2-14-'22 made by J. Masuda

Estimate of Cost for Tatigahana Bridge over Chikuma Gawa, Nagano Ken. 1.

This bridge is to be located on the main highway (幹道) between Nagano and Nakano, crossing Chikuma River. The total length of the crossing is about 11.5 ken. The chief Engineer Nishiike instructs us to make the span as long as possible on account of flood ~~the~~ complain made by the residents. Let us try 20' Roadway instead of 18'.

From the above information there are only two layouts for this crossing.

1st 3-220' Steel spans with 2 intermediate piers.

2nd 4-165' steel spans with 3 intermediate piers.

Let us make comparison of the cost for these two layouts.

220' Steel Span

Loading of Bridge.

Unif. Live Load. $q\ kg/m^2 = \frac{100.000}{170+l}$ max 500 kg/m²

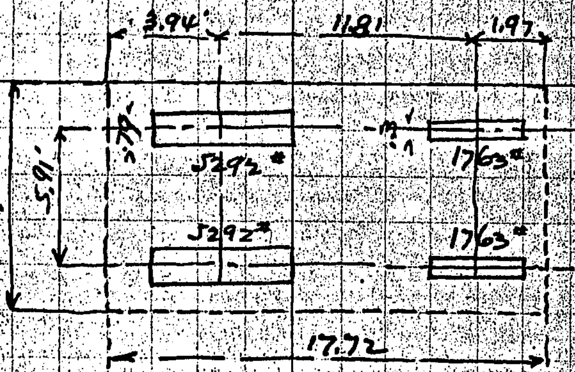
where l = span length of bridge in meter.

Uniform load for 220' span – 86# per sq ft.

Motor Truck Total weight = 14110#

Uniformly distributed load = $\frac{14110}{7.55 \times 17.72}$ = 105.5 #/□'

or $\frac{14110}{17.72}$ = say 800# per lin ft of Truck



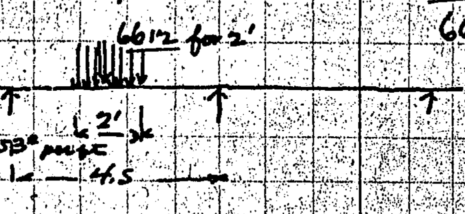
When the trucks are running in series, it is assumed the continuous line of uniform load is on the bridge and take no impact. where the trucks are running side by side take impact as specified below

Impact $I = \frac{L}{L+D}$? L = Live Load D = Dead Load

the impact not over 25% in all cases.

Floor Slab. panel length 4.5'

concentration	5292
Impact 25%	1320
	6612#



Distribution of Load $.79 + 2.58$ = call this 2' (.75 + 1, 1.75)

Dead Load Slab assumed 88# 7" slab including 1" wearing surface. $m = \frac{1}{10} \times 88 \times 4.5^2 = 178$#

Live Load $\frac{6612}{2}$ = 3306# per ft.

moment = 1653 × 2.25 = 3720

Less 1653 × 0.5 = 830

2890

4" wood block pavement	20#
1/2" sand mortar	6
	26#
Slab 6"	75
	101
call this	100#

Shear.

Dead Load 88 × $\frac{4.5}{2}$ = say 200

Live Load 1653

1853

Unit shear = $\frac{1853}{\frac{7}{8} \times 4.85 \times 12}$ = 36#/□" ok

For continuity of slab 0.8 × 2890 = 2310#

Dead Load moment 178

2488'#

Effective Depth = $\sqrt{\frac{2488}{104}} = \sqrt{23.5}$ = 4.85"

Use 6" slab 1" wearing course 7" over all.

steel = $\frac{2488 \times 12}{\frac{7}{8} \times 4.85 \times 17000}$ = .413 □" per ft

use ½"φ bars 6" ctrs = .390 □"

Stringer

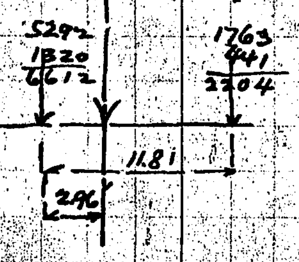
Dead Load 88# × 4.5 = 396#

beam say 84

℄ of gravity 480# per ft

$m = \frac{1}{8} \times 480 \times 20^2$ = 24,000'#

Live Load



Load

6612

2204 × 11.81 = 26050

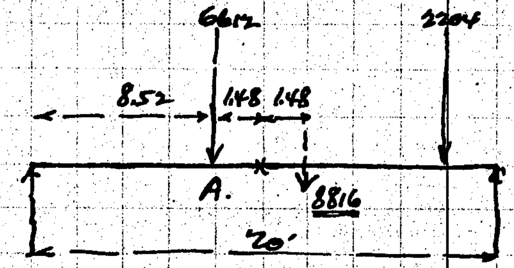
8816 × 2.96' = 26050

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Estimate of cost for Tategahana Bridge over Chikuma Gawa, Nagano Ken.

Stringer (Continued)

Live Load moment.

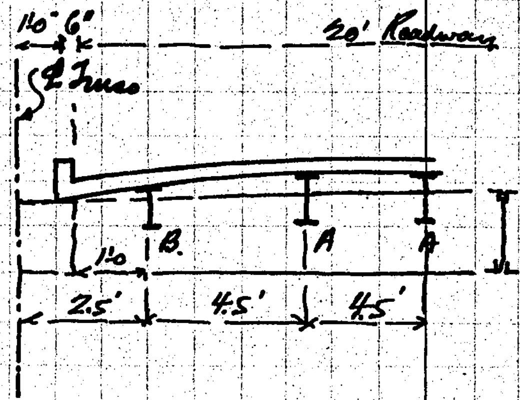


Absolute max at A. 8816 × 8.52²/20 = 32000#
Dead Load moment 24000
VOID
Total moment ---------- 56000#
section modulus req'd = 56000×12/16000 = 42.0

Use 15" I 42# S.M. = 58.9

Try Conc. at middle m = 3306 × 10 = 33060
Dead Load m = 24000
57060 ÷ 16000 = 42.7 S.M. Req'd

Cross Beam. span length 23.0'

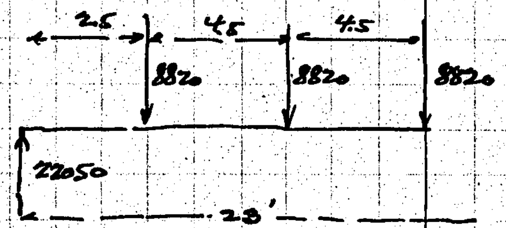


Weight of Roadway slab. 88# × 20 = 1760
curb 2@40 80
1840 ÷ 5 = 368# per stringer

Load on stringer A = 88 × 4.5 = 396#
do 396
(920) (992)

Load on Stringer B. 1840 − 3(396)/2 = 326#
Assuming H.R. 60# 60
386

Assume the load as shown on sketch

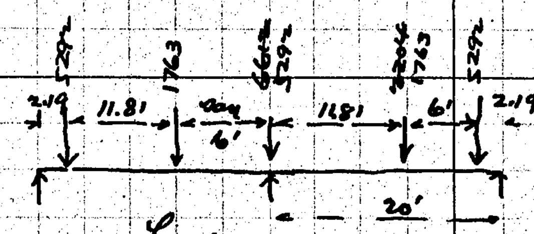


396 × 20 = 7920#
Stringer say 900
8820#

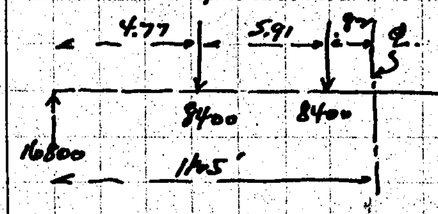
Dead Load moment = 22050 × 11.5 = 253,500
Less. 8820 × 13.5 = 119000
134,500#
Dead Load Beam assumed 1/8 × 150 × 23² = 10,000
144,500

Live Load moment

max Reaction.



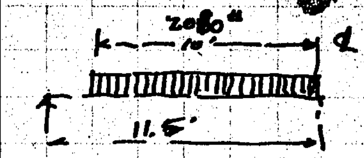
5292
2@5292 × 2.19/20 = 1160
1763 × 14/20 = 1234
1763 × 8.19/20 = 720
8406# per wheel. Call this 8400#



moment. 16800 × 10.67 = 179000
Less 8400 × 5.91 = 49600
129400#

Live Load moment 129400
Dead Load moment 144500
273900#

Try uniform load 2-20' span
load = 104#/0' moment =



20800 × 11.5 = 239000
℄ Less 20800 × 5 = 104000
135000# This moment gives max
Dead Load m 144500
279500#   Use built section for cross beam.

Dead Load End shear 22050
Live Load " " 20800
42850

Try 30 × 3/8 (5/16) web plate = 11.25 / 9.38   1/8 Web = 1.41 / 1.17
30½" back to back L's
Effective Depth = 25.4 − 1.6 = 23.8

11.2 □" req'd for shear. add B.S. Beam
No of Rivets Req'd = 42850/4910 = 9.
7/8"φ
Rivet spacing at end 6562 × 26.5/42850 = 4" (5469, 3.4")

Stress = 279,500 ÷ 2.38 = 117,500
S.R. = 117,500 ÷ 16,000 = 7.33 □" net
7.33 − 1.41 (1.17) = 5.92 (6.16) net
Use 2L5 6 × 3½ × 3/8 = 6.84 gross 6.09 □" net.

weight of one Cross Girder

1 web	30 × 5/16	@ 31.88	× 22.0	=	700
Flanges	4Ls 6×3½×3/8	@ 11.70	× 22.0	=	1030
Stiffeners	16Ls 3½×3½×5/16	@ 7.2	× 2.5	=	288
End Stiffs.	4Ls 3½×3½×3/8	@ 8.5	× 2.5	=	85
Fills	4 Ms 3½ × 3/8	@ 4.47	× 1.92	=	34
Shelf	10Ls 3½×3½×5/16	@ 7.2	× .5	=	36
	Rivet heads + variation	say			150
					2323

weight of metal in Floor System

stringers	5 @ 45# × 222	=		50.000
Int CB	10 @ 2323	=		23230
End CB say	2 @ 2000	=		4000
				77230#

77230 ÷ 222 = say 350# per lin. ft.

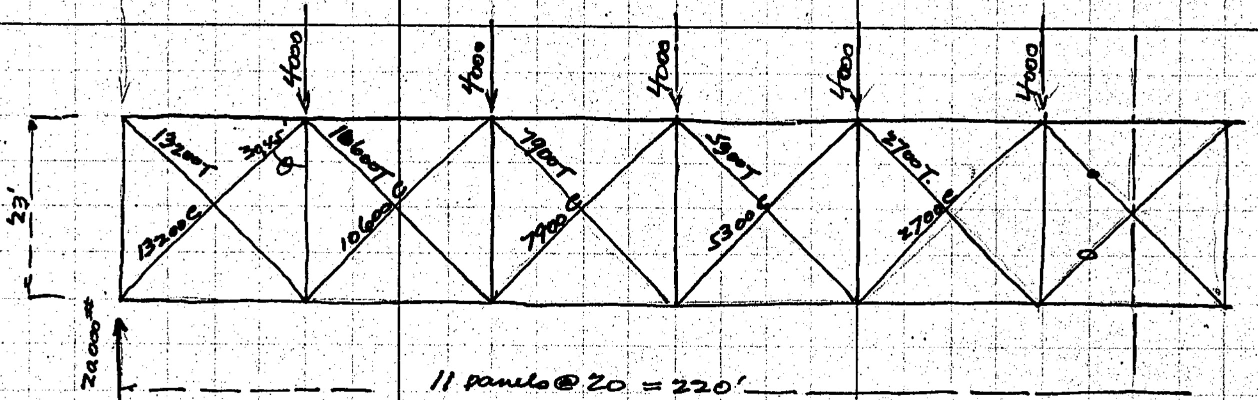
metal in Handrail.

Handrail Post 1L 3×2×5/16 @ 5.0# × say 4.0 = 20.0
connection plates + extra details say 30.0
2 @ 50.0 = 100.0
Handrail. 3Ls 2×2×¼ @ 3.2 × 20 = 192.0
292.0

292. ÷ 20 = say 15# per lin ft.

weight of metal in Handrail.
2 @ 15# × 222 = 6650# per span

Lower Lateral Bracing.



Assume wind load 200# per lin ft which is ample for lower lateral.
Panel concentration = 200 × 20 × 4000#

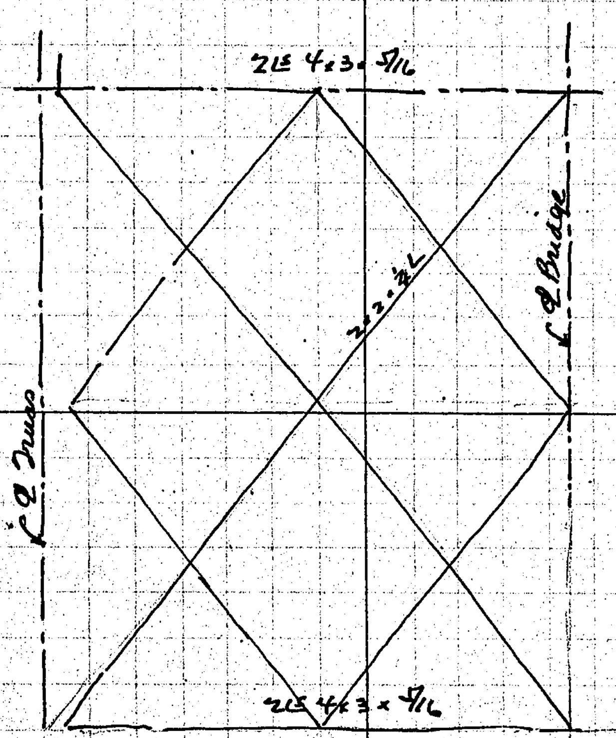
sec θ = 1.32 ~~Try 3½×3½×5/16×15~~ l/r = 150 r req'd = $\frac{15 \times 12}{150}$ = 1.2
Use 2Ls 4×3×5/16 back to back.

weight of Lower Lateral Bracing.
8Ls 4×3×5/16 @ 7.2 × 14.5 = 835
center connection 50
misc say 20
905 @ 11 = 9955#

9955 ÷ 222 = 45# per lin ft.

Upper Lateral Bracing use same as for lower lateral.

Sway Bracing.



2Ls 4×3×5/16	@ 7.2#	× 11.0	=		158
2Ls 4×3×5/16	@ 7.2	× 11.0	=		158
2Ls 2×2×¼	@ 3.2	× 16.0	=		103
4Ls 2×2×¼	@ 3.2	× 8.0	=		103
connection pls + rivets say					150
			2 @	672	= 1344#

10 sway bracing @ 1344 = 13440
Upper Lateral 9 @ 905 = 8145
21585#

97# per lin ft
call this 100# per lin. ft.

Dead Load on bridge

weight of Roadway Slabs	88# × 20 =	1760
curbs	2@40	80
		1840#

Metal in Bridge –

2 Handrails	30# per lin ft
Floor system	350
Lower Laterals	45
Upper Laterals + Sway	100
	525#

From curve the weight of trusses is 740#
Making truss separate add 10%.
7.40 × 1.10 = 8~~90~~# 820
Take this figure for Estimate

Roadway		1840#
Live Load on span	86·20 =	1720
Trusses assumed		800
	Total	4885#

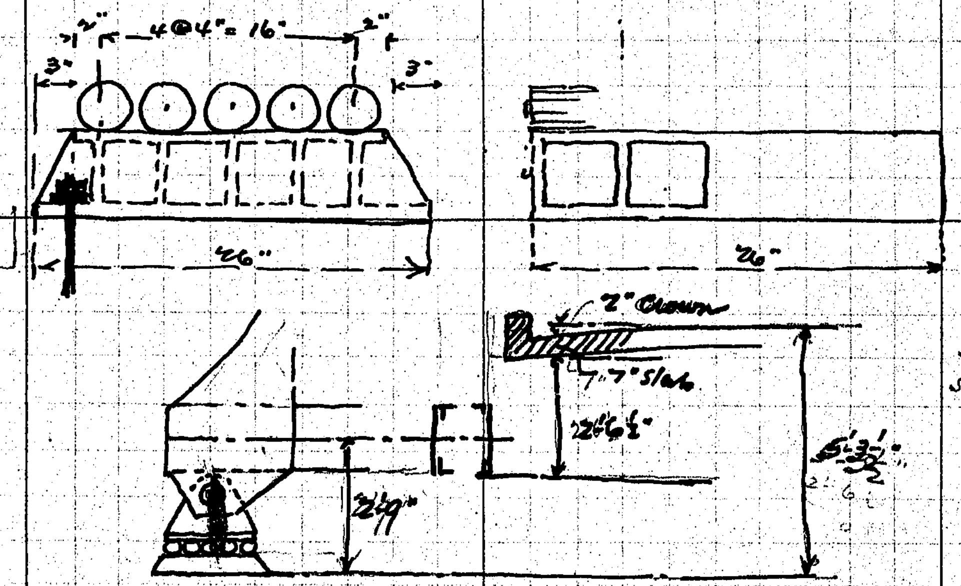
Summary of metal in Bridge

2 Handrails	30
Floor system	350
Lower Laterals	45
Upper Laterals + way	100
2 Trusses	820
On piers.	45
	1390

approximate weight for one span
1390 × 222 = 308,000#
138 tons
3@138 = 414 tons.

Load on pier. Roadway	1840	
metal	1390	
		3230
Live Load		1720
		4950 ÷ 2 = 2475# per truss

Load on pier 2475 × 111 = 274,000#
bearing on masonry 274,000 ÷ 500 = 548 □" 21" × 26" = 546 □"
Length of Roller Good use 4" Roller 2400"
274,000 ÷ 2400 = 114"
Try 3½" Roller 600 · 3½ = 2100 274000 ÷ 2100 = 130 5 Rollers @ 26" each

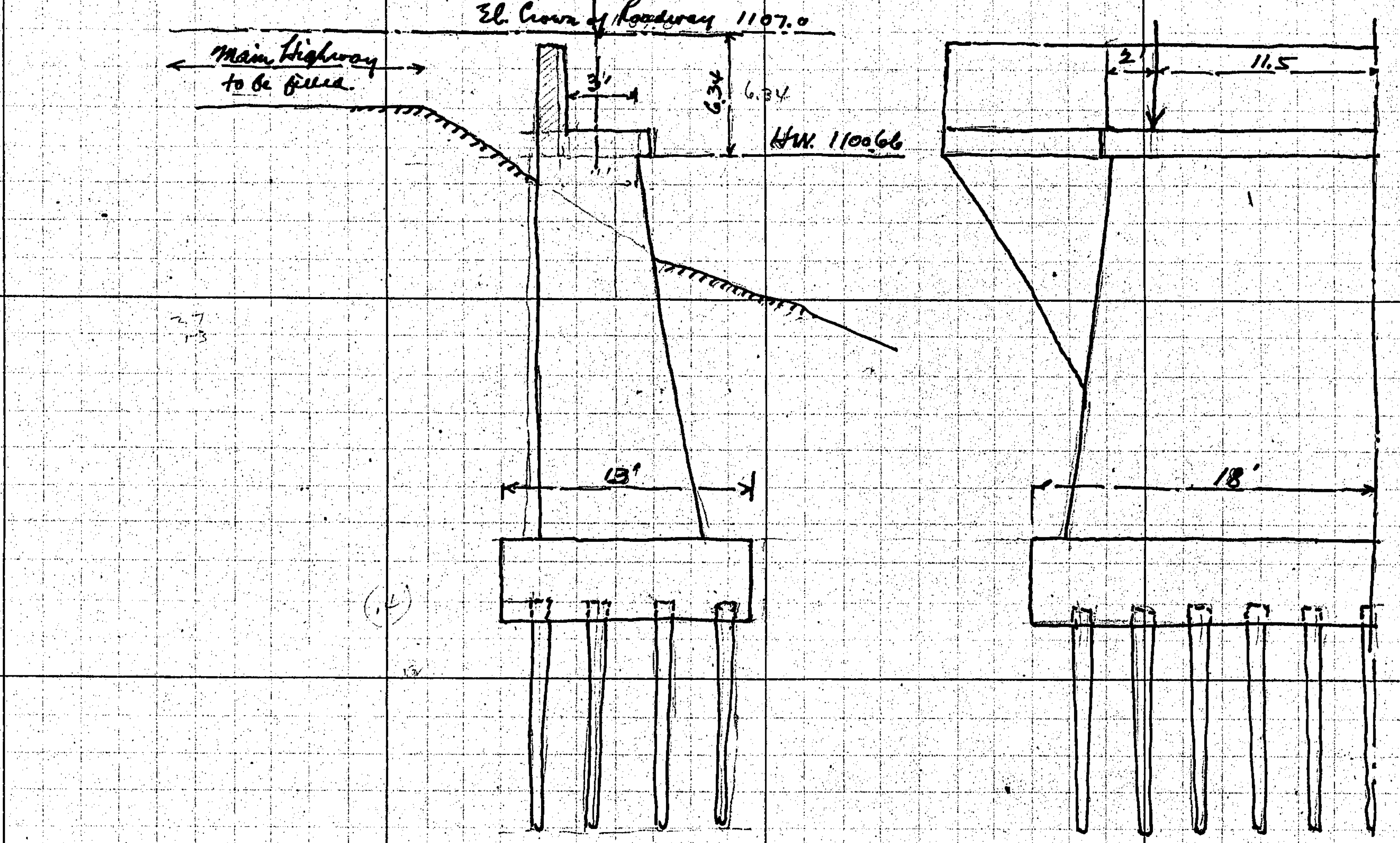


area = 26·26 = 676
unit bearing
= 274,000 / 676 = 410#/□"

3'-0
2-6½
5'-6½"

High water 11.07
11.00.66
6.34

Details of abutment



wall -	5.0 × 1.0 ×	45.0	=	225.0	
coping -	4.5 × 1.0 ×	36.0	=	162.0	
shaft	7' × 3.0 ×	22.0	=	462.0	
base	4 × 13 ×	36.0	=	1875.0	
misc. say				200.0	

2 Abutments @ 13.5 = 27.0 立坪
Reinforcing bars say 5000#
no of piles for 2 abutments = 104

2924.0 ÷ 216 = 13.5 立坪

weight of abutment = 13.5 × 32.400# = 437.000#
Superimposed load 2@ 274.000 = 548.000
985.000#

Bearing = 985000 / 13 × 36 = 2100 #/□'

no of pile used = 52   985000 ÷ 52 = 19000# ok

Amount of Concrete in Floor.

.58 × 21.0 = 11.20
2 - .5 × .5 = .50
11.7 × 666 = 7800 cubic ft or 36.1 立坪 call this 36.5 坪

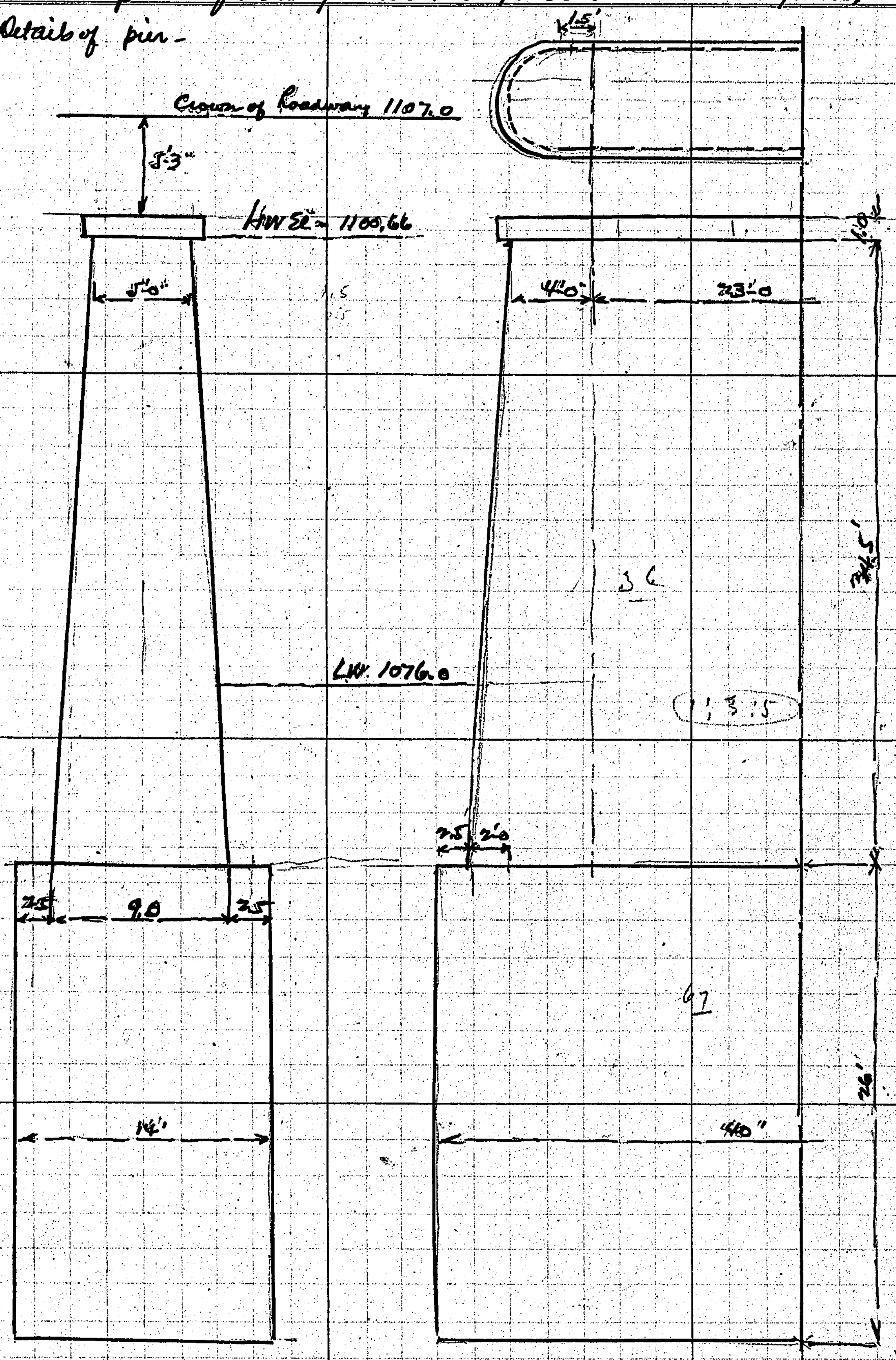
Reinforcing Bars.
straight 2 - ½φ @ .67 × 21.0 = 28.1
bent 2 - ½φ @ .67 × 22.5 = 30.1
58.2 ÷ 1.5 = 39#
curb. 4 - ½φ @ .67   27
Temp. 20 - 3/8φ @ .38   7.6
49.3 call this 50# per lin ft

In floor Reinf. Bars 50 × 666 = 33300#
In abutment " " say 5000
38300# or 17.1 tons.

Excavation. 36 × 13 × 20 = 9350 ÷ 216 = 43 坪
2@ 43 = 86 立坪
Excavation of base of pier say. 150 立坪

Estimate of Cost for Tategahana Bridge over Chikuma Gawa, Nagano Ken. **6**

Details of pier -



volume -
coping. 6 × 26 = 156
6' dia - 28.3
184.3
volume = 184.3 ÷ 216 = .853 cu yd

Shaft
top 5 × 26 = 130
5' dia 19.6
149.6
Bott. 9 × 26 = 234
9' dia 63.6
297.6

149.6
297.6
447.2 ÷ 2 = 223.6 aver
volume = 223.6 × 34.5 = 7700
7700 ÷ 216 = 35.6 cu yd

Base 14 × 40 × 26 = 14,550
14550 ÷ 216 = 67.3 cu yd

Total Concrete
coping .9
shaft 35.6
base 67.3
103.8 cu yd

wt cu yd 216 × 150 = 32400#

weight of pier = 103.8 × 32400 = 3360.000#
Super imposed Load 4@274000 = 1096.000
4456,000

Bearing Pressure = 4456.000 / 14 × 40 = 8000#

4 short ton /□' ok
when pier is sunk into gravel soil

Estimate of cost (3-220' SPANS)

Structural Steel erected & painted	414 tons	@ 350	=	144900.00
Concrete in 2 piers	208 cu yd	@ 320.00	=	66560
Concrete in abutments	27 " "	@ 270	=	7290.00
Timber piling	104 "	@ 12.00	=	1248.00
Concrete in Floor slab.	36.5 cu yd	@ 450.00	=	16425.00
Reinforcing Bars.	17. tons	@ 200.00	=	3400.00
Excavation for abutment	86 cu yd	@ 10.00	=	860.00
Excavation for pier base	150 cu yd	@ 100.	=	15000
				255683
				25568
				281251
230 cu yd Pavement @45.00				10350
				291601.00

111 × 20/6 = 370

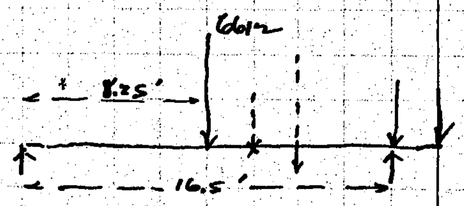
291601.00 ÷ 370 = 788 $/ft

165' span For this layout there are 4 spans and 3 intermediate piers.
Panel length @ 16.5' making 10 panels for the span.

Loading on Bridge same as for 220' span, Use 14110# motor truck.
Uniform load – 93# per sq ft.
Floor slab same as for 220' span
Stringer
Dead Load 88# × 4.5 = 396
Beam say 34
430# per ft

moment = ⅛ × 430 × 16.5² = 14650'#

Live Load



Concentration at center moment 3306 × 8.25 = 27300'#
Dead Load moment 14650
41950'#

section modulus req'd 41950 × 12 / 16000 = 31.4

Use 12" × 31½# I = 36.0

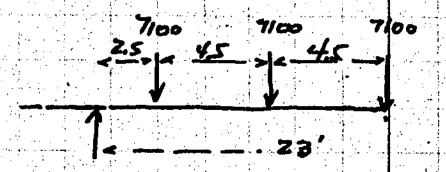
max End shear
Dead Load 430 × 16.5/2 = 3550
Live Load 6612
10162#

weight of stringer 12" I @ 31½# × 16.5 = 520
4 Ls 3½ × 3½ × ⅜ @ 8.5 × .75 = 25.5
Rivet heads say 5.5
551.0

no of Rivets 3 – for single shear
For bearing 10162/8400 = 2 rivets

551.0 ÷ 16.5 = call this 34# per ft.

Cross Beam



Load on stringer 88 × 4.5 = 396
stringer 34
430#

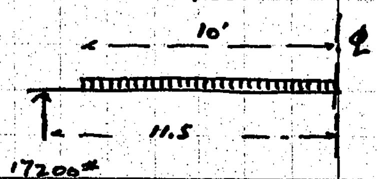
concentration 430 × 16.5 = 7100# assume this loading for all stringer connection.

17750
Dl Beam 1150
18900

moment 17750 × 11.5 = 204.000
Less 7100 × 13.5 = 96.000
108.000#

Dead Load beam ⅛ × 100 × 23² = 6600
114600'#

Try unif. Live Load 2-165' span
load = 104#/o'



moment 17200 × 11.5 = 197800
Less 17200 × 5 = 86000
111500
Total moment 226100'#

Try 30 × 5/16" web plate = 9.36 □" ⅛ web = 1.17
Effective Depth say 25.4 – .16 = 23.8
Stress 226100 ÷ 23.8 = 95.000#
section Req'd = 95000/16000 = 5.94
1.17
4.77 □" net Use 4 × 4 × ⅜ 5.72 □" 4.97 □" net

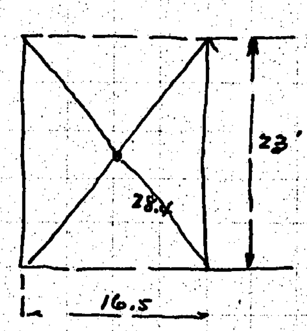
weight of Cross Beam.
1 web. 30 × 5/16 @ 31.88 × 22.0 = 700
Flanges 4Ls 4×4×⅜ @ 9.8 × 22.0 = 860
Details say 600
2160#

weight of metal in Floor System
Stringers 5 @ 34 × 167 = 28400
Int CB 9 @ 2160 = 19.700
End CB 2 @ 2000 = 4000
52100#

52100 ÷ 167 = 312# per lin. ft.

metal in Handrails.
2015# = 30# per lin. ft of span.

Lower Lateral Bracing.



L/r = 150 r req'd = (14.2 × 12)/150 = 113

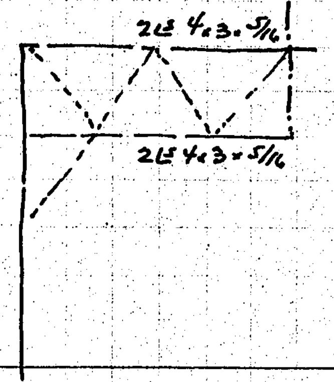
Use 2Ls 4×3×5/16" b.t.b.

weight of lower laterals. (1 panel).

8Ls 4×3×5/16 @ 7.2 × 13.5	=	780
center connection		50
misc say		20
		850 × 10 = 8500

8500 ÷ 167 = 51# per lin ft of span.

Sway Bracing & Upper laterals.



Sway Bracing.

2Ls 4×3×5/16 @ 7.2# × 11.0	=	158
2Ls 4×3×5/16 @ 7.2# × 11.0	=	158
1L 2×2×1/4" @ 3.2 say 30	=	96
Connection pls &c say		120
	2 @ 532 =	1064#

9 sway Br. @ 1064	=	9600
upper lateral 8 @ 850	=	6800
		16400

16400 ÷ 167 = 97# per lin ft of span.
call this 100# per ft.

Dead Load on Bridge –

weight of Roadway slab & curbs = 1840# per ft

metal in Bridge

2 Handrails.	30
Floor system	312
Lower Laterals	51
Upper Laterals & Sway	100
Trusses assumed	650
	1143

	2983
Live Load on Bridge 93 × 20	1860
	4843#

From curve weight of trusses = 520#
add 10% 52
572# per ft.

Total metal in Bridge –

2 Handrails	30
Floor system	312
Lower Laterals	51
Upper Laterals & Sway	100
Trusses	572
On piers say	60
	1125# per ft

approximate weight per span

1125 × 167 = 188.000# 84 tons
4 @ 188.000 = 752.000# 336 tons

Load on Pier

Roadway	1840
metal	1125
Live Load	1860
	4825# per ft

On one bearing $\frac{4825}{2} \times \frac{167}{2}$ = 201.000#

On pier 4 @ 201.000 = 804.000#

Roller End 3½" Roller 600 × 3½ = 2100# per lin. inch.
201000 ÷ 2100 = 96" 4 Rollers @ 24"
make size of shoe 24" × 22". center to center 2'-0".

From this the size of pier is practically same as for 220' span.

Piers. Reduce 6" in thickness and 1'-0" for base.

volume

coping. 6×25 = 150 — 5.5' dia 23.7 — 173.7

volume = 173.7 ÷ 216 = .80

shaft top 4.5×25 = 112.5 — 4.5' dia 15.9 — 128.4

Botty 8.5×25 = 213. — 8.5' dia 56.7 — 269.7

398.1 ÷ 2 = 199.0

Base 13×38×26 = 12850

12850 ÷ 216 = 59.5

vol = 199. × 34.5 = 6850

6850 ÷ 216 = 31.7 立坪

Concrete in pier

coping	0.8
shaft	31.7
base	59.5
	92.0 立坪

3@ 92. = 276 立坪

weight 92 × 32400 = 2980.000

superimposed Load 804.000

3784.000

Bearing $P = \frac{3784000}{13 \times 38} = 7650\#/\square'$ ok

Abutments make same as for 220' span

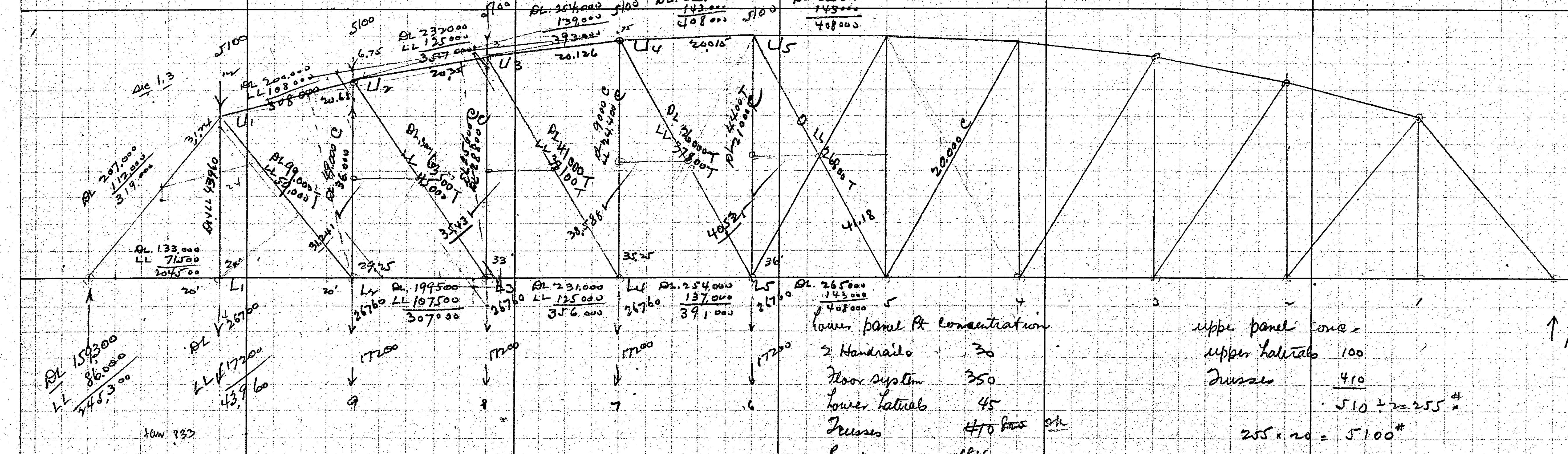
Excavation for Pier Bases. say 220 立坪

Estimate of cost. (4-165' SPANS)

Structural steel erected + painted.	336 tons	@	350.00	= 117600
Concrete in 3 piers	276 坪	@	320.00	= 88320
Concrete in abutments	27 坪	@	270	7290
Timber Piling	104	@	12.00	1248
Concrete in Floor slabs	365 坪	@	450.00	16425
Reinforcing Bars	17. tons	@	200.00	3400
Excavation for Abutments	86 坪	@	10.00	860
Excavation for pier Bases.	220 坪	@	100.00	22000
				257143
			10%	25710
				282853
Wood block Pavement 2×115 = 230 坪 — 230 @ 45.00 = 10350				10350
				293203

293203 ÷ 370 坪 = 792 円/坪

Estimate of Cost for Tatigahana Bridge -



Lower panel Pt concentration

2 Handrails	30
Floor system	350
Lower Laterals	45
Trusses	~~410~~ 410 ok
Roadway	1840
	~~2688~~ ~~2675~~ = 1547 ÷ 2 = 1338 per truss

~~1338 × 20 = 26760#~~

~~1844 × 20 = 3~~

upper panel conc.

upper laterals	100
Trusses	410
	510 ÷ 2 = 255#

255 × 20 = 5100#

Live load = 1720 for span

panel load $\frac{1720}{2} \times 20 = 17200$#

haw 833

1115

$1115 \times \frac{24}{31.24} = 856.00$

$176 \times \frac{29.25}{35.43} = 145.25$

$313 \times \frac{33}{38.585} = 268$

$960 \times \frac{35.25}{40.525} = 835$

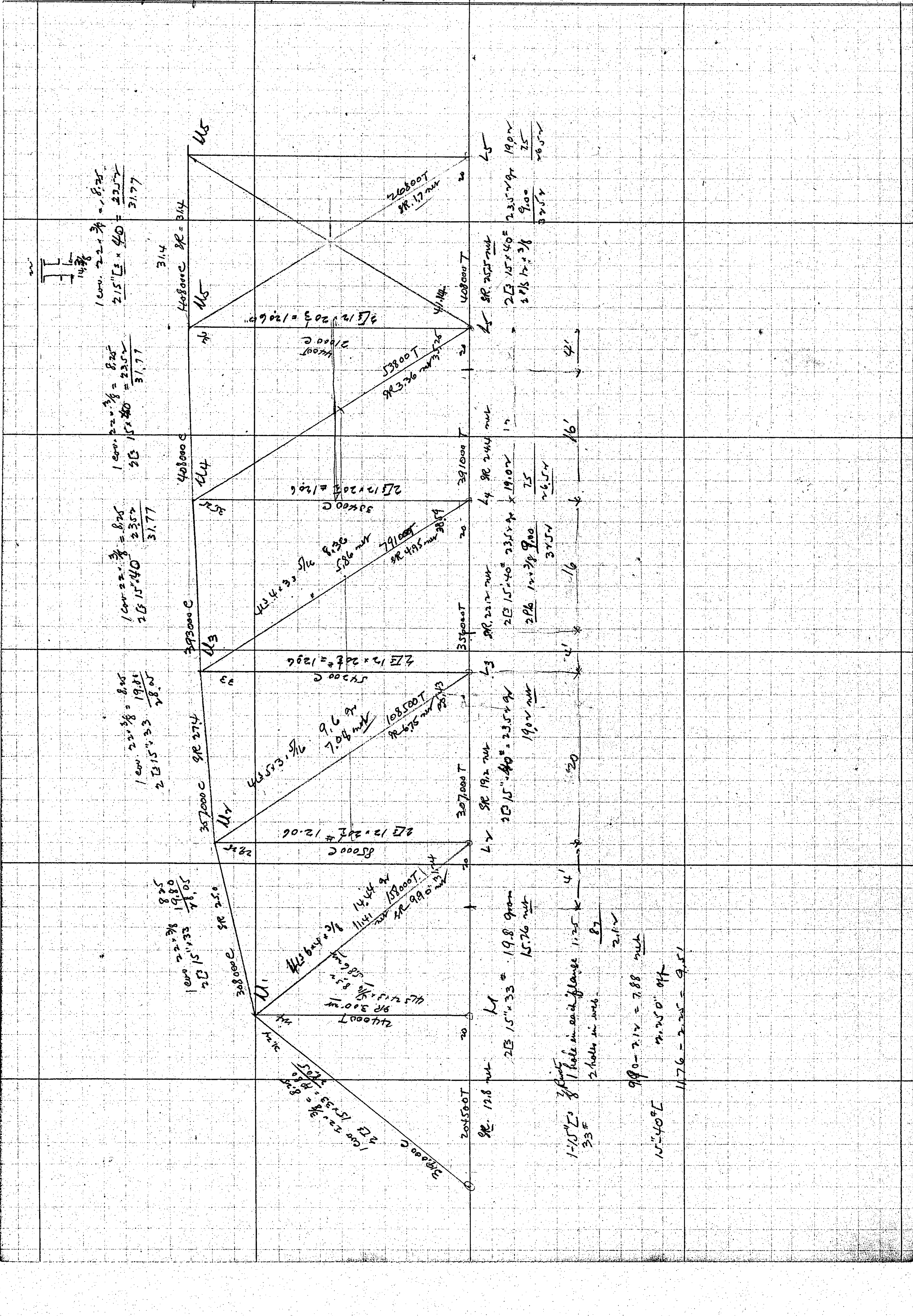
$29.25 \times \frac{20}{20.68} = 28.30$

$33.00 \times \frac{20}{20.35} = 32.4$

$35.25 \times \frac{20}{20.126} = 35.00$

$36.00 \times \frac{20}{20.015} = 35.99$

				Reaction R1	R2
1	1	0.091	× 17200 =	1560	15640
2	3	0.273	" =	4700	29700
3	6	.546	" =	9400	42200
4	10	.909	" =	15600	53700
5	15	1.363	" =	23450	
6	21	1.910	" =	32800	
7	28	2.547	" =	43800	
8	36	3.270	" =	56200	
9	45	4.090	" =	70300	


U1
U2
U3
U4
U5
L1
L2
L3
L4
L5
308000 C
357000 C
393000 C
408000 C
204580 T
307000 T
359000 T
391000 T
408000 T

Estimate of Cost for Zatigahama Bridge.

1/2 Truss.

Top chord	$U_3 - U_5 - M$	31.77 @ 3.4 = 108#	× 50.0	=	5400
" "	$U_1 - U_3$	28.05 @ 3.4 = 92.5#	× 42.0	=	3880
" "	$L_0 - U_1$	28.05 @ 3.4 = 92.5	× 31.0	=	2870
Bottom chord	$L_0 - L_2$	19.8 @ 3.4 = 67.2	× 36.0	=	2420
	$L_2 - L_3$	= 80.0	× 28.0	=	2240
	$L_3 - L_5$	32.52 @ 3.4 = 111.0	× 46.0	=	5110
Diagonals	$U_1 - L_2$	14.44 @ 3.4 = 49.0	× 31.0	=	1520
"	$U_2 - L_3$	9.6 @ 3.4 = 32.6	× 35.0	=	1140
"	$U_3 - L_4$	8.36 @ 3.4 = 28.5	× 38.0	=	1080
"	$U_4 - L_5$	8.36 @ 3.4 = 28.5	× 41.0	=	1180
"	$U_5 - L_5'$	8.36 @ 3.4 = 28.5	× 41.0	=	1180
Hanger	$U_1 - L_1$	8.36 @ 3.4 = 28.5	× 24.0	=	680
post	$U_2 - L_2$	41#	× 29.25	=	1190
"	$U_3 - L_3$	"	× 33	=	1350
"	$U_4 - L_4$	"	× 35	=	1430
"	$U_5 - L_5$	"	× 36	=	1480
					34150#

Main section for one truss 2 @ 34150 = 68300

Details say 33% 22600

90900# ÷ 222 = 410# per ft.

For 2 trusses 820# per lin. ft.

assumed weight OK

Supplement p1.

13

Estimate of Cost for Tategahana Bridge over Chikuma Gawa, Nagano Ken,

Mr. M. Makino instructs us to make this bridge heavy enough to carry loadings for Main Highway (国道).

Try 3-220' spans — Total length edge to edge 115 Ken.

Approximate concrete in Floor.

18 × 665 = 1200 say 56 yd

Reinforcing Bar 75# @ 665 = 50,000# 22.3 tons

Load on bridge —

metal.

2 Handrails	50	
Floor system	400	
Lower laterals	50	
Upper laterals & Sway	100	
Trusses	1035	
On piers	55	
	1690#	1690
Roadway say		2500
Live Load —	32000 / 18	1780
Uniform load —		500
		6470# per ft.

From curve weight of 2 trusses = 900
Add 15% = 135
1035

Total metal in bridge 1690 × 220 = 372000# 166 tons

3 @ 166 = 498 tons call this 500 tons.

Estimate of cost (3-220' spans).

Structural Steel	500 tons	@ 350	=	175000
Concrete in piers, say	220 yd	@ 320	=	70400
Concrete in abutment	30 "	@ 270	=	8100
Timber piling	104 #	@ 12.00	=	1248
Concrete in Floor slab	56 yd	@ 450.00	=	25200
Reinforcing bars say	25 tons	@ 200.00	=	5000
Excavation for abutment	86 yd	@ 10.00	=	8600
Excavation for pier base	150 "	@ 100.00	=	15000
Wood block pavement	230 yd	@ 45.00	=	10350
				318898
				15945
				334843

78500
33000
111000
7000
8000
15
65

334843 ÷ 370 = 905 yen/yd

2916

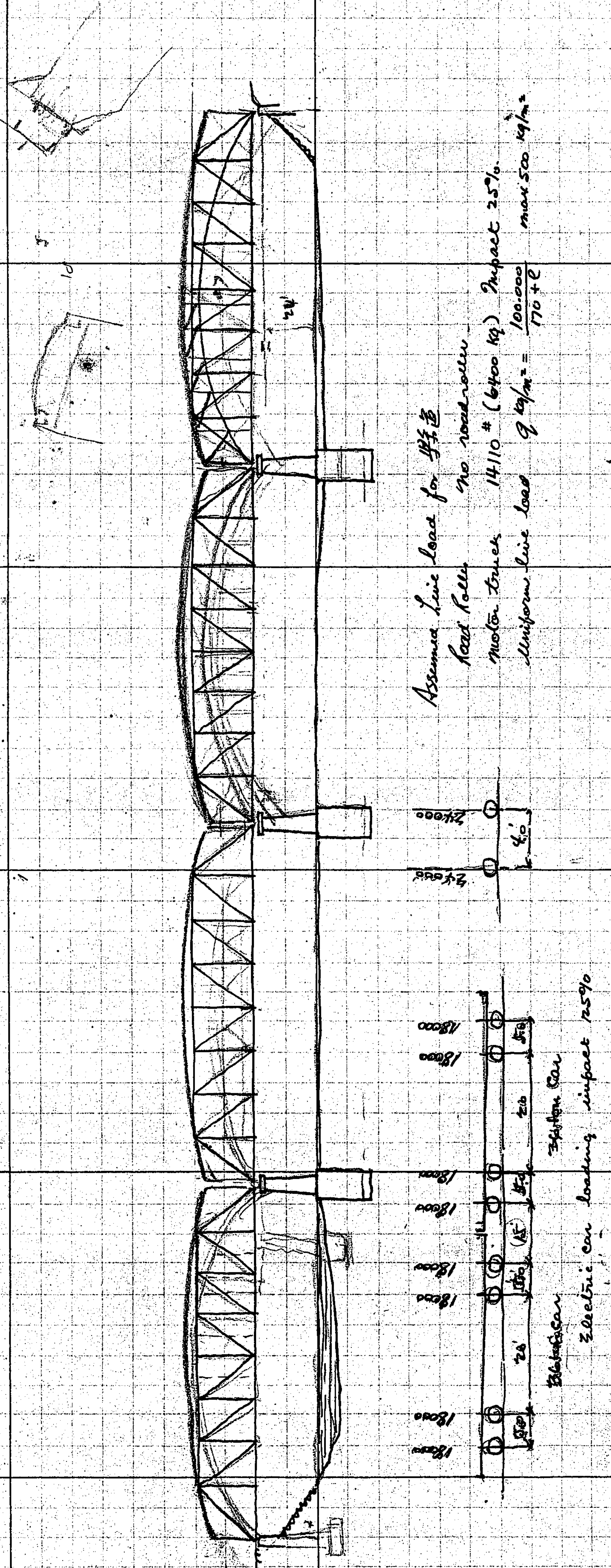
43000

15%

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Sept 24-'23

Highway Bridge Uedabashi, Naganoken



The present Uedabashi which was built 22nd year of Meiji was damaged during flood last Sept 1922. The main damage of the bridge is the settlement of pier which foundation was scoured by the flood, sunk 2.5 down and the pier shaft inclined 2' about at the top toward center line of river.

The present bridge is composed with 5 spans of How trusses with 4 piers and 2 abutments. Two trusses damaged during flood are rather dangerous for the passage of traffic. Even the tempory repair to make the structure safe enough to the traffic will cost 50.000 yens. This bridge is proposed to renew during fiscal years of '13–'16. It is advisable to build the new bridge instead of spending money for temporary repairs and turning down after the completion of new bridge.

The estimated cost of new bridge for 18' roadway is 300,000 yen, However The Electric Car Company which operating car line on the other side of river intends to extend the line cross the river into the City of Uyeda.

The Engineer of the Electric Car Co miss-estimated the cost of new bridge (private) for electric car company. On the side of company it is best to lay rails on the new bridge which will be built by the office of Nagano prefecture and share the cost extra cost of bridge.

Let us estimate the cost of new bridge for 20' clear roadway & one electric car line which will occupy 7'-0" and making the total width of 27' over all.

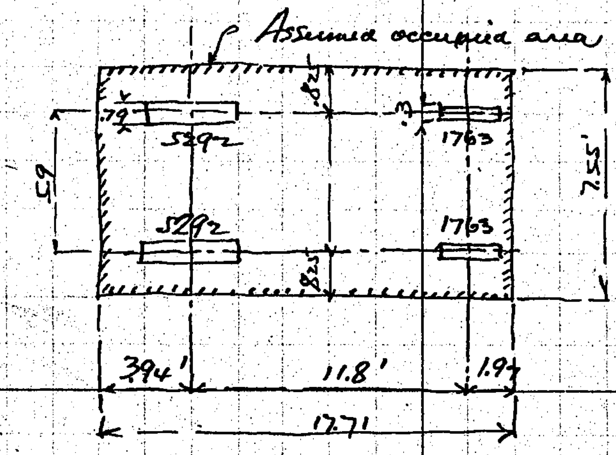
For the opinion of designer it is best to locate the car line on side of roadway but this estimate is based on car line on center of bridge.

The total length of new bridge will be 672ft or 668' between end bearings. For my opinion the spans should be 4 and 3 piers in the ~~bridge~~ river.

Let us make 4 spans @ 166' with 2' space between bearings. The total length will be 670' over all. Panel length = 8@20.75 = 166.

Truss height say 28'-0"

Distribution of motor trucks 14110# (6400 kg).



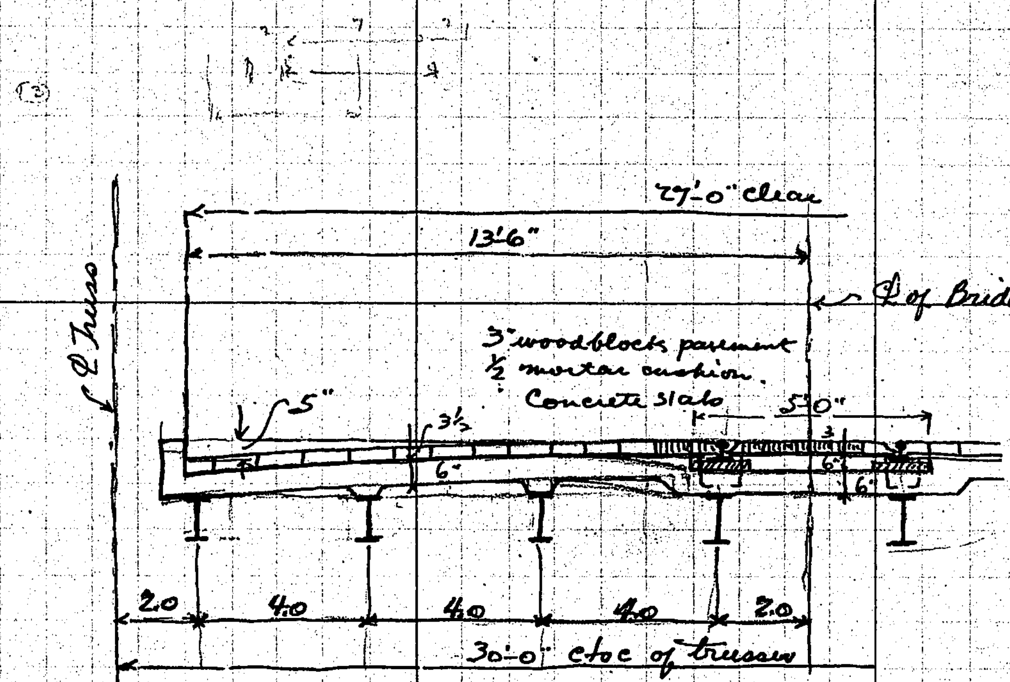
Uniformly distributed load of motor truck

$\frac{14110}{7.55 \times 17.71} = 10.55$ #/□' or $105.5 \times 7.55 = 796$# per lin. ft.
say 800# per ft.

When the motor trucks are running in series, the continuous line of motor truck is assumed on the bridge without impact; when the 2 trucks are running side by side the impact of 25% is allowed.

Uniform load for 166' span 453 kg or 99.66# say 100#/□'

Cross section of Structure.



Roadway slab panel length 4.0'

Concentration of rear wheel =	5292
25% Impact	1320
	6612#

Distribution of wheel load 2'-0"

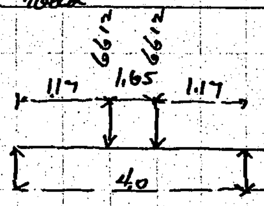
Dead load:—

woodblock pavement 3" =	15
½" cushion	6"
6" concrete assumed	75
	96#/□'
woodblock pavement 3" =	15
½" cushion	6
5½" sand fill	55
6" Concrete slab	75
	151#/□'

$m = \frac{1}{10} \times wl^2 = \frac{1}{10} \times 96 \times 4^2 = 153.5$'#
$= \frac{1}{10} \times 151 \times 4^2 = 242.0$'#

Shear $= 151 \times 2 = 300$#
$96 \times 2 = 192$#

Live Load



moment for concentration $6612 \times 1.17 = 7730$'#
For continuity of slab. take $\frac{2}{3} \times 7730 = 5150$'#
For 1' Strip $5150 \div 2 = 2575$'#
Moment Dead load 242
2817'#

For 600#/□" & 16000#/□" stresses of concrete & steel
Effective depth required $= \sqrt{\frac{2817}{95}} = 5.4$" make depth 6".

Steel required for slab. $= \frac{2817 \times 12}{\frac{7}{8} \times 5.25 \times 16000} = .46$ □" Use ½" bars 5" Centers.

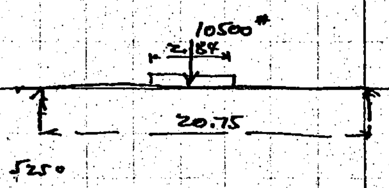
Stringers span length = 20.75'

Dead load say	400
Stringers assumed	45
	445

$m = \frac{1}{8} \times 445 \times 20.75^2 = 24,000$'#
Dead load shear $= 445 \times \frac{20.75}{2} = 4600$#
Distribution say $2\text{-}11" + 12 = 2.84$

Live Load on stringer $6612\# \times \frac{3.18}{4.00} = 5250$
5250
10500#

moment $= \frac{10500}{2} \times 10.37 = \frac{109,000}{2}$'# $= 54500$'#
less. $5250 \times \frac{2.84}{4} = -3700$
50800'#



Summary of Moments

Dead Load m	24000
Live Load m	50800
	74800'#

Sm required $= \frac{74800 \times 12}{16000} = 56.0$ Use 15" I × 42#

Stringers under tracks.

Dead load say 150 × 4 = 600
stringer 60
660#

Live Load concentration say 12000
Impact 25% 5000
17000# (Void).

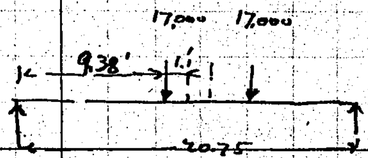
Moment = $\frac{1}{8}$ × 660 × 20.75² = 35600 #
Live Load m 136000
Total 171600 #

Sm = $\frac{171600 \times 12}{16000}$ = 129.0

Use 20" × 80# I Sm = 146.6

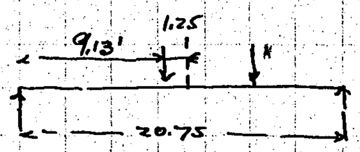
Distribution = 2.6 + 1.0 = 3.6' about
max moment

34000 × $\frac{9.38^2}{20.75}$ = 144,000 #
Less $\frac{17000}{2}$ × $\frac{3.6}{4}$ = −8000
136,000 #



max End shear 17000 × $\frac{16.75}{20.75}$ = 13750
17000
30750

Try Live Load conc. of 9000 × 1.25 = 11250#



22500 × $\frac{9.13^2}{20.75}$ = 90300
less $\frac{11250}{2}$ × $\frac{3.6}{4}$ = 5000
85300 #

Total m = 35600
85300
120,900

Sm = $\frac{120900 \times 12}{16000}$ = 90.

Use 18 × 60# I.

Summary for stringers 6@ 42# 252
2@ 60# = 120
details say 18
390# per lin ft of span.

Floor Beam spacing – 20.75'

Dead Load Floor

wood block pavement 15
½" mortar cushion 6
21# × 27 = say 570#

6" concrete slab say 75# × 30' = 2250#

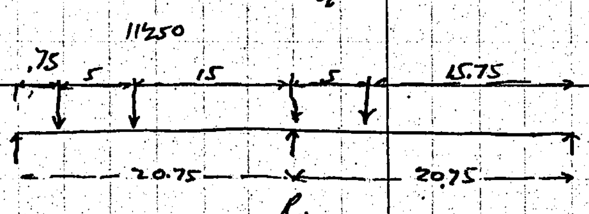
6" sand fill under tracks 60 × 5.0 = 300
2 Rails 2–70# 47#
Stringers 350#
3517 ÷ 27 = 130#/□' average.

Load on Floor Beam
130 × 20.75 = 2700#
Floor Beam say 150
2850

Dead Load moment = $\frac{1}{8}$ × 2850 × 30² = 321,000 #
shear say 2850 × 15 = 43000#

Live Load

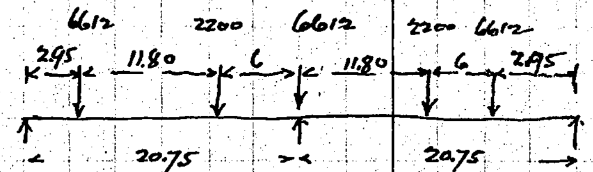
Electric Car loading.



20.75
15.75
5.75
.75
43.00

R_1 = 11250 × $\frac{43.0}{20.75}$ = 23300# one ~~side~~ side per rail.

Motor truck loading



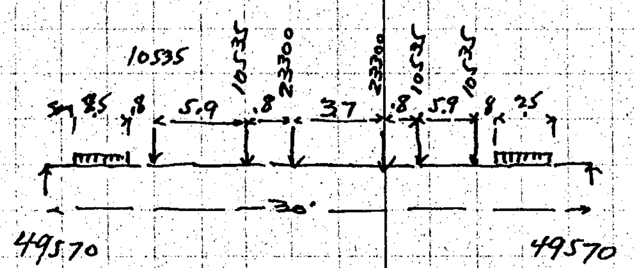
Front wheel 1763 × 1.25 = 2200#

6612 × 2.95
20.75
23.70 ÷ 20.75 = 7550

6612 × 2.95 ÷ 20.75 = 465

2200 × 8.95
14.75
23.70 ÷ 20.75 = 2520
10535#

Uniform loading 100 × 20.75 = 2075# per ft 2075 × 2.5 = 5200#



Reaction 5200
10535
10535
23300
49570

Floor Beam

moment 49570 × 13.15 = 652.000 #
Less 10535 × 7.5 = 79000
5200 × 9.55 = 49700
− 128700
523300 #

Summary of moments
Dead Load Moment 321.000
Live Load Moment 523300
844300 #
Shear DL. 43.000
LL. 49570
92570 #

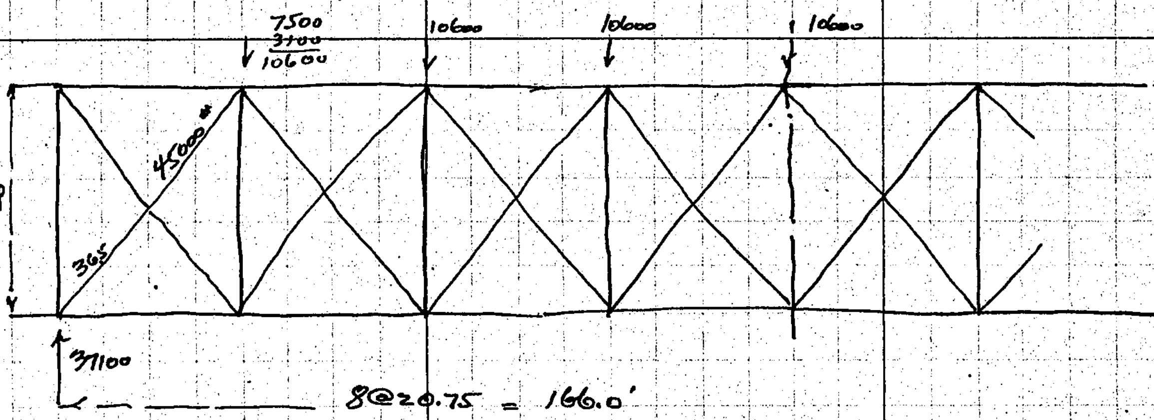
Section of Floor Beam
Section required for shear = 9.25 □" Try 30 × 3/8 = 13.50" f web = 1.69 □" 30¼' b to b of Ls
Effective depth = 30.4 − 1.60 = 28.8' Stress = 844.300 ÷ 28.8 = 292.000 # or 18.25 net
less 1.69
16.56 net
Try 2Ls 6×6×9/16 = 12.86 (11.50) 10.61 net
1 Pl 14 × ½ = 7.00 6.00 − − − − 30 × $\sqrt{\frac{6}{16.61}}$ + 2 = 20'
16.61 □" net

Weight of Intermediate Floor Beam
1 web. 30 × 3/8 @ 38.25 × 29.0 = 1110
Flanges 4Ls 6×6×9/16 @ 21.90 × 29.0 = 2540
Cov. pls. 2Pls 14 × ½ @ 23.80 × 20.0 = 950
Stiffeners 14 Ls 3½ × 3 × 7/16 @ 6.1 × 3.0 = 256
End stiffs 4Ls 3½ × 3½ × 3/8 @ 8.5 × 3.0 = 102
Fills. 4Pls 6½ × 9/16 @ 11.43 × 2.0 = 92
Shelf Ls 12Ls 4×4×3/8 @ 9.8 × .75 = 88
Fills say. 100
Rivet heads & variation 5%. 262
5500 #

5500 ÷ 30 = 183 # per ft of girder
5500 ÷ 20.75 = 265 # per ft of span
7 @ 5500 = 38500
2 @ 5000 = 10000
48500 ÷ 166 = 292 #

Metal in Handrail say 60 # per ft 2 @ 60 = 120 #

Lower Lateral Bracing.



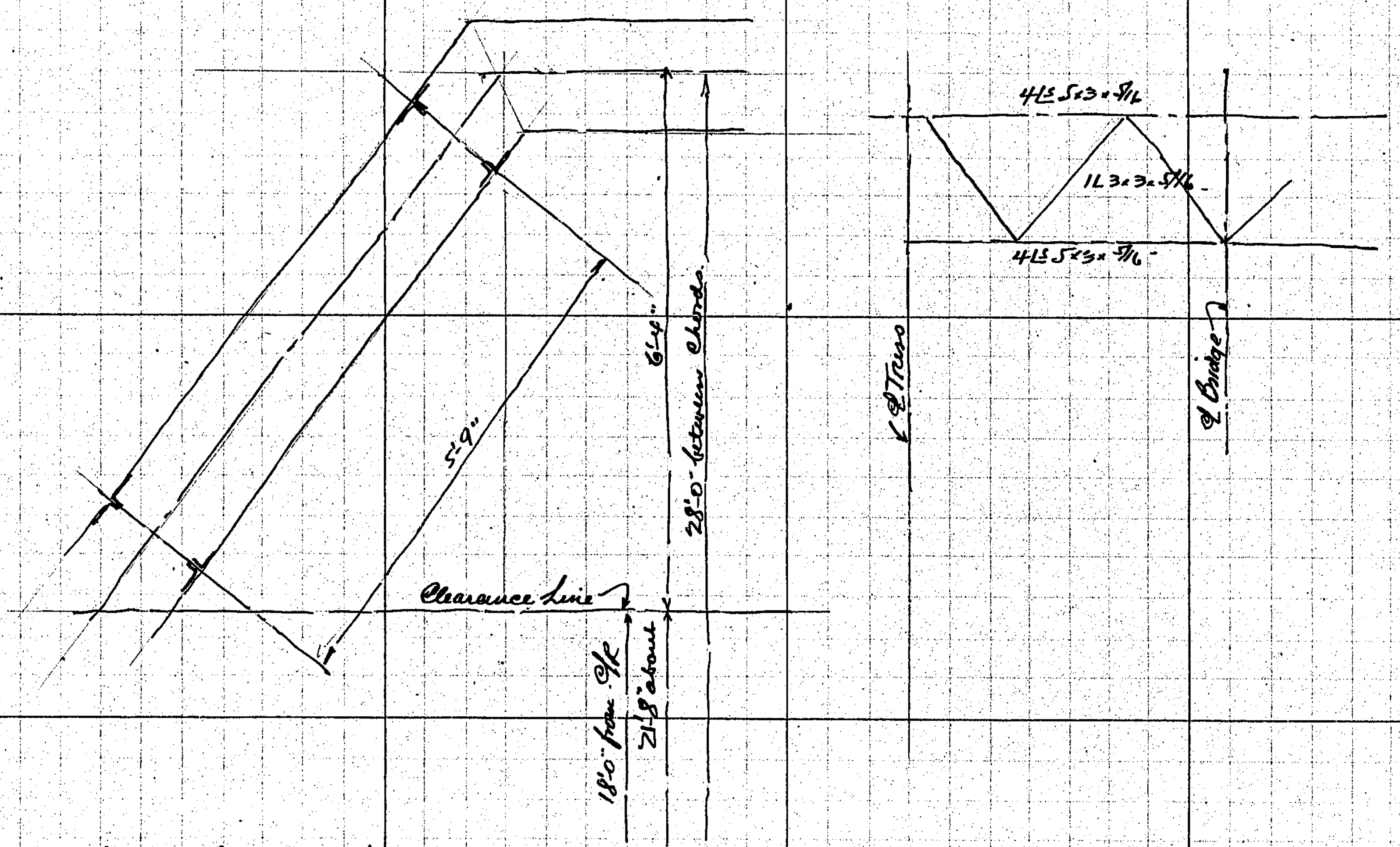
Static wind load 30 #/□'
wind load for bottom chord
Say 5 × 30 = 150 #
moving wind load 12 × 30 = 360 #
concentration
Static 150 × 20.75 = say 3100
moving 360 × 20.75 = say 7500
Sec θ = 1.215
tan θ = 0.69

Section for lower chord 22500 ÷ 16000 = 1.4 □" net
Try 1L 3½ × 3½ × 3/8 = 8.5 # 2.48 □" gr. 1.83 □" net

Weight of Lower Laterals. 2 − 8.5 # × 36.5 = 620 #
Misc connections 200
820 ÷ 20.75 = 40 # per ft call this 50 # per lin ft.

Upper Lateral Bracing −
stiff member for strut effective span length say 29' × 12 = 348" l/r = 150 r = 2.32
Use 4Ls 5 × 3 × 7/16 laced with 5/16" bars. r = 2.42
Diagonal Bracing 36.5/2 = 18.25 220" l/r = 150 r = 1.48
Try 2Ls 5 × 3½ × 7/16" −
Weight of upper lateral bracings.
strut say. 4Ls 5 × 3 × 7/16 @ 8.2 # × 29. = 950 #
Ti pls. say 75
lacing say 240
1265
Diagonals 4Ls 5 × 3½ × 7/16 @ 8.7 # × 35' = 1220
Details say 75
1295 − 1295

Portal Bracing.



weight of Portal Bracing

Top + bottom struts	2 @ 1265	=	2530#
Braces 6 @ 3×3×5/16 @ 6.1# × 8.0 =	300		
Details say	170		
			470
			3000#

Sway Bracing	2Ls 5×3×5/16 @ 8.2 × 29.0 =	475
	6Ls 3×3×5/16 @ 6.1 × 10.0 =	366
	Details say.	170
		1011

call this 1000#
strut say 1300
2300#

Summary for bracing.

Sway Bracings	5 @ 2300	=	11500
Diagonal Bracings	6 @ 1300	=	7800
Portal Bracings	2 @ 3000	=	6000
			25300 ÷ 166 = 152# per lin. ft of span.

Metal in Bridge.

Handrails	2 @ 60# =	120	
Stringers		390	
Floor Beams		292	
Lower Laterals.		50	
Upper Laterals, Sway Br + Portals		152	1004#
Trusses assumed			900
			1904

Dead Load of Floor say 3200#

Live Load 2 lines of motor trucks
2 @ 800 = 1600#
Uniform load 500
Elec Car 1600# 800 impact

Total Live Load	3700
Dead Load Floor	3200
Steel	1900
	8800

Total load ~~4400~~ 4800 per truss
From curve the weight of truss = ~~520~~ 540
For 2 trusses ~~1040~~# 1080 per ft

For one truss ~~4400~~# 4800 9600

metal in Bridge -

Handrails	120
Stringers	390
Floor Beams	292
Lower Laterals	50
Upper Laterals, Sways, & Portals	152
Trusses	1040 (80)
Shoes	50
	2094 call this 2100#
	2134

Approximate weight of metal in one span
2100 × 166 = Say 350,000#
or 156 tons per span.
For 4 trusses 4
624 tons.

Load on pier -

metal	2100
Dead Load floor	3200
Live Load	3700
	9000# per lin ft.

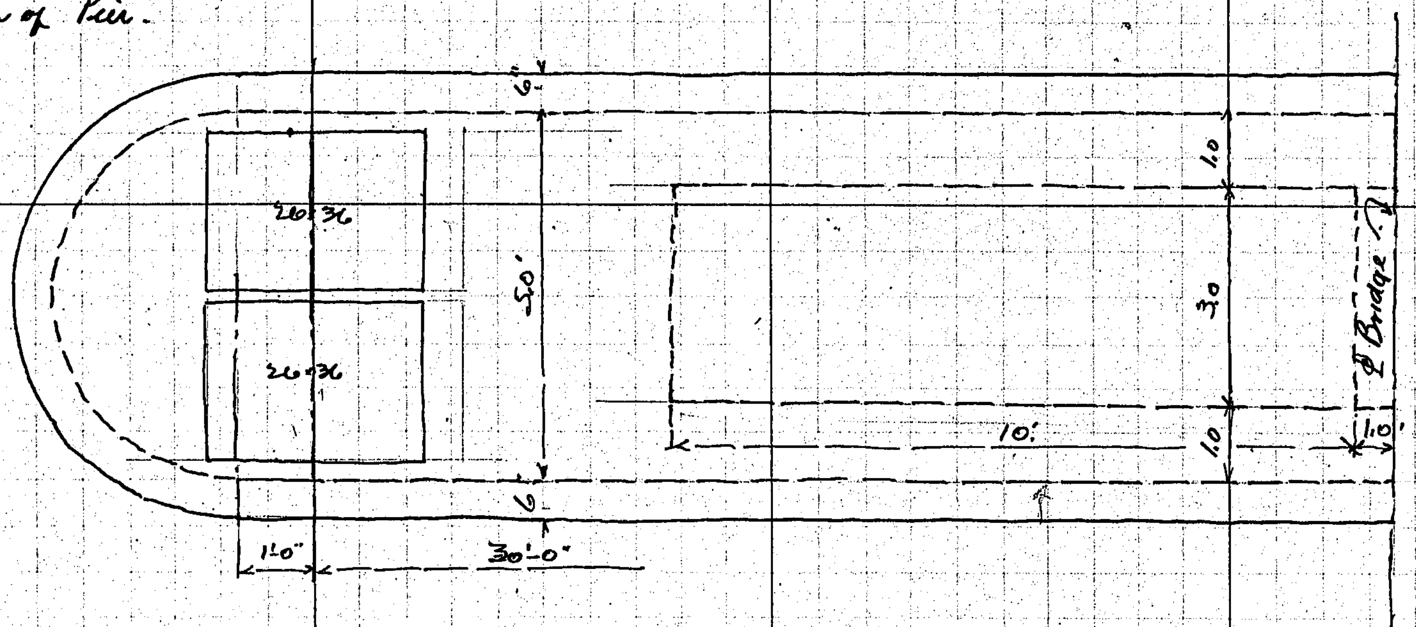
On one bearing $\frac{9000}{2} \times \frac{168}{2} = 378000\#$
× 4
On pier - 1,512,000#

4" Roller - 4 × 600 = 2400#   378000 ÷ 2400 = 157.5 lin inches
5 Rollers . 31.5" "
Try 4½" Rollers 4.5 × 600 = 2700   378000 ÷ 2700 = 140.0 lin inches
5 Rollers 28" " length - 30"

Size of Base Plate   5 @ 4½ = 22½"
2
24½   make plate 26" × 36"

Bearing Pressure = $\frac{378000}{26 \times 36}$ = 403#/□" ok.   Center to center of End bearings 2.25'

Design of Pier.



Coping 1'-0" thick
Height of shaft 30'
Height of base 20'
batter of shaft 1/20
} assumed.

volume of coping
coping 32 × 6 = 192
6' dia 28
220

volume of shaft
Under coping 32 × 5 = 160
5' dia 19
179
32 × 8 = 256
8' dia 50
306
485 ÷ 2 = 242.5 □'

coping -	220
shaft	5470
	5690 × 150 = 855000#

volume of concrete
5690 ÷ 216 = 26.4 cu yd

Pressure at bottom
volume = 242.5 × 30 = 7270 cubic ft.
Depth of water say 25 × 62.5 = 1560#   Less 60 × 30 - 1800
5470 cubic ft.

$m = \frac{1}{12} \times 1560 \times 10^2 = 13{,}000$ 1#

steel required = $\frac{13{,}000 \times 12}{\frac{7}{8} \cdot 27 \cdot 16000}$ = 0.41 □"   Use ¾" □ bars 12" centers

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Reinforcing Steel = 2 @ 3/4"φ @ 1.5# × 28 = 84.0
20 - 5/8"φ @ 1.04 × 10 = 20.8
104.8# × 30 = 3,140#
2 @ 3140 = 6280# 2.8 tons. ~~1.4 tons~~

Concrete 1800 ÷ 216 = 8.3 立坪 @ 250 = 2080
Steel 2.8 tons @ 200 = 540
Saving = 1540 yen

Base of Pier. 14 × 44 × 20 = 12300 cubic ft @ 150 = 1,850,000#
12300 ÷ 216 = 57.0 cubic tsubo

Total load - Superimposed load 1,512,000
Shaft 855,000
Base 1850,000
4,217,000

Soil pressure = $\frac{4217,000}{14 \times 44}$ = 6840#/□' or 3.05 tons/□'

Design of Abutment
Approximate only shaft 25 × 36 × 5 = 4500
base 16 × 36 × 5 = 2900
7400 ÷ 216 = 34 立坪

Excavation for abutments 16 × 36 × 25 = 14400 ÷ 216 = 66.5 立坪
2 @ 66.5 = 133 立坪

Excavation for piers 14 × 44 × 25 = 15400 71 立坪 3 @ 71 = 213 立坪

Wood block pavement $\frac{27 \times 676}{36}$ = 507 坪

Concrete in Floor slab.
slab. 0.5 × 27 = 13.5
piers say 0.5 × 2.5 = 1.25
curbs 2 - 0.5 × 1.3 = 1.30
16.05 × 676# = 10850 cuft - 50 立坪
Reinforcing steel say 25 tons.

Lean sand mortar for track trench 0.5 × 5.0 = 2.5 × 676 = 1690 cuft - 78 坪

Piers - Shaft + coping. 3 @ 26.4 = 79.2 say 80 坪
Reinforcing steel 3 @ 2.8 = 8.4 tons.
Bases. 3 @ 57.0 = 171.0 立坪
Excavation 213.0 立坪
Abutments Concrete 2 @ ~~66.5~~ 34 = ~~133~~ 68 坪
Excavation 2 @ 66.5 = 133 坪

Steel Rails + fastenings 670 × $\frac{2-70}{3}$ #Rail = 31,200# 14 tons

Estimate of cost 27' roadway Electric car line on ℄ Bridge.

1.	Structural Steel	~~624~~ 632 tons	@ 360.00 =	~~224640~~ 227520.00	
2.	Concrete in Floor slab.	50 坪	@ 250.00 =	12500.00	
3.	Reinforcing steel	33.4 tons	@ 200.00 =	6680.00	
4.	woodblock Pavement	507. 坪	@ 40.00 =	20280.00	
5.	Sand mortar in track	7.8 坪	@ 150.00 =	1170.00	
6.	Steel Rails	14.0 tons	@ 150.00 =	2100.00	}
7.	Track Layout	670'	@ 2.50 =	1675.00	}
8.	Concrete in shaft + coping	80 坪	@ 300.00 =	24000.00	
9	Concrete in bases of piers	171 坪	@ 250 =	42750.00	
10	Excavations	213 坪	@ 400 =	85200.00	
11	concrete in abutments	68 坪	@ 220 =	14960.00	
12	Excavation abutment	133 坪	@ 25.00 =	3325.00	
				442160.00	
	Misc Expenses. 5%			17600	
			Total Cost.	459760	

170
170
11900
17
2890
1160
1290

300

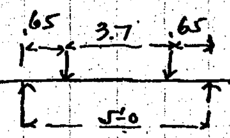
50000

Roadway 18'

Structural Steel	396 tons	@ 360	142500	
Concrete in slab	34.4 坪	@ 250	8600	
Reinforcing Steel.	17.2 tons	@ 200	3440	
woodblock pavement	338 坪	@ 40	13520	
Concrete in shafts of piers	59 坪	@ 300	17700	}
Concrete in bases of piers	116 坪	@ 250	29000	} 114700 / 3
Excavation	170 坪	@ 400	68000	} 38,000
Concrete in abutments	51 坪	@ 220	11220	
Excavation	100 坪	@ 35	3500	
			297480	
			11500	
			308980.00	

17700

Let us make estimate of cost of the bridge for Electric Car line only.
Electric car loading as shown on pp 1.
Span length assumed same as for Kendō highway bridge - say 4-166' panels 8 @ 20.75',
Spacing of stringer 5'-0"
Size of Rail - 60# per yard spacing of tie 7½"
Concentration 24000# wheel conc = 12000# Load distributed over 3 ties
12000 ÷ 3 = 4000 Ties - 6" × 9" × 8'-0"



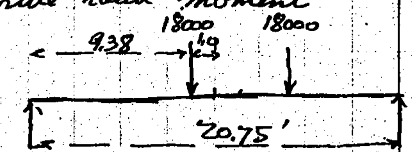
moment = 4000 × .65 = 2600'#
Add 200'#
2800

f = 6 × 2800 × 12 / 9 × 6² = 622 #/□" ok.
Impact 50% 311
933 #/□"

weight of tracks. 2 Rails @ 60 = 120 ÷ 3 = 40#
accessories - 15% 6#
Ties 6"·9" × 8'-0" 120#/2 = 60#
Guard rails .5 × .5 × 2 20#
126# per lin ft
call this 150#

Stringer span length 20.75 Dead load per stringer 150 ÷ 2 = 75
Axle load 24000 Stringer say 75
Impact 50% 12000 150
36000# ÷ 2 = 18000 per rail.

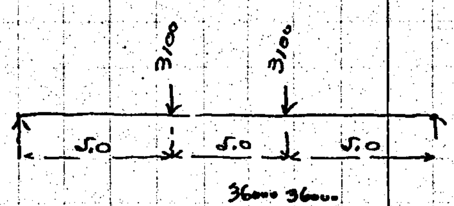
Live Load moment max moment - 36000 × 9.38²/20.75 = 152500'#



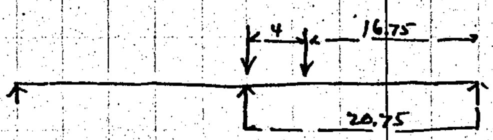
Dead Load moment 1/8 × 150 × 20.75² = 6080
158580'#

Section modulus required 158580 × 12 / 16000 = 119.0
use 20" × 65# I Sm = 117.0.

Center to Center of trusses = 15.0'
Cross beams concentration 150 × 20.75 = 3100#
Dead Load Moment = 3100 × 5 = 15500'#



Live Load moment conc = 32550
moment = 32550 × 5 = 162750'#
Dead Load m 15500
178250'#



36000 × 16.75/20.75 = 29100
36000
65100

Assume web = 27" × 3/8 = 10.10" ⅛ web = 1.26
d = 2.29 - .17 = 2.12 S = 178250 ÷ 2.12 = 84000#
Section required 84000 ÷ 16000 = 5.25
1.26
3.99 □"
use 2L 3½ × 3½ × 3/8 4.96 gr 4.21 net

weight of Intermediate Floor Beam =

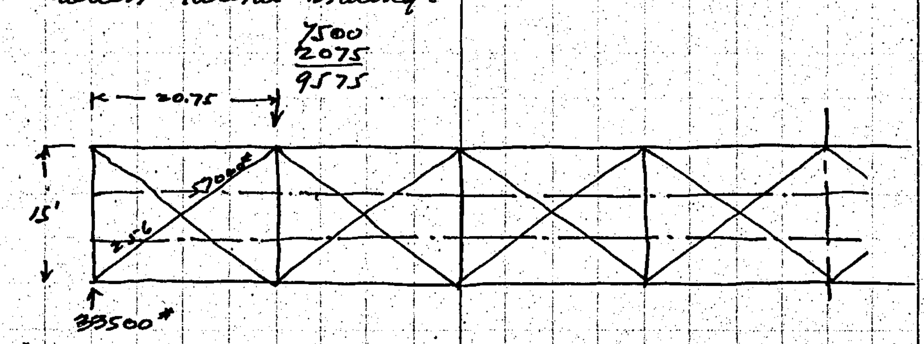
web	1 Pl	27" × 3/8	@ 34.43	×	14.0	=	482.
Flanges	4 Ls	3½ × 3½ × 3/8	@ 8.5	×	14.0	=	476.
End stiffs -	4 Ls	3½ × 3½ × 3/8	@ 8.5	×	2.25	=	48
Fills	4 Pls	3½ × 3/8	@ 4.46	×	1.6	=	29

Rivet heads + variation say 50
1085# ÷ 20.75 = 52# per ft of span

9 @ 1085 = 9750#

Stringers 20" I 65# 2 @ 70 = 140#
details say 5 Total weight = 140 × 166 = say 24000#
70

Lower Lateral Bracing.

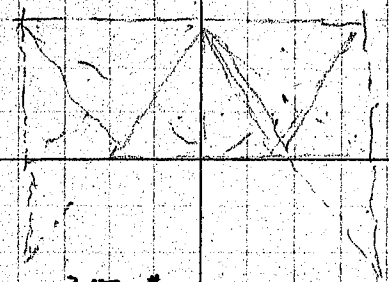


Static wind load 30#/□'
wind load for bottom chord 3⅓×30# = 100#
Concentration 100 × 20.75 = 2075#
moving wind load 12 × 30 = 360
concentration 360 × 20.75 = 7500
secθ = 1.70

28500 C&T 28500 ÷ 16000 = 1.780□" Try 1L 3½ × 3½ × 3/8" @ 8.5# 2.480□" gr 1.83□" net

weight of lower laterals
2Ls 8.5# × 25.0 = 425#
misc connections 200
625# ÷ 20.75 = 30# per lin ft.

Upper Lateral Bracing.
Stiff member for strut Effective span say 14 × 12 = 168" 168 ÷ 150 = 1.12
Try 2Ls 3 × 2½ × 5/16
Use same section for bottom strut of sway bracing
weight of sway bracing
4Ls 3 × 2½ × 5/16 @ 5.6# × 14 = 314#
4Ls 3 × 3 × 5/16 @ 6.1 × 8 = 195
Connections 5@25 125
634# × 5 = 3170#



Diagonals. 25 × 12 = 300 l/r = 150 r = 2.0
Try 2Ls 3½ 5 × 3 × 5/16 @ 8.2 × 25 = 410#
details say 125
535 × 6 = 3210#

Portal Bracing. 8Ls 3 × 2½ × 5/16 @ 5.8# × 14 = 628#
4Ls 3 × 3 × 5/16 @ 6.1 × 8 = 195
connections 150
973 × 2 = 1950#
8330#

8330 ÷ 166 = 50# per lin ft of truss

Metal in Bridge	
stringers	70.0
Floor Beams	60.0
Lower Lateral	30.0
Upper Laterals & Portals	50.0
trusses assumed say	800.0
metal in shoes	50.0
	1060.0#

Dead load on deck 150#
Live Load 72000 car
Impact 100% 72000
144000 ÷ 45 = 3200
3350 ÷ 2 = 1700#
From curve weight of truss 530
350# per truss 2230
call this 400# per lin ft

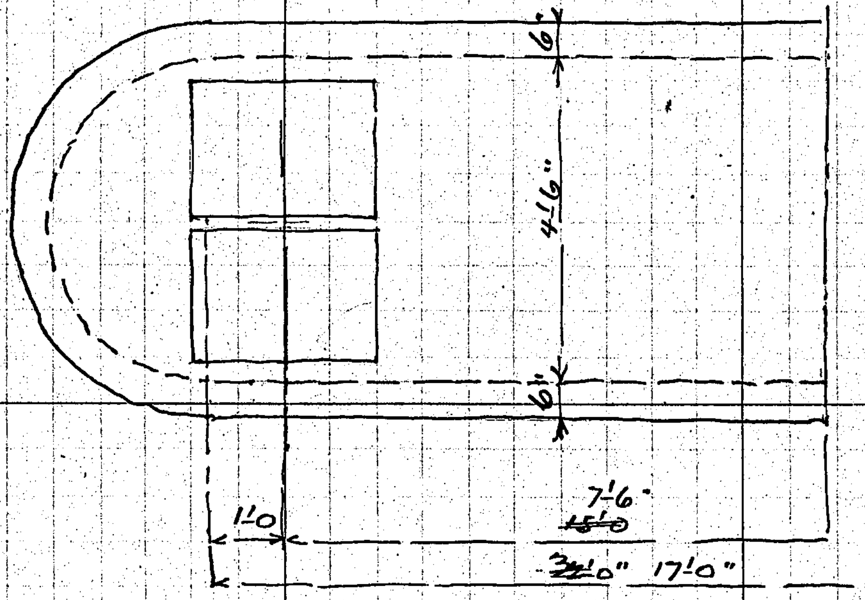
metal in one span = 1060 × 166 = 176,000# or 78.5 tons call this 80 tons

Steel in Bridge = 4 × 80 = 320 tons.

Load on piers 2230 × 2 = 4460 4460 × 166 = 740.000#

Load on bearing shoe 2230 × 166/2 = 185,000#
4" Roller 4 × 600 = 2400 185.000 ÷ 2400 = 77.0 lin inches.
4 Rollers 20" net make bearing plate 20" × 26"
185000/(20×26) ÷ = 356#/□" OK.

Piers.



coping 17 × 5.5 = 93.5
5.5' dia = 23.7
117.2 cubic ft

shaft top area = 17 × 4.5 = 76.5
4.5' dia = 15.9
← 92.4

bottom area = 17 × 7.5 = 127.5
7.5 dia 44.2
171.7
264.1

volume = $\frac{264.1}{2}$ × 30 = $\frac{7920}{2}$ = 3960.

coping 117.
shaft ~~3960~~
~~4077~~ @ 150 = ~~610,000~~#
~~18.9~~ 立坪

Base = 12 × 30 × 20 = 7200 cubic ft @ 150# = 1,080,000
33.3 立坪

Excavation say. 40 立坪

Total load superimposed 740.000
shaft 610.000
bases 1080.000
2,430.000

Max pressure $\frac{2430.000}{12 \times 30}$ = 6750# /□'
3.0 tons /□'

Abutments.
concrete in abutments $34 \times \frac{30}{44}$ = 23 立坪 2@23 = 46 立坪.

Excavation say - 50 立坪 2@50 = 100 立坪.

Steel Rails & fastening. 14 tons.

Ties. 2'-0" centers - $\frac{670}{2}$ = 335 pieces.

Estimate of Cost Electric Car line

1.	Structural Steel	320 tons	@	360.00	=	115200.00
2.	Steel Rails + accessories	14 tons	@	200	=	2800.00
3	Ties –	335 pieces	@	4.00	=	1340.00
4	Concrete in shaft + coping of piers	57 立坪	@	300.00	=	17100.00
5	Concrete in bases of piers	100 立坪	@	250.00	=	25000.00
6	Excavation for piers	120 立坪	@	400.00	=	48000.00
7	Concrete in abutments	46 立坪	@	220.00	=	10120.00
8	Excavations of abutments –	100 立坪	@	135.00	=	13500.00
						223060.00
	Designing Fee + Superintendent			[illegible]%		21150
						244210
						234210

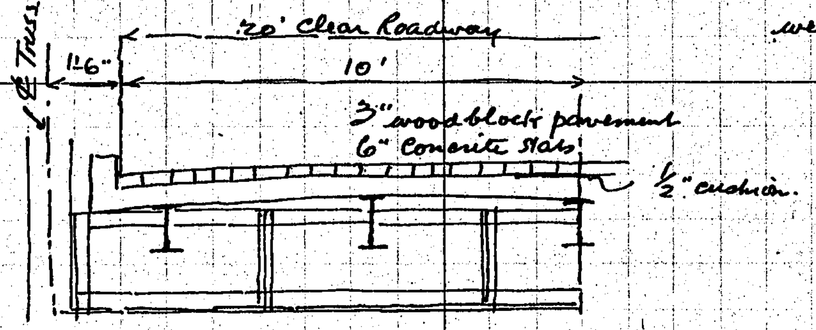
1002

Say Estimated cost of bridge 245,000.00

Highway bridge Roadway 20'0" clear woodblock pavement on concrete slab.
Span length 166' 8 panels @ 20.75' center to center of trusses = 23'0".

Assumed loading – 14110# (6400kg) motor trucks.

Floor slab – 6" thick.
Stringers 15" × 42# I
weight of stringer
5 @ 47# = 23.5# per ft of span



Cross Beam span length 23'0"
Same as for Tategahana Bridge – Intermediate Floor Beam say 2400# per piece.
End Floor Beam say 2200#

7 @ 2400 = 16800
2 @ 2200 = 4400
21200# ÷ 166 = say 130# per ft

Lower Lateral Bracing – 50# per lin ft.
Upper Lateral Bracing, Sway Bracing + Portals say 130# per lin ft.
Handrails 2 @ 60# = 120# per ft.

Metal in Bridge
2 lines of Handrails 2 @ 60 (30) = 120 – 6
Stringers 235
Floor Beams 130
Lower Lateral Bracing 50
Upper Lateral etc 130
Trusses assumed 880 750
Shoes 50
~~1595~~
130
1465

Dead Load
Roadway 100 × 20 = 2000
curbs say 250
2250
Live Load 800 × 1.25 = 1000
1000
Uniform load 100 × 5' = 500
2500
4750#
1595
6345

3172# per truss
assumed truss wt.

metal in one span
Void ~~1595~~ 1535 × 166 = 265,000# (254,00) or say 120 tons (110 ton)

4 spans @ 480 tons. From estimate of Tategahana weight of truss = 930# per ft.
930 × 166²/220² =

1080 × 3172/4800 = 720# per ft call this 750#

Metal in one span 1465 × 166 = 243000# or 108 tons
× 4
428 call this 430 tons

Distance between truss bearings = 2'0".
Top of pier similar to Electric Ry Bridge.

Piers

coping – 25 × 5.5 = 137.5
5.5 dia = 23.7
161.2

shaft 4500
4661 @ 150 = 700.000

volume 4661 ÷ 216 = 21.6 立坪

shaft top 25 × 4.5 = 112.5
4.5' dia 15.9
128.4

bottom 25 × 7.5 = 187.5
7.5' dia 44.2
231.7
360.1 ÷ 2 = 180.0

volume = 180 × 30 = 5400
less 30 × 30 = 900
4500

Base 12 × 20 × 37 = 8900 @ 150 = 1335.000
41.0 立坪
Excavation say 60 立坪

superimposed load 1050.000
shaft 700.000
base 1335.000
3,085.000

Abutment 34 × 37/44 = 28.5 2 @ 28.5 = 57.0 立坪

Unit P = 3085.000 / 12 × 37 = 6950 #/□'
3.10 tons/□'

Excavation say 110 立坪

woodblock pavement 20 × 676 / 36 = 376 坪

Concrete in Floor slab.
slab 0.5 × 20 = 10.0
fillet say .75
curbs 1.25
12.0 × 676' = 8100 cubic ft or 375 立坪

Steel in Floor slab. 19 tons

Estimate of Cost

1.	Structural Steel	430 tons	@ 360.00	=	154800.00
2.	Concrete in Floor Slab.	37.5 立坪	@ 250.00	=	9375.00
3.	Reinforcing Steel	19 tons	@ 200.00	=	3800.00
4	Woodblock Pavement	376 坪	@ 40.00		15040.00
5	Concrete in shaft + coping of piers	65 立坪	@ 300.00		19500.00
6	Concrete in bases of piers	123 立坪	@ 250.00		30750.00
7	Excavation for Pier bases	180 立坪	@ 400.00		72000.00
8	Concrete in abutments	57 立坪	@ 220.00		12540.00
9	Excavation in abutments	110 立坪	@ 35.00		3850.00
					321655.00
					12000
					333,655

183015

20' highway Bridge 333.655.00 58.8%
Elec. Ry Bridge. 234 210.00 41.2%
567 865

459760 × 41.2 = 189.000
× 58.8 = 270.000
460.000
300
160.000

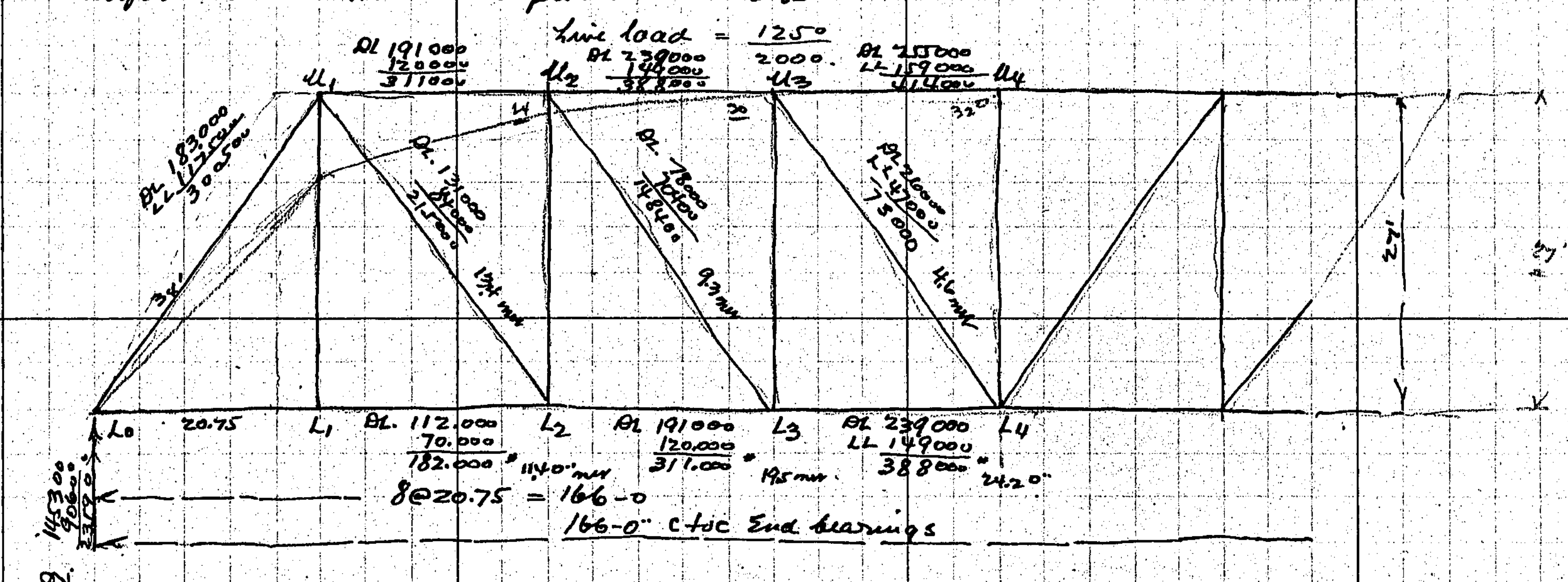
234210
159760
74450.00

Weight of truss -

Uniform dead + Live load per truss = 3250#

Live load = 1250

2000.



$\sec\theta = \frac{34.0}{27} = 1.26$ $\tan\theta = \frac{20.75}{27} = 0.768$

panel concentration Dead Load say 2000 × 20.75 = 41500#

" " Live Load say 1250 × 20.75 = 25900#

			Section					
L0-U1	1 cov. 20 × 7/16	40# [s	34.50	34	34.0 =	3880	3730	
U1-U2	"	33# [s	29.80	"	20.75 =	2100	1925	
U2-U3	"	35# [s	31.58	"	20.75 =	2220	2010	
U3-U4	"	40# [s	33.52	"	20.75 =	2360	2270	
L0-L2	2 [s 15" × 33#	15.56	19.80		41.5 =	2790	2080	
L2-L3	2 [s 15" × 40#	19.38	23.52		20.75 =	1660	1450	
L3-L4	2 [s 15" × 40	19.28 7.50 26.78	32.52	31.08	20.75 =	2290	2200	
U1-L2	4 Ls 5 × 3½ × 3/8, 12 × 3/8 web		18.20		34.0 =	2100	2100	
U2-L3	4 Ls 4 × 3 × 3/8, 12 × 3/8 web	10.67	14.42		34.0 =	1660	1660	35%
U3-L4	4 Ls 4 × 3 × 5/16	5.88	8.36		34.0 =	965	965	
U1-L1	4 Ls 4 × 3 × 5/16	"	8.36		27 =	770	770	
U2-L2	2 [s 9" × 15#	"			27 =	1080	810	
U3-L3	"				27 =	1080	810	
U4-L4	"				13.5 =	540	405	

25490 × 2 = 50,980

40% Details say 20,020

71,000#

(430) per ft

51,000#

35% details 18,000

69,000 ÷ 166 = 415# per ft.

153' 118486#

$775^{\#} \times \frac{5400}{6600} = 620^{\#}$ 6600

$555 \times \frac{54}{66} = 445$ 166' 133.

(40%) 220

665 — 650#

800 530 270

270,000 / 668 = 180,000 / 200,000 ÷ 2240 = 80 / 90 tons @ 350 = 28,000 / 31,500

133' — 85 tons @ 5 = 425 tons

120 tons @ 4 480

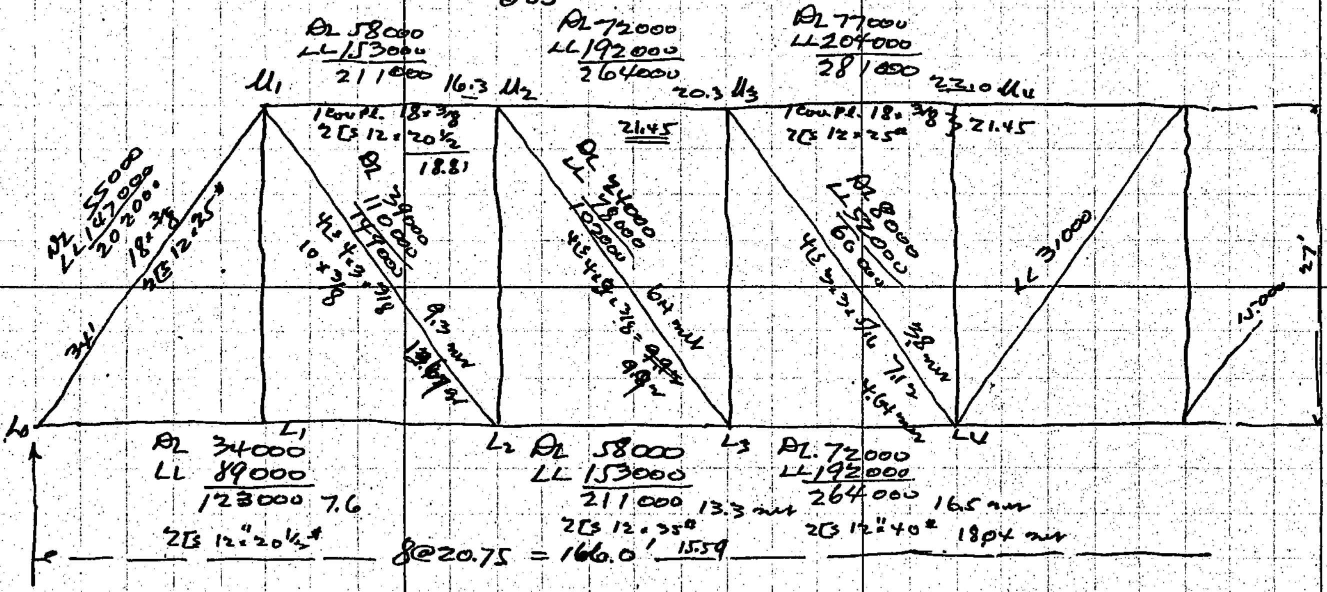
80 tons

Highway Bridge Uyedabashi, Uyeda, Nagonoken.

Single track Electric Ry.

Trusses Live Load on one truss - 1600# Conc. 1600 × 20.75 = 33200

Dead Load 530 / 75 / 605 Conc 605 × 20.75 = 12500



sec θ = 1.26

tan θ = 0.768

Weight of truss	section	3.4			
L0-U1	21.45	× 3.4	× 34.0	=	2480
U1-U2	18.81	"	20.75	=	1330
U2-U3	21.45	"	20.75	=	1520
U3-U4	21.45	"	20.75	=	1520
L0-L2		2- 41#	× 20.75	=	1700
L2-L3		70#	× 20.75	=	1450
L3-L4		80#	20.75	=	1700
U1-L2	13.67	× 3.4	× 34.0	=	1580
U2-L3	9.92	×	× 34.0	=	1140
U3-L4	7.12	× "	× 34.0	=	820
U1-L1	8.36	× "	× 27.0	=	770
U2-L2		30#	× 27.0	=	810
U3-L3		"	× 27.0	=	810
U4-L4		"	× 27.0/2	=	405

18035 × 2 = 36070

38% 13700

49770

49770 ÷ 166 = 300# per lin ft

Stringers	70.0
Floor Beams	60.0
Lower Laterals	30.0
Upper Laterals	50.0
Trusses	600.0
On pier	50.0
	860

860 × 166 = 143000 64 tons

.4

256 tons @ 360 = 92000 yen

Estimate of cost for Saigawa Bashi (犀川橋) between 明科 and 下押野　　1.

This bridge is to be located on the highway between Akashina and Ikeda, Minamiazumigun, naganoken across saigawa seven miles about down ~~stream~~ the city of matsumoto, just below Takasegawa, (高瀬川) ~~stream~~ flow into to Saigawa.

The total length of the crossing is 172 Ken = 1032 ft end to end of bearings. Roadway 18'

arrangement of the span length

1 - 180' span　2 - 60' girder spans encased into concrete

18 - 40' spans of Reinforced Concrete.

Let us make the estimate of cost on this layout.

180
120
720
1020 | 6 ... 170
420
420

18 × 40 = 720

180' Steel span

Loading of Bridge -

Uniform Live Load of kg/cm² = $\frac{100.000}{170+l}$　max 500 kg meter.

where l = span length of bridge in meter

Equivalent formula for ft-lbs unit = $\frac{20480}{170+\frac{l}{3.28}}$ - where l = span length in ft

102.4#/□' for spans under 30 meters or 98.5'

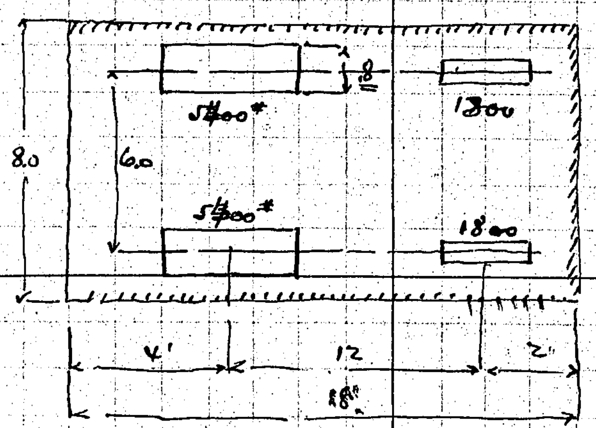
For 180' span = 91#/□'

~~Load Roller concentration 11000 kg (24250#)~~

motor Truck loading　Total weight 14.110# (6400 kg)

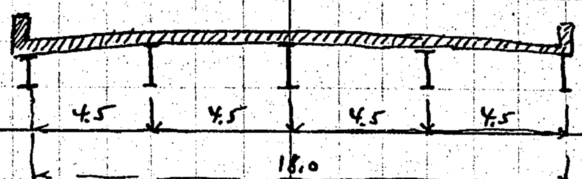
Distribution of wheel concentrations as shown below.

These wheel concentrations modified a little to give the figures round number.



M - 5400 = 10800
2 - 1800 = 3600
14400#

Cross section of Floor



Floor slab - panel length = 4.5'

wheel concentration = 5400#

Impact 1/3 = 1800

7200#

Distribution of this concentration assumed 2'×2'

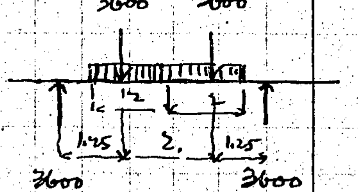
For 1' strip = 7200/2 = 3600#

Live Load moment

m = 3600 × 1.25 = 4500'#

~~Less 3600~~

For continuity of slab 4500 × 0.66 = 2970'#



Revised
see next page.

Dead Load

wood block pavement 3" = 3.6"　18#

cushion sand　7

25#

Slab 6" thick　75

100# per ft

Dead Load moment = 1/10 × 100 × 4.5² = 202

3172'#

Effective depth = $\sqrt{\frac{3172}{94.5}} = \sqrt{33.5} = 5.7"$

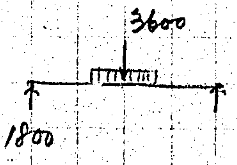
Reinforcement on compression side will reduce unit stress of concrete

Let us make slab 6" thick

Reinforcing bar = $\frac{3172 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = .54 □"

Use 1/2"# bars

1 wheel concentration on one span



For 1' strip $\frac{7200}{2} = 3600$ #

Live Load moment = 1800 × 2.25 = 4050 '#

For continuity of slab take 4050 × 2/3 = 2700 '#

Dead Load for 100 × 4.5² = 200

2900 '#

Effective Depth = $\sqrt{\frac{2900}{104}}$ = 5.3"

make depth of slab 6" with reinforcement on compression side.

Steel Area = $\frac{2900 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = 0.50" use ½" ⌀ bars 4" centers = .590"

Reinforcing bars 1' strip

Longitudinal bars

24 – ½" ⌀ @ .67 × 1.0 = 16.1

2 – ½ @ .67 × 20.0 = 26.8

2 – ½ @ .67 × 22.0 = 29.5

724 #

Lap etc say 5% 36

760 # per lin ft.

Concrete in floor slab.

Floor slab. 18 × 0.5 = 9.0 □'

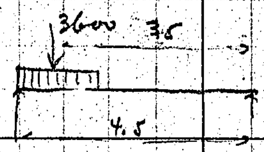
Curbs 2@ 0.5 × 1.25 = 1.25

10.25 □' @150 = 1540 #

10.25 ÷ 216 = .048 坪 per ft.

76 ÷ 0.048 = 1580 # per cubic tsubo

End shear



3600 × $\frac{3.5}{4.5}$ = 2800 # per ft

Dead Load – 100 × 2.25 = 225

3025

Unit shear = $\frac{3025}{14 \times \frac{7}{8} \times 5}$ = 57.5 #/□"

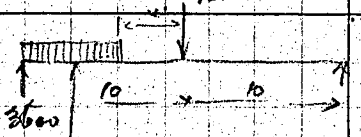
Bond Stress = $\frac{3025}{3 \times 1.57 \times 5}$ = 127 #/□"

Stringer 100 × 4.5 = 450 # m = ⅛ × 500 × 20² = 25000 '#

beam say 50

500

Live Load



m = 3600 × 10 = 36000

Less 3600 × .5 1800

34200

59200

Section modulus required = 59200 × 12 ÷ 16000 = 44.4

Use 15' I beam × 42 # Sm = 58.9

Note: – This uniform load to be added when figuring max moment

5 @ 15" × 42.0 # × 20 = 4200 # per panel

Details say 5@40 = 200

4400 # ÷ 20 = 220 # per lin ft.

Floor Beam span length 21.0'

Roadway pavement + slab 100 #/□'

Stringers say 10

load on Floor beam 110 # × 20 = 2200 # per ft.

Dead Load of Floor Beam say 100

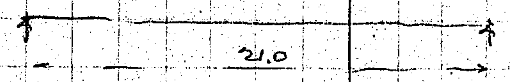
2300 # per ft

Floor Beam Continued -

Dead Load moment

Uniform load = 2300# per ft

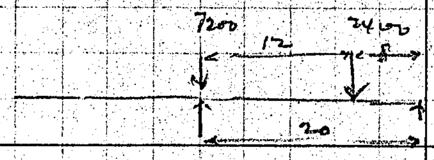
m = 1/8 . 2300 × 21² = 128,000 '#



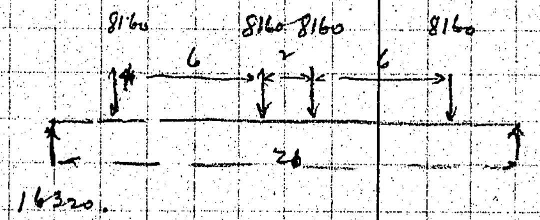
Live Load Moment

wheel concentration

	Rear wheel	Front wheel
	5400	1800
1/3 Impact	1800	600
	7200#	2400#



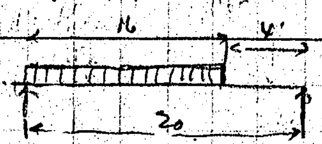
2400 × 8/20 = 960#
7200
8160#



moment = 16320 × 9.5 = 155,000
8160 × 6 = -49,000
Live Load moment 106,000 '#
Dead Load Moment 128,000
234,000#

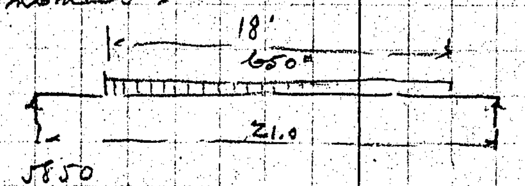
Uniform load 102# per sq ft.

102 × 16 = 1630#



Reaction = 1630 × 8/20 = 650# per ft.

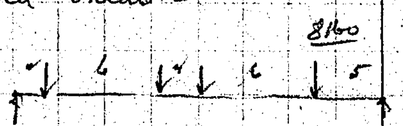
moment -



moment = 5850 × 10.5 = 61500
less 650 × 9 × 9/2 = 26300
35200 '#

Total moment = Live load wheel load 106,000
" " Unif. load 35,200
Dead load 128,000
269,200 '#

End shear -



End shear = 8160 × 48/21 = 18700#
Unif load 650 × 9 = 5850
Dead load shear 2300 × 21/2 = 24200
48750 #

web assumed 27 × 5/16 = 8.44   Unit shear = 48750/8.44 = 5770#/□" -

Gweb = 1.05 □".   No. of Rivets 48750/4810 = 10 Rivets -

Back to back of Ls = 30½   Effective Depth = 2.54 - .14 = 2.40

Stress = 269,200/2.40 = 112000#

AR = 112000/16000 = 7.0 less 1.05 = 5.95 □" net

Use 2Ls 6×3½×3/8" = 6.84 gross or 6.09 □" net

Weight of Intermediate Floor Beam -

1 web	30 × 5/16	@ 31.88	× 20	=	637
Flanges	4Ls 6×3½ × 3/8	@ 11.70	× 20	=	936
Stiffeners	8Ls 3½×3½ × 5/16	@ 7.2	× 2.5	=	144
fillers	8 Pls. 3½ × 3/8	@ 4.46	× 1.9	=	68
End Stiffeners	4Ls 3½ × 3½ × 3/8	@ 8.5	× 2.5	=	85
fillers	4 Pls 3½ × 3/8	@ 4.46	× 1.9	=	34
Shelf Ls	10Ls 4 × 3 × 5/16	@ 7.2	× .67	=	48
	Rivet & variation say				120
					2072

2072 ÷ 20 = 104# per ft of truss
2072 ÷ 21 = 100# per ft of girder

Use same section for End Floor beam.

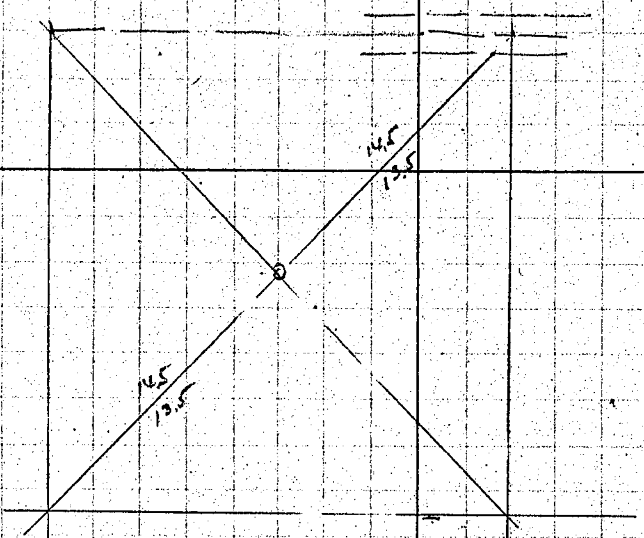
weight of metal in Floor system

Stringers = 220# × 182.5 = 40200#
Floor Beam 8 @ 2070 = 16560
2 @ 2070 = 4140
20700 — 20700
60900 ÷ 182.5 = 334# per lin ft of span

Metal in Handrail - 30# per rail
60# per lin ft of span 60 × 182.5 = say 11000#

Lower Lateral Bracing - 50# per lin ft 50 × 182.5 = 9120#

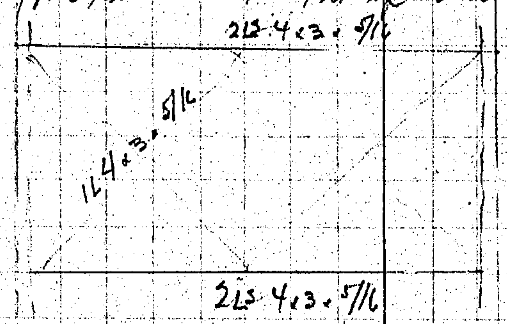
Upper Lateral Bracing -
Diagonals. L



l/r = 150. l = 13.5 × 12 = 162"
r reqd = 162/150 = 1.08

2Ls 4 × 3 × 5/16 @ 7.2 × 13.5 = 195
details say 7.5 × 13.5 = 100
4 @ 295 = 1180#
7 @ 1180 = 8250#

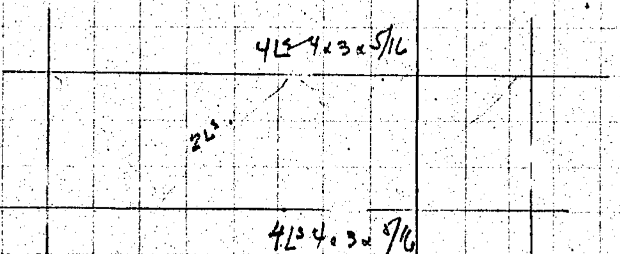
Sway Bracing and Portal -
approximate Height of Sway 10' in average.



Top strut 21 × 12 = 252
r = 252/150 = 1.68 2Ls 4×3×5/16 ⊓ 1.87 about ok

weight of Sway Bracing -
4Ls 4×3×5/16 @ 7.2 × 20 = 576
4Ls 4×3×5/16 @ 7.2 × 13 = 375
Details. 8 @ 25 = 200
1151
6 @ 1151 = 6906#

Portal Bracing -



weight of Portal Bracing -
8Ls 4.3 × 5/16 @ 7.2 × 20 = 1152
8Ls 3.3 × 5/16 @ 6.1 × 7 = 342
Connection pls. 14 @ 25 = 350
Lacing Bars. 5#. 350
2194
say 2200#

Portal 2 @ 2200 = 4400

Summary:- Diagonals. 8250
Sway Bracing 6906
Portal Bracing. 4400
19556 ÷ 182.5 = 107# per lin ft of.
19556 ÷ 180 = 108.5
call this 110# per ft.

Estimate of Cost Saigawa Bashi. 5

180' Truss Span

Roadway - slab -	10.25 @ 150 = 1540		Live Load.	91
Pavement	25# 18 = 450		Assume 1/3 Impact	30
		1990		121#
			121 × 18 = 2180.	
Metal in Bridge				
2 lines of Handrail	60			
Floor system	340		From curve the weight of truss.	
Lower Lateral	50			
upper laterals & sway Br.	110			1400
		560	30%	840#
Live Load.		2180		
Trusses assumed.		840		
		5570#		
Total weight of metal	560			
Trusses	840			
On piers.	45			
	1445#			

Total metal on span = 1445 × 182.5 = 265,000 or 118 tons
add 3% 3.5
121.5 tons.

Live Load on pier.

Roadway 1990
metal 1445
3435
Live Load 2180
5615 ÷ 2 = 2807# per truss

Load on pier per truss 2807 × 90 = 253,000#

Bearing on masonry 253,000 / 500 = 505□"

Use 4" Roller Bearing 4 × 600 = 2400
253000 ÷ 2400 = 105" 5 Rollers 4" - 21 ins. required
2"
23" -
6
29" - Say 30".

8 @ 4" = 20 + 2
make base 22
6
28"

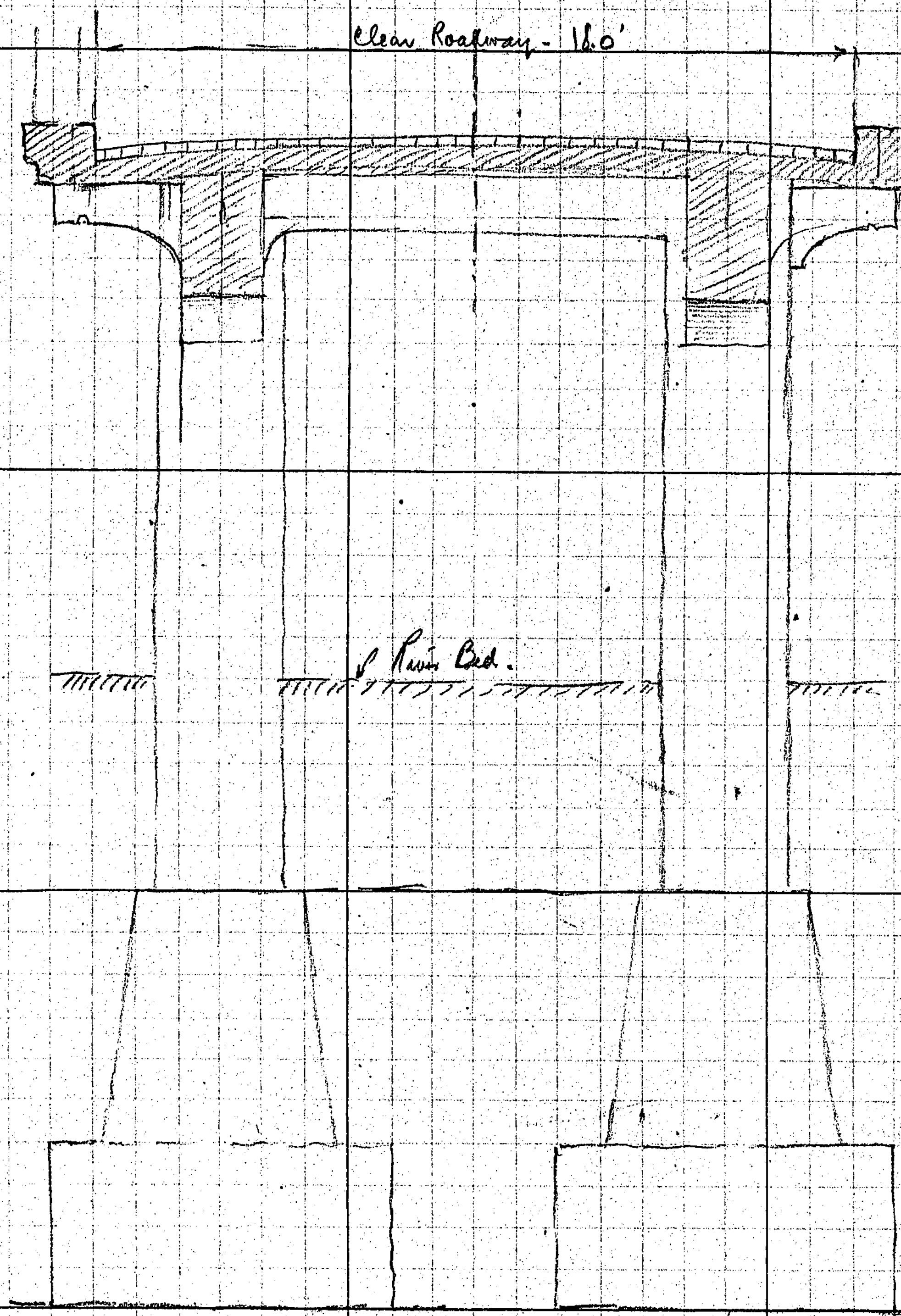
make base 30" × 28" = 840□"

Unit bearing. 253000 ÷ 840 = 300#/□" Ok

3-span continuous 40'. concrete girder.
Loading. 6400 Kg. distribution of wheel loads. 2-5400 = 10800
2-1800 = 3600
14400#.

Uniform load say 100#/□'

Cross section of Bridge.



scale 1/50.

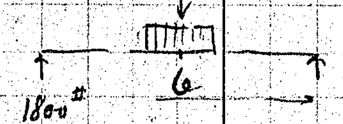
Floor slab.
40' span will be divided into 6-equal panels of 6.67'

Floor Beam 1'-0" wide making clear span of 5.67' or effective span of say 6.0.

wood block pavement	18
cushion	5 8
concrete slab assumed 9" =	112
	135

Dead load moment
$m = \frac{1}{10} \cdot 135 \cdot 6^2 = 486.0$ 1#

Live load moment
concentration 5400
Impact 1/3 1800
7200#
This load assumed distributed over 2.2'
or making 3600# on 1' strip



Approx. max. moment at center
$= 1800 \cdot 3 = 5400$
For continuity $5400 \cdot 0.8 = 4320$
Dead load moment say 490
4810 1#

Effective depth required $= \sqrt{\frac{4810}{100}} = 6.9"$

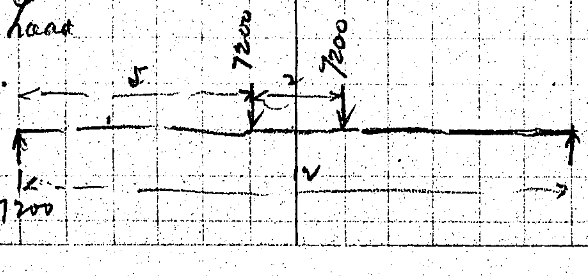
make depth of slab 8" =
Steel Area $= \frac{4810 \cdot 12}{\frac{7}{8} \cdot 7 \cdot 16000} = .590$
1/2"φ bars - 4" centers.

Floor Beam span length 12' between ℄ of main girder.

Dead load -		
	wood block	18
	cushion	8
	Concrete slab 8"	100

Total load - 126#/□' × 6.67 = 840# + 150 = 990# say 1000
Dead load moment $= \frac{1}{10} \cdot 1000 \cdot 12^2 = 14400$ 1#
↳ add weight of

Live Load $m = 7200 \times 5 \times .8 = 28800$
43200 1# for both pos + neg moments



Estimate of cost for Oigawa Bashi –

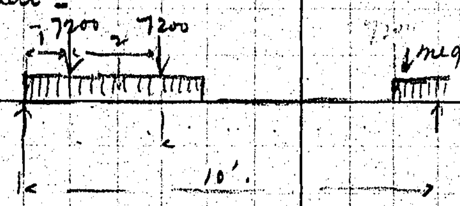
Floor Beam moment 43200[#] as partially fixed beam.

Depth required at connection = $\sqrt{\frac{43200}{400}}$ = 21". make depth at connection – 24" at least.

At center – 20" –

Reinforcing Bars – $\frac{43200 \times 12}{\frac{7}{8} \times 18 \times 16000}$ = 2.06 □" Use – 4 – 7/8" bars = 2.40 □"

End shear –



7200 × 16/10 = say 11500[#]

Dead load say 5000

16500 –

Unit shear at end – $\frac{16500}{\frac{7}{8} \times 18 \times 12}$ = 87[#]/□" – ok.

Reinforcing – 1/2" stirrups 5 or 6 places – all others – 3/8" dia.

Details of cantilever portion as shown on sketch arrangement of reinforcing bars similar to the floor beam.

Main girder – span length 40'. 3 – continuous spans –

Dead load on Bridge – (1/2)

Pavement and slab –	126[#] × 9	=	1132[#]
curb + coping –	1.5 × 2 @ 150	=	450
Cantilever + C.B.	1200[#]/6.67	=	180
Hand rail assumed –			120
			1882[#] per lin ft.
main girder assumed –			600
	700 or more		2482[#] or say 2500[#] per lin ft.

max negative moment = 0.10 × 2500 × 40^2 = 400,000[#]

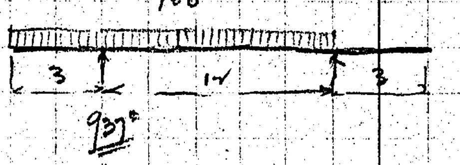
max positive end span = 0.08 × 2500 × 40^2 = 320,000 "

max pos. Center span = 0.025 × 2500 × 40^2 = 100,000 –

Live Load –

say – 100[#] uniform on roadway.

100 × 15 × 7.5/12 = 937[#]



Motor trucks without impact – 14400[#]
distributed over 8 × 18 100[#]/□'

Design – girder for 1000[#] per lin ft of moving live load –

max negative moment = 0.117 × 1000 × 40^2 = 187,000[#]

max pos. End span = 0.10 × 1000 × 40^2 = 160,000 "

max pos. Center span = 0.075 × 1000 × 40^2 = 120,000 "

Summary of moments

	End span +	2nd support –	Center span pos.
Dead Load Moment	320,000	400,000	100,000
Live Load Moment	160,000	187,000	120,000
	480,000[#]	587,000[#]	220,000[#]

Main girder –

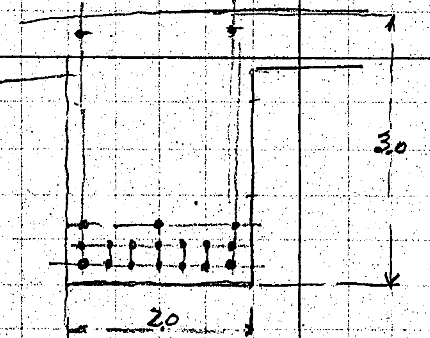
Depth required at support fc = 750%

moment = 587000[#] d = √(587000 / 2×13.4) = √2220 = 47" make depth 60"

Depth at center of span – assumed 3.0 (36")

Reinforcement required = 480.000×12 / (7/8·3½·16000) = 12.90" – Use 17–1"φ = 13.25□"

13.25□ / 24·30 = .184%



End shear – max R 2500
L 1000
3500[#]×20 = 70,000[#] unit shear = 70.000 / (24×56×7/8) = 59.6[#]

Amount of Concrete in span – Floor slab .67·18 = 12.10
curb + coping 6.00
main girder 4×3.33 = 13.3
31.40□×40 = 1266 cub.ft.
Floor B. + Cantilever 16×say 6 = 9.6
1356

1356 ÷ 216 = 6.3 cu.yd. 6.0 yd.

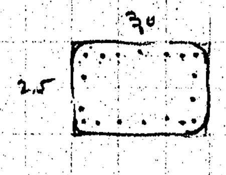
Reinforcing Bars.

Floor slab.	55 – ½"φ @ .67 × 1.0 =	37[#]	
Curb.	12 – ½φ @ .67 × 1.0 =	8[#]	
Transverse.	1 – 3/8 @ .37 × say 25 = say	10[#]	
beam.	240 ÷ 6.67 =	36[#]	
main girder.	17 – 102 × 1.0	102	
		193 × 1.30 = 250[#] per lin.ft.	

250 × 40 = 10,000 ÷ 6.3 = 1590[#] per cu.yd.
10,000 ÷ 2240 = 4.5 tons.

Column load.

3500×40 = 140.000[#]
col. 17×2.5×3×.150 19100
159.100 unit 159100 ÷ 930 = 171[#]/□" O.K.



core area = 6.44 × 144 = 930. 18 – 1"φ bars

Base + Shaft.

Shaft Superimposed load: 159.100

col. 36/12 (16) + 2 = 26. Vol = 26 × 6 = 156 @ 150 = 23,400

Base. 8×8×4 @ .150 = 30700
213,200[#] ÷ 64 = 33.30[#]/□' 1.5 tons/□'

Material for Base shaft + Columns
Concrete

Base =	8 x 8 x 4	=	256 cu ft
Shaft =			156
Col.	2.5 x 3.0 x 17	=	127.5

539.5 x 2 = say 1080 ÷ 216 ÷ 216 = 5.0 cubic tsubo per bent.

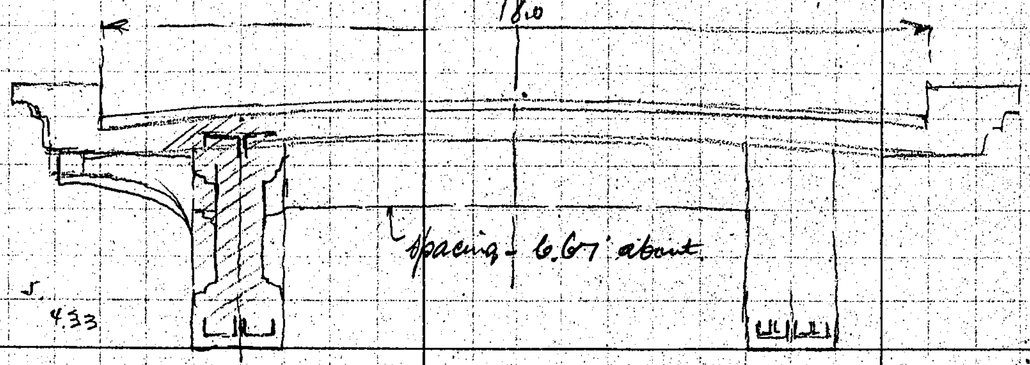
Reinforcing bars. 36' - 1" @ 2.67 x 25 = 2400 #
misc. say 600
3000 # per bent

Excavation say - 8 x 8 x 15 = 960   2 @ 960 = 1920 ÷ 216 = say 9. tsubo

2 - 60' Steel girder spans encased into concrete

Cross section of Bridge.

Assume Floor slab, cross beam etc.
same as for concrete girder span.



Main girder -

Pavement etc	126 x 9	=	1132
curb + coping.			450
Cantilever + Cross Beam			180
Hand rail assumed			120
			1882
Main girder assumed	4.33 x 2 @ 150 =		1300
			3182
Live load say			1000
			4182 # per lin ft

Total moment = 1/8 x 4182 x 60² = 1885,000 '#

Steel Area required for Tension - $\frac{1885,000}{4.25}$ = 443,000 #   SR = $\frac{443,000}{16000}$ = 27.7 □" net

Concrete in span.

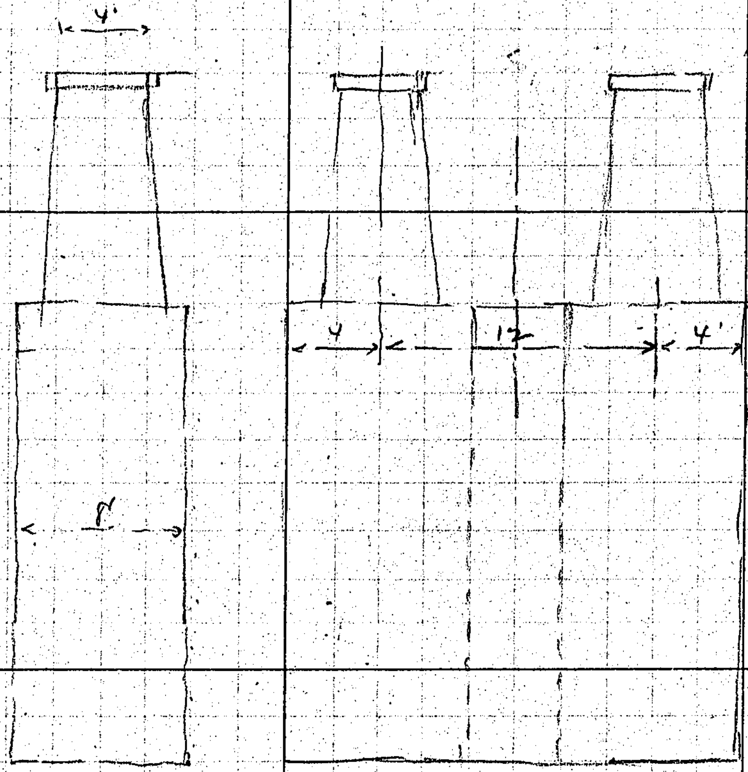
Floor slab.	.67 x 18	=	12.10
curb + coping			6.00
main girder	4 x 4.33	=	17.30

35.40 □ x 60 = 2120
Floor Beam + cantilever 16 x 9 = 144
2264

2264 ÷ 216 = 10.5 cubic tsubo
Reinforcing bars. say 10.5 @ 1500 = 15,750 # or 7.0 tons.

Bridge Pier for 60' girder span.

load on pier - 4182 × 60 = 252000 #



bearing Area = 252,000 / 400 = 630 □"

630 / 24 =

2 @ 252,000 = 504000

Shaft

4' top Area = 12.6

6' bottom Area = 28.3

40.9 ÷ 2 = 20.45 × 10 = 204.5

2 @ 204.5 = 409.0

Base. 8 × 20 × 20 = 3200

3609 ÷ 216 = 16.7 cubic tsubo

Excavation say 20 tsubo

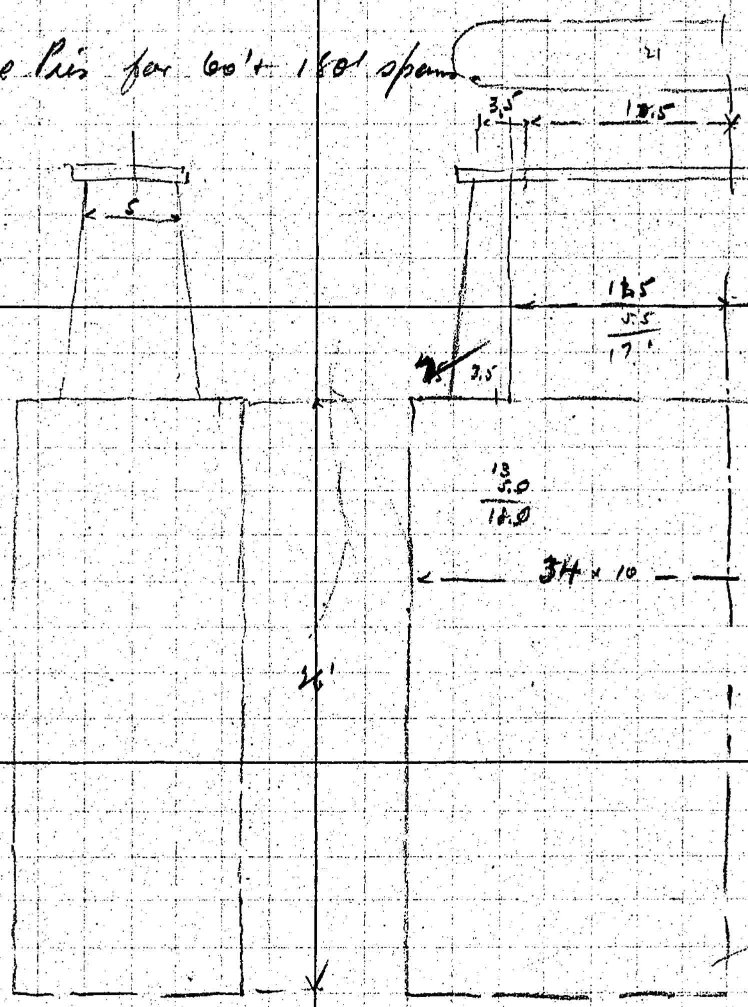
weight of pier - 3609 @ 150 = 541,000

superimposed load - 504000

1,045,000 #

Bearing P = 1045000 / 8 × 20 = 6540 or 2.92 tons -

Bridge Pier for 60' + 180' span



Concrete in pier -

5 × 23 = 115 / 20

13.5

7 × 23 = 161 / 34

199

334 ÷ 2 = 167

volume of shaft = 167 × 10 / 216 = 7.7 cubic tsubo

Base. 10 × 34 × 26 = 8840 / 216 = 41.0 tsubo -

48.7

say 50 tsubo -

Excavation - say 50 T -

Weight of Pier, shaft 1670 @ 150

Base 8840 @

1,051,0 @ 150 = 1,580,000

Superimposed load - 1/2 girder span 252,000

1/2 truss span - 506,000

253.

2,338,000

Unit Bearing = 2,338,000 / 10 × 34 = 6870 # or 3.07 tons/□'

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Estimation of Cost for Shigawa Bridge

	Structural Steel in 180' span say		125 tons.	@ 370	=	46200.00	
180	Floor slab .05 × 182.5		9.1 tsubo	@ 300	=	2730.00	100√30
	Reinforcing bars		6.5 tons	@ 200	=	1300.00	
	Bridge Pier	2@50	100 tsubo	@ 200	=	20000.00	
	Excavation	2@50	100 〃	@ 300	=	30000.00	
	misc. say					3000	
60	2 Piers for 60' girder span	2@20	40 〃	@ 200	=	8000.00	
	Excavation		40 〃	@ 200	=	8000.00	
	Concrete in 60' spans	2@10.5	21. 〃	@ 300	=	6300.00	
	Reinforcing bars	2@7.0 tons	14. tons	@ 200	=	2800.00	
	40' spans. Concrete	18@6.3	113.4 tsubo	@ 300	=	34000.00	
	〃 〃 Steel	18@4.5	81. tons	@ 200	=	16200.00	
	〃 Bent Concrete	18@5.0	90. tsubo	@ 250	=	22500.00	
	Steel	18@1.5	27 tons	@ 200	=	5400.00	
	Excavation	18@9	162 tsubo	@ 70	=	11340.00	
	Misc Structural Steel		15 tons	@ 250	=	3750.00	
	Handrail		1700 ft	@ 2.50	=	4250.00	
	Abutment					2000.00	
	Pavement	170×3 =	510 坪	@ 350	=	17900.00	
						245670.00	
			variation			24567	
						270237.00	
			Misc Expense. say			4763	
						275000.00	

$\frac{275000}{510}$ = 540 円/坪

This bridge spans between Hokari and Makizato across Chikuma River.
Layout as shown on sketch with 2 spans of 70'
1 span of 180
3 spans of 60'.
All spans will be through with 18' Roadway.

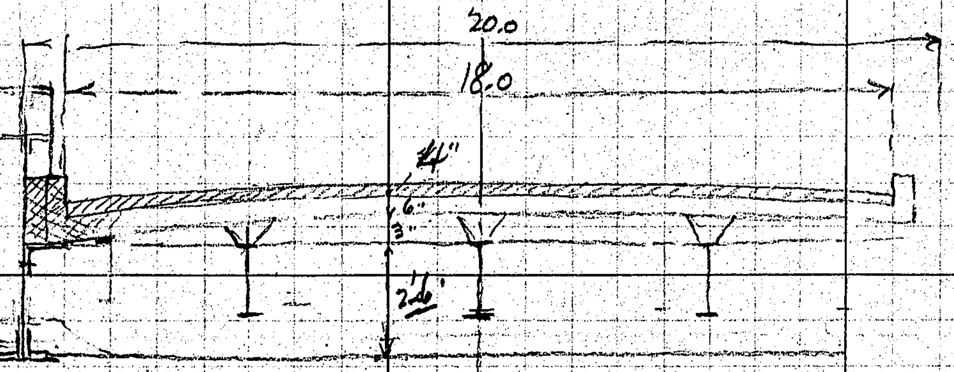
140
180
180
500 ÷ 6 = 84
48
20
18

Loading of bridge –
Uniform load = $\frac{20480}{170 + \frac{l}{3.28}}$ where l = span length in ft

no impact considered.
motor Truck loading. 2 – 5400# + 2 – 1800#
Sketch of distribution as shown on page No 1.

180' span
Estimate same as for 180' span as shown on page No 1.
Summary
Structural steel in 180' span 125 tons.
Floor slab. 0.05 × 182.5 9.1 Tsubo
Reinforcing bars 6.5 tons.

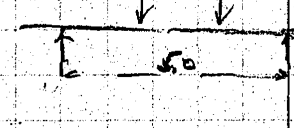
70' Steel girder span



Crown to Bottom of Floor Beam

2'-6
6
4
6
2'-6
6'-4"

2'-6"
1'-1
3'-7"



Floor slab. Span length = 4.75 5.0

wheel concentration 5400
Impact 1/3 1800
7200

Distributed over 2'
For 1' strip $\frac{7200}{2}$ = 3600#

Live Load moment = 3600 × 1.5 = 5400"#
For continuity 5400 × 0.67 = 3620

Dead load in
Floor. 88#
Pavement 25 — 3" wood block + 1/2 cushion
113#

Dead Load Moment = $\frac{1}{10}$ × 113 × 5² = 282
Live Load moment 3620
3902"

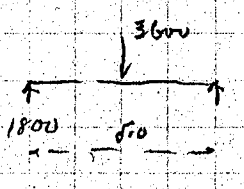
Effective D = $\sqrt{\frac{3902}{100}}$ = 6.25"

make slab – 7½" with compressive steel

Steel Area = $\frac{3902 \times 12}{\frac{7}{8} \cdot 6 \times 16000}$ = .557 □"

use ½" □ bars – 4" centers.

1 wheel load at center of span



Live Load Moment = 1800 × 2.5 = 4500
For continuity 4500 × 2/3 = 3000
Dead Load moment 280
3280

Effective Depth $\sqrt{\frac{3280}{100}}$ = 5.65"

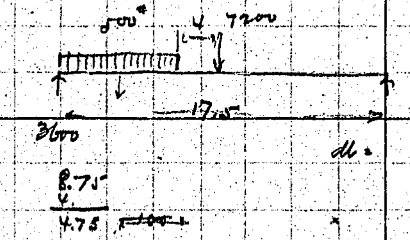
make depth 6½" with steel on comp side

Steel Area = $\frac{3280 \times 12}{\frac{7}{8} \times 5.5 \times 16000}$ = .511 use ½" □ bars – 4" centers.

Stringer span length 17.5

Dead load Floor + pavement 113 × 5 = 565 $m = \frac{1}{8} \times 615 \times 17.5^2 = 23600^{\#}$
Beam say 50
615

Live Load. moment at center 3600 × 8.75 = 31500 #



4.75 × 500 = 2380
$2380 \times \frac{2.38}{17.5} = 324^{\#}$ m = 324 × 8.75 = 2830
3433 0 # 34330
23600
Total 57930

$Sm = \frac{57930 \times 12}{16000} = 43.4$

Use 15" × 42.0# I beams. Sm = 58.9

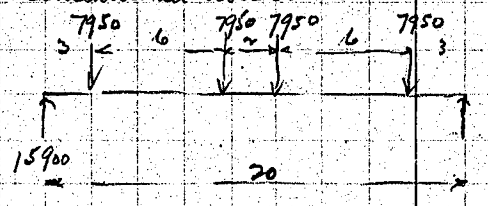
Floor Beam span length 20' Spacing - 17.5 about

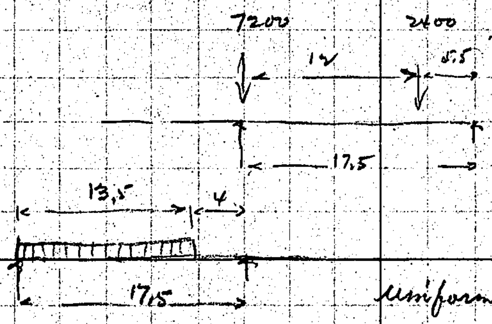
Dead Load Roadway + Pavement + slab 113
Stringers say 10
123 × 17.5 = 2150
Dead Load of Floor Beam say 100
2250

$m = \frac{1}{8} \times 2250 \times 20^2 = 112,500^{\#}$

Live Load

Concentrated load.

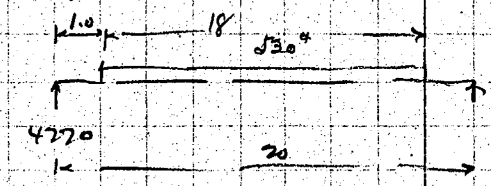




$2400 \times \frac{5.5}{17.5} = 750$
7200
7950

moment = 15900 × 9 = 143000
7950 × 6 = - 47700
95300

Uniform load

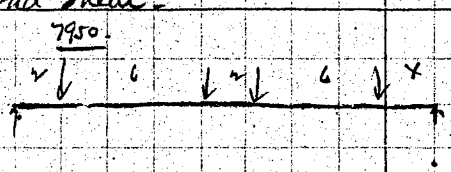


uniform load $102 \times \frac{13.5^2}{2 \times 17.5} = 530^{\#}$ per ft.

moment 4770 × 10 = 47700
Less $530 \times \frac{9^2}{2}$ = 21500
26200 #

Total moment Live Load Conc. 95300
Unif. 26200
Dead Load - m 112500
234000 #

End Shear.



End shear $7950 \times \frac{44}{20}$ = 17500
Unif. 530 × 9 = 4800
Dead Load shear say $\frac{2250}{2} \times 20$ = 22500
44800 #

Web assumed 30 × 5/16 = 9.370" Unit shear = $\frac{44800}{9.37}$ = 4800 #/□"

Backs to back of Ls 30½" Effective D = 2.54 - .14 = 2.40 1/8 Web = 1.175
Stress = $\frac{234000}{2.40}$ = 97500 SR = 6.10 - 1.18 = 4.920" - net

use 2Ls 5 × 3 × 3/8 5.72 gross 4.970" net weight of Inter-mediate Floor Beam

1 web		30 × 9/16	@ 31.88	× 20	=	637
Flanges	4L^s	6 × 3 × 3/8	@ 9.8	× 20	=	785
Stiffeners	8L^s	3½ × 3½ × 9/16	@ 7.2	× 2.5	=	144
Fillers	8Pl^s	3½ × 3/8	@ 4.46	× 1.9	=	68
End stiffs	4L^s	3½ × 3½ × 3/8	@ 8.5	× 2.5	=	85
Fillers	4Pl^s	3½ × 3/8	@ 4.46	× 1.9	=	34
Shelf L^s	18L^s	4 × 3 × 5/16	@ 7.2	× .67	=	39
			Rivits + variation			100
						1892#

1892 ÷ 20 = 94.6 per ft of Beam

1892 ÷ 17.5 = 108.# per ft of main girder

Use same section for End Floor Beam.

weight of metal in Floor system
- Stringers 3@50 = 150# × 70 = 10,500
- Floor Beam 3@ 1900 = 5700
- 2@ 1900 = 3800
- 20,000# ÷ 70 = 286# per lin ft of span.

metal in Handrail. (if used). 30# per ft.
For 2 lines 60# per lin ft of span.

Lower Lateral Bracing. 50# per lin ft of span.

Summary
- metal in Floor system 286
- Handrails. 60
- Lower Lateral. 50
- 396# say 400#.

Main girder.
Dead Load
- Pavement + slab, say 113# × 18 = 2040
- Curbs 375
- metal in Floor system &c 400
- 2815
- Live Load Assumed 2180
- Girder assumed. 400
- 5395 ÷ 2 = 2700

From Curve the weight of girder = 270#
For 2 girders 2@ 270 = 540# per ft.

Metal in 70' girder spans.
- Floor system Complete 286
- Handrails. 60
- Lower Lateral 50
- Girder. 540
- On pier say 40
- 976 × 70 = 68300# 30.5 tons
- Add for variation. 3.5
- 34.0 tons.

60' girder spans From curve weight of girder 230.
- Floor system Complete 286
- Handrails. 60
- Laterals. 50
- Girder. 460
- On pier say 40
- 896 × 60 = 54000# or 24.2 / 2.4 / 26.6 say 27 tons.

Estimate of Cost for Highway Bridge Hokaribashi on Nagano – Matsumoto Line. 15

Structural steel in Bridge.

180' span		125	tons
70' spans	2@ 35	70	tons
60' spans	3@ 27	71	
		266	tons

Concrete in Floor.

Floor	.55 × 18 =	9.9
curbs		2.5
misc fillet		.6
		13.0 ÷ 216 = 0.06 t7 per ft

2-70 = 140
3-60 = 180
320

Concrete in Floor slab = 0.06 × 320 = 19.2 Tsubo

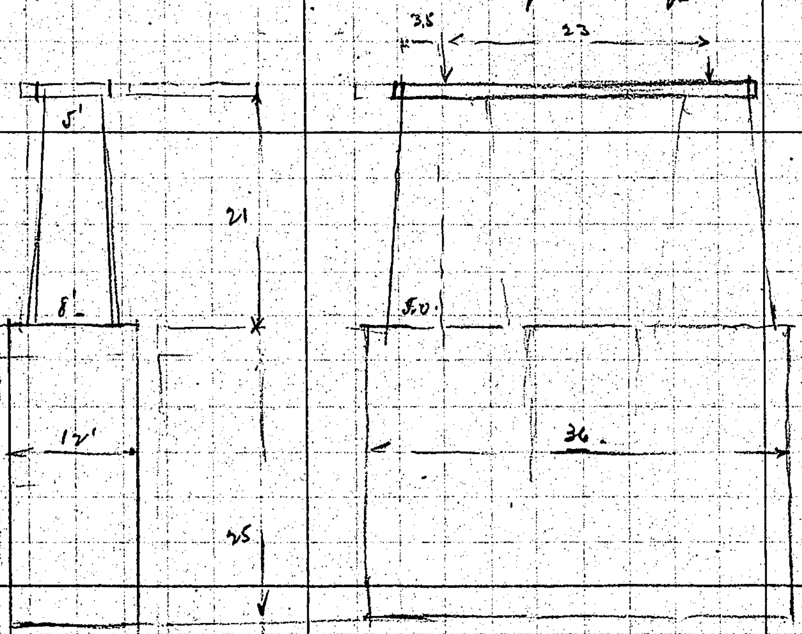
Reinforcing steel 1600 × 19.2 = 30800 = say 14 tons.

Summary

	Concrete	Reinf.
180 span	9.1	6.5
70+60 span	19.2	14.0
	28.3 tsubo.	20.5 tons

Design of substructure.

Boring not made. Ref. the most possible soil will be sand and gravel on top and the bed rock will be reached in a very short distance. Assume soil sand & gravel.



Shaft
5 × 23 = 115
5'φ 20
135 135
8 × 23 = 184
8'φ 50
234 234
369 ÷ 2 = 185

vol = 185 × 21 = 3880.0 @150 = 582,000"
3880 ÷ 216 = 18.00

Base. 12 × 36 × 25 = 10,800. @150 = 1620,000
2,202,000

10800 ÷ 216 = 50 t7

Total vol. Shaft 18
Base 50
68

Excavation say 60 t7

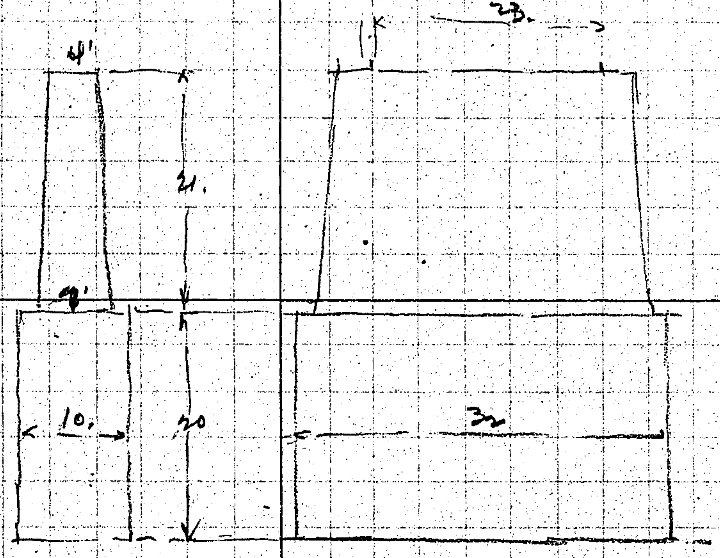
Bearing Pressure on soil – Superimposed Load 180' 506000
70' span say 2 - 5600 × 70/2 196000

702,000
2202,000
2904,000

Unit pressure = 2904.000 / 12 × 36 = 6700 #/' = 3.06 tons

Estimate of Cost for Highway Bridge Hokaribashi - on Nagano-Matsumoto Line - 16.

Bridge Pier Between 70' spans.
Similar Pier to be used for 60' spans. depth of base - 20' Assumed.



Shaft 4×23 = 92
4'dia = 12
104
7×23 = 161
7'dia. 38
199
303÷2 = 151.5

vol. = 151.5×21 = 3180. @150 = 477,000.
3180÷216 = 15.0 cu tsubo
Base. 32×10×20 = 6400 @150 = 960.000
1,437,000
6400÷216 = 30 tsubo

Total vol. Shaft 15
base. 30
45 tsubo. Excavation say - 40 tsubo -

Estimate of Cost.

Structural Steel in Bridge -	266 tons	@ 400.	=	106400	
Concrete in Floor	19.2 t3	@ ~~300~~	=	5750	
Reinforcing Steel -	14.0 tons	@ 240	=	3360	
2 Piers @ 68	136 t3	@ 200	=	27200	
Excavation 2@60 =	120.0 "	@ ~~200~~	=	~~34,000~~ 18,000	96200
3 piers @ (45) - 30	~~135~~ 90 t3	@ 200	=	~~27,000~~	4000
Excavation 3@~~40~~ 25	~~120.0~~ 75	@ ~~150~~ 100	=	~~18000~~ 7500	100200
Pavement 3×83.2	250.0	@ 40	=	25000.	
2 abutments @ 5000				10000	
				~~246710~~ 229210	
				call this ~~250000~~ 230,000	

$\frac{230{,}000}{250}$ = 920 ~~1000~~ B-17

Note { Piers estimated on safe side. Will reduce cost in final design.
If borings made, layout and the design of piers will be revised.

Layout of spans
1- 47.5' concrete girder span. fixed one end.
1- 220' steel truss span
2- 47.5' concrete girder spans. continuous.
Roadway 18' wide concrete slab. woodblocks pavement 3 1/2"

220' span
This will be similar to Tategahana Bridge.
Structural steel in one span say 150 tons.

Concrete in slab = 0.05 × 223 = 11.2 $^{c}/_{y}$
Reinf. in concrete slab say 17,600# or say 8. tons.

Concrete Girder Span.
See page 6 for cross section of structure
Divide the span into 8 panels of 6' each
Floor slab + cross beams same as for Saigawa Bashi - see pp. 6-7

Main girder Span length 47.5 2 continuous spans.
Dead load on bridge - ½ width

Pavement + slab	126 × 9 =	1132
Curb + coping.	1.5 × 2 @150 =	450
Cantilever + Cross Beam		180
Handrail assumed		120
		1882
Main girder assumed.	2 × 3.33 @150	1000
		2882 call this 2900#

Max neg moment at support = 0.125 × 2900 × 47.5² = 820000 '#
" pos. moment of endspan = 0.070 × 2900 × 47.5² = 458,000

Live Load 1000# per lin ft per girder

Max neg moment at support = 0.125 × 1000 × 47.5² = 282,000
Max pos. " endspan = 0.095 × 1000 × 47.5² = 214,000

Summary of moment

	max neg	max Pos
Dead Load	820,000	458,000
Live Load	282,000	214,000
	1,102,000 '#	672,000 '#

Effective depth required for the stress of fs=16000#/" + fc=750#/" $\frac{M}{bd^2}$ = 133

$d=\sqrt{\frac{1,102,000}{2\times133}} = \sqrt{4125} = 64"$. make depth 5.5

Depth at center of span assumed 3.5' = 42" - 5 = 37".

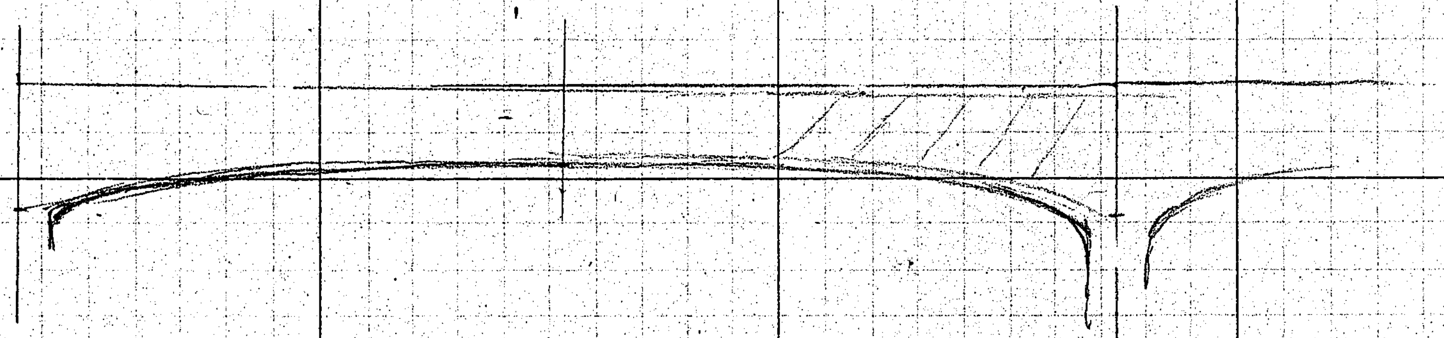
Reinforcing Bars = $\frac{672,000 \times 12}{\frac{7}{8} \times 37 \times 16000}$ = 15.6 □"
use 16 bars of 1 1/8" = 15.9 □"

220
143
363 | 6 | 182
36 | 6 | 6

Estimate of Cost for Koizumi Bashi - on Saigawa line between Nagano + Matsumoto 18

Reinforcement at negative side -

Steel = $\frac{1102,000 \times 12}{\frac{7}{8} \times 60 \times 16000}$ = 15.8 □" Use 16 - 1⅛" bars = 15.9 □" -



Amount of concrete in span

Floor slab -	.67 × 18	=	12.10	
curb + coping			6.00	
main girder	4.25 × 4	=	17.00	
			35.10 × 47.5 =	1670
Floor Beam + Cantilever	16 @ 8			128
				1798

1798 ÷ 216 = 8.33 3 @ 8.33 = 25.0
5.0
30.0 tsubo

Reinforcing Bars.

Floor slab	55 - ½" @ .67	× 1.0	=	37.
Curb	12 - 1½" @ .67	× 1.0	=	8
Transverse bars	1 - 3/8" @ .38	× 25	=	10
Cross Beam				36
main girder	same			140
				231 × 1.3 = 300# per ft

300 × 47.5 = 14,250 or 6.35
1740# per cubic tsubo 3 @ 6.35 = 19.1
1.5
20.6 tons

Column + Base same as for Saigawa Bridge see pp 8

Concrete in col + Base - 5.0 tsubo per bent
Reinf bars in col. say 1.5 tons per bent

Piers assumed same as for Hokari Bashi. see pp. 15

Estimate of Cost

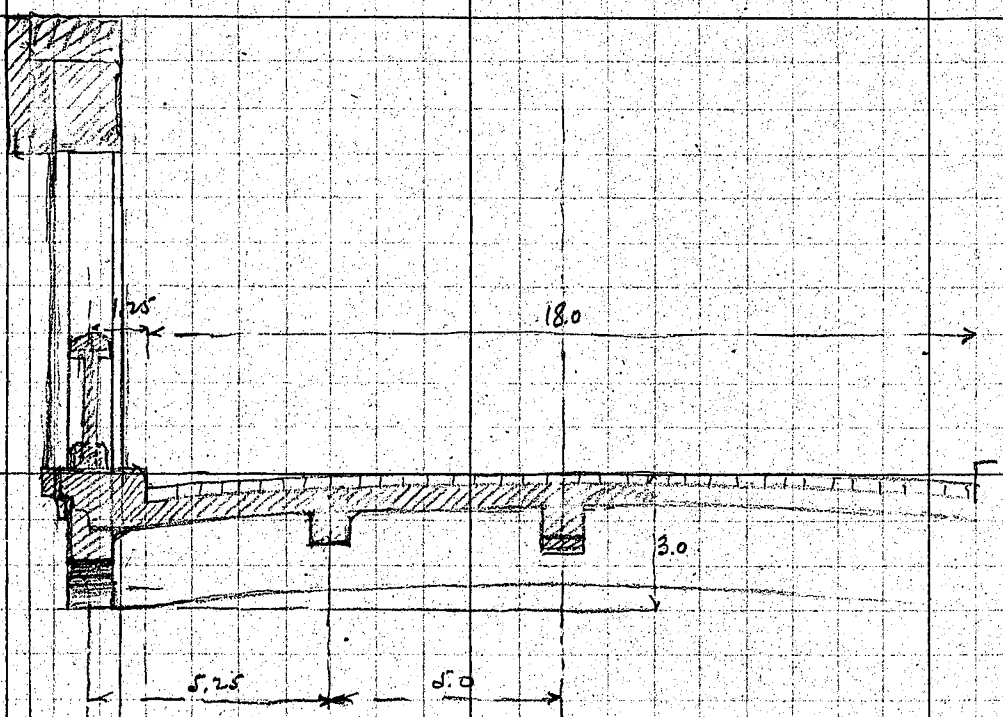
structural Steel		150 tons	@ 400	=	60,000
Concrete in slab		11.2 ㎥	@ 300	=	3360
Reinf.		8.0 tons	@ 240	=	1920
concrete span	Concrete	30.0 tsubo	@ 320	=	9600
	Steel	20.6 tons	@ 240	=	4940
Abutments - 2 @ 3000				=	6000
Concrete Handrail -		285 ft	@ 2.50	=	712
Pavement -	3.61	183 ㎥	@ 40.00	=	7320
2 Piers - concrete	2@50 -	100 ㎥	@ 200.00	=	20000
Excavation -	2@50 -	100 ㎥	@ 200.00	=	20000
Excavation for Bent		10 ㎥	@ 50.00	=	500
					134432 -
variation say					13400
					147832

call this ¥ 150,000.00

Estimate of Cost for Yasuga Bashi (安賀橋) on Saigawa Line between Nagano & Matsumoto 19

Total length of this bridge — 280' = 46.5 Ken
width of Roadway 18 Concrete slab with wood block pavement 3寸
Span will be Reinforced Concrete Arch spans of 132' with rise of 30'
making the bridge half through
Boring not made, but solid bed rock will be found in shallow depth.

Cross section of Bridge.



Concrete slab. span length 5.0 center to center of beams or 4.0 clear between faces of beams. Effective span length say 4.5

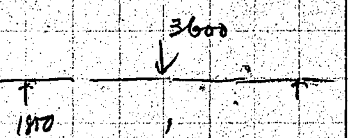
Dead Load

pavement wood block 3寸	18
cushion say	7
concrete slab assumed 6"	75
	100#

$m = \frac{1}{10} \times 100 \times 4.5^2 = 203.^{1\#}$

Live Load conc.	5400
Impact	1800
	7200#

For 1' strip ~~say~~ 3600 1#



moment = 1800 × 2.5 = 4500
For continuity of slab 4500 × 0.8 = 3600
203
3803

Effective $d = \sqrt{\frac{3803}{100}} = \sqrt{38.} = 6.25$"

make slab 7" deep with reinf on compression side.

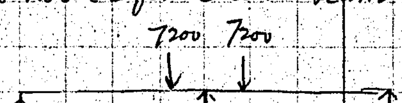
Reinforcing $= \frac{3803 \times 12}{\frac{7}{8} \times 6 \times 16000} = 0.545$ □"

use ½"⌀ bars 4" centers

Intermediate Beam
span length 10.0
Dead load 100 × 5.0 = 500
Beam assumed 150
650#

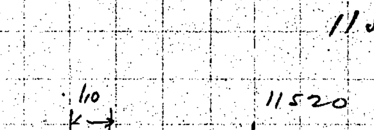
$m = \frac{1}{10} \times 650 \times 10^2 = 6500$ 1#

Live Load for center beam



Transverse.

$7200 \times \frac{4}{5} = 5760$
5760
11520#

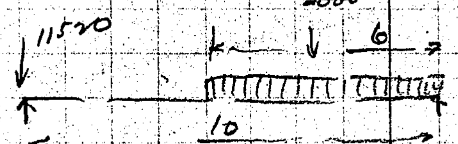


moment = 5760 × 5 = 28800 1#
For continuity 28800 × ⅔ = 19200 1#
Dead load m = 6500
Total 25700 1#

depth $= \sqrt{\frac{25700}{100}} = 16.5$"
2.
18.5

make depth 19" or 12" under slab.

For Shear
uniform load 100 × 5 = 500#



11520
900
12420
3250
15670
LL
DL

Reinforcing bars $\frac{25700 \times 12}{\frac{7}{8} \times 17 \times 16000} = 1.30$"

use 4 - ¾"⌀ bars = 1.30 O.K.

unit shear $= \frac{15670}{12 \times 17 \times \frac{7}{8}} = 88$#/□"
40

Shear to be carried by Stirrups - 48#/□" at end.
⅜" Stirrups $\frac{2200}{12 \times 48}$ = 4" centers at ends required

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Floor Beam span length 20.5 spacing 10.0

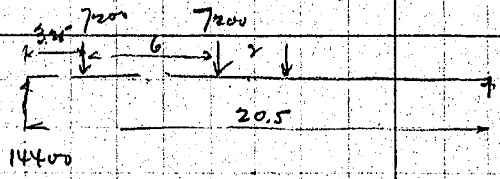
Dead load Floor 500
Beam 150
650 ÷ 5 = 130#

load on Floor Beam = 130 × 10 = 1300
Beam assumed 1.0 × 2.1 @ 150 = 320
1620#

m = 1/8 × 1620 × 20.5² = 85,000 1#

Live Load Concentration 7200# with impact.

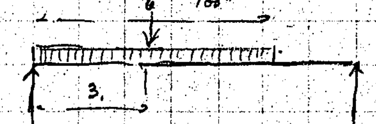
moment = 14400 × 9.25 = 133000
7200 × 6 = -43200
89800 1#



m = 1/8 × 180# × 20.5² = 9500

99300
DL m 85000
184300 1#

uniform load

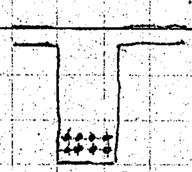


600 × 3/10 = 180

try 2'-6" concrete 30" depth say 28

Steel Area = 184300 × 12 / (7/8 × 28 × 16000) = 5.65

use 8 - 1"□ bars = 6.250" -



Max. End shear

Dead load 1620 × 10.25 = 16600
Fascia girder say 2400
19000# — 19000

Live Load Conc 7200 × 4.5/20.5 = 15800#

Unif 180 × 9 = 1620
17420
36420#

Unit shear = 36420 / (7/8 × 12 × 28) = 124#/□"
40
84

1/2" stirrup. 4400 / (84 × 12) = 4.35" spacing at ends.

Summary of Dead Load

Pavement say .25 × 18 = 4.50
Concrete Slab .58 × 18 = 10.45
beam 1.0 × 1.0 3.00
Fascia 2 @ 3.25 6.50
19.95 say 20 @ 150 = 3000
Floor Beam 2.25 × 3.0 @ 20.0 @ 150 / 10 675
Handrail 300
4425# per lin ft.
Hanger say. 175
4600# ÷ 2 = 2300 per Rib.
Arch Rib assumed 4 × 3 @ 150 - 900
3200

Dead Load stress Span 135 Rise 20
Rise Ratio 20 ÷ 135 = .148

16 × 144 = 2300

Crown thrust 0.95 × 3200 × 135 = 410,000# ÷ [illegible] = 1510
Normal thrust at sp. 1.14 × 3200 × 135 = 492,000

Dead Load moment assumed = 410,000 × .5 = 182,000 1# crown
492,000 × .5 = 246,000 springing

Live Load say 1750# for crown stress + 1000# for springing stress –

Rise Ratio – .148 Ratio crown thickness to springing = 3/6 or reverse 6/3 = 2.0

pos. moment at crown 0.00485 × 1750 × 135² = 155,000 1#

Thrust 0.435 × 1750 × 135 = 103,000 #

Stress at springing

negative moment at sp. 0.02335 × 1000 × 135² = 407,000 1#

Thrust Ts. 0.40 × 1000 × 135 = 54,000 #

" Tc 0.24 × 1000 × 135 = 32400 #

14 – 1" bars @ .78 = 10.9 on one side –

$\frac{21.8 \times 15}{144}$ = 2.27 □' equiv.

2.27 × 1.0² = 2.27

4.5

6.77

Steel % $\frac{21.8}{2 \times 3 \times 144}$ = 2.52 %

Temperature stress –

$I_o = \frac{bd^3}{12} = \frac{2 \times 3^3}{12} = 4.5$ $I_c + I_s = 6.77$

w = 0.0000055 × 40 × 288,000,000 = 63400 #/□'

$T_c = -25.0 \times \frac{63400}{100} \times \frac{6.77}{20^2} = -26800$ #

$M_c = -22.0 \times 20 \times \frac{26800}{100} = +118,000$ 1#

$T_s = 0.83 \times 26100 = 22300$ #

$M_s = +11800 - 20 \times 26100 = 524200$ 1#

Average stress – Dead Load 410,000

Live Load 103,000

Crown 513,000 ÷ 6 = 185,500 #/□' × 0.9 = say 177,000 #/□'

Springing 492,000

54,000

546,000 ÷ 12 = 45500 #/□' × 0.9 = say 41,000 #/□'

Positive moment at crown

Fibre stress due to DL LL and Rib shortening.

		Thrust		moment
Dead Load	+	410,000	Assumed	+ 182,000
Live Load		103,000		+ 155,000
Rib shortening.	–	32,700		– 144,000
		480300		381,000

77,000 ÷ 63400 = 1.22

sec = .79 3/h = $\frac{.79}{3.0}$ = .264

K = .118

$f_c = \frac{381{,}000}{.18 \times 2 \times 3^2}$ = 118,000 #/□' or 775 #/□" [illegible]

Arch ring overstressed, [illegible]

~~Let us change the arch rise and estimate the cost~~

Make span length 135' Rise say 27 Rise Ratio = .20

Dead Load crown thrust = 0.725 × 3200 × 135 = 313,000

normal thrust at sp. = 0.983 × 3200 × 135 = 424,000

moment crown say 313,000 × 0.2 = 62600 1#

Springing " 424,000 × 0.5 = 212,000 1#

Live Load say 1750# for crown stress 1000# for springing –

Ratio Arch ring sp/crown = 6/3 = 2.0

Pos. m at crown 0.0052 × 1750 × 135² = 166,000 1#

Thrust 0.33 × 1750 × 135 = 78000 #

Stress at Springing
req. moment at Sp. = 0.02135 × 1000 × 135² = 389,000 '#
Thrust Ts = 0.36 × 1000 × 135 = 48,500 #
" Tc = 0.18 × 1000 × 135 = 24300 #

Temperature stress = Ic = $\frac{bd^3}{12} = \frac{2 \times 3^3}{12}$ = 4.5 14-1" bars = 10.9° on one side.
$\frac{21.8 \times 15}{144}$ = 2.27
Is = 2.27 × 1² = 2.27
$\frac{4.5}{6.77}$

Wt of E = 63400#/□'
Tc = 23.3 × 63400 × $\frac{6.77}{27^2}$ = −13,700 #
Mc = 20.9 × 27 × $\frac{13700}{100}$ = 77,500 '#
Ts = .74 × 13700 = 9850 #
Ms = 77500 − 27 × 13,700 = 292,500 '#

Average stress Approximate only
Crown D.L. 313,000
L.L. 78,000
391,000 ÷ 6 = 65,000 × 0.95 = 62,000 #

Positive Moment at Crown
Ecc = .815 2/h = $\frac{.815}{3.00}$ = .272
k = say .18

	Thrust	moment
Dead Load	313000	+ 62600
Live Load	78000	+ 166,000
Rib shortening	− 13500	+ 76,000
	377,500	304,600

fc = $\frac{304,600}{.18 \times 2 \times 3^2}$ = 94000 #/□' or 653 #/□"

Springing
Ecc = 1.93 2/h = $\frac{1.93}{6.00}$ = .322
k = .133

	Thrust	moment
Dead Load	424,000	− 212,000
Live Load	48500	− 389,000
Rib short say	− 9800	− 290,000
	462700	− 891,000

fc = $\frac{891,000}{.133 \times 2 \times 6^2}$ = 93000 #/□' or 645 #/□"

Try Arch rib. 2.5 × 3.0 at crown
D.L. = 1125 #
2300
3425 #

Span length 135 Rise 27 Rise Ratio .20
crown thickness 3.0 Spring. 6.75'

Dead Load Stress.
Crown thrust = 0.725 × 3425 × 135 = 336,000 #
Normal St. sp. = 0.913 × 3425 × 135 = 455,000 #

Moment assumed
crown 336000 × 0.2 = 67200 '#
Sp. 455,000 × 0.5 = 227,500 '#

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Estimate of Cost for Yasugabashi on Saigawa Line between Nagano & Matsumoto

Live Load say 1750# for crown 1000 for springing stress
Archring. d/crown = 2.25

Crown Stress Moment 0.00495 × 1750 × 135² = 158,000'#
Thrust 0.33 × 1750 × 135 = 78,000#

Stresses at Springing
neg moment = 0.0224 × 1000 × 135² = 408,000'#
Thrust Ts = 0.36 × 1000 × 135 = 48,500#
Tc = 0.18 × 1000 × 135 = 24300#

Temp Stress 14 - 1" bars one side Is = 2.27
$I_c = \frac{2.5 \times 3^3}{12} = \frac{5.63}{7.90}$ wtδE = 63400#/□'

$T_0 = 26.8 \cdot 63400 \times \frac{7.90}{27^2} = 18,400$#

$M_c = 20.9 \times 27 \cdot \frac{18400}{100} = 104,000$'#

$T_s = .74 \times 18,400 = 13,600$# Steel % $\frac{26.8}{2.5 \times 3.0 \times 144} = 2.0\%$

$M_s = 104,000 - 27 \times 18,400 = 392,000$'#

Average stress Approximate
DL 336000
LL 78000
414000 ÷ $\frac{3 \cdot 2.5}{7.5}$ = 55,000# × .93 = 51,000#/□'

Positive moment at crown Ecc = .775 $e/h = \frac{.775}{3.00}$ = .268, k = .153

	Thrust		Moment
Dead Load	336000	+	67200
Live Load	78000	+	158000
Rib shortening	− 14800	+	84000
	399200		309200

$f_c = \frac{309200}{.153 \times 2.5 \times 3^2}$ = 89500#/□' 620#/□" ok

Negative moment at Springing Ecc = 1.93 $e/h = \frac{1.93}{6.75}$ = .286, k = .125

	Thrust	Moment
Dead Load	455000	227,500
Live Load	48500	408,000
Rib shortening	− 11000	316,000
	493500	951,500

$f_c = \frac{951500}{.125 \times 2.5 \times 6.75^2}$ = 66800#/□' 467#/□" ok

Fibre Stress including Temperature Stress.

Crown Ecc = 1.09 $\frac{1.09}{3.00}$ = .362, k = .164

	Thrust	Moment
	399200	309200
Temp.	− 18400	104000
	380800	413200

$f_c = \frac{413200}{.164 \times 2.5 \times 3^2}$ = 111000#/□' ~ 770#/□" ok

Springing Ecc = 2.8 $e/h = \frac{2.8}{6.75}$ = .415, k = .13

	Thrust	Moment
Springing	493500	951500
	− 13600	392,000
	479900	1343,500

$f_c = \frac{1343500}{.13 \times 2.5 \times 6.75^2}$ = 90300#/□' ~ 630#/□" ok.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Concrete in Arch Ring, 3 × 2.5 × 135 × say 1.2 = 1220 × 1.2 = 1460
1220
2440 ÷ 216 = 11.3 ㎥
2920 13.5

Concrete in Floor Slab etc
Slab & Beam 20.0 × 135 = 2700
Floor Beam 45 × 15 = 675
3375 ÷ 216 = 15.7
27.0 ㎥
29.4

Reinforcing steel. Add for Hangers + 2. = 31.2

Slab	4 – ½" ⌀ @ .67	× 22.0	=	29.5
"	16 – ½ @ .67	× 1.0	=	10.7
"	8 – ½ @ .67	× 1.0	=	5.3
Beam	30 – ¾ @ 1.5	× 1.0	=	45.0
F.B.	8/10 – 1" ⌀ @ 2.67	× .25	=	53.5
	misc bars say			20.0
				164.0#

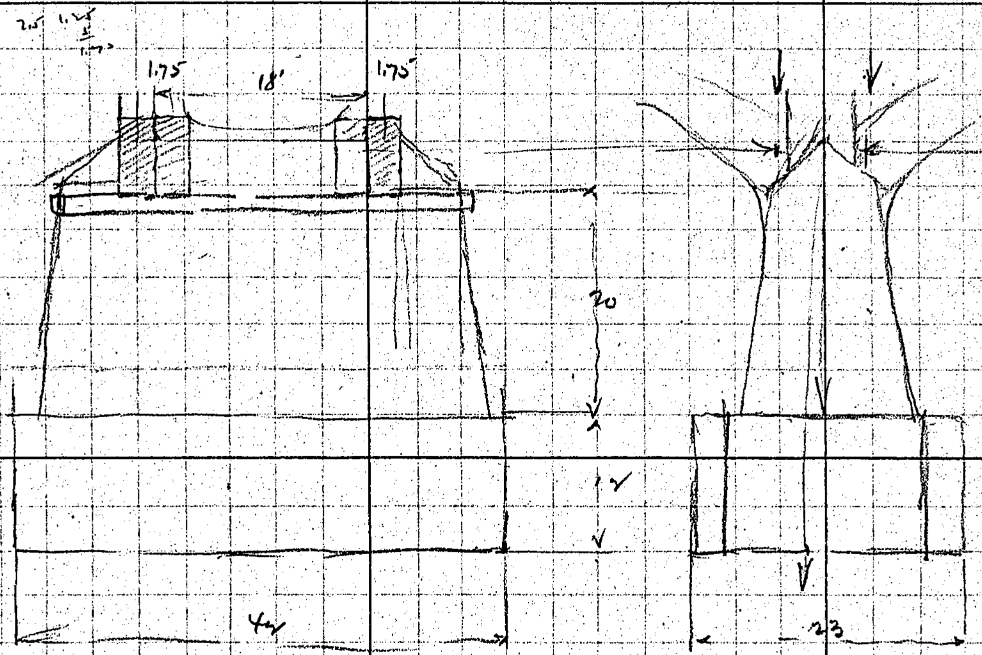
164.0# per ft × 135 = 22200# or 10 tons.
Add 20% 4400
26600 12 tons

1700# per cubic tsubo of concrete

Arch 21.8 @ 3.4 = 74#
74
148 × 1.4 = 2070# × 135 = 28000 ÷ = 2480# per cubic tsubo

12.5 tons

Reinforcing bars in slab 12.0
Arch 12.5
24.5 for one span or say 26 tons.



Superimposed load.
Floor system – 4600 × 140 = 650,000
Arch ring – 2920 @ 150# = 440,000
1,090,000
Live Load – 2000/2 # per ft × 135 = 135000#

Horizontal thrust =

$\frac{1}{8} \times \frac{2000}{270} \times 135^2 = 169,000$#

weight of Pier – 12 × 25 × 36 = 10800 @ 150 = 1620,000
base 12 × 23 × 42 = 11600 @ 150 = 1740,000
23400 3360,000
104 ㎥ 1090,000
135,000
4,585,000#

Horizontal moment $e = \frac{169,000 \times 35}{4,585,000} = 1.29'$

1.29 × 6 = 7.7

Base can be reduced & Concrete in pier say 80 ㎥.
In Abutment say 50 ㎥.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Estimate of Cost for Masugabashi on Saigawa Line between Nagano & Matsumoto 25

Summary of materials.
Concrete in Floor 15.7 × 2 = 31.4 call this 32. Ts
" " Arches. 13.5 × 2 = 27.0 tsubo

Reinforcing Bars say 25 tons @ 2 = 50 tons.
In Abutment + pier say 10
60 tons.

Estimate of Cost

Concrete in Floor slab	32 tsubo	@	350	=	11200
Concrete in Arch Ring	27. tsubo	@	400	=	10800
Reinforcing bars	60 tons.	@	220	=	13200
piers	80 tsubo	@	250	=	20000
Abutment say	50 "	@	250	=	12500
Excavation &c.					15000
Handrail. 560'		@	2.50	=	1400
Finish		@		=	2000
Pavement	140 tsubo	@	.40.00	=	5600
					91700
variation &c say					9170
					100870

call this 100,000 ¥.

US (A) Ts.

Estimate of Cost for 220' simple truss span + 47.5' Concrete girder span.

Structural Steel 220' span	150 tons	@	400.00	=	60.000
Concrete in slabs "	11.2 Ts	@	300.00	=	3360
Reinf. "	8.0 tons	@	240.00		1920
47.5' span concrete	10. Ts	@	320		3200
Steel	7. tons	@	240		1680
abutments					4000
Concrete Handrail					600
Pavement	140	@	40.00		5600
finish say					600
One pier complete					20000
				¥	100960 #
			variation say		10090
				¥	111050

Arch span layout say same as for this layout
Take Estimate - 111000.00 on safe side.

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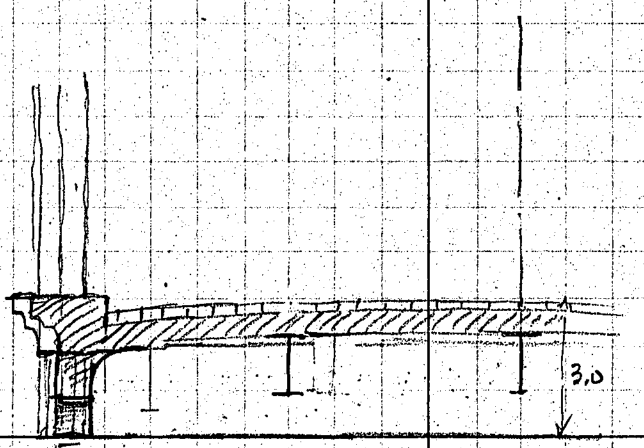
Span 150' Arch on center Retaining walls or girder spans on sides -
Total length - 37K × 30 = 111 FT bridge floor
Concrete slab floor with wood blocks pavement concrete Handrail - 37.
150' Arch span will be of steel; Floor system & hangers are of all steel.
Kendo Loading - throughout

Cross Section of bridge -



Concrete slab 7"
Reinforcing bars 1/2"ϕ 4" centers

Panel length 12.0
Stringer span 12.0

Dead Load -

wood blocks	18
cushion say	7
concrete slab - 7"	88
	113#

Load on stringer 113 × 5 = 565
beam assumed 35
600#

$m = \frac{1}{8} \times 600 \times 12^2 = 10800$ 1#

Live Load moment
concentration = 3600 × 6 = 21600 — 21600
uniform load 32400

$Sm = \frac{32400 \times 12}{16000} = 24.0$

neglect this uniform load

Use 3 - 12" × 31.5# = say 100# per lin ft.

Fascia girder concrete
Approximate Dead Load - Handrail - 150#
coping - 375
Roadway 113 × 2 225
750#

DL m = 1/10 × 750 × 12² = 10800 1#
LL m say 21600 × .8 = 17300
28100

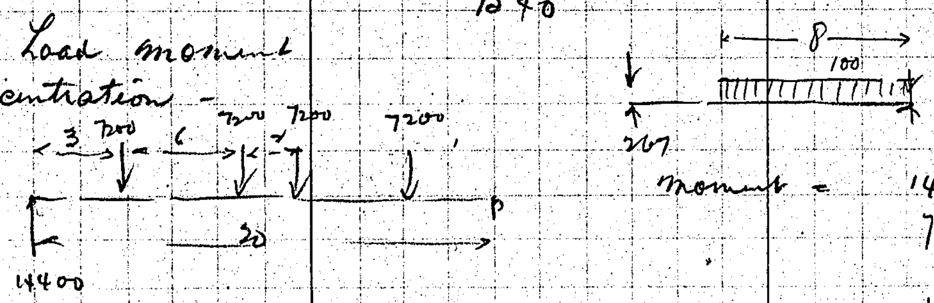
Steel Area = $\frac{28100 \times 12}{\frac{7}{8} \times 24 \times 16000} = 1.00$

Use 4 - 5/8"ϕ bars = 1.20" -

Floor Beam span length - 20' spacing 12'
Dead load 113#
beam say 7
120# × 12 = 1440#
Dead Load FB 100
1540

$m = \frac{1}{8} \times 1540 \times 20^2 = 77,000$ 1#

Live Load moment
Concentration -



800 × 4/12 = 267

moment = 14400 × 9 = 129,600
7200 × 6 = 43200
86400 1#

moment = 1/8 × 267 × 20² = 13300 1#

99,700
77,000
176,700 1#

Back to back of L^s 30½" Effective depth = 2.40
Stress = 176,700 / 2.4 = 73500 SR = 73500 ÷ 16000 = 4.60
web 30 × 5/16 = 9.37 1/8 web = 1.17
3.43

For Flange use 2L^s 3½ × 3½ × 3/8 = 4.96 gross - 4.21 net

Estimate of Cost for Dansuiji Bashi on Saigawa Line between Nagano + Matsumoto

I web	30 × 5/16 @ 31.88	× 20 =	637
Flanges	4Ls 3½ . 3½ × 3/8 @ 8.5	× 20 =	680
Details say			500
			1817# ÷ 12 = 150# per lin ft of span.

Floor system 150
100
250# per lin ft.
Bottom Bracing 50
300# per lin ft.

Dead Load on Arch.

Roadway	113# × 15/2 =	1010
Coping say		375
Handrail		150
Metal in Floor System, 300/2		150
		1685 say 1700# per lin ft of Arch rib.

$Wa = (0.000248D + 0.000416L)l$

$\frac{0.545}{\frac{625}{1.170}} \times 150 = 190^{\#}$ say 200

Arch rib Assumed 500
2200

Dead Load thrust at crown $= \frac{1}{8} \times \frac{2200 \times 150^2}{30} = 206,000^{\#}$

Live Load thrust say $= \frac{1}{8} \times \frac{1500 \times 150^2}{30} = 141000$

347000 ÷ 8000 = 43.0".

4Ls 6 × 6 × 7/8 714		28.6
2 Pls. 30 × 1/2 =		15.0
	43.6 @ 3.40 =	150# per lin ft
	Details say	50
		200# per lin ft

Structural steel in span. Floor system 300
Arch Ribs 400
On pier say 70
770# @ 150 = 116000#
variation say 24000
140000# say 65 tons

Estimate of Cost.

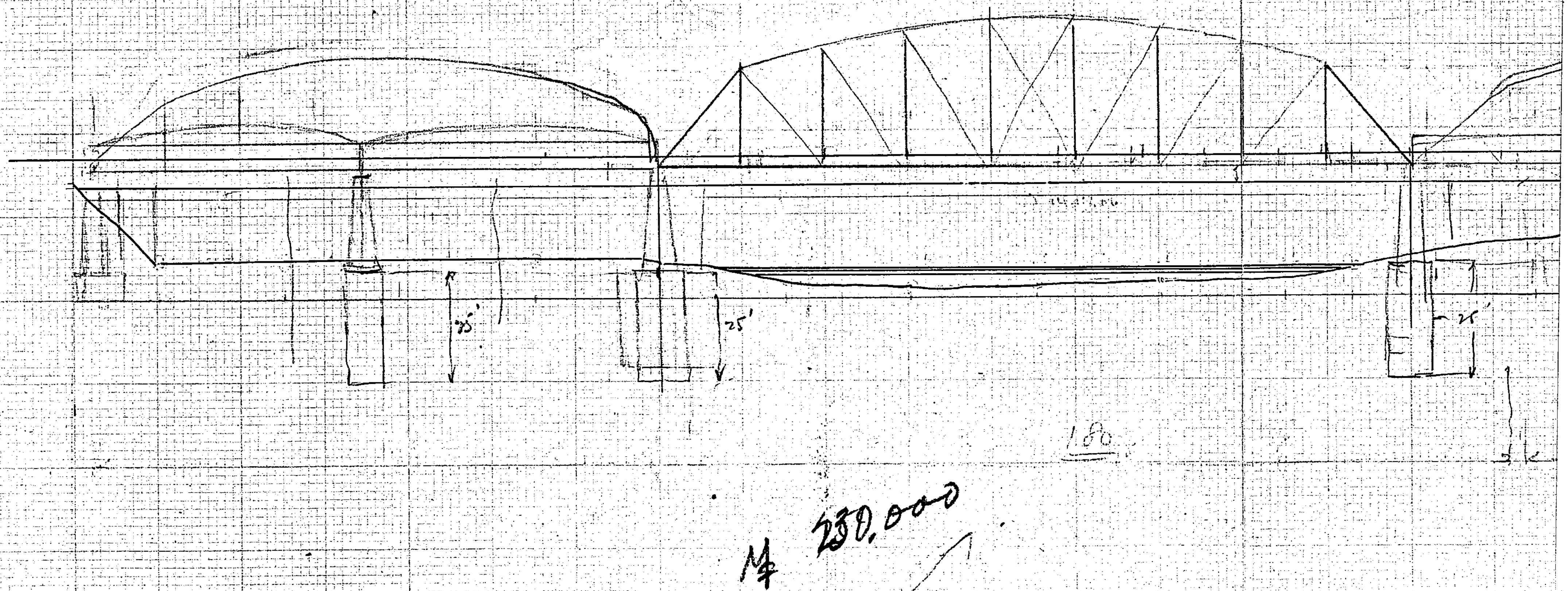
structural steel	65 tons @ 400	=	26,000 ¥
Floor concrete	0.6 × 135 = 8.1 tsubo @ 300	=	2440
" steel	5.8 tons @ 240	=	1400
2 Abutments	2 @ 2000		4000
2 walls	2 @ 2500		5000
Pavement	111 @ 40		4440
Handrail	440 @ 2.5		1100
finish say			600
			44980
		Add	15020
			¥ 60000

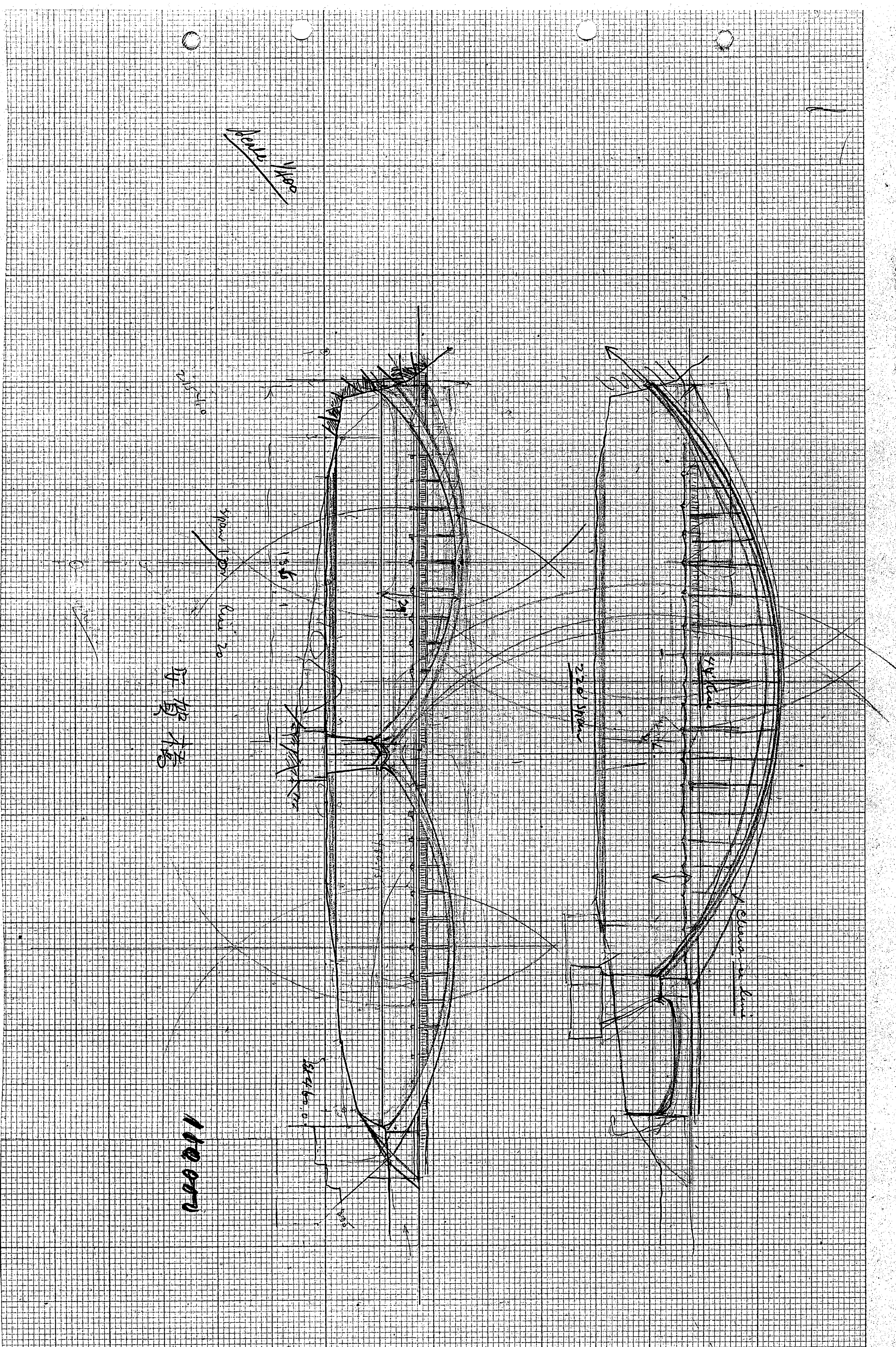
540円/坪.

Summary of Estimate of Cost

	Estimate to be spend	Estimate of Cost		
⊗ 穂刈橋	264.364	240.000 ([illegible] crossed out)	250	920
安坂橋	183.096	140.000	140	785
山王跨線橋	203.505	60.000	111	540
小泉橋	213.510	150.000	157	835
犀川橋	649.970	275.000	510	540
	1514.445	835.000	140	
	835.000			
	679.445			

168
130
38


25'
25'
25'
180
M 230,000



増田淳氏関係資料（独立行政法人 土木研究所蔵）