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1895

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

John Edward Kirkham

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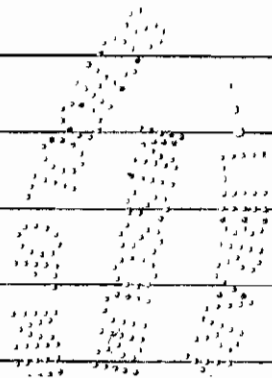
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17 A 210 foot R.R. Bridge

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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

$$W = 5L + 750$$

weight of floor = 400<sup>#</sup> 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  $\perp$  distance to the inclined chords.

The stresses in the lower chords was found by taking moment at the upper chord joints and dividing by the  $\perp$  distance.

were  
The stresses in the web members  
was 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  
panel draw ac || to AB and cb  
|| to AD.

Let  $T$  = stress in AD  
then we get

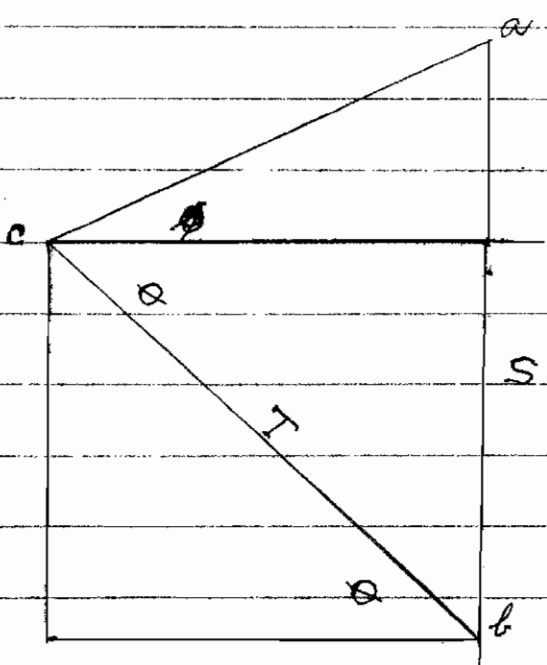


Fig 1

$$T \sin \theta + T \cos \theta \tan \phi = S = \text{shear}$$

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

The stress in BD = the shear

Kind Stress

is shown in Plate I by the diagrams.

Part Second

Plate 1

Design of Upper Chord

Ex 9

Stress = + 36400 #

L = 30'

Used formula

$$H = 5750 - 84 \frac{L}{d}$$

$$P = 8400 - 87 \times \frac{360}{15}$$

P = 6720 = wrought iron

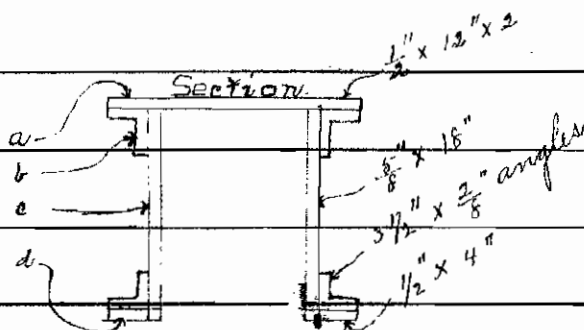
P = 6720 + 20% = steel

P = 8000 (about)

Area required

$$A = \frac{S}{P} = \frac{36400}{8000} = 4.55 = 4.6 \text{ sq in.}$$

Section



2a = 1/2 x 2.4 = 12 sq in.

2c = 5/8 x 3 1/2 = 2.1 sq in. = 4

2d = 2.2 + 2.2

2d = 4 / 47 sq in.

B 6

Stress = + 365000<sup>#</sup>

L = 30' (about)

$$P = 8700 - 84 \times \frac{360}{18}$$

$$P = 8500$$

Area required

$$A = \frac{S}{P} = \frac{365000}{8500} = 46.5 \text{ sq in.}$$

Section

Same as in 69.

B 7

Stress = + 327000<sup>#</sup>

L = 31' (about)

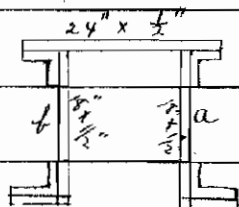
$$P = 8750 - 84 \times \frac{372}{18}$$

$$P = 8500$$

Area required

$$A = \frac{S}{P} = \frac{327000}{8500} = 40 \text{ sq in.}$$

Section



Same as 69 and B 6 except a 3/4\"/&gt;

A.d.

Stress = + 30,000 #

L = 42' (about)

$P = 8750 - 54 \times \frac{507}{18}$

P = 7250

Area Required

$A = \frac{S}{P} = 41.5 \text{ sq in. (about)}$

Section

Same as C.D. and B.L.

N.B.

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

2°

Design of Posts

C.F.

Stress = + 30,000 #

L = 40'

$P = 52,000 - 220 \times \frac{P}{R}$

Take channel no 227. R = 4.49

$P = 52,500 - 220 \times \frac{4.50}{4.5}$

P = 5800 { Safety factor 5 }



Area required

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

$\therefore$  channel no 357 is OK as two of these have an area of 11.5 sq in.

Q Q

$$\text{Stress} = + 90000 \#$$

$$L = 37.5'$$

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

Used channel 357, 30'7" per ft.

$$P = 52500 - 220 \times \frac{430}{4}$$

$$P = 5072 = 5000 \quad \text{Safety factor } 5.7$$

Area required

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

$\therefore$  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.

3°

## Design of Tension Members

{11,000# per sq in. allowed}

F.C.

$$\text{Stress} = + 365,000$$

Area required

$$A = \frac{S}{P} = \frac{365,000}{11,000} = 33.2 \text{ sq in.}$$

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

L.C.

$$\text{Stress} = 61,000$$

Area required

$$A = \frac{S}{P} = \frac{61,000}{10,000} = 6.1 \text{ sq in.}$$

Use 2 eye bars 4" x 3/4" = 3.84 sq in = 6.

B.F.

$$\text{Stress} = 17,500 \#$$

Area required

$$A = \frac{S}{P} = \frac{17,500}{1,250} = 14 \text{ sq in.}$$

Use 4 eye bars 5" x 3/4"

EF

$$\text{Stress} = 32,500 \text{ psi}$$

Area required

$$A = \frac{32,500 \times 2}{11,500} = 5.65$$

Use 2 eye bars  $9" \times 1\frac{1}{2}"$ CD

$$\text{Stress} = 11,500 \text{ psi}$$

Area required

$$A = \frac{11,500}{1,150} = 10 \text{ sq. in.}$$

Use 4 eye bars  $9" \times 1\frac{1}{2}" = 20 \text{ sq. in.}$ 26 and 26 O

$$\text{Stress} = 25,000$$

Area required

$$A = \frac{25,000}{1,150} = 22 \text{ sq. in.}$$

Use 2 eye bars  $9" \times 1\frac{1}{2}" = 27 \text{ sq. in.}$ 26 A

$$\text{Stress} = 5 = 9,1200$$

Area Required

$$A = \frac{75000}{10000} = 7.5 \text{ sq. in.}$$

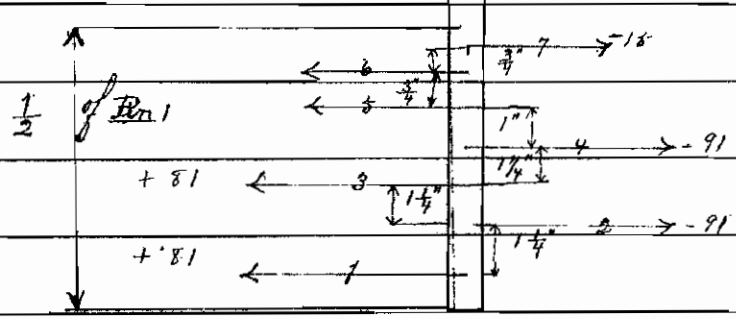
Use 2 eye bars 3' x 1" = 10 sq. in.

4°

Design of Pin at E

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	- 91	+ 91
3	+ 91	+ 88.5
4	- 91	+ 177.5
5	0	+ 157.5
6	0	+ 152.5
7	+ 91	+ 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 { 177.57 } 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 21000 and its bearing area is  $= 2 \times 1\frac{1}{2} = 5\frac{1}{2}$  or a 5" pin.

Allowing 15000# per sq. in.

$$15000\# \times 5\frac{1}{2} = 82500$$

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

Shear

Greatest shear = 5,000  
 allowing 10000<sup>lb</sup> per sq. in.

It requires 5.1 sq. ins

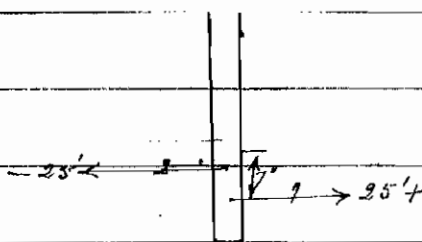
Area of  $5\frac{3}{8}$ " pin = 22.69 sq. in.  
 so it is ok for shear.

So I will use a  $5\frac{3}{8}$ " steel pin at  
 F and also at O. H. d.

Pin at C

Here there are no horizon. components  
 that effects the pin.

So I took the vertical components  
 $\therefore$  the condition is as shown

Bending

Members	Shear	Moments
1	+ 25	0
2	0	2500.

So  $1\frac{1}{2}$ " would do for bending.

Shear

$$\text{Shear} = 25,000$$

$$\text{Area} = \frac{25,000}{15,000} = 1.67 \text{ sq in.}$$

So a 2" would do.

Crushing

$$\text{Bearing area} = \frac{25,000}{15,000} = 1\frac{2}{3} \text{ sq in.}$$

Bearing is here on a 2" pin  
is  $2" \times \frac{3}{4}" = 1\frac{1}{2}$ . 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.

5°

Page 11

## Floor Stringers

$$2 \text{ lb} = 55 = \text{wt of iron}$$

$$= 55$$

$$\text{wt of truck tires etc} = 450$$

$$\text{total D.L per foot} = 715$$

Bending Moment for dead load.

$$M = \frac{715 \times (30)^2}{2 \times 8} = 43650 \text{ ft. lbs.}$$

$$\text{inch pounds} = 43650 \times 12 = 523200$$

Bending moments for live load

$$= 385200 \text{ ft. lbs.}$$

$$\text{inch pounds} = 385200$$

$$\text{Cover height} = 1.41 \sqrt{\frac{M}{ft}}$$

$$h = \frac{523200}{4375200}$$

$$h = 1.41 \sqrt{\frac{4375200}{8500 \times \frac{3}{8}}}$$

$$\text{where } f = 5500$$

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

$$\text{Take } h = 48''.$$

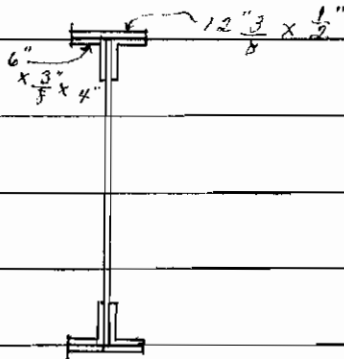


Assume that distance between the center of gravity of flanges is 45"

Total bending moment in inch lbs =  
4375200

Then Area of flanges =  $\frac{4375200}{45 \times 5000} = 12 \text{ sq in.}$

Allow 5000 lbs per sq in.



Area of flange made up of

2 - 2" x 4" x  $\frac{3}{8}$ " angles

1 -  $\frac{1}{2}$ " x  $12 \frac{3}{8}$ " Plate

Shear

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

$$= \frac{1460.1 - 34.5 \times 6 + 75 \times 8.3}{30} = 48000$$

R from dead load = 2820

$$\text{Total Shear} = 48000$$

$$\begin{array}{r} 5820 \\ \hline 23820 \end{array}$$

allow 5000# per sq. in.

$$\text{Area of web} = \frac{23820}{5000}$$

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

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

An excess of 6 sq. in.

Rivets required in flanges

$$\text{End Shear} = 23820$$

$$\text{Spacing } \frac{1}{5} \text{ in the ends} = P = \frac{rV}{S}$$

$P$  = pitch of rivets  $r$  = resistance of rivets

$n$  = center to center of rivets in flanges

$S$  = shear

$$P = \frac{45000 \times 40}{23820} = \frac{1800000}{23820} = 3.8"$$

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

$$\text{Shear in the center} = 12700\#$$

$$\text{Shear at quarter point} = 30700\#$$

# Floor Beams

Max. Moment is gotten by wheel 4

$$M_{max} = \frac{3549.3 + 113 \times 1}{2} - 608.6 = 1122.5$$

Mul. by the factor  $\frac{2}{a}$  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

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

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.

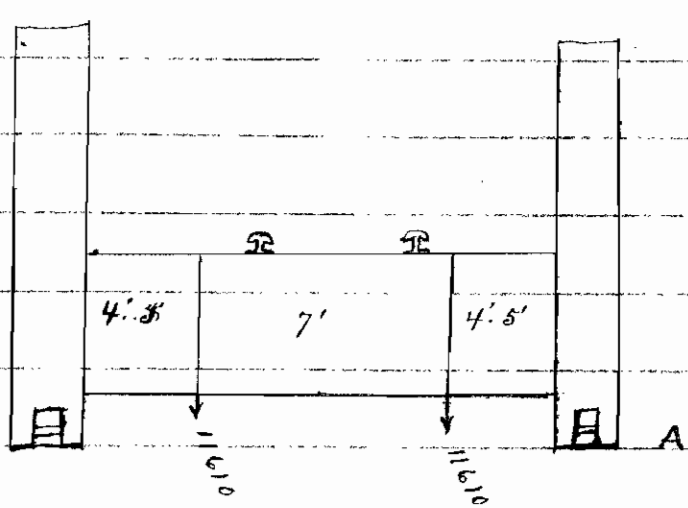
$$M = \frac{Pl^2}{8} = \frac{2500}{\frac{16}{8}} \times (16)^2 = \frac{2500 \times 16}{8} = 5000$$

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

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

Take moments about a



$$11610 \times 4.5' + 11610 \times 11.5' = R.L = R16$$

Moments at centre for stringer  
dead load

$$M = R8' = 5 \times 2245 = 11610 \times 4.5$$

Moments L.L

$$M = 74500 \times 4\frac{1}{2} = 336600$$

Total moments

$$\begin{array}{r} 336600 \text{ L.L.} \\ 5000 \text{ I.L. Beam} \\ \hline 52240 \\ \hline 393845 \end{array}$$

Shear =

$$\begin{array}{r} 1200 = \text{weight of } \frac{1}{2} \text{ beams} \\ 11600 = \text{" " " Stringers} \\ 74500 = \text{live load} \\ \hline 86800 = \text{Total Shear} \end{array}$$

# Area of Flanges

$$\frac{3938.45}{4.5} = 875.21$$

4.5" = distance from c.g. of flange

$$\frac{875.21}{8000} = 11 \text{ sq. in. about}$$

$$\text{Web} = \frac{3}{8} \times 57''$$

## Section

