A Review of the Suspension Bridge at Reading, Penna.

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The bridge in question was erected by the Philadelphia and Reading Railroad Co; and stretches from the depot shedding on the West side, across the company's main line tracks to their property adjoining the freight depot, and thence a pavement furnishes access to Eighth Street.

Thus the bridge, as far as pedestrian travel is concerned, furnishes continuity to Buttonwood Street. Before the bridge was built, passengers to and from the North eastern section of the city were obliged to cross the tracks at Walnut St., two squares farther to the South; and thus the distance to the depot, for residents of Northeast Reading, was inconveniently long by nearly one-half a mile.

The bridge was designed by Roebling Sons & Co., and was erected in the year 1886.

The bridge is provided with a stiffening truss and with stiffening stays, and combines many of the features of larger suspension structures.

Specifications, Loading, Weight.

The specifications by which the designer was guided could not be obtained, so throughout this review, all members investigated for tension, compression, or shear, will have their average ultimate strengths introduced into the discussion, and factors of safety will be sought.

These average ultimate strengths will be taken from Merriman's "Mechanics of Materials" and will be found tabulated on the next page, as will a table of factors of safety usually agreed upon as good practice, and these will also be found in the above work.

Average Ultimate Strengths.

Material.	Tension. S_T lbs. per □"	Compression. S_C lbs. per □"	Shear. S_S lbs. per □"
Timber.	10,000	8000	Long. 600 Trans. 3000
Cast Iron	20,000	90 000	20 000
Wrought Iron	55,000	55 000	50000
Steel.	100,000	150 000	70000

Factors of Safety Used in American Practice:

Material.	Steady Stress [Buildings]	Varying Stress [Bridges]	Shocks. [machines]
Timber	8	10	15
Brick & Stone	15	25	30
Cast Iron	6	10	15
Wrought Iron	4	6	10
Steel	5	7	10

The weight of the bridge, at least the weight of the part between the towers, can be computed, and is made up of two twisted cables of three strands each, the suspenders, the trusses, and the floor system.

The following is a fairly accurate estimate of the weight, taking timber at 40 lbs. per cu. ft. and iron at 450 lbs. per cu. ft.

Weight of Cables	= 12 000 lbs.
Weight of Suspenders	= 2 200 lbs.
Weight of Trusses	= 19 000 lbs.
Floor System	= 16 000 lbs
Total	49 200 lbs

Then the dead weight per linear foot is:–

$$\text{Dead Weight} = \frac{49200}{275} = 180 \text{ lbs. per lineal foot of bridge}$$

$$\text{Then the dead panel load per truss} = \frac{180 \times 6}{2} = 540 \text{ lbs}$$

The Live Load will be taken as a densely packed crowd of people. Prof. Merriman gives the loading for a city highway bridge as 70 lbs. per square foot of floor surface. [See Roofs and Bridges, Part I, p. 45, Art. 29].

Since the footway is $7\frac{1}{2}$' wide, the live load per lineal foot will be:-

$$7.5 \times 70 = 525.0 \text{ lbs.}$$

Then the live panel load would be:- [per truss]

$$\frac{525 \times 6}{2} = 1575 \text{ lbs.}$$

Working with the above ultimate strengths and with the above loading, and with the measured cross-sections; the factors of safety found will be compared with those above tabulated.

Again, the [illegible] will be taken up in the following order:-

1st - The Cables, Suspenders, and Stays.

2nd - The Stiffening Truss.

3rd The Floor System.

4th - The Towers.

5th - The Anchorage.

The Cables, Suspenders, and Stays.

The cables consist of three strands each of smaller galvanized steel cables, twisted into one. Each of these smaller cables is $1\frac{5}{8}$" in diameter. The center deflection of the cable is 22.5 feet, and the distance between centers of towers is 275'. [See Plate I, at end of Volume].

The load is carried to the cable by means of wire cable suspenders each $\frac{3}{4}$" in diameter, but part of the load is carried by stays directly to the anchorage through the medium of a small cable extending from the tower to the anchorage. These stays, three at each end of the truss, extend from the 6th to the 12th panel point, therefore connecting at the 6th, 9th, & 12th panel points respectively. The stay connected with the 12th panel point, the outer stay, is $\frac{3}{4}$" in diameter, while the two inner ones are $\frac{5}{8}$" in diameter.

Now suppose the bridge to be covered with uniform live load. Each panel load would then be:-

$$1575 + 540 = 2015 \text{ lbs.}$$ (see pp's 5+6)

At the panel points where the stays are connected, part of this load goes to the suspenders and part to the stays.

Now we will suppose that the stays are arranged to take so much of the load that a suitable factor of safety remains.

A stay $\frac{3}{4}$" in diameter has an area of $\frac{4}{9}$ □" [very closely]; and taking the tensile strength at 100000 lbs. per sq. in. with a factor of safety of 7 we find:—

$$\text{Allowable Unit Stress} = \frac{100000}{7} = 14200 \text{ lbs. per □"}$$

$$\text{Allowable Stress in } \tfrac{3}{4}\text{" stay} = \tfrac{4}{9} \times 14200 = 6300 \text{ lbs.}$$ [Round No's]

A stay $\frac{5}{8}$" in diameter has an area of $\frac{137}{448}$ □", so under the same conditions as above we find:-

$$\text{Allowable Stress in } \tfrac{5}{8}\text{" stay} = \tfrac{137}{448} \times 14200 = 4410 \text{ lbs.}$$ [Round No's].

Since the stays are arranged to take this much stress before the suspenders takes any of the load, the load that would produce the above stresses, would, in each case be the stress multiplied by the cosine of the angle which the stay makes with the vertical.

The $\frac{3}{4}$" stay makes an angle with vertical whose tangent = $\frac{72}{22.5}$ or angle of 72°39'

The outer $\frac{5}{8}$" stay makes an angle with the vertical whose tan = $\frac{54}{22.5}$ or 67°23'

The inner $\frac{5}{8}$" stay makes an angle with vert. whose tan = $\frac{36}{22.5}$ or 58°

Then the load going to each stay would be:–

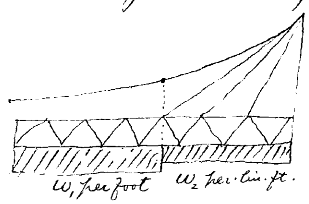
To $\frac{3}{4}$" stay	6300 X cos 72°39' = 6300 X .298	=	1877 lbs
To outer $\frac{5}{8}$" stay	4410 x cos 67°23' = 4410 x .385		1698 lbs
To inner $\frac{5}{8}$" stay	4410 X cos 58° 4410 X .530	=	2337 lbs
	Total from one side at one end	–	5912 lbs
	other	" =	5912 lbs
	Total Load going to stays at one end	=	11824 lbs.

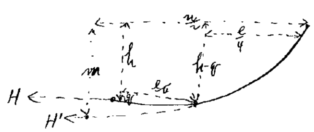
We have found that, when bridge is covered with live load, the uniform load per foot was $[\overset{dead}{180} + \overset{live}{525}]$ = 705 lbs. per foot; and we have just found that 11824 lbs. of the loading from the tower to the 12th panel point, goes to the stays. In other words, the stays take $\frac{11824}{72} = 165$ lbs. per linear foot. The load that goes to the cable, from the tower to 12^{th} panel point, is

$705 - 165 = 540$ lbs. per lineal foot $= w_2$ for convenience.

705 $\qquad\qquad$ w_1 for convenience.

To find the horizontal tension $\underline{H}$ in the lowest point of the cable, and also the greatest tension $\underline{R}$ at the tower, the following was given in a lecture to this class by Proff. Merriman.





Under the loading shown in the accompanying figure, the cable assumes the form of a double parabola, having a common ordinate [h-q], and a common tangent.

We have in the case of a single parabola, with a uniform load of w_1 $\qquad H = \frac{w_1}{2q}\left(\frac{l}{4}\right)^2$ [for the lowest of the above parabola's], [see lectures].

The problem now is to find $\underline{q}$, there being three unknown quantities $\underline{m}, \underline{n}, q$. We must have 3 known conditions between these quantities before they can be determined. These conditions are:-

$$H = H'$$

$$y = y' \quad \text{that is, they have a common ordinate}$$

$$\frac{dy}{dx} = \frac{dy'}{dx'} \quad \text{that is, they have a common tangent.}$$

From lectures we have:-

$$H' = \frac{w_2 n^2}{8m} \quad \text{and} \quad q = \frac{w_1 l^2}{[illegible]\,H}$$

$$\text{and} \quad m - [h-q] = \frac{w_2\left(\frac{n}{2} - \frac{l}{4}\right)^2}{2H}$$

1st Condition gives

$$\frac{dy}{dx} = \frac{w_1 x}{H} = \frac{wl}{2H}$$

$$\frac{dy'}{dx'} = \frac{w_2 x}{H} = \frac{w_2\left(\frac{n}{2} - \frac{l}{4}\right)}{H}$$

$$m = \frac{h(w_1 + w_2)^2}{w_2^2 + 3w_1 w_2}$$

<u>And</u> $q = \frac{w_1}{3w_1 + w_2} h$ <u>and</u> $H = \frac{3w_1 + w_2}{32h} l^2$

Then $R = H \sec\theta$ in which θ is the angle which the tangent to the curve at the tower makes with the ~~vertical~~ horizontal; and to find this angle we have:-

$$\frac{dy'}{dx'} = \frac{w_2 x}{2H}$$

But $x = \dfrac{(2w_1 + w_2)l}{2w_2}$

So $\dfrac{dy'}{dx'} = \dfrac{(2w_1 + w_2)l}{4H} = \tan\theta$

[The above discussion is applicable, for the distribution of w_2 to the quarter point is a feature of this bridge. See drawing, plate I.]

On this bridge w_1 for one cable $= \frac{705}{2} = 352$.

And $w_2 = \frac{540}{2} = 270$.

Now using $H = \frac{3w_1 + w_2}{32h} l^2$ we have:—

$$H = \frac{3 \times 352 + 270}{32 \times 22.5} \times 275 \times 275 = 139301.$$

Then $\dfrac{dy'}{dx'} = \dfrac{(2w_1 + w_2)l}{4H} = \dfrac{975 \times 275}{557204} = .489 = \tan\theta.$

Therefore $\theta = 26^\circ 05'$ and $\sec\theta = \frac{1}{.898} = 1.113$

Therefore $R = 139300 \times 1.113 = 154300$ "

Now the cross-section of the cable $= 3\left[\frac{1}{4}\pi d^2\right]$; d being $1\frac{5}{8}$" strand, so

$= 6.22\ \square''$

Now taking the tensile strength of steel at 100000 lbs. per □"

we get the factor of safety $\frac{622000}{154300} = 4.0+$. The factor of safety taken from the table, page 4, ~~goes~~ is 7. The discrepancy may be explained by the fact that a twisted wire, steel cable will have an ultimate strength far higher than the average ultimate strength of steel; and again the loading may not be the same as that used by the designer.

At all events it may be said that the cable is perfectly safe, and capable of holding any load that may come upon it.

The largest load, one panel load, that may come upon a single suspender is 2015 lbs. [see page 9].

The suspenders are $\frac{3}{4}$" in diameter, or have a cross section of $\frac{4}{9}$ □". Taking the tensile strength at 55000 lbs. per sq. in. we have the ultimate strength of a suspender $= \frac{4}{9} \times 55000 = 24444$ lbs. Now the suspenders are not truly vertical, and the greatest inclination to the vertical is $\tan^{-1}\frac{1.5}{6}$ = angle of about 15°

So the factor of safety $= \frac{24444}{2015 \sec 15^\circ} = \frac{24444}{2015 \times 1.035} = 11.7$, while an ample factor, taken from the table, would be 6.

Truss

The Truss is of the Howe type, the upper and lower chords being made up of jointed timbers, the diagonals being timber compression members, and the verticals, wrought Iron tension rods.

The upper and lower chords are made uniform in size throughout, as are also the diagonals and verticals.

The verticals are $\frac{7}{8}$" in diameter, the diagonals are 3"x4", the upper chord is 4"x8", and the lower chord 6"x8". The complete arrangement of the truss can be seen in Plate II.

When the bridge is loaded with its own weight alone, there is no reaction at the end of the truss, but part of the live load reacts at the tower, as soon as brought upon the bridge.

Burr's Bridges, Article 64, 3rd Edition, treats of the

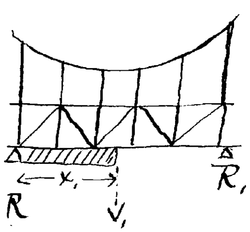
Strains in such a truss, in the following manner:-

Conditions:

A { 1st - Under Live Load, Cable is not deformed.
2nd - All Suspenders are Equally Strained.
3rd - All Live Load goes into the Suspenders.

Moreover we have the 3 ordinary conditions of equilibrium, viz:-

① The Sum of the Vertical Forces = 0
② " " " " Horizontal " = 0
③ " Moments = 0



From first condition of equilibrium we have:-

(a) $R + R_1 + t\,l - w_1 x_1 = 0$ [Letting t = tension in suspenders per foot of span]

And the 3rd condition of equilibrium gives,

(b) $R_1 l + t\,l\,\frac{l}{2} - w_1 x_1 \frac{x_1}{2} = 0$

Under live load, from 3rd of conditions "A" we have:-

$$t\,l = w_1 x_1.$$

Therefore from (a) we get:- $R + R_1 = 0$ or $R = -R_1$
and from (b) follows:-

$$\text{(c)}\quad R_1 = -\frac{w_1 x_1}{2}\left(1 + \frac{x_1}{l}\right)$$

$$\frac{dR_1}{dx_1} = \frac{w_1 x_1}{l} - \frac{w_1}{2}$$

Placing $\frac{dR_1}{dx_1} = 0$ we have $\frac{w_1 x_1}{l} - \frac{w_1}{2} = 0$ whence $x_1 = \frac{l}{2}$

Substituting in (c) this value of x_1 we have for

Maximum $R_1 = \frac{w_1 l}{8}$.

Now the shear at any point, $V = R + t_x - w_1 x$.

And $V_1 = -\frac{w_1 x_1}{2}\left(1 - \frac{x_1}{l}\right)$, and V_1 is a maximum for $x_1 = \frac{l}{2}$, so

(d) Max $V = \frac{w_1 l}{8}$ or $\frac{1}{4}$ Shear of Simple Truss.

From page 13, we see that $w = \frac{2}{3}\left[\frac{352+270}{2}\right] = 204$ lbs. per foot.

Substituting in (d) we have;- [$\frac{2}{3}$, since live load $= \frac{2}{3}$ of Dead + Live Load]

$$V = \frac{204 \times 275}{8} = \frac{2}{3}\left(\frac{85525}{8}\right) = 7130.$$

Any diagonal makes an angle with the vertical whose tangent $= \frac{8.75}{7.50} = 1.1666$ or $\sec^{-1} = 1.53$. We have therefore, the maximum stress in any web member -

$$S = 7130 \times 1.53 = 10908 \text{ lbs.}$$

Now the sectional area of a web piece being 4"×3" = 12 □" and taking the ultimate compressive strength of timber at 8000 lbs. per. sq. in., the strength of our piece is 12 × 8000 = 96000 lbs., and is therefore designed with a factor of safety $= \frac{96000}{10908} = 9$ nearly. By reference to our table of factors we find that good practice would give a factor of 8 to 10; but for timbers which are as carefully selected as bridge timbers are, the factor found above would be far enough on the safe side.

Continuing the discussion of the stiffening truss, remembering that $t l = w_1 x_1$; then the Bending Moment for any section at a distance x from R [Fig. page 16] is:-

$$M = Rx - (w_1 - t)\frac{x^2}{2}$$

But x, for Max M, $= \frac{x_1}{2}$ or -

$$M = \frac{w_1 x_1^2}{8}\left(1 - \frac{x_1}{l}\right) \text{ for a distance } x_1.$$

But
$$\frac{dM}{dx_1} = \frac{w_1 x_1}{4} - \frac{3w_1 x_1^2}{8l} = 0$$

$$2w_1 x_1 l - 3w_1 x_1^2 = 0$$
$$3w_1 x_1^2 = 2w_1 x_1 l$$
$$3x_1 = 2l$$
$$x_1 = \frac{2}{3}l$$

Therefore the Max. Mom. $= M = \frac{4w_1 l^2}{72}\left(1 - \frac{2}{3}\right) = \frac{1}{54} w_1 l^2$ #

Then Max M, for this truss, $= \frac{204 \times 275 \times 275}{54} = 285690$

Since $\frac{M}{d}$ gives the stress for a bending moment M, d being the depth of the truss, there follows —

$$S = \frac{285690}{6} = 47600 \text{ lbs.}$$

For the upper chord, compression, the section $4'' \times 8'' = 32''$ pertains, giving a total ultimate strength of ~~from~~ 32×8000 or 256000 lbs. With this ultimate strength, the factor of safety is $\frac{256000}{47600} = 5.5$ which is rather a low factor of safety for the loading assumed in this review.

The lower chord has a section of 6"×8" = 48□", or a total ultimate strength of 480000 lbs, and therefore the factor of safety is $\frac{480000}{47600} = 10$ which is ample; and this difference between upper and lower chord may be accounted for in the fact that the designer probably added something to the section of the lower chord for flexure brought on by the floor beam which is placed mid-way between panel points. [See drawing Plate II]

The maximum bending moment in a beam of span l loaded with one concentrated load P at the center of span is:- $M = \frac{1}{4} Pl$.

and $S = \frac{Mc}{I}$ #

The load $P = \frac{525 \times 3}{2} = \frac{1575}{2} = 787$ and $M = \frac{787 \times 6}{4} = 1180$

Then $S = \cancel{\frac{1180 \times 4}{256}} = \cancel{\frac{1180}{64}} = \cancel{18.4}$ $\frac{8000}{10} = 800$

Then $800 = \frac{(1180)\frac{1}{2}d}{\frac{bd^3}{12}}$. Assuming b as 1" we find the area necessa-

ry to support the load P, is about 3□"
So in the lower chord the area to be investigated for tensile strain is 48□" - 3□" = 45□", which has an ultimate strength of 18000 × 45 = 450000 lbs. The factor of safety would then be $\frac{450000}{47600} = 9.4$ #.

The Verticals are $\frac{7}{8}$" in diameter, and these are to withstand a vertical shear = $\frac{1}{4}$ the maximum shear of a simple truss or [page 17] $\frac{w.l}{8} = V = 7130$

The cross-section = $\frac{1}{4}\pi d^2 = \frac{3.1416 \times 49}{4 \times 64} = \frac{9}{16}$ □", and the total ultimate strength = $\frac{9}{16} \times 55000 = 30940$. Therefore the factor of safety is $\frac{30940}{7130} = 4.3$, which is a trifle smaller than the usual factor, 6.

The joints of timbers are those usually found in wooden bridge construction, and are made as firm as the whole timber, by the bridge carpenter, by the judicious use of bolts, fish-plates, and the like.

The floor planking is supported by floor beams placed 3' c to c and resting on lower chord [see plate II].
Then each beam supports 3' x 2" x 7'6" of planking, and when covered with live load, an addition $3' \times 7\frac{1}{2}' \times 70 = 1575$ lbs.
$3' \times 2'' \times 7'6'' = 3\frac{3}{4}$ cu. ft, which weighs 150 lbs. So the total load on a floor beam is 1725 lbs, which is equivalent to a uniform load along its length of $\frac{1725}{7.5} = 230$ lbs, per foot. The expression for the Max. Moment $= \frac{wl^2}{8}$. So.

$$\text{Max. M.} = \frac{230 \times 56.25}{8} = 1610$$

Again $S = \frac{Mc}{I}$ or $\frac{600}{10} = \frac{(1610)\frac{1}{2}d}{\frac{bd^3}{12}} = \frac{1610\, d \times 6}{bd^3} = \frac{9660}{bd^2}$.

[For S_s for timber, longitudinally, = 600 & factor = 10]

So $60 = \frac{9660}{bd^2}$; and if b be assumed at 3" we have

$$180 d^2 = 9660 \text{ or -}$$

$$d^2 = 53.66$$

$$d = 7.3''$$

By reference to Plate II we see that the floor beams are alternately 4"x8" and 3"x8".

The Towers, are made up entirely of Angles, Channels, Bars & Plates, and is of the form of a square, truncated pyramid. The legs of the tower are angles 6"×6"×$\frac{7}{16}$" and these are framed together and braced by channels and angles; and the two main columns comprising one tower, are braced by a curved strut. A skeleton drawing is shown in Plate III.

A rough estimate of the weight of one complete tower is:-

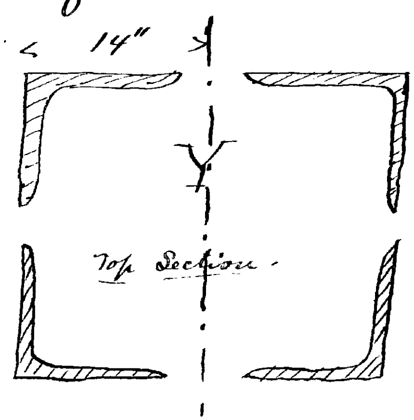
1352' of L's varying from 17 to 3.5 lbs. per foot	wt =	13520 lbs
360' of ['s from 4" to 8" from 28 to 5 " " "	=	5760 "
Weight of 2 Curved Struts	=	3120 "
Total Weight of one tower		22400 "
" " "	=	10 gross tons.

[An estimate made by Ass't Eng. P.R.R.R. made weight of one tower · 20000 lbs.]

The towers, or rather the columns to tower, are 5'3" square at the base and 2'4" square at the top, as can be seen in Plate III.

The towers will now be investigated, both as to strength as a column, and as to stability.

Rankine's Column Formula:- $\frac{P}{A} = \frac{S_c}{1 + q\frac{l^2}{r^2}}$ will be used, in which $q = \frac{1}{36000}$ and l is the length of the column in inches, r is the least radius of gyration in inches, S_c = ultimate unit compressive strength of iron = 55000 lbs. per sq. in., and A = area of section.



From Carnegie's Book of Shapes:-

$I' = 17.68$ [for one angle]

$I = I' + Ay^2$

$I = 88.48$ for one angle

$I = 353.92$ for 4 angles.

Now radius of gyration is:-

$$r = \sqrt{\frac{I}{A}}$$

$$r = \sqrt{\frac{353.92}{20.24}} = \sqrt{17.48}$$

$$r = 4.18$$

Now $\frac{P}{A} = \frac{55000}{1 + \frac{1}{36000} \times \frac{47.5 \times 47.5 \times 144}{17.48}}$

$$\frac{P}{A} = \frac{55000}{1.51} = 36400$$

$$P = 36400 \times 20.24$$

$$P = 736736$$

From page 13 we see that the greatest tension, R, in the cable, is 154000 lbs, and the vertical component only of this tension, goes to towers. Or:-

$$V = R \times \cos 26°05'$$

$$V = \frac{154000}{1.1134} = 138585$$

Now if we divide $\frac{P}{V}$ we get the "columnar" factor of safety of the tower, which is:-

$$z = \frac{P}{V} = \frac{736736}{138585}$$

$$z = 5.3$$

Carnegie, Phipps, & Co., give a factor of 5 for bridge columns, and

and the factor found in our table would be 6. We thus see that the towers are amply safe as far as crushing is concerned.

As to Stability, it may be said that the forces tending to overturn the tower are:–

1st – The normal pressure produced by moving loads, into the coeff. of friction.

2nd – Wind pressure

(1st.) The greatest tension in the cable was found to be [page 13] 154000 lbs. Since the dead load is ~~nearly~~ closely $\frac{1}{3}$ of the live load (actually a little greater than $\frac{1}{3}$), it will not be far wrong to say that $\frac{2}{3}$ of 154000 = 102666 lbs. tension is produced by live load, and the vertical component of this, which is $102666 \times \cos 26°05' = \frac{102666}{1.1134} = 92492$, is the pressure which produces the force of friction F.

Then $F = 92492 \times f$, in which f is the coeff. of friction

From Wood's Mechanics, we find f, for steel on wrought iron is .076. Then 92492 × .076 = F = 7029 lbs. or 14058 for 2 cables. Moreover, the worst direction for stability that F may take, is a horizontal direction, so we will assume that such is its direction.

The horizontal overturning force of the wind will be taken at 40 lbs. per square foot of vertical projection. So the total wind pressure on the tower is, the area of the vertical projection of one side, in feet, into 40. This is:-

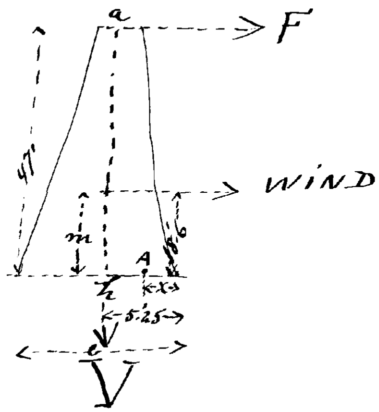
$$\frac{10.5 + 2.4}{2} \times 47 = 12160. \text{ or } 24320 \text{ for 1 complete tower.}$$

The forces tending to keep the tower from overturning are:-

① The vertical component of Maximum Tension - V'

② The Weight of the Tower - W.

V' has been found to be 138585 for one cable or 277170 lbs for 1 complete tower, and W was found = 22400 lbs. Then the resisting weight = V'+W = V = ~~160985 + 138585~~ = 299570 lbs.



Taking a center of moments at A any point in the base, (supposing the resultant of all forces cut the base at A), and call its distance X, from the "toe" of the tower we have the following equation of moments:-

$$V(5.25 - x) = F \times 47 + \text{WIND} \times 18.6.$$

[Note- The lever arm of wind = distance from base to center of gravity of trapezoid = 18.6, by using formula $m = \frac{b+2a}{b+a} \times \frac{1}{3}\,\underline{ah}$] -

Substituting our values of V, F, and Wind, we get:-

$$299570(5.25 - x) = 14058 \times 47 + 24320 \times 18.6$$

$$299570x = 1572742 - 1113078$$

$$299570x = 459664.$$

$$x = 1.5 \text{ feet}$$

Thus we see that the resultant falls within the base by 1.5', which, although preventing overturning, does not fulfill the condition of stability.

viz:- $\underline{X}$ should be at least equal to $\frac{1}{3}b$.

The Anchorage -

The Cables are carried ~~by~~ in a straight line from towers to anchorage, and by means of eye-bars the stress in the cable is communicated to the anchor-plates. There are two such bars 3" square, except at point where turnbuckles are put on, where they are rounded; so it is seen that if the cable is capable of holding the load, the eyebars, being of larger sectional area, must be capable of doing the same work.

A drawing of the anchorage is shown in plate II, with the arrangements of anchor-plates, masonry, and eye-bars.

To find out whether the anchorage is capable of withstanding the pull on the cable, we take, this pull, and resolve it into its horizontal and vertical components. The force resisting the vertical component is the weight of the anchor plates and of the superincumbent masonry. The force resisting the horizontal component is the total weight of the anchorage into the coefficient of friction.

The anchor-plates are $2' \times 2' \times 6\frac{1}{2}$ and $2' \times 1.5' \times 5'$ respectively; or of a volume of 26 cu. ft and 15 cu. ft. respectively. Taking the weight of cast iron at 450 lbs. per cu. ft., the weight of anchor-plates is:-

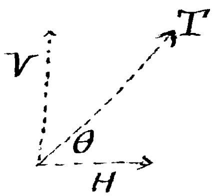
$$[26+15]450 = 18450$$

The Volume of Masonry is $[21 \times 6 \times 8\frac{1}{4}] + [11 \times 6\frac{1}{2} \times 6] = 1468.5$ cu. ft.

Taking the weight of a cu. ft. of masonry at 140 lbs. The weight of the masonry = 205590 lbs.

Then the total weight of the anchorage is:-

$$205590 + 18450 = 224040 \#$$



The vertical component of $T = V = T \sin\theta$

The horizontal component of $T = H = T \cos\theta$.

By measurement θ has been found $= \tan^{-1} \frac{11}{15.75} = .698$

or $\theta = 35°$.

Then $\sin\theta = .573$ and $\cos\theta = .819$

Then $V = \underbrace{154000 \times .573}_{\text{main Cable}} + \underbrace{11824 \times .573}_{\text{Stay Cable}} = 95017$

So the factor of safety would be $\frac{224040}{95017} = 2.3$

Again $H = [154000 + 11824] .819 = 135709$

We can assume a coeff. of friction of almost unity, on account of the bond of the masonry and on account of the external resistance of surrounding earth so that we would get a factor of safety of at least $1\frac{1}{2}$.

From the above we can conclude that the masonry and bed-plates in question form a safe and efficient anchorage.

As a brief summing up it may be said, under the loading assumed

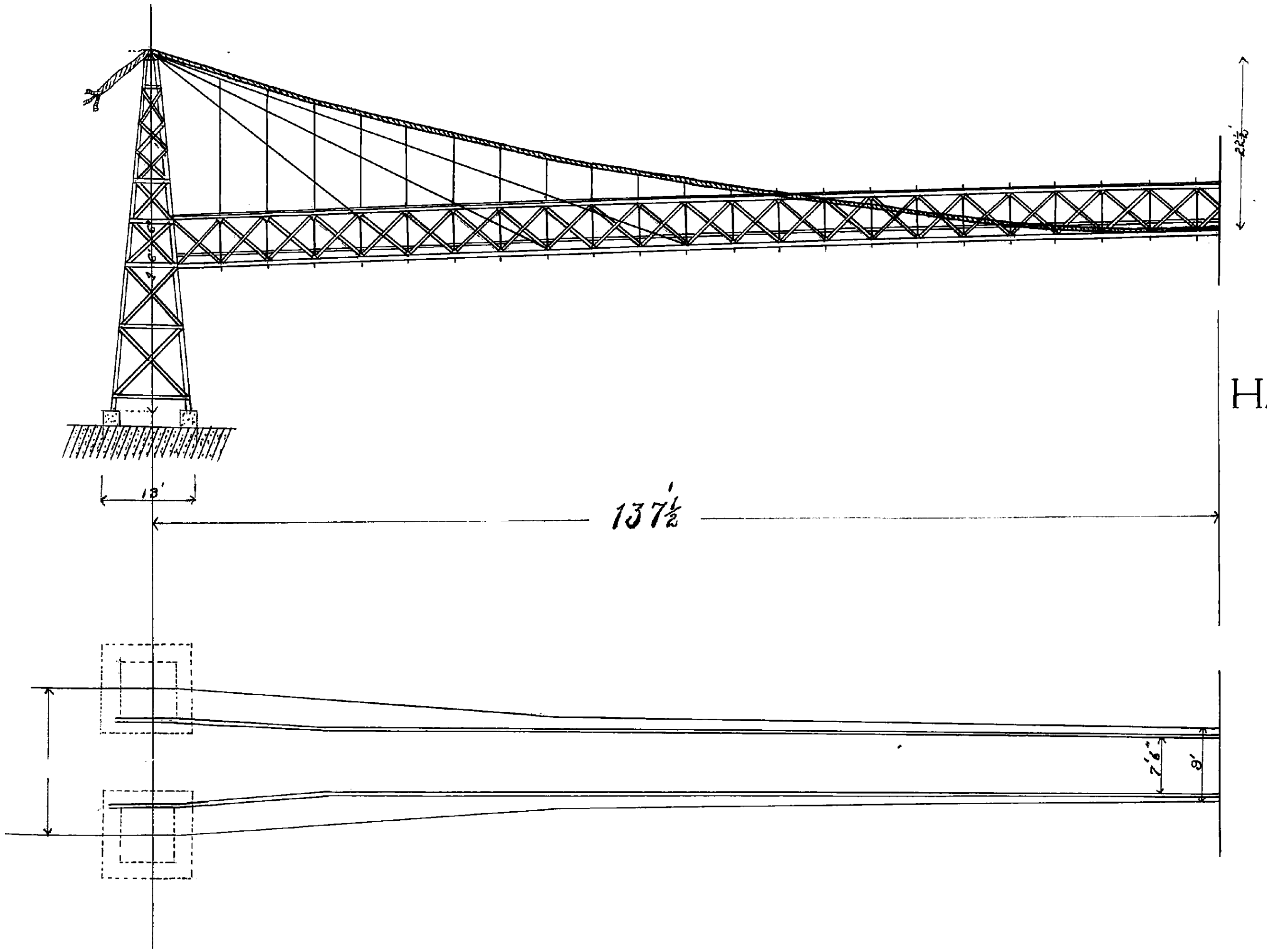
1st That the cables, suspenders, and stays, are of ample strength to do the work assigned them.

2nd The truss, though not dangerously weak, is not of such a strength as good practice would give it; that is the factors of safety found are not quite as large as those allowed in good practice. Especially is this the case with the upper chord.

3rd The Towers are capable of resisting any crushing strain liable to come upon them; and although there is no danger of overturning, yet the condition of perfect stability is not fulfilled.

4th. The Anchorage is designed with an ample factor of safety, but would pull up before the cable would break or before the towers would crush.

PLATE I.

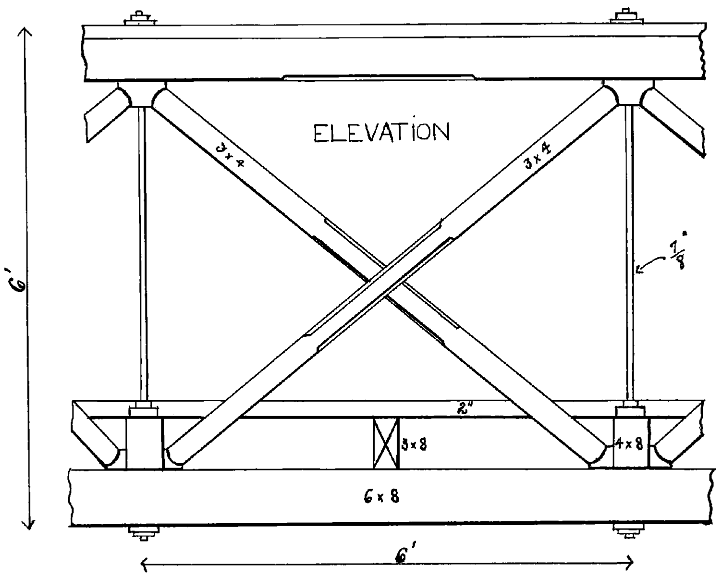


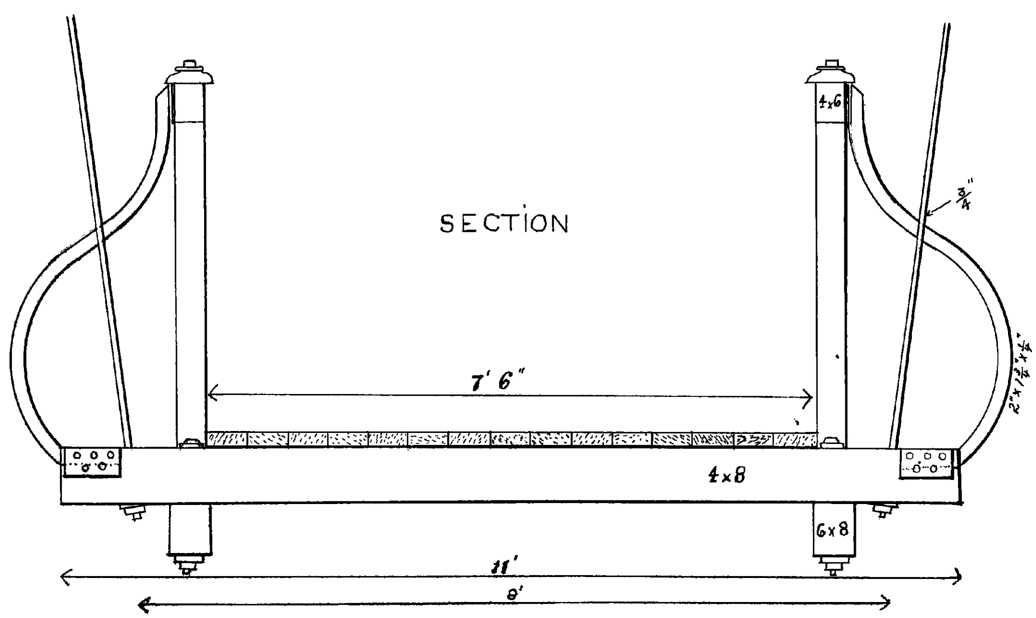
HALF ELEVATION

AND

HALF PLAN.

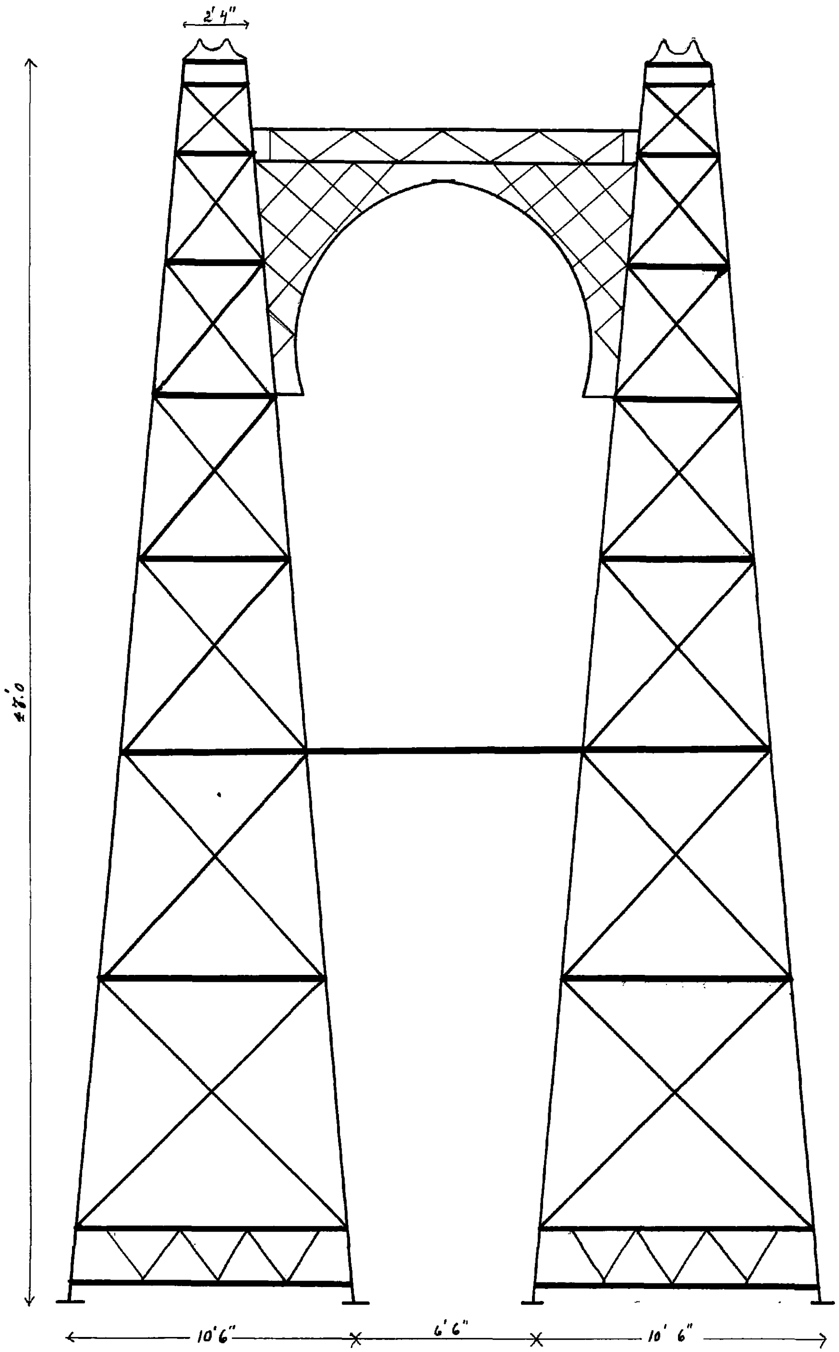
A PANEL OF THE TRUSS.





—NOTE—

LATERAL BRACE EVERY 12'



THE TOWERS.

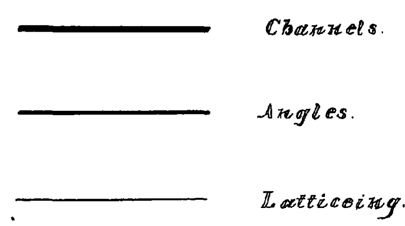
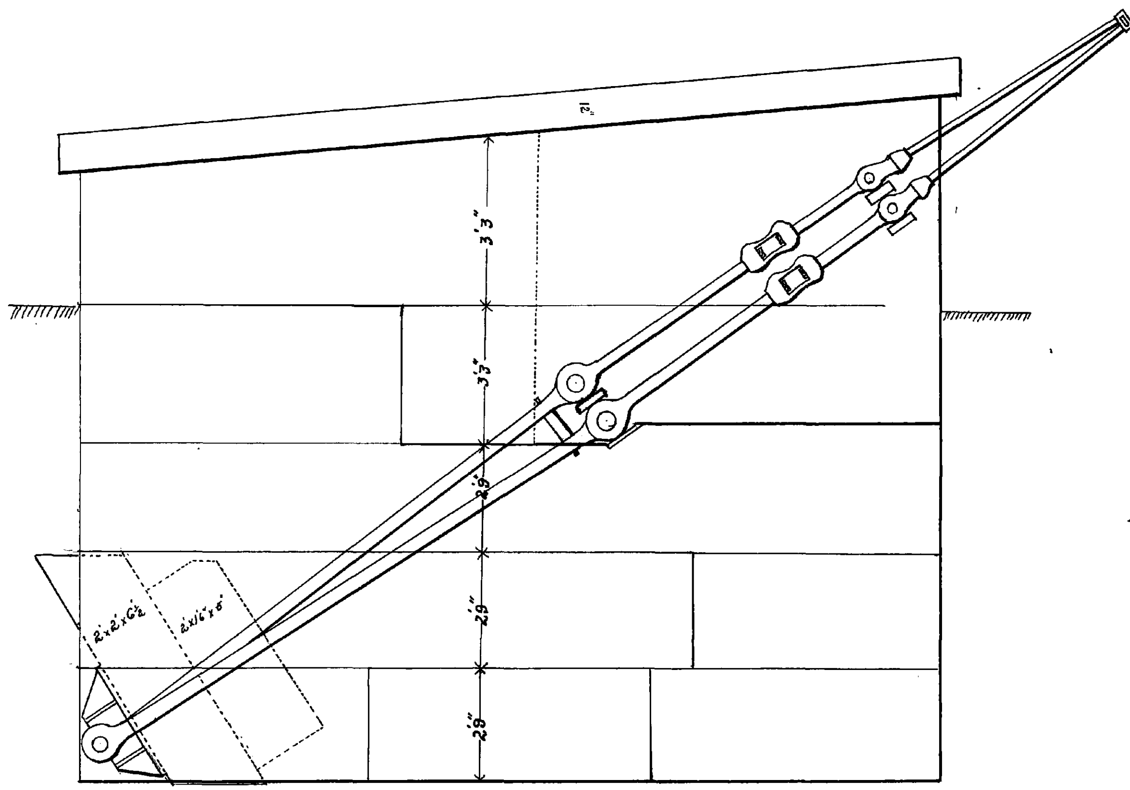


PLATE IV



THE ANCHORAGE.