A DESIGN CHECK OF A TYPICAL 200 FOOT THROUGH RAILWAY TRUSS SPAN

Thesis for the Degree of B. S.

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A Design Check of a Typical 200 Foot Through Railway Truss Span

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by

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Candidate for the Degree of
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Diagrams and Tables

I. INTRODUCTION

The railway bridge, the design of which is to be checked herein, is a single track through Warren type truss with vertical members and riveted connections. It is a straight single span 200 feet long, consisting of eight 25 foot panels, and a height of 32 feet. The distance between center of trusses is 18 feet-2 inches and between stringer is 7 feet.

The plans from which this analysis is made were furnished by the American Bridge Company, Gary, Indiana and consist of:

D-2 : General Design (Sheet 2 of 4)
L-1 : 200' Truss Span Joints
L-2 : 200' Truss Span Joints
L-3 : Portal, Top Chord Bracing, Sway Frames
L-4 : Bottom Chord Bracing and Floor System

As no information is at hand regarding the particular specifications used in the original design, the "Specifications for Steel R ailway Bridges" as published in 1935 by the American Railway Engineering Association will be used as a guide. Any features found not to agree with the above mentioned specifications will be noted and, if possible, an explanation presented to justify the discrepancy.

II. GENERAL FEATURES

MATERIALS:

All structural steel has been used with inserts of USS 12 specified at expansion shoe. This is in agreement with specifications.

TYPE OF BRIDGE:

A riveted truss is the preferred type of bridge for spans 100 feet or longer.

SPACING OF TRUSSES AND GIRDERS:

Trusses: Spaced 18'-2" c.c.
Length of span is 200'-0" c.c. end bearing.
R = 18.17 ÷ 200 = 0.091 o.k.(0.05 min.)

Stringers: 2 stringers used @ 7'-0" o.k.(6'-6" min.)

DEPTH RATIO:

Depth of truss is 32'-0" c.c. of chords

Ratio = 32 ÷ 200 = 0.16 o.k.(0.10 min.)

CLEARANCE:

Designed clearances are found to be either equal to or greater than specified clearances. (See fig. 1).

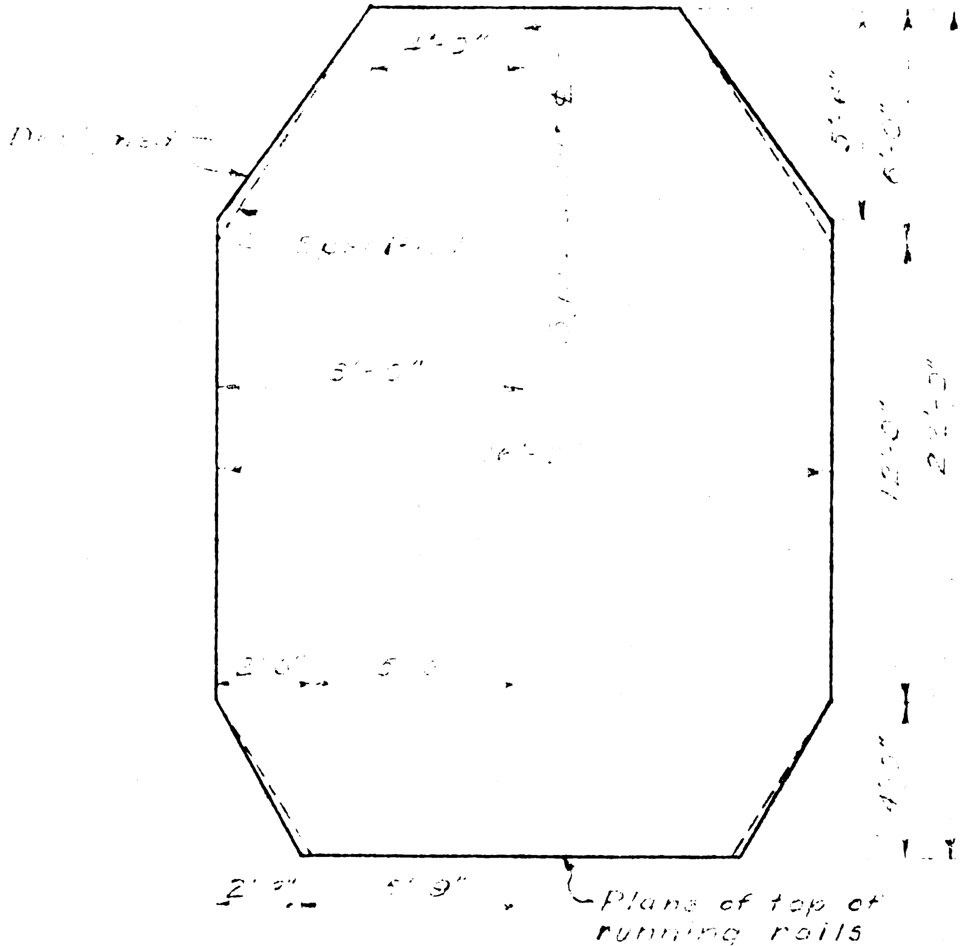


Fig. 1. Clearance

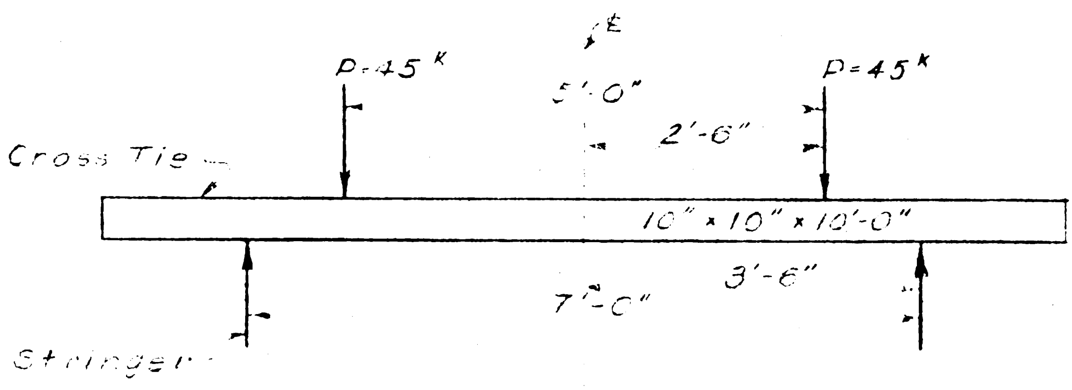


Fig. 2. Floor System

III. LOADS

DEAD LOAD:

Track rails, guard rails & fastenings	= 200 lbs./ft.
Timber ties @ 60 lbs./cu.ft. = (0.75)(7)(60)	= 315 " "
Steel; $w = k(9L + 700) = (1.264)(1800 + 700)$	= 3160 " "
Total dead load	= 3675 lbs./ft.
Assumed total dead load	= 3700 lbs./ft.
Total dead load per truss = $(\frac{1}{2})(3700)(200)$	= 370 kips.
End reaction per truss = $(\frac{1}{2})(370)$	= 185 kips.

It should be noted that the formula, $w = k(9L + 700)$ as found in the "American Civil Engineers' Handbook," 5th edition, is based on an allowable unit stress of 16,000 psi. and therefore will produce a conservative result in that present day practice permits an unit stress of 18,000 psi.

LIVE LOAD:

The live load to be used is the Cooper E-72 as is recommended in the specifications.

IMPACT:

Numerical values for impact are computed by the use of the following formulas:

(a) The lurching effect: A percentage of the static live load stress equal .... $\frac{100}{S}$

S = spacing, in feet, between centers of stringers or trusses; or length, in feet, of floor beams.

(b) The direct vertical effect: A percentage of the static live load stress equal to:

For "L" less than 100 feet............. 100-0.60L

For "L" 100 feet or more...............$\frac{1800}{L-40} + 10$

"L" = length, in feet, center to center of supports for stringers and trusses (chords and main members); or length of floor beams, in feet, for floor beams and floor beam hangers.

IV. FLOOR SYSTEM

The floor system consists of wooden ties placed on stringers as shown in fig. 2. Specifications state that the maximum wheel load on one rail is uniformly distributed over three ties, and is applied without impact. The recommended maximum wheel load is 45,000 lbs. The moment due to this load is:

$$M = \frac{45000}{3} \times \frac{7-5}{2} \times 12 = 180{,}000 \text{ in.lbs.}$$

The moment due to weight of the floor is very small compared to the live load moment and is usually neglected. Cross ties are assumed to be of White oak with an allowable extreme fiber stress in bending equal to 1200 psi. The required section modulus of the tie is

$$\frac{bh^2}{6} = \frac{180{,}000}{1200} = 150 \text{ cu.in.}$$

For a 10 x 10 in. tie, the section modulus equals 167 cu. in. which meets the requirements. Length of tie equals 10'-0" which also agrees with specifications. They are to be spaced not more than six inches apart and secured to prevent bunching.

V. STRINGERS

The stringers are designed as simply supported beams equal in length to the distance between centers of floor beams which, in this case, is 25'-0". In the design, the stringers are 33CB240 with end connections to floor beams of 2 - $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 in. angles 2'-$3\frac{1}{2}$" long.

DEAD LOAD:

Assumed weight of stringers = $1.264[(12.5)(25) + 100]$.... = 522 lbs./ft.

Weight of floor .... = 515 " "

Total dead load.. = 1037 lbs./ft.

Dead load per stringer .... = 518 lbs./ft.

Dead load per stringer used in design.. = 600 " "

MAXIMUM MOMENT:

Dead load center moment = $\frac{600}{8} \times 625$ ... = 47 ft. kips

Live load moment E-72 = $\frac{72}{50} \times 381.3$ .... = 550 " "

Impact = 99.3% of 550 .... = 546 " "

Total maximum moment = 1143 ft. kips

Required section modulus, $S = \frac{1143}{18} \times 12 = 762$ cu.in.

Section modulus of a 33CB240 equals 811 cu. in., therefore, section meets requirements.

END SHEAR:

Dead load $= \frac{600}{2} \times 25$ .... $=$ 7.5 kips

Live load E-72 $= \frac{72}{50} \times 71$ .... $=$ 102.0 "

Impact $=$ 99.3% of 102.0 .... $=$ 101.0 "

Total end shear $=$ 210.5 kips

Unit shearing stress in web $= \frac{210500}{30.7 \times 0.83}$ .....$=$ 8250 psi.
o.k. (allowable-11,000 psi.)

FLANGE BUCKLING:

Ratio $\frac{L}{b} = \frac{25 \times 12}{15.9} = 18.9$ o.k. (allowable $=$ 40)

Allowable unit compressive stress in flange $=$
$18{,}000 - 5(18.9)^2 = 16{,}210$ psi.

Effective area $= (15.9)(1.4) + \frac{25.5}{8}$ ....$=$ 25.44 sq.in.

Actual unit compressive stress $= \frac{1143000}{25.44} \times \frac{12}{32.4} = 16{,}600$ psi.

This shows a slight overstress in the compressive flange but this is negligible as the formula used to calculate allowable stress is empirical and conservative; and also the restraining effect of the timer ties has been neglected.

DIAGONAL WEB BUCKLING:

Ratio $\frac{h}{t} = \frac{30.7}{0.83} = 37$. This is less than 70 and is considered very safe.

END CONNECTION:

The connecting angles are 6 x 8 x 5/8 in. angles 2'-3½" long with the 8 in, leg against the web of the stringer. The number of rivets through the floor beam web is controlled by single shear at 8.1 kips per rivets. The number of rivets required is $\frac{210.5}{8.1} = 26$ rivets. Detail sheet shows 26 rivets which meets the requirements.

LATERAL BRACING:

The stringers are provided with lateral bracing consisting of 3½ x 3½ x 3/8 in. angles arranged as shown in figure 3. Each 25 foot panel is divided by cross frames consisting of 2-3½ x 3½ x 3/8 in. angles and a 24 x 3/8 plate. This agrees with specifications which limits the length of panel without cross frames to 20 feet. The lateral forces carried by this bracing is due to a wind force on the train of 300 lbs. per linear foot of bridge plus one-half of the wind force on the truss, or 840 lbs. per lin. foot. The total force carried by stringer bracing is 1140 lbs. per lin. ft. As the same size angles are used throughout, only that member having the maximum compressive stress need be investigated. In this case, the greatest stressed member is the diagonal "ab". The force is considered a moving load and is applied at the panel points. The load per panel point equals 1140 x 12.6 or 14.25 kips and the shear on section y-y equals 7.13 kips. Diagonal "ab" will have a compressive stress of 7.13 x 88.5/66 = 9.55 kips.

The unsupported length of the member is 88.5 inches and the least radius of gyration of a $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 inch angle is 1.07 inches. The ratio L/r = 88.5/1.07 = 82.6. The allowable unit stress is

$$15000 - (0.25)(82.6)^2 = 15000 - 1700 = 13,300 \text{ psi.}$$

The area required for member "ab" is 7.13/13.3 or 0.54 sq. inch. One $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 in. angle has an area of 2.48 sq. in. which is excessive but it is the smallest angle generally used for this purpose.

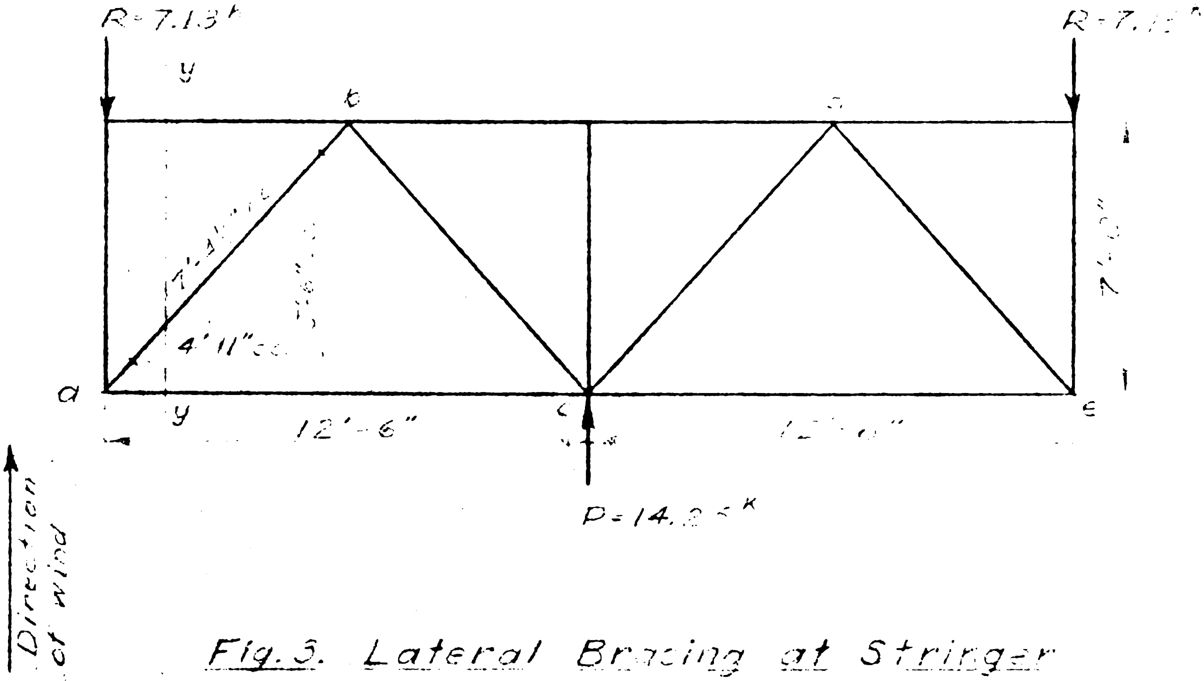


Fig. 3. Lateral Bracing at Stringer

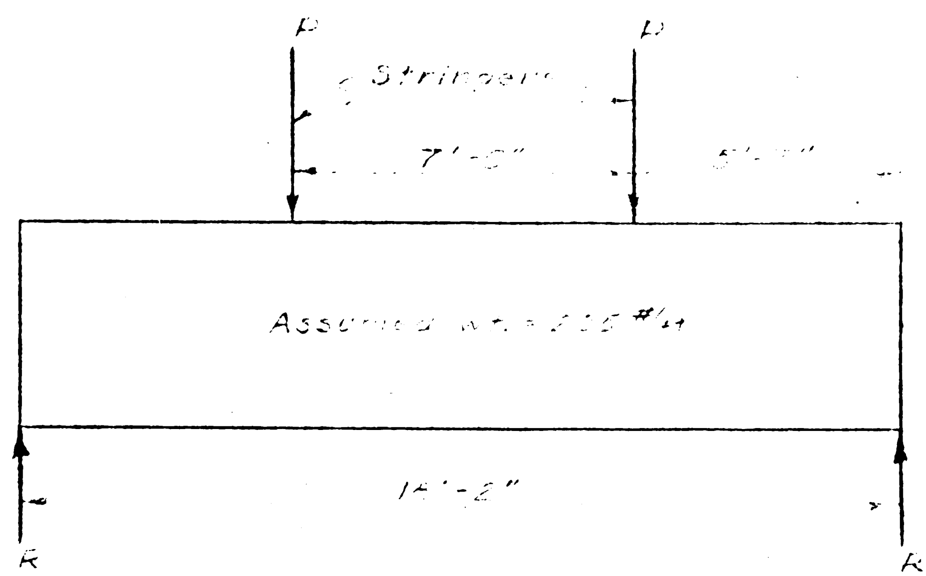


Fig. 4. Intermediate Floor Beam Loading

VI. INTERMEDIATE FLOOR BEAMS

At the panel points are located intermediate floor beams built up of 4 - 6 x 6 x 7/8 in. angles and a 52 x 5/8 in. web. In calculating the stresses, the length of beam is taken as the distance between centers of trusses which is 18'-2" in this cas e. The floor beams transmit to the trusses their own weight plus two concentrated loads from the stringers, as shown in figure 4. The dead load consists of the weight of the floor beam assumed at 225 lbs. per lin. ft. and the weight of one stringer equal to 15000 lbs.

MAXIMUM SHEAR:

Dead load = 18.17 x 225/2 + 15000 .... = 17 kips

Live load = stress in hanger .... = 136 "

Impact = 95% of 136 .... = 130 "

Total shear = 283 kips

MAXIMUM MOMENT:

Dead load = $\frac{(225)(18.17)}{8}$ + (15000)(5.58).... = 93 ft.kips

Live load = 136 x 5.58 .... = 760 " "

Impact = 95% of 760 .... = 720 " "

Total moment = 1573 ft.kips

CHECK OF SECTION:

Required web area = 283/11 .... = 25.8 sq.in.

Furnished web area = 52 x 0.625 .... = 32.5 sq.in. o.k.

Minimum thickness of web plate = 40.5/170.. = 0.24 in.

Actual thickness of web plate .... = 0.625 in. o.k.

Required section modulus = $\frac{1573}{18}$ x 12 ...... = 1050 cu. in.

Total net moment of inertia of section I... = 28030 $in.^4$

Furnished section modulus = $\frac{I}{c} = \frac{28030}{26.25}$ ..... = 1065 cu.in. o.k.

COMPRESSION FLANGE:

Ratio $\frac{L}{b} = \frac{15.59 \times 12}{12.625}$ .... = 14.8 in.

Allowable unit stress = $18{,}000-5(14.8)^2$ .... = 16,900 psi.

Effedtive depth = 52.2-3.64 .... = 48.86 in.

Total flange stress = $\frac{1573 \times 12}{48.86}$ .... = 387 kips

Total compressive area required = 387/16.9. = 22.9 sq.in.

Including 1/8 web area as effective flange area, the total effective area supplied is 19.46 + 4.06 = 23.52 sq.in. o.k.

FLANGE RIVETS:

The total flange stress is assumed to be the same as that at the center of the floor beam which is 387 kips as calculated above. Due to the assumption that the web carries a portion of this stress, the total flange stress may be reduced by the ratio of net flange area to total moment-carrying area. Therefore, the stress in the angles is

$$s = 387 \times \frac{17.49}{21.55} = 314 \text{ kips}$$

The number of rivets required between stringer and end is controlled by bearing on the 5/8" plate and is 314/14.75 or 22 rivets. Plans show 34 rivets being used which is more than enough. The shear between stringers is so small that the rivets are spaced at 6 inches which is the maximum pitch.

INTERMEDIATE STIFFENERS:

Stiffeners are required where the depth of web between flange angles exceeds 60 times its thickness. In this case, the depth of web is 40.5 inches which is greater than the 60 x 0.625 or 37.5 inches. Therefore, stiffeners are required. The spacing of stiffeners as calculated by the formula specified is found to be 94 inches but 72 inches is the maximum spacing permitted. Stiffeners consisting of 2 - $3\frac{1}{2}$ x 5 x 3/8- in. angles have been placed 3'-3" from each end and at the midpoint. These are conservatively placed but this is probably due to need to place one stiffener between the stringer and the end which would require that one be placed between stringers. The midpoint of span is the best place to locate the stiffener between stringers to preserve symetry. Width of outstanding leg of each angle is 5 inches, which is less than 16 times its thickness and greater than 2 + 52.5/30; therefore, size of angle is alright.

END CONNECTIONS:

The connecting angles are 6 x 4 x 5/8-inches with the 6-inch leg riveted with a 10 x 7/8-in. fill plate to the web of the floor beam. The required number of rivets is controlled by bearing on the 5/8-inch web and equals 283/14.75 or 20 rivets. Plans show 26 rivets being used which is sufficient.

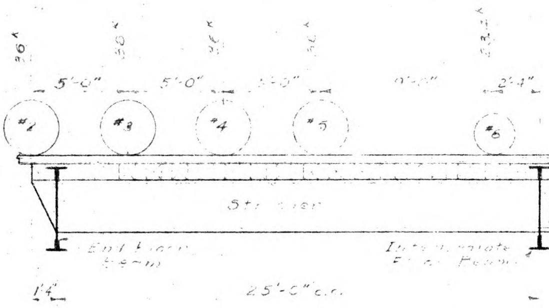


Fig. 5. End Floor Beam - Live Loading

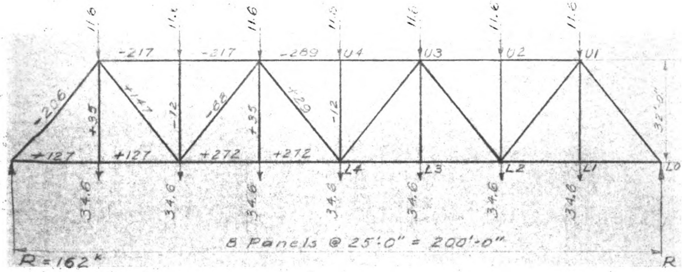


Fig. 6. Dead-load Stresses in Truss

VII. END FLOOR BEAM

The stringers in the end panels are supported by means of end floor beams consisting of 4 - 6 x 6 x 3/4-in. angles and a 52 x ½-in. web plate. The end beam is connected by gusset plates on the foot of the end posts. Stringer brackets are designated for the end floor beams. The dead load is composed of the weight of the floor beam, assumed to be 3000 lbs. and the end reaction per stringer equal to 7500 lbs.

MAXIMUM SHEAR:

Dead load = 1500 + 7500 .... = 9.0 kips

Live load = (See figure 5) .... = 108.5 "

Impact = 94.6% of 108.5 .... = 102.5 "

Total shear... = 220.0 kips

MAXIMUM MOMENT:

Dead load = 7.5 x 5.6 + 18.17 x 3.0/8 ..... = 49 ft.-kips

Live load = 108.5 x 5.6 .... = 609 " "

Impact = 94.6% of 609 .... = 576 " "

Total moment =1234 ft.-kips

CHECK OF SECTION:

Required web area = 220/11................ = 20.0 sq.in.

Actual web area = 52 x 0.5 .... = 26.0 " " o.k.

Required web thickness..................... = 0.24 in.

Actual web thickness .... = 0.50 in. o.k.

Required section modulus = 1234 x 12/18 .... = 822 cu. in.

Total net moment of inertia .... = 23,962 $in.^4$

Actual section modulus = $\frac{I}{c}$ = $\frac{23,962}{26.25}$ .... = 912 cu.in. o.k.

Flange section ratio L/b = 18.17 x 12/12.5 . = 17.45 in.

Allowable stress = $18,000-5(17.45)^2$ .... = 16,500 psi.

Effective depth = 52.5 - 3.56 .... = 48.94 in.

Total flange stress = 1234 x 12/48.94 ...... = 303 kips

Required total area = 303/16.5 .... = 18.4 sq.in.

Including 1/8 web area as effective in compression, the actual area is = 16.88 + 3.25 .... = 20.13 sq.in. o.k.

FLANGE RIVETS:

Stress in angles between stringer and end is equal to 303 x 15.19/18.44 or 250 kips. The required number of rivets as controlled by bearing on the ½" web is 250/11.8 or 22 rivets. Plans show 27 rivets are used. The shear between stringers is so small that the rivets are again spaced at 6 inches.

INTERMEDIATE STIFFENERS:

Stiffeners are required because the depth of web, 40.5", is greater than 60 times the thickness or 30 inches. The spacing of stiffeners as calculated by the formula specified is found to be 71". Plans indicate a stiffener of 2-3½ x 5 x 3/8-in. angle placed at center of span. Since the end floor beam is to be used as a jacking beam, the stiffener placed at bearing point which is 3'-6" from center line of truss, must be designed to transmit the concentrated load of 185 kips (see general design sheet D-2). The moment caused by this load is 185 x 3.5 or 648 ft. kips. Specifications allow that unit design stresses may be increased by 50% for such cases. Therefore, the required effective area is 185/27 or 6.85 sq.in. The design sheet indicates 4 - 5 x $3\frac{1}{2}$ x $\frac{1}{2}$-in. angles are to be used. These angles furnish a gross area of 4 x 4.00 or 16.00 sq.in. and an effective area of 4 x $4\frac{1}{2}$ x $\frac{1}{2}$ or 9.00 sq.in. which is ample. The outstanding leg of the angles is 5-inches which meets specifications in that it is less than 16 x $\frac{1}{2}$ or 8 inches, and more than 2 + 52.5/30 or 3.75 inches.

Here it should be noted that although the design sheet indicates that the stiffener at the jacking point should consist of 4 - 5 x $3\frac{1}{2}$ x $\frac{1}{2}$-in. angles, the detail sheet, L-4, shows only two such angles to be used. The reason for this discrepancy is not immediately apparent so the detail sheet will be assumed to be in error.

The size of stiffener at center of beam is not checked because the shear value is so small. The size was chosen to match the other stiffener angles.

END CONNECTION:

The end connection consists of 2 - 4 x 4 x 5/8-in. angles and 2 - 7 x 3/4-in. fill plates. The outstanding leg of the angle meets the specifications in that it is less than 16 times its thickness or 10" and more than

2 + 52.5/30 or 3.75 inches. Number of rivets required is controlled by bearing on the ½-inch web and equals 220/11.8 or 19 right. Detail plan shows 13 rivets in the angles and 9 rivets in the fill plate or a total of 22 rivets, which is considered sufficient.

VIII. TRUSS

The truss under consideration is of the Warren type with vertical members. The span is 200'-0" center to center of end bearings and is divided into 8 equal panels 25'-0" long. As the truss is symmetrical about the center vertical, stresses will be calculated for one-half of the truss only.

DEAD-LOAD STRESSES:

From the assumed dead load of 3700 lbs. per linear foot, the dead panel load per truss is 25 x 3700/2 or 46.2 kips. Generally, this panel load is assumed to be distributed two-thirds to each lower chord joint, and one-third to each top chord joint. However, in this case, it appears that this distribution has been assumed as three-fourths of the dead panel load or 34.6 kips at each lower chord joint and one-fourth, or 11.6 kips, at each top chord joint (see fig.6). The reaction at one end of the truss due to these loads is 46.2 x 7/2 or 162.0 kips.

The stresses are determined by the general methods of statics. The chords stress is determined by dividing the algebraic sum of the moment on one side of the section about the opposite chord point, "M", by the height of the truss, "h". The web stresses are found by multiplying the shear on the section, "V", by the secant of the angle which the member makes with horizontal. The secant of the angle in this case equals 40.6/32 or 1.27.

LO-U1 = V sec Θ = -162 x 1.27 .... = -206. kips

U1-U3 = M/h = -6940/32 .... = -217 "

U3-U3 = M/h = -9255/32 .... = -289. "

LO-L2 = 162 x 25/32 .... = 127. "

L2-L4 = M/h = +8670/32 .... = 272. "

U1-L2 = V sec Θ = 115.7 x 1.27 .... = 147. "

L2-U3 = V sec Θ = -69.4 x 1.27 .... = -88. "

U3-L4 = V sec Θ = 23.1 x 1.27 .... = 29.4. "

Hangers = by method of joints .... = 34.6. "

Verticals = by method of joints .... = -11.6. "

Reaction = see section III, Loads .... = 185.0. "

LIVE-LOAD STRESSES:

The maximum stresses due to the assumed live loading of the Cooper E-72 type are calculated by use of influence lines for stress (see Figure 7). The position of the load which would produce the maximum stress is first determined, then the stress equals the summation of the wheel load times the ordinate of the influence line under each load. The method of computation will be carried out for one member, after which only a summary will be presented for each member.

Member LO-U1: with train moving toward the left--

;place wheel #3 @ right of L1: $\frac{54}{25}$ is less than $\frac{741}{175}$

;place wheel #3 @ left of L1: $\frac{90}{25}$ is less than $\frac{705}{175}$

;place wheel #4 @ right of L1: $\frac{90}{25}$ is less than $\frac{723}{175}$

;place wheel #4 @ left of L1: $\frac{126}{25}$ is greater than $\frac{687}{175}$

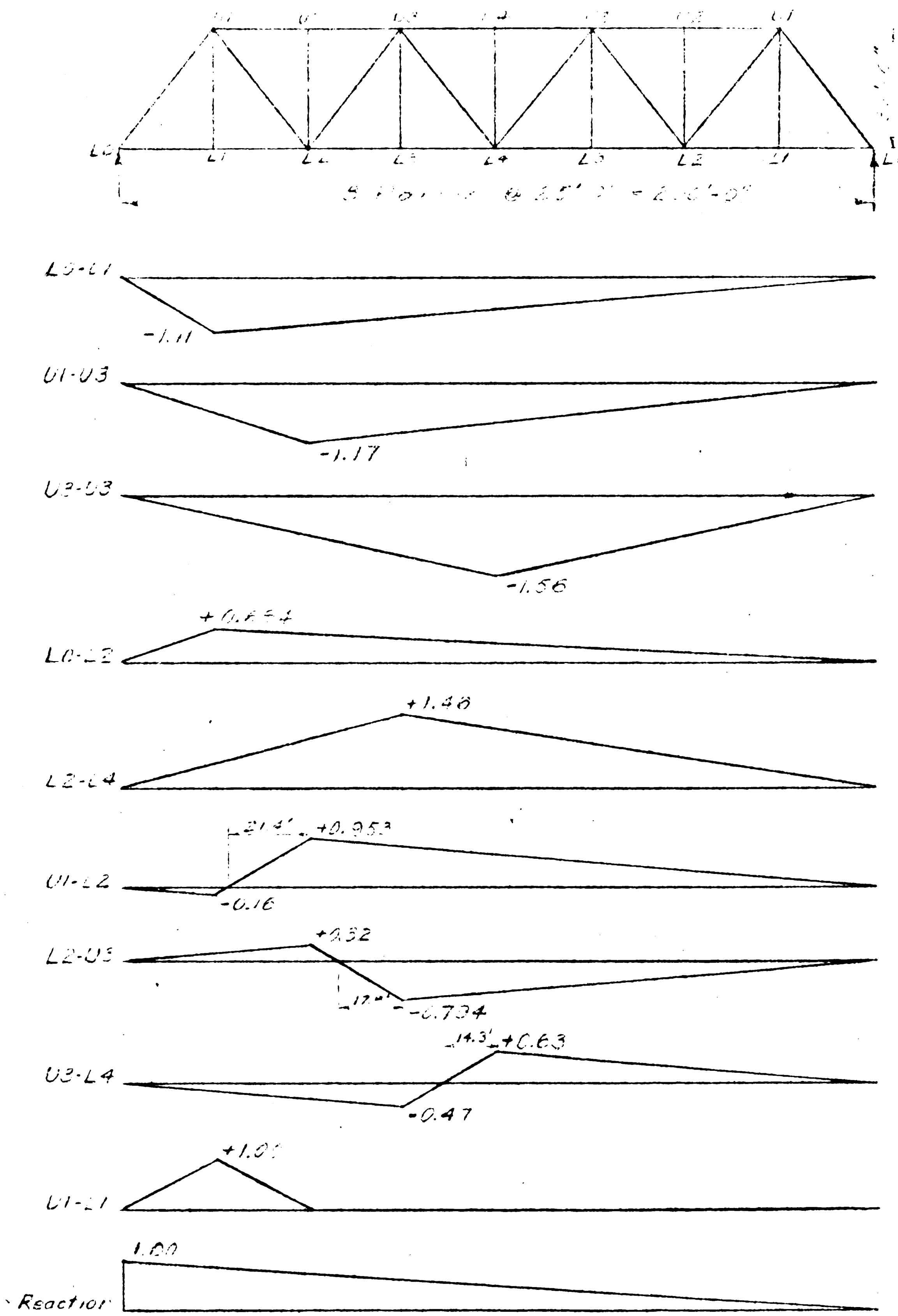


Fig. 7. Influence Lines for Stress in Members and for the [illegible] Reaction

This indicates that the maximum stress in member L0-U1 is when wheel #4 is at L1 with the train moving toward the left. The total stress is

$$-S = \frac{1.11}{175}\left[(18)(7) + (36)(35)\right] + \frac{1.11}{175}\left[(84)(\tfrac{1}{2})(3.6)\right] +$$

$$\frac{1.11}{175}\left[(23.4)(1000) + (36)(831) + (18)(137)\right] = -497 \text{ kips}$$

Maximum stress for other members are as follows:

MEMBER	TRAIN HEADED	WHEEL @ JOINT	STRESS
L0-U1	Left	#4 @ L1	-497.0 kips
U1-U3	"	#7 @ L2	-506.0 "
U3-U3	"	#13 @ L4	-665.0 "
L0-L2	"	#4 @ L1	305.5 "
L2-L4	"	#11 @ L3	630.0 "
U1-L2	"	#3 @ L2	369.0 "
L2-U3	"	#3 @ L3	-270.4 "
	Right	#3 @ L2	48.9 "
U3-L4	Left	#3 @ L4	178.0 "
	Right	#2 @ L3	-102.0 "
Hangers	Left	#4 @ L1	136.0 "
Verticals	--	------	none
Reaction	Left	#2 @ L0	470 "

Reversal of stresses occurs in the diagonals but in member U1-L2, the compressive stress is so small compared to dead load stress that it is neglected. The stress of member L2-U3 in tension is computed with wheel #1 omitted.

IMPACT STRESSES:

Using the formulas for impact as stated in section III, Loads, the following values for stress due to impact are obtained:

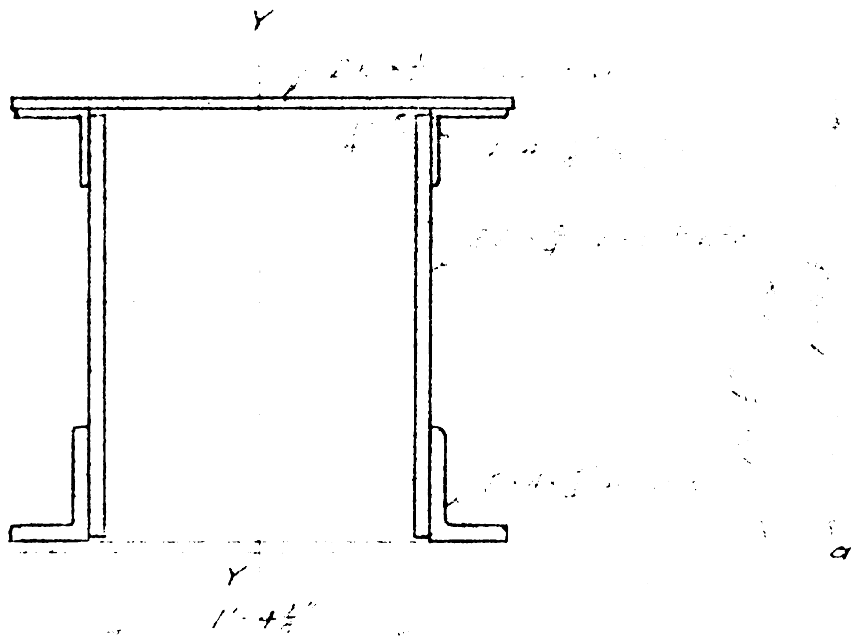
Member				Stress	
L0-U1	= 26.75% of	-497.0	=	-133.0	kips
U1-U3	= " "	-506.0	=	-135.0	"
U3-U3	= " "	-665.0	=	-178.0	"
L0-L2	= " "	305.5	=	81.7	"
L2-L4	= " "	630.0	=	169.0	"
U1-L2	= " "	369.0	=	99.0	"
L2-U3	= " "	-270.0	=	- 72.0	"
	= " "	48.0	=	13.0	"
U3-L4	= " "	178.0	=	48.0	"
	= " "	-102.0	=	- 27.0	"
Hangers	= 94.6% of	136	=	129.0	"
Reaction	= 26.75% "	470	=	126.0"	"

MAXIMUM DIRECT STRESSES:

The total maximum stress in any member is the sum of the stresses due to dead load, live load, and impact as calculated above. The design stress of a member is equal to the total maximum stress for that member except in the case of reversal of stress. In that case, both the maximum tensile and the maximum compressive stress are increased by 50 per cent of the smaller, and the member proportioned so that it will resist either increased resultant stress. The values obtained by the author compare very favorable with those indicated on the design sheets. See table #1 for a summary of stresses.

CHECK OF MEMBERS:

Member L0-U1 is to be checked for a design stress of 833 kips compression. The section is shown on figure 8. To locate the "x-x" axis, the summation of moments about the base or "a-a" axis is made as follows:



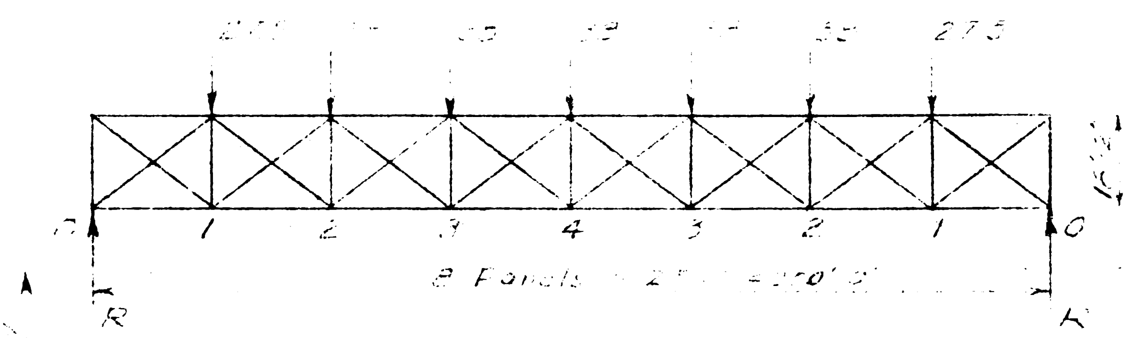


Fig.9. Bottom Lateral Bracing

SECTION		AREA		ARM		MOMENT
Cover Plate 26 x ½	=	13.00 sq.in.	x	22.75	=	295.75 in.^{3}
2 angles 4 x 4 x 3/8	=	5.72 " "	x	21.36	=	122.18 "
2 webs 22 x 3/4	=	33.00 " "	x	11.25	=	371.25 "
2 angles 6 x 4 x 3/4	=	13.88 " "	x	2.08	=	28.87 "
Total A	=	65.60 sq.in.		Total M	=	818.05 in.^{3}

Distance from "a-a" axis to the "x-x" axis equals 818/65.6, or 12.45 inches.

Moment of inertia about the "x-x" axis:

Cover Plate	=	(13)(10.25)	=	1367. in.^{4}
4 x 4 angles	=	(2)(4.4) + (2)(2.86)(8.86)^2..	=	460. "
Web	=	(2)(665.5)+(2)(16.5)(1.25)^2..	=	1382. "
6 x 4 angles	=	(2)(24.5)+(2)(6.94)(10.42)^2..	=	1557. "
		Total I	=	4766. in.^{4}

Radius of gyration r, about the "x-x" axis is the square root of the total moment of inertia divided by the total area which is equal to 8.53 inches.

Moment of inertia about the "y-y" axis:

Cover plate	=	(1/12)(½)(26)...............	=	732.3 in.^{4}
4 x 4 angles	=	(2)(4.4)+(2)(2.86)(9.95)^2...	=	578.8 "
Web	=	(16.5)(142)................	=	2343.0 "
6 x 4 angles	=	(2)(8.7)+(2)(6.94)(9.89)^2...	=	1373.4 "
		Total I	=	5027.5 in.^{4}

Radius of gyration, r = 8.76 in.

Ratio L/r = 487/8.53 = 57.

Allowable unit stress = 15,000-0.25(57)^2 = 14.2 kips/sq.in.

Req'd. area = 833/14.2 .... = 58.8 sq.in.

Furnished gross area .... = 65.6 sq.in. o.k.

Lacing consists of 2 3/4 x ½-in. bars double laced at 45 degree angles. This agrees with specifications which state that where distance across member between rivet lines exceeds 15 inches and a bar with single rivet connection is used, the lacing must be doubled and riveted at intersections. The width of bar exceeds the minimum which is 3 times the diameter of the rivet, or, in this case, 2.625 inches. The thickness of the bar exceeds the minimum which is one-sixtieth of its length between connections, or in this case, 0.486 inches. Using the radius of gyration about the "y-y" axis, the shearing stress normal to the member is

$$V = \frac{P}{100}\left[\frac{100}{L/r + 10} + \frac{L/r}{100}\right] = 17{,}200 \text{ lbs.}$$

$$\text{Actual } \frac{P}{A} = \frac{(\frac{1}{2})(17200)(1.414)}{(2.75)(0.5)} = 8{,}840 \text{ psi.} \qquad \text{o.k.}$$

$$\text{Allowable } \frac{P}{A} = 15000 - \frac{1}{4}\left[\frac{0.7 \times 29.2}{0.29 \times 0.5}\right]^2 = 10{,}030 \text{ psi.}$$

The least ratio L/r of flange between the lacing bar connections is 18.4 which is less than the maximum of 40 and less than the minimum of two-thirds of the L/r of the member, or 38, in this case.

Tie Plates:

Minimum length of tie plates = 20.6 x 1.25 = 25.8 in.

Actual length of tie plates .... = 27.0 in. o.k.

Minimum thickness = 20.6 x 1/50 .... = 0.41 in.

Actual thickness .... = 0.44 in. o.k.

Number of rivets required at end connections is controlled by single shear and equals 833/8.1 or 103 rivets. Actual number of rivets at joint LO is 106 and at joint Ul is 122. Therefore, end connection is satisfactory.

Note: As all of the compression members are similar in design to that of LO-Ul, the checking of the remaining compression members will be summarized here, without repeating the details illustrated with member LO-Ul.

Member U1-U3: (See figure 8)

Actual gross area ....= 60.10 sq.in.

Moment of inertia about "a-a" axis ......= 756.80 cu.in.

Distance from "a-a" axis to "x-x" axis.. = 12.6 in.

Moment of inertia about "x-x" axis...... = 4536 $in.^4$

Radius of gyration about "x-x" axis .... = 8.68 in.

Moment of inertia about "y-y" axis ..... = 4609.5 $in.^4$

Radius of gyration about "y-y" axis .... = 8.76 in.

Ratio L/r = 300/8.68 .... = 35

Allowable unit stress = 15000-1225/4 ... = 14.7 kips/sq.in.

Required area = 859/14.7 .... = 58.3 sq.in.

Furnished area of 60.10 sq.in. is sufficient.

The lacing is the same as in member L0-U1 except that the normal shearing force equals 22,300 lbs. and the actual P/A equals 11,500 psi. With an allowable of 10,030 psi., as in L0-U1, this appears to be a slight overstress but this is satisfactory as the cover plate has not been considered. The tie plates are the same as in member L0-U1 and are therefore satisfactory.

The number of rivets in the end connection at joint U1 is controlled by the rivet value in single shear. The number of rivets required is 859/8.1 or 106 rivets which is less than the 132 rivets furnished.

Member U3-U3: (See figure 8.) As the location of the "x-x" has been calculated twice and found to compare favorably with that used on the detail sheets, the location of this axis will be assumed hereafter to be that as given by the plans.

Actual gross area .... = 76.60 sq.in.

Moment of inertia about "x-x" axis .... = 5222.6 $in.^4$

Radius of gyration about "x-x" axis ...... = 8.3 in.

Moment of inertia about "y-y" axis .... = 6001.3 $in.^4$

Radius of gyration about "y-y" axis....... = 8.85 in.

Ratio L/r = 300/8.3 .... = 36

Allowable unit stress = 15000-13/4 .... = 14.7 kips/sq.in.

Required area = 1132/14.7 .... = 77.0 sq.in.

Furnished area of 76.6 sq.in., while slightly less than that required, is not considered dangerous.

The lacing of this member consists of 5 x 5/8-in. bars single laced. With a normal shear equal to 29,700 lbs., the actual P/A equals 5,500 psi. The allowable P/A as calculated for axial compression is 10,680 psi. This indicates that the lacing is greatly understressed . The reason why such a long bar was indicated for use here is not apparent. moreover, the specification requiring double lacing for members this wide has not been satisfied. The ratio L/r for the flange is found to be less than 36 and is therefore satisfactory. Tie plates are the same size as those previously examined.

Diagonal L2-U3:

Least radius of gyration, r .... = 6.29 in.

Ratio L/r = 487/6.29 = 77.5 ....o.k. (140 max.)

Allowable unit stress .... = 13,500 psi.

Required area 430/13.5 .... = 31.8 sq.in.

Furnished gross area is o.k. .... = 33.96 " "

The lacing and tie plates are checked as above and are considered satisfactory. The number of rivets required is 430/8.1 or 53 rivets at each end. Plans indicate 60 rivets use at L2 and U3 which is satisfactory.

Member L0-L2 is designed for tension and the net area will be the effective area.

Required net area = 515/18 .... = 28.6 sq.in.

Furnished gross area .... = 39.0 sq.in.

Furnished net area (deduct for 2 rivets in each angle and 2 rivets in each plate) .... = 33.0 sq.in. o.k.

Req'd no. rivets at end = 515/8.1 .... = 64 rivets

Furnished number of rivets = (2)(32) .... = 64 rivets O.k.

Member L2-L4:

The required net area is 1072/18 or 59.6 sq.in. The member as designed supplies a gross area of 75.84 sq.in. In calculating the supplied net section, it appears that two rivet holes have been deducted from each angle and four rivet holes from each plate, leaving a net area of 60.34 sq.in. This is very conservative as the rivets through the plates are staggered. The stitch rivets have a pitch of five inches which is less than the maximum specified of 24 times the thickness of the thinnest outside plate, or 16.5 inches in this case.

Member U1-L2:

Req'd. net area = 627/18 .... = 34.9 sq.in.

Furnished gross area .... = 43.94 " "

Furnished net area (deducting 3 holes from
each angle and 2 holes from web).. = 34.94 sq.in. o.k.

Req'd no. rivets at end = 627/8.1 .... = 78 rivets

Furnished rivets at joint U1 .... = 84 " o.k.

Furnished rivets at joint L2 .... = 92 " o.k.

Member U3-L4 is designed for tension as the tensile stress is more than twice the compressive stress. The required net area is 306/18 or 17 sq.in. The furnished gross area is 23.4 sq.in. Deducting 4 rivets from each web, the furnished net area is 19.2 sq.in. which is satisfactory. The number of rivets required at the end connection is 456/8.1 or 57 rivets. Each end has 68 rivets which is more than enough.

Hangers:

Required net area = 301/18 .... = 16.7 sq.in.

Furnished gross area .... = 20.00 " "

Furnished net area (deduct 4 rivet holes) .. = 17.25 " " o.k.

Req'd no. rivets at end = 301/8.1 .... = 38 rivets

Furnished no. rivets .... = 40 " o.k.

The vertical members are not designed for their small stress but rather as a minimum section which will keep the L/r ratio less than 140. The least radius of gyration of a 14CB78 member is 3.0. Hence the L/r ratio is 384/3.0 or 128 which is satisfactory.

The end reaction per truss will require a bearing area on masontry of 781/0.6 or 1300 sq.in.

Table #1 shows the above calculation in tabular form for comparison purposes.

SPLICES:

The top chord splice at joint U3 is a compression splice. The plans indicate the ends of the members are to be finished for bearing, therefore, the splice material need only be designed to transmit one-half of the stress, or 430 kips. The main splice plate alone has an area of 51.9 sq. in. and would be stressed less than 8.3 kips per sq.in. so there is ample material provided. Required number of rivets in single shear is 430/8.1 or 53. Plans show about 87 rivets which make the splice very safe.

The bottom chord splice near joint L4 is a tension splice and must develop the full net strength of the member. This splice is made up of two parts so each part will act spearately.

Part A:

Net area of member = 18.44 + 21.90 ... = 40.34 sq.in.

Net area of splice plates .... = 41.4 " " o.k.

Required load = 40.34 x 18 .... = 730 kips

Load transmitted by rivets .... = 756 kips o.k.

Part B:

Net area of member .... = 20.00 sq.in.

Net area of splice plates .... = 37.50 sq.in. o.k.

Required load = (20.0)(18) .... = 360 kips

Load transmitted by rivets .... = 719 kips

Section between parts A&B will transmit 766 kips and is therefore satisfactory. This type splice appears to be effective in keeping size and number of splicing plates at a minimum and also in producing a rigid connection.

The splice near joint L2 is also a tension splice. The net area of splicing plates is 36.0 sq.in. which is sufficient. The required loading will be 33 x 18 or 594 kips. Number of rivets furnished is 88 which will transmit a load of 713 kips in single shear. Therefore, splice is satisfactory.

GUSSET PLATES:

The size of gusset plates is generally governed by the space required for the placing of rivets. As most end connections are in single shear, the minimum thickness for a gusset plate would be that thickness which would develop a bearing value equal to the shearing value of a rivet. In this case, the value of a 7/8-in. rivet in single shear is 8,100 lbs. and the minimum thickness would be 3/8-inches. However, specifications state that $\frac{1}{2}$-in. is the minimum thickness and the design sheet designates 5/8-in. Upon checking the gusset plate where U1-L2 is connected, it is found that more than twice the required area is furnished. As this is the most highly stressed plate, the other plates are assumed to be safe.

MEMBER	D.L. STRESS	L.L. STRESS	IMPACT STRESS	TOTAL STRESS	DESIGN STRESS	'l'	'r'	l/r	UNIT STRESS	REQ'D AREA	ACTUAL GROSS AREA	ACTUAL NET AREA
L0-U1	-206	-497	-133	-836	-833	487	8.53	57	14.2	58.8	65.60	—
U1-U3	-217	-506	-135	-855	-859	300	8.68	35	14.7	58.3	60.10	—
U3-U3	-289	[illegible]	-175	-1132	-1132	300	8.3	36	14.7	77.0	76.60	—
L0-L2	+127	[illegible]	+82	+515	+515	300	—	—	18	28.6	39.00	33.00
L2-L4	+272	[illegible]	[illegible]	+1071	+1072	300	—	—	18	59.5	75.24	[illegible]
U1-L2	+147	[illegible]	[illegible]	[illegible]	[illegible]	487	—	—	18	34.9	43.04	34.94
L2-U3	[illegible]	-275 +40	-72 +13	-430 -27	[illegible]	487	8.23	77.5	13.5	31.8	33.06	—
U3-L4	[illegible]	+178 [illegible]	+48 -27	+255 [illegible]	+306 [illegible]	497	5.44	[illegible]	+18 -13	[illegible]	[illegible]	[illegible]
HANGERS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	354	—	—	18	16.7	[illegible]	17.25
VERTICAL	[illegible]	—	—	[illegible]	-11	324	[illegible]	128	—	—	[illegible]	—
REACTION	[illegible]	[illegible]	126	751	—	—	—	—	0.0	[illegible]	—	—

Table 1. Summary of Stresses and Properties

IX. BRACING

The bracing to be checked here consists of the upper and lower lateral, the sway, and the portal bracing.

UPPER LATERAL BRACING:

The top lateral bracing is composed of struts at each panel point of 4 - 5 x $3\frac{1}{2}$ x 3/8" angles, and two diagonals in each panel of 4 - $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8" angles. It is assumed that the diagonals take tension only. Specifications state that lateral bracing of compression chords shall be proportioned for a transverse shear in any panel equal to $2\frac{1}{2}$ per cent of the total axial stress in both members in that panel, in addition to the shear from the specified lateral forces.

Diagonals of Panel U1-U3 = 1.7 x 2 x 859 x 0.025 = +73 kips

Diagonals of Panel U3-U3 = 1.7 x 2 x 1132 x 0.025 = +96.3 kips

As these loads agree with those indicated on the design sheet, the above values will be used for use here. However, it appears that the stress due to the specified lateral forces has been omitted.

The diagonals are all alike, therefore only the diagonal with the highest stress will be checked. If it is safe, all diagonals will be safe. The diagonal of panel U3-U3 will require an area of 96/18 or 5.35 sq.in. The designed members are composed of 4 - $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8" angles with one rivet hole per angle. These give a net area of 4(2.48-0.375) = 8.4 sq.in. which is satisfactory. The number of rivets required by stress in the end connection is 96/8.1 = 12 rivets in single shear. Number of rivets required to develope full strength of member is 18 x 8.4/8.1 = 19 rivets. However, as large excess of area has been provided, due to minimum size of angles, the 16 rivets provided are recognized as adequate.

The struts are built up of 4 - 5 x 3½ x 3/8". angles and the highest compressive stress is 56.6 kips. The unsupported length is in the horizontal direction and equals 218-27 or 191 inches. The least permissable radius of gyration is 191/120 or 1.59 inches. The angles are placed with short legs back to back and have a horizontal radius of gyration of 2.27 inches. This permits a working stress of 13,230 psi. The required area is then 56.6/13.23 or 4.3 sq.iin. The angles as designed supply 12.2 sq.in. Number of rivets required in the end connection is 56.6/8.1 or seven rivets. The design uses twelve rivets and is alright.

BOTTOM LATERAL BRACING:

The maximum stress is produced in the bottom lateral bracing with a wind force of 50 lbs. per square foot on the area formed by the vertical projection of the trusses. It is assumed that one-half of the total force is carried by this bracing, and is an uniform moving load concentrated at the panel points on the windward side. The stresses in the floor beams are so small, compared to their previously designed section, that they are neglected. It is also assumed that the diagonals act at the same time with each diagonal taking one-half of the shear in a given panel. (See Fig.9).

Load per interior panel point .... = 38 kips

Load per end panel point .... = 27.5 "

Shear in panel	$\times \frac{1}{2} \times \sec\theta$ or 1.7	=	Stress in diagonal
0-1:	122.5	=	104 kips
1-2:	98.5	=	83.6 "
2-3:	70	=	59.5 "
3-4:	46.2	=	39.4 "

Unsupported length - diagonal 0-1 approx. = 9.0 ft.

Minimum radius of gyration = 108/120 .... = .90 in.

Least radius of gyration of angles ...... = 0.99

Allowable compressive stress .... = 12. kips/sq.in.

Required area 104/12 .... = 8.66 sq.in.

Furnished area 2 - 5 x $3\frac{1}{2}$ x 5/8 angles .. = 9.84 " "

Req'd. no. of rivets = 104/8.1 .... = 13 rivets

Number of rivets furnished .... = 14 rivets o.k.

Diagonals in panel 0-1 appear to be satisfactory. The diagonals of the other panels are checked in a similar manner using their respective stresses.

SWAY BRACING:

The sway bracing at each panel point consists of diagonals of $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8-in. angles and a bottom horizontal member of 2 - 5 x $3\frac{1}{2}$ x 3/8-in. angles. These members are not usually designed for definite stresses but the general form and dimensions are governed by the minimum clearance required.

PORTAL BRACING:

As in the sway bracing the general form and dimensions are governed by the required clearances. In this case the members are built-up of 4 - $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8" angles and a 3/8" plate. Due to limited time, an analysis of the portal effect is not presented here. This presents an interesting problem in indeterminate structures and for a thorough treatment of the subject, the reader is referred to the text, "Modern Framed Structures", Part III, by Johnson, Bryan, and Turneaure.

X. END BEARING

The end bearings consist of a fixed shoe and an expansion shoe. They have adjustable top plates to keep the vertical deflection of the bridge from creating excessive moments in the bearing plate. Both shoes are approximately of the same dimensions and have the same load, so one check applies to both shoes.

Maximum reaction .... = 781 kips

Base area required: 781/600 .... = 1300 sq.in.

Base area furnished 28 x 48 .... = 1344 " " o.k.

Allowable unit stress .... = 18 kips/sq.in.

Minimum cross section 4 x 18 .... = 72 sq.in.

Unit stress 781/72 .... = 10.9 kips/sq.in

Req'd. bearing area at exp. shoe 781/75 ... = 10.4 sq.in.

Length of insert ... .= 30 in.

Width of insert in bearing 10.4/30 .... = 0.347 in.

The plans show an interesting feature which is not intended to be of the typical bridge plans. It is a specially designed hold down which is intended to restrain flood waters from lifting the span off its footings.

XI. CONCLUSION

The foregoing check of the design of a typical bridge span indicates the design to be more than adequate throughout. The only inadequacy noted was in the detailing of the stiffeners at the jacking point on the end floor beam.

In the course of completing this analysis, many references were made to A.R.E.A. Specifications of different years. The author was impressed with the amount of variations between these specifications. One difference particularly noteworthy, was the many formulas proposed for the calculation of stress due to impact. The use of formulas given in the 1948 Specification results in a much higher impact stress than do those used herein.

The author regrets that the limited time allocated for the purpose of writing a thesis prevented a more thorough and detailed analysis. It is believed that much can be learned by the undergraduate civil engineering student in the making of an analysis such as this one. The fundamentals of engineering become more than isolated facts when it is seen how they are applied in actual practice.

BIBLIOGRAPHY

A.I.S.C., "Steel Construction," 5th ed., American Institute of Steel Construction, New York, 1947.

American Railway Engineering Association, "Specifications for Steel Railway Bridges," A.R.E.A., Chicago, 1948.

Burr & Falk, "The Design and Construction of Metalic Bridges," John Wiley & Sons, New York, 1905.

Fuller & Kereckes, "Analysis and Design of Steel Structures," D. Van Nostrand Company, Inc., New York, 1936.

Grinter, Linton E., "Design of Modern Steel Structures," The MacMillan Company, New York, 1941.

Johnson, J.B., "The Theory and Practice of Modern Framed Structures," Parts I & III, 10th ed., John Wiley & Sons, Inc., New York, 1926.

Matthews & Soneson, "Analysis of Framed Structures," McGraw-Hill Book Company, Inc., 1935.

Merriman, Thaddeus, "American Civil Engineers' Handbook," 5th ed., John Wiley & Sons, Inc., New York, 1944.

Sutherland & Bowman, "Structural Theory," 3rd ed., John Wiley & Sons., Inc., New York, 1948.