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1895

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

John Edward Kirkham

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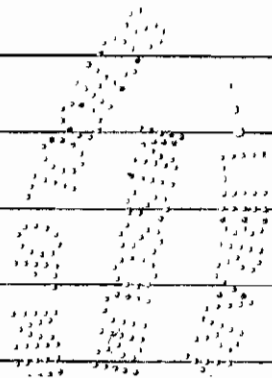
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17 A 210 foot R.R. Bridge

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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⁺ 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 ct
|| to AD.

Let T = stress in AD
then we get

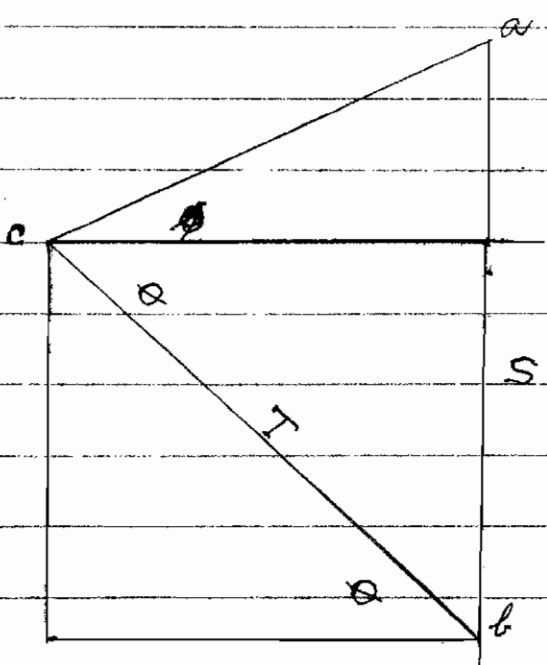


Fig 1

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

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

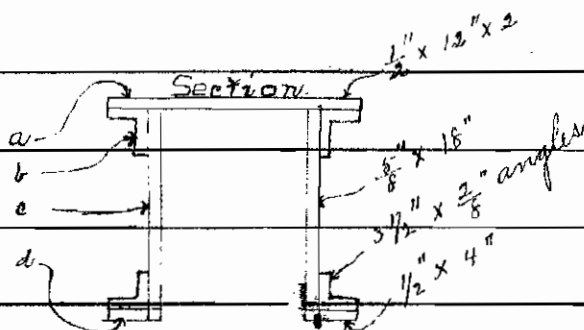
P = 6720 + 20% = steel

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 = 2.69 sq in. = 2.7

2d = 2.2 + 2.2

2d = 4.4 = 4.4 sq in.

47 sq in.

B 6

Stress = + 365000[#]

L = 30' (about)

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

$$P = 8500$$

Area required

$$A = \frac{S}{P} = \frac{365000}{8500} = 46 \text{ sq in.}$$

Section

Same as in 69.

B 7

Stress = + 327000[#]

L = 31' (about)

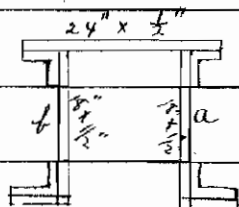
$$P = 8750 - 84 \times \frac{372}{18}$$

$$P = 8500$$

Area required

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

Section



Same as 69 and B 6 except a 3/4\"/>

A.d.

$$\text{Stress} = + 30,000 \text{ #}$$

$$L = 42' \text{ (about)}$$

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

$$P = 7250$$

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.

$$\text{Stress} = + 30,000 \text{ #}$$

$$L = 40'$$

$$P = 52,000 - 220 \times \frac{40}{18}$$

Take channel no 227. $R = 4.49$

$$P = 52,000 - 220 \times \frac{40}{4.5}$$

$$P = 5800 \text{ (Safety factor 5)}$$

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

$$A = \frac{S}{P} = \frac{90000}{5650} = 16 \text{ sq in.}$$

\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.

C.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 sq in = 6.

B at

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

Area required

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

Use 4 eye bars 5" x 3/4"

EF

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

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

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

Stress = $S = 9,000$

Area Required

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

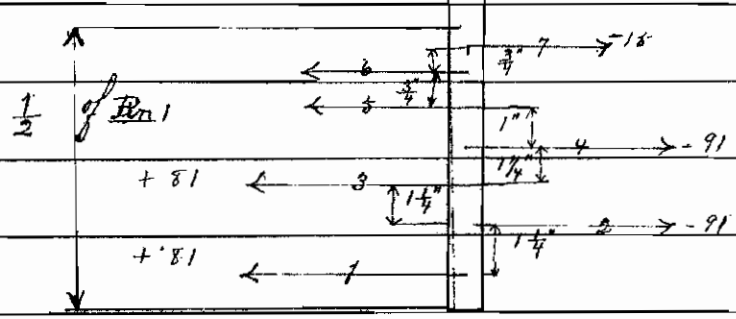
Use 2 eye bars 3' x 1" = 10 sq. in.

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	-10	+157.5
3	+71	+88.5
4	-30	+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 (177.57) 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 184,000

So 5" does for Bending.

Crushing

The greatest stress on any member is 21,000 and its bearing area is $= 2 \times 1\frac{1}{2} = 3\frac{1}{2}$ or a 5" pin.

Allowing 15,000# per sq. in.

$$15,000\# \times 3\frac{1}{2} = 51,750$$

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

Shear

Greatest shear = 5,000
 allowing 10000^{lb} per sq. in.

It requires 5.1 sq. ins

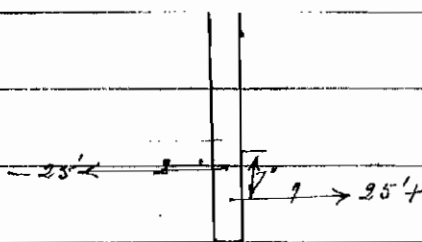
Area of $5\frac{3}{8}$ " pin = 22.69 sq. in.
 so it is ok for shear.

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 $1\frac{1}{4}$ " 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.

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

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

Bending moments for live load

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

$$\text{inch pounds} = 385200 \times 12 = 4622400$$

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

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

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

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

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

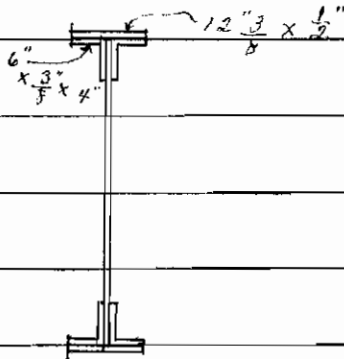
$$\text{Take } h_c = 48''$$

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

$$\begin{array}{r} \text{Total Shear} = 48000 \\ \quad \quad \quad 5820 \\ \hline 53820 \end{array}$$

allow 5000# per sq. in.

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

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

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

An excess of 6 sq. in.

Rivets required in flanges

$$\text{End shear} = 53820$$

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

P = pitch of rivets r = resistance of rivets

n = center to center of rivets in flanges

S = shear

$$P = \frac{45000 \times 40}{53800} = \frac{1800000}{53800} = 3.35"$$

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.

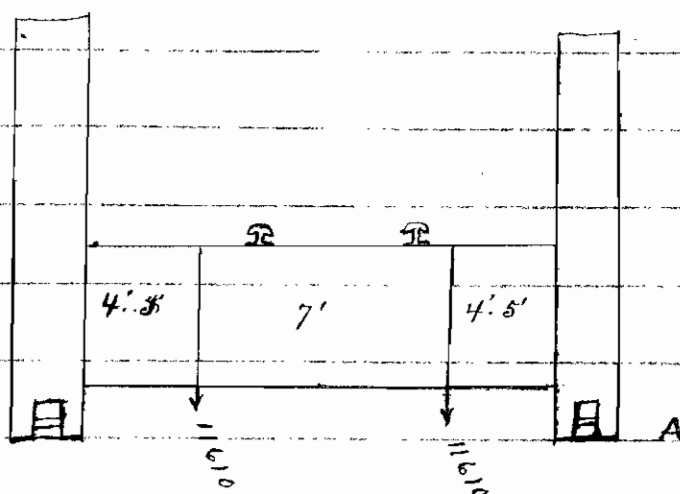
$$M = \frac{Pl^2}{8} = \frac{2500}{\frac{16}{8}} \times (16)^2 = \frac{2500 \times 16}{8} = 5000$$

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

$$M = R8' = 5 \times 245 = 11610 \times 4.5$$

Moments L.L.

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

Total moments

$$336600 \text{ L.L.}$$

$$5000 \text{ I.L. Beam}$$

$$\begin{array}{r} 52240 \\ 393845 = \end{array}$$

Shear =

$$1200 = \text{weight of } \frac{1}{2} \text{ beams}$$

$$11600 = \text{ " " " Stringers}$$

$$74500 = \text{live load}$$

$$\underline{86800} = \text{Total Shear}$$

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

