Final Design of Tateqahana Bridge over Chikuma Gawa, Nagano Ken. 1.

This bridge is to be located on main highway between the city of Nagano and Nakano. The total length of this crossing is about 115 Ken. On account of flood complain made by residents it is advisable to make the span as long as possible reducing the obstruction into min.

We made comparative estimate of cost for 3-220' spans with two intermediate piers and 4-165' Steel spans with three intermediate piers and we found the cost of these two layouts is about the same but the former layout giving less obstruction to the river channel.

After conferring with Mr Nishiike, chief engineer of Nagano Ken, we had permit to go ahead with design of 220' spans. Mr Nishiike instructs us to make roadway 20' wide and road pavement of wood block.

Design of 220' span.
Loading of bridge

$$\text{Unif. Live Load } q^{kg/m^2} = \frac{100.000}{170+l} \quad \text{with max of } 500^{kg/m^2}$$

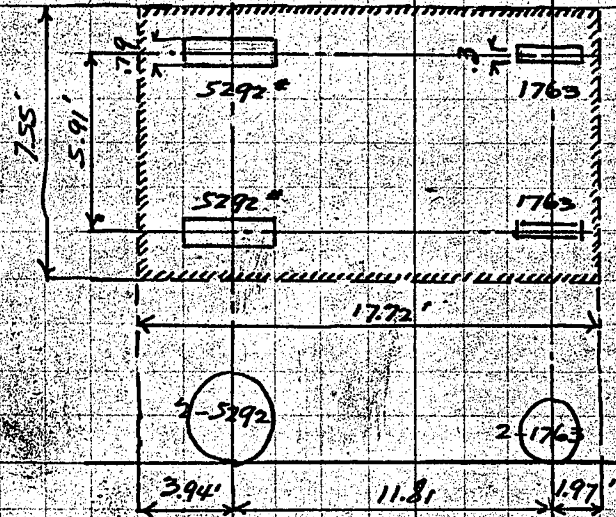
where l = span length in meter

For 220' span the uniform load will be $86^{\#}/\square'$

Motor truck Loading Total weight 6400 kg (14110#)
Distribution of wheel load is as follows.

Uniformly distributed load for 1 line of motor truck

$$\frac{14110}{17.72} = \text{say } 800^{\#} \text{ per lin. ft}$$



when the trucks are running in series, it is assumed the continuous line of the uniform load without impact; when the trucks are running side by side without train of trucks, impact is to be counted as specified below.

$$\text{Impact } I = \frac{L^2}{L+D} \quad L = \text{live load} \quad D = \text{Dead Load}$$

with maximum of 25%

For this bridge 2 lines of motor trucks + 4.9' of uniform load are to be taken.

Assumed working strength

Fibre stress in concrete	$650^{\#}/\square''$
Shear (reinforced)	$120^{\#}/\square''$
Shear (plain)	$40^{\#}/\square''$
Bond stress	$78^{\#}/\square''$
Tension in reinf. bars	$16.000^{\#}/\square''$
Shear in " "	$11.000^{\#}/\square''$
$fs \div fc = 15$	

Structural Steel

Tension		$16000^{\#}/\square''$
Compression		16000 - 70 ℓ/r
Bending Stress		16000
Shear	Shop rivet	12.000
"	field rivet	10.000
Bearing	Shop rivet	24.000
"	field rivet	20.000
Expansion roller		600d
On masonry		600
Specification		AREA 1910

The span is divided into 11 panels @ 20' each and the stringer spacing 4.5' with one ft overhang at ends.

Cross section is as shown in next page.

Roadway Slab	pavement 4" wood block	20#
	⅓" Sand mortar cushion	5
	concrete slab assumed	75
		100# per sq. ft

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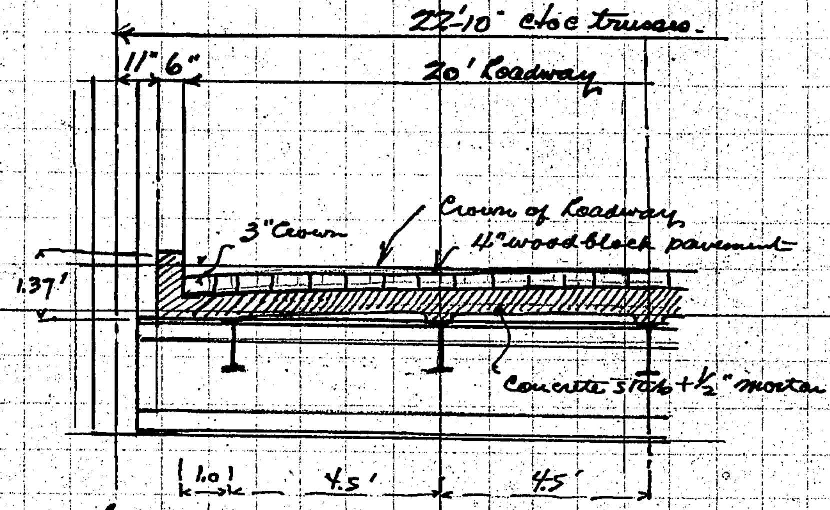
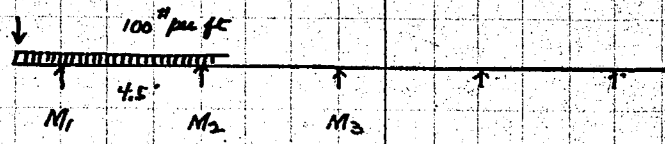
Cross section of Bridge

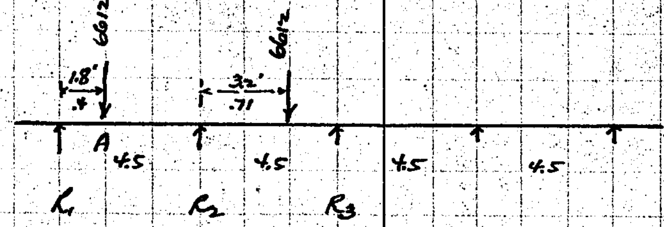


Dead Load.



Live Load	wheel concentration	5292
	Impact 25%	1320
		6612#

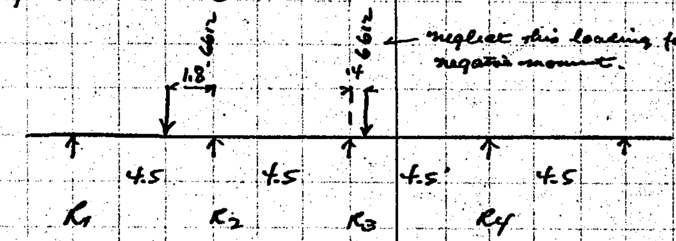
Positive moment

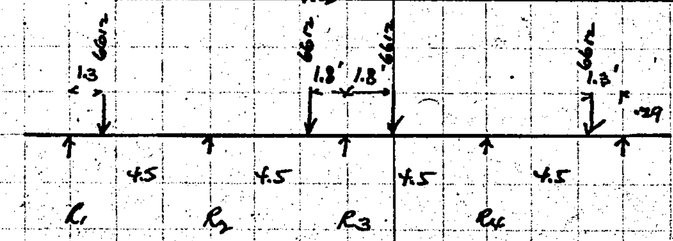


Max shear load on overhanging arm = 6612#

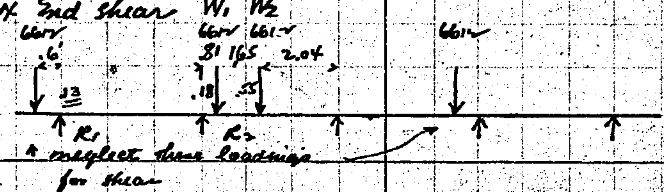
For one ft strip = 6612 ÷ 2 = 3306#

Negative moment





Max End shear



	Reaction R_1	Reaction R_2
W_1	− 0.030	0.820
W_2	− 0.070	0.500
	− 0.100 · 6612 = 660#	1.320 · 6612 = 8720

Shear at R_2 — 660

8.060# For one ft strip 4030#

Weight of curb = 0.5 · 1.37 @ 150 = 103#

Moment of overhang

curb	103 · 1.25	=	129 1#
Roadway	$100 \cdot \frac{1^2}{2}$	=	50
			179 1#

max neg. moment M_2

.107 · 100 · 4.5² = 217

Less $179 \cdot \frac{4}{15}$ = 48

169 1#

Max Pos. moment for end span

Zero shear at 2.09' from end support

Extra reaction due to overhanging moment

$= \frac{179}{4.5} =$ say 40#

moment = 40 · 2.09 = 83

Pos. continuous span

0.077 · 100 · 4.5² = 156

239

Less 179

60 1#

For intermediate span say

0.036 · 100 · 4.5² = 73 1#

Distribution of load 1 + .75 = 1.75 call this 2.0'

Max Live load positive moment at .4 of end span

4.5 · .4 = 1.8'

Reaction R_1 = + 0.510

− 0.047

+ 0.463 · 6612 = 3060#

moment at A = 3060 · 1.8 = 5500 1#

For one ft strip 5500 ÷ 2 = 2750 1#

Longitudinal distribution 2'

Reduction of moment (approx) $\frac{6612}{2 \cdot 2} \cdot 0.5 = 825$ 1#

Resulting moment = 2750 − 825 = 1925 1#

Case I Approx max neg. moment at Second support

Reaction R_1 = + 0.295 · 6612 = 1950 1#

moment + 1950 · 4.5 = + 8770 1#

− 6612 · 1.8 = − 11900

− 3130 1#

For one ft strip 3130 ÷ 2 = 1565 1#

Case II Approx max neg. moment

	Reaction R_1	Reaction R_2
W_1	+ 0.640	0.45
W_2	− 0.062	0.44
	0.578 · 6612 = 3820#	0.89 · 6612 = 5890#

Moment at R_3

R_1	3820	· 9.0	=	34380
R_2	5890	· 4.5	=	26500
				60880
	6612	· 9.5'	=	− 62800
				1920 1#

For symmetrical loading 2 · 1920 = 3840 1#

For one ft strip 1920 1#

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Summary of Stresses

	Neg. moment	Pos. moment	End shear
Dead Load	169	60	280
Live Load	1920	1925	4030
	2089	1985	4310#

End shear Dead load $\frac{5}{8} \times 100 \times 4.5 = 280$

Effective Depth required $= \sqrt{\frac{2089}{107}} = 4.4"$

concrete stress = 650#/□"

Steel stress = 16000#/□"

Use 6" slab effective depth say 5.0"

Steel area $= \frac{2089 \times 12}{\frac{7}{8} \times 5 \times 16000} = .36$ □"

Use ½"# bars 6" ctrs = .39 □"

Unit End shear $\frac{4310}{\frac{7}{8} \times 5 \times 12} = 82$#

Shear of concrete 40

to be carried by stirrups 42#

Use 3/8"# stirrups as required.

Temperature Bars $5 \times 12 \times 0.003 = 0.180$" ½"# bars 15" ctrs about.

Main Reinforcing bars Use straight bars top and bottom 18" centers (Every third bar).

2 bars bent up at support

Stringer span length = 20'0" spacing 4.5'

neglect continuity of slab and take uniform reaction for stringer

Dead Load Floor $100 \times 4.5 = 450$#

Beam assumed 50

500#

End stringer $100 \times \frac{5.5^2}{2} = 1510$

$103 \times 5.75 = 590$

$2100 \div 4.5 = 466$#

$m = \frac{1}{8} \times 500 \times 20^2 = 25000$'#

Beam assumed 50

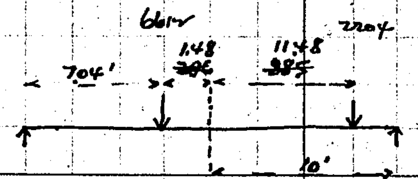
516

$m = \frac{1}{8} \times 516 \times 20^2 = 25800$'#

End shear $= 516 \times 10 = 5160$#

Live Load motor truck. Rear wheel with impact = 6612#

Front wheel " " = 2204#

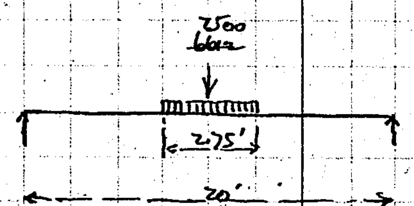


center of gravity

6612

$2204 \times 11.81 = 26050$

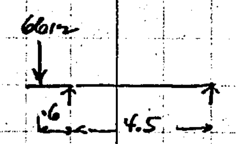
$8816 \times 2.95 = 26050$



Front wheel will drop outside of support, single load at center will give max moment

Longitudinal distribution of load = 2.75'

Load on End stringer neglecting continuity of slab.



$6612 \times \frac{5.1}{4.5} = 7500$#

$2204 \times \frac{5.1}{4.5} = 2500$

moment $3750 \times 10 = 37500$

Less $3750 \times \frac{2.75}{4} = 2580$

34920

max shear $7500 \times \frac{18.63}{20} = 7000$#

$2500 \times \frac{6.72}{20} = 840$

7840

Summary of stresses.

	moment	Shear
Dead Load	25800	5160
Live Load	34920	~~7000~~ 840
	60720'#	~~12160~~ 13000#

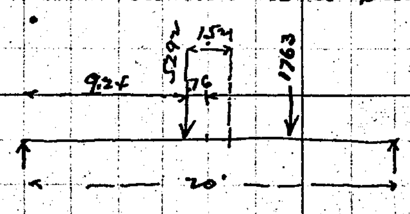
section modulus required $= \frac{60720 \times 12}{16000} = 44.5$ Use 15" × 42# I Sm = 58.9 ok

For shear 1.3□" Use standard connection

Live Load motor trucks coupled without impact Rear wheel 5292

Front wheel $1763 \times 6'$ say $= 10578$

6955 $1.52' = 10578$



moment $6955 \times \frac{9.24^2}{20} = 29700$'#

less $\frac{5292}{2} \times \frac{2.75}{4} = 1820$

27880'# $\times \frac{5.1}{4.5} = 31500$'#

Increase loading by $\frac{5.1}{4.5} = 1.133$

$5292 \times 1.133 = 6000$

$1763 \times 1.133 = 2000$

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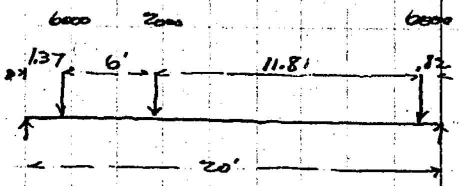
Stringer (Continued)

Max End shear due to live load



$6000 \times \frac{19.45}{20} = 5830$

$2000 \times \frac{6.72}{20} = 672$

6502

Moment and End shear in Case I will give max stress

Floor Beam Span length 22.83'

neglect continuity of Floor slab

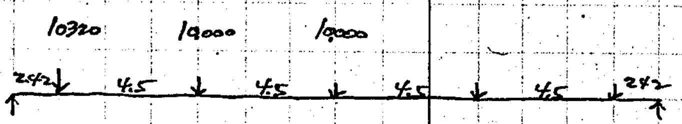
Dead Load

Concentration on int. stringer $100 \times 4.5 = 450$

beam say 50

$500 \times 20 = 10.000$ #

Concentration on End Stringer $516 \times 20 = 10320$



30320 25320
1140 1140
31460 26460

moment due to concentration

moment $\frac{25320}{30320} \times 11.42 =$ 289.000 / 346.000

less $10.000 \times 4.5 = 45000$

$10320 \times 9.0 = 93000$

Dead Load Floor Beam assumed 100# per ft

moment $= \frac{1}{8} \times 100 \times 22.83^2 = 6520$

138000
151000
208000

M. Dead Load Beam say 6500
157500
214500

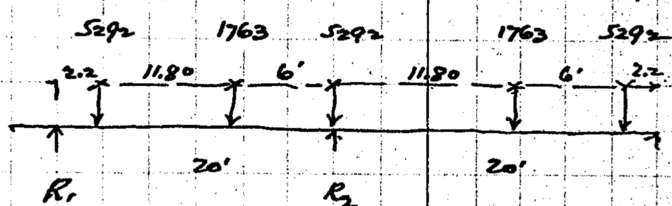
Live Load motor trucks without impact

Reaction R_2 $5292 \times \frac{4.4}{20} = 1200$

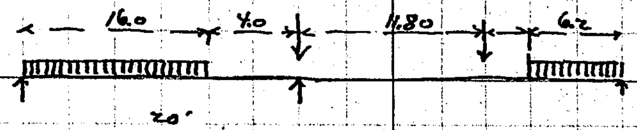
$1763 \times \frac{24.2}{20} = 2140$

5292

8632 #



Live Load motor truck without impact plus uniform load at front and rear.



Reaction from motor truck

$1763 \times \frac{8.2}{20} = 723$

5292

6015

uniform load 2-20' span 102.4#

$102.4 \times \frac{16^2}{2 \times 20} = 655$#

$102.4 \times \frac{6.2^2}{2 \times 20} = 98$#

753#

Uniform load $753 \times \frac{7.55}{2} =$ 2850

8865

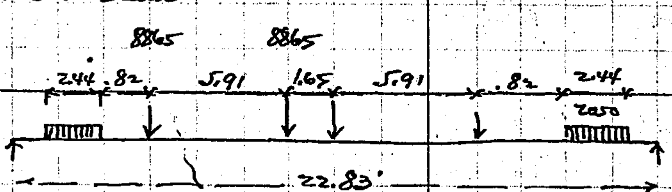
Live Load motor truck with impact

Reaction from motor truck $6015 \times 1.25 = 7520$ #

Floor Beam

uniform load $= 102.4 \times 20 = 2050$ #

Moment due to concentration



$17730 \times 11.42 = 202500$

Less $8865 \times 7.75 = -68700$

133800

17730
5000
22730

M. unif. load $5000\,(11.42 - 8.78) =$ 13200 (2.64)

147000

M. Full uniform load of 2050# per ft

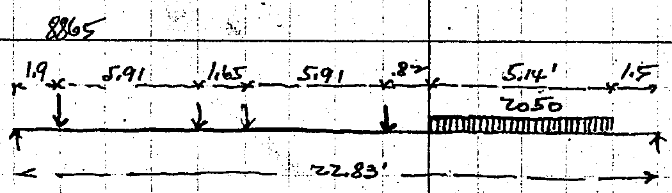
Reaction $2050 \times 10 = 20500$

moment $= 20500 \times 11.42 = 234500$

Less $2 \times 10250 \times 5 = 102500$

132000 #

End shear



23780

Unif. load $2050 \times 5.14 \times \frac{4.07}{22.83} = 1870$

Conc. $8865 \times$ 20.93
15.02
13.37
7.46
$56.78 \div 22.83 = 22000$

23780

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Floor Beam (Continued)

Summary of Stresses

	Moment	End shear
Dead Load	157500	25320
Live Load	147000	23780
	304500 #	49100 #

No of Rivets for end connection

49100 ÷ 4810 = 10 Single shear 7/8" Rivet

Rivet spacing $\frac{5469 \times 26.5}{49100}$ = 2.95" 5/16" web.

Try 30 × 5/16" web = 9.360" 1/8 web = 1.17

Back to back of Ls = 30 1/2" Effective Depth = 25.4 − 1.5 = 23.9 Stress = 304500 ÷ 2.39 = 128000

SR = 128000 ÷ 16000 = 8.00 − 1.17 = 6.83 □" net

Weight of one intermediate Floor Beam

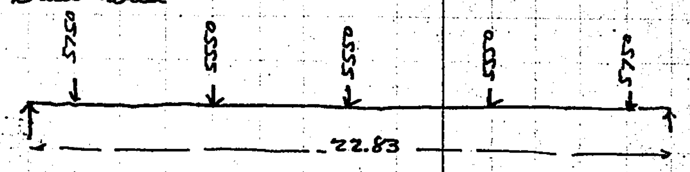
web	30 × 5/16	@ 31.88	× 22.0	=	700
Flanges	4Ls 5 × 3 1/2 × 1/2	@ 13.60	× 22.0	=	1196
Stiffeners	8Ls 4 × 3 × 5/16	@ 7.2	× 2.54	=	144
Fills	4Pls 3 1/2 × 1/2	@ 5.95	× 1.8	=	43
End stiff	4Ls 3 1/2 × 3 1/2 × 3/8	@ 8.5	× 2.4	=	82
Shelf Ls	10Ls 3 1/2 × 3 1/2 × 5/16	@ 7.2	× .67	=	48
Fills	10Pls 8 × 1/2	@ 13.6	× 1.25	=	170
	Rivet heads + variation 5%				124
					2607

2607 ÷ 22.83 = 114# per ft

2607 ÷ 20 = 130.3# per ft of bridge

End Floor Beam Projection of stringer 13"

Dead Load



Conc. on int. stringer 500 × $\frac{21.08^2}{20}$ = 5550

Conc on end stringer 516 × $\frac{21.08^2}{20}$ = 5750

moment due to conc.

Moment 14075 × 11.42 = 161.000

less 5550 × 4.5 = 25000

5750 × 9.0 = 51800

− 76800

84200

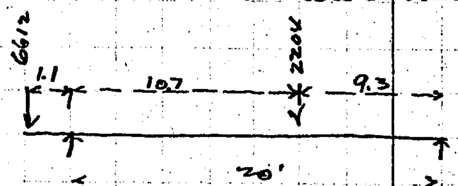
Dead load Floor Beam assumed 100# per ft

moment = 1/8 × 100 × 22.83² = 6500 #

Dead load Beam 6500

90700 #

Live Load one motor truck with impact; no uniform load

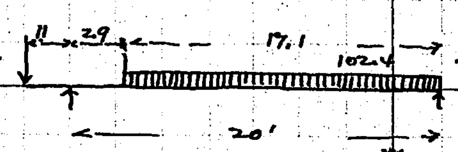


concentration 6612 × $\frac{21.1}{20}$ = 7000

2204 × $\frac{9.3}{20}$ = 1030

8030

Live Load one motor truck without impact followed by unif. load



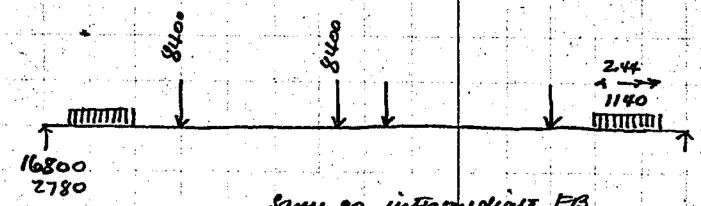
Uniform load 102.4 × $\frac{17.1^2}{2 \times 20}$ = 750#

Conc. 5292 × $\frac{21.1}{20}$ = 5570

750 × $\frac{7.55}{2}$ = 2830

8400# Use this concentration.

Floor Beam



Same as intermediate F.B.

See p. 4

Unif. load = 102.4 × $\frac{21.1^2}{2 \times 20}$ = 1140#

Moment due to concentration

16800 × 11.42 = 192000

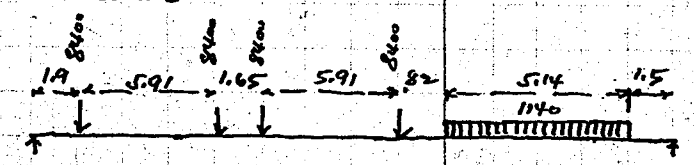
less 8400 × 7.75 = − 65100

126900

Unif. load 2780 × 2.64 = 7300

134200

End Shear



Unif load 1140 × 5.14 × $\frac{4.07}{22.83}$ = 1040

conc. 8400 × $\frac{56.78}{22.83}$ = 20850

21890#

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End Floor Beam (continued)

Summary of Stresses

	Moment	End shear
Dead Load	90700	15215
Live Load	134200	21890
	224900	37105

Try 30 × 5/16" web = 9.360□" 1/8 web = 1.17

Back to back of Ls = 30½" Effective = 239'

Stress = 224900 ÷ 239 = 94000#

SR = 94000 ÷ 16000 = 5.87 − 1.17 = 4.70 net

Use 2Ls 5 × 3½ × 3/8 = 6.10 gr or 5.35□" net

weight of one end Floor Beam

web	30 × 5/16	@ 31.88	× 22.0 =	700	
Flanges	4Ls 5 × 3½ × 3/8	@ 10.4	× 22.0 =	915	
Stiffeners	8Ls 4 × 3 × 5/16	@ 7.2	× 2.5 =	144	
End Stiffs	4Ls 3½ × 3½ × 3/8	@ 8.5	× 2.4 =	82	
Fills	4Pls 3½ × 3/8	@ 4.46	× 1.8 =	32	
Shelf Ls	5Ls 3½ × 3½ × 5/16	@ 7.2	× .67 =	24	
Fills	10Pls 8 × 3/8	@ 10.20	× 1.25 =	128	
	Rivet heads & variation			100	
				2125	2125 ÷ 22.83 = 93# per ft

Cantilever Bracket at End Panel.

Projection 13"

Live Load motor truck with impact 6612# Assume uniform load of 6612 ÷ 1.08 = say 6000#

moment = 6000 × .54 = 3240'# Depth 15" Effective depth say 1.1 Stress = 3540 ÷ 1.1 = 3220#

Dead load .516# × $\frac{108^2}{2}$ = 300 SR = 3220 ÷ 16000 = 0.20□"

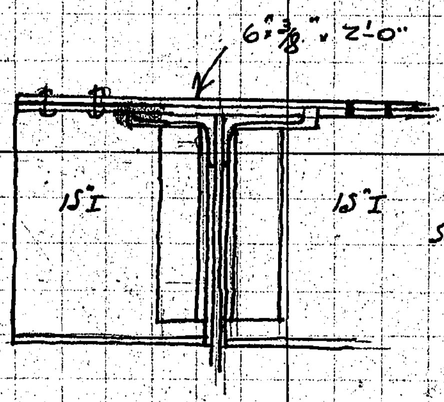
3540'#

weight of one bracket

	15"I	@ 42#	× 1'-0" =	42.0
splice	1Pl. 6 × 3/8	@ 7.65#	× 2.0 =	15.0
conn.	2Ls 3½ × 3½ × 3/8	@ 8.5	× 1.0 =	17.0
	Rivet heads etc	say		6.0
				80.0#

5 brackets @ 80# = 400# at one end

800 for both ends.



Scale 1" = 1'-0"

Hand Rails.

Top Rail 40# side pressure m = 1/8 × 40 × 20² = 2000'# Sm = $\frac{2000 \times 12}{16000}$ = 1.5

Or try 10' panel with center post Use 4 × 3 × 3/8" 8.5#

m = 1/8 × 40 × 10² = 500'# Sm = $\frac{500 \times 12}{16000}$ = .375 bottom rail 3" × 3" × 1/4" @ 4.9#

use 3 × 3 × 1/4" @ 4.9# Sm = .58

verticals 2 × 1½ × 1/4 @ 2.77# spaced 6" ctrs

weight of Handrail

Top rail	3 × 3 × 1/4	@ 4.9	× 1.0 =	4.9
bottom rail	3 × 3 × 1/4	@ 4.9	× 1.0 =	4.9
verticals	3 − 2 × 1½ × 1/4	@ 2.77	× 2.2 =	12.2
Posts	2Ls 3 × 3 × 1/4	@ 4.9	× 8.0/10 =	4.9
				26.9
	Rivet heads etc	say		3.1
				30.0 per lin ft

For 2 lines of handrail @ 30 = 60# per ft

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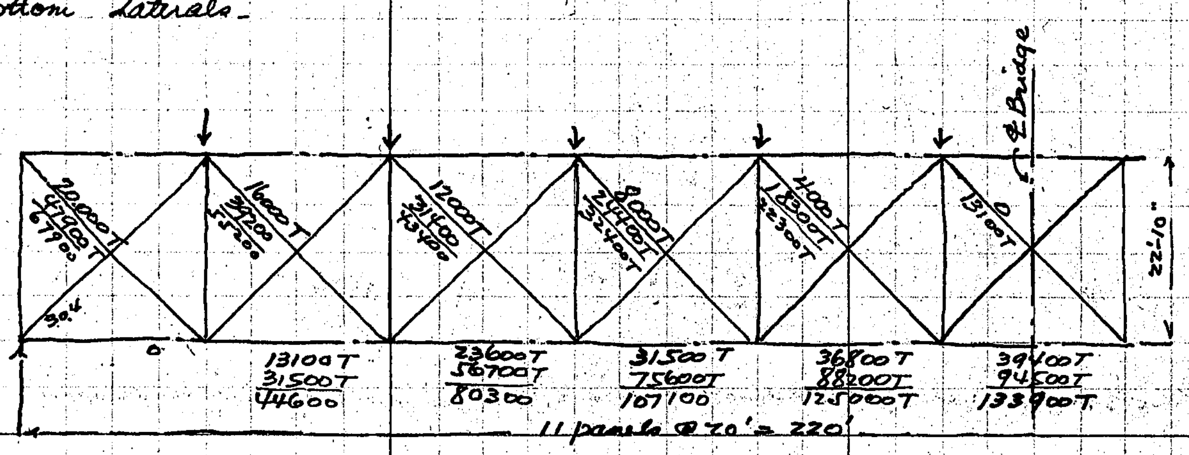
Bottom Laterals.



$\sec\theta = 1.33$

$\tan\theta = .152\ .875$

$W_1 \tan\theta = .875 \times 3000 = 2625$

$W_2 \tan\theta = .875 \times 7200 = 6300$

$W_1 \sec\theta = 3990$

$W_2 \sec\theta = 9560$

Static wind load 30# per sq ft of exposed surface.

Assume 5 × 30# = 150# per lin ft panel conc = 150 × 20 = 3000#

Moving wind load 12' × 30 = 360# per ft panel conc = 360 × 20 = 7200#

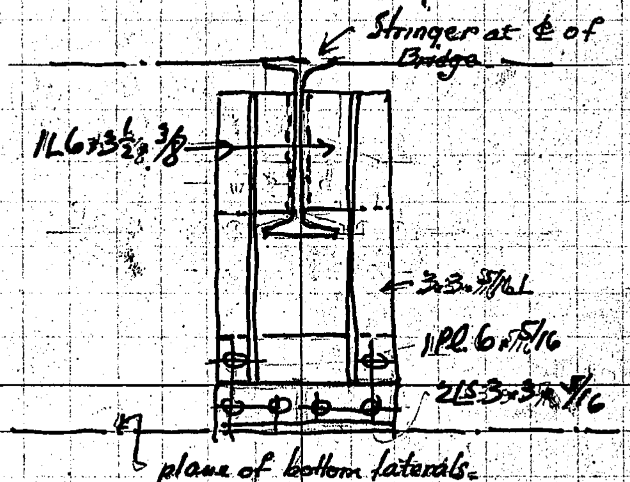
$l/r = 150$

$r = \frac{15 \times 12}{150} = 1.2$

2Ls 4 × 3 × 5/16 riveted back to back

$r = 1.2$ cir. ok

Stresses	Unit stress	S.R. net	section	gross	net
67900	16000	4.24 □"	2Ls 5 × 3½ × 5/16	6.10	5.34
55200	"	3.45	2Ls 4 × 3 × 5/16	4.18	3.56
43400	"	2.71	do		
32400	"	2.02	do		
22300	"	1.40	do		
13100	"	.81	do		



Details of Tie at intersection of bottom lateral bracing.

Weight of Bottom Laterals.

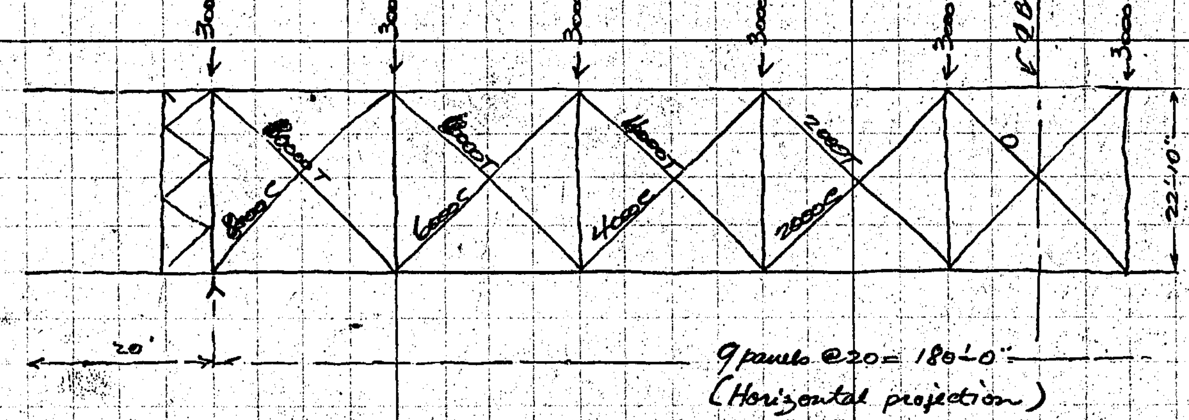
4Ls 5 × 3½ × 5/16 @ 8.7#	× 29.0	= 1010		
18Ls 4 × 3 × 5/16 @ 7.2	× 29.0	= 3750		
center connection + details say		= 200		
		2 @ 4960	=	9920

Details of Tie

2Ls 6 × 3½ × 5/16 @ 9.8	× .83	= 16		
2Ls 3 × 3 × 5/16 @ 7.2	× 1.2	= 17		
2Ls 3 × 3 × 5/16 @ 7.2	× 2.0	= 29		
1Pl 6 × 5/16 @ 6.38	× 1.2	= 8		
Rivet heads etc		10		
		11 @ 80#	=	880
				10800

10800 ÷ 220 = 49# per lin ft of bridge.

Upper Laterals.



Stiff members are used for diagonals

Static wind load 30#/□' assume wind load 150# per lin ft conc = 3000#

On account of curved top chord, the stress in the bracings are somewhat larger than those shown above. Effect is very small neglect this effect

use 4Ls 3 × 2½ × 5/16" @ 5.6#][← lace

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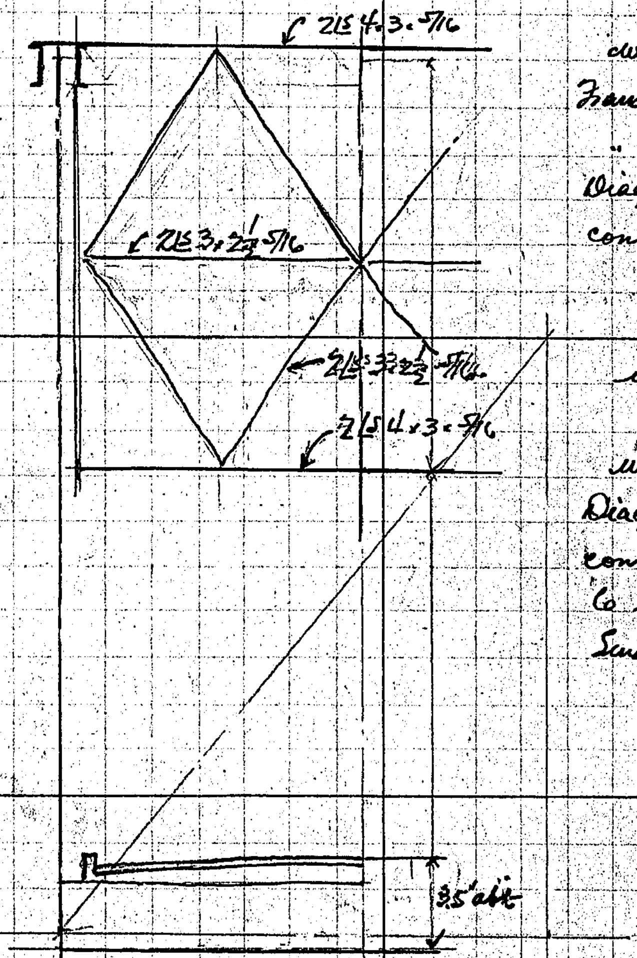
（独立行政法人 土木研究所蔵）

Sway Bracing



Transverse strut length say 22' l/r = 150

r required = $\frac{22 \times 12}{150}$ = 1.76 Use 2 [s 4 x 3 x 5/16" = r = 2.0 abt

weight of sway bracings

Trans.	4 [s 4 x 3 x 5/16	@ 7.2#	x 22.0 =	634
"	2 [s 3 x 2 1/2 x 5/16	@ 5.6	x 22.0 =	246
Diag.	16 [s 3 x 2 1/2 x 5/16	@ 5.6	x 9.0 =	805
connection pls + details say				200
				1885#

For average say 1800# each

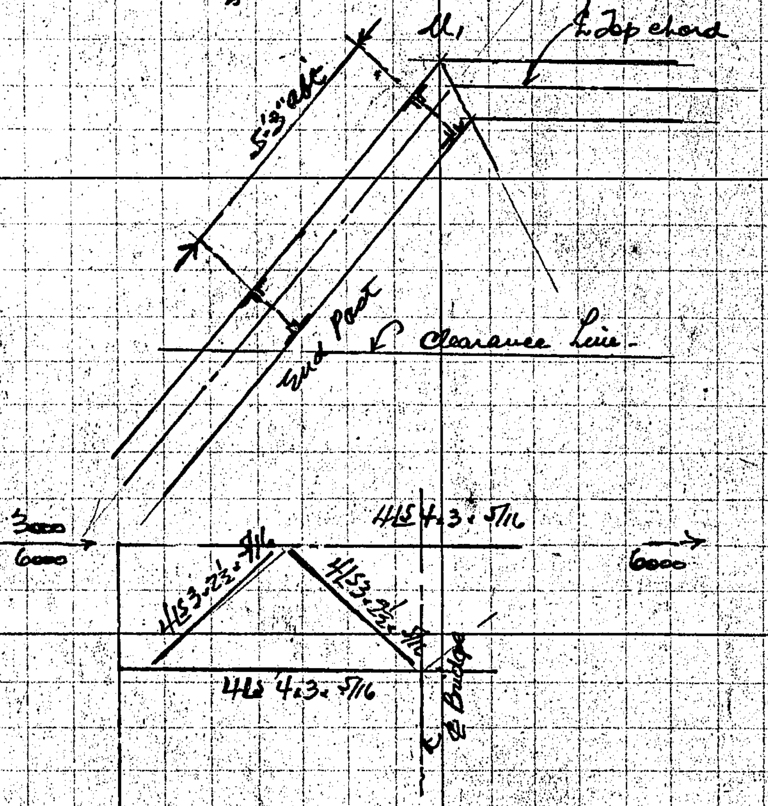
weight of sway bracing 8 @ 1800 = 14400#

weight of upper lateral bracing

Diag.	8 [s 3 x 2 1/2 x 5/16 @ 5.6	x 29.0 =	1300
connection pls say			40
6 tie pls @ 20#			120
Single lacing 8# @ 50' abt			400
			1860#

Upper laterals 9 @ 1860 = 16740#

Portal Bracing



weight of Portal Bracing

8 [s 4 x 3 x 5/16 @ 7.2 x 22.0	=	1270
16 [s 3 x 2 1/2 x 5/16 @ 5.6 x 6.5	=	582
connection pls 5 @ 50	=	250
Lacing bars say 8 x 50 lin ft	=	400
Tie Pls 15 @ 30	=	450
		2952

call this 3000#

2 @ 3000 = 6000#

Summary of weight of metal in Sway bracings Upper laterals & portal bracing

Sway bracing	14400
Upper Laterals	16740
Portal bracing	6000
	37140#

37140 ÷ 220 = 169# per lin ft

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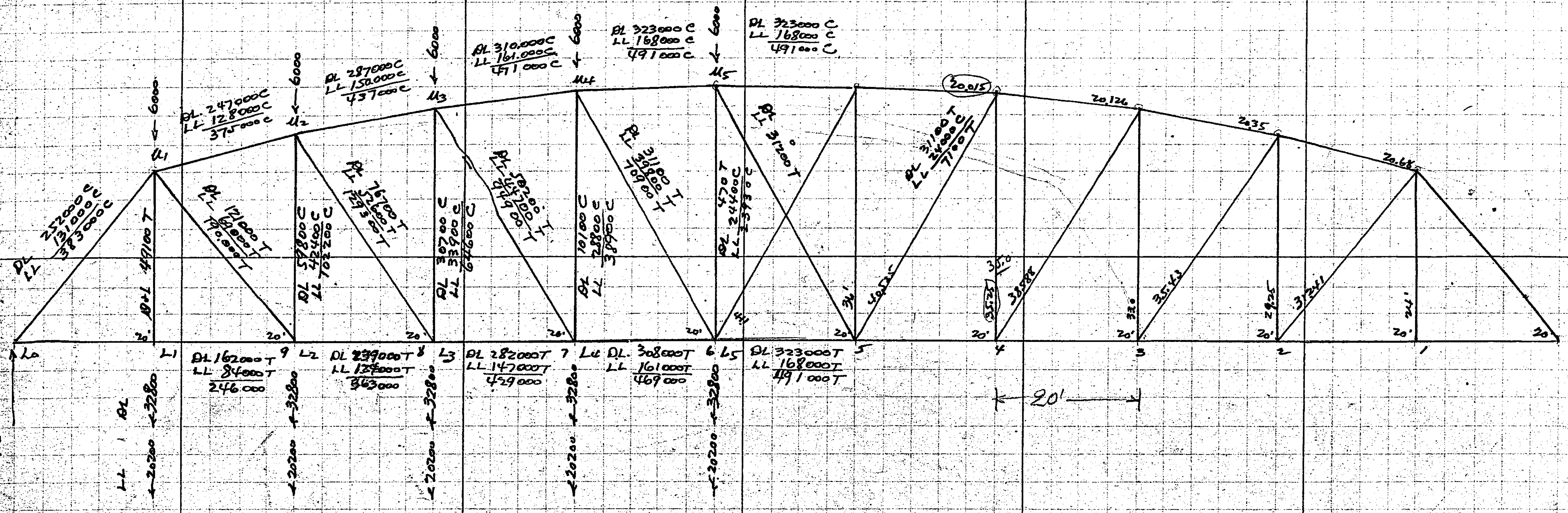

Panel concentration

Lower Panel load

2 Lines of Handrails	2@30 =	60#
Stringers	5@50 =	250
Intermediate Floor beam		130
Lower Laterals		49
Trusses lower half		430
		919

Roadway Pavement + Slab.

100 × 20 = 2000

curb 2@105 = 210

misc fill say 150

2360

3279

Load per truss 3279 ÷ 2 = 1640#

Lower Panel concentration 1640 × 20 = 32800#

Upper Panel Point

upper Laterals	169
Trusses upper half	430
	599 ÷ 2 = 300#

Panel conc = 300 × 20 = 6000#

Live Load

2 Lines of motor truck @800 = 1600

Unif. load 4.9' × 86 = 420

2020#

Panel concentration 2020 × $\frac{20}{2}$ = 20200#

$\sec\theta$ = 31.241 ÷ 24 = 1.30

$\tan\theta$ = 20 ÷ 24 = .833

					R_1	R_2
111.50 × $\frac{24}{31.24}$ = 85.60	1	1	0.091 × 20200 =	1840		
176.0 × $\frac{29.25}{35.43}$ = 145.25	2	3	0.273	5500	38500	
313.4 × $\frac{33.0}{38.588}$ = 268.20	3	6	0.546	11000	49600	
960.0 × $\frac{35.25}{40.525}$ = 835.00	4	10	0.909	18400	62400	
	5	15	1.365	27300		
29.25 × $\frac{20}{20.68}$ = 28.30	6	21	1.910	38600		
33.00 × $\frac{20}{20.35}$ = 32.42	7	28	2.547	51500		
35.25 × $\frac{20}{20.126}$ = 35.00	8	36	3.272	66000		
36.00 × $\frac{20}{20.015}$ = 35.99	9	45	4.090	82500		
	10			101000		

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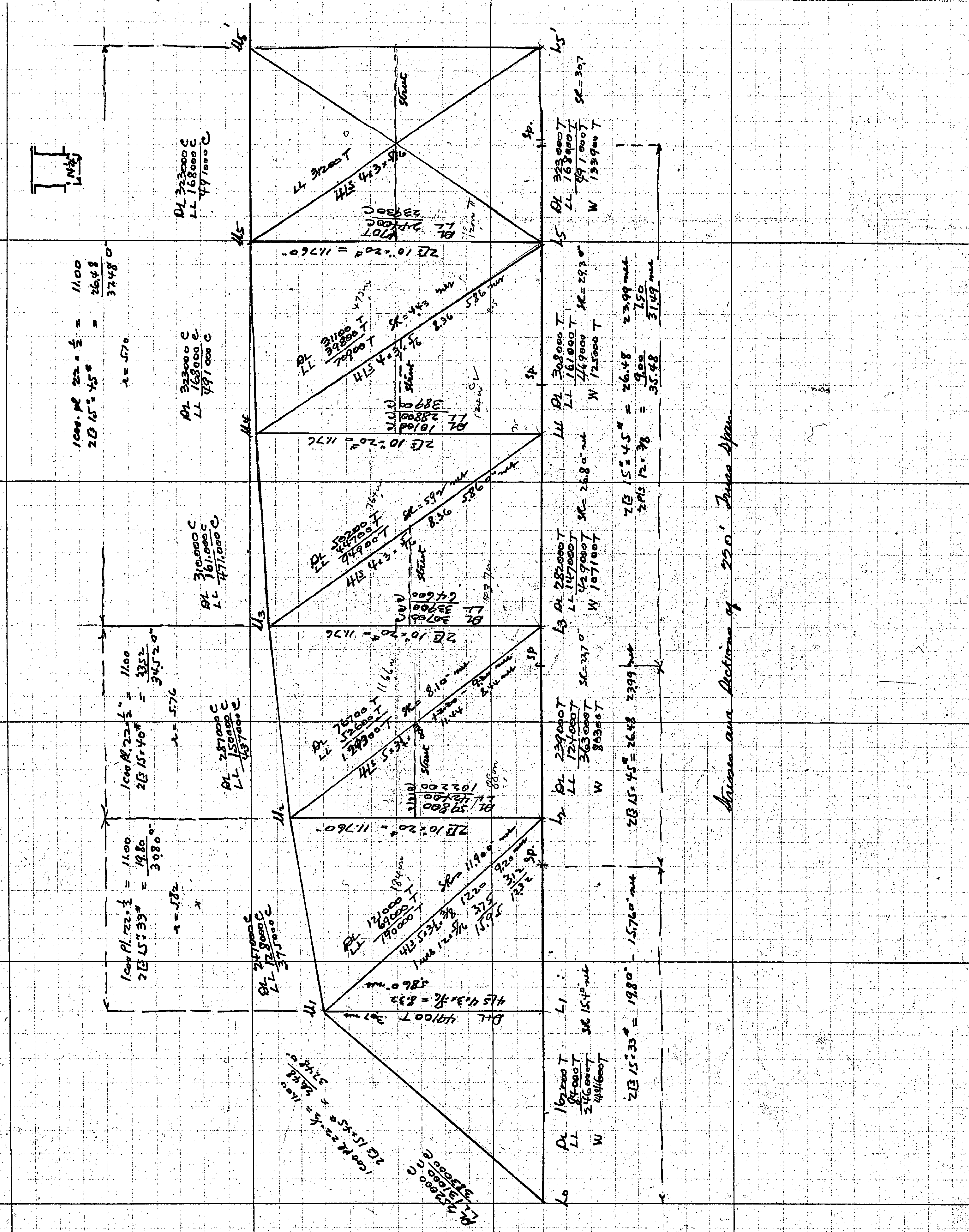

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Top chord section

U_5-U_5' S = 491000# C 1 cov Pl 22 × 1/2 = 11.00 fs = 16000 − 70 l/r l = 20 × 12
2 ⊏ 15 × 45# = 26.48 r = 5.70
37.48
fs = 13060
SR = 491000 ÷ 13060 = 37.60"

U_4-U_5 same as U_5-U_5'

U_3-U_4 S = 471000 C SR = 471000 ÷ 13060 = 36.10"

U_2-U_3 S = 437000 C 1 cov Pl 22 × 1/2 = 11.00 r = 5.76 l = 20.35 × 12
2 ⊏ 15" × 40# = 23.52
34.52 fs = 13040 #/□"
SR = 437000 ÷ 13040 = 33.50 □"

U_1-U_2 S = 375000 C 1 cov Pl 22 × 1/2 = 11.00 r = 5.82 l = 20.68 × 12
2 ⊏ 15" 33# = 19.80 fs = 13020#
30.80
SR = 375000 ÷ 13020 = 28.8 □"

L_0-U_1 S = 383000 C Try 1 cov Pl 22 × 1/2 = 11.00 fs = 16000 − 70 l/r r = 5.70
case I 2 ⊏ 15 × 45# = 26.48 l = 31.2 × 12
37.48 fs = 11400#
SR = 383000 ÷ 11400 = 33.60"

Taking effect of wind stress it seems to be ok with the following section.
case II. 1 cov Pl 22 × 1/2 = 11.00
2 ⊏ 15 × 40# = 23.52
34.52 □"

moment due to wind load: wind load = 7500# point of contra flexure = 12.6'
moment = 7500 × 12.6 = 94500'# fs = $\frac{94500 \times 12 \times 11}{1956.5}$ = 6370 #/□"

Unit stress = 16000 − 70 $\frac{31.2 \times 12}{7.52}$ = 12520 12520 × 1.30 = 16300
6370
9930 #/□"
SR = 383000 ÷ 9930 = 38.6 □"

Try case I. for section
wind load moment = 94500'# fs = $\frac{94500 \times 12 \times 11}{2120.7}$ = 5890 #/□" f = 16300 as above
5890
10410#
SR = 383000 ÷ 10410 = 36.8 □" ok
20500
403500 ÷ 10410 = 38.6 □" ok

Direct load due to wind
15000 × $\frac{31.2}{22.83}$ = 20500#

増田淳氏関係資料
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Metal in bridge (one span)

2 lines of Handrails @ 30# per ft	= 60.0	. 222.25'	=	13340
Stringers 5 @ 50#	= 250.0	. 222.25	=	55600
Intermediate Floor Beams		10 @ 2607	=	26070
End Floor Beams		2 @ 2125	=	4250
Lower Lateral Bracings				10800
Upper Lateral Bracings				16740
Sway Bracings		8 @ 1800	=	14400
Portal Bracings		2 @ 3000	=	6000
Weight of two trusses		860 . 220	=	189200
On piers.		40 . 220	=	8800
				345200

345200 ÷ 2240 = 154 tons for one span

Roadway Pavement and slab	100 . 20 =	2000	
curb	2 @ 105 =	210	
misc fills say		150#	
			2360
Live Load 2 lines of motor trucks	@ 800 =		
Uniform load	4.9' . 86 =	1600	
		420	
			2020
			4380# per ft

Total load on pier 4380 . 222.25 = 974.000
metal in bridge 345200
1319200

For one shoe 1319200 ÷ 4 = 330.000 #

Bearing on masonry 25" . 39" = 975 □"

Unit bearing pressure 330.000 ÷ 975 = 338# per sq inch.

Length of roller 600 d 4" roller 2400# per lin inch

330.000 ÷ 2400 = 137.5 lin. inches net

Use 5 rollers @ 25" net = ~~135~~ 125"

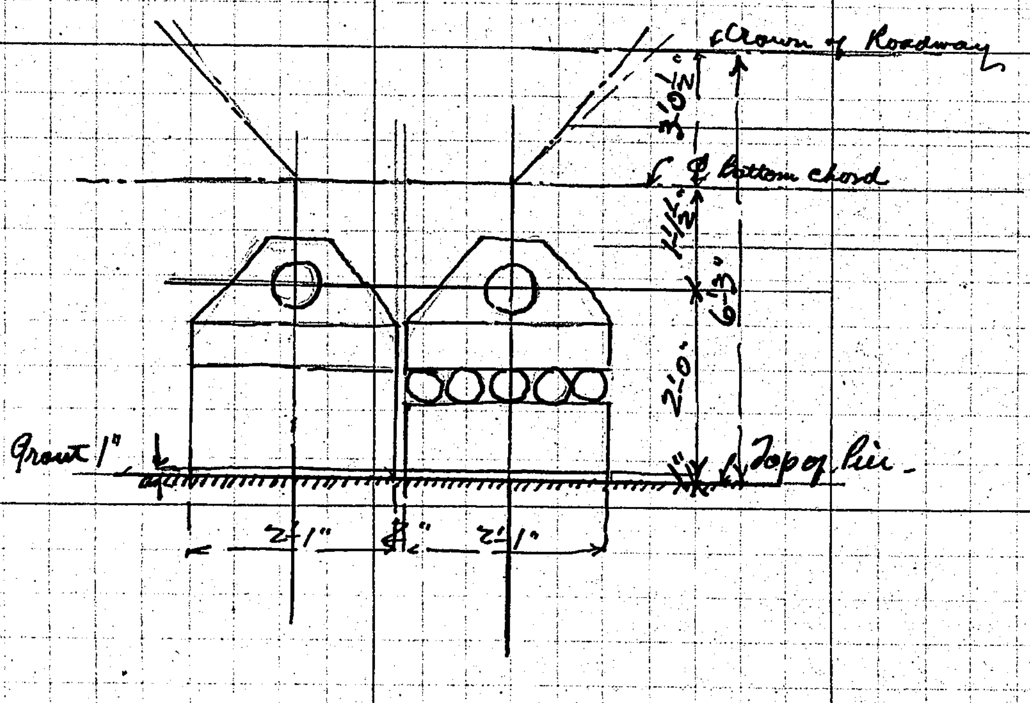
Size of pin at panel point L0 Assume 6" pin Unit bearing 6 . 24000 = 144000#

Thickness of bearing pls 330.000 ÷ 144.000 = 2.29"

1.14 per rib.

Bending moment of pin ? 330.000 . 1.39" = 458000"#

Fibre stress of pin 24.000#/□" 6" pin good for 508900"# Ok



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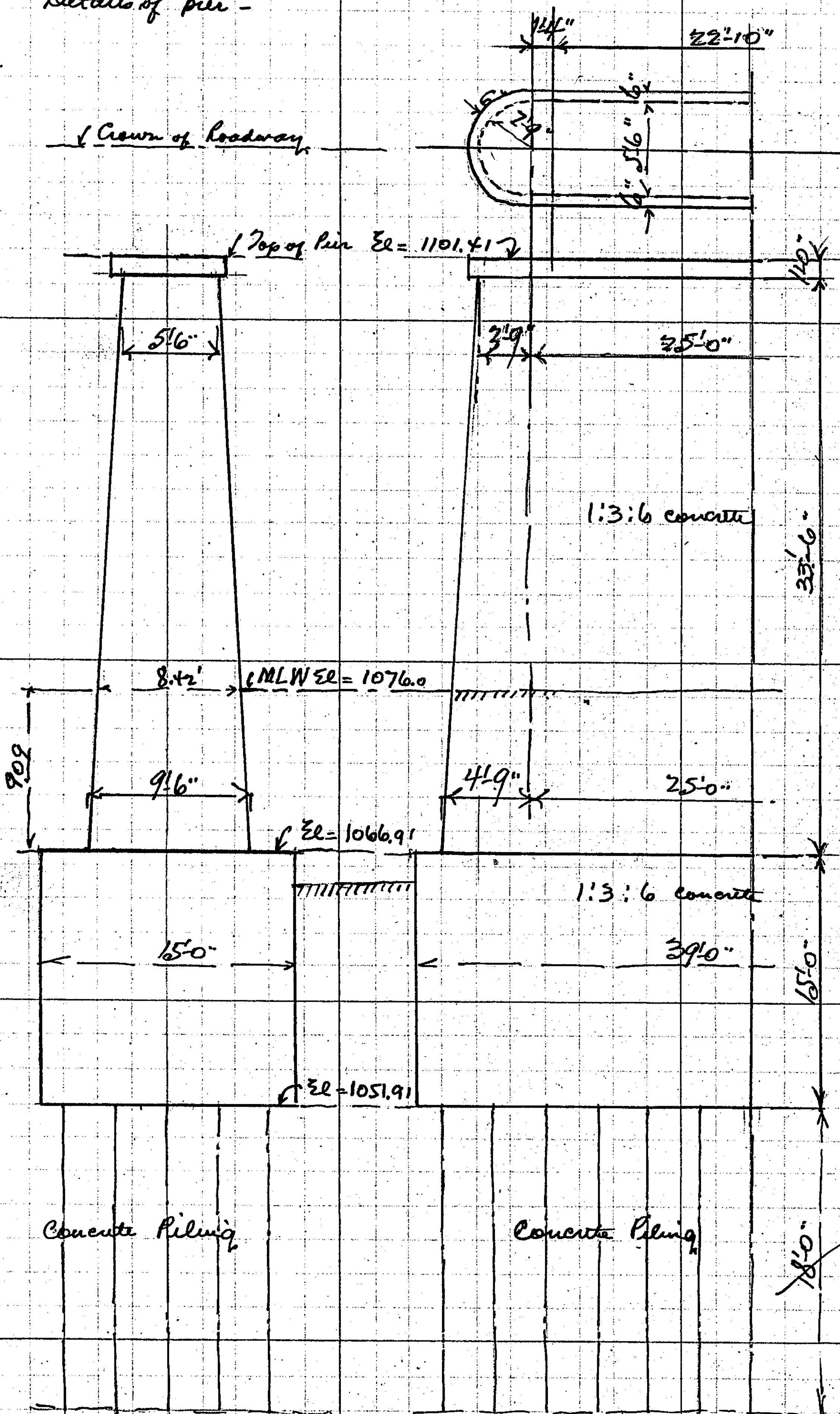
Details of pier -



Volume of Concrete
coping 6 × 25 = 150
6.5' dia 33.2
183.2
vol = 183.2 ÷ 216 = 0.85 立坪

Shaft
top 5.5 × 25 = 137.5
5.5' dia 23.7
= 161.2
Bott 9.5 × 25 = 237.5
9.5' dia 70.9
308.4
469.6
average area = 234.8 □'
vol = $\frac{234.8 \times 33.5}{216}$ = 36.4 立坪

Base $\frac{15 \times 39 \times 15}{216}$ = 40.6 立坪

Total concrete in pier
coping .85
Shaft 36.40
Base 40.60
77.85 cubic 坪
wt of cubic tsubo 216 × 150 = 32,400#

weight of pier = 77.85 × 32.400 = 2,520,000
Superimposed load 1,319,000
3,839,000

Bearing pressure under base
$\frac{3,839,000}{15 \times 39}$ = 6550 #/□'
2.92 tons/□'

Load on concrete piles
3,839,000 ÷ 65 = 59000#
59000 ÷ 2240 = 26.3 tons

No of Concrete piles 13 × 5 = 65
lin. ft of concrete piles 65 × 18 = 1170 lin ft

concrete in shaft below M.L.W. El = 1076.0
area 25 × 8.42 = 211.0
8.42 dia 55.7
266.7
bottom area 308.4
575.1 ÷ 2 = 287.6 □'
volume = $\frac{287.6 \times 9.09}{216}$ = 12.1 立坪

base 40.6
52.7 call this 60 坪 below M.L.W line
25.15 call this 27 坪 above M.L.W line.

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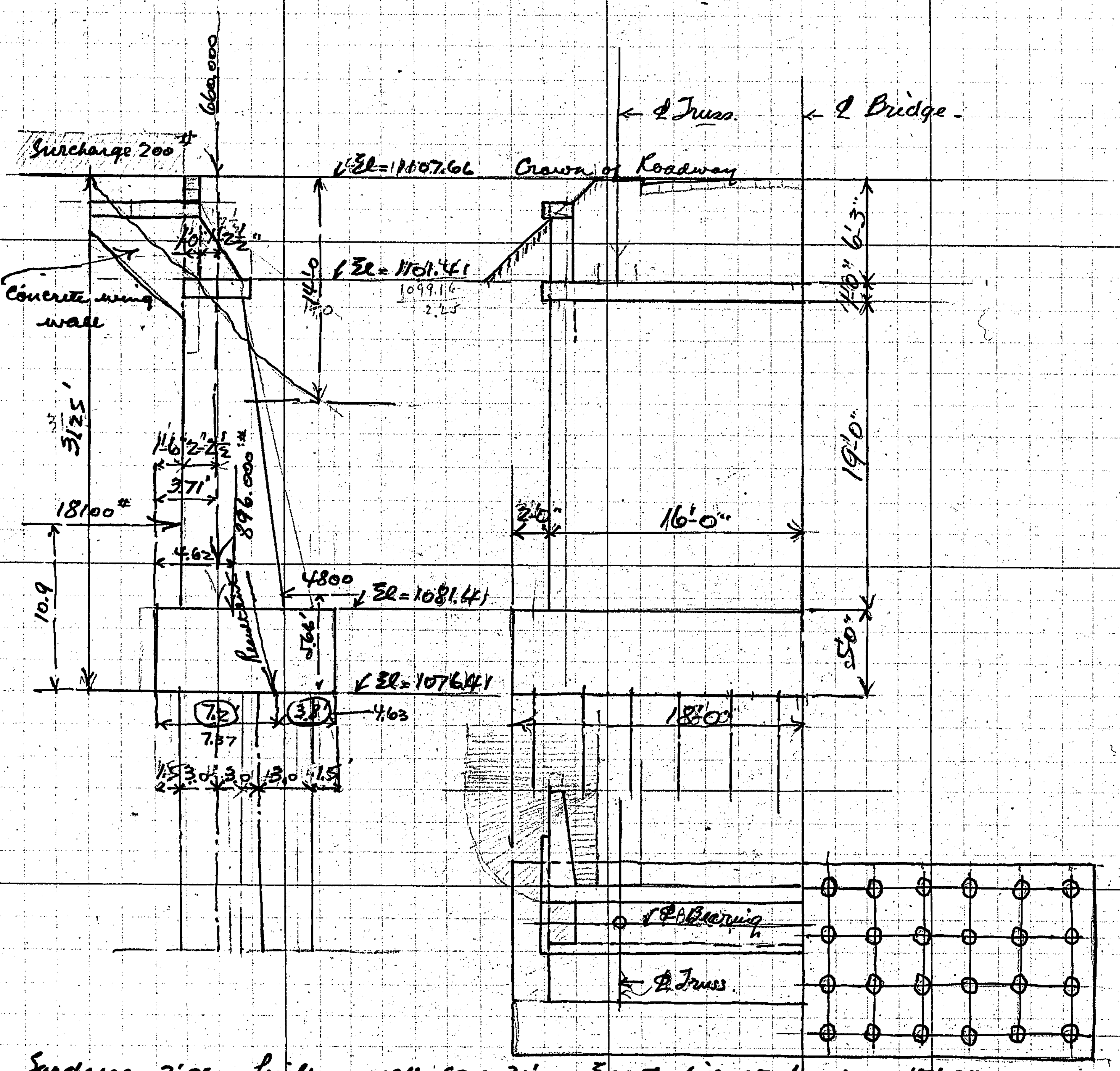
Details of Abutment



Surcharge 2'-0" height of wall say 31' Earth fill at front = 17'-0"
Earth pressure assumed 1/3 of weight. weight of earth assumed 100# per cub. ft
Total pressure on wall = $\frac{1100 + 66}{2} \times 31$ = 18100#
center of gravity of horizontal force = $x = \frac{(33-2)^2}{6(33+2)} = \underline{4.6}'$

Point of application = 15.5 - 4.6 = 10.9'

Earth pressure at front 1700 × 1/3 = 566# Total P = $\frac{566}{2} \times 17$ = 4800#
Point of application of resultant 17 ÷ 3 = 5.66'
Superimposed load 660.000# at centre line of bearing.
Approximate weight of Concrete abutment

				weight		M about heel
back wall	6' × 1' × 32.0'	= 192 cub. ft @ 150	=	28800	× 2.0 =	57600
coping	4.25 × 1 × 33.0	= 140 "	" =	21000	× 3.62 =	76000
shaft	$\frac{3.75 + 8.00}{2} \times 19 \times 32$	= 3580	" =	537000	× ~~7.37~~ 4.43 =	2380000
base	5 × 11 × 36	= 1980	" =	297000	× 5.5 =	1633000
wings	say	80	=	12000	× 0 =	—
		5972		895800#	4.62'	4146600

5972 ÷ 216 = 27.6 cubic tsubo.

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Details of abutment continued. weight Arm

Earth at front approximate $3 \times 12 \times 36' = 1296$ @100 = 129600# 9.5'

Earth at back approximate $1.5 \times 28 \times 36' = 1510$ @100 = 151000# 0.75'

Earth fill at sides of shaft $11 \times 16 \times 4.0 = 704$ @100 = 70400# 5.5'

Moments about heel due to various forces.

Superimposed load	660.000	× 3.71	=	2450.000	
Body of abutment	896.000	× 4.62	=	4130.000	
Earth fills	129600	× 9.50	=	1230.000	
"	151.000	× 0.75	=	113.000	
"	70400	× 5.5	=	387.000	
	1907000				+ 8310.000
Hor. Force	18100 ×	10.90 × 32			+ 6320.000
	4800 ×	5.66 × 32			− 870.000
					13760.000#

Point of resultant at bottom of base $13.760000 \div 1907.000 = 7.20'$ from heel.

From toe $11.0 - 7.20 = 3.80$ inside of middle third

Toe pressure $\frac{1907.000}{11 \times 36}\left(1 \pm \frac{6 \times 1.7}{11}\right) = 9200$#/□' or 350#/□' (4800, .926)

Approximate load on pile at toe $\frac{9200 \times 3.5 \times 3}{2240} = 43$ tons per pile.

This load is too much for single pile.

Let us change the size of base from 11'-0" to 12'-0"

Base = $12 \times 5 \times 36 = 2160$ @ 150	= 324.000#	×	6.0	= 1945 000
shaft	537.000	×	4.43	2380.000
wings	12.000	×	0	
back wall	288.000	×	2.0	576.000
coping	210.000	×	3.62	76000
	922800		4.83'	4458600

Moments about heel due to various forces.

Superimposed load	660.000	× 3.71	=	2450.000	
body of abutment	923 000	× 4.83	=	4460.000	
Earth fills	173 000	× 10.0	=	1730.000	
"	151.000	× 0.75	=	113.000	
"	70400	× 5.5	=	387.000	
	1977.400				+ 9140.000
Hor. Force	18100 × 10.90 × 32 =				+ 6320.000
	4800 × 5.66 × 32 =				− 870.000
					14590.000

Point of Resultant $14590.000 \div 1977.400 = 7.37'$

Edge distance $12.0 - 7.37 = 4.63$

Eccentricity 1.37

Toe pressure $= \frac{1977.400}{12 \times 36}\left(1 \pm \frac{6 \times 1.37}{12}\right) = 6640$#/□' or 1420#/□' (4570, 1.69)

max load on pile 3' spacing $\frac{6640 \times 9}{2240} = 26.6$ tons per pile. ok.

Assuming no horizontal force $9140.000 \div 1977.400 = 4.62$

Load on pile (3' spacing) same as for the above. 26.6 tons per pile.

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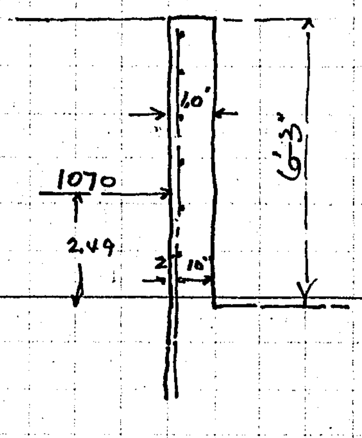
Details of abutment (continued)
Reinforcement of back wall
surcharge 2'



$x = \frac{(H-h)^2}{6(H+h)} = \frac{(8.25-2)^2}{6 \times 10.25} = .63'$

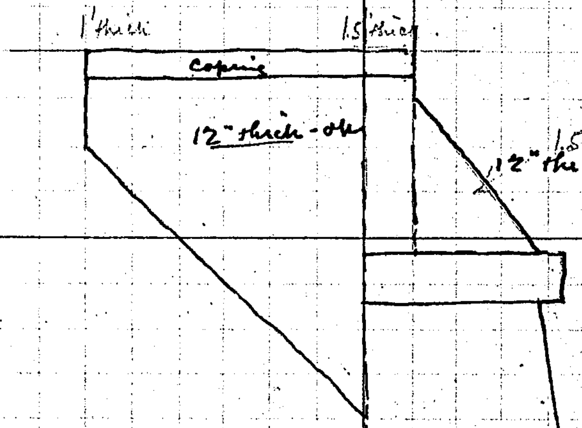
$P = \frac{67+275}{2} \times 6.25 = 1070$

moment $= 1070 \times 2.49 = 2660^{\#}$ $\quad R = \frac{2660 \times 12}{12 \times 10^2} = 26.6$

Reinforcing bar $= \frac{2660 \times 12}{\frac{7}{8} \times 10 \times 16000} = 0.23$ □" use $\frac{1}{2}$"φ bars 1'-0" ctrs

Reinforcing bar at front face 2'-0" ctrs
Temperature bars 18" ctrs each face

Details of wing.



Average thickness of wall say 1.25
projection 6'0"
moment

$2 \times 6 \times 1.25 \times 150 = 2250 \times 3 = 6750$
$18 \times 1.25 \times 150 = 3380 \times 2 = \underline{6760}$
13510

Effective Depth say 84"
Reinforcing bars req'd $= \frac{13510 \times 12}{\frac{7}{8} \times 84 \times 16000} = .137$ □"

use 2-$\frac{1}{2}$"φ bars.

Horizontal Pressure at 4' depth assumed H.P. = say $130^{\#}$
moment $= 130 \times 6 \times 3 = 2340^{\#}$
Effective Depth $= \sqrt{\frac{2340 \times 12}{12 \times 104}} = \sqrt{22.4} = 4.7$" use 12" wall.

Reinforcing bars req'd $= \frac{2340 \times 12}{\frac{7}{8} \times 10 \times 16000} = 0.20$ use $\frac{1}{2}$"φ bars 12" ctrs.

増田淳氏関係資料
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Quantities of materials –
Structural Steel – 152 tons per span (All figures made by Naganoken)
3 spans @ 152 = 456 tons

Reinforcing steel in Floor slab.

Transverse bars straight $\frac{2}{1.5}$ × ½"ϕ @ 0.67# × 21' = 18.8
" " bent $\frac{2}{1.5}$ × ½"ϕ @ 0.67 × 23 = 20.5
~~longitudinal bars~~ 20 × ½ϕ @ 0.67 × 1.0 = 13.4
long. bars at curbs 8 × 3/8 @ 0.38 × 1.0 = 3.1
stirrups. $\frac{2}{1.5}$ × ¼ @ 0.17 × 3.75 = .9
56.7
laps @ 10% 5.67
62.37 call this 63#

63# × 220 = 13860# per span
3 @ 13860# = 41,580# or 18.6 tons
may be ok with 18 tons

Reinforcing steel in abutments
wing 2 – ½"ϕ @ .67 × 16.0 = 21.4
2 – ½"ϕ @ .67 × 13.0 = 17.4
Hor bars 12 – ½"ϕ @ .67 × 7.0av = 56.0
vert bars 6 – ½ @ .67 × 5.5av = 22.0
116.8 × 2 = 233.6
In Front wall vert 66 – ½ @ .67 × 7.7 = 341.0
" " hor 32 @ .67 × 17.0 = 364.0
938.6
For 2 abutments 2 @ 940 = 1880# or .84 ton

Total Reinforcing bars say 19. tons without reinf bars in concrete piling

Concrete in piers Nos 1 + 2
Concrete below El = 1076.0 Base $\frac{15 \times 39 \times 15}{216}$ = 40.60 立坪
Shaft 12.10
52.70 立坪 below M.L.W. line
25.15 立坪 above M.L.W. line.

2 @ 52.70 = 105.4 立坪
2 @ 25.15 = 50.3 立坪

Concrete piling below bottom of base 16' × 65 = 1050 lin ft
2 @ 1050 = 2100 lin ft.

Excavation 1' larger than base
$\frac{17 \times 41 \times 31}{216}$ = 100 立坪 for piers Nos 1 + 2

Concrete in Abutments 28.6 立坪

Excavation $\frac{14 \times 38 \times 23}{216}$ = 57 立坪 2 @ 57 = 114 立坪

concrete piling 16 × 48 = 768 2 @ 768 = 1536 lin ft under base

wood block pavement $\frac{670 \times 20}{36}$ = 372 立坪

Concrete in Floor slab. 21.0 × 0.5 = 10.5
2 – 1.37 × 0.5 = 1.37
fillets .5
$\frac{12.37 \times 670}{216}$ = 38.2 立坪

増田淳氏関係資料
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Estimate of Cost

Structural Steel, erected + painted	456 tons @ 370.00	=	168720.00
Concrete in Floor slab.	38.2 坪 @ 240.00	=	9168.00
Reinforcing Steel in Floor slab + abutments	19.0 tons @ 230.00	=	4370.00
wood block pavement	372.0 坪 @ 45.00	=	16740.00
Concrete in two piers below M. L. W	105.0 坪 @ 350.00	=	36750.00
Concrete in " " above " " "	51.0 坪 @ 260.00	=	13260.00
Concrete piling for piers.	2100.0 尺 @ 12.00	=	25200.00
Excavation for piers.	100.0 坪 @ 270.00	=	27000.00
Concrete in abutments	58.0 坪 @ 220.00	=	12760.00
concrete piling for abutments	1536.0 尺 @ 10.00	=	15360.00
Excavation + backfilling for abutments	114.0 坪 @ 15.00	=	1710.00
		¥	331038.00

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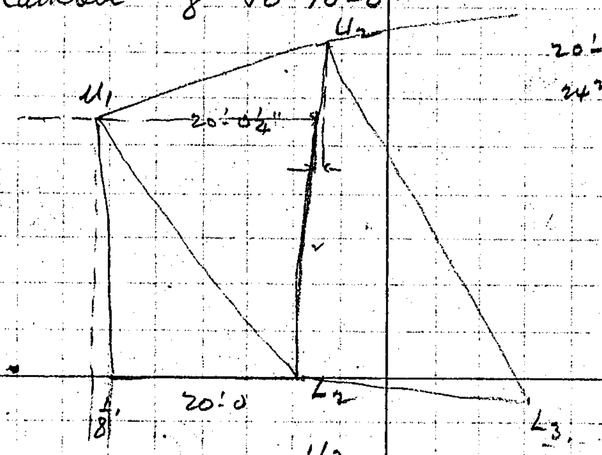
17
26

43

Tategahana Bridge.

Length of bridge members.

Camber 1/8" to 10'-0"



$20'\text{-}0\frac{1}{8}''^2 = 400.4168$

$24^2 = 576.0000$

976.4168 — $31'\text{-}2\frac{31}{32}$ $U_1\text{-}L_2$

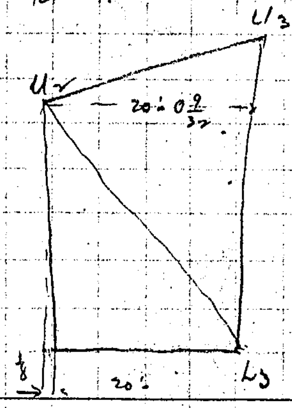
$.325'' \times \frac{5.25}{20} = .0328$ — $\frac{1}{32}''$

$20'\,0\frac{1}{4}$

$20'\,0\frac{9}{32}''^2 = 400.9380$

$5'\text{-}3''^2 = 27.5625$

428.5005 — $20'\text{-}8\frac{23}{32}$ $U_1\text{-}U_2$



$20'\text{-}0\frac{1}{8}''^2 = 400.4168$

$29'\,3'' = 855.5625$

1255.9793 — $35'\text{-}5\frac{9}{32}$ $U_2\text{-}L_3$

$20'\text{-}0\frac{9}{32}''^2 = 400.9380$

$3'\,9'' = 14.0625$

415.0005 — $20'\text{-}4\frac{15}{32}$ $U_2\text{-}U_3$

33

20'-0 1/4 0 1/8

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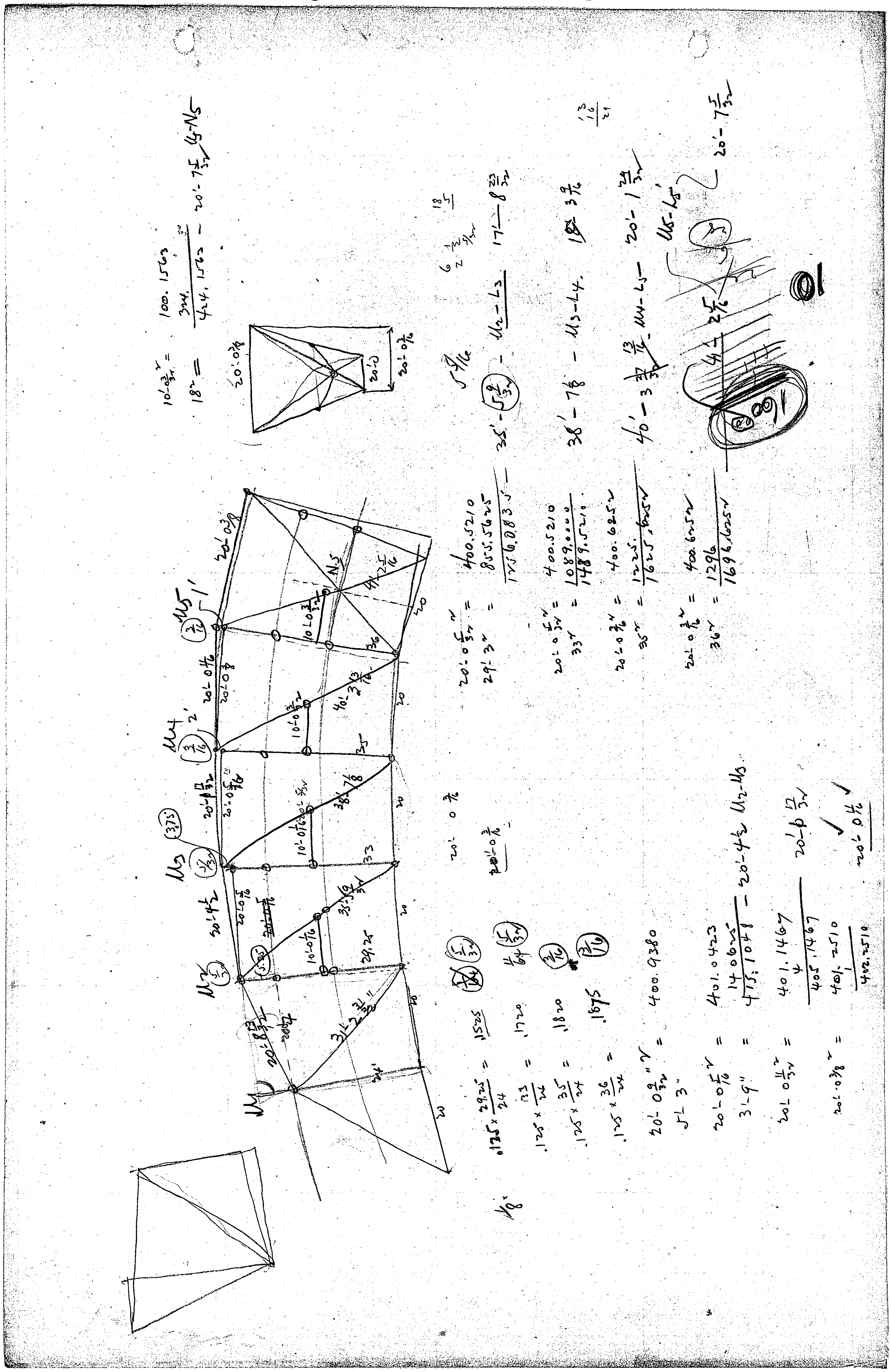

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Tategahana Bridge.

Metal on Bridge.

2 lines of Handrails 2@30# = 60# × 221.75 = 13300

Stringers 5@50 = 250 × 221.75 = 55500

Intermediate Floor beams 10@ 2607 = 26070

End Floor beams 2@ 2125 = 4250

Lower Laterals = 10800

Upper Laterals + Portal Bracings + Sways = 37140

2 Trusses 860 × 220 = 189000

Metal in floor 10@220 = 2200 150

338260

call this 340.000# ÷ 220 = 1550

Floor 2360

3910

Live Load 2020

5930# × 220 = 1310.000

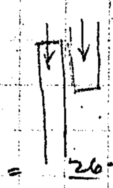
load each bearing = 1310.000 ÷ 4 = 327000#

Length of Roller 4" 4×600 = 2400 length of roller = 327000 ÷ 2400 = 136.00"

5@27" = 135" make 27" net

Try 5" pin. bearing = 24000 × 5 = 120.000

327000 ÷ 120.000 = 2.72" for one rib 1.36" .75 1.61 1/2 + 7/8



Try 6" pin. bearing = 24000 × 6 = 144000 moment = 1.61 × 163.500 = 263,000"#

327000 ÷ 144000 = 2.27" 1.13" each rib.

1/2" + 5/8" .75

moment = 1.375 × 163.500 = 225.000 inch lbs. ok

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Estimate of cost

Structural steel erected + painted	456 tons	@ 340.00		= 155,040.00
concrete in floor slab	38.2 坪	@ 240.00	220	15280.00
Reinforcing steel in floor slab + abutments	19.0 tons	@ 230.00	210	5370.00
wood block pavement	372.0 坪	@ 55.00	45	20460.00
concrete in two piers below M.W.L.	105 坪	@ 350		36750.00
concrete in " " above " " "	51 坪	250	200	12750.00
concrete piling	2100 lin ft	@ 10.00		21000.00
Excavation	100 坪	100.00		10000.00
concrete in abutments	58 坪	250	200	14500.00
concrete piling	1536 lin ft	10.00	8.00	15360.00
Excavation + back filling	114 坪	12.00	10.00	1368.00
				307878.00
Incidentals 12%				36945.36
				344823.36

180

24

4

120
200
24
7
1.7

88
92
1800
10

1800
4
7200

増田淳氏関係資料
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Preliminary Estimate of 村山橋 for Naganoken and Nagano Elec. Ry Co. 1.

This Estimate will serve for the selection of echonomic span length or suitable spans to cross the main stream of river channel. From the figures made out we have to get opinions of 県土木課長, 内務省土木出張所長, 内務省土木試第一技術課長, 鉄道省の電気鉄道局 青木氏 and will decide the spans to be used. and then will make final estimate of cost of the bridge.

This bridge will cross 千曲川 150 ken down stream of the present 村山橋 on the main highway between Nagano and Suzaka. The total length of bridge between end bearings is about 2670' in which 1200' is assumed as river spans and the rest the approach spans. We won't care about the approach spans at present because the approach will be built separate highway and electric tracks from the foundation, and we will estimate later, however, the economical span is 30' or 40' standards of railway bridge and likewise for highway. Either of these or others will be decided by the Engineers of Home Affairs' Dept. considering the water way during flood, we will, of course, furnish data for this decision.

Location of bridge is agreed by the officials concerned in this case and practically settled in the opinion.

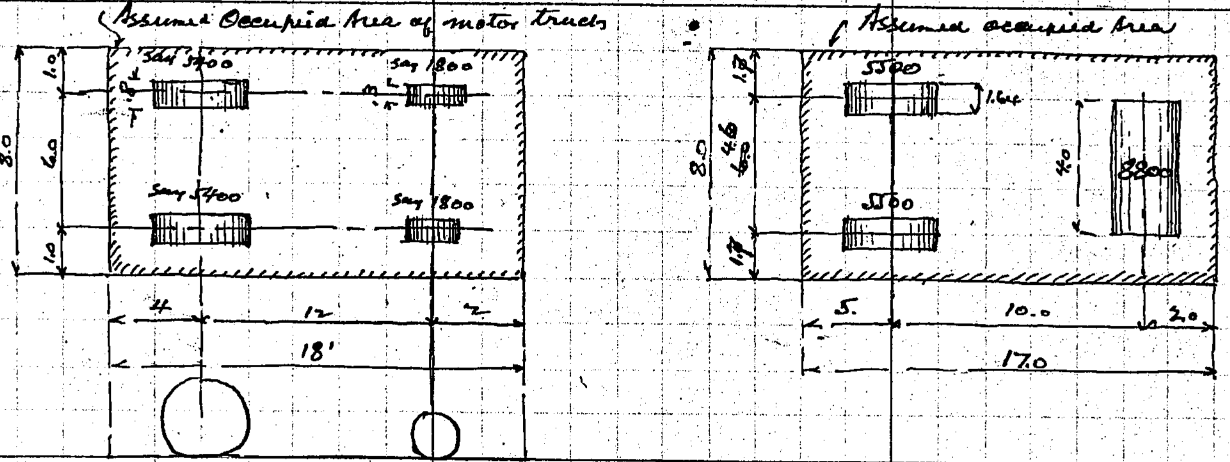
Requirements of bridge on electric railway tracks on down stream side and 18' roadway on up stream side by side with electric railway tracks.

Loading of Electric railway tracks E25 cooper's Loading. Impact

Highway loading

Uniform load $q^{kg/m^2} = \frac{100.000}{170+l}$ but not over $500^{kg/m^2}$ where l = span length in meter.

motor trucks loading 6400 kg (14118) Road Roller 9 tons



Impact allowance no impact for uniform live load and Road roller concentration
1/3 impact for motor trucks loading 35% for truss

Assumed arrangement of motor trucks 2 trucks coupled to-gether with impact preceded by or followed by the uniform load; 2 trucks side by side, the rest of space of roadway occupied by uniform load

Assumed arrangement of road roller 1 road roller on bridge without impact; unoccupied space to be filled by uniform load.

Assumed working strength

Fibre stress in concrete		600 #/□"	for positive moment
" " "		700 #/□"	for negative moment
Direct compression		500 #/□"	
Shear (reinforced)		120 #/□"	
Shear (plain)		57 #/□"	
Punching shear		120 #/□"	
Bond stress		85 #/□"	
Reinforcing bars		16000 #/□"	Tension
Structural Steel			
Tension		16000 #/□"	
Compression		16000 − 70 l/r	for railway side
Fibre stress		16000 #/□"	
Shear	shop rivet	12000 #/□"	
	field rivet	10.000	
Bearing	shop rivet	24000	
	field rivet	20.000	

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Assumed working strength (continued)

Expansion rollers per lin inch d = diameter of roller 600d

Compression on plate girder flange $16000(1-0.012\,\frac{l}{b})$

where l = unsupported length of flange

b = width of flange.

Compression member gross section (Highway only) $16000(1-0.0055\,\frac{l}{r})$

where l = unsupported length of member

r = least radius of gyration.

with max of 14000 #/□" and l/r limited to 100 for main member and ~~120~~ 150 assumed for wind bracings and secondary members.

Miscellaneous data.

Ratio between moduli of elasticities of Steel and Concrete = 15

Expansion Coefficient of Concrete = 0.0000055 per 1° of Fahrenheit

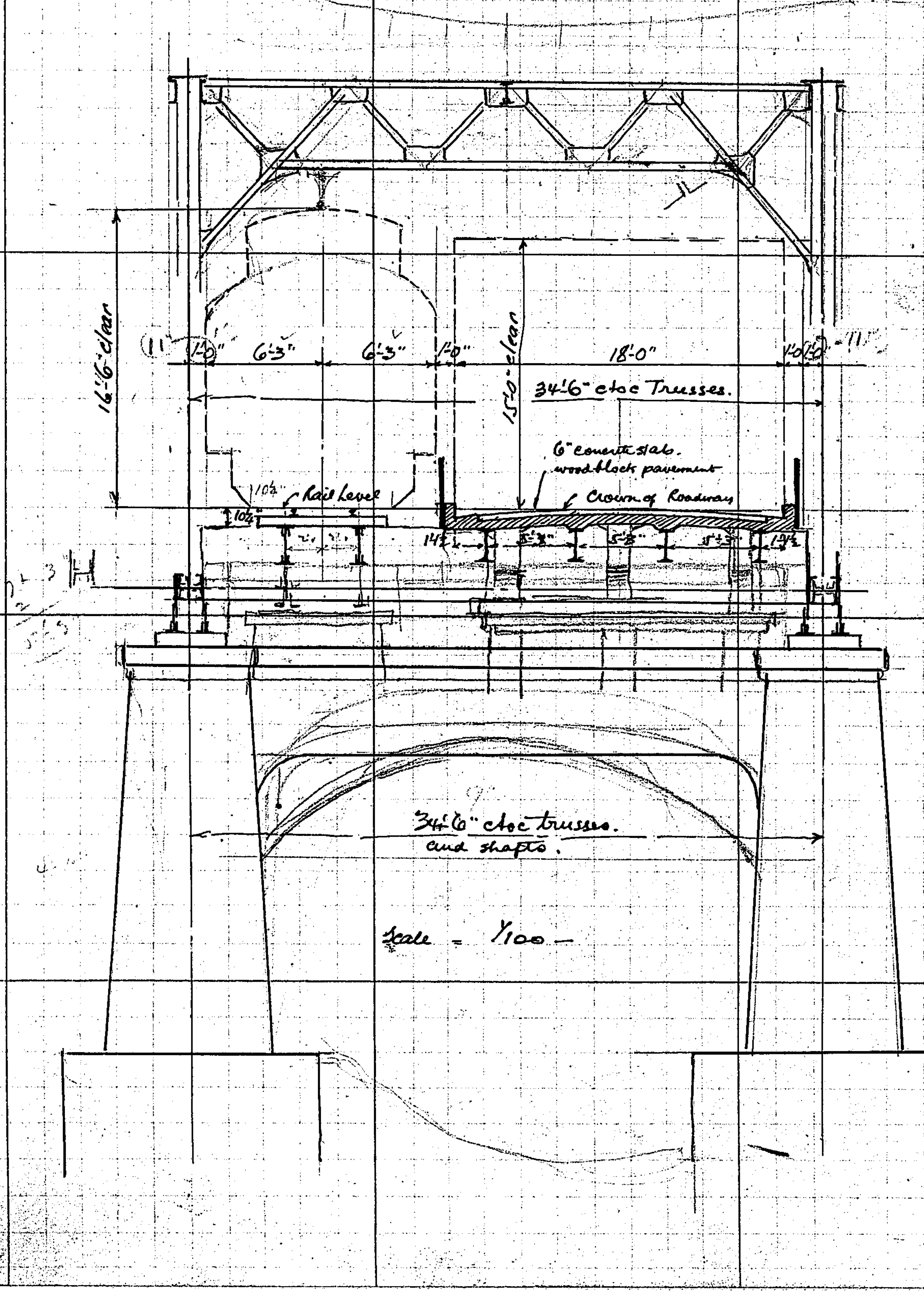
" " Steel = 0.0000060 " " "

Temperature change for concrete structure = ±40° Standard Temperature assumed

" " steel structure = ±60° as 62°F.

Cross section of structure (River span only).

We tried 4 truss design, 3 truss design and 2 truss design and we found the latter will give the minimum cost for the superstructure and also for the substructure. We decided to use 2 truss design and the general cross section assumed as shown on sketch below.



Headway required for railway = 16'-6"

Headway required for highway = 15'-0"

Railway track will be separated from highway which is 18'-0" clear between curb lines and used handrails for both sides of roadway to insure the safety of traffic and passengers.

Span length of bridge.

No1. layout

8 spans @ 146.5' = 1172'-0"

1 span @ 28'0 = 28'-0"

1200'-0"

Space between ℄ bearings on piers assumed 2.5 making span length between end bearings = 144. ±

7 panels @ 20.6'

Layout No 2.

7 spans @ 167.5 = 1172'-0"

1 span @ 28'0 28'-0"

1200'-0"

C to C of end bearings say 165'-0" ±

Panel length 8 @ 20.6' about.

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[I] Let us estimate No 1 layout span length 144'-0" about.

Highway Floor

Concrete slab – span length 5'-3" –

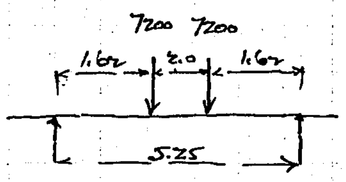
Dead load

3½" wood block pavement	17.5#	Dead load moment = $\frac{1}{10} \times 100 \times 5.25^2$ = 276'#
½" cushion	6.0	Dead load shear = $100 \times \frac{5.25}{2}$ = 262#
concrete slab assumed	75.0	
	98.5	

Call this 100.0# per sq ft of Roadway including filler etc.

Live Load motor truck loading

Rear wheel conc.	5400	Front wheel conc.	1800
⅓ impact	1800	⅓ impact	600
	7200#		2400#



moment = 7200 × 1.62 = 11700'#

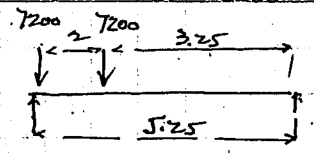
For continuity of slab say 11700 × 0.8 = 9350'#

Distribution of wheel concentration assumed as follows

Effective width = 0.6 × 5.25 + 1.0 = 4.15

moment per ft strip = $\frac{9350}{4.15}$ = 2250'#

Live load end shear



shear 7200 × $\frac{3.25}{5.25}$ = 4460

7200

11660# ÷ 4.15 = 2700#

Summary for Moments and Shears

	moment	Shear
Dead load	276	262
Live load	2250	2700
	2526	2962

Effective Depth required = $\sqrt{\frac{2526}{95}}$ = 4.1"

Covering for protection 1.25

5.35"

use 6" slab with effective depth = 4.75"

Steel Area = $\frac{2526 \times 12}{\frac{7}{8} \cdot 4.75 \times 16000}$ = .457 □" per ft

Unit shear = $\frac{2962}{\frac{7}{8} \cdot 4.75 \times 12}$ = 59.5#/□" ok

For bond stress $\frac{2962}{4.75}$ = 625

625 ÷ 85 = 7.37

Add more bars at support to carry bond stress

Use ½"φ bars 5" centers. 2/3 bent up at support add 1 bar at top every 3rd.

Steel Stringer span length 20.6' spacing of stringers 5'-3"

neglect continuity of slab and take reaction as simple beam.

Dead load floor 100 × 5.25 = 525#

Beam assumed 50

575# per ft

moment = $\frac{1}{8} \times 575 \times 20.6^2$ = 30400'#

If count negative reaction due to overhanging arm the load will be 575 - 50 = 525# per ft

moment = $\frac{1}{8} \times 525 \times 20.6^2$ = 27800'#

Dead Load shear = 525# × $\frac{20.6}{2}$ = 5400# (Intermediate)

Dead Load shear = 617 × $\frac{20.6}{2}$ = 6350# (End stringer)

End stringer

Handrail assumed	40# × 1.75	=	70.0
Curb	75# × 1.50	=	112.5
Gutter	125 × 1.12 = 140 × 0.56	=	78.5
	255		261.0

Extra reaction = $\frac{261.0}{5.25}$ = say 50# per ft

Total reaction	$\frac{100}{2} \times 5.25$ =	262#
direct		255#
from reaction		50
		567#
beam assumed		50
		617#

D.L. m = $\frac{1}{8} \times 617 \times 20.6^2$ = 32800'#

Live Load motor truck loading

Rear wheel 7200# including impact

Front wheel 2400# " "

Spacings of wheels as shown on diagram

Uniform live load say 100#/□' on unoccupied space of motor trucks or road roller

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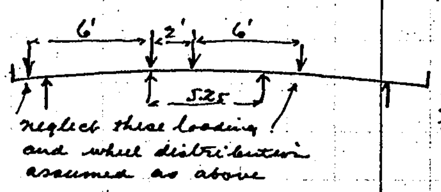
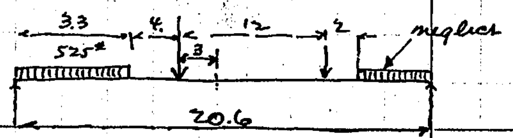
Intermediate Stringer



Rear wheel $7200 \times \frac{3.25}{5.25} =$ 4450#
7200
11650# including impact
Front wheel $\frac{1}{3} \times 11650 =$ 3880#
15530

Live Load moment due to motor trucks loading -
$15530 \times \frac{7.3^2}{20.6} = 40200$#



Moment due to uniform load $139 \times 13.3 = 1850$

Reaction due to U.L. $= \frac{525 \times 3.3^2}{2 \times 20.6} = 139$#
moment motor trucks 40200
42050 1#

End Stringer
load on beam rear wheel 7200#
Front wheel 2400
9600#

Uniform load
$650 \times \frac{325}{525} =$ say 400#

Live Load moment due to motor trucks loading -
$9600 \times \frac{7.3^2}{20.6} = 24800$
M due to unif. load $1850 \times \frac{400}{525}$ say $=$ 1400
26200 1#

Summary for moments

	Intermediate Stringer	End Stringer
Dead Load	27800	32800
Live Load	42050	26200
	69850 1#	59000

Section modulus required $\frac{69850 \times 12}{16000} = 52.3$ $\frac{59000 \times 12}{16000} = 44.2$

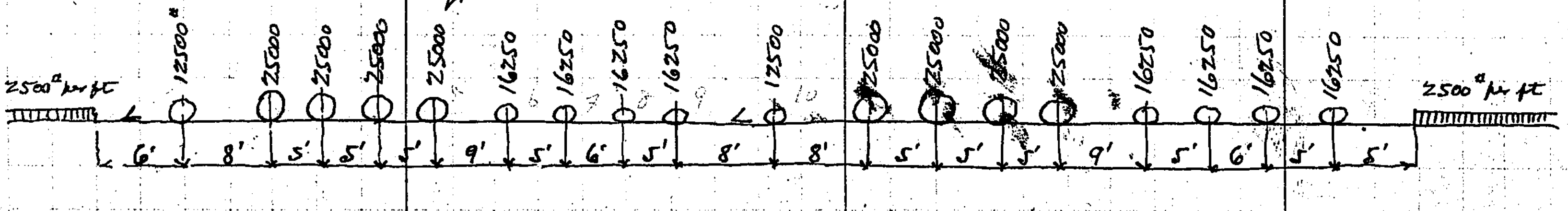
Use 15" × 42# I 15" × 42# I.

connection angles 4 L $3\frac{1}{2}$ $3\frac{1}{2}$ $\frac{3}{8}$ @ 8.5 × 1.0 = 34#
1-15" I × 42# × 20.6 = 865
899
add rivet heads. 5
904 ÷ 20.6 = 44# per ft or say 45# per lin ft.
For stringers. 4 @ 45# = 180# per lin ft of span.

Electric Ry Stringers span length 20.6'
Dead load of Decks.
main rails 60# per yard 2 @ 60 = 120#
Guard rails 2 @ 60 = 120#
Ties 6" 8" 7.0 = 2.34 240# ÷ 3 = 80#
wt $2.34 \times \frac{50}{2} =$ say 60#
Timber guard rails - 2 - .5 × .5 wt 25#
Foot walks 2' at track 2 × 0.1 × 50# 10#
Spikes, splices Hook bolts etc say 10#
195# ÷ 2 = say 100# per ft.
Dead Load beam and lateral bracings assumed per stringer 100
200

Dead Load moment $= \frac{1}{8} \times 200 \times 20.6^2 = 10600$ 1#
Dead Load Shear $= \frac{200}{2} \times 20.6 = 2060$# per stringer

Live Load E25 loading



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Preliminary Estimate of Cost of #4#3 for Naganoken and Nagano Electric Ry Co

Impact coefficient = $\frac{300}{300+L} \times \frac{30}{60}$ where L = loaded length of bridge -

Strieger span length 20.6' spacing 4'-0"
Equivalent uniform live Load = 5130 #
Impact 93.7% — 1500. 4810
33.
9940 ÷ 2 = ~~4970~~
6830. — 3420 -

LL m = $\frac{1}{8} \times 4970 \times 20.6^2$ = 264,000 '#
DL m — 10600
Total 274600 '#
Sm = $\frac{274600 \times 12}{16000}$ = 206.0

Try 27" web. web 27" × 3/8 = 10.1 □"
1/8 web = 1.260 □" d = 2.25 - .15 = 2.10
Stress = 131000 # SR = $\frac{131000}{16000}$ = 8.16
1/8 web 1.26
6.90 □"

Max End shear = 6200 × $\frac{20.6}{2}$ = 64000 # per span
~~DL shear~~ Impact 60000
124000 ÷ 2 = 62000 #
DL Shear 2060
64060 #

use 2 L's 5 × 3½ × ½" = 8.00 gr 7.000 □" net
weight of one stringer

	1 web. 27" × 3/8	@ 34.43	× 20.5	=	705
	4 L's 5 × 3½ × ½	@ 13.6 #	× 20.5	=	1115 #
stiffs	6 L's 3½ × 3½ × 5/16	@ 7.2	× 2.25	=	97 #
End stiff	4 L's 4 × 4 × 7/16	@ 11.3	× 2.10	=	95
fills	4 Pl's 4 × ½	@ 6.80	× 1.67	=	46
	variation + rivet heads 5%				100

2158 # ÷ 20.6 = 105 # per lin ft.
105
25

Top Bracing and Cross Frame -
235 # per lin ft.
call this 240 # per lin ft.

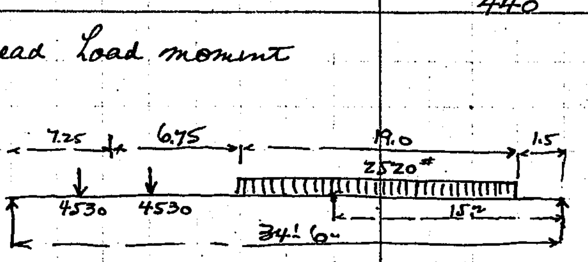
Floor Beam span length 34'-6" spacing 20.6'
Dead Load -
Highway loading -

Roadway	0.5 × 18	=	9.0	
curb	2 - 0.5 × 1.33	=	1.33	
Extra for gutter	2 - .33 × 1.5	=	1.00	
	11.33 @ 150	=	1700 #	
Pavement	235 # × 1.5	=	352	
2 Handrails	2 @ 40	=	80	
4 stringers @ 45		=	180 #	

2312 # ÷ 19 = 122.0 #/0' assumed -
122 × 20.6 = 2520.#

Railway Loading
Deck 200 #
stringers + frames 240
440 # per lin. ft of track - panel load 440 × 20.6 = 9060 #

Dead Load moment



22400
5200
27600

34560
5200
39760

Reaction ! -

2520 × 19.0 = 47900 × 11.0 = 527000
9060 × 27.25 = 247000
56960 — 774000 ÷ 34.5 = 22400 #
22400
34560 ÷ 2520 = 13.7'

max moment = 34560 × 15.2 = 525000 '#
Less. 2520 × $\frac{13.7^2}{2}$ = - 236000
289000 '#

Dead load beam 300 × $\frac{1}{8}$ × 34.5² = 44700
333700 '#

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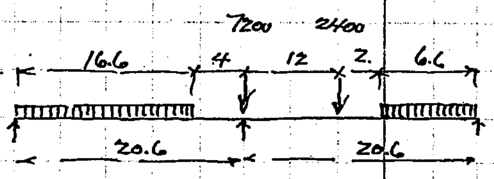
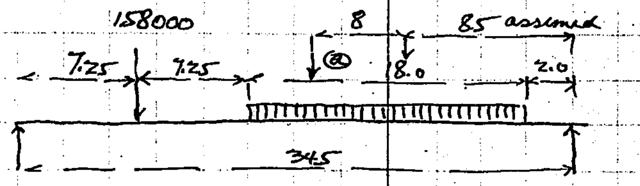
Floor Beam Continued
Live Load
Highway loading



motor trucks conc. 7400 × 8.6/20.6 = 1000
rear wheel 7200
8200 × 2 = 16400#

Unif. load 100 × 6.6²/(2 × 20.6) = 106.
100 × 16.6²/(2 × 20.6) = 670
776# per lin ft.

Railway loading Max Floor Beam Conc 84000
Impact 88% say 74000
158000 per track



Moment due to concentrations
Reaction
16400 × 8.5 = 139400
16400 × 16.5 = 270500
158000 × 27.25 = 4310000
190800
4719900 ÷ 34.5 = 137000

137000
4460
141460

53800
9540
63340

Unif. load 776 × 18 = 14000
14000 × 11/34.5 = 4460
9540 ÷ 776 = 12.3
2
14.3

Moment at (a)
53800 × 16.5 = 888000
16400 × 8 = −131000
757000#
77200
Total moment 834200#

moment 9540 × 14.3 = 136000
776 × 12.3²/2 = 58800
77200#

Summary for moments and shears

	Moment	Shear (Ry)	Shear (Hy)
Dead Load	333700	27600	40000
Live Load	834200	141460	say 65000
	1167900#	169060	105000

48½" b to b of Ls g web = 2.25 0" Effective depth say 3.8
Stress = 1167900 ÷ 3.8 = 307000# SR = 19.20
2.25
16.95 □" net

Use 2 Ls 6 × 6 × 5/8 = 14.22 11.72 □" net
1 Pl. 12½ × 3/8 ½ = 6.25 5.25
16.97 □" net

weight of one Intermediate Floor Beam

web	1 Pl	48 × 3/8	@ 61.2	× 33.25 =	2040
Flanges	4 Ls	6 × 6 × 5/8	@ 24.2	× 33.25 =	3220
Cover Pls	2 Pls	12½ × ½	@ 21.25	× 20.0 =	850
stiffs	14 Ls	5 × 3½ × 9/16	@ 8.7	× 4.0 =	487
End stiffs	4 Ls	4 × 4 × ½	@ 12.8	× 4.0 =	204
fills	4 Pls	4 × 5/8	@ 8.5	× 3.0 =	102
Shelfs	12 Ls	3½ × 3½ × 9/16	@ 7.2	× .67 =	58
Splice	2 Pls.	12 × 3/8	@ 15.30	× 3.0 =	92
"	4 Pls.	5 × 3/8	@ 6.38	× 3.0 =	77

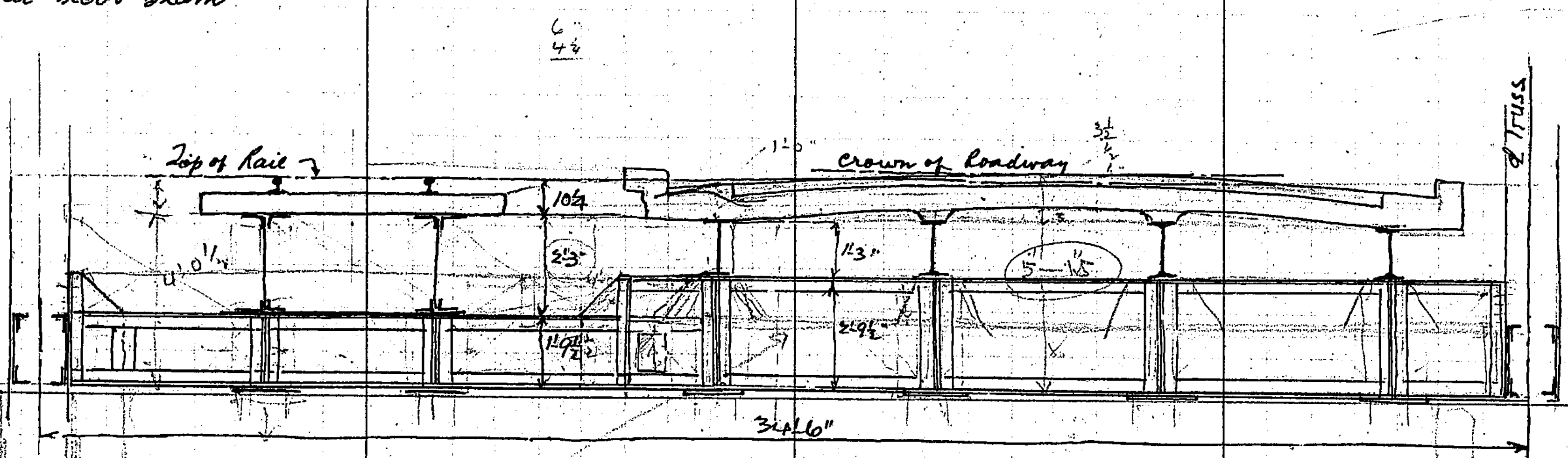
7130
Rivet heads + variation say 270
7400# ÷ 33.25 = 223#

7400/20.6 = 360# per ft

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Preliminary Estimate of Cost of 7@40 ft for Naganoken and Nagano Electric Ry Co

End Floor Beam



Scale 1/50

End Floor beam as shown in sketch will serve as strut only

web	1 Pl.	33 × 5/16	@	35.06	×	20.0	= 700
flanges	2 Ls	3½ · 3½ × 5/16	@	7.2	×	19.0	= 274
	2 Ls	do	@	7.2	×	20.0	= 144
	1 Pl.	21 × 5/16	@	22.31	×	12.5	= 280
	2 Ls	3½ · 3½ × 5/16	@	7.2	×	14.5	= 209
	2 Ls	3½ · 3½ × 5/16	@	7.2	×	13.7	= 197
	1 Pl.	15 × 5/16	@	15.94	×	2.5	= 40
End stiffs	4 Ls	3½ · 3½ × 3/8	@	8.5	×	2.5	= 85
fills	4 Pls.	3½ × 5/16	@	3.19	×	2.2	= 28
Int stiffs	8 Ls	3 · 3 × 3/8	@	7.2	×	1.75	= 100
fills + splice	8 Pls	6 × 5/16	@	6.38	×	1.2	= 61
Int stiffs	16 Ls	3 · 3 × 5/16	@	6.1	×	2.7	= 264
fills	8 Pls	6 × 5/16	@	6.38	×	2.2	= 112
Bearing Pl	1 Pl.	7½ × 3/8	@	9.56	×	6.0	= 57
Base Pl	6 Pls	15 × ½	@	25.50	×	1.25	= 192

2743

5% variation — 187

2930#

Summary for one span

Highway stringers	180 × 146.5	=	26400
Railway stringers	240 × 146.5	=	35200
	420# ×		61.600#
Intermediate Floor Beam	6 @ ~~7400~~	=	~~44400~~
End Floor Beam	2 @ 2930	=	5860

50360 ÷ 146.5 = 343# per ft of bridge

420 " " " "

763# " " " "

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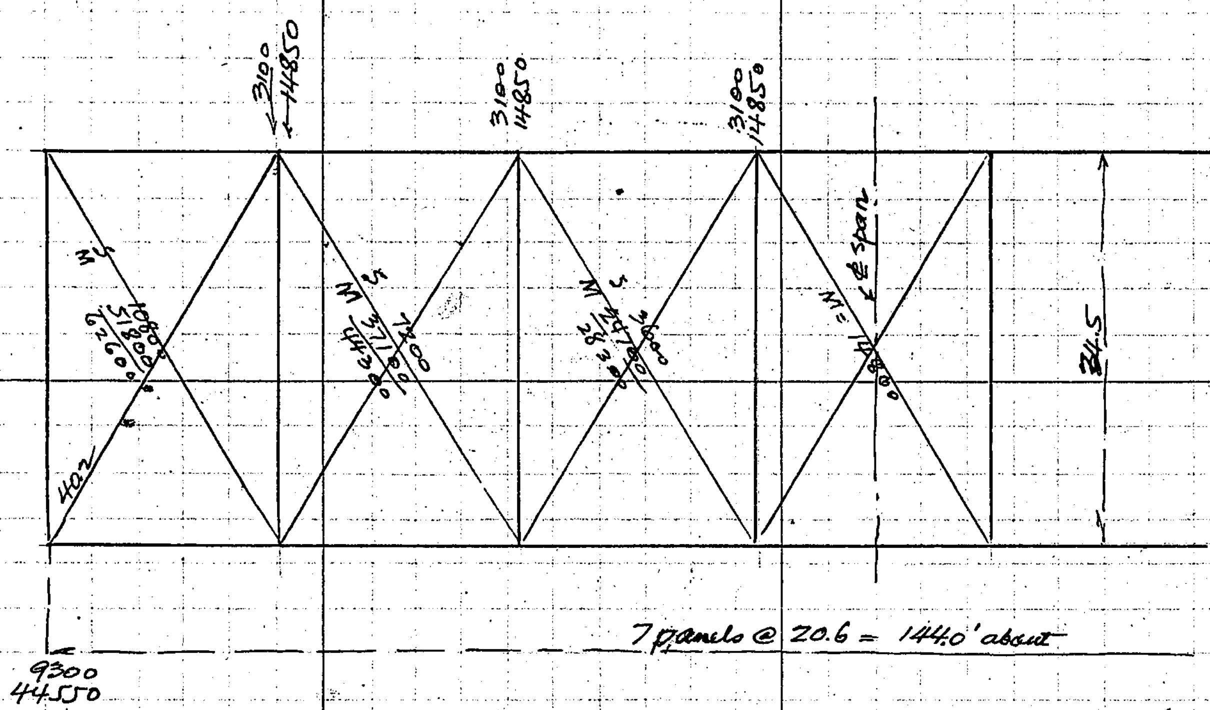
Lower Lateral Bracing -



Static wind load 30# per square ft of Exposed Surface. assumed for highway truss 5 × 30 = 150# per ft
Moving wind load 12 × 30 = 360# per lin ft assumed for highway truss.
Same moving load assumed for railway truss but neglect static load for railway truss.

static load 150# × 20.6 = say 3100# Reaction 3100 × 3 = 9300
moving load 720# × 20.6 = 14850# 14850 × 3 = 44550

Lower Lateral assumed ~~to~~ resist tension only

stress	Unit stress	Sq. net	Section used	gross	net
62600	16000	3.92	2∠s 5 × 3½ × 5/16 ✓	5.12	4.50 ☐"
44300	"	2.71	do	"	"
28300	"	1.77	do	"	"
14800	"	.93	do	.	"

Lateral to be tied under the highway and railway stringers at quarter point to prevent sag

Approximate weight of Lower Laterals -

4∠s 5 × 3½ × 5/16 @ 8.7 × 39.0 = 1360#
center connection 75
4 gussets @ 50 200
misc connection say 100

1735 × 7 = 12150# ÷ 146.5 = 83# per lin ft of bridge.
Or call this 85# or 12500# per span

Upper Lateral Bracings.

Use same section for diagonals 2∠s 5 × 3½ × 5/16 -
Longitudinal strut at center of bridge - 2∠s 6 × 4 × 3/8 in contact -
or use 4∠s 3 × 3 × 5/16 with 6" × 5/16" web plate

weight of strut 4∠s 3 × 3 × 5/16 @ 6.1 × 20.0 = 490#
1 Pl 6 × 5/16 @ 6.38 × 20.0 = 128
details say 22
640#

Diagonal Bracing - 5 @ 1750 = 8750
Long. " strut 5 @ 640 = 3200
11950#

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Sway Bracing - see pp 2 for sketch

Top section	2 Ls 4 × 3 × 5/16	@ 7.2	× 33.5	= 483	
Bottom section	2 Ls 4 × 3 × 5/16	@ 7.2	× 24.5	= 353	
diagonals Braces	4 Ls 4 × 3 × 5/16	@ 7.2	× 11.5	= 331	
diag.	6 Ls 3 × 3 × 5/16	@ 6.1	× 5.0	= 183	
	11 conn @ 30			330	
	Details say			100	
				1780#	

~~4~~ Intermediate sway Bracings @ 1780 = 7120
2 End portals @ 2500 = 5000
12120 - 12120
Diagonals & struts 11950
24070# ÷ 1465 = 165# per ft

Main trusses:
Dead Load reaction of Floor Beam see pp 5

	Ry		Highway	
Ry reaction	27600	Highway reaction	39760	
Lower Lateral	3350 ?		3350 ?	
Top Lateral say	4000 ?		4000 ?	
truss assumed	13000		13000	
	47950 ÷ 20.6 = 2330		60110# ÷ 20.6 = 2920#	

Live Load
Highway loading -
Uniform load $= \dfrac{20480}{170 + \frac{144}{328}}$ = say 96#/□'

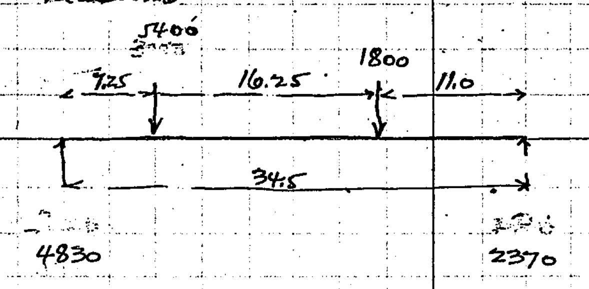
Motor trucks loading without impact ~~14110#~~ 2 @ 5400 = 10800
2 @ 1800 = 3600
14400
Uniformly distributed load $\dfrac{14400}{\cancel{8} \; 18}$ = 800# per ft
or 100# per sq ft

Assume live load for Highway truss 100# per sq ft throughout. or 1800# per lin ft.
panel concentration 1800 × 20.6 = 37100#

Railway loading -
Uniform live load for 144' span E25 3220# per lin ft.
Impact 67.5% 2180
5400# per lin ft.

Impact $I = \dfrac{300}{L+300} \times \dfrac{V}{60}$

Reaction



Void
1800 × 11.0 = 19800
3240 × 27.25 = 88250
5040 108050 ÷ 34.5 = 3130
3130
1910

1800 × 11.0 = 19800
5400 × 27.25 = 147000
7200 166800 ÷ 34.5 = 4830
4830
2370

Summary for loading -

	Ry truss	Highway truss
Dead Load	2330	2920
Live Load	4830	2370
	7160#	5290

From curve weight of one truss
Ry side 600# per lin ft
Hy side 500# per lin ft } for both trusses 1100# per ft.

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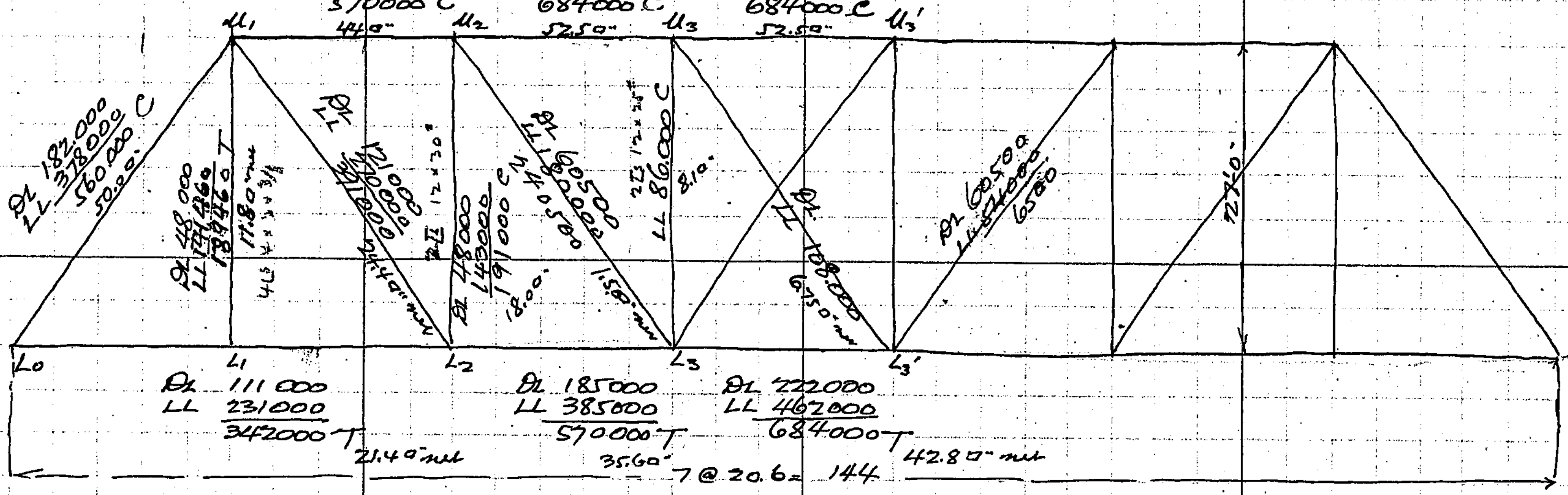
Stresses in Trusses

Railway truss



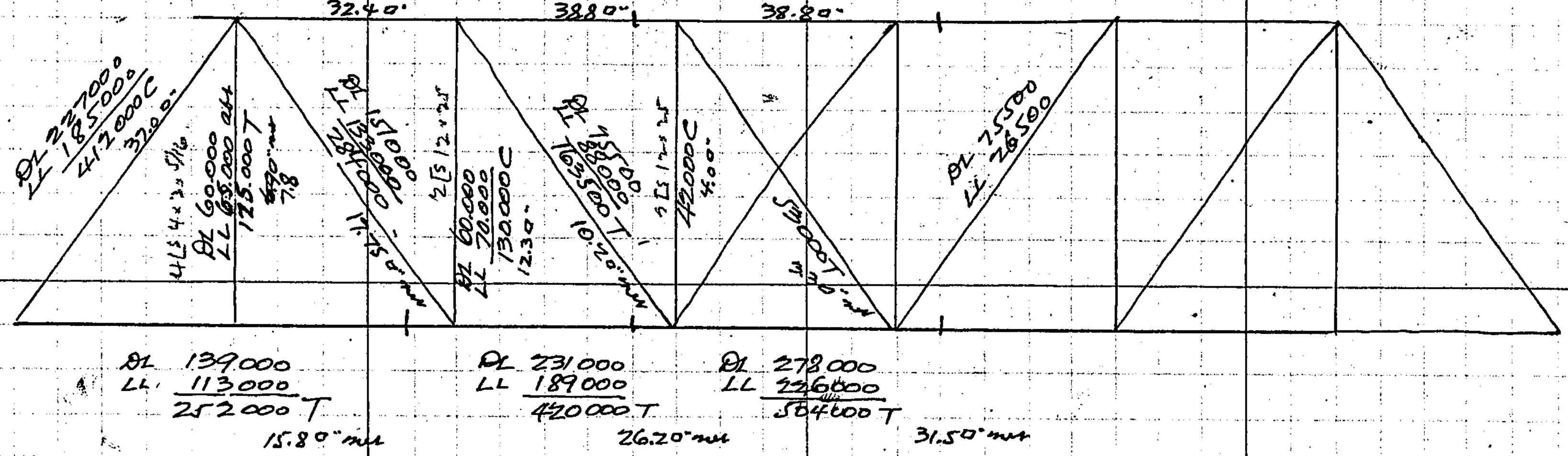
Railway truss Dead Load Panel concentration 48000#

Live Load Panel conc. 4830 × 20.6 = 100.000#

$\sec\theta = \frac{34}{27} = 1.26$

$\tan\theta = \frac{20.6}{27} = 0.77$

Highway truss



Highway truss Dead Load Panel Conc (60.000)#

Live Load Panel Conc 2370 @ 20.6 = 49000#

Top chord 16000 − 70 l/r where l = 20.6 × 12 = 247
= 13100 #/□" about r = 6.0

End Post 16000 − 70 l/r = 112000 #/□"

vert. Post 16000 − 70 l/r = 106000 #/□" r = 4.2 l = 27 × 12

Sectioning of top chord

railway truss

	2 Web pls.	18 × 3/8	38.80□"	use this section for highway truss
	1 cov pl.	21 × 1/2		
	2 Ls	3½ × 3½ × 3/8		
	2 Ls	5 × 3½ × 5/8		
add.	2 pls.	11 × 3/8 = 8.25	47.05	

	2 web pls	18 × 1/2	43.30	use this section for Ry. truss
	1 cov.	21 × 1/2		
	2 Ls	3½ × 3½ × 3/8		
	2 Ls	5 × 3½ × 5/8		
add	2 pls.	11 × 3/8	8.25	
			51.55	

r = 7.17

16000 − 70 l/r = 13600 #/□"

3R 50.4

増田淳氏関係資料
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Railway truss.

Bottom chord

2 Pls. 18 × 1/2	= 18.00	14.00 net		
4 Ls 3 1/2 × 3 1/2 × 3/8	= 9.92	6.92		
	27.92		20.92	
add. 2 Pls. 18 × 1/2	= 18.00		14.00	
	45.92		34.92 –	34.92
add. 2 Pls 11 × 3/8	4.12			3.38
add 2 Pls. 16 × 3/8 1/2	8.00			6.00
	58.04			44.30

hangers	4 Ls 5 × 3 × 3/8	= 11.44	8.44 net					
	14 × 3/8	= 5.25	4.50					
		16.69	12.94					
U_1-L_2	4 Ls 6 × 6 × 1/2	23.00	19.00	U_2-L_3	4 Ls. 5 × 3 1/2 × 7/16		14.12	10.62
	14 × 1/2	7.00	6.00		14 × 3/8		5.25	4.50
		30.00	25.00				19.37	15.12
U_3-L_3	4 Ls 4 × 3 × 3/8	= 10.00						
U_2-L_2	2 [s 12" × 30 #							
U_3-L_3	2 [s 12 × 25 #							

Estimate of weight of truss

member				
$U_2-U_3-U_3'$	51.55	@ 3.4	× 31.0	= 5430
U_1-U_2	43.30	"	× 20.6	= 3030
L_0-U_1	51.55		× 34.0	= 5960
L_0-L_2	27.92		× 41.2	= 3900
L_2-L_3	45.92		× 20.6	= 3200
L_3-L_3'	58.04		× 10.3	= 2030
$U_1 L_1$	16.69		× 26.0	= 1475
U_2-L_2		60 #	× 26.0	= 1560
U_3-L_3		50 #	× 26.0	= 1300
U_1-L_2	30.00	@ 3.4	× 32.0	= 3270
U_2-L_3	19.37	@ 3.4	× 32.0	= 2110
U_3-L_3'	10.00	@ 3.4	× 32.0	= 1090

34355 × 2 = 68710

Details say 36% say 25000

93710 ÷ 146.5 = 640 # per lin. ft.

Truss height assumed as 27'-0"

If the height assume as 28'-0" the chord stress will be reduced 3.7%

Reducing weight 3% from weight of truss 640

20

620 # per lin ft.

増田淳氏関係資料
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Highway truss

Member	Section					
U_1-U_3	38.8	@3.4	×	51.5	=	6780
L_0-U_1	38.8	"	×	34.0	=	4490
L_0-L_2	23.4	"	×	41.2	=	3280
L_2-L_3	33.0	"	×	20.6	=	2310
L_3-L_3	43.0	"	×	10.3	=	1510
U_1-L_1	9.6	"	×	26.0	=	850
U_2-L_2		50#	×	26.0	=	1560
U_3-L_3		50#	×	26.0	=	1300
U_1-L_2	22.0	@3.4	×	32.0	=	2400
U_2-L_3	14.0	"	×	32.0	=	1530
U_3-L_3'	~~10.0~~ 8.4	"	×	32.0	=	920

26930 × 2 = 53860

Details say 38% 20400

74260 ÷ 146.5 = 506#

making the truss height to 28'-0" reduce the section

approximate reduction 30% — 506 / 15

Highway truss 490# per lin ft

Railway truss say 620

1110# per lin ft.

506 — / 640 = / 1146

Weight of shoes.

	Railway truss		Highway truss	
DL.	48000 × 4 =	192000	60.000 × 4 =	240000
LL	100000 × 4 =	400,000	49000 × 4 =	196000
		592000		436000
	call this	600000#	call this	450000#
weight of shoe	3500# Exp.			3000#
	3060# fixed			2500#
	6560			5500#
	call this 7000#		call this	6000

Total weight of shoes 13000# ÷ 146.5 = 89# per lin ft of span

Summary of Structural Steel in span

Highway stringers		26400#
Railway stringers		35200
Floor Beams complete		50300
Lower Laterals		12150
Upper laterals		11950
Sways + Portals		12120
Highway truss	27'	74260
Railway truss	27'	93710
Highway shoes		7000
Railway shoes		6000
		329090

329090 call this 330000# 147.0 tons per span

8 spans @ 147.0 = 1176 tons ✓

増田淳氏関係資料
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Estimate of superstructure of No2 Layout

7 spans @ 167.5' = 1172'-0" This truss span will be divided into 8 panels @ 20.6 each making
1 span @ 28'-0" = ~~28'-0~~ Span length 165'-0" about between end bearings.
1200'-0"

Summary of Steel in one span

Highway Stringers	180# × 167.5	=	30200	
Railway Stringers	240# × 167.5	=	40200	
				70400#
Intermediate Floor Beams	7@7400	=	51800	
End Floor Beams	2@2930	=	5860	
				57660
				128060 ÷ 167.5 = 765# per ft

Lower Lateral Bracings Use 2Ls 5×3½×5/16" throughout
weight per panel 1735# × 8 = 13880#

Upper Lateral Bracings	Diagonal Bracings	6@1750	=	10500
	Longitudinal Struts	6@ 640	=	3840
				14340

Sway Bracings and Portals

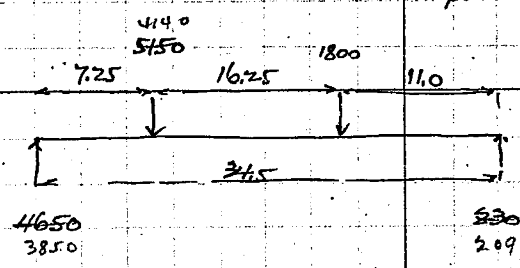
Intermediate Sways	5@2000	=	10000
End Portals	2@3000	=	6000
			16000#

Main truss
Dead Load

	Railway		Highway
DL Floor supps	27600	30000	39760
Lower Laterals say	900		900
Upper Lateral	1200		1200
Sway &c	1000		1000
Truss assumed	14400		12400
	45100		55260 ÷ 206
say	~~2190~~# per ft		2680# per ft
	1450		
	640		

Live Load assumed 100#/□' for highway or 1800# per ft of span

Railway loading M.L.L for 165' span E25 3130#
Impact 64.5% 32.25% 1010
~~2020~~
4140
~~5150~~#



1800 × 11.0 = 19800
4140
~~5150~~ × 27.25 = ~~140500~~ 113000
5940
~~6950~~
4650
2300

5940
3850
2090

~~160300~~ 132800 ÷ 34.5 = ~~4650~~ 3850

Summary for truss loading

	Ry truss	Hy truss	
Dead Load	2190	2680	
Live Load	~~4650~~ 3850	~~2300~~ 2090	
	~~6840~~ 6040#	~~4980~~ 4770#	per ft

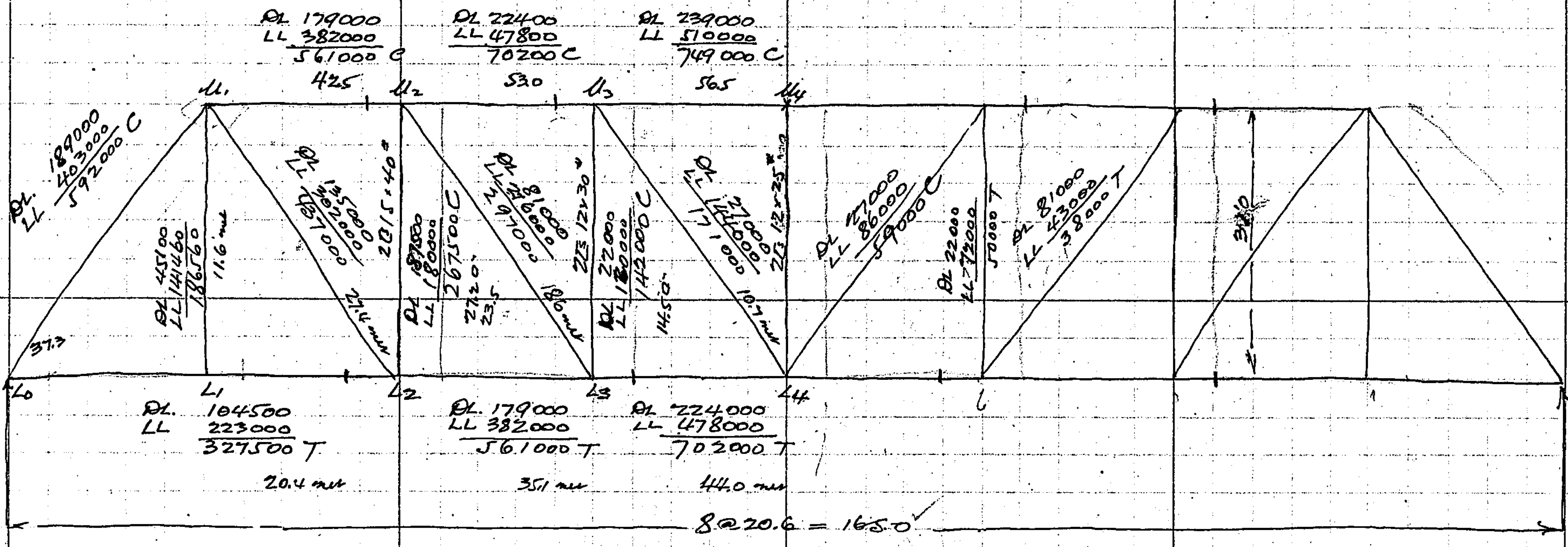
From curves the approximate weight of truss
Ry truss 690# per ft
Hy truss ~~540~~# " "
1230# per ft

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Preliminary Estimate of Cost of 材山村 for Nagano Ken and Nagano Electric Ry Co

Stresses in trusses

Railway truss



Railway truss Dead Load Conc 2190 × 20.6 = say 45000# sec θ = 37.3 / 31.0 = 1.20

Live Load Conc ~~4650~~ × 20.6 = 96000# tan θ = 20.6 / 31.0 = .664

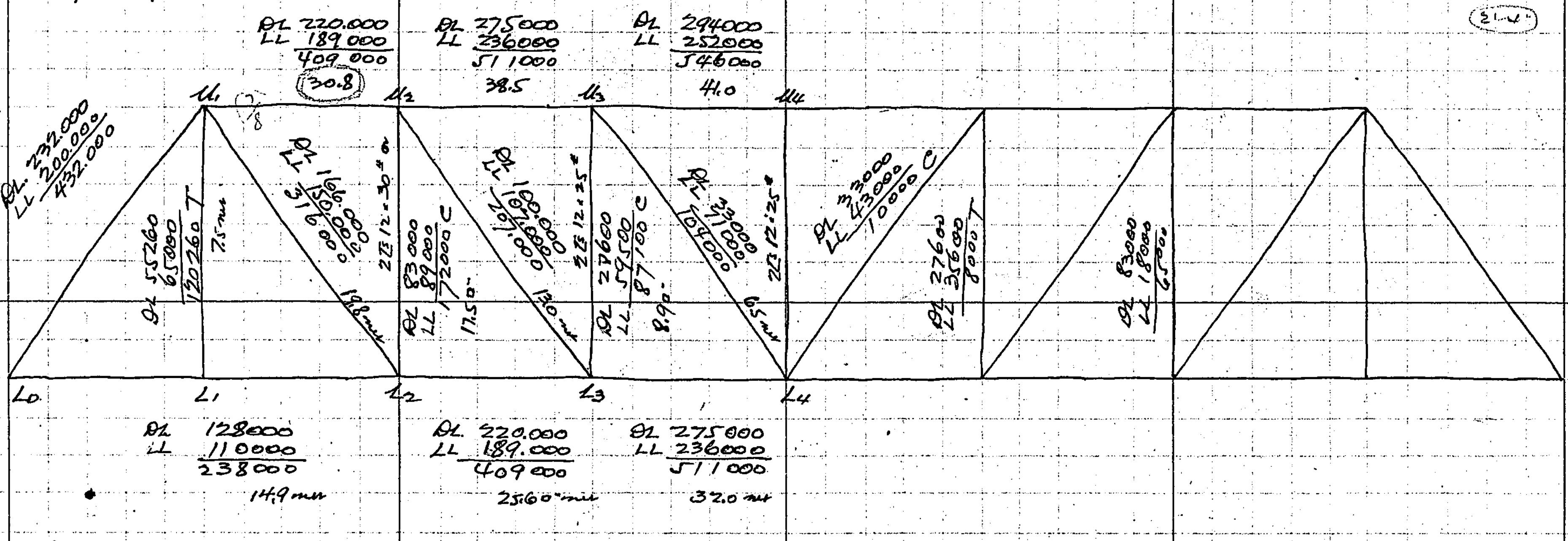
3850
6040

m = 1/8 × 6040 × 165.0² = 2055000

2055000 ÷ 31 = 66250#

5060 gross

Highway truss



Highway truss Dead Load Conc 2680 × 20.6 = 55200# sec θ = 1.20

Live Load Conc ~~2300~~ × 20.6 = 47500# tan θ = .664

2090
4770

Top chords 16000 − 70 l/r where l = 20.6 × 12 = 247

= 13100 #/□" r = say 6.0

End Posts 16000 − 70 l/r l = 37.3 × 12 = 447 r = say 7.0

= 11500 #/□"

Vert. Posts 16000 − 70 l/r l = 31.0 × 12 = 372 r = 4.2

= 9800 #/□"

16000 − 70 l/r = 11350 r = 5.6

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Preliminary Estimate ~~of~~ Cost of [illegible] for Naganoken and Nagano Electric Ry Co.

Railway truss

member	section		Length		
U_1-U_2	43.3	@ 3.4	20.6	=	3040
U_2-U_3	51.55	"	20.6	=	3620
U_3-U_4	56.5		20.6	=	3950
L_0-U_1	56.5	"	37.3	=	7170
L_0-L_2	26.0	"	41.2	=	3640
L_2-L_3	44.0		20.6	=	3080
L_3-L_4	55.0		20.6	=	3850
U_1-L_1	16.6		30.0	=	1700
U_2-L_2		80#	30.0	=	2400
U_3-L_3		60#	30.0	=	1800
U_4-L_4		25#	30.0	=	750
U_1-L_2	35.0		36.0	=	4300
U_2-L_3	24.0		36.0	=	2940
U_3-L_4	16.0		36.0	=	1960

44200 × 2 = 88400#

Details say 37% 32700

121100 ÷ 167.5 = 722#/ft

Highway truss

Member	Section		Length		
U_1-U_2	32.0	@ 3.4	20.6	=	2240
U_2-U_3	38.5		20.6	=	2700
U_3-U_4	41.0		20.6	=	2870
L_0-U_1	41.0	@	37.3	=	5200
L_0-L_2	20.0		41.2	=	2800
L_2-L_3	33.0		20.6	=	2310
L_3-L_4	42.0		20.6	=	2940
U_1-L_1	9.6	@ 3.4	30.0	=	980
U_2-L_2		66#	30.0	=	1980
U_3-L_3		50	30.0	=	1500
U_4-L_4		25	30.0	=	750
U_1-L_2	24.0	@ 3.4	36.0	=	2940
U_2-L_3	17.0		36.0	=	2080
U_3-L_4	14.0		36.0	=	1710

33000 × 2 = 66000

Details say 38% 25000

91000 ÷ 167.5 = 544# per ft

Railway truss 722
Highway truss 544
1266#

Summary of Structural Steel in span

Highway & Railway Stringers	70400
Floor Beams	57660
Lower Laterals	13880
Upper Laterals	14340
Sway & Portal	16000
Railway truss	121100
Highway truss	91000
Shoes ±	14000
	398380 or 178 tons

1352
7 spans @ 178 = 1246 tons
1176
70 tons 100

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

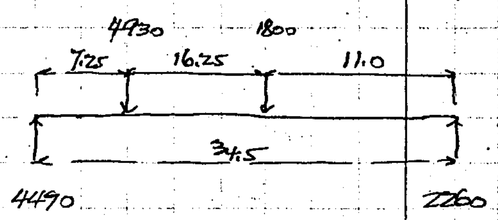
Estimate of superstructure for No 3 Layout

6 spans @ 195.5' = 1172'-0" This truss span will be divided into 9 panels @ 21.4' each
1 span @ 28' = 28'-0" making span length 192.6' between ℄ of end bearings
1200'-0"

Approximate metal in Floor system assumed 770# per lin. ft.
Lower Lateral bracings 1800 × 9 = 16200#
Upper Lateral Bracing 2400 × 7 = 16800#
Sway bracings and Portals
Intermediate sway bracing 6@2500 = 13000
2@3500 = 7000
20000#

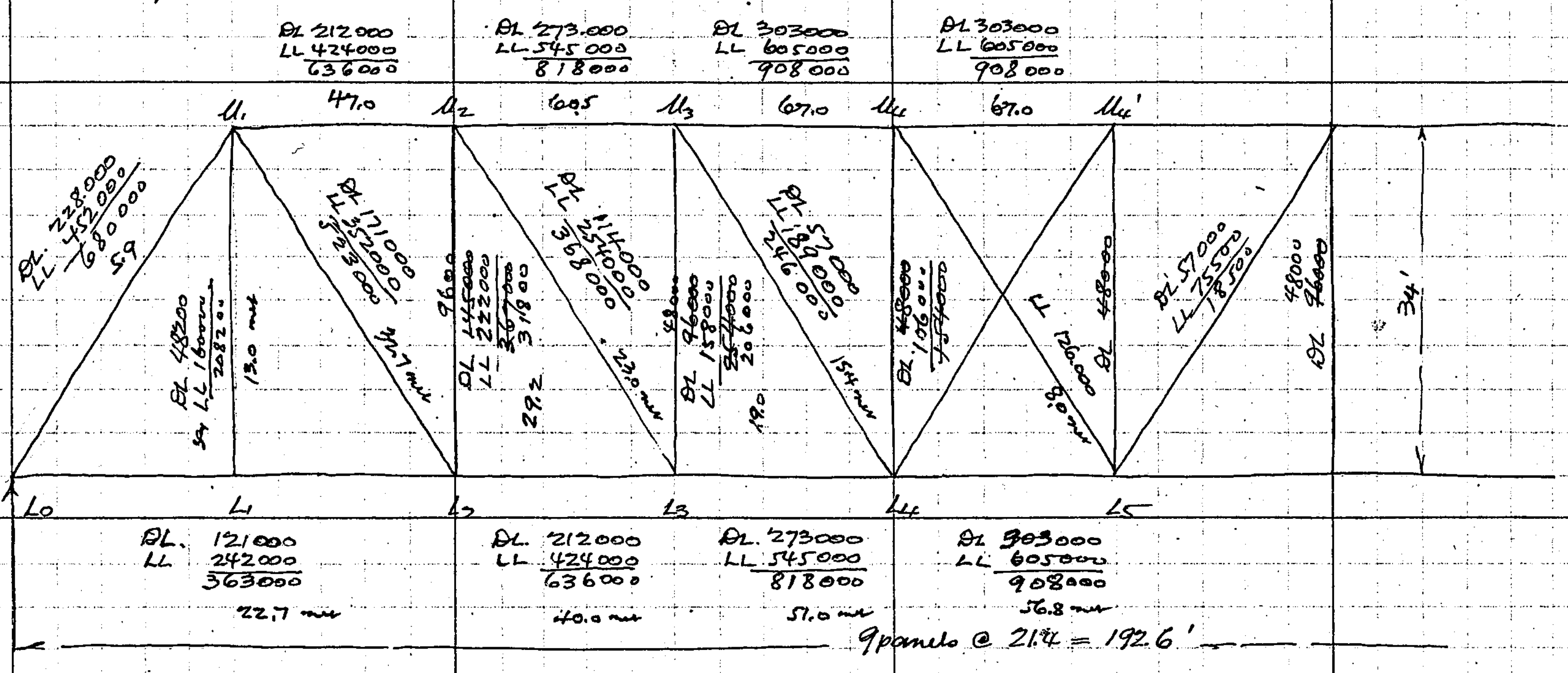
Main truss
Dead Load assumed Railway side 2250# per lin ft
Highway side 2750# " " "

Live Load
Highway loading assumed 100#/□' of roadway or 1800# per ft of span.
Railway loading M.L.L. 192.6' span Cooper's E25 3065#
Impact 61% 1865
4930# per track



1800 × 11.0 = 19800
4930 × 27.25 = 134500
154300 ÷ 34.5 = 4470
6730
4470
2260

Stresses in Trusses
Railway truss

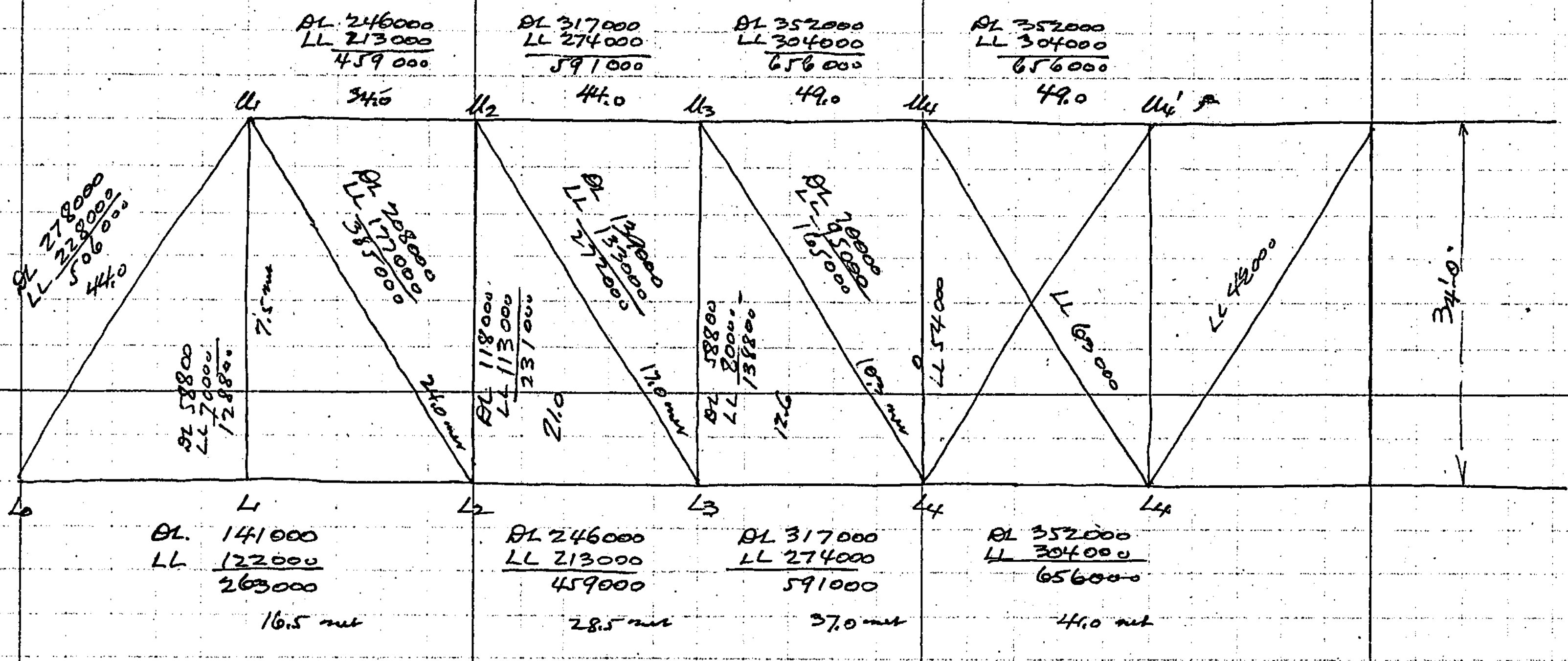


Railway truss Dead Load Panel Conc. 2250 × 21.4 = 48200# sec θ = 40.3/34 = 1.18
Live Load Panel Conc. 4470 × 21.4 = 96000# tan θ = 21.4/34 = 0.63

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Preliminary Estimate of Cost of 村山橋 for NaganoKen and Nagano Electric Ry Co_

Highway truss



Hy truss Dead Load Conc. 2750 × 21.4 = 58800 Sec θ = 1.18

Live Load Conc. 2260 × 21.4 = 48300 tan θ = 0.63

Top chord section. 16000 − 70 l/r = ~~12600~~ 13640 #/" l = 21.4 × 12 = 257" r = 7.6

End Posts 16000 − 70 l/r = 11550 #/" l = 40.2 × 12 = 483 r = 7.6

vert Post 16000 − 70 l/r = 10900 #/" l = 34 × 12 = 408 r = 5.6

vert Post 16000 − 70 l/r = 9200 #/" l = 34 × 12 = 408 r = 4.2

Weight of Railway truss

member	section				
U_1-U_2	47.0	@ 3.4	× 21.4	=	3420
U_2-U_3	60.5		× 21.4	=	4400
$U_3-℄$	67.0		× 32.1	=	7300
L_0-U_1	67.0		× 40.2	=	9120
L_0-L_2	29.0		× 2 × 21.4	=	4220
L_2-L_3	50.0		× 21.4	=	3640
L_3-L_4	65.0		× 21.4	=	4730
$L_4-℄$	72.0		× 10.7	=	2620
U_1-L_1	20.0		× 32.5	=	2220
U_2-L_2		100#	× 32.5	=	3250
U_3-L_3		66#	× 32.5	=	2140
U_4-L_4		50#	× 32.5	=	1630
U_1-L_2	41.0		× 39.0	=	5430
U_2-L_3	30.0		× 39.0	=	3970
U_3-L_4	21.0		× 39.0	=	2780
U_4-L_4'	15.0		× 39.0	=	2000

62870 × 2 = ~~125~~740

details say 37% ~~46500~~

172240 ÷ 1955 = 1880# per ft

making truss depth 36' and using curved chord this weight will be reduced to ~~970~~ 850# about

Revised weight of Hy truss ≒ 166000#

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Weight of Highway truss

Member	Section					
U1-U2	34.0	@ 34	×	21.4	=	2470
U2-U3	44.0		×	21.4	=	3200
U3-℄	49.0		×	32.1	=	5350
L0-U1	49.0		×	40.2	=	6700
L0-L2	21.0		×	2×21.4	=	3060
L2-L3	36.0		×	21.4	=	2620
L3-L4	47.0		×	21.4	=	3420
L4-℄	52.0		×	10.7	=	1900
U1-L1	10.0		×	32.5	=	1100
U2-L2		80#	×	32.5	=	2600
U3-L3		66#	×	32.5	=	2140
U4-L4		50#	×	32.5	=	1630
U1-L2	30.0		×	39.0	=	3970
U2-L3	22.0		×	39.0	=	2910
U3-L4	15.0		×	39.0	=	2000
U4-L4'	10.0		×	39.0	=	1320

47390 × 2 = 94780

Details say 38% 36000

130780 ÷ 195.5 = 668#

From curve this wt ok.

Using 36' depth and curved chord

this will be reduced to 840#

850

~~1610~~#

1490

Revised wt of Highway truss 125000

Summary of Structural Steel in span

Metal in Floor system	770 × 195.5 =	150500
Lower Laterals		16200
Upper Laterals		16800
Sways &c		20000
Railway truss		166000
Highway truss		125000
Shoes		16000
		510500 or 228 tons

6 spans @ 228.0 = 1370 tons } 134 tons (415円)

7 spans @ 178.0 = 1246

8 spans @ 147.0 = 1176 } 70 tons (220円)

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(独立行政法人 土木研究所蔵)

Substructure for Layout No 1.

8 spans @ 146.5 = 1172'-0"

1 span @ 28'-0" on left bank 44'-6" span on right

Dead Load

Floor reaction see pp 5

Railway reaction 22400 ÷ 20.6 = 1090# per lin ft including stringers

Highway reaction 34560 ÷ 20.6 = 1725# " " " "

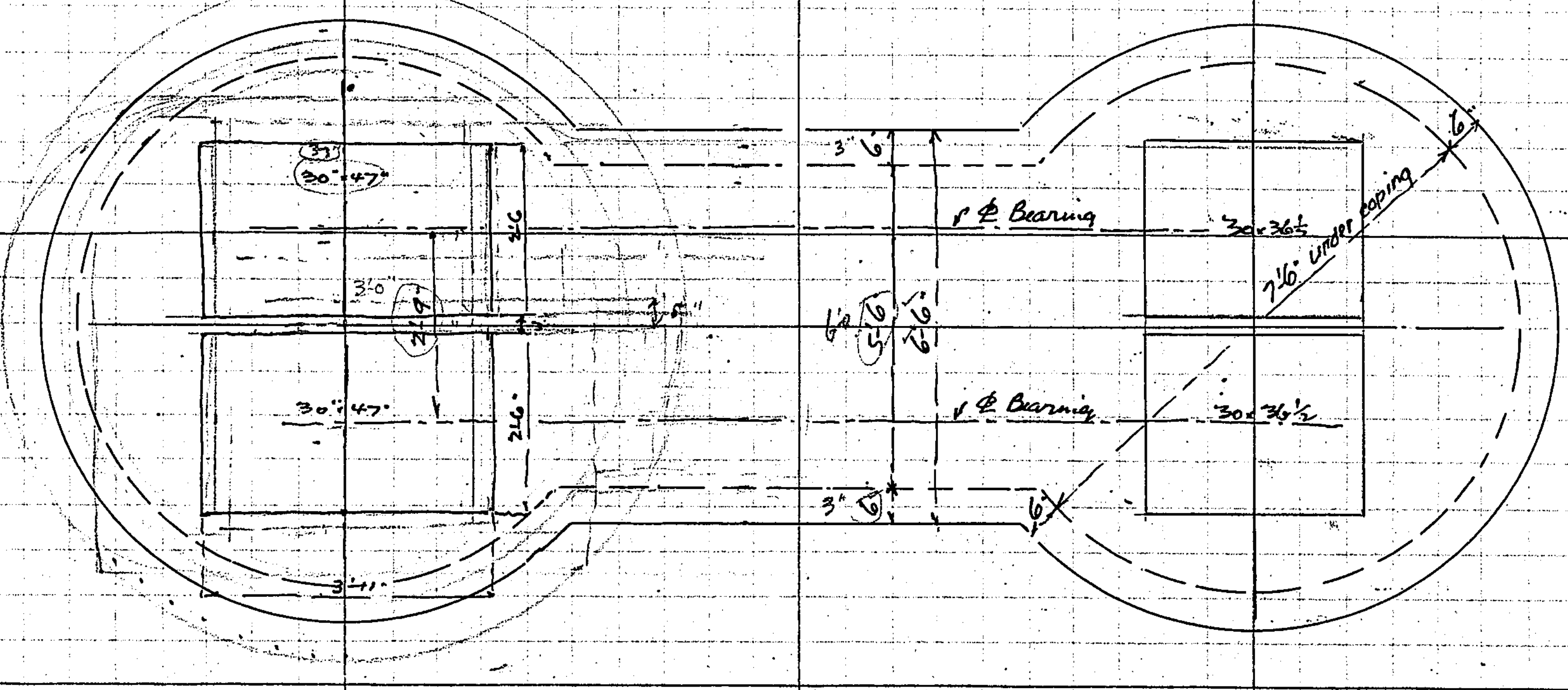
	Railway loading	Highway loading
Floor	1090	1725
Floor Beam	170	170
laterals + Sways.	125	125
truss	640	510
Shoes	48	41
Total D.L.	2073	2571#
Live Load	4830	2370
Tot	6903#	4941#

6040 × 168.5/2 = 507,000

Load on bearing 6903 × 146.5/2 = 512.000# 4941 × 146.5/2 = 360.000#

Bearing area 512.000/500 = 1024□" 360.000/500 = 720□"

Bearing area 512.000/400 = 1280□" 33" × 39" 30" × 43" 360.000/400 = 900□" 33" × 33"



Roller required for bearing, Railway truss

4" roller 600 d = 600 × 4 = 2400# per lin inch

512.000/2400 = 213" net

assume 7 - 4" dia roller with ½" space between rollers 6 spaces @ 4½ = 27"

overhang - 2 @ 3" 6

33"

Length of roller 213/7 = 30½" net

2" groove

10" Extra bearing

42½"

Size of bearing 33 × 42½ = 1400□"

Unit Bearing = 512.000/1400 = 366#/□" ok.

Try 3½" rollers bearing will be 30" × 47"

Highway truss

360000/2400 = 150 lin inches net

7 rollers 150/7 = 21.4"

2

10

33.4" make base 33½"

Size of bearing plate 33" × 33½ = 1105□"

Unit bearing = 360.000/1105 = 326#/□"

7 - 3½" rollers 360000/2100 = 171

171 ÷ 7 = 24½"

2

10

30" × 36½"

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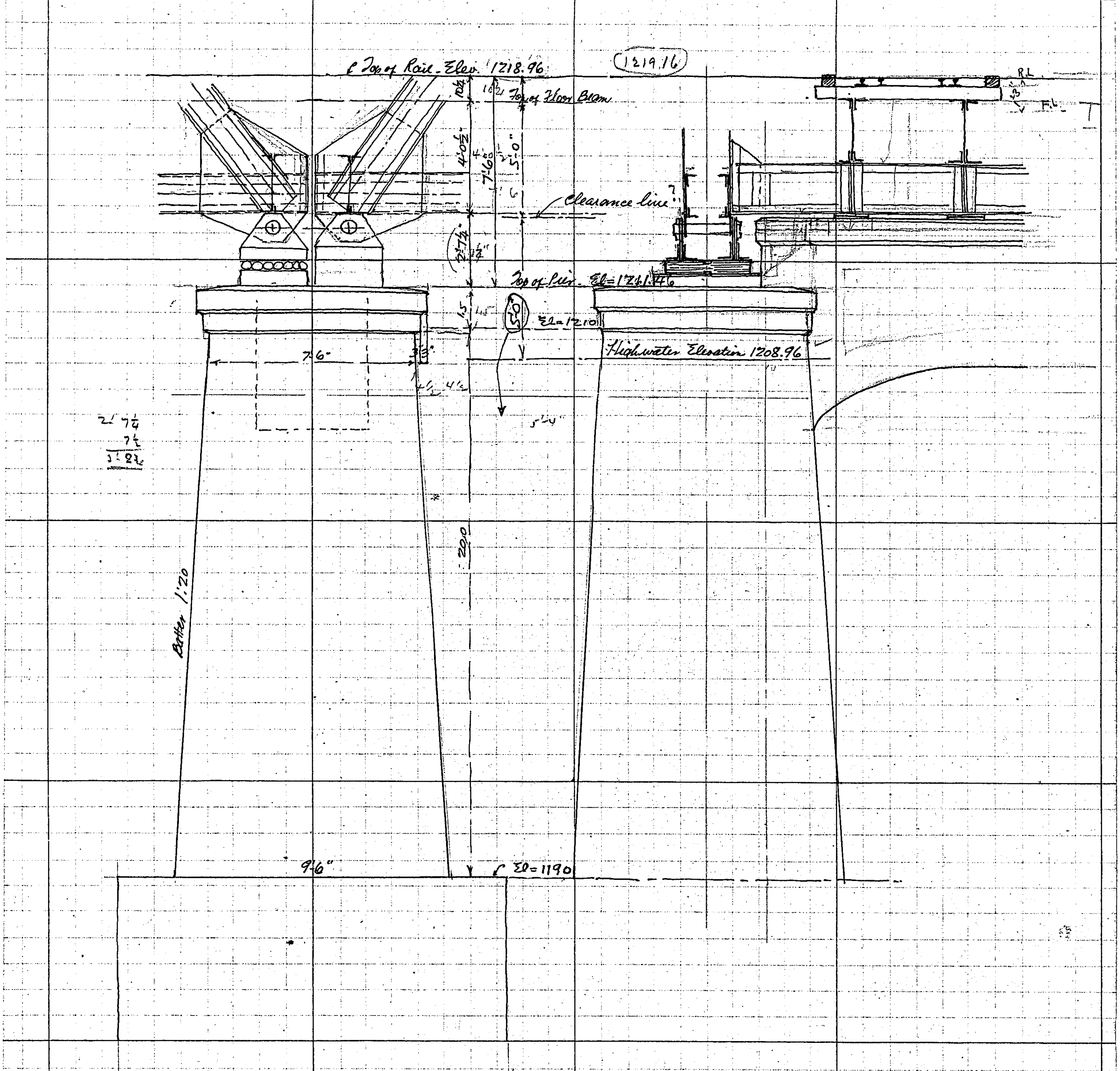

Concrete in Coping of shaft

8.5' dia	=	56.7	× .75	=	42.5	
8.0 dia	=	50.3	× .75	=	37.8	
					80.3	cubic ft

Shaft top area 44.17
bottom 70.88

115.05 ÷ 2 = 57.5 × 20 = 1150.0

1230.3 cubic ft or 5.7 坪

2@ 5.7 = 11.4 坪 370.000#

Strut between shafts coping - 4.5 × .75 = 3.38

4.0 × 5.0 = 20.

approximate 23.38 × 30.5 = 711. cu ft or 3.3 tsubo. 107.000#

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Well - height of well 20' - 25' - 30'
Shell 1.25' thick -
dia of well - 15' dia - 176.7
12.5 dia - 122.7 for 20' for 25 for 30'
54.0 1080 1350 1620
5.0 坪 6.25 坪 7.5 坪
filling - 122.7 2460 3070 3680
11.4 坪 14.20 坪 17.00 坪
Total volume 16.40 坪 20.45 坪 24.50 坪
weight 140# concrete assumed 495000# 618000# 740000#

weight of one shaft 20' 25 30
shaft 185000 185000 185000
beam 53500 53500 53500
~~shell~~ - 495000 618000 740000
733500# 856500# 978500#

20' well - superimposed load Ry superimposed load Hy
superimposed load ~~512000~~ 512000 15.0 | 15' well ~~360.000~~ 360.000 14.0 | 14' well 720000
Pier 733500 733500 669500
~~1759500~~ 1245500 ~~1453500~~ 1093500 1389500
Bottom Area ~~15 dia~~ 15.5 ~~176.7~~ 188.7 ~~176.7~~ 188.7
9320#/□' or 415 tons 7700#/□ or 347 tons. 8400# 375 tons/□'

25' well
superimposed load 1024000 15.0 | 15' well 720.000 14.0 | 14' well 720.000
Pier 856500 856.500 774500
1880500 1576500 1494500
Unit Bearing 10000#/□' 446 tons/□' 8350#/□' 373 tons/□' 9000#/□' 401 tons/□'

30' well -
superimposed load 1024000 15.5 | 15' well 720.000 14.5 | 14' well 720.000
Pier 978500 978500 882500
Total load - 2002500 1698500 1602500
Unit bearing - 10600#/□' or 4.73 tons 9000#/□' or 4.00 tons 9660#/□' 4.32 tons/□'

Let us count frictional resistance of well 300#/□'
Circumference 15' dia 47' @ 300 = 14100#
15
211500#

2002500 -
211500
1791000# ÷ 188.7 = 9500#/□' or 4.25 tons/□'

Try 14' dia well for highway truss -
dia of well - 14' dia 153.9
11.50 dia 103.9 for 20' well 25' well 30' well
shell - 50.0 1000 cuft 1250 cuft 1500 cuft
4.62 坪 5.78 坪 6.93 坪
filling 103.9 2075 2600 3110
9.65 坪 12.0 坪 14.4 坪
Total volume 14.27 坪 17.78 坪 21.33
weight 140# per cubic ft 431000# 536000# 644000#
area of bearing 14.5 = 165 □'
Shaft 185000 185000 185000
53500 53500 53500
669500 774500 882500

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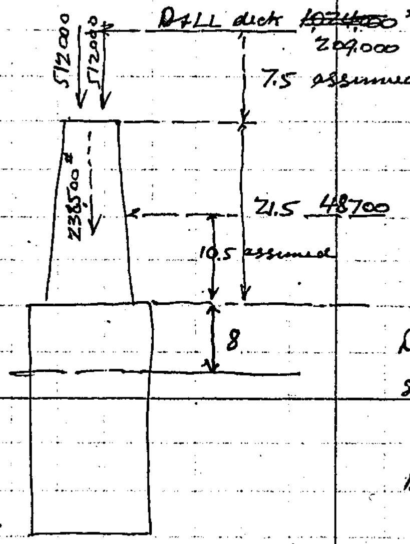
Resistance to Earthquake



Acceleration of falling body $g = 32.2$ ft per sec.²
assumed acceleration of Earthquake $= 2000\ ^{mm}/sec^2 = 6.55'/sec^2$
Strength at the top of base
Moment due to acc $M = \frac{W}{g} ay$
where W = weight
g = acceleration of gravity
a = " " Earth quake motion –
y = lever arm to extreme fibre.

Deck Horizontal thrust $= 1024.000 \times \frac{6.55}{32.2}^{.204} = 209.000^{\#}$
Shaft $238500 \times .204 = 48700^{\#}$

Bending Moment $= 209.000 \times 29.0 = 6060000$
$- \quad 48700 \times 10.5 = \underline{511.000}$
$6571,000\ ^{1\#}$

Moment of Inertia at bottom of shaft $I = 0.049 \times 114^4 = 8290000$ (inch)⁴
Fibre stress $= \frac{6571,000 \times 12 \times 57}{8290.000} = 541\ ^{\#}/_{\square''}$ comp or tension –

Direct load Deck 1024.000
shaft $\underline{238.500}$
$1262.500^{\#} \div 10.207 = 123\ ^{\#}/_{\square''}$

Fibre stress on tension side is too big for concrete; $664\ ^{\#}/_{\square''}$ comp or $418\ ^{\#}/_{\square''}$ tension –
let us reinforce the shaft ~~to resist~~ the tension.
Round shaft is assumed as a square beam of 84" × 105"
Eccentricity $6571.000^{\#} \div 1262,500 = 5.2'$ or 62.4" $\quad e/h = 62.4/105 = .\underline{595}$
Steel assumed 16 – 1"φ bars = 12.6 □" % steel = 12.6/84×105 = 0.143 %
add one more row $\underline{0.143}$
0.286 % –

$R = \frac{M}{bd^2} = \frac{6571.000 \times 12}{12 \times 7 \times 105^2} = 85.$ value of k = 0.~~085~~ 0.107

$f_c = \frac{6571000 \times 12}{0.107 \times 84 \times 105^2} = 785\ ^{\#}/_{\square''}$ ok.

steel stress = 785 × 19 about = 15000 #/□" – ok

steel in shaft	60 – 1"φ bars @ 2.67 × 15'-0"	=	2400
	30 – 1"φ bars @ 2.67 × 25'-0	=	2000
	60 – 1"φ @ 2.67 × 10'-0	=	1600
	misc. say		$\underline{500}$
			6500 # × 2 = 13000 #
	approximate steel in cross beam	say	$\underline{4000}$
			17000 # or call this 7.6 tons per pier.

weight of curb shoes – 70# per lin ft assumed
For 15' shoe 70 × 47 = 3300#
For 14' shoe 70 × 44 = $\underline{3100^{\#}}$
6400# or call this 3.0 tons per pier –

Concrete in shaft 1:2:4 147 yd^3
Concrete in shell of well 15' dia – 1.25' thick.

	20' well	*25' well	30' well
	5.0 yd^3	6.25 yd^3	7.5 yd^3
Concrete filling 1:2:4 8'	4.55 yd^3	4.55 yd^3	4.55 yd^3
Concrete filling 1:3:6	6.70 "	9.65 yd^3	12.50 ~~10.25~~ yd^3
~~14' dia well~~ Excavation from top of well.	16.40 "	20.45 yd^3	24.50 yd^3

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14' well - 1.25' thick

	20'	25'	30'
concrete in shell 1:2:4	4.62 坪	5.78 坪	6.93 坪
Filling 1:2:4 8'	3.85 坪	3.85 坪	3.85 坪
Filling 1:3:6	5.80 坪	8.20 坪	10.60 "
Excavation	14.27 "	17.78 坪	21.33 "

Bearing Pressure due to Earthquake motion - By side -
approximate only - moment taken at 8' below top of well
weight of well neglected - moment = 209.000 × 37.0 = 8.730.000 #
48700 × 18.5 = 900.000
9.630,000 #

moment of Inertia of well I = 0.049 × 180.0⁴ = 51,500,000 (inch)⁴

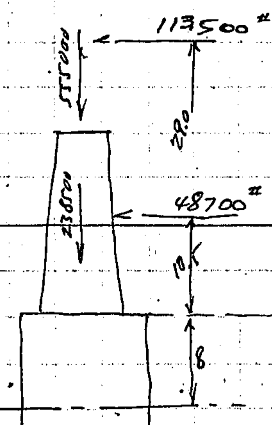
Fibre stress = $\frac{9.630.000 \times 12 \times 90}{51,500.000}$ = 202 #/□"

moment of Inertia of well 15.5 × 12 = 186" I = 0.049 × 186⁴ = 58750,000 (inch)⁴

Fibre stress = $\frac{9.630.000 \times 12 \times 93}{58750.000}$ = 183 #/□" 11.8 tons/□'

It may not possible full live load on bridge during earthquake
Try dead load + load on one span without impact

Dead load 2250# × 146.5 = 330.000
Live Load 3065 × $\frac{146.5}{2}$ = 225.000
555000 #



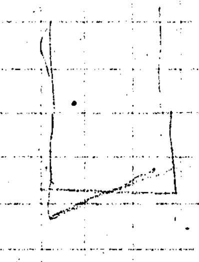
horizontal force 555000 × .2044 = 113500 #

moment = 113,500 × 37.0 = 4200.000
48700 × 18.5 = 900.000
5,100,000

Fibre stress = $\frac{5,100.000 \times 12 \times 93}{58750.000}$ = 97 #/□" or 6.25 tons

Total load superimposed load = 555.000
Shaft 238500
Base 740000
1533500 #

∴ unit pressure = $\frac{1533500}{188.7}$ = 8100 #/□' or 3.62
6.25
9.87 tons



Assuming resistance in circumference of well = 47 × 200 × 20' = 188 000 #
couple = 188000 × 15 × .637 = say 1800.000
Resulting moment = 5.100.000
1800.000
3,300,000

Fibre stress = $\frac{3300.000 \times 12 \times 93}{5,875.000}$ = 6300 #/□' or 2.81 tons

Unit pressure counting friction 1533500
less 188000
1345500 ÷ 188.7 = 7130 or 3.18
5.99 tons

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End Pier :–

Superimposed load truss span 2250 × 146.5/2 = 165000
girder span say 500# × 44.5/2 = 11000
176.000 #

Live load say 4470 × 191.0/2 = 427000
603000 #

Try 13'-0" well ok
Shaft say – 6' at top 8' at bottom
Concrete in shaft and coping

coping 7.0' dia	38.5 × .75 =	28.9	
6.5' dia	33.2 × .75 =	24.9	
		53.8	
Shaft top 28.27			
" bottom 50.26			
78.53 ÷ 2 = 39.26 × 20 =		785.2	
		839.0 cubic ft or 3.74	
		2@3.74 = 7.48 cubic ft	242.000
Cross beam		3.33 tons	107.000
well :– 13' dia	132.7		
10.5 dia	86.6		
	46.1 × 20 = 922 or	42.6 ft	
filling 1:2:4	86.6 × 8 = 692	32.0 ft wt	370.000
" 1:3:6	86.6 × 12 = 1040	48.0 ft	~~719.000~~
Excavation –		122.6 ft	

load on bearing –
shaft 121.000
beam 53500
well – 370000
544500 #
Total – Superimposed load 603000
1,147,500 #

Area of base – 13.5 dia – 143
Unit bearing 1147,500/143 = 8000# or 3.57 tons/□'

Estimate of Cost
1:2:4 concrete 15600 1:3:6 concrete ~~12~~ 1300

Estimate of Cost of one pier 15' well –
20' high –

concrete in beam and shafts	1:2:4	14.7 ft	@ 155.00	=	2280
do forms		53.0 ft	@ 15.00	=	795
Concrete in shell of well	1:2:4	10.0 ft	@ 155.00	=	1550
do forms		48.0 ft	@ 9.00	=	432
Concrete filling	1:2:4	9.10 ft	@ 160.00	=	1460
Concrete filling	1:3:6	13.40 ft	@ 131.00	=	1760
Excavation of well		32.80 ft	@ 180.00	=	5900
Plain excavation say		10.00 ft	@ 5.00	=	50
Reinforcing bars say plain bars –		3.0 tons	@ 200.00	=	600
curb shoes say		3.0 tons	@ 320.00	=	960
					15787
Misc expense – 準備費					1578
					17365
		20% risk & Incidentals			3470
					20835 call this 21000.00

85
15
3
25
131

5.00

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Estimate of cost of one pier 15' well 25' high.

concrete in Beam and shafts	1:2:4	14.7 坪	@	155.00	=	2280.00
do forms		530 面坪	@	15.00	=	795.00
concrete in shell of well	1:2:4	12.50 坪	@	155.00	=	1940.00
do forms		60 面坪	@	9.00	=	540.00
Concrete filling	1:3:6	19.30 坪	@	131.00	=	2520.00
Excavation of well		40.9 坪	@	210.00 ✓	=	8600.00
Reinforcing bars		3 tons	@	200.00	=	600
curb shoes		3 tons	@	320.00	=	960
Concrete filling	1:2:4	9.10	@	160.00	=	1460
						19695.00
準備費						1500.00
Risk 其他 Incidentals.						3500.00
						24695.00 call this 25000.00

Estimate of Cost of one pier 15' well 30' high.

concrete in beam + shafts	1:2:4	14.7 坪	@	155.00	=	2280.00
do forms		530 面坪	@	15.00	=	795.00
concrete in shell of well	1:2:4	15.0 坪	@	155.00	=	2320.00
Concrete filling	1:2:4	9.10 坪	@	160.00	=	1460.00
" "	1:3:6	25.00 坪	@	131.00	=	3280.00
Excavation		49.0 坪	@	250.00 ✓	=	12250.00
Reinforcing bars		3 tons	@	200.00	=	600.00
curb shoes		3 tons	@	320.00	=	960.00
forms for shell of well		7200	@	9.00	=	650.00
						24595
						5000
						29595 call this 30.000.00

Estimate of Cost of one pier 13' well 20' high.

concrete in beam and shafts		10.8 坪	@	155.00	=	1675
do forms say		45.0 坪	@	15.00	=	675
concrete in shell of well	1:2:4	8.5 坪	@	155.00	=	1320
do forms say		36 坪	@	9.00	=	324
concrete filling	1:2:4	6.4 坪	@	160.00	=	1025.00
" "	1:3:6	9.6 坪	@	131	=	1260
Excavation say well		24.5 坪	@	180.00	=	4410.00
plain excavation		20.0	@	15.00	=	300.00
Reinforcing bars		3.00 tons	@	200.00	=	600.00
curb shoes		3.00	@	320	=	930.00
						12219.00
variation in design						1000.00
Incidentals.						2000.00
						15219.00 call this 15000.00

Total cost of piers.

End piers.	13' well	2 @ 15,000	=	30,000	
	15' well 30'	1 @ 30.000	=	30.000	
	" 25'	1 @ ~~25000~~	=	25000	
	" 20'	~~4~~ 5 @ 21000	=	~~84~~ 105 000	
				~~169,000.00~~ 190.000.00 円	for no 1 Layout 8-146.5 spans
				169.000.00	for no 2 Layout 7-167.5 spans

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Preliminary Estimate of Cost of [illegible] for Nagamoken and Nagano Electric Ry Co

Highway Flooring – slab $0.5 \times 19 = 9.5$

gutter 1.0

curb. + coping. say 2.0

$12.5 \div 216 =$ say 0.06 cu yd per lin ft

total volume $0.06 \times 1172 = 70.0$ cu yd

Reinforcing bars say $70 \times 1650 = 115500$# or 52 tons.

Pavement $\frac{15 \times 1172}{36} = 490$ sq yd

Handrail – $2 \times 1172 = 2344$ lin ft

Estimate of Cost of Deck

Concrete in Floor	70.0 cu yd @	155.00	=	10850
Reinforcing bars –	52.0 @	230.00	=	12000
Pavement	490 sq yd @	38.00	=	18600.00
Handrail	2344 lin ft @	9.00	=	21100.00
forms for floor slab	690 cu yd	10.00	=	6900
				69450.00

Approximate Estimate of River span

	8 @ 146.5 span = 1172'	7 @ 167.5 span = 1172'
Steel in structure	1176 tons @ 310 = 364,000.00	1246 tons @ 310 = 387,000.00
Substructure	190,000.00	169,000.00
Highway Floor + Pavement complete	69,500.00	69,500.00
	623,500.00	625,500
without floor	554,000.00	556,000.00

Approximate Estimate of Ry bridge approach use old Estimate

Steel in approach spans	315 tons	@ 240.00	=	75,700.00
girder piers.	32 piers	@ 2800.00	=	89,500.00
abutments	2	@ 2500	=	5000.00
				170200.00

½ cost of River span Approximate only 277000.00

447200.00

[illegible]
67,000
176,600
6,700
443,000

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