1899

Design of a three hinged arch steel bridge

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Thesis

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of

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Subject

Design of a Three Hinged Arch Steel Bridge

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Design

of a

Three Hinged Spandrel Braced Steel Arch Bridge.

Length 280 feet C to C of end pins, 14 panels of 20 feet each, Depth of parabolic lower chord 49 feet C to C of pins. Width 60 feet over all, sidewalk on each side 12 feet, wagon road 36 feet. Three arches spaced 20 feet C to C.

Upper chord horizontal 10 feet above C of middle pin. Floor system calculated for a Live Load of 100 pounds per square foot. Trusses calculated for a Live Load of 80 pounds per square foot.

The roadway to consist of 1/4 inch buckle plates resting on I beams spaced 3 feet C to C and concrete arches 3 inches at crown and 5 inches at abutment, covered with 1 inch layer of asphalt.

Asphalt at 80 pounds per cubic foot $= 1/12 \times 80 = \frac{20}{3}$ pounds per square foot.

Concrete one part Portland Cement, two parts sharp sand, three parts coarse gravel at 130 pounds per cubic foot $= 1/3 \times 130 = \frac{130}{3}$ lbs. per square foot.

Buckle plates at 10 pounds per square foot for 1/4 inches thick. Rivets spaced at 16 times thickness of plates = 4 inches, diameter $\frac{5}{4}t + \frac{3}{16} = \frac{5}{4} \times .48 + \frac{3}{16} = \frac{3}{4}$ inches requiring 120 rivets weighing 35 pounds per 20 feet I beam. Weight of beam assumed at 25.5 pounds per foot.

Total dead load per beam $= 20\left[3\left(\frac{20}{3} + \frac{130}{3} + 10\right) + 25.5\right] + 35 = 4145$ pounds

" live " " " $= 20 \times 3 \times 100 \qquad = 6000$ pounds

$W = 10145$ "

The moment of resistance of the beam $R = \frac{3\ Wl}{2T}$ assuming $T = 12500$ for steel, we have $R = \frac{3 \times 10145 \times 20}{25000} = 24.34$

Carnegie page 93 has a 10 inch I beam weighing 25.5 per foot with $R = 24.7$ of a little stronger than needed, and will answer the purpose.

At the coping between sidewalk and wagon road a plate girder 18 inches deep, Web 1/4 inch is used. Assuming its weight at 665 pounds.

$$M_{max} = \frac{10300 \times 240}{8} = 309000 \text{ inch lbs. } A = \frac{M}{Rd} - \frac{td}{6}$$

$$A = \frac{309000}{10000} - \frac{1 \times 18}{4 \times 6} = .97 \text{ square inches.}$$

Taking angle iron 3 x 3 x 5/16 = 1.58 square inches, weighing 5 pounds per foot. Size of rivets = 5/4t + 3/16 = 5/4, 5/16 + 3/16 = 5/8 inches, rivet holes 2 x 5/16 x 3/4 = .48 or 1.1 square inch of angle left which is ample

Another angle of same size is rivited at roadside 8 7/8 inches above bottom to the plate, but not taken into calculation as at neutral line.

$$\text{Shear} = \frac{10300 \times 4}{2 \times 18} = 1144 \text{ pounds per square inch at ends.}$$

$$\text{Resistance to buckling } \frac{10000}{1 \frac{d^2}{3000t^2}} = \frac{10000}{1 \frac{324 \times 16}{3000}} = 3665 \text{ per square inch.}$$

which is more than the shear no stiffeners are needed.

Weight of girder. Web plate 20/3 x 18 x 1/4 x 10 = 300 pounds

3 angle irons at 5 lbs. per foot 3 x 5 x 20 = 300 "

180 rivets spaced 16 x 1/4 = 4 inch, size 5/8 in. = 39

639 "

To give the driveway a parabolic crossection, iron plates are placed between cross girder and I beam, there are two each of 1 1/8" 2" 2 5/8" 3" and 1 of 3 1/8" weighing $\frac{8\ 1/2 \times 4 \times 18\ 1/2 \times 10}{36}$ = 175 pounds.

Cross Girder.

The total outside load on 20 ft. of the cross girder W = 20/3 x 10145 + 216 = 67849. Least weight depth $= \frac{10\ 1^2}{R} + \sqrt{\frac{6\ Wl}{R} + \left(\frac{10\ 1^2}{R}\right)^2} =$

$$\frac{10 \times 400}{10000} + \sqrt{\frac{6 \times 67850 \times 20}{10000} + \left(\frac{10 \times 400}{10000}\right)^2}$$

d = 28.94 inch, best economic depth = 8/10 x 28.94 = 24 inches.

$$\text{Weight} = \frac{12\ Wl^2 + 2\ Rld^2}{1.2\ Rd - 12l^2} = \frac{12 \times 67850 \times 400 + 2 \times 10000 \times 20 \times 576}{1.2 \times 10000 \times 24 - 12 \times 400} = 1964 \text{ lbs}$$

Total load = 67849 + 1964 = 69813 we assume it at 70000

$$\textbf{M max.} = \frac{70000 \times 240}{8} = 2100000 \text{ inch lbs. Web assumed 3/8 inch thickness.}$$

Area of flange $= \frac{M_{max}}{Rd} - \frac{td}{6} = \frac{2100000}{10000 \times 24} - \frac{3 \times 24}{8 \times 6} =$ **7.25 square inch. Carnegie page 105 we find angles 5 x 4 x 1/2 weighing 13.5 lbs. per ft. area 4.04 square inch. Size of rivet 5/4 x 1/2+3/16 = 13/16 in. diam. Area of rivet holes = 7/8 x 1/2 x 2 = .875 sq. in.** leaving a tensile area of 2 x 4.04 - .875 = 7.205 which is sufficient.

The shear $- \frac{70000}{2} = 35000$ pounds at the end. Area of web = 24 x 3/8 = 9 square inch. but $\frac{350000}{9} = 3888$ pounds per square inch.

Resistance to buckling $= \frac{10000}{1 + \frac{576 \times 64}{3000 \times 9}} = 4228$ which is more per square inch than shear, no stiffeners are needed.

The end of the crossgirder outside the arches acts as a beam fastened at one end, and has about the same uniform load and same max. moment, but as the moment is inversely proportional to the distance from the arch the web is tapered to a width of 18 inches, or that of the angles at the free end.

The webs are spliced at center of girders by double plates 1/4 in. thick, 14 x 8 and 8 rivets 7/8 inch. weighing 12 pounds.

The rivets in the angles are spaced 3 inch. diam. 4/5 x 1/2 + 3/16 = 7/8 inch. we need 4 x 60 = 240 weighing 105 pounds.

Web plate weighs $[(40 \times 2 \times 3/2) + (10 \times 38/12 \times 3/2)]\ 10$	= 1675 lbs.
Angle irons (4 x 60) 13.5	= 3240 "
Rivets, splice plates	= 117
	= 5032 "

The cornice girder outside has only half the, load of that at the coping, but in case the floor buckle plates are not sufficiently rigid, it has to resist some horizontal thrust, web and angles are therefore taken the same strength as the coping girder.

The web extends from lower edge of crossgirder to top of concrete and has a width of 10 + 18 + 5 + 3 = 36 inches.

Weight of web = 3 x 20 x 10	=	600 pounds.
Angles 3 x 2 x 5/16 at 5 lbs. per ft. 5 x 60 = 300		"
180 rivets 5/8 inches	=	39
Total weight of cornice girder		936 pounds.

The pattern of the railing is taken from Johnson plate XX No.8 made of cast iron in one panel of 20 feet. At the end they are screwed to posts consisting of 6 inch.I beams weighing 13 lbs. per foot,which are riveted by angles to the cross girders and cornice girders,and finished by covering with an ornamental bronze shell.

The railing is 4 feet high,the lower edge is fastened to the upper angle of the cornice girder every 2 feet,and the upper one to a nickel plated steel tube 2 inches dimeter and 1/2 inch wall to give security against breaking by shock.

Weight of cast iron panel estimated at	800 pounds.
Bronze shell and connections	48 "
Tube 3 x 2 x 20 x 10/12	100 "
I beam 4 x 13	52 "
Entire weight of 20 feet of railing	1000 "

Cast steel footings between cross girder and I beams for side-walk to be 8 3/4, 9 1/2, and 10 1/4 inch high,weighing 50 pounds each, 300 pounds per panel. 2 Splice plate for I beams every 40 feet 8 x 8 x 1/4 weighing 88 pounds per panel,40 rivets weighing 12 pounds per panel. 80 rivets 1 inch for fastening crossgirder to I beams weighing 52 pounds per panel. Rain or snow water is drained off through gutters alongside the coping which have cast iron inlets at the middle of every panel. Inlets with screen weighing 30 pounds each or 60 pounds per panel.

Total weight of Floor System per panel.

Asphalt 60 x 20 x 20/3	= 8000	pounds.
Concrete 60 x 20 x 130/3	= 52000	"
Buckle plates 60 x 20 x 10	= 12000	"
Buckle plate rivets 20 x 35	= 700	"
Seventeen 10 inch I beams 17 x 510	= 8670	"
2 Coping girders 2 x 639	= 1278	
2 cornice girders 2 x 936	= 1872	
Plates between cross girder and I beam	= 350	
Cross girder	5032	
Railing	2000	
Footing for I beams under side walk	300	
Spice plates for I beam	88	
Rivets for splicing	12	
Rivets for fastening I beams to cross girder	52	
Inlets for gutter	60	
Panel load for floor system	92414	pounds.
per arch	30805	pounds.

The weight of iron Highway bridges in iron per foot is given (page 503) = $\frac{7000+13l}{1100-l}w$ = [illegible] $\times$ 60 =3097 lbs. per foot.

Weight of concrete and Asphalt = 3000

As the bridge is to be of steel we assume total dead load per linear foot = 6000 pounds of the panel dead load for each arch rib =

$$\frac{20 \times 6000}{3} = 40000 \text{ pounds.}$$

$$\text{Panel live load} = \frac{20 \times 60 \times 80}{3} = 32000$$

The stresses in the trusses were found by the graphical method, but to make the diagrams accurate, all the verticals and upper horizontals were checked by the method of moments, thus

$+1/14\ 32000 \times 20 + 46 \times AC = 0$

$$AC = -\frac{32000 \times 20}{14 \times 46} = 994$$

$$AE = -\frac{32000 \times 40}{14 \times 35} = 2612$$

$$AG = -\frac{32000 \times 60}{14 \times 26} = 5275$$

$$AI = -\frac{32000 \times 80}{14 \times 19} = 9623$$

$$AL = -\frac{32000 \times 100}{14 \times 14} = 16449$$

$$AN = -\frac{32000 \times 120}{14 \times 11} = 24935$$

were obtained as stresses in the upper chord due to the vertical reaction at the left support for a live load of 32000 pounds placed at Panel Point 13. and $-\frac{320000}{49} \times 13 + 46 \times AC = 0$

$$AC = \frac{320000 \times 13}{49 \times 46} = 1846$$

$$AE = \frac{320000 \times 24}{49 \times 35} = 4477$$

$$AG = \frac{320000 \times 33}{49 \times 26} = 8289$$

$$AI = \frac{320000 \times 40}{49 \times 19} = 13749$$

$$AL = \frac{320000 \times 45}{49 \times 14} = 20991$$

$$AN = \frac{320000 \times 48}{49 \times 11} = 28497$$

$$AP = \frac{320000 \times 49}{49 \times 10} = 32000$$

for the horizontal reaction.

The stress in the vertical were found by calculation to be for vertical reaction ab = 2286, cd 2832, lf 3461, gh 4132, ik 4692, lm 4753, no 3533.

For horizontal reaction al = 4245, cd = 4607, ef 4953, gh 5187, ik 5070, lm 4128, no 1751.

The diagram for full live load shows that the stress in the upper chord and diagonals are zero and those in the verticals equal to the Panel Load.

The loading for max. stresses was found by diagram ,and either the max. compression or the tension was calculated whichever could be obtained from the stress diagram by simple multiplication, and from this the full live load stress was subtracted to get the other max.

The wind stresses in the lower chords and connected system were obtained by diagram,while the floor system is thought to be sufficiently rigid to take the wind stresses in the upper chords.

The wind stresses in the verticals due to overturning moments transmitted by the sway bracing $= \frac{\frac{1}{2}Pd}{b} = \frac{3000 \times 10}{20}$ =1500 for P Q where P is the Panel Wind load at each chord 20 x 75 = 1500, d the distance between upper and,lower chord and b the distance between trusses. By same formula are obtained NO 1650, LM 2100, IK 2850, GH 3900, EF 5250, CD 6900, AB 8850.

The stresses due to overturning moment in the lower lateral system are found by $\frac{nPd}{b}$ where n is the number of panel loads between panel points,d distance vertically to panel points above. NO. 300, LM 2700, IK 7500, GH 14700, EF 24300, CD 36300, AB 50700.

These stresses were then added as total wind stresses in verticals and used for obtaining the Max. and Min. in the verticals.

Stresses in Sway Diagonals $\frac{\frac{1}{2}Pd}{b} \sec\theta$ $PQ = \frac{3000 \times 10}{20} \times 1.12 =$

1680 ND 1881, LM 2562, IK 3933, GH 6435, EF 8663,

CD 11385, AB 14503 and below intermediate system

EF 5775, CD 9660, AB 16815.

Stresses in Struts of intermediate system $(P+P')\frac{d}{\ell} - P$

AB $= (1500 + 15 \times 59)\frac{59}{46} - 15 \times 59 = 4527$ CD 3183, EF 2204.

The complete stresses in intermediate system are diagramed on sheet by method of Indi~~ves~~.

Designing lower chord by stresses at tabled on sheet they were found too heavy in comparison to Fairmont Park bridge in Philadelphia. By using Cooper's formula,

$$P = 10000 - 45\frac{\ell}{r}\,LL + 20000\,\frac{\ell}{r}\,DL$$

the areas were for Osborns built section 27 inches Web page 57.
OB 72.5, Od 80.6 OF 85.79 OH 88.6 OK 92.6 OM 90.8, 00 86.9.

A second determination disregarding Wind stresses as less than a quarter of combined stresses as recommended by Cooper was made as follows: OB LL 382200, DL 477500, l 286.2, r 9.23, $\frac{l}{r}$ 31.

Unit~~ed~~ stresses 8605 and 17210. area 44.4 + 27.7 = 72.1 square inches. for OD 70, OF 68.3, OH 67.7 OK 67.5, OM 63.8 00 59.

It will be noticed in disregarding wind stresses those sections which were lightest before now require the greatest area, and still except the last two are too heavy by comparison.

I therefore disregarded Cooper's specification as only roughly approximate and calculated O B by the new method.

$$A = \frac{\max S}{\sigma} \qquad \sigma = \frac{p}{n}\left[1 + \frac{u-p}{p}\,\frac{\text{const. } S}{\text{total } S}\right] = 9530\left[1 + \frac{3}{4}\,\frac{477500}{864300}\right]$$

9520 x 1.82 17345.

$$A = \frac{864300}{17345} = 50 \text{ square inches.}$$

It is therefore perfectly safe to take No. 354 page 57, Osburn consisting of 2 Webs 27 x 3/4 and 4 angles 4 x 4 x 3/4 weighing 213 pounds per foot. Area 63.9 square inches for all the Lower Chords.

The Fairmont Park bridge has only 48 square inch in all lower Chords, though the stresses are 636000 to 781000 while here they are 864000 to 933000 including wind stresses.

Weight OB for 6 equal pieces 6 x 213 x 23.85 30480 pounds.

OD 29277, OF 28027, OH 27081, OK 26352, OM 25662, OO 25598.

Weight of all the Webs and Angles in Lower Chord~~s~~ 192377 pounds.

By using Cooper's formula for Upper Chords we have LL 1.8 x 127180

l=240 , r 5.53 $\frac{l}{r}$ 43.4 Unit stress 7047. Area 34.5 which is in excess of a pair of heaviest Channel bars.

The new method gives $A = \frac{\max S}{\sigma_1}$ $\sigma_1 \frac{k}{n(1+c\frac{l^2}{r^2})}\left[1-\frac{k-k'}{k}\,\frac{\max S'}{\max S}\right]$

$\sigma_1 = \frac{9530}{1+\frac{1}{36000}\,\frac{57600}{26.2}}\left[1-\frac{2}{5}\times 1\right]$ $\sigma_1 = 5384$ Area $\frac{127180}{5384} = 23.6$ sq. ins.

adding 6 x 1 x 3/4 for rivets we have 23.6 + 4.5 = 28.1 square inches, requiring 2 Channel bars. 15 inch , 47 pounds per foot for AL

AI = $\frac{121040}{5384}$ = 22..5 + 3.5 = 26 square inches or 15 inch Channel 43 lbs per foot

AN = $\frac{99740}{5384}$ = 18.5 + 3 = 21.5 " " " " 35 lbs "

AG = $\frac{91930}{5384}$ = 17.7 + 2.4 = 20.1 " " " " 33.5 " "

AE , AC and AP need less area than the smallest 15 inch channel of 32 pounds per foot, but are thus taken because the appearance and ease of building and erection would otherwise suffer more than could be gained. It is again referred to Fairmont Park bridge as justification for using smaller areas in Upper chords than specified by Cooper.

Weight of Upper Chords 6 x 20 x 96 = 11280 pounds for AL,

AI 10320, AN 8400, AG 8040, AE 7680, AC 7680, AP 7680.

Weight of all the Channels in Upper Chords 61080 pounds.

The posts were calculated by new method, taking wind stresses into account for AB $\sigma_1 = \frac{9530}{1+\frac{1}{36000}\,\frac{1568760}{26.2}}\left[1-.4\times .66\right]$ $\sigma_1 = 6018$

Area $\frac{137260}{6018} = 22.6$

requiring 15 inch channels 38 pounds per foot.

CD 22 square inch 15 inch Channels 36 pounds per foot.

EF 18.6 - 15 - 32 -

GH 15.6 12 26 - -

IK 12.6 12 21 - -

LM, NO, PQ require less than the smallest 12 inch Channel of 20 pounds which is taken.

Weight AB $= 58 \times 76 \times 6 = 26448$ pounds CD 19440, EF 13056, GH 7800 IK 4536, LM 3120, NO 2400, Pq 2160.

Weight of all the Channels in the Posts 52512 pounds.

The Diagonals are also calculated by the new method for

BC $$\sigma_1 = \frac{9530}{1 + \frac{1}{36000}\frac{152887}{19.4}}\left[1 - \frac{2}{5} \times 1\right] = 4287$$

Area $\frac{68400}{4287} = 15.7 + 3$ for rivet holes 18.7 square inch, 12 inch Channel 31 pounds.

DE 17.1 sq. inch 28.5 pounds FG 154 sq inch 25.5 lbs.

HI 13.3 " " 22 " KL 11 - - - 20 lbs because smallest 12" Chan.

MN 16 " " 26.5

OP required more area than the heaviest 12 inch Channel and was calculated for a 15 inch Channel requiring 21.9 square inch 36.5 pounds.

Weight of Diagonals BC $48.5 \times 62 \times 6 = 18042$ pounds.

DE 13338 , FG 9490, HI 6996 , KL 5520, MN 6840, OP 9290.

Weight of all the Channels in the Diagonals 69516 pounds..

Wind bracing Struts $P = 13000 - 70\frac{l}{r}$ $l = 221.4$ $r = 2.31$, $\frac{l}{r} = 95.6$

$P = 13000 - 70 \times 95 = 6310$. Area $= \frac{3900}{6310}$ 6.2 square inches. Requiring two 6 inch channels 10.5 pounds per foot. I 5.7 square inch 9.5 lbs. per foot. II 4.75 square inch, 6 in. Ch. 8 lbs. per ft., III 3.9 sq. in. 7 lbs. per foot 6 in. CH., IV 3.6 sq. in. 6 lb. per foot 6 in. CH., V 2.4 sq. in. 5 in. Ch. 6 lbs per foot. VI 2 square in, 4 in CH. 5 lbs. per ft. VII 4 in. CH. 5 lb. per foot.

Weight O 4x18.45x 21 1550 lbs.I 1402,II 1180,III 1033,IV 885,,V 885, Vl 738, VII 369.
Weight of all the Struts for Wind bracing between lower Chords 8042 lbs.

Diagonals for Wind bracing. Unit stress 18000 lbs. per square inch.

I $\frac{60300}{18000}$ = 3.36 sq. in. 3 bars 1 3/16 each, 11 pounds per foot.

II 2.78 sq. in. 2 bars 1 5/16 each, 9.02 lbs. III 2.2 sq.in. 2 bars 1 3/16 each 7.38 pounds. , IV 1.72 sq.in. 1 1/2 in. 5.89 lbs.

V 1,2 sq. in. 1 1/4 in.4.9 lbs. , VI .7 sq.in. 15/16 in.2.3 lbs.

VII .3 sq.in. 5/8 in. 1.03 lbs.

Weight I 30 x 8 x 11 = 2640 lbs., II 2092 lbs, III 1682 lbs. IV 1319, V 900, VI 497, VII 223.

Weight of all the Diagonals for Wind bracing between lower Chord 9353 lbs. Struts in intermediate system O 738 lbs., I 885 lbs., II 885, III 885, together 2593 lbs.

Diagonals I .425 sq.in. 3/4 in. bar 1.48 lbs. per ft. II .729 1 in bar 2.62 lbs. per ft. III .945 sq.in. 1 1/8 in. bar 3.31 lbs per ft. Weight I 317 lbs. , II 566 lbs., III 715, together 1598 pounds.

$$\sigma_1 = \frac{9530}{1+\frac{1}{3600}\,\frac{57600}{5.2}}\left[1-\frac{2}{5}\times\frac{21309}{26721}\right] = 4925 \qquad \text{Area} = \frac{26721}{4925} = 5.4$$

Two 6 in. Channels 9 lbs. per ft.
II and III have considerably less stress but are made of lightest 6 in. Channels 7 lbs. per foot for sake of appearance.

Weight I 6 x 20 x 18 2160 lbs. II 1680 lbs. III 1680, together 5520 lbs

Weight of all the channels and rods in intermediate system 9711 pounds.

Sway bracing AB upper .806 sq. in. 1 1/16 in. 2.95 lbs. per foot AB lower .934 1 1/8 in. 3.31 lbs. per ft. CD upper .632 sq.in. 15/16 in. 2.3 lbs. per ft. CD lower .536 sq.in. 13/16 in. 1.73 lbs. per ft. ,EF upper.541 lbs. per ft. 7/8 in. 2 lbs. per ft. EF lower. 351 lbs. per ft. 11/16 in. 1.24 lbs per ft. GH .357 sq.In. 11/16 in 1.24 lbs. per ft. IK .213 sq. in. 9/16 in. 828 lbs. per ft. LM .142 sq.in. 7/16 in. .5 lbs. per ft. NO .105 sq in 3/8 in. .368 lbs. per ft. PQ .0934 sq.in. 3/8 in.368 lbs per ft.

Weight AB upper 8 x 30 x 2.95 = 708 lbs. AB lower 954 lbs. CD upper 552, CD lower 360, EF upper 480, EF lower 198, GH 298, IK 172, LM 88 NO 62, PQ 62.

Weight of all the rods in the Sway bracing 3934 pounds.

Pins. The maximum stresses on the lower pins obtained by combining the maximum in the verticals and lower chords meeting at them are 950000 lbs. Assuming Pins of 12 in. diameter and a unit compressive stress of 12500 pounds per square inch in width of the bearing is = $\frac{950000}{12 \times 12500}$ = 6.3 inches.

The upper pin has three members with a combined stress somewhat lower acting on it and needs 6 inch length of bearing. As all the members have reversed stresses except the lower chords which have compression only, all the joints are riveted and there is no moment on the Pins. Weight of upper pins 3600 pounds.

Rivets. The lower chords require 5/4 x 3/4 + 3/16 = 17/16 in rivets. for riveting angles to webs. Pitch 4 in. Length of chords 900.24 ft. Number of rivets $\frac{900 \times 2 \times 12}{4}$ = 5400 Weight 5400 x .653 = 3526 lbs. Stay plates, thickness 3/8 in, length .50 + $\frac{d}{D}$ 15 .5 x 27 + $\frac{16.5}{27}$ + 1.5 = 16 in. width 26 in. Weight 168 x 43.3 7275 pounds.

Splice plates, inside				3/8 x 30 x 36	Weight	72 x 112.5	= 8100 pounds.
			outside	3/4 x 19 x 20	"	72 x 62	= 4464 "
Bearing plates at Pins			inside	3/8 x 30 x 36	"	18 x 112.5	= 2024 "
"	"	"	" "	1/2 x 30 x 19	"	24 x 79	= 1896 "
"	"	"	" outside	3/4 x 19 x 36	"	24 x 142	= 3409
"	"	"	" "	3/8 x 25.5 x 30"		24 x 79.5	= 1908 "
"	"	"	" "	1/2 x 19 x 24	"	24 x 63.3	= 1520 "

Lattice Bars (phoenix Bridge Comp. table) 3 x 1/2, 5 pounds per ft.

Spacing 22 C to C. Length 24 inches Weight $\frac{1524 \times 2 \times 24 \times 5}{22}$ = 16625 lbs.

Rivets for laticing 1 in. diam. Weight 1659 x.653 = 1083 lbs.

Weight of all the plates, Lattice bars and rivets in lower chords 51830 pounds. The greatest stress in upper chords is 127180 lbs. and as there are splice plates on both sides the rivets have to resist crushing on a 3/4 in. plate which resistance for 1 in. rivets = 9370 pounds requiring $\frac{127180}{9370}$ = 14 rivets, spaced in two rows of 4 and 2 of 3 rivets.

Splice plates inside	3/8 x 30 x 30	Weight	90 x 94	8460 pounds.
" " outside	3/8 x 30 x 14	"	90 x 44	3960 "
Stay plates	3/8 x 23.2 x 16	Weight	168 x 38.3	6434 "

Lattice bars 3 x 3/8 3.75 pounds per ft. spacing 22 in. length 24 in.

Weight $\frac{1375 \times 2 \times 24 \times 3.75}{22}$ = 10410 pounds.

Rivets for latticing 1 in. diam. 975 lbs.

Weight of all the plates, lattice bars and rivets in upper chords 30239 lb

The Verticals and Diagonals are riveted to the inside splice plates of the upper and lower chords and need angles at the outside only. Dimension 3 x 4 x 3/8 82 pounds per foot.

Vertical angles 14 in. long	Weight	60 x 9.56	574 pounds.		
" 11 " "	"	120 x 7.5	900 "		
Stay plates 3/8 x 23.5 x 10	"	60 x 24.5 =	1470	for 15 in. channels.	
" 3.8 x 23.5 x 9	"	130 x 22.1	2917	" 12 "	"

Lattice bars 3 x 3/8 3.75 lbs per ft spacing 22 in. length 24 in for 15 in channels.

Weight $\frac{1312 \times 2 \times 24 \times 3.75}{22}$ = 9916 lbs.

Lattice bars 2 1/4 x 3/8, 2.81 lbs. per ft. spacing22 in. length24 in for 12 in channel

Weight $\frac{1188 \times 2 \times 24 \times 2.81}{22}$ = 7283 lbs.

Rivets for latticing 1400 pounds.

Weight of all the plates, lattice bars, angles and rivets in verticals 24500 pounds.

Diagonals, angles averaging 1.5 ft. Weight 168 x 12.3 2067 pounds.

Stay plates 3/8 x 23.5 x 9. Weight 168 x 22.1 = 3713 .

Lattice bars 2 1/4 x 3/8 , 2.81 lbs per ft. spacing 22 in. length 24 in.

$$\text{Weight} \quad \frac{2388 \times 2 \times 24 \times 2.81}{22} = 14640 \text{ pounds.}$$

Rivets for latticing 1700 pounds.

Weight of all plates,lattice bars,angles and rivets in diagonals 22120 pounds.

Wind Bracing, Struts connected by angles 3 x 4 x 3/8, 8.2 lbs. per ft. to lower chords 64 of 6 in length, 16 of 5 in. length, 32 of 4 in length.

Weight 49.3 x 8.2 = 404 pounds.

Stay plates 1/4 x 27 x 14. Weight 26.1 x 120 = 3132 pounds.

Lattice bars 1 3/4 x 5/16, 1.82 pounds per ft spaced 14 in. double, length 30 inches.

$$\text{Weight} = \frac{1070 \times 2 \times 30 \times 1.82}{14} = 8346 \text{ pounds.}$$

Rivets for latticing 526 pounds.

Weight of all the plates,lattice bars,angles and rivets of Wind struts 12408 pounds.

Diagonals of Wind bracing require 88 sets of clevices,bolts and pins plates for fastening,weighing 40 pounds each, 3520 pounds.

Sway bracing require 80 sets. Weight 3200 pounds. Intermedi ate system,Chords riveted to verticals.

Stay plates 1/4 x 20 x 10. Weight 36 x 13.9 = 600 pounds.

Lattice bars 1 3/4 x 5/16, 1.82 lbs. per foot spaced 22 in,length 24 in.

$$\text{Weight} = \frac{648 \times 2 \times 24 \times 1.82}{22} = 2576 \text{ pounds.}$$

Rivets 320 pounds. Together 3496 pounds.

Struts,angles 3 x 4 x 3/8, 8.2 pounds per ft. 48 of 5 in and 16 of 4 in. length.

Weight 25.3 x 8.2 = 207 pounds.
Stay plates 1/4 x 9 x 6 Weight 64 x 3.75 240 pounds.
Lattice bars 1 3/4 x 5/16, 1.82 lbs per ft spaced 14 in. length 12 in.

Weight $\frac{576 \times 2 \times 14 \times 18\frac{3}{4}}{14}$ = **1788 pounds.**
Rivets 285 pounds **together 2520 pounds.**

Diagonals of intermediate system require 48 sets of clevices,bolts and pin plates. Weight 1920 pounds.

Weight of all the plates,lattice bars,angles clevices,bolts and rivets **in** intermediate system 7936 pounds.

Entire weight of Trusses

		Connections.
Lower Chords	192377	51830
Upper	61080	30239
Verticals	52512	24500
Diagonals.	69516	22120
Wind bracing Struts	8042	12408
Diagonals.	9353	3520
Intermediate system	9711	7936
Sway bracing.	3934	3200
	406525 pounds.	Pins 3600
		159353 pounds.

Weight of Bridge.

Floor system	1293796 pounds.
Trusses	565878
	1859674 pounds.

Dead load per linear foot $\frac{1859674}{280}$ = 6642 pounds.

The bridge is calculated for a dead load of 6000 pounds per linear ft., the excess is 10.7 percent,which would be permissible in any kind of **bridge**,and more so in this,where the **stresses** due to dead load are zero for all members,except the lower chords and verticals,in which an **in**crease of the above amount makes a very slight difference.

Camber. The upper chords have reversed stresses of equal amounts and need only the 1/8 in. per 10 ft. or 1/4 **in.** per panel excess for camber over the Panel length of 20 ft. ,making it 20 ft + 1/4 **in.**

The diagonals also have reversed stresses of equal magnitude and require no change in length.

The verticals have in addition to equal reversed stresses a dead load of 40000 lbs. each $e = \frac{40000}{26000000\left[\frac{119000}{9000} + \frac{40000}{18000}\right]} = .00001$

The verticals must therefore be lengthened by $\frac{1}{100000}$ their length as taken from the detail drawing.

The lower chords have $e = \frac{445960}{26000000\left[\frac{470680}{10000} + \frac{445960}{20000}\right]} = .0000247$

which multiplied with the length and added to it gives the actual required length (corrected) for compression, of lower chord members.

The details of connections and floor system are given in a separate drawing.