Design of Three Spans of a Steel Highway Bridge at New St. Between Bethlehem & So. Bethlehem, Penna.

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Penna.

May /90.

T. A. Straub. '90

Design of Steel Highway Bridge.

Introduction.

The Design of a Steel Highway Bridge is the joint subject for theses of E. H. Beazell & the writer. The latter's part of the work comprises the design of three separate spans; two through spans and one plate girder

The bridge is designed as a new superstructure to take the place of that of the present New St. Bridge between Bethlehem & South Bethlehem. The spans are therefore the same as those of the present. They include five different lengths: one of fifty-five feet; two of one hundred & three feet; one of one hundred & thirteen feet; three of one hundred and twentyone feet; and one of one hundred and fifty-three feet. The latter two are treated in the thesis of E. H. Beazell.

The method of calculation is that taught by Prof. M. Merriman in his "Roofs & Bridges", Part I; and class room lectures. The results only are given; however a general method of deriving them is given in each case. Isami Hiroi's "Plate Girder Construction" is followed in the design of that span.

The through spans being similar in all details excepting the size of the same, only one drawing is made. The sizes of the different pieces is given in the calculations.

Cooper's Highway Bridge Specifications were the ones used.

Design of Plate Girder. Span = 55'
Plate I.

a. Weight of Girder:

Use the formula given by Prof. A. J. DuBois in "Discussion of Weights of Bridges" American Society of Civil Engineers, Feb. 1886.

W = Total external weight in pounds

W_1 = Weight of Plate Girder in pounds.

l = Clear span in feet

d = Depth of Plate Girder in inches

r = Ratio of span to depth both in the same unit

S = Allowable unit stress in lbs per sq inch.

Then will
$$W_1 = \frac{Wlr + 2dSl}{1.2S - lr}$$

$l = 55'$; $d = 4'7"$; $r = 12$; $S = 10000$ lbs

$\therefore W_1 = 10937$ lbs.

b. Dead Load per linear foot of Girder:
This consists of two parts: the floor system & the weight of the girders:—

$$\text{Girder} = \frac{10937}{55} = 200 \text{ lbs per ft.}$$

Floor System: Making a rough approximation on the known position of the stringers, thickness of the plank flooring etc, etc. we have:

$w = 410$ lbs

Total $w = 410 + 200 = 610$ lbs.

c. Live Load per linear foot of Girder.
Total live load at 100 lbs per sq foot, Specifications

$W = 100 \times 35 \times 55 = 192500.$

$w = \frac{192500}{2 \times 55} = 1750$ lbs.

d. Maximum Bending Moments at Panel points due to Dead & Live Loads.

Load the entire bridge for maximum.

$R = 64900$; Panel loads $= 25960$

$M_1 = 571120 = M_4$; $M_2 = 856680$ ft lb $= M_3$.

M_1, M_2, etc refer to the panel points beginning with the left end.

e. Stress at the Panel Points:

$S_1 = \frac{571120}{\frac{55}{12}} + 1\%$ impact $= 137069 = S_4$

$S_2 = \frac{856680}{\frac{55}{12}} + 10\%$ impact $= 205603 = S_3$

f. Sections required in square inches.

Upper flange will equal that of the lower.

At 1 & 4. $a = 11.4\,\square''$

At 2 & 3 $a = 16.7\,\square''$

g. Design of Flange sections:

The lower flange section should be deducted for the rivet holes.

At 1 & 4.

2 L's	$5'' \times 3'' \times {}^{13}/_{16}$	=	$10.3\,\square''$
1 plate	$10'' \times {}^{3}/_{8}''$	=	3.
			$13.3\,\square''$

At 2 & 3

2 L's	$5'' \times 3'' \times {}^{13}/_{16}$	=	$10.3\,\square''$
1 plate	$10'' \times {}^{3}/_{8}''$	=	3
1 "	$10'' \times {}^{1}/_{2}$	=	4.
			$17.3\,\square''$

h. Determination of shear for these sections

At End $V = 51920$; at 1, $V = 25960$
at 2. $V = 000$.

i. Thickness assume & strength investigated
Taking the maximum shear & add 10% for impact. Section required = $11.4''$
Making due allowance for rivets with a 3/8" plate we have a section of $14.5''$ which is large enough.

j. Testing for stiffness according to Cooper's formula $\dfrac{15000}{1+\dfrac{H^2}{3000}} = 1800$, $H = 147$. Therefore

stiffners are required. They will be required particularly at the bearing points. Their position will be seen from the drawing.

K. Design of Lateral & Diagonal Bracing.
wind stress = 150 lb per linear foot.

The diagonals are ~~rods~~ angles. We calculate for these by allowing the wind to blow on one side & finding the loads on the panel points. the ~~rods~~ angles being double. a reverse wind is thus taken into account.

Take angles $3\frac{1}{2}" \times 3" \times \frac{5}{16}"$.

The floor beams will act as diagonal struts.

m. Shearing & Bearing stress of rivets.

We use the $\frac{7}{8}"$ rivet; $\therefore (\frac{7}{8})^2 \times \frac{1}{4} \times \pi \times 9000 = 5300 =$ the allowable shearing stress. For double shear. $s = 10600$; & for bearing stress we have $s = 5000$ hence the last is the one to be considered

n. Rivet Spacing: The limits are $6"$ & $2\frac{5}{8}"$. Two forces acting on the rivets; the direct load & that of the horizontal stress divided by the depth. Assume the floor beam to affect four rivets in the flange, also those of the stiffener which has 11"

P = 25960. & taking 1/3 of this to be supported by the stiffener we have 700 lbs per inch run horizontally. The resultant at different points

End = 944; 1 = 472; & 2 = 0 & finally at the end $\sqrt{944^2+700^2} = 1180$; 1 = 850, & 2 = 700

∴ Rivet spacing from the end to 1. $= \frac{5000}{1180} = 4''$ to 2. = 5″ 3/4; & to 3. = 6″

Let the rivets in the flange stagger with the above which are those connecting the flanges with the web.

o. Splicing plates for the web only will be needed and that at the middle. This will also act as a stiffener.

p. Bed Plates:- Allowable stone work pressure equals 300 lbs per square inch. Consider maximum reaction plus the impact: $\frac{71390}{300} = 238\,\square''$ necessary

This requires a plate 10"×24"×1". This plate requires a slot for the anchor bolt which being stationary necessitates the cutting of this slot to allow for the effect of temperature. Calculation shows 0.7". ∴ slot = 1"×1.7"

Bolts to be 1" in diameter.

q. Design of Stringers:– These to be of oak. & located two feet apart centre to centre. We find the load on each beam to be 1100 lbs. then we have the well known formula $\frac{I}{C} = \frac{M}{S}$; S=1200 lbs.; M=1/8 WL ∴ $\frac{I}{C} = 63$. A beam 6"×8" = 64 which is the necessary section ∴ Stringers to be 6×8" white oak.

r. Design of Floor Beams:– Taking steel eye beams we have the case of an overhanging beam. Using the same formula $\frac{I}{C} = \frac{M}{S}$ in which the second member is known taking M at its maximum. This requires an 8" I beam.

2. Bill of Material & Weight.

4.	Web plates	55" x 3/8" x 32 1/2	8926. lbs.
4.	Flange "	11" x 1/2 x 19'	1393
4	"	11" x 3/8 x 57'	3121
4	End Cover "	10" x 3/8" x 56"	209.
4	Web Splices	10" x 3/8" x 49"	272
8	Connection Plates	2' x 1' x 3/8"	240
4	"	1' x 1' x 3/8"	60
16	"	9" x 9" x 3/8"	133
8	"	9" x 1' x 3/8"	90
8	Bed "	16" x 16" x 1"	560
		Total.	15006.

Angles:

8	Flange Angles	5" x 3" x 13/16 x 57'	9758.
48	Stiffner "	3 x 3 1/2 x 5/16 x 55"	1457.
8	End "	3 x 4 x 5/8 x 55"	525

8	Lateral Brace Angles	3 x 3½ x 5/8 x 18¾	1950 lbs.
2		3 x 3½ x 5/8 x 18'	468
24	Cross	3" x 3" x 3/8 x 18.3	4830.
10		3 x 3 x 3/8 x 17	1903
8	Anchor Bolts, + nuts, etc		20
2210	pro. rivet heads		1105
		Total	22016.

b. Estimate of Cost

8926 lbs cast iron	@ 3¢	$267.80
6099 " plate	@ 2½	152.47
20882 " Angle "	@ 2½	522.05
1105 " rivets "	3¾	41.44
Total wgt. cost of fabrication	@ .75	277.50
" " " " Erection	@ .5	185.00
Painting		15.00
	Total	$1461.27

Floor system

6 'Eye beams 8" deep @ 3¢		$ 130.68
17 Stringers 6X8 X 22 = 14960 bd ft.		
17 " 6X8 X 33 = 28920 " "		
Flooring 35' X 55' X 4½ = 8662.		
4 Guard Rails 6X8 x 27½	5280	
	57822 @ $30	1734.67.

Grand Total 3326.61.

Design of Two Through Pratt Trusses.

Spans = $\begin{cases} 1. & 103' \\ 2. & 113' \end{cases}$ respectively (Plate II)

a. Dimensions & Data for computation

	No 1	No. 2.
Span	103'	103'
Panel Length	14.71	16.16
End Post	24.83	25.71
Depth	20.	20.'
tang θ.	.7352.	.808
Secant θ	1.242	1.286.

b. Dead Loads per Truss.

The weight of the bridge per linear foot was calculated from the formula: $w = 140 + 12b + .2bl - .4l$. from Merriman's "Roofs & Bridges," Part I

b = width of bridge
l = span.
w = wgt per linear ft.

	No. 1.	No. 2.
Wgt of bridge (w)	1120	1190
" " truss (w')	560	595
Panel load per truss	8238	9615
Panel load in Upper Chord.	2746	3205
Panel load " Lower Chord	5492	6410
Reaction	24714	28845

c. Web Stresses due to Dead Load:- These were calculated by finding the reaction & then the vertical shear; the latter multiplied by the secant of θ giving the stress. For the results of these and all stress calculations see strain sheets.

d. Chord Stresses due to Dead Load:- These were calculated by finding the shear for each for each piece, multiplying this by the tangent of θ thus obtaining the increments. Having these the stress is equal to the increment plus the sum of the increments of the preceding pieces or rather those to the left. This is in accordance with the rule from Merriman's Roofs & Bridges.

E. Web Stresses due to Live Load.

	No 1.	No. 2.
Live Load per sq. ft.	100	100
Total Live Load	361500	395500
Live Load per truss	180250	197750
Live Load " panel	25750	28250

The maximum stresses were found by loading to the right in each case. In those panels where the dead load stress is of opposite sign to the calculated live load stress no further calculation is necessary when the dead load stress exceed the live load stresses.

f. Chord stresses due to live load were calculated in the same manner, as those ~~of~~ due to dead load, by the method of chord increments. This can be done because the maximum stresses in the chord are obtained by loading the bridge entirely

g. Wind Stresses:

		No. 1	No. 2
Wind load per ~~square~~ linear ft.	L.	300 lb	300 lb
	U.	150 lb.	150 lb.
Total Wind load.	U.C	11032.	12120
	L.C.	30870	33936
Panel		2207.	2424.

The above shows only that due to the wind load treated as dead load. On the lower lateral bracing a calculation was also made allowing for 150 lbs. per linear foot as a moving load. This was taken into account in the calculations shown on the strain sheet. All the methods of calculating the strains due to wind on the various pieces are similar to those explained

h. For strain sheets see separate plates at the end of the work. The strains due to wind load are worked for one side only; when the wind blows in the opposite direction the other members will be strained to the same degree.

i. Design of Upper Chord & Posts.

For the upper chord not much material will be wasted if we take it of one section throughout its entire section so we consider only the largest strain in any one of the pieces. this will be in the third panel. According to specifications, having A = section; r = radius of gyration; & l = the length in inches:

$$A = \frac{2\,\text{live} + 2 \times \text{dead load stress}}{\left(20000 - 2 \times 40 \frac{l}{r}\right)1.20}$$

the denominator is increased 20% because of the steel of the structure in place of iron.

For the posts the following formula is used.

$$A = \frac{2\,\text{live} + 2 \times \text{dead load stress}}{20000 - 140 \frac{l}{r}}$$

No. 1.	Channels.	Plates or Bars.	A.	
			A_1	A_2
Upper Chord.	9" Deep # 359.	16" × 3/8"	14.7	15
End Post.	10" " # 358	bars. 12" × 1/2 × 2" 5	21	22.
Second Post	8" " # 360	16" × 1/2.	10.4	16.
Third Post	7" " # 361.		8.4	8

No. 2.	Channels.	Plates or Bars	A.
Upper Chord	10" deep #359	16" x 3/8"	18.5" & 19"
End Post.	10" " #358.	16 x 1/2.	21 & 22.
Second Post	8 " " #360	12 x 1/2" x 2"	10.3 & 10.
Third Post.	7 " " #361.	"	8.6 & 8.

m. Design of sections for Lower Chord; & main and counter web members:– The following formula is used in finding the area (A)

$$A = \frac{2 \times \text{live load stress} + \text{dead}}{120(20000)}$$

No. 1.	1st Vertical	Panels. 2nd	3rd	4th
Main.	2 pieces 3"x1"	2 pcs. 4x7/8	2 pcs 3x13/16	2 pcs. 1.50
Counter.	——	——	1 bar 1/40	"
No. 2.				
Main	1 piece 3x1"	2 pce 4x1.	2 pcs 3x7/8	2 bars 1.50
Counter	——	——	1 bar 1/40	"

Lower Chord.	Panels			
	1st.	2nd	3rd	4th.
No 1.	2 pcs. 3X1	2 pcs 3X1.	2 pcs 4X1 1/8	4 pcs 3X7/8
No. 2.	2 pce 3.5X1	2 pce 3.5X1	2 pce 4X1 1/4	4 pce 3.5X1.

n. Upper & Lower lateral rods & Struts.

The upper rods :- $A = \frac{wwid + 10000A}{15000}$ or $A = \frac{wwid}{5000}$

We find this requires a rod 1 1/8" in diameter. for the 1st panel. Made the others the same size

The Lower rods :- The same formula is used.

	1st Panel	2nd Panel	3rd	4th.
No 1	2 1/8" diam.	1 3/4" ∅	1 5/8" ∅	1 1/8" ∅
No. 2.	2 1/4	1 7/8 ∅	1 5/8" ∅	1 1/4 ∅

The upper lateral struts were found in a mansimilar to that of the upper chords: we have four angles whose radius of gyration can be found & by the method of approximation having:

$P = 16000 - 105\frac{l}{r}$ = allowable unit stress. the stress due to tension of the bar is of course taken into consideration. By this we find a convenient section in angles 4 x 4½ placed so that they may be rivetted to the upper chord on either side. They then form a latticed eye beam.

The lower lateral struts are the floor beams.

o. Sizes of Pins:- There are three methods of determining the size of pins; viz, by the method of moments, to resist fracture by bearing, & to resist fracture by shearing. By methods we find the components of the moments in a vertical & horizontal plane & find the resultant of these moments.

Having arranged the joints we can find these moments easily take the maximum in each case, then having M_1 = the horizontal moment & M_2 = the vertical moment we have:

$$M = \sqrt{M_1 + M_2} \quad \& \quad d = \sqrt[3]{\frac{32\,M}{\pi S}}$$

To resist fracture by bearing stress we have the diameter of the pin into the thickness of the bearing surface as the area to resist the pressure. then we have $S = \frac{P}{d \times t}$ or $d = \frac{P}{d \times t}$ in which P = the total pressure; t = the thickness of the bearing surface, and S = the unit bearing stress allowable.

To resist fracture by shearing stress, the area of a section of the pin into the unit stress allowable is the amount of strain resisted. then

$$S = \frac{P}{\pi r^2} \quad \text{or} \quad r = \sqrt{\frac{P}{\pi S}} \;\&\; d = 2\sqrt{\frac{P}{\pi S}}$$

Each pin is calculated for all of these

strains and the maximum diameter deduced is taken as the proper diameter of the pin.

	Upper Chord			Lower Chord			
	1	2	3	1	2	3	4
No. 1	3 5/8" Ø	3 3/8" Ø	3 1/8" Ø	3 1/2 Ø	3 1/4 Ø	3 5/8 Ø	3 3/8 Ø
No. 2	4" "	3 7/8" "	3 3/8 "	4" "	3 7/8 "	4 "	3 1/2 "

p. Design of Eye Bar Heads:– To obtain the proper dimensions we try to follow as closely as possible the advantageous proportions deduced by S. Smith in a series of tests. We have two quantities known the size of the bar in section & the diameter of the pin. Now the third quantity is a simple thing to obtain having given the above reports of tests. This quantity represents the diameter of the circular part of the eye bar head. The following are the results:

<table>
<tr><td></td><td colspan="3">Upper Chord</td><td colspan="4">Lower Chord.</td></tr>
<tr><td></td><td>1</td><td>2</td><td>3</td><td>1</td><td>2</td><td>3</td><td>4.</td></tr>
<tr><td>No 1</td><td>\ 4.3
| 4"</td><td>\ 4.1"</td><td>\ 3.8</td><td>— 4.6</td><td>— 3.75
ʌ 3.75
— 3.5</td><td>— 4.6
| 4.9
— 4.6</td><td>— 4.5
\ 4.2
— 4.5</td></tr>
<tr><td>No. 2</td><td>\ 5"
| 4.5"</td><td>\ 4.3"</td><td>\ 4"</td><td>— 4.7"</td><td>— 4.6
ʌ 4.4
— 4.6</td><td>— 4.7
\ 5.0"
— 5.2</td><td>— 5.3
\ 4.6
— 4.7</td></tr>
</table>

The lines before the different numbers show to which eye bar these various dimensions refer. The number of in each case is that of the ½ diameter of the head.

9. Design of Hangers & Hanger Plates

The hangers are simple bars & are calculated to hold the maximum load coming on the panel point. A small unit stress is taken to allow for the sudden loading. The bars are square in section when in contact with the pins & round the other part.

the calculations require; for,

No 1. $2^{\square}{}'' =$ bars $3/2'' \times 3/2''$ or $2.5''$ ⊘.

No 2. $2.5^{\square}{}'' =$ bars $5/3'' \times 5/3''$ or $3''$ ⊘

The plates are 1" thick & 1'x1' in size.

s. Floor Beams: These are calculated in a manner similar to those of the plate girder using the formula $\frac{I}{c} = \frac{M}{S}$, taking M as a maximum. These resulted in the following selection.

For No 1. 12" heavy I beams.

No 2. 14" " I

t. Stringers: Also calculated as those of the plate girder. with results. 6"x8" section & of white oak.

u. Bed Plates & rollers:- For the bed plates the maximum reaction is to be resisted by these plates. The unit working stress is be that of the stone of the pier which is 250 lbs. per specifications. This requires plates of 500 □" area or 22" square

The rollers are of steel & the pressure per lineal inch should be less than the number expressed by this formula: $500\sqrt{d}$; d representing the diameter of the pins. Taking (9) nine, two inch rollers we have the total resisting power on the above assumed strain per lineal inch = 138600 lbs while the actual pressure is only = 123125

Therefore a nest of nine rollers 2" diameter.

Both the designs of plates and rollers are for both trusses.

v. Estimate of Bill for angles, plates, etc.

As before, No 1 & No 2. will be used to designate the two trusses. Weights taken from Carnegie's "Pocket Book".

Channels.

No Pieces	Location	Size No. 1	No. 2	Section No 1	No 2	Length No 1	No 2	Wgt in lbs. No 1	No 2
4.	End Posts.	10" x #24	10" x #24.	20□	22.	25'	26'	4x500	4x524.
4.	Upper Chord.	9 x 24	10" x #20	15	19	36½.	40½.	4x730	4x984.
8.	Inter. Posts.	8&7" x #15 #17	8&7" x #17 15	10&8	8&10	20½.	20½	4x300	4x340.
								6120	7392.
Angles.									
12.	Later. Struts	4x5 x 3/8 11	4x5 x 3/8 11	3.2	3.2	20'	20'	2640	2640
2	Portal brac.	4x4 x 3/8 10	4x4 x 3/8 x10	3.	3.	11'	11.'	220	220
4	Shoe	5x3 x½ 12.	5x3 x½ 12	4	4.	15"	15"	60	60
								2920	2920
Plates.									
2.	End Posts	16 x ½.	16 x ½.	8□	8□	25'	26'	1332	1276
1	Upper Chord.	16 X 3/8	16 x 3/8	8	8	73'	81.'	1727	2038
6	Hanger Pl.	1" x 1"	1" x 1"	12"	12.	1'	1'	237	237.
4.	shoe	8 x 3/8	8 x 3/8	3	3	18"	18"	60	60
4		12 x 1"	12 x 1"	12	12	19	19.	240	240
								3596	4850

Plates. con.

No Pieces	Location	Size No 1.	Size No 2	Section No 1.	Section No 2.	Length No 1.	Length No 2	Weight No 1	Weight No 2.
4	End Post & U.C	9 x 3/8	9 x 3/8	3 3/8	3 3/8	17"	17"	646	646
28	Posts & U.C.	9 x 1/2	9 x 1/2.	4 1/2	4 1/2	1'	1'	420.	420
20	Posts	10 x 1/2	10 x 1/2.	5"	5"	17	17	453.	453.
8	End Posts	9 x 1/2.	9 x 1/2.	4.5	4.5	15"	15"	150	150
					Forward.			3596	3850
								5265	5520
Eye	Bars.								
4	Lower Chord.	3 x 7/8	3.5 x 1	2.6	3.5	15.5	16.	532	739
4	"	4 x 1 1/8	4 x 1 1/4	4.5	5	15.5	16.	921	1036
8	"	3 x 1	3.5 x 1.	3	3.5	15.5	16.	1228	1478
2.	1st Vertical	3 x 1	3 x 1.	3	3.	21	21.	420	420
4.	2nd Panel.	4 x 7/8	4 x 1.	3.5	4.	23.5	26.5	1373	1495
4	3rd "	3 x 13/16	3 x 7/8.	2.5	2.6	25.5	26.5	990	1059.
								5464	6247

Lateral rods & counters.

		No 1	No 2.
4 Counters for central panel	1.5⊘ & 1.5⊘	1320	1320
2 " 3rd & 5th	1.1/4 & 1 1/4 ⊘	505	505
5 Upper Rods 1 1/8 ⊘ 24' long	=	825	825
7 Lower Rods 2 6/8 ⊘ 24'	=	1733	1733.
		4383	4383

Floor Beams. etc. etc.

	No 1	No 2
6. Floor Beams. 12" deep & 14" deep:	1400	1750
18 Rollers 2" diameter	400	400
28 Nuts 5 lbs.	140	140
10 Connections for U. Rods.	50	50
14 "	140	140
8 Anchor Bolts	16	16.
105' Fencing. @ 75 lbs per ft.	5875	5875.
750 & 770 Lattice bars. @ 1.5 lbs	1125	1155
6 Hangers. @ 55	330	330
	9856	9856

Forward.	9856	9856
Rivet Heads. 2470 & 2687 @ ½ lb.	1235	1344
Turnbuckles & collars.	250	250
14 Pins	700	890
	11340	11400

Floor System:	No 2	No 1.
112 Stringers 6 x 8 by (14.5 & 16' long)	19500	18000
9000 ft flooring @ .3 lbs.	3070	2250
Guard Rails	1050	985
	23520	21235

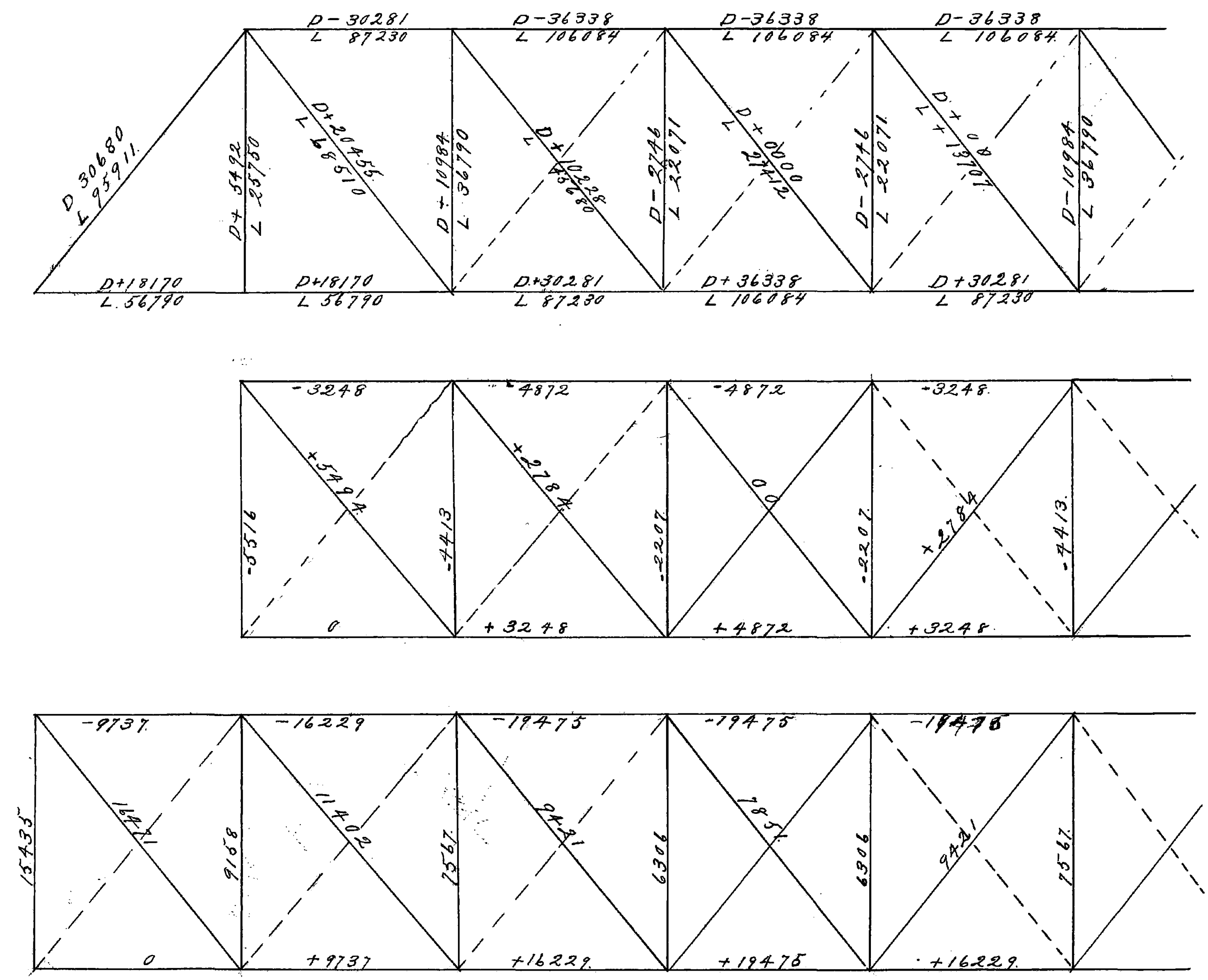
	No 1	No 2
½ Total Weight =	59683	64183
Estimated Total Weight =	119366	128367
Assumed Total Weight =	118600	133570

w. Estimate of Cost.

	Price per lb.	Weight. No 1	Weight. No 2	Cost. No 1	Cost. No 2
Channels	3¢	12240	14684	367.20	440.62
Angles	2.5	5840	5840	146.	146.
Plates	2.5	10520	11040	263.45	276.
Bars	3.5	10928	12494	382.48	437.29
Rods	3.	8766	8766	262.28	262.98
Rivets	3.75	2470	2688	92.65	100.80
Odd pieces		bd ft		340.20	357.80
Floor System	per M. $30	84600	87000	2538.00	2610.00
Wgt cost of fabrication	.75	119366	128367	895.25	962.75
Wgt cost of Erection	.5	119366	128367	569.83	641.84
Total				$5885.02	$6020.07

X. The high price or rather cost, is due to two things: the width of the bridge & the elaborate floor system. Considering the width we have side-walks which will accommidate any large city travel; the same for the wagon road-way. The floor system is very durable and is that of a bridge of any larger city as mentioned before.

NO. 1.



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D+5492 L 25750
D+20455 L 68510
D+10984 L 36790
D+10228 L 42280
D-2746 L 22071
D+0000 L 21412
D-2746 L 22071
D+1370 L 1074
D-10984 L 36790
D+18170 L 56790
D+18170 L 56790
D+30281 L 87230
D+36338 L 106084
D+30281 L 87230
-3248
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-5516
+5494
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+2784
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+2784
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+3248
+4872
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-9737.
-16229
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-19475
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6306
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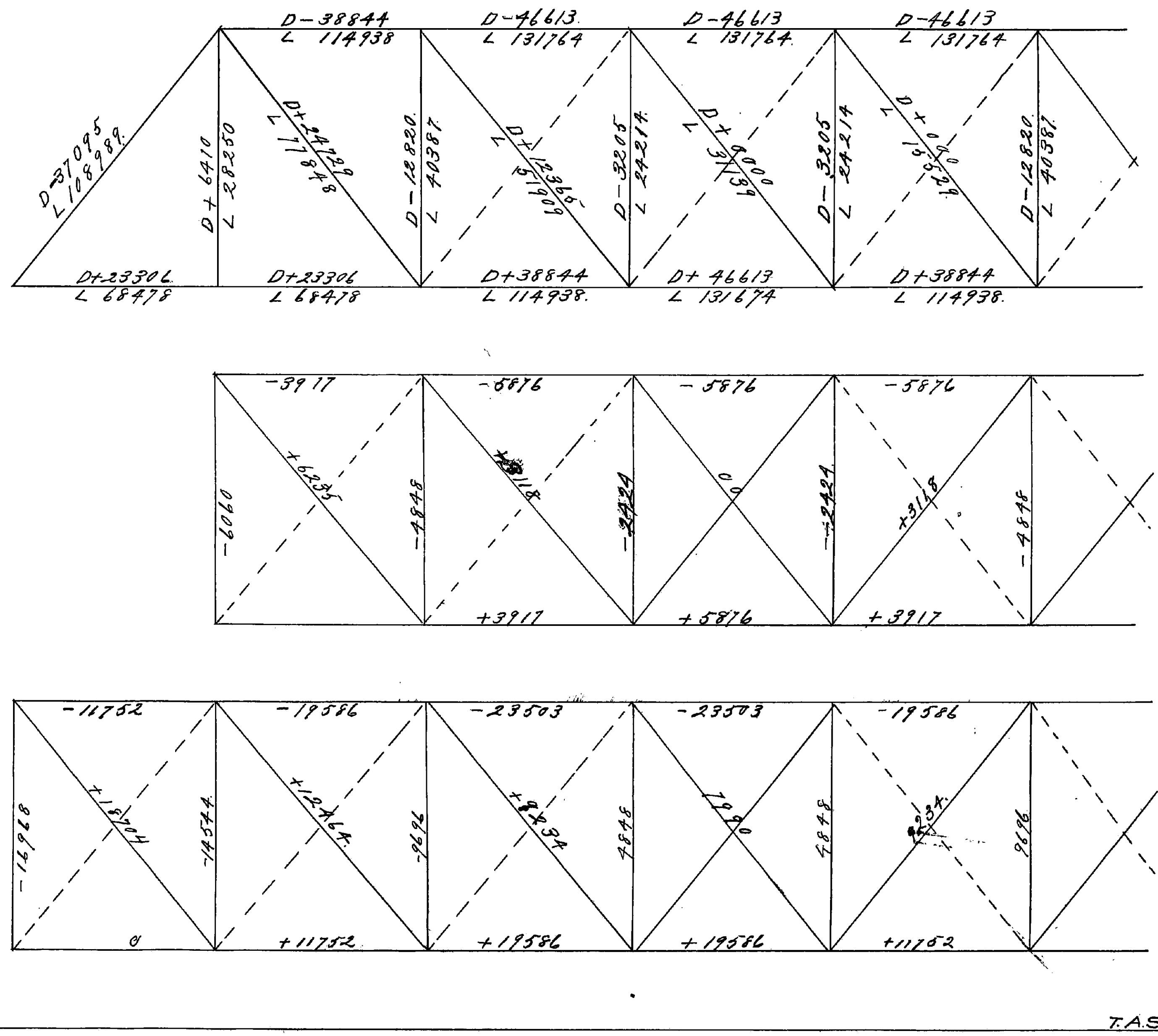
+16229.

T.A.S.

NO. 2.


D-38844
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D+24729
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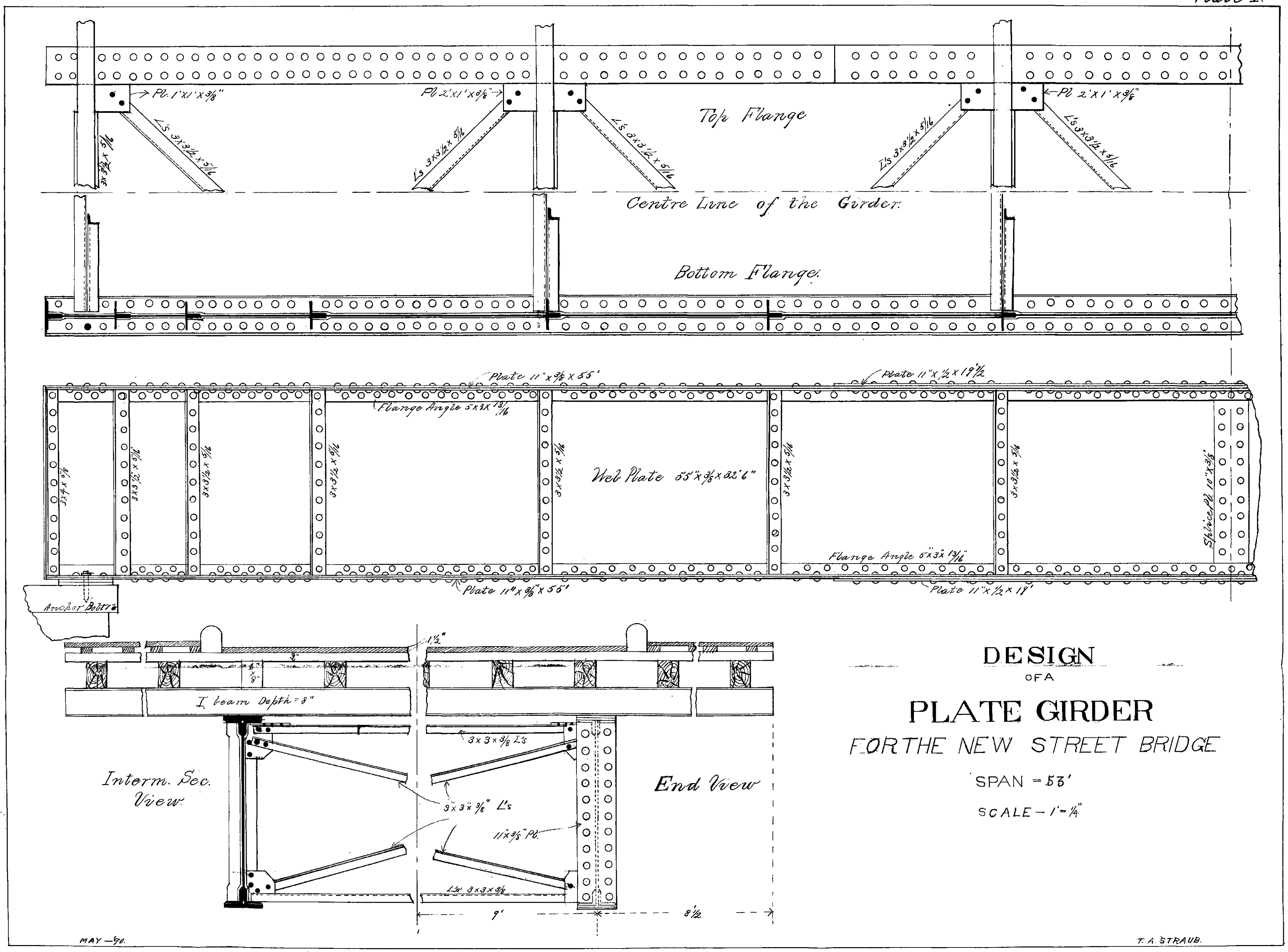
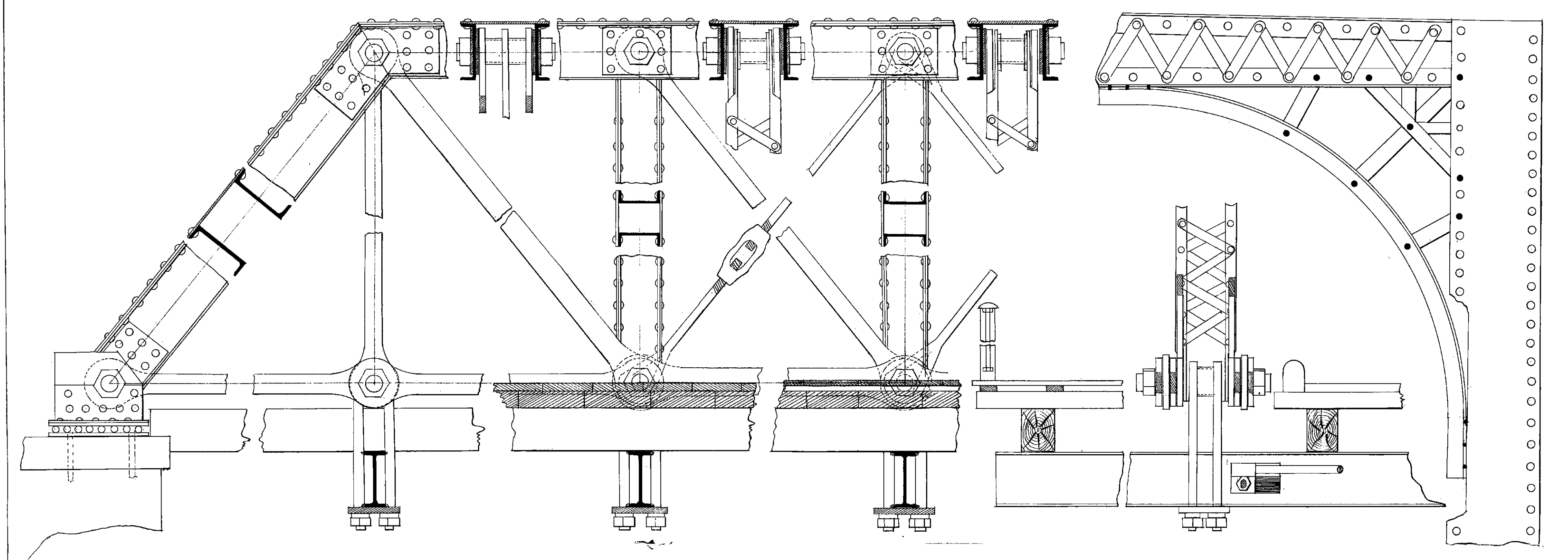
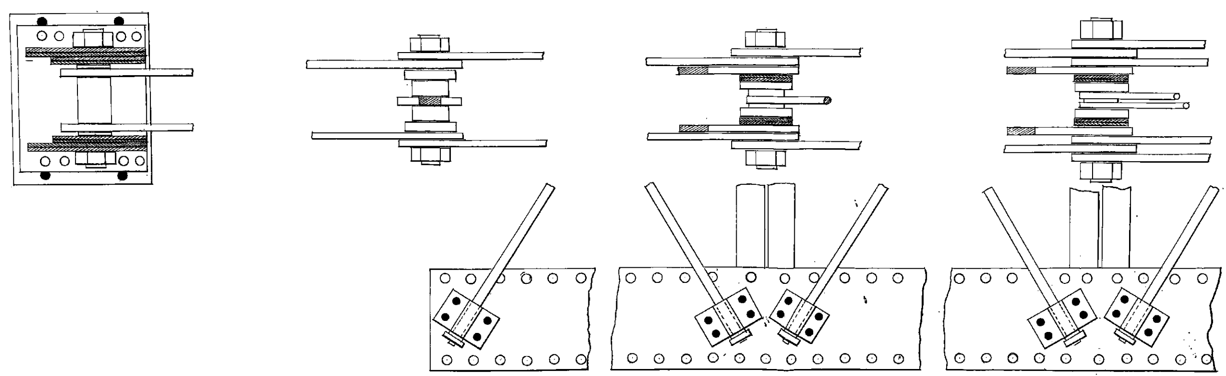
T.A.S.


Pl. 1'x1'x3/8"
Pl 2'x1'x3/8"
Pl 2'x1'x3/8"
L's 3x3½x5/16
Top Flange
Centre Line of the Girder.
Bottom Flange.
Plate 11"x3/8x55'
Plate 11"x½x19½'
Flange Angle 5x3x13/16
3x4x3/8
3x3½x5/16
Web Plate 55"x3/8x32'6"
Splice Pl. 10"x3/8
Flange Angle 5"x3"x13/16
Plate 11"x3/8"x55'
Plate 11"x½x19'
Anchor Bolts
1½"
I beam Depth = 8"
3x3x3/8 L's
3"x3"x3/8" L's
11"x3/8" Pl.
L's 3x3x3/8
9'
8½
Interm. Sec. View
End View
DESIGN
OF A
PLATE GIRDER
FOR THE NEW STREET BRIDGE
SPAN = 53'
SCALE - 1'=¼"
MAY-90.
T. A. STRAUB.





DESIGN
OF A
THROUGH STEEL TRUSS
FOR THE NEW STREET BRIDGE

SPAN—113'

SCALES
SKELETON TRUSS—1'=¼"
DETAILS —1'=1"

MAY. 20—98.

T. A. STRAUB.