DESIGN & DETAILS OF A SINGLE TRACK SINGLE LEAF TRUNNION BASCULE BRIDGE

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LDesign and general details
for a single track, single

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THESIS

DESIGN AND GENERAL DETAILS

FOR

A SINGLE TRACK, SINGLE LEAF, TRUNNION BASCULE BRIDGE.

By H C Anderson
H W Schlinz
T E Richards Jr.

GENERAL DIMENSIONS

Length of trusses - 150'-0", (center line to center line)
Span - 120'-0", (center line to center line)
5 panels at 24' or total of 120'-0".
1 panel at 30'
Required clear channel of stream - 100'-0".
Required river clearance - 14'-6".

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INTRODUCTION.

The main object to be attained by this somewhat peculiar design is to avoid the heavy steel bolsters which are commonly employed for supporting the trunnions of this type of bridge. As will be readily seen from the drawings, this object is realized by using a slotted abutment to accomodate the tail end of the trusses when the bridge is lifted, the tail end of the floor remaining stationary.

It is easy to see that in designing a bridge of this type, parts of which are more or less original and somewhat complicated, considerable experience in bridge work would be necessary in order to arrive at an ecomonical and practical design. Along this line we have been very fortunate in having the kind assistance of Professor M.B.Wells, to whom we are indebted for many ideas and practical hints with regard to design and details of the structure.

LOADING AND SPECIFICATIONS.

A dead load of 22500 pounds was assumed at each panel point and the live load at each panelwas figured as 67500 pounds, using Coopers E 50 loading. The stresses were then calculated with these loads as explained in a later paragraph on "Stresses" and the members and the weight of each was then calculated from this data. Taking into consideration the floor system, members, wind bracing, etc., and roughly estimating connection plates, the dead panel load at each panel point was figured, with result shown on drawing-sheet #1. With these values for dead load at each panel point, the dead load stresses were refigured, and after proper combining of wind, dead and live stresses, the members were figured.

The dead panel loads were figured in the same manner as before but by using the new weights of members. These loads came so close to the former calculation, that the difference would not warrant an entire recalculation of stresses, so these dead panel loads were used in the final calculation of stresses.

The wind stresses were figured for the bridge with a moving load of 450 pounds per foot of bridge and a dead load of 150 pounds per foot of bridge, considering bridge as a simple span and closed. Then with a load of 200 pounds per foot of length the top and bottom bracing and chord wind stresses were figured, the bridge acting as a cantilever. The angle which the bridge will make with the horizontal when up is 70 degrees, but according to Green, when figuring wind stresses on a surface leaving an angle greatbr than 60 degrees, the wind is considered as acting at right angles to that surface so a wind load of 25 pounds per square foot was considered acting along the track and the effect on the trusses calculated. Considering that an open floor system is used and that there is traffic in the river the year round, no snow load was taken into consideration in the loading. In detailing and designing members, portals, sway and lateral tracing, floor beams and stringers, etc., Coopers General Specifications for Steel Railroad Bridges and Viaducts

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ST235

(1906 Edition) were followed excepting in combining stresses ih certain cases as shown later.

STRESSES.

In computing the stresses in the truss members, the live load was taken at 67500 pounds per panel per truss, this being the same as the maximum shear in stringer, although the maximum live load concentration which can occur at a panel is 92500 pounds, this stress having been combined with the dead load stress in the design of the hip vertical. This however would have been too great to be used as the panel load throughout the truss. Impact due to live load was not considered in the design, the live load allowable stress being taken at one half of the value which would be used if impact were considered. In the design of members, Cooper's Specifications for Railway Bridges were used throughout except as noted herewith: All calculations of members were made with the live load unit stresses as a basis, the dead and wind load stresses being reduced to live load equivalent. Thus 2/3 of dead load stresses and 1/2 of wind lateral stresses were combined with the live load stresses, algebraically. If wind load stresses were less than 30 per cent of the other <u>combined</u> stresses they were neglected entirely.

Rivets in members where reversal of stress occurred due to wind load only were figured merely for the maximum combined stress, the reversal being neglected. (See specifications).

In calculating the dead load stresses, the bridge was considered as a cantilever, that is, the counterweight was taken as just balancing the dead weight of the movable leaf, this weight having at that time been found only tentatively. The concurrent stresses were then combined and an approximate design of members obtained, allowances being made for connection plates, laterals, lacing,etc., when the approximate weights of members were found. In this way the corrected panel weights were found and the stresses and sizes of members again calculated, the weights of members being figured accurately.

The floor system and floor beams were designed according to Coopers E 50 loading. The weights of each member of the floor system were found as well as the center of gravity of the whole.The center of gravity of the truss members were also obtained and this combined with the corresponding values for the floor system gave the point at which the mass of the bridge might be considered as concentrated. Knowing this, the amount and distribution of counterweight was found, the center of gravity of the latter being on the line joining the center of gravity of bridge and trunnion,produced. By locating the center of gravity of the counterweight on this line and making the moment of the counterweight about the trunnion equal to total amount of the dead load on the moving leaf,the bridge would be balanced in any position and the power necessary to raise the same would be reduced to the amount necessary to overcome wind load gear losses and journal friction.

The counterweight end of the lower chord was designed to take the compression which would come on it with the bridge in a horizontal position, and also to take the <u>entire</u> tension due to the counterweight when the bridge was raised.

STRESSES.

EMBERS	DEAD LOAD, BRIDGE AS CANTILEVER	MAX. LIVE LOAD. BRIDGE CLOSED	WIND LOAD IN DIRECTION OF TRACK BRIDGE UP	LATERAL WIND LOAD BRIDGE CLOSED	LATERAL WIND LOAD BRIDGE UP	DESIGN FOR (USE LIVE LOAD UNIT STRESSES)
AI	+14100		+2200	+22200		
		-172900	-2200	-22200		-177500#
AB	+32000		+5600	+0		
		-162000	-5600	-7200		-147100#
B6	+32000	-	+5600	+7200		
		-162000	-5600	-7200		-147100#
cD	+70400		+12600	0	+43200	+136200#
		-162000	-12600	-7200	-43200	-162100
DE	+200800	0	+35000			133900
			-35000			
E2	+283100	0	+49500			+188800
			-49500			
e2	+300000	0	+35000			+307000
	-200800	0	35000			240900#
de		+108000	+22400	+0	43200	
	-125600		-22400	-64800	-67500	-129400#
cd		+108000	+22400	+53200		+105200
	-125600		-22400	-64800		-175000#
bc		+162000	+12600	+64800		+163300
	-70400		-12600	-64800		129200#
ab		+108000	+1400	+53200		+129800
	-8800		-1400	-64800		-68900
Ia		+108000	+1400	+0		+125100
	-8800		-1400	-53200		-58320
Ab		+103700	+6700			+100000
	-37200		-6700			-75600#
Cb	+61500	+51900	+11200			+92900
		-51900	-11200			
Cc		+40500	+8800			
	-48000	-40500	-8800			-72500
Dc	+88400	+103700	+15700			+147900
		-17300	-15700			
Dd	+22000	+59100	+3500			+70100
			-3500			
De			+20100	+22000		
	-124200	-175000	-20100	-22000	-69100	-273900
YZ	+6000	+16900	+1000			+20900
			-1000			
dZ	0					
	0	-10800				-10800
Ee		0	+3500			
	-200800	0	-3500			-133900
Aa	+18000	+92500	+3500			+104500
			-3500			

Members indicated by asterisk should be figured for reversal of stress in rivets. (See specifications)

STRESSES (cont'd).

The part of the lower chord marked Ye on the stress sheet was designed as a beam fixed at the outer end, to take lower lateral stresses. The top lateral stresses were calculated for two positions of bridge:- 1st, closed (in which case the bridge was considered as supported at both ends in resisting lateral forces). 2nd, as just off the abutments (in which case the bridge acted as a cantilever, and all lateral forces were transferred to the trunnion through the member De). The first case produced maximum stresses in all laterals except those in panel DC. For this panel the stresses of case 2 were put on the stress sheet for these members, which for the other panels the stresses of case 1 were taken.

OPERATING MACHINERY.

The operating machinery will be located between the two approved girders and beneath the track. It will be protected from the weather by the webs of the girders and by a solid floor built on that part of the girder. The power will be furnished by a compound D.C. Motor of 130 Horse Power, running at 600 R.P.M. under ordinary conditions when there is little or no wind to oppose the lifting of the bridge. The power necessary to lift bridge has been figured on the assumption of a wind pressure of 15# per sq. foot of exposed area, the wind coming in a direction parallel to the track, and the bridge to be raised in 40 seconds. This wind velocity produces a stress in the rack, at pitch line, of 21000# on each truss which must be counteracted by the motor. The efficiency of the entire mechanism was assumed at 60 per cent, this including losses in gears, gear shaft journals and trunnion bearings.

To raise the bridge in 40 seconds through 70 degrees would require a speed along the pitch line of 55 feet per minute. Then allowing 60 per cent efficiency, the foot pounds per minute at the motor would be 55 x 21000 x 100/60=1,925,000 foot pounds which is equivalent to 58.3 Horse Power for each truss or a total of 116.6 Horse Power.

For the design of the teeth of rack, and on the engaging pinion the maximum wind stress of 35000# was used, the tensile strength of cast steel being taken at 5000# per sq. inch in the design. With a rack 18" wide, this gives teeth having a pitch of 6.5" and a height of 4.5". The pinion operating the rack has 10 teeth, the diameter being 20.7" on the pitch circle. On the same shaft with this pinion is a gear of 10" face and 4" pitch and 2.75" height of tooth. It has 75 teeth and is engaged by a pinion of 10 teeth mounted on a 5" shaft. On this same shaft is placed a gear of 95 teeth having an 8" face and 3" pitch. Driving this is a gear on the motor shaft. This has a 3" pitch and is11-1/2" in diameter.

The rack and pinion are to be cast steel, the teeth not being machined. The same applies to the intermediate gears. The pinion on the motor shaft and the corresponding gear are to be machined or cut.

No mention has here been made of brakes, methods of control,

signals and other devices which form part of the equipment in operating houses for bridges of this character, this being too large a subject to be included in a civil engineering thesis of this kind.

Due to the position of the machinery a slot will probably have to be cut in the rear wall of masonry, but as the machinery is not fully designed, this change in the masonry was not shown on the tracing.

Although a 120 Horse Power motor is to be installed, the power necessary to lift the bridge may at times of high wind velocity be much more, but as a compound motor will operate satisfactorily under an overload of 50 per cent this greater horse power is taken care of, and affecting the operation only in so far as it will take perhaps a minute to raise bridge. On the other hand under ordinary conditions the wind load will be considerably less than was assumed and the power required, and also the time of raising bridge, will be materially lessened.

LOCKING DEVICE.

In a bridge of this type, when the trusses are counterbalanced by a weight in such a way that the bridge will remain at any angle to the horizontal if no outside force is exerted upon it, it is necessary that the bridge be locked in some manner before trains or other traffic be allowed to enter upon it. If a locking system of some sort were not used, aside from the apparent danger of a serious accident, there would be a tendency for the bridge to rebound slightly just after a set of trucks had left it, assuming in this case that the train is coming off at the locked end, and by the time the next set came up there would be a tendency for the wheels to jump the small gap thus left with the result that there would be a pounding on the rails such as takes place at rail joints on straight track. This rebounding would be greatly increased if a wind of high velocity were blowing. With the train coming on the bridge there would be a pounding on the rails and an unnecessary jarring of the trusses. With a train going in the opposite direction the result would be practically the same excepting that the pounding would be on the other side of the rail joint.

Without a locking device on the trusses themselves it would be necessary to have a locking device on the operating gears and this would necessitate designing the machinery to take a great part of the live load, in fact we would have a continuous span, with three joints of support and the tooth that would be in contact with the operating rack would have to take all the live load reaction at that point. This would make necessary the designing of much heavier gears and this, along with the cost of replacing pounded rails very often, would increase the cost of construction and maintainance very noticeably.

The locking system used in this case consists in the main of two 3 inch pins 2'-3" long (one at each truss) which are operated by means of electric magnets. One of the magnets is placed on the masonry at each truss and the other on the truss, the

operator of the bridge being able to throw whichever magnet he desires into action by turning a switch, the wiring to be done in such a manner that only one magnet can be charged at a time and the other is always charged.

MASONRY.

The substructure for supporting the bridge will be built of Portland cement, concrete reinforced as shown on the drawings. The abutments are to be also reinforced for temperature and shrinkage stresses. This reinforcing is not shown on the drawings but there is to be placed a 3/4 inch corrugated bar to every square foot of cross section. The abutments are assumed to rest on good clay subsoil with a bearing power of 4 or 5 tons per square foot. No piles are required, under the masonry as the pressure only comes up to about 2 tons per square foot. A row of protecting piles is to be driven around the piers on the channel side. The clear channel will be finally about 101 feet.

METHOD OF BREAKING TRACK.

The way in which the track is to be broken is best shown by the sketches on the stress sheet. In working out the scheme used, attempt was made to make it as simple as possible and to support the rails as rigidly and as near the ends as possible.

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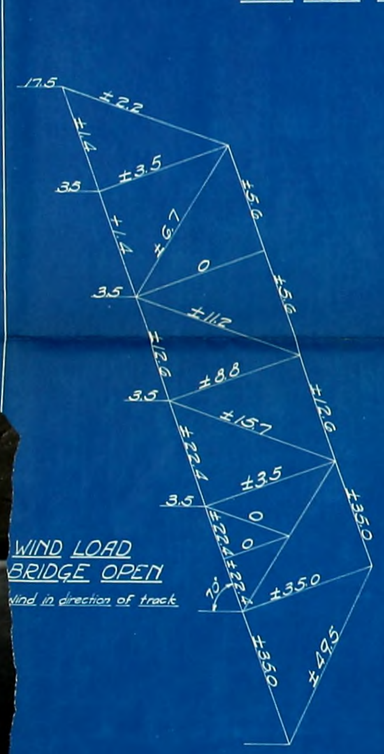
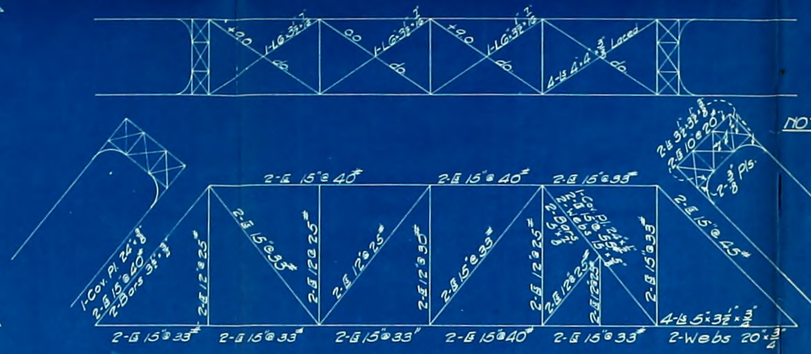
LIVE LOAD - BRIDGE CLOSED

DEAD LOAD - BRIDGE AS CANTILEVER



WIND LOAD
BRIDGE OPEN
Wind in direction of track

BRIDGE OPEN
Counterweight Stress in ez


+20
+46.3
+46.3
+20
+46.3
+46.3
4-L 4"x4"x
do
do
do
Lacing
1-Cov. Pl. 24"x
2-L 5"x3½"x
1-Bar 5"x
2-L 5"x3½"x
2-L 5"x3½"x
2-L 5"x3½

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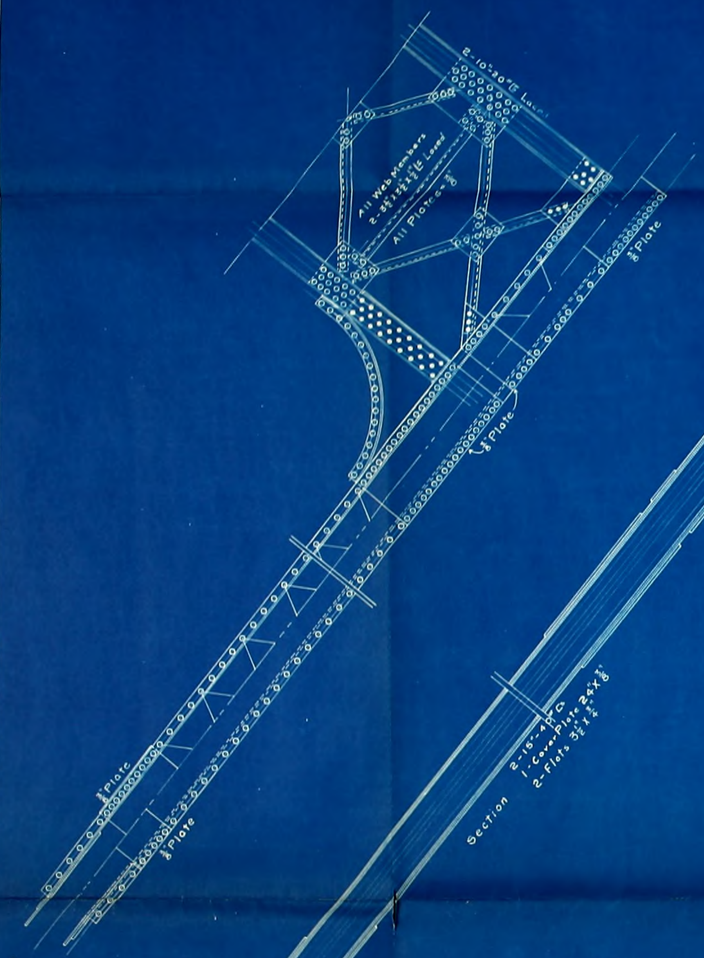

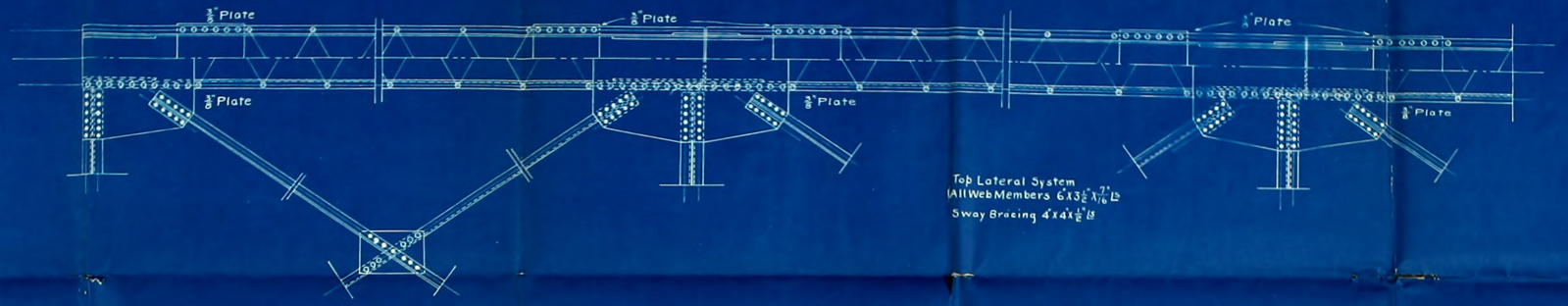

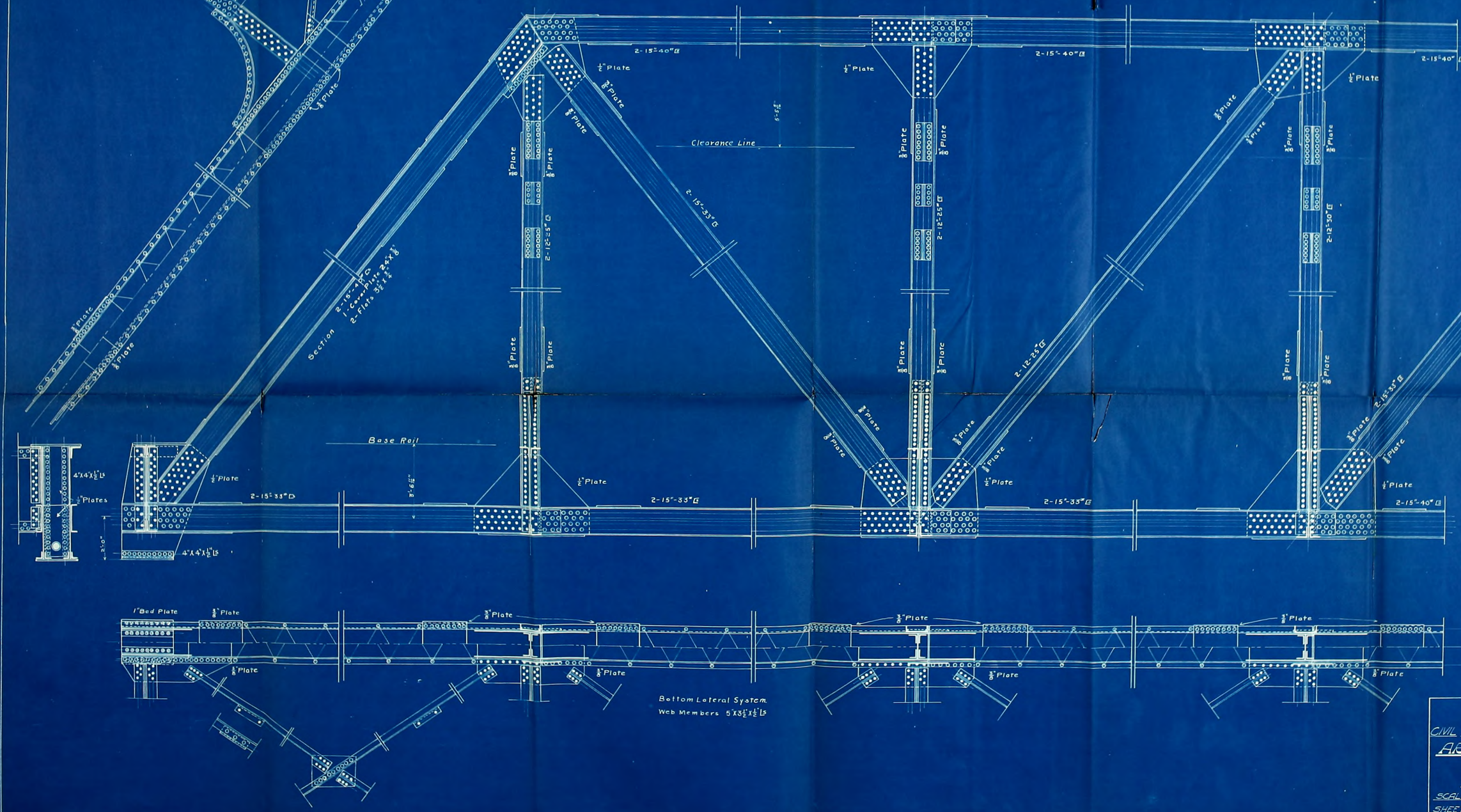

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2-12"-25#C's

2-12"-25#C's

2-15"-50#C's

2-12½"x⅜" Plates

3½"x3½"x⅜" L's For Diaphram

2-12"-25#C's

Base of Rail

2-15"-55#C's

2-15"-35#C's

All Batten Plates ⅜"

All Rivets ⅞"

6"x3½"x⅜" L's

⅜" Plate

6"x3½"x⅜" L's

⅜" Plate

5"x3½"x⅜"x9"

6"x3½"x⅜" L's

⅜" Plate

6"x3½"x⅜" L's

½" Plate

⅜" Sp. Plates

2-15"x½" plates

2-15"-55#C's

2-15"-35#C's

2-15"-35#C's

2-3½"x3½"x⅝" L's, Laced

6"x3½"x⅜" L's

⅜" Plate

5"x7"x¾"x36"

16½" Trunion

Steel Casting For Trunion

3½" Brass Bushing

Pedestal.

Steel Casting

⅜" Plates

10"-25#C's

6"x3½"x⅜" L's

Counterweight on Chord rest

½" Plate

Counterweight on Chord rest

2-3½"x3½"x⅝" L's, Laced

51-2

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Counterweight Blocks 12"x12"x14"

1½" Bolts

5½" Trunnion

8'x7'x1⅜" L's

6'x3½"x⅜" L's

Counterweight on Chord rem'vd

Gear on Motor Shaft

Gear #1

Gear #2

¾" Plate

2-5"x3½" L's

½" Spl. Pl's

2-3½"x3½"x⅝" L's, Laced

2-15"50 #ch

¾" Plate

¾" Sp. Plates

¾" Plate

¾" Plate

3

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Bottom Lateral System
Dotted Lines indicate Top Bracing
Top & Bottom Bracing $\frac{L}{6}$ All 4"x4"x$\frac{1}{2}$"
All Connection plates $\frac{3}{8}$" Thick

℄ Approach Girder "G"

Approach Girder "G" (2 Req'd)

Mat. Plate 1" Thick

41'-0"

20'-6"

20'-6"

8"x8"x$\frac{11}{16}$"Ls

8"x8"x$\frac{11}{16}$"Ls

$\frac{11}{16}$" Filler Plate
8"x8"x$\frac{11}{16}$"Ls

$\frac{1}{2}$" Web
Web Stiffners 5"x3$\frac{1}{2}$"x$\frac{3}{8}$"
$\frac{L}{6}$ (Crimped)

5'-0"

Stringer "S" (8 Req'd)

23'-11$\frac{1}{2}$"

$\frac{11}{16}$" Filler Plate
2-6"x6"x$\frac{11}{16}$"Ls

$\frac{1}{2}$" Web
2-6"x6"x$\frac{7}{16}$"Ls

5"x3$\frac{1}{2}$"x$\frac{3}{8}$" Con Ls

5"x3$\frac{1}{2}$"x$\frac{3}{8}$" Con Ls

5'-0"

Floor Beam "F2" (4 Req'd)

14'-7" (Ab't)

2-6"x6"x$\frac{3}{8}$"Ls

$\frac{3}{8}$" Plate

2-6"x6"x$\frac{7}{16}$"Ls

$\frac{7}{16}$" Web

2-5"x3$\frac{1}{2}$"x$\frac{3}{8}$"Ls ℄ Stringer

2-5"x3$\frac{1}{2}$"x$\frac{3}{8}$"Ls

2-3$\frac{1}{2}$"x3$\frac{1}{2}$"x$\frac{3}{8}$"Ls

2-3"x3"x$\frac{3}{8}$"Ls

6'-7$\frac{1}{2}$"

4'-4"

7'-0"

$\frac{3}{8}$" Plate

Floor Beam "F" (1 Req'd)

14'-8"

℄ Stringer

℄ Stringer

Standard Stringer Connections

$\frac{7}{16}$" Filler Plate
2-6"x6"x$\frac{7}{16}$"Ls

$\frac{1}{2}$" Web
2-6"x6"x$\frac{1}{2}$"Ls

5"x5"x$\frac{3}{8}$" Con Ls

5"x5"x$\frac{3}{8}$" Con Ls

Stringer Bracket "S3" (4 Req'd)

1'-10$\frac{1}{2}$"

20"-80# I Beam

1'-10"

Toilend Stringer "S2" (2 Req'd)

11'-11$\frac{1}{2}$"

6"x6"x$\frac{3}{8}$" Con Ls

20"-80# I Beam

11'-11$\frac{1}{2}$"

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Floor Beam "F" (1 Req'd)

14'-8½"

5"x3"x⅜" Con Ls — 2'-8½" — 5"x3½"x7/16" Ls — ½" Web — 2-5"x3½"x7/16" Ls — 5"x3"x⅜" Con Ls

Standard Stringer Connections

Stringer

7/16" Filler Plate

5"x5"x⅝" Con Ls

4'-0½"

Tail End Stringer "S2" (2 Req'd)

11'-11½"

6"x6"x⅝" Con Ls — 20"-80# I Beam — 6"x6"x⅝" Con Ls

Stringer Bracket "S3" (4 Req'd)

1'-10½" — 20"-80# I Beam — 1'-0" — ⅜" — ½"

16'-0"

7'-0"

4'-7½"

Sway Bracing (5 Req'd)
All connection Plates-⅜"
All Ls 4"x4"x⅜"

5'-10"

4'-8"

11'-7"

30'-0"

6'-6"

⅜" Filler Plate
6"x8"x⅜" Ls

Ls

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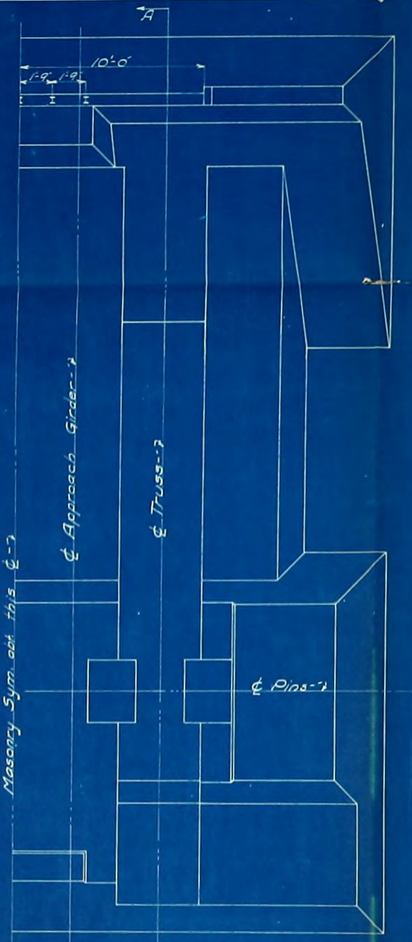
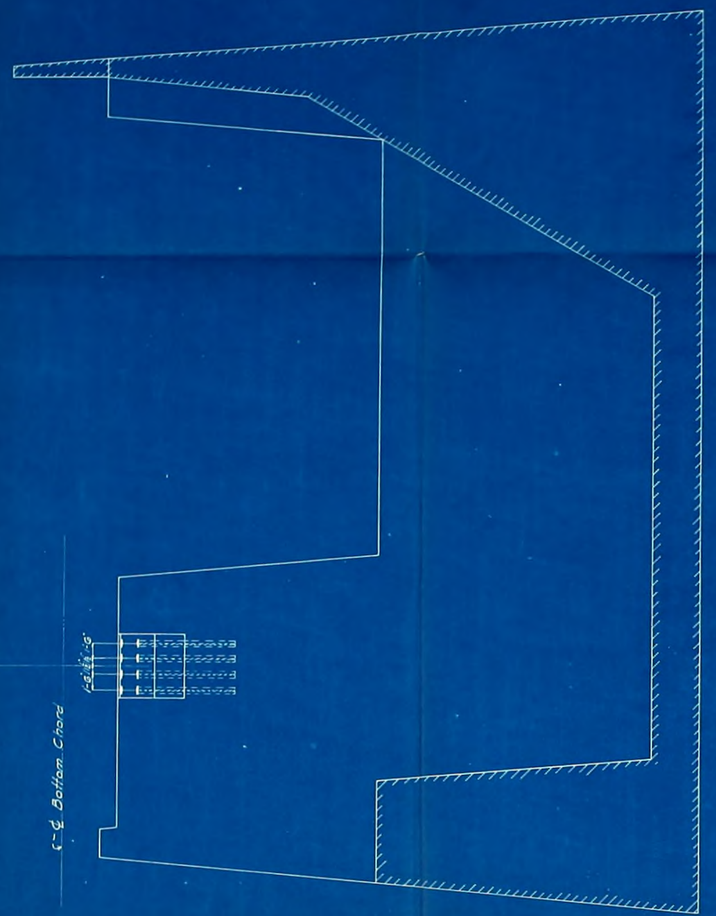
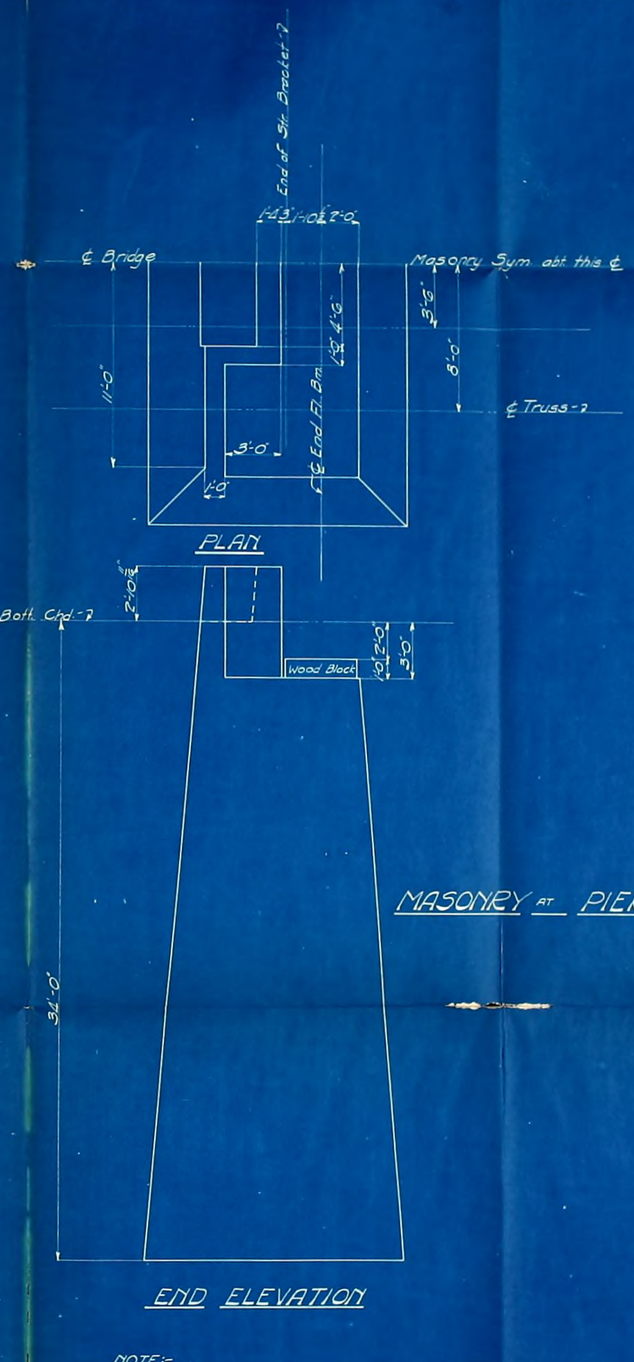


A PLAN

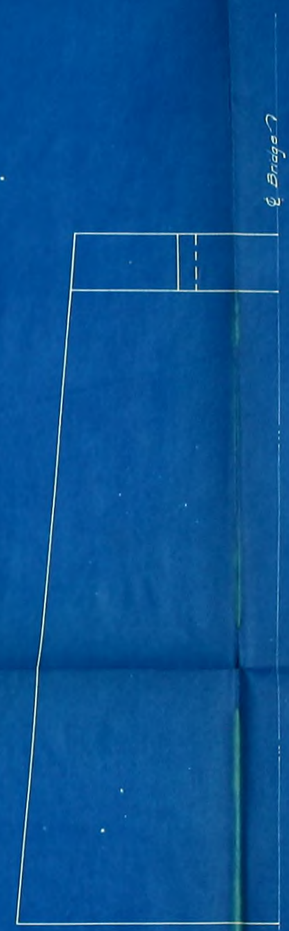


SECTION A-A



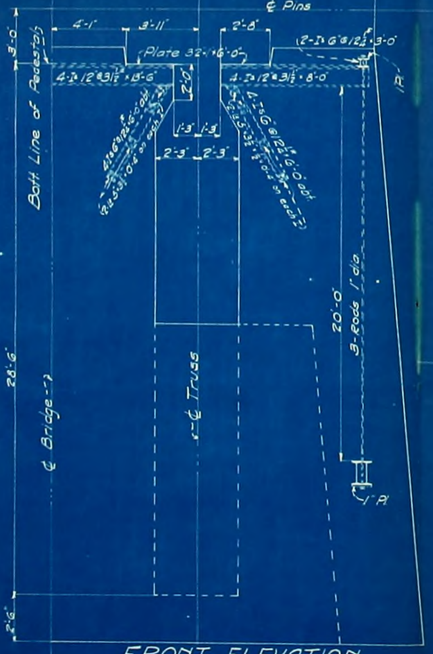
PLAN

MASONRY AT PIER 1

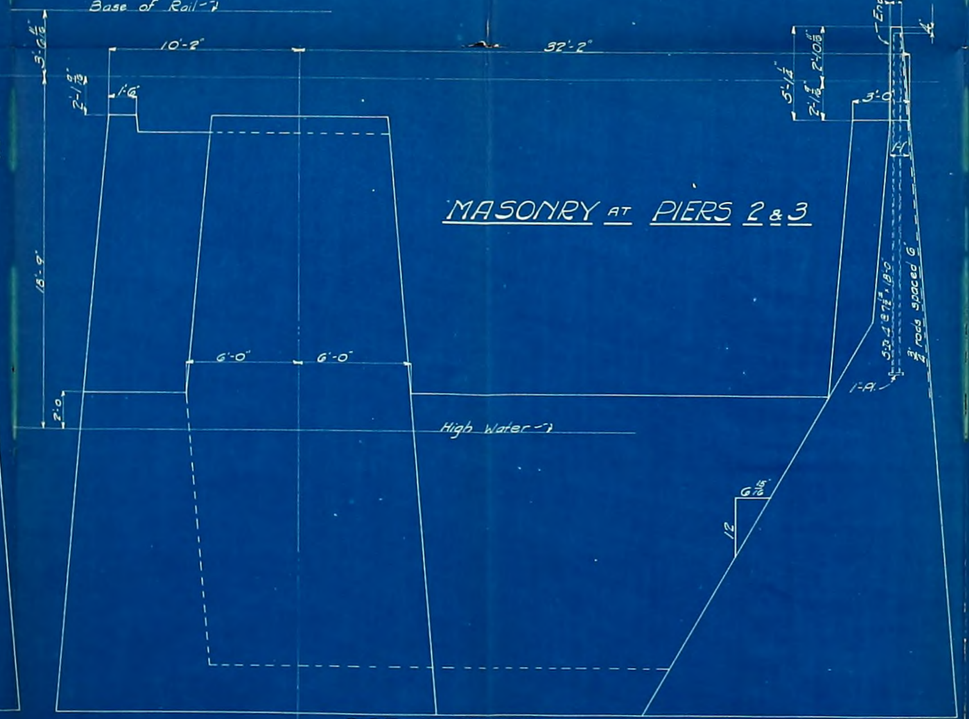


END ELEVATION

FRONT ELEVATION



FRONT ELEVATION



MASONRY AT PIERS 2 & 3

SIDE ELEVATION

NOTE:- All batters 1 in 12 except as shown

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OF
TECHNOLOGY

SCALE 1/4"=1' 4-24-'09
SHEET #5 H.C. Anderson

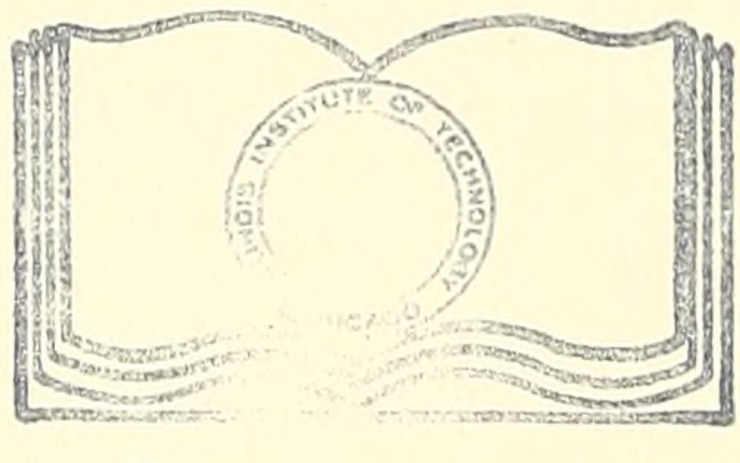
DOUBLE TRACK TRUNION BASCULE BRIDGE FOR ELECTRIC CARS

BY
P. M. LEICHENKO
E. C. HOLDEN
M. L. LOWENBERG

ARMOUR INSTITUTE OF TECHNOLOGY

1912

624
L 53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Leichenko, P. M.
Single leaf, double track
trunion bascule bridge for

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THESIS

A SINGLE LEAF, DOUBLE TRACK TRUNION BASCULE BRIDGE FOR INTERURBAN ELECTRIC CARS.

Presented by

P. M. Leichenko.
E. C. Holden
M. L. Lorenz

to the

PRESIDENT AND FACULTY

of the

ARMOUR INSTITUTE OF TECHNOLOGY

for the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

having completed the prescribed course

of study in

CIVIL ENGINEERING

1912.

Approved by Alfred E. Phillips
Prof. Civil Engineering
F. M. Raymond
Dean of Eng. Studies
L. C. Morrison
Dean of Coll. Studies.

624
L53

THESIS.

Design and General Drawings of a Single Leaf - Double Track Trunion Bascule Bridge for Interurban Cars.

Length of Span, 110 ft. from pin to free end.

Circular Segment, 35 ft. radius.

Clear Span, 75 ft.

5 Panels at 20 ft.

1 Panel at 10 ft.

Height of truss - at free end - 25 ft.

Height of truss - at pin end - 35 ft.

Width - 27 ft. c to c of trusses.

Sidewalks - 5 ft. wide.

Overall width - 39 ft.

LOADING.

Cooper's Class B. Suburban or Interurban Bridges.

For Trusses - Loads as per Table A.

Live Panel Load 48000 lbs.

SPECIFICATIONS.

American Railway Engineering & Maintenance of Way Association.

23478

STRINGERS.

The timbers and flooring were designed first and the weight per stringer calculated.

INTERMEDIATE STRINGERS.

Max. L.L. Bending Moment = 466200 in.lbs.

Max. D.L. " " = 53300

Total " " = 519.500 in.lbs.

$M/S = I/C = \frac{519500}{16000} = 32.3$

Use 12" @ 31.50# I-beams with I/C = 36

TROLLEY STRINGERS.

Max. L.L. Bending Moment = 810,000

" D.L. " " = 120,450

L.L. Impact = 760,000

Total = 1.690,450 inch.lbs.

D.L. End Shear = 2150

L.L. " " = 9000

Impact = 8450

Total=19600 #

Section used - 4 angles 4"x 4" x $\frac{1}{2}$"

1 web. plate 20" x 3/8"

FLOOR BEAM.

Max. D.L. Bending Moment = 1,225,000

" L.L. " " = 3,756,000

Impact = 3,440,200

Total = 8,421,200 inch.lbs.

FLOOR BEAM (cont.)

D.L. End Shear = 14280

L.L. " " = 45000

Impact = 41500

Total = 100780#

Section used 4 angles 6" x 6" x 13/16"

1 web plate 36" x 3/8"

Bridge Closed.

DEAD LOAD STRESSES - COEFFICIENTS.

The counter weight was considered as balancing the D.L. of the bridge to give zero reaction at the free end with bridge unloaded. A D.L. of 1 kip was assumed at each panel point and the D.L. stresses figured analytically, considering the moving leaf as a cantelever. These stresses were used as a table of coefficients when the correct D.L. was figured. See Table 1.

LIVE LOAD STRESSES - COEFFICIENTS.

The L.L. stresses were figured with a L.L. of 1 kip moving from the free end to the pin. The L.L. stresses were also figured with the L.L. moving from the pin to the free end. See Table 1.

THE ABUTMENTS.

The piers and the abutments were designed considering the weight of the bridge, water pressure, and earth pressure. The resultant of these forces fell within the middle third of bases.

TABLE 1.

Dead Load, Live Load, and Wind Load Coefficients for a Load of 1 Kip at Panel Points

Member	Bridge Closed: Dead Load	Bridge Closed: Live Load Moving from Free End L to e	Bridge Closed: Live Load Moving to Free End e to L	Bridge Closed: Wind Load in Chords	Bridge Open: Dead Load	Bridge Open: Wind Load	Bridge Open: # Final Weight
KL	+.640	-1.050	-2.910		+.166	±.65	1920
kL	-.400			±8.150	-.413	±.45	1400
Kk	-.500	+.818	+2.270		-.166	±.50	1000
HK	+.400				-.082	±.45	800
Hk	+1.930	+.233	-1.860		+.503	±2.00	1600
hk	-1.600			±7.800	-1.530	±1.70	1400
Hh	-1.500	-.182	+1.458		-.503	±1.50	1000
GH	+1.600				-.005	±1.70	800
gh	-3.600			±6.600	-2.620	±3.20	2588
Gg	-1.730	-.545	+.408		-.830	±3.70	1000
Fg	+3.300	+1.890	-.220		+1.280	±1.70	1775
gf	-5.500			±5.200	-4.000	±3.30	2588
FG	+3.700				+.240	±5.50	2185
Ff	-2.335	-1.330	+.155		-1.080	±3.80	1450
ef	-7.450			±3.260	-5.350	±2.30	3168
Ef	+3.890	+2.580	-.043		+1.530	±7.50	2340
Ee	+1.000	+1.000	+1.000		+.190	±3.80	1675
FE	+5.590				+.680	±1.00	2185
ec	-7.450				-6.010	±5.70	1584
Ec	-5.300	-2.87	-.955		-1.910	±7.50	3500
ED	+8.960				+1.660	±5.20	1648
Dc	-4.100	For chord			-.710	±9.00	1750
CD	+8.960	stresses			+1.330	±4.00	1990
Cc	-9.100	$\frac{W_1}{W} = \frac{48}{28}$			-.040	±9.00	1150
cb	-2.720	=1.714			+9.380	±3.00	1360
CB	+5.600				+1.420		1990
BA	+6.750				+2.318		1990
Cb	-6.500				-3.400		1680
Bb	-8.830				-4.580		1890
Ab	-1.47				+5.450		1360
Gh	+3.22	+.7	-.525		+1.000		1600
							54366

LATERAL SYSTEMS.

The stresses in the lower chord lateral bracing were figured from a W.L. per panel of 8800#. The train load - cars end to end - = 2400# per lin. ft. of one track. Lateral force on loaded chord = 200# per lin. ft.+ 10% of train load on one track

= 200 +240 = 440# per lin.ft. moving load

20 x 440 = 8800# W.P.L.

The stresses in the upper lateral bracing were figured from a moving load of 200# per lin.ft. W.P.L. = 4000#.

The lateral systems were figured as cantelever trusses.

Bridge Open.

DEAD LOAD STRESSES - COEFFICIENTS.

The bridge was considered raised at an angle of 79° with the horizontal and the D.L. stresses scaled graphically, using one kip at the panel points. The stresses were tabulated as coefficients.

WIND LOAD.

A wind load of 25# per sq.ft. was considered as acting perpendicular to the floor surface. The W.L. stresses from top of bridge perpendicular to floor

have the same coefficients as D.L. stresses for bridge closed, assuming W.L. of 1 kip per panel. W.L. Stresses, with wind from bottom of bridge perpendicular to floor, were scaled graphically. See Table 1, bridge open.

Approximate Stresses.

The D.L. per panel of the North Avenue Bridge and the Archer Avenue Bridge was obtained and used as a basis for assuming the D.L. per panel of this bridge.

A D.L. per panel of 19 kips was assumed and multiplied by the coefficients to obtain the approximate D.L. stresses. A table was made of the L.L., D.L., and W.L. Stresses with bridge open and closed, and the maximum + and − stresses figured for each member according to specifications for an alternating stress.

Next the members were designed and their total weight computed + 30% for details. The weight of flooring and laterals was added in and the total was found to be 27.5 kips, D.L. per panel.

Corrected Stresses.

The D.P.L. was assumed as 28 kips and Table 2 made of all the stresses, and the members were redesigned.

TOTAL WEIGHT OF BRIDGE.

Weight of Members in Table 1.

9.8 kips per panel - truss

17.5 " " " - floor load

0.7 " " " - laterals

28.0 " " " - D.L. Maximum

COUNTER WEIGHT.

The center of gravity of the bridge was located and the center of gravity of the counter weight was placed in line with it through the pin.

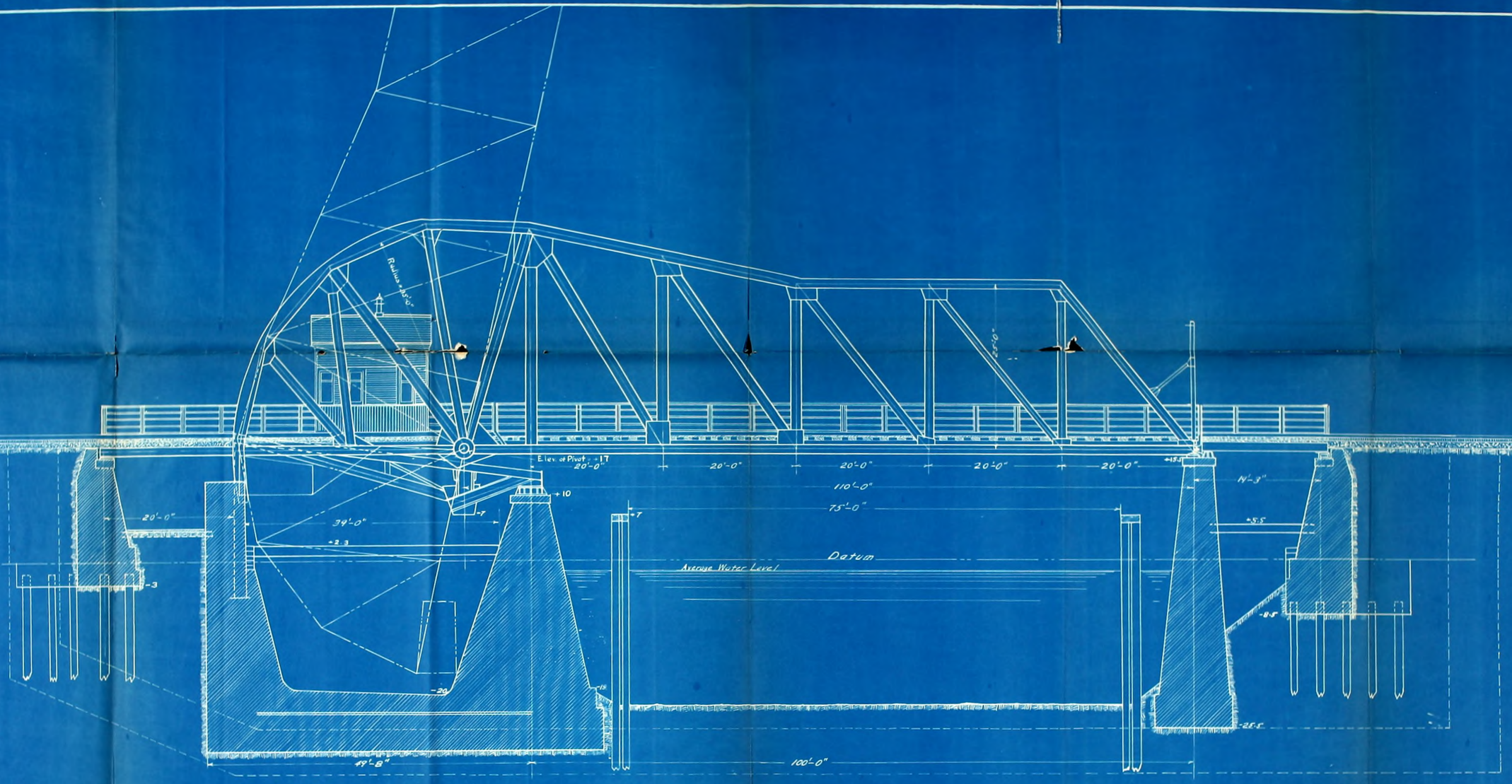
The sides of the counterweight box were designed as plate girders to carry the weight with bridge closed. The bottom of the box was designed as a plate girder to take the weight with bridge open.

PIVOT TRUSS.

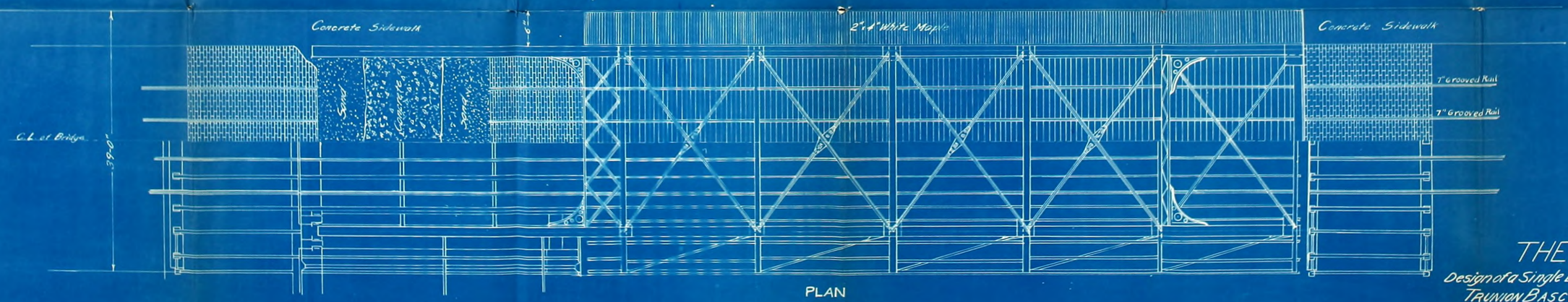
The pivot trusses were designed to carry the weight of the bridge to the abutments and to support the roadway as far as the movable leaf.

TABLE 2. Final Stresses

	Bridge Closed				Bridge Open			
Member	L.L.#	D.L.#	W.L.#	Total #	D.L.#	W.L.#	Total #	Maximum #
LK	+50400 -139,680	+17,800		+93400 -152,980	+4650	±6500	+11,150 -4,000	+93400 -152980
Hk	+11185 -89,280	+54,100		+70880 -58700	+14000	±20000	+34,000 -11,000	+70880 -58700
kL	-19200	-11,200	±81500	+81500 -111900	-11580	±4500	-16,080	+81500 -111900
Kk	+39260 ±108960	-14,000		-147900 +144270	-4650	±5000	-9,650	-14700 +99000
hk	-76,750	-44,700	±78000	-129250	-42800	±17000	-22000	-129250
HK	+19200	+11200		+30,400	-2290	±4500	-7000	-7000 +30400
GH	+76,750	+44,700		+121450	-140	±17000	±17000	-17000 +121460
Hh	-8700 +69984	-42,000		-55000 ±46300	-14000	±15000	-29000	-55000 +46300
gh	-173000	-101500	±66000	-340500	-73400	±37000	-110400	-340500
Gh	+33,600 -25,200	+90100		+123700	+2800	±32000	+60000	-32000 +123700
Gg	+19500 -26,160	-48500		-74660	-23200	±17000	-40200	-74660
fg	-263,800	-154000	±52000	-469800	-112000	±55000	-167000	-469800
Fg	+90820	+92500		+183320	+35800	±33000	+68800	-12000 +183320
FG	+178,000	+103500		+281500	+70000	±38000	+108000	+281500
Ff	+7,400 -63,840	-65,400		-129240	-30200	±23000	-53200	-129240
ef	-356,800	-204,000	±32600	-593400	-149500	±75000	-224500	-593400
Ef	+123,840	+109,000		+232840	+42800	±38000	+80800	+232840
EF	+268000	+156,000		+424000	+19000	±57000	+76000	-45000 +424000
Ee	+48000	+28,000		+76000	+5320	±10000	+15000	- +76000
ce	-356800	-204,000		-560800	-168000	±75000	-243000	-560800
DE		+250,500			+46500	±90000	+136500	-60000 +250500
CD		+250,500			+37300	±90000	+127300	-65000 +250500
BC		+156,500			+39800		+39800	+156500
BA		+189,000			+65000		+65,000	+189000
Ec	-137,760	-148,200		-285960	-53500	±52000	-105,500	-285960
Dc		-114,800			-19890	±40000	-59,890	-114800
Cc		-255,000			-1112	±30000	-31,112	-255000
Cb		-182,000			-95400		-95400	-182000
Bb		-247000			-128200		-128200	-247000
bc		-76200			+262000		+262000	-76200 +262000
Ab		-41200			+152500		+152500	-41200 +152500



LONGITUDINAL SECTION



PLAN

THESIS

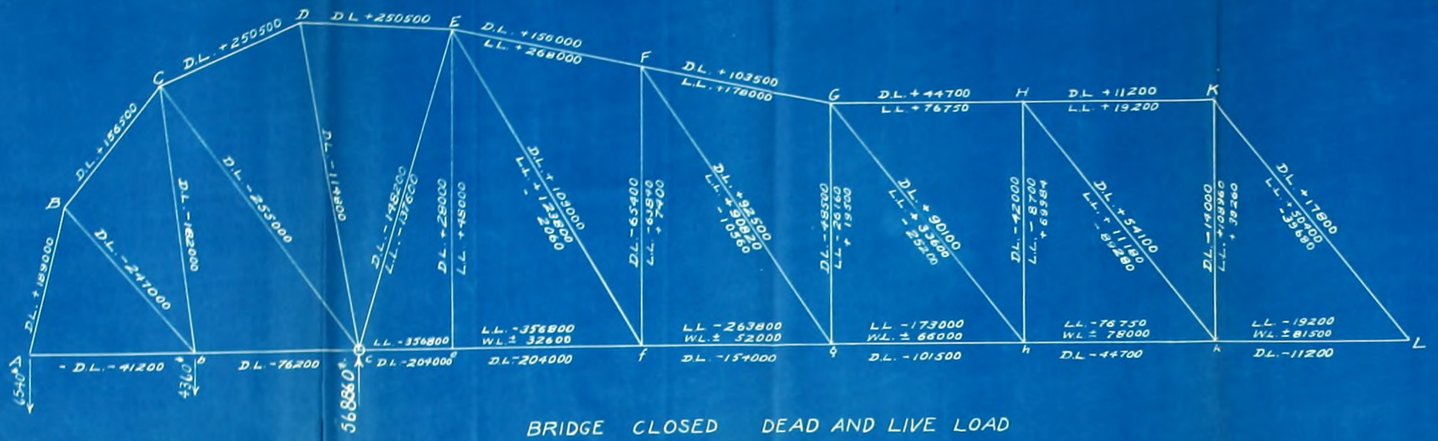
Design of a Single Leaf Double Track TRUNION BASCULE BRIDGE for Interurban Electric Ry. System

by P. Leichenko, E. Holden & M. Loewenberg.

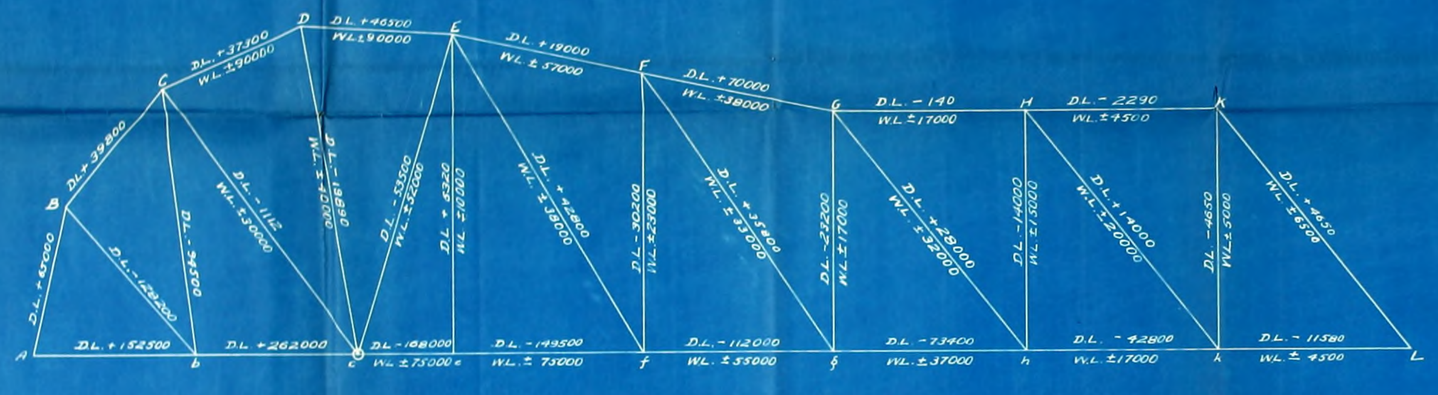
Plan & Elevation

(1) of (6) scale 1/8" = 1'

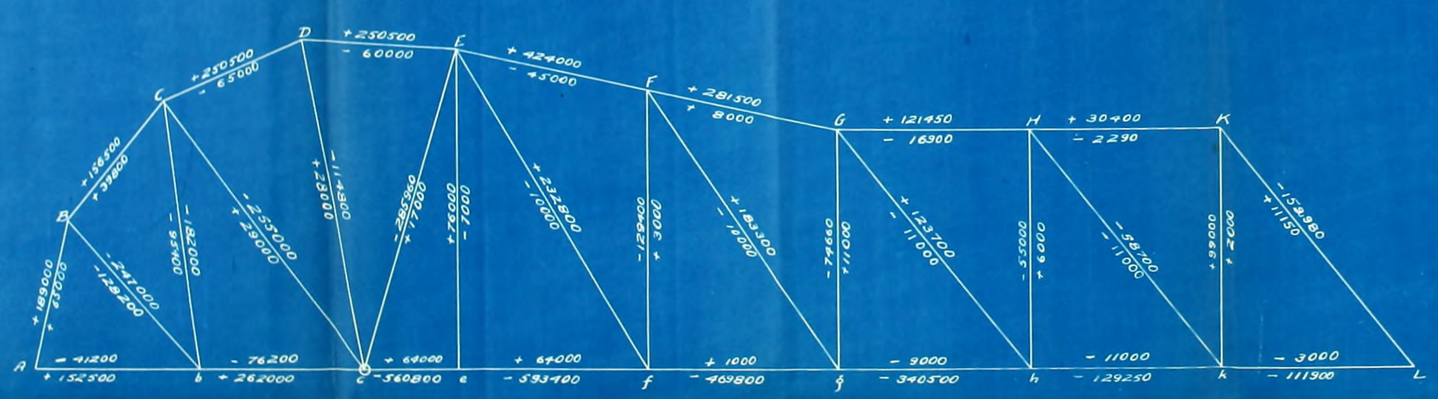
May 1912



BRIDGE CLOSED DEAD AND LIVE LOAD



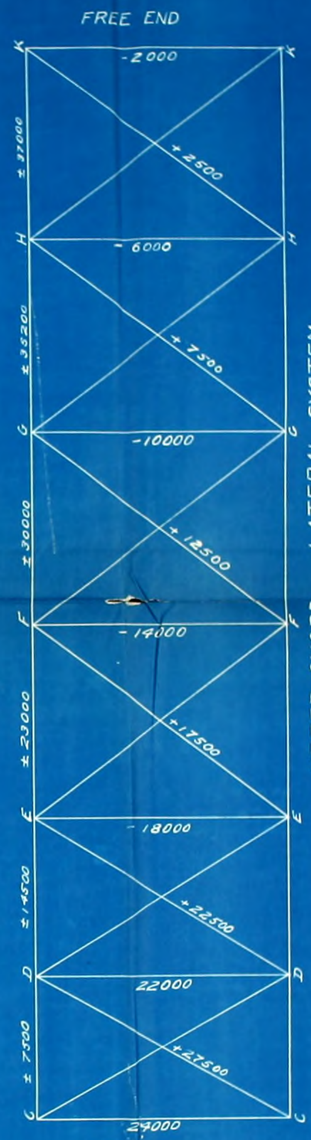
BRIDGE OPEN DEAD AND WIND LOAD



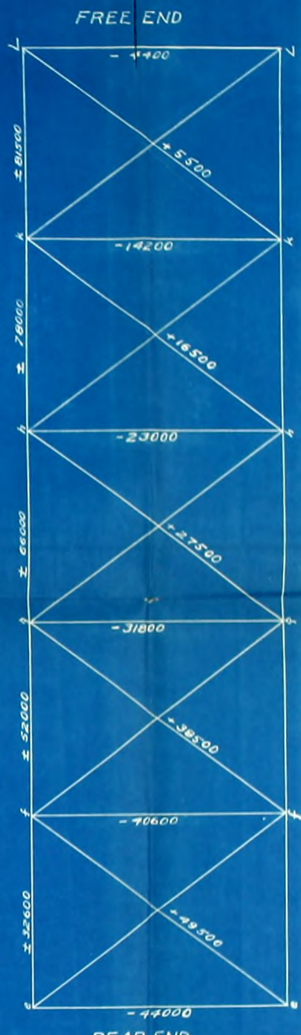
MAXIMUM AND MINIMUM STRESSES

Note ;-
+ Denotes Tension.
- Denotes Compression.

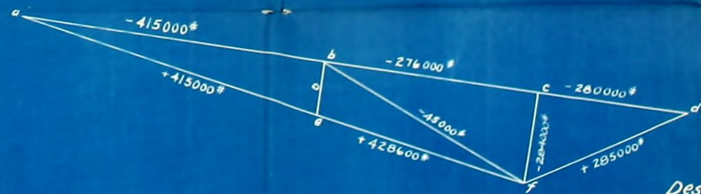
LIVE LOAD = 48000 # per Panel per Truss
DEAD " = 28000 # " " " "
Bottom Chord — WIND " = 8800 # " " " "
Top " " " = 4000 # " " " "



UPPER CHOOD LATERAL SYSTEM



LOWER CHORD LATERAL SYSTEM



PIVOT TRUSS

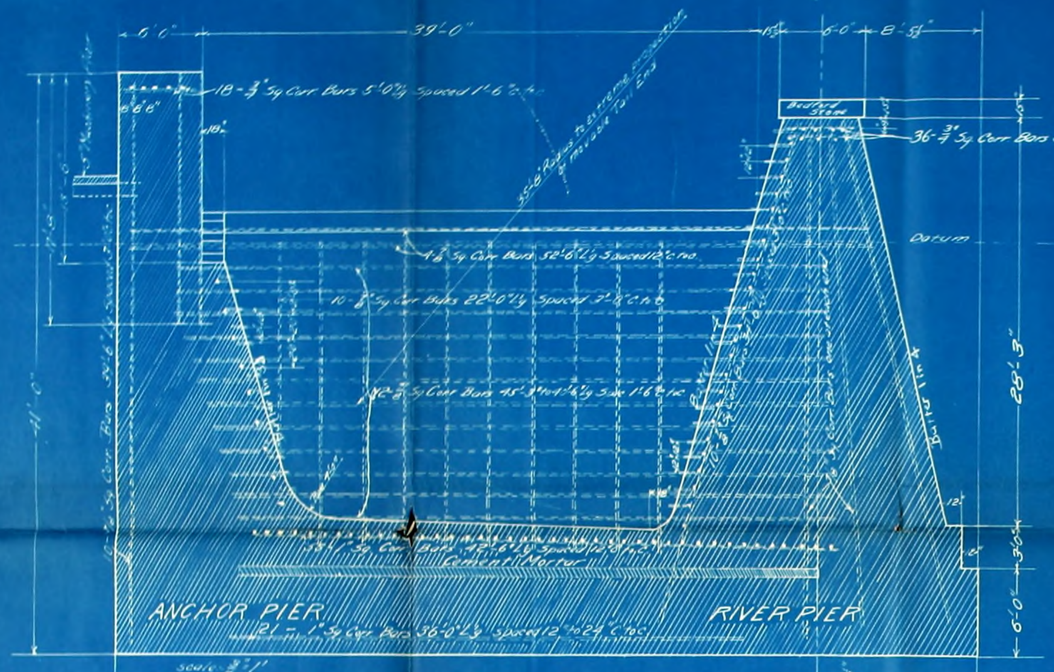
THESIS

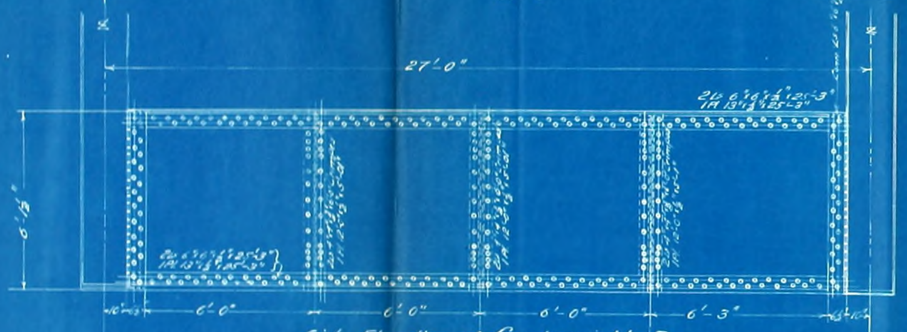
Design of a Single Leaf Double Track
TRUNION BASCULE BRIDGE
for Interurban Electric Ry. Service
by P. Leichenko, E. Holden & M. Loewenberg

Stress Sheet

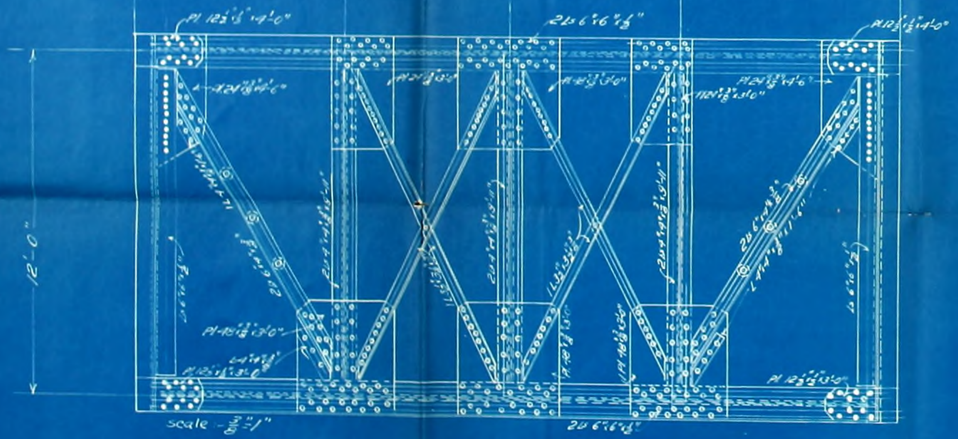
(2) of (6) Scale 1/8" = 1' #

May 1912



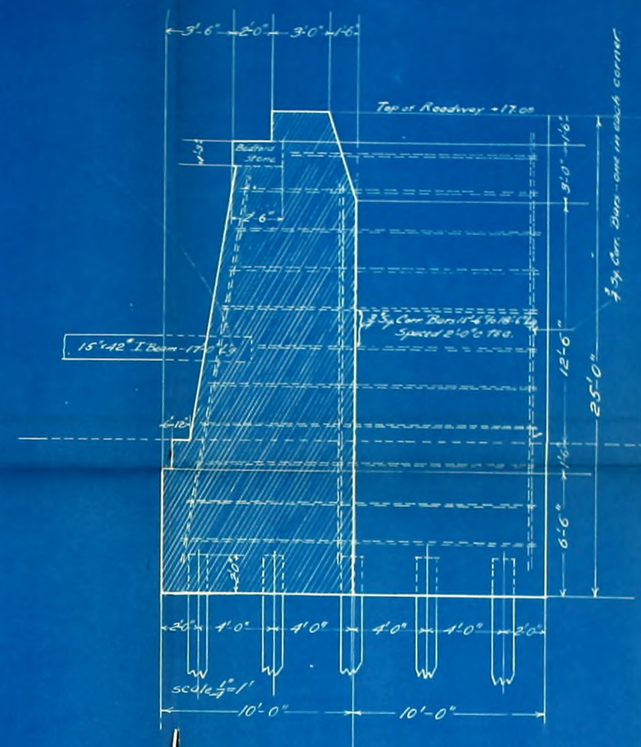


Side Elevation of Counterweight Box

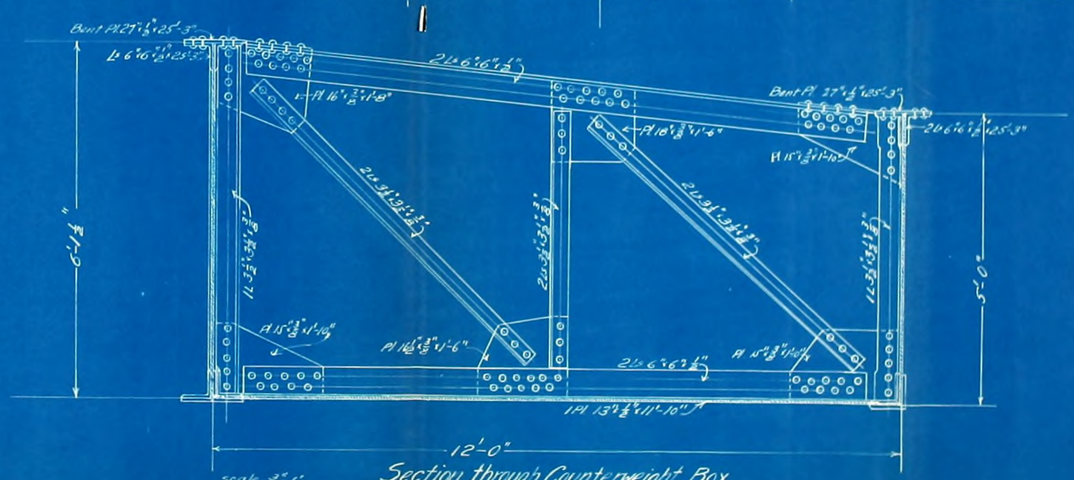


Top View of Counterweight Box

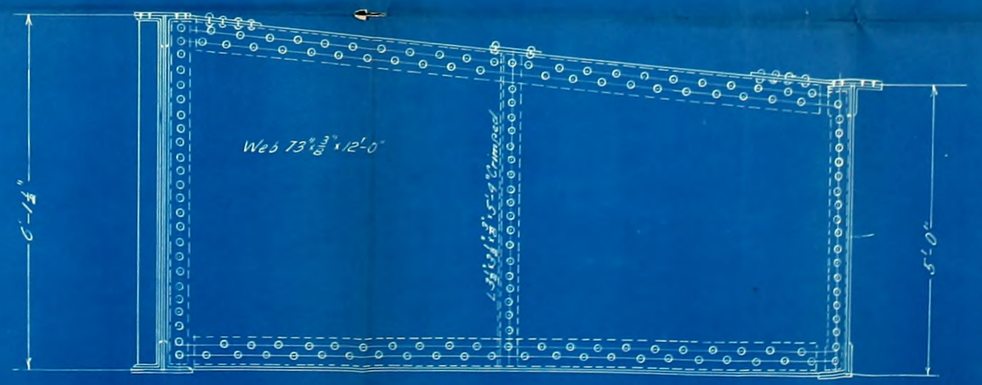
Section Through Abutment



AMOUNT OF COUNTERWEIGHT REQUIRED	
New Steel Rails	25000 lbs
Cast Iron Plates	10000 "
Cement Grout about	10000 "
1635 cub ft of any material weighing about 280 lbs per cub ft	440000 "
Total	500000 lbs



Section through Counterweight Box



End View of Counterweight Box

THESIS

Design of a Single Leaf Double Track
TRUNION BASCULE BRIDGE
for Interurban Electric Ry. System
by P. Leichenko, E. Holden & M. Loewenberg

PIERS & COUNTERWEIGHT BOX DETAILS
scale as indicated — May 1912

1 Pl 19"X22¼"X½"
1 Pl 18"X18½"X½"
2 Pls 2'-0"X18½"X½"
1 Pl 18"X18½"X½"
1 L 4"X4"X½" 22'-0"
Single Lacing 2½"X½"X1'-10"
1 L 4"X4"X½" 10'-10"
20'-0"
20'-0"
20'-0"
6'-3"
4 Ls 4"X4"X½"
2 Pls 13"X⅜"
1 Pl 19"X22½"X½"
2 Ls 4"X4"X½"
2 Ls 6"X6"X⅜"
Wedge Filler
1 Pl 18"X18½"X½"
1 Pl 6'-3"X4'-2"X⅜"
2 Ls 10"@20"
Tie Pl 2'-3"X18½"X⅜"
1 Pl 2'-5"X2'-5"X⅜"
1 Pl 2'-0"X18½"X½"
2 Ls 4"X4"X½"
2 Ls 6"X6"X⅜"
1 Pl 2'-0"X18½"X½"
18"
6'-11"
2 Ls 4"X4"X½"
2 Ls 4"X4"X½"
⅜" Pl.
2 Ls 3½"X13½"X⅜"
2 L 4"X4"X½"
1 Bent Pl 1'-11"X18½"X⅜"
2 Ls 4"X4"X⅜"
1 Pl 18"X18½"X½"
2 Ls 10"@20"
1 Pl 3'-9"X2'-6"X⅜"
1 Bent Pl 15"X18½"X½"
1 Pl 14"X3'-1"X⅜"
All Tie Plates on Posts 12"X11¾"X⅜"
1 Fill Pl 6½"X⅜"-2'-9"
2 Ls 10"@20"
Lace bars 2½"X½"X13"
All Tie Plates on Diagonals 13"X19"X⅜"
2 Ls 12"@25"
Lace bars 2½"X½"X1'-10"
25'-0"
2 Ls 10"@20"
Lace bars 2½"X⅜"X13"
2 Ls 12"X25"
Lace bars 2½"X½"X1'-10"
2 Ls 12"@25"
Lace bars 2½"X½"X1'-10"
1 Pl 2'-9"X3'-0"X⅜"
1 Pl 2'-3"X21¾"X⅜"
1 Pl 2'-3"X21¾"X⅜"
4 Ls 6"X6"X½"
2 Pls 12"X⅜"
1 Pl 2'-3"X21¾"X⅜"
1 Pl 3'-9"X3'-3"X⅜"
1 Pl 2'-3"X21¾"X⅜"
4 Ls 4"X4"X⅜"
2 Pls 12"X⅜"
1 Pl 3'-1"X3'-1"X⅜"
1 Pl 2'-3"X21¾"X⅜"
1 Pl 4'-9"X4'-1"X½"
1 Pl 4'-9"X4'-1"X½"
2 Side Pls 3'-9"X11½"X⅜"
2 Fill. Pls 4½"X⅜"X2'-0"
2 Fill. Pls 11½"X⅜"X2'-0"
1 Pl 4'-9"X4'-1"X½"
20'-0"
20'-0"
20'-0"
2'-2½"
Diaphragms
4 Ls 4"X4"X⅜"
1 Pl. 11"X⅜"X3'-3"
1 Pl 4'-9"X4'-1"X½"
All Laterals
1 L 6"X6"X⅜"
1 Lug L 6"X6"X⅜"
Lacing Bars 2½"X½"X2'-7"
All Laterals
1 L 6"X6"X⅜"
1 Lug L 6"X6"X⅜"
1 Pl 4'-9"X4'-1"X½"
Lacing Bars 2½"X½"X2'-2"

1 Pl 18"x18½"x½"

Single Lacing 2½"x½"x1'-10"

1 L 4"x4"x½" 10'-10"

1 L 4"x4"x½" 22'-0"

20'-0"

6'-3"

18½"

2 L, 4"x4"x½"

2 L, 4"x4"x½"

2 L, 6"x6"x⅞"

1 Pl 2'-6"x18½"x½"

2 L, 4"x4"x½"

6'-11"

⅜" Pl.

2 L, 3½"x13½"x⅜"

2 L, 4"x4"x½"

1 Bent Pl 1'-11"x18½"x⅜"

2 L, 4"x4"x⅜"

1 Pl 18"x18½"x½"

2 L, 10"@20#

1 Pl 3'-9"x2'-6"x⅝"

1 Bent Pl 15"x18½"x½"

1 Bent Pl 2'-7"x18½"x½"

Tie Pls 2'-3"x19"x⅜"

2 L, 4"x4"x½"

2 L, 4"x4"x½"

2 L, 3½"x13½"x⅜"

⅜" Pl

2 L, 4"x4"x½"

2 L, 10"@20"
Lace bars 2½"x⅜"x13"

2 L, 12"x25#
Lace bars 2½"x½"x1'-10"

2 L, 10"@20"
Lace bars 2½"x⅜"x13"

2 L, 12"@30#
Lace bars 2½"x½" 1'-10"

3'-0"

5¾" 10⅜" 10⅜" 5"

2'-4"

10½" 9½" 9½" 10½"

6"

11"

HOLES ½" DIAM.

BOLTS 1½" DIAM.

1'-11"

12"

CORED HOLES
5" DIAM.

CASTING AT FREE END

13'-6"

END PORTAL

2 L, 4"x4"x½"

2 L, 4"x4"x½"

Bars 3"x⅝"

⅜" Pl

2 L, 4"x4"x½"

11"

6'-0"

13'-6"

2 L, 4"x4"x½"

2 L 6"x6"x⅞"

1'-11½"

12½"

1 Pl 3'-9"x3'-3"x⅝"

1 Pl 2'-3"x21¾"x⅜"

4 L, 4"x4"x⅜"
2 Pl, 12"x⅜"

1 Pl 3'-1"x3'-1"x⅝"

1 Pl 2'-3"x21¾"x⅜"

1 Pl 2'-3"x21¾"x⅜"

4 L, 4"x4"x⅜"
2 Pl, 12"x⅜"

1 Pl 2'-3½"x13'-0"x⅜"

1 Pl 2'-3"x21¾"x⅜"

2 Side Pls 3'-9"x11½"x⅜"
2 Fill. Pls 4½"x⅜"x2'-0"
2 Fill. Pls 11½"x⅜"x2'-0"

1 Pl 4'-9"x4'-1"x½"

20'-0"

20'-0"

1 Pl 3'-5"x4'-0"x½"

1 Pl 2'-4"x1'-9¾"x⅜"

End Diaphragm
4 L, 4"x4"x½"
1 Pl 11¾"x⅜"x12"

13"

1'-9¾"

1 Pl 4'-9"x4'-1"x½"

Lacing Bars 2½"x½"x2'-2"

1 Pl 4'-9"x4'-1"x½"

Lacing Bars 2½"x½"x2'-2"

1 Pl 3'-5"x4'-0"x½"

THESIS

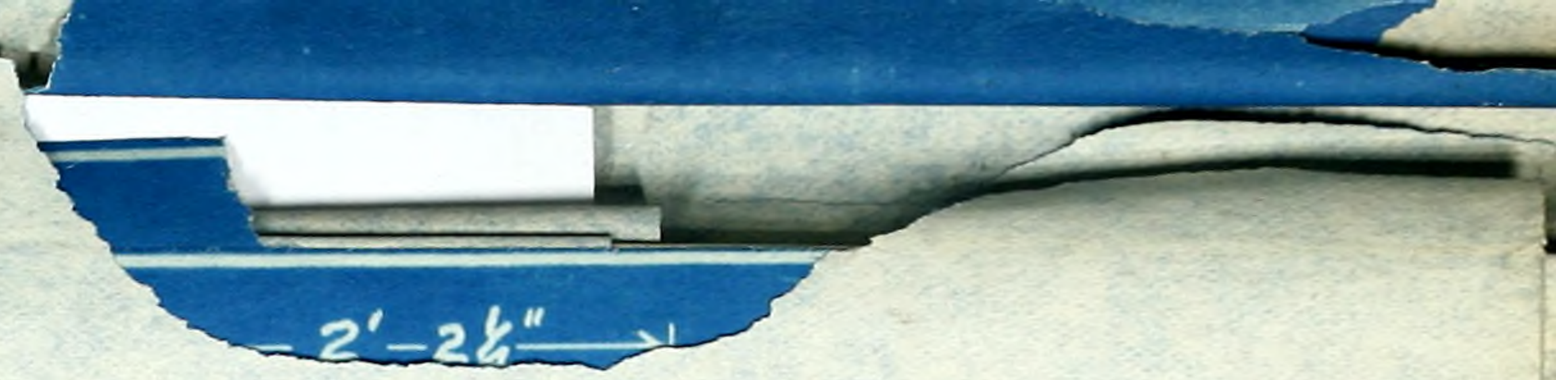
Design of a Single Leaf Double Track
Trunion Bascule Bridge
for Interurban Electric Ry. Service
by P. Leichenko, E. Holden & M. Loewenberg

General Drawing

Scale Outline ½"-1' Details ¾"-1"

May 1912

4 of 6


2'-2½"

Radius to Back of Angles - 35'-8"
Radius to Pitch Circle - 35'-0"

Section A-A

Section B-B

2 Bent Angles — 51 Spaces @ 11" = 46'-9" Measured on Pitch Circle

2 Bars - 4"x⅝" Inside Lock Bars - 3"x⅜"x3'-5"

2 Bars - 4"x⅝" Inside Lock Plates - 7½"x⅜"x21"

2 splices - 3½"x3½"x⅜" Outside Milled Joint

2 splices - 3½"x3½"x⅜" Outside Milled Joint

Ctrs. 2'-0"
2 Pls - 22½"x⅜"
2 L - 4"x4"x⅜"

Ctrs. 1'-6"
2 Pls - 22½"x⅜"
2 L - 4"x4"x⅜"

2 Pls - 3'-0"x⅜"x5'-0"

Lacing Bars 3"x⅜"x24"

Tie Pls. 18"x⅜"x20"

Lacing Bars - 3"x⅜"x22"

20-9" Ctrs.

2-B-16" @ 45"-13'-6"

28-9" Ctrs

2-B-12" @ 30# 13'-6"

Radius of Pitch Circle - 35'-0"

35'-0" Ctrs.

2-B-15" @ 45# - 13'-6"

2-B-12" @ 35# - 13'-6"

Lacing Bars - 3"x⅜"x22"

Tie Pls. 18"x⅜"x20"

Tie Pls. 18"x⅜"x21'-0"

2 Pls. - 21'-0"x⅜"x10'-6"

Batter Line Shows Outline of Deep Truss

Lacing Bars - ⅜"x2½"

2 Pls. - 12"x⅜"

2 L - 6"x6"x⅝"

2 L - 6"x4"x⅝"

5'-0"

12'-0"

17'-6"

35'-0"

7'-3"

6'-1½"

13½"

5'-8"

13'-0"

1" Tap Bolts ⅜" Lock Plates

4 Lock Bars

⅜" Extra Shim Plate Separator

Lacing Bars - ⅜"x2½"x24"

Tie Pls. - 18"x⅜"x20"

Lacing Bars - ⅜"x2½"x22"

Tie Pls. - 18"x⅜"x20"

2 Pls. - 21'-0"x⅜"x10'-0"

2 Bars - 4"x⅝" Inside

2 splices - 3½"x3½"x⅜" Outside Milled Joint

2 Pls. - 21'-0"x⅜"x10'-0"

Lacing Bars - ⅜"x2½"x24"

Tie Pls. - 18"x⅜"x20"

Lacing Bars - ⅜"x2½"x22"

Tie Pls. - 18"x⅜"x20"

Tie Pls. - 18"x⅜"x21'-0"

2 Pls. - 21'-0"x⅜"x10'-0"

Lacing Bars - ⅜"x2½"x24"

Tie Pls. - 18"x⅜"x20"

Lacing Bars - ⅜"x2½"x22"

Tie Pls. - 18"x⅜"x20"

Tie Pls. - 18"x⅜"x21'-0"

2 Pls. - 21'-0"x⅜"x10'-0"

Deflection Plate

2'-6"

THESIS

Design of a Single Leaf Double Track
TRUNNION BASCULE BRIDGE
for Interurban Electric Ry. System
by P. Levchenko, E. Holden & M. Loewenberg

General Drawing May 1912

(5) or (6)

scale 1" = 2'-0"

NL

Posts 4"X4"X4'-0" Spac. 6'-0" c.to c.
Railing 2" Pipe Spac. 9" Top-2"X4"X6'0"
2¾" PLANKS 6" WIDE
NAILING STRIP 5"X3½"X20'-0"
2 L's 6"X6"X½"
I BEAMS 12"@31.5#
1 Bent Plate 6'-0"X12⅜"X½"
2 L's 4"X4"X½"
Fill Pl. 6"X5½"X½"
Fill Pl. 6"X18"X 13/16"
Planking 2¾" X 10"
7" Trolley Rails
Diagonal Timbers 4½" X 6" Spaced 30" c.to c.
FLOORBEAM WEB Pl 36"X⅜"
FLANGES 2 L's 6"X6"X 13/16"
1 Pl 4'-8"X4'-1"X½"
TYPICAL CROSS SECTION
Scale ¾"=1'
1'-0" 3'-6" 1'-6" 1'-3½" 2'-3" 2'-3" 2'-4¼" 2'-4¼" 3'-0" 3'-0" 2'-4¼" 2'
6'-0" 27'-0"

2 L's 6"X6"X1⅛"X19'-11"
2 Fill Pl's 12"X 12"X½"
TROLLEY STRINGER
WEB 20"X⅜"
FLANGE 2 L's 4"X4"X½"
19'-11⅝"

20'-0" 14'-0" 44'-0"
Top of Rail
Top Line of Floor beams
Diaphragm
Web Pl's 16½"X⅜"
4 L's 3½"X3½"X½"
2 L's 15" @ 50#
2 Pl's 8'-6"X 3'-6"X½"
1 Pl 20"X19½"X½"
Tie Plates 15"X2'-0"
4 L's 3½"X3½"X½"
2 Pl's 20"X½"
2 Pl's 2'-6"X1'-10"X½"
4 L's 3½"X3½"X⅜" Laced
2 L's 15" @ 50#
Lattice bars 5"X 7/16"-2'-4"
1 Cover Pl 25"X½"
Diaphragm ½" Pl
4 L's 3½"X3½"X⅜"
2 Web Pl's 22"X⅜"
4 L's 3½"X3½"X⅜"
1 Pl 20"X19½"X½"
Single Lacing 2½"X½"-18"
1 Pl 20"X19½"X½"
2 Pl's 7'-4"
Scale ½"=1'

INTERMEDIATE STRINGER 12" I @31.5#
9'-11 13/16"

PIVOT TRUSS

25"
11½"

FLOORBEAM "A"

FLOORBEAM AT SHORT PANEL "A"
1 WEB PL 36"X⅜"
1 L 6"X6"X 13/16"
FLANGE 2 L's 6"X6"X 13/16"
WEB 36"X⅜"
Scale ¾"=1'

1 Pl 8⅜"X½"X23"
2 L's 4"X4"X½"
FLOOR BEAM
1 Fill Pl 8⅜"X13/16"X19"
1 Pl 23"X13½"X½"
2 L's 4"X4"X½"
TROLLEY STR
BRACKET AT CUT

TROLLEY STRINGER WEB 20"x⅜" FLANGE 2L 4x4x½"

19'-11⅝"

20½"

25'-0"

6"

11¾"

1'-5½"

" Trolley Rails

Diagonal Timbers 4½"X6" Spaced 30" c.to c.

TYPICAL CROSS SECTION
Scale ¾"=1'

3'-0" 3'-0" 2'-4¼" 2'-4¼" 2'-3" 2'-3" 1'-3½" 1'-6" 3'-6" 1'-0"

27'-0"

6'-0"

Diaphragm Pl, 11"x⅜"x 3'-3"
4L 4x4x½"

44'-0"

14'-0" 5'-6" 4'-6"

Top of Rail

Top Line of Floor beams

2'-0"

Pin, 14½" Diam.

12

½

1L 3½"x3½"x½"

Cover Pl 6"x14'-5½"

Diaphragm ½" Pl
4L 3½"x3½"x⅜"
2 Webs 16½"x½"
4L 3½"x3½"x½"

1L 3½"x3½"x½"

End Pl 2'-6"x1'-1½"
1L 3½"x3½"x⅜"

2L 15"@ 50" Lattice bars 5"x⅜"-2'-4"
1 Cover Pl 25"x½"

2Pl, 2'-6"x1'-10"x½"

4L 3½"x3½"x⅜" Laced

Diaphragm ½" Pl
4L 3½"x3½"x⅜"
2 Web Pl 22"x½"
4L 3½"x3½"x⅜"

2Pl, 6'-6"x3'-0"x½"

Tie Plates 25"x20"x½"

1Pl 20"x1'-½"x½"

4L 3½"x3½"x½"
2 Pl 15"x½"

2'-6"

1'-6"

CORED HOLES 6" DIAM

1" Mortar

6'-3"

2 Pl 7'-4"x3'-10"x½"

Scale ½"=1'

PIVOT TRUSS

8'-0"

10" 4@ 8½" 10"

9'-5"

¾" ¾" ¾" 11" 11" 11" 11" 11"

18" 17" 2'-2" 17" 18"

HOLES FOR BOLTS 1½" DIAM. DRIVING FIT

1½"

½"

Tie Plates 25"x20"x½"

2'-0"

19"

2L 6"x6"x⅜"

END FLOORBEAM CONNECTION
Scale ¾"=1'

ANGLE 2L 6"x6"x⅜"
PL 36"x⅜"

FLOOR BEAM "A"
1 Fill Pl 8⅜"x1'-3⅜"x19"

1Pl 8⅜"x½"x23"
2L 4x4x½

1Pl 23"x13⅜"x½"

2L 4"x4"x½"

TROLLEY STRINGER
BRACKET AT CUT IN FLOOR

FLOORBEAM "A"
(1 FILL. Pl 8⅜"x1'-3⅜"x19"

1Pl 8⅜"x½"x23"
2L 4"x4"x½"

1Pl 23"x13⅜"x½"

2L 4"x4"x½"

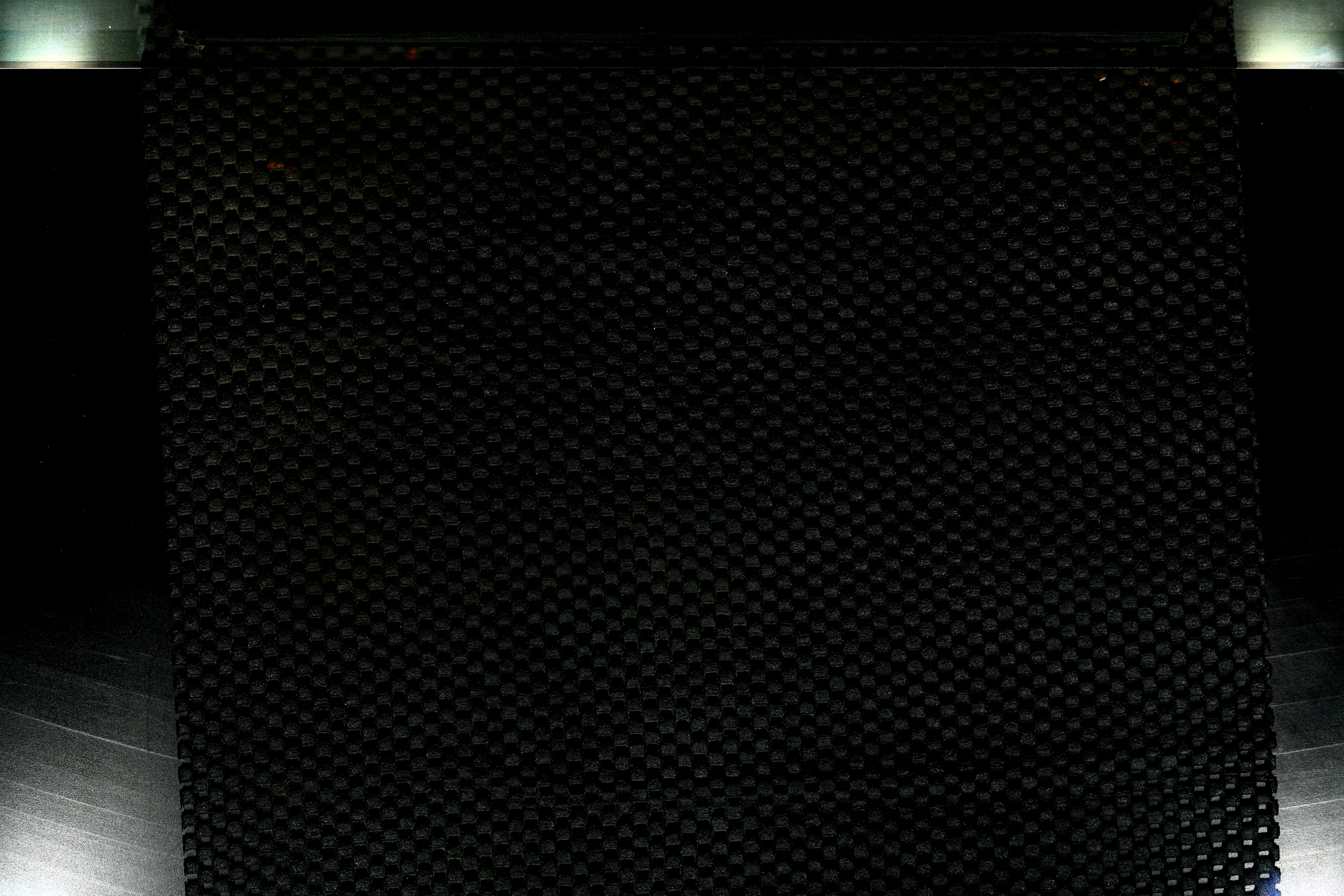
INTERMEDIATE STRINGER
BRACKET AT CUT IN FLOOR

THESIS

Design of a Single Leaf Double Track
TRUNION BASCULE BRIDGE
for Interurban Electric Ry. Service
by P. Leichenko, E. Holden & M. Loewenberg

Pivot Truss & Details
Scale ¾"-1', ½"=1'
May 1912

6 of 6



DESIGN OF A TRUNNION BASCULE BRIDGE

BY
A. E. BREDLAU
P. R. HYNES
J. B. JOHNSON

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bascule bridge

THE DESIGN OF A TRUNNION BASCULE BRIDGE.

A THESIS PRESENTED BY

P. R. Hyne

John B. Johnson.

to the

PRESIDENT and FACULTY

OF THE

ARMOUR INSTITUTE OF TECHNOLOGY

for the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

HAVING COMPLETED THE PRESCRIBED COURSE OF STUDY

IN

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DESIGN OF A TRUNNION BASCULE BRIDGE

for the B. & O. TERMINAL R. R.

SPAN -- 100'- 0" RISE -- 30'- 0"

SINGLE SPAN --- SINGLE TRACK

Distance center to center of Trusses, 17'- 0".

COOPER E-50 LOADING

22765

SS: C2

DESIGN OF A TRUNNION BASCULE BRIDGE.

The subject of bascule bridges is an important one, inasmuch as at present, the old style draw span is becoming obsolete and is being replaced by the bascule bridge.

There are many good reasons for this change, the bascule bridge lacking the disadvantages of the draw span of which the chief objection is the necessity of a center pier, which in a river of heavy traffic and of no great width, as the Chicago River, leaves a minimum of clearance for the passage of boats. There is constant danger of a vessel striking the pier, causing injury to the drum and rollers, which are very expensive to repair. The draw span has all the faults of the partially continous and continuous trusses as are given in Merriman and Jacoby's "Roofs and Bridges, Part IV."

The motion of a draw span is slow, much valuable time being lost to the traffic while the bridge is swinging. Also when the draw span is open, it must be protected throughout its length by protection piers, causing a great expense in piles and timber.

There is some discussion as to whether the protection piers so erected are at all efficient in preventing vessels from striking the pier. The piles are usually driven only five or ten feet in to the river bottom, rising through possibly twenty five feet of water and leaving about five feet exposed. This piling is not braced in any way transversely

so they act as simple cantilevers and cannot stand a very great impact.

The bascule bridge on the other hand allows a maximum clearance between the abutments, the stresses are simple to analyze, and as it is erected vertically, the rerection stresses are low. The bridge can be opened and closed much more quickly than the draw span and only the width of the pier is protected by piling. The method of computing stresses is more certain.

There are several types of bascule bridges, the most common types of which are the "Scherzer Rolling Lift," the Strauss bascule bridge, and the ordinary trunnion bascule bridge.

In the ordinary trunnion bridge, the truss rotates on a trunnion situated at the end of what would be the simple span. The counter weight is attached to that part of the truss extending beyond the pin. When the bridge is open, it must be at an angle of 79° with the horizontal, this usually allows the extreme end of the bridge to fall within the vertical plane of the protection piers. The lever arm of the counterweight must be such that, when the bridge rotates, it will not necessitate too deep or too long a pit.

In the Scherzer bridge, the counterweights are placed above the top chord of the bridge, and the bridge rolls on a segment of a circle instead of rotating on a trunnion. This obviates the necessity of having a pit, but the chief difficulty is to sufficiently stiffen the segment to stand the weight of

the truss. In the Strauss type of bridge, the trunnion is located at the intersection of the top chord and the end post, the counterweight being attached by a link motion so that it moves vertically, the elevation of the center of gravity of the total bridge being fixed.

For the conditions at hand, we took a single leaf span of 100'- 0" from the center of pin to the center of the abutment, and the lever arm of the counterweight was taken at 30'- 0". It was necessary that a line connecting the center of gravity of the truss, and the center of gravity of the counterweight should pass through the center of the trunnion, and then if the moment of the weight equals the moment of the truss, the structure will remain in equilibrium in any position, the power required to operate the bridge being necessary to overcome the friction ofthe moving parts.

* * * * * * * * *

DESIGN OF TRUNNION BRIDGE

Span, c. to c. of bearings - - - - 100'- 0"

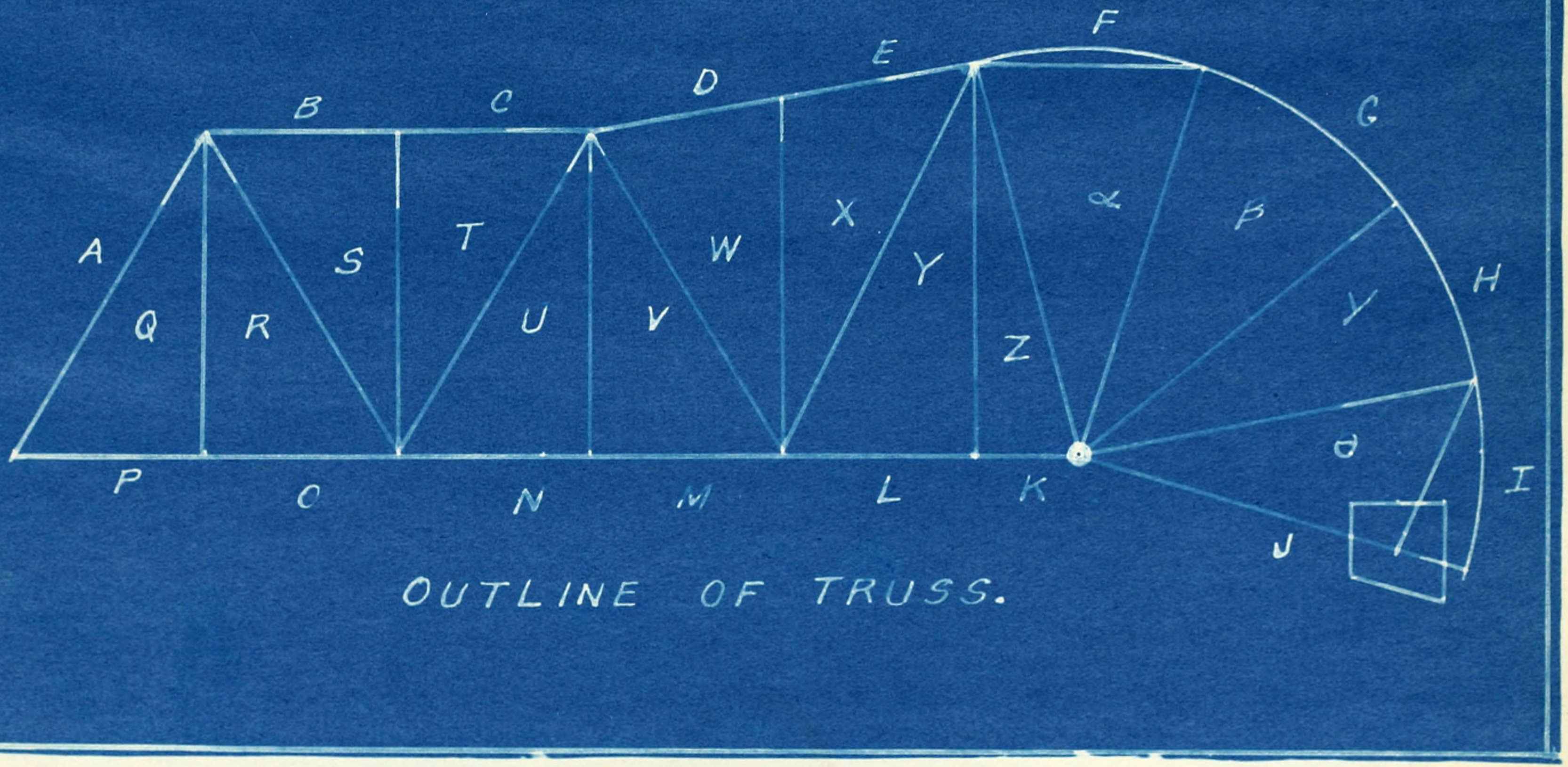
5 panels @ 18'- 0" and one panel at 10'- 0"

Length of counterweight lever arm - - - 30'- 0"

Rise - - - - - - - - - - - - - 30'- 0"

DESIGN OF STRINGER

Total maximum bending moment - - 5, 576,000"/#



OUTLINE OF TRUSS.

Total Shear - - - - - - - 117,115#

Members used

24"x1/2" web plate

2 - 6"x 8"x 3/4" angles @ 33.9#

6"x 8"x 5/8" angles for end connections

DESIGN OF FLOOR-BEAM

Total Bending Moment - - - - - 10, 556,500"#

Total Shear - - - - - - - - - 195,300#

Web - 48"x 1/2" Plate

Angles 6"x 8"x 3/4" for flanges

End Connection Angles, 6"x 8"x 5/8"

Rivets required - - - - - - - 33

In computing the stresses, the bridge was considered as acting as a simple truss when closed and as a cantilever when open. The following were the cases considered, dead load stresses with the bridge horizontal and at angles of 30°, 60°, and 79° with the horizontal; live load stresses with the bridge down; wind stresses (against flooring) bridge at angles of 30°, 60°, and 79° and the wind stresses for the side of the truss.

These stresses were figured analytically by taking moments and resolutions of forces and were checked graphically. The maximum and minimum stresses were computed and final

stresses obtained and the members designed. The preliminary panel load was assumed at 13,500 Lbs. The members being designed, the exact weight of the bridge was computed, and the new dead load panel load obtained after which the true dead load stresses in the members were computed and tabulated. These were then combined with the wind and live load stresses, and the final maximum and minimum stresses were obtained.

Following is the computations showing the exact weight of the bridge ---

Member	AQ	2-15"	@55#	2 x 55 x 35 =	3,850#
		1 Pl.	3/4"x 16.64 x 35 x 480 ÷ 144 =		1,500#
					3,850#
	Za		same		
					1,500#
	RQ	2 - 10"	@ 30#	2 x 30 x 30 =	1,800#
	YZ	"	"	36 x 34 x 30 =	2,180#
	UV	"	"		1,800#
	ST	"	@ 15#	2 x 15 x 30 =	900#
	WX	"	"	2 x 15 x 33.25	998#
	RS	2 - 12"	@ 35#	2 x 35 x 34.99	2,449#
	TU	"	@ 30#	2 x 30 x 34.99	2,100#
	VW	"	"	"	2,100#
	XY	"	@ 35#	2 x 35 x 40.75	2,852#
	PQ	2 - 15"	@ 33#	2 x 33 x 36	2,376#
	RD	"	"	"	2,376#
	UN	"	@ 40#	2 x 40 x 36	2,880#
	VM	"	"	"	2,880#

Member	YL	2 - 15"	@ 35#	2 x 35 x 28 =	1960#
	ZK	"	"	"	1960#
	BS	"	@ 33#	2 x 33 x 36 =	2376#
	TC	"	"	"	2376#
	WD	"	"	2 x 33 x 37.3 =	2484#
	EX	"	"	"	2484#

Increase weight 5% for rivets.

WEIGHT OF STRINGERS

Web = 24"x 1/2"x (18/144) x 480 =		720#
2 - 6"x 8"x 3/4" Angles	2 x 33.9 x 18 =	1,440#
5 % rivets		108#
Total		2,268#

WEIGHT OF THE FLOOR BEAMS

Web = 48 x 1/2 x (17/144) x 480 =		1,360#
2 - 6"x 6" x 5/8" Angles	2 x 28.5 x 17 =	969#
5 % rivets		117#
Total		2,446#

Weight of Truss - - - - - -	39,450#
Weight of Floor Beams per Truss - -	20,098#
Weight of Floor per Truss - - - -	28,500#
Weight of Portal and Bracing - - -	3,000#
Total weight of Truss	91,048#

This gives a value of 910.5# per linear foot of Truss.

True panel load = 910.5 x 18 = 16,389#

DESIGN OF COUNTER WEIGHT

Total weight of the bridge is 182,100# and taking moments about the hinge, we have the equation that

$$182{,}100 \times 50 = X \times 30 - 35 \times 89 \times 17.5$$

$$X = 267{,}163 \text{ kips}?$$

Assuming the weight of cast iron as 350# per cubic foot, the number of cubic feet required will be 267,163/350 which is equal to 763.3. The width of the box is 14'- 0", and the dimensions of the box are 14'x 7'x 8" which gives 784 cu.ft.

In the design of the counter weight box, the box was considered as a beam uniformly loaded and the sides and bottom figured as resisting the bending moment, when the bridge is in an inclined position and the case when the bridge is closed.

DESIGN OF BRIDGE.

In figuring the stresses in the stringers and floor beams, Cooper's E-50 loading was used and the position of live load to produce a maximum stress was ascertained according to the following criterions.

1. Criterion for Maximum Moment in the Stringer.- The heaviest load must be on the span, and a wheel load must be at the section, and this wheel causing the maximum moment must be as far on one side of the section of the center, as the center of gravity of all the loads is on the other.

2. Criterion for Maximum Reaction.- The average load in the panel on the left must be equal to or just greater than the average panel load on the right, including the wheel at the panel point.

3. Criterion for maximum shear in a Panel.- The average load in the panel must be equal to the average load on the span.

4. Criterion for Maximum Moment in the Span.- The maximum moment occurs when the average load upon the span is equal to or just greater than the average load in front of the panel point.

In ascertaining the maximum bendung moment in the truss, Criterion No. 4 was used. Bringing the wheels on from the right ahd up to the panel point, the maximum bending moment occurred when there was a load at the panel point and the average load on the left is less than the load on the whole span. When the load is on the right of the panel point and greater than the load on the whole span, when the load is considered on the left of the panel, then that load causes a maximum bending moment.

In determining the dead load, we assumed the load resulting from the formula

$$P = 5\,L + 350$$

as given in Cooper's Specifications. A graphical determination of the stresses in each member due due a dead panel load

o

DEAD LOAD STRESSES. (Kips)

Assuming Panel Load of 13,500#

Position of bridge to horizontal.

MEMBER	Horizontal	30°	60°	79°
AQ	5.5	5.0	3.0	2.0
QR	9.0	7.8	4.5	3.5
PQ	– 2.8	– 4.5	– 5.5	– 5.3
RS	20.5	18.0	10.5	8.0
SB	13.5	9.5	3.5	1.0
RO	– 2.8	– 9.0	– 13.5	– 14.0
ST	– 4.5	– 4.0	– 2.5	– 2.0
TC	13.5	7.0	1.0	5.0
TU	37.0	31.5	18.5	13.5
UN	– 32.0	– 39.0	– 36.0	– 32.5
UV	9.0	7.8	4.5	3.0
VW	– 42.0	– 37.5	– 23.5	– 18.5
WD	54.5	40.5	17.0	8.0
VM	– 32.0	– 44.0	– 44.0	– 41.0
WX	– 4.5	– 4.0	– 3.0	– 2.0
XY	55.5	49.5	31.0	23.5
XE	54.5	40.5	13.0	3.5
YL	– 77.5	– 89.0	– 77.5	– 69.5
YZ	9.0	7.8	3.0	3.0
Zα	– 76.5	– 66.0	– 38.0	– 27.0
αF	38.0	75.0	32.5	17.5
ZK	– 77.5	– 93.0	– 85.0	– 77.5
αB	– 65.5	– 49.5	– 19.0	– 8.0
BQ	94.5	67.0	22.5	7.5
BY	– 52.0	– 35.5	– 9.0	– 0
YH	89.0	59.5	18.0	3.0
Yε	– 81.5	– 55.5	– 14.5	– 0
εI	104.5	70.5	20.0	2.5
εJ	47.5	93.0	111.5	110.0

TABLE A

LIVE and WIND LOAD STRESSES.

MAXIMUM FLOOR BEAM REACTION FOR HANGERS

MEMBER	Live Load.	Wind Bridge at 30°	Wind Bridge at {19° / 60°	Wind on side
A Q	-407.5	2.4	3.6	
Q R	195.3	4.0	6.1	
P Q	91.5	-1.2	-1.8	± 6.6
R S	1796	-7.1	-10.7	
S B	-2182	4.9	7.4	± 1.9
R O	915	-1.2	-1.8	± 20.0
S T	0	- 0	0	
T C	-2182	4.9	7.4	± 5.1
T U	-45.0 +104.5	11.7	17.8	
U N	230.8	-10.6	-16.6	± 33.2
U V	1953	4.0	6.1	
V W	-45.0 +104.5	-17.0	-25.9	
W D	-1810	18.0	27.1	± 9.5
V M	230.8	-10.9	-16.6	± 46.7
W X	0	0	0	
X Y	1740	21.0	32.1	
X E	-1810	18.0	27.1	± 13.3
Y L	91.5	-26.0	-39.4	± 60.0
Y Z	195.3	3.2	4.8	
Z α	-365.0	-22.2	-33.8	
α F	0	6.0	9.2	
Z K	91.5	-27.0	-39.4	± 65.9
α B		-2.6	-3.7.0	
B G				
B Y				
Y H				
Y θ				
θ I				
θ J				

TABLE B

RESULTANT STRESSES

For Maximum and Σ sec. Specific, Page 6, Article 22.

MEMBER	Maximum Σ Wind + Dead.	LIVE	Maximum	Σ *
AQ	7.4	-407.5	-411.2	414.9
GR	11.8	195.3	+207.1	207.1
PQ	-13.9	91.5	+98.5	105.4
RS	25.4	179.6	192.2	204.7
SB	-17.3	-218.2	-226.9	244.2
RO	-35.3	91.5	109.2	126.8
ST	-4.5	0	-4.5	
TC	17.6	-218.2	-227.0	235.8
TU	43.2	-45.0 +104.5	-132.0	149.5
UN	-84.8	230.8	273.2	315.6
UV	11.8	195.3	207.1	207.1
VW	-54.5	-45.0 +104.5	-131.8	159.1
WD	68.0	-181.0	-215.0	249.0
VM	-107.3	230.8	284.5	338.2
WX	-4.5	0	-4.5	-
XY	70.5	174.0	244.5	244.5
XE	71.8	-181.0	-216.9	252.8
YL	-176.9	91.5	-222.7	268.5
YZ	11.0	195.3	206.3	206.3
Zα	-88.2	-363.0	-407.1	451.2
αF	98.0	0	98.0	98.0
ZK	-190.3	91.5	236.1	281.8
αB	-65.5		-65.5	65.5
BQ	94.5		94.5	94.5
BY	-52.0		-52.0	52.0
YH	89.0		89.0	89.0
yθ	-81.5		-81.5	81.5
θI	1045			
θJ	111.6			

TABLE C

Max.: For reversal of stress, add ½ smaller stress to vary-r for Max* Sum of two for design joint

DEAD LOAD STRESSES (kips)
Actual Dead Panel Load = 16,400#
Position of bridge to Horizontal

MEMBER	Horizontal	30°	60°	79°
AQ	6.7	6.1	3.7	2.4
QR	11.2	9.5	5.5	4.2
PQ	3.4	5.5	6.7	6.5
RS	25.0	22.0	12.8	9.7
SB	16.4	12.3	4.2	1.2
RO	3.4	11.0	16.5	17.0
ST	5.5	4.4	3.0	2.4
TC	16.8	8.5	1.2	6.1
TU	45.0	38.4	22.5	16.4
UN	39.0	42.5	43.8	39.5
UV	11.2	9.5	5.5	3.6
VW	51.0	45.5	28.5	22.5
WD	66.1	49.0	20.6	9.7
VM	39.0	53.6	53.5	50.0
WX	5.5	4.9	3.6	2.4
XY	67.5	60.0	37.5	28.5
XE	66.1	49.5	15.8	4.3
YL	94.0	108.0	94.0	84.5
YZ	11.2	95.0	36	36
Zα	93.0	80.0	46.3	32.8
αF	119.0	91.0	39.5	21.3
ZK	94.0	112.0	103.0	94.1
αβ	79.5	60.0	23.0	9.7
βG	114.5	81.5	27.5	91.0
βY	63.0	40.8	11.0	0
YH	108.0	72.5	21.9	3.6
YΘ	99.0	67.5	17.6	0
ΘI	127.0	85.5	24.4	3.0
ΘJ	57.5	112.0	136.0	134.0

TABLE D

FINAL RESULTANT STRESSES

(Preliminary, See Table C for Data)

MEMBER	Maximum Σ Wind + Dead	LIVE	Maximum	Σ
AQ	9.1	-407.5	412.1	416.6
QR	14.0	195.3	202.5	209.3
PQ	-16.1	91.5	99.6	107.6
RS	29.1	179.6	194.2	208.7
SB	-19.1	-218.2	227.8	237.3
RO	-38.8	91.5	110.3	130.3
ST	-5.5	0	5.5	
TC	19.1	-218.2	227.8	237.3
TU	50.1	-45.0 +104.5	129.6	154.6
UN	-93.3	230.8	277.5	324.1
UV	13.5	195.3	203.1	208.8
VW	-62.5	-45.0 +104.5	135.8	167.0
WD	76.3	-181.0	219.8	257.5
VM	-116.8	230.8	289.2	347.6
WX	-5.5	0	5.5	
XY	81.0	174.0	254.0	254.0
XE	80.8	-181.0	221.4	261.8
YL	-185.9	91.5	231.7	277.4
YZ	12.5	195.3	207.8	207.8
ZX	102.2	-363.0	-414.1	465.2
αF	114.0	0	114.0	114.0
ZK	209.3	91.5	255.1	300.8
αβ	-76.0		76.0	76.0
βθ	144.5		114.5	114.5
βγ	-63.0		63.0	63.0
γH	108.0		108.0	108.0
γθ	99.0		99.0	99.0
θI	127.0		127.0	127.0
θJ	136.0		136.0	136.0

TABLE E

of one kip gave a coefficient from which the actual dead and wind stresses were obtained. Then by multiplying by the dead and wind panel load, the final dead loads were then subsituted and the stresses figured.

In figuring the impact, the formula

$$I = S\frac{300}{300 + L}$$

was used as given in Art.9 of the "American Railway Engineering and Maintenance of Way Association," where "I" is the dynamic increment to be added to the live load stresses, and "L" is the loaded part of the bridge.

The wind load was assumed as 200# per linear foot of the upper chord and 200# plus ten per cent of the specified train load on the bridge (5,000# per foot of track) for the lower chord.

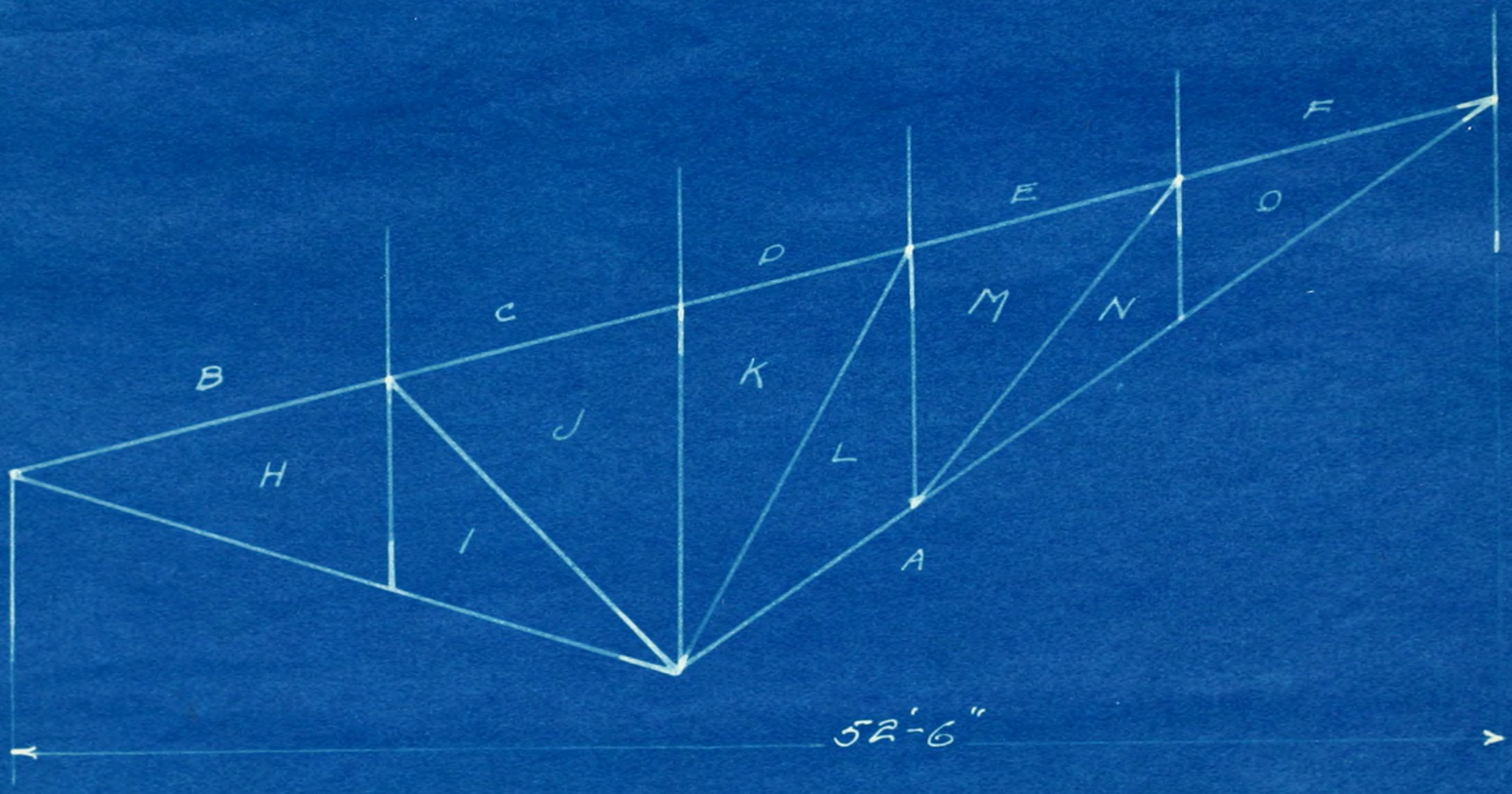
THE PIVOT TRUSS.

When the bridge rotates, the trunnion has to be placed so that the counter weight will not strike the pier. This necessitates a supporting or pivot truss. This truss supports the weight the bridge and also the load from the track across the pit.

The stresses in the various members were found to be as given in the following table.

Member	Stress	Area
AH	403,000	30 sq.in.
AI	403,000	30 sq.in.

OUTLINE OF THE SUPPORTING TRUSS.



Member	Stress	Area
AL	452,000	36.4 sq.in.
AN	582,000	"
AO	582,000	"
BH	380,000	32.5 sq.in
CJ	359,000	"
DK	359,000	"
EM	417,000	"
FD	520,000	"
HI	0	
IJ	31,000	1.9 sq.in.
JK	240,300	15.02 "
KL	113,000	7.06 "
LM	33,000	2,10 "
MN	148,000	9.25 "
NO	0	

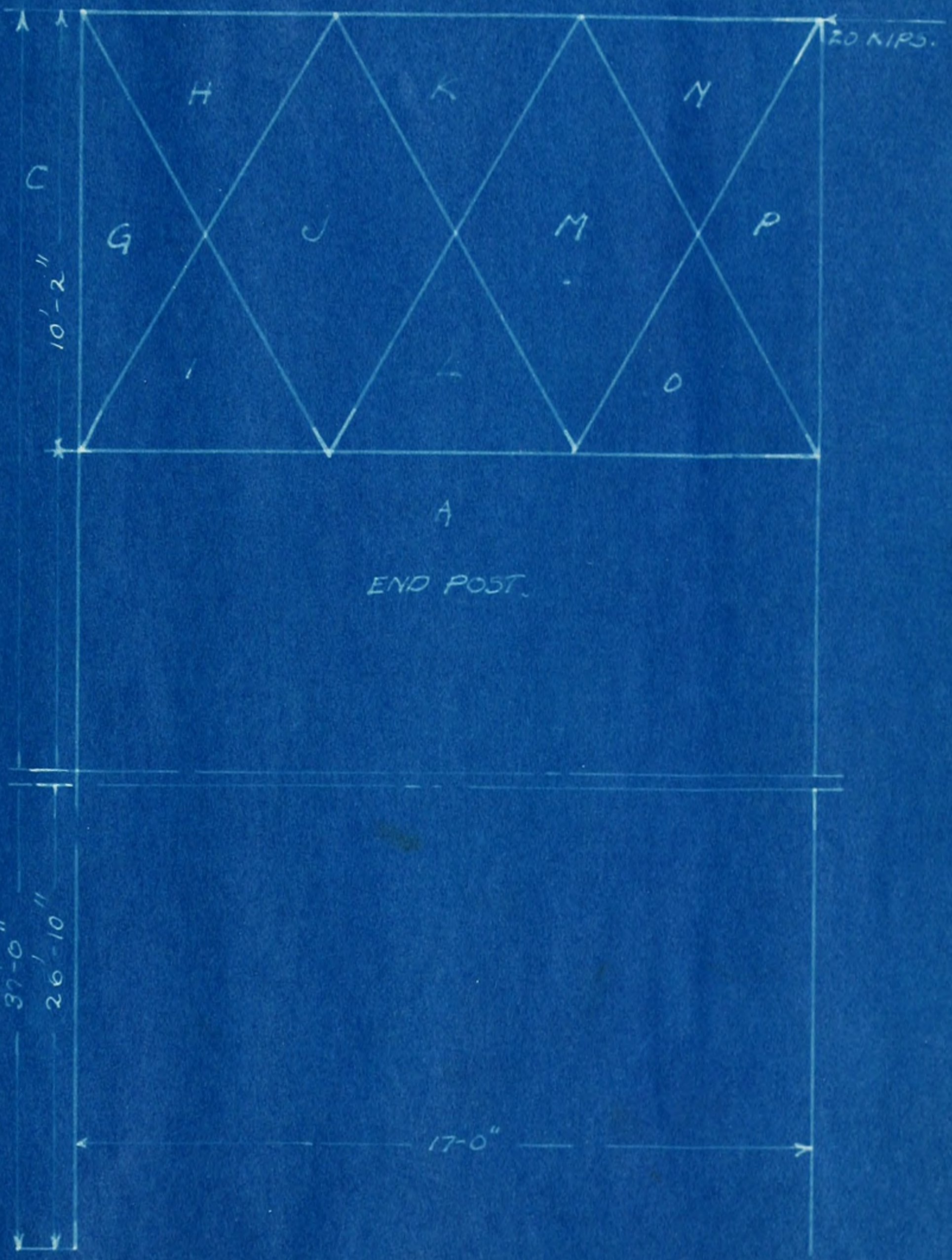
AH & AI - 2 - 15" @ 55# A = 32.36

AL, AN & AO 4 - 4"x 4"x 5/8" Angles and 1 - 15"x 3/4" Pl.

DESIGN OF PORTAL

The outline of the portal is similar to the one in the "Stress Phamphlet" by Prof. Phillips in Fig. 45. The main cross girder has two diagonals in each panel, and the shear that is distributed between them is practically indeterminate. It was necessary to make some arbitrary assumptions, and the probable error will not be great if it is considered that each diagonal in the middle panel takes one half of the shear.

OUTLINE OF PORTAL.



Under these assumptions, the following stresses were determined.

Wind Pressure at the end	20,000#
JL = KM	18,048#
JK = LM	18,048#
HJ = GI	18,048#
IA	9,000#
GH = GI	2,568#
EH	29,352#
CG	2,220#
LA	1,164#
EK	11,376#
MN	17,048#
EN	6,660#
OA	9,000#
IA	9,000#
MO = NP	840#
EP	720#

The portal was made much heavier than necessary to prevent swaying of the bridge in operation.

SPECIFICATIONS

The stresses permitted to be developed in a member are given in the specifications by the Maintenance of Way Association article by article as follows:

Art. 15. Axial Tension - - - - - - - 16,000#

" 16. Axial Compression 16,000 - 70 l/r

where "l" is the length of the member and "r" is the least radius of gyration

" 18. Shearing, shop driven rivets - - 12,000#

" 19. " field rivets - - - 10,000#

" 19. Bearing, shop driven rivets - - 24,000#

" field rivets - - - 20,000#

" 20. Bearing on rollers - - - - 600 d

where "d" is the diameter of the foller in inches.

An allowance for temperature expansion was obtained by using the coefficient .0000056 for a change of temperature of 50°.

$$E = (.0000065)\ t\ l$$

where "t" is the change of temperature = 50°

"l" is the length of the bridge

"E" is the amount of elongation

The bearing value of each pile was assumed as 19 tons.

In all other conditions not covered by these specifications, the "American Railway and Maintenance of Way Association's" specifications must be followed.

ERECTION

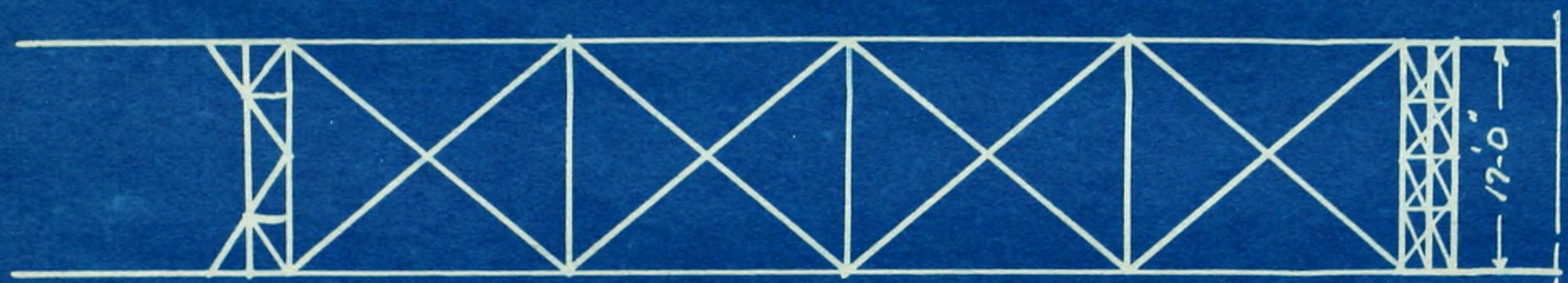
In erecting the bridge, the segment is made in the shop and comes shipped tonthe location in two pieces. The main bridge is then built up piece by piece while the bridge is open. In this way, the erection stresses are low.

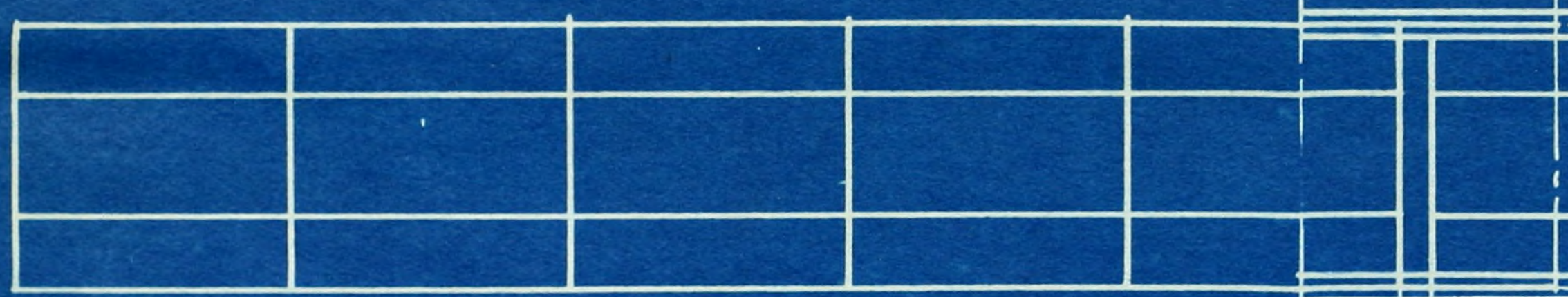
PIER AND ABUTMENTS

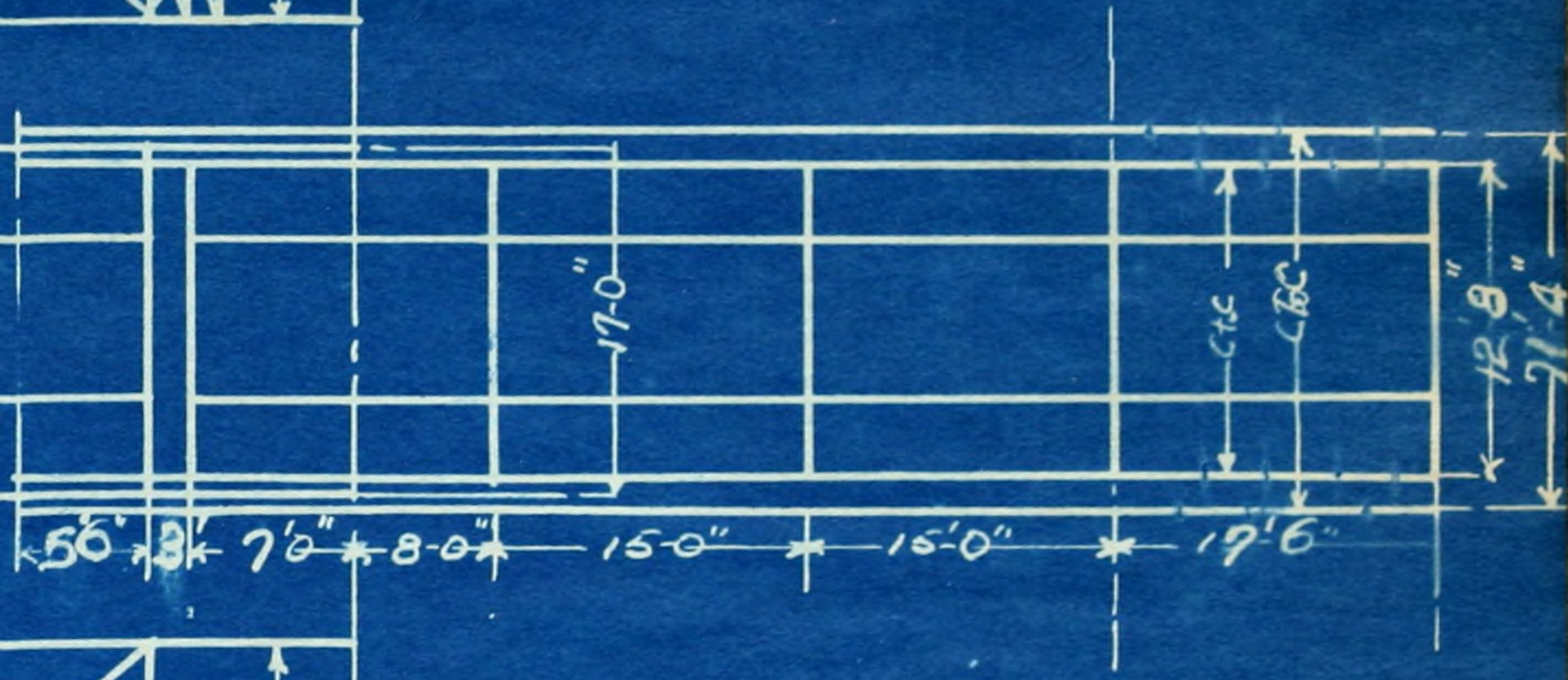
The river pier was assumed as 4'- 0" wide at the top with a batter of 1 1/2" to the foot. The height above the figured base was 31 feet and the water was 18 feet above the base. This gave a horizontal water pressure of 10,000#, and the weight of the concrete in the abutment was 31,400#. The resultant came out to be 33,000# and fell within the middle third of the base.

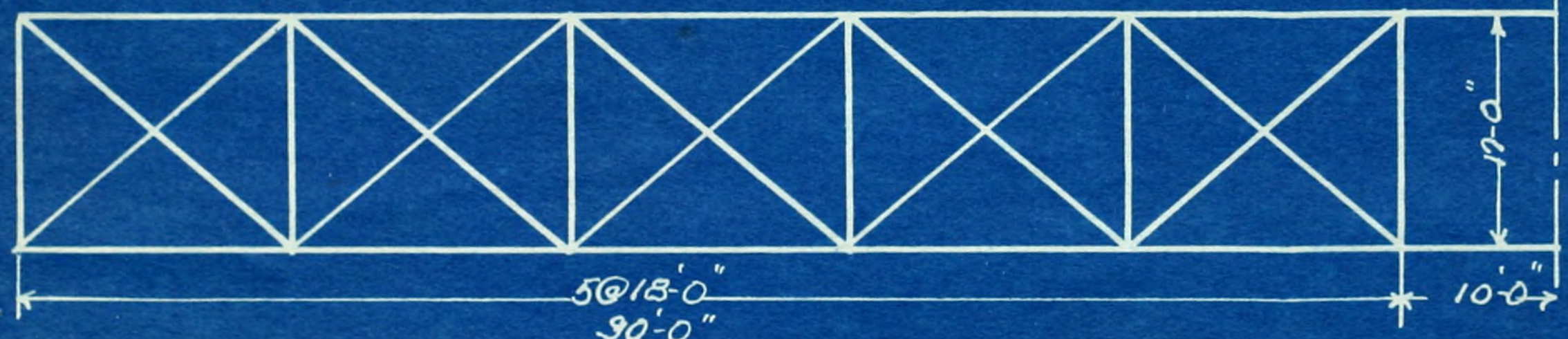
The supporting pier was assumed as 4 feet wide on top and was 25 feet high, the batter on one face being 4" to the foot, the other face being vertical. The resultant earth pressure equalled 10,900#. The weight of the masonry was 33,800# and the final resultant equalled 35,700# and fell 3 inches inside of the middle third.

The west and east abutments were designed so that the resultant fell within the middle third. In figuring the number of piles required , each pile was assumed as being capable of carrying a load of nineteen tons, but a greater number were put in for safety and good design.









OUTLINE OF PORTAL, LATERAL
STRINGERS, FLOOR-BEAMS,
MAIN & SUPPORTING
TRUSSES.

References consulted in obtaining information for the design:

Some new Bascule Bridges, Engineering News, July 18, '07.

Double Track Trunnion Bascule Bridge over Bodine Creek, Staten Island Rapid Transit Railway, Engineering News, Jan. 16,1908.

Remarkable Bascule Bridge at Syracuse, N. Y., Engineering News, Sept. 24, 1908.

Ashland Avenue Bascule Bridge, Chicago, Engineering Record, Oct. 10, 1903.

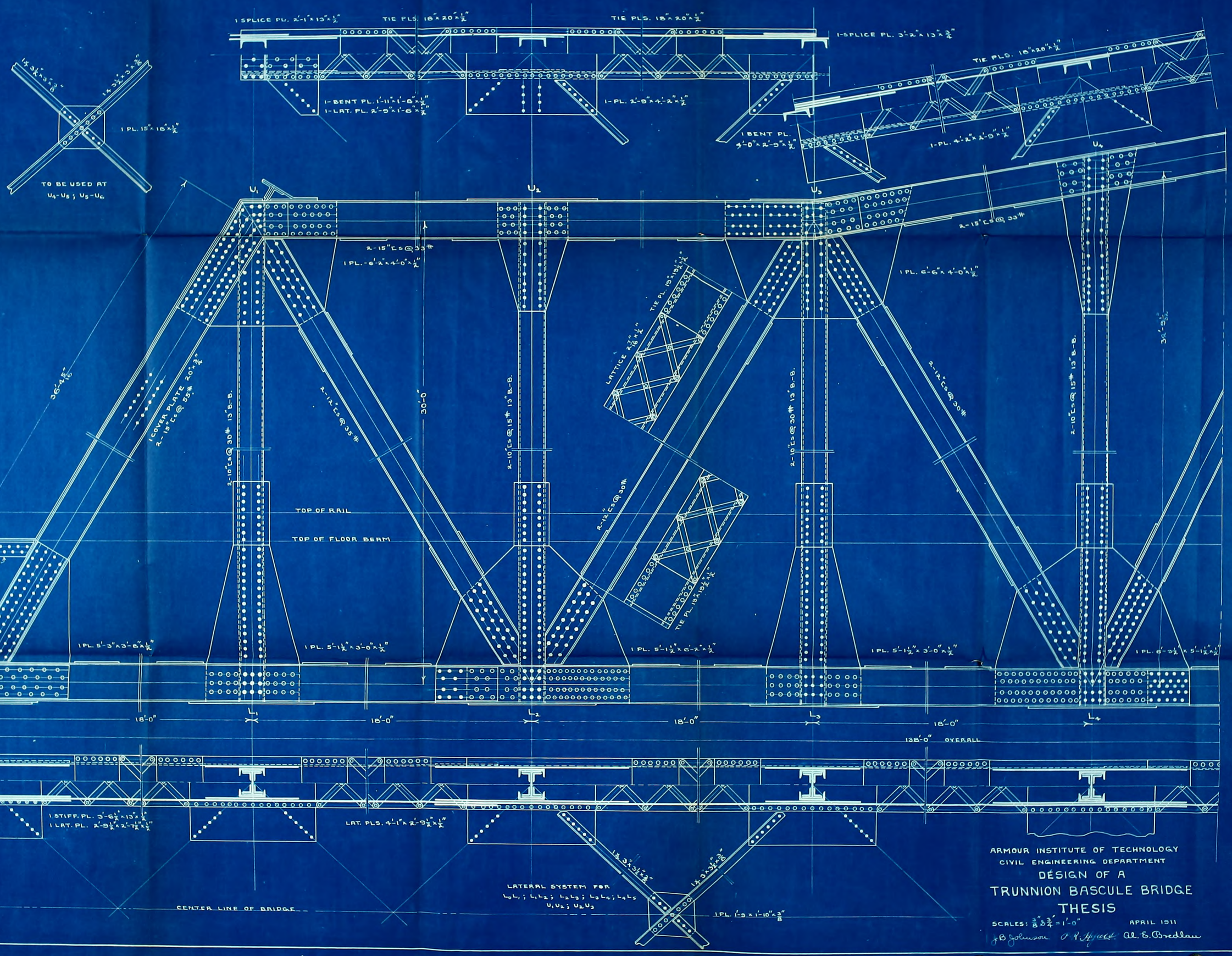
Division Street Bascule Bridge, Chicago, Engineering Record, Aug.20, 1904.

A Trunnion Bascule Bridge, Railroad Gazette, July 11, '02.

Trunnion Bridge for N. & L. E. R. R. at Cleveland, Railway and Engineering Review, Dec. 16, 1905.

PLATE I

Main Girder of Bridge



ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

DESIGN OF A TRUNNION BASCULE BRIDGE THESIS

SCALES: 3/8" & 3/4" = 1'-0" APRIL 1911

J.B. Johnson O.P. Hyneck Al. E. Bredlau

PLATE 2.

Lifting Segment.

U6

20'-0"

20'-0"

U5

U4

U7

2-15"E@33# fabtd.

2 Plates ½x5'-5x6'-0"

1 Plate 5'-0"x2'-8"x½"

3 Plates ½" by 2'-8" by 5'-6" Reqd cut and fastened as shown for THE ARC OF SEGMENT

Filler plates ½" by 2'-8" by 3½" Reqd cut as shown for joints U5 U6 & U7

1 Plate ½"x2'-11"x4'-10"

Notes.

4"x4"x⅝" ∠s are used as the lower and upper flange ∠s of the arc of the segment. A uniform spacing of 3" is used thru out with a variation at the joints to suit the rivet spacing of the plates.

The rack is made of 4 steel pins placed 9" c to c separated by plates ½" by 7" by 17" and held in place by bars ⅝" by 2½" by 3'. Common pin nuts are used held in place by a patented nut lock.

Separators consist of 1¼" standard gas pipe and ⅜" steel rods and ⅜" bolt nuts. Spaced as indicated.

Spacing of Separators. 6'-0" c to c

2-15"E@35#-14 fabd.

2-15"E@35#-13 fabd.

2-10"E@35#-13 fabd.

1-⅛" Cover Plate 8½"

2-15"E@35#-14 fabd.

2-15"E@35#-14 fabd.

35'-6½"

32'-6½"

4'-2"

Top of Rail.

Lattice bars are 2-½"x6" by ½"

37'-0"

2-15"E@35#

Bottom Plate ½"x2'-11"x5'-6"

1 Bottom Plate ½"x15"x18"

The bottom plates are fastened to the shoe by 1¼" Rd. turned bolts.

This is standard for all bottom plates on the spokes of segment. 15" Rd 19.

2-15"E @35#

1 Plate ½"x5'-6"x6'-0"

2-15"E@35#

The SHOE consists of a steel Casting of three parts as shown in detail.

The PIN is 15" in diameter and 4'-0" long made of cast steel.

2-15"E@35#

30'-0"

U8

9'-3"

℄ of Gear which is in mesh with Arc rack.

U9

3'-0"

4'-0"

1 Plate -½"x8'-6"x1'-0"

5"x5"x½" ∠s Use in the construction of the box.

7'-0"

2 Plates ½"x7'-0"x7'-3"

1 Plate -½"x28"x87"

1 Plate -15"x½"

2-15"E@35# placed 14" c to c

1-Plate 4'-2"x21"x½"

1 Plate 4'-2"x 21"x½"

L4

L5

L6

18'-0"

10'-0"

15"

10"

2'-0"

ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT.

THESIS.

DESIGN OF A

TRUNNION BASCULE BRIDGE.

GENERAL DRAWINGS OF SEGMENT.

3/4, 3/8 and 1/2 In. = 1Ft. Scales

Apr-25-1911

CHICAGO ILL.

DRAWING ⑤ of ①

Ad. to. Bradbeer P.R. Hayward J.B. Johnson

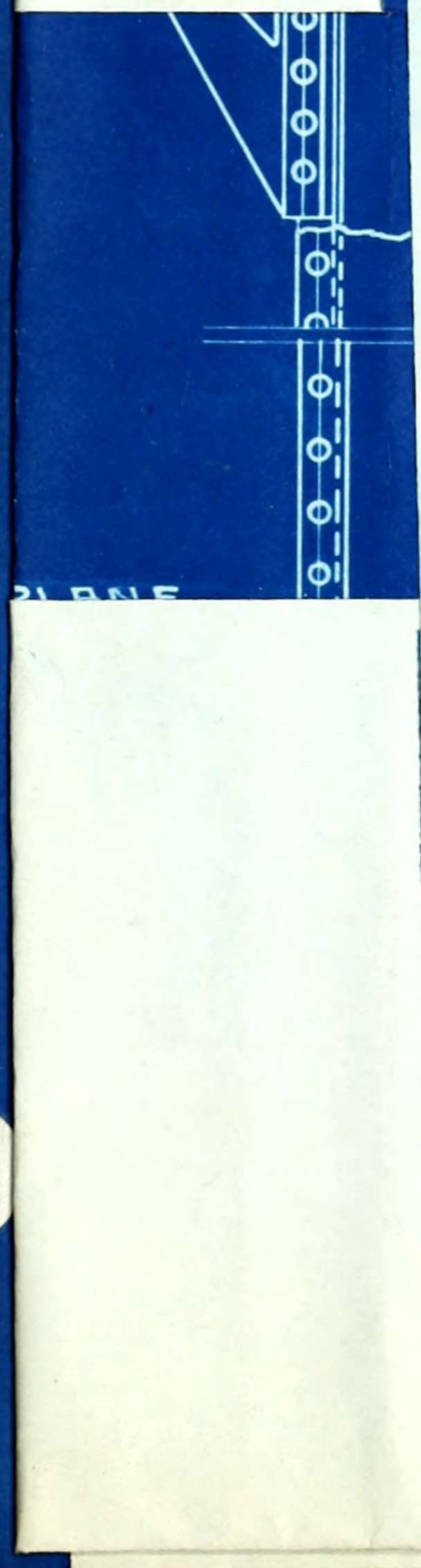


PLATE 3.
Portals & Shoe.

2 ∠s 5×5×½"

2 ∠s 5"×3½"×⅜"

ALL ∠s 3½"×3"×⅜"

ALL ∠s 4"×3"×½"

TIE PL.-15"×14"×½"

LATTICE 2⅞"×½"

6½"

3'-0"

22'-10"

CENTER LINE OF BRIDGE

22'-10" VERTICAL PLANE

LATTICE 2⅞"×½"

SECTION THRU
$U_2L_2 - U_3L_3$

SECTION THRU PLANE
OF END-POST

2 ∠s 5"×3½"×⅜"

2 ∠s-6"×8"×¾"

1 PL.-2'-2½"×4'-0"×½"

1 PL.-2'-11"×1'-6½"×½"

WEB PLATE
11'-0"×4'-0"×½"

1 PL.-2'-11½"×1'-6½"×½"

1 PL.-1'-11½"×4'-0"×½"

ALL ∠s 6"×6"×⅝"

TIE PL. 15"×14"×½"

2 ∠s-8"×6"×⅜"

4'-0"

8"

4'-0"

4'-0"

8'-6"

8'-6"

17'-0"

1½" CORED HOLES

9"

1'-6½"

9"

⅞" DRILLED HOLES

4"

6"

15/16"

4½"

21"

3"

2'-10½"

3"

2½"×4½"×½"

3"

2"

7/8"

1'-4¼"

7/8"

2"

3"

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

DESIGN OF
TRUNNION BASCULE BRIDGE

SCALES: ¾"=1'-0" 1"=1'-0" MAY 1911

J B Johnson P.R. Hyde A.L.E. Bredlau

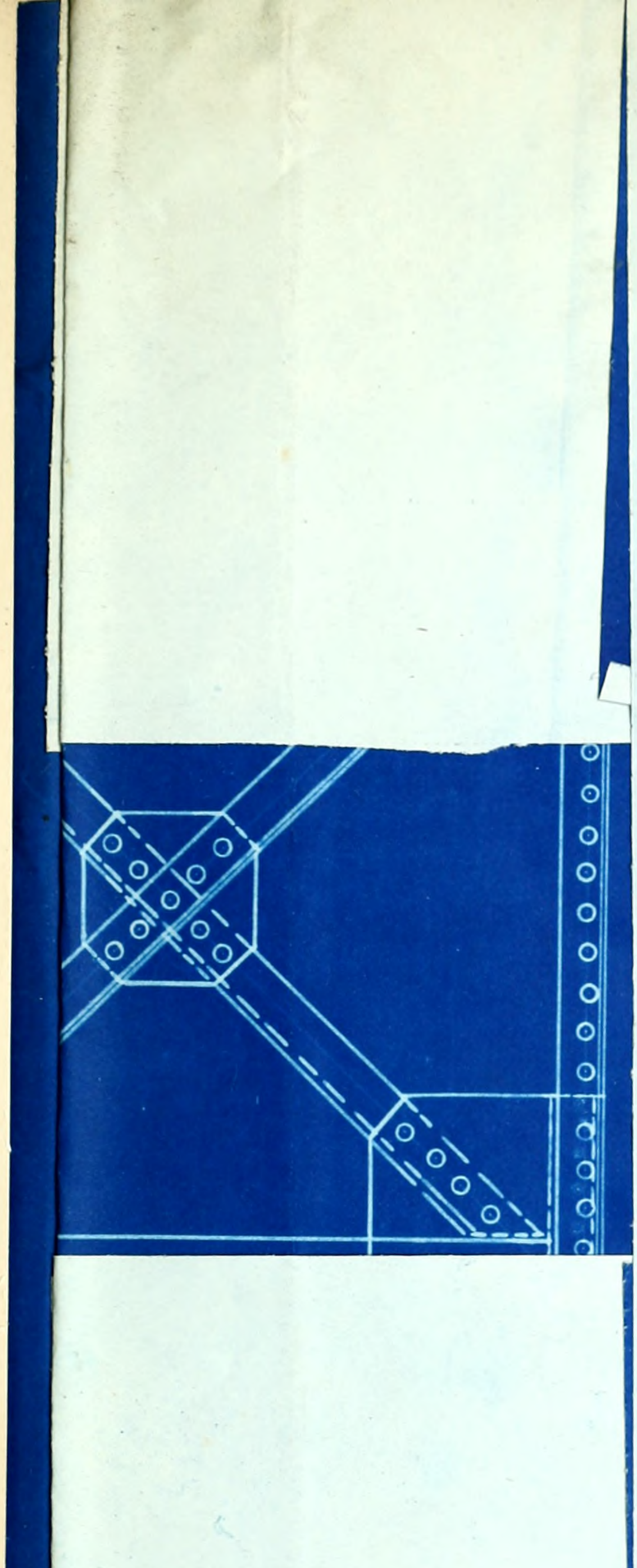
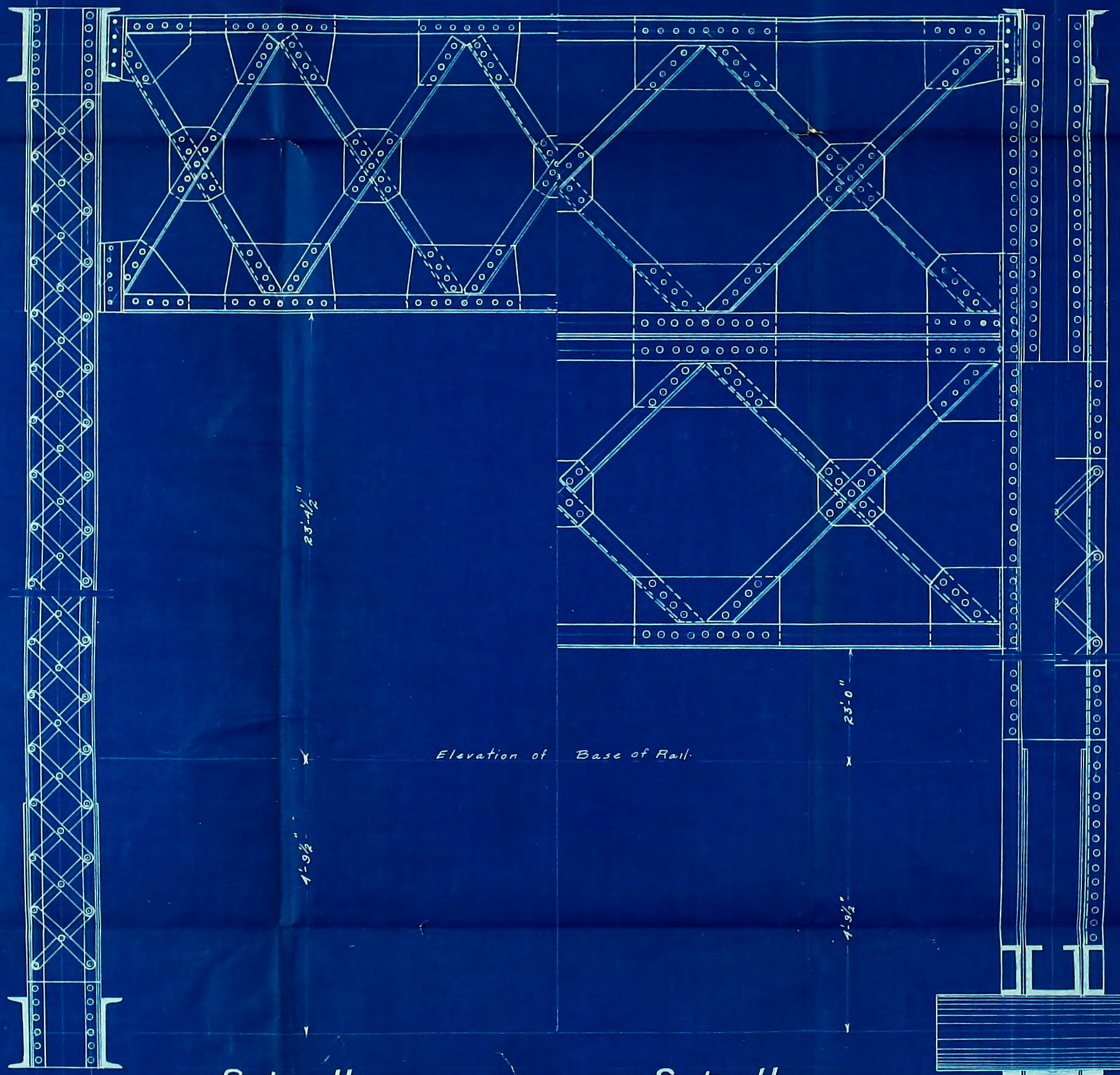


PLATE 4.
Portals.



Section U4.

Section U5.

ARMOUR INSTITUTE OF TECHNOLOGY.
CIVIL ENGINEERING DEPARTMENT.

THESIS

GENERAL DRAWING OF PORTALS.

Scale ¾"=1'-0" May 1911.

Plate (4) of (7)

J.B. Johnson
C.C. Brodbeck
P.R. Hynes

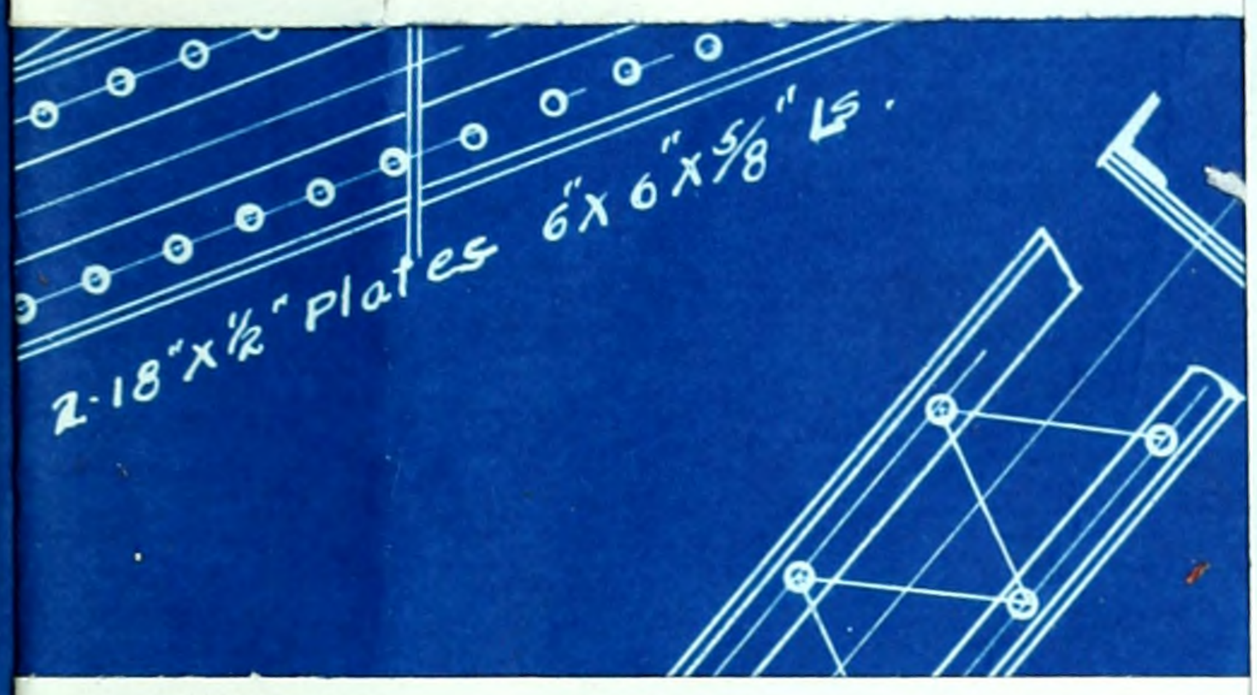
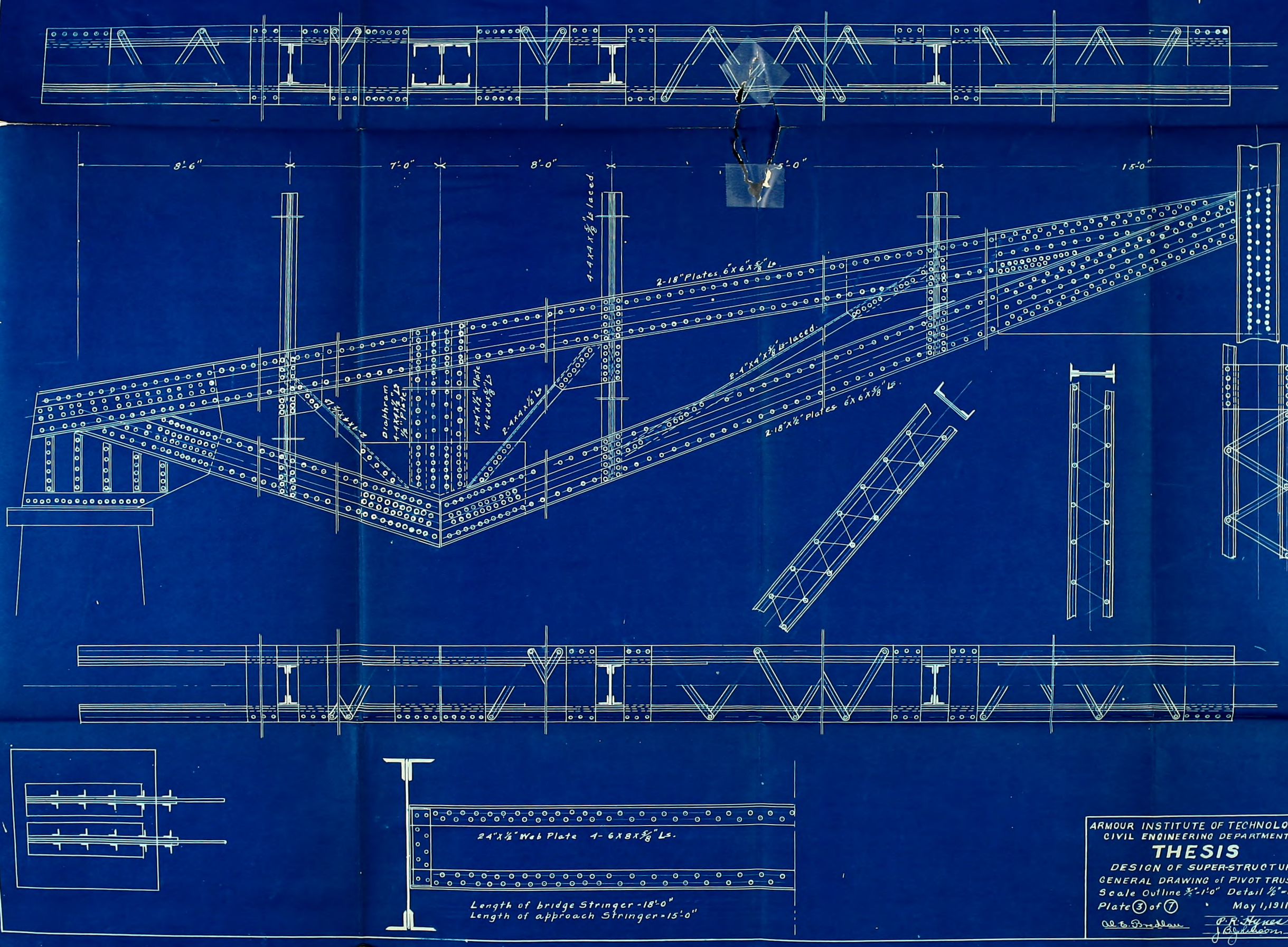


Plate 5

THE Supporting Truss



ARMOUR INSTITUTE OF TECHNOLOGY.
CIVIL ENGINEERING DEPARTMENT.

THESIS

DESIGN OF SUPER-STRUCTURE.

GENERAL DRAWING of PIVOT TRUSSES

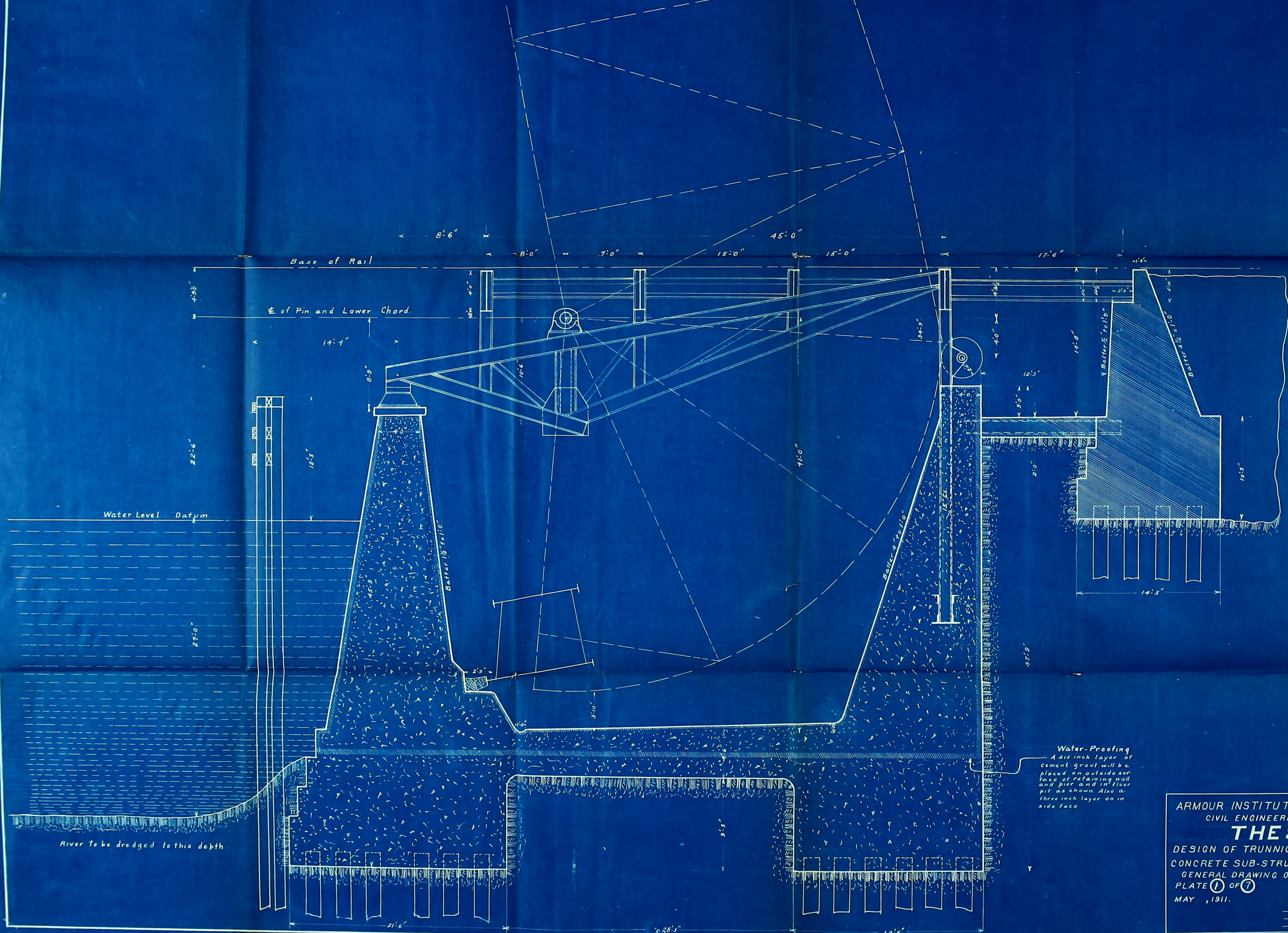
Scale Outline ⅛"-1'-0" Detail ½"-1'-0"

Plate (3) of (7) May 1, 1911

W. E. Bradley — P. R. Hynes
J. Blackburn

PLATE 6

The Substructure



ARMOUR INSTITUTE OF TECHNOLOGY.
CIVIL ENGINEERING DEPARTMENT.

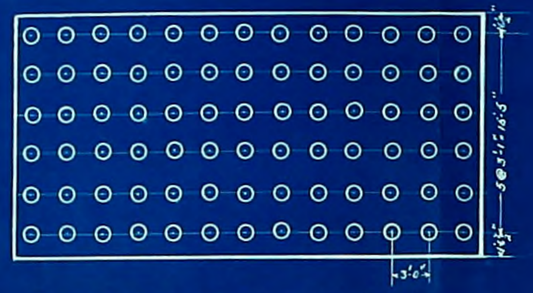
THESIS

DESIGN OF TRUNNION BASCULE BRIDGE
CONCRETE SUB-STRUCTURE LONG. SECT.
GENERAL DRAWING OF PIVOT TRUSS.
PLATE (1) OF (7) SCALE ¼"=1'-0"
MAY, 1911.

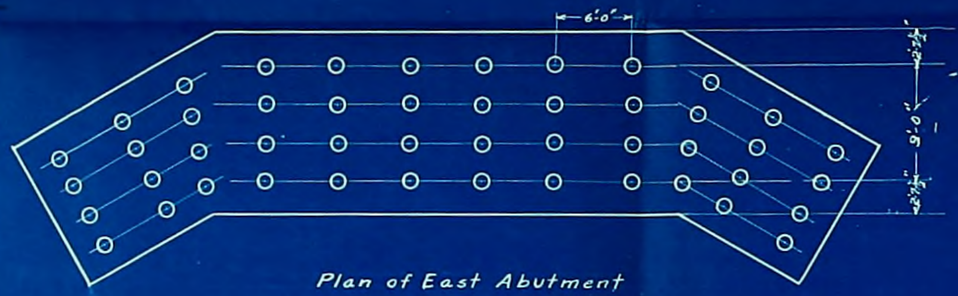
A. B. Johnson
B. A. Spence
W. S. Sheridan

PLATE 7.

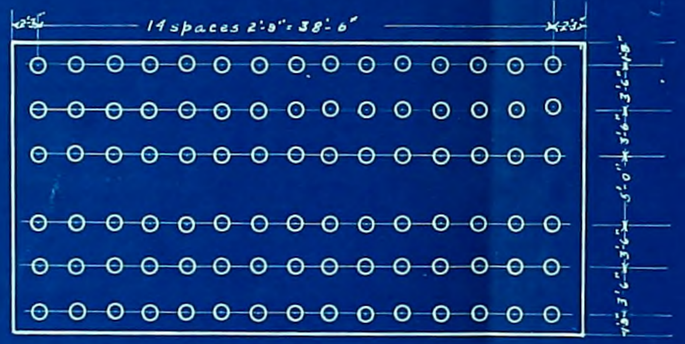
The General Layout.



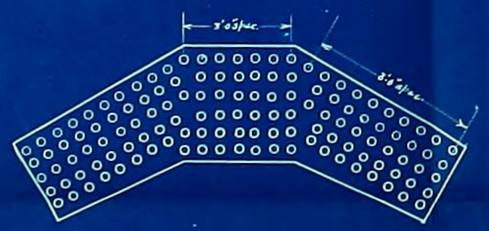
Plan of East Retaining Wall.



Plan of East Abutment

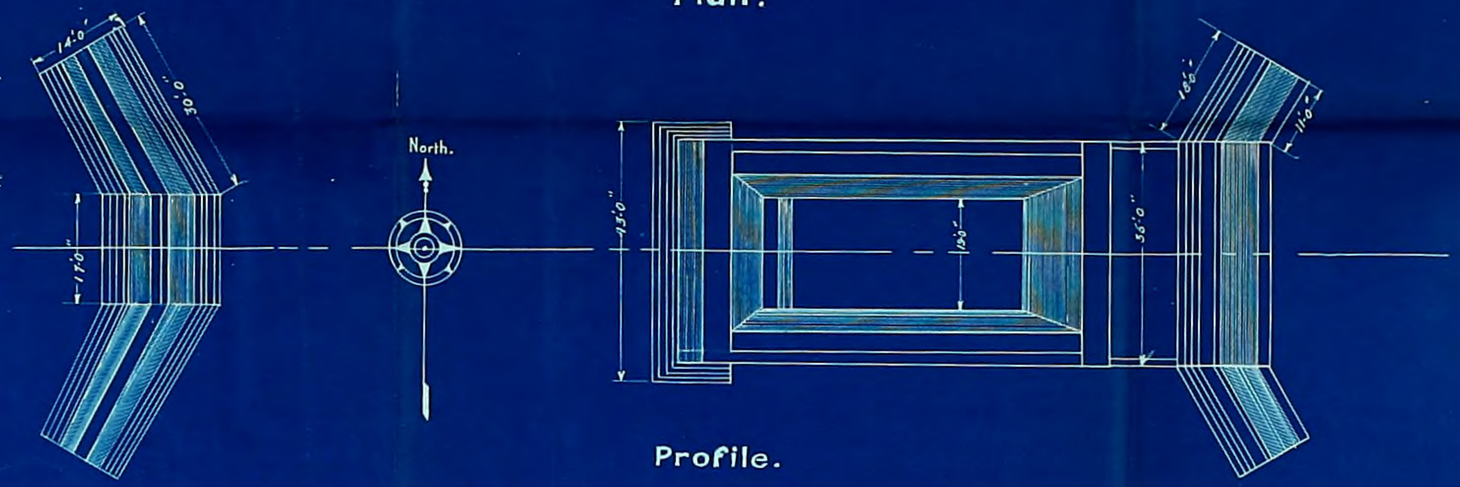


Plan of Pier

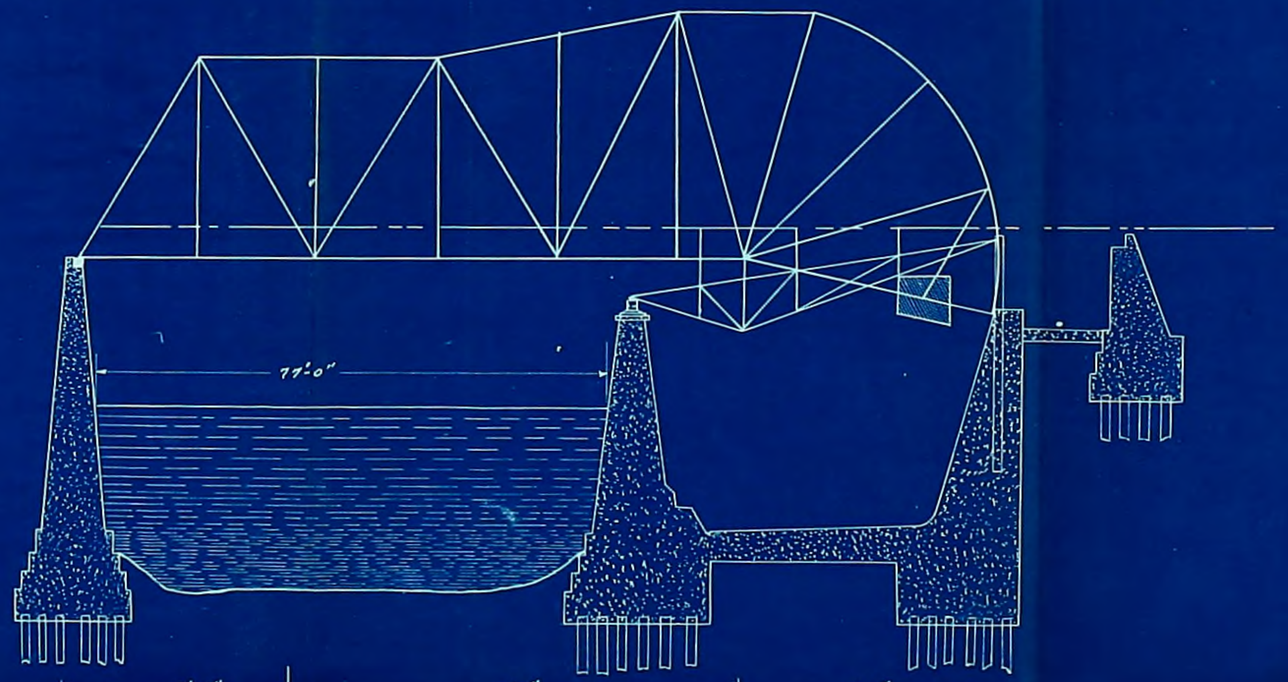


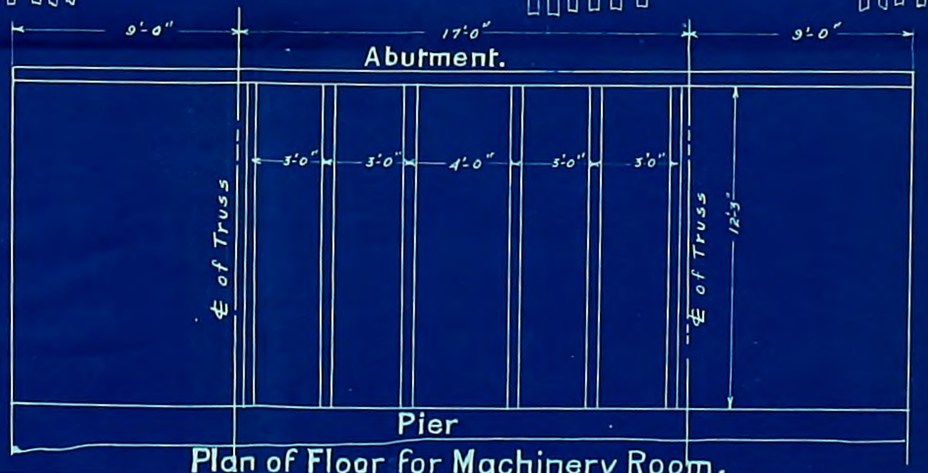
Plan of West Abutment.

Plan.



Profile.





Plan of Floor for Machinery Room.

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT.

THESIS

PLAN VIEW OF SUB-STRUCTURE
SKELETON OUTLINE OF TRUSS
Scale 1/8"=1'-0" and 1/16"=1'-0"
Plate (2) of (7) May 1911.