JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

MADE BY ____ DATE ____ FILE NO ____
CHECKED BY ____ DATE ____ PAGE NO ____

昭和五年七月

三重縣國道弍號路線

揖斐長良川橋基礎工事

設計々算書及材料調書

JIUN MASUDA
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MADE BY J. Masuda DATE 5-6-21 FILE NO Sub.
CHECKED BY M. Jinda DATE 5-6-30 PAGE NO 3

CALCULATIONS FOR

Design of Hi-Nagara-Gawa Bashi for Mie Ken

Superimposed load
Dead Load Roadway

Asphalt block pavement 5cm thick @ 21 = 105
2cm Sand mortar cushion @ 17 = 34
15.5 cm reinforced concrete slab = 372
misc filler etc say 9
520 kg per sq m

For 7.5 meter wide roadway 520 × 7.5 = 3900
Coping 2 @ 173 = 346
Handrails 2 @ 80 = 160
4406 kg per lin. meter

For ½ width 4406 ÷ 2 = 2203 kg/m

Load on shoe 2203 × $\frac{73.76}{2}$ = 81,500
" " structural steel 246000 ÷ 4 = 61500
143,000 kg.

Live Load
Uniform live load $w = \frac{100.000}{170+72.80}$ = say 412 kg/m²

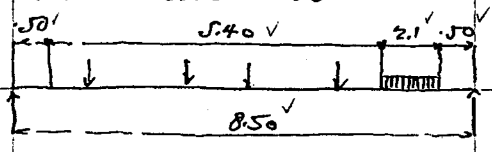
For 7.5 meter wide 412 × 7.5 = 3090 kg per lin. meter

Impact for motor truck Impact = $\frac{20}{60+72.80}$ = say 15.1%

motor truck rear wheel 3000
impact 15.1% 453
3453 × 2 = 6906 kg.

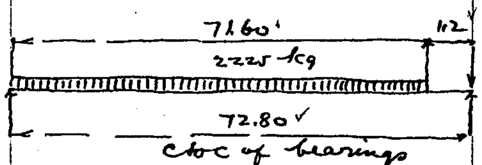
For 2 motor trucks 2 × 6906 = 13810 kg.

Reaction on truss



412 × 2.1 = 865 kg/m 3090 − 865 = 2225 kg/m
uniform load 865 × $\frac{4.55}{8.5}$ = 158 kg
" " 2225 × $\frac{5.3}{8.5}$ = 1390

motor truck rear wheel 13810 × $\frac{5.3}{8.5}$ = 8610 kg.



Load on abutment
uniform load $\frac{2225 \times 71.60^2}{2 \times 72.80}$ = 78500
" 865 × $\frac{72.80}{2}$ = 31600

motor truck rear wheel concentration 13810
123910 call this 124000 kg

max load on one shoe.
uniform load 1390 × $\frac{71.60^2}{2 \times 72.80}$ = 48800
unif. load 158 × $\frac{72.80}{2}$ = 5750
motor truck rear wheels 8610
63160 kg. call this 63200 kg

Summary load on abutment and on one shoe

	On abutment	On one shoe
Dead Load	286000	143000
Live Load	124000	63200
	410000 kg	206200 kg.

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MADE BY K. Inaba DATE 5-6-23 FILE NO Sub
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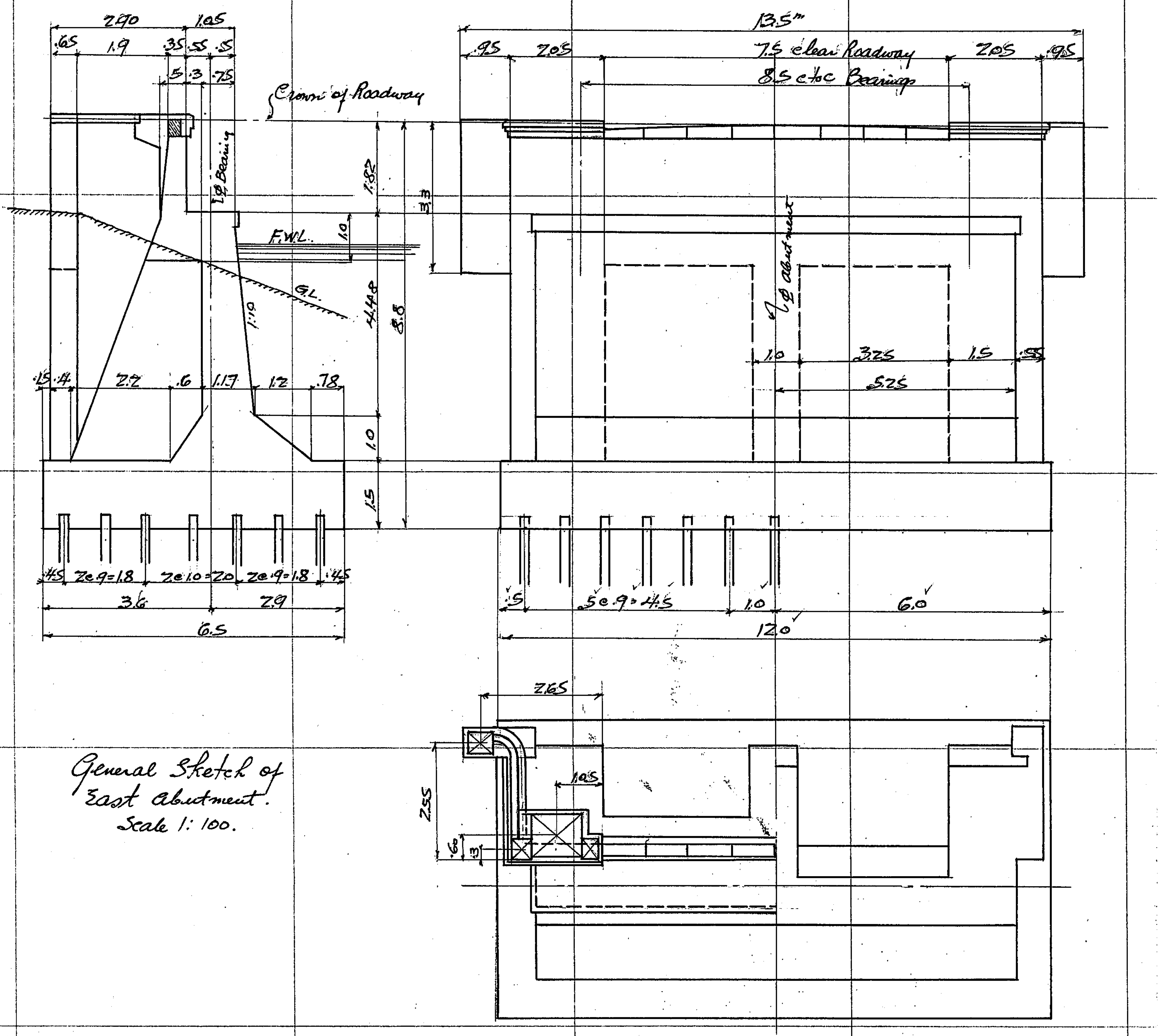
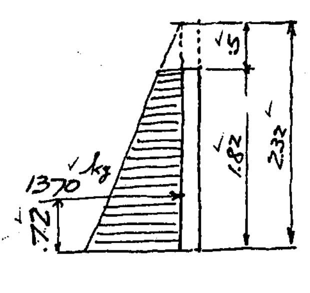
CALCULATIONS FOR

Design of Ibi Nagara Bashi for Mie ken
Design of East Abutment. 長島寄
General dimensions are as shown on sketch below.



General Sketch of East Abutment.
Scale 1: 100.

Design of Parapet wall



Earth pressure at normal state

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

$\frac{1}{3} \cdot 1600 \cdot 2.32 = \frac{1235}{1502} \div 2 = 751$ kg/m² average.

$751 \cdot 1.82 = 1370$ kg per meter strip of wall.

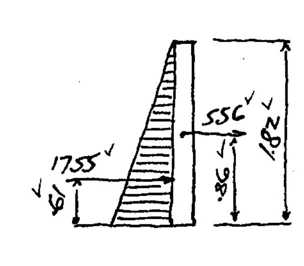
Moment at bottom of wall

$1370 \cdot .72 = 985$ kgm.

Earth pressure during earthquake. k assumed 0.30

$1600 \cdot .662 \cdot \frac{1.82^2}{2} = 1755$ kg

weight of wall. $.425 \cdot 1.82 \cdot 2400 = 1855$ $\quad 1855 \cdot .3 = 556$ kg



moment at bottom of wall.

$1755 \cdot .61 = 1070$

$556 \cdot .86 = \frac{480}{1550}$ kgm per meter strip of wall. $\quad \frac{1550}{1.8} = 860 < 985$

Moment at normal state governs the section of wall.

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CALCULATIONS FOR

Design of Ibi nagara Bashi for Mie Ken.

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

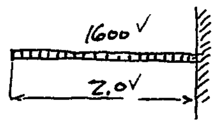
$d = \sqrt{\frac{M}{bR}}$ where $R = 7.18$, $b = 100$ cm

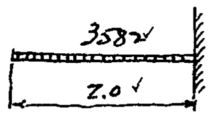
$d = \sqrt{\frac{985 \times 100}{100 \times 7.18}} = 11.7$ cm use 47 cm effective depth with 3 cm insulation

Steel area required $= \frac{985 \times 100}{1200 \times \frac{7}{8} \times 47} = 2.00$ cm² per lin meter.

Use 12 mm φ bars at 30 cm c to c = 3.77 cm² per lin meter · on rear side
12 " " " 60 " = 1.89 " " 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/m²
moment on wall $= 1600 \times \frac{2^2}{2} = 3200$ kgm. Shear $1600 \times 2 = 3200$ kg

Earth pressure during Earthquake $= 1600 \times .662 \times 3.0 = 3180$
Seismic force on wall $.35 \times 2400 \times .3 = 252$
Handrail + post say $1500 \div 3 = 500$ $500 \times 0.30 = 150$
3582 kg per lin m.

moment on wall $= 3582 \times 2.0^2 \div 2 = 7160$ kgm. Shear $= 7160$ kg.
Latter moment and shear governs

Effective depth required $= \sqrt{\frac{7160 \times 100}{100 \times 7.18 \times 1.8}} = \frac{23.5}{\cancel{31.6}}$ cm

Use 32 cm effective depth with 3 cm insulation or 35 cm in total.
Steel area required $= \frac{716000}{1200 \times 1.8 \times \frac{7}{8} \times 32} = 11.9$ cm² per meter strip

Use 19 mm φ bars 24 cm c to c = 11.9
unit shear $= \frac{7160}{100 \times 7/8 \times 32} = 2.56$ kg/cm² ok.

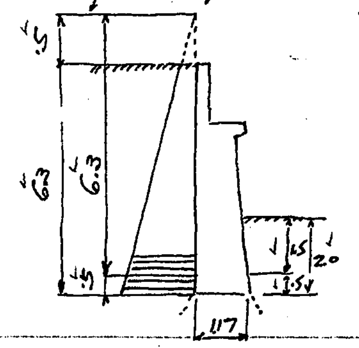
Unit bond $= \frac{7160}{5.97 \times \frac{100}{24} \times \frac{7}{8} \times 32} = 10.28$ " $< 6.0 \times 1.8 = 10.8$ ok.

Section at 1.5 meter below top of wall.
moment due to earth pressure and seismic forces.
$3180 \div 2 = 1590$
252
150
$1992 \times \frac{2^2}{2} = 3984$ kgm.

Steel area required $= \frac{398400}{2160 \times \frac{7}{8} \times 32} = 6.58$ cm² per meter strip.

Use 16 mm φ bars at 25 cm c to c = 8.04 cm² .

Design of Curtain wall. Span length assumed at 4.0 meters.



Earth pressure at normal state $= \frac{1}{3} \times 1600 \times 6.3 = 3360$ kg/m² average.
earth pressure front side $\frac{1}{3} \times 1600 \times 1.5 = -800$
2560

Earth pressure during earthquake $= .662 \times 1600 \times 5.8 = 6150$
Seismic force $1.12 \times 2400 \times .3 = 800$
6950 kg/m² average.

moment on wall $= \frac{6950 \times 4^2}{10} = 11120$ kgm.
Effective depth required $= \sqrt{\frac{11120}{7.18 \times 1.8}} = 29.3$ cm use 117 cm total depth.

Steel area required $= \frac{1112000 \times 100}{2160 \times \frac{7}{8} \times 112} = 5.25$ cm² per meter strip.

Use 16 φ bars at 35 cm c to c = 5.75 .
Shear $= 6950 \times 2.0 = 13900$ kg
Unit shear $= \frac{13900}{100 \times \frac{7}{8} \times 112} = 1.42$ kg/cm² ok.

Unit bond $= \frac{13900}{5.03 \times 2.86 \times \frac{7}{8} \times 112} = 9.85 < 6 \times 1.8 = 10.8$ ok

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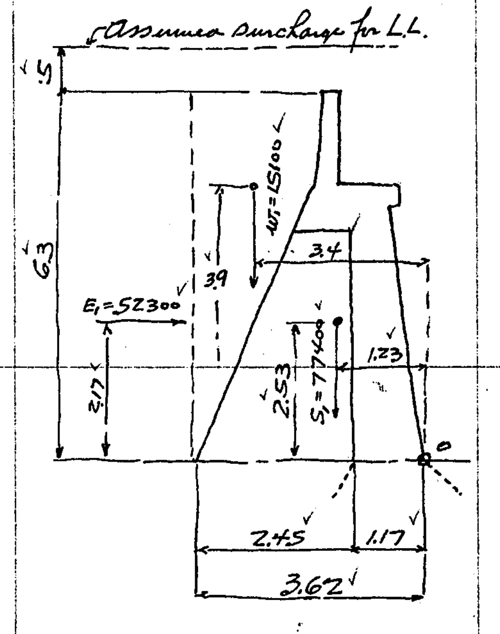
CALCULATIONS FOR

Design of Ibi Nagara Bashi for Mie Ken.

Design of Counterfort at center.

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



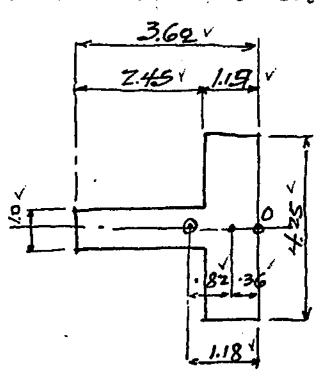
			arm. vert m.		arm hor. m.	
Granite	.25·.3·4.25 = .318 @ 2600 =	830	6.15	5100	1.72	1427
parapet wall	4.25·1.55·.425 = 2.800 @ 2400 =	6720	5.22	35100	1.90	12765
Top beam	1.80·1.0·4.25 = 7.650 @ " =	18350	3.93	72100	1.32	24200
Coping	.1·.3·4.25 = .130 @ " =	310	4.33	1342	.37	115
Curtain wall	1.0·3.48·4.25 = 14.800 @ " =	35500	1.64	58200	.66	23430
Counterfort.	1.875·3.48·1.00 = 6.522 @ " =	15650	1.54	24100	2.11	33020
Sum of concrete	= 31.902 m³	77360 kg	2.53 m	195942	1.23 m	94957
" granite	= .318	call this 77400				

Weight of Earth on counterfort wall.
1.5·1.0·6.3 = 9.45·1600 = 15100 kg

Earth pressure at normal state $\frac{1}{3}\cdot1600\cdot6.8 = 3637$
$\frac{1}{3}\cdot1600\cdot.5 = \underline{267}$
$3904 \div 2 = 1952$ kg/m² average.

Earth pressure $E_1 = 1952\cdot6.3\cdot4.25 = 52300$ kg
Earth pressure on front side neglected.
Earth pressure during Earthquake $.662\cdot\frac{6.3^2}{2}\cdot4.25\cdot1600 = 89300$ kg $= E_1'$

Case 1. Stresses at normal state.



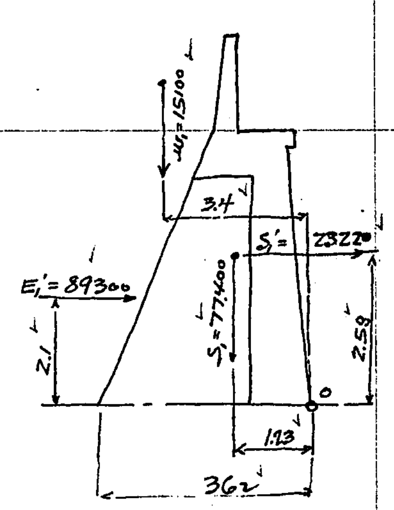
Center of gravity of section
$1.17\cdot4.25 = 4.97\cdot.585 = 2.91$
$1.0\cdot2.45 = 2.45\cdot2.395 = 5.87$
$7.42 \quad 1.18\text{m} \quad 8.78$

Taking moment about point O

Loads	Hor. forces	Vert. forces	Lev. arm	Moments
S_1		77400	1.23 =	95200
w_1		15100	3.40 =	51300
E_1	52300		2.17 =	−113500
	52300	92500	.36 m =	33000

Eccentricity = 1.18 − .36 = .82 m
moment at bottom section = 92500 · .82 = 75800 kgm
Shear " " " = 52300 kg

Case 2. Stresses during Earthquake. $k = 0.300$



Taking moment about point O.

Loads	Hor. forces	Vert. forces	lev. arm	moment.
S_1		77400	1.23 =	+95200
S_1'	23220		2.53 =	−58750
w_1		15100	3.40 =	+51300
E_1'	89300		2.10 =	−187500
	112520 kg	92500 kg	−1.08 m =	99750

Eccentricity $\varepsilon = 1.18 + 1.08 = 2.26$ m
moment at bottom section = 92500 · 2.26 = 209000 kgm.
Shear " " = 112520 kg.
Steel area required for moment $= \frac{209000\times100}{2160\cdot\frac{7}{8}\cdot352} = 31.4$ cm²

Use 9-22 mm∅ bars = 34.2 cm²

$p = \frac{34.2}{425\cdot352} = 0.00023$

$t/d = \frac{1.17}{352} = 0.33$

neutral axis in the flange.

$k = \sqrt{2\times15\times.00023 + (.00023\times15)^2} - .00023\times15$
$= 0.08$

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

$f_s = \frac{209000\times100}{34.2\cdot\frac{7}{8}\cdot352} = 1985$

Direct comp. $= \frac{92500\times15}{742\times10000} = \frac{-19}{1966}$ kg/cm² $< 1200\times1.8 = 2160$ ok.

$f_c = \frac{f_s k}{n(1-k)} = \frac{1985\times.08}{15\times0.92} = 11.50$

Direct comp. $\frac{92500}{742\times10000} = \frac{1.30}{12.80}$ kg/cm² ok.

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CALCULATIONS FOR

Design of Ibi Nagara Bashi for Mie Ken.

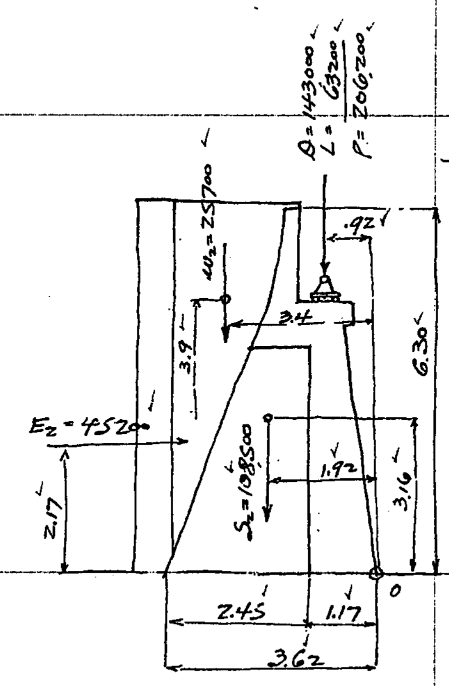
Unit shear = $\frac{112520}{100 \times .973 \times 352} = 3.3$ kg/cm² ok.

Unit bond = $\frac{112520}{691 \times 9 \times .973 \times 352} = 5.3$ " ok.

Design of Counterfort under truss bearing.

Superimposed Load on abutment.

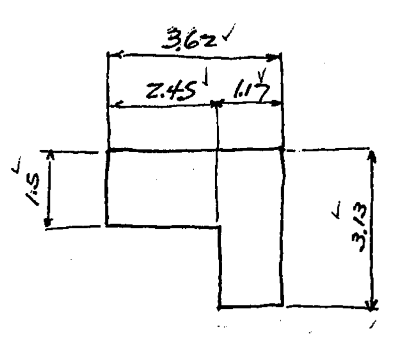
Dead Load	143000
Live Load	63200
	206200 kg on one shoe.



Weight and Center of gravity of Counterfort under truss bearing

				arm vert m.		arm hor m.	
Granite parapet	.25 × .3 × 1.63 =	.122 @ 2600 =	320	6.15	1968	1.72	550
Light pedestal. say		2.500 @ " =	6500	~~5.22~~ 7.60	49350	~~1.90~~ 2.07	13450
posts (3)	3 × .6 × .6 × 1.05 =	1.135 @ " =	2950	6.90	20350	2.52	7440
handrail	2.5 × .2 × 1.0 =	.50 @ " =	1300	6.88	8940	3.57	4640
parapet wall	.425 × 1.52 × 1.63 =	1.053 @ 2400 =	2525	5.22	13175	1.90	4795
top beam	1.8 × 1.0 × 3.13 =	5.63 @ " =	13520	3.93	53080	1.32	17830
Coping front	.1 × .3 × 3.13 =	.094 @ " =	226	4.33	980	.37	85
" end	.1 × .3 × 1.10 =	.033 @ " =	79	4.33	342	.92	73
projection	.65 × ~~1.6~~ 1.25 × 3.3 =	2.680 @ " =	6430	4.73	30400	4.02	25850
" inside	.30 × .65 × 3.00 =	.585 @ " =	1405	1.54	2165	4.02	5650
wing wall.	.35 × 2.90 × 6.38 =	6.475 @ " =	15540	3.19	49550	2.92	45380
"	.425 × 1.9 × 1.70 =	1.372 @ " =	3290	5.43	17870	1.89	6210
Coping	.1 × .3 × 4.45 =	.134 @ " =	322	6.23	2008	2.02	650
projection top.	.625 × .75 × 1.20 =	.562 @ " =	1350	6.07	8200	2.13	2875
front wall.	1.0 × 3.48 × 3.13 =	10.900 @ " =	26150	1.64	42880	.66	17250
Counterfort	1.875 × 3.48 × 1.70 =	11.100 @ " =	26630	~~1.64~~ 1.55	41280	~~.66~~ 2.09	55650
Sum of concrete =		40.618 m³	108537	3.16	342538	1.92	~~185013~~ 208378
Sum of granite =		4.257 call this 108500 kg					

Assumed bottom section



Weight of Earth on Counterfort wall.

1.5 × 1.7 × 6.3 @ 1600 = 25700 kg = w_2

Earth pressure at normal state E_2 = 1952 × 6.3 × 3.68 = 45200 kg

Earth pressure during Earthquake E_2' = 89300 × $\frac{3.68}{4.25}$ = 77400 kg

Center of gravity of section

1.17 × 3.13 =	3.66 ×	.585 =	2.14
1.5 × 2.45 =	3.67 ×	2.395 =	8.79
	7.33 m²	1.49	10.93

Case 1. Stresses at normal state.

Taking moment at point 0.

Loads	Hor. forces	Vert. forces	Lev. arm	Moment
P		206200 ×	.92 =	189800
S_2		108500 ×	1.92 =	208500
w_2		25700 ×	3.4 =	87400
E_2	45200	——— ×	2.17 =	− 98100
	45200	340400	1.14 m	387600

Eccentricity = −1.14 + 1.49 = 0.35 m

moment at bottom section = 340400 × .35 = 119200 kgm

shear " " " = 45200 kg

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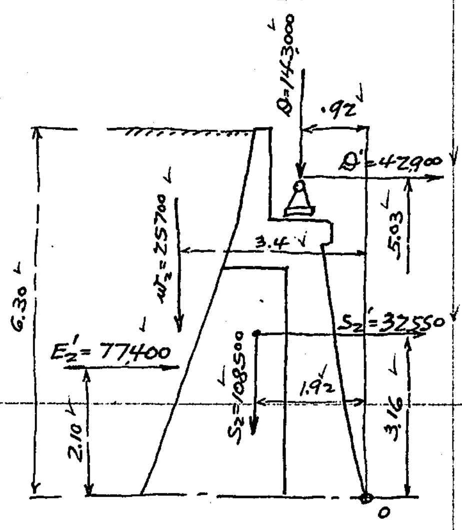
CALCULATIONS FOR

Design of Ibi Nagara Bashi for Mie Ken.

Case 2. Stresses during Earthquake k assumed 0.300 (Seismic forces forward).



Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lev. arm	Moment
D		143,000	× − 0.92 =	− 131,500
D'	42,900		× + 5.03 =	+ 216,000
S_2		108,500	× − 1.92 =	− 208,400
S_2'	32,550		× + 3.16 =	+ 102,900
w_2		25,700	× ∓ 3.40 =	− 87,400
E_2'	77,400		× + 2.10 =	+ 162,500
	152,850	277,200	0.195ᵐ	+ 54,100

Eccentricity = 1.49 + .20 = 1.69ᵐ

moment at bottom section = 277200 × 1.69 = 468,500 kgm

Shear " " = 152,850 kg

Steel area required for moment = $\frac{468500 \times 100}{2160 \times \frac{7}{8} \times 352}$ = 70.4 cm²

Use 14 − 25ᵐᵐφ bars = 68.7 cm²

$f_s = \frac{468500 \times 100}{68.7 \times \frac{7}{8} \times 352}$ = 2210

Direct comp. = $\frac{277200 \times 15}{733 \times 10000}$ = −57 / 2153 kg/cm² < 1200 × 1.8 = 2160 ok.

$p = \frac{68.7}{313 \times 352}$ = .00063

$t/d = \frac{117}{352}$ = .33

neutral axis in the flange.

pn = .00063 × 15 = .00945

$(pn)^2$ = .00009

$k = \sqrt{2 \times .00945 - .00009} - .00945$ = .128

$f_c = \frac{2210 \times .128}{15 \times .872}$ = 21.6

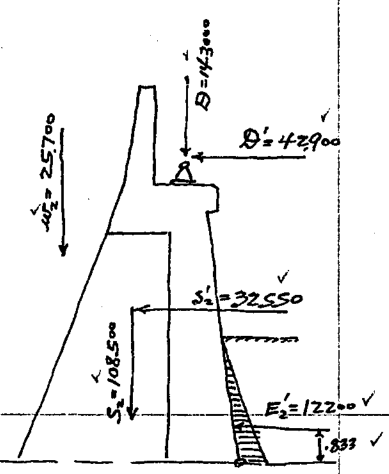
Direct comp. = $\frac{277200}{733 \times 10000}$ = +3.8 / 25.4 kg/cm² ok.

Unit shear = $\frac{152850}{150 \times \frac{7}{8} \times 352}$ = 3.3 ok.

Unit bond = $\frac{152850}{7.85 \times 14 \times \frac{7}{8} \times 352}$ = 4.5 ok.

Case 3 Stresses during Earthquake (Seismic forces backward).

Earth pressure on front side $E_2' = .662 \times \frac{2.5^2}{2} \times 1600 \times .368 = 12200$ kg.



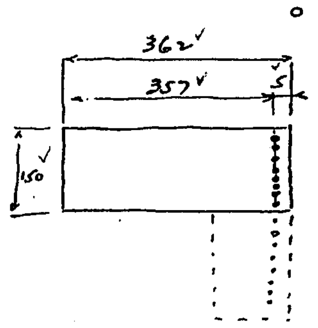
Taking moment about point O.

Load	Hor. forces	Vert. forces	Lev. arm	moment
D		143,000	× − 0.92 =	− 131,500
D'	42,900		× ∓ 5.03 =	− 216,000
S_2		108,500	× − 1.92 =	− 208,400
S_2'	32,550		× − 3.16 =	− 102,900
w_2		25,700	× − 3.40 =	− 87,400
E_2'	12,200		× − 0.833 =	− 10,200
	87,650	277,200	2.73ᵐ	− 756,400

Eccentricity = 2.73 − 1.49 = 1.24ᵐ

moment at bottom section = 277200 × 1.24 = 344,000 kgm

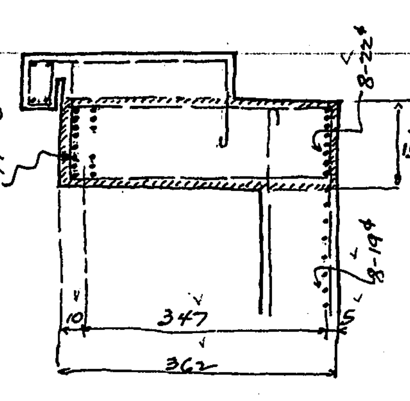
Shear " " = 87,650 kg



Steel area required for moment = $\frac{344000 \times 100}{2160 \times \frac{7}{8} \times 357}$ = 51.0 cm²

Use 8 − 22ᵐᵐφ = 30.4
8 − 19 = 22.7
53.1 cm²

$p = \frac{53.1}{150 \times 357}$ = 0.001, k = .158, j = .947



$f_s = \frac{344000 \times 100}{53.1 \times .947 \times 357}$ = ~~1880~~ 1915

Direct comp = $\frac{277200 \times 15}{733 \times 10000}$ = −56 / 1859 kg/cm² ok.

$f_c = \frac{1915 \times .158}{15 \times .842}$ = 24.0

Direct comp = $\frac{277200}{733 \times 10000}$ = 3.7 / 27.7 kg/cm² ok.

Unit shear & bond ok.

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CALCULATIONS FOR

Design of Ibi Nagara Bashi for Mie ken.

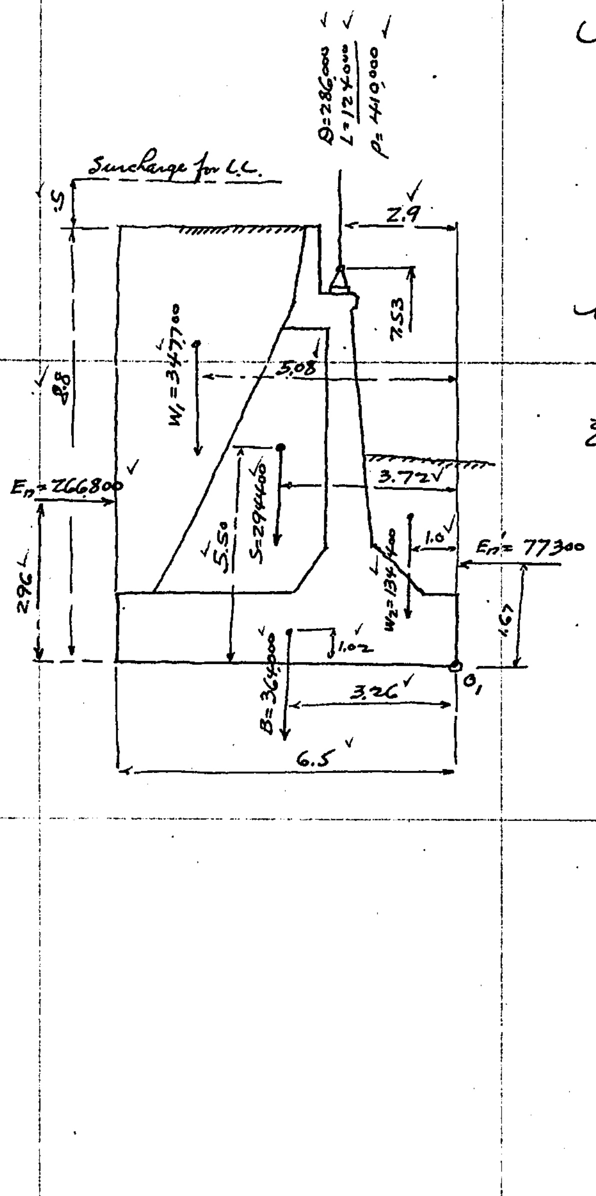
Stability of Abutment as a whole.

Superimposed Loads on abutments

Dead Load	286000
Live Load	124000
	410,000 kg. for one abutment.

Weight and center of gravity of shaft.

weight	Vert. lev. arm	moment	Hor. lev. arm	moment
77400	2.53	195800	1.23	95200
108500	3.16	343000	1.92	208400
108500	3.16	343000	1.92	208400
294400 kg	3.00 m	881800	1.74 m	512000
	2.50		1.98	
	5.50 from toe		3.72 from toe	



Weight of earth fill on rear footing

$2.50 \times 7.30 \times 11.30$ @ $1600 = 330000 \times 5.25 = 1732000$

$0.85 \times 4.48 \times 7.50$ @ $1600 = 45700 \times 3.58 = 163500$

$1.95 \times 2.24 \times 4.0$ @ $1600 = -28000 \times 4.65 = -130200$

347700 kg, 5.08 m from toe. 1765300

Weight of earth fill on front footing Depth assumed 3.5 m

$3.5 \times 2 \times 12$ @ $1600 = 134400$ kg arm 1.0 m from toe

Earth pressure on rear.

normal state $\frac{1}{3} \times 1600 \times .5 = 267$ Surcharge for LL. = 0.5 m assumed.

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

$5222 \div 2 = 2611$ kg/m² average.

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

during earthquake $E_e = .662 \times 1600 \times 8.8^2 \div 2 \times 11.6 = 476000$ kg arm 2.93 m

Earth pressure on front

normal state $E_n' = \frac{1}{3} \times 1600 \times \frac{5^2}{2} \times 11.6 = 77300$ kg arm 1.67 m

during earthquake $E_e' = .662 \times 1600 \times \frac{5^2}{2} \times 11.6 = 153700$ kg " "

Weight and center of gravity of Base.

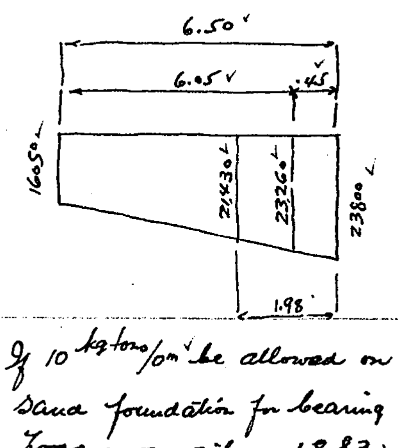
			vert arm	m	hor arm	m
$2.07 \times 1.0 \times 10.5 = 21.75$ @ 2400	=	52200	1.93	100800	2.41	125800
$4.0 \times 1.0 \times 2.3 = 9.20$ @ "	=	22100	2.00	44200	4.80	106000
$2.9 \times .55 \times 2.0 = 3.19$ @ "	=	7700	2.00	15400	4.90	37700
$6.5 \times 1.5 \times 12.0 = 117.00$ @ "	=	282000	0.75	210800	3.25	917000
151.14 m³		364000 kg	1.02 m	371200	3.26 m	1186500

Case 1. Stability at normal state.

See above sketch

Taking moment about point O_1, = toe.

Loads	Hor. forces	Vert. forces		Lev. arm		moment
P		410,000	×	2.90	=	1,189,000
S		294400	×	3.72	=	1095000
W_1		347700	×	5.08	=	1766000
W_2		134400	×	1.00	=	134400
B		364000	×	3.26	=	1,186000
E_n	−266800		×	2.96	=	−790000
E_n'	77300		×	1.67	=	129000
	−189500 kg	1550500 kg	.	3.04 m	=	4709400



If 10 kg tons/m² be allowed on sand foundation for bearing load on one pile 18.83
less $.9 \times .9 \times 10 = -8.10$
10.73 kg tons

Eccentricity $= 3.25 - 3.04 = 0.21$ m

Resultant force within middle third .194

max. toe pressure $= \frac{1550500}{6.5 \times 12.0}\left(1 \pm \frac{6 \times .21}{6.5}\right) = 23800$ kg/m² (or 2.18 tons/m²)
or 16,050 "

max. load on one pile $= .9 \times .9 \times 23260 = 18.83$ kg tons.

Average load on one pile $= \frac{15505}{91} = 17.1$

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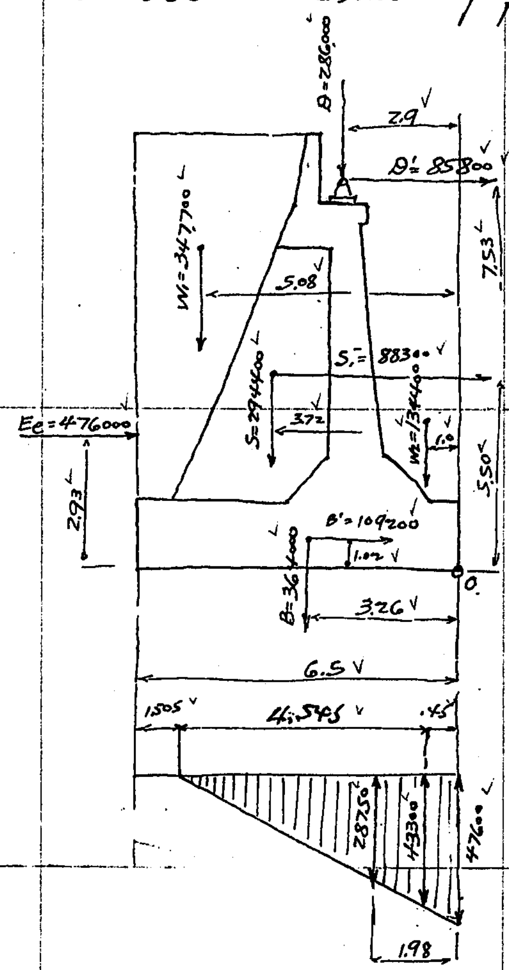
CALCULATIONS FOR

Design of Ibi nagara Bashi for Mie Ken.

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



Taking moment about toe O.

Loads	Hor. forces	Vert. forces	Lev. arm.	moments.
D		286,000 ×	2.90 = +	830,000
D'	85,800	×	7.53 = −	646,000
S		294,400 ×	3.72 = +	1,095,000
S'	88,300	×	5.50 = −	485,000
B		364,000 ×	3.26 = +	1,187,000
B'	109,200	×	1.02 = −	111,000
W_1		347,700 ×	5.08 = +	1,765,000
W_2		134,400 ×	1.00 = +	134,400
Ee	476,000	×	2.93 = −	1,395,000
	759,300	1,426,500	1.665 m +	2,374,400

Eccentricity = 3.25 − 1.665 = 1.585 m

Resultant force outside of middle third, neglecting tension

pressure area = 1.665 × 3 × 12 = 59.9 m²

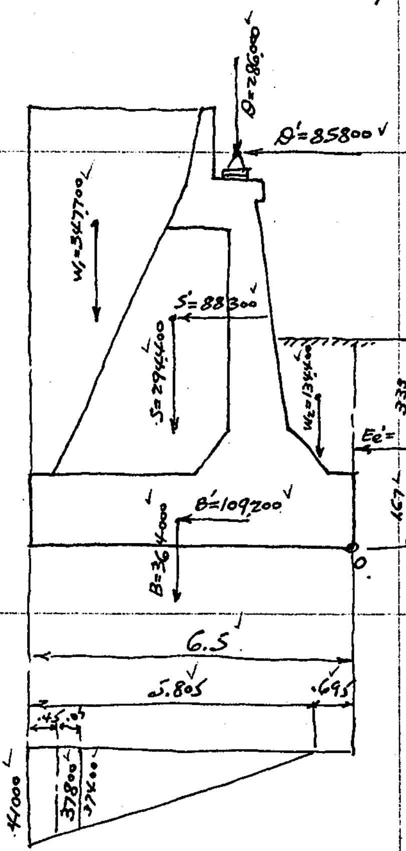
max. toe pressure = $\frac{1,426,500 \times 2}{59.9}$ = 47600 kg/m² (or 4.36 tons/□')

max. load on one pile = 43300 × .9 × .9 = 35.0 kg tons.

If 10.0 × 1.8 = 18. tons/□m be allowed for bearing pressure on sand foundation

max. load on one pile 35.0
less. 18.0 × .9 × .9 = −14.6
20.4 kg tons.

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



Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lev. arm	moment.
D		286,000 ×	2.90 =	830,000
D'	85800	×	7.53 =	646,000
S		294400 ×	3.72 =	1095,000
S'	88300	×	5.50 =	485,000
B		364000 ×	3.26 =	1187,000
B'	109200	×	1.02 =	111,000
W_1		347700 ×	5.08 =	1765,000
W_2		134400 ×	1.00 =	134400
E_e'	153700	×	1.67 =	256,500
	437000	1,426500	4.565 m	6,509,900

Eccentricity = 4.565 − 3.25 = 1.315 m

Resultant force outside of middle third neglecting tension.

pressure area = (−4.565 + 6.5) = 1.935 × 3 × 12.0 = 69.6 □m

max. toe pressure = $\frac{1426,500 \times 2}{69.6}$ = 41,000 kg/m² or (3.75 tons/□').

max. load on one pile = 37800 × .9 × .9 = 30.6 kg tons.

If 10.0 × 1.8 tons/□m be allowed for bearing on sand foundation

max. net load on one pile 30.6
less. 18.0 × .9 × .9 = −14.6
16.0 kg tons.

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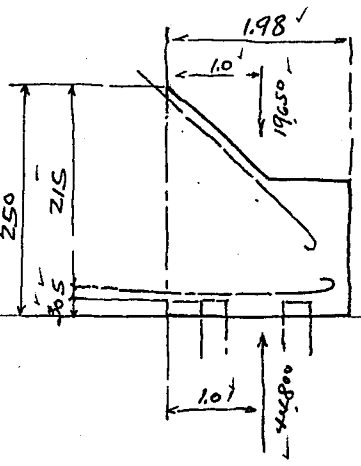
CALCULATIONS FOR

Design of Ibi Nagara Bashi for Mie ken.

Design of footing at toe.



Loads on Cantilever footing at normal state. (Case 1)

upward pressure
21430
23800
45230 ÷ 2 × 1.98 = 44800 kg

weight of earth fill on footing 3.5 × 1600 × 1.98 = − 11,100
" " concrete footing say 1.8 × 2400 × 1.98 = − 8550 } 19650
25,150 kg

moment on footing
44800 × 1.01 = 45250
19650 × 1.00 = 19650
25,600 kgm per meter strip

Shear = 25,150 kg

During Earthquake (Case 2.)

upward pressure
28750
47600
76350 ÷ 2 × 1.98 = 75600 kg arm 1.07

moment on footing
75600 × 1.07 = 80900
19650 × 1.00 = −19650
61250 kgm per meter strip.

Shear 75600 − 19650 = 55950 kg

Case 2 governs the section.

Effective depth required $= \sqrt{\frac{61250 \times 100}{100 \times 1.8 \times 7.18}} = 69.8$ cm

Use eff. depth of 215 cm with 5 cm insulation.

Steel area required $= \frac{61250 \times 100}{2160 \times \frac{7}{8} \times 215} = 15.1$ cm²

Use 25 mmφ bars at 30 cm ctoc = 16.35 cm²

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

unit shear $= \frac{55950}{100 \times .951 \times 215} = 2.74$ kg/cm² ok.

unit bond $= \frac{55950}{7.85 \times 3.333 \times .951 \times 215} = 10.43$ kg/cm² ok < 6.0 × 1.8 = 10.8

Top reinforcement.

negative moment on footing = 19650 kgm

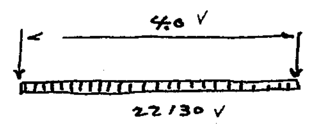
Steel area required $= \frac{19650 \times 100}{2160 \times \frac{7}{8} \times 170} = 6.12$ cm²

use 16 mmφ bars at 30 cm ctoc = 6.71 cm²

Design of footing at heel.

upward pressure during Earthquake. (Case 3) 37400 kg/m²

downward pressure due to earth fill = 7.3 × 1600 = 11670
" " footing = 1.5 × 2400 = 3600
− 15270
22,130 kg/m²



Span length assumed 4.0 meters.

moment $= \frac{1}{10} \times 22130 \times 4.0^2 = 35400$ kgm.

Shear = 22130 × 2 = 44260 kg

steel ratio $= \frac{15.2}{100 \times 115} = .00132$

k = .175; j = .943

effective depth reqd $= \sqrt{\frac{35400 \times 100}{100 \times 1.8 \times 7.18}} = 52.4$ cm use 115 cm

Steel area required $= \frac{35400 \times 100}{2160 \times \frac{7}{8} \times 115} = 16.3$ cm² per meter strip

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

$f_s = \frac{35400 \times 100}{15.2 \times .943 \times 115} = 2149$ kg/cm² ok.

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CALCULATIONS FOR

Design of Ibi nagara Bashi for Mie Ken.

Shear at edge of counter fort wall.

$= 22130 \times 3.25 \div 2 = 35950$ kg

unit shear $= \dfrac{35950}{100 \times .943 \times 115} = 3.32$ kg/cm. ok.

unit bond. $= \dfrac{35950}{6.92 \times 4.0 \times .943 \times 115} = 1.199$ ok. use 1-22φ extra bent bar at end.

Moment for case 3.

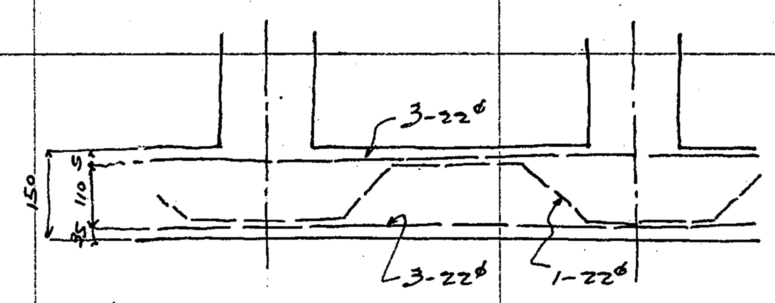
Upward pressure 0

downward pressure $= 15270$ kg/m.

moment $= \dfrac{15270 \times 4.0^2}{10} = 24400$ kgm

Steel area reqd $= \dfrac{24400 \times 100}{2160 \times 7/8 \times 115} = 11.2$ cm²

use 3 – 22φ bars = 11.4 cm² per meter at end.



Reinforcements for 1 extreme meter strip.

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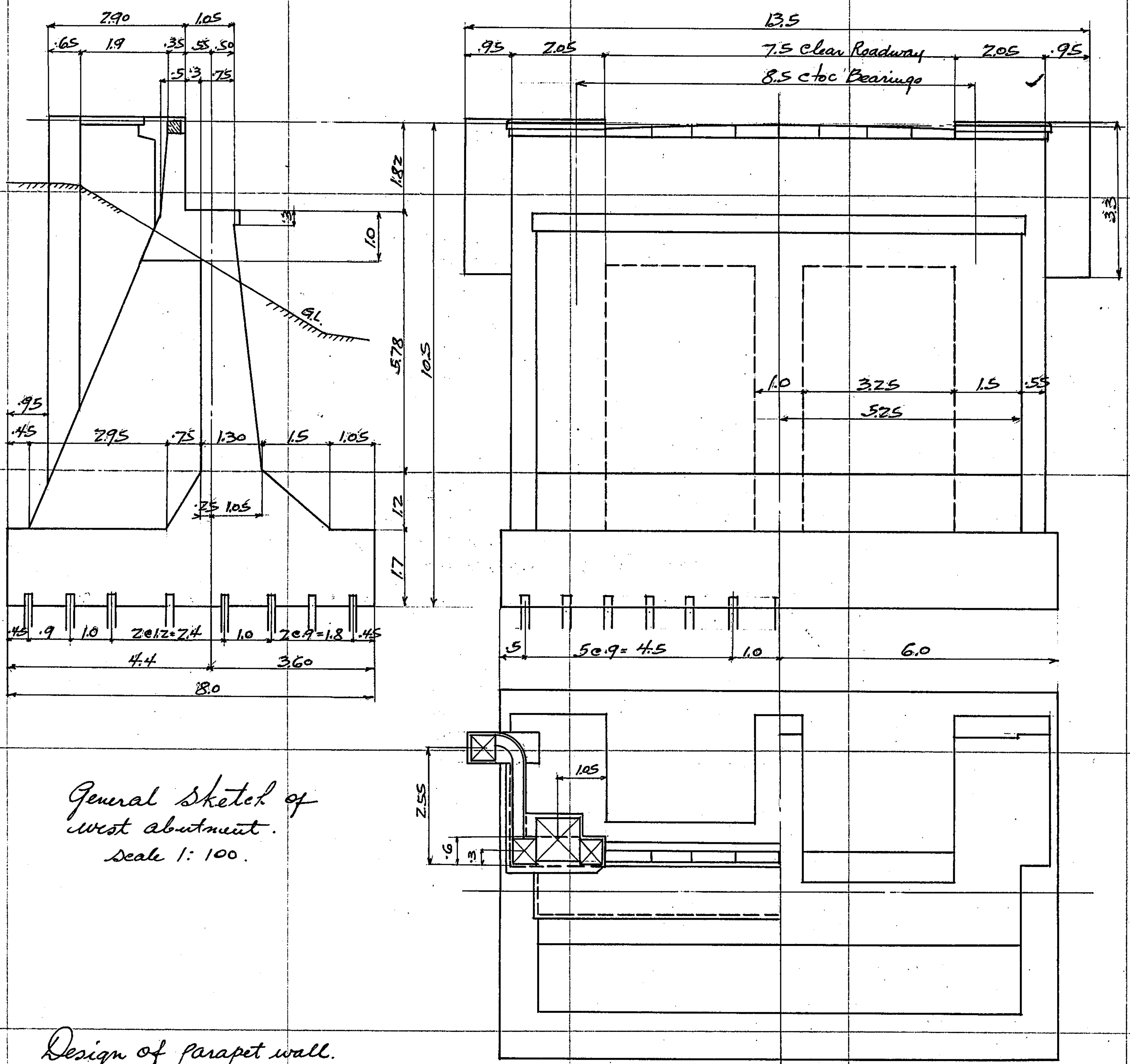
CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.

Design of West Abutment. 桑名寄

General dimensions are as shown on sketch below.



General Sketch of west abutment.
scale 1 : 100.

Design of parapet wall.

All details, same as for East abutment.
See on page 4.

Design of wing wall.

All details same as for East abutment
See on page 4.

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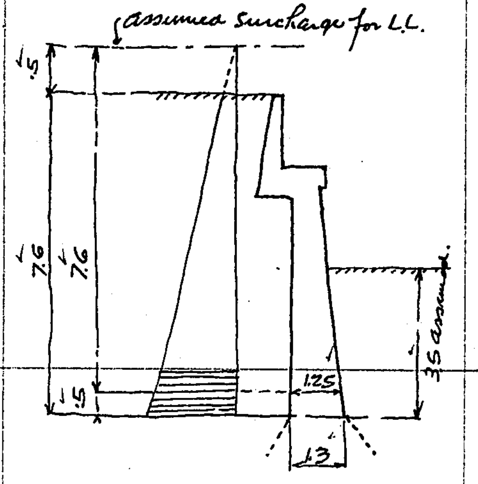
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie ken.

Design of Curtain wall. Span length assumed 4.0 meters



Earth pressure at normal state $1/3 \cdot 1600 \cdot 7.6 = 4050$
$1/3 \cdot 1600 \cdot 3.0 = -1600$
2450 kg/m² average.

Earth pressure during Earthquake. $k = .900$
$.662 \cdot 1600 \cdot 7.1 = 7520$
Seismic force $1.25 \cdot 2400 \cdot .3 = 900$
8420 kg/m² average

moment on wall $= \frac{8420 \cdot 4^2}{10} = 13470$ kgm. shear $= 8420 \cdot 2 = 16840$ kg

Effective depth required $= \sqrt{\frac{13470 \cdot 100}{100 \cdot 7.18}} = 43.3$ cm

use 120 cm effective depth with 5 cm insulation, total depth 125 cm

Steel area required $= \frac{13470 \cdot 100}{2160 \cdot \frac{7}{8} \cdot 120} = 5.93$ cm²

try 16φ bars at 35 cm c to c $= 5.75$ cm²

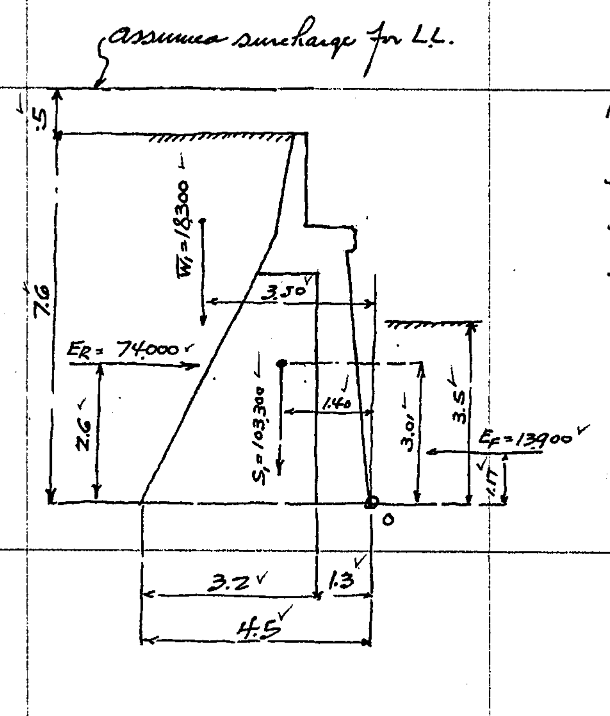
steel ratio $= \frac{5.75}{100 \cdot 120} = .000479$, $j = .95$

$f_s = \frac{13470 \cdot 100}{5.75 \cdot .95 \cdot 120} = 2053$ kg/cm² $< 1200 \cdot 1.8 = 2160$ ok.

Unit shear $= \frac{16840}{100 \cdot .95 \cdot 120} = 1.48$ kg/cm² ok.

Unit bond $= \frac{16840}{5.03 \cdot 2.86 \cdot .95 \cdot 120} = 10.3$ " $< 6.0 \cdot 1.8 = 10.8$ 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	$.25 \cdot .3 \cdot 4.25 = .318 \cdot 2600$	= 830	7.45	6180	1.85	1540
parapet wall	$.425 \cdot 1.55 \cdot 4.25 = 2.800 \cdot 2400$	= 6720	6.52	43780	2.03	13640
Top beam	$1.80 \cdot 1.0 \cdot 4.25 = 7.65$ "	= 18350	5.23	96000	1.45	26600
Coping	$.1 \cdot .3 \cdot 4.25 = .13$ "	= 310	5.63	1750	.50	160
Curtain wall	$1.06 \cdot 4.78 \cdot 4.25 = 21.55$ "	= 51720	2.21	114350	.76	39300
Counterfort	$2.21 \cdot 4.78 \cdot 1.0 = 10.56$ "	= 25350	1.92	48650	2.50	63350
Sum of concrete	= 42.69 m³	103280	3.01 m	310710	1.40 m	144590
granite	= .318	call this 103300 kg				

Weight of earth fill on counterfort wall.
$1.5 \cdot 1 \cdot 7.6 = 11.4 \cdot 1600 = 18300$ 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' } Rear
" " at normal state $2291 \cdot 7.6 \cdot 4.25 = 74000$ kg E_R
" " " $\frac{1600 \cdot 3.5^2}{6} \cdot 4.25 = 13900$ kg E_F } Front
" " during earthquake $.662 \cdot \frac{3.5^2}{2} \cdot 4.25 \cdot 1600 = 27600$ kg E_F'

Case 1. Stresses on counterfort at normal state
Taking moment about point O. in the above sketch

Loads	Hor. forces	Vert. forces		Lev. arm		Moment
S_1		103300	×	1.40	=	144700
W_1		18300	×	3.50	=	64020
E_R	−74000		×	2.60	=	−192500
E_F	13900		×	1.17	=	16260
	−60100	121600		0.267		32480

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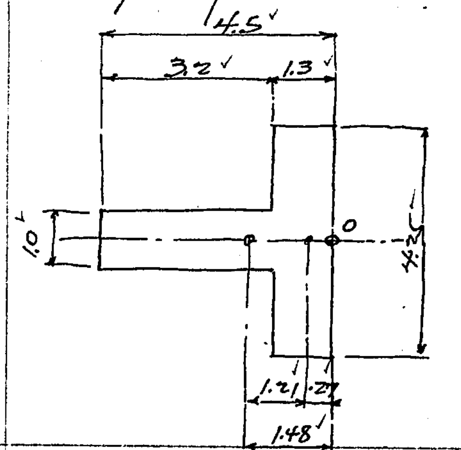
CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.



Center of gravity of section.

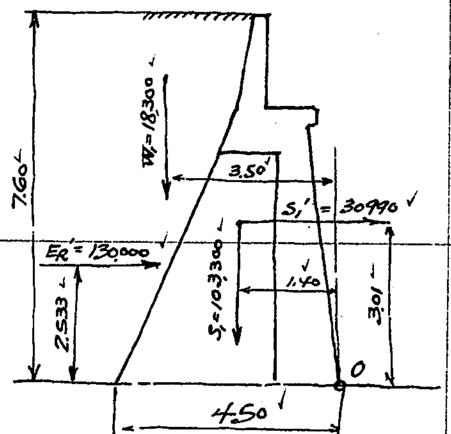
1.30 × 4.25 = 5.52 × .65 = 3.59
1.00 × 3.20 = 3.20 × 2.90 = 9.28
8.72 1.48 12.87

Eccentricity = 1.48 − 0.27 = 1.21 m
moment at bottom section = 121600 × 1.21 = 147000 kgm
Shear = 60100 kg

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

Taking moment about 0.

Loads	Hor. forces	Vert. forces		Lev. arm		Moment
S_1		103300	×	1.40	= +	144750
S_1'	30990	/	×	3.01	= −	93250
W_1		18300	×	3.50	= +	64050
E_R'	130000		×	2.533	= −	329300
	160990	121600 kg		1.76 m	= −	213750



Eccentricity = 1.48 + 1.76 = 3.24 m
moment at bottom of counterfort = 121600 × 3.24 = 394000 kgm.
Shear = 160990 kg

Steel area required for moment = $\frac{394000 \times 100}{2160 \times \frac{7}{8} \times 440}$ = 47.4 cm²

Use 14 − 22φ bars = 53.2 cm²

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

$\frac{t}{d} = \frac{130}{440} = 0.294$

neutral axis in flange.

$k = \sqrt{2 \times .00029 \times 15 + (.00029 \times 15)^2} - .00029 \times 15$
= 0.089

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

$f_s = \frac{394000 \times 100}{53.2 \times .97 \times 440}$ = 1736

Direct comp. = $\frac{121600 \times 15}{872 \times 10000}$ = $\frac{-21}{1714}$ kg/cm² ok.

$f_c = \frac{1735 \times .089}{15 \times (1 - .089)}$ = 9.4

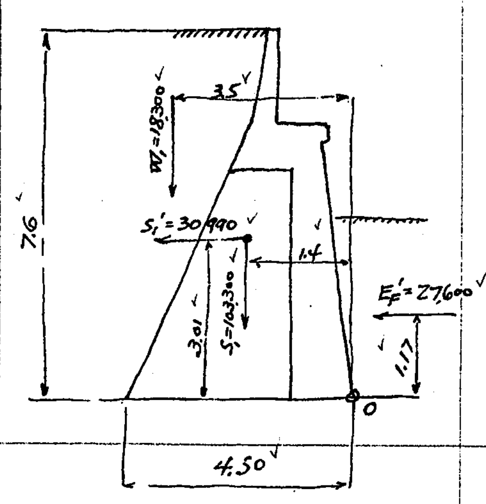
Direct comp. $\frac{121600}{872 \times 10000}$ = $\frac{1.4}{10.8}$ kg/cm² ok.

Unit shear = $\frac{160990}{100 \times .97 \times 440}$ = 3.8 kg/cm² ok

Unit bond = $\frac{160990}{6.91 \times 14 \times .97 \times 440}$ = 3.9 " ok.

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

Loads	Hor. forces	Vert. forces		Lev. arm	Moment
S_1		103300	×	1.40 =	144750
S_1'	30990		×	3.01 =	93250
W_1		18300	×	3.50 =	64050
E_F'	27600		×	1.17 =	32300
	58590	121600		2.75	334350



Eccentricity = 2.75 − 1.48 = 1.27 m
moment at bottom = 121600 × 1.27 = 154500 kgm.
Shear = 58590 kg

Steel area required = $\frac{154500 \times 100}{2160 \times \frac{7}{8} \times 445}$ = 18.4 cm²

Use 8 − 19φ bars = 22.7 cm²

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

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

$(pn)^2 = .00006$

$k = \sqrt{2 \times .0076 + .00006} - .0076$
= 0.115

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

$f_s = \frac{154500 \times 100}{22.7 \times .962 \times 445}$ = 1590

Direct comp. = $\frac{121600 \times 15}{872 \times 10000}$ = $\frac{-21}{1569}$ kg/cm² ok.

$f_c = \frac{1590 \times .115}{15 \times .885}$ = 13.8

Direct comp. = $\frac{1.4}{15.2}$ kg/cm² ok.

Unit shear = $\frac{58590}{100 \times .962 \times 445}$ = 1.37 kg/cm² ok

Unit bond = $\frac{58590}{8 \times 5.97 \times .962 \times 445}$ = 2.87 ok

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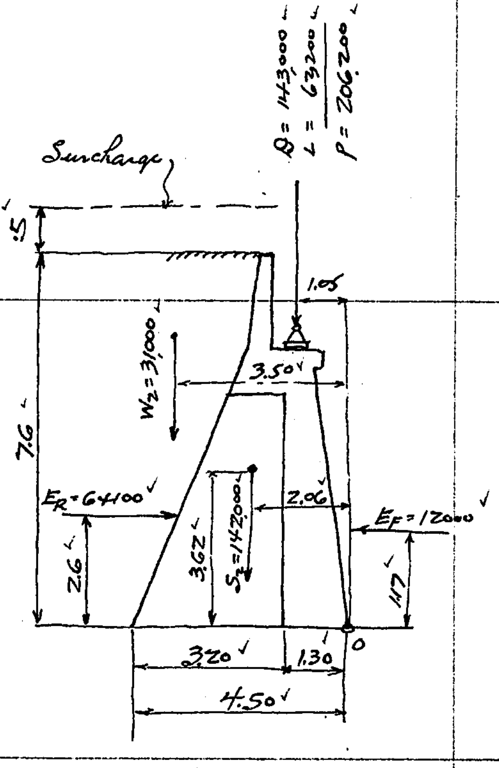
CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.

Design of Counterfort under truss bearing.

Superimposed Loads on abutment.

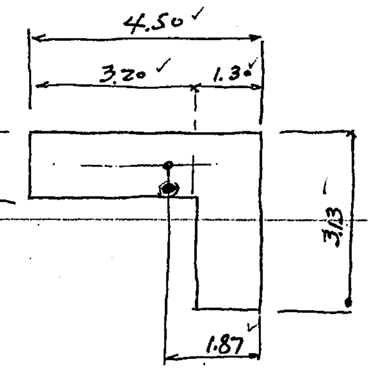
Dead Load	143000
Live Load	63200
	206200 kg on one shoe.



Weight and center of gravity of counterfort under bearing.

				arm vert m		arm	hor m
Granite parapet	.25 × .3 × 1.63 =	.122 @ 2600 =	320	7.45	2380	1.86	590
Light pedestal	say	2.500 @ " =	6,500	8.93	58000	2.20	14300
Posts (3)	3 × .6 × .6 × 1.05 =	1.135 @ " =	2,950	8.21	24200	2.65	7820
Handrails	2.5 × .2 × 1.0 =	.50 @ " =	1,300	8.18	10640	3.70	4810
parapet wall	.425 × 1.52 × 1.63 =	1.053 @ 2400 =	2,525	6.52	16450	2.03	5130
top beam	1.8 × 1.0 × 3.13 =	5.63 @ " =	13,520	5.23	70700	1.45	19600
Coping front	.1 × .3 × 3.13 =	.094 @ " =	226	5.63	1270	.50	110
" end	.1 × .3 × 1.10 =	.033 @ " =	79	5.63	450	1.05	80
projection outside	.65 × 1.25 × 3.30 =	2.680 @ " =	6,430	6.03	38800	4.15	26700
" inside	.30 × .65 × 4.98 =	.839 @ " =	2,015	2.19	4410	4.15	8360
wing wall	.35 × 2.90 × 7.68 =	7.800 @ " =	18,720	3.84	71900	3.05	57050
"	.65 × 1.90 × 1.70 =	2.100 @ " =	5,040	6.73	33900	1.93	9720
Coping	.1 × .3 × 4.45 =	~~.143~~ .134 @ " =	322	7.53	2420	2.15	690
projection top	.6 × .55 × 1.2 =	.396 @ " =	950	7.38	7010	2.50	2380
front wall	1.06 × 4.78 × 3.13 =	15.850 @ " =	38000	2.21	84000	.76	28870
counterfort.	2.21 × 4.78 × 1.70 =	17.960 @ " =	43,100	2.03	87500	2.48	107,000
	Sum of concrete	54.57	141,997	3.62 m	514030	2.06 m	293,210
	" " granite	4.257	call this 142,000 kg				

Assumed section



Weight of Earth fill on counterfort.

$1.5 \times 1.7 \times 7.6 \times 1600 = 31000$ kg $= W_2$

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

$E_F = \frac{1600 \times 3.5^2}{6} \times 3.68 = 12,000$ kg front

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

$E_F' = .662 \times \frac{3.5^2}{2} \times 3.68 \times 1600 = 23,900$ kg front.

Center of gravity of assumed section.

$1.30 \times 3.13 =$	4.07	× .65 =	2.645
$1.50 \times 3.20 =$	4.80	× 2.90 =	13.925
	8.87	1.87 m	16.570

Case 1. Stresses at normal state.

Taking moment about point 0.

Loads	Hor. forces	Vert. forces		Lev. arm		Moment
P		206,200	×	1.05	=	216,500
S_2		142,000	×	2.06	=	292,500
E_R	−64,000		×	2.60	=	−166,500
E_F	12,000		×	1.17	=	14,050
W_2		31,000	×	3.50	=	108,500
	−52,000	379,200		1.23 m		465,050

Eccentricity $= 1.87 - 1.23 = 0.64$ m

moment at bottom section $= 379,200 \times 0.64 = 242,500$ kgm.

Shear $= 52,000$ kg.

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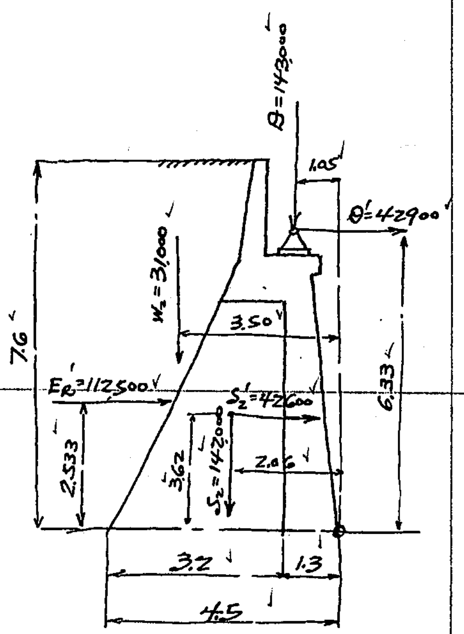
CALCULATIONS FOR

Design of Ibi nagara Bashi for Mie Ken.

Case 2. Stresses during Earthquake (Seismic forces forward)



Taking moment about point O.

D		143000	× 1.05 =	+ 150,100
D'	42900		× 6.33 =	− 271,500
S_2		142000	× 2.06 =	+ 292,500
S_2'	42600		× 3.62 =	− 154,200
W_2		31,000	× 3.50 =	+ 108,500
E_R'	112,500		× 2.533 =	− 785,900
	198,000	316000	.51 m	− 159,600

Eccentricity = 1.87 + .51 = 2.38 m
moment at bottom section = 316000 × 2.38 = 752000 kgm
Shear = 198000 kg

Case 2 governs.

Steel area required for moment = $\frac{752000 \times 100}{2160 \times \frac{7}{8} \times 440} = 90.5$ cm²

use 18 − 25 mm φ bars = 88.3 cm²

Steel ratio $p = \frac{88.3}{313 \times 440} = .00064$
$t/a = \frac{1.30}{440} = .294$

neutral axis in flange.
$p_n = 15 \times .00064 = .0096$
$(p_n)^2 = .000092$

$k = \sqrt{2 \times .00064 \times 15 + .000092} - .0096$
$= \sqrt{.0193} - .0096 = .129$

$j = 1.0 - \frac{.129}{3} = .957$

$f_s = \frac{752000 \times 100}{88.3 \times .957 \times 440} = 2025$

Direct comp. $= \frac{316000 \times 15}{88700} = -54$
1971 kg/cm² ok.

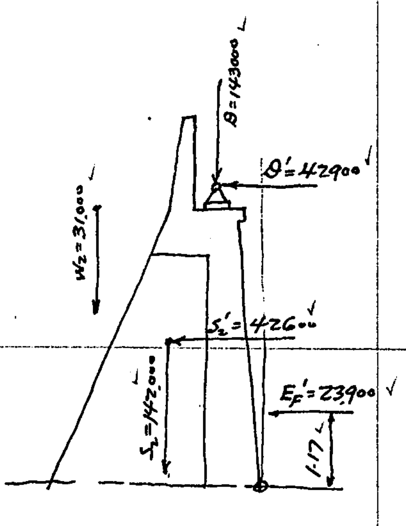
$f_c = \frac{2025 \times .129}{15 \times .871} = 20.00$

Direct comp. $= \frac{316000}{88700} = 3.57$
23.57 kg/cm² ok.

Unit shear = $\frac{198000}{150 \times .957 \times 440} = 3.14$ kg/cm² ok

Unit bond = $\frac{198000}{7.85 \times 18 \times .957 \times 440} = 3.33$ " ok.

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



Loads	Hor. forces	Vert. forces	Lev. arms	Moments about O.
D		143000	× 1.05 =	150,100
D'	42,900		× 6.33 =	271,500
S_2		142000	× 2.06 =	292,500
S_2'	42600		× 3.62 =	154,200
W_2		31000	× 3.50 =	108,500
E_R'	23900		× 1.17 =	28000
	109,400	316000	3.20	1004,800

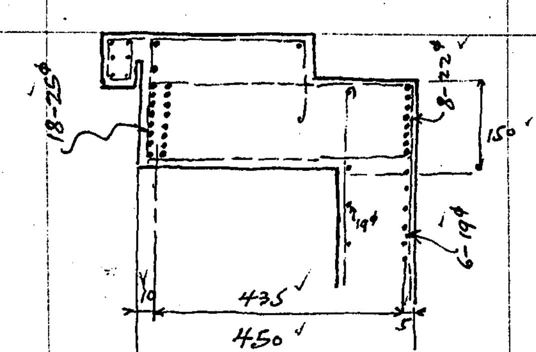
Eccentricity = 3.20 − 1.87 = 1.33 m
moment at bottom section = 316000 × 1.33 = 420,000 kgm
Shear = 109,400 kg

$p = \frac{47.5}{150 \times 445} = .0007$

Steel area reqd for moment = $\frac{420000 \times 100}{2160 \times \frac{7}{8} \times 445} = 50.0$ cm²

$k = .141$; $j = .952$

use 8 − 22φ bars = 30.45
6 − 19φ " = 17.05
47.50 cm²



$f_s = \frac{420000 \times 100}{47.5 \times .952 \times 445} = 2090$

Direct comp $= \frac{316000 \times 15}{150 \times 450} = -70$
2,030 kg/cm² ok.

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CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie ken.

Stability of Abutment

Superimposed Loads on Abutment.

Dead Load.	286000
Live Load.	124000
	410000 kg for one abutment.

Weight and center of gravity of Shaft.

weight	vert. arm	moment.	hor. arm	moment.
103,300	3.01	310,710	1.40	144,590
142,000	3.62	514,030	2.06	292,910
142,000	3.62	514,030	2.06	292,910
387,300 kg	3.46 m	1,338,770	1.89 m	730,410
	2.90		2.55	
	6.36 m		4.44 m	

Weight and center of gravity of Base.

			vert. arm	m	hor. arm	m
2.425 × 1.2 × 10.5 =	30.55 × 2400 =	73,300	2.09	153,200	2.95	216,200
4.0 × 1.2 × 3.1 =	14.88 × 〃 =	35,700	2.30	82,100	5.78	206,500
2.9 × .55 × 1.2 × 2 =	3.83 × 〃 =	9,200	2.30	21,150	5.60	51,500
8.0 × 1.7 × 12.0 =	163.00 × 〃 =	391,000	.85	332,500	4.00	1,565,000
	212.26 m³	509,200 kg	1.16 m	588,950	4.01 m	2,039,200

Weight of earth fill on rear footing

			hor. arm	m
3.35 × 8.80 × 12.0 =	353.5 × 1600 =	565,000	6.32 =	3,570,000
.80 × 5.98 × 11.3 =	54.0 × 〃 =	86,400	4.25 =	367,000
2.9 × 8.8 × .35 × 2 = −	17.9 × 〃 =	−28,600	5.60 =	−160,000
4.0 × 2.6 × 5.98 = −	62.2 × 〃 =	−99,500	5.00 =	−497,000
		523,300 kg	6.27 m	3,280,000

Weight of earth on front footing

4.5 × 2.55 × 12.0 × 1600 = 221,000 kg arm 1.30 m.

Earth pressure at normal state $\frac{1}{3} \cdot 1600 \cdot .5 = 267$

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

$6127 \div 2 = 3064$ kg/m² average.

Rear side pressure $E_R = 3064 \times 10.5 \times 11.6 = 373000$ kg arm 3.65

Front side 〃 $E_F = \frac{1}{6} \cdot 1600 \cdot 6.2^2 \cdot 11.6 = 119000$ kg 〃 2.07

Earth pressure during earthquake

Rear side pressure $E_R' = 0.662 \cdot 1600 \cdot \frac{10.5^2}{2} \cdot 11.6 = 678,000$ kg arm 3.50

Front side 〃 $E_F' = 0.662 \cdot 1600 \cdot \frac{6.2^2}{2} \cdot 11.6 = 236000$ kg 〃 2.07

Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
P		410,000	3.60	1,475,000
S		387,300	4.44	1,720,000
B		509,200	4.01	2,042,000
W_1		523,300	6.27	3,280,000
W_2		221,000	1.30	288,000
E_R	−373,000		3.65	−1,362,000
E_F	119,000		2.07	246,000
	−254,000	2,050,800	3.75 m	7,689,000

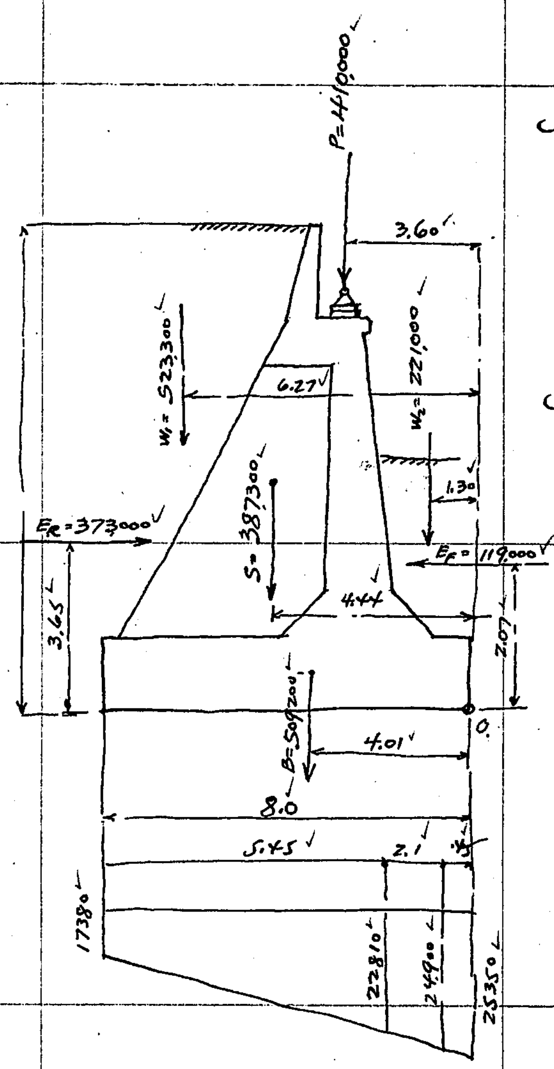
Eccentricity = 4.0 − 3.75 = 0.25 m

~~Resultant force within middle third~~

max. toe pressure $= \frac{2050800}{8.0 \times 12.0}\left(1 \pm \frac{6 \times .25}{8.0}\right)^{.187} = 25350$ kg/m² 2.32 tons/□
or 17380 〃

Max. Load on one pile = 24900 × .9 × .9 = 20.2 kg tons

If 10. kg tons/□ be allowed on sand foundation for bearing
Load on one pile = 20.2 − .9 × .9 × 10 = 12.1 kg tons



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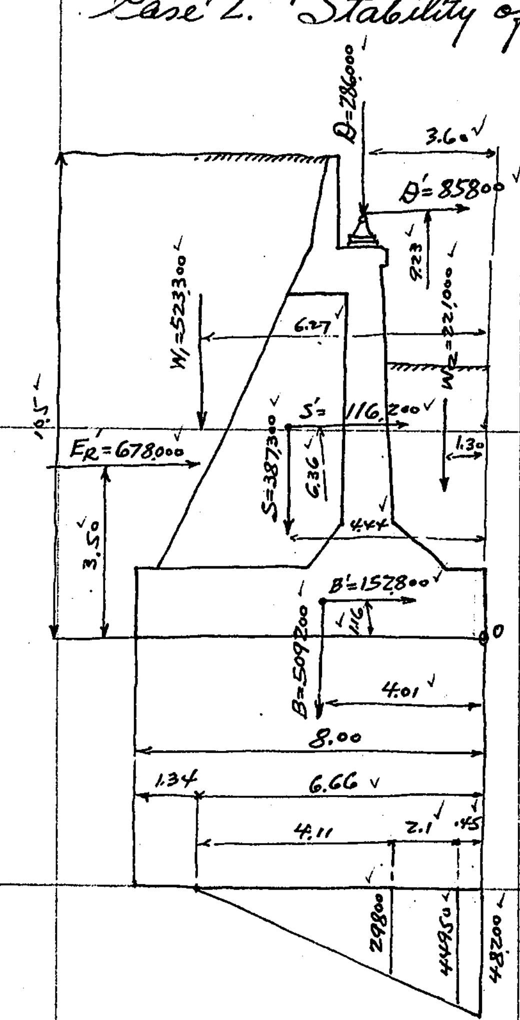
CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.

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



Taking moment about toe O.

Load	Hor. forces	Vert. forces	Lev. arms	Moments
D		286000	3.60	+ 1030000
D'	- 85800		9.23	- 792000
S		387300	4.44	+ 1720000
S'	- 116200		6.36	- 739000
B		509200	4.01	+ 2045000
B'	- 152800		1.16	- 177000
W_1		523300	6.27	+ 3280000
W_2		221000	1.30	+ 287000
ER'	- 678000		3.50	- 2372000
	1032800	1926800	2.22 m	+ 4282000

Eccentricity = 4.0' - 2.22' = 1.78 m

Resultant force outside of middle third, neglecting tension

pressure area 2.22 × 3 × 12.0 = ~~53.3~~ m² 80.0 m²

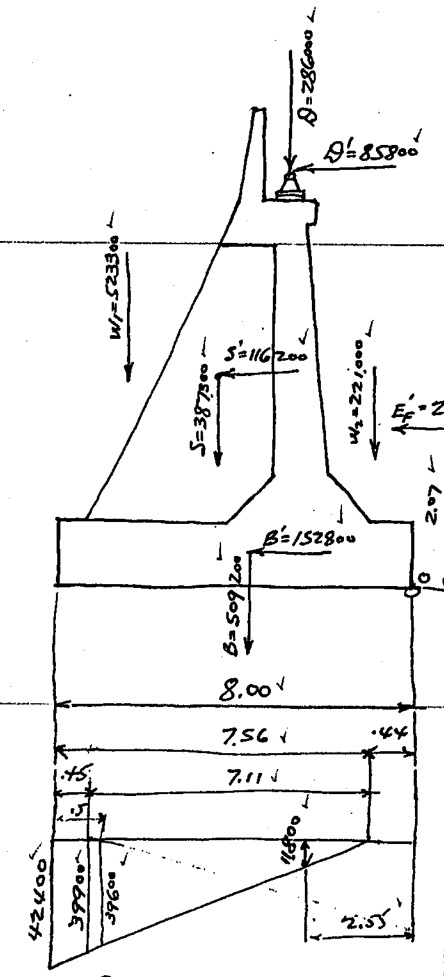
max. toe pressure = $\frac{1926800 \times 2}{80.0}$ = 48200 kg/m² or (4.8 tons/□')

max. load on one pile = 44950 × .9 × .9 = 36.4 kg tons.

If 10.0 × 1.8 = 18 ton/m² be allowed on sand foundation for bearing

max load on one pile = 36.4 - 18 × .9 × .9 = 21.8 kg tons

Case 3. Stability during Earthquake (Seismic forces backward)



Taking moment about toe O.

Load	Hor. forces	Vert. forces	Lev. arms	Moments
D		286000	3.60	1030000
D'	85800		9.23	792000
S		387300	4.44	1720000
S'	116200		6.36	739000
B		509200	4.01	2045000
B'	152800		1.16	177000
W_1		523300	6.27	3280000
W_2		221000	1.30	287000
E_F'	236000		2.07	488000
	590800	1926800	5.48 m	10,558000

Eccentricity = 5.48 - 4.00 = 1.48 m

Resultant force outside of middle third pressure area = 2.52 × 3 × 12 = 90.8 m²

max toe pressure = $\frac{1926800 \times 2}{90.8}$ = 42400 kg/m² (or 3.88 tons/□')

max load on one pile = 39900 × .9 × .9 = 32.3 kg tons.

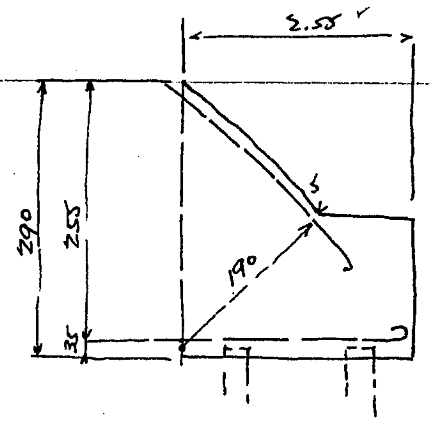
Earth bearing reduced — 32.3 - 18 × .9 × .9 = 15.7

Design of Cantilever footing at toe.

Load on cantilever footing, normal state 23350
~~22810~~
48160 ÷ 2 = 24080 kg/m²

24080 × 2.55 = 61500 kg arm 1.28 m

Earth filling 4.5 × 1600 × 2.55 = - 18380
footing 2.0 × 2400 × 2.55 = - 12250 } 30630 arm 1.28
30870



moment = 30870 × 1.28 = 39500 kgm per meter strip.
Shear = 30870 kg

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CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.

Loads on Cantilever footing during Earthquake. Case 2.

48200
29800
78000 ÷ 2 = 39000 × 2.55 = 99400 kg   arm 1.37 m

upward pressure   99400 kg   arm 1.37 m
Downward ..   30630   arm 1.28
                                68770

moment on footing = 99400 × 1.37 = 136000
                    30630 × 1.28 = −39200
                                    96800 kgm. per meter strip.

Shear .. = 68770 kg

moment and shear during earthquake govern the section.

Effective depth required = $\sqrt{\frac{96800 \times 100}{100 \times 7.18 \times 1.8}}$ = 86.5 cm use 255 cm eff. depth.

Steel area required = $\frac{96800 \times 100}{2160 \times \frac{7}{8} \times 255}$ = 20.1 cm²

use 25ϕ bars at 30 cm c to c = 16.35   Perimeter 3.33 × 7.85 = 26.15
16ϕ " at 60 " " = 3.36   1.67 × 5.03 = 8.40
19.71 cm²   34.55 cm.

Steel ratio $p = \frac{19.71}{255 \times 100}$ = .00077, pn = .00077 × 15 = .0116, $(pn)^2$ = .000135

$k = \sqrt{2 \times .0116 + .000135} - .0116 = 0.141$, $j = 1 - \frac{k}{3} = .953$

$f_s = \frac{96800 \times 100}{19.71 \times .953 \times 255}$ = 2020 kg/cm² ok.

$f_c = \frac{2020 \times .141}{15 \times .859}$ = 22.1 " ok.

Unit shear = $\frac{68770}{100 \times .953 \times 255}$ = 2.8 kg/cm² ok.

Unit bond = $\frac{68770}{34.55 \times .953 \times 255}$ = 8.2 " ok.

Negative moment on Cantilever footing at toe. (Case 3)

upward pressure $\frac{11800 \times 2.11}{2}$ = 12450 kg arm 0.70 m

moment   12450 × 0.7 = + 8710
          30630 × 1.28 = − 39200
          18180 kg.   − 30490 kgm.

Steel area required = $\frac{30490 \times 100}{2160 \times \frac{7}{8} \times 190}$ = 8.5 cm² per meter strip

Use 19 mmϕ bars at 30 cm c to c = 9.45 cm²

Design of Footing at heel.

upward pressure during earthquake (Case 3) 39600 kg/m² average for extreme meter strip

downward pressure Earth   1600 × 8.8 = −14070
                   footing 1.7 × 2400 = − 4070
                                        − 18140
                                        21460 kg/m² upward.

moment $\frac{21460 + 8310}{10} \times 4.0^2$ = 34300 kgm Shear = 21460 × 2 = 42920 kg.

Steel area reqd. = $\frac{34300 \times 100}{2160 \times \frac{7}{8} \times 135}$ = 13.48 cm²

Use 22ϕ bars at 25 cm c to c = 15.22 cm²

downward pressure during earthquake case 2. 18140 kg/m²

use same reinforcements as above.

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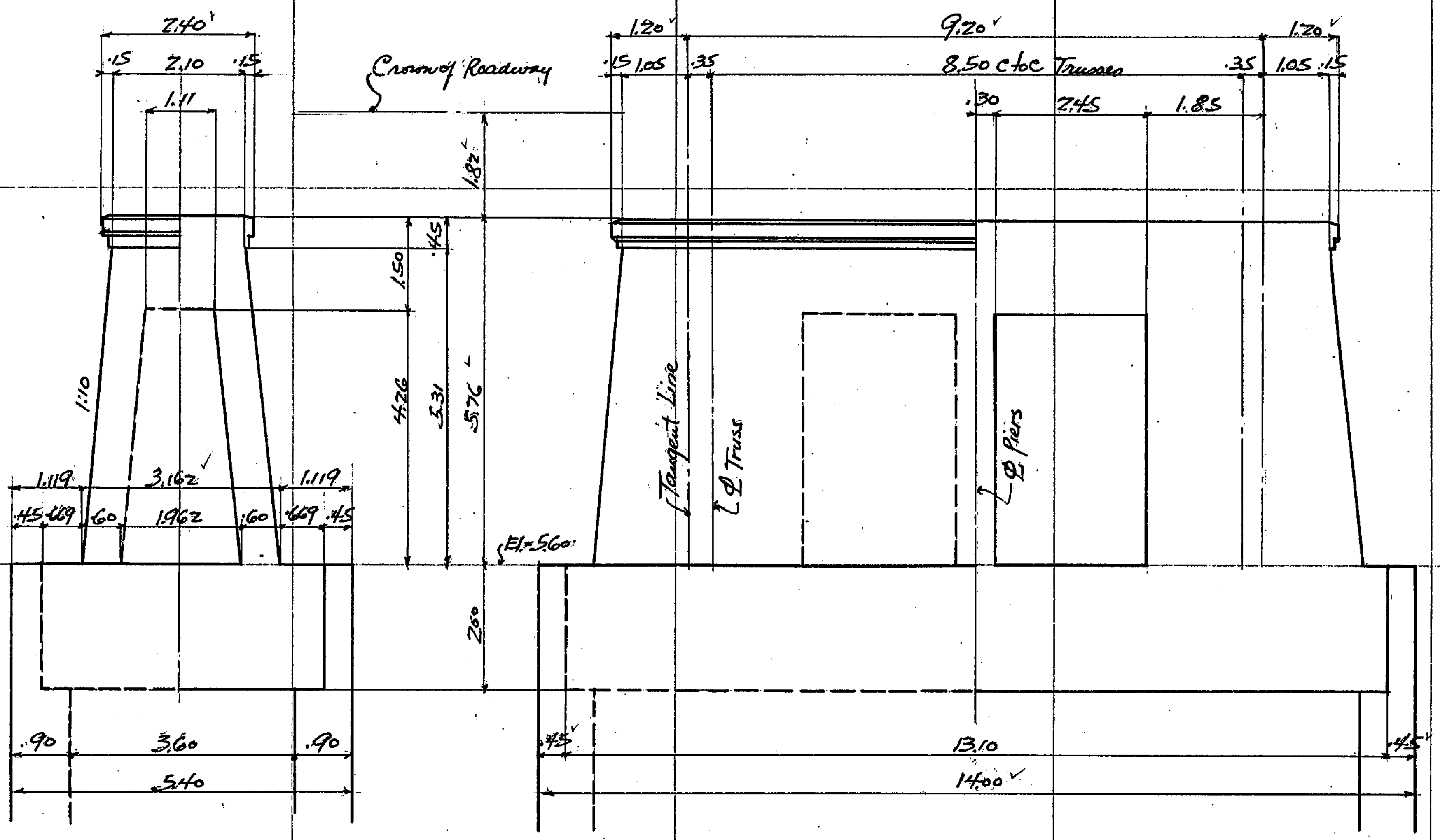
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie ken.

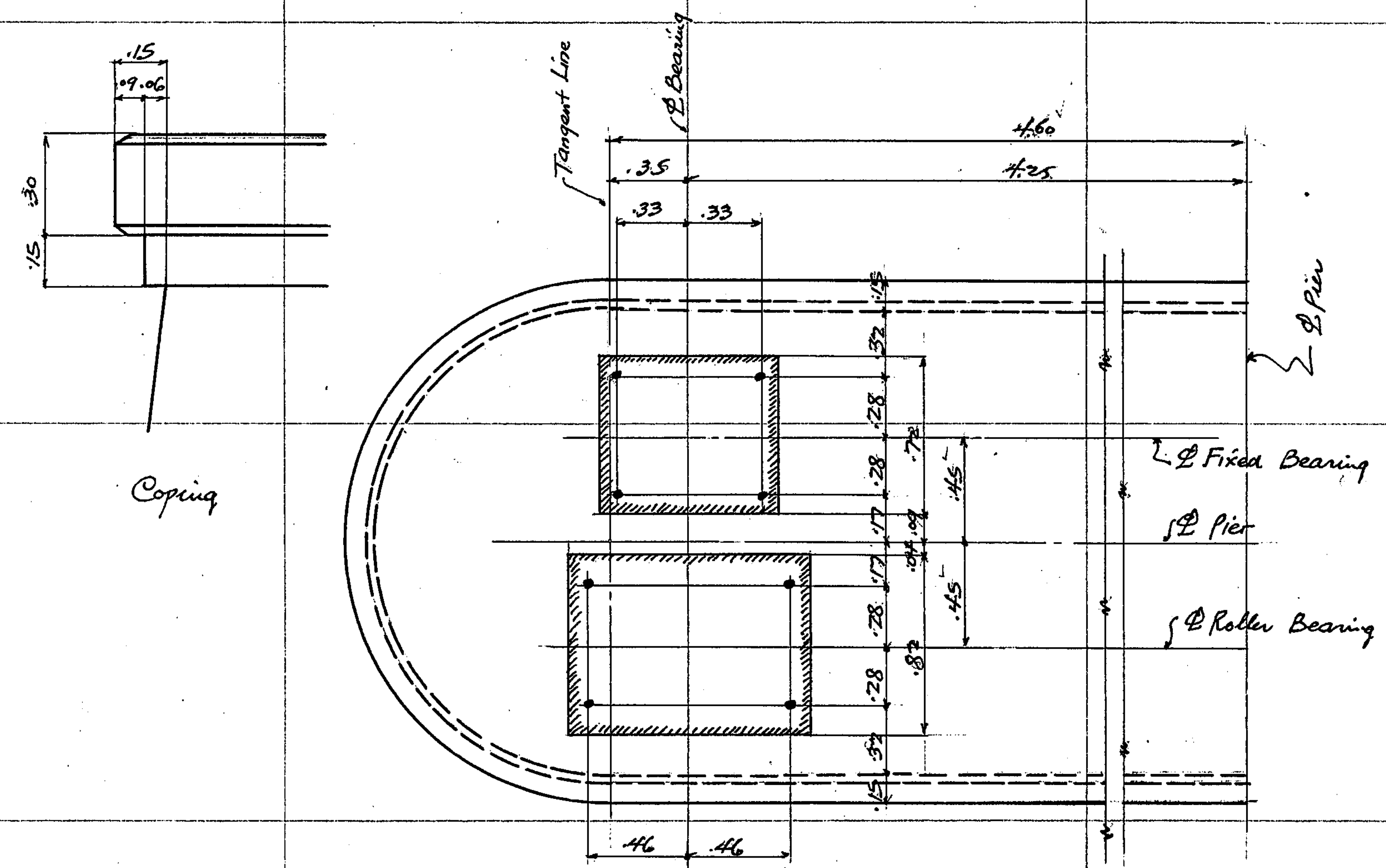
Design of Piers.

Pier shaft.

General dimensions are as shown on sketch below. (Height of shaft for pier P 7).



Scale 1:100



Scale 1:30

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CALCULATIONS FOR

Design of Ibi - nagara Bashi for Mie Ken.

Superimposed Load.

Dead Load. Same as for abutment see on page 3.

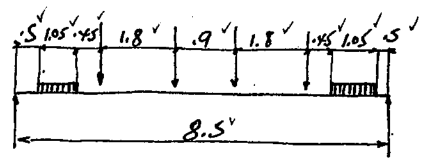
143000 kg on one shoe

or 286000 kg for one-half of Pier shaft.

Live Load Uniform live load $w = \frac{100000}{170+73.2 \times 2} = 315$ kg/m² $315 \times 7.5 = 2360$ kg/lin m

motor truck rear wheel concentration = 3000

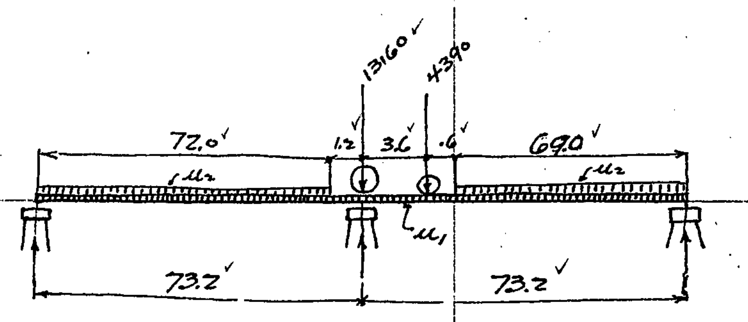
Impact coeff. $= \frac{20}{60+73.2 \times 2} = 9.7\%$ $= \frac{290}{3290} \times 4 = 13160$ kg



front wheel concentration with imp. $= \frac{3290}{3} = 1097 \times 4 = 4390$

unif load on side of motor truck $= 2.10 \times 315 = 660$ kg per lin m. $= M_1$

front and rear of motor truck $= 5.40 \times 315 = 1700$ " $= M_2$



M_2 $\frac{1700 \times 69.0^2}{2 \times 73.2} = 55300$

" $\frac{1700 \times 72.0^2}{2 \times 73.2} = 60200$

M_1 $660 \times 73.2 = 48300$

front wheel $4390 \times \frac{69.6}{73.2} = 4170$

rear wheel $= 13160$

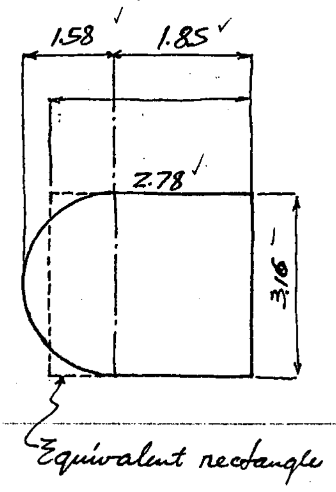
181130 kg for one pier

for one-half of pier = 90600 kg

call this 91000 kg

Summary of Superimposed Dead and Live Load.

Dead Load	286000	572000
Live Load	91000	182000
	377000 kg.	754000 kg
	for ½ of pier	for one pier



Equivalent rectangular section of same moment of inertia.

moment of inertia of assumed section.

Semi. circle $0.0491 \times 3.16^4 \div 2 = 2.450$

rectangle $\frac{1.85 \times 3.16^3}{12} = 4.865$

7.315 m⁴

Length of equivalent rectangle = b.

$\frac{b \times 3.16^3}{12} = 7.315$

$b = \frac{7.315 \times 12}{3.16^3} = 2.78$ m.

Stresses of shaft at bottom section.

Case 1. Stability at normal state for full load.

weight and center of gravity of shaft.

Coping	$2.40^{\phi} \times .30$	=	1.355	× 5.61	7.600
"	$2.40 \times .30 \times 9.20$	=	6.620	× 5.61	37.150
"	$2.22^{\phi} \times .15$	=	.580	× 5.385	3.125
"	$2.22 \times .15 \times 9.20$	=	3.062	× "	16.500
Shaft	$2.631^{\phi} \times 5.31$	=	28.880	× 2.31	66.700
"	$2.631 \times 5.31 \times 9.20$	=	128.500	× 2.47	317.500
Hollow, less	$1.536 \times 2.45 \times 4.26 \times 2$	=	−32.080	× 1.925	−61.800
			136.917 m³	2.827	386.775
Base of shaft.	$4.50 \times 2.0 \times 13.1$	=	118.000	× 1.00	−118.000
			254.917 m³	1.05 m	268.775

weight 136.917 @ 2400 = 329000 kg

weight. 254.917 @ 2400 = 612000 kg

above top of caisson

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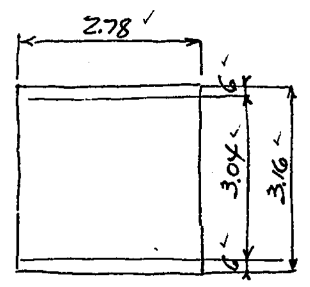
CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.



Superimposed Load on one-half of piershaft. 377,000

weight of " " " 164,500

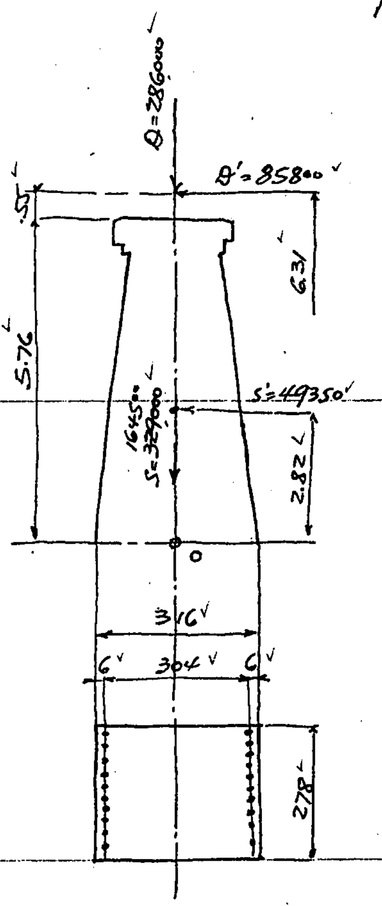
541,500 kg.

Unit bearing pressure at bottom section

$= \frac{541,500}{316 \times 278} = 6.17$ kg/cm² ok. $= f_c$

$f_s = 15 \times 6.17 = 92.5$ kg/cm² ok.

Case 2. Stability of shaft during Earthquake k assumed 0.300.



Dead load for one-half of shaft = 286000 kg = D

seismic force 286000 × .3 = 85800 = D'

weight of one-half of shaft = 164500 = S

seismic force 164500 × .3 = 49350 = S'

Taking moment about center of bottom area O.

Loads	Hor. forces	Vert. forces	Lev. arms	moments
D		286000	0	0
D'	85800		6.31	541,000
S		164500	0	0
S'	49350		2.82	139200
	135,150	450,500 kg	1.517	680,200 kg-m

Reinforcements try 10-25 mm φ bars = 49.1 cm²

for both sides 49.1 × 2 = 98.2

steel ratio $p_0 = 2p = \frac{98.2}{278 \times 316} = .00112$, $d'/h = 6/316 = .019$, $e/h = 151/316 = 0.478$

From the prepared diagrams of combined stresses.

$k = .32$, $L = .075$

$f_c = \frac{M}{Lbh^2} = \frac{680200 \times 100}{.075 \times 278 \times 316^2} = 32.7$ kg/cm² ok.

$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 32.7\left(\frac{310}{.32 \times 316} - 1\right) = 1015$ kg/cm² ok.

Unit shear $= \frac{135150}{278 \times \frac{7}{8} \times 310} = 1.79$ kg/cm² ok

Unit bond $= \frac{135150}{10 \times 7.85 \times \frac{7}{8} \times 310} = 6.35$ " ok.

Assumed section is ample.

Use 22 mm φ bars for upper half of shaft at same spacing as lower half.

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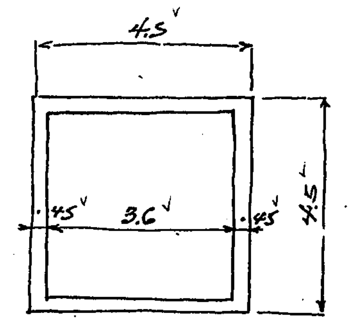
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

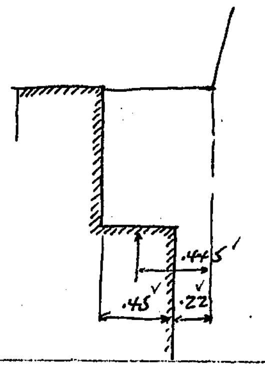
Base of shaft.

At normal state.



max. load = superimposed load = 377,000
weight of shaft 164,500
Base on end partition 4.5×4.5×2.0×2400 = 197,000
638,500 kg

This load assumed to be supported by end partition only at its four sides.
bearing area say.
4.5×4.5 = 20.25
3.6×3.6 = -12.95
7.30 m²



Unit bearing pressure = $\frac{638,500}{7.30 \times 10000}$ = 8.74 kg/cm² ok.

$\frac{638,500}{16.2}$ = 39,400 kg per lin meter of support.

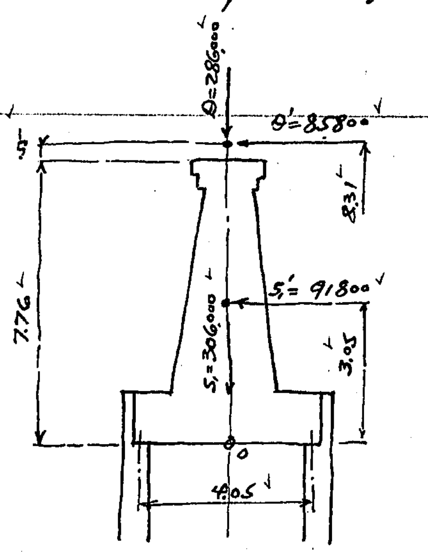
moment on footing = 39400 × .445 = 17,550 kgm/m strip.
Effective depth req'd = $\sqrt{\frac{17550 \times 100}{100 \times 7.18}}$ = 49.4 cm

use effective depth = 190 cm with 10 cm insulation at bottom.
~~Steel area required = $\frac{17,550 \times 100}{1200 \times \frac{7}{8} \times 190}$ = 8.8 cm² per meter strip.~~

use 5 - 25φ bars = 24.5 cm²
unit shear = $\frac{39,400}{100 \times \frac{7}{8} \times 190}$ = 2.37 kg/cm² ok.

unit bond = $\frac{39400}{7.85 \times 5 \times \frac{7}{8} \times 190}$ = 6.0 kg/cm² ok.

During Earthquake k = 0.300. Taking moment about point o.



Loads	Hor. forces	Vert. forces	Lev. arms	moments
D.		286,000	0	0
D'	85,800		8.31	713,000
S.		306,000	0	0
S'	91,800		3.05	280,000
	177,600	592,000	1.68 m	993,000

Direct bearing pressure = $\frac{592,000}{16.2}$ = 36,550 kg/lin m of support.

Bearing pressure due to moment = $\frac{993,000}{4.05 \times 6.55}$ = ±37,450
74,000
or - 900

Unit bearing pressure = $\frac{74,000}{45 \times 100}$ = 16.5 kg/cm² ok.

Steel reqd. for uplift. = $\frac{900}{1200 \times 1.8}$ = .42 cm² per meter.

total steel area = 0.42 × 6.55 = 2.73 cm²
use 10 - 22φ bars = 38.1 cm² ok for ½ shaft.

moment on footing = 74,000 × .445 = 32,900 kgm.
Eff. depth req'd = $\sqrt{\frac{32900 \times 100}{100 \times 7.18 \times 1.8}}$ = 50.5 cm ok

Steel area req'd = $\frac{32900 \times 100}{2160 \times \frac{7}{8} \times 190}$ = 9.16 cm² < 24.5 ok

unit shear = $\frac{74000}{100 \times \frac{7}{8} \times 190}$ = 4.45 kg/cm² ok.

unit bond = $\frac{74000}{7.85 \times 5 \times \frac{7}{8} \times 190}$ = 11.3 " over

p = $\frac{245}{100 \times 190}$ = .0013, j = .942

unit bond = $\frac{74000}{7.85 \times 5 \times .942 \times 190}$ = 10.5 kg/cm² < 10.8 ok.

neg. moment on footing = -900 × .445 = -400 kgm
$2400 \times 2.0 \times .67^2 \div 2$ = -1077
1400

Steel reqd = $\frac{1400 \times 100}{2160 \times \frac{7}{8} \times 190}$ = 0.390 cm²

use 2.5 - 22φ bars = 9.5 cm² ok.

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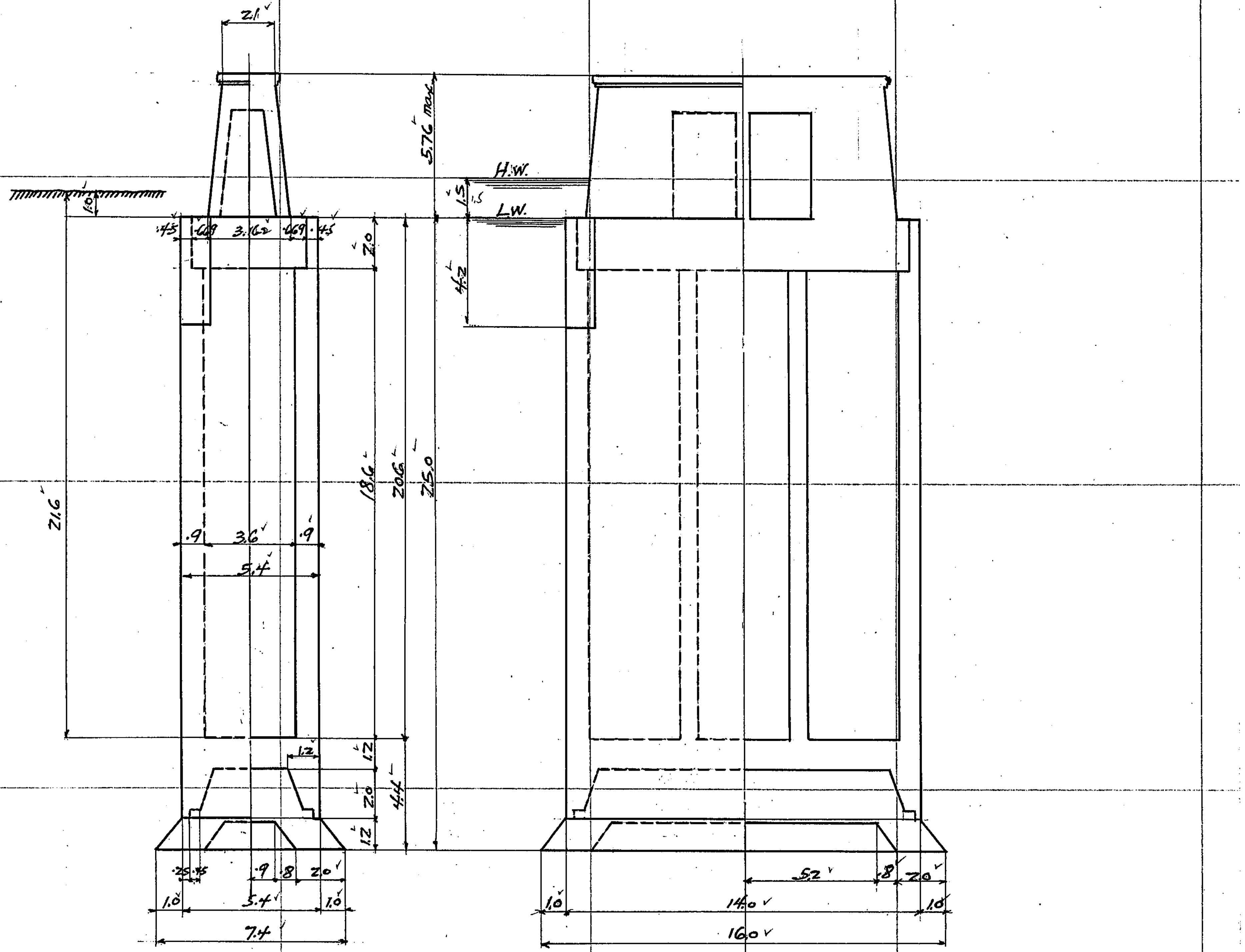
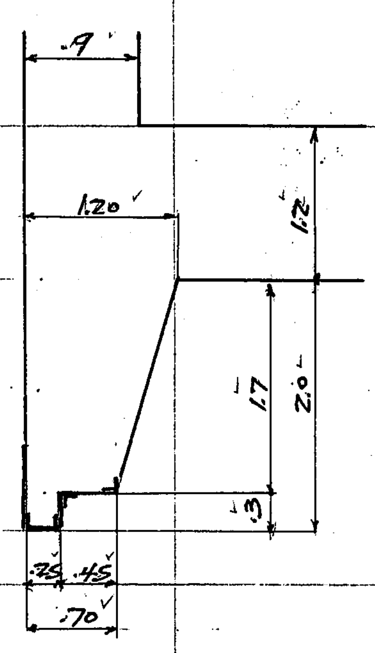
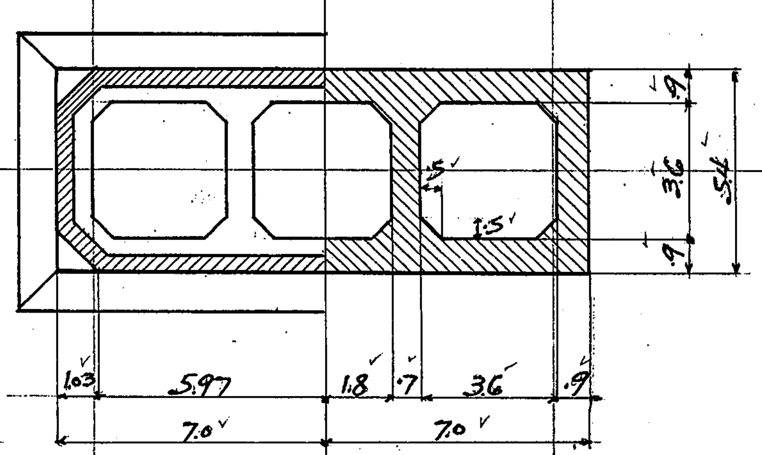
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.
Pneumatic Caissons.
Land Caisson. Reinforced Concrete.







General Sketch for Land Caisson
scale 1:200.

Cutting Edge.
scale 1:60

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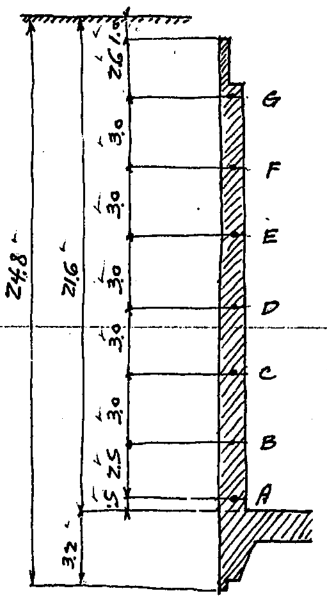
CALCULATIONS FOR

Design of Ibi – Nagara Bashi for Mie Ken.

Side walls of Caisson.



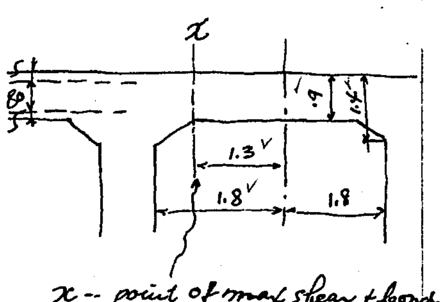
Earth pressure on side wall. $\frac{1}{3} \times 1600 \times h = 533h$ (final pressure after completion)

			water pressure	difference
At section A.	$\frac{1}{3} \times 1600 \times 21.1 =$	11,250 kg/m²	- 21,100	- 9850 kg/m²
〃 〃 B	〃 〃 × 18.6 =	9910	- 18,600	- 8690
〃 〃 C	〃 〃 × 15.6 =	8,310	- 15,600	- 7,290
〃 〃 D	〃 〃 × 12.6 =	6,710	- 12600	- 5,890
〃 〃 E	〃 〃 × 9.6 =	5,120	- 9,600	- 4480
〃 〃 F	〃 〃 × 6.6 =	3,515	- 6600	- 3,085
〃 〃 G	〃 〃 × 3.6 =	1,920	- 3,600	- 1,680

Span length assumed 4.40 meters, moment = $\frac{1}{12} \times p \times 4.4^2 = 1.615\,p$

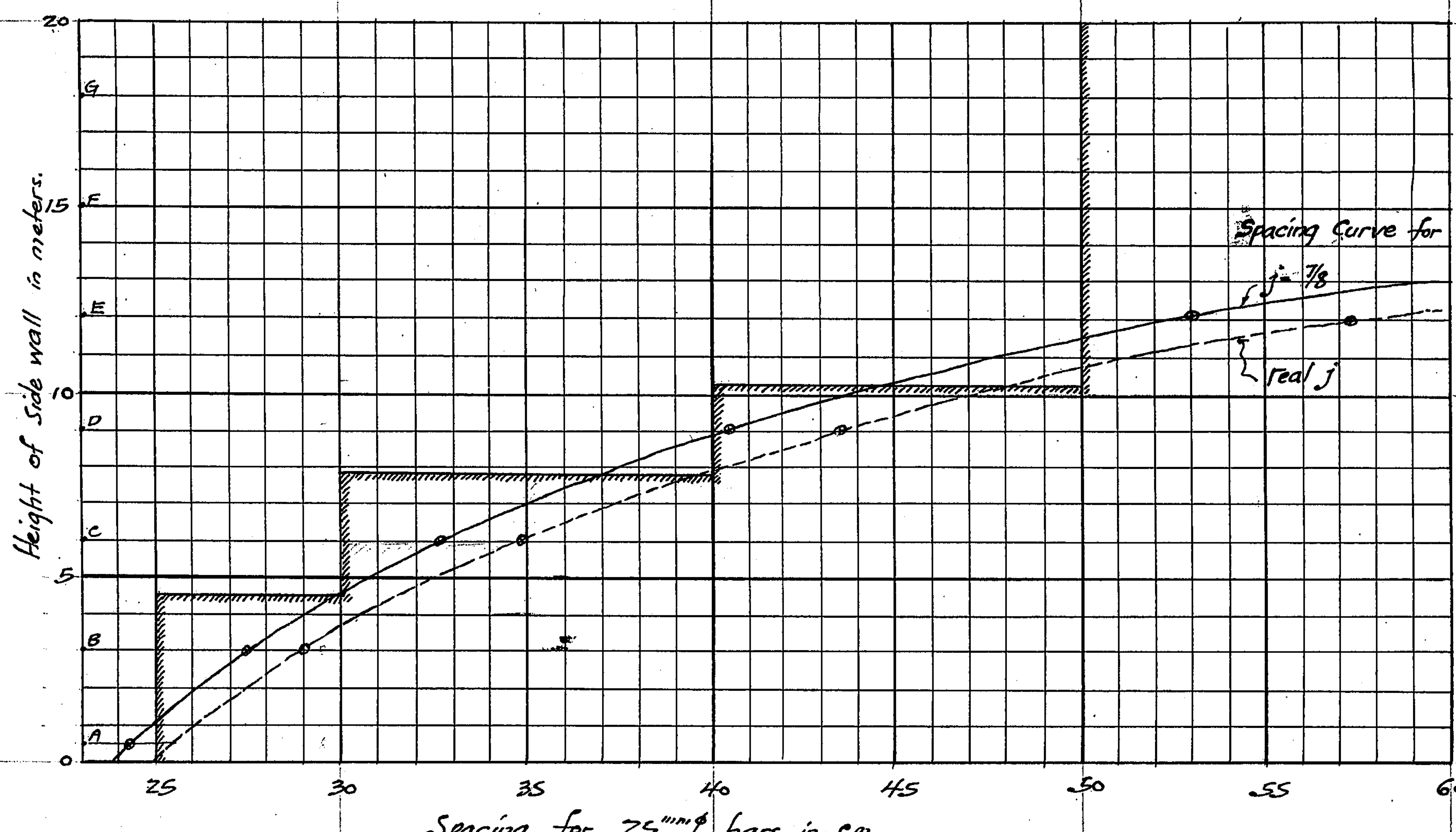
	Earth pressure	moment	Steel reqd.	25φ bars	k	j
At Section A	p= 11,250	18,170 kgm	20.35 cm²	24.15 cm c to c	.231	.923
〃 〃 B	9,910	16,000	17.92	27.40	.220	.927
〃 〃 C	8,310	13,420	15.02	32.70	.202	.933
〃 〃 D	6,710	10,840	12.13	40.45	.181	.940
〃 〃 E	5,120	8,270	9.26	53.00	.164	.946
〃 〃 F	3,515	5,670	6.35	77.30	.145	.950
〃 〃 G	1,920	3,100	3.47	141.50		

Stresses on steel and concrete for side wall.



x – point of max. shear + bond.

at section	k	j	f_s	f_c	Shear at x	max Unit shear	max. unit bond	Perimeter of steel O. cm
〃 〃 A	.231	.923	1,137	23.0	14,620	1.86 kg/cm²	5.73 kg/cm²	32.5
〃 〃 B	.220	.927	1,133	21.4	12880	1.64	5.70	28.7
〃 〃 C	.202	.933	1,127	19.0	10,800	1.36	5.68	24.0
〃 〃 D	.181	.940	1,118	16.5	8,730	1.09	5.63	19.4
〃 〃 E	.164	.946	1,111	14.6	6,660	.83	5.60	14.8
〃 〃 F	.145	.950	—	—	—	—	—	—
〃 〃 G	—	—	—	—	—	—	—	—



Spacing for 25 m/m φ bars. in cm.

Spacing Diagram of Sidewall Reinforcements, 25 m/m φ.

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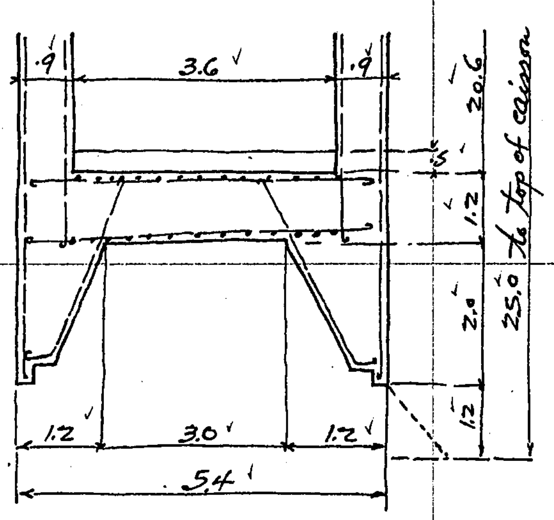
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Design of working chamber
Ceiling Slab.



Span length assumed
transverse span = 3.60 m
longitudinal span = 4.30 m

Theoritical max air pressure = 26.5 @ 1000 = 26500 kg/m²
weight of concrete 1.20 + .5 = 1.70 @ 2400 = $-\underline{4080}$
22420

Load on 3.6 m span $22420 \times (1.5 - \frac{3.6}{4.3}) = 14850$ kg/lin m of span
" 4.3 m " $22420 \times (\frac{3.6}{4.3} - .5) = 7570$ "

transverse moment $= \frac{14850 \times 3.6^2}{10} = 19250$ kgm per meter strip.
longitudinal moment $= \frac{7570 \times 4.3^2}{10} = 14000$ "

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 per meter strip transversaly.

" " $\frac{14000 \times 100}{1200 \times \frac{7}{8} \times 115} = 11.60$ " " longitudinary

Shear
transvers $14850 \times 1.5 = 22300$ kg
longitudinal $7570 \times 1.8 = 13650$ "

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

$j = 0.94$

No. of 25 mm φ bars for one partition $= \frac{15.9 \times 3.6}{4.909} = 11.67$

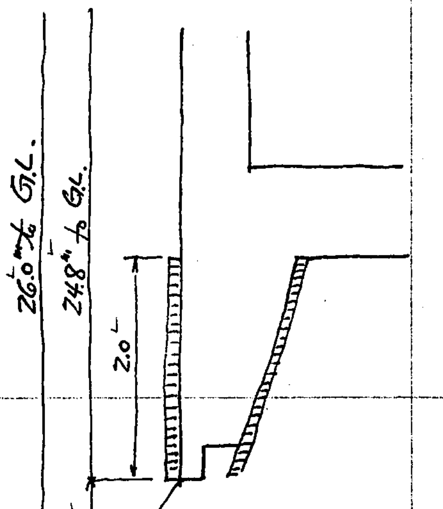
$\frac{11.60 \times 4.3}{4.909} = 10.2$ use 13 bars on both directions

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

Unit bond $= \frac{22300}{\frac{13}{3.6} \times 7.85 \times .94 \times 115} = 7.28$ " ok

実際ノAir pressureハ理論圧力ヨリモ遥カニ小ナルヲ普通トス 本橋ノ如キ粘土盤ノ所ニ於テハ相当減ガスルモノト見ルヲ得ベシ 而シテ之等ノ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 earth pressure $\frac{1}{3} \times 1600 \times 24.8 = 13220$
$\frac{1}{3} \times 1600 \times 22.8 = \underline{12150}$
$25370 \div 2 = 12685$ kg/m² average.

Internal pressure due to air = 26.5 @ 1000 = $\underline{26500}$
13815

moment on wall $= \frac{13815 \times 2.0^2}{2} = 27600$ kgm per meter strip.

Effective depth required $= \sqrt{\frac{27600 \times 100}{100 \times 7.18}} = 62.0$ cm

Use 115 cm eff. depth with 5 cm insulation

Steel area required $= \frac{27600 \times 100}{1200 \times \frac{7}{8} \times 115} = 22.8$ cm² per m strip.

Use 25 φ bars at 20 cm ctoc = 24.55 "

Shear $= \frac{27630}{100 \times .927 \times 115} = 2.59$ kg/cm² ok

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

Unit bond $= \frac{27630}{7.85 \times 5 \times .927 \times 115} = 6.60$ kg/cm² ok.

$j = .927$

Bond stress is about 10 % over, but above stresses are only temporaly stresses during excavation of spread base, and also ~~the~~ air pressure will somewhat be smaller ~~that~~ than the theoritical one assumed in the above calculation.

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

参考. 省線関西線木曽川橋梁ニ於テ実施セラレシ潜函内気圧ハ最高理論圧力 ~~40~~ 39 #/□" (深90呎) ニ対シ実際使用シタル圧力ハ 25 #/□" 内外ナリ 即チ 約65%内外圧力ニテ足レリ.
依而設計図面調製ニ際シ鉄筋量ハ前記ニ比シ多少減ズルモ差支ナキモノト認ム

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken

weight of Caisson

Top 2m
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 = 75500

next 18.6m
5.4 × 14.0 = 75.60
3.6 × 3.6 × 3 = − 38.90
.5 × .5 × 6 = 1.50
38.20 × 18.6 = 710.0 @ 2400 = 1705000
change 1.03 × 1.03 × 2 × 2.2 = −4.67 @ 2400 = −11,200
1693800
1769300 kg

working chamber 5.4 × 14.0 × 1.2 = 90.70
.95 × 1.7 × 35.00 = 56.50
.3 × .25 × 37.80 = 2.84
150.04 @ 2400 = 361,000 kg
Curb shoe say = 5,200
Total weight of caisson = 2135500 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 × 206 = 20200

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
.2 × .1 × 38.0 × 2 = 1.52
35.25 @ 650 = 22900

working shaft. 9.0 tons × 2 = 18000
material lock say 5500
man lock " 3500
misc. pipes, falseworks &c say 4400
31400
74,500 kg

Concrete filling on ceiling slab
3.6 × 3.6 × .5 × 3 = 19.45 @ 2400 = 47000 kg

Summary of caisson weight.
Concrete caisson 2,135,500
forms, Top dam, air locks &c 74500
conc. filling in partition at bottom 47000
2257000 kg During excavation of base.

water filling
3.6 × 3.6 × 3 = 38.9
.5 × .5 × 6 = − 1.5
37.4 @ 1000 = 37400 kg / lin m.

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CALCULATIONS FOR

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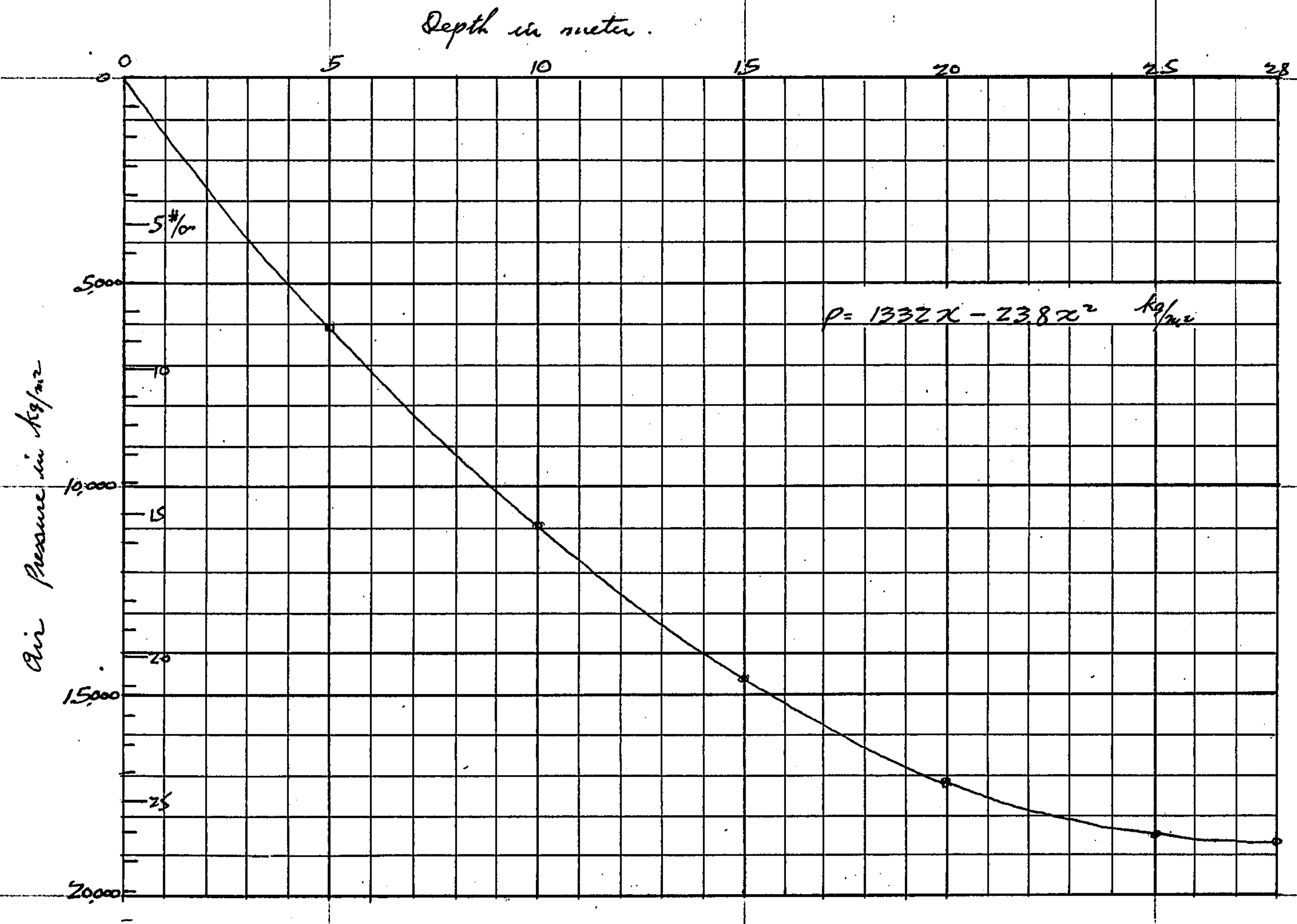
Design of Ibi – nagara Bashi for Mie ken.

pressure in working chamber.

Total weight of caisson = 2257000 kg

潜函内ニ使用スル實圧力ハ必シモ理論圧力ト一致セズ。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圧力調書ヲ基礎トシ適当ナル圧力ヲ推定スレバ大約深サ90呎ニ於テ理論圧力39封度ノ2/3即26封度アレバ充分ナルヲ示セリ。而シテ之レヲ頂点トスル拋物線状ニ累累変化セリ之レヲ米法ニテ示セバ次ノ如シ



max. depth 25.3 m from high tide   Theoritical pressure = 25.3 @ 1000 = 25300 kg/m²

Assumed actual pressure in working chamber = 18,430 " (73%.)

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

Skin friction during sinking work.   Effective depth for friction assumed 22 m.

for 1220 kg/m² (250 #/□') friction   1220 × 38.8 × 22 = 1,040,000 kg

1465 " (300 ") "   1465 × 38.8 × 22 = 1,250,000 kg.

Total downward pressure	friction 1220 kg/m² (250 #/□')	1465 (300)
weight of caisson	2257000	2257000
Skin friction	− 1040,000	− 1,250,000
	1,217,000	1,007,000
water fill 5 m @ 37400 =	187,000	187,000
	1404,000 kg > 1394,000	1,194,000
		187,000
		1381,000 kg

沈下最後ノ4~5m ニ於テ水ヲ5~10m 注入スレバ足ル.

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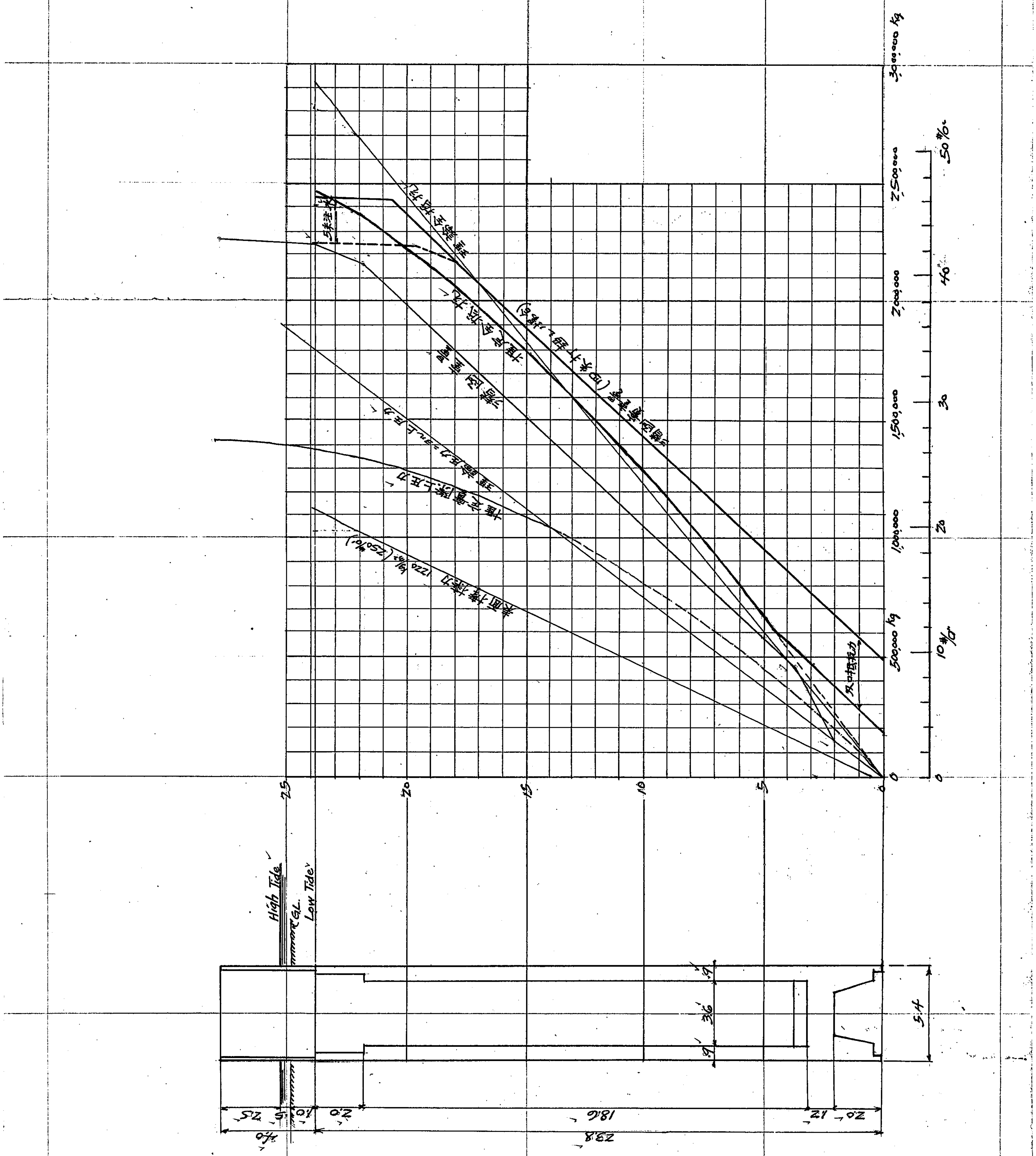
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.
Sinking Diagram of Land Caisson.



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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

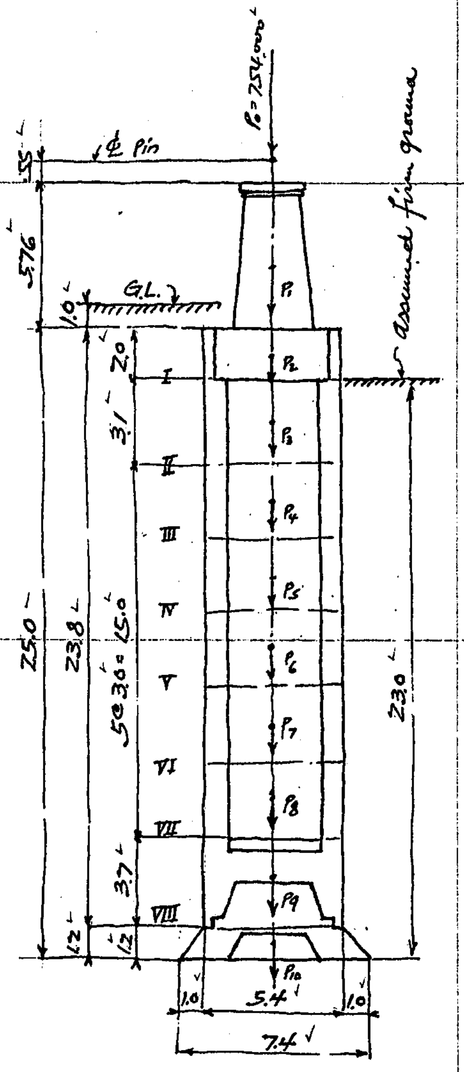
Stability of Pier

Superimposed Loads on pier — Dead Load D= 572,000
Live Load L= 182,000
P_0= 754,000 kg. on one pier.

weight of Shaft (including top fill.) P_1= 612,000 kg — c.g. 1.05 m above top of caisson

Weight of several portions of caisson.

P_2= 75,500 kg — arm 1.0 m from bott.



P_3 = 1693800 × 3.1/18.6 = 282000
water 37400 × 3.1 = 115900
wood frame (difference) = −4300
P_3 = 393600 kg — arm 1.55

水ト木 72@350=2520
コンクリートト木 1.0@1750=1750
=4270 kg
call this 4300 kg

P4, P5, P6, P7 + P8

1693800 × 3.0/18.6 = 273,300
water 37400 × 3.0 = 112200
P4, P5, P6, P7 + P8 = 385,500 kg — arm 1.50

P9 — 14.0 × 5.4 × 3.7 @ 2400 = 672,000 kg — arm 1.85

P10 — 14.0 × 5.4 = 75.60
16.0 × 7.4 = 118.40
194.0 ÷ 2 = 97.0

97.0 × 1.2 @ 2200 = 256000 × .495 = 126700
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
P10 = 237,000 kg — 116,700 — arm 0.49

Center of gravity of pier.

Loads.	~~612,000~~		
P_1	612,000 ×	26.05 =	15,950,000
P_2	75,500 ×	24.00 =	1,812,000
P_3	393,600 ×	21.45 =	8,450,000
P4, P5, P6, P7, P8	1,927,500 ×	12.40 =	23,900,000
P9	672,000 ×	3.05 =	2,050,000
P10	237,000 ×	0.49 =	116,000
	3,917,600 kg	13.35 m	52,278,000

Call this 3,918,000 kg

Seismic force = 3918000 × .3 = 1175000 kg

Stability of pier at normal state.

Superimposed Dead + Live Load = 754,000 kg
weight of pier = 3,918,000
4,672,000

Skin friction 1220 kg/m² (or 250 #/□')
1220 × 38.8 × 21.8 = −1,032,000
3,640,000 kg

unit bearing pressure on soil = 3,640,000 / 7.4 × 16.0 = 30,750 kg/m² (or 2.81 tons/□')

for skin friction of 1465 kg/m² (or 300 #/□')
1465 × 38.8 × 21.8 = −1,240,000
4,672,000
3,432,000

unit bearing pressure on soil = 3432000 / 7.4 × 16.0 = 29,000 kg/m² or (2.65 tons/□')

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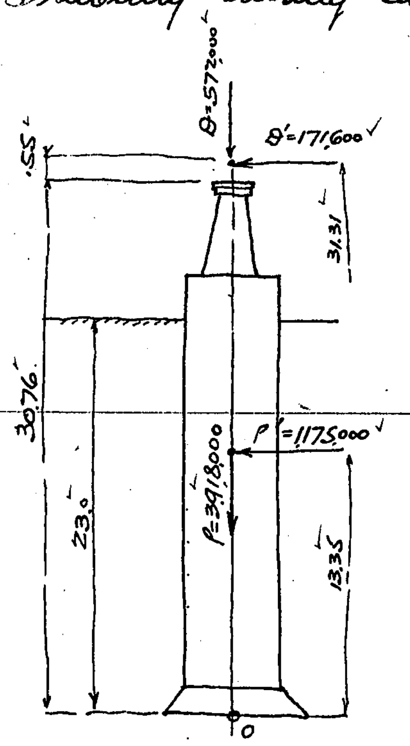
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Stability during Earthquake   k assumed 0.300



moment due to seismic forces about center of base O.

$Q' = 171600 \times 31.31 = 5375000$

$P' = 1175000 \times 13.35 = 15675000$

$M = 21050000$ kgm.

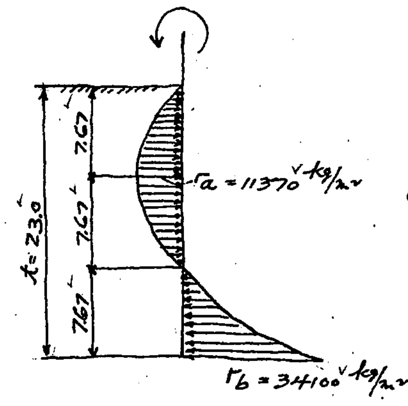
土木学会誌 第十四巻 第四號 五二九頁参照

Sum of vertical Loads

$Q = 572000$

$P = 3918000$

4490000 kg.



$r_b = \frac{12M}{t^2} = \frac{12 \times 21050000}{23.0^2} = 477000$ kg

for one meter strip

$r_b = \frac{477000}{14.0} = 34100$ kg/m²

$r_a = \frac{r_b}{3} = \frac{34100}{3} = 11370$ kg/m²

$11370 \times 14 = 159000$ kg

Passive pressure of earth

$P = wx \frac{1+\sin\phi'}{1-\sin\phi'}$ where $\phi' = \phi - \tan^{-1} k$.

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

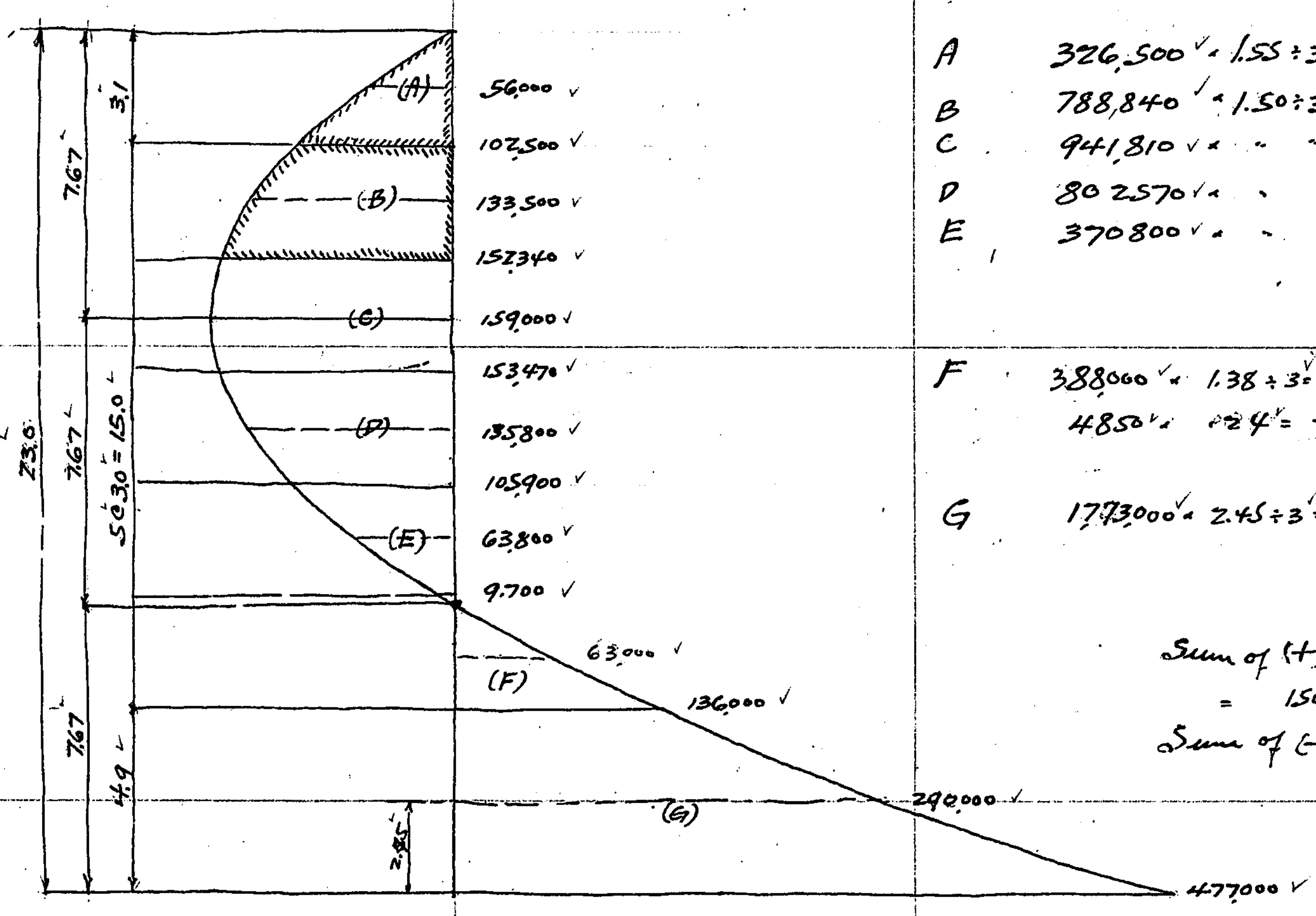
$\phi' = 30° - \tan^{-1} 0.300 = 13°-20'$ $\sin\phi' = \sin 13°-20' = 0.231$

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

Passive pressure at r_a. $P_a = 1600 \times 7.67 \times 1.6 = 19600$ kg/m² > 11370 ok

" " r_b $P_b = 1600 \times 23.0 \times 1.6 = 58800$ " > 34100 ok

Unit bearing pressure on soil $= \frac{4490000}{16.0 \times 7.4} = 37900$ kg/m² or (3.47 tons/□')



Pressure on each section by Simpson's formula.

			arm from bottom
A	326,500 × 1.55 ÷ 3 =	168,800	1.05
B	788,840 × 1.50 ÷ 3 =	394,500	1.41
C	941,810 × " " =	471,000	1.50
D	802,570 × " " =	401,300	1.59
E	370,800 × " " =	185,400	1.92
		1621000 kg	
F	388000 × 1.38 ÷ 3 = 178,500		.92
	4850 × 2.4 = −1,200		
		177,300	
G	1773000 × 2.45 ÷ 3 =	1,448,500	2.02
		1625,800 kg.	

Sum of (+) area (Parabola)

$= 159000 \times 15.34 \times \frac{2}{3} = 1625000$ kg

Sum of (−) area $= -1625000$

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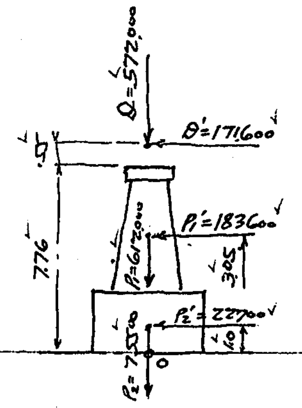
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

moments at several sections during earthquake.

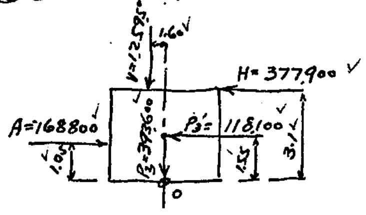
Section I   Z meter below top of caisson.

Taking moment about point O.



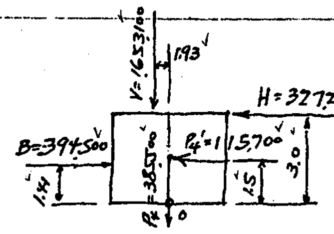
Loads	Hor. Forces	Vert. forces	Lev. arm	moments
D		572,000		0
D'	171,600		8.31	1,427,000
P_1		612,000	0	0
P_1'	183,600		3.05	560,000
P_2		75,500	0	0
P_2'	22,700		1.00	22,700
	377,900 kg	1,259,500 kg	1.60 m	2,009,700 kgm.

Section II.



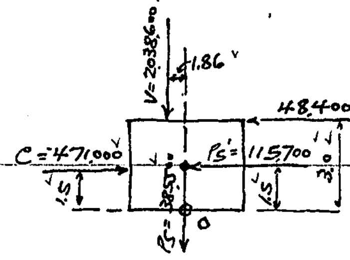
Loads	Hor. Forces	Vert. forces	Lev. arm	moments
V		1,259,500	1.60	2,009,700
H	377,900		3.10	1,171,000
P_3		393,600	0	0
P_3'	118,100		1.55	183,000
A	−168,800		1.05	−177,300
	327,200 kg	1,653,100 kg	1.93 m	3,186,400 kgm

Section III



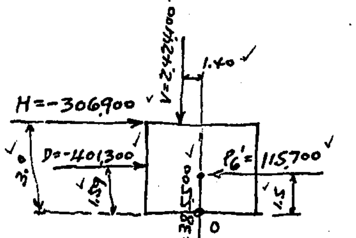
Loads	Hor. Forces	Vert. forces	Lev. arm	moments
V		1,653,100	1.93	3,186,400
H	327,200		3.00	981,000
P_4		385,500	0	0
P_4'	115,700		1.50	173,500
B	−394,500		1.41	−556,000
	48,400 kg	2,038,600 kg	1.86 m	3,784,900 kgm

Section IV



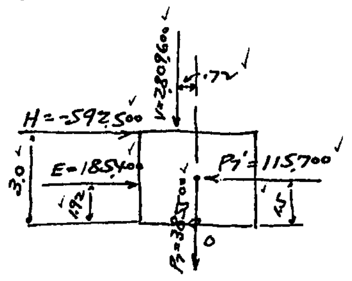
Loads	Hor. Forces	Vert. forces	Lev. arm	moments
V		2,038,600	1.86	3,784,900
H	48,400		3.00	145,200
P_5		385,500	~~1.50~~ 0	0
P_5'	115,700		1.50	173,500
C	−471,000		~~1.50~~	−706,200
	−306,900 kg	2,424,100 kg	1.40 m	3,397,400 kgm

Section V.



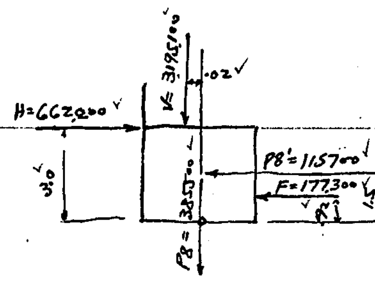
Loads	Hor. Forces	Vert. forces	Lev. arm	moments
V		2,424,100	1.40	3,397,400
H	−306,900		3.00	−920,600
P_6		385,500	0.00	0
P_6'	115,700		1.50	173,500
D	−401,300		1.59	−638,000
	−592,500 kg	2,809,600 kg	0.72 m	2,012,300 kgm

Section VI



Loads	Hor. Forces	Vert. forces	Lev. arm	moments
V		2,809,600	0.72	2,012,300
H	−592,500		3.00	−1,777,500
P_7		385,500	0	0
P_7'	115,700		1.50	173,500
E	−185,400		1.92	−356,000
	−662,200 kg	3,195,100 kg	0.02 m	52,300 kgm

Section VII



Loads	Hor. Forces	Vert. forces	Lev. arm	moments
V		3,195,100	0.02	52,300
H	−662,200		3.00	−1,987,500
P_8		385,500	0	0
P_8'	115,700		1.50	173,500
F	177,300		0.92	163,200
	−369,200 kg	3,580,600 kg	−0.45 m	−1,598,500 kgm

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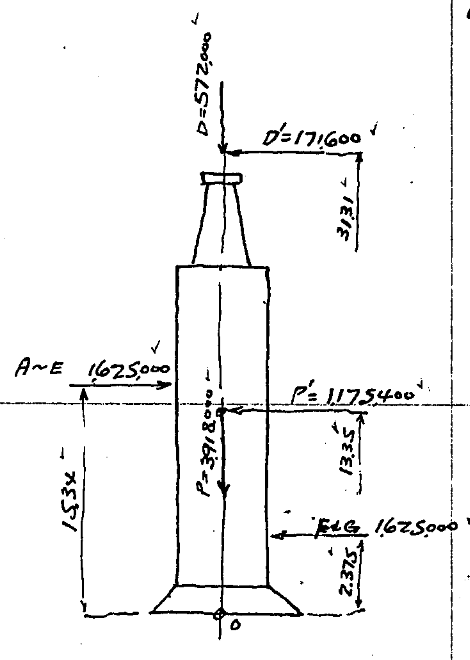
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

moment about center of bottom area.



Loads	Hor. Forces	Vert. Forces	Lev. arm	moment
D		572,000	× 0 =	0
D'	171,600		× 31.31 =	5,370,000
P		3,918,000	× 0 =	0
P'	1,175,400		× 13.35 =	15,690,000
A~E	−1,625,000		× 15.34 =	−24,920,000
F+G	1,625,000		× 2.375 =	3,860,000
	1,347,000 kg	4,490,000 kg		0

Vertical Reinforcements of caisson.

max. moment 3784,900 kgm at section III (8.1 m below top of caisson)
vertical load 2038600 kg
less wt. of water 37,400 × 6.1 = −228,000
1810,600 kg on shell only.

Sectional area of caisson 38.2 m² (page 28)

Direct compression on concrete = 1810,600 ÷ 38.2 ÷ 10000 = 4.74 kg/cm²

Direct compression on steel = 4.74 × 15 = 71.10 "

Bending stresses $\frac{3784900}{4.5}$ = 841,000 kg T or C ~~per lin~~

841,000 ÷ 14.0 = 60000 kg per lin meter of side wall. T or C.
Direct compression 4.74 × 90 × 100 = 42,700 " " C
102,700 kg " fc
or 17,300 " " fs

Unit compression $f_c = \frac{102,700}{90 \times 100}$ = 11.4 kg/cm² ok.

Steel area required for tension = $\frac{17,300}{1200 \times 1.8}$ = 8.00 cm² per meter of side wall.

use 25 mmφ bars at 50 cm c to c on both sides = 4.909 × 4 = 19.65 cm² " " ok.

If the lower 6.7 m of caisson be suspended during excavation steel area required is as follows.

3 m shell with water = 385,500 kg
bottom 3.7 m 672,000
less conc fill 3.6 × 12.2 × 2.0 × 2200 = 193,000 479,000
864,500 kg total wt. of suspended portion.

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

25φ bars 35.7 m × 4 = 141 bars × 4.909 = 692 cm²
19φ bars partition walls. 14 × 2 = 28 " × 2.835 = 79.4
771.4 cm² ok.

Use same vertical reinforcement for shell throughout its whole length.

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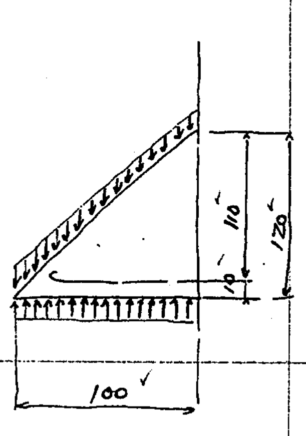
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken

Reinforcements for spread base



Max. upward pressure = 30750 kg/m²
Earth on footing assumed 1.5 m @ 1600 = 24000 kg/m² (including wt. of footing)
6750 kg/m²

moment on footing = 6750 × 0.5 = 3375 kgm per meter strip.
Shear " " = 6750 kg " "

Effective depth required = $\sqrt{\frac{3375 \times 100}{100 \times 7.18}}$ = 21.7 cm

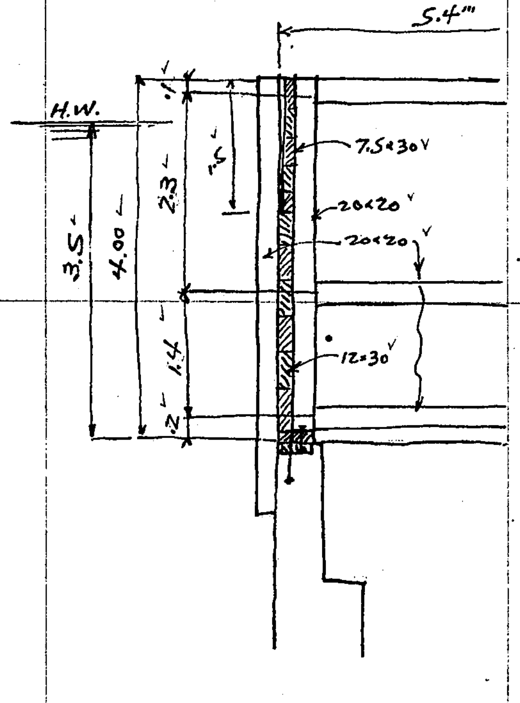
use effective depth of 110 cm
with 10 cm insulation total depth = 120
Steel area required = $\frac{3375 \times 100}{1200 \times \frac{7}{8} \times 110}$ = 2.92 cm²

use 22ϕ bars at 60 cm ctoc = 6.34 cm²

Unit shear = $\frac{6750}{100 \times \frac{7}{8} \times 110}$ = 0.71 kg/cm² ok

unit bond = $\frac{6750}{6.91 \times \frac{7}{8} \times 110}$ = 6.08 kg/cm² ok.

Wooden coffer dam on top of caisson 5.4 m × 14.0 m — 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 = $\frac{3500 \times 2.15^2}{10}$ = 1616 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 m.

Vertical columns. span length at bottom = 1.4 m spacing 2.15 m 2 columns at each panel pt.

pressure = 2.6 @ 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 Sm = $\frac{20 \times 20^2}{6} \times 2$ = 2670 cm³
Shear = 2600 × 2.15 × .7 = 3930 kg
unit shear = $\frac{3930}{20 \times 20 \times 2}$ = 4.9 kg/cm² ok.

Load on strut. approx.
1900 × 1.9 × 2.15 = 7760 kg
unit comp. = $\frac{7760}{20 \times 20}$ = 19.4 kg/cm²
allowable unit comp = $50\left(1 - \frac{l}{60d}\right)$ = 29.1 kg/cm² ok.

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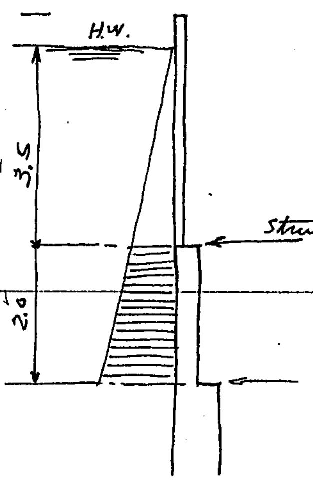
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Vertical reinforcements for top of Caisson required 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/m² on top

" " " 5.5 @ 1000 = 5500 " " bottom.

9000 ÷ 2 = 4500 kg/m² 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 eff. depth with 5 cm insulation

Steel area required $= \frac{1800 \times 100}{1200 \times \frac{7}{8} \times 40} = 4.29$ cm²

use 25φ bars at 50 cm c to c = 9.82 cm²

Shear = 4500 kg

steel ratio $p = \frac{9.82}{100 \times 40} = .00245$

unit shear $= \frac{4500}{100 \times \frac{7}{8} \times 40} = 1.29$ kg/cm² ok

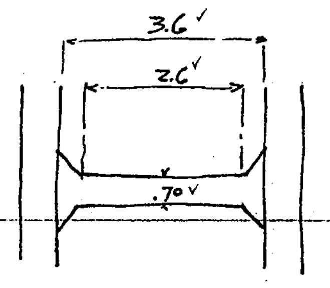
$j = .922$

unit bond $= \frac{4500}{7.85 \times 2 \times \frac{7}{8} \times 40} = 8.2$ "

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

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

Design of partition wall. spacing 4.3 m



Outside earth pressure 21 × 1600 × $\frac{1}{3}$ = 11200 kg/m²

load on wall = 11200 × 4.3 = 48200 kg

Direct compression $= \frac{48200}{70 \times 100} = 6.9$ kg/cm² c.

Unbalance of water fill assumed 10 m max.

unbalance pressure = 10 @ 1000 = 10000 kg/m²

moment $= \frac{10000 \times 2.6^2}{12} = 5640$ kgm

steel ratio $= \frac{11.34}{100 \times 65} = 0.00175$

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

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

use 19φ bars at 25 cm c to c = 11.34 cm²

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

$f_c = \frac{820 \times .202}{15 (1 - .202)} = 13.8$ kg/cm²

Direct compression = 6.9

20.7 kg/cm² ok.

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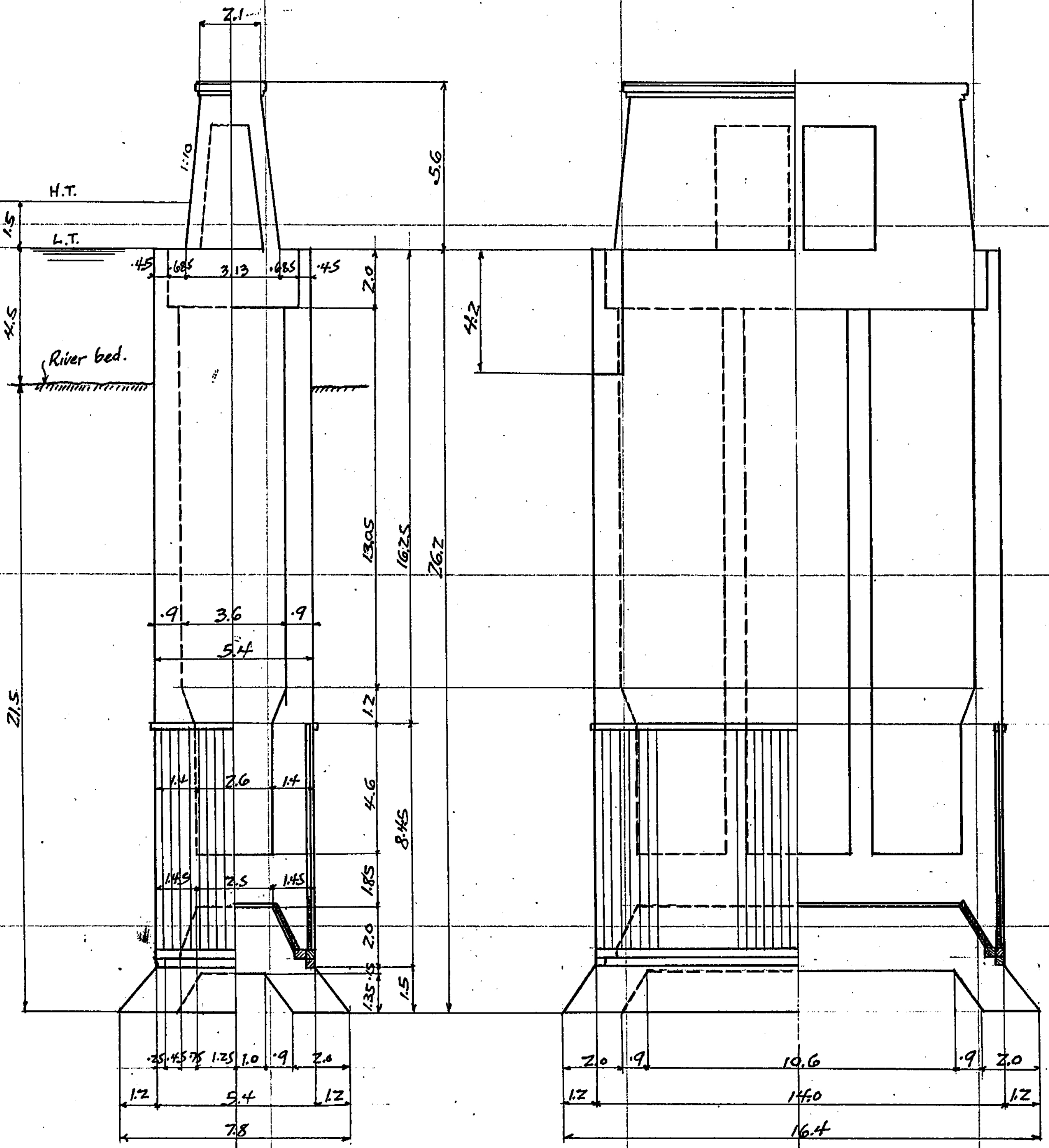
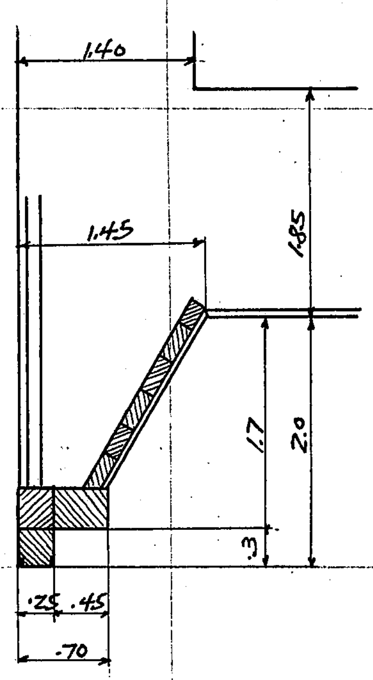
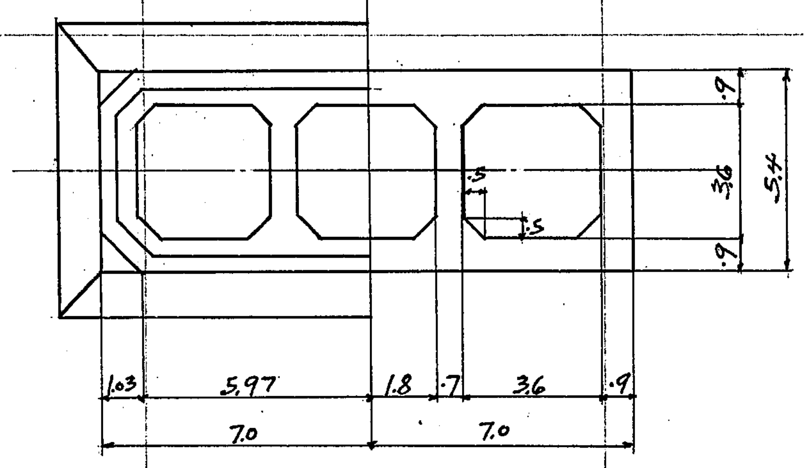
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie ken.
River Caisson. Reinforced Concrete with timber work.
Pier no P3, P4, and P11. 5.4 × 14.0 × 26.2 m Caisson.





Cutting Edge
Scale 1:60



General Sketch of River Caisson. (Pier P4 - P11)
scale 1:200.

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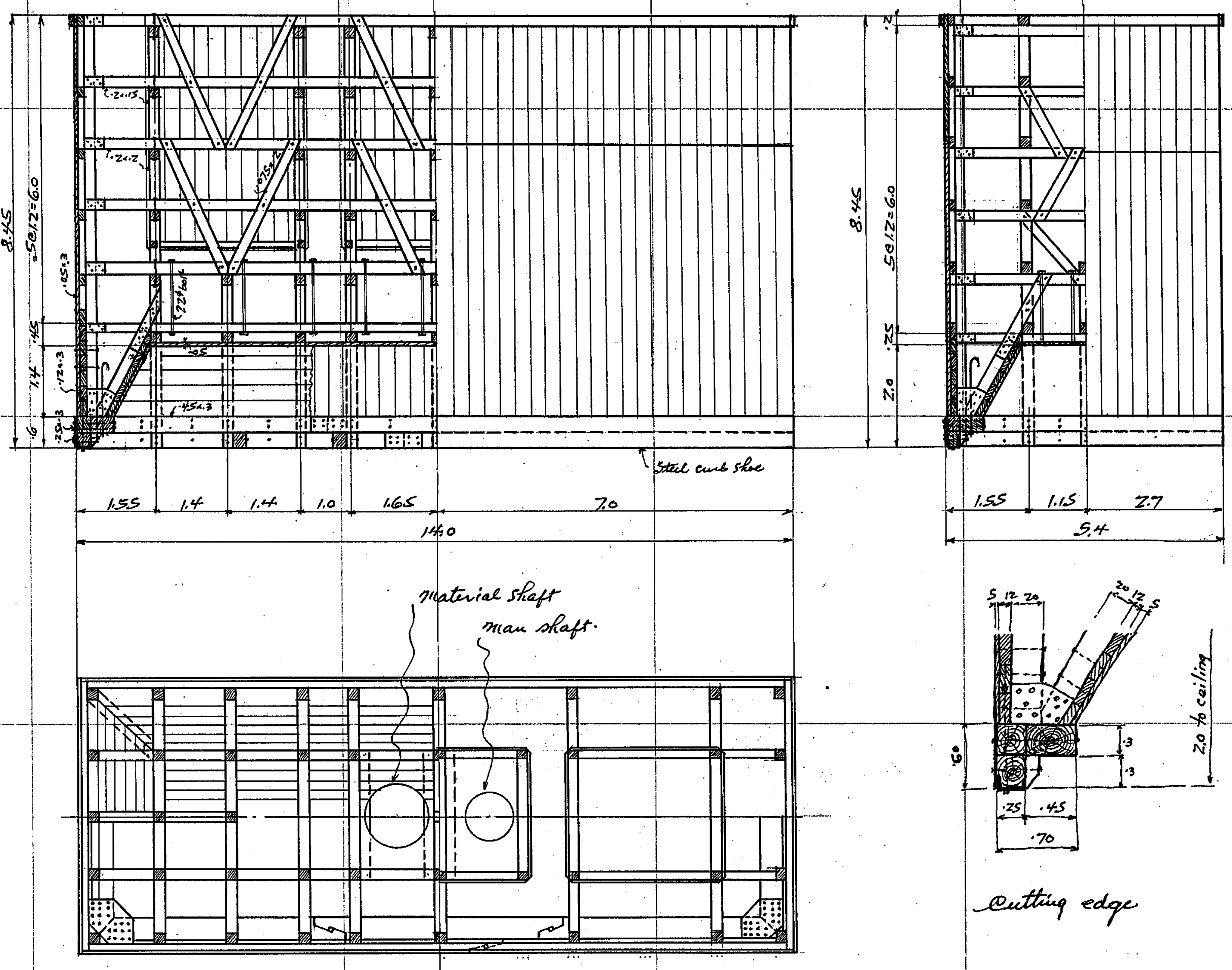
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.
Design of Timber floating caisson 5.4 × 14.0 × 8.45 m
General construction and dimensions are as shown on sketch below.



General sketch of Floating Caisson
Scale 1:100

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie ken.

Weight of Floating caisson. 5.4 × 14.0 × 8.45 m

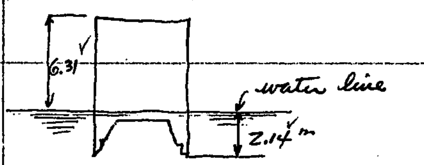
Timber	95.2 m³ @ 650 =	61,820 kg
Bolts, plates nails etc say		5000
Curb shoe say		2,680
working shaft 3 m long =本 etc say.		2000
		71,500 kg

Center of gravity 2.8 m from bottom abt.

weight of water in working chamber.

top area 2.50 × 11.1 = 27.75
bottom " 4.0 × 12.6 = 50.40

78.15 ÷ 2 = 39.08 × 1.4 = 54.70



lower part	50.4 × 0.30 =	15.12
cutting edge 片	49 × 13.5 × 0.30 =	19.88
		89.70
working shaft 口	1.22φ × 0.2 = 0.20	
	.91φ × .2 = 0.13	
		0.33
		90.03 m³ @ 1000 = 90030 kg call this 90000

Total weight of Timber caisson with water in working chamber = 71500 + 90000 = 161,500 kg

Volume of caisson for 1 meter strip. = 5.4 × 14.0 = 75.6 m³ @ 1000 = 75,600 kg for water

case 1. Draft of caisson = $\frac{161,500}{75600}$ = 2.14 meters.

Side ~~workin~~ wall of working chamber under ceiling only filled with concrete and all reinforcements put in.

Concrete. approx. vol. .75 × 1.4 × 34.0 = 35.7 m³ @ 2200 = 78600 kg
reinforcements in floating caisson say 23,400
107,000 kg

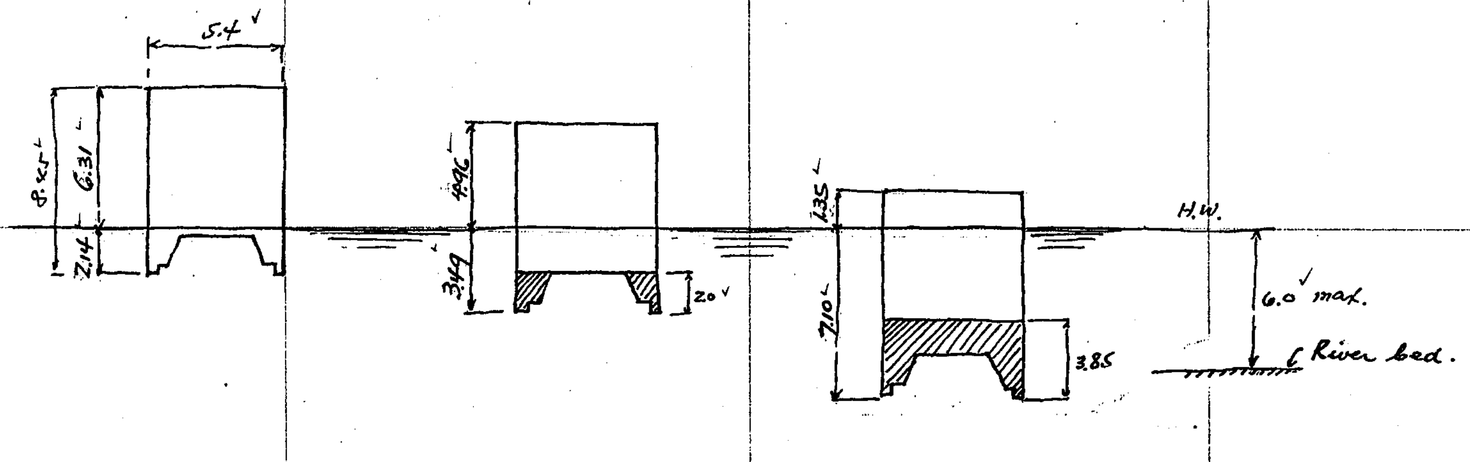
161,500
107,000
263,500 kg

case 2 Draft of caisson = $\frac{263,500}{75600}$ = 3.49 m

Ceiling slab executed. 5.05 × 13.65 × 1.80 = 124.0 m³ @ 2200 = 273,000 kg

case 3. Draft of caisson = $\frac{536,500}{75600}$ = 7.10 m

263,500
273,000
536,500 kg



Empty
Case 1.

Side wall concrete + reinforcements executed.
Case 2

Ceiling slab concrete executed also.
Case 3.

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie ken

Planking of ceiling for working chamber

in case 2. Draft of caisson = 3.49 m water pressure on planking = 1.49 @ 1000 = 1490 kg/m²
span length 1.40 m (Center 1.65 m panel will be shortened by auxiliary cross beams)
moment on planking = $\frac{1490 \times 1.40^2}{10}$ = 292 kgm.

allowable strength of american pine assumed 70 kg/cm² (or 1000 #/□" about)
Section modulus required = $\frac{292 \times 100}{70}$ = 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 5.0 m 5.0 − 2.0 = 3.0 m
upward water pressure = 3.0 @ 1000 = 3000
downward pressure, concrete = 1.8 @ 2400 = 4320
1320 kg/m² downward pressure. ok.

Sidewall planking.

Case 1. draft = 2.14. water pressure 2.14 − .6 = 1.54 @ 1000 = 1540 kg/m²
Case 2 Draft = 3.49 " " 3.49 − .6 = 2.89
2.89 @ 1000 = 2890
Concrete pressure 1.40 @ 2200 = − 3080
− 190 kg/m²

Case 3. draft = 7.10
max. water depth = 6.0 m
Concrete height 3.85
2.15 m @ 1000 = 2150 kg/cm²

Horizontal planking, out side wall span length 1.65 m max.
moment = $\frac{1540 \times 1.65^2}{10}$ = 419 kgm.

Section modulus required = $\frac{419 \times 100}{70}$ = 599 cm³

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

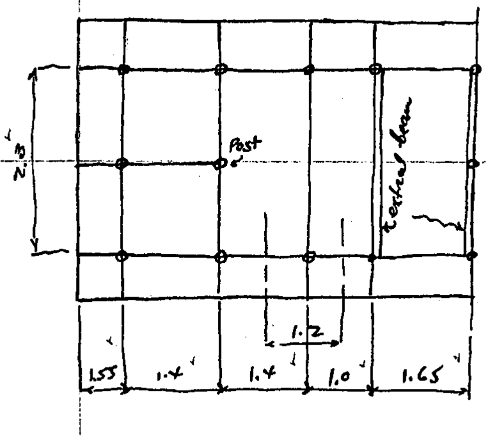
Vertical planking of out side wall. span length 1.2 m
moment = $\frac{2150 \times 1.2^2}{10}$ = 309 kgm Sm. reqd = $\frac{309 \times 100}{70}$ = 442 cm³

for b = 100, d = $\sqrt{\frac{442 \times 6}{100}}$ = 5.15 cm

Use 5 cm planking

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

Cross beams span length = 2.3 m spacing 1.4 m (center panel being shortened by auxiliary cross beam)
max pressure = 1490 kg/m²
load on beam = 1490 × 1.2 = 1785 kg/lin m
moment = $\frac{1785 \times 2.3^2}{10}$ = 944 kgm.



Sm. required = $\frac{944 \times 100}{70}$ = 1348 cm³

width 20 cm d = $\sqrt{\frac{1348 \times 6}{20}}$ = 20.1 cm

Use 20 × 20 cross beam.

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Design of Shell for Caisson.

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

Side wall of caisson.

Depth of earth $= 16.15^{m}$

Surcharge of water $= 6 \cdot \frac{1000}{1600} = 3.75$

19.90^{m} at bottom of side wall. pressure $20 @ \frac{1600}{3} = 10670\ kg/m^2$

moment on wall $= \frac{10670 \cdot 4.3^2}{12} = 16430\ kgm$ per meter strip of wall. eff. depth $= 98^{cm}$

Steel area required $= \frac{16430 \cdot 100}{1200 \cdot \frac{7}{8} \cdot 98} = 16.0\ cm^2$ per meter strip.

use $25^{mm\phi}$ bars at 30^{cm} c to c $= 16.35\ cm^2$

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

Shear on bottom 2^{m} assumed to be transmitted by horizontal + vertical directions

Shear on wall $= 17.5 \cdot 533 \cdot 1.8 = 16760\ kg$.

unit shear $= \frac{16760}{100 \cdot .935 \cdot 98} = 1.83\ kg/cm^2$ ok.

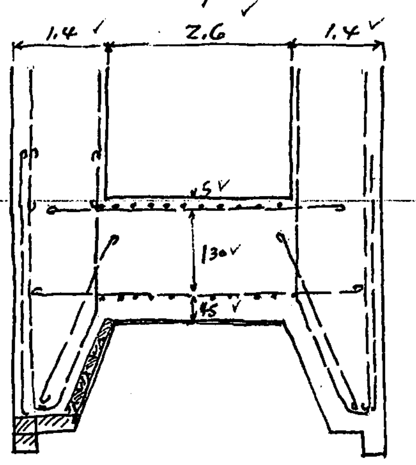
unit bond $= \frac{16760}{7.85 \cdot 3.33 \cdot .935 \cdot 98} = 7.0\ kg/cm^2$

This excess of bond stress will be taken care of by wooden struts at center of span (center partition)

For end partition Shear $= 17.5 \cdot 533 \cdot 1.55 = 14450\ kg$

unit bond $= \frac{14450}{7.85 \cdot 3.33 \cdot .935 \cdot 98} = 6.0\ kg/cm^2$ ok.

Design of working chamber



Height of caisson $= 26.2^{m}$
max. tidal change $= 1.5$
27.7^{m}

Theoritical max. air pressure during high water $= 27.7 @ 1000 = 27700\ kg/m^2$
weight of concrete slab $= 1.8 @ 2400 = -4320$
23380

transverse span length assumed $3.2^{m} = l_1$
longitudinal " " $4.3^{m} = l_2$

Load on shorter span $= 23380 \left(1.5 - \frac{3.2}{4.3}\right) = 17700\ kg$

" " longer " $= 23380 \left(\frac{3.2}{4.3} - 0.5\right) = 5680\ kg$

transverse moment $= \frac{1}{10} \cdot 17700 \cdot 3.2^2 = 18100\ kgm$ per meter strip

" shear $= 17700 \cdot 1.3 = 23000\ kg$

longitudinal moment $= \frac{1}{10} \cdot 5680 \cdot 4.3^2 = 10500\ kgm$

shear $= 5680 \cdot 1.8 = 10200\ kg$

Steel area req'd for transverse span

$= \frac{18100 \cdot 100}{1200 \cdot \frac{7}{8} \cdot 135} = 12.75\ cm^2$ per meter strip

use 25^{ϕ} bars at 25^{cm} c to c $= 19.63\ cm^2$

unit shear $= \frac{23000}{100 \cdot .940 \cdot 135} = 1.81\ kg/cm^2$ ok

unit bond $= \frac{23000}{7.85 \cdot 4.0 \cdot .940 \cdot 135} = 5.78\ kg/cm^2$ ok.

Steel ratio $p = \frac{19.65}{135 \cdot 100} = .00145$

$j = 0.944$

use same bars on top and bottom.

Steel area required for longitudinal span

$= \frac{10500 \cdot 100}{1200 \cdot \frac{7}{8} \cdot 135} = 7.40\ cm^2$ per m strip

use $22^{mm\phi}$ bars at 37.5^{cm} c to c $= 10.15\ cm^2$

unit shear $= \frac{10200}{100 \cdot .95 \cdot 135} = 0.80\ kg/cm^2$ ok

unit bond $= \frac{10200}{6.91 \cdot \frac{100}{37.5} \cdot .95 \cdot 135} = 4.41$ " ok.

Steel ratio $p = \frac{10.15}{135 \cdot 100} = 0.00075$

$j = 0.95$

use same bars on top and bottom.

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

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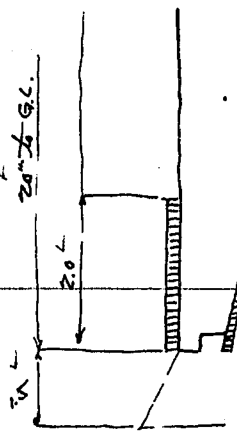
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Cantilever side wall of working chamber.



Depth of earth below river bed = 20.0 m
Surcharge of water = $6.0 \times \frac{1000}{1600}$ = 3.75
23.75 m at bottom of cutting edge.

External earth pressure
at top 21.75 @ 533 = 11600
at bottom 23.75 @ " = 12670
24270 ÷ 2 = 12135 kg/m² average

Internal pressure 26 (27.5) @ 1000 = 27,500
15365

moment on cantilever side wall
$= \frac{1}{2} \times 15365 \times 2.0^2 = 30730$ kgm per meter strip
Shear = 15365 × 2 = 30730 kg

Assume 1/3 of above moment and shear are taken care of by wood frame
Resulting moment = $\frac{2}{3} \times 30730 = 20500$ kgm
" Shear = $\frac{2}{3} \times 30730 = 20500$ kg.

Effective depth req'd = $\sqrt{\frac{20500 \times 100}{100 \times 7.18}} = 53.5$ cm

use eff. depth of 118 cm with 5 cm insulation.
Steel area required = $\frac{20500 \times 100}{1200 \times \frac{7}{8} \times 118} = 16.53$ cm² per meter strip.

use 5 - 25φ bars = 24.55 cm²/m strip
unit shear = $\frac{20500}{100 \times \frac{7}{8} \times 118} = 1.99$ kg/cm² ok.

unit bond = $\frac{20500}{7.85 \times 5 \times \frac{7}{8} \times 118} = 5.06$ " ok.

Weight of caisson. see page 28.
Top 2 m same as for land caisson. = 75,500 kg
next 13.05 m 38.2 × 13.05 = 498.5 @ 2400 = 1,197,000
chamfer − 11200
1,185,800 kg

next 1.2 m top area 38.20
bottom " 5.4 × 14.0 = 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 4.6 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 × 4.6 = 222.0
timber .12 × .2 × 4 × 38 = 3.65
.2 × .2 × 56 × 1.4 = 3.14
− 6.79
215.21 @ 2400 = 516,500 kg.

working chamber 5.3 × 13.9 × 1.8 = 132.5
.73 × 1.40 × 37.0 = 37.8
170.30 m³
timber .2 × .2 × 18 × 5.3 = 3.82
.12 × .85 × 38.5 = 3.92
.2 × .2 × 5 × 13.9 = 2.78
hole 1.22 × 1.8 = 2.07
.91 × 1.8 = 1.17
− 13.76
156.54 @ 2300 = 360,000 kg 鉄筋ナキ故
2,265,000 kg in total.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie-ken

Inside forms	16.25 @ 980	=	15930	
Top wooden dam		=	22900	page 28.
working shaft, locks, pipes &c		=	31400	
			70230 call this	70200 kg

Summary for weight of caisson.

Concrete caisson	2,265,000
Floating caisson	71,500
Forms, dams, air locks &c	70200
	2,406,700 kg During excavation of base.

water filling for upper part. 37400 kg per lin m
" lower " 25.48 @ 1000 = 25500 "

max. depth of cutting edge below high tide 26.2 m
Theoritical air pressure = 26.2 @ 1000 = 26200 kg/m2
Assumed actual pressure in chamber = 18600 " (71%) see page 29. diagram
Total upward pressure = 18600 × 14.0 × 5.4 = 1406000 kg

Skin friction during sinking work: Effective depth for friction. 18.0 m
for 1220 kg/m2 (250%) friction 1220 × 38.8 × 18.0 = 852000 kg
" 1465 " (300 ") " 1465 " " " = 1023000 "

Total downward pressure	friction 1220 kg/m2 (250%)	1465 kg/m2 (300%)
weight of caisson	2406700	2406700
Skin friction	852000	1023000
	1554700	1383700
upward pressure	1406,000	1406000
	148,700 kg	-22300 kg

Bearing area of cutting edge 36.0 × .7 = 25.2 m2
Bearing pressure on cutting edge $= \frac{148700}{25.2} = 5900$ kg/m2 for 1220 kg/m2 friction
" " " $= \frac{22300}{25.2} = -885$ " " 1465 " "

No water filling required for sinking work.

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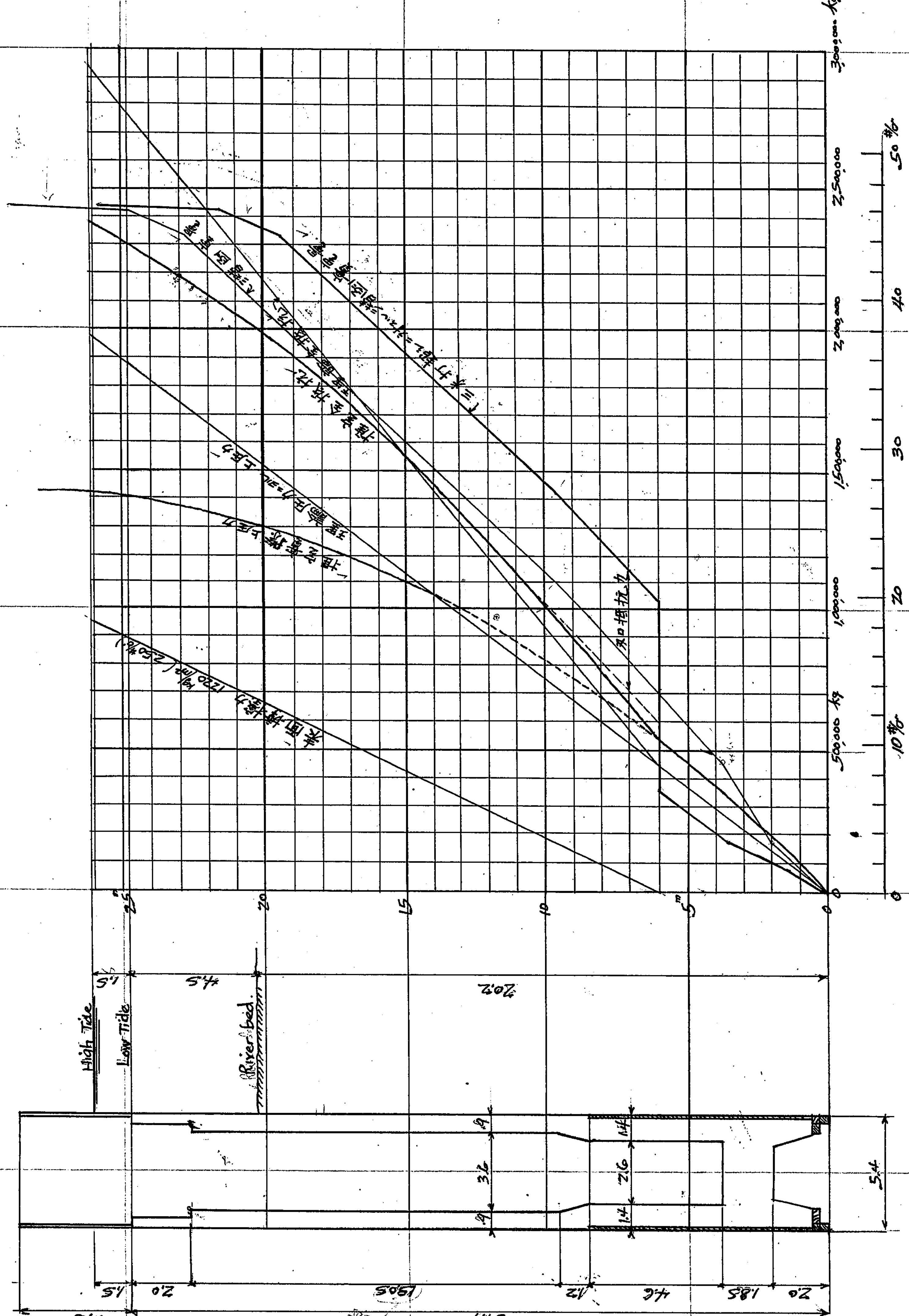
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie-ken.
Sinking Diagram of River Caisson *5.4×14.0×26.2ᵐ Caisson*



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

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Stability of Pier.

Superimposed Loads on pier — Dead Load $D = 572000$ — Same as for Land caisson P31.

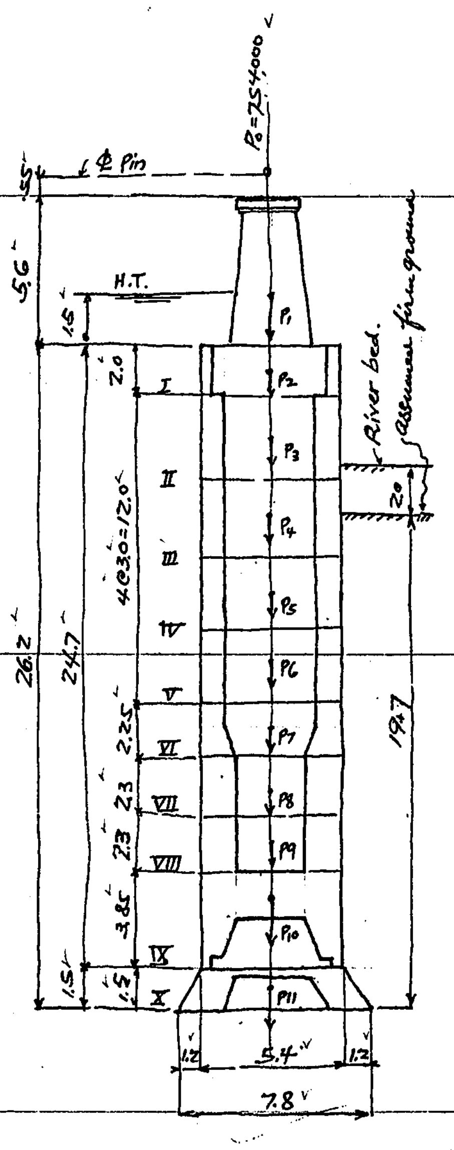
Live Load $L = 182000$

$P_0 = 754000$ kg on one pier.

weight of shaft (including top fill) $P_1 = 612000$ kg c.g. 1.05 above top of caisson

Same as for Land caisson assumed.

weight of several sections of caisson.



P2 same as for land caisson $= 75500$ kg arm 1.0 from bottom

P3, P4, P5, + P6 $\frac{1197000}{13.05} \times 3.0 = 275300$

water $37400 \times 3.0 = \underline{112200}$

387500 kg arm 1.5

P7. conc. $\frac{1197000}{13.05} \times 1.05 = 96300$

water $37400 \times 1.05 = 39300$

conc 127200

water $\frac{37.4 \quad 25.48}{31.44} \times 1.2 \times 1000 = \underline{37700}$

300500 kg arm 1.10

P8 + P9 $516500 \times 2 \div 2 = 258300$

water $25.48 \times 2.3 \times 1000 = 58600$

timbers say $\frac{31500}{2} = \underline{15800}$

332700 kg arm 1.15

P10 concrete 360000

timbers say $\underline{40000}$

400000

Concrete $11.85 \times 3.25 \times 1.4 = 53.9$

$13.05 \times 4.45 \times .6 = 34.8$

timber ?e less (approx) $\underline{-1.6}$

$87.1 @ 2200 = \underline{191700}$

591700 kg arm 1.95 about

P11. $14.0 \times 5.4 = 75.6$

$16.4 \times 7.8 = \underline{127.8}$

$203.4 \div 2 = 101.7$

$101.7 \times 1.5 @ 2200 = 336000$

$10.6 \times 2.0 = 21.2$

$12.4 \times 3.8 = \underline{47.5}$

$68.7 \div 2 = 34.35$

$34.35 \times 1.35 = 46.35 @ 2200 = -102000$

$46.35 @ 1600 = \underline{74200}$

308200 kg arm 0.70

Center of gravity of pier

Loads	weight		arm		moment
P1	612000	×	27.25	=	16680000
P2	75500	×	25.20	=	1903000
P3, P4, P5 + P6	1550000	×	18.20	=	28200000
P7	300500	×	11.05	=	3320000
P8 + P9	665400	×	7.65	=	5082000
P10	591700	×	3.45	=	2040000
P11.	308200	×	0.70	=	216000
	4103300		14.00 m	=	57441000

Call this 4103000 kg

Seismic force $= 4103000 \times .30 = 1231000$ kg.

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CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken

Stability of pier at normal state.

Superimposed Dead and Live Load	=	754,000 kg
weight of pier		4,103,000
		4,857,000

Skin friction 1220 kg/m² (or 25 #/□')

1220 × 38.8 × 18.0 = − 852,000
4,005,000 kg

Unit bearing pressure on soil

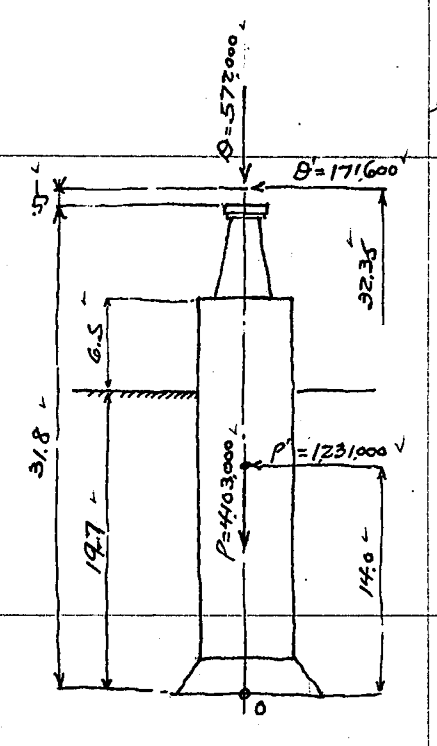
$= \frac{4005000}{16.4 \times 7.8} = \underline{31,350}$ kg/m² (or 2.87 tons/□')

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

1465 × 38.8 × 18.0 = − 1,023,000
4,857,000
3,834,000 kg

Unit bearing pressure on soil

$= \frac{3834000}{16.4 \times 7.8} = \underline{30,000}$ kg/m² (or 2.75 tons/□')



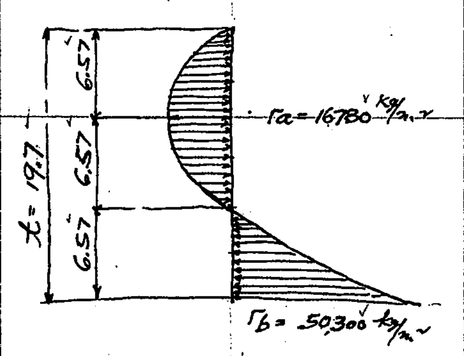
Stability during Earthquake. k assumed 0.300.

moment due to seismic forces about center of base O

D' = 171,600 × 32.35 = 5,550,000
P' = 1,231,000 × 14.00 = 17,230,000
M = 22,780,000 kgm.

Sum of vertical load.

D = 572,000
P. = 4,103,000
4,675,000 kg.



$\sigma_b = \frac{12M}{t^2} = \frac{12 \times 22780000}{19.7^2} = 705,000$ kg.

for 1 meter strip $\sigma_b = \frac{705000}{14.0} = 50,300$ kg/m² (4.6 tons/□')

$\sigma_a = \frac{\sigma_b}{3} = \frac{50300}{3} = 16,780$ " (1.57 ")

16780 × 14 = 235,000 kg

Passive pressure of earth. See page 32.

Depth of earth	= 19.7		~~7.67~~ 6.57
Surcharge of water $6.5 \times \frac{1000}{1600}$ =	4.1		4.10
	23.8 m		10.67 m

Passive pressure at σ_a = 10.67 × 1600 × 1.6 = 27,300 kg/m² > 16780
" " " σ_b = 23.8 × 1600 × 1.6 = 60,900 " > 50,300

上記 σ_b ハ 14m 巾トシテ計算セルモ実際ハ spread base ノ幅 16.4m ナリ。之ニ対スル unit pressure ハ

$\sigma_b' = \frac{705000}{16.4} = 43,000$ kg/m² (or 3.93 tons/□') ok.

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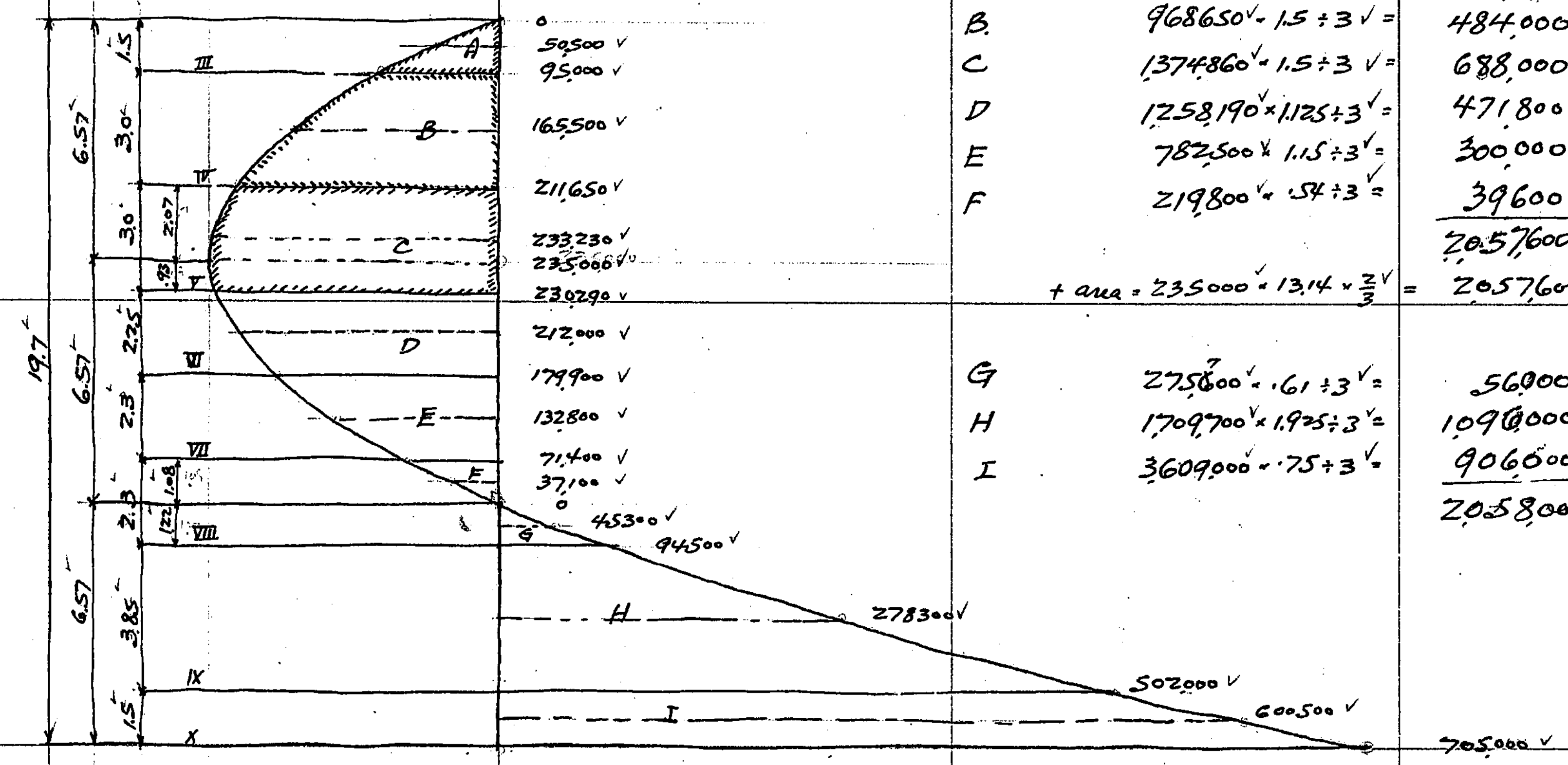
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

Reactional Earth pressure diagram.

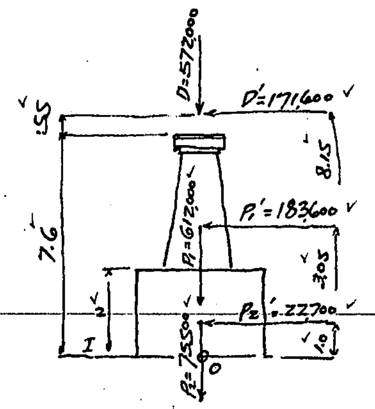


Pressure on each section by Simpson's formula.

A.			Arm from bottom
A	297000 × .75 ÷ 3 =	74200	0.45 m
B	968650 × 1.5 ÷ 3 =	484000	1.32
C	1374860 × 1.5 ÷ 3 =	688000	1.47
D	1258190 × 1.125 ÷ 3 =	471800	~~1.08~~ 1.17
E	782500 × 1.15 ÷ 3 =	300000	1.31
F	219800 × .54 ÷ 3 =	39600	.72
		2057600 kg	
	+ area = 235000 × 13.14 × 2/3 =	2057600 ok.	
G	275800 × .61 ÷ 3 =	56900	0.40
H	1709700 × 1.925 ÷ 3 =	1096000	1.48
I	3609000 × .75 ÷ 3 =	906000	0.71
		2058000 kg.	

Moments at several sections during earthquake.

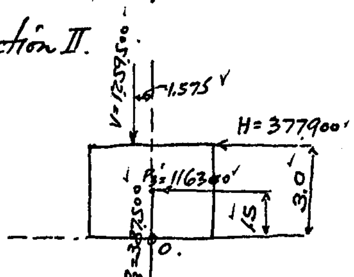
Section I.



Taking moment about point O.

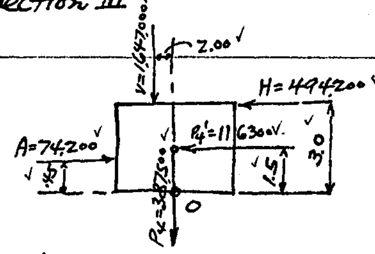
Loads	Hor. forces	Vert. forces	Lev. arms	moment
D		572000	× 0 =	0
D'	171600		× 8.15 =	1399000
P_1		612000	× 0 =	0
P_1'	183600		× 3.05 =	560000
P_2		75500	× 0 =	0
P_2'	22700		× 1.0 =	22700
	H = 377900 kg	V = 1259500 kg	1.575 m	1981700 kgm.

Section II.



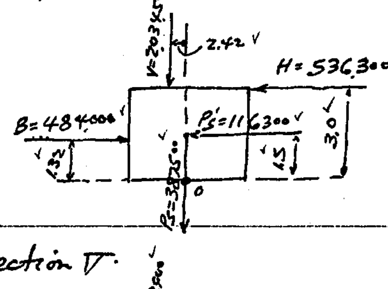
V		1259500	× 1.575 =	1981700
H	377900		× 3.00 =	1134000
P_3		387500	× 0	0
P_3'	116300		× 1.5 =	174500
	494200 kg	1647000 kg	2.00 m	3290200 kgm

Section III



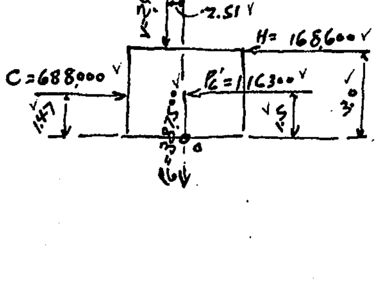
V		1647000	× 2.00 =	3290200
H	494200		× 3.00 =	1482000
P_4		387500	× 0 =	0
P_4'	116300		× 1.50 =	174500
A	− 74200		× .45 =	− 33400
	536300 kg	2034500 kg	2.42 m	4913300 kgm

Section IV.



V		2034500	× 2.42 =	4913300
H	536300		× 3.00 =	1610000
P_5		387500	× ~~1.50~~ =	0
P_5'	116300		× 1.50 =	174500
B	− 484000		× 1.32 =	− 639000
	168600 kg	2422000 kg	2.51 m	6058800 kgm

Section V.



V		2422000	× 2.51	6058800
H	168600		× 3.00	506000
P_6		387500	× 0	0
P_6'	116300		× 1.5	174800
C	− 688000		× 1.47	− 1011000
	− 403100	2809500	2.04 m	5728300 kgm

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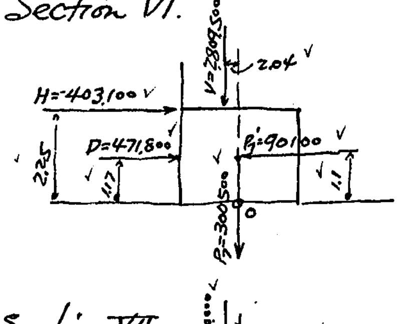
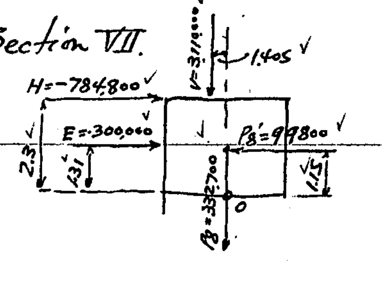
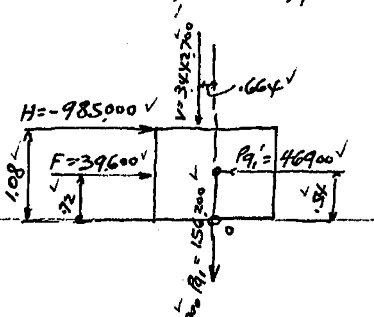
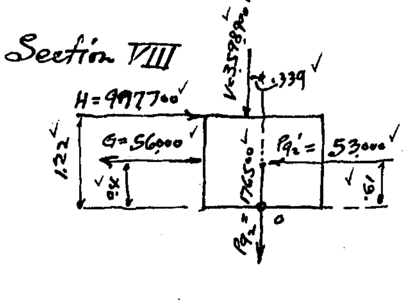
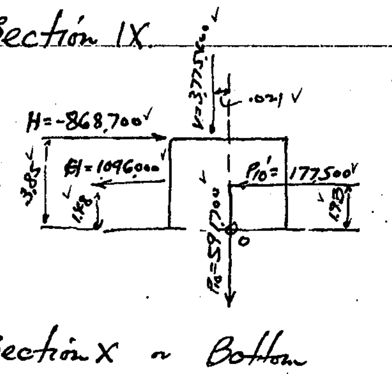
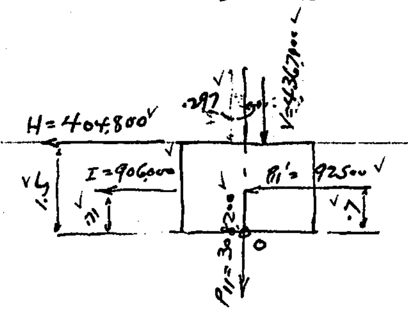
CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie Ken.

	Loads	Hor. forces	Vert. forces	Lev. arms	Moments.
Section VI.	V		2,809,500	2.04	5,728,300
	H	-403,100		2.25	-907,000
	P_7		300,500	0	0
	P_7'	90,100		1.10	99,100
	D.	-471,800		1.17	-551,800
		H= -784,800 kg	V: 3,110,000 kg	1.405	4,368,600 kgm
Section VII.	V		3,110,000	1.405	4,368,600
	H	-784,800		2.30	-1,804,900
	P_8		332,700	0	0
	P_8'	99,800		1.15	114,800
	E	-300,000		1.31	-393,000
		-985,000 kg	3,442,700 kg	.664 m	2,285,500 kgm
Section at ⅓ point from bottom	V		3,442,700	.664	2,285,500
	H	-985,000		1.08	-1,064,500
	P_{9_1}		156,200	0	0
	P_{9_1}'	46,900		.54	25,300
	F	-39,600		.72	-28,500
		-977,700 kg	3,598,900 kg	.339 m	1,217,800 kgm.
Section VIII	V		3,598,900	.339	1,217,800
	H	-977,700		1.22	-1,192,500
	P_{9_2}		176,500	0	0
	P_{9_2}'	53,000		.61	32,300
	G	56,000		.40	22,400
		-868,700 kg	3,775,400 kg	.021 m	80,000 kgm
Section IX.	V		3,775,400	.021	80,000
	H	-868,700		3.85	-3,342,000
	P_{10}		591,700	0	0
	P_{10}'	177,500		1.93	343,000
	H	1,096,000		1.48	1,623,000
		404,800 kg	4,367,100 kg	-0.297	-1,296,000 kgm
Section X ~ Bottom	V		4,367,100	-.297	-1,296,000
	H	404,800		1.50	607,000
	P_{11}		308,200	0	0
	P_{11}'	92,500		.70	64,700
	I	906,000		.71	643,000
		1,403,300 kg	4,675,300 kg	.004 m	-18,700 kgm

計算ノ誤差ニテ 0トナルベキモノナリ

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MADE BY K. Inaba DATE 5-7-8 FILE NO Sub

CHECKED BY M. Jinda DATE 5-7-10 PAGE NO 49

CALCULATIONS FOR

Design of Ibi-Nagara Bashi for Mie Ken.

Vertical Reinforcements of caisson.

max. moment. = 6,058,800 kgm at section IV
vertical load = 2,422,000 kg
less wt. of water = - 336,600
2,085,400 kg net for shell only.

Sectional area of caisson = 38.2 m² (page 28)

Direct compression on concrete = $\frac{2085400}{38.2}$ = 54,600 kg/m² or 5.46 kg/cm²

Bending stress = $\frac{6058800}{4.5}$ = 1,345,000

1345000 ÷ 14.0 = 96,100 kg T or C per lin meter of side wall.

Direct compression 5.46 × 100 × 90 = 49,200 " C "
46,900 kg T " on steel
or 145,300 kg C " on concrete.

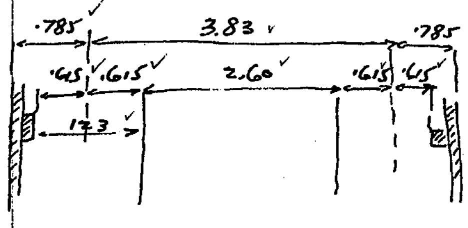
Unit compression on concrete f_c = $\frac{145300}{90 \times 100}$ = 16.15 kg/cm² ok.

Steel area required for tension = $\frac{46900}{1200 \times 1.8}$ = 21.70 cm² per meter of side wall.

use 25 mm φ bars at 45 cm c to c = 21.82 cm² " " ok.

at section VI.

~~max.~~ moment = 4,368,600 kgm
vertical load = 3,110,000 kg
less wt. of water = - 525,800
2,584,200 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 sectional area of concrete (effective).

Direct compression on concrete = $\frac{2584200}{43.64}$ = 59,200 kg/m² or 5.92 kg/cm²

Bending stress = $\frac{4368600}{3.83}$ = 1,140,000

1,140,000 ÷ 14.0 = 81,500 kg T or C per lin m of side wall.

Direct unit compression = 5.92 × 100 × 123 = 72,800
8,700 kg T " on steel.
154,300 " C " on concrete.

Unit compression on concrete = $\frac{154300}{100 \times 123}$ = 12.55 kg/cm² ok.

Steel area required for tension = $\frac{8700}{1200 \times 1.8}$ = 4.03 cm² per lin meter of wall.

use same reinforcements as at section IV. (25φ - 45 cm c to c.)
or (25φ - 50 ")

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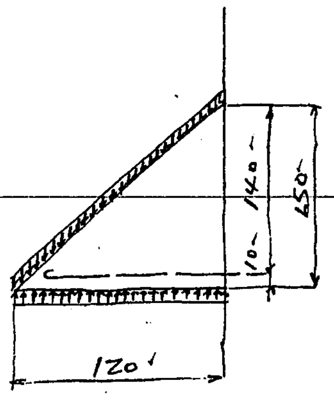
MADE BY K. Inaba DATE 5-7-8 FILE NO Sub

CHECKED BY M. Iinida DATE 5-7-10 PAGE NO 50

CALCULATIONS FOR

Design of Ibi-nagara Bashi for Mie-Ken.

Reinforcements for spread base.



max. upward pressure = 30,000 kg/m^2

Earth on footing assumed 14×1600 = $\underline{22400}$ (including wt of footing)

7600

moment on footing = $\frac{7600}{2} \times 1.2^2 = 5470$ kgm per meter strip.

Shear " " = $7600 \times 1.2 = 9120$ kg

Effective depth required = $\sqrt{\frac{\cancel{9120}^{547} \times 100}{100 \times 7.18}} = 27.6$ cm

use Effective depth of 140 cm with 10 cm insulation.

Steel area required = $\frac{5470 \times 100}{1200 \times \frac{7}{8} \times 140} = 3.73$ cm^2

use 22 mm ϕ bars at 60 cm c to c = 6.34 cm^2

Steel ratio $p = \frac{6.34}{100 \times 140} = .00045$ $j = .95$ about.

unit shear = $\frac{9120}{100 \times .95 \times 140} = 0.69$ kg/cm^2 ok.

unit bond = $\frac{9120}{\frac{6.91}{.6} \times .95 \times 140} = 5.96$ kg/cm^2 ok.

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MADE BY M. Jinda DATE 5-8-1 FILE NO. Sub.
CHECKED BY M. Kojima DATE 5-8-2 PAGE NO. 51

CALCULATIONS FOR

Materials of Ibi Nagara Bashi for Mie-ken

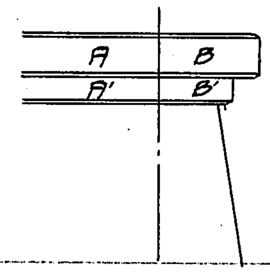
Materials of shaft for P1, P2, P3, P4, P5, P6 & P7

Shaft for P8, P9, P10, P11, P12, P13 & P14 are identical to shaft for P7, P6, P5, P4, P3, P2 & P1 respectively

concrete for shaft 1:2:4 mixture

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	



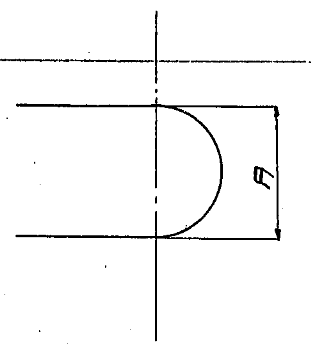
Shaft

Top section

Rectangular	2.10	9.20	1	19.320
Circular	2.10ϕ		1	3.464
				22.784 sq.m

Bottom sections

Piers	A rectangular				circular end		Bottom section
P1	3.051 ×	9.20	28.069	+	7.311	=	35.380 sq.m
P2	3.083	〃	28.364	+	7.465	=	35.829
P3	3.110	〃	28.612	+	7.596	=	36.208
P4	3.131	〃	28.805	+	7.699	=	36.504
P5	3.147	〃	28.952	+	7.778	=	36.730
P6	3.158	〃	29.054	+	7.833	=	36.887
P7	3.163	〃	29.100	+	7.858	=	36.958

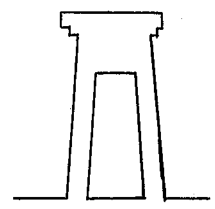


Top section of hollow

2@1.11 × 2.45 = 5.439 sq.m

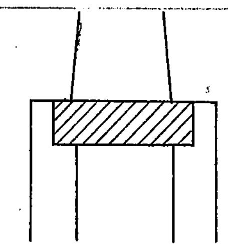
Bottom sections of hollow

Piers	bottom width		length		req'd no		area
P1	1.851	×	2.45	×	2	=	9.070
P2	1.883	×	〃	×	〃	=	9.227
P3	1.910	×	〃	×	〃	=	9.359
P4	1.931	×	〃	×	〃	=	9.462
P5	1.947	×	〃	×	〃	=	9.540
P6	1.958	×	〃	×	〃	=	9.594
P7	1.963	×	〃	×	〃	=	9.619



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

Piers	Top area	Bott a.	mean a	Height	Volume	hollow (less) Top a	Bottom a	Mean a	Height	Volume	Coping	Slab	Total Volume
P1	22.784	35.380	29.082	4.755	138.285	5.439	9.070	7.255	3.705	-26.880	11.626	115.590	238.621 cub.m
P2	〃	35.829	29.307	4.915	144.044	〃	9.227	7.333	3.865	-28.342	〃	〃	242.918
P3	〃	36.208	29.496	5.048	148.896	〃	9.359	7.399	3.998	-29.581	〃	〃	246.531
P4	〃	36.504	29.644	5.155	152.815	〃	9.462	7.451	4.105	-30.586	〃	〃	249.445
P5	〃	36.730	29.757	5.235	155.778	〃	9.540	7.490	4.185	-31.346	〃	〃	251.648
P6	〃	36.887	29.836	5.288	157.773	〃	9.594	7.517	4.238	-31.857	〃	〃	253.132
P7	〃	36.958	29.871	5.315	158.764	〃	9.619	7.529	4.265	-32.111	〃	〃	253.869

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MADE BY M. Iida DATE 5-7-1 FILE NO Sub

CHECKED BY M. Kojima DATE 5-7-2 PAGE NO 52

Revised 5-8-20

CALCULATIONS FOR

Materials of Ibi Nagara Bashi for Mie-Ken

Forms for shaft

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 slab
Side 2 @ 9.20 = 18.40
Circular end 1 @ 2.10φ 6.60
25.00 m

Total length of perimeter at bottom of shaft

Piers	Circular end	Perimeter	Side	Total Perimeter
P1	3.051φ	9.59	18.40	27.99 m
P2	3.083	9.69	〃	28.09
P3	3.110	9.77	〃	28.17
P4	3.131	9.84	〃	28.24
P5	3.147	9.89	〃	28.29
P6	3.158	9.92	〃	28.32
P7	3.163	9.94	〃	28.34

Top length of hollow
4 @ 2.45 + 4 @ 1.11 = 14.24 m

Bottom length of hollow

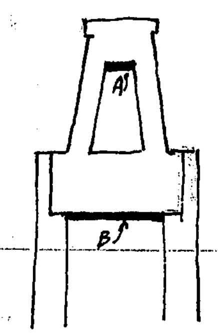
P1	4 @ 2.45	+	4 @	1.851	=	17.20 m
P2	〃	+	4 @	1.883	=	17.33
P3	〃	+	4 @	1.910	=	17.44
P4	〃	+	4 @	1.931	=	17.52
P5	〃	+	4 @	1.947	=	17.59
P6	〃	+	4 @	1.958	=	17.63
P7	〃	+	4 @	1.963	=	17.65

Total area of forms

Piers	Top P	Bottom P	Mean P	Height	Area sq m	Top P	Bottom P	Mean P	Height	Area	Coping	Total area
P1	25.00	27.99	26.50	4.755	126.01	14.24	17.20	15.72	3.705	58.24 m²	15.56 m²	199.81 sq m
P2	〃	28.09	26.55	4.915	130.49	〃	17.33	15.79	3.865	61.03	〃	207.08
P3	〃	28.17	26.59	5.048	134.23	〃	17.44	15.84	3.998	63.33	〃	213.12
P4	〃	28.24	26.62	5.155	137.23	〃	17.52	15.88	4.105	65.19	〃	217.98
P5	〃	28.29	26.65	5.235	139.51	〃	17.59	15.92	4.185	66.63	〃	221.70
P6	〃	28.32	26.66	5.288	140.98	〃	17.63	15.94	4.238	67.55	〃	224.09
P7	〃	28.34	26.67	5.315	141.75	〃	17.65	15.95	4.265	68.03	〃	225.34

埋込型枠

Top of hollow 2 @ 1.11 × 2.45 = 5.44 (A)
Bottom of slab 3 @ 3.60 × 3.60 = 38.88 (B)
Less fillets 6 @ .50 × .50 = -1.50
42.82 sqm



Reinforcements, Plain bars

P1 6.218 kg tons
P2, P3 @ 6.308 〃
P4, P5, P6 & P7 @ 6.487 〃

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MADE BY M. Imida DATE 5-6-15 **FILE NO** sub
CHECKED BY M. Kojima DATE 5-6-16 **PAGE NO** 53
Revised 5-8-20

CALCULATIONS FOR

Materials of Ibi-Nagara-Bashi for Mie-ken

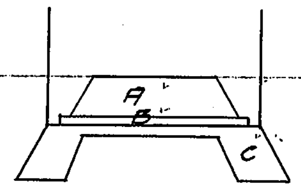
Materials of land caisson

Concrete ~~fill~~ in base 1:3:6 mixture

	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	

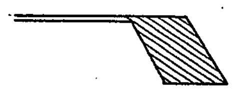
Concrete in working chamber 1:3:6 mixture

	Top area	Bottom area	mean	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 mixture) 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 mixture)

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 part 16.40m 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 part 2.20m

Shell cross sectional area 38.22
Less 2 @ 1.03 × 1.03 = -2.12
36.10 × 2.20 = 79.420 cub.m

Shell top part 2.00m

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 cub.m
Less embeded timber .10 × .20 × 35.36 = -.707
30.673 cub.m

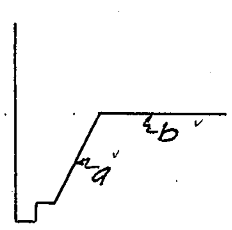
Concrete filling in partition hole (5m depth)

$[3.60^2 \cdot 3 - (1.22\phi + .91\phi + .50^2 \cdot 6)] \cdot 5 = 17.781$ cub.m

grand summary of Concrete for caisson 902.153 cub.m (1:2:4 mixture)

Forms

Inner face of working chamber a $2 @ \frac{11.60+12.60}{2} \times 1.65 = 39.93$
" a $2 @ \frac{3.00+4.00}{2} \times 1.65 = 11.55$
" b $3.00 \times 11.60 = 34.80$
less material shaft 1.22φ = -1.17
less man shaft .91φ = -.65
84.46 sq.m



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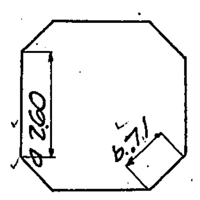
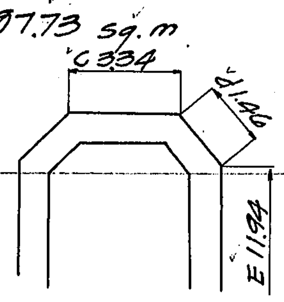
MADE BY M. Jmida DATE 5-6-15 **FILE NO** sub
CHECKED BY M. Kojima DATE 5-6-16 **PAGE NO** 54
Revised 5-8-24

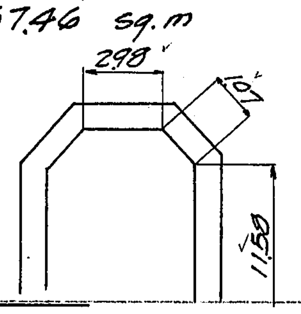
CALCULATIONS FOR

Materials of Ibi-Nagara-Bashi for Mie-Ken

outer face of working chamber		38.80 × 2.59	= 100.49 sq.m
outer side of shell 16.40ᵐ 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 2.20ᵐ 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 2.20ᵐ depth		39.72 × 2.20 = 87.38	
			167.46 sq.m
outside of shell top 2.00ᵐ 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	1,779.74 sq.m
Reinforcements, plain bars	52.356 Kg.tons
Curb shoe structural steel	5.139 Kg.tons

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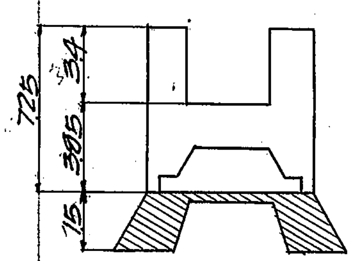
MADE BY M. Kojima DATE 5-6-15 FILE NO sub
CHECKED BY M. Iida DATE 5-6-16 PAGE NO 55
Revised 5-8-21

CALCULATIONS FOR

Materials of Ibi-Nagara-Bashi for Mie-Ken

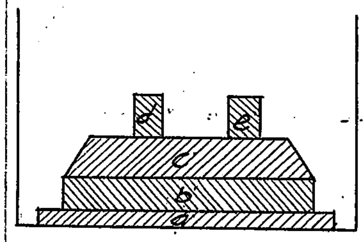
Materials of River caisson
Footing caisson for 7.25m high
Concrete 1:3:6 mixture
Base (1.5m depth)

(14.0 × 5.4 + 16.4 × 7.8) × 1/2 × 1.5 = 152.640
Less (10.6 × 2.0 + 12.4 × 3.8) × 1/2 × 1.35 = -46.116
106.524 cub.m



In working chamber (2.00m depth)

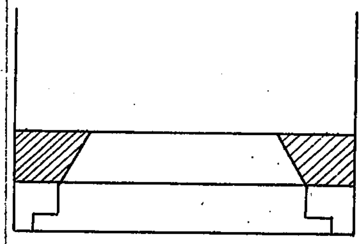
a	13.5 × 4.9 × .3	= 19.845	
Less	(.12 × .2 × 2.4 + .12 × .12 × 4) × 2.4 =	- 0.152	(木材)
〃	.3 × .3 × 4.9 × 4	= - 1.764	(〃)
b	12.6 × 4.0 × .3	= 15.120	
c	(12.6 × 4.0 + 11.1 × 2.5) × 1/2 × 1.4	= 54.705	
Less	.2 × .2 × 2.5 × 4	= - .400	(木材)
d	1.22² × 1.85	= 2.163	
e	.91² × 1.85	= 1.203	
		90.720 cub.m	



Summary 197.244 cub.m

Concrete 1:2:4 mixture
Wall of working chamber (1.45m depth)

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)

(13.9 × 5.3) × 1.8 = 73.670 sq.m × 1.8 = 132.606

Less	28 @ .2 × .2 × 1.8 =	- 2.016		(側柱)
〃	4 @ 〃 〃 13.26 =	- 2.122		(切張 F1, F2)
〃	4 @ 〃 〃 2.68 =	- .429		(〃 F6)
〃	18 @ 〃 〃 4.65 =	- 3.348		(〃 F5)
〃	96 @ .2 × .4 × .075 =	- .576		(切張端継材 I1)
〃	23 @ .2 × .2 × .8 =	- .736		(竪材 E1)
〃	14 @ .2 × .2 × .2 =	- .112		(〃 上部)
〃	6 @ 〃 〃 × .25 =	- .060		(斜材 C1, 上部)
〃	12 @ 〃 × 1.1 × .075 =	- .198	- 15.729	(〃 C2)
〃	36 @ 〃 × 2.0 × 〃 =	- 1.080		(〃 C5)
〃	24 @ 〃 × .3 × 〃 =	- .108		(猫木 C3)
〃	12 @ 〃 × .5 × 〃 =	- .090		(筋違 H1, 下部)
〃	2 @ .12 × .65 × 5.3 =	- .827		(側壁横板 PL16)
〃	2 @ 〃 〃 × 13.66 =	- 2.131		(〃 PL15, 16, 17)
〃	2 @ .12 × .2 × 5.3 =	- .254		(〃 布木 G1)
〃	2 @ 〃 〃 × 13.9 =	- .667		(〃 〃 G1, 2)
〃	10 @ .2 × .8 × .075 =	- .120		(筋違 H3, 下部)
〃	2 @ .12 × 11.1 × .26 =	- .693		(作業室側壁横板 PL1)
〃	2 @ .12 × 2.6 × 2.6 =	- .162		(〃 PL9)

116.877 cub.m

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MADE BY M. Kojima DATE 5-6-15 **FILE NO** sub

CHECKED BY M. Jinda DATE 5-6-16 **PAGE NO** 56

Revised 5-8-21

CALCULATIONS FOR

Materials of Ibi-Nagara-Bashi for Mie-Ken

Shell (1 Lot 3.40m 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

Shell (1 Lot 3.40m depth)
Volume 48.19 × 3.4 = 163.846

Less	.12 × .2 × 13.9 × 6 = - 2.002		(布木 G1, G2)
〃	.12 × .2 × 5.3 × 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		(端継材 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~~ 8.45m high
Lower part 7.25m same as above = 304.388 cub.m
Shell (1 Lot 1.20m depth at top)
Sectional area 48.19 sq.m = 57.828
Volume

Less	.12 × .2 × 13.9 × 2 = -.667		(布木 G1, G2)
〃	.12 × .2 × 5.3 × 2 = -.254		(〃 G1)
〃	.2 × .2 × 1.2 × 18 = -.864		(側柱)
〃	.15 × .2 × 3.46 × 2 = -.208	= -2.581	(切張 F3, F4)
〃	.2 × .2 × 2.06 × 5 = -.412		(〃 F5)
〃	.075 × .2 × .4 × 28 = -.168		(端材 I1)
〃	.05 × .2 × .2 × 4 = -.008		(〃 I2)

55.247
304.388
359.635 cub.m

Reinforcements, Plain bars
For 7.25m floating caisson 20.934 kg. tons
For 8.45m 〃 〃 23.552 〃

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CALCULATIONS FOR

MADE BY M. Kojima DATE 5-6-15 **FILE NO** Sub

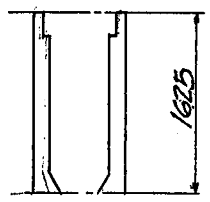
CHECKED BY M. Iida DATE 5-6-16 **PAGE NO** 57

Revised 5-8-21

Materials of Ibi-Nagara-Bashi for Mie-Ken

Materials for shell of River caisson (for all caissons) top 16.25m

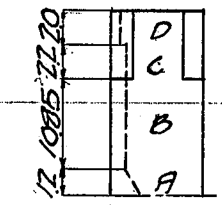
Concrete 1:2:4 mixture



Sectional area

(A) Bottom (1.2m depth) .9 × (140.0 × 2 + 3.6 × 2) = 31.60
 " .7 × 3.6 × 2 = 5.04
 " .5 × .5 × 6 = 1.50 } 38.22
 " fillet .5 × 3.0 × 4 = 6.00

44.22 × 1.2 = 53.064 cub.m



(B) Middle part (10.85m depth)

Sectional area × 10.85 = 38.22 × 10.85 = 414.687 cub.m

(C) Upper part (2.20m depth)

(38.22 − 1.03 × 1.03 × 2) × 2.2 = 36.098 × 2.2 = 79.416 cub.m

(D) 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

Less embeded timber
 15.683 × 2.0 = 31.366
 .10 × .20 × 35.36 = − .707
 30.659 cub.m

Total volume of shell concrete = 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 for all caissons

Summary for River caisson

	26.20m caisson	25.00m caisson
1:2:4 Concrete in floating caisson	359.635	304.388
" shell	577.826	577.826
	937.461 m³	882.214 m³
1:3:6 Concrete in working chamber filling and spread base	197.244 cub.m	197.244 cub.m
Reinforcements in floating caisson	23.552	20.934
" shell	30.211	30.211
	53.763 kg tons	51.145 kg. tons
Floating caisson 図面参照 米松	95.183 cub.m	86.063 cub.m
締付金物	5.066 kg. tons	4.892 kg. tons
Curb shoe	2.644 "	2.644 "

増田淳氏関係資料（独立行政法人 土木研究所蔵）

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MADE BY M. Iida DATE 5-7-1 **FILE NO** sub.
CHECKED BY M. Kajima DATE 5-7-2 **PAGE NO** 58

CALCULATIONS FOR

Revised 5-8-21

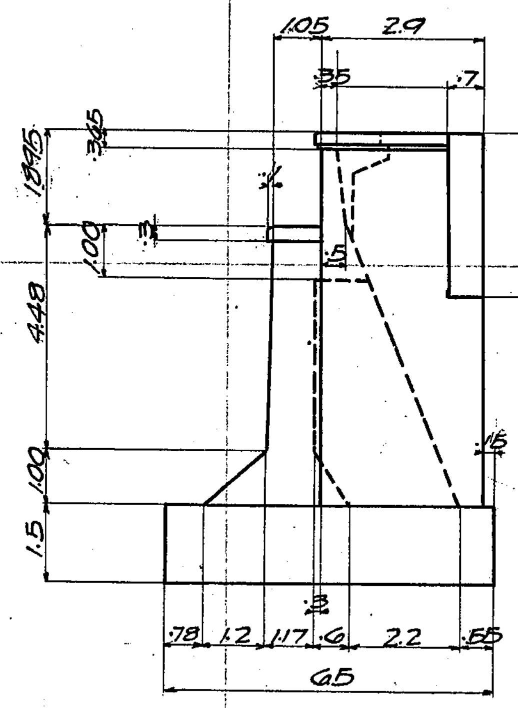
Materials of Ibi-Nagara-Bashi for Mie-Ken

Excavations for Piers
満潮面以下(ケイソン天端ヨリ1.5m上ト假定ス)　River caisson ハ River bed ニ下トス　El. of H.W.L = +7.10~
Sectional area 75.60 sq.m　Depth from H.W.L.

Piers	length	remark	0m-15m	5m-20m	20m-25m	25m以下	base	tatal
P1	25.30m	below high tide	1,134.0	378.0	378.0	22.7	85.2	1,997.9
P2	〃	〃	〃	〃	〃	〃	〃	〃
P3	20.19	below river bed	679.6	〃	〃	90.7	106.5	1,632.8
P4	20.00	〃	665.3	〃	〃	〃	〃	1,618.5
P5	25.30	below high tide	1,134.0	〃	〃	22.7	85.2	1,997.9
P6	〃		〃	〃	〃	〃	〃	〃
P7	〃	〃	〃	〃	〃	〃	〃	〃
P8	〃	〃	〃	〃	〃	〃	〃	〃
P9	〃	〃	〃	〃	〃	〃	〃	〃
P10	〃		〃	〃	〃	〃	〃	〃
P11	21.01	below river bed	741.6	〃	〃	90.7	106.5	1,694.8
P12	25.30	below high tide	1,134.0	〃	〃	22.7	85.2	1,997.9
P13	21.23	below river bed	849.0	〃	〃	0	106.5	1,711.5
P14	25.30	below high tide	1,134.0	〃	〃	22.7	85.2	1,997.9

Materials for East abutment
Concrete 1:2:4 mixture

Parapet wall		.425 × 1.53 × 7.5	=	4.877
〃	(side)	.65 × 1.895 × 2.05 × 2	=	5.050
Wing wall		.35 × 1.55 × 1.895 × 2	=	2.056
〃		〃 × .85 × 4.65 × 2	=	2.767
Column	(outer)	.7 × .95 × 3.3 × 2	=	4.389
〃		〃 × .65 × 7.25 × 2	=	6.598
Less (Stone)		.15 × .2 × .56 × 2	=	-.336
〃	〃	.4 × 〃 × .5 × 2	=	-.080
〃 (top of column)		.44 × 〃 × .7 × 2	=	-.123
Slab under pedestal		.5 × .6 × 1.2 × 2	=	.720
Coping (wing)		.365 × .11 × 4.25 × 2	=	.341
Top beam		1.77 × 1.0 × 10.5	=	18.585
Coping		.3 × .1 × 12.8	=	.384
				45.228
Curtain wall		1.0 × 3.48 × 10.5	=	36.540
Base		2.07 × 1.0 × 〃	=	21.735
〃		6.5 × 1.5 × 12.0	=	117.000
				175.275
Buttress B		(1.16+2.44) × ½ × 3.48 × 2.05 × 2	=	25.682
〃		(2.44+2.20) × ½ × 1.0 × 2.05 × 2	=	9.512
Less		.55 × .3 × 3.6 × 2	=	-1.188
Buttress A		(1.16+2.44) × ½ × 3.48 × 1.0	=	6.264
〃		(2.44+2.20) × 1.0 × 1.0	=	2.320
				42.590



Total Concrete = 263.093 cub.m

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MADE BY M. Kojima DATE 5-6-28 **FILE NO** Sub.

CHECKED BY M. Jinda DATE 5-7-2 **PAGE NO** 59

CALCULATIONS FOR

Materials of Ibi Nagara Bashi for Mie-Ken

Forms

Parapet wall	front & rear	1.53 × 7.5 × 2	= 22.95
〃	side (front)	1.53 × 2.05 × 2	= 6.27
〃	〃 (rear)	1.695 × 1.7 × 2	= 5.76
Bracket	〃 (bottom)	.54 × 1.2 × 2	= 1.30
Wing wall	(outside)	2.9 × 7.01 × 2	= 40.66
〃	(inside)	(2.55+2.40) × ½ × 1.895 × 2	= 9.38
〃	(〃)	(2.40+.40) × ½ × 5.48 × 2	= 15.34
Column	(side)	.365 × .70 × 2	= .51
〃	(outside)	.95 × (3.3 × 2 + .7) × 2	= 13.87
〃	(inside)	.3 × 6.4 × 2	= 3.84
〃	(back)	.65 × 7.2 × 2	= 9.36
Coping	(wing)	.515 × 4.4 × 2	= 4.53
			133.77

Curtain wall	(front)	10.5 × 4.2	= 44.10
〃	(end)	1.26 × 4.18 × 2	= 10.53
Front of wing		.55 × 5.48 × 2	= 6.03
Curtain wall	(rear)	4.54 × 6.5	= 29.51
〃	〃	1.2 × 〃	= 7.80
Bottom of top beam		1.16 × 〃	= 7.54
Coping of 〃		.4 × 12.8	= 5.12
Buttress A & B	(side)	(1.16+2.44) × ½ × 3.48 × 4	= 25.06
〃	(〃)	(2.44+2.2) × ½ × 1.0 × 4	= 9.28
〃 A	(back)	5.80 × 1.0	= 5.80
〃 B	(〃)	〃 × 1.7 × 2	= 19.72
Less 〃	(〃)	.3 × .8 × 2	= - .48
Base		2.07 × 1.0 × 2	= 4.14
Base		1.5 × (24.0+13.0)	= 55.50
			229.65

Total Forms 363.42 sq.m

Reinforcements, Plain bars 68.15 kg.tons

人造洗出仕上

Parapet wall		2.05 × 1.53 × 2	= 6.27
〃	(lower)	.55 × 1.1 × 2	= 1.21
Column	(front)	1.1 × .95 × 2	= 2.09
Wing wall	(side)	(.75+2.65) × ½ × 2.2 × 2	= 7.48
Column	(〃)	.7 × .8 × 2	= 1.12
Coping		.515 × 4.34 × 2	= 4.47
Top face of coping		.15 × 5.59 × 2	= 1.68

Total Finished area 24.32 sq.m

踏掛石, 花崗石 8ヶ × (.25 × .28 × .93) = .521 cub.m

Foundation piles 内地産赤松 末口 21糎長 4.80米 7 × 13 = 91本

Gas pipes 2 - 2½"φ - 100m long.

Rabble for foundation

70 × 12.5 × .60 = 5250 cub.m

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MADE BY M. Kojima DATE 5-7-2 **FILE NO** sub.

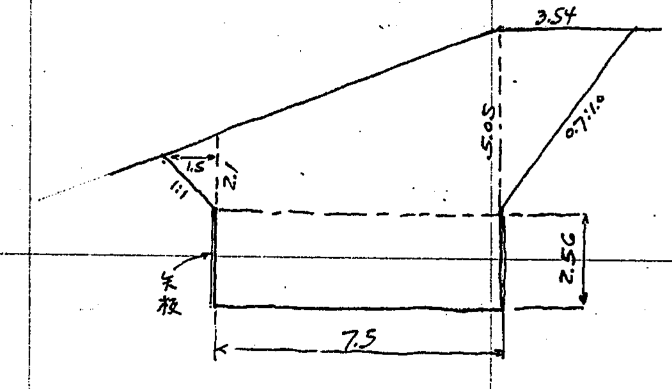
CHECKED BY M. Ymida DATE 5-7-3 **PAGE NO** 60

CALCULATIONS FOR

Materials of Ibi Nagara Bashi for Mie-Ken

Excavation

Average section



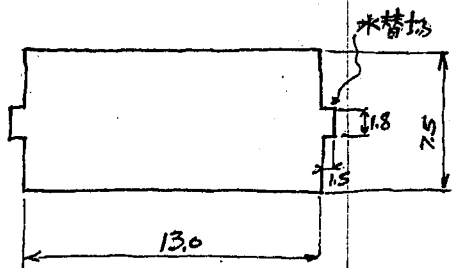
~~Area~~

7.50 × 13.0 × 2.56 = 250. m^3

5.05 × 3.54 ÷ 2 = 8.93
3.58 × 7.5 = 26.85
2.1 × 1.5 ÷ 2 = 1.58
37.36 m^2

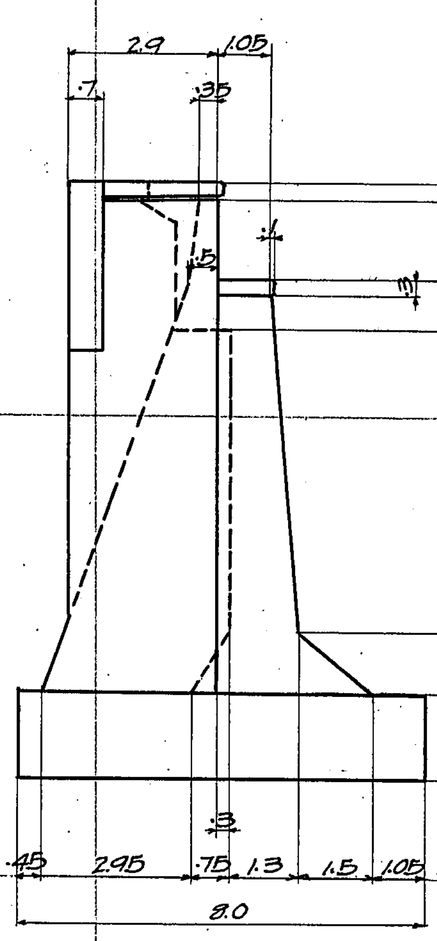
37.36 × 17.0 = 635.
885

水替坊 1.5 × 1.8 × 2.78 × 2 = 15 = 434 ...
900 m^3.



Excav. under L.W. = 265 m^3.
" above " = 635
900

矢板長 (延)
13 × 2 = 26.0
7.5 × 2 = 15.0
1.5 × 4 = 6.0
延長 = 47.0 m



Materials for west abutment
Concrete 1:2:4 mixture.

Parapet wall	4.25 × 1.53 × .75	= 4.877
" (side)	2 × .65 × 1.895 × 2.05	= 5.050
Wing wall	2 × 1.55 × 1.895 × .35	= 2.056
"	2 × .35 × .85 × 4.1	= 2.440
Column (outer)	2 × .7 × .95 × 3.3	= 4.389
"	2 × " × .65 × 6.75	= 6.143
Less (Stone)	2 × .15 × .2 × 5.6	= -.336
" "	2 × .4 × " × .5	= -.080
" (top of column)	2 × .44 × " × .7	= -.123
Slab under pedestal	2 × .5 × .6 × 1.2	= .720
Coping (wing)	2 × .365 × .11 × 4.25	= .341
Top beam	1.8 × 1.0 × 10.5	= 18.900
Coping	.3 × .1 × 12.8	= .384
		44.761

Curtain wall	10.6 × 4.78 × 1.05	= 53.201
Base	24.25 × 1.20 × "	= 30.555
"	8.00 × 1.70 × 12.0	= 163.200
		246.956

Buttress B	(1.22 + .33) × ½ × 4.78 × 2.05 × 2	= 44.291
"	(.33 + 2.95) × ½ × 1.2 × 2.05 × 2	= 15.375
Less	2 × .55 × .3 × 4.90	= -1.617
Buttress A	(1.22 + .33) × ½ × 4.78 × 1.00	= 10.803
"	(.33 + 2.95) × ½ × 1.20 × 1.00	= 3.750
		72.602

Total Concrete 364.319 cub. m

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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MADE BY M. Kojima DATE 5-7-2 **FILE NO** Sub.

CHECKED BY M. Iida DATE 5-7-3 **PAGE NO** 61.

Revised 5-8-21

CALCULATIONS FOR

Materials of Ibi Nagara Bashi for Mie-Ken

Forms

Parapet wall front & rear		1.53 × 7.5 × 2	= 22.95
〃 〃 side	(front)	〃 × 2.05 × 2	= 6.27
〃 〃 〃	(rear)	1.695 × 1.7 × 2	= 5.76
Bracket	(bottom)	.54 × 1.2 × 2	= 1.30
Wing wall	(outside)	2.9 × 8.51 × 2	= 49.36
〃	(inside)	(2.55+2.4)×½×1.895×2	= 9.38
〃	〃	2.40 × ½ × 5.58 × 2	= 13.39
〃	Δ (outside)	.5 × ½ × 1.4 × 2	= .70
Column side		.365 × .70 × 2	= .51
〃	(outside)	.95 × (3.3×2+.7) × 2	= 13.87
〃	(inside)	.3 × 6.1 × 2	= 3.66
〃	(back)	.65 × 7.48 × 2	= 9.72
Coping	(wing)	.515 × 4.4 × 2	= 4.53
			141.4

Curtain wall	(front)	10.5 × 5.5	= 57.75
〃	(end)	1.325 × 5.48 × 2	= 14.52
Front of wing		.55 × 6.98 × 2	= 7.68
Curtain wall	(rear)	5.84 × 6.5	= 37.96
〃	〃	1.40 × 〃	= 9.10
Bottom of top beam		1.22 × 〃	= 7.93
Coping		.4 × 12.8	= 5.12
Buttress A & B (side)		(1.22+3.3)×½×4.78×4	= 43.21
〃 〃 〃		(3.3+2.95)×½×1.2×4	= 15.00
〃 A	(back)	7.6 × 1.0	= 7.60
〃 B	(〃)	〃 × 1.7 × 2	= 25.84
Less		.30 × 1.6 × 2	= -.96
Back of column		.35 × 1.45 × 2	= 1.02
Base		2.35 × 1.2 × 2	= 5.64
〃		1.70 × (2.4+1.6)	= 68.00
			305.41

Total forms 446.81 sq.m

Reinforcements, Plain bars 8829 Kg. tons

人造洗出仕上

Parapet wall		2.05 × 1.53 × 2	= 6.27
〃	(lower)	.55 × 1.4 × 2	= 1.54
Column	(front)	1.1 × .95 × 2	= 2.09
wing wall	(side)	(.2+3.04)×½×2.9×2	= 9.40
Side of column		.365 × .70 × 2	= .51
Coping		.515 × 4.34 × 2	= 4.47
Top face of coping		.15 × 5.59 × 2	= 1.68

Total finished area 25.96 sq.m

踏掛石 花崗石 8ヶ × (.25 × .28 × .93) = .521 cub.m

Foundation piles 内地産赤松末口21糎長サ4.80米 8 × 13 = 104本

Gas pipes 2 - 2½"φ × 1.00m long

Rubble stone of Foundation

8.5 × 12.5 × .60 = 63.75 cub.m

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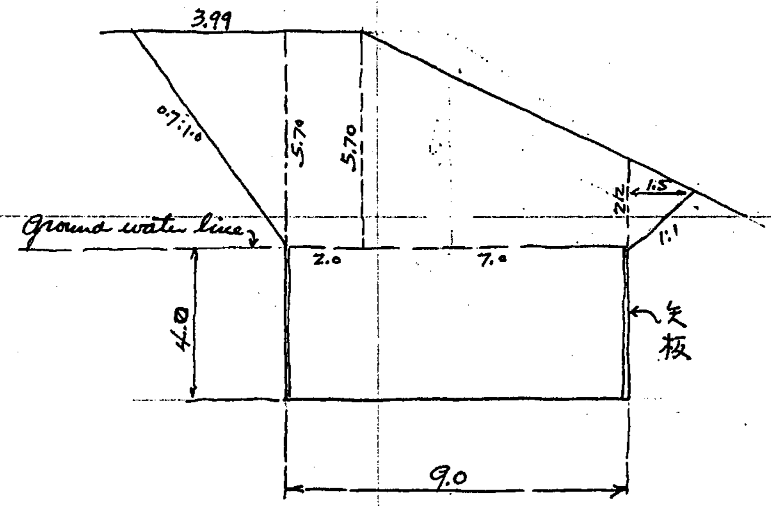
MADE BY K. Inaba DATE 5-7-9 FILE NO sub.
CHECKED BY ________ DATE ________ PAGE NO 62

CALCULATIONS FOR

Materials of Ibi-nagara Bashi for Mie-ken

Excavation

Average section.

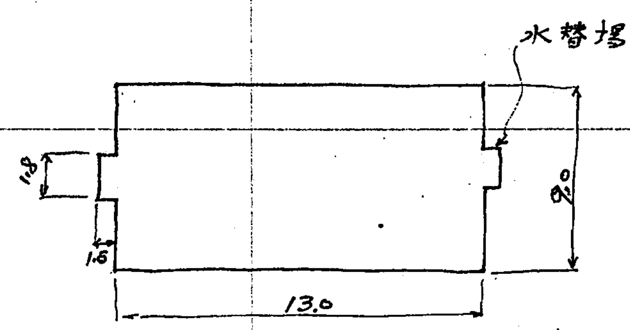


3.99×5.7÷2 = 11.37
5.7×2.0 = 11.40
3.95×7.0 = 27.65
2.2×1.5÷2 = 1.65
52.07 m²

52.07×18.2 = 948 m³ above G.W.

9.0×13.0×4.0 = 468
1.5×1.8×4.3×2 = 23 水替場 = ポンプ
491 below G.W.

total excavation = 1439 m³



矢板 延長
13×2 = 26.0
9×2 = 18.0
1.5×4 = 6.0
50.0 m