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CALCULATIONS FOR

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昭和五年二月

國道壱號線

愛知縣木曽川橋

予算設計々算書

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken.

Total length of bridge 877.71 meters c to c of End bearings 878.81 meters out to out.
13 spans @ 63.42 c to c of pins 1- 40.77 meter span on west bank. center to center of bearings on pier .96 meter.
Clear roadway 7.5 meters clear between curb lines ; paved with asphalt block 5 cm thick on Sand mortar Cushion. Floor slab reinforced concrete ; Handrails of cast iron design.

Assumed Loadings

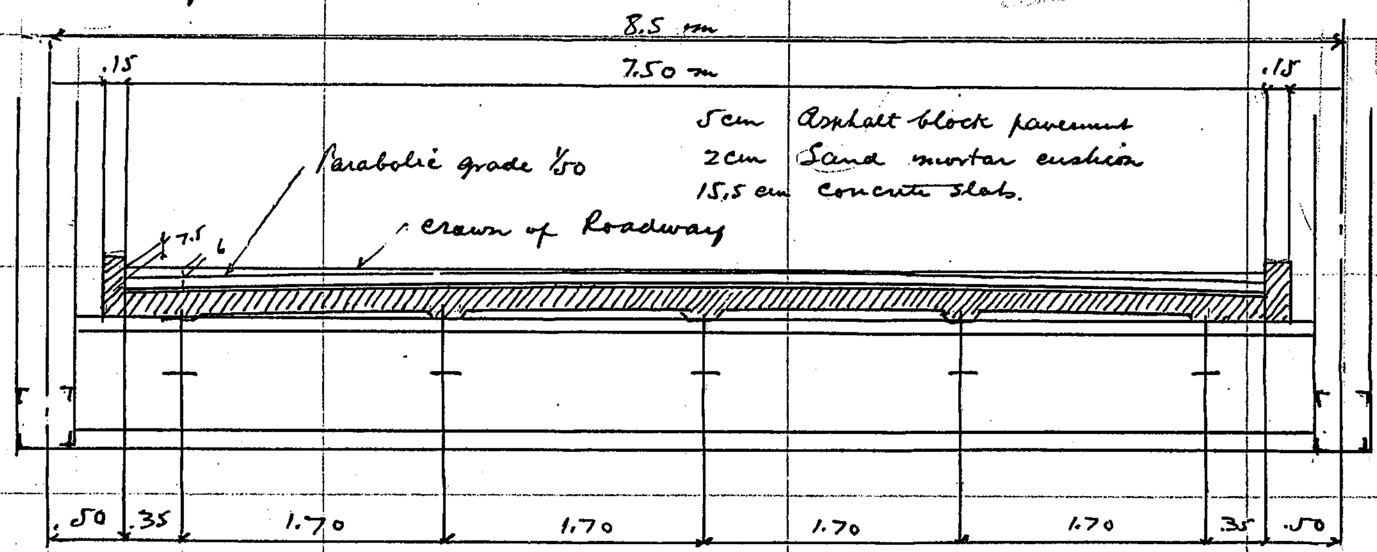
Uniform load on Roadway $w = \frac{100.000}{170 + l} \leq 500$ kg/m²

where l = span length in meter

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

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

Cross Section of Bridge assumed as shown.



Floor Slab span length 1.70 meters

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

Dead Load Moment = $\frac{1}{10} \cdot 520 \cdot 1.70^2 = 150$ kgm

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

Distribution of wheel concentration on slab

Longitudinal distribution a contact between wheel and pavement 20
Distribution 2@7. 14
34 cm

Transverse distribution b = 27 + 14 = 41.0 cm

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

Load per meter strip 3900 ÷ 1.47 = 2650 kg
Uniform live load 500 kg per square meter

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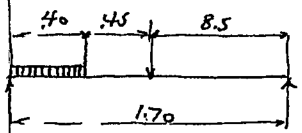
CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken



500 × 0.4 = 200

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

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

uniform load 23.5 × .85 = 20

1145 × .8 = 915 kgm

Dead Load moment 150 "

1065 "

Effective depth req'd $= \sqrt{\frac{1065 \times 100}{100 \times 7.18}} = 12.2$

make depth of slab 15.5 cm insulation at bottom 3 cm

I Beam Stringer Span length 4.53 cm spacing 1.70 meters

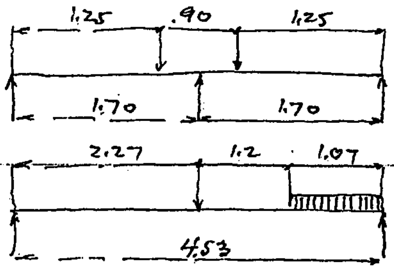
Dead Load floor slab and pavement 520 × 1.70 = 885

beam assumed 75

960 kg per meter

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

Live Load motor truck loading rear wheel $2 \times 3900 \times \frac{1.25}{1.70} = 5750$ kg.



Uniform live load 500 × 1.70 = 835 kg

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

moment motor truck $\frac{5750}{2} \times 2.27$ = 6550

Unif. load 105 × 2.27 = 238

6788

Summary for moment
Dead Load 2460
Live Load 6788
9248 kgm

section modulus req'd $= \frac{924800}{1100} = 840$

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

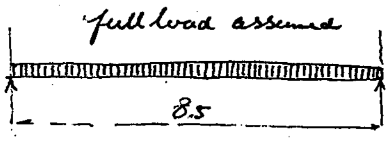
section modulus 1030.

Floor Beam span length 8.5 meters spacing 4.53 meters

Dead Load Floor 520 × 4.53 = 2360

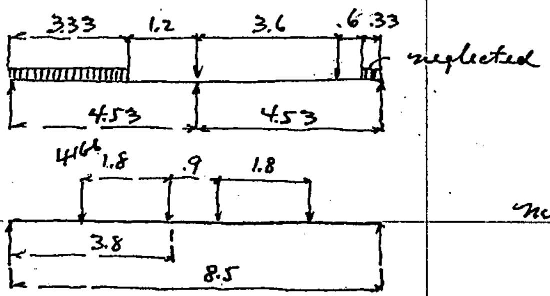
floor beam + stringers 400

2760



Dead Load moment $= \frac{1}{8} \times 2760 \times 8.5^2 = 25100$ kgm

Live Load



motor truck $1300 \times \frac{.93}{4.53}$ = 266

3900

4166 kg.

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

moment motor truck 2 × 4166 × 3.8 = 31700

4166 × 1.8 = − 7500

24200 kgm

Unif. load $\frac{1}{8} \times 610 \times 8.5^2$ = 5500 "

29700 − call this 31000

Dead Load moment 25100 25100

561 56100

Try web plate 800 × 9 = 72.0 1/8 web = 9.0 cm² back to back of Ls 81.0 cm

Effective depth .787

flange stress $= \frac{56100}{.787} = 71200$ kg SR = 71200 ÷ 1200 = 59.3

− 9.0

50.3 net

2 Ls 125 × 90 × 10 = 41.0 − 8.8 = 32.20

1 Pl. 270 × 9 = 24.3 − 4.4 = 19.90

52.10 cm² net

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken

Approximate weight of Intermediate Floor Beam

1 web pl		800 × 9	@ 56.52	× 8.10	=	458
flange	4Ls	125×90×10	@ 16.09	× 8.10	=	520
"	2Pls.	270×9	@ 19.08	× 5.60	=	214
End stiffs	4Ls	125×90×10	@ 16.09	× 0.8	=	52
fills	4Pls	90×10	@ 7.06	× 0.62	=	18
str. conn.	10Ls	100×90×10	@ 14.13	× 0.8	=	113
fills	10Pls.	90×10	@ 7.06	× 0.62	=	44
Int stiffs	8Ls	90×90×10	@ 13.34	× 0.8	=	107
						1526

Rivet Heads and variation $3\frac{1}{2}\%$ 54

1580. kg ÷ 8.10 = 195 kg/m

Approximate weight of End Floor Beam 1380 kg about.

Bottom Lateral Bracing Seismic force k = 0.3

Dead Load , metal assumed

stringers	72 × 5	=	360
floor beam	1580 ÷ 4.53	=	350
Lateral Bracing			100
Trusses lower Half say			900
			1710

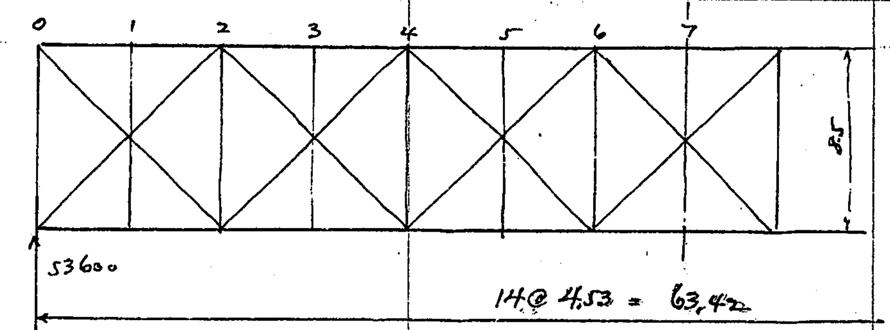
Floor Load	520 × 7.5	=	3900
copings	2 @ 150	=	300
Handrails	2 @ 80	=	160

4360

6070 kg

Panel Concentration 6070 × 4.53 = 27500 @ .3 = 8250 kg

Reaction 8250 × 6.5 = 53600 kg.



4Ls 4.53^2 = 2055
4.25^2 = 1805
38.60 − 6.21

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

						tension only SR					
0-1	53600	× 1.465	=	78600	÷ 2160 =	36.4	39300	÷ 1540	=	25.5	2Ls 130×130×9 = 45.18−9 = 36.18
1-2	46350		=	68000		31.4	34000		=	22.0	2Ls do
2-3	38100		=	55900		25.9	24950				2Ls do
3-4	29850		=	43800		20.2	21900				2Ls 125×90×10 = 41.0 − 10 = 31.0
4-5	21600		=	31700		14.7	15850				2Ls do
5-6	13350		=	19600		9.1	9800				2Ls do
6-7	4100		=	6000		2.8	3000				2Ls do

Approximate weight of lower laterals

4Ls 130×130×9 @ 17.73 × 5.8 = 412

Center connection etc say 38

440 kg per panel

4Ls 125×90×10 @ 16.09 × 5.8 = 373

37

410 kg per panel

Total weight 6 @ 440 = 2640

8 @ 410 = 3280

misc details say 320

6240 ÷ 63.42 = 98.5 call this 100 kg

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Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken.

Top Lateral Bracing

Diagonal length $4.95^2 = 24.50$

$4.25^2 = 18.10$

42.60 – 6.50 meters

Radius of gyration req'd $= \frac{650}{150} = 4.33$

2Ls 100×75×10 r = 48

4Ls 100×75×10 @ 12.95 × 6.0 = 310

Details say 90

400 (350)

For one panel 2×400 = 800 kg

Diagonals	12 @ 800	=	9600 (7000)
strut	6 @ 600	=	2400 } 11,400
Sway Bracings	5 @ 1800	=	9000
Portal Bracings	2 @ 2500	=	5000
			26000 kg.

26000 ÷ 63.42 = 410 kg per lin. meter

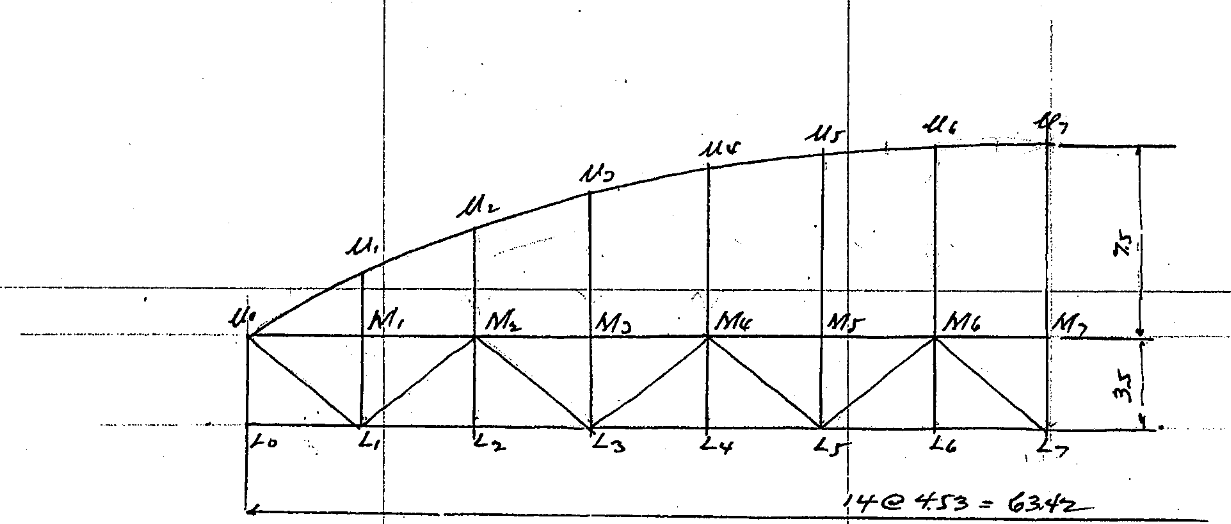
Approximate Dead Load on truss

Structural Steel

stringers	72×5	=	360
Floor Beam	1580 ÷ 4.53	=	350
Lower Laterals			100
Top Laterals			410
trusses assumed			1800
			3020
Floor load complete			4360
			7380 kg per meter

Dead load concentration $= \frac{7380}{2} \times 4.53 = 16700$ kg.

General dimension of truss as shown



Live Load stresses. Uniform live load $\frac{100,000}{170 + 63.42} = 429.0$ kg $429 \times 7.5 = 3220$

For one truss $\frac{3220}{2} \times 4.53 = 7300$

including motor truck say 8000 kg throughout

8000 ÷ 16700 = 48% about

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa Bashi for Aichi-Ken.

Dead Load stresses

chord stresses				web stresses			
U_0-U_1	11.1 c	16700	= 186000 C	U_0-L_1	3.05 T	@ 16700	= 50900 T
U_1-U_2	10.8		= 181000 C	L_1-M_2	2.72 C		= 45400 C
$U_2$$U_3$	10.55		= 176000 C	M_2-L_3	2.35 T		= 39300 T
U_3-U_4	10.35		= 173000 C	L_3-M_4	1.95 C		= 32600 C
U_4-U_5	10.25		= 171000 C	M_4-L_5	1.45 T		= 24200 T
U_5-U_6	10.15		= 170000 C	L_5-M_6	.89 C		= 14900 C
U_6-U_7	10.10 ✓		= 169000 C	M_6-L_7	.27 T		= 4500 T
L_0-L_3	4.55		= 76000 T	U_0-M_2	7.65 T		= 127800 T
L_3-L_5	7.95		= 133000 T	M_2-M_4	3.68 T		= 61500 T
L_5-L_7	9.80		= 164000 T	M_4-M_7	1.00 T		= 16700 T

Live Load stresses as shown on diagram

Top chord section

1 cov Pl. 650 × 13 = 84.50
2 web pls 470 × 13 = 122.00
4 Ls 100 × 100 × 10 = 76.00
282.50 — U_0 - U_7 + L.do

Bottom chord section

2 P/s. 470 × 10 = 94.0 − 20 = 74.0
4 Ls 100 × 100 × 10 = 76.0 − 20 = 56.0
170.0 130.0 — L_0-L_3
2 P/s. 270 × 10 = 54.0 − 10 = 44
224.0 174.0 — L_3-L_5
2 P/s. 470 × 10 = 94.0 − 20 = 74
318.0 248.0 — L_5-L_7

middle chord section

4 Ls 125 × 75 × 10 = 76.0 − 25 = 51.0 -- M_4-M_7
2 P/s. 270 × 9 = 48.6 − 13.5 = 35.1
124.6 86.1 -- M_2-M_3

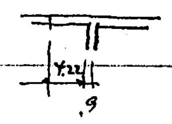
4 Ls 125 × 75 × 10 = 76.0 − 25 = 51.0
4 P/s. 270 × 15 = 162.0 − 45 = 117.0
238.0 168.0 --- U_0-M_2

Diagonal 4.53^2 = 20.5
3.50^2 = 12.25
32.75 — 5.70

Tension members 4 Ls 125 × 75 × 10 = 76.0 − 25 = 51.0 --- M_2-L_3
2 P/s. 270 × 9 = 48.6 − 13.5 = 35.1 ---
124.6 86.1 — net ---- U_0-L_1

4 Ls 100 × 75 × 10 = 66.0 − 20 = 46.0 net M_4 L_5, M_6-L_7

Compression members



1 Pl 270 × 9 = 24.3 = 1475
2 Ls 125 × 75 × 10 = 38.0 × 4.67^2 + 594 = 1422
62.3 2897

$$r = \sqrt{\frac{2897}{62.3}} = 6.82$$

$$P = 1500\left(1 - 0.0055 \times \frac{570}{6.82}\right) = 810 \text{ kg/cm}^2$$

$$SR = 69900 \div 810 = 86.4 \text{ cm}^2$$

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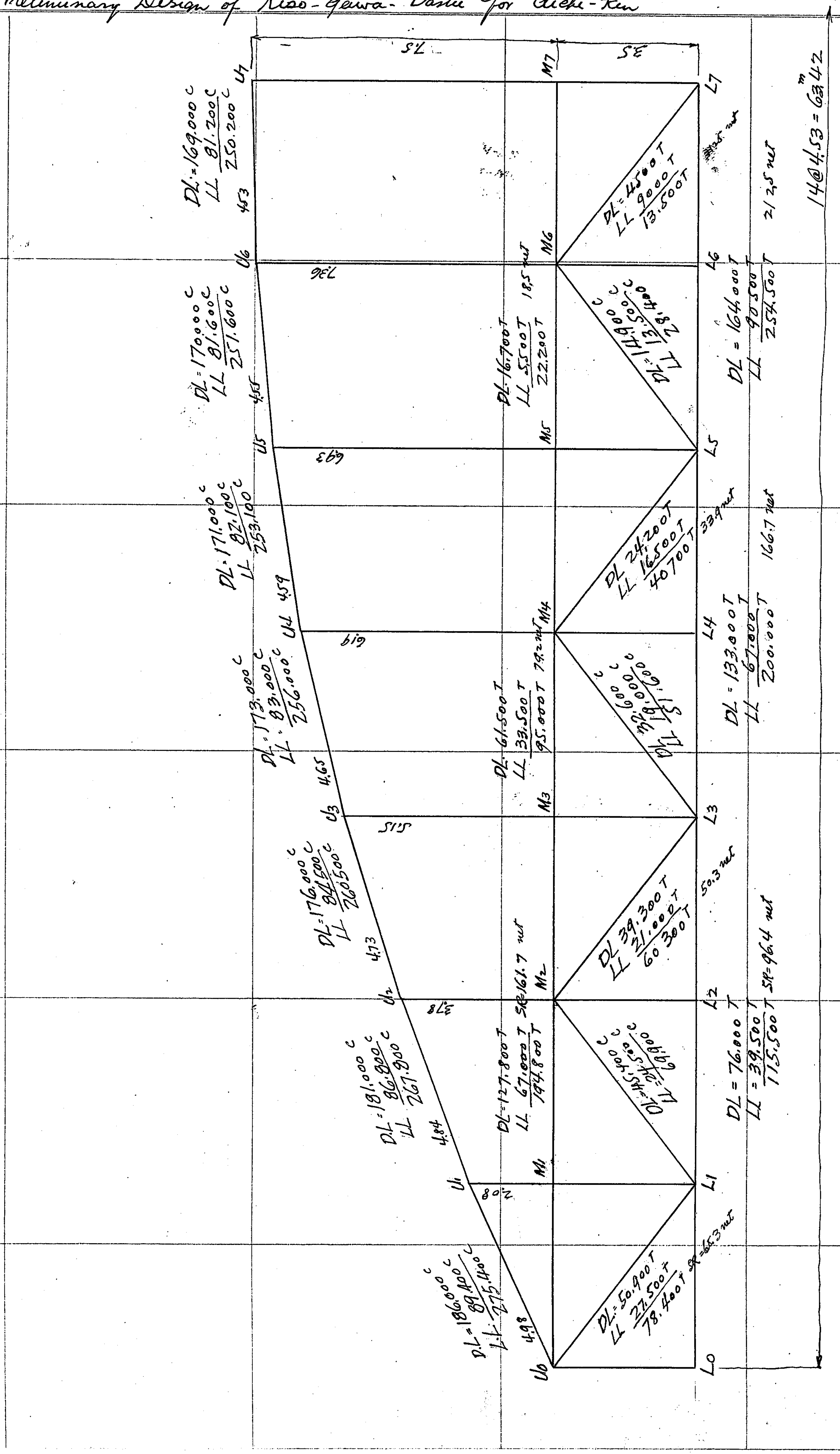
CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-ken



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Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken

Use 2Ls 270 × 9 = 48.6
4Ls 125×75×10 = 76.0
124.6 — gross - L1-M2

2Ls 125×75×10 = 38.0 ∴ $r = \sqrt{\frac{1422}{38}} = 6.12$

1422 · 9

$P = 1500\left(1 - 0.0055 \times \frac{570}{6.12}\right) = 730$

51600 ÷ 730 = 70.7 cm²

Use 4Ls 125×75×10 = 76.0 gross - L3-M4

4Ls 100×75×10 = 66.0 .. L5-M6

Approximate weight of truss

Top chord	282.5	@ .785 × 36.37	=	8070	8070
Bott. ..	170.0	× 13.59	=	1810	
	224.0	× 9.06	=	1570	5610
	318.0	× 9.06	=	2230	
middle chord	238.0	× 9.06	=	1670	
	124.6	× 9.06	=	870	3350
	76.0	× 13.59	=	810	
vertical	76.0	× 58.00	=	3460	3460
Diagonals	124.6	5.70	=	560	
	76.0	5.70	=	340	
	66.0	11.40	=	590	2690
	124.6	5.70	=	560	End post 除く
	76.0	5.70	=	340	
	66.0	5.70	=	300	
				23180 × 2 =	46.360
		Details 35%			16200
					62560

For 2 trusses @ 62560 = 125120 kg.
call this 125.0 tons

125000 ÷ 6342 = 1980 kg per lin. meter of span.

Structural Steel in one span span length 63.42 meters

stringers	360 × 64.5	=	23200	1.0
intermediate F.B.	1580 × 13	=	20540	2.3
End Floor beam	1380 × 2	=	2760	
Bottom lateral			6240	1.3
Top lateral			26000	23.2 — ~~2.~~8 -3
Trusses			125000	10.0
shoes			4500	
misc steel			1500	
			209740	

call this 210. tons.

For 13 spans @ 210 = 2730 tons

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CALCULATIONS FOR

Preliminary Design of Kiso-Gawa-Bashi for Aichi-Ken

Approximate Dead Load on truss span length 40.77
structural steel

stringers	72 × 5	=	360
Floor beam	1580 ÷ 4.53	=	350
Lower Lateral	say		100
trusses assumed			1500
			2310

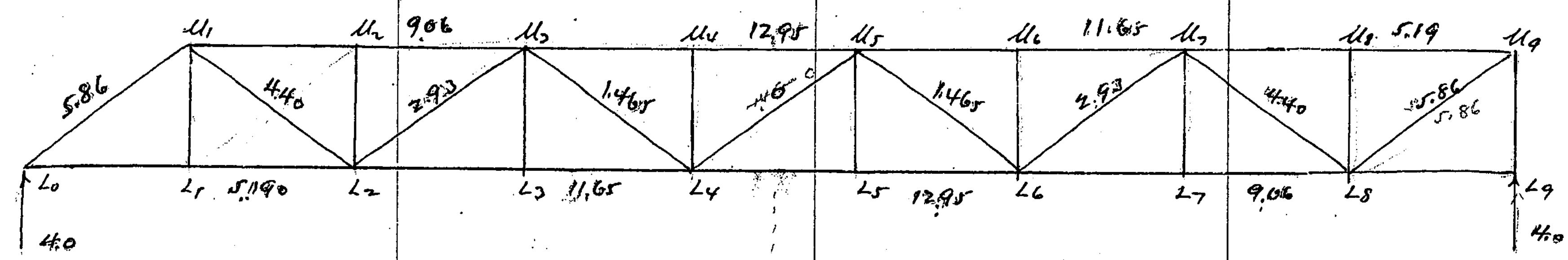
Floor Load complete 4360
6670 kg per meter

Dead Load concentration one truss = $\frac{6670}{2}$ × 4.53 = 15100 kg.

Live Load Uniform live load $w = \frac{100.000}{170+40.77}$ = 475 kg/m²

for 7.5 meter wide 475 × 7.5 = 3565 kg per lin. meter.

motor truck rear wheel concentration 3000
impact $\frac{20}{60+40.77}$ = 18.6% 558
3558 × 2 = 7116



Diagonal length 4.53^2 = 20.55
4.25^2 = 18.05
38.60 - 6.21

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

Dead Load stresses

U1-U3	9.06	@ 15100	=	137000 C
U3-U5	12.95		=	195500 C
L0-L2	5.19		=	78400 T
L2-L4	11.65		=	176000 T
L0-U1	5.86	@ 15100	=	88500 C
U1-L2	4.40			66500 T
L2-U3	2.93			44200 C
U3-L4	1.465			22100 T
L4-U5	0			0

Live Load stresses
motor truck loading.

Uniform load live load concentration = $\frac{3565}{2}$ × 4.53 = 8100 kg.
rear wheel
2 × 7116 × $\frac{5.3}{8.5}$ = 8900
unif. load 2.1 × 475 × $\frac{1.55}{8.50}$ = 182
panel concentration = 182 × 4.53 = 825
8900
9725
8100
1625

use panel concentration 9500 kg throughout

Live load stress 9500 ÷ 15100 = 63%

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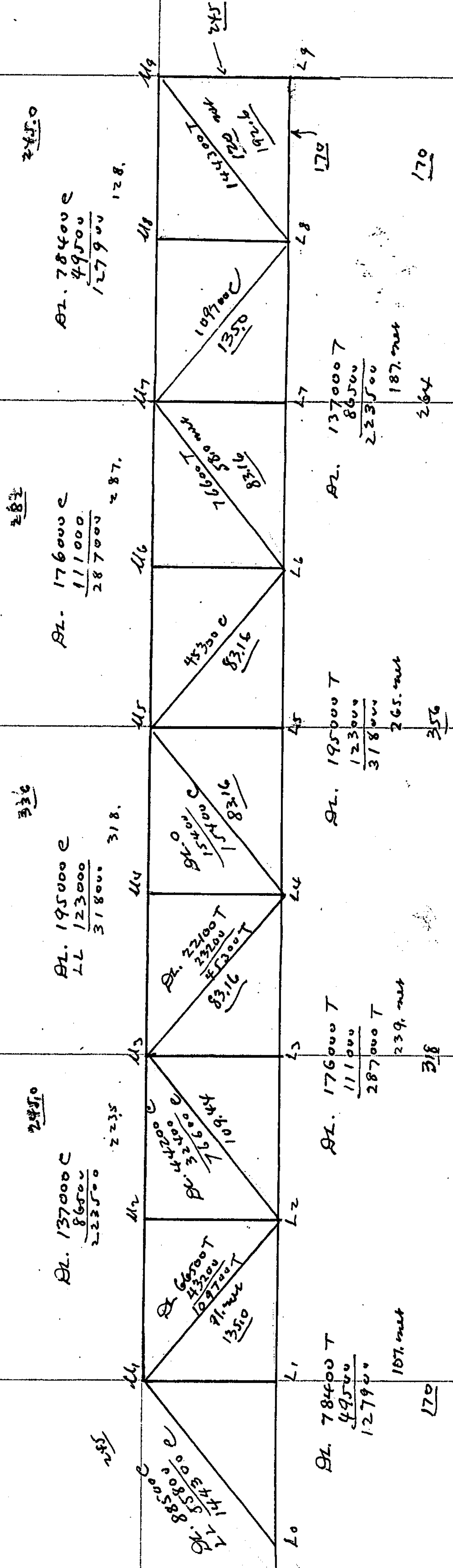
CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken

webstresses

M1-L2 3.11, ~~7.79~~	9500 × 1.465 ×	=	432000
L2-M3 2.33, ~~6.65~~		=	32400
M3-L4 1.67, 5.55		=	23200
L4-M5 1.11, 4.44		=	15400



Chord members

1 cov Pl	=	650 × 13	=	84.5
2 Pls		470 × 13	=	122.5
4 Ls		100 × 100 × 10	=	76.0
2 Pls.		270 × 10	=	54.0
				336.0

1 cov Pl	650 × 13	=	84.5
2 Pls.	470 × 13	=	122.5
4 Ls	100 × 100 × 10	=	76.0
			283.0

1 cov Pl.	650 × 13	=	84.5
2 Pls.	470 × 9	=	84.5
4 Ls	100 × 100 × 10	=	76.0
			245.0

Bottom chord members

2 Pls.	470 × 10	=	94.0	- 20	=	74.0
2 Pls.	470 × 10	=	94.0	- 20	=	74.0
4 Ls	100 × 100 × 10	=	76.0	- 20	=	56.0
			264			204.0

2 Pls.	470 × 10	=	94.0	- 20	=	74.0
4 Ls	100 × 100 × 10	=	76.0	- 20	=	56.0
			170.			130.0

2 Pls.	470 × 10	=	94.0	- 20	=	74.0
2 Pls.	470 × 10	=	94.0	- 20	=	74.0
4 Ls	100 × 100 × 13	=	97.24	- 26	=	71.24
2 Pls.	270 × 13	=	70.20	- 13	=	57.20
			355.44			276.44

2 Pls.	470 × 10	=	94.0	- 20	=	74.0
2 Pls.	470 × 10	=	94.0	- 20	=	74.0
4 Ls	100 × 100 × 10	=	76.0	- 20	=	56.0
2 Pls.	270 × 13 (10)	=	54.0	- 10	=	44.0
			318.0			248.0

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken.

Diagonal members. unsupported length ~~621~~ 570

9, 3.95

2∟s 125 × 90 × 10 = 41.0 × 4.4² + 634 = 1429

$r = \sqrt{\frac{1429}{41}} = 5.90$

$P = 1500\left(1 - 0.0055 \cdot \frac{\cancel{621}\ 570}{5.9}\right) = \cancel{630}\ 703$ kg/cm²

stress = 2 × ~~630~~ 703 × 41.0 = ~~25800~~ 28750 × 2 = ~~51600~~ 57500

~~300 × 100 × 10 [s = 119.1~~

9, 4.96, .45, 5.41

2∟s 150 × 90 × 9 = 41.58 × 5.41² + ~~9~~4.2 = 2188

$r = \sqrt{\frac{2188}{41.58}} = 7.24$

$P = 1500\left(1 - 0.0055 \cdot \frac{\cancel{621}\ 570}{7.24}\right) = \cancel{790}\ 850$

stress = 2 × ~~790~~ 850 × 41.58 = ~~65700~~ 70700

net section 4∟s 150 × 90 × 9 = 83.16 □cm − 18.0 = 65.16 net
6516 × 1200 = 78300 kg.

9, 5.18, .45, 5.63

2∟s 150 × 90 × 15 = 67.5 × 5.63² + 1510 = 3650

$r = \sqrt{\frac{3650}{67.5}} = 7.35$

$P = 1500\left(1 - 0.0055 \cdot \frac{\cancel{621}\ 570}{7.35}\right) = \cancel{802}\ 860.$

stress = 2 × ~~802~~ 860 × 67.5 = ~~108000~~ 129500 — 116000

~~net section~~	4∟s 150 × 90 × 15	= 135.0	− 30	=	105.0 net
	2Pls 320 × 9	= 57.6	− 9	=	48.6
		192.6			153.6 net

approximate weight of truss

Top chord	245	@ .785	× 24.35	=	4690
"	287	@	× 9.06	=	2040
"	318	@	× 9.06	=	2260
Bott. chord	170		× 13.60	=	1810
"	264		× 9.06	=	1875
"	318		× 9.06	=	2260
"	356		× 9.06	=	2530
Diag.	4 − 83.16		× 6.21	=	1630
	1 − 109.00		× 6.21	=	531
	2 − 135.00		× 6.21	=	1320
	1 − 192.6		× 6.21	=	935
verticals	8 − 82.0		× 3.5	=	1800
	1 − 245		× 3.5	=	673
Bracket say					1000
					25354
Details say	40 %				10100
					35454 call this 36000 kg.

36000 ÷ 40.77 = 885 kg. per lin. meter

weight of 2 trusses 36000 × 2 = 72.000 kg.

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken

Approximate structural steel in one span span length 40.77 meters out to out 41.80

stringers	360 × 41.80	=	11450 — 15050	
floor beam intermediate	1580 × 8	=	12640	
" " End	2 × 1380	=	2760	
lateral Bracings			4100	
trusses			72000	
shoes			3000	
misc.			1500	
			107450	call this 108 tons. 111

108.000 (111.000) ÷ 95 = 1140 (1170) kg / tsubo. 111,050

Total structural Steel.

13 @ 210.0 tons	=	2730
1 span		108
		2838 tons.

Approximate Estimate of Deck Construction
Total length of bridge - 878.81 between faces of parapet walls of abutments.

Concrete Slab	.155 × 7.5	=	1.160
coping	.15 × .375	=	.056
filler			.010
			1.226 × 878.81 = 1075 cubic meters
Area of pavement	= 7.5 × 878.81	=	6580 sq meters
Forms say	8.1 × 878.81	=	7110 sq meters
Reinforcing Bars	6580 × 23 kg	=	152.000 kg
Handrails	2 × 80 × 879	=	140500 kg
Finish of coping.	1.40 × 879	=	1230 sq meters.

Approximate Estimate of Deck Construction

Concrete	1075	m³	@	16.30	=	17500
Forms	7110	m²	@	2.00	=	14220
Reinforcing Bars	152	tons	@	120.00	=	18250
Pavement	6580	m²	@	4.75	=	31300
Handrails	140.5	tons	@	320.00	=	45000
Finish	1230	m²	@	4.25	=	5220
Drains	216		@	6.00	=	1296
Entrance Pedestals &c	4		@	1500.00	=	6000
lamps and Electric wiring	890	m	@	10.00	=	8900
						147686 say 148000

structural steel say 2850 tons @ 280 = 798000.00

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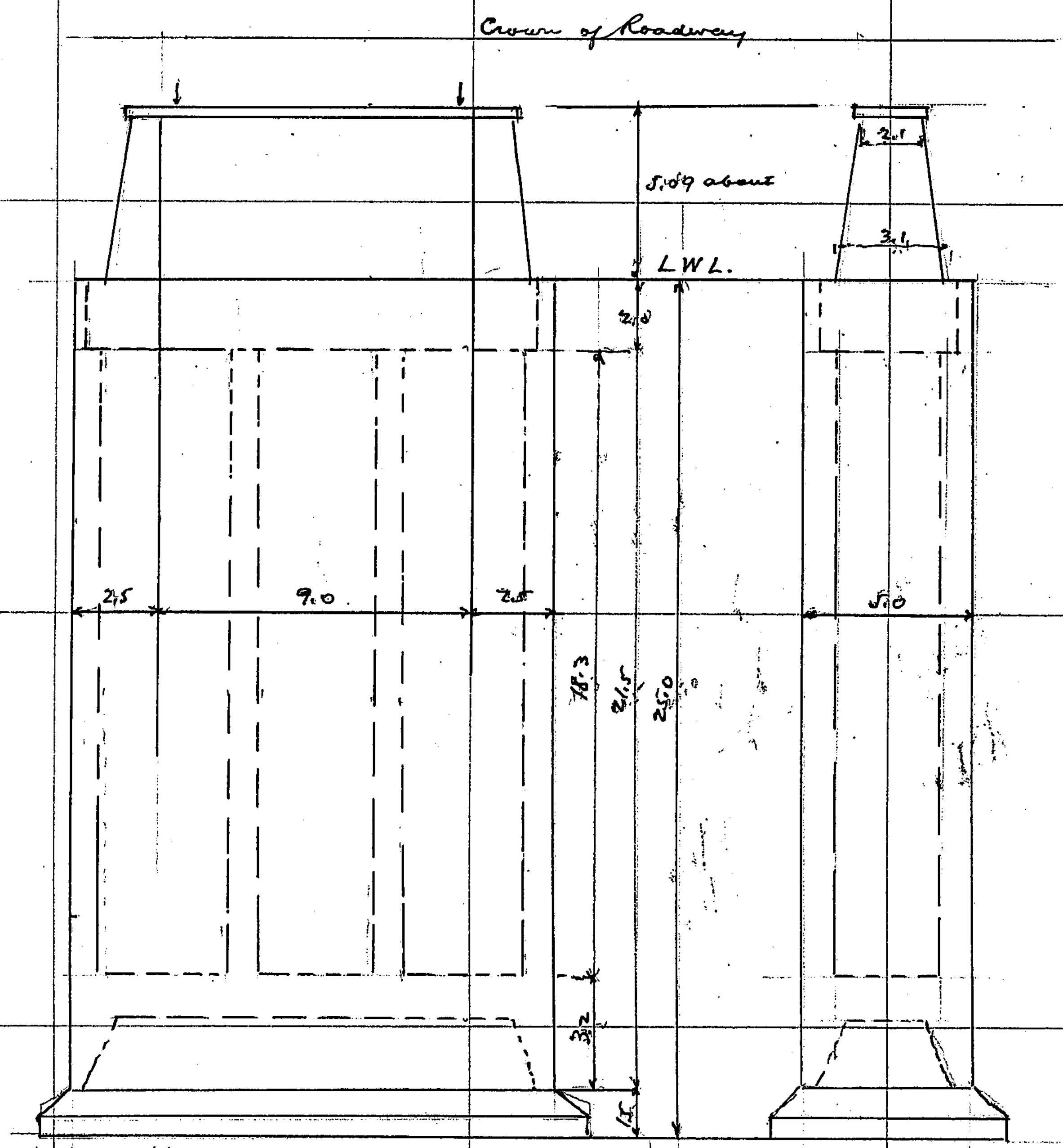
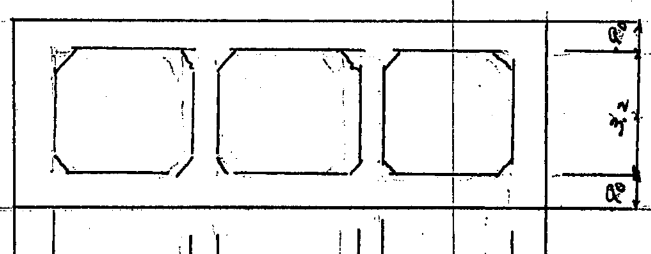
CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken

Piers Superimposed Dead Load structural steel 210.000
Floor 4360 × 64.5 = 281.000
491,000 kg.
Live Load say 3220 × 64.5 = 208.000 "





Approximate Concrete in shaft
2.1^{ϕ} = 3.46
2.1 × 9.0 = 18.90
22.36
3.1^{ϕ} = 7.55
3.1 × 9.0 = 27.90
35.45
57.81 ÷ 2 = 28.9
vol = 28.9 × 5.09 = 147.0 cubic meters

Caisson area 2 - 14.0 × 0.9 = 25.20
2 - 3.2 × 0.9 = 5.76
2 - 3.2 × 0.7 = 4.48
.25 × 6 = 1.50
36.94 sqm

inside filling 5 × 14 = 70.0
36.94
33.06
33.06 ÷ 3 = 11.02 sqm
33.06 × 18.3 = 605.0 cubic meters

volume of Concrete in shell 36.94 × 18.3 = 675.0

Concrete in top portion 70 × 2.0 = 140.0
" " bottom chamber 70 × 3.2 = 224.0
941.0

Concrete in base say 7.0 × 16.0 × 1.5 = 167.0

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa Bashi for Aichi-Ken

weight of Caisson & shaft

shaft	147.0 m^3	@	2200	=	324000
shell	675.0	@	2400	=	1620000
Top	140.0	@	2200	=	308 000
Bottom	224.0		"	=	493.000
Base	167.0		"	-	368,000
water	605.	@	1000	=	605.000
					3,718,000

Superimposed load Dead Load 491.000
Live Load 210.000

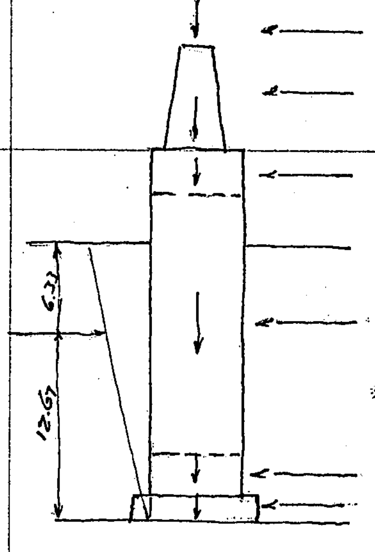
701.000
4419 000 kg.

Area of base 16 · 7 = 112.0 sq meters
Skin friction 350 #/□' or 1710 kg per sq meter
1710 · 38.0 = 65 000 kg per lin. meter of depth

Depth of caisson below firm ground assumed 19.0 meters
Total load 4419000
Less friction 65000 · 19.0 - 1235000
3184 000

Bearing on soil counting skin friction 3184 000 ÷ 112 = 28400 kg/m² or 2.64 tons/□'

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



1	Superimposed load	491.000	× 0.3	=	147300
2	shaft	324.000		=	97.000
3	top concrete	308.000		=	92.500
4	shell + water fill	2225.000		=	667.000
5	bottom concrete	493.000		=	148.000
6	base	368.000		=	110500
		4209.000 ✓			1262.300 ✓

Moment about bottom of base

1	147300 ·	30.9	=	455 0.000
2	97.000 ·	27.5	=	2665.000
3	92500 ·	24.0	=	2220.000
4	667.000 ·	13.85	=	9250.000
5	148.000 ·	3.10	=	459.000
6	110500 ·	.75	=	83.000
	1262.300			19227.000
	1262300 ·	12.67	=	- 16000.000
				3227.000 kg meters

Frictional couple 1710 · 19 · 14 = 455000

Resisting couple 455000 × 5 = - 2280.000
947.000 kg meters

Eccentricity = $\frac{947000}{\cancel{1262300}\ 4209000}$ = .225

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CALCULATIONS FOR

Preliminary Design of Kiso-gawa-Bashi for Aichi-Ken

max pressure = $\frac{4209000}{112}\left(1 \pm \frac{6 \times .225}{7.0}\right)$ = 44900 or 30400
4.18 tons/□' 2.83 tons/□'

Approximate list of materials

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

Area of forms caisson 81.5 × 23.5 = 1915
bottom say 5 × 14 = 70
1985 call this 2000
shaft say 33 × 5.1 = 170
2170 sq meters

Reinforcing Bars 45.0 tons
Curb shoe 4.0 tons
Timber Crib work say 6000 cubic ft
excavation say 5 × 14 × 25.0 = 1750 cubic meters

Estimate of Cost of one pier

concrete 124	1100	m^3	@	16.80	=	18000
" 136	253	m^3	@	13.50	=	3420
forms	2170	m^2	@	2.85	=	6190
Reinforcing Bars	45.	tons	@	120.00	=	5400
Cutting Edge	4	tons	@	200.00	=	800
Timber Crib						12000
Excavation	1750	m^3	@	5.50	=	9630
water fill						300
						55640

Extra cost for Pneumatic Caisson work – – – – – – – – – – 20200
misc Expense 2160
78000.00

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

Abutments 2 @ 12500.00 = 25000.00

Estimate of Cost
Deck construction 148 000
structural Steel 798 000
946 000

Substructure piers 13 @ 78000 = 1014000
abutments 25000
1039000
1985000.00

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CALCULATIONS FOR

昭和五年二月

國道第壹號路線

愛知縣木曽川橋々臺設計々算書

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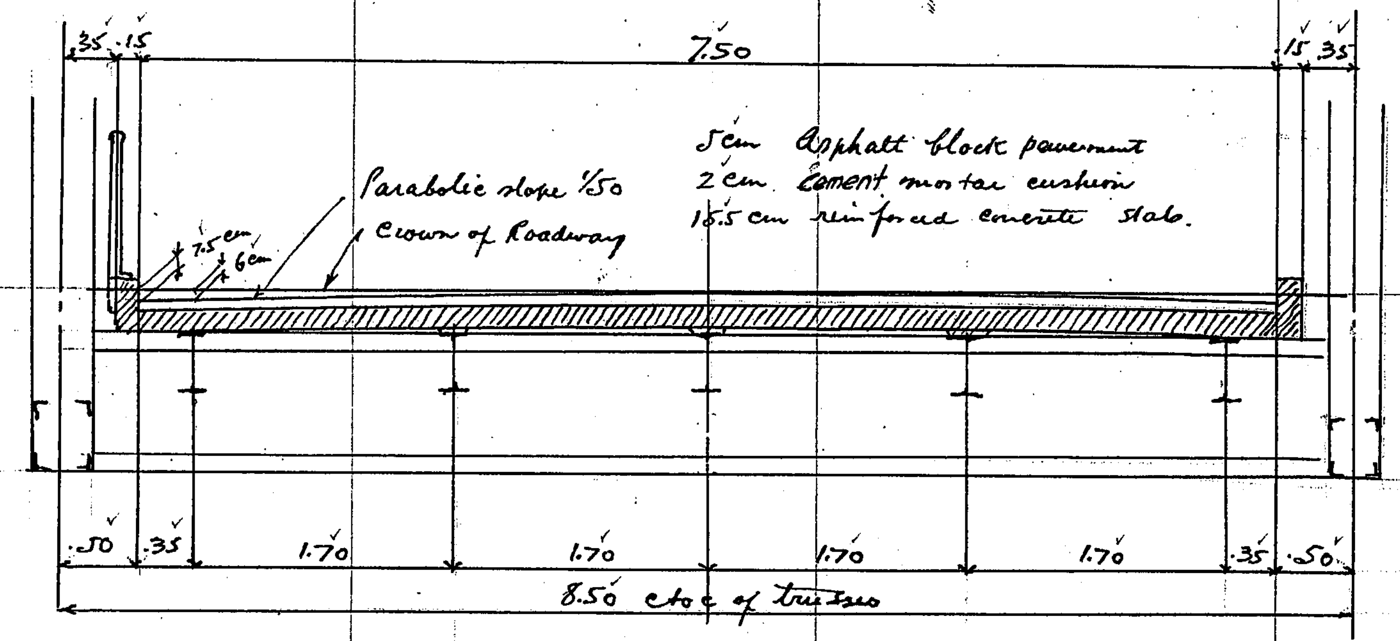
CHECKED BY M. Jiuda DATE 5-2-17 **PAGE NO** 1

CALCULATIONS FOR

Design of Kiso-gawa-Bashi for Aichi-Ken

Assumed cross section of bridge.

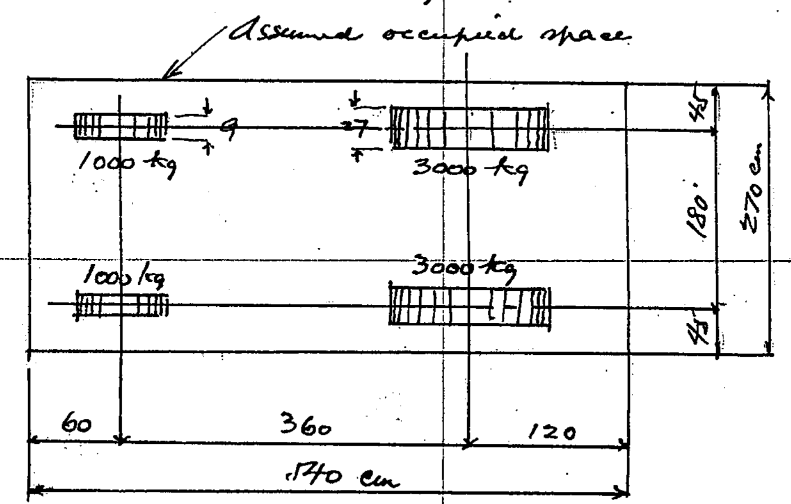


Assumed Loadings

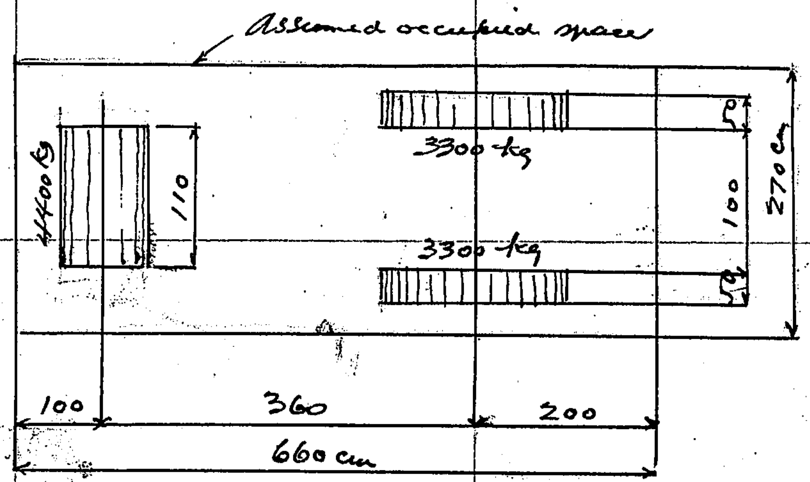
Uniform live load on Roadway $w = \frac{100.000}{170+l} \leq 500$ kg/m²

where l = span length in meter

8 ton motor truck loading



11 ton Road Roller loading



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

Impact for motor truck loading

$\text{Coef} = \frac{20}{60+l}$ where l = loaded length in meter

max impact 30%

No impact for road roller and uniform load

Allowable Working Strength

1:2:4 Concrete

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

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

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CHECKED BY M. Juda DATE 5-2-17 **PAGE NO** 2

CALCULATIONS FOR

Design of Kiso-gawa-Bashi for Aichi-Ken

Superimposed load on abutment span length 63.42 meters.

Dead Load For cross section of bridge see page 1

Asphalt Block Pavement 5cm thick @ 21 = 105
2cm Sand mortar Cushion @ 17 = 34
15.5cm reinforced concrete slab @ 24 = 372
misc concrete say 9
520 kg

Roadway 7.5 meters 520 × 7.5 = 3900
Coping .37 × .165 @ 2400 = say 150 2 × 150 = 300
Handrail assumed 2 × 80 = 160
4360 kg

Structural Steel in one span span length 63.42 m c to c of End bearings 64.38 out to out
185 ton

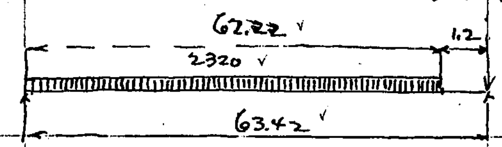
185000 ÷ 64.4 = 2880
Floor load 4360
7240 × 64.4/2 = 234000

for one shoe 234000 ÷ 2 = 117.000 kg.

Live Load Uniform live load = 100.000 / (170 + 63.42) = 429.0 kg 429 × 7.5 = 3220 kg

motor truck rear wheel Concentration 3000
impact = 20 / (60 + 63.42) = 16.2 % 486
3486 × 2 = 6972

For 2 motor trucks 6972 × 2 = 13944 call this 13950 kg.

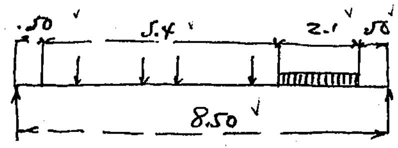


Live load reaction on abutment
Unif. load 2320 × 62.22² / (2 × 63.42) = 70700
900 × 63.42/2 = 28500
motor truck rear wheel 13950
113150

2.1 × 429 = 900 kg.
3220
2320 kg

call this load 115000 kg on abutment

Load on shoe



Uniform load U_1 900 × 1.55/8.50 = 164 kg
Uniform load U_2 2320 × 5.30/8.50 = 1450 kg
motor truck loading 13950 × 5.30/8.50 = 8700 kg

Uniform load U_1 164 × 63.42/2 = 5200
" " U_2 1450 × 62.22² / (2 × 63.42) = 44200
motor truck loading 8700
58100 kg. call this 59000 kg

Summary load 63.42 meter span

	On abutment	On shoe
Dead Load	234 000	117.000
Live Load	115000	59.000
	349 000 kg	176.000 kg.

Size of shoe Roller End 75 × 100 = 7500
unit bearing 176.000 / 7500 = 23.5 kg/cm²

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CALCULATIONS FOR

Design of Kiso-gawa- Bashi for Aichi-Ken

Superimposed load span 40.77 meters

Dead Load	flooring -	7.5 meter wide	520 × 7.5 = 3900
	coping		2 × 150 = 300
	Handrail		2 × 80 = 160
			4360 kg per meter

structural steel in one span 140.000 kg ÷ 41.67 = 3360

7720 " " "

Load on abutment 7720 × $\frac{41.67}{2}$ = 161.000 kg.

Load on shoe 161.000 ÷ 2 = 80500 kg.

Live Load uniform live load = $\frac{100.000}{170 + 40.77}$ = 475 kg/m²

for 7.5 meter wide = 475 × 7.5 = 3565 kg per lin. meter

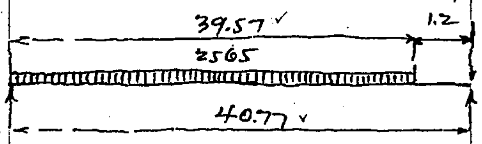
motor truck rear wheel concentration = 3000

impact $\frac{20}{60+40.77}$ = say 19.0 % 570

3570 × 2 = 7140

For 2 motor trucks 7140 × 2 = 14280 call this 14280 kg.

Load on abutment



2.1 × 475 = 1000
3565
2565

uniform load $\frac{2565 \times 39.57^2}{2 \times 40.77}$ = 49300 (493)

Unif. load 1000 × $\frac{40.77}{2}$ = 20400

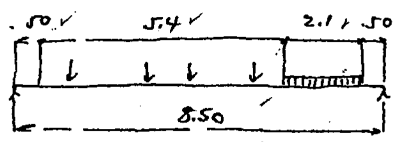
motor truck rear wheel = 14280

83980

call this 84000 kg on abutment

Load on shoe

Reaction on truss



uniform live load U_1 1000 × $\frac{1.55}{8.50}$ = 182 kg

Uniform live load U_2 2565 × $\frac{5.30}{8.50}$ = 1600 "

motor truck loading 14280 × $\frac{5.30}{8.50}$ = 8900 "

Load on shoe

uniform load U_1 182 × $\frac{40.77}{2}$ = 3700

uniform load U_2 1600 × $\frac{39.57^2}{2 \times 40.77}$ = 30700

motor truck loading 8900

43300 kg

Call this 44000 kg

Summary load 40.77 meter span

	On abutment	On shoe
Dead Load	161 000	80.500
Live Load	84 000	44000
	245 000 kg	124500 kg.

Approximate size of Roller shoe.

Unit bearing assumed 23 kg/cm² bearing area req'd = $\frac{124500}{23}$ = 5410

Size of base plate 70 × 80 about = 5600 cm²

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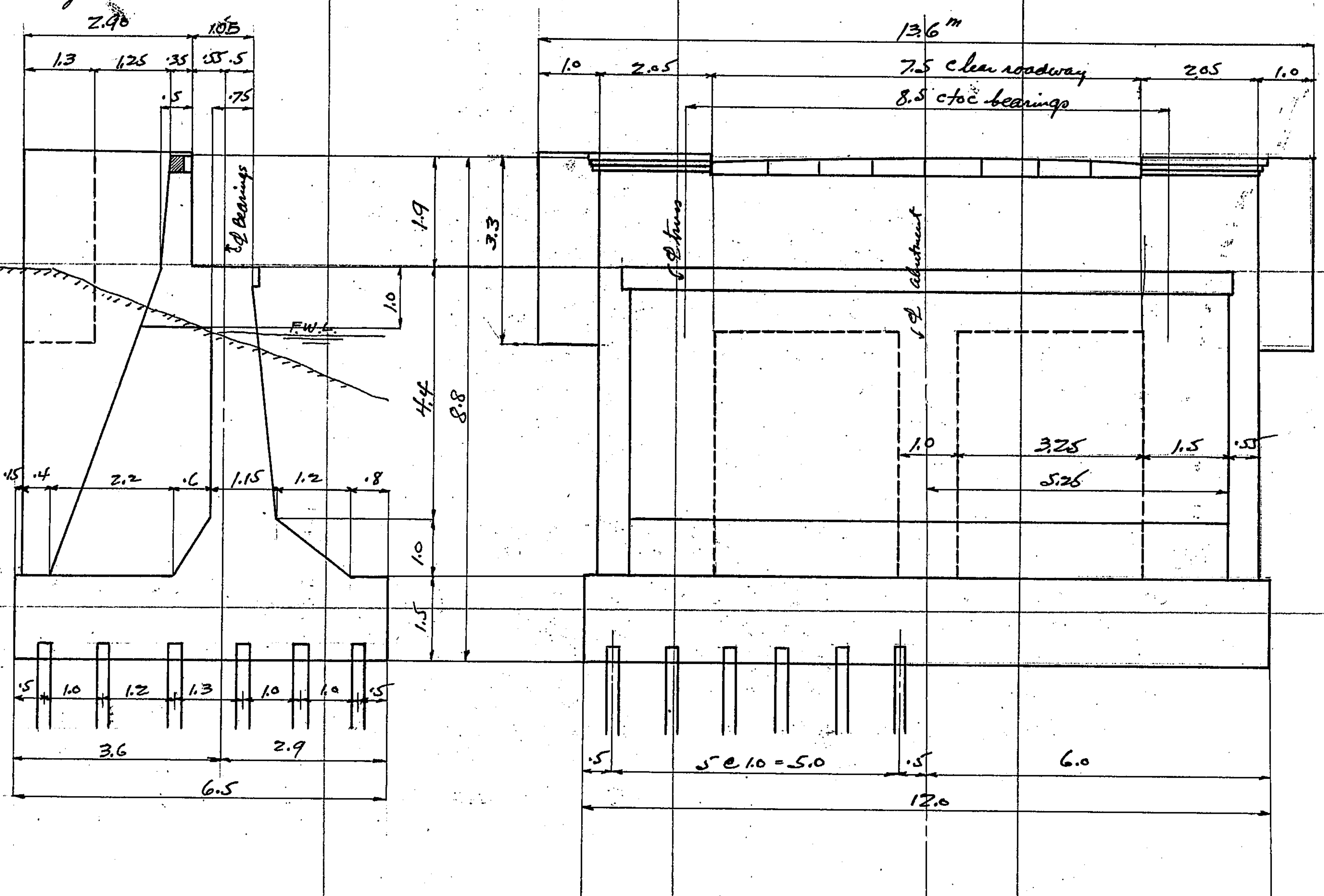
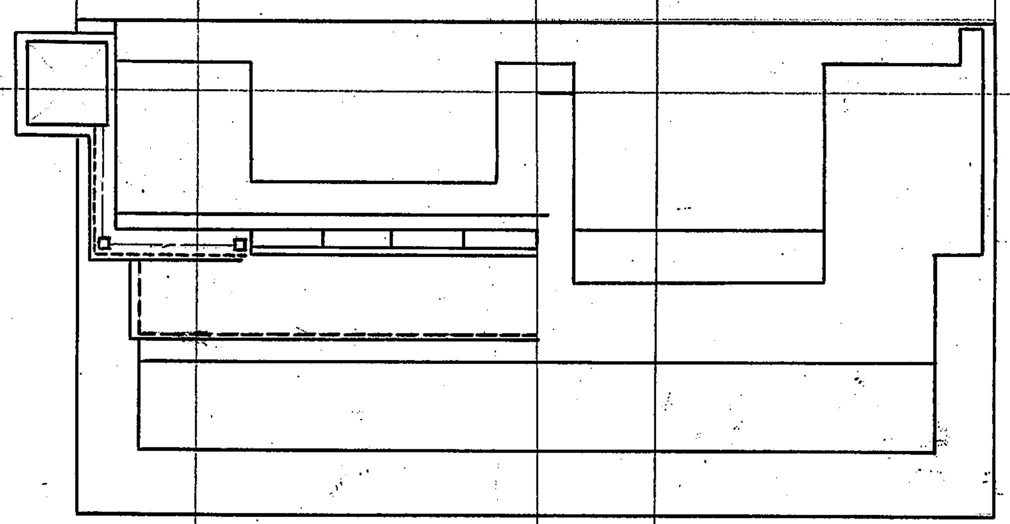
CHECKED BY M. Iida DATE 5-2-17 PAGE NO 4

CALCULATIONS FOR

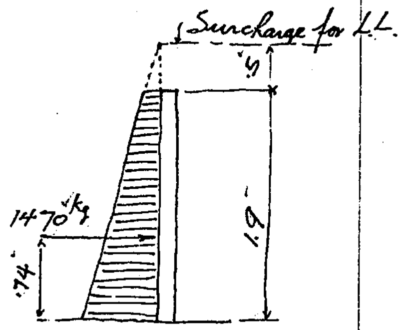
Design of Kisogawa Bashi for Aichiken.
Design of abutment. (East abutment)
General dimensions are as shown on sketch below.



General sketch of abutment.
Scale 1:100.

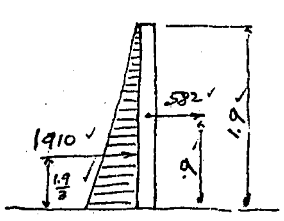


Design of parapet wall.



Earthpressure at normal state
$\frac{1}{3} \times 1600 \times .5 =$ 267
$\frac{1}{3} \times 1600 \times 2.4 =$ 1280
$1547 \div 2 =$ 774 kg/m² average.
$774 \times 1.9 =$ 1470 kg per meter strip of wall.
moment at bottom of wall.
$1470 \times .74 =$ 1088 kgm.

Earth pressure during earthquake $k = 0.3$ assumed
$1600 \times .662 \times \frac{1.9^2}{2} =$ 1910 kg
wt. of wall. $1.9 \times .425 \times 2400 = 1940$
Seismic force $1940 \times .3 = 582$ kg
moment at bottom of wall.
$1910 \times \frac{1.9}{3} =$ 1210
$582 \times 0.9 =$ ~~964~~ 524
1734 kgm. $1734 \div 1.8 = 964$ kgm.



moment at normal state governs the section of wall.

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CHECKED BY M. Iiida DATE 5-2-17 **PAGE NO** 5

CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken.

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

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

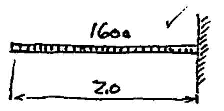
$d = \sqrt{\frac{1088 \times 100}{100 \times 7.18}} = 12.3\ cm$ use $47\ cm$ effective depth with $3\ cm$ insulation.

Steel area required $= \frac{1088 \times 100}{1200 \times \frac{7}{8} \times 47} = 2.2\ cm^2$ per lin meter.

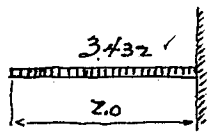
Use $13\ mm\phi$ bars at $30\ cm$ ctoc $= 4.4\ cm^2$ " on rear side
$13\ \phi$ " " 60 " $= 2.2$ " " on front side.

Design of wing wall

Section at 3 meter below top of wall.



Earth pressure at normal state $\frac{1}{3} \times 1600 \times 3 = 1600\ kg/m^2$
moment on wall $= 1600 \times \frac{2^2}{2} = 3200\ kgm$



Earth pressure during earthquake $= 1600 \times .662 \times 3.0 = 3180\ kg/m^2$
Seismic force on wall $= .35 \times 2400 \times .3 = \underline{252}$
3432

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

Seismic moment governs the section

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

Use $32\ cm$ effective depth with $3\ cm$ insulation or $35\ cm$ in total.
Steel area required $= \frac{686400}{1200 \times 1.8 \times \frac{7}{8} \times 32} = 11.35\ cm^2$ per meter strip.

Use $19\ mm\phi$ bars at $25\ cm$ ctoc $= 11.35\ cm^2$ "
Unit shear $= \frac{6864}{100 \times \frac{7}{8} \times 32} = 2.45\ kg/cm^2$ ok

Unit bond $= \frac{6864}{59.7 \times 4 \times \frac{7}{8} \times 32} = 10.2\ kg/cm^2 < 6 \times 1.8 = 10.8$ ok.

Section at $1.5\ m$ from top.

moment due to earth pressure and seismic force
$3180 \div 2 = 1590$
$\underline{252}$
$1842 \times \frac{2^2}{2} = 3684\ kgm.$

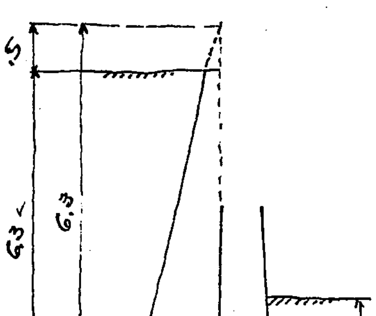
Steel area required $= \frac{368400}{2160 \times \frac{7}{8} \times 32} = 6.08\ cm^2$ per meter strip.

Use $16\ mm\phi$ bars at $25\ cm$ ctoc $= 8.04$

Design of curtain wall.

Span length assumed 4.0 meters

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



Earth pressure during earthquake $= .662 \times 1600 \times 5.8 = 6150$
Seismic force $1.1 \times 2400 \times .3 = \underline{800}$
$6950\ kg/m^2$

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

Effective depth required $= \sqrt{\frac{11120}{7.18 \times \frac{1}{8} 1.8}} = 29.3\ cm$ use $115\ cm$ total depth.

Steel area required $= \frac{1112000}{2160 \times \frac{7}{8} \times 110} = 5.35\ cm^2$ per meter strip at bottom

use $16\ mm\phi$ bars at $35\ cm$ ctoc $= 5.75$ "

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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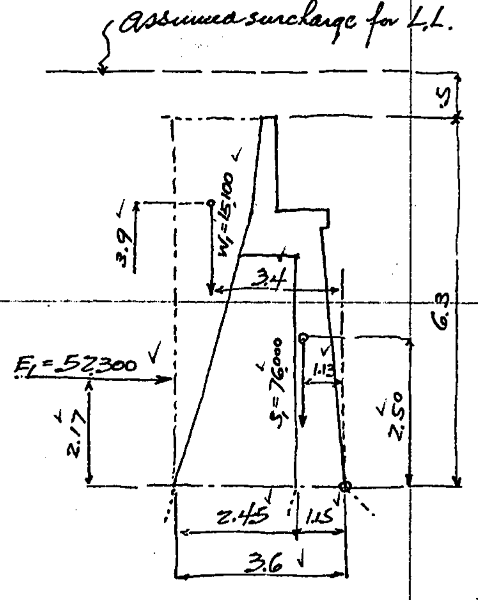
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken.

Design of Counterfort at center. Weight and center of gravity of counterfort at center, width 4.25 m

			arm	Vert m	arm	hor. m
Granite	.25×.3×4.25 = .318 @ 2600 =	830	6.15	5100	1.65	1370
parapet wall.	.425×1.6×4.25 = 2.89 @ 2400 =	6930	5.15	35660	1.66	11500
Top beam	1.8×1.0×4.25 = 7.65 @ " =	18350	3.88	71200	1.29	23650
Coping	.1×.3×4.25 = .13 @ " =	300	4.25	1280	.35	110
Curtain wall	.99×3.4×4.25 = 14.30 @ " =	34300	1.57	53800	.50	17150
Counterfort.	1.875×3.4×1.0 = 6.37 @ " =	15290	1.50	22900	2.09	31900
Sum of concrete	= 31.34 m³	76,000 kg	2.50 m	189940	1.13 m	85680
granite	.318					



Weight of earth on counterfort wall. 1.5×1×6.3 = 9.45 @ 1600 = 15,100 kg

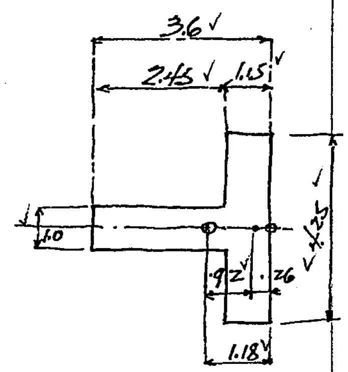
Earth pressure on front side neglected.

Earth pressure on rear side at normal state. $\frac{1}{3}$×1600×6.8 = 3637
$\frac{1}{3}$×1600×.5 = 267
3904 ÷ 2 = 1952 kg/m² average.

Total earth pressure E_1 = 1952×6.3×4.25 = 52,300 kg

Earth pressure during earthquake
E_1' = .662×$\frac{6.3^2}{2}$×4.25×1600 = 89,300 kg

Case 1. Stresses at normal state. Referring to the above sketch and taking moment about point O.



Loads	Hor. forces	Vert. forces		lev. arm		moment
S_1		76000	×	1.13	=	85800
w_1		15100	×	3.40	=	51300
E_1	52300			2.17	=	-113500
	52300	91100		0.26 m		23600

Center of gravity of section

1.15×4.25 = 4.88 × .575 = 2.805
1.0×2.45 = 2.45 × 2.375 = 5.810
7.33 × 1.18 m = 8.615

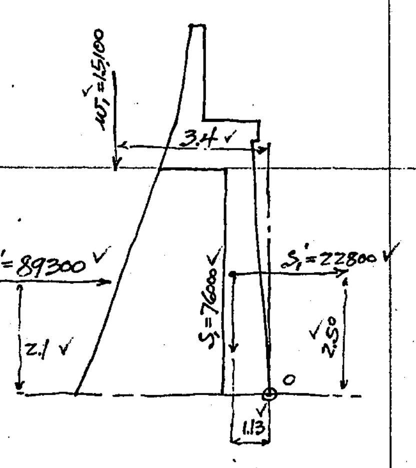
Eccentricity = 1.18 - .26 = 0.92 m

moment at bottom of section = 91100×.92 = 83800 kgm

Shear " " = 52,300 kg

Case 2. Stresses during earthquake.

Taking moment about O



Loads	Hor. forces	Vert. forces		lev. arm		moment
S_1		76000	×	1.13	=	85800
S_1'	22800		×	2.50	=	-57000
w_1		15100	×	3.40	=	51300
E_1'	89300		×	2.10	=	-187500
	112,100 kg	91100 kg		-1.18 m		-107400

Eccentricity = 1.18 + 1.18 = 2.36 m

moment at bottom section = 91100 × 2.36 = 215000 kgm.

Shear " " " = 112100 kg.

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

Use 9-22φ bars = 34.2 cm²

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

Steel ratio $p = \frac{34.2}{425 \times 350}$ = .00023

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

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

Neutral axis in the flange.

$k = \sqrt{2 \times 15 \times .00023 + (.00023 \times 15)^2} - .00023 \times 15$ = 0.08

$j = 1.0 - \frac{.08}{3}$ = .973

$f_c = \frac{f_s k}{n(1-k)} = \frac{2055 \times .08}{15 \times .92}$ = 11.9

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

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JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken.

Unit Shear $= \frac{112100}{100 \times \frac{7}{8} \times 350} = 3.7$ kg/cm² ok.

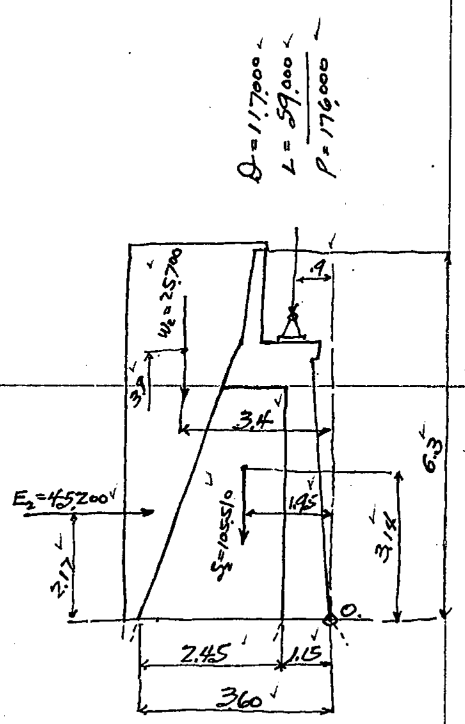
Unit bond $= \frac{112100}{6.91 \times 9 \times \frac{7}{8} \times 350} = 5.9$ ok.

Design of Counterfort under truss bearings.

Superimposed Loads on abutment.

Dead Load	117000
Live Load	59000
	176000 kg for one shoe.

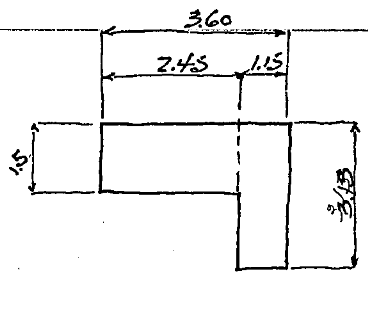
Weight and center of gravity



					arm vert. m		arm hor. m	
Granite	.25 × 3 × 1.63 =	.122 @ 2600 =	320	6.15	1980	1.65	530
Light pedestal	30 =	2.500 @ " =	6500	8.40	54600	3.70	24050
Handrail	3.0 @ .250 =	.75 @ " =	1950	6.80	13250	1.70	3310
parapet wall	.425 × 1.6 × 1.63 =	1.11 @ 2400 =	2660	5.15	13700	1.66	4410
Top beam	1.8 × 1.0 × 3.13 =	5.63 @ " =	13520	3.88	52400	1.29	17440
Coping	.1 × .3 × 3.13 =	.09 @ " =	220	4.25	940	.35	80
Projection	1.0 × 1.3 × 3.3 =	4.28 @ " =	10270	4.75	48750	3.70	38000
wing wall	.35 × 2.9 × 6.4 =	6.50 @ " =	15600	3.20	49900	2.90	45200
Coping	.1 × .3 × 3.0 =	.09 @ " =	220	6.25	1380	1.60	350
front wall	.99 × 3.4 × 3.13 =	10.53 @ " =	25300	1.57	39700	0.50	12650
Counterfort	1.875 × 3.4 × 1.70 =	10.83 @ " =	26000	1.50	39000	2.09	54300
wall	.42 × 2.0 × 1.70 =	1.43 @ " =	3430	4.8	16420	1.65	5640
less:	.2 × .3 × 3.4 =	−.20 @ " =	(−) 480	1.7	(−) 820	1.26	(−) 600
Sum of concrete	=	40.29 m³	105510 kg	3.14 m	331200	1.95 m	205360
" granite	=	3.372 m³					

Weight of earth on counterfort wall.

$1.5 \times 1.7 \times 6.3 \times 1600 = 25700$ kg



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

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

Center of gravity of section

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

Case 1. Stresses at normal state.

Taking moment about point O.

Loads.	Hor. forces	Vert. forces	Lev. arm	moment
P		176000 ×	0.90 =	158500
S_2		105510 ×	1.95 =	205800
W_2		25700 ×	3.40 =	87400
E_2	45200		× −2.17 =	−98000
	45200 kg	307210 kg	1.15 m	353700

Eccentricity $= 1.48 - 1.15 = 0.33$ m

moment at bottom section $= 307210 \times 0.33 = 101400$ kgm

Shear " " " $= 45200$ kg

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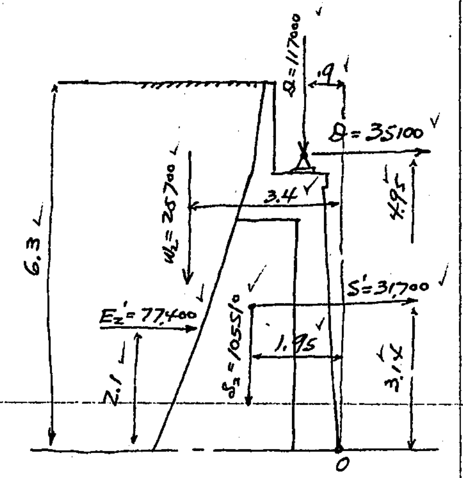
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken.

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



Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		117,000	0.90	− 105,300
D'	35,100		4.95	174,000
S_2		105,510	1.95	− 205,500
S_2'	31,700		3.14	99,500
w_2		25,700	3.40	− 87,400
E_2'	77,400		2.10	162,500
	144,200 kg	248,210 kg	0.15 m	37,800

Eccentricity = 1.48 + .15 = 1.63 m

moment at bottom section = 248210 × 1.63 = 404,500 kgm

Shear = 144,200 kg

Steel area required for moment = $\frac{404500 \times 100}{2160 \times \frac{7}{8} \times 350}$ = 61.1 cm²

Use 14 – 25 mmφ bars = 68.7 cm²

$f_s = \frac{404500 \times 100}{68.7 \times .957 \times 350}$ = 1758 kg/cm²

Direct comp. = $\frac{248210 \times 15}{727 \times 10000}$ = 51

1707 kg/cm² ok.

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

$t/d = 1.15/3.5 = 0.33$

neutral axis in the flange.

$k = 0.128$, $j = 0.~~897~~$ .957

$f_c = \frac{1758 \times .128}{15 \times .872}$ = 17.2

Direct comp. = $\frac{248210}{727 \times 10000}$ = + 3.5

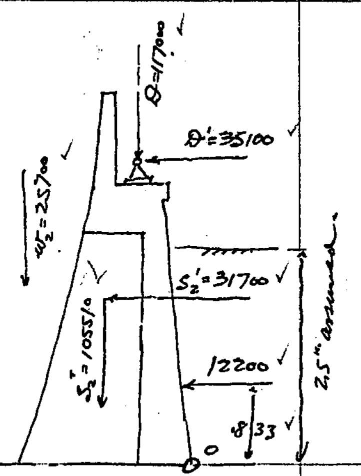
20.7 kg/cm² ok.

Unit shear = $\frac{144200}{150 \times .957 \times 350}$ = 2.9 " ok

Unit bond = $\frac{144200}{7.85 \times 14 \times .957 \times 350}$ = 3.9 " ok.

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

Earth pressure during earthquake on front side = $E_2' = .662 \times \frac{2.5^2}{2} \times 1600 \times 3.68$ = 12200 kg



Taking moment about point O.

Load.	Hor. forces	Vert. forces	Lev. arms	Moments.
D	"	117,000	0.90	105,300
D'	35,100		4.95	174,000
S_2		105,510	1.95	205,500
S_2'	31,700		3.14	99,500
w_2		25,700	3.40	87,400
E_2'	12200 ~~77,400~~		0.833	10,200
	79,000 kg	248,210 kg	2.75 m	681,900

Eccentricity = 2.75 − 1.48 = 1.27 m

moment at bottom section = 248210 × 1.27 = 315,000 kgm.

Shear = 79,000 kg

Steel area reqd for moment = $\frac{315000 \times 100}{2160 \times \frac{7}{8} \times 355}$ = 47.0 cm²

Use 8 – 22φ = 30.4
7 – 19φ = 19.8
50.2 cm²

$p = \frac{50.2}{150 \times 355}$ = .00095 $k = 0.155$ $j = .95$

$f_s = \frac{315000 \times 100}{50.2 \times .95 \times 355}$ = 1860

Direct comp. = $\frac{248210 \times 15}{727 \times 10000}$ = − 51

1809 kg/cm² ok.

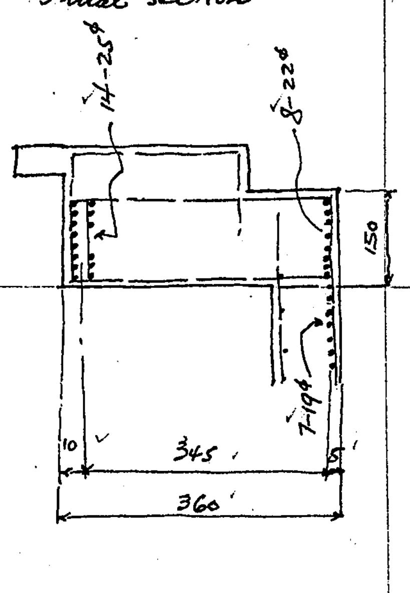
$f_c = \frac{1860 \times .155}{15 \times .845}$ = 22.7

Direct comp. = $\frac{248210}{727 \times 10000}$ = + 3.4

26.1 kg/cm² ok.

Shear & bond ok.

Final section



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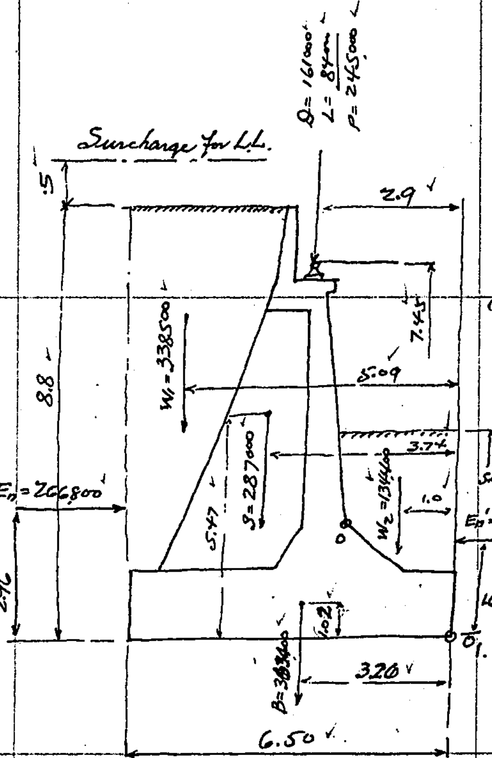
Design of Kisogawa Bashi for Aichi ken.

Stability of Abutment.

Superimposed Loads on Abutment.

Dead Load	161,000
Live Load	84,000
	245,000 kg on one abutment.



Weight and center of gravity of Shaft. arms about point O.

weight	vert. lev. arm	moment	hor. lev. arm	moment
76,000	2.50	189,940	1.13	85,680
105,510	3.14	331,200	1.95	206,080
105,510	3.14	331,200	1.95	206,080
287,020 kg	2.97 m	852,340	1.74 m	497,840

call this 287,000

Weight of Earth fill on Rear footing. arms from point toe O_1.

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

Weight of earth fill on front footing
$3.5 \times 2 \times 12 @ 1600 =$ 134,400 kg arm 1.0 m from toe O_1.

Earth pressure on Rear side

Normal state $\frac{1}{3} \times 1600 \times 0.5 = 267$ Surcharge for L.L. assumed 0.5 m
$\frac{1}{3} \times 1600 \times 9.3 = 4955$
$5222 \div 2 = 2611$ kg/cm² average.

Earth pressure $E_n = 2611 \times 8.8 \times 11.6 = 266,800$ kg arm 2.96 m.
During earthquake $E_e = .662 \times 1600 \times \frac{8.8^2}{2} \times 11.6 = 476,000$ kg " 2.93

Earth pressure on front side
normal state $E_n' = \frac{1}{3} \times 1600 \times \frac{5^2}{2} \times 11.6 = 77,300$ kg arm 1.67 m
during earthquake $E_e' = .662 \times 1600 \times \frac{5^2}{2} \times 11.6 = 153,700$ kg " 1.67

Summary of concrete for one abutment.

counterfort at center	31.34
" side 2 @ 40.29 =	80.58
base	150.91
	262.83 m³

Weight and center of gravity of Base.

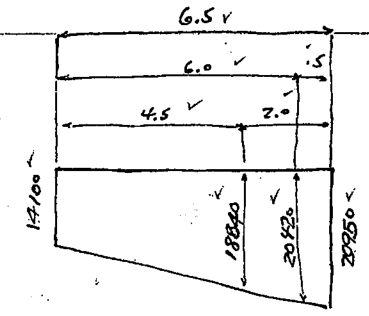
				vert arm	m	hor. arm	m
$2.05 \times 1.0 \times 10.5 =$	21.52 @ 2400 =	51,600	×	1.93	99,500	2.40	124,000
$4.0 \times 1.0 \times 2.3 =$	9.20 @ " =	22,100	×	2.00	44,200	4.80	106,000
$2.9 \times .55 \times 1.0 \times 2 =$	3.19 @ " =	7,700	×	2.00	15,400	4.90	37,700
$6.5 \times 1.5 \times 12.0 =$	117.00 @ " =	282,000	×	0.75	210,800	3.25	917,000
	150.91 m³	363,400 kg		1.02 m	369,900	3.20 m	1,184,700

Case 1. Stability at normal state.

Taking moment about toe O_1.

Refer to above sketch

Loads	Hor. forces	Vert. forces		Lev. arms		moments
P		245,000	×	2.90	=	+ 711,000
S		287,000	"	3.74	=	+ 1,073,000
W_1		338,500	"	5.09	=	+ 1,722,000
W_2		134,400	"	1.00	=	+ 134,400
B		363,400	"	3.20	=	+ 1,184,700
E_n	−266,800		"	2.96	=	− 790,000
E_n'	77,300		"	1.67	=	+ 129,000
	−189,500	1,368,300		3.04 m		4,164,100



Eccentricity = 3.25 − 3.04 = 0.21 m
Resultant force within middle third.

max. toe pressure = $\frac{1,368,300}{6.5 \times 12}\left(1 \pm \frac{6 \times .21}{6.5}\right)$ .194 = 20,950 kg/m² or (1.92 tons/ft²)
or 14,100

Max. load on one pile = $20.42 \times 1.0 \times 1.0 = 20.42$ kg tons.
If 50% be carried by sand foundation, pile load = 10.2 kg tons per pile.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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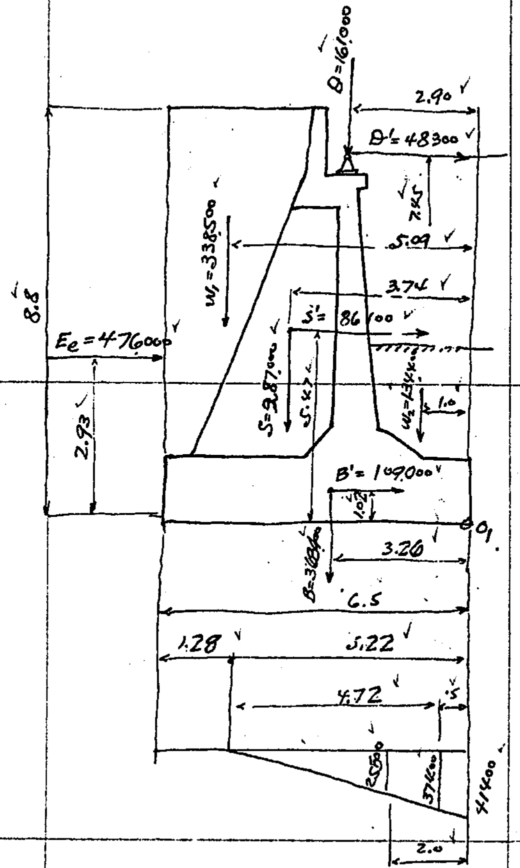
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi ken.

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



Taking moment about toe O_1

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		161,000	× 2.90 =	+ 467,000
D'	48,300		× 7.45 =	− 360,000
S		287,000	× 3.74 =	+ 1,073,000
S'	86,100		× 5.47 =	− 471,000
B		303,400	× 3.26 =	+ 1,183,000
B'	109,000		× 1.02 =	− 111,000
W_1		338,500	× 5.09 =	+ 1,723,000
W_2		134,400	× 1.00 =	+ 134,400
E_e	476,000		× 2.93 =	− 1,395,000
	719,400 kg	1,284,300 kg	1.74 m	2,243,400

Eccentricity = 3.25 − 1.74 = 1.51 m

Resultant force outside of middle third, neglecting tension at heel.

pressure area = 1.74 × 3 × 12 = 62.7 m²

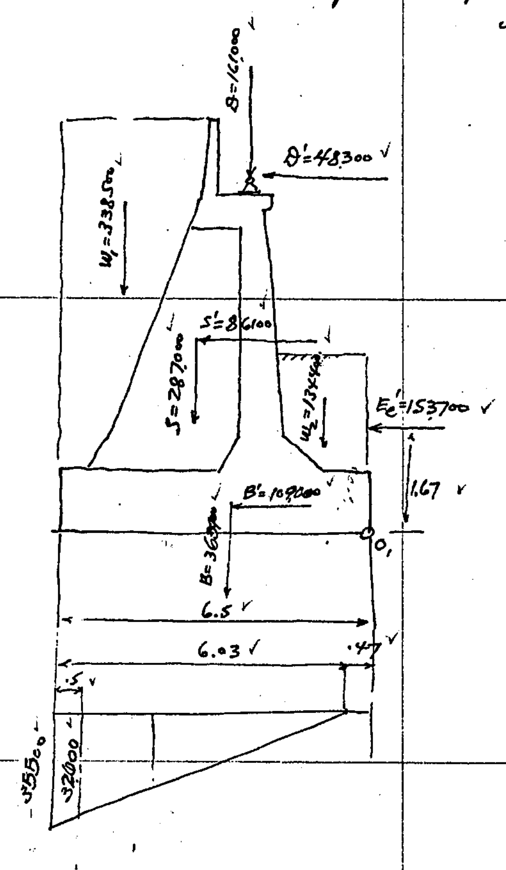
max. toe pressure = $\frac{1284300 \times 2}{62.7}$ = 41400 kg/m² or (3.78 tons/□')

max. load on one pile = 37.4 × 1.0 × 1.0 = 37.4 kg tons.

~~If 50% be assumed to be carried by sand foundation~~

pile load = 37.4 ÷ 2 = 18.7 kg tons per pile.

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



Taking moment about O_1.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		161,000	× 2.90 =	467,000
D'	48,300		× 7.45 =	360,000
S		287,000	× 3.74 =	1,073,000
S'	86,100		× 5.47 =	471,000
B		303,400	× 3.26 =	1,183,000
B'	109,000		× 1.02 =	111,000
W_1		338,500	× 5.09 =	1,723,000
W_2		134,400	× 1.00 =	134,400
E_e'	153,700		× 1.67 =	256,500
	397,100 kg	1,284,300	4.49 m	5,778,900

Eccentricity = 4.49 − 3.25 = 1.24 m

Resultant force outside of middle third, neglecting tension,

pressure area = 2.01 × 3 × 12 = 72.4 m²

max. bearing pressure at heel = $\frac{1284300 \times 2}{72.4}$ = 35500 kg/m² or (3.18 tons/□')

max load on one pile = 32.0 × 1.0 × 1.0 = 32.0

if 50% be assumed to be carried by sand foundation

max pile load = 16.3 kg tons / pile.

Design of Cantilever footing at toe.

Loads on cantilever footing at toe

at normal state case 1.

upward pressure 20950
18840
39790 ÷ 2 × 2 = 39790 kg

Earth fill on footing 3.5 × 1600 × 2 = −11200

footing, concrete 1.8 × 2400 × 2 = −8640

−19840

19950 kg upward.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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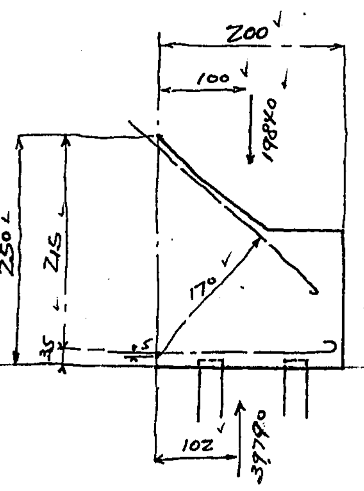
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi ken



moment on footing

39790 × 1.02 = 40550
19840 × 1.00 = −19840
19950 kg   20710 kgm

During Earthquake case 2.

upward pressure
41400
25500
66900 ÷ 2 × 2 = 66900 kg

moment on footing
66900 × 1.02 = 68200
19840 × 1.00 = 19840
47060 kg   48360 kgm

Effective depth req'd = $\sqrt{\frac{48360 \times 100}{100 \times 1.8 \times 7.18}}$ = 61 cm

Use effective depth of 215 cm with 5 cm insulation.

Steel area required = $\frac{48360 \times 100}{2160 \times \frac{7}{8} \times 215}$ = 11.91 cm² per meter strip.

Use 25 mm φ bars at 30 cm ctoc = 16.35 cm²

Steel ratio = $\frac{16.35}{215 \times 100}$ = .0008   k = .147; j = .951

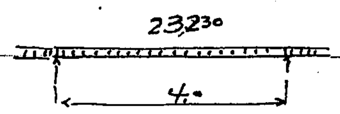
Unit shear = $\frac{47060}{100 \times .951 \times 215}$ = 2.3 kg/cm² ok.

Unit bond = $\frac{47060}{7.85 \times 3.33 \times .951 \times 215}$ = 8.8 " ok. < 6 × 1.8

Top reinforcements.
negative moment on footing 19840 kgm. upward pressure is small & neglected
effective depth being 170 cm.
Steel area req'd. = $\frac{19840 \times 100}{2160 \times \frac{7}{8} \times 170}$ = 6.17 cm²
Use 16 mm φ bars at 30 cm ctoc = 6.71 cm²

Design of footing at heel.

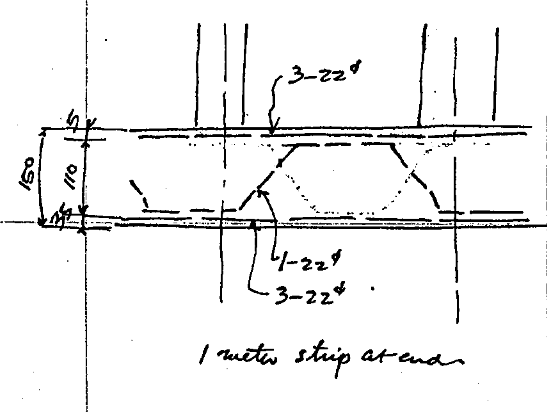
Span length assumed at 4.0 meters.
Upward pressure during earthquake case 3. for extreme meter strip = 35500 kg/m
downward " due to earth fill   7.3 × 1600 = 11670
footing concrete   1.5 × 2400 = 3600
−15270
20230 upward



Moment = $\frac{1}{10}$ × 20230 × 4.0² = 32408 kgm.
Shear = 20230 × 2 = 40460 kg
Eff. depth req'd = $\sqrt{\frac{32400 \times 100}{100 \times 1.8 \times 7.18}}$ = 50.1 cm   use 115 cm for eff. depth.

Steel area req'd = $\frac{32400 \times 100}{2160 \times \frac{7}{8} \times 115}$ = 14.9 cm² per meter strip.

Use 22 mm φ bars at 25 cm ctoc = 15.2 cm²   for extreme 1 meter.



upward pressure at case 2. = 0
downward pressure = 15270 kg/m
moment = $\frac{1}{10}$ × 15270 × 4² = 24400 kgm.
Steel area req'd. = $\frac{24400 \times 100}{2160 \times \frac{7}{8} \times 115}$ = 11.2 cm²

Use 3 − 22φ bars = 11.4 cm² per meter strip at end

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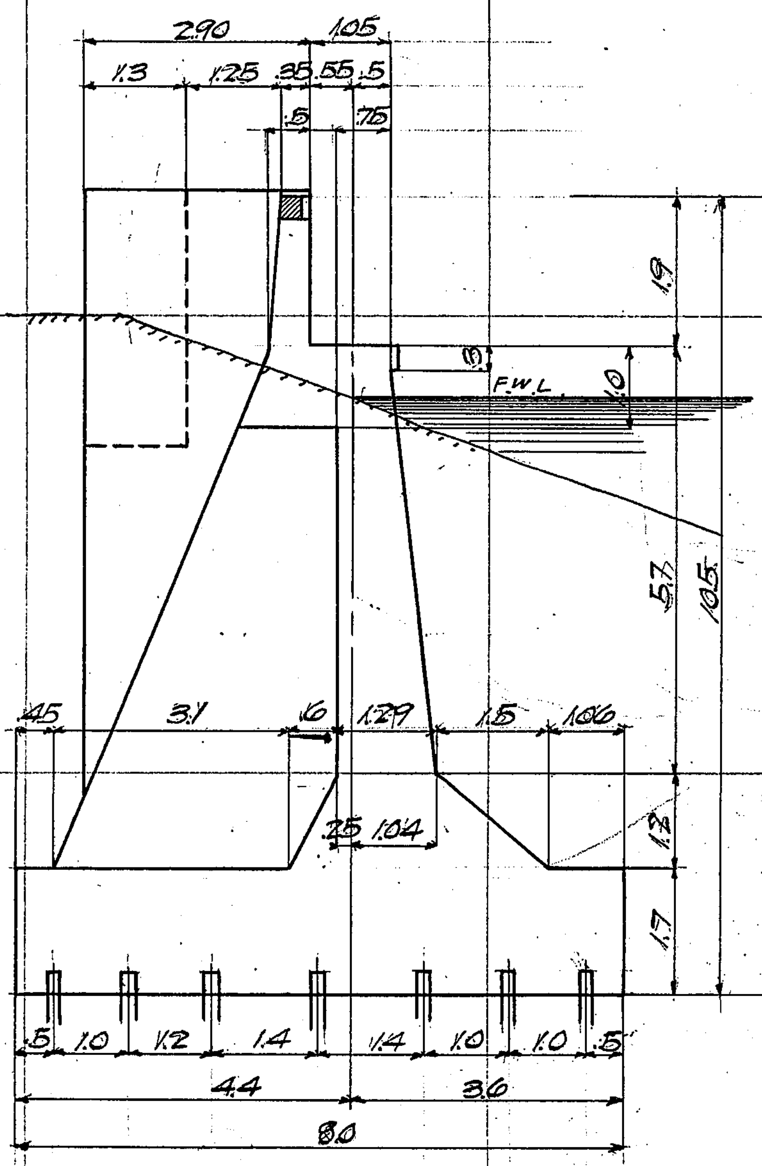
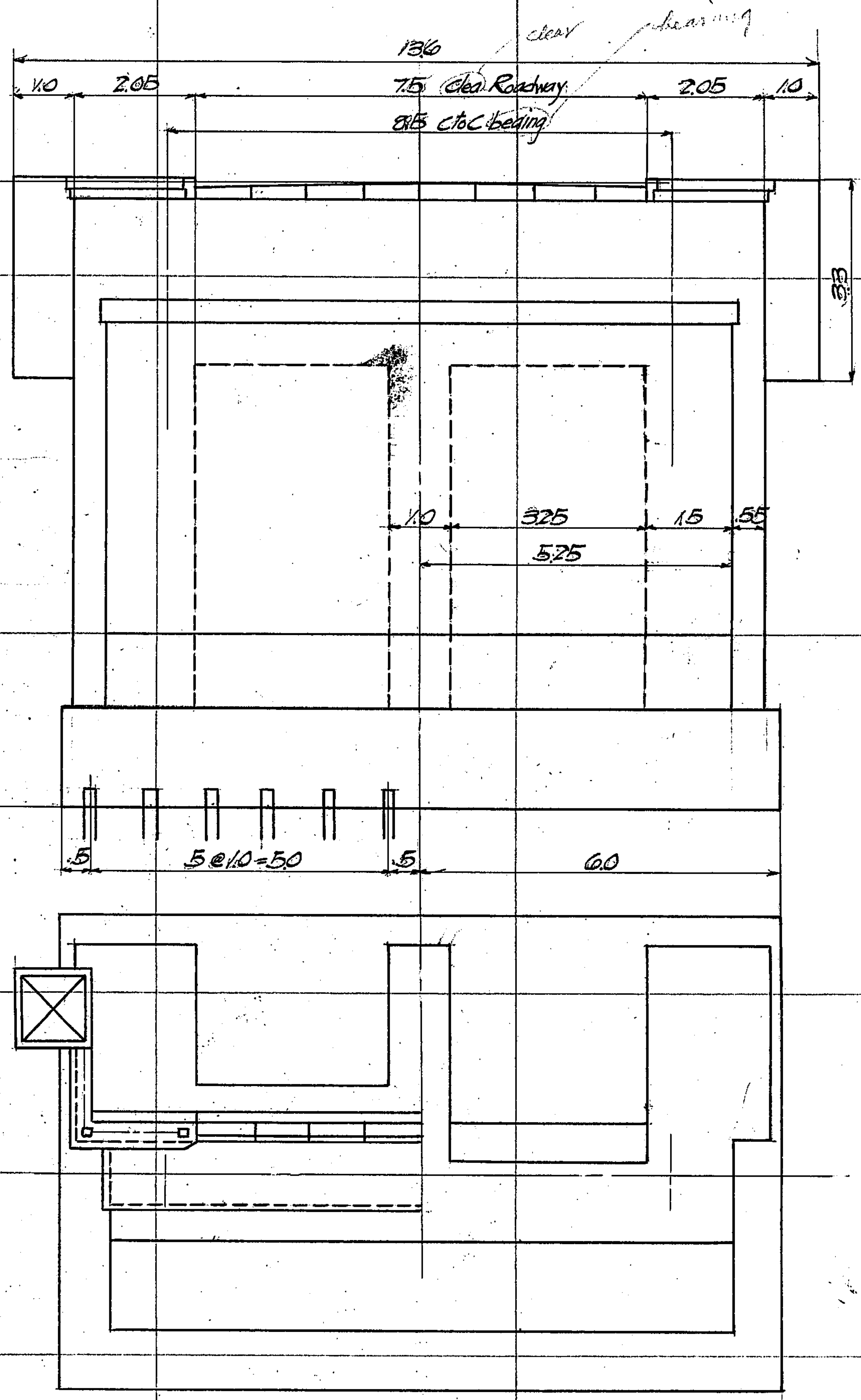
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi ken.

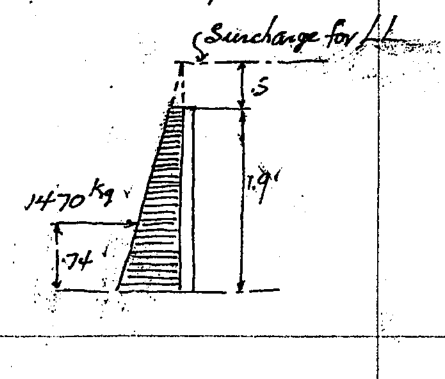
Design of west abutment.
General dimensions are as shown on sketch below.





General Sketch of west abutment
scale 1:100.

Design of Parapet wall.



Earth pressure at normal state

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

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

$1547 \div 2 = 774$ kg/m² average.

$774 \times 1.9 = 1470$ kg per meter strip of wall.

Moment at bottom of wall $1470 \times .74 = 1088$ kgm.

Earth pressure during earthquake. $k = 0.3$ assumed.

$1600 \times .662 \times \frac{1.9^2}{2} = 1910$ kg

Seismic force.

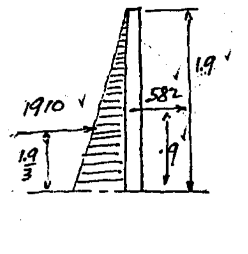
$1.9 \times .425 \times 2400 = 1940 \times .3 = 582$ kg

moment at bottom of wall.

$1930 \times \frac{1.9}{3} = 1210$

$582 \times .9 = 524$

1734 kgm



Moment at normal state governs the section of wall.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CONSULTIEG ENGINEEE
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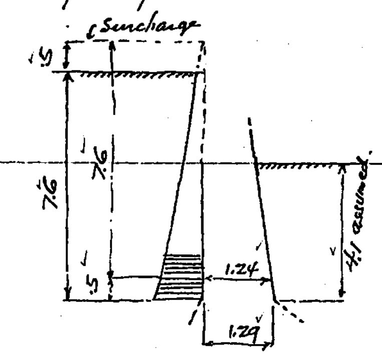
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken

Use same details as for parapet wall of east abutment.

Design of wing wall.
Use same details as for east abutment.

Design of Curtain wall. Span length assumed 4.0 meters



Earth pressure at normal state $= \frac{1}{3} \cdot 1600 \cdot 7.6 = 4050$
$\frac{1}{3} \cdot 1600 \cdot 3.6 = -1920$
2130 kg/m² average.

Earth pressure during earthquake
$.662 \cdot 1600 \cdot 7.1 = 7520$
Seismic force $1.24 \cdot 1200 \cdot .3 = 446$
7966 kg/m² average.

moment on wall $= \frac{7966 \cdot 4^2}{10}$ 12750 kgm. Shear $7966 \cdot 2 = 15932$ kg

Effective depth required $= \sqrt{\frac{12750 \cdot 100}{100 \cdot 7.18 \cdot 18}} = 31.4$ cm

Use 119 cm effective depth with 5 cm insulation ~~top~~ total depth = 124 cm.

Steel area required $= \frac{12750 \cdot 100}{2160 \cdot \frac{7}{8} \cdot 119} = 5.66$ cm² per meter strip.

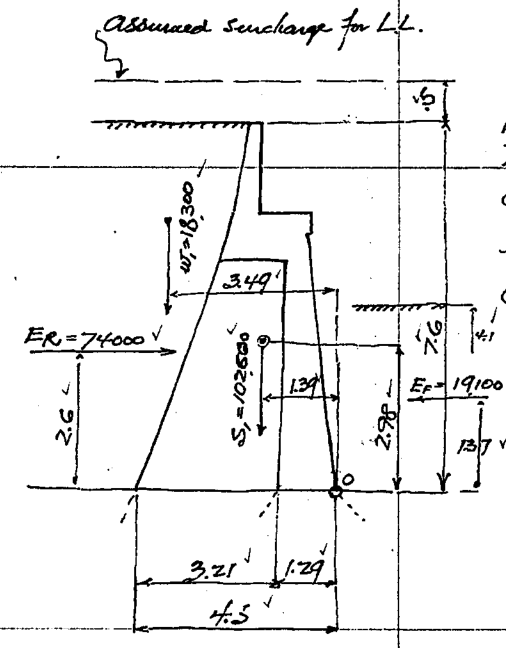
Use 16φ bars at 35 cm ctoc = 5.75 cm²

Unit shear $= \frac{15932}{100 \cdot \frac{7}{8} \cdot 119} = 1.53$ kg/cm² ok.

Unit bond $= \frac{15932}{5.03 \cdot 2.86 \cdot \frac{7}{8} \cdot 119} = 10.65$ kg/cm² < 10.8 ok.

Design of Counterfort at center.

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



					arm	vert. m.	arm	hor. m.
Granite	.25·.3·4.25 =	.318 @ 2600 =	830		7.45	6180	1.81	1500
Parapet wall	.425·1.6·4.25 =	2.89 @ 2400 =	6930		6.45	44670	1.80	12470
Top beam	1.80·1.0·4.25 =	7.65 @ " =	18350		5.18	95100	1.43	26240
Coping	.1·.3·4.25 =	.13 @ " =	300		5.55	1670	.49	150
Curtain wall	1.06·4.7·4.25 =	21.18 @ " =	50800		2.18	110750	.76	38600
Counterfort	2.25·4.7·1.0 =	10.57 @ " =	25350		1.88	47630	2.50	63400
Sum of concrete		42.42 m³	102560 kg		2.98 m	306000	1.39 m	142360
granite		0.318 m³	call this 102600 kg					

Weight of earth fill on counterfort wall.
$1.5 \cdot 1 \cdot 7.6 = 11.4$ @ $1600 = 18300$ kg $= w_1$

Earth pressure at normal state $= \frac{1}{3} \cdot 1600 \cdot .5 = 267$
$\frac{1}{3} \cdot 1600 \cdot 8.1 = 4315$
$4582 \div 2 = 2291$ kg/m² average

Earth pressure during earthquake $.662 \cdot \frac{7.6^2}{2} \cdot 4.25 \cdot 1600 = 130000$ kg $= E_R'$
" " at normal state $= 2291 \cdot 7.6 \cdot 4.25 = 74000$ kg $= E_R$ rear
" " $\frac{1600 \cdot 4.1^2}{6} \cdot 4.25 = 19100$ kg $= E_F$ front.

Case 1. Stresses at Normal state. Taking moment about point O in the above sketch.

Loads	Hor. forces	Vert. forces	lev. arm	moment
S_1		102600	1.39 =	142600
w_1		18300	3.49 =	63800
E_R	− 74000		2.60 =	− 192500
E_F	+ 19100		1.37 =	26200
	− 54900 kg.	120900 kg.	~~3.32 m~~ 0.33 m	40100

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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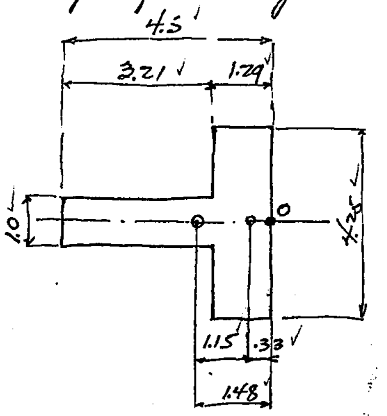
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichiken.



Center of gravity of section

$1.29 \times 4.25 = 5.48 \times .645 = 3.532$

$1.00 \times 3.21 = \underline{3.21} \times 2.895 = \underline{9.298}$

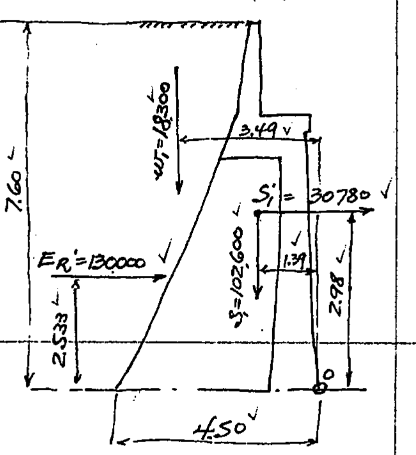
8.69^{m^2} 1.48^m 12.830

Eccentricity $= 1.48 - .33 = 1.15^m$

moment at bottom section $= 120,900 \times 1.15 = 139,000$ kgm.

Shear " " " $= 54,900$ kg

Case 2. Stresses during earthquake (seismic forces forward)



Taking moment about O.

Loads	Hor. forces	Vert. forces	Lev. arm	Moment.
S_1		102,600 ×	1.39 =	142,600
S_1'	− 30780	×	2.98 =	− 91,700
w_1		18,300 ×	3.49 =	63,900
E_R	− 130,000		2.533 =	−329,000
	−160,780 kg	120,900 kg ×	−1.77 m =	−214,200

Eccentricity $= 1.77 + 1.48 = 3.25^m$

moment at bottom of counterfort $= 120900 \times 3.25 = 392,800$ kgm.

Shear $= 160,780$ kg

Steel area required for moment $= \dfrac{392800 \times 100}{2160 \times \frac{7}{8} \times 440} = 47.2\ cm^2$

use 14 − 22 mmφ bars $= \cancel{45.6}\ 53.2\ cm^2$

$f_s = \dfrac{392800 \times 100}{\cancel{45.6}\ 53.2 \times \frac{7}{8} \times 440} = 1920$

Direct comp. $= \dfrac{120900 \times 1.5}{869 \times 10000} = -20$ 1900 kg/cm² ok

$f_c = \dfrac{1920 \times .089}{15(1 - .089)} = 12.5$

Direct comp. $= \dfrac{120900}{8.69 \times 10000} = 1.4$ 13.9 kg/cm² ok.

Unit shear $= \dfrac{160780}{100 \times .97 \times 440} = 3.76$ kg/cm² ok

Unit bond $= \dfrac{160780}{6.91 \times 14 \times .97 \times 440} = 3.90$ ok

Steel ratio $p = \dfrac{53.2}{425 \times 440} = .00029$

$\dfrac{t}{d} = \dfrac{1.29}{4.4} = 0.293$

neutral axis in the flange.

$k = \sqrt{2 \times .00029 \times 15 + (.00029 \times 15)^2} - .00029 \times 15$

$= \sqrt{.00874 + .00002} - .00435$

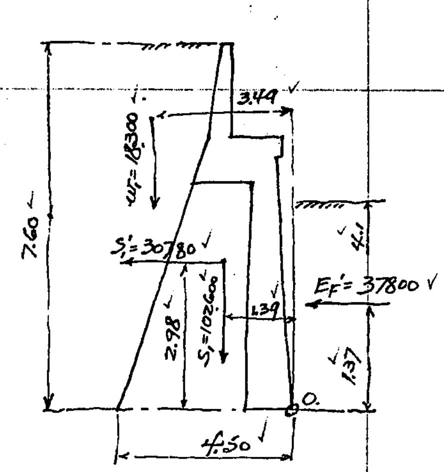
$= 0.089$

$j = 1 - \frac{1}{3}k = 0.97$

Case 3. Stresses during earthquake. (Seismic forces backward).

Earth pressure during earthquake $.662 \times 1600 \times \frac{4.1^2}{2} \times 4.25 = 37800$ kg $= E_F'$

Taking moment about point O.



Loads	Hor. forces	Vert. forces	Lev. arm	Moments.
S_1		102,600 ×	1.39 =	142,600
S_1'	30,780	×	2.98 =	91,700
w_1		18,300 ×	3.49 =	63,900
E_F'	37800	×	1.37 =	51,800
	68,580 kg	120900 ×	2.90 m =	350,000

Eccentricity $= 2.90 - 1.48 = 1.42^m$

moment at bottom $= 120,900 \times 1.42 = 171,700$ kgm

Shear $= 68,580$ kg.

Steel area req'd $= \dfrac{171700 \times 100}{2160 \times \frac{7}{8} \times 445} = 20.4\ cm^2$

use 8 − 19φ bars $= 22.7\ cm^2$

$f_s = \dfrac{171700 \times 100}{22.7 \times .962 \times 445} = 1767$

Direct comp. $= \dfrac{120900 \times 1.5}{869 \times 1000} = -20$ $\Big\}$ 1747 kg/cm² ok.

$f_c = \dfrac{1767 \times .115}{15 \times .885} = 15.3$

Direct comp. $= 1.4$ $\Big\}$ $= 16.7$ ok.

Unit shear $= \dfrac{68580}{100 \times .962 \times 445} = 1.6$ kg/cm² ok

Unit bond $= \dfrac{68580}{8 \times 5.97 \times .962 \times 445} = 3.35$ ok

Steel ratio $p = \dfrac{22.7}{100 \times 445} = .00051$

$pn = .00051 \times 15 = .0076$

$(pn)^2 = .00006$

$k = \sqrt{2 \times .0076 + .00006} - .0076$

$= 0.115$

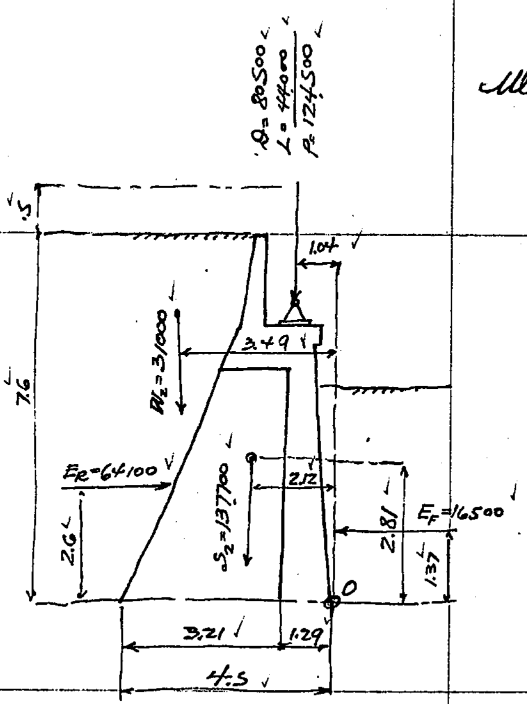
$j = 1 - \dfrac{.115}{3} = .962$

Design of Kiso-gawa Bashi for Aichi Ken.

Design of Counterfort under truss bearings.

Superimposed Loads on abutment.

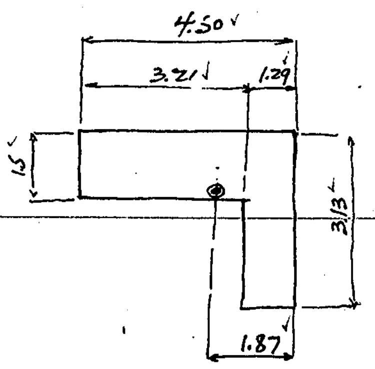
Dead Load	80500
Live Load	44000
	124500 kg. on one shoe.



Weight and center of gravity of counterfort under bearing.

				arm	vert m	arm	hor. m
granite	.25·.3·1.63 =	.122 @ 2600 =	320	7.45	2,380	1.79	570
Light pedestal		= 2.50 @ " =	6500	9.70	63,000	3.84	24960
Handrail	3.0·.250 =	.75 @ " =	1950	8.10	15,800	1.84	3,590
Parapet wall	.425·1.6·1.63 =	1.11 @ 2400 =	2660	6.45	17,150	1.80	4,780
Top beam	1.8·1.0·3.13 =	5.63 @ " =	13520	5.18	70,000	1.43	19,330
coping	.1·.3·3.13 =	.09 @ " =	220	5.55	1,220	.49	110
projection	1.0·1.3·3.3 =	4.28 @ " =	10270	6.05	62,100	3.84	39,400
wing wall	.35·2.9·7.68 =	7.80 @ " =	18720	3.84	71,900	3.04	56,900
coping	.1·.3·3.0 =	.09 @ " =	220	7.55	1,660	1.74	380
front wall	1.06·4.7·3.13 =	15.58 @ " =	37,400	2.18	81,500	.76	28,400
Counterfort	2.25·4.7·1.70 =	17.96 @ " =	43430	1.88	81,000	2.50	107,800
wall	.42·.2·1.70 =	1.43 @ " =	3430	6.10	20,900	1.80	6,170
less	.3·.2·4.7 =	(−) .28 @ " =	(−) 670	2.35	(−) 1,570	1.40	(−) 940
Summary concrete		53.69	137,670	2.81 m	387,040	2.12 m	291,450
" granite		3.372	Call this 137,700 kg				

assumed section.



Weight of Earth fill on counterfort.

$1.5 \times 1.7 \times 7.6 @ 1600 = W_2 = 31{,}000$ kg.

Earth pressure at normal state $= E_R = 2291 \times 7.6 \times 3.68 = 64100$ kg. rear

" $E_F = \frac{1600 \times 4.1^2}{6} \times 3.68 = 16.500$ kg front.

Earth pressure during earthquake $E_R' = .662 \times \frac{7.6^2}{2} \times 3.68 \times 1600 = 112{,}500$ kg rear

" ~~$E_F' = .662 \times \frac{4.1^2}{2} \times$ " " $= 32800$ kg front.~~

Center of gravity of assumed section

$1.29 \times 3.13 = 4.035 \times .645 = 2.605$

$1.5 \times 3.21 = \underline{4.815} \times 2.895 = \underline{13.930}$

8.850^{m^2} 1.87^m 16.535

Case 1. Stresses at normal state

Taking moment about point 0 in ~~the~~ above figure.

Loads	Hor forces.	Vert. forces.		Lev. arm		Moment.
P		124,500	×	1.04	=	129,500
S_2		137,700	×	2.12	=	292,000
E_R	−64100		×	2.60	= (−)	166600
E_F	16,500		×	1.37	=	22600
W_2		31,000	×	3.49	=	108,200
	~~80600~~ −47,600	293,200	:	1.32 m	=	385,700

Eccentricity $= 1.87 - 1.32 = 0.55$ m

moment at bottom section $= 293200 \times .55 = 161{,}300$ kgm

Shear $= 47{,}600$ kg

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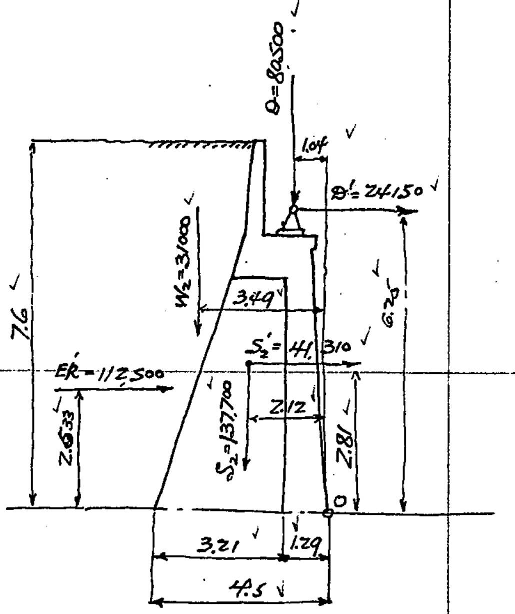
Design of Kisogawa Bashi for Aichi-ken.

Case 2. Stresses during Earthquake (Seismic forces for backward.)



Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lev. arm	Moments.
D		80,500 ×	1.04 =	83,700
D'	24,150	×	6.25 =	− 151,000
S_2		137,700 ×	2.12 =	292,000
S_2'	41,310	×	2.81 =	− 116,100
W_2		31,000 ×	3.49 =	108,200
E_R'	112,500	—— ×	2.533 =	− 285,000
	177,960	249,200 ×	~~4.16~~ −0.27 =	− ~~1036,000~~ 68,200

Eccentricity = 1.87 + 0.27 = 2.14 m

Moment at bottom Section = 249,200 × 2.14 = 533,000 kgm.

Shear = 177,960 kg

Case 2. governs.

Steel area required for moment = $\frac{533000 \times 100}{2160 \times \frac{7}{8} \times 440}$ = 64.1 cm²

Use 14 − 25 mm φ bars = 68.7 cm²

$f_s = \frac{533000 \times 100}{68.7 \times \frac{7}{8} \times 440}$ = 2015

Steel ratio = $p = \frac{68.7}{313 \times 440}$ = .0005

t/d = 1.29/4.40 = .293

neutral axis in flange.

$\ast = pn = 15 \times .0005 = .0075$

$(pn)^2 = .00006$

K = 0.115, j = .962

Direct comp. = $\frac{249200 \times 15}{885 \times 10,000}$ = −42

1973 kg/cm² ok.

$f_c = \frac{2015 \times .115}{15 \times .885}$ = 17.5

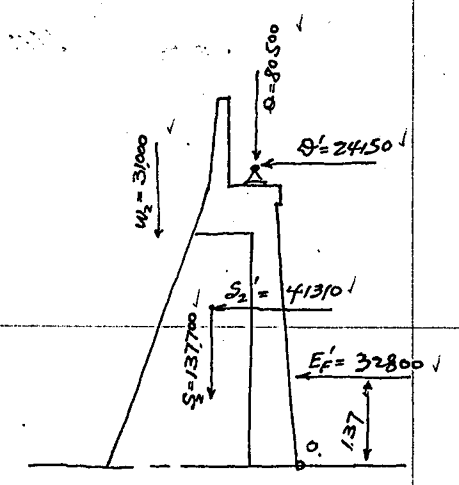
Direct comp. = $\frac{249200}{885 \times 10000}$ = 2.8

20.3 kg/cm² ok.

Unit Shear = $\frac{177960}{150 \times .962 \times 440}$ = 2.8 " ok

Unit bond = $\frac{177960}{7.85 \times 14 \times .962 \times 440}$ = 3.8 " ok.

Case 3. Stresses during Earthquake (Seismic forces backward)



Loads	Hor. forces	Vert. forces	Lev. arm	moment.
D		80,500 ×	1.04 =	83,700
D'	24,150	×	6.25 =	151,000
S_2		137,700 ×	2.12 =	292,000
S_2'	41,310	×	2.81 =	116,100
W_2		31,000 ×	3.49 =	108,200
E_F'	+32,800	×	1.37 =	44,900
	98,260	249,200	3.19 m	795,900

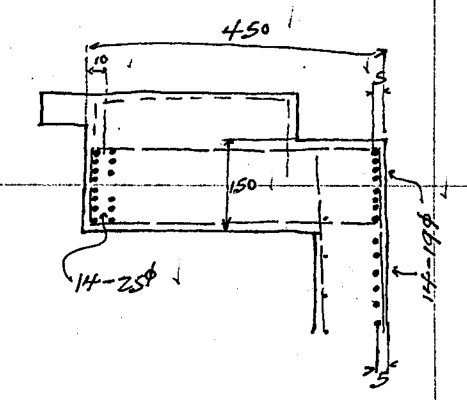
Eccentricity = 3.19 − 1.87 = 1.32 m

moment at bottom section = 249200 × 1.32 = 329000 kgm.

Shear = 98260 kg.

Steel area required for moment = $\frac{329000 \times 100}{2160 \times \frac{7}{8} \times 445}$ = 39.2 cm²

Use 14 − 19φ bars = 39.70 cm²



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JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY K. Inaba DATE 5-2-12 **FILE NO** A

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CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken

Stability of abutment

Superimposed load on abutment.

Dead load.	161,000
Live Load	84,000
	245,000 kg on one abutment.

Weight and center of gravity of shaft.

weight	vert arm	moment	hor. arm	moment
102600	2.98	306,000	1.39	142,360
137700	2.81	387040	2.12	291,450
137700	"	387040	"	291,450
378000 kg	2.86 m	1080080	1.92 m	725,260
	2.9		2.56	
	5.76		4.48	

Summary of concrete

Cantilefoot at center	42.42
" 2@53.69 =	107.38
	149.80
Base	211.2
	361.0 m3

Weight and center of gravity of Base.

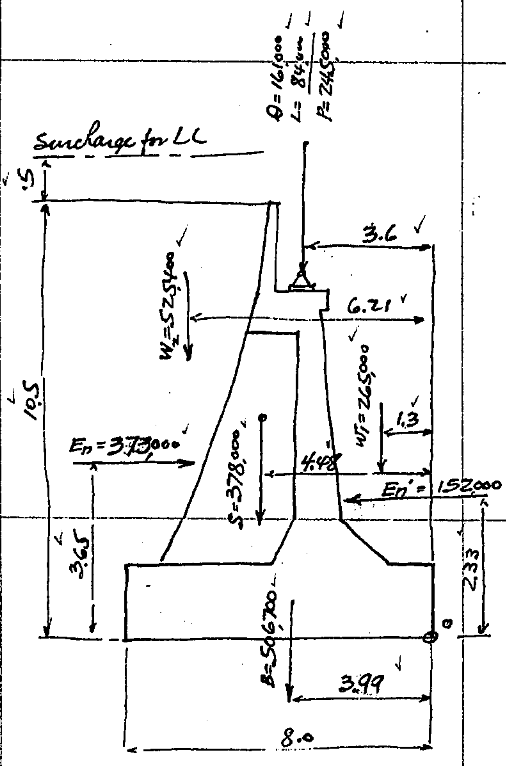
				vert arm	m	hor arm	moment
2.34 × 1.2 × 10.5 =	29.50	@ 2400 =	70800	2.21	156500	2.85	201,800
4.0 × 1.2 × 3.1 =	14.88	@ " =	35700	2.30	82,100	5.70	203,500
2.9 × .55 × 1.2 × 2 =	3.83	@ " =	9200	2.30	21,200	~~6.00~~ 5.60	51,500
8.0 × 1.7 × 12.0 =	163.00	@ " =	391000	.85	332200	4.00	1565,000
	211.21 m3	=	506700 kg	1.17 m	592,000	3.99	2021,800

Weight of earthfill on rear footing

3.35 × 8.80 × 11.30 @ 1600 =	533000 × 6.32 =	3368000
.85 × 5.90 × 11.30 @ " =	90700 × 4.27 =	387000
2.6 × 5.90 × 4.00 @ " =	(−) 98300 × 5.00 =	(−) 492000
	525400 kg 6.21 m	3263000

Weight of earth fill on front footing. Depth assumed. 5.30 m

$5.30 \times 2.6 \times 12.0 @ 1600 = 265,000$ kg



Earthpressure at normal state.

$\frac{1}{3} \times 1600 \times 0.5 = 267$

$\frac{1}{3} \times 1600 \times 11.0 = 5860$

$6127 \div 2 = 3064$ kg/m²

Rearside $E_n = 3064 \times 10.5 \times 11.6 = 373,000$ kg.

frontside $E_n' = \frac{1}{6} \times 1600 \times 7.0^2 \times 11.6 = 152000$

Earthpressure during earthquake.

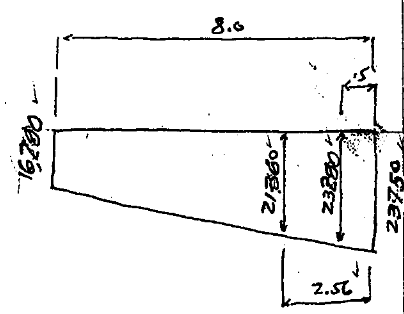
Rear side $E_e = 0.662 \times 1600 \times \frac{10.5^2}{2} \times 11.6 = 678000$ kg

front $E_e' = 0.662 \times 1600 \times \frac{7.0^2}{2} \times 11.6 = 301,000$ kg.

Taking moment about toe o.

Case 1 at normal state.

Load	Hor. forces	Vert. forces		Lev. arm	Moment
P.		245000	×	3.60 =	+ 882000
S		378000	×	4.48 =	+ 1,695000
B		506700	×	3.99 =	+ 2020000
w_1		265000	×	1.30 =	+ 345000
w_2		525400	×	6.21 =	+ 3,260000
E_n	−373,000		×	3.65 =	− 1361000
E_n'	152000		×	2.33 =	+ 354000
	−221,000	1920,100		3.73 m	7195000



Eccentricity $= 4.0 - 3.73 = 0.25$ m

Resultant force within middle third

max. toe pressure $= \frac{1920100}{8.0 \times 12.0}\left(1 \pm \frac{6 \times .25}{8.0}\right) = 23750$ kg/m² (2.17 tons/ft²) or 16,200

max. load on one pile $= 23.20 \times 1.0 \times 1.0 = 23.20$ kgtons.

If 50% be allowed to ~~carry~~ be carried by sand foundation

Pile load $= 23.20 \div 2 = 11.6$ kgtons per pile.

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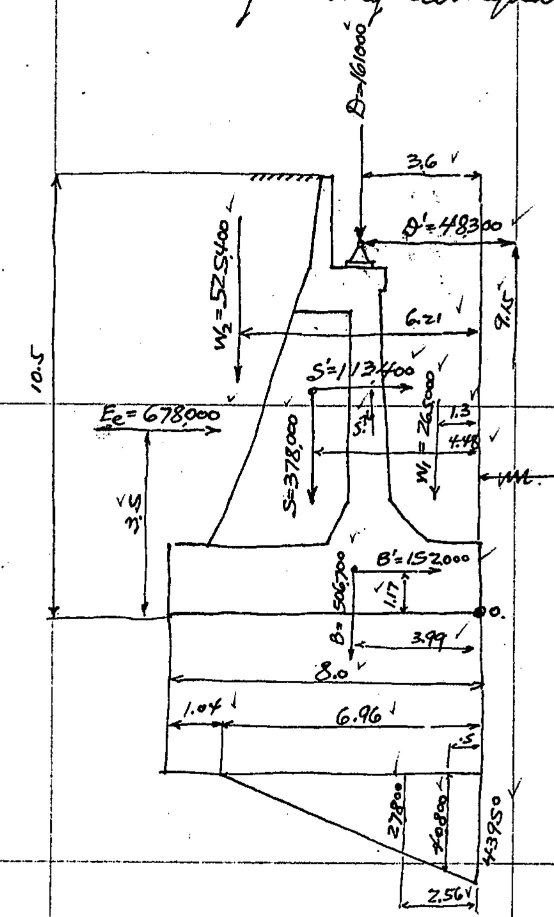
Design of Kisogawa Bashi for Aichi Ken.

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



Taking moment about toe O.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		161,000	× 3.60 =	579,500
D'	48,300		× 9.15 = −	442,000
S		378,000	× 4.48 =	1695,000
S'	113,400		× 5.76 = −	653,000
B		506,700	× 3.99 =	2,020,000
B'	152,000		× 1.17 = −	177,800
W1		265,000	× 1.30 =	344,000
W2		525,400	× 6.21 =	3262,000
Ee	678,000		× 3.50 = −	2370,000
	991,700	1836,100	2.32 m	4257,700

Eccentricity = 4.0 − 2.32 = 1.68 m

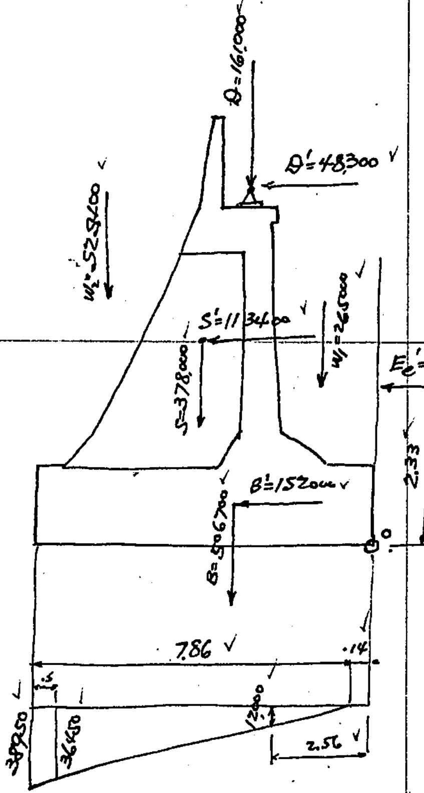
Resultant force outside of middle third. pressure area = 2.32 × 3 × 12.0 = 83.5 m²

max. toe pressure = $\frac{1836,100 \times 2}{83.5} =$ 43950 kg/m² or (4.0 tons/□')

max. load on one pile = 40.8 × 1 × 1 = 40.8 kg tons

$\frac{1}{2}$ · 40.8 = 20.4 kg tons / pile

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



Taking moment about O.

Loads	Hor. forces	Vert. forces	Lev. arm	Moments
D		161,000	× 3.60 =	579,500
D'	48,300		× 9.15 =	442,000
S		378,000	× 4.48 =	1695,000
S'	113,400		× 5.76 =	653,000
B		506,700	× 3.99 =	2020,000
B'	152000		× 1.17 =	177,800
W1		265,000	× 1.30 =	344,000
W2		525,400	× 6.21 =	3262,000
Ee'	301,000		× 2.33 =	701,000
	614,700 kg	1836,100 kg	5.38 m	9874300

Eccentricity = 5.38 − 4.0 = 1.38 m

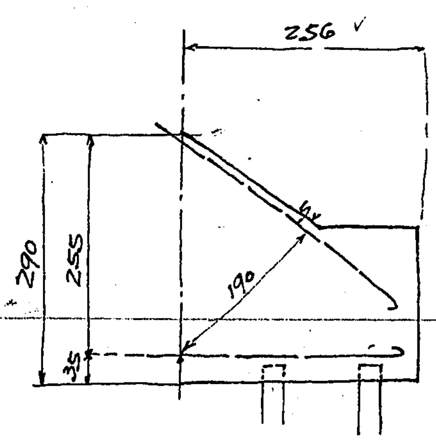
Resultant force outside of middle third. Pressure area 2.62 × 3 × 12 = 94.3 m²

max bearing pressure at heel = $\frac{1.836,100 \times 2}{94.3} =$ 38950 kg/m² ~ (3.56 tons/□')

max pile load = 36.45 × 1 × 1 = 36.45

½ × 36.45 = 18.2 kg tons.

Design of Cantilever footing at toe.



Loads on cantilever footing at normal state.

upward pressure 23750
21360
$\frac{45110}{}$ ÷ 2 = 22555 × 2.56 = 57700 kg arm 1.30 m

weight of earth fill on footing 5.0 × 1600 × 2.56 = 20500

" footing concrete 2 × 2400 × " = 12300

32800 kg arm say 1.25 m

moment on footing = 57700 × 1.3 = 75,000
− 32800 × 1.25 = − 41,000
24900 34,000 kgm. per meter strip.

Shear = 24900 kg

Design of Kisogawa Bashi for Aichi ken.

Loads on cantilever footing during earthquake. case 2.

upward pressure 43950
27800
$71750 \div 2 \times 2.56 = 91800$ kg arm 1.38 m

Downward pressure 32800 " 1.25

moment on footing = $91800 \times 1.38 = 126700$
$-32800 \times 1.25 = -41000$
59000 kg   85700 kgm. per meter strip.

moment and shear during earthquake governs.

Effective depth required $= \sqrt{\frac{85700 \times 100}{100 \times 1.8 \times 7.18}} = 81.5$ cm Use 255 cm effective depth.

Steel area required $= \frac{85700 \times 100}{2160 \times \frac{7}{8} \times 255} = 17.8$ cm² per meter strip of footing.

use 25 mmφ bars at 30 cm c to c = 16.35 cm²

Steel ratio $p = \frac{16.35}{255 \times 100} = .00064$, $pn = .00064 \times 15 = .0096$, $(pn)^2 = .00009$

$k = \sqrt{2pn + (pn)^2} - pn = 0.130$, $j = 1 - \frac{1}{3}k = 0.957$

$f_s = \frac{85700 \times 100}{16.35 \times .957 \times 255} = 2148$ kg/cm² $< 1200 \times 1.8$ ok.

$f_c = \frac{2148 \times .130}{15 \times .870} = 21.4$ " ok

unit shear $= \frac{59000}{100 \times .957 \times 255} = 2.42$ kg/cm² ok.

Unit bond $= \frac{59000}{7.85 \times 3.33 \times .957 \times 255} = 9.25 < 6.0 \times 1.8$ ok.

negative moment on cantilever footing at toe.

upward pressure $\frac{12000 \times 2.42}{2} = 14500$ kg

moment $-32800 \times 1.25 = -41000$
$14500 \times .81 = 11700$
-18300 kg -29300 kgm

Steel area required $= \frac{29300 \times 100}{2160 \times \frac{7}{8} \times 190} = 8.15$ cm² per meter strip of footing.

use 19 mmφ bars at 30 cm ctoc = 9.45 cm²

unit shear $= \frac{18300}{100 \times \frac{7}{8} \times 190} = 1.1$ kg/cm² ok

unit bond $= \frac{18300}{5.97 \times 3.33 \times \frac{7}{8} \times 190} = 5.5$ kg/cm² ok.

Design of footing at heel.

upward pressure during earthquake case 3. 36450 kg/m² for extreme meter strip.

downward pressure earth $1600 \times 8.8 = -14070$
footing $1.7 \times 2400 = -4070$
18310 kg/m² upward.

moment $18310 \times 4.0^2 \div 10 = 29300$ kgm Shear $= 18310 \times 2 = 36620$ kg

Steel area req'd. $= \frac{29300 \times 100}{2160 \times \frac{7}{8} \times 135} = 11.5$ cm²

use 19 mmφ bars 25 cm ctoc = 11.4 cm² for extreme meter strip.

downward pressure during earthquake case 2. 18140 kg/m²

use 19 mmφ bars 25 cm ctoc = 11.4 cm²

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MADE BY K. Inaba DATE 5-2-17 **FILE NO** A

CHECKED BY M. Kojima DATE 5-2-17 **PAGE NO** 20

CALCULATIONS FOR

Materials of Kisogawa Bashi for Aichi Ken.

Materials for East abutment.

Concrete 1:2:4 mixture.

parapet wall	.425 × 1.65 × 7.50	=	5.255
Top beam	1.00 × 1.774 × 5.45 × 2	=	19.340
less (end)	1.08 × 1.0 × .20 × 2	= (−)	.432
Coping	.1 × .3 × 12.8	=	.384
front wall.	.99 × 3.40 × 10.5	=	35.320
parapet end front.	.425 × 1.975 × 1.7 × 2	=	2.853
Coping	.10 × .33 × 3.70 × 2	=	.244
wing wall	.35 × 2.90 × 7.375 × 2	=	14.980
projection under pedestal	1.30 × .95 × 3.30 × 2	=	8.150
Counterfort wall	1.92 × 4.40 × 4.40	=	37.180
less. end.	.30 × .20 × 4.4 × 2	= (−)	.528
Base	2.06 × 1.0 × 10.50	=	21.630
"	1.50 × 6.50 × 12.00	=	[illegible]
			261.376 Cub-meters.

Reinforcements. Plain Bars. 6.663 kg tons. see detail drawing.

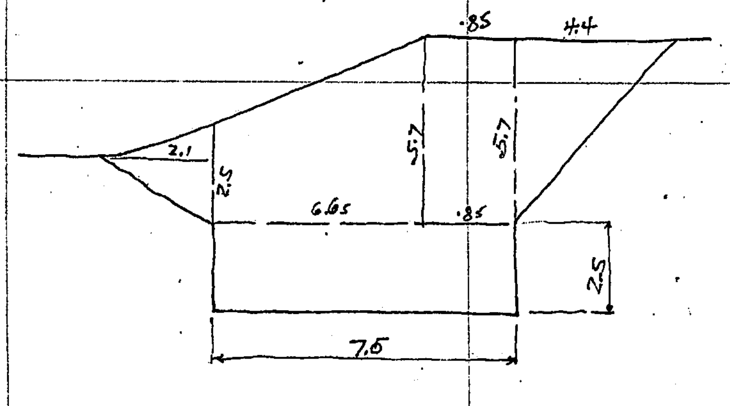
Forms.

Parapet wall	1.65 × 2 × 7.50	=	24.75
" end	1.975 × 2 × 3.75	=	14.80
" wing wall outside	2.90 × 7.375 × 2	=	42.25
" end	.35 × 7.375 × 2	=	5.16
" front	.55 × 5.40 × 2	=	5.94
" projection	.95 × 2 × 3.3 × 2	=	12.54
" "	.95 × 1.3 × 2	=	2.47
" Coping	.15 × 3.75 × 2	=	1.13
" inside	2.475 × 1.975 × 2	=	9.77
" "	1.40 × 5.40 × 2	=	15.12
Curtain wall front	6.0 × 10.5	=	64.00
" side	1.26 × 4.50 × 2	=	11.34
" "	2.06 × 1.0 × 2	=	4.12
Top beam back.	1.10 × 10.90	=	11.99
" bottom	1.18 × 3.25 × 2	=	7.67
Counterfort back	4.40 × 4.80	=	21.11
" Sides	1.92 × 4.4 × 4	=	33.80
Curtain wall back	3.25 × 3.4 × 2	=	22.10
" "	3.25 × 1.20 × 2	=	7.80
Base	1.50 × 12.0 × 2	=	36.00
"	1.5 × 6.5 × 2	=	19.50
			373.86 sq. meters.

Rubbles for Foundation	7.0 × 12.5 × 0.40	=	35.0 cub. meters.
piles 内地產赤松 末口	21cm 長 4.8m 6 × 12	=	72 piles.
granite 踏掛石	.25 × .25 × .93 × 8	=	0.465 cub. meters.

Excavations.

average section.



area.

2.5 × 2.1 ÷ 2	=	2.65
4.1 × 6.65	=	27.25
5.7 × .85	=	4.85
5.7 × 4.4 ÷ 2	=	12.55
		47.40 × 17.5 = 829 cub meters L.W. 以上
7.5 × 13.0 × 2.5		= 244 " L.W. 以下
		1073
	水替坊	7
		1080 cub meters

矢板延長

(7.6 + 13) × 2 = 41

水替坊 3

44 meters.

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MADE BY K. Inaba DATE 5-2-17 FILE NO A
CHECKED BY M. Kojima DATE 5-2-17 PAGE NO 21

CALCULATIONS FOR

Materials of Kisogawa Bashi for Aichi Ken.

人造洗出仕上				
wing wall outside	2.90 × 1.65 × 2	=	9.56	
〃 projection	0.5 × 1.3 × 2	=	1.30	
〃 〃	1.5 × .95 × 2	=	2.85	
〃 coping	.15 × 3.65 × 2	=	1.10	
〃 top.	0.50 × 3.45 × 2	=	3.45	
〃 〃	1.30 × 1.30 × 2	=	3.38	
Parapet front.	2.05 × 1.975 × 2	=	8.08	
〃 〃	.8 × .55 × 2	=	.88	
inside of coping	.20 × 4.25 × 2	=	1.70	
			32.30 sq. meters.	

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MADE BY K. Inaba DATE 5-2-17 **FILE NO** A

CHECKED BY M. Kojima DATE 5-2-17 **PAGE NO** 22

CALCULATIONS FOR

Materials of Kisogawa Bashi for Aichi Ken.

Materials for West abutment.

Concrete 1:2:4 mixture

Parapet wall.	.425 × 1.65 × 7.50	=	5.260
Top beam.	1.00 × 1.795 × 5.45 × 2	=	19.550
less (end)	1.08 × 1.0 × .20 × 2	= (−)	.432
Coping	.10 × .3 × 12.80	=	.384
Curtain wall	1.055 × 4.70 × 10.50	=	52.070
parapet end.	.425 × 1.975 × 1.70 × 2	=	2.855
coping	.10 × .33 × 3.70 × 2	=	.244
wing wall	.35 × 2.90 × 8.875 × 2	=	18.000
projection under pedestal	1.30 × .95 × 3.30 × 2	=	8.150
Counterfort walls.	2.36 × 4.40 × 5.90	=	61.250
less.	.30 × .20 × 5.80 × 2	= (−)	.696
wing end.	1.30 × .5 × .35 ÷ 2 × 2	=	.228
Bearing sheets	1.0 × 2.0 × .1 × 2	=	.400
Base	2.415 × 1.20 × 10.5	=	30.430
〃	1.70 × 8.00 × 12.0	=	163.200
			360.893 cub. meters

Reinforcements. Plain Bars. Kg tons see detail drawing. sheet no. 5.

Forms.

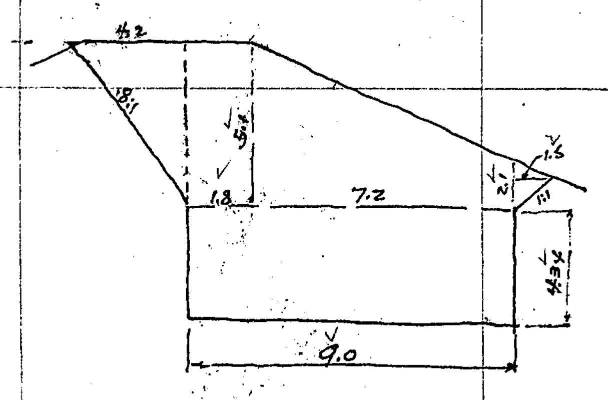
Parapet wall	1.65 × 7.50 × 2	=	24.75
〃 end.	1.975 × 2 × .375	=	14.80
wing wall outside	2.90 × 8.875 × 2	=	51.50
〃 end.	.35 × 8.875 × 2	=	6.21
~~front~~ 〃 front	.55 × 6.90 × 2	=	7.59
〃 projection	.95 × 2 × 3.3 × 2	=	12.53
〃	.95 × 1.3 × 2	=	2.47
〃 Coping	.15 × 3.75 × 2	=	1.13
〃 inside	2.475 × 1.975 × 2	=	9.78
〃	1.2 × 5.6 × 2	=	13.44
〃 edge	1.3 × .5 ÷ 2 × 2	=	.65
Curtain wall front	7.75 × 10.5	=	81.32
〃 sides	1.32 × 5.70 × 2	=	15.05
〃 〃	2.34 × 1.20 × 2	=	5.61
Top beam back	1.10 × 10.9	=	11.99
〃 bottom	1.22 × 3.25 × 2	=	7.93
Counter fort back.	6.40 × 4.40	=	28.15
〃 Sides	2.36 × 5.9 × 4	=	55.70
Curtain wall back.	3.25 × 4.70 × 2	=	30.55
〃	3.25 × 1.45 × 2	=	9.42
Base	1.70 × 12.0 × 2	=	40.80
〃	1.70 × 8.0 × 2	=	27.20
			458.57 sq. meters.

Rubbles for Foundation	8.5 × 12.5 × .40	=	42.50 cub. meters
piles 内地產赤松木口 21cm 長4.8m	6 × 12	=	72.0 本
granite 踏掛石	.125 × .25 × .93 × 8	=	0.465 cub. meters.

Excavation.

area.

4.2 × 5.1 ÷ 2 = 10.71
1.8 × 5.1 = 9.18
3.6 × 7.2 = 25.91
2.1 × 1.5 ÷ 2 = 1.58



47.38 × 17.4 = 824 cub. meters L.W. 17.E

9.0 × 13.0 × 4.34 = 507.2
1.5 × 1.5 × 4.34 = 9.8
517. cub. m L.W. 12.7
1341.

矢板延長 (9.0 + 13.0) × 2 = 44.0
水替协 3.0
47.0 meters.

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MADE BY K. Inaba DATE 5-2-17 **FILE NO** A
CHECKED BY M. Kojima DATE 5-2-17 **PAGE NO** 23

CALCULATIONS FOR

Materials of Kisogawa Bashi for Aichi ken.

人造洗ひ出し (west abutment)

wingwall	outside	2.90 × 1.65 × 2	=	9.56
〃	projection	0.50 × 1.30 × 2	=	1.30
〃	〃	1.50 × .95 × 2	=	2.85
〃	coping	.15 × 3.65 × 2	=	1.10
〃	top	0.50 × 3.45 × 2	=	3.45
〃	〃	1.30 × 1.30 × 2	=	3.38
parapet	front	2.05 × 1.975 × 2	=	8.08
〃	〃	0.8 × .55 × 2	=	.88
inside of coping		.20 × 4.25 × 2	=	1.70
				32.30 sq. meters.

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CHECKED BY ______ DATE ______ PAGE NO ______

CALCULATIONS FOR

昭和五年七月

愛知縣國道第壹號路線

木曽川橋々脚工事設計々算書

附材料計算書

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MADE BY K. Inaba DATE 5-7-12 FILE NO. Sub

CHECKED BY________ DATE________ PAGE NO. 1

CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken.

The total length of the new bridge is 878.81 meters (483 ken) between parapet walls of both abutments. Said total length is divided into 13 equal spans of 63.42 meters (209.3 R or 34.9 ken) and one 40.77 meter span between end bearings under the careful study of economical span length of this bridge.

The width of bridge is 7.5 meter wide clear between curb lines of roadway, paved with asphaltic block on reinforced concrete slab.

The handrails of throughout the bridge are made of cast iron and the pedestals and handrails at entrance over both abutments are of cutstone and ornamental design.

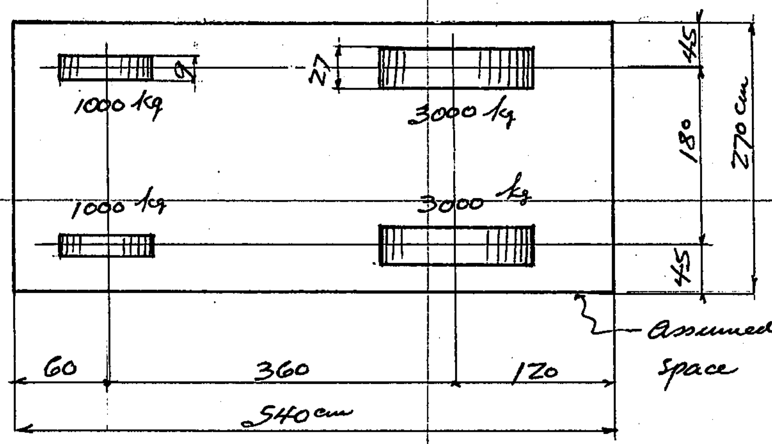
The findings of borings at the bridge site and also at the railway bridge upstream show the layer of fine sand at top ground and then clay soil of low bearing power until the firm sand is reached at a depth of 48 meters below ground line. It is impossible to sink the piers such a depth with the present practice of caisson sinking. Most practical method, therefore, in this case, is to use pneumatic process and carry the piers enough depth to get side friction and spread base to assure the designed safty.

Assumed loadings

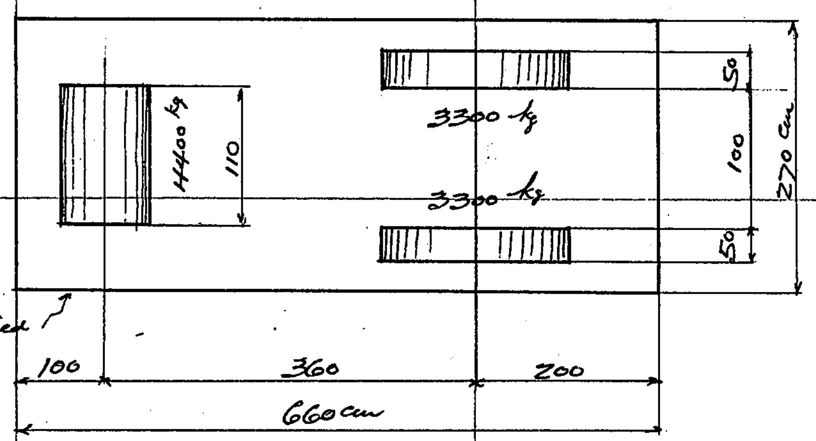
Uniform load on roadway : $w = \frac{100\,000}{170+l} \leq 500$ kg/m²

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

8 ton motor truck loading



11 ton road roller loading.



2 lines of motor traffic on roadway with occupied width of 270 cm each; unoccupied space around the motor trucks shall be filled with uniform load specified above.

One road roller on one span assumed.

Impact for motor truck loading coef $= \frac{20}{60+l}$

where l = loaded length in meter
max. impact 30%.

No impact for road roller and uniform live load.

Allowable working strength.

Concrete 1:2:4 mixture

Direct compression	35 kg/cm²
Fibre stress due to bending	45 "
Combined stress due to direct and bending (compression)	35 "
Punching shear of concrete	9 "
Shear of plain concrete	4 "
Bearing value	45 "
Bond stress for plain bars	6 "
" " deformed	9 "

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MADE BY K. Inaba DATE 5-7-12 **FILE NO** Sub

CHECKED BY ________ DATE ________ **PAGE NO** 2

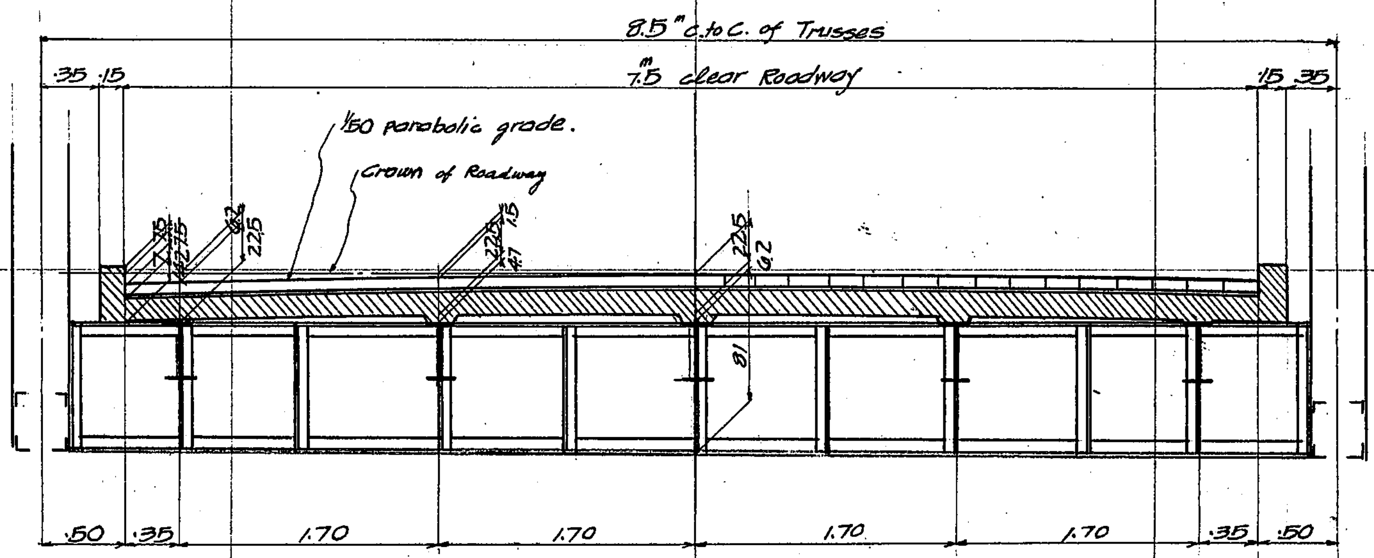
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi Ken.

	Reinforcing bars.	
	Tension or compression	1200 kg/cm²
	shearing strength	900 "
Structural steel		
	Tension net	1200 "
	Extreme fibre stress net	1200 "
	shear of web gross section	900 "
	Compression member	
	$1500(1-0.0055\ l/r)$ not over	1000 "
	where l = length of member in cm.	
	r = least radius of gyration in cm	
	Compression flange of girder	
	$1200(1-0.012\ l/b)$ not over	1100 "
	where l = unsupported length of flange in cm.	
	b = width of flange in cm.	
	Shear on shop driven rivets (machine driven)	850 "
	" " field " and turned bolt (")	750 "
	Shear on pin	900 "
	Bearing on shop driven rivets (machine driven)	1700 "
	" " field " "	1500 "
	" " pin	1800 "
	Roller $45d$ kg/cm where d = dia. of roller in cm	

Considering wind or temperature stress in addition to dead, live and impact stresses the allowable working strengths shall be increased 25%; in case of earthquake increase working strength by 60 %.
Seismic acceleration assumed as 3000 mm/sec² or $k = 0.30$

Cross section of bridge assumed as shown on Sketch below.



Scale 1:50.

5 cm asphalt block pavement.
2 cm cement mortar cushion
15.5 cm concrete slab.

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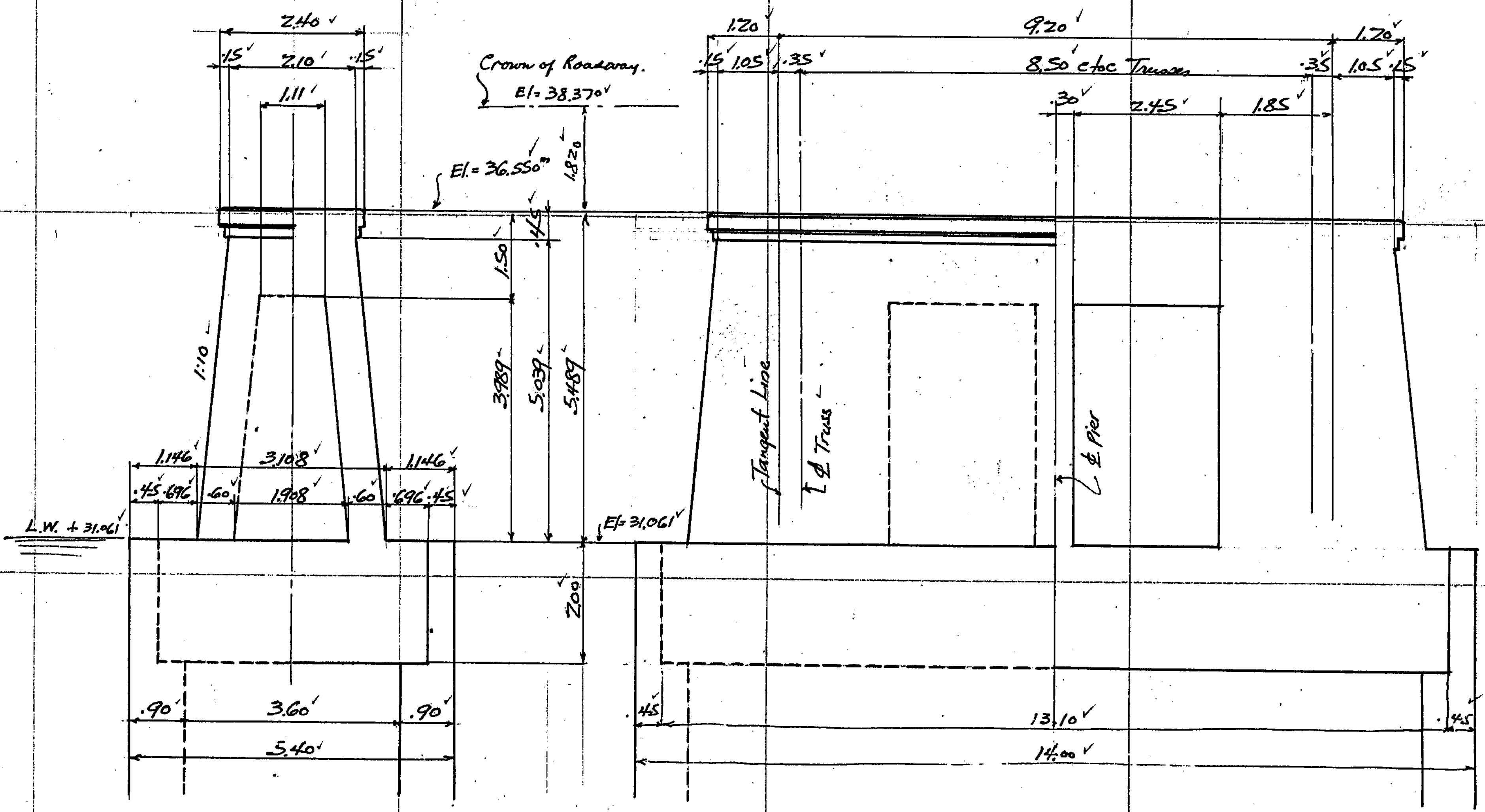
MADE BY K. Inaba DATE 5-5-15 FILE NO. Sub
CHECKED BY M. Jinda DATE 5-7-16 PAGE NO. 3

CALCULATIONS FOR

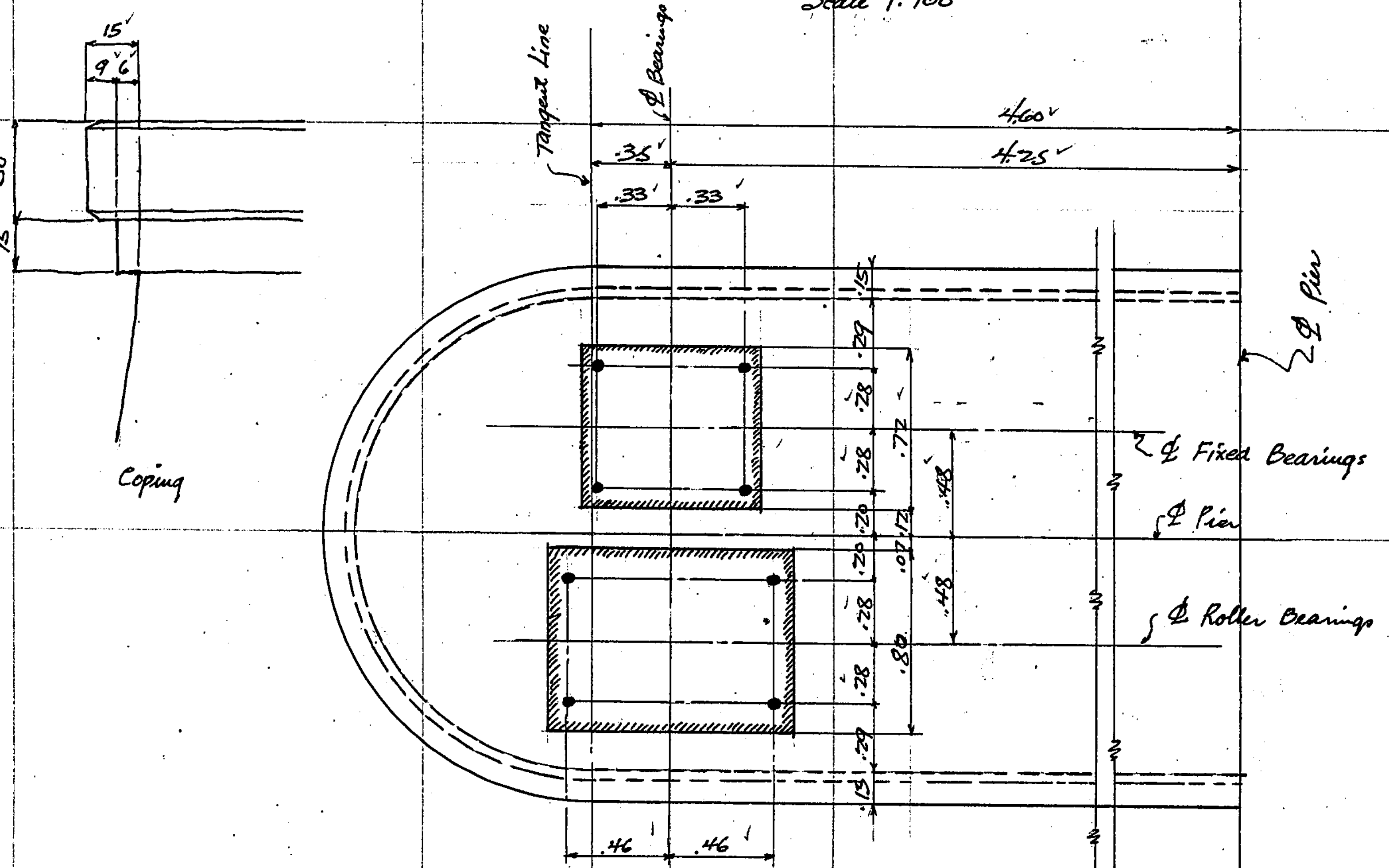
Design of Kisogawa Bashi for Aichi-Ken.

Design of piers

General dimensions are as shown on sketch below. (Height of shaft for Pier P8).



Scale 1:100



Scale 1:30

General Sketch of Pier.

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MADE BY *K. Inaba* DATE 5-5-15 FILE NO Sub

CHECKED BY *M. Jinda* DATE 5-7-16 PAGE NO 4

CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi-ken

Design of Shaft.

Weight and Center of gravity of shaft.

Coping.	2.40² · .30 =	1.355	× 5.339 =	7.230
"	2.40 · .30 · 9.20 =	6.620	× " =	35.320
"	2.22² · .15 =	.580	× 5.114 =	2.97
"	2.22 · .15 · 9.20 =	3.062	× " =	15.66
Shaft.	2.604² · 5.039 =	26.850	× 2.205 =	59.20
	2.604 · 5.039 · 9.20 =	120.600	× 2.358 =	284.50
Hollows less	1.509 · 2.45 · .3989 · 2 = (−)	29.470	× 2.305 =	−67.90
		129.597^{m^3} @ 2400 = 311,000 kg	× 2.60^m	336.98
Base of shaft.	4.50 · 2.00 · 13.1 =	118.000	× 1.00 =	−118.00
	Total concrete for shaft =	247.597^{m^3} @ 2400 = 594,000 kg	× 0.88^m =	218.98

above top of caisson

Superimposed Loads on shaft.

Dead Load 4 shoes @ 128000 = 512,000 kg on one pier.

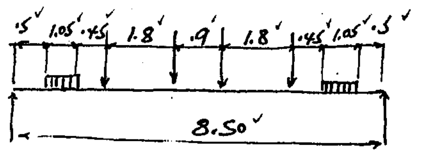
Live Load.

Uniform live load $= \frac{100,000}{170+126.84} = 337$ kg/m² loaded length assumed 2·63.42

Motor truck rear wheel concentration = 3000 kg

Impact. $= \frac{20}{60+126.84} = 10.7\%$ = 320

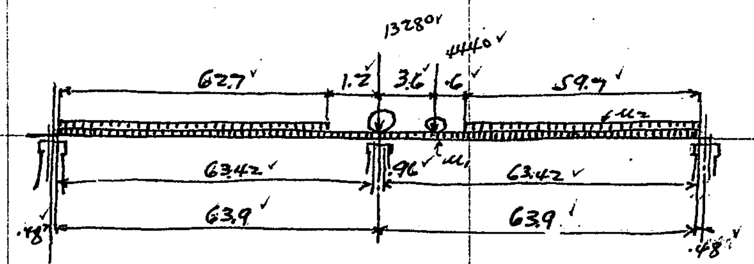
3320 kg × 4 = 13,280 kg



Front wheel concentration with impact say $\frac{3320}{3}$ = 1110 kg. × 4 = 4440

Unif. load on side of motor truck = 2.10 · 337 = 707 kg per lin. m. = M_1

Front and rear of truck = 5.4 · 337 = 1820 = M_2



M_2 $\frac{1820 \cdot 59.7^2}{2 \cdot 63.42} =$ 51,100

" $\frac{1820 \cdot 62.7^2}{2 \cdot 63.42} =$ 56,400

M_1 707 · 63.9 = 45,100

Front wheel 4440 · $\frac{60.3}{63.42}$ = 4,220

rear wheel = 13,280

170,100 kg for one pier.

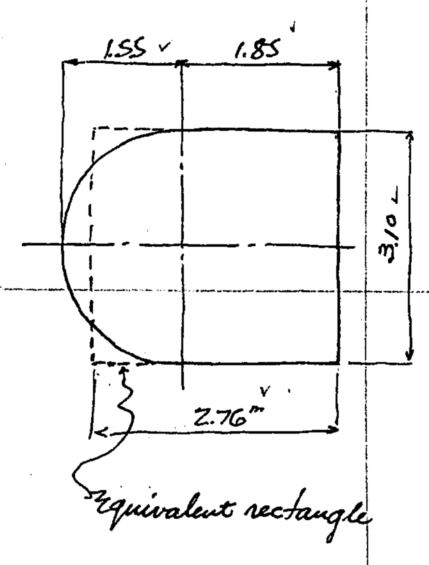
Call this 170,000

Summary of Superimposed Dead and Live Load.

Dead Load	512,000	÷2	256,000
Live Load	170,000	÷2	85,000
	682,000 kg on one pier.		341,000 kg on ½ shaft.

Stresses at bottom of shaft.

assumed bottom section.



Equivalent rectangular section of same moment of inertia.

moment of inertia of assumed section

semi. circle $0.0491 \cdot 3.1^4 \div 2 = 2.267$

rectangle $\frac{1.85 \cdot 3.1^3}{12} = 4.590$

6.857^{m^3}

length of equivalent bread rectangle b.

$\frac{b \cdot 3.1^3}{12} = 6.857$

$b = \frac{6.857 \cdot 12}{3.1^3} = 2.76^m$.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY K. Inaba DATE 5-5-16 FILE NO Sub

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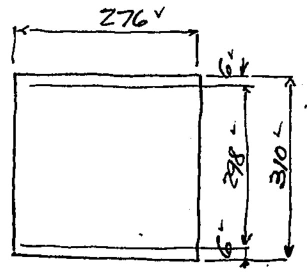
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi-Ken.

Stresses of shaft at bottom section

Case 1. Stability at normal state for full load.

Superimposed load = 341,000 kg on ½ shaft.
weight of ½ shaft = 155,500 kg
496,500 kg



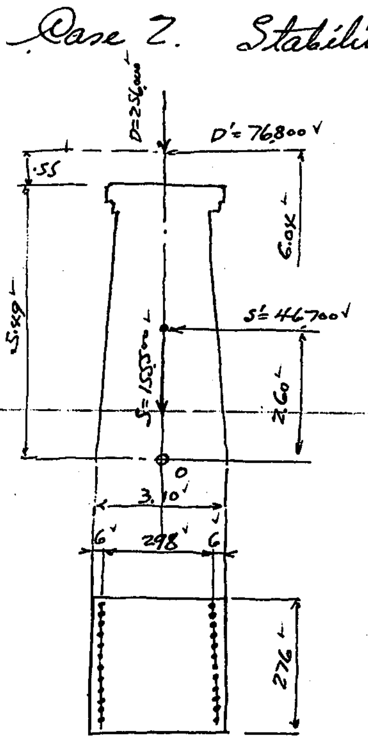
max. load on bottom section = 496,500 kg.

Unit bearing stress = $\frac{496,500}{310 \times 276}$ = 5.8 kg/cm² ok. = fc.

f_s = 5.8 × 15 = 87.0 kg/cm² C ok.

Case 2. Stability during Earthquake. k assumed 0.300

Dead load on one-half shaft = 256,000 kg = D
Seismic force 256000 × .3 = 76,800 " = D'
weight of ½ shaft = 155,500 " = S
Seismic force 155500 × .3 = 46,700 kg. = S'



Taking moment about center of bottom area. 0

Loads	Hor. forces	Vert. forces	Lever arms	moments.
D		256,000	0	0
D'	76800		6.04	465,000
S		155,500	0	0
S'	46,700		2.60	121,500
	123,500 kg	411,500 kg.	1.44 m	586,500

Reinforcements. try 10 - 25 mm φ = 49.1 cm²
49.1 × 2 = 98.2 cm² for both sides

Steel ratio $p_0 = 2p = \frac{98.2}{276 \times 310} = .00115$ $d'/h = \frac{6}{310} = .02$ $z/h = \frac{144}{310} = 0.465$

From the prepared diagrams of combined stress
k = .34   L = .073

$f_c = \frac{M}{Lbh^2} = \frac{586,500 \times 100}{.073 \times 276 \times 310^2} = 30.6$ kg/cm² ok.

$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 30.6\left(\frac{304}{.34 \times 310} - 1\right) = 867$ kg/cm² ok.

Unit shear = $\frac{123500}{276 \times \frac{7}{8} \times 304}$ = 1.7 kg/cm² ok

Unit bond = $\frac{123,500}{10 \times 7.85 \times \frac{7}{8} \times 304}$ = 5.92 " ok.

Assumed section is ample.

Use 22φ bars for top half of shaft at same spacing.

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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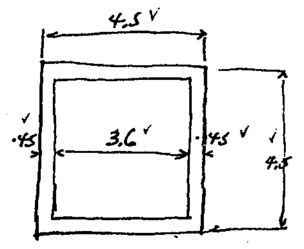
CALCULATIONS FOR

Design of Kissgawa Bashi for Aichi-ken

Base of shaft.

At normal state.

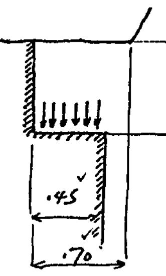


max. load = 341,000 kg (this load assumed to be supported by end partition only.)

bearing area say
4.5 × 4.5 = 20.25
3.6 × 3.6 = 12.95
7.30 m²

Unit bearing pressure = $\frac{341000}{7.3 \times 10000}$ = 4.7 kg/cm² ok.

$\frac{341000}{16.2}$ = 21100 kg per lin m of support.



moment on footing = 21100 × .475 = 10020 kgm per meter strip.

Effective depth req'd = $\sqrt{\frac{10020 \times 100}{100 \times 7.18}}$ = 37.4 cm

use eff. depth 190 cm with 10 cm insulation at bottom.

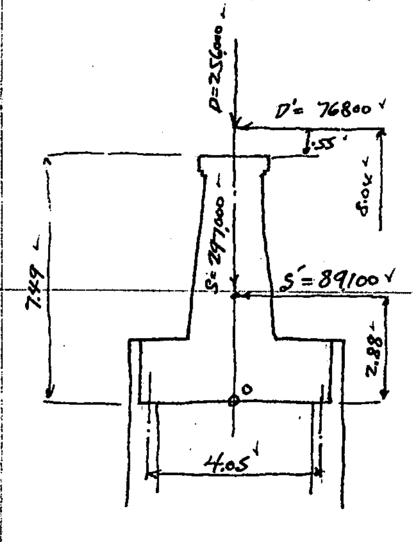
Steel area required = $\frac{10020 \times 100}{1200 \times \frac{7}{8} \times 190}$ = 5.02 cm² per meter strip

Use 5-25φ bars = 24.5 cm²

unit shear = $\frac{21100}{100 \times \frac{7}{8} \times 190}$ = 1.27 kg/cm² ok

unit bond = $\frac{21100}{5 \times 7.85 \times \frac{7}{8} \times 190}$ = 3.23 " ok.

During earthquake. k = .300.



Taking moment about O.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
D		256,000	0	0
D'	76800		8.04	617,000
S		297,000	0	0
S'	89,100		2.88	256,500
	165900	553000 kg	1.58 m	873,500

Direct bearing press. = $\frac{553000}{16.2}$ = 34100 kg per lin meter of support.

bearing press. due to moment = $\frac{873500}{4.05 \times 4.5}$ = ± 47900

Total pressure on support = + 82000 kg per lin m
or − 13800

unit bearing pressure = $\frac{82000}{45 \times 100}$ = 18.2 kg/cm² C. ok.

steel required for uplift = $\frac{13800}{1200 \times 1.6}$ = 7.19 cm² per meter

total steel area = 7.19 × 4.5 = 32.3 cm² for ½ shaft.
use 10 − 22φ bars = 38.1 cm² ok. for ½ shaft.

Moment on footing = 82000 × .475 = 38900 kgm

Eff. depth req'd = $\sqrt{\frac{38900 \times 100}{100 \times 7.18}}$ = 73.6 cm ok.

Steel area req'd. = $\frac{38900 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 190}$ = 12.18 < 24.5 ok.

~~unit shear = $\frac{82000}{100 \times \frac{7}{8} \times 190}$ = 4.94 kg/cm² < 4×1.6=6.4 ok~~

unit bond = $\frac{82000}{5 \times 7.85 \times \frac{7}{8} \times 190}$ = 12.55 kg/cm²

for ½ shaft.

unit bond = $\frac{82000 \times 4.5}{30 \times 7.85 \times \frac{7}{8} \times 190}$ = 9.42 kg/cm² < 6×1.6 = 9.60 ok.

neg. moment on footing = −13800 × .475 = −6,550 kgm

2400 × 2.0 × 0.72² ÷ 2 (wt. of footing) = −1250

7800 kgm.

Steel area req'd = $\frac{7800 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 190}$ = 2.44 cm²

~~use 7.5 − 22φ bars = 9.5 cm²~~

~~2400 × 2 × .72 = 3450~~ + 138 m = 17250 kg shear.

unit bond = $\frac{17250}{25 \times 6.91 \times \frac{7}{8} \times 190}$ = 6.0 kg/cm² ok.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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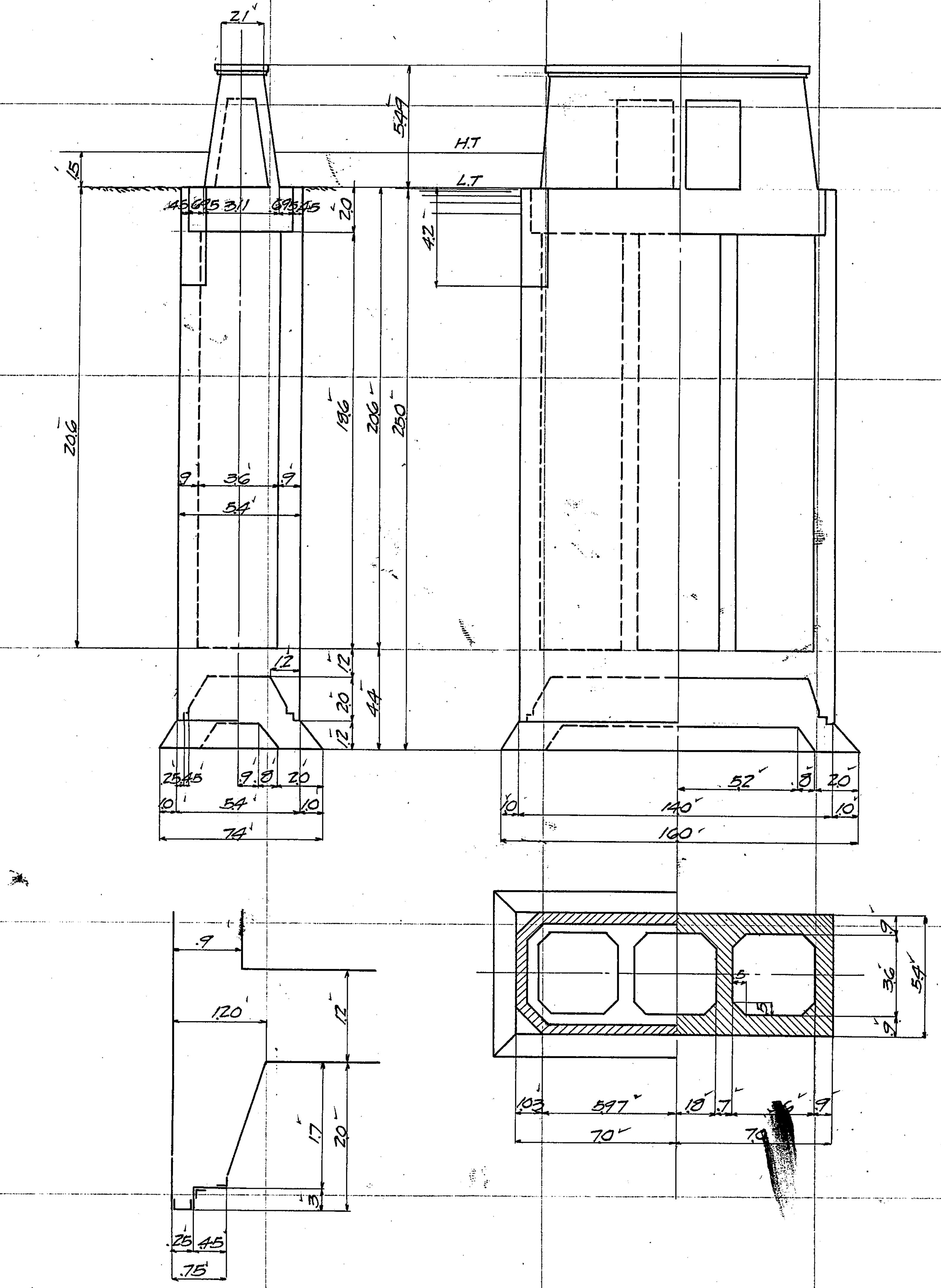
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken.
Pneumatic Caissons.
Land Caisson, Reinforced Concrete



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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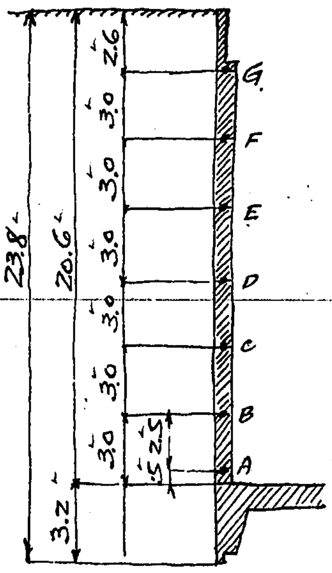
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken.

Sidewalls of Caisson



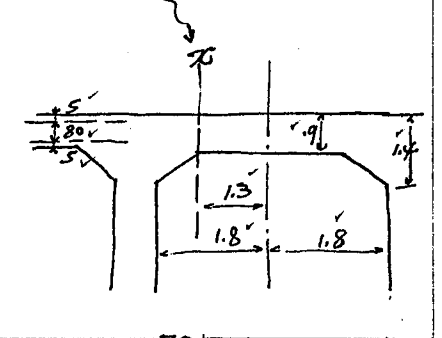
Earth pressure on side wall $\frac{1}{3} \times 1600 \times h = 533h$

At section			Earth pressure	water pressure	Final pressure after completion difference
A	1/3 × 1600 ×	20.1 =	10,700 kg/m²	−20,100 kg/m²	−9400 kg/m²
B	" "	17.6 =	9380	−17,600	−8220
C	" "	14.6 =	7780	−14,600	−6820
D	" "	11.6 =	6180	−11,600	−5420
E	" "	8.6 =	4580	−8,600	−4020
F	" "	5.6 =	2985	−5600	−2615
G	" "	2.6 =	1385	−2600	−1215

Span length assumed 4.40 m, moment = $\frac{1}{12} P \times 4.4^2 = 1.615P$ kgm.

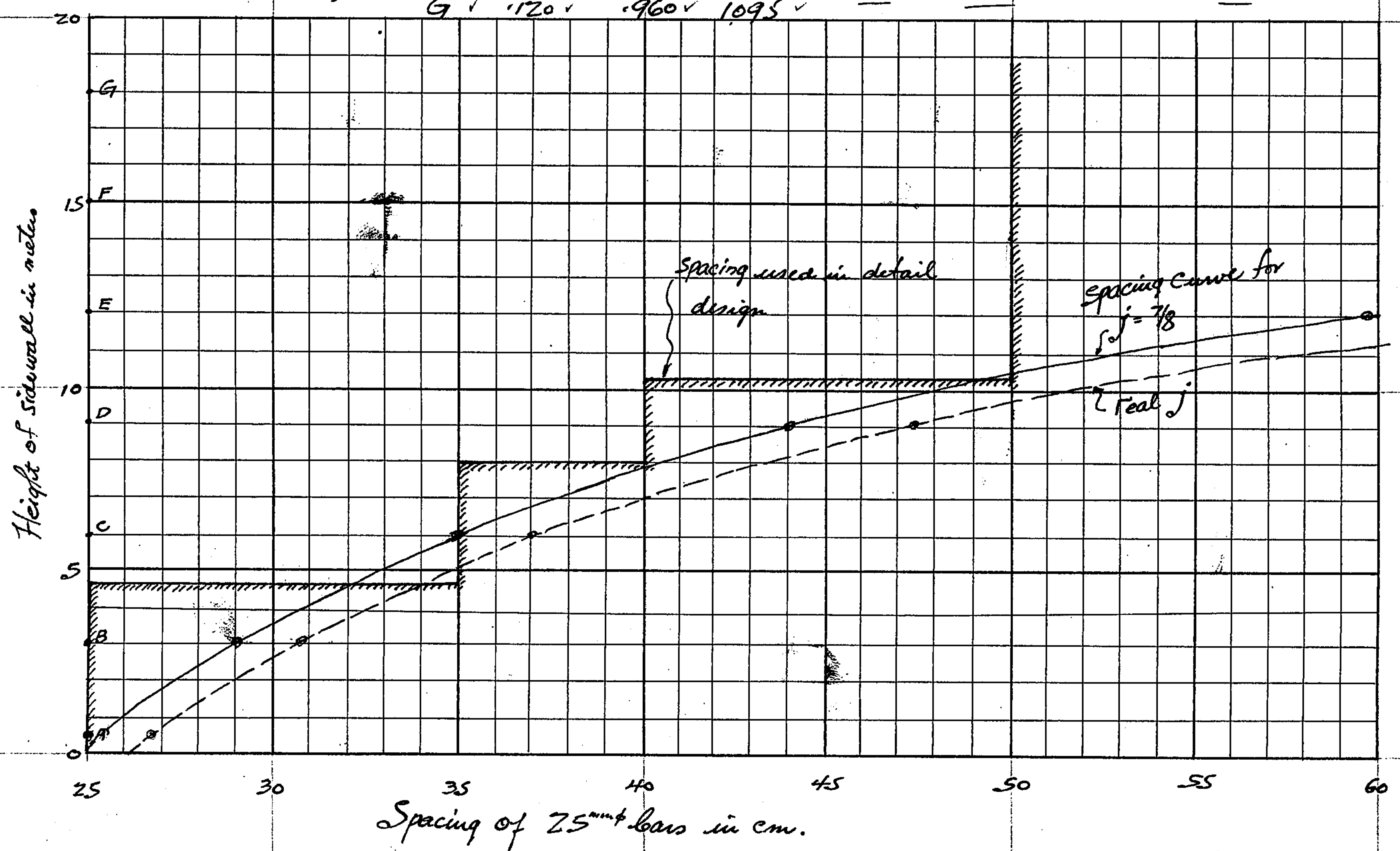
At section	Earth pressure	moment	Steel req'd for j=7/8	25 φ bars	k.	j.
A	P= 10700	17280 kgm	19.35 cm²	25.4 cm ctoc	.227	.925
B	9380	15150	16.96	29.0	.217	.928
C	7780	12560	14.07	34.9	.194	.935
D	6180	9980	11.17	44.0	.175	.942
E	4580	7390	8.27	59.6	.158 alt	.947 alt
F	2985	4820	5.40	90.9	.135	.954
G	1385	2235	2.50	196.4	.120	.960

x - point of max shear + bond



Stresses on steel and concrete for side wall.

At section	k.	j.	f_s	f_c	Shear at x.	max unit shear	max unit bond	Perimeter of steel
A	.227	.925	1136 kg/cm²	22.3	13900	1.77 kg/cm²	5.82 kg/cm²	30.4 cm
B	.217	.928	1133	21.0	12200	1.55	5.72	27.1
C	.194	.935	1124	18.1	10,110	1.27	5.65	22.5
D	.175	.942	1117	15.8	8030	1.00	5.65	17.7
E	.158	.947	1110	13.9	5950	0.73	5.53	13.2
F	.135	.954	1102	—	—	—	—	—
G	.120	.960	1095	—	—	—	—	—



Spacing Diagram of Sidewall Reinforcements 25mmφ.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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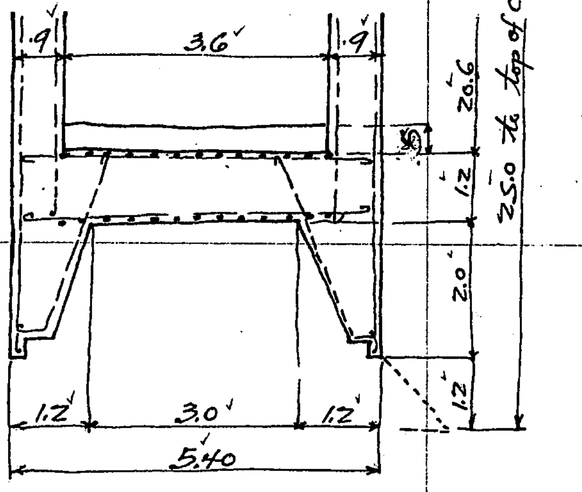
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Design of working chamber.

Ceiling Slab.



Span length assumed as follows.

transverse span = 3.60 meters

longitudinal span = 4.30

Theoritical air pressure = 26.5m @ 1000 = 26500 kg/m²

weight of concrete 1.20+.5 = 1.70 @ 2400 = - 4080

22420

Load on 3.6m span $22420 \times (1.5 - \frac{3.6}{4.3}) =$ 14850 kg/lin m of span

" " 4.3 " $22420 \times (\frac{3.6}{4.3} - 0.5) =$ 7570 "

transverse moment = $\frac{14850 \times 3.6^2}{10}$ = 19250 kgm per meter strip

longitudinal moment = $\frac{7570 \times 4.3^2}{10}$ = 14000 kgm " "

Effective depth required = $\sqrt{\frac{19250 \times 100}{100 \times 7.18}} = 51.8$ cm

Use 115 cm effective depth with 5 cm insulation.

Steel area required = $\frac{19250 \times 100}{1200 \times \frac{7}{8} \times 115}$ = 15.9 cm²/m strip transversely

$\frac{14000 \times 100}{1200 \times \frac{7}{8} \times 115}$ = 11.6 " longitudinary

no. of bars 25mm in diameter for one partition = $\frac{15.9 \times 3.6}{4.909}$ = 11.67 bars

$\frac{11.60 \times 4.3}{4.909}$ = 10.20 "

Shear

transverse 14850 × 1.5 = 22300 kg

longitudinal 7570 × 1.8 = 13650 "

Steel ratio = $\frac{4.909 \times 13}{360 \times 115}$ = .00155

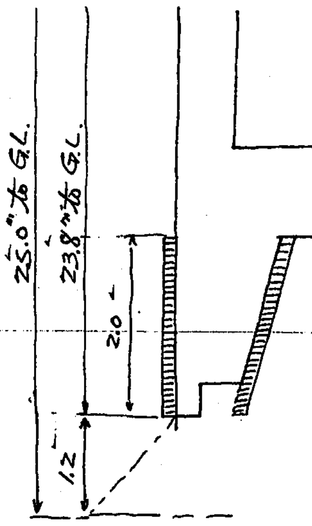
j = 0.94

Unit shear = $\frac{22300}{100 \times .94 \times 115}$ = 2.06 kg/cm² ok

Unit bond = $\frac{22300}{\frac{13}{3.6} \times 7.85 \times .94 \times 115}$ = 7.28 kg/cm² ok. Use 13 - 25φ bars in both directions.

實際ノAir pressureハ理論壓力ヨリモ遥カニ小ナルヲ普通トス 本橋ノ如ク粘土盤ノ所ニ於テハ相当減小スルモノト見ルヲ得ベシ 而シテ上記ノStressハ一時的ノモノナルヲ以テbond stressハ多少超過セルモ差支ナキモノト認ム

Cantilever side wall of working chamber.



External average pressure $\frac{1}{3} \times 1600 \times 22.8$ = 12150 kg/m²

Internal air pressure 26.5 @ 1000 = 26500

resulting internal pressure = 14350 "

moment on wall = $\frac{14350 \times 2.0^2}{2}$ = 28700 kgm per meter strip.

Effective depth required = $\sqrt{\frac{28700 \times 100}{100 \times 7.18}}$ = 63.2 cm

Use 115 cm effective depth with 5 cm insulation.

Steel area required = $\frac{28700 \times 100}{1200 \times \frac{7}{8} \times 115}$ = 23.8 cm² per m strip.

Use 25φ bars at 20 cm ctoc = 24.55

Unit shear = $\frac{28700}{100 \times .927 \times 115}$ = 2.70 kg/cm² ok.

Unit bond = $\frac{28700}{7.85 \times 5 \times .927 \times 115}$ = 6.87 " ok.

Steel ratio = $\frac{24.55}{100 \times 115}$ = .00214

j = .927

Bond stress is about 1.5% over, but above stresses are only temporaly stresses during excavation of spread base, and also the air pressure will somewhat be smaller than the theoritical one assumed in the above calculation.

For outside reinforcements use 25φ bars at 40~50 cm ctoc.

参考 省線関西線木曽川橋梁ニ於テ實施セル潜函内気壓ハ最高理論壓力 39#/□"(深90尺) ニ対シ實際使用シタル壓力ハ25#/□"内外ナリ 即チ約65%内外ノ壓力ニテ足レリ

依而本橋脚設計図面調製ニ際シ都合ニヨリテハ鐵筋量ヲ多少減ズルモ差支ナキモノト認ム

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken.

Weight of Caisson

Top 2 meters
5.4×14.0 = 75.60
4.5×13.1 = −58.90
1.03×1.03×2 = −2.12
.76×.76×2 = 1.15
15.73×2 = 31.46 @ 2400 = 75,500 kg

next 18.6 m
5.4×14.0 = 75.60
3.6×3.6×3 = −38.90
.5×.5×6 = 1.50
38.20×18.6 = 710.00 @ 2400 = 1,705,000

Chamfer 1.03×1.03×2 × 2.20 = −4.67 @ = −11,200
1,693,800
1,769,300 kg

Working chamber 5.4×14.0×1.2 = 90.70
.95×1.7×35.0 = 56.50
.30×.25×37.8 = 2.84
150.04 @ 2400 = 361,000 kg

Curb shoe say 5,200 kg
Total weight of caisson = 2,135,500 kg

Inside forms
.1×.1×8 = .080 = .080
.04×3.6×4 = .576
.05×.3×3.6×4×2.5 = .540
.05×.2×1.6×4×2.5 = .160
.05×.1×3.0×4×2.5 = .150
1.506 @ 650 = 980 × 20.6 = 20,200

Top wooden dam
.2×.2×4.2×40 = 6.72
.12×4.0×38.0 = 18.25
.2×.2×4.9×21 = 4.12
.2×.2×14×4 = 2.24
.2×.2×3.5×12 = 1.68
.075×.2×3.0×16 = .72
.20×.1×38.0×2 = 1.52
35.25 @ 650 = 22,900

Working shaft, locks &c.
Shaft 9.0×2 = 18,000
material lock say 5,500
man lock " 3,500
misc pipes, false works &c say 4,400
31,400
74,500 kg

Concrete filling on ceiling slab
3.6×3.6×.5×3 = 19.45 @ 2400 = 47,000 kg

Summary of Caisson weight.
Concrete caisson = 2,135,500
forms, top dam, air locks &c 74,500
Concrete filling in partition at bottom = 47,000
2,257,000 kg during excavation of base.

Water filling
3.6×3.6×3 = 38.90
.5×.5×6 = −1.50
37.40 @ 1000 = 37,400 kg per lin meter of caisson shell.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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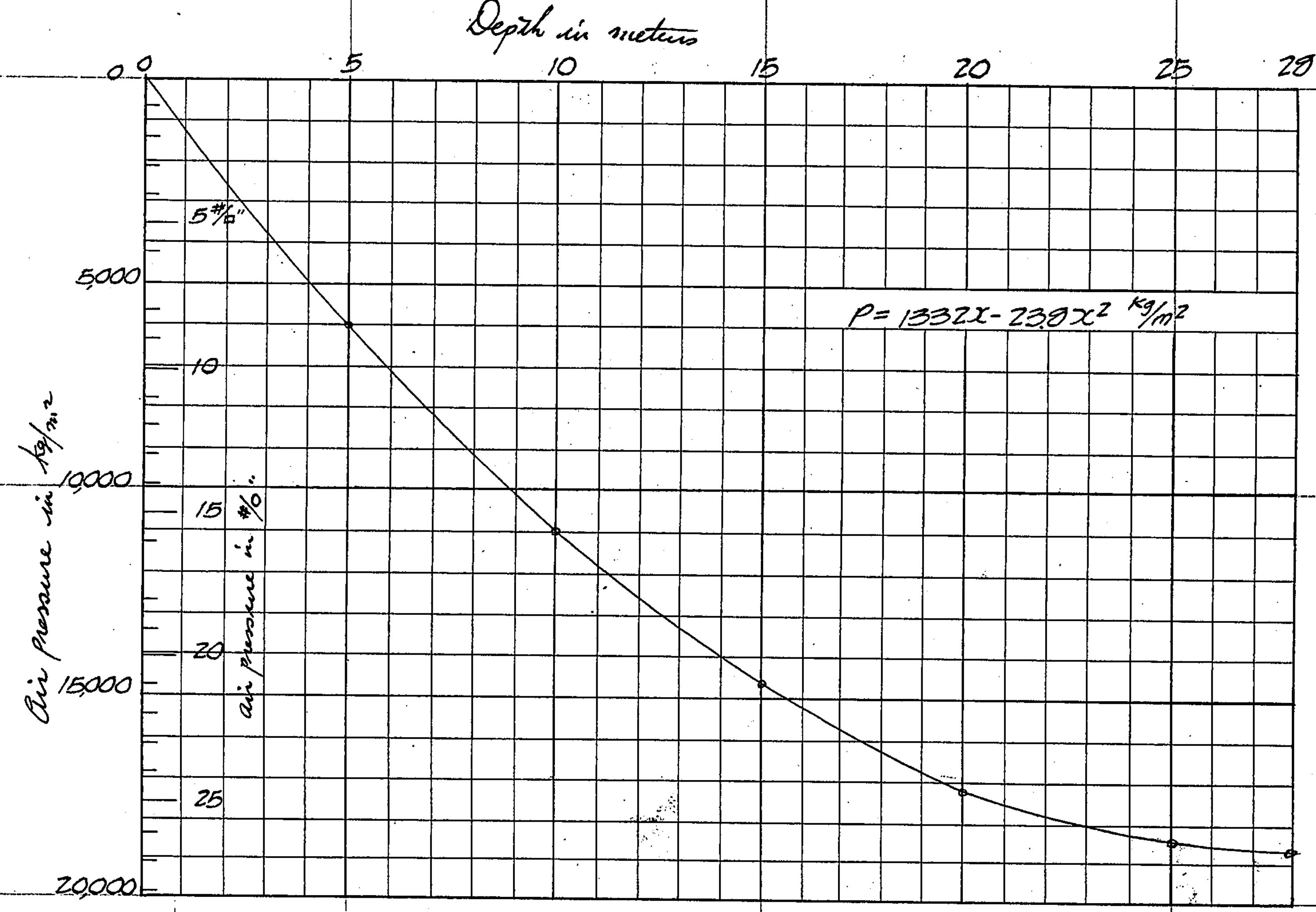
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Pressure in working chamber.

Total weight of caisson = 2257000 kg.

潜函内ニ使用スル實壓力ハ必ズシモ理論壓力ト一致セズ 参考ノ為メ関西線木曽川鐵道橋潜函工事ニ於ケル使用實壓力調書ヲ基礎トシ適當ナル圧力ヲ推定スレバ 大約 深サ90呎ニテ理論圧力 39 #/□" ノ 2/3 即チ 26 #/□" アレバ充分ナルヲ示セリ 而シテ之レヲ図示スル拋物線状ニ變化セリ之レヲ採用シテ示セバ左ノ如シ



Max. depth 25.3 m from high tide. Theoritical air pressure = 25.3 @ 1000 = 25,300 kg/m²

Assumed actual pressure in working chamber = 18430 " (73%)

Total upward pressure = 18430 × 14.0 × 5.4 = 1394,000 kg

Skin friction during sinking work Effective depth of friction assumed 21 meters

For friction of 1220 kg/m² (250 #/□') 1220 × 38.8 m × 21 = 994,000 kg.

" 1465 " (300 ") 1465 × 38.8 × 21 = 1193,000 "

Total downward pressure	friction 1220 kg/m² (250)		1465 (300)
Weight of caisson	2257000		2257000
Skin friction	- 994,000		- 1193000
	1,263,000		1,064,000
water fill 5 m @ 37400 =	187000	10 m @ 37400 =	374000
	1450,000 kg > 1394000		1438,000 kg > 1394000

沈下最後ノ 4~5m ニ対シ水ヲ 5~10m 注入スレバ足ル

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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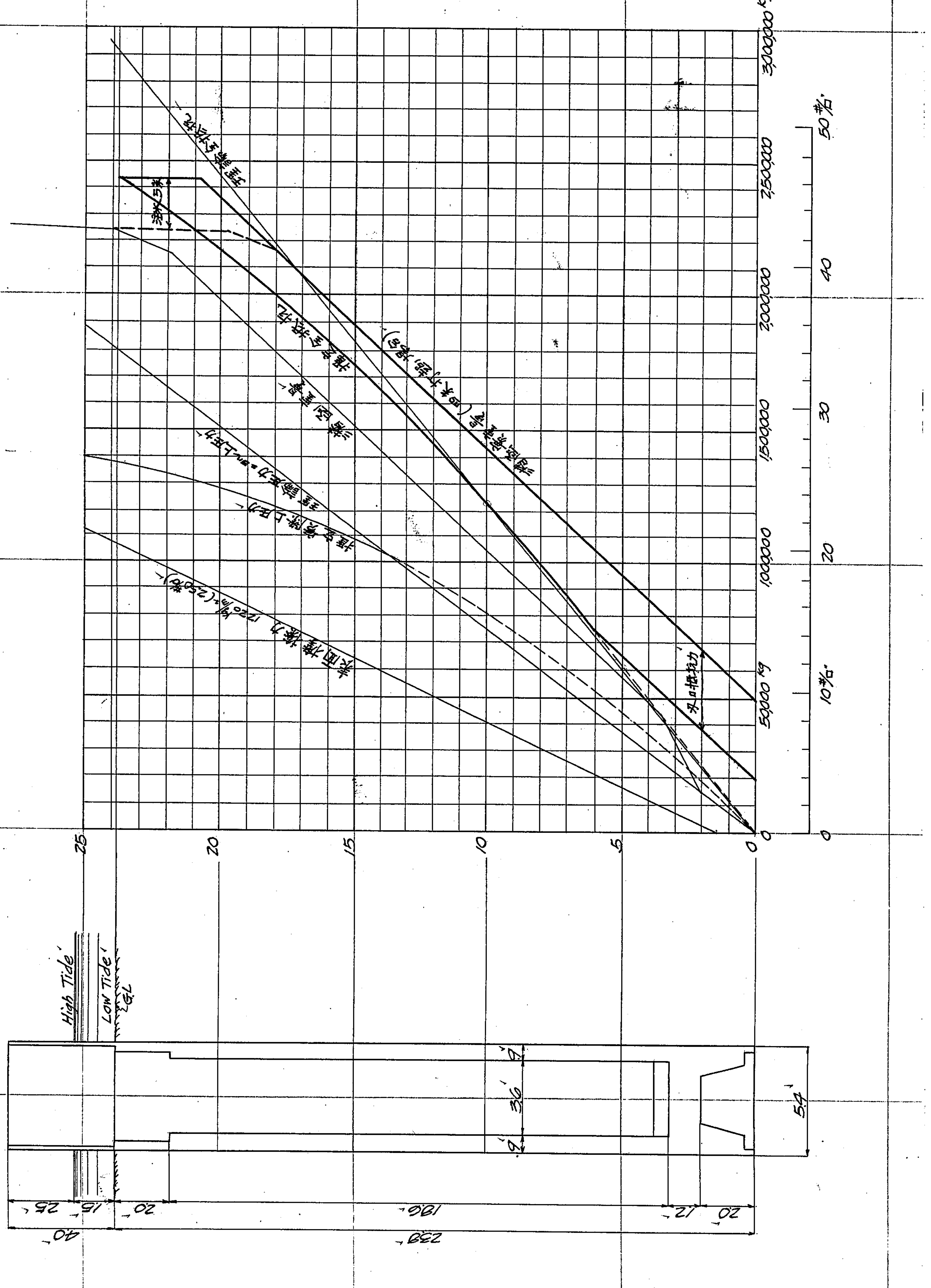
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken.
Sinking Diagram of Sand Caisson



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken.

Stability of Pier

Superimposed Loads on Pier   Dead Load D = 512,000

Live Load L = 170,000

P_0 = 682,000 kg on one pier.

weight of shaft including top fill = P_1 = 594,000 kg.   arm 0.88 m above top of caisson

weight of several sections of caisson.

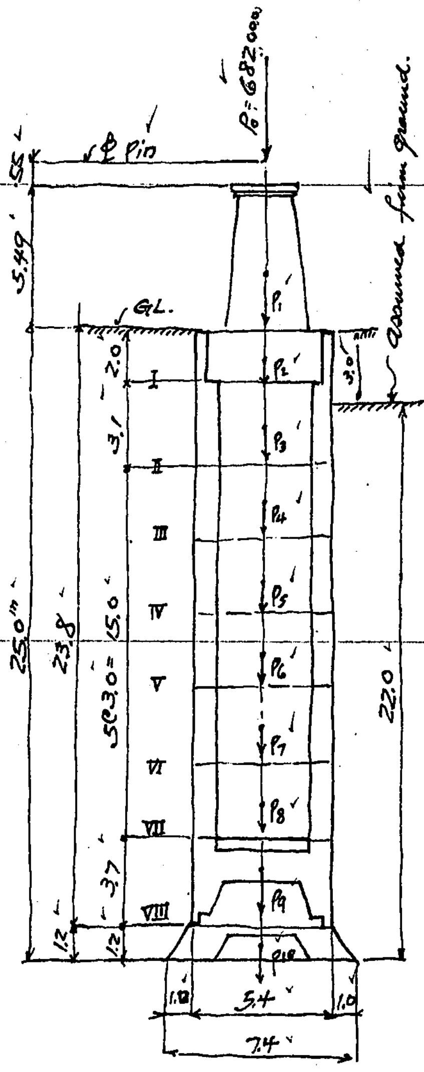
P_2 = 75,500 kg   arm 1.00 m from bottom.

P_3 = 1693800 × 3.1/18.6 = 282,000   水上木ノ重 7.2 m³ @ 350 = 2520

water 37400 × 3.1 = 115,900   コンクリート木ノ重 1.0 @ 1750 = 1750

wood frame (difference) = − 4,300   − 4270 kg

P_3 = 393,600 kg   arm 1.55   Call this 4300



P_4, P_5, P_6, P_7 + P_8.

1693800 × 3.0/18.6 = 273,300

water 37400 × 3.0 = 112,200

$P_4, P_5, P_6, P_7 + P_8$ = 385,500 kg   arm 1.50

P_9. 14.0 × 5.4 × 3.7 @ 2400 = 672,000 kg   arm 1.85

P_{10}. 14.0 × 5.4 = 75.60

16.0 × 7.4 = 118.40

194.0 ÷ 2 = 97.0

97.0 × 1.2 @ 2200 = 256,000 × .495 = 126,700

10.4 × 1.8 = 18.70

12.0 × 3.4 = 40.80

59.50 ÷ 2 = 29.8

29.8 × 1.05 = 31.4 @ 2200 = −69,500 × .528 = −36,700

31.4 @ 1600 = 50,500 × .528 = −26,700

P_{10} = 237,000 kg   116,700   arm 0.49 m

Center of gravity of pier

Loads			
P_1	594,000 ×	25.88 =	15,380,000
P_2	75,500 ×	24.00 =	1,812,000
P_3	393,600 ×	21.45 =	8,450,000
$P_4, P_5, P_6, P_7 + P_8$	1,927,500 ×	12.40 =	23,900,000
P_9	672,000 ×	3.05 =	2,050,000
P_{10}	237,000 ×	0.49 =	116,000
	3,899,600	13.10 m	51,708,000

Call this 3,900,000 kg

Seismic forces = 3,900,000 × .3 = 1,170,000 kg.

Stability of Pier at normal state.

Superimposed Dead and Live Load = 682,000

weight of pier 3,900,000

4,582,000 kg

Skin friction 1220 kg/m² (or 250#/□')

1220 × 38.8 × 20.8 = − 984,000

3,598,000

unit bearing pressure on soil = 3,598,000 / (7.4 × 16.0) = 30400 kg/m² (or 2.78 tons/□')

For skin friction of 1465 kg/m² (= 300#/□')

1465 × 38.8 × 20.8 = − 1,182,000

4,582,000

3,400,000 kg

unit bearing pressure on soil = 3,400,000 / (7.4 × 16.0) = 28700 kg/m² (or 2.63 tons/□')

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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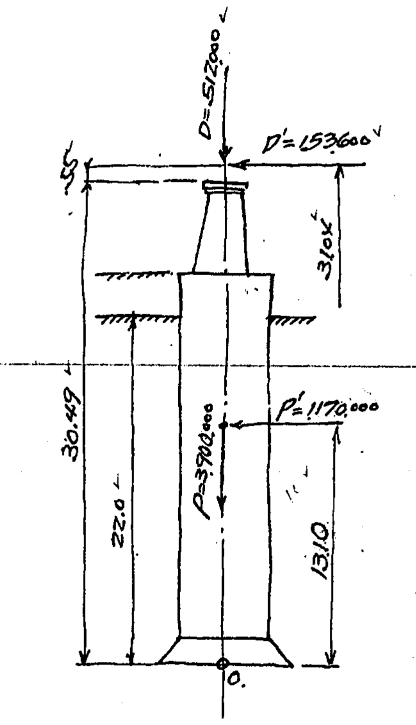
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi Ken.
Stability during Earthquake. K assumed 0.300

土木学会誌第十四巻第十四號五二九頁参照

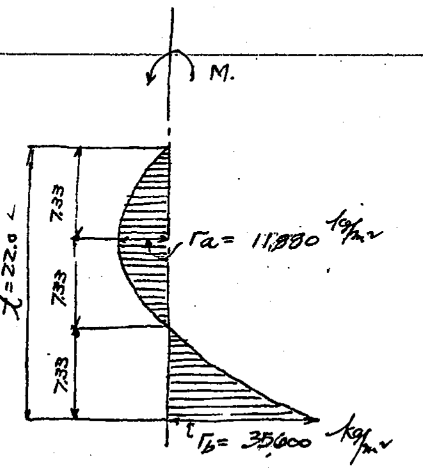
Moment due to seismic forces about center of base O.

D' H7 $153600 \times 31.04 = 4770000$
P' $1170000 \times 13.10 = 15330000$
$M = 20100000$ kgm



Sum of vertical loads.

$D = 512000$
$P = 3900000$
4412000 kg



$r_b = \frac{12M}{l^2} = \frac{12 \times 20100000}{22.0^2} = 498500$ kg

$r_a = \frac{r_b}{3} = 498500 \div 3 = 166200$ kg

for one meter strip.

$r_b = 498500 \div 14.0 = 35600$ kg/m²
$r_a = 166200 \div 14.0 = 11880$

Passive pressure of earth

$p = w \cdot x \, \frac{1+\sin\phi'}{1-\sin\phi'}$ where $\phi' = \phi - \tan^{-1} k$.

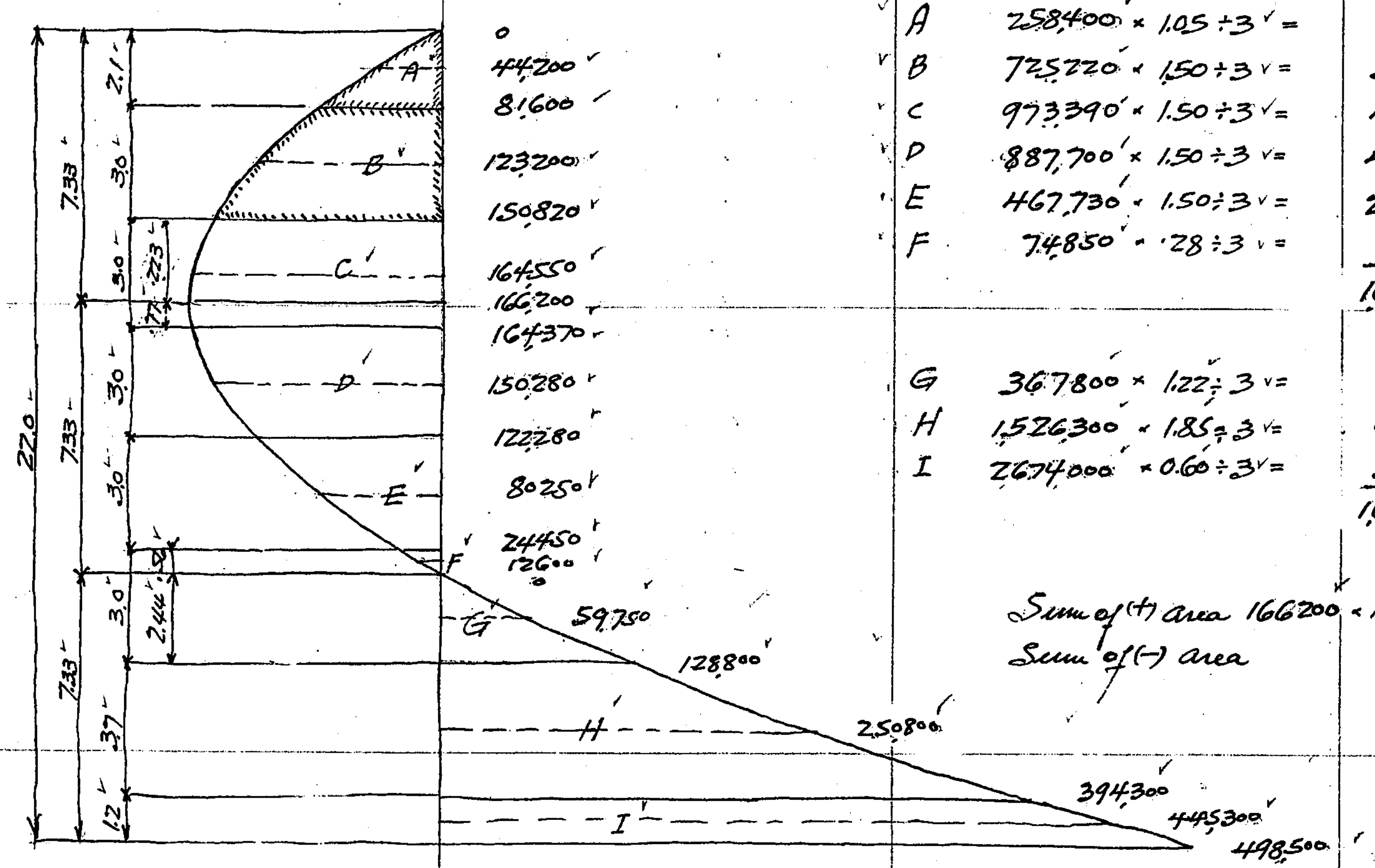
ϕ = angle of repose of earth at normal state assumed 30°

$\phi' = 30° - \tan^{-1} 0.300 = 13°\text{-}20'$ $\sin\phi' = \sin 13°\text{-}20' = 0.231$

$\frac{1+\sin\phi}{1-\sin\phi} = \frac{1.231}{.769} = 1.60$

Passive pressure at r_a $p_a = 1600 \times 7.33 \times 1.60 = 18800$ kg/m² > 11880 ok
" " " r_b $p_b = 1600 \times 7.33 \times 1.60 \times 3 = 56300$ ~~37550~~ > 35600 ok

Unit bearing pressure on soil = $\frac{4412000}{16.0 \times 7.4} = 37250$ kg/m² or (3.47 tons/□')



Pressure on each section by Sympson's formula.

			arm from bottom
A	$258400 \times 1.05 \div 3 =$	90400 Kg	0.72 m
B	$725220 \times 1.50 \div 3 =$	362600	1.38
C	$973390 \times 1.50 \div 3 =$	486700	1.48
D	$887700 \times 1.50 \div 3 =$	443900	1.60
E	$467730 \times 1.50 \div 3 =$	233900	1.67
F	$74850 \times .28 \div 3 =$	7000	.37
		1624500 Kg	
G	$367800 \times 1.22 \div 3 =$	149500	.80
H	$1526300 \times 1.85 \div 3 =$	940600	1.50
I	$2674000 \times 0.60 \div 3 =$	534800	.55
		1624900 Kg	

Sum of (+) area $166200 \times 14.66 \times \frac{2}{3} = 1624500$ kg
Sum of (−) area $= -1624500$ kg

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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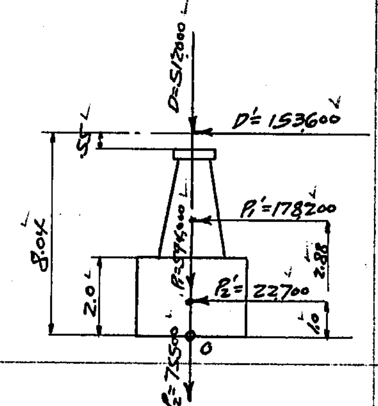
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken.

moments at several sections during earthquake.

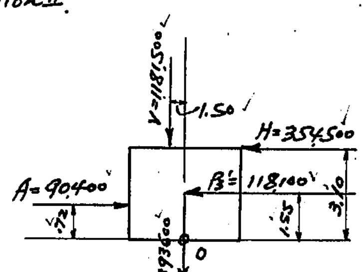
Section I

Taking moments about point O.



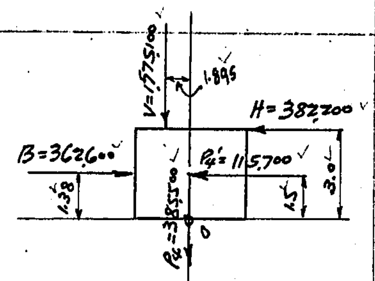
Loads	Hor. forces	Vert. forces	Lev. arms	Moments about O.
D		512000	.0	0
D'	153600		8.04	1235000
P_1		594000	0	0
P_1'	178200		2.88	513000
P_2		75500	0	0
P_2'	22700		1.00	22700
	H=354500 kg	V=1181500 kg	1.50 m	1770700. kgm

Section II



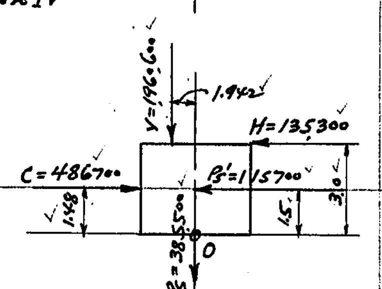
V		1181500	1.50	1770700
H	354500		3.10	1099000
P_3		393600	0	0
P_3'	118100		1.55	183100
A	− 90400		.72	65100
	382200 kg	1575100 kg	1.895 m	2987700 kgm

Section III



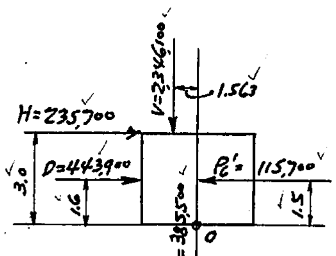
V		1575100	1.895	2987700
H	382200		3.00	1147000
P_4		385500	0	0
P_4'	115700		1.50	173500
B	362600		1.38	− 500300
	135300 kg	1960600 kg	1.942 m	3807900 kgm

Section IV



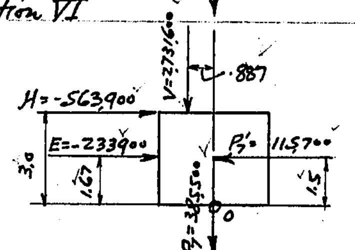
V		1960600	1.942	3807900
H	135300		3.00	406000
P_5		385500	0.00	0
P_5'	115700		1.50	173500
C	− 486700		1.48	− 720000
	− 235700 kg	2346100 kg	1.563 m	3667400 kgm

Section V.



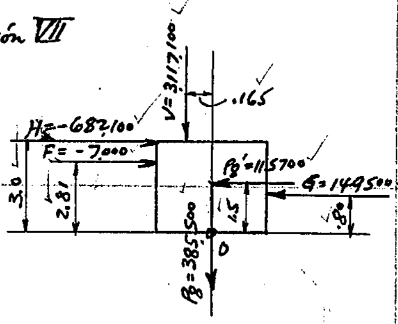
V		2346100	1.563	3667400
H	− 235700		3.00	− 707000
P_6		385500	0	0
P_6'	115700		1.50	173500
D	− 443900		1.60	− 710000
	− 563900 kg	2731600 kg	0.887 m	2423900 kgm

Section VI



V		2731600	0.887	2423900
H	− 563900		3.00	− 1692000
P_7		385500	0	0
P_7'	115700		1.50	173500
E	− 233900		1.67	− 390500
	− 682100 kg	3117100 kg	0.165 m	514900 kgm

Section VII



V		3117100	0.165	514900
H	− 682100		3.00	− 2047000
P_8		385500	0	0
P_8'	115700		1.50	173500
F	− 7000		2.81	− 19700
G	+ 149500		0.80	119500
	423900 kg	3502600 kg	− 0.359 m	− 1258800 kgm

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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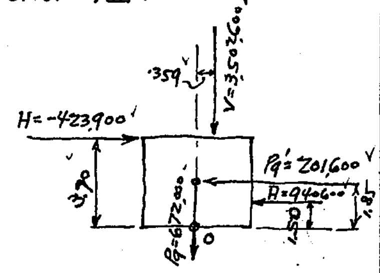
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken

Section VIII.



Loads	Hor. forces	Vert. forces	Lev. arms	moments abt. O
V		3502600	× −.359 =	− 1258800
H	− 423900		× 3.70 =	− 1570000
Pq		672000	× 0 =	0
Pq'	201600		× 1.85 =	373000
H̄	940600		× 1.50 =	1410000
	718300 kg	4174600 kg	−.253	− 1045800 kgm

moment at bottom assumed 0. in the first assumption of this calculation.

Vertical Reinforcements of Caisson

max. moment 3807900 kgm call this 3810000 at section III. (8.1 m below top of caisson).
vertical load 1960600 kg " 1960000 "
less weight of water $37400 \times 6.1 =$ − 228000
1732000 kg on shell only.

Sectional area of caisson 38.2 m² (see page 10)
Direct compression on concrete = $1732000 \div 38.2 \div 10000 = 4.54$ kg/cm²
Bending stress $= \frac{3810000}{4.5} = 847000$ kg

$847000 \div 14.0 = 60500$ kg per lin m of side wall T or C
Direct compression $4.54 \times 90 \times 100 = 40850$ " " C
101350 kg " C on concrete
or 19650 kg " T on steel.

Unit compression $f_c = \frac{101350}{90 \times 100} = 11.3$ kg/cm² ok.

Steel area required for tension $= \frac{19650}{1200 \times 1.6} = 10.24$ cm² per m of side wall.

use 25 mmϕ bars at 50 cm c to c on both sides $= 4.909 \times 4 = 19.65$ cm² .. ok.

If the lower 6.7 m of caisson be assumed to be suspended during excavation, steel area req'd is as follows:
3 m shell with water filling = 385500
bottom 3.7 m 672000
less conc. fill $3.6 \times 12.2 \times 2.0 @ 2200 =$ 193000
479000
864500 kg total weight of suspended portion

Steel area required $= \frac{864500}{1200} = 720$ cm²

25ϕ bars, 35.2 m c 4 = 141 bars @ 4.909 = 692.0
19ϕ bars partition wall 28 " @ 2.835 = 79.4
771.4 cm² ok.

Use same vertical reinforcements for shell throughout its length.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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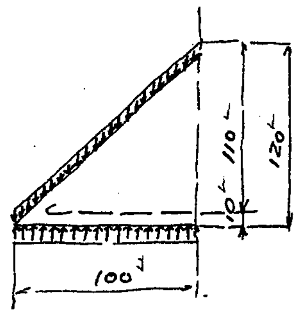
CALCULATIONS FOR

Design of Kisogawa Bashi for Aichi Ken

Reinforcements for spread base.



max. upward pressure $= 30400$ kg/m²
Earth filling assumed $15 @ 1600$ $= \underline{24000}$ (including wt. of footing)
6400 ..

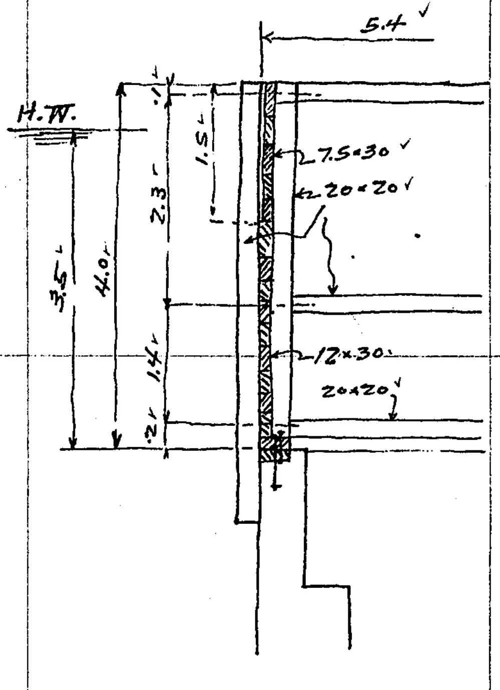
moment on footing $= 6400 \times 0.5^2 = 3200$ kgm per meter strip.
shear " " $= 6400$ kg ..
Effective depth required $= \sqrt{\frac{3200 \times 100}{100 \times 7.18}} = 21.1$ cm

use effective depth of 110 cm with 10 cm insulation.
~~Steel area required $= \frac{3200 \times 100}{1200 \times \frac{7}{8} \times 110} = 2.77$ cm²~~
use 22 mmφ bars at 60 cm ctoc $= 6.34$ cm²

Unit shear $= \frac{6400}{100 \times \frac{7}{8} \times 110} = 0.67$ kg/cm² ok.

unit bond $= \frac{6400}{\frac{6.91}{.6} \times \frac{7}{8} \times 110} = 5.77$ " ok.

Wooden coffer dam on top of Caisson.



$5.4^m \times 14.0$ — 4.0^m deep.
water depth during an ordinary flood water assumed 3.5^m
water pressure at bottom $3.5 @ 1000 = 3500$ kg/m²
planking, span length assumed 2.15 m
moment $= \frac{3500 \times 2.15^2}{10} = 1,616$ kgm per meter strip

Section modulus required $= \frac{1616 \times 100}{80} = 2020$ cm³

$d = \sqrt{\frac{2020 \times 6}{100}} = 11.52$ cm

use 12 cm planking.

At section 1.5^m below ~~top~~ H.W. water pressure $= 1500$ kg/m²
moment $= \frac{1500 \times 2.15^2}{10} = 693$ kgm per meter strip.

S.m. required $= \frac{693 \times 100}{80} = 866$ cm³

$d = \sqrt{\frac{866 \times 6}{100}} = 7.21$ cm
use 7.5 cm planking for top 1.5 meters.

Vertical Columns. Span length at bottom $= 1.40$ m, spacing 2.15^m. 2-columns at each panel pt.

pressure $= 2.6 \times 1000 = 2600$ kg/m² average
moment $= \frac{2600 \times 1.4^2}{10} \times 2.15 = 1095$ kgm

S.m. required $= \frac{1095 \times 100}{80} = 1370$ cm³

use 2-columns 20×20
S.m. $= \frac{20 \times 20^2}{6} \times 2 = 2,670$ cm³ ok.

Shear $= 2600 \times 2.15 \times .7 = 3930$ kg
unit shear $= \frac{3930}{20 \times 20 \times 2} = 4.9$ kg/cm² ok.

Load on strut approximately
$1900 \times 1.9 \times 2.15 = 7760$ kg
unit compression $= \frac{7760}{20 \times 20} = 19.4$ kg/cm²

Allowable unit compression
$= 50\left(1 - \frac{l}{60d}\right) = 29.1$ kg/cm² ok.

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JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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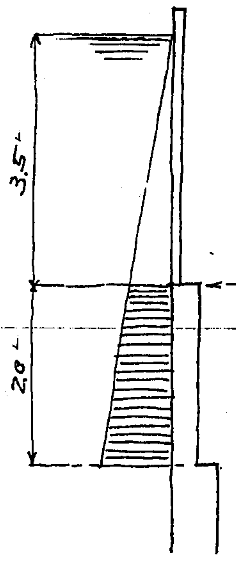
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken
Vertical Reinforcements in top wall of caisson for water pressure during execution.



Height of wall 2.0 m, thickness 45 cm, effective depth 40 cm
water pressure on wall 3.5 @ 1000 = 3500 kg/cm² on top
5.5 @ 1000 = 5500 " " bottom.
9000 ÷ 2 = 4500 average.

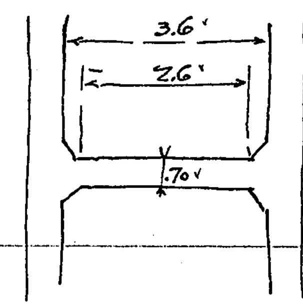
moment $= \frac{1}{10} \times 4500 \times 2.0^2 = 1800$ kgm per meter strip.
Effective depth required $= \sqrt{\frac{1800 \times 100}{100 \times 7.18}} = 15.9$ cm
use 40 cm effective depth with 5 cm insulation
Steel area required $= \frac{1800 \times 100}{1200 \times \frac{7}{8} \times 40} = 4.29$ cm²
use 25 mm φ bars at 50 cm c to c = 9.82 cm²
Shear = 4500 kg
unit shear $= \frac{4500}{100 \times \frac{7}{8} \times 40} = 1.29$ kg/cm² ok.

Steel ratio $p = \frac{9.82}{100 \times 40} = 0.00245$
$j = 0.922$

unit bond $= \frac{4500}{7.85 \times 2 \times .922 \times 40} = 7.77$ kg/cm²

This stress is only temporaly and if we allow 30% over stress allowable bond = 6.0 × 1.3 = 7.8 ok.

Design of partition wall. spacing 4.3 meters.



Outside earth pressure $= 20 \times 1600 \times \frac{1}{3} = 10670$ kg/m²
load on wall = 10670 × 4.3 = 45900 kg
Direct compression $= \frac{45900}{70 \times 100} = 6.6$ kg/cm² c

unbalance of water fill assumed 10 m max.
unbalance pressure = 10 @ 1000 = 10000 kg/m²
moment $= \frac{1000 \times 2.6^2}{12} = 5640$ kgm.

Steel area required $= \frac{5640 \times 100}{1200 \times \frac{7}{8} \times 65} = 8.28$ cm²

use 19φ bars at 25 cm c to c = 11.34 cm²
steel ratio $p = \frac{11.34}{100 \times 65} = 0.00175$

$j = .933$, $k = .202$

$f_s = \frac{5640 \times 100}{11.34 \times .933 \times 65} = 820$ kg/cm² ok

$f_c = \frac{820 \times .202}{15(1 - .202)} = 13.8$ "
Direct compression = 6.6
20.4 kg/cm² c ok.

Assumed section is ample.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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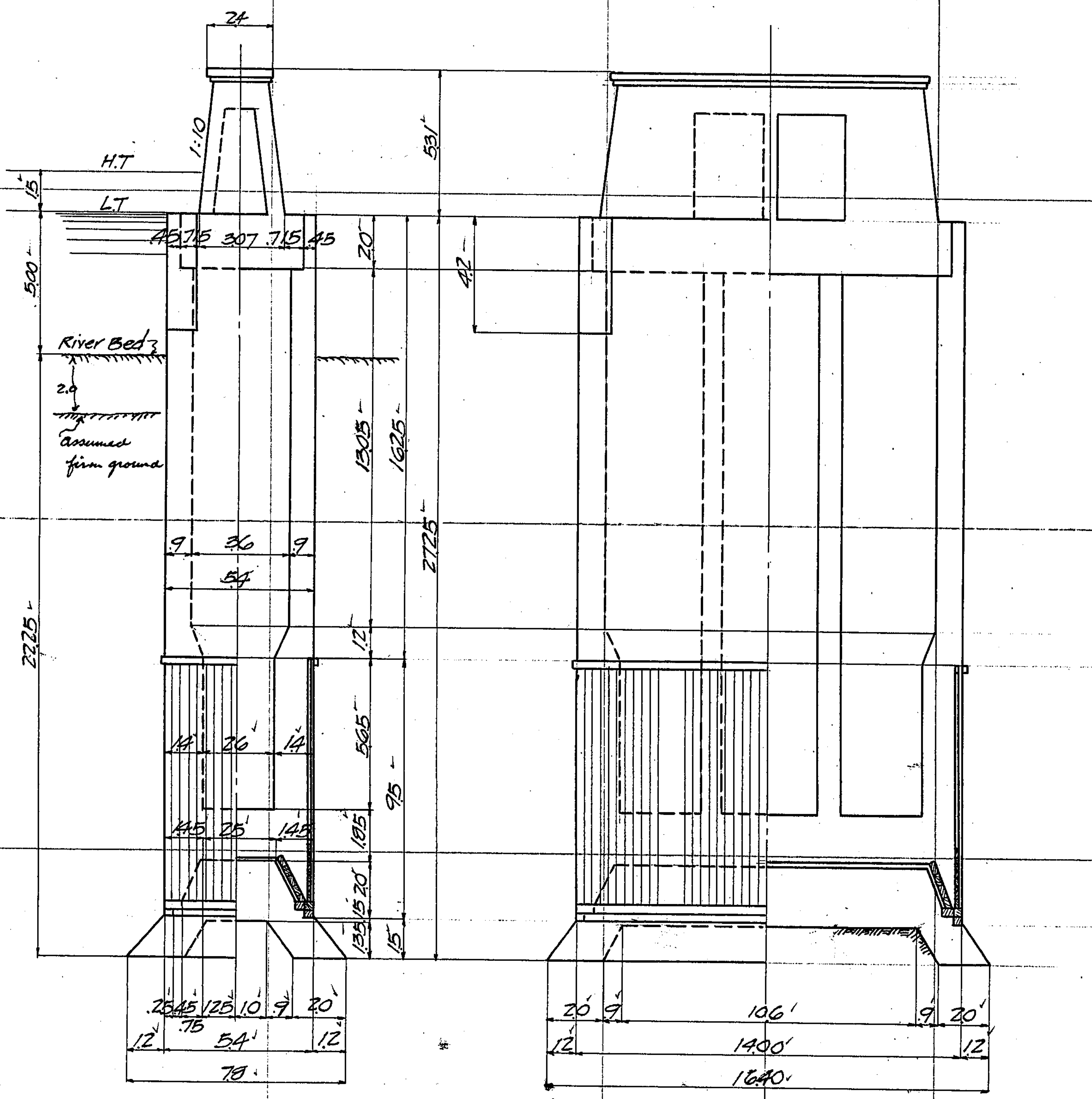
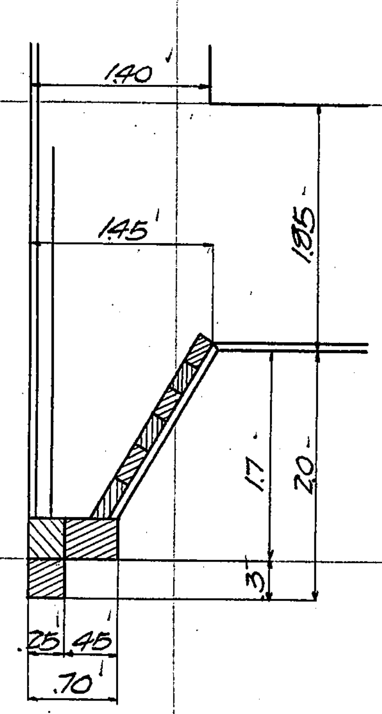
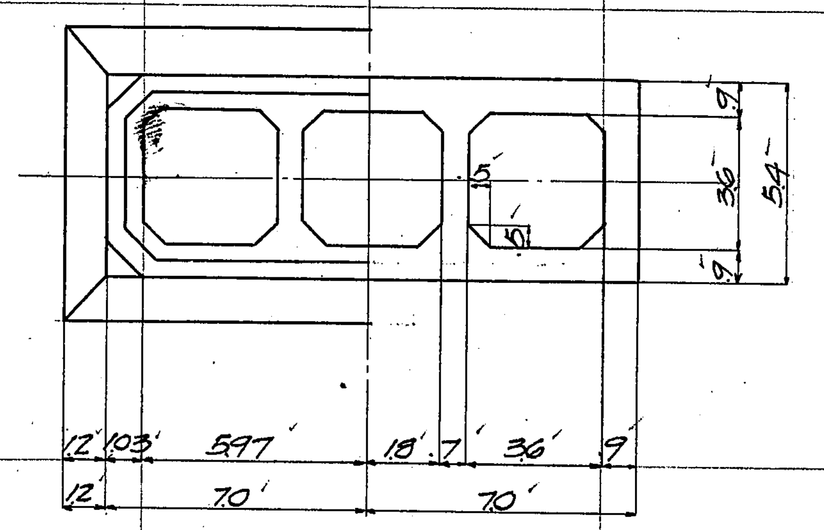
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken
River Caisson. Reinforced Concrete with timber work.
Pier P11. 5.4 × 14.0 × 27.25 m Caisson.





Cutting Edge.
Scale 1:60.



General Sketch of River Caisson (Pier P11)
Scale 1:200.

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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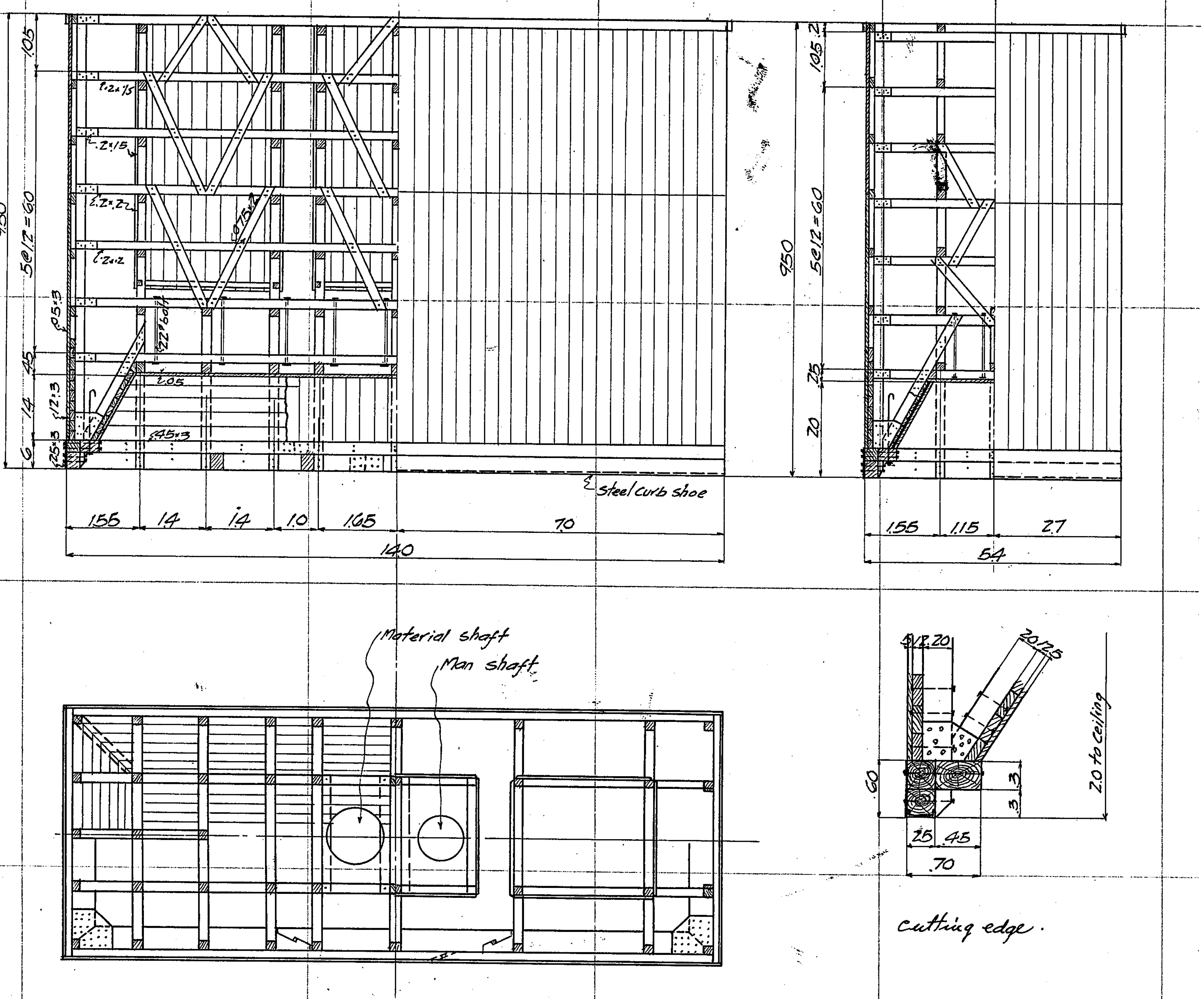
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.
Design of Timber Floating caisson 5.4 × 14.0 × 9.5 m
General dimensions and constructions are as shown on sketch below.



cutting edge.

General sketch of Floating Caisson.
Scale 1: 100.

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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-Ken.

Weight of Floating Caisson. 5.4 × 14.0 × 9.5 meter timber caisson.

Timbers say	103.3 m^3 @ 650 =	67,200
Bolts, plates, nails &c say		5,200
Curb shoe, say		2,600
Working shaft	2-10 m long say	2,000
		77,000 kg

Center of gravity 3.10 m from bottom about

Weight of water in working chamber

top area 2.50 × 11.1 = 27.75

bottom " 4.00 × 12.6 = 50.40

78.15 ÷ 2 = 39.08 × 1.4 = 54.70

lower part 50.40 × 0.3 = 15.12

Cutting edge ½ 4.90 × 13.50 × 0.30 = 19.88

89.70

working shaft ½ 1.22φ × 0.20 say = 0.20

.91φ × 0.20 = 0.13

0.33

90.03 m^3 @ 1000 = 90,030 kg call this 90,000 kg

Total weight of Timber caisson with water in working chamber = 90,000 + 77,000 = 167,000 kg.

Volume of caisson for 1 meter strip = 5.40 × 14.0 = 75.60 m^3 @ 1000 = 75,600 kg for water.

Draft of Caisson

Case 1. Draft for Empty caisson.

$$\text{Draft} = \frac{167000}{75600} = 2.21 \text{ meters}$$

Case 2. Draft for side wall of working chamber casted and all reinforcements arranged.

Concrete approx. volume 7.5 × 1.4 × 3.40 = 37.5 m^3 @ 2200 = 78,600

reinforcements for floating caisson say 26,400

105,000 kg

wt for case 1. 167,000

Total weight. 272,000 kg

$$\text{Draft} = \frac{272000}{75600} = 3.60 \text{ meters}$$

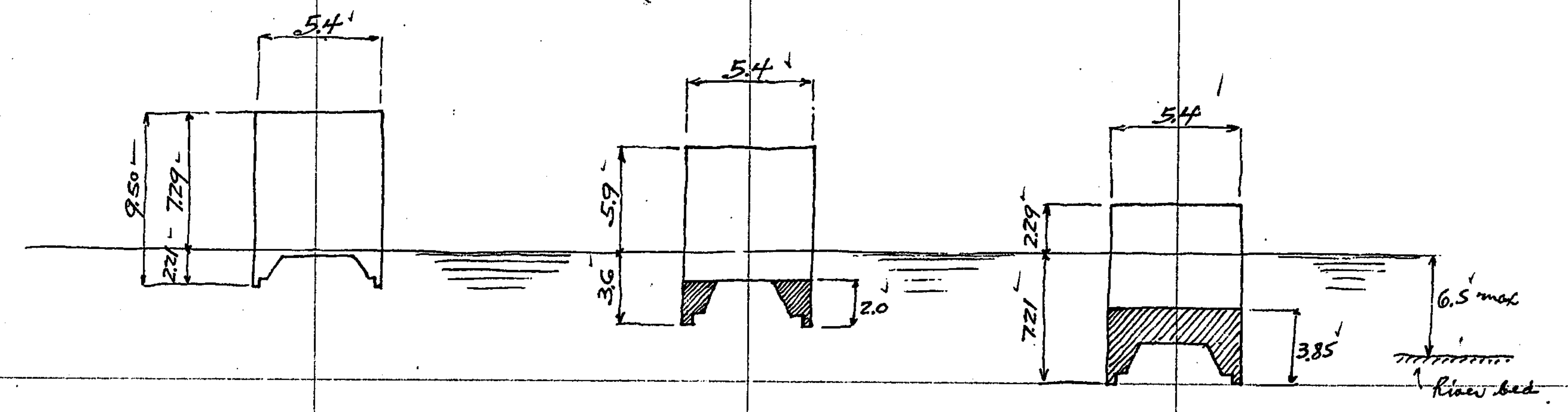
Case 3. Ceiling slab executed.

Ceiling slab concrete 5.05 × 13.65 × 1.80 = 124.0 m^3 @ 2200 = 273,000

272,000

545,000 kg

$$\text{Draft} = \frac{545000}{75600} = 7.21 \text{ meters}$$



Case 1.
Empty.

Case 2.
Side wall concrete casted + reinforcements arranged.

Case 3
Ceiling slab concrete executed also.

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CONSULTIEG ENGINEER
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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Planking of ceiling for working chamber.
in case 2. Draft of caisson = 3.60 water pressure on planking = 1.6 × 1000 = 1600 kg/m²
Span length 1.40 meters (Center 1.65m panel will be shortened by temporaly beams)
moment on planking = $\frac{1600 \times 1.40^2}{10}$ = 313 kgm

Allowable strength of american pine for bending stresses assumed 75 kg/cm² (1050#/□" about)
Section modulus required = $\frac{313 \times 100}{75}$ = 417 cm³
for b = 100 cm
thickness of planking $d = \sqrt{\frac{417 \times 6}{100}} = 5.00$ cm

Ceiling slab executed. min draft of caisson at L.W. assumed 5.0m 5.0 − 2.0 = 3.0m
upward water pressure = 3.00 × 1000 = 3000
downward pressure, concrete = 1.8 × 2400 = −4320
−1320 kg/m² downward pressure ok.

Side wall planking
Case 1. Draft 2.21m, water pressure 2.21 − .6 = 1.61 × 1000 = 1610 kg/m²

Case 2. Draft 3.60 " " 3.60 − .6 = 3.00 × 1000 = 3000
concrete pressure 1.40 × 2200 = −3080
− 80 kg/m²

Case 3. Draft 7.21
max water depth = 6.50m
concrete height = 3.85
2.65 × 1000 = 2650 kg/m²

Horizontal planking of outside wall. span length 1.65m max.
moment = $\frac{1610 \times 1.65^2}{10}$ = 438 kgm

Section modulus required = $\frac{438 \times 100}{75}$ = 585 cm³

Use 12cm planking S.m. = $\frac{100 \times 12^2}{6}$ = 2400 cm³ ok

Vertical planking of outside wall span length = 1.2m
moment = $\frac{2650 \times 1.2^2}{10}$ = 381 kgm S.m. required = $\frac{381 \times 100}{75}$ = 508 cm³

for b = 100, $d = \sqrt{\frac{508 \times 6}{100}} = 5.52$ cm
This stress is only temporaly, use 5 cm planking.

Horizontal planking for inside wall of working chamber
use 12cm planking horizontal with a vertical 5cm sheathing.

Cross Beams. span length = 2.3 meters spacing 1.4m (center panel being shortened by temporaly extra cross beams)

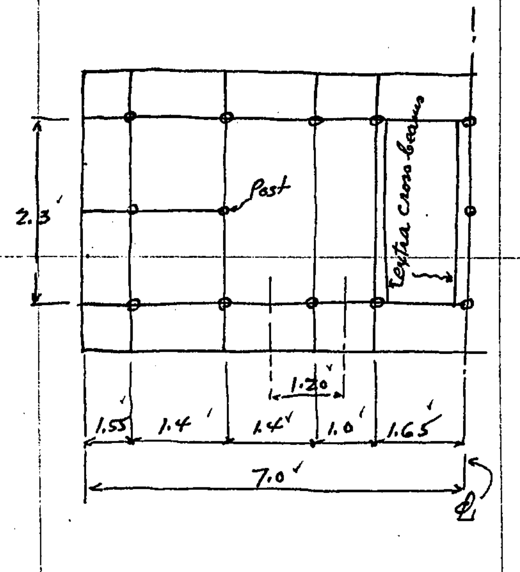
max pressure 1600 kg/m²
load on beam = 1600 × 1.2 = 1920
wt. of beam + planking say 60
1860 kg per lin m.

moment = $\frac{1860 \times 2.3^2}{10}$ = 984 kgm

S.m. required = $\frac{984 \times 100}{75}$ = 1312 cm³

width of beam 20 cm $d = \sqrt{\frac{1312 \times 6}{20}} = 19.83$ cm

Use 20 × 20 cm cross beams.



Design of Kiso-gawa Bashi for Aichi-ken.

Design of Shell for Caisson

For upper 16.25 meters of caisson, use same details as for Land Caisson.

Side wall of Caisson.

Depth of earth = 16.90 m
Surcharge of water $6.5 \times \frac{1000}{1600}$ = 4.06
20.96 m at bottom of side wall.

Average pressure for the lowest 1 m strip = $20.46 \times \frac{1600}{3}$ = 10900 kg/m²

moment on wall = $\frac{10900 \times 4.3^2}{12}$ = 16800 kgm. per meter strip of wall, effective depth = 98 cm

Steel area required = $\frac{16800 \times 100}{1200 \times \frac{7}{8} \times 98}$ = 16.3 cm² per meter strip.

Use 25 mm φ bars at 30 cm c to c = 16.35 cm²

Steel ratio $p = \frac{16.35}{100 \times 98}$ = .0017, $j = 0.935$

Shear on bottom 2 m assumed to be transmitted both to horizontal + vertical directions

max. shear on wall = $18.46 \times 5.33 \times 1.8$ = 17700 kg

If 20% of above shear be assumed to be taken care of by wood-struts at center

Shear on concrete wall = 17700×0.8 = 14150 kg

unit shear = $\frac{14150}{100 \times .935 \times 98}$ = 1.54 kg/cm² ok

unit bond = $\frac{14150}{7.85 \times 3.33 \times .935 \times 98}$ = 5.90 " ok.

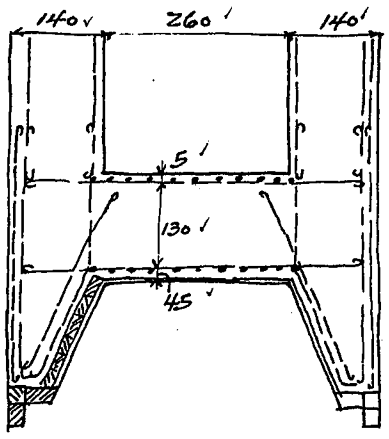
For end partition Shear = $18.46 \times 5.33 \times 1.55$ = 15250 kg

unit shear = $\frac{15250}{100 \times .935 \times 98}$ = 1.67 kg/cm² ok.

unit bond = $\frac{15250}{7.85 \times 3.33 \times .935 \times 98}$ = 6.35 " this will somewhat be reduced by timber frame ok.

Design of working chamber.

Height of caisson = 27.25
max tidal change say = 1.50
28.75 m



Theoritical max. air pressure during H.T. = 28.75×1000 = 28750
weight of concrete slab = 1.8×2400 = −4320
planking to − 30
24400 kg/m²

transverse span length 3.2 m = l_1, longitudinal span length = 4.3 m = l_2 assumed

Load on transverse span = $24400 \times (1.5 - \frac{3.2}{4.3})$ = 18450 kg
" " longitudinal " = $24400 \times (\frac{3.2}{4.3} - 0.5)$ = 5950 kg

transverse moment = $\frac{1}{10} \times 18450 \times 3.2^2$ = 18900 kgm per m. strip.
" shear = 18450×1.3 = 23970 kg "

longitudinal moment = $\frac{1}{10} \times 5950 \times 4.3^2$ = 11,000 kgm. "
shear = 5950×1.8 = 10,700 kg

Steel area required for transverse span
= $\frac{18900 \times 100}{1200 \times \frac{7}{8} \times 135}$ = 13.35 cm² per m. strip

Use 25φ bars at 25 cm c to c = 19.65 cm² "

steel ratio $p = \frac{19.65}{135 \times 100}$ = .00145, $j = 0.944$

unit shear = $\frac{23970}{100 \times .944 \times 135}$ = 1.88 kg/cm² ok

unit bond = $\frac{23970}{7.85 \times 40 \times .944 \times 135}$ = 5.98 " ok.

Use same bars on top + bottom

Steel area required for longitudinal span
= $\frac{11000 \times 100}{1200 \times \frac{7}{8} \times 135}$ = 7.76 cm² per m. strip.

Use 22φ bars at 37.5 cm c to c = 10.15 cm² .

steel ratio $p = \frac{10.15}{135 \times 100}$ = .00075, $j = 0.95$

unit shear = $\frac{10700}{100 \times .95 \times 135}$ = 0.83 kg/cm² ok

unit bond = $\frac{10700}{\frac{6.91}{.375} \times .95 \times 135}$ = 4.53 " ok.

use same bars on top and bottom

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CONSULTIEG ENGINEER
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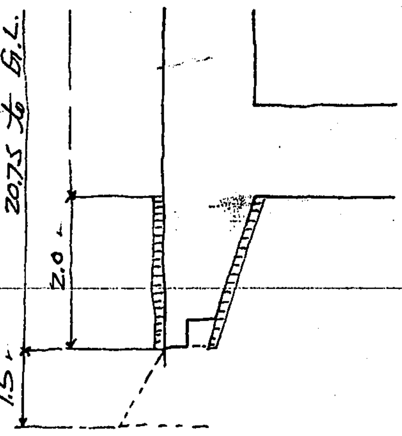
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken

Cantilever Side wall of working chamber.



Depth of earth below river bed = 20.75 m

Surcharge of water $6.5 \times \frac{1000}{1600}$ = 4.05

24.80 m at bottom of cutting edge.

External earth pressure

at top 22.8 @ 533 = 12,150

at bottom 24.8 @ 533 = 13,210

25,360 ÷ 2 = 12,680 kg/m² average.

Internal theoritical air pressure 28.75 @ 1000 = 28,750

16,070 " "

moment on cantilever side wall

$= \frac{1}{2} \times 16070 \times 2.0^2 =$ 32,140 kgm per m strip

Shear = 16070 × 2.0 = 32,140 kg " "

assume 1/3 of above moment and shear are taken care of by timber framing.

Resulting moment on conc. = $\frac{2}{3} \times 32140 =$ 21,400 kgm

" " shear " " = " = 21,400 kg

Effective depth required $= \sqrt{\frac{21400 \times 100}{100 \times 7.18}} =$ 54.6 cm

use effective depth of 118 cm with 5 cm insulation

Steel area required = $\frac{21400 \times 100}{1200 \times \frac{7}{8} \times 118}$ = 17.27 cm² per m. strip.

Use 5 - 25φ bars = 24.55 cm²

unit shear = $\frac{21400}{100 \times \frac{7}{8} \times 118}$ = 2.07 kg/cm² ok

unit bond = $\frac{21400}{7.85 \times 5 \times \frac{7}{8} \times 118}$ = 5.28 " ok.

Weight of Caisson see page. 10

Top 2 meters same as for land caisson = 75,500 kg

next 13.05 m 38.2 × 13.05 = 498.5 @ 2400 = 1,197,000

chamfer less - 11,200

1,185,800 kg

next 1.2 m top area 38.20

bottom 5.4 × 14 = 75.60

2.6 × 3.6 = - 9.36

2.6 × 3.1 × 2 = -16.12

50.12

88.32 ÷ 2 = 44.16 × 1.2 @ 2400 = 127,200 kg

next 5.65 m, (in floating caisson)

5.3 × 13.9 = 73.70

2.6 × 3.6 = - 9.36

2.6 × 3.1 × 2 = -16.12

48.22 × 5.65 = 272.55

timber .2 × .2 × 70 × 1.4 = 3.92

.12 × .2 × 5 × 38 = 4.56

.2 × .2 × 18 × 5.65 = 4.07

- 12.55

260.0 @ 2400 = 624,000 kg

Working chamber 5.3 × 13.9 × 1.8 = 132.5

.73 × 1.40 × 37.0 = 37.8

170.30 m³

timber .2 × .2 × 5.3 × 18 = 3.82

.12 × .85 × 38.5 = 3.92 } -12.68

.2 × .2 × 6 × 13.9 = 2.78 } - 3.24 --- holes for shafts

.2 × .2 × 1.8 × 30 = 2.16

154.38 @ 2400 = 370,000 kg

total wt = 2,382,500 kg for concrete.

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CONSULTIEG ENGINEER
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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Inside forms 16.25 @ 980 = 15,930
Top wooden dam = 22900 see page 10
Working shaft, locks, pipes to say = 31400
70,230 call this 70,200 kg

Summary for weight of caisson
Concrete caisson 2,382,500 page 24
Floating caisson 77,000 page 21
Forms, dams, air locks &c 70,200
2,529,700 kg During excavation of base.

Water filling for upper part 37,400 kg per lin m.
for lower part 25.48 @ 1000 = 25,500 " "

Max. depth of Cutting edge below high tide = 27.25 meters.
Theoritical air pressure = 27.25 @ 1000 = 27,250 kg/m²
Assumed actual pressure in chamber =
= 1332 × 27.25 − 23.8 × 27.25² = 18,620 kg/m² (68 %) see page 11. diagram
Total upward pressure = 18620 × 14.0 × 5.40 = 1408,000 kg

Skin friction during sinking work Effective depth of friction assumed 18.8 m
for 1220 kg/m² (250%) friction 1220 × 38.8 × 18.8 = 890,000 kg
" 1465 " (300 ") " 1465 × 38.8 × 18.8 = 1,068,000 kg

Total downward pressure	friction 1220 kg/m²	1465 kg/m²
weight of caisson	2,529,700	2,529,700
skin friction	890,000	1068,000
	1,639,700	1,461,700
upward pressure	1408,000	− 1408,000
	231,700 kg	53,700 kg

Bearing area of cutting edge 36.0 × .7 = 25.2 m²
Bearing pressure on cutting edge = $\frac{231700}{25.2}$ = 9200 kg/m² (0.84 kg/cm²) for 1220 kg/m² friction
" " " = $\frac{53700}{25.2}$ = 2130 " for 1465 " "

No water filling will be required for sinking work.

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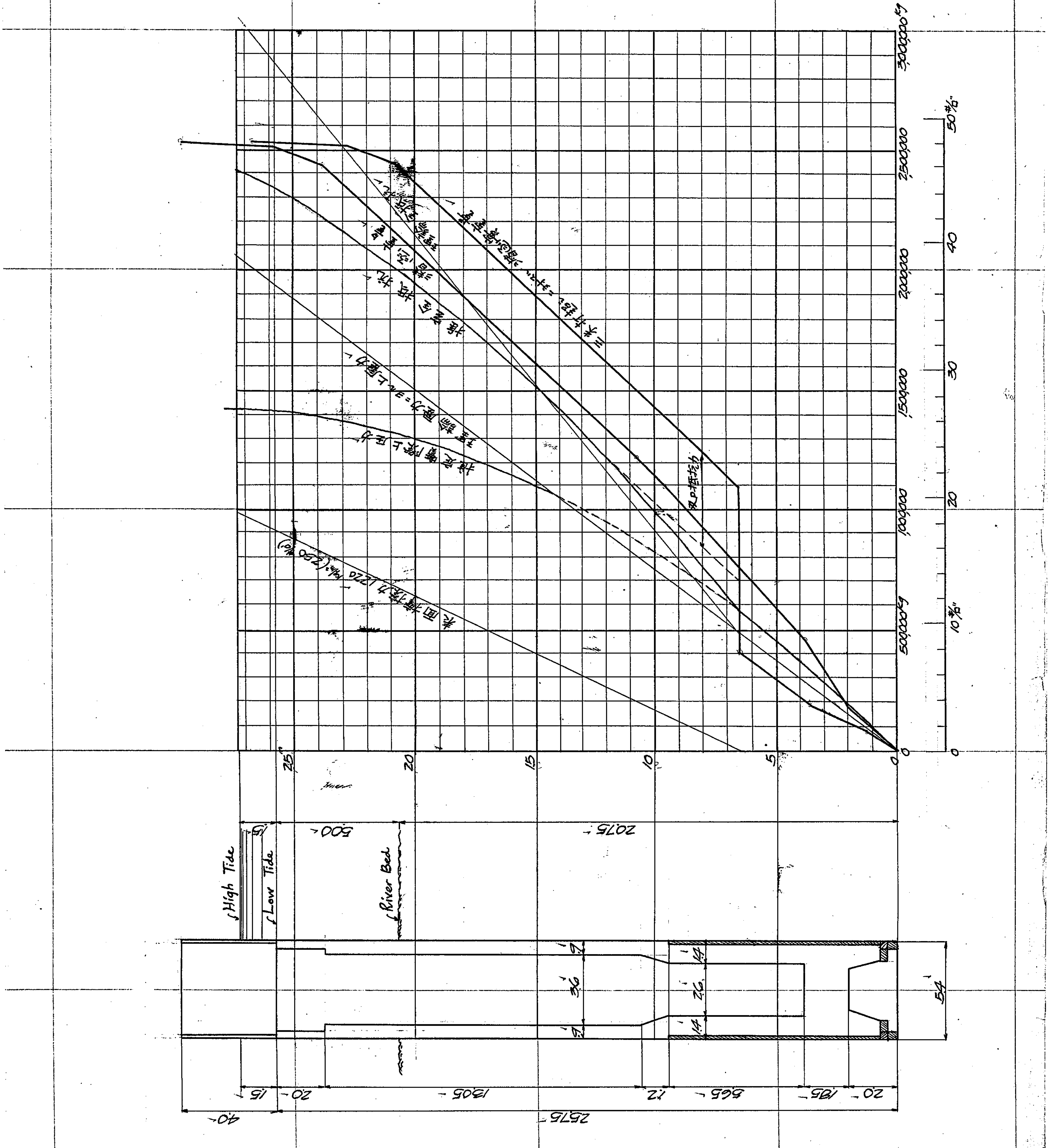
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.
Sinking diagram of River Caisson 5.4×14.0×27.25 m
for Pier P11.



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CALCULATIONS FOR

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Design of Kiso-gawa Bashi for Aichi-Ken.

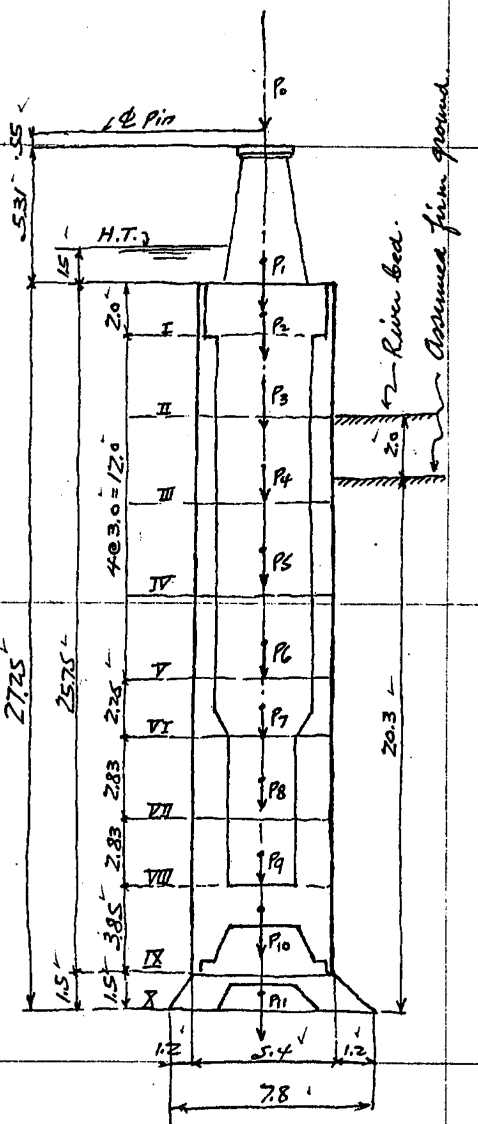
Stability of Pier.

Superimposed Loads on Pier. Dead Load D = 512,000
Live Load L = 170,000
P0 = 682,000 kg on one pier

Weight of shaft including top fill say P1 = 583,000 kg arm 0.86 m above top of caisson

weight of several sections of caisson:
P2 = 75,500 kg arm 1.00 m from bottom.

P3, P4, P5, + P6 1197000 / 13.05 × 3.0 = 275300
water 37400 × 3.0 = 112200
387,500 kg arm 1.50 m.



P7. Concrete 1197000 / 13.05 × 1.05 = 96300
water 37400 × 1.05 = 39300
concrete 127,000
water 37.4 − 25.48 = 31.44 × 1.2 @ 1000 = 37,700
300,500 kg arm 1.10 m.

P8 + P9. 624000 ÷ 2 = 312000
water 25.48 × 2.83 @ 1000 = 72000
timbers say 36000 ÷ 2 = 18000
402,000 kg arm 1.41 m.

P10 Concrete (P2+4) 370,000
timbers say 41,000
411,000
Concrete 11.85 × 3.35 × 1.4 = 53.9
" 13.05 × 4.45 × .6 = 34.8
timbers less approx. − 1.6
87.1 @ 2200 = 192,000
603000 kg arm 1.95 about.

P11. 14.0 × 5.4 = 75.60
16.4 × 7.8 = 127.80
203.40 ÷ 2 = 101.7
101.7 × 1.5 @ 2200 = 336,000
10.6 × 2.0 = 21.2
12.4 × 3.8 = 47.5
68.7 ÷ 2 = 34.35
34.35 × 1.35 = 46.35 @ 2200 = −102,000
46.35 @ 1600 = 74200
308,200 kg arm 0.70 m

Center of Gravity of pier

Loads	weight		arm		moment
P1	583000	×	28.11	=	16,400,000
P2	75500	×	26.25	=	1,980,000
P3 P4 P5 + P6	1,550,000	×	19.25	=	29,830,000
P7	300,500	×	12.10	=	3,640,000
P8 + P9	804,000	×	8.18	=	6,580,000
P10	603,000	×	3.45	=	2,080,000
P11	308,200	×	0.70	=	216,000
Total wt. =	4224200 kg	×	14.38 m	=	60,726,000

Seismic force = 4224200 × .3 = 1267300 kg

Seismic moment = 1267300 × 14.38 = 18218,000 kgm.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Stability of Pier at normal state.

Superimposed Dead and Live Load = 687,000 kg
weight of Pier = 4,224,200
4,906,200 kg

Skin friction 1220 kg/m² (250 #/□')
1220 × 38.8 × 18.8 = − 890,000
4,016,200 kg

unit bearing pressure on soil = $\frac{4016200}{16.4 \times 7.8}$ = 31400 kg/m² or (2.87 tons/□')

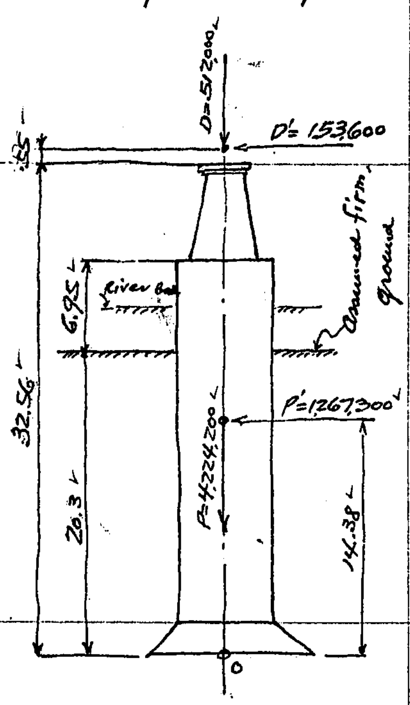
Skin friction 1465 kg/m² (300 #/□')
1465 × 38.8 × 18.8 = − 1068000
3838,200 kg

unit bearing pressure on soil = $\frac{3838200}{16.4 \times 7.8}$ = 30000 kg/m² or (2.74 tons/□')

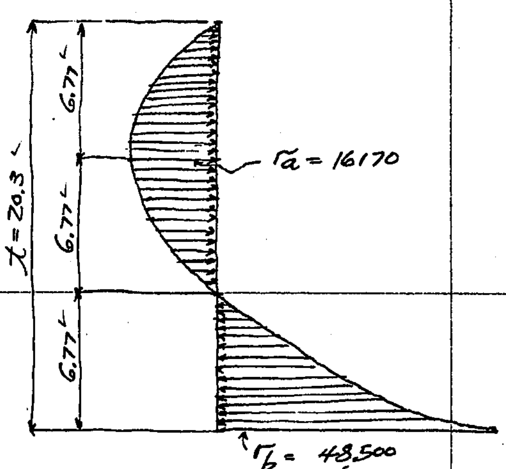
Stability during Earthquake k assumed 0.300.

moment due to seismic forces about center of base O.

D' = 153,600 × ~~38.06~~ 33.11 = 5,090,000
P' = 1267300 × 14.38 = 18,218,000
1420,900 kg M = 23,308,000 kgm



Sum of vertical load.
D = 512000
P = 4,224,200
4,736,200 kg



$\sigma_b = \frac{12M}{t^2} = \frac{12 \times 23308000}{20.3^2}$ = 679000 kg

$\sigma_a = \frac{\sigma_b}{3}$ = 679000 ÷ 3 = 226,300 kg

for one meter strip

σ_b = 679000 ÷ 14.0 = 48,500 kg/m² (4.43 tons/□')
σ_a = 226300 ÷ 14.0 = 16,170 " (1.48 ")

Passive bearing pressure of earth. see page 14.

Depth of earth	= 20.3	6.77
Shrinkage of water min. 6.95 − 1/16 =	4.4	4.4
	24.7 m	11.17 m

passive bearing pressure at σ_a = 11.17 × 1600 × 1.60 = 28600 kg/m² > 16170 ok
" " " " σ_b = 24.7 × 1600 × 1.60 = 62000 " > 48500 ok.

上記 σ_b ハ 14ᵐ 幅トシテ計算セルモ実際 spread base ノ下端幅ハ 16.4ᵐ ナリ 之レニヨレバ
unit bearing pressure σ_b' = $\frac{679000}{16.4}$ = 41400 kg/m² or (3.79 tons/□') ok.

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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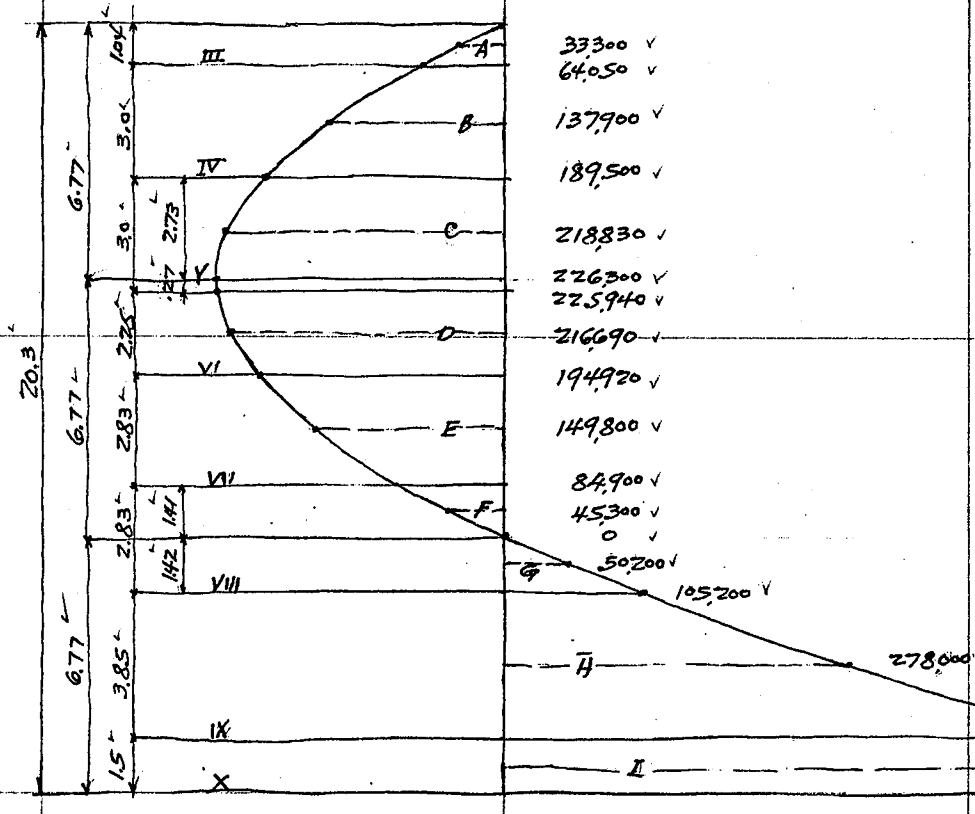
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi Ken.

Reactional pressure diagram



Pressure on each section by sympsons' formula.

			arm from bottom.
A	197250 × .52 ÷ 3 =	34,200	.347 m
B	805150 × 1.50 ÷ 3 =	402,600	1.25
C	1290760 × 1.50 ÷ 3 =	645400	1.41
D	1287660 × 1.125 ÷ 3 =	483,000	1.15
E	879020 × 1.415 ÷ 3 =	414,500	1.60
F	266100 × .705 ÷ 3 =	62,600	.94
		2042300 kg	

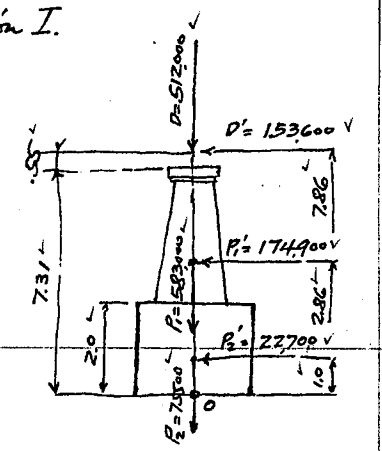
+ Area = 226300 × 13.54 × 2/3 = 2042000 kg

G	306000 × .71 ÷ 3 =	72,400	.47
H	1707000 × 1.925 ÷ 3 =	1095000	1.49
I	3499600 × .75 ÷ 3 =	875000	0.89
		2042400 kg	

− area = 2042000 kg

Moments at several sections during earthquake. k assumed 0.300

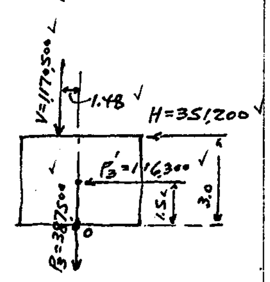
Section I.



Taking moment about 0.

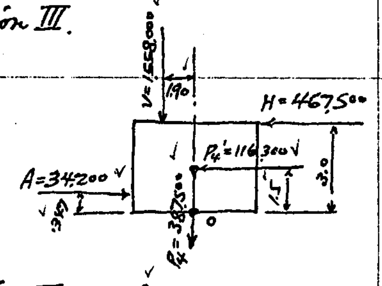
Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
D		512000	0	= 0
D'	153600		7.86	= 1207500
P_1		583000	0	= 0
P_1'	174900		2.86	= 500000
P_2		75500	0	= 0
P_2'	22700		1.00	= 22700
	H = 351200 kg	V = 1170500 kg	1.48 m	= 1730200 kgm.

Section II



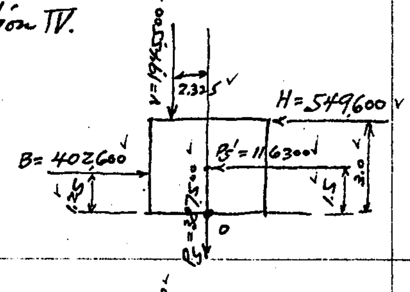
Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
V		1170500	1.48	= 1730200
H	351200		3.00	= 1053600
P_3		387500	0	= 0
P_3'	116300		1.50	= 174500
	H = 467500 kg	V = 1558000 kg	1.90 m	2958300 kgm.

Section III.



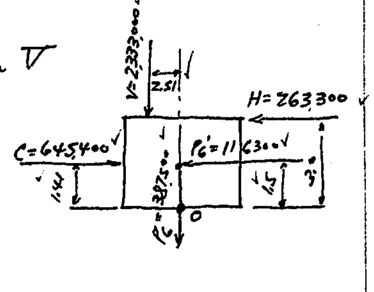
Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
V		1558000	1.90	= 2958300
H	467500		3.00	= 1402500
P_4		387500	0	= 0
P_4'	116300		1.50	= 174500
A	− 34200		.347	= − 11900
	H = 549600 kg	V = 1945500 kg	2.325 m	4523400 kgm

Section IV.



Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
V		1945500	2.325	= 4523400
H	549600		3.00	= 1649000
P_5		387500	0	= 0
P_5'	116300		1.50	= 174500
B	− 402600		1.25	= − 503200
	H = 263300 kg	V = 2333000 kg	2.51 m	= 5843700 kgm.

Section V



Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
V		2333000	2.51	= 5843700
H	263300		3.00	= 789900
P_6		387500	0.00	= 0
P_6'	116300		1.50	= 174500
C	− 645400		1.41	= − 910000
	H = − 265800 kg	V = 2720500 kg	2.17 m	5898100 kgm.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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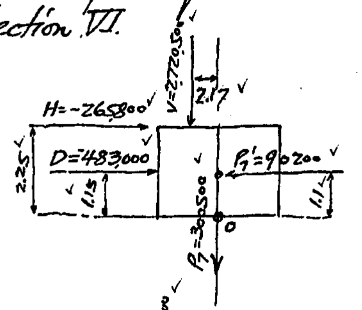
CALCULATIONS FOR

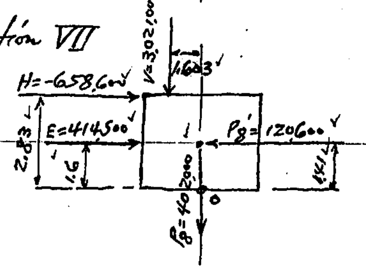
Design of Kiso-gawa Bashi for Aichi Ken

Section VI.



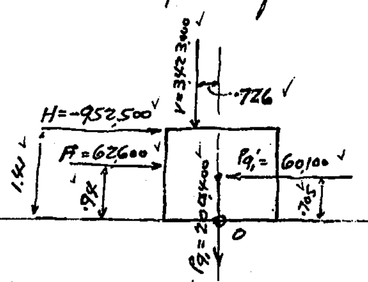
Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		2720,500	2.17	5898,100
H	−265,800		2.25	−598,000
P_7		300,500	0	0
P_7'	90,200		1.10	99,200
D.	−483,000		1.15	−555,000
	H = −658,600 kg	V = 3021,000 kg	1.603 m	4844,300 kgm.

Section VII



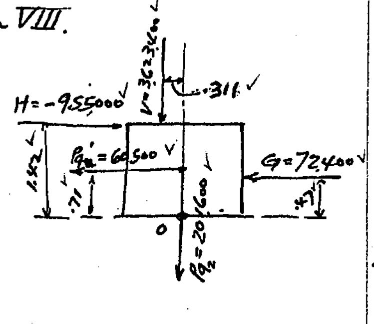
Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		3021,000	1.603	4844,300
H	−658,600		2.83	−1865,000
P_8		402,000	0	0
P_8'	120,600		1.41	170,200
E	−414,500		1.60	−663,000
	H = −952,500 kg	V = 3423,000 kg	0.726 m	2486,500 kgm

Section at 1/3 point from bottom.



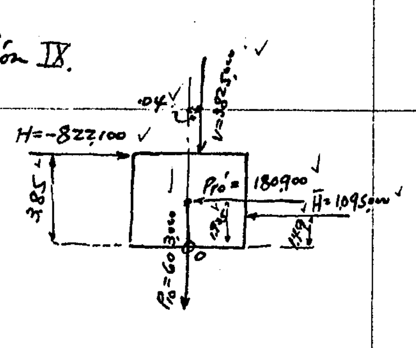
Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		3423,000	0.726	2486,500
H	−952,500		1.41	−1342,500
P_9		200,400	0.00	0
P_9'	60,100		0.705	42,300
F	−62,600		0.94	−58,800
	H = −955,000 kg	V = 3623,400 kg	0.311 m	1,127,500 kgm

Section VIII.



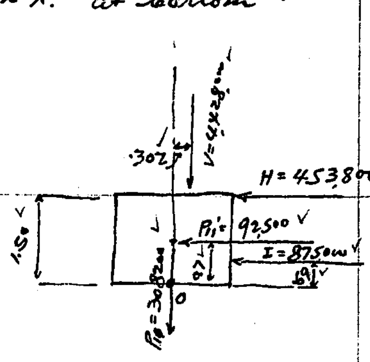
Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		3623,400	0.311	1,127,500
H	−955,000		1.42	−1355,500
$P_{9'}$		201,600	0.00	0
$P_{9'}'$	60,500		0.71	43,000
G	72,400		0.47	34,000
	H = −822,100 kg	V = 3825,000 kg	−0.040 m	−151,000 kgm

Section IX.



Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		3825,000	−.040	−151,000
H	−822,100		3.85	−3163,000
P_{10}		603,000	0.00	0
P_{10}'	180,900		1.925	348,000
$\bar{H}$	1095,000		1.49	+1630,000
	H = 453,800 kg	V = 4428,000 kg	.302 m	−1336,000 kgm

Section X. at bottom.



Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		4428,000	.302	−1336,000
H	453,800		1.50	680,000
P_{11}		308,200	0.00	0
P_{11}'	92,500		0.70	64,700
I	875,000		0.69	603,000
	H = 1421,500 kg	V = 4736,200 kg		+ 11,700 kgm

計算上，誤差ニテ 0トナルベキモノナリ 誤差僅少ニ付訂正セズ

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MADE BY K. Inaba DATE 5-7-18 FILE NO Sub.

CHECKED BY M. Iida DATE 5-7-19 PAGE NO 31.

CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi Ken

Vertical Reinforcements of Caisson.

max. moment = 5898,100 kgm at section V.
for which Vert. load 2720,500 kg
less wt. of water 112200×4 = 448800
2271,700 kg net for shell only.

Sectional area of caisson = 38.2 m² see page 10

Direct compression on concrete = $\frac{2271.700}{38.2}$ = 59500 kg/m² or 5.95 kg/cm²

Bending stress = $\frac{5898100}{4.5}$ = 1311,000 kg

1311,000 ÷ 14 = 93700 kg T or C per lin m of side wall

Direct compression 5.95×100×90 = 53550
40,150 kg T on steel
or 147,250 kg C on concrete.

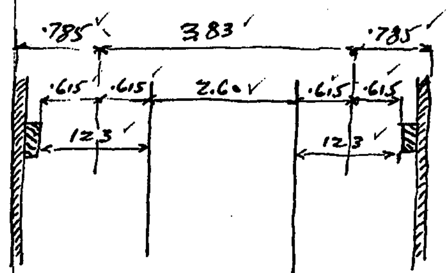
Unit compression on concrete f_c = $\frac{147.250}{90\times100}$ = 16.37 kg/cm² ok.

Steel area required for tension = $\frac{40150}{1200\times1.6}$ = 20.90 cm² per meter of side wall.

Use 25mmφ bars at 45cm c to c = 21.82 cm² ok.

at section VI

moment = 4844,300 kgm
vert. load = 3021,000
less water = − 525800
2495,200 kg for shell only.



Sectional area of caisson 1.23 × 13.66 × 2 = 33.60
1.23 × 2.60 × 2 = 6.40
40.00 sq.m

Partition wall. 2.60 × .70 × 2 = 3.64
43.64 m² Total effective sectional area of caisson.

Direct compression on concrete = $\frac{2495200}{43.64}$ = 57200 kg/m² or 5.72 kg/cm²

Bending stress = $\frac{4844300}{3.83}$ = 1265,000

1265000 ÷ 14 = 90400 kg T or C per lin m of side wall.

Direct compression = 5.72×100×123 = 70400 C
20,000 kg T on steel
or 160800 C on concrete

Unit compression on concrete f_c = $\frac{160800}{100\times123}$ = 13.07 kg/cm² ok.

Steel area required for tension = $\frac{20000}{1200\times1.6}$ = 10.42 cm² per lin m of side wall.

Use same reinforcements as for section V. (25φ bars − 45~50cm c to c)

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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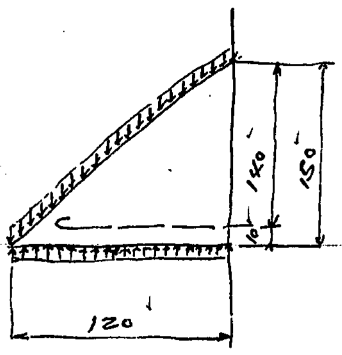
MADE BY K. Inaba DATE 5-7-18 FILE NO Sub.

CHECKED BY M. Imida DATE 5-7-19 PAGE NO 32

CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Reinforcements for Spread base.



max. upward pressure = 31400 kg/m² page 28

Earth on footing assumed 15×1.6 m = 24000 including wt. of footing.

7400 "

moment on footing = $\frac{7400 \times 1.2^2}{2}$ = 5330 kg per meter strip.

Shear " " 7400 × 1.2 = 8880

Effective depth required = $\sqrt{\frac{5330 \times 100}{100 \times 7.18}}$ = 27.3 cm

use effective depth of 140 cm with 10 cm insulation

Steel area required = $\frac{5330 \times 100}{1200 \times \frac{7}{8} \times 140}$ = 3.63 cm² per m strip.

Use 22 mmφ bars at 60 cm c/c = 6.34 cm²

Steel ratio $p = \frac{6.34}{100 \times 140}$ = .00045, j = .95 about.

Unit shear = $\frac{8880}{100 \times .95 \times 140}$ = .67 kg/cm² ok

Unit bond = $\frac{8880}{\frac{6.91}{.6} \times .95 \times 140}$ = 5.80 kg/cm² ok.

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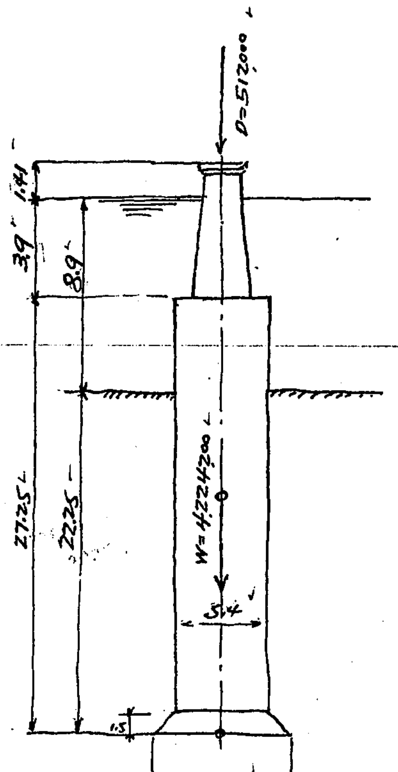
MADE BY K. Inaba DATE 5-7-18 FILE NO Sub.

CHECKED BY M. Jinda DATE 5-7-19 PAGE NO 33

CALCULATIONS FOR

Design of Kiso-gawa Bashi for Aichi-ken.

Stability of pier no 11. during Flood water.



Superimposed Dead Load = 512,000 kg

weight of pier, page 27. = 4,224,200

4,736,200 kg.

Bouyancy on pier (Full bouyancy assumed on safe side

volume of caisson 5.4×14.0×25.75= 1947

base 6.6×15.2×1.5= 151

shaft. 2.68×11.0×3.9= 115

2213 cub m.

Bouyancy = 2213×1000 = 2,213,000 kg

mean velocity of Flood water assumed 2.50 m/sec (or 8.25'/sec.)

End area of submerged portion of pier

Shaft 2.68×3.9 = 10.45

Caisson 5.4×5.0 = 27.00

37.45 sq. m.

M.V. = A.V.× w/g ×v

momentum of flowing water against pier

g = 9.8 m/sec²

w = 1000 kg/m³

v = 2.5 m/sec.

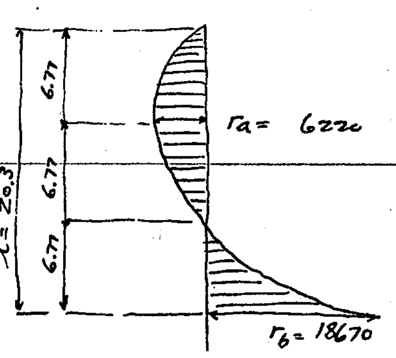
= 37.45 × 2.5 × 1000/9.8 × 2.5 = 23,880 kg on pier

call this 24,000 kg. lev. assumed at middle point

22.25 + 8.9/2 = 26.7

moment on pier = 24,000 × 26.7 = 641,000 kgm = M.

Reactional earth pressure (transverse).



$r_b = \frac{12M}{t^2} = \frac{12 \times 641000}{20.3^2} = 18,670$ kg

$r_a = \frac{r_b}{3} = \frac{18670}{3} = 6,220$ kg

for one meter strip

r_b = 18670 ÷ 5.4 = 3,460 kg/m² < 28,600 ok page 28

r_a = 6220 ÷ 5.4 = 1150 " < 62,000 ok "

Total vertical load = 4,736,200

bouyancy = 2,213,000

2,523,200 kg

Unit bearing pressure on soil = $\frac{2523200}{16.4 \times 7.8}$ = 19750 kg/m² or (1.81 tons/□') ok.

The designed pier is safe during Flood water.

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MADE BY M. Junda DATE 5-6-3 **FILE NO** Sub

CHECKED BY M. Kojima DATE 5-6-4 **PAGE NO** 34

CALCULATIONS FOR

Materials of Kisogawa-Bashi for Aichi-Ken

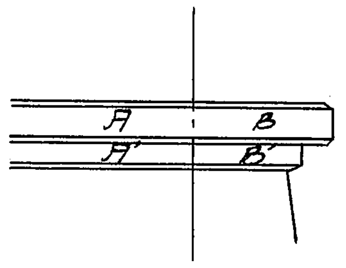
Materials of Shaft for P1, P2, P3, P4, P5, P6 & P7

Shaft for P8, P9, P10, P11, P12 & P13 are identical to Shaft for P7, P6, P5, P4, P3 & P2 respectively

Concrete for Shaft 1:2:4 mixture

Projection under shoe for P1 only

2 @ .85 × 1.10 × .12 = .224 cub. m



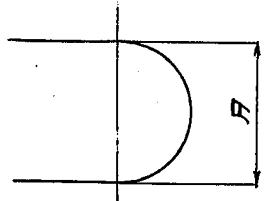
Coping

	Section	length	req'd no.	Volume	remarks
Coping rectangle	2.40 × .30	9.20	1	6.624	A
〃	2.22 × .15	9.20	1	3.064	A'
Coping circular end	2.40φ	.30	1	1.357	B
〃	2.22φ	.15	1	.581	B'
		Total		= 11.626 cub. m.	

Shaft

Top section

rectangular	2.10 × 9.20	1	19.320
circular	2.10φ	1	3.464
			22.784 sq. m.

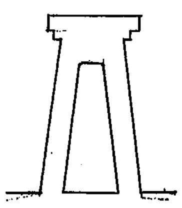


Bottom sections.	A rectangular		circular end	Bottom section
P1	2.985 × 9.20	= 27.462	+ 6.998	34.460
P2	3.020 × 〃	= 27.784	+ 7.163	34.947
P3	3.049 × 〃	= 28.051	+ 7.301	35.352
P4	3.073 × 〃	= 28.272	+ 7.417	35.689
P5	3.090 × 〃	= 28.428	+ 7.499	35.927
P6	3.102 × 〃	= 28.538	+ 7.557	36.095
P7	3.108 × 〃	= 28.594	+ 7.587	36.181

Top Section of hollow

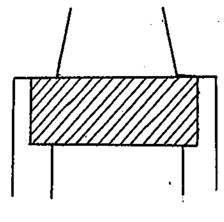
2 @ 1.11 × 2.45 = 5.439 sq. meters

Bottom Sections of hollow



Piers	bottom width	length	required	area
P1	1.785	× 2.45	× 2	= 8.747
P2	1.820	× 〃	× 〃	= 8.918
P3	1.849	× 〃	× 〃	= 9.060
P4	1.872	× 〃	× 〃	= 9.173
P5	1.890	× 〃	× 〃	= 9.261
P6	1.902	× 〃	× 〃	= 9.320
P7	1.908	× 〃	× 〃	= 9.349

volume of Slab at Bottom



4.50 × 13.10 × 2.00 = 117.900

−2 @ .76 × .76 × 2.00 = 2.310 less corner

115.590 cub. m.

Total volume of concrete

Pier	Top A	Bottom A	Mean A	Height	volume	Top A (hollow less)	Bottom A (hollow less)	Mean A (hollow less)	Height (hollow less)	Volume (hollow less)	Coping	Slab	Shoe bed	Total volume
P1	22.784	34.460	28.622	4.423	126.595	5.439	8.747	7.093	3.373	−23.925	11.626	115.590	.224	230.110
P2	〃	34.947	28.866	4.599	132.755	〃	8.918	7.179	3.549	−25.478	〃	〃		234.493
P3	〃	35.352	29.068	4.746	137.957	〃	9.060	7.250	3.696	−26.796	〃	〃		238.377
P4	〃	35.689	29.237	4.863	142.180	〃	9.173	7.306	3.813	−27.858	〃	〃		241.538
P5	〃	35.927	29.356	4.951	145.342	〃	9.261	7.350	3.901	−28.672	〃	〃		243.886
P6	〃	36.095	29.440	5.010	147.494	〃	9.320	7.380	3.960	−29.225	〃	〃		245.485
P7	〃	36.181	29.483	5.039	148.565	〃	9.349	7.394	3.989	−29.495	〃	〃		246.286

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MADE BY M. Imida DATE 5-6-3 FILE NO Sub

CHECKED BY M. Kojima DATE 5-6-4 PAGE NO 35

CALCULATIONS FOR

Revised 5-8-21

Materials of Kisogawa-Bashi for Aichi-Prefecture

Forms for Shaft

Projection under shoe for P1 only

2 @ .13 × 4.30 = 1.12 sq.m.

Coping

Side	2 @ .60 × 9.20	= 11.04	
circular end	1 @ 2.40ϕ × .60	= 4.52	
		15.56 sq.m.	

Shaft, Total length of perimeter at top of Shaft

Side	2 @ 9.20	= 18.40	
circular end	1 @ 2.10ϕ	= 6.60	
		25.00 sq.m	

Total length of perimeter at bottom of Shaft

Pier	Circular end	Perimeter	Side	Total perimeter
P1	2.985ϕ	9.38	+ 18.40	= 27.78
P2	3.020ϕ	9.49	+ 〃	= 27.89
P3	3.049ϕ	9.58	+ 〃	= 27.98
P4	3.073ϕ	9.65	+ 〃	= 28.05
P5	3.090ϕ	9.71	+ 〃	= 28.11
P6	3.102ϕ	9.75	+ 〃	= 28.15
P7	3.108ϕ	9.76	+ 〃	= 28.16

Top length of hollow

4 @ 2.45 + 4 @ 1.11 = 14.24 m

Bottom length of hollow

P1	4 @ 2.45	+ 4 @ 1.785	= 16.94
P2	〃	+ 4 @ 1.820	= 17.08
P3	〃	+ 4 @ 1.849	= 17.20
P4	〃	+ 4 @ 1.872	= 17.29
P5	〃	+ 4 @ 1.890	= 17.36
P6	〃	+ 4 @ 1.902	= 17.41
P7	〃	+ 4 @ 1.908	= 17.43

Total area of forms

Pier	Top l	Bottom l	Mean l	Height	area	Top l	Bottom l	Mean l	Height	area	Coping	Shoe bed	Total area
P1	25.00	27.78	26.39	4.423	116.72	14.24	16.94	15.59	3.373	52.59	15.56	1.12	185.99 sq.m
P2	〃	27.89	26.45	4.599	121.64	〃	17.08	15.66	3.549	55.58	〃		192.78
P3	〃	27.98	26.49	4.746	125.72	〃	17.20	15.72	3.696	58.10	〃		199.38
P4	〃	28.05	26.53	4.863	129.02	〃	17.29	15.77	3.813	60.13	〃		204.71
P5	〃	28.11	26.56	4.951	131.50	〃	17.36	15.80	3.901	61.64	〃		208.70
P6	〃	28.15	26.58	5.010	133.17	〃	17.41	15.83	3.960	62.69	〃		211.42
P7	〃	28.16	26.58	5.039	133.94	〃	17.43	15.84	3.989	63.19	〃		212.69

埋込型枠

Top of hollow	2 @ 1.11 × 2.45	= 5.44	
Bottom of slab	3 @ 3.60 × 3.60	= 38.88	
less fillets	6 @ .50 × .50	= -1.50	
		42.82 sq.m	

Reinforcements, plain bars

P1	6.097 Kg tons
P2 & P3	6.220 〃
P4, P5, P6 & P7	6.332 〃

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MADE BY M. Jinda DATE 5-6-5 **FILE NO** Sub

CHECKED BY M. Kojima DATE 5-6-6 **PAGE NO** 36

CALCULATIONS FOR

Revised 5-8-21

Materials of Kisogawa-Bashi for Aichi-Prefecture

Materials of land caisson

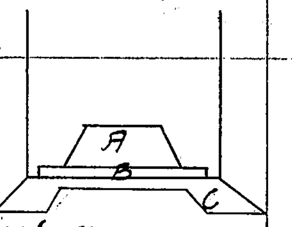
Concrete 1:3:6 mixture

Concrete fill in base

	Top area	Bottom area	Mean area	Height	Volume
C	5.40 × 14.00 = 75.60	7.40 × 16.00 = 118.40	97.00	1.20	116.400
	1.80 × 10.40 = 18.72	3.40 × 12.00 = 40.80	29.76	1.05	− 31.248 less earth volume
					85.152 cub. m.

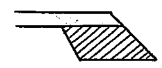
(1:3:6 mix)

Concrete in working chamber	Top area	Bottom area	Mean area	Height	Volume
A	3.00 × 11.60 = 34.80	4.00 × 12.60 = 50.40	42.60	1.70	72.420
B	4.90 × 13.50 =		66.15	.30	19.845
Material shaft	1.22φ		1.17	1.20	1.404
Man shaft	.91φ		.65	1.20	.780
					94.449 cub. m



Total concrete of bottom fill. (1:3:6 mix.) 179.601 cub. m.

Volume of base concrete fill per lin. meter 1.90 × 1.20 = 2.28 cub. m.



Concrete for caisson (1:2:4 mix)

Working chamber and slab

Total volume 5.40 × 14.00 × 3.20 = 241.920

less hollow Same as working chamber = (−) 94.449

147.471 cub. m.

Shell lower parts 16.40 deep

		Cross sectional area	depth
Shell	5.40 × 14.00 =	75.60	
less hollow	3 @ 3.60 × 3.60 = (−)	38.88	
fillet	6 @ .50 × .50 =	1.50	
		38.22 × 16.40	= 626.808 cub m.

Shell upper 2.20 m

Shell cross sectional area 38.22

less corner 2 @ 1.03 × 1.03 = (−) 2.12

36.10 × 2.20 = 79.420 cub. m.

Shell top 2.00 depth

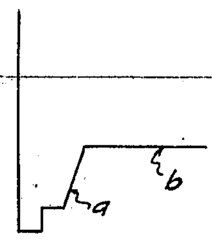
Shell	5.40 × 14.00 =	75.60	
less corner	2 @ 1.03 × 1.03 = (−)	2.12	
less	4.50 × 13.10 = (−)	58.95	
fillet	2 @ .76 × .76 =	1.16	
		15.69 × 2.00	= 31.380
less embeded timber .10 × .20 × 35.36			= − .707
			30.673 cub m

1:2:4 Concrete filling in partition hole (5 m depth) = {3.60 × 3 − (1.22φ + .91φ + .50 × 6)} × 5 = 17.781

Grand summary of concrete for caisson = 902.153 cub. m (1:2:4 mix)

Forms

Inner face of working chamber	a	2 @ (11.60 + 12.60)/2 × 1.65	=	39.93
"	a	2 @ (3.00 + 4.00)/2 × 1.65	=	11.55
"	b	3.00 × 11.60	=	34.80
less material shaft		1.22φ	= (−)	1.17
less man shaft		.91φ	= (−)	.65
				84.46 sq. m



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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY *M. Jinda* DATE *5-6-5* FILE NO *Sub*

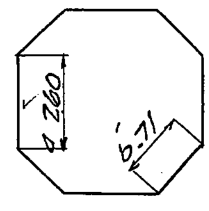
CHECKED BY *M. Kojima* DATE *5-6-6* PAGE NO *37*

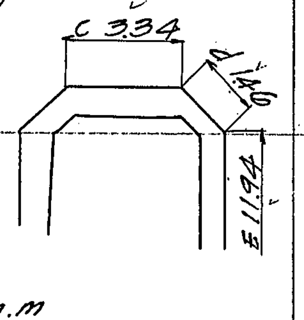
Revised 5-8-21

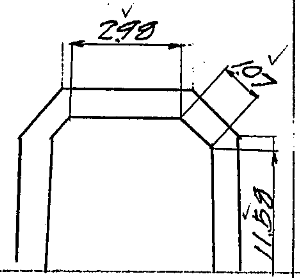
CALCULATIONS FOR

Materials of Kisogawa-Bashi for Aichi-Prefecture

Outer face of working chamber	38.80 × 2.59 =		100.49 sq.m
Outside of Shell 16.40 depth		38.80 × 16.40 = 636.32	
Inside of Shell 〃 〃	a 12 @ 2.60 = 31.20		
〃	b 12 @ .71 = 8.52		
		39.72 × 16.40 = 651.41	
			1,287.73 sq.m
outside of Shell 2.20 depth	c 2 @ 3.34 = 6.68		
	d 4 @ 1.46 = 5.84		
	E 2 @ 11.94 = 23.88		
		36.40 × 2.20 = 80.08	
Inside of Shell 2.20 depth		39.72 × 2.20 = 87.38	
			167.46 sq.m
Outside of Shell top 2.00 depth		36.40 × 2.00 = 72.80	
Inside of Shell 〃	2 @ 11.58 = 23.16		
〃	2 @ 2.98 = 5.96		
〃	4 @ 1.07 = 4.28		
		33.40 × 2.00 = 66.80	
			139.60 sq.m
	Grand Summary of Forms of Caisson		= 1,719.74 sq.m
Reinforcements,	Plain bars		52.358 Kg tons
Curb Shoe	structural steel		5.139 Kg tons







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CALCULATIONS FOR

Revised 5-8-21

Materials of Kisogawa-Bashi for Aichi-Prefecture

Materials for shell of River Caisson (for all caisson) top 16.25m

Concrete 1:2:4 mixture

		Sectional area	
Bottom (1.2m depth)	.9 × (14.0 × 2 + 3.6 × 2)	= 31.68	38.22
〃	.7 × 3.6 × 2	= 5.04	
〃	.5 × .5 × 6	= 1.50	
〃 fillet	.5 × 30 × 4	= 6.00	
		44.22 × 12 = 530.64 cub.m	

Middle part (10.85m depth)
sectional area × 10.85m = 38.22 × 10.85 = 414.687 cub.m

Upper part (2.20m depth)
(38.22 − 1.03 × 1.03 × 2) × 2.2 = 36.098 × 2.2 = 79.416 cub.m

Top wall (2.00m depth)

Side	.45 × 14.00 × 2	= 12.600
End	.45 × 4.50 × 2	= 4.050
Corner	.76 × .76 × 2	= 1.155
less	1.03 × 1.03 × 2	= −2.122
		15.683 × 2.0 = 31.366
less embeded timber	.10 × .20 × 35.36	= −.707
		30.659

Total volume of shell = 577.826 cub.m

Reinforcements, Plain bars 30.211 kg.tons

Forms

outside	upper	(5.97 × 4 + 3.34 × 2 + 1.46 × 4) × 4.10	= 149.24
〃		(14.00 × 2 + 5.40 × 2) × 12.05	= 467.54
Inside	top	(5.79 × 4 + 2.98 × 2 + 1.08 × 4) × 2.00	= 66.88
〃		2.60 × 12 × 14.35	= 447.72
〃	fillet	.71 × 12 × 14.25	= 121.41
		Total	= 1,252.79 sq.m

Excavations

Piers	Base		Shell (sect' Area)		Length	Volume	Total Volume		内訳 満潮面以下（ケーソン先端ヨリ1.5m上ト假定ス） 0m–15m	15–20m	20–25m	25以下	Base
P1	8.52	+	75.60	×	25.30	1,912.7	1,997.9	below high tide					
P2	〃	+	〃	×	〃	〃	〃	〃	1134.0	378.0	378.0	22.7	8.52
P3	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P4	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P5	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P6	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P7	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P8	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P9	〃	+	〃	×	〃	〃	〃	〃	〃	〃	〃	〃	〃
P10	10.65	+	〃	×	20.99	1,586.8	1,693.3	below river bed	830.8	〃	〃	0	10.65
P11	〃	+	〃	×	20.60	1,557.4	1,663.9	〃	631.4	〃	〃	170.0	〃
P12	〃	+	〃	×	20.48	1,548.3	1,654.8	〃	792.3	〃	〃	0	〃
P13	8.52	+	〃	×	25.30	1,912.7	1,997.9	below high tide	1,134.0	〃	〃	22.7	8.52

360.

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CALCULATIONS FOR

Revised 5-8-21

Materials of Kisogawa-Bashi for Aichi-Prefecture

Materials of River caisson

Concrete 1:3:6 Mixture

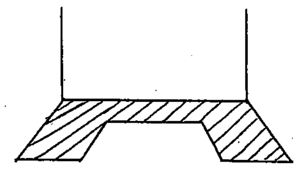
Floating caisson for 7.25m

Base (1.5m depth)

(14.0 × 5.4 + 16.4 × 7.8) × 1/2 × 1.5 = 152.640

less (10.6 × 2.0 + 12.4 × 3.0) × 1/2 × 1.35 = -46.116

106.524 cub.m



In working chamber (2.0m depth) (1:3:6 mix)

a 13.5 × 4.9 × .3 = 19.845

less (.12 × 2 × 24 + .12 × .12 × 4) × .24 = - 0.152 (木材)

〃 .3 × .3 × 4.9 × 4 = - 1.764 (〃)

b 12.6 × 4.0 × .3 = 15.120

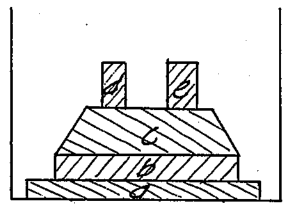
c (12.6 × 4.0 + 11.1 × 2.5) × 1/2 × 1.4 = 54.705

less .2 × .2 × 2.5 × 4 = - .400 (木材 F7)

d 122φ × 1.85 = 2.163

e .91φ × 1.85 = 1.203

90.72 cub.m



Concrete 1:2:4 mixture

summary 197.244 cub.m.

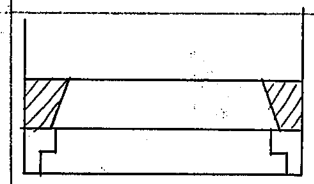
Wall of working chamber (1.45m depth) (1:2:4 mix)

13.66 × 5.06 × 1.45 = 100.223

less (13.0 × 4.4 + 11.5 × 2.9) × 1/2 × 1.45 = - 65.649 (作業室)

〃 .2 × .2 × 1.45 × 56 = - 3.248 (木材)

31.326 cub.m.



Slab (1.8m depth)

Sectional Area

(13.9 × 5.3) × 1.8 = 73.670 sq.m × 1.8 = 132.606

less	.2 × .2 × 1.8 × 28 = -2.016				(側柱)
〃	〃	13.26 × 4 = -2.122			(切張 F1, F2)
〃	〃	2.68 × 4 = -.429			(〃 F6)
〃	〃	4.65 × 18 = -3.348			(〃 F5)
〃	.075 × .2	.4 × 96 = -.576			(切張端縦材 I1)
〃	.2 × .2	.8 × 23 = -.736			(竪材 E1)
〃	〃	.2 × 14 = -.112			(〃 〃)
〃	〃	.25 × 6 = -.060			(斜材 C1, 上部)
〃	.075 × .2	1.1 × 12 = -.198	-15.729		(〃 C2)
〃	〃	2.0 × 36 = -1.080			(〃 C5)
〃	〃	.3 × 24 = -.108			(猫木 C3)
〃	〃	.5 × 12 = -.090			(筋違 H1, 下部)
〃	.12 × .65	5.3 × 2 = -.827			(側壁横板 PL16)
〃	〃	13.66 × 2 = -2.131			(〃 PL 16.17.15)
〃	〃 × .2	5.3 × 2 = -.254			(〃 布木 G1)
〃	〃	13.9 × 2 = -.667			(〃 〃 G2, G1)
〃	.075 × .2	.8 × 10 = -.120			(筋違 H3, 下部)
〃	.12 × 2.6	11.1 × 2 = -.693			(作業室側壁横板 PL1)
〃	〃	2.6 × 2 = -.162			(〃 〃 PL9)

116.877 cub.m

Shell (1 Lot 3.4m depth)

Sectional area

Side wall 1.35 × (13.9 × 2 + 2.6 × 2) = 44.55

Partition wall .7 × 2.6 × 2 = 3.64

48.19 sq.m

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CALCULATIONS FOR

Revised 5-8-21

Materials of Kisogawa-Bashi for Aichi-Prefecture

Shell (1Lot 3.4m depth)

Volume 48.19 × 3.4 = 163.846

less	.12	× .2	× 3.9	× 6	= -2.002		(布木 G1, G2)
〃	.12	× .2	× .53	× 6	= -.763		(〃 G1)
〃	.2	× .2	× 3.4	× 18	= -2.448		(側柱)
〃	.2	× .2	× 3.46	× 2	= -.277	= -7.661	(切張 F1, F2)
〃	.2	× .15	× 3.46	× 4	= -.415		(〃 F3, F4)
〃	.2	× .2	× 2.06	× 15	= -1.236		(〃 F5)
〃	.075	× .2	× .4	× 84	= -.504		(端緒材 I1)
〃	.05	× .2	× .2	× 8	= -.016		(〃 I2)

156.185 cub. m

Total volume of (Concrete for) 7.25m floating caisson 304.388 cub. m

Floating caisson for 9.50m

lower part 7.25m same as above = 304.388 cub. m

Shell (1Lot 2.25m depth top)

Sectional area 48.19 sq. m

volume 48.19 × 2.25 = 108.428

less	.12	× .2	× 13.9	× 4	= -1.334		(布木 G1, G2)
〃	.12	× .2	× .53	× 4	= -.509		(〃 G1)
〃	.2	× .2	× 2.25	× 18	= -1.620		(側 柱)
〃	.15	× .2	× 3.46	× 4	= -.415	= -5.054	(切張 F3, F4)
〃	.2	× .2	× 2.06	× 10	= -.824		(〃 F5)
〃	.075	× .2	× .4	× 56	= -.336		(端緒材 I1)
〃	.05	× .2	× .2	× 8	= -.016		(〃 I2)

103.374 cub. m
304.388

Total volume of 9.50m floating caissons = 407.762 cub. m

Reinforcements, Plain bars
For 7.25m floating caisson = 20.934 Kg. tons
For 9.50m floating caisson = 26.693 Kg. tons

Summary of River caisson

		25.75m Caisson	23.5m caisson
1:2:4 Concrete	in floating caisson	407.762	304.388
	in Shell	577.826	577.826
		985.588 m^3	882.214 m^3
1:3:6 Concrete	in working chamber filling and spread base	197.244 m^3	197.244 m^3
Reinforcements	in floating caisson	26.693	20.934
	in shell	30.211	30.211
		56.904 Kg. tons	51.145 Kg. tons
Floating caisson	米松	103.256 m^3	86.063 m^3
	締付金物	5.246 Kg tons	4.951 Kg. tons
	curb shoe	2.644 Kg tons	2.644 Kg. tons

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CALCULATIONS FOR

昭和五年八月

愛知縣木曾川橋梁架設工事用假棧橋

應力及材料計算書

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MADE BY K. Inaba DATE 5-8-12 **FILE NO** 9
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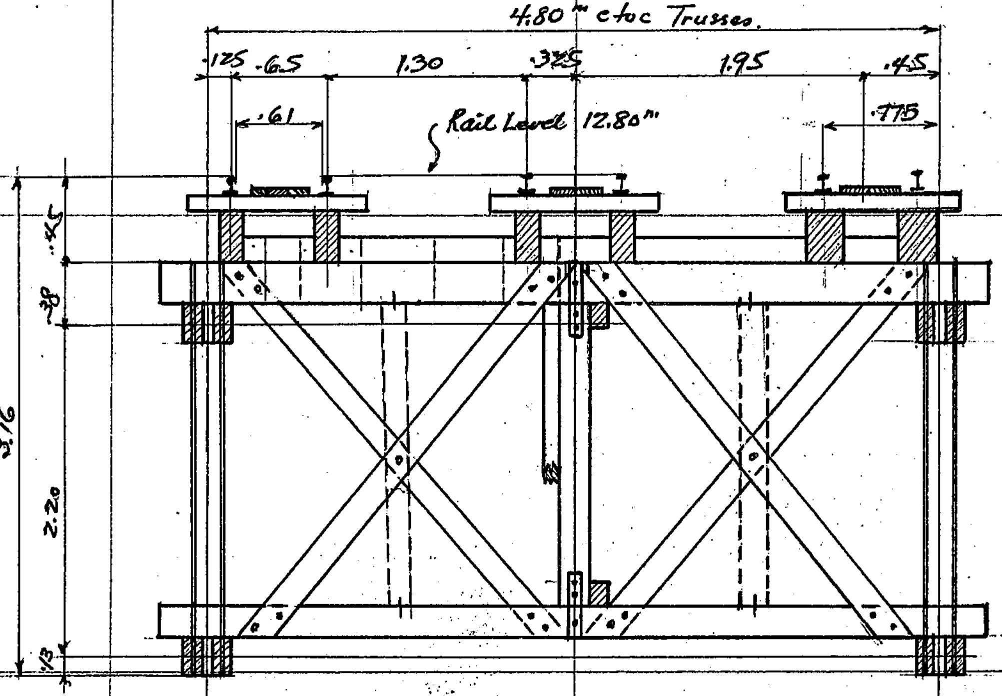
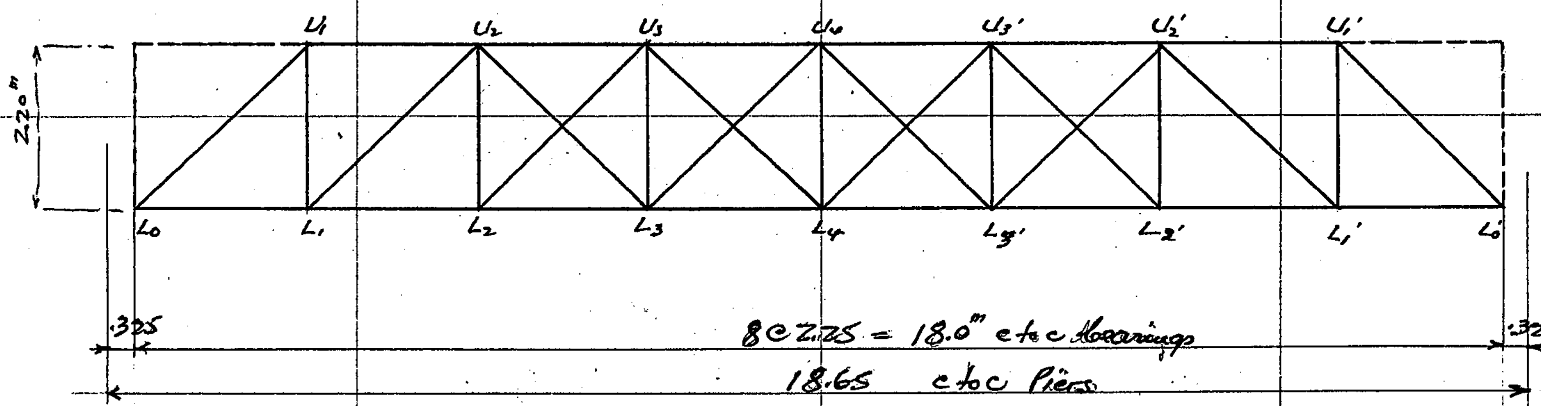
CALCULATIONS FOR

工事用假棧橋計算書

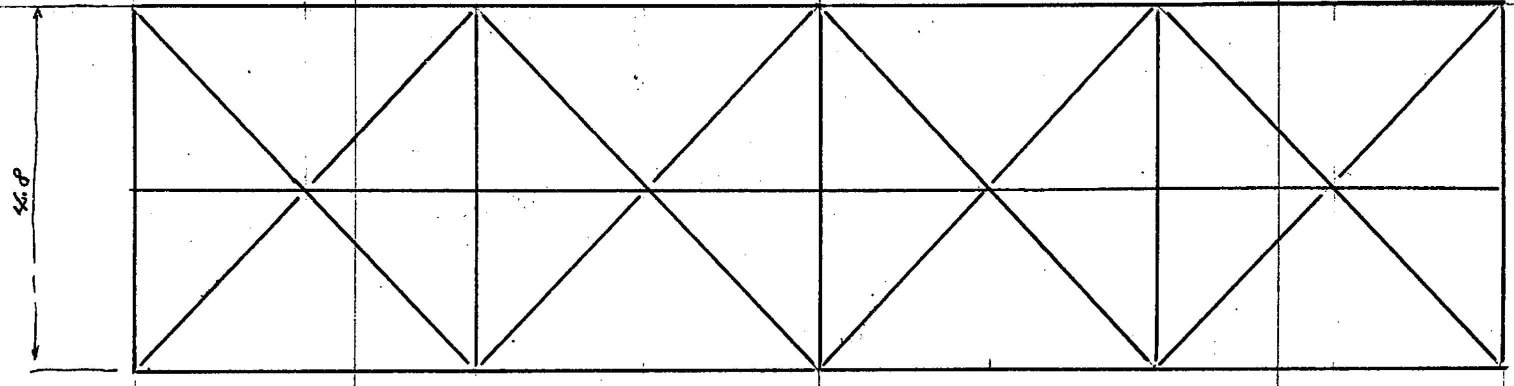
Design of wooden truss. Total length of bridge c to c of end bearings = 1100 meters about.

47 spans @ 18.0m = 846.0
46 clearances @ .65 = 29.91
875.9 meters c to c end bearings.





Skeleton of truss.



Lower Lateral bracings.

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CALCULATIONS FOR

工事用假棧橋計算書

Design of stringers.

Stringers under air lock loads. span length 4.5 meters.

Dead Load:

Track assumed $\frac{1}{2} \times 50 = 25$

beam " 50

75 kg per lin meter.

Dead Load moment $= \frac{1}{8} \times 75 \times 4.5^2 = 190$ kgm

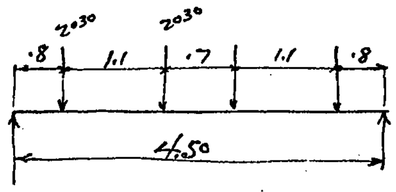
Dead Load shear $= \frac{1}{2} \times 75 \times 4.5 = 170$ kg.

Live Load

max. concentration on wheel due to air lock. $= \frac{1}{4} \times 6500 = 1625$

25% impact $= 405$

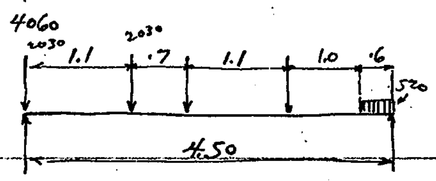
2030 kg



moment $= 4060 \times \overset{1.90}{\cancel{2.75}} = 7710$

$2030 \times 1.1 = -2230$

5480 kgm.



Reaction $2030 \times 3.40 = 6900$

$2030 \times 2.70 = 5480$

$2030 \times 1.60 = 3250$

$520 \times 0.6^2 \div 2 = 94$

$15724 \div 4.50 = 3500$

2030

End shear $= 5530$ kg

Summary of moments and shears.

	moment	end shear
Dead Load	190	170
Live Load	5480	5530
	5670 kgm	5700 kg

Section modulus req'd $= \frac{5670 \times 100}{85} = 6670$ cm³

use 40×25 cm stringer. $S_m = \frac{25 \times 40^2}{6} = 6670$ cm³. OK

Shear stress $= \frac{5530}{30 \times 25} = 7.4$ kg/cm² ok.

Cut off 10 cm at support.

Stringers under material trucks. span length 4.5 meters.

Dead Load: – Track assumed $\frac{1}{2} \times 50 = 25$

beam " 30

55 kg per lin m

Dead Load moment $= \frac{1}{8} \times 55 \times 4.5^2 = 139$ kgm

Dead Load shear $= \frac{1}{2} \times 55 \times 4.5 = 124$ kg.

Live Load.

Load for one truck assumed 1500 kg, for one stringer $1500 \div 2 = 750$ kg

25% impact $= 187$

937 kg

Assuming this load to be distributed uniformly over the length of 1.8 meters.

Uniform load $= 937 \div 1.8 = 520$ kg per lin meter.

Live Load moment $= \frac{1}{8} \times 520 \times 4.5^2 = 1317$ kgm

Live Load shear $= \frac{1}{2} \times 520 \times 4.5 = 1170$ kg.

Summary of moments and shears.

	moment	shear
Dead Load	139	124
Live Load	1317	1170
	1456 kgm.	1294 kg

Section modulus req'd $= \frac{1456 \times 100}{85} = 1715$ cm³

use 30×15 cm $S_m = \frac{15 \times 30^2}{6} = 2250$ cm³ ok.

Unit shear $= \frac{1294}{15 \times 30} = 3$ kg/cm² ok.

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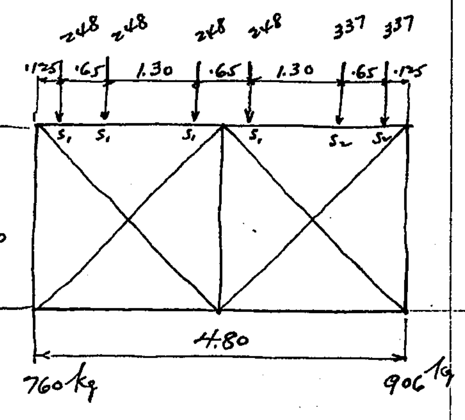
CALCULATIONS FOR

工事用假棧橋計算書

Design of Cross Frame
Dead Load.

Span length 4.8 meters, Spacing 4.5 meters
Stringer concentration on frame.
Stringers S_1 tracks = Stringer/frame 55 × 4.5 = 248 kg
" S_2 " " 75 × 4.5 = 337 "



Dead load of frame assumed. 110 kg per lin m.
Moment on frame due to stringer concentration
Reaction
337 × .90 = 304
248 × 13.50 = 3346
3650 ÷ 4.8 = 760 kg left reaction
1666 − 760 = 906 kg right "

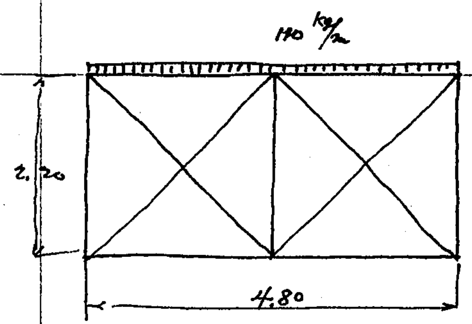
moment.
760 × 2.40 = 1825
248 × 4.225 = −1050
775 kgm.

Load on center panel point
585 × .125 = 73
585 × .775 = 453
496 × 2.075 = 1030
1556 ÷ 2.40 = 650 kg

$\sqrt{2.2^2+2.4^2} = \sqrt{10.60} = 3.26^m$
$\frac{3.26}{2.2} = 1.48^m$

Bending moment on top chord.
Reaction 248 × .325 = 81
337 × 4.225 = 1425
1506 ÷ 2.40 = 628 kg.

moment 628 × .775 = 487
337 × .65 = −219
268 kgm. at S_2



Moment on frame due to own weight.
Load on center panel point.
110 × 2.4 = 264 kg. Reaction = 132 kg
moment at center = 132 × 2.4 = 317 kgm.
Bending moment on top chord.
$= \frac{1}{8} \times 110 \times 2.4^2$ = 80 kgm.
Shear $\frac{1}{2} \times 110 \times 2.4$ = 132 kg.

Summary for Dead Load moments, shears, and reactions

	moment at center	moment on top chord.	Shear	load on center
Stringer Concentration	775	268	628	650
Own weight	317	80	132	264
	1092 kgm	348 kgm	760 kg	914 kg

Chord ~~stress~~ = $\frac{1092}{2.20}$ = 496 kg C or T.

Diagonal ~~stress~~ = $\frac{914}{4}$ × 1.48 = 338 kg C or T for one member.

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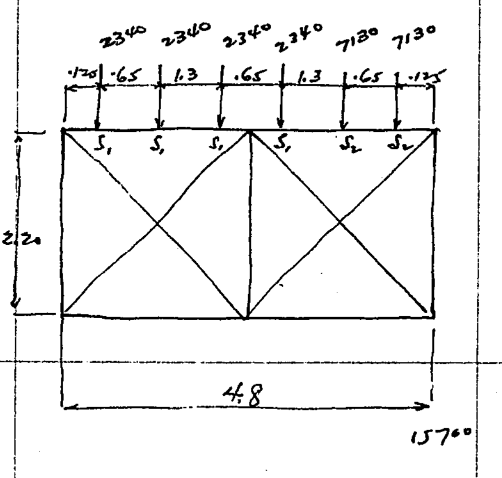
CALCULATIONS FOR

4

工事用仮桟橋計算書

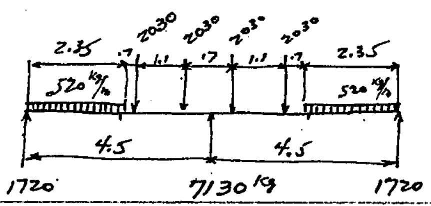
Live Load.



Uniform load on S_1 = 520 kg per lin meter for one rail.
Concentration on S_1 = 2030 kg for one rail.

Stringer concentration S_1 520 × 4.50 = 2340 kg

S_2



2030 × 3.05 = 6190
2030 × 4.15 = 8420
$\frac{520 \times 2.35^2}{2}$ = 1435
16045 ÷ 4.5 × 2 = 7130 kg

Reaction on truss.
2340 × 5.70 = 13350
7130 × 8.70 = 62000
75350 ÷ 4.8 = 15700 kg

Moment at center
15700 × 2.40 = 37700
7130 × 3.90 = − 27800
2340 × .325 = − 790
9110 kgm

Load on center panel pt.
2340 × 2.975 = 6960
2340 × 2.075 = 4860
7130 × 0.90 = 6420
18240 ÷ 2.4 = 7600 kg

Moment on top chord.
2340 × .325 = 790
7130 × 3.90 = 27800
28590 ÷ 2.4 = 11900 kg shear

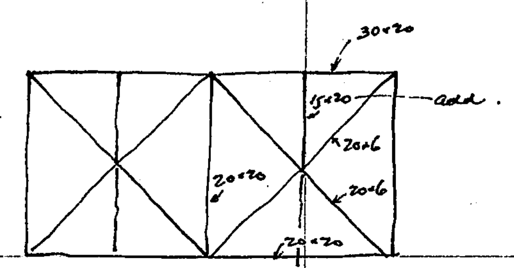
11900 × .775 = 9220
7130 × .65 = − 4630
4590 kgm.

Summary of moments + shears.

	Moment at center	m. on top chord	Shear on top chord.	Load on center panel pt.
Dead Load	1,092	348	760	914
Live Load.	9110	4590	11900	7600
	10,202 kgm	4938 kgm	12,660 kg	8,514 kg

Chord stress = $\frac{10202}{2.2}$ = 4650 kg T or C use 30 × 30 with center vertical col.

Diagonal stress = $\frac{8514}{4}$ × 1.48 = 3150 kg C or T. use 20 × 6



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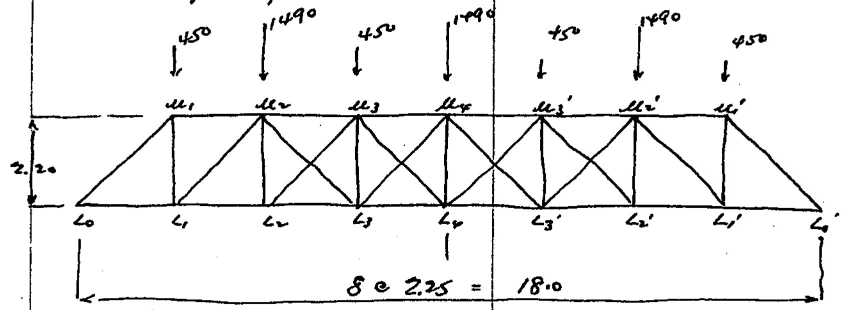
CALCULATIONS FOR

工事用假橋計算書

Design of Main Truss.



3135 kg

Dead Load. Cross frames on u_2, u_4 + u_2' only.

Tracks and cross frame.
panel load. tracks + stringer 906
frame wt. 132
1038 kg

call this 1,040 kg

Truss assumed 200 kg per lin m
panel concentration = 200 × 2.25 = 450 kg u_1, u_3
1490 kg u_2, u_4

Reaction = 450 × 2 = 900
1490 × 1.5 = 2235
3135 kg

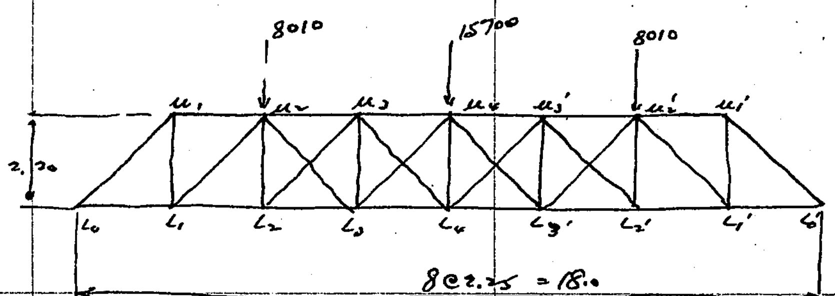
moment at center
3135 × 9.0 = 28220
450 × 2.25 × 4 = − 4050
1490 × 2.25 × 2 = − 6710
17460 kgm

max chord stress = $\frac{17460}{2.2}$ = 7950 kg T or C

Shear = 3135 kg Diagonal length = $\sqrt{2.25^2+2.2^2}$ = 3.15
Coef. = $\frac{3.15}{2.2}$ = 1.43.

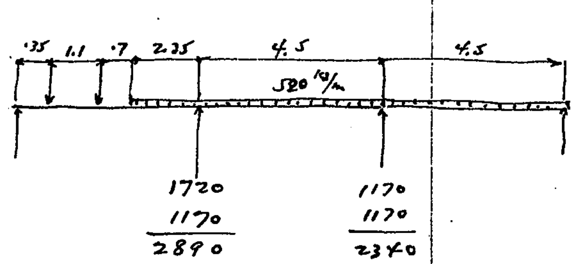
Stresses in diagonals
L_0–u_1 3135 × 1.43 = 4480 kg
L_1–u_2 2685 × 〃 = 3840 〃
L_2–u_3 1195 × 〃 = 1710 〃
L_3–u_4 745 × 〃 = 1070 〃

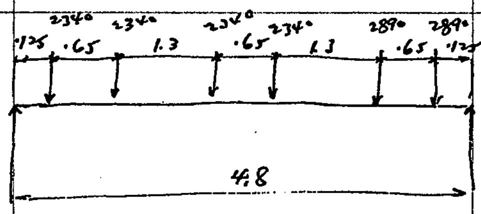
Live Load



15860

max concentration on truss = 15700 kg page 12



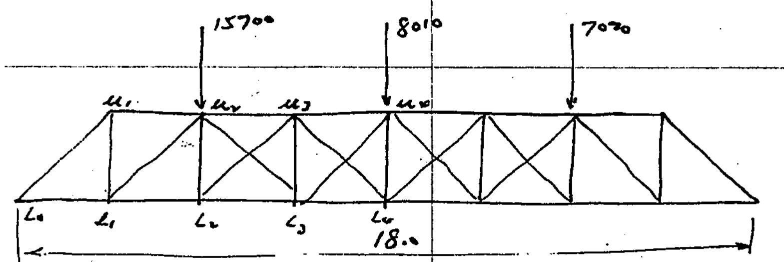


moment at center
15860 × 9.0 = 142700
8010 × 4.5 = − 36000
106700 kgm

max chord stress = $\frac{106700}{2.2}$ = 48500 kg C or T.

Reaction 2340 × 5.70 = 13330
2890 × 8.70 = 25120
38450 ÷ 4.8 = 8010 kg

Panel load for unif load = 2340 × 3 = 7020 kg



~~6020~~
17550

Shear
Reaction 15700 × 2.25 × 6 = + 212,000
8010 × 2.25 × 4 = + 72,000
7020 × 2.25 × 2 = + 31,600
~~108,400~~ 315,600 ÷ 18 = ~~6020~~ 17,550 kg

Stress in L_0–u_1 + L_1–u_2
~~6020~~ 17550 × 1.43 = ~~8610~~ 25100 kg C

15700 ÷ 2 = ~~8850~~ 7850
8010 × $\frac{1}{4}$ = 2000
9850

Stress in L_2–u_3, L_3–u_4
9850 × 1.43 = 14100 kg C.

15700 × $\frac{1}{4}$ = 3930 kg

Stress in counter = 3930 × 1.43 = 5600 kg C

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(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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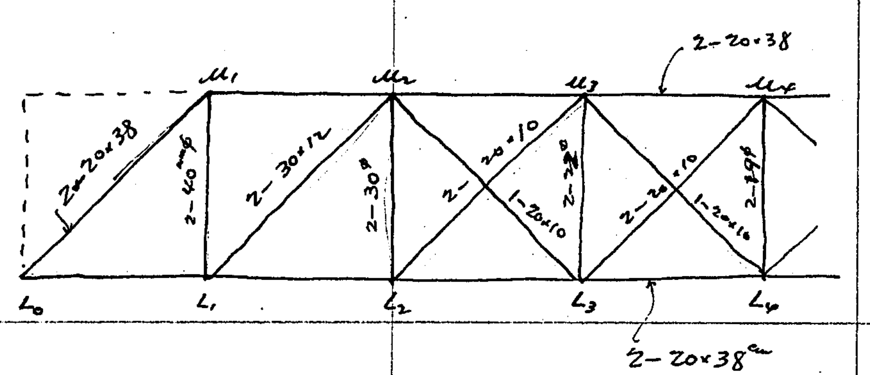
CALCULATIONS FOR

工事用假棧橋計算書

Summary for stresses on truss members.

	Chord.	Diagonal L_0-U_1	L_1-U_2	L_2-U_3	L_3-U_4	counter U_3-L_4
Dead Load	7950	4480	3840	1710	1070	1070
Live Load.	48500	25100	25100	14100	14100	-5600
	56450 kg T or c	29580 kg c	28940 kg c	15810 kg c	15170 kg c	4530 c

	Vertical. U_1-L_1	U_2-L_2
Dead Load	2685	1195
Live Load.	17550	9850
	20235 kg T	11,045 kg T



Lateral bracing
Steel bars.
$L_0 \sim L_2$ 1-30φ
$L_2 - L_4$ 1-22φ

Materials in one span (approximate figure)
米松材

Chords top & bottom	4	—	.20×.38×18.6 =	5.650
Diagonals. L_0-U_1	4	—	.20×.38×2.7 =	.821
〃 L_1-U_2	4	—	.12×.30×2.7 =	.389
〃 L_2-U_3 & L_3-U_4	8	—	.10×.20×2.7 =	.432
〃 counters	4	—	.10×.20×2.7 =	.216
Column on L_0	2	—	.25×.25×1.8 =	.225
chord blocks.	14	—	.30×.70×.55 =	1.615
bearing blocks	2	—	.55×.30×1.0 =	.330
				9.678×2 = 19.356 m³

Cross frame.

Chord. top	1	—	.20×.30×5.5 =	.330
〃 bott.	1	—	.20×.20×5.5 =	.220
posts	1	—	.20×.20×2.0 =	.080
〃	2	—	.15×.20×2.0 =	0.120 ~~.060~~
Diagonals	4	—	.06×.20×3.3 =	.158
				.908×5 = 4.540

23.896 call this 24.0 m³

Approximate weight of steel.

vertical member.	8 —	40mmφ × 2.8	@ 9.86 =	221.
〃	8 —	30 〃 × 2.8	@ 5.54 =	124
〃	~~4~~8 —	22φ × 2.8	@ 2.98 =	67
〃	4 —	19φ × 2.8	@ 2.22 =	25
nuts & washers.	~~56~~ 28 sets	@	= 12.0 =	336
bottom lateral	4 —	30φ × 7.0	@ 5.54 =	155
〃	4 —	22φ × 7.0	@ 2.98 =	83
nuts & washers	8 sets	@ 3	=	24
frame, plates	10 Pls	75×9	@ 5.30 × 1.3 =	69
Splice pls.	4 sets	@	40 =	160
bolts say				600
				1,864
錆止め say				36
				1,900 kg for one span.

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CALCULATIONS FOR

工事用假棧橋設計

Tracks.

12# rails with accessories 3^{lines}@ 18.65 = 55.95 meters

Sleepers	140 @ .1 × .1 × 1.2	=	1.68
Stringers	4 @ .15 × .30 × 18.6	=	3.34
Stringers	2 @ .25 × .40 × 18.6	=	2.72
Struts	5 @ .15 × .15 × 4.0	=	.45
			$8.19\ m^3$

Planking 3 × .5 × 18.65 = $28.0\ m^2$

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CALCULATIONS FOR

工事用假棧橋計算書

Estimate of Cost for Piers.
Piers on land and in shallow water (A)

米松	8	.30 × .25 × 1.10 =	.660	支承台
〃	6	.30 × .30 × 2.10 =	1.135	支承桁
〃	2	.30 × .25 × 6.20 =	.930	梁木
〃	4	.20 × .15 × 1.80 =	.216	布木
〃	4	.20 × .06 × 1.30 =	.062	〃
〃	4	.20 × .06 × 2.50 =	.120	筋違
〃	4	〃 〃 〃 =	.120	〃
〃	4	.20 × .10 × 3.0 =	.240	〃
〃	1	.20 × .10 × 2.0 =	.040	継木
〃	2	.25 × .20 × 7.20 =	.720	梁木下部
〃	4	.20 × .15 × 1.80 =	.216	布木
〃	8	.20 × .06 × 2.00 =	.192	挟楞
			4.651 m³	
杉丸太柱	12	末口 18cmφ × 2.40		
松丸太杭	12	〃 × 5.50		
手鋏	16	75×9 × 0.70	60	
鋲	32	6" 鋲	15	
ボールト		250 kg.	250	
			325 kg	

Cost of one pier.

米松材	4.651 m³	@	50.0 =	232.5 円	仕拵組立共
杉丸太柱	12 本	@	5.0 =	60.0	〃 〃
松杭	12 〃	@	10.0 =	120.0	〃 〃 打込共
金物	0.325 kg ton	@	200.0 =	65.0	〃 取付共
根掘	6.0 m³	@	1.0 =	6.0	
				483.5	
取外片付				56.5	
				540.0	
塵芥除 仕拵建込杭打及撤去費 杭ニ支建布及筋違付ケ 2 @ 90 =				180.0	
				720.0 円	

(B) 構造全部同一ニシテ 杭長 7.2m トス

杭差額 18 @ 5 =	90	
水中仕掛	40	
		130
		720
塵芥除共.		850.0円

Piers in deep water. (C)

米松		4.80 m³ abt.	@	50.0 =	240.0
杉丸太柱		12 本	@	5.0 =	60.0
松杭	末口 21cm × 11.0m	18 本	@	35.0 =	630.0
					930.0
撤去					100.0
					1030.0
塵芥除 杭 28φ × 9m ニ鍵 杭方撤去 ニ基		2 @ 135		=	270.0
					1300.0 円

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CALCULATIONS FOR

工事用假桟橋計算書 ニ之

60 piers required.
Total cost. 塵芥除去

Piers A.	42 piers	@	720.00	=	30,240	
〃 B	8	@	850.00	=	6,800	
〃 C	10	@	1300.00	=	13,000	
	60				50,040	円
両詰取付	2	@	80	=	160	
					50,200	円

古材處分收入金

Piers A	42	@	200.00	=	8,400	
〃 B	8	@	250.00	=	2,000	
〃 C	10	@	450.00	=	4,500	
					14,900	円
			差引損料		35,300	円