# ANALYSIS OF STRESSES IN PEDESTRIAN & UTILITY BRIDGE ACROSS RED CEDAR RIVER AT MICHIGAN STATE COLLEGE

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE

Robert E. Miller

1948

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# Analysis of Stresses
# in Pedestrian & Utility Bridge
# Across Red Cedar River
# at Michigan State College

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

Robert E. Miller

Candidate for the Degree of

Bachelor of Science

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PEDESTRIAN AND UTILITY BRIDGE OVER THE RED CEDAR AT M.S.C.
37030

# INTRODUCTION

This half through pony truss bridge which is located on the Michigan State College campus spans the Red Cedar River behind the stores and electrical engineering buildings. The bridge was primarily built to carry steam pipes and electrical cables across the river to the main part of the campus. Its secondary purpose is to act as a footbridge. This bridge has proven to be very useful for students going to and from the temporary classroom buildings which preside on the south side of the river.

Originally the college had planned to construct a suspension bridge to carry the utilities only. When it was decided to use it also as a footbridge, a plate girder bridge was the next thought in everyone's mind, but the State Board of Agriculture decided that a less expensive bridge should be built. Therefore, this half-through pony truss, costing approximately $21,670 compared to an estimated $27,000 for the plate girder bridge was built.

The two 106 Ft. girders of this bridge were completely built and welded at the Jarvis Engineering Works in Lansing and transported one at a time by truck and trailer to the location on the campus where the bridge now stands. The process of transporting the bridge and placing it are shown by illustrations in this thesis. The abutments built of reinforced concrete were ready for the truss upon arrival, and the girders were swung into position by cranes. The floor beams, floor system, struts and the lower lateral system were field welded after the girders were in position.

The author acknowledges his indebtedness to all of those who helped him in the analysis. Mr. Hebblewhite of the Jarvis Engineering Works was very cooperative, as were all those in the Civil Engineering Department - Professors C. L. Allen, C. A. Miller, L. A. Roberts were particularly helpful. Thanks should also be given to Mr. A. Howell for the loan of a set of specifications, and members of the Reniger Construction Company.

June 1948 Robert E. Miller

# COST OF BRIDGE BREAKDOWN

## Abutments

| | |
|---|---|
| Reinforcing Steel - - - - - - - - - - - - - - | $1,500.00 |
| Concrete Work - - - - - - - - - - - - - - - - - | 4,670.00 |
| Excavation, Backfill, etc. - - - - - - - - - | 3,000.00 |

## Superstructure

| | |
|---|---|
| Structural Steel - - - - - - - - - - - - - - - | $11,300.00 |

## Floor

| | |
|---|---|
| T-Tri-Lok - - - - - - - - - - - - - - - - - - - | $1,200.00 |
| | $21,670.00 |

## INVESTIGATION OF LOADS, DEAD LOAD UTILITY WEIGHTS

**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. |

**Electric Cables**

| | |
|---|---|
| 3 Conductor Power Cable | 6.65#/ft. |
| 4" Conduit | 10.50 |
| 5 Cables @ | 17.5/ft. |
| | 88#/ft. |

**Telephone Cables**

| | |
|---|---|
| 3 Cables @ 15#/ft (Estimated) | 45#/ft. |
| Total = | 940#/ft. |

## INVESTIGATION OF LOADS, DEAD LOAD

The dead load in this bridge I divided into two parts; that due to the utilities and that due to the bridge components. As shown in the following pages, the dead load due to the utilities was found to be 940# per lineal foot and 636# per lineal foot for the remainder of the bridge, a total of 1576#'s per lineal foot. Actual size and weights of components were obtained from representatives of the college Building and Utilities Department, and Landscape and Planning Department as follows:

### Conduits for Steam Pipes

(Data obtained from Al Howell, College Designing Engineer)

| Conduit | O.D. Conduit | O. D. Connective | Weight |
|---|---|---|---|
| 12" | 13 1/4" | 15" | 50#/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]

### Electric Cables

(Data obtained from R. Noonan, College Electrical Engr.)

3" Conductor Power cable of Type to be used - - - - -6.653#/ft.

4" Conduit for shielding electric cables - - - - - -10.8#/ft.

### Floor System - T Tri-Lok

(Data from Carnegie Pocket Companion )

Wt. of Standard 2" T Tri-Lok @ 4" c-c

With concrete - - - - - - - - - - - - -38#/ '

## INVESTIGATION OF LOADS, DEAD LOAD

Wt. of individual members were computed as follows:

(Letters in left hand column denote sections as shown on blueprints)

Vertical diagonals:

| | | |
|---|---|---|
| (T) | 10" ⊓ 15.3# x 9'-10" | = 150.5# |
| (V) | 10" ⊓ 15.3# x10'- 0" | = 153.0# |

Lateral diagonals

| | | |
|---|---|---|
| (L1) | L 3½ x 3½ x 5/16 (7.2) (10'-9 1/8") | = 77.5# |
| (L2) | L 3½ x 3½ x 5/16 (7.2) (11'-2") | = 80.4# |

Lower Chord Struts

| | | |
|---|---|---|
| (B2) | 4 WF 10 (9'-2"x10) | = 91.7# |

Gusset Plates

| | | |
|---|---|---|
| (b) | 8"x3/8"x 1' - 5½" @ 490# /cf. | = 14.7# |
| (c-1) | 6"x3/8"x 1' - 1" @ 490# /cf | = 8.3# |
| (c) | 6"x3/8#x 0' - 11" @ 490# /cf | = 7.1# |
| (f) | 8"x3/8"x 1' - 2½" @ 490# /cf | = 12.3# |
| (d) | 6"x3/8"x 0' - 3½" @ 490# /cf | = 2.2# |
| (g) | 6"x3/8"x 1' - 7" @ 490# /cf | = 12.2# |
| *(e) | 8"x3/8"x 0' - 9½" @ 490# /cf | = 8.1# |
| *(s) | 6"x3/8"x 0' - 6" @ 490# /cf | = 3.8# |
| (t) | 6"x3/8"x 1' - 5" @ 490# /cf | = 10.9# |
| (v) | 6"x3/8"x 0'8" @ 490# /cf | = 5.1# |
| (p) | 6"x3/8"x 1' - 4" @ 490# /cf | = 10.2# |

Splice Plates

| | | |
|---|---|---|
| (a) | 10"x3/8"x 1'- 0" @ 490#/cf | = 12.8# |

Spacer Plates

| | | |
|---|---|---|
| (m) | 6"x3/8"x 0' - 8" @ 490#/cf | = 5.1# |
| (n) | 10"x3/8"x 0' - 9½" @ 490#/cf | = 10.1# |

**Floor Beam**

| | | | |
|---|---|---|---|
| (B-1) | 8"I 18.4# x 9' - 3" | = | 170.2# |

**Side Plate**

| | | | |
|---|---|---|---|
| (P-1) | 48"x3/16"x 6' - 10" @ 490# | = | 209.7# |
| (P 2) | 48"x3/16"x 7' - 4" @ 490# | = | 225.1# |

**Verticals**

| | | | |
|---|---|---|---|
| (N) | 10" WF 21 x 7' - 11½" | = | 166.7# |
| (S) | 10" WF 15 x 7' - 11½" | = | 119.1# |

**End Posts**

| | | | |
|---|---|---|---|
| (M) | 10" WF 49 7' - 11½" | = | 389.0# |

**Curb L's**

| | | | |
|---|---|---|---|
| (F) | 1L4x6x½ x 52'-6" (16.2)(52.5) | = | 850.5# |
| (E) | 1L4x6x½ x 26'-6" (16.2)(26.5) | = | 429.3# |

**Railing**

| | | | |
|---|---|---|---|
| (B) | 12"⊓ 80.7x60' - 00" | = | 1242.0# |
| (A) | 12"⊓ 20.7x45' - 6" | = | 941.85# |
| (H) | 3" ⊔ 4.1x52' - 3" | = | 214.23# |
| (HA) | 3" ⊔ 4.1x52' - 11" | = | 216.96# |

## INVESTIGATION OF LOADS, DEAD LOAD
## Wt. of Entire Bridge
## Vertical Truss

**Railing**

| | | | |
|---|---|---|---|
| (A & B) | (2183.85)(4) | | 8,735.4# |
| (H) | (214.23)(4) | | 856.92# |
| (HA) | (216.96)(4) | | 867.84# |
| | | | 10,460.16# |

**Verticals**

| | | | |
|---|---|---|---|
| (N) | (166.7)(4) | = | 666.8# |
| (S) | (119.1)(22) | = | 2,620.2# |
| | | | 3,287.0 |

**End Posts**

| | | | |
|---|---|---|---|
| (M) | (389.0)(4) | = | 1,556.0# |

**Diagonals**

| | | | |
|---|---|---|---|
| (V) | (153.0)(24) | = | 3,672.0# |
| (T) | (150.5)(4) | = | 602.0# |

**Gusset Plates**

| | | | |
|---|---|---|---|
| (b) | (8) (14.7#) | = | 117.6# |
| (c-1) | (8) (8.3#) | = | 66.4# |
| (c) | (40)(7.1#) | = | 284.0# |
| (d) | (40)(7.1#) | = | 8.8# |
| (g) | (4)(12.2#) | = | 48.8# |
| (f) | (8)(12.3#) | = | 98.4# |
| (e) | (8)(8.1) | = | 64.8# |
| | | | 688.8# |

**Splice Plate**

| | | | |
|---|---|---|---|
| (a) Rail Splice | (4)(12.8#) | = | 51.2 |
| **Bottom Chord (Vert. Truss)** | | | |
| (E) | (4)(429.3#) | = | 1,717.2# |
| (F) | (4)(850.5#) | = | 3,402.0# |
| **Batten Plates (Btm Chord)** | | | |
| (m) | (20)(5.1#) | = | 102.0# |
| (n) | (8)(10.1#) | = | 80.8# |

**Bottom Lateral System**

| | | | |
|---|---|---|---|
| Struts | | | |
| (B2) | (13)(91.7#) | = | 1,192.1# |
| **Laterals** | | | |
| (L1) | (2)(77.5#) | = | 155.0# |
| (L2) | (12)(80.4#) | = | 964.8# |
| **Gusset Plates** | | | |
| (S) | (13)(3.8#) | | 49.4# |
| (T) | (11)(10.9#) | | 119.9# |
| (V) | (2)(5.1#) | | 10.2# |
| (P) | (2)(10.2#) | | 20.4# |

**Floor System**

| | | | |
|---|---|---|---|
| (B1) | (15)(170.2#) | = | 2,553.0# |
| Concrete & T-Tri-Lock | = (9')(105.5)(38)= | | 36,081 # |
| | | | 66,765 |

St./Panel = $\frac{66,765}{14}$ = 4,768.6#

Uniform load/lin ft. = $\frac{4,769}{7.5}$ = 636#

## INVESTIGATION OF LOADS, LIVE LOAD

The live load was quite a problem to decide upon. The plans called for adherence to the fourth edition of "Standard Specifications for Highway Bridges" as adopted by the American 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 trusses 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 trusses. However, these loadings were for sidewalks, not specifically for footbridges. 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 trusses. The third Edition also stipulated that all parts of the footbridge should be designed for a live loading of 100 pounds per square foot. Since there was no specific statement as to what to use on a footbridge in the fourth Edition, the author was forced to use his own discretion. Since the bridge is in big use during many activities on campus, the author feels that it would be more likely to use the third Edition's loading of 100 pounds per square foot.

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. Sincethere is the possibility of having extreme loads on this footbridge such as after football games in Michigan State's enlarged football stadium which is very near, or even students passing between

between classes, impact was 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 percent for impact. 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 lbs. 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 lbs. per sq. foot. For the 9 foot walkway, this gives 900 lbs. 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 truss than the specified sidewalk loading of 100 pounds per square foot. Similarly, a single truck of greater weight could cross safely.

<u>Sidewalk - loading of 900#/ft.</u>

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

$M = 1/8\ wl^2 = 1/8\ (900)(104)^2 = 1{,}216{,}000$ ft. lbs

<u>Highway loading H-15</u> (480#/ft plus a Concentrated load of 19,500# for shear and 13,500# for moment)

$V = \frac{1}{2}\ wl \times P = 1/2\ (480)(104)(19{,}500) = 44{,}460\#$

$M = 1/8\ wl^2 \times \frac{Pl}{4} = 1/8\ (480)(104)^2 \times \frac{(13{,}500)(104)}{4}$

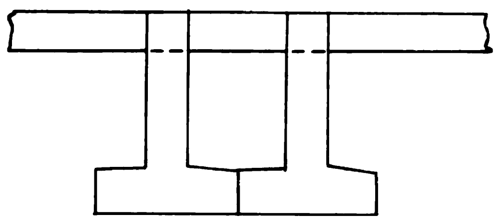
$= 1{,}000{,}000$ ft.lbs.

## DETERMINATION OF STRESSES, FLOOR SYSTEM & FLOOR BEAM

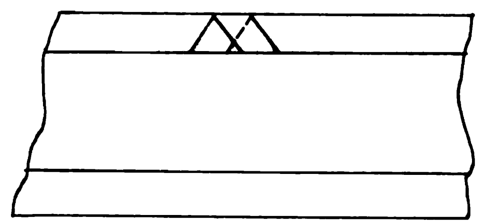
**Stress in T-Tri-Lok due to 100#/sq. ft. loading on 750 foot span**

(Data from Carnegie Pocket Companion)

| | | | |
|---|---|---|---|
| Concrete | $f_c$ = 405 psi. | (700 psi allowable) |
| Steel | $f_s$ = 3888 psi. | (18,000 psi. allowable) |



End View



Side View

2" T-Tri-Lok

| | | |
|---|---|---|
| Sidewalk | L.L. = | 100 lbs. /sq. ft. |
| Sidewalk | D.L. = | 38 lbs. /sq. ft. |
| Sidewalk loading | = | 138 lbs. /sq. ft. |

Length of panel = 7.50'

& Uniform sidewalk ld/lin. ft. of floor beam

= (138)(7.50) = 1035#/ft.

& Dead Load of Floor Beam

| | | |
|---|---|---|
| Utility load /lineal foot of bridge | | 940#/ft. |
| Length of panel | | 7.50' |
| Utility load /panel | = | 7050 lbs. |

Equivalent uniform utility load/lineal foot

of floor beam = $\frac{7050}{9.167}$ = 769#/ft

1804#/ft

Shear in floor beam

V= $\frac{1}{2}$ wl = 1/2 (1804)(9.17) = 8271.3#

S= $\frac{8271.3}{5.34}$ = 1,549.0#/ in. (allow 11,000)

Moment in floor beam:

$$M = 1/8\,Wl^2 = 1/8\,(1804)(9.17)^2 = 18{,}962.1 \text{ ft. lbs.}$$

$$S = M/z = \frac{(12)(18{,}962)}{14.2} = 16{,}024\#/\square'' \quad (\text{allow. } 18{,}000)$$

# DETERMINATION OF STRESSES, VERT. TRUSS

D.L. + L.L. = 1576 + 900 = 2476#/ lin. ft.

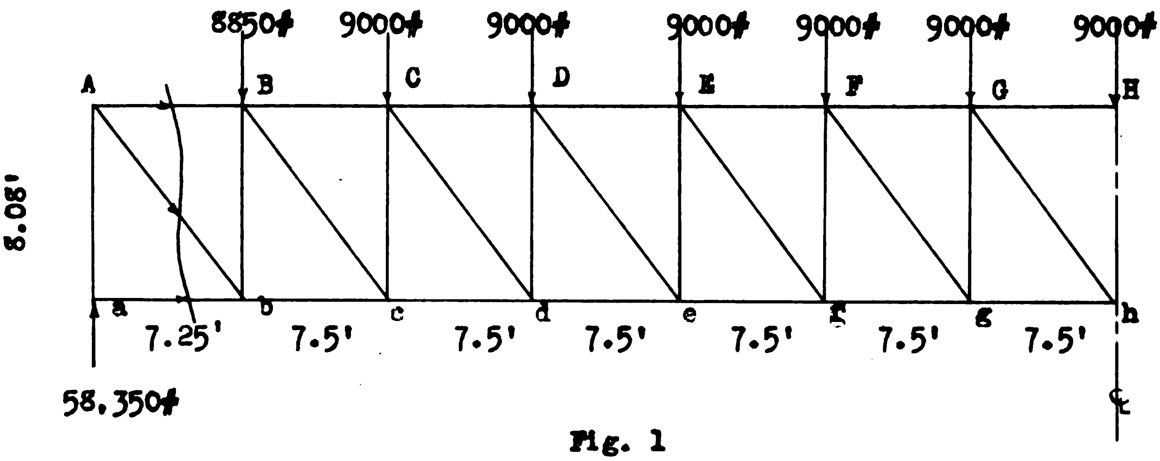



Fig. 1

L.L. + D.L./Truss = $\frac{(2476)(7.5)}{2}$ = 9000#

**Upper Chord Members** (By Method of Section)
See Fig. 1

(Example)

Cut Section through desired chord member (AB) wanted and two other members (AB & ab) take moment about where two other members cross (b) Forces to left of section

(AB)(8.08) + (58,350)(7.25) = 0

AB = 52,356 lbs. c

(BC)(8.08) - (8850)(7.5) + (58,350)(14.75) = 0

BC = 98,303 lbs. c

(CD)(8.08) - (8850)(15') - (9000) (7.5') + (58,350)(22.25) = 0

CD = 135,895 lbs. c

(DE)(8.08) + (58,350)(29.75) - (8850)(22.5) -(9000) (15) - (9000)(7.5) = 0

DE = 165,135 lbs. c

(EF)(8.08) + (58,350)(37.25) - (8850)(30) - (9000)(22.5) - (9000)(15) - (9000)(7.5)= 0

$EF = 186{,}018$ lbs c

$$(FG)(8.08) + (58{,}350)(44.75)-(8850)(37.5)-(9000)(30)-(9000)(22.5)-(9000)(15)-(9000)(7.5)= 0$$

$FG = 198{,}550$ lbs. c

$$(GH)(8.08) + (58{,}350)(52.25)-(8850)(45)-(9000)(37.5)-(9000)(30)-(9000)(22.5)-(9000)(15) -(9000)(7.5) = 0$$

$GH = 202{,}727\#$ c

$s = P/A$

$$s = \frac{202{,}727}{2(6.03)} = 16{,}810 \text{ psi} \quad (\text{allow} = 18{,}000 \text{ psi})$$

## DETERMINATION OF STRESSES, VERT. TRUSS LOWER CHORD MEMBERS

(By Method of Sect.) See Fig. 1

$(ab)(8.08) = 0$

$ab = 0$

$(bc)(8.08) - (58,350)(7.25) = 0$

$bc = 52,356$ lbs. T

$(cd)(8.08) + (8850)(7.5) - (58,350)(14.75) = 0$

$cd = 98,303$ lbs. T

$(de)(8.08) + (8850)(15) + (9000)(7.5) - (58,350)(22.25) = 0$

$de = 135,895$ lbs. T

$(ef)(8.08) + (8850)(22.5) + (9000)(15) + (9000)(7.5) - (58,350)(29.75) = 0$

$ef = 165,135$ lbs. T

$(fg)(8.08) + (8850)(30.0) + (9000)(22.5) + (9000)(15) + (9000)(7.5) - (58,350)(37.25)$

$fg = 186,018$ lbs. T

$(gh)(8.08) + (8850)(37.5) + (9000)(30) + (9000)(22.5) + (9000)(15) + (9000)(7.5) - (58,350)(44.75) = 0$

$gh = 198,550$ lbs. T

$S_{(gh)} = \frac{P}{A} = \frac{198,550}{2(4.75)}$ = 20,900 lbs. (allow 18,000 psi)

$(fg)\ S = \frac{P}{A} = \frac{186,018}{2(4.75)}$ = 19,580 lbs. (allow 18,000 psi)

## DETERMINATION OF STRESSES, VERT. TRUSS

**VERTICALS** (By Method of Shear)
See Fig. 1

Cut section just to left of vertical member (Bb) under analysis and take summation of all vertical forces to left of this cut section

(-58,350)#c = Bb

Cc = (58,350)# -8850# = 49,500#c

Dd = 58,350# - 8850 - 9000 = 40,500#c

Ee = 58,350 - 8850 - 9000 - 9000 = 31,500#c

Ff = 58,350 - 8850 - 9,000-9000-9000 = 22,500#c

Gg = 58,350-8850-9000-9000-9000-9000= 13,500#c

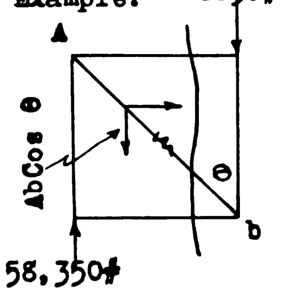
Hh = (58,350-8850-9000-9000-9000-9000-9000)2 = 9000#c

(Bb) $S = \frac{P}{A} = \frac{58,350}{6.19}$ = 9,426 psi (allow = 18,000 psi)

**DIAGONALS** -(By Method of Shear)
See Fig. 1

Cut section through diagonal under analysis and two other chord members. Take summation of Vert. forces to left of cut section.

Example: 8850#




R - abcos θ = 0

$58,350 - Ab\ \frac{8.08}{10.85}$

**Ab** = 78,851 lbs. T

$58,350\# - 8850\# - Bc\ \frac{8.08}{11.02} = 0$

**Bc** = 67,530 lbs. T

$58,350 - 8850 - 9000 - Cd\ \frac{8.08}{11.02} = 0$

**Cd** = 55,250 lbs. T

DIAGONALS (cont'd)

$$58,350 - 8850 - 9000 - 9000 - De\frac{8.08}{11.02} = 0$$

**De** = 42,974 lbs. T

$$58,350 - 9950 - 9000 - 9000 - 9000 - Ef\frac{8.08}{11.02} = 0$$

**Ef** = 30,696 lbs T

$$58,350 - 8850 - 9000 - 9000 - 9000 - 9000 - Fg\frac{8.08}{11.02}$$

**Fg** = 18,417 lbs. T

$$58,350 - 8850 - 9000 - 9000 - 9000 - 9000 - 9000 - Gh\frac{8.08}{11.02}$$

**Gh** = 6,139 lbs. T

(Ab) $S = \frac{P}{A} = \frac{78,851}{4.47} = 17,629$ psi (allow = 18,000 psi)

HAULING 12TON - 106FT. GIRDER FOR BRIDGE OVER
3177-2 RED CEDAR RIVER AT M.S.C. - JARVIS ENG. Co. 7-31-'47.

## DETERMINATION OF STRESSES, LOWER LATERAL SYSTEM (Due to Winds)

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

(300#/ft.)(7.5) = 2250#/ Panel

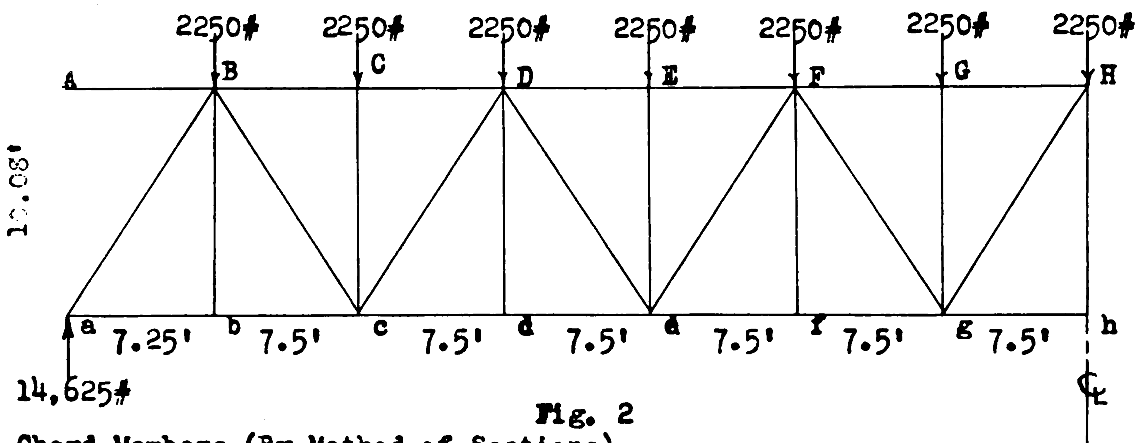



Fig. 2

Chord Members (By Method of Sections)
See Fig. 2

(AB)(10.08') = 0

AB = 0

(Bc)(10.08) + (14,625)(14.75)-(2250)(7.5) = 0

Bc = 19,726 lbs. c

(CD)(10.08) + (14,625)(14.75) - (2250)(7.5) = 0

CD = 19,726 lbs. c

(DE)(10.08)+(14,625)(29.75)-(2250)(22.5)-(2250)(15)
-(2250)(7.5) = 0

DE = 33,119 lbs. c

(EF)(10.08) + (14,625)(29.75)-(2250(22.5)_(2250)(15)
- (2250)(7.5) = 0

EF = 33,119 lbs. c

$$(FG)(10.08)+(14,625)(44.75)-(2250)(37.5)-(2250)(30) - (2250)(22.5)-(2250)(15)-(2250)(7.5) = 0$$

FG = 39,815 lbs. c

GH = 39,815 lbs. c

$$S = \frac{P}{A}$$

$$S = \frac{39,815}{2(4.75)} = 4191 \text{ psi. (allow} = 18,000 \text{ psi)}$$

## DETERMINATION OF STRESSES, LOWER LATERAL SYSTEM

Chord Members (cont'd)

$-(ab)(10.08) + (14,625\#)(7.25) = 0$

$ab = 10,519$ lbs. T

$-(bc)(10.08) + (14,625)(7.25) = 0$

$bc = 10,519$ lbs. T

$-(cd)(10.08)+(14,625)(22.25)-(2250)(15)-(2250)(7.5) = 0$

$cd = 27,260$ lbs. T

$-(de)(10.08)+(14,625)(22.25)-(2250)(15)-(2250)(7.5) = 0$

$de = 27,260$ lbs. T

$-(ef)(10.08)+(14,625)(37.25)-(2250)(30)-(2250)(22.5)$
$-(2250)(15)-(2250)(7.5) = 0$

$ef = 37,404$ lbs. T

$-(fg)(10.08)+(14,625)(37.25)-(2250)(30)-(2250)(22.5)$
$-(2250)(15)-(2250)(7.5) = 0$

$fg = 37,404$ lbs. T

$-(gh)(10.08)+(14,625)(52.25)-(2250)(45)-(2250)(37.5)$
$-(2250)(30)-(2250)(22.5)-(2250)(15)-(2250)(7.5) = 0$

$gh = 40,652$ lbs. T

(gh) $S = \frac{P}{A} = \frac{40,652}{2(4.75)} = \underline{4,278}$ psi (allow 18,000 psi)

## DETERMINATION OF STRESSES, LOWER LATERAL SYSTEM

Struts (By Method of Joints)
See Fig. 2

Bb = 0

Cc = 2250

Dd = 0

Ee = 2250

Ff = 0

Gg = 2250

Hh = 0

$$S = \frac{P}{A} = \frac{2250}{2.93} = 768\# \quad \text{(allow = 18,000 psi)}$$

Diagonals (By Method of Shear)
See Fig. 2

$$R - aB \cos \theta = 0$$

$$14,625\# - aB \frac{10.08}{12.13} = 0$$

$$aB = 17,620\# c$$

$$14,625\# - 2250 - Bc \frac{10.08}{12.56}$$

$$Bc = 15,469\# \ T$$

$$14,625 - 2250 - 2250 + cD \frac{10.08}{12.56} = 0$$

$$cD = 12,656\# c$$

$$14,625 - 2250 - 2250 - 2250 - De \frac{10.08}{12.56} = 0$$

$$De = 9,844\# \ T$$

$$14,625 - 4(2250) + eF \frac{10.08}{12.56} = 0$$

$$eF = 7,031\# \ c$$

Diagonals (cont'd)

$$14,625 - 5(2250) - Fg \frac{10.08}{12.56} = 0$$

$$Fg = 4219\# \ T$$

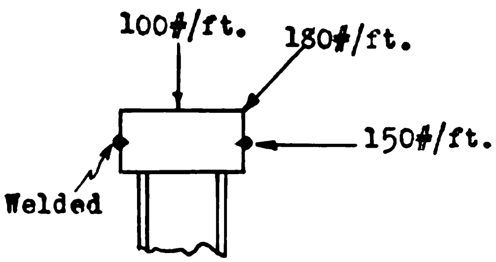
$$14,625 - 6(2250) - gH \frac{10.08}{12.56} = 0$$

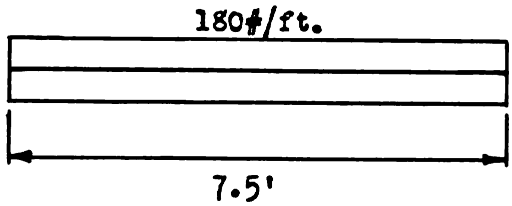
$$gH = 1400\# \ c$$

$$S = \frac{P}{A} = \frac{17,620}{2.09} = 8,430 \ \#/\square'' \quad \text{(allow 18,000 psi)}$$

## DETERMINATION OF STRESSES, RAILINGS

As per Specification 1 the top of the railings is higher than the 3 ft. minimum 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#/ft, as shown below.




Unit stresses in railing were computed as follows:

Shear in railing: $V = 1/2\,(180)(7.5) = 675.0\#$

Stress due to shear: $S = \frac{V}{A} = \frac{675.0}{6.03} = 112$ psi.

Moment in railing:

$$M = 1/8\,(180)(7.5)^2 = 1266 \text{ ft. lbs.}$$

$$S = \frac{Mc}{I} = \frac{M}{Z}$$

$$= \frac{(1266)(12)}{1.7} = 8{,}936 \text{ psi (allow } 14{,}500 \text{ psi)}$$

## DETERMINATION OF STRESSES, END BEARING

Refer to Specifications 10 to 15 inclusive, for the design of the end bearings. The specifications are followed very closely. Expansion must be allowed for at the rate of $1\frac{1}{4}$ inches for every 100 feet or 1.3 inches for this span of 104 ft. 3 in. are allowed. Bronze sliding expansion bearings are provided. The anchor bolts prevent any lateral moment. The bolts extend into the masonry the required 12". The truss is supported on metal plates so that the bottom chord is 6 inches above bridge seat. The base plate is 12" x 15", giving a pressure on the masonry of

$$\frac{(1576\#)(104')}{2} = 81{,}952\# \text{ on each truss}$$

$$\frac{81{,}952\#}{12'' \times 15''} = 455 \text{ psi (allow 1000 psi)}$$

## SUMMARY OF STRESSES

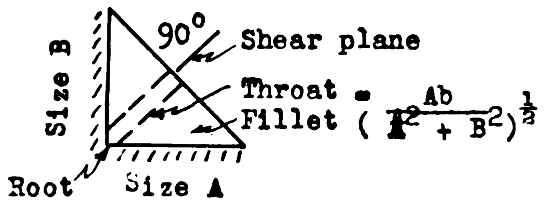
The unit stress for each member, found by assuming the application of a live load of 100 pounds per square foot of sidewalk area, are listed below. The allowable unit stress, according to the A.A.S.H.O specifications, are listed opposite for comparison. All stresses are in pounds per square inch

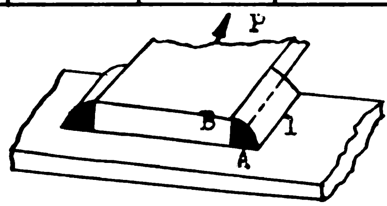
| Member under Consideration | Stress as found | Allow Stress |
|---|---|---|
| T-Tri-Lok Flooring (Concrete) | 405 | 700 |
| (Steel) | 3888 | 18,000 |
| Floor Beam (due to shear) | 1,549 | 11,000 |
| Floor Beam (due to moment) | 16,024 | 18,000 |
| Vertical Truss (using largest stressed member) | | |
| Upper Chord | 16,810 | 18,000 |
| Lower Chord | 20,900 | 18,000 |
| Verticals | 9,426 | 18,000 |
| Diagonal | 17,629 | 18,000 |
| Lower Lateral System (largest stressed member) | | |
| Chord (windward side) | 4,191 | 18,000 |
| Chord | 4,278 | 18,000 |
| Struts | 768 | 18,000 |
| Diagonal | 8,430 | 18,000 |
| Railing | 8,936 | 14,500 |
| End Bearing (masonry) | 455 | 1,000 |

## INVESTIGATION OF WELDS

"Code of Fusion Welding" of the American Welding Society specifies the following safe working strengths per linear inch for fillets of various sizes:

| Size (A = B) | 1/4" | 5/16" | 3/8" | 1/2" | 5/8" | 3/4" |
|---|---|---|---|---|---|---|
| Strength, lbs/inch | 2,000 | 2,500 | 3,000 | 4,000 | 5,000 | 6,000 |




Lower Lateral System (See Stress Diagrams and Detail Sheet)

Diagonal aB on gusset plate V and P

Total stress in Diagonal aB = 17,620#

Req'd length of 5/16" fillet = $\frac{17,620 \text{ lb.}}{2500}$ = 7.5"

(V) actual length = 10.8"

(P) actual length = 12"

Diagonal Bc on gusset plate P and t

Total stress in diag. = 15,469#

Req'd length of 5/16" fillet = $\frac{15,469 \text{ lbs.}}{2500}$ = 6.2"

(P) actual length = 8.4"

(t) actual length = 8.4"

(other diagonal welds are repetition of Diagonal Bc on gusset plate t)

Struts

Total stress in strut Cc = 2250#

Req'd length of 1/4" fillet = $\frac{2250}{2000}$ = 1.1"

(P) actual length = 9.6"

(t) actual length = 12"

(all other struts welds are repetition of this, and struts taking no stresses require no definite length of weld.

VERTICAL TRUSS SYSTEM

(see stress diagrams & detail sheet)

Diagonals (Gusset plate on each side)

Total stress in diagonal Ab = 78,851

Reg'd length of 5/8" fillet = $\frac{78,851}{5,000}$ = 15.7"

Reg'd length each side = 15.7/2 = 7.85"

(b) actual length = 14.4" (one side)

(f) actual length = 19.2" (one side)

Total stress in diagonal Bc = 67,530 lbs.

Reg'd length of 5/8" fillet = $\frac{67,530}{5000}$ = 13.5"

Reg'd length each side = 13.5/2 = 6.75"

(c) actual length = 12" (one side)

(c) actual length = 12" (one side)

Verticals

Total stress in vertical Bb = 58,350#

Reg'd length of 3/8" fillet = $\frac{58,350}{3000}$ = 19.4"

Reg'd length each side plate = 19.4/2 = 9.7"

Actual length on plate cl = 21.0"

Actual length on plate f = 29"

Total stress in Vertical Cc = 49,500#

Req'd length of 3/8" fillet = $\frac{49,500}{3000}$ = 16.5"

Req'd length per side = 8.25"

Actual length on plate c = 13"

(All remaining vertical welds are identical to the above)

All welds are well on the safe side and are for the most part very much under designed. Approximately 1/4" is added to the computed length necessary, to allow for starting and stopping the arc

## INVESTIGATION OF DEPTH RATIO

The width of the truss, according to Specification 9 should not be less than 1/10 the length of the span.

Minimum Depth Ratio 1/10 x 104 = 10.4' = 124.8 inches

Minimum Depth at center 8 ft. 6 in = 102.0 inches

SWINGING THE 12 TON - 106 FT. GIRDER ACROSS THE RED CEAAR RIVER AT MICH. STATE COLLEGE.

37033

# CONCLUSION

The main question in everyone's mind is, "Does this bridge satisfy 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, and if so the allowable stresses are not exceeded. However, the author feels that the third edition of these same said specifications apply more to this bridge,as explained previously in the investigation of live loads. The allowable stresses are still not exceeded as shown previously.

I am comparing 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 as shown on the preceeding page. It will be noted that the two greatest stressed lower chord members in the vertical truss were a trifle high for the allowable, but are considered to be within a reasonable limit, and thereby are safe. It is possible that the author might have used too high a value for the live load. 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 stresses in the chord members well below the allowable.

The reason for the large difference in the utility loads used by the author and the designing engineer is evident when the designing conditions are known. The designing engineer did not have a definite knowledge of just what type and size of utilities were to be carried by the bridge. 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. The possibility of the structure ever receiving such extreme loads is so rare, that the engineer was quite justified in not changing his design.

The main disagreement of the design of this bridge with the specifications is the depth ratio. Specifications call for a depth of 1/10 of the span which for this bridge would be 10.4 feet. The actual design was 8.5 feet deep. The author feels that since the bridge is such a short span, and that the entire bridge is under-designed enough to compensate for the difference. This specification would apply more to a long span truss bridge, which would receive greater wind loads and would have more live and dead loads which would tend to make the bridge jack-knife in the middle. Deflection will also enter into this matter also. Upon talking to Mr. Hepplewhite, the designing engineer, I found his opinion to be that the bridge was ample in depth for that length of span.

The welded connections on the web members of the vertical truss were done at the Jarvis Engineering factory, and were analyzed in the previous pages to be of great enough strength to resite any shear or bending which might occur in the bridge. The field welding of the lower lateral system also checked within the limits.

The 3/16" plate which is welded to the inside of the vertical floor down to lower chord members was not considered in the stresses of this bridge, as this was placed there for the purpose of hiding the utilities beneath the bridge. The author, however, did realize that some of the stress is taken up by this plate, but since the designing engineer ignored this in his design, the author did likewise. The author did include the weight of the plate in determining the dead load of the bridge.

Considering that a primary objective 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 objective fulfilled. The author was surprised concerning the many special problems presented by a footbridge. This thesis brought out quite clearly the importance and difficulty which arises in choosing of specifications and applied loads.

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EXCERPTS FROM SPECIFICATIONS

ıbstantial railings along each side of the bridge
ided for the protection of traffic. The top of the
be not less than 3 feet above the finished surface
y adjacent to the curb, or if on a sidewalk, not
eet above sidewalk floor.

shall be designed to resits a horizontal force of
150#/lin. ft. of bridge, applied at the top of the
a vertical force of not less than 100#/linear ft.

he dead load shall consist of the weight of the
plete including the roadway, sidewalks, car tracks,
ts, cables, and other public utility services.

and ice load is considered to be offset by an ac-
crease of live load and impact and shall not be in-
under special conditions.

owing weights are to be used in computing the dead

- - - - - - - - - - - - - - - - - - - - - - 490 lbs./cu. foot
in or reinforced - - - - - - - - - - - 150 lbs./cu. foot
arth & gravel - - - - - - - - - - - - 100 lbs./cu. foot
d, earth, gravel or ballast - - - - 120 lbs./cu. foot

he live load shall consist of the weight of the ap-
load of vehicles, cars or pedestrians.

ing - Sidewalk floors, stringers and their immedi-
shall be designed for a live load of 85 lbs. per
f 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 ft.)

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

P = Live Load per sq. ft. (Max. 60 psf)

L = Loaded length of sidewalk in feet

W - Width of sidewalk in feet

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 $1\frac{1}{2}$ 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.

8. Compression Members - shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

9. Depth Ratio - The ratio of the depth to the length of span, for trusses, shall be not less than 1/10.

10. End Bearings - Expansion ends shall be firmly secured against

lifting or lateral movement. Fixed bearings shall be firmly anchored. Spans of less than 70 ft. may be arranged to slide upon metal plates with smooth surfaces. Spans of 70 ft. or more shall be provided with rollers or rockers, or else with bronze sliding expansion bearings.

11. Expansion - provision shall be made for expansion and contraction at the rate of $1\frac{1}{4}$" for every 100 ft. The expansion ends shall be secured against lateral movement.

12. Anchor Bolts - Anchor bolts for trusses and girders shall not be less than $1\frac{1}{2}$" in diameter and shall extend into the masonry not less than 15 inches. Washers shall be used under the nut.

13. Sole Plates - Sole plates of girders and trusses shall not be less than 3/4" thick.

14. Bronze or Copper Alloy Sliding Expansion Bearings- Bronze or copper alloy sliding plates shall be chamfered at the ends. They shall be held securely in position, usually by being inset into the metal of the pedestals and sole plates. Provision shall be made against any accumulation of dirt which will obstruct free movement of the span.

15. Allowable Bearing on Masonry - Bridge seats - under hinged rockers and bolsters (not subjected to high edge loading by a deflecting beam, 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 above amount.

16. Thickness of Metal - The minimum thickness of structural steel shall be 5/16 inch, except for fillers, railing and unimportant details. Gusset plates shall be not less than 3/8 inch thick

17. Floor beams - Floor beams, preferably, shall be at right angles to the trusses, and shall be rigidly connected thereto.

18. Lateral Bracing - Half through truss spans shall have top and bottom lateral bracing.

19. Half Through Truss Spans - The vertical truss members and the floor beam connections of half-through truss spans shall be proportioned to resist a lateral force, applied at the top chord panel points of the truss, determined by the following equation:

$$R = 150 ( A + P )$$

R = Lateral force in lbs.

A = Area of cross section of chord (D")

P = Panel length in feet

20. Camber - The length of the truss members shall be such that the camber will be equal to or greater than the deflection produced by the dead load.

21. Number of Trusses or Girders - Preferably, through spans shall have only two trusses, arches or girders.

22. Spacing of Trusses and Girders - Main trusses shall be spaced a sufficient distance apart center to center, to be secure against overturning by the assumed lateral forces.

23. Effective Span - For the calculation of stresses span lengths shall be assumed as follows:

Trusses, distance between centers of bearings

Floorbeams, distance between centers of trusses

24. Effective Depth - For the calculation of stresses, the effective

depth of a truss shall be assumed as, the distance between centers of gravity of the chords.

25. Fillers (welded)- Shall be designed according to specifications of the American Welding Society, "Welded Highway and Railway Bridges".

26. Permissible unit stresses (lbs/sq. inch) allowable compression in splice -

material, gross section - - - - - - - - - - - - - - - 18,000

Shear in girder webs, gross section - - - - - - - - - 11,000

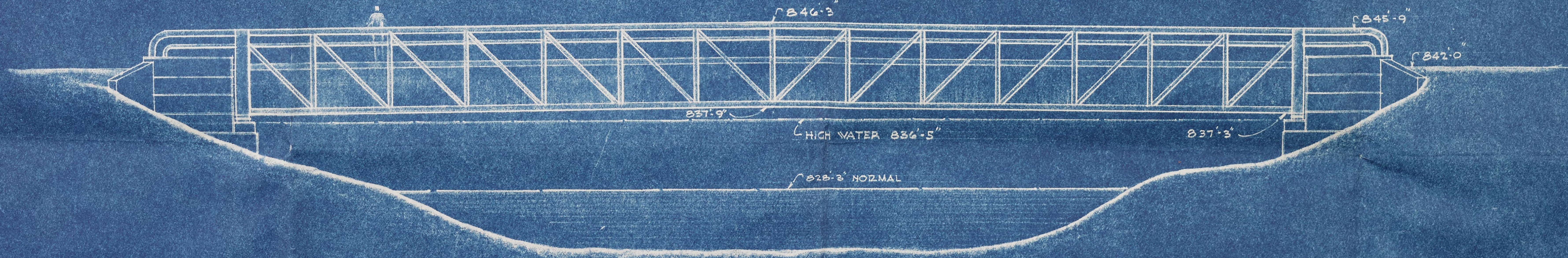
Diagonal tension in webs of girders and rolled beams, at sections where maximum shear and bending occur simultaneously - - - - - - 18,000

Bearing on expansion rollers and rockers, pounds per linear inch:

Diameters up to 25 inches - - - $\frac{P-13,000}{20,000}$ 600d

d = diameter of roller or rocker, in inches

P = Yield point in tension of steel in the roller or the base whichever is the lesser




# TRUSSED UTILITY & FOOT BRIDGE

MICHIGAN STATE COLLEGE

SCALE 1/8" = 1'-0"

CLAUD R. ERICKSON
CONSULTING ENGINEER
LANSING

4-23-47

Pocket has:
2 Blueprints
sheet no. 1
sheet no. 2
1 Plan

3 1293 03150 6946

1293 03150 6953