C. E. Thesis case

DESIGN OF A 270-FOOT PIN CONNECTED DOUBLE TRACK BRIDGE.

Submitted by

Signature redacted

June 1924

Cambridge, Mass.
May 31st, 1924

Professor A. L. Merrill,
Secretary of the Faculty,
Massachusetts Institute of Technology.

Dear Sir:—

In compliance with the requirements for graduation, I herewith submit a thesis, entitled, — Design of a 270-ft. Pin Connected Double Track Bridge.

I desire to express my gratitude to Prof. H. L. Bowman for his valuable advice.

Respectfully submitted,

Signature redacted

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INTRODUCTION

The following computations are for a two-hundred-seventy-foot double track railway bridge of the through pin connected type. The specifications followed were those of the A.R.E.A. for 1920. This span was chosen because it represents a typical span of a pin connected bridge. The double track system was used to accommodate the present and future railways in China and a loading of Cooper E 65 was provided for the traffic development which will likely occur in any railroad in the near future.

The estimated weight of this structure is 2,007,900 pounds and the assumed weight from which dead stresses were computed was 2,080,000 pounds. The dead stresses are approximately 2⅞ per cent of the total stress. This would mean that only about 16,000 pounds of dead the weight is over-assumed and it is on the safe side. Hence no redesign is necessary.

Assuming the bridge to cost nine cents a pound, the cost of this structure would be about $180,700.00.

STRINGERS

Dead Load = 488 #/'

D.S. = 30'×488×$\frac{1}{2}$ = 7,320 #

L.S. = 102,370

L.S. = 97.1% = 99,500

209,090

÷ 10,000 = 20.909 ⅢⅢ

use web 60"×$\frac{3}{8}$" = 22.5 ⅢⅢ

$I = \frac{300}{300+\frac{30^2}{100}} = 97.1\%$

D.M. = $\frac{488\times30^2}{8}$ = 54,900 '#

L.M. = 668,250 '#

L.M. = 97.1% = 649,000 '#

1,363,150 '#

÷ 57.35 = 23,800

÷ 16,000 = 1.49

× 12 = 17.87 ⅢⅢ

$-\frac{22.5}{8}$ = 2.81

15.06 ⅢⅢ needed in tension flange

$f_c = 14{,}000 - 200\times\frac{12\times12}{12.375} = 11{,}670$

23,800

÷ 11,670 = 2.04

× 12 = 24.45

− 2.81 2.81

21.64 ⅢⅢ needed in compression flange

Use 2 Ls 6×6×$\frac{13}{16}$ } Top flange
1 Pl. 14×$\frac{3}{8}$ }
2 Ls 6×6×$\frac{13}{16}$ Bottom flange

Area 2 Ls	=	18.18
−4×1×$\frac{13}{16}$	=	3.25
		14.93

Needed for tension 16.06 sq in
furnished 14.93 sq in net

Area 2 Ls	=	18.18
+ 1 Pl.	=	5.25
		23.43

Needed for compression 21.64 sq in
Furnished 23.43 sq in gross.

Use
Web 60"×$\frac{3}{8}$"
Top Flange 2 Ls 6"×6"×$\frac{13}{16}$" 1 Pl. 14"×$\frac{3}{8}$"
Bottom Flange 2 Ls 6"×6"×$\frac{13}{16}$"

FLOOR BEAMS (INTERMEDIATE)

Dead F.B. Load = 500 #/'

D.S. = 22878 #

L.S. = 315600

I.S. = 282000

620,478

$I = \frac{300}{300 + \frac{60^2}{100}} = 89.2\%$

÷ 10,000 = 62.05 ▭

Web $84'' \times \frac{3}{4}'' = 63$ ▭

D.M = 191,700 '#

L.M = 2,840,400

I.M = 2,540,000

5,572,100 '#

÷ 82.47 = 67,500

÷ 16,000 = 4.22

× 12 = 50.65 ▭

$- \frac{63}{8} = \frac{7.88}{42.77}$ ▭ needed in tension flange

$f_c = 14{,}000 - 200 \times \frac{6.5 \times 12}{16.75} = 13{,}070$

67,500

÷ 13,070 = 5,165

× 12 = 62.00 ▭

− 7.88

54.12 needed in compression flange

Use 2Ls 8×8×1
1 Pl. 18 × 5/8
1 Pl. 18 × 3/4
} Top Flange

2Ls 8×8×1
1 Pl. 18 × 5/8
1 Pl. 18 × 1/2
} Bottom Flange

Area 2Ls 30.00 in²
1 Pl. 11.25
1 Pl. 19.50
50.75
−2×1(2+.625+.5) = 6.25
44.50 in²

Needed for tension 42.77 in²
Furnished 44.50 in²

2Ls 30.00 in²
1 Pl 13.50
1 Pl. 11.25
54.75 in²

Needed for compression 54.12 in²
Furnished 54.75 in²

Use for floorbeam
Web 84" × 3/4"
Top Flange 2Ls 8"×8"×1" 1 Pl 18" × 5/8" 1 Pl 18" × 1/2"
Bottom Flange 2Ls 8"×8"×1" 1 Pl. 18" × 3/4" 1 Pl. 18" × 5/8"

FLOOR BEAM (END)

D.S. = 15,314 #

L.S. = 204,800

L.S. = 199,200

419,300

$I = \frac{300}{300 + \frac{30^2}{100}} = 97.1\%$

D.M. = 172,700 '#

L.M. = 1,740,000

I.M. = 1,690,000

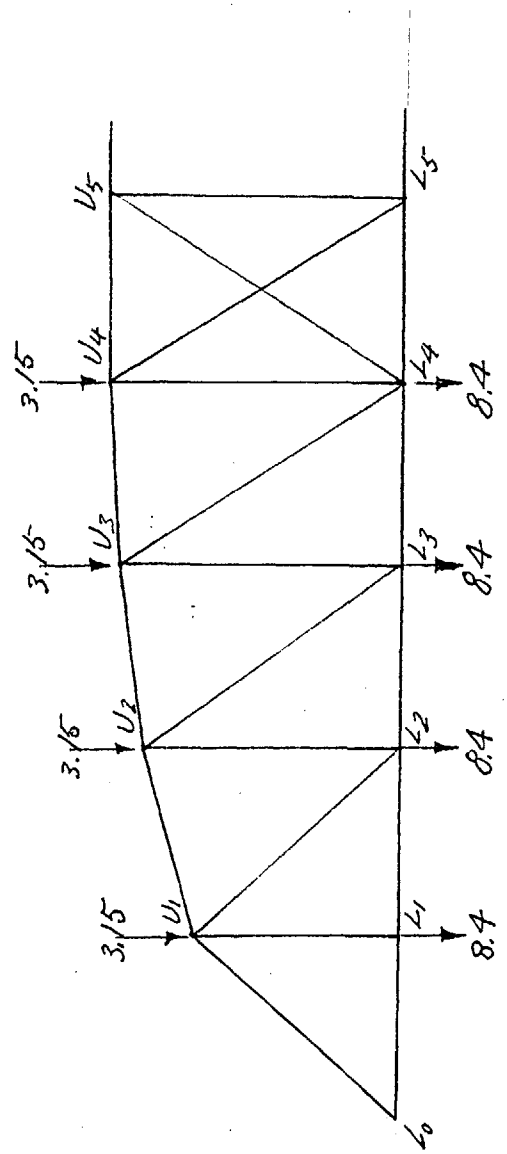
3,602,700 #

TRUSS

Dead load per truss = 3850 #/,

Bottom panel load = $30\left(3850-\frac{2100}{2}\right)=84000$ #

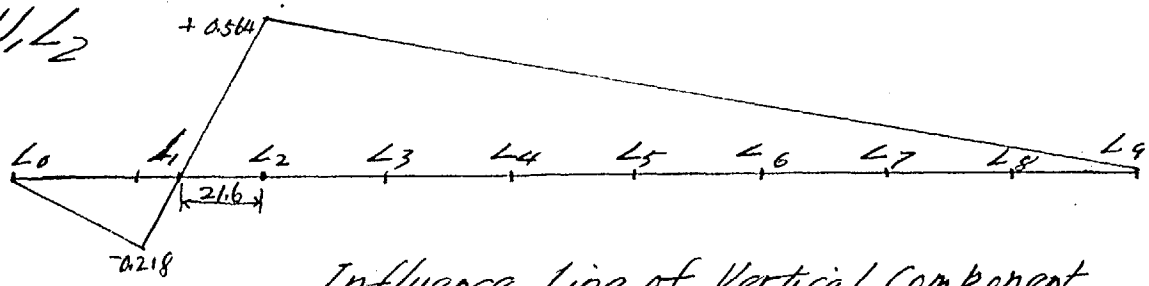
Top panel load = $\frac{2100}{2}\times 30 = 31500$ #



SAMPLE COMPUTATION

OF L. STRESS IN MEMBERS.

U_1L_2



Influence Line of Vertical Component of stress (Live) in U_1L_2

	Increase in V.C	Decrease in V.C.
With load ④ at L_2 move up load ⑤ to L_2	$347.75 \times \frac{5}{210} \times .564 + 112.5 \times 3.25 \times \frac{5}{210} \times .564$	$< 113.75 \times \frac{5}{30} \times .782$
load ③ to ④	$(380 + 124 \times 3.25)\frac{5}{210} \times .564$	$< 81.25 \times \frac{5}{30} \times .782$
load ② to ③	$(412.5 + 109 \times 3.25)\frac{5}{210} \times .564$	$> 48.75 \times \frac{5}{30} \times .782$

$\therefore$ load ③ at L_2 gives max.

$\frac{16.25}{21.6} \times .564 \times 3.6$	$= 1.53$
$+ \frac{32.5}{21.6} \times .564 \times 11.6$	$= 9.85$
$+ \frac{32.5}{210} \times .564 (210 + 205 + 200 + 159 + 154 + 149 + 144)$	$= 106.50$
$+ \frac{21.125}{210} 1.564 (191 + 186 + 180 + 175 + 135 + 130 + 124 + 119)$	$= 70.30$
$+ \frac{16.75}{210} 1.564 \times 167$	$= 7.29$
$+ \frac{11.4}{210} \times \frac{11.4}{2} \times 3.25 \times .564$	$= 56.75$
Total Vertical Component	$= 252.22$

Total live stress $= \frac{46.1}{35} \times 252.22 \times 2 = 665$ kips.

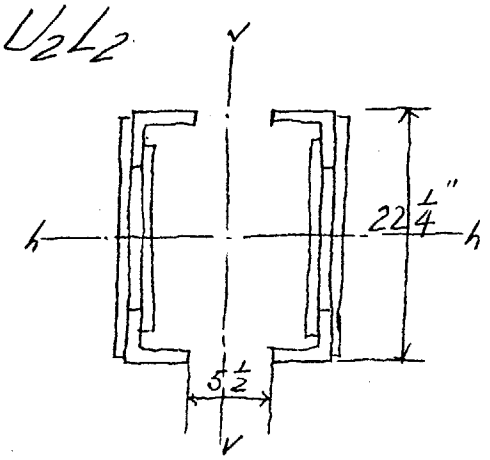
Impact $= \frac{300}{300 + \frac{223}{100}^2} \times 665 = 250$ kips

STRESS SHEET

BAR	DEAD LOAD			LIVE LOAD			GRAND TOTAL (KIPS)
	INDEX	Mut.	D. STRESS	L. STRESS	IMPACT	TOTAL	
L_0U_1	-462	46.1/35	-609	-1213	377	-1590	-2199
U_1U_2	-111.4	30.5/5.6	-607	-1178	376	-1554	-2161
U_2U_3	-78.8	30.2/33	-721	-1370	447	-1817	-2538
U_3U_4	-28.2	30/1.1	-769	-1460	505	-1965	-2734
U_4U_5	-897	30/35	-769	-1596.8	552.5	-2149	-2918
U_1L_1	+84	1	+84	+275.8	246.2	+522	+606
U_1L_2	+234.1	46.1/35	+309	+665	250	+915	+1224
U_2L_2	-150.1	1	-150.1	-379.6	169.6	-549	-699
U_2L_3	+182.7	50.5/40.6	+190	+507	227	+734	+924
U_3L_3	-69	1	-69	-297.4	157.6	-455	-524
U_3L_4	+87.3	53.1/43.4	+106	+426	226	+652	+758
U_4L_4	-3.3	1	-3.3	-249.4	152.6	-402	-405
U_4L_5	0		0	+333	204	+537	+537
L_0L_1	+462	30/35	+396	+790	246	+1036	+1432
L_1L_2	+462	30/35	+396	+790	246	+1036	+1432
L_2L_3	+697	30/35	+598	+1157	369	+1526	+2124
L_3L_4	+827.5	30/35	+709	+1361	444	+1805	+2514
L_4L_5	+897	30/35	+769	+1278	442	+1720	+2489
L_3U_4	-87.3	54/45	-105	+125	93	218	+113

DESIGN OF MEMBERS

U_2L_2



2 Webs $22 \times \frac{5}{8}$
4 Ls $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$
2 Fillers $15 \times \frac{1}{2}$
2 Webs $18 \times \frac{3}{8}$

A	×	Arm^2	=	I_{hh}
27.50		0		1110 + 0
16.00		0		281 + 0
13.00		10.07		1320
13.50		0		365 + 0
69.00				3076

$$\rho_{hh} = \sqrt{\frac{3076}{69}} = 6.69$$

A		Arm		I_{vv}
27.50	×	6.563^2	=	1186
16.00	×	6^2	=	540
13.00	×	5.19^2	=	350
13.50	×	5.56^2	=	417
				2493

$$\rho_{vv} = \sqrt{\frac{2493}{69}} = 6.01$$

$$f_c = 16000 - 70 \times \frac{40.6 \times 12}{6.01} = 10{,}330 \ \#/\square''$$

Required Area $= \frac{699}{10.33} = 67.6 \ \square''$

Furnished Area $= 69 \ \square''$

U_3L_3

2 Webs $22 \times \frac{5}{8}$
4 L's $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$
2 Fillers $15 \times \frac{1}{2}$

$\rho_{VV} = \sqrt{\frac{2076}{55.0}} = 6.11$

$f_c = 16{,}000 - 70 \times \frac{43.9 \times 12}{6.11} = 9950$

Area required $= \frac{524}{9.95} = 52.6$

Area furnished $= 55.0$ in²

U_4L_4

2 Webs $22 \times \frac{1}{2}$
4 L's $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$
2 Fillers $15 \times \frac{3}{8}$

$\rho_{VV} = \sqrt{\frac{1618}{43.25}} = 6.11$

$f_c = 16000 - 70 \times \frac{45 \times 112}{6.11} = 9810$

Area required $= \frac{405}{9.81} = 41.3$ in²

Area furnished $= 43.25$ in²

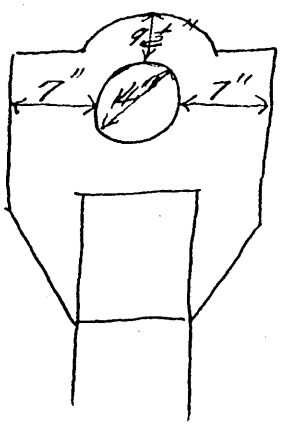
U, L,

2 Webs $22 \times \frac{5}{8} = 27.50^{\square"}$
4 Ls $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2} = 13.00$
2 Fillers $8 \times \frac{1}{2} = 15.00$
55.50
$-8 \times 1 \times 1 = 8.00$
$47.50^{\square"}$

Area required $= \frac{606}{16} = 37.9^{\square"}$
Area furnished $= 47.5^{\square"}$

Hanger plate at U,
$\frac{1.40 \times 37.9}{2} = 26.5^{\square"}$ Use 2-14"x2" pl.

$\frac{1.00 \times 37.9}{2} = 19^{\square"}$ Width of pl above pin hole $= 9.5" = 9\frac{1}{2}"$



$\frac{606}{2 \times 7.2} = 49$ rivets required at U,

$55.5 - 2 \times (14 \times 1.125) = 23.5^{\square"}$
$46 \quad 47.5 - 23.5 = 22.5^{\square"}$ Use 2 pri plates $\frac{7}{8}$" thick at L,

*U_1L_2

$\frac{1224}{16} = 75.5^{\text{m}}$

$\frac{75.5}{14} = 6.39''$

$\frac{6.39}{2} = 2.7''$

Use 4 - 14 × 1½ eye bars.

U_2L_3 $\frac{924}{16 \times 14} = 4.1$

Use - 2 - 14 × 2 eye bars

U_3L_4 $\frac{758}{16 \times 14} = 3.4''$

Use 2 - 14 × 1¾ eye bars.

U_4L_3 (counter) $\frac{113}{16 \times 14} = 0.5''$

Use 1 - 14 × 1½ eye bars.

U_4L_5 $\frac{537}{16 \times 14} = 2.4''$ Use 2 - 14 × 1½ eye bars.

*In the whole design the pins are assumed as 14" in diameter in order to secure the minimum number of eyebars.

L_0L_1 & L_1L_2

$\frac{1432}{16 \times 14} = 6.4''$ Use $4-14 \times 1\frac{5}{8}$ eyebars

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L_2L_3

$\frac{2124}{16 \times 14} = 9.5''$ Use $4-14 \times 1\frac{5}{2}$, $2-14 \times 1\frac{3}{4}$ } eye bars

---

L_3L_4

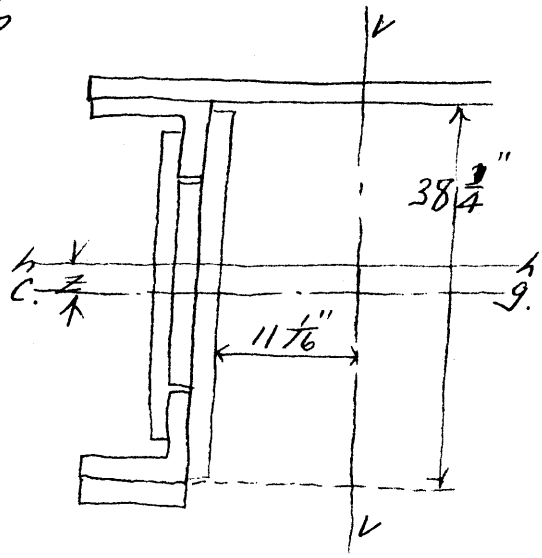
$\frac{2514}{16 \times 16} = 9.8''$ Use $4-16 \times 1\frac{1}{2}$, $2-16 \times 2$ } eye bars

---

L_4L_5

$\frac{2489}{16 \times 16} = 9.74''$ Use $4-16 \times 1\frac{1}{2}$, $2-16 \times 2$ } eye bars

$U_1 U_2$



Cover $36 \times \frac{1}{2}$
2 Webs $38 \times \frac{3}{4}$
4 Ls $6 \times 6 \times \frac{1}{2}$
2 Fillers $26 \times \frac{1}{2}$
2 Bot. Pl. $6 \times \frac{1}{2}$
2 Webs $34 \times \frac{3}{8}$

A	Arm	M	I cg
18.0	19.25	+347	6670
57.0	0	0	7080
23.5	17.32	0	1460
26.0	0	0	7050
6.0	19.25	-116	2220
26.3	0	0	2460
156.8 in²		+231	26940

$$z = \frac{231}{156.8} = 1.47"$$

$$I_{hh} = 26940 - 1.47^2 \times 156.8 = 26600$$

$$\text{Area required} = \frac{2161}{13.84} = 155.7 \text{ in}^2$$

$$\text{Area furnished} = 156.8 \text{ in}^2$$

$U_1 U_2$ (continued)

A	Arm	I_{vv}
18.0	0	1940
57.0	11.42	7430
23.5	12.06	4270
26.0	13.49	3780
6.0	14.81	1310
26.3	12.50	4110
156.8		22840

$$\rho_{vv} = \sqrt{\frac{22840}{156.8}} = 12.09$$

$$t_c = 16{,}000 - 70 \times \frac{30.5 \times 12}{12.09} = 13890$$

Area required $= \frac{2161}{13.89} = 155.7$ □"

Area furnished $= 156.8$ □"

$\frac{2161}{336} = 6.43"$ $= 6.43"$

Thickness of built up section $= \frac{4.375"}{2.06"}$

Use 2 pin plates outside of section $1-1"$ $1-1\frac{1}{16}"$

$\frac{2161}{6.43} \times 1.063 = 357$

$\div 7.2 = 50$ rivets required

$U_2 U_3$

Cover	$36 \times \frac{1}{2}$
2 Webs	$38 \times \frac{3}{4}$
4 ∠s	$6 \times 6 \times \frac{5}{8}$
2 Fillers	$26 \times \frac{5}{8}$
2 Bot. Pl.	$6 \times \frac{5}{8}$
2 Webs	$34 \times \frac{5}{8}$

A	Arm	I_{vv}
18.0		1940
57.0	11.42	7430
28.4	13.54	5210
32.5	12.12	4770
7.5	14.81	1640
42.5	12.94	6040
185.9 in²		28590

$$\rho_{vv} = \sqrt{\frac{28590}{185.9}} = 12.4 \qquad Z = \frac{20^2}{185.9} = 1\tfrac{1}{8}''$$

$$f_c = 16{,}000 - 70 \times \frac{30.2 \times 12}{12.4} = 13{,}950$$

$$\text{Area required} = \frac{2538}{13.95} = 182 \text{ in}^2$$

$$\text{Area furnished} = 185.9 \text{ in}^2$$

$$\frac{2538}{336} = 7.56'' \qquad = \quad 7.56$$

$$\text{Thickness of built up section} = \frac{3.75}{3.81''}$$

$U_2 U_3$ (continued)

Use 2 - 2" pui plates.

$\frac{2538}{7.56} \times 1.9 = 640$

$\div 7.2 = 89$ rivets

L_0U_1

Same section as U_2U_3

$f_c = 16{,}000 - 70 \times \frac{46.1 \times 12}{12.4} = 12{,}880$

Area required $= \frac{2199}{12.88} = 171$ □"

Area furnished $= 185.9$ □"

Test for effect of wind stresses:—

Bending Moment $= 175\,000$ #

$f = \frac{P}{A} + \frac{Mc}{I} = \frac{219\,000}{185.9} + \frac{175\,000 \times 12 \times 18}{27030}$

$= 1180 + 1400 = 13{,}200$ #/□" actual

$1.25 \times 12{,}880 = 16{,}100$ #/□" allowable

$\frac{2199}{338} = 6.5"$ $=$ $6.5"$

Thickness of built up section $= \frac{3.0"}{3.5"}$

Use 2-$1\frac{3}{4}"$ pin plates.

$\frac{2199}{6.5} \times 1.75 = 592$

$\div 7.2 = 83$ rivets required

$U_3 U_4$

Cover $36 \times \frac{1}{2}$
2 Webs $38 \times \frac{3}{4}$
4 Ls $\angle 6 \times 6 \times \frac{3}{4}$
2 Fillers $26 \times \frac{3}{4}$
2 Bot. Pl. $6 \times \frac{3}{4}$
2 Webs $34 \times \frac{5}{8}$

$$\rho_{vv} = \sqrt{\frac{30450}{199.25}} = 12.37$$

$$z = \frac{347 - 9 \times 19.375}{199.25} = 0.87$$

$$f_c = 16{,}000 - 70 \times \frac{30 \times 12}{12.37} = 13{,}960$$

Area required $= \frac{2734}{13.96} = 196$ in²

Area furnished = 199.25 in²

$\frac{2734}{336} = 8.14''$ = 8.14"

Thickness of built up section = $\frac{4.25''}{3.90''}$

Use 2-2" pin plates.

$\frac{2734}{8.14} \times 1.95 = 654$
$\div 7.2 = 91$ rivets required

$U_4 U_5$

Cover $36 \times \frac{1}{2}$
2 webs $38 \times \frac{3}{4}$
4 L's $6 \times 6 \times \frac{3}{4}$
2 Fillers $26 \times \frac{3}{4}$
2 Bot. Pl. $6 \times \frac{3}{4}$
2 webs $36 \times \frac{3}{4}$

$$\rho_{vv} = \sqrt{\frac{32410}{210.75}} = 12.4 \qquad Z = \frac{173}{210.75} = 0.82$$

$$f_c = 16{,}000 - 70 \times \frac{30 \times 12}{12.4} = 13{,}965$$

Area required $= \frac{2918}{13.965} = 209^{\square''}$

Area furnished $= 210.75^{\square''}$

$\frac{2918}{336} = 8.5''$ $=$ $8.5''$

Thickness of built up section $= \frac{4.5''}{4.0''}$

Use 2-2" pin plates

$\frac{2918}{8.5} \times 2 = 687$
$\div 7.2 = 96$ rivets required

LATERALS

*Top laterals:-

$P_T = \frac{43.5 \times 12}{200} = 2.61$ use 2 Ls $3 \times 2\frac{1}{2} \times \frac{3}{8}$ $2\frac{1}{2}\lfloor^{3}$ $2\frac{1}{2}\lfloor_{3}$

$P_C = 3.1$ use 4 Ls $7 \times 3\frac{1}{2} \times \frac{3}{8}$ $\frac{7}{3\frac{1}{2}}\lfloor\frac{7}{3\frac{1}{2}}$ ⊥

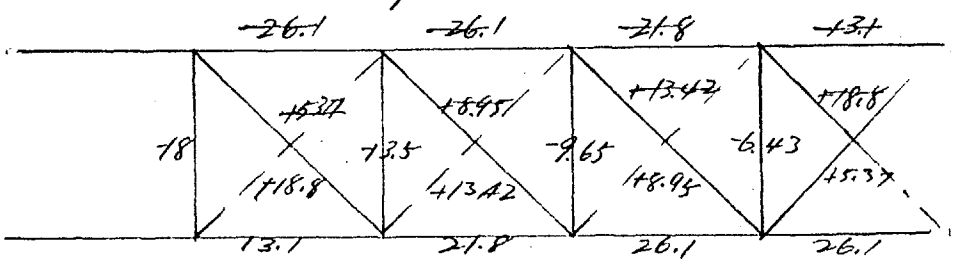
*Bottom laterals:-

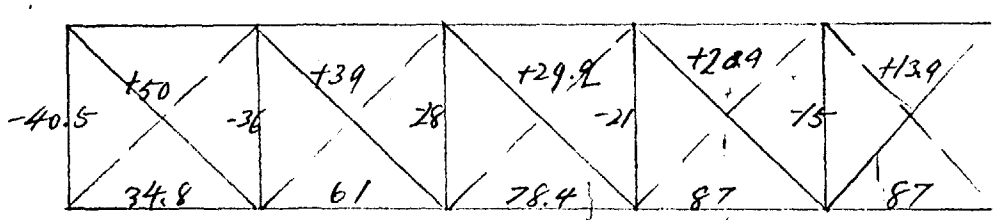
$P_T = 1.31$ use 2 Ls $8 \times 3\frac{1}{2} \times \frac{7}{16}$ $7\lfloor_{8}^{3\frac{1}{2}}$

Compression members are floor beams.

** The actual stresses in laterals as shown below are way within the limits of ~~25%~~ 60% of the stresses the above angles can carry therefore only the least radius of gyration is the controlling factor.

Top Laterals

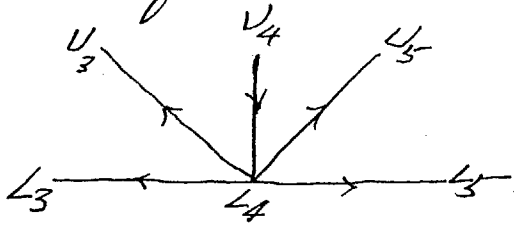




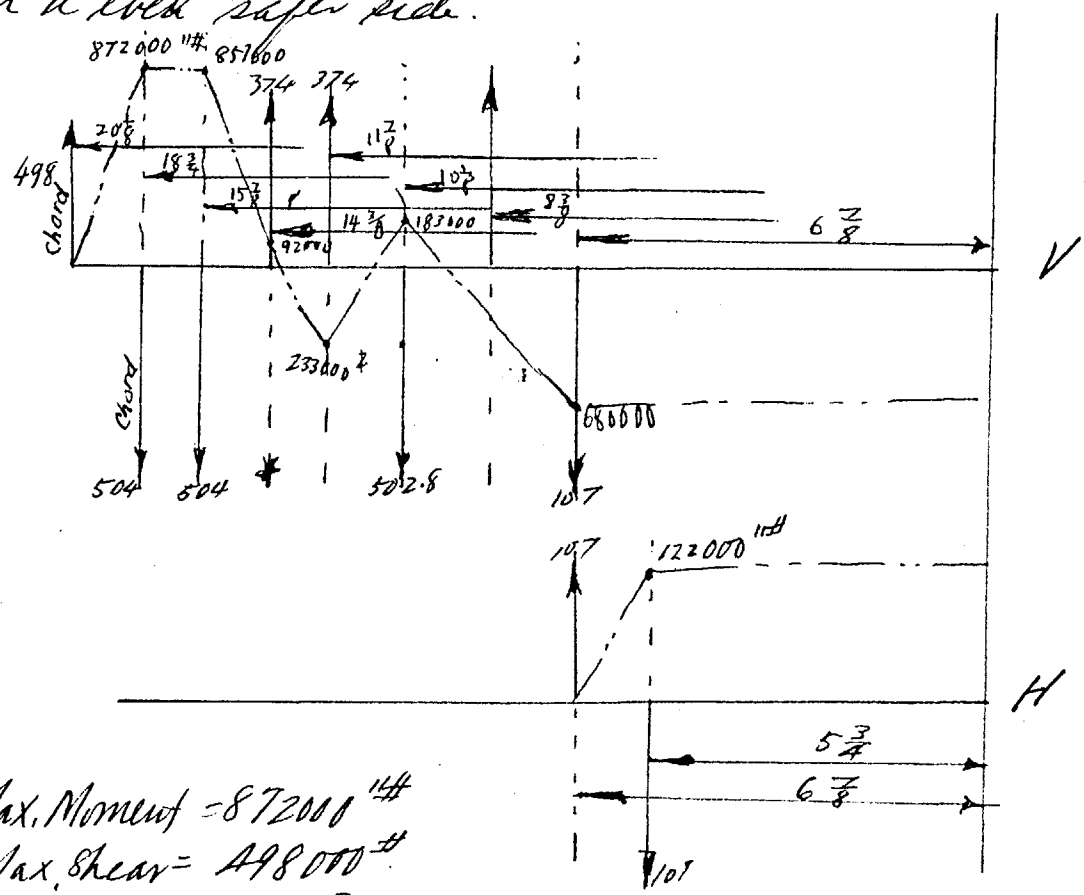
Bottom Laterals

PINS

Test of Moment and shear in pin at L_4



First try the case by assuming all the bars to be stressed to their respective maximums. If this can be safely resisted by the pin all the other cases must be on a much safer side.



Max. Moment = 872000 ##

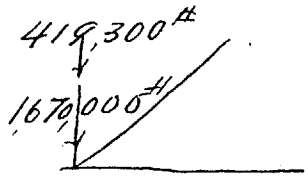
Max. Shear = 498000 #

Allowable B.M = $(14)^3 \times 0.098175 \times 16000 = 4,350,000$ ##

Area required for shear = 49.8 □"

Area furnished = $7^2 \times \pi = 154$ □" The reason for using such large pins is to keep down the number of eye bars used.

ROLLERS & BEARING PLATES



$\frac{2,089,300}{600 \times 10} = 350''$

Use 7-4'-2" rollers 10" dia.

$\frac{2089300}{600 \times 144} = 24.6^{\square''}$

Use 5'x5'x2" plate.

ACTUAL WEIGHT

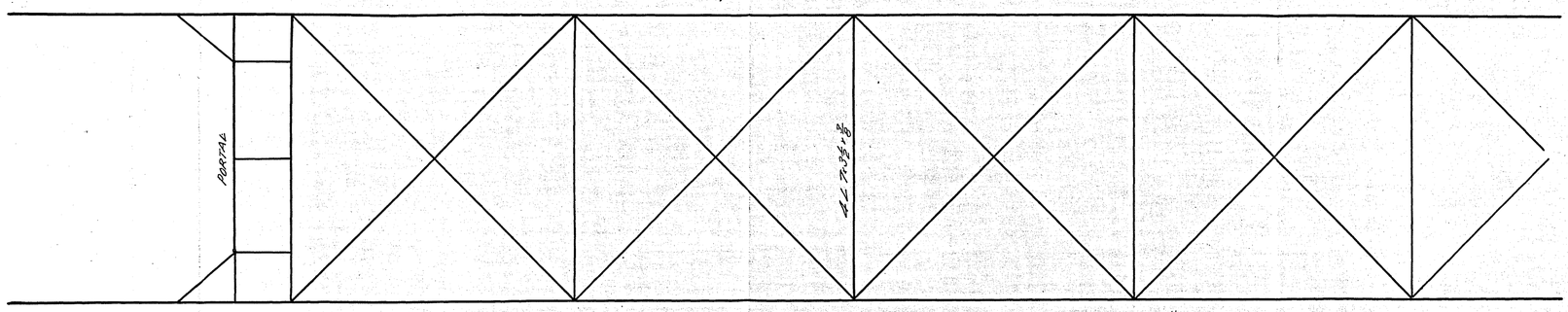
$U_4 U_5$	3.4 x 30 x 210.75 x 2	=	43 000
$U_3 U_4$	3.4 x 30 x 199.25 x 4	=	81 250
$U_2 U_3$	3.4 x 30.2 x 185.9 x 4	=	76 400
$U_1 U_2$	3.4 x 30.5 x 156.8 x 4	=	65 100
$L_0 U_1$	3.4 x 46.1 x 185.9 x 4	=	116 700
$U_1 L_1$	3.4 x 35 x 47.5 x 4	=	22 600
$U_2 L_2$	3.4 x 40.6 x 69 x 4	=	38 200
$U_3 L_3$	3.4 x 43.9 x 55 x 4	=	32 800
$U_4 L_4$	3.4 x 45 x 43.25 x 4	=	26 500
$U_1 L_2$	3.4 x 46.1 x 84 x 4	=	52 800
$U_2 L_3$	3.4 x 50.5 x 56 x 4	=	38 500
$U_3 L_4$	3.4 x 53.1 x 49 x 4	=	35 400
$U_4 L_5$	3.4 x 54 x 42 x 4	=	30 900
$U_4 L_3$	3.4 x 54 x 21 x 4	=	15 460
$L_0 L_1$	3.4 x 30 x 91 x 4	=	37 100
$L_1 L_2$	3.4 x 30 x 91 x 4	=	37 100
$L_2 L_3$	3.4 x 30 x 133 x 4	=	54 250
$L_3 L_4$	3.4 x 30 x 160 x 4	=	65 250
$L_4 L_5$	3.4 x 30 x 160 x 2	=	32 600
			901 900
	Details +50%		451 000
			1352 900

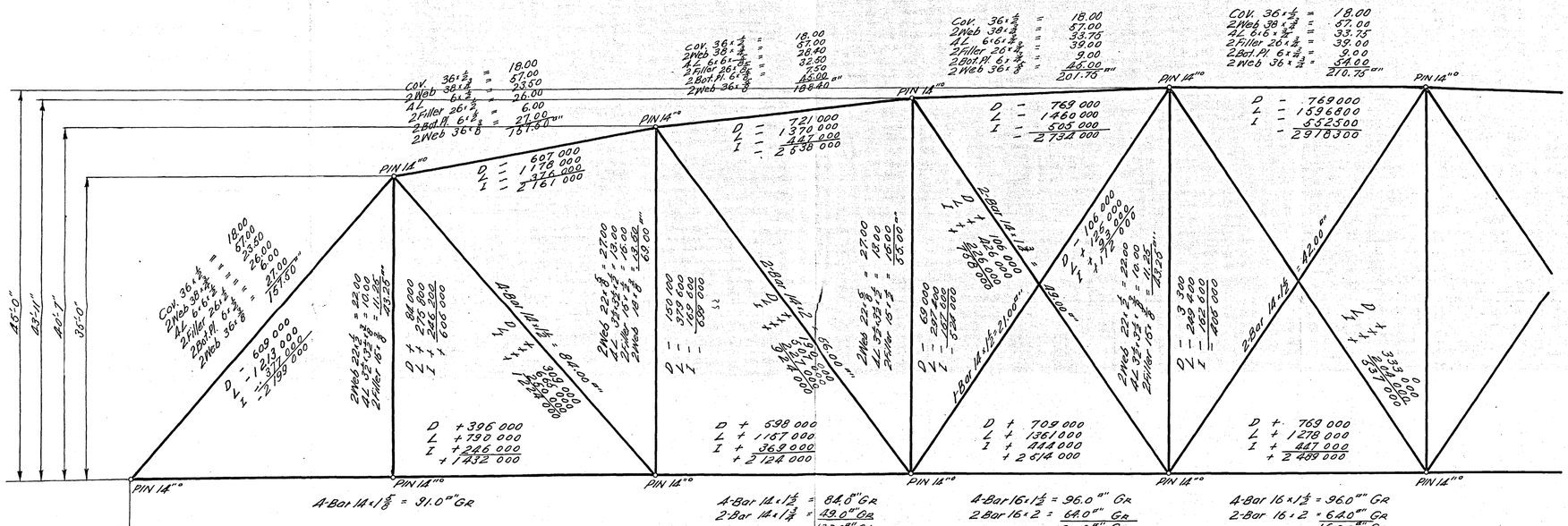
Floor system = 1640 x 10 + 4 x 7238 x 9
+ 850 x 270 = 655 000

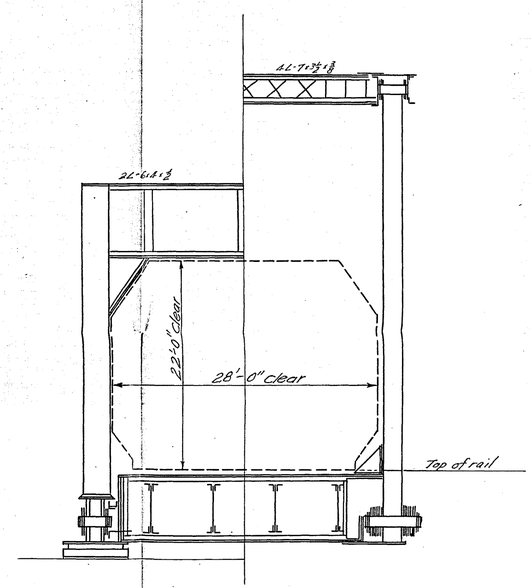
Total weight 2,007,900#

Assumed = 7700 x 270 = 2,080,000#

Top Laterals 2L-3×2½×⅜ (Diagonals)

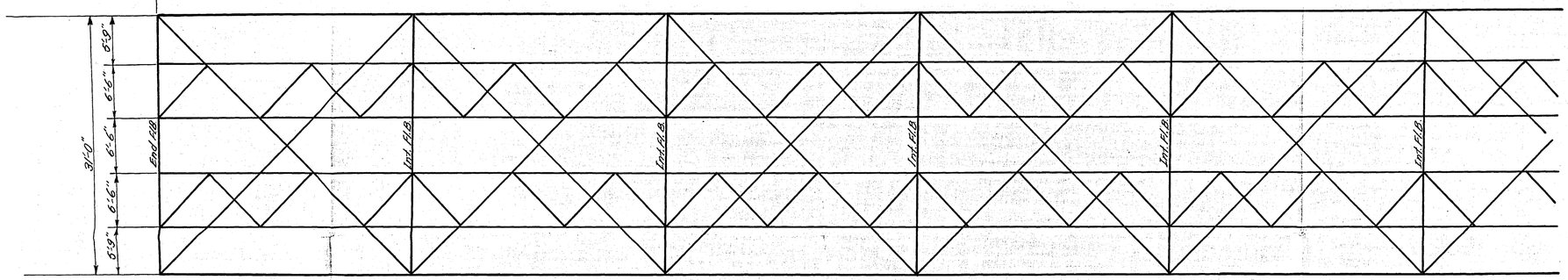






HALF END VIEW HALF SEC.

9 Panels of 30'-0" = 270'-0" C.C. END PINS

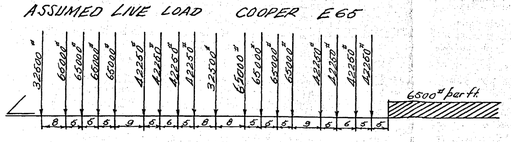


Bottom Laterals 2L 8×3½×7/16
Stringer Laterals L 3½×3½×⅜

ASSUMED DEAD LOAD

TRUSS	5220
FLOOR	1570
TRACK	850
	7620 # per lin ft.

ASSUMED LIVE LOAD COOPER E65



END FLOOR BEAMS

SHEAR	D	15 314#	MOMENT	D 2 075 000"#
	L	204 800#		L 20 850 000"#
	I	199 200#		I 20 300 000"#
		419 300#		43 225 000"#

INTERMEDIATE FLOOR BEAMS

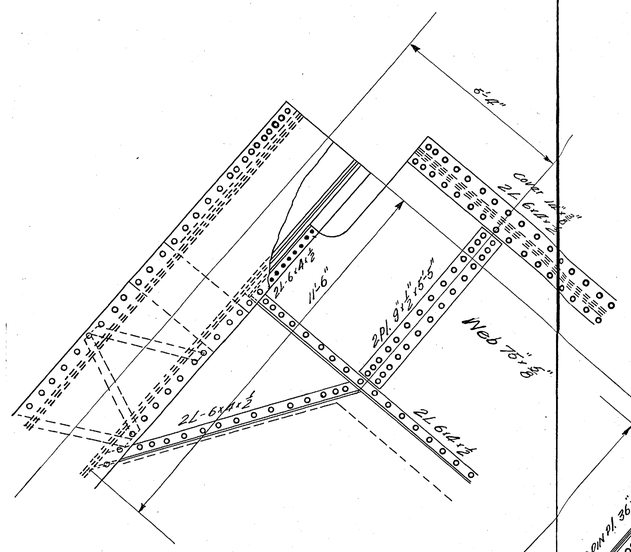
SHEAR	D	22878#	MOMENT	D 2 300 000"#
	L	316 600#		L 34 100 000"#
	I	282 000#		I 30 500 000"#
		620 300#		66 900 000"#
÷10000	=	62.05""	+82.47	= 810 000#
Web 84×¾	=	63.00""	÷16000	= 50.6"" NET.
			⅓ Web	= 7.9""
			2L-8×8×1	= 26.0""
			1 Pl. 18×⅞	= 9.25
			1 Pl. 18×½	= 7.00
				50.75 NET.

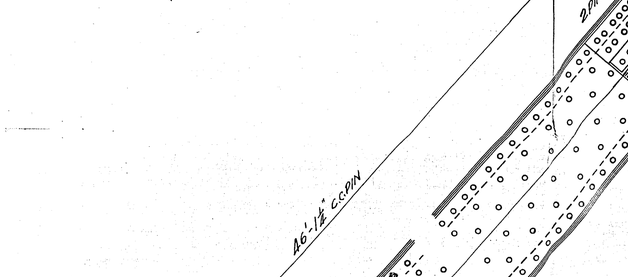
STRINGERS

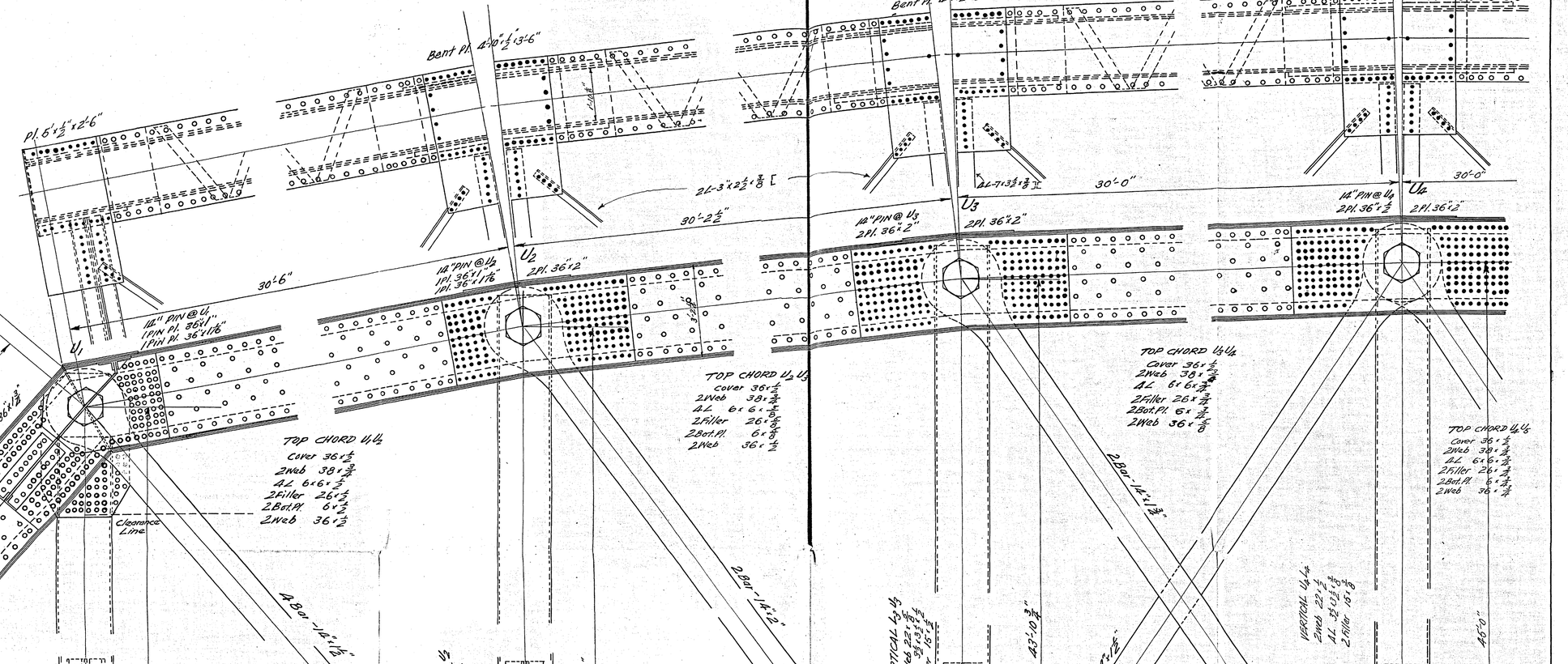
SHEAR	D	7320#	MOMENT	D 658800"#
	L	102 370#		L 7 919 000"#
	L	99 600#		I 7 788 000"#
		209 090#		16 365 800"#
÷10000	=	20.9""	÷ 57.4	= 285 000#
Web 60×⅜	=	22.5""	÷16 000	= 17.8""
			⅔ Web	= 2.82""
			2L-6×6×13/16	= 14.93
				17.75 NET.

SPECIFICATIONS — A.R.E.A. 1920
RIVETS — ⅞"
TIES — 8"×10"×10'-0"

DOUBLE TRACK
THRU PIN CONNECTED SPAN
270'-0" c to c. END PINS
Stress Sheet
Scale ⅛"=1' 5-29-24
C.W. Chen







DOUBLE TRACK THRU. PIN CONNECTED SPAN 270'-0" c to c. END PINS

A.R.E.A. 1920 SPECIFICATIONS
COOPER E65 LOADING
Scale 1/2" = 1'
5-27-24
C.W. Chen