DESIGN OF A SWING BRIDGE

BY

WILLIAM ANTON THEODORSON

THESIS

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

William Anton Theodorson

ENTITLED Design of a Swing Bridge

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE

OF Bachelor of Science in Civil Engineering.

Ira O. Baker.

HEAD OF DEPARTMENT OF Civil Engineering.

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Design of a Swing Bridge.

Introduction.

Even to the casual observer, the crowded condition of the bridges in the business district of Chicago is noticeable. This is especially true of the Dearborn Avenue bridge, which is used principally by the heavily laden trucks coming from the freight houses on the north bank of the river and by the numerous delivery wagons from the commission houses on the south bank. Another feature which one can not fail to notice is the lack of adequate room for the passage of vessels. The nearness of the Dearborn Avenue bridge to the mouth of the river, and the fact that most of the warehouses, and lumber and coal yards of the city are located on the river causes the bridge to be opened at frequent intervals. This usually results in a blockade of vehicles on the approaches to the bridge.

The bridge which now occupies the site is an iron structure 185 feet long and 19.25 feet between trusses. It was erected at Wells Street shortly after the great Chicago fire. On March 20, 1888, it was moved to its present location.

It is proposed to design the superstructure of a bridge for this place, which shall be longer and wider than the present structure.

Description.

From a careful study of swing bridges described in engineering periodicals, and of working drawings, it has been decided to use the following form. The bridge will have four driveways between two trusses. The span will be 240 feet center to center of end pins, and will be divided into 16 panels each 13.75 feet long and one center panel 20 feet long. The depth of trusses will be 20 feet at the ends and 28 feet at the middle. The top chord of the middle panel will be horizontal. The width is to be 37 feet between centers of trusses. There will be a 7-foot sidewalk outside of the trusses on each side of the bridge. The trusses will be of the Pratt type.

The main trusses will be supported at two points each on two transverse girders, which will in in turn be placed on two longitudinal girders and these will rest upon four bolsters, each having three points of support on the turntable rim. The points of support of the bolster will be so placed as to distribute the weight of the bridge equally over twelve points on the turn-table rim.

The metal work will be of medium steel. The roadway will be planked with one course of five-inch, long-leaf yellow pine planks, and over this will be layed a course of three-inch, white oak planks. The sidewalks will be made of one course of three-inch white oak planks.

Plate I, page 5, shows the outlines of the main and lateral trusses. Tables 1 and 2, Plate II, page 6, give the lengths of all members, their moment arms, and the angle of the member with the horizontal.

Specifications.

The bridge will be designed according to "Specifications for Highway Bridges of Chicago."

PLATE I.

DIAGRAMS OF MAIN & LATERAL TRUSSES.

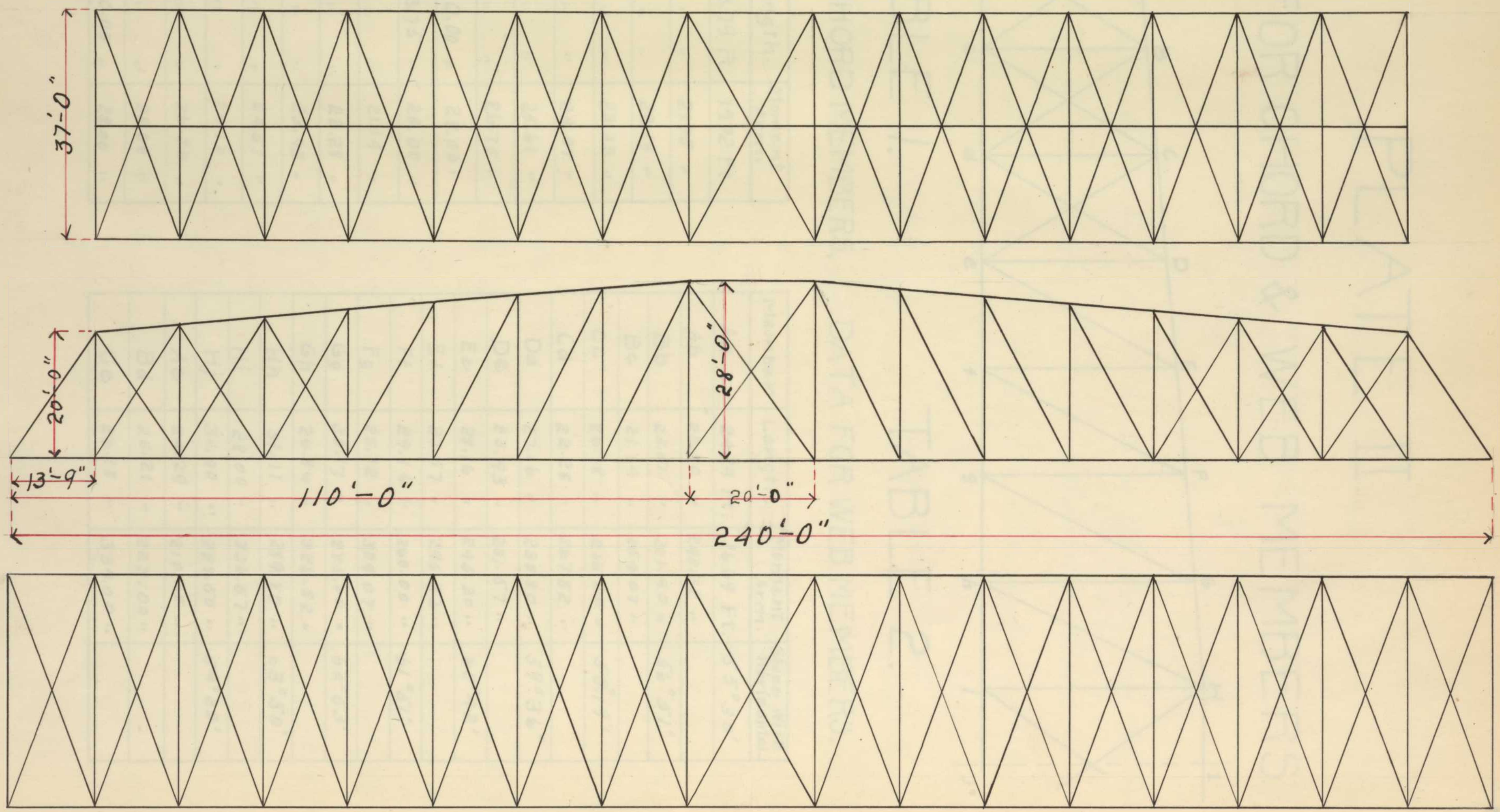


PLATE II.

DATA FOR CHORD & WEB MEMBERS.

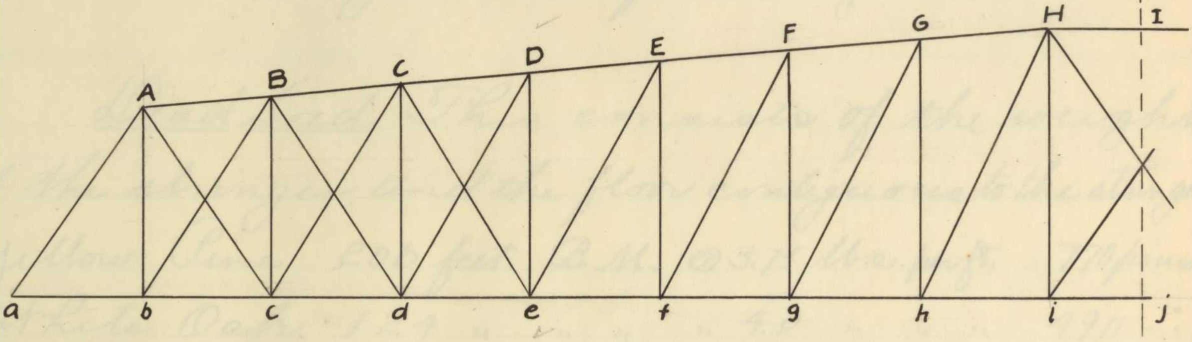


TABLE 1.

DATA FOR CHORD MEMBERS.

Member.	Length.	Moment Arm.
AB	13.79 Ft.	19.92 Ft.
BC	" "	21.05 "
CD	" "	22.19 "
DE	" "	23.33 "
EF	" "	24.47 "
FG	" "	25.61 "
GH	" "	26.75 "
HI	20.00 "	28.00 "
ab	13.75 "	20.00 "
bc	" "	21.14 "
cd	" "	22.28 "
de	" "	23.43 "
ef	" "	24.57 "
fg	" "	29.16 "
gh	" "	26.86 "
hi	" "	28.00 "
ij	20.00 "	28.00 "

TABLE 2.

DATA FOR WEB MEMBERS.

Member	Length.	Moment Arm.	Slope With Horizontal.
Aa	24.29 Ft.	186.69 Ft.	55°30′
Ab	20.00 "	240.32 "	
Bb	25.51 "	201.40 "	56°51′
Bc	21.14 "	254.07 "	
Cc	26.18 "	216.00 "	58°19′
Cd	22.28 "	267.82 "	
Dd	27.16 "	230.80 "	59°36′
De	23.43 "	281.57 "	
Ee	28.16 "	245.20 "	60°42′
Ef	27.57 "	295.32 "	
Ff	29.16 "	260.00 "	61°51′
Fg	25.72 "	309.07 "	
Gg	30.17 "	275.00 "	62°53′
Gh	26.86 "	322.82 "	
Hh	31.11 "	289.80 "	63°50′
Hi	28.00 "	336.57 "	
Hj	34.40 "	288.50 "	54°28′
Ac	24.29 "	210.00 "	
Bd	25.21 "	223.00 "	
Ce	26.18 "	339.00 "	

Design of Stringers.

I. Stringer 13.75 feet Long.

Dead Load. This consists of the weight of the stringer and the floor contiguous to the stringer.

Yellow Pine	206 feet. B.M.	@ 3.75 lbs. per ft.		770 pounds
White Oak	124 " " "	" 4.0 " " "		490 "
Stringer I Beam	13.75 feet long.	" 31.5 " " "		435 "

Dead load uniformly distributed = 1695 pounds.

Live Load. It will be assumed that a rear wheel of the specified steam roller is over the middle of the stringer. The weight on a wheel of the roller is 11500 pounds. Of this weight the floor will transmit 2,500 pounds concentrated load to the adjacent stringers. This reduces the roller load on the joist under consideration to 11500 − 2500 = 9000 pounds concentrated load, or an equivalent uniformly distributed load of 18000 pounds.

Total Load. The combined live and dead load uniformly distributed on the stringer is 1695 + 18000 or 19695 pounds.

Size of Stringer. The permissible unit stress on the flanges is $11000(1+\frac{min}{2 max}) = 11,350$ pounds. 8.

$M \div S = (19,695 \times 13.75 \times 12) \div (8 \times 11,350) = 35.6 = I \div c$ or the section modulus of the required I beam. From Cambria Steel Book it is found that a 12-inch I weighing 31.5 pounds per foot is large enough to take the stress.

<u>End</u> <u>Shear</u>. Live load shear = 9000 pounds
Floor " " = 630 "
Stringer " " = 215 "
Total end shear = 9845 pounds.

The allowable shear on 7/8" rivits = 4510 pounds.
The bearing value of 7/8" rivit in web of stringer = 4922 pounds.
The number of rivits connecting web of stringer to floor beam is $9845 \div 4510 = 2+$ say three. 3"x3"x3/8" connecting angles will be used. The number of rivits required in web of floor beam is four.

Weight of Stringer.

Name of Piece.	Number of Pieces.	Length in Feet.	Cross-Section	Weight per Foot.	Weight Pounds.
I Beam Stringer	1	13.75	12" I	31.5	430.0
Connecting Angles	4	1.0	3"x3"x3/8"	7.2	29.0
Rivits	32		24.3 per. C.		8.0
					Total 467

2. Stringer 20 Feet Long.

Dead Load. This consists of weight of flooring and stringer.

Yellow Pine 300 feet B.M. @ 3.75 lbs. per foot 975 pounds.
White Oak 180 " " " " 4.00 " " " 720 "
Stringer I Beam 20 feet long @ 42.00 " " " 840 "
Dead load uniformly distributed = 2535 pounds.

Live Load. The same assumption is made in this case as was made for the 13.75 foot stringer.

Total Load. The combined live and dead load uniformly distributed over stringer is 2535 + 18000 or 20535 pounds.

Size of Stringer. The permissible unit stress on flanges of stringer $= 11000(1+\frac{min}{2\,max}) = 11680$ pounds. $M \div S = (20535 \times 20 \times 12) \div (8 \times 11680) = 52.7 = I \div c$ or the section modulus of the required section. From Cambria Steel Book it is found that a 15-inch I weighing 42 pounds per foot has a section modulus of 58.9. This section will be used.

End Shear. Shear due to live load = 9000 pounds
" " " dead " = 1265 "
Total shear = 10,265 pounds.

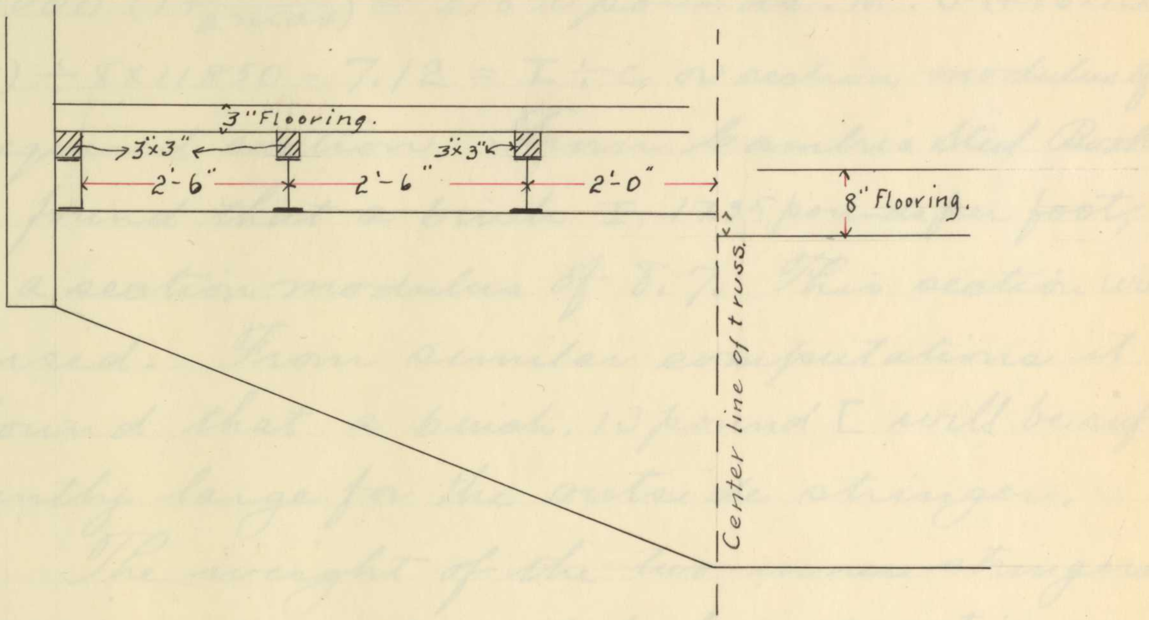
The allowable single shear for 7/8" shop rivets is 4510 pounds.
" " " " " " field " " 3600 pounds.

The allowable bearing on 7/8" shop rivits on .41 in. plate = 5335 pounds.
" " " " 7/8" field " " 3/8 in. " = 3940 pounds.
The connecting angles to be used will be 3" x 3" x 3/8". The number of rivits required in web of I and connecting angles will be 10265 ÷ 5335 = 2+, but in order to keep the pitch of rivits equal to or less than 6 inches it will be necessary to use 4 rivits, giving a pitch of 4 inches which is preferable. The number of rivits in web of floor beam will be 10265 ÷ 3600 = 3, but 4 rivits will be used.

Weight of Stringer.

Name of Piece.	Number of Pieces	Length in Feet.	Cross-Section	Weight per Foot.	Weight Pounds.
Stringer I Beam	1.	20.0	15" I	42	840
Connecting Angles.	4	1.25	3 x 3 x 3/8	7.2	36
Rivits	32		7/8"	24.3 per C.	8
					Total 884

Design of Sidewalk Stringers.



1. Joist for 13.75 Foot Panel.

Dead Load. This is the weight of floor, nailing piece and I beam stringer.

Yellow Pine	100 ft. B.M.	@ 3.75 lbs. per foot	= 375 pounds.
White "	10.3 " "	@ 2.5 " " "	= 26 "
Stringer I Beam	13.75 " long.	@ 17.25 " " "	= 235 "
		Total dead load	= 636 pounds.

Live Load. This consists of 100 pounds per square foot of sidewalk or per stringer uniformly distributed it is 13.75 x 2.5 x 100 = 3450 pounds.

The combined live and dead load is 636 + 3450 or 4086 pounds.

Size of Stringer. The permissible unit stress is $11000\left(1+\frac{min}{2max}\right) = 11850$ pounds. $M \div S = (4086 \times 12 \times 13.75) \div 8 \times 11850 = 7.12 = I \div c$ or section modulus of the required section. From Cambria Steel Book it is found that a 6 inch I, 17.25 pounds per foot, has a section modulus of 8.7. This section will be used. From similar computations it is found that a 6 inch, 13 pound [will be sufficiently large for the outside stringer.

The weight of the two inner stringers is 235 pounds each and of the outer stringer it is 177 pounds.

In order to elevate the sidewalk above the street level by about 7 inches, the web of the cantilever end of the floor beam will be made 3 inches higher than the main portion of web.

Joists for 20 Foot Panel.

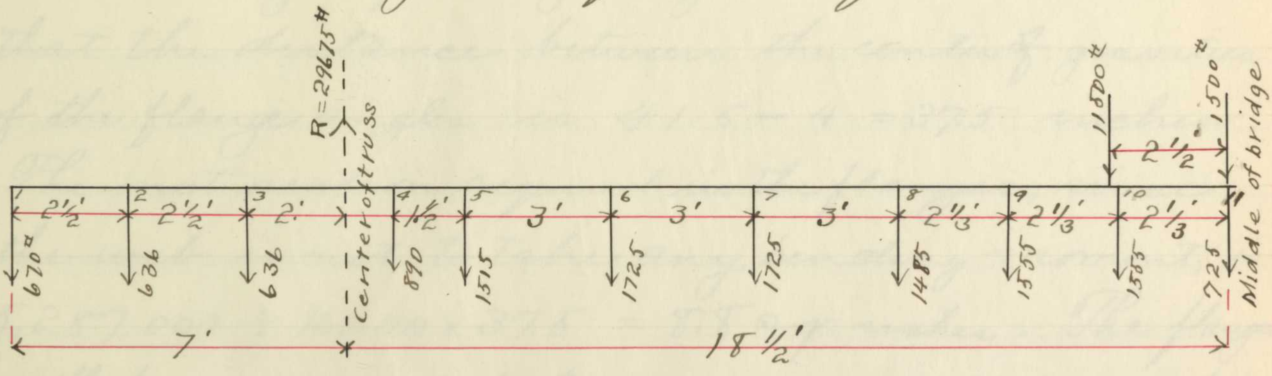
By a process similar to the above it was found that an 8" 20¼ lb. I is required for the two inner joists. An 8" 16.25 lbs. [will be used for the outer joist.

The web of the cantilever portions of the two floor-beams adjacent to the 20 foot panel is to be made one inch higher than the web of the main part of beam.

Design of Floor-Beams.

It is assumed that there is no live load on the sidewalks. A street roller is assumed to be at the middle of the floor-beam and in such a position as to give a maximum bending moment at the middle of the beam. The weight of the floor-beam is assumed at 140 pounds per foot.

Diagram of Loading.



The loads suspended from the line are the dead loadings and those above the line are the live loads. The reaction is shown as acting in the center of the truss.

<u>Moments</u> of <u>Loads</u> and R. About Center of Beam.

Moment of R = 550,000 foot pounds.

" " loads to the right of R = 131,315 " ".

" " weight of beam = 24,000 " "

" " loads to the left of R. = 45,900 " "

The combined bending moment at middle is (550,000 + 45900) − (131,315 + 24000) = 440,585 foot pounds or 5,287,000 inch pounds.

The economic depth of floor-beam from (Johnson's Modern Framed Structures, page 300) is $1.41 \sqrt{M \div ft}$. in which M = bending moment at middle of beam. f = allowable unit stress in flanges and t = thickness of web. f in this case $= 10000 \left(1 + \frac{min}{2 max}\right) = 16200$ pounds per sq. inch.

<u>Depth</u> $= 1.41 \sqrt{5{,}287{,}000 \div 16200 \times 3/8} = 41.5$ inches.

Design of Flange Angles. It will be assumed that the distance between the center of gravities of the flange angles is 41.5 − 4 = 37.5 inches. The net section required in the flanges, since the web is not to take any bending moment, is 5,287,000 ÷ 16200 × 37.5 = 8.8 sq. inches. The flanges will be composed of two 6" × 4" × 9/16" angles, with the four inch leg attached to the web of beam.

The gross area of angles is 10.62 sq. inches.

The area to be deducted for rivets is <u>1.12 " "</u>

Net area of section = 9.5 sq. inches.

The actual unit stress = 5,287,000 ÷ 9.5 × 37.5 = 14800 pounds.

<u>Pitch of Rivits in Flanges</u>. The allowable shear on 7/8 inch rivits is 4500 pounds. The allowable bearing value of 7/8 inch rivit on 3/8 inch plate is 4920 pounds. By the formula (Johnson's Modern Framed Structures, page 302) pitch = rh ÷ S. From this formula the following pitches have been found for the lower flange;-

The pitch 0 feet to 6 feet from the middle is 6 inches.
" " 6 " " 10 " " " " " 5 " .
" " 10 " " 15 " " " " " 4 " .
" " 15 " " 18 " " " " " 3 " .

The pitch in the upper flange will be 4 inches.

<u>End Shear</u>. The maximum shear will occur when there is a live load of 100 pounds per square foot on the roadway. The shear is equal to the live load plus the weights of beam, stringers and flooring, or to 13.75 × 18.50 × 100 + 2500 + 11,165 = 39100 pounds. The number of rivits required in web of floor beam = 39100 ÷ 4500 = 9, the allowable stress per rivit being 4500 pounds. The allowable shear on web plate = $6500(1 + \frac{min}{2\,max})$ = 7550 pounds per square inch.

Gross area of plate = 41.5 × 3/8 = 15.56 sq. inches.

Area of rivit holes to be deducted is 9 × 3/4 = 3.37 sq. inches

The net area of web plate is $15.56 - 3.37 = 12.19$ square inches. The actual unit shearing stress is $39100 \div 12.19 = 3200$ pounds. The connecting angles to be used will be 3"×3"×3/8".

Weight of Floor-Beam.

Name of Piece	Numbers of Pieces.	Length in Feet.	Cross-Section	Weight per Foot	Weight. Pounds.
Web plate	1	51	3/8"×41.5"	52.9	2700
Flange angles	4	51	4"×6"×9/16"	18.1	3700
Connecting Angles	4	3.5	3"×3"×3/8"	7.2	100
Rivets	800		7/8" ⊙	24.3 per. C.	194.
					Total 6694

Design of Lateral Trusses.

In computing the stresses in the lateral trusses, it will be assumed that the bridge acts as two simple spans.

Stresses in Upper Lateral System.

When the bridge is closed, the wind on the upper chord will be taken down to the abutments by the end posts and the tower posts. When the draw is open, the wind will travel along the upper laterals to the tower posts, down which it will go to the center pier.

<u>Dead Load</u>. The dead load on the top chord is 150 pounds per foot, or $150 \times 13.75 = 2060$ pounds per panel. For the middle panel the dead loading is 150×20 or 3000 pounds. The stress diagrams for the upper lateral system is shown on page 18.

Sections for Upper Lateral System.

The allowable unit tensil stress $= 15000 \left(1 + \frac{min}{2\ max}\right)$

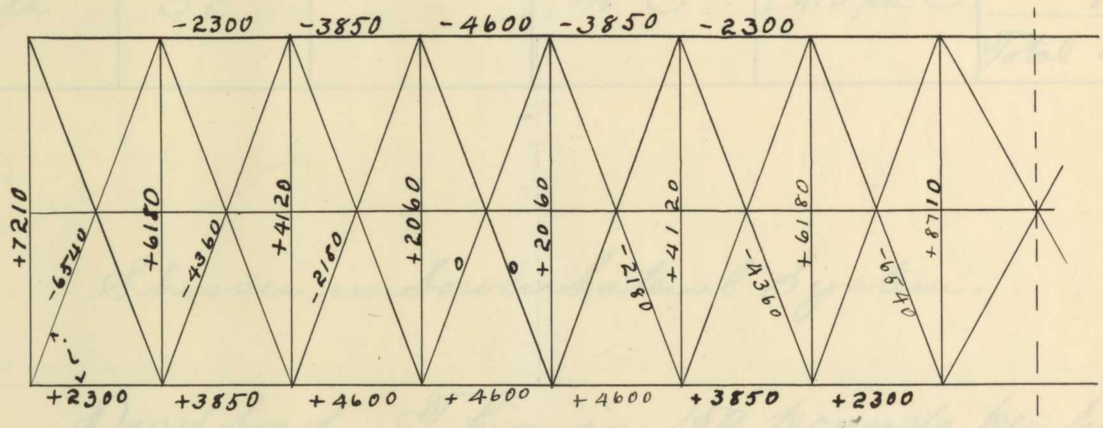
PLATE III.

STRESSES IN UPPER LATERALS.

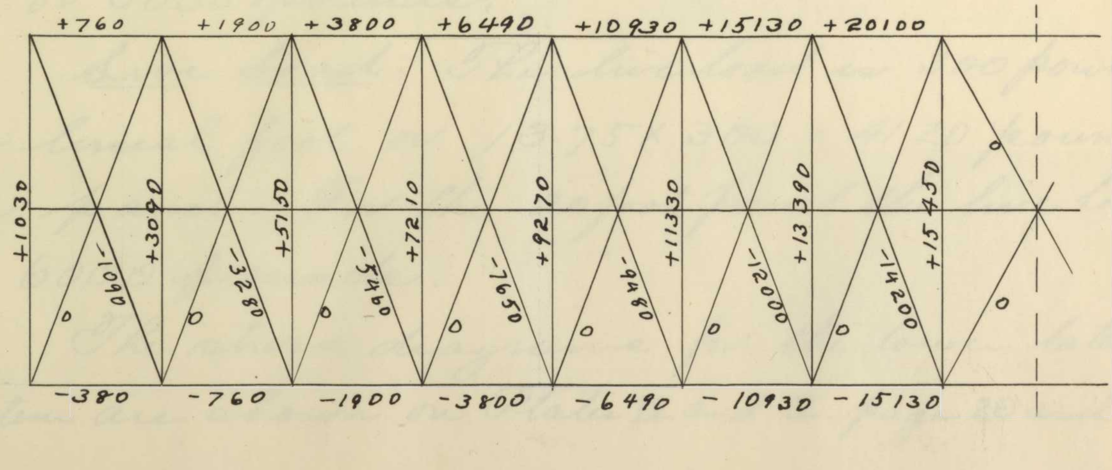
DATA

$i = 20°30'$ $Tan.i = .372$ $Sec.i = 1.060$ $Sin.i = .350$ $Cos.i = .936$

STRESS DIAGRAM-BRIDGE CLOSED



STRESS DIAGRAM-BRIDGE OPEN.



It is seen from the stress diagrams that 19. sections one inch square are sufficient, but since stiff lateral systems are preferable, 3" x 3" x 3/8" angles will be used for the tension members.

Weight of Upper Lateral Tension Members per Panel.

Name of Piece	Number of Pieces	Length in Feet	Cross-Section	Weight per Foot.	Weight Pounds.
Angles	2	39.4	3 x 3 x 3/8"	7.2	565
"	4	1.0	"	"	29
Rivits	32		7/8" ⊙	24.3 per C	8
					Total 602

Stresses in Lower Lateral System.

Dead Load. This is 150 pounds per lineal foot or 13.75 x 150 = 2060 pounds per panel. For the middle panel the dead load is 20 x 150 or 3000 pounds.

Live Load. The live load is 300 pounds per lineal foot or 13.75 x 300 = 4120 pounds per panel. For the 20 foot panel the live load is 6000 pounds.

The stress diagrams for the lower lateral system are shown on Plates IV and V page 20 and 21

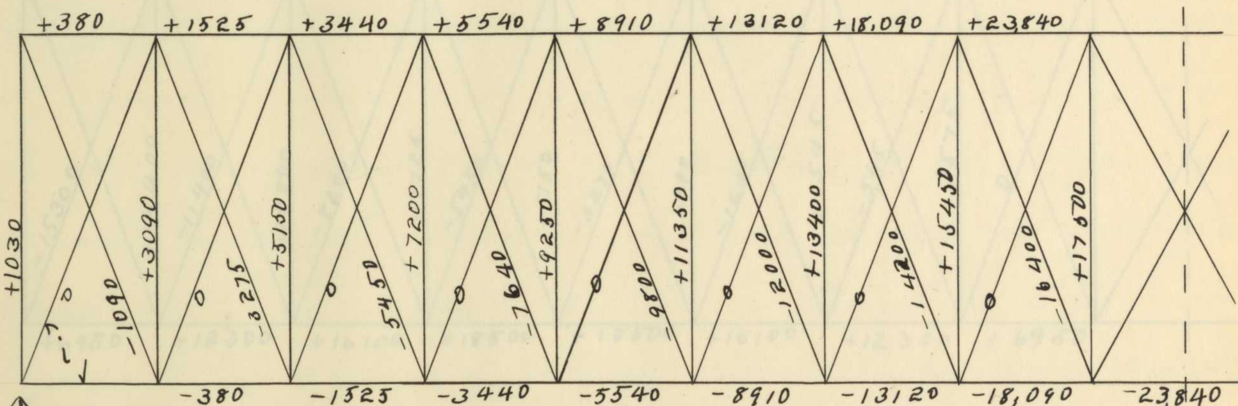
PLATE IV

STRESSES IN LOWER LATERALS

DATA

$i = 20°30'$ $\quad Tan\, i = .372$ $\quad Sec\, i = 1.060$ $\quad Sin\, i = .350$ $\quad Cos\, i = .936$

STRESS DIAGRAM FOR DEAD LOAD. BRIDGE OPEN.



STRESS DIAGRAM FOR DEAD LOAD BRIDGE CLOSED

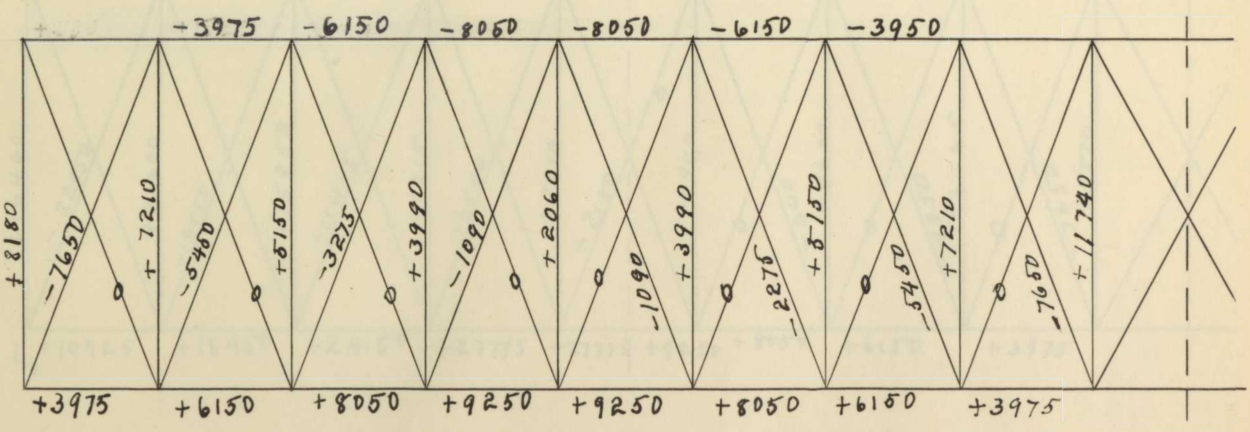
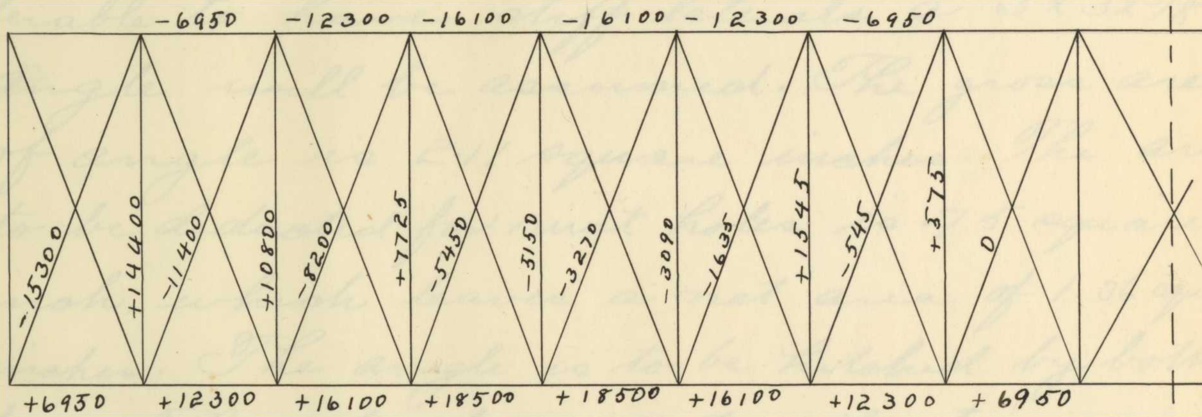


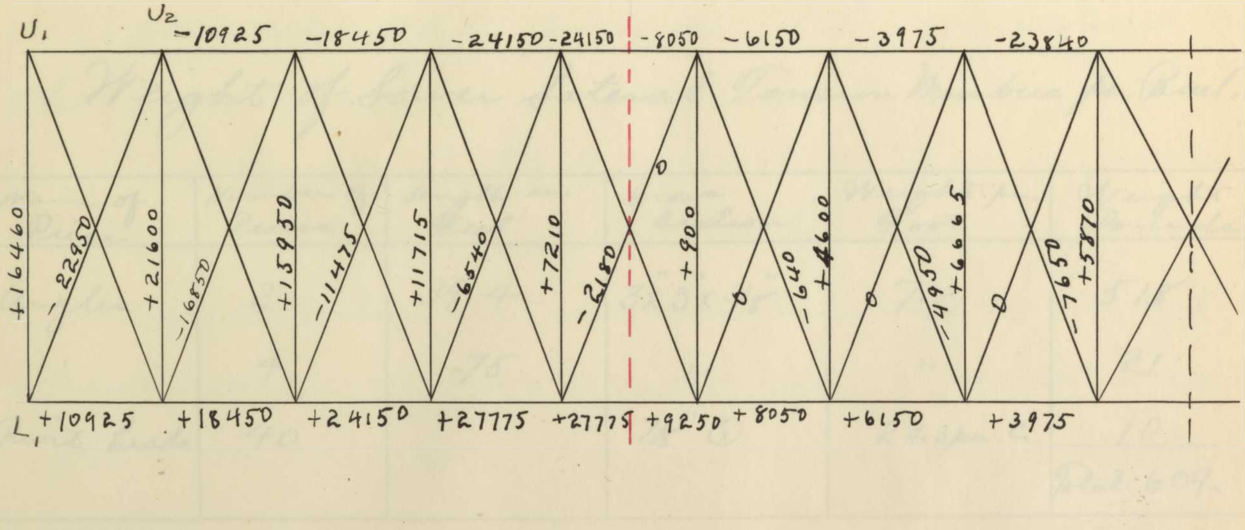
PLATE V.

STRESSES IN LOWER LATERALS.

STRESS DIAGRAM FOR LIVE LOAD.



MAXIMUM AND MINIMUM STRESSES.



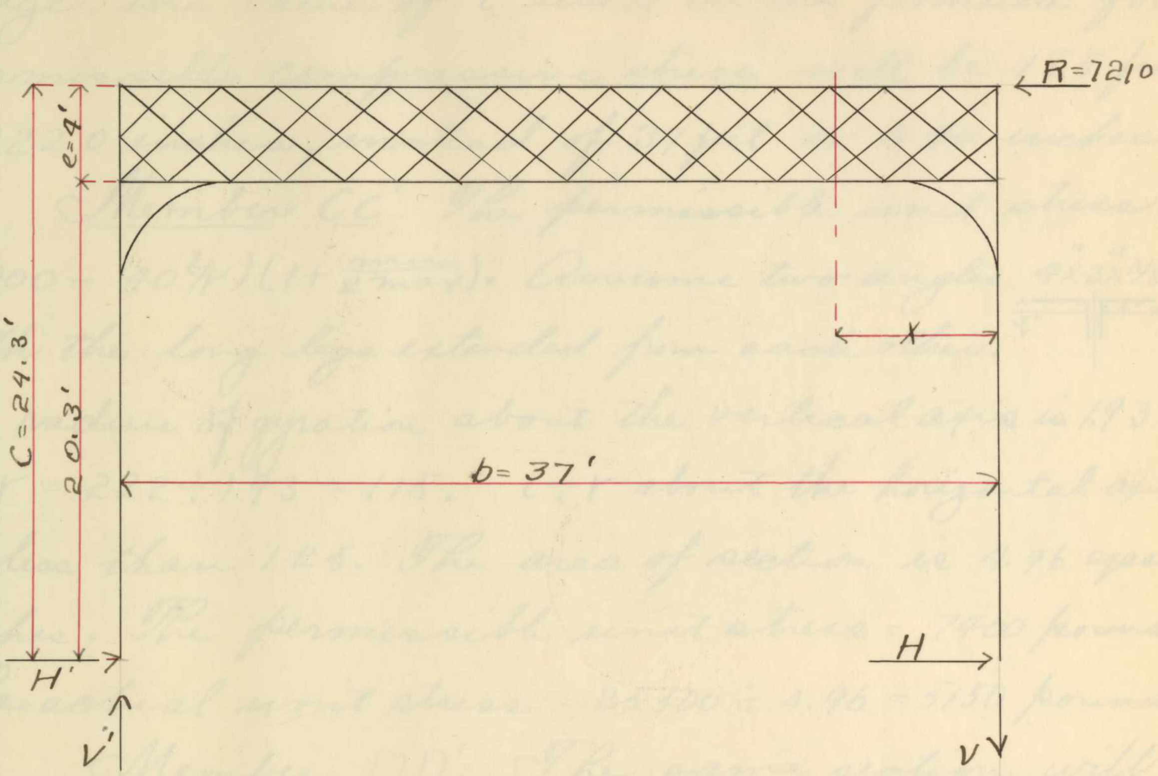
Sections for Lower Lateral System.

Member L_1U_2. The maximum stress is 22950 pounds. The minimum is 7650 pounds. The permissible unit tensil strength is $15000(1+\frac{min}{2\,max}) = 17400$ pounds. The net section required to take the stress is $22950 \div 17400 = 1.3$ square inches. Since it is preferable to have stiff laterals a $3'' \times 3'' \times 3/8''$ angle will be assumed. The gross area of angle is 2.11 square inches. The area to be deducted for rivit holes is .75 square inch, which leaves a net area of 1.36 square inches. The angle is to be hitched by both legs. The actual unit tensil stress will be 16850 pounds. The same section is to be used for all the other tension members.

Weight of Lower Lateral Tension Members per Panel.

Name of Piece	Number of Pieces	Length in Feet	Cross-Section	Weight per Foot	Weight Pounds.
Angles	2	39.4	$3'' \times 3'' \times 3/8''$	7.2	518
"	4	.75	"	"	21
Rivit heads	40		$7/8''$ ⊙	24.3 per C.	10
					Total 609.

Design of Portal.

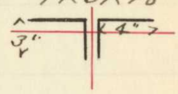


Wind Load. The wind pressure R which the portal must transmit to the abutments consists of the wind load upon one half of the upper chord or $3\frac{1}{2} \times 2060 = 7210$ pounds. It is assumed that the end post is not fixed.

Stresses in Portal. $H = R \div 2 = 7210 \div 2 = 3605$ lbs. $V = V' = (7210 \times 24.3) \div 37 = 4740$ lbs. Stress in CC' (by formula in Johnson's Modern Framed Structures page 112) $= \frac{R}{2} + \frac{V}{e}\left(\frac{b}{2} - x\right)$, ($x = 0$ in this case) $= 7210 \div 2 + (4740 \times 37) \div (2 \times 8) = 25{,}500$ lbs. Stress in DD' $= Hc \div e - Vx \div e = 3605 \times 24.3 \div 4 = 21660$ lbs. ($x = 0$).

Design of Sections. Since a strut will extend longitudinally through the middle of the bridge, the value of l used in the formula for permissible compressive stress will be 18.5 feet or 222.0 inches, instead of 37 feet or 444 inches.

Member CC'. The permissible unit stress = $(12000 - 40\,l/r)(1 + \frac{min}{2\,max})$. Assume two angles 4"x3"x3/8" with the long legs extended from each other.



The radius of gyration about the vertical axis is 1.93. $l \div r = 222 \div 1.93 = 115$. $l \div r$ about the horizontal axis is less than 125. The area of section is 4.96 square inches. The permissible unit stress = 7400 pounds. The actual unit stress = 25500 ÷ 4.96 = 5150 pounds.

Member DD'. The same section will be used here as was used for CC'.

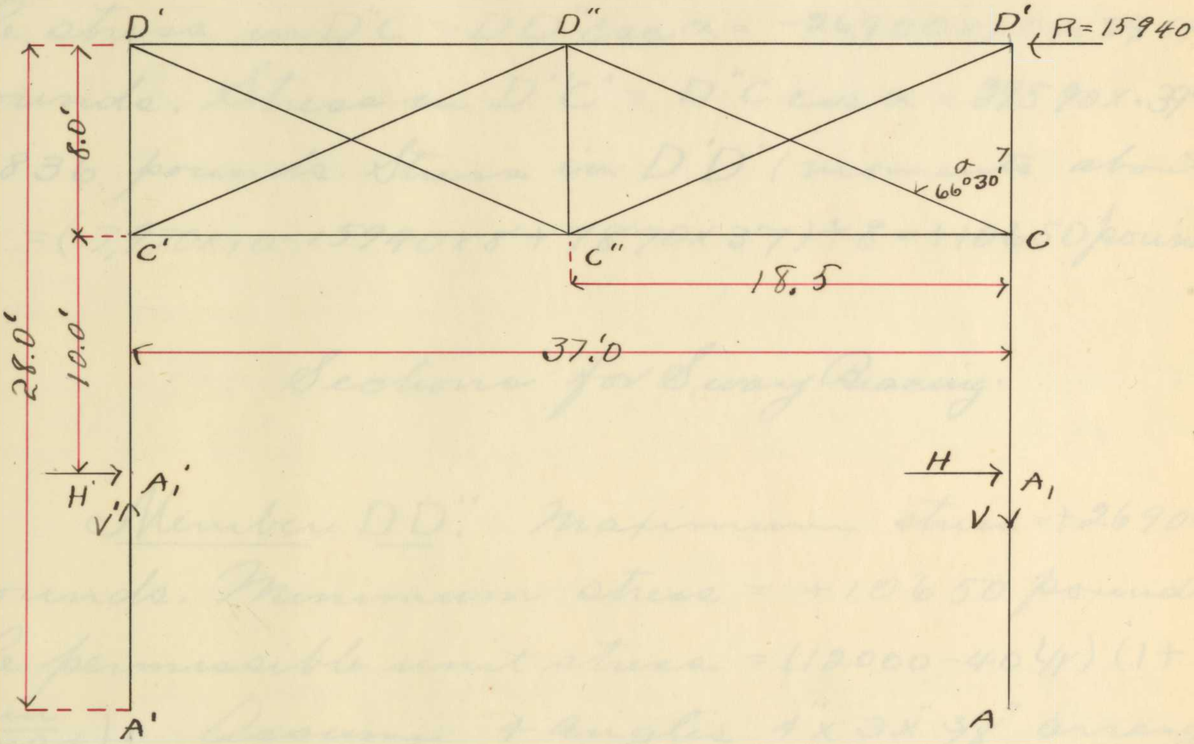
The web members will be made of 3"x3"x3/8" angles.

Weight of Portal.

Name of Piece.	Number Pieces	Length in Feet	Cross Section	Weight per Foot.	Weight Pounds.
Angles	4	37	4"x3"x3/8"	8.5	1260
"	30	5.6	3"x3"x3/8"	7.2	1210
"	4	5.0	"	"	144
Rivet heads	400		7/8" ⊙	24.3 per C.	97
					Total 2711

Design of Sway Bracing.

Sway Bracing for Middle Panel.

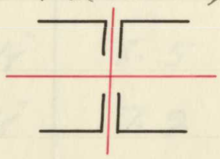


Wind Load. The wind pressure R applied at the top of the post AD, is a maximum when the bridge is open and equals the pressure on one arm of the bridge plus one half the wind one the middle panel or $(7 \times 13.75 + 10) \times 150 =$ 15940 pounds. It is assumed that the post is fixed at the lower end and that the reactions H and H' are applied at a point 10 feet above the bottom of of post.

Stresses in Sway Brace. $H = H' = R \div 2 =$

15940 ÷ 2 = 7,970 pounds. Taking moments about A_1', V = (15940×18) ÷ 37 = 7780 pounds. The stress in DD'', (moments about C) = (15940×8×7970×10) ÷ 8 = 26900 pounds. The stress in CC'' (moments about D'') = (−7970×18×7780×37) ÷ 8 = 18050 pounds. The stress in D''C = DD'' csc α = −26,900×1.09 = 29,590 pounds. Stress in D''C'' = D''C cos α = 29590×.39 = 11,836 pounds. Stress in D'D'' (moments about C') = (−7,970×10 − 15940×8 + 7870×37) ÷ 8 = +10650 pounds.

Sections for Sway Bracing.

Member DD''. Maximum stress = +26900 pounds. Minimum stress = +10650 pounds. The permissible unit stress = $(12000 - 40\,l/r)\left(1 + \frac{min}{2\,max}\right)$. Assume 4 angles 4" × 3" × 3/8" arranged as shown in figure.  The vertical distance between the centers of gravity of the angles is 6 inches, this makes the depth 8 inches. The least r is about the vertical axis and is 1.93. The permissible unit stress is 8900 pounds. The actual unit stress = 2700 pounds.

Member D''C''. Stress = 11,360 pounds. Assume one angle 3" × 3" × 3/8". r = .91 l = 96 inches. The per-

missible unit stress $= (12000 - 40\frac{l}{r}) = 7720$ pounds.
The actual unit stress $= 11360 \div 2.11 = 5400$ pounds.

Member D"C: Maximum stress $= -29590$ pounds. Minimum stress $= 0$. The permissible unit stress $= 15000(1 + \frac{\min}{2\max}) = 15,000$ pounds. The section required $= 29,590 \div 15000 = 1.97$ sq. inches. One angle 4"×4"×3/8" will be used. The gross area $= 2.86$ sq. inches. Deducting 0.75 sq. inch for rivets leaves a net area of 2.11 sq. inches. The actual unit stress $= 14,000$ pounds.

Member CC": The same section will be used for this section as was used for DD".

Weight of Sway Brace.

Name of Piece	Number of Pieces.	Length in Feet.	Cross Section	Weight per Foot.	Total Weight Pounds.
Angles	8	37	4"×3"×3/8"	8.5	2500
"	1	8'	3"×3"×3/8"	7.2	57
"	4	20	"	"	575
Lacing	74	1.0	3/8"×2"	2.5	185
Rivets	150		7/8" O	24.3 per C.	36
					Total 3354

Intermediate Sway Bracing.

As the sway braces will not take any wind stress, but is only to keep the trusses rigid they will be made of minimum sections. The upper strut will be made of the same sections as for the middle bracing. The lower strut will be two - 3" x 3" x 3/8" angles. The vertical post in middle of bracing will consist of one 3" x 3" x 3/8 angle. The tension members are to be made of 1"x1" rods. As the longitudinal struts through the middle of the bridge are used only for helping the rigidity of the struts in the sway bracing, they will be sufficiently strong if each is made of two 3" x 3" x 3/8" angles. The depth of bracing will be 4 feet.

Weight of Sway Bracing.

Name of Piece	Number of Pieces	Length in Feet	Cross Section	Weight per Foot.	Weights Pounds.
Angles.	4	37	4"x3"x3/8"	8.5	1260
"	2	37	3 x 3x3/8	7.2	530
"	4	13.75	"	"	396
"	1	4	"	"	28
Rods.	4	19	1"x1"	3.4	258
Turnbuckles	4			4.75	19
Lacing					50
Rivets	100		7/8" O	24.3 per. C.	25
					Total 2561

Design of Main Trusses.

Computation of Stresses.

Dead Load. To obtain the dead load, the floor system was first designed, and from this the weight suspended from the lower chord pins was found. To determine an approximate weight of the trusses the weight of the Wells Street bridge was computed. The street traffic on the Wells Street and Dearborn Avenue bridges is the same in both amount and nature. The Wells Street bridge was presumably designed under specifications very similar to those used in the present design.

The Wells Street bridge was designed in December 1887. It is a steel structure of three Pratt trusses, spaced 21 feet between centers. The depth of trusses at the ends is 20 feet and at the middle 28 feet. The panel length is 14 feet. Cantilevers projecting from the floor-beams carry the sidewalks, which are 7 feet wide. The pavement is the same as that on the bridge to be designed. The joists are 12 inches deep and weigh 31.5 pounds per foot.

The weight per foot per truss of the

Wells Street bridge is 690 pounds, or for the
three trusses 2070 pounds. The weight of the other parts is approximately the same as for the bridge to be designed.

The weight per foot per truss of the proposed bridge was assumed at 800 pounds. The above method of finding the weight of trusses is very crude, but it is the best available.

Assumed Weight of Proposed Bridge.

Joists	3500	pounds per panel per truss.
Floor-Beams	3350	" " " "
Joists for sidewalks	650	" " " "
Flooring on roadway	7800	" " " "
" " sidewalk	1150	" " " "
Nailing pieces and railing	370	" " " "
Wheel guards, rails, spikes, etc.	1475	" " " "
Lateral trusses and sway braces	1900	" " " "
Truss	11020	" " " "

Total weight = 31215 pounds per panel per truss. The load on the joints adjoining the 20 foot panel was assumed at 36300 pounds.

Reactions. The following formulas, which are demonstrated in Johnson's Modern Framed

Structures page 194, were used in finding the reactions. They are based on the assumption of a uniform moment of inertia.

$$R_1 = P[1 - K - \{l \div (4l + 6l_2)\}(K - K^3)]$$

$$R_4 = -P[\{l \div (4l + 6l_2)\}(K - K^3)]$$

R_1 = reaction at end of loaded arm.

R_4 = " " " " unloaded " .

P = load applied successively at each joint in loaded arm. In this case P = 1000 pounds.

K = the ratio of the distance from R_1 to loaded joint, to the length of one arm or 110 feet.

l = length of one long arm = 110 feet.

l_2 = " " middle panel = 20 feet.

Table III gives the values of K, $K - K^3$, R_1 and R_4 for different positions of the load P. The values given for R_1 and R_4 have been calculated by assuming 1000 pounds at each joint in succession. The web members Hj and Ii (Plate II) have been disregarded in getting the reactions, as they only perform the function of stiffeners, when the bridge is open. These members recieve no definite stresses.

Table III.

Reactions.

K	K − K³	R₁ Pounds	R₄ Pounds.
1.25	.118	+851	−24.2
.250	.234	+704	−46.0
.375	.322	+562	−63.2
.500	.375	+426	−73.7
.625	.381	+300	−75.0
.750	.328	+186	−64.4
.875	.205	+86	−39.4
		+3175	−385.9

<u>Stresses</u>. The stresses were computed for the following positions and loadings.

Case A. Bridge open dead load only acting. This gives a maximum tension in the upper chord and the maximum compression in the lower chord.

Case B. Bridge closed, ends raiseds, dead load only acting. This gives the maximum compression in the upper chord, and the maximum tension in the lower chord due to dead load.

Case C. Bridge closed, ends raised, one arm fully loaded the other unloaded. This gives the maximum compression in the upper chord and the maximum tension in the lower chord due to the live load.

Case D. Bridge closed, ends raised, one arm fully loaded the other partly loaded. This gives the maximum tension in the upper chord and the maximum compression in the lower chord, due to the live load on both arms.

Table IV gives the chord stresses for the above cases, and also the maximum positive and negative stresses. Table V gives the web stresses for the above cases, and also the maximum positive and negative stresses.

TABLE IV.

STRESSES IN CHORDS.

MEMBER	DEAD LOAD.		LIVE LOAD.		MAXIMUM	
	CASE A. POUNDS.	CASE B. POUNDS.	CASE C. POUNDS.	CASE D. POUNDS.	+ POUNDS.	− POUNDS.
AB	−9,900	+5,800	+76,000	−9,300	134,000	19,200
BC	−38,200	+94,500	+120,000	−17,700	214,500	55,900
CD	−84,500	+99,000	+139,000	−25,500	238,000	110,000
DE	−142,000	+88,500	+136,000	−31,900	224,500	173,000
EF	−216,000	+63,300	+112,000	−37,900	175,300	252,900
FG	−294,000	+23,000	+72,300	−43,600	95,300	337,600
GH	−386,000	−30,100	+18,550	−46,600	18,550	432,600
HI	−480,000	−92,200	−84,400	−88,700	0	568,700
ab	+8200	−64,700	−75,600	+9,300	17,500	140,300
bc	+39,200	−64,700	−75,600	+17,600	56,800	140,300
cd	+83,700	−89,500	−166,000	+25,000	108,700	255,500
de	+140,000	−149,000	−139,000	+31,750	171,750	288,000
ef	+216,000	−64,700	−225,000	+37,800	253,800	289,700
fg	+292,000	−23,000	−62,800	+43,400	335,400	85,800
gh	+382,000	−30,200	−18,000	+48,400	430,400	48,200
hi	+480,000	+93,100	+48,000	+106,000	586,000	0
ij	+480,000	+93,100	+48,000	+89,400	569,000	0

TABLE V.

STRESSES IN WEB MEMBERS.

MEMBER	DEAD LOAD		LIVE LOAD		MAXIMUM.	
	CASE A. POUNDS.	CASE B. POUNDS.	CASE C. POUNDS.	CASE D POUNDS.	+ POUNDS.	— POUNDS.
Aa	−17,600	+103,000	+133,000	−16,800	236,00	34,600
Ab	+13,700	−30,800	−35,000	0	13,700	65,800
Bb	−53,300	0	0	−23500	0	76,800
Bc	+42,300	+46,000	+48,700	+18,600	94,700	0
Cc	−85,200	0	0	−37,200	0	122,400
Cd	+63,800	0	+29700	+32,000	95,800	0
Dd	−116,500	−19,300	0	−55,200	0	171,700
De	+95,000	+15,800	0	+46,000	141,000	0
Ee	−144,000	−54,700	−18,500	−79,500	0	223,500
Ef	+119,500	+44,500	+15,400	+66,000	175,500	0
Ff	−170,500	−85,500	−8,250	−105,500	0	275,000
Fg	+144,000	+72,300	+7,000	+88,600	232,600	0
Gg	−196,000	−115,500	0	−133,500	0	339,500
Gh	+165,500	+98,000	+2,120	+114,000	279,500	0
Hh	−221,800	−145,000	0	−163,000	0	384,800
Hi	+189,000	+123,400	0	+140,500	329,500	0
Ac	0	−55,800	−85,000	0	0	140,000
Bd	0	−18,600	−55,500	0	0	74,100
Ce	0	0	−23,400	0	0	23,400

Design of Truss Members.

Tension Members.

Member Bb. The maximum stress is 76,400 pounds. The minimum stress is 0 pounds. The permissible unit tensil stress is 10,000 pounds. Area required = 76,400 ÷ 10,00 = 7.64 sq. inches. Two bars 4 in. x 1 in. and weight per foot 13.6 pounds will be assumed. The stress due to the bending effect of the weight of the member is (by the formula page 155 Johnson's Modern Framed Structures) $f = My \div (I + \frac{Pl^2}{10E})$.

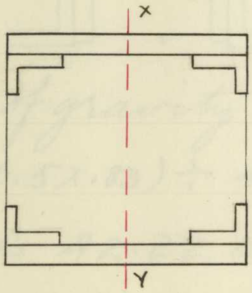
$$f = (1/8 \times 13.6 \times 25.21 \times 13.75 \times 12 \times 2) \div \left(5.33 + \frac{38,200(25.21 \times 12)^2}{10 \times 28,000,000}\right)$$

= 890 pounds. But since this stress is less than 10% of the permissible unit stress, it will be neglected. The actual unit tensil stress is 76,400 ÷ 8 = 9,800 pounds.

In a similar manner the section for the remaining tension members were computed. The sections for the different members of the truss are shown on Plate VI.

Compression Members.

Member Hi tower post. The maximum stress is 329,000 pounds. Minimum stress is 123,400 pounds. The permissible unit compressive stress $= (11000 - 40\,l/r)(1 + \frac{min}{2\,max}) = 13,100 - 47.5\,l/r$. $l = 336$ inches. Assume a section as shown. Web plate is



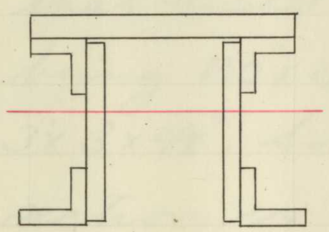
15" x 3/4". Angles 3" x 4" x 5/8" having the 4 inch leg rivited to the web plate. The area of the section is 38.42 sq. inches. The weight per lineal of section is 131.0 pounds.

I about axis $XY = 1133.36$ $r = \sqrt{I \div A} = 5.42$. The permissible unit stress $= 13100 - 47.5\ 336/5.42 =$ $13100 - 2950 = 10,150$ pounds. The actual unit stress $= 329,000 \div 38.42 = 8560$ pounds.

In a similar manner the sections for the remaining compression members were found and are shown on the same plate as the tension members.

Tension and Compression Members.

End Post. The maximum stress of greater kind $= +236,000$ pounds. Maximum stress of lesser kind = -34.600 pounds. The permissible unit compressive stress $= (10{,}000 - 40\,l/r)\left(1 - \frac{1}{2}\,\frac{\text{max stress of lesser kind}}{\text{'' '' '' greater ''}}\right)$ $= 8750 - 35\,l/r$. The permissible unit tensil stress $= 11{,}000\left(1 - \frac{1}{2}\,\frac{\text{max stress of lesser kind}}{\text{'' '' '' greater ''}}\right) = 11{,}000 \times .875 = 9625$ pounds. Assume a section as shown. Cover plate 3/8" x 20".



Web plates 3/4" x 16". Upper angles 3" x 3" x 3/8". Lower angles 4" x 3" x 1/2". The distance above the bottom of the section at which the center of gravity lies is $\Sigma \frac{a\,z}{A} = (16.43 \times 7.5 + 4.22 \times 15.41 + 24 \times 8.1 + 6.5 \times .83) \div 42.22 = 9.3$ inches. The area of section is 42.22 sq. inches. The moment of inertia about gravity axis $= 1553.02 = I$ Radius of gyration $(r) = \sqrt{\frac{I}{A}} = 6.2$. The stress due to bending effect of the weight of member $= f = \frac{My}{I - \frac{PL^2}{10E}}$. The weight per foot = 143.7 pounds. The length of end post = 24.29 feet. $f = (\frac{1}{8} \times 143.7 \times 24.29 \times 13.75 \times 12 \times 8) \div (1553.02 - \frac{236{,}000(24.29 \times 12)^2}{.10 \times 28{,}000{,}000}) = 380$ pounds. The permissible unit stress = 7060 pounds. Actual unit stress = 5500 pounds. Since f is less than 10% of the allowable unit stress it will be neglected.

Top Chord.

Member AB. The maximum stress of

greater kind = (134,000 + 2300) = + 136,300 pounds. Maximum stress of lesser kind = − 19,200 pounds. The allowable compressive stress = (10250 − 37 $\frac{1}{4}$ r) pounds. The allowable tensil stress = 10250 pounds. The section will be of similar design to that used in the end post. The cover plate being 20" x 3/8". Web plates 16" x 3/8". Upper angles 3" x 3" x 3/8". Lower angles 3" x 3" x 3/8". The area of section is 28.22 square inches. Weight per foot = 93. pounds. The center of gravity is 11.1 inches above the bottom of the section. $I = 911.04$ $r = \sqrt{\frac{I}{A}} = 5.5$. The allowable compressive stress = 9140 pounds per sq. inch. The actual compressive stress = 4830 pounds per sq. inch. The stress due to bending = $f = My \div (I - \frac{Pl^2}{32E}) = 300$ pounds, and since this is less than 10% of the allowable stress it will be neglected. The other members of the upper chord were computed in a similar manner.

Lower Chord.

<u>Member ab</u>. Maximum stress of greater kind = −(140,300 + 10,925) = −151,225 pounds. The maximum stress of lesser kind = +17,500

pounds. The allowable unit tensil stress
= 10360 pounds. Allowable compressive stress = 10360 − 37.7 l/r. Assume a section similar to the end post but substituting lacing bars for the cover plate. The web plates will be 16" x 3/8". The four angles are to to be 3" x 3" x 3/8". The gross area of section = 20.44 square inches. The area to be deducted for rivits = 4.5 sq. inches. The net area = 15.94 sq. inches. The weight per foot = 68.6 pounds. The direct stress is 9550 pounds. Stress due to bending = 200 pounds, which is less than 10% of the permissible stress and will be neglected. In a similar manner the remaining members of the lower chord were computed.

Estimate of Weight of Truss.

Member	Weight per Foot	Total Weight.
Aa	143 pounds.	4000 pounds.
AB	93 "	1480 "
BC	131 "	2080 "
CD	117 "	1865 "
DE	131 "	2080 "
EF	152 "	2430 "
FG	152 "	2430 "
GH	184 "	2940 "
ab	69 "	1440 "
bc	83 "	1720 "
cd	140 "	2840 "
de	197 "	4035 "
ef	251 "	5200 "
fg	163 "	3370 "
gh	291 "	6025 "
hi.	305 "	6260 "
Bb	27 "	755 "
Cc	43 "	1180 "
Dd	61 "	1830 "
Ee	64 "	1980 "
Ff	77 "	2460 "
	Carried Forward	58400 pounds.

Member	Weight per Foot	Total Weight.
		Brought Forward 54,400 pounds
Gg	46 pounds	1660 pounds
Hh	51 "	1750 "
Ac	41 "	1100 "
Bd	24 "	660 "
Cc	9 "	260 "
Bc	40 "	1200 "
Cd	50 "	1560 "
De	70 "	2300 "
Ef	70 "	2410 "
Fg	90 "	3240 "
Gh	110 "	4130 "
Hi	131 "	5100 "
Ab.	26 "	780 "
	Total	84550 pounds

The weight per foot per truss is therefore $84550 \div 110.0 = 768$ pounds.

Pins.

The sizes of the pins are shown on Plate I. The distance from the centre of the web plate at which the pin should be placed is $l_1 = l - AW \div S$ (Johnson's Modern Framed Structures page 325). For the end post l, $=+1.27$ inches, or the pin is to be placed 1.27 inches above the center of the web plate. The pins of the top chord will be placed the same distance above the middle of the web plate. The lower chord pins will be placed in the center of the web.

Design of Center Pier.

The pier will consist of two parts as shown in Plan, Plate VII. The outer part will have an annular cross-section, and will be wider at the bottom than at the top, the outer face being plumb and the inner one battered. The inner part of the pier will be of the form of a frustum of a cone. The two parts will be entirely disconnected.

The pier will be 42 feet in diameter at the base of masonry. The height is 34 feet. The foundation will be composed of piles. Concrete will be placed between the piles to a depth of two feet below their heads and one foot above. The permissible load per pile according to the specifications is

10 tons. The maximum load comes on the pier when the bridge is loaded with 100 pounds per square foot of floor. The dead load is 533,000 pounds and the live load 825,000 pounds. The total load is 1,358,000 pounds or 679 tons. To carry this load will require 68 piles. The piles will be distributed in the following manner, see Plan, Plate VII. In the circle whose diameter is 40 feet will be placed 30 piles spaced approximately 4 feet on centers; in the 36-foot circle will be placed 25 piles, and in the 30-foot circle 20 piles spaced approximately 4 feet on centers.

The permissible pressure on the masonry is 250 pounds per square inch. It will be assumed that the turntable will rest on a plate one foot wide. The pressure on the masonry will then be 82 pounds per square inch. The pressure on the bottom of the pier will be 82+34 or 116 pounds per square inch. The pressure on the piles is well within the allowable. The pressure on the inner part of the pier is indeterminate, but does not exceed the permissible pressure.

Design of End Pier.

Plate VII shows a foundation plan and a front and side elevation.

The dead load is 171,000 pounds and the live load 192,000 pounds. The total load is 363,000 pounds or 181.5 tons. This will require 19 piles. The piles are to be placed in two rows 5 feet apart. In each row will be placed 15 piles spaced 3½ feet on centers. The pressure on the masonry and on the piles is well within the limits.

Conclusion.

Owing to the limited time which the author has had at his disposal, no attempt has been made to design the details, such as turntable, turning devices and other mechanisms belonging to a swing bridge. For the same reason no attempt has been made to estimate the cost.

Finis.

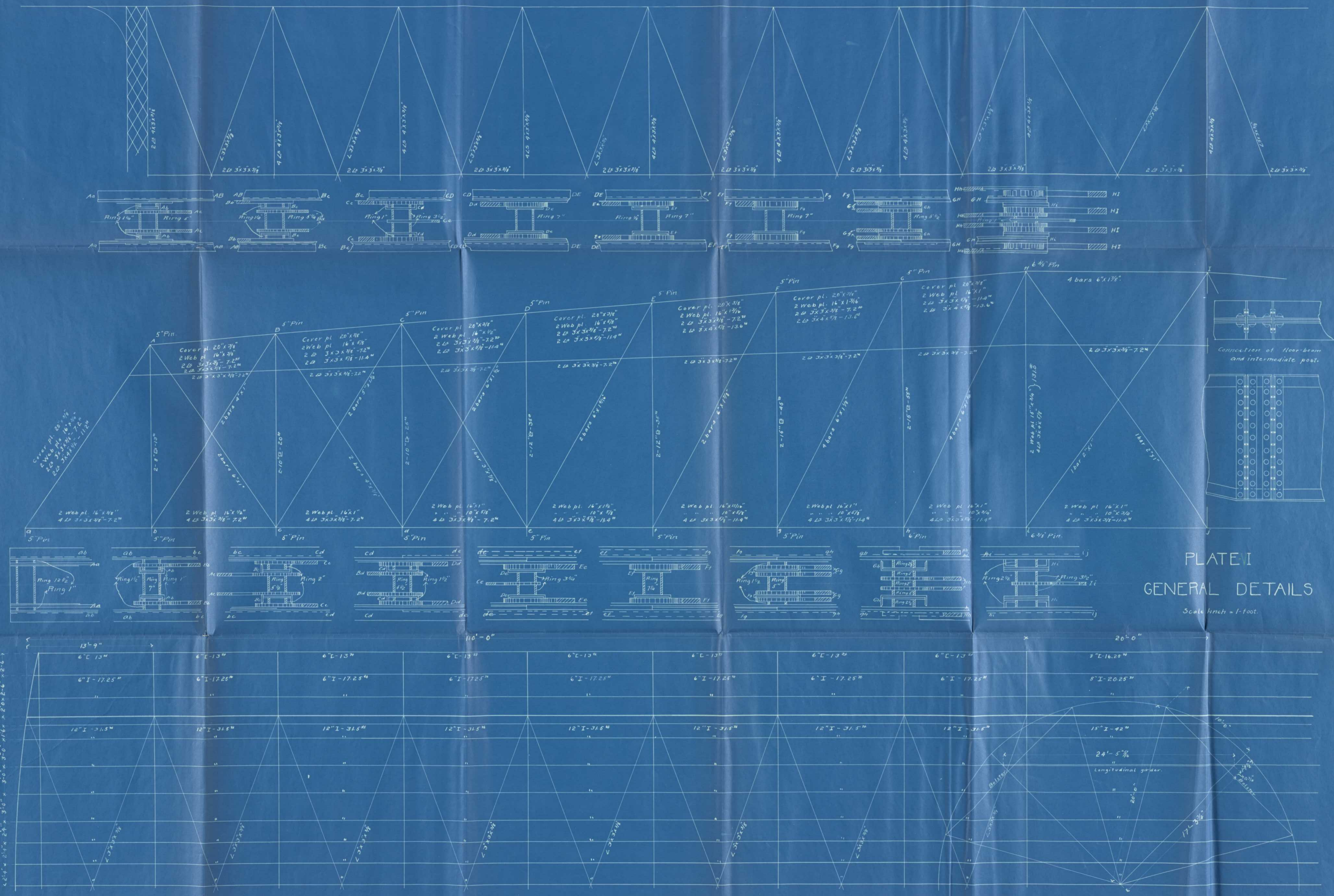
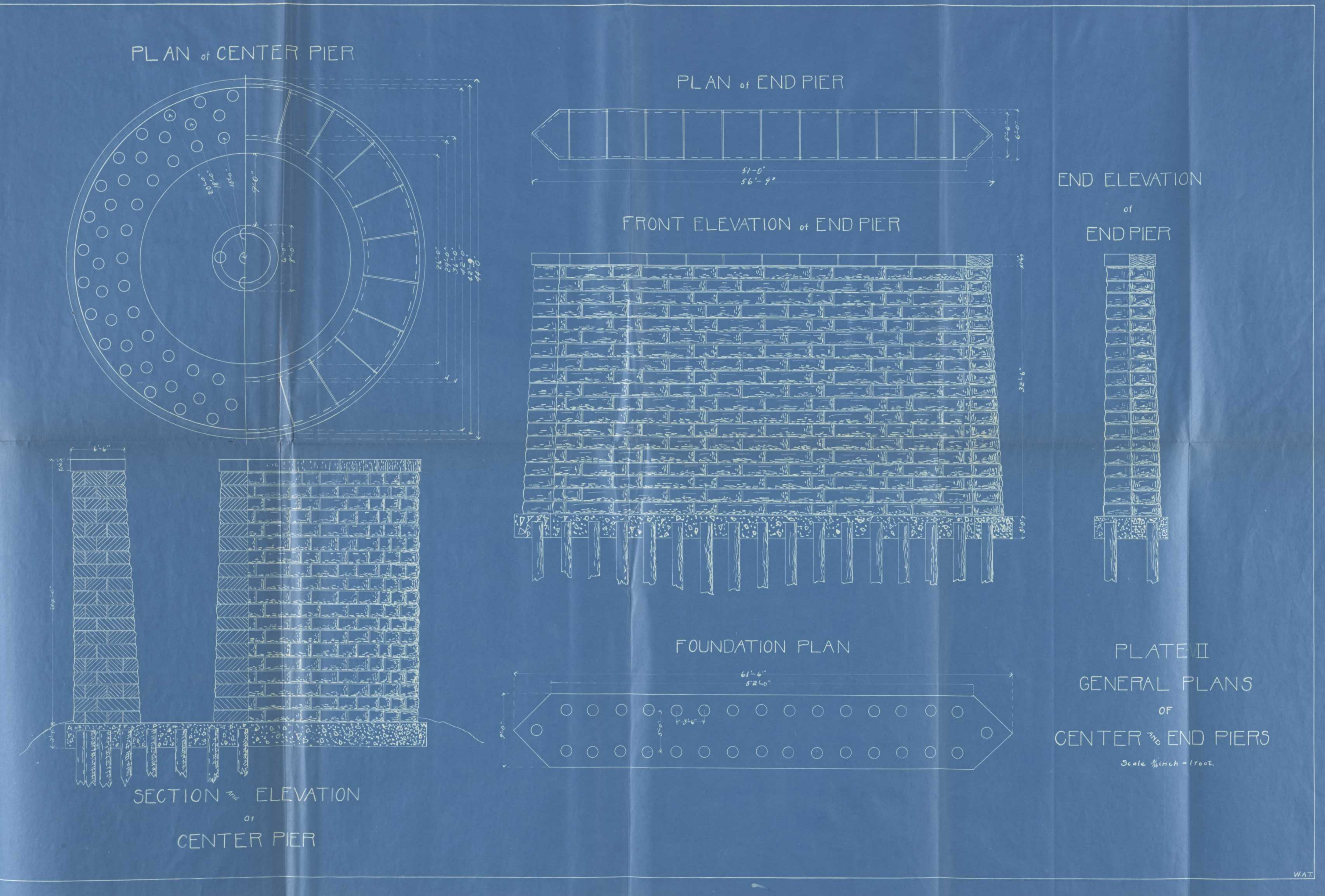


PLATE VI
GENERAL DETAILS
Scale 1-inch = 1-foot.


PLAN of CENTER PIER
PLAN of END PIER
51-0'
56'-9"
FRONT ELEVATION of END PIER
32'-6"
END ELEVATION
of
END PIER
6'-6"
SECTION AND ELEVATION
of
CENTER PIER
FOUNDATION PLAN
61'-6"
52'-0"
3'-6"
PLATE VII
GENERAL PLANS
OF
CENTER AND END PIERS
Scale 3/16 inch = 1 foot.
W.A.T.