AN ANALYSIS OF THE PROPOSED PEDESTRIAN AND UTILITY BRIDGE ACROSS THE RED CEDAR RIVER

Thesis for the Degree of B. S.
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Lester R. Shelden

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An Analysis of the Proposed

Pedestrian and Utility Bridge

Across the Red Cedar River

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BY

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INTRODUCTION

This bridge is actually under consideration by Michigan State College, to be built across the Red Cedar River behind the Ground Building. The purpose of the bridge is primarily to carry steam pipes and electric cables across the river from the new steam generation plant to the main part of the campus. A secondary purpose is to act as a footbridge. It is to be particularly useful to those students going to and from the temporary steel classrooms.

The original plans were to build a suspension bridge for the utilities only. When it was decided to also use the structure as a footbridge, the plate girder bridge was designed. Although everyone concerned favored the plate girder bridge, the State Board of Agriculture decided a less expensive one should be built. Thus the present plans call for a half-through truss, costing $12,000 to $15,000 compared to an estimated $20,000 for the plate girder bridge.

Time did not permit the author to wait for the final decision as to which type of design was to be used. Therefore, this analysis is of the plate girder type which was favored so much. The present indications are that this type will not be used, much to the chagrin of the author, who had hoped to see the realization of the plans he had worked on.

The author acknowledges his indebtedness to all those who helped him in the analysis. Mr. A. Howell, the engineer, who originally designed the bridge, was very cooperative, as were all those in the Civil Engineering Department. Professors C. L. Allen and L. A. Roberts were particularly helpful. Thanks should also be given to the Michigan State Highway Department for the loan of a set of specifications.

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Investigation of Loads, Live Load

The amount of the live load proved to be quite a problem. The plans called for adherence to the fourth edition of "Standard Specifications for Highway Bridges" as a dopted by the A merican Association of State Highway Officials. These specifications call for a live load of 85 pounds per square foot on the sidewalk while designing the flooring and floor beam. When designing the girders and other members, the live load should be determined by the formula:

$$P = \left(30 + \frac{3000}{L}\right)\left(\frac{55-W}{50}\right)$$

This gives a live load of only 54 pounds per square foot for designing the main girders. However, these loadings were not specifically for footbridges, which seem to be a special case. The third edition of the same specifications call for slightly higher loadings in each case -- 100 pounds per square foot for flooring and its immediate supports and a formula similar to the above for the girders. But the third edition also stipulated that all parts of footbridges should be designed for a live loading of 100 pounds per square foot; no reduction being made for the main girders. In the absence of a specific statement concerning footbridges in the fourth edition, the author was forced to use his own discretion. Feeling that during some college activities, the live loading on this bridge would be more that the usual footbridge loading, the third edition's loading of 100 pounds per square foot was decided upon.

A large amount of thought was also spent in consideration of impact on footbridges. Spec. 5 calls for no impact with sidewalk loads but this again was for highway and railway bridges with sidewalks. The possibilities of having extreme loads on this footbridge at such times as the Water Carnival, Freshman - Sophomore tug-o-war, or when football games let out, were considered. It was decided that while some impact would very likely be present, the

100 pounds per square foot was generous enough to allow for any loading which might exist, plus an allowance of 50 to 100 per cent for impact. This allowance for impact offsets a possible reduction in loading due to the large width. It was not thought possible that all 9 feet of the sidewalk would ever be loaded to a packed condition, which would be necessary to produce 100 pounds per square foot on the sidewalk, and still have the load moving so that impact should be considered. Thus the live load, with possible impact included, was set at 100 pounds per square foot. For the 9 foot walkway, this gives 900 pounds per lineal foot of bridge.

The possibility of a vehicle crossing the bridge presented itself, since the walkway has sufficient clearance. When examined with this in mind, as shown below, it was discovered that an H-15 highway loading would produce less bending moment and shear on the main girder than the specified sidewalk loading of 100 pounds per square foot. Similarly, a single truck of greater weight could cross safely.

Sidewalk-loading of 900 #/ft.

$$V = \tfrac{1}{2}\, wl = \tfrac{1}{2}\, 900 \times 104 = 46{,}800\ \#$$

$$M = 1/8\, wl^2 = 1/8\ 900 \times (104)^2 = 1{,}216{,}800 \text{ ft. lb.}$$

Highway loading H-15 (480 #/ft plus a concentrated load of 19,500# for shear and 13,500# for moment.)

$$V = \tfrac{1}{2}\, wl \times P = \tfrac{1}{2}\, 480 \times 104 \times 19{,}500 = 44{,}460\#$$

$$M = 1/8\, wl^2 \times \tfrac{1}{4}\, Pl = 1/8\ 480\ (104)^2 \times \tfrac{1}{4}\ 13{,}500 \times 104 = 1{,}000{,}000 \text{ ft lb}$$

Investigation of Loads, Dead Load

The dead load was divided into two parts: that due to the utilities and that due to the bridge components. As shown below, the dead load was found to be 940# per foot due to the utilities and 927# per foot for the remainder of the bridge, a total of 1867# per lineal foot. Actual size and weights of components were obtained from representatives of the college Building and Utilities Dept. and Landscape and Planning Dept. as follows:

Electric Cables.

(Data obtained from R. Noonon, college electrical engineer)

3 conductor power cable of type to be used 6.683#/ft

4" conduit for sheilding electric cables...10.8#/ft

Conduits for Steam Pipes

(Data obtained from Al Howell, college designing engineer)

Conduit	O.D. Conduit	O.D. Connector	Weight
12"	13 [illegible]"	15"	60#/ft.
15"	16 3/4"	18"	75#/ft.
22"	23 3/4"	24"	115#/ft.

Weight of molded fiber insulation (from Al Howell) 60#/ft 3

Floor System - T Tri-Lok

(Data from Carnegie Pocket Companion)

Weight of standard 2" USS T Tri-Lok @ 4" c-c.

with concrete...........28#/sq. ft.

Investigation of Loads, Dead Load

Utility Weights

Electric cables		
3 conductor power cable	6.65 #/ft	
4" conduit	10.8	
5 cables @	17.5 #/ft	88 #/ft
Telephone cables		
3 cables @ 15 #/ft (estimated)		45 #/ft
8" Return pipe		
8" standard pipe (AISC Handbook)	29 #/ft	
12" Conduit	50	
insulation	19	
water	22	120 #/ft
8" H.P. Steam pipe		
8" standard pipe (AISC handbook)	29 #/ft	
15" conduit	75	
insulation	38	
steam	11	153 #/ft
12" H.P. Steam pipe		
12" standard pipe (AISC handbook)	50 #/ft	
22" conduit	115	
insulation	78	
steam	24	
2 pipes @	267 #/ft	534 #/ft
		940 #/ft

Investigation of Loads, Dead Load

Weight of One Panel

(Panel 6)

Main Girder Flange		2650 #
Main Girder Web		1140
Railing		256
Floor System		2520
Floor Beam		190
Flange Side Plates		52
Lateral Braces		108
Stiffeners		190
Fillers		106
Facia		40
	Total	7650 #

Equivalent uniform load per lineal foot of bridge:

$$\frac{\text{Weight per panel}}{\text{Length of panel}} = \frac{7650\#}{8.25\text{ Ft}} = 927\ \#/\text{ft.}$$

Investigation of Loads, Dead Load

The weights of the Individual members were computed as follows:

Girder Web	4'6" x 3/8" Plate	8.26 ft @ 490#/cu.ft.	570.2 lb.
	4'7" x 3/8" Plate	8.25 ft @ 490#/cu.ft.	580.6
	4'8½"x 3/8" Plate	8.25 ft @ 490#/cu.ft.	598.3
	4'11½"x3/8" Plate	8.25 ft @ 490#/cu.ft.	626.9
	5'3" x 3/8" Plate	8.25 ft @ 490#/cu.ft.	664.2
	5'7½"x 3/8" Plate	8.25 ft @ 490#/cu.ft.	710.4
Girder Flange	ST12WF 161	8.25 ft @ 161#/ft.	1328.3
	ST12WF 106	8.25 ft @ 106#/ft.	874.5
Flange Side Plate	3" x 5/16" Plate	8.25 ft @ 490#/cu.ft.	26.4
Stiffeners, Inside	5 3/16" x 3/8" Plate	4 ft.7 in. @ 490#/cu.ft.	50.4
	5½" x 3/8" Plate	5 ft. 7 in. @ 490#/cu.ft.	62.6
Stiffeners, Outside	5¼" x 3/8" Plate	4 ft. 7 in. @ 490#/cu. ft.	33.7
Stiffeners, End	5½" x ½" Plate	6 ft @ 490#/cu.ft.	56.2
	5¼" x ½" Plate	6 ft @ 490#/cu.ft.	53.8
Fillers	3" x 9/16" Plate	4 ft. 7 in. @ 490#/cu.ft.	26.4
	3" x ¼" Plate	5 ft. 7 in. @ 490#/cu.ft.	14.4
	14" x ¼" Plate	6 ft @ 490#/cu.ft.	71.6
Facia	5" x ¼" Plate	4 ft. 7 in. @ 490#/cu.ft.	19.5
	5" x ¼" Plate	6 ft @ 490#/cu.ft.	25.6
Web Splice	3½" x 5" x 3/8" Angle	4 ft. 1½" 10.4#/ft.	50.0
	3½" x ½" Plate	4 ft. 11½ in. @ 490#/cu.ft.	29.6
	6½" x ½" Plate	4 ft. 11½ in. @ 490#/cu.ft.	54.9
	3½" x ¼" Plate	4 ft. 11½" @ 490#/cu. ft.	44.3
	14" x 3/8" Plate	3 ft. @ 490#/cu.ft.	53.7
	5" x 5" x 7/8" Angle	3 ft. 7½ in. @ 27.2#/ft.	98.5
	5½" x ½" Plate	2 ft. @ 490#/cu.ft.	18.7
	5½" x ¼" Plate	2 ft. @ 490#/cu.ft.	9.4

Curb angle 6" x 6" x 5/8" Angle 8.25 ft @ 14.9#/ft. 123.0

Lateral braces $3\frac{1}{2}$" x $3\frac{1}{2}$" x 5/8" Angle 16.7 ft @ 3.5 /ft. 58.9

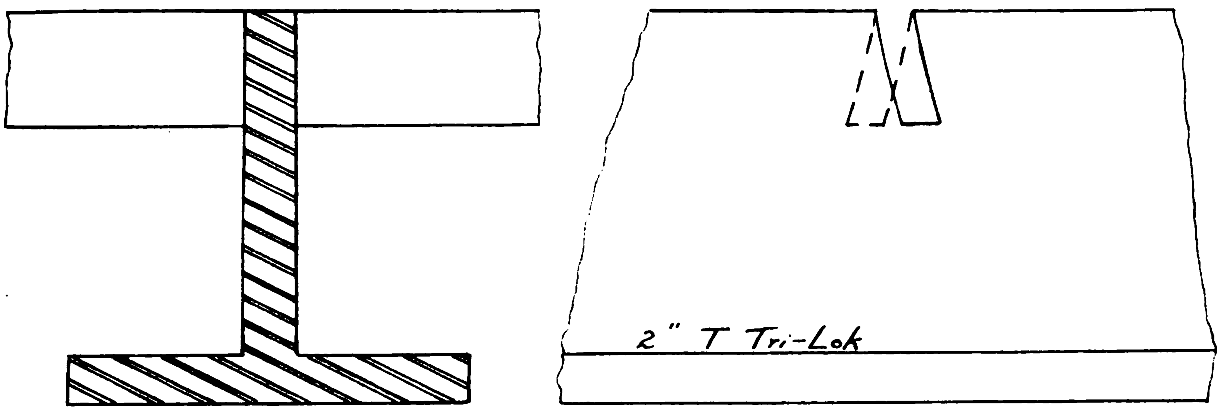
<u>Determination of Stresses, Floor System and Floor Beam</u>

Stress in T Tri-Lok due to 100 #/sq.ft. loading on 8.25 ft. span.

(Data from Carnegie Pocket Companion)

concrete f_c = 489 psi. (700 psi. allowable)

steel f_s = 4896 psi. (18,000 psi. allowable)



Sidewalk live load	100 #/sq.ft.
Sidewalk dead load	38
Sidewalk loading	138 #/sq.ft.
Length of panel	8.25 ft.

Uniform sidewalk load per lineal foot of floor beam 1159 #/ft.

Dead load of floor beam

Utility load per lineal foot of bridge 940 #/ft.

Length of panel 8.25 ft.

Utility load per panel 7755 #

Equivalent uniform utility load per lineal foot of floor beam 862 #/ft.

2022 #/ft.

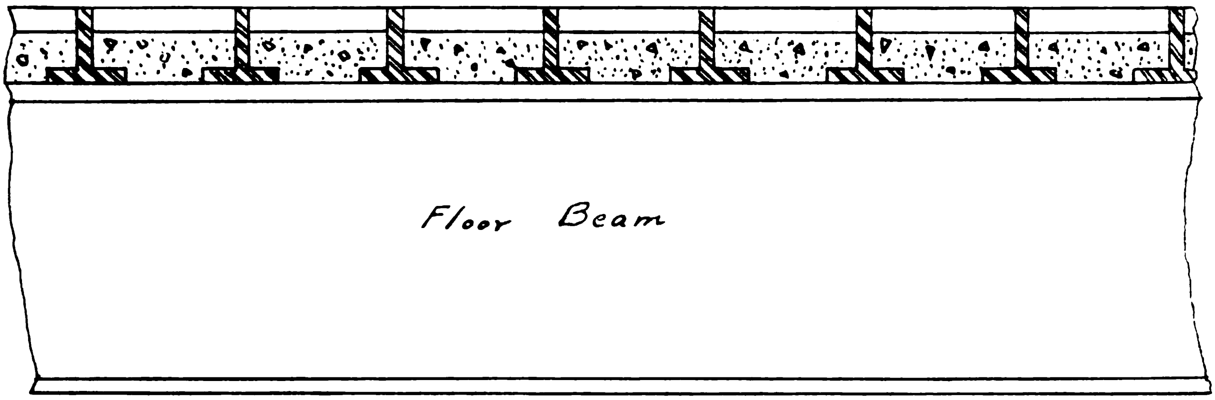
Shear in Floor Beam: $V = \frac{1}{2} wl = \frac{1}{2} 2022 \times 9 = 9{,}099$ #

$$S = \frac{V}{A} = \frac{9{,}099}{6.13} = 1{,}500 \text{ psi. (allowable 11,000 psi.)}$$

Moment in Floor Beam: $M = 1/8\ wl^2 = 1/8\ 2020 \times 81 = 20{,}473$ ft - lb

$$S = \frac{M}{Z} = \frac{12 \times 20{,}473}{18} = 13{,}650 \text{ psi. (allowable 18,000 psi.)}$$

Determination of Stresses, Floor Beam Connections



Floor beam to stiffener connection:

End reaction of floor beam (shear found on previous page) 9,099 #

Weight of curb angle 123

V = 9,222 #

Length of Welds = 2 x 4 13/16 = 9 5/8" = 9.63"

Stress of welds = $S = \dfrac{V}{(.707 \times 3/8)l} = \dfrac{9{,}222 \times 8}{.707 \times 9.63 \times 3} = 3{,}610$ psi.

Allowable (AISC handbook) 13,600 psi.

Stiffener to filler and filler to girder web connections:

Length of welds = 2 x 4 x 3 = 24"

Stress in welds = $S = \dfrac{9{,}222 \times 4}{.707 \times 24} = 2{,}170$ psi.

Allowable (AISC handbook) 13,600 psi.

Determination of Stresses, Main Girder Web

The web of the main girder was designed for adherence to Spec. 11 and 27. The shear was not a deciding factor, as shown below. The uniform load, dead and live, is 900 (live load) x 1867 (dead load) or 2767 lb per ft.

Shear at end of bridge = V = ½ wl = ½ 2767 x 104 = 143,884 #

Shear for each girder = ½V = 71, 942 #

Stress due to shear = S = $\frac{71,\ 942}{6 \times 12 \times 3/8}$ = 2,664 psi.

The minimum web thickness is 3/8 inch, according to Spec. 11. As Spec. 27 states, the web must be more than D/20, in which D is the distance between flanges. Since the width of the web varies, D will vary from 41 inches at the center to 59 inches at the end. As checked below, the web thickness does not follow the specifications at the end. This discrepancy exists only in the first one-half panel and is minor.

at end D ÷ 20 = 59 ÷ 20 = .38 in.

at center D ÷ 20 = 41 ÷ 20 = .32 in.

actual web thickness = 3/8 in. = .375 in.

The width of the web, according to Spec. 10, should be not less than 1/25 the length of the span. This is satisfactory, as shown.

depth ratio 1/25 x 104 = 4.16 ft. = 49.9 inches

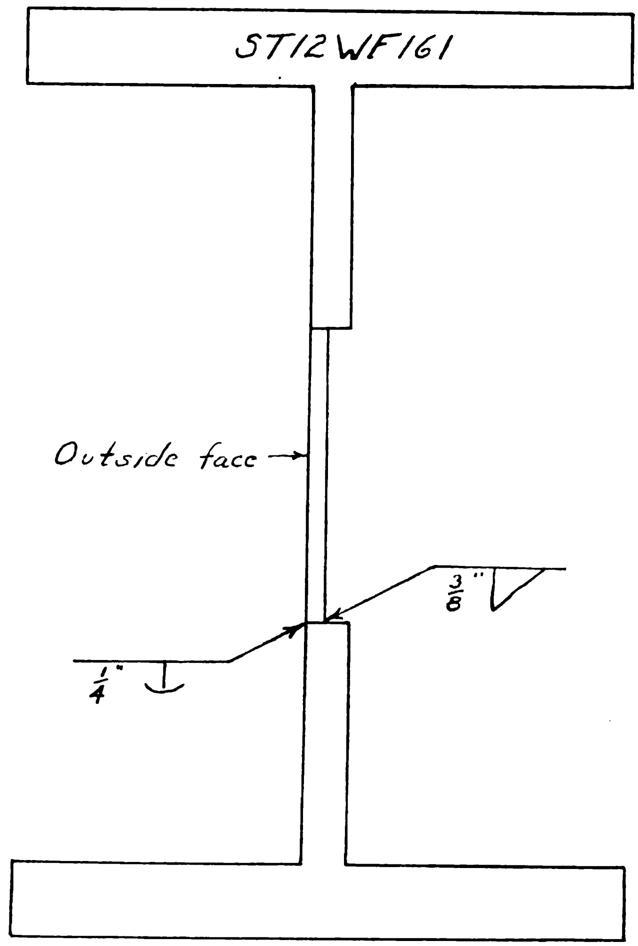
Minimum depth at center 4 ft. 6 in. = 54 inches

Determination of Stresses, Main Girder Flange

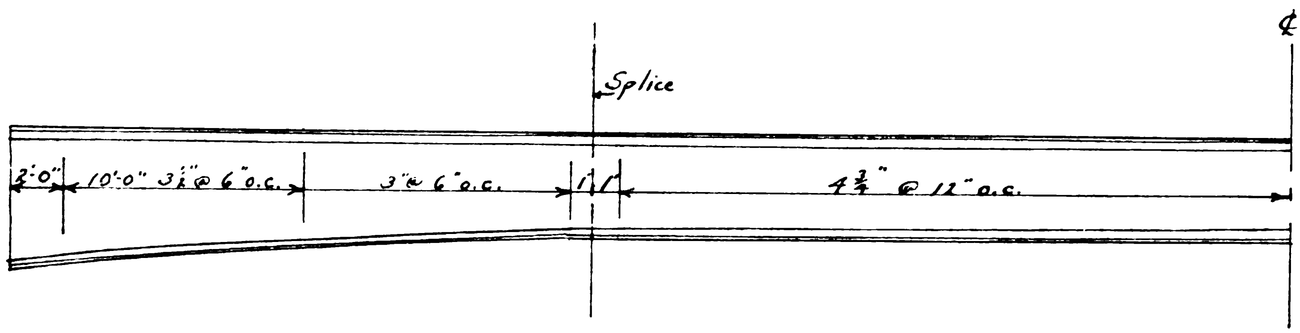
The flanges were analyzed according to the moment of inertia method, as specified in Spec. 25 and 26.

Bending Stress = $\frac{\text{Moment and center } (1/8\ wl2)}{\text{Depth of section Area of Section}}$ = $\frac{2767\ (104)\ 2\ 12}{8x54x23.38x2}$ = 17,790 ps i.

According to Spec. 8, the allowable compressive stress is 18,000 - 5 L2 ÷ b 2, with an L ÷ b ratio less than 40. With L = 120 inches and b = 12 inches, L ÷ b = 10 and the allowable stress is 17,500 psi. As found above, this stress


ST12WF161
Outside face
3/8"
1/4"

Web Welds


¢
Splice
2'-0"
10'-0" 3½" @ 6" o.c.
3" @ 6" o.c.
1' 1'
4¾" @ 12" o.c.

is exceeded. The possibility of this girder failing because of this is slight. Only all the flange and 1/8 web was considered in the moment of inertia method. By considering the entire web as taking bending, the stress would be below the allowable. This is not according to the specifications, however, and the author would not advise using such a narrow depth of section at the center. An extra 2 inches in the depth of the web would not upset the rest of the design and would bring the stress below the maximum allowable.

<u>Determination of Stresses, Main Girder Weld</u>

The horizontal increment of shear was computed and the connection is checked against that. A sample computation follows:

$$H.I. = \frac{V \cdot A_f}{h\,(A_f \times 1/8\ A_w)} = 1000\ \#/\text{in.}$$

$$\text{Stress in welds} = \frac{6 \times 1000}{.707\,(3/8\ 5.5 \times 8 \times \tfrac{1}{2})} = 3{,}030\ \text{psi.}$$

Allowable (A.I.S.C. handbook) 13,600 psi.

<u>Determination of Stresses, Stiffeners</u>

According to Spec. 31, the outstanding legs of stiffener should be more than 2 inches plus 1/30 the depth of the girder and should not exceed sixteen times their thickness. Thus, the length of outstanding legs must fall between 3.8 inches and 6 inches, as computed below. All stiffeners fall in this bracket, their sizes ranging from 5 5/16 to 5½ inches.

$$2 \times \frac{D}{30} - 2 \times \frac{54}{30} - 3.8''$$

$$16t - 16 \times 3/8 - 6''$$

Stiffeners are placed every 4 ft. 3 in., well below the depth of web limit of 4 ft. 6 in.

The end stiffeners are designed for bearing on their outer flanges only, as stated in Spec. 30.

2 - 5½" x ½" plates 5.5 sq. in.

2 - $5\frac{1}{2}$" x $\frac{1}{2}$" Plates   5.75 sq. in.

2 - 5" x $\frac{1}{4}$" Plates $\underline{2.5}$ sq. in.
                                   13.75 sq. in.

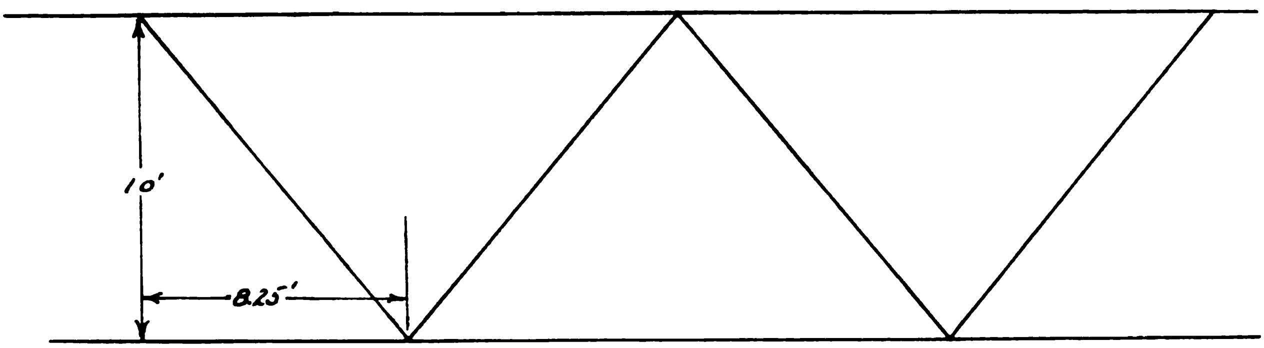
The total load on the end stiffeners will be one fourth.

The total weight of the bridge or 71, 942 06.

$$\text{Bearing Stress} - \frac{71,\ 942\ \#}{15.75\ \text{sq. in.}} = 5,225\ \text{psi.}$$

(allowable 18,000 psi.)

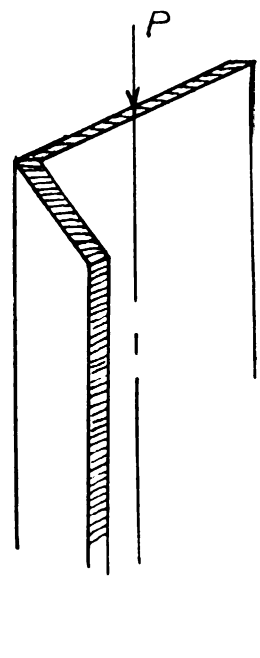
<u>Determination of Stresses, Lateral Bracing</u>



The specifications state that lateral bracing should be designed for a wind load of 300 pounds per lineal foot. (Refer to Spec. 7.) Each lateral would hold the wind load of one panel in either tension or compression.

300 #/ft 8.25 ft = 2475 # per panel

There would be no danger of buckling under this load. The stress was computed for an eccentric loading as shown below.



$$S = \frac{V}{A} \pm \frac{Mc}{I}$$

$$= \frac{2475}{2.48} \times \frac{2475 \times 1.25 \times 3.63}{2.9}$$

$$= 4,870\ \text{psi.}$$

The allowable stress was computed from Spec. 8, the formula for riveted columns being used. L/r = 130

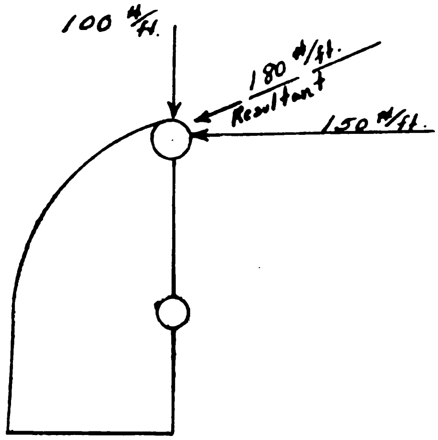
$$S = 15,000 - \frac{1}{4}\frac{L^2}{r^2}$$

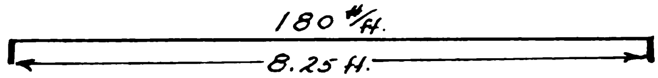
$$S - 15,000 - \frac{1}{4}\frac{(11.63)^2}{1.07}$$

$$S - 14,750\ \text{psi.}$$

Determination of Stresses, Railings

As per Spec. 1, the top of the railings is 3 feet above the sidewalk. Also, the railing is to be designed to withstand a vertical force of 100 pounds per foot and a horizontal force of 150 pounds per foot, as shown below.





The unit stresses in the railing were computed as follows:

Shear in railing: $V = \frac{1}{2}\ 180 \times 8.25 = 742.5\ \#$

Stress due to Shear: $S = \frac{V}{A} = \frac{742.5}{2.25} = 333$ psi.

Moment in railing: $M = 1/8\ 180\ (8.25)^2 = 1530$ ft. lb.

$$S = \frac{Mc}{I} = \frac{12 \times 1530 \times 1.5}{3.017} = 9{,}150 \text{ psi.}$$

(allowable 14,500 psi.)

The strength of the railings appeared quite sufficient but the welds were checked as follows, considering only the horizontal force:

$$S = \frac{150 \times 8.25}{.707 \times 3 \times \frac{1}{4}} = 2340 \text{ psi.}$$

(allowable 13,600 psi.)

Determination of Stresses, End Bearings

Refer to Spec. 15 to 20 inclusive, for the design of the end bearings. The specifications are followed very well. Expansion must be allowed for at the rate of 1¼ inches for every 100 feet or 1.3 inches for this span of 104 feet. 3 inches are allowed. Bronze sliding expansion bearings are provided. The anchor bolts prevent any lateral movement. The bolts extend into the masonry the required 12 inches and a 4" x 6" x 3/8" angle adds to their stability. The girder is supported on metal plates so that the bottom chord is 6 inches above the bridge seat. The base plate is 13" x 14½", giving a pressure on the masonry of

$$\frac{71,942\ \#}{13 \times 14\frac{1}{2}\ \text{sq. in.}} = 380\ \text{psi.}$$

(allowable 1000 psi.)

Summary of Stresses

The unit stresses for each member, found by assuming the application of a live load of 100 pounds per square foot of sidewalk area, are listed below. The allowable unit stress, according to the A.A.S.H.O. specifications, are listed opposite for comparison. A ll stresses are in pounds per square inch.

Member under consideration	Stress as found	Allowable stress
T Tri-Lok Flooring (concrete)	489	700
(steel)	4,696	18,000
Floor beam (due to shear)	1,500	11,000
(due to bending)	13,650	18,000
Floor Beam Connections	3,610	1 3,600
	2,170	13,600
Main Girder Web (shear)	2,664	11,000
Main Girder Flange (bending)	17,790	17,500
Main Girder Welds	3,020	13,600
End Stiffeners (bearing)	5,225	18,000
Lateral Bracing	4,870	11,750
End Bearings (masonry)	380	1,000

CONCLUSION

In answer to the question, "Does this bridge fulfill the required specifications?" the answer is yes. The fourth edition of the specifications calls for a live load of 85 pounds per square foot on sidewalks and makes no other statement concerning footbridges. From this, one might conclude that 85 pounds per square foot should be used for this bridge. The allowable stresses are not exceeded with such as applied load. However, as explained in the investigation of live loads, the author felt that the statement concerning footbridges in the third edition of the same specifications should apply to this case. As it turned out, all unit stresses were very satisfactory, sa ve for the flexure stress in the main girder flange,.

A comparison of the allowable stresses, according to the fourth edition of "Standard Specifications for Highway Bridges," and the unit stresses found by applying a live load of 100 pounds per square foot is shown on the preceeding page. It will be noted that the unit stress in the main girder flange is a bove the allowable. A live load of 96 pounds per square foot could be applied without exceeding the allowable stress for the flange. It is possible that the live load used by the author was too high. A more intensive knowledge of the development of this design will present the possibility that the bridge was slightly under-designed. The designing engineer used a utility load of 700 pounds per lineal foot of bridge. The utility loading as found in this analysis was 940 pounds per foot. There is quite a difference there and could bring the unit str ss in the flange well below the allowable.

The reason for the large difference in the two utility loads in evident when the designing conditions are known. The designing engineer did not have a definite knowledge of the type and amount of utilities that were to be put across. The pipe sizes and types were changed several times after the design was started. The utilities shown on the plans included with this analysis are not exactly the utilities which the bridge was designed for. The engineer considered the changes minor enough and the bridge over-designed enough to carry the extra utility load. Strictly speaking, this was not so; but the resulting over-load is so slight, and the possibility of the structure ever receiving such extreme live loads is so rare, that the engineer was quite justified in not changing his design. This will be further borne out by the fact that the present plans represent a further change in the arrangement of utilities. The author offers no explanation for the numerous changes.

Except for the girder flange, the design follows the specifications quite satisfactorily. The welded construction allows ample strength for all connections. The secondary members are over-designed in order to fulfill the requirements of the specifications concerning minimum sizes.

Considering that a primary purpose of this analysis was for the benefit of the author, in obtaining experience in structural design and formal reports, the time was well spent and the rewards were more than expected. The author was quite enlightened concerning the special problems presented by footbridges. This problem brought out quite clearly, the importance and difficulty which arises in the choosing of specifications and applied loads. It is regretted that all the "little things" learned by the author in writing this could not be presented on paper for the benefit of others.

Excerpts from "Standard Specifications for Highway Bridges"

1. Railings. Substantial railings along each side of the bridge shall be provided for the protection of traffic. The top of the railing shall be not less than 3 feet above the finished surface of the roadway adjacent to the curb, or if on a sidewalk, not less than 3 feet above the sidewalk floor.

Railings shall be designed to resist a horizontal force of not less than 150 pounds per linear foot of bridge, applied at the top of the railing, and a vertical force of not less than 100 pounds per linear foot.

2. Dead Load. The dead load shall consist of the weight of the structure complete, including the roadway, sidewalks, car tracks, pipes, conduits, cables, and other public utility services.

The snow and ice load is considered to be offset by an accompanying decrease of live load and impact and shall not be included except under special conditions.

The following weights are to be used in computing the dead load:

Steel	490 lb/cu.ft.
Concrete, plain or reinforced	150 lb/cu.ft.
Loose sand, earth and gravel	100 lb/cu.ft.
Compacted sand, earth, gravel or ballast	120 lb/cu.ft.

3. Live Load. The live load shall consist of the weight of the applied moving load of vehicles, cars or pedestrians.

4. Sidewalk Loading. Sidewalk floors, stringers and their immediate supports, shall be designed for a live load of 85 pounds per square foot of sidewalk area. Girders, trusses, arches and other members shall be designed for the following sidewalk live load per square foot of sidewalk area: (for spans over 100 feet)

$$P = \left(30 \times \frac{3000}{L}\right)\left(\frac{55-W}{50}\right)$$

P = live load per square feet (maximum 60 psf.)

L = loaded length of sidewalk in feet.

W = width of sidewalks in feet.

Excerpts from "Standard Specifications for Highway Bridges"

5. Impact. Live load stresses, except those due to sidewalk loads and centrifugal, tractive, and wind forces, shall be increased by an allowance for dynamic, vibratory, and impact effects.

6. Longitudinal Force. Provision shall be made for the effect of a longitudinal force of 10 per cent of the live load on the structure, acting 4 feet above the floor.

7. Wind Loads. The wind force on the structure shall be assumed as a moving horizontal load equal to 30 pounds per square foot on 1½ times the area of the structure as seen in elevation, including the floor system and railings, and on one-half the area of all trusses or girders in excess of two in the span. The total assumed wind load shall be not less than 300 pounds per linear foot in the plane of the loaded chord and 150 pounds per linear foot in the plane of the unloaded chord on truss spans, and not less than 300 pounds per lineal foot on girder spans.

8. Allowable Stresses in Structural Steel and Rivets.

Axial tension, net section .... 18,000 psi.

Tension in extreme fibers of rolled shapes, girders

and built sections subject to bending .... 18,000 psi.

Tension in bolts at root of thread .... 13,500 psi.

Axial compression, gross section, stiffeners

of plate girders .... 18,000 psi.

Columns: The permissible unit stress in concentrically loaded columns having values of L/r not greater than 140 may be computed from the following approximate formulas.

Riveted ends .... $15,000 - \frac{1}{4} L^2/r^2$

Pin ends .... $15,000 - 1/3\ L^2/r^2$

L = length of member, in inches; r = least radius of gyration.

Compression in extreme fibers of rolled shapes, girders, and built sections subject to bending (for values of L/b not greater than 40) .... 18,500 - 5 L^2/b^2

8. Allowable Stresses in Structural Steel and Rivets.

Allowable compression in splice material, gross section .... 18,000 psi.

Stress in extreme fibers in pins .... 27,000 psi.

Shear in girder webs, gross section .... 11,000 psi.

Shear in power-driven rivets and pins .... 13,500 psi.

Shear in turned bolts .... 11,000 psi.

Bearing in pins .... 24,000 psi.

Bearing on power-driven rivets, milled stiffeners, and other

steel parts in contact .... 27,000 psi.

Bearing on pins subjected to rotation (not due to deflection) . 12,000 psi.

Bearing on turned bolts .... 20,000 psi.

9. Allowable Pressure on Masonry.

Bridge seats - under hinged rockers and bolsters (not subjected to high edge loading by a deflecting beam, girder or truss) .... 1,000 psi.

The above bridge seat unit stress will apply only where the edge of the bridge seat projects out at least 3 inches beyond edge of the shoe or plate. Otherwise, the unit stresses permitted will be 75% of the a bove amount.

10. Depth Ratio. The ratio of the depth to the length of span, for plate girders, shall be not less than 1/25.

11. Thickness of Metal. The minimum thickness of structural steel shall be 5/16 inch, except for fillers, railings and unimportant details. Gussets shall be not less than 3/8 inch thick. Metal subjected to marked corrosive influence shall be of greater thickness.

12. Strength of Connections. No connection, except for lattice bars and hand-rails, shall contain less than three rivets. All connections and splices, whether in tension or compression, shall be proportioned to develop the full strength of the members, and no allowance in excess of 50% shall be made for milled ends of compression members. Splices shall be as near the panel points

as practicable and, in general, shall be on that side of the panel point which is subjected to the smaller stress.

13. Compression Members. Compression members shall be designed so that the metal shall be concentrated as far as feasible in the webs and flanges, and so that the center of gravity of the section may be as near the center line of the member as practicable.

The thickness of web plates of compressive members shall be not less than one-thirtieth of the transverse distance between the lines of rivets connecting them to the flanges. The thickness of cover plates of compression members and cover plates on the compression flanges of plate girders preferably shall not be less than one-fortieth of the transverse distance between the lines of rivets connecting them to the flanges, but the minimum may be one-fiftieth of this distance, provided the width of the plate between the connecting lines of rivets in excess of forty times the thickness shall not be considered as effective in resisting stress.

14. Outstanding Legs of Angles. The widths of the outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following: In girder flanges and main members carring axial stress, twelve times the thickness. In bracing and other secondary members, sixteen times the thickness.

15. Expansion. Provision shall be made for expansion and contraction at the rate of $1\frac{1}{4}$ inches for every 100 feet. The expansion ends shall be secured against lateral movement.

16. End Bearings. Expansion ends shall be firmly secured against lifting or lateral movement. Fixed bearings shall be firmly anchored. Spans of less than 70 feet may be arranged to slide upon metal plates with smooth surfaces. Spans of 70 feet or more shall be provided with rollers or rockers, or else with bronze sliding expansion bearings.

Excerpts from "Standard Specifications for Highway Bridges."

17. Rollers. Expansion rollers shall not be less than four inches in diameter for spans of 100 feet or less, and this minimum shall be increased not less than one inch for each additional 100 feet and proportionally for intermediate lengths. They shall be connected by substantial side-bars and shall be effectively guided so as to prevent lateral movement, skewing, or creeping. The rollers and bearing plates shall be protected from dirt and water as far as possible, and the construction shall be such that water shall not be retained and that the roller nests may be inspected and cleaned with the least difficulty.

18. Pedestals and Shoes. Pedestals and shoes shall be used and be designed to secure rigidity and stability and to distribute the reaction uniformly over the entire bearing area. They shall be made preferably of cast steel or structural steel. The bottom bearing widths shall not exceed the top bearing widths by more than twice the depth of the pedestal and, when involving pin bearings, this depth shall be measured from the center of the pin. Where built pedestals and shoes are used, the web plates and the angles connecting them to the base plates shall not be less than 5/8 inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely.

19. Anchor Bolts. Anchor bolts for trusses and girders shall not be less than $1\frac{1}{4}$ inches in diameter and shall extend into the masonry not less than 12 inches. Washers shall be used under the nut. Anchor bolts subjected to tension, as in viaduct towers, shall engage 50 per cent more masonry than is required by the uplift.

20. Sole Plates. Sole plates of girders and trusses shall not be less than $\frac{3}{4}$ inch thick.

21. Masonry Bearings. Girders and trusses on masonry shall be so supported on metal plates or pedestals that the bottom chords will be above the bridge seat, preferably not less than 6 inches.

Excerpts from "Standard Specifications for Highway Bridges."

22. Floor System. Floor-beams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Spans with floor systems preferably shall have end floor-beams. When end floor-beams are not used, the end panel stringers shall be secured in correct position by end struts securely connected to the stringers and to the main girders or trusses. The end panel lateral bracing shall be rigidly attached to the main girders, or trusses, and shall be attached to the end struts.

23. Minimum Size of Angles. The smallest angle used in bracing shall be 3 x $2\frac{1}{2}$ inches.

24. Lateral Bracing. Bottom lateral bracing shall be provided in all spans except I-beam spans and deck plate girder spans of 50 feet or less.

25. Through Plate Girder Spans. Through plate girder spans shall be stiffened against lateral deformation by means of gusset plates, or knee braces with solid webs, attached to the stiffener angles and floor beams. These braces generally shall extend to the clearance line. If the unsupported length of the inclined edge of the gusset plate exceeds 60 times its thickness, the gusset plate shall have one or two stiffening angles riveted along its edge.

26. Design of Plate Girders. Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity. One-eighth of the gross section of the web, if the web is effectively spliced, may be considered as flange section. For girders having unusual sections, the moment of inertia method shall be used.

27. Flange Sections. The flange angles shall form as large a part of the area of the flange as practicable, side plates shall not exceed except where flange angles angles exceeding 7/8 inch in thickness otherwise would be required.

The gross area of the compression flange shall not be less than the gross area of the tension flange.

Flange plates shall be of equal thickness, or shall decrease in thickness from the flange angles outward. No plate shall have a thickness greater than

Excerpts from "Standard Specifications for Highway Bridges."

27. Flange Sections.

that of the flange angles.

If the flange plates are used, at least one plate on the top flange shall extend the full length of the girder, except where the flange is to be covered with concrete. Any additional flangle plates shall extend at least one foot beyond the theoretical end, and there shall be a sufficient number of rivets at each end of each plate to develop its full stress value before the end of the next outside plate is reached.

28. Thickness of Web Plates. The thickness of web plates, except those to be encased in concrete, shall be not less than D/20, in which D is the distance in inches between flanges.

29. Flange Splices. Splices in flange parts shall not be used except by special permission of the engineer. Not more than one part shall be spliced at the same cross-section. If practicable, splices shall be located at points where there is an excess of section. The net section of the splice shall exceed by 10 per cent the net section of the part spliced. Flange angle splices shall consist of two angles, one on each side of the girder.

30. Web Splices. Web plates shall be spliced symmetrically by plates on each side. The splice shall be equal to the web in strength in both shear and moment. The splice plates for shear shall be the full depth of the girder between flanges. In the splice, there shall be not less than two rows of rivets on each side of the point.

31. End Stiffeners. Over the end bearings of plate girders, there shall be stiffener angles, the outstanding legs of which shall extend as nearly as pra cticalbe to the outer edge of the flange angles. End stiffeners shall be proportioned for bearing on the outstanding legs of the flange angles, no allowa nce being made for the portions of the legs fitted to the fillets of the f lange angles. End stiffeners shall be arranged, and there shall be sufficient number of rivets in their connections to the web, to transmit the entire end rea ction

Excerpts from "Standard Specifications for Highway Bridges."

51. End Stiffeners.

to the bearings. They shall not be crimped.

52. Intermediate Stiffeners. Webs shall be stiffened by angles riveted thereto in pairs on opposite sides, with outstanding legs not exceeding sixteen times their thickness, nor less than 2 inches plus one-thirtieth of the depth of the girder. Intermediate stiffeners shall be placed at points of concentrated loading and at intervals not exceeding the depth of the web, nor 6 feet.

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