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THESIS

ANALYSIS OF PERE MARQUETTE R. R. BRIDGE
AT GRAND LEDGE, MICHIGAN

E. P. NORTH H. M. COBURN

1922

Bridges

Analysis of Pere Marquette R. R. Bridge

at Grand Ledge , Michigan.

A Thesis

submitted to the Faculty of

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by

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Candidates for the degree of

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INTRODUCTION

The subject of this thesis being an analysis of a railroad bridge , it may be well to give a short general description of the structure, its location, construction and type of traffic to which it is subjected.

The bridge in question is located on one of the main lines of the Pere Marquette Railroad, being known as Bridge D 100.2 over the Grand River at Grand Ledge, Michigan. The structure was constructed by the Pennsylvania Steel Co. of Steelton, Pa., in the year 1904. It consists of three single track , deck lattice spans of the Pratt type , each having six panels of 20 feet for one span making a total of 120 feet per span. At each end of the bridge there is an approach span consisting of 50 ft. deck plate girders. The total track span over the river is therefore approximately 470 feet. The distance from the track level to the surface of the river is about 60 feet.

The location is an ideal one for a bridge of this type, the banks of the river being very high and steep and the outcropping bed rock on the banks makes a very foundation for the abuttments. The bed of the river is also of solid rock and the water is comparatively shallow.

The bridge is subjected to fairly heavy loads, the line being one of the main freight routes of Central Michigan.

As the bridge is inside of the yard limits of Grand Ledge, much starting and stopping of switching cars is done in mid span.

The structure, although at date, being about 22 years old, seems to be in very good condition, and shows that it has recieved constant care and attention. Reference is made to the photographs of the structure among the follow -ing pages.

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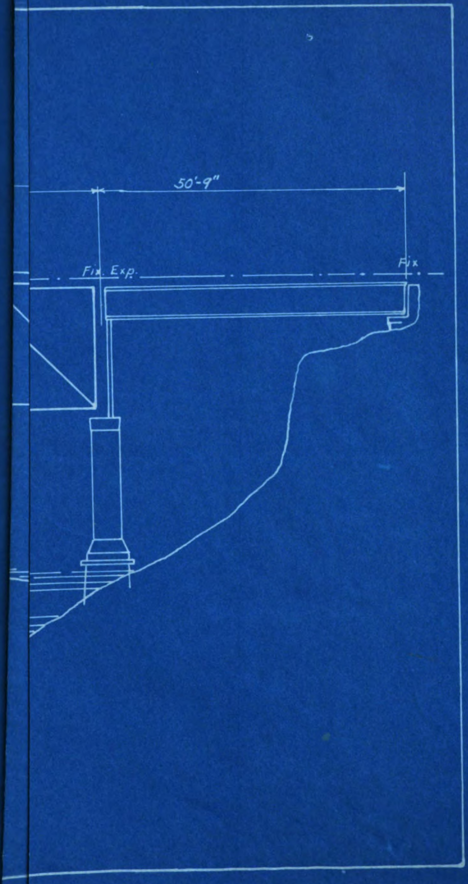
LIST OF DRAWINGS

FOREWORD

In the following analysis an attempt has been made to analyze the main members of the structure with as much detail as possible. The structure was designed to Cooper's 1901 specifications, however the analysis was made according to Cooper's 1906 specifications, which may explain some slight differences in sectional area etc.

Reference is hereby made to the various drawings of the sections in question, which follow the analysis of each particular member. The drawings of the upper chord and end post as well as the lower chord and lateral bracing are to be found in the pocket on the back cover.

The writers are greatly indebted to H. K. Vedder, Professor of Civil Engineering at the Michigan Agricultural College for many helpful aids and suggestions; and to Chas. S. Sheldon, Engineer of Bridges and Structures, P. M. R. R. for aid in securing plans and specifications.


50'-9"
Fix. Exp.
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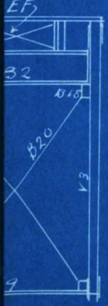

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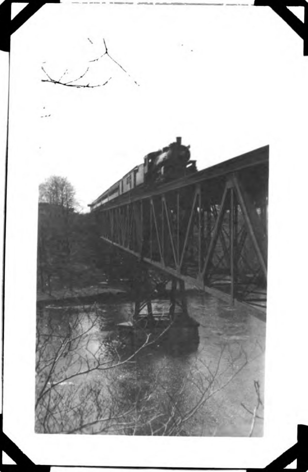
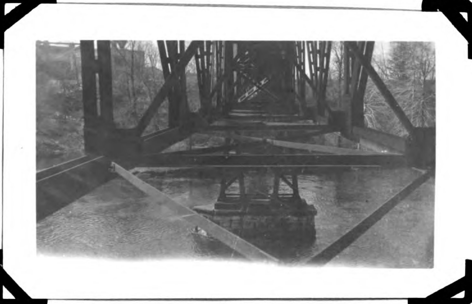
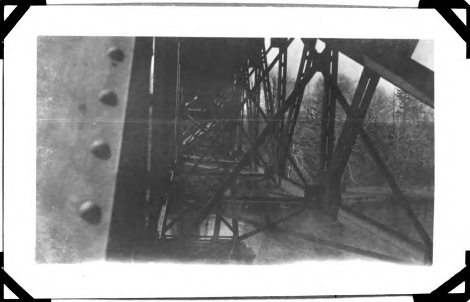
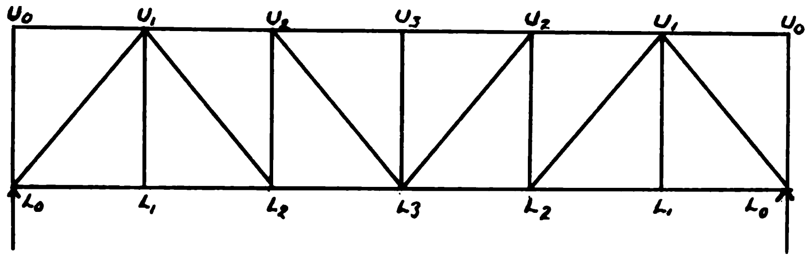






DEAD LOAD STRESSES IN TRUSS MEMBERS



Data on Truss

Span, center to center of end pins------------------ 120'- 0"

Depth, between centers of chords-------------------- 22'- 0"

Width, between centers of trusses------------------- 13'- 0"

Number of panels------------------------------------ 6

Panel length-- 20'- 0"

Length of end posts, c to c of pins----------------- 29'-8 7/8"

Sec Θ--- 1.351

Loading

Dead load per lineal foot of bridge----------------- 2400 #

Dead load per panel of each truss------------------- 24000 #

Load per upper panel point-------------------------- 18000 #

Load per lower panel point-------------------------- 6000 #

End reaction due to dead load----------------------- 60000 #

Dead Load Stresses

Member	Compression	Tension
Uo Lo	6000 #	
Lo L1		54600 #
L1 L2		54600 #
L2 L3		87300 #
U1 U2	87300 #	
U2 U3	98000 #	
Lo U1	81000 #	
U1 L1		6000 #
U2 L2	30000 #	
U3 L3	18000 #	
U1 L2		48700 #
U2 L3		16300 #

COMPUTATIONS FOR DEAD LOAD STRESSES

Diagonals

Shear in Lo U1 ------------------------------------		60000 #
Stress in Lo U1 ---------------	60000 x 1.351	81000 #
Shear in U1 L2 ---------------	60000 - 24000	-36000 #
Stress in U1 L2 ---------------	36000 x 1.351	-48700 #
Shear in U2 L3 ----------------	60000 - 48000	-12000 #
Stress in U2 L3 ---------------	12000 x 1.351	-16300 #

Vertical Posts

With a distribution of dead load of 18000 # at each upper panel point and 6000 # at each lower panel point, the stress in post Uo Lo would be that caused by the difference between 9000 # at Uo and 3000 # at U1, or 6000 # compressive stress. The dead load stress in Uo L1 would be that caused by the load of 6000 # at U1, or 6000 # tensile stress. The dead load stress in U2 L2 is equal to the shear in the section or 60000 # - 30000 # which is 30000 # compressive stress. The dead load stress in U3 L3 is equal to that caused by the dead load of 18000 # at U3 or 18000 # compressive stress.

Chords

The dead load chord stresses were computed by the method of moments. A section is passed thru the chord in question and moments taken about the intersection of the other two members cut by the section.

Passing a vertical section thru Lo L1, sum M about U1 = 0

60000 x 20 - 22 Lo L1 = 0

Stress in Lo L1 = -54600 #   L1 L2 = Lo L1

Passing a vertical section thru U1 U2, sum M about L2 = 0

60000 x 40 - 24000 x 20 - 22 U1 U2 = 0

Stress in U1 U2 = +87300 #   -L2 L3 = +U1 U2

Passing a vertical section thru U2 U3, sum M about L3 = 0

60000 x 40 - 24000 x 20 - 24000 x 40 - 22 U2 U3 = 0

Stress in U2 U3 = +98000 #

LIVE LOAD STRESSES IN TRUSS MEMBERS

The live load stresses are those due to a loading equivalent to Cooper's Standard E-50, consisting of two 177 ½ Ton engines followed by a uniform train load of 5000 # per lineal foot. No impact was figured in any of the computations on stresses.

Live Load Stresses in Web Members

Coopers E-50 Loading

Panel	Wheel at rt. end panel	Mom. at rt. support Kip feet	Mom. at rt. end panel Kip feet	Shear Kips	Stress Pounds	Mem.
LoL1	4	23789.0	600.0	167.9	227000	U1Lo
L1L2	3	15077.05	287.5	111.1	-150000 111000	U1L2 U2L2
L2L3	3	9359.4	287.5	63.5	86000 63500	U2L3 U3L3

Live Load Chord Stresses

Sect	Wheel at sect	Length of train Feet	Moment of rt. support Kip Feet	Moment at section Kip Feet	Bending Moment Kip Feet	Stress	Chord
U1L1	4	9.1	23789.0	600.0	3365.0	153,000	LoL1 L1L2
U2L2	6	3.08	21560.0	2050.0	5136.0	233,000	U1U2 L2L3
U3L3	10	7.0	23001.1	5790.0	5710.6	259,600	U3U2

COMPUTATIONS FOR LIVE LOAD STRESSES

Diagonal Lo U1 :

The largest possible shear in any section occurs when the live load on the panel is $\frac{1}{m}$ of the live load on the bridge, m being the number of panels in the span of the bridge.

Wheel 4 at L1 gives load of 62.5 - 87.5 Kips in panel

Length of train on bridge = 118.0 ft.

Load on bridge = 377.75 Kips

$\frac{1}{m}$ of 377.75 = 62.96 Kips

Since 62.96 is between 62.5 and 87.5 the criterion is satisfied by the above position.

Moment at right support = 23789.0 Kip ft.

Moment at right end of panel = 600.0 Kip ft.

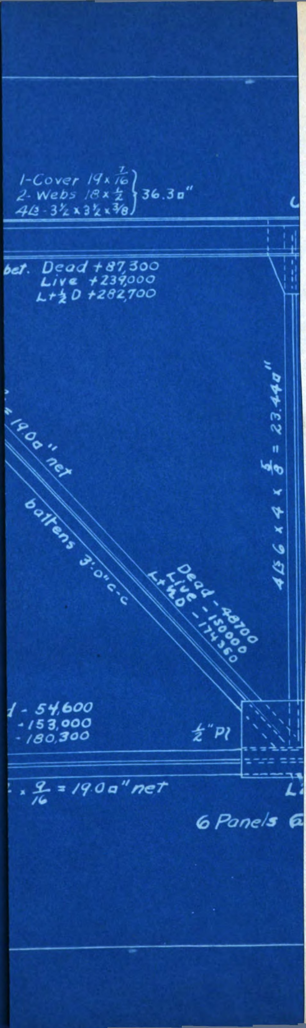
Shear in section equals Moment at right support minus six times the moment at the right end of the panel divided by the length of the bridge.

$$VLoL1 \quad \frac{23789 - (6 \times 600)}{120} = 167.9$$

Stress in LoU1 = 167.9 x 1351 = 227000 #

The above method was used in calculating the other web members and the results were tabulated in table on preceding page. The stresses computed check with those used in design to within a few pounds. All of above were taken to the greatest one hundred pounds.

The moments were taken from the moment diagram of Cooper's E-50 loading.


1-Cover 19 x 7/16
2-Webs 18 x 1/2
4∠-3½ x 3½ x 3/8
36.3□"
bet. Dead +87,300
Live +239,000
L+½D +282,700
19.0□" net
battens 3'0" c-c
Dead -48700
Live -150000
L+½D -174360
4∠ 6 x 4 x 5/8 = 23.44□"
d - 54,600
-153,000
-180,300
½" Pl
x 9/16 = 19.0□" net
6 Panels

COMPUTATIONS FOR LIVE LOAD STRESSES

Chord Members

Chord LoL1 :

Criterion for greatest chord stress is satisfied when the load on the left segment of the span is to the total load on the bridge as the length of the left segment is to the total length of the span.

Wheel 4 at panel point L1 = 62.5 - 87.5

Length of train on bridge = 9.1 ft.

Load on bridge = 355 + (9.1 x 2.5) = 377.75 Kips

According to above criterion the load causing the greatest stress is one-sixth of 377.75 Kips or 62.96 Kips, as this is between 62.5 and 87.5 the criterion is satisfied.

Moment at rt. support $= 20455 + (355 \times 9.1) + 1.25 \times \overline{9.1}^2$
$= 23789.0$ Kip feet

Moment ar section 600 Kip feet

Bending moment $= 1/6 \times 23789 - 600 = 3365$ Kip feet

Stress in LoL1 $= \frac{3365}{22} = 153000$ #

The above method was used in computing the stresses in the other chords and the results are tabulated in Table III. The live load chord stresses all check with those used in the design except L2L3 which has a difference of 6000 #. Moments and loadings were taken from Cooper's Moment Diagram for E-50 loading. A drawing of this diagram may be found on page 122 of Merriman and Jacoby Vol. I.

ANALYSIS OF COMPRESSION MEMBERS

Compression members shall be proportioned by the following allowed unit strains;

For Medium Steel

Chord segments $P = 10000 - 45\ l/r$ for live load.
$P = 20000 - 90\ l/r$ for dead load.

All posts of through bridges $P = 8500 - 45\ l/r$ for live load.
$P = 17000 - 90\ l/r$ for dead load.

All posts of deck bridges $P = 9000 - 40\ l/r$ for live load.
$P = 18000 - 80\ l/r$ for dead load.

End posts are not to be considered chord segments.

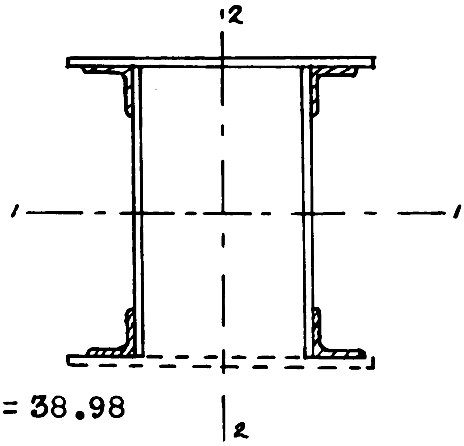
Lateral Struts and rigid bracing $P = 13000 - 60\ l/r$.

P = the allowed strain in compression per square inch of cross-section, in pounds.

l = the length of compression member, in inches, c. to c., of connections

r = the least radius of gyration of the section, in inches.

SECTION OF INCLINED END POST - Lo U1



1 Cover (ba) 19 x $\frac{1}{2}$

2 Webs (bd) 18 x $\frac{1}{2}$

4 Ls (bc) $3\frac{1}{2}$ x $3\frac{1}{2}$ x 7/16

10 3/4" clear between Webs

l = 29' - 8 7/8" = 356 7/8"

Total Area of gross section = 38.98

Axis 1-1 I of cover plate = $19 \times (.5)^3/12$ = .198

Ad^2 of cover plate = $9.5 \times (9.375)^2$ = 835.050

I of 4 Ls = 4 x 3,3 = 13.200

Ad^2 of 4 Ls = $4 \times 2.87 \times 8.085^2$ = 750.413

I of 2 Webs = $2 \times \frac{1}{2} \times 18^3/12$ = 486.000

Moment of Inertia of gross sec. = 2084.861 in.^{4}

Axis 2-2 I of cover plate = $\frac{1}{2} \times 19^3/12$ = 285.791

I of 2 Webs = $2 \times 18 \times .5^3/12$ = .375

Ad^2 of 2 Webs = $2 \times 9 \times 5.625^2$ = 569.520

I of 4 Ls = 4 x 3.3 = 13.200

Ad^2 of 4 Ls = $4 \times 2.87 \times 6.915^2$ = 549.168

Moment of Inertia of gross sec. = 1418.054 in.^{4}

Least Radius of Gyration $= \sqrt{\frac{1418.054}{38.98}}$ = 6.02 in.

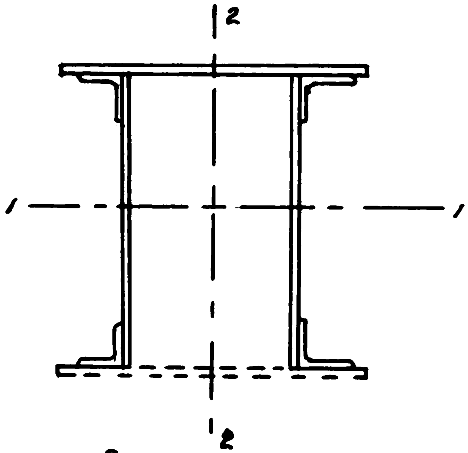
$P = 9000 - 40 \frac{l}{r} = 9000 - \frac{40 \times 356.875}{6.02}$ = 6621.00 #/in.^{2}

Stress in Lo U1 = 267500.00 #

$\frac{267500}{6621}$ = 40.4 in.^{2} reqd. 39.0 in.^{2} used in gross section.

The formula used is from Cooper's 1906 Specifications while the structure was designed to Cooper's 1901.

SECTION OF UPPER CHORD U1 U2



1 Cover (bo) 19 x 7/16 = 8.31

2 Webs (bn) 18 x $\frac{1}{2}$ = 18.00

4 Ls (bk) $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 = 9.92

1 = 20' = 240"

10 5/8" clear bet. webs

Total area gross section = 36.23 in.^{2}

From the design of Lo U1 we know that the least radius of gyration is about axis 2-2.

Axis 2-2	I of cover plate $7/16 \times 19^3/12$	=	250.060
	I of 2 webs $2 \times 18 \times .5^3/12$	=	.375
	Ad^2 of 2 webs $2 \times 9 \times 5.562^2$	=	556.830
	I of 4 Ls 4×2.9	=	11.600
	Ad^2 of 4 Ls $4 \times 2.48 \times 6.822^2$	=	461.676
	Moment of Inertia of gross sec.	=	1280.541

Radius of gyration $= \sqrt{\frac{1280.5}{36.23}} = 5.94$ in.

$P = 10000 - 45 \frac{l}{r} = 10000 - \frac{45 \times 240}{5.94} = 8182$ #/in.^{2}

$\frac{282700}{8182} = 34.6$ In.^{2} reqd. Gross section = 36.3 in.^{2}

SECTION OF UPPER CHORD U2 U3

4 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$x 7/16

2 webs 18 x $\frac{1}{2}$

Cover 19 x 7/16

10 3/4" clear between webs

Least moment will be about

axis 2-2 same as in predeeding work.

Axis 2-2 I of cover plate (bu) $7/16 \times 19^3/12 = 250.060$

I of 2 webs (bt) $2 \times 18 \times .5^3/12 = .375$

Ad^2 of 2 webs (bt) $2 \times 9 \times 5.562^2 = 556.830$

I of 4 Ls (bp) $4 \times 3.3 = 13.200$

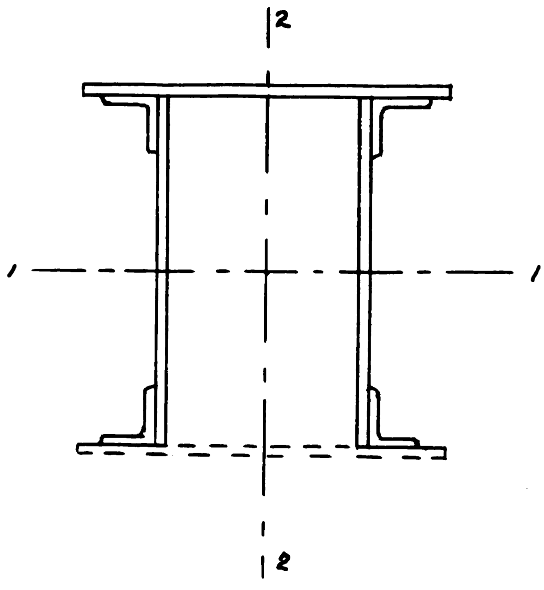
Ad^2 of 4 Ls (bp) $4 \times 2.87 \times 6.852^2 = \underline{538.974}$

Moment of Inertia of gross sec. $= 1359.439$

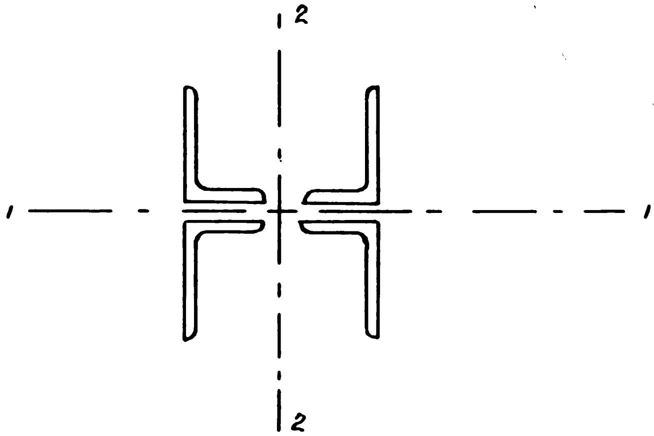
Radius of gyration $= \sqrt{\frac{1359.4}{37.71}} = 6.05$ in.

$P = 10000 - 45 \frac{1}{r} = 10000 - \frac{45 \times 240}{6.05} = 8125$ #/in.^{2}

$\frac{310000}{8215} = 37.7$ in.^{2} reqd. Gross section $= 37.8$ in.^{2}



SECTION VERTICAL POST U2 L2 (V2)



4 Ls (ce) 6 x 4 x 5/8 = 23.44 in.^{2}

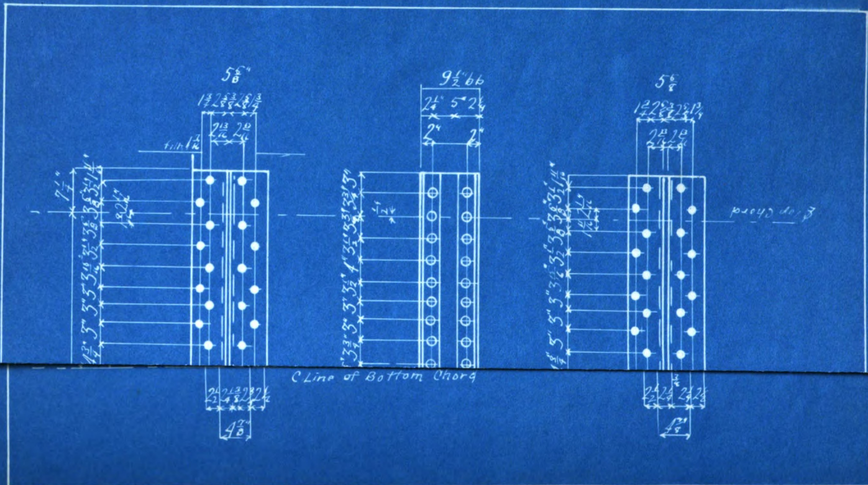
l = 22' = 264 "

Axis 1-1	I of 4 Ls (ce) 4 x 21.1	=	84.400
	Ad^2 of 4 Ls (ce) 4 x 5.86 x 2.21^2	=	115.090
	Moment of Inertia of gross section	=	199.490
Axis 2-2	I of 4 Ls (ce) 4 x 7.5	=	30.000
	Ad^2 of 4 Ls (ce) 4 x 5.86 x 3.22^2	=	242.838
	Moment of Inertia of gross section	=	272.838

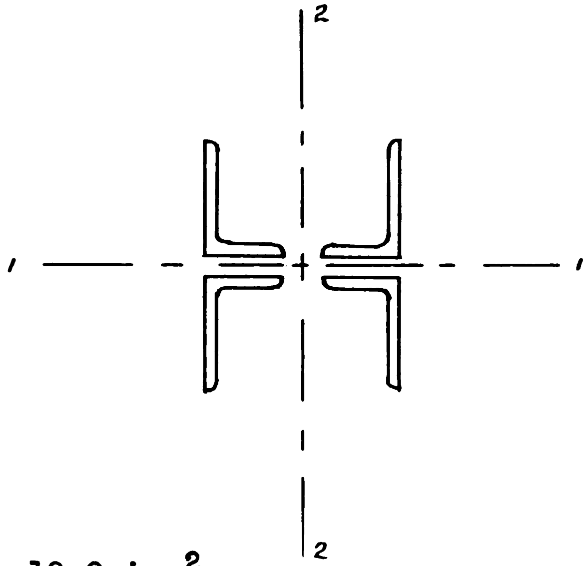
Least radius of gyration = 2.92 in.

$P = 9000 - \frac{40 \times 264}{2.92} = 5384$ #/ in.^{2} Allowable unit stress.

$\frac{126000}{5384} = 23.4$ in.^{2} reqd. Gross section = 23.44 in.^{2}


C Line of Bottom Chord

SECTION OF VERTICAL POST U3 L3 (V3)



4 Ls 6 x 4 x $\frac{1}{2}$ = 19.0 in.^{2}

1 = 22' = 264"

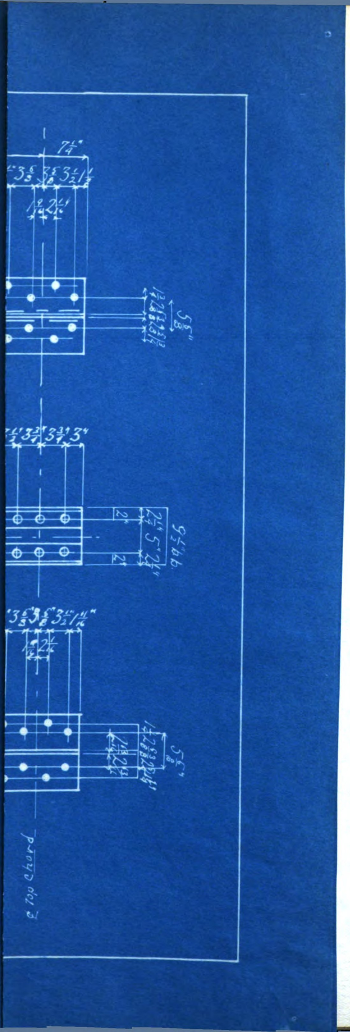
From the previous post V2 we know that the least radius of gyration is about axis 1-1.

Axis 1-1 I of 4 Ls (ca)	4 x 17.4	=	69.600
Ad^2 of 4 Ls	4 x 4.75 x 2.177^2	=	90.041
Moment of Inertia of gross section		=	159.641

Radius of gyration = 2.90 in.

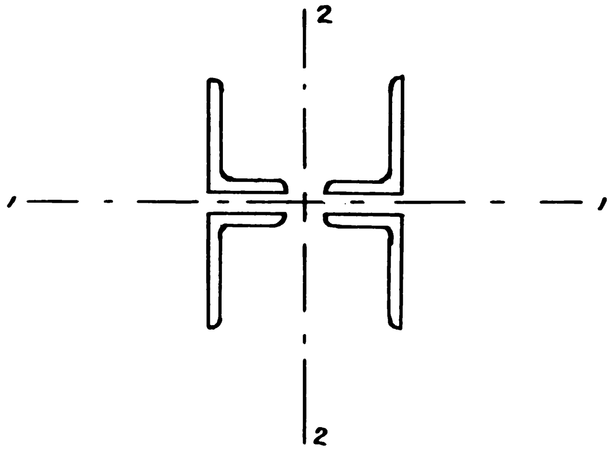
$P = 9000 - \frac{40 \times 264}{2.90} = 5359$ #/in.^{2}

$\frac{91000}{5359} = 17.0$ in.^{2} reqd. Gross section = 19.0 in.^{2}



℄ Top Chord
9 1/2" b.b.

SECTION OF VERTICAL POST Uo Lo (V)



As before least radius of gyration is about axis 1-1.

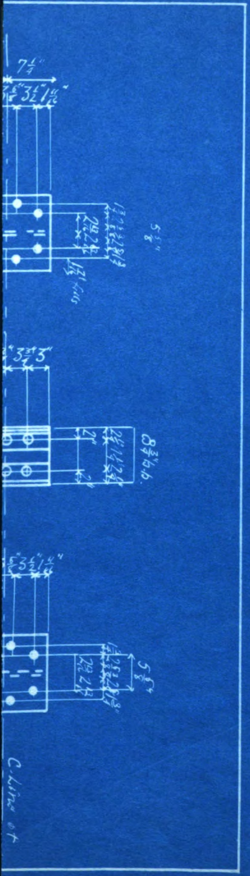
4 Ls (cm) 6 x 4 x 3/8 = 14.4 in.^{2}

l = 22' = 264"

Axis 1-1	I of 4 Ls (cm) 4 x 13.5	=	54.000
	Ad^2 of 4 Ls (cm) 4 x 3.61 x 2.127^2	=	65.145
	Moment of Inertia of gross section	=	119.145
	Radius of gyration = 2.87 in.		

$P = 9000 - \frac{40 \times 264}{2.87} = 5321$ #/in.^{2}

$\frac{68000}{5321} = 12.8$ in.^{2} reqd. Gross section = 14.4 in.^{2}


C. Line

ANALYSIS OF TENSION MEMBERS

All parts of the structures shall be proportioned in tension by the following allowed unit strains, net sections:

For medium steel	Pounds per sq. in.	
Floor beams, hangers when permitted-----------	6000	
Longitudinal, lateral and sway bracing, for lateral forces---------------------------	18000	
Longitudinal, lateral and sway bracings, for live load--------------------------------	12000	
Solid rolled beams, used as cross floor beams and stringers---------------------	10000	
Bottom flanges of plate girders chords and webs of lattice and pin-	Dead load.	Live load.
connected trusses-------------------	20000	10000
Verticals carrying floor beams-----------	16000	8000

The required sectional area of any member is obtained by dividing the sum af (Live $\frac{1}{2}$ dead load strains) by the live load unit strains since the live load unit strains are equal to $\frac{1}{2}$ of the dead load unit strains.

In members subjected to tensile strains full allowance shall be made for reduction of section by rivit-holes, screw threads, etc.

7/8" rivets are used throughout, therefore a 1" hole is to be deducted.

SECTION OF DIAGONAL U1 L2 (D)

Stress in U1 L2 = 174360 # Allowable unit stress 10000 #/in.^{2}

$\frac{174350}{10000}$ = 17.43 in.^{2} reqd. net sedtion.

4 Ls 6 x 4 x 9/16 = 4 x 5.31 = 21.24 in.^{2} Gross section.

4 rivet holes @ .5625 = 2.25

21.24 - 2.25 = 18.99 in.^{2} Actual net section.

SECTION OF DIAGONAL U2 L3 (D1)

Stress in U2 L3 = 138650 #

$\frac{138650}{10000}$ = 13.87 in.^{2} reqd. net section.

4 Ls 6 x 4 x 3/8 = 4x 4.75 = 19.0 in.^{2}

4 rivet holes @ .5 = 2.0

19.0 - 2.0 = 17.0 Actual net section.

SECTION OF LOWER CHORD Lo L1 AND L1 L2

Stress in Lo L1 = 180300 #

$\frac{180300}{10000}$ = 18.03 in.^{2} reqd. net section.

4 Ls 6 x 4 x 9/16 = 4 x 5.31 = 21.24 in.^{2}

4 rivet holes @ .5625 = 2.25

21.24 - 2.25 = 18.99 in.^{2} Actual net section.

SECTION OF LOWER CHORD L2 L3

Stress in L2 L3 = 282700 #

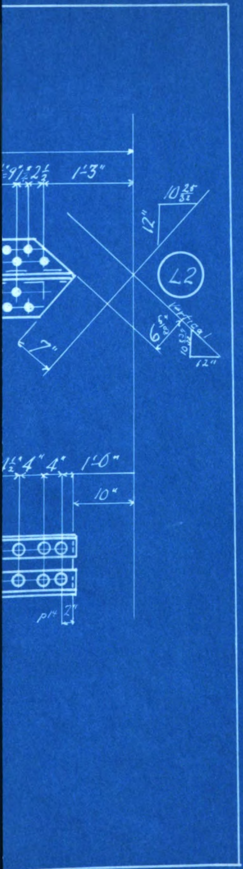
$\frac{282700}{10000}$ = 28.27 in.^{2} reqd. net section.

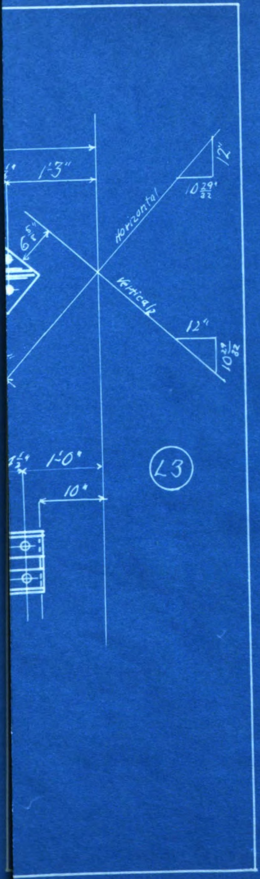
4 Ls 6 x 4 x 5/8 = 4 x 5.86 = 23.44 in.^{2}

2 pl. 13 x 3/8 2 x 4.875 = 9.75 "

33.19 "

8 rivet holes = 4.0 in.^{2} 33.19 - 4.0 = 29.19 in.^{2} net sec.


1'-3"
12"
L2
Vertical
7"
12"
4"
4"
1'-0"
10"
2"



1'-3"
Horizontal
Vertical
12"
1'-0"
10"
L3

SECTION OF HANGER U1 L1 (V1)

Stress in U1 L1 = 90000 #

$\frac{90000}{10000}$ = 9.0 in.^{2} reqd. net section.

4 Ls 6 x 4 x 3/8 = 4 x 3.61 = 14.44 in.^{2} Gross section.

4 rivet holes @ .375 = 1.50

14.44 - 1.50 = 12.94 in.^{2} Actual net section.

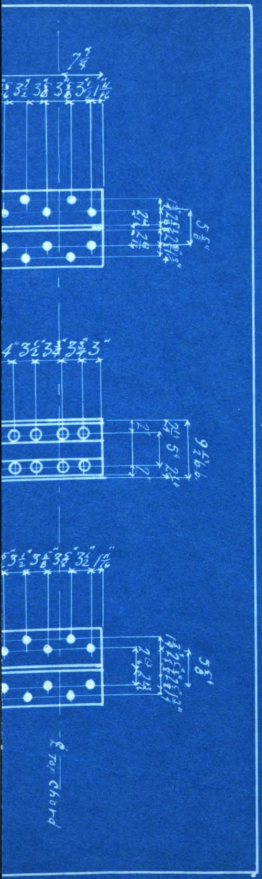
ANALYSIS OF LATERAL BRACING

To provide for wind and vibrations from high-speed trains:

The top lateral bracing in deck bridges, and the bottom lateral bracing in through bridges shall be proportioned to resist a lateral force of 600 pounds for each foot of the span; 450 pounds of this to be treated as a moving load, and as acting on a train of cars, at a line 6 feet above base of rail.

The bottom lateral bracing in deck bridges, and the top lateral bracing in through bridges, shall be proportioned to resist a lateral force of 200 pounds for each linear foot for spans up to 200 feet.

All lateral, sway and portal bracing must be made of shapes capable of resisting compression as well as tension, and must have riveted connections.


℄ Top Chord

UPPER LATERAL BRACING

Since the lateral diagonals are stiff members designed to take both tension and compression it will be assumed that the shear in each panel is equally divided between the two diagonals - P 163 M and J Vol. II.

Shear in first panel = 36000 - 6000 = 30000 #

Stress in B10-11-12 = 30000 x 1.834 = 55000

Stress in B10 etc. = 55000 ÷ 2 = 27500

3/4 of 27500 = 20625# = live load stress in B10.

1/4 of 27500 = 6875 = dead " " " ".

The other diagonals in the upper lateral bracing are all designed for the stresses in the first panel.

Stress in H10 = 6000 #

Stress in H11-12 = 12000 #

Section of diagonal B10

1 L 5 x $3\frac{1}{2}$ x 3/8 = 3.05 in.^{2} Gross section.

2 rivet holes @ .375 = .75 in.^{2}

2.30 in.^{2} Actual net section.

$\frac{20625}{12000} = 1.72$ in.^{2} $\qquad \frac{6875}{18000} = .38$ in.^{2}

Required net section = 2.1 in.^{2}

Section of Strut H10

1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 = 2.48 in.^{2} Gross section.

2 rivet holes = .75 $\qquad$ 1 = 13' = 256"

1.73 in.^{2} Actual net section.

For lateral struts and rigid bracing $P = 13000 - 60\frac{l}{r}$

r for 1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 = 1.07

$\frac{6000}{4253} = 1.41$ in.^{2} reqd. net section.

$$P = 13000 - \frac{60 \times 256}{1.07} = 4253 \ \#/\text{in.}^2$$

Section of Strut H11

2 Ls $3\frac{1}{2} \times 3\frac{1}{2} \times 3/8 = 2 \times 2.48 = 4.96$ in.^{2} Gross section.

2 rivet holes @ .75 = 1.50

$4.96 - 1.50 = 3.46$ in.^{2} Actual net section.

$r = 1.06$ in.

$P = 13000 - \frac{60 \times 156}{1.06} = 4170$ #/in.^{2}

$\frac{12000}{4170} = 2.87$ in.^{2} reqd. net section.

Section of strut H12 is same as H11.

LOWER LATERAL BRACING

The lower lateral bracing in deck bridges shall be proportioned to resist a lateral force of 200 pounds per lineal foot for spans up to 200 feet.

$L = 200 \times 20 = 4000$ # per panel. sec $\theta = 1.834$

Shear in first panel $= 12000 - 2000 = 10000$ #

Stress in L8-9-10 $= 10000 \times 1.834 = 18340$

Section of Diagonal L8

1 L $3\frac{1}{2} \times 3\frac{1}{2} \times 3/8 = 2.48$ in.^{2}

2 rivet holes = .75

$2.48 - .75 = 1.73$ in.^{2} Actual net section.

$\frac{18340}{18000} = 1.02$ in.^{2} reqd. net section.

Sections of other diagonals same as L8.

Section of Strut H8

2 Ls $5 \times 3\frac{1}{2} \times 3/8 = 6.10$ in.^{2} $r = 1.44$ in.

2 rivet holes = .75 $P = 13000 - \frac{60 \times 156}{1.44} = 6500$

$6.10 - .75 = 5.35$ in.^{2} Actual net section.

H8 will take stress of $5.35 \times 6500 = 34775$ #

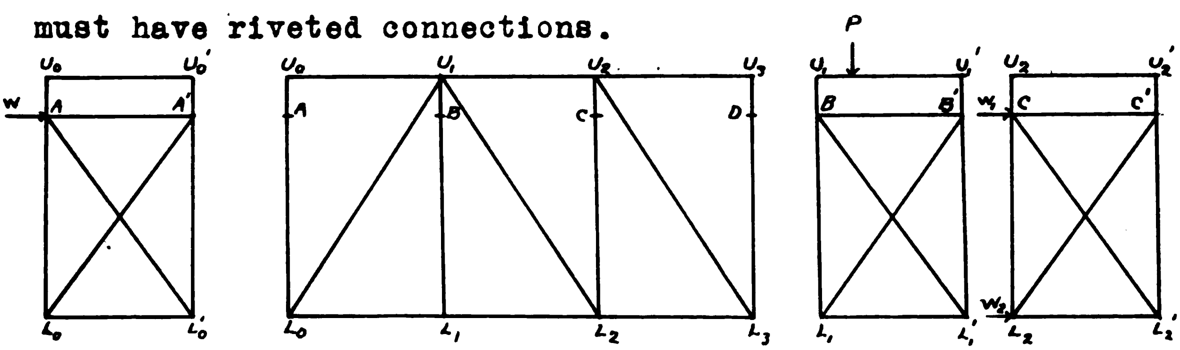
Extra sectional area is needed to stiffen lower section.

ANALYSIS OF SWAY BRACING

All deck bridges shall have transverse bracing at the ends, and at each panel.point of sufficient strength to carry half the maximum stress increment due to lateral and centrifugal forces.

All members of the web, lateral, longitudinal or sway systems must be securely riveted at their intersections to prevent sagging and rattling.

All lateral, sway and portal bracing must be made of shapes capable of resisting compression as well as tension, and must have riveted connections.



Panel C L2 L2' C'

W1 and W2 represent the wind loads at each panel, W2 being the smaller. The difference W1 - W2 would distort the panel, if the sway bracing did not prevent it, and hence one-half this difference may be considered to act as shear across the rectangle.

Both diagonals are tension rods and C L2' receives the tensile stress $\frac{1}{2}(W1 - W2) \sec \theta$.

W1 = Panel wind load at top = 12000 #

W2 = " " " bottom = 4000 #

Shear across rectangle = $\frac{1}{2}$ (W1 - W2) = 4000#

Stress in diagonal CL2' = 4000 x sec θ

Sec θ = 1.23   4000 x 1.23 = 4920#

Panel B L1 L1' B'

Another function of the sway bracing is to prevent the distortion of the bridge under eccentric load. In rectangle B L1 L1' B' an eccentric load may come. The reactions of this load at the ends of the floor beam B B' are unequal, and since the trusses are made alike their deflections would be proportional to these reactions unless they were equalized by the sway bracing. If R and R' be these reactions, one-half of R - R' acts as shear across the rectangle, and produces the tensile stress $\frac{1}{2}$ (R - R') sec θ in the tie B L1'.

R = 36000   R' = 12000   sec θ = 1.23

Stress in B L1' = $\frac{1}{2}$ (36000 - 12000) 1.23 = 29880 #

Allowable stress = 18000 #/in.^{2}

$\frac{29880}{18000}$ = 1.65 in.^{2} required.

1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 = 2.48 in.^{2}

2 rivet holes   .75

2.48 - .75 = 1.73 in.^{2} Actual net section.

The stress in this last piece being greater than the one first computed, this was used throughout the bridge except in the end panels.

End Panel A Lo Lo' A'

The bracing in the end rectangle A' Lo' Lo A must be heavier than the sway bracing between intermediate posts, because its function is to carry wind pressure and in some cases cintrifugal load from the upper latteral system to the abutment. If the sway bracing equalizes the wind loads between the two chords, as suggested above, the horizontal force W os one-fourth of the total wind pressure on the bridge; if the upper lateral bracing carries all the wind pressure specified for the upper chord, the W is one-half of this wind pressure.

The stress in A' Lo = W sec θ

W = $\frac{1}{2}$ of total wind pressure for upper chord = 34000 #

Sec θ = 215.375 ÷ 172.5 = 1.245

34000 x 1.245 = 42330 # Allowable stress = 18000 #/in.^{2}

$\frac{42330}{18000}$ = 2.3 in.^{2} Required net section.

1 L 5 x $3\frac{1}{2}$ x 3/8 = 3.05 in.^{2}

2 rivet holes = .75

3.05 - .75 = 2.30 in.^{2} Actual net section.

The extra sectional area in members of the sway bracing adds additional stiffness to the cross section of the bridge.

FLOOR BEAM (FB)

Web Plate

The span of the floor beam (effective length) shall be considered as the perpendicular distance between the center lines of the trusses = 13.0'. The floor beam carries in addition to its weight, two concentrated loads 3'-6" from its center, each load consisting of the maximum sum of the adjacent reactions of the stringers on both sides and of the track it supports, and the corresponding live load. From the analysis of the stringers the total dead load of one of the stringers was found to be 12600 #.

The equivalent uniform live load must be taken for a span of two panel lengths or 40 feet, and by means of Table I, Coopers Specifications 1906, it is found to be 9048 # per lineal foot of track, for E-50 loading. This would give a uniform live load on each stringer of 4524 # per lineal foot, or the total live load at each stringer connection would be 4524 x 20 = 90480 #. The weight of the floor beam will be assumed as being 2500#. The total maximum vertical shear would be 90480 + 1250 + 12600 = 104330 #. The allowable shearing stress is 10000 #/in.^{2}

The net area required to resist this vertical shear is

104330 ÷ 10000 = 10.43 in.^{2} Required net section.

1 web plate 34 x $\frac{1}{2}$ = 17.0 in.^{2}

6 rivet holes = 3.0

17.0 - 3.0 = 14.0 in.^{2} Actual net section.

The web plate used is plenty safe and the excess area will do away with the necessity of stiffeners, as the span is short.

Flanges

The bending moment due to the two concentrated loads equals 103080 x 3 = 309240 ft. #, and that due to the weight of the floor beam is 4026 ft. #, making a total of 313302 ft. #. Assuming an effective depth of 30.58 inches and allowing a unit stress of 15000 #/ in.^{2} the required net area of the lower flange is as follows:-

$$\frac{313302 \times 12}{30.58 \times 15000} = 8.2 \text{ in.}^2$$ Required net section.

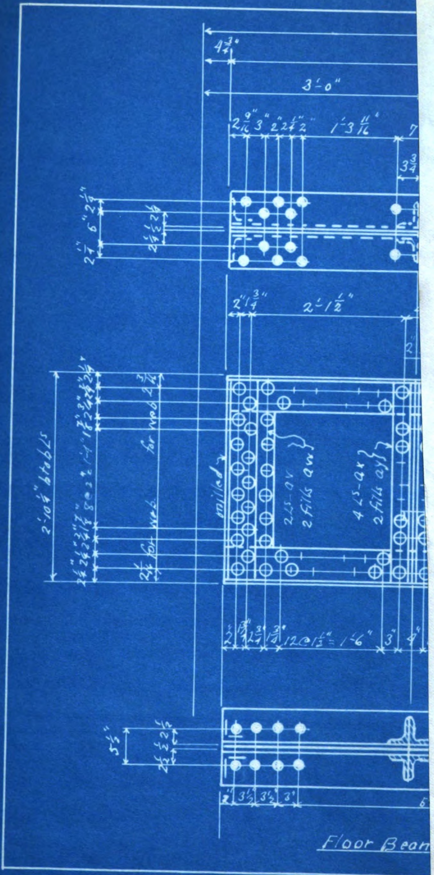
2Ls 6 x 6 x 9/16 = 6.43 x 2 = 12.86 in.^{2} gross section.

2 rivet holes @ .5625 = 1.13

12.86 - 1.13 = 11.73 in.^{2} Actual net section.

In the analysis of the web plate an allowance was made for 6 rivets , which is the maximum number used, a uniform pitch of 2-3/16" being used throughout.

All floor beams used in the structure are identical in section and vary only a few inches in length. The other floor beams are F B1 and F B2. Four Ls 5 x 3½ x 3/8 are used as stiffeners under each stringer connection, which acts as a column with an excess of steel in cross-section. The floor beams are connected to the posts by means of 2Ls 5x 5 x3/8, a rivet pitch of 2" is used. See estimate of total weight for actual weight of floor beam.



3'-0"
1'-3 11/16"
2'-1 1/2"
12@1 1/2" = 1'-6"
milled
for web
2 ∠s-av
2 Fills aw
4 ∠s-ax
2 Fills ay
2'-10 1/4" Back to Back
Floor Beam

TRACK STRINGERS

The span of the stringer equals the panel length of the truss or 20 feet. The loading is a load of 120000 # equally distributed on two pairs of driving wheels spaced 6 feet center to center.

The maximum bending moment for one stringer equals 247500ft.# (Table I Coopers Specifications) The total dead load on one stringer equals 12600 #. The dead load moment equals 30500 ft. #. The total moment equals 30500 + 247500 = 278000 ft.#

The diagram of end shears gives 60000 # for a span of 20 feet. The shear due to dead load is 6300 #, thus giving a total vertical shear of 66300 #.

Allowable shearing stress is 10000# / in.^{2}

$\frac{66300}{10000} = 6.63$ in.^{2} Required net section.

1 Web plate 32 x $\frac{1}{2}$ = 16.0 in.^{2} gross section.

7 rivets @ 1/2 = 3.5

16.0 - 3.5 = 12.5 in.^{2} Actual net section.

This section is large enough so that stiffeners are not necessary. As flange angles 6 inches wide are most suitable for stringers without cover plates, the clear distance between flange angles is 26 inches. The extra material in the web plate increases the stiffness of the stringer.

Flanges

The specified allowable tensile stress is 10000 #/ in.^{2}

Bending moment to be resisted by the lower flange 278000 ft.#

$\frac{278000 \times 12}{28.58 \times 10000} = 11.7$ in.^{2} Required net section.

2 Ls 6 x 6 x 9/16 2 x 6.43 = 12.86 in.^{2} gross section.

2 rivet holes @ .562 = 1.12

12.86 - 1.12 = 11.74 in.^{2} Actual net section.

The effective depth is taken as 28.58 inches. (The distance between the centers of gravity of the flange areas will be considered as the effective depth of all plate girders.) Cooper specifies that plate girders shall be proportioned upon the supposition that the bending or chord stresses are resisted entirely by the upper and lower flanges, and that the shearing or web stresses are resisted entirely by the web plate ; no part of the web plate shall be estimated as flange area.

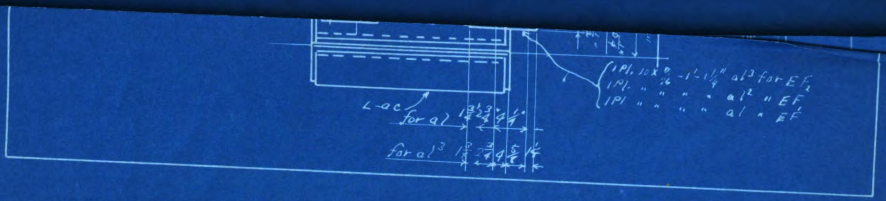
Rivet Pitch in Flanges.

The maximum vertical shear at the end is 66300 #, and the increment of flange stress per lineal inch is-----

$$\frac{11.74}{11.7} \times \frac{66300}{28.58} = 2530 \# .$$

The vertical load on the flange is $\frac{24024}{32} = 750$

The resultant of these horizontal and vertical components is 2450 pounds. The allowable bearing of a 7/8 inch rivet in a ½ inch web is 7880 #, and hence the theoretic rivet pitch at the end is 7880 / 2450 = 3.2 in. The pitch used runs from 2 inches at the ends to 5 inches at the center. The sections of all stringers ,S, S1 and S2 are the same except for a slight variation in length.



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ESTIMATE OF WEIGHT

Material for one Span

End Post LO U1 Four required.

2Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 7/16 x 28'-9" @ 9.8 # -----------	563.50 #
2Ls " " " x 28'-$4\frac{3}{4}$" @ 9.8 -------------	556.55
2Wbs 19" x $\frac{1}{2}$"x 23'-$2\frac{1}{4}$" @ 32.3 -----------------	1495.17
1Cov " " " " @ 32.3------------------	747.58
2Pl $42\frac{1}{4}$" x 15/16 x 4'-$8\frac{1}{4}$" ---------------------	103.73
1Pl 19" x 3/8" x 1'-6" @ 24.23 ----------------	36.35
1Pl 19" x 3/8" x 2'-$0\frac{1}{2}$" @ 24.23 ---------------	49.43
1Pl " " x 1'-$7\frac{3}{4}$" @ 24.23 ---------------	39.88
50 Lat. bars $2\frac{1}{2}$" x 7/16" x 1'-5 11/16" @ .5 ----	25.00
Total weight of one post --------------	3617.17 #

Upper Chord UO U1 Four required.

4Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 5/16 x 18'-$7\frac{1}{2}$" @ 7.2 # -----------	535.68 #
2Wbs 18" x 5/16" x 18'-$7\frac{1}{2}$" @ 19.13 -------------	711.36
1Pl 19" x 3/8" x 2'-2" @ 24.23 ----------------	52.34
1Pl " " x 2'-$2\frac{1}{4}$" @ 24.23 ---------------	52.82
2Pl " " x 1'-7" @ 24.23 ----------------	77.34
2Pl $30\frac{3}{4}$" x $\frac{1}{2}$" x 1'-$10\frac{1}{4}$" @ 52.28 ---------------	193.75
44 Lat. bars $2\frac{1}{2}$" x 7/16" x1'-5 11/16" @ .5 -----	22.00
Total weight of one section -------------	1645.29 #

Upper Chord U1 U2 Four required.

4 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 21'-$4\frac{1}{2}$" @ 8.5 # -------------	727.77 #
2 webs 18 x $\frac{1}{2}$ x 21'-$4\frac{1}{2}$ @ 30.60 # -----------	1309.68
1 cover 19" x 7/16" x 21'-$4\frac{1}{2}$" @ 28.26 # ----	604.76
2 pl. 47 3/4 x $\frac{1}{2}$ x 6'-0" @ 81.18 # ---------	974.16
2 pl. 19 x 3/8 x 1'-6" @ 24.23 # -----------	72.69
36 lat. bars (same as above) --------------	18.00
2 pl. 47 3/4 x $\frac{1}{2}$ x 4'-1 3/4" @ 81.18 # -----	663.97
Total weight of one section -----------	4372.03 #

Upper Chord U2 L2 Two required.

4 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 7/16 x 41'-$1\frac{1}{2}$" @ 9.8 # ------	1612.10 #
2 webs 18 x $\frac{1}{2}$ x 41'-$1\frac{1}{2}$" @ 30.6 # -----------	2516.85
1 cover 19 x 7/16 x 41'-$1\frac{1}{2}$" @ 28.26 # ------	1162.19
4 pl. 19 x 3/8 x 1'-6" @ 24.23 # -----------	145.38
36 lat. bars (same as before) --------------	18.00
2 pl. 30 3/4 x $\frac{1}{2}$ x 2'-8 1/4" @ 52.28 # -----	280.85
Total weight of section----------------	5735.37 #

Lower Chord B C1 Four required.

4 Ls 6 x 4 x 9/16 x 38'-$8\frac{1}{2}$" @ 18.1 # -------	2800.87 #
15 pl. 6 x 3/8 x 0'-$8\frac{1}{2}$" @ 7.65 # -----------	78.72
2 pl. 40 x $\frac{1}{2}$ x 6'-3" @ 68.0 # --------------	850.00
2 pl. 22 x 3/4 x 0'-7 7/8" @ 56.1 # --------	73.61
2 Ls 6 x 4 x 3/4 x 2'-$1\frac{1}{2}$" @ 23.6 # ---------	100.30
1 pl. $17\frac{1}{2}$ x 3/8 x 2'-$3\frac{1}{2}$" @ 22.31 # ---------	51.31
1 pl. $20\frac{1}{2}$ x 3/8 x 2'-$8\frac{1}{2}$" @ 26.14 # ---------	70.22
Total weight of one section -----------	4025.03 #

Lower Chord B C2	Two required.
4 Ls 6 x 4 x 5/8 x 40'-0" @ 20.0 # ------	3200.00 #
2 pl. 13 x 3/8 x 46'-3" @ 16.58 # -------	1533.65
14 pl. 9 x 3/8 x 0'-8½" @ 11.48 # -------	110.25
6 pl. 10 x 3/8 x 2'-10½" @ 12.75 # ------	229.00
2 pl. 35½ x ½ x 4'-5½" @ 60.35 # --------	543.15
Total weight of one section --------	5616.05 #

Post V	Four required.
4 Ls 6 x 4 x 3/8 x 21'-7¼" @ 12.3 # -----	1057.80 #
2 pl. 12 x 3/8 x 2'-6 7/8" @ 15.3 # -----	76.50
6 bat. 6 x 3/8 x 0'-8¼" @ 7.65 # --------	230.00
1 pl. 8¼ x 3/8 x 6'-11" @ 10.52 # -------	73.64
1 pl. 8¼ x 3/8 x 2'-2¼" @ 10.52 # -------	22.72
Total weight of one section --------	1460.66 #

Hanger V1	Four required.
4 Ls 6 x 4 x 3/8 x 22'-0½" @ 12.3 # -----	1082.40 #
1 pl. 9 x 3/8 x 6'-11" @ 11.48 # --------	80.36
1 pl. 9 x 3/8 x 1'-4½" @ 11.48 # --------	17.22
1 pl. 12½ x ½ x 2'-1 3/4" @ 21.25 # -----	45.90
1 pl. 12½ x 9/16 x 1'-6 7/8" @ 23.91 # --	47.82
1 pl. 12 x ½ x 1'-5 3/4" @ 20.40 # ------	40.80
4 bat. 6 x 3/8 x 0'-9" @ 7.65 # ---------	22.95
Total weight of one section --------	1337.45 #

Post V2 Four required.

4 Ls 6 x 4 x 5/8 x 22'-0$\frac{1}{4}$" @ 20.0 # ------ 1760.00 #

1 pl. 12 x $\frac{1}{2}$ x 1'-5 3/4" @ 20.4 # -------- 30.60

1 pl. 9 x 3/8 x 2'-1" @ 11.48 # ---------- 23.88

1 pl. 9 x 3/8 x 6'-11" @ 11.48 # --------- 80.36

6 bat. 6 x 3/8.x 9" @ 7.65 # ------------- 34.43

Total weight of one section --------- 1929.27 #

Post V3 Two required.

4 Ls 6 x 3$\frac{1}{2}$ x $\frac{1}{2}$ x 22'-0$\frac{1}{4}$" @ 15.3 # ------- 1346.40 #

1 pl. 9 x 3/8 x 2'-1" @ 11.48 # ---------- 23.87

1 pl. 9 x 3/8 x 6'-11" @ 11.48 # --------- 80.36

6 bat. 6 x 3/8 x 9" @ 7.65 # ------------- 34.43

Total weight of one section --------- 1485.06 #

Diagonal D Four required.

4 Ls 6 x 4 x 9/16 x 28'-1 3/4" @ 18.1 # -- 2038.78 #

1 pl. 9 x 3/8 x 3'-8" @ 11.48 # ---------- 42.02

1 pl. 9 x 3/8 x 3'-3$\frac{1}{4}$" @ 11.48 # --------- 37.31

6 bat. 6 x 3/8 x 0'-9" @ 7.65 # ---------- 34.42

Total weight of one section --------- 2152.53 #

Diagonal D1 Four required.

4 Ls 6 x 3½ x ½ x 28'-1 3/4" @ 15.3 # ------ 1723.39 #

1 pl. 9 x 3/8 x 3'-2¼" @ 11.48 # ----------- 36.74

1 pl. 9 x 3/8 x 2'-5 3/8" @ 11.48 # -------- 34.32

11 bat. 6 x 3/8 x 0'-9" @ 7.65 # ----------- 63.19

Total weight of one section ----------- 1857.64 #

Lower Lateral Bracing Two required.

1 L 3½ x 3½ x 3/8 x 21'-9½" @ 8.5 # -------- 215.30 #

1 L 3½ x 3½ x 3/8 x 9'-6 3/4" @ 8.5 # ------ 80.75

1 L 3½ x 3½ x 3/8 x 11'-8 3/8" @ 8.5 # ----- 99.88

1 pl. 7 x 3/8 x 2'-1 3/4" @ 8.93 # --------- 19.31

2 Ls 3½ x 3½ x 3/8 x 0'-6 3/4" @ 8.5 # ----- 8.50

2 Ls 3½ x 3½ x 3/8 x 22'-5" @ 8.5 # -------- 382.50

2 Ls 3½ x 3½ x 3/8 x 11'-0¼" @ 8.5 # ------- 18.75

2 Ls 3½ x 3½ x 3/8 x 10'-11¼" @ 8.5 # ------ 18.65

4 Ls 3½ x 3½ x 3/8 x 0'-6 3/4" @ 8.5 # ----- 20.40

2 pl. 7 x 3/8 x 2'-1" @ 8.93 # ------------- 35.72

1 L 5 x 3½ x 3/8 x 2'-10¼" @ 10.4 # -------- 31.20

Total weight of one section ----------- 930.96 #

Lower Strut H B Two required.

2 Ls 5 x 3½ x 7/16 x 11'-4" @ 13.5 # ------- 305.91 #

2 Ls 5 x 3½ x 7/16 x 10'-0" @ 13.5 # -------- 270.00

8 lat. bars 2 3/4 x 5/8 x 2'-1 5/8" @ .5 # -- 4.00

2 pl. 23 x ½ x 3'-2" @ 39.1 # -------------- 247.11

4 Ls 6 x 4 x 3/8 x 1'-8¼" @ 12.3 # --------- 86.10

Total weight of one section ----------- 913.12 #

Lower Strut H9 Five required.

2 Ls 5 x 3½ x 3/8 x 11'-5" @ 10.4 # --------	239.20 #
2 pl. 14½ x 3/8 x 1'-5" @ 18.49 # ----------	55.47
4 Ls 3½ x 3½ x 3/8 x 1'-5" @ 8.5 # ---------	51.00
Total weight of one section -----------	345.67 #

Upper Latteral Strut H10 Two required.

1 L 3½ x 3½ x 3/8 x 14'-7" @ 8.5 # ---------	124.10 #

Strut H11 Two required.

2 Ls 3½ x 3½ x 3/8 x 14'-7" @ 8.5 # --------	248.20 #
4 fills 3 x 3/8 x 1'-0½" @ 3.83 # ----------	15.62
Total weight of one section -----------	263.82 #

Strut H12 Three required.

2 Ls 3½ x 3½ x 3/8 x 14'-7" @ 8.5 # --------	124.10 #

Sway Bracing in Intermediate Posts Five required.

2 Ls 3½ x 3½ x 3/8 x 19'-7 5/8" @ 8.5 # ----	334.22 #
4 Ls 3½ x 3½ x 3/8 x 0'-6½" @ 8.5 # --------	17.00
1 pl. 7 x 3/8 x 0'-8 3/4" @ 8.93 # ---------	6.25
2 pl. 13½ x 3/8 x 1'-3½" @ 17.21 # ---------	43.03
4 Ls 4 x 3½ x 3/8 x 1'-0" @ 9.1 # ----------	36.00
4 Ls 4 x 3½ x 3/8 x 1'1½" @ 9.1 # ----------	40.04
Total weight of one section -----------	476.54 #

Upper Latteral Bracing Four required.

1 L 5 x 3½ x 3/8 x 21'-9 3/4" @ 8.5 # ------ 184.96 #

1 L 5 x 3½ x 3/8 x 10' -7" @ 8.5 # --------- 124.10

1 L 5 x 3½ x 3/8 x 10'-7½" @ 8.5 # --------- 125.00

1 pl. 8½ x ½ x 2'-10½" @ 14.03 # ----------- 42.00

2 Ls 3½ x 3½ x 3/8 x 0'-9½" @ 8.5 # -------- 12.65

2 Ls 3½ x 3½ x 3/8 x 0'-11 3/4" @ 8.5 # ---- 17.00

2 Ls 3½ x 3½ x 3/8 x 0'-9½" @ 8.5 # -------- 12.65

1 pl. 14 x 3/8 x 3'-9¼" @ 17.85 # ---------- 66.94

1L 4 x 3½ x 3/8 x 1'-3" @ 9.1 # ------------ 11.38

1 L 4 x 3½ x 3/8 x 2'-3¼" @ 9.1 # ---------- 20.48

1 pl. 14 x 3/8 x 3'-6½" @ 17.85 # ---------- 62.47

½ L 4 x 3½ x 3/8 x 3'-1 1/8" @ 9.1 # ------- 13.75

½ pl. 12½ x 3/8 x 3'-7" @ 15.94 # ---------- 55.79

Total weight of one section ----------- 749.17 #

Bracing of Inclined End Post Four required.

2 Ls 3½ x 3½ x 3/8 x 15'-9" @ 8.5 # -------- 267.75 #

6 pl. 9 x 3/8 x 1'-6¼" @ 11.48 # ----------- 103.32

2 Ls 3½ x 3½ x 3/8 x 7'-8½" @ 8.5 # -------- 130.22

3 pl. 9 x 3/8 x 1'-6¼" @ 11.48 # ----------- 51.66

2 Ls 3½ x 3½ x 3/8 x 7'-8½" @ 8.5 # -------- 130.22

3 pl. 9 x 3/8 x 1'-6¼" @ 11.48 # ----------- 51.66

4 pl. 14½ x 3/8 x 2'-0 3/8" @ 18.49 # ------ 148.00

2 pl. 12 x 3/8 x 1'-9 5/8" @ 15.3 # -------- 55.08

1 fill. 3 x ½ x 1'-6 7/8" @ 5.1 # ---------- 7.65

2 pl. 12 x 3/8 x 1'-9 5/8" @ 15.3 # -------- 55.08

Total weight of one section ----------- 1000.64 #

Strut H5 Two required.

4 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 11'-$4\frac{1}{4}$" @ 8.5 # -------	391.00 #
8 lat. bars $2\frac{1}{2}$ x 3/8 x 11'-$4\frac{1}{4}$" @ 3.19 # ----	292.68
2 pl. 18 x 3/8 x 1'-3" @ 22.95 # -----------	57.38
Total weight of one section -----------	741.06 #

Strut H6 Two required.

4 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 11'-$4\frac{1}{4}$" @ 8.5 # -------	391.00 #
8 lat. bars (same as above) @ 3.19 # -------	292.68
2 pl. 18 x 3/8 x 3/8 x 11'- $4\frac{1}{4}$" @ 22.95 # --	57.38
Total weight of one section -----------	741.06 #

Sway Bracing in Vertical End Post Two required.

2 Ls 5 x $3\frac{1}{2}$ x 3/8 x 18'-2 3/8" @ 10.4 # ----	37.79 #
4 Ls $3\frac{1}{2}$ x 3 x 3/8 x 0'-$11\frac{1}{2}$" @ 7.9 # --------	31.60
1 pl. $10\frac{1}{2}$ x 3/8 x 0'-$8\frac{1}{2}$" @ 13.39 # ---------	8.97
2 pl. $15\frac{1}{4}$ x 3/8 x 1'-5" @ 19.45 # ----------	58.35
4 Ls 4 x $3\frac{1}{2}$ x 3/8 x 1'-$3\frac{1}{4}$" @ 9.1 # ---------	45.50
4 Ls 4 x $3\frac{1}{2}$ x 3/8 x 1'-$1\frac{1}{2}$" @ 0.1 # ---------	42.96
Total weight of one section -----------	225.17 #

Stringer S Four required.

4 Ls 6 x 6 x 9/16 x 19'-11 3/4" @ 21.9 # ---	1732.00 #
4 Ls 6 x 6 x $\frac{1}{2}$ x 2'-7 1/8" @ 19.6# ---------	203.84
4 fill. 6 x 9/16 x 1'-$8\frac{1}{4}$" @ 11.48 # --------	76.23
1 web 32 x $\frac{1}{2}$ x 19'-11 3/4" @ 54.4 # --------	1088.00
Total weight of one section -----------	3100.07 #

Stringer S1 Four required.

4 Ls 6 x 6 x 9/16 x 19'-11 3/4")

4 Ls 6 x 6 x 9/16 x 2'-7 1/8")

4 fill. 6 x 9/16 x 1'-8$\frac{1}{4}$") Same as S.

1 web 32 x $\frac{1}{2}$ x 19'-11 3/4") 3100.07 #

Stringer S2 Two required.

4 Ls 6 x 6 x 9/16 x 21'-1" @ 21.9 # --------	1846.61 #
4 Ls 6 x 6 x $\frac{1}{2}$ x 8'-7 1/8" @ 19.6 # --------	203.84
1 web 32 x $\frac{1}{2}$ x 21'-1" @ 54.4 # -------------	1088.00
4 fill. 6 x 9/16 x 1'-8$\frac{1}{4}$" @ 11.48 # --------	76.23
Total weight of one section -----------	3214.68 #

Stringer S3 Two required.

Same as stringer S2 3214.68 #

Floor Beam F B Four required.

4 Ls 6 x 6 x 9/16 x 12'-1 3/8" @ 21.9 # ----	1059.96 #
1 web 34 x $\frac{1}{2}$ x 12'-1 3/8" @ 57.8 # ---------	699.38
4 Ls 5 x 5 x 5/8 x 2'-9 1/8" @ 20.0 # ------	220.00
4 fills 8 x 9/16 x 1'-10$\frac{1}{4}$" @ 15.3 # --------	122.40
8 Ls 5 x 3$\frac{1}{2}$ x 3/8 x 2'-9 1/8" @ 10.4 # -----	228.80
4 fills 7 x 9/16 xl'-10$\frac{1}{4}$" @ 13.39 # --------	107.00
Total weight of one section -----------	2437.54 #

End Frame for Stringers Seven required.

2 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 5'-$11\frac{1}{4}$" @ 8.5 # -------	102.00 #
1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 5'-$9\frac{1}{2}$" @ 8.5 # ---------	48.88
2 pl. 10 x 7/16 x 1'-$1\frac{1}{4}$" @ 14.88 # ---------	32.74
Total weight of one section -----------	183.62 #

Floor Beam F B1 Two required.

4 Ls 6 x 6 x 9/16 x 12'-$3\frac{1}{4}$" @ 21.9 # -------	1073.10 #
1 web 34 x $\frac{1}{2}$ x 12'-$3\frac{1}{4}$" @ 57.8 # ------------	708.05
4 Ls 5 x 5 x 5/8 x 2'-9 1/8" @ 20.0 # ------	220.00
4 fills 8 x 9/16 x 1'-$10\frac{1}{4}$" @ 15.3 # --------	122.40
8 Ls 5 x $3\frac{1}{2}$ x 3/8 x 2'-9 1/8" @ 10.4 # -----	228.80
4 fills 7 x 9/16 x 1'-$10\frac{1}{4}$" @ 13.39 # -------	107.00
Total weight of one section -----------	2459.35 #

Floor Beam F B2 One required.

4 Ls 6 x 6 x 9/16 x 12'-$2\frac{1}{2}$" @ 21.9 # -------	1068.72 #
1 web 34 x $\frac{1}{2}$ x 12'-$2\frac{1}{2}$" @ 57.8 # ------------	705.16
4 Ls 5 x 5 x 5/8 x 2'-9 1/8" @ 20.0 # ------	220.00
4 fills 8 x 9/16 x 1'-$10\frac{1}{4}$" @ 15.3 # --------	122.40
8 Ls 5 x $3\frac{1}{2}$ x 3/8 x 2'-9 1/8" @ 10.4 # -----	228.80
4 fills 7 x 9/16 x 1'-$10\frac{1}{4}$" @ 13.39 # -------	107.00
Total weight of one section -----------	2452.08 #

Lateral Bracing for Stringers   Two required.

9 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 8'-9" @ 8.5 # --------- 669.38 #

6 pl. $10\frac{1}{2}$ x 3/8 x 1'-$10\frac{1}{4}$" @ 13.39 # -------- 160.68

6 pl. $10\frac{1}{2}$ x 3/8 x L8 -$1\frac{1}{2}$" @ 13.39 # -------- 88.37

1 L 5 x $3\frac{1}{2}$ x 3/8 x 21'-9" @ 10.4 # --------- 226.20

2 pl. $12\frac{1}{2}$ x 3/8 x 2'-2" @ 15.94 # ---------- 68.63

2 Ls 4 x 4 x 3/8 x 1'-$0\frac{1}{2}$" @ 9.8 # ---------- 19.60

2 Ls $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 0'-$11\frac{1}{2}$" @ 8.5 # ------- 17.00

1 L 5 x $3\frac{1}{2}$ x 3/8 x 10'-10" @ 10.4 # -------- 20.80

1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 0'-11" @ 8.5 # --------- 8.50

1 pl. $8\frac{1}{2}$ x $\frac{1}{2}$ x 2'-$7\frac{1}{2}$" @ 14.45 # ------------ 37.57

1 L 5 x $3\frac{1}{2}$ x 3/8 x 10'-$1\frac{1}{2}$" @ 10.4 # -------- 10.50

1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 0'-$11\frac{1}{4}$" @ 8.5 # -------- 8.50

1 L $3\frac{1}{2}$ x $3\frac{1}{2}$ x 3/8 x 0'-9 3/16" @ 8.5 # ----- 6.46

Total weight of one section ----------- 1342.19 #

ESTIMATE OF WEIGHT

4 end posts L O U1	4 x 3617.17	14468.68 #
4 upper chord Uo U1	4 x 2645.29	6582.16
4 " " U1 U2	4 x 4372.03	17488.12
2 " " U2 U2	2 x 5735.37	11470.74
4 Lower " B C1	4 x 4025.03	16100.12
2 " " B C2	2 x 5616.05	11232.10
4 Post V	4 x 1460.66	5842.64
4 Hanger V1	4 x 1337.45	5349.80
4 Post V2	4 x 1929.27	7717.08
2 " V3	2 x 1485.06	2970.12
4 Diagonal D	4 x 2152.53	8610.12
4 " D1	4 x 1857.64	7430.56
Lower Lateral	2 x 930.96	1861.92
2 " Strut H8	2 x 913.12	1826.24
5 " " H9	5 x 345.67	1728.35
5 Upper " H10-H12	5 x 124.10	620.50
2 " " H11	2 x 263.82	527.64
" Lateral	4 x 749.17	2996.68
Sway bracing Int.	5 x 476.54	2383.70
" " End	2 x 225.17	450.34
Bracing Inclined Post	4 x 1000.64	4000.66
4 Strut H5-H6	4 x 741.06	2964.24
8 Stringer S-S1	8 x 3100.07	24800.56
4 " S2-S3	4 x 3214.68	12858.72
7 End Frame	7 x 183.62	1285.34

Bracing for Stringer	2 x 1342.19	2684.38 #
4 Floor Beam F B	4 x 2437.54	9750.16
2 " " F B1	2 x 2459.35	4918.70
1 " " F B2	1 x 2452.08	2452.08
Weight of Tradk	20 x 220.00	4400.00
Total weight		197770.45 #
Rivet Heads @ 6% of total weight		12623.65
Total weight		210394.10 #

210394 ÷ 120 = 1754.6 #/ lin. ft. of Bridge.

BIBLIOGRAPHY

Books

MERRIMAN and JACOBY, Roofs and Bridges-Part I-Stresses

MERRIMAN and JACOBY, Roofs and Bridges-Part III
Bridge Design

THEODORE COOPER, General Specifications for
Steel Railroad Bridges 1906

CARNEGIE STEEL CO. Hand Book

CONCLUSION

In concluding this analysis, it may be said that the structure, from the standpoint of design is an exceedingly safe one. Specifications were followed closely and all cross-sectional areas are safely in excess of those required. The design was made without allowing for impact, which may be explained by the fact that at the time the bridge was designed this practise was not closely followed. We can offer no explanation for the live load stress, which may be noted on hanger V1. A general idea of the structure may be gained by reference to the photographs which are to be found in the fore part of this book. As has been before noted, the bridge although being over twenty years old, appears to be in very good condition.

It is the intention of the writers to construct a model of this bridge, to one-eighth scale. The material used being a special molding of cross-section representing the different sizes of angles in the structure. The various sizes of angles being reduced to three sizes with a minimum thickness of one-eighth inch. At the time of writing the model spoken of is not completed.

2/14
Gagie