JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

昭和四年四月

大阪府淀川十三橋予算設計書

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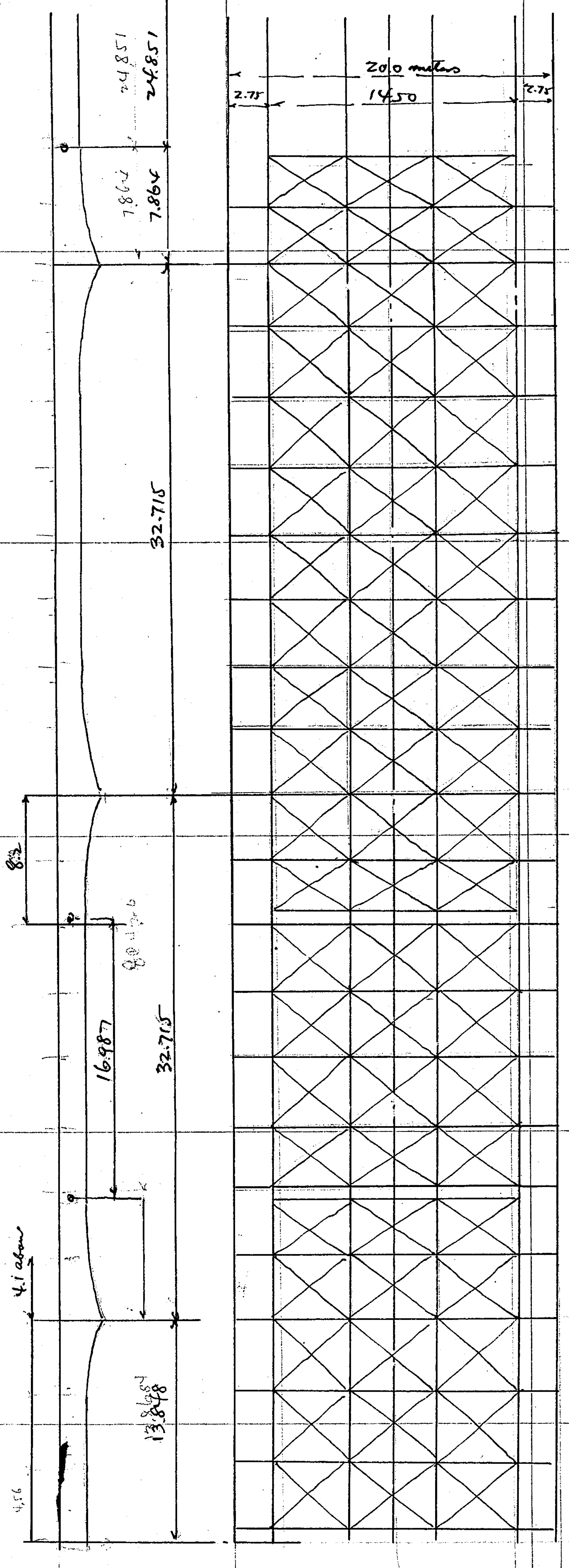
CALCULATIONS FOR

Preliminary Estimate of cost Juso-Bashi for Osaka-Fu.



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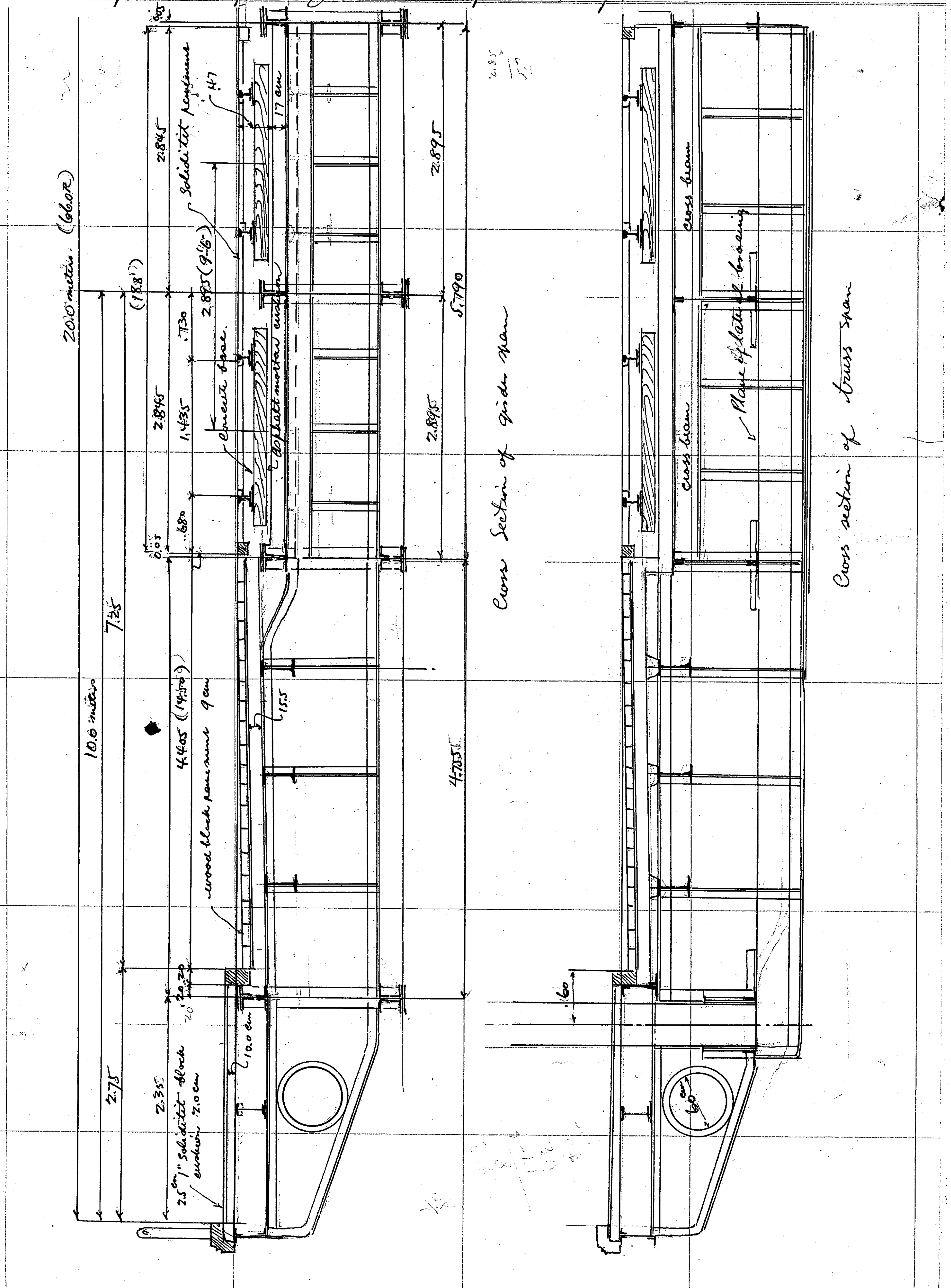
CALCULATIONS FOR

Preliminary Estimate of Cost. Juso-Bashi for Osaka-fu.



増田淳氏関係資料（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Preliminary Estimate of Cost Jiuso-Bashi for Osaka-fu.

Girder span

Slab under Electric Rwy track span length 4.57/3 = 1.52 meter.

Dead Load Total depth .55 load per sq meter = .55 × 2400 = 1320 kg.

Dead Load moment = 1/10 × 1320 × 1.52² = 304.0 kgm.

Dead Load shear = 1/2 × 1320 × 1.52 = 1000 kg.

Live Load. motor truck loading Rear wheel concentration 4500 kg.

30% impact 1350

5850 kg.

distribution of wheel concentration.

Contact between wheel & pavement = 20

2@ ~~3~~8 = 96

longitudinal a = 1.16

Transverse b = 39+ ~~96~~ 76 = ~~1.35~~ 1.15

For 2 wheels side by side. b = 0.90+.39 + ~~.96~~ .76 = ~~222~~ 202

Effective width E = 2/3 × ~~1.52~~ 1.52 + ~~222~~ 202 = ~~323~~ 3.03 meters.

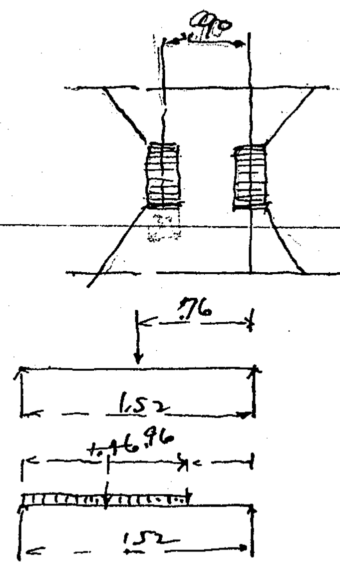
for one wheel say ~~1.62~~ 1.52

load per meter strip 5850 ÷ ~~1.62~~ 1.52 = ~~3610~~ 3850 kg.

moment = ~~3610~~ 3850 / 2 × .76 = ~~1370~~ 1460 kgm

for continuity of slab. ~~1370~~ 1460 × 0.8 = ~~1095~~ 1170 kgm

shear say ~~3610~~ 3850 × ~~.96~~ 1.04 / 1.52 = ~~2230~~ 2640.



Summary for moments and shears

	moments	shear
Dead Load	304	1000
Live Load	~~1095~~ 1170	~~2230~~ 2640
	~~1399~~ 1474 kgm	~~3230~~ 3640 kg.

Effective depth = $\sqrt{\frac{1399 \times 100}{100 \times 7.18}}$ = 14.0 cm

Effective say 14.5 cm & use 17 cm slab.

insulation at bottom 2.5 cm

Electric Car Loading. water car max axle load 9000 kg. single wheel load 4500 kg.

Distribution. 1.00 + .76 = 1.76.

Transverse. 2.10 + .76 = 2.86

axle ~~wheel~~ concentration 9000

impact. 30%. 2700

11700 ÷ 2.86 = 4090 kg per lin meter.

This load spread over 1.76 meter.

For one meter square 4090 ÷ 1.76 = 2320 kg.

Live Load moment = 1/10 × 2320 × 1.52² = 536 kgm.

Roadway Slab. Span length 1.19 meter

Dead Load. wood block pavement 9 cm @ 10 = 90

mortar cushion. 2 cm @ 17 = 34

Concrete slab 15 cm @ 24 = 360

484

misc say 16

500 kg per sq meter

CALCULATIONS FOR

Preliminary Estimate of Cost Jui-sō-Bashi for Osaka-fu.

Dead Load moment $= \frac{1}{10} \times 500 \times 1.19^2 = 60.2$ kgm
shear $= \frac{1}{2} \times 500 \times 1.19 = 298$ kg

Live Load motor truck loading 5850 with impact.

distribution $a = 20 + 22 = 42$ cm
$b = 39 + 22 = 61$ cm
Effective width $= \frac{2}{3} \times 1.19 + 42 = 1.21$
load per meter strip $= \frac{5850}{1.21} = 4840$
$m = \frac{4840}{2} \times .595 = 1440 \times 0.8 = 1150$ kg.
shear $= 4840 \times \frac{.89}{1.19} = 3620$

4840
1.19
.61 .58
1.19

Summary for moments and shears

	moment	shear
Dead Load	60	298
Live Load	1150	3620
	1210 kgm	3918 kg.

Effective depth $= \sqrt{\frac{1210 \times 100}{100 \times 7.18}} = 13$ cm
insulation at bottom 1.3 / 2.5
depth of slab. — — 15.5

Revised — Dead Load pavement 9 @ 10 = 90
mortar cushion 2 @ 17 = 34
15.5 cm slab @ 24 = 372
496
14
510 kg per sq meter.

Sidewalk slab. span length max 1.35 meters
Dead Load concrete 10 cm @ 24 = 240
3 cm @ 21 = 63
303
misc 7
310 kg.
Live load 500
810 kg.

Dead and Live load moment $= \frac{1}{10} \times 810 \times 1.35^2 = 148$
Effective depth $= \sqrt{\frac{14800}{7.18}} = 4.5$ cm
2.0
6.5 cm

Stringer under "Electric" Ry Track span length 2.895 spacing. $\frac{4.2}{3} = 1.40$ for girder
Dead Load $1320 \times 1.40 = 1850$
beam say 200
2050 $m = \frac{1}{8} \times 2050 \times 2.895^2 = 2150$ kgm

Live Load
4500
1350
5850
.73 1.435 .73
1.76
1.42 1.42

L.L. moment $= 5850 \times .73 = 4270$
D.L. " 2150
6420 kgm

$Sm = \frac{6420.00}{1100} = 582.0$

$\frac{.98}{1.42} = .69$

$m = \frac{1}{8} \times 2320 \times 1.42^2 = 5850$
$5850 \times .69 = 4040$
2150
6190

$Sm = \frac{619000}{1100} = 562.$

use 300×150 @ 48.8 $= Sm = 633.2$

At panel point. depth of girder 1.00 meter.
flange section 2LS $75 \times 75 \times 9$ @ 12.69 = 25.38
2
50.76
web 1000×0.8 = 80.00
130.76 @ .785 = 103
Details say 25%. 27
130 kg.

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CALCULATIONS FOR

Preliminary Estimate of Cost Jiu-so Bashi for Osaka-fu.

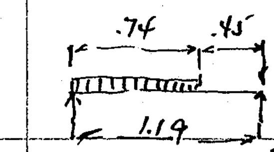
Approximate weight of cross beam under track
for one panel
2Is 300×150 @ 48.34 × 2.9 = 280
Connection Ls 8Ls @ 6 kg 48
1 Cross beam 130 × 2.9 = 380
708
call this 710 kg per panel.

For two tracks — 710 × 2 = 1420
1420 ÷ 4.2 = 340 kg per lin meter.

Stringer for Roadway Span length 4.2 meter spacing 1.19 meter
Dead Load Slab & pavement 510 × 1.19 = 607
beam assumed 73
680

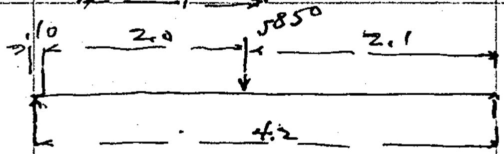
$m = \frac{1}{8} \cdot 680 \cdot 4.2^2 = 1500$ kgm

Live Load motor truck loading — with impact 5850 kg.
uniform live load. 600 kg per square meter

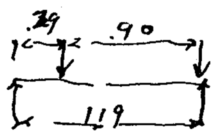


$\frac{600 \times .74^2}{2 \times 1.19} = 138$ kg.

moment = $\frac{5850}{2} \cdot 2.1$ = 6150



" = $\frac{1}{8} \cdot 138 \cdot 4.2^2$ = 304
6454



$\frac{1.000}{1.328} \times 5850 = 7270$ $m = \frac{7270}{2} \cdot 2.1 = 7620$
Dead Load m 1500
9120 kgm.

Section modulus req'd = $\frac{912000}{1100}$ = 829
~~Use 350×150 I @ 58.54 kg Sm = 870.6~~
14" × 6" I @ 68.5 kg. Sm = 1030.

Approximate weight 14"×6" I @ 68.5
details say 5.5
74.0 kg
6
444 kg per lin meter.

Sidewalk stringer 12" I. 300 × 150 I @ 48.34 kg.

Fascia stringer 2Ls 90×90×10 @ 13.34 = 26.68
450 × 8 × .785 = 28.20
54.88
details say 5.12
60.00 kg
55
135.0 kg per lin. meter.

Summary 2 @ 135 = 270 kg per lin meter.

Cantilever bracket and cross beam under sidewalk and motor traffic way.
web. 120.0 × 0.8 cm @ .785 = 75.3
4Ls 90·90·10 @ 13.34 = 53.4
details say 30.0
158.7 call this 160 kg.
weight = 2 × 160 × 7.3 = 1230 kg.
1230 ÷ 4.2 = 294 kg per lin meter of span

CALCULATIONS FOR

Preliminary Estimate of Cost Jiu-sō Bashi for Osaka-fu.

Summary of weight of stringers and floor beam

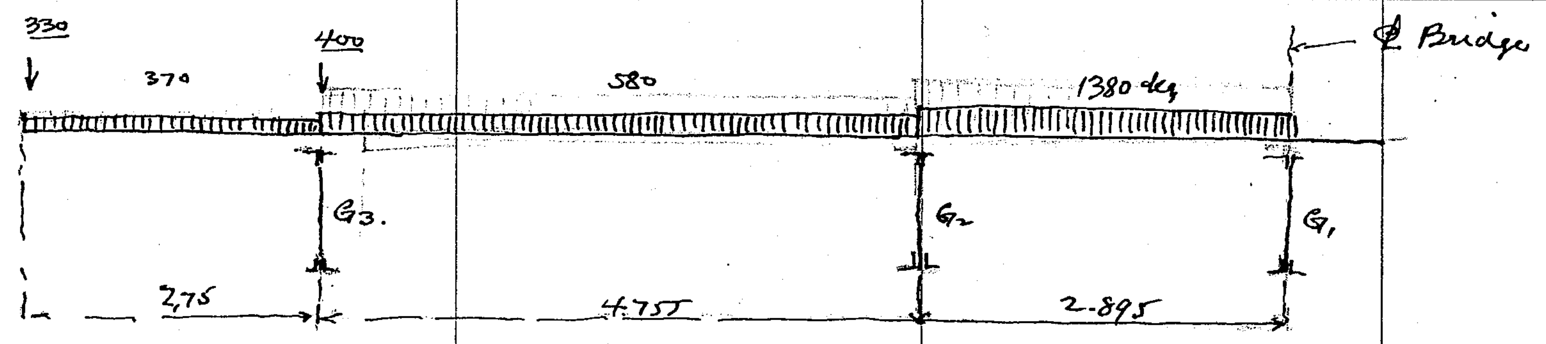
Under Electric Ry Track	340	kg.
Under motorway - stringers.	444	
Under sidewalk stringers	270	
Cross beam and cantilever on Roadway	294	
	1348	kg per lin meter of span.

Lateral Bracing between main girders.

4L5 125 × 90 × 10	@ 16.09	× 7.16	=	460
8L5	"	× 6.35	=	817
Connections 14 @ 15			=	210
3 @ 10			=	30
				1517 kg.

1517 ÷ 4.2 = 362 kg.
misc. strut. &c. 100
462 kg per lin meter.

Dead Load on main girders.



Sidewalk. Slab and pavement say 310
Stringer say 60
370 - kg

Handrail 80 kg.
Coping stone 250
330 kg per lin. meter.

at curb stone 400 kg per lin meter
Roadway slab and pavement 510
structural Steel say 70
580 kg per sq meter

Roadway at Electric Ry track 1320
structural steel 60
1380 kg per sq meter.

Dead Load on G3.

	370 × 2.75 =	1020
		330
Extra for cantilever		484
at curb line.		400
From Roadway	$580 \times \frac{4.755}{2}$ =	1380
		3614 kg.
Girder assumed		700
		4314 kg per lin meter.

$370 \times 2.75 = 1020 \times \frac{1.37}{4.755} = 294$

$330 \times \frac{2.75}{4.755} = 190$

484

Preliminary Estimate of Cost Jiu-sō-Bashi for Osaka-fu

Load on girder G2
From Roadway 1380
From track side 1380 × 1.45 = 2000
3380
Less Cantilever effect 484
2896
main girder assumed 700
3596 kg per lin meter.

Load on girder G1
1380 × 2.9 = 4000
main girder 700
4700 kg per lin meter.

Live Load motor truck loading 12 tons
impact 3.6
15.6 ÷ (6.60 × 2.70) = 875 kg per sq meter.
Assume Equiv. uniform live load 750 kg per sq meter throughout

Live Load on girder G3
Unif. live load. 500 × 2.75 = 1375
1375 × 1.37/4.75 = say 400
From Roadway 750 × 4.755/2 = 1785
3560 kg per lin meter.

Live Load on girder G2
750 × 4.755/2 = 1785
750 × 1.45 = 1090
2875 kg per lin meter

Live Load on girder G1
750 × 2.9 = 2180 kg per lin meter.

Summary for load on girders

	G3	G2	G1
Dead Load	4314	3596	4700
Live Load.	3560	2875	2180
	7874	6471	6880 kg

Suspended span span length 17.0 meter.
G1 m = 1/8 × 6880 × 17.0² = 249,000 kg meter.
G2 m = 1/8 × 6471 × 17.0² = 234.000

DL. 4700 × 8.5 = 40.000
LL. 2180 " = 18500
DL. 3596 " = 30500
LL. 2875 " = 24400

Cantilever arm
G1 Dead Load 4700 × 7.86²/2 = 145000 40.000 × 7.86 = 315.000
Live Load 2180 × 7.86²/2 = 67300 18500 × " = 145500
212300
460500
212300
672800

G2 Dead Load 3596 × 7.86²/2 = 111000 30500 × 7.86 = 240.000
Live Load 2875 × " = 88700 24400 × 7.86 = 192.000
199700
432000
199700
631700

Anchor span span length 32.72
G1 Dead Load m = 1/8 × 4700 × 32.72² = 630.000
less ~~465.000~~ 460.000
170.000
~~485.000~~
Live Load = 1/8 × 2180 × 32.72² = 292.000
~~797.000~~
462000 kg meter.

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Preliminary Estimate of Cost Jiu-sō Bashi for Osaka-fu.

G2. Dead Load 1/8 × 3596 × 32.72² = 480.000
less ~~111.000~~ 351
129.000
~~369.000~~

Live Load 1/8 × 2875 × 32.72² = 386.000
~~755.000~~ 515.000 kg m

Outside girder G3.
Suspended span 1/8 × ~~4314~~ 1874 × 17.0² = ~~156.000~~ 284.000
DL. 4314 × 8.5 = 36600
LL. 3560 × " = 30200

Cantilever arm DL. 4314 × 7.86²/2 = 133.500
LL. 3560 × 7.86²/2 = 110.000
243.500 kgm
36600 × 7.86 = 288.000
30200 × " = 237.000
525.000
243.500
768.500

Anchor span
DL. 1/8 × 4314 × 32.72² = 577.000
Less ~~133.500~~ 421.500
155.500
~~443.500~~
LL. 1/8 × 3560 × 32.72² = 480.000
~~923.500~~ 635.500 kgm

Section of Anchor span max pos. moment = 515.000 kgm
web assumed 170 × 13 = 221. 1/8 web = 27.60 cm back to back of Ls = 171 cm
Effective depth 1.534
flange stress = 515.000 / 1.534 = 336000 kg.
SR = 336000 ÷ 1200 = 280.00
27.6
252.4 net

4Ls 150 × 150 × 15 = 171.00 − 30.0 = 141.00
2Pls. 350 × 19 = 133.00 − 112.00
304.00 253.00 a cm net

Section of Cantilever arm moment = 672800 kgm
web assumed 250 × 13 = 325 1/8 web = 40.6 back to back of Ls 251.
Effective depth 2.31
flange stress = 672800 / 2.31 = 292.000 kg.
SR = 292.000 = 244.0
40.6
203.4 kg cm²

4Ls 150 × 150 × 15 = 171.00 − 30.0 = 141.00
1Pl. 350 × 19 = 66.50 − 56.00
237.50 197.00 cm²

At center. main section
304
304
221
829 @ .785 = 650

at pier
237.5
237.5
325.0
800.0 @ .785 = 625 kg.

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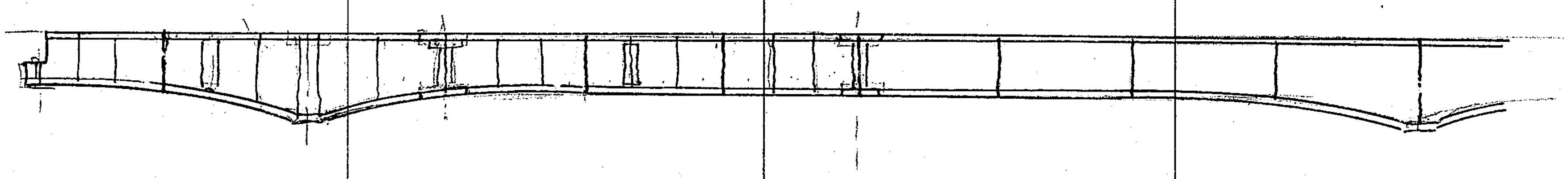
CALCULATIONS FOR

Preliminary Estimate of Cost Juso Bashi for Osaka-fu



Details of girder 78 Ls 125 × 90 × 10 @ 16.09 × 180 = 2260
web splice 7 @ 300 = 2100
flange splice 3 @ 700 = 2100
fillets say 1000
Details at hinge. say 2000
9460 ÷ 48.5 = 197

main section. 650
details say 35% 228
For girders G_1 and G_2 878 kg per lin meter.

~~Suspen~~
For girder G_3. Center moment 635500
web assumed 1.90 × 1.3 = 247. 1/8 web = 30.9 back to back of Ls 191.
Effective depth 1.73

flange stress = $\frac{635500}{1.73}$ = 367000

section reqd = $\frac{367000}{1200}$ = 306.0
30.5
275.5 □ net

moment at pier. 768500 kgm
web assumed 270 × 1.3 = 352 1/8 web. = 44.0
Effective depth 2.53

flange stress = $\frac{768500}{2.53}$ = 304.0
44
260.0 net

Flange section same as for G_1 and G_2 for approximate weight say 920 kg/meter.

Suspended span G_1 = 249000
G_2 = 234000
Depth say 170 × 10 = 170 1/8 web = 21.30
Effective depth say 1.60

Flange stress = $\frac{249000}{1.60}$ = 156000

SR = $\frac{156000}{1200}$ = 130.0
21.3
108.7 □ cm net

4 Ls 150 × 150 × 11 = ~~31.79~~ 127.16 − 22.0 = 105.16 net
105.16
web 170.00
380.32 @ .785 = 300
Details say 40 % 120
420 kg.

For G_1 and G_2 420 kg per lin ft.
" G_3 450 " " " " assumed.

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Preliminary Estimate of Cost Jiuso-Bashi for Osaka-fu

Suspended span 24.85 meter

G_1 = 1/8 × 6880 × 24.85² = 531000 kgm
G_2

main section will be same as for anchor span

Approximate weight of main girder.

Suspended span	G_1	420	anchor span	G_1	880
	G_2	420		G_2	880
	G_3	450		G_3	920

24.85 meter suspended span same as anchor span
13. meter simple beam same as 17.0 suspended span

4 × 48.5 = 194.0 2 × 17.0 = 34.0
4 × 24.85 = 100.0 26.9
294.0 60.9

G_1 and G_2 3 × 294 × 880. = ~~517.0~~ 777
3 × 60.9 × 420 = ~~51.0~~ 77
568.0 854

G_3 2 × 294 × 920 = 540
2 × 60.9 × 450 = 55
354.9
595
1449 tons

stringer floor beams & laterals. 1348 + 462 = 1810 × 354.9 = 642.0
main girders. 1449.0
shoes and misc. 70.0
2161.0 tons.

girder span over Canal - span length say 25.2 meter ~~26.0 meters out to out~~

G_1 and G_2 3 × 880 = 2640
G_3 2 @ 920 = 1840
4480 × 26.0 = 117.0
stringers &c 1810 × 26.0 = 47.0
~~164.0~~ tons + shoes. say 172
~~2161.0~~ 2161
~~2325.0~~ 2333 tons

Call this 2350 tons

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CALCULATIONS FOR

Preliminary Estimate of Cost Jinso-Bashi for Osaka-Fu.

Tied Arch Span 64.2 meter 4.59 panel. 14 panels.

Stringer under side walk. 2@135 = 270 kg per lin meter.

Roadway stringers
Dead Load. moment $\frac{1}{8} \cdot 680 \cdot 4.59^2 = 1770$ kgm
Live Load see p5 motor truck $\frac{5850}{2} \cdot 2.30 = 6720$
$\frac{1}{8} \cdot 138 \cdot 4.59^2 = 364$
7084 say 7100
1770
8870

$\frac{7270}{2} \cdot 2.3 = 8360$
1770
10130

Section modulus reqd = $\frac{1013000}{1100} = 922$
Use 14"·6" I @ 68.5 kg Sm = 1030

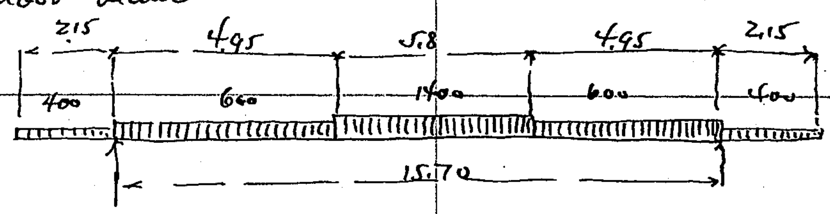
6@ 73 = 438
girder at curb. say
2@ 100 = 200
638 kg per lin. meter.

Summary.
sidewalk ~~438~~ 270
Roadway 638
track 580
1488

Framing under Electric Rwy Track

Transverse Joist. 4@ 50 × 2.9 = 580 ÷ 4.59 = say 130
3 longitudinal beams @ 150 450
580 kg per lin meter.

Floor beam



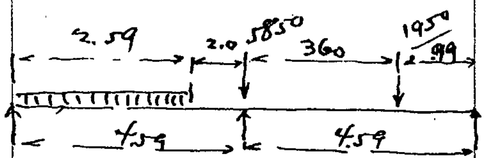
1400 × 2.9 = 4060
600 × 4.95 = 2970
7030

moment = 7030 · 7.85 = 55200
2970 · 5.37 = − 16000
~~4660~~ ~~2030~~ · 1.45 = − 6000 − 2940
~~36260~~ 33200 kgm.
− 923
~~35337~~ 32277

Cantilever moment $400 \cdot \frac{2.15^2}{2} = 923$

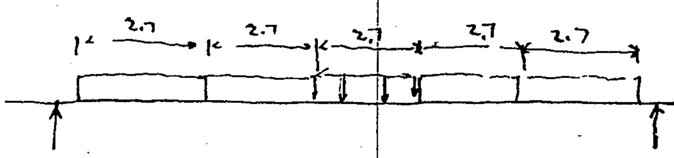
For 4.59 m panel ~~35337~~ 32277 · 4.59 = ~~162.000~~ 148.000 kgm
Dl. beam $\frac{1}{8} \cdot 600 \cdot 15.70^2$ = 18500
~~180500~~ 166500 kgm

Live Load ~~motor truck~~ loading. 5850 kg. per wheel



$1950 \cdot \frac{.99}{4.59} = 420$
5850
6270

Unif. $\frac{600 \cdot 2.59^2}{2 \cdot 4.59} = 439$ kg.



as unif. 2 × 6270 ÷ 2.7 = ~~2320~~ 4640
440
2760

4640
440
5080

$m = \frac{1}{8} \cdot \overset{5080}{\cancel{2760}} \cdot 15.70^2 =$ ~~850000~~ 157000
~~180500~~ 166500
~~337500~~ 323500

157000
166500
323500

web assumed ~~1500~~ 1700 · ~~10~~ 13 = ~~150~~ 221 1/8 web = ~~18.7~~ 27.6
Effective depth 1.68

flange stress = $\frac{\cancel{337500}\ 323500}{\cancel{1.40}\ 1.68}$ = ~~240.0~~ 18.7 ~~221.3~~

~~240.0~~
~~27.6~~
~~182.4~~

193.0
27.6
165.4

Preliminary Estimate of Cost Juu-sō-Bashi for Osaka-fu

assumed section.

2L 150×150×19 = 106.78 − 19 = 87.78
1Pl. 350×15 = 52.50 − 7.5 = 45.00
52.50 − 7.5 = 45.00

211.78
211.78
221.00
644.56 @ .785

177.78
~~177.98~~
221.00
576.56 @ .785 = ~~452~~
238
690 kg.

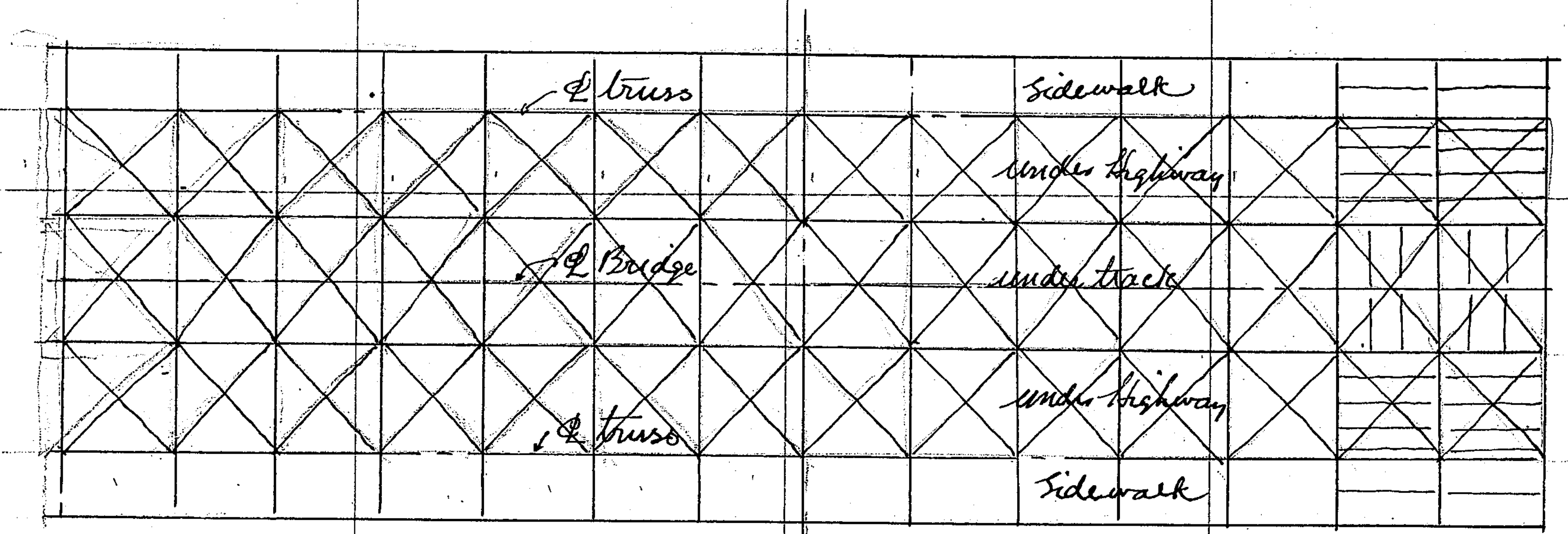
Details 52%

45%

505.0
228
733.0

approximate wt of Floor beam 690 (733) × 15.70 = 10850 11500
cantilever 2 @ ~~350~~ 500 × 2.15 = 1070 1070
11920 ~~call this 12000~~ 12570 12500

12600 ÷ 4.59 = ~~2620~~ 2720 kg.



Bottom Lateral Bracing. 520 kg × 64.0 = 33.000 kg.

Top lateral, sway and portal bracing – 78.000

Structural steel in one span
stringers 1488 × 65 = 96.700
floor beams. 15 × 12.500 = 187.500
Bottom laterals. 33.000
Top Lateral sway + Portal say 78.000
395.200 ton

395.000 ÷ 64 = 6180 kg.

Weight of floor.
1320 × 2.895 = 3820
510 × 4.755 = 2420
310 × 2.75 = 850
Handrail & coping 330
curb stone &c 400
7820 for ½ width
~~Structural steel except truss~~ 3090
one truss assumed. 3000
13910

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CALCULATIONS FOR

Preliminary Estimate of Cost Jiuso Bashi for Osaka-fu.

Dead Load stress of Ribbed Tied Arch design.

$m = 1/8 \times 13910 \times 64.2^2 = 7,170,000$ kgm

Depth of truss. 12.0 m stress $= \frac{7170.000}{12} = 597.000$ at ℄

at End $1.25 \times 597.000 = 747.000$ kg.

Live Load.

$500 \times 2.75 = 1375$
$600 \times 7.25 = 4350$
5725

extra say 1000
6725

$m = 1/8 \times 6725 \times 64.2^2 = 3490.000$
stress $= \frac{3470.000}{12} = 290.000$ kg at ℄

at End $1.25 \times 290.000 = 363.000$

at center. DL 597000 at End DL 747.000 at 1/4 pt say
LL 290.000 LL 363.000
887.000 1,110,000 1000,000 kg.

Depth say 2.0 meter. max moment due to live load at 1/4 point.
Panel load 21500 for 1/20 of span

moment $= 0.560 \times \frac{21500 \times 64.2}{2} = 387,000$ kgm

stress $= \frac{387.000}{2} = 193500$ kg.

Design section for 1.387.000 kg. ÷ 1000 = 1387 cm². for top chord.

For tie. 887000 ÷ 1200 = 740.0 net
887.0 gross about

$1.1 \times 1400 \times 0.785 = 1210$
$890 \times 0.785 = 700$
1910
50%. 950
2860
Hanger ~~in average~~ 310
3170 kg.
3170
~~Floor system + laterals~~. $6340 \times 64.2 = 407,000$

Floor system + laterals. 395.000
shoes &c 4@4 16.000
818.000

Call this 820 tons.

5 spans @ 820 = 4100 tons

Estimate of Cost of structural steel = 4100 @ 275 = 1,170.000
girder spans. ~~2350~~ 2700 @ 230 = 541,000
1,711,000

1,170.000
600.000
1770.000
250.000
2020.000

3800 @ 270 = 1,026.000
2700 @ 225 = 607.000
1637.000
250.000
1887.000

CALCULATIONS FOR

Preliminary Estimate of Cost Juso-Bashi for Osaka-Fu.

Estimate of Deck Construction -

Concrete. 2 - .10 × 2.75 = .550
2 - .155 × 4.80 = 1.49
2 - .17 × 2.9 = .99
3.03
misc say .17
3.20

Stone Masonry coping - 2 - .40 × .30 = .24
curb 2 - .20 × .30 = .12
~~track 2 - .20 × .25 = .10~~
.50 cubic meter / meter.

Reinforcing Bars. under track 5.79 @ 30 kg. = 174
9.50 @ 25 = 238
5.50 @ 12 = 66
478 call this 480 kg / meter

Form 20.0 square meters.

Pavement sidewalk. 2 × 2.55 = 5.10 sq meter Soliditit.
Roadway 2 × 4.4 = 8.80 " " woodblock

Handrails. 160 kg per lin. meter

Right of way for Electric Car tracks.
pavement. 4.90 square meter.
Concrete filling 1:3:6 - .32 × 2 × 2.84 = 1.81 cubic meters
sleeper and rails + accessories.

Estimate of Cost (1 meter strip)

Concrete	3.20 cubic meter	@	17.43 (43.50)	= 55.8	× 730	=	40.750
~~Stone~~	.36 ~~.40~~ "	@	~~162.0~~ ~~66~~	= ~~58.4~~ 15.7		=	11540 ~~42600~~
Reinf. bars.	.480 ~~kg~~ tons	@	165.00	= 79.0			57600
forms.	20	@	1.50	= 30.0			21600
Pavement Sidewalk	5.10	@	3.30 .80	= 16.8	4.07		3000 ~~12300~~
" Roadway	8.80	@	~~11.00~~ 4.50	= 96.8	39.60		28900 ~~70600~~
Handrails.	.160	@	360.00	= 57.6			42000
							287450 205350
	Pedestals	4 @ 4000					16000 42000
	"	4 @ 4000					16000 247350
	Wiring						10000
							329450

Track.

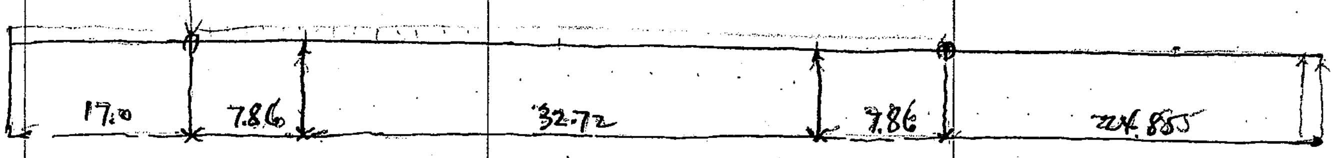
pavement.	4.9 sq meter	@	4.50	= 22.0	× 730	=	16,100
Concrete	1.81	@	15.36	= 27.8		=	20.300
							36400
Rails - accessories	50 × 4 = 200	@	130.00	= 26.0		=	19,000
sleeper				15.0		=	11000 } 33000
				5.0		say =	3000
							69400
							70.000.00

Preliminary Estimate of Cost Jiuso Bashi for Osaka-fu.

Design of pier.

Dead load.	G_1	4700	—	4700
	G_2	2 × 3600	=	7200
	G_3	2 × 4300	=	8600
				20500 kg per lin meter.
Live Load	G_1	2180		2180
	G_2	2 × 2875	=	5750
	G_3	2 × 3560	=	7120
				15050 kg per lin. meter.

max load on pier.



max dead Load say

$20500 \times \frac{32.72}{2} = 336.000$

" $\times$ 7.86 = 161.000

$20500 \times \frac{24.85}{2} = 254.000$

~~354.000~~ 751.000 kg.

7.86 × 15050 = 118500

12.42 × 15050 = 187.000

max live load

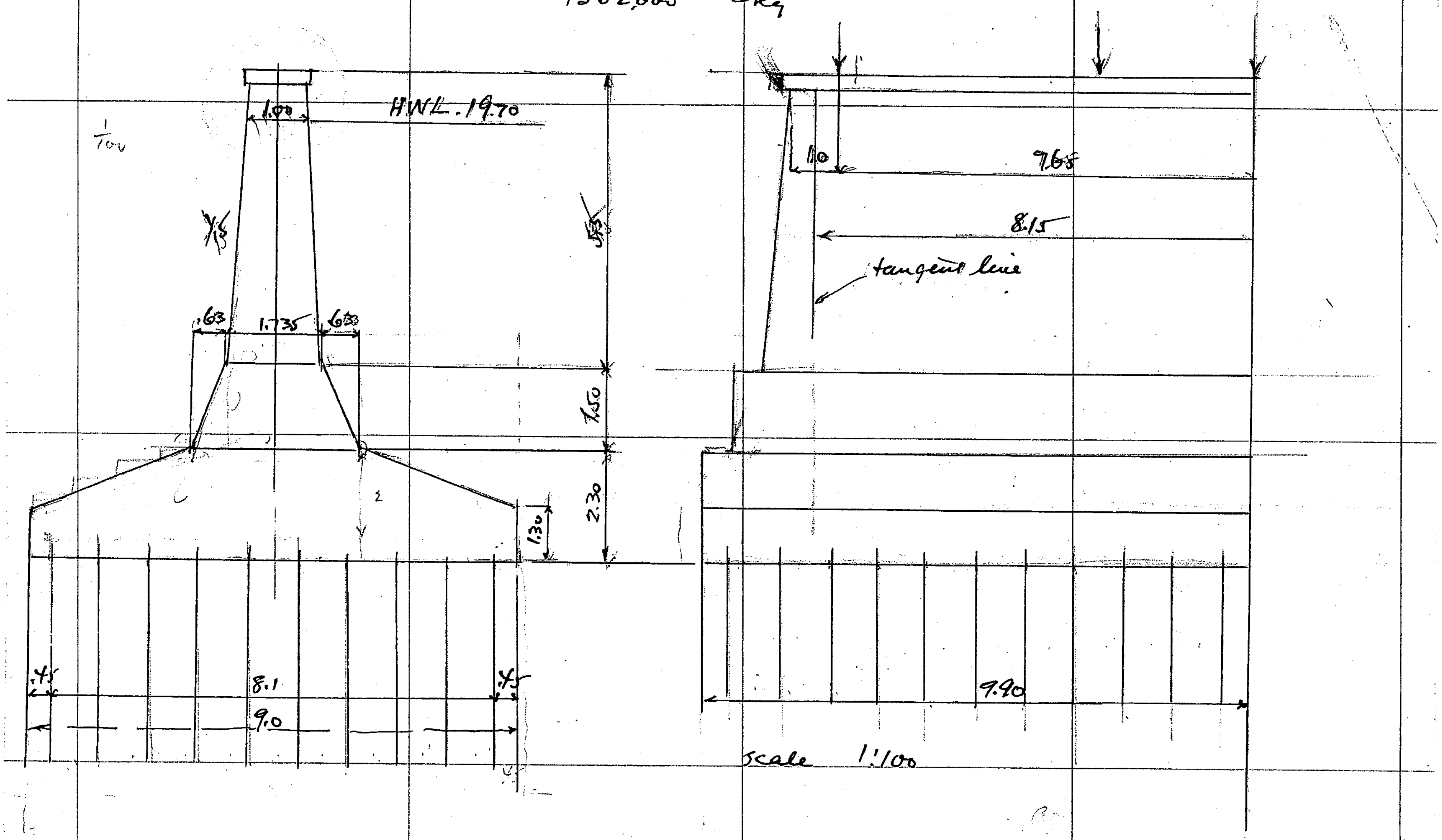
$15050 \times \frac{32.72}{2} = 246.000$

$118.500 \times \frac{36.65}{32.72} = 133.000$

$187000 \times \frac{40.58}{32.72} = 232.000$

611.000 kg

Dead Load 751000
Live Load 611000
1362,000 kg



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CALCULATIONS FOR

Preliminary Estimate of Cost Juso-Bashi for Osaka-fu.

volume of Concrete

top area = 1⁰ - .78
1.00 × 16.30   16.30
17.08 — 17.08
1.735² = 2.36
1.735 × 16.30 = 28.30
30.66 — 30.66
47.74 ÷ 2 = 23.87

vol = 23.87 × 5.5 = 131.00 @ 2400 = 314,000

~~top area~~ average: 3000, 1.735, 4.735, 2.31 × 19.3 = 45.7

volume = 45.7 × 1.5 = 68.5 @ 2400 = 164,000

Base

3.0
9.0
12.0 ÷ 2 = 6.0
1.3 × 9 = 11.7
17.7

vol = 17.7 × 19.8 = 350 @ 2400 = 840,000

131
68.5
350
549.5

31.4
16.4
84.0
131.8 tons

weight = 549.5 @ 2400 = 1320.0 tons
Superimposed load 1362.0
2682. tons

Base area 9 × 19.8 = 178.0   2682 / 178 = 15,100 tons/□m
1.4 tons per sq ft.
1.4 × 9 = 12.6 tons per pile.

Bearing Pressure during Earthquake

Superimposed Dead Load	751,000 × 0.3	= 225,000	× 9.50	=	2,120,000
pier	314,000	= 94,000	× 6.55	=	615,000
	164,000	49,000	× 3.05	=	150,000
	840,000	252,000	× 1.10	=	277,000
	2,069,000		Ecc = 1.53		3,162,000

Soil Pressure = 2069,000 / 178. (1 ± 6 × 1.53 / 9) = 23600 or 0.
11650, 1.02
2.2 tons/ft²

2.2 × 9 = 19.8 tons per pile.

average dia of pile say .8 × 3.14 = 2.51   2.51 @ 300# = 750#

length of pile = 12.6 × 2200 / 750 = 37.0

approximate area of forms.

Base	60 × 2.3	=	138.
Shaft.	41.4 × 7.0	=	290.
			428

Reinforcing bars.

moment	225,000 ×	7.20	=	1,620,000
	94,000 ×	4.25	=	400,000
	49000 ×	.75	=	36,000
	1,229,000	1.67		2,056,000 kg-meter.

fibre stress = 1,229,000 / 3 × 19 (1 ± 6 × 1.67 / 3) = 93,800 C or 58500 T
2160, 334
9.38 kg/cm²   5.05 kg/cm²   71.5#/□"
505 / 2161

m = 2,056,000 / say 18 = 114,000 kg m per lin meter.   114,000 × 1/505

depth say 2.5   stress = 114,000 / 2.5 = 45,600 kg / 2060 = 22.2 □ cm per lin. meter

19mm area = 2.83   8 cm spacing.

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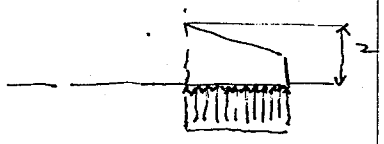
CALCULATIONS FOR

Preliminary Estimate of Cost Jiuso-Bashi for Osaka-fu.

Approximate weight 360 - 19 mm @ 2.22 kg. × 9.0 = 7200
~~Reinf. in base say~~ 7000
~~14200~~ kg.

#



15.100 × $\frac{3^2}{2}$ = 68.000 kg m

steel area req'd = $\frac{68000}{2.0 \times 7/8 \times 1200}$ = 32.4

22 mm bars - 3801 cm .400 @ 2.98 × 9.0 = 10.700
133 @ 2.22 × 9.0 = 2700
13.400
7.200
20 8 00 call this 22.0 tons.

割栗 2φ × 10 × 0.5 = 105 cubic meters.
Excavation 21 × 10 × 5.0 = 1050 cubic meter
No of piles. 220 本

Approximate cost of one pier for girder span

Concrete	550	cubic meters	@	17.43 =	9600
Reinf. bars.	22	tons.	@	130.00 =	2860
forms.	428	sq meters	@	2.00 =	850
割栗	105	cubic meters	@	4.00 =	420
Excavation	1050	cubic meters	@	2.00 =	2100
piles.	220		@	55.00 =	12100
					27930
		misc say			2070
					30000

Abutment say 30.000
abutments for canal span 40.000

piers	10 @ 30.000	=	300.000 円
abut	2 @ 30.000	=	60.000
"	2 @ 40.000	=	80.000
			440.000
	6 @ 100.000	=	600.000
			1,040.000

Super structure - 1711.000 円
Deck const. 330.000

2041.000
1040.000
3081000 円

Track construction 70.000
3151000

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CALCULATIONS FOR

Preliminary Estimate of Cost Juusō-Bashi for Osaka-Fu.

Design of River Pier.

Dead Load. floor and pavement and others $1390 \times \frac{65}{2} = 452.000$

Live Load. $2 - 500 \times 2.75 = 2750$

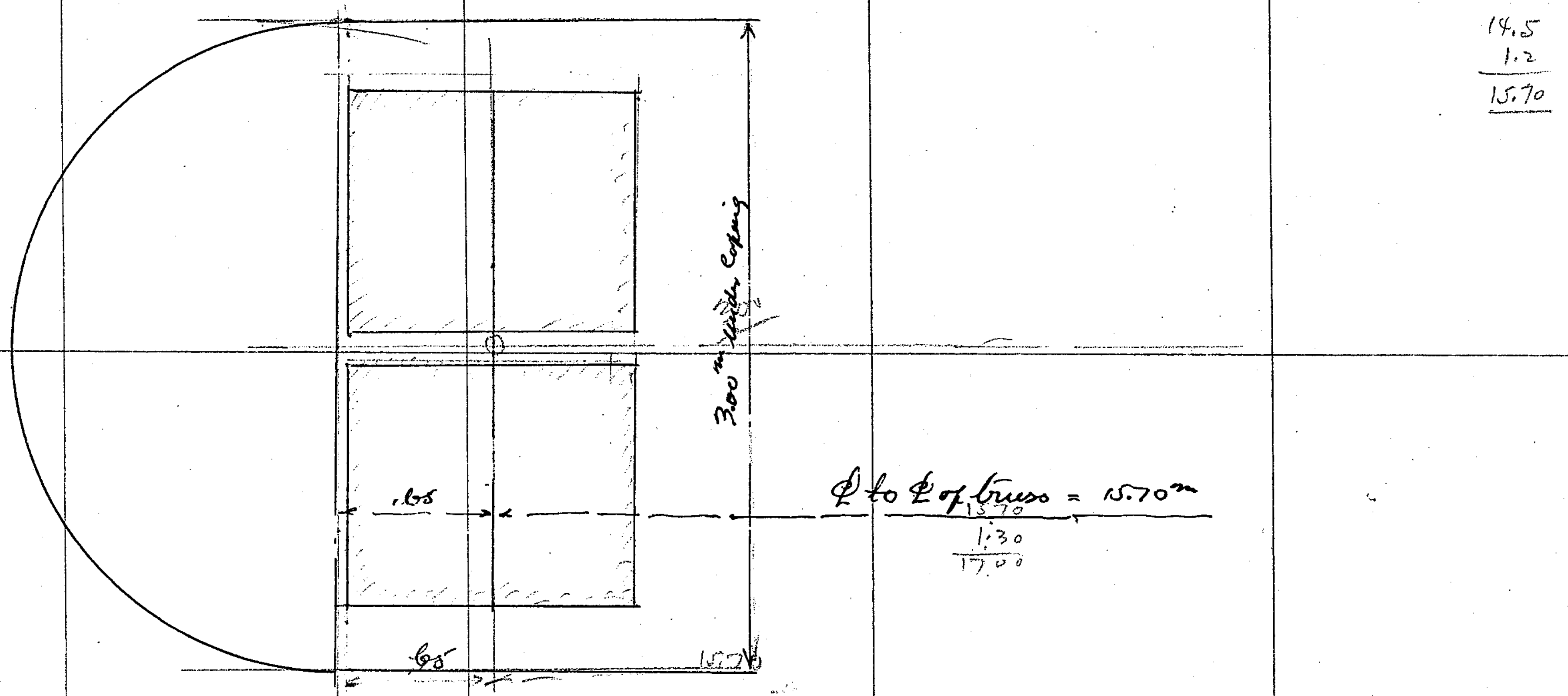
$600 \times 14.5 = 8700$

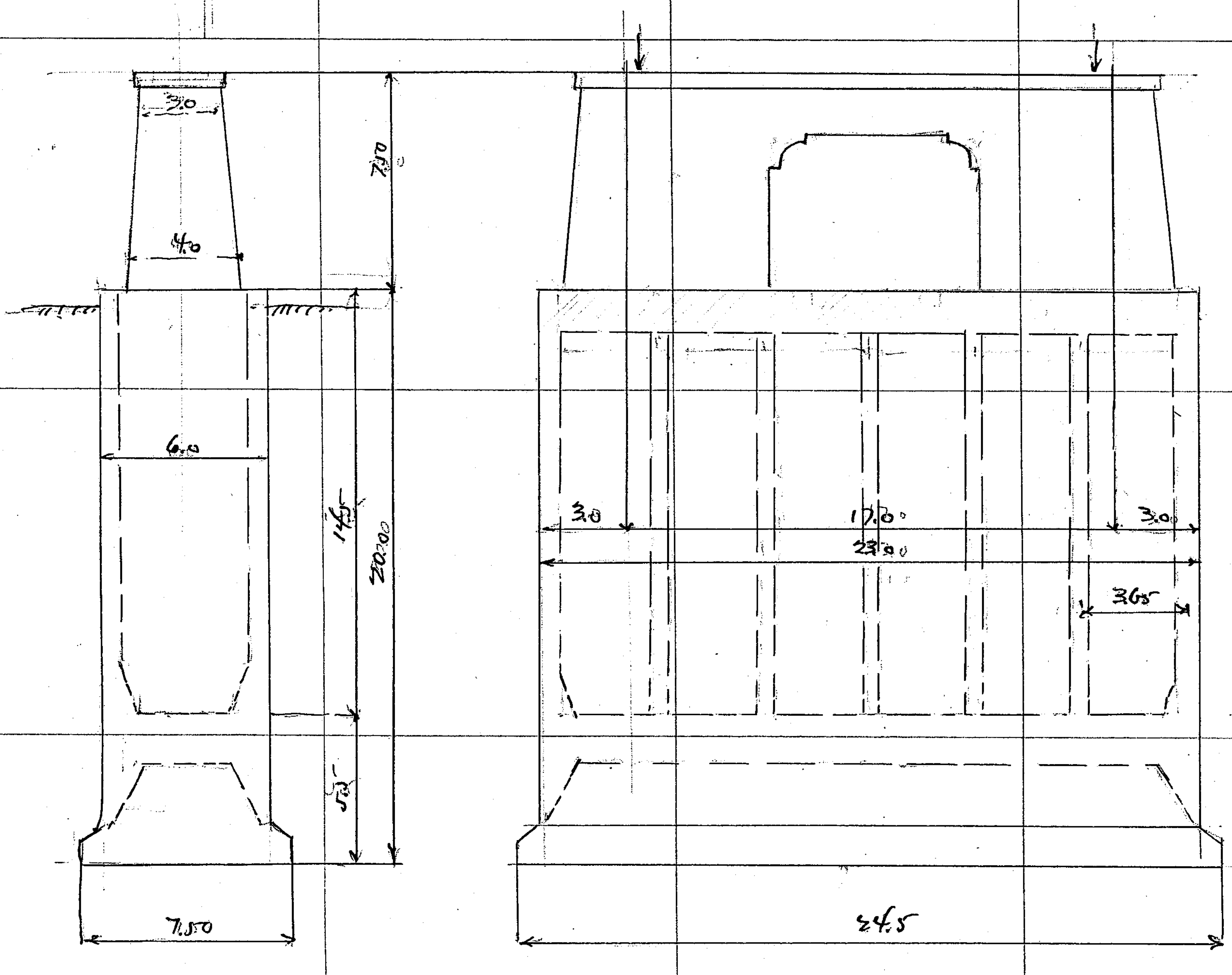
$11450 \div 2 = 5725 \times \frac{65}{2} = 186.000$

638.000 kg.

load on pier. $4 \times 638.000 = 2550.000$ kg

area for bearing. ~~452~~ $\frac{638.000.0}{\frac{30}{45}} = \frac{1420.00}{21.200}$ sq cm 1.19 1.20×1.20.





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CALCULATIONS FOR

Preliminary Estimate of Cost Jiuso-Bashi for Osaka-fu.

Shaft. Top area - 3□ - 7.09
3 × 17 = 51.
58.09

58.1
80.5
138.6 ÷ 2 = 69.3 -

Bottom area 4□ - 12.5
4 × 17 = 68.0
80.5

vol = 69.3 × 7.5 = 520.
less 7 × 6 × 2 = 84
436 cubic meters.

area of forms. (8 + 40) × 7.5 = 360 sq meters.
Reinforcing bars. 15.0 tons.

approx. cost 436 @ 17.43 = 7600
360 @ 2.00 = 720
20 ton @ 135 = 2700
11020 .00

Caisson.
Concrete. outside walls. 2 - 1.0 × 6.0 = 12.0
" " 2 - 1.0 × 21.0 = 42.0
Partitions 5 - 0.6 × 4.0 = 12.0
filler. 12 × 0.09 = 1.1
67.1 sq meters

23.0 × 6 = 138
67.1
70.9

volume 67.1 × 14.5 = 970.0 cubic meters.

working chamber - with fill. 23 × 6 = 138 @ 5.5 = 760
projection. 1.2 × 60 = 72
832 cubic meters

Top filling. concrete - 70.9 × 1.5 = 107. cubic meters

Sand filling - - - - 70.9 × 13.0 = 920 cubic meters

Total vol of concrete shaft 436 @ 2400 = 1050.000
Caisson 970
832
107
1909 @ 2400 = 4580.000
Sand fill. 920 @ 1700 = 1565.000
7195.000
Superimposed dead & live loads 2550.000
9745.000 tons.

Base area = 7.5 × 24.5 = 184.0

Unit bearing = $\frac{9745000}{184}$ = 53.000 kg/m²
4.92 tons/ft²

Surface friction - 250#/□' 1400 kg per sq meter perimeter. 12
1400 × 58 = 81.000 kg. 46
58.0

18 meter @ 81. = 1455 tons

9745
- 1455
8290.000 kg ÷ 184 = 45.000 kg/m²
4.19 tons/ft²

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CALCULATIONS FOR

Preliminary Estimate of Cost Juso-Bashi for Osaka-fu.

Superimposed Dead load	1810.000 × 0.3	= 542.000	× 28.0	=	15,200.000
shaft	1050.000 ×	= 315.000	× 23.7	=	7460.000
well -	6145.000 ×	= 1840.000	× 10.0	=	18400.000
		2697.000			41,060.000

Horizontal reaction = 2697.000
moment = 2,697.000 × 12.0 meter — 32300.000
8,760.000

frictional couple. 1400 × 18 × 23. = 580.000 kg.
Couple = 580.000 × 6 = — 3,480.000
5,280.000

$$\Sigma ce = \frac{5280.000}{8290.000} = .638$$

$$\text{Unit bearing} = \frac{8290.000}{184}\left(1 \pm \frac{6 \times .638}{7.5}\right) = 68.000 \ \text{kg/m}^2$$

(450.00 above 8290; .51 above fraction)

6.31 tons/ft²

Reinforcing steel 60.0 tons.
forms. ~~2~~840 square meters.
curb shoe 20.0 tons.

Estimate of Cost
(Caisson only).

Concrete	1909	@ 17.43	=	33.300
Reinf. bars.	60 tons	@ 135.00	=	8100
forms.	2840	@ ~~1.70~~ 1.50	=	4250
curb shoe	20 tons	@ 240.00	=	4800

50450
2000

浮島費

Sinking 25000
77400.00

shaft 11020
88470.00

call this 90.000

Substructure	10 @ 30.000	=	300.000
abut	2 @ 30.000	=	60.000
"	2 @ 40.000	=	80.000
			440.000
	6 @ 90.000	=	540.000
			980.000

steel - 1,711.000
Deck - 330.000

2041.000
980.000
3,021.000
4% 120.000
3,141.000

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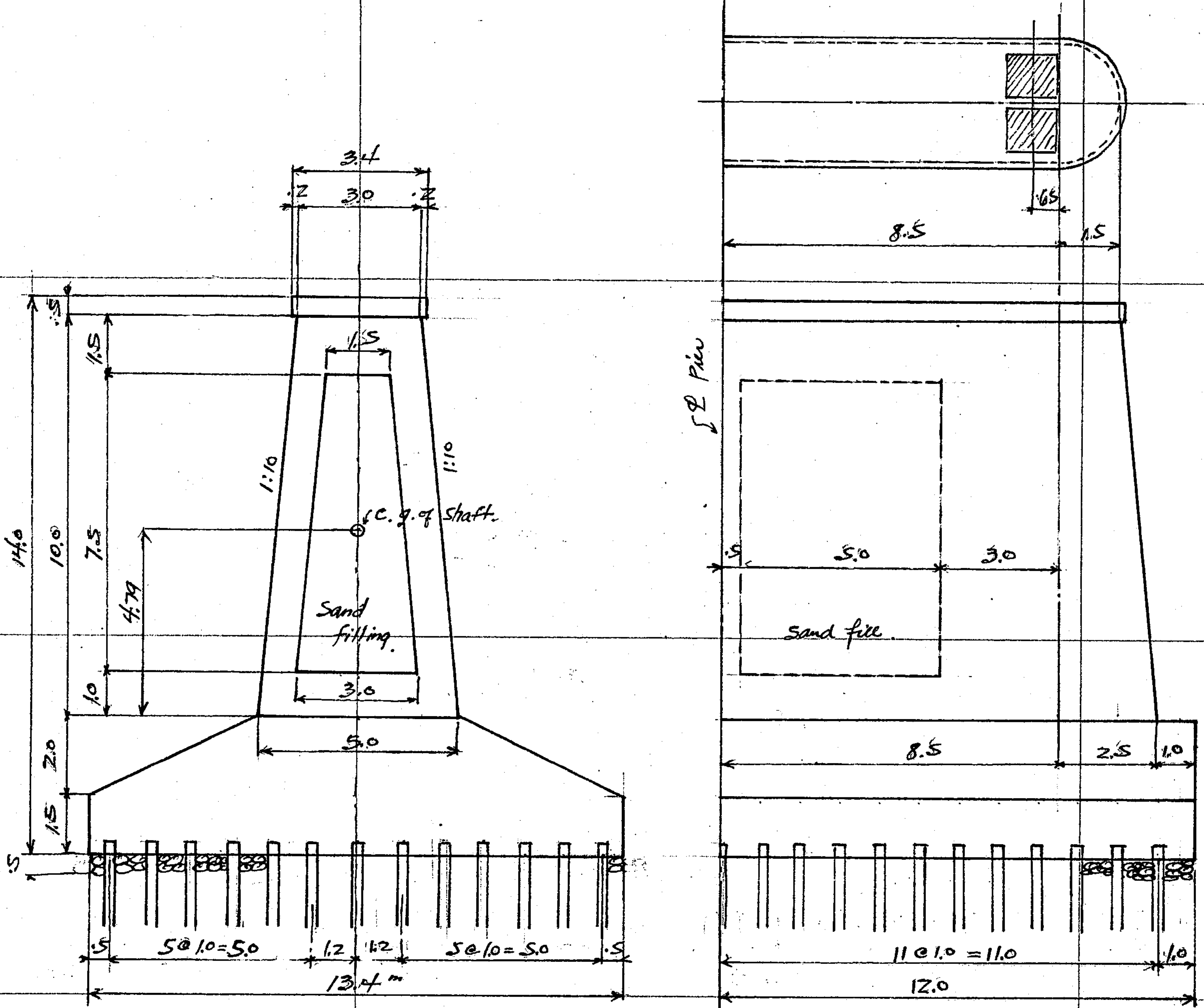
CALCULATIONS FOR

Preliminary Estimate of cost. Jiuso Bashi for Osaka Fu.

Design of Pier for Arch span



Volume of Concrete and Center of gravity of Shaft.

Coping	3.4 × 0.5 × 17.0 =	28.90	@ 2,400 =	69,400 kg	× 10.26 =	712,000
〃 Circular ends.	3.4φ × 0.5 =	4.54	@ 〃 =	10,900	× 10.26 =	112,000
Shaft.	4.0 × 10.0 × 17.0 =	680.00	@ 〃 =	1,632,700	× 4.59 =	7,500,000
〃 Circular ends.	4.0φ × 10.0 =	125.66	@ 〃 =	301,500	× 4.20 =	1,265,000
Hollow less.	2.25 × 7.5 × 5 × 2 = (−)	168.70	@ 〃 =	(−)404,500	× 4.34 = (−)	1,755,000
		670.40 m³		1,610,000 kg		
Sand filling		168.70 m³	@ 1,700 =	287,000 〃	× 4.34 =	1,246,000
				1,897,000 kg	× 4.79 m =	9,080,000

Volume of Concrete and Center of gravity of Base.

upper layer	9.2 × 2 × 24 =	441.5	@ 2200 =	971,000	× 2.35 =	2,280,000
lower 〃	1.5 × 13.4 × 24 =	482.0	@ 2200 =	1,060,000	× .75 =	795,000
	Base	923.5 m³		2,031,000 kg	× 1.51 =	3,075,000
	Shaft	670.4				
	Total concrete for one pier =	1,593.9 m³ (265立坪)				

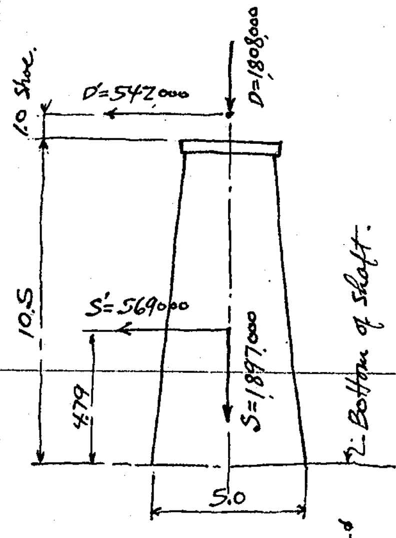
Super Imposed Loads on Pier.

Dead Load	457,000 × 4 =	1,808,000
Live Load	186,000 × 4 =	744,000
	638,000 kg	2,552,000 kg on one pier

(3)

Preliminary Estimate of cost. Jiu-so Bashi for Osaka Fu

Stability of Pier Shaft during earthquake, K assumed 0.3.



D.L. of Super Structure	D= 1,808,000		×	0	=	0
its Seismic force	D'= ~~542000~~	542,000	×	11.5	=	6,230,000
Weight of Shaft	S = 1,897,000		×	0	=	0
its Seismic force	S' =	569,000	×	4.79	=	2,730,000
	3,705,000 kg	1,111,000 kg				8,960,000 kgm.

Eccentricity $\varepsilon = \dfrac{8,960,000}{3,705,000} = 2.42^{m}$

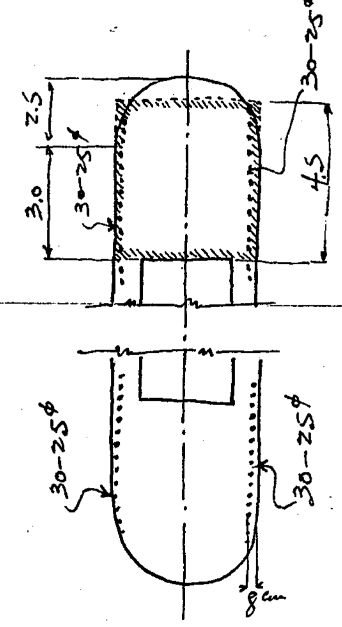
Use 30 – 25mmφ bars on both sides for one shaft $= 147.5^{cm^2}$

Steel ratio $= p_0 = 2p = \dfrac{147.5\times4}{500\times450\times2} = 0.00131$

$d'/h = 8/500 = 0.016$, $\varepsilon/h = 2.42/5.0 = .484$

From the prepared diagrams of combined stresses, we obtain as follows:

$k = .35$, $L = .078$



$f_c = \dfrac{M}{Lbh^2} = \dfrac{8960,000\times100}{.078\times900\times500^2} = 50.1^{kg}/_{cm^2} < 35\times1.8 = 63^{kg}/_{cm^2}$ ok

$f_s = n f_c\left(\dfrac{d}{kh}-1\right) = 15\times50.1\left(\dfrac{492}{.35\times500}-1\right) = 1,360^{kg}/_{cm^2} < 1200\times1.8 = 2160$ ok.

Unit shear $= \dfrac{1,111,000}{900\times\frac{7}{8}\times492} = 2.87^{kg}/_{cm^2}$ ok.

Unit bond $= \dfrac{1,111,000}{7.854\times60\times\frac{7}{8}\times498} = 5.48^{kg}/_{cm^2}$ ok.

Assumed section of shaft is ample.

Stability of shaft at normal state.

Super imposed load	=	2,552,000	D.L+L.L.
wt. of Shaft	=	1,897,000	
		4,449,000 kg	on one pier with no eccentricity assumed.

Bearing area assumed $2\times450\times500 = 450,000^{cm^2}$

Unit compression on concrete $= \dfrac{4,449,000}{450,000} = 9.9^{kg}/_{cm^2}$ ok.

Stability of Pier as a whole.

Case 1. Stability at normal state.

Super imposed D.L + L.L	=	2,552,000
wt. of Shaft	=	1,897,000
wt. of base	=	2,031,000
		6,480,000 kg on one pier with no eccentricity assumed

Bearing pressure on foundation $= \dfrac{6,480,000}{13.4\times12.0\times2} = \underline{20160}^{kg}/_{m^2}$ or $(1.84^{tons}/_{\square'})$ ok

max. load on one pile $= \dfrac{6,480,000}{299 \text{ piles}} = 21,700^{kg}$ or $\underline{21.7}$ kg tons on one pile.

Skin friction on pile $= \dfrac{21,700}{31\times3.14\times1500} = .148^{kg}/_{cm^2}$ or $1,480^{kg}/_{m^2} = 303^{\#}/_{\square'}$

Case 2. Stability during Earthquake, k = 0.3

				lev. arm		moment at center of base.
Super imposed Dead load	1808,000		×	0	=	0
its seismic force		542,000	×	15.0	=	8,130,000
wt. of shaft	1,897,000		×	0	=	0
its seismic force		569,000	×	8.29	=	4,720,000
wt. of base	2,031,000		×	0	=	0
its. seismic force		610,000	×	1.51	=	920,000
	5,736,000 kg	1,721,000 kg				13,770,000 kgm

Eccentricity $\varepsilon = 13,770,000 \div 5,736,000 = 2.40^{m}$

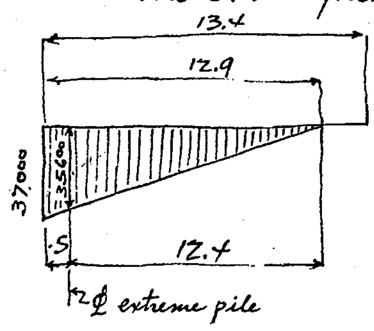
Resultant force out of middle third.

(3)

Preliminary Estimate of Cost. Jinso Bashi for Osaka Fu.

Neglecting tension at heel. Pressure area = $\left(\frac{13.4}{2}-2.4\right)\times 3 = 12.9\times 24 = 310$ sq.m

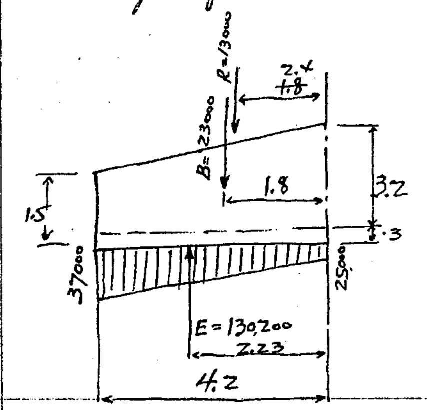
Max. toe pressure = $\frac{5.736000\times 2}{310} = \underline{37000}$ kg/m² or (3.38 tons/□') during earthquake.



Max. load on one pile = $35600\times 1\times 1 = \underline{35.6}$ kg.tons during earthquake. Ok.

Skin friction on pile = $\frac{35600}{31\times 3.142\times 1500} = .244$ kg/cm² or 2440 kg/m² = $\underline{499}$ #/□' of pile surface.

Design of Cantilever Footing.



During Earthquake.

upward pressure. $\frac{(37000+25000)}{2}\times 4.2 = 130200$ kg per lin meter of base.

down ward pres. footing $\frac{3.5+1.5}{2}\times 4.2 = 10.5 @ 2200 = 23000$ kg "

" rubble $\frac{3.0+1.0}{2}\times 4.2 = 8.2 @ 1600 = 13000$ " "

moment at fixed pt.

R = −13000 × 2.4 = − 31200

B = − 23000 × 1.8 = − 41400

E = 130200 × 2.23 = + 290600

218000 kgm per lin meter of base.

Effective depth req'd. = $\sqrt{\frac{218000\times 100}{100\times 7.18}} = \sqrt{30400} = 175$ cm

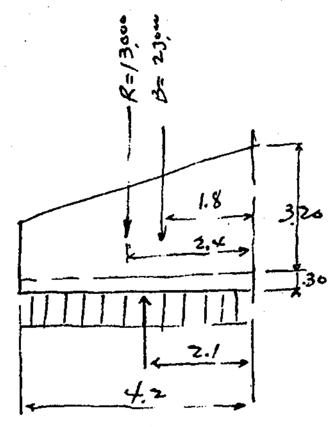
Use 320 cm. effective depth.

Steel area req'd = $\frac{218000\times 100}{1200\times 1.8\times \frac{7}{8}\times 320} = 36.$ cm²

Use 8−25φ = 39.2 cm² per lin meter of base.

Unit Shear = $\frac{94200}{100\times \frac{7}{8}\times 320} = 3.35$ kg/cm² ok

~~Unit bond = $\frac{94200}{8\times 7.854\times \frac{7}{8}\times 320} = 5.35$ kg/cm² ok.~~



At normal state.

upward pressure	=	20160 × 4.2 = 84800	× 2.1	=	178000
wt. of footing	=	= −23000	× 1.8	= −	41400
wt. of rubble		= −13000	× 2.4	= −	31200
		48800 kg			105400 kgm/m of base.

218000 ÷ 1.8 = 121000 kgm. > 105400

above detail is ample.

(4)

Preliminary Estimate of Cost. Jinso Bashi for Osaka Fu.

Estimate of Cost for Pier.

Concrete 1:2:4	1594 m^3	@	17.43	27,800
Reinforcements, plain.	30 tons	@	135.00	4,050
Forms.	800 m^2	@	2.00	1,600
Sand filling	170 m^2	@	4.00	680
piles	299 ft.	@	55.00	16,450
Rubble. base 下部捨石	560 m^2	@	4.00	2240
Excavation	1610 "	@	2.00	3220
Sheet piles ラルセン ガニ型 12 m	75 m	@	250.00	18,750
筆假締切.	110 m	@	550.–	5,500
				80,290 円

Estimate of Cost of bridge (pile foundation throughout)

Substructure.	Girder piers	10 @ 30.000	= 300.000	
	abutments	2 @ 30.000	= 60.000	
	"	2 @ 40.000	= 80.000	
				440.000
	truss piers say	6 @ 80.000	=	480.000
				920 000 円

Steel say	1711.000	
Deck	330.000	
		2041 000
Substructure		920.000
		2961 000
Add. misc exp. 4%		120.000
		3081 000 円

CALCULATIONS FOR

Preliminary Estimate of Cost Jiuso-Bashi for Osaka-fu

Pneumatic Caisson Pier. Depth 27.5 meters
Superimposed load Dead and live load 638.000
Shaft same as p19
Caisson 27.5 meter width say 7.0 meter.

23 × 7 = 161
72.1
88.9

Concrete

Outside walls.	2. - 1.0 × 7.0	=	14.0
" "	2 - 1.0 × 21.0	=	42.0
Partition walls.	5 - 0.6 × 5.0	=	15.0
filler.			1.1
			72.1 sq meter

Volume = 72.1 × 22.0 = 1590 cubic meters

working chamber with fill 23 × 7 = 161 @ 5.5 = 885.
Projection say 1.5 × 60 = 90
975 cubic meter

Top filling Concrete 88.9 × 1.5 = 133.0 cubic meters
Sand filling 88.9 × 20.5 = 1820.0 cubic meters

Total volume of concrete.
shaft 436 @ 2400 = 1050.000
Caisson 1590
975
133
2698 @ 2400 = 6470.000
Sand fill 1820 @ 1700 = 3100.000
10620 000
Superimposed load 2550.000
13,170,000 kg.

Base area = 9.0 × 25.0 = 225.
1,317,000 ÷ 225 = 58500 kg/m2
5.45 tons / ft2

Surface friction. 200#/□' 1400 kg per sq meter.
60 × 1400 = 84000 kg
25.5 × 84 = 2140.

14
46
60

1,3170.000
2140.000
1,1030,000 kg.

Unit bearing. 11030.000 ÷ 225 = 49000 kg/m2
4.56 tons/ft2

Using water fill. shaft 1050.000
caisson 6470.000
water fill 1820.000
9340.000
Superimposed load 2550.000
11,890,000

unit bearing = 11,890.000 ÷ 225 = 52800 kg/m2
4.90 ~~tons~~ /ft2

Counting friction of caisson
Surface friction - 350#/□' 1710 kg per sq meters
60 × 1710 = 106.000 kg per lin meter.
25.5 × 106 = 2700. tons
11.890.
2700
9190.000 ÷ 225 = 40800 kg per sq meter
3.8 tons per sq ft.

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CALCULATIONS FOR

Preliminary Estimate of Cost Juso - Bashi for Osaka-fu.

Bearing during Earthquake

Superimposed Dead Load	1810.000 × 0.3	= 542.000	× 36.00	=	19,500.000
shaft	1050.000 ×	= 315.000	× 31.20	=	9820.000
well	8290.000 ×	= 2480.000	× 13.75	=	34100.000
Horizontal force		3337.000			63420,000
moment	3.337.000 × 17.0				− 56700.000
					6720,000

frictional couple 1710 × 23 × 25.5 = 1,000,000 kg.
Couple 1,000.000 × 7 − 7.000.000
− 280.000

Call this 00 6.720.000 / 3500.000 / 3220.000

Unit bearing same as ordinary case
Not counting ½ frictional couple

$Σee = \frac{\cancel{6720.000}\ 3220.000}{11.150.000} = .289$

$\text{Unit bearing} = \frac{11.150.000\ (4960)}{225}\left(1 \pm \frac{6 \times .289}{9}\right) = 59300$ kg per sq meter
55 tons per sq ft

Reinforcing steel. 83 tons.
forms 4450 sq meters
Curb shoes &c 60 tons.
working and material shafts. 20.0 tons.

Estimate of cost
(Caisson only)

Concrete	2698	@	17.43	=	47000
Reinf. bars.	83 tons	@	135.00	=	11200
forms	4450	@	1.50	=	6680
Curb shoes	60 tons	@	220.00	=	13200
shafts	20 tons	@	240.00	=	4800
					82880

学雑費 2000
84880

Sinking 30000

Caisson Complete 114880
Shaft Complete 11020
125900.00

Call this 126000.00

Substructure	10 @ 30.000	=	300.000
abut	2 @ 30.000	=	60.000
"	2 @ 40.000	=	80.000
			440.000
River piers	6 @ 126000	=	756.000
			1196.000

6 .. 120,000
2020.000
3220.000

steel 1711000
Deck 330.000

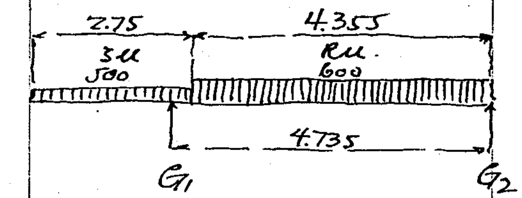
1886000
1200.000
3.086.000

2041.000
3,237,000
4% misc expense. 130.000
3,367,000

~~with revised cost of Deck const.~~
steel 1711.000
Deck 247.000
1,958,000 — 1,958.000
substructure 1196.000
3,154,000
4% misc exp. 126.000
3,280,000

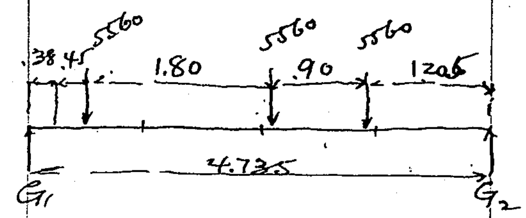
Design of Jiuso-Bashi for Osaka-Prefecture

Live Load 500 kg per sq meter on sidewalk
600 " " " on Roadway
motor truck rear wheel concentration 4500
impact $\frac{20}{60+25.23} = 23.5\%$ 1060
5560 kg.

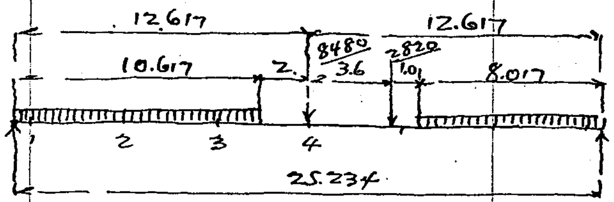


front wheel with impact $\frac{5560}{3} = 1853$ kg.

Uniform load on sidewalk. $2.75 \times 500 = 1375$
load on G1 $1375 \times \frac{5.73}{4.735} = 1665$ kg per meter
Uniform load on Roadway $4.355 \times 600 = 2615$
load on G1 $2615 \times \frac{2.178}{4.735} = 1200$ kg per meter



motor truck loading rear wheel $5560 \times \frac{7.215}{4.735} = 8480$ kg.
front wheel $\frac{1}{3} \times 8480 = 2820$ kg.
11300



moment at center of span
motor truck loading R. $8480 \times \frac{1}{2} = 4240$
$2820 \times \frac{9.017}{25.234} = 1050$
5290

moment $= 5290 \times 12.617 = 66800$ kgm

$10.617 \times 1200 = 12750$ kg
$8.017 \times 1200 = 9620$ "

Uniform live load R.L.
Reaction $12750 \times \frac{19.925}{25.234} = 10080$ ($\cancel{7.308}$)
$9620 \times \frac{4.009}{25.234} = 1525$
11605

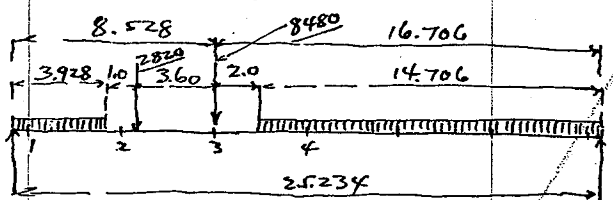
moment $= 11605 \times 12.617 = 146300$
$12750 \times 7.308 = 93300$
53000 kgm.

Summary for Live Load moment at Center

motor truck	66800
R.L.	53000
S.L.	132600
	252400 kgm

Uniform live load S.L. $\frac{1}{8} \times 1665 \times 25.234^2 = 132600$ kgm

Moment at panel point (3)



motor truck loading Reaction $8480 \times \frac{8.528}{25.234} = 2870$
$2820 \times \frac{4.928}{25.234} = 550$
3420
11300
3420
7880

moment $= 3420 \times 16.706 = 57100$ kgm

$1200 \times 3.928 = 4710$
$1200 \times 14.706 = 17650$

Uniform live load R.L. Reaction $4710 \times \frac{1.964}{25.234} = 366$
$17650 \times \frac{17.881}{25.234} = 12500$
12866

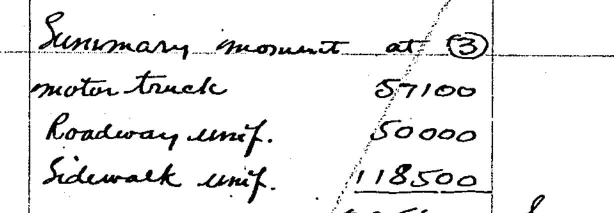
moment $12866 \times 16.706 = 215000$
$17650 \times 9.353 = -165000$
50.000 kgm

Summary moment at (3)

motor truck	57100
Roadway unif.	50000
Sidewalk unif.	118500
	225600 kgm

Unif. load S.L. 1665 kg.
moment $\frac{1665}{2} \times 8.528 \times 16.706 = 118500$ kgm

Moment at panel point (2)



motor truck loading Reaction $8480 \times \frac{4.439}{25.234} = 1490$
$2820 \times \frac{.839}{25.234} = 97$
1587 kg.

moment $= 1587 \times 20.795 = 33000$ kgm

Uniform live load R.L. Reaction $22600 \times \frac{9.40}{25.234} = 8400$

$1200 \times 18.795 = 22600$ kg.

moment $= 8400 \times 4.439 = 37300$ kgm

Uniform live load S.L. 1665 kg/m
moment $= \frac{1665}{2} \times 4.439 \times 20.795 = 76800$ kgm

Summary moment at (2)

motor truck	33000
Roadway Unif.	37300
Sidewalk Unif.	76800
	147100 kgm

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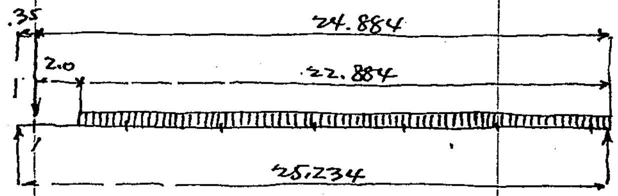
CALCULATIONS FOR

Design of Jūsō-Bashi for Osaka-Prefecture.

Moment at ①



motor truck loading Reaction $8480 \cdot \frac{24.884}{25.234} = 8350$

Moment $= 8350 \times 0.35 = 2920$ kgm

Uniform load R.U. $1200 \cdot \frac{22.884^2}{2 \times 25.234} =$ ~~13000~~ 12450

Moment $=$ ~~13000~~ 12450 $\times 0.35 =$ ~~4550~~ 4350 kgm

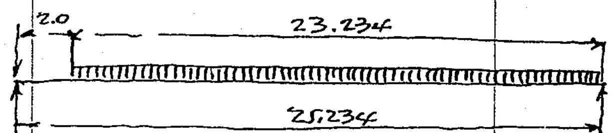
Unif. load S.U. 1665 kg/m

moment $= \frac{1665}{2} \cdot 0.35 \cdot 24.884 = 7260$ kgm.

Summary moment at ①

motor truck loading	2920
Unif. Roadway	~~4550~~ 4350
Unif. Sidewalk	7260
	~~14730~~ 14530 kgm

Live Load End shear



Unif. load	$1200 \cdot \frac{23.234^2}{2 \times 25.234}$	$= 12850$
Unif. load	$\frac{1665}{2} \cdot 25.234$	$= 21000$
Motor truck loading		8480
		42330 kg.

Summary for moments and shears

	④	③	②	①	End shear
Dead Load	394000	352000	228000	21400	62300
Live Load	252400	225600	147100	~~14730~~ 14530	42330
	646400 kgm	577600	375100	35930	104630 kg.

Max load on shoe girder span out to out $= 25.774$

Dead Load on shoe say $\frac{4930}{2} \cdot 25.774 = 63500$

Live Load say 43000

106500 kg on bearing.

main girder G_2 span length 25.234 meters

Dead Load.

From roadway side. Reaction page 13. 4204

Direct on girder $361 \cdot 4.089 = 1480$

beam say $120 \cdot \frac{4.735}{2} = 284$

conduit pipes + water main neglected $7898 \div$ ~~4.735~~ $= 1670 \times 3/4$ 5968

neg. reaction due to Cantilever moment $3/4 \cdot$ ~~2330~~ 1670 assumed $=$ ~~-1750~~ -1250

$4218 \div 4089 =$ ~~1030~~ 1150 kg.

4718

From Car Track side Dead Load slab only $1187 \cdot 2.895 = 3430$

Int. cross beam 316

Cross beam at panel 427

$743 \div 4.089 = 181$

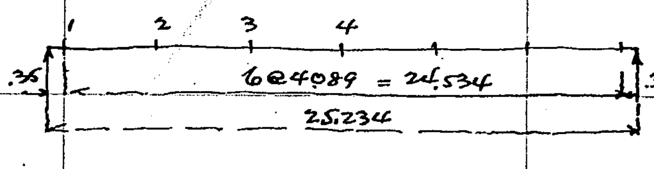
Lateral Bracing between G_2-G_3 46

$3657 \div 2 = 1829$

Lateral Bracing between G_1-G_2 say 46

main girder assumed 1000

~~3905~~ 4025 kg.



moment at 4 $\frac{1}{8} \cdot$ ~~3905~~ $4025 \cdot 25.234^2 =$ ~~124.000~~ 320.000 kgm

3 $\frac{\cancel{3905}\ 4025}{2} \cdot 8.528 \cdot 16.706 =$ ~~278.000~~ 287.000 "

2 $\frac{\cancel{3905}\ 4025}{2} \cdot 4.439 \cdot 20.795 =$ ~~180.000~~ 186.000 "

1 $\frac{\cancel{3905}\ 4025}{2} \cdot 0.35 \cdot 24.884 =$ ~~17000~~ 17500 "

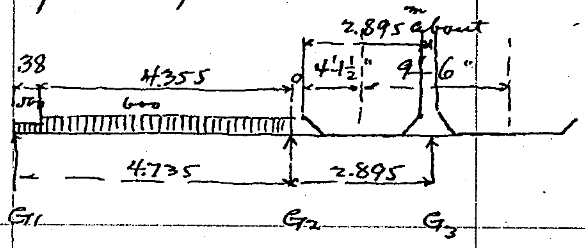
End shear $= \frac{\cancel{3905}\ 4025}{2} \cdot 25.234 =$ ~~49200~~ 50700 kg.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Design of Jiusō-Bashi for Osaka-Prefecture.

Live Load Roadway side 500 kg per sq meter on sidewalk
600 " " " " " Roadway
motor truck loading Rear wheel with impact 5560 kg
front wheel with impact 1855 kg.

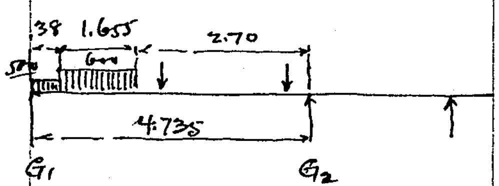
Full uniform load.



Reaction on G_2 $\frac{500 \cdot .38^2}{2 \cdot 4.735}$ = 8

$600 \cdot \frac{4.355 \cdot 2.556}{4.735}$ = 1410

1418 kg per meter



Reaction on G_2 side walk load = 8

$600 \cdot \frac{1.655 \cdot 1.202}{4.735}$ = 252

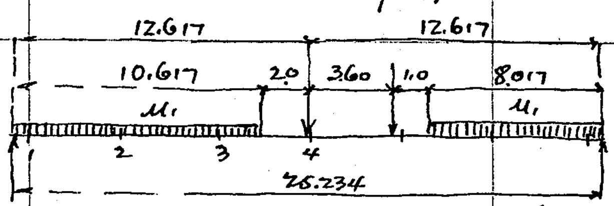
260 kg.

full unif. load 260 kg per meter M_2
at rear or front 1418 − 260 = 1158 kg. M_1

motor truck loading rear wheel with impact $2 \cdot 5560 \cdot \frac{3.385}{4.735}$ = 7950 kg.

front wheel $\frac{1}{3} \cdot 7950$ = 2650 kg.

moment at center of span



motor truck loading Reaction 7950 ÷ 2 = 3975

$2650 \cdot \frac{9.017}{25.234}$ = 945

4920

moment = 4920 · 12.617 = 62000 kgm

1158 · 10.617 = 12350 kg

1158 · 8.017 = 9290 kg.

Uniform load M_1

Reaction 12350 · $\frac{19.925}{25.234}$ = 9700

9290 · $\frac{4.009}{25.234}$ = 1472

11172

Summary moment at center

motor truck	62000
Uniform M_1	51000
Uniform M_2	20800
	133800 kgm

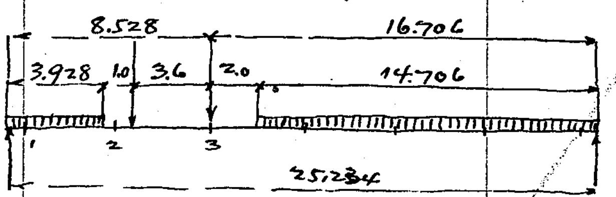
moment = 11172 · 12.617 = 141000

12350 · 7.308 = 90000

51000 kgm

Unif. load M_2 $\frac{1}{8} \cdot 260 \cdot 25.234^2$ = 20800 "

moment at panel point (3)



motor truck loading Reaction 7950 · $\frac{8.528}{25.234}$ = 2690

2650 · $\frac{4.928}{25.234}$ = 517

3207

moment = 3207 · 16.706 = 53500 kgm

Uniform live load M_1 1158 kg/m

1158 · 3.928 = 4550 kg

1158 · 14.706 = 17000

Reaction 4550 · $\frac{1.964}{25.234}$ = 354.0

17000 · $\frac{17.881}{25.234}$ = 12050

12404

Summary moment at (3)

motor truck	53500
Unif. M_1	48000
Unif. M_2	18500
	120000 kgm

Moment 12404 · 16.706 = 207000

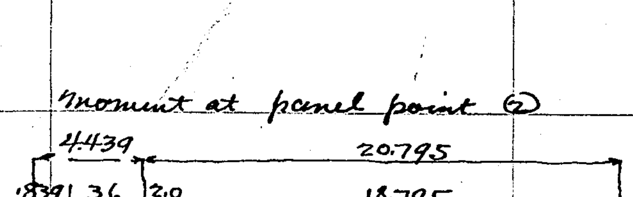
17000 · 9.353 = 159000

48000 kgm

Uniform live load M_2 260 kg/m

moment = $\frac{260}{2} \cdot 8.528 \cdot 16.706$ = 18500 kgm

moment at panel point (2)



motor truck loading Reaction 7950 · $\frac{4.439}{25.234}$ = 1400

2650 · $\frac{.839}{25.234}$ = 88

1488

moment = 1488 · 20.795 = 30900 kgm

M_1 1158 · 18.795 = 21700 kg.

Unif. load M_1 Reaction 21700 · $\frac{9.40}{25.234}$ = 8060 kg.

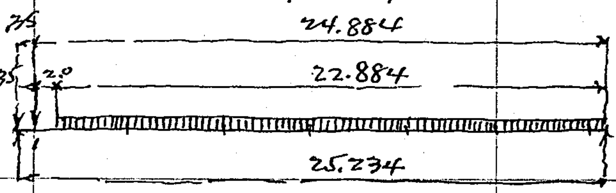
moment = 8060 · 4.439 = 35800 kgm

Design of Jiu-so-Bashi for Osaka-Prefecture

Summary moment at 2
motor truck 30900
Unif. load U_1 35800
Unif. load U_2 12000
78700 kgm

Uniform live load U_2 260 kg.
moment = $\frac{260}{2} \times 4.439 \times 20.795$ = 12000 kgm

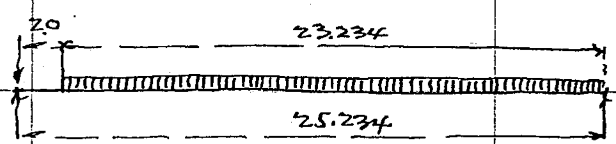
moment at panel point ①



motor truck loading Reaction $7950 \times \frac{24.884}{25.234}$ = 7840 kg.
moment = 7840×0.35 = 2740 kgm
Uniform load U_1 $1158 \times \frac{22.884^2}{2 \times 25.234}$ = 12000 kg.
moment = 12000×0.35 = 4200 kgm
Uniform load U_2 260 kg/m
moment = $\frac{260}{2} \times 0.35 \times 24.884$ = 1138 kgm

Summary moment at ①
motor truck 2740
Unif load U_1 4200
Unif. load U_2 1138
8078 kgm

Live Load End shear



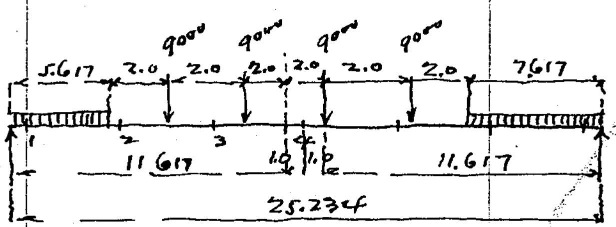
Uniform load U_1 $1158 \times \frac{23.234^2}{2 \times 25.1234}$ = 11450
" " U_2 $\frac{260}{2} \times 25.234$ = 3280
motor truck loading 7950
22680 kg.

max load on shoe Girder span 25.774 out to out
~~Dead Load say~~ Live load say 23.000 kg.

Live ~~Load~~ Electric Car Loading Class B water sprinkler coupled caused by Uniform live load at front and rear of car
Uniform live load 6.00×2.895 = 1740 kg. per meter.

impact for car loading 23.5 %

Absolute max moment near ℄ span



Electric Car loading Reaction $4 \times 9000 \times \frac{11.617}{25.234}$ = 16550
moment 16550×11.617 = 192.800
9000×2 = 18.000
174200 kgm

Uniform load.
1740×5.617 = 9760
1740×7.617 = 13250

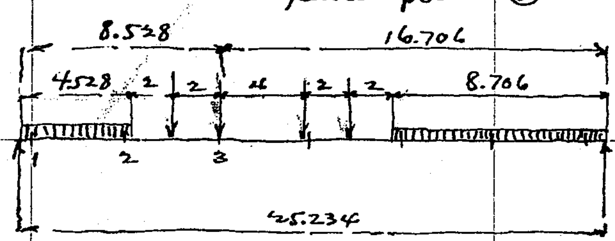
Reaction $9760 \times \frac{2.809}{25.234}$ = 1088
$13250 \times \frac{21.425}{25.234}$ = 11250
12338 kg.

moment = 12338×11.617 = 143200
13250×7.808 = -103500
39700 kgm

Summary moment max
Electric Car 174200
Uniform load 39700
impact 23.5% 40900
254800 ÷ 2 = 127400 kgm

174100
40900
215100

moment at panel point ③



Electric Car Loading.
Reaction = $4 \times 9000 \times \frac{14.706}{25.234}$ = 21000 kg.
moment = 21000×8.528 = 179000
9000×2.0 = 18.000
161,000 kgm
impact 23.5 % 37800
198800

Unif load 1740×4.528 = 7870
1740×8.706 = 15150
23020

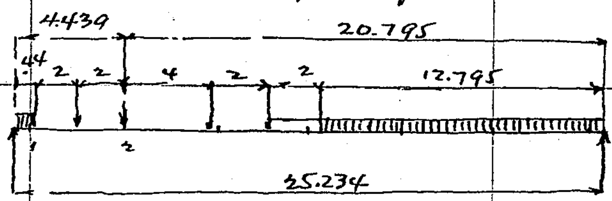
Design of Jiu-so-Bashi for Osaka-Prefecture

Summary moment
Electric Car 198800
Unif. load 34000
232800
232800 ÷ 2 = 116400 kgm

Uniform live load Reaction 7870 × 22.970/25.234 = 7150
15150 × 4.353/25.234 = 2620
9770

moment = 9770 × 8.528 = 83300
7870 × 6.264 = −49300
34000 kgm

Moment at panel point no 2



Electric Car loading 4 × 9000 × 18.795/25.234 = 26800

moment = 26800 × 4.439 = 119.000
9000 × 2. = 18.000
101.000
impact 23.5 % = 23700
124.700 kgm.

Unif. load 1740 × .44 = 765
1740 × 12.795 = 22260
23025

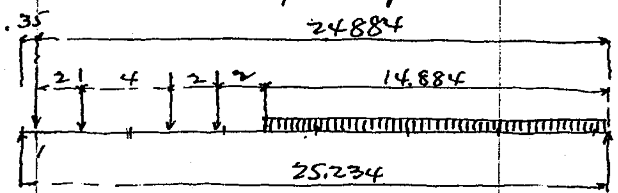
Reaction. 765 × 25.014/25.234 = 760
22260 × 6.40/25.234 = 5650
6410 kg.

Summary moment
Elec. Car 124.700
Unif. load 25220
149920

moment 6410 × 4.439 = 28450
765 × 4.22 = 3230
25220 kgm

149920 ÷ 2 = 74960 kgm

Moment at panel point no 1



Electric Car Loading Reaction 4 × 9000 × 20.884/25.234 = 29800

moment = 29800 × 0.35 = 10400
impact 23.5% 2440
12840 kgm

Uniform load Reaction = 1740 × 14.884²/(2 × 25.234) = 7620

Summary moment
Electric Car 12840
Unif. load 2670
15510 kgm ÷ 2 = 7755

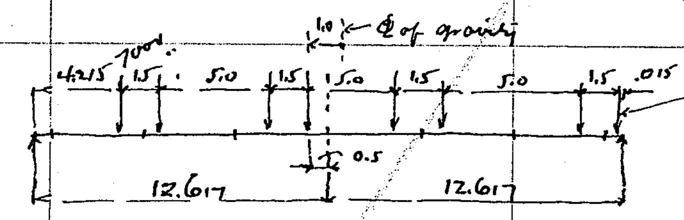
Moment = 7620 × 0.35 = 2670 kgm

max End shear

Electric car loading 4 × 9000 × 21.234/25.234 = 30300
Unif. load 1740 × 15.234²/(2 × 25.234) = 7130
8020
38320 kg.
impact 23.5% 45450 ÷ 2 = 22725 kg.

Live Load Electric Car Loading class D Bogie Car Continuous Total weight of one car 28.000 kg impact 23.5 %.

Absolute max moment near center.

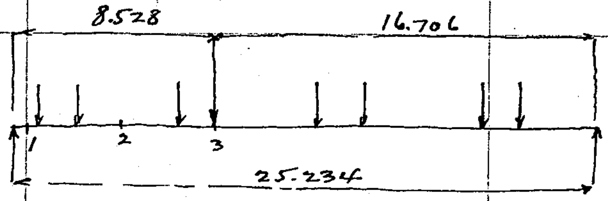


this load neglected.

Reaction 7 × 7000 × 12.117/25.234 = 23500

moment = 23500 × 12.117 = 285000
7000 × 16.0 = 112000
173 000
impact 23.5% 40600
213600 ÷ 2 = 106800 kgm

moment at panel point at 3



Reaction 8 × 7000 × 14.206/25.234 = 31500 kg.

moment = 31500 × 8.528 = 267.000
7000 × 16 = 112.000
145000
impact 23.5% 34100
179100 ÷ 2 = 89550 kgm

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CALCULATIONS FOR

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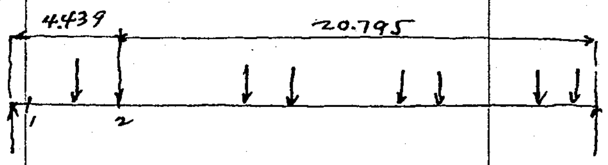
moment at panel point (2)



Reaction $8 \times 7000 \times \frac{11.795}{25.234} = 26200$

moment $26200 \times 4.439 = 116.000$

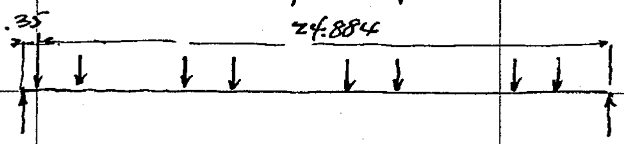
$7000 \times 1.5 = 10.500$

105500

impact 23.5% 24650

$130150 \div 2 = 65075$ kgm

moment at panel point (1)



Reaction $8 \times 7000 \times \frac{14.384}{25.234} = 31800$

moment $31800 \times .35 = 11100$

impact 23.5% 2610

$13710 \div 2 = 6855$ kgm

max End shear $8 \times 7000 \times \frac{14.734}{25.234} = 32700$

impact 23.5% 7700

$40400 \div 2 = 20200$ kg.

Summary for moment and shear

	moment at 4	3	2	1	End shear
Dead Load	320.000	287000	186000	17500	50700
Live Load Roadway	133800	120000	78700	8078	22680
L.L. Elec. Car Side	127400	116400	74960	7755	22725
	581200	523400	339660	33333	96005

max load on shoe Dead Load $\frac{4025}{2} \times 25.774 = 51900$

Live Load Highway side — say 23000.

" " Elec. Car side say 23000

97900 kg.

Design of main girder G3

Dead Load - See page 19 on G2 3657

main girder assumed 1000

4657 kg. per lin. meter.

moment at 4 $\frac{1}{8} \times 4657 \times 25.234^2 = 370.000$ kgm

3 $\frac{4657}{2} \times 8.528 \times 16.706 = 332.000$ "

2 $\frac{4657}{2} \times 4.439 \times 20.795 = 214.000$ "

1 $\frac{4657}{2} \times 0.35 \times 24.884 = 20200$ "

End shear $\frac{4657}{2} \times 25.234 = 58700$ kg

Live Load figured on page 21 and 22

Summary for moment and shear

	moment at 4	3	2	1	End shear
Dead Load	370.000	332000	214000	20200	58700
Live Load	254800	232800	149920	15510	45450
	624800	564800	363920	35710	104150

max load on shoe Dead Load $\frac{4657}{2} \times 25.774 = 60000$

Live Load say 46000

106000 kg.

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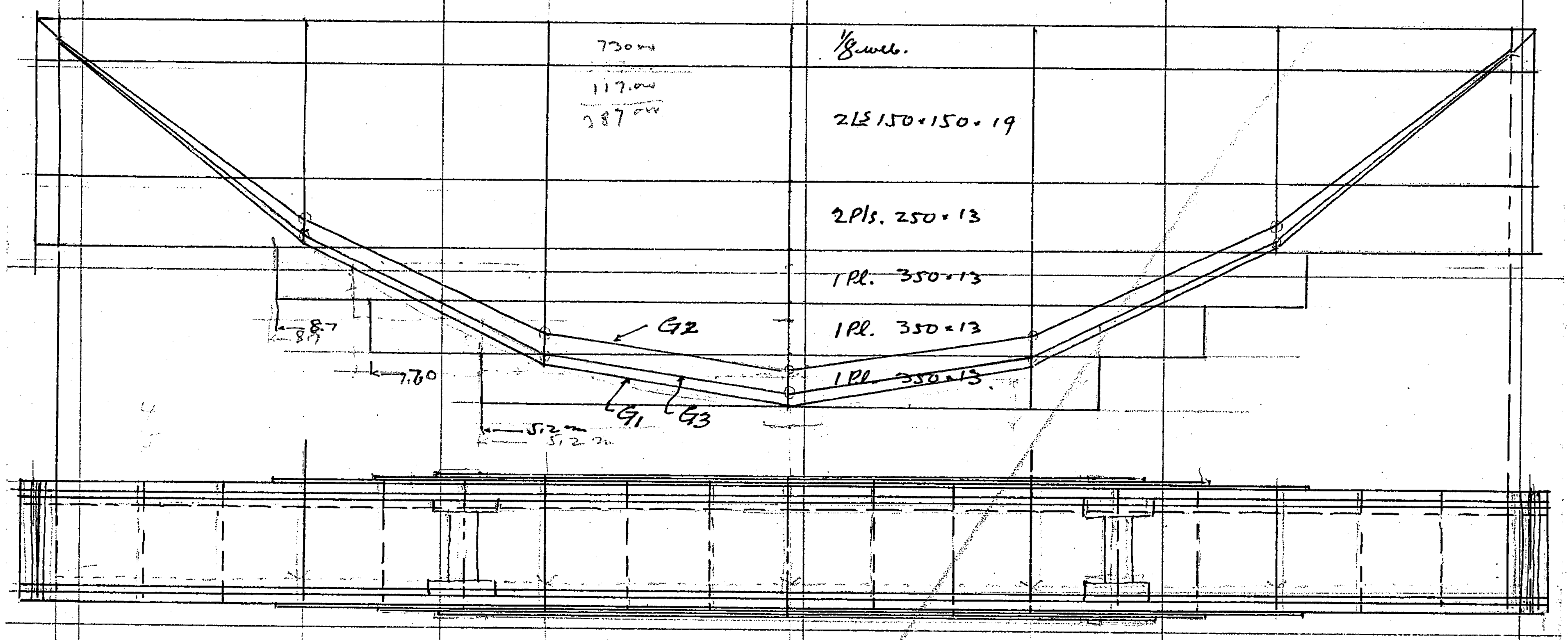
CALCULATIONS FOR

Design of Jiuso-Bashi for Osaka Prefecture.

Moment diagram for main girders G1, G2 and G3.



Section of main girder.
~~web assumed~~ 2000 × 13 = 260.0 □cm 1/8 web = 32.5 □cm
flanges assumed

2L 150×150×19	= 106.78	− 19.0	= 87.78	@ 4.36	= 465.
2Pls 250×13	= 65.00	− 13.0	= 52.00	@ 13.00	= 845
3Pls 350×13	= 136.50	− 19.5	= 117.00	@ 1.95	= −266
	308.28		256.78 □ net	339cm	1044

2 @ 3.39 = 6.80 cm

Effective depth 201.0 − 6.8 = 194.2 cm

G1 m = 646.400 (3cov. pls.) flange stress = $\frac{646400}{1.942}$ = 333000 SR = $\frac{333000}{1200}$ = 278.0
32.5
245.5 □cm net

G2 m = 581.200 (2cover pls.) flange stress = $\frac{581200}{\cancel{1.942}\ 1.92}$ = ~~299000~~ 302000 SR = $\frac{\cancel{299000}\ 302000}{1200}$ = ~~249.5~~ 252.0
32.5
~~217.0~~ 219.5 □cm net

G3 m = 624800 (3cov. pls.) flange stress = $\frac{624800}{1.942}$ = 321000 SR = $\frac{321000}{1200}$ = 267.5
32.5
235.0 □cm net

moment of inertia of section (gross).

4L	150×150×19	= 213.56	× 96.1²	+ 4392	= 1874.400
2×2 side Pls.	250×13	= 130.00	× 87.5²	+ 4060	= 1.000.000
2×3 cov pls.	350×13	= 273.00	× 102.5²		= 2870.000
1 web pl.	2000×13	= 260.00			866.000
		876.56			6,710.400 cm⁴

201

Unit stresses for main girders.

G1 $\frac{646.40000 \times 104.4}{6.710.400}$ = 1005 kg/cm²

G2 $\frac{581.200.00 \times \cancel{104.4}\ 103.1}{\cancel{6.710.400}\ 5.720.000}$ = ~~903~~ 1050 kg/cm²

G3 $\frac{62480000 \times 104.4}{6710400}$ = 970 kg/cm²

Approximate Deflection. DL LL

$\frac{5 \times 25.234^2 \times M}{48 \times 21.000.000 \times 6710400}$ = 3.04 = 1.85 + 1.19 $\frac{1}{830}$

$\frac{5}{48}$ do $\frac{25.234^2 \times M}{21.000.000 \times 5.720.000}$ = ~~2.74~~ 3.28 = ~~1.51 + 1.23~~ 1.76 + 1.46 $\frac{1}{\cancel{920}\ 780}$

Same as G1 do × M = 2.94 = 1.74 + 1.20 $\frac{1}{860}$

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Approximate weight of main Girders G_1-G_2 and G_3.

1 web.	1 Pl.	200 × 13	@ 204.10	× 25.77	=	5260	
flange	4 Ls	150×150×19	@ 41.91	× 25.77	=	4320	
side Pls.	4 Pls.	250 × 13	@ 25.51	× 25.77	=	2630	
Cover Pls.	2 Pls.	350 × 13	@ 35.72	× 17.40	=	1245	15197
"	2 Pls.	350 × 13	@ "	× 14.00	=	1000	
"	2 Pls.	350 × 13	@ "	× 10.40	=	742	
End stiffs	8 Ls	130×130×12	@ 23.36	× 2.00	=	374	
fills	4 Pls.	260 × 19	@ 38.78	× 1.70	=	264	
"	4 Pls.	540 × 13	@ 55.11	× 1.50	=	330	3547
Stiffs	38 Ls	125×90×10	@ 16.09	× 2.00	=	1225	
fills.	38 Pls.	90 × 13	@ 9.19	× 1.50	=	524	
shelf Ls	2 Ls	125 × 90×10	@ 16.09	× 25.8	=	830	
web splice	8 Pls.	230 × 13	@ 23.47	× 1.2	=	226	
"	4 Pls.	450 × 13	@ 45.92	× 1.0	=	184	11623
fills.	4 Pls.	90 × 19	@ 13.42	× 1.7	=	91	
side Pls.	8 Pls.	100 × 19	@ 14.92	× 1.0	=	119.5	
flange	8 Ls	150×150×19	@ 41.91	× 2.5	=	838	
	4 Pls.	350 × 15	@ 41.20	× 1.0	=	165	
Sole Pls.	2 Pls.	350 × 19	@ 52.2	× .54	=	56	1076
Rivit heads and variations			5%			1020	
						21443	kg.

$21443 \div 25.77 = 834$ kg per lin meter.

Summary for weight of structural steel in span on Canal (25.234 meter)

stringer	S_1	2@63.5	= 127		
	S_2	52 × 2	= 104		
	S_{3-4-5}	74 × 6	= 442		
			675 × 25.77	=	17400
Cross beams under tracks.		160 × 24		=	3840
Cross beam Cantilever etc at panel point		3200×7		=	22100 ✓
Lateral Bracings under Roadway		1117 × 6		=	6700
" " " Sidewalk					500
main girders		5@ 21440	=	less coupl etc	107200 (−2100)
shoes +					7000
expansion Joint etc say					2000
					164940 kg.

$164940 \div 25.77 = 6400$ kg per lin. meter.

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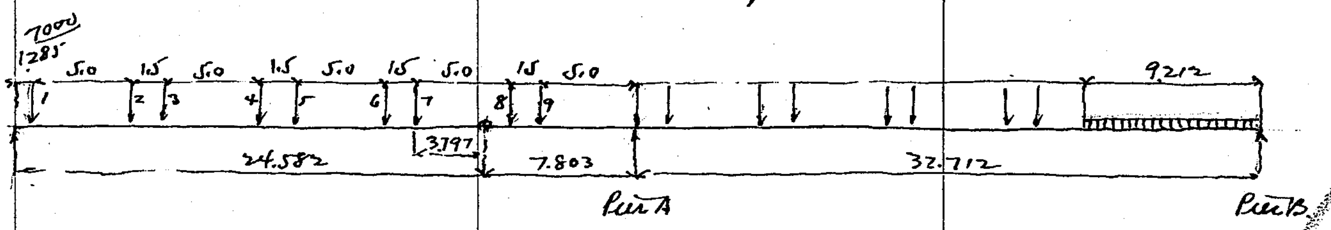
CALCULATIONS FOR

Design of Jiuso-Bashi for Osaka-Prefecture

max load on shoe Electric Car Loading



Load on Pier A. loading assumed as above.

Reaction at hinge 1-7 $7000 \times \frac{82.495}{24.582} = 23500$

Load on cantilever $2 \times 7000 = 14000$

37500

impact 21.6% 8100

45600 kg.

moment at Pier A. $23500 \times 7.803 = 183500$

$2 \times 7000 \times 5.75 = 80500$

264000

impact - 21.6% 57000

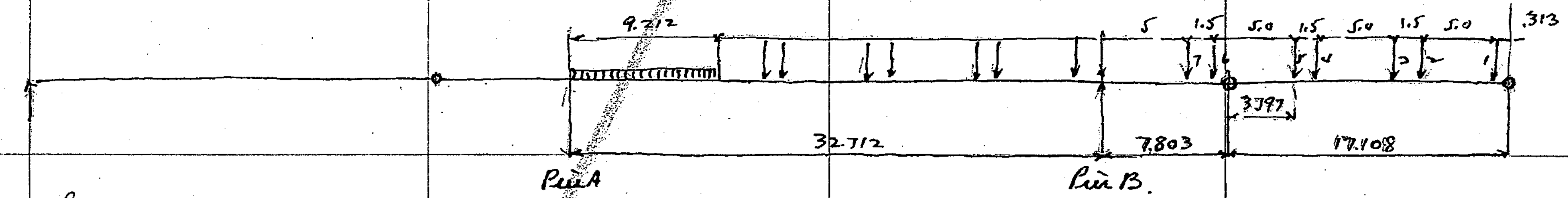
321000

Load on Pier A. Cantilever 45600

Due to moment $\frac{321000}{32.712} =$ 9820

Between piers see p. 52 48580

104000 $\div 2 =$ 52000 kg.



Load on pier B. Reaction at hinge 1-5 $7000 \times \frac{37.565}{17.108} = 15300$

2×7000 14000

29300

impact 21.6% assumed 6330

35630 kg.

moment at Pier B. $15300 \times 7.803 = 119200$

$2 \times 7000 \times 5.75 = 80500$

199700

impact 21.6% 43100

242800

Summary Load on pier B Cantilever 35630

Due to moment $\frac{242800}{32.712} =$ 7420

Between piers p52 48580

91630 $\div 2 =$ 45815 kg.

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CALCULATIONS FOR

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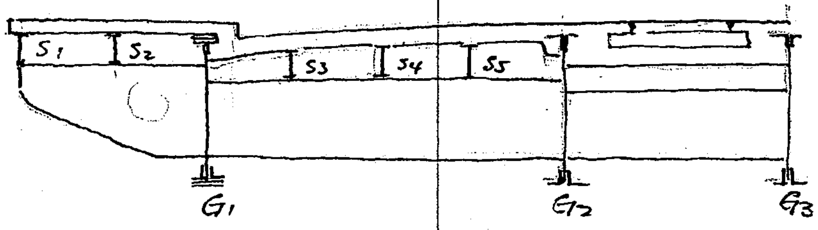


Design of stringer S1 span length 4.40 meter spacing 1.26 meters
See page 8

Dead Load moment = 1/8 × 521 × 4.40^2 = 1260
Live Load moment = 1/8 × 245 × 4.40^2 = 592
1852 kgm

Dead Load shear = 1/2 × 521 × 4.40 = 1190
Live Load shear = 1/2 × 245 × 4.40 = 540
1730 kg.

Depth of girder at center 35.5 cm back to back of angles
web = 350 × 8 = 280 □ cm 1/8 web = 3.50 cm^2 Effective depth 0.312 m

Section reqd = $\frac{1852}{0.312 \times 1200}$ = 4.95
3.50
1.45 □ cm net

Use 1L 75×75×9 = 12.69 cm^2 − 1.98 = 10.71 □ cm net
approximate weight of stringer S1 63.5 kg per lin. meter
or say 63.5 × 4.4 = 280 kg.

Stringer S2 span length 4.40 meters spacing 1.26 meters see page 9

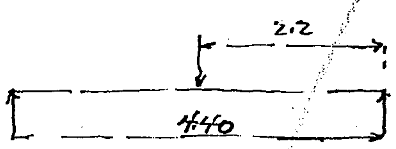
		moment		shear
Dead Load	1/8 × 430 × 4.4^2	= 1040	430 × $\frac{4.40}{2}$	= 945
Live Load	630	= 1525	630	= 1385
Conduit pipes etc	180	= 436	180	= 396
water main assumed	370	= 896	370	= 814
		3897 kgm		3540 kg

Section modulus reqd = $\frac{389700}{1100}$ = 354

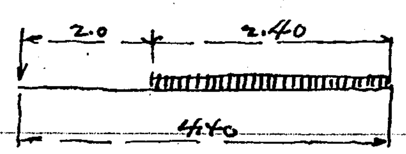
Use 300 × 150 I @ 48.34 kg Sm 633.2

Roadway stringers S3-4-5 span length 4.40 meters spacing 1.184 meters see page 10
Dead Load 700 kg per lin. meter
Dead Load moment = 1/8 × 700 × 4.40^2 = 1695 kgm
Dead Load shear = 1/2 × 700 × 4.40 = 1540 kg.

Live Load motor truck wheel concentration on stringer p10 7255 kg.
moment = $\frac{7255}{2}$ × 2.2 = 7980 kgm



max End Shear



Unif. load 600 × 1.184 = 710 kg.
Reaction $\frac{710 \times 2.40^2}{2 \times 4.40}$ = 465
motor truck loading 7255
7720 kg.

Summary for moments and shears

	moments	shears
Dead Load	1695	1540
Live Load	7980	7720
	9675 kgm	9260 kg.

Section modulus reqd = $\frac{967500}{1100}$ = 880.0

Use 14" × 6" I @ 68.5 kg Sm = 1030. about

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CALCULATIONS FOR

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Cross beam under Electric Ry Tracks span length 2.895 spacing 1.10 meters
Use 300 × 150 I @ 48.34 kg.

Cross beam at panel point under Electric Ry Tracks.
Use same section as for span over canal
Depth of girder 1.310 meters b to b of L^s web 1300 × 8
flange L^s $2L^s$ 90 × 75 × 9.
See page 11 for approximate weight etc of this cross beam.

Design of Cantilever spacing 4.40.

Dead Load page 12 Dead Load moment $10710 \times \frac{4.400}{4.089} = 11520$
Cantilever beam say 317
11837 kgm

End Shear $6135 \times \frac{4.400}{4.089} = 6600$
Cantilever beam say 252
6852 kg.

Live load. Proportional moment $5760 \times \frac{4.400}{4.089} = 6200$ kgm
shear $3570 \times \frac{4.400}{4.089} = 3840.$ kg.

Summary for moments and shears

	moments	shear
Dead Load	11837	6852
Live Load	6200	3840
	18037 kgm	10692 kg

Depth of Cantilever Bracket at connection – 1.395
web plate 138. × 0.8 = 110.4
Effective depth say 1.355

flange stress = $\frac{18037}{1.355} = 13320$

section reqd for tension $13320 \div 1200 = 11.10$ □cm net
" comp. $13320 \div 1000 = 13.32$ □cm gross

Section reqd for tension plate $\frac{18037 \; \cancel{13320}}{1.375 \times 1200} = 10.96$ □cm net

Stress say $\frac{18037}{1.375} = 13140$ kg.

approximate weight of Cantilever beam say 400 kg.

Design of Roadway Floor Beam max moment same as cantilever moment = 18037
Depth of girder 1.170 web assumed 116 × .8 = 92.7 □cm
Effective depth say 1.13 m
flange stress = $\frac{18037}{1.13} = 16000$
section reqd = $16000 \div 1200 = 13.30$ □cm

Use $2L^s$ 90 × 75 × 9 = 28.08 – 7.92 = 20.16 □cm net

approximate weight of floor beam say 692 kg see page 15

Summary for cross beams and Cantilever Brackets

Cantilever Brackets	2@400	=	800
Roadway Floor Beam	2@692	=	1384
Floor beam under track at panel point.	2@485	=	970
			3154 kg

call this 3200 kg.

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CALCULATIONS FOR

Design of Jiu-sō-Bashi for Osaka Prefecture.

Bottom Lateral Bracing — For horizontal force assume 6270 kg per meter see page 16.

Panel Concentration 6270 × 4.4 = 27600 kg.

Shear for 2nd panel 27600 × 1.5 = 41400 kg.

for one set 41400 ÷ 3 = 13800.

max diagonal stress = 13800 × 1.35 = 18600

Section reqd for tension $\frac{18600}{2160}$ = 8.60 Try 1L 130 × 130 × 9 = 22.59 − 4.5 = 18.09 cm²

no of rivets $\frac{18600}{5120}$ = 3.6.

Approximate weight of Bottom Lateral Bracing

4Ls 130 × 130 × 9 @ 17.73 × 6.00 = 425

4Ls " @ " × 3.25 = 230

connection pls and Ls say 480

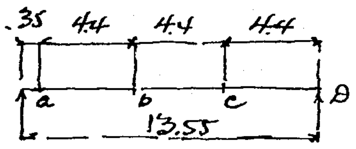
1135 kg.

Bracing under sidewalk stringers same as shown on page 17

approximate weight say 500 kg per span

Design of main girder DG1

Dead Load assumed same as CG1 page 37 4636 kg per lin. meter



moment at a $\frac{4636}{2}$ × .35 × 13.2 = 10700 kgm

moment at b $\frac{4636}{2}$ × 4.75 × 8.80 = ~~110000~~ 97000 "

moment at C " × 9.15 × 4.4 = 93500 "

moment at Center " × 6.78 × 6.78 = 106200 "

End reaction say $\frac{4636}{2}$ × 13.55 = 31400 kg.

Live Load 500 kg per square meter on sidewalk

600 " " " " roadway

motor truck rear wheel Concentration 4500

impact $\frac{20}{60 + 13.55}$ = 27.2% 1225

5725 kg.

front wheel concentration ⅓ × 5725 = 1910 "

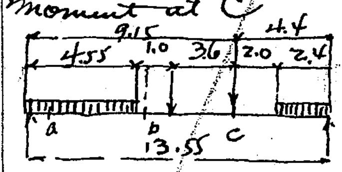
See page 18 Uniform load on sidewalk Reaction on girder 1665 kg

" " " Roadway " " " 1200 "

motor truck loading see page 18 5725 × $\frac{7.215}{4.735}$ = 8730 kg.

Front wheel ⅓ × 8730 = 2910 "

moment at C



motor truck loading Reaction 8730 × $\frac{9.15}{13.55}$ = 5890

2910 × $\frac{5.55}{13.55}$ = 1190

7080 kg.

1200 × 4.55 = 5460

1200 × 2.40 = 2880

moment = 7080 × 4.4 = 31200 kgm

Uniform load Rd. 5460 × $\frac{2.28}{13.55}$ = 920

2880 × $\frac{12.35}{13.55}$ = 2630

3550 kg.

Summary moment at C

motor truck 31200

Unif. Roadway 6380

" Sidewalk 33500

71080

moment = 3550 × 4.4 = 15600

2880 × 3.2 = −9220

6380 kgm

Uniform load Sw. 1665 kg per lin. meter

moment = $\frac{1665}{2}$ × 9.15 × 4.4 = 33500 kgm

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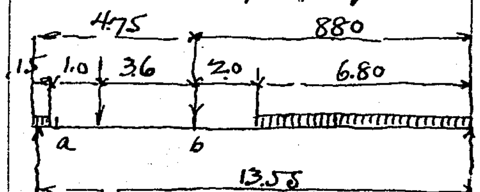
CALCULATIONS FOR

Design of Jiuso-Bashi for Osaka Prefecture.

moment at panel point b.



motor truck loading Reaction 8730 × 4.75/13.55 = 3060

2910 × 1.15/13.55 = 247

3307 kg.

moment = 3307 × 8.80 = 29000 kgm

Uniform load Roadway 180 × 0.075/13.55 = ~~10~~

1200 × .15 = 180

1200 × 6.80 = 8160

8160 × 10.15/13.55 = 6110

~~6120~~ 6110 kg.

Summary moment at b

moment = ~~6120~~ 6110 × 8.80 = ~~53900~~ 53700

~~6800~~ 8160 × 5.40 = ~~36700~~ 44100

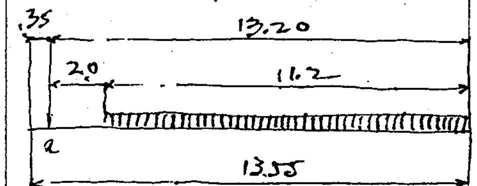
~~17200~~ 9600 kgm

motor truck	29000
Unif. Roadway	~~17200~~ 9600
Unif. Sidewalk	34800
	~~81000~~ ~~80800~~ 73400 kgm

Uniform load Sidewalk 1665 kg.

moment = 1665/2 × 4.75 × 8.80 = 34800 kgm

moment at panel point a



motor truck loading Reaction 8730 × 13.20/13.55 = 8510

moment = 8510 × .35 = 2980 kgm

Uniform load Roadway $m = \frac{1200 \times 11.2^2}{2 \times 13.55} \times 0.35 = 1940$

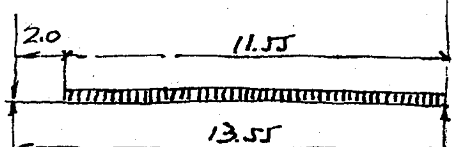
Summary moment at a

motor truck	2980
Unif. Roadway	1940
" Sidewalk	3850
	8770 kgm

Unif. load on Sidewalk 1665 kg.

moment = 1665/2 × 0.35 × 13.20 = 3850

Live Load End shear



Uniform Roadway 1200 × 11.55²/(2 × 13.55) = 5900

Uniform Sidewalk 1665 × 13.55/2 = 11300

motor truck loading 8730

25930 kg.

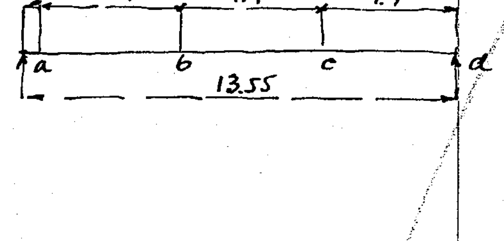
Summary for moments and shears

	a	b	c	shear	Load on shoe say
Dead Load	10700	97000	93500	31400	32000
Live load	8770	81000	71080	25930	26500
	19470	178000	164580	57330	58500 approx.

max load on shoe say 58500 kg.

Design of main girder DG2 span length 13.55 meters

Dead Load 3725 kg per lin. meter page 38



moment at a 3725/2 × 0.35 × 13.20 = 8600 kgm

b " × 4.75 × 8.80 = 77900 "

c " × 9.15 × 4.40 = 75000 "

moment at center 3725/2 × 6.78 × 6.78 = 85500 "

End Reaction = 3725/2 × 13.55 = 25200 kg.

Live Load Roadway side see page 20 full uniform load 260 kg. per meter U_2

~~at rear or front of motor truck~~ 1158 " " " U_1

motor truck reaction on girder 2 × 5735 × 3.385/4.735 = 8200 kg

front wheel 1/3 × 8200 = 2730 kg.

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(独立行政法人 土木研究所蔵)

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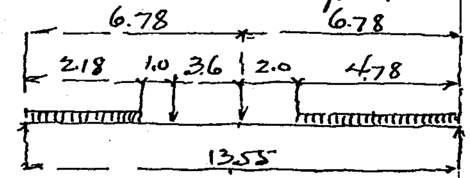
CALCULATIONS FOR

Design of Jiu-sō-Bashi for Osaka-Prefecture

moment at center of span



1158 · 2.18 = 2520
1158 · 4.78 = 5530

Summary moment at Center
motor truck ~~33000~~ 32200
Unif U_1 7900
Unif U_2 5960
46860 kgm

motor truck Reaction 8200 ÷ 2 = 4100
2930 · $\frac{3.18}{13.55}$ = 687
4787 kg.

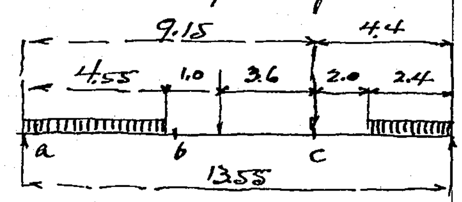
moment = 4787 · 6.78 = ~~33000~~ 32200 kgm

Uniform load U_1 Reaction 2520 · $\frac{1.09}{13.55}$ = 202
5530 · $\frac{11.16}{13.55}$ = 4550
4752

moment = 4752 · 6.78 = 32200
5530 · 4.39 = -24300
7900 kgm

Uniform load U_2 260 kg.
moment = 1/8 · 260 · 13.55² = 5960 kgm

moment at panel point C



motor truck Reaction 8200 · $\frac{9.15}{13.55}$ = 5530
2930 · $\frac{5.55}{13.55}$ = ~~1200~~ 1120
~~6730~~ 6650 kg.

moment = ~~6730~~ 6650 · 4.4 = ~~29600~~ 29300 kgm

Uniform load U_1 Reaction 5260 · $\frac{2.28}{13.55}$ = 886
2780 · $\frac{12.35}{13.55}$ = 2535
3421 kg.

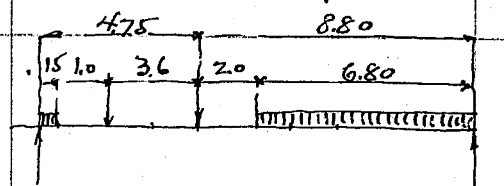
1158 · 4.55 = 5260
1158 · 2.40 = 2780

Summary moment at C
motor truck 29300
Unif. U_1 6200
Unif U_2 5230
~~41030~~ 40730 kgm

moment = 3421 · 4.4 = 15100
2780 · 3.2 = -8900
6200 kg.

Uniform load U_2 260 kg.
moment = $\frac{260}{2}$ · 9.15 · 4.4 = 5230 kg.

moment at Panel point b



motor truck loading Reaction 8200 · $\frac{4.75}{13.55}$ = 2875
2730 · $\frac{1.15}{13.55}$ = 232
3107

moment = 3107 · 8.80 = 27300 kgm

1158 · 0.15 = 174
1158 · 6.80 = 7870

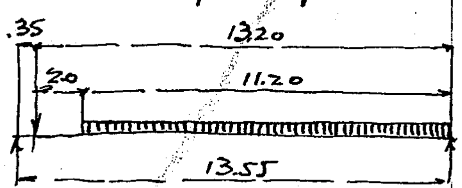
Uniform load U_1 Reaction 174 · $\frac{0.75}{13.55}$ = say 0
7870 · $\frac{10.15}{13.55}$ = 5900

Summary moment at b
motor truck 27300
Unif. U_1 9400
Unif. U_2 5430
42130 kg.

moment = 5900 · 8.80 = 51900
7870 · 5.40 = -42500
9400 kgm

Uniform load U_2 260 kg
moment = $\frac{260}{2}$ · 4.75 · 8.80 = 5430 kgm

moment at panel point a



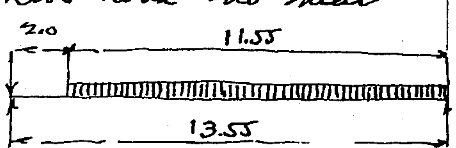
motor truck loading Reaction 8200 · $\frac{13.20}{13.55}$ = 7980
moment = 7980 · 0.35 = 2790 kgm

Uniform load U_1 m = $\frac{1158 \cdot 11.2^2}{2 \cdot 13.55}$ · 0.35 = 1880

Uniform load U_2 260 kg.
m = $\frac{260}{2}$ · 0.35 · 13.20 = 600

summary moment at a
motor truck 2790
Unif. U_1 1880
Unif U_2 600
5270 kgm

Live load End shear



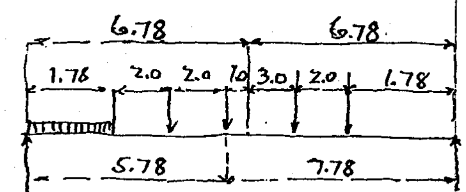
Uniform load U_1 1158 · $\frac{11.55^2}{2 \cdot 13.55}$ = 5700
" " U_2 260 · $\frac{13.55}{2}$ = 1760
motor truck 8200
15660 kg.

Design of Jiu-so-Bashi for Osaka-Prefecture.

Live Load Electric Car loading and uniform load between G_2 and G_3.
Class B Electric Car coupled covered by uniform live load at front and rear.

impact 27.2 %
Uniform live load 6.00 · 2.895 = 1740 kg per meter.

Absolute max moment — Electric car loading Reaction $4 \times 9000 \cdot \frac{5.78}{13.55} = 15380$ kg.



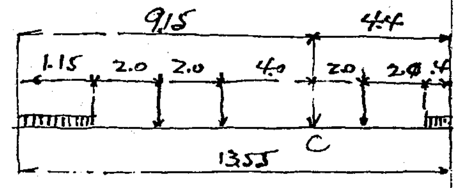
moment = 15380 · 5.78 = 88900
9000 · 2.0 = 18.000
70900
impact 27.2 %. 19300
90200 kgm

Summary moment
Electric Car 90200
Unif. load 1580
91780

91780 ÷ 2 = 45890 kgm

Uniform load $\frac{1740 \cdot 1.78^2}{2 \times 13.55} \cdot 7.78 = 1580$ kgm.

Moment at panel point c — Electric car loading Reaction $4 \times 9000 \cdot \frac{7.15}{13.55} = 19000$ kg



moment = 19000 · 4.4 = 83600
9000 · 2 = 18000
65600 kgm
impact 27.2 %. 17850
83450 kgm

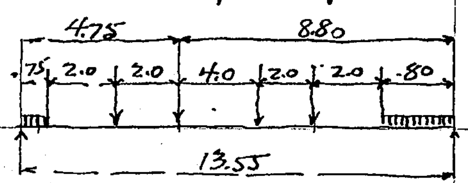
Summary moment at c
Electric Car 83450
Unif. ~~3790~~ 460
~~87240~~ 83910

~~87240~~ 83910 ÷ 2 = ~~43620~~ 41955 kgm

Uniform load
1740 × 1.15 = 2000 kg   c $\frac{0.57}{13.55}$ = ~~840~~ 84
1740 · 0.40 = 695 "   c $\frac{13.35}{13.55}$ = 685
~~1525~~ 769

moment = ~~1525~~ 769 · 4.4 = ~~6710~~ 3380
695 · 4.2 = −2920
~~3790~~ 460 kgm

Moment at panel point b. — Electric car loading $4 \times 9000 \cdot \frac{6.80}{13.55} = 18100$ kg.



moment = 18100 · 4.75 = 86000
9000 · 2.0 = 18000
68000
impact 27.2 %. 18500
86500 kgm

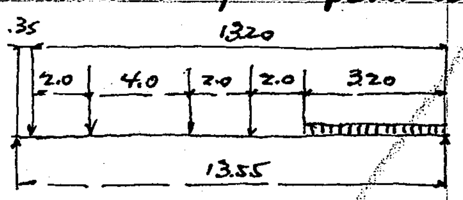
Summary moment at b
Electric Car 86500
Unif. 500
87000

87000 ÷ 2 = 43500 kgm

Unif. load
1740 · 0.75 = 1305 · $\frac{13.17}{13.55}$ = 1268
1740 · 0.80 = 1390 · $\frac{.40}{13.55}$ = 41
1309 kg.

moment 1309 · 4.75 = 6210
1305 · 4.375 = 5710
500

Moment at panel point a — Electric Car loading $4 \times 9000 \cdot \frac{9.20}{13.55} = 24450$



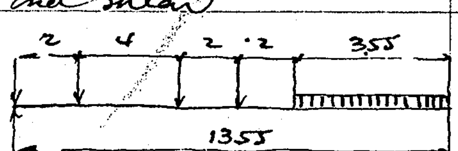
moment = 24450 · 0.35 = 8550 kgm
impact 27.2 % 2320
10870

Unif load $\frac{1740 \cdot 3.20^2}{2 \cdot 13.55} \cdot 0.35 = 230$ kgm

Summary moment at a
Electric Car 10870
Unif. 230
11100

11100 ÷ 2 = 5550 kgm.

End shear — Electric Car $4 \times 9000 \cdot \frac{9.55}{13.55} = 25400$



impact 27.2 % 6900

Uniform load $1740 \cdot \frac{3.55^2}{2 \times 13.55} = 808$
32108

say 32100 ÷ 2 = 16050

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CALCULATIONS FOR

Design of Jūsō-Bashi for Osaka-Prefecture

Summary for moments and shears DG2

	a	b	c	max	shear
		moment at.			
Dead Load	8600	77900	75000	85500	25200
Live Load Roadway side	5270	42130	40730	46060	15660
" " Car side.	5550	43500	41955	45890	16050
	19420	163530	157685	177450 kgm	56910 kg

max load on shoe say Dead Load 26200
Live Load 32800
59000 kg.

Design of main girder DG3.
Dead Load 4357 kg per lin. metre.

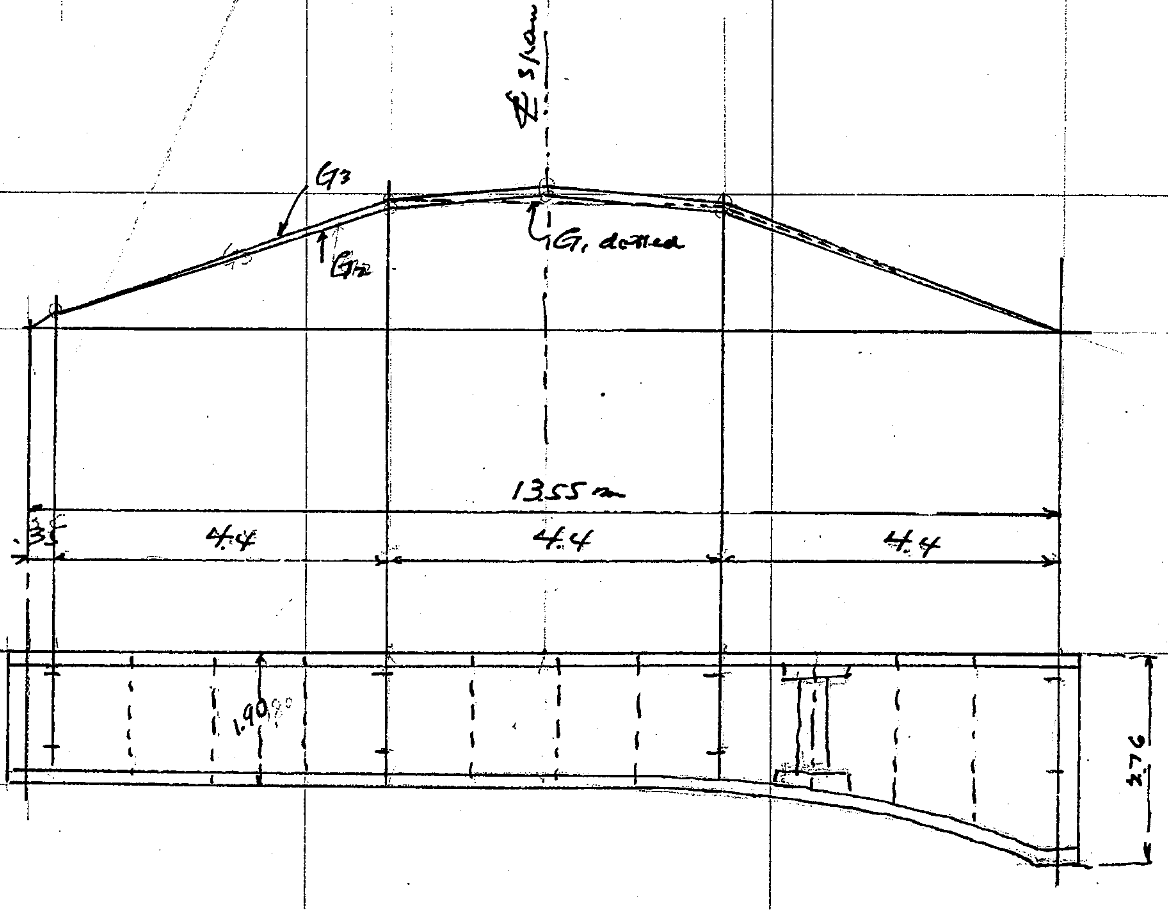
moment at a $\frac{4357}{2} \cdot 0.35 \cdot 13.20 = 10050$ kgm
b $\cdot 4.75 \cdot 8.80 = 91000$ "
c $\cdot 9.15 \cdot 4.4 = 87600$ "
center. $\cdot 6.78 \cdot 6.78 = 100000$ "

End shear $\frac{4357}{2} \cdot 13.55 = 29500$ kg.

Summary for moments and shears DG3.

	a	b	c	max	shear
		moment at			
Dead Load	10050	91000	87600	100.000	29500
Live Load	11100	87000	83910	91780	32100
	21150	178000	171510	191780 kgm	61600 kg

Load on shoe say Dead Load 31000
Live Load 33000
64000 kg.



Design of Juso Bashi for Osaka-Prefecture.

Section of girder DG
web assumed 1890 × 13 = 246.0 □cm 1/8 web = 30.70 □cm
flange assumed 2Ls 150×150×11 = 63.58 − 5.5 = 58.08 □ net
Effective depth 1900 − 81 = 1819

DG1	m = 178000 (max m) at b.	flange stress = $\frac{178000}{1.819}$ = 97800	SR = $\frac{97800}{1200}$ =	81.50 / 30.70 / 50.80 net
DG2	= 177450 at center	= $\frac{177450}{1.819}$ = 97800	= $\frac{97800}{1200}$ =	81.50 / 30.70 / 50.80 net
DG3	= 191780 at center	= $\frac{191780}{1.819}$ = 105000	= $\frac{105000}{1200}$ =	87.50 / 30.70 / 56.80 net

moment of inertia of section
4Ls 150×150×11 = 2×63.58 × 95.4² + 265.2 = 1.163.000
1Pl. 1890 × 13 = 246.00 730.000
1,893 000

Unit stress in gross section — approx deflection

DG1	$\frac{17800000 \times 99.5}{1.893.000}$ = 935 kg/cm²	$\frac{5}{48} \cdot \frac{1355^2 \times M}{21.000.000 \times I}$ =	.855 = (.390 + .465)	1/1390
DG2	$\frac{17745000 \times 99.5}{1.893.000}$ = 933 "	=	.85 = (.44 + .41)	1/1595
DG3	$\frac{19178000 \times 99.5}{1.893.000}$ = 1010 "	=	.92 = (.44 + .48)	1/1475

Approximate weight of main girder.

1-web	1Pl 2000 × 13	@	204.10	× 10.51	=	2145
	1Pl. 2750 × 13	@	280.64	× 3.58	=	1005
	4Ls 150×150×11	@	24.95	× 14.20	=	1420
End stiffs	4Ls 130×130×9	@	17.73	× 1.90	=	135
	4Ls 130×130×9	@	"	× 2.75	=	195
fills.	2Pls 350 (av) × 11	@	30.22	× 1.60	=	97
	2Pls. "	@	30.22	× 2.45	=	148
Int stiff	24Ls 125×90×10	@	16.09	× 2.00 say	=	772
web splice						350
flange splice						150
Sole plate						100
Shelf						450
Rivet heads + misc details say 5%						350
						7317 kg.

Summary for weight of structural steel in one span DG.

stringers	675 × 14.09	=	9520
cross beams	160 × 18	=	2880
Floor beams and Cantilevers	4 × 3200	=	12800
Bottom Lateral Bracings	1135 × 3	=	3410
Bracing under sidewalk		=	500
main girders	5 @ 7317	=	36600
misc			1000
shoes.			5000
			71.710 kg.

Total steel in girder span

Span over canal			164.94
AG span	4 × 180.29	=	721.16
BG	4 × 381.96	=	1527.84
CG	2 × 88.61	=	176.22
DG	2 × 71.71	=	143.42
			2733.58 tons

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CALCULATIONS FOR

大阪府十三橋
タイドアーチ計算
プレリミナリーノ分
一ヨリ四迄ハ本設計書ニ挿入
ガーダー部検定計算

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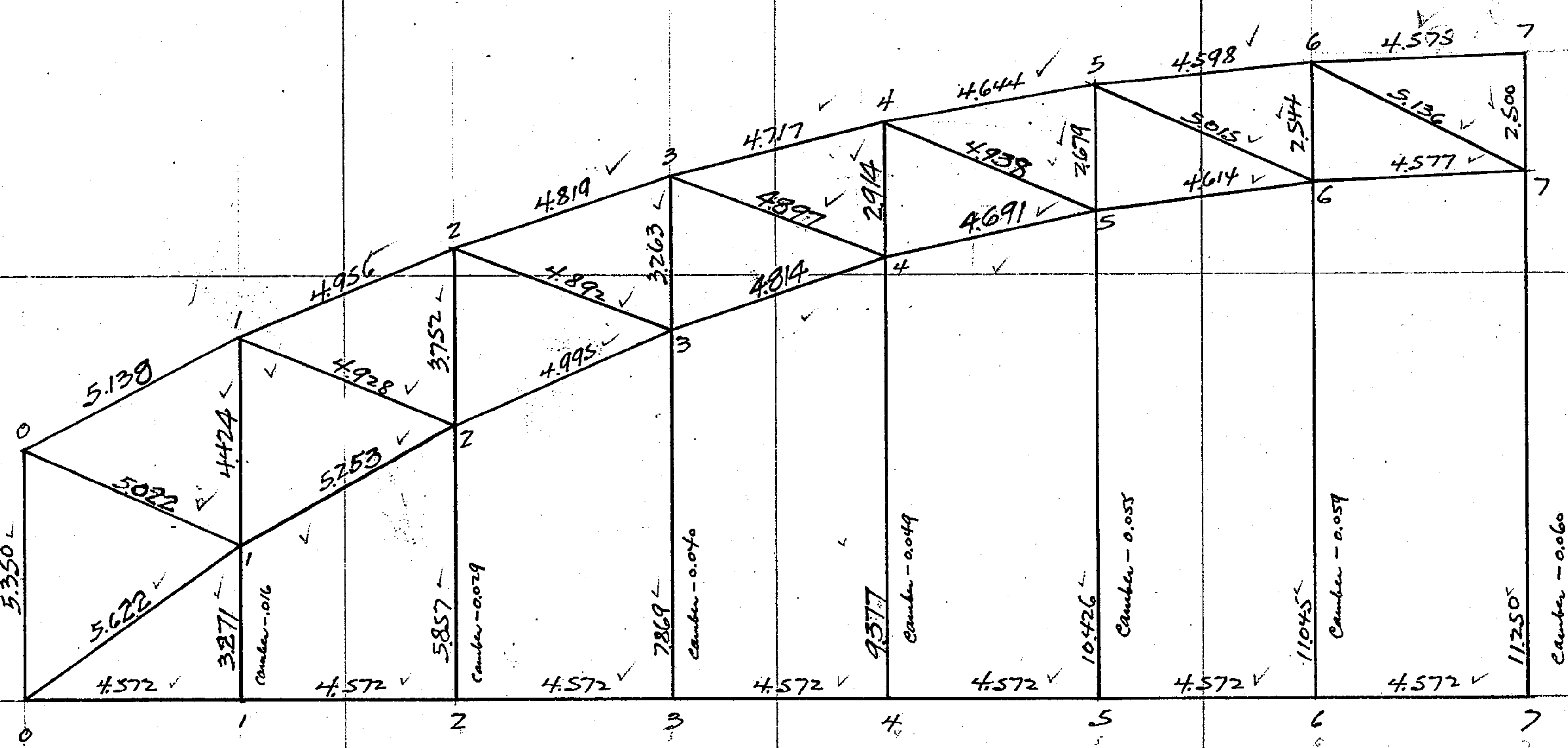
CALCULATIONS FOR



Dead Load Stresses:

H. Approximate D.L. $= 13{,}150^{kg}$ per lin meter for one truss. Panel load $= 13150 \times 4.572 = 60{,}100^{kg}$

moment. $\frac{1}{8} \times 13150 \times 64.008^2 = 6{,}740{,}000$

$$\frac{6{,}740{,}000}{13.75} = 491{,}000^{kg}$$

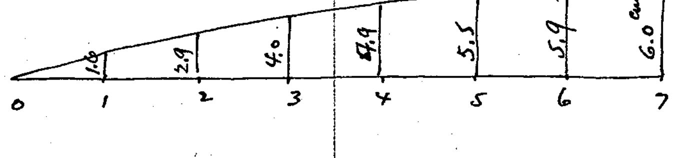
Approx. D.L. $H = 491{,}000 \times .965 = 475{,}000$ kg.

End Reaction $R = 60100 \times 6.5 = 390{,}700$ "

By the Graphical Solutions, D.L. stresses of Several members are as follows.

	Upper chords	Lower chords	Diagonals		Verticals	Hangers	Ties	Load on shoe
0–1	56,000 C	590,000 C	55,000 T	0–0	47,500 C	60,100 T	475,000 T	390,700
1–2	134,500 C	497,000 C	80,000 T	1–1	56,500 C	〃		30,100
2–3	240,000 C	$\overset{3}{4}$89,000 C	111,000 T	2–2	62,000 C	〃		420,800 kg/C
3–4	346,000 C	268,000 C	115,000 T	3–3	48,000 C	〃		
4–5	458,000 C	148,000 C	125,000 T	4–4	36,000 C	〃		
5–6	512,000 C	29,000 C	61,000 T	5–5	13,500 C	〃		
6–7	537,000 C	27,500 T	31,500 T	6–6	31,000 T	〃		
				7–7	33,000 T	〃		

Camber of Bridge 6.0 cm at center.



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CALCULATIONS FOR

Preliminary Design of Jusō Bashi for Osaka prefecture.

Live Load.

Uniform Live Load on Roadway $= \frac{120000}{170+64} = 513$ kg/m²

do on Side walk $= \frac{100000}{170+64} = 428$

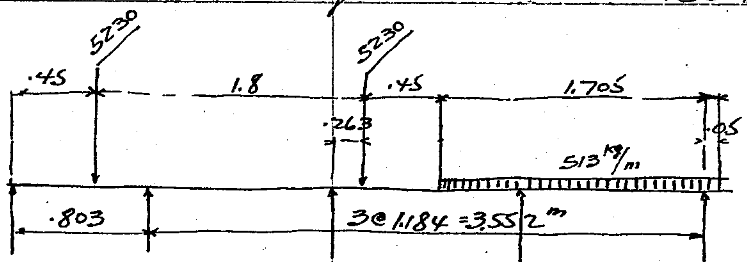
motor truck Rear wheel concentration 4500 kg

Impact coeff. $= \frac{20}{60+64} = 16.1\%$ 730

5230 kg × 2 = 10460 kg for one truss.

Motor truck front wheel concentration with impact

Say 5230 ÷ 3 = 1750 kg × 2 = 3,500 kg



Uniform Load on Road way.

Side of truck 513 × 1.705 = 875 kg/m of span

front + rear 513 × 4.405 = 2260

Sidewalk 428 × 2.75 = 1175

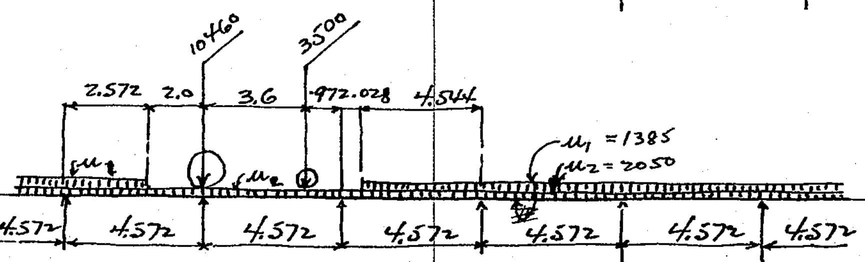
2260 − 875 = 1385

Total load

875 + 1175 = 2050 Side of truck

2260 + 1175 = 3435 front + rear of truck

1385



u_1 = 1385 kg/lin m. front + rear of truck only loaded.

u_2 = 2050 " loaded throughout

u_1	{3165 / 2560}	1000	3125	3165 / 3165	3165 / 3165	
u_2	9380	9380	9380	9380	9380	
Rear wheel		10460				
front		745	2755			
	15105	21585	15260	15710	15710	15710
Assume	15700	15700	15700	15700	15700	15700
		5900				

Electric car loading 5 cars @ 28000 = 140000 kg

140000 ÷ 64 = 2190

impact 16.1% = 352

2542 kg/lin m.

panel load. = 2542 × 4.572 = 11620 kg

Total panel live load.

Motor truck + unif. load. 15700

Electric car loading 11620

27320 kg

Call this 27500 kg

Single concentration 7000 kg

Live Load Stresses:—

upper chord stresses.

$U_6 - U_7$ Loading. full load

Approximate Horizontal thrust H.

Influence ordinates for H.

panel pt.		load 27500	load 7000
1	.215	2.455 % × $\frac{7000}{27500}$ =	0.62 %
2	.418	4.77	1.21
3	.604	6.90	1.75
4	.764	8.72	2.24
5	.898	10.25	2.61
6	.980	11.20	2.85
7	1.000	11.41	2.90

Influence area = 8.758

27500 ÷ 4.572 = 6020 kg/lin meter of span.

moment $= \frac{1}{8} \times 6020 \times 64.008^2 = 3,084,000$ kgm

$H = \frac{3084000}{13.75} = 224,500$ kg

take 0.965 of above value.

H = 224500 × .965 = 216500 kg. for full load.

H for $U_6 - U_7$

full panel loads of 27500 H = 216,500 kg = 216,500

Single load of 7000 at 7. H = .029 × 216,500 = 6,300

H = 222,800 kg

End reaction of 27,500 × 6.5 = 178,700

7000 × ½ = 3,500

182,200 kg

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CALCULATIONS FOR

Preliminary Design of Juso Bashi for Osaka prefecture.

Moment at 7.

182200 × 32.004 = + 5830000
27500 × 4.572 × 21 = − 2640000
222800 × 11.26 = − 2508000
+ 682000 ÷ 2.5 = 273000 kg C —— U_6-U_7

U_5-U_6 Loading 1,2,3,4,5,6,7,6,5,4,3,2

H = 216500 × .97545 = 211000
" × .0285 = 6200
217200 kg

Reaction
27500 × 5.5 = 151300
27500 × 13/14 = 25500
7000 × 8/14 = 4000
180800 kg

moment at 6.

180800 × 4.572 × 6 = + 4960000
27500 × 4.572 × 15 = − 1887000
217200 × 11.045 = − 2400000
+ 673000 ÷ 2.52 = 267000 kg C —— U_5-U_6

U_4-U_5 Loading 1,2,3,4,5,6 ---- 5

H = 216500 × .77155 = 167000
" × .0261 = 5650
172650

Reaction
27500 × ~~43~~ 81/14 = 159200
7000 × 9/14 = 4500
163700 kg

Moment at 5.

163700 × 4.572 × 5 = + 3740000
27500 × 4.572 × 10 = − 1258000
172650 × 10.426 = − 1802000
+ 680000 ÷ 2.645 = 257000 kg C —— U_4-U_5

U_3-U_4. Loading 1,2,3,4,5,6,7

H = 216500 × .55705 = 120500
" × .0224 = 4850
125350

Reaction
27500 × 70/14 = 137500
7000 × 10/14 = 5000
142500

moment at 4.

142500 × 4.572 × 4 = + 2605000
27500 × 4.572 × 6 = − 764000
125350 × 9.456 = − 1185000
+ 656000 ÷ ~~2.74~~ 2.83 = ~~239500~~ 232000 kg C —— U_3-U_4

U_2-U_3. Loading 1,2,3,4,5,6,7.

H = 216500 × .55705 = 120500
" × .0175 = 3800
124300

Reaction
27500 × 70/14 = 137500
7000 × 11/14 = 5500
143000

moment at 3.

143000 × 4.572 × 3 = + 1962000
27500 × 4.572 × 3 = − 377500
124300 × 7.869 = − 978500
+ 606000 ÷ 3.105 = 195100 kg C —— U_2-U_3

U_1-U_2 Loading 1,2,3,4,5,6

H = 216500 × .44295 = 95900
" × .0121 = 2600
+ 98500

Reaction
27500 × ~~45~~ 63/14 = 123800
7000 × 12/14 = 6000
129800

moment at 2.

129800 × 4.572 × 2 = + 1187000
27500 × 4.572 × 1 = − 125800
98500 × 5.857 = − 576500
+ 484700 ÷ 3.45 = 140500 kg C —— U_1-U_2

~~Reaction~~

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M_0-M_1. Loading ①2 3 4 5

H = 216,500 × .33095 = 71,600
  " × .0062 = 1340
  72,940

Reaction
27500 × 55/14 = 108,000
7000 × 13/14 = 6500
114,500

Moment at 1.

114,500 × 4.572 × 1 = + 523,500
72940 × 3.266 = − 238,500
+ 285,000 ÷ 3.925 = 72,700 kg C —— M_0-M_1

Lower Chord Members.

L_0-M_1. Loading Full Load with motor truck rear wheel concentration at center panel.
Same as for M_0-M_1.
H = 222,800 kg   End Reaction 182,200 kg

Moment at M_0.
182,200 × 0 = + 000000
222,800 × 5.35 = − 1192,000
− 1192,000 ÷ 4.36 = 273,500 kg C ---- L_0M_1 ✓

M_1-M_2. Loading 1,2,3,4,5,⑥,7,6,5,4,3

H = 216500 × .92775 = 200,900
  " × .0285 = 6,200
  206,100

Reaction
27,500 × 66/14 = 129,700
7000 × 6/14 = 3000
132,700

Moment at M_1.
132,700 × 4.572 × 1 = + 607,000
207,100 × ~~3.266~~ 7.695 = − 1,595,000
− 988,000 ÷ 3.84 = 257,000 kg C ---- M_1-M_2 ✓

M_2-M_3. Loading 1,2,3,4,5,⑥,7,6,5,4

H = 216,500 × .85875 = 186000
  " × .0285 = 6,200
  192,200

Reaction
27,500 × 55/14 = 108,100
7000 × 6/14 = 3000
111,100

Moment at M_2
111,100 × 4.572 × 2 = + 1,017,000
192,200 × 9.609 = − 1,845,000
− 828,000 ÷ 3.45 = 240,000 kg C —— M_2-M_3 ✓

M_3-M_4. Loading 1,2,3,4,⑤,6,7,6,5

H = 216,500 × .77155 = 167,000
  " × .0261 = 5650
  172,650

Reaction
27,500 × 45/14 = 88,400
7000 × 5/14 = 2,500
90,900

Moment at M_3.
90,900 × 4.572 × 3 = + 1,246,000
172,650 × 11.132 = − 1,924,000
− 678,000 ÷ 3.10 = 218,700 kg C —— M_3-M_4 ✓

M_4-M_5
(neg moment) Loading 1,2,3,4,⑤,6,7,6

H = 216,500 × .66905 = 144,800
  " × .0261 = 5,650
  150,450

Reaction
27,500 × 36/14 = 70,700
7000 × 5/14 = 2500
73,200

Moment at M_4
73,200 × 4.572 × 4 = + 1,338,000
150,450 × 12.291 = − 1,849,000
− 511,000 ÷ ~~2.77~~ 2.84 = ~~184,500~~ 180,000 kg C —— M_4-M_5 ✓

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M4-M5 (Pos. moment)

Loading 1 2 3 ④ 5

H = 216,500 × .33095 = 71,600
× .0224 = 4800
76,400

Reaction
27,500 × 55/14 = 108,100
7000 × 10/14 = 5000
113,100

moment at U4
113100 × 4.572 × 4 = + 2070,000
27,500 × 4.572 × 6 = − 754,000
76,400 × 12.291 = − 939,000
+ 377000 ÷ ~~2.77~~ 2.84 = ~~136,100~~ 132,800 kg T M4-M5.

M5-M6 (neg. moment)

Loading 1 2 3 ④ 5 6 7

H = 216,500 × .55705 = 120,600
× .0224 = 4800
125,400

Reaction
27,500 × 28/14 = 55,000
7000 × 4/14 = 2,000
57,000

moment at U5
57000 × 4.572 × 5 = + 1303,000
125400 × 13.105 = − 1645,000
− 342,000 ÷ 2.66 = 128,700 kg C M5-M6.

M5-M6 (Pos. moment)

Loading 1 2 3 4 ⑤ 6

H = 216,500 × .44295 = 93,000
× .0261 = 5650
98650

Reaction
27,500 × ~~55~~ 63/14 = ~~108,100~~ 123,700
7000 × 9/14 = 4500
~~112600~~ 128,200

moment at U5
~~112,600~~ 128,200 × 4.572 × 5 = + ~~2575000~~ 2,930,000
27,500 × 4.572 × 10 = − 125,8000
98,650 × 13.105 = − 1,293,000
379000 ÷ 2.66 = 142,500 kg T M5-M6.

M6-M7. (neg. moment)

Loading 1 2 ③ 4 5

H = 216,500 × ~~.44295~~ .3395 = 73,500
× .0175 = 3800
77,300

Reaction
27,500 × 15/14 = 29450
7000 × 3/14 = 1500
30950

moment at U6
30950 × 4.572 × 6 = + 848,500
77300 × 13.589 = − 1,050,000
− 201,500 ÷ 2.54 = 79300 kg C --- M6-M7

M6-M7. (Pos. moment)

Loading 6 7 ⑥ 5 4 3 2 1

H = 216,500 × .66.905 = 144,800
× .0285 = 6,170
150,970

Reaction
27,500 × 76/14 = 149,400
7000 × 8/14 = 4,000
153,400

moment at U6.
153,400 × 4.572 × 6 = + 4208,000
150,970 × 13.589 = − 2051,000
27,500 × 4.572 × 15 = − 1,885,000
+ 272,000 ÷ 2.54 = 107,000 kg T —— M6-M7.

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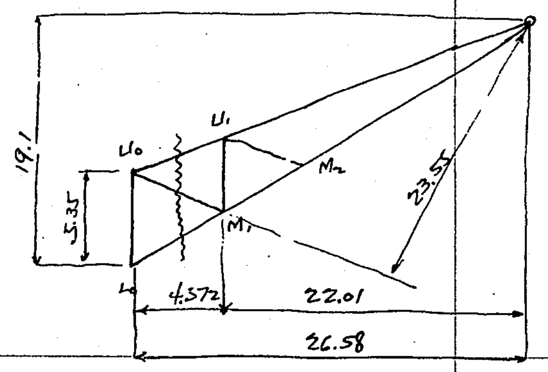
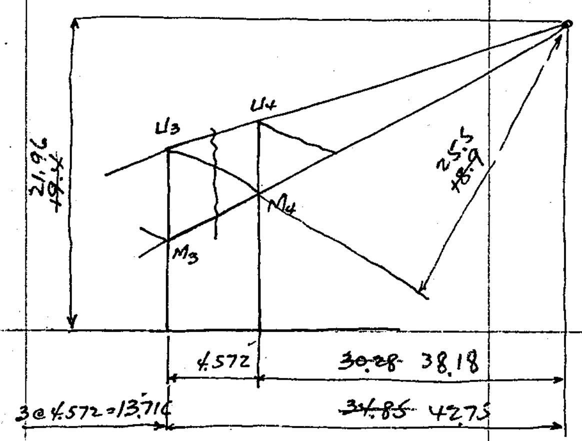
Diagonal members.
U_0-M_1.



Center of moment being found by graphical method.
Loading ①, 2, 3, 4, 5 Reaction
H = 216500 × .33095 = 71300 27500 × 59/14 = 108,100
" × .0062 = 1340 7000 × 13/14 = 6500
72640 114600

Pos. moment.
114600 × 26.58 = + 3046000
72640 × 19.1 = − 1388000
+ 1658000 ÷ 23.55 = 70400 kg T −− U_0-M_1.

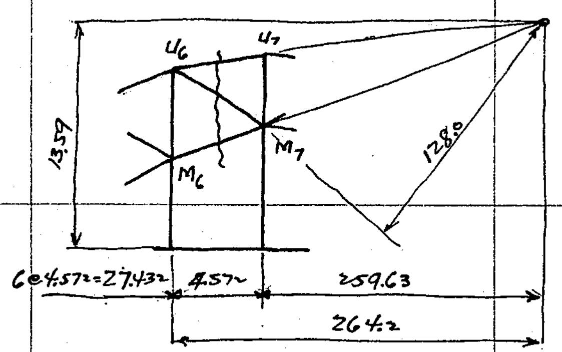
U_3-M_4.



Loading ④, 5, 6, 7, 6, 5, 4, 3, 2, 1 Reaction
H = 216500 × .85875 = 186000 27500 × 55/14 = 108100
" × .0224 = 4850 7000 × 10/14 = 5000
190850 113100

Pos. moment
113100 × ~~48.57~~ 56.47 = + ~~5495000~~ 6390000
190850 × ~~49.4~~ 21.96 = − ~~3700000~~ 4190000
+ ~~1795000~~ 2200000 ÷ ~~18.9~~ 25.5 = ~~95000~~ 86300 kg T −−− U_3-M_4

U_6-M_7



Loading ⑦, 6, 5, 4, 3, 2, 1 Reaction
H = 216500 × .55705 = 120600 27500 × 28/14 = 55000
" × .029 = 6280 7000 × 7/14 = 3500
126880 58500

Pos. moment
58500 × 291.63 = + 17050000
126880 × 13.59 = − 1725000
+ 15325000 ÷ 128 = 119700 kg T −−− U_6-M_7

Loading 1, 2, 3, 4, 5, ⑥ Reaction
H = 216500 × .44295 = 93000 27500 × 63/14 = 123700
" × .0285 = 6200 7000 × 8/14 = 4000
99200 127700

291.63 × 6 = 1749.79
4.572 × 21 = − 96.01
1653.78

Neg. moment.
127700 × 291.63 = + 37200000
99200 × 13.59 = − 1348000
27500 × 1653.78 = − 45460000
~~− 9608000 ÷ 128 = 75100 kg C −− U_6-M_7~~
7000 × 264.2 = − 1850000
11458000 ÷ 128 = 89600 kg C −− U_6-M_7

Vertical members.
U_0-L_0.

Loading 5, 4, 3, 2, ① same as for U_0-M_1
H = 72640, R = 114600
+ moment at M_1.
114600 × 4.572 = + 524000
72640 × 3.266 = − 237400
+ 286600 ÷ 4.572 = 62700 kg C −− U_0-L_0

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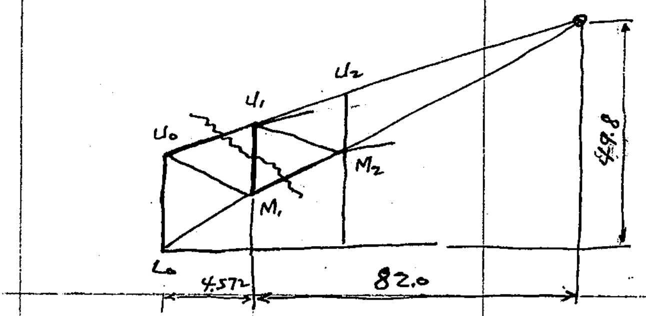
Vertical member.

$U_1 - M_1$.



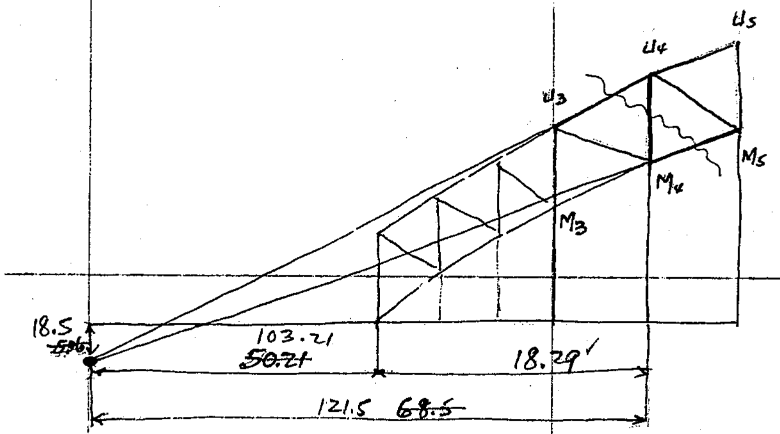
Loading ①2,3,4,5,6

H = 216,500 × .44295 = 93000
" × .0062 = 1340
94340

Reaction
27,500 × 63/14 = 123,700
7000 × 13/14 = 6,500
130,200

Pos. moment.
130,200 × 86.572 = + 11,260,000
94340 × 49.8 = − 4,700,000
27,500 × ~~423.42~~ 82.0 = − 2255,000
7000 × 82.0 = − 574,000
+ 3731,000 ÷ 82.0 = 45,500 kg C $U_1 - M_1$.

Vertical $U_4 - M_4$. (参2)



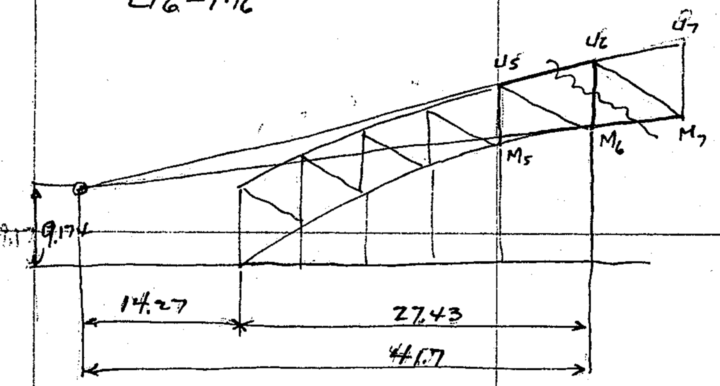
Loading ④5,6,7,6,5,4,3,2,1

H = 216,500 × .85875 = 186000
" × .0224 = 4850
190,850

Reaction
27500 × 55/14 = 108,100
7000 × 10/14 = 5000
113100

pos moment
113,100 × ~~50.21~~ 103.21 = + ~~5680,000~~ 11,660,000
190,850 × ~~5.06~~ 18.5 = − ~~966000~~ 3,520,000
27500 × ~~68.5~~ 121.5 = − ~~1883,000~~ 3340,000
7000 × ~~68.5~~ 121.5 = − ~~480000~~ 850,000
+ ~~2351,000~~ 3,950,000 ÷ ~~68.5~~ 121.5 = ~~34,300~~ 32,500 kg C $U_4 - M_4$

$U_6 - M_6$



Loading ⑥7,6,5,4,3,2,1

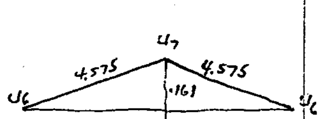
H = 216,500 × .66905 = 144,900
" × .0285 = 6300
151,200

Reaction
27,500 × 36/14 = 70700
7000 × 8/14 = 4000
74,700

Pos moment
74,700 × 14.27 = + 1,065,000
151,200 × 9.174 = + 1,387,000
27,500 × 41.7 = − 1,147,000
7000 × 41.7 = − 292,000
+ 1,013,000 ÷ 41.7 = 24,300 kg C $U_6 - M_6$

$U_7 - M_7$

max stress in $U_7 - U_6$ = 273000 kg C



Stress in vertical $U_7 - M_7$
$= 273000 \times \frac{.161}{4.575} \times 2 =$ 19200 T — $U_7 - M_7$

$U_6 - M_6$ Tensile stress

14.27 × 5 = 71.35
15 × 4.572 = 68.58
139.93

Loading ⑤4,3,2,1

H = 216,500 × .33095 = 71,600
" × .0261 = 5650
77,250

Reaction
27,500 × 55/14 = ~~29450~~ 108,100
7000 × 9/14 = 4500
~~33950~~
112,600

neg. moment
~~33950~~ 112,600 × 14.27 = + 1607,000
77250 × 9.174 = + 709,000
7000 × ~~37.13~~ = − 260,000
27,500 × ~~41.7~~ 139.93 = − 3847,000
− 1791,000 ÷ 41.7 = 43,000 kg T $U_6 - M_6$.

Preliminary Design of Jusō Bashi for Osaka prefecture.

Summary of Dead and Live Load Stresses of Truss members.

Upper chords.	U_0-U_1	U_1-U_2	U_2-U_3	U_3-U_4	U_4-U_5	U_5-U_6	U_6-U_7	
Dead Load	61,000 C	147,000 C	260,000 C	372,000 C	474,000 C	543,000 C	570,000 C	
Live Load	72,700 C	140,500 C	195,100 C	~~239,500~~ 232,000 C	257,000 C	267,000 C	273,000 C	
Total	133,700 C	287,500 C	455,100 C	~~611,500~~ C 604,000	731,000 C	810,000 C	843,000 C	
Lower chords	L_0-M_1	M_1-M_2	M_2-M_3	M_3-M_4	M_4-M_5	M_5-M_6	M_6-M_7	
Dead Load	584,000 C	485,000 C	370,000 C	$24\overset{1}{2},000$ C	$11\overset{3}{7},000$ C	9,000 C	66,000 T	
Live Load	273,500 C	257,000 C	240,000 C	218,700 C	{ ~~184,500~~ 180,000 C / ~~136,100~~ 132,800 T }	{ 128,700 C / 142,500 T }	{ 79,300 C / 107,000 T }	
Total	857,500 C	742,000 C	610,000 C	~~460,700~~ C 459,700	~~301,500~~ C 293,000 (~~58,100~~ T) 60,800	137,500 C (136,500 T)	173,000 T (35,300 C)	
Diagonals.	U_0-M_1	U_1-M_2	U_2-M_3	U_3-M_4	U_4-M_5	U_5-M_6	U_6-M_7	
Dead Load	60,000 T	87,000 T	119,000 T	$12\overset{7}{3},000$ T	114,000 T	83,000 T	33,000 T	
Live Load	70,400 T			~~95,000~~ 86,300 T			{ 119,700 T / 89,600 C }	
Total	130,400 T			~~218,000~~ T 213,300			152,700 T (67,600) C	
Verticals.	U_0-L_0	U_1-M_1	U_2-M_2	U_3-M_3	U_4-M_4	U_5-M_5	U_6-M_6	U_7-M_7
Dead Load	52,000 C	61,000 C	66,000 C	51,000 C	37,000 C	6,000 C	22,000 T	35,000 T
Live Load	62,700 C	45,500 C	say (42,000) C		~~34,300~~ 32,500 C		{ 43,000 T / 24,300 C }	19,200 T
Total	114,700 C	106,500 C	(108,000 C)		~~71,300~~ C 69,500		65,000 T (9,600 C)	54,200 T

Max. Stress on tie = D.L. = 475,000 T
L.L. = 222,800 T
697,800 T.

Max. Load on Shoes.

D.L. 6.5 @ 60100 = 390,700
$\frac{1}{2}$ @ " = 30,100
420,800
Shoe say 1,200
D.L. = 422,000 C

L.L.
7.0 @ 27500 = 192,500
rear wheel extra 7,000
199,500

L.L. Call this 200,000 C
Total Load on one shoe = 622,000 kg. C

Max. Load on Hanger
motor truck rear wheel concentration = 4500
Impact coef. = $\frac{20}{60+9.144}$ = 29% = 1300
5800 kg × 2 = 11,600 kg
front wheel con. with impact = say 5800 × $\frac{1}{3}$ = 1930 kg × 2 = 3,860 kg.
unif. live load on roadway = 600 kg/m²
M_1 ~~1000~~ do Side walk = 500 "
M_2 ~~4380~~

Unif. load on Roadway
Side of motor truck = 600 × 1.705 = 1025 kg/m of span
front + rear " = 600 × 4.405 = 2645 "
Side walk = 500 × 2.75 = 1375 "
Total load. 1025 + ~~2645~~ 1375 = 2400 = M_2 loaded throughout
2645 + 1375 = 4020
−2400
1620 = M_1 front + rear of truck only loaded.

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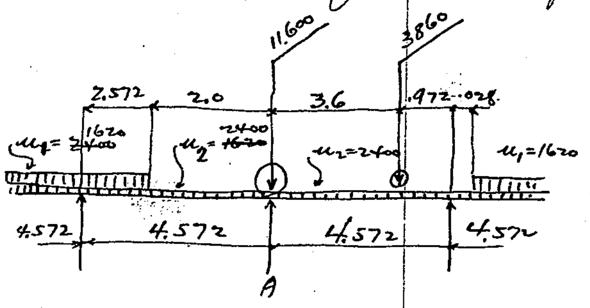


Reaction of floor Beam A.

unif load u_1, $\cancel{1620}\ 2400 \times 4.572 = \cancel{7410}\ 10,970$

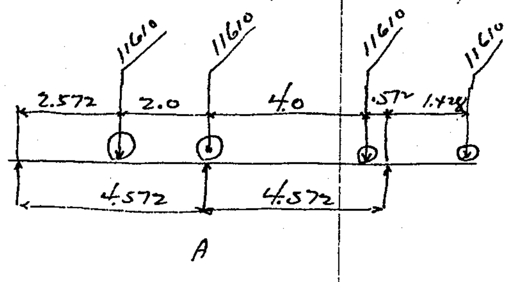
" u_2 $\dfrac{\cancel{2400}\ 1620 \times 2.572^2}{2 \times 4.572} = \cancel{1740}\ 1220$

rear wheel $= 11,600$

front wheel $3860 \times \dfrac{.972}{4.572} = 820$

$24610 = 24610$ unif load + truck

Electric Car Loading Water sprinkler double line.



Concentration for one axle. 9000 kg

Impact 29% = 2610

11610 kg

Concentration on A.

$11610 \times \dfrac{2.572}{4.572} = 6,530$

$11610 \times \dfrac{.572}{4.572} = 1,450$

$11610 \times 1 = 11610$

19590 kg for one truss.

24610

44,200 kg

Summary of Load on Hanger.

Dead Load = 60,100 kg

Live Load = 44,200 "

104300 kg

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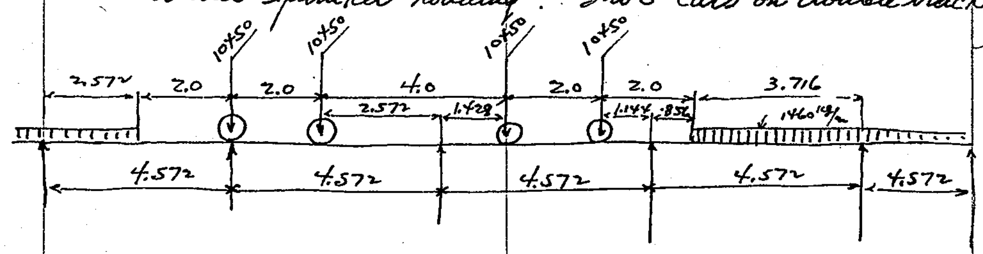
Electric Car Loading.
Water splinker loading. Two cars on double tracks.



Concentration on one axle for single line 9000 kg
Impact 16.1 % = 1,450
10,450 kg

Unif. load. 513 × 5.69 ÷ 2 = 1460 kg per lin. m. for one truss

2700	1050			3,340	6680
3,340	10450		2210	3210	
	5,880	4570			
		7,180	3270		
		2620	7830		
~~2700~~ 6,040	17,380	14,370	13,310	6,550	6680
15,105	21,585	15,260	15,710	15,710	15,710
21,145	38,965	29,630	29,020	22260	22390

Panel Loads from Electric Railway.
" " " Roadway + side walk.

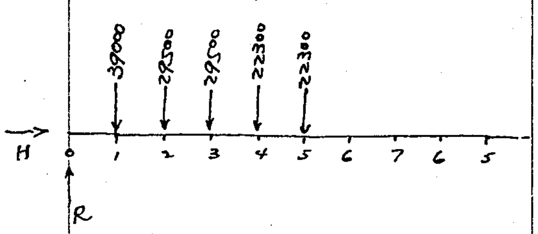
22,300	39,000	29,500	29,600	22300 ~~22300~~	22300 ~~22300~~

Assume total panel loads.

Check of Web stresses for Water splinker Loading.

Diagonal members.

U_0-M_1.



Percentage of H (= 216,500) due to Single panel load.

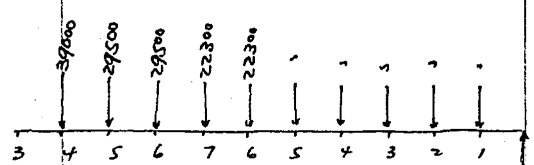
panel pt		27,500 kg	39,000 kg	29,500 kg	22300 kg
1	.215	2.455 %	3.98 %	2.63 %	1.99 % of 216,500 kg
2	.418	4.77	6.76	5.12	3.86
3	.604	6.90	9.79	7.40	5.59
4	.764	8.72	12.37	9.35	7.07
5	.898	10.25	14.53	11.00	8.30
6	.980	11.20	15.88	12.00	9.07
7	1.000	11.41	16.18	12.24	9.25
					45.13

H.
3.98
5.12
7.40
7.07
8.30
31.87 % × 216,500 = 69,000 = H.

Reaction
39000 × 13/14 = 36,200
29500 × 23/14 = 48,450
22300 × 19/14 = 30,250
114,900 = R.

Pos. moment.
114,900 × 26.58 = + 3,052,000
69000 × 19.10 = − 1,318,000
+ 1,734,000 ÷ 23.55 = 73600 kg T — U_0-M_1.

U_3-M_4

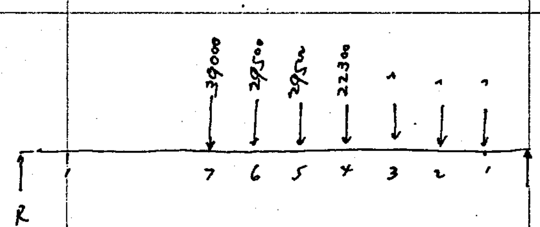


H. 12.37
11.00
12.00
45.13
80.50 % × 216500 = 174,200 kg

Reaction
39,000 × 10/14 = 27,880
29500 × 17/14 = 30,750
22300 × 28/14 = 44600
103,230 kg

Pos. moment.
103,230 × 48.57 = + 5,015,000
174200 × 19.40 = − 3,385,000
+ 1,640,000 ÷ 18.9 = 86800 kg T — U_3-M_4

U_6-M_7.



H. 16.18
23.00
18.51
57.69 % × 216,500 = 125,000

Reaction
39000 × 7/14 = 19,500
29500 × 11/14 = 23,200
22300 × 10/14 = 15,940
58,640

Pos. moment
58640 × 291.63 = + 17,100,000
125,000 × 13.59 = − 1,700,000
+ 15,400,000 ÷ 128 = 120,200 kg T —— U_6-M_7

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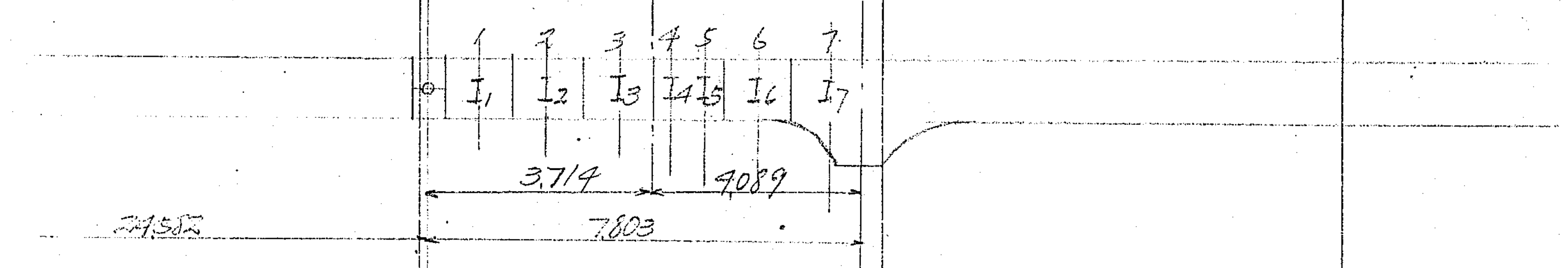
SEIYU BLDG, TOKIO

MADE BY M. Juda DATE 1-8-17 FILE NO.

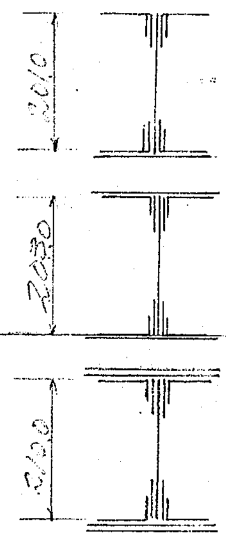
CHECKED BY DATE PAGE NO.

CALCULATIONS FOR

Deflection of anchor span
cantilever arm for suspended 24.582 meter span.
revised BGCA1 under sidewalk



General equation $\delta = \int \frac{M m dx}{EI}$
moment of Inertia



4 L's 150×150×19 = 213.56 × $\overline{96.1}^2$ + 4392 = 1,974,400
4 Pl's 250 ×13 = 130.00 × $\overline{87.5}^2$ + 4060 = 1,000,000
1 Pl. 350 ×13 = 45.50 × $\overline{101.2}^2$ = 461,000
1 Pl. 2000 × 13 = = 866,000
4,301,400 = $I_1 = I_2$

4 L's 150×150×19 = 213.56 × $\overline{97.1}^2$ + 4392 = 1,974,400
4 Pl's 250 ×13 = 130.00 × $\overline{88.5}^2$ + 4060 = 1,000,000
2 Pl's 350 ×13 = 91.00 × $\overline{102.1}^2$ = 930,000
1 Pl. 2,020 × 13 = = 866,000
4,770,400 = I_3

4 L's 150×150×19 = 213.56 × $\overline{100.6}^2$ + 4392 = 2,164,400
4 Pl's 250 × 13 = 130.00 × $\overline{92.0}^2$ + 4060 = 1,104,000
4 Pl's 350 ×13 = 182.00 × $\overline{106.3}^2$ = 2,060,000
1 Pl 2,090 ×13 = 990,000
6,318,400 = I_4

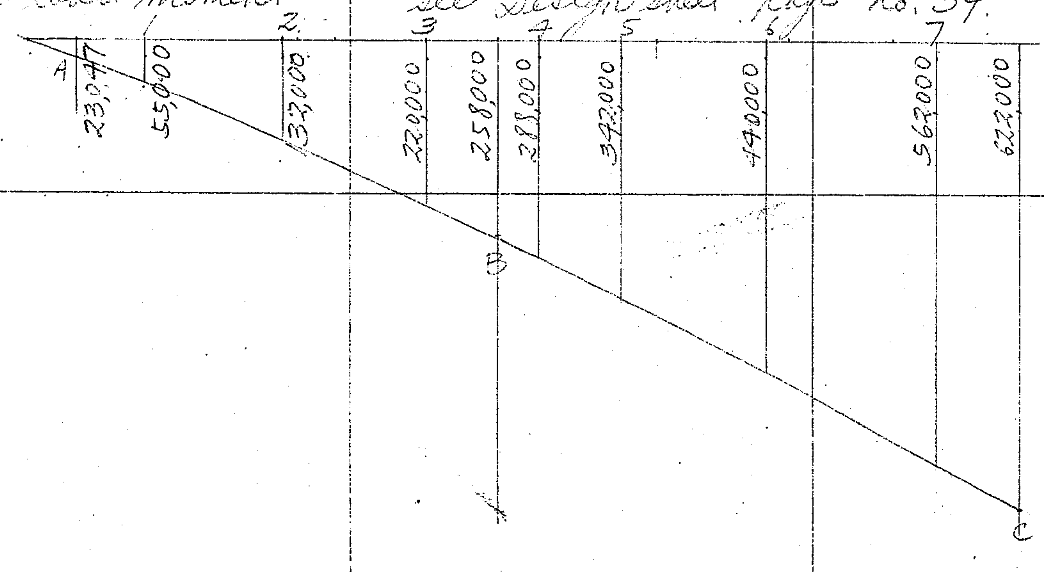
I_5 3 Cov. Pl's + 2130 b to b. of flange L's
4 L's 150×150×19 = 213.56 × $\overline{102.1}^2$ + 4392 = 2,224,400
4 Pl's 250× 13 = 130.00 × $\overline{73.5}^2$ + 4060 = 1,139,100
6 Pl's 350 ×13 = 274.00 × $\overline{108.5}^2$ = 3,220,000
1 Pl. 2,120 ×13 = 1,035,000
7,618,500 = I_5

I_6 4 Cov. Pl's + 2340 b to b of flg. L's
4 L's 150×150×19 = 213.56 × $\overline{112.7}^2$ + 4392 = 2,714,400
4 Pl's 250 ×13 = 130.00 × $\overline{104.0}^2$ + 4060 = 1,409,100
8 Pl's 350 ×13 = 364.00 × $\overline{119.6}^2$ = 5,200,000
1 Pl 2,330 × 13 = 1,370,000
10,693,500 = I_6

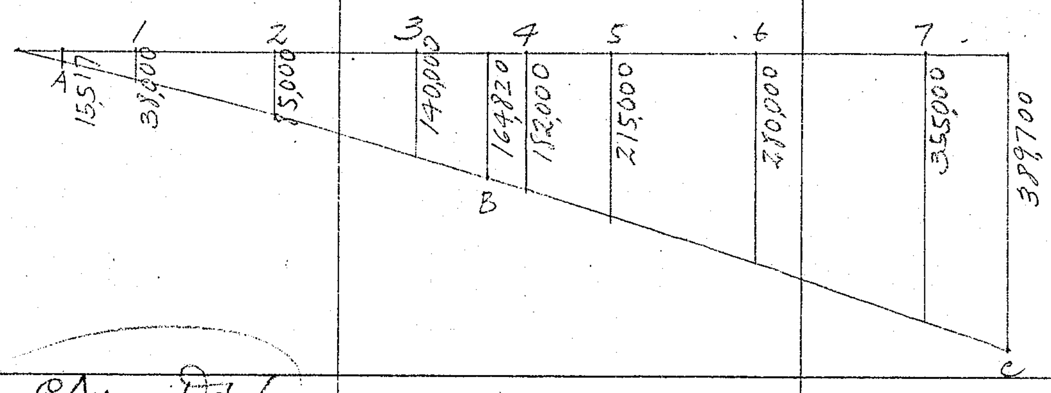
I_7 4 Cov. Pl's + 2660 b to b of flg. L's
4 L's 150×150×19 = 213.56 × $\overline{128.6}^2$ + 4392 = 3,524,400
4 Pl's 250 ×13 = 130.00 × $\overline{120.0}^2$ + 4060 = 1,874,100
8 Pl's 350 ×13 = 364.00 × $\overline{135.6}^2$ = 6,690,000
1 Pl. 2650 ×13 = 1,880,000
13,968,500 = I_7

Dead load moment see Design sheet page no. 39.

CA1 Dead load Cantilever arm only



Live load moment see design sheet page no. 34



CA1 live load
cantilever arm only

CA1 D+L

point	deadload moment	live load moment	unit moment (m)	dx	I	$\frac{Md\,m\,dx}{EI}$	$\frac{Ml\,m\,dx}{EI}$
1	5,500,000	3,800,000	− 92.7	154.7	4,301,400	0.00874	0.00603
2	13,200,000	8,500,000	− 202.2	95.0	4,301,400	0.0282	0.0181
3	22,000,000	14,000,000	− 314.0	128.5	4,770,400	0.0886	0.0563
4	28,800,000	18,200,000	− 400.2	44.0	6,318,400	0.0383	0.0242
5	34,200,000	21,500,000	− 467.9	91.4	7,618,500	0.0913	0.0574
6	44,000,000	28,000,000	− 581.7	136.3	10,693,500	0.1555	0.0990
7	56,200,000	35,500,000	− 715.1	130.4	13,968,500	0.1790	0.1130
						0.58963	0.37403

4,00,000
2,400,000

$0.589 \times \frac{355}{562} = .372$

589
372
96

mark BTCH2 under roadway

$I_1' = I_2' = 4{,}301{,}400$
$I_3' = 4{,}770{,}400$
$I_4' = 6{,}318{,}400$
$I_5' = 7{,}618{,}500$

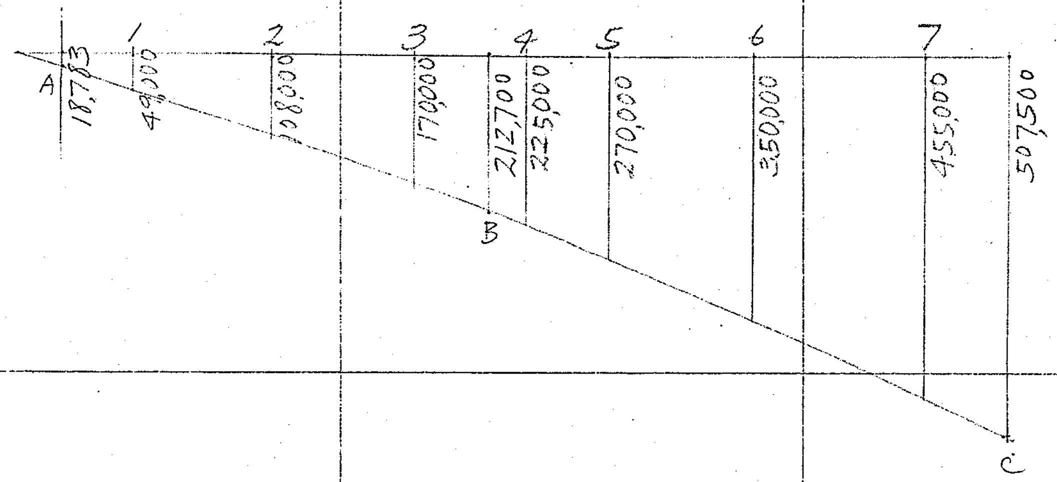
I_6' ... 3 cov. P/s + 2,340 b. to b of flg. Ls

4 Ls	150×150×19 = 21.356 × $\overline{112.7}^2$ + 4392		= 2,714,400
4 P/s	250 × 13 = 130.00 × $\overline{104.0}^2$ + 4060		= 1,409,100
6 P/s	350 × 13 = 274.00 × $\overline{119.0}^2$		= 3,880,000
1 Pl	2300 × 13		= 1,370,000
			9,373,500 = I_6'

I_7' ... 3 cov. P/s + 2,660 b to b of flg. Ls

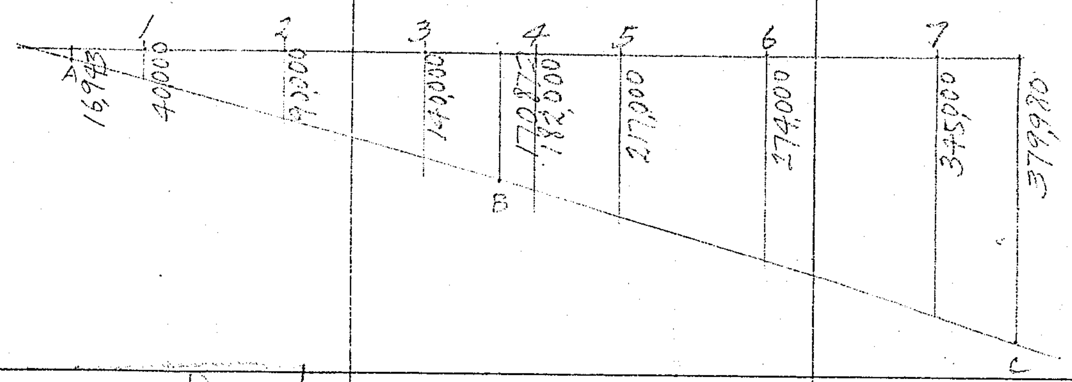
4 Ls	150×150×19 = 21.356 × $\overline{128.6}^2$ + 4392		= 3,524,400
4 P/s	250 × 13 = 130.00 × $\overline{120.0}^2$ + 4060		= 1,874,100
6 P/s	350 × 13 = 274.00 × $\overline{135.0}^2$		= 4,980,000
1 Pl	2650 × 13		= 1,880,000
			12,258,500 = I_7'

Dead load moment see design sheet page no. 36.



CA2 Dead Load.
cantilever arm only

Live load moment see design sheet page no. 36



CA2 Live load
cantilever arm only

CA2 D+L

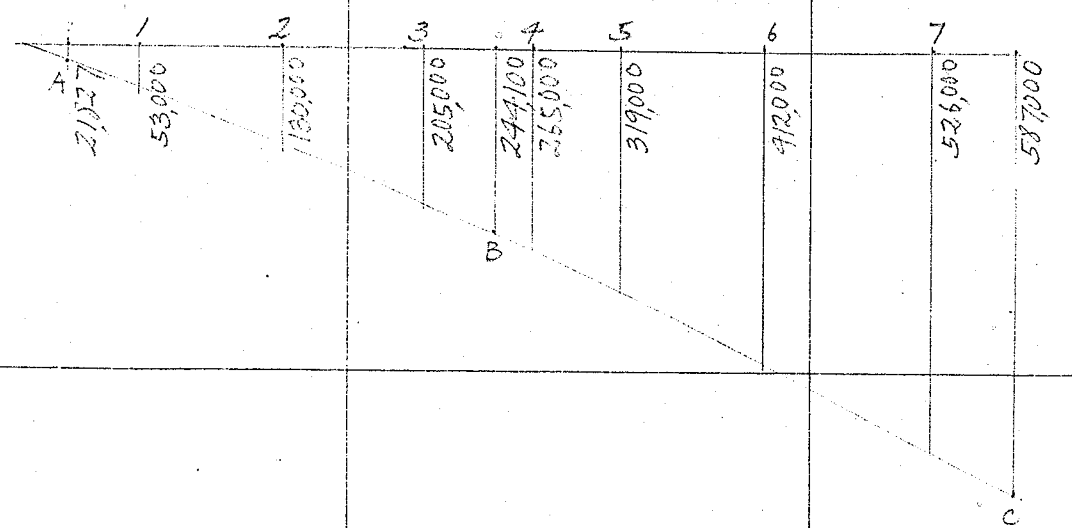
point	dead load moment	live load moment	unit moment (m)	dx	I	$\frac{M_d m dx}{EI}$	$\frac{M_l m dx}{EI}$
1	49,000	40,000	92.7	154.7	4,301,400	0.00775	0.00634
2	108,000	70,000	202.2	95.0	4,301,400	0.0230	0.01918
3	170,000	140,000	314.0	128.5	4,770,400	0.0685	0.0565
4	225,000	182,000	400.2	44.0	6,318,400	0.02995	0.0242
5	270,000	217,000	467.9	91.4	7,618,500	0.0722	0.0580
6	350,000	274,000	581.7	136.3	9,373,500	0.1414	0.1115
7	455,000	345,000	715.1	130.4	12,258,500	0.1650	0.1251
						0.5078	0.40082

91

mark BGC3 under Electric Ry tracks

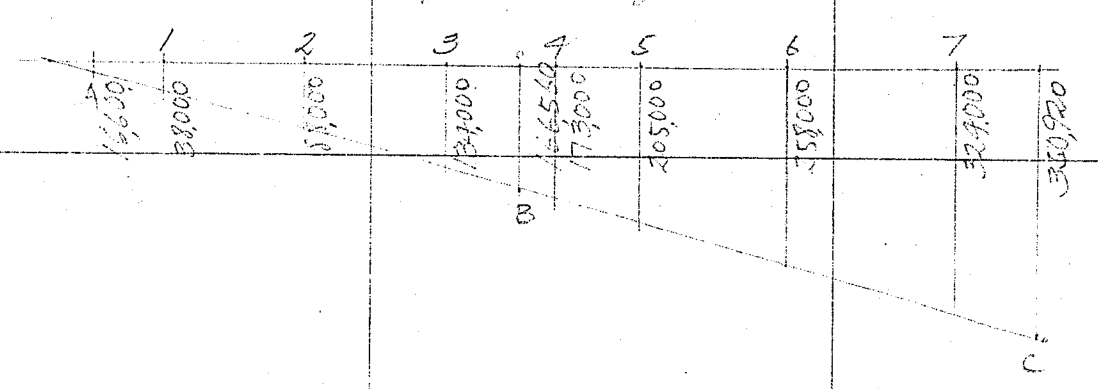
$I_1'' = I_2'' = 4301400$
$I_3'' = 4,770,400$
$I_4'' = 6,318,400$
$I_5'' = 7,618,500$
$I_6'' = 9,373,500$
$I_7'' = 12,258,500$

Dead load moment see design sheet page no 36.



CA3 Dead Load
cantilever arm only

Live load moment see design sheet page no. 36



CA3 Live load
cantilever arm only

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MADE BY M. Jiida DATE 4-8-19 **FILE NO** 4

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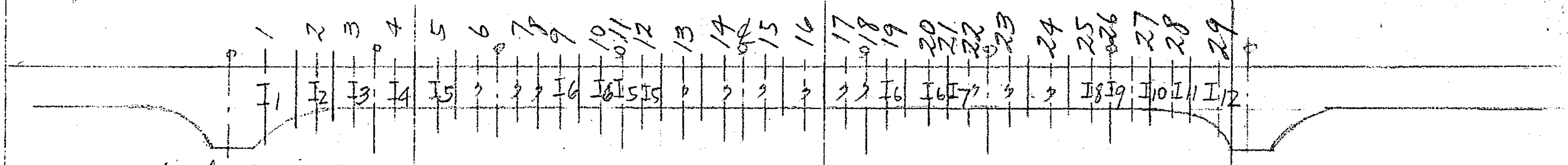
CALCULATIONS FOR

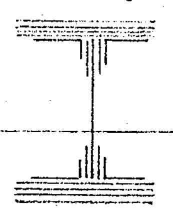
[CA3 D+L]

point	deadload moment	liveload moment	unit moment (m)	dx	I	DL $\frac{Mm\,dx}{EI}$	LL $\frac{M_L m\,dx}{EI}$
1	53,000	38,000	92.7	154.7	4,301,400	0.00838	0.00601
2	130,000	88,000	202.2	95.0	4,301,400	0.0277	0.01875
3	205,000	134,000	314.0	128.5	4,770,400	0.0826	0.0540
4	265,000	173,000	400.2	44.0	6,318,400	0.03521	0.02399
5	317,000	205,000	467.9	91.4	7,618,500	0.0853	0.0548
6	412,000	258,000	581.7	136.3	9,373,500	0.1663	0.1041
7	526,000	324,000	715.1	130.4	12,258,500	0.1910	0.1175
						0.59649	0.37915

Deflection of Cantilever arm due to moment of ~~intermediate~~ anchor span. mark BGCA1



moment of Inertia



I_1 ... 4 Cov. P/s + 2,670 b. to b. of flg. Ls.
4 Ls 150×150×19 = 213.56 × $\overline{129.1}^2$ + 4392 = 3,574,400
4 P/s. 250 ×13 = 130.00 × $\overline{120.5}^2$ + 4060 = 1,894,100
8 P/s. 350 ×13 = 364.00 × $\overline{136.1}^2$ = 6,749,000
1 Pl. 2660 ×13 = 2,035,000
14,252,500 = I_1

(BG.1)

I_2 ... 5 Cov. P/s. + 2,360 b. to b. of flg. Ls
4 Ls 150×150×19 = 213.56 × $\overline{114.6}^2$ + 4392 = 2,804,400
4 P/s. 250 ×13 = 130.00 × $\overline{105.0}^2$ + 4060 = 1,436,100
10 P/s. 350 ×13 = 455.00 × $\overline{121.3}^2$ = 6,700,000
1 Pl 2350 ×13 = 1,405,000
12,345,500 = I_2

I_3 ... 5 cov. P/s + 2,130 b. to b. of flg. Ls.
4 Ls 150×150×19 = 213.56 × $\overline{102.1}^2$ + 4392 = 2,230,400
4 P/s. 250 ×13 = 130.00 × $\overline{93.5}^2$ + 4060 = 1,139,100
10 P/s 350 ×13 = 455.00 × $\overline{109.8}^2$ = 5,470,000
1 Pl. 2120 ×13 = 1,031,000
9,870,500 = I_3

I_4 ... 3 Cov. P/s. + 2,030 b. to b. of flg. Ls
4 Ls 150×150×19 = 213.56 × $\overline{97.1}^2$ + 4392 = 1,974,400
4 P/s. 250 ×13 = 130.00 × $\overline{88.5}^2$ + 4060 = 1,000,000
6 P/s. 350 ×13 = 274.00 × $\overline{103.5}^2$ = 2,930,000
1 Pl. 2020 ×13 = 886,000
6,790,400 = I_4

I_5 ... 3 cov. P/s. + 2,010 b. to b. of flg. Ls
4 Ls 150×150×19 = 213.56 × $\overline{96.1}^2$ + 4392 = 1,974,400
4 P/s 250 ×13 = 130.00 × $\overline{87.5}^2$ + 4060 = 1,000,000
6 P/s. 350 ×13 = 274.00 × $\overline{102.5}^2$ = 2,880,000
1 Pl 2000 ×13 = 866,000
6,720,400 = I_5

I_6 ... 4 cov. P/ + 2,010 b. to b. of flg. Ls
4 Ls 150×150×19 = 213.56 × $\overline{96.1}^2$ + 4392 = 1,974,400
4 P/s. 250 ×13 = 130.00 × $\overline{87.5}^2$ + 4060 = 1,000,000
8 P/s. 350 ×13 = 364.00 × $\overline{103.1}^2$ = 3,870,000
1 Pl. 2,000 ×13 = 866,000
7,710,400

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MADE BY *M. Iida* DATE *4-8-19* **FILE NO** *5*

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CALCULATIONS FOR

I_7 — 2 Cov. P/s + 2010 b. to b. of flg. Ls
4Ls 150×150×19 = 213.56 × $\overline{96.1}^2$ + 4392 = 1,974,400
4P/s 250 × 13 = 130.00 × $\overline{87.5}^2$ + 4060 = 1,000,000
4P/s 350 × 13 = 182.00 × $\overline{101.8}^2$ = 1,880,000
1P/ 2000 × 13 = 866,000
5,720,400 = I_7

I_8 2 Cov. P/s + 2020 b to b of flg. Ls
4Ls 150×150×19 = 213.56 × $\overline{96.6}^2$ + 4392 = 1,999,400
4P/s 250 × 13 = 130.00 × $\overline{88.0}^2$ + 4060 = 1,009,000
4P/s 350 × 13 = 182.00 × $\overline{102.3}^2$ = 1,910,000
1P/ 2010 × 13 = 880,000
5,898,400 = I_8

I_9 2 Cov. P/s + 2080 b to b of flg. Ls
4Ls 150×150×19 = 213.56 × $\overline{99.6}^2$ + 4392 = 2,125,400
4P/s 250 × 13 = 130.00 × $\overline{90.5}^2$ + 4060 = 1,067,100
4P/s 350 × 13 = 182.00 × $\overline{105.3}^2$ = 2,022,000
1P/ 2070 × 13 = 960,000
6,174,500 = I_9

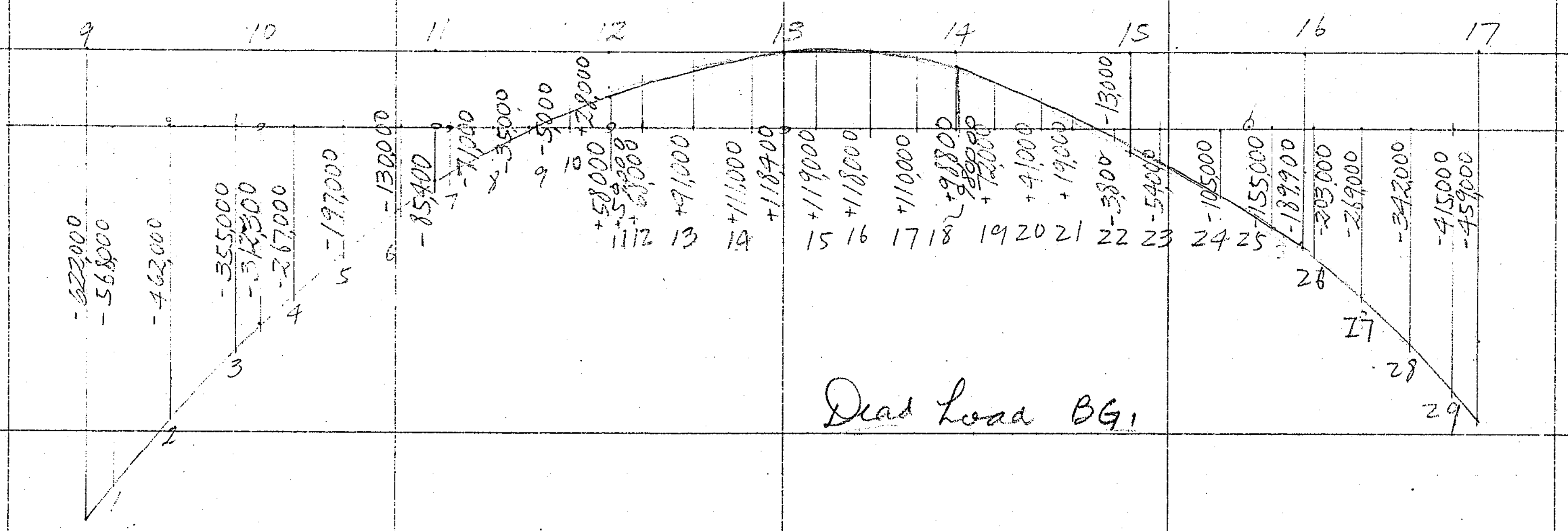
I_{10} — 3 Cov. P/s + 2210 b to b of flg. Ls
4Ls 150×150×19 = 213.56 × $\overline{106.1}^2$ + 4392 = 2,424,400
4P/s 250 × 13 = 130.00 × $\overline{97.5}^2$ + 4060 = 1,239,100
6P/s 350 × 13 = 274.00 × $\overline{112.5}^2$ = 3,470,000
1P/ 2200 × 13 = 1,150,000
8,283,500 = I_{10}

BG1

I_{11} — 2 Cov. P/s + 2410 b to b of flg. Ls
4Ls 150×150×19 = 213.56 × $\overline{116.1}^2$ + 4392 = 2,888,400
4P/s 250 × 13 = 130.00 × $\overline{107.5}^2$ + 4060 = 1,505,100
4P/s 350 × 13 = 182.00 × $\overline{121.8}^2$ = 2,695,000
1P/ 2400 × 13 = 1,500,000
8,588,500 = I_{11}

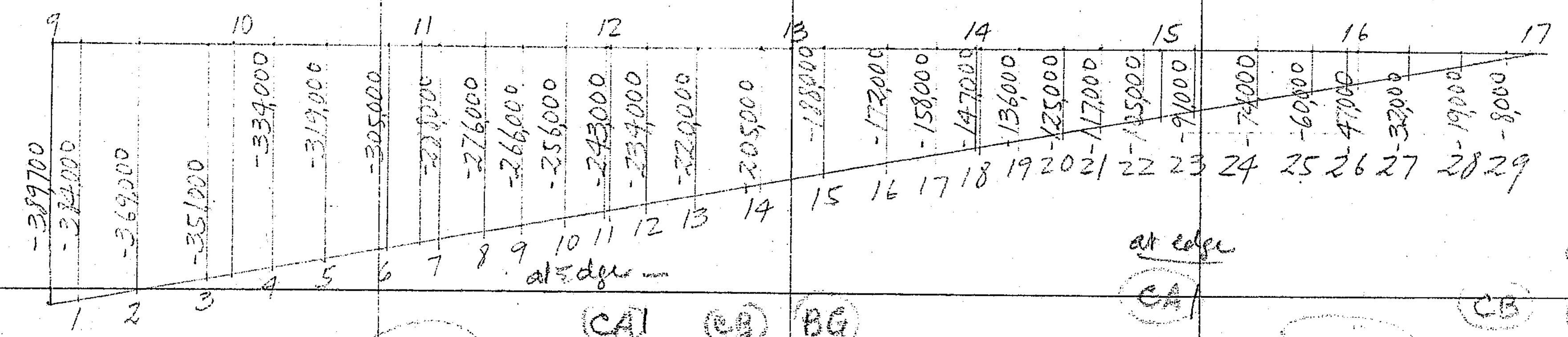
I_{12} — 2 Cov. P/s + 2670 b to b of flg. Ls
4Ls 150×150×19 = 213.56 × $\overline{129.1}^2$ + 4392 = 3,574,400
4P/s 250 × 13 = 130.00 × $\overline{120.5}^2$ + 4060 = 1,894,100
4P/s 350 × 13 = 182.00 × $\overline{134.8}^2$ = 3,300,000
1P/ 2660 × 13 = 2,035,000
10,803,500 = I_{12}

Dead load moment



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Negative live load moments see design sheet page no. 45



CA1 CB BG CA1 CB BG

point	dead load moment	live load moment	unit moment (m)	dx'	I	Md m dx / EI	Ml m dx / EI
1	-568,000	-384000	7670	1224	14252,500	-0.1780	-0.1205
2	-462,000	-369000	7350	1443	12,345,500	-0.1892	-0.1511
3	-355,000	-351,000	7000	1596	9870,500	-0.1912	-0.1890
4	-267,000	-334,000	6660	1130	6790,400	-0.1412	-0.1768
5	-197,000	-319,000	6380	1363	6,720,400	-0.1215	-0.1965
6	-130000	-305,000	6040	1363	"	-0.0758	-0.1781
7	-71,000	-288000	5760	950	"	-0.0375	-0.1118
8	-35,000	-276,000	5540	950	"	-0.0131	-0.1030
9	-5,000	-266000	5340	915	7,710,400	-0.0015	-0.0802
10	+28,000	-256000	5100	1015	"	+0.0090	-0.0817
11	+59,000	-243000	4890	856	6,720,400	+0.0200	-0.0720
12	+68000	-234,000	4670	856	"	+0.0151	-0.0520
13	+91,000	-220,000	4410	1363	"	+0.0388	-0.0940
14	+111,000	-205,000	4075	1422	"	+0.0455	-0.0840
15	+119,000	-188,000	3740	1363	"	+0.0430	-0.0640
16	+118,000	-172000	3408	1363	"	+0.0389	-0.0566
17	+110,000	-158000	3140	950	"	+0.0232	-0.0334
18	+980000	-147,000	2938	950	"	+0.0194	-0.0291
19	+72,000	-136,000	2715	915	7,710,400	+0.0111	-0.0209
20	+41,000	-125000	2480	1015	5,720,400	+0.0076	-0.0262
21	+19000	-117,000	2280	856	"	+0.0031	-0.0191
22	-38,000	-105000	2050	856	"	-0.0055	-0.0153
23	-54000	-91,000	1780	1363	"	-0.0109	-0.0184
24	-105,000	-74000	1452	1442	"	-0.0184	-0.0129
25	-155,000	-60,000	1168	1031	5898,400	-0.0151	-0.0059
26	-203,000	-47000	920	1031	6,174,500	-0.0149	-0.0035
27	-269000	-32,000	648	1180	8283,500	-0.0118	-0.0014
28	-342,000	-19,000	37.65	1051	8588,500	-0.0075	-0.0009
29	-415,000	-8,000	12.6	1051	10,803,500	-0.0024	-0.0001
						-0.7508	-1.9987

14250
12340
9870
6790
6720
6720
117170
78000

summary

	CA1 dead load	CA1 live load
	+0.5896	-0.3740
	+0.7508	-1.9987
	+1.3404	-2.3727

CB1
0.5359
0.5624
1.0983

73000

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY ________ DATE ________ FILE NO 6

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CALCULATIONS FOR Dead load Deflection

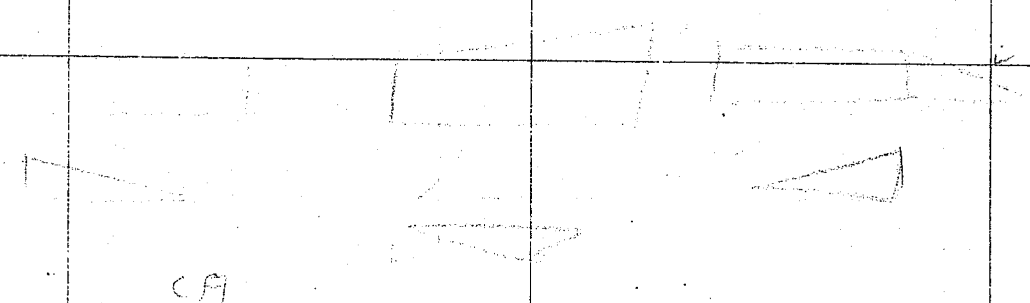
32712

		unit moment		Deflection		unit moment		Deflection			
		at edge of CB1	at cent.					at edge	at edge	at cent.	at cent.
		CB1	BG1	CB1	BG1	CB2&3	BG2&3	CB2	CB3	BG2	BG3
M	1	− 14.6	+ 30.6	−0.0034	−0.0071	20.9	43.8	+ −0.0066	−0.0076	−0.0139	−0.0161
	2	− 46.5	97.3	−0.0119	−0.0251	52.8	110.5	+ −0.0079	−0.0089	−0.0165	−0.0187
	3	− 82.8	173.3	−0.0226	−0.0473	75.7	158.8	+ −0.0124	−0.0142	−0.0261	−0.0298
	4	− 116.8	244.8	−0.0247	−0.0519	108.2	227.9	−0.0293	−0.0330	−0.0616	−0.0695
	5	145.2	303.8	−0.0278	−0.0577	145.2	303.8	−0.0227	−0.0241	−0.0475	−0.0505
	6	178.0	371.9	−0.0223	−0.0467	178.0	371.9	−0.0172	−0.0172	−0.0359	−0.0359
	7	− 205.0	429.5	−0.0098	−0.0205	205.0	429.5	−0.0080	−0.0081	−0.0168	−0.0170
	8	− 228.0	476.8	−0.0054	−0.0112	228.0	476.8	−0.0038	−0.0031	−0.0080	−0.0065
	9	248.0	519.4	−0.0007	−0.0015	248.0	519.4	−0.0003	+0.0017	−0.0006	+0.0034
	10	271.5	567.7	+0.0048	+0.0099	271.5	567.7	+0.0044	+0.0068	+0.0092	+0.0130
	11	293.8	614.4	+0.0105	+0.0219	293.8	614.4	+0.0087	+0.0100	+0.0182	+0.0208
	12	314.0	657.2	+0.0129	+0.0271	314.0	657.2	+0.0118	+0.0147	+0.0247	+0.0207
+M	13	340.8	712.7	+0.0300	+0.0626	340.8	712.7	+0.0257	+0.0292	+0.0536	+0.0610
	14	374.0	782.3	+0.0418	+0.0874	374.0	782.3	+0.0347	+0.0388	+0.0726	+0.0705
	15	407.5	782.3	+0.0467	+0.0898	407.5	782.3	+0.0382	+0.0394	+0.0733	+0.0755
	16	441.0	712.7	+0.0503	+0.0812	441.0	712.7	+0.0409	+0.0419	+0.0615	+0.0674
	17	467.0	657.2	+0.0346	+0.0787	467.0	657.2	+0.0280	+0.0302	+0.0394	+0.0424
	18	489.0	614.9	+0.0322	+0.0404	489.0	614.4	+0.0270	+0.0306	+0.0339	+0.0383
	19	510.0	567.7	+0.0207	+0.0230	510.0	657.7	+0.0196	+0.0236	+0.0252	+0.0303
	20	534.0	519.4	+0.0185	+0.0180	534.0	519.4	+0.0222	+0.0284	+0.0215	+0.0276
	21	554.0	476.8	+0.0074	+0.0064	554.0	476.8	+0.0118	+0.0174	+0.0102	+0.0153
	22	576.0	429.5	−0.0196	−0.0116	576.0	429.5	+0.0021	+0.0062	+0.0015	+0.0044
	23	604.0	371.9	−0.0370	−0.0228	604.0	371.9	−0.0199	−0.0192	−0.0122	−0.0118
	24	638.0	303.8	−0.0781	−0.0372	638.0	303.8	−0.0621	−0.0620	−0.0296	−0.0296
	25	666.0	244.0	−0.0640	−0.0234	666.0	244.0	−0.0678	−0.0766	−0.0248	−0.0281
	26	690.0	192.4	−0.1118	−0.0311	690.0	192.4	−0.0922	−0.1040	−0.0257	−0.0290
−M	27	717.0	135.9	−0.1309	−0.0248	717.0	135.9	+ −0.1090	−0.1227	−0.0206	−0.0232
	28	744.0	78.9	−0.1545	−0.0158	744.0	78.9	−0.1225	−0.1460	−0.0130	−0.0168
	29	769.0	+ 26.3	−0.1484	−0.0050	769.0	26.3	−0.1206	−0.1410	−0.0041	−0.0048
				(−) 0.5624	+0.1057			−0.4272	−0.4688	+0.0879	+0.1033
								+	+		

should be + sign

−.7877
+.3189

−.3873
+.4906



CA
CB
BT

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MADE BY M. Jinda DATE 4-8-22 FILE NO 7

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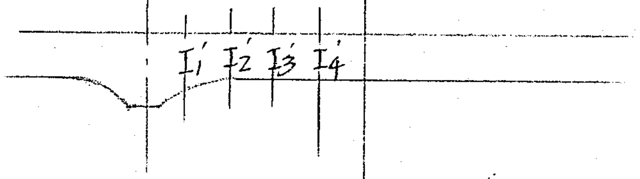
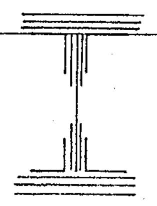
CALCULATIONS FOR

Deflection of cantilever arm due to moment of intermediate span. mark BGCA2



moment of inertia



I_1' .. 3 Cov. Pls + 2600 b. to b. of flg. Ls
4Ls 150×150×19 = 21.356 × $\overline{125.64}^2$ + 4392 = 3,366,400
4Pls 250 × 13 = 130.00 × $\overline{117.0}^2$ + 4060 = 1,784,100
6Pls. 350 × 13 = 274.00 × $\overline{132.0}^2$ = 4,780,000
1Pl. 2590 × 13 = = 1,880,000
11,810,500 = I_1'

I_2' .. 4 Cov. Pls + 2330 b. to b. of flg Ls
4Ls 150×150×19 = 21.356 × $\overline{112.14}^2$ + 4392 = 2,694,400
4Pls 250 × 13 = 130.00 × $\overline{103.5}^2$ + 4060 = 1,395,100
8Pls. 350 × 13 = 364.00 × $\overline{119.1}^2$ = 5,170,000
1Pl. 2320 × 13 = 1,351,000
10,610,500 = I_2'

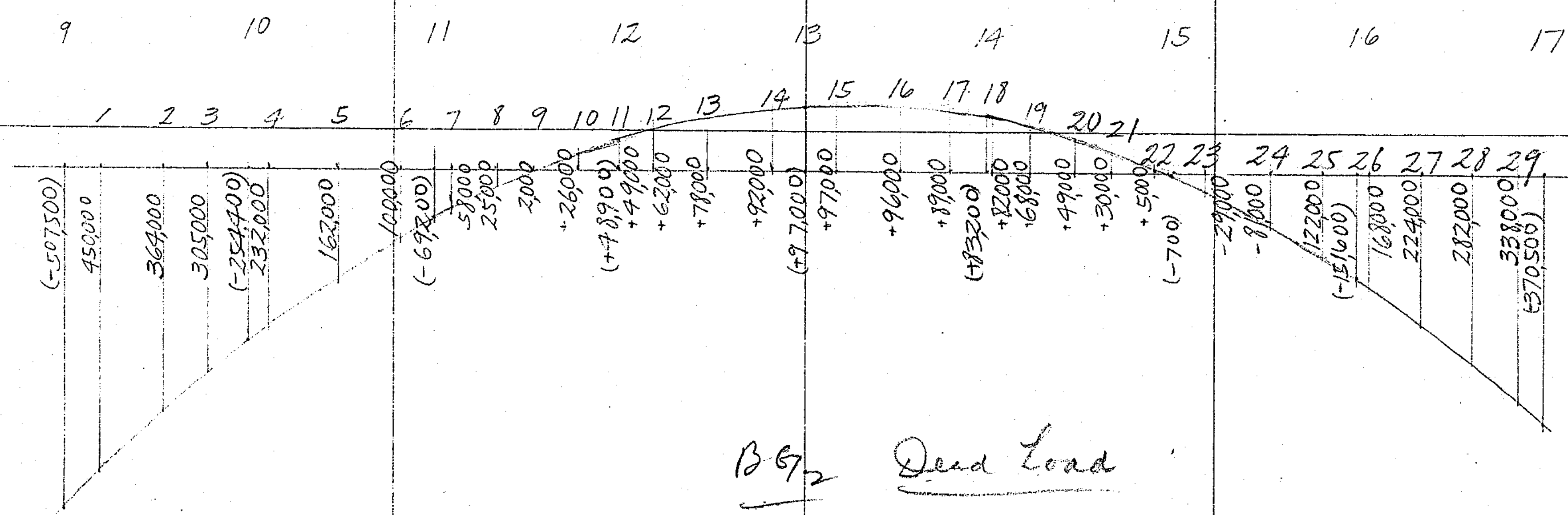
I_3' .. 4 Cov. Pls + 2160 b. to b. of flg. Ls.
4Ls 150×150×19 = 21.356 × $\overline{103.64}^2$ + 4392 = 2,294,400
4Pls 250 × 13 = 130.00 × $\overline{95.0}^2$ + 4060 = 1,176,100
8Pls 350 × 13 = 364.00 × $\overline{110.6}^2$ = 4,450,000
1Pl. 2150 × 13 = 1,077,000
8,997,500 = I_3'

BG2

I_4' .. 3 cov. Pls + 2050 b. to b. of flg. Ls
4Ls 150×150×19 = 21.356 × $\overline{98.14}^2$ + 4392 = 2,030,400
4Pls 250 × 13 = 130.00 × $\overline{89.5}^2$ + 4060 = 1,044,100
6Pls 350 × 13 = 274.00 × $\overline{104.45}^2$ = 2,990,000
1Pl. 2040 × 13 = 919,000
6,983,500 = I_4'

other moment of inertia same as previous sheet.

Dead load moment see design sheet page no. 54



BG2 Dead Load

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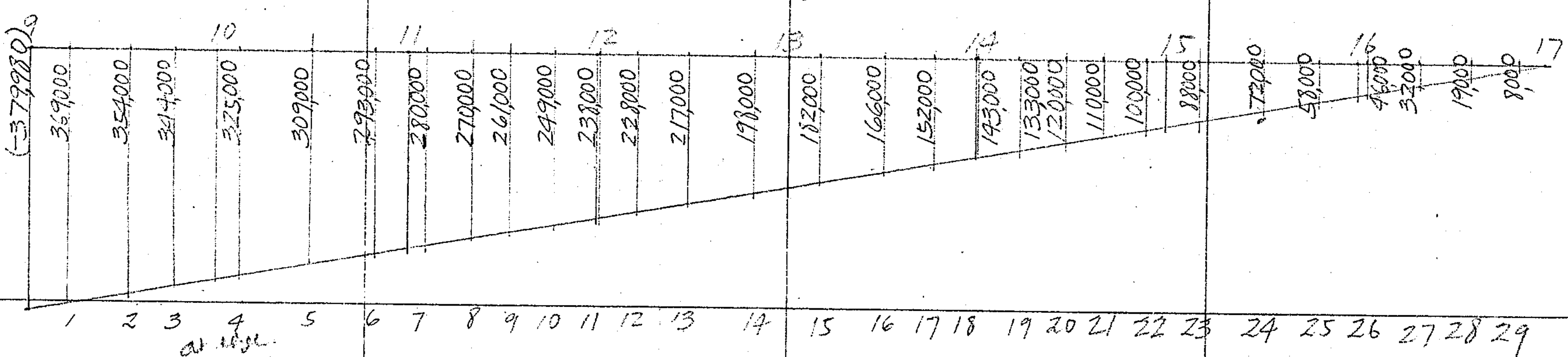
CALCULATIONS FOR

Live load negative moment see design sheet page no. 54



point	at abut. (CA) dead load moment	live load moment	unit moment (m)	dx	I	(CF) $\frac{M d\, m\, dx}{EI}$	$\frac{M l\, m\, dx}{EI}$
1	-450,000	369,000	762.0	175.2	11,810,500	-0.2420	-0.1985
2	-364,000	354,000	729.0	91.5	10,610,500	-0.1089	-0.1059
3	-305,000	344,000	705.0	101.5	8,997,500	-0.1157	-0.1304
4	-232,000	325,000	673.0	171.1	6,983,500	-0.1825	-0.2554
5	-162,000	309,000	638.0	136.3	6,720,400	-0.1000	-0.1908
6	-100,000	293,000	604.0	"	"	-0.0583	-0.1707
7	-58,000	280,000	576.0	95.0	"	-0.0225	-0.1086
8	-25,000	270,000	554.0	"	"	-0.0081	-0.0876
9	-2,000	261,000	534.0	91.5	7,710,400	-0.0006	-0.0786
10	+26,000	249,000	510.0	101.5	"	+0.0073	-0.0796
11	+49,000	238,000	489.0	85.6	6,720,400	+0.0145	-0.0705
12	+62,000	228,000	467.0	"	"	+0.0176	-0.0674
13	+78,000	217,000	441.0	136.3	"	+0.0333	-0.0927
14	+92,000	198,000	407.5	142.2	"	+0.0378	-0.0813
15	+97,000	182,000	374.0	136.3	"	+0.0350	-0.0656
16	+96,000	166,000	340.8	"	"	+0.0316	-0.0546
17	+89,000	152,000	314.0	95.0	"	+0.0188	-0.0321
18	+82,000	143,000	293.8	"	"	+0.0162	-0.0282
19	+68,000	133,000	271.5	91.5	7,710,400	+0.0104	-0.0204
20	+49,000	120,000	248.0	101.5	5,720,400	+0.0103	-0.0252
21	+30,000	110,000	228.0	85.6	"	+0.0049	-0.0179
22	+5,000	100,000	205.0	"	"	+0.0007	-0.0146
23	-29,000	88,000	178.0	136.3	"	-0.0059	-0.0178
24	-81,000	72,000	145.2	149.2	"	-0.0142	-0.0126
25	-122,000	58,000	116.8	103.1	5,898,400	-0.0119	-0.0057
26	-168,000	46,000	92.0	"	6,174,500	-0.0123	-0.0034
27	-224,000	32,000	64.8	118.0	8,283,500	-0.0099	-0.0014
28	-282,000	19,000	37.65	105.1	8,588,500	-0.0062	-0.0004
29	-338,000	8,000	12.6	"	10,803,500	-0.0020	-0.0000
						-0.6626	~~-2.0179~~

summary

	CAZ dead load	live load
	0.5078 ✓	-0.4008
	0.6626 ✓	-2.0179
	1.1704 ✓	-2.4187

CBZ
0.4351
0.4272
0.8623

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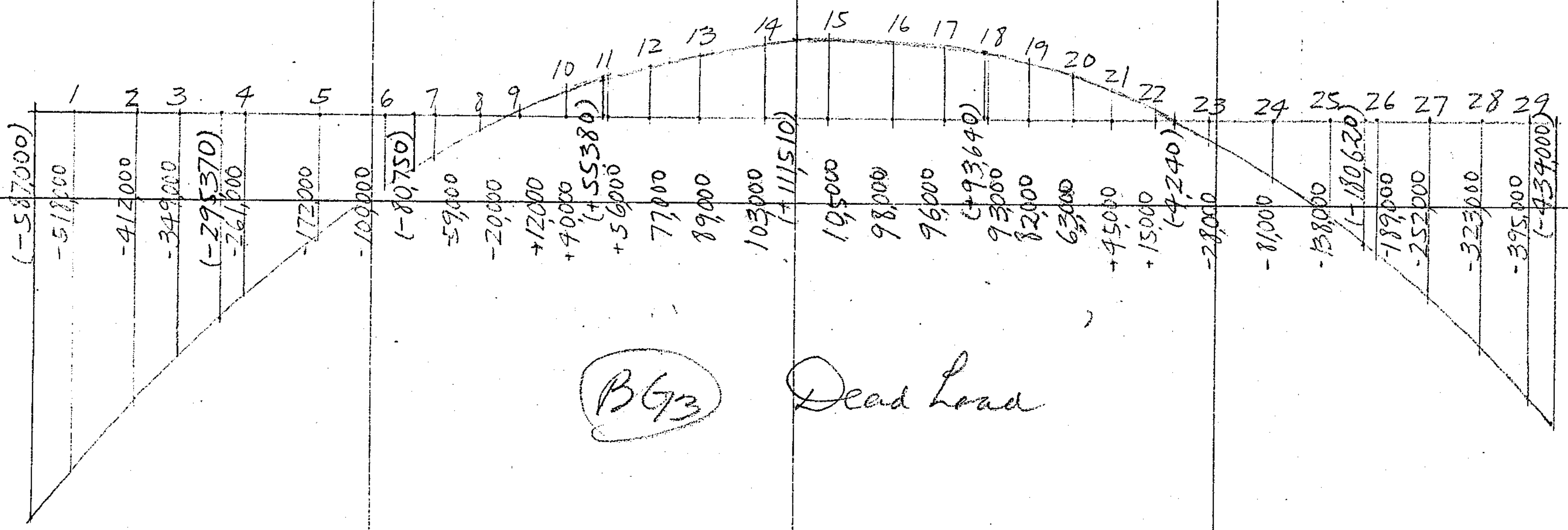
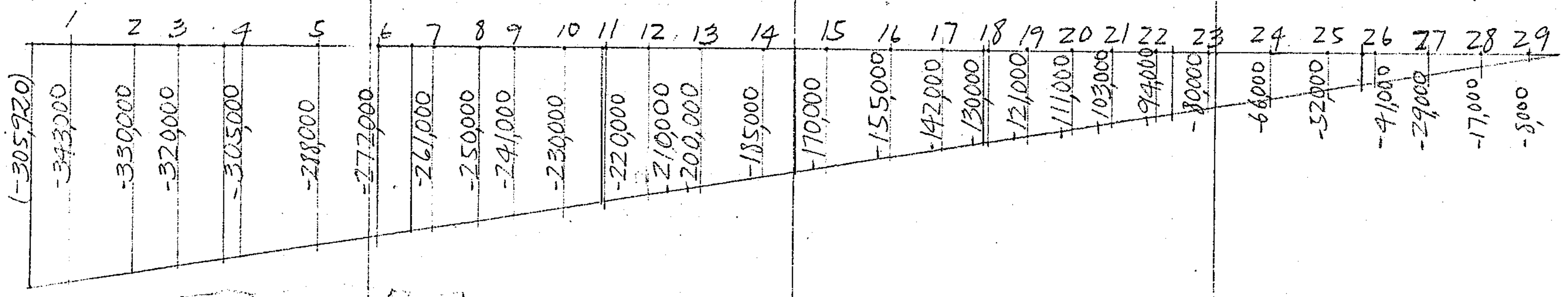
Deflection of cantilever arm due to moment of intermediate span, mark BGCA3
moment of inertia same as previous sheet
Dead load moment see design sheet page no.



Live load negative moment see design sheet page no.



(CA3 dead load)

point	dead load moment	live load moment	unit moment (m)	dx	I	CA $\frac{M_d m\,dx}{EI}$	$\frac{M_l m\,dx}{EI}$
1	-518,000	-343,000	762.0	175.2	11,810,500	-0.2779	-0.1845
2	-412,000	-330,000	729.0	91.5	10,610,500	-0.1230	-0.0984
3	-349,000	-320,000	705.0	101.5	8,997,500	-0.1325	-0.1214
4	-261,000	-305,000	673.0	171.1	6,983,500	-0.2052	-0.2400
5	-172,000	-288,000	638.0	136.3	6,720,400	-0.1062	-0.1778
6	-100,000	-272,000	604.0	〃	〃	-0.0583	-0.1590
7	-59,000	-261,000	576.0	95.0	〃	-0.0229	-0.1012
8	-20,000	-250,000	554.0	〃	〃	-0.0075	-0.0932
9	+12,000	-241,000	534.0	91.5	7,710,400	+0.0036	-0.0726
10	+40,000	-230,000	510.0	101.5	〃	+0.0128	-0.0735
11	+56,000	-220,000	489.0	85.6	6,720,400	+0.0165	-0.0652
12	+77,000	-210,000	467.0	〃	〃	+0.0218	-0.0595
13	+89,000	-200,000	441.0	136.3	〃	+0.0378	-0.0852
14	+103,000	-185,000	407.5	142.2	〃	+0.0423	-0.0760
15	+105,000	-170,000	374.0	136.3	〃	+0.0361	-0.0615
16	+98,000	-155,000	340.8	〃	〃	+0.0323	-0.0511
17	+96,000	-142,000	314.0	95.0	〃	+0.0203	-0.0300
18	+93,000	-130,000	293.8	〃	〃	+0.0184	-0.0256
19	+82,000	-121,000	271.5	91.5	7,710,400	+0.0126	-0.0185
20	+63,000	-111,000	248.0	101.5	5,720,400	+0.0132	-0.0232
21	+45,000	-103,000	228.0	85.6	〃	+0.0071	-0.0168
22	+15,000	-94,000	205.0	〃	〃	+0.0022	-0.0137
23	-28,000	-80,000	178.0	136.3	〃	-0.0057	-0.0162
24	-81,000	-66,000	145.2	144.2	〃	-0.0142	-0.0115
25	-138,000	-52,000	116.8	103.1	5,898,400	-0.0134	-0.0051

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MADE BY M. Juda DATE 9-8-26 **FILE NO** 10

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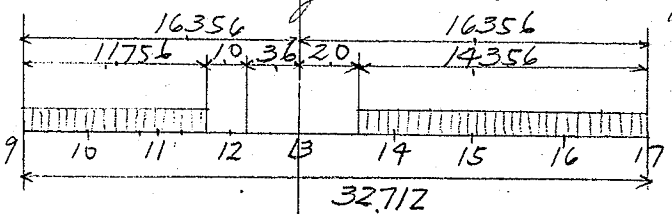
CALCULATIONS FOR

point	deadload moment	live load moment	unit moment (m)	dx	I	Mdmdx / EI	Mlmdx / EI
26	-189,000	-41,000	920	1031	6174,500	-0.0138	-0.0030
27	-252,000	-29,000	648	1180	8283500	-0.0111	-0.0013
28	-323,000	-17,000	37.65	1051	8588,500	-0.0073	-0.0004
29	-395,000	-8,000	126	"	10803500	-0.0023	0
						-0.7243	-1.8854

summary

	CA3 dead load	live load	CB3
	+0.5965	-0.3792	+0.5119
	+0.7243	-1.8854	+0.4688
	+1.3208	-2.2646	+0.9807

Deflection of anchor span for BGCA1
Deflection due to dead load same as for previous design.
Live load moment, loading, reaction + etc see design sheet No. 45.



Reaction at 17
motor truck reaction 5,260 Kgo
uniform load " 15,985 Kgo
unif. load reaction of side walk 27,200
48,445 Kgo

Reaction at 9
motor truck reaction 5,870 Kgo
uniform load " 15,315
unif. load R of side walk 27,200
48,385 Kgo

unif. load
1200 × 11,756 = 14,100 Kgo
1200 × 14,356 = 17,200 Kgo
1200 + 1,665 = 2,865 Kgo

moment at panel point 13
411,000 Kgm.

moment at " " 12
48,385 × 12,267 = 592,500
14,100 × 6,389 = (-) 90,000
1665 × 12267 × 6134 = (-) 125,000
377,500 Kgm

moment at panel point 11
48385 × 8,178 = 387,000
2,865 × 8,178 × 4,089 = (-) 95,700
291,300 Kgm.

moment at panel point 10
48385 × 4089 = 197,800
2,865 × 4,089 × 2,045 = (-) 23,950
173,850 Kgm.

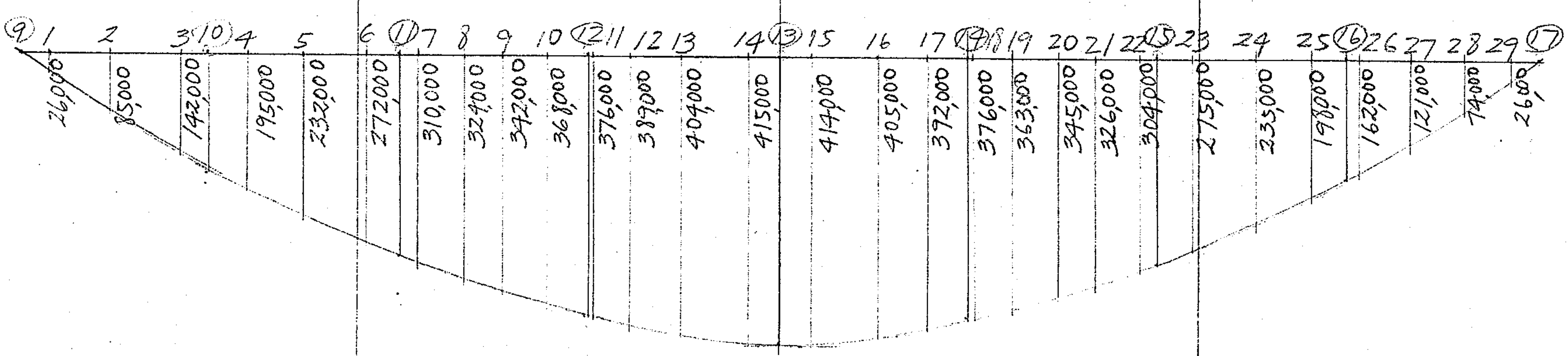
moment at 9 = 0

moment at 14
48,445 × 12,267 = 593,000
2,865 × 12,267 × 6,134 = (-) 214,700
378,300 Kgm.

moment at 15
48445 × 8,178 = 395,500
2865 × 8,178 × 4,089 = (-) 95,700
299,800 Kgm

moment at 16
48,445 × 4,089 = 197,900
2,865 × 4,089 × 2,045 = (-) 24,000
173,900 Kgm.

moment at 17 = 0



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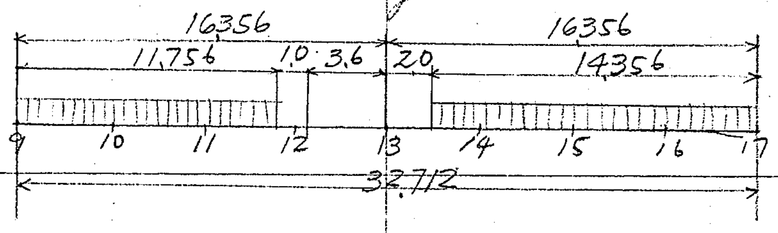
point	live load moment	unit moment (m)	dx	I	$\frac{M\,m\,dx}{EI}$ ✓
1	26,000	30.6	1224	14,252,500	0.000
2	85,000	97.3	1443	12,345,500	0.004
3	142,000	173.3	1596	9,870,500	0.019
4	195,000	244.8	1130	6,790,400	0.038
5	232,000	303.8	1363	6,720,400	0.068
6	272,000	371.9	〃	〃	0.098
7	310,000	429.5	950	〃	0.089
8	324,000	476.8	〃	〃	0.104
9	342,000	519.4	915	7,710,400	0.100
10	368,000	567.7	1015	〃	0.131
11	376,000	614.4	856	6,720,400	0.140
12	389,000	657.2	〃	〃	0.155
13	409,000	712.7	1363	〃	0.278
14	415,000	782.3	1922	〃	0.325
15	414,000	782.3	1363	〃	0.313
16	405,000	712.7	〃	〃	0.279
17	392,000	657.2	950	〃	0.173
18	376,000	614.4	〃	〃	0.155
19	363,000	567.7	915	7,710,400	0.117
20	345,000	519.4	1015	5,720,400	0.151
21	326,000	476.8	856	〃	0.111
22	304,000	429.5	〃	〃	0.093
23	275,000	371.9	1363	〃	0.116
24	235,000	303.8	1442	〃	0.086
25	198,000	244.0	1031	5,898,400	0.040
26	162,000	192.9	〃	6,174,500	0.025
27	121,000	135.9	1180	8,283,500	0.011
28	74,000	78.9	1051	8,588,500	0.003
29	26,000	26.3	〃	10,803,500	0.000

positive Deflection 3.222 = ΣD′

GB 1

Deflection of anchor span for BGCA2
Deflection due to dead load same as for previous design.
Live load moment, loading reaction + etc see design sheet No. 48



Reaction at 17

motor truck loading	4,931
unif. load	15,422
unif. load	4,250
	24,603 kgs.

Reaction at 9

motor truck loading	5,509
unif. load	14,778
unif. load	4,250
	24,537 kgs.

moment at 13
224,400 kgm
1,158 + 260 = 1,418 kgs

moment at 12

24,537 × 12,267 =	300,500
1,158 × 11,756 × 6,389 = (−)	86,800
260 × 12,267 × 6,134 = (−)	19,500
	194,200 kgm

moment at 14

24,603 × 12,267 =	302,000
1,418 × 12,267 × 6,134 = (−)	106,500
	195,500 kgm

moment at 11

24,537 × 8,178 =	200,200
1,418 × 8,178 × 4,089 = (−)	47,400
	152,800 kgm

moment at 15

24,603 × 8,178 =	201,200
1,418 × 8,178 × 4,089 = (−)	47,300
	153,900 kgm

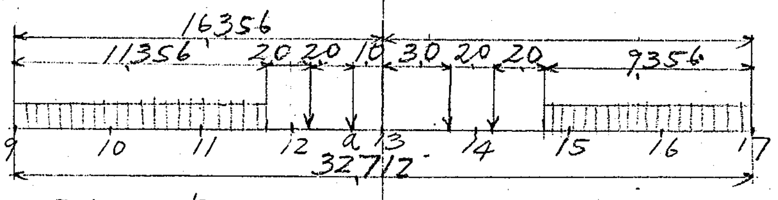
moment at 16

24,603 × 4,089 =	100,700
1,418 × 4,089 × 2,045 = (−)	11,800
	88,900 kgm

moment at 10
24,537 × 4089 = 100,100
1,918 × 4.089 × 2.045 = (−) 11,800
88,300 kgm

moment at 17 = 0
moment at 9 = 0

Live load moment due to Electric car see design sheet No. 50



Reaction at 17
car loading 19,100
impact 21.6% 4,120
unif. load reaction 17,370
40,590 kgs.

Reaction at 9
car loading 16,900
impact 21.6% 3,650
unif. load reaction 18,680
39,230 kgs.

9,000 × 1.216 = 10,920 kgs

moment at a
389,600 ÷ 2 = 194,800 kgm.

moment at 14
40,590 × 12,267 = 498,000
10,920 × 0.911 = (−) 9,900
1,740 × 9,356 × 7,589 = (−) 123,500
364,600 ÷ 2 = 182,300

moment at 12
39,230 × 12,267 = 481,000
1,740 × 11,356 × 6,589 = (−) 130,000
351,000 ÷ 2 = 175,500 kgm

moment at 15
40,590 × 8,178 = 332,000
1,740 × 8,178 × 4,089 = (−) 58,100
273,900 ÷ 2 = 136,950

moment at 11
39,230 × 8,178 = 321,000
1,740 × 8,178 × 4,089 = (−) 58,100
262,900 ÷ 2 = 131,450 kgm

moment at 16
40,590 × 4,089 = 166,000
1,740 × 4,089 × 2,045 = (−) 14,500
151,500 ÷ 2 = 75,750

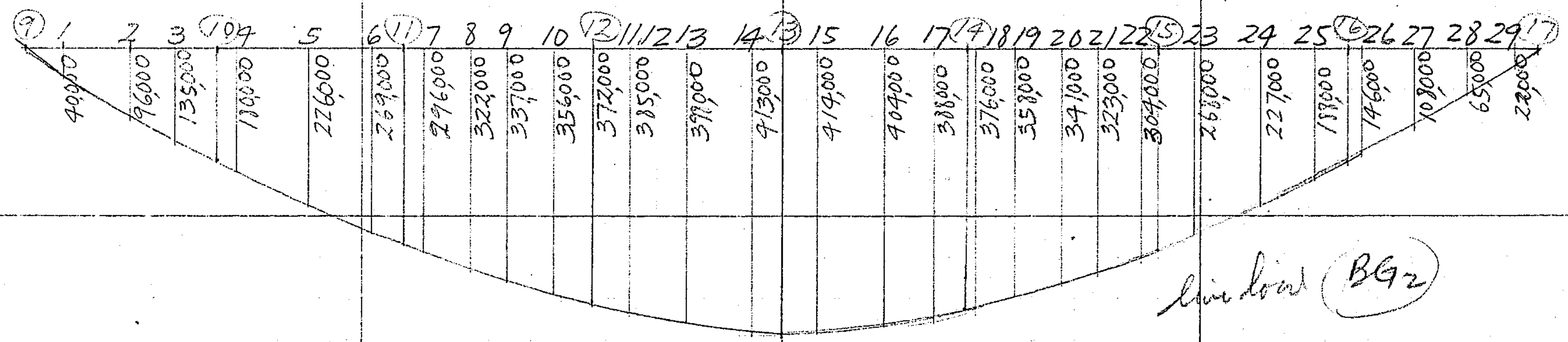
moment at 10
39,230 × 4,089 = 160,500
1,740 × 4,089 × 2,045 = (−) 14,500
146,000 ÷ 2 = 73,000 kgm

moment at 17 = 0

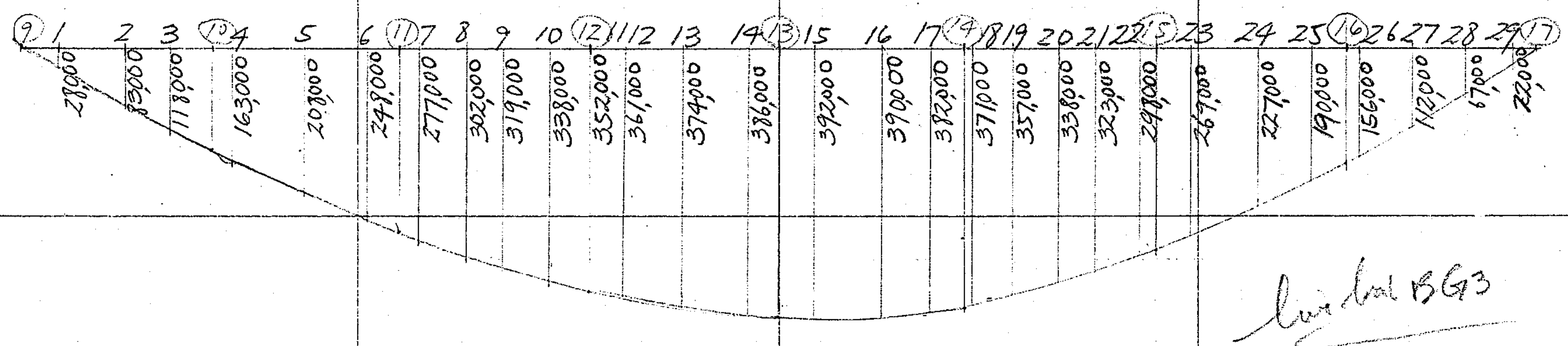
moment at 9 = 0

summary for moment

9	10	11	12	13	14	15	16	17
0	88,300	152,800	194,200	224,400	195,500	153,900	88,900	0
0	73,000	131,450	175,500	194,800 (say)	182,300	136,950	75,750	0
0	161,300	284,250	369,700	419,200	377,800	290,850	164,650	0



live load (BG2)



live load BG3

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MADE BY *M. Jinda* DATE ______ **FILE NO** 13

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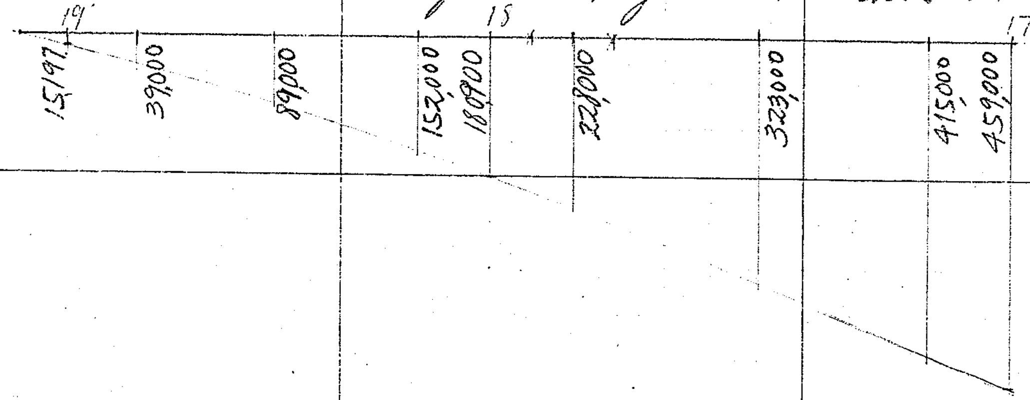
CALCULATIONS FOR

Live load Deflection for BGCA2 & BGCA3

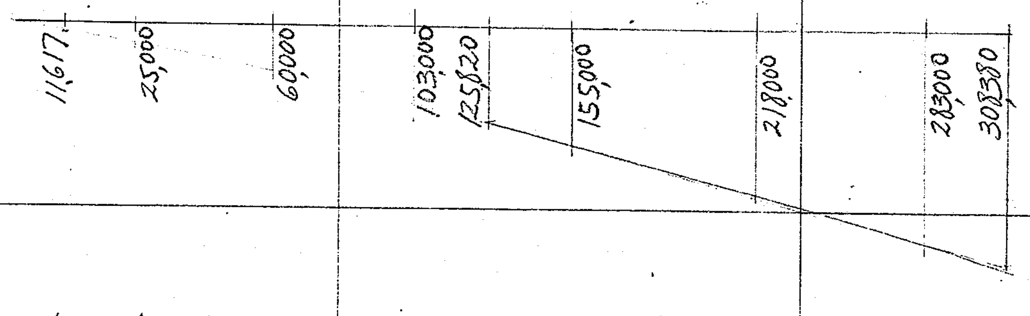
point	live load moment for BGCA1	for BGCA2	(m)	dx	I	BGCA2 $\frac{Mlmdx}{EI}$	BGCA3 $\frac{Mlmdx}{EI}$
1	40,000	28,000	438	175.2	11,810,500	0.001	0.000
2	96,000	83,000	1105	91.5	10,610,500	0.004	0.004
3	135,000	118,000	1588	101.5	8,997,500	0.015	0.010
4	180,000	163,000	2279	117.1	6,983,500	0.033	0.030
5	226,000	208,000	3038	136.3	6,720,400	0.066	0.061
6	269,000	248,000	371.9	"	"	0.097	0.089
7	296,000	277,000	4295	95.0	"	0.085	0.080
8	322,000	302,000	4768	"	"	0.104	0.097
9	337,000	319,000	5194	91.5	7,710,900	0.099	0.094
10	356,000	338,000	5677	101.5	"	0.127	0.121
11	372,000	352,000	6144	85.6	6,720,400	0.138	0.131
12	385,000	361,000	6572	"	"	0.153	0.144
13	398,000	374,000	7127	136.3	"	0.274	0.258
14	413,000	386,000	7823	142.2	"	0.326	0.304
15	414,000	392,000	"	136.3	"	0.313	0.296
16	404,000	390,000	7127	"	"	0.278	0.268
17	387,000	382,000	6572	95.0	"	0.172	0.169
18	376,000	371,000	6149	"	"	0.155	0.153
19	358,000	357,000	5677	91.5	7,710,400	0.115	0.115
20	341,000	338,000	5194	101.5	5,720,400	0.150	0.148
21	323,000	323,000	4768	85.6	"	0.110	0.110
22	304,000	298,000	4295	"	"	0.093	0.091
23	268,000	269,000	3719	136.3	"	0.113	0.113
24	227,000	227,000	3038	144.2	"	0.083	0.083
25	188,000	190,000	2440	103.1	5,898,900	0.038	0.039
26	146,000	156,000	1924	"	6,174,500	0.022	0.024
27	108,000	112,000	1359	118.0	8,283,500	0.010	0.010
28	65,000	67,000	789	105.1	9,588,500	0.003	0.003
29	22,000	22,000	263	"	10,803,500	0.000	0.000

positive Deflection $\Sigma D = 3.177'$ $3.045'$

Deflection of cantilever arm suspended girder 17.106 meter span
~~Dead~~ live load moment see design sheet page no. 41. mark BGCB1



CB1 Dead load



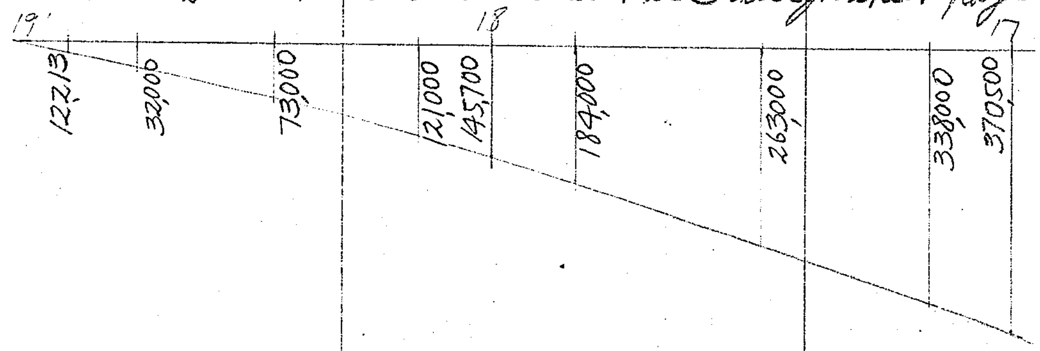
CB1 live load

$I_1' = I_2' = 4,301,900$ $I_5' = 8,093,500$
$I_3' = 4,305,400$ $I_6' = 10,623,500$
$I_4' = 5,379,900$

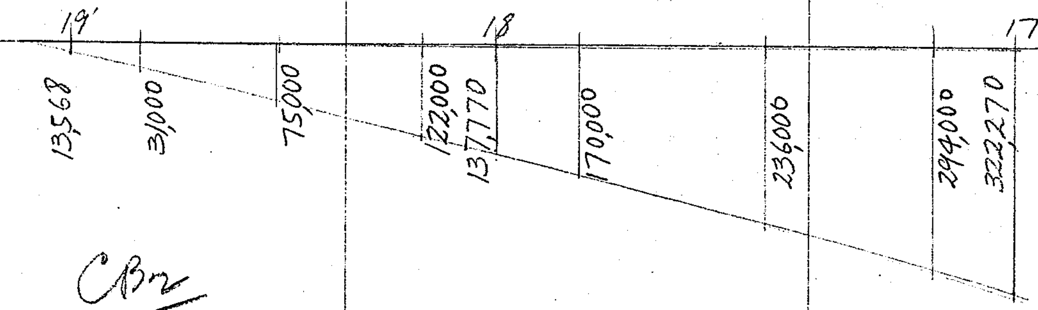
CB1

point	dead load moment	live load moment	unit moment (m)	dx	I	$\frac{M_d m dx}{EI}$	$\frac{M_l m dx}{EI}$
1	39,000	25,000	92.7	154.7	4,301,400	0.0062	0.0040
2	89,000	60,000	202.2	95.0	4,301,400	0.0190	0.0128
3	152,000	103,000	314.0	128.5	4,305,400	0.0676	0.0458
4	228,000	155,000	434.1	135.4	5,379,900	0.1189	0.0805
5	323,000	218,000	581.7	136.4	8,093,500	0.1509	0.1019
6	415,000	283,000	715.1	130.4	10,623,500	0.1733	0.1183
				6)37002 6180		ΣD=0.5359	0.3633

mark BGCB2 Dead & live load moment see design sheet page no. 43.



CB2 Dead Load

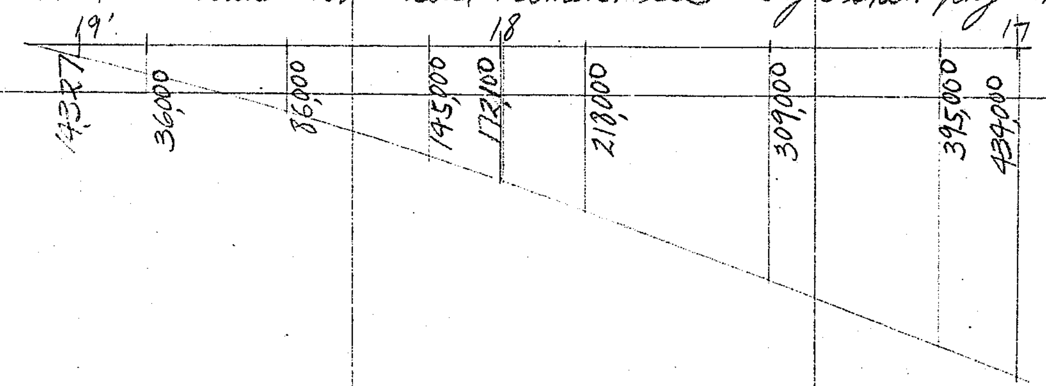


CB2 Live load

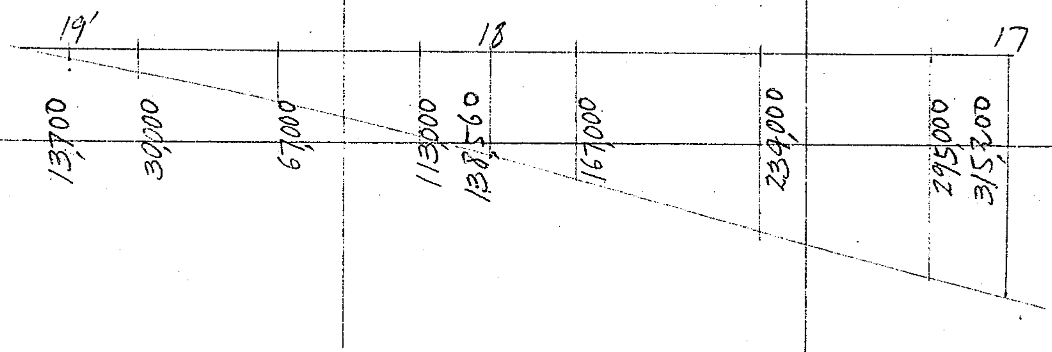
CB2

point	dead load moment	live load moment	unit moment (m)	dx	I	$\frac{M_d m dx}{EI}$	$\frac{M_l m dx}{EI}$
1	32,000	31,000	92.7	154.7	4,301,400	0.0051	0.0049
2	73,000	75,000	202.2	95.0	4,301	0.0156	0.0160
3	121,000	122,000	314.0	128.5	4,305,400	0.0541	0.0545
4	184,000	170,000	434.1	135.4	5,379,900	0.0956	0.0884
5	263,000	236,000	581.7	136.4	8,093,500	0.1232	0.1107
6	338,000	294,000	715.1	130.4	10,623,500	0.1415	0.1232
					Same	ΣD= 0.4351	0.3977

mark BGCB3 Dead & live load moment see design sheet page no. 43



CB3 Dead load



CB3 Live Load

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CALCULATIONS FOR

CB3

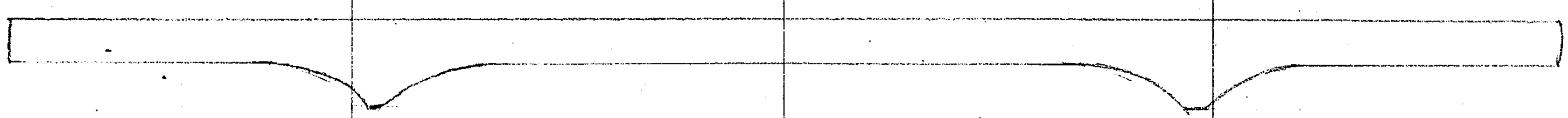
point	dead load moment	live load moment	unit moment (m)	dx	I	MDmdx / EI	MLmdx / EI
1	-36000	30000	-927	154.7	4301400	0.0057	0.0048
2	86000	67000	2022	95.0	"	0.0184	0.0143
3	145000	113000	3140	128.5	4305400	0.0646	0.0503
4	218000	167000	4341	135.4	5379900	0.1132	0.0868
5	309000	234000	5817	136.4	8093500	0.1448	0.1095
6	395000	295000	7151	130.4	10623500	0.1652	0.1235
						ΣD=0.5119	0.3892

~~negative live load deflection at cantilever arm due to intermediate moment~~ span

~~mark BGCB1 = 1581~~
~~mark BGCB2 = 1713~~
~~mark BGCB3 = 1672~~

~~summary~~

~~BGCB1~~		~~BGCB2~~		~~BGCB3~~	
~~dead load~~	~~live load~~	~~dead load~~	~~live load~~	~~dead load~~	~~live load~~
~~-0.5359~~	~~-0.3633~~	~~0.4351~~	~~0.3977~~	~~-0.5119~~	~~0.3892~~
~~-0.7508~~	~~-1.5810~~	~~0.6626~~	~~1.7130~~	~~-0.6985~~	~~1.6720~~
-1.2867	-1.9443	-1.0977	-2.1107	-1.2104	-2.0612



	dead l.	live d.		dead	live(pos)	live(neg)		dead	live
CG1	+1.3404	+3.3312	BG1	+0.1057	+3.222	-3.3521	CB1	+1.0983	+3.4174
CG2	+1.1704	+3.4323	BG2	+0.0879	+3.177	-3.4033	CB2	+0.8623	+3.5033
CG3	+1.3208	+3.1470	BG3	+0.1033	+3.045	-3.1620	CB3	+0.9807	+3.3165

1.0983
3.4174
4.5157

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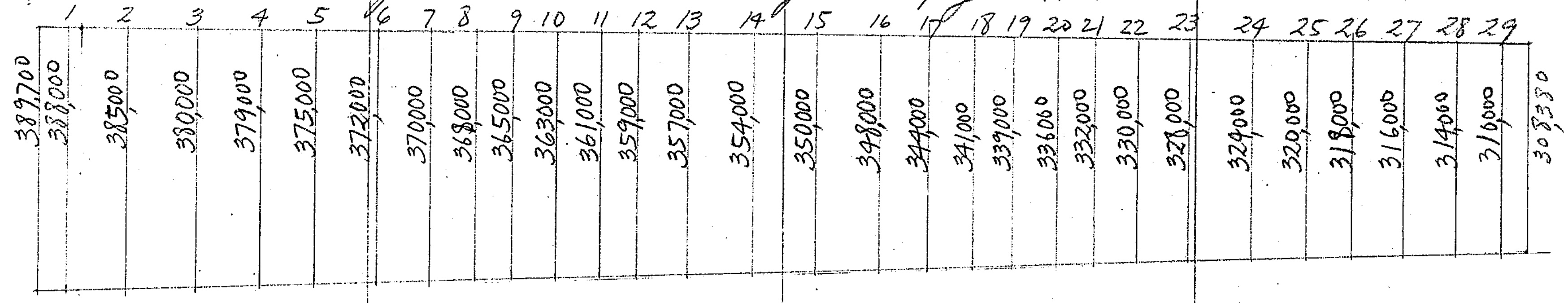
CALCULATIONS FOR

Deflection of live load negative moment see design sheet page no. 45.



point	live load moment	unit moment (m) CA1	CB1	BG1	dx	I	Deflection CA1	CB1	BG1
1	-388,000	767.0	146	30.6	122.4	14,252,500	-0.1220	-0.0023	-0.0048
2	-385,000	735.0	46.5	97.3	144.3	12,395,500	-0.1578	-0.0099	-0.0208
3	-380,000	700.0	82.8	173.3	159.6	9,870,500	-0.2042	-0.0242	-0.0506
4	-379,000	666.0	116.8	244.8	113.0	6,790,400	-0.1995	-0.0350	-0.0733
5	-375,000	638.0	145.2	303.8	136.3	6,720,400	-0.2310	-0.0526	-0.1099
6	-372,000	604.0	178.0	371.9	〃	〃	-0.2172	-0.0640	-0.1338
7	-370,000	576.0	205.0	429.5	95.0	〃	-0.1439	-0.0511	-0.1070
8	-368,000	554.0	228.0	476.8	〃	〃	-0.1371	-0.0564	-0.1181
9	-365,000	534.0	248.0	519.4	91.5	7,710,400	-0.1101	-0.0511	-0.1070
10	-363,000	510.0	271.5	567.7	101.5	〃	-0.1161	-0.0617	-0.1291
11	-361,000	489.0	293.8	614.4	85.6	6,720,400	-0.1070	-0.0642	-0.1344
12	-359,000	467.0	314.0	657.2	〃	〃	-0.1018	-0.0683	-0.1431
13	-357,000	441.0	340.8	712.7	136.3	〃	-0.1520	-0.1198	-0.2459
14	-354,000	407.5	374.0	782.3	192.2	〃	-0.1458	-0.1338	-0.2795
15	-350,000	374.0	407.5	782.3	136.3	〃	-0.1266	-0.1380	-0.2645
16	-348,000	340.8	441.0	712.7	〃	〃	-0.1195	-0.1482	-0.2392
17	-344,000	314.0	467.0	657.2	95.0	〃	-0.0744	-0.1108	-0.1559
18	-341,000	293.8	489.0	614.4	〃	〃	-0.0674	-0.1125	-0.1411
19	-339,000	271.5	510.0	567.7	91.5	7,710,400	-0.0565	-0.1043	-0.1161
20	-336,000	248.0	534.0	519.4	101.5	5,720,400	-0.0704	-0.1515	-0.1472
21	-332,000	228.0	554.0	476.8	85.6	〃	-0.0539	-0.1310	-0.1128
22	-330,000	205.0	576.0	429.5	〃	〃	-0.0482	-0.1354	-0.1010
23	-328,000	178.0	604.0	371.9	136.3	〃	-0.0662	-0.2260	-0.1382
24	-324,000	145.2	638.0	303.8	144.2	〃	-0.0565	-0.2490	-0.1183
25	-320,000	116.8	666.0	294.0	103.1	5,898,400	-0.0311	-0.1776	-0.0650
26	-318,000	92.0	690.0	192.4	〃	6,174,500	-0.0234	-0.1748	-0.0486
27	-316,000	64.8	717.0	135.9	118.0	8,283,500	-0.0139	-0.1536	-0.0291
28	-314,000	37.7	744.0	78.9	105.1	8,578,500	-0.0069	-0.1362	-0.0145
29	-310,000	12.6	769.0	26.3	〃	10,803,500	-0.0018	-0.1108	-0.0038
							-2.9572	-3.0541	-3.3521

summary	CA1 live load	CB1 live load
	-0.3740	-0.3633
	-2.9572	-3.0541
	-3.3312	-3.4174

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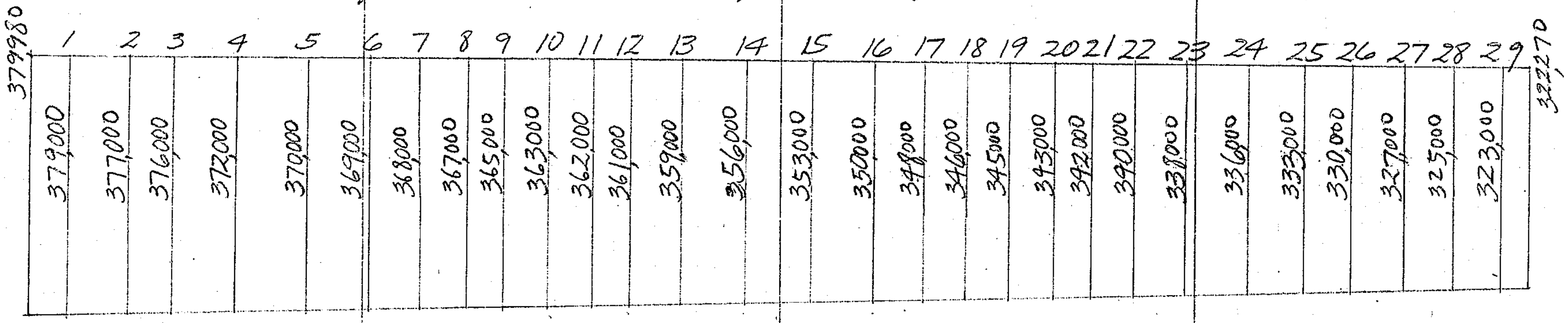
Deflection of live load negative moment see design sheet page no 54. mark CA2, CB2, BG2



point	live load moment	unit moment (m) CA2	CB2	BG2	dx	I	Deflection CA2	CB2	BG2
1	-379000	7620	209	438	1752	11810,500	-0.2040	-0.0056	-0.0117
2	-377000	7290	528	1105	915	10610,500	-0.1128	+0.0081	-0.0171
3	-376000	7050	757	1588	1015	8997,500	-0.1430	+0.0153	-0.0321
4	-372000	6730	1082	2279	1711	6983,500	-0.2920	+0.0471	-0.0990
5	-370000	6380	1452	3038	1363	6720400	-0.2280	+0.0518	-0.1085
6	-369000	6040	1780	3719	〃	〃	-0.2150	+0.0634	-0.1325
7	-368000	5760	2050	4295	950	〃	-0.1428	+0.0507	-0.1062
8	-367000	5540	2280	4768	〃	〃	-0.1368	+0.0563	-0.1179
9	-365000	5340	2480	5194	915	7,710400	-0.1110	+0.0511	-0.1071
10	-363000	5100	2715	5677	1015	〃	-0.1160	+0.0616	-0.1290
11	-362000	4890	2938	6144	856	6720400	-0.1072	+0.0644	-0.1348
12	-361000	4670	3140	6572	〃	〃	-0.1021	+0.0686	-0.1439
13	-359000	4410	3408	7127	1363	〃	-0.1528	+0.1180	-0.2465
14	-356000	4075	3740	7823	1422	〃	-0.1461	+0.1341	-0.2805
15	-353000	3740	4075	7823	1363	〃	-0.1272	+0.1387	-0.2661
16	-350000	3408	4410	7127	〃	〃	-0.1152	+0.1491	-0.2409
17	-348000	3140	4670	6572	95	〃	-0.0736	+0.1094	-0.1541
18	-346000	2938	4890	6144	〃	〃	-0.0684	+0.1140	-0.1431
19	-345000	2715	5100	6577	915	7,710,400	-0.0529	-0.0995	-0.1282
20	-343000	2480	5340	5194	1015	5720400	-0.0726	-0.1564	-0.1519
21	-342000	2280	5540	4768	856	〃	-0.0555	-0.1325	-0.1162
22	-340000	2050	5760	4295	〃	〃	-0.0496	-0.1398	-0.1040
23	-338000	1780	6040	3719	1363	〃	-0.0684	-0.2320	-0.1428
24	-336000	1452	6380	3038	1442	〃	-0.0585	-0.2570	-0.1222
25	-333000	1168	6660	2440	1031	5898,400	-0.0324	-0.1850	-0.0676
26	-330000	920	6900	1924	〃	6174500	-0.0242	-0.1811	-0.0505
27	-327000	648	7170	1359	1180	8283500	-0.0144	-0.1590	-0.0301
28	-325000	377	7440	789	1051	8588,500	-0.0071	-0.1410	-0.0149
29	-323000	126	7690	263	〃	10803,500	-0.0019	-0.1150	-0.0039
							-30315✓	-31056✓	-34033✓

Summary	CA2 live load	CB2 〃
	0.4008	0.3977
	30315	31056
	34323	35033

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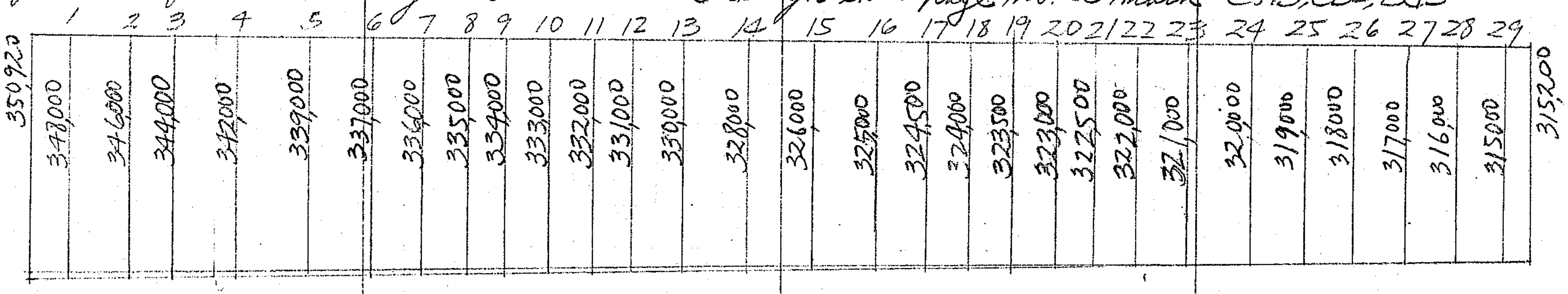
CALCULATIONS FOR

Deflection of live load negative moment see design sheet page no. 55 mark CA3, CB3, BG3



point	live load moment	unit moment (m) CA3	CB3	BG3	dx	I	Deflection CA3	CB3	BG3
1	- 348000	- 7620	209	438	1752	11810500	+0.1875	-0.0051	-0.0108
2	346000	7290	528	1105	915	10610500	-0.1035	-0.0075	-0.0157
3	344000	7050	757	1588	1015	8997500	-0.1305	-0.0141	-0.0294
4	342000	6730	1082	2279	1711	6983500	-0.2690	-0.0433	-0.0910
5	339000	6380	1452	3038	1363	6720400	-0.2090	-0.0476	-0.0995
6	337000	6040	1780	3719	"	"	-0.1968	-0.0580	-0.1210
7	336000	5760	2050	4295	950	"	-0.1305	-0.0463	-0.0970
8	335000	5540	2280	4768	"	"	-0.1250	-0.0514	-0.1075
9	334000	5340	2480	5194	915	7710400	-0.1009	-0.0468	-0.0980
10	333000	5100	2715	5677	1015	"	-0.1065	-0.0566	-0.1185
11	332000	4890	2938	6144	856	6720400	-0.0984	-0.0591	-0.1237
12	331000	4670	3140	6572	"	"	-0.0938	-0.0631	-0.1320
13	330000	4410	3408	7127	1363	"	-0.1409	-0.1089	-0.2275
14	328000	4075	3740	7823	1922	"	-0.1350	-0.1238	-0.2589
15	326000	3740	4075	7823	1363	"	-0.1147	-0.1289	-0.2463
16	325000	3408	4410	7127	"	"	-0.1072	-0.1389	-0.2240
17	324500	3140	4670	6572	950	"	-0.0686	-0.1021	-0.1438
18	324000	2938	4890	6144	"	"	-0.0640	-0.1067	-0.1340
19	323500	2715	5100	6577	915	7710400	+0.0495	-0.0932	-0.1201
20	323000	2480	5340	5194	1015	5720400	-0.0666	-0.1460	-0.1419
21	322500	2280	5540	4768	856	"	-0.0529	-0.1275	-0.1099
22	322000	2050	5760	4295	"	"	+0.0470	-0.1322	-0.0983
23	321000	1780	6040	3719	1363	"	-0.0649	-0.2200	-0.1355
24	320000	1452	6380	3038	1942	"	+0.0558	-0.2454	-0.1168
25	319000	1168	6660	2940	1031	5898400	+0.0310	-0.1773	-0.0649
26	318000	920	6900	1924	"	6174500	-0.0232	+0.1747	-0.0486
27	317000	648	7170	1359	1180	8283500	-0.0139	-0.1542	-0.0291
28	316000	377	7440	789	1051	8588500	-0.0069	-0.1370	-0.0145
29	315000	126	7690	263	"	10803500	-0.0018	-0.1121	-0.0038
							-2.7948	+2.9273	-3.1620

(+)

summary

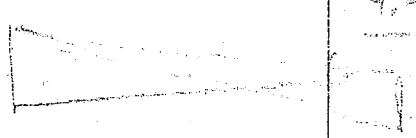
	CA3	CB3
live load		"
	0.3792	0.3892
	2.7948	2.9273
	3.1740	3.3165

Dead load	Live
+ 0.4688	+ 2.927
0.5199	+ .389
0.9887	2.216
	.9887

4.304

4.7
.136
6.06

2.72
1.36



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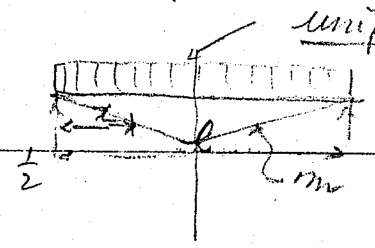
CALCULATIONS FOR

Deflection of Cant. Beam Uniform load Cantilever arm $\delta_{max} = \frac{1}{4}\frac{Ml^2}{EI}$

				DL	LL
CA1	$\frac{1}{4} \times \frac{1,011,70000 \times 780^2}{2,100,000 \times 7,420,000}$	= 0.99	=	.607	+ .383
CA2	$\frac{1}{4} \times \frac{88740000 \times 780^2}{2100,000 \times 6960,000}$	= 0.925	=	.530	+ .375
CA3	$\frac{1}{4} \times \frac{93792000 \times 780^2}{2100,000 \times 6960,000}$	= 0.975	=	0.610	+ .365
				D	L
CB1	$\frac{1}{4} \times \frac{76738000 \times 780^2}{2100,000 \times 6180,000}$	= .900		.540	.360
CB2	$\frac{1}{4} \times \frac{69277,000 \times 780^2}{2100,000 \times 6180,000}$	= .810		.432	+ .378
CB3	$\frac{1}{4} \times \frac{74920,000 \times 780^2}{2100,000 \times 6180,000}$	= .876		.507	+ .369

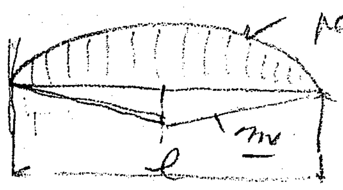
Deflection for uniform moment throughout



uniform moment M

$$\delta = \int_0^l \frac{Mm\,dx}{EI} = 2\int_0^{\frac{l}{2}} \frac{Mm\,dx}{EI}$$

$$= 2\frac{M}{EI}\int_0^{\frac{l}{2}} \frac{1}{2}x\,dx.$$



parabolic curve M.

$$= \frac{M}{EI}\left[\frac{x^2}{2}\right]_0^{\frac{l}{2}} = \frac{M}{EI}\frac{l^2}{8} = \frac{6}{48}\frac{Ml^2}{EI}$$

$$d = \frac{5}{48}\frac{Ml^2}{EI}$$

Anchor span
BG girder between piers Dead Load

BG1
622,000
459 000
1081000 ÷ 2 = 540500
+ 118400
658900

$\frac{5}{48} \times \frac{658900000 \times 3272^2}{2,100,000 \times 7,300,000}$ = 4.790 down
$\frac{6}{48} \times \frac{540500 \times 3272^2}{2,100,000 \times 7,300,000}$ = 4.700 up
average .090 down

BG2
507500
370500
878000 ÷ 2 = 439000
97000
536000

likewise –

3.90 D
3.83 U
0.07 Down

BG3
587000
434000
1021000 ÷ 2 = 518500
111500
627000

4.550
4.500
.050 down

BG girder between piers Live Load

	BG1	BG2	BG3
neg LLm at Center of span	– 349000	– 343640	– 319420
Pos. LLm at Center	411000	+ 410200	+ 389600

Live Load Positive

BG1 $\frac{5}{48} \times \frac{411000000 \times 3272^2}{2100,000 \times 6710,000}$ = 3.24 down

BG2 × M = 3.23

BG3 × M = 3.07

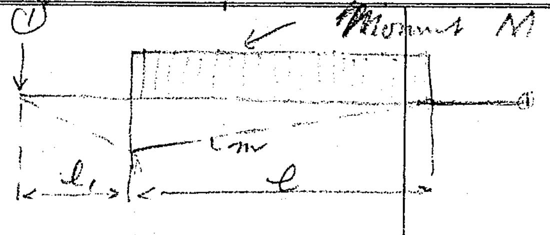
Live Load negative –

$\frac{6}{48} \times \frac{349000000 \times 3272^2}{2100,000 \times 671}$ = 3.32 upward

= 3.26 upward

= 3.03 upward

Deflection of Cantilever arm due to negative moment of Anchor span.



$$d = \int_0^l \frac{M m\,dx}{EI} = \frac{M l_1}{EI}\int_0^l \frac{x}{l}\,dx$$

$m = l_1$

$$= \frac{M l_1}{EI}\left[\frac{x^2}{2l}\right]_0^l$$

$$= \frac{M l_1}{EI}\times\frac{l}{2} = \frac{1}{2}\,\frac{M l_1 l}{EI}$$

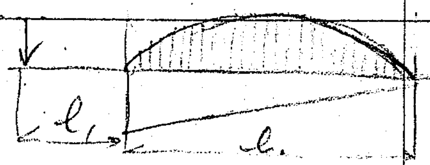
where m = moment due to unit load about support.

both arms fully loaded.
Deflection approximate only.

CA1 or CB1.	$\frac{1}{2}\times\frac{34{,}900{,}000\times3272\times780}{2{,}100{,}000\times7{,}300{,}000}$	= 2.90	cm
CA2 or CB2	$\frac{1}{2}\times$ 34,364,000	= 2.85	"
CA3 or CB3	$\frac{1}{2}$ 31,942,000	= 2.65	"

W

$$d = \int_0^l \frac{M m\,dx}{EI}$$ W Total weight.

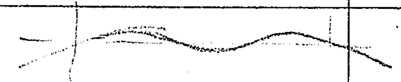


$$M = \frac{W}{2}x\left(1-\frac{x}{l}\right).$$

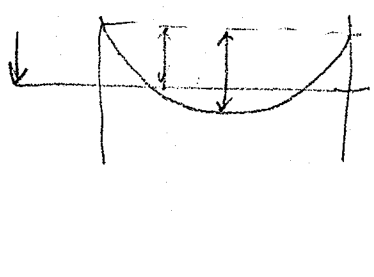
$$m = l_1\frac{x}{l}.$$

$$d = \frac{W l_1}{2EI}\int_0^l x\left(1-\frac{x}{l}\right)\frac{x}{l}\,dx.$$

$$= \frac{W l_1}{2EI}\left\{\int_0^l \frac{x^2}{l}dx - \int_0^l \frac{x^3}{l^2}dx\right\}.$$



$$= \frac{W l_1}{2EI}\left\{\left[\frac{x^3}{3l}\right]_0^l - \left[\frac{x^4}{4l^2}\right]_0^l\right\}$$



$$= \frac{W l_1}{2EI}\left\{\frac{l^2}{3}-\frac{l^2}{4}\right\}$$ max $M = \frac{Wl}{8}$

$$= \frac{W l_1}{2EI}\times\frac{l^2}{12} = \frac{W l^2 l_1}{24EI} = \frac{M l\, l_1}{3EI}.$$

CA1 or CB1.	negative moment down, positive up.	$\frac{1}{2}\times\frac{54{,}050{,}000\times3272\times780}{2{,}100{,}000\times7{,}300{,}000}$ =	4.500	down
		$\frac{1}{3}\times\frac{65{,}890{,}000\times3272\times780}{2{,}100{,}000\times7{,}300{,}000}$ =	3.66	up
			.84 Δ	
CA2 or CB2		$\frac{1}{2}\times$ 43,900,000	3.664	down
		$\frac{1}{3}\times$ 53,600,000	2.98	up
			.680 Δ	
CA3 or CB3.		$\frac{1}{2}\times$ 51,550,000	4.300	down
		$\frac{1}{3}\times$ 62,900,000	3.48	up
			.82 Δ	

CA1 or CB1	$\frac{1}{3}\times\frac{41{,}100{,}000\times3272\times780}{2{,}100{,}000\times7{,}300{,}000}$	=	2.27
CA2 or CB2	41,020,000	=	2.26
CA3 or CB3.	38,960,000	=	2.15

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CALCULATIONS FOR

Summary for deflections.

Dead Load deflection.

CA. BG. CB.

CA₁ .607 D. / .840 / 1.447 — 0.090 down — CB₁ .540 / .840 / 1.380 cantilever due to neg. mom. BG

CA₂ .530 / .680 / 1.210 — 0.070 down — CB₂ .432 / .680 / 1.112

CA₃ .610 / .820 / 1.430 — 0.050 down — CB₃ .507 / .820 / 1.327

Live Load.

CA₁ .383 / 2.900 down / 3.283 — 2.27 up — down — 3.24; up — 3.32 — 2.27 up — CB₁ .360 / 2.900 down / 3.26

CA₂ .375 / 2.850 down / 3.225 — 2.26 up — down — 3.23; up — 3.26 — 2.26 up — CB₂ .378 / 2.850 down / 3.228

CA₃ .365 / 2.650 down / 3.015 — 2.15 up — down — 3.07; up — 3.03 — 2.15 up — CB₃ .369 / 2.650 down / 3.019

Dead & Live Load Deflection

CA₁ 1.447 / 3.283 / 4.730 — 2.72 — BG₁ 3.24 / 0.09 — D / 3.33 — CB₁ 1.380 / 3.260 / 4.640 — 124 – 5.88 1/556

CA₂ 1.210 / 3.225 / 4.435 — 2.90 — BG₂ 3.23 / 0.07 / 3.30 — CB₂ 1.112 / 3.228 / 4.340 — 116 – 5.50 1/593

CA₃ 1.430 / 3.015 / 4.445 — 2.66 — BG₃ 3.070 / 0.050 / 3.12 — CB₃ 1.327 / 3.019 / 4.346 — 126 – 5.606 1/556

one span fully loaded

CA₁ 1.447 / 1.833 / 3.280 / 1.640 / 2.720 / 4.360 ← 1/750 — BG₁ – 3.33 1/980 — CB₁ 1.380 / 1.810 / 3.190 / 1.240 / 4.430 — 1/738

CA₂ 1.210 / 1.800 / 3.010 / 1.505 / 2.900 / 4.405 ← 1/741 — BG₂ 3.30 1/990 — CB₂ 1.112 / 1.803 / 2.915 / 1.160 / 4.075 — 1/800

CA₃ 1.430 / 1.690 / 3.120 / 1.560 / 2.660 / 4.220 ← 1/771 — BG₃ – 3.12 1/1048 — CB₃ 1.327 / 1.694 / 3.021 / 1.260 / 4.281 — 1/762

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CALCULATIONS FOR

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昭和五年七月

大阪府十三橋比較設計予算書

人道橋及電車人道並用橋比較設計

電車橋設計及予算

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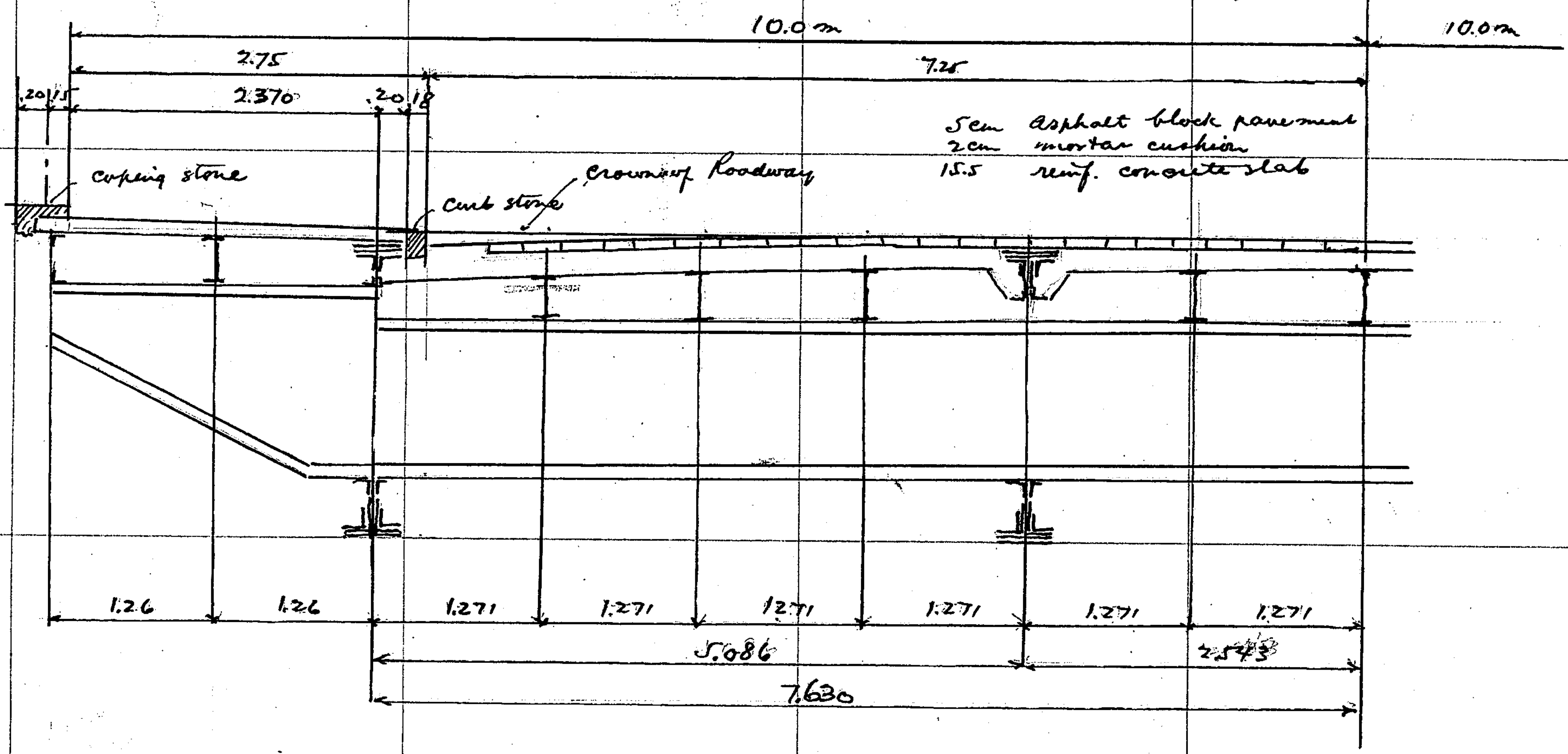
CALCULATIONS FOR

Design of Jiiso-Bashi (Highway Loading).

Girder span over Nagara - Canal.
span length 6@ 4.089 = 24.534 meters

Cross Section of bridge.



Roadway slab. span length 1.271 meters
Dead Load assumed

5 cm	asphalt block pavement	@ 21	=	105
2 cm	mortar cushion	@ 22	=	44
15.5 cm	slab	@ 24	=	372
misc. load	say			9
				530 kg per lin. meter

Dead Load moment $\frac{1}{8} \times 530 \times 1.271^2 = 107.$ kgm

Live Load motor truck loading

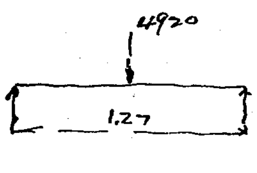
Rear wheel concentration 4500
impact 30% 1350
5850 kg

Front wheel $5850 \div 3 = 1950$ kg

Distribution longitudinal a 34 cm
transverse b 39+14 = 53 cm

Effective width $E = \frac{2}{3}l + a$
$= \frac{2}{3} \times 1.271 + .34 = 1.19$ meters

Load per meter strip for rear wheel $= \frac{5850}{1.19} = 4920$ kg



Live Load moment $= \frac{4920}{2} \times .635 = 1560$
for continuity of slabs $1560 \times 0.8 = 1250$ kgm
Dead Load 107
1357 kgm

$d = \sqrt{\frac{1357 \times 100}{100 \times 7.18}} =$ 13.80
15.50
1.7 insulation at bottom of slab.

Details same as for original design.

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CALCULATIONS FOR

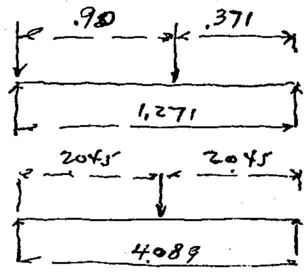
Design of Jinso-Bashi for Highway Loading.

Stringer span length 4.089 meters Spacing 1.271 meters under driveway
Dead Load

Floor load $530 \times 1.271 = 675.$
Stringer assumed 74
749 kg

Dead Load Moment $= \frac{1}{8} \times 749 \times 4.089^2 = 1570$ kgm

Live Load motor truck rear wheel with impact 5850
front " " " 1950
Uniform live load 600 kg per square meter



Reaction $5850 \times \frac{.371}{1.271} = 1710$
5850
7560

LL moment $= \frac{7560}{2} \times 2.045 = 7720$ kgm
Dead Load Moment $= 1570$
9290 kgm

Section modulus required $= \frac{929000}{110} = 845$
300 × 150 I @ 58.54 kg section modulus = 870.6
or 14" × 6" I @ 68.5 " " 1030.0 about.

Use same stringer as for original design.

Cross beam and Cantilever Bracket
Same as for original design

Approximate weight of Cross Frame complete

Cantilever Brackets 2 @ 400 = 800
Roadway Cross beam 3 @ $692 \times \frac{5.086}{4.735}$ = 2220
3020 per panel point.

Approximate weight of stringers
Fascia stringer 2 @ 63.5 = 127
stringer under sidewalk 2 @ 52.0 = 104
stringer under Roadway 9 @ 74.0 = 666
897 kg per lin. meter
Cantilever & Cross Beams 3020 ÷ 4.089 = 740
1637 kg per lin. meter

Bottom Lateral Bracing
Under Roadway 1117 ÷ 4.089 = 273 kg
Under sidewalk 20
293 kg.

Same as for original design.

main girder G_1 and G_2
Dead Load on main girders.
On G_1 see page 17
Cantilever load 7160
11028 ÷ 5.086 = 2170
9330 ÷ 4.089 = 2280

Floor load 530
80
$610 \times \frac{5.086}{2}$ = 1550
Direct on girder assumed 150
Lateral Bracing say 56
Girder assumed 1000
5036 kg per lin. meter

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Design of Jiuso-Bashi for highway Loading

On G_2

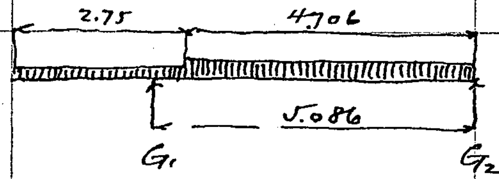
Floor	610 · 5.086 =	3100
Direct load say		180
Lateral Bracing		90
main girder assumed		1000
		4370
Less reaction	2170 ÷ 4.089 =	−530
		3840 kg per lin. meter

Dead Load moment $G_1 = \frac{1}{8} \cdot 5056 \cdot 24.534^2 = 380.000$ kgm

$G_2 = \frac{1}{8} \cdot 3840 \cdot 24.534^2 = 288.000$ kgm
4370 328 000

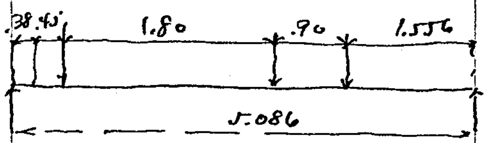
Live Load
500 kg per sq meter on sidewalk
600 " " " " Roadway
motor truck rear wheel concentration 4500
impact 23.5% 1060
5560 kg
front wheel 5560 ÷ 3 = 1855 kg.

Load on G_1



Uniform load on sidewalk 2.75 · 500 = 1375
Load on G_1 $1375 \cdot \frac{6.081}{5.086} = 1645$ kg

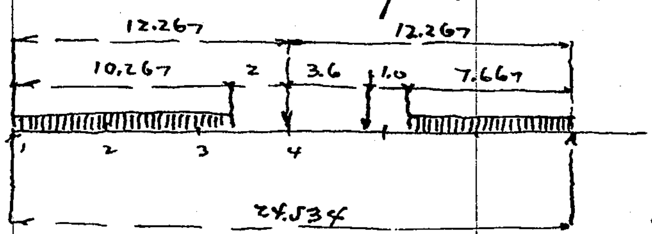
Uniform load on Roadway 4.706 · 600 = 2825
Reaction on G_1 $2825 \cdot \frac{2.353}{5.086} = 1310$
Reaction on G_2 1310
1515



motor truck loading Rear wheel $5560 \cdot \frac{8.668}{5.086} = 9500$ kg

Front wheel 9500 ÷ 3 = 3170 .

moment at Center of Span



motor truck loading
Reaction $\frac{9500}{2}$ = 4750
$3170 \cdot \frac{8.667}{24.534}$ = 1120
5870 kg.

moment 5870 · 12.267 = 72.000 kgm

Uniform load on Roadway 1310 kg per lin. meter

10.267 · 1310 = 13450
7.667 · 1310 = 10000

Reaction $13450 \cdot \frac{19.40}{24.534}$ = 10650
$10.000 \cdot \frac{3.833}{24.534}$ = 1560
12210 kg.

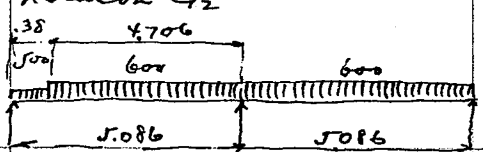
Summary moments

motor truck	72.000
Roadway Unif.	54000
Sidewalk Unif.	123500
	249.500 kgm

moment 12210 · 12.267 = 150.000
13450 · 7.134 = −96.000
54.000 kgm

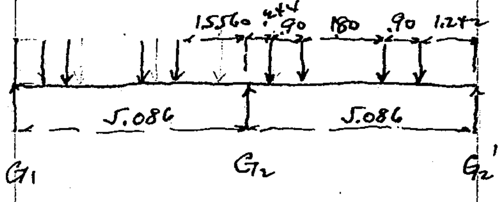
Uniform load on sidewalk 1645 kg per lin. meter
moment $\frac{1}{8} \cdot 1645 \cdot 24.534^2$ = 123,500 kgm

Load on G_2



Uniform load on sidewalk $500 \cdot \frac{.38 \cdot .19}{5.086} = 7.1$

Uniform load on Roadway $600 \cdot \frac{\cancel{9.742} \; 4.706 \cdot \cancel{4.646} \; 2.733}{5.086} = \cancel{5650}$ 1520
1527



motor truck loading
1st Span Reaction on G_2 3@5560 = 16680 − 9500 = 7180
2nd Span $5560 \cdot \frac{12.168}{5.086}$ = 13300
20480 kg

Front wheel 20480 ÷ 3 = 6820 kg

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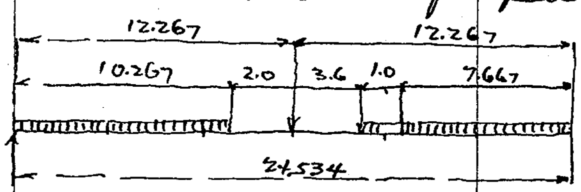
CALCULATIONS FOR

Design of Juso-Bashi for Highway Loading

Moment at center of span — motor truck



Reaction 20480 ÷ 2 = 10240

6820 × $\frac{8.667}{24.534}$ = 2410

12650

moment 12650 × 12.267 = 155 000

10.267 × 1527 = 1565

7.667 × 1527 = 1170

moment due to unif. load 54000 × $\frac{1527}{1310}$ = 62800

217800 kgm

Summary moments

motor truck loading 155000

Uniform live load 62800

217800 kgm

For Girder G2 neglecting negative reaction

DL. moment = ⅛ · 4370 × 24.534² = 328000 kgm

Summary moments for main Girders

	G1 girder	G2 girder
DL. moment	380.000	328000
LL "	249500	217800
	629500	545800 kgm

web assumed 2000 × 13 = 260.0 cm² ⅛ web = 32.5 cm²

Girder G1 flange stress = $\frac{629500}{1.942}$ = 324000 SR = $\frac{324000}{1200}$ = 270.0 − 32.5 = 237.5 net

Girder G2 flange stress = $\frac{545800}{1.92}$ = 284500 SR = $\frac{284500}{1200}$ = 238.0 − 32.5 = 205.5

Considering deflection of girder some allowance is taken into account.

Flange sections G1

2L^s 150 × 150 × 19	=	106.78	−	19.0	=	87.78
2Pls. 250 × 13	=	65.00	−	13.0	=	52.00
3Pls. 350 × 13	=	136.50	−	19.5	=	117.00
		308.28				256.78 net

Flange section G2

2L^s 150 × 150 × 19	=	106.78	−	19.0	=	87.78
2Pls. 250 × 13	=	65.00	−	13.0	=	52.00
2Pls. 350 × 13	=	91.00	−	13.0	=	78.00
		262.78				217.78 net

Girder G3 in original design = 20.4 tons to be reduced in weight of main girders

5 girders 99800

G3 girder 20400

79400 kg

Summary for structural Steel in Girder span

Stringers	897.0 × 25.07	= 22500
Cantilever + Cross Beam	3020 × 7	= 21140
Lateral Bracings		6700
"		500
main girders complete		79400
shoes		4000
Expansion Joints etc		1500
		135,740 kg.

135,740 ÷ 156840 = 86.5%. 13.5%.

Reduction of Steel 2557 tons × 13.5% = 345.0 tons

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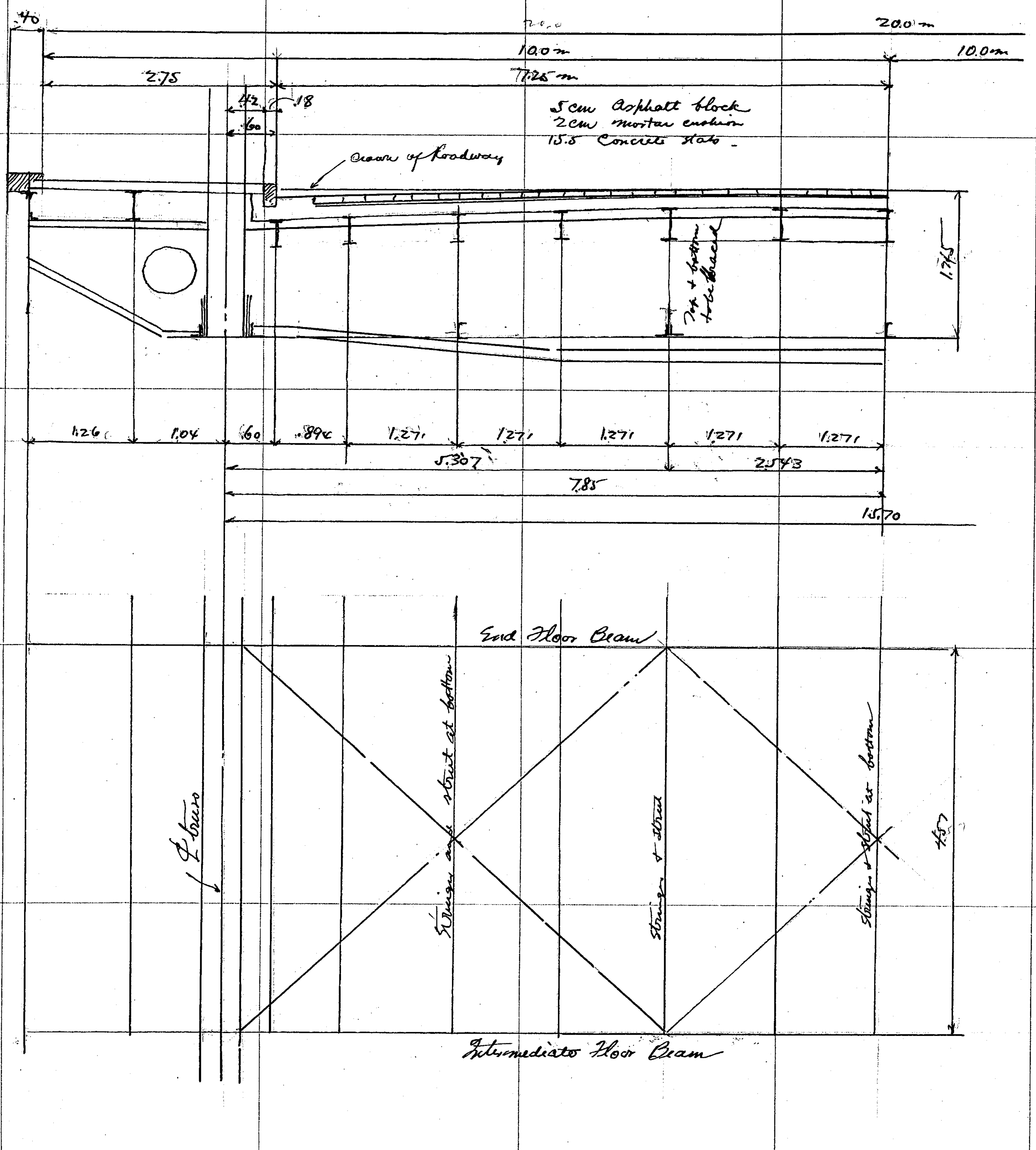
CALCULATIONS FOR

Design of Juso-Bashi for Highway Loading

Truss span span length 64.008 meters between end bearings 14 panels @ 4.57 m

Cross Section of truss Span



Design of Jiu-so-Bashi for Highway Loading

Stringers under sidewalk and roadway
under sidewalk fascia 57.0 kg per meter with details
stringer intermediate 300 × 150 I @ 48.34 kg
Under roadway 14" 6" I @ 68.5 kg
Details of sidewalk same as for original design
" " roadway 15.5 cm reinforced concrete slab throughout.

Design of Cantilever Bracket same as original design.

	Moment		Shear
Dead Load moment	10586	shear	7518
Live Load moment	6000	"	4410
	16586 kgm		11928 kg.

Approximate weight 405 kg for both cantilever . 810 kg.

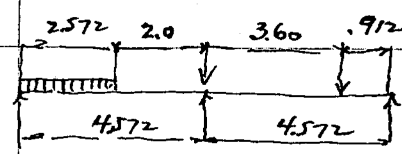
Design of intermediate floor beam. span length 15.70 meters Spacing · 4.572 meters
Dead Load

Floor slab and pavement 530
stringers $\frac{74}{1.27}$ = say 60
590 kg per square meter

misc floor load say $\frac{10}{600}$ × 4.572 = 2740
Girder & bracing assumed 660
3400 kg per lin. meter

Dead Load moment $\frac{1}{8}$ · 3400 · 15.70² = 105000 kgm
– Cantilever moment 10590
94410 kgm

Live Load motor truck loading rear wheel with impact say 5850 kg
front " " " 1950 "



Uniform load 600 kg per square meter

moment at center of span see page 83. 148570 kgm

Summary for moments

Dead Load 94410
Live Load 148570
242980 call this 243000 kgm

Back to back of flange L^s 1.760 web 1.750 × 9 = 157.5 cm² $\frac{1}{8}$ web = 19.7 cm²
flange stress = $\frac{243000}{1.73}$ = 140500 section required = $\frac{140500}{1200}$ = 117.0
– 19.7
97.3 cm² net

2L^s 150 × 150 × 15 = 85.5 – 15.0 = 70.50
1 Pl. 350 × 10 = 35.0 – 5.0 = 30.00
100.50 cm² net.

Approximate weight of intermediate floor beam
Original design 7779
less – 779
7000 kg.

weight of floor beam and cantilever brackets
floor beam 7000
Cantilevers 810
7810 kg.

End floor beams and cantilever brackets without shoe 3400 kg

Design of Jūso-Bashi for Highway Loadings

Bottom Lateral Bracings

longitudinal strut	3 @ 228	=	683
	2 @ 500	=	1000
			1683

Summary for Bottom Lateral Bracings

Diagonals	6 @ 1741	=	10450
"	8 @ 1520	=	12180
struts	14 @ 1683	=	23600
			46230
Bracing under sidewalk			500
			46730 kg.

46730 ÷ 64 = 730 kg per lin. meter.

Top Lateral Bracings

Sways	9 @ 3600	=	32400
Portals	2 @ 5560		11120
longl struts	10 @ 419		4190
diagonals.	40 @ 410		16400
misc say			890
			65000 kg.

65.000 ÷ 64 = 1016 kg per meter

Dead Load structural Steel

stringers	S_1	2 × 57.0	=	114	
	S_2	2 × 50.0	=	100	
		13 × 74.0	=	960	
				1174	
Intermediate floor beam		7810 ÷ 4.57	=	1710	4630
Bottom Lateral Bracings				730	
Top Lateral Bracings				1016	
trusses assumed					6500
					11130

Floor load.

Handrail	2 @ 90	= 180	
Coping stone and slab under	2 @ 206	= 412	
Sidewalk slab + wearing course	300 × 2.42 × 2	= 1450	
Curb stone + concrete wall etc		540	
Roadway slab.	530 × 14.5	= 7680	
		10262	10262
			21392 say 21400

for one truss 21400 ÷ 2 = 10700 kg.

$\frac{10700}{13150}$ = 81.5 %.

Live Load

Uniform live load on Roadway = $\frac{120.000}{170+64}$ = 513 kg/m²

" " " " Sidewalk = $\frac{100.000}{170+64}$ = 428 "

motor truck loading Rear wheel Conc. 4500

impact = $\frac{20}{60+64}$ = 16.1 = 730

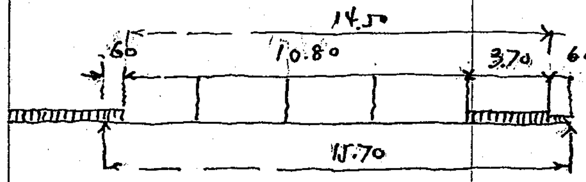
5230 × 2 = 10460

front wheel with impact 5230 ÷ 3 = 1750

2 @ 1750 = 3500

Design of Juso-Bashi for Highway Loading

Uniform load



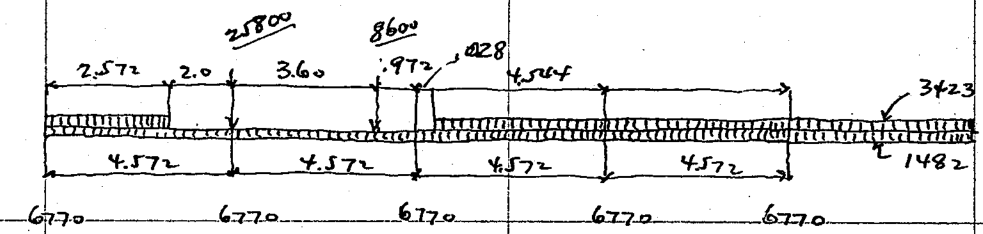
Sidewalk 428 × 2.75 = 1180
Roadway 513 × (3.70 × 2.45)/15.70 = 297
Sidewalk 428 × (.60 × .30)/15.70 = 5
1482 in case of full load
1482 × 4.57 = 6770

motor truck loading 10460 × 4 × 9.70/15.70 = 25800 kg.

front wheel 25800 ÷ 3 = 8600 kg.

Full Uniform load Sidewalk 1180
5
Roadway 513 × 7.25 = 3720
4905 kg.

Panel concentration 4905 × 4.57 = 22400



4905
1482
3423 × 2.572 = 8800
3423 × 4.544 = 15600

25800
1830 — — 6770
2470 7750 - - - 7850
7840 - - - 15680
36870 21290 22460 22450
22450
14420

Assume panel load 22450 throughout and extra load 14000 at panel point for motor truck loading.

middle chord

4 Pls. 660 × 15 = 396.0
4 Ls 150 × 150 × 19 = 213.56
2 Pls. 360 × 19 = 137.00
746.56

2 Pls. 660 × 15 = 198.0
4 Ls 150 × 150 × 15 = 171.0
369.0

2 Pls. 360 × 15 = 108.0
477.0

4 Pls. 660 × 15 = 396.0
4 Ls 150 × 150 × 15 = 171.0
567

2 Pls. 360 × 15 = 108
675.0

Diagonal

4 Ls 150 × 150 × 15 = 171.0
30.0
141.0

4 Ls 150 × 150 × 11 = 127.16
22.
105.16

Vertical

4 Ls 150 × 100 × 9 = 86.76 − 18.0 = 68.76
1 Pl. 410 × 9 = 36.90 − 4.5 = 32.40
123.66 101.16 net

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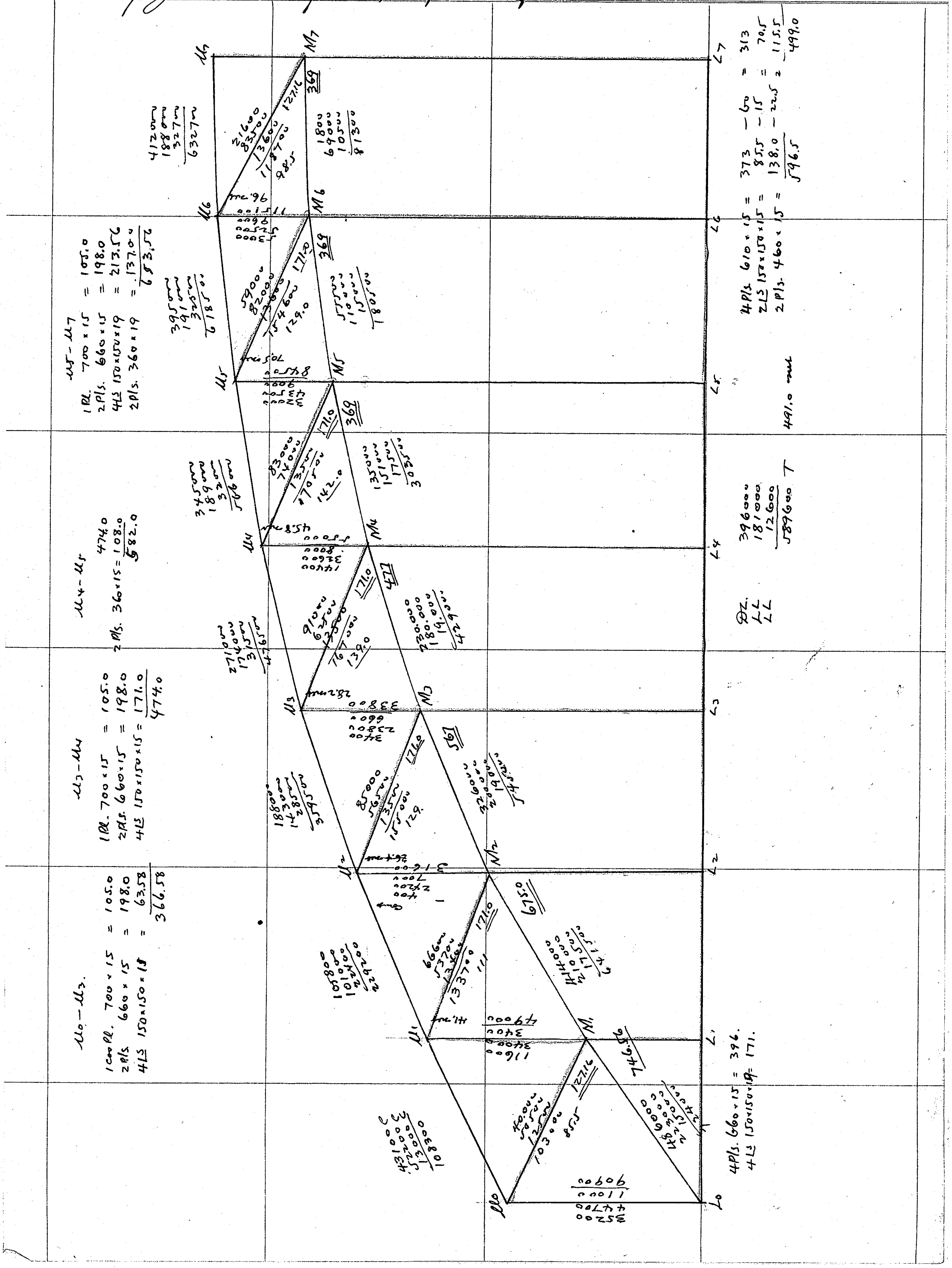
CALCULATIONS FOR

Design of Jiuso-Bashi for Highway Loading.



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

CALCULATIONS FOR

Design of Jiuso Bashi for Highway Loading.

Approximate weight of truss.

	member	section	length		
	L_0-U_0	366.58 @ .785 ×	5.335	= 1540	
	U_0-U_1	"	5.138	1480	
Top chord	U_1-U_2	"	4.956	1430	
	U_2-U_3	"	4.819	1390	
	U_3-U_4	474.0	4.717	1750	
	U_4-U_5	582.0	4.644	2120	
	U_5-U_6	653.56	4.598	2360	
	U_6-U_7	"	4.575	2340	14410
	L_0-M_1	746.56	5.622	3300	
	M_1-M_2	675.0	5.253	2780	
Middle	M_2-M_3	567.0	4.995	2220	
Chord	M_3-M_4	477.0	4.814	1800	
	M_4-M_5	369.0	4.691	1360	
	M_5-M_6	"	4.614	1330	
	M_6-M_7	"	4.577	1320	14110
	U_0-M_1	127.16	5.022	500	
	U_1-M_2	171.0	4.928	660	
Diagonal,	U_2-M_3	"	4.892	655	
	U_3-M_4	"	4.897	655	
	U_4-M_5	"	4.938	660	
	U_5-M_6	"	5.015	670	
	U_6-M_7	127.16	5.136	510	4310
	$U_1-M_1-L_1$	123.66	7.695	745	
verticals +	$U_2-M_2-L_2$	"	9.609	930	
Hanger	$U_3-M_3-L_3$	"	11.132	1080	
	$U_4-M_4-L_4$	"	12.291	1190	
	$U_5-M_5-L_5$	"	13.105	1270	
	$U_6-M_6-L_6$	"	13.589	1310	
½	$U_7-M_7-L_7$	"	13.750	665	7190
Tie		5965	32.00		15000

55020 × 2 = 110000

Details say ~~60%~~ 55% 60500

170500

For two trusses 2 @ 170.5 = 341.0 tons

Summary for structural steel in one span

stringers	1174 × 65.4	= 77000
Intermediate Floor Beam	13 × 7810	= 101600
End Floor Beam	2 × 4200	8400
Bottom Lateral Bracings		46.730
Top Lateral sways & Portals		65.000
trusses		341.000
Steel Shoes		10.000
misc steel say		1500
		651.230

call this 651.0 tons.

Original design	750
Highway design	651
	99.0 tons

5 spans @ 99 = 495 tons. / 750 × 5 = 13.2 %

Design of Juso-Bashi (Estimate of Cost).

area of pavement 5.69 × 703.36 = 4000 sq. meters

Estimate of Cost Combined Highway and Electric Ry design and Highway only.

structural steel girder span	345 tons @ 230.00	=	– 79.350.00
truss span	495 tons @ 275.00	=	– 136.125.00
境界石			– 5.960.00
			– 221435.00
Pavement	4000 @ 4.70		+ 18800.00
			– 202635.00

Difference of Cost between two Designs 202.635.00 円

Note:- Cost of Electric railway tracks is not included in Estimate of Cost for Electric & Highway design.

Comparison of Design Loading Highway Design and Highway & Ry Combined.

Girder span Highway Loading.

Dead Load for highway design

Handrails	2@90	=	180	
Copings & slab under	2 @ 206	=	412	
Sidewalk slab & wearing course	300 × 2.75 × 2	=	1650	
Extra at curb	2 × 230	=	460	
Roadway slab	530 × 14.5	=	7700	
				10402
Structural Steel	135740 ÷ 25.07	=		5400
				15802 kg per meter

Dead Load for highway & Ry.

Handrails		180	
Copings & slab under		412	
sidewalk slab & wearing course		1650	
Extra at Curb		460	
Roadway slab	2 – 530 × 4.355	4610	
	Extra filler	220	
~~Structural steel~~ Car tracks	815 × 2.895 × 2	4700	
			12232
Structural steel	156840 ÷ 25.07 =		6250
			18482 kg per meter

Live Load for highway design.

On sidewalk 2.75 × 2 × 500 = 2750
On roadway 3.70 × 600 = 2220
4970 kg

front and rear of motor truck 10.8 × 600 = 6480
motor truck rear wheel with impact 5560 × 8 = 44480 kg.
front wheel 44480 ÷ 3 =

moment at center of span
by proportion truck $72000 \times \frac{44480}{9500}$ = 338 000
Unif. load $54000 \times \frac{6480}{1310}$ = 267 000
605 000 kg m

Equivalent unif. load $w = \frac{8 \times 605.000}{24.543^2}$ = 8050 kg per lin. meter
full unif. load shown above 4970
Total 13 020 kg per meter.

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CALCULATIONS FOR

Estimate of Cost Jiuso-Bashi

Live Load for highway + Ry combined
- On sidewalk: 2.75 × 2 × 500 = 2750
- On roadway: 1.655 × 2 × 600 = 1990
- 4740
- motor truck + Unif. load front + rear 8050 ÷ 2 = 4025
- Electric Car Equiv. Unif. $\frac{2 \times 243200 \times 8}{24.5342}$ = 6450
- 15215 kg.

Summary Loads for Girder span

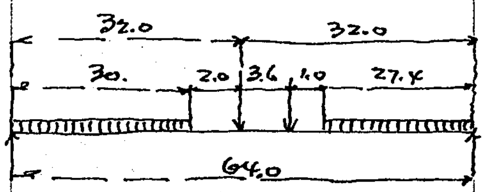
	Highway Design		Hy + Ry Design		
Dead Load	15802		18482		
Live Load	13020		15215		
	28822	÷	33697	= 85.5 %	Less 14.5 %.

Truss Spans
Dead Load for highway loading — Dead Load 21400 kg per lin. meter

Live Load
- On sidewalk: 2.75 × 2 × 428 = 2360
- On roadway: 3.70 × 513 = 1900
- 4260 kg

Unif. load front and rear of motor truck 10.8 × 513 = 5550 kg

motor truck loading
Rear wheel with impact 5230 × 8 = 41840 kg
Front " " " 1750 × 8 = 14000 kg



motor truck $\frac{41840}{2}$ = 20920
14000 × $\frac{28.4}{64}$ = 6220
27140

Moment = 27140 × 32 = 868 000 kgm

30.0 × 5550 = 166500
27.4 × 5550 = 152000

Uniform load
Reaction 166500 × $\frac{49}{64}$ = 127500
152000 × $\frac{13.7}{64}$ = 32500
160000

moment = 160000 × 32. = 5120.000
166500 × 17 = 2830.000
2290.000
motor truck moment 868.000
3158 000 kgm

Equivalent Unif. Load w = $\frac{8 \times 3158000}{64^2}$ = 6160 kg.
4260
Summary load 10420 kg per lin. meter

Truss Span
Highway + Ry Combined — Dead Load 26300 kg per lin. meter

Live Load
- On sidewalk: 2.75 × 2 × 428 = 2360
- On roadway: 1.655 × 2 × 513 = 1680
- 4040
- motor truck + Unif. load rear + front 6160 ÷ 2 = 3080
- Electric car loading with impact 2542 × 2 = 5084
- 12204 kg

Summary Loads for Truss Span

	Highway Design		Hy + Ry Design		
Dead Load	21400		26300		
Live Load	10420		12204		
	31820	÷	38504	= 82.5 %	Less 17.5 %

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CALCULATIONS FOR

Estimate of Cost Jiuso-Bashi for Osaka Prefecture

Estimated Cost	Girder span proper	Truss Span Proper
Steel	616.786	1021.416
floor etc	157.600	136.000
	774386 円	1157416 円

Estimated Cost of	Girder pier & abutment	Truss Piers
cost	414.370	811.985
Misc Exp.	14.000	27500
	428370.00	839485.00

For super structure	774386	× 14.5%	=	112.000
	1157416	× 17.5%	=	202.000
	1931802			314.000.00 円
For Substructure	428370	× 14.5%	=	62000
	839485	× 17.5%	=	147000
	1267855			209.000.00
				523000 円

Total Cost Supr	1931802	
Sub	1267855	
	3199657	% $\frac{523000}{3199657}$ = 16.4% in average.

Estimate of Cost Jiuso-Bashi

Electric Ry Bridge - Span length same as for Highway Design.

Live Load 28.0 ton car continuous for truss
18.0 ton water car coupled for floor system.

Equivalent uniform load say 6450 kg/m with impact for double tracks

Dead Load Deck construction double track 2-400 #/lin ft
or 1200 kg per lin meter
structural steel say 2000

3200
Live Load 6450
9650 kg per lin. meter

Approximate weight of structural steel

				weight of one span
Girder span	24.5 meter span	1720 #/ft	2560 kg/meter	63.0 tons
"	13.0 meter span	1200 "	1800 "	23.4 "
"	32.0 "	2050 "	3060 "	980 "
truss span	64.0 "	2760 "	4100 "	262.0 "

Girder spans:- 1 span reqd 24.5 m 63.0
2 spans 13.0 46.8
10 spans 32.0 980.0
1089.8 call this 1090 tons
truss spans 5 spans 64.0 m @ 262.0 tons 1310
2400 tons

Estimate of Cost girder span 1090 @ 230.00 = 251 000
truss span 1310 @ 275.00 = 360.000
611,000.00

Substructure

for girder span Highway Design outside bearings 15.26 m
Railway Design " " 8.5 m about

$8.5/15.26 = 55.7\%$

for truss span by well sinking and separate shaft
total cost assumed 40% of Highway design

Girder piers + abutments 428000 . 55.7% = 240.000
truss piers 839000 . 40.0% = 335.000
575,000.00

Total Estimated Cost Structural Steel 611.000
Substructure 575 000
1186,000.00

The above estimated cost is not included the cost of track construction + and both approaches