1895

A complete design of a 210 foot R.R. bridge

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Thesis

For the Degree

of

B. Sc. in C. E.

A Complete Design of A 210 foot R.R. Bridge

By

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1895

Preface.

I have divided the subject into two divisions, viz. Part I and Part II.

Part I treats of the maximum stresses and the methods employed in finding them.

Part II treats of the design of the members to resist the stress.

Part I.

Stresses

Dead Load

Use the formula

$w = 5l + 350$.

weight of floor = 400# per foot

$P = 5l + 750$

$P = 1800$

weight on one truss $= \frac{1800}{2} = 9000$ per ft

The Dead Load diagram in Plate I shows the dead load stresses.

Live Load

I used the concentrated wheel loads as shown in Plate I.

The stress in the upper chords was found by taking moments at the lower joints and dividing by the ⊥ distance to the inclined chords.

The stresses in the lower chords was found by taking moments at the upper chord joints and dividing by the ⊥ distance

The stresses in the web members ~~was~~ were found by taking the max shear in the panels.

To illustrate this take the panel AHDB Plate I

lay of ab (Fig 1) = shear in the pannel draw ac ∥ to AB and cb ∥ to AD

Let T = stress in AD

then we get.

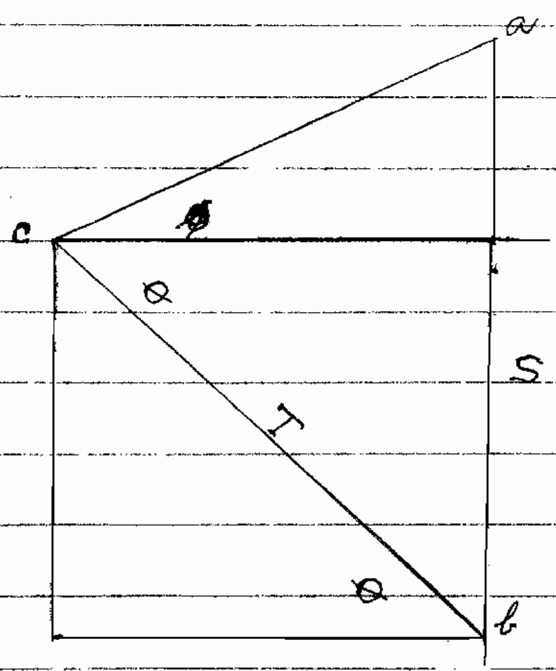


Fig 1

$T \sin\theta + T \cos\theta \tan\phi = S =$ Shear

From this we find T, which is shorter than the methods commonly used.

The stress in BD = the shear

Wind Stress

Is shown in Plate I by the diagrams.

Part Second

Plate 1

Design of Upper Chord

C D

Stress = + 364600#

L = 30′

Used formula

$$P = 8400 - 84\frac{L}{d}$$

$$P = 8400 - 84 \times \frac{360}{18}$$

$P = 6720$ = wrought iron.

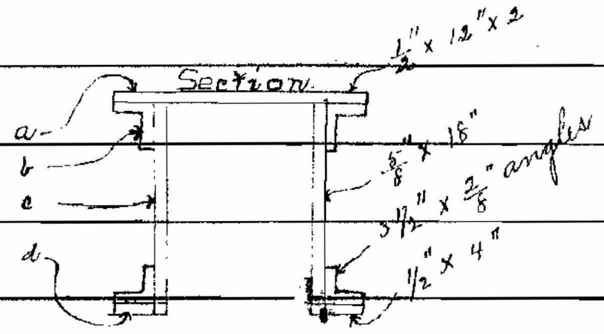
$P = 6720 + 20\%$ = steel.

$P = 8000$ (about)

Area required

$$A = \frac{S}{P} = \frac{364600}{8000} = 45.5 = 46 \text{ sq in.}$$

Section



$a = \frac{1}{2}'' \times 24 = 12$ sq in.

$4b = \frac{3}{8} \times 3\frac{1}{2} = 9$ sq in. $= 9$ = 9

$2c$ = = 22 22

$2d$ = = 4

47 sq in.

B C.

Stress = + 365000#

$\mathcal{L}$ = 30' (about)

$$P = 8400 - 84 \times \frac{360}{18}$$

$$P = 8000$$

Area required

$$a = \frac{S}{P} = \frac{365000}{8000} = 46 \text{ sq in.}$$

Section

Same as in C D.

B A.

Stress = + 327000#

$\mathcal{L}$ = 31' (about)

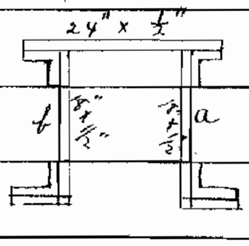
$$P = 8400 - 84 \times \frac{372}{18}$$

$$P = 8000$$

Area required

$$a = \frac{S}{P} = \frac{327000}{8000} = 40 \text{ sq in.}$$

Section



Same as C D and B C except a and b

AD.

Stress = + 351000#

L = 42' {about}

$P = 8750 - 84 \times \frac{504}{18}$

$P = 7250$

Area required.

$A = \frac{S}{P} =$ 41 sq in. (about)

Section

Same as CD. and BC.

N.B.

We use this excess area to resist the wind pressure transmitted to the portal.

2°

Design of Posts.

CF

Stress = + 50000#

L = 40'

$P = 52500 - 220 \frac{L}{R}$

Take channel no 557. $R = 4.49$

$P = 52500 - 220 \times \frac{480}{4.5}$

$P = 5800$ {Safety factor 5}

Area required

$$a = \frac{S}{P} = \frac{50000}{5800} = 8.7 = 9$$

∴ channel no 357 is Ok as two of these have an area of 11.8 sq in.

B C

Stress = + 90000#

L = 37.5'

$$P = 52500 - 220\frac{L}{R}$$

Used channel 357. 30½# per ft.

$$P = 52500 - 220 \times \frac{450}{4}$$

P = 5572 = 5600 {Safety factor 5}

Areal required

$$a = \frac{S}{P} = \frac{90000}{5600} = 16 \text{ sq in.}$$

∴ channel 357 is Ok as two of these have an area of 17.5 sq in.

There are eight posts in the Bridge 4 as the first design and 4 as the second design.

5° **Design of Tension Members**

(15000# per sq in. allowed)

F E.

Stress = + 365000.

Area required

$$A = \frac{S}{P} = \frac{365000}{15000} = 36.5 \text{ sq in.}$$

Use 4 eye bars 9" x 1 1/8" = 40 sq ins.

C E.

Stress = 65000

Area required

$$A = \frac{S}{P} = \frac{65000}{10000} = 6 \text{ sq in.}$$

Use 2 eye bars 4" x 3/4 = 3 sq in = 6.

B F

Stress = 14500#

Area Required

$$A = \frac{S}{P} = \frac{14500}{1000} = 14. \text{ sq.in.}$$

Use 4 eyebars 5" x 3/4"

O F.

Stress = 325 000#

Area required

$A = \frac{325\,000}{10000} = 32.5$

Use 2 eyebars 7" x 1 1/2"

A O.

Stress = 11 000#

Area required

$A = \frac{11\,000}{1100} = 19$ sq. in.

Use 4 eyebars 5 x 1 = 20 sq. in.

I H and H O.

Stress = 250 000

Area required

$A = \frac{25\,000}{10000} = 25$ sq. in.

Use 2 eyebars 9" x 1 1/2 = 27 sq. in.

H A

Stress = S = 9 000

Area required

$$A = \frac{75000}{15000} = 9 \text{ sq in}$$

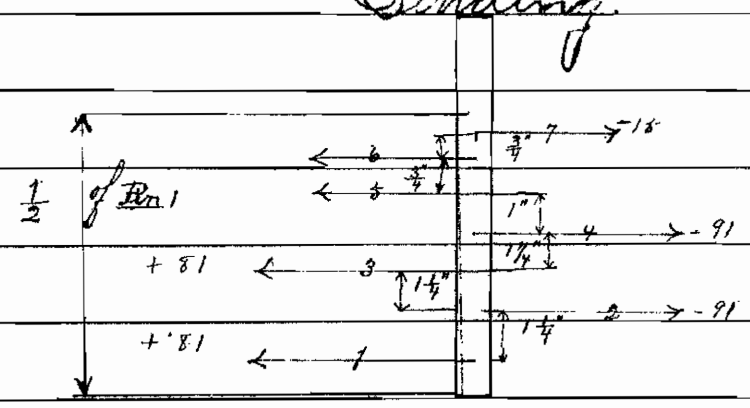
Use 2 eye bars $5'' \times 1'' = 10$ sq. in.

4°

Design of Pin at F.

I will analyze this pin as a beam
1° for Bending.
2° for Crushing
3° for Shear.

Bending.



Members	Shear	Moments.
1	+81	0
2	-10	+101
3	+71	+88.5
4	-20	+177.5
5	0	+157.5
6	0	+152.5
7	+5	+152.5

This analysis comes from this, that the bending moment at any point in a beam {considering the pin as a beam} is equal to the bending moment at any other section + the shear at that section into its arm about the centre of gravity of the section in question + the intervening loads into their respective arms about the C.G. of the section in question.

From this we see that 180 {177.5} is the greatest bending moment on half of the pin and as the members are symmetrical the other half will have the same bending moment.

By Carnegie's table we see that a 5" Pin will stand 184000

So 5" does for Bending.

Crushing

The greatest stress on any member is 91 000 and its bearing areas is $= 5 \times 1\frac{1}{8} = 5\frac{5}{8}$ on a 5" pin.

Allowing 15 000# per sq. in.

$15\,000^{\#} \times 5\frac{5}{8} = 8$...

So as 5" is too small it takes a $5\frac{3}{8}$".

Shear.

Greatest shear = 81,000
allowing 10000# per sq. in.

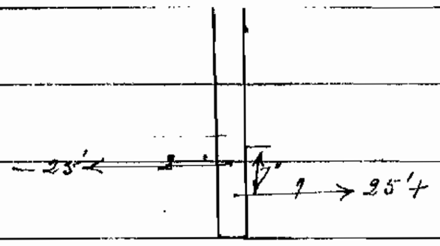
It requires 8.1 sq. ins

Area of $5\frac{3}{8}$" pin = 22.69 sq. in.
So it is ok for shear

So I will use a $5\frac{3}{8}$" steel Pin at F and also at C. H. J.

Pin at C

Here there are no Horizon. components that effects the pin.
So I took the vertical components
∴ the condition is as shown



Bending.

Members	Shear	Moments.
1	+ 25	0
2	0	2500.

So $1\frac{1}{4}$" would do for bending.

Shear

Shear = 25000

Area = $\frac{25000}{10000}$ = 2.5 sq. in.

So a 2" would do.

Crushing.

Bearing area = $\frac{25000}{15000}$ = $1\frac{2}{3}$ sq. in.

Bearing is here on a 2" pin is 2" x $\frac{3}{4}$" = $1\frac{1}{2}$. So a 2" is too small.
I used a 3" Pin.

The pins of A and B was analyzed the same way by taking the Vertical Components.

Floor Stringers

[illegible] = wt of iron

= [illegible]

wt of truck ties etc = 450

total D.L. per foot = 715

Bending Moment for dead load.

$$M = \frac{775 \times (30)^2}{2 \times 8} = 43600 \text{ ft. lbs.}$$

Inch pounds = $43600 \times 12 = 523200$

Bending moments for live load

[illegible]000 ft lbs

Inch pounds = 3,852,000

Econ Height = $1.41 \sqrt{\frac{m}{ft}}$

$$m = \frac{\begin{array}{r}523200 \\ 3852000\end{array}}{4375200}$$

$$h = 1.41 \sqrt{\frac{4375200}{8000 \times \frac{3}{8}}}$$

where f = 8000

t = $\frac{3}{8}$

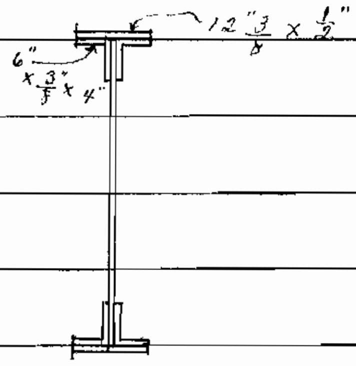
Take h = 48".

Assume that distance between the center of gravity of flanges is 45"

Total bending moment in inch lbs =
4375200

Then Area of flanges = $\frac{4375200}{45 \times 8000} = 12$ sq in.

Allow 8000 lbs. per sq in.



Area of flange made up of

2 - 6" x 4" x $\frac{3}{8}$" angles
1 - $\frac{1}{2}$" x 12 $\frac{3}{8}$" Plate.

Shear

Live load shear is a max at the end with wheel 1 on the end and is

$$= \frac{1460.1 - 34.8 \times 8 + 78 \times 3.3}{30} = 48000$$

R from dead load = 5820

Total Shear = 48000
$\underline{5820}$
53820

Allow 5000# per sq. in.

Area of web = $\frac{53820}{5000}$

But the web must not be less than $\frac{3}{8}$" therefore the web has an excess

$48" \times \frac{3}{8}" = 18$ sq in

An excess of 6 sq. in.

Rivets required in flanges

End Shear = 53820

Spacing $\frac{7}{8}$" in the ends = $P = \frac{rh}{S}$

P = pitch of rivets r = resistance of rivets
h = center to center of rivets in flanges
S = Shear

$\phi = \frac{4500 \times 45}{53800} = \frac{202500}{53800} = 3.8"$

Space them 3" out to 4 feet, then 4" for 4 feet then 6" apart in the center.

Shear in the center = 12700#

Shear at quarter point = 30700#

Floor Beam

Max. Moment is gotten by wheel 4

$$M_{max} = \frac{3349.3 + 113 \times 1}{2} - 608.6 = 1122.5$$

Mul. by the factor $\frac{2}{d}$ here $= \frac{2}{30}$.

$$1122.5 \times \frac{2}{30} = 74.8 = L.L.$$

Hence two such loads applied at the centre of stringers gives Max stress.

The depth $48'' + 4'' \times 2 + \frac{1}{2} \times 2 = 57''$

Assume D.W. of beam = 2500
the width of bridge = 16' to center to center of truss and the stringers are 7' apart.

Taking moments about the center.

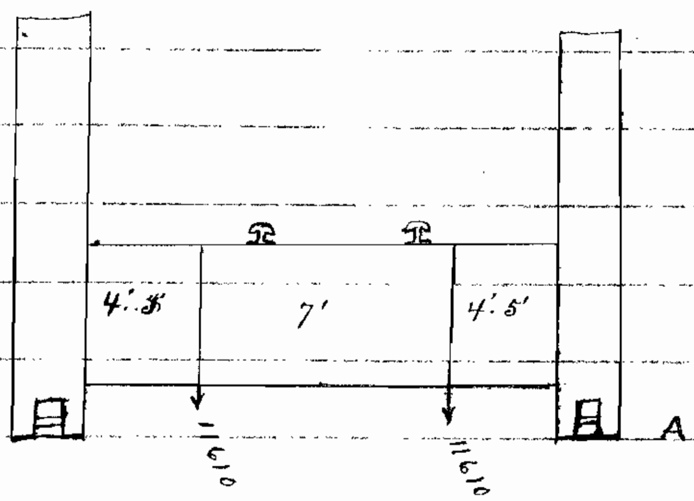
$$M = \frac{Pl^2}{8} = \frac{\frac{2500}{16} \times (16)^2}{8} = \frac{2500 \times 16}{8} = 5000$$

The next thing to take into account is the dead load of stringers and track system.

Let this = 775 lbs per foot as we found in the design of the stringer

$\therefore 30 \times \frac{775}{2} = 11610$ lbs.

Take moments about a



$11610 \times 4'.5' + 11610 \times 11'.5' = Rl = R16$

Moments at centre for stringer dead load

$M = R8' = 52245 = 11610 \times 4.5$

Moments L.L.

$M = 74800 \times 4\frac{1}{2} = 336600$

Total moments

336600 L.L.
5000 D.L. Beam
52245
393845

Shear =

1200 = weight of ½ beams.
11600 = " " Stringers.
74000 = Live Load
86800 = Total Shear

Area of Flanges

$$\frac{393845}{4.5} = 87521$$

4'.5" = distance from c.g of flange

$$\frac{87521}{8000} = 11 \text{ sq in (about)}$$

Web $= \frac{3}{8}" \times 57"$

Section