JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

京都府御幸大橋

仝　桂橋

仝　算設計計算書

大正十五年十月

428

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY J. Masuda DATE 15-9-30 FILE NO.

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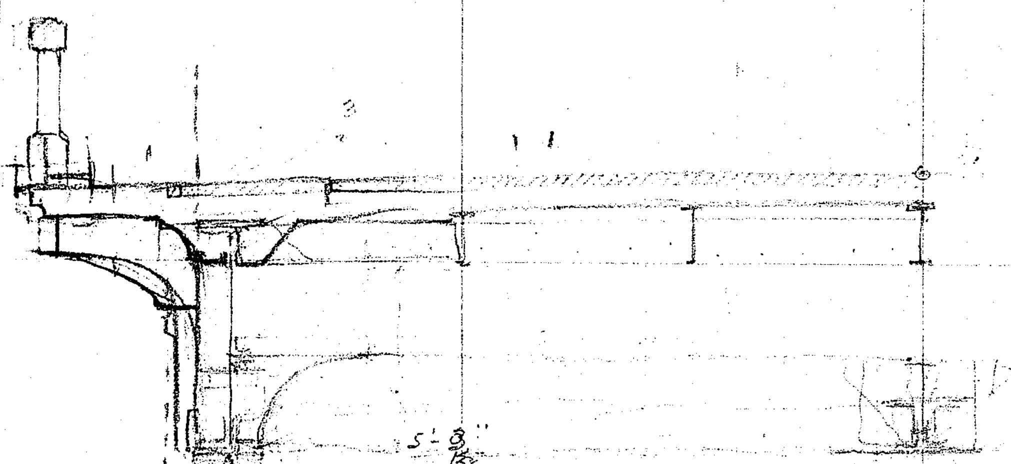
CALCULATIONS FOR

Preliminary Estimate of Goko-bashi for Kioto Prefecture.

Roadway 36.0 curb to curb. span length 100'-0" center to center of end supports.
20' cantilever 60' center girder.



5'-3"
13
8
6'2"

6@5'3 = 31'6"

30'0 - c to c of girders.

Dead Load

asphaltic pavement	2"	25 ~~15~~
Concrete	6"	75
misc.		5
		105 ~~95~~ #/□'

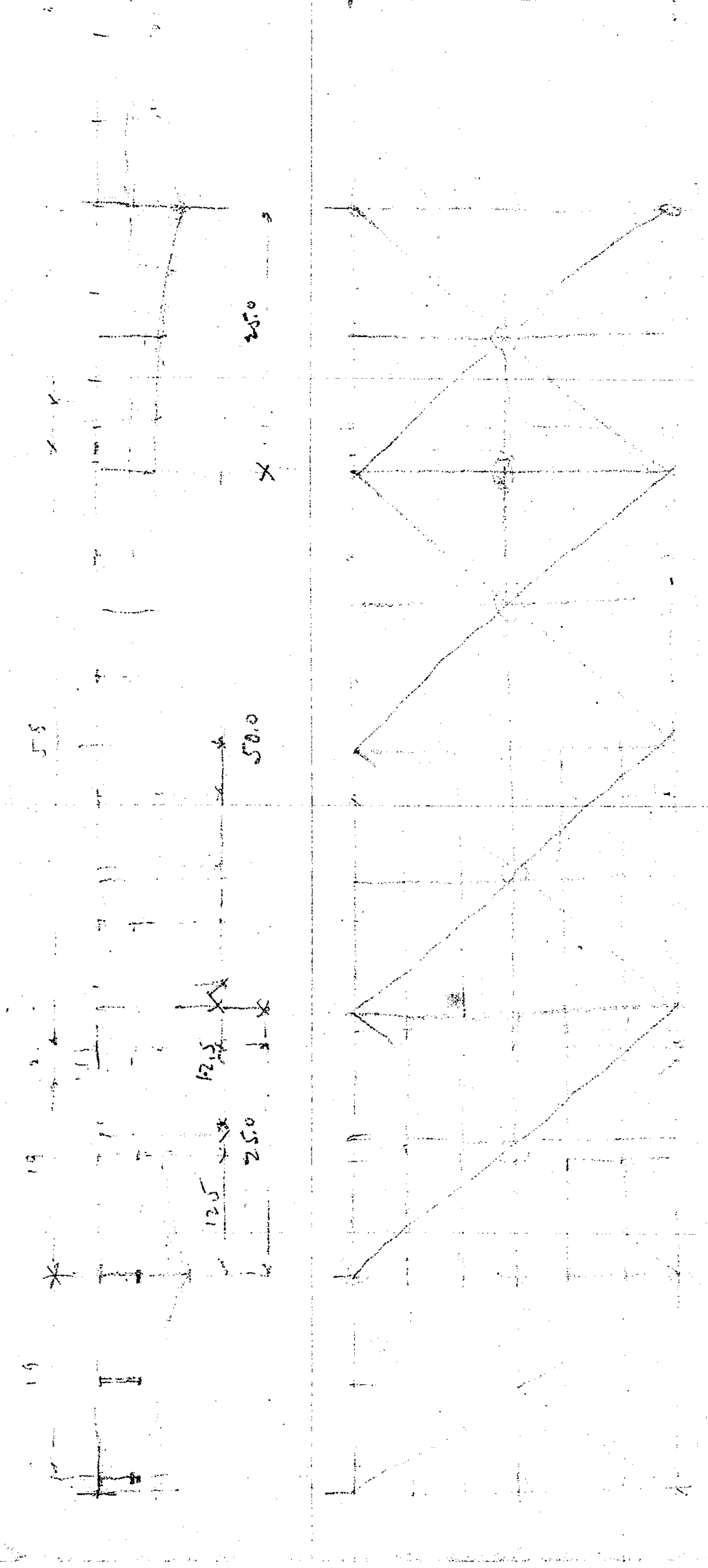
105 × 36 = 3780 (~~95 × 30 = 2850~~)
Coping - 300
Handrail - 200 ~~100~~
misc. ~~200~~
4380 ~~3550~~ ÷ 2 = 2190 ~~1775~~ # per lin ft.

metal.
stringer - 5@35# = 175#
Cross beam 280#
Lateral Bracing 60
Girder assumed 900 ~~600~~
1415 ~~1115~~ ÷ 2 = 760 ~~560~~#
2190 ~~1775~~
2900 ~~2335~~#

2900 ~~2335~~# per lin ft. for dead load

1150
865
2015

Live load say 120 × 18 ~~15~~ = 2160 ~~1800~~# per lin ft.



Center span 50'

DL. moment = 1/8 × 2900 ~~2335~~ × 50² = 905,000 ~~730,000~~ 1#
LL. moment = 1/8 × 2160 ~~1800~~ × 50² = 675,000 ~~562,000~~
5060 ~~4135~~ 1,580,000 ~~1,292,000~~ 1#

Reaction =
DL. Reaction = 1/2 × 2900 ~~2335~~ × 50 = 72500 ~~1168~~ ~~58400~~
LL. Reaction = 1/2 × 2160 ~~1800~~ × 50 = 54000 ~~45000~~

Depth say 51" × 3/8" = 19.10
1/8 web = 2.40" Effective depth = 4.10
flange stress = 1,580,000 ~~1,292,000~~ ÷ 4.10 = 385,000 ~~315,000~~#
Section required = 385,000 ~~315,000~~ ÷ 17000 = 22.60 ~~18.5~~ □"
2.4
20.2 ~~16.1~~ □" net

2Ls 6.6. 3/4 ~~5/8~~ = 16.88 ~~14.32~~ - ~~2.5~~ = 13.88 ~~11.72~~
1Pl. 13. 5/8 = 8.10 - 1.25 = 6.85
24.98 ~~22.32~~ 20.73 ~~18.57~~ □" net

Approximate weight of girder
web - 19.10
flange - ~~44.64~~
69.10 ~~63.74~~ @3.4 = 235 ~~217~~#
25% - 60 ~~54~~
295 ~~271~~#

For two girders 2@295 ~~271~~ = 590 ~~542~~# per ft.
For one span 590 ~~542~~ × 50 = 29500 ~~27000~~#

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MADE BY J. Masuda DATE 15-9-30 FILE NO.
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CALCULATIONS FOR

Preliminary Estimate of Goko-bashi for Kioto Prefecture.

Center span 100' over hanging portion 26.0'

Dead Load DL. ~~5400~~ 7250#

25.0

~~400~~# per ft. 450

DL. moment = ~~5400~~ 7250 × 25.0 = ~~1460.000~~ ~~1460.000~~ 1,810.000

~~400~~ 2435 2900 × $\frac{25^2}{2}$ = ~~125.000~~ ~~260.000~~ 902.000

1.585,000 2,220,000 2,712,000

LL. moment = ~~4500~~ 5400 × 25 = ~~1.125.000~~ 1350.000 2025.000

~~1800~~ 2160 × $\frac{25^2}{2}$ = 562.000 675.000 2025.000 4,737.000

1.687.000

Total moment ~~3.907.000~~

Depth 8.0' 96 × ½ = 48.0 ⅛ web = 6.0 Effective depth say 7.8

flange stress = ~~3907.000~~ 4737.000 ÷ 7.8 = ~~501.000~~ 606.000 SR = ~~501.000~~ 606.000 ÷ 17000 = ~~29.5~~ 35.7

~~6.0~~ 6.0

~~23.5~~ net 29.70

use 2Ls 6×6.~~⅝~~ ¾ = ~~14.22~~ 16.88 − ~~2.5~~ = 13.88 11.72

1Pl. 13.~~⅝~~ ¾ = ~~8.10~~ 9.75 − 1.25 = ~~6.85~~ 8.50

1Pl. 13.~~⅝~~ ¾ = ~~8.10~~ 9.75 − 1.25 = ~~6.85~~ 8.50

~~30.42~~ ~~25.42~~ □" net.

36.38 □" 30.38

approximate weight of girder

web − 48.0

flanges ~~60.84~~ 72.76

~~108.84~~ 120.76 @ 3.4 = ~~370~~# 410

30% ~~110~~ 120

~~480~~# 530 per lin ft. or say 480# per lin. ft.

Center span 100' Dead load say ~~2435~~ 2900# per lin ft.

DL. moment = ⅛ × ~~2435~~ 2900 × 100² = 3630.000 ~~3040.000~~

DL. neg. moment = 2712.000 ~~2220.000~~

~~820,000~~ 918.000 # net 918.000

Live Load moment = ⅛ × ~~1800~~ 2160 × 100² = ~~2250.000~~ 2700.000

~~3.070.000~~ 3,618,000 #

Depth of girder assumed 60 × ½ = 30.0 □" ⅛ web = 3.75 □"

Effective depth say 4.8 flange stress = 3070,000 ÷ 4.8 = 640.000.

section required = 640.000 ÷ 17000 = 37.60

3.75

33.85 □" net

2Ls 6×6. ¾ = 16.88 13.88

1Pl. 14 × ¾ = 10.50 9.0

1Pl. 14 × ¾ = 10.50 9.0

31.88 □" net

Approximate weight of girder.

web − 30.0

flanges 63.76

93.76 @ 3.4 = 320

30% 100

420#

100' span 420# × ~~100~~ 50 = ~~42000~~ 21.000

2@480 × $\frac{100}{25}$ = ~~96000~~ 48000

370 × 50 = 13500

~~130500~~ ÷

82500 ÷ 200 = 412.5# call this 420# per lin ft.

増田淳氏関係資料
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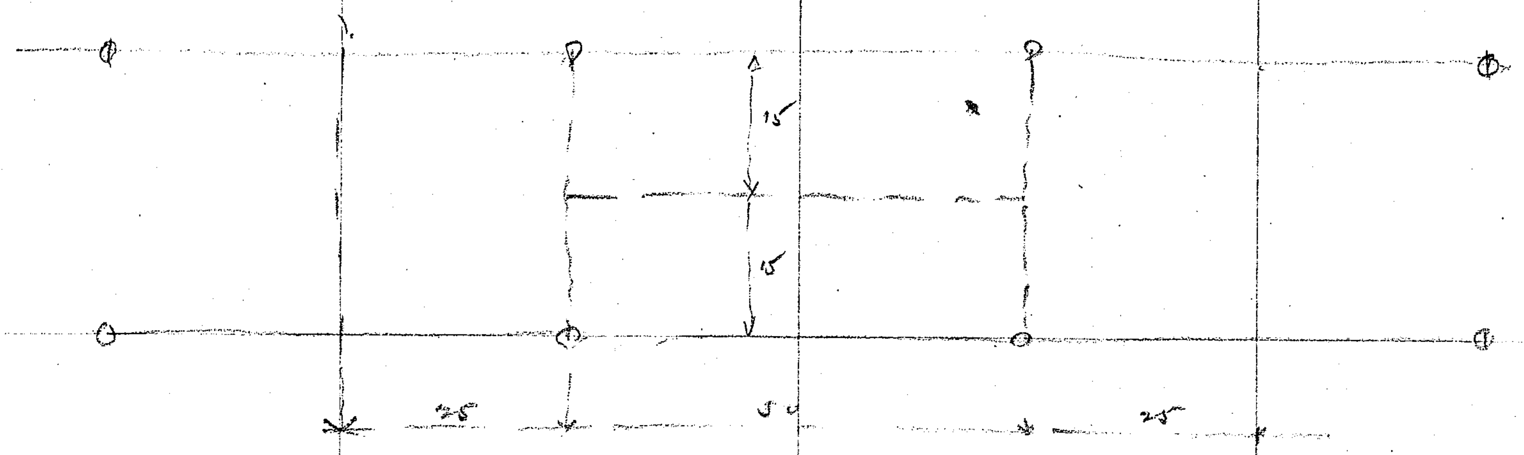
CALCULATIONS FOR

Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

100' center span



Center girder span length say 50'-0".

Dead Load. 105# × 15.0 = 1575#

stringers 3@35# = 105#

Laterals. 30× — 30

Girder say 250

385 — 385

1960# per lin ft.

moment = 1/8 × 1960 × 50² = 611.000 '#

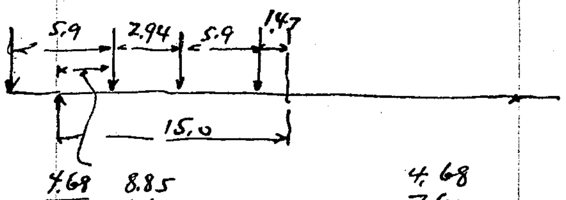
shear = 1/2 × 1960 × 50 = 49.000 #

Live Load.

motor truck rear wheel — 6600

30% impact 1980

8580 #



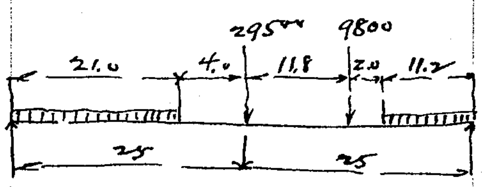
8580 × 25.82/15.0 = 14750#

4.68 8.85 1.47 10.32

4.68 7.62 13.52 25.82

2@14750 = 29500# on girder.

Front wheel = 29500 ÷ 3 = 9800#



Uniform load 100 × 15 = 1500# per lin ft.

1500 × 21. × 10.5/50.0 = 6600#

1500 × 11.2 = 16800#

R = 16800 × 44.4/50.0 = 14900#

21500

moment due to motor truck = 29500 ÷ 2 = 14750.0

9800 × 13.2/50.0 = 2590

17340 × 25.0 = 434000 '#

moment due to uniform load — 21500 × 25 = 538.000

16800 × 19.4 = −326.000

212000

646000 '#

Full uniform load of 120# per sq ft. 120 × 15 = 1800#

m = 1/8 × 1800 × 50² = 562000 '#

Summary for moments D.L. m = 611,000

L.L. m = 646.000

1.257.000 '#

Depth say 51" × 3/8 = 19.10

gross = 2.4

Section required 1.257.000 ÷ 4.10 = 306.000 ÷ 17000 = 18.0

2.4

15.6 □" net

2 Ls 6 × 6 × 5/8 = 14.22 — 11.72

1 Pl. 13 × 5/8 = 8.10 — 6.85

22.32 □" 18.57 net

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CALCULATIONS FOR

Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

Approximate weight of center girder.

flanges 2 @ 22.32 = 44.64

web. 19.10

63.74 @ 3.4 = 217

25%. 53

270#

weight of one girder 270 × 50 = 13500#

Cross beam. span length. 30'-0"

Dead Load Concentration from main girder 49000#

Uniform from stringers 115#

girder say. 200

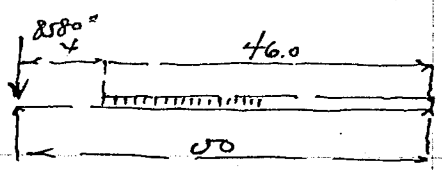
315

moment = 1/8 × 315 × 30² = 35400 1#

49000/2 × 15 = 368000

403400 1#

Live Load moment.



460 × 100 = 4600 × 23/50 = 2120#

Stringer Concentration 2120 × 5 = 10600#

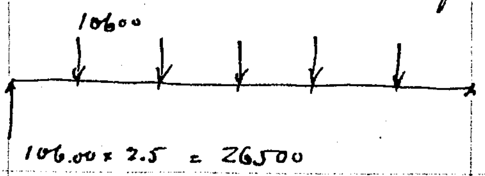
moment due to Concentration.

Reaction 8580 × 3 = 25800# m = 25800 × 15 = 387000

8580 × 19.15 = 165000

222000 1# 222000

moment due to uniform load.



m = 26500 × 15 = 398000

10600 × 15 = 159000

239000 239000

461000 1#

D.L.m 403400

864400 1#

Depth of girder 51" × 3/8 = 19.10 1/8 web = 2.40"

flange stress = 864400 ÷ 4.1 = 211000 ÷ 17000 = 12.40

2.40

10.00"" net.

Use 2L 6×6×5/8 = ~~13.66~~ 14.22 - 10. 11.72

Approximate weight of girder.

2 @ 14.22 = 28.44

web. 19.10

47.54 @ 3.4 = 161.5

Details say 25%. 40.5

202.0# per lin ft.

Ordinary Cross beam 48 × 3/8 = 18.00

flanges. 4L 3½ × 3½ × 3/8 = 10.0

28.0 @ 3.4 = 95.0

details say 25% 24

120# per lin ft.

120 × 30 / 12 = 300# per lin ft. of span.

120 @ 30 = 3600#

100' outside girder span

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CALCULATIONS FOR

Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

Dead Load moment.

Uniform dead load 2900 - 1960/2 = say 1900#

m = 1/8 × 1900 × 100² = 2.380.000'#

~~2900~~ 1000# per ft for 25' at end. ~~2900~~ 1000 × 25 = ~~72500~~ 25000#

moment at center - ~~72500~~ 25000 × (50 - 37.5) = ~~905000~~ 312000'#

Concentration at 25' 1000 × 25 = 25000#

moment 25000 × 25 = 625000

~~1530000~~ 937000
2380.000
~~3910,000~~ '#
3,317,000 '#
2712.000
605,000 '#

Total live load. 2160# per ft.
less 900
1260#

m = 1/8 × 1260 × 100² = 1.575.000.

900# per ft for 25' at end. 900 × 25 = 22500#

moment at center 22500 × 12.5 = 282000

Concentration at 25' 900 × 25 = 22500

22500 × 25 = 562000

844000'#
1575.000
2,419,000
605.000
3,024000'#

Depth of girder assumed 60 × 1/2 = 30.0 □" 1/8 web = 3.75

depth say 4.8 flange stress = 3024.000 ÷ 4.8 = 630.000# ÷ 17000 = 37.1 □"
3.75
33.35

2∠s 6.6.3/4 = 16.88 - 13.88
2Pls. 14 × 3/4 = 21.00 - 18.00
37.88 31.88 □" net.

Approximate weight of girder.
web. 30.0
flanges 63.76
93.76 @ 3.4 = 320
30%. 100
420#

Summary.

Cantilever portion - 530 × 50 }
530 × 50 } = 53000
420 × 50 = 21000
295 × 50 = 14750
88750 ÷ 200 = 444.0# per lin ft.

For both side 888# per ft.

Extra for center span. 202 × 50 = 10100
Cross beam say 2@3600 = 7200
17300# ÷ 200 = say 87#
888
975# per lin ft.
Call this 1000# per ft

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CALCULATIONS FOR

Preliminary Estimate of Goko-bashi - for Kioto Prefecture

Approximate weight of structural steel.

Stringers	175	175
Floor beam	280	300
Lateral Bracing	60	60
Girder assumed	840	1000
misc.	50	50
	1405# per lin ft.	1585

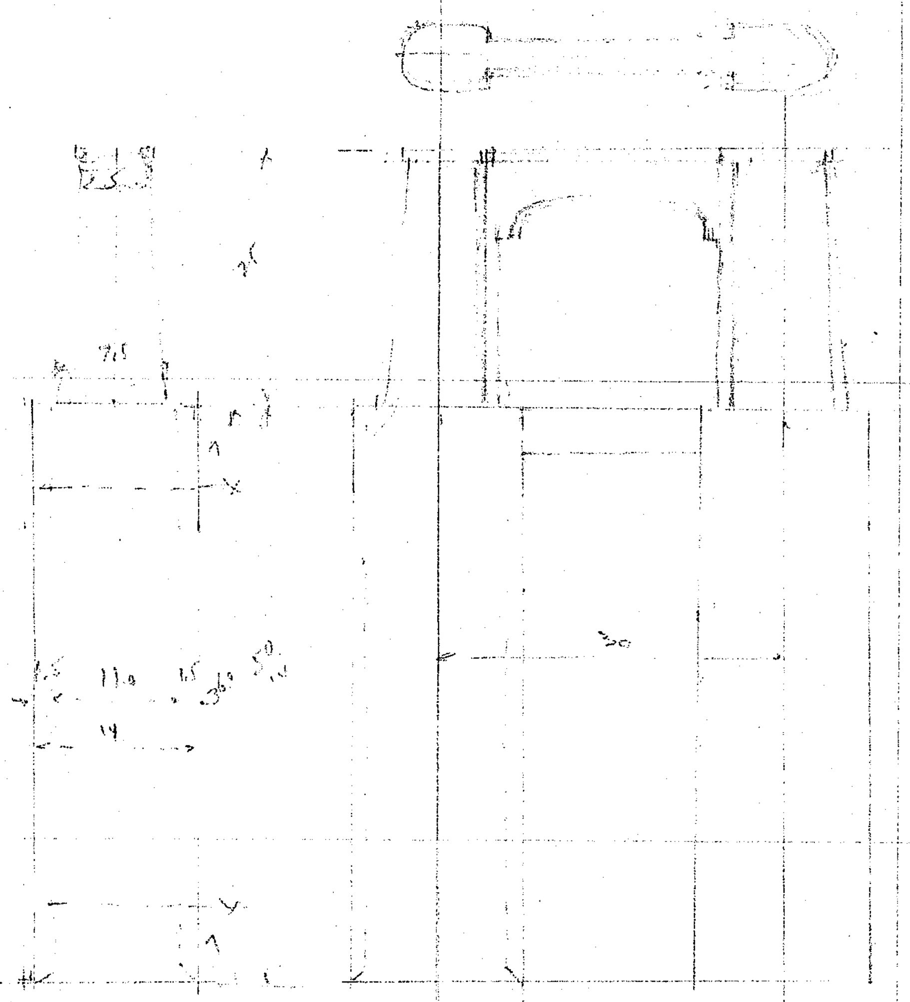
Total length of bridge - ~~2080'~~ 2020

Total wt = 1405 × 2080 = 2920,000
2920.000 ÷ 2240 = 1300 tons
10% variation 130
1430 tons

Total weight - say 1585 × 2020 = 320,000
1430 tons

Load on pier.
Dead Load 2@ 2477 × 100 = 495,500
Live Load say 3600 × 100 = 360.000

855,500# on pier.



Approximate volume of Concrete shaft.
5 dia = 28.2
7.5 dia = 44.2
72.4 ÷ 2 = 36.2
vol = 36.2 × 25.0 = 905, cubic ft
5 × 5 = 25.0
5 × 7.5 = 37.5
72.5 ÷ 2 = 36.2
vol = 36.2 × 25.0 = 905
2@ 905 = 1810 cubic ft.

volume in web -
3 × 25 × 20 = 1500 -

Total vol. 905
1810
1500
4215 ÷ 216 = 19.5 3/4
weight = 19.5 @ 32400 = 631,000#

volume of well 14 dia 154.0
11 dia 95.0
59.0 × 50 = 2950 2@ 2950 = 5900 cubic ft

Top filling and Bottom filling. 2@ 95.0 × 14 = 2660 cubic ft.

Intermediate filling 2@ 95 × 36.0 = 6850 cubic ft.

volume of Concrete

shell.	5900	27.3 3/4	@ 32400 =	885,000
filling	2660	12.3 " "	@ 30200 =	372.000
Sand filling	6850	31.7 " "	@ 21600 =	685,000
				1,942,000
		weight of shaft		631.000
				2,573,000
		Superimposed dead load		495.500
		" live load.		360.000
				3,428,500#

Base area 2@ 154.0 = 308.0
Unit pressure = 3,428.500 ÷ 308. = 11120#/□' or 4.96 tons/□'

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CALCULATIONS FOR

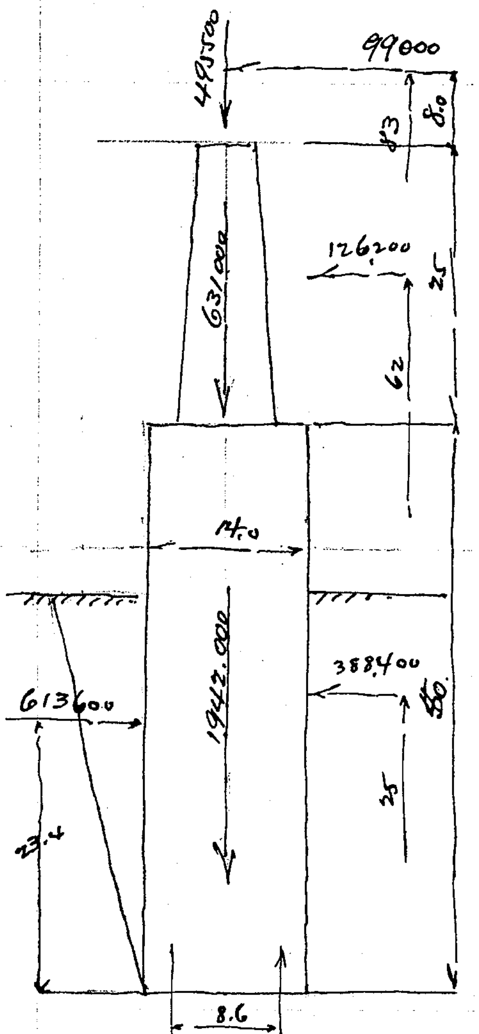
Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

Frictional Resistance on surface of well. 44 × 200 = 8800#
Firm ground assumed 25' Total friction = 2-8800 × 25 = 440,000#
Resulting load on bottom = 3428 500
440.000
2,988,500

35'?

Unit pressure = 2988500 ÷ 308 = 9700#/□' or 4.33 tons/□'

Earthquake k = 0.2 assumed.



load	H.F.		arm		
495500	99000	×	83.0	=	8,200.000
631000	126200	×	62.0	=	7820.000
1942000	388400	×	25.0	=	9700.000
3068500	613600				25,720 000
Less m	613600 × 23.4			=	14350.000
					1,1,370,000 ′#

Frictional Resistance 2 @ 8800 × 35 = 616000#

Frictional couple. = 308,000 × 8.6 = 2640.000

Resultant moment = 11,370.000
− 2640.000
8,730,000 ′#

Moment of inertia of bottom area = 0.049 × 14⁴ = 1885 ′⁴
2 @ 1885 = 3770

Fibre stress = $\frac{8730.000 \times 7}{1885 \times 2}$ = 16300 #/□' or 7.30 tons/□'

Direct load. 3068500 ÷ 308 = 9950#/□' or 4.50
11.80 tons/□'

Volume of Excavation 2 @ 154 × 50 = 15400 cubic ft or 71.4 cubic tsubo

Form for well. 44 + 34 = 78 2 @ 78 × 50 = 7800 □' or 217 面坪

Form for shaft approximate only. 70 面坪

Shoes say 2 tons. Reinforcing bars. 10.0 tons.

Approximate Estimate of one pier.

Concrete in shaft.	19.5 立坪	@	110.00	=	2145.00
forms.	70.0 面坪	@	10.00	=	700.00
Concrete in well.	27.3 立坪	@	110.00	=	3000.00
Concrete in filling 1:2:4	12.3 "	@	110.00	=	1350.00
Concrete in filling 1:4:8	31.7 "	@	50.00	=	1585.00
forms				=	500.00
Excavation.	71.4 立坪	@	50.00	=	3570.00
Curb shoe	2 tons	@	220.00	=	440.00
Reinforcing bars	10 tons	@	160.00	=	1600.00
					14890.00

call this 15,000.00 per pier.

Approximate Cost of one abutment = 13000.00

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CALCULATIONS FOR

Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

Approximate cost of deck construction.

Floor slab and coping. 0.5 × 36. = 18.0
Coping. 2.0
Extra Concrete 2.0
22.0 × 6/216 = 0.61 立坪 per lin. 間

Total volume of Concrete in Floor slab. 0.61 × ~~2070~~ 350 間 = 214 立坪

Pavement = 6 × 350 = 2100 面坪. form say 2400 面坪
Reinforcing bars say 214 @ .72 tons = 155 tons.

Approximate cost of deck.

Concrete in slab.	214 立坪	@	110.00	=	23500
Reinforcing bars	155 tons	@	160.00	=	24800
forms	2400 面坪	@	6.00	=	14400
pavement	2100 面坪	@	16.00	=	33600
finish of coping and curb line	400 面坪	@	12.00	=	4800
drains and expansion Joints & misc.					3500
Handrails.	700 ken	@	50.00	=	35000
Electric wiring & lighting.					5000
monuments &c.	8	@	500.00	=	4000
					148,600.00

Summary for substructure.

Piers	9 @	15.000	=	135000.00
"	11 @	15000	=	165000.00
Abutments	4 @	13000	=	52000.00
				352.000.00

Structural Steel 1410 tons @ 240.00 = 338000.00

Estimate of Cost

substructure	352.000.00
structural steel	338.000 —
Deck.	148 600
	838.600.00

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MADE BY J. Masuda DATE 15-10-24 FILE NO.

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CALCULATIONS FOR

Preliminary Estimate of Yokō-Bashi for Kioto-Prefecture

General layout

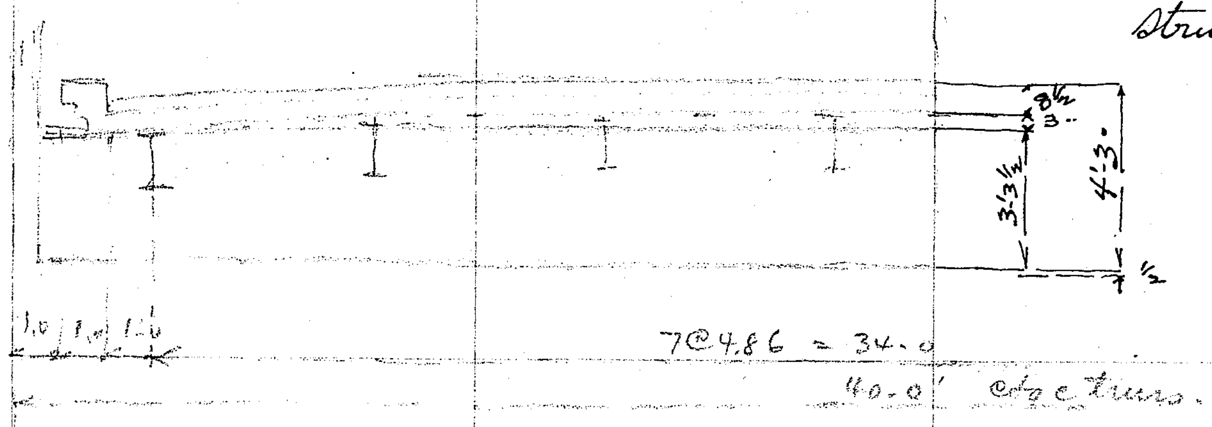
Ujigawa-Crossing.	2-47'-0"	girders	94'-0
	6@129'-0"	spans =	774-0
			868-0"
Kizugawa-Crossing.	2@61.5	girders =	123'-0
	8@129'-0"	spans =	1032-0
			1155'-0"

Truss span span length 127'-0"

12 panels @ 10.56' = 127'-0"

Cross section of bridge as shown below in sketch

Stringers 12"×5" @ 31.99# I.



Floor Beam

Asphaltic pavement	2"	25
6" Concrete slab.		75
Laterals + stringers say		10
		110# per square ft.

Dead Load panel Concentration - 110 × 10.56 = 1165# per lin ft.

girder assumed 200

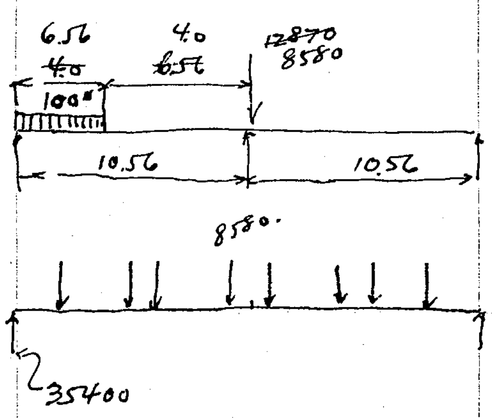
1365#

Dead Load moment = $\frac{1}{8} \cdot 1365 \cdot 40^2 = 273000'^{\#}$

Live Load moment motor truck 6600 for rear wheel.

Impact 1980

8580#



Front wheel 2890#

Uniform load - $656 \cdot \frac{3.28}{10.56} = 204^{\#}$ per lin ft.

Reaction due to motor truck loading.

4@8580 = 35400#

$m = 35400 \times 20 = 708000'^{\#}$

$35400 \times 8.85 = 313000$

$395000'^{\#}$

Uniform live load $m = \frac{1}{8} \cdot 200 \cdot 40.0^2 =$ 40000

$435000'^{\#}$

Dead Load moment = 273000

$708000'^{\#}$

Depth of girder assumed $39\frac{1}{2}$" ~~web assumed~~ $39 \times \frac{3}{8} = 14.60$" $\frac{1}{8}$ web = 1.830

Effective depth say 3.1 flange stress = 228.000 sk = 13.40"

1.83

11.57" net.

Use	2 L 6×4×½ =	9.50	7.50
	Pl. 12½ × ½ =	6.25	5.25
		15.75#"	12.75

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Preliminary Estimate of Goko-Bashi for Kioto-Prefecture.

Approximate weight of floor beam

flanges 2 @ 15.75 = 31.50
web. 14.60
46.10 @ 34 = 1570
Details say 43
200# per lin. ft.

weight of one floor beam = 200 × 39 = 7800#

Dead Load of floor

asphaltic concrete 2" 25
6" Reinforced Concrete slab 75
100#

Floor 100 × 36 = 3600
Coping 300
Handrails 200
misc details say 100
~~4020~~ 4200#

metal in span

stringers 8 @ 35 = 280
Cross beam 740
Laterals. 80
~~trusses~~ assumed 1200
2300
~~6520~~ 6500# ÷ 2 = ~~3160~~ 3250# per lin. ft of span

moment at center = ⅛ × 3250 × 127² = 6.550.000 '#

Live Load 120# per sq ft assumed 120 × 18 = 2160# per lin ft of one truss

moment at center = ⅛ × 2160 × 127² = 4.350.000
DL m 6.550.000
10,800,000 '#

Depth of truss 16.0 Stress = 10.800.000 ÷ 16.0 = 675.000
675.000 ÷ 14000 = 48.30□"

Top and bottom chord members. 2 @ 50.0 = 100 @ 34 = 340# per ft.
verticals. 4 Ls 5 × 3 × 3/8 - @ 9.8 = 29.4 × 15. - 440#
12 @ 440 = 4950#

End post. 5400 × 127/2 = 343000# SR = 343000 ÷ 14000 = 24.50"
24.5 × 34 × 8 = 665#
2 @ 665 = 1330#

Diagonals say. 600# × 12 = 7200

Top and bottom chord. 340 × 127 = 43200
vertical. 5000
End posts. 1330
diagonals. 7200
56730
details say 40% 22700
79430# call this 80.000

For two trusses 160.000 ÷ 127 = 1260# per lin. ft.

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CALCULATIONS FOR

Preliminary Estimate of Goko-Bashi for Kioto-Prefecture.

approximate weight of one span

stringers	280 × 129	=	36100
Floor beam say	11 @ 7800	=	85700
	2 @ 6500	=	13000
Lateral Bracings		=	15000
trusses	1260 × 129	=	162500
shoes &c.			10000
			322300# or 144 tons.

14 spans @ 144 = 2020 tons.

Girder span.

60' girder span.

Dead Load floor load - 4200
metal say 1500
5700 ÷ 2 = 2850# per lin ft.

L.L. 120# × 18 = 2160
5010

$m = \frac{1}{8} \times 5010 \times 60^2 = 2260000$ 1#

Depth of girder say 5.0 flange stress = 2.260.000 ÷ 4.7 = 482.000#

482000 ÷ 15400 = 31.2 □" gross
31.2
web. 60 × 3/8 = 22.5
84.9 @ 3.4 = 290#
Detail say 30% - 87
377# per lin ft.

weight of girder 2 @ 377 × 62 = 45800
stringers 175
floor beam 280
Lateral 60
shoes &c 50
565 × 62 = 35000
80800 ÷ 2240 = 36.1 tons.

47' girder span

$m = \frac{1}{8} \times 5010 \times 47^2 = 1.380.000$ 1#

Depth say 4.0 flange stress = 1.380.000 ÷ 3.8 = 364000#

$\square$" = 364000 ÷ 15400 = 23.6
23.6
48 × 3/8 = 18.0
65.2 @ 3.4 = 222
Detail say 30% 66
288#

weight of girder 2 @ 288 × 49 = 28200
other metal = 565 × 49 = 27700
55900 ÷ 2240 = 25.0 tons.

Total weight of structural steel

14 spans of trusses	@ 144	=	2020
2 @ 60' span	@ 36	=	72
2 @ 47 span	@ 25	=	50
			2142 tons.

call this 2150 tons.
1430
720

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CALCULATIONS FOR

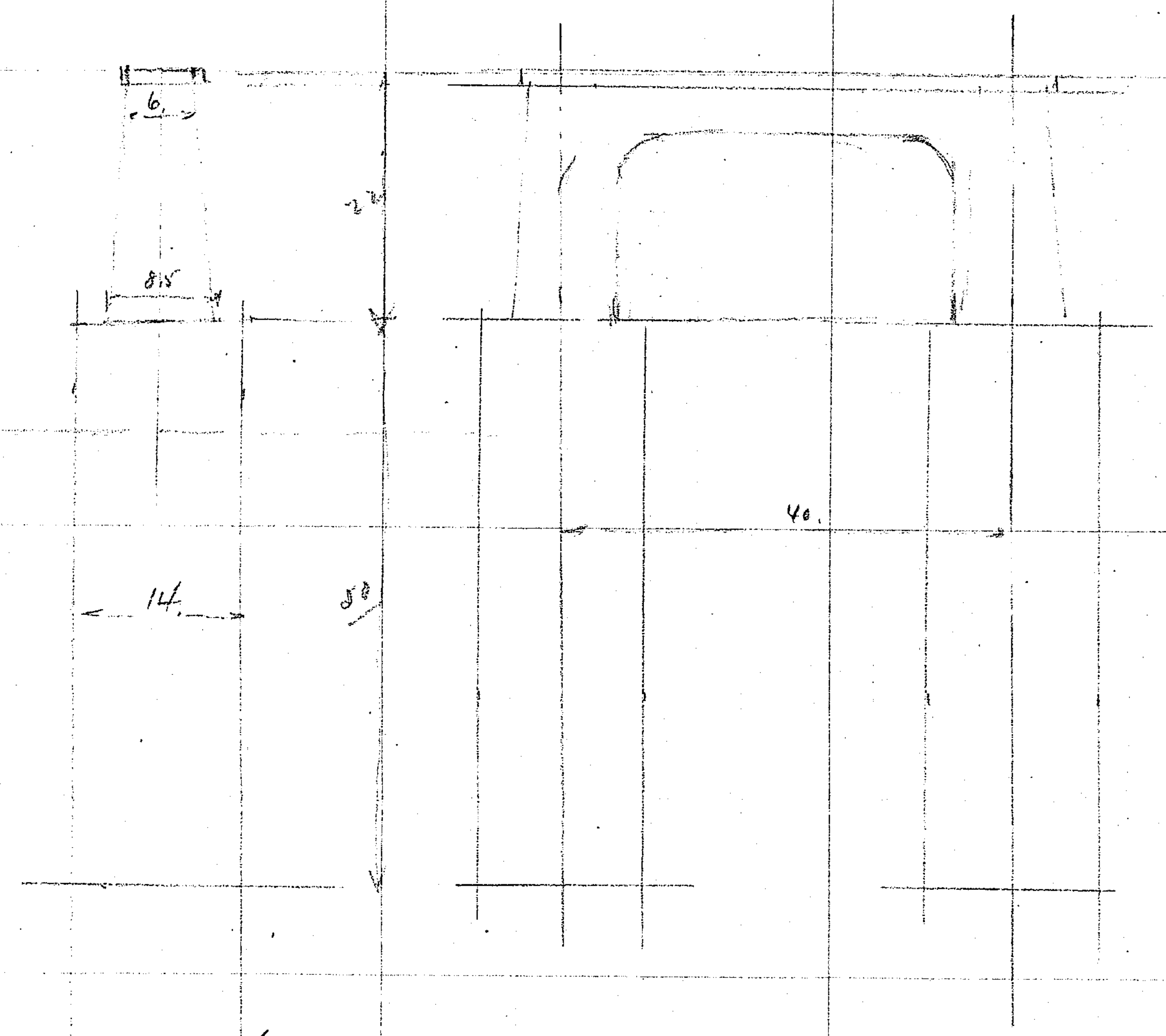
Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

Load on pier.

Dead Load. 2 @ ~~2900~~ 3250 = ~~5400~~ 6500 × 129 = ~~696000~~ 840000 #

Live Load 3600 × 129 = 465000 #

~~1161000~~ 1305000 # on pier.



Approximate weight of pier.

vol of Concrete.

6' dia 28.2 vol = 42.5 × 22 = 935

8.5 dia. 56.7

84.9 ÷ 2 = 42.5

6.0

8.5

14.5 ÷ 2 = 7.25   7.25 × 22 × 10 = 1600

Top strut. 5 × 6 × 30 = 900

web. 2.5 × 17 × 30 = 1275

4710 cubic ft or 21.8 立坪

weight = 21.8 @ 32400 = 706,000 #

volume of well. 14' dia 154.0

11. dia 95.0

59.0 × 50 = 2950   2 @ 2950 = 5900 cubic ft

Top and bottom fillings 2 @ 95 × 36.0 = 6850 cubic ft

Intermediate filling 2 @ 95 × 14 = 2660 " "

Volume of Concrete shell 5900 27.3 立坪 @ 32400 = 885.000

filling 2660 12.3 " @ 30200 = 372.000

Sand filling. 6850 31.7 " @ 30200 = 960.000

2217000

Shaft 706.000

2923.000

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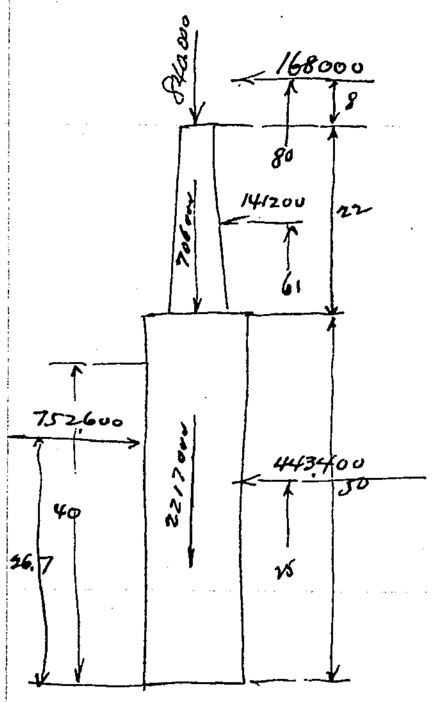
CALCULATIONS FOR

Preliminary Estimate of Yokō-Bashi for Kioto Prefecture.

weight of pier -	2.923.000
Superimposed D.L.	840.000
Live load	465.000
	4.228.000 #

Base area. 2 @ 154.0 = 308 Unit pressure = 4228000 ÷ 308 = 13700 #/□' or 6.11 tons.

Earthquake 0.2



load	H.F.		arm		moment
840.000	168.000	×	80	=	13450.000
706.000	141200	×	61		8600.000
2217.000	443400	×	25		11.100.000
3763000	752600				33150.000
	752600 × 26.7 =				20.100.000
					13.050.000 ′#

Frictional resistance 2 @ 8800 × 40 = 705000
Frictional couple - 352500 × 8.6 = 3030.000

Resulting moment 13.050.000
3030.000
10.020.000 ′#

moment of inertia of bottom area = 0.049 × 14⁴ = 1885
Fibre stress = $\frac{10.020.000 \times 7.}{1885 \times 2}$ = 18600 or 8.3 tons.

Direct load - 3763000 ÷ 308 = 12200 or 5.45
13.75 tons/□'

Bearing pressure is too high let us try 15.0 well instead of 14'

Volume of well.

shell.	15' dia.	201.	
	12' dia	113	
		88 × 50 = 4400	2 @ 4400 = 8800 cubic ft.
Top and bottom filling		2 @ 113 × 14 = 3160	cubic ft.
Intermediate filling		2 @ 113 × 36 = 8140	cubic ft.

Shell.	8800 cubic ft or	40.7 3.77	@ 32400	=	1320.000
top and bottom filling	3160	14.6 "	@ 30200	=	441.000
Intermediate filling	8140	37.6 "	@ 30200	=	1135.000
					2.896.000
		Shaft			706.000
					3.602.000 #
	Superimposed dead load				840.000
	" live load				465.000
					4.907.000

Base area. 15.5 ϕ 188 × 2 = 376 □'
Unit bearing = 4.907.000 ÷ 376 = 13050 #/□' or 5.83 tons/□.

Earthquake k = 0.2

load	H.F.		arm	moment
840.000	168.000	×	80	13450.000
706.000	141200	×	61	8600.000
2896.000	579200	×	25	14500.000
3442 000	888400			36550.000 ′#
	888400 × 26.7 =			23.700.000
				12.850.000 ′#

Frictional resistance 47. @ 200 = 9400# per ft × 40 = 376000 #
For ~~two wells.~~ 752.000 #

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CALCULATIONS FOR

Preliminary Estimate of Goko-Bashi for Kioto Prefecture.

Frictional couple = 376.000 × 9.22 = 3470.000 "#

Resulting moment 12850.000
3470.000
9380.000 "#

Moment of inertia of bottom section = 0.049 × 15.5⁴ = 57800 × 0.049 = 2820

Fibre stress = $\frac{9380.000 \times 7.75}{2 \times \cancel{5780}\ 2820}$ = ~~6280~~ "12900" = 5.75 tons.

direct load 3.442.000 ÷ 376 = 9140 = 4.10
9.85 tons/□

volume of Excavation 2 @ 188 = 376 @ 50 = 18800 or 84.0 立坪

Forms for well (47+38) × 50 = 4250 2 @ 4250 = 8500 - 236 面坪

Form for shaft say 90 面坪
Shoes say 2.2 tons Reinforcing bars say 12. tons.

Approximate Estimate of Cost of one pier.

Concrete in shaft	21.8 立坪	@ 110.00	= 2398 00		
forms.	90.0 面坪	@ 8.00	= 720 00		
Concrete in well.	~~27.3~~ 40.7 立坪	@ 110.00	= ~~3003~~ 00	4477	
Concrete filling 1:2:4	~~12.3~~ 14.6 "	@ 110.00	= ~~1353~~ 00	1606	
" 1:4:8	~~31.7~~ 37.6 "	@ 65.00	= ~~2060~~ 00	2450	
forms.			= 600 00		
Excavation	84.0 立坪	@ 50.00	= 4200 00		
Curb shoe	2.2 tons	@ 220.00	= 485 00		
Reinforcing bars.	12. tons	@ 150.00	= 1800 00		
			16619 00 (2117)	18736 00	

Call this 19.000 00

Approximate Cost of one abutment = 13000 yen

Summary for substructure

16 piers @ 19000 = ~~272.000~~ 304 000
4 abutments @ 13000 = 52 000
~~324000~~ 356 000 00 356000

Structural Steel - say 2150 tons @ 260.00 560.000 00 250.00 538000
Cost of Deck structure 148600 148600
1064600 1042600 00

girder 838000
226600

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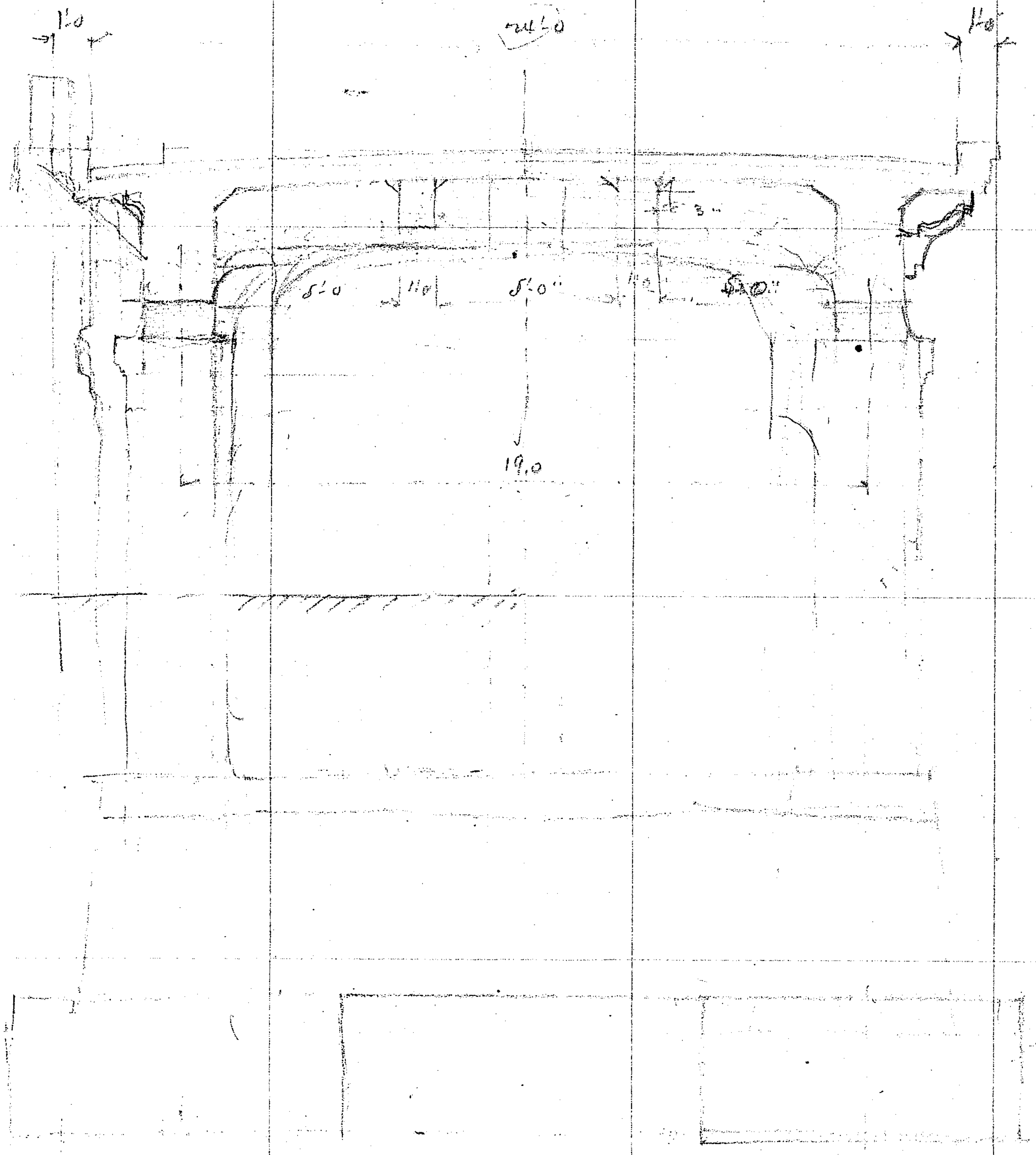
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Approach Concrete spans.
15 @ 31. = 465-0".
Cross section of bridge as shown on sketch below.



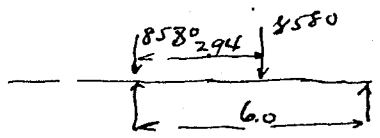
13'-6
2-0
2-0
17-6

Floor slab.
Dead Load. 2" asphaltic Concrete pavement = 24
6" Reinforced Concrete slab. = 75
99 call this 100#

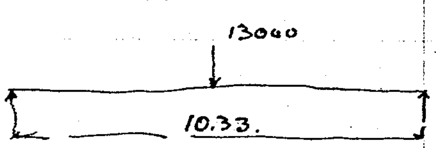
Longitudinal stringer span length 10'-4"
Dead Load 100 × 6 = 600
stem say 180
780#

$m = \frac{1}{10} \times 780 \times 10.33^2 = 8330$ '#

Live Load motor truck loading rear wheel with impact 8580"
max load on stringer



$8580 \times \frac{3.07}{6.00} = 4400$
8580
12980 call this 13000#



$m = \frac{13000}{2} \times 5.16 = 33600$ '# × 0.8 = 26900
8330 8330
41930 '# 35230 '#

Effective depth say. $\sqrt{\frac{35230}{102}} = 18.7"$.

make depth 18".

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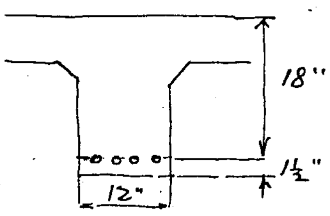
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture

Reinforcing steel reqd = $\frac{35230 \times 12}{18 \times 7/8 \times 17000}$ = 1.58

use 4-3/4-bars = 1.76 □"



Design of Cross beam span length 19.0' spacing - 10.33'

Dead Load stringer Connection - 780 × 10.33 = 8050#

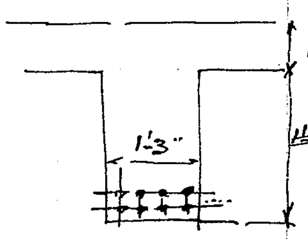
Dead Load beam assumed. 1.25 × 2.5 = 3.12 @150 = 470# per lin ft.

moment = 8050 × 6.5 = 52300'#
$\frac{1}{8}$ × 470 × 19² = 21200
73500'#

Effective depth as square beam $\sqrt{\frac{73500}{102 \times 1.25}} = \sqrt{577} = 24"$.

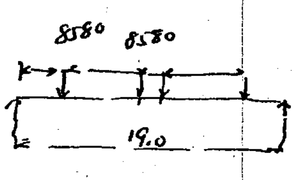
Reinforcing steel = $\frac{73500 \times 12}{24 \times \frac{7}{8} \times 17000}$ = 2.48

6-3/4 = 2.640 □"



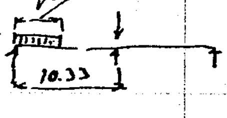
$\frac{39"}{3}$
29"

~~1.75~~ × 1.25 = 2.17; ~~2.19~~ 2.72 @150 = ~~330~~ 407# per lin ft.



R = 17160.
m = 17160 × 9.5
− 17160 × 4.42
5.08 = 87000'#

Dead Load = 8050.
330 × $\frac{19}{2}$ = 3140
11190 say 11200# - from floor beam unif.



377 × 100 = 377 × $\frac{188}{10.33}$ = 68.5
Call this 70# × 6 = 420#

Uniform load - slab. 6.0 × 100 = 600.
Coping say - 150
beam say. 2 × 2.5 @150 = 750
1500# per lin ft.
Handrail say 150
1650.
Extra for beam. 200
1850# per lin ft.

m = 420 × 6.5 = 2720
87000
89720
DL m 73500
163,220

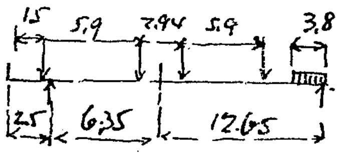
Effective d = $\sqrt{\frac{163220}{102 \times 1.25}}$ = 36".

m = $\frac{1}{8}$ × 1850 × 31.² = 178,000'#
m = 11200 × 10.33 = 116.000
294.000'#

Try 30" beam.
steel = $\frac{163220 \times 12}{\frac{7}{8} \times 27 \times 17000}$ = 4.86.
use 8-7/8 bars = 4.8 □"

~~Depth as square beam~~ $\sqrt{\frac{294.000}{102 \times 2.}}$ =

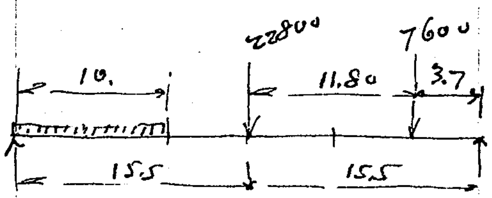
Live Load.



Live Load motor truck
4 × 8580 × $\frac{12.65}{19.00}$ = 22800#

Rear wheel 22800 ÷ $\frac{1}{3}$ = 7600#

moment = 7600 × $\frac{3.7}{31.0}$ = 900
11400
12300#



m = 12300 × 15.5 = 191000'#

1200# × 10. = 12000#
m = 12000 × 5 = 60000
251000'#

DL m. 294000
545000'#

For Continuity of beam 545.000 × 0.8 = 436.000'#

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CALCULATIONS FOR

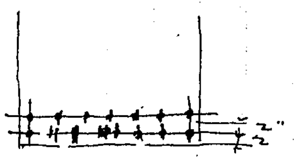
Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Effective depth as square beam. $\sqrt{\frac{436.000}{2 \times 102}} = \sqrt{2140} = 46"$ at support.

. Depth assumed 36" at center of girder :— 32" effective.

steel area $= \frac{436.000 \times 12}{\frac{7}{8} \times 32 \times 17000} = 11.0$

Use 14 - 1" bars = 11.0 □"



at support. steel area $= \frac{~~436.000~~\ 545.000 \times 12}{\frac{7}{8} \times 54" \times 17000} = 8.2$ □"

approximate volume of Concrete. Reinf. bars.

Slab.	~~24 × 0.5 × 31.~~	=	~~372~~	2800.	
main beam	4 × 2.5 × 31.	=	310	6000.	11 900 — 5.3 tons per span
filler say.	1.1 × 31	=	34		
Extra - say		=	100.		
floor beam	3 - 1.25 × 2.17 × 17	=	139.	2100.	
stringer		=	73.	1000	
			1028 ÷ 216 =	4.8 立坪 per span call this - 5.0 立坪	
Coping say	2 × 31	=	62		
			1090 ÷ 216 =	5.05 call this - 5.3 立坪	

Column 2.5 × 2.5

Dead load Concrete	$\frac{5.3}{2}$ @ 32400	=	86,000
pavement.	25 × 12 × 31	=	9000
Handrail.	100 × 31	=	3100
			98100 call this 100.000#

Height of Col. say 16.0

Earthquake. 0.2 HF = 20.000#

moment = 8 × 20.000 = 160.000 '#

Direct load. 100.000 ÷ 900 = 111#/□".

Reinforcing bars for moment $= \frac{160.000 \times 12}{7/8 \times 27 \times 17000 \times 1.8} = 2.65$

Reinforcing bars. 900 × 0.01 = 9.0 □" -

Col. reinforcement say 12.0 × 3.4 = 41# per ft say 50# per ft

50 × 25 = 1250# for 2 cols 2500#

weight of Col. 2.5 × 2.5 × 14.0 = 87.5

16
36
52 ÷ 2 × 26. × 5 = 130.0

Base. 9 × 9 × 4 = 324.0

web. 5 × 2 × 7 = 70.0

611.5 ÷ 216 = 2.85 2 @ 2.85 = 5.70 立坪

load 2.85 @ 32400 = 92500

superimposed. load. 100000

192500 #

Live Load say 1200 × 31 = 37000

229500 call this 230.000 #

Bearing pressure $= \frac{230.000}{9 \times 9} = 2840\#/□'$

杭ニ五寸 = 間物 230.000 ÷ 9 = 25600 or 11.4 tons per pile.

Reinforcing bars. in web & base say. 1000

Col. say 2500

3500 ÷ 2240 = 1.56 tons.

Summary for one span

Concrete	5.3	Reinf.	5.3
	5.7		1.56
	11.0 立坪		6.86 say 7 tons.

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CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Concrete	11.0 @ 15	=	165 立坪
Reinf.	6.86 @ 15	=	103 tons.
pavement	$4 \times \frac{465}{6}$	=	310 面坪
Handrail.	2 @ 465′	=	930 lin ft.

Forms. slab. 26′
stringer 5
beam 12
Cross beam 17

60 ÷ 36 = 1.67 面坪 per ft. × 465 = 775 面坪

Columns	10 × 15	=	150
pedestal.	20 × 5	=	100
base.	36 × 4	=	144
web.	2-7×5	=	70

464 call this 500 for two cols 1000 ÷ 36 = 28 面坪

28 × 15 = 420 面坪
775 " "
Total 1195 call this 1200 面坪

Piling 18 × 14 = 252 本

Excavation. $\frac{15 \times 28 \times 15}{216}$ = 29.2 立坪 14 @ 29.2 = 410 立坪

Estimate of cost of approach span

Concrete -	165 立坪	@ 110.00	=	18,150.00	
Reinf. bars.	103 tons	@ 150.00	=	15450.00	
forms	1200 面坪	@ 12.00	=	14400.00	15.00 + 3600
pavement	310 "	@ 16.00	=	4950.00	
finish of coping &c	80 "	@ 12.00	=	960.00	
drain + expansion Joint				1000.00	
Handrail -	155 間	@ 70.00	=	9300.00	
Electric wiring -				1500.00	
monuments		2 @ 500	=	1000.00	
excavation -	410 立坪	@ 10.00	=	4100.00	
piling -	252 本	@ 20		5040.00	
				75850.00	
1 abutment say				6000	
				81850.00	call this 82000.00
					3600
					85600

85600 ~~82000.00~~ ÷ 310 = 276 ~~264~~.00 per 面坪

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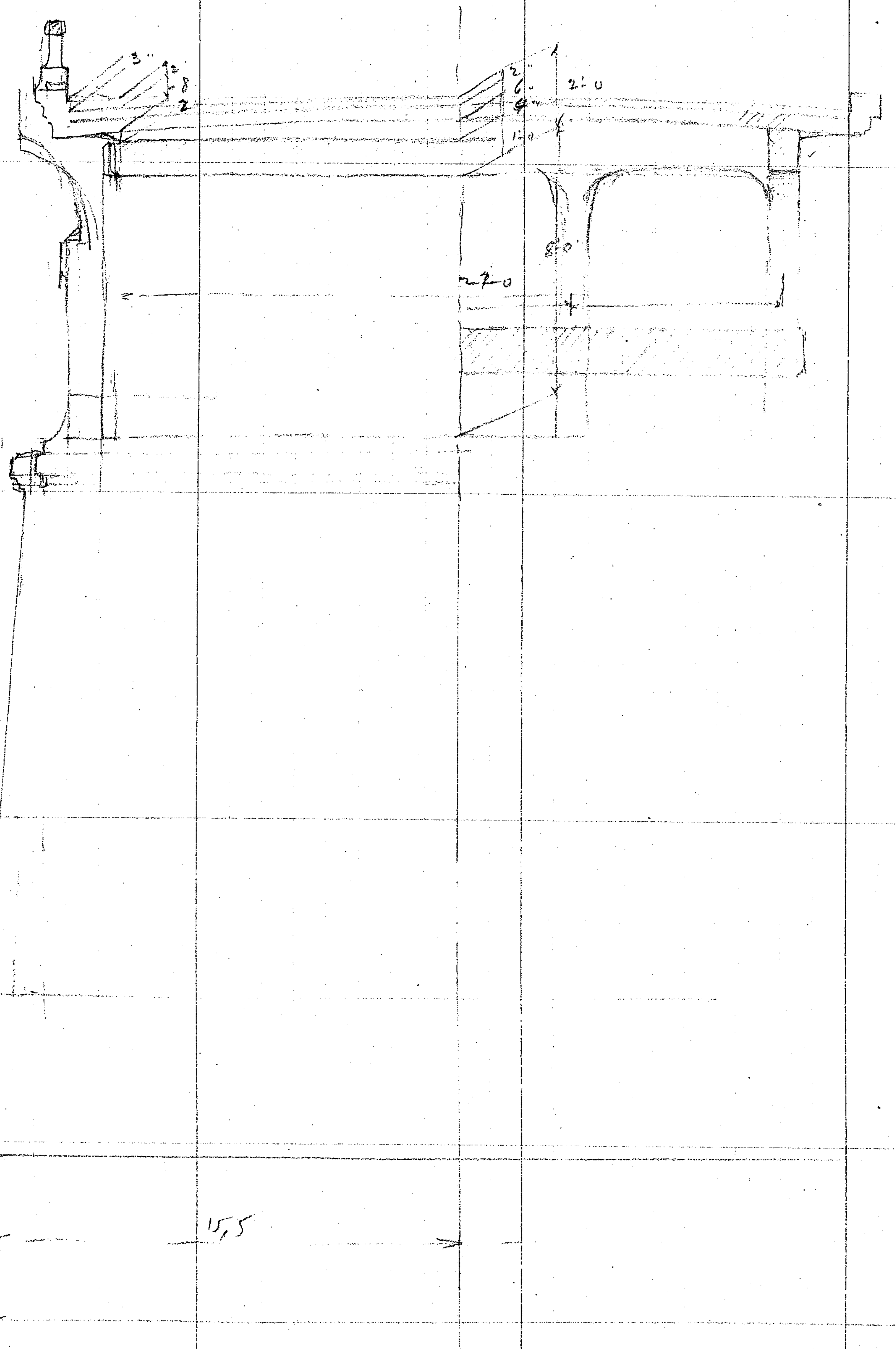
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

River span total length 555' 8 - 69.5 - arch spans.

Clear span 65.0'

Cross section of bridge as shown on sketch below



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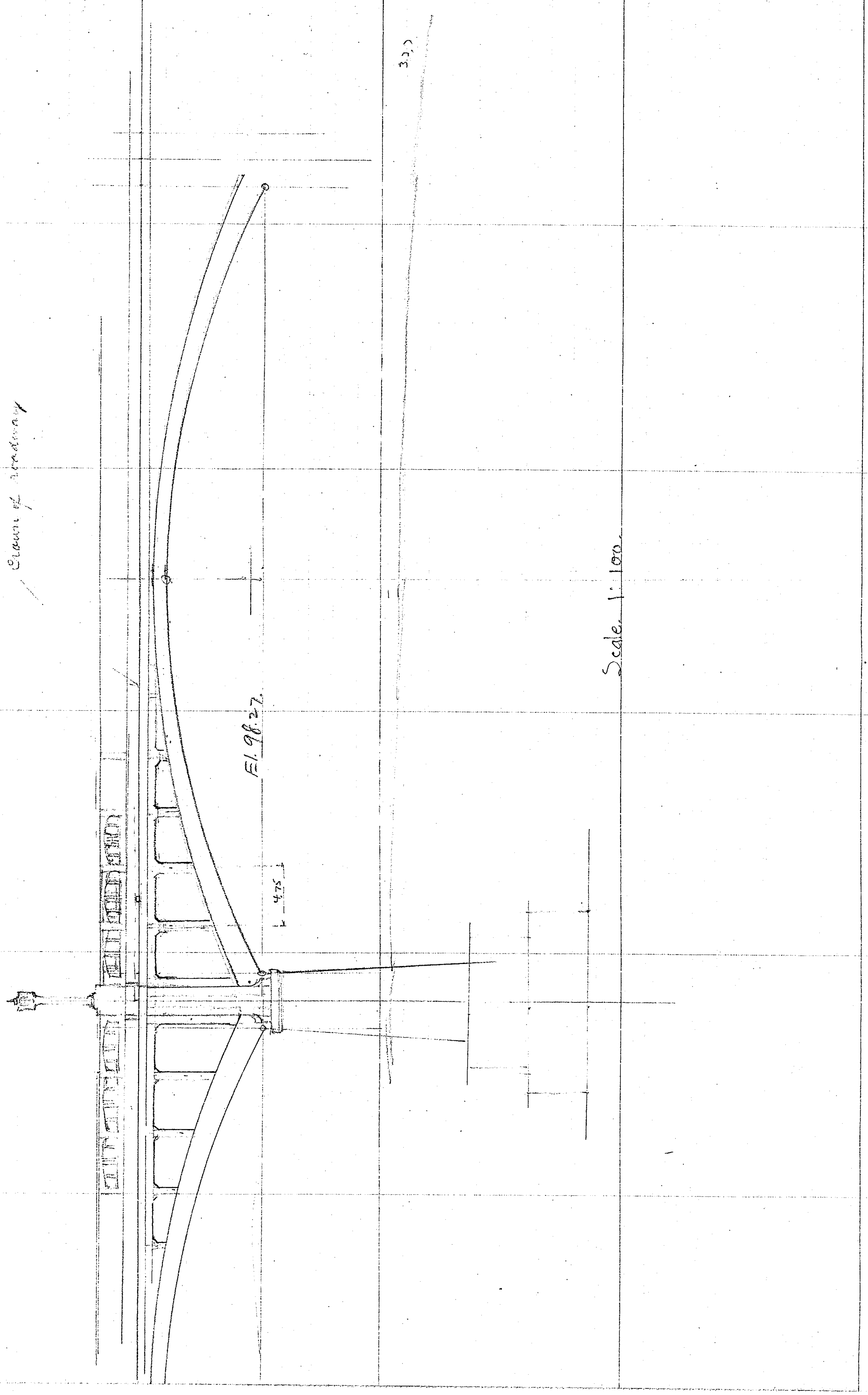
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.



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CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Concrete slab. 6" thick throughout.
Transverse beam 10" thick.
approximate Concrete -

24 × 0.5 = 12.0 } 14.0
Coping - 2.0
Concrete beam $\frac{83 \times 21 \times 3.0}{4.75}$ = 11.0
25.0 sq ft.
say 30.0 sq ft. @ 150 = 4500 #
Handrail say 200
4700

Arch ring - average depth say 1.35 × 22 = 30.0 @150 = 4500
9200

Bending moment at center = $\frac{1}{8}$ × 9200 × 66.5² = 5,080,000 '#
Rise of arch ring say 7.5
Horizontal thrust = 5.080.000 ÷ 7.5 = 678.000 #
vertical reaction = 9200 × $\frac{66.5}{2}$ = 306,000 #

Live Load 100 × 24 = 2400 # per lin ft.
Moment = $\frac{1}{8}$ × 2400 × 66.5² = 1.330.000 '#
Horizontal thrust 1.330.000 ÷ 7.5 = 177.000 #
vertical reaction = 2400 × $\frac{66.5}{2}$ = 80.000 #

For Horizontal thrust add motor truck loading at center of span
2 motor trucks. 4 @ 8580 = say 35000 #
moment = 17500 × 33.25 = 582 000 '#
Horizontal thrust = 582.000 ÷ 7.5 = 77500 #
vertical reaction = 17500 #

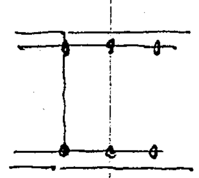
Dead Load	Horizontal thrust	678.000
Live Load	uniform	177.000
" "	motor truck	77500
		932,500 #

Cross section of arch ring at crown 1.0 × 22.0 = 22.0
For one ft strip 932500 ÷ 22. = 42400 #
Direct load = 42400 ÷ 144 = 294 #/□"
Bending stress = Eccentricity say .25 m = 42400 × 0.25 = 10600 '#

fibre stress = $\frac{10600 \times 12 \times 6 \times 12}{12 \times \frac{12^{3}}{24}}$ = 442 #
294
736 #/□".

For Eccentricity 0.15 m = 42400 × .15 = 6360
Fibre stress = 442 × $\frac{6360}{10600}$ = 266.
294
560 #/□". ok

Reinforcing bars.



1" □ bars 6" centers. .78 × 4 = 3.12 □"
15
47.00 □"
144
3.12 ÷ 144 = 2.17 %. Reinforcement 191.0.

moment of inertia 2 - 1.56 × 15 × 4² = 753
$\frac{1}{12}$ × 12 × 12³ = 1728
2481

Direct stress = 42400 ÷ 191 = 222 #
fibre stress = $\frac{10600 \times 12 \times 6}{2481}$ = = 307
529 #/□". ok

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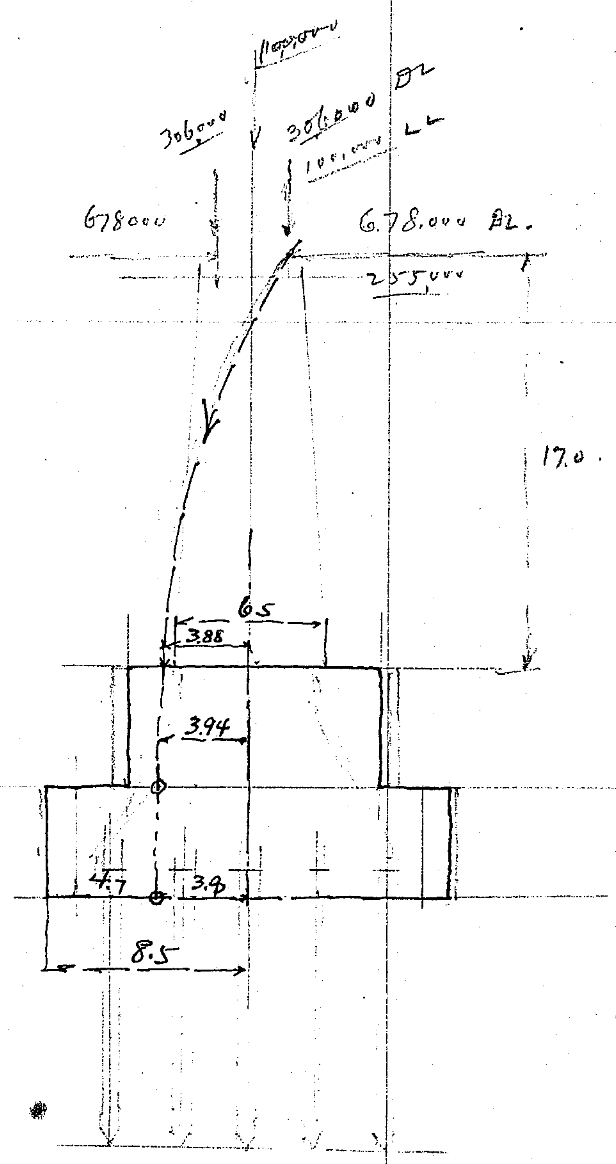
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Design of pier.

80,000
17,500
97,500

Total load. on pier - 100.000
Dead load 2@306.000 = 612,000
live load. 100.000
812,000 #



Weight of shaft.
$\frac{4.5 + 6.5}{2} = 5.5$

5.5 × 26 × 17 = 2430, @150 = 365,000 #
812.000
Total on base - 1,177,000 #

moment due to live load.
255.000 × 17.0 = 4340.000
Eccentricity = $\frac{4340.000}{1.177.000}$ = 3.88'

Base. 11 × 5 × 31.0 = 1650 @ 150 = 247,000
1,177.000
1,424,000 #

moment 255.000 × 22 = 5610.000
Eccentricity = 5610.000 ÷ 1424.000 = 3.94'

Base. 15 × 5 × 31.0 = 2325 @150 = 350.000
1424.000
1,774,000 #

Moment 255000 × 27 = 6.900.000
Eccentricity = 6900.000 ÷ 1774000 = 3.90'

Assuming the width of base 17'-0" instead of 15.
Base 17 × 5 × 31 = 2640 @150 = 396.000.
1424.000
1,820,000

Eccentricity = 6900.000 ÷ 1.820.000 = 3.80'

Pressure area = 4.7 × 3 × 31 = 437.0 □'

Unit bearing say $\frac{1820.000 \times 2}{437}$ = 8300 # 3.7 tons

Revised - Base. 12 × 5 × 31. = 1860 @ 150 = 279.000
18 × 5 × 33 = 2970 @150 = 445.000
724.000
1 177.000
1,901,000 #

Ecc = 6,900.000 ÷ 1901.000 = 3.62

9.0 - 3.62 = 5.38 5.38 × 3 × 33 = 533.

Bearing Pressure = $\frac{1901.000 \times 2}{533}$ = 7120 #/□' or 3.2 tons per sq ft.

3.2 × 9 = 28.8 per pile. Use pile #.62 18.0 ton.

No of piling - 6 × 11 = 66 # per pier.

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CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Approximate Concrete in one pier
Above springing - say 2.0
shaft 2430 ÷ 216 = 11.25
base. 4830 ÷ 216 = 22.40
35.65 call this 36. cu yd

~~Reinforcing bars~~ Concrete in super structure
60 × 67 = 4000 ÷ 216 = 18.6 cu yd
Reinforcing bars in slab etc say 4.5 tons. 4.5, 8.75 } 13.25 tons.

In arch ring. 280 × 70 = 19600# or 8.75 tons. 4.00
In pier. say. 300# × 30 = 9000 ~~call this~~ 4.00 17.25 tons
Excavation - 18 × 33 × 20 / 216 = 55.0 cu yd
~~Total Concrete~~

Approximate Cost of one pier.

Concrete in pier say -	36.0 cu yd	@	110.00	=	3960
Reinforcing bars	4.0 tons	@	150.00	=	600
forms. say.	53. Fd. yd	@	12.00	=	636
Excavation -	55 cu yd	@	18.00	=	990.
piling	66 #	@	~~30~~.00	=	1980
					~~8166~~.00 8500.00

call this ~~9500~~.00 per pier.

abutment assumed. 12000.

9000 @ 10.00 = 9000; 3500 @ 20.00 = 9000, 18

Super structure.

Concrete -	8 @ 18.6 cu yd =	149 cu yd	@	110.00	=	16400.
Reinforcing bars	8 @ 13.25 =	106 tons	@	150.00	=	15900
forms say.		1500 Fd yd	@	~~18~~.00	=	~~27000~~ 17000
pavement.		370 Fd yd	@	16.00	=	~~59200~~ 5920 5920
finish of coping.						1200
drains + expansion joint ~~+ misc~~ say						1500
Handrails.		185 ken	@	60.00	=	11100.
Electric wiring.						1500
monuments -						1000
						~~134800~~.00 71520

substructure 85~~00~~
7 piers @ ~~9500~~ = ~~66500~~.00 59500
2 abutments @ ~~12000~~ 10000 = ~~24000~~ 20,000
80800

Super structure complete — ~~135.000~~ 71.5100
substructure — 80.800
~~215800~~.00
151 800.00

Total Cost of River span ~~215800~~ 151800.00 ÷ 370 = ~~580~~ 410 tsubo
" " " approach 85600
~~300900~~ 290000.00
237400.00

370
310
680.

~~300900~~ 290000 240.000 ÷ 680 = ~~440~~.00 353 yen per tsubo.

360.
320
680

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CALCULATIONS FOR

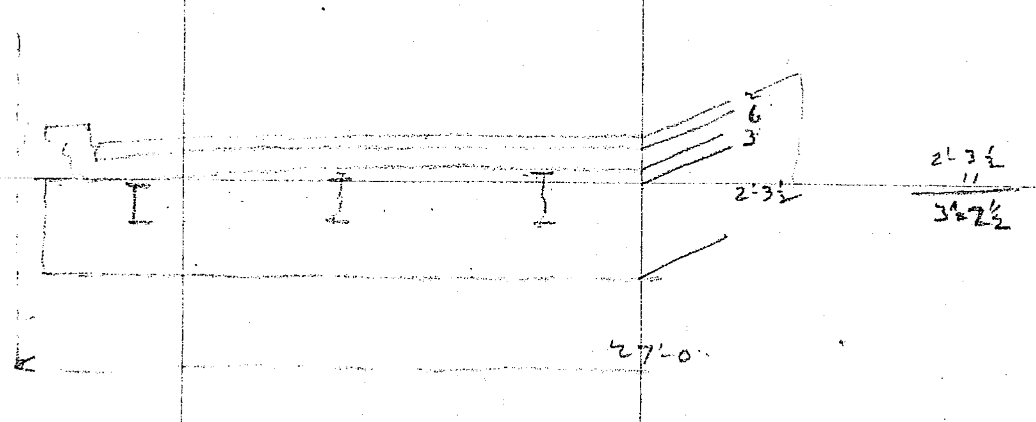
Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Design of truss span

111 c to c of piers. 109 c to c of End Bearings.

panels divided into 10 - 10.9 each

Cross section of bridge -



Floor 2" asphaltic pavement 24

6" Reinforced Concrete 75

say 100.

stringer 12" 5" @ 31.99#

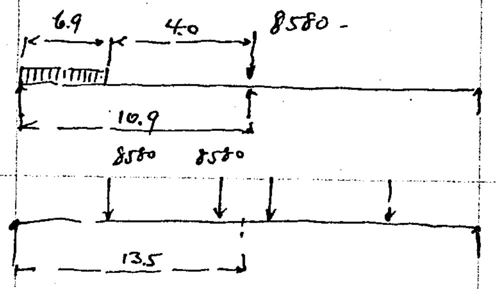
Floor beam Dead Load 110 × 10.9 = 1200# per lin ft.

Floor beam assumed 180

1380# per ft.

D.L. moment = ⅛ × 1380 × 27.0² = 126000'#

Live Load



uniform load 690# × 3.45/10.9 = 218 say 220#

m = ⅛ × 220 × 27² = 20.000'#

m = 17160# × (13.5 - 4.42) 9.08 = 156.000

20.000

Total Live Load m - 176.000

DL m - 126.000

302.000'#

~~web assumed 30" × 3/8 = 11.25□" ⅛ web = 1.41~~

web assumed 27" × 3/8 = 10.12 ⅛ web = 1.27

Effective depth say 2.14 flange stress = 141000 SR = 141000 ÷ 17000 = 8.30

1.27

7.03

2Ls 5 × 3½ × ½ - 8.00 - or 7.00 □" net.

weight of girder = 10.12

16.00

26.12 @ 3.4 = 89.

30%. 30

119# per lin ft.

120 × 26 = 3120# per piece. 3120 ÷ 10.9 = 286# per lin. ft.

Lateral Bracing say 60# per lin ft.

Design of ~~floor~~ truss

Dead Load. flooring 100 × 24 = 2400

Coping say 300

Handrails say 200

2900# per lin ft.

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CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Dead load metal.
Stringer 6 @ 35 = 210
floor beam 286
Lateral system 60
trusses assumed 1000
1556
2900
4456 ÷ 2 = 2230# per lin ft of truss.

Live load 120 × 12 = 1440
3670#

moment = ⅛ × 3670 × 109² = 5440.000#
Depth 15.0

Stress = 5440.000 ÷ 15.0 = 363,000
SR = 363 000 ÷ 14000 = 26.00□"

Top and bottom section 2 @ 26.0 = 52.
vertical + diagonal. 45
107 @ 3.4 = 365#
Details say 40% 145
510# ok

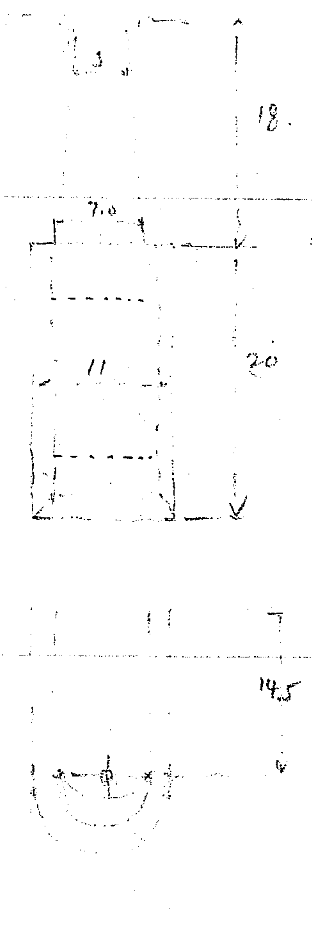
For two trusses 2 @ 510 = 1020#

Approximate weight of structural steel –
stringers = 210 × 111 = 23310
floor beam 11 @ 3120 = = 34320
Laterals. 6600
trusses. 1020 × 109 = = 111000
shoes + c. 5000
180230 ÷ 2240 = 80.5 tons.

5 spans @ 80.5 = 402.5 tons.
call this 410.0 tons.

Design of pier.
Load on pier – Dead Load 3670 × 111 = 407,000
Live Load 2880 × 111 = 320.000
727,000#

Shafts.
5.0 dia 19.6
7.0 dia 38.5
58.1 ÷ 2 = 29.0
6 × 29 = 174.0
203. × 18.0 = 3660. ÷ 216 = 17.0 立坪



well.
11.0φ 95.0
8.5φ 56.7
38.3 □'
29 × 2.5 = 72.5
110.8 × 20.0 = 2220. ÷ 216 = 103 立坪

Top and bottom filling.
56.7
8.5 × 29.0 247.0
303.7 × 10 = 3037 ÷ 216 = 14.0 立坪
Intermediate filling 14.0 "

Excavation 95.0
11 × 29 = 319.0
414.0 × 24 say = 9950 ÷ 216 = 45.6 立坪

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CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

weights of pier

shaft	17.0	@ 32400	=	551.000
well shell	10.3	@ 32400	=	334.000
filling	14.0	@ 32400	=	454.000
"	14.0	@ 30200	=	423.000
				1,211.000
				551.000
				1,762.000
Dead and Live Loads.				727.000
				2,489.000 #

Bottom area = 414.

2,489.000 ÷ 414 = 6000 #/□' or 2.68 tons per sq ft.

Estimate of cost of one pier.

Concrete in shaft.	17.0	@	110.00	=	1870.00
well say	10.3	@	110.00	=	1135.00
filling say	14.0	@	110.00	=	1540.00
"	14.0	@	65.00	=	910.00
Reinforcing bars say	6.0 tons	@	150.00	=	900.00
forms say					500.00
Curb shoes	2.5	@	220.00	=	550.00
Excavation say	45	@	20.00	=	900.00
					8305.00

Estimate of Deck.

Concrete in floor	0.5 × 24 =	12.0		
Coping.		2.0		
		14.0 × 555/216	=	36.0 立坪

Reinforcing bars. 26. tons.
forms say 410 坪.

Estimate of Cost

Concrete	36.0 立坪	@ 110.00	=	3960
Reinf. bars.	26. tons	@ 150.00	=	3900
forms	410	@ 7.00	=	2870
pavement	370 平坪	@ 16.00	=	5920
finish of coping				1200
drains + Exp. Joints &c				1500
Handrails	185 Ken	@ 60.00		11100
Electric wiring				1500
monuments				1000
				32950 call this 33000

Substructure.	5 piers @ 8300	=	41500
structural steel.	410 tons 260.00	=	106800.
	1 abutment.		8000.
Deck.			33000
			189300 call this 190.000.00
	approach span		85600
			275600.00

41500
8000
49500

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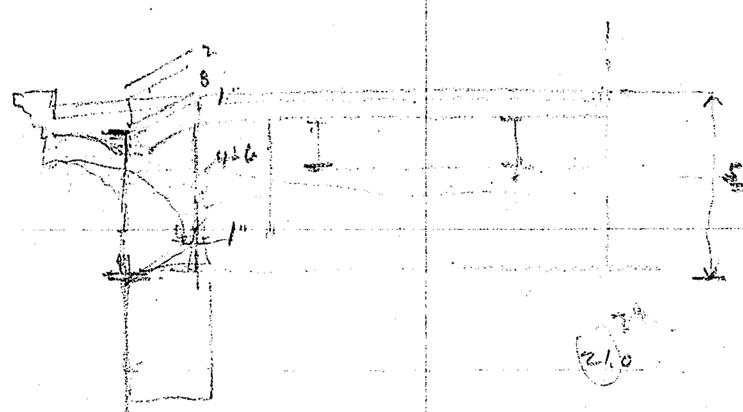
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Design of girder span 7 spans @ 79'-0" = 553'
Call this 80' span overhang - 20' center span 40'-0"
60' simple span 80' span with 2-20' hanging arm
Cross section of bridge.



Roadway - 2" asphalt 24
6" Concrete 75
say 100# per sq ft.

Stringer 4 @ 35# = 140#

Beam 210#
Lateral 60
410# per lin ft.
Girder assumed. 600
1010

Dead Load of floor. 100 × 24 = 2400
Coping - 300
Handrail - 200
2900
1010
3910 ÷ 2 = say 1950# per lin ft of one girder.

60' girder span
Dead Load - m = ⅛ · 1950 × 60² = 880 000
Live Load say 120 × 12 = 1440# per lin ft.
m = ⅛ × 1440 × 60² = 647,000
1,527,000'#

Depth 54" × 3/8 = 20.25 ⅛ web = 2.53 □"
Effective depth 4.3 flange stress = 1527,000 ÷ 4.3 = 356.000#
Section required = 356.000 / 17000 = 21.0
2.53
18.47 □" net.

2∟s 6 × 6 × 3/4 = 16.88 - 13.88
1 Pl. 14 × ½ = 7.00 6.00
23.88 19.88 □" net.

approximate weight of 60' girder -
flanges. 48.
web. 20
68 @ 3.4 = 232#
Details say 30% . 68
300# per lin ft of one girder.
600# per lin ft of span

40' Girder span
Dead Load m = ⅛ · 1950 × 40² = 390,000
Live Load m = ⅛ · 1440 × 40² = 288,000
678,000'#

Depth of girder - 54 × 3/8 - 20.25 ⅛ web = 2.53 Effective depth say 4.3'
flange stress = 678,000 ÷ 4.3 = 158,000# SR = 158,000 ÷ 17000 = 9.30
2.53
6.77 □".

use 2∟s 6 × 6 × 7/16 - 10.12 - 9.25 □"
10.12
20.25
40.5 @ 3.4 = 138
30% - 42
180# per lin ft.

For two girders 360# per lin ft of span

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CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Cantilever arm 20' for 60' span

End Concentration. D.L. ½ × 1950 × 60 = 58,500

L.L. ½ × 1440 × 60 = 43200

3390# 101,700

m = 101700 × 20 = 2040.000

$3390 \times \frac{20^2}{2}$ = 677,000

2717,000 1#

Depth. 6.5' 78" × ½ = 39.0 □" ⅛ web = 4.87 □"

Effective depth say 6.3 flange stress = 2,717.000 ÷ 6.3 = 430.000

section reqd = 430.000 ÷ 17000 = 25.3

4.87

20.43 □"

2 Ls 6×6 × 3/4 = 16.88 – 13.88

1 Pl. 14 × 3/4 = 10.50 9.00

27.38 22.88

27.38

39.00

93.76 @ 3.4 = 320#

30% – say 80

400#

For two girders – 2 @ 400 = 800# per lin ft.

Center span 80'-0" –

Dead Load – say 2100#

moment = ⅛ × 2100 × 80² = 1,680.000 1#

Less. $2100 \times \frac{20^2}{2}$ = 420.000 1260.000

$2100 \times \frac{40}{2} \times 20$ = 840.000 420 000 1#

1260 0000

Live Load. 1440# per lin ft.

moment = ⅛ × 1440 × 80² = 1150.000

1570,000 1#

Depth of girder 54 × ½ = 27.0 □" ⅛ web = 3.38 □"

Effective depth 4.3 flange stress = 1570.000 ÷ 4.3 = 365.000#

Section required = 365000 ÷ 17000 = 21.50

3.38

18.12 □" net.

Use 2 Ls 6×6 × 3/4 = 16.88 13.88

1 Pl 14 × ½ = 7.00 6.00

23.88 19.88

flanges say 48.

web 27.

75 @ 3.4 = 255#

details say 30% say 75

330# per lin ft.

For two girders 660# per lin ft.

Approximate weight of structural steel in girder.

II –	60' span –	600#	= 36000#	2 @ 36000	= 72000#
	40' span –	360	= 14400#	2 @ 14400	= 28800
	20' span –	800	= 16000	12 @ 16000	= 192000
	40' center span	660	= 26400	3 @ 26400	= 79200
					372000#

Stringers – 140 × 555 = 77600#

Floor beam say 230# × 555 = 128000

Laterals. say 60 × 555 = 33300

238900#

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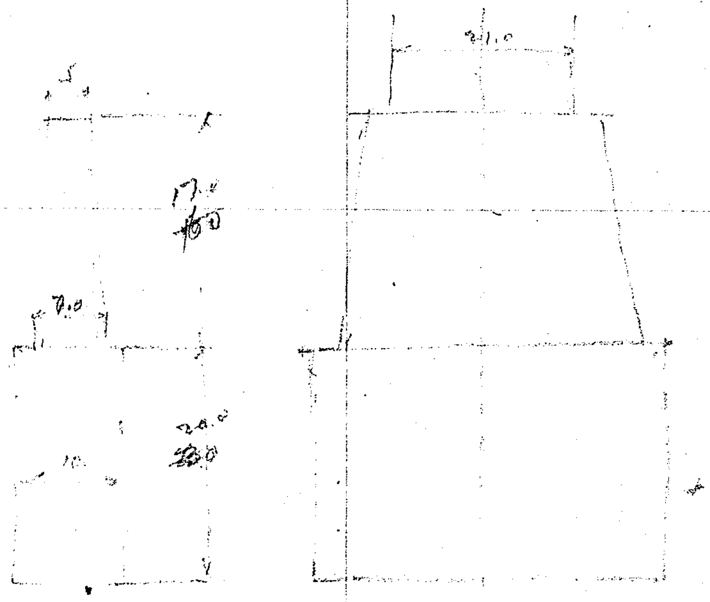
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Summary for structural Steel.

Girders	372000
floor system	238900
shoes &c	40000
	650900 ÷ 2240 = 290 tons

Pier.



top area - 5'φ = 19.6
5×21 = 105.0
124.6

bottom 7'φ = 38.5
7×21 = 147.0
185.5

124.6
185.5
310.1 ÷ 2 = 155.0

vol = 155 × 17.0 = 2640 ÷ 216 = 12.2 立坪

well shell -
10'φ = 78.5
10×21 = 210.0
288.5

Inside 7.5φ - 44.2
7.5×21 = 157.5
201.7

288.5
201.7
86.8 □'

vol of shell - 868 × 20 = 1740 ÷ 2160 = 8.05 立坪

top and bottom filling - 201.7 × 10 = 2017 ÷ 216 = 9.3 立坪
Intermediate filling - 9.3 "

Weight of pier	shaft	12.2	@	32400	=	395.000
	shell	8.0	@	32400	=	259.000
	top & bottom filling	9.3	@	32400	=	301.000
	Int. filling	9.3	@	30200	=	281.000
						1.236.000
Superimposed load	4000					555.000
	2880					1.791.000
	6880 × 80 = 555000					

Unit bearing - 1.791.000 ÷ 288.5 = 6220 #/□' or 2.78 tons/□'

Estimate of cost of one pier.

Concrete in shaft	12.2 立坪	@	110.00	=	1345
shell	8.0	@	110.00	=	880
filling	9.3	@	110.00	=	1020
"	9.3	@	65.00	=	605
Reinf. bars say	4.5 tons	@	150	=	675
forms say					400
curb shoes	2.0	@	220.00	=	440
Excavation -	33.0 立坪	@	20.00	=	660.
					6025 - call this 6000.00 yen

Substructure -

7 @ 6000 =	42000
1 abut @	8000
	50000.00
Superstructure steel 290 tons @ 240 -	69500.00
Cost of Deck	33000
	152500.00
Approach span	85600
	238100.00

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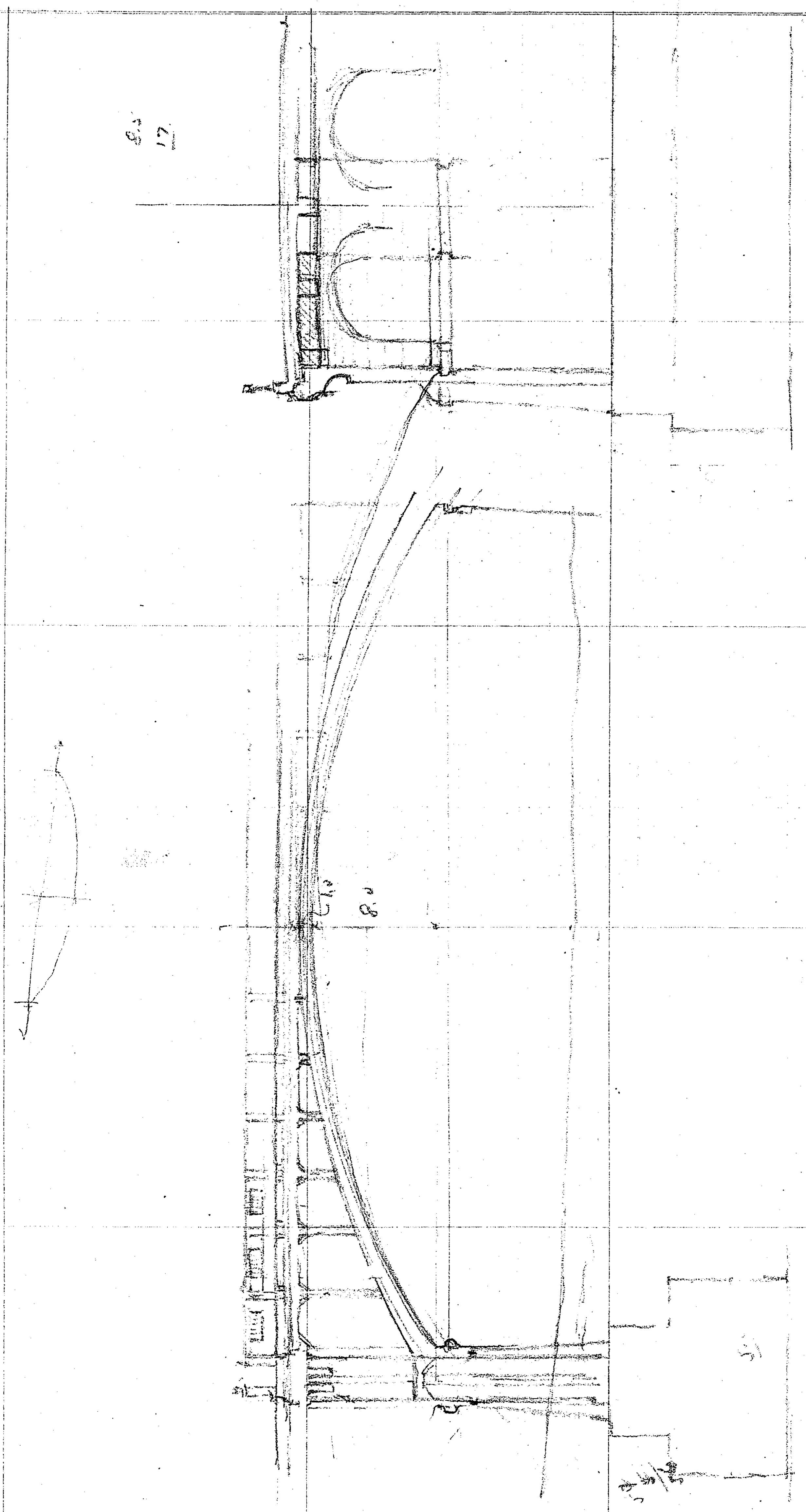

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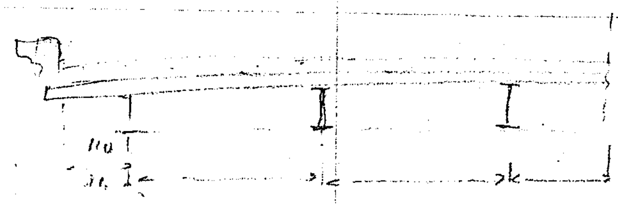
CALCULATIONS FOR

Preliminary Estimate of Katsura-Bashi for Kioto Prefecture.

Design of tied arch.
span length 136' 14@ 9.7
Center to center of trusses 27'0" Roadway 24'0".

Floor 2" Asphalt block 24
1/2 cushion 6
Reinforced Concrete 75
105# call this 110# per sq ft.

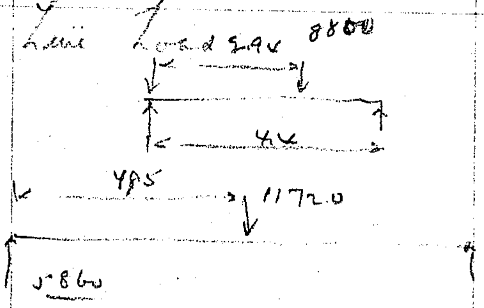
4.4' spacing



Stringer span length 9.7' 4.4 × 110 = 485#
30
515#

m = 1/8 × 515 × 9.7² = 6050'#

Live Load 8800
4.4 / 2.94 / 1.46
6600 / 2200 / 8800#
8800 × 1.46/4.4 = 2920
8800
11720#



LL m = 5.86 × 4.85 = 28400'#
DL m 6050
34450'#

Sm = 34450 × 12 / 15400 = 26.8

Use 12" I × 31.99

Floor beam. 3000# ÷ 9.7 = 310# per lin ft.
Lateral Bracings. 60# per lin ft.

Stringers 6 × 35 = 195
Floor beam 310
lateral Bracings 60
565# per lin ft.

Design of truss.
Dead load floor. Flooring 110 × 24 = 2640
Coping say 300
Handrails say 200
3140# per lin ft.

Floor system 565
Top laterals 75
trusses assumed 1500
2140# per lin ft.
Flooring 3140
5280# ÷ 2 = 2640# per ft of truss.
Live load say 120 × 12 = 1440
4080# per ft of one truss

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CALCULATIONS FOR

Preliminary Design of Katsura-Bashi for Kioto Prefecture.

Span length 136' Height 21'

moment = $\frac{1}{8}$ · 4080 · 136² = 9,450,000'#
500

2,000,000 26 1,000,000 stress = 9,450,000 ÷ 21 = 450,000#
 SR = 450,000 ÷ 14,000 = 32.00"

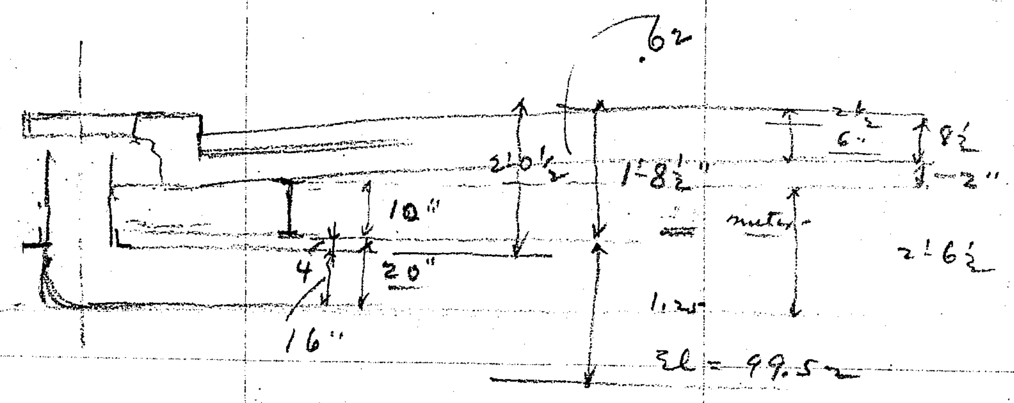
32.0 @ 3.4 = 110# × 5.6 = 615# per lin. ft.
32.0
32
2 @ 615 = 1230# per ft.

110
110
110
50
384 × 1.40 = 530#
100
630

trusses - 1230#
floor - 565
top lateral 75
1870# × 138 = 258,000# or 115 tons.

4 @ 115 = 460 tons.

structural steel.	460	@ 270 =	124,000
4 piers	@ 9000		36,000
1 abutment.			8000
1 Deck construction		say	33000
			201,000
approach span			85600
			286600



1.71

99.52
.62
1.25
101.39
99.82
1.57 "5.15"

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CALCULATIONS FOR

1085'
5
545' 14
1366
14
12
25
24

4.15
166°

14
1.5
2.075

6.9'

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CALCULATIONS FOR

昭和二年八月成

京都府桂橋設計々算書

及材料調書

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture.

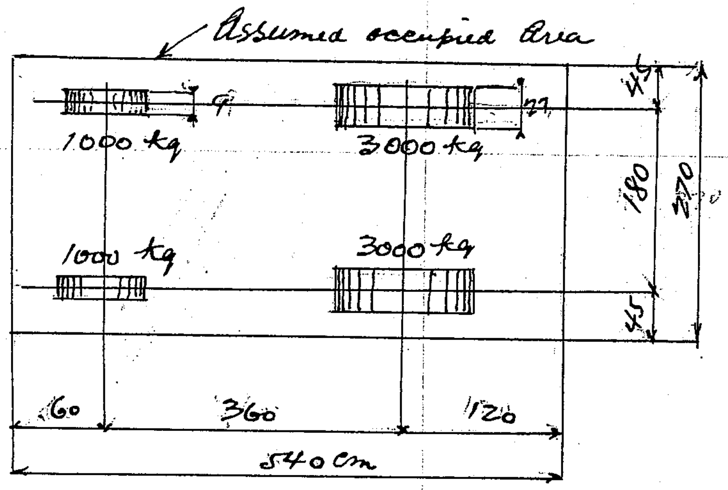
This bridge is on the crossing of Katsura-gawa on No 18 main national highway from City of Kioto to Tottori prefecture. The total crossing is 311.75 meters between edges of banks and 305.875 meters between faces of parapet walls of abutments. The bridge consists of 7 plate girder spans of Gerber-träger and 5 sets of 3 continuous girders, each span length of 8.75 meters. The latter will be made of reinforced concrete.
The width of roadway 7.5 meters between curb lines, with ornamental hand rails on both sides. The pavement will be of asphalt block 2" = 5c thick on mortar cushion. The floor will be of reinforced concrete construction on steel stringers or monolithic with concrete beam

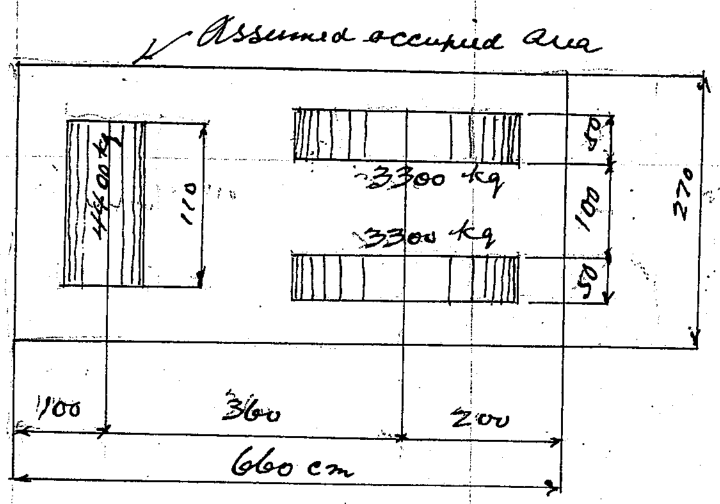
Assumed loadings

Uniform load on Roadway $w = \frac{100.000}{170+l} \leqq 500\ ^{kg}/\square^{m}$

where w = Uniform load in kg per square meter
l = span length in meter

Motor truck loading 8 ton motor truck Road roller loading 11 ton





2 rows of motor traffic on roadway with occupied width of 270 cm each;
Unoccupied space besides motor trucks to be filled with uniform load
Impact for motor truck loading

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

No impact for roadroller and uniform load.
Road Roller assumed One on one span

Allowable working strength
Structural steel or Reinforcing bars.

Tension	1200 kg/cm²	or	17000 #/□
Extreme fibre stress	1200 kg/cm²	"	"
Shear of web gross section	900 "	"	12800

Compression member

$1500\ (1 - 0.0055\ l/r)$ or not over $1.000\ ^{kg}/cm^2$
where l = length of member in cm
r = least radius of gyration in cm²

Equivalent formula in inch-lbs
$21300\ (1 - 0.0055\ l/r)$ or not over $14000\ \#/\square$

Compression flange of plate girder
$1200\ (1 - 0.012\ l/b) \leqq 1100\ ^{kg}/cm^2$
where l = Unsupported length of flange in cm
b = width of flange in centi-meter.

Equivalent formula in inch-lbs
$17000\ (1 - 0.012\ l/b) \leqq 15400\ \#/\square$

Shearing on shop driven rivets (machine driven) 12000 #/□
Shear on field driven rivets and turned bolts (machine driven) 10.000 #/□

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture

Extreme fibre stress of pin	24000	#/□"
Bearing on shop rivets	24000	"
Bearing on field rivets and turned bolts	20000	"
Bearing on pin	24000	"

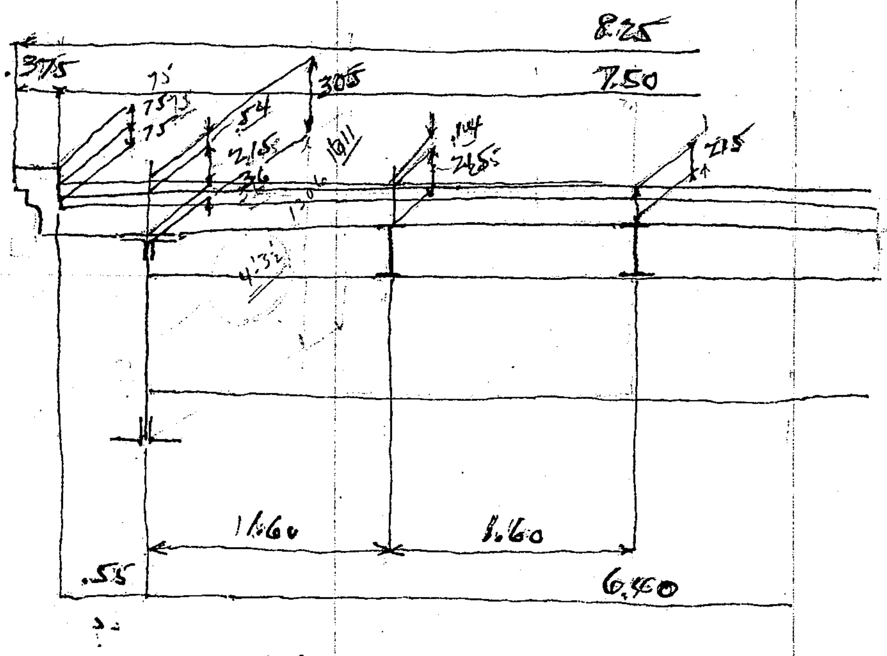
Expansion roller 45d kg/cm where d = diameter of roller in cm
In inch-lbs. 610d # per lin inch where d = diameter of roller in inches

Strength of Concrete 1:2:4 mixture

Bearing	45 kg/cm²	or	640 #/□"
Compressive fibre stress	45 "		640 "
Shear for plain concrete	4 "		58 "
Punching shear	9 "		128 "
Bond stress of plain bar	6 "		85 "
Bond stress of deformed bar	9 "		128 "
Shear for reinforced concrete	9 "		128 "

Considering wind and temperature stresses in addition to dead live and impact stresses, the allowable working strength shall be increased 25%.
Considering earthquake, the working strength shall be increased 80%.
Seismic Acceleration assumed 2000 mm/sec²

Design of Girder span
Cross section of girder span as shown on sketch below.



Floor slab. span length 1.60 meters
Dead Load

5C	Asphaltic block pavement	@ 21 kg.	= 105
1.5C	mortar cushion	@ 22. kg.	= 33
15. C	Concrete slab.	@ 24. "	= 360
	miscellaneous Concrete say		12
			510 kg per sq. meter

Dead Load moment $= \frac{1}{10} \times 510 \times 1.6^2 = 131.0$ kgm
Dead Load shear $= \frac{1}{2} \times 510 \times 1.6 = 408.0$ kg.

Live Load motor truck loading

Rear wheel concentration	= 3000 kg.	Front wheel Conc.	=	1000
30% impact	= 900	30% impact	=	300
	3900			1300 kg.

Distribution of wheel concentration on slab.
Thickness of pavement and mortar cushion = 6.5 cm
Longitudinal distribution a Contact between wheel + pavement 20
distribution 2 @ 6.5 = 13
a = 33

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture

Transverse distribution b $27^{c} + 13 = 40$

Effective width $E = \frac{2}{3}(l+b) + a$ where l = span length

$= \frac{2}{3}(1.6 + .40) + .33 = 1.66$

Say Effective width assumed 1.5 meter

Load per meter strip $= 3900 \div 1.5 = 2600$ kg.

moment per meter strip $= 1300 \times \frac{1.6}{2} = 1040$ kg-meter

For Continuity of slab. $0.8 \times 1040 = 832.0$ kg-meter

End shear

$2600 \times \frac{.50}{1.60} = 812$

$2600 \times \frac{1.40}{1.60} = 2275$

3087 kg.

2600 2600

.20 .90 .50

1.60

Summary for moment and shear

	moment	shear
Dead Load	131	408
Live Load	832	3087
	963 kgm	3495 kg.

Effective depth required for slab.

$f_s = 1200$ kg/cm² $f_c = 45$ kg/cm²

$K = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{bK}}$

where $K = 7.18$

$d = \sqrt{\frac{963 \times 100}{100 \times 7.18}} = 11.6$ cm

Use 15c slab.

Effective depth say 12.5 c

Steel area $= \frac{963 \times 100}{\frac{7}{8} \times 12.5 \times 1200} = 7.32$ per lin. meter

$\frac{1}{2}''\phi$ bars. $\frac{1.27 \times 100}{7.32} = 17.3$ centimeter sp.

Use 15 centimeter spacing.

Unit shear $= \frac{3487}{\frac{7}{8} \times 12.5 \times 100} = 3.18$ kg/cm²

Dia.	area		circumference
$\frac{1}{2}''$ = 12.7 mm	.1963 □"	= 1.27 □cm	4.0 cm
$\frac{3}{8}$ = 9.53 "	.110 □"	= 0.71	3.0 cm

Bond stress.

Use $\frac{1}{2}''$ bars 15cm } spacing
$\frac{3}{8}''$ bars 15cm

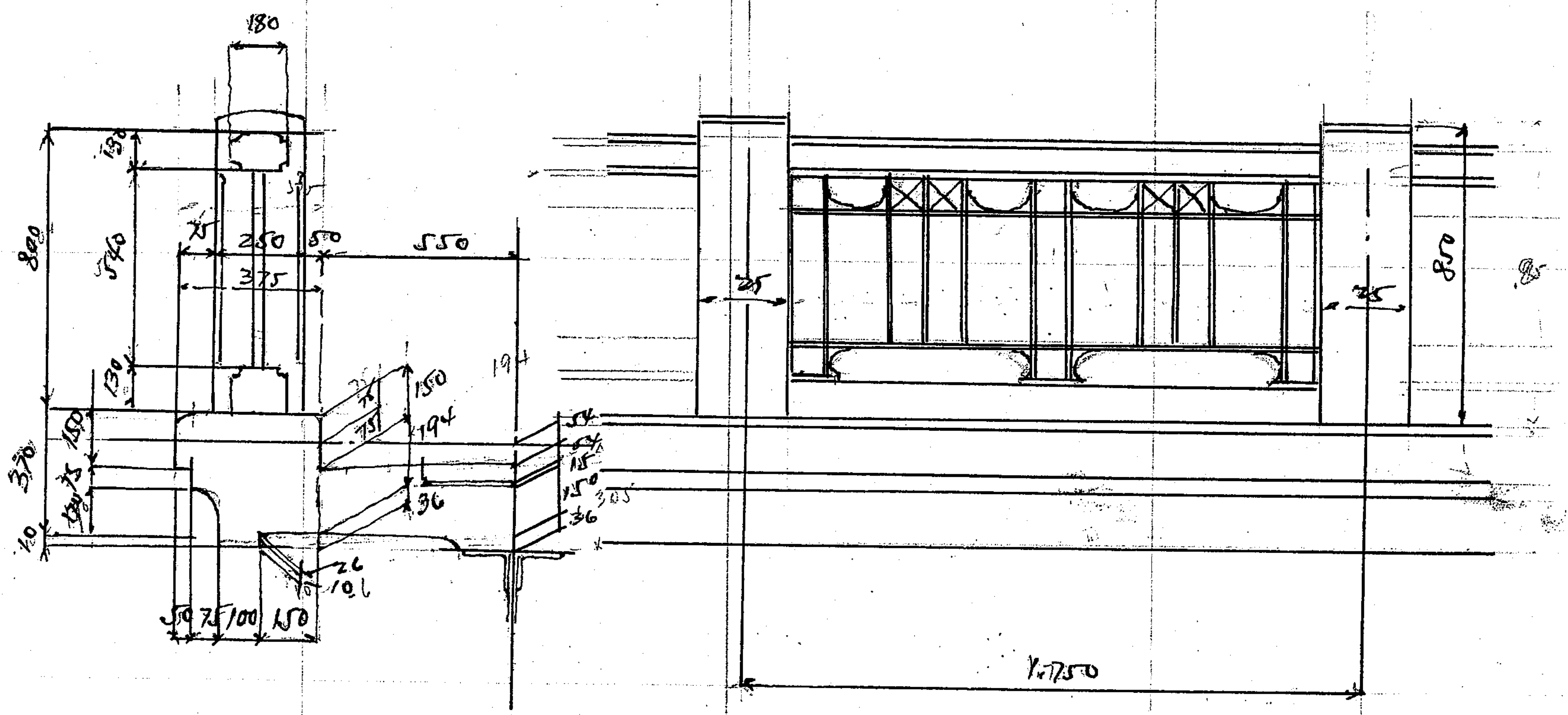
Circumferencial area $4.0 \times 6.67 = 26.7$

$3.0 \times 6.67 = 20.0$

46.7 □cm

Unit bond stress $= \frac{3495}{\frac{7}{8} \times 12.5 \times 46.7} = 6.83$ kg/cm² Use deformed bars in slab.

Overhanging slab beyond main girder.



Details of Coping and Handrail. Scale 1/20

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto Prefecture

Handrail. Concrete post and top and bottom rails with cast iron panel.
Spacing of post 3.5 ÷ 2 = 1.75 meter
Approximate volume of concrete in Handrail.

Post	0.25 × 0.25 × 0.85	=	0.094
Top and bott rails	2 - .130 × .18 × 1.50	=	0.070
			0.164 @ ~~2400~~ 2400 kg = ~~360~~ 394 kg.

Approximate weight of ornamental cast iron panel.

Hor.	4 - .020 × .030 × 1.50	=	.0036
vert etc	12 - .020 × .030 × 0.50 about	=	.0036
			.0072 @ 7850 = 52
			446 kg.

446 ÷ 1.75 = 255 kg per lin meter

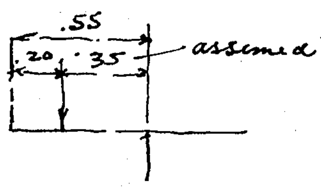
Approximate weight of coping.

.150 × .344	=	0.051
.100 × .370	=	0.037
.200 × .125 about	=	0.025
		0.113 @ 2400 = 272 kg per meter

Dead Load moment

	load		arm		
Handrail	255	×	.725	=	185
Coping	272	×	.725	=	197
slab + pavement 510 × .55 =	280	×	.275	=	77
	807 kg				459 kg meter

Live Load



Distribution of wheel concentration assumed
2 @ .35 + .20 = .90 meter at center line of girder.
Motor truck rear wheel concentration with impact = 3900 kg.
Load per ~~ft~~ meter strip = 3900 ÷ .90 = 4340 kg.
Live Load moment = 4340 × .35 = 1520 kgm

Live Load shear assumed 4340 kg per meter strip

Summary for moments and shears

	moment		shear
Dead Load	459	say	807
Live Load	1520		4340
	1979 kgm		5147 kg.

Effective depth required for $f_s = 1200$ kg/cm² , $f_c = 45$ "

$d = \sqrt{\frac{1979 \times 100}{100 \times 7.18}} = 16.6$ centimeters.
Insulation 2.5 "
18.1 "

Depth of slab above back of flange angle of main beam = 18.6 centimeter OK.
Effective depth say 18.5 cm

Steel area $= \frac{1979 \times 100}{7/8 \times 18.5 \times 1200} = 10.20$ □cm per lin meter

15c spacing 1/2" bars	1.27 □cm	×	6.67	=	8.45
sp do 3/8 "	0.71 "	×	6.67	=	4.73
					13.18 □cm per meter strip.

Circumferential area	1/2"	4.0	× 6.67	=	26.7
	3/8"	3.0	× 6.67	=	20.0
					46.7

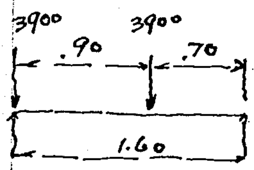
Unit bond stress $= \frac{5147}{7/8 \times 18.5 \times 46.7} = 6.82$ kg/cm² Use deformed bars in slab to carry necessary bond stress.

Design of I beam stringer span length 3.5 meters spacing 1.6 meters.
Dead Load
floor slab and pavement 510 × 1.6 = 816
beam assumed 32# I beam with details 52
868 kg per meter

Dead Load moment = 1/8 × 868 × 3.5² = 1330 kgm
Dead Load shear = 1/2 × 868 × 3.5 = 1520 kg.

Design of Katsura-Bashi for Kioto-Prefecture.

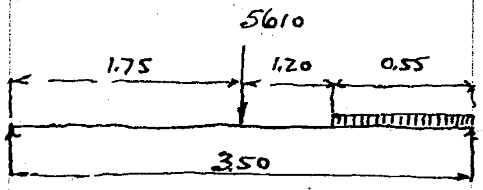
Live Load motor truck rear wheel concentration with impact = 3900
" " Front " " " " = 1300



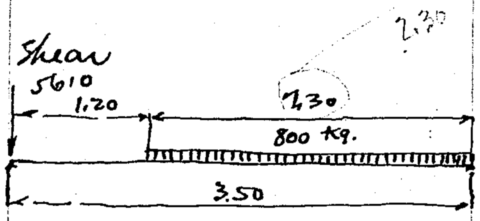
Load on Stringer 3900 × $\frac{.70}{1.60}$ = 1710
3900
5610 kg.

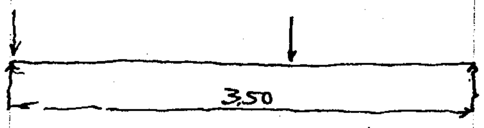
Uniform live load = 500 × 1.6 = 800 kg per lin meter of span

Live Load Moment



moment
due to motor truck 2805 × 1.75 = 4910
Due to unif. load 800 × 0.55 = 440*
440 × $\frac{0.275}{350}$ × 1.75 = 60
4970 kgm





Reaction due to Unif. load $\frac{800 \times 2.30^2}{2 \times 3.50}$ = ~~193~~ 604

Due to motor truck 5610 ---- 5610
~~5803~~
6214

Summary for moments and shears.

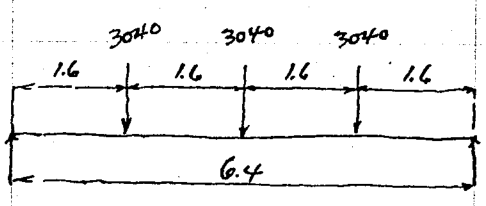
	moment	shear
Dead Load	1330	1520
Live Load	4970	6214 ~~5803~~
	6300 kgm	~~7323~~ 7734 kg

× 7.23 = 45600# 2.2 = ~~16100~~ # 17100

Section modulus req'd = $\frac{45600 \times 12}{15400}$ = 35.50 Use 12" × 5" @ 31.99# Sm = 36.69

weight of stringer with detail 35# per lin ft or 52 kg.
3 @ 52 = 156 kg per lin meter.

Intermediate Floor Beam span length 6.4 meter spacing - 3.5 meter
Dead Load
Dead Load Concentration - floor 510 × 1.60 = 816
52
868 kg



868 × 3.5 = 3040 kg.

moment = 4560 × 3.2 = 14600
3040 × 1.6 = -4870
9730 kgm

3040
1520
4560
800
5360

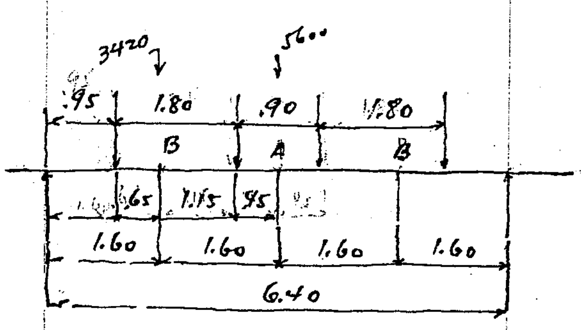
250 × 3.2 = 800

Dl. beam $\frac{1}{8}$ × 250 × 6.4² = 1280
11010 kgm

End shear = 5360 kg.

Live Load.

Motor truck rear wheel with impact 3900. kg.
Load on B. 3900 × $\frac{.95}{1.60}$ = 2320
3900 × $\frac{.45}{1.60}$ = 1100
3420 kg.



Load on A 3900 × $\frac{1.15}{1.60}$ = 2800
2800
5600 kg.

Uniform load 800 × $\frac{1.30^2}{2 \times 3.50}$ = 193.

Stringer Concentration = 193

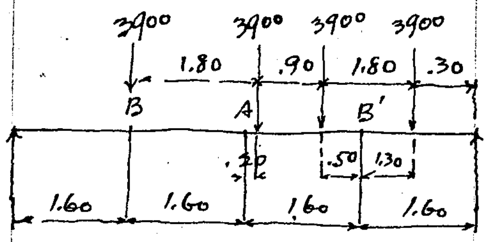
Moment due to motor truck 6220 × 3.2 = 19900
3420 × 1.6 = -5470
14430 kgm

Moment due to Unif. load 193 × 1.5 × 3.2 = 927
193 × 1.6 = -309
618
15048 kgm

CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture.

Max End shear



Concentration on B - 3900 kg.
Concentration on A. $3900 \times \frac{1.90}{1.60} = 4625$ kg
Concentration on B' $3900 \times \frac{1.60}{1.60} = 3900$ kg

End shear = 3900 + 2313 = 6213. kg.
193 × 1.5 = 290
6503 kg

Summary for moments and shears

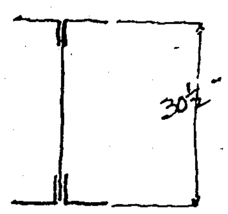
	Moment	Shear
Dead Load	11010	5360
Live Load	15048	6503
	26058 kgm	11863 kg.
	× 7.23 = 188 000#	26100 lbs.

web assumed = 30 × 5/16 = 9.38 □" 1/8 web = 1.17
Effective depth = 2.40
flange stress = 188 000 ÷ 2.4 = 78.300#
Section req'd = 78300 ÷ 17.000 = 4.61 □"
1.17
3.54 □" net.

Try ~~Use~~ 2∠s 5 × 3½ × 3/8 = 6.10 - 5.44 □" net.
Unsupported length of flange 1.6 meter or 5.25' or 63"
Unit stress $= 17000 \left(1 - 0.012 \times \frac{63}{10.31}\right) = 15750$ #/□" use 15400 #/□" for top flange of girder.

Gross section of flange = 6.10
1/8 web. 1.17
Good for ~~moment of~~ stress 7.27 @ 15400 = 111.500#

Moment of inertia of girder in gross section. Try 2∠s 4 × 3½ × 3/8 = 5.34 □"



1 web. 30 × 5/16 = 703.
4∠s 4 × 3½ × 3/8 = 10.68 □" × 14.29² + 12.0 = 2180
2885 in⁴

fibre stress $= \frac{188000 \times 12 \times 15.25}{2885} = 11950$ #/□"

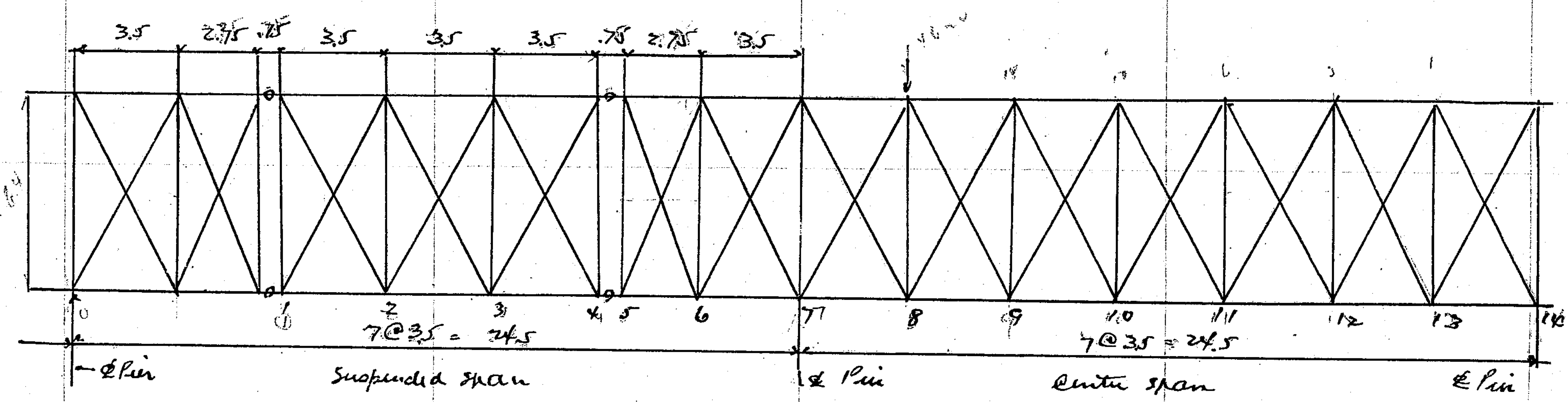
Unit flange stress $= 17000 \left(1 - 0.012 \times \frac{63}{8.31}\right) = 15400$ #/□" OK.

Approximate weight of intermediate floor beam.
Span length 6.4 m = 21.0'

	1 web.	30 × 5/16	@ 31.88#	× 21.0	=	670
	4∠s	4 × 3½ × 3/8	@ 9.1	× 21.0	=	765
Stiffs	14∠s	3 × 3 × 5/16	@ 6.1	× 2.5	=	214
End stiff	2∠s	5 × 3½ × 3/8	@ 10.4	× 2.5	=	52
fills	2 fills	5 × 3/8"	@ 6.38	× 1.9	=	24
stringer seat	3 pls.	8½ × ½	@ 14.45	× 1.0	=	43
	Rivet heads + variation 5%					87
						1855# or 843 kg.

843 ÷ 6.4 = 132 kg per meter of girder
843 ÷ 3.5 = 241 kg per meter of main span.

Lateral Bracing



note:- Panels of anchor span same as for center span

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto Prefecture.

wind pressure
400 + 200 = 600 kg per lin meter of span
Panel Concentration 600 × 3.5 = 2100 kg.

Earthquake Horizontal force
Approximate Dead Load

Slab and pavement	510 × 7.5	=	3825
Copings	2 @ 272	=	544
Handrails	2 @ 255	=	510
			4879 call this 4880 kg.

Structural steel — structural steel 1717

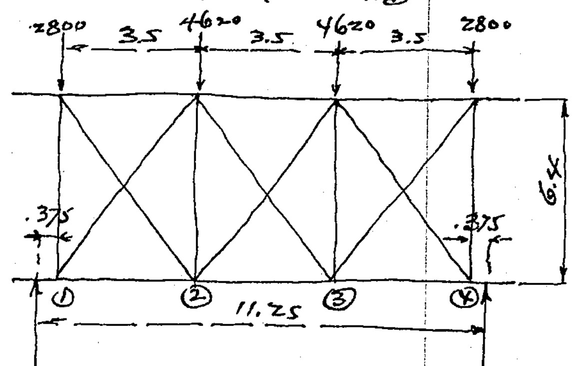
Stringers	156
floor beam	241
Lateral system assumed	120
main girder assumed	1200
	1717

6597 say 6600 kg per meter of span

Panel Dead Load Concentration 6600 × 3.5 = 23100 kg
Horizontal force due to Earthquake 23100 × 0.2 = 4620 kg. panel load.

Design lateral system for Earthquake force
6600 × 0.2 = 1320 kg per lin. meter

Suspended Span
sec θ = 7.35 ÷ 6.4 = 1.15

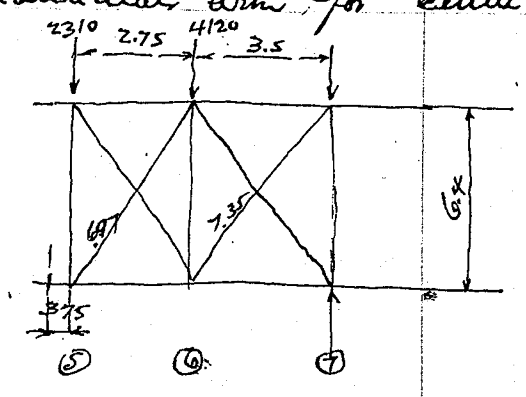


panel Concentration 1320 × 3.5 = 4620 kg
1320 × 2.125 = 2800 ″

		Shear		Stress
Shear for 3-4 say		4620 × 1.15	=	5320 kg

Shear for 2-3 assumed ⅓ × 4620 = 1540
Stress = 1540 × 1.15 = 1770 kg.

Cantilever arm for centre span



Panel Concentration 1320 × 1.75 = 2310 kg.
1320 × 3.125 = 4120 ″

Shear for 5-6.
4600
2800
2310
9710 Diagonal stress = 9710 × 1.09 = 10.620 kg.
4120
6-7 13830 Diagonal stress = 13830 × 1.15 = 15.930 kg.

Extra reaction at 7
moment at 7

7420 × 6.625	=	49100	
2310 × 6.25	=	14420	
4120 × 3.50	=	14420	
		77940 ÷ 24.5	= 3180 kg

			Shear			
7-8	4620 × 3	=	13860			
			3180			Diagonal stress
			17040	× 1.15	=	19600 kg
8-9	4620 × $\frac{15}{7}$	=	9900			
			3180			
			13080	× 1.15	=	15050 kg.
9-10	4620 × $\frac{10}{7}$	=	6600			
			3180			
			9780	× 1.15	=	11250 kg.
10-11	4620 × $\frac{6}{7}$	=	3960			
			3180			
			7140	× 1.15	=	8200 kg.

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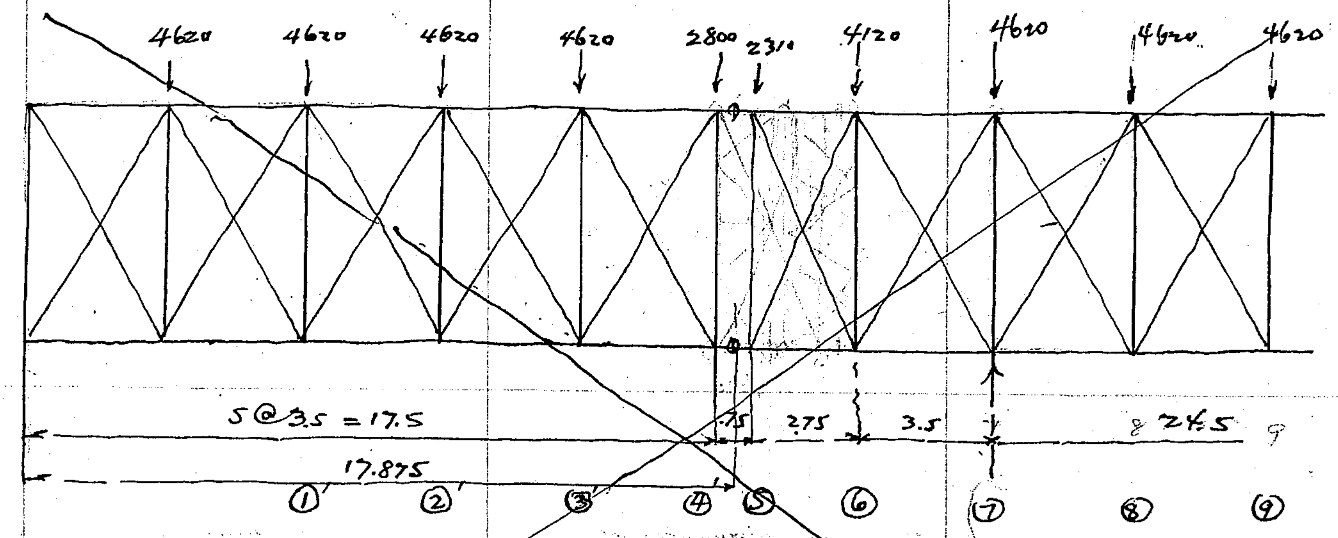
CALCULATIONS FOR

Design of Katsura-Bashi for Kioto Prefecture

Side span



VOID

Panel	shear		stress in diagonal.
1'-2'	2770 × 1.15	=	3180 kg.
2'-3'	5550 × 1.15	=	6380
3'-4'	9240 × 1.15	=	10600
5'-6'	14350 × 1.09	=	15650
6'-7	18470 × 1.15	=	21200

This span length changed to 24.5 same as for center span.

Extra reaction at 7 due to cantilever moment
Reaction from side span assumed as
2@4620 = 9240
2800
12040

12040 × 6.625 = 79700
2310 × 6.250 = 14420
4120 × 3.50 = 14420
108540 kgm

108540 ÷ 24.5 = 4425

Center span carrying side hanging span.

			shear		Diagonal stress
7'-8'	4620 × 3 = 13860 + 4425 =		18285 × 1.15	=	21000 kg.
8'-9'	4620 × 15/7 = 9900 + 4425 =		14325 × 1.15	=	16500 "
9'-10'	4620 × 10/7 = 6600 + 4425 =		11025 × 1.15	=	12700 "
10'-11'	4620 × 6/7 = 3960 + 4425 =		8385 × 1.15	=	9520 "

Summary for max stress in lower lateral Bracings.

				Unit stress	SR.	No. Rivet 3/4"
Suspended span 11.25 meter	2-3	1770 kg	= 3900 *	30600	.130 □" net	0.5
	3-4	5320 "	= 11700		.38	1.5
Side span	1'-2'	3180 "	= 7000 *		.23	0.9
	2'-3'	6380 "	= 14050		.46	1.8
	3'-4'	10600 "	= 23300 *		.76	2.9
Cantilever arm	5'-6'	15650	= 34400		1.12	4.3
	6'-7'	21200	= 46600		1.52	5.9
Center span	7'-8'	21000	= 46200		1.51	5.8
	8'-9'	16500	= 36300		1.19	4.6
	9'-10'	12700	= 28000		0.91	3.5
	10'-11'	9520	= 20900		0.68	2.6

Use 6-3/4" Rivets each connection.
Section for diagonal Bracing. 2Ls 5×3×5/16 = 4.80 □" gross.
Radius of gyration = 1.61 ⊥⊥
Unsupported length 7.35 × 3.28 = 24.1' $l/r = \frac{24.1 \times 12}{1.61} = 180$

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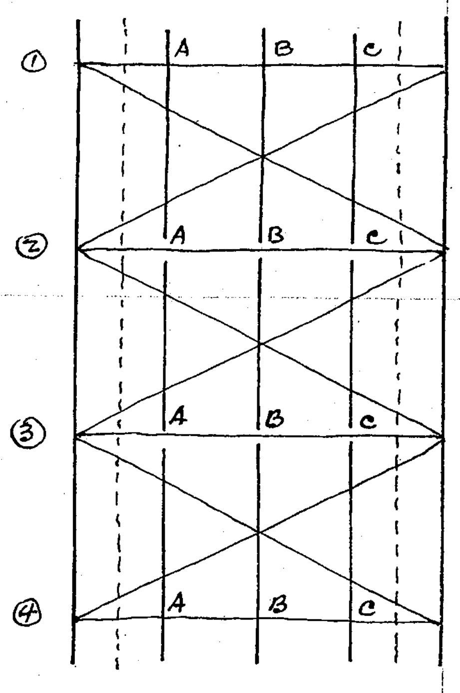
Design of Katsura-Bashi for Kioto-Prefecture

Approximate weight of Lower Laterals for one standard panel.

2L5 5×3×5/16 @ 8.2 × 23 =	377
do	377
Center Connection	25
Connection at ends 4@50 =	200
Rivet heads and detail say	50
	1029# per panel

1029 ÷ 11.5 = 90# per lin ft.
1029 ÷ 2.2 = 467 kg.
467 ÷ 3.5 = 134 kg per lin meter of span.

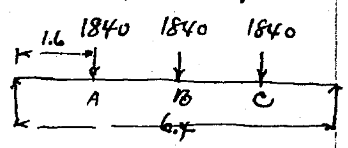
Design of main girder.
Suspended span Span length 3@3.5 = 10.5 add .75 = 11.25 meters.
Dead Load.



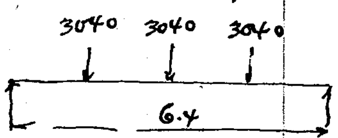
load on stringer
floor slab + pavement 510 × 1.6 = 816
stringer say 52
868 kg. per stringer

Concentration ① ABC. 868 × 2.125 = 1840 kg.
" ② ABC 868 × 3.5 = 3040 "

Load at panel point ①



Load at panel point ②



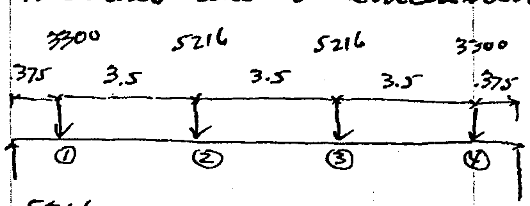
R = 1840 × 1.5 = 2760 R = 3040 × 1.5 = 4560.

panel Concentration ①
flooring. 2760
floor beam 843 ÷ 2 = 422
lateral Bracing 467 ÷ ~~4~~ 2 = ~~234~~ 117
~~3416~~
3299

panel Concentration ②
flooring 4560
floor beam 422
lateral Bracing 234
5216 kg.

call this 3300 kg.

Moment due to concentrated load.



5216
3300
8516

8516 × 3.875 = 33000
3300 × 3.50 = 11550
moment at ② 21450 kgm
Moment at ① 8516 × 0.375 = 3190 "

Uniform dead Load

Slab	510 × 1.6/2 =	408
Cantilever slab.	510 × .55 =	260
Coping.		272
Handrail		255
main girder assumed		450
		1645 kg per lin meter

Moment due to uniform load
at center = 1/8 × 1645 × 11.25² = 26000 kgm

End shear = 1/2 × 1645 × 11.25 = 9250 kg.
moment at ② = 1645/2 × 3.875 × 7.375 = 23500 kgm
" " ① = 1645/2 × 0.375 × 10.875 = 3360 kgm

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto Prefecture.

Summary for moments.

	at ①	at 2	at center	
Due to concentration	3190	21450	21450	
" " uniform load	3360	23500	26000	
	6550	44950	47450	kgm

End reaction Due to conc. 8516
" " Unif. 9250
17766 kg.

Shear at 1 Conc. 5216
9250 - 1645 × 0.375 = 8634
13850 kg.

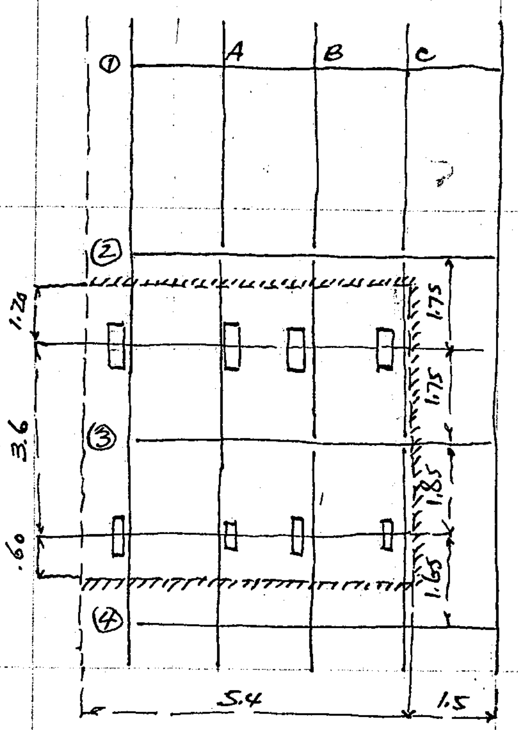
Live Load.

motor truck loading
impact coef = $\frac{20}{60+11.25}$ = 28.1 %.

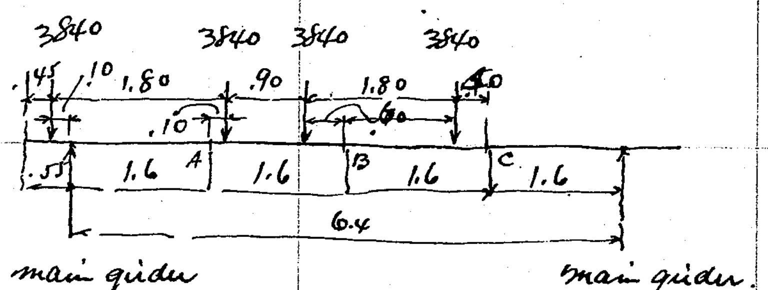
motor truck rear wheel 3000 kg.
impact 28.1 % 840
3840

motor truck front wheel 1000
impact 280
1280 kg.

Uniform live load. 500 kg per sq meter.



moment at center of span.
Rear wheel of motor truck at center of span.



Direct on main girder Rear wheel .3840
due to moment
$3840 \times \frac{.10}{6.4}$ = say 60
3900 kg.

For front wheel.
1300 kg

On stringer A. $3840 \times \frac{2.1}{1.6}$ = 5040 " 1680 "

On stringer B. $3840 \times \frac{1.5}{1.6}$ = 3600 " 1200 "

On stringer C $3840 \times \frac{1.2}{1.6}$ = 2880. " 960 "

Concentration at 2 A. 5040 × ½ = 2520
B 3600 × ½ = 1800
C 2880 × ½ = 1440

Load on 2
2520 × 3/4 = 1890
1800 × ½ = 900
1440 × ¼ = 360
3150 kg.

Concentration at 3 A $1680 \times \frac{1.65}{3.5}$ = 792
2520
3312

B $1200 \times \frac{1.65}{3.5}$ = 565
1800
2365

C $960 \times \frac{1.65}{3.5}$ = 452
1440
1892

Load on 3.
3312 × 3/4 = 2480
2365 × ½ = 1182
1892 × ¼ = 473
4134 kg.

CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture.

Concentration at 4

A $1680 \times \frac{1.85}{3.5} = 886$

B $1200 \times \frac{1.85}{3.5} = 632$

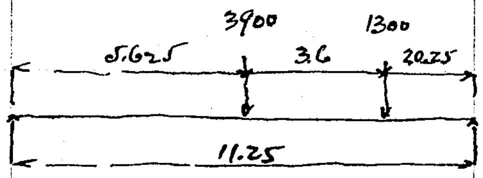
C $960 \times \frac{1.85}{3.5} = 507$

Load on 4

$886 \times 3/4 = 665$
$632 \times 1/2 = 316$
$507 \times 1/4 = 127$
1108 kg.

Moment due to motor truck

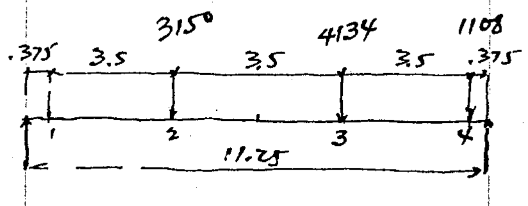
Direct load on main girder



Reaction $1300 \times \frac{2.025}{11.25} = 234$
$3900 \div 2 = 1950$
2184

moment $= 2184 \times 5.625 = 12300$ kgm

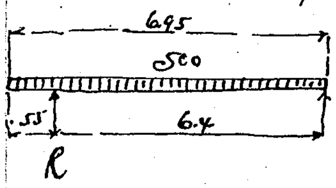
Panel load on main girder



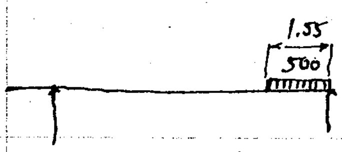
Reaction. $1108 \times 0.375 = 416$
$4134 \times 3.875 = 16000$
$3150 \times 7.375 = 23200$
$39616 \div 11.25 = 3520.$ kg

moment $= 3520 \times 5.625 = 19800$
$3150 \times 1.75 = -5510$
14290 kgm

Uniform load.
Load on main girder.

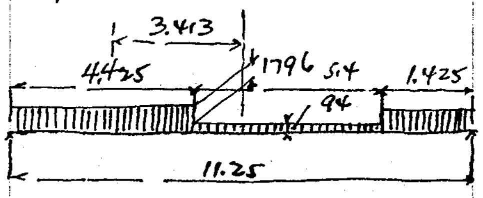


$R = \frac{500 \times 6.95^2}{2 \times 6.4} = 1890$ kg per lin meter.



$R = \frac{500 \times 1.55^2}{2 \times 6.4} = -94$ kg per lin meter
1796 " " " "

Load assumed direct on main girder.



moment at center. unif. $\frac{1}{8} \times 94 \times 11.25^2 = 1490$

Partial load. $6542 \times 5.625 = 36800$
$7950 \times 3.413 = -27100$
9700
11190

$R = \frac{1796 \times 1.425^2}{2 \times 11.25} = 162$

$\frac{1796 \times 4.425 \times 9.038}{11.25} = 6380$ (7950)
6542 kg.

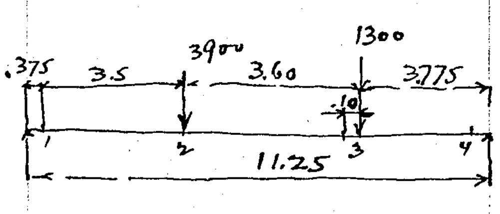
Summary for Live Load Moment at center of span

Due to motor truck	direct	12300
" "	Concentration	14290
Due to unif. load.		11190
		37780 kg meter

Live Load moment at panel point No 2.
Motor truck rear wheel at panel point No 2.
Direct load 3900 kg for rear 1300 kg for front wheel.
Due ~~to Concentration~~.

$3840 \times$ 2.0
3.8
4.7
$10.5 \div 6.4 = 6300$ for rear wheel.
2100 for front wheel.

Moment due to direct load.



Reaction $1300 \times \frac{3.775}{11.25} = 436$
$3900 \times \frac{7.375}{11.25} = 2560$
2996 kg.

Moment $= 2996 \times 3.875 = 11600$ kgm

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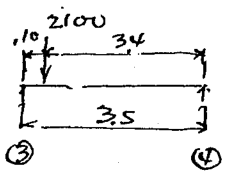
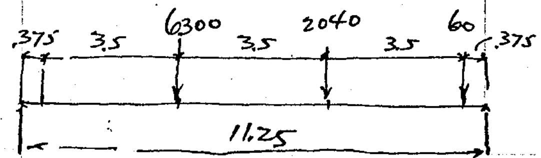
Moment due to concentrated load between main girders.
Panel concentration -



③ 2100 × 3.4/3.5 = 2040

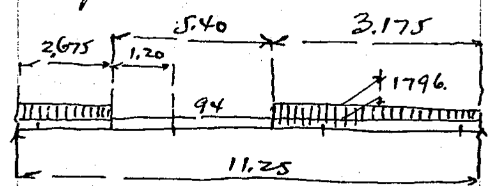
④ 2100 − 2040 = 60



Reaction 60 × .375 ÷ 11.25 = 2
2040 × 3.875 ÷ 11.25 = 705
6300 × 7.375 ÷ 11.25 = 4130
4837 kg.

moment = 4837 × 3.875 = 18750

Uniform load.



1796 × 2.675 = 4810.
1796 × 3.175 = 5700

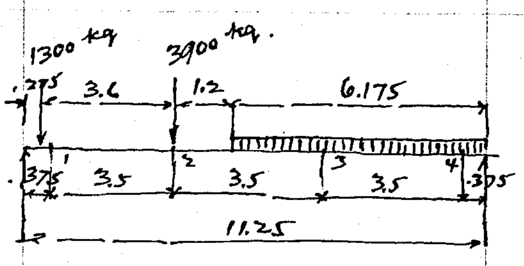
Reaction = ~~4810~~ 5700 × 1.587 ÷ 11.25 = ~~678~~ 802
~~5700~~ 4810 × 9.912 ÷ 11.25 = ~~5020~~ 4230
~~5698~~ kg. 5032

moment ~~5698~~ 5032 × 3.875 = 19500
4810 × 2.537 = 12200
7300 kgm

94/2 × 3.875 × 7.375 = 1340
8640 "

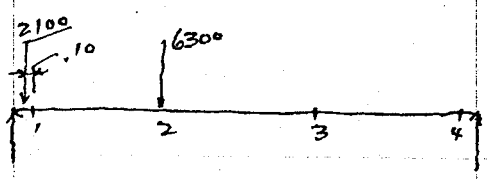
Summary for live load moment at ②
Due to motor truck direct 11600
" " panel load 18750
Due to uniform load 8640
38990

Live Load Moment at Panel Point No 2.
Rear wheel at panel point No 2 and front wheel at left of span
Direct on main girder
moment due to direct motor truck loading



Reaction 1300 × .275/11.25 = 32
3900 × 3.875/11.25 = 1343
1375

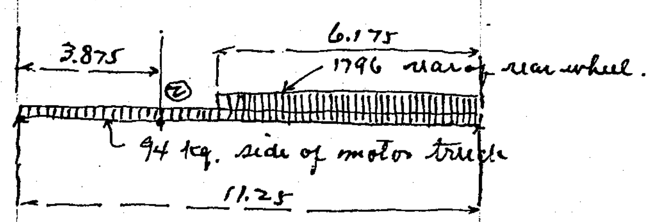
moment 1375 × 7.375 = 10120 kgm



Motor truck loading between main girders.

moment = 10120 × 6300/3900 = 16400 kgm.

Uniform load



Reaction 1796 × 6.175 = 11100 kg.
11100 × 3.087/11.25 = 3040.

moment = 3040 × 3.875 = 11800
" unif. 94/2 × 3.875 × 7.375 = 1340
13140 kgm

Summary for moment at ②

Direct motor truck loading on girder 10120
due to panel concentration 16400
Uniform live load. 13140
39660 kgm
Use this moment for max.

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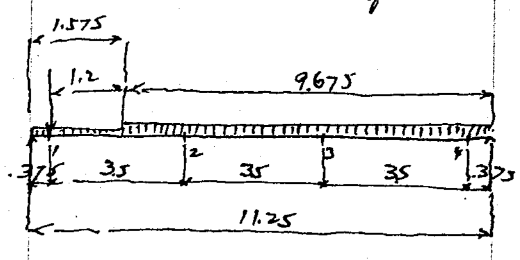
Live Load moment at panel point no 1.

Motor truck loading



Rear wheel	direct on girder		3900
" "	from floor beam		6300
			10200 kg.
Reaction =	10200 × $\frac{10.875}{11.25}$	=	9850 kg
Moment =	9850 × .375	=	3690 kgm

Uniform live load

1796 × 9.675 = 17400 Reaction = 17400 × $\frac{4.838}{11.25}$ = 7480

moment = 7480 × .375 =		2800
$\frac{94}{2}$ × .375 × 10.875	=	190
		2990 kgm

Summary for Live Load moment at panel point no 1.

Moment due to motor truck	3690
" " " Unif. load	2990
	6680 kgm.

Summary for Dead and Live Load moments.

	1	2	Center	
Dead Load moment	6550	44950	47450	
Live Load moment	6680	39660	37780	← motor truck rear wheel at center
	13230	84610	85230 kgm	
	95800 #	612000 #	617000 #	

Live Load shear

At 3nd. Rear wheel at panel point no 1. shear at ①

motor truck loading.		9850	− 6300	=	3550
Unif. load.		7480		=	7480
" "	$\frac{94}{2}$ × 11.25 =	530	− 35	=	495
		17860			11525

Summary for Dead and Live Load shears.

	End shear	Shear at 1
Dead Load	17766	13850
Live Load	17860	11525
	35626 kg	25375 kg.
	78300 lbs.	55700 lbs.

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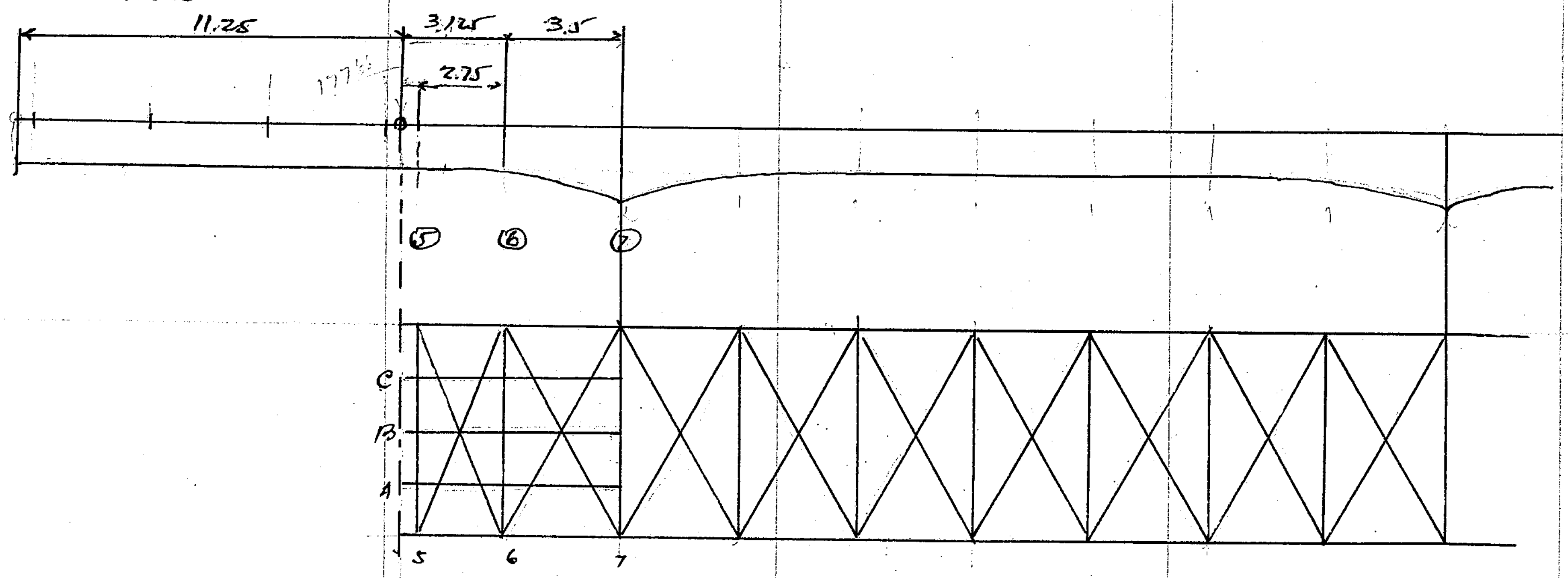
Design of Katsura-Bashi for Kioto-Prefecture

Cantilever Arm for Center Span. carrying 11.25 meter suspended span.
Dead Load



See page 9. load on stringer
Concentration 5 ABC 868 × 1.75 = 1520
6 868 × 3.125 = 2710.

Load at 5	1520 × 1.5 =	2280	Load at 6	2710 × 1.5 =	4065
floor beam		422			422
Lateral Bracing.		117			234
		2819			4721
	call this	2820			4720

Concentration at hinge Reaction from 11.25 suspended span = 17766 kg.

Uniform load on main girder assumed 1645 kg per lin meter of span.
Dead Load moment at 6.

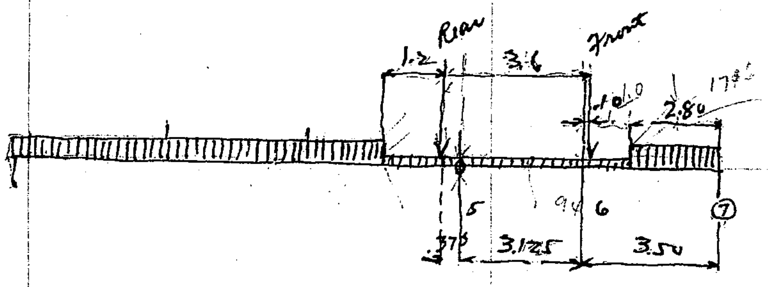
				Shear
from floor	2820 × 2.75	=	7750	2820
from hinge	17766 × 3.125	=	55500	17766
Unif.	$1645 \times \frac{3.125^2}{2}$	=	8050	5150
			71300 kgm	25736 kg.

Dead Load moment at 7

				Shear
from floor	2820 × 6.25	=	17600	2820
" "	4065 × 3.50	=	14250	4065
hinge	17766 × 6.625	=	117500	17766
Unif.	$1645 \times \frac{6.625^2}{2}$	=	36100	10900
			185450 kgm	35551 kg

Live Load motor truck loading impact same as for suspended span assumed.
Uniform load per lin meter of main girder 1890 kg.

Live Load at hinge 17860 load from motor truck assumed on end floor beam



Live Load moment at 6.
Reaction at Hinge 17860 × 3.125 = 55900
Unif. $94 \times \frac{3.125^2}{2}$ = 450
56350 kgm

Shear 17860
Unif. 290
18150 kg.

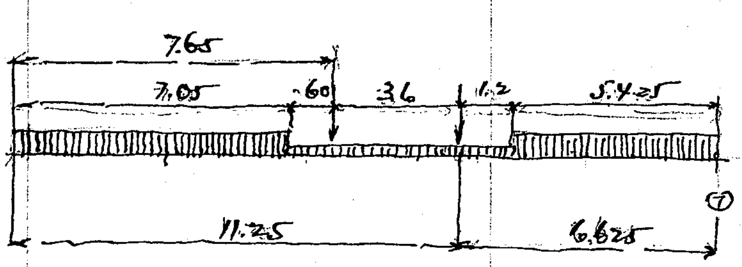
Design of Katsura-Bashi for Kioto-Prefecture.

Live Load moment at 7

Front wheel 2100 from floor beam panel load 2040 at 6 60 kg at 7
1300 direct on girder.

Moment due to motor truck

direct	1300 ×	3.4	=	4420
from floor beam	2040 ×	3.5	=	7150
Unif.	$1796 \times \frac{2.8^2}{2}$		=	7050
"	$96 \times \frac{6.625^2}{2}$		=	2060
				20680 kgm
Load at hinge	17860 × 6.625		=	118300
				138980 kgm



Reaction at hinge
motor truck 1300
2100
$3400 \times \frac{7.65}{11.25}$ = 2320
3900
6300
12520 kg.
4500
17020 kg.

Uniform load $1796 \times \frac{7.05^2}{2 \times 11.25}$ = 3970
$\frac{94}{2} \times 11.25$ = 530
4500

Moment at 7 of cantilever span

Uniform load	$1796 \times \frac{5.425^2}{2}$	=	26450
	$94 \times \frac{6.625^2}{2}$	=	2060
			28510
at hinge	17020 × 6.625	=	112800
			141310 kgm.

Shear. 1796 × 5.425 = 9750
94 × 6.625 = 622 } 10372
17020
27392 kg. call this 27400 kg.

Summary for moments and shears.

	6	7	shear 5-6	6-7
Dead Load	71300	185450	25736	35551
Live load	56350	141310	18150	27400
	127650	326760 kgm	43886 kg	62951 kg
	922.000 #	2360.000 #	96500 #	138000 #

~~Cantilever Arm for Center span carrying 17.875 meter Suspended span.~~
~~Dead Load.~~
~~Dead Load reaction at hinge = 28154 kg.~~
~~Dead Load moment at 6'~~

				shear
from floor	2820 × 2.75 =	7750		2820
at hinge	28154 × 3.125 =	88000		28154
Unif.	$1645 \times \frac{3.125^2}{2}$ =	8050		5150
		103800 kgm		36124 kg

VOID

~~Dead Load moment at 7'~~

from floor	2820 × 6.25 =	17600	2820
	4065 × 3.50 =	14250	4065
	28154 × 6.625 =	186500	28154
	$1645 \times \frac{6.625^2}{2}$ =	36100	10900
		254450 kgm	45939 kg

CALCULATIONS FOR

Design of Katsura Bashi for Kioto Prefecture.

Live Load

Live load at Hinge. 25040 kg.

Moment at 6'

Reaction at Hinge 25040 × 3.125 = 78300

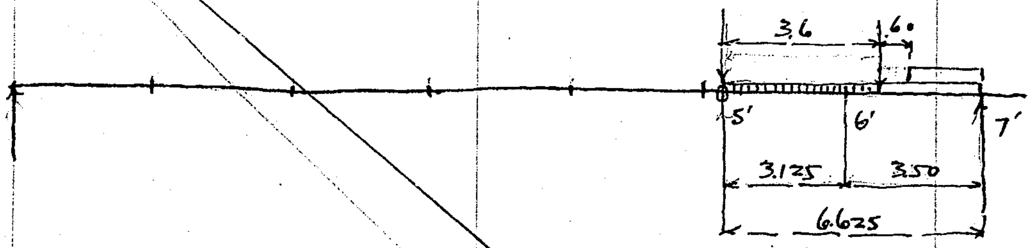
Unif. load $94 \times \frac{3.125^2}{2}$ = 450

78750 kgm

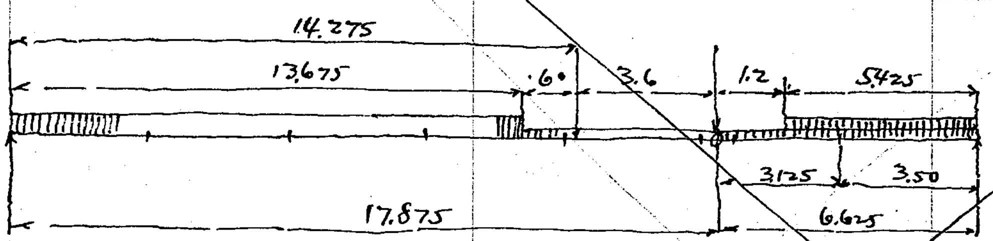
Shear 25040

Unif. 290

25330 kg.



Moment at 7'



Reaction at Hinge

Front wheel 1300

2100

$3400 \times \frac{14.275}{17.875}$ = 2720

3900

6300

12920

Uniform load. 10240

23160 kg.

Uniform load $1796 \times \frac{13.675^2}{2 \times 17.875}$ = 9400

$\frac{94}{2} \times 17.875$ = 840

10240

Moment at 7' load on Cantilever moment 28510

load at Hinge 23160 × 6.625 = 153000

181510 kgm

Shear Unif. 10372

at Hinge 23160

33532 kg.

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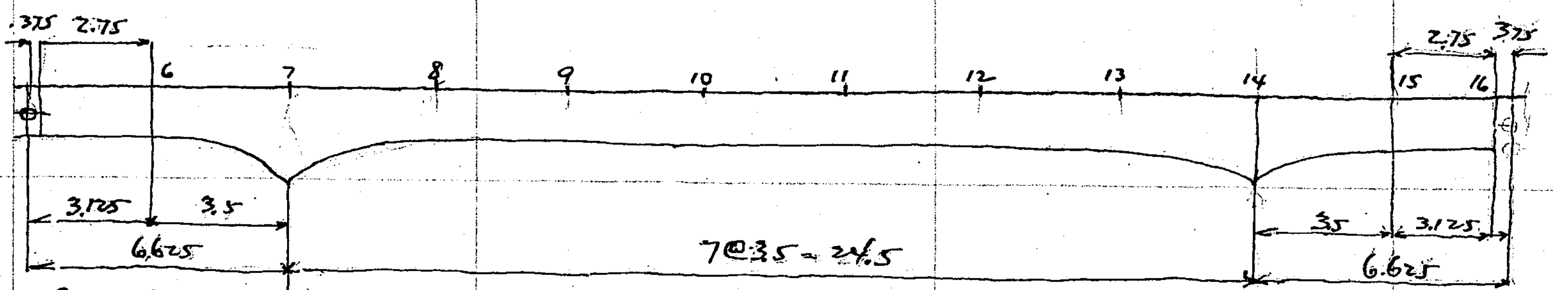
Summary for moments and shears

	moment at 6'	moment at 7'	shear at 5-6	shear 6-7
Dead Load	103800	254450	36124	45939
Live Load	78750	181510	25330	33532
	182550 kgm	435960 kgm	61454 kg	79471 kg
	1320.000 #	3150.000 #	135000 #	175.000 #

Center Span with 11.25 suspended span on both ends.

Dead Load.



Cantilever Arm Center Span Cantilever Arm

Concentration from floor beam 5216 kg.

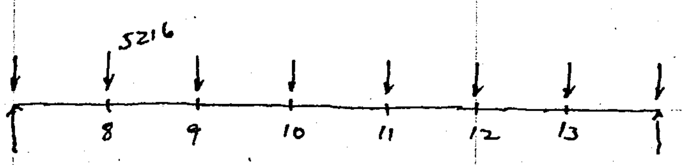
Uniform load on main girder main girder assumed 550

See page 9 Uniform load. 1745 kg per lin meter.

moment at 8 15648 × 3.5 = 54800 kgm

Moment at 9 5216 × 5 × 3.5 = 91200 "

" " 10 5216 × 6 × 3.5 = 109600 "



5216
3
15648

15648

moment at 8 $\frac{1745}{2} \times 3.5^2 \times 6$ = ~~18300~~ " 64100 "

" " 9 " $\times 3.5^2 \times 2 \times 5$ = ~~30500~~ " 107000 "

" " 10 " $\times 3.5^2 \times 3 \times 4$ = ~~36650~~ 128200

" at center $\frac{1}{8} \times 1745 \times 24.5^2$ = 131000 "

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture

Dead Load shear

			Uniform load			Summary
7-8	5216 × 3 =	15648	7-8	1745 × 12.25 =	21400	37048
8-9	" × 2 =	10432	8-9	21400 − 1745 × 3.5 = 6100	15300	25732
9-10	" × 1 =	5216	9-10	15300 − 6100 =	9200	14416

Summary for Dead Load Positive moments

7	8	9	10	Center
0	54800	91200	109600	109600
0	64100	107000	128200	131000
0	+ 118900	+ 198200	+ 237800	+ 240600
− 185450	− 185450	− 185450	− 185450	− 185450
− 185450	− 66550	+ 12750	+ 52350	+ 55150

Live Load motor truck loading impact = $\frac{20}{60+24.5}$ = 23.7 %

Rear wheel 3000
impact 23.7 % 710
3710

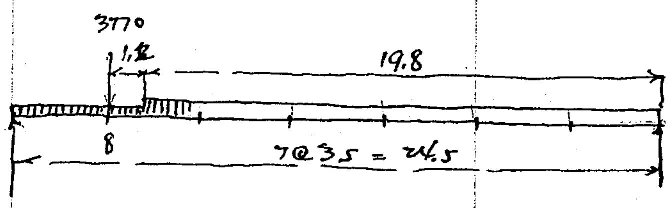
Direct on main girder $3710 \times \frac{6.5}{6.4}$ = 3770 kg.

Front wheel 3770 ÷ 3 = 1256 kg.

Motor truck rear wheel $3710 \times \frac{10.5}{6.4}$ = 6100 kg
Front wheel 6100 ÷ 3 = 2033 kg

Uniform live load 500 kg per sq. meter.

Live load moment at 8 motor truck loading 3770
6100
9870 kg



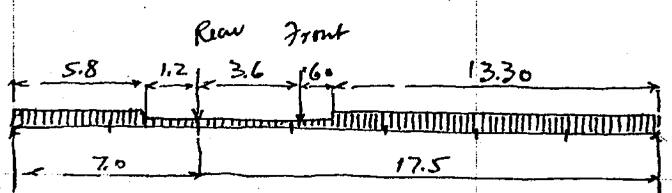
Reaction = 9870 × 6/7 = 8470 kg.
moment = 8470 × 3.5 = 29600 kgm

Uniform load 1796 × 19.8 = 35600 R = $35600 \times \frac{9.9}{24.5}$ = 14350 kg.
moment = 14350 × 3.5 = 50250
$\frac{94}{2} \times 3.5^2 \times 6$ = 3460
53710 kgm

Summary for Live Load Moment at 8
motor truck loading 29600
Uniform load 53710
83310 kgm

Live Load moment at 9

Rear wheel − 9870
Front wheel − 3289



Reaction = $9870 \times \frac{17.5}{24.5}$ = 7050
$3289 \times \frac{13.90}{24.5}$ = 1870
8920

moment = 8920 × 7.0 = 62400 kgm

Uniform load 1796 × 13.30 = 23900 Reaction 23900 × 6.65 ÷ 24.5 = 6480
1796 × 5.80 = 10400 10400 × 21.6 ÷ 24.5 = 9180
15660 kg.

moment 15660 × 7.0 = 109500
10400 × 4.1 = − 42600
66900
$\frac{94}{2} \times 7.0 \times 17.5$ = 5750
72650

Summary for Live Load moment at 9
motor truck loading 62400
Uniform load 72650
145050 kgm

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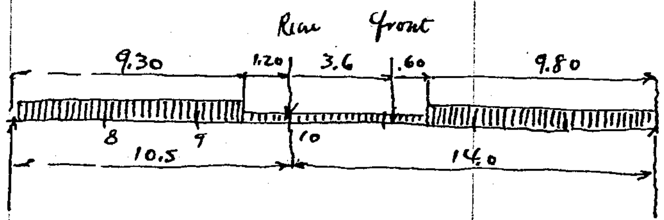
Live Load Moment at 10



Rear wheel Concentration 9870
Front " " 3289

Reaction 9870 · 4/7 = 5640
3289 × 10.40/24.5 = 1400
7040

Moment = 7040 · 10.5 = 74000 kgm

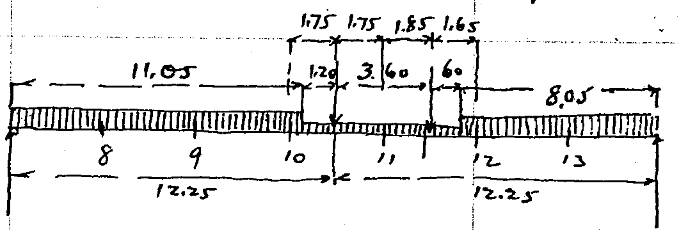
Uniform load 1796 · 9.8 = 17600 Reaction 17600 · 4.9 ÷ 24.5 = 3520
1796 · 9.3 = 16700 16700 × 19.85 ÷ 24.5 = 13550
17070

Moment = 17070 · 10.5 = 179200
Less 16700 · 5.85 = 97700
81500

94/2 · 10.5 × 14.0 = 6910
88410

Summary for moment at 10
motor truck loading 74000
Uniform load 88410
162410 kgm

Live Load Moment at center of span.



motor truck direct on girder.
1256 · 8.65 ÷ 24.5 = 443
3770 ÷ 2 = 1885
2328

Moment = 2328 × 12.25 = 28500 kgm

Motor truck loading between girder.
panel Concentration. Rear wheel at 10 ~~3770~~ 6100 ÷ 2 = 3050

at 11 6100 ÷ 2 = 3050
2033 · 1.65/3.5 = 958
4008

at 12 2033 - 958 = 1075

Reaction at end 1075 · 2/7 = 308
4008 · 3/7 = 1720
3050 · 4/7 = 1740
3768

moment 3768 · 12.25 = 46100
Less 3050 × 1.75 = 5340
40760 kgm

Uniform load 1796 · 8.05 = 14450 Reaction 14450 × 4.025/24.5 = 2375
1796 · 11.05 = 19850 19850 · 18.975/24.5 = 15400
17775

Moment = 17775 · 12.25 = 217500
19850 · 6.725 = 133500
84000 kgm

Summary for moment at center
motor truck loading 28500
" " " 40760
Uniform load 84000
153260 kgm

Summary for Positive LL moment + Dead Load m

	7	8	9	10	center
Dead Load	-185450	-66550	+12750	+52350	+55150
Live Load		+83310	+145050	+162410	+153260
	-185450	+16760	+157800	+214760	+208410
	-1,340,000 in#	+1,210,000 in#	+1,140,000 in#	+1,555,000 in#	+1,505,000 in#

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CALCULATIONS FOR

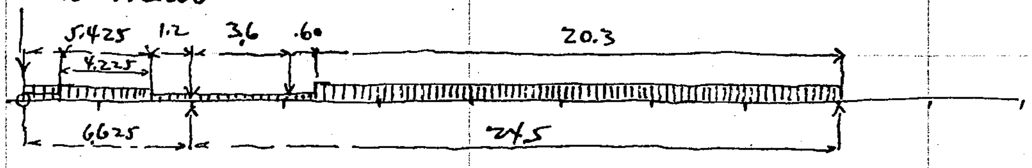
Design of Katsura-Bashi for Kioto-Prefecture.

Summary for Dead Load and Negative Live Load moments

	7	8	9	10	center	
Dead Load	− 185450	− 66550	+ 12750	+ 52350	+ 55150	
Live Load	− 141310	− 141310	− 141310	− 141310	− 141310	
	− 326760	− 207860	− 128560	− 80760	− 86860	kgm
	− 2360000	− 1500000	− 930000	− 643000	− 622000	
	2360000	1500000	930000	843000	622000	ft lbs

Live Load Shear. approximate only

End shear



motor truck rear 9870
front $3289 \times \frac{20.9}{24.5} = 2800$
12670 kg.

Unif. load $1796 \times 20.3 = 36400$
Reaction $36400 \times \frac{10.15}{24.5} = 15100$
$\frac{94}{2} \times 24.5 = 1060$
16160

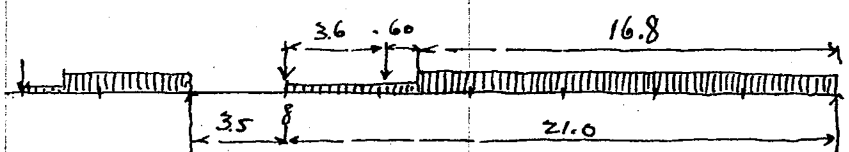
Extra reaction due to Cantilever moment

moment = $94 \times \frac{6.625^2}{2} = 2060$
$1796 \times \cancel{5.425}\ 4.225 \times \frac{\cancel{3.912}\ 2.312}{} = \cancel{25600}\ 25100$
$17020 \times 6.625 = 112800$
$139960 \div 24.5 = 5710$ kg

Summary for End shear

motor truck	12670
Unif.	16160
extra reaction	5710
	34540 kg

Shear at 8



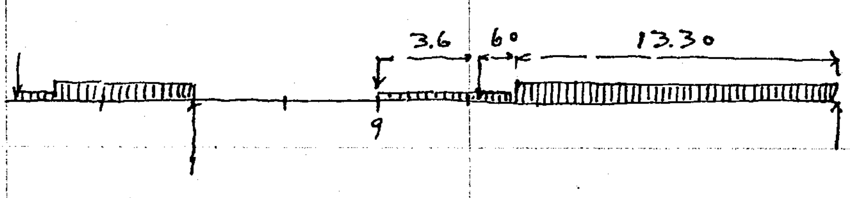
motor truck
Rear $9870 \times 6/7 = 8470$
$3289 \times \frac{17.40}{24.5} = 2340$
10810

Uniform load $1796 \times 16.8 = 30200$ R $30200 \times \frac{8.4}{24.5} = 10350$
$94 \times 21.0 = 1980$ $1980 \times \frac{10.5}{24.5} = 840$
11190
22000 kg.

Extra reaction $141310 \div 24.5 = 5770$
27770 kg.

Shear at 9

motor truck loading R 8920

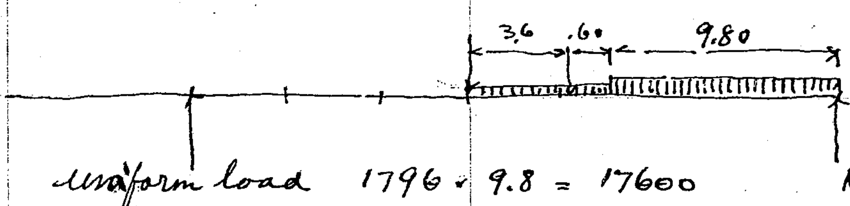


Uniform load $1796 \times 13.30 = 23900$ R = $23900 \times 6.65 \div 24.5 = 6480$
$1150 - 660 = \cancel{820}\ 490$
6970
~~7300~~
~~16220~~ 15890

Extra reaction 5770 5770
~~21990~~ 21660 kg.

Shear at 10

motor truck loading
Reaction 7040



uniform load $1796 \times 9.8 = 17600$ R = $17600 \times 4.9 \div 24.5 = 3520$
$1150 - 990$ 160
3680
10720

Extra reaction 5770
16490 kg.

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto-Prefecture.

Summary for shears

	7-8	8-9	9-10	10-11
Dead Load	37048	25732	14416	
Live Load	34540	27770	21660	16490
	71588	53502	36076 kg	
	157000 #	118000 #	77200 #	

End Anchor span span length 24.5 meters
Span length same as for center span; Cantilever arm on one side only.
Moments at various panel point reduced from calculation of center span.

Dead Load moment.

	7	8	9	10	11	12	13	14
Simple span	+118900	+198200 (+118900)	+198200	+237800	+237800	+198200	+118900	0
Cantilever	0	−26500	−53000	−79500	−106000	−132500	−159000	−185450
	0	+92400	+145200	+158300	+131800	+65700	−40100	−185450

Dead Load and positive live Load moments combined

	7	8	9	10	11	12	13	14	
Dead Load	0	+92400	+145200	+158300	+131800	+65700	−40100	−185450	
Live Load	0	+83310	+145050	+162410	+162410	+145050	+83310	0	
	0	+175710	+290250	+320710	+294210	+210750	+43210	−185450	kgm
		1.270.000	2.100.000	2320.000	2.140.000	1.525.000	312.500	1.340.000	ft lbs

Dead Load and negative live load moments combined

	7	8	9	10	11	12	13	14	
Dead Load	0	+92400	+145200	+158300	+131800	+65700	−40100	−185450	
LL Cant.		−20200	−40400	−60600	−80800	−101000	−121200	−141310	
		+72200	+104800	+97700	+51000	−35300	−161300	−326760	kgm.
		522000	758000	706000	369000	255000	1.170.000	2.365000	‘#

For moment diagram see next page.

Load on shoe (On Piers).

Dead Load
End shear center span 37048 kg
Cantilever arm and ½ suspended span 39650
weight of shoe say 3600
say 80300 kg.

Live Load
Center span and Cantilever arm p19 34540
from suspended span 18300
52840
133.140 kg or say 133200 kg
Design shoes for load of 293040 #

Bearing area of shoe 24×25 = 600□" $\frac{293040}{600}$ = 488 #/□"

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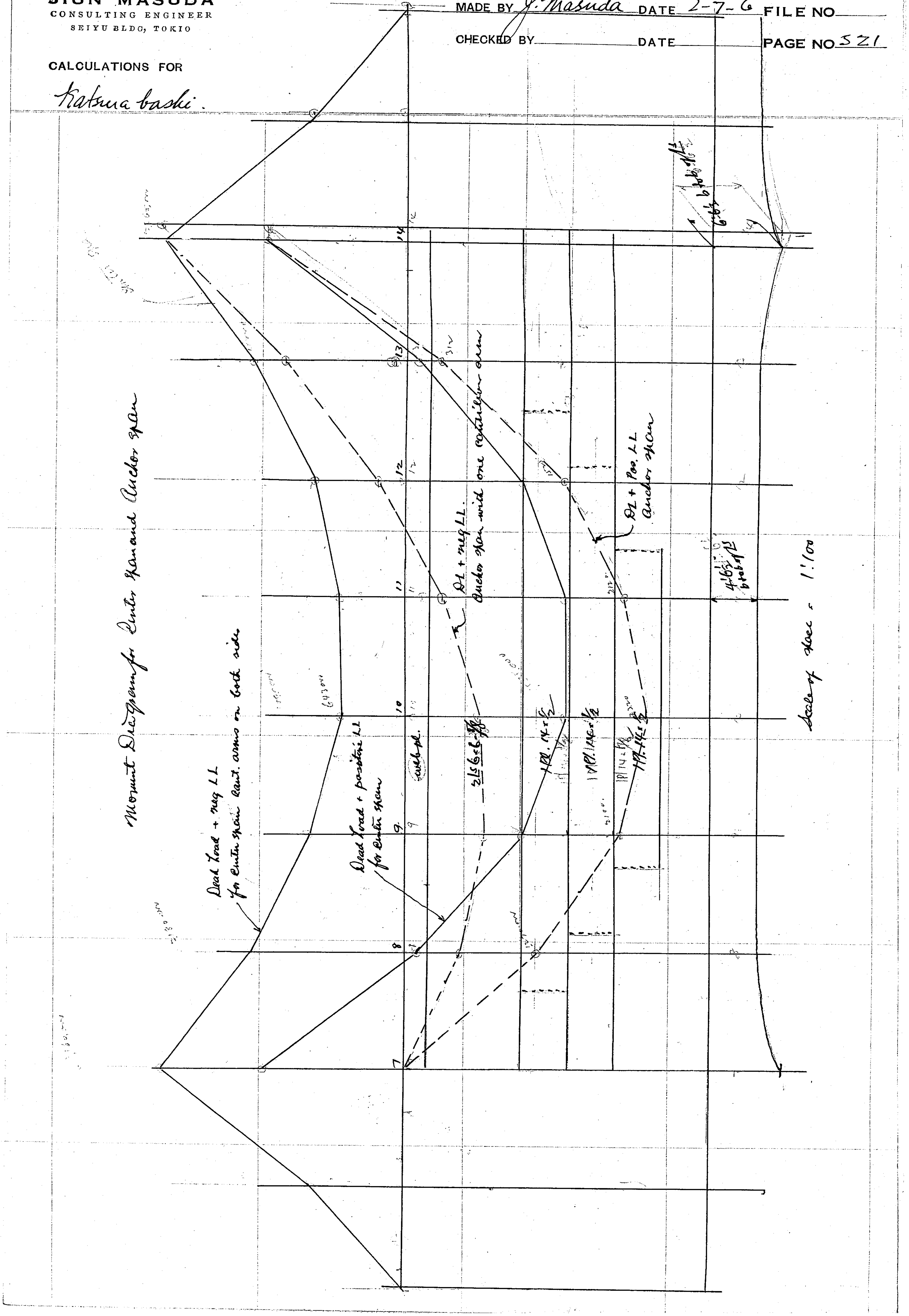
CALCULATIONS FOR
Katsura bashi.



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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto Prefecture

Sections of main girders.

Suspended span. m = 617000 "# s = 78300 #
~~web assumed~~ 54 × 3/8 = 20.250 □" 1/8 web = 2.53 □" 4'-6½" b to b of Ls.
Effective depth = 4.26 flange stress = 617000 ÷ 4.26 = 145000 #
SR = 145.000 ÷ 17000 = 8.52
1/8 web 2.53
5.99 □" net
use 2Ls 6×6×½ = 11.50 □" or 10.50 □" net
used ½ metal instead of 3/8 on acct of main girder.

Cantilever arm panel point no 6. m = 922.000 "#
~~web assumed~~ 56 × 7/16 about = 24.5 □" 1/8 web = 3.06 □" b to b of Ls = 4'-8½"
Effective depth = 4.56' flange stress = 922.000 ÷ 4.56 = 202.000 #
SR = 202.000 ÷ 17000 = 11.90
− 3.06
8.84 □" net.
2Ls 6×6×5/8 = 14.22 − 12.97 □" net
Unit stress in gross section
Gross section 14.22
3.06
17.28
202.000 ÷ 17.28 = 11700 #/□"
l = 11'-6" = 138"
b = 12 7/16
l/b = 11.1
Allow Unit stress 17000 (1 − 0.012 l/b) = 14700 #/□" ok

~~Cantilever arm~~ at support (Pier). m = 2360.000 "#
~~web assumed~~ 78 × 7/16 = 33.200 □" 1/8 web = 4.15 □" 6'-6½" b to b of Ls
Effective depth assumed = 6.40 flange stress = 2360.000 ÷ 6.40 = 369.000 #
SR. = 369.000 ÷ 17000 = 21.70
− 4.15
17.55 □" net
2Ls 6×6×5/8 = 14.22 − 11.72
1Pl. 14×½ = 7.00 − 6.00
21.22 □" 17.72 □" net
21.22
4.15
25.37
Allowable unit stress for Compressive flange = 17000 (1 − 0.012 l/b) = 15000 #/□"
b = 14 l = 138"
Unit stress in Gross = 369.000 ÷ 25.37 = 14550 #/□" ok

Center Span moment at 10 pos. m = 1.555.000
½ neg. m = 321.500
1.876.500 "#
Depth of web 4'-6" web. 54 × 7/16 = 23.65 1/8 web = 2.95 □" depth = 4'-6½" b to b of Ls
Effective depth say 4.4 flange stress = 426000 #
SR = 426000 ÷ 17.000 = 25.00
2.95
22.05 □" net
2Ls 6×6×5/8 = 14.22 11.72
1Pl. 14×½ = 7.00 6.00
1Pl 14×½ = 7.00 6.00
28.22 23.72 □" net

for other section of girder see moment diagram

Anchor span moment at 10 pos. m = 2.320.000
~~½ neg m = 353.000~~
~~2.653.000 "#~~
Depth 4'-6½" b to b of Ls. web = 54 × 7/16 = 23.65 □" 1/8 web = 2.95 □"
Effective depth = 4.50 stress = ~~2653.000~~ 2320.000 ÷ 4.50 = ~~588.000~~ 515.000 #
SR = ~~588~~ 515 000 ÷ 17000 = ~~34.60~~ 30.30
~~2.95~~ 2.95
~~31.65~~ □" net 27.35

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto Prefecture.

Section used	2 L̲s 6 × 6 × 5/8	=	14.22	11.72
	1 Pl 14 × 1/2	=	7.00	6.00
	1 Pl. 14 × 1/2	=	7.00	6.00
	1 Pl. 14 × 1/2	=	7.00	6.00
			35.22	~~29.72~~ net

Approximate deflection of main girders.
Moment of inertia of girder section.

Suspended span

1 web. 54 × 3/8		4920
4 L̲s 6 × 6 × 1/2	= 23.00 × 25.57² + 80 =	15080
		20000 (")⁴

Center span

1 web 54 × 7/16		5740
4 L̲s 6 × 6 × 5/8	= 28.44 × 25.52² + 96.8 =	18596
4 Pls 14 × 1/2	= 28.00 × 27.75² =	21600
		45936 (")⁴

Anchor span

2 Pls. 14 × 1/2	= 14.00 × 28.25² =	11180
		57116 (")⁴

Cantilever Arm 3'-9" out of ℄ Pier depth assumed 5'-9½" b to b of L̲s

1 web 69 × 7/16		= 12000
4 L̲s 6 × 6 × 5/8	= 28.44 × 33.02² + 96.8 =	31150
2 Pls. 14 × 1/2	= 14.00 × 35.00² =	17200
		60350

Cantilever arm 11'-6" out of ℄ Pier

1 web 56 × 7/16		= 6402
4 L̲s 6 × 6 × 5/8	= 28.44 × 26.52² + 96.8 =	20098
		26500

Deflection of Suspended span. Dead load + full Live Load.

Simple Beam $\Delta = \frac{5}{384}\frac{wl^4}{EI}$ where $m = \frac{wl^2}{8}$

$\Delta = \frac{5}{48} \times m \times \frac{l^2}{EI}$ max moment $= 617000^{\prime\#}$

$\Delta = \frac{5}{48} \times \frac{617.000 \times 12 \times 37^2 \times 12^2}{30.000.000 \times 20.000} = 0.254''$

Deflection of Cantilever Arm

Cantilever beam

$\Delta = \frac{wl^4}{8EI}$ where $m = \frac{wl^2}{2}$ Average m of I assumed $= 40.000^{('')4}$

$\Delta = \frac{1}{4} \times m \times \frac{l^2}{EI}$ $m = 2.360.000^{\prime\#}$ $l = 21.75'$

$\Delta = \frac{1}{4} \times \frac{2.360.000 \times 12 \times 21.75^2 \times 12^2}{30.000.000 \times 40.000} = 0.40''$ for uniform load

for concentration at end $0.40 \times \frac{4}{3} = 0.53''$

Deflection of center span — span length between zero moments 58' for Pos m.
Cantilever arm $\frac{13.5'}{11.0}$ at both ends for neg m.

Deflection due to pos. m $\Delta = \frac{1.555.000 \times 12 \times 58^2 \times 12^2}{30.000.000 \times 46.000} \times 5/48 = .68''$

Deflection due to neg m $\Delta = \frac{1.340.000 \times 12 \times \overset{11.0}{\cancel{13.5}}{}^2 \times 12^2}{4 \times 30.000.000 \times 40.000 \text{ assumed}} = \overset{0.59}{\cancel{0.088}}''$ for unif.

$\overset{.59}{0.\cancel{088}} \times \frac{4}{3} = \cancel{0.12}''\ 0.08$ for concentration at end

Total say $\overset{0.68}{0.\cancel{57}} + \overset{.08}{\cancel{.12}} = \cancel{0.69}''\ 0.76''$

$\Delta/l = \frac{0.\cancel{69}{}^{76}}{80 \times 12} = \frac{1}{\cancel{1390}}\ \frac{1}{1260}$

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CALCULATIONS FOR

Design of Katsura-Bashi for Kioto prefecture

Deflection of anchor span   span length between zero moments $21.7^m \times 3.28 = 71'$

Cantilever arm 9' for negative moment

$$\Delta = \frac{2.320.000 \times 12 \times 71^2 \times 12^2}{30.000.000 \times 57100} \times 5/48 = 1.23''$$

Δ = for neg moment say 0.07

$1.30''$

$$\Delta/l = \frac{1.30}{80 \times 12} = 1/740$$

The above deflections are approximate only, camber to be given to main girder figuring exact deflection due to dead load + ½ live load to give smooth camber

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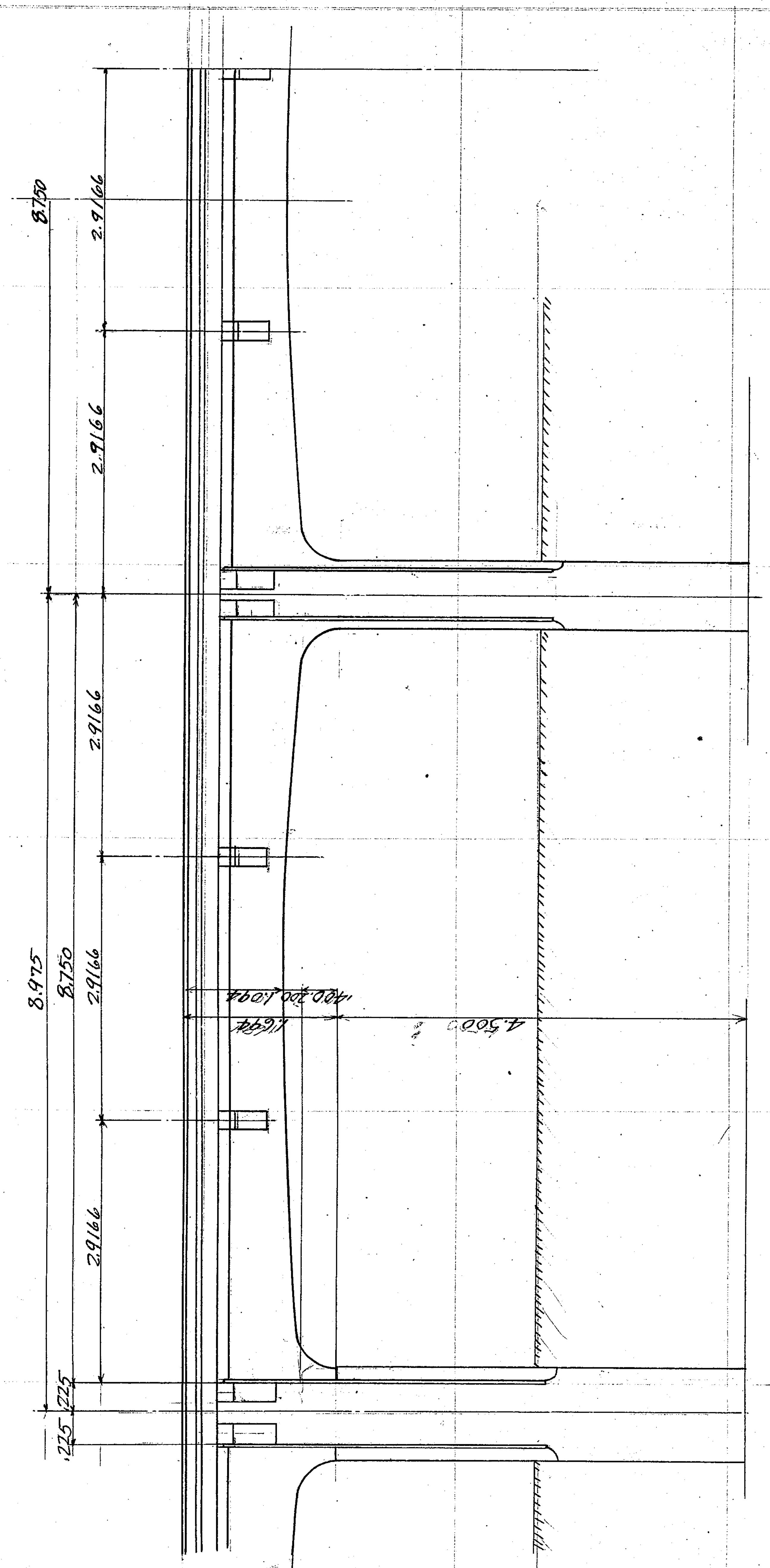
CALCULATIONS FOR



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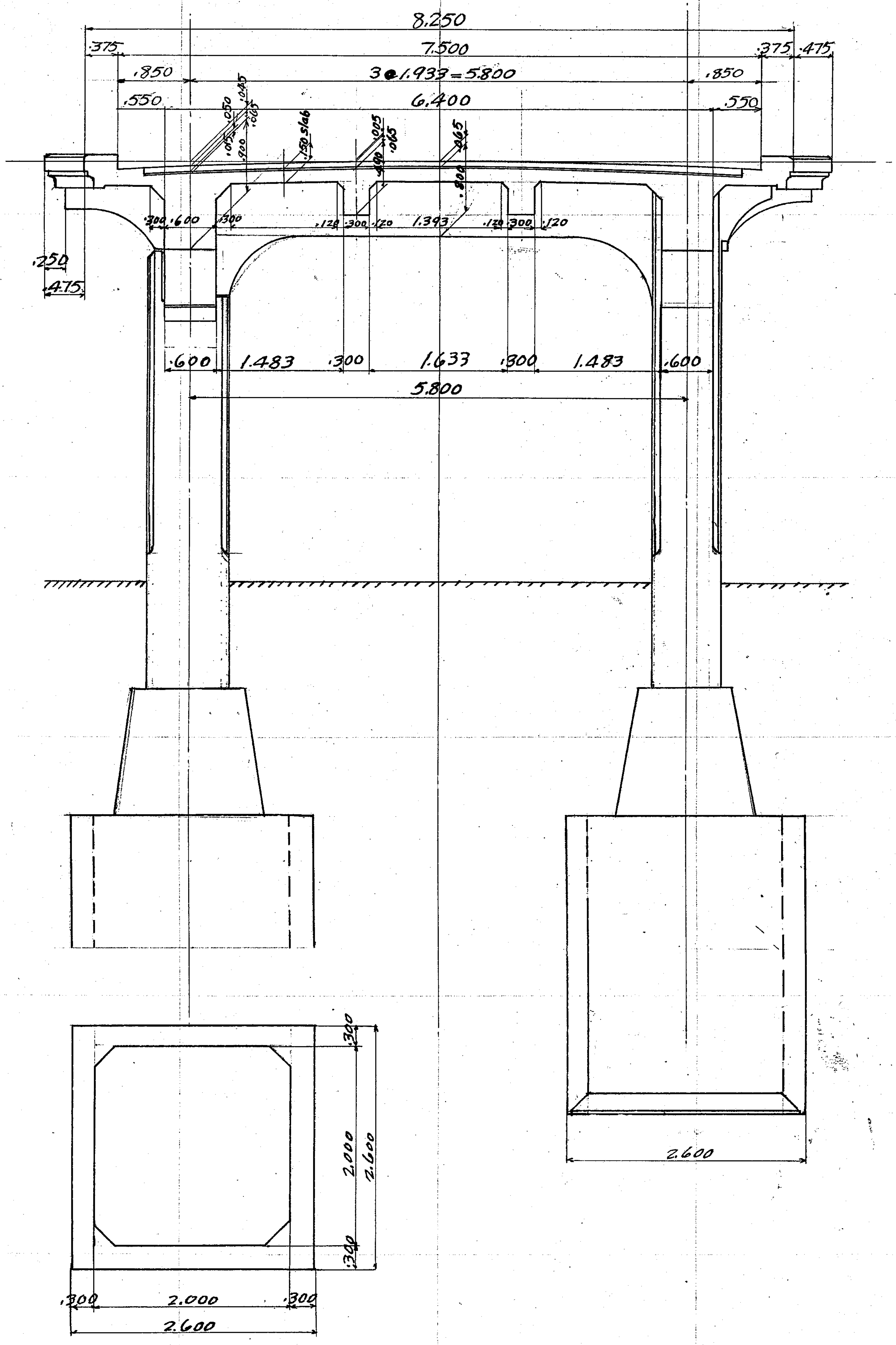
CALCULATIONS FOR



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CALCULATIONS FOR

Katsura Bashi for Kyotofu

Design of Reinforced concrete girder spans.

Floor slab span length = 1.933 meters.

Dead Load 5c asphaltic block pavement @ 21kg = 105
1.5c mortar cushion @ 22kg = 33
15c concrete slab @ 24kg = 360
Miscellaneous concrete say 12
510 kg per sq. meter.

Dead Load moment = $\frac{1}{10} \times 510 \times 1.933 = 191$ kgm

Dead Load shear = $\frac{1}{2} \times 510 \times 1.933 = 493$ kg.

Live Load. Motor truck loading

Rear wheel concentration = 3000 kg Front wheel conc. = 1,000
30 % impact = 900 30% impact = 300
3900 kg 1,300 kg

Distribution of wheel concentration on slab.

Thickness of pavement and mortar cushion = 6.5 cm

Longitudinal distribution a, contact between wheel & pavement = 20
Distribution 2@6.5 = 13
a = 33 cm

Transverse distribution b, = 27c + 13 = 40 cm

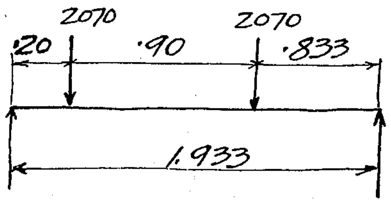
Effective width $E = \frac{2}{3}(l+b)+a$ where l = span length
$= \frac{2}{3}(1.933+.40)+.33 = 1.885$ m.

Load per meter strip = 3900 ÷ 1.885 = 2,070 kg

moment per meter strip = $1,035 \times \frac{1.933}{2} = 1,000$ kgm

For continuity of slab $0.8 \times 1000 = 800$ kgm.

End shear $2,070 \times \frac{.833}{1.933} = 892$ kg



$2070 \times \frac{1.733}{1.933} = 1855$
2,747 kg

Summary for Moment and Shear

	Moment	Shear
Dead Load	191	493
Live Load	800	2,747
	1091 kgm	3240 kg

Effective depth required for slab

$f_s = 1200$ kg/cm², $f_c = 45$ kg/cm²

$R = \frac{M}{bd^2}$, $d = \sqrt{\frac{M}{bR}}$

where R = 7.18

$d = \sqrt{\frac{1091 \times 100}{100 \times 7.18}} = 12.34$ cm

use 15 cm slab

Effective depth say 12.5 cm

Steel area req'd = $\frac{1091 \times 100}{\frac{7}{8} \times 12.5 \times 1200} = 8.31$ cm²/lin. meter

$\frac{1}{2}$"φ bars $\frac{1.27 \times 100}{8.31} = 15.3$ cm spacing

use 15 cm spacing

Unit shear = $\frac{3240}{\frac{7}{8} \times 12.5 \times 100} = 2.96$ kg/cm²

Dia	area	circumference
$\frac{1}{2}$" = 12.7 mm	.1963□" = 1.27 cm²	4.0 cm
$\frac{3}{8}$" = 9.53	.110 = 0.71	3.0

Bond stress

use $\frac{1}{2}$"φ bars 15 cm } spacing
$\frac{3}{8}$"φ " 15 "

Circumferential area $4.0 \times 6.67 = 26.7$
$3.0 \times 6.67 = 20.0$
46.7 cm²

Unit bond stress = $\frac{3240}{\frac{7}{8} \times 12.5 \times 46.7} = 6.35$ kg/cm² use deformed bars for slab.

Inside of Main girder, effective depth of slab 27.5 cm say

use $\frac{1}{2}$"φ bars 15 cm spacing only

unit bond stress = $\frac{3240}{\frac{7}{8} \times \cancel{12.5}\ 27.5 \times 26.7} = 5.04$ kg/cm² ok

Check for negative moment on the cross beam.

Load acting to cause bending moment on the longer span of the slab

Shorter span = 1.933 m longer span = 2.917 m

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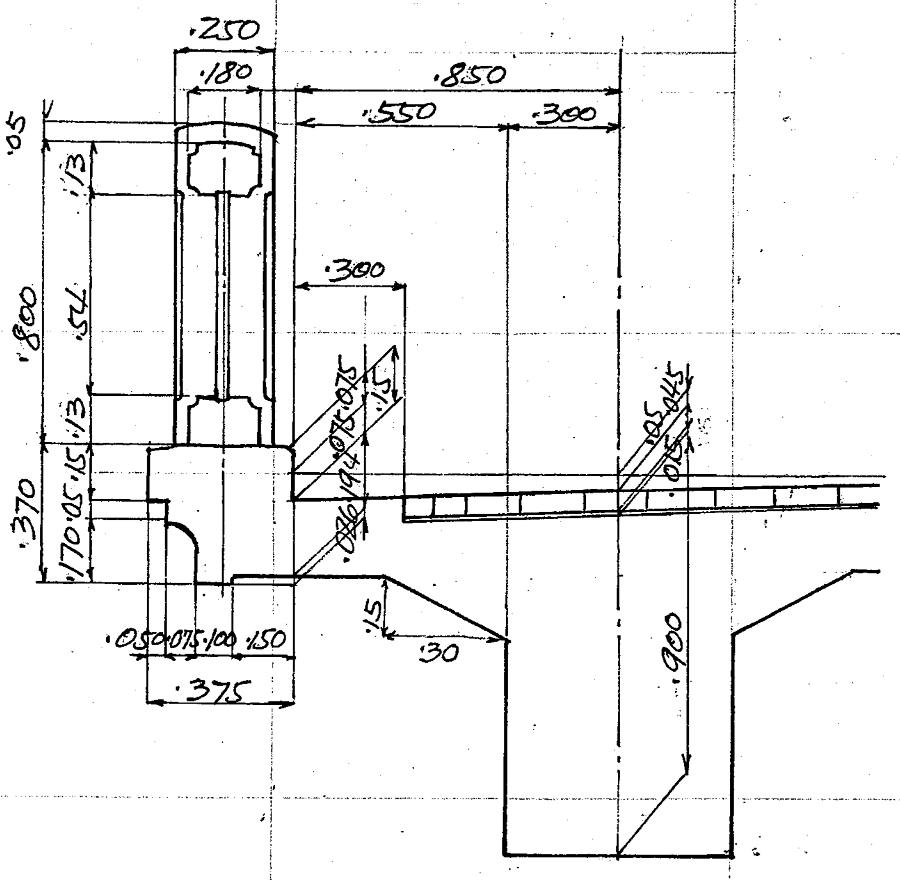
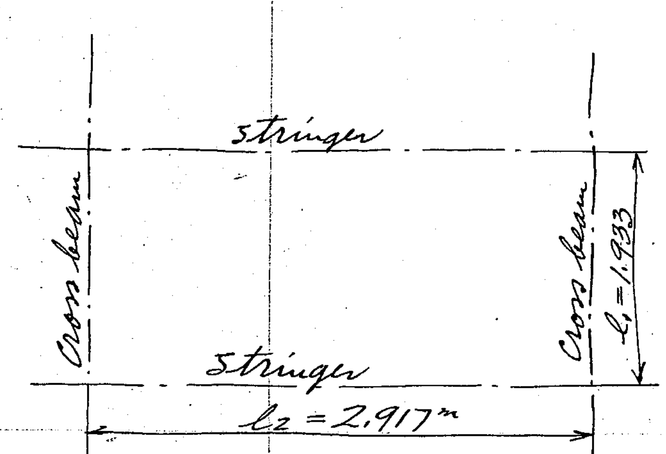
CALCULATIONS FOR

Overhanging slab beyond main girder.





$Coeff. = \frac{l_1}{l_2} - .5 = \frac{1.933}{2.917} - .5 = 0.163$

$510 \times 0.163 = 83$ kg/$\square^m$

Dead Load m $= \frac{1}{10} \times 83 \times 2.917^2 = 70$ kgm

Live Load concentration for longer span $= 3900 \times .163 = 636$ kg

Effective width say 2m for 2 wheels

Live Load per meter strip $= 636$ kg

Live Load m $= 318 \times \frac{2.917}{2} = 464$ kgm

Summary for moment

Dead Load m = 70

Live " " = 464

534 kgm

Steel req'd for neg. m. $= \frac{534 \times 100}{1200 \times \frac{7}{8} \times 12.5} = 4.07$ cm²

use $\frac{1}{2}''$ bars $\frac{1.27 \times 100}{4.07} = 31.3$ cm. spacing

Overhanging slab beyond main girder.

Handrail. see page S4

approx. weight of Handrail = 255 kg/lin meter

" " Coping = 272 "

" " slab & pavement = 280 "

Dead Load moment.

	load		arm		moment
Handrail	255	×	.725	=	185
coping	272	×	.725	=	197
Slab & pavement	280	×	.275	=	77
	807 kg.				459 kgm

Live Load

Distribution of wheel concentration assumed
2@ .35 + .20 = .90 meter
at face of main girder.

Motor truck rear wheel concentration with impact.
= 3,900 kg

Load per meter strip = 3900 ÷ 0.9 = 4,340 kg

Live Load moment = 4340 × .35 = 1,520 kgm

Live Load Shear assumed 4,340 kg per meter strip

Summary of moment & shear

	moment		shear
Dead Load	459	say	807
Live Load	1,520		4340
	1,979 kgm		5,147 kg

unit shearing stress $= \frac{5147}{\frac{7}{8} \times 27.5 \times 100} = 2.14$ kg/cm² OK.

Effective depth req'd for $f_s = 1200$, $f_c = 45$

$d = \sqrt{\frac{1979 \times 100}{100 \times 7.18}} = 16.6$ cm

Insulation = 2.5

18.1

Depth of slab use 15 cm slab with 15 cm fillet or 30 cm in total

Effective depth at face of main girder = 27.5 cm

Steel area $= \frac{1979 \times 100}{\frac{7}{8} \times 27.5 \times 1200} = 6.85$ ☐cm/m

$\frac{1}{2}''$ ☐ bars 15 cm spacing

$1.27 \times 6.67 = 8.45$ ☐cm / meter strip

Circumferential area 26.7 ☐cm/meter

unit bond stress $= \frac{5147}{\frac{7}{8} \times 27.5 \times 26.7} = 8.0$ kg/cm² OK

use deformed bars in slab to carry necessary bond stress.

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CALCULATIONS FOR

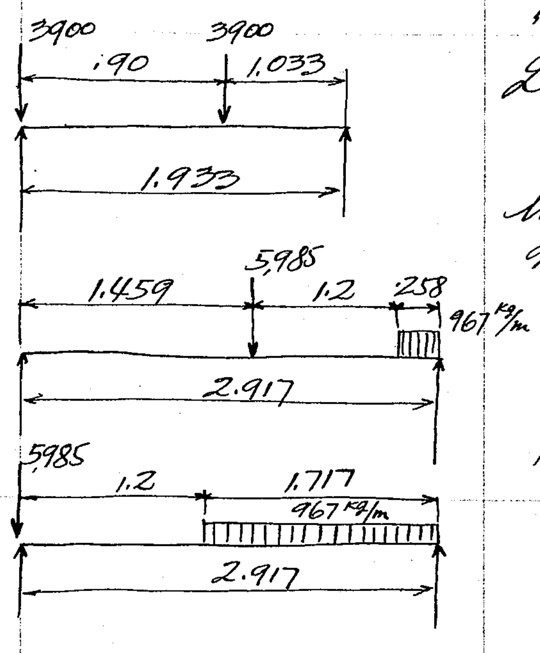
Katsura Bashi for Kyotofu.

Design of Stringers. 2.917 meter span. 1.333 meter spacing

Dead Load floor slab and pavement 510 × 1.933 = 986
Stem of beam assumed .34 × .30 @ 2400 = 245
fillet say 6
1237 kg/meter

Dead Load moment = $\frac{1}{10}$ × 1237 × 2.917² = 1,052 kgm
Dead Load shear = $\frac{1}{2}$ × 1237 × 2.917 = 1,805 kg

Live Load



Motortruck rear wheel concentration with impact = 3900 kg
" " front " " " = 1,300 "

Load on stringer = 3900 × $\frac{1.033}{1.933}$ = 2,085 kg
3,900
5,985 kg

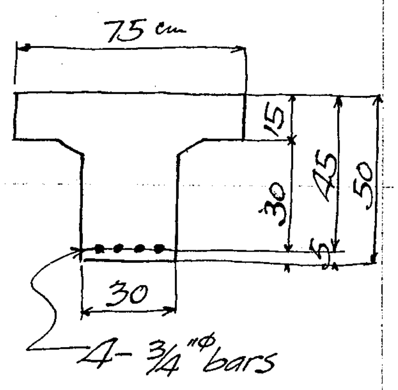
Uniform live load = 500 × 1.933 = 967 kg per lin meter of span.
Moment due to motortruck = 2993 × 1.459 = 4,365
" " unif. load = $\frac{250 \times .129 \times 1.459}{2.917}$ = 16
4381 kgm

for continuity 4,381 × 0.8 = 3,505 kgm

Reaction due to uniform load = $\frac{967 \times 1.717^2}{2 \times 2.917}$ = 488
due to motor truck 5,985
6473 kg

Summary for moments and shears

	Moment	Shear
Dead Load	1,052	1,805
Live Load	3,505	6,473
	4557 kgm	8,278 kg



steel area req'd = $\frac{4557 \times 100}{1200 \times \frac{7}{8} \times 45}$ = 9.67 □cm

use 4 - 3/4"φ = 11.4 □cm

steel ratio $p = \frac{11.4}{75 \times 45}$ = 0.0034 $t/d = \frac{15}{45}$ = 0.333

neutral axis in the flange.
K = .271, j = .910

Steel stress = $\frac{4557 \times 100}{11.4 \times .910 \times 45}$ = 977 kg/cm² OK

conc. stress = $\frac{f_s k}{n(1-k)} = \frac{977 \times .271}{15 \times .729}$ = 24.2 " OK

unit shear = $\frac{8278}{30 \times .91 \times 45}$ = 6.73 " use stirrups

unit bond = $\frac{8278}{6 \times 4 \times .91 \times 45}$ = 8.42 use deformed bars to carry necessary bond stress.

Stirrup spacings.
It is assumed that stirrups take 2/3 shearing stresses
use 3/8"φ = .952 cmφ u-stirrups $A_s = 2 \times 0.713 = 1.426$ □cm
Spacing of stirrup $s = \frac{3}{2} \cdot \frac{A_s f_s j d}{V} = \frac{3}{2} \cdot \frac{1.426 \times 1200 \times .91 \times 45}{8278}$
= 12.7 cm

for 1/2"φ = 1.27 cmφ u-stirrups $A_s = 2 \times 1.267 = 2.534$ □cm
$S = \frac{3}{2} \frac{2.534 \times 1200 \times .91 \times 45}{8278}$ = 22.6 cm

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CALCULATIONS FOR

Katsura-Bashi for Kyotofu.

Design of Cross Beam. Span length = 5.8 meters, spacing = 2.917 meters.

Dead Load.

Dead Load concentration = 1,237 Kg/meter see page

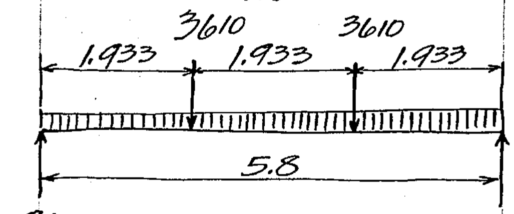
1,237 × 2.917 = 3,610 Kg

Stem of cross beam assumed 65 cm × 38 cm

.65 × .38 @ 2400 = 592

fillet say 8

600 Kg per lin meter.



3610
1740
5350

600 × 2.9 = 1,740 Kg

Dead Load moment

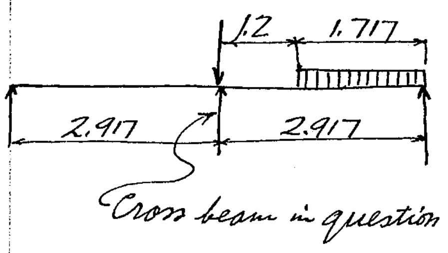
due to stringer concentration = 3610 × 1.933 = 6990

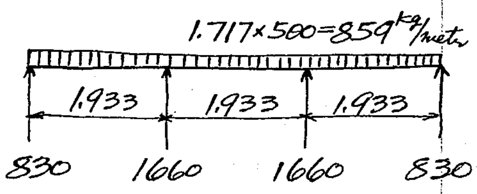
" " Stem and fillet = ⅛ × 600 × 5.8² = 2,522

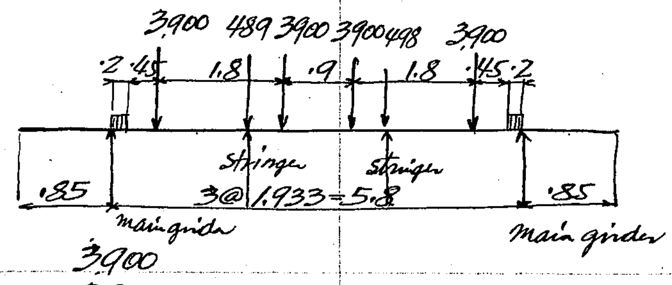
9,512 Kgm

End shear 5,350 Kg

Live Load.







Stringer concentration on crossbeam

$1660 \times \frac{859}{2.917} = 489$ Kg

3,900
3,900
489
8,289

Unif. load at both ends neglected.

Live Load moment

8289 × 2.45 = 20,300

less 3900 × 1.8 = − 7,020

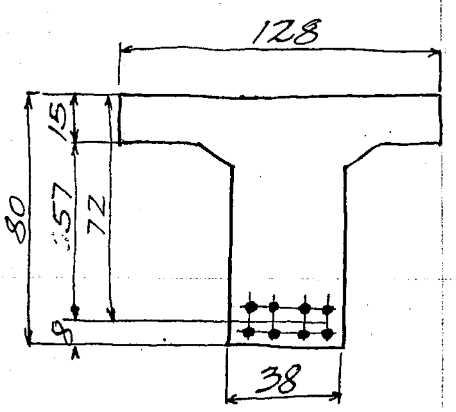
" 489 × 0.517 = − 253

13,027 Kgm

End shear = 8,289 Kg

Summary of moments and shears.

	moment	shear
Dead Load	9,512	5350
Live Load	13027	8289
	22,539 Kgm	13,639 Kg



Section assumed as left figure.

Steel area required = $\frac{22,539 \times 100}{1200 \times \frac{7}{8} \times 72}$ = 29.8 cm²

use 8 − ⅞″ϕ (2.22 cmϕ) bars = 31.01 ocm

steel ratio $p = \frac{31.01}{72 \times 128} = 0.0033$ $\frac{t}{d} = \frac{15}{72} = 0.21$

K = 0.275, j = 0.92

$f_s = \frac{22539 \times \cancel{0.275} \times 100}{31.01 \times 0.92 \times 72}$ = 1,096 Kg/cm² OK

$f_c = \frac{1096 \times 0.275}{15 \times 0.725}$ = 22.8 Kg/cm² OK

unit bond = $\frac{13639}{7 \times .92 \times 72}$ = 29.5 for one bar

29.5 ÷ 9 = 4 bars required at end for deformed bars.

unit shear = $\frac{13639}{38 \times .92 \times 72}$ = 5.42 Kg/cm² Stirrups necessary

use ½″ϕ (1.27 cmϕ) U-stirrups, $A_s = 2 \times 1.267 = 2.534$ ocm

stirrup spacing = $\frac{3}{2} \cdot \frac{2.534 \times 1200 \times .92 \times 72}{13,639}$ = 22.1 cm

use 20 cm spacing at support

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CALCULATIONS FOR

Katsura-Bashi for Kyotofu.

Negative moment due to continuity of cross beam to main girder.

Taking care for neg. moment of $\frac{8}{20}$ of positive moment at center of span.

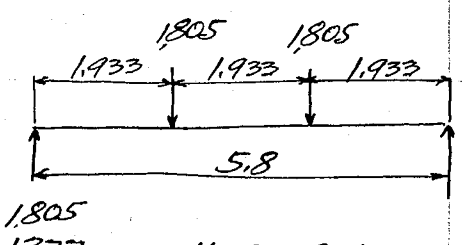
Dead Load moment $9{,}512 \times 8/20 = -\ 3{,}805$ kgm Steel area req'd $= \frac{9015 \times 100}{1200 \times \frac{7}{8} \times 75} = 11.45$ cm²

Live Load moment $13{,}027 \times 8/20 = -\ \underline{5{,}210}$

-9015 kgm use 3 - 7/8"φ (2.22 cm²φ) = 11.65 cm²

Cross beams at Expansion Joint.

Dead Load.



Span length = 5.8 meters, spacing 2.917 meters.

Dead Load concentration = 1237 kg/m.

$1239 \times \frac{2.917}{2} = 1{,}805$ kg.

Stem assumed $.65 \times 30 @ 2400 = 468$

fillet say $\underline{7}$

475 kg/meter.

1,805
$\underline{1{,}377}$ $475 \times 2.9 = 1{,}377$ kg
3,182 kg

Dead Load moment

due to stringer concentration $= 1805 \times 1.933 = 3{,}490$

" " Stem and fillet $= \frac{1}{8} \times 475 \times 5.8^2 = \underline{2{,}000}$

5,490 kgm

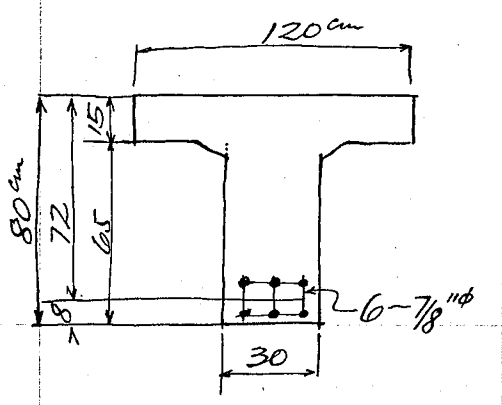
End shear 3,182 kg

Live Load moment and shear same as for intermediate cross beams see page

Summary for moments and shears.

Assumed section as follows.

	Moment	Shear
Dead Load	5,490	3,182
Live Load	13,027	8289
	18,517 kgm	11,471 kg



Steel ratio $p = \frac{6 \times 3.88}{120 \times 72} = 0.0027$ $\frac{t}{d} = \frac{15}{72} = 0.21$

$k = 0.24$ $j = 0.92$

$f_s = \frac{18{,}517 \times 100}{6 \times 3.88 \times .92 \times 72} = 1{,}200$ kg/cm² OK.

$f_c = \frac{1200 \times .24}{15 \times .76} = 25.3$ " OK

unit bond stress $= \frac{11{,}471}{7 \times .92 \times 72} = 24.75$ for one bar

$\frac{24.75}{9} = 3$ bars necessary at end.

Unit shear $= \frac{11471}{30 \times .92 \times 72} = 5.77$ kg/cm² use stirrups.

It is assumed that stirrups take 2/3 of shearing stress.

Stirrup spacing $s = \frac{3}{2} \cdot \frac{2.5\ 34 \times 1200 \times .92 \times 72}{11471}$

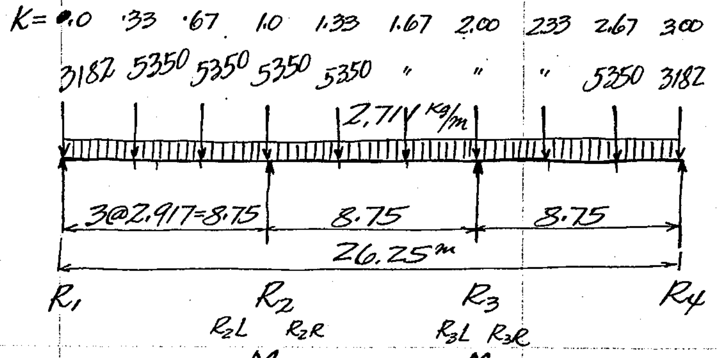
$= 26.3$ cm.

use the same spacings as for intermediate cross beams.

For Negative reinforcements at fixture of cross beam to main girders, use same details as for intermediate cross beams.

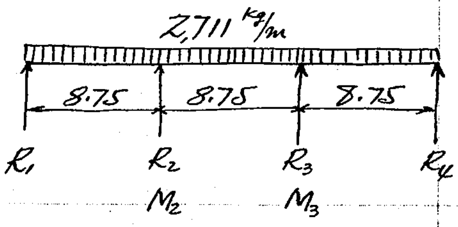
Katsura-Bashi for Kyotofu

Design of Main girder. Span length 8.75 meters, 2 girders.
Dead Load. 3 span continuous beam with both ends free.



Dead Load Concentration = 5,350 kg See page C6
End crossbeam " = 3,182 kg
Uniform load
Handrail 185 kg/meter
Coping 277 " $510 \times \frac{1.933}{2} = 493$
Slab and pav. 773 " $\frac{280}{773}$
1235 kg per meter
Stem of main girder assumed 75 cm deep at center & 115 cm at end mean 95 cm
Stem .95×.60@2400=1368
fillet .15×.30@ " =108
1476 kg per meter
Total = 2,711 kg per meter of span.

Uniform load moment + shear.



By the Theorem of Three Moments, for uniform load.

$$M_1 + 4M_2 + M_3 = -\frac{wl^2}{2}$$

we have

$$0 + 4M_2 + M_3 = -\frac{wl^2}{2}$$

$$M_2 + 4M_3 + 0 = -\frac{wl^2}{2}$$

Solving these formulas

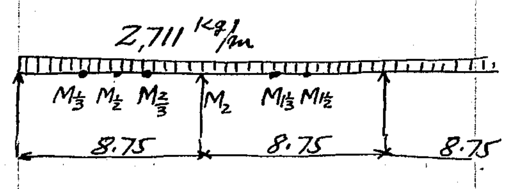
$$M_2 = -\frac{wl^2}{10}, \quad M_3 = -\frac{wl^2}{10}.$$

$$R_1 = R_4 = \frac{wl}{2} - \frac{wl}{10} = \frac{4}{10}wl$$

$$R_{2L} = R_{3R} = \frac{wl}{2} + \frac{wl}{10} = \frac{6}{10}wl$$

$$R_{2R} = R_{3L} = \frac{wl}{2} + \frac{wl}{10} - \frac{wl}{10} = \frac{5}{10}wl.$$

$$R_2 = R_3 = \frac{11}{10}wl.$$



Shears and Reactions

$$R_1 = R_4 = \frac{4 \times 2711 \times 8.75}{10} = 9{,}500\ kg$$

Shear $R_{2L} = R_{3R} = \frac{6 \times 2711 \times 8.75}{10} = \mp 14{,}250$

$$R_{2R} = R_{3L} = \frac{5 \times 2711 \times 8.75}{10} = \pm 11{,}860$$

$$R_2 = R_3 = 26{,}110\ kg$$

$$M_{\frac{4}{10}} = 9{,}500 \times \frac{8.75 \times 4}{10} - \frac{2711}{2} \times \left(\frac{8.75 \times 4}{10}\right)^2 = +15{,}660\ kgm$$

Bending moments

$$M_2 = M_3 = -\frac{2711 \times 8.75^2}{10} = -20{,}750\ kgm$$

Moment at $\frac{1}{3}$ point from left end support, call this $M_{\frac{1}{3}}$

$$M_{\frac{1}{3}} = 9500 \times \frac{8.75}{3} - \frac{2711}{2} \times \left(\frac{8.75}{3}\right)^2 = +16{,}160\ kgm$$

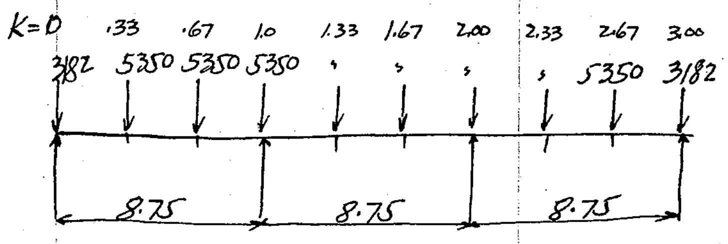
$$M_{\frac{1}{2}} = 9500 \times \frac{8.75}{2} - \frac{2711}{2} \times \left(\frac{8.75}{2}\right)^2 = +15{,}575$$

$$M_{\frac{2}{3}} = 9500 \times \frac{8.75 \times 2}{3} - \frac{2711}{2} \times \left(\frac{8.75 \times 2}{3}\right)^2 = +9{,}250\ kgm$$

$$M_{1\frac{1}{3}} = 9500 \times 8.75 \times \frac{4}{3} - \frac{2711}{2} \times \left(\frac{8.75 \times 4}{3}\right)^2 = \cancel{+76} + 26110 \times \frac{8.75}{3} = +2{,}150\ kgm$$

$$M_{1\frac{1}{2}} \quad 9500 \times 8.75 \times \frac{3}{2} - \frac{2711}{2}\left(\frac{8.75 \times 3}{2}\right)^2 + 26110 \times \frac{8.75}{2} = +5{,}200\ kgm$$

Dead Load moment due to cross beam concentration.



By the prepared diagram

load	k	M_2 unit load	$10M_2/l$ (moment for 10m span)
3182	.00	- .000	0
5350	.33	- .780	-4,170
"	.67	- .990	-5,295
"	1.00	- .000	0
"	1.33	- .790	-4,225
"	1.67	- .540	-2,888
"	2.00	- .000	0
"	2.33	+ .242	+1,295
"	2.66	+ .200	+1,070
3182	3.00	+ .000	0
			-14,213

$$M_2 = M_3 \doteqdot -14213 \times \frac{8.75}{10} = -12{,}440\ kgm.$$

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CALCULATIONS FOR

Katsura Bashi for Kyotofu.

Dead Load Reactions due to Crossbeam concentrations.

Reactions R_1 & R_4

Loads	K.	$R_1=R_4$ unit load	(R_1 or R_4)
3182	0	+ 1.00	+ 3,182
5,350	.33	+ .59	+ 3,158
5,350	.67	+ .23	+ 1 230
〃	1.00	0	0
〃	1.33	− .08	− 428
〃	1.67	− .06	− 321
〃	2.00	0	0
〃	2.33	+ .025	+ 134
〃	2.67	+ .025	+ 134
3182	3.00	0	0
		$R_1=R_4$ =	+ 7,089 kg

Reactions R_2 & R_3

Loads	$R_2=R_3$ unit load	(R_2 or R_3)
3182	+ 0	0
5350	+ .505	+ 2702
〃	+ .890	+ 4760
〃	+ 1,000	+ 5350
〃	+ .700	+ 4120
〃	+ .360	+ 1926
〃	0	0
〃	− .150	− 803
〃	− .120	− 642
3182	0	0
$R_2=R_3$ =		+ 17,413 kg

End shears.

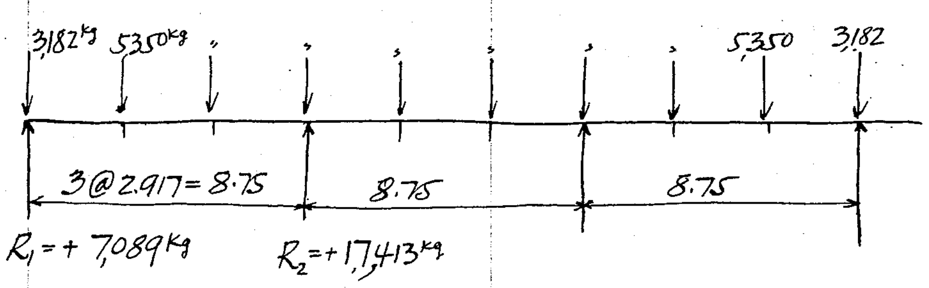
R_{2L} & R_{3R}

Load	K	Shear unit load	Shear
3182	0	0	0
5350	.33	− .41	− 2,195
〃	.67	− .77	− 4,120
〃	1.00	− 1.00	− 5,350
〃	1.33	− .078	− 417
〃	1.67	− .055	− 294
〃	2.00	0	0
〃	2.33	+ .025	+ 134
〃	2.67	+ .020	+ 107
3182	3.00	0	0
		$R_{2L}=R_{3R}$ =	− 12,135 kg

R_{2R} & R_{3L}

Load	Shear unit load	Shear
3,182	0	0
5,350	+ .098	+ 524
〃	+ .122	+ 652
〃	+ 1.000	+ 5,350
〃	+ .691	+ 3,697
〃	+ .309	+ 1,654
〃	0	0
〃	− .122	− 652
〃	− .098	− 524
3182	0	0
$R_{2R}=R_{3L}$ =		+ 10,701

Dead Load Positive Moments due to Crossbeam concentrations at several Points.



At 1/3 l from left support, call this moment $M_{\frac{1}{3}}$.

$$M_{\frac{1}{3}} = 7089 \times \frac{8.75}{3} - 3182 \times \frac{8.75}{3} = +12,390 \text{ kgm}$$

at 2/3·l from left support. 〃 $M_{\frac{2}{3}}$

$$M_{\frac{2}{3}} = 7089 \times \frac{8.75}{3} - 3182 \times \frac{8.75 \times 2}{3} - 5350 \times \frac{8.75}{3} = +7,140 \text{ kgm}$$

$$M_{1\frac{1}{3}} = \frac{(7089-3182)\,8.75 \times 4}{3} + 17,413 \times \frac{8.75}{3} - \frac{5350 \times 8.75 \times 6}{3}$$

$$= +2,740 \text{ kgm}$$

$$M_{\frac{1}{2}} = (7089-3182)\frac{8.75}{2} - \frac{5350 \times 8.75}{6} = +9,360 \text{ kgm}$$

$$M_{1\frac{1}{2}} = (7089-3182)\,8.75 \times 1.5 + 17.413 \times \frac{8.75}{2} - \frac{5350 \times 8.75 \times 8}{3}$$

$$= +3,047 \text{ kgm}$$

$$M_{\frac{4}{10}} = (7089-3182)\frac{8.75 \times 4}{10} - 5350 \times 8.75 \times .067$$

$$= +10,560 \text{ kgm}$$

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MADE BY K. G. DATE 2-7-2 FILE NO

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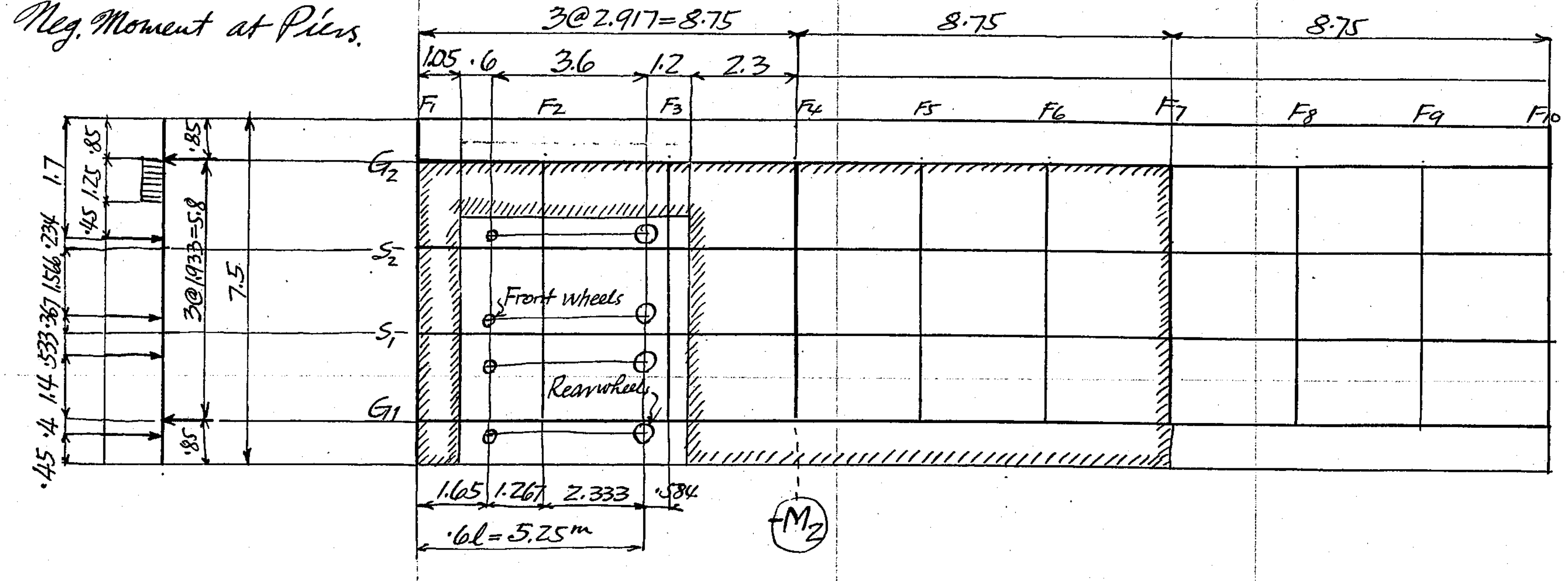
CALCULATIONS FOR

Katsura-Bashi for Kyoto-fu.

Live Load
Neg. Moment at Piers.



By the aid of Prepared diagrams, we find the rear wheels of motor trucks at 0.6l distant from the left end, to cause max. neg. moment at 2nd support for the following load.

Motor truck loads.

Impact coefficient $= \frac{20}{60+l} = \frac{20}{68.75} = 29.1\%$

where l = span length in meter.

Motor truck rear wheel $3000 \times 1.291 = 3870$ kg

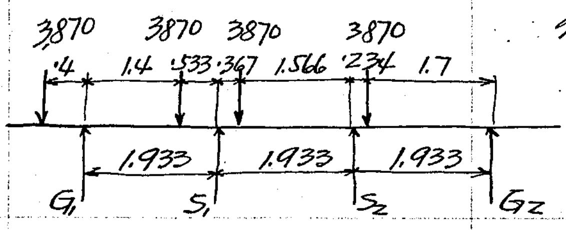
" " front " $1000 \times 1.291 = 1,290$ "

Uniform loads.

$w = \frac{120,000}{170+l} = \frac{120,000}{178.75} = 670$ kg/m²

take at 600 kg/m².

Rear wheel concentration on stringers and main girder G1.

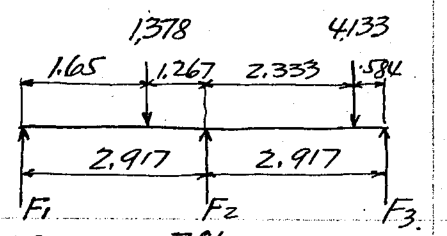


Extra load on main girder $\frac{3870 \times 0.4}{5.8} = 267$ kg.

G1	S1	S2	G2
3870	3137	733	
1067	2803	3400	470
4937	5940	4133	470 kg
add 267			
5,204 kg			

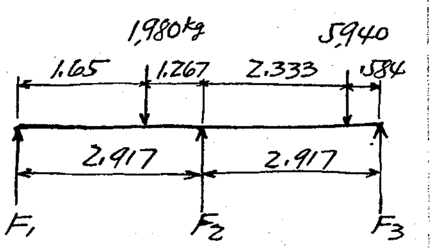
Front wheel concentration on stringers & main girder G1.
Take ⅓ of rear wheel concentrations. or
1,735 kg 1,980 1,378 157 kg.

Wheel concentrations on cross beams F1, F2 & F3.



Stringer S2

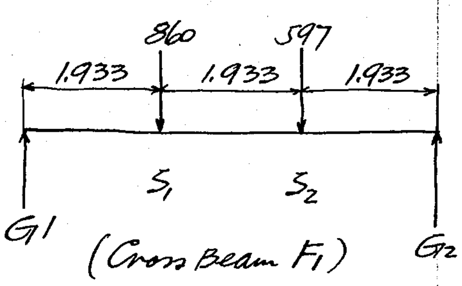
F1	F2	F3
597	781	
	828	3305
597 kg	1609	3305 kg

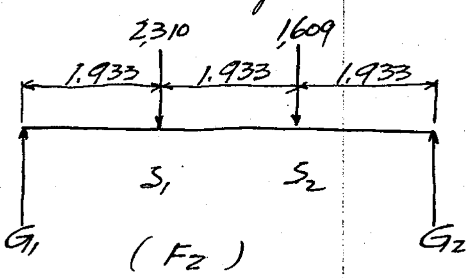


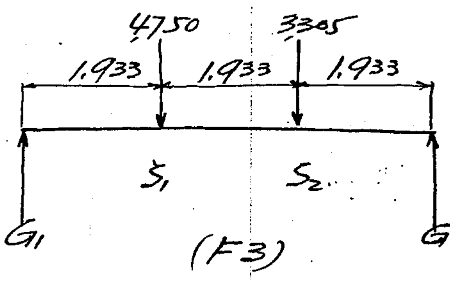
Stringer S1.

F1	F2	F3
860	1120	
	1,190	4,750
860	2,310	4,750

Motor truck concentrations on Main girders.

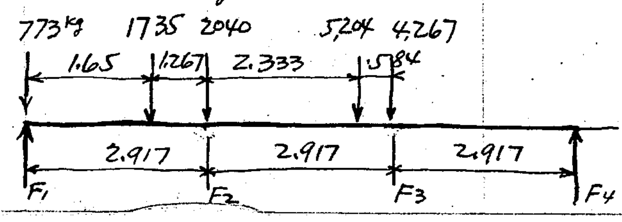






F1: G1	F1: G2	F2: G1	F2: G2	F3: G1	F3: G2
573	287	1,504	806	3165	1,585
200	397	536	1073	1102	2203
773 kg	684 kg	2040	1,839	4267.	3788 kg.

Resulting wheel loads on main girder G1.



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MADE BY K. J. DATE 2-7-3 FILE NO.
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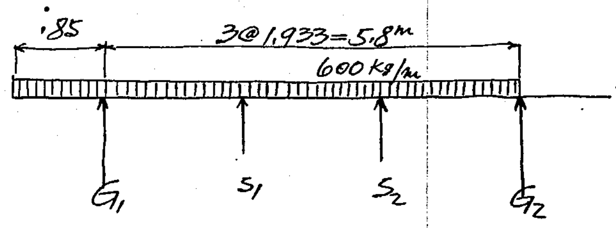
CALCULATIONS FOR

Katsura-Bashi for Kyotofu.

Uniform Live Load Concentrations.

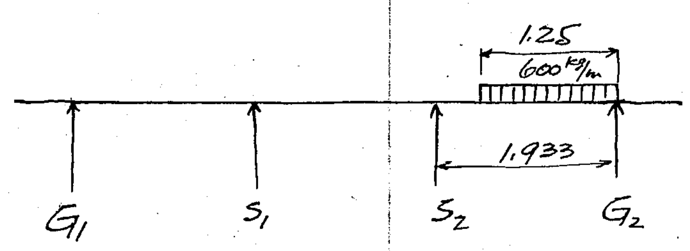
Uniform Load on the 1st. span before and behind of motor truck.



G1	S1	S2	G2
580	1,160	1,160	580
510			
add 37			
1,127 kg.	1,160	1,160	580

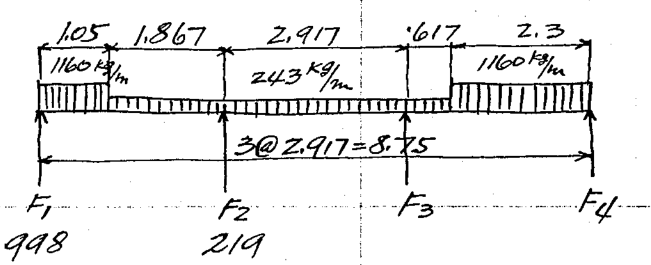
Extra load on G1 $= \frac{600 \times .85 \times .85}{2 \times 5.8} = 37$ kg.

Uniform load on the G_2 side of motor truck.



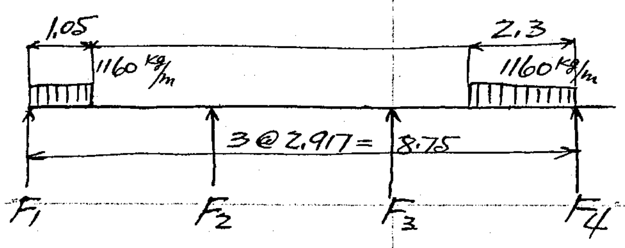
G1	S1	S2	G2
		243	507

Concentration on Floor Beam due to stringer S_2



F1	F2	F3	F4
998	219		
145	309		
	355	355	
		134	16
		1,050	1616
1,143 kg.	883	1,539	1,632

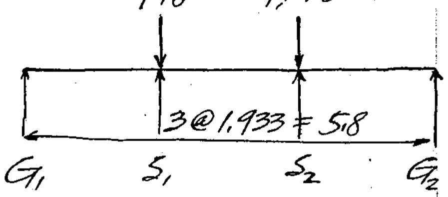
Concentration on floor beam due to stringer S_2



F1	F2	F3	F4
998	219	1,050	1,616

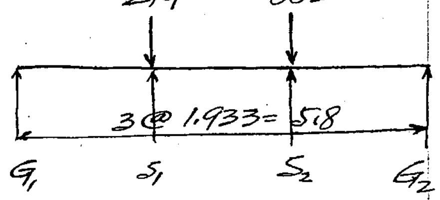
Concentration on Main Girder due to floor beams.

Floor Beam F1



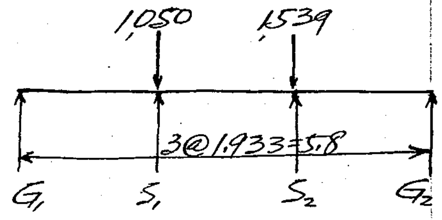
G1	G2
665	333
381	762
1,046	1,095

Floor Beam F2



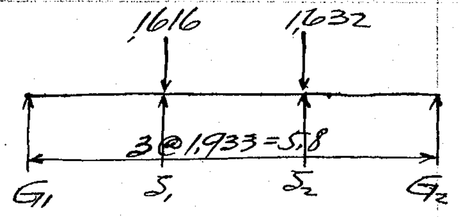
G1	G2
146	73
294	589
440	662

Floor Beam F3



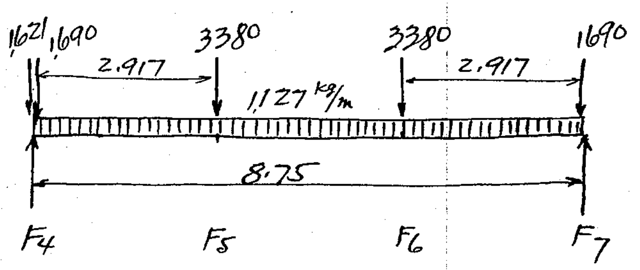
G1	G2
700	350
513	1026
1,213	1,376

Floor Beam F4



G1	G2
1,077	539
544	1088
1,621	1,627

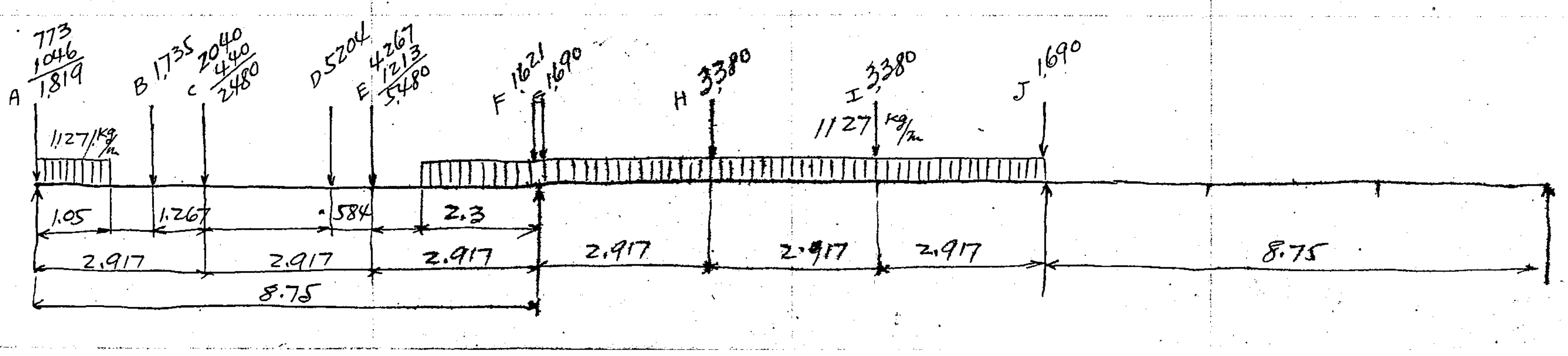
Full Uniform Load on the next span.



Floor Beam concentration due to F4 & F7
$600 \times 1.933 \times \frac{2.917}{2} = 1,690$ kg

do. F5 & F6.
$600 \times 1.933 \times 2.917 = 3,380$ kg

Combined Loading for M_2 max.



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MADE BY K. J. DATE 2-7-3 FILE NO
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CALCULATIONS FOR

Katsura Bashi for Kyotofu

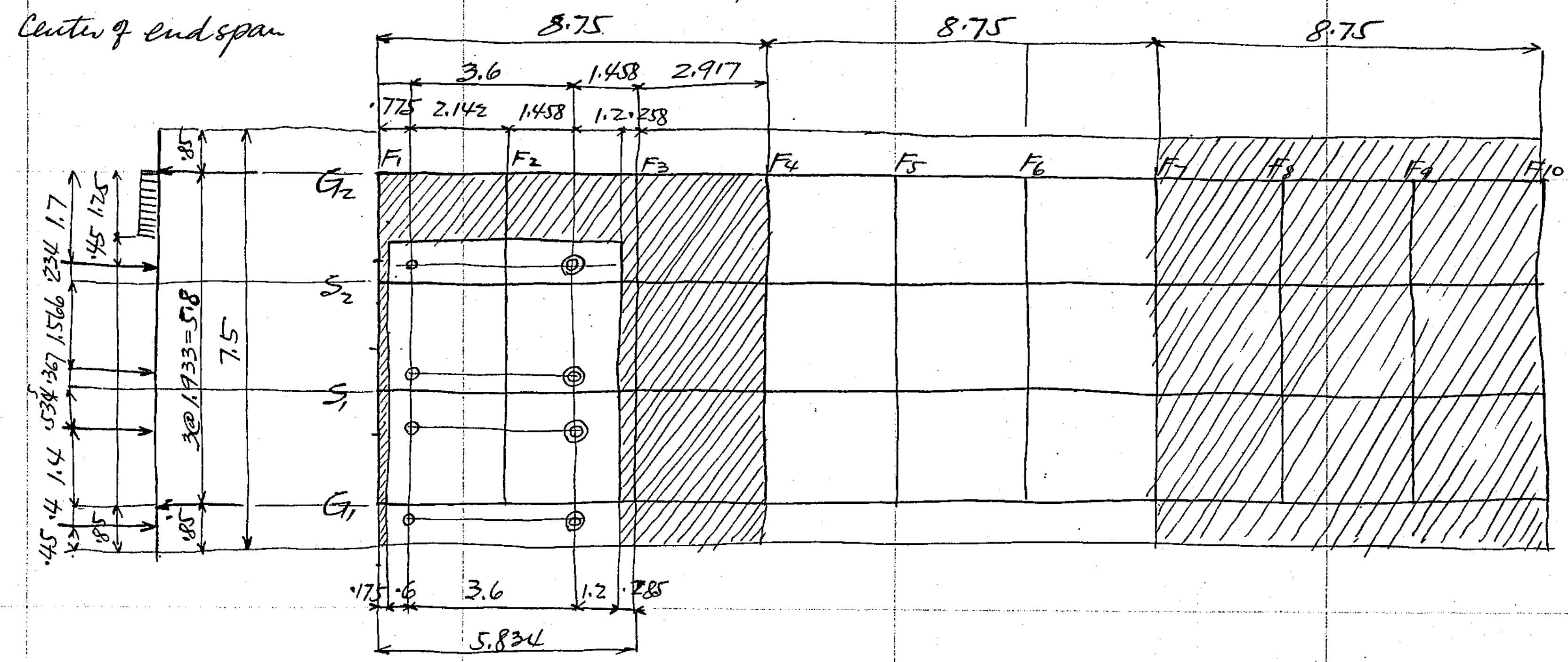
Max. negative moment M_2 (or $=M_3$).
By the prepared influence diagram.

	Loads	K.	M_2 unit load	$M_2 \frac{10}{l}$
A	1,819	0	0	0
B	1,735	.19	-.49	- 850
C	2,480	.33	-.78	- 1,935
D	5,204	.60	-1.024	- 5,325
E	5480	.66	-1.00	- 5480
F	1,621	1.00	0	0
G	1,690	1.00	0	0
H	3380	1.33	-.79	- 2670
I	3,380	1.67	-.54	- 1825
J	1,690	2.00	0	0
				~~-24,971~~
				- 18,085

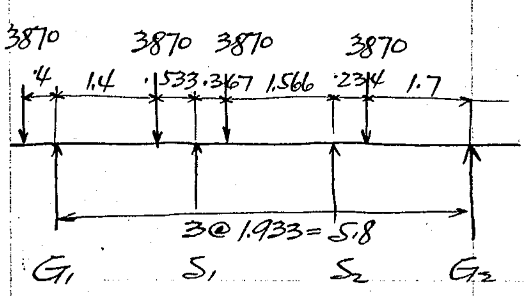
Uniform load	K	M_2 unit load	$M_2 \frac{10}{l}$
1,127 × 1.05 = 1,183	.06	- .17	- 201
1,127 × 2.3 = 2,590	.87	- .55	- 1,425
1,127 × 2.917 = 3,288	1.17	- .58	- 1,907
" = 3,288	1.50	- .75	- 2,465
" = 3,288	1.83	- .27	- 888
			-6,886
			-18,085
			-24,971

Live Load max $M_2 = M_3 = -24,971 \times \frac{8.75}{10}$
$= -21,840$ kgm.

Live Load Pos. Moment at Center of end span



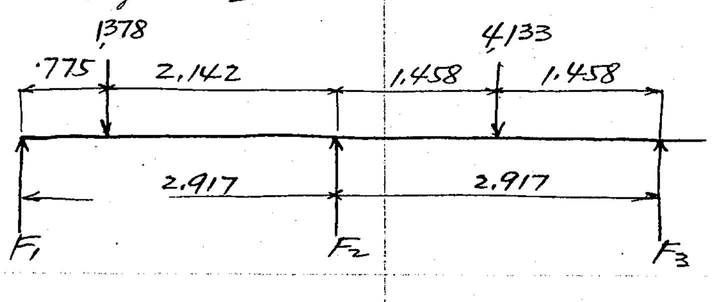
Wheel concentrations on Stringers and Main girders



	G1	S1	S2	G2
	5204	5940	4133	470
	1,735	1,980	1,378	157

Wheel concentrations on floor beam F1, F2 & F3

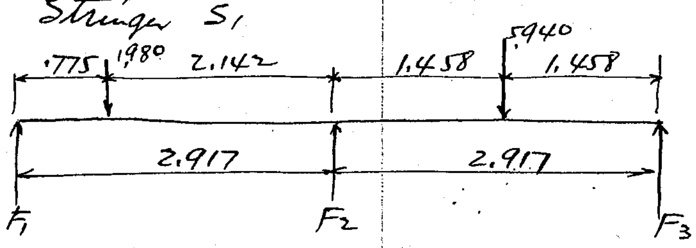
Stringer S2



Rear wheel concentration.
Front wheel "

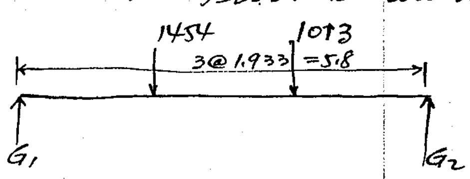
1,013 | 365 / 2067 / 2432 | 2067

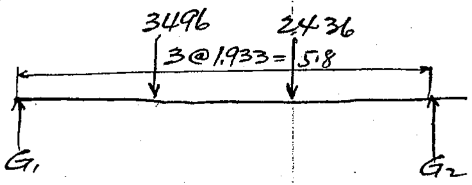
Stringer S1

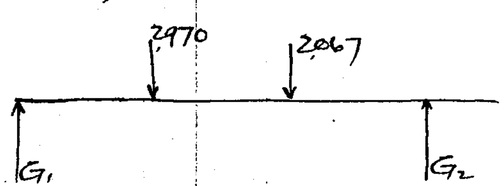


1,454 | 526 / 2970 / 3,496 | 2970

Wheel Load concentrations on main girders due to Floor Beams







G1	G2	G1	G2	G1	G2
970	484	2330	1166	1967	1003
338	675	814	1622	689	1,378
1,308	1,159	3,144	2,788	2656	2,381

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MADE BY K. Inaba DATE 2-7-3 FILE NO

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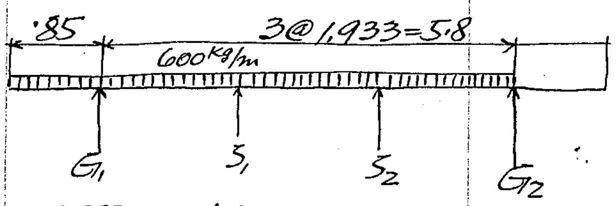
CALCULATIONS FOR

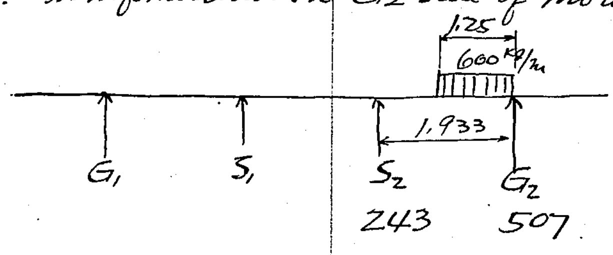
Katsura Bashi for Kyotofu.

Uniform Live Load concentrations.

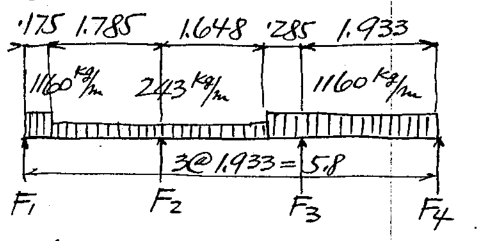
Uniform loads before and behind of motor truck. Uniform load on G_2 side of motor truck.



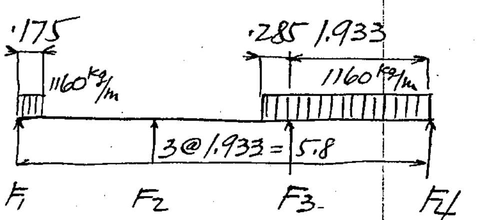
1127 1160 1160 580 See p 11



Concentration on Floor Beams due to stringer S_2. Concentrations on floor beams due to stringer S_1

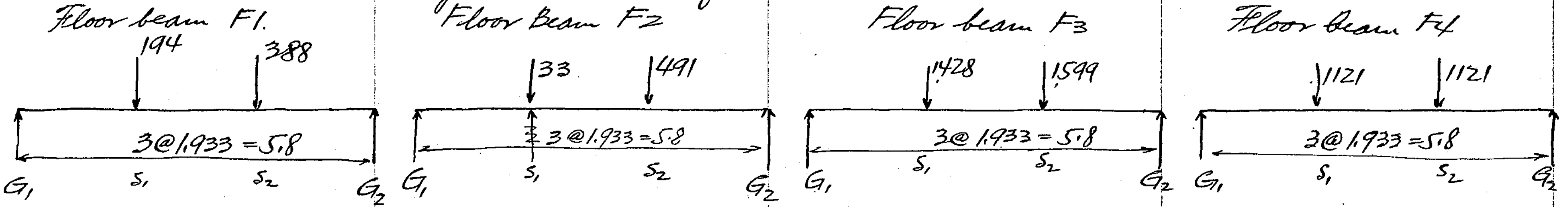


F_1	F_2	F_3	F_4
194	9		
194	233		
	234	177	
	24	307	
		1121	1121
388	491	1599	1121



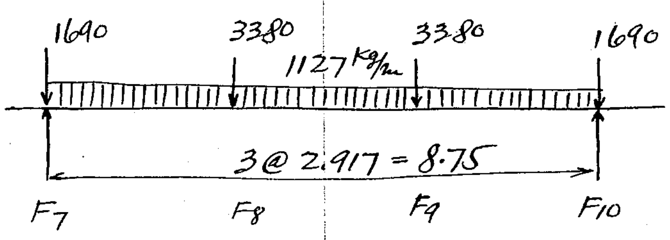
F_1	F_2	F_3	F_4
194	9		
	24	307	
		1121	1121
194	33	1428	1121

Concentration on Main Girder due to floor beam.

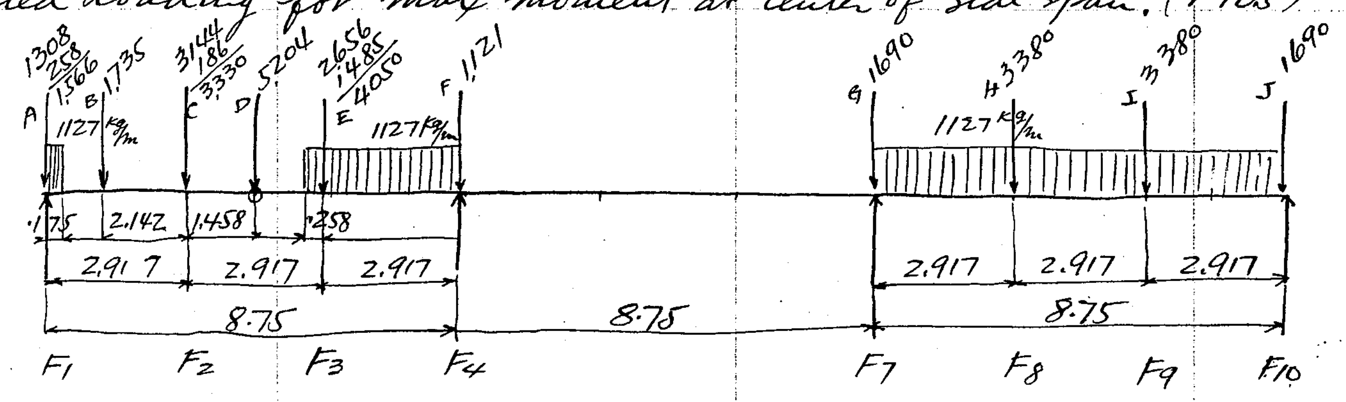


F1 G_1	F1 G_2	F2 G_1	F2 G_2	F3 G_1	F3 G_2	F4 G_1	F4 G_2
129	65	22	11	952	476	1121	1121
129	259	164	327	533	1066		
258	324	186	338	1485	1542	1121	1121

Uniform load on the 3rd span same as for 2nd span for M2. previously discussed.



Combined Loading for max moment at center of side span. (M_{05})



Katsura Bashi for Kyotofu.

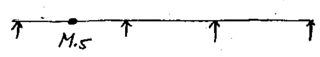
Max. Pos. Moment at center of end span ($M_{0.5}$)

Loads		K.	$M_{.5}$ Unit load	$M_{.5}\frac{10}{\ell}$
A	1566	0	0	0
B	1735	.09	.33	+573
C	3330	.33	1.25	+4160
D	5204	.50	2.04	+10,640
E	4050	.67	1.16	+4700
F	1121	1.00	0	0
G	1690	2.00	0	0
H	3380	2.33	.13	+ 440
I	3380	2.67	.10	+ 338
J	1690	3.00	0	0
				+20,851

Uniform loads	K	$M_{.5}$ unit load	$M_{.5}\frac{10}{\ell}$
1127 × .175 = 197	.01	.03	+ 6
1127 × 3.175 = 3580	.83	.58	+ 2076
1127 × 2.917 = 3288	2.17	.09	+ 296
1127 × 2.917 = 3288	2.50	.13	+ 428
" = 3288	2.83	.06	+ 197
			+ 3,003
			20,851
			23,854

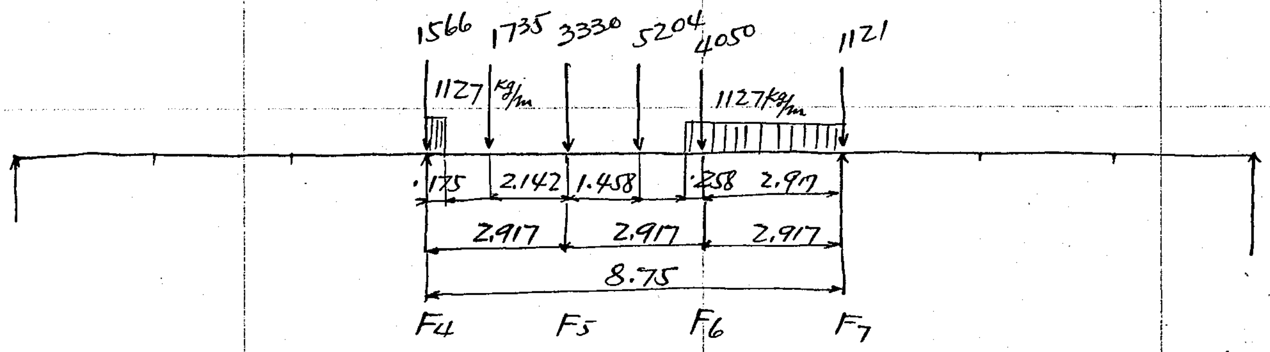
Max Live Load moment at center of end span

$+M_{0.5} = +23,854 \times \frac{8.75}{10} = +\underline{20,900}$ kgm.



Live Load Pos. moment at center of center span ($M_{1.5}$)

All Loadings are just the same as for $M_{0.5}$ loaded at center span instead of side span and get off the uniform loads on the third span.

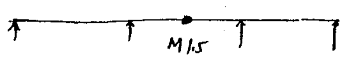


Max. Pos. moment $M_{1.5}$

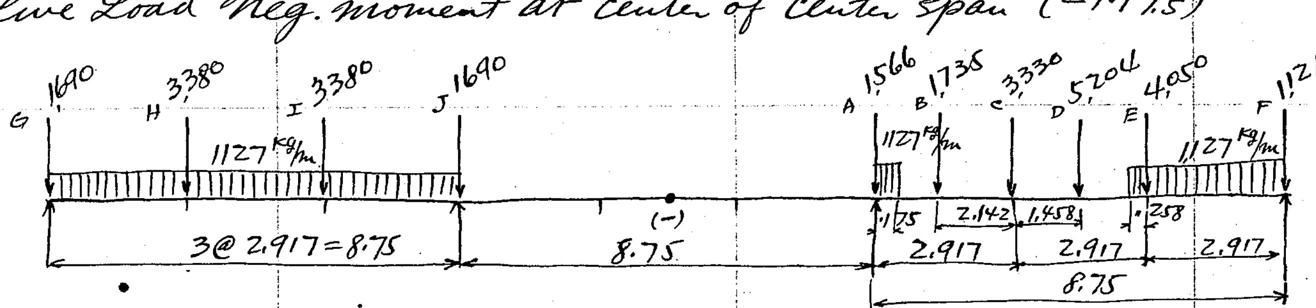
Loads	K	$M_{1.5}$ Unit load	$M_{1.5}\frac{10}{\ell}$
1566	1.00	0	0
1735	1.09	.20	+ 347
3330	1.33	.97	+ 3230
5204	1.50	1.75	+ 9,120
4050	1.67	.99	+ 4,010
1121	2.00	0	0
197	1.01	.03	+ 6
3580	1.82	.46	+ 1648
			+ 18,361

Max Live Load pos moment

$+M_{1.5} = +18361 \times \frac{8.75}{10} = +16,060$ kgm



Max. Live Load Neg. moment at center of center span ($-M_{1.5}$)

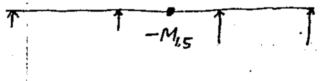


Loads		K.	(−) $M_{1.5}$ Unit load	(−) $M_{1.5}\frac{10}{\ell}$
G	1690	0	0	0
H	3380	.33	.29	− 981
I	3380	.67	.37	− 1250
J	1690	1.00	0	0
A	1566	2.00	0	0
B	1735	2.09	.16	− 277
C	3330	2.33	.37	− 1231
D	5204	2.50	.375	− 1954
E	4050	2.67	.29	− 1176
F	1121	3.00	0	0
				− 6,869

Unif. loads	K	(−) $M_{1.5}$ Unit load	(−) $M_{1.5}\frac{10}{\ell}$
3288	.17	.165	− 542
"	.50	.375	− 1,233
"	.83	.26	− 855
197	2.01	.02	− 4
3580	2.82	.175	− 626
			− 3260

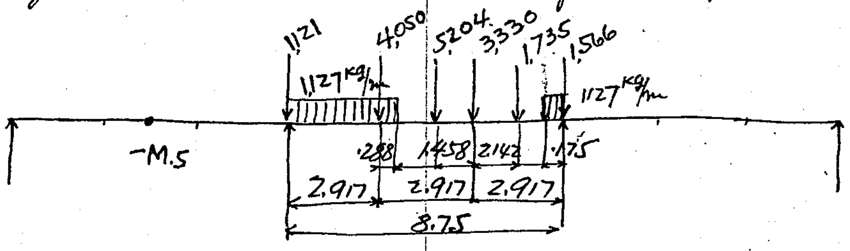
Max. Neg. L.L. moment at center of center span

$(-) M_{1.5} = -(6869 + 3260) \times \frac{8.75}{10} = -8865$ kgm



Katsura-Bashi for Kyotofu

L.L. Negative moment at center of side span ($-M_{0.5}$)



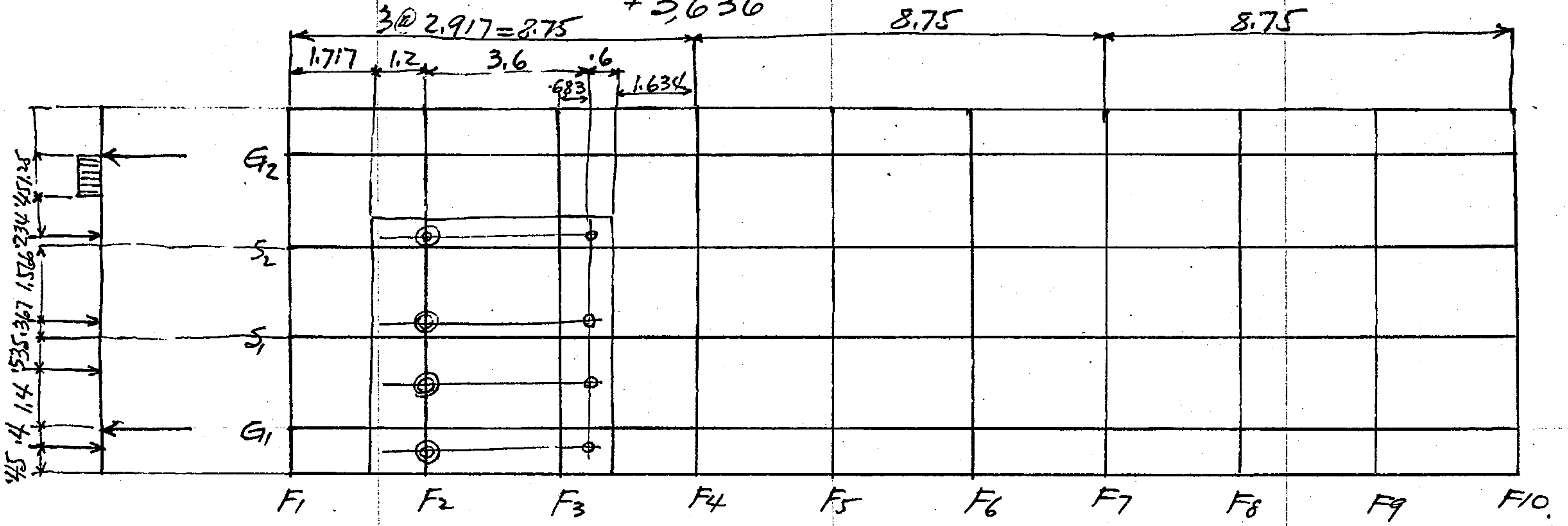
Loads	K	M.5 unit load	$M.5\ \frac{10}{l}$
1735	1.91	.065	− 113
3330	1.67	.270	− 900
5204	1.50	.375	−1,952
4,050	1.33	.392	−1,588
197	1.99	.010	− 2
3,580	1.18	.300	−1,074
			−5,629

$$-M_{0.5} = -5629 \times \frac{8.75}{10} = -4,925 \text{ kgm.}$$

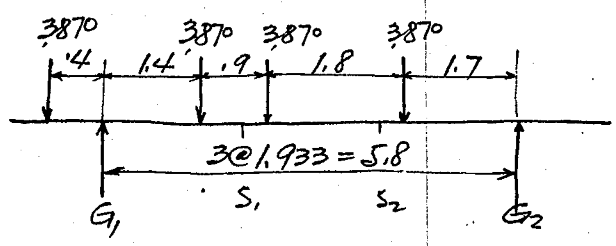
Max. Live Load Pos. moment at 2nd support $+M_2$.
Apply the same loadings on the 3rd span instead of 2nd span.

Loads	K.	$+M_2$ unit load	$+M_2\ \frac{10}{l}$
1,735	2.91	.065	+ 113
3,330	2.67	.173	+ 591
5,204	2.50	.25	+1302
4,050	2.33	.245	+ 992
197	2.99	.01	+ 2
3,580	2.18	.175	+ 636
			+3,636

$$+M_2 = +3636 \times \frac{8.75}{10} = +3180 \text{ Kgm.}$$



Wheel Load concentration on main girders.



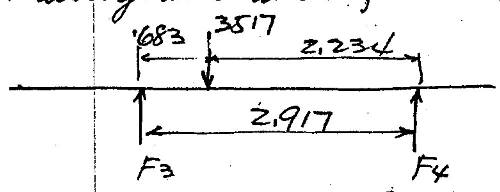
$3870 \times 1.7 = 6,580$
$\times 3.5 = 13,550$
$\times 4.4 = 17,030$
$\times 6.2 = 24,000$
$61,160 \div 5.8 = 10,550$ kg

10,550
3,517

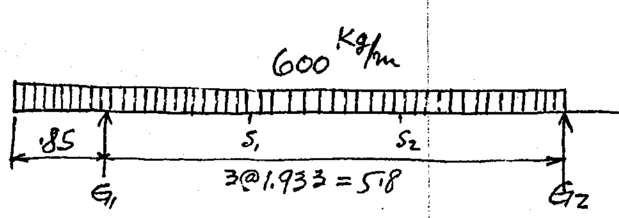
4,930 for Rear wheel
1,643 " front "

front wheel $= \frac{10550}{3} = 3,517$ kg.

Floor beam concentration on main girder due to front wheel.

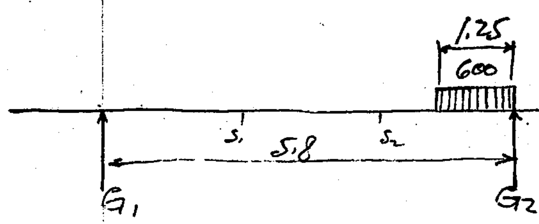


Uniform Load on Main girder G1.
Uniform load on before and behind of motortrucks.



2,288 1,707

$$\frac{600 \times 6.65 \times 6.65}{2 \times 5.8} = 2,288 \text{ kg/m.}$$



81. 669.

$$\frac{600 \times 1.25 \times 1.25}{2 \times 5.8} = 81.$$

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CALCULATIONS FOR

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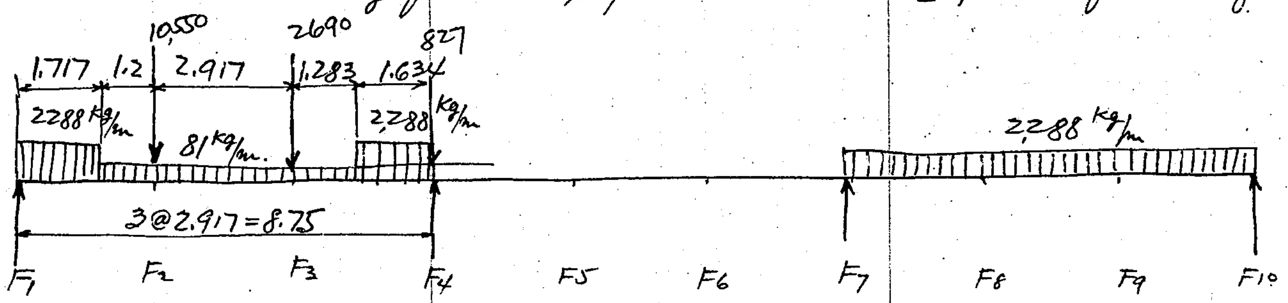
Katsura Bashi for Kyotofu.

Combined Loading for Max. pos. moment at $\frac{1}{3}$ point from left end. (+M0.33)

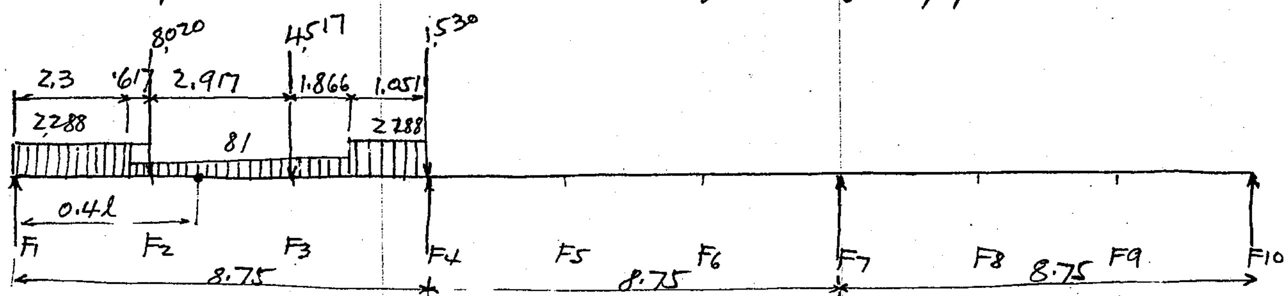


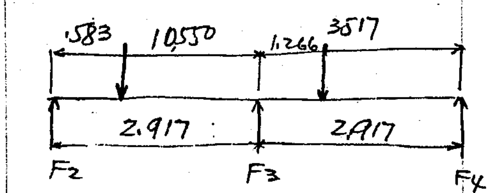
Loads	K	+M.33 unit load	M.33 $\frac{10}{l}$	Unif loads	K	M.33 unit load	M.33 × $\frac{10}{l}$
10,550	.33	1.95	+20,540	2288×1.717=3926	.10	.59	+ 2332
2,690	.67	.78	+2,098	81×1.2 = 97	.26	1.54	149
827	1.00	0	0	81×2.917 = 236	.50	1.33	− 314
			+ 22,638	81×1.283 = 104	.74	.58	60
			4492	2288×1.634=3,737	.91	.16	598
			+ 27,130	2288×2.917 = 6,675	2.17	.05	334
				" = 6675	2.50	.07	467
				" = 6675	2.83	.035	238
							+4492

Live Load max pos. m

$M.33 = +27130 \times \frac{8.75}{10} = 23,720$ Kgm.

Live Load pos. max moment at 0.4 l from left support.





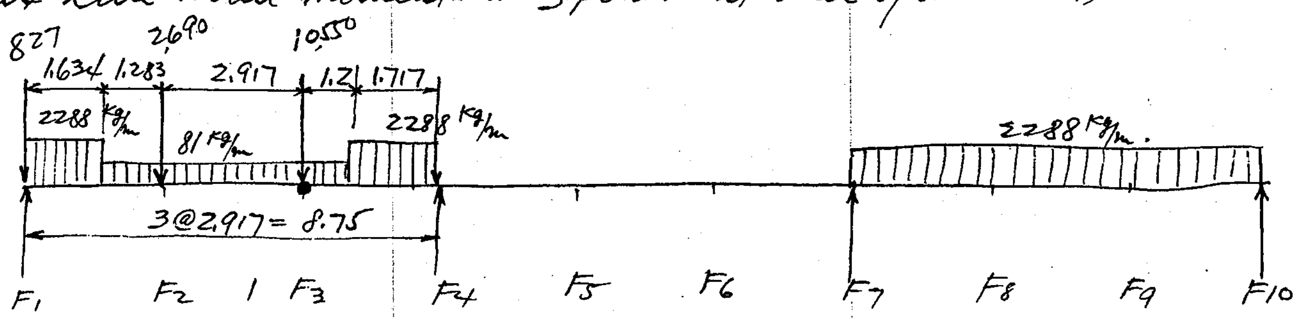
F2	F3	F4
8020	2530	1530
	1987	
8020	4517	1530

Loads	K	+M.4 unit load	+M.4 $\frac{10}{l}$	Unif load.	K.	+M.4 unit load.	+M.4 $\frac{10}{l}$
8,020	.33	1.68	13,460	2288×2.3 = 5260	.13	.65	3,420
4,517	.67	.92	4,150	81×.617 = 50	.30	1.53	77
1,530	1.00	0	0	81×2.917 = 236	.50	1.60	378
			17,610	81×1.866 = 153	.77	.59	90
			5,819	2288×1.051 = 2,405	.93	.16	385
			+ 23,429	2288×2.917 = 6,675	2.17	.07	467
				" = 6,675	2.50	.10	668
				" = 6675	2.83	.05	334
							+5,819

L.L. max m. $+M.4 = 23429 \times \frac{8.75}{10}$

$= +20,500$ Kgm.

Max Live Load Moment at $\frac{2}{3}$ point at End span (+M.67)



Loads	K.	+M.67 Unit load	+M.67 $\frac{10}{l}$	Unif. loads	K,	+M.67 unit load	+M.67 · $\frac{10}{l}$
827	.00	0	0	2288×1.634 = 3737	.09	.15	+ 560
2690	.33	.58	+1,560	81×1.283 = 104	.26	.43	45
10,550	.67	1.62	+17090	81×2.917 = 236	.50	1.00	236
			+18,650	81×1.2 = 97	.74	1.14	111
			+ 4,734	2288×1.717 = 3926	.90	.36	1,413
			+23,384	"×2.917 = 6,675	2.17	.11	734
				" = 6,675	2.50	.17	1,134
				" = 6,675	2.83	.075	501
							+4,734

Live Load pos. m. $+M.67 = 23384 \times \frac{8.75}{10}$

$= +20,460$ Kgm.

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CALCULATIONS FOR

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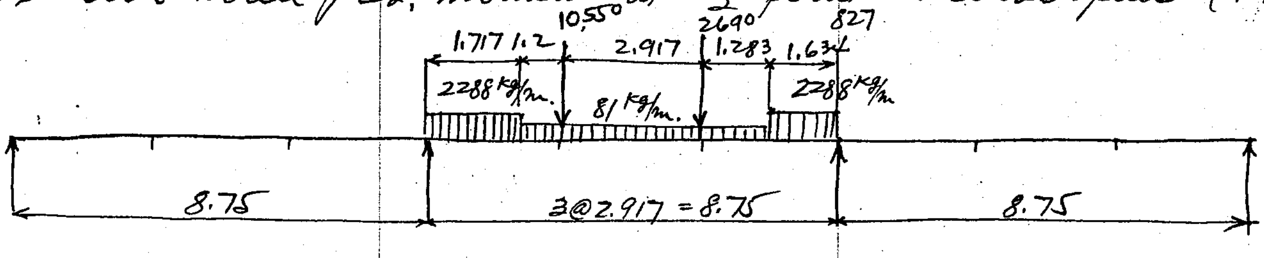
Katsura Bashi for Kyotofu.

Max Live Load pos. moment at $\frac{1}{3}$ point in center span ($+M_{1.33}$)



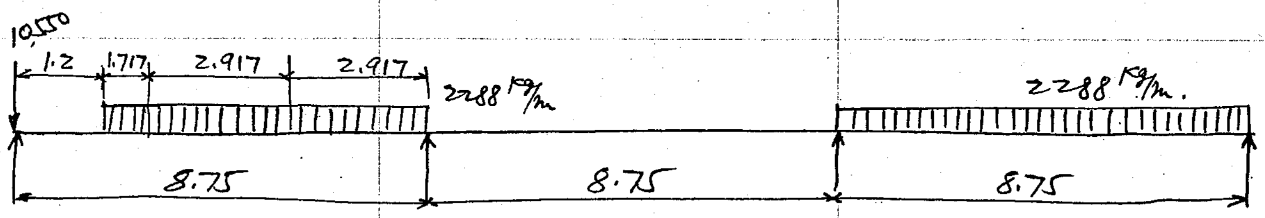
Loads	K	$+M_{1.33}$ unit load	$+M_{1.33}\frac{10}{l}$	Unif. load	K.	$+M_{1.33}$ unit load	$M_{1.33}\frac{10}{l}$
10550	1.33	1.53	+16,140	3926	1.1	·355	1,394
2690	1.67	'48	1,291	97	1.26	1.12	109
827	2.00	0	0	236	1.50	·92	217
			+17,431	104	1.74	·35	36
			+ 2,167	3737	1.91	·11	411
			+19,598				+2,167

Live Load max pos. moment $M_{1.33} = 19{,}598 \times \frac{8.75}{10} = +17{,}148$ kgm

Live Load Shears and Reactions

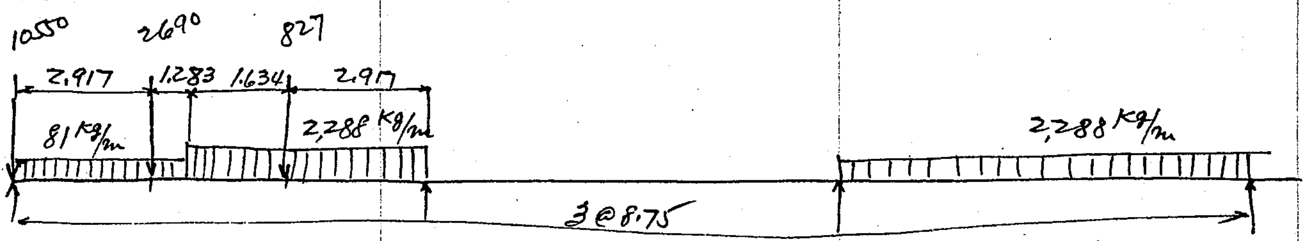
Max Reaction or End shear at end support R_1.

Case 1.



Loads	K	R_1 unit load	R_1	Unif. load	K.	R_1 unit load	R_1
10550	0	1.00	10550 kg.	2288×1.717 = 3,926	·235	·706	2,772
			6464	2288×2.917 = 6,675	·50	·400	2,670
		$R_1 =$	+17,014 kg.	" = 6,675	·83	·100	668
				" = 6,675	2.17	·017	114
				" = 6,675	2.50	·025	167
				" = 6,675	2.83	·011	73
							6,464

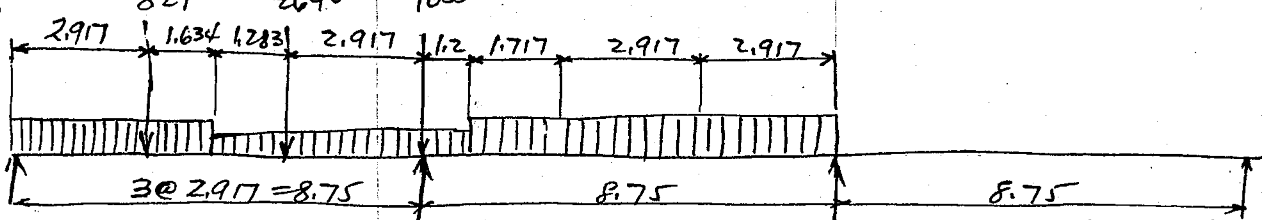
Case 2.



Loads	K	R_1 unit load	R_1	unif. load	K	R_1 unit load	R_1
10550	·000	1.00	+ 10,550	81× 2.917 = 236	·17	·791	+ 187
2690	·33	·588	1,581	81 × 1.283 = 104	·41	·500	52
827	·67	·236	195	2288 × 1.643 = 3737	·57	·329	1,229
			+12,326	2288 × 2.917 = 6,675	·83	·100	668
			2,490	" = 6,675	2.17	·017	114
		$R_1 =$	+14,816 kg.	" = 6,675	2.50	·025	167
				" = 6,675	2.83	·011	73
							+2,490

Live Load Max. Reaction at 2nd support R_2 or R_3.

Case 1.



Loads	K	R_2 unit load	R_2	unif. load	K	R_2 unit load	R_2
827	·33	·510	+422	6675	·17	·263	+1,735
2,690	·67	·886	+2383	3737	·43	·643	+2402
10,550	1.00	1.000	+10,550	104	·59	·820	+ 85
			+ 13,355	236	·83	·981	+232
			+ 12,897	97	1.07	·975	+ 95
		$R_2 (= R_3) =$	+ 26,252 kg	3926	1.235	·865	+3394
				6675	1.5	·575	+ 3840
				6675	1.83	·164	+ 1094
							+12,897

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

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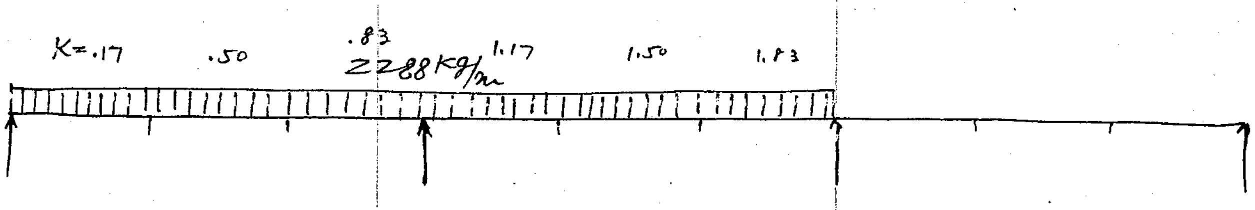
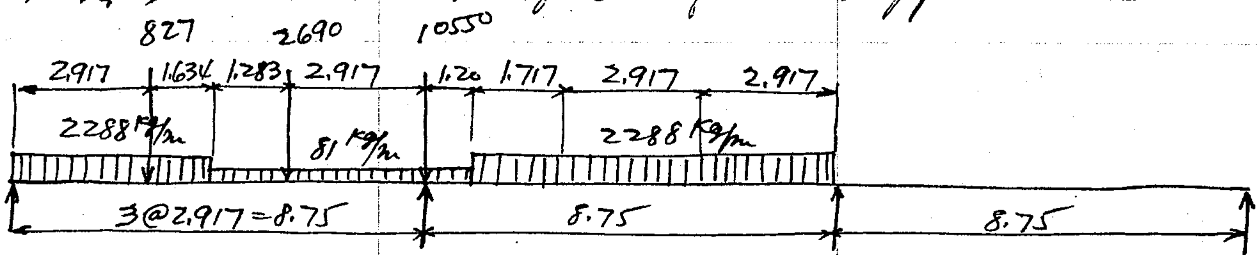
Katsura Bashi for Kyotofu.

Case 2.



$6675 \times (.263 + .725 + .981 + .918 + .575 + .164) = 24,200 kg$

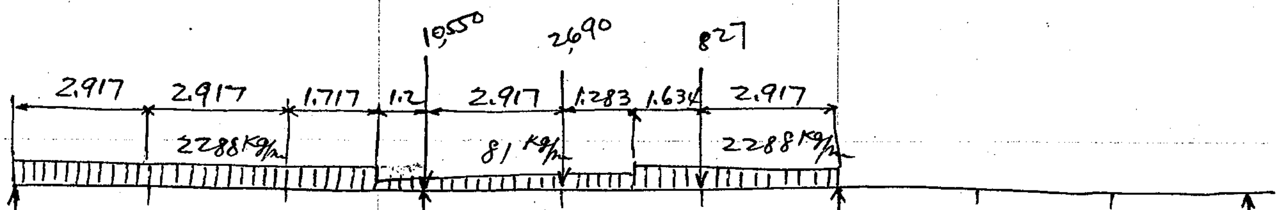
Max L.L. End shear at left side of 2nd support $-R_2'$ or $+R_3''$



Loads	K	R_2' unit load	R_2'
827	.33	.503	− 416
2690	.67	.764	− 2055
10,550	1.00	1.000	− 10550
			− 13,021
			− 4,374
		$-R_2'$ or $=+R_3''$ =	− 17,395 kg

unif load	K.	R_2' unit load	R_2'
6675	.17	.176	− 1,175
3737	.43	.527	− 1,970
104	.59	.690	− 72
236	.83	.900	− 212
97	1.07	.026	− 3
3926	1.235	.068	− 267
6675	1.50	.075	− 501
6675	1.83	.026	− 174
			− 4,374

Live Load Max End Shears at Right side of 2nd support. $+R_2''$ or $=-R_3'$



Loads	K	R_2'' unit load	R_2''
10550	1.000	1.000	+ 10,550
2690	1.33	.691	1,860
827	1.67	.309	256
			+ 12,666
			+ 4,352
			+ 17,018 kg

Unif load	K	R_2' unit load	R_2''
6675	.17	.043	287
6675	.50	.125	834
3926	.764	.103	404
97	.93	.04	4
236	1.17	.909	210
104	1.41	.605	63
3737	1.57	.423	1,580
6675	1.83	.137	970
			+ 4,352

Summary of moments and shears for main girder.

	Moments										Reactions		End Shears.		
	End span					Center span			At Supports		End sup.	Int sup.	End sup	Int. supports	
	$+M_{K=\frac{1}{3}}$	$+M_{K=.4}$	$+M_{K=\frac{1}{2}}$	$-M_{K=\frac{1}{2}}$	$+M_{K=\frac{2}{3}}$	$+M_{K=1\frac{1}{3}}$	$+M_{K=1\frac{1}{2}}$	$-M_{K=1\frac{1}{2}}$	$-M_2=M_3$	$+M_2$	$+R_L$	$+R_2$	$+R_1$ Right	$+R_2$ left	$+R_2$ right
D.L. unif.	+ 16160	+15660	+15.575		+ 9,250	+2,150	+5,200	+5,200	-20750		+ 9,500	+ 26110	+9500	-14,250	+11860
D.L. concent.	+ 12,390	+10560	+9,360		+7,140	+2,740	+3,047	+3047	-12,440		+ 7,087	+17,413	+7089	-12,135	+10701
Total D.L.	+28,550	+26220	+24,935	+24,935	+16,390	+4,890	+8,247	+8247	-33,190	-33,190	+16,589	+43,523	+16,589	-26,385	+22,561
L.L.	+23,720	+20,500	+20,900	-4,925	+20,460	+17,145	+16,060	-8865	-21,840	+ 3180	+17,014	+26252	+17014	-17,395	+17,018
Total	+52,270	+46,720	+45,835	+20,010	+36,850	+22,035	+24307	- 618	-55,030	+30,010	+33,603	+69,775	+33,603	-43,780	+39,579

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Katsura Bashi for Kyotofu.

Equivalent Uniform Loads

Dead Load Uniform load 2711 kg/m

Floor Beam concentration = 5350 kg

Conversing to unif. load $5350 \div 2.917 = 1834$

2711

Total dead unif. load on main girder = 4545 kg/m ---- Wd.

$Wd \cdot l^2 = 4545 \times 8.75^2 = 347,500$

Live Load.

Neg. moment M_2. = 21,840 kgm

for Equiv. unif. load $M = 0.11667 Pl^2$

Equiv. unif. load $P_1 = \frac{21840}{.11667 \times 8.75^2} = 2,445$ $P_1 l^2 = 187,300$

Neg. moment $M_{1.5} = -8865$ kgm.

for Equiv. unif load $M = 0.05 Pl^2$

Equiv. unif. load $P_{-1.5} = \frac{8865}{.05 \times 8.75^2} = 2,315$ $P_{-1.5} l^2 = 177,200.$

Neg. moment $M_{.5} = -4925$

for equiv. unif load $M = .025 Pl^2$

Equiv. unif load $P_{-.5} = \frac{-4925}{.025 \times 8.75^2} = 2,574$ $P_{-.5} l^2 = 197,000$

Positive moment $M_{\frac{1}{3}} = +23,720$ kgm

for equiv. unif. load $M = 0.0942 Pl^2$

Equiv unif load $P_{+\frac{1}{3}} = \frac{23720}{.0942 \times 8.75^2} = 3,288$ $P_{+\frac{1}{3}} l^2 = 251,700$

Positive moment $M_{\frac{1}{2}} = +20900$ kgm

for equiv. unif load $M = 0.10 Pl^2$

equiv. unif load. $P_{+\frac{1}{2}} = \frac{20900}{.1 \times 8.75^2} = 2,730$ $P_{+\frac{1}{2}} l^2 = 209,000$

Positive moment $+M_{.4} = +20,500$

for equiv. unif. load = $0.10 Pl^2$

equiv. unif. load = $P_{+.4} = \frac{20500}{.10 \times 8.75^2} = 2675$ $P_{+.4} l^2 = 205,000$

Pos. moment $M_{\frac{2}{3}} = +20,460$ kgm

for equiv. unif. load $M = .0783 Pl^2$

Equiv. unif. load $P_{+\frac{2}{3}} = \frac{20460}{.0783 \times 8.75^2} = 3412$ $P_{+\frac{2}{3}} l^2 = 261,300$

Pos. moment $+M_2 = +3180$

for equiv. unif load $M = 0.01667 Pl^2$

equiv. unif load $P_{+1} = \frac{3180}{.01667 \times 8.75^2} = 2,491$ $P_{+1} l^2 = 190,800$

Positive moment $M_{1\frac{1}{3}} = +17,145$ kgm

for equiv. unif. load $M = .0616 Pl^2$

Equiv. unif load $P_{+1\frac{1}{3}} = \frac{17145}{.0616 \times 8.75^2} = 3632$ $P_{+1\frac{1}{3}} l^2 = 278,100$

Pos. moment $M_{1\frac{1}{2}} = +16,060$

for equiv. unif. load $M = 0.075 Pl^2$

equiv. unif. load $P_{+1\frac{1}{2}} = \frac{16060}{.075 \times 8.75^2} = 2,795$ $P_{+1\frac{1}{2}} l^2 = 214,000$

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CALCULATIONS FOR

Katsura Bashi for Kyotofu.

Moments at several points through main girders (calcurated by equivalent unif. load)

		Dead Load moments			Live Load neg. moment			Live Load Pos. moments		
	x/l	Coeff. of W_dl^2	By equiv. load	By actual load	Coef. of Pl^2	By equiv. load	By actual load	Coef. of Pl^2	By equiv. load	By actual load
End span.	.10	+ .035 W_dl^2	+ 12,170		.005 $P_{.5}l^2$	− 985		.040 $P_{+\frac{1}{3}}l^2$	+ 10,060	
	.20	+ .060	+ 20,850		.010 "	− 1,970		.070 "	+ 17,600	
	.30	+ .075	+ 25,070		.015 "	− 2,955		.090 "	+ 22,620	
	.33	+ .0778	+ 27,030	+ 28,550	.0167 "	− 3,290		.0942 "	+ 23,720	+ 23,720
	.40	+ .080	+ 27,800		.020 "	− 3,940		.100 $P_{+\frac{1}{2}}l^2$	+ 20,900	+ 20,500
	.50	+ .075	+ 25,070	+ 24,935	.025 $P_{.5}l^2$	− 4,925	− 4,925	.100 "	+ 20,900	+ 20,900
	.60	+ .060	+ 20,850		.030 "	− 5,910		.090 "	+ 18,840	
	.67	+ .0438	+ 15,220	+ 16,390	.0333 "	− 6,560		.0783 $P_{+\frac{2}{3}}l^2$	+ 20,460	+ 20,460
	.70	+ .035	+ 12,170		.035 "	− 6,890		.070 "	+ 18,300	
	.80	+ .000	+ 0		.04022 "	− 7,920		.04022 "	+ 10,500	
	.85	− .02125	− 7,380		.04898 $P_{1}l^2$	− 9,175		.02773 $P_{+1}l^2$	+ 5,300	
	.90	− .045	− 15,640		.06542 "	− 12,260		.02042 "	+ 3,900	
	.95	− .07125	− 24,750		.08831 "	− 16,540		.01706 "	+ 3,260	
	1.00	− .100	− 34,750	− 33,190	.11667 $P_{1}l^2$	− 21,840	− 21,840	.01667 "	+ 3,180	+ 3,180
Center span	1.05	− .07625	− 26,500		.09033 "	− 16,910		.01408 "	+ 2,690	
	1.10	− .0550	− 19,110		.07014 "	− 13,130		.01514 $P_{+1\frac{1}{3}}l^2$	+ 4,210	
	1.15	− .03625	− 12,600		.05678 "	− 10630		.02053 "	+ 5,700	
	1.20	− .0200	− 6,950		.0500 $P_{1.5}l^2$	− 8,865		.0300 "	+ 8,350	
	1.2764	− .0000	0		.0500 "	− 8,865		.0500 "	+ 13,900	
	1.30	+ .005	+ 1,740		.0500 "	− 8,865		.0550 "	+ 15,300	
	1.33	+ .0118	+ 4200	+ 4,890	.0500 "	− 8,865		.0616 "	+ 17,145	+ 17,145
	1.40	+ .020	+ 6,950		.0500 "	− 8,865		.0700 $P_{+1\frac{1}{2}}l^2$	+ 14,980	
	1.50	+ .025	+ 8,690	+ 8,247	.0500 "	− 8,865	− 8,865	.0750 "	+ 16060	+ 16060

$W_d = 4,545$ kg/m
$W_dl^2 = 347,500$ kgm

$P_{.5} = 4925$ $P_{.5}l^2 = 197,000$
$P_{1} = 2445$ $P_{1}l^2 = 187,300$
$P_{1.5} = 2315$ $P_{1.5}l^2 = 177,200$

$P_{+\frac{1}{3}} = 3,288$ $P_{+\frac{1}{3}}l^2 = 251,700$
$P_{+.4} = 2,675$ $P_{+\frac{1}{2}}l^2 = 209,000$ ← $P_{+.4}l^2 = 205,000$
$P_{+\frac{1}{2}} = 2730$ $P_{+\frac{2}{3}}l^2 = 261,300$
$P_{+\frac{2}{3}} = 3412$ $P_{+1}l^2 = 190,800$
$P_{+1} = 2,491$ $P_{+1\frac{1}{3}}l^2 = 278,100$
$P_{+1\frac{1}{3}} = 3,632$ $P_{+1\frac{1}{2}}l^2 = 214,000$
$P_{+1\frac{1}{2}} = 2,795$

Summary of Dead & Live Load Moments

	x/l	Negative M.	Positive M.
End span	.10	+ 11,185 kgm	+ 22230
	.20	+ 18,800	+ 38,450
	.30	+ 22,115	+ 47,690
	.33	+ 23,740	+ 52,270
	.40	+ 23,860	+ 48,700
	.50	+ 20,010	+ 45,970
	.60	+ 14,940	+ 39,690
	.67	+ 8,660	+ 36,850
	.70	+ 5,280	+ 30,470
	.80	− 7,920	+ 10,500
	.85	− 16,555	− 2,080
	.90	− 27,900	− 11,740
	.95	− 41,290	− 21,490
	1.00	− 56,590	− 30,100
Center span.	1.05	− 43,410	− 23,810
	1.10	− 32,240	− 14,900
	1.15	− 23,230	− 6,900
	1.20	− 15,815	+ 1,400
	1.2764	− 8,865	+ 13,900
	1.30	− 7,125	+ 17,040
	1.33	− 4,665	+ 22035
	1.40	− 1,915	+ 21,930
	1.50	− 618	+ 24,750

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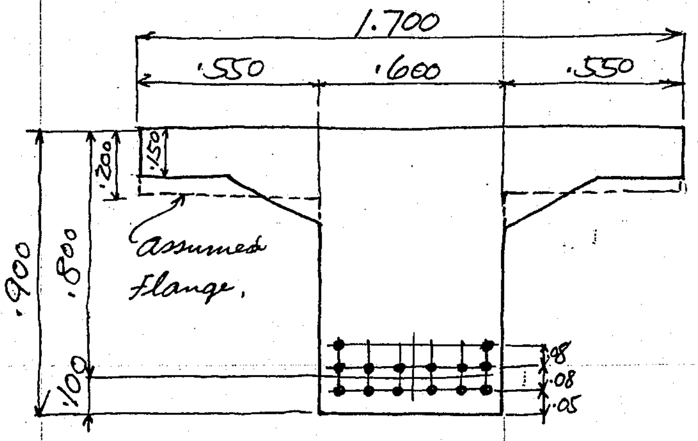
CALCULATIONS FOR

Katsura Bashi for Kyotofu.

Section of Main girder.



at $\frac{1}{3}l$ from end support in End span.

$+M_{\frac{1}{3}} = 52,270^{kgm}$.

Steel area req'd $= \dfrac{52,270 \times 100}{1200 \times \frac{7}{8} \times 80} = 62.2^{\square cm}$

use $14 - 1''^{\phi}\,(2.54^{cm\phi}) = 70.9^{\square cm}$.

Steel ratio $p = \dfrac{70.9}{170 \times 80} = 0.0052$

$t/d = .20/.80 = 0.25$

$K = .332,\ j' = .90$

$f_s = \dfrac{52,270 \times 100}{70.9 \times 0.90 \times 80} = 1,024\ ^{kg}/_{cm^2}$ OK

$f_c = \dfrac{f_s K}{n(1-K)} = \dfrac{1024 \times .332}{15 \times .668} = 340$ " OK

At $\frac{1}{2}l$ from end support in end span

$+M_{\frac{1}{2}} = +45,970^{kgm}$

Steel area req'd $= \dfrac{45970 \times 100}{1200 \times \frac{7}{8} \times 80} = 54.9^{\square cm}$

use $12 - 1''^{\phi}\,(2.54^{cm\phi}) = 60.8\ cm^2$.

at $\frac{2}{3}l$

$+M_{\frac{2}{3}} = +36,850^{kgm}$

Steel area req'd $= \dfrac{36850 \times 100}{1200 \times \frac{7}{8} \times 80} = 43.9^{\square cm}$

use $10 - 1''^{\phi} = 50.65\ cm^2$

at $\frac{1}{3}l$ from end support in End span

$+M_{1\frac{1}{3}} = +22035^{kgm}$

Steel area req'd $= \dfrac{22035 \times 100}{1200 \times \frac{7}{8} \times 80} = 26.2\ cm^2$

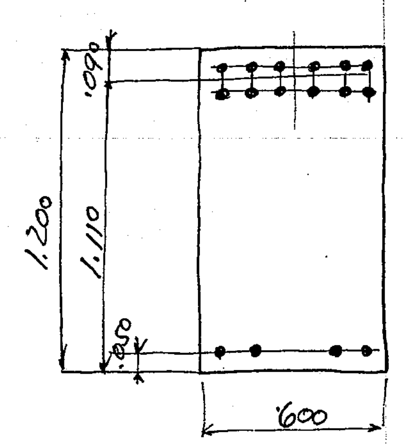
use $6 - 1''^{\phi} = 30.4^{\square cm}$

At center of center span

$+M_{1\frac{1}{2}} = +24,750^{kgm}$

Steel area req'd $= \dfrac{24750 \times 100}{1200 \times \frac{7}{8} \times 8} = 29.5^{\square cm}$

use $8 - 1''^{\phi} = 40.52^{\square cm}$



Negative Reinforcements at Pier (2nd support)

$-M_{1.0} = -56,590^{kgm}$

Steel area req'd $= \dfrac{56590 \times 100}{1200 \times \frac{7}{8} \times 111} = 48.6^{\square cm}$

use $12 - 1''^{\phi} = 60.8^{\square cm}$

Steel ratio $p = \dfrac{60.8}{60 \times 111} = 0.00913$ $\quad p' = \frac{p}{4}$ assumed $= .00228$

$d'/d = .05$ say

By the prepared diagram

$K = .38,\ j' = .884$

$f_s = \dfrac{56590 \times 100}{60.8 \times .884 \times 111} = 949\ ^{kg}/_{cm^2}$ OK

$f_c = \dfrac{949 \times .38}{15 \times .62} = 38.8\ ^{kg}/_{cm^2}$ OK

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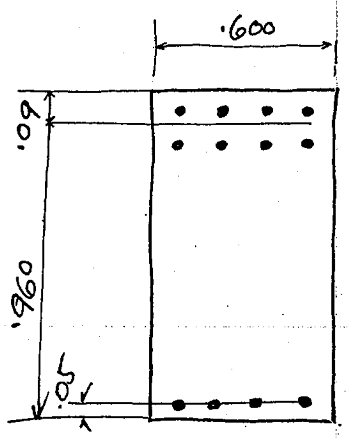
Katsura Bashi for Kyotofu.
Section of main girders continued.



At 0.1l from 2nd support in center span.

$-M_{1.1} = -32,240^{kgm}$

steel req'd $= \frac{32240\times100}{1200\times\frac{7}{8}\times96} = 32.0^{\square cm}$

use 8-1"ϕ $= 40.52^{\square cm}$

$p = \frac{40.52}{60\times96} = 0.00704 \quad p' = \frac{p}{4} = 0.00176 \quad \frac{d'}{d} = .105$

$K = .355, \; j = .892$

$f_s = \frac{32240\times100}{40.52\times.892\times96} = 929 \; {}^{kg}/_{cm^2}$ OK

$f_c = \frac{929\times.355}{15\times.645} = 32.7$ " OK

At 0.85l in end span & 0.2l from 2nd support in center span

$-M_{.85} = -16,555^{kgm}$

steel area req'd $= \frac{16555\times100}{1200\times\frac{7}{8}\times95} = 16.6 \; cm^2$

use 4-1"ϕ $= 20.25 \; cm^2$

Shearing and Bond stresses at various points in the main girder.

End shear at End support $V = 33603 kg$ (R_{1R})

Unit shear $v = \frac{33603}{60\times\frac{7}{8}\times115} = 5.56 \; {}^{kg}/_{cm^2}$

use $\frac{1}{2}$"ϕ (1.27cmϕ) u-stirrups $A_s = 2\times1.267 = 2.534 \; cm^2$

min. stirrup spacing $s = \frac{3}{2}\cdot\frac{A_s f_s j d}{V} = \frac{3}{2}\cdot\frac{2.534\times1200\times\frac{7}{8}\times115}{33603}$ $= 13.6 \; cm.$

$\frac{2}{3}$ of total shear be taken by stirrups assumed.

For 5/8"ϕ u-stirrups $A_s = 2\times1.979 = 3.958^{cm^2}$

min stirrup spacing $s = \frac{3}{2}\cdot\frac{3.958\times1200\times\frac{7}{8}\times115}{33603}$

$= \underline{21.3} \; cm$

bond stress $= \frac{33603}{7.98\times\frac{7}{8}\times115} = 41.85 \; {}^{kg}/_{cm^2}$ for 1-1"ϕ bar

no. of bars req'd for bond stress $= \frac{41.85}{9} = 4.65$ bars

use 6 deformed bars.

Shear at Left side of 2nd Support $V_{2L} = -43780 = R_{2L}$

unit shear $v = \frac{43,780}{60\times\frac{7}{8}\times115} = 7.26 \; {}^{kg}/_{cm^2}$

use 5/8"ϕ u-stirrups $s = \frac{3}{2}\cdot\frac{3.958\times1200\times\frac{7}{8}\times115}{43780} = 16.4 \; cm$

bond stress $= \frac{43780}{7.98\times\frac{7}{8}\times115} = 54.5 \; {}^{kg}/_{cm^2}$ for 1-1"ϕ bar

for 12-1"ϕ bars $u = \frac{54.5}{12} = 4.55 \; {}^{kg}/_{cm^2}$ ok

Shear at Right side of 2nd support.

$V_{2R} = 39579$

unit shear $v = \frac{-39579}{60\times\frac{7}{8}\times115} = 6.56 \; {}^{kg}/_{cm^2}$

use 5/8"ϕ u-stirrups $s = \frac{3}{2}\cdot\frac{3.958\times1200\times\frac{7}{8}\times115}{39579} = \underline{18.1} \; cm$

bond stress $= \frac{39579}{7.98\times\frac{7}{8}\times115} = 49.3 \; {}^{kg}/_{cm^2}$

for 12-1"ϕ bars $u = \frac{49.3}{12} = 4.11 \; {}^{kg}/_{cm^2}$. OK.

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CALCULATIONS FOR

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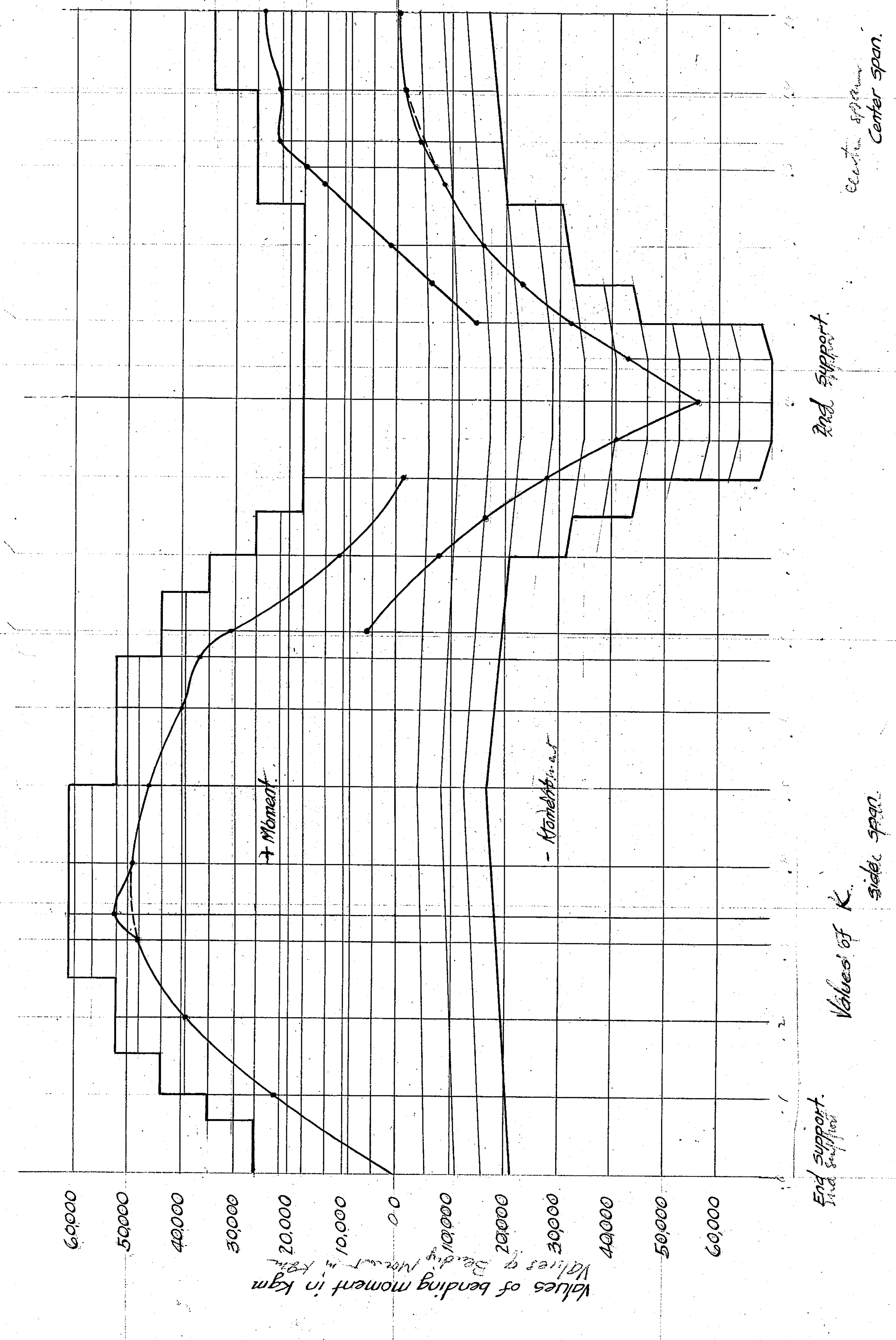

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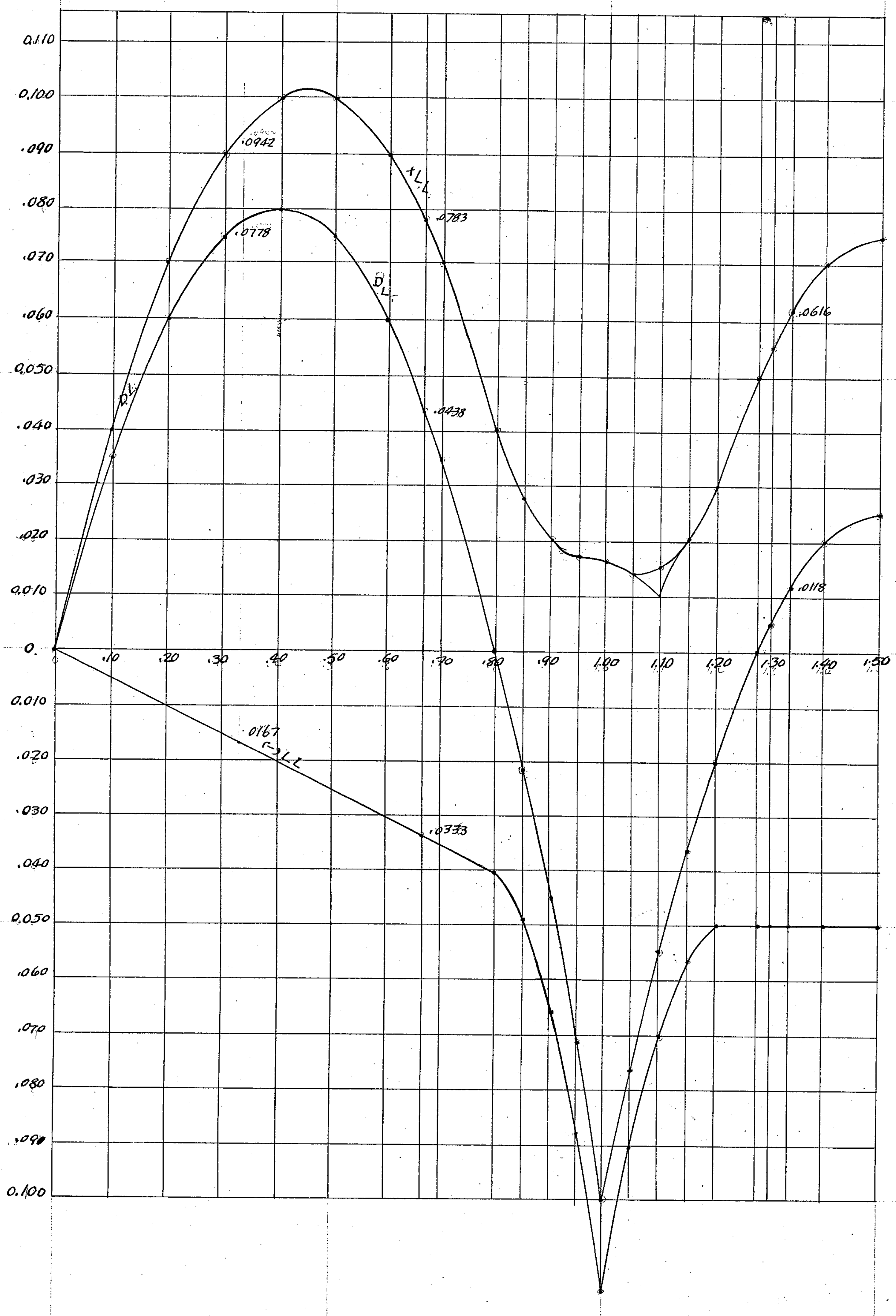
CALCULATIONS FOR



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Katsura Bashi for Kyotofu
Design of Pier for Concrete Girder Spans.
Center of gravity of floor
Taking moments about a plane below 1 meter from Crown of Roadway

Load	weight per span	Lever arm	moment
Handrail	2 @ 255 × 8.75 = 4,460 kg	1.48 m	6,000 kgm
Coping	2 @ 272 × 8.75 = 4,750	.93	4,420
Light pole say 2 @ 130	= 260	3.30	860
Light pedestal	2 × .5 @ 2600 = 2,600	1.50	3,900
Pavement	7.5 m × 8.75 @ 140 = 9,180	0.93	8,540
Slab	7.5 × 8.75 @ 370 = 24,280	0.82	19,900
Stringer	2 × 7.4 @ 250 = 3,700	0.57	2,110
Int. Cross beam	5.2 × 2.5 @ 600 = 7,800	0.42	3,280
End " "	5.2 @ 475 = 2470	0.42	1,040
Main Girder	2 × 8.75 @ 1,476 = 25,800	0.27	6,970
Total weight of floor / span	= 85,300 kg		57,620 kgm ÷ 85,300 kg = 0.676 m

Load on one col. $= 85.300 \times \frac{1.5}{2} = 64,000$ kg Center of gravity of floor 0.324 m below Crown of Roadway

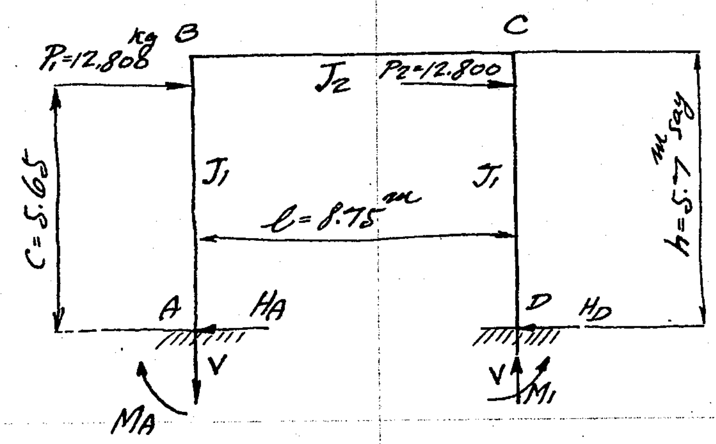
Center of gravity of structure above bottom of Pier Column
Taking moment about bottom of Column

Loads	weight	Lever arm	moment
Floor	64,000 kg	5.88 m	376000 kgm
Column · 8 × · 8.5 @ 2200	= 7,000	2.5	17500
	71,000 kg		393500 kgm ÷ 71,000 = 5.65 m

Center of Gravity 5.65 m above bottom of Col.

Seismic Stability, seismic force parallel to bridge
Acceleration of earth quake = 2.000 mm/sec²
The structure is assumed to act as following fig..

Seismic force on one col. = 64.000 × 0.2 = 12,800 kg.
Moments and reactions for P_1 only
See Kleinlogel's Rahmenformeln P. 94



$$Y = \frac{3PC\delta k}{l(6k+1)}$$

$$H_A = P - H_D$$

$$H_D = \frac{P\delta^2}{2(k+2)}\left[3(k+1) - \delta(2k+1)\right]$$

$$M_A = -\frac{PC\delta}{2}\left[\frac{2}{\delta} - \frac{3+2k-\delta(k+1)}{k+2} - \frac{3k}{6k+1}\right]$$

$$M_D = +\frac{PC\delta}{2}\left[\frac{3+2k-\delta(k+1)}{k+2} - \frac{3k}{6k+1}\right]$$

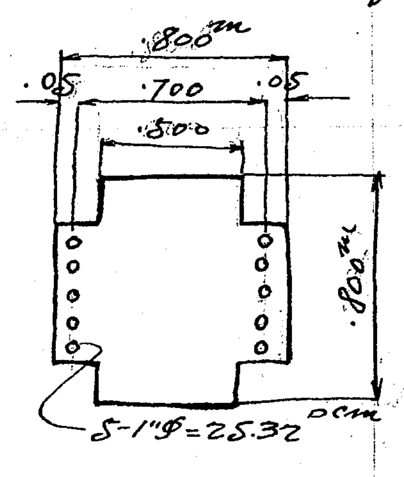
$$M_B = M_A - H_D h + PC$$

$$M_C = M_D - H_D h$$

$$M_P = M_A + H_A C$$

where $k = \frac{J_2}{J_1}\cdot\frac{h}{l}$ $\delta = \frac{c}{h}$

Assumed section of Pier column



Moment of Inertia of Column J_1

Concrete $\frac{\cdot 50 \times \cdot 80^3}{12}$ = $.02135\ (m)^4$

" $\frac{\cdot 30 \times \cdot 50^3}{12}$ = $.003125$

Reinf. $14 \times \frac{25.32}{10000} \times 2 \times \cdot 35^2 = .00890$

$J_1 = 0.03318\ (m)^4$

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CALCULATIONS FOR

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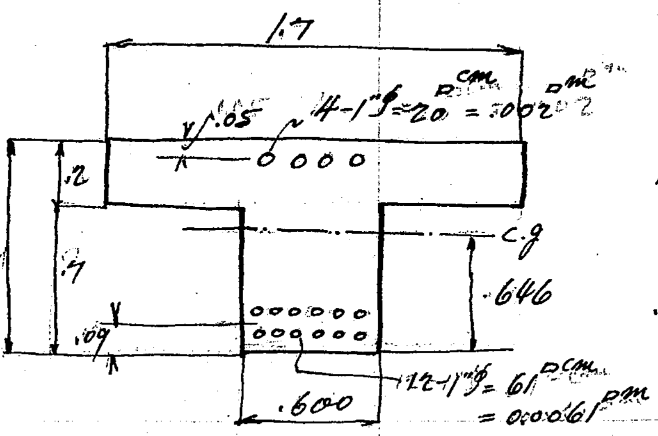


Moment of Inertia of beam
Center of gravity of section

Area	Lever arm	moment about bottom of stem
1.7×.2=.34	.80	.272
.6×.7=.42	.35	.147
.002×14=.028	.85	.024
.0061×14=.081	.09	.073
.869 m²		.516 ÷ .869 = .646

Moment of Inertia J_2

Flange $\frac{1.7\times .2^3}{12}+1.7\times .2\times .154^2 = .00945$

Stem $\frac{.6\times .7^3}{12}+.6\times .7\times .296^2 = .05395$

Top Reinf. $.002\times 14\times .204^2 = .00117$

Bottom " $.0061\times 14\times .556^2 = .0264$

$J_2 = .091\ (m)^4$

$$K=\frac{J_2}{J_1}\cdot\frac{h}{l}=\frac{.091}{.03318}\times\frac{5.7}{8.75}=1.79$$

$$\delta=\frac{c}{h}=\frac{5.65}{5.70}=0.99$$

$$V=\frac{3Pc\delta k}{l(6k+1)}=\frac{3\times 12{,}800\times 5.65\times .99\times 1.79}{8.75(6\times 1.79+1)}=3{,}760\ kg$$

$$H_D=\frac{P\delta^2}{2(k+2)}\left[3(k+1)-\delta(2k+1)\right]=\frac{12{,}800\times 0.99^2}{2\times 3.79}(3\times 2.79-.99\times 4.58)=6360\ kg$$

$$H_A=P-H_D=12{,}800-6360=6440\ kg$$

$$M_A=-\frac{Pc\delta}{2}\left[\frac{2}{\delta}-\frac{3+2k-\delta(k+1)}{k+2}-\frac{3k}{6k+1}\right]=-\frac{12800\times 5.65\times .99}{2}\left(\frac{2}{.99}-\frac{3+3.58-.99\times 2.79}{3.79}-\frac{5.37}{11.74}\right)$$

$$=-35{,}800\times 0.554=-19{,}850\ kgm$$

$$M_D=+\frac{Pc\delta}{2}\left[\frac{3+2k-\delta(k+1)}{k+2}-\frac{3k}{6k+1}\right]=+35.800(1.008-.458)=+19.700\ kgm$$

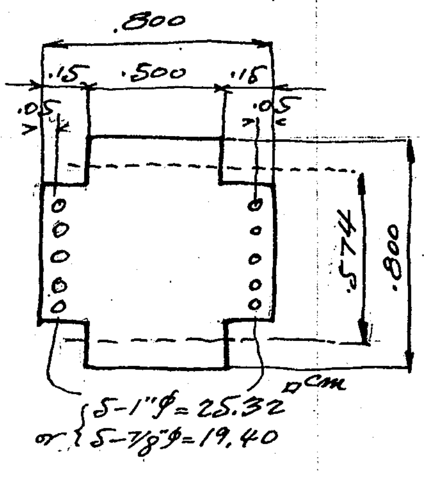
$$M_B=M_A-H_Dh+Pc=-19.850-6{,}360\times 5.7+12800\times 5.65=+16{,}200\ kgm$$

$$M_C=M_D-H_Dh=19.700\ kgm-36.250=-16{,}550\ kgm$$

$$M_P=M_A+H_Ac=-19{,}850+6{,}440\times 5.65=+16{,}500\ kgm$$

Summary of Moment and Shear

	M_A	M_D	M_B	M_C	M_{P_1}	M_{P_2}	Thrust H_A	H_D	Reaction Shears V
Dead Load	neglected	do	(−33,190 on beam only)	(−33,190 on beam only)			neglect	"	64,000
Seismic force P_1	−19,850	+19.700	+16,200	−16,550	+16,500		6,440	6,360	3,760
" " P_2	−19.700	+19,850	+16.550	−16,200		−16,500	6,360	6,440	3,760
	−39,550	+39.550	+32,750	−32,750	+16,500	−16,500	12,800	12,800	71,520 kg
for beam			−440	−65,940					



Equivalent rectangular section, width b'
moment of inertia of the section (Concrete only) = 0.02448 (m)⁴ See P.26

$$\frac{b'h^3}{12}=0.02448$$

$$b'=\frac{0.02448\times 12}{.8^3}=0.574\ m$$

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Katsura Bashi for Kyotofu

Stresses of Column during earth quake

$M_A = M_D' = +39,550$ kgm Shear for column $= H_A = 12,800$ kg

Direct load $V = 71,520$ kg

$\frac{d'}{h} = \frac{.05}{.80} = .062$ Eccentricity $e = \frac{39,550}{71,520} = .553$, $\frac{e}{h} = \frac{.553}{.8} = .692$

$p_o = \frac{25.32 \times 2}{.574 \times .8 \times 10,000} = 0.011$

$k' = .485$ $k'' = .398$

$k = k' - 5(\frac{d'}{h})(k' - k'') = .485 - 5 \times 0.062 \times .087 = .458$

$A_2 = .063$ $B_2 = .0795$ $C_2 = 100 \times 0.011 \times .063 + 0.0795 = .0693 + .0795 = 0.1488$

$f_c = \frac{M}{C_2 b h^2} = \frac{39.550}{.1488 \times .574 \times .8^2} = 723,000$ kg/□m $= 72.3$ kg/□cm ok
(allowable $f_c = 45 \times 1.8 = 81$. kg/□cm)

$f_s = n f_c (\frac{d}{kh} - 1) = 15 \times 72.3 (\frac{.75}{.458 \times .8} - 1) = 1,138$ kg/□cm ok
(allowable $f_s = 1200 \times 1.8 = 2160$ kg/□cm)

try $5 - 7/8"\phi = 19.4$ □cm on both sides

$P_o = .0085$

$k' = .455$ $k'' = .375$

$k = .455 - 5 \times .062 \times .08 = .4302$

$B_2 = .0765$ $A_2 = .0668$ $C_2 = 100 \times .0085 \times .0668 + .0765 = 0.133$

$f_c = \frac{39,550}{0.133 \times .574 \times .8^2} = 810,000$ kg/□m $= 81$ kg/□cm

$f_s = 15 \times 81 (\frac{.75}{.4302 \times .8} - 1) = 1,432$ kg/□cm OK

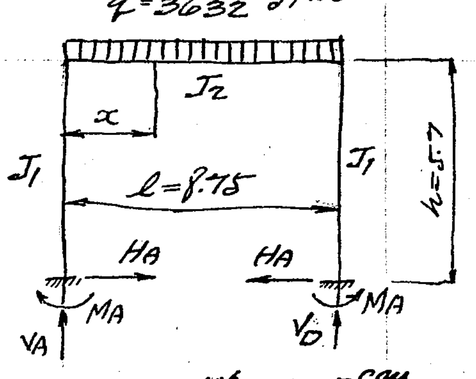
Use Same details through out the length of column

Max. Live Load

Equivalent unit load for Max Pos. live load of beam $= 3632$

moment $= 0.0616\, q l^2$ See P. 20

Probably this load will cause max live load moment on col.



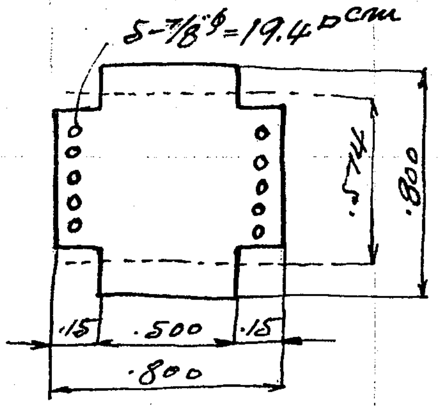
$k = \frac{J_2}{J_1} \times \frac{h}{l} = 1.79$

$V_A = V_D = \frac{ql}{2} = \frac{3632 \times 8.75}{2} = 15900$ kg

$H = \frac{q l^2}{4h(k+2)} = \frac{3632 \times 8.75^2}{4 \times 5.7 \times 3.79} = 3,220$ kg

$M_B = M_C = -\frac{q l^2}{6(k+2)} = \frac{3632 \times 8.75^2}{6 \times 3.79} = -12,250$ kgm

$M_A = M_D = +\frac{q l^2}{12(k+2)} = +6,125$ kgm



Live Load Stresses

$M_L = -12,250$ kgm Dead load moment neglected

Directed load

Live load 15,900 kg
Dead load 64,000
  79,900 Call this 80,000 kg

$\frac{d'}{h} = 0.062$ $e = \frac{12,250}{80,000} = 0.153$ m, $\frac{e}{h} = \frac{.153}{.8} = .191$

$p_o = 0.0085$

$k' = 1.03$ $k'' = .92$

$K = 1.03 - 5 \times .062 \times .11 = 0.995$

$B_2 = .0835$ $A_2 = 0.0285$ $C_2 = 100 \times .0085 \times .0285 + .0835 = 0.1077$

$f_c = \frac{12,250}{.1077 \times .574 \times .8^2} = 310,000$ kg/m² $= 31$ kg/cm² OK

$f_s =$ negligible

CALCULATIONS FOR

Katsura Bashi for Kyotofu

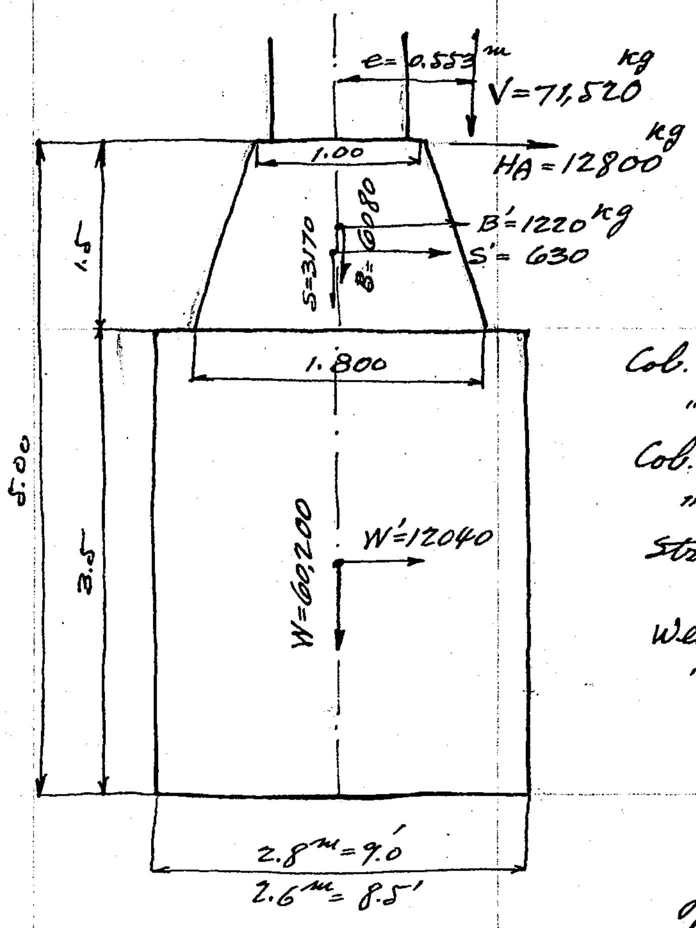
Design of Pier Foundation

Seismic stability

acceleration of earthquake = 2,000 $^{mm}/_{sec^2}$

$M_A = 39,500^{kgm}$, $H_A = 12800^{kg}$, Vert. load on Col. $V = 71520^{kg}$

ecc. $e = 0.533^m$



Weight

Column base $B = 1.3 \times 1.3 \times 1.5 @ 2400 = 6080$

Strut $S = .6 \times 1.0 \times 2.2 @ " = 3170$

Well $W = 2.8 \times 2.8 \times 3.5 @ 2200 = 60,200$

Lever arm of seismic force from bottom of base

$\frac{1.5}{3}\ \frac{1.8+2}{2.8} = .68 + 3.5 = 4.18$

$.5 + 3.5 = 4.00$

$\frac{3.5}{2} = 1.75$

Taking moment about Center of bottom of well

		Hor. loads	Vert. loads	Lever arm	moment
Col. load	V		71,520	0.553	39,550
"	H	12,800		5,00	64,000
Col. base	B		6,080	0.	0
"	B'	1,220		4.18	5,100
~~Strut~~	S		3,170	0.	0
"	S'	630		4.00	2,520
Well	W		60,200	0	0
"	W'	12,040		1.75	21,080
		$\Sigma H = 26,690^{kg}$	$\Sigma V = 140,970^{kg}$		$132,250^{kgm}$

Eccentricity $e = \frac{132.250}{140,970} = 0.94^m$

$\frac{e}{b} = \frac{0.94}{2.8} = .336$ tension occurs

Neglecting tension, max toe pressure

$\frac{2 \times 140,970}{2.8 \times (1.4 - .94) 3} = 72,900^{kg}/m^2 = 14,930^{\#}/\square' = 6.67^{tons}/\square'$ OK

Try well $2.6 \times 2.6 \times 3.5^m$ (8.5' 角)

$W = 2.6 \times 2.6 \times 3.5 @ 2200 = 52,000^{kg}$

$W' = 10,400^{kg}$

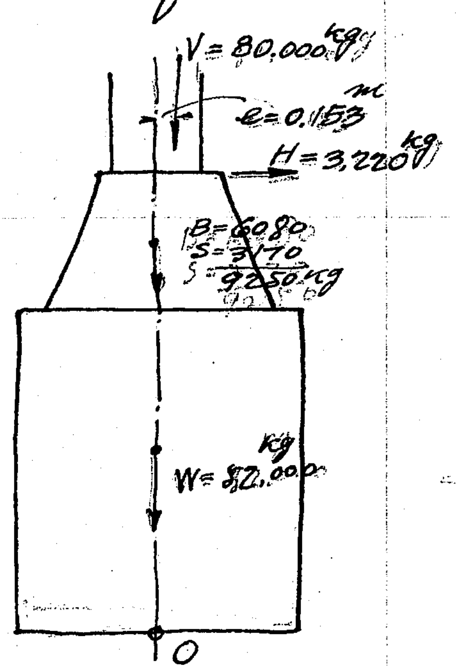
$\Sigma H = 25,050$ $\Sigma V = 132.770$ $\Sigma M = 129.370^{kgm}$

Ecc. $e = \frac{129370}{132,770} = 0.974$

Neglecting tension, max toe pressure

$= \frac{2 \times 132,770}{2.6 \times (1.3 - .974) \times 3} = 104,400^{kg}/m^2 = 21,400^{\#}/\square' = 9.55^{tons}/\square'$

Case of live load max.



		~~Hor. loads~~ Hor. loads	Vert. loads	Lever arm	Moment about O
Col. load	V		80,000	0.153	12,250 kgm
" "	H	3,220		5,000	16,100
Col. base	B		6,080	0	0
Strut	S		3,170	0	0
Well	W		52,000	0	0
		$\Sigma H = 3220$	$\Sigma V = 141,250^{kg}$		$28,350^{kgm}$

Eccentricity $e = \frac{28.350}{141,250} = 0.205^m$

$\frac{e}{b} = \frac{0.205}{2.6} = 0.077$ Comp. over whole section

Section modulus $= \frac{2.5 \times 2.5^2}{6} = 2.6$

max toe pressure $= \frac{141.250}{2.5 \times 2.5} \pm \frac{28,350}{2.6}$

$= 22,600 \pm 10,900$

$= 33500^{kg}/m^2 = 6870^{\#}/\square' = \underline{\underline{3.07}}^{tons}/\square'$

or $= 11700$ "

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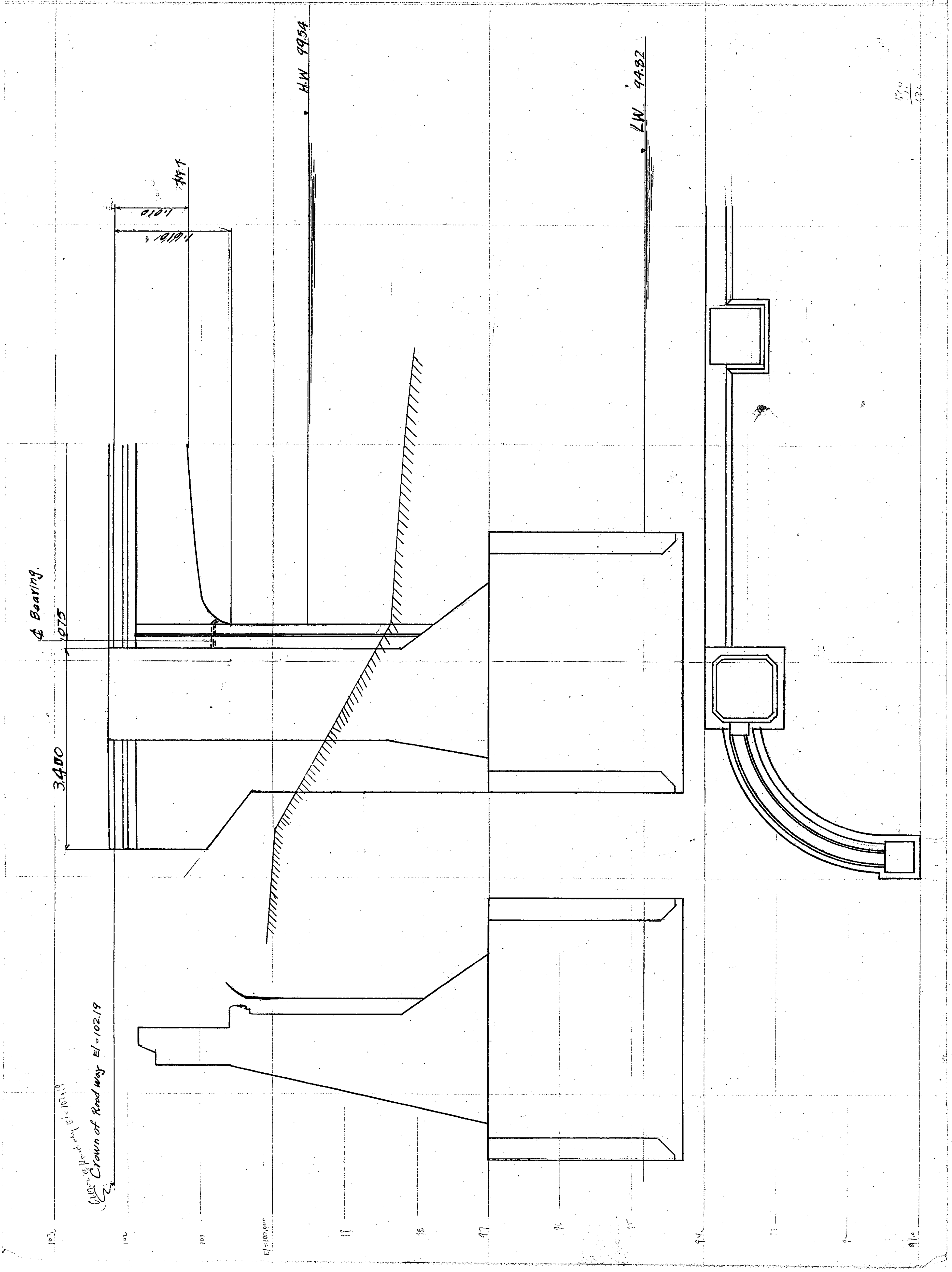
CALCULATIONS FOR



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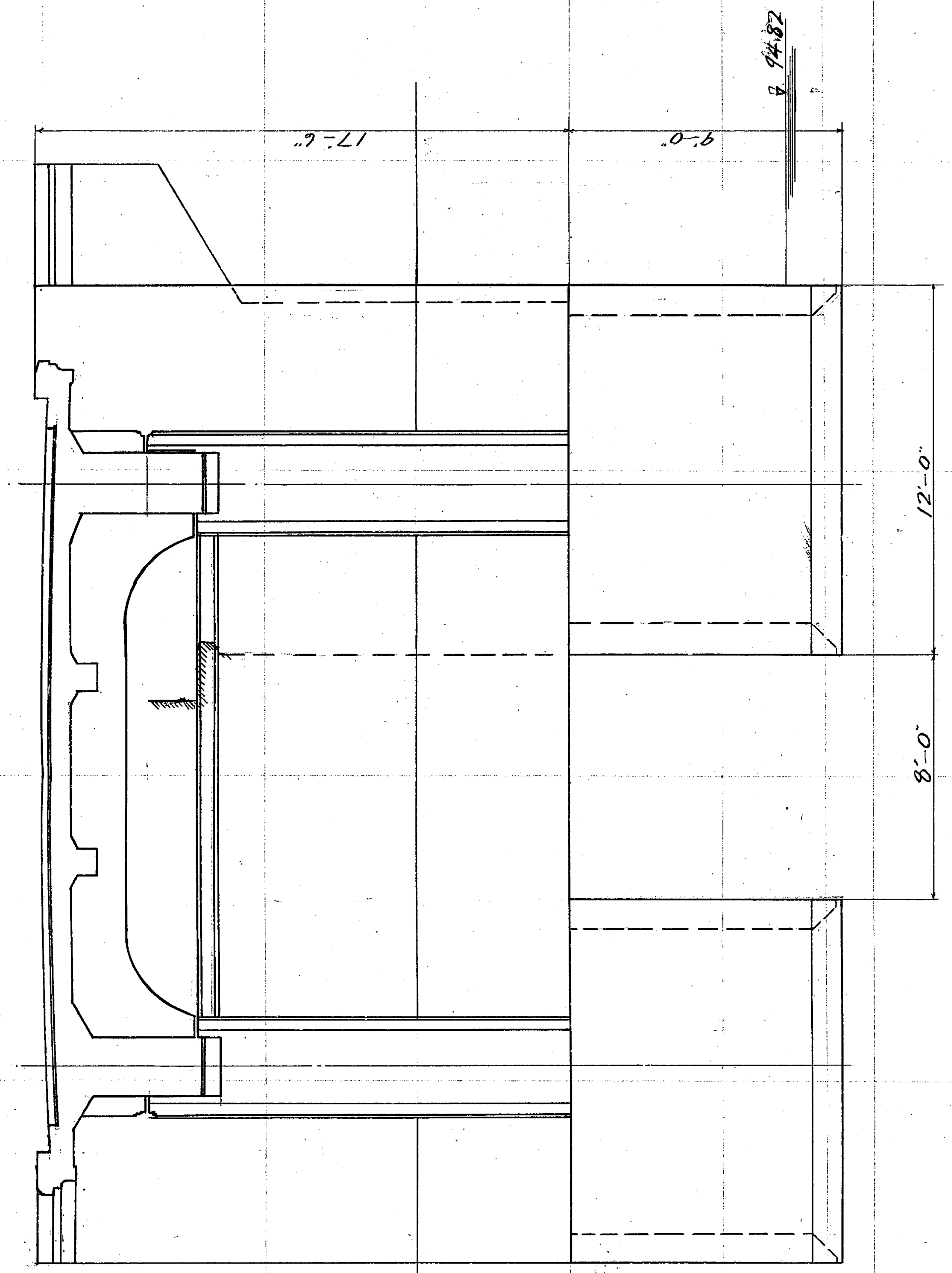
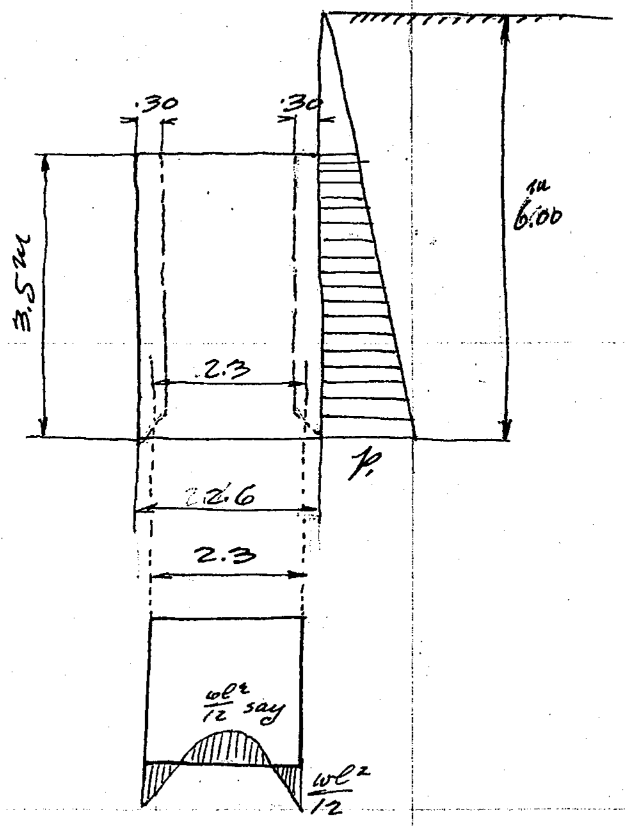
CALCULATIONS FOR



Katsura Bashi for Kyotofu

Design of Caisson for Pier 2.6^m square 3.5^m deep



Earth pressure at bottom of caisson

$= \frac{1600^{kg} \times 6}{3} = 3200\ ^{kg}/m^2$

Bending moment due to earth pressure on side wall

$= \frac{1}{12} \times 3200 \times 2.3^2 = 1410\ ^{kg}m.$

Thickness 30^{cm} or effective depth 27.5^{cm}

Steel area req'd $= \frac{1410 \times 100}{1200 \times \frac{7}{8} \times 27.5} = 4.88\ ^{\square cm}$/meter strip

Use $\frac{1}{2}''\phi$ Bars spacing $= \frac{1.267}{4.88} \times 100 = 26^{cm}$

at 1.5^m from bottom

earth pressure $= \frac{1600 \times 4.5}{3} = 2400\ ^{kg}/m^2$

moment $= \frac{1}{12} \times 2400 \times 2.3^2 = 1058\ ^{kg}m$

steel req'd $= \frac{1058 \times 100}{1200 \times \frac{7}{8} \times 27.5} = 2.665\ ^{\square cm}$/meter strip

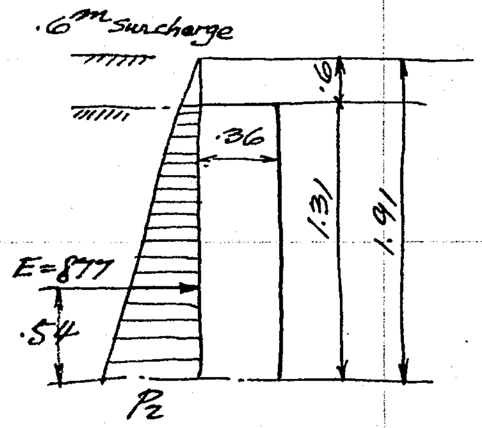
Use $\frac{1}{2}''\phi$ Bars spacing $= \frac{1.267}{2.665} \times 100 = 47.5^{cm}$

Design of Abutment
Prapet wall

Case 1

Stability at normal case

0.6^m surcharge due to live load assumed



$P_1 = .6 \times 1600 \times \frac{1}{3} = 320$

$P_2 = 1.91 \times 1600 \times \frac{1}{3} = 1.020\ ^{kg}/m^2$

$E = \frac{320+1020}{2} \times 1.31 = 877\ ^{kg}$/meter strip

pt. of application of P

$x_1 = \frac{1.31(1020+640)}{3(1020+320)} = .54$

Moment at bottom of wall

$877 \times .54 = 474\ ^{kg}$/meter strip

Effective depth say 33^{cm}

Steel area req'd $= \frac{474 \times 100}{1200 \times \frac{7}{8} \times 33} = 1.37\ ^{\square cm}$

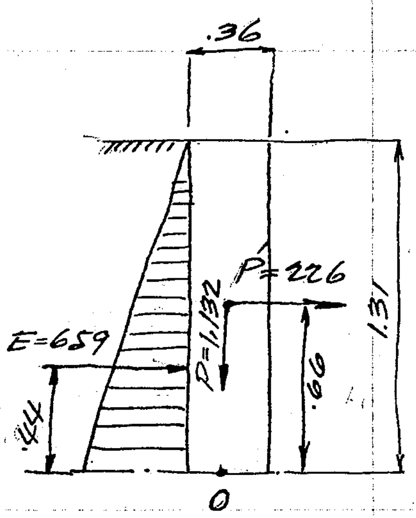
Use $\frac{1}{2}''\phi$

Spacing $= \frac{100 \times 1.267}{1.37} = .925^m$

Use 45^{cm} Spacing Both sides

Case 2

Stability during earthquake of which acceleration $= 2.000\ ^{mm}/sec^2$



$E = \frac{wh^2}{2} \times .48 = .24 \times 1600 \times 1.31^2 = 659\ ^{kg}$/meter strip

weight of parapet wall $= .36 \times 1.31 \times 2400 = 1.132^{kg}$

moment about O

Loads	Lever arm	moment about O
P = 1.132	0	0
E = 659	.44	290
P' = 226	.66	149
		$439^{kg}m$

Details for Case 1 discussed above are quite safe for this case.

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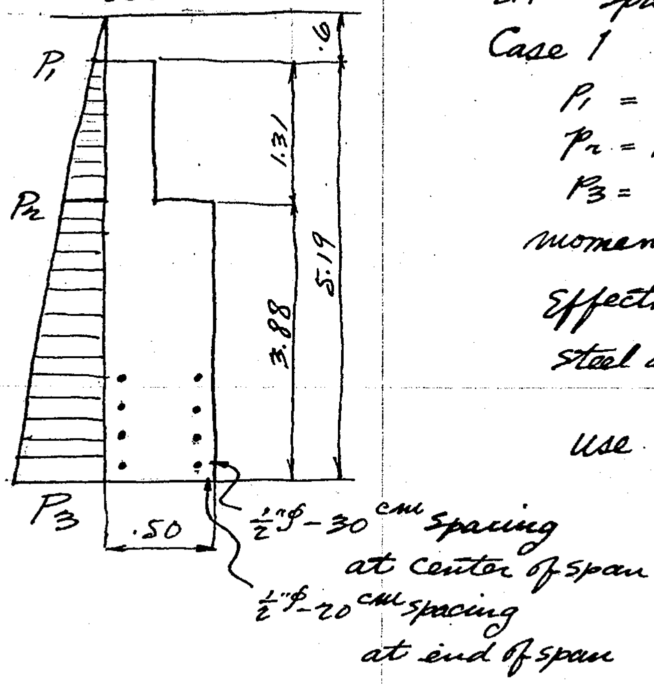
CALCULATIONS FOR

Katsura Bashi for Kyotofu

Curtain wall



2.4 m span

Case 1 Normal case

$P_1 = .6 \times 1600 \times \frac{1}{3} = 320\ kg/m^2$

$P_2 = 1.91 \times 1600 \times \frac{1}{3} = 1,020$ "

$P_3 = 5.79 \times 1600 \times \frac{1}{3} = 3,090$ "

moment at bottom 1m strip $= \frac{3090 \times 2.4^2}{10} = 1,780\ kgm$

Effective depth say 47 cm   End shear $= 3090 \times 2.4 = 7,420\ kg$

steel area req'd $= \frac{1,780 \times 100}{1200 \times \frac{7}{8} \times 47} = 3.60\ cm^2$

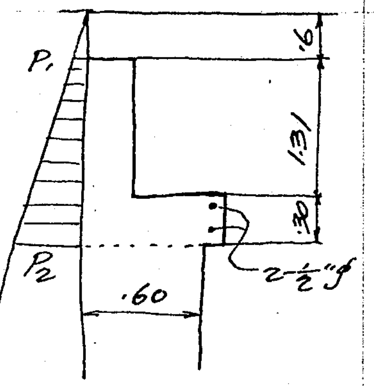
Use $\frac{1}{2}$"φ - 30 cm spacing = 4.22 cm² ok

Unit shear $= \frac{7420}{100 \times \frac{7}{8} \times 47}$

Unit bond $= \frac{7420}{4 \times \frac{7}{8} \times 47} = 45.1\ kg$ for 1 bar

No. of bars req'd for bond stress at end for deformed bars $= \frac{45.1}{9} = 5$ or 20 cm spacing

At top of curtain wall



$P_1 = .6 \times 1600 \times \frac{1}{3} = 320\ kg/m^2$

$P_2 = 2.21 \times 1600 \times \frac{1}{3} = 1,180$ "

$E = \frac{320 + 1180}{2} \times 1.61 = 1,207\ kg$/lin. meter

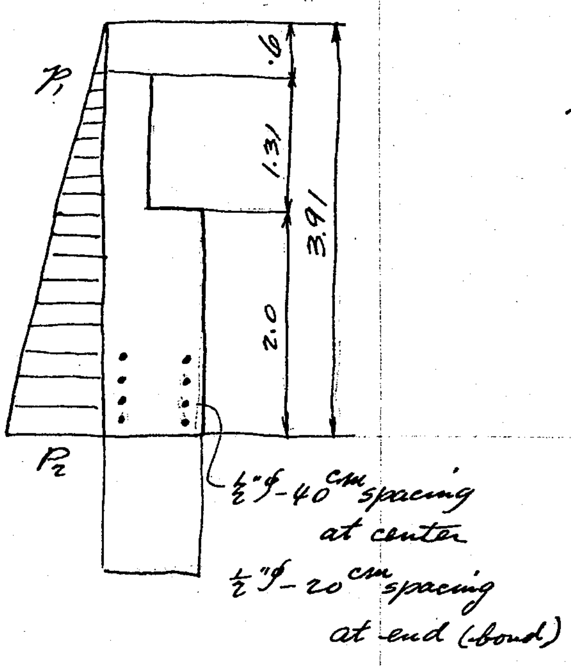
effective depth of beam say 57 cm

moment $= \frac{1}{10} \times 1207 \times 2.4^2 = 695\ kgm$

steel area req'd $= \frac{695 \times 100}{1200 \times \frac{7}{8} \times 57} = 1.16\ cm^2$

Use 2-$\frac{1}{2}$"φ = 2,534 cm²

At 2 meter from top of curtain wall



$P_1 = \qquad = 320$

$P_2 = 3.91 \times 1600 \times \frac{1}{3} = 2,085$

$E \qquad = 2,405 \div 2 \times 2.31 = 2,780\ kg/m.$

moment $= \frac{2780 \times 2.4^2}{10} = 1,600\ kg\ m$

End shear $= 2,780 \times 1.2 = 3,085\ kg$

steel req'd $= \frac{1600 \times 100}{1200 \times \frac{7}{8} \times 47} = 3.24\ \square cm.$

Use $\frac{1}{2}$"φ deformed bars

spacing $= \frac{100 \times 1.267}{3.24} = 39\ cm$

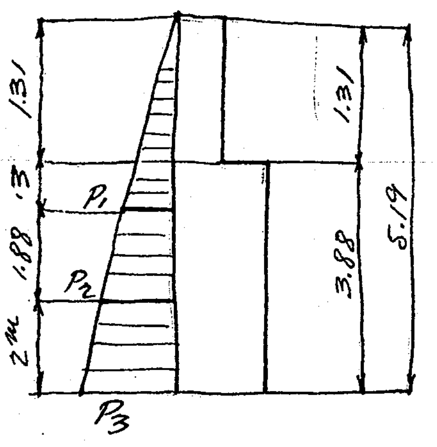
Unit shear $= \frac{3085}{100 \times \frac{7}{8} \times 47} = 0.75\ kg/\square cm$ ok

bond stress $= \frac{3085}{4 \times \frac{7}{8} \times 47} = 18.75\ kg/\square cm$ for 1 bar

req'd spacing of $\frac{1}{2}$"φ deformed bar

$= \frac{100 \times 9}{18.75} = \underline{48}\ cm$ for bond stress

Use $\underline{30}$ cm spacing



Case 2 During earthquake k = 0.2

Earth pressure during earth quake

$P_1 = 0.48 \times 1600 \times 1.61 = 1,230\ kg/m^2$ wt. of curtain wall

$P_2 = 0.48 \times 1600 \times 3.49 = 2,680$ " $= 1 \times .5 \times 2400 = 1200\ kg/m$

$P_3 = 0.48 \times 1600 \times 5.19 = 3,980$ "

at bottom 1m strip loads on beam { earth pessure = 3,980 kg/m; seismic force of wall = 240 " } 4,220 "

moment $= \frac{1}{10} \times 4220 \times 2.4^2 = 2,430\ kgm$

steel req'd $= \frac{2,430 \times 100}{2160 \times \frac{7}{8} \times 47} = 2.74\ \square cm$ Less than case 1 (3.6) ok

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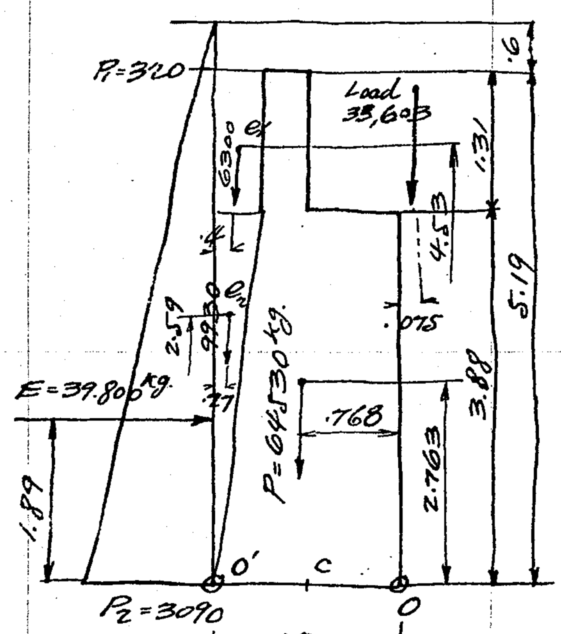
CALCULATIONS FOR

Katsura Bashi for Kyotofu

Shaft Case 1 Normal case

Load on shaft

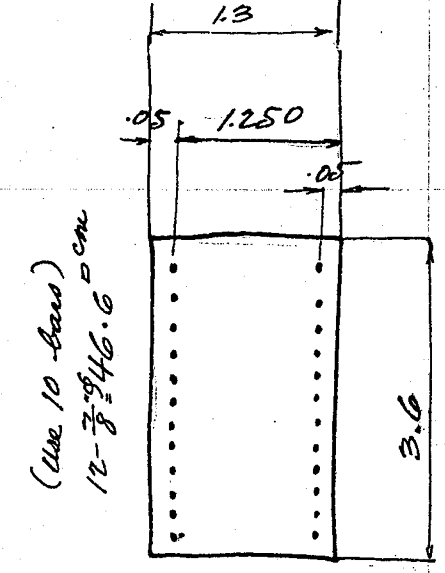


Reaction - max reation on shaft = 33,603 kg See P.19

C.G. of structure above top of wall

				Hor. lever arm from face O		
Parapet wall	.36 x 1.31 x 3.4 @ 2400	= P	= 3,850 kg.	x .32	=	1,232 kgm
Curtain "	.5 x 1.2 x 3.88 @ 2400	= C	= 5,590	x .25	=	1,398
Shaft	$\frac{.5+1.3}{2}$ x 3.88 x 2.55 @ 2400	= S_1	= 21,400	x .48	=	10,270
"	.35 x .5 x 1.31 @ 2400	= S_2	= 550	x .25	=	138
Column	1.05 x 1.1 x 5.265 @ 2400	= Cl_1	= 14,600	x .55	=	8,030
" under beam	1.0 x .3 x 3.88 @ 2400	= Cl_2	= 2,800	x −.15	=	− 420
Wing wall	.6 x 5.265 x .8 @ 2400	= W_1	= 6,060	x 1.50	=	9,090
"	.5 x 1.5 x 2.7 @ 2400	= W_2	= 4,860	x 2.80	=	13,600
Light pedestal	.8 x .8 x 2.0 @ 2400	= Lp	= 3,070	x .55	=	1,690
Handrail	500 kg x 3.5 m	= Hr	= 1,750	x 2.60	=	4,550
		Total	= 64,530 kg.			49,578

Distance of Resultant form O = $\frac{49,578}{64,530}$ = .768 m



Vertical dist. of pt. of application of resultant

		Lever arm		moment about O
P	3850	x 4.54	=	17,480
C	5590	x 1.94	=	10,840
S_1	21400	x 1.65	=	35,330
S_2	550	x 4.54	=	2,500
Cl_1	14600	x 2.63	=	38,400
Cl_2	2800	x 1.94	=	5,430
W_1	6060	x 2.63	=	15,940
W_2	4860	x 4.76	=	23,150
L	3070	x 6.26	=	19,220
Hr	1750	x 5.76	=	10,080

Vert. dist. from O = 178,370 ÷ 64,530 = 2.763 m

$p_0 = \frac{46.6 \times 2}{1.25 \times 3.6 \times 10,000} = .0021$

$\frac{d'}{h} = \frac{.05}{1.30} = .0385, \quad \frac{e}{h} = \frac{.758}{1.30} = .582$

$k = 0.335 \quad k'' = .283$

$k = 0.335 - .5 \times .0385 \times .052 = .325$

$A_2 = .099 \quad B_2 = .0635$

$C_2 = 100 \times .0021 \times .099 + .0635 = .0843$

$f_c = \frac{M}{C_2 b h^2} = \frac{86,700 \times 100}{.0843 \times 360 \times 130^2} = 16.9$ kg/cm² ok

$f_s = n f_c \left(\frac{d}{kh} - 1\right) = 15 \times 16.9 \left(\frac{1.25}{.325 \times 1.3} - 1\right)$

$= 496.5$

Use 10-$\frac{7}{8}$"φ bars

Earth pressure

$E = \frac{320+3090}{2} \times 5.19 \times 4.5^{m}$ (width say) $= 39,800$ kg

$e_1 = .8 \times 1.31 \times 3.75 \times 1600 = 6,300$

$e_2 = .4 \times 3.88 \times 4.0 \times 1600 = 9,930$

Moment about O'

	Loads	arm	moments
L	33,603	1.375	46,200
P	64,530	.532	34,350
e_1	6,300	.40	2,520
e_2	9,930	.27	2,680
E	39,800	1.89	75,200
	ΣV = 114,360 kg		160,950 kgm ÷ 114,360 kg

ΣH = 39,800

= 1.408 m

Eccentricity = 1.408 − .65 = .758 m

Bending moment at bottom of shaft

= 114,360 x .758 = 86,700 kgm

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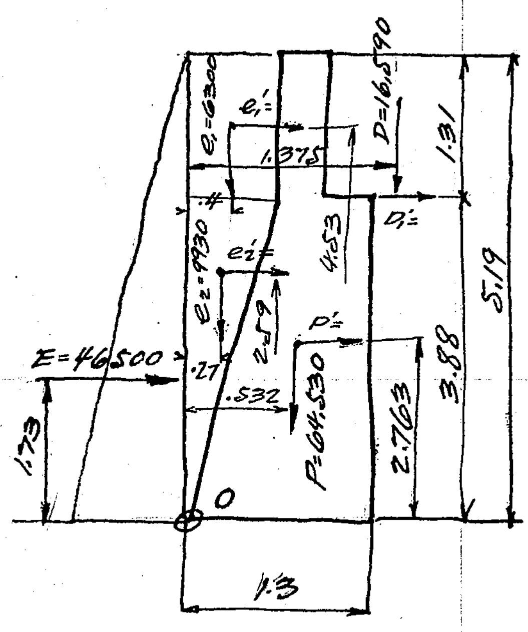
CALCULATIONS FOR

Katsura Bashi for Kyotofu

Case 2 During earthquake of which accelation = $2000^{mm/sec^2}$



	Vert. loads	Hor. loads	lever arm	moments
D	16,590		1.375	22,800 kg.m.
D'		3,320	3.88	12,880
P	64,530		.532	34,320
P'		12,960	2.763	35,800
e_1	6,300		.40	2,520
e_1'		1,260	4.53	5,710
e_2	9,930		.27	2,680
e_2'		1,990	2.59	5,150
E		46,500	1.73	80,400
	ΣV=97,350	ΣH=66,030		202,260 kg.m

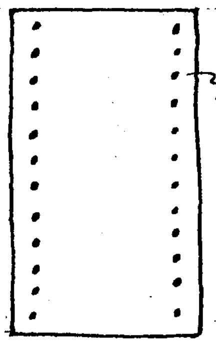
Distance of resultant from O = 202,260 ÷ 97,350 = 2.08 m

Eccentricity = 2.08 − .65 = 1.43 m

Earth pressure = $\dfrac{.48 \times 1600 \times 5.19^2 \times 4.5}{2}$ = 46,500 kg.

Bending moment at bottom of shaft = 97,350 × 1.43 = 139,200 kgm.

$\dfrac{e}{h} = \dfrac{1.43}{1.3} = 1.10$ $\quad \dfrac{d'}{h} = \dfrac{.05}{1.3} = .0385$ $\quad P_0 = \dfrac{466 \times 2}{125 \times 36} = .0021$



$k' = .230 \quad k'' = .205$

$k = .230 - 5 \times .0385 \times .025 = .225$

$A_2 = .061 \quad B_2 = .0475 \quad C_2 = .21 \times .061 + .0475 = .0603$

$f_c = \dfrac{139,200 \times 100}{.0603 \times 360 \times 130^2} = 38.0$ kg/cm² OK

$f_s = 15 \times 38 \left(\dfrac{1.25}{.225 \times 130} - 1\right) = 1,870$ kg/cm² ok

Case of no dead load from supper structure

Moment about O = 202,260 − 35,680 = 166,580 kgm

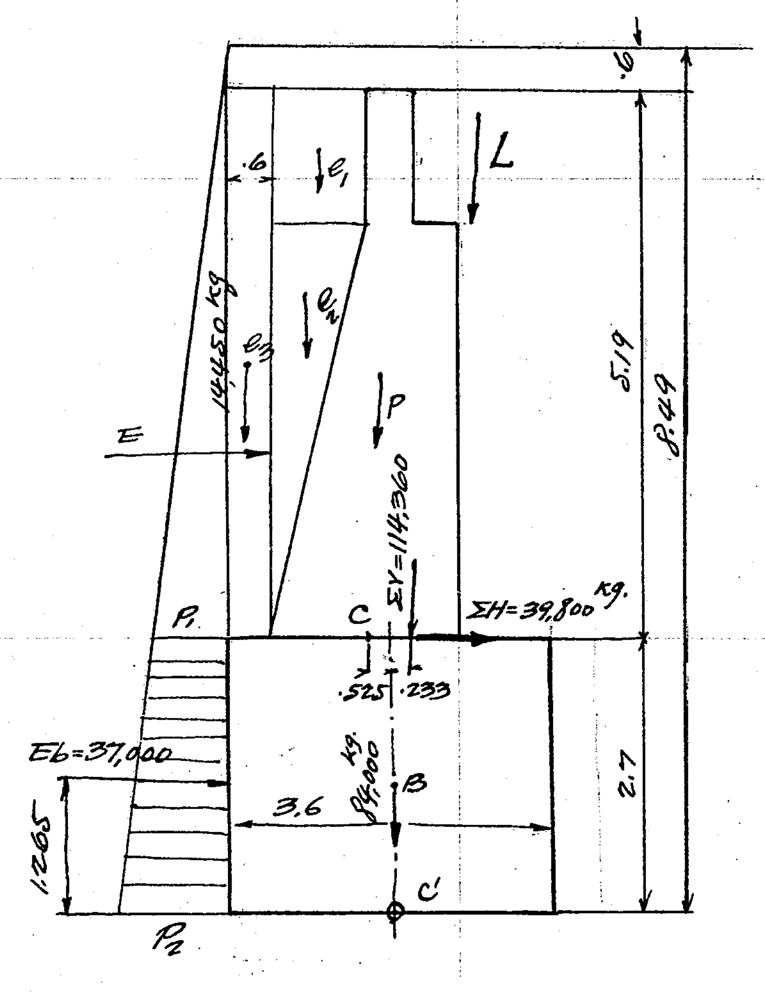
ΣV = 80,760 kg $\quad \Sigma H$ = 62,710 kg.

Dist. from O = $\dfrac{166580}{80760}$ = 2.065 m

Ecc. = 2.065 − .65 = 1.415 m

Foundation

Case 1 Stability of abutment at bottom of base in normal state



$e_3 = .6 \times 2.9 \times 5.19 \times 1,600 = 14,450$ kg

$B = 3.6 \times 3.6 \times 2.7 \times 2400 = 84,000$

Earth pressure on base only

$P_1 = \dfrac{5.79 \times 1600}{3} = 3090$

$P_2 = \dfrac{8.49 \times 1600}{3} = 4530$

$E_b = \dfrac{3090 + 4530}{2} \times 2.7 \times 3.6 = 37,000$ kg.

Moment about C'

	Hor. loads	Vert. loads	arm	moment
V		114,360	+.233	+26,650
H	39,800		+2.70	+107,500
e_3		14,450	−1.475	−21,300
B		84,000	0	0
E_b	37,000		+1.265	46,800
ΣH	76,800	ΣV=212,810		+159,650 kgm.

Eccentricity = $\dfrac{159,650}{212,810}$ = 0.75 m $\quad a = 1.8 - .75 = 1.05$

Neglecting tension

max. toe pressure = $\dfrac{2 \times 212,810}{1.05 \times 3 \times 3.6}$ = 37,550 kg/m² = 3.44 tons/□' OK

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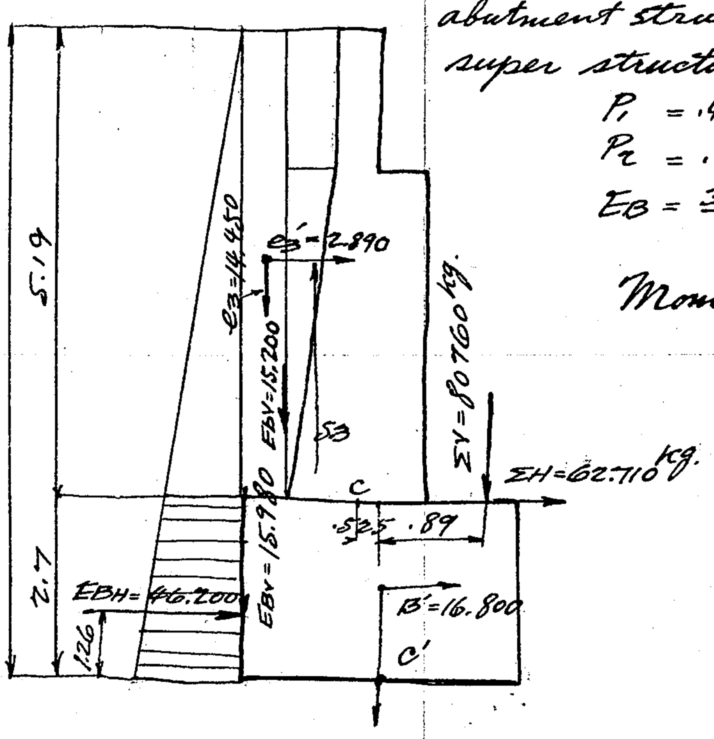
CALCULATIONS FOR

Katsura Bashi for Kyotofu

Case 2 Stability of abutment during earthquake
Earth pressure due to back fill and seismic force and weight of abutment structure acting toward river side, no dead load from super structure, with,—



$P_1 = .48 \times 1600 \times 5.19$
$P_2 = .48 \times 1600 \times 7.89$
$E_B = \frac{3985+6060}{2} \times 2.7 \times 3.6 = 48,850$ kg
$E_{bV} = 48850 \times .327 = 15,980$ kg
$E_{bH} = 48850 \times .945 = 46,200$

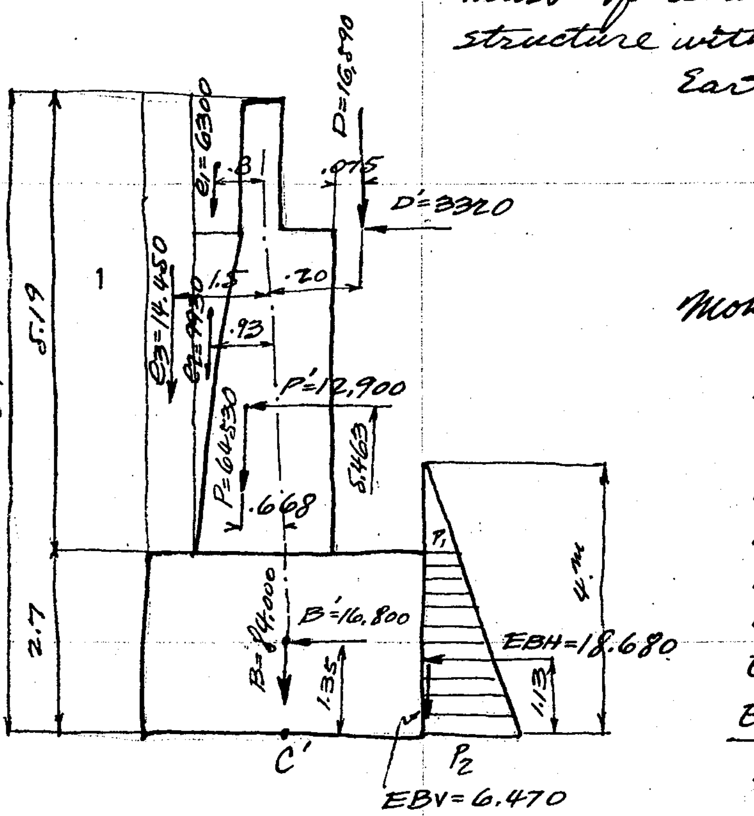
Moment about C'

	Hor. loads	Vert. loads	arm	moment
e_3		14,450	−1.475	−21,300
e_3'	2890		+5.30	+15,320
Y		80,760	+.89	+71,850
H	62,710		+2.70	+169,400
B		84,000	0	0
B'	16,800		+1.35	+22,680
EBH	46,200		+1.26	+58,200
EBV		15,980	−1.80	−28,800
EbV		15,200	−1.175	−17,850
	ΣH=128,600	ΣV=210,390		ΣM= 269,500

$\text{Eccentricity} = \frac{269,500}{210,390} = 1.28 \qquad a = 1.8 - 1.28 = .52$

Neglecting tension
$\text{Max. toe pressure} = \frac{2 \times 210,390}{3 \times .52 \times 3.6} = 74,900\ \text{kg/m}^2 = 6.85\ \text{tons/□'}$

Case 3 Stability of abutment during earthquake
Mass of abutment body and dead load from super structure with no earth pressure.



Earth press. $p_1 = .48 \times 1600 \times 1.3 = 1,000$
$p_2 = .48 \times 1600 \times 4.0 = 3,070$
$EB = \frac{1000+3070}{2} \times 2.7 \times 3.6 = 19,760$ kg.
$EBV = 19,760 \times .327 = 6,470$
$EBH = 19,760 \times .945 = 18,680$

Moment about C'

	Hor. loads	Vert. loads	arm	moment
D		16,590	−.20	−3,320
D'	3,320		+6.58	+21,850
P		64,530	+.668	+43,100
P'	12,900		+5.463	+70,500
B		84,000	0	0
B'	16,800		+1.35	+22,700
EBH	18,680		+1.13	+21,100
EBV		6,470	−1.80	−11,650
	ΣH = 51,700	ΣV=171,590		+164,280

$\text{Ecc.} = \frac{164,280}{171,590} = .958^{m} \qquad a = 1.8 - .958 = .842$

Neglecting tension
$\text{Max. heel press.} = \frac{2 \times 171,590}{3 \times .842 \times 3.6} = 37,750\ \text{kg/m}^2 = 3.45\ \text{tons/□'}$ OK

If the weight of earth on the heel of foundation be taken into account,—

	Hor. loads	Vert. loads	arm	moment
	51,700	171,590		+164,280
e_1		6,300	+.8	+ 5,040
e_2		9,930	+.93	+ 9,230
e_3		14,450	+1.50	+ 21,680
	ΣH=51,700	ΣV=202,270		200,230

$\text{Ecc.} = \frac{200,230}{202,270} = 0.99^{m} \qquad a = 1.8 - .99 = .81^{m}$

Neglecting tension
max. heel pressure
$= \frac{2 \times 202,270}{3 \times .81 \times 3.6} = 46,300\ \text{kg/m}^2$
$= 4.24\ \text{tons/□'}$
OK.

Katsura Bashi for Kyotofu.

Case 4 Seismic forces toward river side

Section modulus of bed area
$= \frac{3.6 \times 3.6^2}{6} = 7.77 (m^3)$

Max toe press.
$= \frac{195.800}{3.6 \times 3.6} \pm \frac{74250}{7.77}$
$= 15,110 \pm 9560$
$= 24,670 \, kg/m^2 = 2.255$ tons/□' ok
or $= 5,550$ " $= 0.503$ "

	Hor. load	Vert. load	arm	moment
D		16,590	+ .20	+ 3,320
D'	3,320		+ 6.58	+ 21,850
P		64,530	+ .668	− 43,100
P'	12,900		+ 5.463	70,500
B		84,000	0	0
B'	16,800		+ 1.35	+ 22,700
e_1		6,300	− .8	− 5,040
e_1'	1,260		+ 7.23	+ 9,110
e_2		9,930	− .93	− 9,230
e_2'	1,990		+ 5.29	+ 10,520
e_3		14,450	− 1.50	− 21,680
e_3'	2,890		+ 5.30	+ 15,300
	ΣH = 39,160	ΣY = 195,800		ΣM = 74,250

Wing wall 2^m deep

Earth press. $= \frac{2 \times 1600 \times 2}{3 \times 2} = 1,070 \, kg$

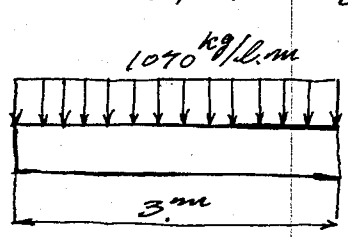
ecc. $= \frac{74250}{195.800} = 0.38^m$

moment $= \frac{1070 \times 3^2}{2} = 4,810$ kgm/2meter strip or 2410 kgm/meter strip

Thickness of wall say 0.6^m effective depth say 57 cm.

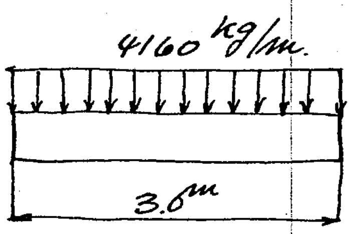
Steel req'd $= \frac{2410 \times 100}{1200 \times \frac{7}{8} \times 57} = 4.02 \, □cm$

Use $\frac{5}{8}$"φ spacing $= \frac{100 \times 1.979}{4.02} = 51$ cm at fixture, Use 30 cm spacing at fixture.



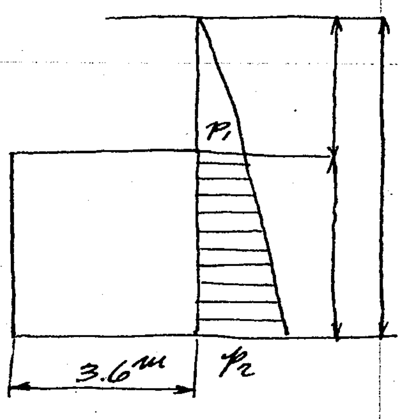
At middle of span
moment $= 2410 \times \frac{1}{4} = 600$ kgm
steel req'd $= \frac{600 \times 100}{1200 \times \frac{7}{8} \times 25} = 2.29 \, cm^2$

Use 5/8"φ, spacing $= \frac{100 \times 1.979}{2.29} = 86$, cm use 60 cm spacing at center of span.

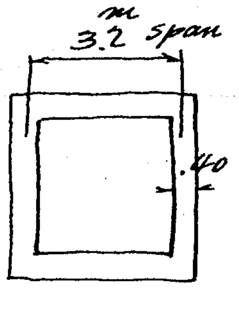


During earth quake
earth pressure $= \frac{.48 \times 1600 \times 2 \times 2}{2} = 1535$ kg/2meter strip
Seismic force
due to Handrail $500 \times .2 = 100$
due to wall $.6 \times 1.0 \times 2400 = 1440$
3075 kg/2meter strip $= 1540$ kg/meter strip
moment at fixture $= 1540 \times 3^2 \times \frac{1}{2} = 6,930$ kgm
steel Area req'd $= \frac{6,930 \times 100}{2160 \times \frac{7}{8} \times 57} = 6.45 \, cm^2$

Use 5/8"φ spacing $\frac{100 \times 1.979}{6.45} = 30.5$ cm
take 30 cm spacing at fixture



Caisson for foundation 3.6^m square 2.7^m deep
Earth pressure at bottom of caisson $= \frac{1600 \times 4.5}{3} = 2,400 \, kg/m^2$
" " " 1^m from bottom $= \frac{1600 \times 3.5}{3} = 1870 \, kg/m^2$
moment $= \frac{1}{12} \times 2400 \times 3.2^2 = 2,050$ kgm at bottom
" $= \frac{1}{12} \times 1870 \times 3.2^2 = 1,600$ " at 1^m above bottom



Use 40 cm thickness, effective depth say 37.5 cm
At bottom steel req'd $= \frac{2050 \times 100}{1200 \times \frac{7}{8} \times 37.5} = 5.21 \, □cm.$
use $\frac{1}{2}$"φ
spacing $= \frac{1.267 \times 100}{5.21} = 24.3$ cm. at bottom

At bottom $p = \frac{1.267 \times 4}{40 \times 100} = 0.00127$ $k = .175$ $j = .942$
$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 2050 \times 100}{.175 \times .942 \times 100 \times 37.5^2} = 17.7 \, kg/m^2$ ok

at 1^m from bottom steel required $= \frac{1600 \times 100}{1200 \times \frac{7}{8} \times 37.5} = 40.6$ cm

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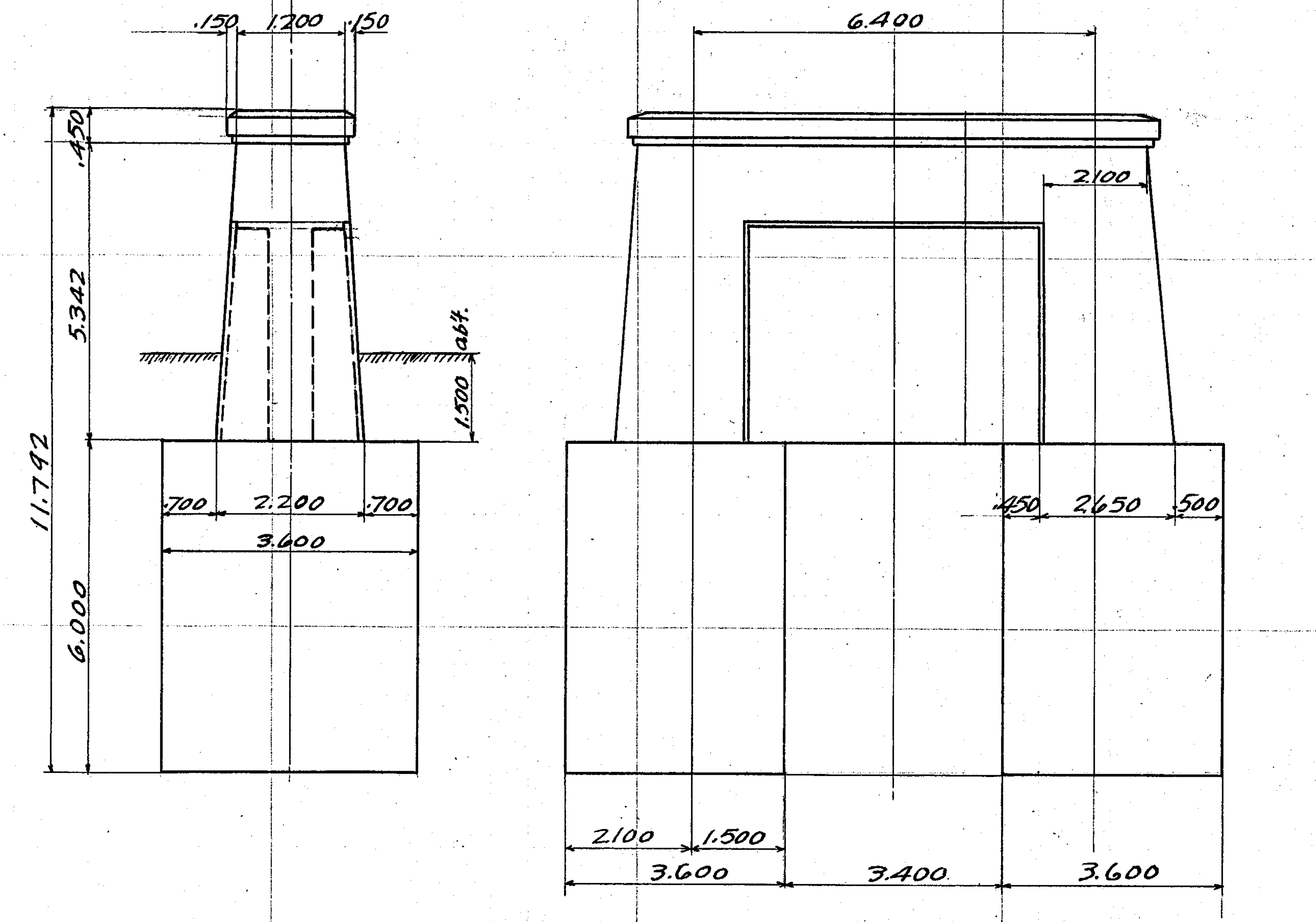
CALCULATIONS FOR

Katsura Bashi for Kyotofu
Design of piers for River span



Approximate weight of Pier on one well — lever arm from bottom of shaft

Shaft coping (c) $.45 \times 1.4 \times 4.2$ @ 2400 = 6,350 × 5.57 = 35,350 kgm
shaft (s) $\frac{1.85+2.35}{2} \times \frac{1.2+2.2}{2} \times 5.34$ @ 2400 = 45,770 × 2.36* = 108,000
wall (W) $1.05 \times 1.3 \times 4.3$ @ 2400 = 14,090 × 4.79 = 67,500
" (W_2) $3.84 \times .6 \times 4.3$ @ 2400 = 23,800 × 1.92 = 45,700
90,010 kg — 256,550 kgm

256,550 kgm ÷ 90,010 kg = 2.85 m

Dead load from super structure
from center span = 37,048 kg
" side span = 39,650 kg
wt of shoe say = 3,600
80,300 kg

Live load
from center span 34,540
" side span 18,300
52,840 kg

Total = 133,140 kg call this 133,200 kg

Weight of well $3.6 \times 3.6 \times 6.0$ @ 2400 = 186,800 kg

Extra load on shoe during earthquake
$80300 \times .2 \times 2 = 16060 \times 2 = 32,120$
$\frac{16060 \times 2.25 \times 2}{24.5} = 1,480^{kg} \times 2 = 2,960$ kg.

Note mark *
C.G. of shaft
try at 2.36 m from bottom

Upper vol. = $1.6 \times 2.98 \times 2.1 = 10.00$
lower vol. = $1.95 \times 2.36 \times 2.2 = 10.11$

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CALCULATIONS FOR

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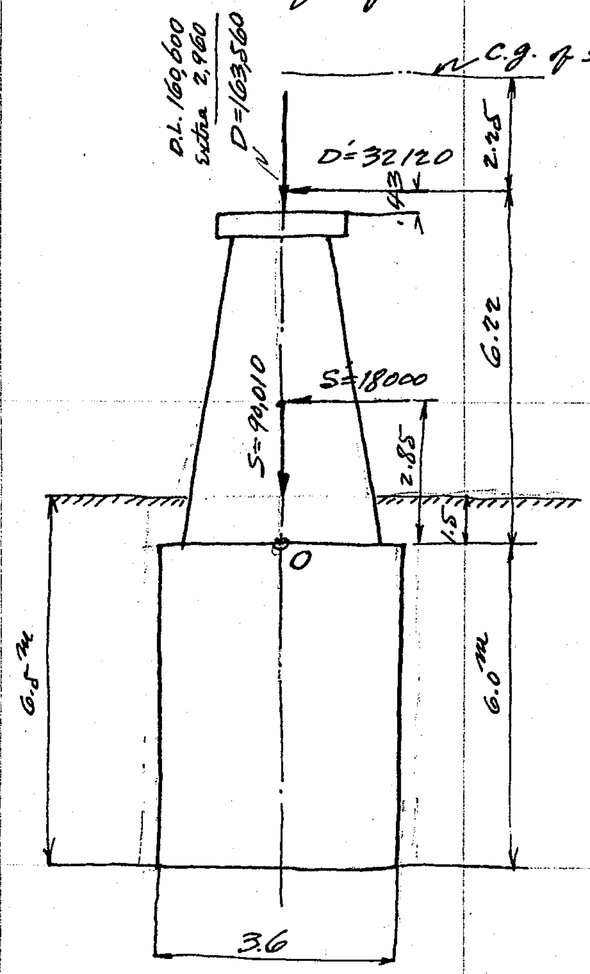
Stability of Pier Shaft during Earthquake $k = 0.2$



Moment about O

	Hor. loads	Vert. loads	lever arm	moment
D		163.560	× 0	= 0
D'	32.120		× 6.22	= 200,000
S		90,010	× 0	= 0
S'	18.000		× 2.85	= 50,650
	ΣH = 50,120 kg	ΣV = 253,570 kg		250,650 kgm

$$Ecc. = \frac{250.650}{25,3,570} = 0.99$$

$$\frac{e}{h} = \frac{.99}{2.2} = 0.45 \quad \frac{d'}{h} = \frac{.05}{2.2} = 0.02275$$

$$p_0 = 2p = \frac{22.8 \times 2}{220 \times 225} = .00092 \quad r = 1.1 - .05 = 1.05$$

$$k^3 - 3\left(\tfrac{1}{2} - \tfrac{e}{h}\right)k^2 + 6np_0k\frac{e}{h} = 3np_0\left(\frac{e}{h} + 2\frac{r^2}{h^2}\right)$$

$$k^3 - 3(.5 - .45)k^2 + 6 \times 15 \times .00092 \times .45k = 3 \times 15 \times .00092\left(.45 + 2\frac{1.05^2}{2.2^2}\right)$$

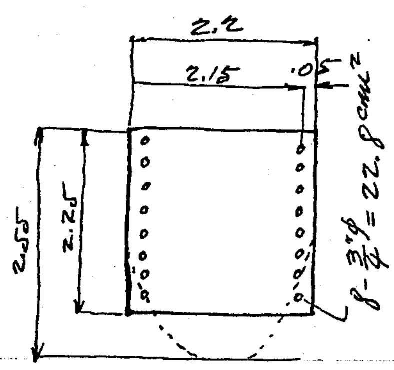
$$k^3 - 0.15k^2 + .0373k = .0375$$

Try with $k = 0.35$

$0.04288 - .01835 + .01306 = .03759$ ok $k = 0.35$

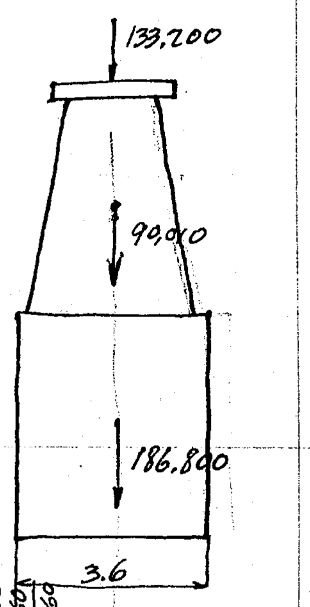
$$L = \left[\frac{np_0r^2}{kh^2} + \frac{k}{12}(3 - 2k)\right]$$

$$= \frac{15 \times .00092 \times 1.05^2}{0.35 \times 2.2^2} + \frac{.35}{12}(3 - 2 \times .35) = .0761$$



$$f_c = \frac{M}{Lbh^2} = \frac{250,650 \times 100}{0.0761 \times 225 \times 220^2} = 30.3\ kg/cm^2 \text{ ok}$$

$$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 30.3\left(\frac{215}{.35 \times 220} - 1\right) = 813\ kg/m^2 \text{ ok}$$



In case of max. Live load

max. load 133.200 kg

$$f_c = \frac{133,200}{225 \times 220} = 2.7\ kg/cm^2 \quad \text{ok}$$

Stability at bottom of well

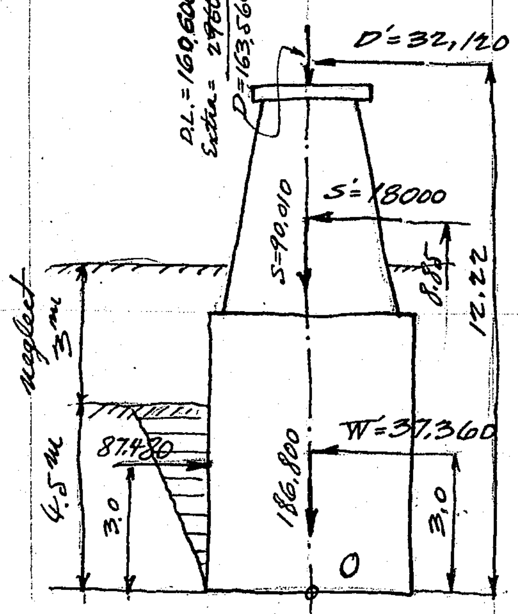
Case 1 max live load

super imposed load	= 133,200 kg
wt of shaft	= 90,010
wt of wall	= 186.800
Total	410,010

max. Bearing pressure on foundation $= \frac{410,010}{3.6 \times 3.6} = 31,650\ kg/m^2$ or $2.9\ tons/\square'$

Case 2 During earthquake $k = 0.2$

moment about O



	Hor. loads	Vert. loads	lever arm	moment ab't O
D		163.560	0	0
D'	32,120		12.22	393.000
S		90,010	0	0
S'	18,000		8.85	159.300
W		186,800	0	0
W	37.360		3.00	112,000
	87,480 kg	440,370 kg		664.300 kgm

Frictional resistance of wall 1200 kg/m² assumed (250 #/□')

Frictional resistance $= 3.6 \times 4.5 \times 1200 \times 4 = 19.450 \times 4 = 77,800$ kg

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Frictional couple resisting overturning moment = 19450 × 3.6 = 70.000 kgm
Resisting moment due to earth pressure = 87480 × 3 = 262.500 kgm
Resulting moment and vertical load

Vert. load		moment about 0
440,370 kg.		664.300 kgm
less friction 77,800	less frictional couple	70,000
	" earth pressure moment	262,500
362,570		331, 800 kgm

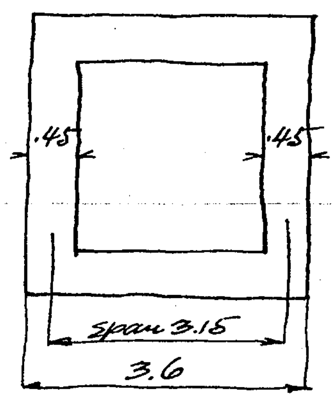
$$\text{Eccentricity} = \frac{331.800}{362,570} = 0.915^m \qquad a = 1.8 - .915 = .885$$

Neglecting tension

$$\text{Max toe pressure} = \frac{2 \times 362,570}{3 \times .885 \times 3.6} = 75,900^{kg/m^2} = 6.95^{Tons/□'} \text{ Ok}$$

Design of well 3.6^m square 6.0^m high

Earth pressure at bottom $\frac{1}{3} \times 1600 \times 7.5 = 4.000^{kg/□m}$
" at 2^m from bottom $\frac{1}{3} \times 1600 \times 5.5 = 2,870$ "
" " 4^m " " $\frac{1}{3} \times 1600 \times 3.5 = 1,870$ "



Reinforcements

(A) at bottom. moment $= \frac{4000 \times 3.15^2}{12} = 3.305^{kgm}$
(B) at 2^m from bott. $m = \frac{2870 \times 3.15^2}{12} = 2,375$ "
(C) at 4^m " " $m = \frac{1870 \times 3.15^2}{12} = 1,545$ "

$$\text{Steel area req'd for (A)} = \frac{3305 \times 100}{1200 \times \frac{7}{8} \times 42} = 7.5^{□cm}$$

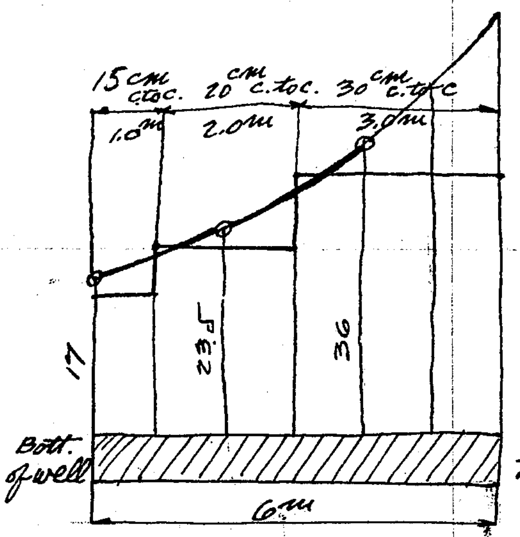
Use $\frac{1}{2}$"ϕ, spacing $= \frac{1.267 \times 100}{7.5} = 17^{cm}$

$$\text{Steel area req'd for (B)} = \frac{2,375 \times 100}{1200 \times \frac{7}{8} \times 42} = 5.39^{□cm}$$

Use $\frac{1}{2}$"ϕ spacing $= \frac{1.267 \times 100}{5.39} = 23.5^{cm}$ c. to c.

$$\text{Steel area req'd for (C)} = \frac{1.545 \times 100}{1200 \times \frac{7}{8} \times 42} = 3.51^{□cm}$$

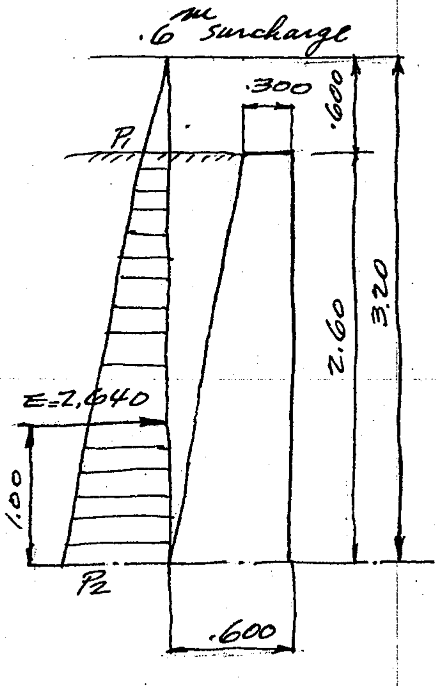
Use $\frac{1}{2}$" spacing $= \frac{1.267 \times 100}{3.51} = 36^{cm}$ c. to c.



Reinf. spacing diagram

Design of Abutment for steel span
Parapet wall Case 1 At normal state

Earth pressure $p_1 = \frac{1}{3} \times .6 \times 1600 = 320^{kg/m^2}$
" $p_2 = \frac{1}{3} \times 3.2 \times 1600 = 1,710$ "
" $E = \frac{320 + 1,710}{2} \times 2.6 = 2.640^{kg/m}$ strip



$$\text{Pt. of application of } E = \frac{2.6(1710 + 640)}{3(1710 + 320)} = 1.0^m$$

Moment $= 2,640 \times 1. = 2640$ kgm.

Use $\frac{1}{2}$"ϕ bars 30 cm spacing $= 4.22^{cm^2}$/meter strip

or $p = \frac{1.267}{57 \times 30} = .00074$

$k = 0.141 \qquad j = .952$

$$f_s = \frac{2640 \times 100}{4.22 \times .952 \times 57} = 1,152^{kg/m^2} \text{ Ok}$$

$$f_c = \frac{1152 \times .141}{15 \times .859} = 12.6^{kg/m^2} \text{ Ok}$$

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CALCULATIONS FOR

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Case 2 During earthquake $k=0.2$

Earth pressure $= \frac{wh^2}{2} \times .48 = .24 \times 1600 \times 2.6^2 = 2,600$ kg

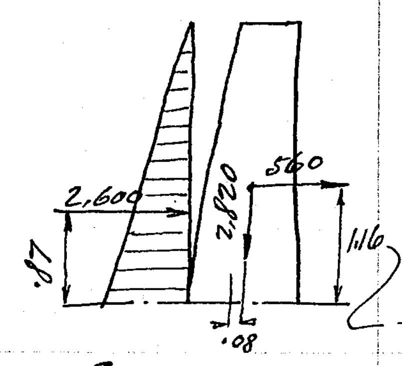
wt of wall $= .45 \times 2.6$ @ $2400 = 2,820$ kg

Moment $= 2600 \times .87 = 2,260$

$2820 \times .08 = 226$

$560 \times 1.16 = 650$

$3,136$



$f_s = \frac{3136 \times 100}{4.22 \times .952 \times 57} = 1370$ kg/cm² OK

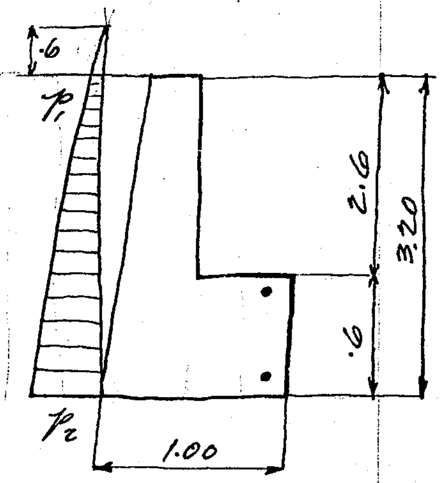
$\frac{2.6(.6+.6)}{3\ (.6+.3)} = 1.16$

Curtain wall Case 1 At normal state

Earth pressure $p_1 = 320$ kg/m²

$p_2 = \frac{1}{3} \times 3.8 \times 1600 = 2028$

$E = \frac{320+2028}{2} \times 3.2 = 3760$ kg/lin meter



Moment $= \frac{1}{10} wl^2 = \frac{1}{10} \times 3760 \times 4^2 = 6,020$ kgm

Steel area req'd $= \frac{6020 \times 100}{1200 \times \frac{7}{8} \times 57} = 10.05$ □cm

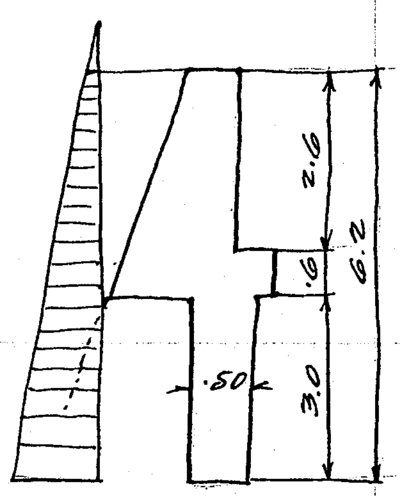
Use 5-5/8"φ = 9.90

Earth pressure at bottom $= \frac{1}{3} \times 6.8 \times 1600 = 3,620$ kg/□m

" " at 1.5m from bottom $= \frac{1}{3} \times 5.3 \times 1,600 = 2,820$ "

Moment at bott. $= \frac{1}{10} \times 3,620 \times 4^2 = 5,800$ kgm

" at 1.5m from " $= \frac{1}{10} \times 2,820 \times 4^2 = 4,480$ "



Steel req'd at bottom $= \frac{5800 \times 100}{1200 \times \frac{7}{8} \times 47} = 11.75$ cm²

Use 3/4"φ Spacing $= \frac{100 \times 2.85}{11.75} = 25$ cm. use 25cm spacing
take care for bond stress.

Steel req'd at 1.5m from bott. $= \frac{4480 \times 100}{1200 \times \frac{7}{8} \times 47} = 9.08$ cm²

Spacing $= \frac{100 \times 2.85}{9.08} = 31.5$ cm use 30cm spacing

Max end shear at bottom $= 3620 \times 2.0 = 7240$ kg

Unit shear $= \frac{7,240}{100 \times \frac{7}{8} \times 47} = 1.76$ kg/cm² OK

Unit bond $= \frac{7,240}{6 \times \frac{7}{8} \times 47} = 29.4$ kg/cm² for one bar OK

Case 2 During earthquake $k=.2$

Earth pressure $= .48 \times 1600 \times 6.2 = 4760$ kg/□m

Max moment $= \frac{1}{10} \times 4760 \times 4^2 = 7,610$ kgm

End shear $= 4,760 \times 2 = 9520$ kg

Steel req'd $= \frac{7610 \times 100}{2160 \times \frac{7}{8} \times 47} = 8.57$ cm² OK

Unit shear $= \frac{9,520}{100 \times \frac{7}{8} \times 47} = 2.32$ kg/cm² OK

bond $= \frac{9,520}{6 \times \frac{7}{8} \times 47} = 38.6$ kg/□cm for one bar

Spacing req'd for bond $= \frac{100 \times 9}{38.6} = 23.3$ cm OK

Use dowels to anchor wall to caisson
and the spacing of bars for curtain wall
at bottom shall be 30cm.

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CALCULATIONS FOR

Katsura Bashi for Kyotofu

Shaft Case 1 Normal state

Load on shaft

Super imposed load on shaft

Dead load = 25,410

Live load = 25,040

total = 50,450 kg

Approx. weight and c.g. of abutment structure above top of caisson.

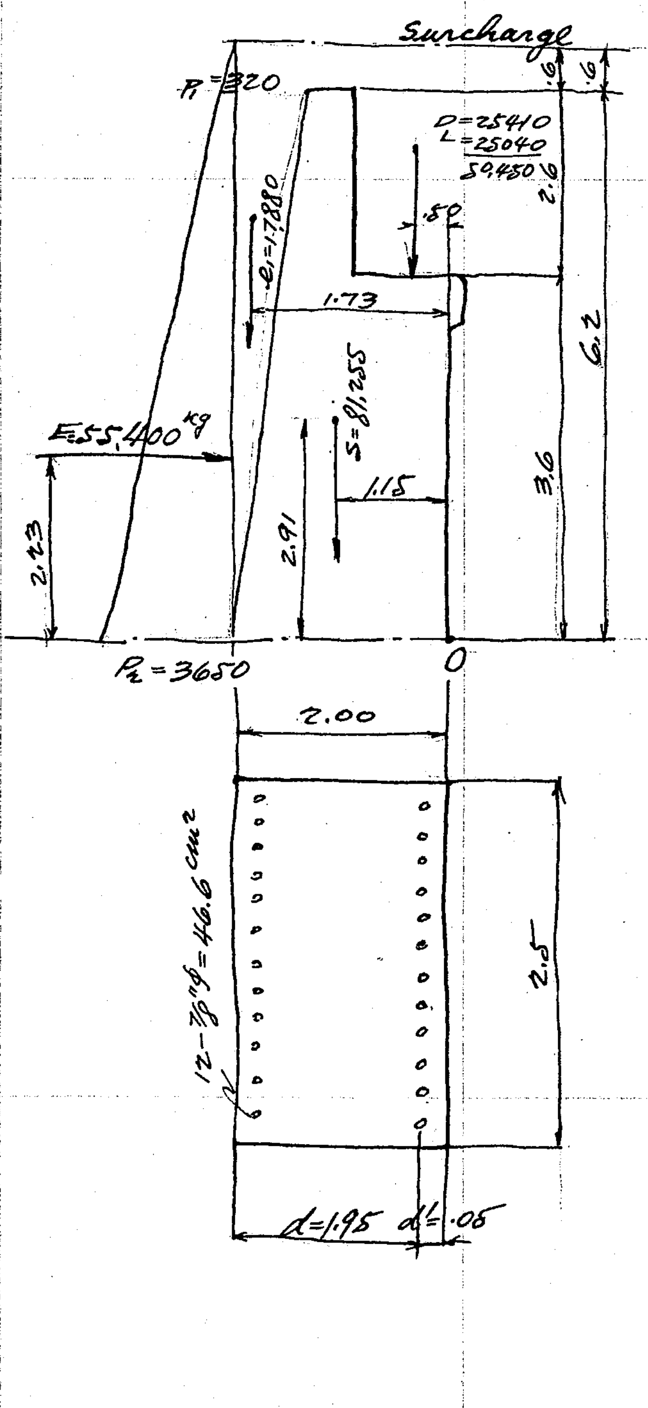
	dimension	weight	Hor. Lev. arm	Hor. M. ab't toe.	Vert. lev. arm	Vert. m. ab't toe.
Parapet wall	.5×2.6×3.75@2400 =	11,700	1.04	12,180	4.80	56,150
Curtain wall	.6×1.0×4.0@2400 =	5,760	1.0	5,760	3.35	19,300
" "	.5×3.05×4.@2400 =	11,000	.75	8,250	1.53	16,830
Shaft	1.7×3.6×2.5@2400 =	36,700	.86	31,600	1.70	62,400
Column	.5×4.6×½×2.5@2400 =	2,900	1.60	4,640	5.00	14,500
Wing wall	.6×.6×6.2@2400 =	5,355	2.10	11,250	3.10	16,620
"	.3×1.5×2.8@2400 =	3,020	3.40	10,280	5.45	16,480
Light pedestal	.8×.8×2.0@2400 =	3,070	1.30	3,990	7.20	22,100
Handrail	500 kg × 3.5 =	1,750	3.30	5,770	6.70	11,720
	Total wt.	81,255		$\Sigma M_h = 93,720$		$\Sigma M_v = 236,100$
				$\div 81,225 = 1.15$ m		$\div 81,225 = 2.91$ m
				behind the toe of shaft		above top of caisson

Earth pressure $= p_1 = \frac{1}{3} \times 1600 \times 0.6 = 320$ kg/m²

" $= p_2 = \frac{1}{3} \times 1600 \times 6.8 = 3650$ "

$\therefore \quad E = \frac{320+3650}{2} \times 6.2 = 12,300 \text{ kg} \times 4.5 = 55,400 \text{ kg}$

$e_1 = \frac{.8 \times 6.2}{2} \times 4.5 \times 1600 = 17,880$ kg.



Taking moment about 0

Loads	Hor. loads	Vert. loads	lever arm	moment ab't 0
Sup. imposed load		50,450	0.5	25,230
S		81,255	1.15	93,500
e_1		17,880	1.73	30,930
E	55,400		−2.23	123,500
		149,585		26,160

Eccentricity $= 1.0 - \frac{26,160}{149,585} = 0.825$ m

Bending moment at bottom of shaft

$= 149,585 \times .825 = 123,400$ kgm

$\frac{e}{h} = \frac{.825}{2.0} = .413 \quad \frac{d'}{h} = \frac{.05}{2.0} = .025$

$p_0 = 2p = \frac{46.6 \times 2}{200 \times 250} = .00187$

$k' = .45 \quad k'' = .38 \quad k = .45 - 5 \times .025 \times .07 = .441$

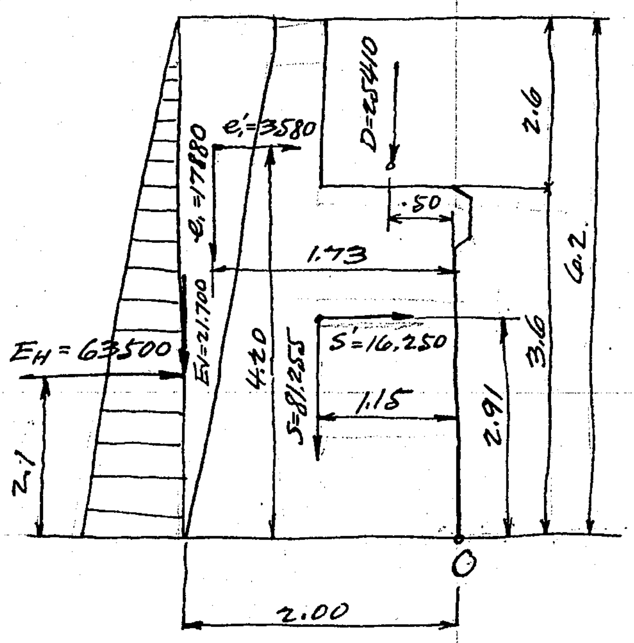
$A_2 = .076 \quad B_2 = .0775 \quad C_2 = 0.187 \times .076 + .0775 = .0917$

$f_c = \frac{123,400 \times 100}{.0917 \times 250 \times 200^2} = 13.45$ kg/cm² ok

$f_s = n f_c \left(\frac{d}{kh} - 1\right) = 15 \times 13.45 \left(\frac{1.95}{.441 \times 2} - 1\right) = 244.4$ kg/cm² ok

Katsura Bashi for Kyotofu

Case 2 During Earthquake $k=0.20$



Earth pressure $= \frac{.148 \times 1600 \times 6.2^2}{2} \times 4.5 = 66,400^{kg}$

$EV = .0785 \times 1600 \times 6.2^2 \times 4.5 = 21,700$

$EH = .2293 \times 1600 \times 6.2^2 \times 4.5 = 63,500$

Moment about toe O

Loads	Hor. loads	Vert. loads	Arm	Moment
D		25,410	−.5	−12,700
S		81,255	−1.15	−93,400
S'	16,250		+2.91	+47,300
e_1		17,880	−1.73	−30,900
e_1'	3,580		+4.20	+15,030
EH	63,500		+2.10	+133,300
EV		21,700	−2.00	−43,400
	ΣH = 83,330	ΣV = 146,245		ΣM = +15,230

Eccentricity $= 1.0 + \frac{15,230}{146,245} = 1.04^m$

Moment at bottom of shaft $= 146,245 \times 1.04 = 152,200^{kgm}$

$\frac{e}{h} = \frac{1.04}{2} = .520 \quad \frac{d'}{h} = .025 \quad p_0 = 0.00187$

$k' = .34 \quad k'' = .28 \quad k = .34 - 5 \times .025 \times .06 = .333$

$A_2 = .101 \quad B_2 = .065 \quad C_2 = .187 \times .107 + .065 = .085$

$f_c = \frac{165800 \times 100}{.085 \times 250 \times 200^2} = 19.5\ kg/cm^2$ Ok

$f_s = 15 \times 19.5 \left(\frac{1.95}{.333 \times 2} - 1\right) = 564\ kg/cm^2$ Ok

Foundation Case 1

Stability of abutment during earthquake toward river side

Caisson $4.2^m \times 3.6^m$ 3.5^m deep

$e_2 = 3.6 \times .4 \times 6.2 \times 1600 = 14,300^{kg}$

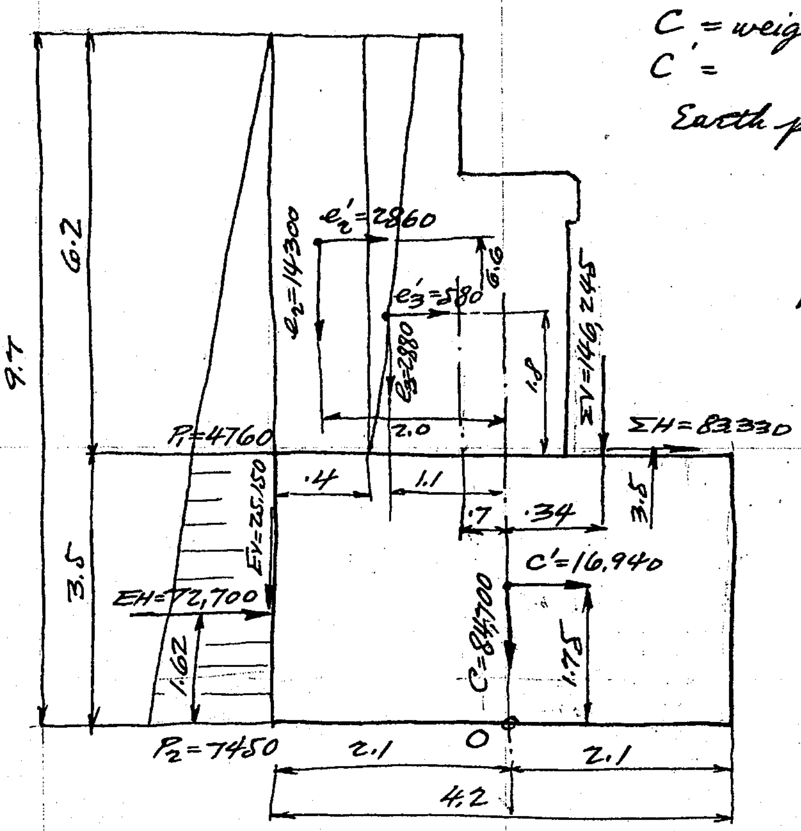
$e_2' = 2,860^{kg}$

C = weight of foundation $= 4.2 \times 3.6 \times 3.5 \times 1600 = 84,700^{kg}$

$C' = 16,940^{kg}$

Earth pressure $p_1 = .48 \times 1600 \times 6.2 = 4,760\ kg/m^2$

$p_2 = .48 \times 1600 \times 9.7 = 7,450$ "



$E = \frac{4760 + 7450}{2} \times 3.5 \times 3.6 = 76,900^{kg}$

$EH = 76,900 \times .945 = 72,700$

$EV = 76,900 \times .327 = 25,150$

Taking Moment about O

Loads	Hor. loads	Vert. loads	arm	moment abt O
e_2		14,300	−2.0	−28,600
e_2'	2,860		+6.6	+18,900
e_3		2,880	−1.1	−3,100
e_3'	580		+5.3	+3,100
V		146,245	+.34	+49,700
H	83,330		+3.50	+291,500
C		84,700	0	0
C'	16,940		+1.75	+29,600
EV		25,150	−2.10	−52,800
EH	72,700		+1.62	+117,700
	ΣH = 176,410	ΣV = 273,275		ΣM = +426,000

Eccentricity $= 1.55 \qquad a = 2.1 - 1.56 = .54$

Neglecting tension at heel

Max toe pressure $= \frac{2 \times 270,395}{3 \times .54 \times 3.6} = 94,000\ kg/m^2 = 8.6\ tons/\square'$ Ok

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MADE BY K. Inaba DATE 2-7-30 FILE NO.
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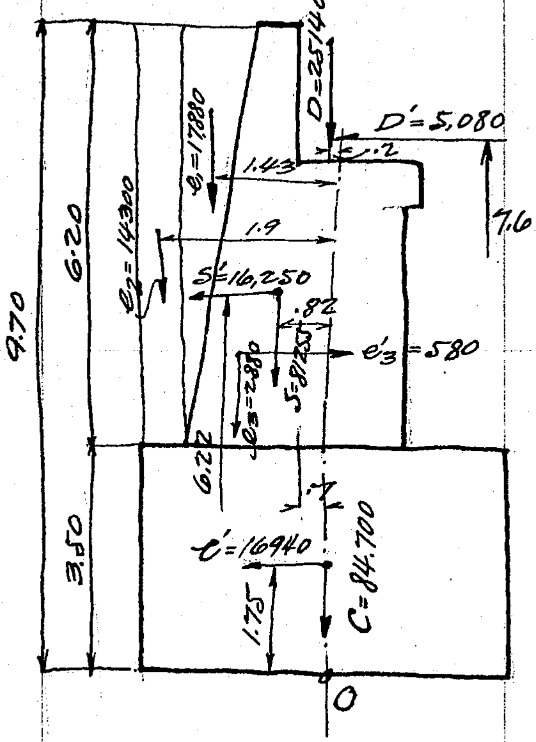
CALCULATIONS FOR

Katsura Bashi for Kyotofu

Case 2 Earthquake toward bank
Taking moment about O



Loads	Hor. loads	Vert. loads	arm	moment abt O
D		25.410	+2.0	+ 5,100
D'	5,080		+7.6	+ 38,600
e_1		17,880	+ 1.43	+25,600
e_2		14,300	+1.9	+27,150
e_3		2,880	−1.1	− 3,100
e_3'	580		+5.3	+ 3,100
S		81,255	+ 0.82	+ 66,600
S'	16,250		+ 6.22	+101,000
C		84,700	0	0
C'	16,940		+1.75	+ 29,600
	$\Sigma H = 38,850$	$\Sigma V = 226,425$ kg.		$\Sigma M = +293,650$ kgm

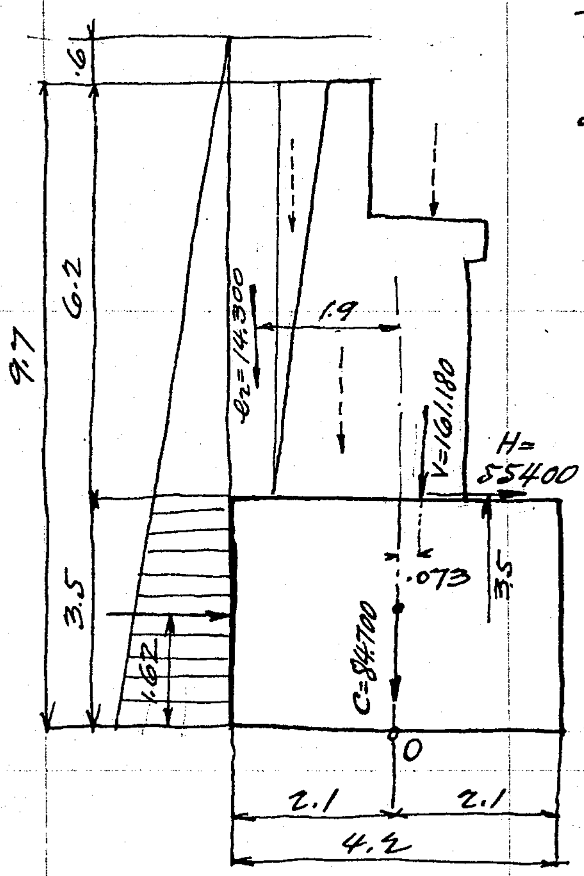
$$\text{Eccentricity} = \frac{293,650}{226,425} = 1.298^{m} \qquad a = 2.1 - 1.298 = 0.802^{m}$$

Neglecting tension at heel

$$\text{Max toe pressure} = \frac{2 \times 226,425}{3 \times .802 \times 3.6} = 52,200 \ \text{kg/m}^2 = 4.77 \ \text{Tons/□'}$$

OK

Case 3 At normal state
Taking moment abt O



Loads	Hor. loads	Vert. loads	arm	moment abt O
V		149.585	+0.073	+ 10,910
H	55,400		+ 3.50	+194,000
e_2		14,300	− 1.90	− 27,200
e_3		2,880	−1.1	− 3,100
C		84.700	0	0
E	57,800		+1.62	+ 93,600
	$\Sigma H = 113,200$	$\Sigma V = 251.465$ kg		$\Sigma M = + 268,210$ kgm

$$\text{Eccentricity} = \frac{268,210}{251.465} = 1.07^{m} \qquad a = 2.1^{m} - 1.07^{m} = 1.03^{m}$$

Neglecting tension at heel

$$\text{Max toe pressure} = \frac{2 \times 251,465}{3 \times 1.03 \times 3.6} = 45,200 \ \text{kg/m}^2 = 4.14 \ \text{tons/□'}$$

OK

$$\text{Earth press. } p_1 = \frac{1600}{3} \times 6.8 = 3,630$$

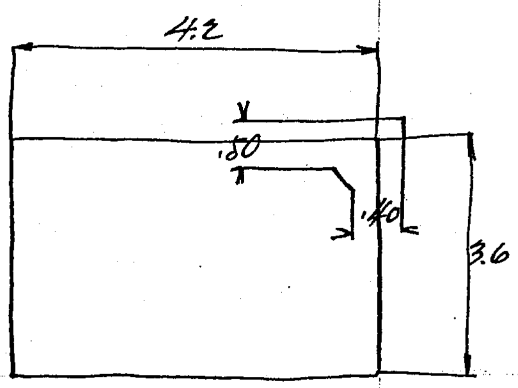
$$'' \quad p_2 = \frac{1600}{3} \times 10.3 = 5,490$$

$$X = \frac{3.5\,(5490 + 7260)}{3\,(5490 + 3630)} = 1.62$$

$$E = \frac{1}{2} \times (3630 + 5490) \times 3.5 \times 3.6 = 57.800$$

Katsura Bashi for Kyotofu

Design of Well $3.6 \times 4.2 \times 3.5^{m}$ deep



(A) Earth press. at bottom $\frac{1}{3} \times 16000 \times 6 = 3200 \ kg/m^2$

(B) " " 1.5^{m} above bottom $\frac{1}{3} \times 16000 \times 4.5 = 2{,}400$

(C) " " 3.0^{m} " " $\frac{1}{3} \times 16000 \times 3.0 = 1{,}600$

Bending moment Long side

A moment $= \frac{3200 \times 3.7^2}{12} = 3.650 \ kgm$ or $\frac{3200 \times 3.15^2}{12} = 2.645 \ kgm$

B $= \frac{2400 \times 3.7^2}{12} = 2{,}740$ $\frac{2400 \times 3.15^2}{12} = 1{,}985$

C $= \frac{1{,}600 \times 3.7^2}{12} = 1.825$ $\frac{1600 \times 3.15^2}{12} = 1{,}322$

Reinforcements

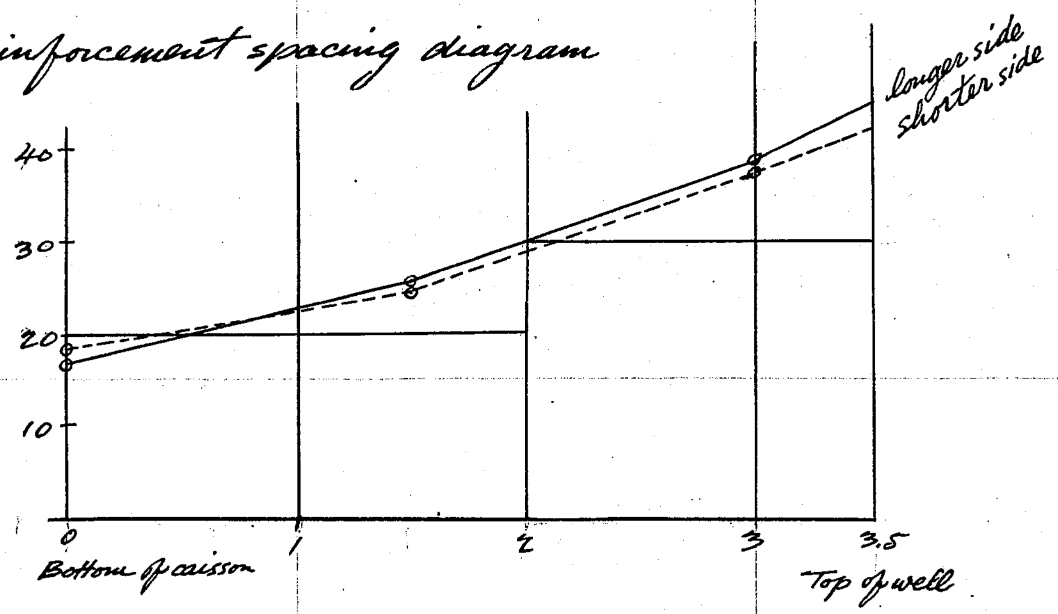
Longer side / shorter side

A $\frac{.3650 \times 100}{1200 \times \frac{7}{8} \times 37} = 9.4$ $\frac{5}{8}''\phi$ $\frac{1.267 \times 100}{9.4} = 21.^{cm}$ spacing $\frac{2{,}645 \times 100}{1200 \times \frac{7}{8} \times 37} = 6.82$ $\frac{1}{2}''\phi$ $\frac{1.267 \times 100}{6.82} = 18.6^{cm}$ spacing

B $\frac{2400 \times 100}{1200 \times \frac{7}{8} \times 37} = 6.18$ $\frac{''}{6.18} = 32''$ " $\frac{1.985 \times 100}{''} = 5.06$ $\frac{''}{5.06} = 25.$ "

C $\frac{1600 \times 100}{1200 \times \frac{7}{8} \times 37} = 4.12$ $\frac{''}{4.12} = 48''$ " $\frac{1.322 \times 100}{''} = 3.4$ $\frac{''}{3.4} = 37.5$ "

Reinforcement spacing diagram



shear $= 3200 \times 3.7 = 11{,}800 \ kg$

unit shear $= \frac{11.800}{100 \times \frac{7}{8} \times 37} = 3.65 \ kg/cm^2$ OK

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MADE BY R. I DATE 2-7-24 FILE NO.
CHECKED BY M. K DATE 2-7-26 PAGE NO. M 1

CALCULATIONS FOR

List of Materials for Katsura-Bashi for Kyoto-Fu

MAIN GIRDER G1 R/L 4 Reqd

1	Web Pls.	78" × 7/16"	17'10"	@ 116.	2,069
2	Flg Ls	6" × 6" × 5/8"	17'10	@ 24.2	863
2	" "	"	18'1"	" 24.2	875
4	Ls	5" × 5" × 3/8"	6'5 7/8"	" 12.3	319
2	"	5" × 3½" × 3/8"	5'1½"	" 10.4	107
2	Fills.	19" × 5/8"	5'6¼"	" 40.38	446
2	Ls	5" × 3½" × 3/8"	5'9"	" 10.4	120
2	Fills.	3½" × 5/8"	4'9¾"	" 7.44	72
2	Ls	5" × 3½" × 3/8"	5'0¾"	" 10.4	105
2	Fills	3½" × 5/8"	4'1¾"	" 7.44	61
2	Ls	5" × 3½" × 3/8"	4'7½"	" 10.4	96
1	Fills	7¾" × 5/8"	3'8¼"	" 16.47	61
1	"	3½" × 5/8"	3'8¼"	" 7.44	27
2	Ls	5" × 3½" × 3/8"	4'5¼"	" 10.4	92
2	Fills.	3½" × 5/8"	3'6½"	" 7.44	53
1	Cov. Pl.	14" × ½"	9'8"	" 23.8	230
1	Cov. Pl.	14" × ½"	2'11 5/8"	" 23.8	71
1	Cov. Pl.	14" × ½"	16'6"	" 23.8	393
1	Cov. Pl.	14" × ½"	7'11¼"	" 23.8	190
1	L	4" × 3½" × 3/8"	2'10"	" 9.1	26
1	L	"	3'5½"	" "	31
1	L	"	3'6"	" "	32
1	L	"	3'1 7/8"	" "	29
1	Pl.	19" × 5/16"	1'11¾"	" 20.19	40
1	"	18¾" × 5/16"	1'11 3/8"	" 19.93	39
1	Fill.	8¾" × 3/8"	2'7"	" 11.16	29
1	"	8¾" × 3/8"	1'4 3/8"	" 11.16	15
429	Shop Rivets. 7/8"φ	grip 1 7/8" abt.		@ 0.86	369
164	Field Riets 7/8"φ	" 2¼" "		" 0.92	151
					7011 × 4 = 28,041

MAIN GIRDER G2 R/L 4 Req'd.

2	Web. Pls.	54" × 7/16"	23'1¾"	@ 80.3	3,718
4	Flg Ls	6" × 6" × 5/8"	23'1¾"	" 24.2	2,241
4	"	"	23'1¾"	" "	2,241
24	Ls	5" × 3½" × 3/8"	4'5¼"	" 10.4	1,108
20	Fills.	3½" × 5/8"	3'6½"	" 7.44	527
4	"	7¾" × 5/8"	3'6½"	" 16.47	233
2	Cov. Pls.	14" × ½"	23'3½"	" 23.8	1,109
2	"	"	7'6¾"	" "	360
2	"	"	9'11¼"	" "	473
2	"	"	19'8¼"	" "	937
2	"	"	11'3¾"	" "	538
4	"	"	23'1¾"	" "	2,204
2	"	"	19'8"	" "	936
2	Ls	6" × 4" × 3/8"	1'4"	" 12.3	33
2	"	"	1'6"	" "	37
2	"	"	1'6¾"	" "	38
2	"	"	1'2"	" "	29
2	Pls.	18¾" × 5/16"	1'11 3/8"	" 19.93	79
2	"	"	2'0 3/8"	" "	81
1124	Shop Rivets 7/8"φ	grip abt. 2¼"		@ 0.92	1031
208	Field Rivets 7/8"φ	" " 2"		" 0.88	181
					18134 × 4 = 72,536

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY M. K DATE 2-7-25 FILE NO.
CHECKED BY Y. H DATE 2-7-26 PAGE NO M2

CALCULATIONS FOR

List of Materials for Katsura-Bashi for Kyoto-fu

MAIN GIRDER G3 R/L — 6 Req'd.

1	Web. Pl.	78" × 7/16"	22'8 3/8"	@ 116.	2,633
1	〃	64" × 7/16"	16'10 5/16"	〃 95.2	1,605
2	Flg ∟s	6" × 6" × 5/8	22'8 3/8"	〃 24.2	1,099
2	〃	〃	15'11 1/16"	〃 〃	773
2	〃	〃	23'0"	〃 〃	1113
2	〃	〃	16'11 1/2"	〃 〃	821
4	∟s	5" × 5" × 3/8"	6'5 1/4"	〃 12.3	317
2	〃	5" × 3 1/2 × 3/8"	6'5 1/4"	〃 10.4	134
2	Fills.	19" × 5/8	5'6 1/4"	〃 40.38	446
4	∟s	5" × 3 1/2 × 3/8	5'9"	〃 10.4	239
4	〃	〃	5'0 3/4"	〃 〃	210
4	〃	〃	4'7 1/2"	〃 〃	193
8	〃	〃	4'5 1/4"	〃 〃	369
6	Fills.	3 1/2 × 5/8"	3'6 1/2"	〃 7.44	158
2	〃	19 3/4" × 5/8"	3'6 1/2"	〃 41.97	297
2	Fills.	7 3/4" × 5/8"	3'8 1/4"	〃 16.47	122
2	〃	3 1/2" × 5/8"	3'8 1/4"	〃 7.44	54
4	〃	〃	4'1 3/4"	〃 〃	124
4	〃	〃	4'9 3/4"	〃 〃	143
2	∟s	4" × 3 1/2 × 3/8"	2'10"	〃 9.1	52
4	〃	〃	1'2 1/2"	〃 〃	44
2	〃	〃	3'6"	〃 〃	64
2	〃	〃	3'2"	〃 〃	58
1	Pl.	19" × 5/16"	3'3 3/4"	〃 20.19	67
2	Pls.	23 1/2" × 5/16"	1'10"	〃 24.97	91
1	Pl.	16" × 5/16"	1'9 3/4"	〃 17.00	31
1	∟	6" × 4" × 3/8"	1'1"	〃 12.3	13
4	∟s	3 1/2 × 3 1/2 × 3/8"	2'0 1/2"	〃 8.5	69
2	〃	4" × 3 1/2" × 3/8"	1'0"	〃 9.1	18
2	〃	6" × 4" × 1/2"	0'10"	〃 16.2	27
2	Fills.	10" × 1/2"	1'7 1/2"	〃 17.0	55
1	Pl.	15" × 1/2"	1'10 5/8"	〃 25.5	48
1	Cov. Pl.	14" × 1/2"	24'8 1/4"	〃 23.8	588
1	〃	〃	25'0"	〃 〃	595
1056	Shop Rivets 7/8"φ	grip ab't	2"	@ 0.88	929
154	Field Rivets 7/8"φ	〃 〃	2	〃 〃	136
					13,735 × 6 = 82,410

MAIN GIRDER G4 R/L — 6 Req'd

2	Web. Pls.	54" × 3/8"	18'6 1/4"	@ 68.9	2,552
2	Flg ∟s	6" × 6" × 1/2"	37'0 5/16"	〃 19.6	1,452
2	〃	〃	35'2 1/16"	〃 〃	1,379
2	Spl. pls.	12 1/2" × 1/2"	2'5 3/4"	〃 21.25	105
4	〃	6 5/8" × 1/2"	2'0 1/2"	〃 10.42	85
8	∟s	3 1/2 × 3 1/2 × 3/8	2'0 5/8"	〃 8.5	139
4	Fills	10" × 1/2"	1'7 1/4"	〃 17.0	109
4	〃	19 3/4" × 1/2"	3'6 1/2"	〃 33.58	475
20	∟s	5" × 3 1/2" × 3/8"	4'5 1/4"	〃 10.4	924
14	Fills.	3 1/2" × 1/2"	3'6 1/2"	〃 5.95	295
2	〃	7 3/4" × 1/2	3'6 1/2	〃 13.18	93
2	∟s	6" × 4" × 3/8"	1'1"	〃 12.3	27
4	∟s	6" × 4" × 1/2"	0'10"	〃 16.2	64
4	〃	4" × 3 1/2" × 3/8"	1'0"	〃 9.1	36

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MADE BY M. K DATE 2-7-26 FILE NO ______
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CALCULATIONS FOR

List of Materials for Katsura-Bashi for Kyoto-fu

2	Pls.	18¾" × 5/16"	2' 1"	@ 19.93	83
2	Pls.	15 3/8" × 5/16"	1' 9¼"	@ 16.87	61
1	Pls.	15" × 1/2"	1' 11"	@ 25.5	98
2	Ls	6" × 4" × 3/8"	1' 6¾"	〃 12.3	38
2	〃	〃	1' 6½"	〃 〃	38
492	Shop Rivets 7/8"	grip ab't 1 3/8"		〃 0.86	423
64	Field Rivets 7/8"	〃 〃 1 3/8"		〃 0.69	44
					8,510 × 6 = 51,060

MAIN GIRDERS G5 R/L 6 Req'd.

1	Web Pl.	78" × 7/16"	22' 8 3/8"	@ 116.	2,633
1	〃	64" × 7/16"	16' 10 7/16"	〃 95.2	1,605
2	Flg. Ls	6" × 6" × 5/8"	22' 8 3/8"	〃 24.2	1,099
2	〃	〃	15' 10 7/16"	〃 〃	768
2	〃	〃	16' 11½"	〃 〃	821
2	〃	〃	23' 0"	〃 〃	1,113
1	Cov. Pl.	14" × 1/2"	24' 8¼"	〃 23.8	588
1	〃	〃	25' 0"	〃 〃	595
4	Ls	5" × 5" × 3/8"	6' 5¼"	〃 12.3	317
2	〃	5" × 3½" × 3/8"	6' 5¼"	〃 10.4	134
2	Fills.	19" × 5/8"	5' 6¼"	〃 40.38	446
4	Ls	5" × 3½" × 3/8"	5' 9"	〃 10.4	239
4	〃	〃	5' 0¾"	〃 〃	210
4	〃	〃	4' 7½"	〃 〃	193
8	〃	〃	4' 5¼"	〃 〃	369
4	Fills.	3½" × 5/8"	4' 9¾"	〃 7.44	143
4	〃	〃	4' 1¾"	〃 〃	124
2	〃	〃	3' 8¼"	〃 〃	54
6	〃	〃	3' 6½"	〃 〃	158
2	〃	7¾" × 5/8"	3' 8¼"	〃 16.47	122
2	〃	19¾" × 5/8"	3' 6½"	〃 41.97	297
2	Ls	4" × 3½" × 3/8"	2' 10"	〃 9.1	52
4	〃	〃	1' 2½"	〃 〃	44
2	〃	〃	3' 6"	〃 〃	64
2	〃	〃	3' 2"	〃 〃	58
1	Pl.	19" × 7/16"	3' 3¾"	〃 20.19	67
2	Pls.	23½" × 7/16"	1' 10"	〃 24.97	91
1	Pl.	16" × 7/16"	1' 9¾"	〃 17.0	31
1	L	6" × 4" × 3/8"	1' 1"	〃 12.3	13
4	Ls	3½" × 3½" × 3/8"	2' 0½"	〃 8.5	69
2	〃	6" × 4" × 1/2"	0' 10"	〃 16.2	27
2	〃	4" × 3½" × 3/8"	1' 0"	〃 9.1	18
2	Fills.	10" × 1/2"	1' 7½"	〃 17.0	55
1	Pl.	15" × 1/2"	1' 11"	〃 25.5	49
1056	Shop Rivets 7/8"	grip ab't.	2"	〃 0.88	949
154	Field Rivets 7/8"	〃 〃	2"	〃 0.88	136
					13,751 × 6 = 82,506

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SEIYU BLDG, TOKIO

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CALCULATIONS FOR

List of Materials for Katsura-Bashi for Kyoto-fu

MAIN GIRDER GG$^{R}_{L}$ 4 Req'd

2	Web Pls.	54" × 7/16"	23'1¾"	@ 80.3	3,718
4	Flg Ls	6" × 6" × 5/8"	23'1¾"	〃 24.2	2,241
4	〃	〃	23'1¾"	〃 〃	2,241
2	Cov. Pls.	14" × ½"	19'8"	〃 23.8	936
2	〃	〃	19'8"	〃 〃	936
2	〃	〃	7'2½"	〃 〃	343
2	〃	〃	23'1¾"	〃 〃	1,102
2	〃	〃	23'1¾"	〃 〃	1,102
24	Ls	5" × 3½" × 3/8"	4'5¾"	〃 10.4	1,108
20	Fills.	3½" × 5/8"	3'6½"	〃 7.44	527
4	Fills.	7¾" × 5/8"	3'6½"	〃 16.47	233
2	Ls	6" × 4" × 3/8"	1'4"	〃 12.3	33
2	〃	〃	1'6"	〃 〃	37
2	〃	〃	1'6¾"	〃 〃	38
2	〃	〃	1'2"	〃 〃	29
2	Pls.	18¾" × 7/16"	1'11⅜"	〃 19.93	79
2	〃	〃	2'0⅜"	〃 〃	81
1156	Shop Rivets ⅞"φ	grip ab't	2"	〃 0.88	1017
208	Fied Rivets ⅞"φ	〃 〃	2"	〃 〃	183
					15,984 × 4 = 63,936

SPLICE SC1 24 Req'd

4	Ls	6" × 6" × 5/8"	3'5½"	@ 24.2	335
2	Fills	14" × ½"	3'5½"	〃 23.8	165
4	Pls.	6" × 5/8"	2'0½"	〃 12.75	104
2	〃	12½" × 5/8"	2'5¾"	〃 26.56	132
					736 × 24 = 17,664

SPLICE SC2 12 Req'd.

2	Ls	6" × 6" × 5/8"	3'5½"	@ 24.2	167
2	Pls.	6" × 5/8"	2'0½"	〃 12.75	52
2	〃	12½" × 5/8"	3'5½"	〃 26.56	184
2	〃	6¼" × 5/8"	2'2½"	〃 13.28	59
2	Ls	6" × 6" × 5/8"	3'7½"	〃 24.2	176
					638 × 12 = 7,656

Total of Main girders = 405,809.

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MADE BY Y. H. DATE 2-7-26 FILE NO
CHECKED BY R. I DATE 2-7-27 PAGE NO M5

CALCULATIONS FOR

LATERAL BRACING

86	Ls	3" × 3 × 5/16"	10' 7 3/4"	@	6.1	5,585
86	"	"	10' 2 3/4"	"	"	5,366
86	"	5" × 3" × 5/16"	21' 10 1/2"	"	8.2	15,425
12	"	"	20' 7 1/2"	"	"	2,030
12	"	3" × 3" × 5/16"	9' 11 1/8"	"	6.1	727
12	"	"	9' 6 1/8"	"	"	696
43	Pls.	12" × 5/16"	2' 3"	"	12.75	1234
6	"	11 1/2" × 5/16"	2' 5 1/4"	"	"	179
2818	Shop Rivets 3/4"	grip abt 3/4"		"	0.43	1,212
2448	Field Rivet 3/4"	" 3/4"		"	0.43	1,053
						33,507

Total of Lateral bracing = 33,507.

STRINGER

6	Is	12" × 5"	12' 3"		31.99	2,350
105	"	"	11' 5"		"	38,349
9	"	"	12' 8 1/2"		"	3,660
9	"	"	10' 2 1/4"		"	2,938
9	"	"	10' 1"		"	2,908
9	"	"	12' 8 1/2"		"	3,660
252	Pls.	9 1/2" × 5/16"	1' 1"		10.09	2,754
1548	Fied Rivets 3/4"	grip abt. 1"		@	0.46	712
						57,331

Total of Stringer = 57,331.

FLOOR BEAMS FB2~~-FB6~~ 48 Req'd.

2	Flg Ls	4" × 3 1/2 × 3/8	20' 8 3/4"		9.1	377
2	"	"	20' 0"		"	364
2	Ls	"	0' 9 3/8"		"	15
2	Pls.	7 1/2" × 5/16"	0' 9 3/8"		7.97	13
2	Ls	5" × 3 1/2" × 3/8"	3' 0 3/4"		10.4	64
2	Fills	5" × 3/8"	2' 6 1/4		6.38	32
1	Web. pl.	37" × 5/16"	20' 8 3/4"		39.31	815
3	Pls.	8 3/8" × 1/2"	1' 0"		14.24	43
14	Ls	3" × 3" × 5/16"	3' 1 1/2"		6.1	267
231	Shop Rivets. 3/4"	grip abt 1 1/2"		@	0.54	125
33.5	Field Rivets. 3/4"	" " 1"		"	0.46	15
28	Field Rivets 7/8"	" " 1 1/4		"	0.5	14
						2144 × 48 = 102,912

FLOOR BEAMS FB1 8 Req'd

4	Fills.	4" × 3 1/2" × 3/8"	20' 8 3/4"		9.1	775
1	Web Pl.	37" × 5/16"	20' 8 3/4"		39.31	815
3	Pls.	8 3/8 × 1/2	1' 0"		14.24	43
14	Ls	3" × 3" × 5/16"	3' 1 1/2"		6.1	267
4	Ls	5" × 3 1/2" × 3/8	3' 0 3/4"		10.4	128
4	Fills	5" × 3/8	2' 6 1/4"		6.38	64
243	Shop Rivets 3/4"	grip ab't. 1 1/2"		@	0.54	131
52	Field Rivets 3/4"	" " 1		"	0.46	24
						2227 × 8 = 17,816

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MADE BY M. K DATE 2-8-2 FILE NO

CHECKED BY R. I DATE 2-8-2 PAGE NO M6

CALCULATIONS FOR

BRACKET BR1 — 12 Req'd.

2	Ls	5" × 3½" × ⅜"	2'-5½"	10.4	51
2	Fills	5" × ⅜"	2'-2¼"	6.38	28
2	Ls	4" × 3½" × ⅜"	2'-7½"	9.1	48
2	Ls	〃	3'-1"	〃	56
1	Fill.	8⅜" × 5/16"	0'-11½"	8.9	9
1	Web. Pl.	29¼" × 5/16"	2'-6⅞"	31.08	80
28	Shop Rivets	¾"φ grip abt 1⅛"		0.47	13
					285 × 12 = 3,420

BRACKET BR2 — 4 Req'd

2	Ls	5" × 3½" × ⅜"	1'-8"	10.4	35
2	Fills	5" × ⅜"	1'-4¾"	6.38	18
2	Ls	4" × 3½" × ⅜"	2'-7½"	9.1	48
2	Ls	〃	2'-7"	〃	47
1	Fill.	8⅜" × 5/16"	0'-11½"	8.9	9
1	Web. Pl.	20" × 5/16"	2'-6⅞"	21.25	55
24	Shop Rivets	¾"φ grip abt. 1⅛"		0.47	11
					223 × 4 = 892

Total of Floor beam = 12,5040

EXPANSION JOINT EJ1 — 3 Req'd.

2	Ls	3" × 3" × ⅜"	24'-10"	7.2	358
1	Web Pl.	9" × 5/16"	12'-3½"	9.56	117
1	〃	〃	12'-6½"	〃	120
2	Checkered Pls.	11¼" × ½"	7'-1½"	20.06	286
2	〃	〃	5'-3	〃	211
15	Anchor bolts	¾"φ	1'-2	@ 2.12	32
227	Shop Rivets	¾"φ grip ¾" abt.		@ 0.43	98
					1,222 × 3 = 3,666.

EXPANSION JOINT EJ2 — 3 Req'd

1	L	6 × 3½" × ⅜"	24'-10"	11.7	291.
1	L	3" × 3" × ⅜"	24'-10"	7.2	179
1	Web Pl.	9" × 5/16"	12'-6½"	9.56	120
1	〃	〃	12'-3½"	〃	117
2	Bars	2" × ½"	7'-2"	3.4	49
1	〃	〃	10'-2"	〃	36
15	Anchorbolts	¾"φ	1'-2"	@ 2.12	32
227	Shop Rivets	¾"φ grip ¾" abt.		@ 0.43	98
					922 × 3 = 2,766

EXPANSION JOINT EJ3 — 1 Req'd.

2	Ls	3 × 3 × 3/8	24-10"	7.2	358
1	Web	9 × 5/16	12-3½	9.56	117
1	〃	〃	12-6½	〃	120
2	Checkerd Pl.	10" × ½"	7-1½	17.83	254
2	〃	〃	5-3	〃	187
15	Anchor Bolts	3/4"φ	1-2"	2.12	32
227	Shop rivets	3/4"φ grip abt. 3/4"		0.43	98
					1,166

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MADE BY ____ DATE ____ FILE NO ____

CHECKED BY ____ DATE ____ PAGE NO M 7

CALCULATIONS FOR

EXPANSION JOINT EJ4 1 Req'd.

1	L	6 × 3½ × 3/8	24'-10"	11.7	291
2	Web Pls	8½ × 5/16	12'-5	9.03	224
1	L	3 × 2 × 1/4	14-8½	4.1	60
2	Ls	"	0-9	"	6
2	Bars	2 × ½	7-2	3.4	49
1	"	"	10-6"	3.4	36
24	Anchor Bolts	3/4"φ	1'-6	2.62	63
198	Shop rivets	3/4"φ grip abt. 3/4"		0.43	85
					814

Total of expansion joint = 8,412

BRACKET SBR1 R/L 2 Req'd.

1	L	4 × 2½ × 3/8	3-2"	7.81	25
1	Pl.	25 × 5/16	4-6	26.56	120
1	L	8 × 3½ × 9/16	5-3	21.0	110
22	Shop rivets	3/4"φ grip abt. 3/4"		0.43	9
9	Field "	" " " "		"	4
					268 × 2 = 536

BRACKET SBR2 R/L 24 Req'd.

1	L	4 × 2½ × 3/8	3'-5"	7.81	25
1	Pl.	25 × 5/16	4'-6"	26.56	120
1	L	4 × 3 × 3/8	5-3	8.5	45
22	Shop rivets	3/4"φ grip abt. 3/4"		0.43	9
9	Field "	" " " "		"	4
					203 × 24 = 4,872

BRACKET SBR3R 84 Req'd.

1	L	4 × 2½ × 3/8	1'-7¼"	7.81	13
1	Pl.	23½ × 5/16	2-1	24.97	52
1	L	4 × 3 × 3/8	3-7	8.5	30
12	Shop rivets	3/4"φ grip abt. 3/4"		0.43	5
7	Field rivets	" " " "		"	3
					103 × 84 = 8,652

Total of bracket = 14,060

CAST STEEL PIN BLOCK CP1 12 Req'd.

1	Cast steel		@226.	226.
8	Turned bolts	1¼"φ × 0'-3¾"	3.	24
				250 × 12 = 3,000

ROLLER SHOE CRS1 6 Req'd.

1	~~Cast steel topshoe~~		~~548~~	~~548~~
2	" " Dust guards		40	80
1	" " Bed Pl.		296	296
6	Bolts	7/8"φ × 0'-3¼"	1.14	7
				931 × 6 = 5,586

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(独立行政法人 土木研究所蔵)

CALCULATIONS FOR

ROLLER NEST RN2 6 Req'd.

6	Rollers	5"φ 3 × 1'-9½"		@	91.	546
4	Pls.	1½ × ½"	1-8¾		4.4	18
24	Screws	½"φ	0-1½		0.083	2
						566 × 6 = 3,396 ✓

DUST GUARD PLATE DG1 12 Req'd.

1	Pl.	5½ × 5/16	1'-10½"	11.	11
5	Bolts	5/16"φ × 0'-1¼"		0.06	1
					12 × 12 = 144 ✓

CAST STEEL FIXED SHOE CFS1 6 Req'd.

1	Shoe	888	888
			888 × 6 = 5,328 ✓

SIDE SPAN ROLLER SHOE SRS1 2 Req'd.

1	Cast steel top shoe	399	399
2	〃 〃 Dust guards	33	66
1	〃 〃 Bed Pl.	223	223
4	Bolts 7/8"φ × 0'-3½"	1.26	5
			693 × 2 = 1,386 ✓

ROLLER NEST RN1 2 Req'd.

4	Roller	4½"φ × 1'-7½"	86.5	346
2	Pls	3½ × ½" × 1'-5½"	8.6	17
8	Screws	7/8"φ × 0'-1¾"	0.29	2
				365 × 2 = 730 ✓

DUST GUARD PLATE DG2 4 Req'd.

1	Pl.	5 × 5/16 × 1'-8"	9.	9
5	Bolts	5/16"φ × 0'-1½"	0.06	1
				10 × 4 = 40. ✓

SIDE SPAN FIXED SHOE SFS1 2 Req'd.

1	Shoe	445	445
			445 × 2 = 890 ✓

CAST STEEL SHEAR BLOCKS 12 Req'd.

1	Shear Pl.	56	56
1	Bracket	91	91
			147 × 12 = 1,764 ✓

CAST STEEL FIXED BEARING BLOCKS 6 Req'd.

1	Top brock	26	26
1	Bottom 〃	34	34
2	7/8"φ × 0'-6"	1.6	3
4	Washers 2½"φ × 1/8"	0.16	1
			64 × 6 = 384 ✓

PIN & NUTS PN1 — 12 Req'd.

1	Pin.	4½"φ × 0'-11½"	@ 52	52	
2	Nuts		4.6	9	
				61 × 12 = 732 ✓	

PIN & NUTS PN2 — 4 Req'd.

1	Pin.	4½"φ × 0'-9¼"	42	42
2	Nuts		4.6	9
				51 × 4 = 204 ✓

ANCHOR BOLT AB1 — 64 Req'd.

1	Bolts	41.	41
			41 × 64 = 2,624 ✓

CAST STEEL SLIDE BEARING BLOCKS — 6 Req'd.

1	Top block		28	28
1	Bottom "		34	34
2	Bolts	⅞"φ × 0'-6"	1.6	3
4	Washers	2½"φ × ⅛"	0.16	1
				66 × 6 = 396

Total of shoes, bearing blocks & accessories = 26,604

Grand total summary.

Main girders	405,809 #
Lateral bracings	33,507
Stringers	57,331
Floor beam	125,040
expansion joint	8,412
bracket	14,060
cast steel shoes, bearings & accessories.	26,604
	670,763 # = 299.448 Tons
Expansion joint	8.412
	291.036 Tons

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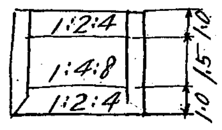
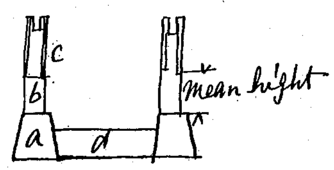
MADE BY M.K DATE 2-8-8 FILE NO.
CHECKED BY Y.H DATE 2-8-10 PAGE NO M10

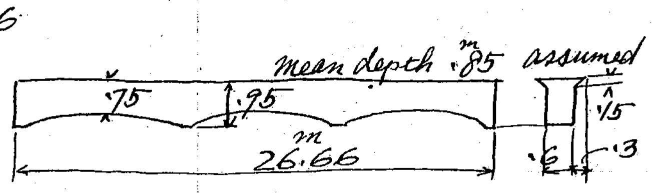
CALCULATIONS FOR

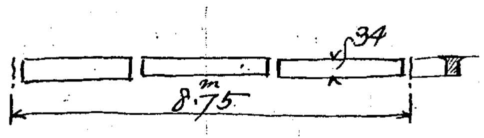
Volume of Concrete for Concrete spans

Name	No.	Section	Length	Unit vol	Total Vol.	Remarks
Well 28 reqd (P1 to P14)			m	cub.m	cub.m	
Shell	1	0.3×9.2+.15×.15×2	3.28	9.35	9.35	Concrete 1:2:4
fills	2	2.0×2.0−.15×.15×2	1.00	3.96	7.92	〃 〃
fill	1	.3×.3×1/2	2.3×4.	.41	0.41	〃 〃
					17.68×28 = 495.04	
fill	1	2.0×2.0−.15×.15×2	1.50	5.92	5.92	Concrete 1:4:8
					5.92×28 = 165.76	
Piers P1. P2. P4. P5. P7. P8. P10. P11. P13 and P14 10 reqd						
a	2	1.3×1.3	1.50	2.54	5.08	
b	2	.8×.8	1.57	1.00	2.00	mean for 10 Piers
c	2	.77×.77	3.15	1.87	3.74	
d	1	1.0×.6	4.40	2.64	2.64	
					13.46×10 = 134.60	
Piers P3. P6. P9 and P12 4 reqd						
a	2	1.4×1.50	1.50	3.15	6.30	
b	2	1.0×.90	1.59	1.43	2.86	mean for 4 Piers
c	2	.98×.87	2.65	2.26	4.52	
d	1	1.0×.6	4.19	2.51	2.51	
					16.19×4 = 64.76	
Main Beam 5 reqd (East Abutment to Pier No15)						
	2	.6×.85	26.66	13.60	27.20	
	2	.15×.3	〃	1.20	2.40	
					29.60×5 = 148.00	
Stringers 15 reqd (East Abutment to Pier No15)						
	2	.34×.30	7.63	.78	1.56	
	2	.10×.12	〃	.09	.18	
					1.74×15 = 26.10	
Cross Beam CB1, 10 reqd.						
	1	.63×.38	5.20	1.24	1.24	
	1	.10×.12	〃	.06	.06	
	2	.50×.50×1/2	.38	.05	.10	
	4	.30×.17	.675	.03	.12	Brackets
					1.52×10 = 15.20	
Cross Beam CB2 30 reqd						
	1	.63×.38	5.20	1.24	1.24	
	1	.10×.12	〃	.06	.06	
	2	.30×.17	.675	.03	.06	Brackets
					1.36×30 = 40.8	
Cross Beam CB3 10 reqd						
	1	.63×.30	5.20	.98	.98	
	1	.10×.12	〃	.06	.06	
	2	.40×.60×1/2	.30	.04	.08	
	2	.30×.20	1.15	.07	(.14×9/10) = .13	9 Brackets
					1.25×10 = 12.5	









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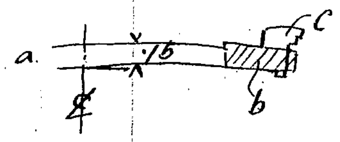
SEIYU BLDG, TOKIO

MADE BY M.K DATE 2-8-8 FILE NO
CHECKED BY Y.H DATE 2-8-10 PAGE NO M11

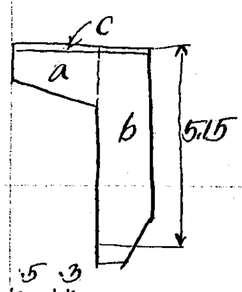
CALCULATIONS FOR

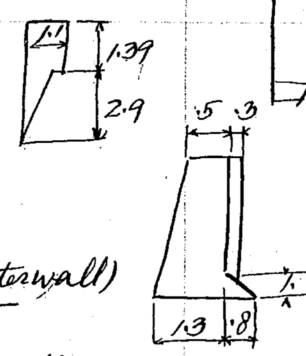
Name	NO	Section	Length	Unit vol.	Total vol.	Remarks
Slab	4 reqd	(Pier No.1 to 15)	m	cub.m	cub.m	
a	1	.15 × 7.2	26.66	28.79	28.79	
b	2	.194 × .48	″	2.48	4.96	
c	2	.15 × .375	″	1.49	2.98	
c′	4	.375 × .475	0.48	0.09	0.36	
					37.09 × 4 = 148.36	



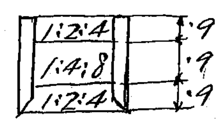
Name	NO	Section	Length	Unit vol.	Total vol.	Remarks
Slab	1 reqd	(Nearest to Abutment)				
a	1	.15 × 7.2	26.66	28.79	28.79	
b	2	.194 × .48	26.60	2.47	4.94	
c	2	.15 × .375	26.57	1.49	2.98	
c′	2	.375 × .475	0.48	0.09	0.18	
					36.89	

Name	NO	Section	Length	Unit vol.	Total vol.	Remarks
East Abutment		1 reqd				
a	2	.30 × 1.50	2.90	1.31	2.62	Wing wall
b	2	.48 × .70	5.15	1.73	3.46	″
c	2	.08 × .375	3.20	0.10	0.20	″
	2	1.10 × 1.05	1.39	1.61	3.22	Column
	2	1.05 × .59 × 1/2	2.90	0.90	1.80	″
	2	(1.3 + .50) × 1/2 × 3.88	3.60	12.58	25.16	Shaft
	2	.8 × 1.0 × 1/2	3.60	1.44	2.88	″
	2	.3 × .9	3.10	0.84	1.68	″
	1	.5 × 3.88	2.40	4.66	4.66	″ (Center wall)
	1	.09 × .3	4.80	0.13	0.13	″
	1	.36 × 1.15	7.50	3.11	3.11	Parapet wall.
	2	.5 × .15	1.20	0.90	0.18	
					49.11	





Name	NO	Section	Length	Unit vol.	Total vol.	Remarks
Caisson	2 Reqd	for East Abutment				
Shell	1	.4 × 12.8 + .2 × .2 × 2	2.50	13.00	13.00	Concrete 1:2:4
fills	2	2.8 × 2.8 − .2 × .2 × 2	0.90	6.98	13.96	″
″	1	.4 × .3 × 1/2	3.2 × 4	0.77	0.77	″
					27.73 × 2 = 55.46	
fill	1	2.8 × 2.8 − .2 × .2 × 2	.90	6.98	6.98	Concrete 1:4:8
					6.98 × 2 = 13.96	



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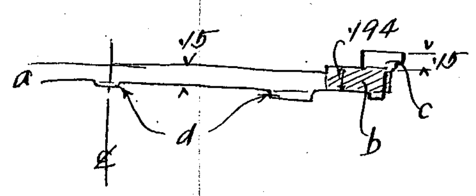
MADE BY M.K DATE 2-8-7 FILE NO.

CHECKED BY Y.H DATE 2-8-10 PAGE NO M12

CALCULATIONS FOR

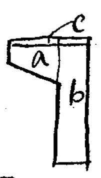
Volume of Concrete for Girder spans

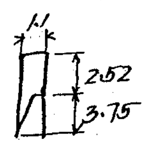
Name	No	Section	Length	Unit Vol.	Total Vol.	Remarks
Slab No.1	1 reqd		m	Cub.m	Cub.m	
a	1	.15 × 7.2	18.16	19.61	19.61	
b	2	.194 × .48	〃	1.69	3.38	
c	2	.15 × .375	〃	1.02	2.04	
c′	2	.35 × .475	0.64	0.11	0.22	
d	1	.016 × .84	18.16	0.24	0.24	
					25.49	
Slab No.2	6 reqd					
a	1	.15 × 7.2	13.55	14.63	14.63	
b	2	.194 × .48	〃	1.26	2.52	
c	2	.15 × .375	〃	.76	1.52	
c′	2	.35 × .475	.95	.16	.32	
d	1	.016 × .84	13.55	.18	.18	
					19.17	
Slab No.3	3 reqd					
a	1	.15 × 7.2	10.81	11.67	11.67	
b	2	.194 × .48	〃	1.01	2.02	
c	2	.15 × .375	〃	.61	1.22	
d	1	.016 × .84	〃	.14	.14	
					15.05	
Slab No.4	2 reqd					
a	1	.15 × 7.2	10.95	11.83	11.83	
b	2	.194 × .48	〃	1.02	2.04	
c	2	.15 × .375	〃	.61	1.22	
d	1	.016 × .84	〃	.14	.14	
					15.23	
Slab No.5	1 reqd					
a	1	.15 × 7.2	17.79	19.21	19.21	
b	2	.194 × .48	〃	1.65	3.30	
c	2	.15 × .375	〃	1.00	2.00	
d	1	.016 × .84	〃	0.23	0.23	
					24.74	

Summary

					Cub.m
Slab	No. 1	1 @	25.49	=	25.49
〃	No. 2	6 @	19.17	=	115.02
〃	No. 3	3 @	15.05	=	45.15
〃	No. 4	2 @	15.23	=	30.46
〃	No. 5	1 @	24.74	=	24.74
			Total		240.86

Name	No	Section	Length	Unit Vol.	Total Vol.	Remarks
West Abutment	1 reqd.					
a	2	1.50 × .30	3.20	1.44	2.88	Wing Wall
b	2	.47 × .57	6.27	1.68	3.36	〃
c	2	.08 × .375	3.40	0.10	0.20	〃
	2	1.10 × 1.05	2.52	2.91	5.82	Column
	2	(1.05 × .40) × 1/2	3.75	.79	1.58	〃





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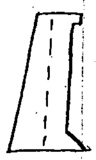
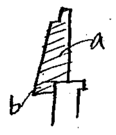
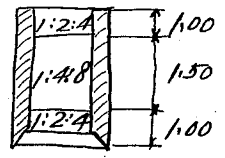
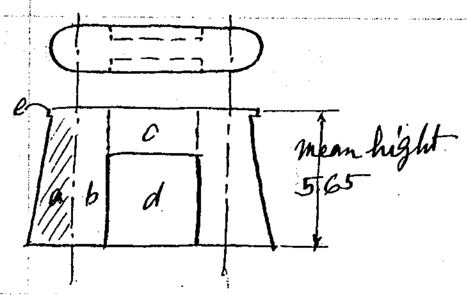
CALCULATIONS FOR

Name	NO	Section	Length	Unit vol.	Total vol.	Remarks
			m	cub.m	cub.m	
	2	·35 × ·09	3.75	1.18	2.36	Column
	2	(1.42+1.8)×½×3.75	2.50	15.09	30.18	Shaft
	2	·8 × 1.0 × ½	2.50	1.00	2.00	〃
	1	·5 × 3.75	3.90	7.31	7.31	〃 (Center wall)
	1	·09 × ·3	11.60	0.31	0.31	〃
a	1	(·58 + ·4)×½×2.30	7.50	8.45	8.45	Parapet wall
b	1	·45 × ·7	3.90	1.23	1.23	〃
					62.32	
Caisson for West Abutment 2 reqd						
Shell	1	·4 × 1.4 + ·2² × 2	3.3	18.74	18.74	Concrete 1:2:4
fills	2	3.4×2.8 − ·2²×2	1.0	9.44	18.88	〃 〃
〃	1	·3 × ·4 × ½	3.6×4	·86	0.65	〃
					38.27 × 2 = 76.54 cub.m	
fill	1	3.4×2.8 − ·2²×2	1.5	14.16	14.16	Concrete 1:4:8
					14.16 × 2 = 28.32 cub.m	
Pier #15 to 21. 7 reqd						
a	2	1.04²×π×½ − ·4×1.04	5.65	7.25	14.50	mean for 7 Piers
b	2	(2.24+1.11)×½×5.65	1.35	12.78	25.56	
c	1	(1.41+1.11)×½×1.55	4.30	8.40	8.40	
d	1	·6 × 4.3	4.10	10.58	10.58	mean for 7 Piers
e	1	·13 × ·45	19.20	1.12	1.12	
					60.16 × 7 = 421.12 cub.m	
Wells for P15 to 21 14 reqd						
Shell	1	·45×12.6 + ·2² × 2	5.8	33.35	33.35	Concrete 1:2:4
fills	2	2.7×2.7 − ·2² × 2	2.0	14.42	28.84	〃
〃	2	4.5 × ·3	3.2	4.32	8.64	〃
					70.83 × 14 = 991.62 cub.m	
fill	1	2.7×2.7 − ·2²×2	4.0	28.84	28.84	Concrete 1:4:8
					28.84 × 14 = 403.76 cub.m	









Katsura Bashi for Kyotofu.
Material List.
Concrete for Handrail.

Names	Req'd. no	Section	Length	Total vol.	Remarks
Top & Bott rails	4	.13×.18	1.80	.17	P1
"	632	"	1.50	22.20	P2, P3, P4, P6, P7, P10, P11, P13, P14, P16 & P17
"	4	"	1.60	.15	P5
"	48	"	1.275	1.43	P8, P9, P12 & P15
"	4	"	1.955	.23	P18
Intermediate post	322	.25×.25	0.85	17.10	
Light Pedestal	22	.7×.7	.95	10.20	on pier.

Handrail on span, Total = 51.48 cub m { 22.28 for concrete span; 29.20 for steel span.

Names	Req'd. no	Section	Length	Total vol.	Remarks
Top & Bott rails	8	.13×.18	3.05	.57	
Intermediate post	8	.25×.25	.54	.27	
Panel Wall	12	.08×.54	.92	.48	
End post	4	.35×.135	.85	.42	
post	4	.125×.175	.85	.15	Post next to light pedestal
Light Pedestal	4	.525×.525	1.60	1.77	Shaft
"	16	.05×.20	.25	.04	moulding
"	4	.07×.18	.80	.18	Base of shaft
"	4	.9×.9	.95	3.07	"

Handrail on abutment, Total = 6.95 cub.m

Total concrete for Handrail = 58.43 cub m { 25.76 for concrete span; 32.67 " steel "

Grand total Volume of Concrete.

	Cub. m
Slab: on steel spans	240.86
" Concrete spans	185.25
	426.11
Main Beams	148.00
Stringers	26.10
Cross Beams	68.50
	242.60
Hand Rails	58.43
Total	727.14

Total 727.14 { for concrete span 453.61; steel span 273.53

West Abutment	{ 138.86	for Concrete	1:2:4
	28.32	" "	1:4:8
Piers for steel spans	{ 1412.74	" "	1:2:4
	403.76	" "	1:4:8
Total for 1:2:4	1551.60		
" " 1:4:8	432.08		

East Abutment	{ 104.56	for Concrete	1:2:4
	13.96	" "	1:4:8
Piers for Concrete spans	{ 694.40	" "	1:2:4
	165.76	" "	1:4:8
Total for 1:2:4	798.96		
1:4:8	179.72		

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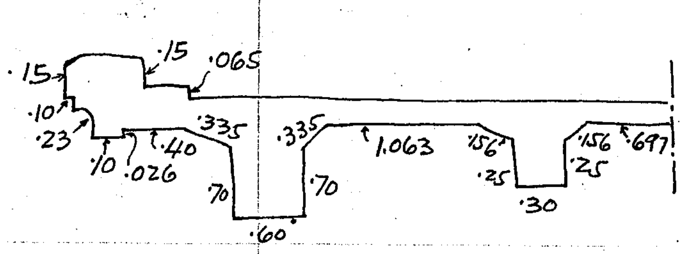
CALCULATIONS FOR

Katsura Bashi for Kyotofu.
Material List for Concrete girder spans.
Forms for floor system, 3 continuous spans.

Names	Reqd No.	Width	Length	Area	Total area	
End of gutter	2	.065	26.7	1.74	3.48	
Curb	2	.15	26.7	4.00	8.00	
Coping	2	.606	26.7	16.18	32.36	
Bott. of floor	1.	11.884	26.7	317.40	317.40	
Projection	2	.606	.95	.58	1.16	
〃	2	.475	1.1	.52	1.04	
Cross beam, sides.	8	.65×2	5.2	6.76	54.08	
〃 〃	2	.65×2	5.2	6.76	13.52	
End, curve	2	.9×2	.4	.72	1.44	
Less Stringer stem	36	.25	.3 −	.08	− 2.88	
〃 〃 〃	36	.10	.42 −	.04	− 1.44	
〃 Bracket area.	8	.17	1.1 −	.19	− 1.52	Bracket on Int. piers
〃 〃	4	.20	1.81 −	.36	− 1.44	〃 〃 End 〃
	12	.17	1.2 −	.20	− 2.40	〃 〃 main beams.
〃 Crossbeam, end	8	.38×2	.65 −	.49	− 3.92	
〃 〃 〃	2	.30×2	1.30 −	.78	− 1.56	
Bracket sides	12	.35×2	.60	.42	− 5.04	Bracket fixed to beams only



Total = 422.36 ㎡ = (127.8 面坪)

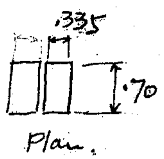
For East End 3 continuous spans, less projections − 2.20
Total = 420.16 ㎡ = (127.1 面坪)

Summary of forms.

East End 3 continuous spans	1 @ 420.16	=	420.16
West End 〃 〃 〃	1 @ 422.36	=	422.36
Intermediate 〃 〃 〃	3 @ 422.36	=	1,267.08
			2,109.60 ㎡ = (640 面坪)

Forms of Handrail for Intermediate 3 continuous spans.

Names	Reqd No.	Width	Length	Area	Total area	Remarks.
Top rail	30	.44	1.500	.66	19.80	
Bott rail	30	.26	1.500	.39	11.70	
Int. post	28	1.00	.85	.85	23.80	
Light pedestal	4	2.07	.95	1.97	7.88	On pier
					63.18 ㎡ = (19.1 面坪)	



East End 3 continuous spans

Top rail	28	.44	1.500	.66	18.48	
〃	2	.44	1.795	.79	1.58	Top rail next to light pedestal
Bott rail	28	.26	1.500	.39	10.92	
〃	2	.26	1.795	.47	.94	Bott rail next to light pedestal
Int. post	28	1.00	.85	.85	23.80	
Light pedestal	2	2.07	.95	1.97	3.94	on pier
					59.66 = (18.1 面坪)	

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CALCULATIONS FOR

Katsura Bashi for Kyotofu
Material list for Concrete Girder spans.
Forms for Handrail
Handrail on abutment (East abutment)

Names	Req'd no.	Width	Length	Area	Total area	Remarks
Light pedestal	2	2.10	1.60	3.36	6.72	Shaft
"	8	.30	.30	.09	.72	moulding
"	2	.07	3.20	.22	.44	Base of shaft
"	2	3.60	1.13	4.07	8.14	"
Post	2	.60	.85	.51	1.02	Post next to light pedestal
End post	2	1.40	.85	1.19	2.38	
Int. post	4	.38	.54	.21	.84	
Top rail	2	.44	3.05	1.34	2.68	
Bott rail	2	.28	3.05	.85	1.70	
Panel wall	6	1.08	.92	.99	5.94	
					30.58 □m = (9.25 面坪)	

Summary of Forms for Handrail

East End 3 continuous spans 1 @ 59.66 = 59.66
Intermediate 3 continuous spans 4 @ 63.18 = 252.72
Handrail on abutment 1 @ 30.58 = 30.58
Total = 342.96 □m = (103.7 面坪)

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CALCULATIONS FOR

Katsura Bashi for Kyotofu.
Material List for Concrete girder spans.
Forms. Pier no. 1, 2, 4, 5, 7, 8, 10, 11, 13 & 14.
Pier # 1.

Names	Reqd no.	width	length	area	total area	Remarks
Bracket	4	.3×2	.6	.36	1.44	sides
〃	4	.17	1.0	.17	.68	faces.
Column	2	.8×4	3.65	11.79	23.58	
〃	8	.10	1.2	.12	.96	top. side of main beam.
〃	2	.60	.75	.45	.90	under bracket.
〃	2	.20	.57	.11	.22	between brackets, front
〃	2	.20	1.20	.24	.48	〃 〃 rear
〃 base	2	1.3×4	1.50	7.80	15.60	
〃 struts	1	3.2	4.40	14.08	14.08	
					57.94 □m (17.5 面坪)	

Caisson	2	2.6×4	3.20	33.30	66.60	Out sides
〃	2	2.0×4	3.20	25.60	51.20	Insides.
					117.80 □m (35.6 面坪)	

Total form for Pier # 1 = 175.74 □m (= 53.1 面坪)

Forms for Piers		Pier Structure		Caisson		Total
	P1	57.94 □m	+	117.80	=	175.74 □m
	P2	〃 +2×3.2×.066= 58.36	+	〃	=	176.16
	P4	〃 + 〃 ×.189= 59.15	+	〃	=	176.95
	P5	〃 + 〃 ×.242= 59.48	+	〃	=	177.28
	P7	〃 + 〃 ×.337= 60.10	+	〃	=	177.90
	P8	〃 + 〃 ×.377= 60.36	+	〃	=	178.16
	P10	〃 + 〃 ×.445= 60.78	+	〃	=	178.58
	P11	〃 + 〃 ×.471= 60.96	+	〃	=	178.76
	P13	〃 + 〃 ×.511= 61.22	+	〃	=	179.02
	P14	〃 + 〃 ×.524= 61.30	+	〃	=	179.10
				Total		= 1,777.65 □m (538.0 坪)

Pier no. 3, 6, 9 & 12. piers at expansion joint.
Pier # 3.

Names	Reqd no.	Width	Length	area	Total area	Remarks
Brackets	4	.6×2	.70	.84	3.36	sides
〃	4	.20	1.40	.28	1.12	faces.
Columns	2	3.80	3.78	14.36	28.72	
〃	8	.20	.30	.06	.48	Top. side of main beam.
〃	2	1.30	.30	.39	.78	Under bracket.
〃	2	.30	.70	.21	.42	Between bracket front
〃	2	1.50	1.20	.18	.36	〃 〃 rear
〃 base	2	5.80	1.50	8.70	17.40	
〃 strut	1	3.20	4.20	13.44	13.44	
					66.08 □m = (20.00 面坪)	

Caisson Same as above 117.80 □m = (35.6 面坪)

Total form for Pier # 3 = 183.88 □m = (55.6 面坪).

Forms for Piers		Pier Structure		Caisson		Total
	P3	66.08	+	117.80	=	183.88
	P6	〃 + 3.8×.162×2= 67.31	+	〃	=	185.11
	P9	〃 + 〃 ×.283×2= 68.23	+	〃	=	186.03
	P12	〃 + 〃 ×.363×2= 68.84	+	〃	=	186.64
				Total	=	741.66 □m = (224.5 面坪)

Total form for Pier # 1 to 14 Inclusive = 2,519.31 □m = (762.5 面坪)

Katsura Bashi for Kyotofu.
Material List for concrete girder spans.
Forms for East Abutment.

Names	Reqd. No.	Width	Length	Area	Total area	Remarks.
Parapet wall.	1.	1.2×2	6.4	15.36□m	15.36□m	Both sides
Curtain wall	1.	3.88×2	2.4	18.63	18.63	〃 〃
〃	1	.1×1	4.8	.48	.48	Bott. of coping
〃	1.	.5	2.4	1.20	1.20	〃 〃 wall.
Shaft.	2	3.6×2	3.88	27.92	55.84	Both faces
Column	4	.3	3.88	1.16	4.64	Both sides of column under girders.
〃	4	.3	.3	.09	.36	〃
Column	2	1.5×2	1.3	3.90	7.80	Both faces of col. under Light Pedestal.
Shaft	2	.4	3.88	1.55	3.10	Inner side
Column	2	1.55	5.22	8.09	16.18	Side of col under Light pedestal
〃	2	.6	1.3	.78	1.56	Inner side 〃
〃	2	.3	2.9	.87	1.74	〃 〃
〃	2	.23	1.1	.25	.50	〃 〃
〃	2	.5	1.0	.50	1.00	Bott. of column 〃 〃
Wing wall	2	.7×2	5.22	7.31	14.62	Both sides
〃	2	.3	3.24	.97	1.94	End plane.
〃	2	1.5	2.6	3.90	7.80	Both sides.
〃	2	.3	1.0	.30	.60	End plane of wall.
〃	2	.3	3.0	.90	1.80	Bott. plane 〃
〃	2	.15	3.3	.50	1.00	Bott. of coping

Form of abutment structure, Total = 156.15□m = (47.3面坪)

Caisson	2	2.8×4	2.4	26.88	53.76	Inside
〃	2	3.6×4	2.4	34.55	69.10	Outside
〃	2	.3×4	2.8	3.36	6.72	Cutting edge
					129.58□m = (39.2面坪)	

Total area of form = 285.73□m = (86.5面坪)

Summary of forms for concrete girder spans

Floor	2,109.60□m	= (640.00面坪)
Handrail	342.96	= (103.70)
Piers	2,519.31	= (762.50)
abutment	285.73	= (86.50)
	5,257.60□m	= (1592.70面坪)

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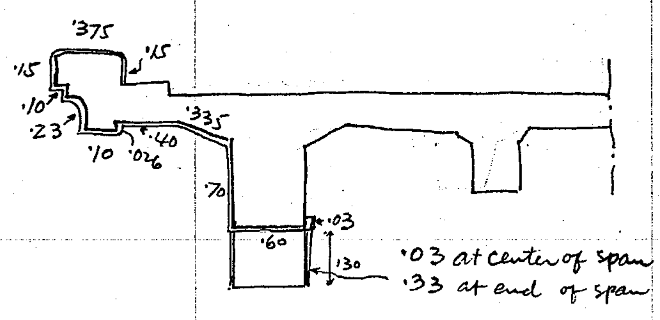
CALCULATIONS FOR

Katsura Bashi for Kyotofu

Material List for Concrete Girder spans.
Artificial Granite finish
Slab. Intermediate 3 continuous spans.

Names	Req'd no	Width	Length	Area	Total area
Curb.	2	.15	26.70	4.00	8.00
Coping	2	.981	26.70	26.20	52.40
overhanging slab	2	.735	26.70	19.63	39.26
Stem of Beam	2	1.45	24.20	35.10	70.20
Back of Beam	2	.18	24.20	4.36	8.72
projection side	2	.606	.95	.58	1.16
" top & Bott.	4	.475	1.10	.52	2.08
Bracket side	12	.35x2	.6	.42	5.04
" face	12	.17	1.10	.19	2.28
Less Bracket area	12	.17	1.2	- .20	- 2.40
" Bott rail "	30	.18	1.50	- .27	- 8.10
" Int. post "	28	.25	.25	- .06	- 1.68
" Light pedestal "	2	.70	.70	- .49	- .98
					175.98 □m = (53.2面坪)



Bracket on beam only.

For East End 3 continuous spans less projection - 1.62
add pedestal + .49
174.85 □m = (52.9面坪)

Summary of artificial granite finish

East End 3 continuous spans	1 @ 174.85	=	174.85
West " " "	1 @ 175.98	=	175.98
Intermediate " " "	3 @ 175.98	=	527.94
	Total =		878.77 □m = (265.80面坪).

Handrail for Intermediate 3 continuous spans.

Names	Req'd no	Width	Length	area	Total area
Top rail	30	.62	1.50	.93	27.90
Bott "	30	.44	1.50	.66	19.80
Int. post	28	1.00	.85	.85	23.80
"	28	.25	.25	.06	1.68
Light pedestal	4	1.37	.95	1.30	5.20
"	2	.70	.70	.49	.98
less	2	.50	.50	- .25	- .50 Bott. area of lamp pole.
					78.86 □m = (23.9面坪)

East End 3 continuous spans

Top rail	28	.62	1.50	.93	26.05
"	2	.62	1.795	1.11	2.22
Bott. rail	28	.44	1.50	.66	18.48
"	2	.44	1.795	.79	1.58
Int. post.	28	1.00	.85	.85	23.80
"	28	.25	.25	.06	1.68
Light pedestal	2	1.37	.95	1.30	2.60
"	1	.70	.70	.49	.49
less	1	.50	.50	.25	- .25 Bott. area of lamp pole.
					76.65 □m = (23.2面坪)

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CALCULATIONS FOR

Katsura Bashi for Kyotofu.

Material List for Concrete girder spans.

Artificial granite finish.

Handrail on abutment. (East abutment)

Names	Reqd. no.	Width	Length	area	Total area	Remarks
Light pedestal	2	2.10	1.60	3.36	6.72	Shaft.
〃	8	·30	·30	·09	·72	moulding
〃	2	·07	3.20	·22	·44	base of shaft
〃	2	3.60	1.13	4.07	8.14	〃
[illegible]	2	·10	1.40	·14	·28	Top of shaft
〃	2	·15	3.00	·45	·90	Top of base.
Post	2	·60	·85	·51	1.02	post next to light pedestal
〃	2	·175	·25	·04	·08	
End post	2	1.40	·85	1.19	2.38	
〃	2	·35	·35	·12	·24	
Int. post	4	·38	·54	·21	·84	
Top rail	2	·54	3.05	1.65	3.30	
Bott rail	2	·36	3.05	1.10	2.20	
Panel wall	6	1.08	·92	·99	5.94	
					33.20 □m = (10.05面坪)	

Summary of Artificial granite finish for Handrail.

East End 3 continuous spans	1 @	76.65	= 76.65
Intermediate 〃 〃 〃	4 @	78.86	= 315.44
Handrail on abutment	1 @	33.20	= 33.20
			425.29 □m = (128.7面坪)

Pier for Concrete girder span

Pier Nos. 1, 2, 4, 5, 7, 8, 10, 11, 13 & 14

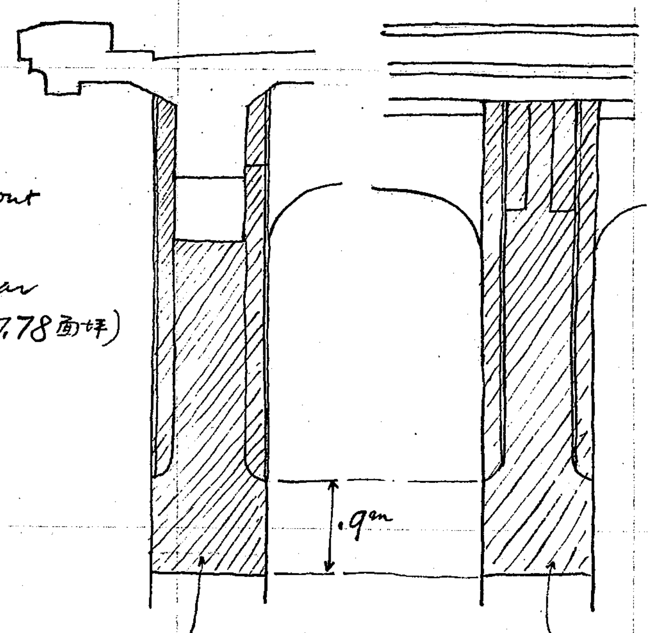
Names	Reqd. no.	Width	Length	area	Total area
Brackets sides	4	·3×2	·6	·36	1.44
〃 face	4	·17	1.0	·17	·68
Column	2	·8×4	3.3	10.56	21.12
〃 Top.	2	1.0	1.2	1.20	2.40
less Bracket area	4	·17.	·6	− ·10	− ·40
Column Top.	2	·70	·33	·23	·46
					25.70 □m = (7.78面坪)

Pier Nos. 3, 6, 9 & 12

Names	Reqd. no.	Width	Length	area	Total area
Brackets sides	4	·6×2	·7	·84	3.36
〃 face	4	·2	1.4	·28	1.12
Column	2	3.80	3.3	12.54	25.08
〃 Top. front.	2	1.1	1.2	1.32	2.64
less bracket area	4	·2	·6	− ·12	− ·48
Column, Top. rear	2	·5	·33	·17	·34
					32.06 □m = (9.70面坪)

Front

Rear



Portion hatched shall be finished

Summary of Artificial granite finish for Piers

Pier Nos	1, 2, 4, 5, 7, 8, 10, 11, 13 & 14	10 @	25.70	= 257.00
〃 〃	3, 6, 9 & 12	4 @	32.06	= 128.24
				385.24 □m = (116.5面坪)

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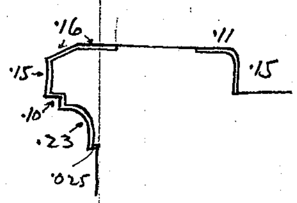
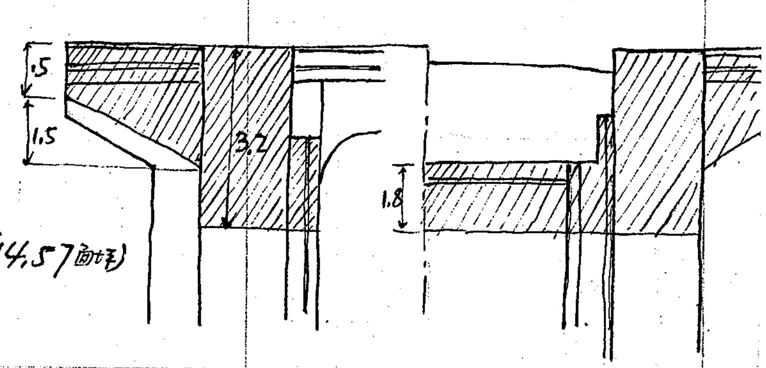
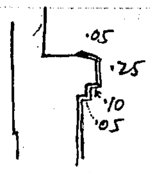
CALCULATIONS FOR

Katsura Bashi for Kyotofu.
Material List for concrete girder spans.
Artificial granite finish
East abutment.

Names	Req'd no.	width	Length	area	Total area	Remarks.
Coping	2	·665	3.15	2.10	4.20	
" End	2	·58	·45	·26	·52	
Curb	2	·26	3.15	·82	1.64	
" End	2	·225	·50	·11	·22	
End plane of Coping	2	·42	·375	·16	·32	
" " wing	2	·30	·125	·04	·08	
Wing wall	2	·875	3.6	3.15	6.30	
Column under lamp	2	2.95	3.2	9.44	18.88	
" curb.	2	·225	1.40	·32	·64	
Column under beam	2	1.60	1.80	2.88	5.76	
"	2	·40	·30	·12	·24	
Coping	1.	·45	4.80	2.16	2.16	
Curtain wall	1.	1.50	4.80	7.20	7.20	
					48.16 □m = (14.57 坪)	







Summary of Artificial granite finish for concrete girder spans.

Slab, copings & Beams	878.77 □m	= (265.80 坪)
Handrails	425.29	= (128.70 ")
Piers	385.24	= (116.50 ")
Abutments	48.16	= (14.57 ")
	1,737.46 □m	= (525.57 坪)

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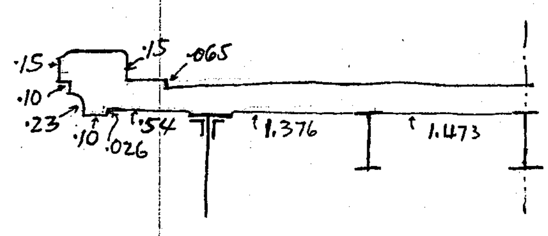
CALCULATIONS FOR

Katsura Bashi for Kyotofu.

Material List for Steel spans.

Forms for Slab. Intermediate span span length = 24.5 m.

Names	Req'd no.	Width	Length	Area	Total area	Remarks.
End of gutter	2	.065	24.5	1.59	3.18	
Curb	2	.15	24.5	3.68	7.36	
Coping	2	.606	24.5	14.85	29.70	
" projection	2	.606	.9	.55	1.10	
Slab	1	6.78	24.5	166.30	166.30	
" projection	2	.64	.48	.31	.62	
					208.26 ㎡ = (63.0 面坪)	



East End span span length = 24.5 m.

End of gutter	2	.065	24.694	1.61	3.22
Curb	2	.15	24.694	3.70	7.40
Coping	2	.606	24.694	14.96	29.92
" projection	2	.606	.90	.55	1.10
Slab	1	6.78	24.694	167.50	167.50
" projection	2	.64	.48	.31	.62
					209.76 ㎡ = (63.35 面坪)

West End span span length = 24.5 m

End of gutter	2	.065	24.80	1.61	3.22
Curb	2	.15	24.73	3.71	7.42
Coping	2	.606	24.73	14.99	29.98
" projection	2	.606	.45	.27	.54
Slab	1	6.78	24.80	168.20	168.20
" projection	2	.32	.48	.15	.30
					209.66 ㎡ = (63.40 面坪)

Summary of forms for floor system.

East End span	1 @	209.76 =	209.76
West End span	1 @	209.66 =	209.66
Intermediate span	5 @	208.26 =	1,041.30
			1,460.72 ㎡ = (442.0 面坪)

Handrail on spans.

Intermediate spans ~~& East End span~~.

Names	Req'd no.	Width	Length	Area	Total area	Remarks.
Top rail	24	.44	1.50	.66	15.84	
"	4	.44	1.275	.56	2.24	
Bott. rail	24	.26	1.50	.39	9.36	
"	4	.26	1.275	.33	1.32	
Int. post	26	1.00	.85	.85	22.10	
Light pedestal	2	2.80	.95	2.66	5.32	
					56.18 ㎡ = (17.00 面坪)	

West End span.

Top rail	24	.44	1.50	.66	15.84
"	2	.44	1.275	.56	1.12
Bott. rail	24	.26	1.50	.39	9.36
"	2	.26	1.275	.33	.66
Int. post	26	1.00	.85	.85	22.10
Light pedestal	2	2.80	.95	2.66	5.32
Top rail	2	.44	1.955	.86	1.72
Bott rail	2	.26	1.955	.51	1.02
					57.14 ㎡ = (17.3 面坪)

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CALCULATIONS FOR

Katsura Bashi for Kyotofu.
Material List for steel span
Forms for Handrail East endspan.

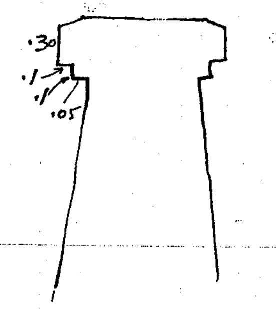
Names	Req'd no	Width	Length	area	Total area	Remarks.
Top rail	24	.44	1.50	.66	15.84	
〃	2	〃	1.275	.56	1.12	
〃	2	〃	1.375	.61	1.22	
Bott rail	24	.26	1.50	.39	9.36	
〃	2	〃	1.275	.33	.66	
〃	2	〃	1.375	.36	.72	
Int post	26	1.00	.85	.85	22.10	
Light pedestal	2	2.07	.95	1.97	3.94	
〃	1	2.80	.95	2.66	2.66	
					57.62 □m = (17.44面坪)	

Summary of forms for Handrail.

East end span	1 @	57.62	= 57.62
West 〃 〃	1 @	57.14	= 57.14
Intermediate 〃	5 @	56.18	= 280.90
Handrail on west abutment	1 @	30.58	= 30.58 Same as for Handrail on East abutment
			426.24 □m = (129.0面坪)

Forms for Pier.
Pier # 16.

Names	Req'd no	Width	Length	area	Total area	Remarks.
Coping	2	.55	9.30	5.11	10.22	
Shaft	2	5.342	9.80	52.35	104.70	
depression, side	4	0.6	4.24	2.55	10.20	
〃 top.	2	.4	3.40	1.36	2.72	
			Total for shaft =		127.84 □m = (38.65面坪)	
Caisson	1	3.6×4	5.7	82.10	82.10	out side
〃	4	2.3	6.05	13.92	55.68	Inside
fillet	4	.28	6.05	1.69	6.76	
					144.54 □m = (43.75面坪)	



Forms for Piers

	Pier shaft		Caisson	
P16	127.84 +		+ 144.54×2 =	416.92 □m
P17	〃 −	24.4 × .028	+ 〃 ×2 =	416.24
P18	〃 −	〃 × .086	+ 〃 ×2 =	414.82
P19	〃 −	〃 × .172	+ 〃 ×2 =	412.72
P20	〃 −	〃 × .288	+ 〃 ×2 =	409.89
P21	〃 −	〃 × .433	+ 〃 ×2 =	406.36
P15	127.84 + 144.54×2 +		13.50 =	430.42 See following calculation.
			Summary for Piers =	2,907.37 □m (880.0面坪)

Extra concrete on Pier 15 for Bearing of shoes.

Base of shoes	2	5.4	.448	2.42	4.84
〃 Conc. beam	2	2.9	1.246	3.61	7.22
〃	4	1.2	.30	.36	1.44
					13.50 □m

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CALCULATIONS FOR

Katsura Bashi for Kyotofu.
Material List for steel spans.
Forms for West Abutment.

Names	Reqd.no.	Width	Length	area	Total area	Remarks
Parapet wall	1.	2.33	7.50	17.48	17.48	Front side
"	1.	3.03	3.90	11.82	11.82	Rear "
Curtain wall	1	.40	3.80	1.52	1.52	Coping
"	1.	3.45	3.90	13.46	13.46	Front side
"	1	3.05	3.90	11.90	11.90	Rear "
"	1	.50	3.90	1.95	1.95	Top.
"	1	.50	3.90	1.95	1.95	Bott.
Shaft. coping	2	.40	3.90	1.56	3.12	
" . Front	2	3.70	3.45	12.76	25.52	
add slope	2	2.50	.30	.75	1.50	
Shaft rear	2	1.80	6.10	10.98	21.96	
" inside	2	.65	3.05	1.98	3.96	
Column front	2	1.05	2.52	2.65	5.30	
" "	2	.35	3.75	1.31	2.62	
" Side	2	1.10	6.27	6.90	13.80	
" rear	2	.45	6.27	2.82	5.64	
" "	2	0.32	6.27	2.01	4.02	
" front	2	.07	2.29	.16	.32	
" Curb.	2	.22	1.10	.24	.48	
Wing side	4	.57	5.9	3.36	13.44	
" "	4	1.13	3.03	3.42	13.68	
" End	2	.30	5.90	1.77	3.54	
" "	2	.30	.63	.19	.38	
" bott.	2	3.5	.30	1.05	2.10	
" Coping	2	.5	3.60	1.80	3.60	
" curb.	2	.22	3.60	.79	1.58	
" end	2	.4	.38	.15	.30	

Forms of Abutment structure, Total = 186.94 □m = (56.5 面坪)

Caisson	2	3.5	3.0	10.50	21.00	Inside longer side
"	2	3.5	2.4	8.40	16.80	" Shorter side
"	4	3.5	.28	.98	3.92	" fillets.
"	2	3.2	4.2	13.45	26.90	Outside longer side
"	2	3.2	3.6	11.52	23.04	" shorter side

Forms for one caisson, Total = 91.66 □m = (27.74 面坪)
for 2 Caissons 2@91.66 = 183.32 □m = (55.48 面坪)

Total Form of West abutment = 186.94 + 183.32 = 370.26 □m = (111.98 面坪)

Summary of Forms for steel girder spans.

Floor	1,460.72 □m	= (442.00 面坪)
Handrail	426.24 "	= (129.00)
Piers	2,907.37 "	= (880.00)
Abutment	370.26 "	= (111.98)
Total	= 5,164.59 □m	≒ (1,562.98 面坪)

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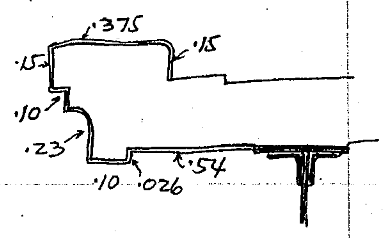
CALCULATIONS FOR

Katsura Bashi for Kyotofu.
Material List for Steel girder spans.
Artificial Granite finish.
Finish for slab. Intermediate spans span length 24.5 m.



Names	Reqd. No.	Width	Length	Area	Total area	Remarks
Curb	2	.15	24.5	3.68	7.36	
Coping, Top	2	.375	24.5	9.19	18.38	
" Side & Bott.	2	.606	24.5	14.85	29.70	
" Projection on pier	2	.606	.9	.55	1.10	
Slab bott.	2	.540	24.5	13.23	26.46	
" Projection on pier	2	.64x2	.48	.61	1.22	Top & Bott.
Less Bracket area	12	.105	.50	− .05	− .60	Bracket on span, top plane
" "	4	.10	.98	− .10	− .40	" " pier, " "
" Bottom rail	24	.18	1.50	− .27	− 6.48	Intermediate Bott rails.
" Int. post	26	.25	.25	− .06	− 1.56	
" Light pedestal	2	.70	.70	− .49	− .98	
" Bott rail	4	.18	1.275	− .23	− .92	End Bott rails.
				Total =	73.28 □m = (22.15 面坪)	

East End span

Names	Reqd. No.	Width	Length	Area	Total area	Remarks
Curb	2	.15	24.794	3.72	7.44	C to C of Pier # 15 + 16 = 24.694 add .10
Coping top.	2	.375	24.794	9.29	18.58	
" Side & Bott	2	.606	24.794	15.08	30.16	
" Projection on pier	2	.606	.9	.55	1.10	
Slab Bott.	2	.54	24.794	13.40	26.80	
" Projection on pier	2	.64+.74	.48	.66	1.32	Top & Bott.
Less Bracket area	12	.10	.50	.05	− .60	Bracket on span, Top plane.
"	2	.10	.98	.10	− .20	" " Pier # 16
"	2	.20	.98	.20	− .40	" " " 15
Less Bott rail	24	.18	1.50	.27	− 6.48	Int. Bott. rail
" "	2	.18	1.2875	.23	− .46	West End rail
" "	2	.18	1.469	.26	− .52	East " "
" Int. post	26	.25	.25	.06	− 1.56	
" Light pedestal	2	.45	.70	.32	− .64	on pier # 15
" "	2	.35	.70	.25	− .50	" " # 16
				Total =	74.04 □m = (22.4 面坪)	

West End span

Names	Reqd. No.	Width	Length	Area	Total area	Remarks
Curb.	2	.15	24.73	3.71	7.42	
Coping top	2	.375	24.73	9.27	18.54	
" Side & Bott.	2	.606	24.73	14.98	29.96	
" projection on pier	1	.606	.90	.55	.55	
Slab Bott	2	.54	24.73	13.35	26.70	
" projection on pier	2	.64	.48	.31	.62	Top & Bott
Less Bracket area	12	.10	.50	.05	− .60	Bracket on span, top plane
" "	2	.10	.98	.10	− .20	" " pier 21. "
" Bott rail	24	.18	1.50	.27	− 6.48	Int. Bott. rail
" "	2	.18	1.275	.23	− .46	East End Bott rail
" "	2	.18	1.955	.35	− .70	West " " "
" Int. post	26	.25	.25	.06	− 1.56	
" Light pedestal	2	.35	.70	.25	− .50	
				Total	= 73.29 □m = (22.18 面坪)	

Summary of Artificial Granite finish for Slab.

Intermediate spans -	5 @	73.28 □m	=	366.40 □m
East End "	1 @	74.04	=	74.04
West " "	1 @	73.29		73.29
				513.73 □m = (155.33 面坪)

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

Katsura Bashi for Kyotofu.
Material List for Steel girder spans.
Intermediate spans. Handrail finish.

Name	Reqd no.	Width	Length	area	Total area	Remarks
Toprail	24	·62	1.50	·93	22.32	
Toprail	4	·62	1.275	·79	3.16	
Bott. rail	24	·44	1.50	·66	15.84	
〃	4	·44	1.275	·56	2.24	
Int. post.	26	1.00	·85	·85	22.10	
[illegible]	26	·25	·25	·06	1.56	
Light pedestal	2	2.80	·95	2.66	5.32	
〃	2	·70	·70	·49	·98	
less	2	·50	·50	·25	−·50	Bott. area of lamp pole.
				Total =	73.02 □m = (22.10 面坪)	

East End span.

Top rail	24	·62	1.50	·93	22.32
〃	2	·62	1.275	·79	1.58
[illegible]	2	·62	1.469	·91	1.82
Bott. rail	24	·44	1.50	·66	15.84
[illegible]	2	·44	1.275	·56	1.12
〃	2	·44	1.469	·65	1.30
Intermediate post	26	1.00	·85	·85	22.10
〃	26	·25	·25	·06	1.56
Light pedestal	2	1.37	·95	1.30	2.60
〃	2	1.40	·95	1.33	2.66
〃	2	·70	·35	·25	·50
〃	1	·70	·335	·23	·23
less Bott of lamp.	2	·50	·50	·25	−·50
				Total =	73.13 □m = (22.15 面坪)

West End span

Top rail	24	·62	1.50	·93	22.32
〃	2	·62	1.275	·79	1.58
〃	2	·62	1.955	1.21	2.42
Bott rail	24	·44	1.50	·66	15.84
〃	2	·44	1.275	·~~56~~	~~1.08~~ 1.12
〃	2	·44	1.955	·86	1.72
Intermediate post	26	1.00	·85	·85	22.10
〃	26	·25	·25	·06	1.56
Light pedestal	2	1.40	·95	1.33	2.66
〃	2	·70	·35	.245	·490
Less Bott of lamp	2	·25	·50	·25	−·50
				Total =	71.31 □m = (21.58 面坪)

Summary for Artificial granite finish for Handrail.

Intermediate spans	5 @	73.02	=	365.10 □m	
East End span	1 @	73.13	=	73.13	
West 〃 〃	1 @	71.31	=	71.31	
Handrail on abutment	1 @	33.20	=	33.20	same as for East abutment.
				542.74 □m = (164.28 面坪)	

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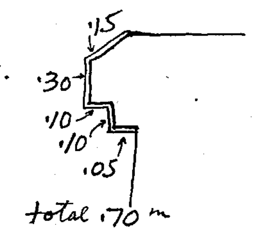
Katsura Bashi for Kyotofu.

Material List for Steel girder spans.

Artificial granite finish for piers.

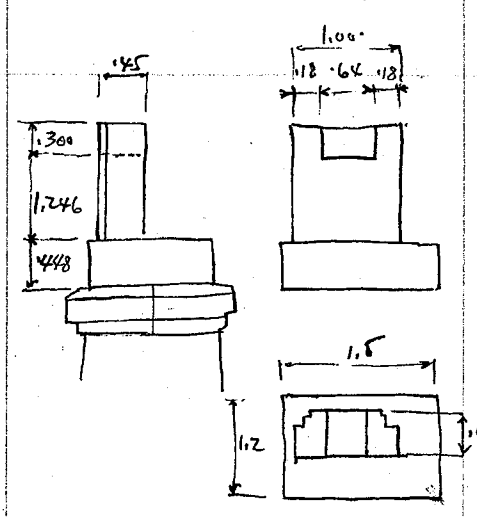
Pier # 16, 17, 18, 19, 20 & 21

Names	Reqd. no	Width	Length	area	Total area	Remarks
Coping	2	·70	7.00	4.90	9.80	Both sides. Straight.
〃	4	·70	1.20	·84	3.36	〃 Curved ends.
					Total = 13.16 □m = (3.98 面坪)	



Pier # 15

Coping	2	·70	7.00	4.90	9.80	
〃	4	·70	1·20	·84	3.36	
Base of shoes	2	·448	5.40	2.42	4.84	sides
〃 Top.	2	1.20	1.50	1.80	3.60	
Column	2	2.90	1.246	3.62	7.24	for conc. beam.
〃	4	1.26	·30	0.38	1.52	〃
				Total =	30.36 □m = (9.19 面坪)	

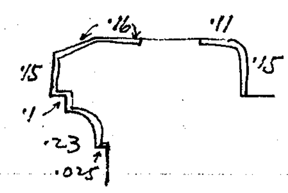


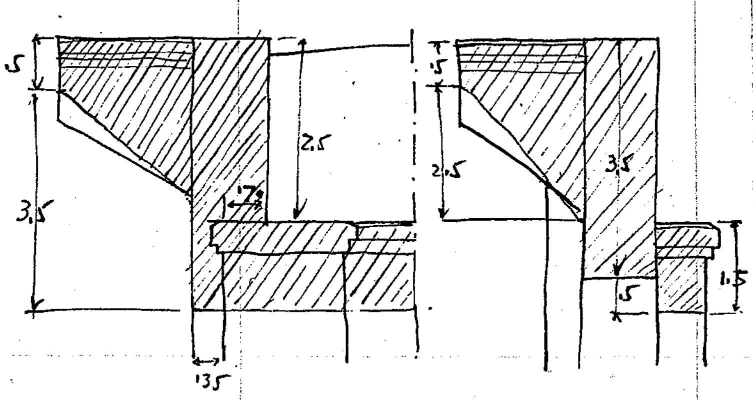
Summary for Articial granite finish for Piers

Pier-nos. 16, 17, 18, 19, 20 & 21 6 @ 13.16 = 78.96
〃 15 1 @ 30.36 = 30.36
109.32 □m = (33.1 面坪)

Artificial granite finish for West abutment.

Name	Reqd. no	Width	Length	area	Total area	Remarks
Coping	2	·665	3.15	2.10	4.20	wing
〃 2nd	2	·58	·45	·26	·52	〃
Curb	2	·26	3.15	·82	1.64	〃
〃 End	2	·225	·50	·11	·22	〃
End plane of coping	2	·42	·375	·16	·32	〃
〃 〃 wing	2	·30	·125	·04	·08	〃
Wing wall	2	1.38	3.60	4.97	9.94	
Column under lamp	2	1.1	3.50	3.85	7.70	
〃 rear	2	·45	3.5	1.58	3.16	
〃 front	2	·35	4.0	1.40	2.80	
〃 〃	2	·70	2.5	1.75	3.50	
Shaft. coping	2	·45	3.9	1.76	3.52	
〃 Sides	2	3.73	1.2	4.47	8.94	
Curtain wall, coping	1.	·45	3.8	1.71	1.71	
〃 face	1.	1.20	3.9	4.68	4.68	
Curb. side of col.	2	·225	1.4	·32	·64	
					53.57 □m = (16.20 面坪)	





Summary of Artificial granite finish for Steel girder spans.

Slab & coping	513.73 □m	= (155.33 面坪)
Handrails	542.74	= (164.28 〃)
Piers	109.32	= (33.10)
Abutment.	53.57	= (16.20)
Total	= 1,219.36 □m	÷ (368.91 面坪)

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MADE BY K. Inaba DATE 2-8-9 FILE NO. ______
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CALCULATIONS FOR

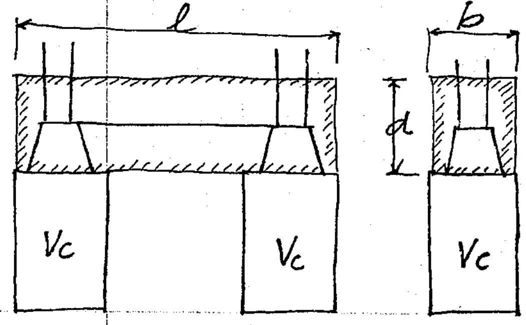
Katsura Bashi for Kyotofu.

Material List.

Excavations. (橋脚ノ堀鑿土量ハ沈井ノ容積ト沈井ノ巾ニ其両端ノ距離ヲ乗ジ更ニ沈井天端ノ根入ヲ乗ジタルモノトノ和ト定ム 即チ下図ニ於テハ 堀鑿土量 = $2V_c + b.d.l$ 但シ橋ム沈井天端以上ヲ五分法ヲ以テ切ルモノトス.)

Piers for Concrete girder spans.

$2V_c = 2\times2.6\times2.6\times3.5 = 47.3\ m^3$



Pier nos.	l		b		d		Vol.		2VC		Total Vol.
P1	8.4	×	2.6	×	1.9	=	41.5	+	47.3	=	88.8
P2	〃		〃		1.1		24.0		〃		71.3
P3	〃		〃		1.1		24.0		〃		71.3
P4	〃		〃		1.9		41.5		〃		88.8
P5	〃		〃		1.9		41.5		〃		88.8
P6	〃		〃		1.9		41.5		〃		88.8
P7	〃		〃		2.7		59.0		〃		106.3
P8	〃		〃		2.5		54.6		〃		101.9
P9	〃		〃		2.5		54.6		〃		101.9
P10	〃		〃		2.5		54.6		〃		101.9
P11	〃		〃		2.5		54.6		〃		101.9
P12	〃		〃		2.7		59.0		〃		106.3
P13	〃		〃		2.7		59.0		〃		106.3
P14	〃		〃		2.7		59.0		〃		106.3

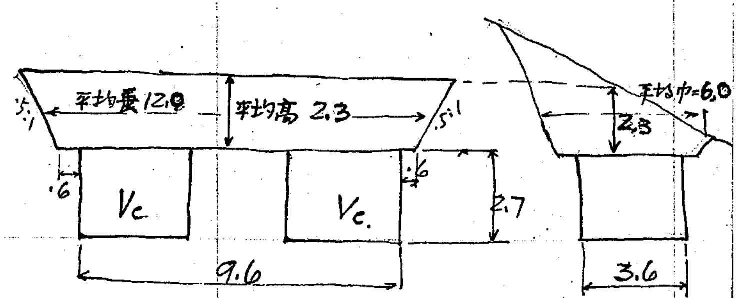
Total = 1,330.6 cub. meter. = (221.4 立坪)

Abutment for concrete girder spans (East abutment)

$2V_c = 2\times3.6\times3.6\times2.7 = 70.0$ cub.m.

Vol above top of caisson } = $12.0\times2.3\times6.0$ = 165.5

Total = 235.5 cub meter = (39.2 立坪)

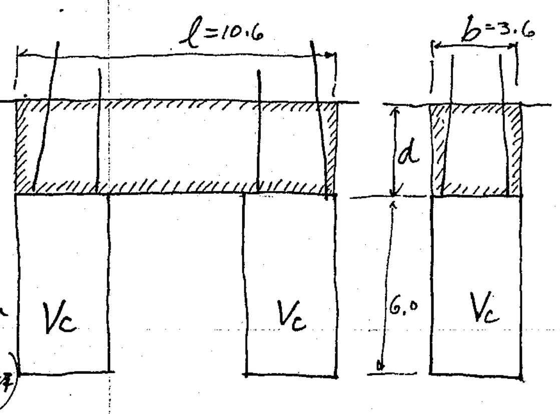


Piers for steel girder span.

Pier nos.	l		b		d		vol		2Vc		Total Vol.
P15	10.6	×	3.6	×	3.8	=	145.1	+	155.5	=	300.6
P16	〃		〃		1.8		68.7		〃	=	224.2
P17	〃		〃		1.8		68.7		〃	=	224.2
P18	〃		〃		1.6		61.0		〃	=	216.5
P19	〃		〃		2.2		83.9		〃	=	239.4
P20	〃		〃		0.0		0		〃	=	155.5
P21	〃		〃		1.4		53.4		〃	=	208.9

Total = 1,569.3 cub.m = (261.0 立坪)

$2V_c = 3.6\times3.6\times6.0\times2 = 155.5\ m^3$

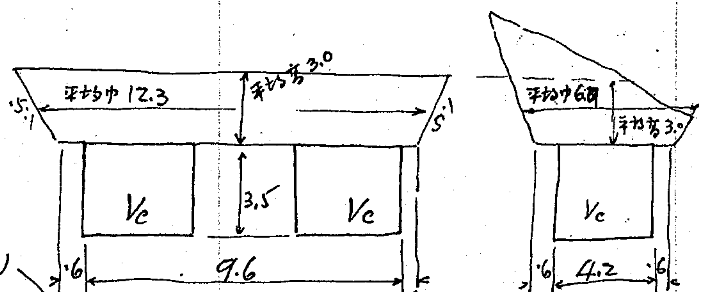


Abutment for steel girder span.

$2V_c = 2\times3.6\times4.2\times3.5 = 105.8$ cub. meter

Vol. above top of caisson } = $12.3\times3.0\times6.9$ = 254.6

Total = 360.4 cub meter = (60.0 立坪)



Summary of Excavation for Sub structures.

Piers for concrete girder spans	1,330.6 cub meter	= (221.4 立坪)
〃 〃 steel 〃 〃	1,569.3	= (261.0)
Abutment for concrete 〃 〃	235.5	= (39.2)
〃 〃 steel 〃 〃	360.4	= (60.0)
Total =	3,495.8 cub m	≑ (581.6 立坪)

Wellヲ除キタル土量
668.4 cub. meter
480.8
165.5
254.6

増田淳氏関係資料 (独立行政法人 土木研究所蔵)

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MADE BY J. Matano DATE 2-8-9 FILE NO.
CHECKED BY DATE PAGE NO. M29

CALCULATIONS FOR

Katsura Bashi for Kyotofu.

BRONZE LAMP POST

22	Bronze lamp globes	@ 45.#	980#
10	" expansion lamp posts	" 210.	2,100
12	" girder span " "	" 200.	2,400
			5,480# = 2.446 Tons = 663.08円

MAIN LAMP POST

4	Bronze top light.	@ 52	208
16	" bracket.	@ 16.5	264
			472# = 0.216 Ton = 57.10円

HAND RAIL

		length in meters	Wt. one piece	Tot. Wt.	Rise in meters	Mark
2	Cast iron hand rail	1.820	@ 185.#	= 370#	0.014	P1
28	" " " "	1.520	134.	3,752	.012	P2
60	" " " "	1.520	"	8,040	.008	P3
58	" " " "	1.520	"	7,772	.003	P4
2	" " " "	1.620	145	290	.002	P5
2	" " " "	1.489	130	260	0	P6
24	" " " "	1.520	134	3,216	0	P7
2	" " " "	1.300	124	248	0	P8
8	" " " "	1.300	"	992	.002	P9
46	" " " "	1.520	134	6,164	.003	P10
2	" " " "	1.550	143	286	.003	P11
8	" " " "	1.300	124	992	.005	P12
46	" " " "	1.520	134	6,164	.006	P13
2	" " " "	1.550	143	286	.006	P14
6	" " " "	1.300	124	744	.008	P15
46	" " " "	1.520	134	6,164	.010	P16
2	" " " "	1.550	143	286	.010	P17
2	" " " "	1.975	200#	400	.014	P18
6	" " expansion joint.		Wt. one set 175#	1,050		
				47,476# = 21.195 Tons = 5744.596円		

NAME PLATES.

4	Bronze	Name of Bridge & date of completion	@ 14#	= 56#
1	"	Names of Responsible Designer, Engineers & Contractors of this bridge	@ 27#	= 27#
				83# = 10.05円

CAST IRON DRAIN PIPE

58	Grate	@ 15#	870#
58	Pipe	@ 35#	2030
		50#	2900# = 1.295 Tons

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MADE BY I. Matano DATE 2-8-11 FILE NO

CHECKED BY M. Kojima DATE 2-8-13 PAGE NO. M30

CALCULATIONS FOR

List of Materials for Katsura-Bashi for Kyoto-fu

List of Rainforcements

	Tons
East Abutment	3.267
West Abutment	4.335
Pier No. 1.2.4.5.7.8.10.11.13 and 14	32.688
Pier No. 3.6.9 and 12	12.470
Pier No. 15 to 21	30.270
Floor of Concrete spans	60.909
Floor Slab of steel spans	25.366
Lamp posts and Hand rails	9.416
	178.721

List of Curb Shoes

			Tons
CS1	for Pier No. 1 to 14	28 req'd.	12.740
CS2	for West Abutment		1.450
CS3	for Pier No. 15 to 21	14 req'd.	9.366
CS3	for East Abutment		1.338
			24.896

Expansion joint and Shoe for Concrete spans

		Tons
Expansion joints for slab	5 req'd.	1.210
Shoes of East Abutment		0.308
Shoes of Concrete Spans		2.168
		3.686