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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

昭和四年十一月

揖斐長良川橋予算設計書

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bashi for Miye Ken

Total length of bridge 3640' = 1110 meters
clear roadway 7.0 meters.

Railway Bridge at up stream 15 - 200 clear span } 16 spans.
1 - 120 clear span }

making 15 spans $\frac{1110}{15}$ = 73.8 meter 73. meters c to c of end bearings about 240'

making 16 spans $\frac{1110}{16}$ = 69.4 meter 68.5 meters " " " " about 225'

Loading 2nd class

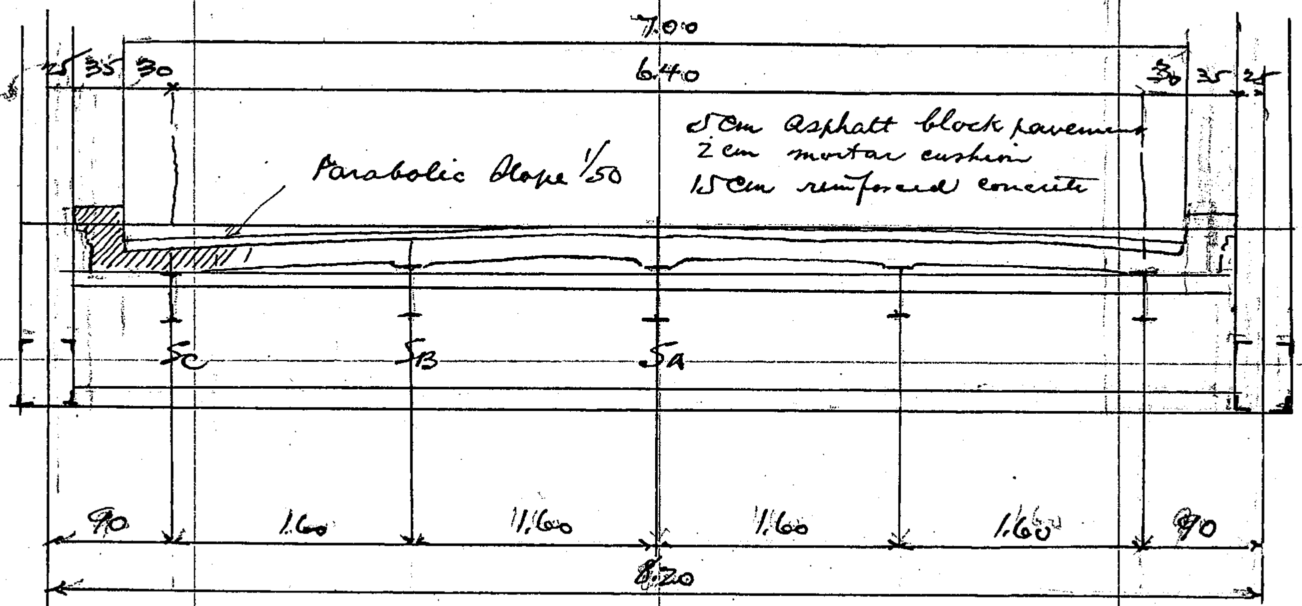
Uniform load = $\frac{100.000}{170+l}$ ≤ 500 kg/m²

motor truck front wheel 1000 kg } 8.0 ton truck
rear wheel 3000 kg }

Road roller 11 tons

impact Coef = $\frac{20}{60+l}$ max 30%

Cross section of bridge. assumed as shown on sketch below:—



7.00
1.20
8.20

Floor Slab. span length 1.60 meters
Dead Load

5cm	asphalt block pavement	@ 21 kg	105
2cm	mortar cushion	@ 22	44
15cm	concrete slab	@ 24	360
misc filler.	say		11
			520 kg/m²

Weight of Coping 230 kg per lin meter
Weight of Handrail 70 kg per lin. meter

Dead Load of flooring complete

520 × 7.0	=	3640
230 × 2	=	460
70 × 2	=	140
		4240 kg per lin. meter

I Beam Stringer Same as for Hagiwara Bashi.

Span length 6.0 meters spacing 1.60 meters
Center Stringer Section modulus = $\frac{1313800}{1100}$ = 1194

400 × 150 I @ 72.01 kg S_m = 1199

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-Ken

製鉄所 Section | 16" × 6" | 28.109 × 3.28 = 92 kg per lin. meter
Section modulus = 1480

40 × 16 = 6.40 meter

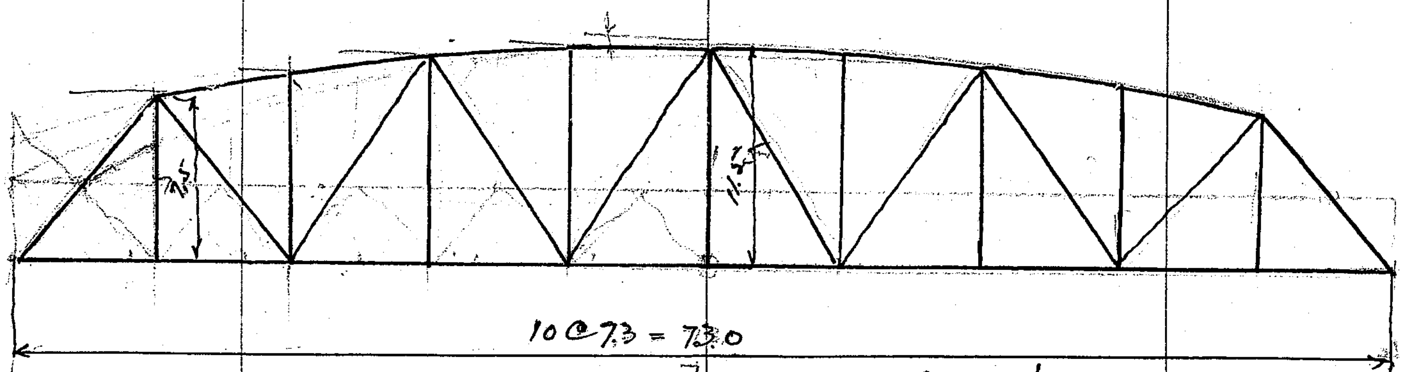
Spacing of Floor-beam 6.0 meter. See design of Hagiwara-Bashi.
weight of intermediate floor beam
200 × 7.8 = 1560 or say 1600 kg.
weight of End Floor Beam 1500 kg.

Floor system stringers 5 @ 95 = 475
1600 ÷ 6.0 = 267
692 call this 700 kg per lin meter

Bottom Lateral Bracings say 120 kg per lin. meter

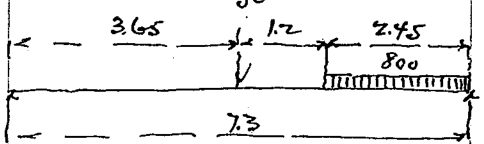
Top Lateral Bracings Sways and Portal. 300 kg per lin. meters

Design of Simple Span



Scale 1/400

Design of stringer. span length 7.3 meters
Center stringer 5A. See Hagiwara-Bashi. $R_1 = \frac{800 \times 2.45^2}{2 \times 7.3} = 329$
front wheel neglected.
moment due to motor truck 2800 × 3.65 = 10200
" " " Unif. load 329 × 3.65 = 1200
11400



Dead Load 930 kg per meter moment = 1/8 × 930 × 7.3² = 6200
17600 kg m

Section modulus reqd = $\frac{1760000}{1100}$ = 1600

Yawata section 18" × 7" ~~108.2~~ 111.5 kg per lin. meter Sm = 2100
Ratio depth / span = 1/16.25

Stringer 5C	H.R. coping.	70 × .45		31.5
		230 × .45	=	103.5
		156 × .15	=	23.4

~~456~~
~~386~~
180
~~466~~
556
416
100
~~576~~
1072

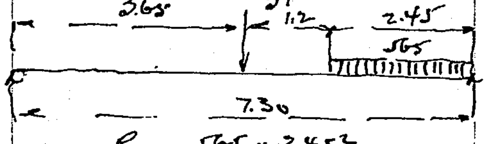
~~126.9~~ 158.4 ÷ 1.60 = 99 80

slab & pavement

stringer assumed

Dead Load moment = 1/8 × ~~576~~ 1072 × 7.3² = ~~3440~~ 7150

Live Load Uniform load = $\frac{500 \times 1.90^2}{2 \times 1.60}$ = 565 kg.
motor truck concentration on stringer
motor truck loading 1950 × 3.65 = 7100
232 × 3.65 = 845
7945
DL.m 7150
15095



$R = \frac{565 \times 2.45^2}{2 \times 7.30} = 232$

Section modulus reqd = $\frac{1509500}{1100}$ = 1370
use 1 I 16" × 6" = 92. kg

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CALCULATIONS FOR

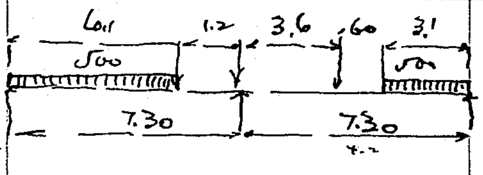
Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-Ken

Stringers 3 @ 120 = 360
2 @ 100 = 200
560 kg per lin. meter

Floor Beam Span length 8.20 spacing 7.3 meter
Dead Load flooring 520 × 7.3 = 3800 kg.
beam assumed 250
4050 kg per meter
moment = 1/8 · 4050 · 8.2^2 = 34000 kgm.

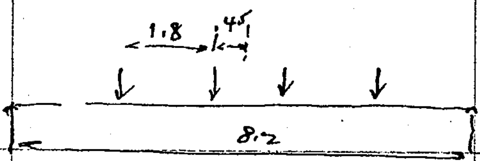
Live Load motor truck loading. 3900.
1300 × 3.7 ÷ 7.30 = 660

3900
660
550
~~4450~~
4560



Uniform live load 500 · $\frac{3.12}{2 \times 7.30}$ = 330
500 · $\frac{6.12}{2 \times 7.30}$ = 1270
1600 kg.

approximate moment of floor beam
motor truck loading 2 × 4560 × 4.1 =
2 × 4560 × -1.30
2.75 = 25100



Unif. load 1/8 · 1600 · 8.2^2 = 13450
38550
Dead load m 34000
72550

Try web plate 800 × 9 = 72.0 1/8 web = 9.0 Effective depth 81.0 - 2.28 = 78.72 cm
flange stress = $\frac{72550}{.7872}$ = 92300 kg. sR = 77.0 - 9.0 = 68.0

2Ls 125 × 90 × 13 = 52.52 - 11.44 = 41.08
1Pl 270 · 13 = 35.10 - 5.72 = 29.38
87.62 70.46 □cm net

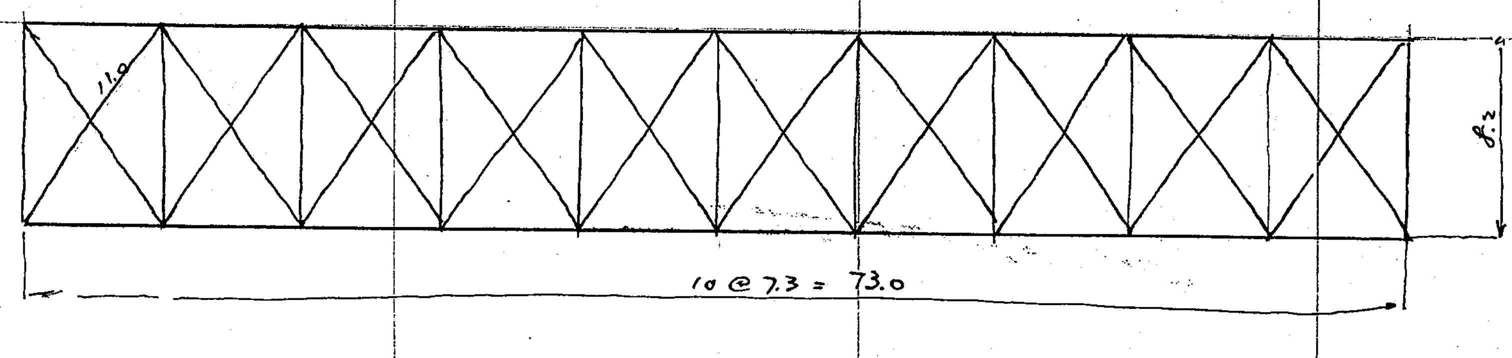
approximate weight of intermediate floor beam
web and flange Ls 138.96 × 7.7 = 1070
cover plate 2 Pls. 270 × 13 @ 27.6 × 5.50 = 303
1373
details say 327
1700 kg.

End Floor beam use flange Ls 125 × 90 × 10
Cov. pls 270 × 9
approximate weight of End floor beam say 1520

Total Steel in floor beam
Intermediate floor beam 9 @ 1700 = 15300
End floor beam 2 @ 1520 = 3040
18340

average weight = 18340 ÷ 73 = 251 kg per meter
stringers 560
811

Bottom Lateral Bracings



$\frac{11.0}{8.2}$ = 1.34

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miyeken.

Approximate Dead Load

Flooring		520 × 7.0	=	3640
Coping	2 @ 230		=	460
Handrails	2 @ 70		=	140
				4240 kg

Structural Steel

Stringers & Floor beam	810
Bottom & Top laterals &c	450
Trusses assumed	2000
	3260 — 3260
	7500 kg per lin meter

Seismic force assumed 3000 mm/sec² 7500 × 0.3 = 2250 kg.

panel concentration 2250 × 7.3 = 16400 kg.

Panel	shear				
0–1	74000 × 1.345 =	99500 ÷ 2160 =	46.0	2 Ls 150 × 150 × 11 =	63.58 − 11.0 = 52.58 net
1–2	57500	77300 =	35.8	2 Ls 130 × 130 × 9 =	45.18 − 9.0 = 36.18 "
2–3	41000	55000 =	25.5	2 Ls 125 × 90 × 10	= 41.00 − 10.0 = 31.00
3–4	24600	33000 =	15.3	2 Ls 125 × 75 × 10	= 38.00 − 10.0 = 28.00
4–5	8200	11000 =	5.1	do.	

Approximate weight of Lower Lateral Bracing.

panel 0–1 2 Ls 150 × 150 × 11 @ 24.95 × 10.5 = 523
4 Ls " × 5.0 = 500
Center connection 31
Stringer connections 4 @ 10 40
misc details & rivet heads 16
1110 kg.

Panel 1–2 2 Ls 130 × 130 × 9 @ 17.73 × 10.5 = 373
4 Ls " × ~~10.0~~ 5 = 355
details say 80
808 kg.

Panel 2–3 2 Ls 125 × 90 × 10 @ 16.09 × 10.5 = 338
4 Ls " × ~~10.0~~ 5 = 322
Details say 80
740 kg.

Panel 3–4 2 Ls 125 × 75 × 10 @ 14.91 × 10.5 = 313
4 Ls do × 5.0 = 298
611 kg.

Total weight of Steel

2 @	1110	=	2220
2 @	808	=	1616
2 @	740	=	1480
4 @	611	=	2444
			7760 kg.

7760 ÷ 73 = 106 kg per lin. meter

Upper Lateral Bracings. Length of Lateral 7.5^2 = 56.25
4.1^2 = 16.80
73.05 — 8.55 meters

radius of gyration required = $\frac{855}{150}$ = 5.70

4 Ls 125 × 75 × 10 = 76.0 @ .785 × 8.0 = 477
details say 123
600 kg.

2 @ 600 kg. = 1200 kg.

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bashi for Miyagi-Ken

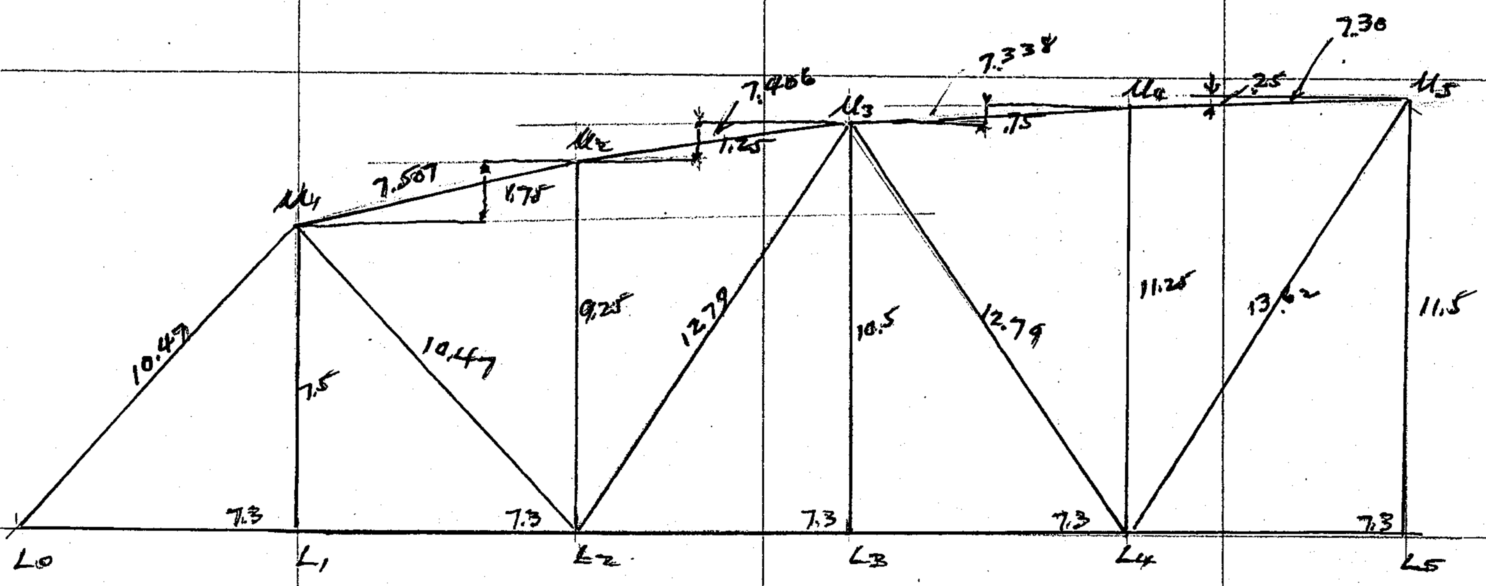
cross strut at U2 and U4 approximate weight 600 kg.

Sway Bracing. at U3 and U5 approximate weight 2030 "
Portal bracings 2400 "

approximate weight.

diagonals	16 @ 600	=	9600
strut.	4 @ 600	=	2400
Sway	3 @ 2030	=	6090
Portal Bracing	2 @ 2400	=	4800
			22890 kg.

22890 ÷ 73 = 314 kg per lin. meter of span



Lever arms U1-U2 $9.25 \times \frac{7.30}{7.507} = 9.00$

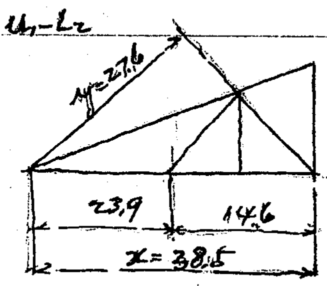
U2-U3 $9.25 \times \frac{7.30}{7.406} = 9.10$

U3-U4 $11.25 \times \frac{7.300}{7.338} = 11.20$

U4-U5 $11.25 \times \frac{7.300}{7.300} = 11.25$ say

L1-U1 $\sec\theta = \frac{10.47}{7.5} = \cancel{1.46}\ 1.395$

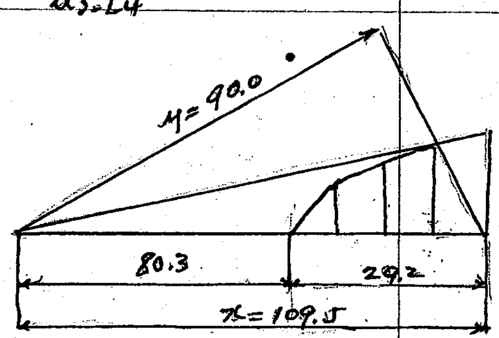
U1-L2



$x = 9.25 \times \frac{7.3}{1.75} = 38.5$

$y = 38.5 \times \frac{7.5}{10.47} = 27.6$

U3-L4



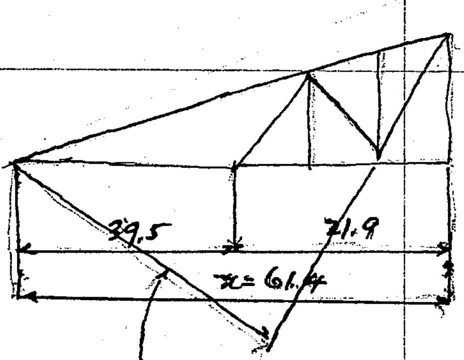
$x = \frac{11.25 \times 7.3}{.75} = 109.5$

$y = \frac{109.5 \times 10.5}{12.79} = 90.0$

L2 U3

$x = \frac{105 \times 7.3}{11.25} = 61.4$

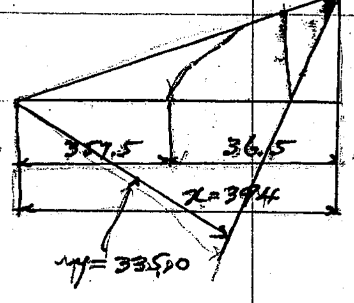
$y = \frac{54.1 \times 10.5}{12.79} = 44.5$



L4-U5

$x = \frac{11.5 \times 7.30}{.25} = 394$

$y = \frac{396.9 \times 11.5}{13.62} = 335$



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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bashi for Miye-Ken

Dead Load stresses due to unit load

Chord stress

					Diagonals
End Post	L_0-U_1	4.50×1.395	=	6.28	U_1-L_2 $4.5 \times 23.9 = 107.5$ 31.2 $76.3 \div 27.6 = 2.76$
moment at U_1	L_0-L_2	$4.5 \times \frac{7.30}{7.50}$	=	4.38	
moment at L_2	U_1-U_2	$8.0 \times \frac{7.30}{9.00}$	=	6.50	L_2-U_3 $4.5 \times 39.5 = 177.8$ -101.4 $76.4 \div 44.5 = 1.715$
	U_2-U_3	$8.0 \times \frac{7.30}{9.10}$	=	6.42	
moment at U_3	L_2-L_4	$10.5 \times \frac{7.30}{10.50}$	=	7.30	U_3-L_4 $4.5 \times 80.3 = 364.0$ -284.7
moment at L_4	U_3-U_4	$12.0 \times \frac{7.30}{11.20}$	=	7.80	$76.3 \div 90.0 = 0.850$
	U_4-U_5	$12.0 \times \frac{7.30}{11.25}$	=	7.78	L_4-U_5 4.5×357.5
moment at U_5	L_4-L_5	$12.5 \times \frac{7.30}{11.5}$	=	7.95	$76.3 \div 335.0 = 0.228$

Dead Load panel Concentration $3750 \times 7.3 = 27400$ kg.

Chord stresses

					Diagonals			
L_0-U_1	6.28	$\times 27400$	=	172200	U_1-L_2	2.76	$\times 27400$ =	75600
U_1-U_2	~~4.38~~ 6.50		=	178000	L_2-U_3	1.715	=	47000
U_2-U_3	6.42		=	176000	U_3-L_4	0.850	=	23300
U_3-U_4	7.80		=	214000	L_4-U_5	0.228	=	6250
U_4-U_5	7.78		=	213000				
L_0-L_2	4.38		=	120000				
L_2-L_4	7.30		=	200.000				
L_4-L_5	7.95		=	218.000				

Live Load on truss

Uniform live load $w = \frac{100.000}{170+73} = 411.0$

motor truck loading rear wheel 3000

front wheel 1000

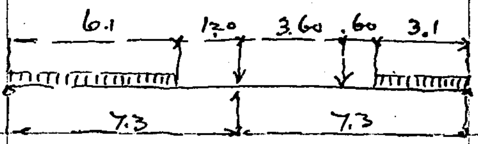
Impact $I = \frac{20}{60+73} = 15.\%$ 20% impact assumed

panel concentration full uniform load $411 \times \frac{7.0}{2} \times 7.3 = 21000$ kg.

for one truss 10500 kg

Extra concentration due to motor truck loading. 3000
 600

rear wheel 3600 front wheel 1200



motor truck loading. $1200 \times \frac{3.7}{7.3} = 607$

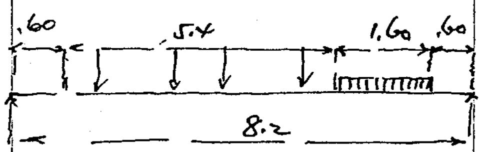
3600

4207 kg.

Uniform load $411 \times \frac{3.1^2}{2 \times 7.3} = 280$ $411 \times 7.3 = 3000$ kg per meter.

$411 \times \frac{6.1^2}{2 \times 7.3} = 1050$

1330



4207
4
$16828 \times \frac{4.90}{8.20} = 10100$

$1330 \times 3.5 = 4650$

14750
10500
4250 Extra concentration say 4500 kg. on one truss.

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bashi for Miye Ken.

Live Load stresses Uniform Concentration 10500 kg.
Chord stresses

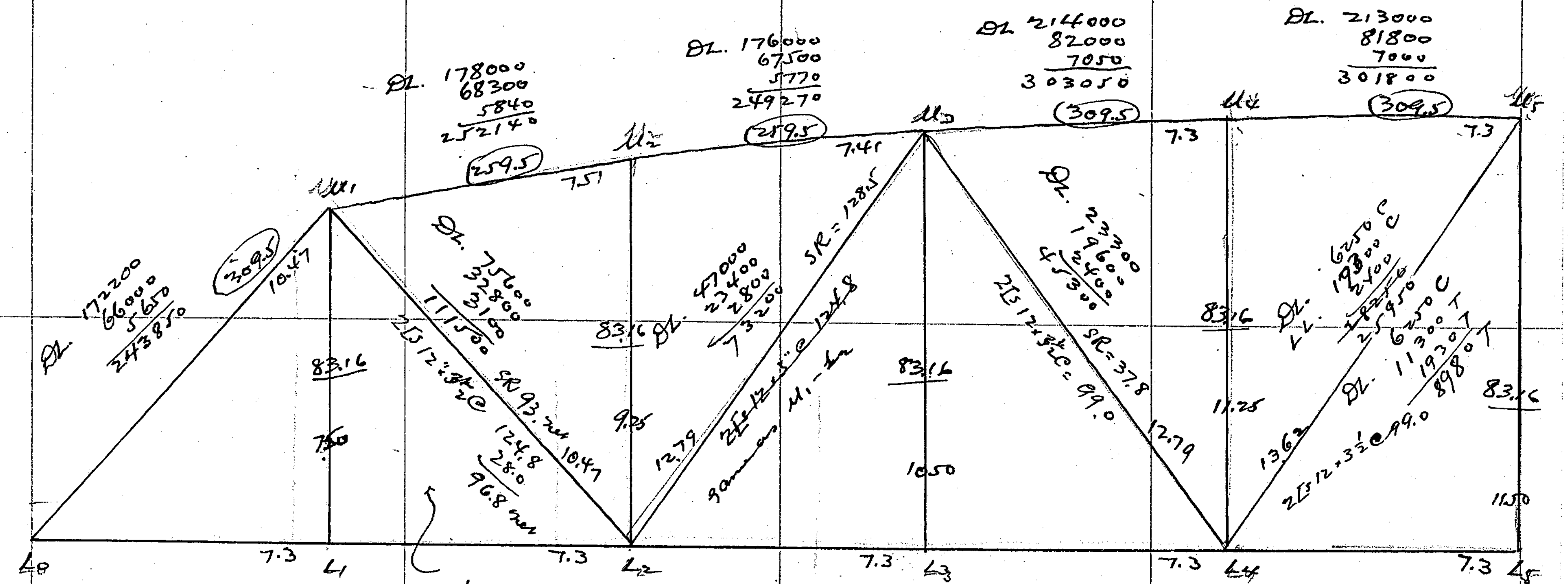
L_0-U_1	6.28	× 10500 =	66000	U_1-L_2	3.6 × $\frac{23.9}{27.6}$ =	.312	× 10500 =	32800
U_1-U_2	6.50		68300					
U_2-U_3	6.42		67500	L_2-U_3	2.8 × $\frac{39.5}{44.5}$ =	2.23	=	23400
U_3-U_4	7.80		82000	U_3-L_4	2.1 × $\frac{80.3}{90.0}$ =	1.87	=	19600
U_4-U_5	7.78		81800					
L_0-L_2	4.38		46000	L_4-U_5	1.5 × $\frac{357.5}{335.}$ =	1.64	=	17300
L_2-L_4	7.30		76700	U_5-L_4'	1.0 × $\frac{357.5}{335.}$ =	1.07	=	11300
L_4-L_5	7.95		83500					

Live Load stresses due to Extra Concentration 4500 kg.
Chord stresses

L_0-U_1	4500 ×	0.9	× 1.395	=	5650
U_1-U_2		0.8	× 14.6 ÷ 9.00	=	5840
U_2-U_3		0.8	× 14.6 ÷ 9.10	=	5770
U_3-U_4		0.6	× 29.2 ÷ 11.20	=	7050
U_4-U_5		0.6	× 29.2 ÷ 11.25	=	7000
L_0-L_2		0.9	× 7.3 ÷ 7.5	=	3940
L_2-L_4		0.7	× 21.9 ÷ 10.5	=	6550
L_4-L_5		0.5	× 36.5 ÷ 11.5	=	7130

Diagonals

U_1-L_2	4500 × 0.8	× 0.865	=	3100
L_2-U_3	0.7	0.887	=	2800
U_3-L_4	0.6	0.892	=	2400
L_4-U_5	0.5	1.070	=	2400
U_5-L_4'	0.4	1.070	=	1930



2Pls. 380 × 9 = 68.40
4LS 90×90×10 = 68.00
136.40
− 38.00
98.40 net

DL. 120.000	DL. 20.0000	DL. 218.000
46000	76700	83500
3940	6550	7130
169940 SR = 142.	283250 SR 236.0	308630 SR = 257.0
135.0 184	297.0 306.	297.0 338.0

1000 plate 650 × 13 = 84.50
2Pls. 450 × 11 = 99.00
4LS 100×100×10 = 76.00
259.50
2 Side Pls. 250 × 10 = 50.00
309.50

L_2U_3. 2LS 12×

verticals & Hangers
4LS 150×90×9 @ 20.79 = 83.16

Bottom chord

2Pls.	450 × ~~11~~ 12 =	108 ~~99.0~~	− ~~4.0~~ ~~22.0~~	= ~~88.0~~	84. ~~77.0~~	} 140.	
4LS	100×100×10 =	76.0	− 20.0 ~~10.0~~	= ~~66.0~~	56.0		
2Pls.	250 × 10 =	50.0	− ~~5.0~~ 10	= 45.0	40.0		
2Pls.	400 × 9 =	72.0	− ~~9.0~~ 18	= 63.0	54.		
		~~297.0~~		~~262.0~~	227.0	234	
2 Pls	400 × ~~11~~	~~44~~ 32	− 6.0 8.0		+8.0 245	24 ~~252~~ 258	

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bridge for Miye-Ken

Approximate weight of truss.

	section				
Lo-U1	309.5	@ .785	× 10.47	=	2550
U1-U2	259.5		× 7.51	=	1530
U2-U3	259.5		× 7.41	=	1510
U3-U4	309.5		× 7.30	=	1780
U4-U5	309.5		× 7.30	=	1780
Lo-L2	175.0		× 14.60	=	2010
L2-L3-L5	297.0		× 21.90	=	5100
U1-L2	136.4		× 10.47	=	1120
L2-U3	136.4		× 12.79	=	1370
U3-L4	99.0		× 12.79	=	990
L4-U5	99.0		× 13.62	=	1060
U1-L1 etc	83.16		× ~~82.75~~ 45.25	=	~~5400~~ 2960

184 × 14.6 @ .785 = 2110
306 × 14.6 @ .785 = 3520
338 × 7.3 @ .785 = 1940
7570
7110
460

4540

~~26200~~ 23760 × 2 = ~~52400~~ 460 24220

Details say 40 % ~~21000~~

for one truss ~~73400~~

" two " ~~146800~~

~~47520~~ 48440
~~19000~~ 19300
~~66520~~ 67740
~~133040~~ 135480

135480
~~133040~~
~~146800~~ ÷ 73 = ~~2010~~ 1855 kg per lin. meter

Approximate weight of Structural Steel in one truss span

Stringers ~~560~~ 490 × 74.0 =	~~41500~~ 36300 − 5200	36300	
Floor beams	18340	~~18340~~ 18365	
bottom lateral bracings	7760	7760	
Upper lateral & Sway Portals	22890	22890	5480 3040 2440
Trusses	~~146800~~	~~133040~~ 5480	
	~~237790~~	232090 ~~228330~~	Revised Final
Shoes say	4500	4500 4500	36300
	~~241790~~	236590 ~~228330~~ kg. 2440	18340
		215270	7760
			22890
			135480
			4500
			225270

225.27 × 15 = **3380 tons**

Revised design for stringers
Center stringers M = 17600 kgm
web 420 × 9 = 37.8 1/8 web = 4.73 depth 43 cm Effective d = .395

flange stress = $\frac{17600}{.395}$ = 44500 3K $\frac{44500}{1200}$ = 37.2
4.73
32.47

2L 125 × 75 × 10 = 38.0 − 4.4 = 33.6 cm net

approximate weight 38 × 2 = 76
37.8
113.8 @ .785 = 89.5 kg.

End connection 5.0

Rivet heads 3½ 3.5
98.0 kg per lin. meter.

Try Side Stringers. 2L 100 × 75 × 10 @ 16.5 = 33.0
d = 38 cm d = 34 flange stress = $\frac{15100}{.34}$ = 44500

try same section as for center stringers
approximate weight 98 × 5 = 490 kg per lin. meter.

560
490
70 kg × 74 = 5200 kg.

~~241790~~
5200
236590 for one span

157 百平 1.37
$\frac{\cancel{236.59}\ 215.27}{157}$ = ~~1.50~~ ~~1.5~~ tons per tsubo

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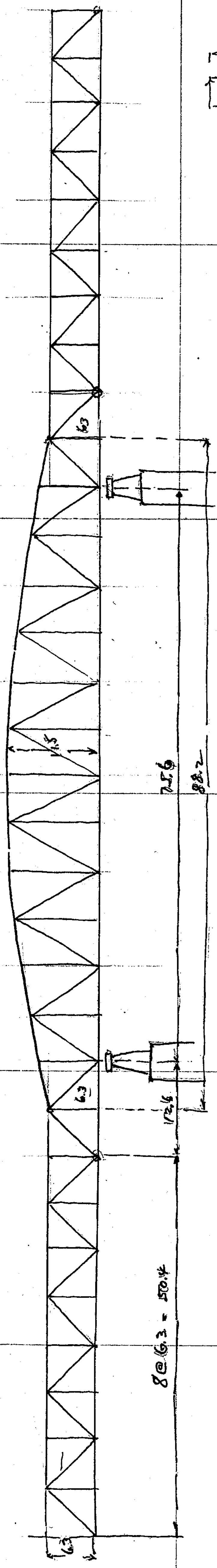
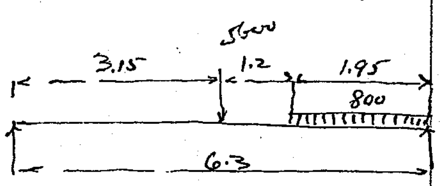
CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bashi for Miye-Ken



Stringer Span length 6.30 meters

$R = \frac{800 \times 1.95^2}{2 \times 6.3} = 241$



moment motor truck	2800 × 3.15	=	8820
	241 × 3.15	=	760
			9580
D.L. m.	1/8 × 930 × 6.3²	=	4620
			14200

Section modulus req'd = $\frac{1420000}{1100}$ = 1290

use 1 I 16" × 6" = 92 kg.

weight of stringer 5 @ 95 = 475 kg per lin. meter.

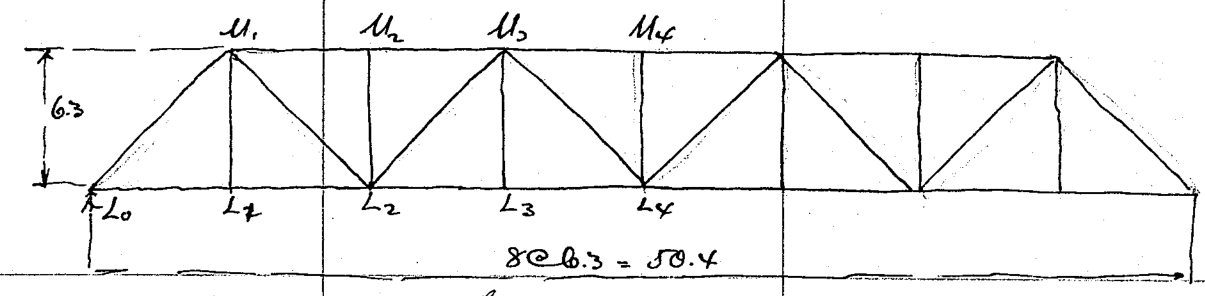
Floor beam intermediate 1700 kg.

$\frac{1700}{6.3}$ = 270 kg per lin meter

Bottom Lateral Bracing. say 110 kg per lin. meter.

Approximate Dead Load

panel dead load 3750 × 6.3 = 23600 kg.



Reaction 23600 × 3.5 =

Dead Load stress

L0-U1	23600 × 3.5 × 1.41	=	116500
L0-L2 ~~U1-U2~~	23600 × 3.5	=	82500
U1-U2	23600 × 6.0	=	141600
L2-L4	23600 × 7.5	=	177000
U3-U4	23600 × 8.0	=	189000
U1-L2	23600 × 2.5 × 1.41	=	83000
L2-U3	× 1.5	=	50000
U3-L4	× 0.5	=	16600

75.6
25
1512

Live Load stresses.

Panel Load $w = \frac{100.000}{170 + 50.4}$ = 453 × 3.5 = 1585

panel Concentration say 1585 × 6.3 = 10,000 kg.

Extra ~~load~~ Concentration due to motor truck say 4500 kg

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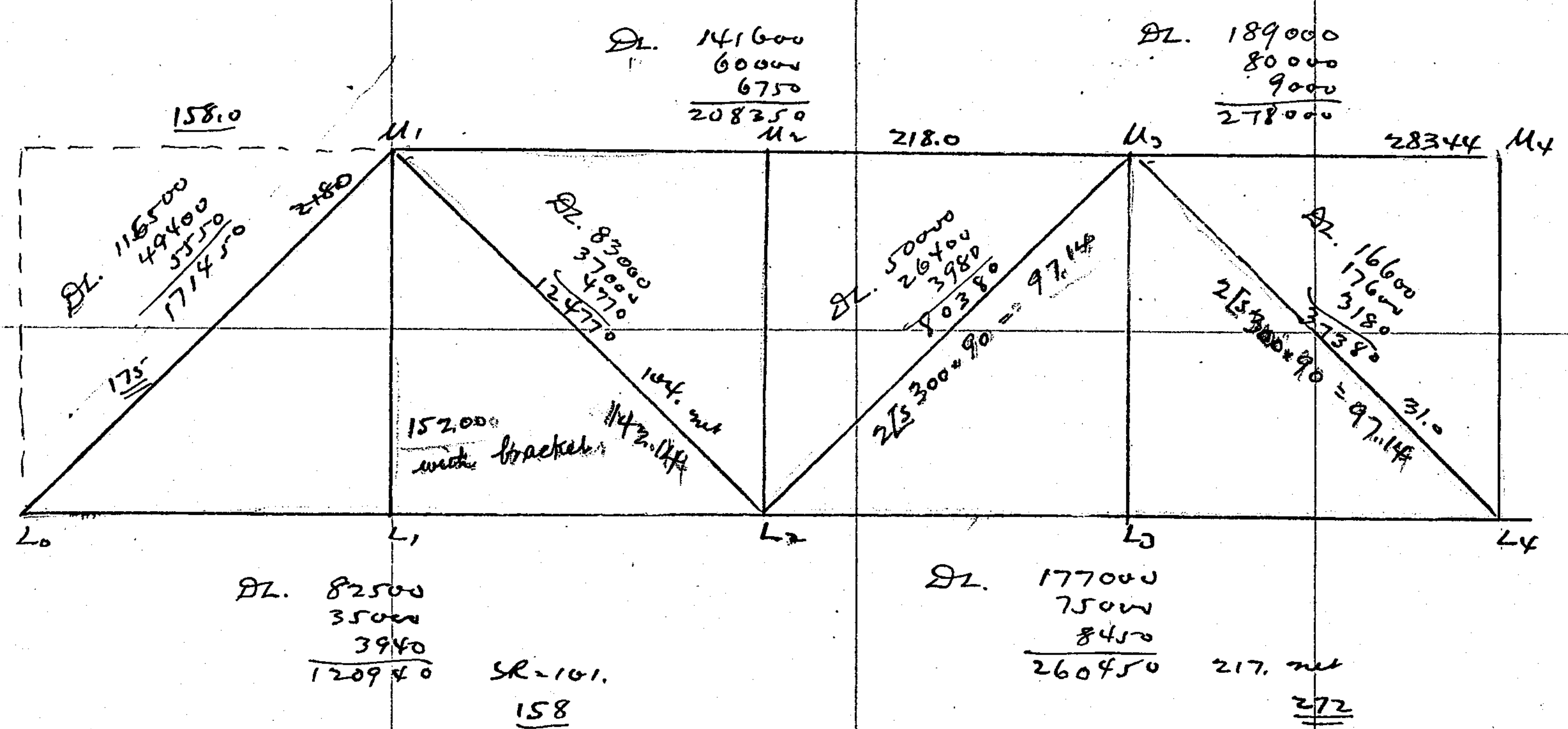
Lo-U1	10000	× 3.5 × 1.41	=	49400
Lo-L2		× 3.5	=	35000
U1-U3		× 6.0	=	60.000
L2-L4		× 7.5	=	75.000
U3-U4		× 8.0	=	80.000
U1-L2	10000	× 21/8 × 1.41	=	37000
L2-U3		× 15/8 "	=	26400
U3-L4		× 10/8 "	=	17600

Extra Concentration 4500 at panel point.

Lo-U1	4500	× 7/8 × 1.41	=	5550
Lo-L2		× 7/8 ×	=	3940
U1-U3		× 6/8 × 2	=	6750
L2-L4		× 5/8 × 3	=	8450
U3-U4		× 4/8 × 4	=	9000
U1-L2	4500	× 6/8 × 1.41	=	4770
L2-U3		× 5/8 "	=	3980
U3-L4		× 4/8 "	=	3180



Top chord section

Cov plate	600 × 10	=	60.0
2Pls.	410 × 10	=	82.0
4Ls	100 × 100 × 10	=	76.0
			218.0
Cov. Plate	600 × 10	=	60.0
2Pls.	410 × 10	=	82.0
4Ls	100 × 100 × 13	=	86.84
2Pls.	210 × 13	=	54.60
			283.44

Bottom chord section

2 Pls.	410 × 10	-	82.0	- 10	=	72.
4Ls	100 × 100 × 10		76.0	- 20	=	56
			158.0			128
2 Pls.	210 × 10	=	42.0	- 10	=	32
2 Pls.	360 × 10	=	72.0	10	=	62
			274			222
2Pls.	380 × 13	=	99.0			
4Ls	100 × 100 × 10	=	76.0			
			175.0			

4Ls	125 × 90 × 10	=	82.00
1Pl.	400 × 9	=	36.00
2Ls	90 × 90 ×		34.00
			152.00

2[s	300 × 90	=	97.14	- 22	=	75.14
2Pls.	250 × 9	=	45.00	- 9	=	36.00
			142.14			111.14

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara Bashi for Miye-Ken

Approximate weight of truss

158 × 3	@ .785	× 6.3	=	2340
218 × 2	@ "	× 6.3	=	2650
283.44	@ "	× 6.3	=	1400
272.0 × 2	@ "	× 6.3	=	2700
175	@	× 8.9	=	1220
142	@	× 8.9	=	1000
97.14 × 2	@	× 8.9	=	1350
152 × 4.5		× 6.3	=	3380

16040 × 2 = 32080

40% details 12800

44880 ÷ 50

for two trusses 89760 kg. ÷ 50.4 = 1780

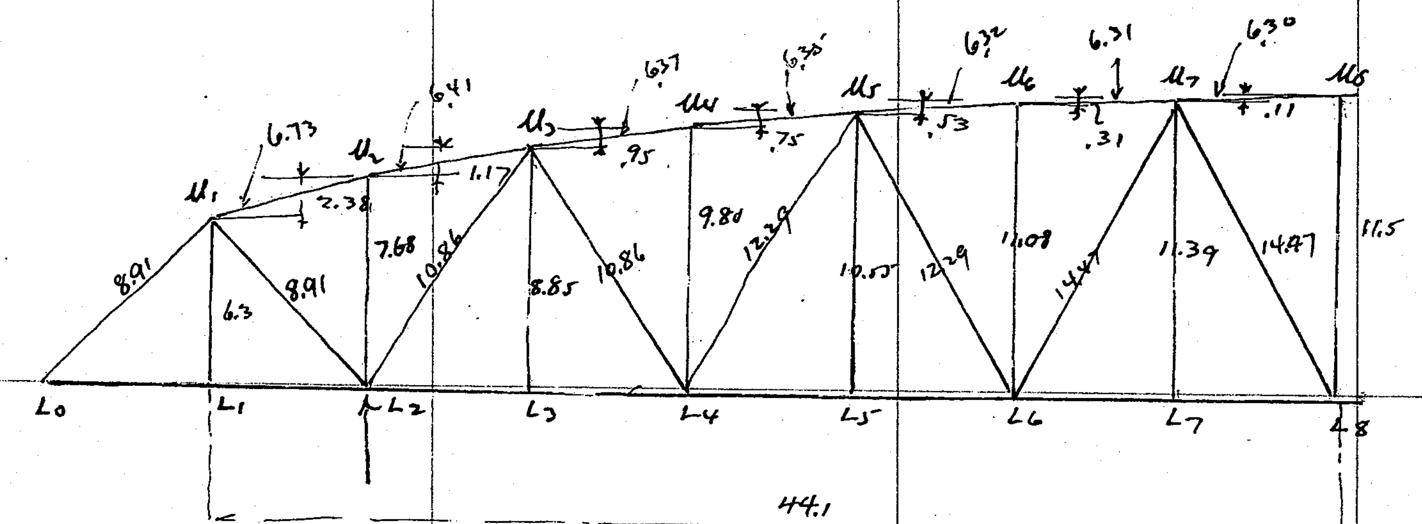
Approximate weight of structural steel in one span

stringers	475	× 50.4	=	24000
floor beams	1700 × 10		=	17000
lower laterals	110 × 50.4			5550
trusses				89760
misc. say				2000
				138310 kg.

$\frac{138310}{50.4}$ = 2740 kg per lin meter

$\frac{138310}{107.27}$ = 1.29 tons per tsubo

Design of Anchor Span



Lever Arms

U1-U2	7.68 × $\frac{6.3}{6.73}$	=	7.18
U2-U3	7.68 × $\frac{6.3}{6.41}$	=	7.54
U3-U4	9.80 × $\frac{6.3}{6.37}$	=	9.62
U4-U5	9.80 × $\frac{6.3}{6.35}$	=	9.70
~~U5-U6~~	~~11.08~~ × $\frac{6.3}{6.32}$	=	~~11.05~~
U6-U7	11.08 × $\frac{6.3}{6.31}$	=	11.08
U7-U8	11.50 × $\frac{6.3}{6.3}$	=	11.50

1 1
2 3
3 6
4 10
5 15
6 21
7 28
8 36
9 45
10 55
11 66

75.6
25.2
100.8

12.6

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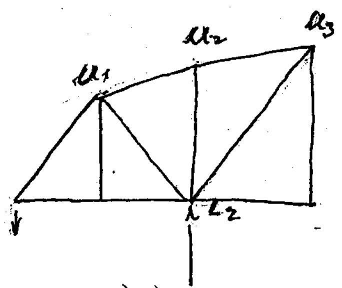
CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-ken



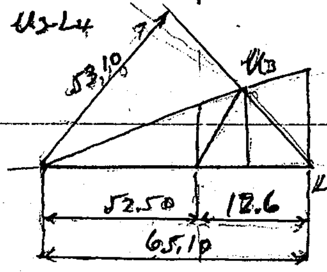
U1-L2 shear $= \frac{5.30}{7.68} = .69$

U1-L2 stress $= .690 \times 1.41 = .973$

L2-U3 shear $= \frac{8.85}{10.02} = .885$

stress $= .885 \times \frac{10.86}{885} = 1.086$

U3-L4

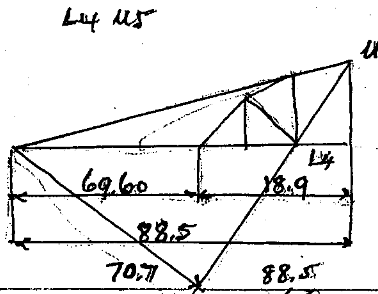


$x = 9.80 \times \frac{6.3}{.95} = 65.10$

65.10
12.60
52.50

$y = 65.10 \times \frac{8.85}{10.86} = 53.10$

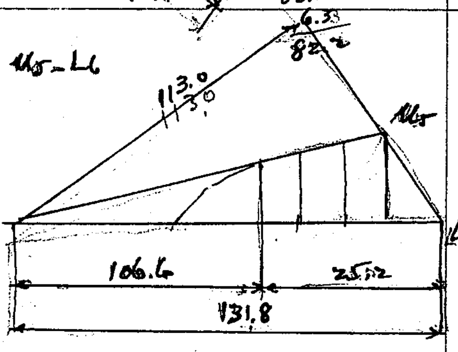
L4 U5



$x = 10.55 \times \frac{6.3}{.75} = 88.50$

$y = 82.2 \times \frac{10.55}{12.29} = 70.70$

U5-L6



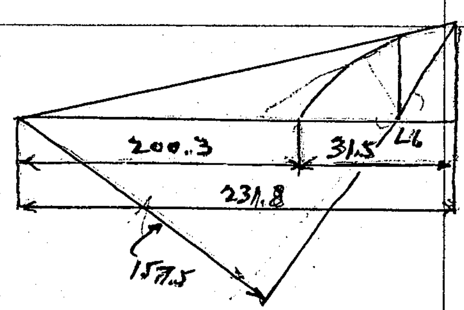
$x = 11.08 \times \frac{6.3}{.53} = 131.8$

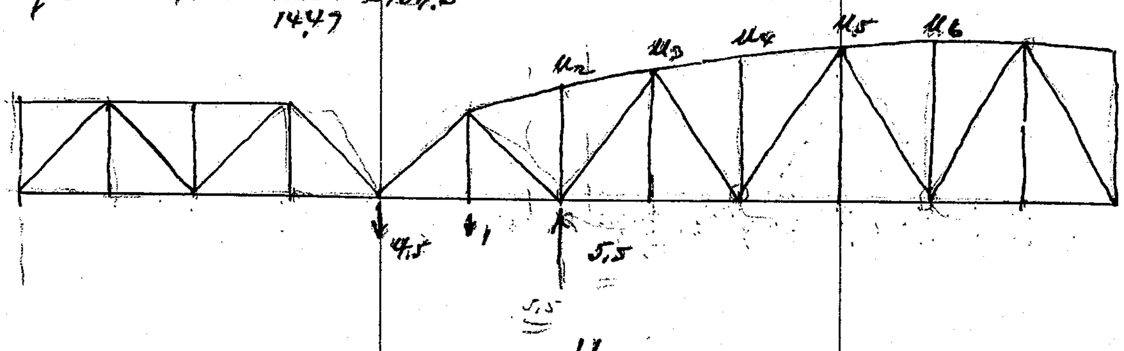
$y = 131.8 \times \frac{10.55}{12.29} = 113.0$

L6-U7

$x = 11.39 \times \frac{6.30}{.31} = 231.8$

$y = 200.3 \times \frac{11.39}{14.47} = 157.5$





Unit stress

Top chord stresses: —

$L_0U_1 = 4.50 \times 1.41 = 6.35$ Tension

$U_1U_2 = 4.50 \times 2 = 9.0$

$1.00 \times 11 = 1.0$

$10.0 \times \frac{6.3}{7.18} = 8.80$ tension

$U_2U_3 = 10.0 \times \frac{6.3}{7.54} = 8.38$ — tension

$U_3U_4 = 11.0 \times 2 = 22$

$4.5 \times 4 = 18.0$

4.0 / 22.8   22 / 0

$U_4U_5 = 0.0$

$U_5U_6 = 11.0 \times 4 = 44$

$4.5 \times 6 = 27.0$; 11.0 / 38.0

38

$6.0 \times \frac{6.3}{11.05} = 3.42$ C

U_6U_7 ———— $6.0 \times \frac{6.3}{11.08} = 3.41$ C

$U_7U_8 \quad 11 \times 6 = 66.0$

$4.5 \times 8 = 36$; 22 / 58

58

$8.0 \times \frac{6.3}{11.5} = 4.38$ C

11.

Lower chord stresses: —

$L_0L_2 \quad 4.5 \times \frac{6.3}{6.3} = 4.50$ Compression

$L_2L_4 \quad 11 \times 1 = 11$

4.5×3 2. / 13.5 / 15.5   15.5

$4.5 \times \frac{6.3}{8.85} = 3.20$ Comp

$L_4L_6 \quad 11 \times 3.0 = 33.0$

4.5×5 7.0 / 22.5 / 29.5   29.5

$3.5 \times \frac{6.3}{10.55} = 2.09$ Tension

$L_6L_8 \quad 11 \times 5.0 = 55$

$4.5 \times 7 =$ 16.0 / 31.5 / 47.5   47.5

$7.5 \times \frac{6.3}{11.39} = 4.15$ tension

CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-ken

Diagonal stress.

U_1-L_2	.973 × 5.5	= 5.35	Comp	
L_2-U_3	1.086 × 5.5	= 5.97	Comp	
U_3-L_4	52.50 × 11.0 = 577.5 58.80 × 1.0 = 58.80 46.20 4.5 × 39.9 = 179.50	−284.5 293.0 × 5310	= 5.52 Tension	284.5 58.8 225.7
L_4-U_5	69.60 × 11.0 63.3 75.9 82.2 4.5 × 57.0 = 256.5 577.9	= 765.0 477.9 287.1 70.7	= 4.05 Comp	63.3 256.5 319.8
U_5-L_6	106.6 × 11 = 4.5 × 94.0 = 422.5 100.3 112.9 119.2 125.5 880.4	1173.0 880.0 293.0 113.0	= 2.60	522.5
L_6-U_7	200.3 × 11 4.5 × 187.7 = 845.0 194.0 206.6 212.9 219.2 225.5 1903.2	= 2203.3 1903.2 300.1 ÷ 157.5	= 1.91	845.0 1.94 1639
U_7-L_8	$0.5 \times \frac{14.47}{11.5}$	= 0.63.		

Dead Load stress. Panel concentration say 3,750 × 6.3 = 23600

chord stress.

L_0-U_1	6.35 T	@ 23600	=	151,000
U_1-U_2	8.80 T		=	208000
U_2-U_3	8.38 T		=	198000
U_3-U_4	0			—
U_4-U_5	0			—
U_5-U_6	3.42 C		=	81000
U_6-U_7	3.41 C		=	81000
U_7-U_8	4.38 C		=	103500
L_0-L_2	4.5 C			106000
L_2-L_4	3.20 C			75500
L_4-L_6	2.09 T			49500
L_6-L_8	4.15 T			98000

U_1-L_2	5.35	C	23600	= 126,000
L_2-U_3	5.97	C		= 141,000
U_3-L_4	5.52	T		= 130,000
L_4-U_5	4.05	C		= 95600
U_5-L_6	2.60	T		= 61500
L_6-U_7	1.91	C		= 45000
U_7-L_8	0.63	T		= 15000

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-Ken.

Live Load stresses.

Concentration. 411 × 7.0 × 6.3 = 181 00
for one truss = 9000
including motor truck say 1100 for convenience sake

chord stresses $\frac{1100}{2360} = 46.5\%$

stresses due to negative moment.
cantilever arm. 46.5% of Dead Load.
center span both arm loaded
moment = 4.5 × 2 = 9.00
1.00

U1-U2	7.16		= 9670 T
U2-U3	63.0 × 1100 / 7.54		= 9200 T
U3-U4	9.62		= 7200 T
U4-U5	9.70		= 7150 T
U5-U6	11.05		= 6270 T
U6-U7	11.08		= 6260 T
U7-U8	11.50		= 6020 T
L2-L4	8.85		= 7800 C
L4-L6	10.55		= 6550 C
L6-L8	11.39		= 6100 C

10.0 × 6.3 = 63.0

L2-L4 5.5 × $\frac{6.3}{8.85}$ = 3.92
U3-U4 10 × $\frac{6.3}{9.62}$ = 6.55
10 × $\frac{6.3}{9.70}$ = 6.50
L4-L6 13.5 × $\frac{6.3}{10.55}$ = 8.06
U5-U7 16.0 × $\frac{6.3}{11.05}$ = 9.12
" 16.0 × $\frac{6.3}{11.08}$ = 9.10
L6-L8 17.5 × $\frac{6.3}{11.39}$ = 9.70
U7-U8 18.0 × $\frac{6.3}{11.50}$ = 9.85

chord stresses due to load between pins

L2-U3	6.77 × 1100	=	7450 C
U3-U4	6.55	=	7200 C
U4-U5	6.50	=	7150 C
U5-U6	9.12	=	10000 C
U6-U7	9.10	=	10100 C
U7-U8	9.70	=	10700 C
L2-L4	3.92	=	4310 T
L4-L6	8.06	=	8900 T
L6-L8	9.85	=	10850 T

Diagonal Stresses load on one arm

Reaction. 4.5 × $\frac{14}{12}$ = 5.25
$\frac{13}{12}$ = 1.08
6.33
− 5.50
.83
$\frac{66}{12}$ 5.5
L2-U3. 6.33 × 1.23 = 7.8

U3-L4. $\frac{55}{12}$ 6.33 + 4.58 = 10.91 × 52.5 = 572.0 − 225.7 = 346.3 ÷ 53.10 = 6.52

U4-U5. $\frac{45}{12}$ = 6.33 + 3.76 = 10.09 × 69.60 = 704.0 − 319.8 = 384.2 ÷ 70.7 = 5.43

U5-L6. $\frac{36}{12}$ 6.33 + 3. = 9.33 × 106.6 = 992.0 − 522.5 = 469.5 ÷ 113 = 4.15 4.50 about

L6-U7 $\frac{28}{12}$ 6.33 + 2.34 = 8.67 × 200.3 = 1740.0 − 1039. = 701.0 ÷ 157.5 = 4.45 3.45 about

U7-L8. $\frac{21}{12}$ = 6.33 + 1.75 = 8.08 − 5.5 = 2.58
2.58 × $\frac{14.47}{11.39}$ = 3.28

L2-U3	7.8	× 1100	= 8600
U3-L4	6.52		= 7170
U4-U5	5.43		= 5970
U5-L6	4.50		= 4950
L6-U7	3.45		= 3800
U7-L8	3.28		= 3600

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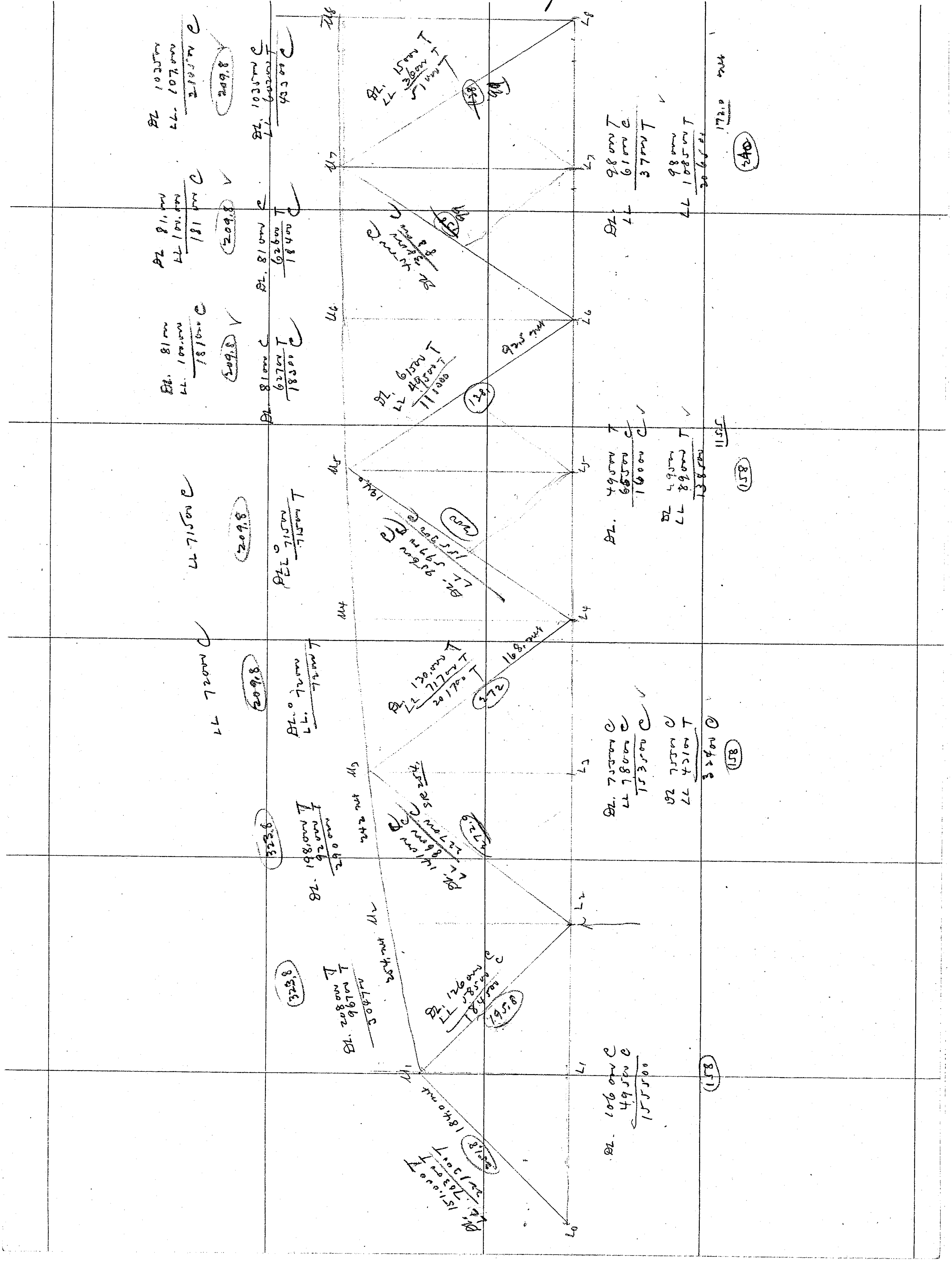
CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-Nagara-Bashi for Miye-Ken



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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-ken

Top chord section.

cov plate	600 × 10	= 60.0 − 10 = 50.0	
2Pls.	410 × 9	= 73.8 − 9 = 63.8	
4Ls	100 × 100 × 10	= 76.0 − 20 = 56.0	
		209.8	160.8
2Pls.	210 × 10	= 42.0 − 10 = 32.0	
2Pls.	360 × 10	= 72.0 − 20 = 52.0	
		323.8	244.8

20 / 4 / 24 60 / 24 / 36

Bottom chord

2Pls.	410 × 10	82.0	− 10 =	72
4Ls	100 × 100 × 10	76.0	− 20 =	56
		158.0		128
2Pls.	210 × 10	42.0	− 10 =	32
	360 × 9	72.0 − 18	=	54
		272.0		214

158 − 128
82 − 72
240 200

$U_1 - L_2$. length. 8.91

r = 0.36 × 41 = 14.8 l/r = 60

= 0.45 × 36 = p = 1500 (1 − 0.0055 × 60) = 1000

2Pls.	410 × 10	=	82.0
4Ls	100 × 100 × 10	=	76.0
2Pls.	210 × 9	=	37.8
			195.8

$L_2 - U_3$ — $1500\left(1 - 0.0055\,\frac{1086}{14.8}\right) = 895$ SR 254.0

$U_3 - L_4$ $1500\left(1 - .0055\,\frac{1229}{14.45}\right) = 800$ SR = 194

2[380 × 100 × 13 69.39 = 138.78

2Pls 320 × 10 = 62.00

200.78

approximate weight.

$L_0 - U_1$	251.8	@ .785	×	8.91	=	1750
$U_1 - U_3$	323.8	@ 〃	×	13.14	=	3340
U_3 —	209.8	@	×	32.00	=	5260
$L_0 - L_6$	158.0	@ .785	×	37.8	=	4700
$L_6 - L_8$	240.	@	×	12.6	=	2370
$U_1 - L_2$	195.8		×	8.91	=	1370
$L_2 - U_3$	270			10.86	=	2300
$U_3 - L_4$	270			10.86	=	2300
$L_4 - U_5$	200			12.29	=	1930
$U_5 - L_6$	138			12.29	=	1330
$L_6 - U_7$	99			14.47	=	1130
$U_7 - L_8$	99			14.47	=	1130
verticals	83.16	@ .785	×	72.4	=	4740
at L_0						1000
vert.						410
						35060 × 2 = 70120

46
56
29
58.6

Details 40% 28000

98120

two trusses 2

196240

98120 ÷ 104.8 = 9700

194.0

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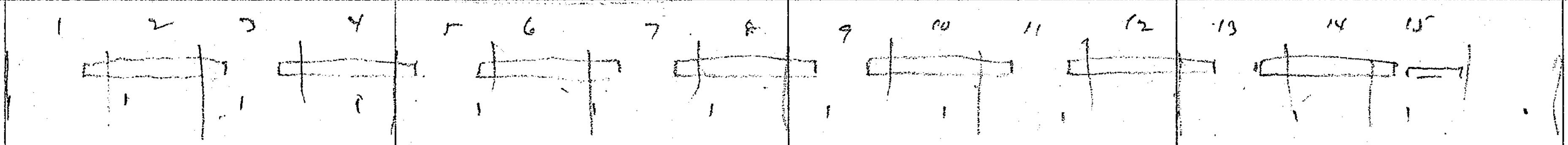
CALCULATIONS FOR

Preliminary Estimate of Cost Hi-nagawa Bashi for Miye-ken.

stringers	475 × 100.8	= 48,000	
floor beams		28,500	
lower laterals	120 × 100	= 12000	
Top laterals	330 × 88	= 29000	
trusses		196,240	
shoes	say	6000	
		319,740	320 tons.



anchor spans 7 @ 320. = 2240
8 @ 139 = 1110
3350

7 @ 315 = 2200
8 @ 120 = 960
3160

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-Ken

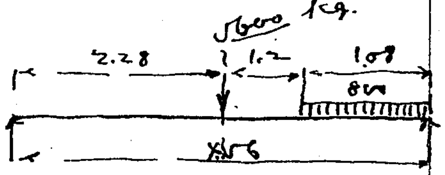
Stabbogen (Arch with stiffening truss)

span length 73.0 16@4.56 = 73.0

Cross section of bridge see page 1.

design of stringer span length 4.56 meters

Center stringer SA Uniform load $R_1 = \frac{800 \times 1.08^2}{2 \times 4.56} = 102.$



moment = 102 × 2.28 = 233.

2800 × 2.28 = 6390

6623

Dead Load 1/8 × 900 × 4.56² = 2340

8963 kgm

Section modulus required $\frac{896300}{1100}$ = 815.

use 350 × 150 × 9 @ 58.54 kg. section modulus 870.5

weight of stringers ~~62~~ × 5 = 310 kg per lin. meter

Design of floor beam span length 8.20 meter

Dead Load flooring 520 × ~~7.3~~ 4.56 = ~~3800~~ 2370

beam assumed 250 250

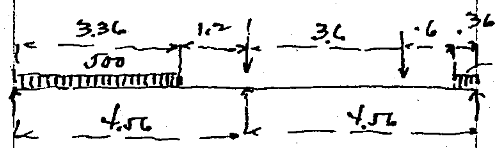
4050 2620

m = 1/8 × ~~4050~~ 2620 × 8.2² = 22000 kgm

Live Load motor truck loading 1300 × $\frac{.96}{4.56}$ = 273

3900

4173 kg.



neglected Uniform load $500 \times \frac{.36^2}{2 \times 4.56}$ = 71

$500 \times \frac{3.36^2}{2 \times 4.56}$ = 620

691

moment motor truck loading

2 × 4173 × 4.1 =

2 × 4173 × 1.35

2.75 = 23000

Unif load 1/8 × 700 (say) × 8.2² = 5880

28880 kgm

Dead Load moment 22000

50880 kgm

web assumed 900 × 9 = 81.0 1/8 web = 10.1 d = .863

flange stress = $\frac{50880}{.863}$ = 59000 SR = 49.2

10.1

39.1 net

2 L's 125 × 90 × 13 = 52.52 − 11.44 = 41.08 net.

52.52

81.00

186.04

weight of main section

168.04 @ .785 × 7.94 = 1050

Details say 350

1400 kg

End floor beam say 1400 − 200 = 1200 kg

stringers 310 × 74.0 = 23000

floor beams 15 × 1400 = 21000

2 × 1200 2400

misc 1000

25400 — 25400

48400 kg.

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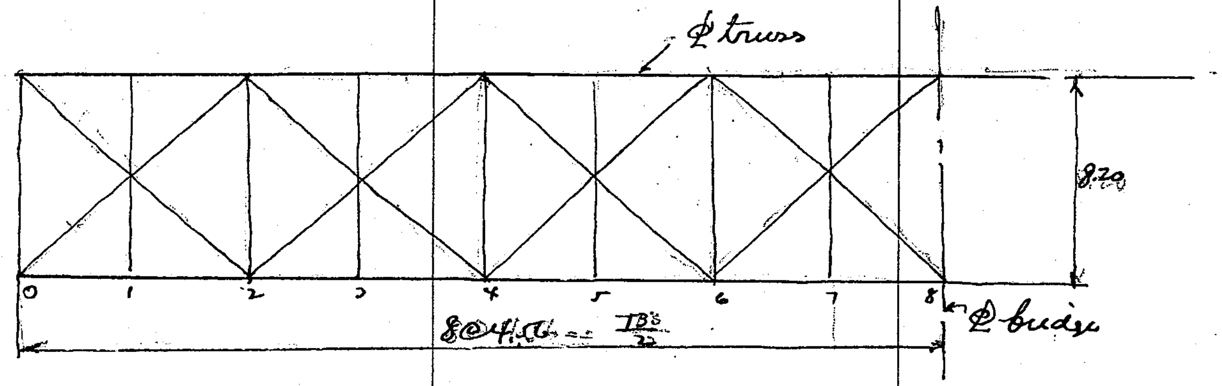
CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-Ken.

Lower Lateral Bracings.



$4.1^2 = 16.8$
$4.56^2 = 20.8$
37.6 — 6.1 meter
$\frac{6.1}{4.1} = 1.49$

Seismic force assumed 3000 mm/sec² 7500 × 0.3 = 2250 kg per meter
Panel concentration 2250 × 4.56 = 10250 kg

		shear	stress ×1.49		SR	
Panel	0-1	77000	115000	÷ 2160 =	53.2	2Ls 150×150×11 = 63.58 − 11.0 = 52.58 net
	1-2	66700	99500		46.0	do
	2-3	56500	84000		39.0	do
	3-4	46200	69000		32.0	2Ls 130×130×9 = 45.18 − 9.0 = 36.18
	4-5	36000	53700		24.9	2Ls 125×90×10 = 41.0 − 10.0 − 31.00
	5-6	25600	38200		17.7	do
	6-7	15400	23000		10.7	do
	7-8	5100	7600		3.5	do
	~~8-9~~					

Approximate weight of Lower Lateral.

4Ls 150×150×11 @ 24.95 × 5.8 = 580
Center connection say 50
630

4Ls 130×130×9 @ 17.73 × 5.8 = 412
38
450

4Ls 125×90×10 @ 16.09 × 5.8 = 373
37
410

Total weight
6 × 630 = 3780
2 × 450 = 900
8 × 410 = 3280
7960
misc say 440
8400 kg. 115 kg per lin. meter

Top Lateral Bracings
Diagonal length.
$5.2^2 = 27.0$
$4.1^2 = 16.8$
43.8 — 6.6 meter

Radius of gyration reqd $\frac{6.6}{150} = 4.4$ 2Ls 100×75×10 — r = 4.8 above

4Ls 100×75×10 @ 12.98 × 6.0 = 310
Details say 90
400 2@400 = 800

Diagonals	12 @ 800	= 9600 kg.
Strut. ~~Diagonals~~	6 @ 400	= 2400
Sway.	5 @ 1800	= 9000
Portal bracings.	2 @ 2200	= 4400
		25400

25400 ÷ 73.0 = 348.0

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Miye-ken.

main truss. stresses by graphic.

Dead Load panel Concentration 4.56 × 3750 = 17100 kg.

chord stresses

Lo-U1	12.35 C × 17100	= 211000 C		U1-L2	0.15 T × 17100	= 2560 T
U1-U2	13.75	= 235000 C		L2-M3	0.38 C	6500 C
U2-U3	13.2C	= 227000 C		M3-L4	0.55 T	9400 T
U3-U4	12.86	= 220.000 C		L4-M5	0.50 C	8500 C
U4-U5	12.58	= 215.000 C		M5-L6	0.39 T	6700 T
U5-U6	12.35	= 211.000 C		L6-M7	0.28 C	4800 C
U6-U7	12.23	= 210.000 C		M7-L8	0.17 T	2900 T
U7-U8	12.18 C	= 208.000 C				
Lo-L2	9.83 T	= 168.000 T		U1-M3	2.23 T	38.000 T
L2-L4	10.50 T	= 180.000 T		M3-M5	1.52 T	26.000 T
L4-L6	11.40 T	= 195.000 T		M5-M7	0.81 T	13800 T
L6-L8	12.03 T	= 206.000 T				

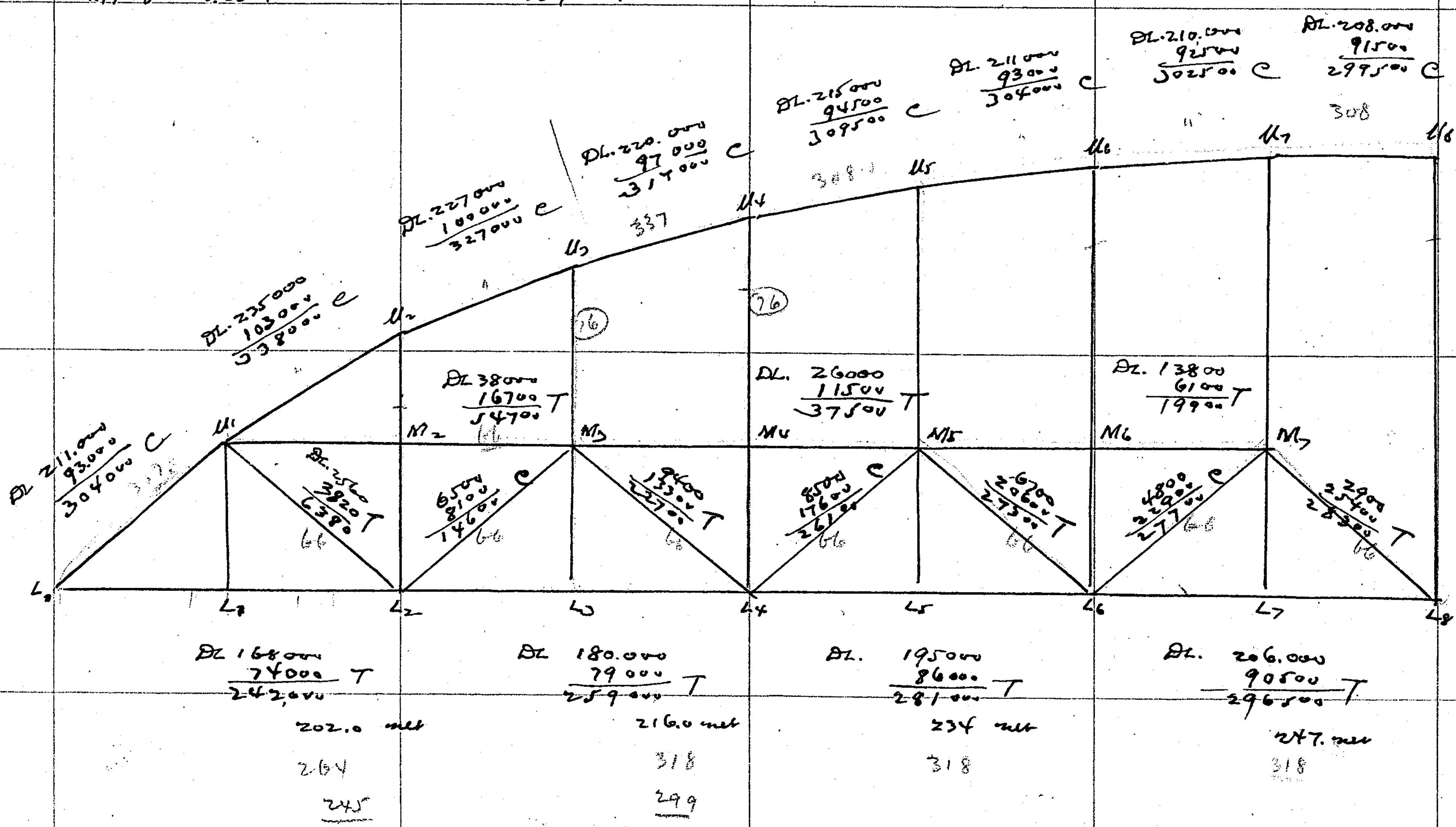
Live Load stresses :- Panel Concentration. 411 × 7.0/2 × 4.56 = 6550

assume 7500 including motor loading.

say 44 %. of DL. stress.

web stress

U1-L2	0.51 T	× 7500 =	3820 T
L2-M3	1.08 C		8100 C
M3-L4	1.78 T		13300 T
L4-M5	2.35 C		17600 C
M5-L6	2.75 T		20600 T
L6-M7	3.05 C		22900 C
M7-L8	3.25 T		25400 T



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Preliminary Estimate of Cost Ibi-nagara-Bashi for Miye-Ken.

Top chord members

1 cov. Pl. 650 × 13 = 84.50
2 Pls. 470 × 13 = 122.50
4 Ls 100 × 100 × 10 = 76.00
283.00
2 Pls. 270 × 10 = 54.
337 ✓

1 cov. Pl. 650 × 13 = 84.50 | 84.5
2 Pls. 470 × 10 9 = 94.00 | 84.5
76.00 | 76.0
54.00 | 54.0
308.00 ✓ | 299.0

Bottom chord members

2 Pls. 470 × 13 = 122.50 − 26 = 96.50
4 Ls 100 × 100 × 10 76.00 − 56.00
2 Pls. 270 × 10 = 54.00 − 10 44.00
196.50

2 Pls. 470 × 10 = 94.0 − 20 = 74.0
2 Pls. 470 × 10 = 94 74.0
4 Ls 100 × 100 × 10 = 76 56.0
2 Pls. 270 × 10 54.0
318.0 ✓ 204.0 ✓ net
44
248.0 ✓ net

2 Pls. 470 × 9 = 84.5 − 9 (18) = 75.5
84.5 − 9 = 75.5
76.0 − 20 = 56.0
245.0 207.0
54.0 − 10 = 44
299 251.0

318
54
264

Diagonal 4.56^2 = 208
3.50^2 = 12.25
33.05 − 5.75 meter.

Il required for diagonal $\frac{575}{120}$ = 4.79

2 Ls 100 × 75 × 10 @ 16.50 = 33.0 , 362^2 + 432 / 320 / 752 = 752

$r = \sqrt{\frac{752}{33}} = 4.77$

9
317
45
3.62

4 Ls 100 × 75 × 10 @ 16.50 = 66.0

middle Chord 54700 ÷ 1200 = 45.6 net.

4 Ls 100 × 75 × 10 @ 66.00 − 20 = 46.0 net.

verticals. 4 Ls 125 × 75 × 10 = 76.0 ~~net~~. gross

approximate weight of truss.

Top chord.	337	@ .785	× 21.2	= 5550	} 10390	− 9150		
	308	" "	× 20.0	= 4840				
bottom	260 245		× 9.12	= 1750 1890	} 8720	7110		
"	318 299.		× 27.40	= 6430 6830 8180				
middle	66	" .785	× 27.40	= 1420	} 3500	− 4540		
diagonal	66	"	× 5.75 × 7	= 2080				
verticals	76.	"	× 67.3	= 4020	4020	− 2960		

~~26630~~ × 2 = 53260
38% details. 20200
73460
× 2
146920 ≒ 147 tons

26090 × 2 = 52180
38% 19800
71980 72000
× 2
144.0 tons

164

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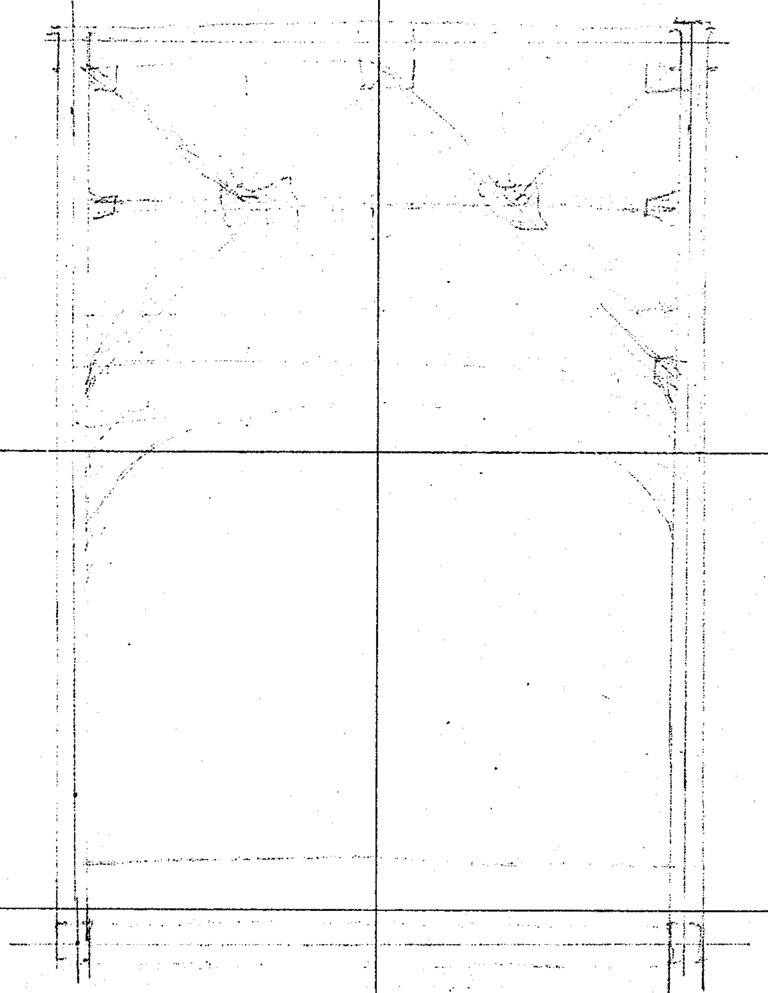
CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara Bashi for Mieye-Ken

approximate weight of steel

stringers	23000
floor beams	25400
bottom Laterals	7960
Top Laterals	25400 - - 3000
trusses	144000
shoes	4500
	230260 227.00

~~230~~ 227 × ~~15~~ = ~~3450~~ 3400 tons.



Diagonal

2 Ls 100 × 75 × 10 @ 12.95 × 6 = 155
1 Pl. 250 × 9 @ 17.66 × 6 = 106
gusset plate 25
286

say 290 × 2 = 580

12 @ 580 = 6950. kg.

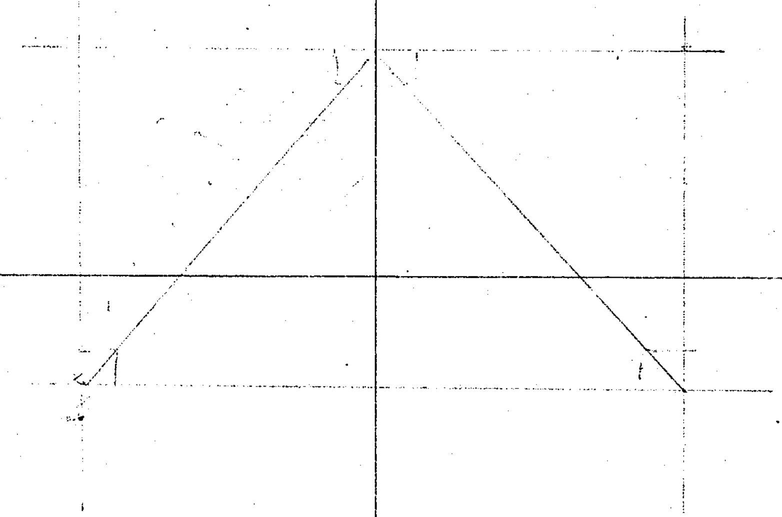
Strut.

4 Ls 100 × 75 × 10 @ 12.95 × 7.80 = 405
3 tie plates 470 × 9 @ 33.2 × 4.70 47
connection Ls 25
Lacings 14 kg. 6.5 = 91
568

570 × 6 = 3420

Sway. 2 Ls 125 × 90 × 10 @ 16.09 × 24 = 772
plate 9 @ 15 178
strut. 568
1458

Call this 1500 × 5 = 7500



Summary.

Diagonal.		6950
~~strut~~		3420
Sway	5 @ 1500	7500
portal.	2 @ 2200 =	4400
		22270
	say	22400.

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DL Reaction 7.5

moment at L_8

$7.5 \times 36.5 = 273.7$

$1 \times 4.5625 \times 28 = 127.7$

146.0

$H = \frac{146.0}{12.0} = 12.17$

LL, $M_1 - L_2$

$R = 1 \times 4.5625 \times \frac{105}{73} = 6.56$

m at L_7

$6.56 \times 36.5 = 239.5$

$1 \times 4.5625 \times 21 = 95.8$

$143.7 \div 12 = 11.97$

6.56
6.25
$.31$

12.17
$.14$
12.03

$M_7 - L_8$ $R = \frac{36}{16} = 2.25$

$2.25 \times 36.5 = \frac{82.1}{12} = 6.84$

2.25
0.30
1.95

$\frac{55}{16} = 3.44$

$3.44 \times 36.5 = 125.5$

$4.5625 \times 3 = 13.7$

$\frac{111.8}{12} = 9.32$

3.44
1.77
1.67

$L_2 - M_3$

$R = \frac{1 \times 91}{16} = 5.69$

$5.69 \times 36.5 = 207.7$

$1 \times 4.5625 \times 15 = 68.4$

$\frac{139.3}{12} = 11.62$

5.69
5.04
$.65$

$M_3 - L_4$

$R = \frac{78}{16} = 4.88$

$4.88 \times 36.5 = 178.2$

$1 \times 4.5625 \times 10 = 45.6$

$\frac{132.6}{12} = 11.06$

4.88
3.8
1.08

$L_4 - M_5$

$R = \frac{66}{16} = 4.12$

$4.12 \times 36.5 = 150.5$

$4.5625 \times 6 = 27.4$

$\frac{123.1}{12} = 10.27$

4.12
2.70
1.42

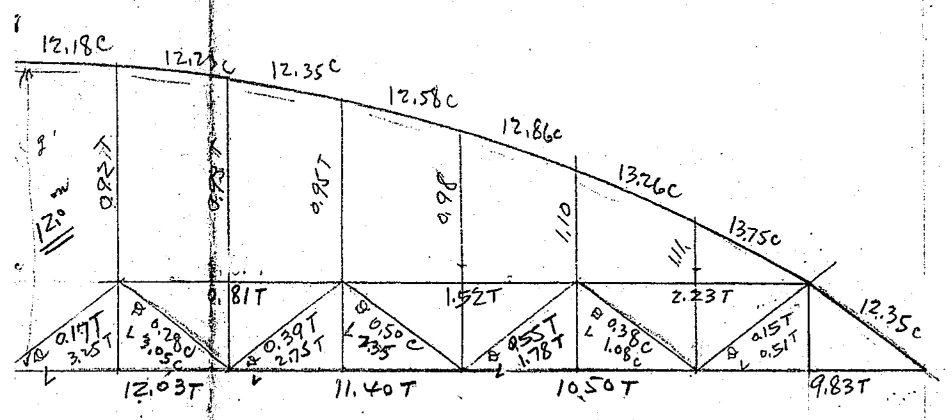
$R = \frac{45}{16} = 2.81$

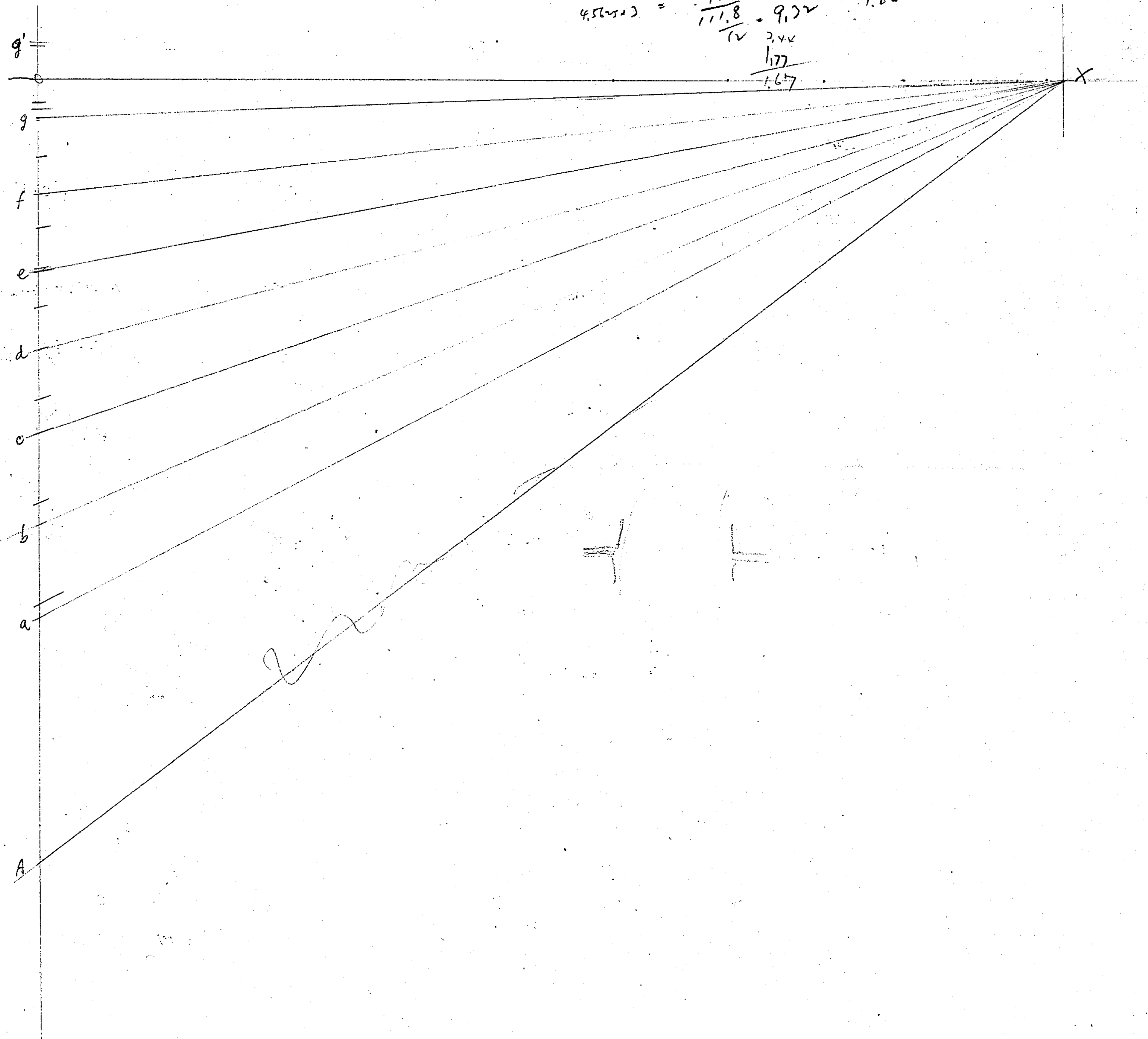
$2.81 \times 36.5 = 102.7$

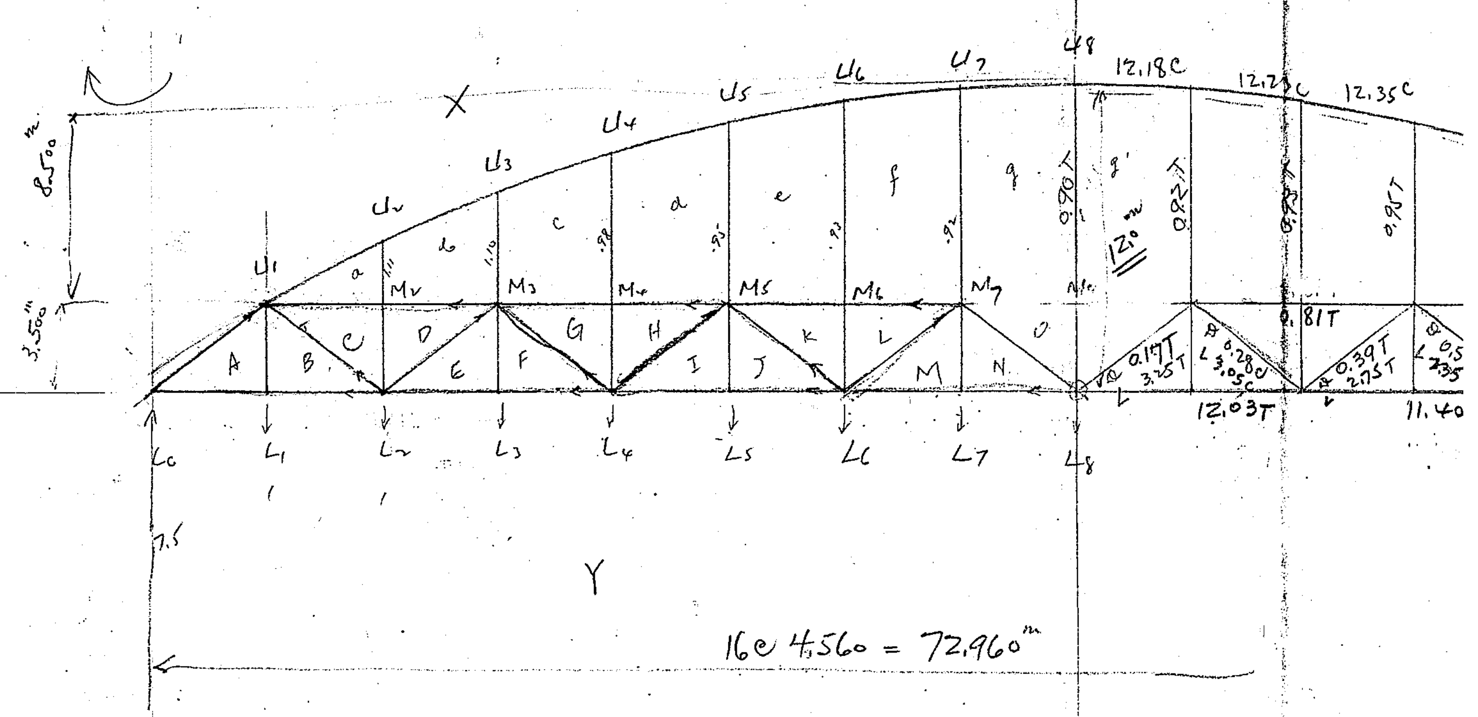
4.6

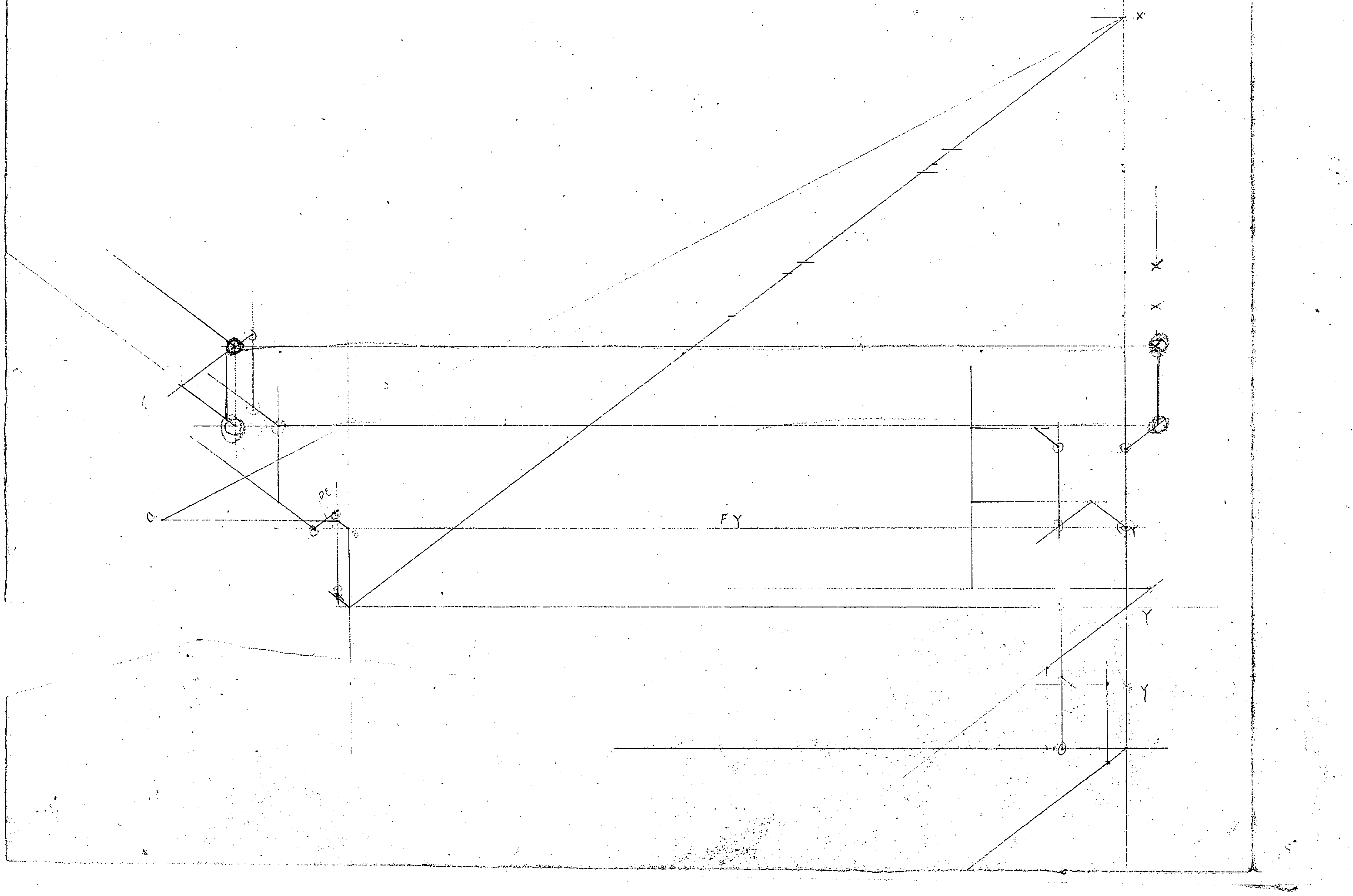
$\frac{98.1}{12} = 8.18$

2.81
0.95
1.86






X
U_1 U_2 U_3 U_4 U_5 U_6 U_7 U_8
M_2 M_3 M_4 M_5 M_6 M_7
L_0 L_1 L_2 L_3 L_4 L_5 L_6 L_7 L_8
A B C D E F G H I J K L M N
a b c d e f g g'
8,500
3,500
12,18C
12,35C
0.90T
0.81T
0.39T
12,03T
Y
16@4,560 = 72,960


FY
Y
Y

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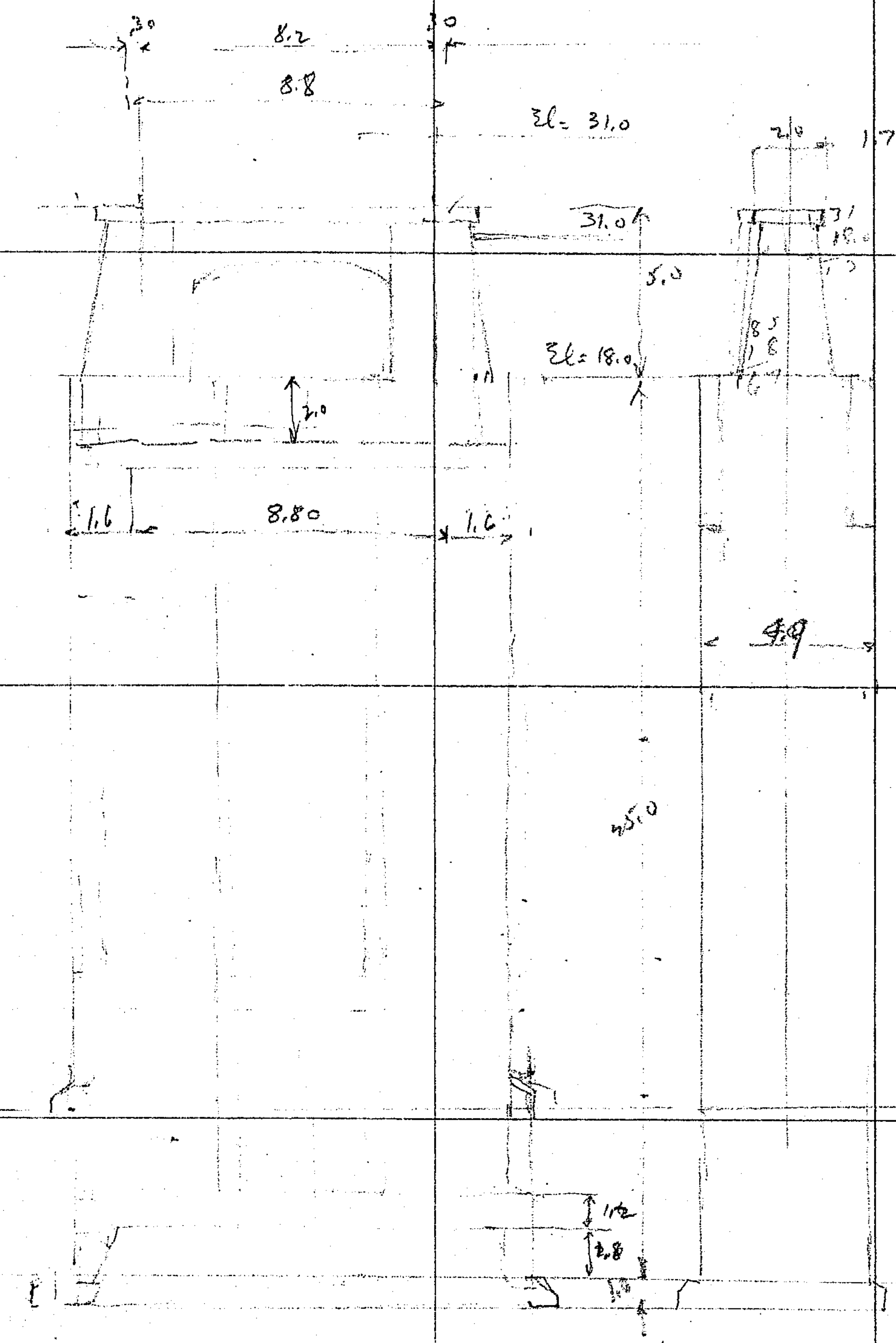
CALCULATIONS FOR

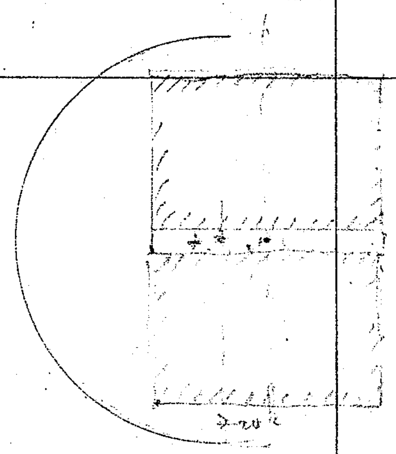
Preliminary Estimate of Cost Hei-nagara Bashi for Mie-ken

superimposed load
one truss DL. 3750
LL. 1450
5200 × 74/2 = 200,000

bearing on concrete 200,000/450,000 = .444 sq. meter

60 × 90
70 × 80



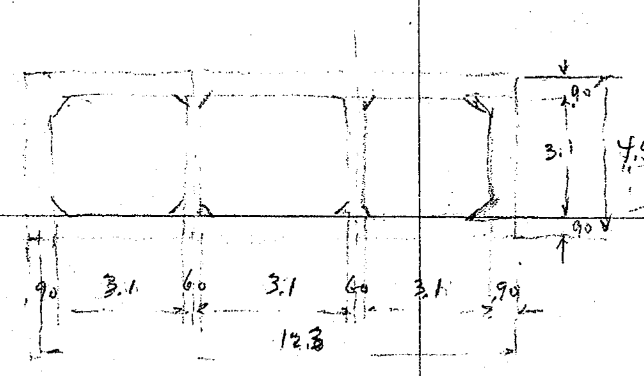


8.80
4.50
13.30
3
10.3 | 3
9 | 3.11
13

1.25
9.0
2.15

12.0
8.8
3.2
1.6

2.9
.3
8.7
3
11.7
8.8
2.3



4.5
1.8
2.7

1.8
1.2
3.0

Concrete in shaft.

1.7^2		2.27
1.7 × 8.8 =		14.90
		17.17
2.5^2 =		4.9
2.5 × 8.8		22.0
		26.9
		17.2
		44.1

22.1 × 5 = 110 cubic m

Caisson area = 12.3 × 2 × .9 = 22.2
3.1 × 2 × .9 = 5.60
3.1 × 2 × .6 = 3.7
31.5
.60 .36 × 6 2.2
33.7 × 21.0 = 708.

60.2
33.7
26.5

Top filling = 4.9 × 12.3 × 2 = 120
bottom = 120
6.9 × 14.3 = 100
220

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CALCULATIONS FOR

Preliminary Estimate of Cost Ibi-nagara-Bashi for Miye-ken

Concrete 110 @ 2400 = 264,000

710, 120, 220 } 1050 @ 2200 = 2310,000

1050
110
1160

water. 26.5 × 21. = 556 556.000

3130.000

superimposed load 200. × 4 800.000

3930.000

area of base = 99 Unit bearing = 39.700 kg/m² 3.7 tons/□'

Skin friction: $\frac{300 \times 3.28^2}{2.2}$ = 1470

4.9 × 2 = 9.8
12.3 × 2 = 24.6
34.4

$\frac{350 \times 3.28^2}{2.2}$ = 1710 × 34.4 = 58.50 tons per lin meter

58.50 × 18 = 1050.000

Counting friction. 3930.000
1050.000
2880.000 Unit bearing 2910 kg/m² 2.7 tons/□'

area of forms.
34.4
32 × 12 = 38.4
72.8 × 23.0 = 1680 sq meters
22 × 5 = 110
1790 sq meters

approximate steel in caisson.
span l.... 4.0 Pressure. 20 × 1700 × 1/3 = 11300 kg.

m = $\frac{1}{12}$ × 11300 × 4² = 15100 kgm

Reinf. bars. $\frac{1510000}{7/8 \times 80 \times 1200}$ = 18.0

19mm bars. 2.83 6.35 15 cm centers.

verticals 25 4.90 60本 @

Reinforcing bars 45 tons

Curb shoe ~~4~~ tons
~~timber~~ ~~extra~~ ~~work~~ 3' thick
2 × 12.3 × .90 = 22.2
2 × 3.1 × .90 = 5.6
27.8 × 2 = 55.6

Ceiling = 1.0 × 12.3 × 4.9 = 60.0
extra 30.0
145.6 say 150 cubic meters.

$\frac{5400}{12}$ = 450 150 × 36 = 5400 cubic ft.

Concrete.	1160 cubic meters	@	$17\underline{00}$	=	19700
Reinf.	45 ton	@	130	=	5850
form	1500 sq meter -	@	3^{00}	=	4500
cutting Edge	$\underline{4}$ ton	@	200	=	800
~~timber~~ crib -					12000
Excavation -	1570 cubic m	@	5^{50}	=	8650
					51500
					25000
					76500

1500
250
500
400
2200
5000
3000
1000
700
600
2500
5000
500
2000
25150 say 25000 =

14 @ 76500 = 1070000
2 @ 25,000 = 50,000
1120,000 -

flooring - Concrete. .15 × 7.0 = 1.05
1.15
1.2 × $\underline{74}$ = 90. Cubic meters

Concrete	90.0 cubic m	@	17.00	=	1530
Reinf.	11.5 tons @		150^{00}	=	1730
forms.	68.0 sq meter.		2^{50}	=	1700
pavement.	518 @		4^{75}	=	2460
Handrails.	10.5 tons @		320	-	3360
misc.					" 200
					10980

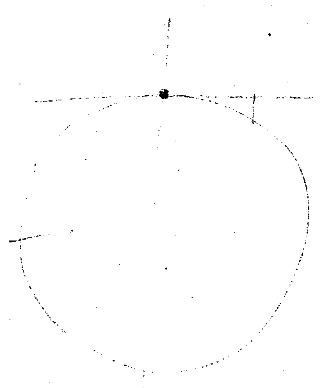
21.2 yen/m²

$\frac{10980}{157}$ = $\underline{70}^{00}$

Electric wiring &c. → 720

11700

Deck construction 15 @ 11700 = 177.000
Substructure
14 @ 76500 } = 1120,000
2 @ 25,000 }
steel - 3400 @ $2\underline{55}$ 870.000
2167 000
misc expense 6% 133 000
2300,000 -



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$x^2+y^2=R^2$

$x^2+(R+y)^2=R^2$

$x^2+\cancel{R^2}+2Ry+y^2=\cancel{R^2}$

$x^2+2Ry+y^2=0$

$R=64.1844$

$x^2+128.3688y+y^2=0$

$x=31.92,\quad y=-8.5$

$1019.5-109+72.2=$

$\cancel{x^2+y+y^2}$

$\sqrt{b^2-4ac}$ / $2a$

$x^2-3x+2=0$

$11,155.39\ (105.62$

52.81

64.184
-52.81
11.374

116.994

$128.3688^2=16,478.55$
-5323.16
11155.39

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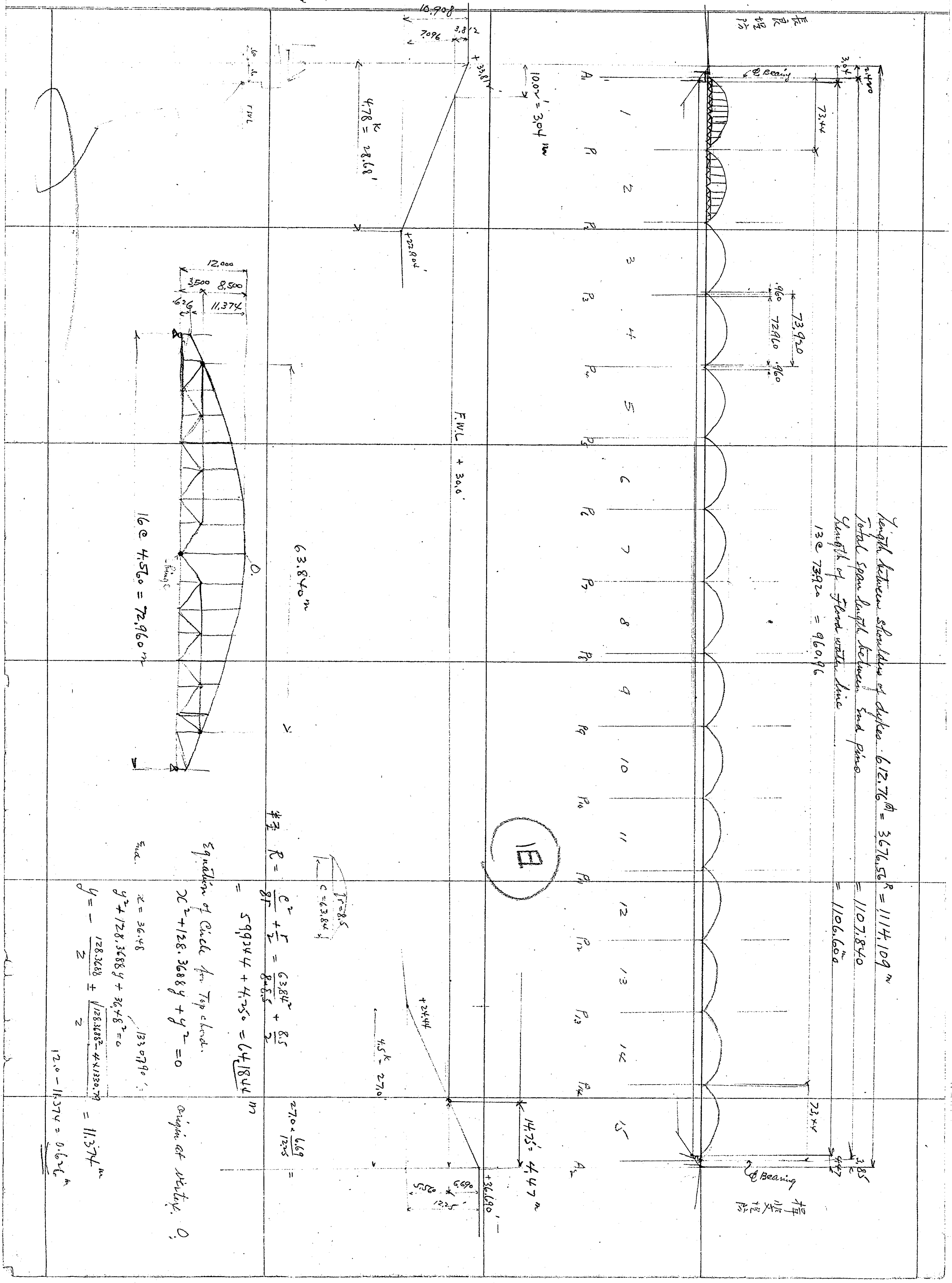

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（独立行政法人 土木研究所蔵）

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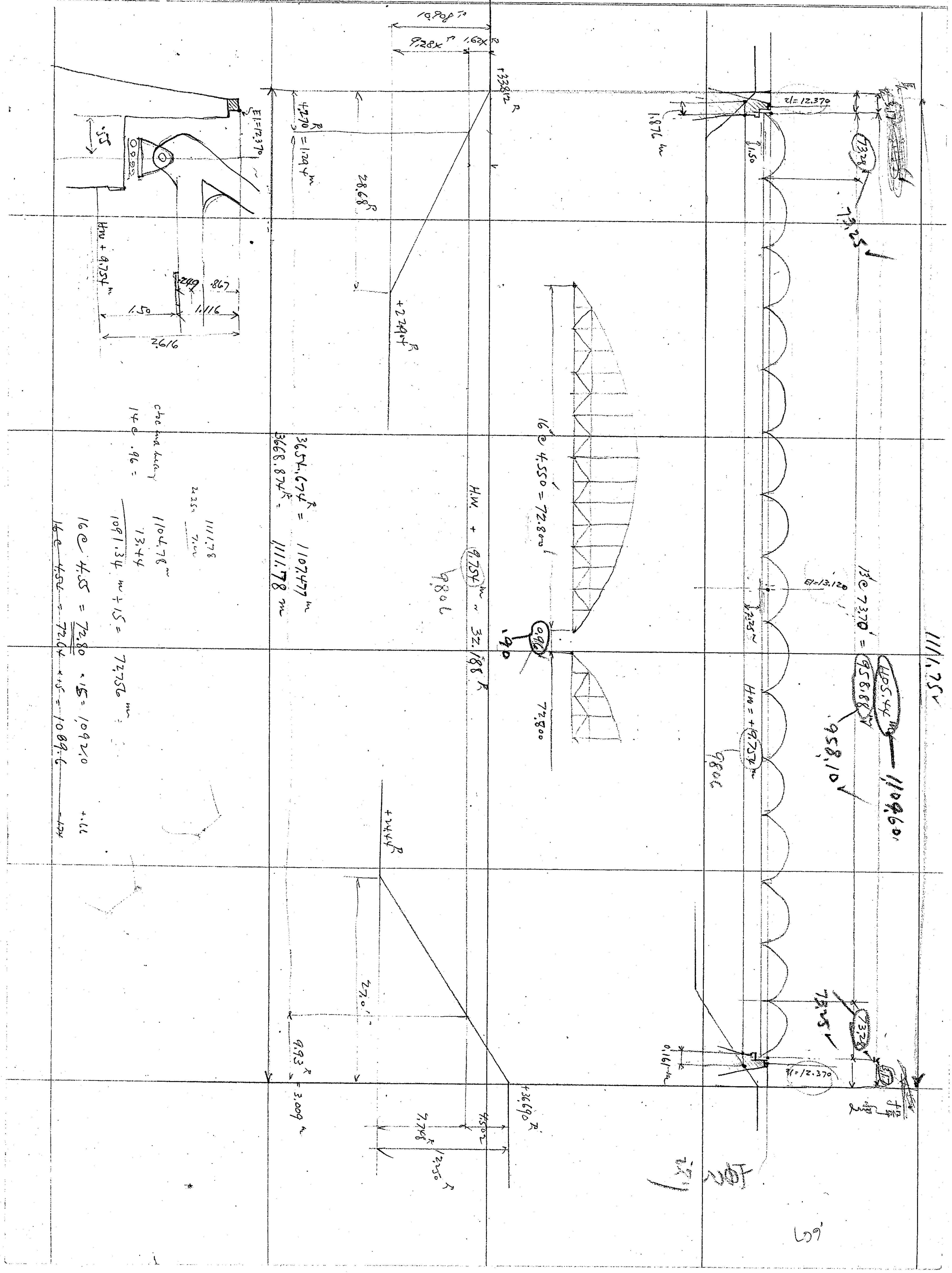
CALCULATIONS FOR



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三
圖
係

計画洪水位　満水位

揖斐川　三七尺三

長良川　三〇八〇

九・八〇尺

基準點ハ東京湾中等潮位

3.713
3.050
.663

9.091
663
9.754

(11)

[illegible]常願寺川[illegible]基線[illegible]測量ノ精密[illegible]
[illegible]
[illegible]技手[illegible]二十日間[illegible]
[illegible]二回測量[illegible]
[illegible]

測量ノ方法

三回ノ「ゲーヂ」及「ボックスゲーヂ」ニテ各回測定シ温度及張力
ノ更正ヲナシタリ「ボックスゲーヂ」[illegible]
[illegible]（[illegible]）

1. 温度ノ更正

$$\Delta t = +\,a\,(T_m - T_0)\,L$$

Δt = 温度ニヨル更正長 (cm.)
L = 実測長 (cm)
T_0 = 20°C 標準温度
T_m = 測定時温度 (C.)
a = 0.0000117 鋼鉄巻尺ノ膨張係数

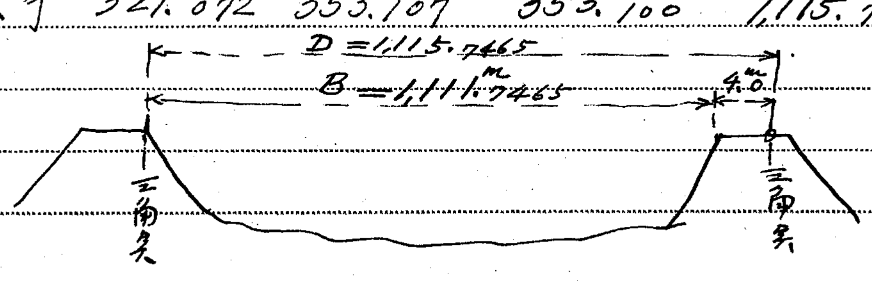
(12)

2. 緊張力ノ更正

$$\Delta p = +\frac{P}{S \cdot E} L$$

Δp = 緊張力ニヨル更正長 (cm)
E = 2,000,000 kg/sq.cm. 鋼鉄製巻尺ノ弾性係数
L = 実測長 (cm.)
S = 0.02 cm^2 鋼鉄製巻尺ノ断面積 (cm^2)
P = 基線測定ニ於ケル緊張力

測量者	基線	検査基線	三角測量ヨリ算出ノ検査基線	D	B	温度
西部技手	321.073 m	353.106 m	353.144 m	1,115.812 m	1,111.812 m	20°C
谷技手	321.072 m	353.107 m	353.100 m	1,115.7465 m	1,111.7465	20°C



average 1,111.779 m

尚ニ [illegible] 水平なる板の上にて 24°C の時測定したる
値にして [illegible] 基線及検査基線の長は

	[illegible]	[illegible]	[illegible]
基線	三三一.〇七三米	三三一.〇七〇	三三一.〇八三
検査基線	三五二.一〇七	三五三.〇八四	三五二.〇九九

右の結果は [illegible]
[illegible]

五月十八日 [illegible]

増田淳様

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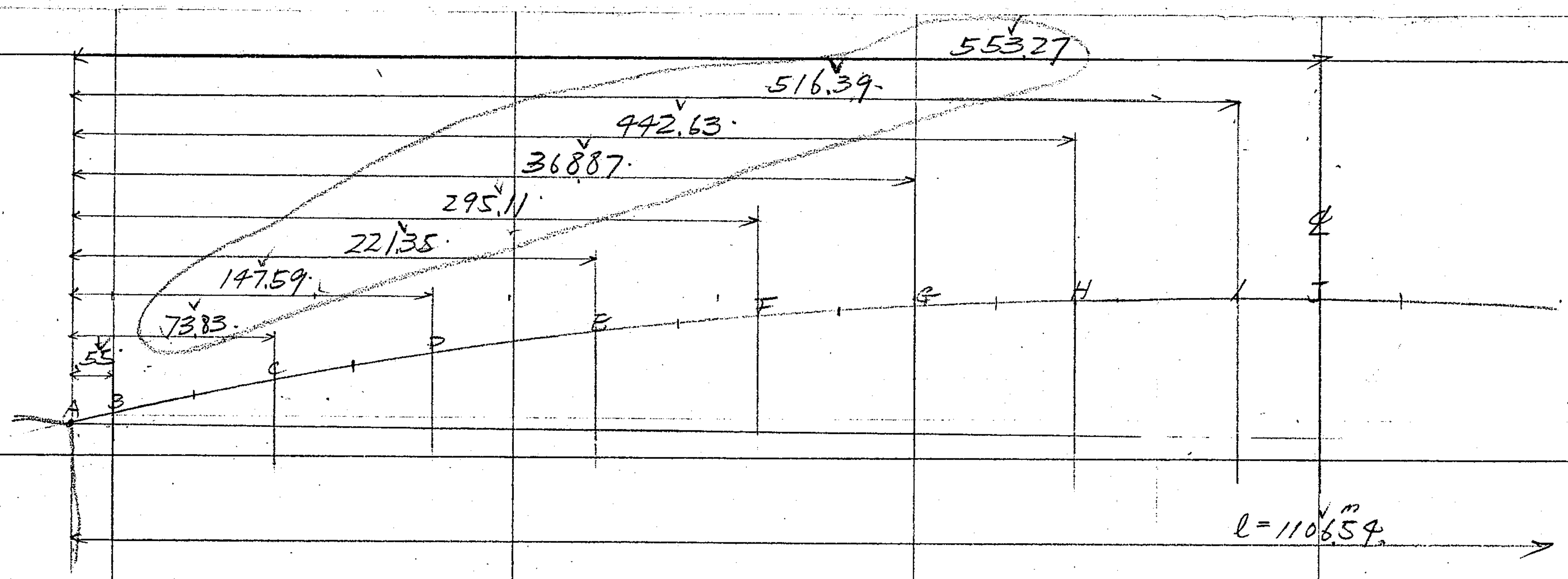
CALCULATIONS FOR

Nagaragawa for Mie-ken



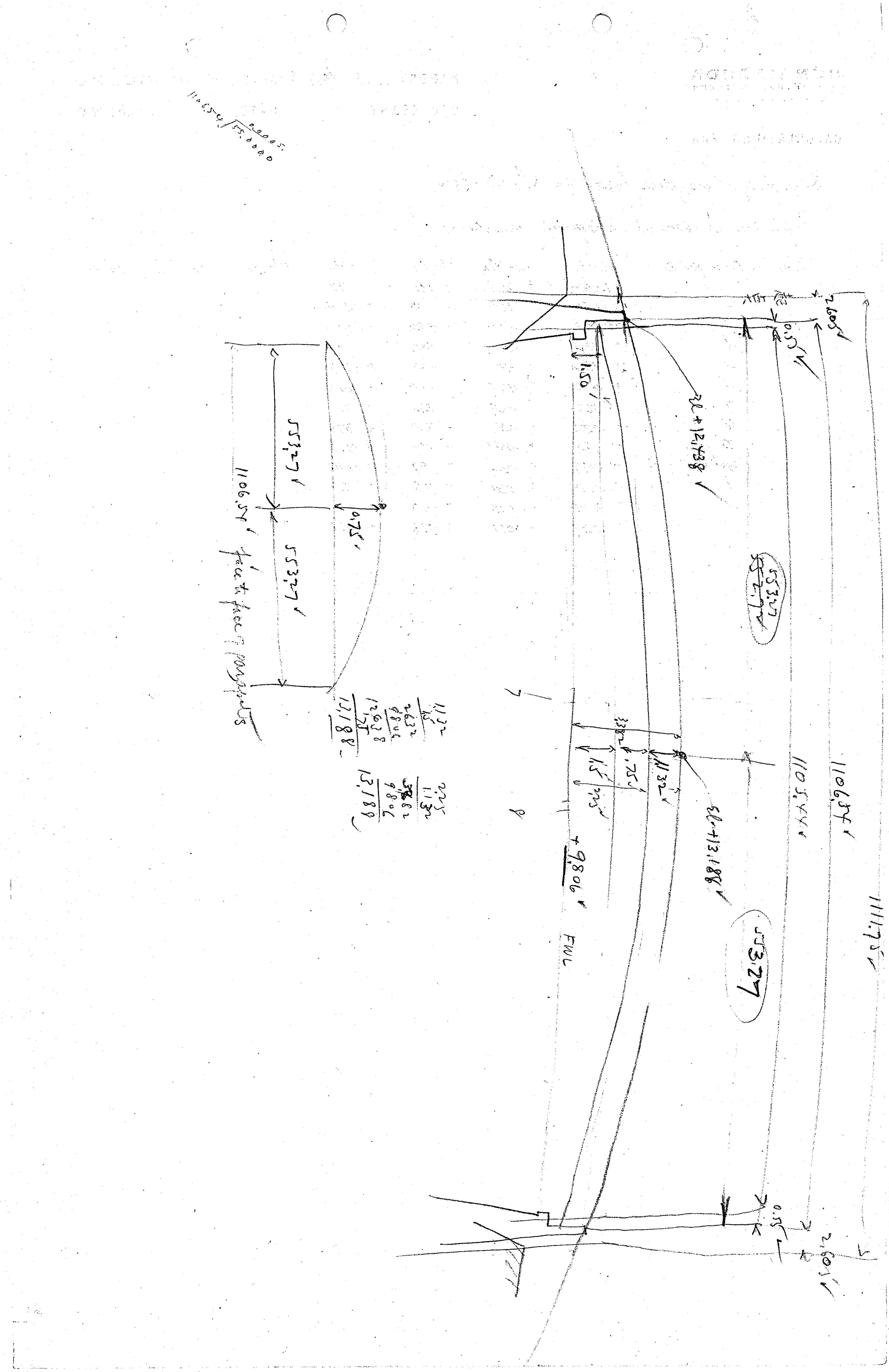
考十三ヶ
実施済ム

Equation of parabola $y = 4fC(1-C)$ where y = ordinate of parabola, f = rise of parabola, $C = \frac{x}{l}$

1132

	x	$C=\frac{x}{l}$	1−C	4fC(1−C)	Crown of roadway elevations	−1.82 =	Crown of roadway elevations	top of pier
A				0	11.306	~~9.486~~	12.438	
B	.55	0.00050	0.99950	0.001	11.307	~~9.487~~	12.439	10.619
C	73.83	0.06672	0.93328	0.187	11.493	9.673	12.625	10.805 P1
D	147.59	0.13338	0.86662	0.347	11.653	9.833	12.785	10.965 P2
E	221.35	0.20004	0.79996	0.470 0.480	~~11.776~~ 11.786	9.966	12.918	11.098 P3
F	295.11	0.26670	0.73330	0.587	11.893	10.073	13.025	11.205 P4
G	368.87	0.33335	0.66665	0.667	11.973	10.153	13.105	11.285 P5
H	442.63	0.40001	0.59999	0.720	12.026	10.206	13.158	11.338 P6
I	516.39	0.46667	0.53333	0.747	12.053	10.233	13.185	11.365 P7
J				0.750	12.056	~~10.236~~	13.188	~~11.368~~

Elevation top of pier = +5.60 m



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H.W.L.S.T. ——— 0.9^{m}

H.W.L.N.T. ——— 0.3^{m}

0 ——— 平均水面

L.W.L.N.T. ——— -0.3^{m}

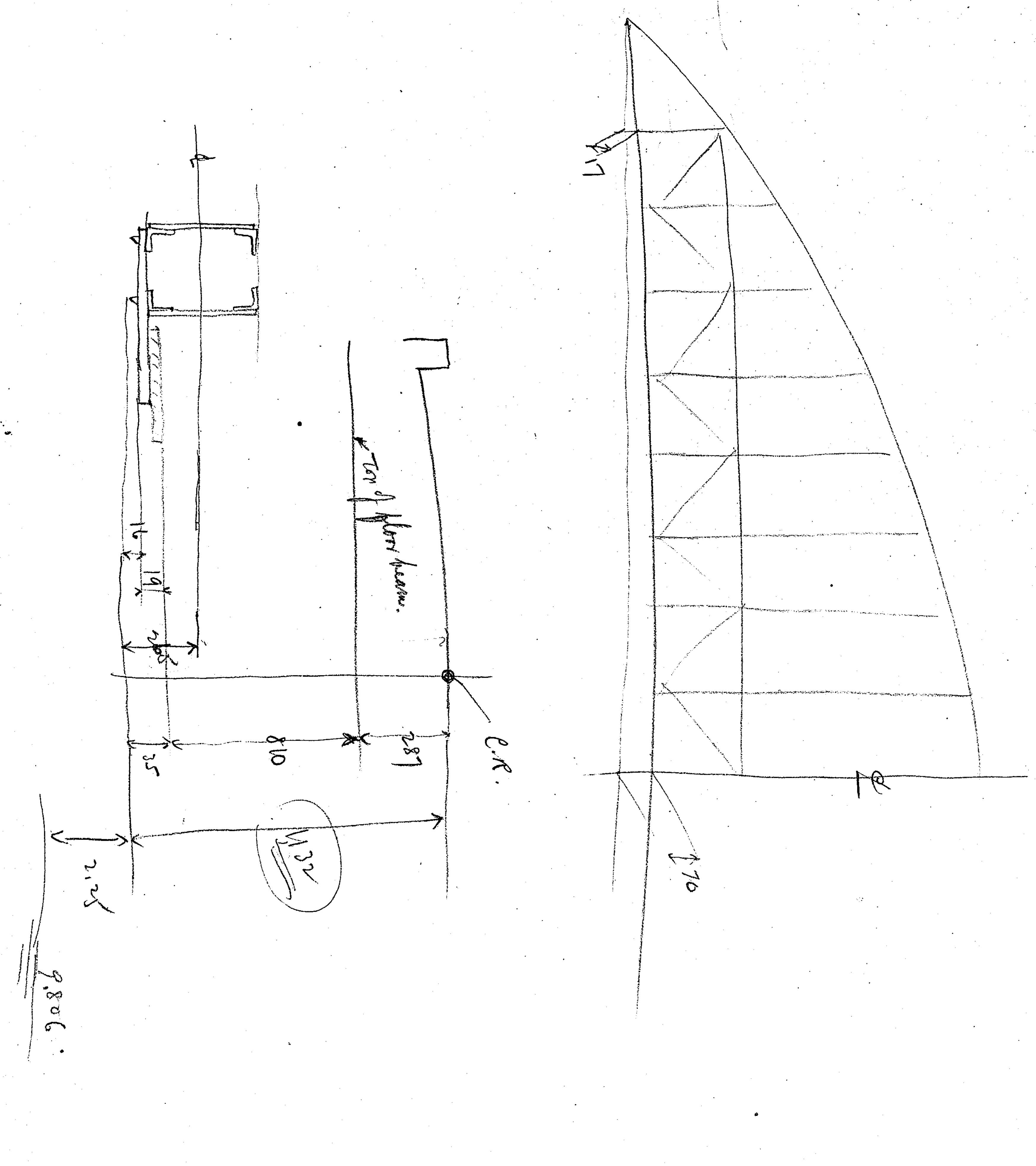
L.W.L.S.T. ——— -0.9^{m}

水位

木曽川

石川も、

地図より調査



Top of floor beam.
C.R.
810
287
35
1132
2.25
9.806

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CALCULATIONS FOR

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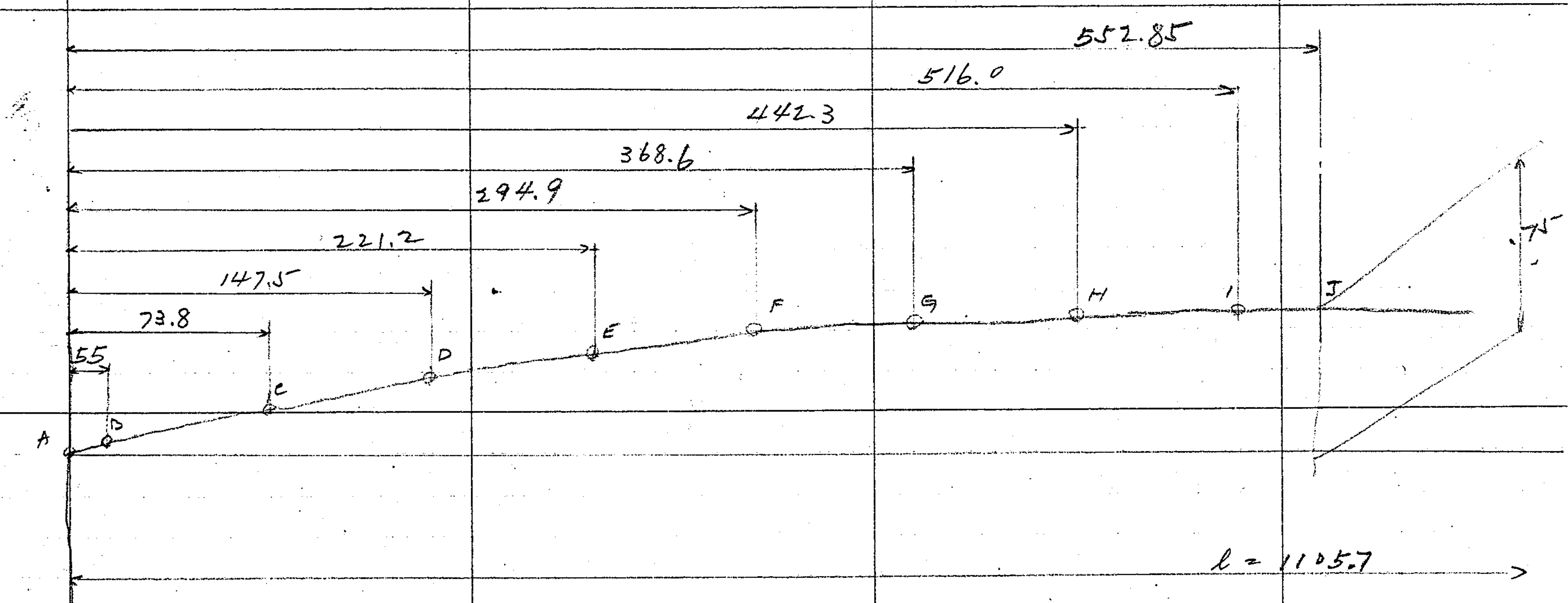

	X	$c = \frac{x}{l}$	$1-c$	$4Fc(1-c)$
A				
B	55	0.00050	0.99950	0.001
C	73.8	0.06675	0.93325	0.187
D	147.5	0.13340	0.86660	0.347
E	221.2	0.20005	0.79995	0.480
F	294.9	0.26671	0.73329	0.587
G	368.6	0.33336	0.66664	0.667
H	442.3	0.40002	0.59998	0.720
I	516.0	0.46667	0.53333	0.747
J	552.85	0.50000	0.5	0.750

F. W. L. = 3.743 (中等潮位)　9.8034 (設計ニ使用ノD.L.ニヨリ)

-0.222

MLW

M. W. L. = 0.2132 (中等潮位)　6.2738

(サイフォン上端)

小潮平均

L. W. T (~~中潮~~) ▽ -0.1231 (中等潮位) ~~打合セノ限界~~ 5.8296 打合セノ最高限度

= -0.3378 (〃)　5.7228

L. W. T (大潮) = -0.4638 (〃)　5.5968

平均

係藤工事サイフォン(実測セズ)ノ推定高

L. L. W. L = -1.3068 ▽ 4.739 (設計ノD.L.ニ依リ測定セル検見川サイフォン上端) 4.7538

LW

昭和四年水位

月	新満月		弦月	
	満潮	干潮	満潮	干潮
1	1.433 m	−0.070	0.590	+0.095
2	1.485	−0.020	0.860	0.190
3	1.615	0.125	0.660	0.180
4	1.975	0.550	0.900	0.570
5	1.810	0.590	1.240	0.795
6	1.860	0.325	1.023	0.467
7	1.910	0.300	1.065	0.315
8	1.935	0.275	0.925	0.390
9	1.995	0.300	1.095	0.850
10	2.140	0.565	1.115	0.895
11	2.040	0.355	0.615	0.380
12	1.797	0.217	0.605	0.085
平均	1.833	0.293	0.890	0.419

以上零点ノ高 −0.M7568 (中等潮位)
縣ノD.L.ヨリノ高 +5.M3038

年	新満月 干潮	弦月 干潮
昭和四年	0.293	0.419
〃三年	0.229	0.559
二年	0.241	0.554
元年	0.189	0.417
大正14年	0.258	0.545
〃3	0.386	
〃2	0.485	

大正十四年迄ハ大潮小潮ノ別ナク平均ナリ
一ヶ月干潮ノ平均

前略御免被下度　扨過日御出張の際御指示の水位工区より調査
提出致し候処偶同日居合有之ニ本課より課員を出張せしめ
調査いたし候処全様の結果ニ有之候ニ付為昨日小生儀名古屋
土木出張所ニ出張調査致し候為甚だ遅延致し候段
悪しからず思召被下度　扨水位は別表ニ記の通りに
通りニ有之先ニ御送附申上候 K.S.S.T は桑名出張所（内務省）ニて
量水標零尺の標高を謬り居り候為め今回報告のものニ大干潮面以下三尺五寸ニ設計
訂正被下度く鉄道省「ゲイツ」の上端は最低水位
示されたる由聞き及び候　尚揖斐長良両線の復線工事
準備の為め施工せる「ゲイツ」及木曽川の「ゲイツ」は最低水位大干潮面
~~本年中~~ニ施工せる由~~聞~~なれば之にて測定せし
揖斐川橋梁「ゲイツ」の上端は四、七三九ニ御座候（復線工事
基礎は之より一尺五寸高き筈）「ゲイツ」の高さニ付内務省土木出
張所ニて設計後異議有之候ては困却と存じ

所長ト面談致シ候処鉄道省のものと同様ニ為されたくとの事
ニ候ひしが 揖斐長良のものと木曽川及び復旧事の基
礎ケイブルの高は一致せざるを以て標高により決定致し方
協議致し候結果五、八三米(中善瀬仕ノ基準ニスレバ(一)〇、三二)
より低き事に於ては差支無き事ニ決定致し候る別表
水位参酌の上決定可被下度 次ニ橋梁足場は
橋杭二十五呎七吋ニ桟橋は径ヲ六十五呎六吋二連及
七十六呎六吋一連を交互ニ使用したる鉄道省の例ニ
做ひ ~~[illegible]~~ 桁は鉄道省より借用の見込みにて設計可
被下度 本日土木課長と協定致し候 名古屋土木出張所長
の意見は鉄道省より小ならざる様為されたくとの事ニ
有之候 先は右御報知迄

匆々

西岡[illegible]太郎

稲葉健三殿

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CONSULTIEG ENGINEER
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CALCULATIONS FOR

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昭和五年二月

國道弐號線

三重縣揖斐長良川橋梁

予算設計々算書

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CALCULATIONS FOR

1099.4

Preliminary Design of Ibi-nagara-Bashi for Mie Ken

Total length of bridge 1099.40 meters between End pins of bearings. 1100.5 out to out
Divided into 15 spans, 72.48 meters between end bearings, ctoc of bearings on pier = 1.041 m
Clear roadway 7.5 meters between curb line; paved with asphalt block 5 cm thick on mortar cushion. Floor slab of reinforced concrete. Handrail of cast iron design.

Assumed Loadings

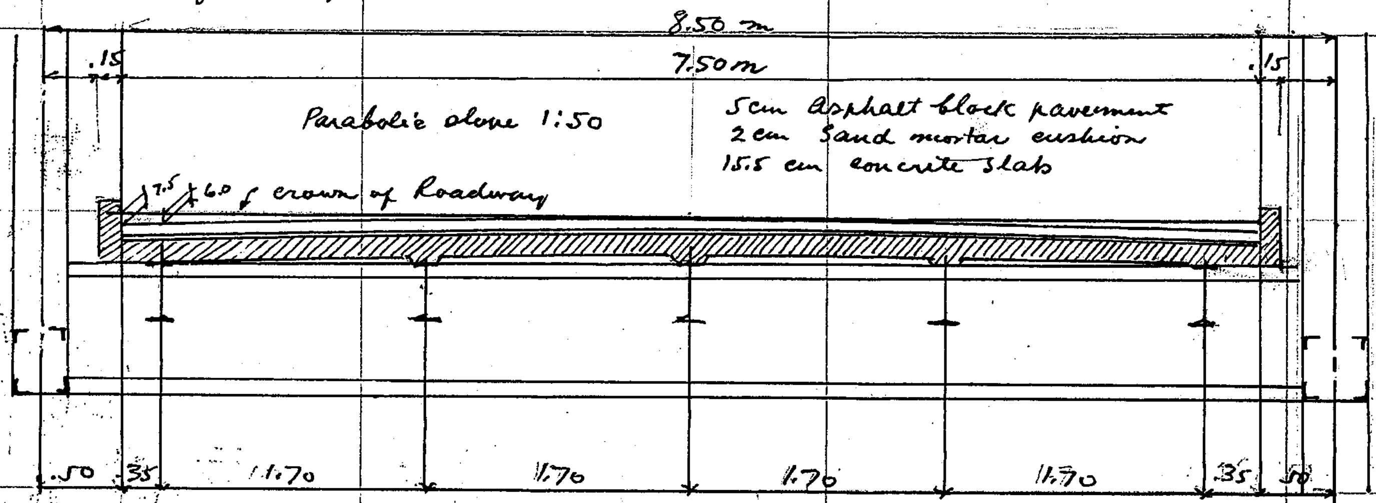
Uniform load on Roadway $w = \frac{100.000}{170+l} \leq 500$ kg/m²

where l = span length in meter.

8 ton motor truck loading	11 ton Road Roller
Rear wheel concentration 3000 kg each	front roller 4400 kg
Front wheel concentration 1000 "	rear roller 2-3300 kg.

Impact for motor truck loading $\text{Coef} = \frac{20}{60+l}$ where l = loaded length in meter

Cross Section of Bridge



Floor slab span length 1.70 meters

Dead Load

5cm Asphalt block pavement	@ 21 kg	=	105
2cm Sand mortar cushion	@ 17 "	=	34
15.5cm Concrete slab	@ 24	=	372
misc say			9
			520 kg/m²

Dead Load moment $= \frac{1}{10} \times 520 \times 1.7^2 = 150$ kgm

Live Load motor truck rear wheel concentration 3000
30% impact 900
3900 kg.

Distribution of wheel concentration on slab
pavement + cushion = 7cm

Longitudinal distribution a Contact between wheel + pavement 20
Distribution 2@7.0 = 14
34.0 cm

Transverse distribution b = 27+14 = 41.0 cm

Effective width $\varepsilon = \frac{2}{3}l + a = \frac{2}{3} \cdot 1.70 + .34 = 1.133$
.34
1.47

Load per meter strip = 3900 ÷ 1.47 = 2650 kg.
Uniform live load = 500 kg/m²

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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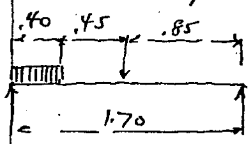
CALCULATIONS FOR

Preliminary Design of Hi-nagara Bashi for Mieken



motor truck loading $\frac{2650}{2} \times .85 = 1125$

unif. load $235 \times .85 = 20$

$1145 \times 0.8 = 915$ kg.m

Dead Load moment 150

1065

$500 \times 0.4 = 200$

$200 \times \frac{.20}{1.70} = 23.5$

Effective depth reqd $= \sqrt{\frac{1065 \times 100}{100 \times 7.18}} = 12.2$

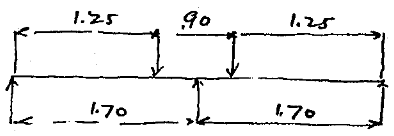
make depth of slab 15.5
insulation 3.0
12.5

I beam stringer span length 4.53 spacing 1.70 meter

Dead Load floor slab and pavement $520 \times 1.70 = 885$
beam assumed 75
960 kg per meter

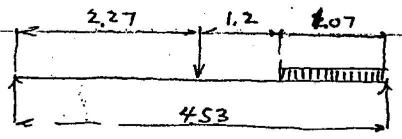
Dead Load moment $= \frac{1}{8} 960 \times 4.53^2 = 2460$

Live Load motor truck $2 @ 3900 \times \frac{1.25}{1.70} = 5750$ kg



Uniform live load $= 500 \times 1.70 = 835$ kg.

$\frac{835 \times 1.07^2}{2 \times 4.53} = 105.$



moment motor truck $\frac{5750}{2} \times 2.27 = ~~13100~~$ 6550
$105 \times 2.27 = ~~238~~$ 238
$~~13338~~$ 6788

Summary moment
Dead load 2460
Live load 6788 $~~13338~~$
$~~15798~~$ say $~~15800~~$
9248

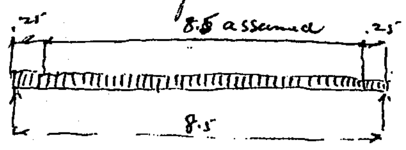
section modulus reqd $= \frac{924800}{1100}$ ($~~1580000~~$) $= 840$

14" × 6" @ 46.01# or 69.0 kg per meter

Section modulus = 1030.

Floor Beam span length 8.5 meters spacing 4.53 m

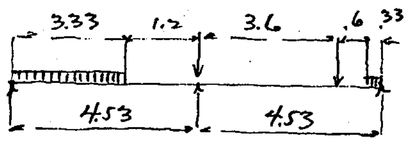
Dead Load floor $520 \times 4.53 = 2360$
floor beam 220
2580 kg.
stringer say 200
2780



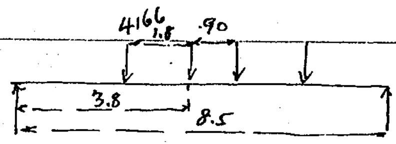
$D.L.\ m = \frac{1}{8} \times 2780 \times 8.5^2 = \frac{25100}{~~23300~~}$ kgm

Live Load

Motor truck $1300 \times \frac{.93}{4.53} = 266$
3900
4166 kg.



Uniform load $500 \times \frac{3.33^2}{2 \times 4.53} = 610$



$2 \times 4166 \times 3.8 = 31700$
$4166 \times 1.8 = -7500$
24200

Unif. load $\frac{1}{8} \times 610 \times 8.5^2 = 5500$
29700 call this 31000

Dead Load 25100
56100 kgm

Try web plate $800 \times 9 = 72.0$ 1/8 web $= 9.0$ back to back of Ls $= 81.0$ cm

Effective depth 78.7

Flange stress $= \frac{56100}{.787} = 71200$ $SR = 71200 \div 1200 = 59.3$
9.0
50.3 cm²

2Ls $125 \times 90 \times 10 = 41.00 - 8.8 = 32.20$
$270 \times 9 = 24.3 - 4.4 = 19.90$
52.10

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

CALCULATIONS FOR

Preliminary Design of Hoi-Nagara-Bashi for Mieken

Approximate weight of Intermediate floor beam

1 web plate		800 × 9	@	56.52	× 8.10	=	458
Flange	4 Ls	125 × 90 × 10	@	16.09	× 8.10	=	520
〃	2 Pls	270 × 9	@	19.08	× 5.6	=	214
End stiffs	4 Ls	125 × 90 × 10	@	16.09	× 0.8	=	52
Fills	4 Pls	90 × 10	@	7.06	× 0.62	=	18
Stringer Conn	10 Ls	100 × 90 × 10	@	14.13	× 0.8	=	113
Fills	10 Pls	90 × 10	@	7.06	× 0.62	=	44
Int. Stiffs	8 Ls	90 × 90 × 10	@	13.34	× 0.8	=	107
							1526
		Rivet heads and variation 3½%					54
							1580. kg ÷ 8.10 = 195 kg per meter

Approximate weight of End Floor Beam 1380 kg about.

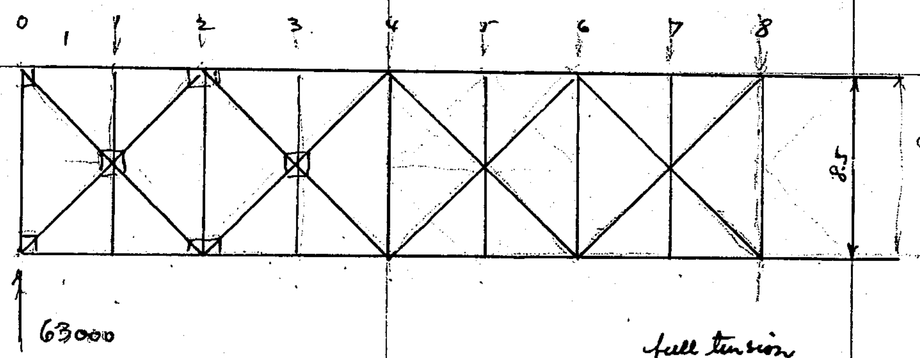
Bottom Lateral Bracing Seismic force k = 0.3

Dead Load metal assumed

stringers	72 × 5	=	360
floor beam	1580 ÷ 4.53 =		350
lateral Bracings			100
Trusses lower half say			1000
			1810 kg

Floor Load	520 × 7.5	=	3900
Copings	2 @ 150	=	300
Handrails	2 @ 80	=	160
			4360
			6170 kg.

Panel Concentration 6170 × 4.53 = 28000 @ 0.3 = 8400 kg.



8400 × 7.5 = 63000

$4.53^2 = 20.55$
$4.25^2 = 18.05$
$38.60 = 6.21$

$\frac{6.21}{4.25} = 1.465$

								Comp. 3R Gross	
					full tension 3R				
0-1	63000 @ 1.465 =	92300	÷ 2160 =		42.7	46150	÷ 1540 =	30.0	2 Ls 130 × 130 × 9 = 45.18 − 9 = 36.18
1-2	54600	80000	=		37.0	40.000		26.0	2 Ls do
2-3	46200	67600	=		31.3	33800			2 Ls ~~125 × 90 × 10~~ do
3-4	37800	55500	=		25.7	27750			2 Ls ~~125 × 90~~ do
4-5	29400	43000	=		19.9	21500			2 Ls 125 × 90 × 10 = 41.0 − 10.0 = 31.0
5-6	21000	30800	=		14.3	15400			2 Ls do

Approximate weight of lower laterals

4 Ls 130 × 130 × 9 @ 17.73	× 5.8	=	412	
Center connection &c say			38	
			450	kg per panel
4 Ls 125 × 90 × 10 @ 16.09	× 5.8	=	373	
			37	
			410	kg per panel

Total weight	8 @ 450	=	3600	
	8 @ 410	=	3280	
misc details say			320	
			7200 kg	÷ 72.48 = 99.5 kg

Call this 100 kg per lin meter

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CALCULATIONS FOR

Preliminary Design of Hi-Nagara-Bashi for Mieken

Top Lateral Bracing

Diagonal length $4.95^2 = 24.50$
$4.25^2 = 18.10$
$42.60 - 6.50$ meters

Radius of gyration reqd $= \frac{650}{150} = 4.33$

2LS $100 \times 75 \times 10$ $r = 48$

4LS $100 \times 75 \times 10$ @ $12.95 \times 6.0 = 310$
Details say 90
400 for one piece

For one panel $2 \times 400 = 800$ kg

Diagonals	12 @ 800 =	9600
Strut	6 @ 600	2400
Sway bracings	5 @ 1800 =	9000
Portal Bracings	2 @ 2500 =	5000
		26000 kg.

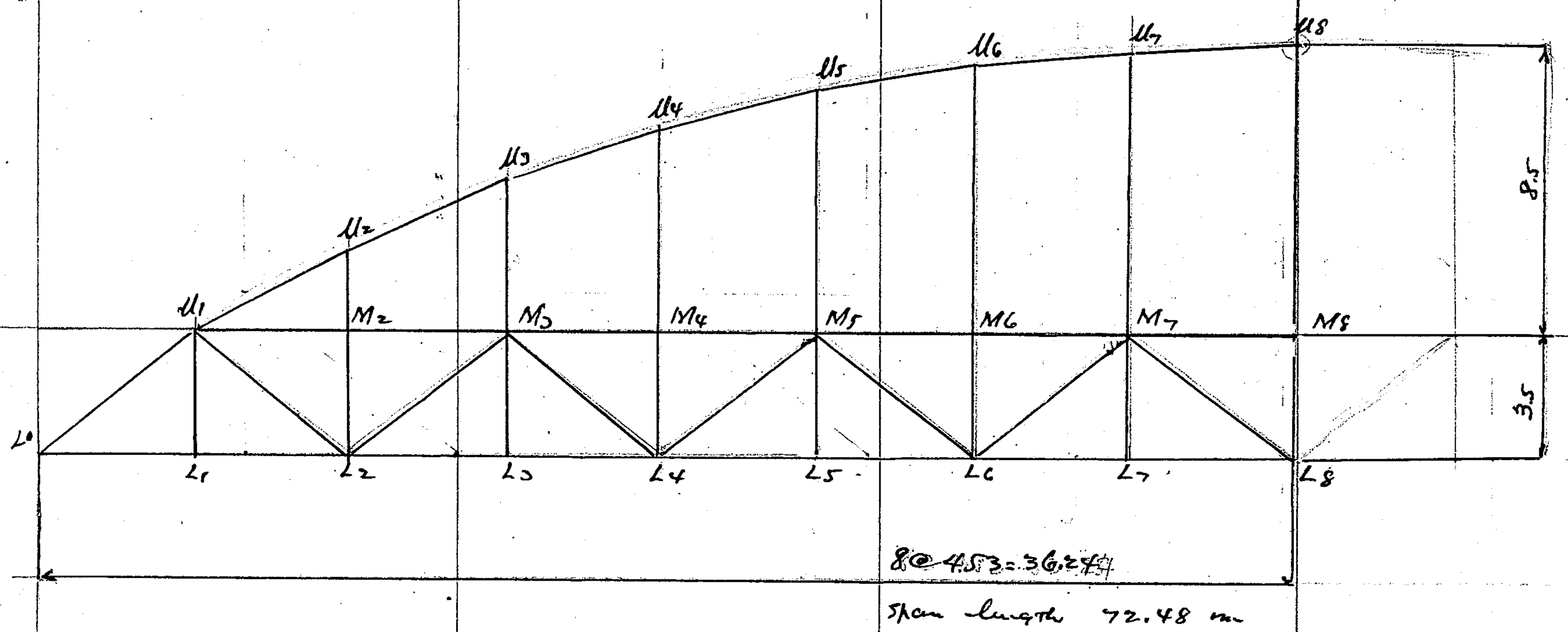
$26000 \div 73 = 356$ kg per lin. meter of span

Approximate Dead Load on truss
structural steel

Stringers	72×5 =	360
Floor beam	$1580 \div 4.53$ =	350
Lower lateral Bracings		100
Top Lateral Bracings		356
trusses assumed		2054
		3220
Floor Load complete		4360
		7580 kg. per lin. meter

Dead Load Concentration $= 7580 \times 4.53 =$ say 34200
For one truss say 17100 kg

General dimensions of truss as shown



All stresses by graphic.

Live Load Stress $412 \times \frac{7.5}{2} \times 4.53 = 7000$ assume 7500 kg including motor truck loading or say 44% of DL. stress.

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CALCULATIONS FOR

Preliminary Design of Ibi-Nagara-Bashi for Mieken

Dead Load stresses

Chord Stresses

L0-U1	12.35 C	@ 17100	=	211.000 C				
U1-U2	13.75 C		=	235.000 C				
U2-U3	13.26 C		=	227.000 C				
U3-U4	12.86 C		=	220.000 C				
U4-U5	12.58 C		=	215.000 C				
U5-U6	12.35 C		=	211.000 C				
U6-U7	12.23 C		=	210.000 C				
U7-U8	12.18 C		=	208.000 C				
L0-L2	9.83 T		=	168.000 T				
L2-L4	10.50 T		=	180.000 T				
L4-L6	11.40 T		=	195.000 T				
L6-L8	12.03 T		=	206.000 T				

web Stresses

U1-L2	0.15 T	@ 17100	=	2560 T
L2-M3	0.38 C			6500 C
M3-L4	0.55 T			9400 T
L4-M5	0.50 C			8500 C
M5-L6	0.39 T			6700 T
L6-M7	0.28 C			4800 C
M7-L8	0.17 T			2900 T
U1-M3	2.23 T			38000 T
M3-M5	1.52 T			26000 T
M5-M7	0.81 T			13800 T

Live Load web Stresses

U1-L2	0.51 T	@ 7500	=	3820 T
L2-M3	1.08 C		=	8100 C
M3-L4	1.78 T		=	13300 T
L4-M5	2.35 C		=	17600 C
M5-L6	2.75 T		=	20600 T
L6-M7	3.05 C		=	22900 C
M7-L8	3.75 T		=	25400 T

Top chord sections

1 cov. Pl.	650 × 13	=	84.50
2 Pls.	600 × 13	=	156.00
4 Ls	100 × 100 × 10	=	76.00
			316.50

1 cov. Pl.	650 × 13	=	84.50
2 Pls.	600 × 13	=	156.00
4 Ls	100 × 100 × 13	=	97.24
			337.74

Bottom chord section

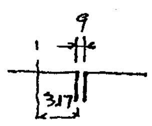
2 Pls.	470 × 10	=	94.0	−	20	=	74.0
2 Pls.	470 × 10	=	94.0	−	20	=	74.0
4 Ls	100 × 100 × 10	=	76.0	−	20	=	56.0
			264.0				204.0
2 Pls.	270 × 10	=	54.0	−	10	=	44
			318.0				248.0

Diagonal

$4.53^2 = 20.5$

$3.50^2 = 12.25$

$32.75 - 5.70$



2 Ls 100 × 75 × 10 @ 1650 = 330 + 3.62^2 + 320, 432, 752

$$r = \sqrt{\frac{752}{33}} = 4.77$$

$$f = 1500\left(1 - 0.0055 \times \frac{570}{4.77}\right) = 516 \text{ kg/cm}^2$$

max stress $27700 \div 516 = 53.7 \ \square^{cm}$

use 4 Ls 100 × 75 × 10 = 66.0 $\square^{cm}$ gross.

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CALCULATIONS FOR

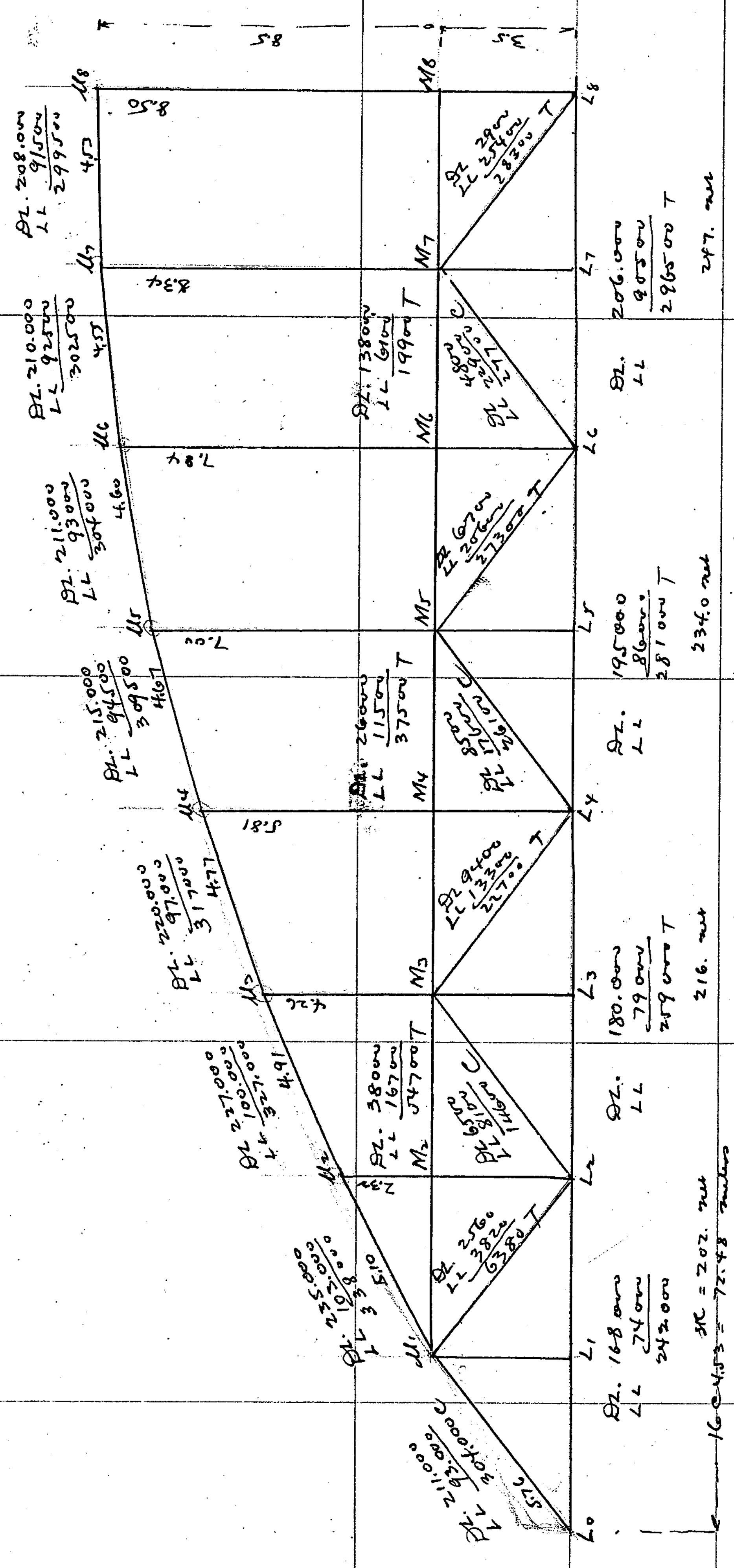
Preliminary Design of Ibi-nagara-Bashi for Mieken.

middle chord max stress = 54700 T SR = 45.60 cm net

use 4L 125 × 75 × 10 = 76.0 − 20 = 56.0 net

verticals 4L 125 × 75 × 10 = 76.0 − 20 = 56.0 net

Stress diagram as shown



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Preliminary Design of Ibi-Nagara-Bashi for Mie Ken

Approximate weights of truss.

Top chord	337.74	@ .785	× 15.77	=	4170	
	316.50	@ .785	× 23.12	=	5750	
Bottom chord	264.0	@ .785	× 9.06	=	1880	
	318.0	@ .785	× 27.18	=	6800	
middle chord	83.16 ~~76.~~	@	× 31.71	=	1890	
vertical	76	@	× 66.10	=	3940	
Diagonals.	~~76~~ 66	"	× 40.32	=	2090	
					26520 × 2 =	53040
			Details 35%.			18600
						71640

For two trusses 71640 × 2 = 143280
Call this 144 tons.
144000 ÷ 72.48 = 1990 kg per lin. meter of span.
167

Structural steel in one span

stringers	360 × 73.5	=	26450	26800	26,000
floor beam intermediate	1580 × 15	=	23700	27100	27.100
" " End	1380 × 2	=	2760		
Lateral bottom			7200	8100	8.100
" top			26000	23500	25.000
trusses			144000	150000	152.000
shoes.			4500	4000	4.000
misc. steel			1500	600	.600
			236110	236100	

call this 236.0 tons 241800
15 @ 242 = 3630 tons
236.000 ÷ 167 = 1.41 tons / tsubo

Total steel 236.0 × 15. = 3540 tons
Taking 8% allowance 285
3825 tons.

236100
8
244100

Dead Load on pier.

Floor load 4360 × 73.5 = 320.000
structural steel 236.000
556 000 kg.

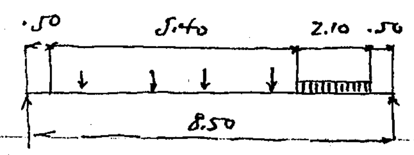
Dead Load on shoe 556.000 ÷ 4 = 139.000 kg.

Live Load $w = \frac{100.000}{170+72.48} = 412$ $412 \times 7.5 = 3090$ kg per lin. meter.

Live load on pier assumed full uniform load on 2 spans with same intensity as for one span neglecting motor truck loading.
load on pier 3090 × 73.5 = 227.000 ~~tons~~ kg.

max live load on shoe motor truck rear wheel 3000
impact $\frac{20}{60+72.48} = 15.1\%$ 453
3453 × 2 = ~~96~~ 6906 kg

For 2 motor trucks 6906 × 2 = 13810 kg

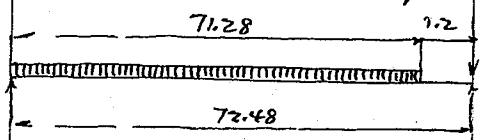


2.1 × 412 = 865
3090 − 865 = 2225

Reaction on truss
~~Uniform load 865 × $\frac{1.55}{8.50}$ = 158 kg~~
Uniform load 2225 × $\frac{5.3}{8.5}$ = 1390 kg
motor truck rear wheel 13810 × $\frac{5.3}{8.5}$ = 8450

CALCULATIONS FOR

Preliminary Design of Ibi-Nagara-Bashi for Mieken



Uniform load	1390	$\times \frac{71.28^2}{2 \times 72.48}$	=	48800
Uniform load	158	$\times \frac{72.48}{2}$	=	5720
motor truck				8450
				62970 call this 63000 kg.

Summary load

	On one pier	On one shoe
Dead Load	556000	139000
Live Load	227000	63.000
	783 000 kg.	202,000 kg

Approximate Estimate of Deck construction. Total length of bridge 1100.5 meters between faces of parapet walls of abutments.

Concrete Slab.	.15.5 × 7.5	=	1.160
coping.	.15 × .375	=	.056
fillet			.010
			1.226 × 1100.5 = 1350 cubic meters

Area of pavement = 7.5 × 1100.5 = 8250 sq meters

Forms say = 8.1 × 1100.5 = 8910 sq meters

Reinforcing Bars. 8250 × 23 kg = 190,000 kg.

Handrails. 2200 meters × 80 = 176.000 kg.
finish of coping 2 - .70 × 1100 = 1540 sq meters.

Approximate Estimate of Deck construction

concrete	1350 cubic meters	@	1690	= 22800
forms	8910 sq meters	@	2^{00}	= 17820
reinforcing bars	190. tons	@	120^{00}	= 22800
pavement	8250 sq meters	@	4^{75}	= 39200
Handrails	176. tons	@	320^{00}	= 56400
Drains	250	@	6^{00}	= 1500
Entrance Pedestals + Rails	4	@	1500^{00}	= 6000
wiring & lamp	1110 meters	@	10^{00}	= 11100
finish of coping	1540 m²	@	4^{25}	= 6550 ^{00}
				184170^{00} Say 185000

Estimate of Cost structural Steel 3825 tons @ 280^{00} = 1071000 ^{00}

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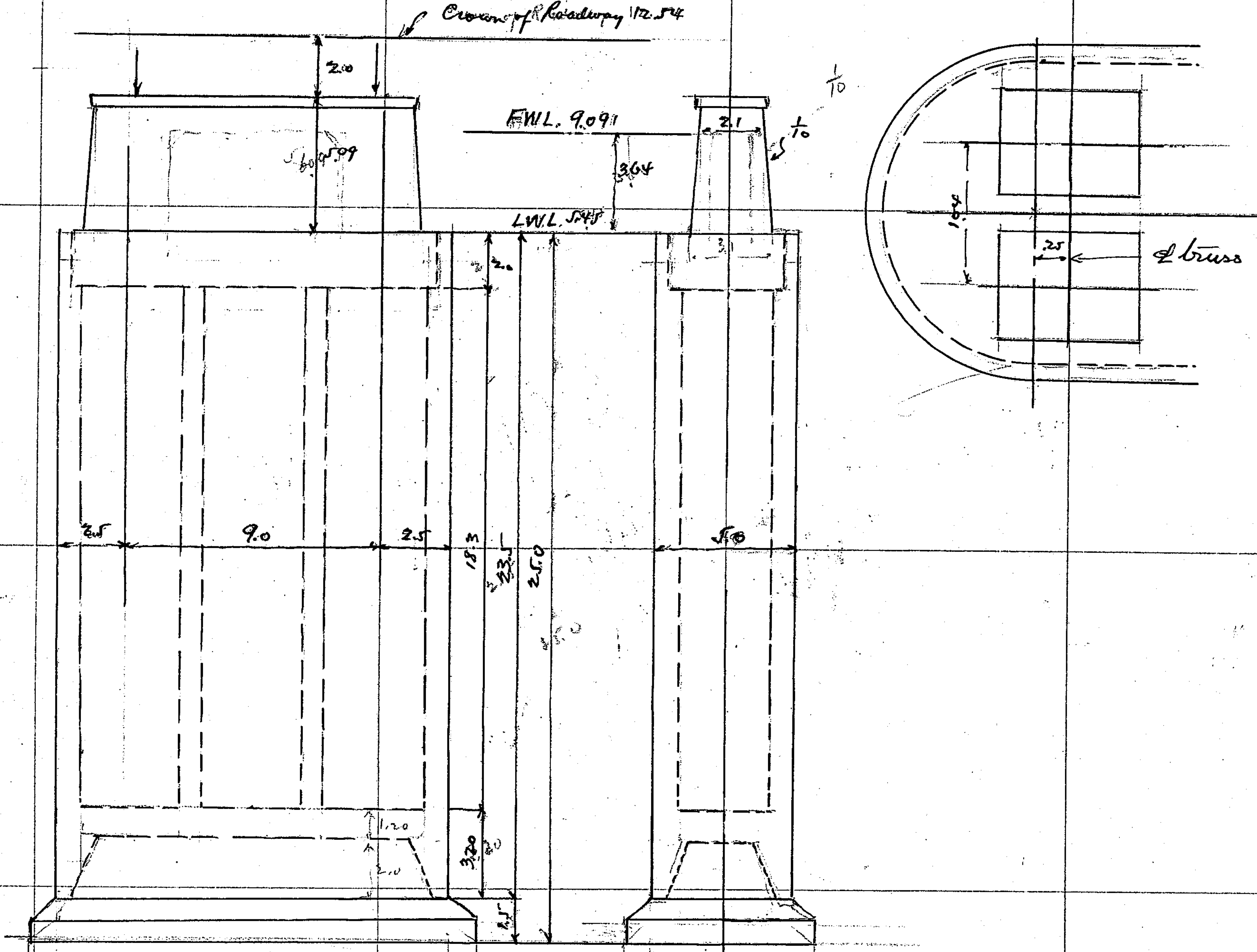
CALCULATIONS FOR

Preliminary Design of Ibi-Nagara-Bashi for Mieken

Piers Superimposed Dead Load 556000 kg
Live load 227000 kg.

max load on shoe 202,000 kg.



Approximate Concrete in shaft.

$2.1^{\phi} = 3.46$

$2.1 \times 9 = \underline{18.90}$

22.36

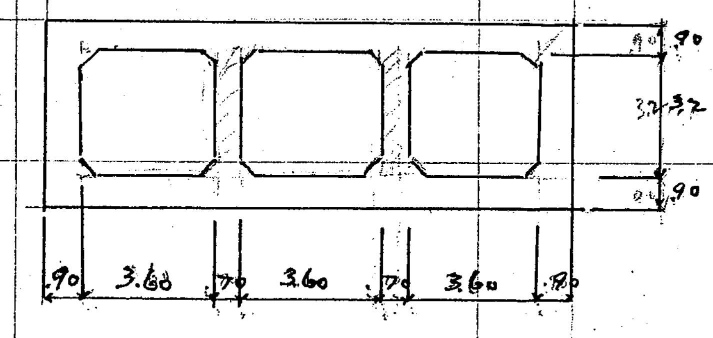
$3.1^{\phi} = 7.55$

$3.1 \times 9.0 = \underline{27.90}$

$\underline{35.45}$

$57.81 \div 2 = 28.9$

volume $= 28.9 \times 5.09 = 147.0$ cubic meters



Caisson area $= 2 - 14.0 \times 0.9 = 25.20$

$2 - 3.2 \times 0.9 = 5.76$

$2 - 3.2 \times 0.7 = 4.48$

$.25 \times 6 = \underline{1.50}$

36.94 sq m

inside filling $= 5 \times 14 = 70.0$

$\underline{-36.94}$

33.06 sq m.

$33.06 + 3 = 11.02$ sq m

volume of concrete in shell shaft $36.94 \times 18.3 = 675.0$

$33.06 \times 18.3 = 605.0$

Concrete in top filling $70.0 \times 2.0 = 140.0$

" " bottom filling $70.0 \times 3.2 = \underline{224.0}$

941.0

Concrete in base say $7.0 \times 16.0 \times 1.5 = 167.0$

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CALCULATIONS FOR

Preliminary Design of Ibi-Nagara-Bashi for Mie Ken

Weight of pier

shaft	147,000	@	2200	=	324 000
Shell	675.0	@	2400	=	1620.000
Top	140.0	@	2200	=	308.000
bottom	224.0	@	"	=	493.000
base	167.0	@	"	=	368.000
water	605.0	@	1000	=	605.000
					3718.000

Superimposed load Dead Load 556000
Live Load 227000 — 783.000

4501 000 kg.

Area of base 16 × 7. = 112.0 square meters

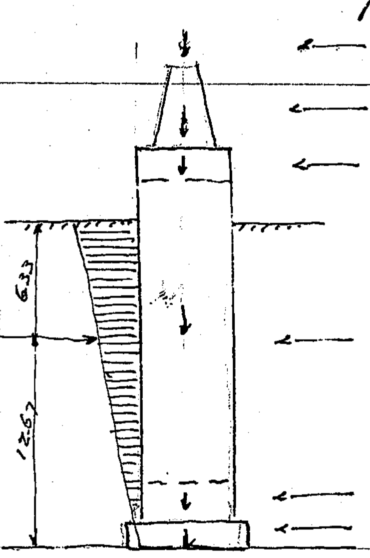
Skin friction 350 #/□' or 1710 kg/m²
1710 × 38.0 = 65000 kg per meter of depth

Caisson below firm ground assumed 19.0 meters

Total load 4501.000
less friction 65000 × 19.0 −1235.000
3266 000 kg.

Bearing on Soil counting skin friction 3266000 ÷ 112 = 29200 kg/m² or 2.72 tons/□'

Stability during Earthquake Horizontal Force assumed k = 0.3
Caisson into firm ground 19.0 meters



1	Superimposed Dead Load	556.000 × 0.3	=	166800
2	shaft	324.000	=	97.000
3	top concrete	308.000	=	92500
4	shell + water fill	2225.000	=	667.000
5	bottom concrete	493.000	=	148.000
6	base	368.000	=	110500
		4274000 ✓		1281.800 ✓

Moment about bottom of base

1	166800	× 30.9	=	5150.000
2	97000	× 27.5	=	2665.000
3	92500	× 24.0	=	2220.000
4	667000	× 13.85	=	9250.000
5	148000	× 3.10	=	459.000
6	110500	× .75	=	83.000
	1281800			19827.000

1281800 × 12.67 = −16200.000

3627.000 kg meters

Frictional couple 1710 × 19 × 14 = 455000

Resisting couple 455000 × 5 = −2280.000

1347000

Eccentricity = $\frac{1347000}{4274000}$ = .315 m

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CALCULATIONS FOR

Preliminary Design of ~~Shi~~-nagara-Bashi for Mieken

max toe Pressure = $\frac{4274000}{112}\left(1 \pm \frac{6 \cdot .315}{7}\right)$ = 48500 kg/m² 4.5 tons/○'

approximate list of materials

Concrete	shaft	147.0				
	shell	675.0	136 concrete	say	253	cubic meters
	top	140.0	124	"	1100	"
	bottom	224.0				
	base	167.0				
		1353.0				

Area of forms

Caisson	81.5 × 23.5 =	1915			
bottom	say 5×14	70			
		1985	call this	2000	
Shaft	say 33 × 5.1			170	
				2170	sq meters

Reinforcing Bars 45.0 tons
Curb shoe 4.0 "
Timber crib work say 6000 cubic ft
Excavation say 5 × 14.0 × 25.0 = 1750 cubic meters

Estimate of Cost of one pier

concrete	124	1100 m³ @	16.90 =	18600	
	136	253 m³ @	13.80 =	3490	
forms		2170 sq @		5420	
Reinforcing bars		45 tons @		5400	
Cutting Edge		4 tons @		800	
Timber work				12000	
Excavation		1750 m³ @		9630	
water fill				300	
					55640

Extra cost for Pneumatic Caisson work ———— 20200

Working shaft rent	1200	misc	2160
Air Hose & pipes rent	400		78000.00
Derrick &c setting removing & temporary work	4000		
Electric Power	1500		
Electric wiring & Equipment	2000		
Pontoon and Launches rent	700		
misc working materials	2500		
workmen's salary & Expense	2500		
Hospital Equipment & Expense	1200		
Working machine & Tools rent	1700		
Repair work	500		
Launching caisson complete	2000		
	20200		

Abutments 2 @ 12500 = 25000.00

Estimate of cost

Deck construction		185.000	
structural steel		1071.000	
			1256000
Substructure	14 @ 78000 =	1092 000	
	2 @ 12500 =	25000	
			1117000
			2373000.00

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CALCULATIONS FOR

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昭和五年二月

國道壹號路線

三重縣揖斐長良川橋梁

橋臺設計々算書

及材料調書

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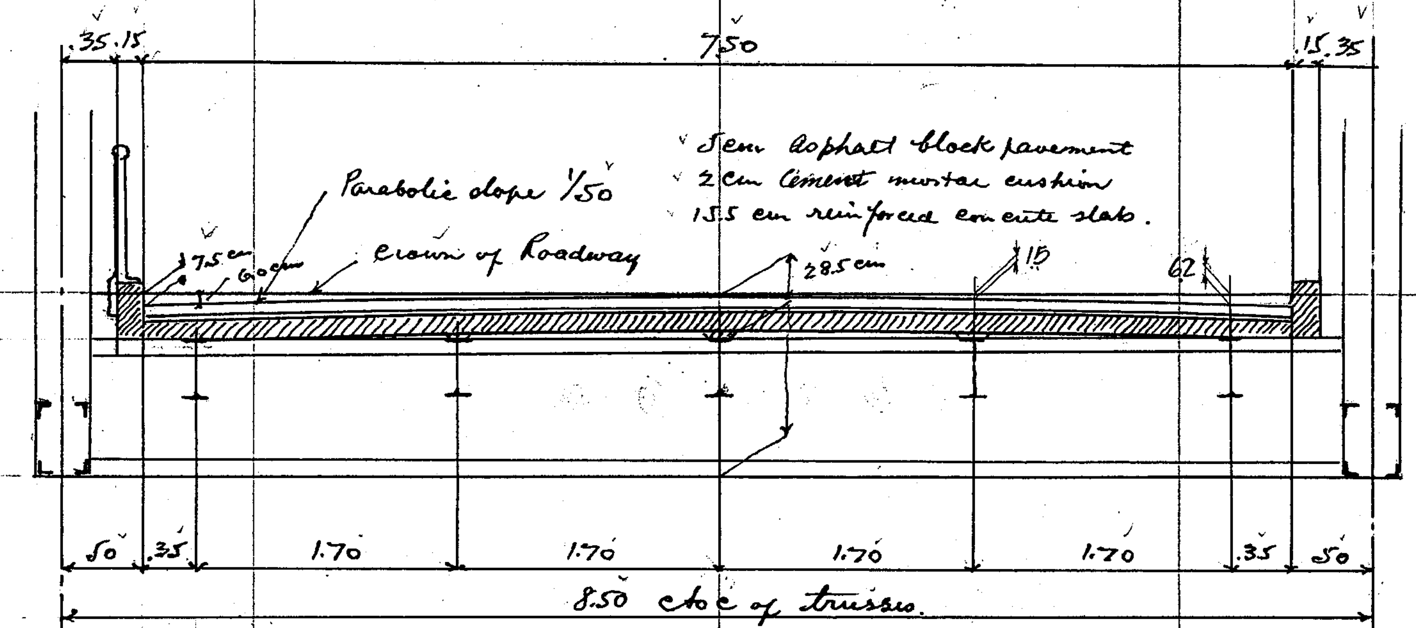
CALCULATIONS FOR

Design of Ibi-Nagara-Bashi for Mieken

Assumed Cross Section of Bridge.

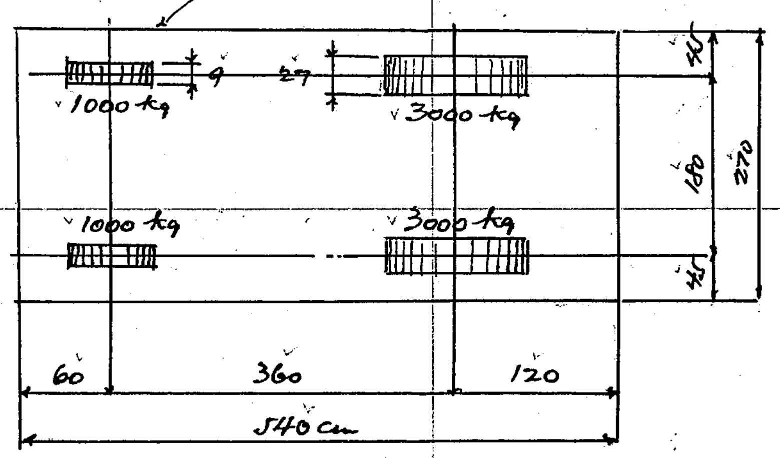


Assumed loadings

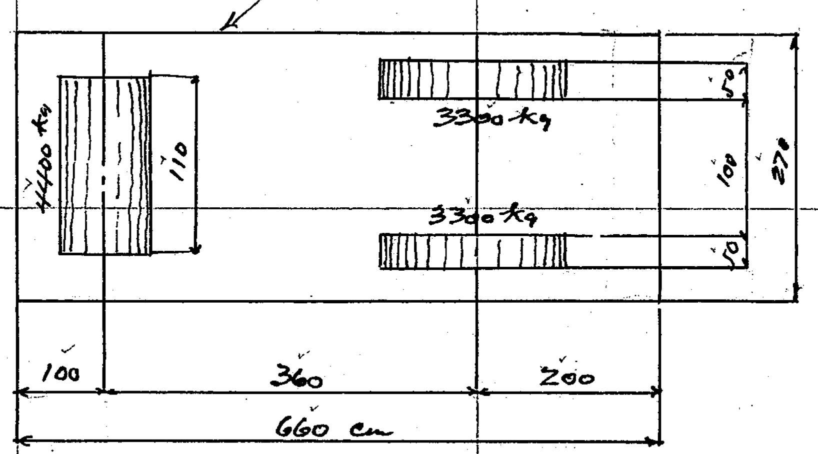
Uniform load on Roadway $w = \frac{100.000}{170+l} < 500$ kg/m²

where l = span length in meter

8 ton motor truck loading



11 ton Road Roller Loading.



2 lines of motor traffic on roadway with occupied width of 270 cm each. One motor truck each traffic line for one span; Unoccupied space around motor trucks shall be filled with uniform load specified above.

Road Roller one on each span

Impact for motor truck loading

Coef $= \frac{20}{60+l}$ where l = loaded length in meter
max impact 30%

No impact for road roller and uniform load.

Allowable working strength

1:2:4 Concrete

Direct compression	35	kg/cm²
Fibre stress due to bending	45	"
Combined stress direct and bending	35	"
Punching shear of Concrete	9	"
Shear of plain concrete	4	"
Bearing value	45	"
Bond stress for plain bar	6	"
Reinforcing Bar Tension	1200	"
Shear	900	"

Seismic force 3000 mm/sec² or $k = 0.3$

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CALCULATIONS FOR

Design of Hei-nagara-Bashi for Mie Ken

Superimposed load

Dead Load

For Cross section of floor see page 1

Roadway Asphalt block pavement 5 cm thick @ 21 = 105
2 cm Sand mortar cushion @ 17 = 34
15.5 cm reinf. concrete slab @ 24 = 372
misc filler etc say 9
520

For 7.5 meter wide roadway 520 × 7.5 = 3900
Coping .37 × .165 @ 2400 = say 150 2 × 150 = 300
Handrail assumed 2 × 80 = 160
4360 kg per lin. meter

structural Steel in one span say 240.000 kg or 240.000 ÷ 73.5 = ~~3300~~ 3270
~~7660~~ 7630 kg per lin. meter

Dead Load on abutment ~~7660~~ 7630 × $\frac{\cancel{72.9}\ 73.5}{2}$ = 280000 kg
for one shoe 280000 ÷ 2 = ~~139500~~ 140,000 kg

Live Load

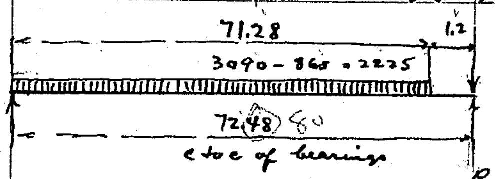
Uniform live load w = $\frac{100.000}{170 + 72.48}$ = 412 kg/m²

for 7.5 meter wide 412 × 7.5 = 3090 kg per lin. meter

Impact for motor truck Impact = $\frac{20}{60 + 72.48}$ = 15.1 %

motor truck rear wheel 3000
Impact 15.1 % 453
3453 × 2 = 6906

For 2 motor trucks 2 × 6906 = 13810 kg



2.1 × 412 = 865

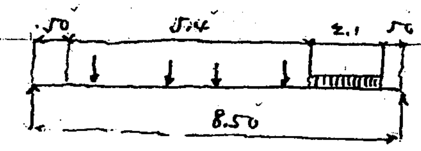
Reaction R unif. load $\frac{\cancel{3090}\ 2225 \times 71.28^2}{2 \times 72.48}$ = 78000

865 × $\frac{72.48}{2}$ = 31300

motor truck loading rear wheels 13810
123110 kg

Call this live load 125000 kg

Load on shoe



Uniform load 865 × $\frac{1.55}{8.5}$ = 158 kg

Uniform load 2225 × $\frac{5.3}{8.5}$ = 1390 kg

motor truck rear wheel = 13810 × $\frac{5.3}{8.5}$ = 8450

Load on shoe Unif. 1390 × $\frac{71.28^2}{2 \times 72.48}$ = 48800
Unif. 158 × $\frac{72.48}{2}$ = 5720
motor truck 8450
62970 call this 63000 kg.

Summary load

	On abutment	On one shoe
Dead Load	280.000	140.000
Live Load	125.000	63.000
	405.000 kg.	203.000 kg

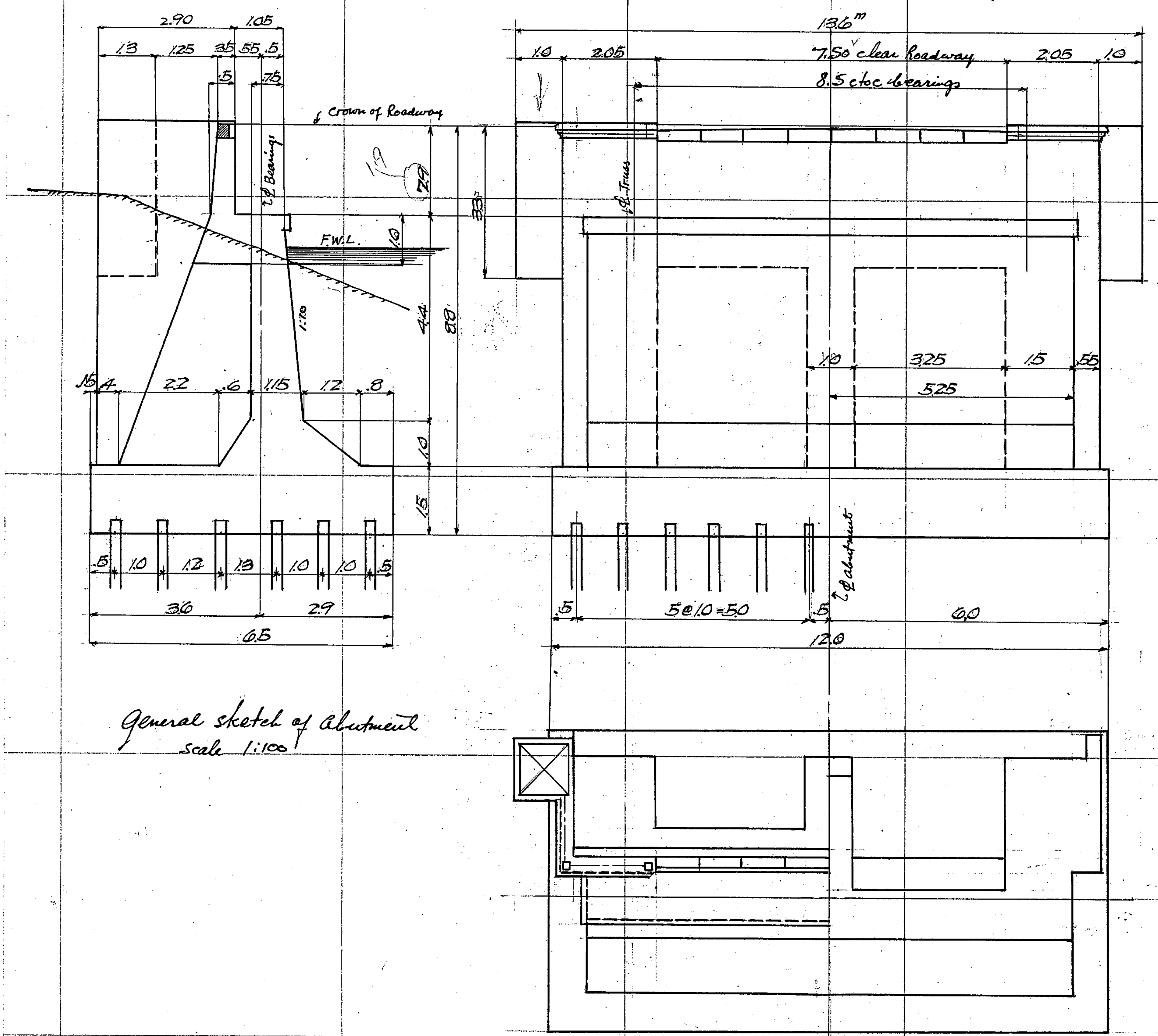
Size of Roller shoe assumed 75 × 100 = 7500 cm²

unit bearing = 203000 = 27.1 kg/cm²

CALCULATIONS FOR

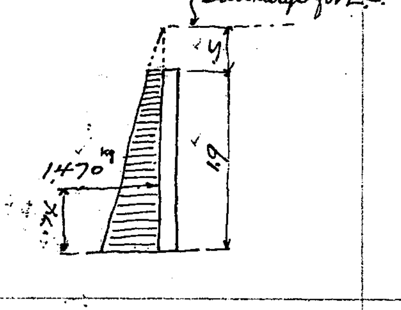
Ibi-Nagara Bashi for Mie ken.

Design of abutment.
General dimensions are as shown on sketch below.



General sketch of abutment
scale 1:100

Design of Parapet wall.



Earth pressure at normal state.

$\frac{1}{3} \times 1600 \times .5 = 267$

$\frac{1}{3} \times 1600 \times 2.4 = \underline{1280}$

$1{,}547 \div 2 = 774 \ kg/m^2$ average

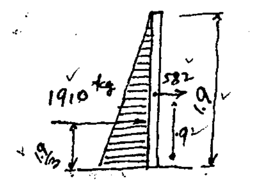
$774 \times 1.9 = 1{,}470$ kg per meter strip of wall.

moment at bottom of wall.

$1470 \times .74 = 1{,}088$ kgm.

Earth pressure during earthquake. k assumed 0.30

$1600 \times .662 \times \frac{1.9^2}{2} = 1{,}910$ kg



Moment at bottom of wall.

$1910 \times \frac{1.9}{3} = 1210$ kgm
$\underline{524}$
$1{,}734$

$\cancel{1210}\ 1734 \div 1.8 = \cancel{673}\ 964$ kgm.

wt. of wall $1.9 \times .425 \times 2400 = 1940$
$1940 \times .3 = 582$
$582 \times .9 = 524$ kgm

Moment at normal state governs the section of wall.

Ibi - Nagara Bashi for Mie ken.

Effective depth required for $f_c = 45\ kg/cm^2$ + $f_s = 1200$

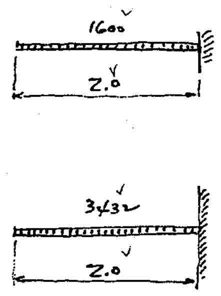
$d = \sqrt{\frac{M}{bR}}$ where $R = 7.18$

$d = \sqrt{\frac{1088 \times 100}{100 \times 7.18}} = 12.3\ cm$ use 47 cm effective depth with 3 cm insulation

Steel area required $= \frac{108800}{1200 \times \frac{7}{8} \times 47} = 2.2\ cm^2$ per lin meter

use 13 mmϕ bars at 30 cm ctoc = 4.4 cm² " on rear side
13 mmϕ " " 60 " = 2.2 " on front side

Design of wing wall.



Section at 3 meter below top of wall.

Earth pressure at normal state $= \frac{1}{3} \times 1600 \times 3 = 1600\ kg/m^2$

moment on wall $= 1600 \times \frac{2^2}{2} = 3200\ kgm$.

Earth pressure during earthquake $= 1600 \times .662 \times 3.0 = 3180\ kg/m^2$

Seismic force on wall $= .35 \times 2400 \times .3 = \underline{252}$
3432

Moment on wall $= 3432 \times \frac{2^2}{2} = 6864\ kgm$ Shear $= 3432 \times 2 = 6864\ kg$

Latter moment governs

effective depth required $= \sqrt{\frac{686400}{100 \times 7.18 \times 1.8}} = \frac{23.1}{~~30.9~~}\ cm$

Use 32 cm effective depth with 3 cm insulation or 35 cm in total.

Steel area required $= \frac{686400}{1200 \times 1.8 \times \frac{7}{8} \times 32} = 11.35\ cm^2$ per meter strip

Use 19 mmϕ bars at 25 cm ctoc = 11.35 cm² "

unit shear $= \frac{6864}{100 \times \frac{7}{8} \times 32} = 2.45\ kg/cm^2$ ok.

Unit bond $= \frac{6864}{5.97 \times 4 \times \frac{7}{8} \times 32} = 10.2\ kg/cm^2 < 6.0 \times 1.8 = 10.8$ ok.

Section at 1.5 m from top.

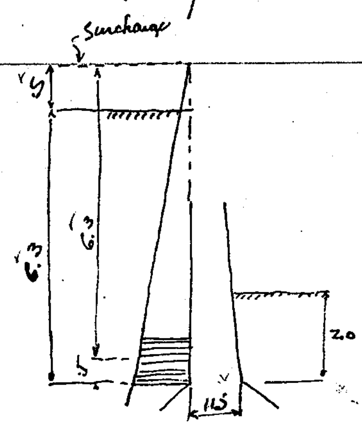
moment due to earth pressure + seismic force.

$3180 \div 2 = 1590$
252
$1842 \times \frac{2^2}{2} = 3684\ kgm$.

Steel area req'd $= \frac{368400}{2160 \times \frac{7}{8} \times 32} = 6.08\ cm^2$ per meter strip

use 16 mmϕ bars at 25 cm ctoc = 8.04 "

Design of Curtain wall span length assumed 4.0 meters.



Earth pressure at normal state $= \frac{1}{3} \times 1600 \times 6.3 = 3360\ kg/m^2$ average.
$\frac{1}{3} \times 1600 \times 1.5 = \underline{-800}$
2560

Earth pressure during earthquake $= .662 \times 1600 \times 5.8 = 6,150$

Seismic force $1.1 \times 2400 \times .3 = \underline{800}$
6950 kg/m²

moment on wall $= \frac{6950 \times 4^2}{10} = 11,120\ kgm$

Effective depth req'd $= \sqrt{\frac{11,120}{7.18 \times 1.8}} = 29.3\ cm$ use 115 cm total thickness

Steel area req'd. $= \frac{1112000}{2160 \times \frac{7}{8} \times 110} = 5.35\ cm^2$ per meter strip.

use 16ϕ bars at 35 cm ctoc = 5.75

Shear = 6950 × 2 = 13900 kg.

Unit shear $= \frac{13900}{100 \times \frac{7}{8} \times 110} = 1.45\ kg/cm^2$ ok.

Unit bond $= \frac{13900}{5.03 \times 2.86 \times \frac{7}{8} \times 110} = 10.03\ kg/cm^2 < 6.0 \times 1.8$ ok.

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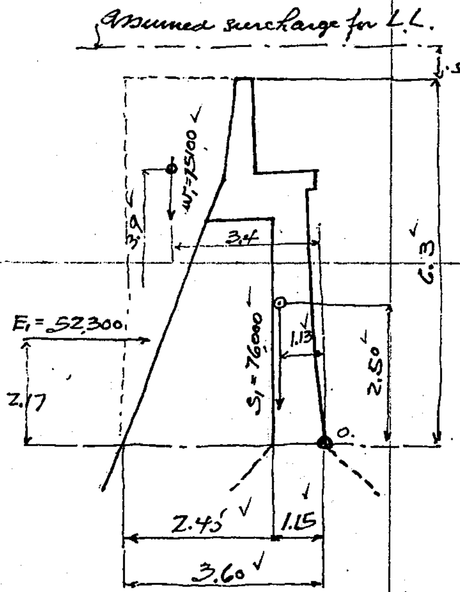
CALCULATIONS FOR

Ibi-Nagara Bashi for Mie ken.

Design of Counterfort at center.

weight and center of gravity of counterfort at center. width 4.25 m

			arm vert m		arm hor m.	
granite	.25 × .3 × 4.25 = .318 @ 2600 =	830	6.15	5100	1.65	1370
parapet wall	.425 × 1.6 × 4.25 = 2.89 @ 2400 =	6930	5.15	35660	1.66	11,500
Top beam	1.80 × 1.0 × 4.25 = 7.65 @ 〃 =	18350	3.88	71,200	1.29	23650
Coping	.1 × .3 × 4.25 = .13 @ 〃 =	300	4.25	1,280	.35	110
Curtain wall	.99 × 3.4 × 4.25 = 14.30 @ 〃 =	34,300	1.57	53,800	.50	17,150
Counterfort	1.875 × 3.4 × 1.0 = 6.37 @ 〃 =	15,290	1.50	22900	2.09	31,900
	Sum of concrete = 31.34 m³	76,000 kg	2.50	189940	1.13 m	85680
	granite 0.318 m³					



Weight of earth on counterfort wall. Earth pressure on front side neglected.

$1.5 \times 1 \times 6.3 = 9.45 @ 1600 = 15100$ kg

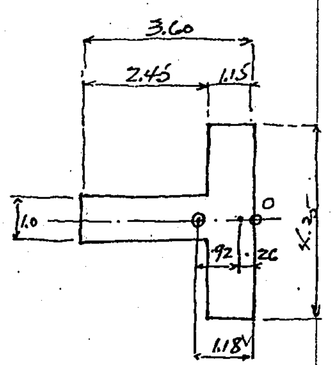
Earth pressure at normal state $\frac{1}{3} \times 1600 \times 6.8 = 3637$

$\frac{1}{3} \times 1600 \times .5 = \frac{267}{3904 \div 2} = 1952$ kg/m² average.

$E_1 = 1952 \times 6.3 \times 4.25 = 52300$ kg

Earth pressure during earthquake $.662 \times \frac{6.3^2}{2} \times 4.25 \times 1600 = 89300$ kg $= E_1'$

Case 1. Stresses at normal state.



Taking moment about point O. in the above sketch.

Loads	Hor. forces	Vert. forces	lev. arm	moment
S_1		76000 ×	1.13 =	85,800
w_1		15,100 ×	3.40 =	51,300
E_1	52,300		2.17 ×	−113,500
	52,300	91,100	0.26 m	23,600

Center of gravity of section

$1.15 \times 4.25 = 4.88 \times .575 = 2.805$

$1.0 \times 2.45 = 2.45 \times 2.375 = 5.810$

$7.33 \times 1.18\text{ m} = 8.615$

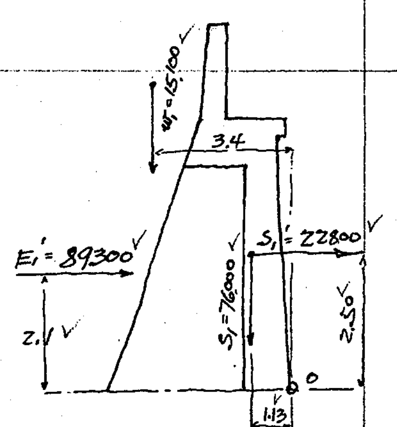
Eccentricity $= 1.18 - .26 = 0.92$ m

Moment at bottom section $= 91100 \times .92 = 83800$ kgm.

Shear 〃 〃 $= 52300$ kg

Case 2 Stresses during earthquake.

Taking moment about point O.



Loads	Hor. forces	Vert. forces	lev. arm	moment
S_1		76,000 ×	1.13 =	85,800
S_1'	22800	×	2.50 =	−57,000
w_1		15,100 ×	3.40 =	51,300
E_1'	89,300		2.10 =	−187,500
	112,100 kg	91,100 kg ×	−1.18 m =	−107,400

Eccentricity $= 1.18 + 1.18 = 2.36$ m

moment at bottom section $= 91100 \times 2.36 = 215,000$ kgm

Shear 〃 〃 $= 112,100$ kg.

Steel area req'd for moment $= \frac{215000 \times 100}{2160 \times \frac{7}{8} \times 350} = 32.6$ cm²

Use 9-22φ bars $= 34.2$ cm²

$f_s = \frac{215000 \times 100}{34.2 \times \frac{7}{8} \times 350} = 2055$ kg/cm²

Direct comp. $= \frac{91100 \times 15}{7.33 \times 10000} = \frac{-19}{2036}$ kg/cm² ok

$f_c = \frac{f_s k}{n(1-k)} = \frac{2055 \times .080}{15 \times .920} = 11.9$

Direct comp. $= \frac{91100}{7.33 \times 10000} = \frac{1.2}{13.1}$ kg/cm² ok.

$p = \frac{34.2}{425 \times 350} = 0.00023$

$\frac{t}{d} = \frac{1.15}{3.5} = 0.33$

Neutral axis in the flange.

$k = \sqrt{2 \times 15 \times .00023 + (.00023 \times 15)^2} - .00023 \times 15$

$= 0.08$

$j = 1 - \frac{1}{3}k = .973$

Ibi – Nagara Bashi for Mie Ken.

Unit shear $= \dfrac{112100}{100 \times \frac{7}{8} \times 350} = 3.7\ \text{kg/cm}$ ok.

Unit bond $= \dfrac{112100}{6.91 \times 9 \times \frac{7}{8} \times 350} = 5.9\ \text{kg/cm}^2$ ok.

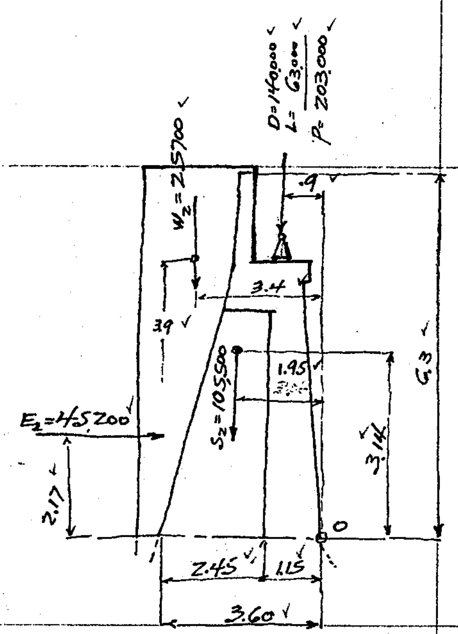
Design of Counterfort under truss bearing.

Superimposed load on abutment

Dead Load	140,000
Live Load	63000
	203000 kg for one shoe.

Weight and center of gravity of Counterfort under truss bearing.

					arm	vert. m	arm	hor. m
Granite	.25 × 3 × 1.63 =	.122 @	2600 =	320	6.15	1980	1.65	530
Light pedestal		2.500 @	" =	6500	8.40	54600	3.70	24050
Handrail	3.0 m × 250 kg =	.750 @	" =	1950	6.80	13250	1.70	3310
parapet wall	.425 × 1.6 × 1.63 =	1.11 @	2400 =	2660	5.15	13700	1.66	4410
Top beam	1.8 × 1.0 × ~~1.63~~ 3.13 =	5.63 @	" =	13520	3.88	52400	1.29	17440
Coping	.1 × .3 × 3.13 =	.09 @	" =	220	4.25	940	.35	80
Projection	1.0 × 1.3 × 3.30 =	4.28 @	" =	10270	4.75	48750	3.70	38000
Wing wall.	.35 × 2.9 × 6.4 =	6.50 @	" =	15600	3.20	49900	2.90	45200
Coping	.1 × .3 × 3.0 =	.09 @	" =	220	6.25	1380	1.60	350
Front wall	.99 × 3.4 × 3.13 =	10.53 @	" =	25300	1.57	39700	0.50	12650
Counterfort	1.875 × 3.4 × 1.70 =	10.83 @	" =	26000	1.50	39000	2.09	54300
wall	.42 × 2.0 × 1.70 =	1.43 @	" =	3420	4.8	16420	1.65	5640
less	.3 × .2 × 3.4 =	(–) .20 @	" =	(–) 480	1.7	(–) 1820	1.75	(–) 600
Sum of concrete		40.29 m³		105540 kg	3.14 m	331200	1.95 m	205360
" granite		3.372		Call this 105800 kg				



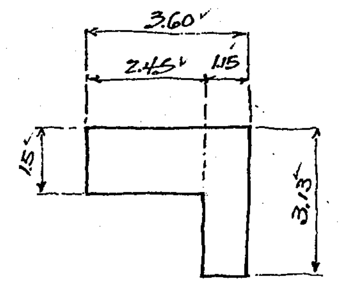
Weight of earth on counterfort wall.

$1.5 \times 1.7 \times 6.3 \ @\ 1600 = 25700\ \text{kg}$

Earth pressure at normal state $E_2 = 1952 \times 6.3 \times 3.68 = 45200\ \text{kg}$

Earth pressure during earthquake $E_2' = 89300 \times \frac{3.68}{4.25} = 77400$

assumed section



Center of gravity of section.

1.15 × 3.13 =	3.60	× .575 =	2.07
1.5 × 2.45 =	3.67	× 2.375 =	8.71
	7.27 m²	× 1.48 m =	10.78

Case 1. Stress at normal state.

Taking moment about point O.

Loads	Hor. forces	Vert forces.		lev. arm		moment.
P.		203000	×	0.90	=	182600
S_2		105500	×	1.95	=	205800
w_2		25700	×	3.40	=	87400
E_2	45200		×	– 2.17	=	– 98000
	45200	334200		1.13		377800

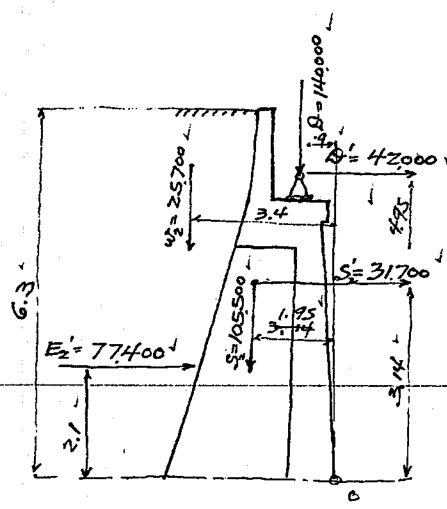
Eccentricity $= -1.13 + 1.48 = 0.35\ \text{m}$

moment at bottom section $= 334200 \times 0.35 = 117000\ \text{kgm}$.

Shear $= 45200\ \text{kg}$

Ibi-Nagara Bashi for Mie Ken.

Case 2. Stresses during Earthquake. (Seismic forces forward).



Taking moment about point O.

Loads	Hor. forces	Vert. forces		Lev. arm		Moment.
D		140,000	×	0.9	=	− 126000
D'	42000		×	4.95	=	208000
S_2		105500	×	1.95	=	− 205500
S_2'	31700		×	3.14	=	99500
w_2		25700	×	3.40	=	− 87400
E_2'	77400		×	2.10	=	162500
	151100	271200	×	0.189 m	=	51100

Eccentricity = 1.48 + 0.19 = 1.67 m

moment at bottom section = 271200 × 1.67 = 452500 kgm.

Shear " " " = 151100 kg.

Steel area required for moment = $\frac{452500 \cdot 100}{2160 \times \frac{7}{8} \times 350}$ = 68.5 cm²

$p = \frac{68.7}{313 \times 350} = .00063$

use 14 − 25 mmφ bars = 68.7 cm²

$\frac{t}{d} = \frac{1.15}{3.5} = 0.33$

$fs = \frac{452500 \cdot 100}{68.7 \times \frac{7}{8} \times 350} = 2150$

neutral axis in the flange

$pn = .00063 \times 15 = .00945$

$(pn)^2 = .00009$

Direct comp. = $\frac{271200 \times 15}{727 \times 10000}$ = −56

2094 kg/cm² < 1200 × 1.8 = 2160 ok.

$k = \sqrt{2 \times .00945 - .00009} - .00945$

$= .128$

$fc = \frac{2150 \times .128}{15 \times .872} = 21.2$

Direct comp. = $\frac{271200}{727 \times 10000}$ = 3.8

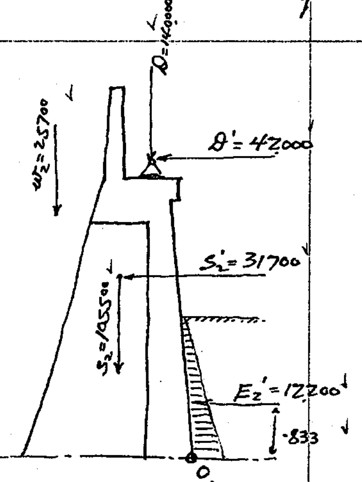
25.0 kg/cm² < 35 × 1.8 = 63. ok

Unit Shear = $\frac{151100}{150 \times \frac{7}{8} \times 350}$ = 3.3 kg/cm² ok.

Unit bond = $\frac{151100}{7.85 \times 14 \times \frac{7}{8} \times 350}$ = 4.5 kg/cm² ok.

Case 3. Stresses during Earthquake (seismic forces backward)

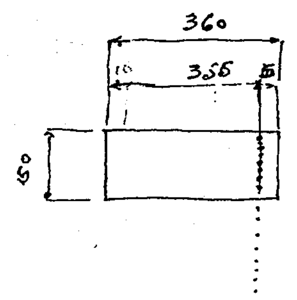
Earthpressure front side $E_2' = .662 \times \frac{2.5^2}{2} \times 1600 \times 3.68$ = 12,200 kg



Taking moment about point O.

Load.	Hor. forces	Vert. forces		Lev. arms		Moments.
D		140,000	×	0.90	=	126000
D'	42000		×	4.95	=	208000
S_2		105500	×	1.95	=	205500
S_2'	31700		×	3.14	=	99500
w_2		25700	×	3.40	=	87400
E_2'	12200		×	.833	=	10200
	85900	271200	×	2.72 m	=	736600

Eccentricity = 2.72 − 1.48 = 1.24 m



moment at bottom section = 271200 × 1.24 = 336200 kgm

Shear " " " = 85900 kg

Steel area required for moment = $\frac{336200 \times 100}{2160 \times \frac{7}{8} \times 355}$ = 50.1 cm²

use 8 − 22φ = 30.4

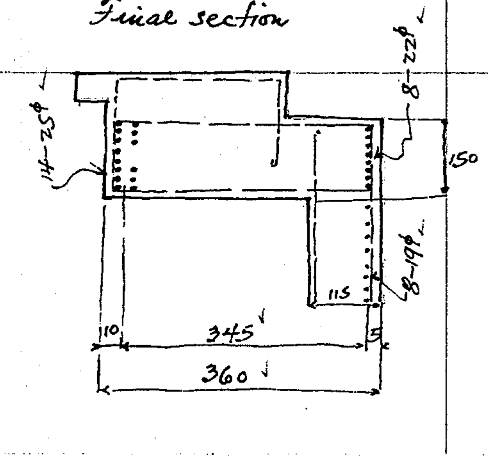
8 − 19φ = 22.7

53.1 cm²

$p = \frac{53.1}{150 \times 355}$ = 0.001 $k = .158$ $j = .947$

Final section

$fs = \frac{336200 \times 100}{53.1 \times .947 \times 355}$ = 1880



Direct comp. = $\frac{271200 \times 15}{7.27 \times 10000}$ = −56

1824 kg/cm² ok.

$fc = \frac{1880 \times .158}{15 \times .842}$ = 23.5

Direct comp. = $\frac{271200}{7.27 \times 10000}$ = 3.8

27.3 kg/cm² ok.

Unit shear & bond ok.

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CHECKED BY M. Jinda DATE 5-2-19 PAGE NO 8

CALCULATIONS FOR

Ibi-nagara Bashi for Mie Ken.

Stability of abutment

Summary of Concrete for one abutment

Counterfort, center	31.34
" side 2×40.29 =	80.58
Base	150.91
	262.83 m^3

Superimposed Load on abutment.

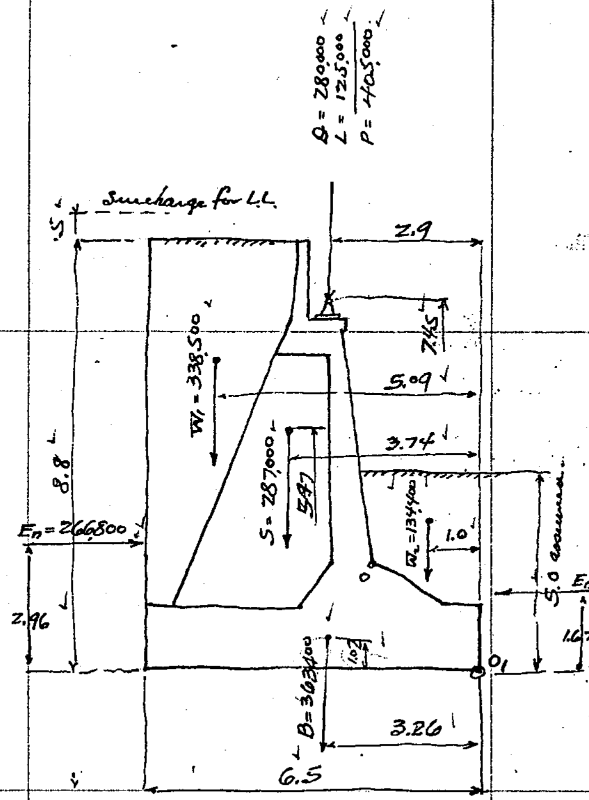
Dead Load	280000
Live Load	125000
	405000 kg for one abutment.

Weight and center of gravity of shaft.

weight	vert. lev. arm	moment	hor. lev. arm	moment
76000	2.50	189940	1.13	85680
105500	3.14	331200	1.95	206,080
105500	3.14	331200	1.95	206,080
287000 kg.	2.97 m	852340	1.74 m from pt. O	497840

Weight of earth fill on rear footing.

2.5 × 7.3 × 11.30 @ 1600 =	330,000 ×	5.25 =	1,732,000
.85 × 4.4 × 11.30 @ 1600 =	67,600 ×	3.58 =	242,000
2.1 × 4.4 × 4.0 @ 1600 = (−)	59,100 ×	4.25 =	− 251,000
	338,500 kg	5.09 m from toe.	1,723,000



Weight of earth on front footing. Depth assumed 3.5 m

3.5 × 2 × 12 @ 1600 = 134,400 kg arm 1.0 m from toe.

Earth pressure on rear.

normal state $\frac{1}{3} \times 1600 \times 0.5 = 267$ Surcharge for LL = 0.5 m assumed

$\frac{1}{3} \times 1600 \times 9.3 = \underline{4955}$

$5222 \div 2 = 2611$ kg/m² average

Earth pressure $E_n = 2611 \times 8.8 \times 11.6 = 266800$ kg arm 2.96 m

during earthquake $E_e = .662 \times 1600 \times \frac{8.8^2}{2} \times 11.6 = 476,000$ kg arm 2.93 m

Earth pressure on front.

normal state $E_n' = \frac{1}{3} \times 1600 \times \frac{5^2}{2} \times 11.6 = 77,300$ kg arm 1.67 m

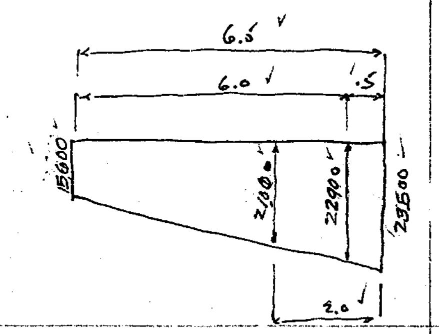
during earthquake $E_e' = .662 \times 1600 \times \frac{5^2}{2} \times 11.6 = 153,700$ kg " "

Weight and center of gravity of Base.

			vert. arm	m	hor. arm	m
2.95 × 1.0 × 10.5 =	21.52 @ 2400 =	51600 ×	1.93	99500	2.40	124,000
4.0 × 1.0 × 2.3 =	9.20 @ 2400 =	22100 ×	2.00	44200	4.80	106,000
2.9 × .55 × $\frac{1.0}{7.4}$ × 2 =	3.19 @ 2400 =	7700 ×	2.00	15400	4.90	37700
6.5 × 1.5 × 12.0 =	117.00 @ 2400 =	282000 ×	0.75	210,800	3.25	917,000
	150.91 m^3	363,400 kg	1.02 m	369,900	3.26 m	1,184,700

Case 1. Stability at normal state.

see above sketch.



Taking moment about toe O₁.

Loads	Hor. forces	Vert. forces		lev. arms		Moment
P.		405,000	×	2.90	=	+ 1,175,000
S		287,000	×	3.74	=	+ 1,073,000
W₁		338,500	×	5.09	=	+ 1,722,000
W₂		134,400	×	1.00	=	+ 134,400
B.		363,400	×	3.26	=	+ 1,184,700
Eₙ	− 266,800		×	2.96	=	− 790,000
Eₙ'	77,300		×	1.67	=	+ 129,000
	344,100 / −189,500	1,528,300		3.03 m		4,628,100

Eccentricity = 3.25 − 3.03 = 0.22 m

Resultant force within middle third

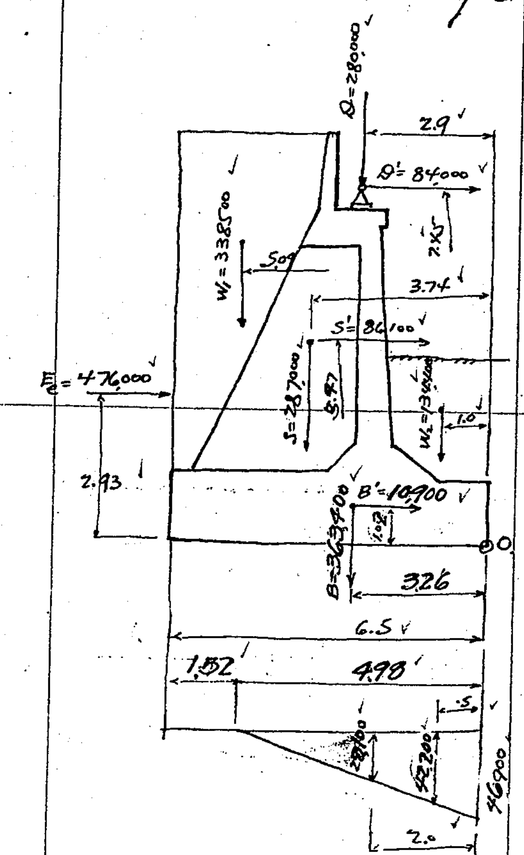
max toe pressure = $\frac{1528300}{6.5 \times 12.0}\left(1 \pm \frac{6 \times .22}{6.5}\right) = \begin{matrix} 23500 \\ 15600 \end{matrix}$ kg/m² $= \begin{matrix}(2.14 \text{ tons/} \square) \\ (1.43 \quad)\end{matrix}$ ok (.203)

max. load on one pile = 2290 × 1.0 × 1.0 = 22.90 kg tons.

If 50% be carried by sand foundation pile load = 11.5 kg tons per pile.

Ibi-Nagara Bashi for Mie ken.

Case 2. Stability during Earthquake (Seismic forces forward)



Taking moment about toe o.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		280,000	2.90 =	+ 812,000
D'	84000		7.45 =	− 626,000
S		287,000	3.74 =	+ 1,073,000
S'	86100		5.47 =	− 471,000
B		363000	3.26 =	+ 1,183,000
B'	109000		1.02 =	− 111,000
W_1		338500	5.09 =	+ 1723000
W_2		134400	1.00 =	+ 134400
E_e	476000		2.93 =	− 1395000
	755100	1403300	1.66 =	+ 2322400

Eccentricity = 3.25 − 1.66 = 1.59 m

Resultant force outside of middle third, neglecting tension

Pressure area = 1.66 × 3 × 12 = 59.8 m²

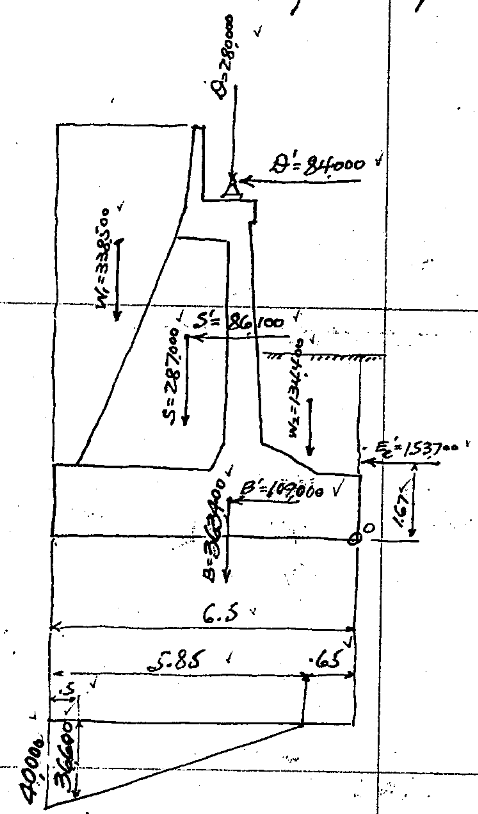
max. toe pressure = $\frac{1403300 \times 2}{59.8}$ = 46900 kg/m² or (4.28 tons/□')

max. load on one pile = 42.2 × 1.0 × 1.0 = 42.2 kg tons.

If 12.0 × 1.8 = 21.5 kg tons/□m be carried by sand foundation

Pile load = 42.2 − 21.5 = 20.7 kg tons per pile

Case 3. Stability during Earthquake (Seismic forces backward).



Taking moment about O.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		280,000	2.90 =	812000
D'	84000		7.45 =	626000
S		287,000	3.74 =	1,073000
S'	86100		5.47 =	471,000
B		363,400	3.26 =	1,183000
B'	109000		1.02 =	111,000
W_1		338,500	5.09 =	1,723000
W_2		134400	1.00 =	134400
E_e'	153700		1.67 =	256,500
	432,800	1403300	4.55 =	6389900

Eccentricity = 4.55 − 3.25 = 1.30 m

Resultant force outside of middle third, neglecting tension,

Pressure area = 1.95 × 3 × 12 = 70.2 m²

max bearing pressure at heel = $\frac{1403300 \times 2}{70.2}$ = 40000 kg/m² or (3.66 tons/□')

max. load on one pile = 36.6 × 1 × 1 = 36.6 kg tons.

If 21.5 kg tons/□m be carried by sand foundation

Load on pile = 36.6 − 21.5 = 15.1 kg tons per pile.

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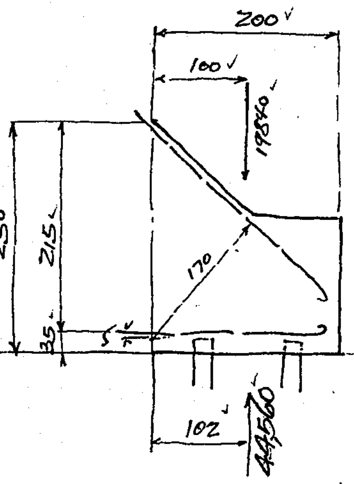
CALCULATIONS FOR

Ibi-Nagara Bashi for Mie ken.

Design of Cantilever footing at toe.



Loads on cantilever footing at normal state (case 1)

upward pressure 21060, 23500, 44560 ÷ 2 × 2 = 44560 kg

wt. of earth fill on footing 3.5 × 1600 × 2 = − 11,200
wt. of footing concrete 1.8 × 2400 × 2 = − 8,640 } 19840 kg
24720 kg

Moment on footing

44560 × 1.02 = 45500
19840 × 1.0 = 19840
moment = 25,660 kgm per meter strip
Shear = 24720 kg

During earthquake (case 2)

upward pressure 28100, 46900, 75000 ÷ 2 × 2 = 75000 kg

moment on footing

75000 × 1.09 = 81700
19840 × 1.0 = 19840
moment = 61860 kgm per meter strip
Shear = 55160 kg

Effective depth req'd. = $\sqrt{\dfrac{61860 \times 100}{100 \times 1.8 \times 7.18}} = 69.7$ cm

use effective depth of 215 cm with 5 cm insulation.

Steel area required = $\dfrac{61860 \times 100}{2160 \times \frac{7}{8} \times 215} = 15.22$ cm²

use 25 mmφ bars at 30 cm ctoc = 16.35 cm²

steel ratio $p = \dfrac{16.35}{215 \times 100} = .0008$, $k = .147$, $j = .951$

Unit shear = $\dfrac{55160}{100 \times .951 \times 215} = 2.70$ kg/cm² ok.

unit bond = $\dfrac{55160}{7.85 \times 3.33 \times .951 \times 215} = 10.4 < 6 \times 1.8 = 10.8$ ok.

Top reinforcements.

negative moment on footing = 19840 kgm. effective depth = 170 cm.

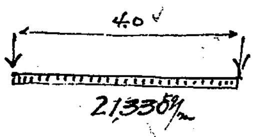
Steel area req'd. = $\dfrac{19840 \times 100}{2160 \times \frac{7}{8} \times 170} = 6.17$ cm²

use 16 mmφ bars at 30 cm ctoc = 6.71 cm².

Design of footing at heel.

upward pressure during earthquake (case 3) for extreme meter strip = 36600 kg/m

downward pressure due to earth fill = 7.3 × 1600 = 11670
" " footing = 1.5 × 2400 = 3600
15270
21330 kg/m



Span length assumed 4.0 m

moment = $\frac{1}{10} \times 21330 \times 4.0^2 = 34100$ kgm

Shear = 21330 × 2 = 42660 kg.

effective depth req'd. = $\sqrt{\dfrac{34100 \times 100}{100 \times 1.8 \times 7.18}} = 51.3$ cm use 115 cm

Steel area req'd. = $\dfrac{34100 \times 100}{2160 \times \frac{7}{8} \times 115} = 15.7$ cm² per meter strip.

use 22 mmφ bars at 25 cm ctoc = 15.2 cm² for extreme 1 meter.

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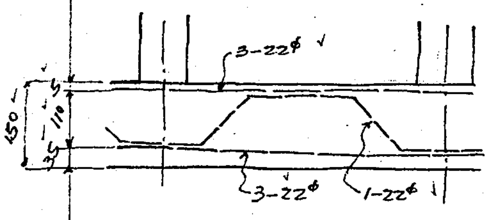
CALCULATIONS FOR

Ibi-Nagara Bashi for Mie-ken.



1 meter strip at end.

upward pressure on rear footing = 0 for case 2.

downward pressure = 15270 kg/m

moment = $\frac{1}{10} \times 15270 \times 4^2 = 24400$ kgm

Steel area req'd = $\frac{24400 \times 100}{2160 \times \frac{7}{8} \times 115} = 11.2$ cm²

Use 3-22φ bars = 11.4 cm² per meter at end.

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CALCULATIONS FOR

Materials of Ibi-Nagara Bashi for Mie Ken.

Materials of Abutment.
Concrete 1:2:4 mixture.

parapet wall ✓	.425✓ × 1.75✓ × 7.50✓	=	5.580✓
Top beam ✓	1.06 × 1.774✓ × 5.45 × 2✓	=	19.340✓
less (end)	1.08✓ × 1.0✓ × .2✓ × 2✓	= (−)	.432✓
Coping ✓	.1✓ × .3✓ × 12.8✓	=	.384✓
front wall ✓	.985✓ × 3.3 × 10.5✓	=	34.100✓
Parapet end (front) ✓	.425✓ × 2.075✓ × 1.70✓ × 2✓	=	2.997✓
coping ✓	.10✓ × .38✓ × 3.70✓ × 2✓	=	.244✓
wing wall ✓	.35✓ × 2.90✓ × 7.375✓ × 2✓	=	14.980✓
projection under pedestal	1.30 × .95✓ × 3.30 × 2✓	=	8.580✓
Counterfort walls ✓	1.92✓ × 4.4✓ × 4.30✓	=	36.320✓
less	.30✓ × .20✓ × 4.30 × 2✓	= (−)	.516✓
Base	2.05✓ × 1.00✓ × 5.25 × 2✓	=	21.520✓
〃	1.50 × 6.50 × 12.00✓	=	117.000✓

260.097✓ cub. meters.

Reinforcements, Plain Bars. 6.649 kg tons. see drawings.

Forms.

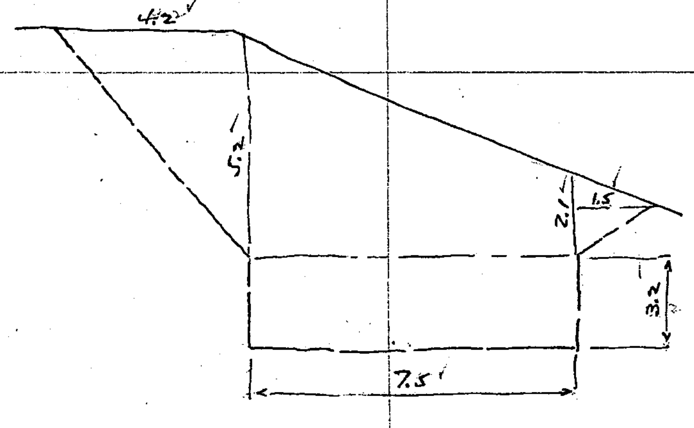
parapet wall ✓	1.75 × 2 × 7.50✓	=	26.25✓
〃 end.	2.075 × 2 × 3.75✓	=	15.55✓
wing wall outside	2.90✓ × 7.375 × 2✓	=	42.75✓
〃 end.	.35✓ × 7.375 × 2✓	=	5.16✓
〃 front.	.55✓ × 5.30✓ × 2✓	=	5.83✓
〃 projection	.95✓ × 2 × 3.3 × 2✓	=	12.54✓
〃	.95✓ × 1.3✓ × 2✓	=	2.47✓
〃 coping	.15✓ × 3.75✓ × 2✓	=	1.13✓
〃 inside	2.475 × 2.075 × 2	=	10.27✓
〃 〃	1.40✓ × 5.30✓ × 2	=	14.84✓
Curtain wall front.	6.00✓ × 10.5✓	=	63.00✓
〃 Sides	1.21✓ × 4.40✓ × 2✓	=	10.65✓
〃 〃	1.98✓ × 1.0✓ × 2✓	=	3.96✓
Top Beam back.	1.10✓ × 10.9✓	=	11.99✓
〃 bottom	1.18✓ × 3.25✓ × 2✓	=	7.67✓
Counterfort back.	4.4✓ × 4.70✓	=	20.70✓
〃 Sides	1.92✓ × 4.3✓ × 4✓	=	33.05✓
Curtain wall back.	3.25✓ × 3.3✓ × 2✓	=	21.45✓
〃 〃	3.25✓ × 1.2✓ × 2✓	=	7.80✓
Base	1.5✓ × 12.0✓ × 2✓	=	36.00✓
〃	1.5✓ × 6.5✓ × 2✓	=	19.50✓

372.56✓ sq. meters

Rubbles for Foundation	7.0✓ × 17.5✓ × 0.60✓		=	52.5✓ cub. meters.
Piles 内地產赤松 末口	21cm 長 4.8m✓	6 × 12✓	=	72✓ piles.
Granite 踏掛石.	.25✓ × .25✓ × .93✓ × 8✓		=	0.465✓ cub. meters.

Excavation.

average section



Average section
5.2✓ × 4.2✓ ÷ 2✓ = 10.9✓
3.65✓ × 7.5✓ = 27.4✓
2.1✓ × 1.5✓ ÷ 2✓ = 1.6✓
39.9✓ × 17.5✓ = 698✓ cub. meters L.W. 以上.

7.5✓ × 13.0✓ × 3.2✓ = 312.✓
水替坊 1.5 × 1.5 × 3.2 = 7✓
319✓ L.W. 以下
1,017✓ cub. meters

矢板延長 (7.5✓ + 13✓) × 2 = 41.✓
水替坊 3✓
44✓ meters.

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CALCULATIONS FOR

Materials of Ibi-Nagara Bashi for Mie ken.

人造洗出仕上

wing wall. outside	2.9 × 1.75 × 2 =	10.15
〃 projection	0.5 × 1.3 × 2 =	1.30
〃 〃	1.6 × .95 × 2 =	3.04
〃 coping	.15 × 3.65 × 2 =	1.10
〃 top.	0.50 × 3.45 × 2 =	3.45
〃 〃	1.3 × 1.3 × 2 =	3.38
Parapet. front	2.05 × 2.075 × 2 =	8.50
〃	.8 × .55 × 2 =	.88
inside of coping	0.2 × 4.25 × 2 =	1.70
		33.50 sq. meters.

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CALCULATIONS FOR

MADE BY ______ DATE ______ FILE NO ______
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昭和五年八月

國道第壹號路線

三重縣揖斐長良川橋梁假棧橋

設計々算書並ニ鋼材表

三重縣揖斐長良川橋假棧橋設計

We have decided as follows:-

Total Length of Temporaly bridge 1,100 meters more or less.

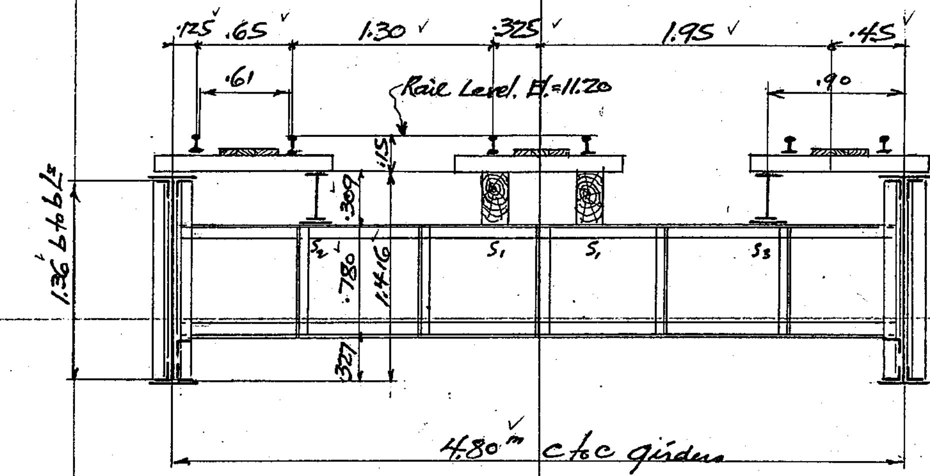
15 girder spans	@	20.0	=	300
44 " "	@	18.0	=	792
58 clealances	@	0.1	=	5.8
				1,097.8 meters between ends of girders.

3 - tracks 61cm guage (24") 12# rails.

Max. load on one truck = 6.5 kg-tons. 2 cars on one span in single line. 25% impact max.

Load due to transportation of earth and gravel ∝ 1.5 kg-tons fully loaded. " " max.

Crossection of Bridge as shown on sketch below.



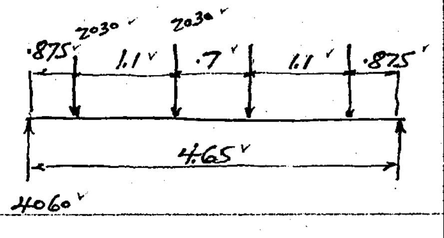
Design of I-beam stringers span length 4.65 meters for 20m girder.

Dead Load
Track assumed $\frac{1}{2} \times 50 = 25$
Beam " 50
75 kg per lin meter

Dead Load moment $= \frac{1}{8} \times 75 \times 4.65^2 = 203$ kgm.
Dead Load shear $= \frac{1}{2} \times 75 \times 4.65 = 175$ kg

Live Load

Max. concentration on one wheel $= \frac{1}{4} \times 6500 = 1625$
25% impact say 405
2030 kg.



moment $= 4060 \times 4.65 \div 2 = 9440$
$2030 \times .35 = -\ 710$
$2030 \times 1.45 = -\ 2940$
5790 kgm.

End shear $= 4,060$ kg

Summary of moments and shears.

	moments	shear
Dead Load	203	175
Live Load	5790	4060
	5993 kgm	4235 kg

Section modulus required $= \frac{599300}{1100} = 545$ cm³

Use 300×150 I @ 48.34 kg Sm. = 633.2 cm³ ok.

Design of wooden stringers for center track.

This track assumed to be used for transportation of materials only.

Dead Load
Track assumed $\frac{1}{2} \times 50 = 25$
beam " 30
55 kg per lin m

Dead Load moment $= \frac{1}{8} \times 55 \times 4.65^2 = 149$ kgm
Dead Load shear $= \frac{1}{2} \times 55 \times 4.65 = 128$ kg.

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CALCULATIONS FOR

三重縣揖斐長良川橋假棧橋設計

Live Load

Load for one truck = 1500 kg
For one stringer = 1500 ÷ 2 = 750
25 % impact = 187
937 kg

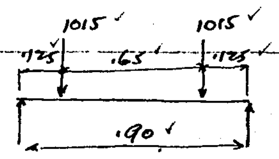
Assuming this load to be uniformly distributed on the length of 1.8 meters
uniform load = 937 ÷ 1.8 = 520 kg per lin m.
Live Load moment = $\frac{1}{8} \times 520 \times 4.65^2$ = 1,405 kgm
" " shear = $\frac{1}{2} \times 520 \times 4.65$ = 1,208 kg

Summary of moments and shears

	moments	shears
Dead Load	149	128
Live Load	1405	1208
	1554 kgm	1336 kg

Allowable unit stress of wood assumed 85 kg/cm²
Section modulus req'd = $\frac{1554 \times 100}{85}$ = 1830 cm³
Use 15cm × 30cm beam Sm = $\frac{15 \times 30^2}{6}$ = 2250 cm³ ok.
unit shear = $\frac{1336}{15 \times 30}$ = 3 kg/cm² ok.

Design of Cross sleeper
max wheel concentration with impact = 2030 kg.
This load assumed to be resisted by 2 sleepers.
load on one sleeper = 2030 ÷ 2 = 1015 kg.

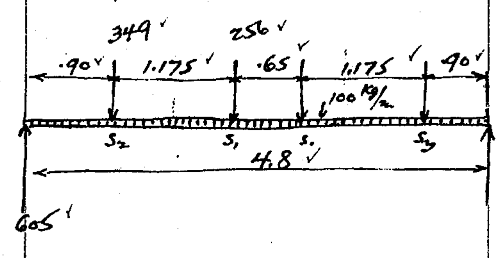


moment = 1015 × .125 = 127 kgm
Shear = 1015 kg.
Section modulus required = $\frac{127 \times 100}{85}$ = 150 cm³
Use 10cm × 10 sleeper, Sm = $\frac{10 \times 10^2}{6}$ = 167 cm³ ok.
unit shear = $\frac{1015}{10 \times 10}$ = 10.15 kg/cm² ok.

Design of Intermediate floor beam: span length 4.80 meters, spacing 4.65 meters.

Dead Load

Concentration on stringers.
wood stringer S_1 55 × 4.65 = 256 kg
I beam " $S_2 + S_3$ 75 × 4.65 = 349 "
floor beam assumed 100 kg per lin meter.

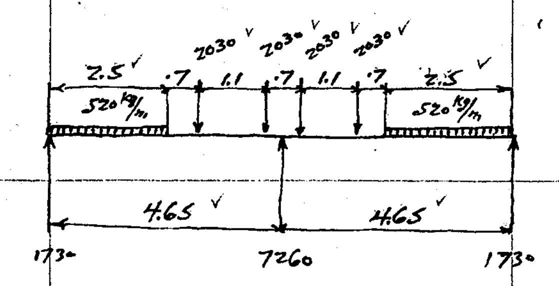


Dead Load moment.
605 × 2.075 = 1255
349 × 1.175 = − 410
845 kgm
$\frac{1}{8} \times 100 \times 4.8^2$ = 288
1133 kgm.

Dead Load shear 100 × 2.4 = 240
605
845 kg

Live Load

max. wheel concentration with impact = 2030 kg.
uniform live load 520 kg per lin meter.
Stringer concentrations
$S_1 + S_2$ 520 × 4.65 = 2420 kg



S_3 " 2030 × 3.2 = 6500
2030 × 4.3 = 8730
$\frac{520 \times 2.5^2}{2}$ = 1625
16,855 ÷ 4.65 = 3630

Reaction = 2 × 3630 = 7260 kg

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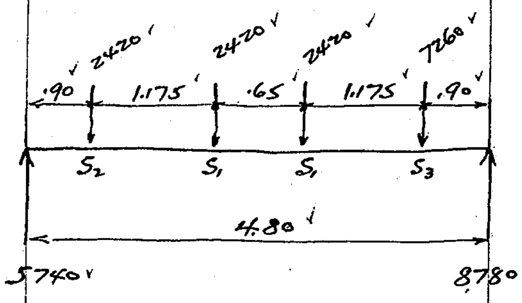
CALCULATIONS FOR

三重縣揖斐長良川假棧橋設計



Reaction

7260 × 3.900 =	28300
2420 × 2.725 =	6600
2420 × 2.075 =	5020
2420 × 0.900 =	2180
	42100 ÷ 4.8 = 8780 kg

Moment under S_1.

8780 × 2.075 =	18220
7260 × 1.175 =	− 8530
	9690 kgm.

right side Shear 8780 kg

5740 × 2.075 =	11900
2420 × 1.175 =	− 2840
	9060 kgm.

left side Shear 5740 kg

Summary for moments and shears.

	moments	shears.
Dead Load	1133	845
Live Load	9690	8780
	10823 kgm	9625 kg

web assumed 77.0 cm × .8 = 61.6 cm² $\frac{1}{8}$ web = 7.70 cm²
Back to back of Ls = 78.0 cm
Effective depth say 73.7
Flange stress = $\frac{10823 \times 100}{73.7}$ = 14700 kg cm T

Flange area reqd = $\frac{14700}{1200}$ = 12.25 cm² net
$\frac{1}{8}$ web = − 7.7
4.55 cm² net.

Use 2Ls 75 × 75 × 9 = 25.38 − 3.96 = 21.42 cm² ok.

Approximate weight of Intermediate floor beam.

Flange	4Ls	75 × 75 × 9	e	9.96 × 4.70 =	187
web	1Pl.	770 × 8	e	48.36 × 4.80 =	232
End connection	2Ls	150 × 90 × 9	e	16.32 × 0.62 =	20
stiffeners	8Ls	75 × 65 × 8	e	8.28 × 0.78 =	52
base of stringers	2Pls	310 × 9	e	21.90 × 0.30 =	13
					504

Rivet heads and variations say 3½% abt = 16
520 kg

For End floor beams, use same details as for intermediate one above decided.

Lateral Bracings.

Use 1 L 125 × 75 × 10. l/r = 688/3.96 = 174 ok.

Approximate weight of Lateral bracings.

Diagonals.	8Ls	125 × 75 × 10	e	14.91 × 6.30 =	751
Center conn.	4Pls	270 × 9	e	19.08 × 0.51 =	39
conn. to girder	10pls	345 × 9	e	24.37 × 0.60 =	146
					936

Rivet heads and variation say 1½% 14
950 kg for one span

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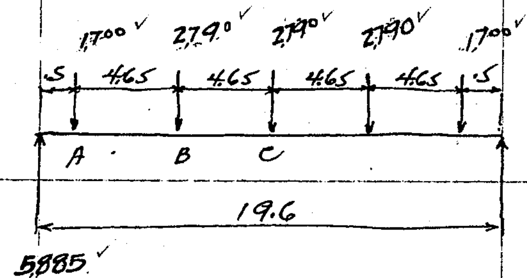
CALCULATIONS FOR

三重縣揖斐長良川橋假桟橋設計

Design of Main Girders - Span length 19.60 meters c to c bearings, 20.0 meter out to out.
Dead Load

Tracks	3 @ 50 =	150.
Stringers S_1	2 @ 30 =	60
〃 $S_2 + S_3$	2 @ 50 =	100
floor beam 520 ÷ 4.65	=	112
lateral bracings	950 ÷ 19.6 =	49
main girders assumed	2 @ 325 =	650
Air pipes &c say		79
		1200 kg per lin meter

For one girder 1200 ÷ 2 = 600 kg
Let us assume the load concentrated at panel point.
Intermediate panel concentration = 600 × 4.65 = 2790 kg for one girder
End 〃 = 600 × 2.83 = 1700 kg



moment at A. 5885 × 0.5 = 2940 kgm.

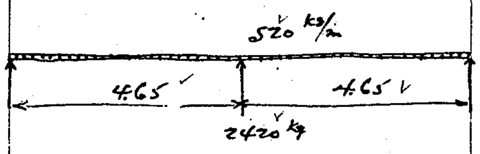
moment at B. 5885 × 5.15 = 30300
1700 × 4.65 = - 7900
22400 kgm.

moment at C 5885 × 9.80 = 57,600
1,700 × 9.30 = - 15800
2,790 × 4.65 = - 12,980
28820 kgm

End shear = 5885 kg

Load on shoe say 10.0 @ 600 = 6,000
Shoe say 150
6,150 kg

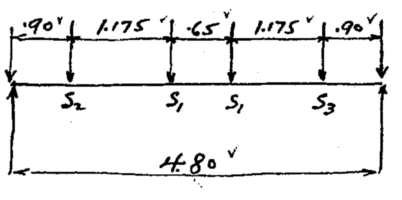
Live Load.
Stringer concentration on floor beam due to 520 kg unif. load.



520 × 4.65 = 2420 kg intermediate panel pt.
520 × 3.03 = 1580 kg end 〃 〃

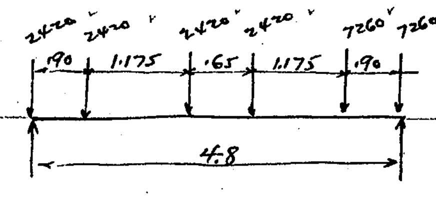
Due to concentration and unif load.
7260 kg see page 2.
~~1730~~ 〃
1730 + 2420/2 = 2940 kg

Floor beam concentration on main girder.
Unif. load only. 2420 × 3 = 7260 kg int. panel pt.
1580 × 3 = 4740 〃 end 〃

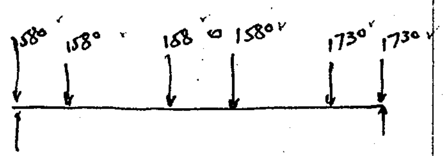


Unif. load + concentration

2420 × 5.7 ÷ 4.8 = 2875
7260 × 3.9 ÷ 4.8 = 5900
7260
16035 kg. panel pt. between concentrations



2420 × 5.7 ÷ 4.8 = 2875
2940 × 3.9 ÷ 4.8 = 2390
2940
8205 kg panel pt next to concentration.



1580 × 5.7 ÷ 4.8 = 1875
1730 × 3.9 ÷ 4.8 = 1405
1730
5010 kg end.
520 × 7 × 2 = 730 unif load outside of bearing
5740 kg

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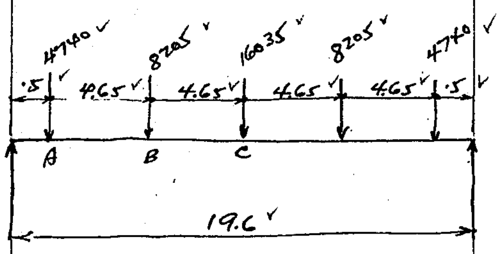
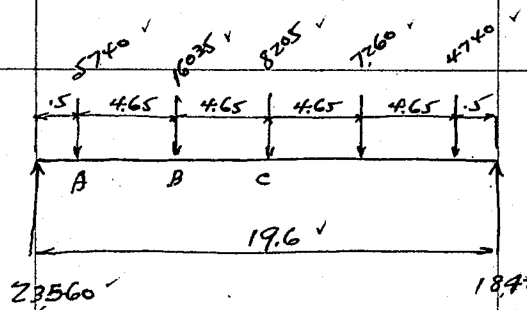
三重縣揖斐長良川橋假棧橋設計

Moment at panel pt. C



20963 × ~~2.40~~ 9.80 = 205500
4740 × 9.3 = − 44100
8205 × 4.65 = − 38100
123300 kgm

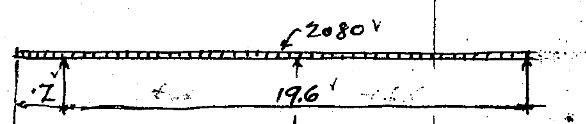
panel pt. B.
Reaction = 4740 × 19.1 = 90500
7260 × 14.45 = 105000
8205 × 9.80 = 80400
16035 × 5.15 = 82500
5740 × .5 = 2870
361,270 ÷ 19.6 = 18420 kg



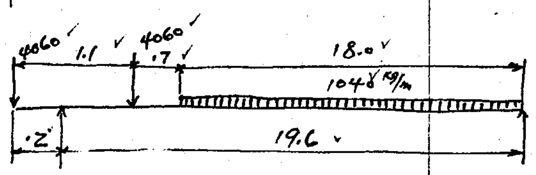
41,980
18420
23,560 kg

moment at B. 23560 × 5.15 = 121000
5740 × 4.65 = 26700
94300 kgm

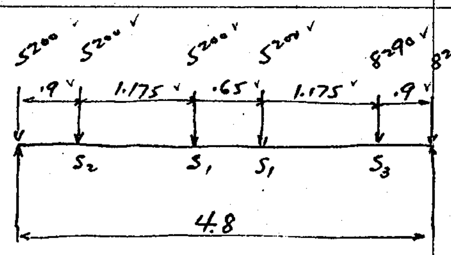
max. Load on shoe.
Reaction due to uniform load S_1 S_2 + left girder.
uniform load = 520 × 4 = 2080 kg per lin m. for 2 lines.
Load on shoe = $\frac{2080 \times 19.8^2}{2 \times 19.6}$ = 20800 kg or 4 × 5200



Reaction due to truck and unif. load on S_3 – right girder



4060 × 18.7 = 75900
4060 × 19.8 = 80400
$\frac{1040 \times 18.0^2}{2}$ = 168500
324800 ÷ 19.6 = 16580 kg or 2 × 8290



max. load on shoe.
5200 × .9 = 4680
5200 × 2.075 = 10790
5200 × 2.725 = 14170
8290 × 3.90 = 32300
61,940 ÷ 4.8 = 12900
8290
21200 kg

For end shear use same value on safe side.

Summary of moments and shears.

	moments at C	~~Shea~~ M at B.	End shear	Load on shoe
Dead Load	28820	22400	5885	16150
Live Load.	123300	94300	21200	21200
	152,120 kgm	116700 kgm	27085 kg	27350 kg

三重縣揖斐長良川橋假棧橋設計

Section of main girders
Depth of web plate 135 cm web area 135 × 9 = 121.5 cm², 1/8 web area = 15.2 cm²
136 cm back to back of Flange Ls. Effective depth say 136.0 − 4 = 132.0 cm

Flange stress = $\frac{152120}{132}$ = 115300 kg cm T

Flange section required = $\frac{115300}{1200}$ = 96.3
1/8 web = −15.2
81.1 cm² net

Use 2 Ls 150 × 150 × 11 = 63.58 − 11.00 = 52.58
1 cov. pl. 340 × 13 = 44.20 − 6.5 = 37.70
107.78 cm² gr 90.28 cm² net

Unit stress = $\frac{115300}{90.28 + 15.2}$ = 1095 kg/cm² net ok

Approximate weight of main girder

4 Ls	150 × 150 × 11	@ 24.95	× 20.0	=	1,996
2 cov. pls.	340 × 13	@ 34.70	× 12.2 m ell	=	847
1 web pl	1350 × 9	@ 95.378	× 20.0	=	1907
					4750
Details + Rivet heads say 32 %					1520
					6,270 kg 314 kg/lin m.

Approximate weight of steel in one span. (20 m span).

Stringers	2 × 50 × 20	=	2,000
Floor beams	5 @ 520	=	2600
lateral bracing			950
main girders	2 @ 6270	=	12,540
Shoes say	4 @ 175	=	700
			18,790 kg call this 18.80 kg tons.

Approximate weight of steel in 18 m span. Use same depth and similar section.

Stringers	2 × 50 × 18	=	1,800
Floor beams	5 @ 520	=	2600
lateral bracing	say		920
main girders	2 @ 5390	=	10780
Shoes say	4 @ 175	=	700
			16800 kg or 16.80 kg tons.

Summary of structural steel for the whole bridge.

20 m span	15 spans @ 18.80	=	282.
18 m span	44 " @ 16.80	=	739
			1021 kg tons 本橋用鋼材
Less field rivets .143 × 59		=	−8.5
			1012.5 kg tons.
machine bolts say 59 @ .158 = 9.3			
turned bolts say 59 @ .228 = 13.5			22.8
			1035.3 kg tons. 仮棧橋用鋼材

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CALCULATIONS FOR

MADE BY ________ DATE ________ FILE NO ________
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昭和五年九月

三重縣揖斐長良川橋梁

基礎工事假棧橋用鋼鈑桁

材料調書

徑間 十八米突及二拾米突

4

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MADE BY I.M. DATE ____ FILE NO ____
CHECKED BY R.I. DATE 5-8-11 PAGE NO M1

CALCULATIONS FOR

Material list of temporary bridge for Ibinagaragawa-bashi (20.0m Plate girder)

No.	Description.	Section in Mm.	Length in Mm.	Wt. of One Meter.	Wt. of Main Section in Kgs.	Wt. of Details in Kgs.	Total wts.	Remarks.
	GIRDER	G1L & G1AL		4 Req'd.				
4	Flg Ls.	150x150x11	10,000	24.95	998.0			
1	Web Pl.	1,350 x 9	10,000	95.378	953.8			
2	Cov Pls.	340 x 13	6,310	34.697	437.9			
4	Ls	125x90x10	1,338	16.09		86.1		
2	Fills	180 x 11	1,055	15.543		32.8		
16	Ls.	125x75x10	1,360	14.91		324.4		
1	Conn. Ls	150x100x9	155	17.02		2.6		
2	"	"	305	"		10.4		
1	"	"	230	"		3.9		
1	Sole Pl.	400 x 25	460		2,389.7	+ 460.2	= 2,849.9	
							x 4	
							11,399.6	
	SPLICE AT CENTER			2 Req'd.				
2	Pls.	340 x 13	1,050	34.697		72.9	296.3	
4	Ls.	150x150x15	930	33.55		124.8		
4	Pls	230 x 13	630	23.472		59.1		
2	"	330 x 13	586	33.677		39.5		
2	Fills.	75 x 13	1,055	7.654		16.1		
2	Ls	125x75x10	1,308	14.91		39.0		
1	Conn. L.	100x75x10	230	12.95		3.0		
1	"	"	305	"		3.9		
							358.3	
							x 2	
							716.6	
	Summary of Main Girders				12,116.2			
	FLOOR BEAM FB1			4 Req'd.				
2	Flg Ls	75x75x9	4,780	9.96		95.2		
2	"	"	4,530	"		90.2		
1	Web Pl.	770 x 8	4,780	48.356		231.1		
10	Ls	75x65x8	780	8.28		64.6		
2	"	125x90x10	620	16.09		20.0		
2	Pls	290 x 9	310	20.489		12.7		
							513.8	
							x 4	
							2,055.2	
	FLOOR BEAM FB2			1 Req'd				
2	Flg Ls	75x75x9	4,730	9.96		94.2		
2	"	"	4,480	"		89.2		
1	Web Pl.	770 x 8	4,730	48.356		228.7		
10	Ls	75x65x8	780	8.28		64.6		
2	"	125x75x10	515	14.91		15.4		
2	Pls.	290 x 9	310	20.489		12.7		
							504.8	
	Summary of Floor beams				2,560.0			
	STRINGERS							
4	Is	300x150@48.34 kg	5,345			1,033.5		
4	"	"	4,640			897.2		
12	Pls	170 x 9	230	12.011		33.2		
							1,963.9	

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CHECKED BY R.I DATE 5.8-11 PAGE NO M2

CALCULATIONS FOR

Material list of temporary bridge for Ibinagaragawa-bashi (20.0 m Plate girder)

LATERAL BRACINGS 1 Req'd.

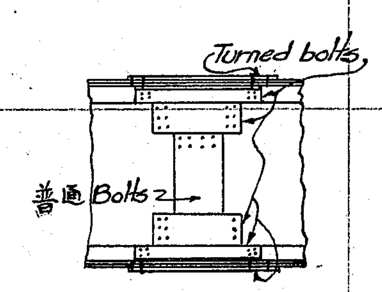
4	Ls	125x75x10	6250	14.91	372.8	
8	"	"	3075	"	366.8	
4	Pls	280 × 9	520	19.782		41.1
4	"	300 × 9	550	21.195		46.6
4	"	"	625	"		53.0
2	"	305 × 9	625	21.548		26.9
					739.6 + 167.6 =	907.2

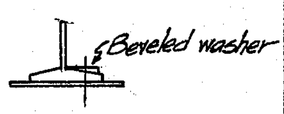
SHOES & ANCHOR BOLTS

4	Cast iron shoes		@ 140	560.0	
4	Pl 70 × 30	160	10.486	10.6	
16	Anchor bolts 25φ	700	@ 3.13	50.1	
16	Washers 100 × 9	100	7.065	11.3	
8	Bolts 22φ	115	@ 0.650	5.2	
					637.2

BOLTS (假橋用)

280	22.5φ Turned bolts length abt.	75	@ 0.6	168.0	
210	22φ bolts " "	60	@ 0.5	105.0	
316	19φ " " "	50	@ 0.32	101.1	
40	Beveled washers 50 × 12 × 50		@ 0.18	7.2	
					381.3





Grand summary of 20.0 m Plate girder

		Kgs.	
Summary of	Main girders.	12,116.2	
	Floor beams.	2,560.0	
	Stringers.	1,963.9	
	Bracings.	907.2	
	Shoes and Anchor bolts.	637.2	
	Bolts. (假橋用)	381.3	
		18,565.8	or 18.566 Kg tons
	Shop rivet-heads	580.0	or .580
			19.146

増田淳氏関係資料（独立行政法人 土木研究所蔵）

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MADE BY R. I DATE ______ FILE NO ______

CHECKED BY I.M. DATE 5-8-11 PAGE NO M3

CALCULATIONS FOR

Material list of temporary bridge for Ibinagaragawa-bashi (18.0m Plate girders)

No.	Description	Section in Mm.	Length in Mm.	Wt. of One Meter	Wt. of Main Section in Kgs.	Wt. of Details in Kgs	Total wt.	Remarks
	GIRDER		G2E & G2AE		4 Req'd.			
4	Flg Ls.	150.150.11	9.000	24.95	898.2			
1	Web Pl.	1,350 × 9	9.000	95.378	858.4			
2	Cov Pls.	340 × 9	4.400	24.021	211.4			
4	End stif. Ls	125.90.10	1.338	16.09		86.1		
2	Fills	190 × 11	1.055	16.407		34.6		
14	Stif Ls	125.75.10	1.360	14.91		283.9		
1	Sole Pl.	400 × 25	460	78.50		36.1		
2	Conn Ls	150.100.9	230	17.02		7.8		
1	〃	〃	305	〃		5.2		
					1,968.0	+ 453.7	= 2,421.7	
							× 4	
							9,686.8	
	SPLICE				2 Req'd.			
4	Ls	150.150.15	930	33.55		124.8		
4	Pls	230 × 13	630	23.472		59.1		
2	〃	330 × 13	586	33.677		39.5		
2	Fills	75 × 13	1.055	7.654		16.1		
2	Pls	340 × 9	810	24.021		38.9		
2	Ls	125.75.10	1.308	14.91		39.0		
1	Conn Ls	100.75.10	230	12.95		3.0		
1	〃	〃	305	〃		3.9		
							324.3	
							× 2	
							648.6	
	Summary of Main Girder				10,335.4			
	FLOOR BEAM		FB 1.		2 Req'd.			
2	Flg Ls.	75.75.9	4780	9.96		95.2		
2	〃	〃	4530	〃		90.2		
1	Web Pl.	770 × 8	4780	48.356		231.1		
10	Ls.	75.65.8	780	8.28		64.6		
2	〃	125.90.10	620	16.09		20.0		
2	Pls.	290 × 9	310	20.489		12.7		
							513.8	
							× 2	
							1,027.6	
	FLOOR BEAM		FB 2		1 Req'd.			
2	Flg Ls.	75.75.9	4730	9.96		94.2		
2	〃	〃	4480	〃		89.2		
1	Web Pl.	770 × 8	4730	48.356		228.7		
10	Ls.	75.65.8	780	8.28		64.6		
2	〃	125.75.10	515	14.91		15.4		
2	Pls.	290 × 9	310	20.489		12.7		
							504.8	
	FLOOR BEAM		FB3		2 Req'd.			
4	Flg Ls.	75.75.9	4510	9.96		179.7		
1	Web Pl.	770 × 8	4770	48.356		220.7		
10	Ls.	75.65.8	780	8.28		64.6		
2	Pls.	290 × 9	310	20.489		12.7		
2	Fills.	120 × 8	245	7.536		3.7		

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CALCULATIONS FOR

Material list of temporary bridge for Ichinagaragawa-bashi (18.0^m. Plate girder)

2	Fills.	120 × 8	310	7.536		4.7
2	〃	〃	1.338	〃		20.2
						512.3
						× 2
						1,024.6

Summary of Floor beams. 2,557.0

STRINGER — 1 Req'd.

4	Is	300×150 @ 48.34 Kgs	4.595		888.5	
4	〃	〃	4.390		848.9	
12	Pls	170 × 9	230	12.011		33.2
					1,737.4 +	33.2 = 1,770.6

LATERAL BRACING — 1 Req'd.

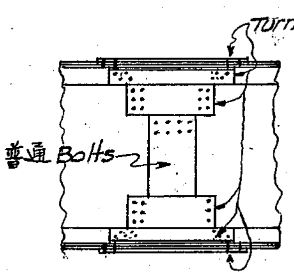
4	Ls.	125×75×10	6.110	14.91	364.4	
8	〃	〃	3.005	〃	358.4	
4	Pls.	280 × 9	520	19.782		41.1
4	〃	310 × 9	410	21.902		35.9
4	〃	300 × 9	625	21.195		53.0
2	〃	305 × 9	625	21.548		26.9
					722.8 +	156.9 = 879.7

SHOES & ANCHOR BOLTS

4	Cast iron shoes			@ 140	560.0
4	Pls.	70 × 30	160	16.486	10.6
16	Anchor bolts.	25^φ	700	@ 3.13	50.1
16	Washers.	100 × 9	100	7.065	11.3
8	Bolts.	22^φ	115	@ 0.650	5.2
					637.2

BOLTS (假橋用)

264	22.5^φ Turned bolt	Length ab't. 75	@	0.6	158.4
270	22^φ Bolts	〃 〃 60	@	0.5	135.0
300	19^φ 〃	〃 〃 50	@	0.32	96.0
40	Beveled washers	50 × 12 × 50	@	0.18	7.2
					396.6



Grand Summary of 18.0^m Plategirder

	Kgs.	
Girder.	10,335.4	
Floor beams	2,557.0	
Stringers	1,770.6	
Lateral bracings	879.7	
Shoes & Anchor bolts.	637.2	
Bolts	396.6	Kgtons.
	16,576.5	or 16.577
Shop rivet heads	440.0	or .440
		17.017

Summary of structural steel

20^M. girders	15 spans	@	19.146	287.190
18^M. girders	44 〃	@	17.017	748.748
				1,035.938 Kgtons

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CALCULATIONS FOR

MADE BY ________ DATE ________ FILE NO ________

CHECKED BY ________ DATE ________ PAGE NO ________

昭和五年八月

三重縣府縣道一八米及二〇米鋼鈑桁

設計々算書並ニ鋼材表

府縣道径間十八米及二十米鈑桁橋計算書

Design of 18 meter and 20 meter girder spans

Design of floor for girder span

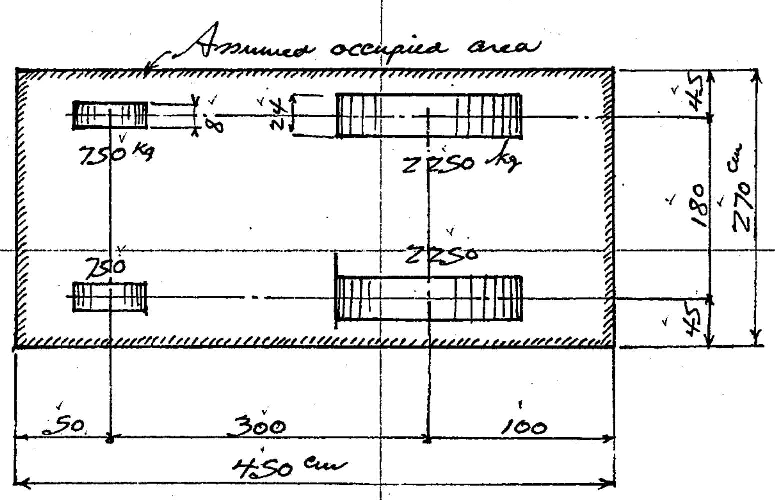
Clear width of Roadway = 5.40 meters
Pavement 3.8 cm asphaltic block on reinforced concrete slab.

Assumed Loadings

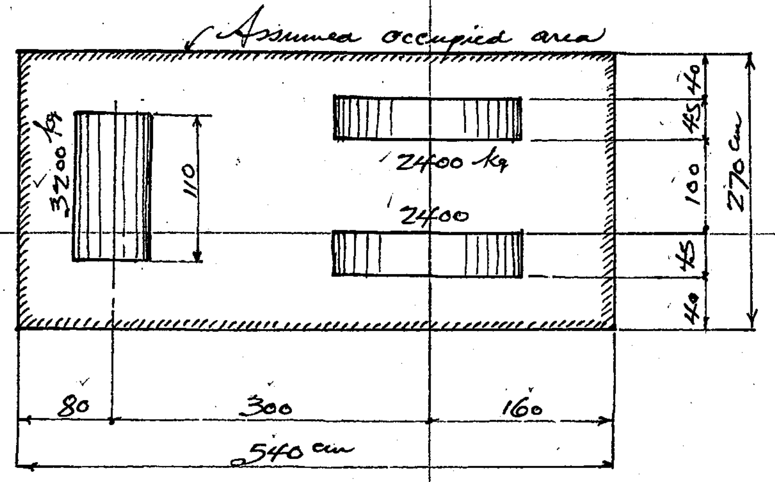
Uniform load on Roadway $w = \frac{100000}{170+l} \leq 500$ kg/m²

where w = Uniform load in kg per sq. meter.
l = span length in meters.

6 ton motor truck loading



8 ton road roller loading



Two rows of motor traffic on roadway with occupied width of 270 cm ; Unoccupied space arround the motor truck shall be filled with uniform load specified above.
One road roller on one span.
Impact for motor truck loading Coef = $\frac{20}{60+l}$ where l = loading length in meter
max. impact 30 %.

No impact for road roller and uniform live load.

Allowable working strength
Structural steel or Reinforcing bars.

Tension, net	1200 kg/cm²
Extreme fibre stress, net	1200 "
Shear of web, gross section	900 "
Compression member $1500\,(1-0.0055\, l/r)$ not over	1000 "

where l = length of member in cm
r = least radius of gyration in cm

Compression flange of girder $1200\,(1-0.012\, l/b)$ not over	1100 "
Shear on shop driven rivets (machine driven)	850 "
" " field " (") and turned bolts	750 "
Shear on pin	900 "
Bearing on shop driven rivets (machine driven)	1700 "
" " field "	1500 "
" " pin	1800 "

Rollers 45d kg where d = diameter of roller in cm

Concrete 1:2:4 mixture.

Direct compression	35 kg/cm²
Fibre stress due to bending	45 "
Combined stress direct and bending	35 "

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MADE BY K. Inaba DATE 5-7-29 FILE NO B&C

CHECKED BY M.K DATE 5-8-6 PAGE NO 2

CALCULATIONS FOR

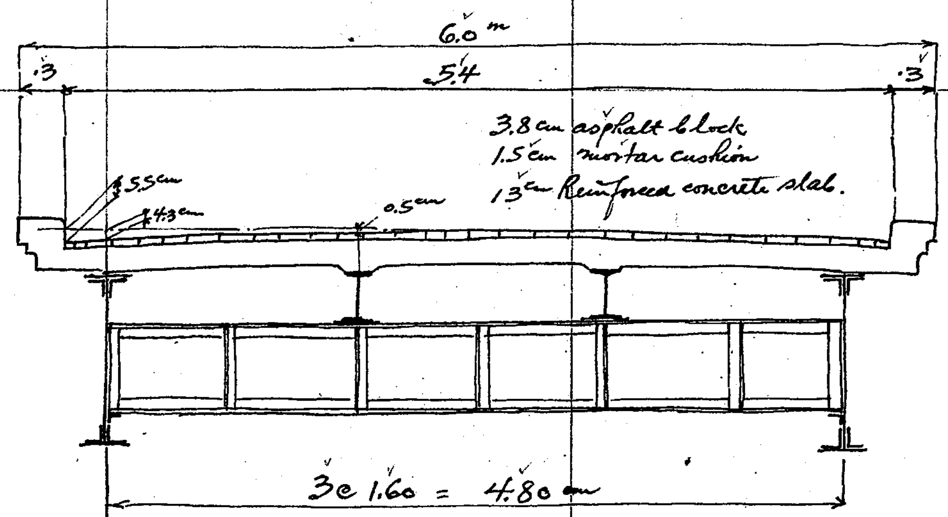
Design of 18 meter and 20 meter girder spans

Concrete 1:2:4 mixture.

Punching shear of Concrete	9 kg/cm²
Shear of plain concrete	4 "
Bearing	45 "
Bond stress for plain bars	6 "
" " " deformed "	9 "

For combined wind or temperature stress with dead, live, and impact stress, the allowable working strength shall be increased 25%. In case of earthquake, increase unit stress by 80%. Seismic acceleration assumed 3000 mm/sec² or k = 0.30

Design of 18.0 meter girder span.
Cross Section of Bridge as shown on sketch below.



Design of Floor slab. span length = 1.60 meters.
Dead load

Asphalt block pavement	3.8 cm	@ 21 kg	=	80
mortar cushion	1.5 cm	@ 17 "	=	26
Concrete slab	13.0 cm	@ 24 "	=	312
				418
misc concrete say			=	12
				430 kg/m²

Dead load moment $= \frac{1}{10} \times 430 \times 1.60^2 = 110$ kgm

Dead load shear $= \frac{1}{2} \times 430 \times 1.60 = 345$ kg

Live load motor truck loading. Rear wheel concentration 2250
Impact 30% = 675
2925 kg

Front wheel concentration $\frac{2925}{3} = 975$ kg

Distribution of wheel concentration
Contact between wheel and pavement 20.0
distribution 2×5.3 = 10.6
Longitudinal distribution a = 30.6 cm
Transverse distribution $b = 24 + 10.6 = 34.6$ cm

Effective width $E = \frac{2}{3} \cdot l + a$ where l = span length in meters.
$= \frac{2}{3} \times 1.60 + .306 = 1.37$ meters.
Load per meter strip $= 2925 \div 1.37 = 2140$ kg

Uniform load 500 kg/sq. meter.

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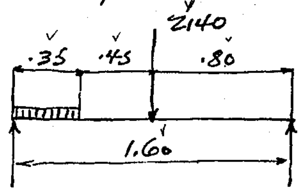
MADE BY K. Inaba DATE 5-7-29 **FILE NO** B&C

CHECKED BY M.K DATE 5-8-6 **PAGE NO** 3

CALCULATIONS FOR

Design of 18 meter and 20 meter Girder Spans

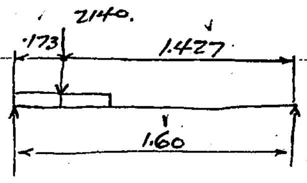


Uniform load $\frac{500 \times 0.35^2}{2 \times 1.60} = 19.1$ kg

moment at center of span
Due to uniform load $19.1 \times 0.8 = 15$
Due to motor truck $\frac{2140}{2} \times 0.8 = 856$
871 kgm.

For continuity of slab moment $= 0.8 \times 871 = 697$ kgm

End shear.



$2140 \times \frac{1.427}{1.60} = 1870$ kg

Summary for moments and shears.

	moments	shears
Dead Load	110	345
Live Load	697	1870
	807 kgm	2215 kg

Effective depth required for $f_s = 1200$ kg/cm² and $f_c = 45$ kg/cm²
$R = \frac{M}{bd^2}$, $d = \sqrt{\frac{M}{bR}}$ where $R = 7.18$ $d = \sqrt{\frac{807 \times 100}{100 \times 7.18}} = 10.6$ cm

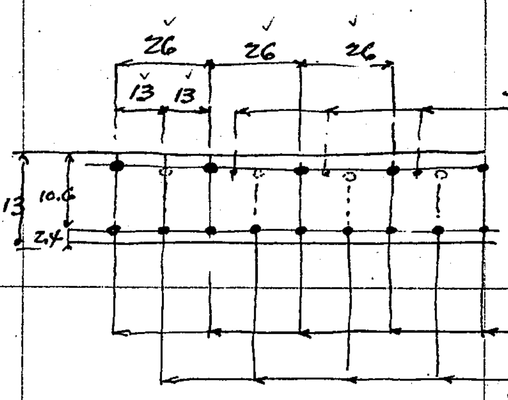
Use 13 cm slab with insulation at bottom of 2.4 cm.

Steel area required $= \frac{807 \times 100}{1200 \times \frac{7}{8} \times 10.6} = 7.25$ cm² per meter strip.

use 12 mm φ bars at 13 cm ctoc = 8.70 cm² ok.

Perimeter of reinforcing bars.

dia.	circumf.		
12 mm	3.77	× 7.70 =	29.00 cm



Unit bond $= \frac{2215}{29.0 \times \frac{7}{8} \times 10.6} = 8.24$ kg/cm² ok for deformed bars.

if plain bars are used. add bond bars 9φ at 26 cm ctoc
perimeter of reinforcing bars

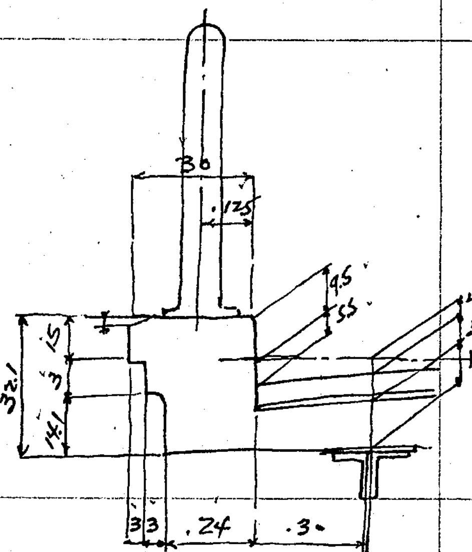
12 mm	3.77 × 7.70 =	29.00	
9 "	2.83 × 3.85 =	11.00	
		40.00	cm

Unit bond $= \frac{2215}{40.0 \times \frac{7}{8} \times 10.6} = 5.96$ kg/cm² Ok for plain bars.

Unit shear $= \frac{2215}{100 \times \frac{7}{8} \times 10.6} = 2.39$ kg/cm² ok.

Overhanging Slab beyond main girder.

Approximate weight of Handrail 70 kg per lin meter
" " Coping 192 " "



Dead Load	load			
Handrail	70.	× 0.425 =	29.8	
Coping	192	× say .44 =	84.5	
Slab and pavement 430 × .3 =	129	× .15 =	19.4	
	391 kg	.34 ~	133.7	

Live Load motor truck rear wheel at 15 cm in side of curb line assumed
Distribution or effective width on main girder assumed thus
$2 \times 0.30 + 0.20 = 0.80$ m

Transverse distribution $24 + 10.6 = 34.6$ say 35 cm
Load on 30 cm $= 2925 \times \frac{30}{35} = 2500$ kg

for one meter strip $= 2500 \div .8 = 3125$ kg per meter strip.
Live Load moment $= 3125 \times .15 = 467$ kgm.
Shear assumed punching shear sheared length along girder 80
Transverse length $2 \times 30 = \frac{60}{140}$ cm
for one meter $3125 \div 1.40 = 2235$ kg

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Design of 18 meter and 20 meter Girder spans

Summary for moments and shears.

	moments	shears.
Dead Load	134	391
Live Load	467	2235
	601 kgm	2626 kg

Effective depth required $= \sqrt{\frac{601\times100}{100\times7.18}} = 9.15$ cm

Use 13 cm slab, eff. depth 10.60 cm.

Unit shear $= \frac{2626}{100\times\frac{7}{8}\times10.6} = 2.83$ kg/cm² ok.

Steel area required $= \frac{601\times100}{1200\times\frac{7}{8}\times10.6} = 5.40$ cm² per meter strip.

use 12 mm φ bars at 13 cm c to c $= 8.70$ cm² ok.

for bond stress.

main bars 12 mm φ $3.77\times7.70 = 29.00$

bond bars 9 mm φ $2.83\times7.70 = 21.80$

$50,80$ cm = total perimeter of bars.

unit bond $= \frac{2626}{50.8\times\frac{7}{8}\times10.6} = \underline{5.6}$ kg/cm² ok for plain bars.

for deformed bars use 2 – 9φ bond bars about, total perimeter $2\times2.83 =$ 29.00 + 5.66 = 34.66

unit bond $= \frac{2626}{34.66\times\frac{7}{8}\times10.6} = \underline{\underline{8.2}}$ kg/cm² ok for deformed bars.

Temperature bars in slab.

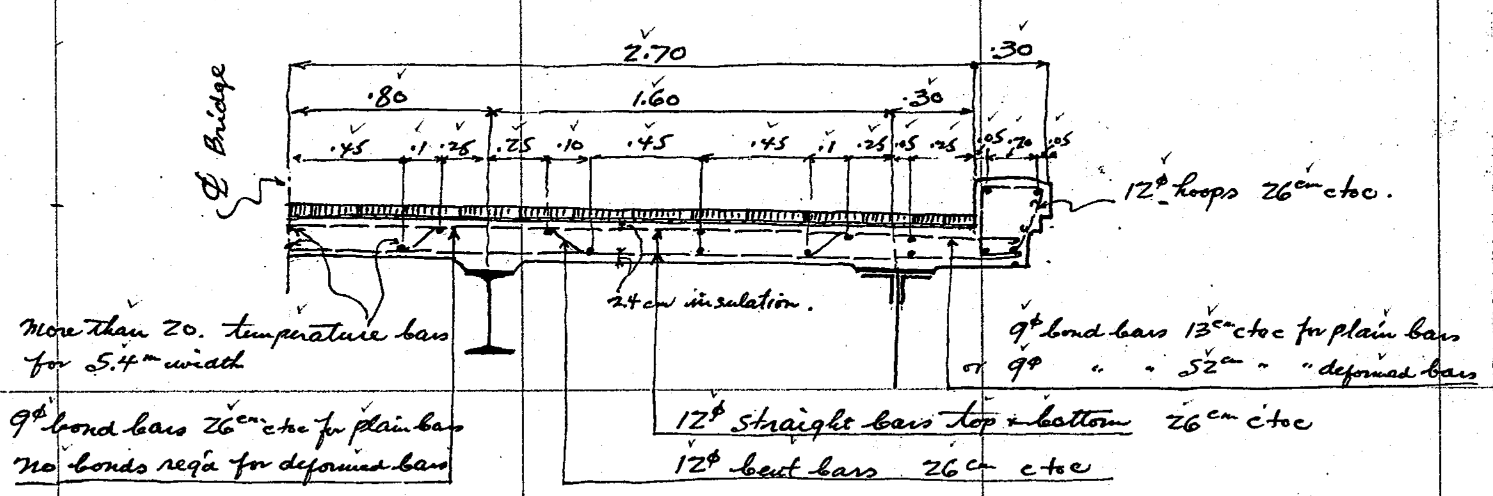
Steel area $\frac{3}{1000}$ of cross sectional area of slab concrete.

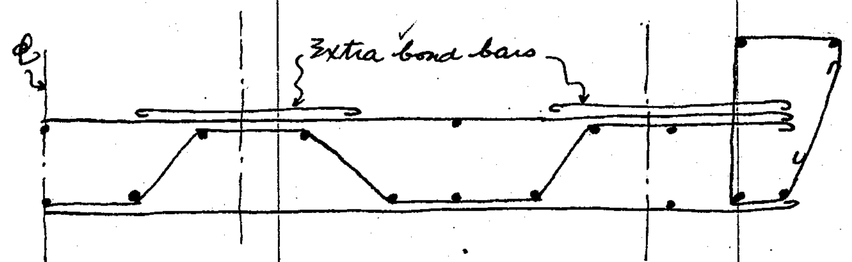
Sectional area of floor slab. $= 13\times540 = 7020$ cm²

Sectional area of temperature bars $= \frac{3}{1000}\times7020 = 21.0$ cm²

12 mm φ bars, required no $= \frac{21.0}{1.13} = 18.6$ bars.

Use more than 20 bars. longitudinally.





General sketch for slab reinforcements.

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CALCULATIONS FOR

Design of 18 meter girder span

Design of I Beam Stringer span length = 4.40 meters. spacing 1.60 meters.

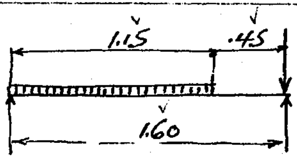
Dead Load

Floor slab and pavement 430 × 1.60 = 688
Stringer assumed 50
738 call this 740 kg/lin meter.

Dead Load moment = 1/8 × 740 × 4.4² = 1,790 kgm
Dead Load shear = 1/2 × 740 × 4.4 = 1,630 kg

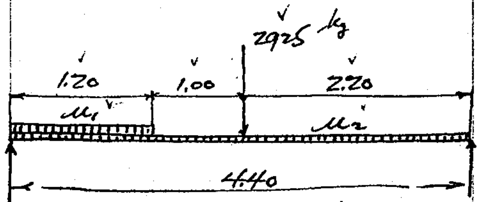
Live Load

motor truck rear wheel concentration with impact = 2925 kg
Uniform live load = 500 kg per sq. meter.

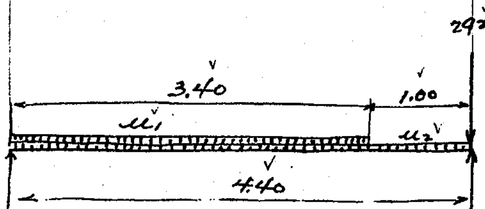


load on stringer = $\frac{500 \times 1.15^2}{2 \times 1.60}$ = 207 kg ------- U_2

full uniform load = 500 × 1.60 = 800
less − 207
593 kg ------- U_1



moment.
Due to motor truck $\frac{2925}{2}$ × 2.20 = 3220
Uniform load U_1 $\frac{593 \times 1.20^2}{2 \times 4.4}$ × 2.20 = 214
Unif. load U_2 1/8 × 207 × 4.4² = 501
3935 kgm.



End shear.
Uniform load U_1 $\frac{593 \times 3.40^2}{2 \times 4.40}$ = 780
Uniform load U_2 207 × 4.4 ÷ 2 = 455
motor truck loading 2925
4160 kg

Summary for moments and shears.

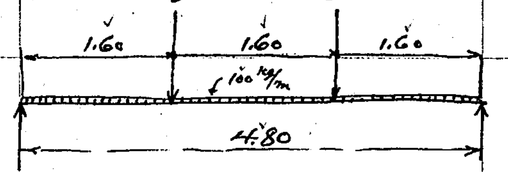
	moments	shears
Dead Load	1790	1630
Live Load	3935	4160
	5725 kgm	5790 kg

Section modulus required = $\frac{5725 \times 100}{1100}$ = 521.0 cm³
use 300 × 150 I @ 48.34 kg S_m = 633.2 cm³ ok.

Unit stress = $\frac{572500}{633.2}$ = 905 kg/cm² C or T ok.
Unit shear = $\frac{5790}{30 \times .8}$ = 241 kg/cm² ok.

Design of Intermediate floor beam. span length 4.80 meters, spacing 4.40 meters.

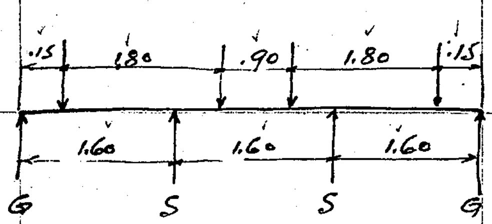
Dead Load Concentration on stringer = 740 × 4.40 = 3260 kg
floor beam assumed 100 kg per lin meter of span.



Dead Load moment
due to stringer concentration 3260 × 1.60 = 5220
due to weight of floor beam 1/8 × 100 × 4.8² = 288
5508 kgm

End shear from concentration = 3260
from beam wt. 1/2 × 100 × 4.8 = 240
3,500 kg

Live Load motor truck loading, rear wheel concentration with impact = 2925 kg
front wheel " = 975 kg



Load on S. $\frac{.15}{1.60}$ = 0.094
1.000
1.094 × 2925 = 3200 kg — rear wheel
1.094 × 975 = 1065 kg — front wheel.

Uniform live load 500 × 1.60 = 800 kg on stringer

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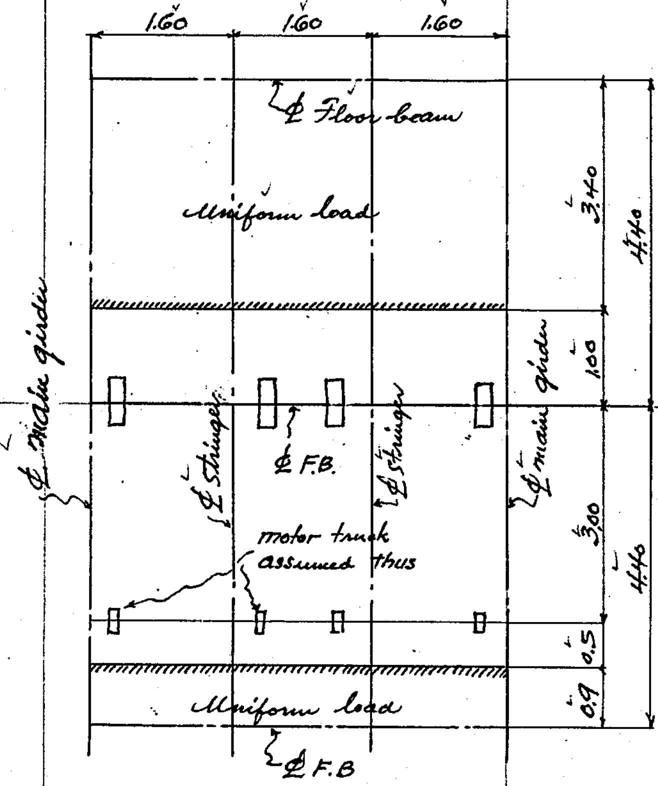
CALCULATIONS FOR

Design of 18 meter Girder Span

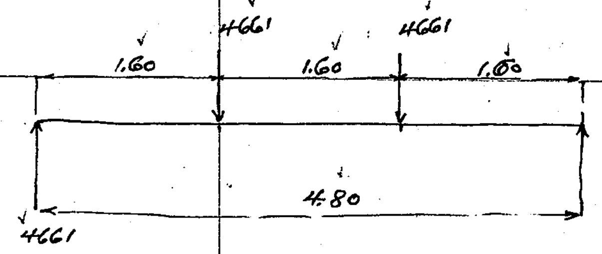


Concentration on Floor-beam
motor truck loading rear wheel 3200
front " $1065 \times \frac{1.4}{4.4} =$ 338
3538 kg

Uniform load $\frac{800 \times 3.4^2}{2 \times 4.4} = 1050$

$\frac{800 \times 0.9^2}{2 \times 4.4} = 73$

1123
4661 kg



moment = $4661 \times 1.60 = 7460$ kgm.
max end shear =
Stringer concentration 4661 kg

Summary for moments and shears.

	moments	end shears.
Dead Load.	5508	3500
Live Load	7460	4661
	12968 kgm	8161 kg

web assumed $77.0 \times .8 = 61.6$ cm² $\frac{1}{8}$ web $= 7.70$ cm²
Back to back of Ls 78.0 cm
effective depth say $78.0 - 4.3 = 73.7$ cm
flange stress $= \frac{12968}{.737} = 17600$ kg cm T

flange area required $= \frac{17600}{1200} = 14.65$
$\frac{1}{8}$ web $= \frac{-7.70}{6.95}$ net

use 2Ls $75 \times 75 \times 9 = 25.38 - 3.96 = 21.42$ cm² net
unit shear on web $= \frac{8161}{61.6} = 132$ kg/cm² ok

22ϕ rivets for conn. $= \frac{8161}{2640} = 3.1$ use 8 about.

Approximate weight of Intermediate floor beam

Flange	4Ls	75×75×9	@ 9.96	× 4.70	=	187
web.	1Pl	770×8	@ 48.36	× 4.80	=	232
End conn.	2Ls	150×90×9	@ 16.32	× 0.62	=	20
stiffeners	8Ls	75×65×8	@ 8.28	× 0.78	=	52
base of stringers	2Pl.	310×9	@ 21.90	× 0.30	=	13
						504

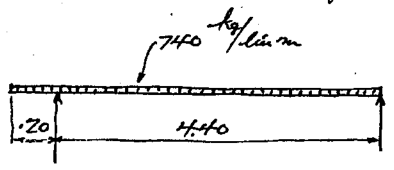
Rivet heads and variations say 3½ % about 16
520 kg

$520 \div 4.8 = 108$ kg/lin m.

CALCULATIONS FOR

Design of 18 meter Girder span

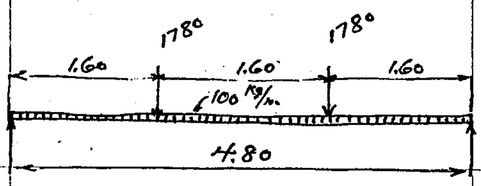
Design of End floor beam, Span length = 4.80 meters, Spacing 4.40 meters



Stringer Concentration on floor beam

$\frac{740 \times 4.60^2}{2 \times 4.40} = 1780$ kg

floor beam assumed 100 kg per lin m



Dead Load moment due to

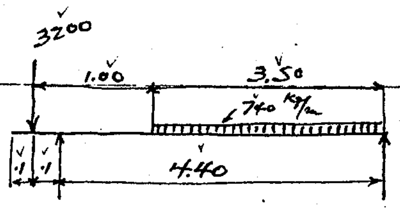
Stringer concentration	=	1780 × 1.60 =	2,850
weight of floor beam	1/8 × 100 × 4.8²	=	290
			3,140 kgm

End shear

from concentration		1780
weight of floor beam	$\frac{100}{2} \times 4.8 =$	240
		2020 kg

Live Load.

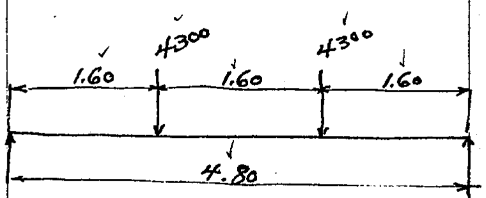
Rear wheel concentration on stringer = 3200 kg
front " " " " = 1065 "



Stringer concentration due to rear wheel. $\frac{3200 \times 4.5}{4.4} = 3270$

" " " unif load $\frac{740 \times 3.5^2}{2 \times 4.4} = 1030$

4300 kg



moment = 4300 × 1.60 = 6880 kgm.

End shear stringer concentration 4300 kg.

Summary for moments and shears.

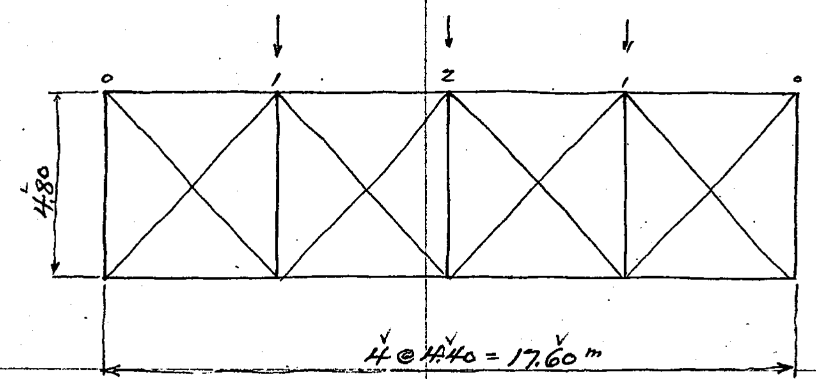
	moments	End shears.
Dead Load	3,140	2020
Live Load	6880	4300
	10,020 kgm	6,320 kg

Use same details as for intermediate floor beam.

CALCULATIONS FOR

Design of 18 meter girder span

Design of Lateral Bracings.



Diagonal length

$4.80^2 = 23.04$
$4.40^2 = 19.36$
$\overline{42.40}$

Diagonal length $= \sqrt{42.40} = 6.51$ meters.

Coefficient $= \frac{6.51}{4.80} = 1.357$

Wind load assumed 400 kg/lin meter of bridge. Panel load = 400 × 4.4 = 1760 kg
Shear at panel 0-1 1760 × 1.5 = 2640 kg
Stress in diagonal = 2640 × 1.357 = 3580 kg T.

Seismic stresses. Acceleration assumed 2500 mm/sec² or k = 0.250
Dead Load
Floor slab and pavement between main girders 4.8 @ 430 = 2065
Handrails, Copings, + floor outside of " " 2 @ 391 = 782
2847

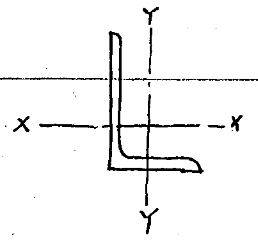
Stringers 2 @ 50 = 100
floor beam 520 ÷ 4.4 = 118
lateral bracing assumed = 55
main girder assumed 635
908
3755 kg per lin meter.

Panel load = 3755 × 4.40 = 16530 kg
Horizontal panel load = 16530 × 0.25 = ~~409~~ 4130 kg
Shear in panel 0-1 = 4130 × 1.5 = 6200 kg
Stress in diagonal = 6200 × 1.357 = 8410 kg T

Seismic stresses govern the section of diagonal member.
Sectional area required = $\frac{8410}{1200 \times 1.8}$ = 3.90 cm² net

19ϕ rivet no. required = $\frac{8410}{2126 \times 1.8}$ = 2.2 use 3-19ϕ rivets.

use 1L 125 × 75 × 10 = 19.00 − 2.2 = 16.80 cm² net ok



Radius of gyration about
X-X axis $r_x = 3.96$ cm $l/r_x = 651/3.96 = 165 < 200$ ok.
Y-Y $r_y = 2.07$ $l/r_y = 326/2.07 = 158$

Approximate weight of Lateral Bracings.

Diagonals	8Ls	125 × 75 × 10	@	14.91 × 6.10	=	728
Center conn.	4Pls	270 × 9	@	19.08 × 0.51	=	39
Conn. to girders	10Pls	345 × 9	@	24.37 × 0.60	=	146
						913
	Rivet heads and variation say 1½ % add					17
						930 kg

930 ÷ 17.6 = 53 kg per lin meter.

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MADE BY K. Inaba DATE 5-8-1 FILE NO B
CHECKED BY M. K DATE 5-8-6 PAGE NO 9

CALCULATIONS FOR

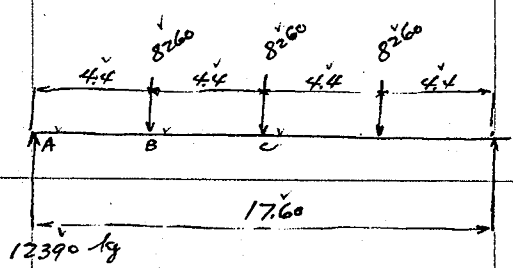
Design of 18 meter Girder span

Design of main girder. Span length 17.60 meters, c to c of bearings, 18.00 meters out to out.

Dead Load
Floor and pavement between main girders 4.80 @ 430 = 2065
Handrails, copings, and floor beyond " " 2 @ 391 = 782
2847

Stringers 2 @ 50 = 100
Floor beams 520 ÷ 4.4 = 118
lateral bracing 55
main girders assumed 635
908
3755 kg per lin meter

For one girder 3755 ÷ 2 = 1878 kg per lin meter.
Let us assume the load concentrated at panel point
Intermediate panel concentration = 1878 × 4.40 = 8260 kg for one girder
End " " = 1878 × 2.20 = 4130 kg " "



Reaction = 8260 × 1.5 = 12390 kg

moment at B. 12390 × 4.40 = 54500 kgm

moment at C 12390 × 8.80 = 109000
8260 × 4.40 = − 36350
72650 kgm

Max. End shear = 12390 kg
Dead Load on shoe or bearing = 1878 × 9.0 = 16900
add for shoe say 100
17000 kg for one shoe.

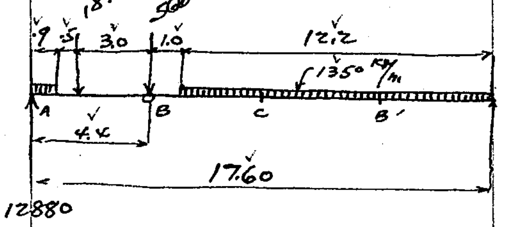
Live Load: Uniform live load = 500 kg per sq. meter.
Impact for motor truck loading = $\frac{20}{60+17.6}$ = 25.8 %

motor truck rear wheel concentration = 2250
25.8 % impact 580
2830 kg

Front wheel with impact = 2830 ÷ 3 = 945

Uniform live load = 500 × 2.7 = 1350 kg per lin meter.
motor truck concentration
Rear wheels 2 @ 2830 = 5660 kg
Front " 2 @ 945 = 1890 "

moment at B.
Reaction 5660 × 3/4 = 4245
1890 × 16.2/17.6 = 1740
$\frac{1350 \times 12.2^2}{2 \times 17.6}$ = 5710
1350 × .9 × $\frac{17.15}{17.6}$ = 1185
12880 kg



moment 12880 × 4.40 = 56700
1890 × 3.00 = − 5670
1350 × .9 × 3.95 = − 4800
46230 kgm

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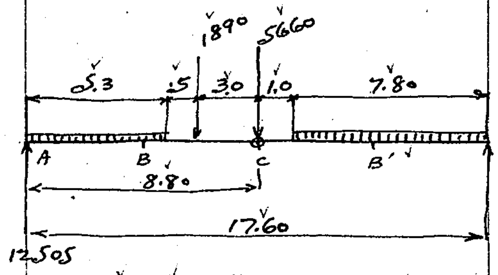
MADE BY K. Inaba DATE 5-8-1 FILE NO B
CHECKED BY M.K DATE 5-8-6 PAGE NO 10

CALCULATIONS FOR

Design of 18 meter Girder span



$1350 \times 5.3 = 7150$, $1350 \times 7.8 = 10530$ kg

moment at C
Reaction

$5660 \div 2$	=	2830
$1890 \times 11.8/17.60$	=	1267
$7150 \times 14.95/17.60$	=	6073
$10530 \times 3.90/17.60$	=	2335
		12505 kg

moment.

12505×8.80	=	110,000
1890×3.00	= −	5,670
7150×6.15	= −	43,950
		60,380 kgm

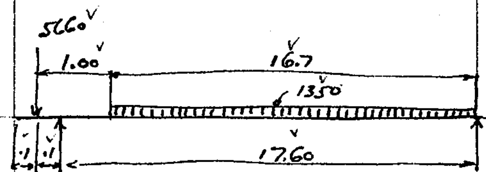
max. End shear, Say rear wheel at ℄ bearing

Uniform load $\frac{1350 \times 16.6^2}{2 \times 17.6} = 10,570$

motor truck 5,660

16,230 kg

max. Load on shoe or bearing



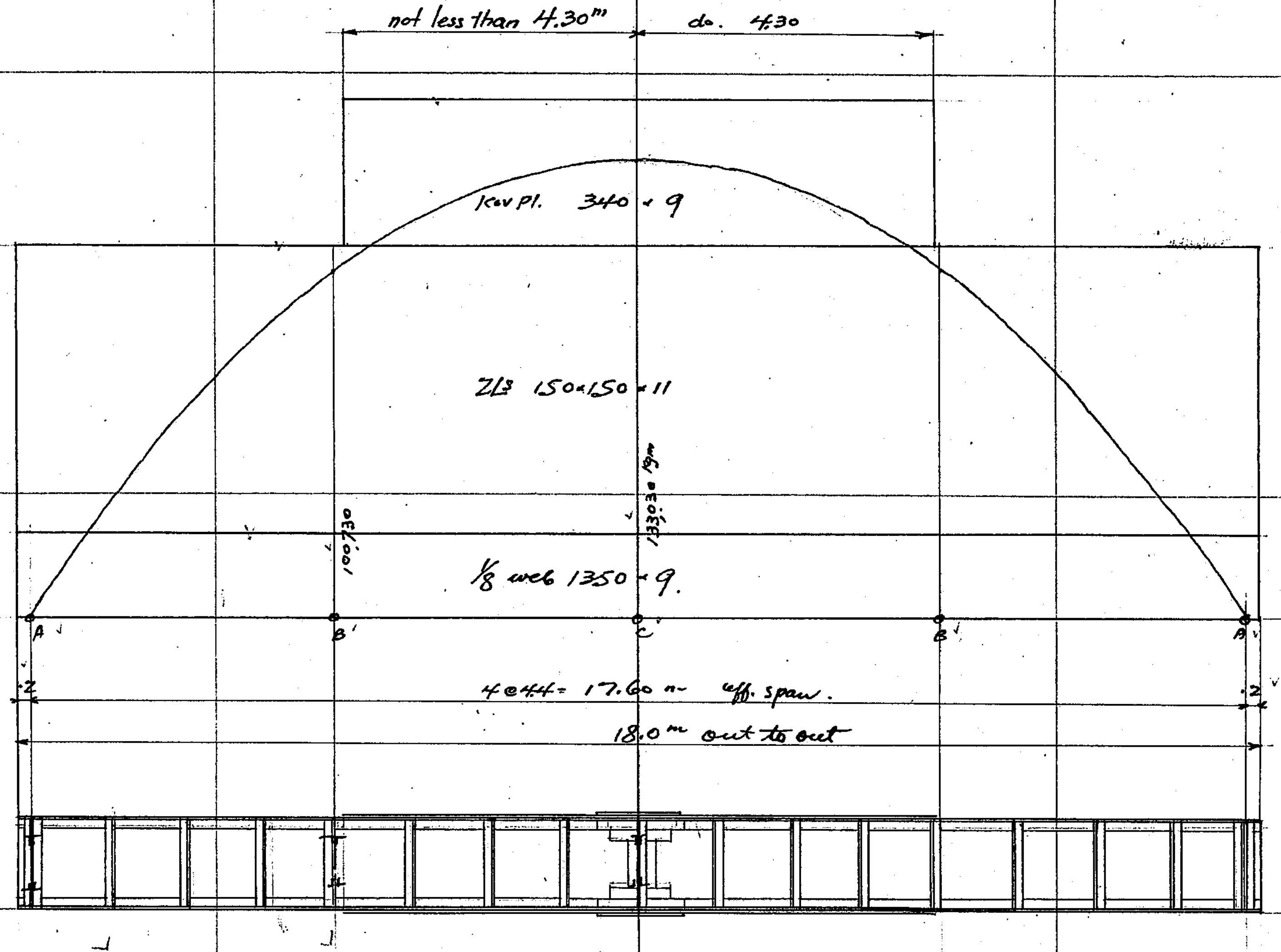
Reaction $\frac{1350 \times 16.7^2}{2 \times 17.6} = 10,700$

5,660

16,360 kg on one shoe.

Summary for moments and shears.

	moment B	moment C	End shear	Load on shoe
Dead Load	54,500	72,650	12,390	17,000
Live Load	46,230	60,380	16,230	16,360
	100,730 kgm	133,030 kgm	28,620 kg	33,360 kg



Moment Diagram.
Scale of space 1 : 100
Scale of moment $\frac{1}{20}$ in = 100,000 kgm.

CALCULATIONS FOR

Design of 18 meter Girder span

Section of Main Girder

web plate 1350 × 9 web area = 135 × .9 = 121.5 cm², 1/8 web area = 15.20 cm²
total depth 136 cm back to back of Ls Effective depth 136.0 − 4 = 132 cm about.

flange stress = $\frac{133030}{1.32}$ = 100,800 kg C or T.

flange area required = $\frac{100800}{1200}$ = 84.10 cm² net
1/8 web −15.20
68.90

use 2 Ls 150 × 150 × 11 = 63.58 − 11.00 = 52.58
1 cov. Pl. 340 × 9 = 31.60 − 4.50 = 27.10
95.18 gr 79.68 cm² net.
15.2
110.38

Unit tension = $\frac{100800}{79.68+15.2}$ = 1063 kg/cm² ok

Unit compression = $\frac{100800}{95.18+15.2}$ = 914 kg/cm² ok

Unit shear on web = $\frac{28620}{121.5}$ = 236 " ok

allowable unit compression on flange = $1200\left(1 - 0.012 \times \frac{440}{34}\right)$ = 1014 kg/cm² C

.155

13.

Approximate weight of main girder.

CALCULATIONS FOR

Design of 20 meter Girder span

Design of I Beam stringer, span length 4.65 meters, spacing 1.60 meters.

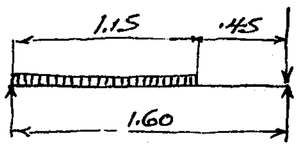
Dead Load.

floor slab and pavement $430 \times 1.60 =$ 688
stringer assumed 50
738 call this 740 kg per lin meter.

Dead Load moment $= \frac{1}{8} \times 740 \times 4.65^2 =$ 2000 kgm
Dead Load shear $= \frac{1}{2} \times 740 \times 4.65 =$ 1720 kg

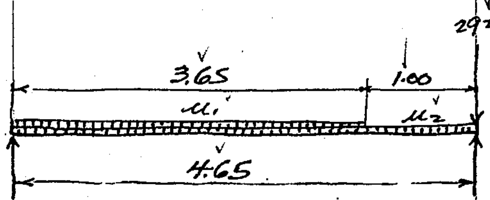
Live Load.

motor truck rear wheel concentration with impact. = 2925 kg.
Uniform live load = 500 kg per sq. meter.



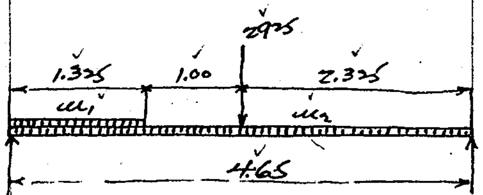
load on stringer $= \frac{500 \times 1.15^2}{2 \times 1.60} =$ 207 kg ------- M_2

full uniform load $= 500 \times 1.6 =$ 800
less M_2 = -207
593 kg ------- M_1



End shear
Uniform load M_1 $\frac{593 \times 3.65^2}{2 \times 4.65} =$ 840

Uniform load M_2 $207 \times 4.65 \div 2 =$ 490
motor truck loading 2925
4245 kg



moment.
Due to motor truck $\frac{2925}{2} \times 2.325 =$ 3400
Uniform load M_1 $\frac{593 \times 1.325^2}{2 \times 4.65} \times 2.325 =$ 260
Uniform load M_2 $\frac{1}{8} \times 207 \times 4.65^2 =$ 560
4220 kgm

Summary for moments and shears.

	moments	shears.
Dead Load	2000	1720
Live Load	4220	4245
	6220 kgm	5965 kg

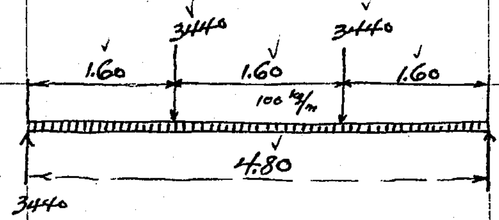
Section modulus required $= \frac{622000}{1100} =$ 565.0 cm³

Use 300×150 I @ 48.34 kg $S_m = 633.2$ cm³ ok

Unit stress $= \frac{622000}{633.2} =$ 983 kg/cm² C or T ok.
Unit shear $= \frac{5965}{30 \times .8} =$ 249 kg/cm² ok.

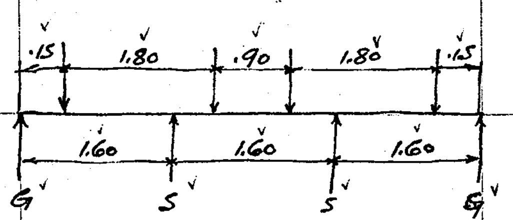
Design of Intermediate Floor Beam. Span length 4.80 meters, Spacing 4.65 meters.

Dead Load. Concentration on stringer $= 740 \times 4.65 =$ 3440 kg.
floor beam assumed 100 kg per lin meter of span
Dead Load moment
due to stringer concentration $3440 \times 1.60 =$ 5500
due to weight of floor beam $\frac{1}{8} \times 100 \times 4.8^2 =$ 288
5788 kgm.



End shear from concentration 3440
from weight of beam $\frac{100}{2} \times 4.8 =$ 240
3680 kg

Live Load motor truck loading, rear-wheel concentration with impact = 2925 kg
front wheel " " " = 975 "



Load on S $\frac{.15}{1.6} =$ 0.094
1.000
$1.094 \times 2925 =$ 3200 kg --- rear wheel.
$1.094 \times 975 =$ 1065 kg --- front "

Uniform live load $= 500 \times 1.60 =$ 800 kg on stringer.

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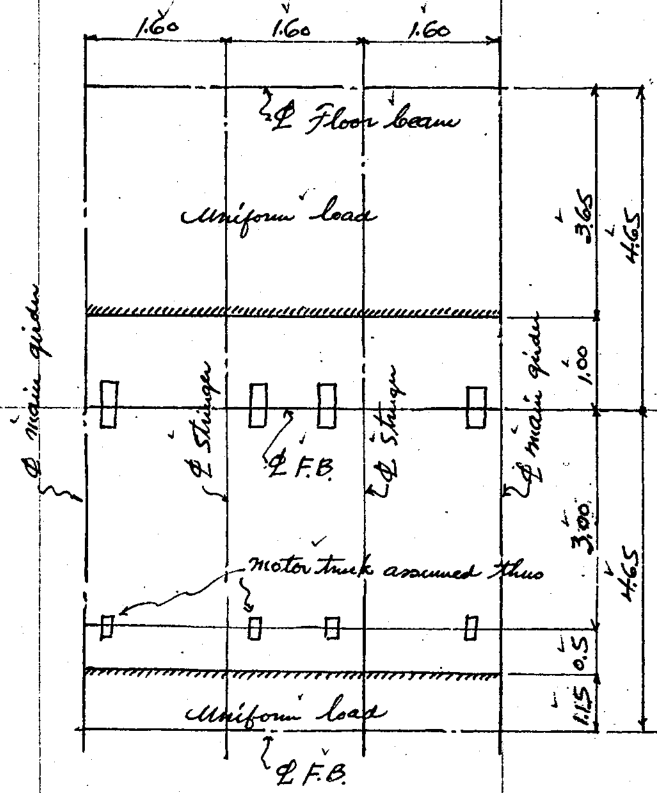
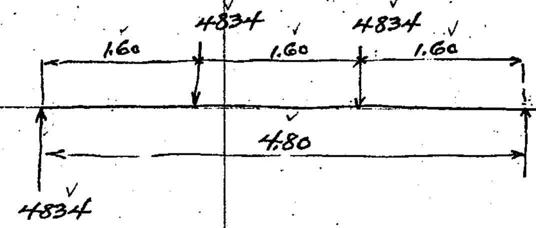
CALCULATIONS FOR

Design of 20 meter Girder span



Concentration on Floor beam
motor truck loading, rear wheel. 3200
front wheel $1065 \times \frac{1.65}{4.65} =$ 378
3578 kg

Uniform load. $\frac{800 \times 3.65^2}{2 \times 4.65} =$ 1146
" $\frac{800 \times 1.15^2}{2 \times 4.65} =$ 110
1256
4834 kg



moment = $4834 \times 1.60 =$ 7730 kgm

End shear 4834 kg

Summary for moments and shears.

	moments	Shears.
Dead Load	5788	3680
Live Load	7730	4834
	13518 kgm	8514 kg

Web assumed $77 \times 0.8 = 61.6$ cm² $\frac{1}{8}$ web $= 7.70$ cm²
Back to back of Ls 78.0 cm
effective depth say $78.0 - 4.3 =$ 73.7 cm
flange stress $= \frac{13510}{.737} =$ 18300 kg cm T

Section required $= \frac{18300}{1200} =$ 15.25 cm² net
$\frac{1}{8}$ web area $= -7.70$
7.55 cm² net.

use 2Ls $75 \times 75 \times 9 = 25.38 - 3.96 = 21.42$ cm² net

Unit shear on web $= \frac{8514}{61.6} =$ 138 kg/cm² ok

22φ rivet no reqd conn. $= \frac{8514}{2640} = 3.2$ use 8 about

Approximately weight of Intermediate Floor Beam

Flange	4Ls	75×75×9	@	9.96 ×	4.70	=	187
web	1Pl.	770 × 8	@	48.36 ×	4.80	=	232
End conn	2Ls	150×90×9	@	16.32 ×	.62	=	20
Stiffeners	8Ls	75×65×8	@	8.28 ×	.78	=	52
base of stiffeners	2Pls	310×9	@	21.90 ×	.31	=	13

504
Rivet heads + variation say $3\frac{1}{2}$% about 16
520 kg

$520 \div 4.8 =$ 108 kg/lin meter ok.

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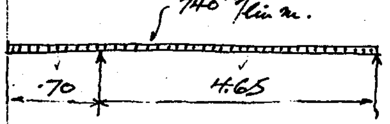
CALCULATIONS FOR

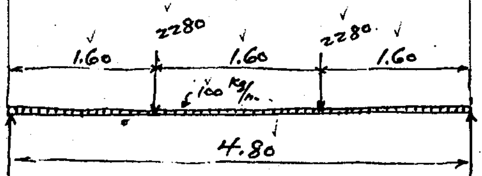
Design of 20 meter Girder Span

Design of End Floor Beam. Span length 4.80 meters, Spacing 4.65 meters.



Stringer concentration on floor beam.

$\frac{740 \times 5.35^2}{2 \times 4.65} = 2280$ kg



Floor beam assumed 100 kg per lin m.

Dead Load moment due to

Stringer concentration $= 2280 \times 1.60 = 3645$

weight of floor-beam $= 1/8 \times 100 \times 4.8^2 = \underline{288}$

3933 kgm

End Shear

from concentration 2280

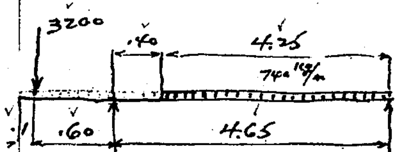
" weight of floor-beam $\frac{100}{2} \times 4.8 = \underline{240}$

2520 kg

Live Load

Rear wheel concentration on stringer = 3200 kg

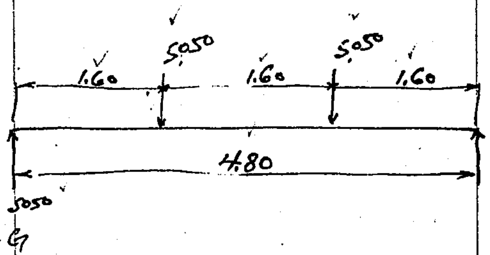
front " " " = 1065 kg



Stringer concentration due to rear wheel. $\frac{3200 \times 5.25}{4.65} = 3610$

" " " unif. load $\frac{740 \times 4.25^2}{2 \times 4.65} = \underline{1440}$

5050 kg



moment $= 5050 \times 1.60 = 8080$ kgm

End shear = 5050 kg

Summary for moments and shears

	moments	shears
Dead Load	3933	2520
Live Load	8080	5050
	12013 kgm	7570 kg

Use same details as for Intermediate Floor beam.

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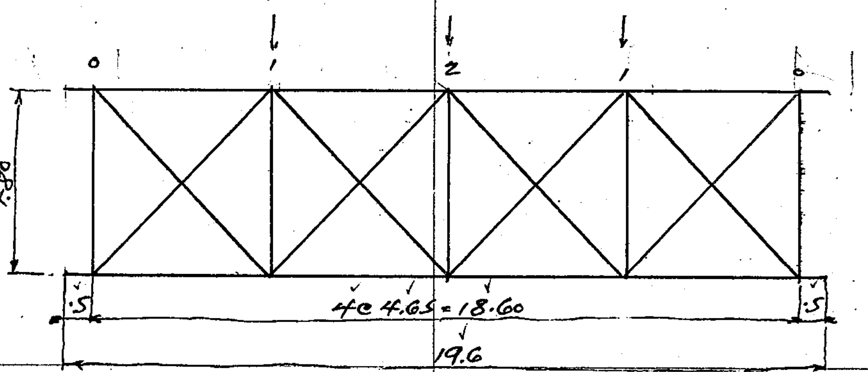
CALCULATIONS FOR

Design of 20 meter Girder Span

Design of Lateral Bracings.



Diagonal Length

$4.80^2 = 23.04$
$4.65^2 = \underline{21.62}$
44.66

Diagonal length $= \sqrt{44.66} = 6.68$ meters

Coefficient $= \frac{6.68}{4.80} = 1.392$

Wind load assumed 400 kg/lin. m. of bridge. Panel load = 400 × 4.65 = 1860 kg
End shear at 0–1. 1860 × 1.5 = 2790 kg
Stress in diagonal = 2790 × 1.392 = 3,880 kg T

Seismic stresses. acceleration 2500 mm/sec² or k = 0.250
Dead Load

Floor slab and pavement between main girders	4.8 @ 430 =	2065	
Handrails, copings, + floor outside of " "	2 @ 391 =	782	
			2847
Stringers	2 @ 50 =	100	
Floor beam	500 ÷ 4.65 =	108	
lateral bracings assumed	=	50	
main girders assumed	=	650	
			908
			3755 kg per lin m.

Panel load = 3755 × 4.65 = 17470 kg
Horizontal panel load = 17470 × .25 = 4370 kg
Shear in panel 0–1 = 4370 × 1.5 = 6560 kg
Stress in diagonal member = 6560 × 1.392 = 9130 kg T

Seismic stress governs the section of diagonal member.
Sectional area required $= \frac{9130}{1200 \times 1.8} = 4.23$ cm² net.

19φ rivet no. required $= \frac{9130}{2120 \times 1.8} = 2.40$ use 3-19φ rivets

use 1L 125×75×10 = 19.00 − 2.2 = 16.8 cm² net ok.

Radius of gyrations about

X–X axis $r_x = 3.96$ cm $\qquad l/r_x = 668/3.96 = 169 < 200$ ok.
Y–Y axis $r_y = 2.07$ cm $\qquad l/r_y = 334/2.07 = 161$

Approximate weight of Lateral Bracings.

Diagonals	8Ls 125×75×10	@	14.91 × 6.30 =	751	
Center conn	4 pls 270 × 9	@	19.08 × 0.51 =	39	
Conn. to girders	10 pls 345 × 9	@	24.37 × 0.60 =	146	
				936	
Rivet heads and variation say 1½ %			=	14	
				950 kg	

950 ÷ 18.6 = 51 kg per lin m

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CALCULATIONS FOR

Design of 20 meter Girder span

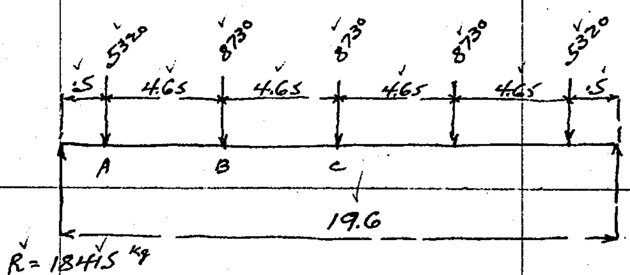
Design of Main girders. Span length 19.60 meters c to c Bearings 20.0 m out to out.

Dead Load: Floor and pavement between main girders 4.8 @ 430 = 2065
Handrails, copings, and floor beyond main girders 2 @ 391 = 782
2847

Stringers 2 @ 50 = 100
Floor beams 500 ÷ 4.65 = 108
lateral bracings = 51
main girders assumed = 650
909

3756 kg per lin m.

For one girder 3756 ÷ 2 = 1878 kg per lin meter
Let us assume the load concentrated at panel point.
Intermediate panel concentration = 1878 × 4.65 = 8730 kg. for one girder.
End " " = 1878 × 2.83 = 5320 "



moment at A. 18415 × 0.5 = 9208 kgm

moment at B. 18415 × 5.15 = 94800
5320 × 4.65 = - 24750
70050 kgm

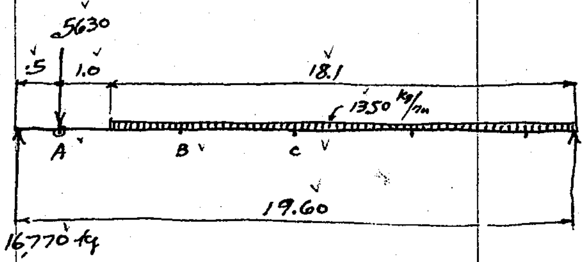
moment at C 18415 × 9.80 = 180400
5320 × 9.30 = - 49450
8730 × 4.65 = - 40600
90350 kgm.

max. end shear = 18415 kg
Dead Load on shoe or bearing = 1878 × 20.0 ÷ 2 = 18780
add for shoe say 120
18900 kg on one shoe.

Live Load Uniform live load = 500 kg per sq. meter.
Impact for motor truck loading = $\frac{20}{60+19.6}$ = 25.1 %

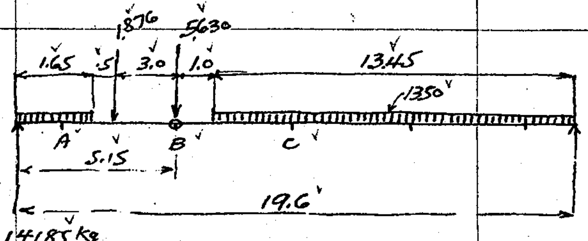
motor truck rear wheel concentration = 2250
25.1 % impact = 565
2815 kg

Front wheel concentration with impact = 2815 ÷ 3 = 938 kg
Uniform live load = 500 × 2.70 = 1350 kg per lin meter.
motor truck concentration
Rear wheels 2 @ 2815 = 5630 kg
front " 2 @ 938 = 1876 kg



moment at A.
Reaction $5630 \times \frac{19.1}{19.6}$ = 5485
$\frac{1350 \times 18.1^2}{2 \times 19.6}$ = 11285
16770 kg

moment = 16770 × 0.50 = 8385 kgm



moment at B.
Reaction $5630 \times \frac{14.45}{19.6}$ = 4150
$1876 \times \frac{17.45}{19.6}$ = 1670
$\frac{1350 \times 1.65 \times 18.775}{19.6}$ = 2135
$\frac{1350 \times 13.45^2}{2 \times 19.6}$ = 6230
14185 kg

moment = 14185 × 5.15 = 73000
1876 × 3.00 = - 5630
1350 × 1.65 × 4.325 = - 9630
57740 kgm

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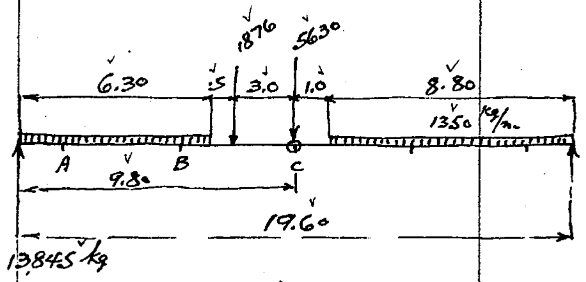
CALCULATIONS FOR

Design of 20 meter Girder Span



1350 × 6.3 = 8500, 1350 × 8.8 = 11880

moment at C

Reaction

5630 ÷ 2		=	2815
1876 ×	$\frac{12.8}{19.6}$	=	1225
8500 ×	$\frac{16.45}{19.6}$	=	7135
11880 ×	$\frac{4.4}{19.6}$	=	2670
			13845 kg

moment

13845 ×	9.80	=	135700
1876 ×	3.00	= −	5630
8500 ×	6.65	= −	56500
			73570 kgm

Max End shear · say Rear wheel at ℄ bearing

Uniform load $\frac{1350 \times 18.6^2}{2 \times 19.6} = 11920$

motor truck 5630

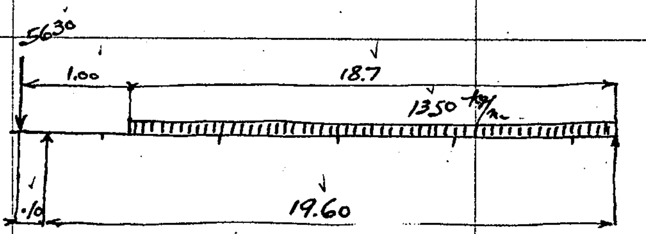
17550 kg

max. Load on shoe or bearing Reaction

$\frac{1350 \times 18.7^2}{2 \times 19.6} = 12050$

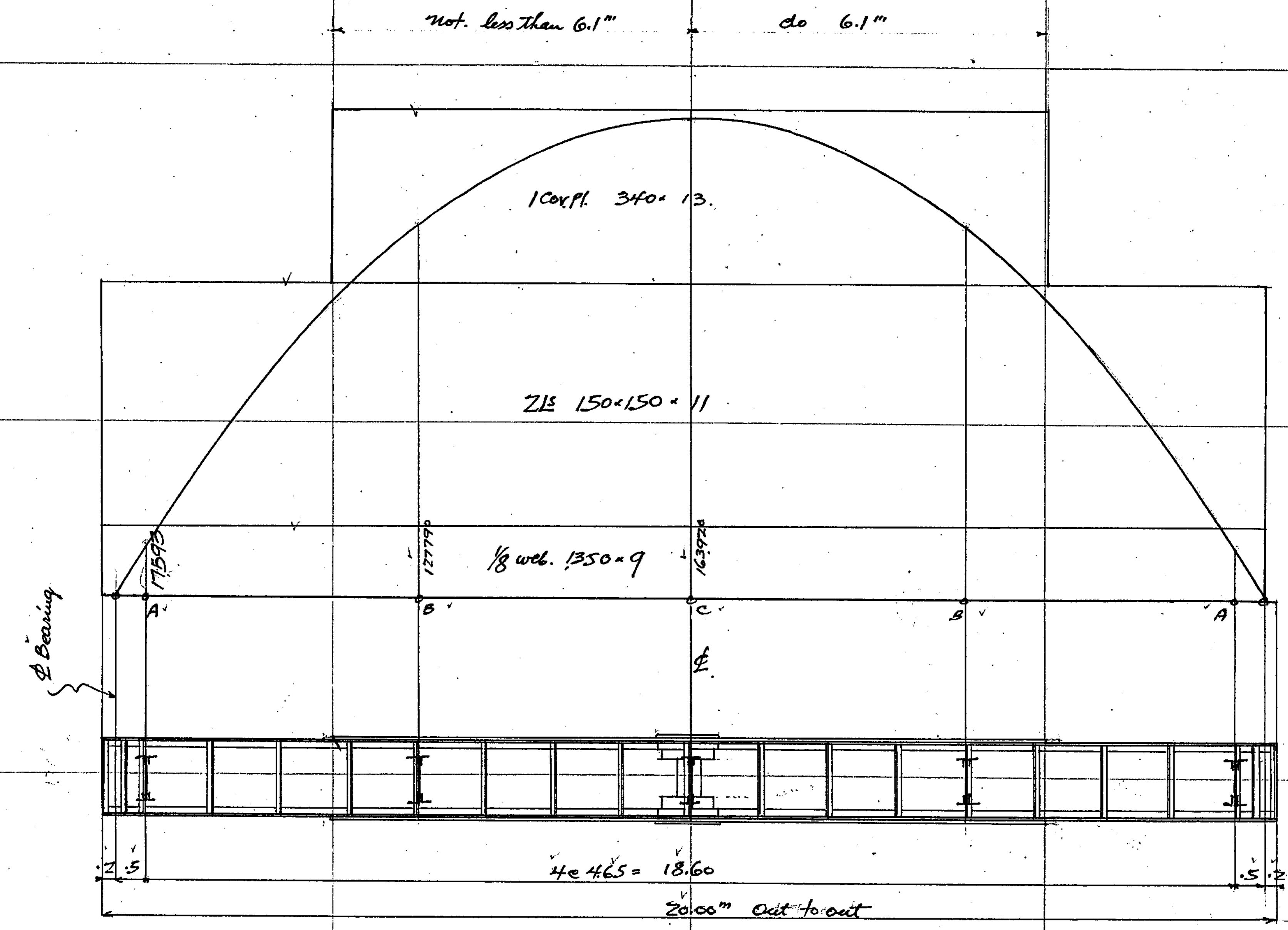
5630

17680 kg on one shoe.



Summary for moments and shears.

	moment A	moment B	moment C	End shear	Load on shoe
Dead Load	9208	70050	90350	18415	18900
Live Load	8385	57740	73570	17550	17680
	17593 kgm.	127790 kgm.	163920 kgm.	35965 kg	36580 kg



Moment Diagram. Scale of space 1/100

∴ moment = $\frac{1}{20}$ m = 100,000 kgm.

Design of 20 meter Girder span

Section of Main Girder

Depth of web pl. 135 cm, Web area $135 \cdot 9 = 121.5$ cm² $\frac{1}{8}$ web area = 15.2 cm²

136 cm back to back of flange Ls Effective depth $136.0 - 4 = 132.0$ about.

flange stress $= \frac{163920}{1.32} = 124000$ kg C or T

Section required $= \frac{124000}{1200} = 103.4$ cm² net

$\frac{-15.2}{88.2}$ cm² net

Use 2 Ls $150 \cdot 150 \cdot 11 = 63.58 - 11.00 = 52.58$

1 Pl. $340 \cdot 13 = \frac{44.20}{107.78 \text{ cm}^2 \text{ gr}} - 6.50 = \frac{37.70}{90.28 \text{ cm}^2 \text{ net}}$ ok

Unit tension $\frac{124000}{90.28+15.2} = \frac{124000}{105.48} = 1178$ kg/cm² T ok

Unit compression $\frac{124000}{107.78+15.2} = \frac{124000}{122.98} = 1005$ kg/cm² C ok

Unit shear on web $= \frac{35965}{121.5} = 296$ " ok

allowable unit comp. on flange $= 1200\left(1 - 0.012 \cdot \frac{465}{34}\right) = 1005$ (.164)

1800
200,000

$9 \times \frac{7.5}{10} \cdot 7.$

60
220,000

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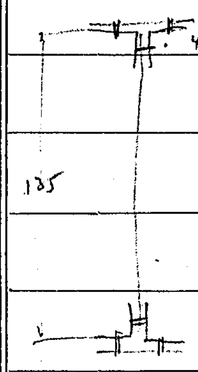
増田淳事務所
電話丸ノ内七七七番



2Ls $52.58 \times 67.5^2 = 241,000 \times 2 = 482,000$

$660 \times 2 = 1,300 \times 2 = 2,600$

1Pl. $37.70 \times 68^2 = 174,000 \times 2 = 349,000$

web 135 $\frac{135^3 \times 0.7}{12} = 143,500$

$976,100\ cm^4$

$$S = \frac{M}{I}y = \frac{163,920 \times 100 \times 68}{976,000} = 1140$$

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CALCULATIONS FOR
Material list,
20 Meter Plate Girder Span for Mie Ken.

No.	Discription	Section in Mm.	Length in Mm.	Wt. of One Meter	Wt. of Main Section in Kgs	Wt. of Details in Kgs	Total Wt.	Remarks
			GIRDER $G1^{R}_{L}$ & $G1A^{R}_{L}$		4 Req'd.			
4	Flg Ls	150 × 150 × 11	10,000	24.95	998.0			
1	Web Pl.	1,350 × 9	10,000	95.378	953.8			
2	Cov. Pls.	340 × 13	6,310	34.697	437.9			
4	Ls	125 × 90 × 10	1,338	16.09		86.1		
2	Fills	180 × 11	1,055	15.543		32.8		
16	Ls	125 × 75 × 10	1,360	14.91		324.4		
1	Conn. Ls	150 × 100 × 9	155	17.02		2.6		
2	″	″	305	″		10.4	36.1	
1	″	″	230	″		3.9		
1	Sole Pl.	400 × 25	460		2,389.7 +	460.2	= 2,849.9	2886.0
						36.1	× 4	
						496.3	~~11,399.6~~	
							11544.0	
			SPLICE AT CENTER		2 Req'd.			
2	Pls.	340 × 13	1,050	34.697		72.9		
4	Ls	150 × 150 × 15	930	33.55		124.8		
4	Pls.	230 × 13	630	23.472		59.1		
2	″	330 × 13	586	33.677		39.5		
2	Fills	75 × 13	1,055	7.654		16.1		
2	Ls	125 × 75 × 10	1,308	14.91		39.0		
1	Conn. L	100 × 75 × 10	230	12.95		3.0		
1	″	″	305	″		3.9		
							358.3	
							× 2	
							716.6	
		Summary of Girder		~~12,116.2~~ 12260.6				
			FLOOR BEAM FB1		4 Req'd.			
2	Flg Ls	75 × 75 × 9	4,780	9.96		95.2		
2	″	″	4,530	″		90.2		
1	Web Pl.	770 × 8	4,780	48.356		231.1		
10	Ls	75 × 65 × 8	780	8.28		64.6		
2	″	125 × 90 × 10	620	16.09		20.0		
2	Pls.	290 × 9	310	20.489		12.7		
							513.8	
							× 4	
							2055.2	
			FLOOR BEAM FB2		1 Req'd.			
2	Flg Ls	75 × 75 × 9	4,730	9.96		94.2		
2	″	″	4480	″		89.2		
1	Web Pl.	770 × 8	4,730	48.356		228.7		
10	Ls	75 × 65 × 8	780	8.28		64.6		
2	″	125 × 75 × 10	515	14.91		15.4		
2	Pls.	290 × 9	310	20.489		12.7		
							504.8	
		Summary of Floor beams		2560.0				
			STRINGERS					
4	Is	300 × 150 @ 48.34 Kgs.	5,345			1,033.5		
4	″	″	4,640			897.2		
12	Pls.	170 × 9	230	12.011		33.2		
							1,963.9	

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CALCULATIONS FOR
Material list,
20 Meter Plate girder span for Mie Ken.

LATERAL BRACINGS

					1 Reqd.		
4	L	125 × 75 × 10	6,250	14.91	372.8		
8	"	"	3,075	"	366.8		
4	pls.	280 × 9	520	19.782		41.1	
4	"	300 × 9	550	21.195		46.6	
4	"	"	625	"		53.0	
2	"	305 × 9	625	21.548		26.9	
					739.6 +	167.6	= 907.2

SHOES & ANCHOR BOLTS

4	Cast iron shoes			@ 140.0	560.0	
4	pls.	70 × 30	160	16.486	10.6	
16	Anchor bolts.	25φ	700	@ 3.13	50.1	
16	Washers	100 × 9	100	7.065	11.3	
8	Bolts	22φ	115	@ 0.650	5.2	
						637.2

RIVETS HEADS.

5,060	22φ	Shop rivet heads.	@ 0.0964	487.8	
980	22φ	Field " "	@ "	94.5	
1,450	19	Shop " "	@ 0.06464	93.7	
640	19	Field " "	@ "	41.4	
					717.4

Grand Summary of 20 Meter Plate Girder

	Kgs.
Girder	~~12,116.2~~ 12260.6
Floor beams	2,560.0
Stringers	1,963.9
Lateral bracings	907.2
Shoe & Anchor bolts	637.2
	~~18,184.5~~ 18328.9
River Heads. —	717.4
	~~18,901.9~~ or 18.902 Kgtons (伸縮金物ヲ含マズ)
	19 046.3

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CALCULATIONS FOR

Material list,
18 Meter Plate Girder Span for Mie Ken.

No.	Description.	Section in Mm.	Length in Mm.	Wt. of One Meter	Wt. of Main section in Kgs.	Wt. of Details in Kgs	Total Wt.	Remarks.
			GIRDER G2^R_L & G2A^R_L			4 Reqd.		
4	Flg ∟s	150 × 150 × 11	9.000	24.95	898.2			
1	Web Pl.	1350 × 9	9.000	95.378	858.4			
2	Cov. Pls	340 × 9	4.400	24.021	211.4			
4	End stif. ∟s	125 × 90 × 10	1.338	16.09		86.1		
2	Fills	190 × 11	1.055	16.407		34.6		
14	Stif. ∟s	125 × 75 × 10	1.360	14.91		283.9		
1	Sole Pl.	400 × 25	460	78.50		36.1		
2	Conn. ∟s	150 × 100 × 9	230	17.02		7.8		
1	"	"	305	"		5.2		
					1.968.0 +	453.7	= 2421.7	
							× 4	
							9.686.8	
			SPLICE		2 Reqd.			
4	∟s	150 × 150. 15	930	33.55		124.8		
4	Pls.	230 × 13	630	23.472		59.1		
2	"	330 × 13	586	33.677		39.5		
2	Fills.	75 × 13	1.055	7.654		16.1		
2	Pls.	340 × 9	810	24.021		38.9		
2	∟s	125 × 75 × 10	1.308	14.91		39.0		
1	Conn. ∟s	100 × 75 × 10	230	12.95		3.0		
1	"	"	305	"		3.9		
							324.3	
							× 2	
							648.6	
		Summary of Girder			10,335.4			
			FLOOR BEAM FB1		2 Reqd.			
2	Flg. ∟s	75 × 75 × 9	4.780	9.96		95.2		
2	"	"	4.530	"		90.2		
1	Web Pl.	770 × 8	4.780	48.356		231.1		
10	∟s	75 × 65 × 8	780	8.28		64.6		
2	"	125 × 90 × 10	620	16.09		20.0		
2	Pls.	290 × 9	310	20.489		12.7		
							513.8	
							× 2	
							1.027.6	
			FLOOR BEAM FB2		1 Reqd.			
2	Flg ∟s	75 × 75 × 9	4.730	9.96		94.2		
2	"	"	4.480	"		89.2		
1	Web Pl.	770 × 8	4.730	48.356		228.7		
10	∟s	75 × 65 × 8	780	8.28		64.6		
2	"	125 × 75 × 10	515	14.91		15.4		
2	Pls	290 × 9	310	20.489		12.7		
							504.8	
			FLOOR BEAM FB3		2 Reqd.			
4	Flg. ∟s	75 × 75 × 9	4.510	9.96		179.7		
1	Web Pl.	770 × 8	4.770	48.356		226.7		
10	∟s	75 × 65 × 8	780	8.28		64.6		
2	Pls.	290 × 9	310	20.489		12.7		
2	Fills	120 × 8	245	7.536		3.7		

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CALCULATIONS FOR
Material list,
18 Meter Plate Girder Span for Mie Ken.

2	Fills	120 × 8	310	7.536		4.7	
2	〃	〃	1.338	〃		20.2	
							512.3
							× 2
							1,024.6
		Summary of Floor beams			2,557.0		
			STRINGER		1 Req'd		
4	Is	300×150 @48.34 Kgs.	4.595		888.5		
4	〃	〃	4.390		848.9		
12	Pls.	170 × 9	230	12.011		33.2	
					1,737.4 +	33.2	= 1,770.6
			LATERAL BRACING		1 Req'd.		
4	Ls	125×75×10	6.110	14.91	364.4		
8	〃	〃	3.005	〃	358.4		
4	Pls.	280 × 9	520	19.782		41.1	
4	〃	310 × 9	410	21.902		35.9	
4	〃	300 × 9	625	21.195		53.0	
2	〃	305 × 9	625	21.548		26.9	
					722.8 +	156.9	= 879.7
			SHOES & ANCHOR BOLTS				
4	Cast iron shoes			@ 140.0		560.0	
4	Pls.	70 × 30	160	16.486		10.6	
16	Anchor bolts	25φ	700	@ 3.13		50.1	
16	Washers	100 × 9	100	7.065		11.3	
8	Bolts	22φ	115	@ 0.650		5.2	
							637.2
			RIVET HEADS				
3,550	22φ shop rivet heads			@ 0.0964		342.2	
1,070	22φ Field 〃 〃			@ 〃		103.1	
1,450	19φ Shop 〃 〃			@ 0.06464		93.7	
600	19φ Field 〃 〃			@ 〃		38.8	
							577.8

Grand summary of 18 Meter Plate Girder

	Kgs.
Girder	10,335.4
Floor beams	2,557.0
Stringers	1,770.6
Lateral bracings	879.7
Shoe & Anchor bolts	637.2
	16,179.9
Rivet heads	577.8
	16,757.7 or 16.758 Kgtons (伸縮金物ヲ含マズ)

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CALCULATIONS FOR

三重縣揖斐長良川橋假材橋設計

Estimate of Cost for Piers.
Piers on land and in shallow water (A)

米松	8	.30 × .25 × 1.10 =	.660	支承台
〃	6	.30 × .30 × 2.10 =	1.135	支承桁
〃	2	.30 × .25 × 6.20 =	.930	梁木
〃	4	.20 × .15 × 1.80 =	.216	布木
〃	4	.20 × .06 × 1.30 =	.062	〃
〃	4	.20 × .06 × 2.50 =	.120	筋違
〃	4	〃 〃 〃 =	.120	〃
〃	4	.20 × .10 × 3.0 =	.240	〃
〃	1	.20 × .10 × 2.0 =	.040	繋木
〃	2	.25 × .20 × 7.20 =	.720	梁木脚
〃	4	.20 × .15 × 1.80 =	.216	布木
〃	8	.20 × .06 × 2.00 =	.192	挟楔
			4.651 m³	

杉丸太柱	12	末口18cm × 2.40	
松丸太杭	12	〃 × 5.50	
平鈑	16	75×9 × 0.70	60
鋲	32	6" 鋲	15
ボールト		250 kg	250
			325 kg

Cost of one pier

米松材	4.651 m³	@ 50.0 =	232.5 円	仕拵組立共
杉丸太柱	12 本	@ 5.0 =	60.0	〃 〃
松杭	12 〃	@ 10.0 =	120.0	〃 〃 打込共
金物	0.325 kg ton	@ 200.0 =	65.0	〃 取付共
根掘	6.0 m³	@ 1.0 =	6.0	
			483.5	
取外片付			56.5	
			540.0	
塵芥除 仕拵建込材料搬出 杭ニ支建布及筋違付其 2@90 =			180.0	
			720.0 円	

(B) 構造全部同一ニシテ 杭長 7.2m トス

杭差較	18 @ 5 =	90
水中当格		40
		130
		720
塵芥除共		850.0 円

Piers in deep water (C)

米松		4.80 m³ all	@ 50.0 =	240.0
杉丸太柱		12 本	@ 5.0 =	60.0
松杭	末口21cm × 11.0m	18 本	@ 35.0 =	630.0
				930.0
撤去				100.0
				1030.0
塵芥除	杭28ᵈ・9ᵐ ニ対シ 杭打搬出 二基	2 @ 135 =		270.0
				1300.0 円

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CALCULATIONS FOR

三重縣揖斐長良川橋假橋設計

60 piers required.
Total cost. 塵芥除去

Piers A	42 piers	@ 720.00	=	30240	
〃 B	8	@ 850.00	=	6800	
〃 C	10	@ 1300.00	=	13000	
	60			50040	円
両詰取付	2	@ 80	=	160	
				50200	円

古材處分收入金

Piers A	42	@ 200.–	=	8400	
〃 B	8	@ 250.–	=	2000	
〃 C	10	@ 450.–	=	4500	
				14900	円

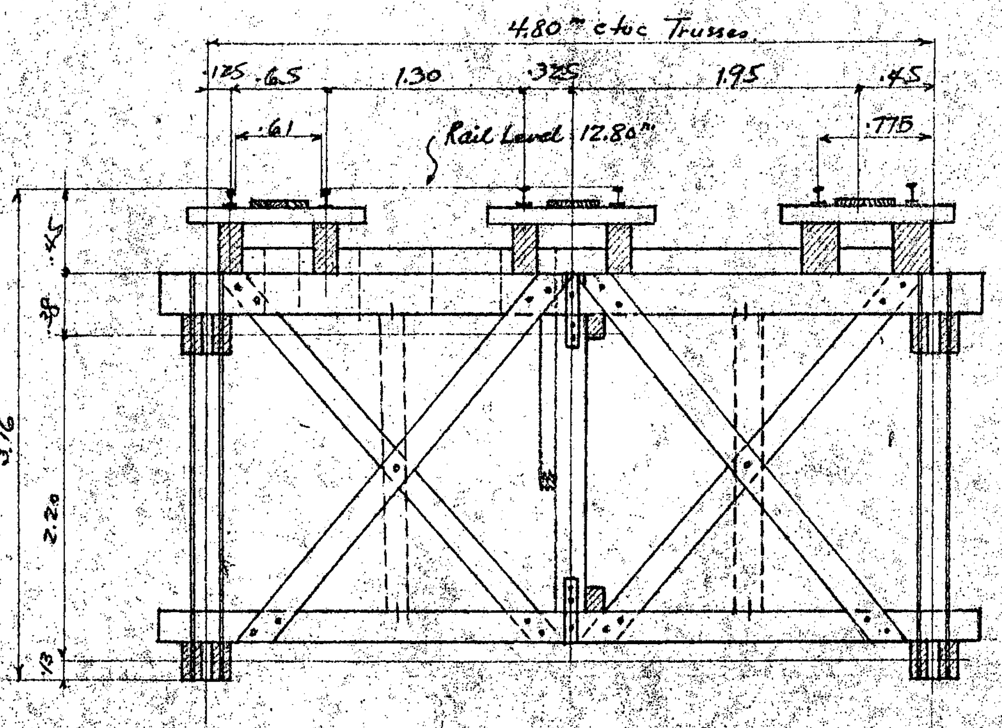
差引損料 35300 円

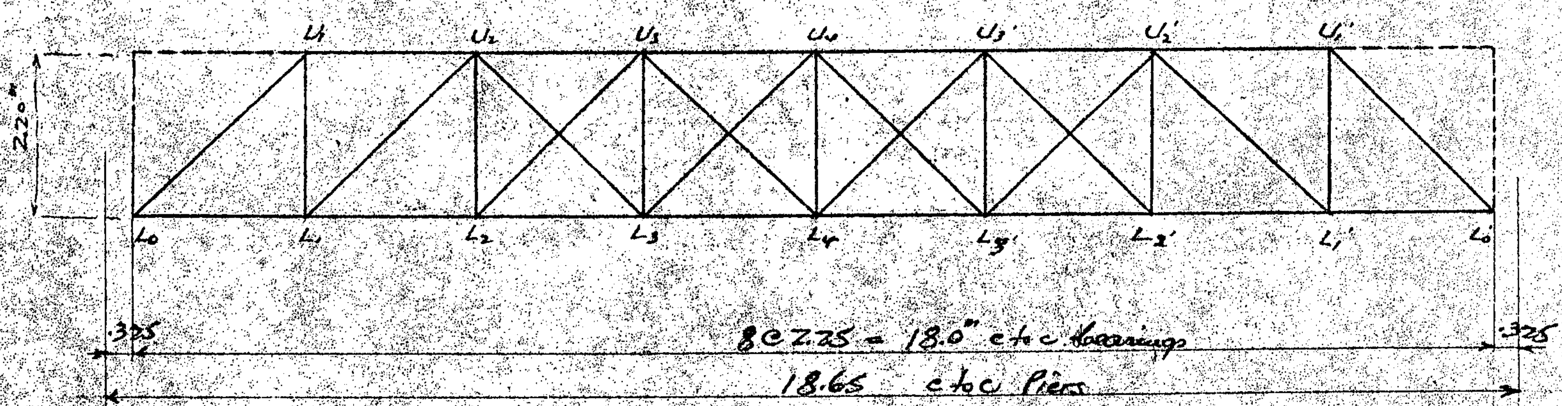
CALCULATIONS FOR

三重縣揖斐長良川橋假桟橋設計

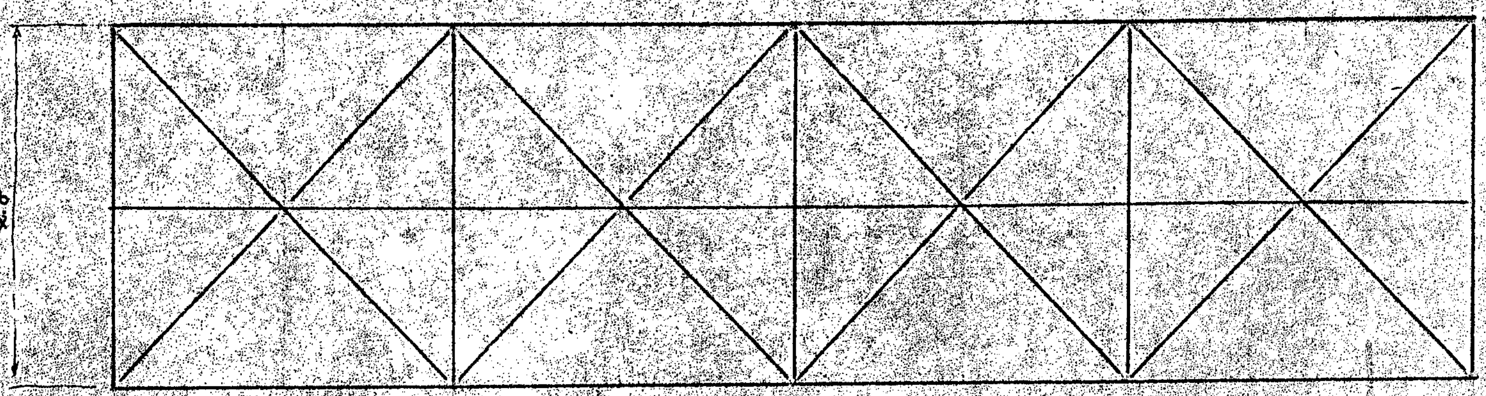
Design of wooden truss. Total length of bridge c to c of end bearings = 1100 metres about.

59 spans @ 18.0 m = 1062.0
58 clearance @ .65 = 37.7
1099.7 metres c to c end bearings.





Skeleton of truss.



Lower Lateral bracing.

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CALCULATIONS FOR

三重縣揖斐長良川假棧橋設計

Design of stringers.

Stringers under air-lock loads. span length 4.5 meters

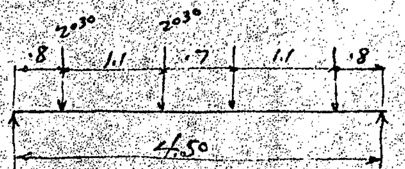
Dead Load:

Track assumed ½ · 50 = 25
beam 50
75 kg per lin meter

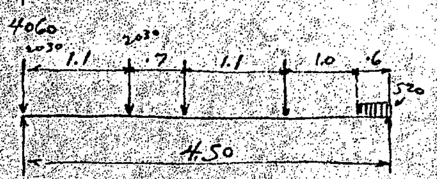
Dead Load moment = ⅛ · 75 · 4.5² = 190 kgm
Dead Load shear = ½ · 75 · 4.5 = 170 kg

Live Load

Max. concentration on wheel due to air lock = ¼ · 6500 = 1625
25% impact = 405
2030 kg



moment = 4060 · ~~2.75~~ 1.90 = 7710
2030 · 1.1 = −2230
5480 kgm.



Reaction 2030 · 3.40 = 6900
2030 · 2.70 = 5480
2030 · 1.60 = 3250
520 · 0.6² ÷ 2 = 94
15724 ÷ 4.50 = 3500
2030
End shear = 5530 kg

Summary of moments and shears.

	moment	end shear
Dead Load	190	170
Live Load	5480	5530
	5670 kgm	5700 kg

Section modulus req'd = 5670 · 100 / 85 = 6670 cm³

Use 40 · 25 cm stringer. Sm = 25 · 40² / 6 = 6670 cm³ OK

Shear stress = 5530 / (30 · 25) = 7.4 kg/cm² OK.

Cut off 10 cm at support.

Stringers under material trucks. span length 4.5 meters

Dead Load: — Track assumed ½ · 50 = 25
beam 30
55 kg per lin m

Dead Load moment = ⅛ · 55 · 4.5² = 139 kgm
Dead Load shear = ½ · 55 · 4.5 = 124 kg.

Live Load.

Load for one truck assumed 1500 kg, for one stringer 1500 ÷ 2 = 750 kg
25% impact = 187
937 kg

Assuming this load to be distributed uniformly over the length of 1.8 meter.
Uniform load = 937 ÷ 1.8 = 520 kg per lin meter.

Live Load moment = ⅛ · 520 · 4.5² = 1317 kgm
Live Load shear = ½ · 520 · 4.5 = 1170 kg.

Summary of moments and shears.

	moment	shear
Dead Load	139	124
Live Load	1317	1170
	1456 kgm.	1294 kg

Section modulus req'd = 1456 · 100 / 85 = 1715 cm³

Use 30 · 15 cm Sm = 15 · 30² / 6 = 2250 cm³ ok.

Unit shear = 1294 / (15 · 30) = 3 kg/cm² ok.

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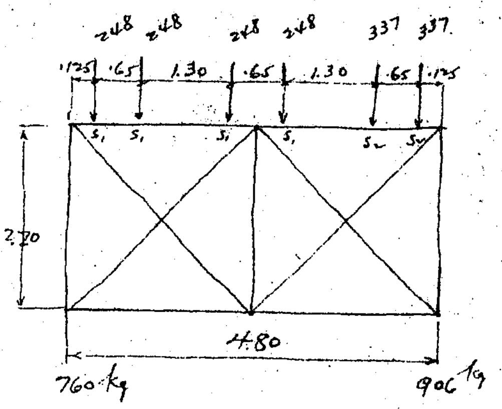
CALCULATIONS FOR

三重縣揖斐長良川假棧橋設計

Design of Cross Frame
Dead Load.

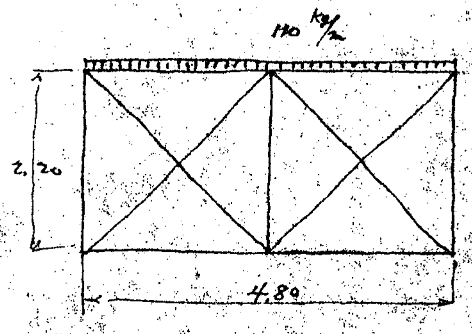


Span length 4.8 meters, Spacing 4.5 meters.
Stringer concentration on frame
Stringers S_1 track = $\frac{\text{stringer}}{\text{frame}}$ 55 × 4.5 = 248 kg
" S_2 " 75 × 4.5 = 337
Dead load of frame assumed 110 kg per lin m.
Moment on frame due to stringer concentration
Reaction
337 × .90 = 304
248 × 13.50 = 3346
3650 ÷ 4.8 = 760 kg left reaction
1666 − 760 = 906 kg right "
moment.
760 × 2.40 = 1825
248 × 4.275 = −1050
775 kgm.
Load on center panel point
585 × .125 = 73
585 × .775 = 453
496 × 2.075 = 1030
1556 ÷ 2.40 = 650 kg

$\sqrt{2.2^2+2.4^2} = \sqrt{10.60} = 3.26^m$

$\frac{3.26}{2.2} = 1.48^m$

Bending moment on top chord
Reaction 248 × .325 = 81
337 × 4.275 = 1425
1506 ÷ 2.40 = 628 kg.
moment 628 × .775 = 487
337 × .65 = −219
268 kgm. at S_2



Moment on frame due to own weight
Load on center panel point
110 × 2.4 = 264 kg Reaction = 132 kg
moment at center = 132 × 2.4 = 317 kgm.
Bending moment on top chord
$\frac{1}{8}$ × 110 × 2.4² = 80 kgm.
Shear $\frac{1}{2}$ × 110 × 2.4 = 132 kg.

Summary for Dead Load moments, shears and reactions

	moment at center	moment on top chord	Shear	load on center
Stringer Concentration	775	268	628	650
Own weight	317	80	132	264
	1092 kgm	348 kgm	760 kg	914 kg

~~Chord stress = $\frac{1092}{2.20}$ = 496 kg C or T~~

~~Diagonal stress = $\frac{914}{4}$ × 1.48 = 338 kg C or T for one member.~~

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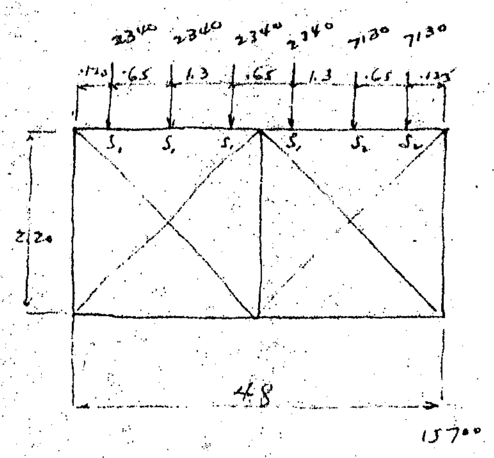
CALCULATIONS FOR

三重縣揖斐長良川橋假棧橋設計

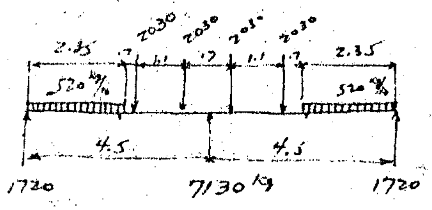
Live Load.

Uniform load on S_1 = 520 kg per lin meter for one rail.
Concentration on S_1 = 2030 kg for one rail.



Stringer concentration S_1 520 × 4.50 = 2340 kg

S_2



2030 × 3.05 = 6190
2030 × 4.15 = 8420
$\frac{520 \times 2.35^2}{2}$ = 1435
16045 ÷ 4.5 × 2 = 7130 kg

Reaction on truss

2340 × 5.70	=	13350
7130 × 8.70	=	62000
		75350 ÷ 4.8 = 15700 kg

Moment at center

15700 × 2.40	=	37700
7130 × 3.90	=	− 27800
2340 × .325	=	− 790
		9110 kgm

Load on center panel pt.

2340 × 2.975	=	6960
2340 × 2.075	=	4860
7130 × 0.90	=	6420
		18240 ÷ 2.4 = 7600 kg

Moment on top chord.

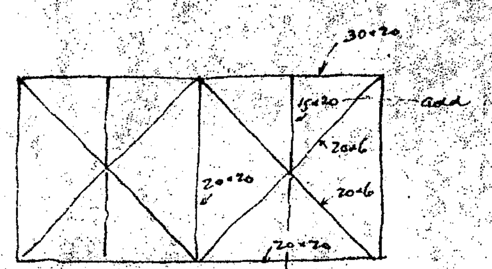
2340 × .325	=	790
7130 × 3.90	=	27800
		28590 ÷ 2.4 = 11900 kg shear
11900 × .775	=	9220
7130 × .65	= −	4630
		4590 kgm.

Summary of moment & shears.

	moment at center	m on top chord	shear on top chord	load on center panel pt.
Dead Load	1092	348	760	914
Live Load	9110	4590	11900	7600
	10202 kgm	4938 kgm	12660 kg	8514 kg

Chord stress = $\frac{10202}{2.2}$ = 4650 kg T & C   use 30 × 30 with center vertical col.

Diagonal stress = $\frac{8514}{4}$ × 1.48 = 3150 kg C or T   use 20 × 6



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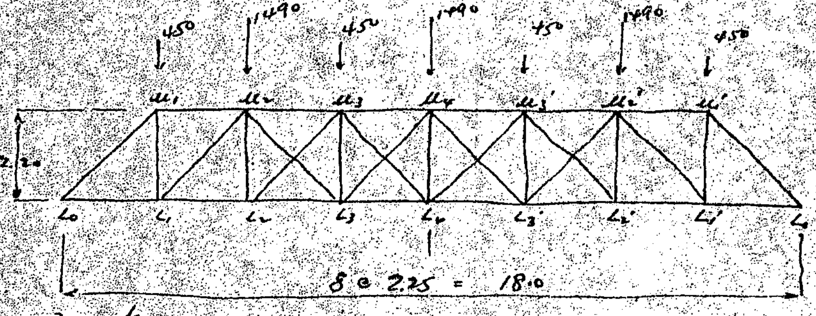
CALCULATIONS FOR

三重縣椙窪長良川橋假棧橋設計

Design of Main Truss.



3135 kg

Reaction = 450 × 2 = 900
1490 × 1.5 = 2235
3135 kg

moment at center
3135 × 9.0 = 28220
450 × 2.25 × 4 = − 4050
1490 × 2.25 × 2 = − 6710
17460 kgm

max chord stress = $\frac{17460}{2.2}$ = 7950 kg T or C

Dead Load Cross frames on M_2, M_4 & M_2' only

Tracks and cross frame
panel load tracks + stringer 906
frame wt 132
1038 kg

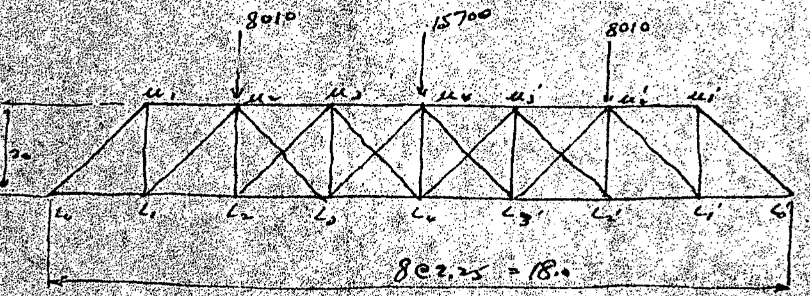
call this 1040 kg

Truss assumed 200 kg per lin m
panel concentration = 200 × 2.25 = 450 kg M_1, M_3
1490 kg M_2, M_4

Shear = 3135 kg Diagonal length = $\sqrt{2.25^2+2.2^2}$ = 3.15
Coef. = $\frac{3.15}{2.2}$ = 1.43.

Stresses in diagonals
L_0–M_1 3135 × 1.43 = 4480 kg
L_1–M_2 2685 × 〃 = 3840 〃
L_2–M_3 1195 × 〃 = 1710 〃
L_3–M_4 745 × 〃 = 1070 〃

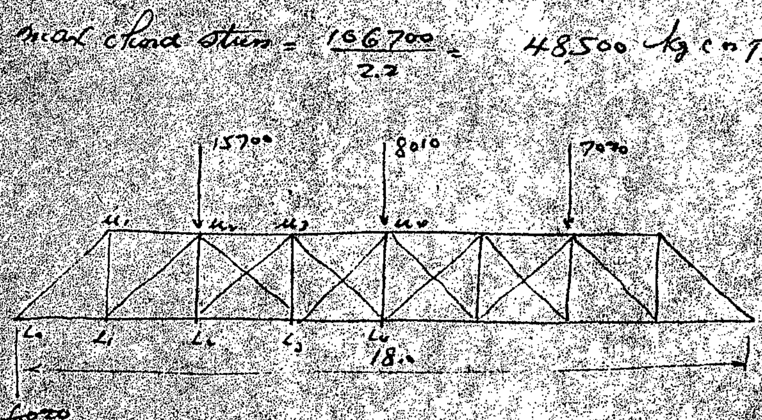
Live Load



15860

moment at center
15860 × 9.0 = 142700
8010 × 4.5 = − 36000
106700 kgm

max chord stress = $\frac{106700}{2.2}$ = 48500 kg C or T.

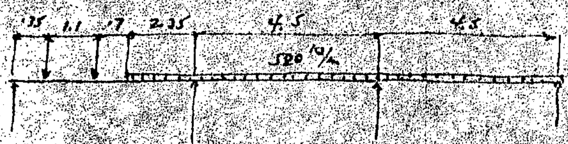


6020
17550

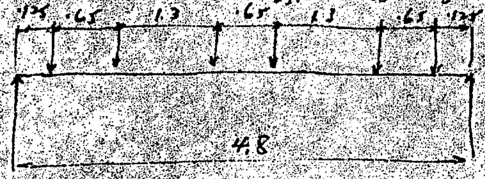
15700 × ½ = ~~8850~~ 7850
8010 × ¼ = 2000
9850

15700 × ⅛ = 3930 kg

max concentration on truss = 15700 kg page 12



1720 1170
1170 1170
2890 2340



Reaction 2340 × 5.70 = 13330
2890 × 8.70 = 25120
38450 ÷ 4.8 = 8010 kg

panel load for unif load = 2340 × 3 = 7020 kg
Shear
Reaction 15700 × 2.25 × 6 = + 212000
8010 × 2.25 × 4 = + 72000
7020 × 2.25 × 2 = + 31600
~~108400~~ 315600 ÷ 18 = ~~6020~~ 17550 kg

Stress in L_0–M_1 & L_1–M_2
~~6020~~ 17550 × 1.43 = ~~8610~~ 25100 kg C

Stress in L_2–M_3, L_3–M_4
9850 × 1.43 = 14100 kg C.

Stress in counter = 3930 × 1.43 = 5600 kg C

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CALCULATIONS FOR

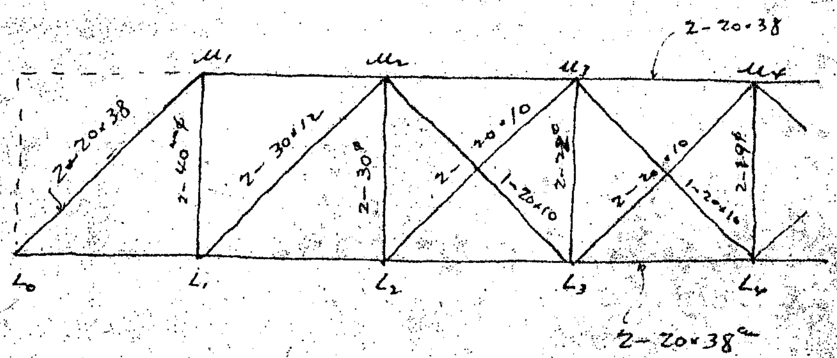
三重縣揖斐長良川橋假材構設計

Summary for stresses on truss members.

	Chord.	Diagonal L_0-U_1	L_1-U_2	L_2-U_3	L_3-U_4	Counter U_3-L_4
Dead Load	7950	4480	3840	1710	1070	1070
Live Load	48500	25100	25100	14100	14100	5600
	56450 kg T or C	29580 kg C	28940 kg C	15810 kg C	15170 kg C	4530 C

Vertical.

	U_1-L_1	U_2-L_2
Dead Load	2685	1195
Live Load	17550	9850
	20235 kg T	11045 kg T



Lateral bracing
Steel bars.
$L_0 \sim L_2$ 1-30φ
$L_2 - L_4$ 1-22φ

Materials in one span (approximate figure)
米松材

Chords top + bottom	4	−	.20 × .38 × 18.6 =	5.650
Diagonals L_0-U_1	4	−	.20 × .38 × 2.7 =	.821
〃 L_1-U_2	4	−	.12 × .30 × 2.7 =	.389
〃 L_2-U_3 + L_3-U_4	8	−	.10 × .20 × 2.7 =	.432
〃 counters	4	−	.10 × .20 × 2.7 =	.216
Column on L_0	2	−	.25 × .25 × 1.8 =	.225
Chord blocks	14	−	.30 × .70 × .55 =	1.615
bearing blocks	2	−	.55 × .30 × 1.0 =	.330
				9.678 × 2 = 19.356 m^3

Cross frame.

Chord top	1	−	.20 × .30 × 5.5 =	.330
〃 bott.	1	−	.20 × .20 × 5.5 =	.220
posts	1	−	.20 × .20 × 2.0 =	.080
〃	2	−	.15 × .20 × 2.0 =	.120
Diagonals	4	−	.06 × .20 × 3.3 =	.158
				.908 × 5 = 4.540
				23.896 call this 24.0 m^3

Approximate weight of steel

vertical member	8 −	40 mmφ × 2.8	@ 9.86	=	221
	8 −	30 × 2.8	@ 5.55	=	124
	8 −	22φ × 2.8	@ 2.98	=	67
	4 −	19φ × 2.8	@ 2.22	=	25
nuts + washers	28 sets		@ 12.0	=	336
bottom lateral	4 −	30φ × 7.0	@ 5.55	=	155
	4 −	22φ × 7.0	@ 2.98	=	83
nuts + washers	8 sets		@ 3	=	24
frame plates	10 pls.	75 × 9	@ 5.30 × 1.3	=	69
Splice pls.	4 sets		@ 40	=	160
bolts say					600
					1864
錆止め say					36
					1900 kg for one span.

CALCULATIONS FOR

三重縣揖斐長良川假設橋設計

Tracks

12# rails with accessories 3@1100 = 3300 meters about

Sleepers	[illegible]	3000 sleepers	.10 × .10 × 1.20"	=	36.00
Stringers		4 –	.15 × .30 × 1100	=	180.00
Stringers		2 –	.25 × .40 × 1100	=	220.00
Struts		295 –	.2 × .2 × 4.0	=	47.
					483.0 ÷ 59 = 8.19 m³ per span
Planking		3 × .50 × 1100	= $\frac{16500}{59}$ m²	=	28.0 m²