JIUN MASUDA
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JIJI BLDG, TOKYO

MADE BY_____DATE_____FILE NO_____

CHECKED BY_____DATE_____PAGE NO_____

CALCULATIONS FOR

昭和五年十一月
京都府
京阪國道鴨川橋梁材料計算書

JIUN MASUDA
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MADE BY K. Inaba DATE 5-10-25 FILE NO
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CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

The Bridge is located at the crossing of the Kamogawa for the new Kei-Han National Road which is in the course of execution.
The bridge site is just at the downstream of the Nara Electric Railway Bridge 1,110" about.
The center line of the new bridge is 74° oblique to the stream line and also to the banks of the river. The roadway is 11.0 meters wide and is located at the center of the bridge both sides of which are 2 sidewalks 2.0 meters wide each. The total width of roadway is 15.0 meters.
The structure will have 6 arch spans each 18.0 meters in effective. The total length of the bridge is 115.812 meters back to back of parapet walls of the abutments.
The grade shall be of a parabolic curve for the portion of arch spans giving a camber of 0.3 meter.

Assumed Loadings.

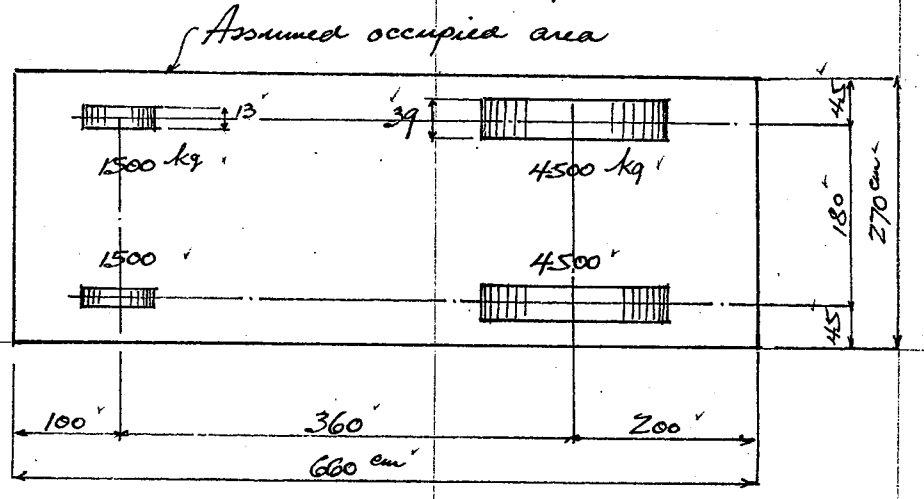
Uniform live load on Roadway $w = \frac{120000}{170+l} \leqq 600$ kg per sq. meter.

Uniform live load on Sidewalk $w = \frac{100000}{170+l} \leqq 500$ " " "

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

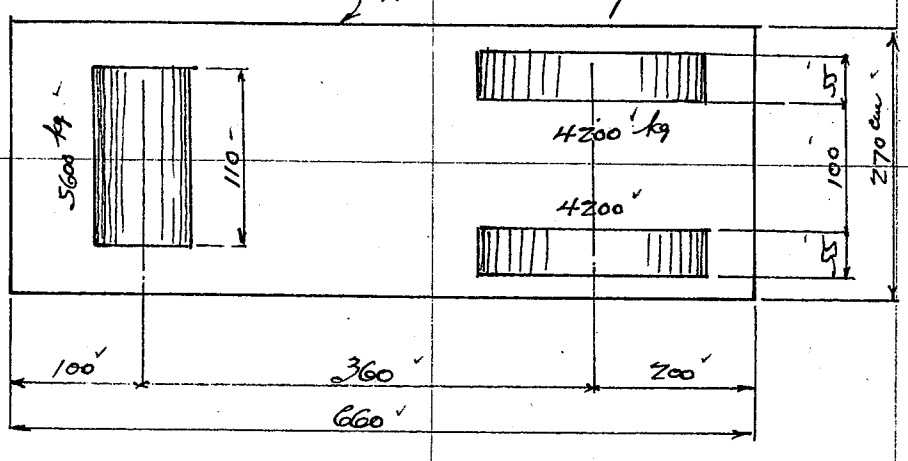
Motor truck loadings.

12 ton motor truck loading.



4 rows of motor traffic on roadway with occupied width of 270 cm each. Unoccupied space around the motor truck shall be filled with said uniform load specified.

14 ton Road roller loading.



One road roller on one span Around the occupied space of road roller shall be fully loaded with uniform live load specified above.

Impact for motor truck loading

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

No impact for road roller and uniform live load.

Seismic load.

Acceleration of Earthquake specified at 2500 m.m per sec^2 or say $k = 0.250$.

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CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

Allowable working strengths.

Tension in steel	1200	kg/cm²
Compression in steel	1200	"

Concrete 1:2:4 mixture.

Direct compression	35	kg/cm²
Fibre stress due to bending	45	"
Combined stress due to direct and bending, Compression member	35	"
punching shear of concrete	9	"
Shear of plain concrete	4	"
Bearing value	45	"
Bond stress, plain bars	6	"
" " deformed bars.	9	"

Weight of materials assumed

Cast iron	7250	kg. per cub. meter
Bronze	8,555	" " "
Reinforced concrete	2400	" " "
Steel	7850	" " "
plain concrete	2,200	" " "
Cement mortar	1,700	" " "
Stone (granite)	2600	" " "
Sand	1,700	" " "
Earth	1,600	" " "
wood	650	" " "
asphalt pavement	2,100	" " "

Considering wind or temperature stress in addition to dead, live and impact stress, the allowable working strength increased 25 %. In case of considering seismic stress increase unit stress as 60 %.

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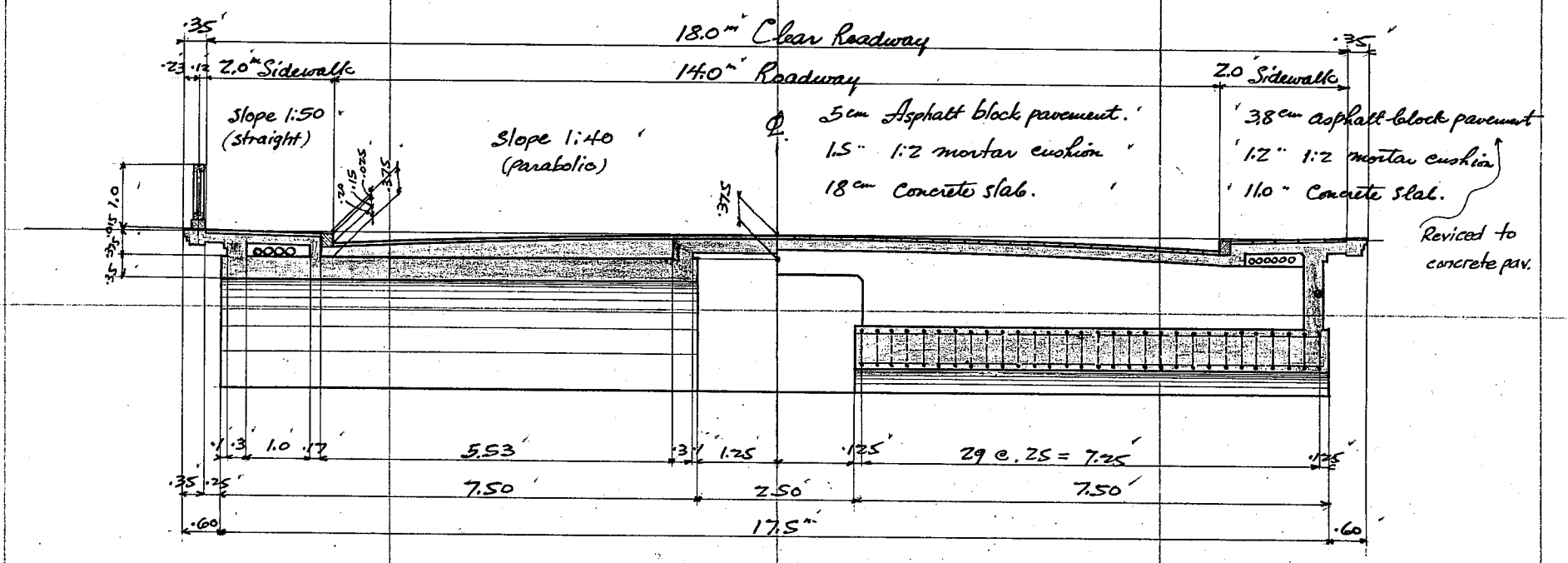
CALCULATIONS FOR

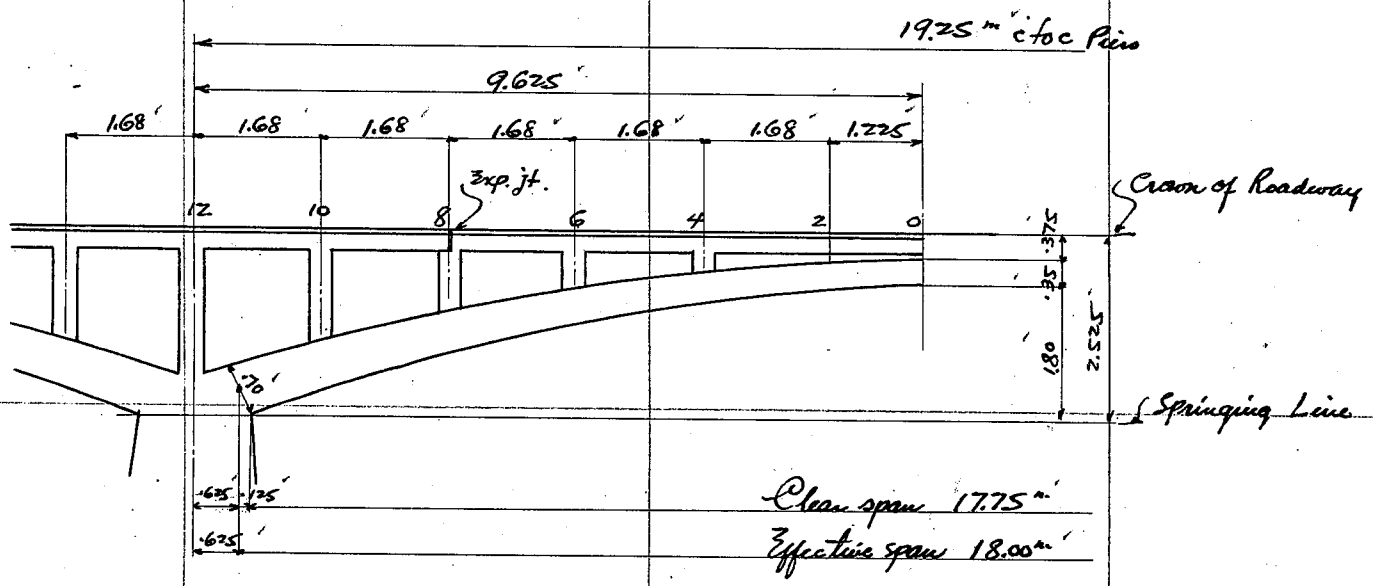
Design of Jōnan Bashi for Kyoto Prefecture.
Cross section of Bridge assumed as shown on sketch below.



Half section at Crown.

Half section near springing.

Scale 1:100.



Longitudinal Section

Scale 1:100.

Design of Floor system

Floor slab between panel point 0 to 4. Span length assumed 2.90 meters between both arch rings.

Dead load.

5cm	Asphalt block pavement	@ 21 kg	= 105.0
1.5	1:2 mortar cushion	@ 17	= 25.5
23	concrete slab	@ 24	= 552.0
	variation say		7.5
			690.0 kg per sq. meter

Dead load moment = 1/10 × 690 × 2.9² = 580 kgm
Dead load shear = 1/2 × 690 × 2.9 = 1000 kg

Live load :- motor truck loading

Rear wheel concentration	=	4,500
30 % impact	=	1350
		5850 kg

Front wheel concentration with impact say
1/3 × 5850 = 1,950 kg.

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CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

Distribution of wheel concentration on slab.

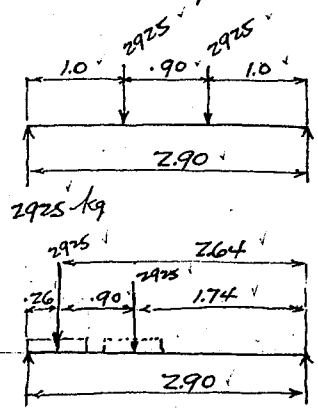
Thickness of pavement and cushion = 6.5 cm

Longitudinal distribution a. Contact between wheel and pavement = 20
distribution 2×6.5 = 13
a = 33 cm

Transverse distribution b
Effective width of slab $E = \frac{2}{3} \cdot l + a$ where l = span length.
$= \frac{2}{3} \times 2.90 + .33$
$= 2.26$ meters use 2.0 meters.
39.0 + 2×6.5 = 52
b = 52 cm

Load per meter strip of slab = 5850 ÷ 2.0 = 2925 kg

Moment per meter strip.



moment = 2925 × 1.0 = 2925
For continuity of slab moment = 0.8 × 2925 = 2340 kgm

max. 2nd shear.
Rear wheel $2925 \times \frac{2.64}{2.90} = 2660$
$2925 \times \frac{1.74}{2.90} = 1755$
4415 kg

Summary for moments and end shears.

	moment	2nd shear.
Dead Load	580	1000
Live Load	2340	4415
	2920 kgm	5415 kg

Effective depth required for $f_s = 1200$ kg/cm², $f_c = 45$ kg/cm²
$R = \frac{M}{6d^2}$ or $d = \sqrt{\frac{M}{6R}}$ where $R = 7.18$

$d = \sqrt{\frac{2920 \times 100}{100 \times 7.18}} = 20.15$ cm

use 20.0 cm effective depth with 3 cm insulation.

Steel area required $= \frac{M}{f_s j d} = \frac{2920 \times 100}{1200 \times \frac{7}{8} \times 20.} = 13.92$ cm²

use 16 mm φ bars at 13 cm c to c = 15.48 cm² / m strip.

Steel ratio $p = \frac{15.48}{20 \times 100} = .00774$

$k = .379$

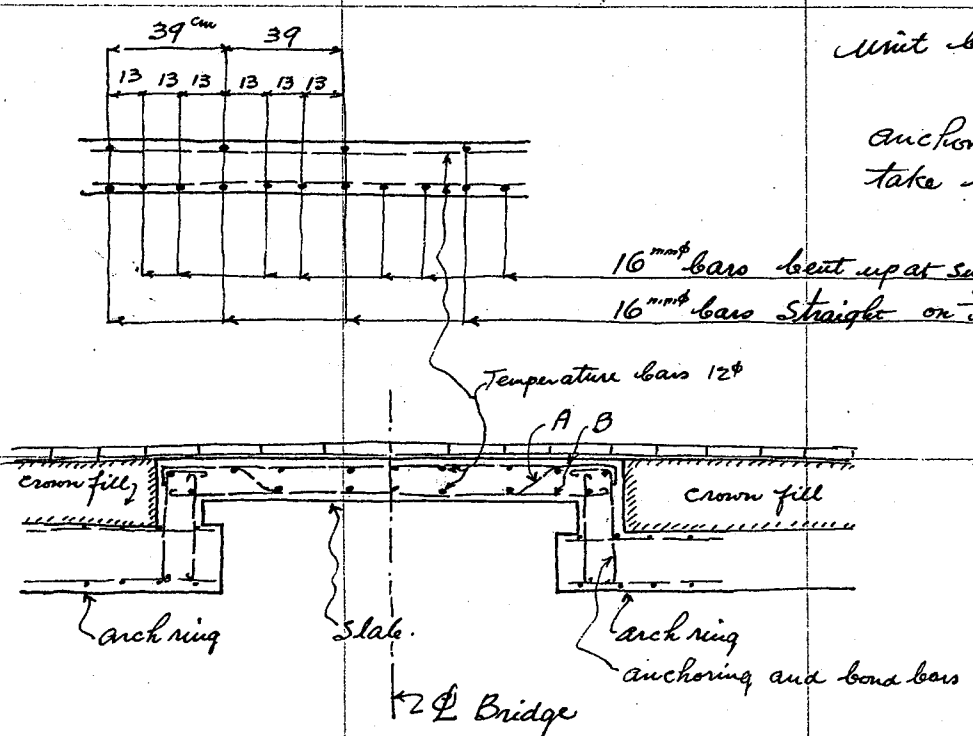
$f_s = \frac{M}{A_s j d} = \frac{2920 \times 100}{15.48 \times \frac{7}{8} \times 20} = 1077$ kg/cm² ok

$f_c = \frac{f_s k}{n(1-k)} = \frac{1077 \times .379}{15(1-.379)} = 43.8$ kg/cm² ok.

Unit Shear $= \frac{5415}{100 \times \frac{7}{8} \times 20} = 3.09$ kg/cm² ok

Unit bond $= \frac{5415}{2 \times \frac{5.03}{.13} \times \frac{7}{8} \times 20} = 4.00$ kg/cm² ok.

anchor bars at support shall be prolonged enough to take bond stress.



For temperature bars
23 × 300 = 6900 cm² concrete area
Steel area req'd. $= \frac{6900}{1000} \times 3 = 20.7$ cm²

use 18 − 17 mm φ bars = 20.5 cm²

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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Roadway floor slab between panel points 4 and 12. Span length 1.68 meters.

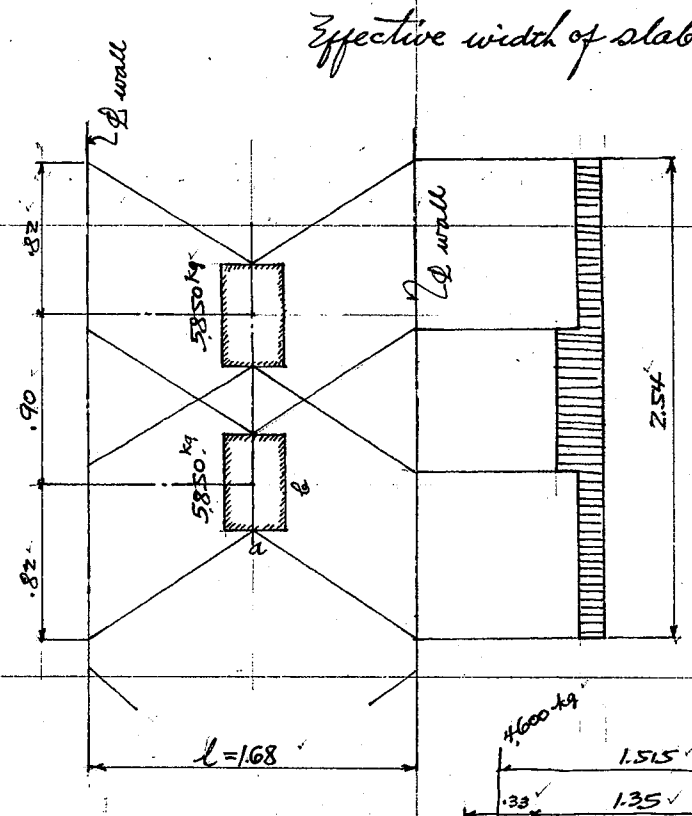
Dead Load :-

5 cm	Asphalt block pavement	@ 21 kg	=	105.0 kg
1.5 "	1:2 mortar cushion	@ 17	=	25.5 "
18 "	Concrete slab	@ 24.	=	432.0 "
	variation say		=	7.5 "
				570 kg per sq. meter.

Dead Load moment = 1/10 × 570 × 1.68² = 160 kgm.
Dead Load shear = 1/2 × 570 × 1.68 = 480 kg

Live Load :- motor truck loading
Rear wheel concentration with impact = 5850 kg.
Distribution of wheel concentration a = 33 cm (see page 4)
b = 52 "

Effective width of slab $E_1 = \frac{2}{3} \cdot l + b$.
$= \frac{2}{3} \times 1.68 + .52 = 1.64$ meters for one wheel.



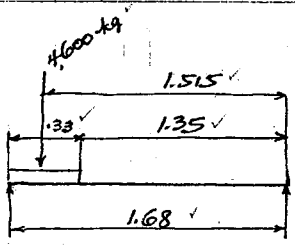
Effective width for 2 consecutive wheels in the nearest distance
$E_2 = 1.64 + 0.90 = 2.54$ metres

Load per meter strip of slab.
$= 5850 \times 2 \div 2.54 = 7430$ 4600 kg

Moment per meter strip of slab.
$\frac{4600}{2} \times \frac{1.68}{2} = 1930$

For continuity of slab moment = 1930 × .80 = 1545 kgm

Max. end shear.
$4600 \times \frac{1.515}{1.68} = 4150$ kg



Summary for moments and end shears.

	Moment	Shear
Dead Load	160	480
Live Load	1545	4150
	1705 kgm	4630 kg.

Effective depth required $d = \sqrt{\frac{1705 \times 100}{100 \times 7.18}} = 15.4$ cm

Use 15.5 cm effective depth with 2.5 cm insulation

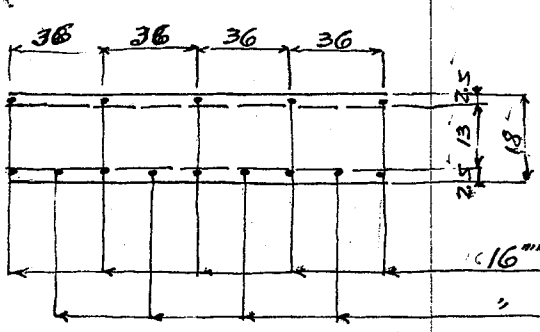
Steel area required = $\frac{1705 \times 100}{1200 \times \frac{7}{8} \times 15.5} = 10.47$ cm²

use 16 mm ϕ bars at 18 cm c to c = 11.20 cm²

Unit shear = $\frac{4630}{100 \times 7/8 \cdot 15.5} = 3.42$ kg/cm². ok

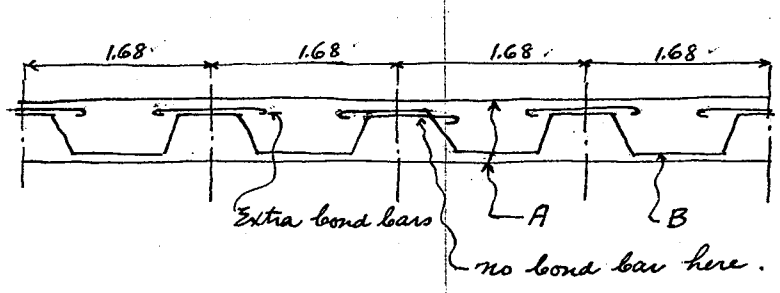
Perimeter of bars for bond stress.

main bars	16ϕ 18 c to c	5.03 × 5.56 =	28.0
extra bond bars	" "	" " " =	28.0
			56.0 cm



Unit bond = $\frac{4630}{56 \times \frac{7}{8} \cdot 15.5} = 6.08$ kg/cm²

Practically ok



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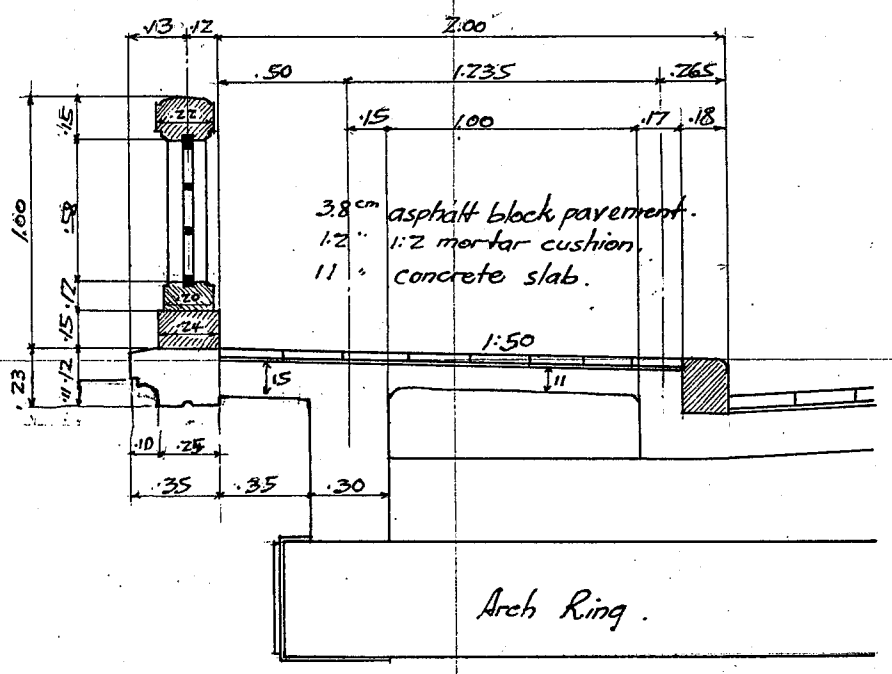
CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Design of Sidewalk slab



Dead Load moment

Handrail	385	× .62 =	239
Coping	165	× .66 =	109
Slab + pavement	230	× .25 =	58
	780 kg	.52	406 kgm.

Live Load.

Uniform live load on sidewalk = 500 kg/m²

moment = $\frac{500 \times .5^2}{2}$ = 63 kgm

shear = 500 × .5 = 250 kg.

Horizontal force on handrail 70 kg per lin meter.

moment = 70 × 1.0 = 70 kgm.

total live load moment = 63 + 70 = 133 kgm.

Summary for moments and shears.

	moment	end shear.
Dead Load	406	780
Live Load.	133	250
	539 kgm	1030 kg.

Effective depth required $d = \sqrt{\frac{539 \times 100}{100 \times 7.18}}$ = 8.7 cm

Use 12.5 cm eff. depth with 2.5 cm insulation, Total depth = 15.0 cm.

Steel area required = $\frac{539 \times 100}{1200 \times \frac{7}{8} \times 12.5}$ = 4.11 cm² per meter strip.

use 12 mm ϕ bars at 15 cm c to c = 7.54 " " "

unit shear = $\frac{1030}{100 \times \frac{7}{8} \times 12.5}$ = .94 kg/cm² ok.

unit bond = $\frac{1030}{377 \cdot 6.67 \cdot \frac{7}{8} \times 12.5}$ = 3.74 " " ok.

Design of Overhanging slab beyond spandrel wall.

Dead Load :-

Approximate weight of Handrail for 3.13 m panel.

Granite

Coping	.24 × .16	=	.0384
bott. rail	.12 × .20	=	.0240
top rail	.15 × .22	=	.0330
	.0954 × 2.76	=	.263
Int. posts.	.15 × .16 × .58 × 2	=	.028
Panel "	.30 × .37 × 1.08	=	.170
	.411 m³		

.411 @ 2600 = 1070 kg.

Cast iron grate say 3 @ 45 = 135

1,205 kg

weight per lin m = 1205 ÷ 3.13 = 385 kg.

Concrete Coping

.30 × .23 @ 2400 = 165 kg

Overhanging Slab and pavement.

asphalt block	3.8 cm	@ 21 kg =	79.6
mortar cushion	1.2 "	@ 17 " =	20.4
Slab	15 "	@ 24 " =	360.0
			460. kg

460 × .5 = 230 kg.

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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture

Sidewalk Slab

Dead load: —

3.8 cm	Asphalt block pavement	@ 21 kg =	79.8
1.2 "	1:2 mortar cushion	@ 17 " =	20.4
11.0 "	Concrete slab	@ 24 " =	264.0
			5.8
			370.0 kg/sq. meter.

Dead load moment = 1/10 × 370 × 1.235² = 56 kgm
Dead load shear = 1/2 × 370 × 1.235 = 230. kg

Live load. Uniform live load on sidewalk = 500 kg/m²

Live load moment = 1/10 × 500 × 1.235² = 76 kgm
Live load shear = 1/2 × 500 × 1.235 = 309 kg.

Summary for moments and shears.

	Positive moment	negative moment	Shear.
Dead load	56	56	230
Live load.	76		309
	−406	−539	539/1.235 = 437
	−274 kgm	−483 kgm.	996 kg

Effective depth required $d = \sqrt{\frac{483 \times 100}{100 \times 7.18}} = 8.2$ cm

use effective depth of 8.5 cm with insulation of 2.5 cm total depth = 11.0 cm.
Steel area required = $\frac{483 \times 100}{1200 \times \frac{7}{8} \times 8.5} = 5.41$ cm² per meter strip.
use 12 mm ϕ bars at 15 cm c to c = 7.54 " " " on top.
unit shear = $\frac{976}{100 \times \frac{7}{8} \times 8.5} = 1.31$ kg/cm² ok.
unit bond = $\frac{976}{3.77 \times 6.67 \times \frac{7}{8} \times 8.5} = 5.21$ " ok.

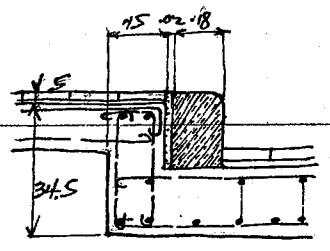
Uplift force on support due to max. negative moment.
= $\frac{-483}{1.235} = 391$ kg per meter strip.
Steel area required = $\frac{391}{1200} = .326$ cm²
use 12 mm ϕ bars at 30 cm c to c = 3.76 cm²

temperature bars. $11 \times 123.5 \times \frac{3}{1000} = 4.08$ cm²
use 4 − 12 ϕ bars = 4.52 cm²

Beam along curb stone supporting sidewalk slab. span length = 1.68 meters.
moment = 1/10 × 391 × 1.68² = 110 kgm.
Shear = 1/2 × 391 × 1.68 = 329 kg.

Effective depth required = $\sqrt{\frac{329 \times 100}{15 \times 7.18}} = 17.5$ cm

use 31 cm effective depth with 2.5 cm insulation total depth 34.5 cm
Steel area required = $\frac{329 \times 100}{1200 \times \frac{7}{8} \times 32} = .98$ cm²
use 2 − 12 mm ϕ bars = 2.26 cm²



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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

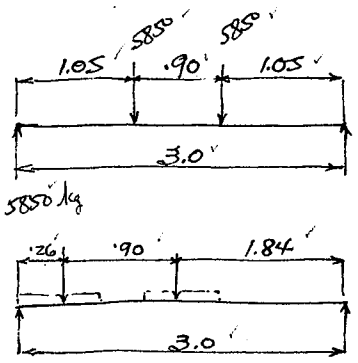
Design of Cross beams at panel points 6 to 12. Span length 3.0 meters, spacing 1.68 meters.

Dead load :-

Floor slab and pavement	1.68 × 570	= 958
Stem of beam assumed	.30 × .47 × 2400	= 338
		1296 call this 1300 kg per lin m.

Dead load moment = 1/10 × 1300 × 3.0² = 1,170 kgm
Dead load shear = 1/2 × 1300 × 3.0 = 1,950 kg.

Live load :- motor truck rear wheel concentration with impact = 5850 kg.



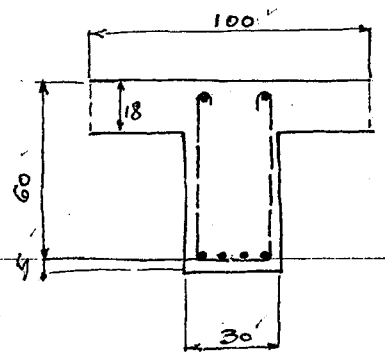
Live load moment 5850 × 1.05 = 6140
For continuity of beam moment = 0.8 × 6140 = 4,910 kgm.

max. end shear.

5850 × 1.84	=	10770
5850 × 2.74	=	16030
		26800 ÷ 3.0 = 8940 kg

Summary for moments and shears.

	Moments	Shears.
Dead load	1170	1950
Live load	4910	8940
	6080 kgm	10890 kg



try reinforcement 4 - 19 mm φ bars = 11.34 cm² t/d = 18/60 = .30
Steel ratio ρ = 11.34 / (60 × 100) = .00189 k = .22 .22 × 60 = 13.2 < 18

Neutral axis in the flange.
Effective depth required = √(6080 × 100 / 100 × 7.18) = 29.1 cm

use effective depth 60 cm with an insulation of 5 cm total depth 65 cm
Steel area required = 6080 × 100 / (1200 × 7/8 × 60) = 9.65 cm²

use 4 - 19 mm φ bars = 11.34 cm²
unit shear = 10890 / (30 × 7/8 × 60) = 6.9 kg/cm²

use 12 mm φ U stirrups As = 1.13 × 2 = 2.26 cm²
Stirrup spacing S = 3/2 · As fs j d / V = 3/2 · 2.26 × 1200 × 7/8 × 60 / 10890 = 19.6 cm at support.

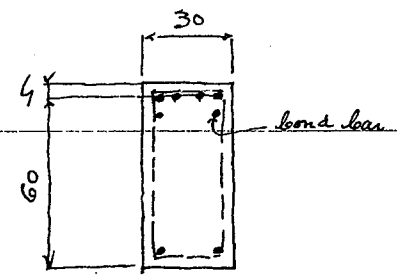
Perimeter of bars for bond = 10890 / (6.0 × 7/8 × 60) = 34.6 cm

use 6 - 19 φ bars = 35.8 cm

Negative moment at support assumed 6080 kgm
Effective depth required = √(6080 × 100 / 30 × 7.18) = 53.1 cm

use 60 cm eff. depth with 5 cm insulation
Steel area required = 6080 × 100 / (1200 × 7/8 × 60) = 9.65 cm²

use 4 - 19 mm φ bars = 11.34 cm²
with 2 - 19 " extra bond bars on top.



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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Cross beam at panel point 4. span length = 3.0 meters.

Dead load :-

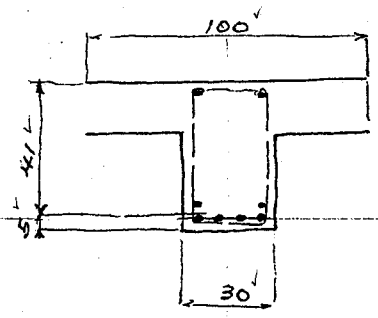
Floor slab and pavement say 1.0 @ 570 = 570
Stem of beam assumed .30 × .28 @ 2400 = 202
772 Call this 770 kg per lin m.

Dead load moment = 1/10 × 770 × 3.0² = 693 kgm
Dead load shear = 1/2 × 770 × 3.0 = 1,155 kg.

Live load :- Live load moments and shears assumed same as for cross beams at 6 to 12.

Summary of moments and shears.

	moment	end shear.
Dead load	693	1,155
Live load	4910	8940
	5603 kgm.	10,095 kg



Neutral axis in the flange.
Effective depth required = $\sqrt{\frac{5603 \times 100}{100 \times 7.18}}$ = 27.95 cm

Use 41 cm eff. depth with 5 cm insulation. total depth 46 cm.

Steel area required = $\frac{5603 \times 100}{1200 \times \frac{7}{8} \times 41}$ = 13.0 cm²

use 6-19φ = 17.0 cm²

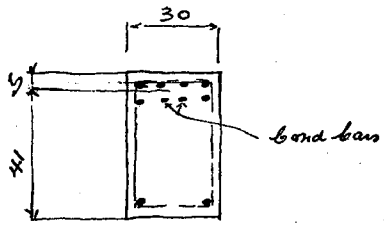
unit shear = $\frac{10095}{30 \times \frac{7}{8} \times 41}$ = 9.36 kg/cm²

use 12 mm φ U-Stirrups A_s = 1.13 × 2 = 2.26 cm²

Stirrup spacing $s = \frac{3}{2} \times \frac{2.26 \times 1200 \times \frac{7}{8} \times 41}{10095}$ = 14.5 cm c to c at support.

Negative moment assumed same as pos. moment or 5603 kgm.

Effective depth required = $\sqrt{\frac{5603 \times 100}{30 \times 7.18}}$ = 51.0 cm



use effective depth 41.0 cm with 5 cm insulation.

For double reinforced beam.

d'/d = 5/41 = .122 $\quad p_1 = \frac{17.0}{30 \times 41} = 0.0138$, $\quad p_2 = \frac{1}{3} p_1$

From the prepared diagrams. $j = .87$, $k = .42$

$f_s = \frac{5603 \times 100}{17.0 \times .87 \times 41}$ = 924 kg/cm² ok

$f_c = \frac{924 \times .42}{15(1 - .425)}$ = 44.6 kg/cm² ok.

Perimeter of bars reqd. for bond = $\frac{10095}{6.0 \times .86 \times 41}$ = 47.7 cm

required no. of 19 mm φ bars for bond = $\frac{47.7}{5.97}$ = 8.0

use 2 − 19φ extra bond bars at support.

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MADE BY K. Inaba DATE 5-10-29 FILE NO
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CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

Design of Transverse spandrel walls. spacing 1.68 meters.

Dead load :—

Road way floor slab and pavement	1.68 × 7.0	@ 570	=	6,703 ✓
Sidewalk " " " "	1.68 × 2.0	@ 370	=	1,245 ✓
Curb stone	1.68 × .18 × .21	@ 2600	=	165 ✓
beam under curb stone say	1.68 × .17 × .140	@ 2400	=	275 ✓
Handrail	1.68	@ 385	=	646 ✓
Coping	1.68	@ 165	=	277 ✓
telephone pipe lines	1.68	@ 100	=	168 ✓
stem of Cross beam	1.35 × .47 × .30	@ 2,400	=	456 ✓
spandrel wall	7.00 × .30 × 1.20	@ 2,400	=	6,045 ✓
miscellaneous say			=	20 ✓
				16,000 ✓ kg

Live load — Motor truck rear wheel concentration with impact 5850 ✓ kg

For 4 wheels @ 5850 = 23,400 ✓ kg

Uniform live load on sidewalk 2 × 500 × 1.68 = 1,700 ✓

Uniform live load on roadway 1.6 × 600 × 1.68 = 1600 ✓

Load on wall = 26,700 ✓ kg with no. moment

Wheels at center of panel and Uniform loads on one side of wall.

Moment due to wheels 23400 × 1.68 ÷ 4 × .8 = 7,870 ✓

do Unif. load, Sidewalk 500 × 2 × 1.68² ÷ 10 = 280 ✓

" " " roadway 600 × 1.6 × 1.68² ÷ 10 = 270 ✓

8420 ✓ kgm.

Vertical load on wall for this case = ½ × 26,700 = 13,350 kg

Summary for vertical load and moment on wall.

	max. vertical load	vertical load	moment.
Dead Load	16,000 ✓	16000 ✓	0 ✓
Live Load	26,700 ✓	13350 ✓	8420 ✓
	42,700 ✓ kg	29,350 kg ✓	8,420 kgm

Try thickness of wall 30 cm ✓ Sectional area of wall = 30 × 700 = 21,000 ✓ cm²

1. Max. unit compression = $\frac{42700}{21000}$ = 2.03 ✓ kg/cm² ok.

2. Eccentricity ε = $\frac{8420}{29,350}$ = 0.287 ✓ m

Try reinforcements 29-12 ✓ mm φ bars on both sides.

$A_s = A'_s$ = 32.8 cm² ✓ $p_0 = \frac{32.8 \times 2}{30 \times 700}$ = .00313 ✓

$f_c = \frac{8420}{.084 \times 700 \times 30^2}$ = 15.9 ✓ kg/cm² ok

$f_s = 15 \times 15.9 \left(\frac{.27}{.26 \times .30} - 1\right)$ = 586 ✓ kg/cm² ok.

ε/h = $\frac{28.7}{30}$ = 0.95 ✓

d'/h = 3/30 = 0.10 ✓

From the prepared diagrams.

k = .260 ✓ , L = .084 ✓

Use Reinforcements, Vertical bars 12 ✓ mm φ bars at 25 cm ✓ c to c on both sides of wall.

horizontal bars 12 mm φ " " 40 " " about on " "

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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Design of Arch Ring.

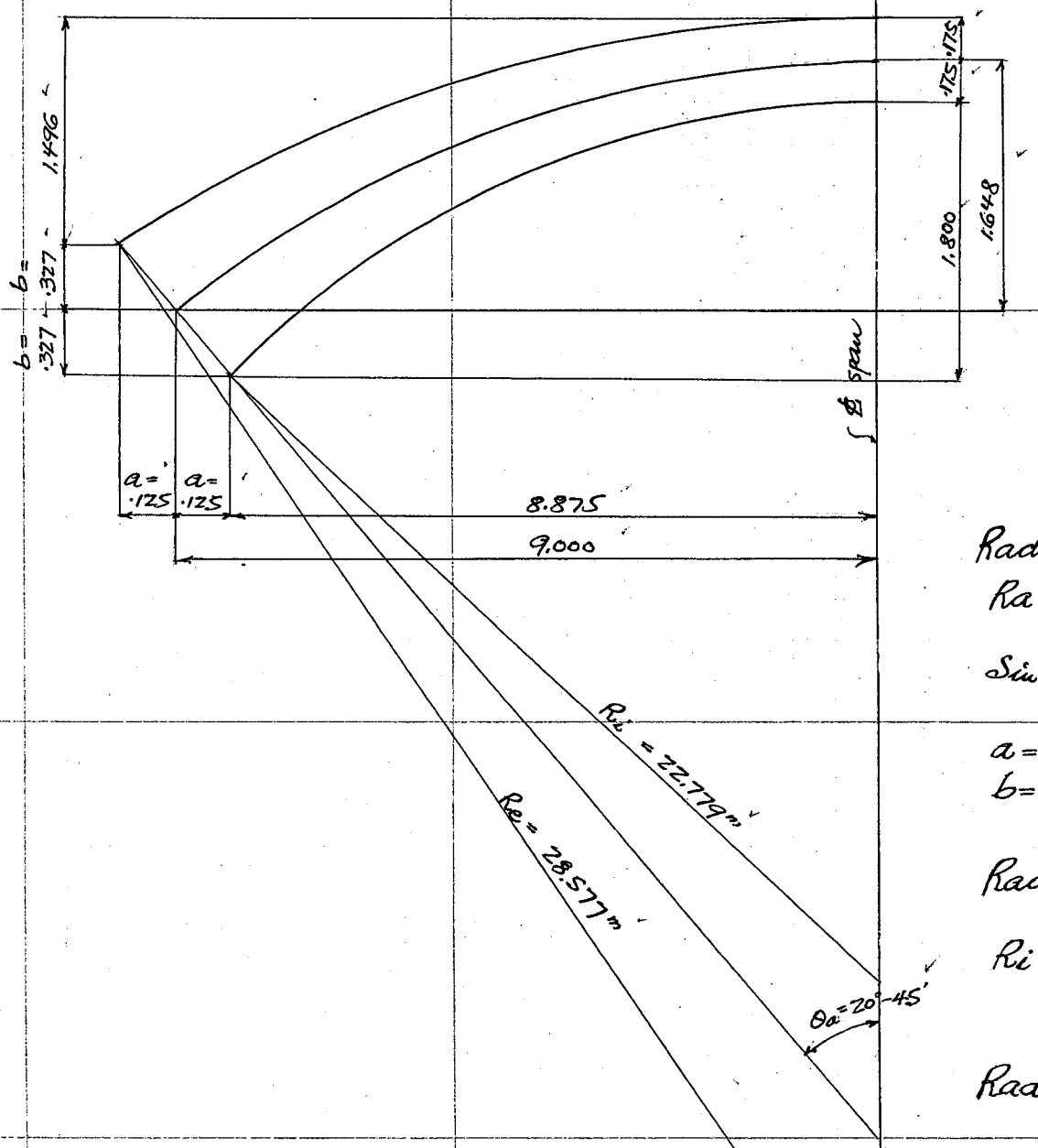
After several investigations on economical standpoint, we have selected the following arch ring as one of the most economical and suitable one to the bridge proposed.

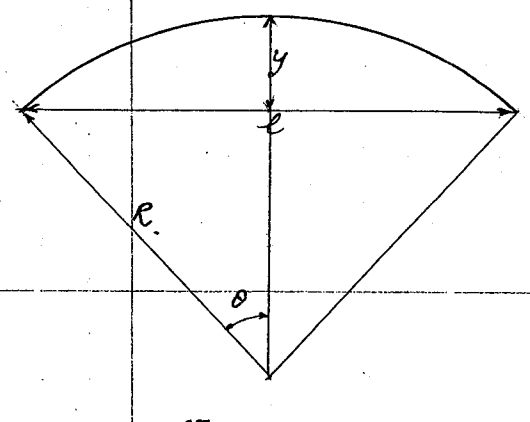
Span length for Neutral axis = 18.00 meters.
Rise " " " = 1.648 " "

Rise ratio = $\frac{1.648}{18.00} = \frac{1}{10.9}$ or 0.092

Thickness of arch ring at crown = 0.35 meter
" " " at springing = 0.70 "

Thickness ratio = $\frac{70}{35} = 2.00$





$$R = \frac{y}{2} + \frac{\ell^2}{8y}$$

Radius of neutral axis (True neutral axis is not circle)

$$Ra = \frac{1.648}{2} + \frac{18.0^2}{8 \times 1.648} = 25.399 \text{ meters.}$$

$$\sin\theta a = \frac{9.00}{25.399} = 0.35434, \quad \theta a = 20°-45'$$

$a = .35 \sin\theta a = .35 \times .35434 = .124$ call this .125 m
$b = .35 \cos\theta a = .35 \times .93514 =$ .327 m

Radius of Intrados curve

$$Ri = \frac{1.800}{2} + \frac{17.75^2}{8 \times 1.800} = 22.779 \text{ meters}$$

Radius of Extrados curve.

$$Re = \frac{1.496}{2} + \frac{18.250^2}{8 \times 1.496} = 28.577 \text{ meters}$$

Length of neutral axis. $S =$

$$2R\theta = 2 \times 25.399 \times \pi \times \frac{20°-45'}{180°-00'} = 18.406 \text{ meters}$$

or $\frac{S}{2} = 9.206$ meters

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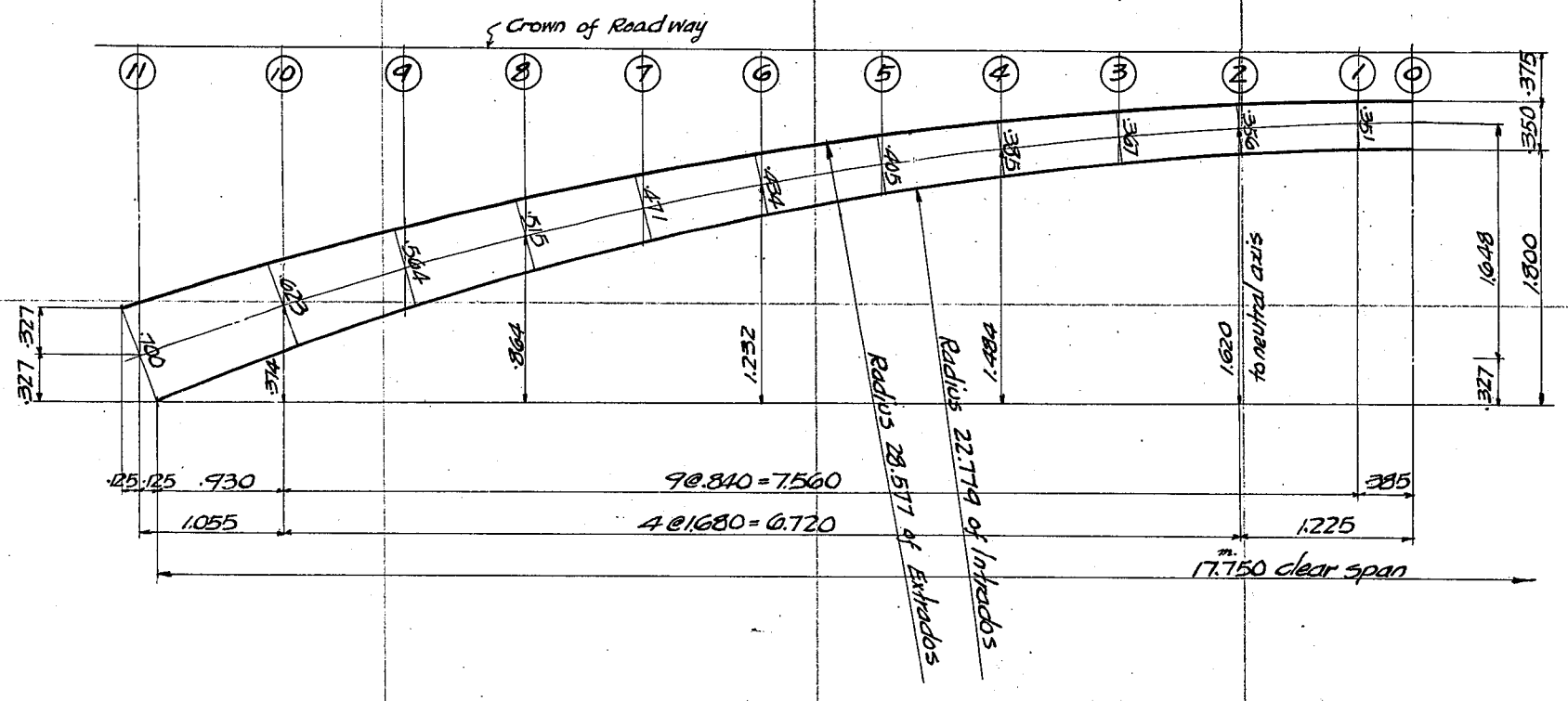
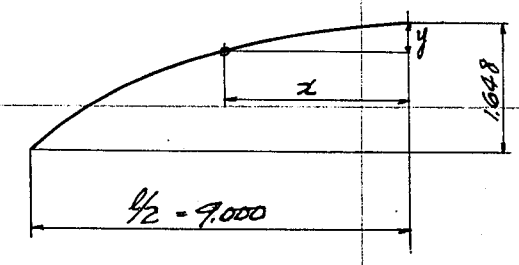
CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.





Panel point		x	y	Thickness	ds
0	Crown	0.000	0.000	0.350	0.193
1		0.385	0.003	0.351	.612
2		1.225	0.030	.356	.842
3		2.065	0.084	.367	.843
4		2.905	0.166	.385	.847
5		3.745	0.278	.405	.849
6		4.585	0.418	.434	.855
7		5.425	0.587	.471	.860
8		6.265	0.786	.515	.868
9		7.105	1.015	.564	.875
10		7.945	1.276	.623	1.000
11	springing	9.000	1.650	.700	.559

$\frac{s}{2} = 9.203$

$s = 18.406$

Design of Arch ring

Reinforcements $25^{mm.\phi}$ bars $25^{cm.}$ c. to c. $= 19.63 \times 2 = 39.26^{\square cm}$ on both side

Equivalent area of 1 Meter strip Reinforcements $= \frac{39.26 \times 14}{10000} = .055^{\square m}$ near Crown, $0.110^{\square m}$ near Springing

		d	d^3	$I_c = \frac{1}{12} d^3$	$(\frac{s}{2} - .045)^2$		Is	Ic + Is
Crown	0	.350	.0429	.00357	.0169	× .055	.00093	.00450
	1	.351	.0432	.00360	.0170	× "	.00094	.00454
	2	.356	.0451	.00376	.0177	× "	.00097	.00473
	3	.367	.0494	.00412	.0192	× "	.00106	.00518
	4	.385	.0571	.00476	.0218	× "	.00120	.00596
	5	.405	.0664	.00553	.0248	× "	.00136	.00689
	6	.434	.0817	.00681	.0296	× "	.00163	.00844
	7	.471	.1045	.00871	.0363	× "	.00200	.01071
	8	.515	.1366	.01138	.0452	× "	.00249	.01387
	9	.564	.1794	.01495	.0562	× .110	.00618	.02113
	10	.623	.2418	.02015	.0710	× "	.00781	.02796
Springing	11	.700	.3430	.02858	.0930	× "	.01023	.03881

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CALCULATIONS FOR

Design of Jōsan Bashi for Kyoto Prefecture

Miscellaneous terms for calculating H_o, M_o, and V_o.

Points		x	x^2	y	y^2	ds	I	$\frac{ds}{I}$	$x\frac{ds}{I}$	$x^2\frac{ds}{I}$	$y\frac{ds}{I}$	$y^2\frac{ds}{I}$
Crown	0	0	0	0	0	.193	.00450	42.89	0	0	0	0
	1	.385	.1482	0.003	.00001	.612	.00454	134.80	51.90	19.98	.04	.001
	2	1.225	1.5006	0.030	.0009	.842	.00473	178.01	218.06	267.12	5.34	.16
	3	2.065	4.2642	.084	.0071	.843	.00518	162.74	336.06	693.96	13.67	1.16
	4	2.905	8.4390	.166	.0276	.847	.00596	142.11	412.83	1199.27	23.59	3.92
	5	3.745	14.0250	.278	.0773	.849	.00689	123.22	461.46	1728.16	34.26	9.52
	6	4.585	21.0222	.418	.1747	.855	.00844	101.30	464.46	2129.55	42.34	17.70
	7	5.425	29.4306	.587	.3446	.860	.01071	80.30	435.63	2363.28	47.14	27.67
	8	6.265	39.2502	.786	.6178	.868	.01387	62.58	392.06	2456.28	49.19	38.66
	9	7.105	50.4810	1.015	1.0302	.875	.02113	41.41	294.22	2090.42	42.03	42.66
	10	7.945	63.1230	1.276	1.6282	1.000	.02796	35.77	284.19	2257.91	45.64	58.24
Springing	11	9.000	81.0000	1.650	2.7225	.559	.03881	14.40	129.60	1166.40	23.76	39.20
				Summary.		9.203		1,119.53	3,480.47	16,372.33	327.00	238.91

		Crown 0				Point ①				Point ②			
Points	x	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
0	0	0	0	0	0								
1	.385	.385	51.90	19.98	.01	0							
2	1.225	1.225	218.06	267.12	6.54	.84	149.53	183.17	4.49	0			
3	2.065	2.065	336.06	693.96	28.23	1.68	273.40	564.58	22.97	.84	136.70	282.29	11.48
4	2.905	2.905	412.83	1199.27	68.53	2.52	358.12	1,040.33	59.45	1.68	238.74	693.55	39.63
5	3.745	3.745	461.46	1728.17	128.30	3.36	414.02	1550.51	115.11	2.52	310.51	1,162.88	86.34
6	4.585	4.585	464.46	2129.55	194.13	4.20	425.46	1950.73	177.83	3.36	340.37	1,560.59	142.26
7	5.425	5.425	435.63	2363.29	255.73	5.04	404.71	2195.58	237.59	4.20	337.26	1829.65	197.99
8	6.265	6.265	392.06	2456.26	308.18	5.88	367.97	2305.31	289.24	5.04	315.40	1975.98	247.92
9	7.105	7.105	294.22	2090.43	298.62	6.72	278.28	1977.16	282.44	5.88	243.49	1730.01	247.14
10	7.945	7.945	284.19	2257.89	362.61	7.56	270.42	2148.48	345.04	6.72	240.37	1909.76	306.70
Sp.	9.000	9.000	129.60	1166.40	213.84	8.615	124.06	1116.50	204.69	7.775	111.96	1007.64	184.73
		Summary	3,480.47	16,372.32	1,864.72		3,065.97	15,032.35	1,738.85		2,274.80	12,152.35	1,464.19

		Point ③				Point ④				Point ⑤			
Points	x	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
3	2.065	0											
4	2.905	.84	119.37	346.78	19.82	0							
5	3.745	1.68	207.01	775.25	57.56	.84	103.51	387.63	28.78	0			
6	4.585	2.52	255.28	1170.44	106.70	1.68	170.18	780.29	71.13	.84	85.09	390.15	35.57
7	5.425	3.36	269.81	1463.72	158.39	2.52	202.36	1,097.79	118.79	1.68	134.90	731.86	79.20
8	6.265	4.20	262.84	1646.65	206.60	3.36	210.27	1,317.32	165.28	2.52	157.70	987.99	123.96
9	7.105	5.04	208.71	1482.87	211.83	4.20	173.92	1,235.72	176.53	3.36	139.14	988.58	141.22
10	7.945	5.88	210.33	1671.04	268.36	5.04	180.28	1,432.32	230.03	4.20	150.23	1,193.60	191.69
Sp.	9.000	6.935	99.86	911.74	164.78	6.095	87.77	789.91	144.82	5.255	75.67	681.05	124.86
		Summary	1,633.21	9,468.49	1,194.04		1,128.29	7,040.98	935.36		742.73	4,973.23	696.50

		Point ⑥				Point ⑦				Point ⑧			
Point	x	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
6	4.585	0											
7	5.425	.84	67.45	365.93	39.60	0							
8	6.265	1.68	105.13	658.66	82.64	.84	52.57	329.33	41.32	0			
9	7.105	2.52	104.35	741.43	105.92	1.68	69.57	494.29	70.61	.84	34.78	247.15	35.31
10	7.945	3.36	120.19	954.88	153.35	2.52	90.14	716.16	115.01	1.68	60.09	477.44	76.68
Sp.	9.000	4.415	63.58	572.18	104.90	3.575	51.48	463.32	84.94	2.735	39.38	354.46	64.98
		Summary	460.70	3,293.08	486.41		263.76	2,003.10	311.88		134.25	1,079.05	176.97

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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture

		Point ⑨				Point ⑩			
Point	x	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$	m	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
9	7.105	0							
10	7.945	.84	30.05	238.72	38.34	0			
SP	9.000	1.895	27.29	245.59	45.03	1.055	15.19	136.73	25.07
		Summary	57.34	484.31	83.37		15.19	136.73	25.07

Crown Thrust $H_o = \frac{\int\frac{ds}{I}\int my\frac{ds}{I} - \int\frac{mds}{I}\int y\frac{ds}{I}}{2\left[\int\frac{ds}{I}\int y^2\frac{ds}{I} - \left(\int y\frac{ds}{I}\right)^2\right]} = \frac{A}{B}$

$B = 2(1,119.53 \times 238.891 - (327.0)^2) = 32,1034.$

Loaded pt.	$\int\frac{ds}{I}$	$\int my\frac{ds}{I}$	Product	$\int m\frac{ds}{I}$	$\int y\frac{ds}{I}$	Product	A	B	
0	1,119.53 ×	1,864.72	2,087,610	− 3,480.47 ×	327.00	1,138,114	944,496 ÷	321.034	= 2.9576
1	〃	1,738.85	1,946,695	− 3,065.97	〃	1,002,572	944,123	〃	= 2.9409
2	〃	1,464.19	1,639,205	− 2,274.80	〃	743,860	895,345	〃	= 2.7889
3	〃	1,194.04	1,336,764	− 1,633.21	〃	534,060	802,704	〃	= 2.5004
4	〃	935.36	1,047,164	− 1,128.29	〃	368,951	678,213	〃	= 2.1126
5	〃	696.50	779,753	− 742.73	〃	242,873	536,880	〃	= 1.6723
6	〃	486.41	544,551	− 460.70	〃	150,649	393,902	〃	= 1.2270
7	〃	311.88	349,159	− 263.76	〃	86,250	262,909	〃	= .8189
8	〃	176.97	198,123	− 134.25	〃	43,900	154,223	〃	= .4804
9	〃	83.37	93,335	− 57.34	〃	18,750	74,585	〃	= .2323
10	〃	25.07	28,067	− 15.19	〃	4,967	23,100	〃	= .0720
SP.	〃								

Crown Moment $M_o = \frac{-H_o 2\int y\frac{ds}{I} + \int m\frac{ds}{I}}{2\int\frac{ds}{I}} = \frac{C}{D}$ $\quad D = 2,239.06$

Loaded pt.	H_o	$\int y\frac{ds}{I}$	Product	$\int m\frac{ds}{I}$	C	D	
0	−2 × 2.9576	327.0	− 1,934.27	+ 3480.47	+ 1,546.20	+ 2,234.06	= + 0.6906
1	2.9409		− 1,923.35	+ 3065.97	+ 1,142.62	〃	= + 0.5103
2	2.7889		− 1,823.94	+ 2274.80	+ 450.86	〃	= + 0.2014
3	2.5004		− 1,635.26	+ 1633.21	− 2.05	〃	= − 0.0009
4	2.1126		− 1,381.64	+ 1128.29	− 253.35	〃	= − 0.1132
5	1.6723		− 1,093.68	+ 742.73	− 350.95	〃	= − 0.1567
6	1.2270		− 802.46	+ 460.70	− 341.76	〃	= − 0.1526
7	.8189		− 535.56	+ 263.76	− 271.80	〃	= − 0.1214
8	.4804		− 314.18	+ 134.25	− 179.93	〃	= − 0.0804
9	.2323		− 151.92	+ 57.34	− 94.58	〃	= − 0.0422
10	.0720		− 47.09	+ 15.19	− 31.90	〃	= − 0.0142
SP.	0		− 0	+ 0	0	〃	= 0

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CALCULATIONS FOR

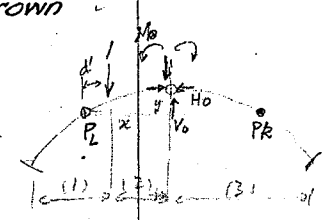
Design of Jōnan Bashi for Kyoto Prefecture.

Crown Shear $V_o = \frac{\int mx \frac{ds}{I}}{2\int x^2 \frac{ds}{I}} = \frac{E}{F}$ $F = 32,744.66$

Loaded pt.	E		F		V_o	$1 - V_o$
0	16,372.32	÷	32,744.66	=	0.5000	0.5000
1	15,032.35	÷	〃	=	0.4591	0.5409
2	12,152.35		〃	=	0.3711	0.6289
3	9,468.49		〃	=	0.2892	0.7108
4	7,040.98		〃	=	0.2150	0.7850
5	4,973.23		〃	=	0.1519	0.8481
6	3,293.08		〃	=	0.1006	0.8994
7	2,003.10		〃	=	0.0612	0.9388
8	1,079.05		〃	=	0.0330	0.9670
9	484.31		〃	=	0.0148	0.9852
10	136.73		〃	=	0.0042	0.9958
SP.	0		〃	=	0	0

Moment at various points for a unit load
let x and y Cordinates of center of sections
d' lever arm of unit load about center of sections, origin at Crown

For left hand sections $M_L = M_o + Hoy + V_o x - d'$ — (1)
" right hand sections $M_R = M_o + Hoy - V_o x$ — (2)
between load + crown $M_L = M_o + Hoy + V_o x$ — (3)



Moment at various points

Points	M_o	Point 1	x = .385	y = .003			Point 2	x = 1.225	y = .030		
		Hoy	$V_o x$	d'	M_L	M_R	Hoy	$V_o x$	d'	M_L	M_R
0	+ 0.6906	0.0089	0.1925	.385	.5070	.5070	0.0887	0.6125	1.225	.1668	.1668
1	+ 0.5103	0.0088	0.1768		.6959	.3423	0.0882	0.5624	.84	.3209	.0361
2	+ 0.2014	0.0084	0.1429		.3527	.0669	0.0837	0.4546		.7397	-.1695
3	- 0.0009	0.0075	0.1113		.1179	-.1049	0.0750	0.3543		.4284	-.2802
4	- 0.1132	0.0063	0.0828		-.0241	-.1897	0.0634	0.2634		.2136	-.3132
5	- 0.1567	0.0050	0.0585		-.0932	-.2102	0.0502	0.1861		.0796	-.2926
6	- 0.1526	0.0037	0.0387		-.1102	-.1876	0.0368	0.1232		.0074	-.2390
7	- 0.1214	0.0025	0.0236		-.0953	-.1425	0.0246	0.0750		-.0218	-.1718
8	- 0.0804	0.0014	0.0127		-.0663	-.0917	0.0144	0.0404		-.0256	-.1064
9	- 0.0422	0.0007	0.0057		-.0358	-.0472	0.0070	0.0181		-.0171	-.0533
10	- 0.0142	0.0002	0.0016		-.0124	-.0156	0.0022	0.0051		-.0069	-.0171

Point	M_o	Point 3	x = 2.065	y = .084			Point 4	x = 2.905	y = .166		
		Hoy	$V_o x$	d'	M_L	M_R	Hoy	$V_o x$	d'	M_L	M_R
0	+ 0.6906	0.2484	1.0325	2.065	-.0935	-.0935	0.4910	1.4525	2.905	-.2709	-.2709
1	+ .5103	0.2470	0.9480	1.680	.0253	-.1907	.4882	1.3337	2.520	-.1878	-.3352
2	+ .2014	0.2343	0.7663	.840	.3620	-.3306	.4630	1.0780	1.680	.0624	-.4136
3	- .0009	0.2100	0.5972		.8063	-.3881	.4151	0.8401	.840	.4143	-.4259
4	- .1132	0.1775	0.4440		.5083	-.3797	.3507	0.6246		.8639	-.3889
5	- .1567	0.1405	0.3137		.2975	-.3299	.2776	0.4413		.5622	-.3204
6	- .1526	0.1031	0.2077		.1582	-.2572	.2037	0.2922		.3433	-.2411
7	- .1214	0.0688	0.1264		.0738	-.1790	.1359	0.1778		.1923	-.1633
8	- .0804	0.0404	0.0681		.0281	-.1081	.0797	0.0959		.0952	-.0966
9	- .0422	0.0195	0.0306		.0079	-.0533	.0386	0.0430		.0394	-.0466
10	- .0142	0.0060	0.0087		.0005	-.0169	.0120	0.0122		.0100	-.0144

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Point	Mo	Point 5 x=3.745 y=.278 Hoy	Vox	d'	ML	MR	Point 6 x=4.585 y=.418 Hoy	Vox	d'	ML	MR
0	+.6906	0.8222	1.8725	3.745	-.3597	-.3597	1.2363	2.2925	4.585	-.3656	-.3656
1	+.5103	.8176	1.7193	3.360	-.3128	-.3914	1.2293	2.1050	4.200	-.3554	-.3654
2	+.2014	.7753	1.3898	2.520	-.1535	-.4131	1.1658	1.7015	3.360	-.2913	-.3343
3	-.0009	.6951	1.0831	1.680	.0973	-.3889	1.0452	1.3260	2.520	-.1497	-.2817
4	-.1132	.5873	0.8052	.840	.4393	-.3311	.8831	0.9858	1.680	.0757	-.2158
5	-.1567	.4649	.5689		.8771	-.2607	.6990	.6965	.840	.4088	-.1542
6	-.1526	.3411	.3767		.5652	-.1882	.5129	.4613		.8216	-.1010
7	-.1214	.2277	.2292		.3355	-.1229	.3423	.2806		.5015	-.0597
8	-.0804	.1336	.1236		.1768	-.0704	.2008	.1513		.2717	-.0309
9	-.0422	.0646	.0554		.0778	-.0330	.0971	.0679		.1228	-.0130
10	-.0142	.0200	.0157		.0215	-.0099	.0301	.0193		.0352	-.0034

Point	Mo	Point 7 x=5.425 y=.587 Hoy	Vox	d'	ML	MR	Point 8 x=6.265 y=.786 Hoy	Vox	d'	ML	MR
0	+.6906	1.7361	2.7125	5.425	-.2858	-.2858	2.3247	3.1325	6.265	-.1172	-.1172
1	+.5103	1.7263	2.4906	5.040	-.3128	-.2540	2.3115	2.8763	5.880	-.1819	-.0545
2	+.2014	1.6371	2.0132	4.200	-.3483	-.1747	2.1921	2.3249	5.040	-.3216	.0686
3	-.0009	1.4617	1.5689	3.360	-.3243	-.1021	1.9653	1.8118	4.200	-.4238	.1526
4	-.1132	1.2401	1.1664	2.520	-.2267	-.0395	1.6605	1.3470	3.360	-.4657	.2003
5	-.1567	.9816	0.8241	1.680	-.0310	.0008	1.3144	0.9517	2.520	-.4106	.2060
6	-.1526	.7202	.5458	.840	.2734	.0218	.9644	.6303	1.680	-.2379	.1815
7	-.1214	.4807	.3320		.6913	.0273	.6437	.3834	.840	.0657	.1389
8	-.0804	.2820	.1790		.3806	.0226	.3776	.2067		.5039	.0905
9	-.0422	.1364	.0803		.1745	.0139	.1826	.0927		.2331	.0477
10	-.0142	.0423	.0228		.0509	.0053	.0566	.0263		.0687	.0161

Point	Mo	Point 9 x=7.105 y=1.015 Hoy	Vox	d'	ML	MR	Point 10 x=7.945 y=1.270 Hoy	Vox	d'	ML	MR
0	+.6906	3.0020	3.5525	7.105	.1401	.1401	3.7739	3.9725	7.945	.4920	.4920
1	+.5103	2.9850	3.2619	6.720	.0372	.2334	3.7526	3.6475	7.560	.3504	.6154
2	+.2014	2.8307	2.6367	5.880	-.2112	.3954	3.5586	2.9484	6.720	-.0116	.8116
3	-.0009	2.5379	2.0548	5.040	-.4482	.4822	3.1905	2.2977	5.880	-.3927	.8918
4	-.1132	2.1443	1.5276	4.200	-.6413	.5035	2.6957	1.7082	5.040	-.7493	.8743
5	-.1567	1.6974	1.0792	3.360	-.7401	.4615	2.1339	1.2068	4.200	-1.0160	.7704
6	-.1526	1.2454	0.7148	2.520	-.7124	.3780	1.5657	0.7993	3.360	-1.1476	.6138
7	-.1214	.8312	.4348	1.680	-.5354	.2750	1.0449	.4862	2.520	-1.1103	.4373
8	-.0804	.4876	.2345	.840	-.1983	.1727	.6130	.2622	1.680	-.8853	.2703
9	-.0422	.2358	.1052		.2988	.0884	.2964	.1176	.840	-.4682	.1366
10	-.0142	.0731	.0298		.0887	.0241	.0919	.0334		.1111	.0443

Point	Mo	Springing x=9.000 y=1.650 Hoy	Vox	d'	ML	MR
0	+.6906	4.8800	4.5000	9.000	1.0706	1.0706
1	+.5103	4.8525	4.1319	8.615	.8797	1.2309
2	+.2014	4.6017	3.3399	7.775	.3680	1.4632
3	-.0009	4.1257	2.6028	6.935	-.2074	1.5220
4	-.1132	3.4858	1.9350	6.095	-.7874	1.4376
5	-.1567	2.7543	1.3671	5.255	-1.2853	1.2355
6	-.1526	2.0246	.9054	4.415	-1.6376	.9666
7	-.1214	1.3512	.5508	3.575	-1.7944	.6790
8	-.0804	.7927	.2970	2.735	-1.7257	.4153
9	-.0422	.3833	.1332	1.895	-1.4207	.2079
10	-.0142	.1188	.0378	1.055	-.9126	.0668

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CALCULATIONS FOR

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Dead Load on Arch Ring. Load on One ring.

Floor.

Roadway floor slab and pavement		7.0 @ 570	=	3,990
Sidewalk " " "		2.0 @ 370	=	740
Curb stone	.18×.21	@ 2600	=	98
beam under curb.	.18×.35	@ 2400	=	151
Handrail	say		=	385
Coping	"		=	165
Telephone pipe lines.	say	4@25	=	100
		misc. say (余分人道其他)		71
				5700 kg per lin meter of span.

Transverse spandrel wall and cross beam at center.

Panel point 4	Spandrel wall	.30×7.00×.22 @ 2400	= 1108		
	Cross beam	.30×1.35×.28 @ "	= 272	1,380 kg	
Panel point 6	Spandrel wall	.30×7.00×.44 @ 2400	= 2,215		
	Cross beam	.30×1.35×.47 @ 2400	= 457	2,672 kg	12896 14896 29792 9508 39300 39300 78600
Panel point 8	Spandrel wall	.30×7.00×.77 @ 2400	= 3880		
	Cross beam		= 457	4,337 kg	
Panel point 10	Spandrel wall	.30×7.00×1.20 @ 2400	= 6050		
	Cross beam		= 457	6,507 kg	

Longitudinal Spandrel wall

Panel point 0	Crown.	.193×.215×.75 @ 2400	=	75 kg
1		.612×.22×.75 @ "	=	242 "
2		.84×.24×.75 @ "	=	365 "
3		.84×.29×.75 @ "	=	438 "
4		.84×.37×.53 @ "	=	395 "
5		.84×.45×.30 @ "	=	272 "
6		.84×.57×.30 @ "	=	345 "
7		.84×.72×.30 @ "	=	435 "
8		.84×.90×.30 @ "	=	544 "
9		.84×1.10×.30 @ "	=	665 "
10		.947×1.33×.30 @ "	=	907 "
11	springing	.528×1.66×.30 @ "	=	631 "
				5312

Crown filling

Panel point 0	crown	.22×.193×.553 @ 2200	=	516 kg
1		.225×.612×" @ "	=	1,675 "
2		.245×.84×" @ "	=	2,500 "
3		.295×.84×" @ "	=	3,010 "
4		.335×.27×" @ "	=	1,100 "
				8801

Floor and pavement for Panel pts 0 to 4

Roadway slab and pavement	1.65 @ 690	=	1,138
Road way pavement on fill	5.35 @ 131	=	701
Sidewalk floor slab and pavement	2.0 @ 370	=	740
Curb stone		=	98
beam under curb	say	=	151
Handrail		=	385
Coping		=	165
Telephone pipe lines		=	100
	misc. say	=	22
			3500 kg per lin meter of span.

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arch Ring.

panel point 0	crown	7.50 ×	.35 ×	.193	@ 2400	=	1215 kg
1		" ×	.351 ×	.612	@ "	=	3870
2		" ×	.356 ×	.842	@ "	=	5395
3		" ×	.367 ×	.843	@ "	=	5570
4		" ×	.385 ×	.847	@ "	=	5870
5		" ×	.405 ×	.849	@ "	=	6190
6		" ×	.434 ×	.855	@ "	=	6680
7		" ×	.471 ×	.860	@ "	=	7290
8		" ×	.515 ×	.868	@ "	=	8040
9		" ×	.564 ×	.875	@ "	=	8880
10		" ×	.623 ×	1.000	@ "	=	11210
11	springing	" ×	.700 ×	.559	@ "	=	7040

Summary of Panel Dead Loads.

Panel point 0 or crown.
- Longitudinal spandrel wall = 75
- Crown filling = 516
- Floor .193 @ 3500 = 675
- arch ring = 1215
- 2481 kg

Panel Point 1.
- Longitudinal spandrel wall = 242
- Crown filling = 1675
- Floor .612 @ 3500 = 2143
- arch ring = 3870
- 7930 kg

Panel point 2.
- Longitudinal spandrel wall = 365
- Crown filling = 2500
- Floor .84 @ 3500 = 2940
- arch ring = 5395
- 11,200 kg

Panel point 3.
- Longitudinal spandrel wall = 438
- Crown filling = 3010
- Floor .84 @ 3500 = 2940
- arch ring = 5570
- 11,958 kg

Panel point 4.
- Floor. .84 @ 5700 = 4790
- " 1.42 @ 3500 = 1470
- Transverse spandrel wall + cross beam = 1380
- Longitudinal spandrel wall = 395
- Coun filling = 1100
- arch ring = 5870
- 15,005 kg

Panel point 5.
- Longitudinal spandrel wall = 272
- arch ring = 6190
- 6462 kg

Panel point 6.
- Floor 1.68 @ 5700 = 9580
- Transv. sp. wall + C.B. = 2672
- longitudinal sp. wall. = 345
- arch ring = 6680
- 19,277 kg

Panel Point 7.
- Longitudinal sp. wall. = 435
- arch ring = 7290
- 7725 kg

Panel Point 8.
- Floor = 9580
- Transv. sp. wall + C.B. = 4337
- longit'dl. sp. wall = 544
- arch ring. = 8040
- 22,501 kg

Panel Point 9.
- Longit'dl. sp. wall. = 665
- arch ring = 8880
- 9545 kg

Panel point 10.
- Floor = 9580
- Transv. sp. wall + C.B. = 6507
- longit'dl sp. wall. = 9071
- arch ring. = 11,210
- 28,204 kg

Panel Point 11 or Springing.
- Longit'dl. sp. wall = 631
- arch ring = 7040
- 7671 kg

Use panel loads in round nos. as follows.

Panel points		Panel loads.
0	crown ------	2480 kg (or 4960/2)
1	--- --- ---	7930
2	— —— —	11,200
3	— ——— --	11,960
4	—— -- ——	15,010
5	--- ——	6460
6	- —— — --	19,280
7	— ---- —	7730
8	— -----	22,500
9	——— --	9550
10	— ———	28,200
11	spr. ——— —	7670

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CALCULATIONS FOR

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Dead load moments at various panel points.

Crown

	Load		Mo		M.
0	2480	+	.6906	+	1,713
1	7,930	+	.5103	+	4,047
2	11,200	+	.2014	+	2,255
3	11,960	−	.0009	−	11
4	15,010	−	.1132	−	1,698
5	6460	−	.1567	−	1,012
6	19,280	−	.1526	−	2,942
7	7,730	−	.1214	−	939
8	22,500	−	.0804	−	1,810
9	9,550	−	.0422	−	403
10	28,200	−	.0142	−	400
11			.0000		0
				+	8015
				−	9215
				−	1,200×2 = −2,400 kgm.

Panel Point	Load	Panel Pt. 2 M unit		Panel Pt. 2 M	Panel Pt. 4 M unit		Panel Pt. 4 M	Panel Pt. 6 M unit		Panel Pt. 6 M	Panel Pt. 8 M unit		Panel Pt. 8 M
11L													
10	28,200	−	.0069	− 195	+ .0100	+	282	+ .0352	+	992	+ .0687	+	1,937
9	9,550	−	.0171	− 163	+ .0394	+	376	+ .1228	+	1173	+ .2331	+	2,228
8	22,500	−	.0256	− 576	+ .0952	+	2,142	+ .2717	+	6110	+ .5039	+	11,330
7	7,730	−	.0218	− 169	+ .1923	+	1,486	+ .5015	+	3,875	+ .0657	+	508
6	19,280	+	.0074	+ 143	+ .3433	+	6,620	+ .8216	+	15,840	− .2379	−	4,585
5	6460	+	.0796	+ 514	+ .5622	+	3,660	+ .4088	+	2,640	− .4106	−	2,652
4	15,010	+	.2136	+ 3,209	+ .8639	+	12,970	+ .0757	+	1137	− .4657	−	6,980
3	11,960	+	.4284	+ 5120	+ .4143	+	4,950	− .1497	−	1,790	− .4238	−	5,070
2	11,200	+	.7397	+ 8,260	+ .0624	+	699	− .2913	−	3,264	− .3216	−	3,600
1	7,930	+	.3209	+ 2545	− .1878	−	1,490	− .3594	−	2,820	− .1819	−	1,442
0	4,960	+	.1668	+ 827	− .2709	−	1,343	− .3656	−	1,815	− .1172	−	581
1	7930	+	.0361	+ 286	− .3352	−	2,658	− .3654	−	2,897	− .0545	−	432
2	11,200	−	.1695	− 1,898	− .4136	−	4,630	− .3343	−	3,740	+ .0686	+	768
3	11,960	−	.2802	− 3,350	− .4259	−	5,090	− .2817	−	3,370	+ .1526	+	1,825
4	15,010	−	.3132	− 4,700	− .3889	−	5,830	− .2158	−	3,237	+ .2003	+	3,005
5	6460	−	.2926	− 1,890	− .3204	−	2,070	− .1542	−	995	+ .2060	+	1,330
6	19,280	−	.2390	− 4,610	− .2411	−	4,650	− .1010	−	1,948	+ .1815	+	3,500
7	7,730	−	.1718	− 1,328	− .1633	−	1,262	− .0597	−	461	+ .1389	+	1,074
8	22,500	−	.1064	− 2,400	− .0966	−	2,175	− .0309	−	695	+ .0905	+	2,035
9	9,550	−	.0533	− 509	− .0466	−	445	− .0130	−	124	+ .0477	+	455
10	28,200	−	.0171	− 482	− .0144	−	406	− .0034	−	96	+ .0161	+	454
11R													
				+ 20904			+ 33,185			+ 31,767			+ 30449
				− 22,270			− 32,049			− 27,252			− 25,342
				− 1,366 kgm.			+ 1,136 kgm.			+ 4,515 kgm.			+ 5,107 kgm.

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Panel Points	Loads.	Panel Point 10 M unit	Panel Point 10 M.	Springing M unit	Springing M.
11 L					
10	28,200	+ .1111	+ 3,135	− .9126	− 25,720
9	9,550	− .4682	− 4,470	− 1.4207	− 13,575
8	22,500	− .8853	− 19,910	− 1.7257	− 38,800
7	7,730	− 1.1103	− 8,590	− 1.7944	− 13,860
6	19,280	− 1.1476	− 22,130	− 1.6376	− 31,580
5	6,460	− 1.0160	− 6,570	− 1.2853	− 8,310
4	15,010	− .7493	− 11,240	− .7874	− 11,820
3	11,960	− .3927	− 4,695	− .2074	− 2,480
2	11,200	− .0116	− 130	+ .3680	+ 4,120
1	7,930	+ .3504	+ 2,778	+ .8797	+ 6,975
0	4,960	+ .4920	+ 2,440	+ 1.0706	+ 5,310
1	7,930	+ .6154	+ 4,880	+ 1.2309	+ 9,760
2	11,200	+ .8116	+ 9,090	+ 1.4632	+ 16,400
3	11,960	+ .8918	+ 10,670	+ 1.5220	+ 18,200
4	15,010	+ .8743	+ 13,125	+ 1.4376	+ 21,580
5	6,460	+ .7704	+ 4,975	+ 1.2355	+ 7,980
6	19,280	+ .6138	+ 11,830	+ .9666	+ 18,640
7	7,730	+ .4373	+ 3,380	+ .6790	+ 5,250
8	22,500	+ .2703	+ 6,080	+ .4153	+ 9,340
9	9,550	+ .1366	+ 1,305	+ .2079	+ 1,985
10	28,200	+ .0443	+ 1,250	+ .0668	+ 1,885
11 R			+ 74,938		+ 127,425
			− 77,735		− 146,145
			− 2,797 kgm		− 18,720 kgm

Dead Load Thrust at several panel points.

Panel Pt.	Load	Ho. unit.	Ho.	Ho	cos θ	Normal thrust	Vert. shear	Sin θ	nor. T.	Total T
0 Cr.	7,480	7.9576	7335	358,660	× 1.000 =	358,660 kg	0	× .000 =	0 kg	358660
1	7,930	7.9409	23340				6,450			
2	11,200	7.7889	31200	"	.999 =	357,000	16,010	.031 =	500	357,500
3	11,960	7.5004	29920				27,590			
4	15,010	7.1126	31,700	"	.993 =	356,000	41,080	.115 =	4720	360,720
5	6,460	1.6723	10,800				51,810			
6	19,280	1.2270	23650	"	.984 =	353,000	64,680	.181 =	11,710	364,710
7	7,730	.8189	6,330				78,190			
8	22,500	.4804	10,805	"	.969 =	347,500	93,300	.248 =	23,150	370,650
9	9,550	.2323	2220				109,330			
10	28,200	.0720	2030	"	.949 =	340,600	128,200	.316 =	40,500	381,100
11 Spr.	7,670	.0	0	"	.935 =	339,000	149,970	.356 =	53,400	392,400
	149,970		179,330							
			179330							
			358,660 kg							

Summary of Dead Load stresses. (in round numbers.)

	Crown 0.	2	4	6	8	10	Springing 11.
Moment	− 2,400 kgm	− 1370	+ 1140	+ 4570	+ 5110	− 2800	− 18720 kgm
Normal thrust	358,660 kg	357,500	360720	364,710	370,650	381,100	392,400 kg
Eccentricity	.007 m	.004	.003	.012	.014	.007	.048 m

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CALCULATIONS FOR

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Live Load Stresses :-

Uniform live Load on Roadway = $\frac{170000^{\vee}}{170+18} = 638^{\vee}$ use $600^{\vee}$ kg/m²

Do on Sidewalk = $\frac{100000^{\vee}}{170+18} = 532^{\vee}$ use $500^{\vee}$ "

Motor truck loading. Impact coefficient = $\frac{20}{60+18}^{\vee} = 25.7^{\vee}\%$

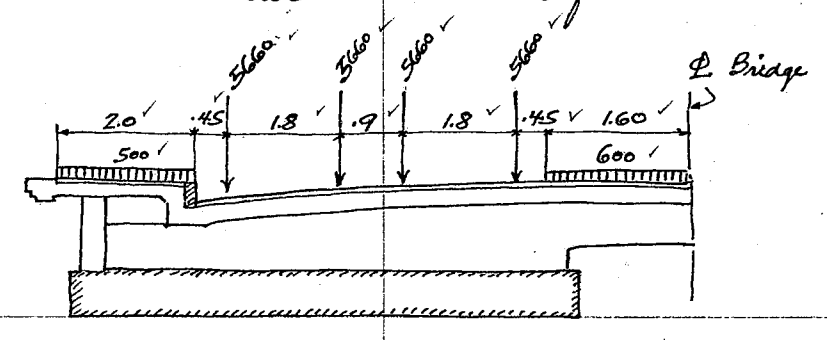
motor truck rear wheel concentration = $4500^{\vee}$

25.7% impact = $1160^{\vee}$

$5660^{\vee}$ kg.

front wheel with impact say $\frac{5660^{\vee}}{3} = 1890^{\vee}$ "

Load manner assumed as follows.



2 motor trucks on one arch rib.

Rear wheels 4 @ 5660 = 22640 kg

Front " 4 @ 1890 = 7560 "

Unif load on Road way

Side of motor truck 600 × 1.60 = 960 kg/lin m.

Front + rear of " " 600 × 7.0 = 4200 "

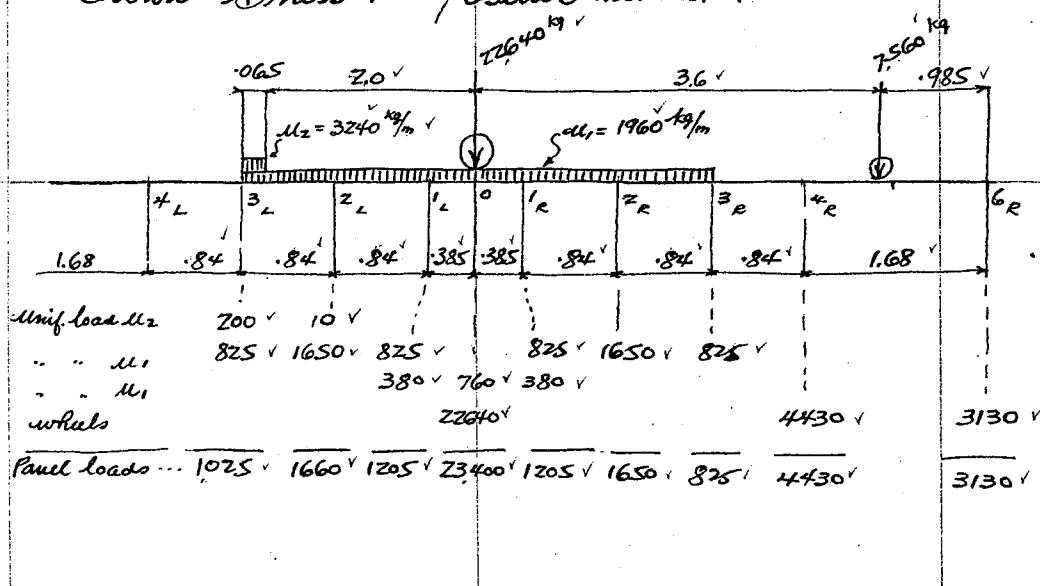
Uniform load on Sidewalk.

2.0 × 500 = 1000 kg/lin m.

Uniform load on sidewalk and side of truck = 1000 + 960 = 1960 kg/lin m = M_1

" " " front + rear of truck = 4200 − 960 = 3240 " = M_2

Crown Stress. Positive moment.



Point	Load	M unit load	Moment	H unit load	H Thrust
3L	1,025 kg	−.0009	− 1	2.5004	2,565
2L	1,660	+.2014	+ 334	2.7889	4,625
1L	1,205	+.5103	+ 615	2.9409	3,545
0	23,400	+.6906	+ 16,150	2.9576	69,200
1R	1,205	+.5103	+ 615	2.9409	3,545
2R	1,650	+.2014	+ 332	2.7889	4,600
3R	825	−.0009	− 1	2.5004	2,065
4R	4,430	−.1132	− 502	2.1126	935
6R	3,130	−.1526	− 478	1.2270	3,840
			+ 18,046		94,920 kg
			− 982		
			+ 17,064 kgm		
		call this	+ 17,060 kgm.		

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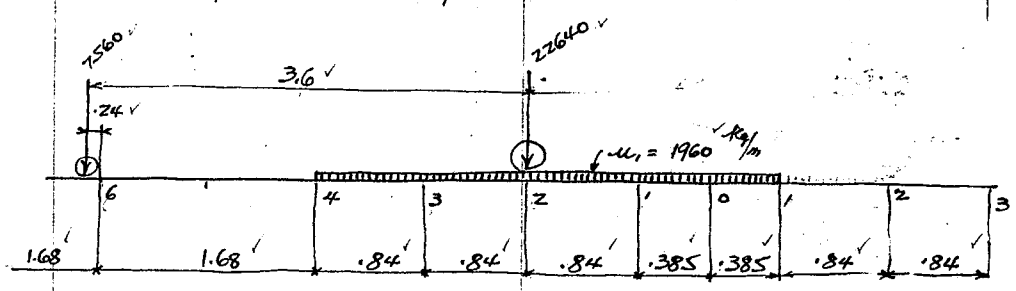
Panel Point 2: positive moment.



6480 825 1650 1650 1205 760 380

22640

1080 6480 825 1650 24290 1205 760 380

Point	Load	M unit load	Moment	H. unit load	Hor. thrust	Shear unit load	Vert. Shear
8	1.080	− .0256	− 28	.4804	.518	.9670 − 1.0 = −.0330	+ 36
6	6480	.0074	48	1.2270	7.950	.8994 − 1.0 = −.1006	= 652
4	825	.2136	176	2.1126	1.742	.7850 − 1.0 = −.2150	− 177
3	1650	.4284	707	2.5004	4130	.7108 − 1.0 = −.2892	− 477
2	24290	.7397	17950	2.7889	67,700	.6289	15270
1	1205	.3209	386	2.9409	3540	.5409	651
0	760	.1668	127	2.9576	2250	.5000	380
1	380	.0361	14	2.9409	1120	− .5409 + 1.0 = .4591	175

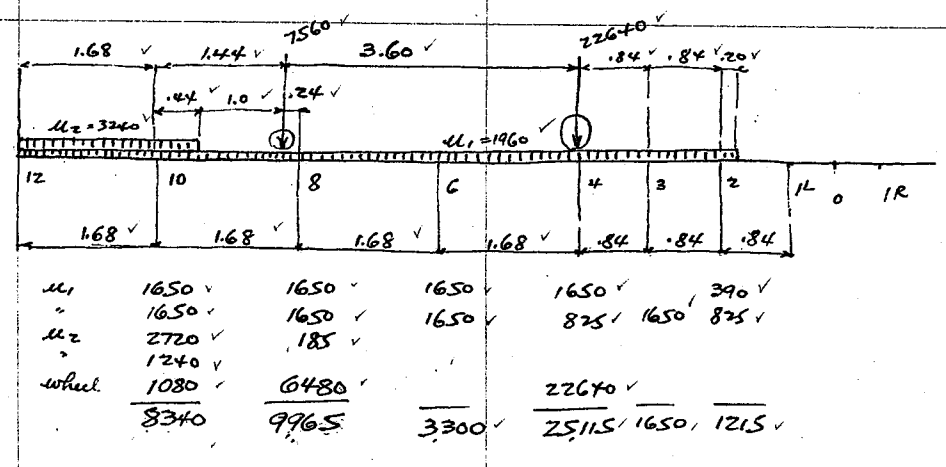
+ 19408
− 28
+ 19380 kgm.

88950 kg

+ 16476
− 1342
+ 15134
call this + 15130 kg

Normal thrust
88950 × .999 = 88900
15130 × .031 = 470
89370 kg.

Panel Point 4. Positive moment.



u_1	1650	1650	1650	1650	390
"	1650	1650	1650	825	1650	825
u_2	2720	185			
"	1240				
wheel	1080	6480		22640	
	8340	9965	3300	25115	1650	1215

Points	Load	M unit load	moment	H unit load	Hor. thrust	Shear unit load	Vert. shear
10L	8340	.0100	83	.0720	600	− .0042	− 35
8	9965	− .0952	1950	.4804	4790	− .0330	− 330
6	3300	.3433	1132	1.2270	4050	− .1006	− 332
4	25115	.8639	21,700	2.1126	53,050	.7850	19720
3	1650	.4143	685	2.5004	4125	.7108	1173
2	1215	.0624	75	2.7889	3385	.6289	764

+ 24625 kgm

70,000 kg

+ 21657
− 697
+ 20960 kg

Normal thrust
70000 × .993 = 69500
20960 × .115 = 2410
71910 kg

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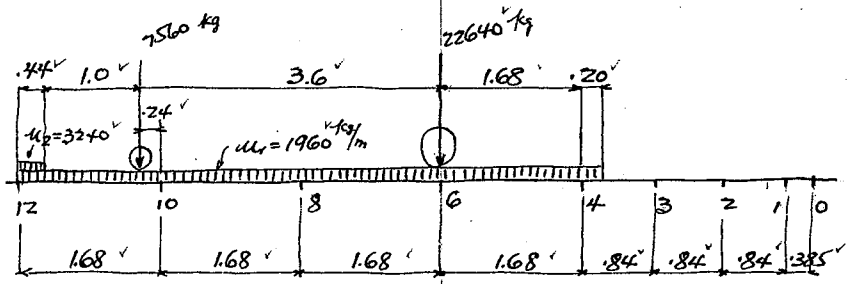
Panel Point 6. Positive moment.

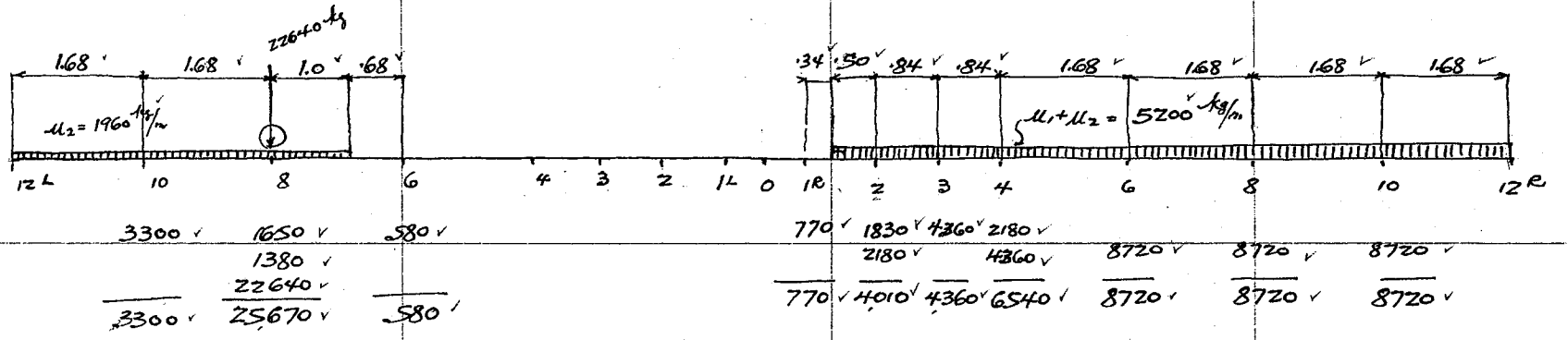


3300 3300 3300 2040
190
6480 22640
9970 3300 25940 2040

Points	Load	M. unit load	moment	H. unit load	Hor. thrust	Shear unit load	Vert. shear
10	9970	.0352	351	.0720	720	−.0042	− 42
8	3300	.2717	896	.4804	1585	−.0330	− 109
6	25940	.8216	21300	1.2270	31,800	+.8996	23320
4	2040	.0757	153	2.1126	4310	+.7850	1600
			+ 22700 kgm		38415 kg		+ 24920
				call this	38420 kg		− 151
							24769
						call this	+ 24770 kg

Normal thrust
38420 × .984 = 37800
24770 × .181 = 4480
42280 kg

Panel Point 8. Positive moment.



3300 1650 580
1380
22640
3300 25670 580

770 1830 4360 2180
2180 4360 8720 8720 8720
770 4010 4360 6540 8720 8720 8720

Points	Load	M. unit load	moment	H. unit load	Hor. thrust	Shear unit load	Vert. shear
10L	3300	.0687	227	.0720	238	−.0042	− 14
8L	25670	.5039	12930	.4804	12325	.9670	24820
6L	580	−.2379	− 138	1.2270	711	.8994	522
1R	770	−.0545	− 42	2.9409	2265	.4591	+ 354
2R	4010	.0686	275	2.7889	11180	.3711	1488
3	4360	.1526	665	2.5004	10915	.2892	1262
4	6540	.2003	1309	2.1126	13820	.2150	1405
6	8720	.1815	1582	1.2270	10690	.1006	878
8	8720	.0905	789	.4804	4185	.0330	288
10R	8720	.0161	140	.0720	627	.0042	37
			+ 17917		66956 kg		+ 31054
			− 180				− 14
			+ 17737 kgm	call this	66960		+ 31040 kg
		call this	+ 17740 kgm				

Normal thrust
66960 × .969 = 64830
31040 × .248 = 7690
72520 kg

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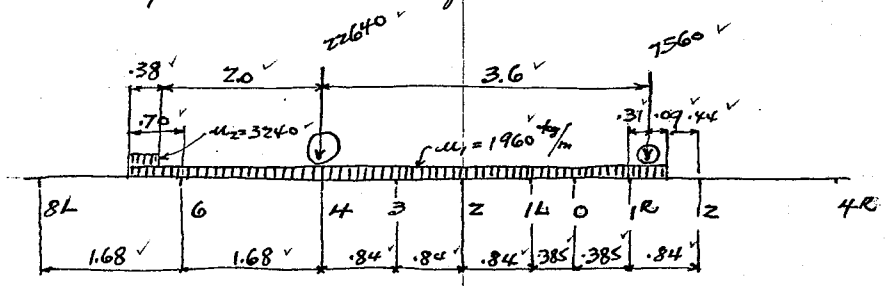
Panel point 8. Negative moment.

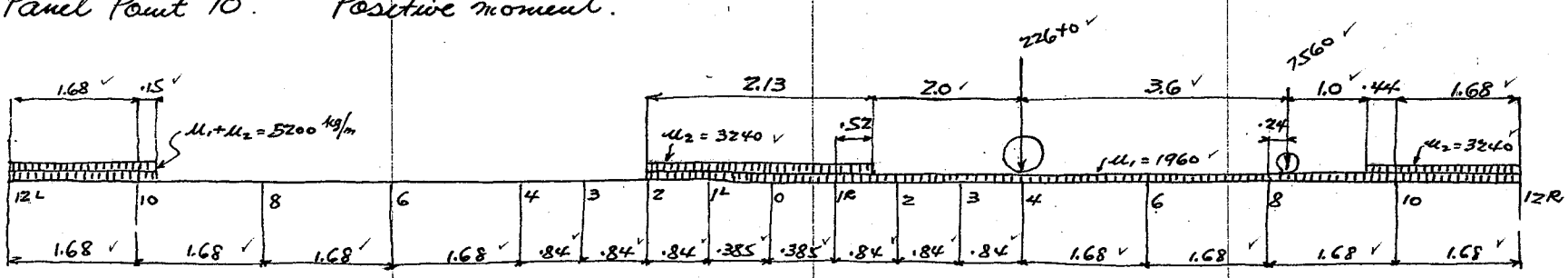


1650 1650 1650 1650 825 760 380
285 1085 825 380 600 180
375 855 22640
4770 2790
660 3590 25115 1650 1650 1205 760 5750 2970

Points	Load	M unit load	Moment	H unit load	Hor. Thrust	Shear unit load	Vert. Shear
8L	660	.5039	+ 333	.4804	317	.9670	638
6	3,590	−.2379	− 854	1.2270	4,405	.8994	3230
4	25,115	−.4657	− 11,700	2.1126	53,060	.7850	19,710
3	1,650	−.4238	− 699	2.5004	4127	.7108	1,173
2	1,650	−.3216	− 530	2.7889	4,600	.6289	1,038
1L	1,205	−.1819	− 219	2.9409	3,545	.5409	652
0	760	−.1172	− 89	2.9576	2,246	.5000	380
1L	5,750	−.0545	− 313	2.9409	16,920	.5409	3,110
2R	2,970	.0686	+ 204	2.7889	8280	.6289	1867
			+ 537		97,500 kg		31,798 kg
			− 14,404				
			− 13,867				
		call this	− 13870 kgm				

Normal thrust
97,500 × .969 = 94,500
31,798 × .248 = 7,880
102380 kg.

Panel Point 10. Positive moment.



say 5150

2180 3180 2000 1000 825 1650 825 3300 3300 3300
825 825 1650 190 1240
1165 510 22640 2720
6480 1086
5150 2180 3180 2000 2990 2170 1650 25115 3300 9970 8340

Points	Load	M unit load	Moment	H unit load	Hor. Thrust	Shear unit load	Vert. Shear
10L	5,150	.1111	573	.0720	370	.9958	5,125
2L	2,180	−.0116	− 25	2.7889	6075	.6289	1,370
1L	3,180	.3504	1,114	2.9409	9350	.5409	1720
0	2,000	.4920	984	2.9576	5915	.5000	1,000
1R	2,990	.6154	1,840	2.9409	8795	.4591	1,375
2	2,170	.8116	1,760	2.7889	6050	.3711	805
3	1650	.8918	1,474	2.5004	4125	.2892	477
4	25,115	.8743	21,970	2.1126	53,070	.2150	5400
6	3,300	.6138	2,025	1.2270	4045	.1006	332
8	9,970	.2703	2,720	.4804	4,785	.0330	329
10R	8,340	.0443	370	.0720	600	.0042	35
			34,805 kgm		103,180 kg		17,968 kg
		call this	34,810 kgm	Normal thrust	103,180 × .949 =		97850
					17,968 × .316 =		5680
							103,530 kg

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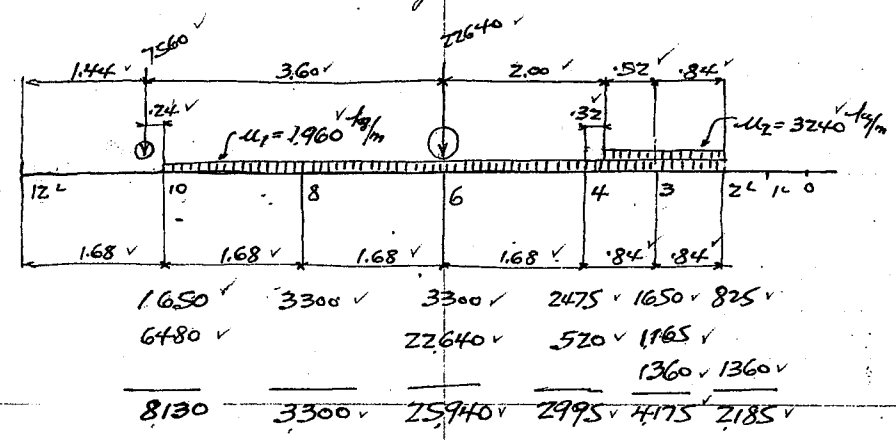
Panel Point 10. Negative moment.



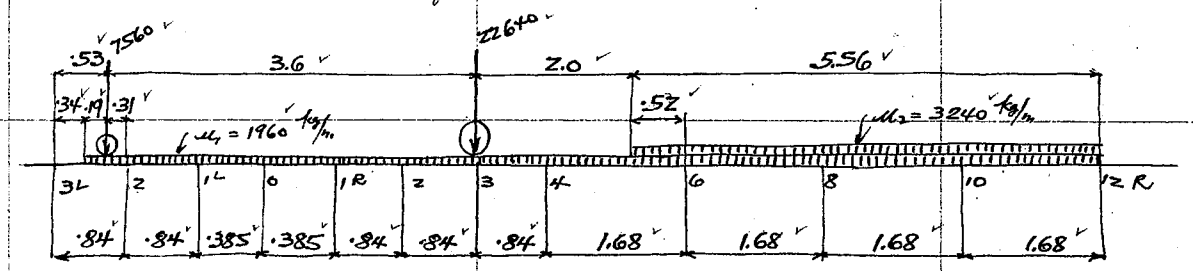
1650 3300 3300 2475 1650 825
6480 22640 520 1165
1360 1360
8130 3300 25940 2995 4175 2185

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	Shear unit load	Vert. Shear.
10 L	8130	+ .1111	+ 903	.0720	585	.9958	8,100
8	3,300	− .8853	− 2920	.4804	1,585	.9670	3,190
6	25940	− 1.1476	− 29750	1.2270	31,800	.8994	23320
4	2,995	− .7493	− 2,245	2.1126	6330	.7850	2,350
3	4175	− .3927	− 1640	2.5004	10,450	.7108	2,970
2 L	2185	− .0116	− 25	2.7889	6090	.6289	1,370
			− 35,677		56,840		41,300 kg.
		call this	− 35,680 kgm.				

Normal Thrust

56,840 × .949 = 53,800
41,300 × .316 = 13080
66,880 kg

Panel Point 11 (Springing) Positive moment.



290 690
2790 4770 380 770 380 1650 1650 825
825 825 825 22640 1650
260
3080 6285 1205 770 1205 1650 24290 2735

3300 3300 3300
1425
2720 5440 5440
7445 8740 8740

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	Shear, unit load	Vert. Shear.
3 L	3,080	− .2074	− 639	2.5004	7,705	.7108	2,190
2	6285	.3680	2,312	2.7889	17,520	.6289	3950
1 L	1,205	.8797	1060	2.9409	3,545	.5409	652
0	770	1.0706	825	2.9576	2275	.5000	385
1 R	1205	1.2309	1484	2.9409	3,545	.4591	553
2	1650	1.4632	2415	2.7889	4600	.3711	612
3	24290	1.5220	36,950	2.5004	60720	.2892	7,020
4	2735	1.4376	3930	2.1126	5775	.2150	588
6	7445	.9666	7200	1.2270	9130	.1006	750
8	8740	.4453	3630	.4804	4197	.0330	288
10 R	8740	.0668	584	.0720	629	.0047	37
			59751		119641 kg		17025 kg
		call this	59750 kgm.	call this	119640		

Normal Thrust

119640 × .935 = 111800 kg
17025 × .356 = 6060
117860 kg

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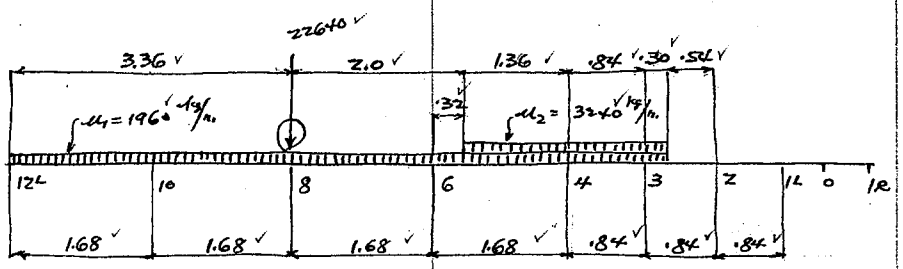
Panel Point 11 (Springing) negative moment.



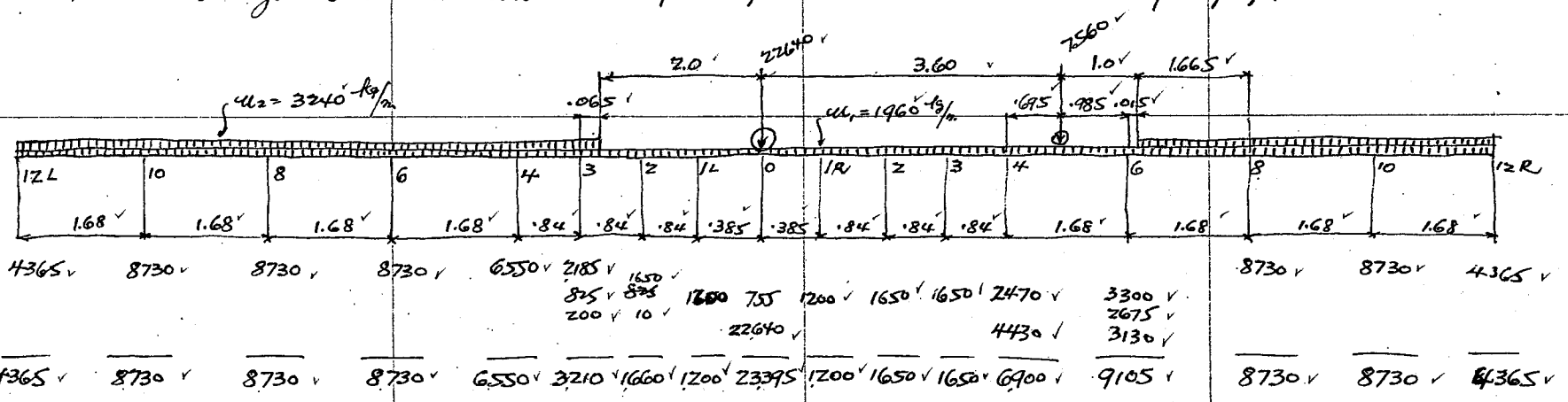
3300 3300 3300 2475 825
22640 485 105
1785 2625 800 175
1360 1360

3300 25940 5085 6460 3470 280

Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	Shear unit load	Vert. Shear
10L	3300	− .9126	− 3,010	.0720	238	.9958	3,287
8	25,940	− 1.7257	− 44,750	.4804	12,450	.9670	25,060
6	5,085	− 1.6376	− 8,325	1.2270	6,235	.8994	4,570
4	6,460	− .7874	− 5,090	2.1126	13,650	.7850	5,070
3	3,470	− .2074	− 720	2.5004	8,680	.7108	2,468
2L	280	.3680	+ 103	2.7889	782	.6289	175
			61,790		42,035 kg		40,630 kg
		call this	− 61,790 kgm	call this	42,040		

Normal thrust
42,040 × .935 = 39,300
40,630 × .356 = 14,460
53,760 kg.

Maximum Horizontal Thrust with the corresponding moment and vert. shear at springing.



4365 8730 8730 8730 6550 2185
825 825 1650
200 10 1650 755 1200 1650 1650 2470 3300
22640 2675
4430 3130

4365 8730 8730 8730 6550 3210 16600 1200 23395 1200 1650 1650 6900 9105 8730 8730 4365

Point	Load	M. unit load	Moment	H unit load	Hor. Thrust	Shear unit load	Vert. Shear
10L	8730	− .9126	− 7970	.0720	628	.9958	8710
8	8730	− 1.7257	− 15090	.4804	4190	.9670	8440
6	8730	− 1.6376	− 14290	1.2270	10700	.8994	7850
4	6550	− .7874	− 5180	2.1126	13820	.7850	5140
3	3210	− .2074	− 665	2.5004	8030	.7108	2280
2	1660	.3680	610	2.7889	4625	.6289	1043
1L	1200	.8797	1055	2.9409	3530	.5409	650
0	23395	1.0706	25000	2.9576	69100	.5000	11700
1R	1200	1.2309	1477	2.9409	3530	.4951	595
2	1650	1.4632	2405	2.7889	4600	.3711	612
3	1650	1.5220	2500	2.5004	4125	.2892	477
4	6900	1.4376	9900	2.1126	14560	.2150	1483
6	9105	.9666	8700	1.2270	11170	.1006	916
8	8730	.4153	3600	.4804	4200	.0330	276
10R	8730	.0668	580	.0720	630	.00442	35
			+ 55827		157438 kg		50207
			− 43195	call this	157400	call this	50200 kg
		call this	12632				
			+ 12600 kgm				

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CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

Temperature Stresses.

Crown Section $H_o = \dfrac{E.w.t.l.\int \frac{do}{I}}{Z[\int \frac{do}{I}\int \frac{y^2 do}{I} - (\int \frac{y\,do}{I})^2]}$

where $E = 1{,}400{,}000.000$ kg/m²
Coef. of Exp. $w = 0.000012$ for 1°C
Variation of temperature ± 15°C
l = span length 18.00 meters.
E.w.t = 252000 kg/m²

For fall of 15° C in temperature.

$H_o = -\dfrac{252000 \times 18.00 \times 1119.53}{321034} = -15800$ kg for 7.5 meter wide $-15800 \times 7.5 = -118500$ kg

$M_o = -\dfrac{H_o \int \frac{y\,do}{I}}{\int \frac{do}{I}} = 118.500 \times \dfrac{327.00}{1119.53} = 34600$ kgm

Temperature stress at various panel points.

Panel Point					Moment			Normal thrust
2	M=	34600	−	118,500 × .030	= + 31,040 kgm	− 118,500	× .999	= − 118,300 kg.
4	M=	34600	−	" × .166	= + 14,930	"	× .993	= − 117,600
6	M=	34600	−	" × .418	= − 14,950	"	× .984	= − 116,600
8	M=	34600	−	" × .786	= − 58,600	"	× .969	= − 114,800
10	M=	34600	−	" × 1.276	= − 116,600	"	× .949	= − 112,500
11 (springing)	M=	34600	−	" × 1.650	= − 160,900	"	× .935	= − 110,800

Average stress in Arch Ring.

Amount of average stress in arch ring for entire span is only approximate. Average stresses are found by the diagram prepared by Mr. J. Masuda, the same diagrams are published in Concrete Engineer's Handbook by Hool and Johnson's (on page 685)

Concrete area at crown .3500
Equivalent steel area .0550
for 7.5 meter wide -- .4050 × 7.50 = 3.04 Sq. meters

Thickness ratio = $\dfrac{\text{depth at springing}}{\text{depth at crown}} = \dfrac{.700}{.350} = 2.00$, Rise ratio = $\dfrac{1.65}{18.00} = 0.092$

Average stresses.

		Hor. thrust at Crown			Coef. for average stress		Average stresses.
Dead load stresses		358,660	÷ 3.04 =	118,000	@	.89	= 105,000 kg/m²
Live load stresses							
Crown	+ moment	94,920	÷ 3.04 =	31,200	@	.90	= 28,100
panelpt. 2	+ moment	88,950	÷ 3.04 =	29,250	@	.90	= 26,300
panel pt. 4	+ moment	70,220 70,000	÷ 3.04 =	23,100	@	.90	= 20,800
panel pt. 6	+ moment	38,420	÷ 3.04 =	12,640	@	.90	= 11,380
Panel pt. 8	+ moment	66,960	÷ 3.04 =	22,040	@	.90	= 19800
	− moment	97,500	÷ 3.04 =	32,050	@	.90	= 28,850
Panel pt. 10	+ moment	103,180	÷ 3.04 =	33,950	@	.88	= 29900
	− moment	56,960	÷ 3.04 =	18,740	@	.90	= 16,850
		56,840					
Springing (11)	+ moment	119,640	÷ 3.04 =	39,350	@	.88	= 34,600
	− moment	42,040	÷ 3.04 =	13,820	@	.90	= 12,430
For Temperature	15°c fall	−118,500	÷ 3.04 =	38,950	@	.86	= − 33,500

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CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Fibre stresses in Arch Ring.

Crown Section. Positive Moment.

	Thrust	moment	Average stress
Dead Load	+ 358,660	− 2,400	+ 105,000
Live Load	+ 94,920	+ 17,060	+ 28,100
Rib Shortening	− 55,200	+ 16,120	− 15,630
	+ 398,380	+ 30,780	+ 117,470

$117470 \div 252000 = .466$, $\frac{e}{h} = .077$, $\frac{z}{h} = \frac{.077}{.35} = .220$

$d/h = 4.5/35 = .129$, $p_0 = zp = \frac{39.26}{3500} = .0112$, $h = 35$, $b = 750^{cm}$

$k = .903$, $L = .116$

$f_c = \frac{30780 \times 100}{.116 \times 750 \times 35^2} = 28.9\ kg/cm^2 < 45.0$ ok

$f_s = 15 \times 28.9 \left(\frac{30.5}{.903 \times 35} - 1\right) =$ negligible

Dead Load	+ 358,660	− 2,400	+ 105,000
Live Load	+ 94,920	+ 17,060	+ 28,100
Temperature	− 118,500	+ 34,600	− 33,500
Rib Shortening	− 41,300	+ 12,070	− 11,690
	+ 293,780	+ 61,330	+ 87,910

$87910 \div 252000 = .349$, $\frac{e}{h} = .209$, $\frac{z}{h} = \frac{.209}{.35} = .597$

$d/h = .129$, $p_0 = .0112$, $h = 35$, $b = 750^{cm}$

$k = .461$, $L = .131$

$f_c = \frac{61330 \times 100}{.131 \times 750 \times 35^2} = 51.0\ kg/cm^2 < 45 \times 1.25 = 56.3$ ok

$f_s = 15 \times 51.0 \left(\frac{30.5}{.461 \times 35} - 1\right) = 680$ " ok.

Panel Point Z, Positive moment.

	Thrust	moment	Average stress
Dead Load	+ 357,500	− 1370	+ 105,000
Live Load	+ 89,370	+ 19,380	+ 26,300
Rib shortening	− 54,500	+ 14,270	− 15,410
	+ 392,370	+ 32,280	+ 115,890

$115890 \div 252000 = .460$, $\frac{e}{h} = .081$, $h = .356$, $\frac{z}{h} = .228$

$d/h = 4.5/35.6 = .127$, $p_0 = \frac{39.26}{3650} = .0110$, $b = 750$

$k = .882$, $L = .117$

$f_c = \frac{32280 \times 100}{.117 \times 750 \times 35.6^2} = 29.1\ kg/cm^2$ ok.

$f_s = 15 \times 29.1 \left(\frac{31.1}{.882 \times 35.6} - 1\right) =$ Negligible ok.

Dead Load	+ 357,500	− 1,370	+ 105,000
Live Load	+ 89,370	+ 19,380	+ 26,300
Temperature	− 118,300	+ 31,040	− 33,500
Rib shortening	− 40,500	+ 10,630	− 11,480
	+ 288,070	+ 59,680	+ 86,320

$86320 \div 252000 = .343$, $\frac{e}{h} = .206$, $\frac{z}{h} = \frac{.206}{.356} = .578$

$k = .466$, $L = .130$

$f_c = \frac{59680 \times 100}{.130 \times 750 \times 35.6^2} = 48.3\ kg/cm^2$ ok

$f_s = 15 \times 48.3 \left(\frac{31.1}{.466 \times 35.6} - 1\right) = 636$ " ok

Panel Point 4 Positive moment.

	Thrust	moment	Average stress
Dead Load	+ 360,720	+ 1140	+ 105,000
Live Load	+ 71,910	+ 24,625	+ 20,800
Temperature	− 117,600	+ 14,930	− 33,500
Rib Shortening	− 38,000	+ 4,820	− 10,830
	+ 277,030	+ 45,515	+ 81,470

$81470 \div 252000 = .323$, $\frac{e}{h} = .164$, $\frac{z}{h} = \frac{.164}{.385} = .426$

$d/h = 4.5/38.5 = .117$, $p_0 = \frac{39.26}{3850} = .010$, $h = 38.5$, $b = 750$

$k = .554$, $L = .125$

$f_c = \frac{45515 \times 100}{.125 \times 750 \times 38.5^2} = 37.8\ kg/cm^2$ ok

$f_s = 15 \times 37.8 \left(\frac{34}{.554 \times 38.5} - 1\right) = 293$ " ok.

Panel Point 6 Positive moment

	Thrust	moment	Average stress
Dead Load	+ 364,710	+ 4520	+ 105,000
Live Load	+ 42,280	+ 22,700	+ 11,380
Temperature	+ 116,600	+ 14,950	+ 33,500
Rib shortening	− 61,200	− 7850	− 17,590
	+ 462,390	+ 34,320	+ 132,790

$132790 \div 252000 = .525$, $\frac{e}{h} = .073$ $\frac{z}{h} = \frac{.073}{.434} = .168$

$h = .434$, $b = 750$

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Design of Jonan Bashi for Kyoto Prefecture.

Panel Point 8 Positive moment.

	Thrust	moment	Average stress.
Dead Load	+ 370,650 ✓	+ 5,110 ✓	+ 105,000 ✓
Live Load	+ 72,520 ✓	+ 17,740 ✓	+ 19,800 ✓
Temperature	+ 114,800 ✓	+ 58,600 ✓	+ 33,500 ✓
Rib shortening	− 63,600 ✓	− 32,500 ✓	− 18,600 ✓
	+ 494,370 ✓	+ 48,950 ✓	+ 139,700 ✓

139700÷252000 = .554, Σcc = .099, z/h = .099/51.5 = .194
d'/h = 4.5/51.5 = .0875, p = 39.26/5150 = .0076, h = 51.5 cm, b = 750 cm

Panel Point 8 Negative moment.

	Thrust	moment	Average stress
Dead Load	+ 370,650 ✓	+ 5,110 ✓	+ 105,000 ✓
Live Load	+ 102,380 ✓	− 13,870 ✓	+ 28,850 ✓
Temperature	− 114,800 ✓	− 58,600 ✓	− 33,500 ✓
Rib shortening	− 40,400 ✓	− 20,640 ✓	− 11,780 ✓
	+ 317,830 ✓	− 88,000 ✓	+ 88,570 ✓

88570÷252000 = .352, Σcc = .277, z/h = .277/51.5 = .538
k = .470, L = .1120
$f_c = \frac{88000 \times 100}{1122 \times 750 \times 51.5^2} = 36.3$ kg/cm² ok.
$f_s = 15 \times 36.3 \left(\frac{47}{.47 \times 51.5} - 1\right) = 513$ " ok.

Panel Point 10. Positive moment.

	Thrust	moment	Average stress
Dead Load	+ 382,100 ✓	− 2,800 ✓	+ 105,000 ✓
Live Load	+ 103,530 ✓	+ 34,810 ✓	+ 29,900 ✓
Temperature	+ 112,500 ✓	+ 116,600 ✓	+ 33,500 ✓
Rib shortening	− 66,400 ✓	− 68,800 ✓	− 19,750 ✓
	+ 531,730 ✓	+ 79,810 ✓	+ 148,650 ✓

148650÷252000 = .590, Σcc = 1.50, z/h = .150/.623 = .241
d'/h = 4.5/62.3 = .0723, $p_0 = \frac{78.52}{6230} = .0126$, h = 62.3, b = 750 cm
k = .897, L = .129
$f_c = \frac{79810 \times 100}{129 \times 750 \times 62.3^2} = 21.2$ kg/cm² ok.
$f_s = 15 \times 21.3 \left(\frac{57.8}{.899 \times 62.3} - 1\right) = 11$ " ok.

	Thrust	moment	Average stress
Dead Load	+ 382,100 ✓	− 2,800 ✓	+ 105,000 ✓
Live Load	+ 103,530 ✓	+ 34,810 ✓	+ 29,900 ✓
Rib shortening	− 53,200 ✓	− 55,200 ✓	− 15,840 ✓
	+ 432,430 ✓	− 23,190 ✓	+ 119,060 ✓

119060÷252000 = .473, Σcc = .054, z/h = .054/.623 = .087

Panel Point 10. Negative moment

	Thrust	moment	Average stress.
Dead Load	+ 382,100 ✓	− 2,800 ✓	+ 105,000 ✓
Live Load	+ 66,880 ✓	− 35,680	+ 16,850 ✓
Temperature	− 112,500 ✓	− 116,600 ✓	− 33,500 ✓
Rib shortening	− 34,780 ✓	− 36,050 ✓	− 10,370 ✓
	+ 301,700	− 191,130	+ 77,980 ✓

77980÷252000 = .309, Σcc = .631, z/h = .631/.623 = 1.013
k = .403, L = .159
$f_c = \frac{191130 \times 100}{159 \times 750 \times 62.3^2} = 41.2$ kg/cm² ok.
$f_s = 15 \times 41.2 \left(\frac{57.8}{.403 \times 62.3} - 1\right) = 80.5$ kg/cm² ok.

Dead Load	+ 382,100 ✓	− 2,800 ✓	+ 105,000 ✓
Live Load	+ 66,880 ✓	− 35,680 ✓	+ 16,850 ✓
Ribs shortening	− 48,000 ✓	− 49,700 ✓	− 14,300 ✓
	+ 400,980 ✓	− 88,180	+ 107,550 ✓

107550 ÷ 252000 = .427, Σcc = .220, z/h = .220/.623 = .353

Springing (Pt. 11) Positive moment.

	Thrust	moment	Average stress.
Dead Load	+ 392,400 ✓	− 18,720 ✓	+ 105,000 ✓
Live Load	+ 117,860 ✓	+ 59,750 ✓	+ 34,600 ✓
Temperature	+ 110,800 ✓	+ 160,900 ✓	+ 33,500 ✓
Rib shortening	− 67,200 ✓	− 97,500 ✓	− 20,310 ✓
	+ 553,860 ✓	+ 104,430 ✓	+ 152,790 ✓

152790÷252000 = .606, Σcc = .189, z/h = .189/.70 = .270
d'/h = 4.5/70 = .0643, $p_0 = \frac{78.52}{7000} = .0112$, h = 70., b = 750 cm
k = .849, L = .130
$f_c = \frac{104430 \times 100}{130 \times 750 \times 70^2} = 21.9$ kg/cm² ok
$f_s = 15 \times 21.9 \left(\frac{65.5}{.849 \times 70} - 1\right) = 33$ " ok.

Dead Load	+ 392,400 ✓	− 18,720 ✓	+ 105,000 ✓
Live Load	+ 117,860 ✓	+ 59,750 ✓	+ 34,600 ✓
Rib shortening	− 54,200 ✓	− 78,600 ✓	− 16,400 ✓
	+ 456,060 ✓	− 37,570 ✓	+ 123,200 ✓

123200÷252000 = .489, Σcc = .082, z/h = .082/.70 = .117

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Springing (Point 11) Negative moment.

	Thrust	moment	average stress.
Dead Load	+ 392,400 ✓	− 18,720 ✓	+ 105,000 ✓
Live Load	+ 53,760 ✓	− 61,790 ✓	+ 12,430 ✓
Temperature	− 110,800 ✓	− 160,900 ✓	− 33,500 ✓
Rib shortening	− 32,600 ✓	− 47,280 ✓	− 9,860 ✓
	+ 302,760 ✓	− 288,690 ✓	+ 74,070 ✓

74070 ÷ 252000 ✓ = .294 ✓, Σcc = .954 ✓, $\Sigma/h = \frac{.954}{.70}$ ✓ = 1.362 ✓

d'/h = .0643 ✓, $p_0 = \frac{78.52}{7000}$ ✓ = .0112 ✓, h = 70.0 ✓, b = 750 cm ✓

k = .361 ✓, L = .158 ✓

$f_c = \frac{288690 \times 100}{.158 \times 750 \times 70^2}$ ✓ = 49.7 ✓ kg/cm² ok. < 56.3 ✓

$f_s = 49.7 \times 15 \left(\frac{6.55}{.361 \times 70} - 1\right)$ ✓ = 1185 ✓ kg/cm² ok. < 1500 ✓

Dead Load	+ 392,400 ✓	− 18,720 ✓	+ 105,000 ✓
Live Load	+ 53,760 ✓	− 61,790 ✓	+ 12,430 ✓
Rib shortening	− 45,600 ✓	− 66,200 ✓	− 13,780 ✓
	+ 400,560 ✓	− 146,710 ✓	+ 103,650 ✓

103650 ÷ 252000 ✓ = .4115 ✓, Σcc = .366 ✓, $\Sigma/h = \frac{.366}{.70}$ ✓ = .523 ✓

k = .536 ✓, L = .145 ✓

$f_c = \frac{146710 \times 100}{.145 \times 750 \times 70^2}$ ✓ = 27.6 ✓ kg/cm² ok < 45

$f_s = 15 \times 27.6 \left(\frac{6.55}{.536 \times 70} - 1\right)$ ✓ = 309 ✓ " ok < 1200

If steel area be changed as follows at springing.

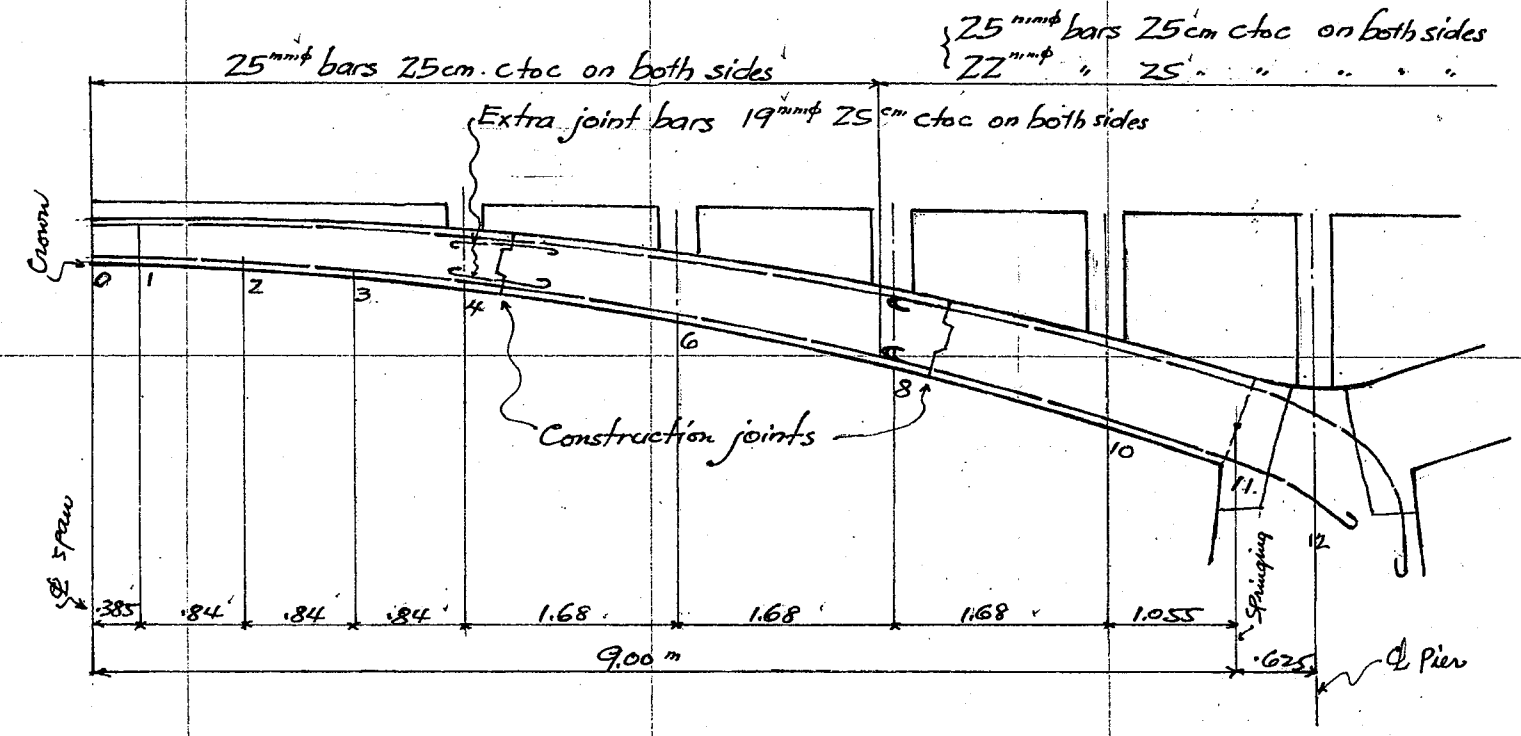
4 − 25 mmϕ bars on both sides = 39.26 ✓
4 − 22 " " " " = 30.41 ✓
69.67 cm²/m strip.

steel ratio $p_0 = \frac{69.67}{70 \times 100}$ ✓ = 0.010 ✓

k = .347 ✓, L = .149 ✓

$f_c = \frac{288690 \times 100}{.149 \times 750 \times 70^2}$ ✓ = 52.7 ✓ kg/cm² < 56.3 ok

$f_s = 15 \times 52.7 \left(\frac{65.5}{.347 \times 70} - 1\right)$ ✓ = 1342 ✓ kg/cm² < 1500 ✓ ok.



Reinforcement Diagram of Arch Ring.

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CALCULATIONS FOR

Design of Joran Bashi for Kyoto Prefecture.

Design of Pier.

weight and center of gravity of super structure. — lever arm from crown of Roadway.

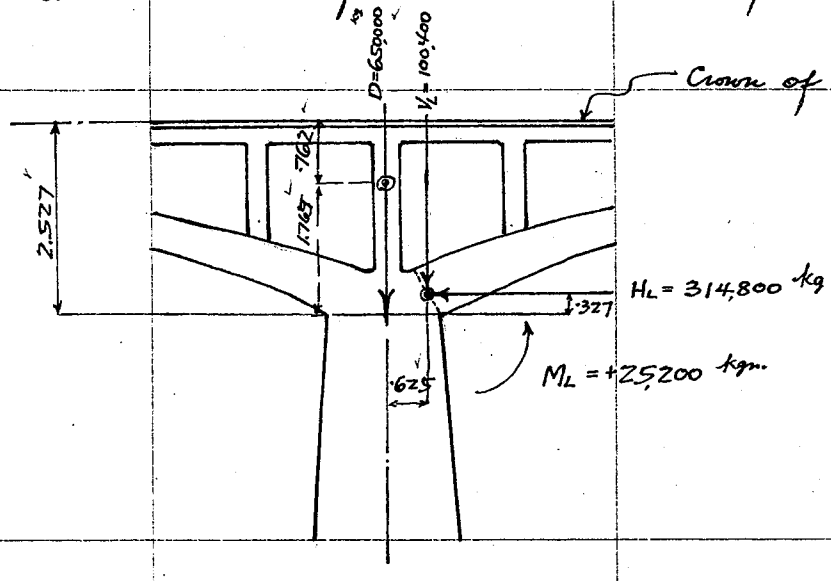
Roadway floor slab and pavement say	14.0'c	570'×19.25	=	153,500	×	−.22	= −	33,750
Sidewalk " " "	4.0'c	370'×19.25	=	28,500	×	−.09	= −	2,565
Curb stone	2'c	98' "	=	3,770	×	−.13	= −	490
beam under curb	2'c	151' "	=	5,810	×	−.32	= −	1,860
handrails	2'c	385' "	=	14,810	×	+.40	= +	5,930
coping	2'c	165' "	=	6,350	×	−.10	= −	635
telephone pipe lines &c say	2'c	171' "	=	6,580	×	−.33	= −	2,170
transverse spandrel wall + cross beam	2'c	39300	=	78,600	×	−.80	= −	62,900
longitudinal spandrel wall say	4'c	5310	=	21,240	×	−.60	= −	12,740
crown filling say $\frac{1}{3}$	4'c	8800	=	11,740	×	−.40	= −	4,700
Arch ring panel point 0 crown	4 c	1,215	=	4,860	×	−.55	= −	2,675
" " 1	4 c	3,870	=	15,480	×	−.55	= −	8520
" " 2	4 c	5,395	=	21,580	×	−.58	= −	12,460
" " 3	4 c	5570	=	22,280	×	−.63	= −	14,040
" " 4	4 c	5870	=	23,480	×	−.72	= −	16,900
" " 5	4 c	6,190	=	24,760	×	−.83	= −	20,550
" " 6	4 c	6680	=	26,720	×	−.97	= −	25,480
" " 7	4 c	7,290	=	29,160	×	−1.14	= −	33,250
" " 8	4 c	8040	=	32,160	×	−1.34	= −	43,100
" " 9	4 c	8880	=	35,520	×	−1.57	= −	55,800
" " 10	4 c	11,210	=	44,840	×	−1.83	= −	82,000
" " 11 springing	4 c	7040	=	28,160	×	−2.10	= −	59,100
extra weight for light pole and columns.	2 c	5,050	=	10,100	×	−.55	= −	5,550
				650,000 kg		−0.762'	−	495,305

Superimposed loads on Pier

Dead load 650,000 kg on ℄ pier. = D

Live load
max. horizontal thrust = 157400×2 = 314,800 kg = H_L
Corresponding vert shear = 50200×2 = 100,400 " = V_L
" moment at spr. = 12,600×2 = 25,200 kgm = M_L

moments and thrusts for Dead load and temperature cancells on both sides.



Seismic force for k=0.250

D' = 650000×.25 = 162,500 kg

tan 16° = 0.1675
sec 16° = 1.0403
cos 16° = 0.96126

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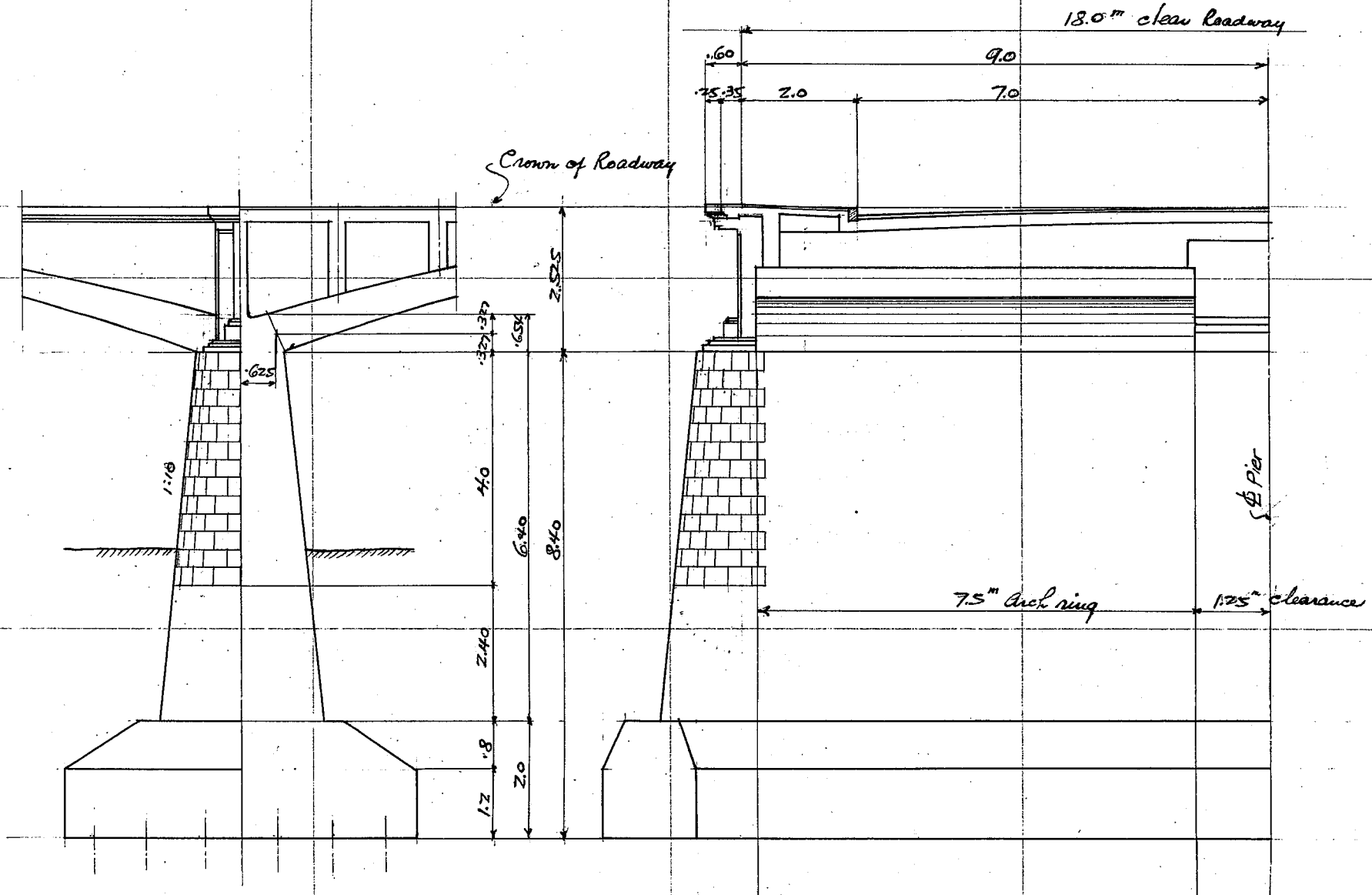
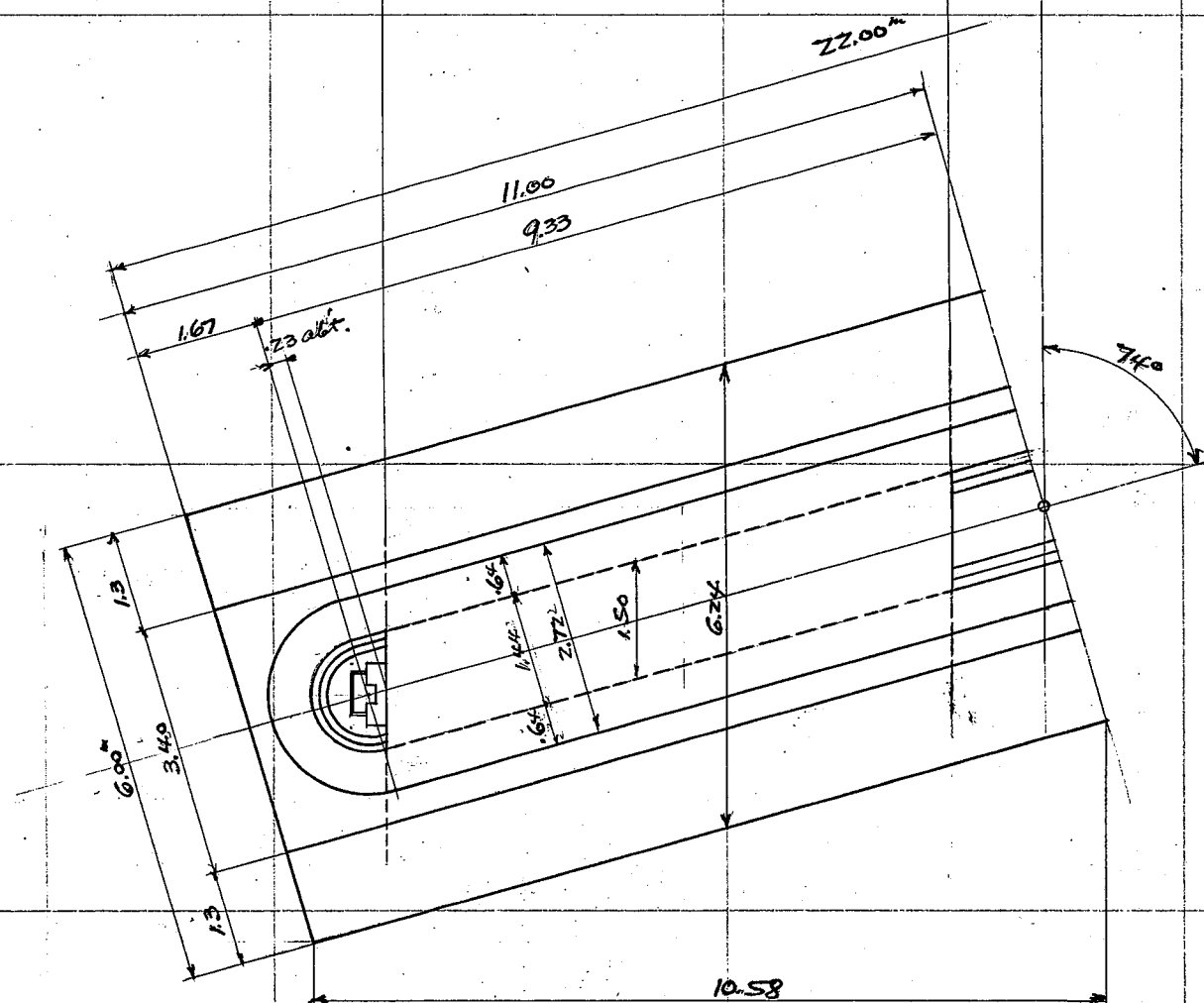
CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

General dimensions of Pier assumed as shown on sketch below.



General sketch of Pier.
Scale 1:100.



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CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

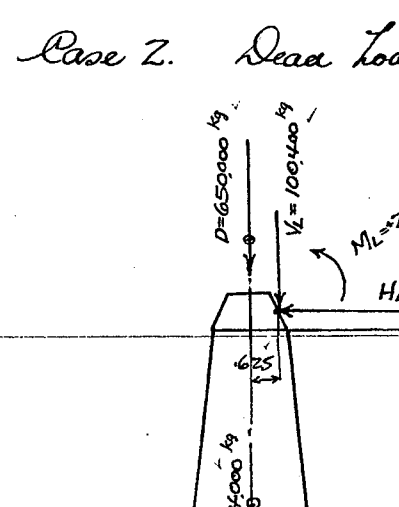
Approximate weight and center of gravity of Pier.

Shaft.-

Core between arch rings	1.20 × .654 ×	18.20 =	14.28 × @	2,400 =	34,300 ×	6.43 =	220,500
Shaft between tangent lines	2.08 × 6.40 ×	18.66 =	248.42 × @	2,400 =	596,000 ×	2.87 =	1,710,000
" circular ends	2.08 ϕ ×	6.40 =	21.70 × @	2,400 =	52,100 ×	2.60 =	135,500
Granite ornaments say					1,600 ×	6.55 =	10,500
				S =	684,000 kg	3.03 m	2,076,500

Base.

4.70 × .80 ×	22.0 =	82.70 × @	2,400 =	198,500 ×	1.56 =	309,500
6.00 − 1.20 ×	22.0 =	158.40 × @	2,400 =	380,500 ×	0.60 =	228,300
		241.10 m³	B =	579,000 kg	0.94 m	537,800



Seismic forces for k = 0.250.

Due to shaft	684,000 × 0.250 =	171,000 kg = S'
Due to base	579,000 × 0.250 =	145,000 " = B'

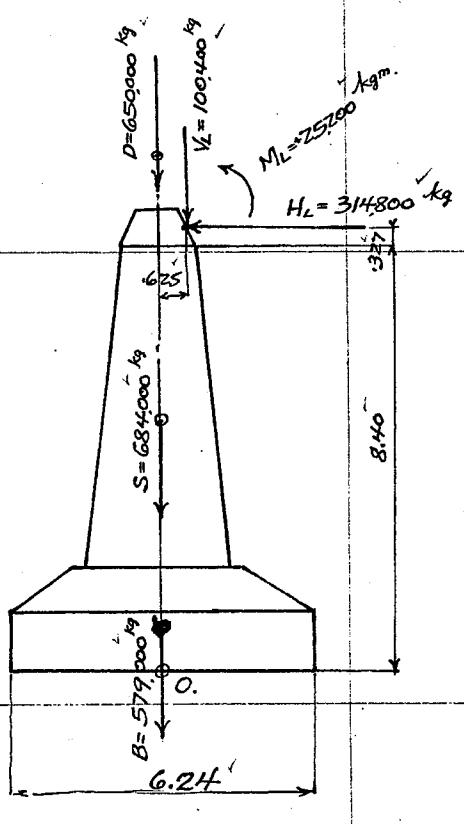
Stability of Pier.

Case 1. Dead load only.

Superimposed Dead Load on pier	=	650,000
weight of Pier shaft	=	684,000
" " Pier base	=	579,000
		1,913,000 kg on one pier.

Bearing pressure on subsoil = $\frac{1913000}{6.0 \times 22.0}$ = 14,500 kg/m² or (1.325 tons/□')

Case 2. Dead load plus live load.



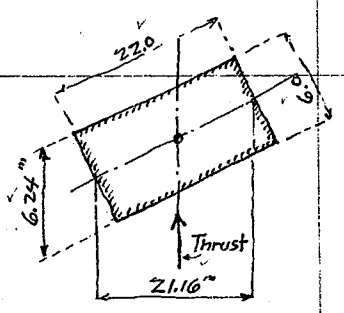
Taking moment about center of base O.

Load	Hor. forces	Vert. forces	Lever arms	Moments
D		650,000 ×	0 =	0
V_L		100,400 ×	−.625 =	−62,800
H_L	314,800	×	8.727 =	+2,747,000
S		684,000 ×	0 =	0
B		579,000 ×	0 =	0
M_L			=	+ 25,200
	314,800 kg	2,013,400 kg	+1.345 m	+ 2,709,400 kg.m.

Resultant force outside of middle third, neglecting tension.

Pressure area = $\left(\frac{6.24}{2} - 1.345\right) 3 = 5.325 \times 21.16 = 112.7$ sq. meters.

max. toe pressure = $\frac{2,013,400 \times 2}{112.7}$ = 35,700 kg/m² or (3.26 tons/□')



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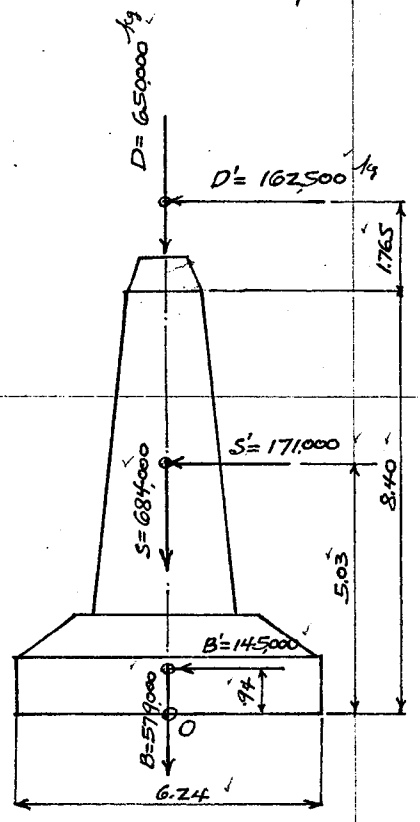
CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

Case 3. Stability during Earthquake. k assumed = 0.250



Taking moment about center of base O.

Load	Hor. forces	Vert forces	Lev. arms	Moments.
D		650,000 ×	0 =	0
S		684,000 ×	0 =	0
B		579,000 ×	0 =	0
D'	162,500	×	10.165 =	1,651,000
S'	171,000	×	5.03 =	860,000
B'	145,000	×	0.94 =	136,300
	478,500 kg	1913,000 kg	1.384 m	2,647,300 kgm.

Resultant force outside of middle third neglecting tension.
Pressure area = (3.12 − 1.384) × 3 = 5.208 × 21.16 = 110.3 sq. meter

max. toe pressure = $\frac{1913000 \times 2}{110.3}$ = 34,650 kg/m² or (3.17 tons/o').

Case 4. Stability during Execution, Arch ring on one side of the pier only.

Hor. thrust and vert. load due to arch ring.

Point	Load.	Ho unit	Thrust Ho.
0 cr.	1,215	2.9576	3,590
1	3,870	2.9409	11,420
2	5,395	2.7889	15,050
3	5,570	2.5004	13,930
4	5,870	2.1126	12,400
5	6,190	1.6723	10,350
6	6,680	1.2270	8,190
7	7,290	.8189	5,970
8	8,040	.4804	3,860
9	8,880	.2323	2060
10	11,210	.0720	810
11 Sp.	7040	.0000	0
	77,250		87,630
	77,250		87,630
	R = 154,500 kg		175,260 kg.
			175,260
		Ho	350,520 kg

Taking moment about O.

Loads	Hor. forces	Vert forces	Lev. arms	Moments.
R		154,500 ×	−.625 =	− 96,500
S		684,000 ×	0 =	0
B		579,000 ×	0 =	0
Hr.	350,520	×	+ 8.727 =	+ 3056500
	350,520 kg	1417,500 kg	+ 2.087 m	+ 2960,000

Resultant force outside of middle third, neglecting tension.
Pressure area = (3.12 − 2.087) × 3 = 3.10 × 21.16 = 65.60 m²
max. bearing pressure on sub soil
= $\frac{1417500 \times 2}{65.60}$ = 43200 kg/m² or (3.95 tons/o')

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MADE BY K. Inaba DATE 5-11-8 FILE NO
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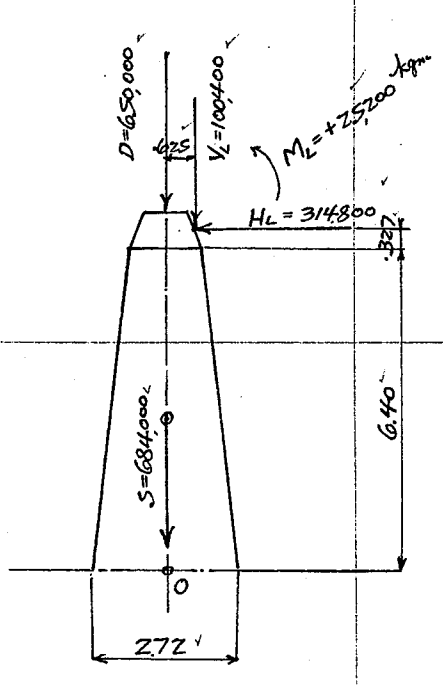
CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.
Design of Pier Shaft.
Case 1. Dead load plus live load.



Taking moment about center of bottom section O.

Load	Hor. Forces	Vert. Forces		Lev. arms		Moments.
D		650,000	×	0	=	0
V_L		100,400	×	−.625	=	− 62,800
S		684,000	×	0	=	0
H_L	314,800		×	+6.727	=	+2,118,000
M_L						+ 25,200
	314,800 kg	1,434,400 kg		1.452 m		+2,080,400
for 1 m strip (÷18.66)	16,870 kg	76,900 kg		1.452 m		111,500 kgm

Try reinforcements 22 mm⌀ bars 25 cm c to c = 15.20 cm²

$p_0 = \frac{15.20 \times 2}{100 \times 272} = 0.00112$, $z/h = 1.452/2.72 = .534$, $d'/h = 5/272 = .0184$

$k = .270$ $L = .071$

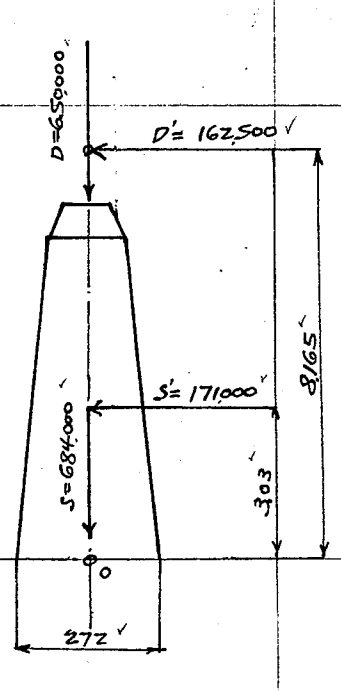
$f_c = \frac{111{,}500 \times 100}{.071 \times 100 \times 272^2} = 21.2$ kg/cm² ok.

$f_s = 15 \times 21.2 \left(\frac{26.7}{.270 \times 272} - 1\right) = 838$ " ok.

unit shear $= \frac{16870}{100 \times \frac{7}{8} \times 267} = 0.72$ " ok

unit bond $= \frac{16870}{6.91 \times 4 \times \frac{7}{8} \times 267} = 2.60$ " ok.

Case 2. Stresses during Earthquake. k assumed 0.250.



Load	Hor. Forces	Vert. forces		Lev. arm		moments.
D		650,000	×	0	=	0
S		684,000	×	0	=	0
D'	162,500		×	8.165	=	1,327,000
S'	171,000		×	3.03	=	518,000
	333,500	1,334,000		1.384 m		1,845,000
for 1 m strip ÷ 18.66	17,870 kg	71,500 kg		1.384 m		98,900 kgm

$z/h = 1.384/2.72 = .509$, $p_0 = .00112$, $d'/h = .0184$
$k = .295$, $L = .0725$

$f_c = \frac{98900 \times 100}{.0725 \times 100 \times 272^2} = 18.5$ kg/cm² ok

$f_s = 15 \times 18.5 \left(\frac{267}{.295 \times 272} - 1\right) = 64.5$ " ok.

unit shear $= \frac{17870}{100 \times \frac{7}{8} \times 267} = 0.77$ " ok

unit bond $= \frac{17870}{6.91 \times 4 \times \frac{7}{8} \times 267} = 2.77$ " ok.

Case 3. Stresses during Execution. Arch ring on one side of the pier only.

Load	Hor. forces	Vert forces	Lev. arm		Moments.
R		154,500	× −.625	=	− 96,500
S		684,000	× 0	=	0
H_R	350,520		× +6.727	=	+2,356,500
	350,520 kg	838,500 kg	2.696 m		2,260,000 kgm
for 1 m strip ÷ 18.66	18,780 kg	44,900 kg	2.696 m		121,100 kgm

Design of Jōnan Bashi for Kyoto Prefecture.

$z/h = 2.696/2.72 = .99$, $P_0 = .00112$, $d'/h = .0184$
$k = .18$, $L = .061$

$f_c = \frac{121,100 \times 100}{.061 \times 100 \times 272^2} = 26.8$ kg/cm² — ok

$f_s = 15 \times 26.8 \left(\frac{267}{.18 \times 272} - 1\right) = 1787$ " — ok

unit shear $= \frac{18780}{100 \times \frac{7}{8} \times 267} = 0.80$ " — ok

unit bond $= \frac{18780}{6.91 \times 4 \times \frac{7}{8} \times 267} = 2.91$ " — ok.

If the thickness of wall be measured along the direction of thrust or bending $h = 272 \times 1.04 = 283$ cm

$z/h = 2696/283 = .95$, $P_0 = \frac{30.40}{283 \times 100} = .00108$, $d'/h = 5/283 = .0177$

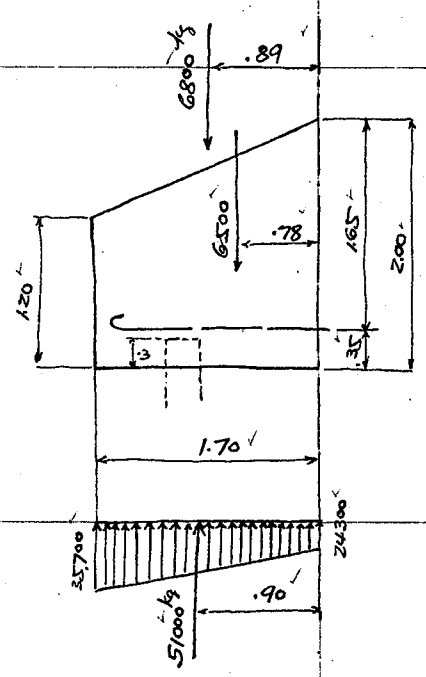
$k = .220$, $L = .064$

$f_c = \frac{121100 \times 100}{.064 \times 100 \times 283^2} = 23.6$ kg/cm² — ok

$f_s = 15 \times 23.6 \left(\frac{288}{.22 \times 283} - 1\right) = 1225$ kg/cm² — ok.

These stresses are only temporaly ones during execution of arching.
There will be no danger in increasing allowable stresses by 50 % or less.

Design of Cantilever footing of base.



Upward pressure due to D.L. + L.L.

		lev. arm	moment
$\frac{35700 + 24300}{2} = 30000 \times 1.70$	51000 kg	× .90 =	+ 45900 kgm

downward pressure due to wt. of footing

1.60×1.70 @ 2400	− 6500	× .78 =	− 5070
do due to earth on footing 2.50×1.70 @ 1600	− 6800	× .89 =	− 6050
Shear =	+ 37700 kg		+ 34780 kgm

Effective depth required $= \sqrt{\frac{34780 \times 100}{100 \times 7.18}} = 69.6$ cm
use 165 cm eff. depth.

Steel area required $= \frac{34780 \times 100}{1200 \times \frac{7}{8} \times 165} = 20.1$ cm² per meter strip.

Perimeter for bond

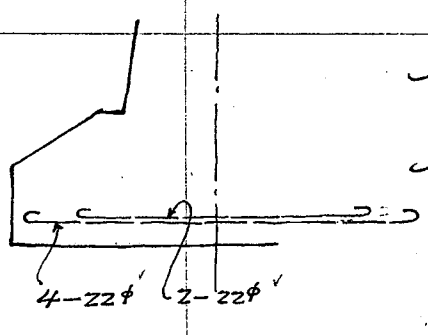
use 4 − 22 mmφ bars @ 38.0 = 15.20 — 4c 6.91 = 27.64
2 − 22 mmφ " @ " = 7.60 — 2c 6.91 = 13.82
22.80 cm — 41.46 cm

Steel ratio $p = \frac{22.80}{165 \times 100} = .0014$

$k = .181$, $j = .940$

$f_s = \frac{34780 \times 100}{22.80 \times .940 \times 165} = 984$ kg/cm² — ok.

$f_c = \frac{984 \times .181}{15(1 - .181)} = 14.5$ " — ok
.819



Unit shear $= \frac{37700}{100 \times .94 \times 165} = 2.43$ " — ok

Unit bond $= \frac{37700}{41.46 \times .94 \times 165} = 5.86$ " — ok.

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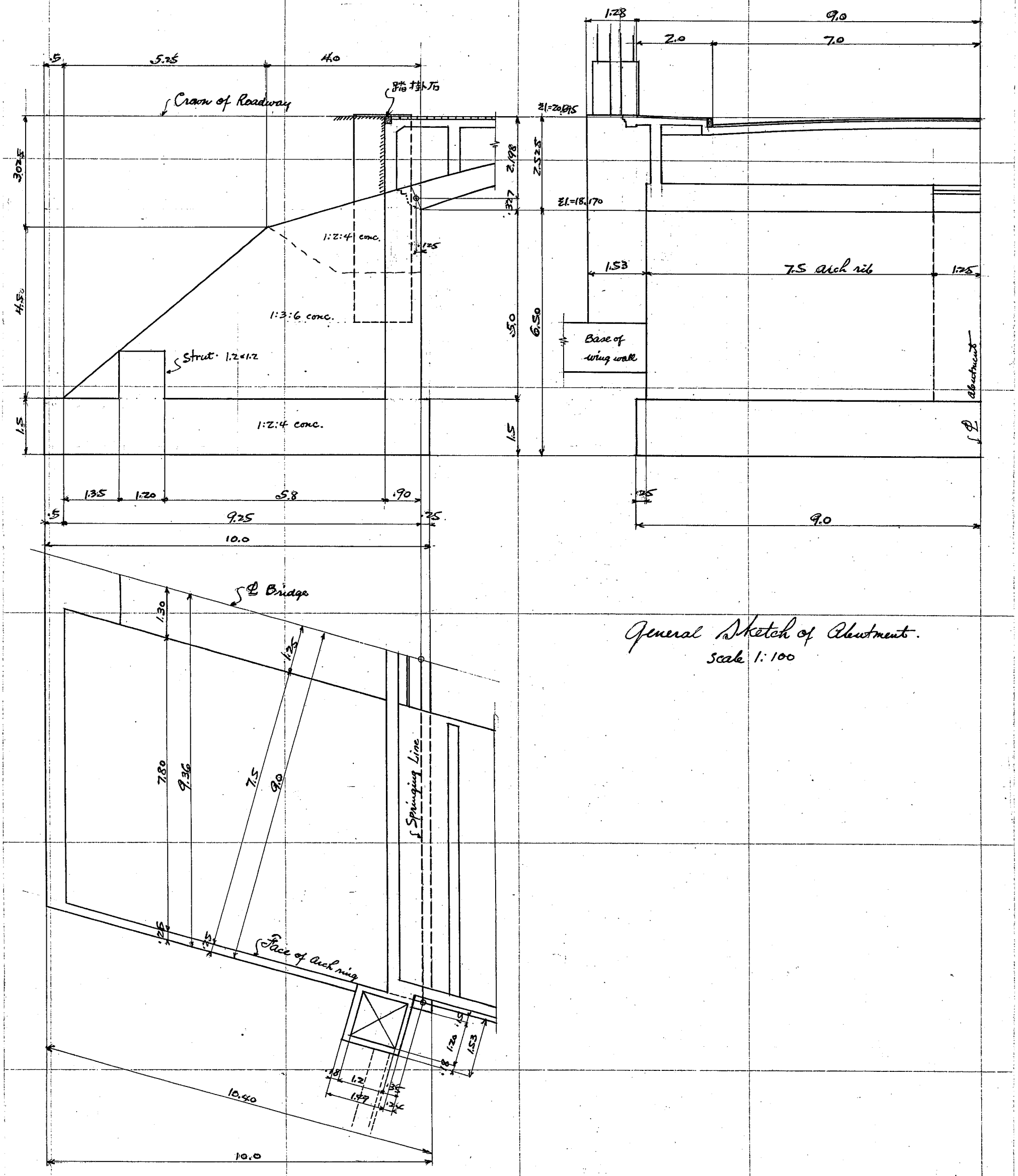
CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.
Design of Abutment.
General dimensions assumed as shown on sketch below.



General Sketch of Abutment.
Scale 1:100

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CALCULATIONS FOR

Design of Jōsan Bashi for Kyoto Prefecture.

Superimposed loads on Abutment.

			Seismic force	below crown of Roadway
Weight of Superstructure assumed	$\frac{1}{2}$ × 650000	= 325,000 kg = D	D' = 81,300 kg	.762 m
Horizontal thrust due to D.L.	2 @ 358660	= 717,300 " = H_D		2.525 m
Dead load moment at spr.		= − 18700 kgm = M_D		
Live load on abutment.		100,400 kg = V_L		
Live load hor. thrust		314800 " = H_L		2.525
Live load moment on springing		+ 25200 kgm = M_L		
Temperature. horizontal thrust	± 2 @ 118500	= ± 237,000 kg = H_T		2.525
" moment at springing	± 2 @ 160900	= ± 321,800 kgm = M_T		

Approximate weight and center of gravity of Abutment body.

			kg	hor. arm	moment	about toe. vert. arm	moment
Body	4.00 × 1.65/2 × 7.8 × 2	= 35.85 @ 2,400 =	86,000	1.58	136,000	6.38	548,000
"	6.625 × 4.5 × 7.8 × 2	= 465.00 @ 2,200 =	1,023,000	3.74	3,830,000	3.45	3,530,000
Front wall.	.90 × 5.3 × 1.3 × 2	= 12.40 @ 2,400 =	29,800	0.70	21,000	4.15	124,000
Strut	1.20 × 1.20 × 1.3 × 2	= 3.74 @ " =	9,000	7.55	68,000	2.10	19,000
base	10.00 × 1.50 × 9.36 × 2	= 280.70 @ 2,200 =	617,000	5.00	3,085,000	0.75	463,000
Column under pedestals.	1.49 × 1.53 × 5.5 × 2	= 25.07 @ 2,400 =	60,200	1.20	72,000	6.29	379,000
" base say	4.0 × 1.30 × 2.0 × 2	= 20.80 @ " =	50,000	1.60	80,000	7.89	145,000
light pedestals say	2 @ 12,500 kg	=	25,000	1.20	30,000	10.80	270,000
		843.56 m³ A =	1,900,000 kg	3.855 m	7,322,000	2.880 m	5478000

arm length along ℄ bridge (1.04) = 4.010 m

Weight and center of gravity of Earth filling on body.

No surcharge due to live load.

			hor. arm	moment	vert. arm	moment
on back of body	3.1 × 2.55 × 7.8 × 2	= 123.2	2.54	313	7.78	952
" "	5.275 × 5.25 × 7.8 × 2	= 432.0	7.24	3130	6.24	2698
on base. (at center)	8.35 × 2.60 × 7.525	= 163.4	5.33	871	5.26	860
" " (on both sides)	.52 × 7.525 × 10.00	= 39.1	5.00	196	5.26	206
" " (at heel)	.52 × 18.20 × 7.525	= 71.2	9.75	694	5.26	374
		828.9 m³	6.28 m	5204	6.15 m	5090
W_D =	828.9 @ 1600	= 1326000 kg	× 1.04			
arm along ℄ bridge =			6.53 m			

Earth pressure at normal state, average width of abutment assumed 20.0 m angle of repose ϕ assumed 30°.
Dead load only. $E_D = \frac{1600 \times 9.03^2}{6} = 21,750 \times 20.0 = 435000$ kg = arm 3.01 m above toe.

Earth pressure during Earthquake.

$E_E = .559 \times \frac{1600 \times 9.03^2}{2} = 36500 \times 20.0 = 730000$ kg arm 3.01 m on rear
$E_E' = .559 \times \frac{1600 \times 6.0^2}{2} = 16100 \times 20.0 = 322000$ " " 2.00 on front.

Surcharge due to live load. assumed $\frac{600}{1600} = .375$ m for total width uniformly.
9.025
9.400 total depth of earth.

			hor. arm	moment	vert. arm	moment
Surcharge earth fill	.375 × 8.85 × 18.72 @ 1600 =	100,000 kg	5.575	557,500	9.213	921,300
		1326000	6.280	8330000	6.15	8158700
W_{DL}		1426,000 kg	6.23	8887,500	6.36 m	9080,000
			× 1.04			
			6.48 m			

Earth pressure at normal state with live load surcharge.
$E_{DL} = \frac{1600 \times 9.4^2}{6} = 23500 \times 20 = 470000$ kg arm 3.12 m above toe.

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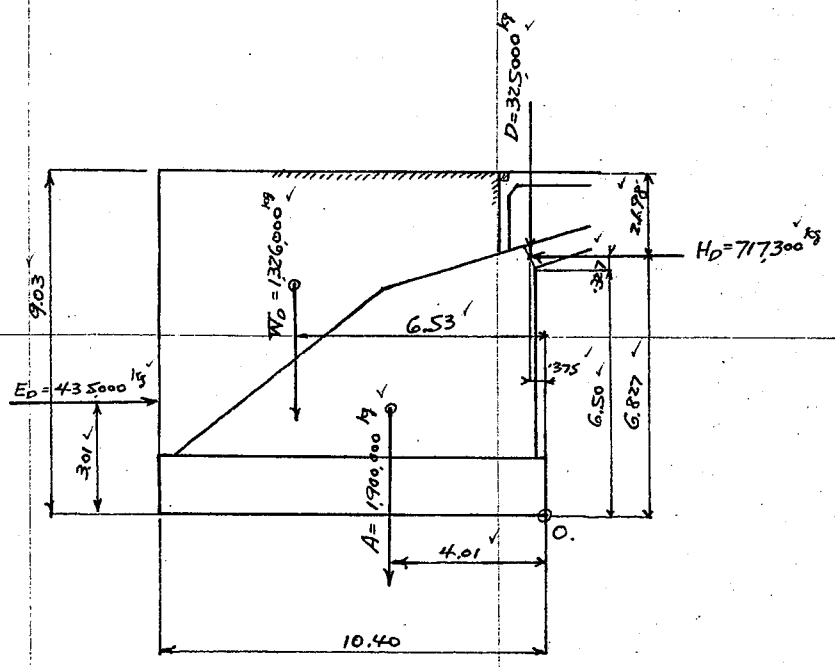
CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture

Stability of Abutment.

Case 1. Dead Load only.



Taking moment at toe O. (No arc springing neglected)

Load	Hor. forces	Vert. forces	Lev. arms	Moments.
D		325,000 ✓	+ 0.375 ✓ = +	122,000 ✓
A		1900,000 ✓	+ 4.010 ✓ = +	7620,000 ✓
W_D		1,326,000 ✓	+ 6.530 ✓ = +	8.658,000 ✓
H_D	717,300 ✓		+ 6.827 ✓ = +	4895,000 ✓
E_D	− 435,000 ✓		+ 3.010 ✓ = −	1,310,000 ✓
	282,300 kg ✓	3,551,000 kg ✓	+ 5.63 m ✓ +	19,985,000 ✓

Eccentricity ε = 5.63 − 5.20 = + 0.43 m ✓ < $\frac{10.40}{6}$ = 1.733 m ✓

Resultant forces within middle third

max. toe pressure = $\frac{3551000}{18.0 \times 10.40}\left(1 \pm \frac{6 \times .43}{10.4}\right)$ = 23,700 or (2.17 ton/sq.ft.)
or = 14,300

Case 2. Dead Load plus Temperature 15° rise.

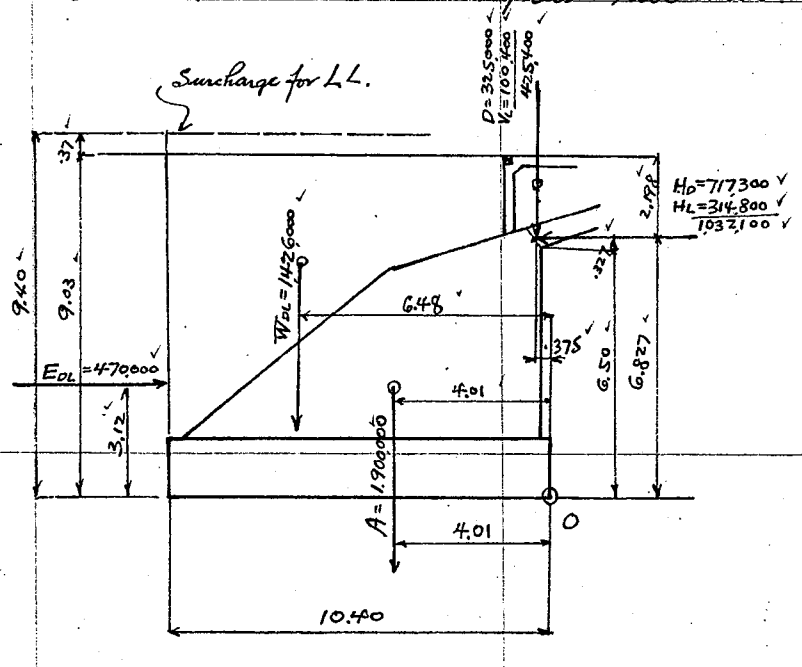
	Load	Hor. forces	Vert. forces	Lev. arms	moments.
Summary for case 1.		+ 282,300 ✓	3,551,000 ✓	+ 5.63 ✓ = +	19,985,000 ✓
Temperature thrust	H_T	+ 237,000 ✓		+ 6.827 ✓ = +	1,618,000 ✓
moment at springing	M_D+M_T			+	303100 ✓
		+ 519,300 kg ✓	3,551,000 kg ✓	+ 6.17 m ✓ +	21,906,100 ✓

Eccentricity ε = + 6.17 − 5.20 = + .97 m ✓

Resultant force within middle third.

max. toe pressure = $\frac{3551,000}{18.0 \times 10.40}\left(1 \pm \frac{6 \times .97}{10.4}\right)$ = 29,650 kg/sq.m = (2.71 ton/sq.ft.)
or 8360 ✓

Case 3. Dead Load plus Live Load



Taking moment about toe O.

Load		Hor. forces	Vert. forces	Lev. arms	moments.
D+V_L			425,400 ✓	× + 0.375 ✓ = +	159,500 ✓
A			1,900,000 ✓	× + 4.010 ✓ = +	7,620,000 ✓
W_{DL}			1426000 ✓	× + 6.480 ✓ = +	9240,000 ✓
H_D+H_L		1032,100 ✓		× + 6.827 ✓ = +	7,045,000 ✓
E_{DL}		− 470,000 ✓		× + 3.120 ✓ = −	1466000 ✓
M_D+M_L				= +	6500 ✓
		+ 562,100 kg ✓	3,751,400 kg ✓	+ 6.02 m ✓ +	22,605,000 ✓

Eccentricity ε = 6.02 − 5.2 = + 0.82 m ✓

Resultant force within middle third.

max. toe pressure = $\frac{3,751,400}{18.0 \times 10.40}\left(1 \pm \frac{6 \times .82}{10.4}\right)$ = 29,550 kg/sq.m or (2.70 ton/sq.ft.)
or 10,550 ✓

Case 4. Dead Load, Live Load plus Temperature +15° rise.

	Load	Hor. forces	Vert. forces	Lev. arms	Moments.
Summary for Case 3.		+ 562,100 ✓	3,751,400 ✓	+ 6.02 ✓ = +	22,605,000 ✓
Temperature thrust	H_T	+ 237,000 ✓		+ 6.827 ✓ = +	1,618,000 ✓
" moment	M_T			+	321,800 ✓
		+ 799,100 kg ✓	3,751,400 kg ✓	6.54 ✓ +	24,544,800 ✓

Eccentricity ε = 6.54 − 5.2 = 1.34 m ✓

Resultant force within middle third.

max. toe pressure = $\frac{3751,400}{18.0 \times 10.4}\left(1 \pm \frac{6 \times 1.34}{10.4}\right)$ = 35,550 kg/sq.m (= 325 ton/sq.ft.)
4,550 ✓

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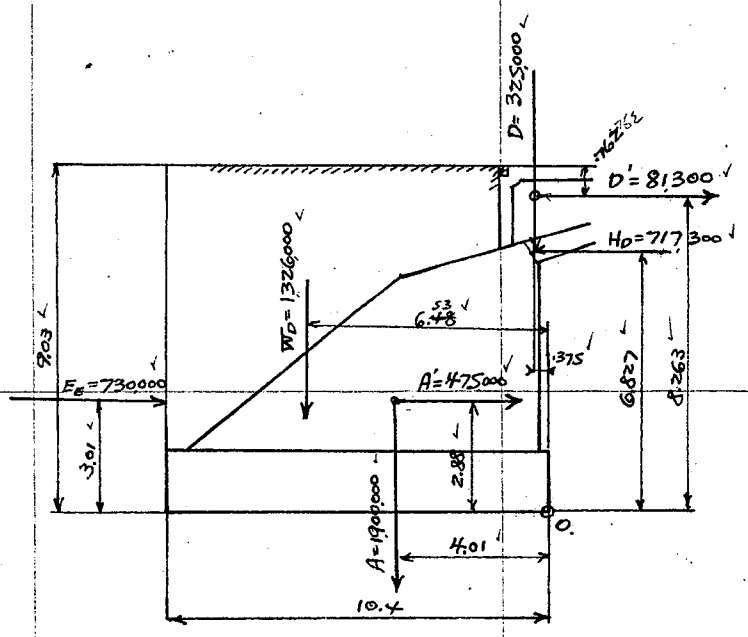
CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Case 5. Stability during Earthquake. Seismic forces toward river side.



Taking moment about toe O.

Loads	Hor. Forces	Vert. Forces	Lev. arm	Moment.
D		325.000 ✓ ↓	+ .375 ✓ =	+ 122.000 ✓
A		1900.000 ✓ ↓	+ 4.010 ✓ =	+ 7620.000 ✓
W_D		1326.000 ✓ ↓	+ 6.530 ✓ =	+ 8658.000 ✓
D'	− 81.300 ✓		+ 8.263 ✓ =	− 672.000 ✓
H_D	+ 717.300 ✓		+ 6.827 ✓ =	+ 4895.000 ✓
A'	− 475.000 ✓		+ 2.880 ✓ =	− 1368.000 ✓
E_E	− 730.000 ✓		+ 3.010 ✓ =	− 2198.000 ✓
M_D			=	− 18.700 ✓
	− 569.000 ✓ kg	3551.000 kg ✓	4.80 m ✓	+ 17038.300 ✓

Eccentricity $\varepsilon = -5.20^{\checkmark} + 4.80^{\checkmark} = -0.40^{\checkmark}$ m

Resultant force within middle third. .231 ✓

max. toe pressure $= \frac{3551.000^{\checkmark}}{18.0 \times 10.4}\left(1 \pm \frac{6 \times .40^{\checkmark}}{10.4}\right) = 23400^{\checkmark}$ kg/m² or (2.14 tons/o.)
or 14.600 ✓ "

Case 6. Seismic forces toward river side with temperature thrust −15 fall.

Loads	Hor. Forces	Vert. forces	Lev. arm	moments.
Summary for case 5.	− 569.000 ✓	3551.000 ✓ ↓	4.80 ✓ =	+ 17038.300 ✓
Temperature thrust H_T	− 237.000 ✓		6.827 ✓ =	− 1618.000 ✓
M_T				− 321.800 ✓
	− 806.000 kg ✓	3551.000 kg ✓	4.25 m ✓	+ 15098.500 ✓

Eccentricity $\varepsilon = 4.25 - 5.20^{\checkmark} = -0.95$ m ✓

max. toe pressure $= \frac{3551000^{\checkmark}}{18.0 \times 10.4}\left(1 \pm \frac{6 \times .95^{\checkmark}}{10.4}\right) = 29400^{\checkmark}$ kg/m² or (2.69 tons/o.)
or = 8590 ✓ "

Case 7. Seismic forces toward shore side.

Taking moment about toe O.

Loads	Hor. Forces	Vert. Fors	Lev. arms	moments.
D		325.000 ✓ ↓	.375 ✓ =	122.000 ✓
A		1900.000 ✓ ↓	4.010 ✓ =	7620.000 ✓
W_D		1326.000 ✓ ↓	6.530 ✓ =	8658.000 ✓
D'	+ 81.300 ✓		8.263 ✓ =	672.000 ✓
H_D	+ 717.300 ✓		6.827 ✓ =	4895.000 ✓
A'	+ 475.000 ✓		2.880 ✓ =	1368.000 ✓
E_E'	+ 322.000 ✓		2.000 ✓ =	644.000 ✓
M_D				− 18.700 ✓
	+ 1595.600 kg ✓	3551.000 ✓	+ 6.75 m ✓	+ 23960.300 ✓

$\frac{\Sigma H}{\Sigma V} = \frac{1595.600^{\checkmark}}{3551000^{\checkmark}} = 0.45^{\checkmark}$

Ecc. $= 6.75^{\checkmark} - 5.2^{\prime} = +1.55$ m ✓

Resultant force within middle third

max. toe pressure. $= \frac{3551.000^{\checkmark}}{18.0 \times 10.4}\left(1 \pm \frac{6 \times 1.55^{\checkmark}}{10.4}\right) = 36000^{\checkmark}$ kg/m² or (3.29 tons/o.)
= 1950 ✓ "

Case 8. Seismic forces toward shore side with temperature + 15° rise.

Loads	Hor. Forces	Vert forces	Lev. arm	moments.
Summary for case 6.	+ 1595.600 ✓	3551.000 ✓	+ 6.75 ✓	+ 23960.300 ✓
Temp. thrust H_T.	+ 237.000 ✓		6.827 ✓	+ 1618.000 ✓
" moment M_T				+ 321.800 ✓
	+ 1832.600 kg ✓	3551.000 ✓	+ 7.30 m ✓	+ 25900.100 ✓

$\frac{\Sigma H}{\Sigma V} = \frac{1832600}{3551000} = 0.516$

Ecc. $= 7.30 - 5.70^{\checkmark} = +2.10$ m ✓ Resultant force outside of middle third.

pressure area $= 3.10 \times 3 \times 18.0^{\checkmark} = 167.4$ m² ✓

max. toe pressure $= \frac{3551000 \times 2^{\checkmark}}{167.4} = 42450^{\checkmark}$ kg/m² or (3.88 tons/o.)

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MADE BY K. Inaba DATE 5-11-11 FILE NO
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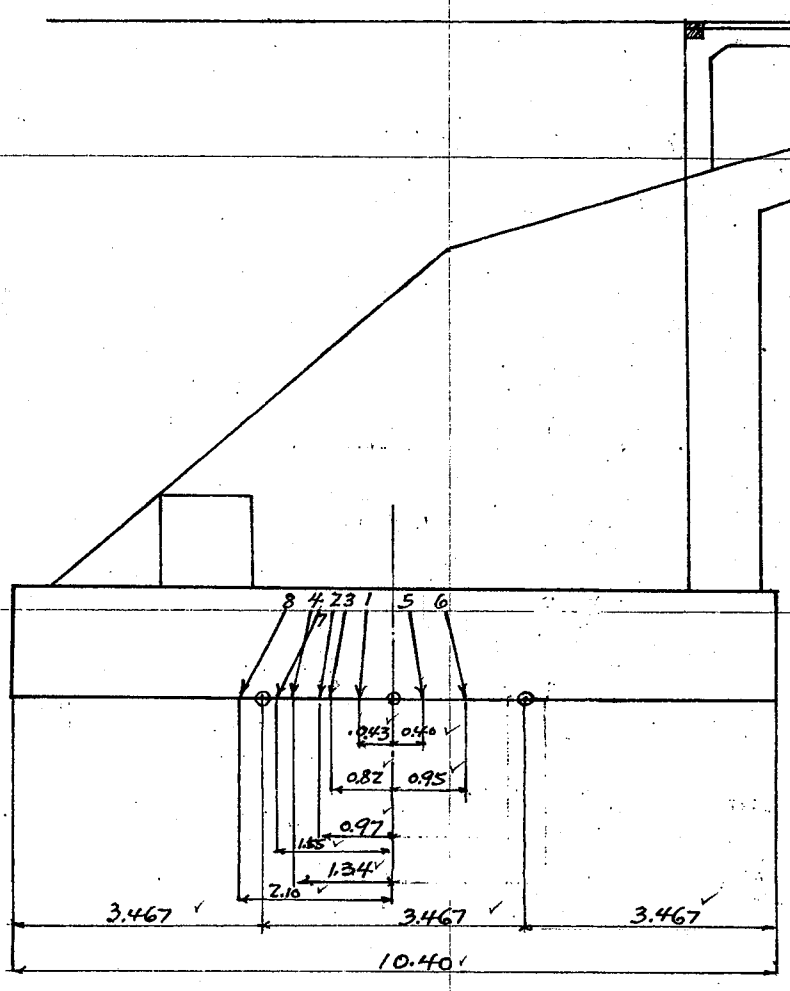
CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.
Summary of Eccentricities and Max. bearing pressures.



Cases.		max. bearing pressure	Eccentricity	$\Sigma H / \Sigma V$
Case 1	Dead Load only	23,700 kg/m² (2.17 tons/ft²)	+ 0.43 m	0.08
2	Dead Load + Temperature.	29,650 " (2.71 ")	+ 0.97	0.15
3	Dead Load + Live Load	29,550 " (2.70 ")	+ 0.82	0.15
4	Dead Load, Live Load + Temperature.	35,550 " (3.25 ")	+ 1.34	0.21
5	Dead Load + Seismic forces, River side.	23,400 " (2.14 ")	− 0.40	0.16
6	Dead Load + Seism. forces with Temperature.	29,400 " (2.69 ")	− 0.95	0.23
7	Dead Load + Seismic forces, Shore side	36,000 " (3.29 ")	+ 1.55	0.45
8	Dead Load + Seism. forces with Temperature.	42,450 " (3.88 ")	+ 2.10	0.52

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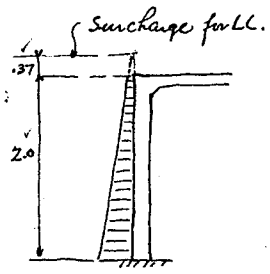
CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

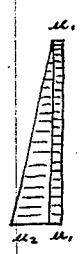
Design of Parapet wall.



Earth pressure on wall. $.37 \times \frac{1600}{3} = 200 = M_1$

$2.37 \times \frac{1600}{3} = 1260$

diff. $1060 = M_2$



Moment due to

M_1 $\frac{200 \times 2.0^2}{10} = 80.$

M_2 $0.8 \times \frac{1060 \times 2.0^2}{15.6} = 218$

298 kgm.

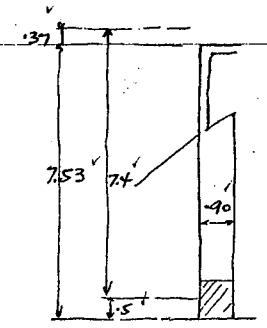
Eff. depth req'd. $= \sqrt{\frac{298 \times 100}{100 \times 7.18}} = 6.45$ cm

Use 27 cm eff. depth with 3 cm insulation. total depth 30 cm.

Steel area req'd $= \frac{298 \times 100}{1200 \times \frac{7}{8} \times 27} = 1.05$ cm² per meter strip.

use 4-12 mmϕ bars $= 4.56$ cm² on both sides.

Design of Front wall



span length 2.50m. both ends fixed assumed.

Earth pressure on wall $\frac{1600}{3} \times 7.4 = 3950$ kg/m

Moment at normal state $= \frac{1}{12} \times 3950 \times 2.5^2 = 2050$ kgm.

Shear " " $= \frac{1}{2} \times 3950 \times 2.5 = 4935$ kg

Earth pressure during earthquake $.559 \times 1600 \times 7.03 = 6290$ kg/m

Seismic force on wall $.9 \times 2400 \times .25$ $= 540$

6830

Seismic moment $= \frac{1}{12} \times 6830 \times 2.5^2 = 3560$ kgm

" Shear $= \frac{1}{2} \times 6830 \times 2.5 = 8540$ kg

Seismic stresses govern the section.

Effective depth required $= \sqrt{\frac{3560 \times 100}{1.6 \times 100 \times 7.18}} = 17.6$ cm

use 85 cm eff. depth with 5 cm insulation

Steel area required $= \frac{3560 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 85} = 2.5$ cm²

use 2 - 19 mmϕ bars $= 5.67$ cm² on both sides

Unit shear $= \frac{8540}{100 \times \frac{7}{8} \times 85} = 1.15$ kg/cm² ok

Unit bond $= \frac{8540}{5.97 \times 2 \times \frac{7}{8} \times 85} = 9.60$ " ok. $6.0 \times 1.6 = 9.6$ allowable bond.

Base reinforcements.

	max. bearing pressure.	downward earth pres.	wt. of base.	Summary
Case. 3. (normal state)	29,550	−12,650	− 3600	+ 13,300 upward
Case 8 (earthquake.)	42,450	− 12,050	− 3600	+ 26,800 "

Moment on base $= \frac{1}{12} \times 26800 \times 2.5^2 = 13960$ kgm

Shear " $= \frac{1}{2} \times 26800 \times 2.5 = 33500$ kg

Eff. depth req'd. $= \sqrt{\frac{13960 \times 100}{100 \times 1.6 \times 7.18}} = 34.9$

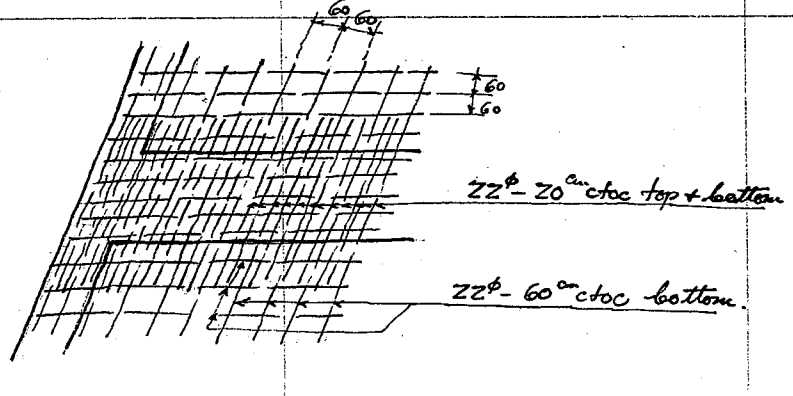
use 115 cm eff. depth.

Steel area req'd $= \frac{13960 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 115} = 7.2$ cm²/m strip.

use 5-22 mmϕ $= 19.0$ cm²

Unit shear $= \frac{33500}{100 \times \frac{7}{8} \times 115} = 3.33$ kg/cm² ok

Unit bond $= \frac{33500}{6.91 \times 5 \times \frac{7}{8} \times 115} = 9.6$ " ok, $6.0 \times 1.6 = 9.6$



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CONSULTING ENGINEER
JIJI BLDG, TOKYO

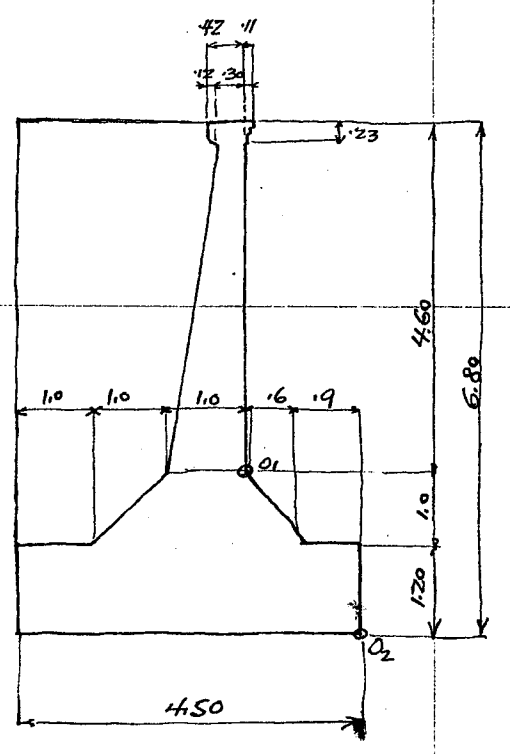
MADE BY K. Iraba DATE 5-11-17 FILE NO
CHECKED BY M. Jinda DATE 5-11-18 PAGE NO 43

CALCULATIONS FOR

Design of Jonan Bashi for Kyoto Prefecture.

Design of wing wall.

General crossection assumed as shown on sketch bellow



Weight and center of gravity of wall.

			Hor. arm	abt O1	Vert. arm	about O
Coping	.23×.5 = .115		× .19	.022	.450	.517
wall	.65×4.37 =	2.840	× .357	1.015	1.79	5.085
		2.955	.351	1.037	1.898	6.0 5.602

2.955 c 2400 = 7100 kg

			arm abt O2		arm abt O2	
base	1.80×1.0 = 1.80		2.11	3.80	1.626	2.925
	1.20×4.5 =	5.40	2.35	12.15	.60	3.240
		7.20	2.22 m	15.95	.857 m	6.165

7.20 c 2400 = 17,300 kg

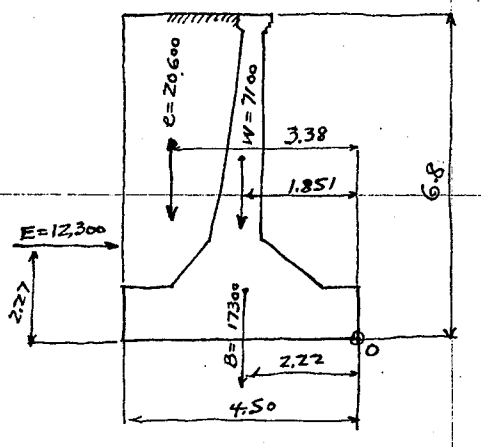
Earth filling on rear footing.

1.3 × 5.6 = 7.30	× 2.90	=	21.2
1.0 × 5.6 = 5.60	× 4.00	=	22.4
12.90 c 1600 = 20,600	3.38		43.6

Earth pressure at normal state = $\frac{1600 \times 6.8^2}{6}$ = 12,300 kg

Earth pressure during earthquake = $.559 \times \frac{1600 \times 6.8^2}{2}$ = 20,700 kg

Stability of wing wall at normal state taking moment about point o.



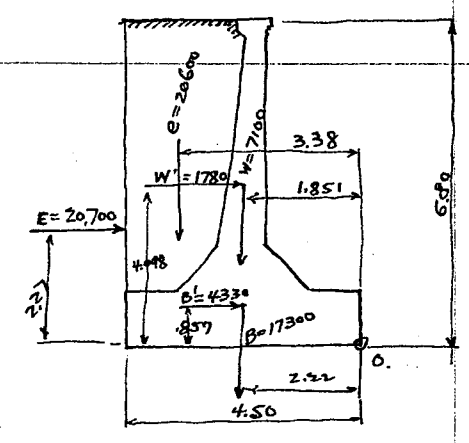
Load	Hor. forces	Vert. forces	Lev. arms	Moments
W		7,100 ×	1.851 =	13140
B		17,300 ×	2.22 =	38,400
e		20,600 ×	3.38 =	69,500
E	12,300	×	−2.27 =	−27900
	12,300 kg	45000 kg	2.07 m	93140

Eccentricity = 2.25 − 2.07 = 0.18 m

Max. toe pressure = $\frac{45000}{1.0 \times 4.50}\left(1 \pm \frac{6 \times .18}{4.50}\right)$ = 12,400 kg/m² or (1.13 ton/ft²)

≈ 7600 ≈ (.70 ton/ft²)

Stability during Earthquake



Load	Hor. forces	Vert. forces	Lev. arm	Moments	Seism. force in reverse
W		7,100 ×	1.851 = +	13140	+ 13140
B		17,300 ×	2.22 = +	38,400	+ 38400
e		20,600 ×	3.38 = +	69,500	+ 69500
W'	1780	×	4.098 = −	7,290	+7290
B'	4330	×	0.857 = −	3,710	+3710
E	20.700	×	2.27 = −	46.950	
	26,810 kg	45000 kg	1.40 +	63090	+ 132,040

Eccentricity = 2.25 − 1.40 = .85 m

Resultant force outside of middle third

pressure area = 1.40 × 3 = 4.20

max. toe pressure = $\frac{45000 \times 2}{1.0 \times 4.20}$ = 21,400 kg/m² or (1.96 ton/ft²)

Earthquake in reversed direction.

Hor. forces	Vert. forces	arm	moments
6110	45,000	2.93	132,040

Eccentricity = 2.93 − 2.25 = .68 m

Max. bearing pressure = $\frac{45000}{1.0 \times 4.5}\left(1 \pm \frac{6 \times .68}{4.5}\right)$ = 19070 kg/m² (1.25 ton/ft²)

≈ 930

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MADE BY K. Imaba DATE 5-11-17 FILE NO
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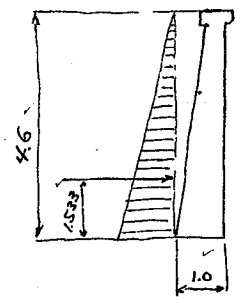
CALCULATIONS FOR

Design of Jōnan Bashi for Kyoto Prefecture.

Wall Reinforcements



Earth pressure at normal state $\frac{1600 \times 4.6^2}{6} = 5650$ kg

moment on wall $= 5650 \times 1.533 =$ 8660 kgm/m strip.

Earth pressure during Earthquake $= \frac{.559 \times 4.6^2 \times 1600}{2} = 9470$ kg

moment on wall due to

Earth pressure $= 9470 \times 1.533 = 14500$

seismic force on wall $= 1780 \times 1.898 = 3380$

shear $= 11250$ kg 17880 kgm/m strip.

Seismic moment governs the section of wall.

Effective depth required $= \sqrt{\frac{17880 \times 100}{16 \times 100 \times 7.18}} = 39.5$ cm

use Effective depth of 95 cm with 5 cm insulation.

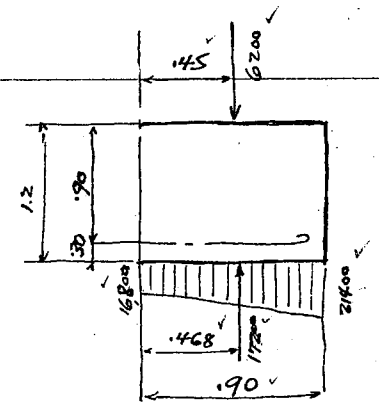
Steel area required $= \frac{17880 \times 100}{16 \times 1200 \times \frac{7}{8} \times 95} = 11.20$ cm²

use 22 mm φ bars at 30 cm c to c $= 12.68$

unit shear $= \frac{11250}{100 \times \frac{7}{8} \times 95} = 1.36$ kg/cm² ok

unit bond $= \frac{11250}{6.91 \times 3.33 \times \frac{7}{8} \times 95} = 5.88$ ok.

Design of Cantilever footing at toe.



max. upward pressure

21,400
16,800
38,200 ÷ 2 = 19,100 × 0.9 = 17,200 kg

Downward pressure

weight of footing 1.2 @ 2400 = 2,880

" earth on footing 2.5 @ 1600 = 4000

6880 × .9 = 6,200 kg

moment on footing

upward pressure 17,200 × .468 = 8050

downward − 6200 × .450 = − 2,790

shear = 11,000 kg moment = 5260 kgm.

Steel ratio $= \frac{9.45}{90 \times 100} = .00105$

Steel area required $= \frac{5260 \times 100}{1.6 \times 1200 \times \frac{7}{8} \times 90} = 3.49$ cm²/m strip.

$j = .946$

use 19 mm φ bars at 30 cm c to c $= 9.45$ " "

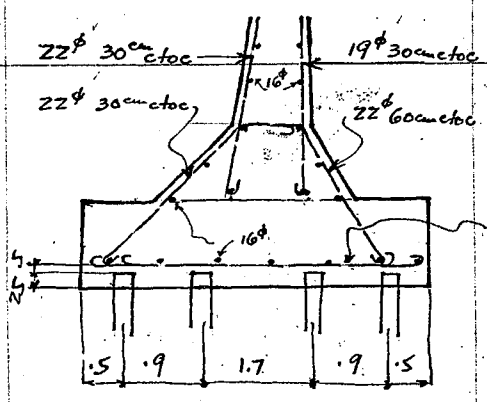
unit shear $= \frac{11000}{100 \times .946 \times 90} = 1.3$ kg/cm² ok

unit bond $= \frac{11000}{5.97 \times 3.33 \times .946 \times 90} = 6.5$ " ok allowable bond $= 6.0 \times 1.6 = 9.6$

Cantilever footing at heel.

For positive moment use the same reinforcement as above.

negative moment.



Earth on footing 5.6 @ 1600 = 8960

footing 1.2 @ 2400 = 2880

11,840 × 0.5 = 5920 kgm

Steel req'd $= \frac{5920 \times 100}{1.6 \times 1200 \times \frac{7}{8} \times 90} = 3.92$ cm²

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CALCULATIONS FOR

MADE BY_____DATE_____FILE NO_____
CHECKED BY_____DATE_____PAGE NO_____

昭和五年十一月

京都府

京阪國道鴨川橋梁設計々算書

附材料計算書

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

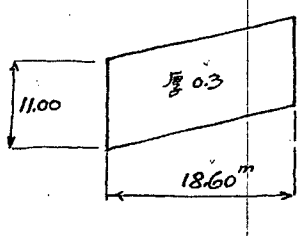
MADE BY M. Kojima DATE 5-11-23 FILE NO
CHECKED BY M. Jinda DATE 5-11-26 PAGE NO 45

Materials of Jonan Bashi for Kyoto Prefecture.

鴨川橋梁材料計算書

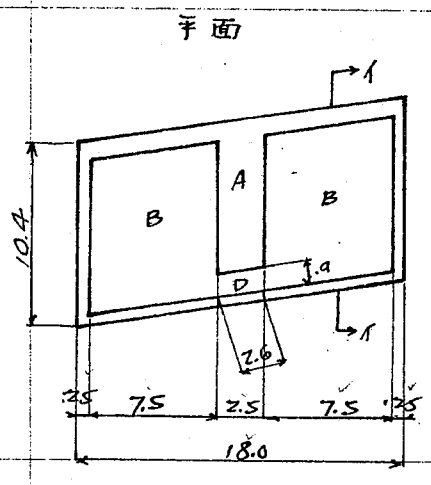
第壹編　橋臺工　(一基當り)　(Sheet no. 10 參照)
　第壹章　基礎工.
　　第壹節　基礎杭
　　　木口 21 糎 長 6.50米　生杭丸太　112本

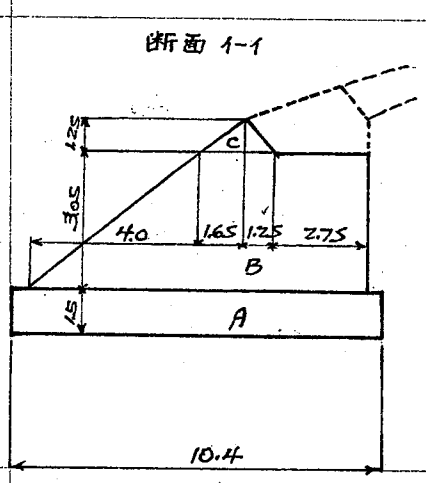
　第貳節　基礎栗石



18.60 × 11.00 × 0.30 = 61.4 cub. meters.

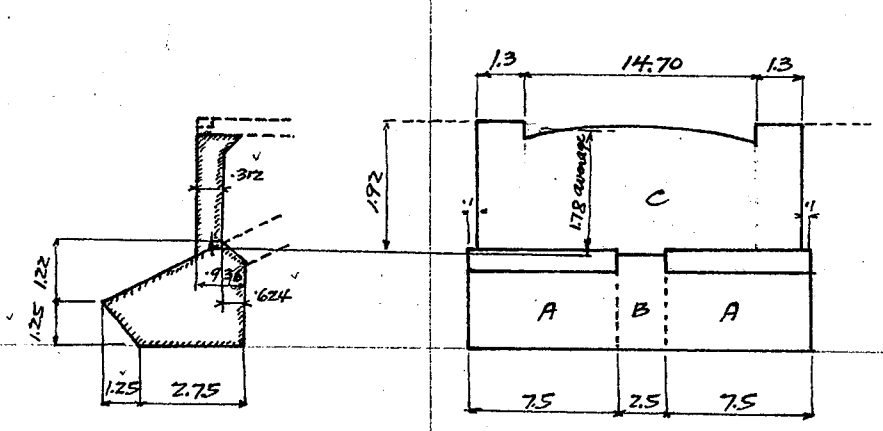
第貳章　躯体工
　第壹節　混凝土
　　第壹款　配合 1:3:6 混凝土.





A 部	10.4 × 18.0 × 1.5	=	280.80
B 〃	(5.65+9.65)÷2 = 7.65 × 3.05 × 7.5 × 2	=	349.98
C 〃	½ × 2.90 × 1.25 × 7.5 × 2	=	27.18
D	2.6 × 1.9 × 3.05	=	7.14
			665.10
	隅石体積	約	− 0.06
	1:3:6 混凝土 合計		665.04 m³

　第貳款　配合 1:2:4 混凝土.



A 部	(2.75+4.0)÷2 = 3.375 × 1.25 × 7.5	=	31.64
	(.70+1.23)÷2 = .965 × .624 × 7.5	=	4.52
	½ × 1.22 × 3.376 × 7.5	=	15.45
	51.61 × 2	=	103.22
B 部	.936 × 2.2 × 2.5	=	5.15
C 部	1.3 × 1.92 × .312 × 2	=	1.56
	14.7 × 1.78 × .312	=	8.16
fillet	.15 × .15 × ½ × 17.37	=	.20
			9.92
			118.29
	隅石体積 減	約 2 × .35 × .17 × 1.99	= − .24
	1:2:4 混凝土 合計		118.05 m³

第貳節　鐵筋（丸鋼）

Mark	Req'd. no.	Size	Length	Unit wt.	Piece wt.	Total wt	Remarks.	
A 1	84	22㎜φ	5.95m	2.98 kg/m	17.73 kg	1,489.32 kg	Base, bottom	
A 2	54	〃	7.10	〃	21.16	1,142.64	〃 〃	〃
A 3	76	〃	4.40	〃	13.11	996.36	〃 at center	〃
A 4	10	19	4.15	2.22	9.21	92.10	Body	〃
A 5	42	22	7.20	2.98	21.46	901.32	〃	〃
A 6	30	19	6.80	2.22	15.10	453.00	〃	〃
A 7	28	22	2.35	2.98	7.00	196.00	〃 Springing	〃
A 8	32	〃	2.50	〃	7.45	238.40	〃 dowels	〃
A 9	32	〃	7.90	〃	23.54	753.28	〃	〃
A 10	60	25	5.40	3.85	20.79	1,247.40	Arch ring	〃

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MADE BY M. Juida DATE 5-11-25 FILE NO
CHECKED BY M. Kojima DATE 5-11-27 PAGE NO 46

CALCULATIONS FOR

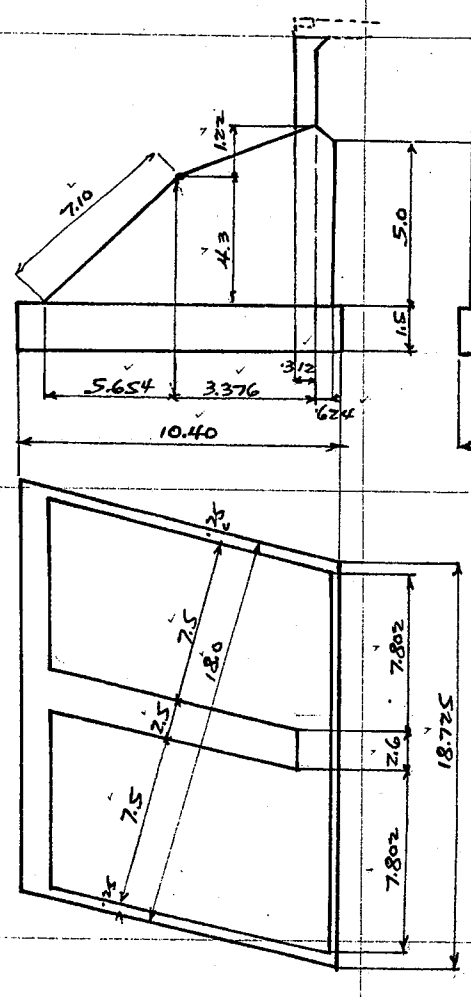
Materials of Jonan Bashi for Kyoto Prefecture.

Mark	Reqd. no.	Size	Length	Unit wt.	piece wt.	Total wt.	Remarks.
A 11	60	2.5φ	3.60m	3.85	13.86 kg	831.60	Arch ring
A 12	116	2.2	5.90	2.98	17.58	2039.28	"
A 13	64	1.6	2.85	1.57	4.48	286.72	dowels for stirrups
A 14	16	"	3.00	"	4.71	75.36	Stirrups
A 15	16	"	3.90	"	6.12	97.92	"
A 16	16	"	2.70	"	4.24	67.84	"
A 17	16	"	3.65	"	5.73	91.68	"
A 18	16	"	4.45	"	6.99	111.84	"
A 19	16	"	5.20	"	8.16	130.56	"
A 20	16	"	5.75	"	9.03	144.48	"
A 21	16	"	4.45	"	6.99	111.84	"
A 22	16	"	3.00	"	4.71	75.36	"
A 23	22	"	8.00	"	12.56	276.32	body
A 24	32	"	4.80	"	7.54	241.28	Stirrups
A 25	69	1.2	2.75	.88	2.42	166.98	Parapet wall
A 26	69	"	3.10	"	2.73	188.37	"
A 27	65	"	.95	"	.84	54.60	" fillet.
A 28	76	"	6.45	"	5.68	147.68	Parapet wall
A 29	4	"	5.10	"	4.49	17.96	"
A 30	8	"	1.40	"	1.23	9.84	End of Parapet
A 31	8	"	2.00	"	1.76	14.08	"

17,691.41 kgs. or 17.691 kg tons

内 2.5φ & 2.2φ 12 983.560 kg 1.9φ & 1.6φ 12 2,256.30 kg 1.2φ 12 599.51 kg

第参節 型枠



Base	18.725 × 1.5 × 2	=	56.2
"	10.40 × 1.5 × 2	=	31.2
body front.	18.204 × 5.0	=	91.0
" rear	7.1 × 7.802 × 2	=	110.8
" "	2.6 × 5.58	=	14.5
" Sides	4.3 × 5.654 × ½ × 4	=	48.6
" "	4.91 aor. × 3.376 × 4	=	66.3
" "	5.765 × .624 × 2	=	6.6
" " less	5.47 × .312 × 2	=	− 3.4
Parapet	1.92 × 1.352 × 4	=	10.4
"	1.80 × 15.292 × 2	=	55.1

487.3 sq. meters

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MADE BY M. Kojima DATE 5-11-23 FILE NO
CHECKED BY M. Jinda DATE 5-11-26 PAGE NO 47

CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

第四節　隅石

花崗石.

8 e .48 × .25 × .30 = .29
8 e .36 × .25 × .30 = .22
.51 cub. meter.

仕上面積.

表面仕上　2 e .60 × 2.40 = 2.88
合端 3cm　16 e .03 × 1.80 = .86
3.74 sq. meters

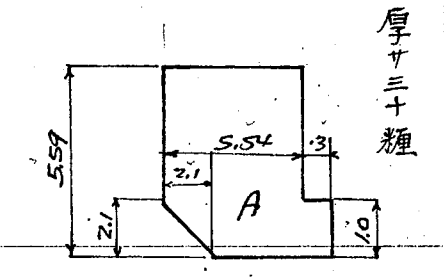
中切仕上・却　32 e ½(.10 + .23) × .35 = 1.85 "

第參章　翼壁工

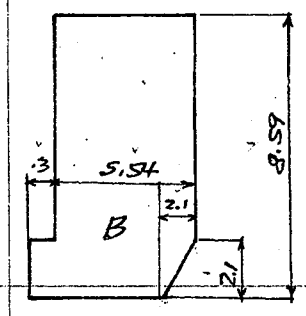
第壹節　基礎杭

末口 18糎 長 5.0米　生杭 丸太
A 翼壁　14
B 〃　22
計 36 本

第貳節　基礎栗石



厚サ三十糎



A 翼壁

5.59 × 5.54 = 31.0
less 2.1 × 2.1 × ½ = − 2.2
1.0 × .3 = .3
29.1 × .3 = 8.73

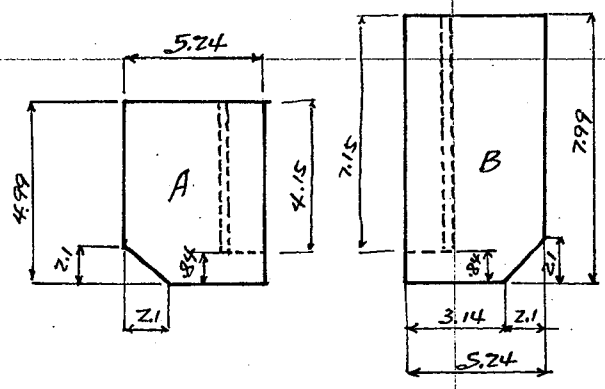
B 翼壁.

8.59 × 5.54 = 47.6
less 2.1 × 2.1 × ½ = − 2.2
1.0 × .3 = .3
45.7 × .3 = 13.71

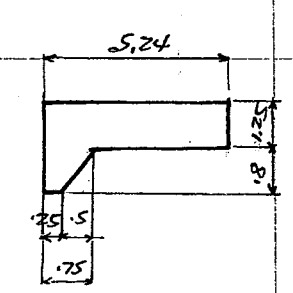
合計　22.44 cub. m.

第參節　混凝土.

第壹款　配合 1:3:6 混凝土.



平面



断面

A 翼壁

4.99 × 5.24 × 1.25 = 32.68
less 2.1 × 2.1 × ½ × 1.25 = − 2.76
.5 × .8 × 4.15 = 1.66
31.58

B 翼壁.

7.99 × 5.24 × 1.25 = 52.33
less 2.1 × 2.1 × ½ × 1.25 = − 2.76
.5 × .8 × 7.15 = 2.86
52.43

AとB合計　84.01 cub. m.

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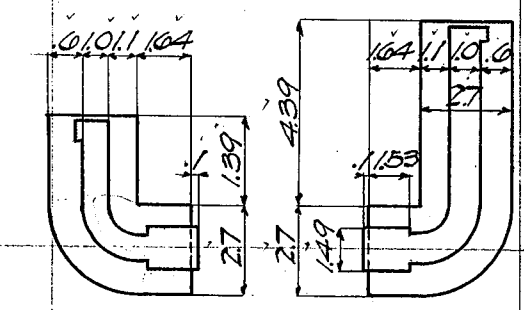
MADE BY M. Kojima DATE 5-11-23 FILE NO
CHECKED BY M. Ueda DATE 5-11-26 PAGE NO 49

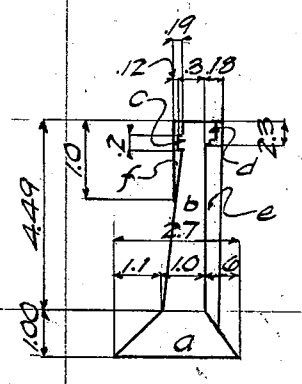
CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture.

第弐 築　　配合 1:2:4 混凝土



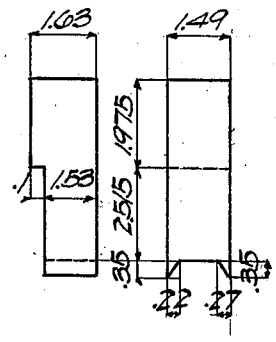
平　面



断　面

A 翼壁

断面積	a	(2.7+1.0)½ × 1.00	=	1.85 sq.m
	b	(1.0+.3)½ × 4.449	=	2.92
	c	.2 × .12	=	.02
	d	.23 × .11	=	.03
容積	a	1.85 × (1.64+1.39+2.72) (5.75)	=	10.64 cu.m
	b	2.92 × (1.29+.11+3.1) (4.5)	=	13.14
	c	.02 × 3.5	=	.07
	d	.03 × 4.3	=	.13
	e	.18 × 4.5 × .47	=	.38
	f	.19 × .80 × ½ × .47	=	.04
	Column	1.49 × 1.53 × 2.515	=	5.73
		1.49 × 1.63 × 1.975	=	4.80 } 10.66
		1.53 × .35 × (.22+.27) (1.49) × ½	=	.13
				35.06 cub.m



B 翼壁

容積	a	1.85 × (1.64+4.39+2.72) (8.75)	=	16.19
	b	2.92 × (4.29+.11+3.1) (7.5)	=	21.90
	c	.02 × 6.5	=	.13
	d	.03 × 7.3	=	.22
	e	same as for A	=	.38
	f	"	=	.04
	Column	"	=	10.66
				49.52 cub.m

A, B 両翼壁合計 = 84.58 "

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MADE BY M. Jinda DATE 5-11-25 FILE NO
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CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture.

第四節 鐵筋 (丸鋼)

Mark	Reqd. no	Size	Length	Unit wt	Piece wt	Total wt	Remarks.
Common Reinforcements for both wing walls A + B.							
W 1	14	1.9 cm φ	4.05 m	2.22 kg/m	8.99 kg	125.86 kg	
W 2	10	〃	5.00	〃	11.10	111.00	
W 3	7	1.6	5.20	1.57	8.16	57.12	
W 4	11	1.9	4.80	2.22	10.00	116.60	
W 5	7	〃	3.15	〃	0.99	48.93	
W 6	5	〃	3.95	〃	8.77	43.85	
W 7	11	1.6	3.90	1.57	6.12	67.32	
W 8	13	2.2	6.25	2.98	18.63	242.19	
W 9	10	〃	3.75	〃	11.18	111.80	
W 10	11	〃	3.85	〃	11.47	126.17	
W 11	17	1.9	3.75	2.22	8.33	141.61	
W 12	18	〃	3.50	〃	7.77	139.86	
W 13	11	〃	3.05	〃	6.77	74.47	
W 14	17	1.6	3.00	1.57	4.71	80.07	
W 15	2	〃	4.00	〃	6.28	12.56	
W 16	22	〃	3.45	〃	5.42	119.24	
W 19	22	1.2	3.60	.88	3.17	69.74	
W 20	2	1.6	1.80	1.57	2.83	5.66	
W 21	10	1.2	1.40	.88	1.23	12.30	
						1,706.35 kg	
Special Reinforcements for wing wall A.							
W 17	11	1.6	3.55	1.57	5.58	61.38	
W 18	11	1.6	2.15	1.57	3.38	37.18	
						98.56 kg	
Special Reinforcements for wing wall B.							
W 1	10	1.9	4.05	2.22	8.99	89.90	
W 2	10	〃	5.00	〃	11.10	111.00	
W 5	5	〃	3.15	〃	0.99	34.95	
W 8	5	2.2	6.25	2.98	18.63	93.15	
W 9	5	〃	3.75	〃	11.18	55.90	
W 10	10	〃	3.85	〃	11.47	114.70	
W 11	10	1.9	3.75	2.22	8.33	83.30	
W 13	10	〃	3.05	〃	6.77	67.70	
W 14	10	1.6	3.00	1.57	4.71	47.10	
W 22	10	〃	5.10	〃	8.01	80.10	
W 23	10	〃	6.45	〃	10.13	101.30	
W 24	17	〃	3.70	〃	5.81	98.77	
						977.87 kg	

Total Reinforcements for wing wall A = 1706.35 + 98.56 = 1804.91
〃 〃 〃 〃 〃 B = 1706.35 + 977.87 = 2684.22
4489.13 kg
or 4.489 kg tons

内 2.2 cm φ 1.21 1.224 kg tons
1.9 & 1.6 〃 3.101
1.2 〃 0.164

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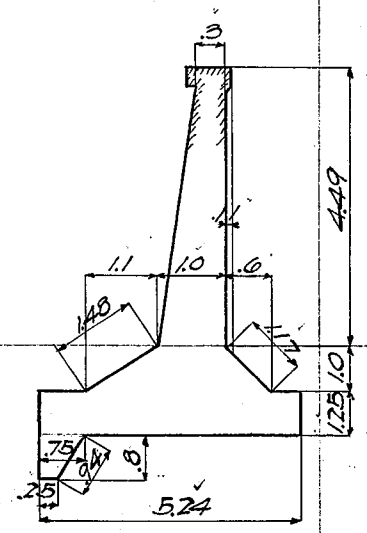
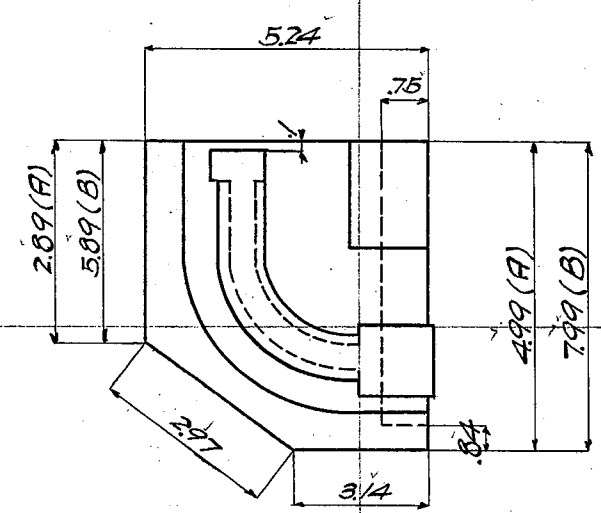
CALCULATIONS FOR

Materials for Jōnan Bashi for Kyoto Prefecture.

第五節　型枠



断面



平面

A 翼壁

Base	(3.14 + 2.97 + 2.89 + 5.24) × 1.25 (14.24)	=	17.8	
〃	(.75 + .25) × ½ × .8 × 2	=	.8	
〃 〃	.94 × 4.15	=	3.9	
outside fillet	1.17 × 7.0	=	8.2	
inside 〃	1.48 × 4.0	=	5.9	
wall, outside	4.49 × 6.7	=	30.1	
〃 inside	4.50 × 5.0	=	22.5	
end face	(2.7 + 1.0) ½ × 1.0	=	1.9	
〃 〃	(1.11 + .41) ½ × 4.49	=	3.4	
Coping	.19 × 4.40	=	.8	
〃	.11 × 3.40	=	.4	
Column	1.12 × 2.15	=	2.4	
				98.1 ㎡

B 翼壁

Base	(3.14 + 2.97 + 5.89 + 5.24) × 1.25 (17.24)	=	21.60	
〃	(.75 + .25) × ½ × .8 × 2	=	.8	
〃	.94 × 7.15	=	6.7	
outside fillet	1.17 × 10.0	=	11.7	
inside 〃	1.48 × 7.0	=	10.2	
wall outside	4.49 × 9.7	=	43.6	
〃 inside	4.50 × 8.0	=	3.6	
end		=	5.3	
Coping	.19 × 7.40	=	1.4	
〃	.11 × 6.40	=	.7	
Column	1.12 × 2.15	=	2.4	
				140.4 ㎡

A,B 両翼壁 合計 = 238.5 ㎡

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MADE BY M. Jinda DATE 5-11-27 FILE NO
CHECKED BY M. Kajima DATE 5-11-29 PAGE NO 51

CALCULATIONS FOR

Materials for Jonan Bashi for Kyoto Prefecture.

第六節　縁石（花崗石）

A 翼壁

品種	長	幅	厚	單位	員数	体積	体積合計
花崗石	.38	.20	.21	本	1	.016	.016
〃	.90	.24	〃	〃	3	.045	.135
〃	.80	.19	〃	〃	1	.032	.032
							0.183 m³
B 翼壁							
花崗石	.38	.20	.21	〃	1	.016	.016
〃	.90	.24	〃	〃	3	.045	.135
〃	.95	.19	〃	〃	4	.038	.152
							0.303 m³
				A,B 兩翼壁 合計		=	0.486 m³

仕上面積

表面仕上　合端 3粗

.39 × 3.97	=	1.55
2 × .03 × 3.97	=	.24
10 × .03 × .39	=	.12
A 翼壁 計		1.91 m²

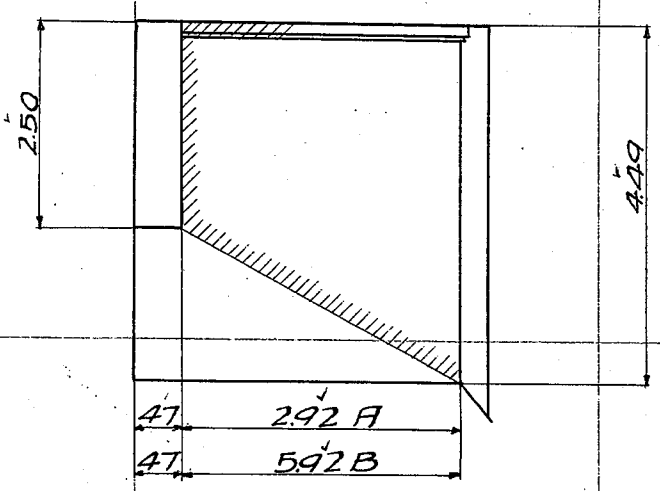
素面仕上　合端 3粗

.39 × 6.97	=	2.72
2 × .03 × 6.97	=	.42
16 × .03 × .39	=	.19
B 翼壁計		3.33 m²
A,B 兩翼壁計	=	5.24 m²

中切仕上　A 壁　10 @ .15 × .18 = .27
B 〃　16 @ .15 × .18 = .43
A,B 兩翼壁 計 = 0.70 m²

荒切仕上　A壁　.33 × 3.88 = 1.28
B 〃　.33 × 6.88 = 2.27
3.55 m²

第七節　人造洗出仕上（厚 1.5 cm）



A 翼壁

親柱下柱	1.73 × 4.49 =	7.77
翼壁	½(2.50 + 6.99 + 4.49) × 4.3 =	15.03
地覆	.11 × 4.3 =	.47
		23.27 m²

B 翼壁

親柱下柱	1.73 × 4.49 =	7.77
翼壁	½(2.5 + 4.49) × 7.3 =	25.51
地覆	.11 × 7.3 =	.80
		34.08 m²
A,B 兩翼壁 合計		57.35 m²

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MADE BY M. Jiuda DATE 5-11-21 FILE NO
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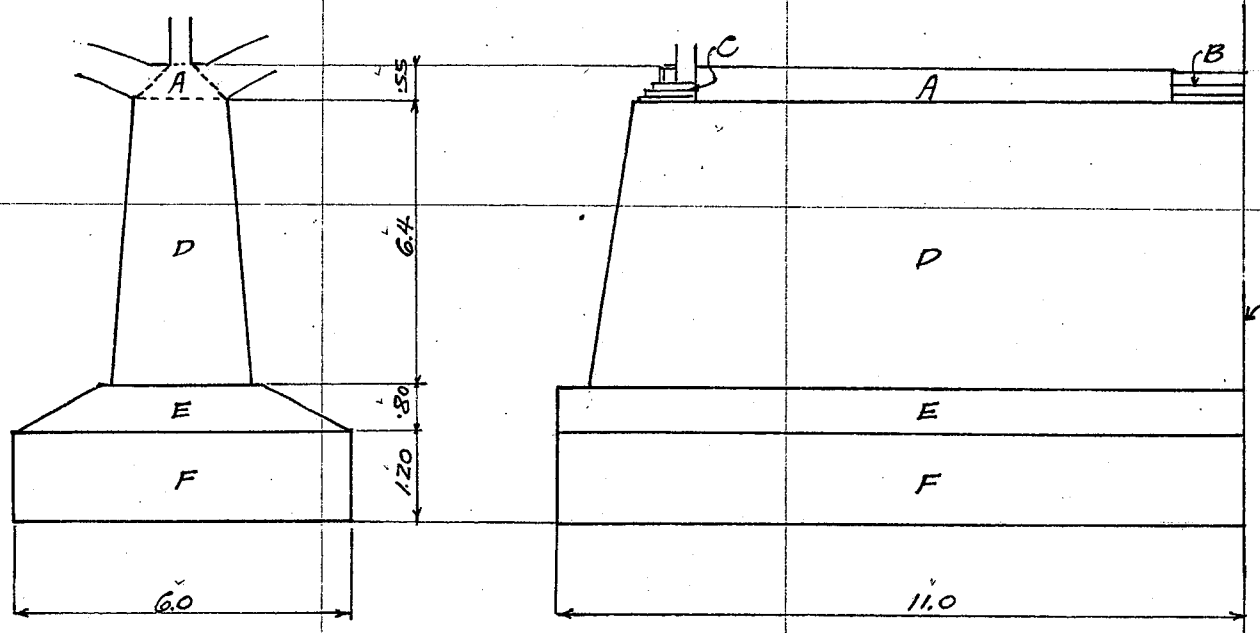
CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture
第貳編 橋脚 (一基當リ)
第壹章 橋脚躯体
第壹節 混凝土 (配合 1:2:4) (Sheet 8 參照)



A 部	.300					
	1.442					
	1.742 ÷2 =	.871 ×	.55 ×	7.803 × 2 =	7.48	Top core between arch ribs.
B 部		1.282 ×	.35 ×	2.60 =	1.17	" " " at center.
C 部	Say	2 ×	0.10 m³	=	0.20	back of nose stones.
D 部	1.442					
	2.722					
	4.164 ÷2 =	2.082 ×	6.40 ×	18.67 =	248.77	between tangent lines
		2.082 ℄ ×	6.40 =		21.79	circular ends
					279.44	
	張石体積減額				− 9.04	
			差引		270.37	cub. meters.

第貳節 鐵筋 (丸鋼) (Sheet No.8 參照)

Mark	Reqd. no.	Size	Length	Unit wt.	Piece wt.	Total wt.	Remarks.		
P1	88	2.2 cm℄	6.30 m	2.98 kg/m	18.77 kg	1,651.76 kg	Base		
P2	44	"	4.85	"	14.45	635.80	"		
P3	27	"	8.20	"	24.44	659.88	"		
P4	280	"	5.10	"	15.20	4,256.00	Shaft		
P5	20	"	5.15	"	15.35	307.00	"	end.	
P6	20	"	4.40	"	13.11	262.20	"	"	
P7	49	1.9	6.90	2.22	15.32	750.68	"		
P8	38	"	7.30	"	16.21	615.98	"		
P9	4	"	6.20	"	13.76	55.04	"	band	
P10	6	"	5.75	"	12.77	76.62	"	"	
P11	4	"	4.55	"	10.10	40.40	"	"	
P12	4	"	4.25	"	9.44	37.76	"	"	
P13	4	"	3.90	"	8.66	34.64	"	"	
P14	4	"	3.60	"	7.99	31.96	"	"	
P15	4	"	3.30	"	7.33	29.32	"	"	
P16	6	"	7.95	"	17.65	105.90	"	top	
P17	4	"	4.10	"	9.10	36.40	"	"	
P18	120	2.5	4.00	3.85	15.40	1848.00	arch	ring	
P19	120	"	2.65	"	10.21	1,225.20	"	"	
P20	116	2.2	6.20	2.98	18.48	2,143.68	"	"	

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MADE BY M. Jiuda DATE 5-11-21 FILE NO
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CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture.

Mark	Required no.	Size	Length	Unit wt.	Piece wt.	Total wt.	Remarks.
PZ1	116	2.2$^{\phi}$	4.80	2.98	14.30	1,658.80	arch ring
PZ2	76	1.2	1.15	0.88	1.02	77.52	spandrel wall at center
PZ3	44	"	1.70	"	1.50	66.00	" " "
PZ4	16	"	1.30	"	1.14	18.24	dowels at ends on top "
						16,624.78 kg or 16.625 kg tons.	
						内 25$^{\phi}$と22$^{\phi}$ 14.648	19$^{\phi}$ 1.815 12$^{\phi}$ 0.162

第参節　張石，花崗石 (Sheet no. 9 参照)

名稱	品種	寸法 長	幅	厚	單位	員数	一個當り体積	合計体積
1	花崗石	.473^{m}	.265^{m}	.300^{m}	箇	14	.038 m^3	.532 m^3
1a	"	.476	.265	"	"	2	.038	.076
1b	"	.411	.250	"	"	16	.031	.496
2	"	.461	.265	"	"	14	.037	.518
2a	"	.331	.260	"	"	2	.026	.052
2b	"	.431	.265	"	"	2	.034	.068
2c	"	.308	.250	"	"	16	.023	.368
3	"	.449	.265	"	"	14	.036	.504
3a	"	.452	.265	"	"	2	.036	.072
4	"	.437	.265	"	"	14	.035	.490
4a	"	.319	.260	"	"	2	.025	.050
4b	"	.419	.260	"	"	2	.033	.066
5	"	.425	.265	"	"	14	.034	.476
5a	"	.428	.265	"	"	2	.034	.068
6	"	.413	.265	"	"	14	.033	.462
6a	"	.307	.260	"	"	2	.024	.048
6b	"	.407	.260	"	"	2	.032	.064
7	"	.401	.264	"	"	14	.032	.448
7a	"	.404	.264	"	"	2	.032	.064
8	"	.389	.264	"	"	14	.031	.434
8a	"	.295	.260	"	"	2	.023	.046
8b	"	.395	.260	"	"	2	.031	.062
9	"	.377	.264	"	"	14	.030	.420
9a	"	.380	.264	"	"	2	.030	.060
10	"	.365	.263	"	"	14	.029	.406
10a	"	.283	.258	"	"	2	.022	.044
10b	"	.383	.258	"	"	2	.030	.060
11	"	.353	.263	"	"	14	.028	.392
11a	"	.356	.263	"	"	2	.028	.056
12	"	.341	.263	"	"	14	.027	.378
12a	"	.271	.258	"	"	2	.021	.042
12b	"	.371	.258	"	"	2	.029	.058
13	"	.329	.262	"	"	14	.026	.364
13a	"	.332	.262	"	"	2	.026	.052
14	"	.317	.262	"	"	14	.025	.350
14a	"	.259	.256	"	"	2	.020	.040
14b	"	.359	.256	"	"	2	.028	.056
15	"	.305	.261	"	"	14	.024	.336
15a	"	.308	.261	"	"	2	.024	.048
16	"	.293	.261	"	"	14	.023	.322
16a	"	.247	.256	"	"	2	.019	.038
16b	"	.347	.256	"	"	2	.027	.054
張石計							9.040 m^3	304^{t}

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MADE BY M. Kojima DATE 5-11-5 FILE NO
CHECKED BY M. Jinda DATE 5-11-10 PAGE NO 54

CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture.

笠石及柱裝飾用石材 (Sheet no 9 参照)

名稱	品質	寸法 長	幅	厚	單位	員数	一個當リ体積	合計体積
17	花崗石	.505	.325	.244	個	6	.040	.240
17a	〃	.550	.350	.244	〃	2	.047	.094
17b	〃	.413	.300	.244	〃	2	.030	.060
17c	〃	.700	.320	.150	〃	2	.034	.068
18	〃	.540	.400	.394	〃	2	.085	.170
19	〃	.300	.394	.300	〃	4	.035	.140
20	〃	.400	.300	.171	〃	4	.021	.084
21	〃	.400	.300	.294	〃	16	.035	.560
22	〃	.400	.300	.137	〃	4	.016	.064
23	〃	.314	.228	.150	〃	2	.011	.022
24	〃	.594	.228	.150	〃	4	.020	.080
25	〃	.228	.150	.134	〃	2	.005	.010
				笠石及柱裝飾石材 計			1.592 m³	50 t

仕上面積 (張石笠石及柱石材)

躯体 表面仕上 $\frac{1}{2}\times(4.293+2.756)\times 4.80 = 16.92$
合端仕上(3cm) $32@\frac{1}{2}\times(4.293+2.756)\times .03 = 3.38$
〃 $19@\ .03\times 4.80 = 2.74$
上段表面 $1@\ .08\times 2.80 = .22$
$23.26\times 2 = 46.52$ sq meters

合端中切仕上(背面ヲ除ク) $6.12\times\frac{20.5}{3} = 41.82\times 2 = 83.64$ sq meters

笠石及柱裝飾用石

笠石
- 表面仕上 .244 × 2.44 = .60
- 〃 .30 × 1.98 = .59
- 合端仕上(3cm) 10 @ .04 × .30 = .12
- 〃 (〃) .03 × (1.51+2.44) = .12
- 表面仕上 .394 × 1.03 = .41
- 〃 .30 × .54 = .16
- 合端仕上(3cm) .03 × 2.55 = .08
- 表面仕上 .2 × .3 × ½ = .03
- 合端仕上(3cm) .03 × .8 = .02

柱裝飾石
- 表面仕上 2 @ .85 × 1.70 = 2.89
- 〃 .228 × 1.48 = .34
- 合端(3cm) 4 @ .03 × 1.70 = .20
- 〃 (〃) 2 @ .03 × 1.48 = .09
- 〃 (〃) 28 @ .03 × .88 = .74
- 8 @ .03 × .23 = .06

$6.45\times 2 = 12.90$ m²

合端中切仕上(背面ヲ除ク)

柱裝飾石
- .27 × .48 = .13
- .36 × .78 = .28
- 2 @ .22 × 1.70 = .75
- 2 @ .12 × 1.48 = .36
- 28 @ .27 × .37 = 2.80
- 8 @ .12 × .23 = .22

笠石
- .27 × 1.98 = .53
- 10 @ .24 × .25 = .60
- .22 × 1.50 = .33
- .12 × 1.00 = .12
- 2 @ .08 = .16

$6.28\times 2 = 12.56$ m²

第四節 合端モルタル (配合 1:2)

張石 横目地 $(4.148+2.803)\div 2 = 3.476 \times .01 \times .22 \times 15 = .115$
竪 〃 $.235 \times .01 \times 4.89 \times 8.5 = .098$
柱石 $.75 \times .01 \times 1.40 = .011$
柱 $.30 \times .01 \times .40 \times 14 = .017$
〃 $.15 \times .01 \times .23 \times 44 = .001$
〃 $.15 \times .01 \times 1.65 \times 2 = .005$
〃 笠石 $.33 \times .01 \times .54 = .002$
片側 計 .249
両側 計 $2@.249 = 0.498$ m³

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

MADE BY M. Jinda DATE 5-11-22 FILE NO
CHECKED BY M. Kojima DATE 5-11-24 PAGE NO 55

CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture.

第貳章　基礎石　(Sheet no 8 參照)
第壹節　混凝土　(配合 1:3:6)

E部	(3.40+6.00)÷2 =	4.70 × .80 × 22.00	=	82.72 ✓
F部		6.00 × 1.20 × 22.00	=	158.40 ✓
				241.12 ✓ cub. meters.

第貳節　木杭
末口 21 糎長 6.50 米　生松丸太　102 本

第參節　栗石
6.60 × .30 × 22.60 = 44.75 cub. meters.

第參章　型枠　(Sheet no. 8 參照).
第壹節　橋脚躯体用

B部	.50 × 2.601 × 2	=	2.60 ✓		注意　張石ヲ先ヅ築造シ
D〃	4.89 × 18.152 × 2	=	177.5 ✓	上半	然ル後混凝土ヲ填充ス
〃	1.51 × 18.666 × 2	=	56.4 ✓	下半	計劃ニ依リ注ヲ張石背後ノ
〃	2.571 φ × 1.51	=	12.0 ✓	〃 角端	型枠ヲ見込マズ
	小計		✓ 248.5 m²		

第貳節　基礎部

側面	1.20 × 22.0 × 2	=	52.8 ✓
端面	1.20 × 6.0 × 2	=	14.4 ✓
	小計		67.2 m²
	型枠合計		✓ 315.7 sq. meters.

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CALCULATIONS FOR

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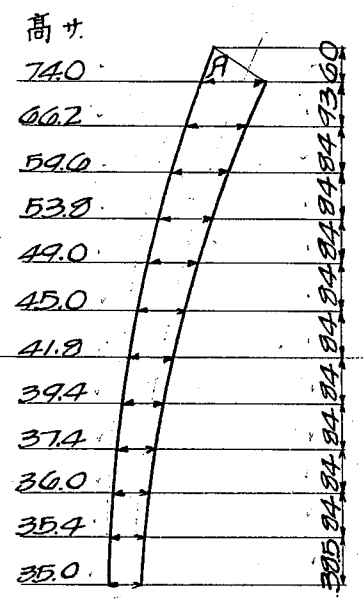
第参編 第壹節 拱環 (1径間分) 混凝土 (配合 1:2:4)

$\frac{1}{2} \times .740 \times .60 = .22$
$.701 \times .93 = .65$
$.629 \times .84 = .53$
$.567 \times \,〃 = .48$
$.514 \times \,〃 = .43$
$.470 \times \,〃 = .39$
$.434 \times \,〃 = .36$
$.406 \times \,〃 = .34$
$.384 \times \,〃 = .32$
$.367 \times \,〃 = .31$
$.357 \times \,〃 = .30$
$.352 \times .385 = .14$

側面型板 4.25 (×8 = 3400平方米)

4.47 × 2 = 8.94 sq.m.

Volume for one span. 8.94 × 7.50 × 2 = 134.10 cub. meters.



第貳節 鐵筋 (丸鋼)

Mark	Reqd. no. one rib	Size	Length	Unit wt	Piece weight	Total wt.	Remarks.
R1	30	2.5	10.00	3.85	38.50	1,155.00	longitudinal bars
R2	30	〃	8.70	〃	33.50	1,005.00	〃 〃
R3	60	〃	5.60	〃	21.56	1,293.60	〃 〃
R4	60	〃	6.30	〃	24.26	1455.60	〃 〃
R5	116	1.9	1.50	2.22	3.33	386.28	Extra bars 〃
R6	82	〃	8.05	〃	17.87	1465.34	transverse bars 〃
RS1×2	120	1.2	.85	.88	.75	90.00	Stirrups 〃
RS3×4	120	〃	.75	〃	.66	79.20	〃 〃
RS5-6	120	〃	.65	〃	.57	68.40	〃 〃
RS 7×8	120	〃	.60	〃	.53	63.60	〃 〃
RS9~13	270	〃	.55	〃	.48	129.60	〃 〃

total for one rib = 7,191.62
〃 〃 one span 2c 7191.62 = 14,383.24 kg
or 14.383 kg tons.

内 2.5φ... 9.818, 1.9φ... 3.704, 1.2φ... 0.862

第参節 人造洗出仕上

厚3糎仕上1割 A割 $\frac{1}{2}(.38+.74) \times .35 = .20$
face of ring $= 4.25$ 上記混凝土断面積表参照
$4.45 \times 4 = 17.80$ sq. m.

厚1.5粗仕上1割 extrados 巾 0.10 × 長 9.252 = .93
intrados 0.20 × 9.117 = 1.82
$2.75 \times 4 = 11.00$ sq. m

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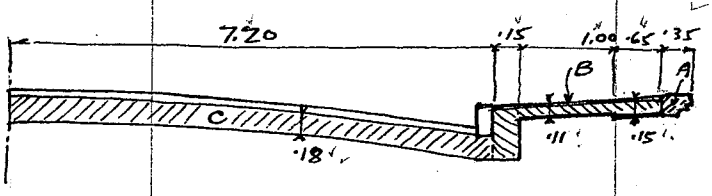
CALCULATIONS FOR

Materials of Jōgan Bashi for Kyoto Prefecture.

第四編 床版及横桁
第壹章 混凝土 (配合 1:2:4)
第壹節 床版



断面積
Coping A = 0.0620 sq.m

Sidewalk slab B .15 × .65 = .098
.11 × 1.00 = .110
.15 × .35 = .053
0.261 sq.m

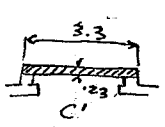
Roadway slab C .18 × 14.4 = 2.592 sq.m

Slab no 1. 2 - Required

A	2 @ .062 × 2.825	=	.35	
B	2 @ .261 × 3.511	=	1.83	
Bracket	1 @ ½(.68+1.10)×.15×1.45	=	.19	on abutment.
"	1 @ .35×.20×2.75	=	.19	
"	1 @ ½(.19+.60)×.15×1.45	=	.09	
"	1 @ .35×.20×1.60	=	.11	
less	2 @ .35×.15×.62	=	−.07	親柱ノ部
C	1 @ 2.592×3.511	=	9.10	
less	1 @ .20×.105×14.00	=	−.29	踏掛版
			11.50 × 2 =	23.00 cub.m

Slab no 2. 6 - Required

A	2 @ .062×12.52	=	1.55	
B	2 @ .261×12.52	=	6.54	
C	2 @ 2.592×3.511	=	18.20	
C'	1 @ .23×3.30×5.498	=	4.17	Crown fill ノ下床版
			30.46 × 6 =	182.76 cub.m



Slab no 3. 5 - Required

A	2 @ .062×6.71	=	.83	
B	2 @ .261×6.71	=	3.50	
C	1 @ 2.592×6.71	=	17.39	
brackets	4 @ .164×.30×.85	=	.17	under lamp posts.
"	2 @ .26×.195×1.02	=	.10	
			21.99 × 5 =	109.95 cub.m

全床版混凝土合計 315.71 cub.m

第貳節 横桁

BM1.	.23×.30×2.809	=	.19	×12本 =	2.28
BM2	.47×.30×2.809	=	.40	×29 =	11.60
BM3	.47×.30×2.809	=	.40		
	½@.15×.15×2.809	=	.03		
			.43	×12 =	5.16

全横桁混凝土合計 19.04 cub.m

58 page 17枚
5枚目

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CALCULATIONS FOR

Materials for Jōnan Bashi for Kyoto Prefecture

第貳章 鐵筋(丸鋼)
第壹節 床版鉄筋

	mark	Req'd no.	Size	Length	unit wt.	Piece weight	Total weight	Remarks
Slab no 1	F 1	78	16φ	3.60	1.57	5.65	440.70	Straight bars
	F 2	40	"	3.75	"	5.89	235.60	bent bars
	F 3	78	"	1.15	"	1.81	141.18	bond bars
	F 4	78	"	.80	"	1.26	98.28	" "
	F 5	18	12	4.50	.88	3.96	71.28	Straight bars "
	F 6	36	"	6.10	"	5.37	193.32	" "
	F 7	16	"	2.10	"	1.85	29.60	Sidewalk on abutment
	F 8A	15	"	1.85	"	1.63	24.45	" "
	F 8B	15	"	1.45	"	1.28	19.20	" "
	F 9	38	"	3.55	"	3.13	118.94	Sidewalk
	F 10	42	"	2.50	"	2.20	92.40	"
	F 11	22	"	2.25	"	1.98	43.56	"
	F 13	40	9	1.10	.49	.54	21.60	hoops
	F 14	42	12	.55	.88	.49	20.58	curb line
	F 15	42	"	.85	"	.75	31.50	"
				Summary for 2 – Slab no 1 =			1,582.19 × 2 =	3,164.38
Slab no 2	F 16	120	16	3.75	1.57	5.89	706.80	Straight bars
	F 17	60	"	3.95	"	6.20	372.00	bent bars
	F 3	176	"	1.15	"	1.81	318.56	bond bars
	F 5	32	12	4.50	.88	3.96	126.72	Straight bars "
	F 6	76	"	6.10	"	5.37	408.12	" "
	F 10	168	"	2.50	"	2.20	369.60	Sidewalk
	F 11	86	"	2.25	"	1.98	170.28	"
	F 12	4	"	1.20	"	1.06	4.24	" "
	F 13	168	9	1.10	.49	.54	90.72	hoops
	F 14	88	12	.55	.88	.49	43.12	curb line
	F 15	88	"	.85	"	.75	66.00	"
	F 18	76	"	4.00	"	3.52	267.52	Sidewalk
	F 19	38	"	5.80	"	5.10	193.80	" "
	F 20	26	16	4.20	1.57	6.59	171.34	bent bars
	F 21	14	12	6.90	.88	6.07	84.98	Straight bars
	F 22	36	16	4.10	1.57	6.44	231.84	" "
	F 23	20	"	4.30	"	6.75	135.00	bent bars
	F 24	15	"	4.05	"	6.36	95.40	Roadway
	F 25	15	"	3.65	"	5.73	85.95	"
				Summary for 6 – Slab no 2 =			3,941.99 × 6 =	23,651.94
Slab no 3	F 3	156	16	1.15	1.57	1.81	282.36	bond bars
	F 4	156	"	.80	"	1.26	196.56	" "
	F 10	90	12	2.50	.88	2.20	198.00	Sidewalk
	F 11	46	"	2.25	"	1.98	91.08	"
	F 12	4	"	1.20	"	1.06	4.24	" "
	F 13	90	9	1.10	.49	.54	48.60	hoops
	F 14	90	12	.55	.88	.49	44.10	Curb line
	F 15	90	"	.85	"	.75	67.50	"
	F 26	156	16	4.05	1.57	6.36	992.16	Straight bars
	F 27	80	"	4.20	"	6.59	527.20	bent bars
	F 28	8	12	1.25	.88	1.10	8.80	bracket on pier
	F 29	22	16	1.45	1.57	2.28	50.16	"
	F 30	18	12	1.55	.88	1.37	10.96	"
	F 5	34	"	4.50	"	3.96	134.64	transverse bars
	F 6	681	"	6.10	"	5.37	365.16	"
	F 31	8	"	1.10	"	.97	7.76	bracket on pier
	F 32	76	"	3.70	"	3.26	247.76	Sidewalk
				Summary for 5 – Slab no 3 =			3,277.04 × 5 =	16,385.20
				内 16φ ... 24.775		12.9φ 18.427		43,201.52 or 43.201 tons

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CALCULATIONS FOR

Materials of Jonan Baski for Kyoto Prefecture.

第貳節　橫桁鉄筋

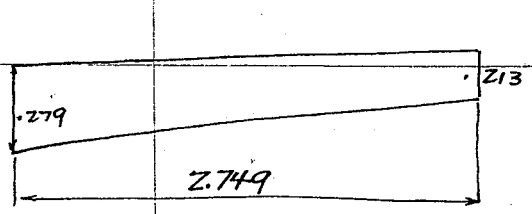
Mark	Reqd. no.	Size	Length	unit wt.	Piece wt.	Total wt.	Remarks.
Cross beam BM1		(12 req'd)					
B1	4	1.9 cmϕ	4.00 m	2.22 kg/m	8.88 kg	35.52 kg	Straight
B2	2	"	4.20	"	9.32	18.64	bent
B3	2	"	4.30	"	9.55	19.10	"
B4	4	"	2.10	"	4.66	18.64	bond
B6	20	1.2	1.50	.88 ✓	1.32	26.40	Stirrups
B12	2	1.9	4.00	2.22 ✓	8.88	17.76 ✓	Straight
						136.06	
Cross Beam BM2		(29 req'd.)					
B1	4	1.9	4.00	2.22	8.88	35.52	Straight
B4	4	"	2.10	"	4.66	18.64	bond
B5	2	"	4.50	"	9.99	19.98	bent
B7	14	1.2	1.90	.88	1.67	23.38	Stirrups
B12	2	1.9	4.00	2.22	8.88	17.76	Straight
						115.28	
Cross beam BM3		(12 req'd.)					
B1	7	1.9	4.00	2.22	8.88	62.16	Straight
B4	2	"	2.10	"	4.66	9.32	bond
B8	1	"	4.50	"	9.99	9.99	bent
B9	2	"	2.20	"	4.88	9.76	"
B10	14	1.2	1.50	.88	1.32	18.48	" Stirrups
B11	14	"	2.00	"	1.76	24.64	" "
B12	2	1.9	4.00	2.22	8.88	17.76	Straight
						152.11	

Summary

BM1	12	@	136.06	= 1,632.72
BM2	29	@	115.28	= 3,343.12
BM3	12	@	152.11	= 1,825.32
			6801.16 kg	
		or	6.801 kg tons.	

計 1.9ϕ --- 5.289
1.2ϕ --- 1.512

第參章　拱頂填充混凝土（配合 1:3:6）



2 @ $\frac{1}{2}$ (.279 + .213) × 2.749 × 5.55 = 7.50 cub.m.

for 6 spans. 6@2 × 7.50 = 90.00 ; Crown filling

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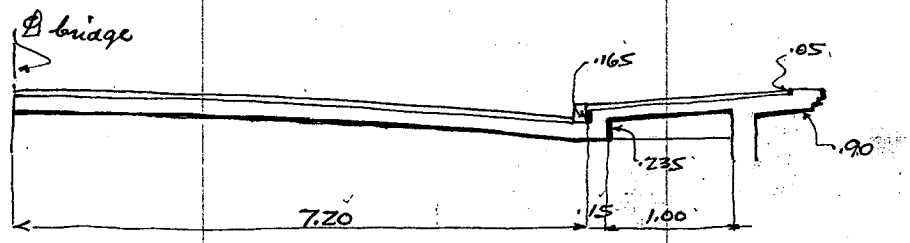
CALCULATIONS FOR

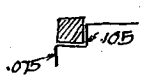
Materials of Jonan Bashi for Kyoto Prefecture.

第四章　型枠
第壹節　床版



Slab no.1. 2-required.

Coping and bracket	2 c .95 × 2.830	= 5.4	
sidewalk	2c 1.235 × 3.204	= 7.9	
longitudinal beam	2c .15 × 2.736	= .8	bottom.
roadway	1c 14.40 × 2.736	= 39.4	
bracket	½ × (.85+1.5) × 1.65	= 1.8	
〃	½ × (.30+.80) × 1.65	= .9	
〃	(1.65+1.35) × .175	= .5	
〃	(.3+1.65) × .175	= .3	
〃	.195 × 2.20	= .4	
〃	.195 × 1.40	= .3	
Curb line	.170 × 6.11	= 1.0	
〃	.170 × 5.16	= .9	
Side	.18 × 14.563	= 2.6	歩道 地覆 1ヶ所
	62.2 × 2	= 124.4 sq. meters.	



Slab no 2. 6-required.

Coping and bracket	2 c .95 × 12.53	= 23.8	
sidewalk and curb line	2 c 1.40 × 12.53	= 35.1	
road way	2 c 14.7 × 2.736	= 80.4	
inside faces (137=7½×18)	4 c (5.498+5.774)×.18	= 8.1	
roadway (〃)	2.7 × 5.498	= 14.8	
	162.2 × 6	= 973.2 sq. meters.	

Slab no 3. 5-required.

Coping	2 c .05 × 6.71	= .7	
Coping + bracket	2 c .90 × 6.11	= 11.0	
bracket	2 c .21 × 1.12	= .5	
〃	2 c .24 × .26	= .1	
〃	8 c .13 × .75	= .8	
〃	4 c .30 × .88	= 1.1	
Sidewalk curb line	2 c 1.40 × 6.72	= 18.8	
roadway	5.472 × 14.70	= 80.4	
	113.4 × 5	= 567.0 sq. meters	

全床版型枠合計 = 1,664.0 〃

第弐節　横桁

Cross beam BM1.	2 c .23 × 2.809	= 1.3	
	1 c .30 × 2.601	= .8	
	2.1 m² × 12 ヶ所	= 25.2	
Cross beam BM2 + BM3.	2 c .47 × 2.809	= 2.6	
	1 c .30 × 2.809	= .8	
	3.4 × 41	= 139.4	
		164.6 sq. meters.	

全横桁型枠合計 =

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CALCULATIONS FOR

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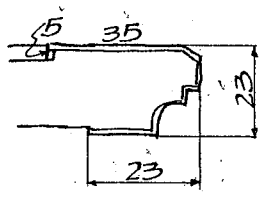
第五章　人造洗ち仕上

Slab no 1.　2 req'd.

2 × .86 × 2.825 = 4.86 × 2 = 9.72 sq.m.



Slab no 2　6 required.

2 × .86 × 12.52 = 21.53 × 6 = 129.18 sq.m

Slab no 3.　5 required

2 × .05 × 6.71	= .67	
2 × .81 × 6.11	= 9.90	
2 × .26 × 1.06	= .55	
2 × .23 × 1.12	= .52	
2 × .26 × .24	= .12	
4 × .13 × .85	= .44	
4 × .13 × .75	= .39	
4 × .30 × .58	= .70	
13.29 × 5	= 66.45 sq.m	
	205.35	

Handrail 1 底面差引スケ手面積

東柱	60 ㎝ c .30 × .37	= 6.66
電燈台	10 〃 c .60 × .84	= 5.04
bottom rail	12 〃 c .24 × 16.56	= 47.69
		− 59.39

差引　全床版人造洗ち仕上合計 = 145.96 sq.m

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CALCULATIONS FOR

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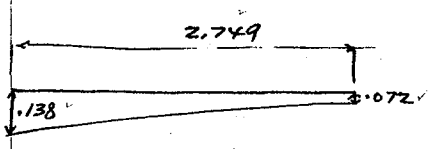
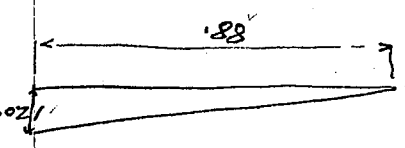
第五編　拱側壁.
第壹章　混凝土（配合 1:2:4）

内側拱側壁　（一径3分）.



4c ½ ·(.072+.138) ×.30 ×2.749 = 0.35 m3

中央部拱側壁



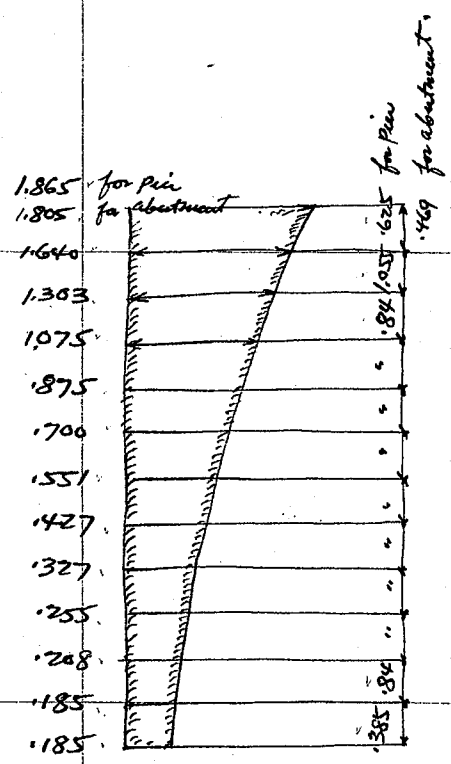
4c ½ ×.021 ×.15 ×.88 = 0.01 m3

外側拱側壁

橋台上＝アル部分　1.723 × .469 = .808 m2
橋脚上　〃　1.753 × .625 = 1.096 〃

各径3＝共通1部分.

1.472	×	1.055	= 1.553
1.189	×	.84	= .999
.975	×	.84	= .819
.788	×	.84	= .662
.626	×	.84	= .526
.489	×	.84	= .411
.377	×	.84	= .317
.291	×	.84	= .244
.232	×	.84	= .195
.197	×	.84	= .165
.185	×	.385	= .071
			5.962



両読径3

2c .808 × .30 =		.48
2c 1.096 × .30 =		.66
4c 5.962 × .30 =		7.15
.10 × .24 × 1.845 =		.04
2c .15 × .15 × .85 =		.04
		8.37 m3

両読以外径3.

4c 1.096 × .30 =		1.32
4c 5.962 × .30 =		7.15
2c .10 × .24 × 1.845 =		.09
2c .15 × .15 × .85 =		.04
		8.60 m3

全拱側壁混凝土合計

内側拱側壁	6c	.35 =	2.10
中央部 〃 〃	6c	.01 =	0.06
外側拱側壁（両読）	2c	8.37 =	16.74
〃 〃（両読以外）	4c	8.60 =	34.40
			53.30 Cub meter.

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MADE BY I. M. DATE 5-11-23 FILE NO
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CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture.

第貳章 鐵筋(丸鋼)

中間拱側壁

Mark	Req'd no.	Size.	Length	Unit wt.	Piece wt.	Total wt.	Remarks.
WL 1	45	1.6$^{\phi}$	1.90	1.57	2.98	134.10	inner wall.
WL 2	25	"	1.10	"	1.73	43.25	"
WL 3	36	1.2$^{\phi}$	1.50	0.88	1.32	47.52	stab bars
WL 4	12	"	1.80	"	1.58	18.96	vertical bars
WL 5	20	"	1.50	"	1.32	26.40	"
WL 6	24	"	1.20	"	1.06	25.44	"
WL 7	34	"	1.05	"	.93	31.62	"
WL 8	4	"	6.80	"	5.98	23.92	horizontal bars
WL 9	4	"	3.20	"	2.82	11.28	"
WL 10	23	"	1.00	"	.88	20.24	middle wall
WL 11	40	"	1.50	"	1.32	52.80	"
WL 18	4	"	1.30	"	1.14	4.56	corner

440.09 × 2 = 880.18 Kgs

橋脚上側壁

WL 13	14	1.2$^{\phi}$	1.70	0.88	1.50	21.00	vertical & horizontal bars
WL 14	12	"	1.60	"	1.41	16.92	vertical bars
WL 15	12	"	1.30	"	1.14	13.08	"
WL 16	14	"	3.70	"	3.26	45.64	horizontal bars
WL 17	4	"	3.30	"	2.90	11.60	"
WL 12	12	"	2.05	"	1.81	21.72	vertical bars

130.56 × 2 = 261.12 Kgs

両詰側壁

WL 13	8	1.2$^{\phi}$	1.70	0.88	1.50	12.00	vertical & horizontal bars
WL 14	6	"	1.60	"	1.41	8.46	vertical bars
WL 15	6	"	1.30	"	1.14	6.84	"
WL 16	6	"	3.70	"	3.26	19.56	horizontal bars
WL 17	2	"	3.30	"	2.90	5.80	"

52.66 × 2 = 105.32 Kgs

全拱側壁 鐵筋合計

中間拱側壁	6 @ 880.18	=	5,281.08
橋脚上側壁	5 @ 261.12	=	1,305.60
両詰側壁	2 @ 105.32	=	210.64
			6,797.32 Kg or 6.797 Kg tons

内
1.6$^{\phi}$ ------- 2.128
1.2$^{\phi}$ ------- 4.669

第參章 型枠

内側拱側壁 8 @ ½ (.097 + .163) × 2.749 = 2.90
中央部拱側壁 8 @ ½ × .021 × .88 = .1
外側拱側壁

外側径間(両詰)

	4 @	.808	= 3.2	
	8 @	5.962	= 47.7	
less	4 @	.312 × 1.303	= -1.6	支壁、分差引
"	4 @	.312 × .875	= -1.1	"
"	4 @	.312 × .551	= -.7	"
"	4 @	.312 × .327	= -.4	"
	4 @	1.096	= 4.4	
less	1 @	.312 × 1.845	= -.6	
	2 @	.10 × 1.845	= .4	
			51.3	

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両詰以外径間

	8 @	1.096	= 8.8	
	8 @	5.962	= 47.7	
less	2 @	.312 × 1.845	= − 1.2	支壁1ヶ所引
〃	4 @	.312 × 1.303	= − 1.6	
〃	4 @	.312 × .875	= − 1.1	
〃	4 @	.312 × .551	= − .7	
〃	4 @	.312 × .327	= − .4	
	4 @	.10 × 1.845	= .7	
			52.2	

全拱側壁型枠合計

内側拱側壁	6 @	2.9	= 17.4
中央部 〃	6 @	.1	= .6
外側 〃 (両詰)	2 @	51.3	= 102.6
〃 〃 (両詰以外)	4 @	52.2	= 208.8
			329.4 sq. meters

第四章 人造洗出仕上

両詰径間

	2 @	.808	= 1.62	
	4 @	5.962	= 23.85	
	2 @	1.096	= 2.19	
less	1 @	.84 × 1.745	= − 1.47	張石1部
			26.19	

両詰以外径間

4 @	~~8~~	1.096	= 4.38
4 @		5.962	= 23.85
2 @	.84	× 1.745	= − 2.93
			25.30

全拱側壁人造仕上合計

両詰径間	2 @	26.19	= 52.38
両詰以外径間	4 @	25.30	= 101.20
			153.58 sq meters

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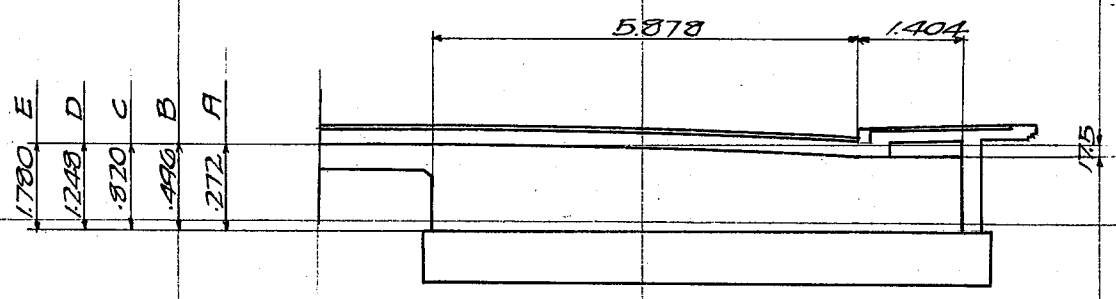
CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第六編　支壁
第壹章　混凝土．(配合 1:2:4)



A 壁　Σc .097 × .30 × 7.282 = .42
Σc ⅔ × .175 × .3 × 5.878 = .141
.83 × 12 = 9.96

B 壁　Σc .321 × .30 × 7.282 = 1.40
.41
1.81 × 12 = 21.72

C 壁　Σc .645 × .30 × 7.282 = 2.82
.141
1@ .15 × .15 × 5878 = .13
3.36 × 12 = 40.32

D 壁　Σc 1.073 × .30 × 7.282 = 4.69
.41
5.10 × 12 = 61.20

E 壁　Σc 1.605 × .30 × 7.282 = 7.01
141
7.42 × 5 = 37.10

全支壁 混凝土合計 = 170.30 cub metres

第貳章　鐵筋 (丸鋼)

Transverse spandrel walls on span.

mark	Req'd. no.	Size	Length	Unit wt.	Piece wt.	Total wt.	Remarks.	
WT.1	14	1.6 cmϕ	1.70 m	1.57 kg/m	2.67	37.38	vertical	⌐
WT2	16	〃	1.80	〃	2.83	45.28	〃	〃
WT3	7	〃	1.00	〃	1.57	10.99	〃	⌒
WT4	8	〃	1.10	〃	1.73	13.84	〃	〃
WT5	22	1.2	1.15	.88	1.02	22.44	〃	〃
WT6	26	〃	1.20	〃	1.06	27.56	〃	〃
WT7	11	〃	1.55	〃	1.37	15.07	〃	〃
WT8	13	〃	1.65	〃	1.46	18.98	〃	〃
WT9	22	〃	1.50	〃	1.32	29.04	〃	〃
WT10.	26	〃	1.60	〃	1.41	36.66	〃	〃
WT11	64	〃	1.40	〃	1.23	78.72	〃	〃
WT12	6	〃	0.70	〃	.62	3.72	〃	〃
WT13	8	〃	0.95	〃	.84	6.72	〃	〃
WT14	8	〃	1.30	〃	1.14	9.12	〃	〃
WT15	12	〃	7.80	〃	6.86	82.32	horizontal	〃
WT16	11	〃	1.75	〃	1.54	16.94	vertical	⌐
WT17	13	〃	1.85	〃	1.63	21.19	〃	〃
WT18	24	〃	1.00	〃	.88	21.12	fillet	⌒
WT19	4	〃	.95	〃	.84	3.36	〃	〃
						500.45 × 4 = 2001.80 kg		for 1 span

16φ 0.430　12φ 1.572

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CALCULATIONS FOR

Materials for Jōnan Bashi for Kyoto Prefecture.

Transverse spandrel wall or pier.

WT1	8	1.2φ	1.70m	.88	1.50	12.00	vertical
WT2	22	〃	1.80	〃	1.58	34.76	〃 〃
WT20	26	〃	1.90	〃	1.67	43.42	〃 〃
WT15	8	〃	7.80	〃	6.86	54.88	horizontal 〃
						145.06 × 2 = 290.12	for one wall.

全支壁 鐵筋合計

径7上、モ	6 @ 2001.80	= 12,010.80	注意 桁台上ノ踏掛石
桁脚上、モ	5 @ 290.12	= 1,450.60	下ノ支壁ヽ桁台ニ見込ム
		13,461.40 kg	内 1.6φ ---- 2.580
		or 13.461 kg tons.	1.2φ ---- 10.882

第参章 型枠

A壁	4 @ .097 × 6.814	= 2.6	
	4 @ 2/3 × .175 × 5.566	= 2.6	
		5.2 × 12 =	62.4
B壁	4 @ .321 × 7.282	= 9.4	
	4 @ 2/3 × .175 × 5.878	= 2.7	
		12.1 × 12 =	145.2
C壁	4 @ .645 × 7.282	= 18.8	
		2.7	
	2 @ .312 × .35	= .2	
		21.7 × 12 =	260.4
D壁	4 @ 1.073 × 7.282	= 31.3	
		2.7	
	2 @ .312 × .778	= .5	
		34.5 × 12 =	414.0
E壁	4 @ 1.605 × 7.282	= 46.8	
		2.7	
	2 @ .312 × 1.31	= .8	
		50.3 × 5 =	251.5
	全支壁 型枠合計		1133.5 sq. meters

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MADE BY M. Kojima DATE 5-12-1 FILE NO
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CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第七編　排水工　(Sheet no. 6 参照)
第一節　橋脚排水工（10箇所）
第一款　鋳鉄蓋

15 × 15.6 × 2 − .8 × 11 × 7 × 2 = 345 cm³
1個当り重量　345 × .00725 = 2.501 Kgs
合計重量　10 @ 2.501 = 25.010 Kgs

第二款　瓦斯管　3"ϕ × 22m ---------- 10本
1本重量　7.575 × 22 × 1.499 = 24.7990 Kgs
合計重量　10 @ 24.799 = 247.990 Kgs

第二節　橋面排水工（24箇所）
第一款　鋳鉄
a 蓋　35 × 18 × 2.5 = 1575
12 @ 2 × 9 × 2.5 = 540
1035 cm³

1個重量　.00725 × 1035 = 7.504

b 漏斗　2.5 × (33 + 16.7) × 2 × 2 = 497
1.2 × {(33 + 16.7) × 2 × + 11.2 × π} ÷ 2/3 × 1.5 = 1614.96
11.2 × π × 1.2 × 5 = 211.12
2323.08 cm³

1個重量　.00725 × 2,323.08 = 16.842

c パイプ　11 × π × 1 × 1.31 = 4527.05 cm³

1個重量　.00725 × 4,527.05 = 32.821
1組重量　57.167 Kgs

合計重量　24 @ 57.167 = 1,372.01 瓩

第二款　排水枡縁混凝土 (wire mesh ヲ含ム)
配合 1:1:2　表面仕上
(.065 + .055) ÷ 2 × .12 × (.060 + .018 × 2) = .0069 m³

合計体積　24 @ .0069 = 0.166 立方米

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MADE BY S. Matano DATE 5-11-29 FILE NO
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CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第八編　高欄材料計算 (Sheet no. 12 参照)

第一章　親柱　一基当り材料

第一節　石材立積 (花崗石)

石材立積計算表 (立積ハ小数以下三位迄以下切捨トス)

	長(米)	巾(米)	厚(米)	一個当り立積(立方米)	数量(個)	立積(立方米)
縁石	.77	.26	.21	.042	1	.042
〃	1.38	.26	.21	.075	1	.075
第一列(下ヨリ)	1.41	.45	.45	.286	4	1.144
〃	1.41	.30	.30	.127	4	.508
第二列	.15	.33	.33	.016	4	.064
〃	1.30	.30	.30	.117	4	.468
〃	1.55	.25	.25	.097	4	.388
〃	.40	.30	.30	.036	4	.144
第三列	.90	.25	.30	.068	2	.136
〃	.90	.25	.70	.158	2	.316
						3.285 立米

第二節　石材仕上計算

第一款　表面及合端仕上計算 (小叩仕上)

縁石	.47 × 1.97 =	.93	(表面)	
〃	.18 × .26 =	.05	〃	
〃	.03 × 6.00 =	.20	(合端)	
第一列	4 e 1.06 × 1.41 =	5.98	(表面)	
〃	4 e .12 × .78 =	.37	〃	
〃	4 e .30 × 1.41 =	1.69	〃	
〃	4 e .03 × 4.00 =	.48	(合端)	
〃	4 e .03 × 3.42 =	.41	〃	
第二列	4 e .20 × .66 =	.53	〃	
〃	4 e .50 × 1.55 =	3.10	(表面)	
〃	4 e .54 × 1.70 =	3.67	〃	
〃	4 e .30 × .12 =	.14	〃	
〃	4 e .05 × .50 =	.10	〃	
〃	4 e .03 × 1.40 =	.17	〃	
〃	4 e .03 × 3.95 =	.47	(合端)	
〃	4 e .03 × 5.40 =	.65	〃	
第三列	2 e 1.30 × .90 =	1.98	〃	
〃	2 e .21 × .70 =	.29	(表面)	
〃	2 e .30 × .90 =	.54	〃	
〃	2 e .30 × .30 =	.18	〃	
〃	2 e .03 × 3.60 =	.22	(合端)	
〃	2 e .03 × 3.00 =	.18	〃	
		22.33 平方米		

第二款　中切仕上計算

縁石	2 e .15 × .18 =	.05	
〃	2 e .23 × .18 =	.08	
第一列	8 e .27 × 1.38 =	2.98	
〃	8 e .27 × 1.38 =	2.98	
〃	4 e .27 × .30 =	.32	
〃	4 e .30 × .30 =	.36	
第二列	4 e .30 × .30 =	.36	
〃	8 e .22 × .22 =	.39	
〃	8 e .22 × 1.70 =	2.99	
〃	8 e .15 × 1.70 =	2.04	
〃	12 e .27 × .30 =	.97	

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CALCULATIONS FOR

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第三列	4 @ .19 × .60 =	.46	
〃	2 @ .15 × .25 =	.08	
〃	8 @ .17 × .90 =	1.22	
		1.528 平方米	

第三節　混凝土（配合 1:3:6）(Sheet no. 12 参照)
.40 × .4025 × 4.00 = .694 立方米

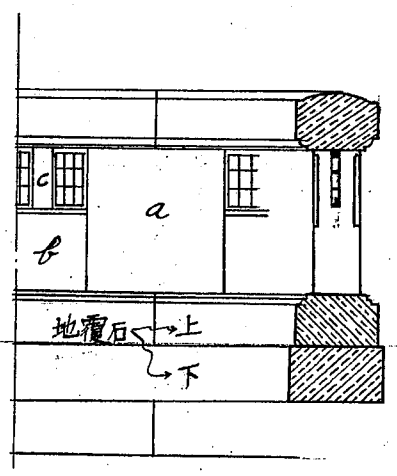
第四節　橋名鈑 (Sheet no. 12 参照)
青銅製橋名又ハ竣功年月日鈑取付金具共　1枚

第五節　電灯 (Sheet no. 12 参照)
青銅金具ランプ・ガラス及附属品共　上部　1箇
仝　下部　4箇
親柱内部配線用ガスパイプ　1式

第貳章　袖高欄（壹組当り材料 上流長サ3.5米下流長サ6.5米ニテ壹組）
第一節　石材立積（花崗石）
石材立積計算表

長サ三五米ノ分

石材名稱	長(米)	巾(米)	厚(米)	一個当立積	数量(個)	立積(立方米)
笠石（円形）	.60	.22	.15	.020	1	.020
〃　〃	1.00	.26	.15	.039	3	.117
〃	.80	.20	.15	.024	1	.024
a （円形）	.58	.53	.14	.043	4	.172
b （円形）	.38	.54	.14	.029	3	.087
b	.38	.48	.12	.022	1	.022
a（親柱寄リ）	.58	.327	.12	.023	1	.023
a（袖柱寄リ）	.58	.12	.085	.006	1	.006
c （円形）	.20	.115	.095	.002	3	.006
c	.20	.11	.09	.002	1	.002
地覆石（上部）（円形）	.60	.22	.12	.016	1	.016
〃　〃　〃	1.00	.26	.12	.031	3	.093
〃　〃	.80	.20	.12	.019	1	.019
地覆石（下部）（円形）	1.09	.30	.16	.052	1	.052
〃　〃　〃	1.00	.29	.16	.046	3	.138
〃　〃　〃	.325	.24	.16	.012	1	.012
袖柱	1.36	.60	.47	.384	1	.384
						1.193 立方米



長サ六、五米ノ分

石材名稱	長(米)	巾(米)	厚(米)	一個当立積	数量(個)	立積(立方米)
笠石（円形）	.60	.22	.15	.020	1	.020
〃　〃	1.00	.26	.15	.039	3	.117
〃	.95	.20	.15	.029	4	.116
a （円形）	.58	.53	.14	.043	4	.172
a （親柱寄リ）	.58	.327	.12	.023	1	.023
a （袖柱寄リ）	.58	.235	.12	.016	1	.016
a	.58	.47	.12	.033	3	.099
b （円形）	.38	.54	.14	.029	3	.087
b	.38	.48	.12	.022	4	.088
c （円形）	.20	.115	.095	.002	3	.006
c	.20	.11	.09	.002	4	.008
地覆石（上部）（円形）	.60	.22	.12	.016	1	.016
〃　〃　〃	1.00	.26	.12	.031	3	.093
〃　〃	.95	.20	.12	.023	4	.092

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MADE BY S. Matano DATE 5-12-2 FILE NO
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CALCULATIONS FOR

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地覆石(下部)(円形)	1.09	.30	.16	.052	1	.052	
〃 〃 〃	1.0	.29	.16	.046	2	.092	
〃 〃 〃	.98	.27	.16	.042	1	.042	
〃 〃	.95	.24	.16	.036	3	.108	
〃 〃	.475	.24	.16	.018	1	.018	
袖 柱	1.36	.60	.47	.384	1	.384	
							1.649 立方米

石材立積合計

長 35m 分 1.193
〃 6.5m 分 1.649
2.842 立方米

第貳節 石材仕上計算

第一款 表面及合端仕上計算(小叩仕上)

長サ三.五米分

笠石		.50 ×	4.212	=	2.44	表面
窓内面	9 @	.10 ×	.75	=	.68	〃
a	2 @	.58 ×	2.107	=	2.44	〃
b	2 @	.38 ×	2.105	=	1.60	〃
c	8 @	.11 ×	.20	=	.18	〃
袖 柱	2 @	1.07 ×	1.36	=	2.91	〃
〃		.47 ×	.60	=	.28	〃
地覆石		.68 ×	4.212	=	2.86	〃
笠石	10 @	.03 ×	.58	=	.17	合端
窓内面	9 @	.02 ×	.77	=	.14	〃
a	4 @	.03 ×	3.64	=	.44	〃
〃(親柱寄り)		.03 ×	3.00	=	.09	〃
〃(袖柱寄り)		.03 ×	2.40	=	.07	〃
b	4 @	.03 ×	2.60	=	.31	〃
c	16 @	.03 ×	.28	=	.13	〃
地覆石	10 @	.03 ×	.30	=	.09	〃
〃	10 @	.03 ×	.35	=	.11	〃
〃	6 @	.03 ×	4.212	=	.76	〃
笠石		.03 ×	4.814	=	.14	〃
					15.94	平方米

長サ六.五米分

笠石		.58 ×	7.212	=	4.18	表面
窓内面	15 @	.10 ×	.75	=	1.13	〃
a	2 @	.58 ×	3.667	=	4.25	〃
b	2 @	.38 ×	3.545	=	2.69	〃
c	14 @	.11 ×	.20	=	.31	〃
袖 柱	2 @	1.07 ×	1.36	=	2.91	〃
〃		.47 ×	.60	=	.28	〃
地覆石		.68 ×	7.212	=	4.90	〃
笠石	16 @	.03 ×	.58	=	.28	合端
窓内面	15 @	.02 ×	.77	=	.23	〃
a	7 @	.03 ×	3.64	=	.76	〃
〃(親柱寄り)		.03 ×	3.00	=	.09	〃
〃(袖柱寄り)		.03 ×	3.00	=	.09	〃
b	7 @	.03 ×	2.60	=	.55	〃
c	28 @	.03 ×	.28	=	.24	〃
地覆石	16 @	.03 ×	.66	=	.32	〃
〃	6 @	.03 ×	7.212	=	1.30	〃
笠石(下端)		.03 ×	8.30	=	.25	〃
					24.76	平方米

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CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

石材仕上合計

長サ 3.5m 1分　　15.84
〃 6.5m 1分　　24.76
40.60 平方米

第貳款　合端中切仕上計算　(Sheet no. 12 参照)

長サ三五米1分

笠石		10	@ .08 × .13	=	.10
〃			.06 × 1.07	=	.11
〃			.03 × .20	=	.01
a		4	@ .06 × 1.58	=	.38
a (親柱寄り)			.06 × 1.27	=	.08
〃 (袖柱寄り)			.06 × 1.07	=	.06
b		4	@ .06 × 1.18	=	.28
〃		4	@ .03 × .05	=	.01
c		4	@ .03 × .10	=	.01
地覆石			.06 × 42.12	=	.25
〃		2	@ .14 × 42.12	=	1.18
〃		10	@ .18 × .25	=	.45
袖柱下面周					.095
					30.15 平方米

長サ六五米1分

笠石		16	@ .08 × .13	=	.17
〃			.06 × 3.20	=	.19
〃			.03 × .35	=	.01
a		7	@ .06 × 1.58	=	.66
a (親柱寄り)			.06 × 1.27	=	.08
〃 (袖柱寄り)			.06 × 1.37	=	.08
b		7	@ .06 × 1.18	=	.50
b		7	@ .03 × .05	=	.01
c		7	@ .03 × .10	=	.02
地覆石			.06 × 72.12	=	.43
〃		2	@ .14 × 72.12	=	2.02
〃		16	@ .18 × .25	=	.72
袖柱下面周					.095
					49.95 平方米

中切仕上合計

長サ 3.5m 1分　　30.15
〃 6.5m 1分　　49.95
80.00 平方米

第三節　鋳鉄格子及装飾　(Sheet no. 12 参照)

鋳鉄格子箱込共　24枚
青銅製装飾鈑箱込共　4枚

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MADE BY S. Matano DATE 5-12-2 FILE NO
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CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第三章　橋上高欄両側分(全部)材料計算 (Sheet no. 13 参照)

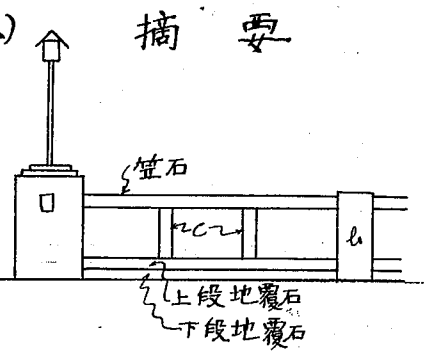
第一節　石材立積 (花崗石)

石材立積計算表

石材名稱	長(米)	巾(米)	厚(米)	箇当り立積	数量	立積(立方米)
笠石	2.78	.20	.15	.083	72	5.976
柱 (a)	1.23	.04	.60	.620	10	6.200
間柱(b)	1.08	.37	.30	.120	60	7.200
小間柱石(c)	.60	.18	.15	.016	144	2.304
上段地覆石	.995	.20	.12	.021	144	3.024
〃	.97	.20	.12	.023	72	1.656
下段地覆石	1.38	.24	.16	.053	144	7.632

33.992 立方米

摘　要



第二節　石材仕上計算

第一款　表面及合端(3㎝)仕上計算　(小叩仕上)

笠石及地覆石	72	@	1.14	2.76	= 226.54	表面
〃	72	@	.36	2.46	= 63.76	〃
柱 a	10	@	.84	3.06	= 25.70	〃
〃 〃	10	@	.60	2.46	= 14.76	〃
間柱 b	60	@	.37	2.46	= 54.61	〃
〃 〃	60	@	.30	2.16	= 38.88	〃
小間柱石(c)	144	@	.66	.58	= 55.12	〃
笠石及地覆石小口	72	@	.03	4.88	= 10.54	合端
b伸縮目地面	48	@	.30	1.08	= 15.55	〃
c	72	@	.03	4.90	= 10.37	〃

515.03 平方米

第二款　合端中切仕上計算

笠石及地覆石小口	72	@	.15	1.26	= 13.61	
c	72	@	.08	.96	= 5.53	
上段地覆石	72	@	.14	2.76	= 27.82	

46.96 平方米

第三節　鋳鉄格子 (Sheet no. 13 参照)

鋳鉄格子　216 枚

第一款　高欄鋳鉄格子　重量　1枚分

立積　外縁　27.6 × 3 × 3 = 2,484
模様　49.7 × 1.5 × 3 = 2,192
4,676 cm³

重量　.00725 × 4,676 = 33.901 ㎏

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Materials for Jōnan Bashi for Kyoto Prefecture

第二款　高欄鋳鉄格子　ペンキ塗面積(全表面) 1枚分

表面	外縁(表裏)	.276 × .03 × 2	= .1656
〃	(外縁)	.288 × .03	= .0864
〃	(内縁)	.249 × .03	= .0747
中横桟延長		.79 × 2 + .655 + .542 = 2.757	4.97
〃 縦〃〃		.125 × 3 + .355 × 4 + .16 × 2 = 2.115	
面積(表裏)		4.97 × .015 × 2	= .1491
〃	(内外)	(4.97 × 2 − .015 × 52) × .03	= .2688
			.741 平方米

第四節　青銅製飾鋲 (Sheet No. 13 参照)
青銅製飾鋲　20 枚

第五節　電灯 (Sheet No. 13 参照)
青銅金具台付ランプ、ガラス及附属品共　10 基

注意 L 柱 / 伸縮目地用エラスタイトハ 76 頁床版第四節ニ記載ス

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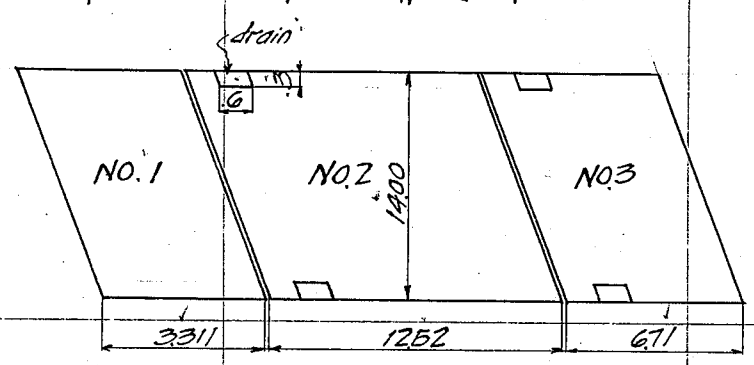
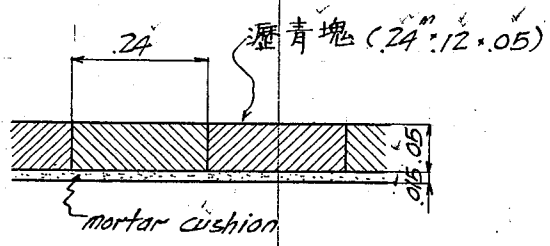
MADE BY M. Kojima DATE 5-11-28 FILE NO
CHECKED BY S. Matano DATE 5-12-3 PAGE NO 74

CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第九編　橋面鋪裝　(Sheet no. 5 参照)
第一節　瀝青塊鋪裝(車道)





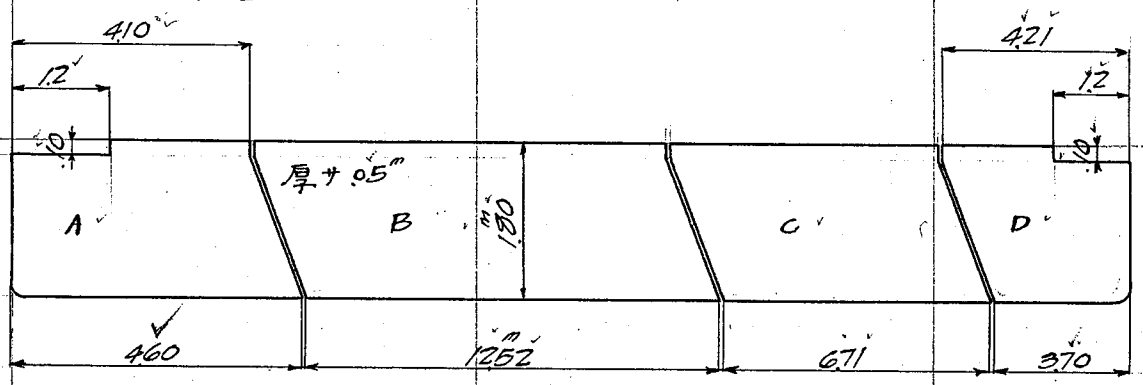
面積 NO.1 3311 × 1400 = 464 m²

NO.2 1252 × 1400 = 1753
less .6 × .3 × 2 = − .4
1749 m²

NO.3 671 × 1400 = 939
− 4
935 m²

面積合計 NO.1 2 @ 464 = 928
NO.2 6 @ 1749 = 10494
NO.3 5 @ 935 = 4675
16097 平方米

第二節　混凝土鋪裝 (歩道)
配合 1:1:2



A　½ × (460 + 410) × 180 = 783 m²
less 1.2 × 0.1 = .12
771

B　1252 × 180 = 2254

C　671 × 180 = 1208

D　½ × (421 + 370) × 180 = 712
less = − .12
700

面積合計
A　2 @ 771 = 1542
B　12 @ 2254 = 27048
C　10 @ 1208 = 12080
D　2 @ 700 = 1400
42070 平方米

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MADE BY M.K DATE 5-12-2 FILE NO
CHECKED BY I.M. DATE 5-12-3 PAGE NO 75

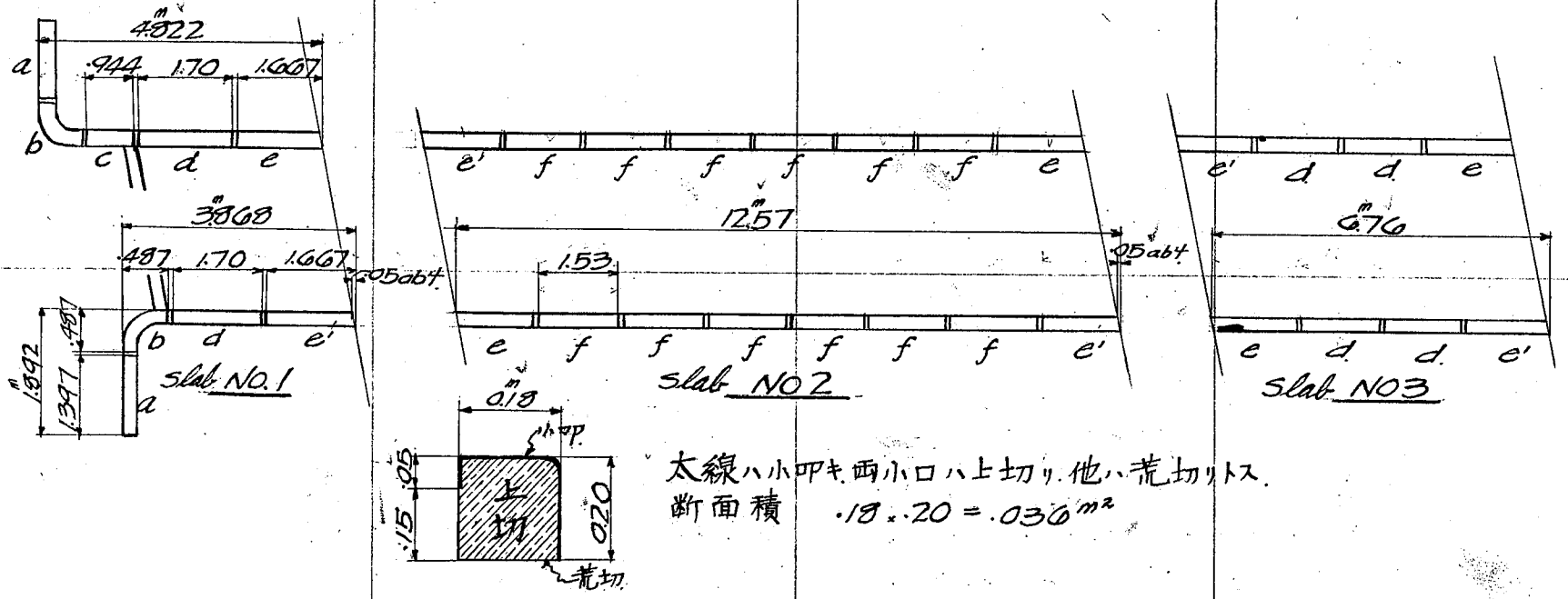
CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第參節 石 材

第壹款 歩車道境界石 (花崗石)

1. Slab NO.1. 目地 8mm. obt.



太線ハ小叩キ西小口ハ上切リ.他ハ荒切リトス.
断面積 .18×.20 = .036 m²

石材体積

Slab NO1. 2 Reqd.

名称	断面積(m²)	長(m)	員数	1個当体積(m³)	合計体積(m³)
a	.036	1.397	2	.050	.100
b	.35×.20	.700	2	.049	.098
c	.036	.944	1	.034	.034
d	〃	1.700	2	.061	.122
e&e'	〃	1.667	2	.060	.120
					.474×2 = .948 m³

Slab NO2. 6 Reqd.

e&e'			4	.060	.240
f	.036	1.530	12	.055	.660
					.900×6 = 5.400 m³

Slab NO3. 5 Reqd.

c&e'			4	.060	.240
d.			4	.061	.244
					.484×5 = 2.42

合計体積 8.768 m³

加工面積

表面及合端3cm仕上 小叩

Slab NO1. (.18+.05+.20)×6.682+5.736 = 5.340×2 = 10.68
〃 NO2 (〃 〃 〃)×12.464×2 = 10.719×6 = 64.31
〃 NO3. (〃 〃 〃)×6.686×2 = 5.750×5 = 28.75
103.74 m²

合端仕上. 上切

Slab NO1. .18×.20×18 = .648×2 = 1.30
〃 NO2. 〃 〃 32 = 1.152×6 = 6.91
〃 NO3. 〃 〃 16 = .576×5 = 2.88
11.09 m²

裏面仕上. 荒切

Slab NO1. (.15+.18)×(6.682+5.736) = 4.098×2 = 8.20
〃 NO2. (〃 〃)×12.464×2 = 8.226×6 = 49.36
〃 NO3. (〃 〃)×6.686×2 = 4.413×5 = 22.07
79.63 m²

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MADE BY M. Kojima DATE 5-12-2 FILE NO
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CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecutre

据付モルタル (1:3)
下敷モルタル及背面厚サ 0.02 接手用モルタル厚サ 0.008
境界石延長

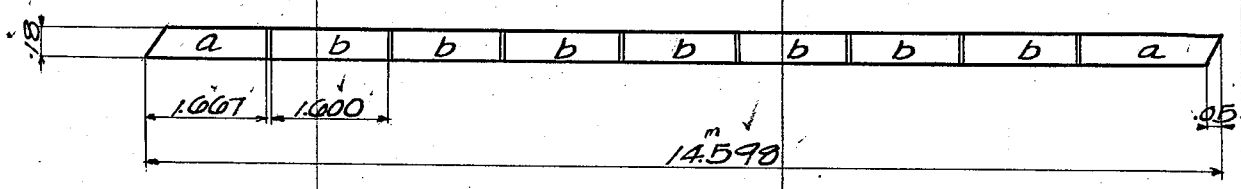
4.822 + 3.868 + 1.720 × 2	= 12.13 × 2	= 24.26
12.52 × 2	= 25.04 × 6	= 150.24
6.71 × 2	= 13.42 × 5	= 67.10
		241.60m

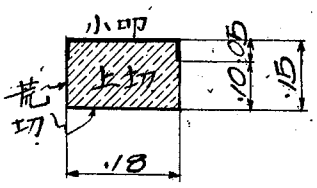
接手全数 (5+4)×2+7×2×6+3×2×5 = 132 個所

モルタル体積

.02 × (.20+.20) × 241.60	= 1.933 m³	
.18 × .20 × .008 × 132	= .038	
	1.971 m³	

第弐款 踏掛石 (花崗石) 両詰分 目地 8mm (Sheet No 5. 参照)





断面
太線ハ小叩キ両小口ハ上切リ
他ハ荒切リトス.

石材体積

a.	.18 × .15 × 1.607 = .045 × 4	= .180
b.	" " × 1.600 = .043 × 14	= .602
		.782 m³

加工面積

小叩	(.18+.05+.05) × 14.484 × 2	= 8.111
上切	.18 × .15 × .18 × 2	= .972
荒切	(.18+.10+.10) × 14.484 × 2	= 11.008

据付ケモルタル (1:3)
下敷モルタル及ビ横面 厚サ 0.02 接手用モルタル 厚サ .008
体積

.02 × (.108+.18) × 14.564 × 2	= .168 m³
.008 × .18 × .15 × 10 × 2	.004
	.172 m³

第四節 伸縮用 Elastite

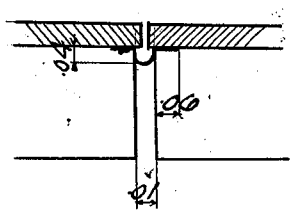
第壹款 床版接手 12 箇所 (厚サ 3/8") (Sheet no.5 参照)
平均高ハ 0.15 トス.
面積 .15 × 10.02 × 12 = 28.836 m².

第弐款 拱側壁両側 24 箇所 (厚サ 3/8") (Sheet no.7 参照)
巾 .47 高サ .9
面積 .47 × .9 × 24 = 10.152 m²

第参款 高欄接手 24 箇所 (厚サ 3/8") (Sheet no.13 参照)
面積 .30 × .105 × 24 = 7.56 m²
面積総合計 46.548 m²

第五節 伸縮用接手金物 (銅鈑) (Sheet no.5 参照).
歩道 24 箇所
延巾 .21 一本ノ長サ 2.13
単位重量 4.882 kg/m

合計面積 .21 × 2.13 × 24 = 10.735 m²
重量 4.882 × 10.735 = 52.408 kgs



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CALCULATIONS FOR 鉄筋コンクリート造 { arch influence surface / 三等拱影響面 / アーチ系

MADE BY M. Iwida DATE 5-10-34 FILE NO
CHECKED BY DATE PAGE NO 1

25 mm ϕ bar, 25 cm c to c = 1963 × 2 = 39.26 on both sides

Design of arch ring — 1 meter strip — Reinforcements $= \frac{39.26 \times 14}{10000} = .055$

	d	d^3	$I_c = \frac{1}{12} d^3$	$(\frac{d}{2} - .045)^2$		I_s	$I_c + I_s$
Crown 0	.350	.0429	.00357	.0169	× .055	.00093	.00450
1	.351	.0432	.00360	.0170	× "	.00094	.00454
2	.356	.0451	.00376	.0177	× "	.00097	.00473
3	.367	.0494	.00412	.0192	× "	.00106	.00519
4	.385	.0571	.00476	.0218	× "	.00120	.00596
5	.405	.0664	.00553	.0248	× "	.00136	.00699
6	.434	.0817	.00681	.0296	× "	.00163	.00844
7	.471	.1045	.00871	.0363	× "	.00200	.01071
8	.515	.1366	.01138	.0452	× "	.00249	.01387
9	.564	.1794	.01495	.0562	× .110	.00618	.02113
10	.623	.2418	.02015	.0710	× "	.00781	.02796
Springing 11	.700	.3430	.02858	.0930	× "	.01023	.03881

Points	x	x^2	y	y^2	ds	I	$\frac{ds}{I}$	$x\frac{ds}{I}$	$x^2\frac{ds}{I}$	$y\frac{ds}{I}$	$y^2\frac{ds}{I}$
Crown 0	0	0	0	0	.1931	.00450	42.89	0	0	0	0
1	.385	.1482	0.003	.00001	.612	.00454	134.80	51.90	19.98	.04	.001
2	1.225	1.5006	0.030	.0009	.842	.00473	178.01	218.06	267.12	5.34	.16
3	2.065	4.2642	0.084	.0071	.843	.00518	162.74	336.06	693.96	13.67	1.16
4	2.905	8.4390	.166	.0276	.847	.00596	142.11	412.83	1199.27	23.59	3.92
5	3.745	14.0250	.279	.0773	.849	.00689	123.22	461.46	1728.16	34.26	9.52
6	4.585	21.0222	.418	.1747	.855	.00844	101.30	464.46	2129.55	42.34	17.70
7	5.425	29.4306	.587	.3446	.860	.01071	80.30	435.63	2363.28	47.14	27.67
8	6.265	39.2502	.786	.6179	.868	.01387	62.58	392.06	2456.28	49.19	38.66
9	7.105	50.4810	1.015	1.0302	.875	.02113	41.41	294.22	2090.42	42.03	42.66
10	7.945	63.1230	1.276	1.6282	1.000	.02796	35.77	284.19	2257.91	45.64	58.24
Springing 11	9.000	81.0000	1.650	2.7225	.559	.03881	14.40	129.60	1166.40	23.76	39.20
				Summary	9.203		1,119.53	3480.47	16372.33	327.00	238.91

Points of Division — Crown 0

Points of Division	m	x	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
0	0	0	0	0	0
1	.385	.385	51.90	19.98	.01
2	1.225	1.225	218.06	267.12	6.54
3	2.065	2.065	336.06	693.96	28.23
4	2.905	2.905	412.83	1199.27	68.53
5	3.745	3.745	461.46	1728.17	128.30
6	4.585	4.585	464.46	2129.55	194.13
7	5.425	5.425	435.63	2363.29	255.73
8	6.265	6.265	392.06	2456.26	308.18
9	7.105	7.105	294.22	2090.43	298.62
10	7.945	7.945	284.19	2257.89	362.61
sp	9.000	9.000	129.60	1166.40	213.84
		Summary	3480.47	16372.32	1864.72

Points of Division 1

Points of Division	m	x	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
1	0	.385	0	0	0
2	.84	1.225	149.53	183.17	4.49
3	1.68	2.065	273.40	564.58	22.97
4	2.52	2.905	358.12	1,040.33	59.45
5	3.36	3.745	414.02	1550.51	115.11
6	4.20	4.585	425.46	1950.73	177.83
7	5.04	5.425	404.71	2195.58	237.59
8	5.88	6.265	367.97	2305.31	289.24
9	6.72	7.105	278.28	1977.16	282.44
10	7.56	7.945	270.42	2148.48	345.09
sp	8.615	9.000	124.06	1116.50	204.69
			3,065.97	15,032.35	1,738.85

Points of Division 2

Points of Division	m	x			
2	0	1.225	0	0	0
3	.84	2.065	136.70	282.29	11.48
4	1.68	2.905	238.74	693.55	39.63
5	2.52	3.745	310.51	1162.88	86.34
6	3.36	4.585	340.37	1560.59	142.26
7	4.20	5.425	337.26	1829.65	197.99
8	5.04	6.265	315.40	1975.98	247.92
9	5.88	7.105	243.49	1730.01	247.14
10	6.72	7.945	240.37	1909.76	306.70
sp	7.775	9.000	111.96	1007.64	184.73
		Summary	2274.80	12152.35	1464.19

Points of Division 3

Points of Division	m	x			
3	0	2.065	0	0	0
4	.84	2.905	119.37	346.78	19.82
5	1.68	3.745	207.01	775.25	57.56
6	2.52	4.585	255.28	1170.44	106.70
7	3.36	5.425	269.81	1463.72	158.39
8	4.20	6.265	262.84	1646.65	206.60
9	5.04	7.105	208.71	1482.87	211.83
10	5.88	7.945	210.33	1671.04	268.36
sp	6.935	9.000	99.86	911.74	164.78
			1,633.21	9,468.49	1,194.04

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CALCULATIONS FOR

矯川橋

Point Division 4

Points Division	m	x	$m\frac{ds}{I}$	$mx\frac{ds}{I}$	$my\frac{ds}{I}$
4	0	2.905	0	0	0
5	.84	3.745	103.51	387.63	28.78
6	1.68	4.585	170.18	780.29	71.13
7	2.52	5.425	202.36	1,097.79	118.79
8	3.36	6.265	210.27	1,317.32	165.28
9	4.20	7.105	173.92	1,235.72	176.53
10	5.04	7.945	180.28	1,432.32	230.03
sp	6.095	9.000	87.77	789.91	144.82
Summary			1,128.29	7,040.98	935.36

Point Division 5

Points Division	m	x			
5	0	3.745	0	0	0
6	.84	4.585	95.09	390.15	35.57
7	1.68	5.425	134.90	731.86	79.20
8	2.52	6.265	157.70	987.99	123.96
9	3.36	7.105	139.14	988.58	141.22
10	4.20	7.945	150.23	1,193.60	191.69
sp	5.255	9.000	75.67	681.05	124.86
Summary			742.73	4,973.23	696.50

Point Division 6

Points Division	m	x			
6	0	4.585	0	0	0
7	.84	5.425	67.45	365.93	39.60
8	1.68	6.265	105.13	658.66	82.64
9	2.52	7.105	104.35	741.43	105.92
10	3.36	7.945	120.19	954.88	153.35
sp	4.415	9.000	63.58	572.18	104.90
Summary			460.70	3,293.08	486.41

Point Division 7

Points Division	m	x			
7	0	5.425	0	0	0
8	.84	6.265	52.57	329.35	41.32
9	1.68	7.105	69.57	494.29	70.61
10	2.52	7.945	90.14	716.16	115.01
sp	3.575	9.000	51.48	463.32	84.94
Summary			263.76	2,003.10	311.88

Point Division 8

Points Division	m	x			
8	0	6.265	0	0	0
9	.84	7.105	34.78	247.15	35.31
10	1.68	7.945	60.09	477.44	76.68
sp	2.735	9.000	39.38	354.46	64.98
Summary			134.25	1,079.05	176.97

Point Division 9

Points Division	m	x			
9	0	7.105	0	0	0
10	.84	7.945	30.05	238.72	38.34
sp	1.895	9.000	27.29	245.59	45.03
Summary			57.34	484.31	83.37

Point Division 10

Points Division	m	x			
10	0	7.945	0	0	0
sp	1.055	9.000	15.19	136.73	25.07
Summary			15.19	136.73	25.07

Crown Thrust $H_0 = \dfrac{\int\frac{ds}{I}\int my\frac{ds}{I} - \int\frac{mds}{I}\int y\frac{ds}{I}}{2\left(\int\frac{ds}{I}\int y^2\frac{ds}{I} - \left(\int y\frac{ds}{I}\right)^2\right)} = \dfrac{A}{B}$ 267,445.64

106.729

$B = 2(1119.53 \times 238.891 - (327.0)^2) = 321,037$

Loaded pt.	$\int\frac{ds}{I}$	$\int my\frac{ds}{I}$	Product		$\int m\frac{ds}{I}$		$\int y\frac{ds}{I}$	Product	A		B	
0	1119.53 ×	1,864.72	2,087,610	−	348.047	×	327.00	1,138,114	949,496	÷	321,034	= 2.9576 0
1	〃	1,738.85	1,946,695	−	306.597	〃		1,002,572	944,123	〃	〃	= 2.9409 1
2	〃	1,464.19	1,639,205	−	227.480	〃		743,860	895,345	〃	〃	= 2.7889 2
3	〃	1,194.04	1,336,764	−	163.321	〃		534,060	802,704	〃	〃	= 2.5004 3
4	〃	935.36	1,047,164	−	112.829	〃		368,951	678,213	〃	〃	= 2.1126 4
5	〃	696.50	779,753	−	74.273	〃		242,873	536,880	〃	〃	= 1.6723
6	〃	486.41	544,551	−	46.070	〃		150,649	393,902	〃	〃	= 1.2270 6
7	〃	311.88	349,159	−	26.376	〃		86,250	262,909	〃	〃	= .8189
8	〃	176.97	198,123	−	13.425	〃		43,900	154,223	〃	〃	= 1.4804 8
9	〃	83.37	93,335	−	5.734	〃		18,750	74,585	〃	〃	= .2323
10	〃	25.07	28,067	−	1.519	〃		4,967	23,100	〃	〃	= 1.0720 10
sp												0

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MADE BY Mr. Jisuda DATE 5-10-27 FILE NO
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CALCULATIONS FOR

鴨川橋

Crown Moment $M_0 = \dfrac{-H_0 2\int y\frac{ds}{I} + \int m\frac{ds}{I}}{2\int\frac{ds}{I}} = \dfrac{C}{D}$ D = 223906

Loaded pt.		H_0	$\int y\frac{ds}{I}$		product		$\int m\frac{ds}{I}$		C		D		
0	-2×	2.9576	327.0	−	1,934.27	+	348047	+	1,546.20	÷	223906	=	+0.6906
1		2.9409		−	1,923.35	+	306597	+	1.142.62	〃		=	+0.5103
2		2.7889		−	1,823.94	+	227480	+	450.86	〃		=	+0.2014
3		2.5004		−	1,635.26	+	163321	−	2.05	〃		=	−0.0009
4		2.1126		−	1,381.64	+	1128.29	−	253.35	〃		=	−0.1132
5		1.6723		−	1,093.68	+	742.73	−	350.95	〃		=	−0.1567
6		1.2270		−	802.46	+	460.70	−	341.76	〃		=	−0.1526
7		.8189		−	535.56	+	263.76	−	271.80	〃		=	−0.1214
8		.4809		−	314.18	+	134.25	−	179.93	〃		=	−0.0804
9		.2323		−	151.92	+	57.34	−	94.58	〃		=	−0.0422
10		.0720		−	47.09	+	15.19	−	31.90	〃		=	−0.0142
ap		0.		−	0	+	0		0	〃		=	0.

Crown Shear $V_0 = \dfrac{\int mx\frac{ds}{I}}{2\int x\cdot\frac{ds}{I}} = \dfrac{E}{F}$ F = 32744.66

Loaded pt.	E		F		V_0	$1 - V_0$	
0	16,372.32	÷	32744.66	=	0.5000	0.5000	0
1	15,032.35	÷	〃	=	0.4591	0.5409	1
2	12,152.35	÷	〃	=	0.3711	0.6289	2
3	9,468.49		〃	=	0.2892	0.7108	3
4	7,040.98		〃	=	0.2150	0.7850	4
5	4,973.23		〃	=	0.1519	0.8481	
6	3,293.08		〃	=	0.1006	0.8994	6
7	2,003.10		〃	=	0.0612	0.9388	
8	1,079.05		〃	=	0.0330	0.9670	8
9	484.31		〃	=	0.0148	0.9852	
10	136.73		〃	=	0.0042	0.9958	10
ap	0		〃	=	0	0	

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MADE BY M. Jiuda DATE 5-10-27 FILE NO
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CALCULATIONS FOR

鴨川橋

Moment at various points for 1 unit load
let x and y cordinates of center of sections
d' lever arm of unit load about center of sections, origin at crown

For left hand sections $M_L = M_0 + Hoy + Vox - d'$
" right hand sections $M_R = M_0 + Hoy - Vox$

moment at various points

Springing x = 9.000' y = 1.650'

Point Dist.	M_0	Hoy	Vox	d'	M_L	M_R
0	+.6906	4.8800	4.5000	9.000	1.0706	1.0706
1	+.5103	4.8525	4.1319	8.615	.8797	1.2309
2	+.2014	4.6017	3.3399	7.775	.3680	1.4632
3	-.0009	4.1257	2.6028	6.935	-.2074	1.5220
4	-.1132	3.4858	1.9350	6.095	-.7874	1.4376
5	-.1567	2.7593	1.3671	5.255	-1.2853	1.2355
6	-.1526	2.0246	.9054	4.415	-1.6376	.9666
7	-.1214	1.3512	.5508	3.575	-1.7944	.6790
8	-.0804	.7927	.2970	2.735	-1.7257	.4153
9	-.0422	.3833	.1332	1.895	-1.4207	.2079
10	-.0142	.1188	.0378	1.055	-.9126	.0668

Point 10 x = 7.945' y = 1.276'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	3.7739	3.9725	7.945	.4920	.4920
+.5103	3.7526	3.6475	7.560	.3504	.6154
+.2014	3.5586	2.9484	6.720	-.0116	.8116
-.0009	3.1905	2.2977	5.880	-.3927	.8918
-.1132	2.6957	1.7082	5.040	-.7493	.8743
-.1567	2.1339	1.2068	4.200	-1.0160	.7704
-.1526	1.5657	.7993	3.360	-1.1476	.6138
-.1214	1.0449	.4862	2.520	-1.1103	.4373
-.0804	.6130	.2622	1.680	-.8853	.2703
-.0422	.2964	.1176	.840	-.4682	.1366
-.0142	.0919	.0334		.1111	.0443

Point 9 x = 7.105' y = 1.015'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	3.0020	3.5525	7.105	.1401	.1401
+.5103	2.9850	3.2619	6.720	.0372	.2334
+.2014	2.8307	2.6367	5.880	-.2112	.3954
-.0009	2.5379	2.0548	5.040	-.4482	.4822
-.1132	2.1443	1.5276	4.200	-.6413	.5035
-.1567	1.6974	1.0792	3.360	-.7401	.4615
-.1526	1.2454	.7148	2.520	-.7124	.3780
-.1214	.8312	.4348	1.680	-.5354	.2750
-.0804	.4876	.2345	.840	-.1983	.1727
-.0422	.2358	.1052		.2988	.0884
-.0142	.0731	.0298		.0887	.0291

Point 8 x = 6.265' y = .786'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	2.3247	3.1325	6.265	-.1172	-.1172
+.5103	2.3115	2.8763	5.880	-.1819	-.0545
+.2014	2.1921	2.3249	5.040	-.3216	.0686
-.0009	1.9653	1.8118	4.200	-.4238	.1526
-.1132	1.6605	1.3470	3.360	-.4657	.2003
-.1567	1.3144	.9517	2.520	-.4106	.2060
-.1526	.9644	.6303	1.680	-.2379	.1815
-.1214	.6437	.3934	.840	.0657	.1389
-.0804	.3776	.2067		.5039	.0905
-.0422	.1826	.0927		.2331	.0477
-.0142	.0566	.0263		.0687	.0161

Point 7 x = 5.425' y = .587'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	1.7361	2.7125	5.425	-.2858	-.2858
+.5103	1.7263	2.4906	5.040	-.3128	-.2540
+.2014	1.6371	2.0132	4.200	-.3483	-.1747
-.0009	1.4677	1.5689	3.360	-.3243	-.1021
-.1132	1.2401	1.1664	2.520	-.2267	-.0395
-.1567	.9816	.8241	1.680	-.0310	.0008
-.1526	.7202	.5458	.840	.2734	.0218
-.1214	.4807	.3320		.6913	.0273
-.0804	.2820	.1790		.3806	.0226
-.0422	.1369	.0803		.1745	.0139
-.0142	.0423	.0228		.0509	.0053

Point 6 x = 4.585' y = .418'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	1.2363	2.2925	4.585	-.3656	-.3656
+.5103	1.2293	2.1050	4.200	-.3554	-.3654
+.2014	1.1658	1.7015	3.360	-.2913	-.3343
-.0009	1.0452	1.3260	2.520	-.1497	-.2817
-.1132	.8831	.9358	1.680	.0757	-.2158
-.1567	.6990	.6465	.840	.4088	-.1542
-.1526	.5129	.4613		.8216	-.1010
-.1214	.3423	.2806		.5015	-.0597
-.0804	.2008	.1513		.2717	-.0309
-.0422	.0971	.0679		.1228	-.0130
-.0142	.0301	.0193		.0352	-.0034

Point 5 x = 3.745' y = .278'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	.8222	1.8725	3.745	-.3597	-.3597
+.5103	.8176	1.7193	3.360	-.3128	-.3914
+.2014	.7753	1.3898	2.520	-.1535	-.4131
-.0009	.6951	1.0831	1.680	.0973	-.3889
-.1132	.5823	.8052	.840	.4393	-.3311
-.1567	.4649	.5689		.8771	-.2607
-.1526	.3411	.3767		.5652	-.1882
-.1214	.2277	.2292		.3355	-.1229
-.0804	.1336	.1236		.1768	-.0704
-.0422	.0646	.0554		.0778	-.0330
-.0142	.0200	.0157		.0215	-.0099

Point 4 x = 2.905' y = .166'

M_0	Hoy	Vox	d'	M_L	M_R
+.6906	.4910	1.4525	2.905	-.2709	-.2709
+.5103	.4882	1.3337	2.520	-.1878	-.3352
+.2014	.4630	1.0780	1.680	.0629	-.4136
-.0009	.4151	.8401	.840	.4143	-.4259
-.1132	.3507	.6246		.8639	-.3889
-.1567	.2776	.4413		.5622	-.3204
-.1526	.2037	.2922		.3433	-.2411
-.1214	.1359	.1778		.1923	-.1633
-.0804	.0797	.0959		.0952	-.0966
-.0422	.0386	.0430		.0394	-.0466
-.0142	.0120	.0122		.0100	-.0144

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

MADE BY M. Jinda DATE 5-10-27 FILE NO
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CALCULATIONS FOR

		Point 3	x = 2.065	y = .084			Point 2	x = 1.225	y = .030		
Divi.	Mo	Hoy	Vox	d'	ML	MR	Hoy	Vox	d'	ML	MR
0	+0.6906	0.2484	1.0325	2.065	-.0935	-.0935	0.0887	0.6125	1.225	.1668	.1668
1	+0.5103	0.2470	0.9480	1.680	.0253	-.1907	0.0882	0.5624	.84	.3209	.0361
2	+0.2014	0.2343	0.7663	.840	.3620	-.3306	0.0837	0.4546		.7397	-.1695
3	-0.0009	0.2100	0.5972		.8063	-.3881	0.0750	0.3543		.4289	-.2802
4	-0.1132	0.1775	0.4440		.5083	-.3797	0.0634	0.2634		.2136	-.3132
5	-0.1567	0.1405	0.3137		.2975	-.3299	0.0502	0.1861		.0796	-.2926
6	-0.1526	0.1031	0.2077		.1582	-.2572	0.0368	0.1232		.0074	-.2390
7	-0.1214	0.0688	0.1264		.0738	-.1790	0.0240	0.0750		-.0218	-.1718
8	-0.0804	0.0404	0.0681		.0281	-.1081	0.0144	0.0404		-.0256	-.1064
9	-0.0422	0.0195	0.0306		.0079	-.0533	0.0070	0.0181		-.0171	-.0533
10	-0.0142	0.0060	0.0097		.0005	-.0169	0.0022	0.0051		-.0069	-.0171
		Point 1	x = .385	y = .003							
0	+0.6906	0.0089	0.1925	.385	.5070	.5070					
1	+0.5103	0.0088	0.1768		.6959	.3423					
2	+0.2014	0.0084	0.1429		.3527	.0669					
3	-0.0009	0.0075	0.1113		.1179	-.1049					
4	-0.1132	0.0063	0.0828		-.0241	-.1897					
5	-0.1567	0.0050	0.0585		-.0932	-.2102					
6	-0.1526	0.0037	0.0387		-.1102	-.1876					
7	-0.1214	0.0025	0.0236		-.0953	-.1425					
8	-0.0804	0.0014	0.0127		-.0663	-.0917					
9	-0.0422	0.0007	0.0057		-.0358	-.0472					
10	-0.0142	0.0002	0.0016		-.0124	-.0156					

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CALCULATIONS FOR

$M_L = M_o + H_{oy} + V_{ox} - d'$

$M_R = M_o + H_{oy} - V_{ix}$

	0	1	2	3	4	5	6	7	8	9	10	
M_o	+.6906	+.5103	+.2014	−.0009	−.1132	−.1567	−.1526	−.1214	−.0804	−.0422	−.0142	
H_{oy}	4.8800	4.8525	4.6017	4.1257	3.4858	2.7593	2.0246	1.3512	.7927	.3833	.1188	
M_o+H_{oy}	5.5706	5.3628	4.8031	4.1248	3.3726	2.6026	1.8720	1.2298	.7123	.3411	.1046	
V_{ox}	4.5000	4.1319	3.3399	2.6028	1.9350	1.3671	.9054	.5508	.2970	.1332	.0378	(11)
$M_o+H_{oy}+V_{ox}$	10.0706	9.4947	8.1430	6.7276	5.3076	3.9697	2.7774	1.7806	1.0093	.4743	.1424	
d'	9.000	8.615	7.775	6.935	6.095	5.255	4.415	3.575	2.735	1.895	−1.055	
M_L	1.0706	0.8797	0.3680	−0.2074	−0.7874	−1.2853	−1.0376	−1.7944	−1.7257	−1.4207	−.9126	
M_R	1.0706	1.2309	1.4632	1.5220	1.4376	1.2355	.9666	.6790	.4153	.2079	.0668	
M_o	+.6906	+.5103	+.2014	−.0009	−.1132	−.1567	−.1526	−.1214	−.0804	−.0422	−.0142	
H_{oy}	3.7739	3.7526	3.5586	3.1905	2.6957	2.1339	1.5657	1.0449	.6130	.2964	.0919	
M_o+H_{oy}	4.4645	4.2629	3.7600	3.1896	2.5825	1.9772	1.4131	.9235	.5325	.2542	.0777	
V_{ox}	3.9725	3.6475	2.9484	2.2977	1.7085	1.2068	.7993	.4862	.2622	.1176	.0334	
$M_o+H_{oy}+V_{ox}$	8.4370	7.9104	6.7084	5.4873	4.2907	3.1840	2.2124	1.4097	.7947	.3718	.1111	(10)
d'	7.945	7.560	6.720	5.880	5.040	4.200	3.360	2.520	1.680	.840		
M_L	.4920	.3509	−.0116	−.3927	−.7493	−1.0160	−1.1476	−1.1103	−.8853	−.4682	.1111	
M_R	.4920	.6154	.8116	.8918	.8743	.7704	.6138	.4373	.2703	.1366	.0443	
M_o	+.6906	+.5103	+.2014	−.0009	−.1132	−.1567	−.1526	−.1214	−.0804	−.0422	−.0142	
H_{oy}	3.0020	2.9850	2.8307	2.5379	2.1443	1.6974	1.2454	.8312	.4876	.2358	.0731	
M_o+H_{oy}	3.6926	3.4953	3.0321	2.5370	2.0311	1.5407	1.0928	.7098	.4072	.1936	.0589	
V_{ox}	3.5525	3.2619	2.6367	2.0548	1.5276	1.0792	.7148	.4348	.2345	.1052	.0298	(9)
$M_o+H_{oy}+V_{ox}$	7.2451	6.7572	5.6688	4.5918	3.5587	2.6199	1.8076	1.1446	.6417	.2988	.0887	
d'	7.105	6.720	5.880	5.040	4.200	3.360	2.520	1.680	.840			
M_L	.140	.0372	−.2112	−.4482	−.6413	−.7401	−.7124	−.5354	−.1983	.2988	.0887	
M_R	.140	.2334	.3954	.4822	.5035	.4615	.3780	.2750	.1727	.0884	.0291	
M_o	+.6906	+.5103	+.2014	−.0009	−.1132	−.1567	−.1526	−.1214	−.0804	−.0422	−.0142	
H_{oy}	2.3247	2.3115	2.1921	1.9653	1.6605	1.3144	.9644	.6437	.3776	.1826	.0566	
M_o+H_{oy}	3.0153	2.8218	2.3935	1.9644	1.5473	1.1577	.8118	.5223	.2972	.1409	.0424	
V_{ox}	3.1325	2.8763	2.3249	1.8118	1.3470	.9517	.6303	.3834	.2067	.0927	.0263	(8)
$M_o+H_{oy}+V_{ox}$	6.1478	5.6981	4.7184	3.7762	2.8943	2.1094	1.4421	.9057	.5039	.2331	.0687	
d'	6.265	5.880	5.040	4.200	3.360	2.520	1.680	.840				
M_L	−.1172	−.1819	−.3216	−.4238	−.4657	−.4106	−.2379	.0657	.5039	.2331	.0687	
M_R	−.1172	−.0545	.0686	.1526	.2003	.2060	.1815	.1389	.0905	.0477	.0161	
M_o	+.6906	+.5103	+.2014	−.0009	−.1132	−.1567	−.1526	−.1214	−.0804	−.0422	−.0142	
H_{oy}	1.7361	1.7263	1.6371	1.4677	1.2401	.9816	.7202	.4807	.2820	.1369	.0423	
M_o+H_{oy}	2.4267	2.2366	1.8385	1.4668	1.1269	.8249	.5676	.3593	.2016	.0942	.0281	
V_{ox}	2.7125	2.4906	2.0132	1.5689	1.1664	.8241	.5458	.3320	.1790	.0803	.0228	(7)
$M_o+H_{oy}+V_{ox}$	5.1392	4.7272	3.8517	3.0357	2.2933	1.6490	1.1134	.6913	.3806	.1745	.0509	
d'	5.425	5.040	4.200	3.360	2.520	1.680	.840					
M_L	−.2858	−.3128	−.3483	−.3243	−.2267	−.0310	.2734	.6913	.3806	.1745	.0509	
M_R	−.2858	−.2540	−.1747	−.1021	−.0395	.0008	.0218	.0273	.0226	.0139	.0053	

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CALCULATIONS FOR

$M_L = M_o + H_{oy} + V_{ox} - d'$ $M_R = M_o + H_{oy} - V_{ox}$

	0	1	2	3	4	5	6	7	8	9	10	
M_o	+.6906	+.5103	+.2014	-.0009	-.1132	-.1567	-.1526	-.1214	-.0804	-.0422	-.0142	
H_{oy}	.12363	.12293	.11658	.10452	.8831	.6990	.5129	.3423	.2008	.0971	.0301	
	1.9269	1.7396	1.3672	1.0443	.7699	.5423	.3603	.2209	.1204	.0549	.0159	
V_{ox}	2.2925	2.1050	1.7015	1.3260	.9858	.6965	.4613	.2806	.1513	.0679	.0193	
	4.2194	3.8446	3.0687	2.3703	1.7557	1.2388	.8216	.5015	.2717	.1228	.0352	(6)
$-d'$	4.585	4.200	3.360	2.520	1.680	.830						
	-.3656	-.3554	-.2913	-.1497	.0757	.4088	.8216	.5015	.2717	.1228	.0352	
	-.3656	-.3654	-.3343	-.2817	-.2158	-.1542	-.1010	-.0597	-.0309	-.0130	-.0034	
M_o'	+.6906	+.5103	+.2014	-.0009	-.1132	-.1567	-.1526	-.1214	-.0804	-.0422	-.0142	
H_{oy}	.8222	.8176	.7753	.6951	.5873	.4649	.3411	.2277	.1336	.0646	.0200	
	1.5128	1.3279	.9767	.6942	.4741	.3082	.1885	.1063	.0532	.0224	.0058	
V_{ox}	1.8725	1.7193	1.3898	1.0831	.8052	.5689	.3767	.2292	.1236	.0554	.0157	
	3.3853	3.0472	2.3665	1.7773	1.2793	.8771	.5652	.3355	.1768	.0778	.0215	(5)
$-d'$	3.745	3.360	2.520	1.680	.840							
	-.3597	-.3128	-.1535	.0973	.4393	.8771	.5652	.3355	.1768	.0778	.0215	
	-.3597	-.3914	-.4131	-.3889	-.3311	-.2607	-.1882	-.1229	-.0704	-.0330	-.0099	
M_o	+.6906	+.5103	+.2014	-.0009	-.1132	-.1567	-.1526	-.1214	-.0804	-.0422	-.0142	
H_{oy}	.4910	.4882	.4630	.4151	.3507	.2776	.2037	.1359	.0797	.0386	.0120	
	1.1816	.9985	.6644	.4142	.2375	.1209	.0511	.0145	-.0007	-.0036	-.0022	
V_{ox}	1.4525	1.3337	1.0780	.8401	.6264	.4413	.2922	.1778	.0959	.0430	.0122	
	2.6341	2.3322	1.7424	1.2543	.8639	.5622	.3433	.1923	.0952	.0394	.0100	(4)
$-d'$	2.905	2.520	1.680	.840								
	-.2709	-.1878	.0624	.4143	.8639	.5622	.3433	.1923	.0952	.0394	.0100	
	-.2709	-.3352	-.4136	-.4259	-.3889	-.3209	-.2411	-.1633	-.0966	-.0466	-.0144	
M_o	+.6906	+.5103	+.2014	-.0009	-.1132	-.1567	-.1526	-.1214	-.0804	-.0422	-.0142	
H_{oy}	.2484	.2470	.2343	.2100	.1775	.1405	.1031	.0688	.0404	.0195	.0060	
$M_o + H_{oy}$	.9390	.7573	.4357	.2091	-.0643	-.0162	-.0495	-.0526	-.0400	-.0227	-.0082	
V_{ox}	1.0325	.9480	.7663	.5972	.4440	.3137	.2077	.1264	.0681	.0306	.0087	(3)
$M_1 + H_{oy} + V_{ox}$	1.9715	1.7053	1.2020	.8063	.5083	.2975	.1582	.0738	.0281	.0079	.0005	
d'	2.065	1.680	.840									
M_L	-0.0935	0.0253	.3620	.8063	.5083	.2975	.1582	.0738	.0281	.0079	.0005	
M_R	-0.0935	-0.1907	-.3306	-.3881	-.3797	-.3299	-.2572	-.1790	-.1081	-.0533	-.0169	
M_o	+.6906	+.5103	+.2014	-.0009	-.1132	-.1567	-.1526	-.1214	-.0804	-.0422	-.0142	
H_{oy}	.0887	.0882	.0837	.0750	.0634	.0502	.0368	.0246	.0144	.0070	.0022	
$M_o + H_{oy}$	.7793	.5985	.2851	.0741	-.0498	-.1065	-.1158	-.0968	-.0660	-.0352	-.0120	
V_{ox}	.6125	.5624	.4546	.3543	.2634	.1861	.1232	.0750	.0404	.0181	.0051	(2)
	1.3918	1.1609	.7397	.4284	.2136	.0796	.0074	-.0218	-.0256	-.0171	-.0069	
d'	1.225	.84										
M_L	0.1668	.3209	.7397	.4284	.2136	.0796	.0074	-.0218	-.0256	-.0171	-.0069	
M_R	0.1668	.0361	-.1695	-.2802	-.3132	-.2926	-.2390	-.1718	-.1064	-.0533	-.0171	

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CALCULATIONS FOR

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$M_L = M_o + H_{oy} + V_{ox} - d'$ $M_R = M_o + H_{oy} - V_{ox}$

	0	1	2	3	4	5	6	7	8	9	10	
M_o	+.6906	+.5103	+.2019	-.0009	-.1132	-.1567	-.1526	-.1214	-.0804	-.0422	-.0142	
H_{oy}	.0089	.0089	.0084	.0075	.0063	.0050	.0037	.0025	.0014	.0007	.0002	
	.6995	.5191	.2098	.0066	-.1069	-.1517	-.1489	-.1189	-.0790	-.0415	-.0140	
V_{ox}	.1925	.1768	.1429	.1113	.0828	.0585	.0387	.0236	.0127	.0057	.0016	(1)
	.8920	.6959	.3527	.1179	-.0241	-.0932	-.1102	-.0953	-.0663	-.0358	-.0124	
	.385											
	.5070	.6959	.3527	.1179	-.0241	-.0932	-.1102	-.0953	-.0663	-.0358	-.0124	
	.5070	.3423	.0669	-.1047	-.1847	-.2102	-.1876	-.1425	-.0917	-.0472	-.0156	(2)

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MADE BY K. I DATE 5-11-14 FILE NO
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CALCULATIONS FOR

Jōnan Bashi for Kyoto Prefecture

Estimate of cost.

Approximate - Cost of Pier.

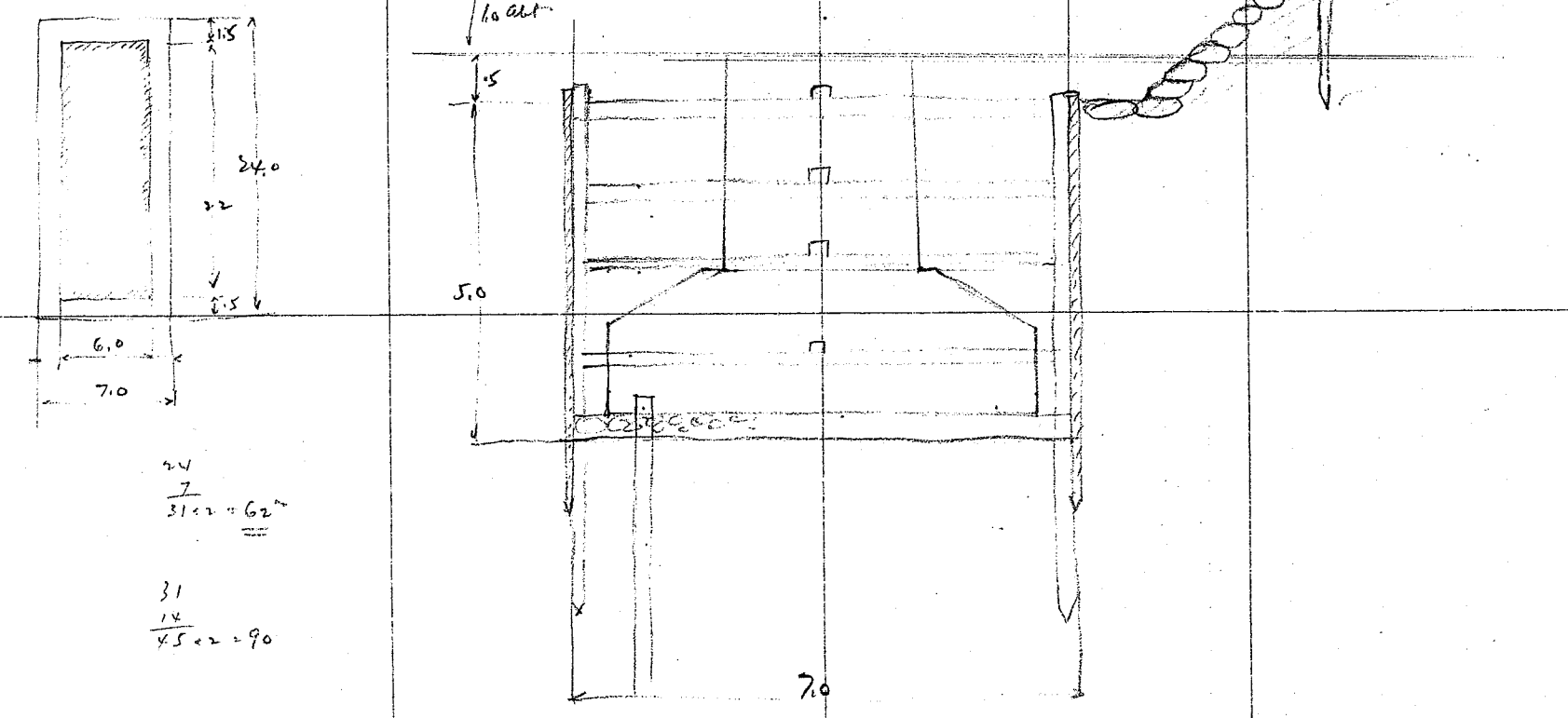
Shaft

Shaft concrete 1:2:4 mix.		285	m³	@	15.78	=	4,500. ✓
" Reinforcements		14.0	kg/tom.	@	141.	=	1,975 ✓
Stone facing		7.03	m²	@	70.	=	492 ✓
Coping		156	"	@	70.	=	39 ✓
W I	262 × 0.5 =	131	㎡	@	2.5	=	328
" Φ13	262 × 0.15	40	"	@	1.40	=	56
							50
							1600
							7600.

Base.

Piles	24 Conc × 6.5ᵐ	85	本	@	13.0 / 10.0 (72)	=	1955 -
掘石		45	m³	@	3.8	=	170 .
コンクリート 1:3:6		241	"	@	12.43	=	2995 ✓
" 捨圍ﾒﾅｶｽ		80	㎡	@	1.4	=	110
							5230 ✓

締切ｼｭﾛｾﾞ奥



24 / 7 / 31×2 = 62ᵐ

31 / 14 / ×5×2 = 90

矢板.	高6米 至 7.5cm	62.	m	@	22.0	=	1370.
水替		50	日	@	20.0	=	1000
假ﾒﾅｶｽ		90	ᵐ	@	7.0	=	630
土嚢基礎							300
							3300
床堀ｺﾝ	24×7×7 = 1175 / 125	1300	m³	@	1.2	=	1560
							1560
矢板工.	6.4×40 = 256 / 2.2×56 = 124	380	m²	@	1.02	=	580
							18270
					misc say		230
					Cost of one pier	=	18500 円 ②

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CALCULATIONS FOR

Jōnan Bashi for Kyoto Prefecture.

Estimate of Cost for Abutment.

Concrete 1:2:4		170 m³	@ 15.78	=	2,680
〃 1:3:6		714 〃	@ 12.43	=	8880
Reinforcement		15 ton	@ 142.	=	2130
コンクリート打設用人工	人工	290 人	@ 1.4	=	410
					14100
袖工					2730
基礎 掘さく 袖					
杭打 杭打ち	21cm φ × 6.5m	112 本	@ 23	=	2580
雲丹	10 + (10.6×20 3×.3)=	75 m³	@ 3.8	=	285
袖用杭	18 × 5.0	36 本	@ 13.	=	465
					3330
締切 水替		60 日	@ 20	=	12m
締切					8m
					2000
床堀 掘さく		2000 m³	@ 1.2	=	24m
両袖					1100
埋戻工	掘さく	400 m³	@ 1.52	=	3,330
	両袖	220	@ 1.52	=	610
					330
					940
上屋工					350
					27000

Estimate of cost for Arch ring.

Concrete 1:2:4		120 m³	@ 15.78	=	1890
〃 打設用人工		6.0 人	@ 1.40	=	85
Reinforcements		15.1 ton	@ 142.	=	2145
人工		40 m²	@ 3.63	=	145
					4265 for one span

Floor slab. 1:2:4

Conc. 1:2:4	17.0×18	=	2.52
aver.	4.0 × .12	=	.48
cap.	2×.2×.35	=	.14
	2×.35×.2	=	.14
	3.28 m² × 19.25	=	63.1 m³
less.	.18 × 5.0 ×2 ×.5	=	- 9.9
			3.2 m³ × 6 = 319 m³
Cross fill 1:4:8	88m × 4 ÷ 24m	=	14.7 × 6 = 88 〃
Reinforcement.			
	18 × 19.25 = 346		
less	.35 × 5.5 × 2 = 89		
	287 × 6 = 1720 m²	@ .023	= 39.6 kg for one
Forms	20 × 19.25 × 6 = 2310		
	59 × 6 = 354		
	1956		= 200 m²
人工 etc	1.8 × 19.25 × 6		= 200 m²

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CALCULATIONS FOR

Jōsui Bashi for Kyoto Prefecture

Estimate of Cost for floor.

Slab concrete 1:2:4	320 m³	@ 15.78	=	5050
crown fill one 1:4:8	90 "	@ 9.10	=	820
reinforcements	40 tons	@ 142.	=	5680
forms.	2000 m²	@ 1.5	=	3000
人 4/3	200 m²	@ 3.63	=	730
コンクリート打込(ECC) 1:2:4 と 1:4:8	200 人	@ 1.40	=	280
				15,560 円

Estimate of Cost for Spandrel walls beams etc.

Transverse wall + beam wt. 78600 one span
Longitudinal wall - wt. 21300
99900 ÷ 2.4 t/m³ = 41.7 × 6 = 250, m³
Reinforcements 3.3 × 6 = 20, tons
forms. 250 @ 7.0 = 1750 + 250 allowance = 2000 m²
人 4/3 35 × 6 × 2 = 420 m²

Cost.

concrete 1:2:4	250 m³	@ 15.78	=	3950
Reinforcement	20 tons	@ 142.	=	2840
forms	2000 m²	@ 1.52	=	3040
人 4/3	420 m²	@ 3.63	=	1530
conc 打込	125 人	@ 1.40	=	180
				11,540 円

Summary of Cost for floor arching walls etc.

Arching complete	6 spans	@ 4265	=	25,600 円
floor construction compl			=	15560
Spandrel walls + crossbeams etc			=	11,540
drains	7 m/m	@ 59	=	400
handrels. complete.			=	21200
pavement 土工, 舗装工, コンクリート, etc			=	14060
Centering			=	22000
				110,300 円

3

Summary for Costs.

橋台工	2 @	27,000	= 54000	2 @ 25,750 =	51,500
橋脚工	5 @	18,500	= 92,500	5 @ 24,200 =	121,000
橋体工	1 a.		= 110,300		110,300
雑工事費			1600		1,600
石垣			258,400		285,000
			1.6m		
			260,000		

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CALCULATIONS FOR

Jōnan Bashi for Kyoto Prefecture

Estimate of Cost for floor, archrings, walls &c.

Arch rings complete 6 spans @ 4265 = 25,600 ¥

= 8830
= 2020
= 900

= 13,750

= 8,400 @ 4¥ = 33,600
= 7,200 @ 4¥ = 28,800
= 1,400 @ 4¥ = 5,600

= 68,000

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CALCULATIONS FOR

鴨川架橋橋梁材料計算書

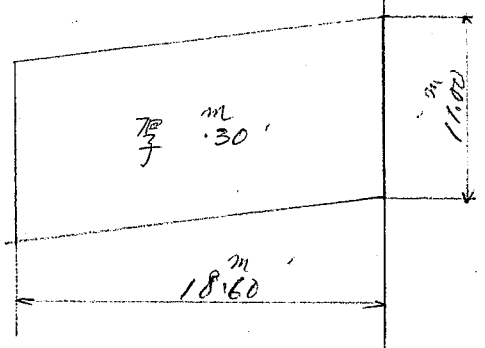
第一編 橋台 (一基当り)

第一章 基礎工

第一節 基礎杭

形状寸法 0.21 (末口) × 7.50 (長) 生松丸太
使用本数 8 × 14 = 112 本

第二節 基礎割栗石

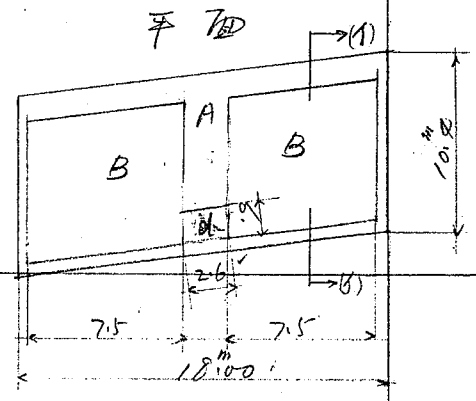


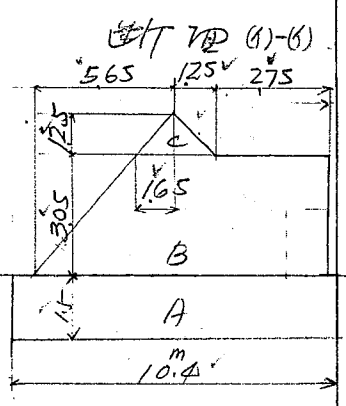
土積 18.6 × 11.0 × .3 = 61.38 m³

第二章 躯体工

第一節 混凝土

第一項 配合 1:3:6



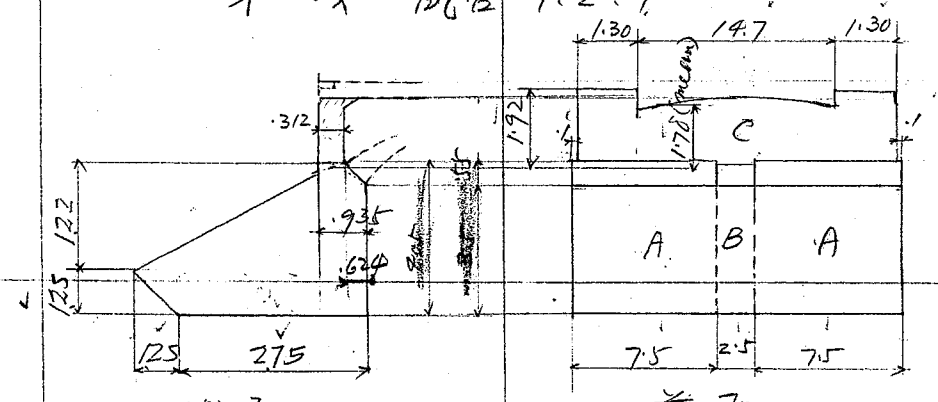


A. 10.4 × 18.0 × 1.5 = 280.80 m³

B ½ × (5.65 + 9.65) × 3.05 × 7.50 = 174.99
C ½ × 2.90 × 1.25 × 7.50 = 13.59
188.58
∴ 2e 188.58 = 377.16 m³
d 2.6 × 9 × 3.05 = 7.14

less facing stone abt = .06
合計 665.04 m³

第二項 配合 1:2:4



A. ½ × (2.75 + 4.00) × 1.25 × 7.50 = 31.64
½ × (.70 + 1.23) × .624 × 7.50 = 4.52
½ × 1.22 × 3.376 × 7.50 = 15.45
51.61
∴ 2e 51.61 = 103.22 m³

B. .936 × 2.2 × 2.5 = 5.15 m³

C 1.3 × 1.92 × .312 × 2 = 1.56
14.7 × 1.78 × .312 = 8.16 — 10.05 m³ 9.92
fillet .15 × .15 × ½ × 14.7 = .33 17.37 .20
less. stone facing = 2e .35 × .17 × 1.99 = — .24
合計 11.805 m³

第二節 鉄筋

別紙・鉄筋材料表ヲ写ス事

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CALCULATIONS FOR

List of Reinforcements for abutments

Mark	NO.	Dia.	Unit Wt.	Length	piece wt.	Total Wt.	Remark
A 1	84	22	2.98	595	17.73	1489.32	
A 2	54	〃	〃	710	21.16	1142.64	
A 3	76	〃	〃	440	13.11	996.36	
A 4	10	19	2.22	440 415	1311 9.21	131.10	92.10
A 5	42	〃 22	3.9	720	21.46	901.32	
A 6	30	19	2.22	680	15.10	453.00	
A 7	28	22	2.98	235	7.00	196.00	
A 8	32	〃	〃	250	7.75 7.45	248.00	238.40
A 9	32	〃	〃	775 90	2310 54	739.20	753.28
A 10	60	25	3.85	540	20.79	1247.40	
A 11	60	〃	〃	360	13.86	831.60	
A 12	116	22	2.98	590	17.58	2039.28	
A 13	64	16	1.59	285	4.48	286.72	
A 14	16	〃	〃	300	4.71	75.36	
A 15	〃	〃	〃	390	6.12	97.92	
A 16	〃	〃	〃	270	4.24	67.84	
A 17	〃	〃	〃	365	5.73	91.68	
A 18	〃	〃	〃	445	6.99	111.84	
A 19	〃	〃	〃	520	8.16	130.56	
A 20	〃	〃	〃	575	9.03	144.48	
A 21	〃	〃	〃	445	6.99	111.84	
A 22	〃	〃	〃	300	4.71	75.36	
A 23	22	〃	〃	800	12.56	276.32	
A 24	32	〃	〃	480	7.54	241.28	
A 25	69	12	.88	275	2.42	166.98	
A 26	69	〃	〃	310	2.73	188.37	
A 27	65	〃	〃	95	.84	54.60	
A 28	26	〃	〃	645	5.68	147.68	
A 29	4	〃	〃	525 10	462 4.49	1848	17.96
A 30	8	〃	〃	140	1.23	9.84	
A 31	8	〃	〃	200	1.76	14.08	
						12,726.45	12691.41 Kg

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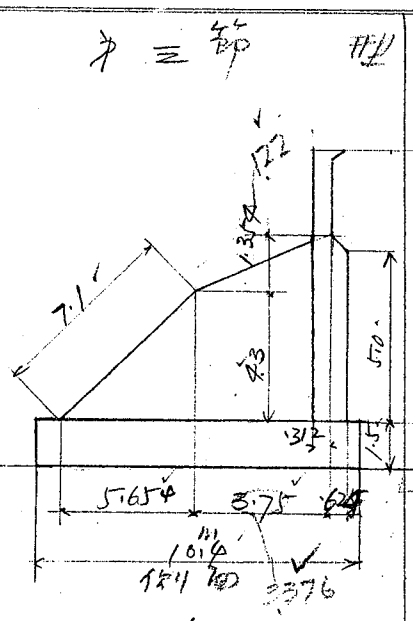
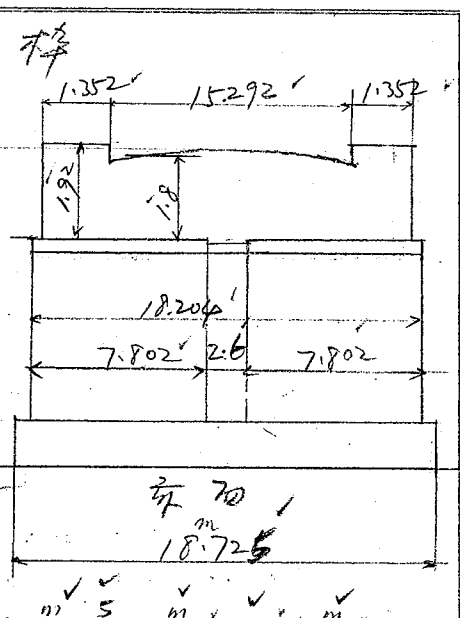
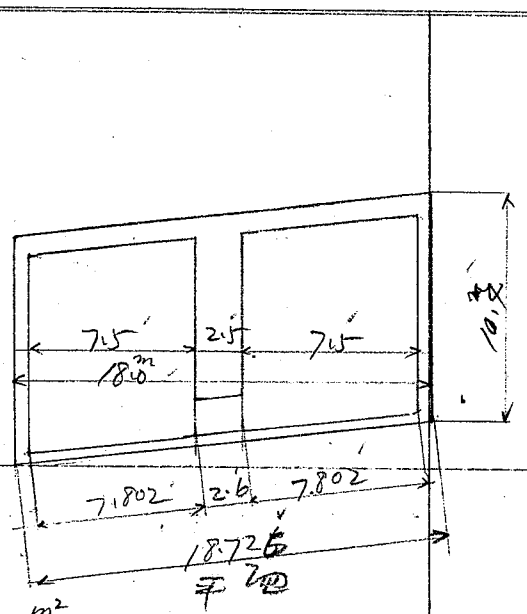
MADE BY M.Kojima DATE 5-11-23 FILE NO
CHECKED BY M.Janda DATE 5-11-26 PAGE NO 2

CALCULATIONS FOR







Bed	(18.726+10.14)×2×1.5 =	87.4	✓ ✓
body (front)	18.204×5.0 =	91.0	✓ ✓
〃 (Rear)	7.1×7.802×2 =	110.8	✓ ✓
〃	2.6×5.2 − 5.58 =	13.5 − 18.5	✓ ✓
〃 (side)	4.3×5.654×½×4 =	48.6	✓ ✓
〃 〃	(4.34+5.656)×½×3.75×4 =	73.7 − 66.3	✓ ✓
〃 〃	5.327×0.624×2 =	6.7 − 6.6	✓ ✓
〃 〃 (less)	5.8×0.312×2 =	− 3.4	✓ ✓
parapet wall	1.92×1.352×4 =	10.4	✓ ✓
〃	1.8×15.292×2 =	55.1	✓ ✓ ✓
	合計	493.8 m²	487.3

4863

第四節 橋脚

48	4.16×0.25×0.30×8 =	2.9 m³	✓
	0.312×0.7×1.1×8 =	1.9 − 2.2	✓
	合計	4.4 m³	
		5.1	

土工部 橋台 石積仕上面積

表面仕上	2c.60×2.40 = 2.88	✓
合端 〃	16c.03×1.80 = 0.86	✓
	3.74 m²	
中詰仕上	32c½(10+23)×35 = 185	✓ m²

第三章 翼壁工

第一節 基礎杭

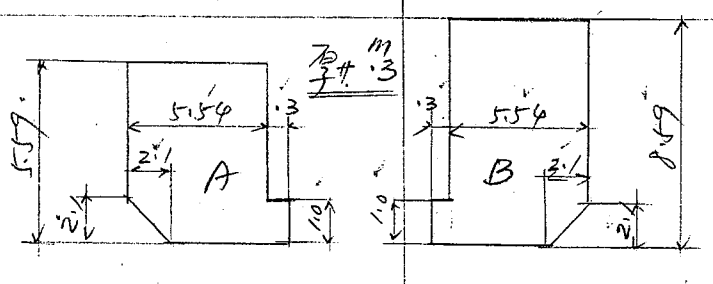
形状寸法

A 側 使用本数
B 〃 〃

0.18(径)×5.00(長)

	生杭
14本	✓
22本	✓
合計 40本	✓
36	✓

第二節 基礎割栗石



A 翼壁	5.59×5.56 =	31.0 m²	✓
(less)	2.1×2.1×½ =	− 2.2	✓
	1.0×0.3 =	0.3	✓
	29.1×0.3 =	8.73 m³	✓
B 翼壁	8.59×5.54 =	47.6	✓
(less)	2.1×2.1×½ =	− 2.2	✓
	1.0×0.3 =	0.3	✓
	45.7×0.3 =	13.71 m³	✓
合計		22.44 m³	

打合せ ✓

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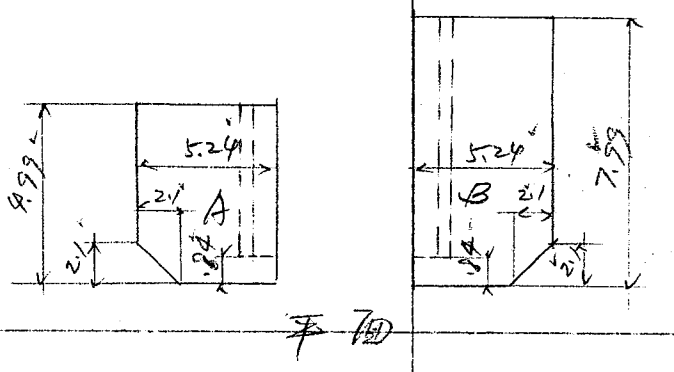
MADE BY M. Kawai DATE 5-11-23 FILE NO 3
CHECKED BY Jun Masuda DATE 5-11-26 PAGE NO

CALCULATIONS FOR

中＝節 三民凝土
中一項 配合 1:3:6



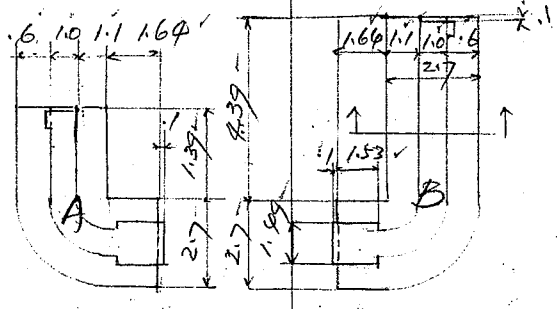
平面 斷面

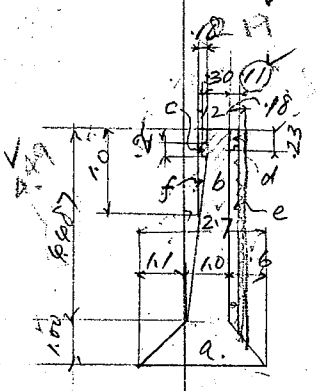
A 翼壁 4.99'×5.24'×1.25 = 31.22 m³ 3268 ✓
less 2.1'×2.1'×½×1.5 = − 3.31 − 276 ✓
(.75+1.25)×½×.8×4.15' = 1.66' m³
37.57 3158 ✓

B 翼壁 7.99'×5.24'×1.25 = 62.82 5282 ✓
less 2.1'×2.1'×½×1.5 = − 3.31 − 276 ✓
(.75+1.25)×½×.8×7.15 = 2.86 ✓
62.35 m³ 5242 ✓

AB 合計 99.92 m³ 8401 ✓ ✓ ✓

中＝項 配合 1:2:4

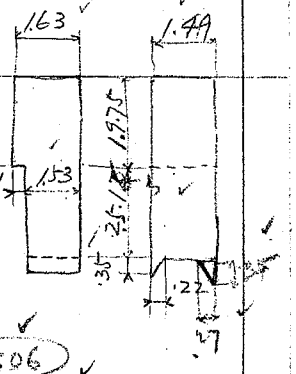




平面 斷面

A 翼壁 section area (2.7+1.0)×½×1.100' = 1.85 m² a
(1.0'+.3')×½×4.487' = 2.92 b
.2'×.1'2' = .02 c
.23'×.111 = .03 d

Volume a 1.85'×(1.64'+1.39'+2.8') = 10.79 m³ 10.64
" b 2.92'×(1.29'+.11'+3.19') = 13.14 ✓ ✓
" c .02'×3.5' = .07 ✓ ✓
" d .03'×4.3' = .13 ✓ ✓
" e .18 .11 ×4.5'×.47' = .23 .38
" f .8'×1.0'×½×.47' = .04 ✓ ✓
Column 1.49'×1.53'×2.518 = 5.73 ✓ ✓
" 1.49'×1.63'×1.975' = 4.80 10.59 10.66
" 1.53'×.35'×.22'×½ (+.27) = .96 ✓ ✓
.13 m³ 34.99 3506



B 翼壁 Volume a. 1.85×(1.64+4.39+2.72) = 16.34 16.19
b. 2.92×(4.29+.11+3.79) = 21.90 ✓ ✓
c .02×6.5 = .13 ✓ ✓
d .03×7.3 = .22 ✓ ✓
e = .23 .38 ✓ ✓
f = .04 ✓ ✓
Column = 10.59 10.66 ✓
49.45 m³ 4952 ✓

A.B. 合計 84.44 m³ 8458 ✓ ✓

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MADE BY M. Jmida DATE 5-11-25 FILE NO (4)
CHECKED BY mk DATE 5-11-27 PAGE NO 5

CALCULATIONS FOR

List of Reinforcements for wing wall

Mark	No.	Dia	Unit wt.	Length	piece wt.	total wt	Remark
Common reinforcement for Both wing walls							
		Cm/ϕ	Kg/m	m	Kg	Kg	
W 1	14	1.9	2.22	4.05	8.99	125.86	
W 2	10	〃	〃	5.00	11.10	111.00	
W 3	7	1.6	1.57	5.20	8.16	57.12	
W 4	11	1.9	2.22	4.80	10.60	116.60	
W 5	7	〃	〃	3.15	6.99	48.93	
W 6	5	〃	〃	3.95	8.77	43.85	
W 7	11	1.6	1.57	3.15 (3.30)	4.95 (5.11)	54.45 (56.21)	67.32
W 8	13	2.2	2.98	6.25	18.63	242.19	
W 9	10	〃	〃	3.75	11.18	111.80	
W 10	11	〃	〃	3.85	11.47	126.17	
W 11	17	1.9	2.22	3.75	8.33	141.61	
W 12	18	〃	〃	3.50	7.77	139.86	
W 13	11	〃	〃	3.05	6.77	74.47	
W 14	17	1.6	1.57	3.00	4.71	80.07	
W 15	2	〃	〃	4.00	6.28	12.56	
W 16	22	〃	〃	3.45	5.42	119.24	
W 19	22	1.2	.88	3.20 (3.60)	2.82 (3.17)	62.04	69.74
W 20	4 (2)	1.6	1.57	1.80	2.83	11.32	5.66
W 21	10	1.2	.88	1.40	1.23	12.30	
						1,691.44 (1696.0)	1706.35 kg
Special reinforcements for Wing Wall A							
W 17	11	1.6	1.57	3.55	5.58	61.38	
W 18	〃	〃	〃	2.15	3.38	37.18	
						98.56 kg	
Special reinforcements for Wing wall B							
W 1	10	1.9	2.22	4.05	8.99	89.90	
W 2	10	〃	〃	5.00	11.10	111.00	
W 5	5	〃	〃	3.15	6.99	34.95	
W 8	5	2.2	2.98	6.25	18.63	93.15	
W 9	5	2.2	〃	3.75	11.18	55.90	
W 10	10	〃	〃	3.85	11.47	114.70	
W 11	10	1.9	2.22	3.75	8.33	83.30	
W 13	10	〃	〃	3.05	6.77	67.70	
W 14	10	1.6	1.57	3.00	4.71	47.10	
W 22	10	〃	〃	5.10	8.01	80.10	
W 23	10	〃	〃	6.45	10.13	101.30	
W 24	17	〃	〃	3.70	5.81	98.77	
						977.87 kg	

Total Reinforcements for wing wall A = 1691.44 (1696.0) + 98.56 = 1,790.00 Kgo (1794.0) — 1804.91

Total Reinforcements for wing wall B = 1691.44 + 977.87 = 2669.31 Kgo (2673.8) — 2,684.22

4489.13 kg

or 4.489 kg tons

1706.35 kg

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MADE BY M.K DATE FILE NO 5
CHECKED BY M.J DATE 5-11-27 PAGE NO 4A

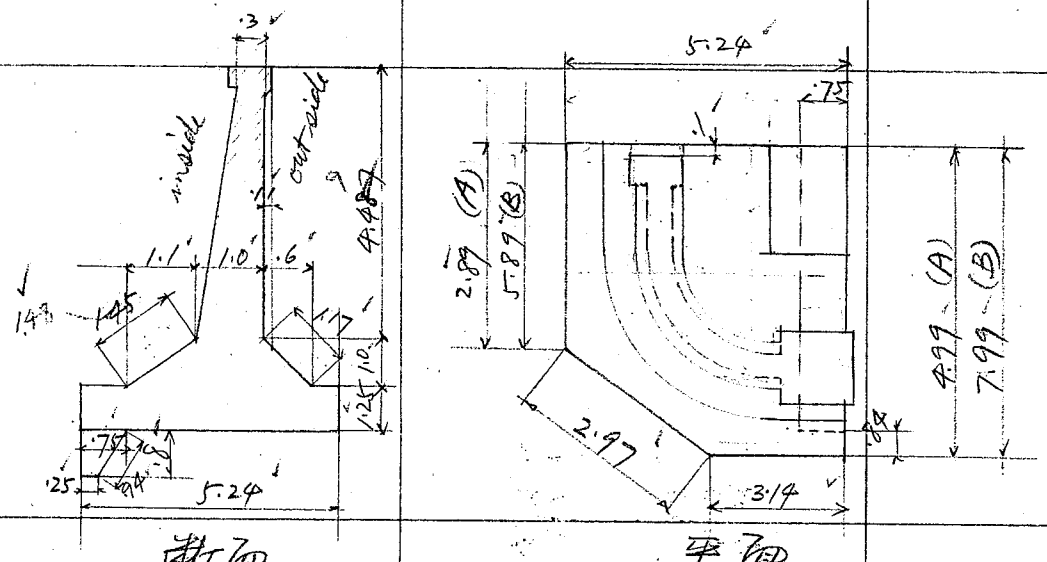
CALCULATIONS FOR

第四節 鉄筋

第一項 A 擁壁

第二項 B 擁壁 } 別紙 鉄筋材表ノ寄セコンクリート

第三節 型枠



断面図 平面図

A 擁壁

Bed	(3.14+2.97+2.89+5.24)×1.25	=	17.8 m²
〃 (▽)	(.75+.25)×½×.8×2	=	.8
〃 〃	.94×4.15	=	3.9
out. slope	1.17×7.0	=	8.2
in. 〃	1.48×4.0	=	5.89
wall (out.)	4.49×6.7	=	30.1
〃 (in.)	4.50×5.0	=	22.5
end	(2.7+1.0)×½×1.0 + (1.11+4.1)×½×4.49	=	3.4 5.3
coping	.19×4.40 + .11×3.40	=	1.2 4.25 m²
		=	24
			98.1 sq. meters

B 擁壁 Column 1/2 × 2/5

Bed	(3.14+2.97+5.89+5.24)×1.25	=	21.6
〃 (▽)	(.75+.25)×½×.8×2	=	.8
〃 〃	.94×7.15	=	6.7
out. slope	1.17×10.0	=	11.7
in. 〃	1.48×7.0	=	10.2
wall (out)	4.49×9.7	=	43.6
〃 (in.)	4.50×8.0	=	36.0
end		=	6.4 5.3
coping	.19×7.40 + .11×6.40	=	2.1 13.4.0 m²
〃	AB合計		3.26 m²
column			24
			140.4
	AB合計		238.5 sq. meters

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MADE BY M. Jmida DATE 5-11-29 FILE NO
CHECKED BY M.K DATE 5-11-29 PAGE NO

CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

橋脚石積仕上面積 (see sheet no 9. (十基分)

躯体

表面仕上 $\frac{1}{2}\times(4.293+2.756)\times 4.80 = 16.92$

合端仕上 $32\text{c}\ \frac{1}{2}\times(4.293+2.756)\times .03 = 3.38$ } 6.12

〃 $19\text{c}\ .03 \times 4.80 = 2.74$ (.22)

上段打 $1\text{c}\ .08\times 2.80$ =

2304 sq. meters
23.26×2 = 46.52

中切仕上(靖国方向) $6.12 \times \frac{20.5}{3} =$ 41.82 sq meters
×2
83.64

笠石及側壁石 (表裏縁同石)

笠石
表面仕上 .244 × 2.49 = .60 側面
〃 .30 × 1.98 = .59
合端仕上 10c .04 × .30 = .09 .12
〃 .03 × (1.51 + .244) = .12
表面仕上 .394 × 1.03 = .41
〃 .30 × .54 = .16 .08
合端仕上 .030 × 2.55 = .11

表面仕上 .2 × .3 × ½ = .03
合端仕上 .03 × .8 = .02 〃

控柱側壁
表面仕上 2c .85 × 1.70 = 2.89
〃 .228 × 1.48 = .34
合端 4c .03 × 1.70 = .20
〃 2c .03 × 1.48 = .09
〃 28c .03 × .88 = .74
〃 8c .03 × .23 = .06

中切仕上 側壁控柱
.27 × .48 = .13 .06
.36 × .78 = .28
2c .22 × 1.70 = .75
2c .12 × 1.48 = .44 .36
28c .27 × .37 = 2.80
8c .12 × .23 = .23 .22
.27 × 1.98 = .53

(亀岡方向 陸)

笠石
10c .24 × .25 = .60
.22 × 1.50 = .33
.12 × 1.0 = .12
2@ .08 = .16

6.45
×2
12.90 ㎡

6.28
×2
12.56 ㎡

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MADE BY me. Jmida DATE 5-11-27 FILE NO 6
CHECKED BY _____ DATE _____ PAGE NO 48

CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

第六節 翼壁縁石
A 翼壁縁石

品種	長	巾	厚	單位	員數	体積	体積合計
花崗石	.38	.20	.21	末	1	.016	.016
〃	.90	.24	〃	〃	3	.045	.135
〃	.80	.19	.2	〃	1	.032	.032
〃	.47	〃	〃	〃	1	.019	.019
							~~.202~~ cub meters .183

B 翼壁縁石

花崗石	.38	.20	.21	末	1	.016	.016
〃	.90	.24	〃	〃	3	.045	.135
〃	.95	.19	〃	〃	4	.038	.152
〃	.47	〃	〃	〃	1	.019	.019
							~~.322~~ cub meters .303

AB 合計 ~~.623~~ cub. meters .486

第七節 石積 ~~表面~~ 仕上

A 翼壁
表面仕上 .39 × 3.97 = 1.55
合端, 2c .03 × 3.97 = .24
10c .03 × .39 = .12
1.91 sq meters

B 翼壁
表面仕上 .39 × 6.97 = 2.72
合端, 2c .03 × 6.97 = .42
16c .03 × .39 = .19
3.33 sq.m

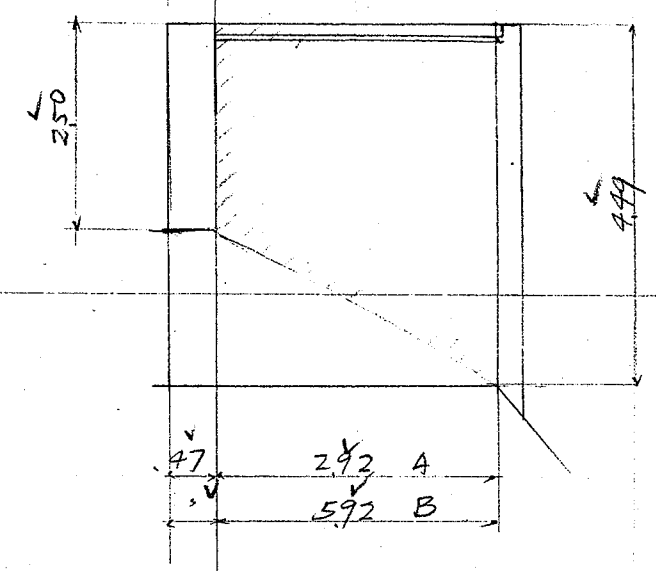
兩翼壁合計 5.24 sq meters

中切仕上
A 翼壁
10c .15 × .18 = .27 sq.m

B 翼壁
16c .15 × .18 = .43 sq.m

兩翼壁合計 .70 sq. meters

第八節 人造洗出仕上



A 翼壁
親柱 1.73 × 4.49 = 7.77
wing wall ½ × (2.50 + 4.49) × 4.30 = 15.03
coping .11 × 4.30 = .47
23.27

B 翼壁
親柱 1.73 × 4.49 = 7.77
wing wall ½ (2.50 + 4.49) × 7.30 = 25.51
coping .11 × 7.30 = .80
34.08

兩翼合計 57.35 sq. meters

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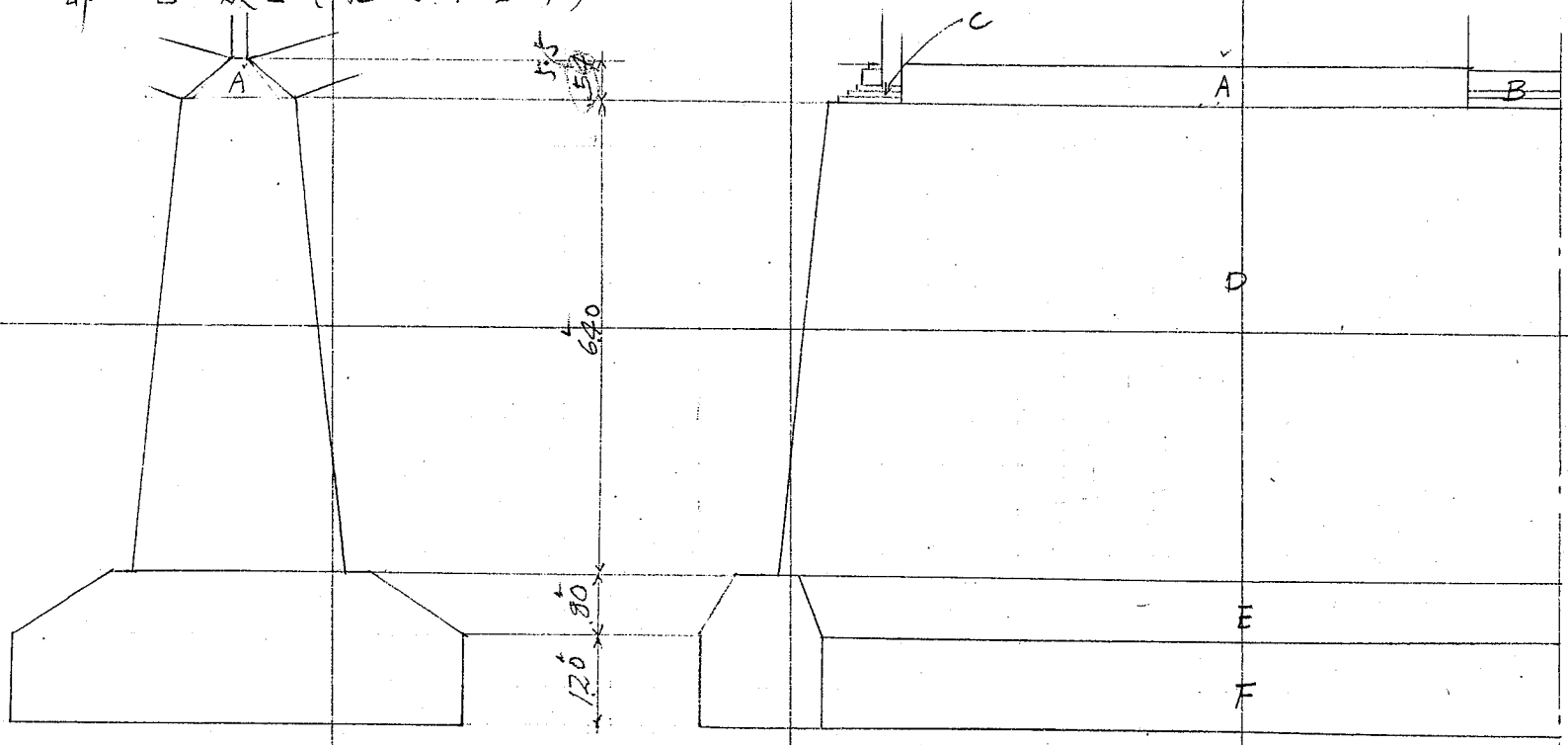
MADE BY M. Fujita DATE 5-11-21 FILE NO
CHECKED BY M.K DATE 4 4 26 PAGE NO 45

CALCULATIONS FOR

Materials of Ionae-Bashi for Kyoto Prefecture

第貳編 橋脚 (一基ニ付)
第一章 橋脚
第一節 混凝土 (配合 1:2:4)



A 部	2	c	½(.30 + 1.442) × .53 × 7.50	=	6.92	pier top
B 部	1	c	1.282 × .60 × 2.60	=	1.92	,
C 部	2	e	volume ≒ 0.1	=	0.20	ornaments
D 部	1	c	½(1.442 + 2.722) × 6.40 × 18.67	=	248.77	rectangular
	1	c	{¼(1.442 + 2.722)}² × π × 6.40	=	21.79	circular end
					279.60	280.29
					−9.04	張石ノ体積
			計		270.56	cub. meters
						271.20

第二節 鉄筋

List of Reinforcements ヲ書ク

第三節 張石 (Side of spandrel wall ヲ含ム)

List of Stone facing ヲ書ク

第四節 合端モルタル (表目地)

Side of pier	15	@	½(3.45 + 2.29) × .006 × 2.7	=	.057	.115 横目地
,	8.5	@	2.35 × .006 × 4.89	=	.059	.098 縦目地
top of pier			.75 × .006 × 1.40	=	.006	.011
Side of spand. wall	14	@	.30 × .006 × .40	=	.010	.017
,	4	@	.15 × .006 × .23	=	.001	.001
,	2	@	.15 × .006 × 1.65	=	.003	.004
top of pier			.33 × .006 × .54	=	.001	.002
						.248

2c .248 = .498 cub. metre for one pier

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MADE BY M. Jinda DATE 5-11-22 FILE NO
CHECKED BY M.K DATE PAGE NO 6

CALCULATIONS FOR

Materials of Iowan Bashi for Kyoto Prefecture

第二章 基礎

第一節 混凝土 (配合 1:3:6)

E部 $\frac{1}{2} \times (3.40 + 6.00) \times .80 \times 22.00 = 82.72$

F部 $6.00 \times 1.20 \times 22.00 = 158.40$

241.12 cub. meters

第二節 杭

長 7.50 (6.5) 末口 21㎝ {生松丸太}

一基ニ付 85本 (別紙図面参照)

第三節 栗石

$6.60 \times .30 \times 22.60 = 44.75$

$600 \times .30 \times 2200 = 3960$ cub. meters

第三章 型枠

第一節 橋脚部 (張石ノ部ヲ除ク)

B部	$2 \times 2.50 \times 2.601$	=	2.6	
D部	$2 \times 4.89 \times 18.152$	=	177.5	upper parts
D部	$2 \times 1.51 \times 18.666$	=	56.4	lower parts
D部	$2.571 \times \pi \times 1.51$	=	12.0	lower circular parts
	小計		248.5	

第二節 基礎部

F部 $2 \times 1.20 \times 28.00 = 67.2$

型枠面積 315.7 sq. meters

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MADE BY M.K DATE 5-11-5 FILE NO
CHECKED BY M.J DATE PAGE NO

CALCULATIONS FOR

橋脚 躯体張石 一基分(両側)

名称	工種	a	b	h	単位	員数	体積(1個)	体積(合計)	単価	金額
1	花崗石	.473	.265	.30	米	14	.038	.532		
1a	〃	.476	.265	.30	〃	2	.038	.076		
1b	〃	.411	.25	.30	〃	16	.031	.496		
2	〃	.461	.265	.30	〃	14	.037	.518		
2a	〃	.331	.26	.30	〃	2	.026	.052		
2b	〃	.431	.265	.30	〃	2	.034	.068		
2c	〃	.308	.25	.30	〃	16	.023	.368		
3	〃	.449	.265	.30	〃	14	.036	.504		
3a	〃	.452	.265	.30	〃	2	.036	.072		
4	花崗石	.437	.265	.30	米	14	.035	.490		
4a	〃	.319	.26	〃	〃	2	.025	.050		
4b	〃	.419	〃	〃	〃	2	.033	.066		
~~4c~~	〃	.308	.25	〃	〃	14	.023	.332		
5	〃	.425	.265	〃	〃	14	.034	.476		
5a	〃	.428	〃	〃	〃	2	〃	.068		
~~5b~~	〃	.411	.25	〃	〃	12	.031	.372		
6	〃	.413	.265	〃	〃	14	.033	.462		
6a	〃	.307	.26	〃	〃	2	.024	.048		
6b	〃	.407	〃	〃	〃	2	.032	.064		
7	〃	.401	.264	〃	〃	14	〃	.448		
7a	〃	.404	〃	〃	〃	2	〃	.064		
~~7b~~	〃	.411	.25	〃	〃	2				
8	〃	.389	.264	〃	〃	14	.031	.434		
8a	〃	.295	.26	〃	〃	2	.023	.046		
8b	〃	.395	〃	〃	〃	2	.031	.062		
9	〃	.377	.264	〃	〃	14	.030	.420		
9a	〃	.38	〃	〃	〃	2	〃	.060		
10	〃	.365	.263	〃	〃	14	.029	.406		
10a	〃	.283	.258	〃	〃	2	.022	.044		
10b	〃	.383	〃	〃	〃	2	.030	.060		
11	〃	.353	.263	〃	〃	14	.028	.392		
11a	〃	.356	〃	〃	〃	2	〃	.056		
12	〃	.341	〃	〃	〃	14	.027	.378		
12a	〃	.271	.258	〃	〃	2	.021	.042		
12b	〃	~~.301~~ .371	〃	〃	〃	2	~~.027~~ .029	~~.054~~ .058		
13	〃	.329	.262	〃	〃	14	.026	.364		
13a	〃	.332	〃	〃	〃	2	〃	.052		
14	〃	.317	〃	〃	〃	14	.025	.350		
14a	〃	.259	.256	〃	〃	2	.020	.040		
14b	〃	~~.351~~ .359	〃	〃	〃	2	~~.027~~ .028	~~.054~~ .056		
15	〃	.305	.261	〃	〃	16	.024	.336		

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CALCULATIONS FOR

Pier

List of Reinforcements (PIER 1/1 ずつ)

mark	NO.	dia cm	Length m	unit wt kg/m	piece wt kg	Total wt kg
P1	88	2.2	6.30	2.98	18.77	1,651.76
P2	44	〃	4.45	〃	14.95	635.80
P3	27	〃	~~8.35~~ 8.30	〃	~~24.88~~ 24.44	~~671.8~~ 659.88
P4	280	〃	5.10	〃	15.20	4,256.00
P5	20	〃	5.15	〃	15.35	307.00
P6	20	〃	4.40	〃	13.11	262.20
P7	49	1.9	6.90	2.22	15.32	750.68
P8	38	〃	7.30	〃	16.21	615.98
P9	4	〃	6.20	〃	13.76	55.04
P10	~~6~~ 6	〃	5.75	〃	12.77	~~51.08~~ 76.62
P11	4	〃	~~5.45~~ 5.5	〃	~~12.10~~ 12.10	~~48.40~~ 40.40
P12	4	〃	~~5.15 4.40~~ 4.25	〃	~~11.43 9.77~~ 9.44	~~45.7 39.08~~ 37.76
P13	4	〃	~~4.80 4.10~~ 3.90	〃	~~10.60 9.10~~ 8.66	~~42.4 36.40~~ 34.64
P14	4	〃	~~4.50 3.80~~ 3.60	〃	~~9.99 8.44~~ 7.99	~~40.0 33.76~~ 31.96
P15	~~6~~ 4	〃	~~4.20 3.45~~ 3.30	〃	~~9.32 7.66~~ 7.33	~~55.9 45.96~~ 29.32
P16	6	〃	7.95	〃	17.65	105.90
P17	4	〃	4.10	〃	9.10	36.40
P18	120	2.5	4.00	3.85	15.40	1,848.00
P19	120	〃	2.65	〃	10.21	1,225.20
P20	116	2.2	6.20	2.98	18.48	2,143.68
P21	116	〃	4.00	〃	14.30	1,658.80
P22	76	1.2	1.15	0.88	1.02	77.52
P23	44	〃	1.70	〃	1.50	66.00
P24	16	〃	1.30	〃	1.14	18.24

~~16667.5~~ Kg or ~~16.6675~~ Kg tons for 1 pier

~~1662876~~ ~~1662876~~ Kg tons

~~16624.78~~ ~~16.6278~~

16,624.78 Kg tons

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CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture.

第参編　拱環　(一径間当り)

第一節　混凝土　(配合 1:2:4)

1/2	.740	× .60	=	.22	
	.701	× .93	=	.65	
	.629	× .84	=	.53	
	.567	× 〃	=	.48	
	.514	× 〃	=	.43	
	.470	× 〃	=	.39	
	.434	× 〃	=	.36	4.25
	.406	× 〃	=	.34	
	.384	× 〃	=	.32	
	.367	× 〃	=	.31	
	.357	× 〃	=	.30	
	.352	× .385	=	.14	
				4.47 × 1/2 =	1788 / 8.94 sq.m

8.94 × 7.50 × 2 = 134.10 cub meters

方サ

740	A	60
662		93
596		84
538		84
490		84
450		84
418		84
394		84
374		84
360		84
354		385
350		

第二節　鉄筋

第三節　人造洗

三種厚 h

一七種厚 h

		Mean width		piece wt			unit wt
60	R1	30	25	10.0	38.5	1155.0	3.85
	R2	30	〃	8.7	33.5	1005.0	〃
	R3	66	〃	5.6	21.56	1293.00 / 1230	〃
	R4	66	〃	6.3	24.26	1455.6 / 1601.2	〃
116	R5	180	19	2.0	150 / 77 / 3.33	380.28 / 410.6	2.22
	~~R6~~	~~360~~	12	.8	.7	252.0	
	~~R7~~	~~390~~	〃	.6	.53	206.7	
	R8	82	19	7.8	17.32 / 17.87	1420.2 / 1465.4	2.22
				8.05			

(世古ノ設計)

						piece wt
	RS1&2	120	12	.85	.88	897.60	.748
	RS3-4	120	〃	.75	〃	792.00	660
	RS5-6	〃	〃	.65	〃	686.40	572
	RS7-8	〃	〃	60	〃	633.60	528
	RS9-13	270	〃	.55	〃	1306.80	4.84
						~~4316.40~~	
						14.077.2 ~~×2~~	
		6				22154kg/m. 2215444 kg/span	

Summarizing for all span 6 × 2215444 = 13292640 kg

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CALCULATIONS FOR

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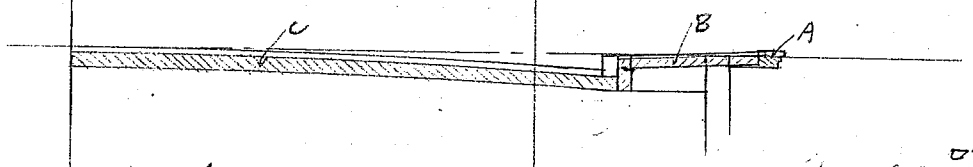
Materials of Jonan Bashi for Kyoto Prefecture

第四編 床版及横桁
第一章 混凝土 (配合 1:2:4) ((neglected drain holes)
第一節 床版



Cross sectional area A =0.062 m²
B .15 × .65 = .098
.11 × 1.00 = .110
.15 × .35 = .053
0.264
C .18 × 14.40 = 2.592

Slab No1. 2-Required

A	2	e	.062 × 2.845 =	.35	
B	2	e	.261 × 3.511 =	1.83	
Bracket	1		½(.68 + 1.10)×.15×1.45 =	.19	on abutment
	1	e	.35 × .20 × 2.75 =	.19	
less	2	e	.10 × .15 × .62 =	-.02 -.07	親柱, 部分
Bracket	1		½(.19 + .60)×.15×1.45 =	.09	
	1	e	.35 × .20 × 1.60 =	1.12 -.11	
C	1	e	2.592 × 3.511 =	9.10	
less	1	e	.20 × .105 × 14.00 =	-.29	踏掛石

1150 (1.15) (1.256) cub. meters
2 e (1.256) = 2.512 - 23.00

Slab No2 6-Required

A	2	e	.062 × 12.52 =	1.55	
B	2	e	.261 × 12.52 =	6.54	
C	2	e	2.592 × 3.511 =	18.20	
	1	o	.23 × 3.30 × 5.498 =	4.17	crown fill, 間

30.46
6 e 30.46 = 182.76

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13 - 18.4 - 54
17 — 24.0 — 7.0

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CALCULATIONS FOR

List of Reinforcements for Slab

Mark	NO	Dia.	Unit wt.	Length	piece wt	Total wt.	Remarks	Slab NO.1 2-Required
F1	78	16	1.57	360	5.65	440.70		
F2	40	〃	〃	375	5.89	235.60		
F3	78	〃	〃	115	1.81	141.18		
F4	78	〃	〃	80	1.26	98.28		
F5	18	12	.88	450	3.96	71.28		
F6	36	〃	〃	610	5.37	247.02	193.32	
F7	16	〃	〃	210	1.85	29.60		
F8A	15	〃	〃	185	1.63	24.45		
F8B	15	〃	〃	145	1.28	19.20		
F9	38	〃	〃	355	3.13	118.94		
F10	42	〃	〃	250	2.20	92.40		
F11	22	〃	〃	225	1.98	43.56		
F13	40	9	.49	110	.54	21.60		
F14	42	12	.88	55	.49	20.58		
F15	42	〃	〃	85	.75	31.50		
						1635.89 × 2 = 3271.78		3164.38
						1582.19		
F16	120	16	1.57	375	5.89	706.80		Slab NO.2 6-Required
F17	60	〃	〃	395	6.20	372.00		
F3	176	〃	〃	115	1.81	318.56		
F5	32	12	.88	450	3.96	126.72		
F6	76	〃	〃	610	5.37	408.12		
F10	168	〃	〃	250	2.20	369.60		
F11	86	〃	〃	225	1.98	170.28		
F12	4	〃	〃	120	1.06	4.24		
F13	168	9	.49	110	.54	90.72		
F14	88	12	.88	55	.49	43.12		
F15	88	〃	〃	85	.75	66.00		
F18	76	〃	〃	400	3.52	267.52		
F19	38	〃	〃	580	5.10	193.80		
F20	26	16	1.57	420	6.59	171.34		
F21	14	12	.88	690	6.07	84.98		
F22	36	16	1.57	410	6.44	231.84		
F23	20	〃	〃	430	6.75	135.00		
F24	15	〃	〃	405	6.36	95.40		
F25	15	〃	〃	550 (365)	550	5.73 / 82.50	85.95	
						3939.54 × 6 = 23631.24		23651.94
						3941.99		
F3	156	16	1.57	115	1.81	282.36		Slab NO.3 5-Required
F4	156	〃	〃	80	1.26	196.56		
F10	90	12	.88	250	2.20	198.00		
F11	46	〃	〃	225	1.98	91.08		
F12	4	〃	〃	120	1.06	4.24		
F13	90	9	.49	110	.54	48.60		
F14	90	12	.88	55	.49	44.10		
F15	90	12	〃	85	.75	67.50		
F26	156	16	1.57	405	6.36	992.16		
F27	80	16	〃	420	6.59	527.20		
F28	8	12	.88	125	1.10	8.80		
F29	22	16	1.57	145	2.28	50.16		
F30	8	12	.88	155	1.37	10.96		
F5	34	〃	〃	450	3.96	134.64		
F6	68	〃	〃	610	5.37	365.16		
F31	8	〃	〃	110	.97	7.76		
F32	76	〃	〃	370	3.26	247.76		
						3277.04 × 5 = 16385.20		
						43288.22 Kgs		43201.52 Kg

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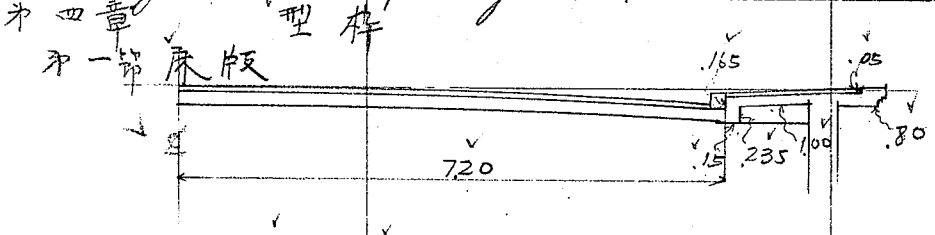
CALCULATIONS FOR

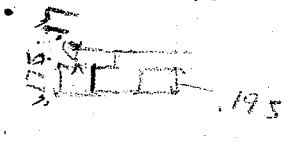
Materials of Jonan Bashi for Kyoto Prefecture.

第四章 型枠
第一節 床版



Slab No 1. 2-Required

Coping + bracket	2 e	.85 × 2.830	=	4.8	
side walk	2 e	1.235 × 3.204	=	7.9	
longitudinal beam	2 e	.15 × 2.736	=	.8	bottome side
roadway	1 e	1440 × 2.736	=	394	
bracket		$\frac{1}{2}$ × (.85 + 1.35) × 1.65	=	1.8	under sidewalk on abutment
〃		$\frac{1}{2}$ × (.30 + .80) × 1.65	=	.9	
〃		(1.65 + 1.35) × .175	=	.5	
〃		(.30 + .80) × .175	=	(2) .3	
〃		.195 × 2.20	=	(43) .4	
〃		.195 × 1.40	=	(27) .3	
curb line		.175 × 6.11	=	1.0	
〃		.175 × 5.16	=	.9	
side	2 ×	.105 × 14.563	=	3.1	62.1 踏掛石



683 sq. meters
2 e 683 = (366) sq. meters
1242

Slab No 2 6-Required

coping and bracket	2 e	.85 × 12.53	=	21.3	
sidewalk + curb line	2 e	1.40 × 12.53	=	35.1	
roadway	2 e	1440 × 2895 2736	=	(823) 788	
Inside	4 e	(5.498 + 5.774) × .18	=	8.1	at crown fill
roadway		2.70 × 5.186	=	14.0	between crown fill
				1608 1573	sq. meters
		6 e 1608	=	9648	sq. meters
				9438	

Slab No 3 5-Required

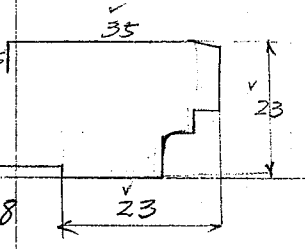
coping	2 e	.05 × 6.71	=	.7	inside
coping + bracket	2 e	.80 × 6.11	=	9.8	
Bracket	2 e	.21 × 1.12	=	.5	side
Bracket	2 e	.24 × .26	=	.1	bottom side
〃	8 e	.13 × .75	=	.8	beam side
〃	4 e	.30 × .88	=	1.1	beam bottom side
sidewalk - curb line	2 e	1.40 × 6.72	=	18.8	
roadway		5.472 × 14.70	=	80.4	included under longinal beam
				1122	
		5 e 1122	=	5610	sq. meters
				1629.0	
				16629	sq. meters

全床版型枠合計

第＝節 横桁 ——(12)ヶ所入
第五章 人造洗出仕上

Slab No 1 2-Required

	2 e	.86 × 2.825	=	4.86	
	2 e	4.86	=	9.72	



Slab No 2 6-Required

	2 e	.86 × 12.52	=	21.53	
	6 e	21.53	=	129.18	

Slab No 3 5-Required

2 e	.05 × 6.71	=	.67		
2 e	.81 × 6.11	=	9.90		
2 e	.26 × 1.06	=	.55		
2 e	.23 × 1.12	=	.52		
2 e	.26 × .24	=	.12		

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CALCULATIONS FOR

materials of Jouan Bashi for Kyoto Prefecture.

4 e .13 × .85 = .44 ✓
4 e .13 × .75 = .39 ✓
4 e 30 × .58 = .70 ✓
1329 ✓
5 e 1329 ✓ = 6645 ✓

Bracket side
bottom side

人造洗出仕上合計 20535 ✓ sq. meters

~~或掛 3. elastic~~

ト.T2	60	@	130×.37	= 6.66
ナT2	10	@	160×.84	= 5.04
bolt nut	12	@	124×18.41	= 53.02
				64.72
				205.35
				140.63

9205
9205
18410

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MADE BY M. Fujida DATE 5-11-26 FILE NO 11
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CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

木ニ節 横桁 (コンクリート)

BM 1 ~~.24~~ .23 × .30 × 2.809 = ~~.24~~ .19

BM 2 .47 × .30 × 2.809 = .40

BM 3 .47 × .30 × 2.809 = .40
½ c .15 × .15 × 2.809 = .03
.43

全横桁合計 BM1 12 c ~~.24~~ .19 = ~~2.88~~ 2.28
BM2 29 c .40 = 11.60
BM3 12 c .43 = 5.16
~~19.64~~ 19.04 cub meters

木ニ節 横桁

BM1 ~~2.0~~ 2 c ~~.23~~ .23 × 2.809 = ~~1.3.16~~ 13
1 c .30 × 2.809 = .8
~~2.4~~ 2.1

BM2c3 2 c .47 × 2.809 = 2.6
1 c .30 × 2.809 = .8
3.4

全横桁合計 BM1 12 c ~~.24~~ .21 = ~~2.88~~ 2.52
BM2c3 41 c .34 = 13.94
~~16.82~~ 16.46 sq. meters

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MADE BY M. Jmida DATE 5-11-23 FILE NO
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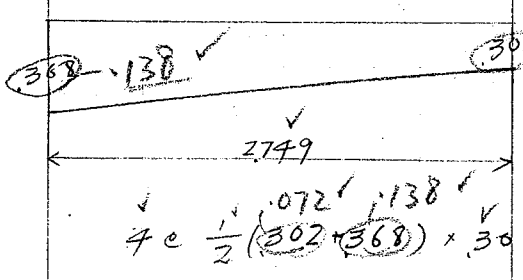
CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

第五編 拱側壁

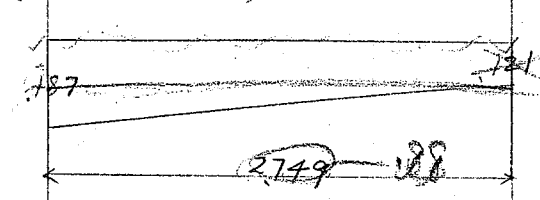
第一章 混凝土 (配合 1:2:4)

内側拱側壁 (一径間分)



4 e $\frac{1}{2}$ (.302 + .368) × .30 × 2.749 = 1.11 cub meters

中部拱側壁 (一径間分)



4 e $\frac{1}{2}$ (.021 + .187) × .15 × 2.749 = .025 cub meters

外側拱側壁 (一径間分) × 2

on abutment 1.723 × 4.69 = .808

on pier 1.738 × .625 = 1.086
1.753 1.096

共通部分

1.472	1.055				
1.457	.84	=	1.224	1.553	1.865 for pier
1.189	1.169	=	.982	.999	1.895 for abutment
.975	.955	=	.802	.819	1.630
.788	.768	=	.645	.662	1.283 1.303
.626	.606	=	.509	.526	1.055 1.075
.489	.469	=	.394	.411	.855 .875
.377	.357	=	.300	.317	.680 .700
.291	.271	=	.228	.246	.534 .551
.232	.212	=	.178	.195	.407 .427
.197	.177	=	.149	.165	.307 .327
.185	.385	=	.069	.071	.235 .255
					.188 .208
					.185
					.185
			5.975	5.962	

1.055 .625 .84 .84 .84 .84 .84 .84 .84 .84 .385 .84

外側径間

2 e .808 × .30 = .48

2 e 1.086 1.096 × .30 = .65 .66

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CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

.077 .163

第三章 型枠

内側拱側壁

8 c ½ (.302 + .368) × 2.749 = 7.4 — 8.3 — 2.9

中部拱側壁

8 c ½ (.121 + .187) × 2.749 = 3.4 — .1

外側拱側壁

外側径間

4 c .808 = 3.2
8 c 5.475 5.962 = 43.8 — 47.7

less	4 c .312 × 1.283	1.303	=	1.6	cross wall
"	4 c .312 × .855	.875	=	1.1	"
"	4 c .312 × .531	.551	=	.7	"
"	4 c .312 × .307	.327	=	.4	"
	4 c 1.086 1.096		=	43 — 4.4	
less	1 c .312 × 1.845		=	.6	
	2 c .10 × 1.845		=	4	
				473 — 51.3	

中部径間

8 c 1.086 1.096 = 8.7 — 8.8
8 c .5475 5.962 = 43.8 — 47.7

less	2 c .312 × 1.845		=	1.2	cross wall
"	4 c .312 × 1.283	1.303	=	1.6	"
"	4 c .312 × .855	.875	=	1.1	"
"	4 c .312 × .531	.551	=	.7	"
"	4 c .312 × .307	.327	=	.4	"
	2 c .10 × 1.845		=	4 — 4.7	
	14			47.9 — 52.2	

全径間合計

2.8
6 c 74 = 444 — 43.8 — 17.4
6 c 34 5.962 = 204 — .6
2 c 473 5.13 = 946 — 102.6
4 c 479 = 1916 — 208.8
5.212

1510 sq. meters
325.8 — 329.4

第四章 人造洗出仕上

外側径間

2 c .808 = 1.62
4 c 5.475 5.962 = 21.90 — 23.85
2 c 1.086 1.096 = 2.17 — 2.19
less 1 c .84 × 1.745 = — 1.47 facing stone
2422 — 26.19

中部径間

1.096
4 c 1.086 = 4.34 — 4.38
4 c 5.475 5.962 = 21.90 — 23.85
2 c .84 × 1.745 = — 2.93
2331 — 25.30

全径間合計

26.19
2 c 2422 = 4874 — 52.38
4 c 2334 = 9324 — 50.60 — 101.20
25.30
14168 sq. meters
102.98 — 153.58

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CALCULATIONS FOR

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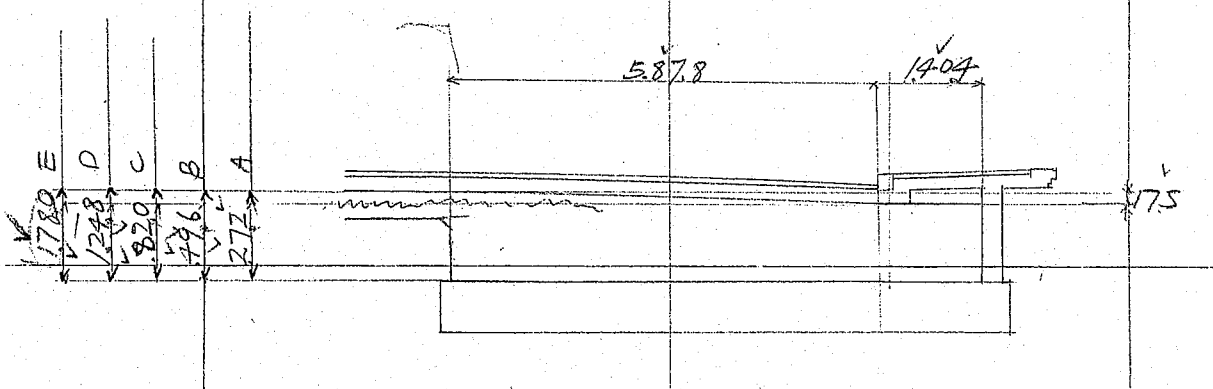
Materials of Jonan Bashi for Kyoto Prefecture

第六綱 支壁
第一章 混凝土 (配合 1:2:4)



A
2 e .097 × .30 × 7.282 = .42
2 e 2/3 × .175 × .30 × 5.878 = .41
.83

B
2 e .321 × .30 × 7.282 = 1.40
.41
1.81

C
2 e .645 × .30 × 7.282 = 2.82
.41
1 e .15 × .15 × 5.878 = .13
3.36

D
2 e 1.073 × .30 × 7.282 = 4.69
.41
5.10

E
2 e 1.605 × .30 × 7.282 = 7.01
.41
7.42

全径間合計 A 1/2 e .83 = 9.96
B 1/2 e 1.81 = 21.72
C 1/2 e 3.36 = 40.32
D 1/2 e 5.10 = 61.20
E 1/2 e 7.42 = 37.10
170.30 cub. meters.

第二章 鉄筋

第三章 型枠

		transverse		spandrel wall	
WT1	28	14	1.70	1.57	
WT2	32	16	1.80	〃	
3	14	7	1.00	〃	

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D 4 e 1.073 × 7.282 = 31.3
2 c 3/2 × .778 = 2.7
5
34.5

E 4 e 1.605 × 7.282 = 46.8
2 c 3/2 × 1.310 = 2.7
8
50.3

全径間合計

A	12 W	e 52	=	624
B	12	e 12.1	=	145.2
C	12	e 21.7	=	~~254.4~~ 260.4
D	12	e 34.5	=	414.0
E	5	e 50.3	=	251.5

~~1127.5~~ sq. meters
1,123.5

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CALCULATIONS FOR

Materials for Jonan Bashi for Kyoto prefecture.

(sheet no 6)

第七節 排水工

第一節 橋脚排水工 (10箇所)

第一項 鋳鉄蓋

15×15.6×2 − .8×11×7×2 = 345 cm³

1個当り重量 345×.00725 = 2.501 kg

合計重量 2.501×10 = 25.010 kg.

第二項 亜鉛管 3"φ × 2.2" — 10本

1本重量 7.575×2.2×1.488 = 24.798 kg

合計重量 24.798×10 = 247.980 kg. ~~0.248 ton~~

第二節 橋面 排水工 (24箇所)

第一項 鋳鉄

a. 蓋 35×18×2.5 − .62×9×12×25 = 1035.0 cm³

1個重量 .00725×1035.0 = 7.504 kg

b. 漏斗 2.5×(33+16.7)×2×2 = 497 cm³

1.2×[(33+16.7)×2 + 11.2×π] ÷ 2/3 × 15 = 16196 cm³

11.2×π×1.2×5 = 211.12 cm³

2323.08 cm³

1826.08

1個重量 .00725×1826.08 = 13.239 kg

c. パイプ 11×π×1×131. = 4527.05 cm³

1個重量 .00725×4527.05 = 32.82 kg

1組重量 46.810 kg

合計重量 96.0×10×24 = 23040 kg

1,123.44 kg

1,372.01

第二項 排水枡 鉄筋混凝土 (wire mesh 含む)

配合 1:1:2 表面 finish

(.065+.055) ÷ 2 × .2 × (.060+.018×2) = .0007 .0069 m³

合計体積 .0007×24 = .017 m³

.0069

0.166

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CALCULATIONS FOR

Materials of Iwau Bashi for Kyoto Prefecture

/

第八編 高欄材料計算 (sheet no 12 参照)

第一章 親柱一基ニ付材料

第一節 石材立積 (花崗石)

石材立積計算表 (立積小数以下三位迄以下切捨ト)

		長(米)	中(米)	厚(米)	一個ニ付積	数量個	立積(立方米)
縁石		1.77	.27, .26	.21	.042	1	.042
〃		1.38	.27, .26	.21	.075	1	.075
第一列	(下ヨリ)	1.40, 1.41	.45	.45	.286	4	1.144
〃		1.46, 1.41	.30	.30	.127	4	.508
第二列		.15	.33, .33	.33, .33	.016	4	.064
〃		1.30—1.70	.30	.30	.117	4	.468
〃		1.55	.25	.25	.097	4	.388
第三列		.90	.25	.30	.068	2	.136
〃		.90	.25	.70	.158	2	.316
第二列ニ入ル		.40	.30	.30	.036	4	.144

3.285 cub. meters

第二節 石材仕上計算

第一款 表面及合端仕上計算 (小叩仕上)

縁石		48.47 × 2.14	1.97 =	.93	(表面)
〃		out 2.18 × .27, .26 =		.05	〃
〃		2.03 × 2.14	5.67 = 6.30	.17—.20	(合端)
第一列		4@ 1.06 × 1.41 =		5.98	(表面)
〃		4@ .12 × .78 =		.37	〃
〃		4@ .30 × 1.41 =		1.69	〃
〃		4@ .03 × 4.32 = 4.00		.43—.48	(合端)
〃		4@ .03 × 3.42 =		.41	〃
〃		~~8@ .03 × 1.41 =~~			
第二列		4@ .15 × 20 .33 = 66		.20—.53	(表面)
〃		4@ .50 × 1.55 =		3.10	〃
〃		4@ .54 × 1.70 =		3.67	〃
〃		4@ .30 × .12 =		.14	〃
〃		4@ .05 × .50 =		.10	〃
〃		4@ .03 × 1.40 =		.17	(合端)
〃		4@ .03 × 3.95 = 3.40		.40—.47	〃
〃		4@ .03 × 5.16 =		.62—.65	〃
第三列		2@ .10 × 1.30 .90 =		1.98	(表面)
〃		2@ .21 × .70 =		.29	〃
〃		2@ .30 × .90 =		.54	〃
〃		2@ .30 × .30 =		.18	〃
〃		2@ .03 × 3.60 =		.22	(合端)
〃		2@ .03 × 3.00 =		.18	〃

2.02 sq. meters
~~2.98~~ 22.33

第二款 (合端) 中切仕上計算

縁石	2@ .15 × .18 =		.05	
〃	2@ .23 × .18 =		.04—.08	
第一列	8@ .30 × 1.41 1.38 =		~~.38~~—2.98	
〃	8@ .27 × .41 =		~~.05~~—2.98	
〃	4@ .27 × .30 =		.32	
〃	4@ .30 × .30 =		.36	
第二列	4@ .30 × .30 =		.36	
〃	~~4@ .24 × .24~~		~~.23~~	
〃	4@ .22 × .22 =		.19—.39	

8

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CALCULATIONS FOR

Materials of Jōnan kaikan for Kyoto prefecture. 2

木ニ引	8v@	.22 ×	1.70 =	2.99	
〃	8v@	.15 ×	1.70 =	2.04	
〃	12v@	.27 ×	.30 =	.97	
木三引	4@	.19 ×	.60 =	~~.23~~ .46	
〃	2@	.15 ×	.25 =	.08	
〃	8'@	.20 → .17 ×	.90 =	~~1.44~~ 1.22	

(573) sq. meters
15.28

第三節 混凝土 (配合 ~~1:2:4~~ 1:3:6) (Sheet 12 参照)

.40 × (40) × 4.00 = (64) Cub. meters
.4025 .6400

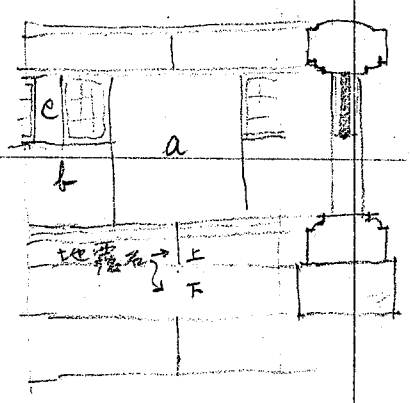
第四節 橋名鈑 (Sheet no. 12 参照)
青銅製橋名及竣功年月日鈑取付金具共 1枚

第五節 電灯 (Sheet no. 12 参照)
青銅製具ランプ・ガラス及附属品共 上部 1箇
下部 4箇
1式
親柱内部配線用ガスパイプ

第貳章 袖高欄 壱組当り材料 (上流及下流ニテ壱組) (長サ3.5ᵐ ト 6.5ᵐ ニテ壱組)
第壱節 石材立積 (花崗石)
石材立積計算表

石材名称	長サ(米)	巾(米)	厚(米)	一個当立積(立方米)	個数(個)	立積(立方米)
笠石(円形)	.6	.22	.15	.020	1	.020
〃 (〃)	1.0	.26	.15	.039	3	.117
〃	.8	.12	.15	.024	1	.024
a (円形)	.58	.53	.14	.043	4	.172
b (〃)	.38	.54	.14	.029	3	.087
b	.38	.48	.12	.022	1	.022
a (親柱寄)	.58	~~.32~~	.12	.023	1	.023
a (袖柱寄)	.58	.12	.085	.006	1	.006
c (円形)	.20	.115	.095	.002	3	.006
c	.20	.11	.09	.002	1	.002
地覆石(上流側形)	.60	.22	.12	.016	1	.016
〃 (〃)(〃)	1.0	.26	.12	.031	3	.093
〃 (〃)	.8	.20	.12	.019	1	.019
地覆石(下流)(円形)	1.09	.30	.16	.052	1	.052
〃 〃 〃	1.0	.29	.16	.046	~~3~~	~~.092~~ .138
〃 〃 〃	~~1.37~~ 1.25	~~.28~~ .29	.16	~~.059~~ .012	1	~~.059~~ .012
袖柱	1.36	.60	.47	.384	1	.384

1.194 立方米
3



笠石(円形)	.6	.22	.15	.020	1 .020
〃 〃	1.0	.26	.15	.039	3 .117
〃	.95	.20	.15	.029	4 .116
a (円形)	.58	.53	.14	.043	4 .172
a (親柱寄)	.58	.327	.12	.023	1 .023
a (袖柱寄)	.58	.235	.12	.016	1 .016
a	.58	.47	.12	.033	3 .099

長サ3.5ᵐ一ケ

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CALCULATIONS FOR

隅川橋

3

長H 6.5 分	b (円形)	.38 ✓	.54 ✓	.14 ✓	.029 ✓	3 ✓	.087 ✓		
	b	.38 ✓	.48 ✓	.12 ✓	.022 ✓	4 ✓	.088 ✓		
	c (円形)	.20 ✓	.115 ✓	.095 ✓	.002 ✓	3 ✓	.006 ✓		
	c	.20 ✓	.11 ✓	.09 ✓	.002 ✓	4 ✓	.008 ✓		
	地覆石(上部)(円形)	.60 ✓✓	.22 ✓	1/2 ✓	.016 ✓	1 ✓	.016 ✓		
	〃 〃 〃	1.0 ✓✓	.26 ✓	1/2 ✓	.031 ✓	3 ✓	.093 ✓		
	〃 〃	.95 ✓	.20 ✓	1/2 ✓	.023 ✓	4 ✓	.092 ✓		
	〃 (下部)(円形)	.09 ✓	.30 ✓	.16 ✓	.052 ✓	1 ✓	.052 ✓		
	〃 〃 〃	1.0 ✓	.29 ✓	.16 ✓	.046 ✓	2 ✓	.092 ✓		
	〃 〃 〃	.98 ✓	.27 ✓	.16 ✓	.042 ✓	1 ✓	.042 ✓		
	〃 〃	.95 ✓	.24 ✓	.16 ✓	.036 ✓	3 ✓	.108 ✓		
	〃 〃	.475 ✓	.24 ✓	.16 ✓	.018 ✓	1 ✓	.018 ✓		
	袖柱	1.36 ✓	.60 ✓	.47 ✓	.384 ✓	1 ✓	.384 ✓		
							1.649 主方米		

石材主積 合計

長 3.5 分 — 1.194 主方米
〃 6.5 分 — 1.649
2.843

第貳節 石材仕上計算

才卷款 表面及合端共仕上計算 (小叩仕上)

長H 3.5 分	笠石		.58 ✓ ×	4.212 ✓	=	2.44 ✓		表面
	窓内面	9@	.10 ✓ ×	.75 ✓	=	.68 ✓		〃
	a	2@	.58 ✓ ×	2.107 ✓	=	2.44 ✓		〃
	b	2@	.38 ✓ ×	2.105 ✓	=	1.60 ✓		〃
	c	8@	.11 ✓ ×	.120 ✓	=	.18 ✓		〃
	袖柱	2@	1.07 ✓ ×	1.36 ✓	=	1.46 2.91 ✓		〃
	〃		.47 ✓ ×	.60 ✓	=	.28 ✓		〃
	地覆石		.68 ✓ ×	4.212 ✓	=	2.86 ✓		〃
	笠石	10@	.03 ✓ ×	.58 ✓	=	.17 ✓		合端
	窓内面	9@	.02 ✓ ×	.77 ✓	=	.14 ✓		〃
	a	4@	.03 ✓ ×	3.64 ✓	=	.44 ✓		〃
	〃 (親柱寄)		.03 ✓ ×	3.00 ✓	=	.09 ✓		〃
	〃 (袖柱寄)		.03 ✓ ×	2.40 ✓	=	.07 ✓		〃
	b	4@	.03 ✓ ×	2.60 ✓	=	.31 ✓		〃
	c	16@	.03 ✓ ×	.28 ✓	=	.13 ✓		〃
	地覆石	10@	.03 ✓ ×	.30 ✓	=	.09 ✓		〃
	〃	10@	.03 ✓ ×	.35 ✓	=	.11 ✓		〃
	〃	6@	.03 ✓ ×	4.212 ✓	=	.76 ✓		〃
	笠石	2	.03 ✓ ×	4.814 ✓	=	.14 ✓		〃
				工17		1439 sq. meters		
						1.584 ✓		
長H 6.5 分	笠石		.58 ✓ ×	7.212 ✓	=	4.18 ✓		表面
	窓内面	15@	.10 ✓ ×	.75 ✓	=	1.13 ✓		〃
	a	2@	.58 ✓ ×	3.667 ✓	=	4.25 ✓		〃
	b	2@	.38 ✓ ×	3.545 ✓	=	2.69 ✓		〃
	c	14@	.11 ✓ ×	.20 ✓	=	.31 ✓		〃
	袖柱	2@	1.07 ✓ ×	1.36 ✓	=	1.46 2.91 ✓		〃
	〃		.47 ✓ ×	.60 ✓	=	.28 ✓		〃
	地覆石		.68 ✓ ×	7.212 ✓	=	4.90 ✓		〃
	笠石	16@	.03 ✓ ×	.58 ✓	=	.28 ✓		合端
	窓内面	15@	.02 ✓ ×	.77 ✓	=	.23 ✓		〃
	a	7@	.03 ✓ ×	3.64 ✓	=	.76 ✓		〃
	〃 (親柱寄)		.03 ✓ ×	3.00 ✓	=	.09 ✓		〃
	〃 (袖柱寄)		.03 ✓ ×	3.00 ✓	=	.09 ✓		〃

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CALCULATIONS FOR 陽明橋

4

	b	7@	.03 ×	2.60	=	.55	合端
	c	28@	.03 ×	.28	=	.24	〃
	地覆石	16@	.03 ×	.66	=	.32	〃
	笠石(下端)	6@	.03 ×	7.212	=	1.30	〃
			.03 ×	8.30	=	.25	〃
				石材仕上合計		2.33 平方米	2.476
					3.5.分	1.439 平方米	1.584
					6.5.分	2.331	2.476
						3.770	4.060
本或款	笠石	10@	.08 ×	.13	=	.10	
	〃		.06 ×	1.807	=	.11	
	〃		.03 ×	.20	=	.01	
	a	4@	.06 ×	1.58	=	.38	
長 3.5.分	a(親柱寄)		.06 ×	1.27	=	.08	
	〃(袖柱寄)		.06 ×	1.07	=	.06	
	b	4@	.06 ×	1.18	=	.28	
	〃	4@	.03 ×	.05	=	.01	
	c	4@	.03 ×	.1	=	.01	
	地覆石		.06 ×	4.212	=	.25	
	〃	2c	.14 ×	4.212	=	.59	1.13
	〃	10@	.18 ×	.25	=	.45	
	袖柱下面周圍					2.33 平方米	
						0.95	2.92
	笠石	16@	.08 ×	.13	=	.17	
	〃		.06 ×	2.99	=	.18	.19
	〃		.03 ×	.35	=	.01	
	a	7@	.06 ×	1.58	=	.66	
6.5.分	a(親柱寄)		.06 ×	1.27	=	.08	
	a(袖柱寄)		.06 ×	1.37	=	.08	
	b	7@	.06 ×	1.18	=	.50	
	b	7@	.03 ×	.05	=	.01	
	c	7@	.03 ×	.10	=	.01	0.02
	地覆石		.06 ×	7.212	=	.43	
	〃	2c	.14 ×	7.212	=	1.01	2.02
	〃	8@ 16	.18 ×	.25	=	.36	.72
						3.7 平方米	
	中切仕上合計				3.5.分	2.33 平方米	5.08 4.89
					6.5.分	3.70	5.08 4.89
						6.03	8.00 7.81

第三節 鑄鉄格子及装飾 (Shut no. 12 参照)

鑄鉄格子箝込共 24枚
青銅製装飾鈑箝込共 4枚

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CALCULATIONS FOR 鉄 橋上高欄両 両全部 部材料

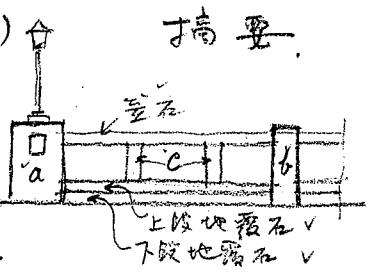
第三章 橋上高欄両側分(全部)材料計算 (sheet no. 13 参照)

第一節 石材立積 (花崗石)

石材立積計算表

石材名称	長(米)	巾(米)	高(米)	一箇割立積(立方米)	箇数(個)	立積(立方米)
笠石	2.78	.20	.15	.083	72	5.976
柱 (a)	1.23	.84	.60	.620	10	6.200
間柱(b)	1.08	.37	.30	.120	60	7.200
電燈台石(c)	.60	.18	.15	.016	144	2.304
上段地覆石	.895	.20	.12	.021	144	3.024
〃	.97	.20	.12	.023	72	1.656
下段地覆石	1.38	.24	.16	.053	144	7.632
						33.992 立方米

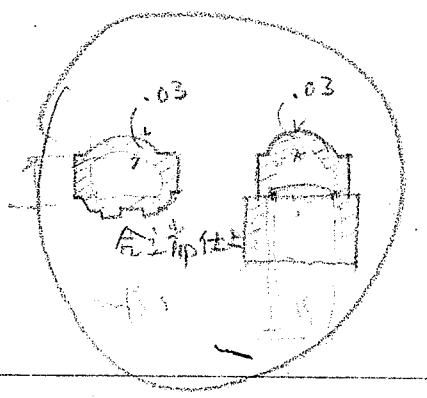
摘要



第二節 石材仕上計算

第一款 表面及合端仕上計算 (小叩仕上)

笠石及地覆石	72 @ 1.14	×	2.76	=	226.54 / 223.26	表面
〃	72 @ .36	×	2.46	=	63.76	〃
柱 a	10 @ .84	×	3.06	=	25.70	〃
〃 b	10 @ .60	×	2.46	=	14.76	〃
間柱	60 @ .37	×	2.46	=	54.61	〃
電燈台石	60 @ .30	×	2.16	=	38.88	〃
〃 c	144 @ .66	×	.58	=	55.12	〃
笠石及地覆石小口	72 @ .03	×	4.88	=	10.54	合端
b伸縮目地面	48 @ .30	×	1.08	=	15.55	〃
c	72 @ .03	×	4.80	=	10.37	〃
					512.55 平方米	
					515.83	



第二款 (実際) 中切仕上計算

笠石及地覆石小口	72 @ .15	×	1.26	=	13.61	
c	72 @ .08	×	.96	=	5.53	
上段地覆石	72 @ .14	×	2.76		27.82	
					46.96 平方米	

第三節 鋳鉄格子 (Sheet no. 13 参照)

鋳鉄格子 216 枚

△ ネーヂュカン入

第四節 青銅装飾鈑 (Sheet no. 13 参照)

青銅装飾鈑 20 枚

第五節 電灯 (Sheet no. 13 参照)

青銅金具並ガラス及附属品共 10 基

注意 b柱, 伸縮目地用エラスタイト, 床版ヲ第四節 (Page no.) = 記載ス

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CALCULATIONS FOR Materials of Torun Bashi for Kyoto prefecture

第2カ所(鋳鉄)

高欄格子 ペット塗面積(全表面) 1枚分.

			m²
主板	外縁(表裏) 2.32×.03×2	=	.1692 → .1656
〃	(外縁) 2.88×.03	=	.0864
〃	(内〃) 2.55×.03	=	.0765 → .0747
中桟	縦桟 .78×2+.67+.56+.16	=	2.85 → 2.357
〃	縦〃 .125×3+.355×4+.21×2	=	2.215 → 2.115
	面積(表裏) 5.07×.015×2	=	4.87 → 4.877
	〃 (内縁) (5.07×2−.015×5.2)×.03	=	.1461
			.1421
			.2808 → .2688
			.7650 m²
			.7416

一枚分表面積

第一カ所

高欄鋳鉄格子 重量 1枚分

主桟 外縁 276×3×3 = 2484.
横桟 487×1.5×3 = 2192
4676 cm²

重量 .00725×4676 = 33.901 kgo

割り形 形 型 枠

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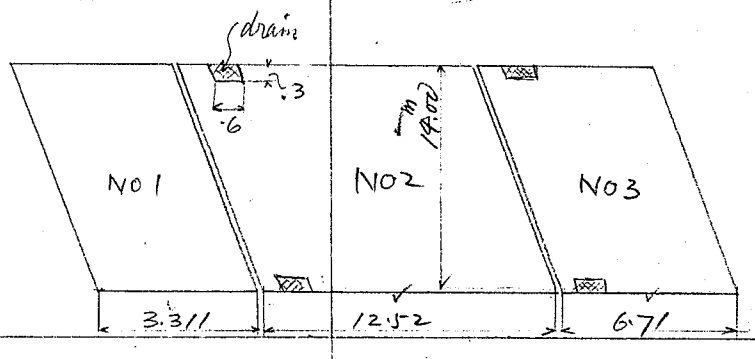
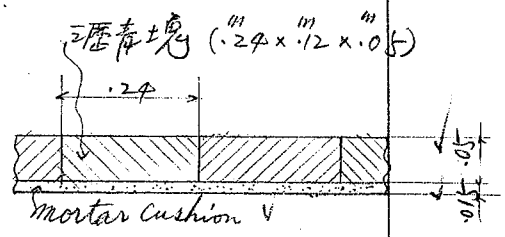
CALCULATIONS FOR

Materials of Yonan Bashi for Kyoto prefecture

第九編 橋面鋪装 (sheet no 4 参照)
第一節 瀝青塊鋪装(車道)





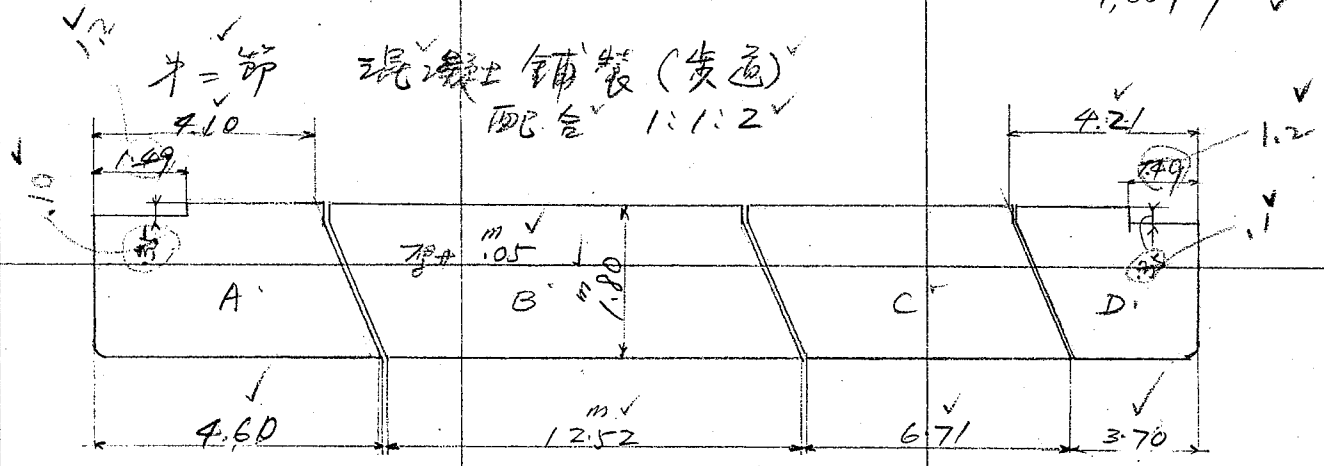
面積 NO1 3.311 × 14.00 = 46.4 m²

NO2 12.52 × 14.00 = 175.3
less .6×.3×2 = −.4
174.9 m²

NO3 6.71 × 14.00 = 93.9
−.4
93.5 m²

面積合計
NO1 2@ 46.4 = 92.8
NO2 6@ 174.9 = 1,049.4
NO3 5@ 93.5 = 467.5
1,609.7 m²

第二節 現場打鋪装(歩道)
配合 1:1:2



A = (4.60+4.10)/2 × 1.80 = 7.83 m²
less 1.49 × .1 × 1.2 = −.12
7.71

B = 12.52 × 1.80 = 22.54

C = 6.71 × 1.80 = 12.08

D = (4.21+3.70)/2 × 1.80 = 7.12
less 1.49 × .1 × 1.2 = −.12
7.0

面積合計
A 2@ 7.71 = 15.42
B 12@ 22.54 = 270.48
C 10@ 12.08 = 120.80
D 2@ 7.0 = 14.0
420.7

体積合計 = 419.10 × .05 = 20.96 m³

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CALCULATIONS FOR

Materials of Jōran Bashi for Kyoto prefecture.

石材体積

	名称	断面積(m²)	長(m)	員数	1個当体積(m³)	合計体積(m³)
slab NO1	a	.036	1.374 1.397	2	.049 .050	.098 .100
	b	.37×.20	.700	2	.052 .0049	.104 .098
	c .35	.036	.906 .944	1	.0324	.0324
	d	〃	1.700	2	.061	.122
	e & e'	〃	1.667	2	.060	.120
						.474
						.474×2=.948 m³
slab NO2	6 Regid					
	e & e'			4	.060	.240
	f	.036	1.530	12	.055	.660
						.900×6=5.400 m³
slab NO3	5 Regid					
	e & e'			4	.060	.240
	d			4	.061	.244
						.484×5=2.42

合計体積 8.774 m³ 8.768

加工面積

素面及合端3cm巾 slab NO1 (1.18+.05+.20)×(6.667+5.761)=5.340 m² = 5.349×2=10.69 10.68
〃 NO2 (〃 〃 〃)×12.464×2 = 10.719×6=64.31
〃 NO3 (〃 〃 〃)×6.686×2 = 5.750×5=28.75
103.78 m² 4

合端ICC 上切 slab NO1 .18×.20×18 = .648×2=1.30
〃 NO2 〃 〃 ×32 = 1.152×6=6.91
〃 NO3 〃 〃 ×16 = .576×5=2.88
11.09 m²

裏面ICC 元切 slab NO1 (.15+.18)×(6.667+5.761)=.4098 ×2=8.20
〃 NO2 (〃 〃)×12.464×2 = 8.226×6=49.36
〃 NO3 (〃 〃)×6.686×2 = 4.413×5=22.07
79.63 m²

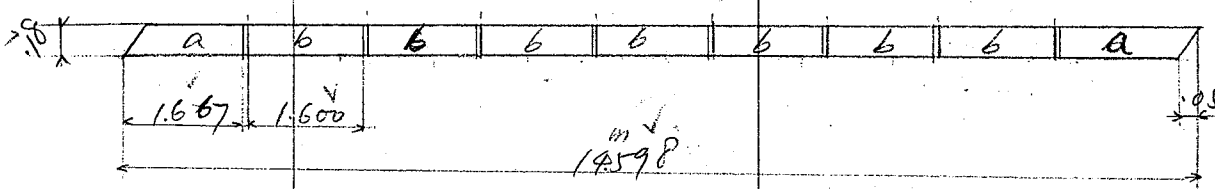
指付モルタル (1:3)
下敷モルタル(底面) 厚0.015 m 0.02 4.8 3.868 12.13
連結用モルタル 厚0.008 m
土留厚を足す 4.807+3.893+1.720×2=12.14×2=24.28 24.26
12.52×2 = 25.04×6=150.24
6.71×2 = 13.42×5=67.10
241.62 m 241.60

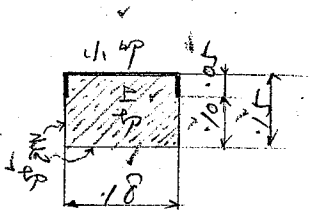
Joint金敷 (5+4)×2+7×2×6+3×2×5=32 個所

モルタル体積 .02 .015×(.20+.20)×241.60 = .1450 m³ 1.933
.18×.20×.008×132 = .038
.1488 m³ 1.971

高沢

歩=2尺 踏掛石 (花崗石)両詰分 目地 8mm (Shut no.5 表2)





断面
太線ハ小口+両小口ハ上ヤ
他ハ元ヤリトス.

石材体積 a .18×.15×1.667 = .045×4 = .180
6 〃 〃 ×1.600 = .043×14 = .1602
14484 .782 m³

加工面積 小口 (.18+.05+.05)×14534×2 = 8.139 m² 8.111
上ヤ .18×.15×18 = .486
元ヤ (.18+.10+.10)×14534×2 = 11.046 11.008
14484

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CALCULATIONS FOR

Materials of Jōsan Bashi for Kyoto prefecture

拱柱ヶモルタル (1:3)
下敷モルタル及横面 厚サ 0.02 連続用モルタル 厚サ .008
体積 .02×(.108+.18)×14.5-64×2 = .168 m³
.008×.18×.15×10×2 = .004
.172 m³

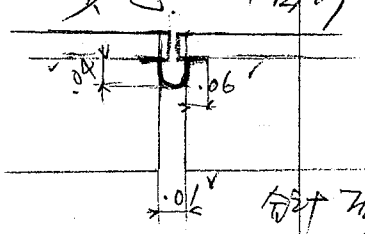
半回節 伸縮用 Elastite

第一級 床版 joint 12箇所 (厚サ 0.01) (Shut no. 5 参照)
3/8 inch
平均高ハ 0.15 ×12
合計 面積 0.245×16.02×12 = 47.099 → 28.836 m²
23+35 24
款
拱側壁 両側

第二級 支壁 24箇所 (厚サ 0.01) (Shut no 7 参照)
巾 .47 高サ .9
合計面積 .47×.9×24 = 10.152 m².

第三級 高欄 joint 24箇所 (厚サ 0.01) (Shut no. 13 参照)
合計面積 .30×1.05×24 = 7.56 m²
面積総合計 64.811 m² → 46.548

半回節 伸縮用 Joint金物 (銅板) 一組 (Shut no. 5 参照)
支点 24箇所



延巾 .21 一本ノ長サ 2.13
単位重量 4.882 kg/m
合計面積 .21×2.13×24 = 10.735 m²
重量 4.882×10.735 = 52.408 kg

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MADE BY Kudara DATE 5-10-18 FILE NO 1
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CALCULATIONS FOR

Preliminary Design of Kamogawa Bashi for Kyoto Inc.

Approximate Dead Load

Floor.

5cm Asphalt block pavement	@ 21	=	105	
15cm mortar cushion	@ 17	=	25.5	
18cm Concrete slab	@ 24	=	432.	
misc conc.			17.5	
			580.0	kg/m²

Floor and pavement	Roadway	5.5 @ 580	=	3190	for one ring
	side walk.	2.0 @ 32	=	64	
	Curb stone	.18×.25 @ 2600	=	117	
	under curb.	.30×.25 @ 2400	=	180	
	coping	.35×.35 @	=	294	
	Handrail	.18×1.0 @ 26m	=	468	
	water main, gas pipes etc say		kg × 40m =	200	
				4515	
				85	
				4600	kg/linear of span for one ring

for one Panel 1.5m @ 4.6m = 6900

Longitudinal Wall.

D.35×4 = 1.40m thick say.

		Thickness	Length	Height			
crown	A	1.40	× 1.5	× .15×½ =	1.60	@ 24m =	880 kg.
	B	"	"	× .20 =	1.20		1010
	C	"	"	× .30 =	1.80		1510
	D	"	"	× .40 =	2.40		2020
	E	"	"	× .70 =	4.20		3530
	F	"	"	× 1.2m =	2.52		6050
Spr	G	"	× 1.33	× 1.6 =	2.96		7300
					9.00	m³.	

Arch Ring	A	25m × 6.3/4.0	=	3940 kg for one ring
	B	5050		7960
	C	5500		8660
	D	6050		9520
	E	7m		11350
	F	8.5m		13380
	G	6m		9450

Summary for panel loads (for one ring)

	Spr. G	F	E	D	C	B	crown A
Floor	5300	6900	6900	6900	6900	6900	3450
Longitudinal wall	7300	6050	3530	2020	1510	1010	880
arching	9450	13380	11350	9520	8660	7960	3940
	22050	26330	21780	18440	17070	15870	8270
call this	22100	26300	21800	18400	17100	15900	8300

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CALCULATIONS FOR

Kanagawa Bashi

Dead load Thrust = moment Thrust.

Panel	load	Ho unit	Ho.
A	8300	2.930	24320
B	15900	2.714	43100
C	17100	2.102	35900
D	18400	1.311	24120
E	21800	.624	13600
F	26300	.143	3760
G	22100	0	0
	129 tm		144800 kg
			1448 tm
			289,600

H end = 2896 tm × 1.0 = 2896 tm kg

Head = 2896 tm × 0.935 = 270800
46 tm
317,000 kg.

V. Sum Σ = 0 + 0 = 0

V sum Σ = 1299 tm × .035 = 46 tm

Hor. thrust = 289,600 kg = Hc
nor. thrust at G = 317,000 "

Moments at crown A.

panel	load	M int	m.
A	830 tm	+.712 m	+5910
B	1590 tm	+1.124	+1970
C	1710 tm	-.119	-2035
D	1840 tm	-.157	-2890
E	2180 tm	-.096	-2090
F	2630 tm	-.026	-685
G	2210 tm	0	0
			+7880
			-7700
			180
			180
			+360 kgm

moment at spring G.

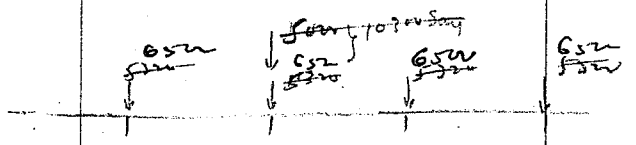
G			
E	263 tm	-1.209	-31780
E	218 tm	-1.643	-35800
D	184 tm	-1.489	-27400
C	171 tm	-.724	-12380
B	159 tm	.206	+3280
A	166 tm	1.044	+17320
B	159 tm	1.498	+23850
C	171 tm	1.426	+24380
D	184 tm	.995	+18300
E	218 tm	.511	+11150
F	263 tm	.129	+3390
G			+101670
			-107360
			-5690 kgm

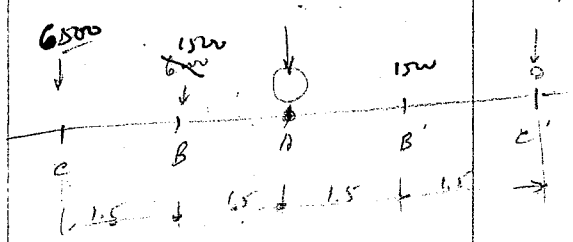
Live Load.

unif. load = 600 / 500 / 400 / 500 kg/sq m roadway.

5.5 @ 6 tm = 33 tm
2 @ 4 tm = 8 tm
41 tm
43 tm kg × 1.5 = 6450 / 5320 kg. = panel load one ring

6.60 × 6 m × 5.5 × 4 = 218 tm = 14,832
8,000 × 2 × 1.25 = 20,000
5,150 kg single concentration





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CALCULATIONS FOR

Kanegawa Bashi.

Live load moment + Thrust for Crown

Panel pt.	load		M_A			X_A	
B	~~6000~~ 1500	+ .124	745	~~190 1086~~	2.714	~~16300~~	4070
A	~~15000 24000~~	+ 0.712	10700	17100	2.930	43900	70,300
B'	1500	+ .124	—	~~190 1086~~	2.714	1050	4070
C'	7500	− .119	−595	−890	2.102		15750
			+ ~~10850~~ 16590	kgm		~~70700~~ 94190	kg.

Spring G.

Moment

C	~~32006500~~	− .724	− 3850	4700	
D	~~3200 1500~~	− 1.489	− 7920	2230	
E	~~10300 24000~~	− 1.643	− 16930	39450	
F	~~4300 1500~~	− 1.209	− 6890	1810	
			~~− 35090~~	kgm.	
			− 48,190		

Thrust

			H		V.					
C	~~4320 6500~~	2.102	~~4,190~~ 1380		+ .214	1140 1490	H cos α	30960 ~~35350~~	× .995 =	~~33700~~ 1890
D	~~4320 1500~~	1.311	6980	1,970	+ .112	600 170	V sin α	~~2270~~ 2810	× .315 =	~~800~~ 1100
E	~~10300 24000~~	1.624	6420	14970	+ .047	480 1130			N =	~~29500~~ kg
F	~~4320 1500~~	.143	760	220	+ .009	50 70				29,900
			~~35350~~	30960		~~2270~~ 2810				

Summary for moments + thrusts

	Crown		Springing	
	moment	normal thrust	moment	nor. thrust
D.L,	+ 360	2896 kg	−5690	317 kg
L.L.	16590	~~94190~~ 29900	− ~~37810~~ 35080	~~24300~~ 24300
	+ ~~10850~~	~~70700~~		
	+ 11210 kgm.	360,300 kg	− ~~40,180~~ kgm	~~54,500~~ 541,500 kg
for one meter of rib	16950	383,790	53,880	346 kg
~~(6.0 m)~~ 5.5	+ ~~1870~~	60 m	− ~~6800~~	~~5700~~
	+ 3080 kgm	+ ~~69.8~~ m kg	− 98 m	63 m

Average thrust

Equiv. Crown Area 0.425

Average thrust $\frac{60980}{60000} \times 1.85 = \frac{120000}{1.425}$ kg/m²
140 m²

For 2@6.00 m ribs

9 m.

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CALCULATIONS FOR

Kamogawa bashi

Stresses at crown (for 1 m strip).

	thrust	moment	average stress
D.L+L.L.	~~69800~~ 60000	~~3080~~ +1870	~~140.0 kg~~ 120.0
Temp.	− 15.6 m'	+4060	− 15.9 m'
rib shr.	− ~~7210~~ 6070	+1880	− ~~9270~~ 6.2 m'
	~~38320~~ 46990	+7510	+~~97900~~ 116720
		9020	

$\frac{15940}{20.2000 + 15940} = .0394$

$\frac{116720}{979 \text{ m}} \div 25^{2} \text{ cm} = .463$ ~~.589~~

$\Sigma cc = \frac{9020}{\frac{75^{10}}{28330} + \frac{}{46990}} = .192$ ~~.196~~

$\Sigma/h = \frac{.192}{.350} = .55$ ~~.57~~ $d'/h = \frac{5}{35} = .143$ ✓

$P_0 = 2p = \frac{50}{35m} = 0.0143$

$k = .150$ ✓ $\angle = 11°33'$ ✓

$f_c = \frac{9020.00}{\frac{75^{1}000}{11335 \times 100 \times 35^{2}}} = \frac{54.2}{48.3}$ kg/cm² < 56.2

$f_s = 15 \times \frac{54.2}{48.3}\left(\frac{30}{.50 \times 35} - 1\right) = \frac{580}{486}$ kg/cm²

Stress at springing

	thrust	moment	average stress
D.L+L.C.	~~63000~~ ~~57000~~	98 m	~~133.0 m~~ ~~114.0 m~~
Temp.	− 14.6 m'	− ~~68.0 m~~ 21.00	− 15.9 m'
rib shr.	− ~~6720~~ 5220	− ~~9310~~ 3.7 m	− ~~6820~~ 5.8 m
	~~37060~~ 41970	~~356 m~~ 40210	~~92780~~ 110150

$\frac{110150}{92280} \div 25^{2} \text{ cm} = .441$ ~~.366~~

$\Sigma cc = \frac{40210}{\frac{33^{5}60}{37060}} = .960$ ✓

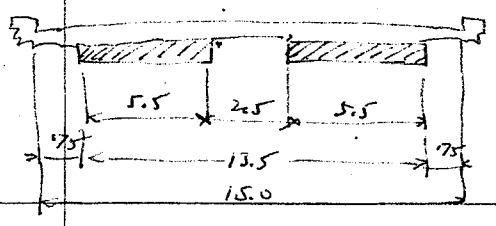
$z/h = \frac{1960}{.70} = 1.375$ ✓ $d'/h = .071$ ✓

$P_0 = .0100$ ✓ $k = .334$ ✓ $\angle = .147$ ✓

$f_c = \frac{40210 \text{ m}}{\frac{33^{5}6 \text{ m}}{.147 \times 100 \times 70^{2}}} = \frac{55.8}{49.4}$ kg/cm² ✓ ok < 56.2 ✓

$f_s = 15 \times \frac{55.8}{49.4}\left(\frac{.65}{.334 \times 70} - 1\right) = \frac{1490}{1318}$ kg/cm² $\frac{1.13}{1.28}$ < 1500

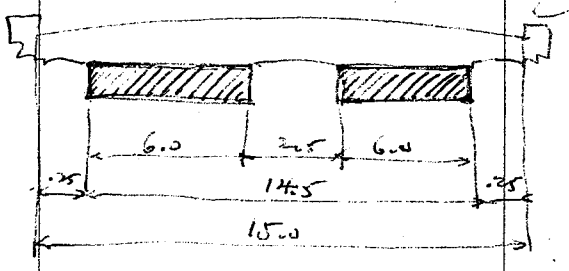
(Each rib assumed 2 @ 5.5" = 11.0" wide)



For 6.0" ribs

Crown stress $f_c = 49.7$ kg/cm² — 89% of allowable working stress
$f_s = 530$ "

Springing $f_c = 51.1$ kg/cm² } 91% of allowable strength
$f_s = 1365$ "



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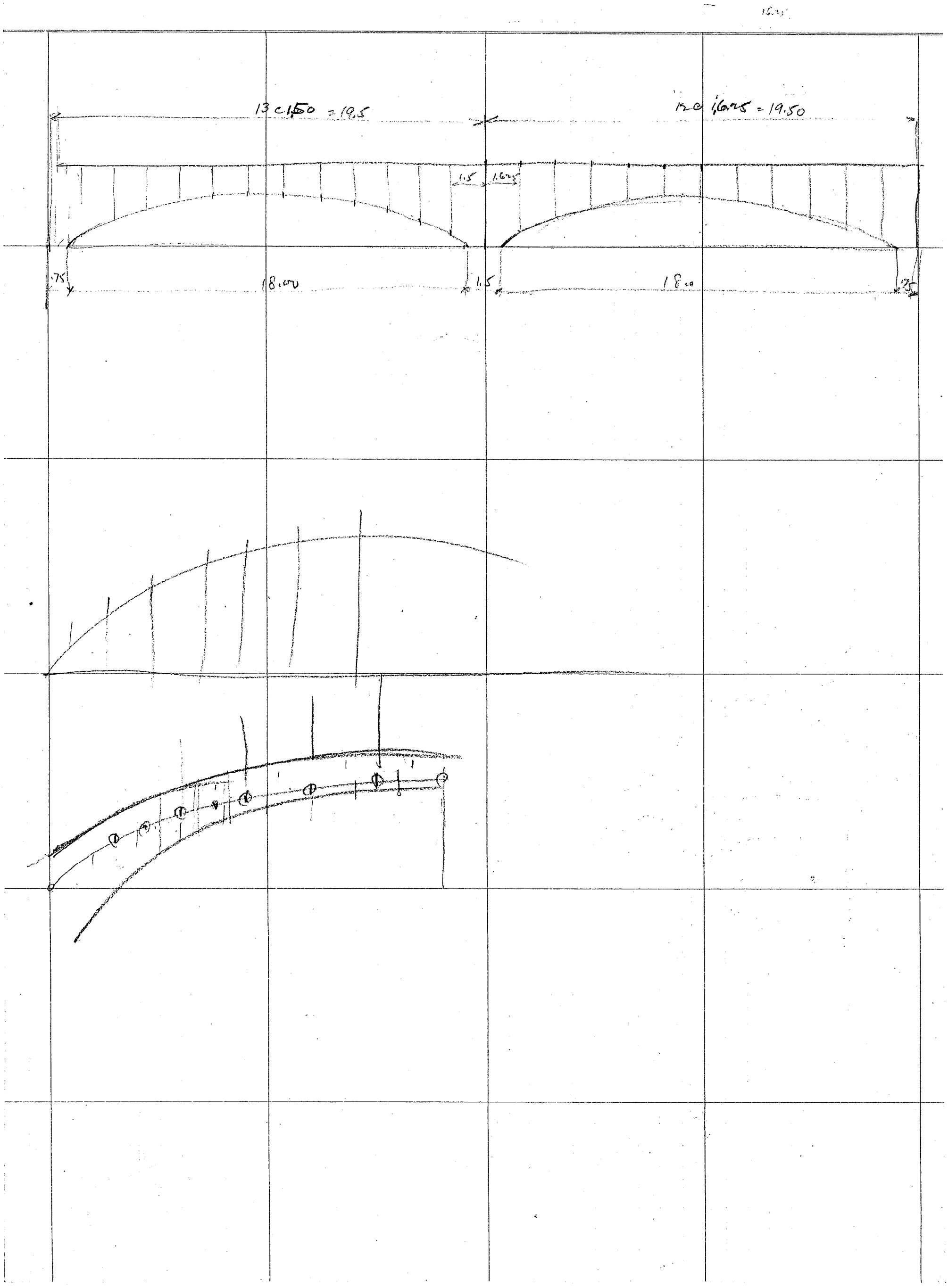
CALCULATIONS FOR



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CALCULATIONS FOR

Kamogawa Bashi for Kyoto Fu.

Approximate Dead Load panel concentrations.

Road way floor.

5 cm asphalt block pavement	@ 21	=	105
1.5" mortar cushion	@ 17	=	25.5
17 cm Concrete slab	@ 24	=	408.
misc concrete say		=	11.5
			550.0 kg/m²

Sidewalk floor.

3.8 cm asphalt block pav.	@ 21	=	80.
1.2" mortar cushion	@ 17	=	20.5
10 cm concrete slab	@ 24	=	240.0
misc. conc say		=	9.5
			350. kg/m²

Coping and handrail

Coping .35 × .20 @ 2400 = 170 kg/lin m coping

Handrail for one panel. (3.13 m)

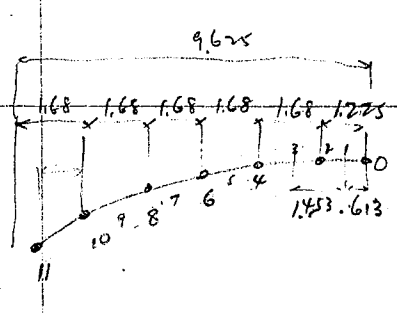
Granite
.20 × .14 × 2.76 = .0773
.11 × .20 × 2.76 = .0607
.24 × .18 × 2.76 = .1192
.15 × .18 × .6 × 2 = .0324
.37 × .30 × 1.08 = .1200
.4096 @ 2600 = 1065

Casting
4 × 3 × 2.76 = 33.10
1.5 × 3. × 6.40 = 28.80
6190 cm³ @ .00725 = 45 × 3 = 135
1200

for 1 meter 1200 ÷ 3.13 = 385 kg/lin m. handrail

Floor Load.

Roadway floor.	5.5 m @ 550 =	3025
Sidewalk floor	1.82 @ 350 =	637
Coping		170
Handrail		385
Slab under curb stone	.17 × .33 = .056 .065 × .15 = .010 .066 @ 2400 =	157
curb stone	.18 × .22 @ 2600 =	103
		4477
Telephone pipes Electric wiring etc	4 @ 25 = say	100 23
		4600 kg per lin m.



Panel load due to floor.

550 × 4.0 × .613 = 1350

Panel pt.			kg	add extra thickness of slab at curb	
0	.613	@ 4600 =	2820	+ 60 =	2880 − 1350 = 1530
2	1.453	@ 4600	6690	+ 140 =	6830
4	1.68	@ "	7730	+ 80 =	7810
6	1.68	@ "	7730		7730
8	1.68	@ "	7730		7730
10	1.68	@ "	7730		7730

$\frac{2.7}{2} \times .103$ @ 2400 = 100.

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CALCULATIONS FOR

Kamogawa Bashi for Kyoto Pref.

Crown fill.				
panel pt.	0	.3 × 4.0 × .613 @2200 =	1615	kg
'	2	.34 × 4.0 × 1.453 @ " =	4350	"
'	4	.38 × 4.0 × 1.84 @ " =	2810	"
Spandrel wall.	0	.30 × .57 × .613 @2400 =	250	
Longitudinal wall	2	.30 × .69 × 1.453 " =	720	
	4	.30 × 1.15 × 1.84 " =	700	
	6	.30 × .50 × 1.68 " =	610	
	8	.30 × .80 × 1.68 " =	970	
	10	.30 × 1.30 × 1.68 " =	1570	
Cross wall.	0	—		
	2	—		
	4	0.30 × .34 × 5.70 @ 2400 =	1400	
	6	" × .50 × 5.70 " =	2050	
	8	" × .80 × 5.70 " =	3280	
	10	" × 1.30 × 5.70 " =	5330	
Cross beam at centre.		.30 × .40 × 1.35 @2400 =	390	at 4, 6, 8, 10
Arch Ring	0	.35 × .613 × 6.0 @ 2400 =	3090	
	2	.37 × 1.453 × " @ " =	7750	
	4	.40 × 1.68 " " =	9670	
	6	.43 × 1.68 " " =	10400	
	8	.53 × 1.68 " " =	14040	
	10	.64 × 1.37 " " =	12620	

Summary for Dead Panel Loads.

	Panel Points					
	0	2	4	6	8	10
Floor	1530	6830	7810	7730	7730	7730
Crown fill	1615	4350	2810	—	—	—
Longitudinal wall	250	720	700	610	970	1570
Cross wall	—	—	1400	2050	3280	5330
Cross beam	—	—	390	390	390	390
Arch ring	3090	7750	9670	10400	14040	12620
	6,485	19,650	22,780	21,180	26,410	27,640
In round nos.	6,500 kg	19,700	22,800	21,200	26,400	27,600 kg.

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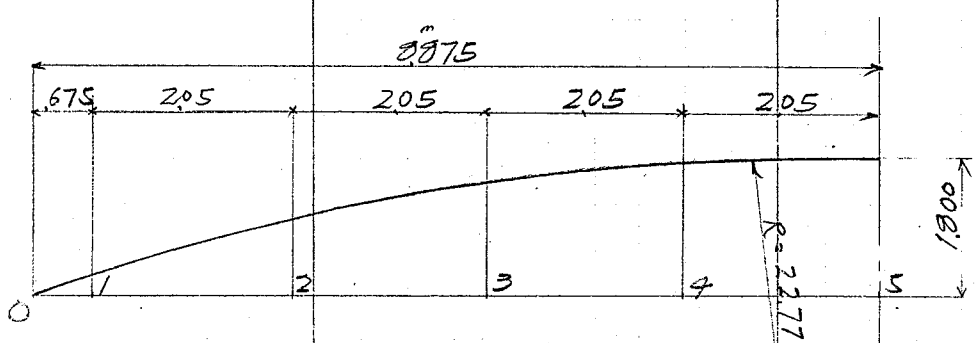
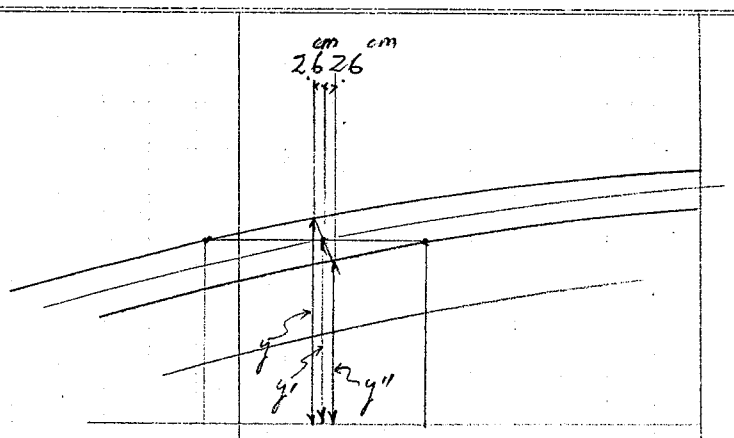
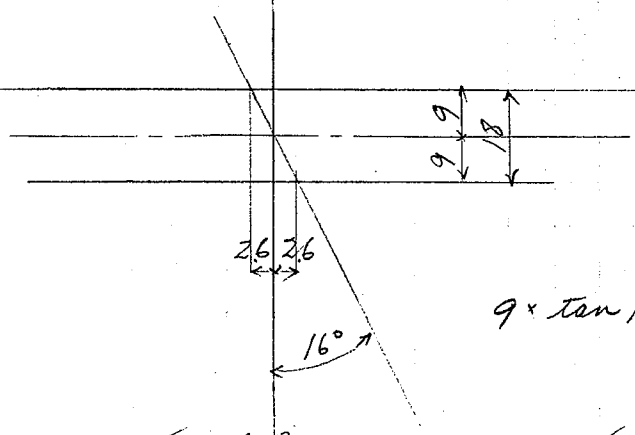
CALCULATIONS FOR

Jonan Bashi for Kyoto-Prefecture







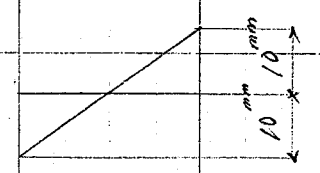
$9 \times \tan 16° = 9 \times 0.2867454 = 2.6\ cm$

20.979

$y_1 = 1.80 - 22.779 + \sqrt{22.779^2 - 8.226^2} = \cancel{113}\ 263'$,

$y_1' \quad + \sqrt{22.779^2 - 8.200^2} = \cancel{123}\ 273'$

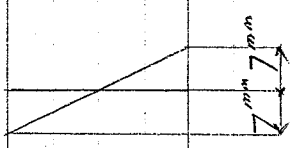
$y_1'' \quad + \sqrt{22.779^2 - 8.174^2} = \cancel{133}\ 283'$



$y_2 \quad + \sqrt{22.779^2 - 6.176^2} = \cancel{797}\ 947'$

$y_2' \quad + \sqrt{22.779^2 - 6.15^2} = \cancel{804}\ 954'$

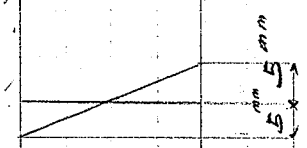
$y_2'' \quad + \sqrt{22.779^2 - 6.124^2} = \cancel{811}\ 961'$



$y_3 \quad + \sqrt{22.779^2 - 4.126^2} = \cancel{1.273}\ 1,423'$

$y_3' \quad + \sqrt{22.779^2 - 4.10^2} = \cancel{1278}\ 1,428'$

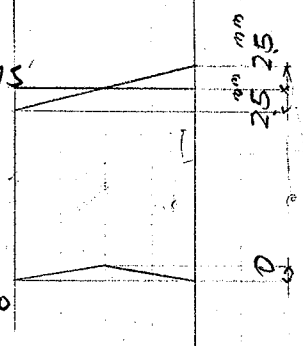
$y_3'' \quad + \sqrt{22.779^2 - 4.074^2} = \cancel{1283}\ 1,433'$



$y_4 \quad + \sqrt{22.779^2 - 2.076^2} = \cancel{1.55}\ 1,705'$

$y_4' \quad + \sqrt{22.779^2 - 2.05^2} = \cancel{1.575}\ 1,707.5'$

$y_4'' \quad + \sqrt{22.779^2 - 2.024^2} = \cancel{1.560}\ 1,710'$



$y_5 \quad + \sqrt{22.779^2 - .026^2} = \cancel{1.650}\ 1,800$

2.10 cm