昭和四年七月

大阪府十三橋下部工設計々算書

JIUN MASUDA
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JIJI BLDG, TOKYO

MADE BY K. Inaba DATE 4-5-24 FILE NO

CHECKED BY M. Jinda DATE 4-6-10 PAGE NO A1

CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka Prefecture

Design of Abutment.

Datas: Clear width of Roadway 20.0 meters; 5.69 meter Electric light way at center, 4.405 meter carriage ways on both sides and 2.75 meter side walks on both ends.

Span Length. 25.234 meters between End Bearings.

Assumed Loadings

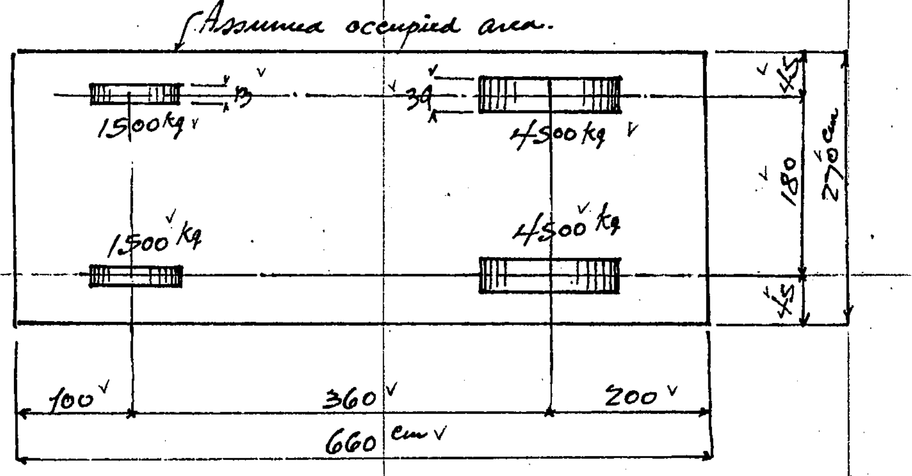
Uniform live load on Roadway $w = \frac{120000}{170+l} \geqq 600$ kg per sq. meter.

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

where l = span length in meters.
w = uniform live load in kg. per square meter.

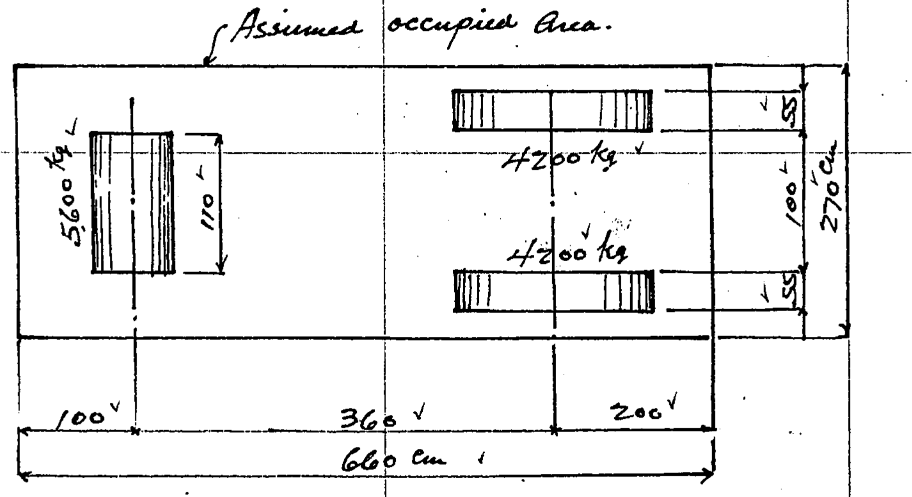
Motor truck and Roadroller Loadings.

12 ton motor truck loading



4 rows of motor traffic on Roadway with occupied width of 270 cm each Space around the assumed occupied Space of motor truck shall be filled with said uniform load.

14 ton Road Roller loading



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

Impact for motor truck and Electric Car loading.

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

No impact for road roller and Uniform live load.

Electric Car Loading

Kinds of Car	Axle distances					Axle load.	Total weight.
4 wheel car class A (continuous)	3 m	0	2 m	0	3 m	7,400 kg	14,800 kg
Water splinker class B (coupled)	2 m	0	2 m	0	2 m	9,000 kg	18,000 kg
Bogie Car class C (continuous)	2.2 m	1.2 m	4.2 m	1.2 m	2.2 m	5,600 kg	22,400 kg
Low platform Bogie car class D (continuous)	2.5 m	1.5 m	5.0 m	1.5 m	2.5 m	7,000 kg	28,000 kg

Impact for Electric car loading as described above.

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CHECKED BY M. Jinda DATE 4-6-18 PAGE NO A2

CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.

Allowable working strength.

Structural steel or Reinforcing Bars.

Tension	1,200 kg/cm²
Extreme fibre stress	1,200 "
Shear of web, gross section	900 "
Compression member	
$1500\,(1-0.0055\,l/r)$ not over	1,000 "

where l = length of member in cm
r = least radius of gyration in cm*

Compression flange of girder	
$1200\,(1-0.012\,l/b)$ not over	1,100 "

where l = unsupported length of flange in cm.
b = width of flange in cm.

Shear on shop driven rivets (machine driven)	850 "
" " field " " and turned bolts (machine driven)	750 "
Shear on pin	900 "
Bearing on shop driven rivets	1,700 "
" " field " "	1,500 "
" " pin	1,800 "

Roller 45d kg per lin cm. where d = dia. in cm.

Concrete 1:2:4 mixture

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

Weight of materials assumed.

cast iron	7250 kg per cub. meter.
steel	7850 " "
cast steel	7860 " "
Reinforced concrete	2400 " "
Plain concrete	2200 " "
cement mortar	1700 " "
Granite	2600 " "
Sand	1700 " "
Earth	1600 " "
asphalt pavement	2100 " "
wood-block pavement	1000 " "

Considering wind or temperature stress in addition to dead, live and impact stresses the allowable working strength increased 25%. In case of considering seismic stress increase unit stress 80%.

Seismic acceleration assumed 3000 kg/sec² or say $k = 0.3$

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MADE BY K. Inaba DATE 4-6-27 FILE NO
CHECKED BY M. Jinda DATE 4-7-24 PAGE NO A3

CALCULATIONS FOR

Design of Jinoo Bashi for Osaka prefecture.
Design of Piers for Girder span.

Piers P_2 to P_5 + P_{12} to P_{15}
Superimposed Loads on pier.
Dead Load :-
Floor.

Conduit pipes and watermains 425 ✓
Handrail 90 ✓
Coping stone + slab under coping 206 ✓
Sidewalk slab including wearing course. 300✓×2.37✓ = 710 ✓
G_1 to curb stone .38×.375@2400 = 342 ✓
Roadway slab, pavement &c. 530✓×4.405✓ = 2335 ✓
Extra concrete at G_2 137 ✓
Electric car track concrete slab. 1.55@24✓ = 372 ✓
" " " Construction 815 ✓
1187✓×2.845✓ = 3380 ✓
Miscellaneous concrete &c say 25 ✓
7650✓×2✓ = 15,300✓ kg per lin meter.

Structural steel for Anchor span + Cantilever arms.
Stringers. S_1 64 ✓
" S_2 52 ✓
" S_3, S_4 + S_5 3@74 = 222 ✓

Cross Beams.
under electric Ry. track 2✓@160✓ = 320 ✓
" " " at panel pt. 485 ✓
cantilever bracket 400 ✓
under roadway 692 ✓
1897✓÷4.089✓ = 464 ✓
802✓×2✓ = 1,604 ✓

Bottom Lateral Bracings 14180 ✓
bracing under sidewalk 500 ✓
14,680✓÷48.318✓ = 304 ✓
main girders assumed 5@1000 = 5000 ✓
6,908✓ Call this 6,900✓ kg/lin m.

Structural steel for Suspended span.
Stringers and floor beams. same as above. 1,604 ✓
Bottom lateral bracings 5000 ✓
Bracing under sidewalk 500 ✓
5,500 ÷ 17.108 = 321 ✓
main girders 5@635 = 3175 ✓
5,100✓ kg/m

Summary of Dead Load on pier
Anchor span with cantilever arms.
Floor complete 15,300 ✓
Structural steel 6,900 ✓
22,200✓ kg/lin m.

Suspended span
Floor complete 15,300 ✓
Structural steel 5100 ✓
20,400✓ kg/lin m.

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CALCULATIONS FOR

Design of Juiso Bashi for Osaka prefecture.

Live Load:-

Uniform load on Roadway $w = \frac{120000}{170+l}$ Loaded length l. taken at 32.712 m on safe side

$= \frac{120,000}{170+32.712} = 592$ use 600 kg/m²

Uniform load on sidewalk $= \frac{100,000}{170+32.712} = 494$ use 500.

Electric car truck loading. Class D. 28,000 ÷ 13 = 2155

Impact. coef. $= \frac{20}{60+32.712} = 21.6\%$ 465

2620 Call this 2700 kg per lin meter for double tracks.

Summary of Live Loads

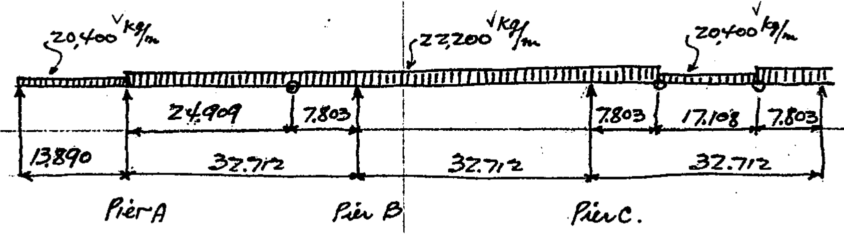
Uniform load on Roadway = 600 × 4.405 × 2 = 5290

" " Sidewalk = 500 × 2.75 × 2 = 2750

Electric car loading 2700 × 2 = 5400

13,440 kg per lin meter.

Dead Load on Piers



Simple spans.

20400 × 13.89 ÷ 2 = 141,700 kg

22200 × 24.909 ÷ 2 = 276,500 kg

20400 × 17,108 ÷ 2 = 174,500 kg

Load on pier A 141,700

276,500

418,200 kg

Load on B. + C. unbalance from cantilever arm 276,500

174,500

102,000

Reaction due to unbalance moment. 102000 × 7.803 ÷ 32.712 = ± 24,400 kg

" " balanced load. 22200 × 32.712 ÷ 2 = 363000

22,200 × 7.803 = 173300

174,500

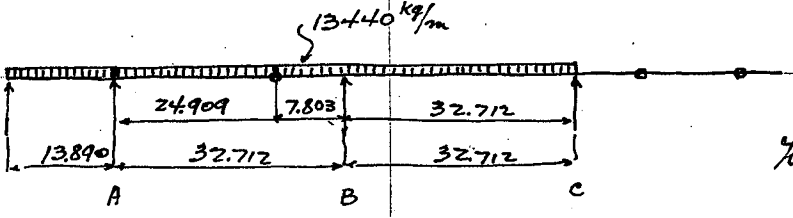
710,800

Load on pier B = 735,200 kg

Load on pier C = 686,400 kg

Live Load on Piers.

Piers A + B



Load on Pier A.

13440 × 13.89 ÷ 2 = 93,300 kg

13440 × 24.909 ÷ 2 = 167,500

260,800 kg Call this 261,000 kg

Load on Pier B

167,500 × 40.515 = 6790,000

$13440 \times 40.515^2 \div 2 =$ 11830,000

18,620,000 ÷ 32.712 = 569,000 kg

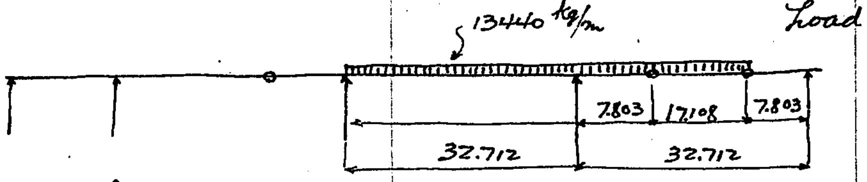
Piers C.

Load on Pier C.

93300 × 40.515 = 3780,000

$13,440 \times 40.515^2 \div 2 =$ 11,830,000

15,610,000 ÷ 32.712 = 477,000 kg



Summary of Dead Load + Live Load on Piers.

	Pier A	Pier B	Pier C.
Dead Load.	418,200	735,200	686,400
wt. of Shoes say	11,800	9800	9600
	430,000	745,000	696,000
Live Load	261,000	569,000	477,000
	691,000 kg	1314,000 kg	1,173,000 kg

Note.

Pier A ---- P1 + P16

Pier B ---- P2, P5, P12 + P15

Pier C ---- P3, P4, P13 + P14

use same details for Pier B and Pier C.

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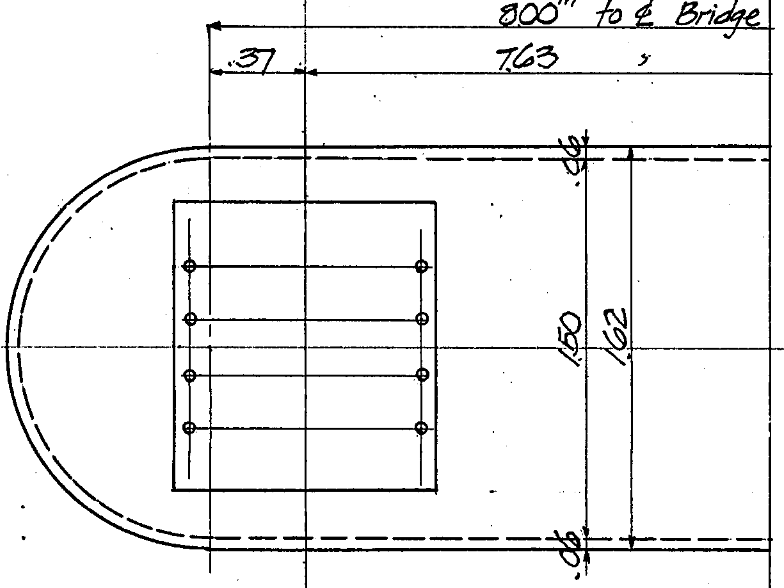
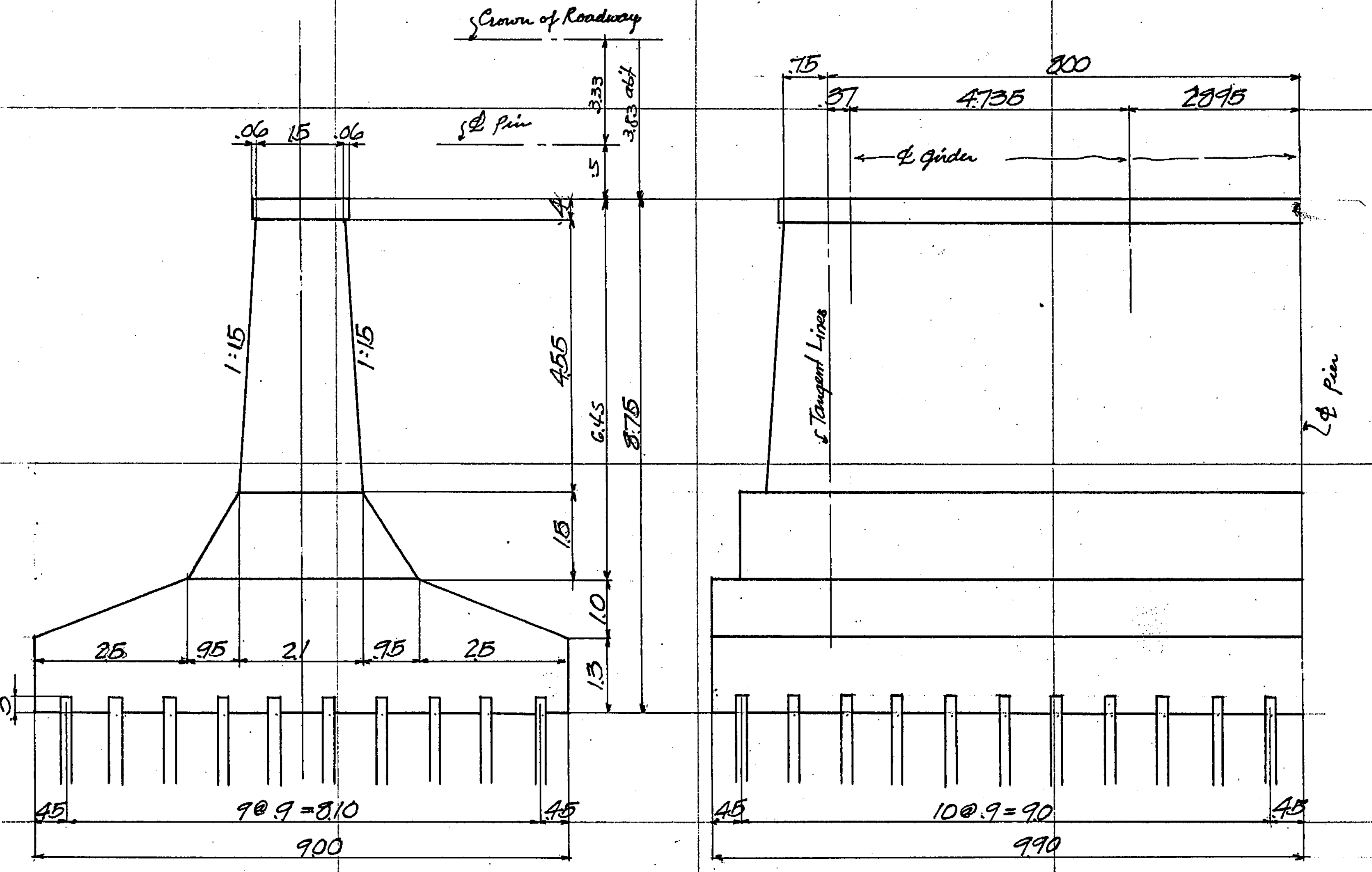
CHECKED BY M. Jinda DATE 4-7-24 PAGE NO A5

CALCULATIONS FOR

Design of Jiuso Bashi for Osaka Prefecture
Design of Pier B&C Piers P2 to P5 and P12 to P15



Detail of Top of Pier.
Scale 1:30.



Sketch of Girder Pier.
Scale 1:100.

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CALCULATIONS FOR

Design of Jiuso Bashi for Osaka prefecture.

Volume and weight of Concrete. 1:2:4 mixture

Coping $1.62 \times 0.4 \times 16.0 = 10.37 \times 4.75 = 49.20$

" $1.62^{\phi} \times 0.4 = 0.83 \times 4.75 = 3.94$

Shaft. $1.80 \times 4.55 \times 16.0 = 131.10 \times 2.15 = 282.00$

" $1.80^{\phi} \times 4.55 = \underline{11.57} \times 2.03 = \underline{23.48}$

$153.87^{m^3} \times \underline{2.33}^{m} = 358.62$ weight $153.87 @ 2400 \doteqdot 369,000$ kg

$153.87 \times 1.5 = \underline{231.00}$

Base. $3.05 \times 1.5 \times 19 = \underline{86.85}^{m^3} \times .705 = \underline{61.20}$

$\underline{\underline{240.72}}^{m^3} \times \underline{\underline{2.70}}^{m} = 650.82$ weight $240.72 @ 2400 = 578,000$ kg

Base. $6.50 \times 1.0 \times 19.8 = 128.70 \times 1.726 = 222.00$

" $1.30 \times 9.0 \times 19.8 = \underline{231.50} \times 0.65 = \underline{150.50}$

$360.20^{m^3} \times \underline{\underline{1.03}}^{m} = 372.50$ weight $360.20 @ 2400 = \underline{864,000}$ kg

Total concrete for one pier $= \underline{\underline{600.92}}^{m^3}$ Total wt of one pier $= 1,442,000$ kg.

Superimposed Load on pier

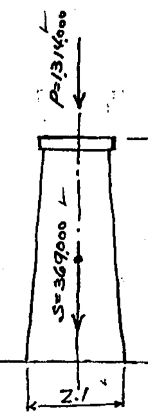
Dead Load. 745,000 kg Seismic force $= 745,000 \times .3 = 223,500$ kg.

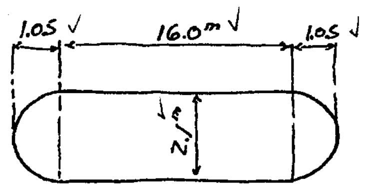
Live Load $\underline{569,000}$

1,314,000 kg

Stability of Shaft.

Case 1. Stability at Normal State.





Bottom area.

$2.1 \times 16.0 = 33.60$

$2.1^{\phi} = \underline{3.46}$

$37.06^{\square m}$

Loads on Shaft.

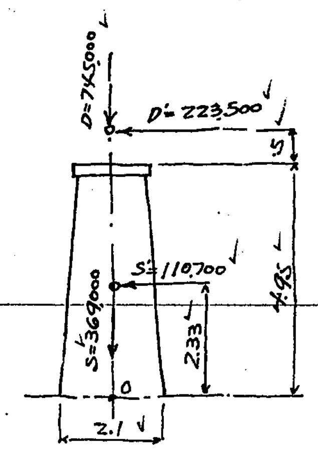
Superimposed Dead + Live Load $= 1,314,000$

weight of Shaft $= \underline{369,000}$

1,683,000 kg

Unit compression on concrete $= \dfrac{1683000}{37.06 \times 10,000} = 4.54$ kg/cm² ok.

Case 2. Stability during Earthquake. $k = 0.3$ assumed.



Taking moment about center of bottom area. 0.

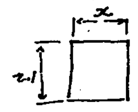
Loads.	Horizontal forces	Vertical forces	Lever arm	moment.
D		745,000 ×	0 =	0
D'	223,500	×	5.45 =	1,218,000
S		369,000 ×	0 =	0
S'	$\underline{110,700}$	×	2.33 =	$\underline{257,500}$
	ΣH = 334,200 kg	ΣV = 1,114,000 kg ÷	$\underline{1.325}^{m}$ =	1,475,500 kgm

Bottom section.

Equivalent rectangle of same moment of inertia

moment of inertia of circle $2.1^{\phi} = 0.0491 \times 2.1^4 = 0.955^{m^4}$

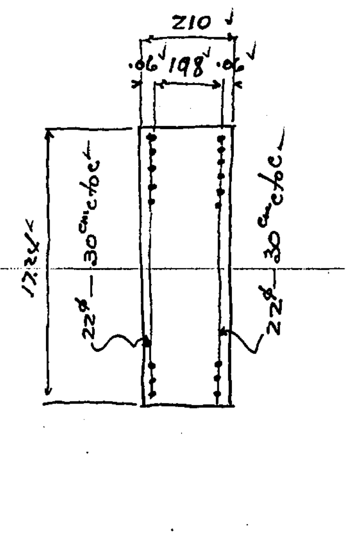
rectangle



moment of inertia $\dfrac{x \times 2.1^3}{12} = 0.772x$.

$0.772x = 0.955$

$x = \dfrac{0.955}{0.772} = 1.24$



Total length of rectangle $= 1.24 + 16.0 = 17.24^{m}$. area $= 17.24 \times 2.1 = 36.2^{\square m}$

Try reinforcement

22^{ϕ} bars at 30cm c to c. $= 3.801 \times 3.33 \times 17.24 = 218.1$ cm²

$e/h = 1.325/210 = .63$, $d'/h = 6/210 = .029$, $p_0 = 2p = \dfrac{218.1 \times 2}{362,000} = .00121$

From the prepared diagrams of Combined Stress. (direct & bending)

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MADE BY K. Inaba DATE 4-6-28 FILE NO

CHECKED BY M. Jinda DATE 4-7-25 PAGE NO A7

CALCULATIONS FOR

Design of Jiuso Bashi for Osaka prefecture.

$k = .228,\quad L = .068$

$f_c = \dfrac{M}{L.b.h^2} = \dfrac{1475500 \times 100}{.068 \times 1724 \times 210^2} = 28.6$ kg/cm² Ok

$f_s = nf_c\left(\dfrac{d}{kh} - 1\right) = 15 \times 28.6\left(\dfrac{204}{.228 \times 210} - 1\right) = 1400$ kg/cm² $< 1200 \times 1.8 = 2160$ ok.

unit shear $= \dfrac{334200}{1724 \times \frac{7}{8} \times 204} = 1.09$ kg/cm² ok

unit bond $= \dfrac{334200}{6.91 \times 3333 \times 1724 \times \frac{7}{8} \times 204} = 4.72$ kg/cm² ok.

Assumed section is ample.

Section at 6.45 meters below top of pier.

Case 1. Stability at Normal State.

Bottom area. $4.0 \times 19.0 = 76.0$ sq.m $= 760{,}000$ cm²

Load.

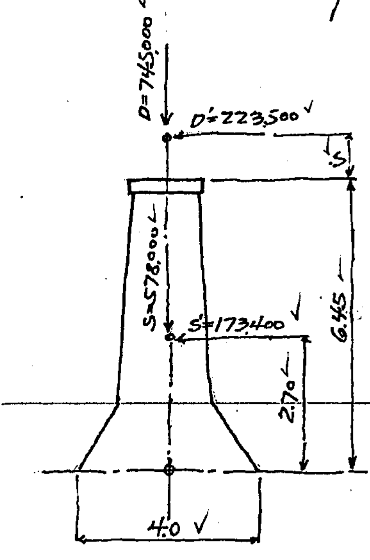
Superimposed D.L. + L.L. = 1,314,000

wt. of pier Above this section = 578,000

1,892,000 kg

Unit compression on concrete $= \dfrac{1892000}{760{,}000} = 2.49$ kg/cm² ok.

Case 2. Stability during Earthquake.



Loads	Hor. forces	Vert. forces	Lev. arm	Moment.
D		745,000	× 0 =	0
D'	223,500		× 6.95 =	1,553,000
S		578,000	× 0 =	0
S'	173,400		× 2.70 =	468,000
	396,900 kg	1,323,000 kg	× 1.53 m =	2,021,000 kgm.

$e/h = 1.53/4.0 = .383,\quad d'/h = 6/400 = 0.015$

Try reinforcements ~~22~~ 19φ bars at 30 cm c to c $= 2.835 \times 3.333 \times 19.0 = 179.5$ cm²

$p_0 = 2p = \dfrac{179.5 \times 2}{400 \times 1900} = .0005$

From the prepared diagrams, $K = 0.423,\ L = .079$

$f_c = \dfrac{2021{,}000 \times 100}{.079 \times 1900 \times 400^2} = 8.42$ kg/cm² ok.

$f_s = 15 \times 8.42\left(\dfrac{394}{.423 \times 400} - 1\right) = 168$ kg/cm² ok.

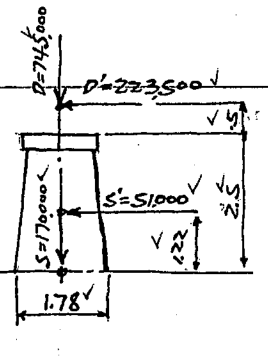
Section at 2.5 meter below top.

Concrete coping $11.20 \times 2.3 = 25.75$

Shaft $1.64 \times 2.1 \times 16.0 = 55.05 \times 1.02 = 56.15$

" $1.64^2 \times 2.1 = 4.43 \times .99 = 4.39$

$70.68 \times 1.22 = 86.29$ weight $70.68 \times 2400 = 170{,}000$ kg



Loads	Hor. forces	Vert. forces	Lev. arm	Moment.
D		745,000	× 0 =	0
D'	223,500		× 3.0 =	670,000
S		170,000	× 0 =	0
S'	51,000		× 1.22 =	62,200
	274,500 kg	915,000 kg	× 0.8 m =	732,200 kgm

$e/h = 80/178 = .45\qquad d'/h = 6/178 = .034$

Reinforcement Try, 19φ bars at 30 cm c to c. $= 2.835 \times 3.333 \times 17. = 161.5$ cm²

$p_0 = 2p = \dfrac{2 \times 161.5}{178 \times 1700} = .0011,\quad K = .363,\quad L = .079$

$f_c = \dfrac{732{,}200 \times 100}{.079 \times 1700 \times 178^2} = 17.2$ kg/cm² ok

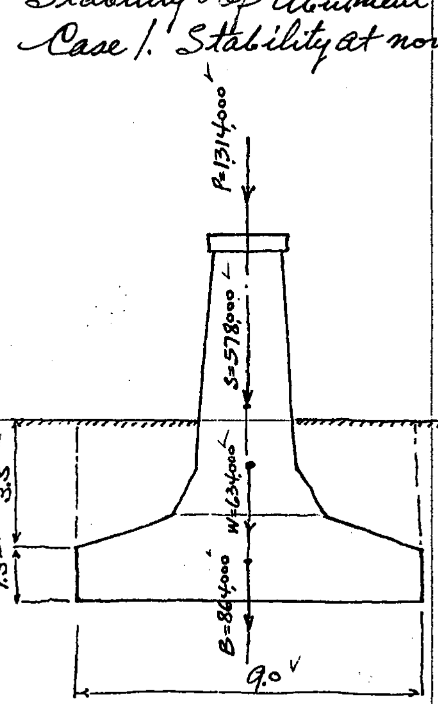
$f_s = 15 \times 17.2\left(\dfrac{172}{.363 \times 178} - 1\right) = 428$ kg/cm² ok.

unit shear $= \dfrac{274500}{1700 \times \frac{7}{8} \times 172} = 1.07$ " ok unit bond $= \dfrac{274500}{5.97 \times 3.333 \times 17 \times \frac{7}{8} \times 172} = 5.4$ kg/cm² ok.

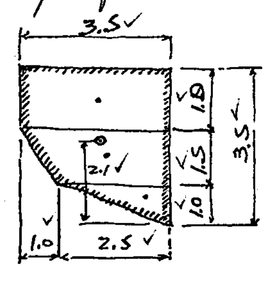
Design of Jinso Bashi for Osaka prefecture.

Stability of Abutment as a whole.

Case 1. Stability at normal state.



weight of earth on Base.



3.5×1.0 = 3.5
3.0×1.5 = 4.5
2.5×.5 = 1.25
9.25^{om}×19.8 @ 1600 = 296,000 kg = $\frac{W}{2}$
earth on end 1.30×4.0×2.5 @ 1600 = 21,000
317,000 kg = $\frac{W}{2}$

~~3.5×3.0 = 10.50~~
~~4.5×1.79 = 8.05~~
~~1.25×.66 = .83~~
~~9.36×2.1 = 19.38~~

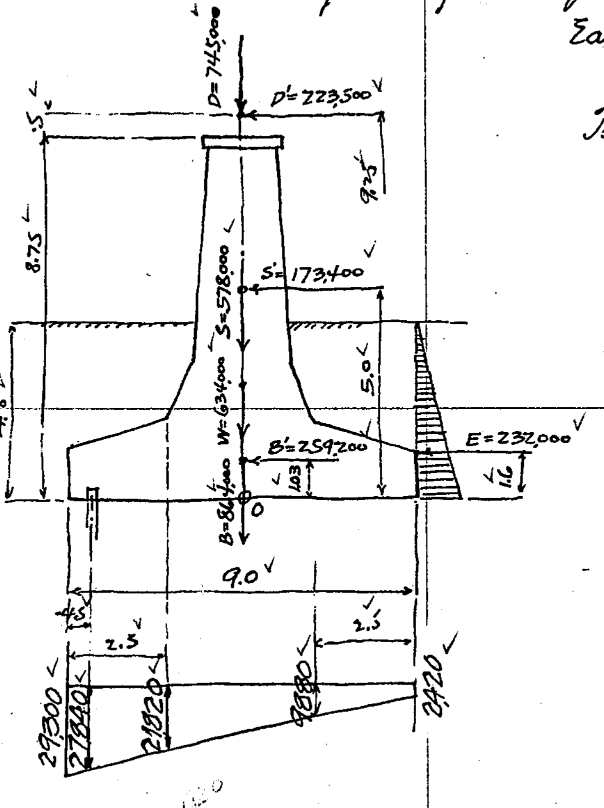
max Loads.
Superimposed D.L.+L.L. = 1,314,000 = P
wt. of Shaft = 578,000 = S
wt. of Earth on base = 634,000 = W
wt. of Base = 864,000 = B
3390,000 kg.

Bottom area of Base.
= 9.0×19.8 = 178.2^{om}

Unit bearing pressure = $\frac{3390,000}{178.2}$ = 19,030 kg/m² (or 1.74 tons/□)

max. load on one pile = 19.03×.9×.9 = 15.4 kg tons.

Case 2. Stability during Earthquake K=0.3 assumed



Earth pressure during Earthquake = $\frac{0.662\,wh^2}{2}$ = $\frac{.662\times1600\times4.8^2}{2}$ = 12,200 kg/m
E = 12200×19.0 = 232,000 kg

Taking moment about center of Base O.

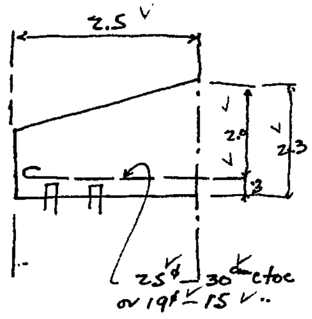
Loads	Hor. forces	Vert. forces	Lever arm	moment.
D		745,000	0	0
D'	223,500		9.25	2070,000
S		578,000	0	0
S'	173,400		5.00	867,000
W		634,000	0	0
B		864,000	0	0
B'	259,200		1.03	267,000
E	232,000		1.60	371,000
	ΣH= 888,100 kg	ΣV= 2,821,000 kg	1.27 m	3,575,000 kgm

Resultant force within middle third.

max. toe pressure = $\frac{2,821,000}{19.8\times9.0}\left(1\pm\frac{6\times1.27}{9.0}\right)$
= 29,300 kg/m² C or (2.68 tons/□)
or = 2,420 " C

max. Load on one pile = 28,240×.9×.9 = 22.9 kg tons.

Design of Cantilever footing



		at end.		at fixture.
Earthquake				
upward pressure		29,300		21,820
wt. of footing	1.3@2400 =	−3120	2.3@2400 =	−5,520
wt. of earth	3.5@1600 =	−5600	2.5@1600 =	−4,000
		20,580		12,300

moment = $\frac{20,580+12,300}{2}$×2.5×1.35 = 55,400 kgm (÷1.8 = 30,800)

	at end.	at fixture.
Normal state		
upward pressure	19,030	19,030
	−3120	−5,520
	−5600	−4000
	10,310	9,510

moment = $\frac{10,310+9,510}{2}$×2.5×1.27 = 31,450 kgm Shear = 24,800 kg

Steel area reqd = $\frac{31450\times100}{1200\times\frac{7}{8}\times200}$ = 15.0 cm²/m

use 25mmφ bars at 30cm ctoc = 16.35
(or 19mmφ " " 15cm " = 18.90)

p = $\frac{16.35}{200\times100}$ = .0008, k = .147, j = .949

fs = $\frac{31450\times100}{16.35\times\frac{7}{8}\,.949\times200}$ = ~~1100~~ 1015 kg/cm² ok

fc = $\frac{\cancel{1100}\ 1015\times.147}{15(1-.147)}$ = 11.7 kg/cm² ok

Moment at normal state controls the section.

unit shear = $\frac{24800}{100\times.949\times200}$ = 1.31 kg/cm² ok

unit bond = $\frac{24800}{7.85\times3.333\times.949\times200}$ = 5.0 kg/cm² ok.

増田淳氏関係資料
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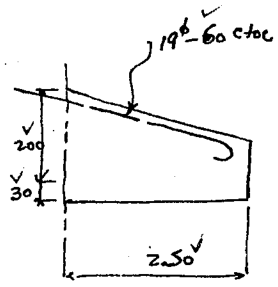
MADE BY K. Inaba DATE 4-6-29 **FILE NO**
CHECKED BY M. Jinda DATE 4-7-25 **PAGE NO** A9

CALCULATIONS FOR

Design of Jiuso Bashi for Osaka Prefecture.

Top reinforcement for cantilever footing.



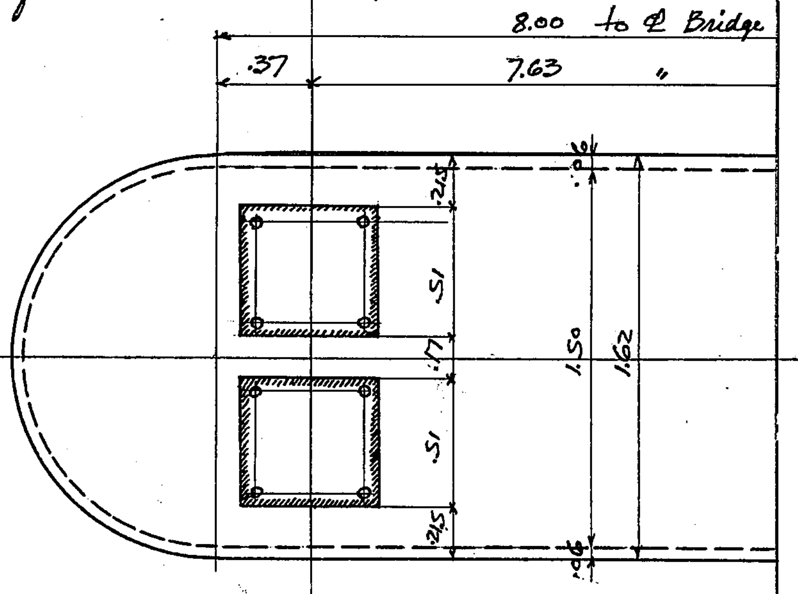
	at end.	at fixed point
upward pressure during earthquake at heel =	+2420	+9,800
wt. of concrete	-3,120	-5520
" " earth on the footing	-5600	-4000
	-6300	+ 360

moment = $\frac{-6300+360}{2} \times 2.5 \times 1.63 = 12,100$ kgm shear = 7,420 kg

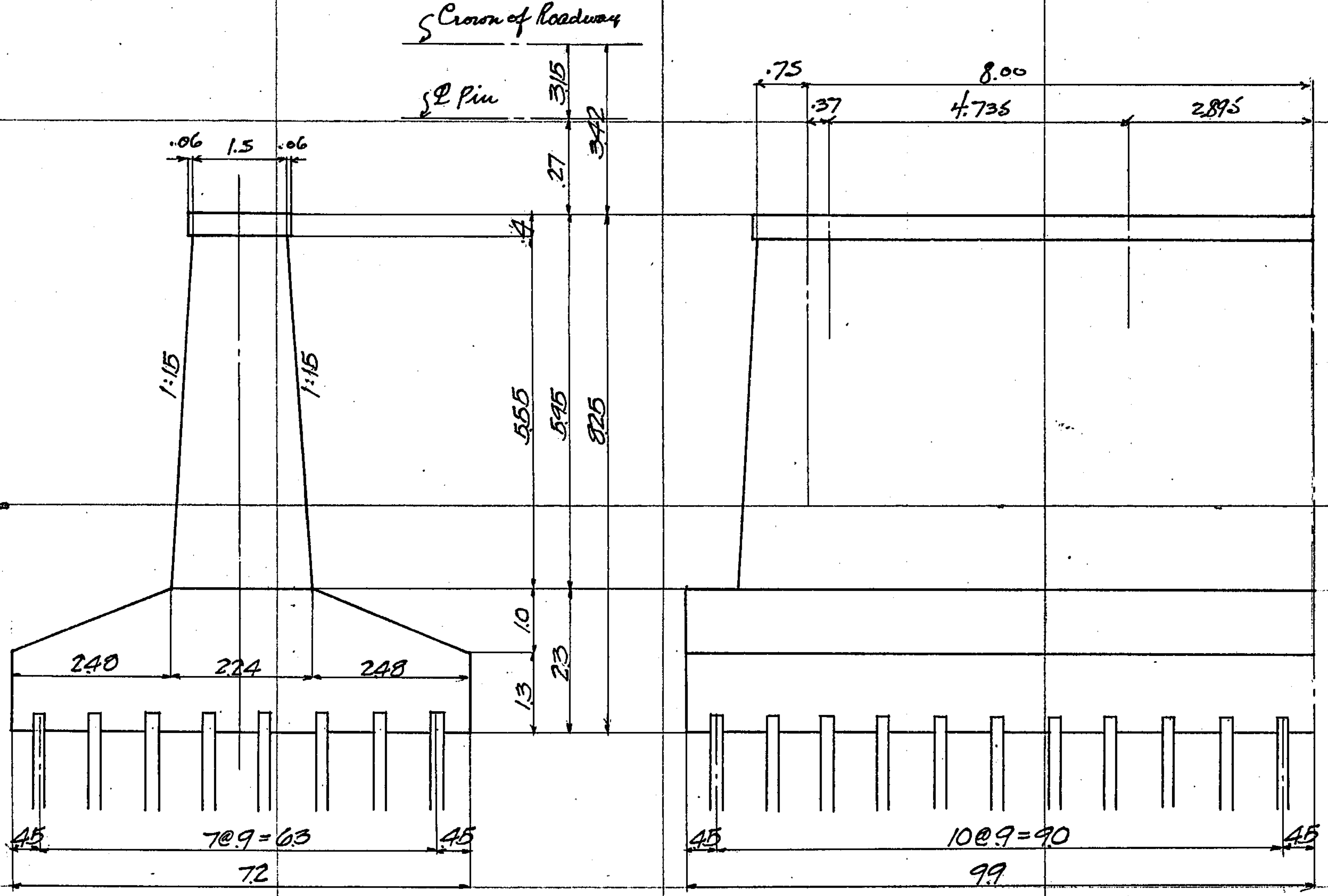
Steel area req'd. = $\frac{12100 \times 100}{2160 \times \frac{7}{8} \times 200} = 3.20$ cm² per meter strip of base.

use 19φ bars at 60 cm ctoc = 4.73

Design of Pier A. (Piers P1 + P16)



Detail of Top of Pier.
Scale 1:30.



Sketch for Girder piers P1 + P16
Scale 1:100.

Design of Jiuso Bashi for Osaka prefecture.

Volume and weight of Concrete 1:2:4 mixture.

Coping $1.62 \times 0.4 \times 16.0 = 10.37 \times 5.75 = 59.6$
" $1.62^{\phi} \times 0.4 = 0.83 \times$ " $= 4.8$
Shaft. $1.87 \times 5.55 \times 16.0 = 166.10 \times 2.61 = 433.5$
" $1.87^{\phi} \times 5.55 = 15.25 \times 2.42 = 36.9$
$192.55^{m^3} \times 2.78^{m} = 534.8$ weight of Shaft $= 192.55 @ 2400 = 462,000$ kg

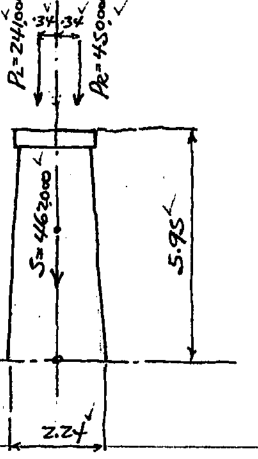
Base $4.72 \times 1.0 \times 19.8 = 93.40 \times 1.71 = 159.7$
" $7.20 \times 1.3 \times 19.8 = 185.40 \times .65 = 120.4$
$278.80^{m^3} \times 1.01^{m} = 280.1$ weight of Base $=$ ~~280.1~~ 278.8 @ $2400 =$ ~~672,000~~ 669,000

Total concrete for one pier $= 471.35$ cub meters.

Superimposed Loads on Pier.

		Loads on Bearing Shorter span	on Longer span.
Dead Load	$= 430,000$	$14\overset{17}{7}100+5900 = 147,600$	$276500+5900 = 282,400$
Live Load	$= 261,000$	$93,300$	$167,500$
	$691,000$ kg.	$240,900$	$449,900$
		Call this $241,000$ kg	$450,000$ kg

Stability of Shaft.
Case 1 Stability at normal State.



Bottom Section
Equivalent rectangle of same moment of inertia.
moment of inertia of circle $2.24^{\phi} = 0.0491 \times 2.24^4 = 1.235^{m^4}$
rectangle moment of inertia $\frac{x \times 2.24^3}{12} = 0.937x$

$0.937x = 1.235$
$x = \frac{1.235}{.937} = 1.32$ $1.32+16.0 = 17.32^{m}$.

Area $= 17.32 \times 2.24 = 38.8^{\square m}$ moment of inertia $= \frac{17.32 \times 2.24^3}{12} = 16.2^{m^4}$

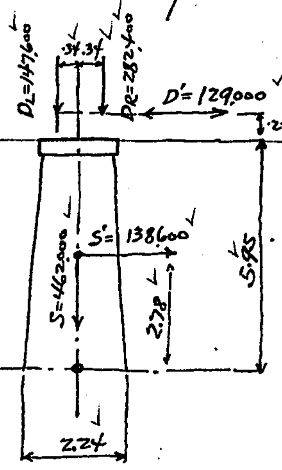
Loads on piers

$P_L = 241,000 \times -.34 = -82000$
$P_R = 450,000 \times .34 = +153000$
$S = 462000 \times 0 = 0$
$1,153,000 \times .062^{m} = +71,000$ kgm

$$f_c = \frac{1,153,000}{38.8}\left(1 \pm \frac{6 \times \overset{.166}{.062}}{2.24}\right) = 34,700 \text{ kg/m}^2 \; C = 3.47 \text{ kg/cm}^2 \text{ ok}$$

or 24800 " $C = 2.48$ "

Case 2 Stability during Earthquake.



Loads	Hor. forces	Vert. forces	Lever arm	Moment.
D_L		$147600 \times$	$-.34 =$	-50200
D_R		$282400 \times$	$.34 =$	$96,000$
D'	$129,000$	$\times$	$6.22 =$	$802,000$
S		$462,000 \times$	$0 =$	0
S'	138600	$\times$	$2.78 =$	$385,500$
	$267,600$ kg	$892,000$ kg $\times$	$1.38^{m} =$	$1,233,300$ kgm.

$\frac{e}{h} = 1.38/2.24 = .616$, $\frac{d'}{h} = 6/224 = .026$

Try reinforcement 19^{ϕ} bars at 30^{cm} c to c. $= 2.835 \times 3.333 \times 17.32 = 163.6^{cm^2}$

$p_0 = 2p = \frac{163.6 \times 2}{1732 \times 224} = .00084$, $k = .23$ $L = .061$

$$f_c = \frac{1,233,300 \times 100}{.061 \times 1732 \times 224^2} = 23.3 \text{ kg/cm}^2 \text{ ok}$$

$$f_s = 15 \times 23.3\left(\frac{218}{.23 \times 224} - 1\right) = 1132 \text{ kg/cm}^2 \text{ ok.}$$

$$\text{Unit Shear} = \frac{267,600}{1732 \times \frac{7}{8} \times 218} = .81 \text{ " ok}$$

$$\text{Unit bond} = \frac{267600}{1732 \times 5.965 \times 3333 \times \frac{7}{8} \times 218} = 4.1 \text{ " ok}$$

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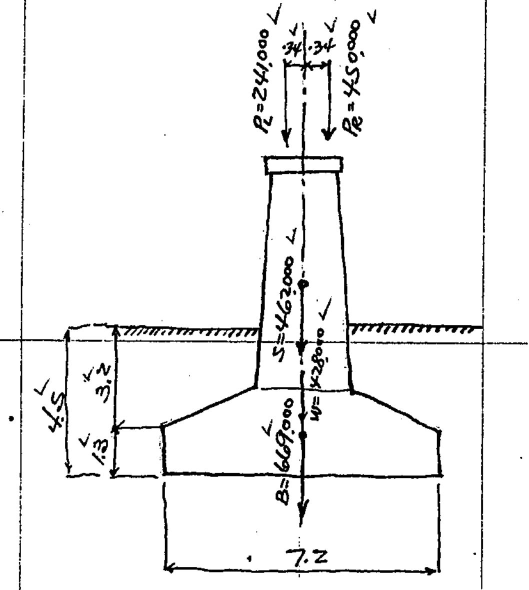
CALCULATIONS FOR

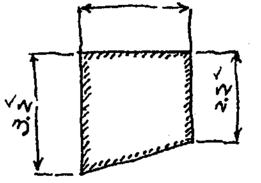
Design of Jiuso Bashi for Osaka Prefecture.

Stability of Pier as a whole.

Case 1. Stability at Normal State.

Weight of earth on footing. 2.7 × 2.5 × 19.8 @ 1600 = 214,000 × 2 = 428,000 kg say = W





Superimp. loads.	241,000 ×	.34 =	− 82,000
"	450,000 ×	.34 =	+ 153,000
wt. of shaft.	462,000		0
" base	669,000		0
" earth on footing	428000		0
	2250,000 kg ×	.032 m =	71,000 kgm.

max. toe pressure = $\frac{2250,000}{7.2 \times 19.8}\left(1 \pm \frac{6 \times .032}{7.2}\right)$ = 16200 kg/m² (1.48 tons/□′)
15400

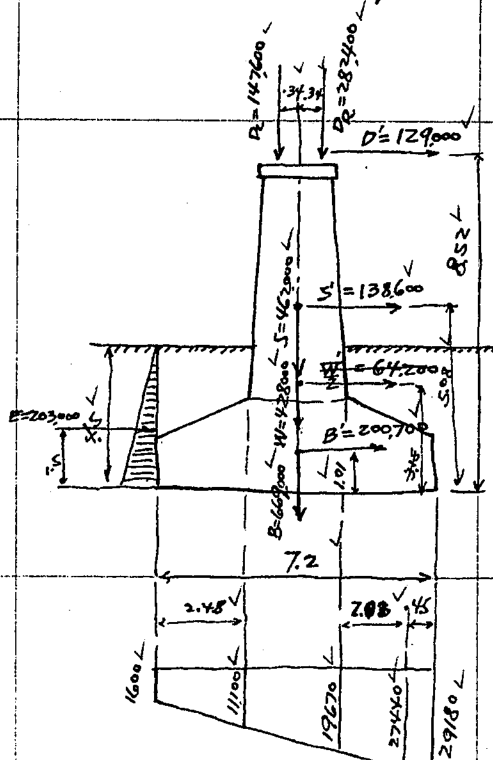
max. load on one pile = 16.2 × .9 × .9 = 13.1 kg tons.

- Case 2. Stability during Earthquake.

~~Seismic force due to earth on footing = $\frac{428000}{2}$ × .3 = 64,200 kg (one side only).~~

Earth pressure during earthquake. = $\frac{.662 \times 1600 \times 4.5^2}{2}$ = 10,700 kg/lin m

E = 10700 × 19.0 say = 203,000 kg



moment about center of Base

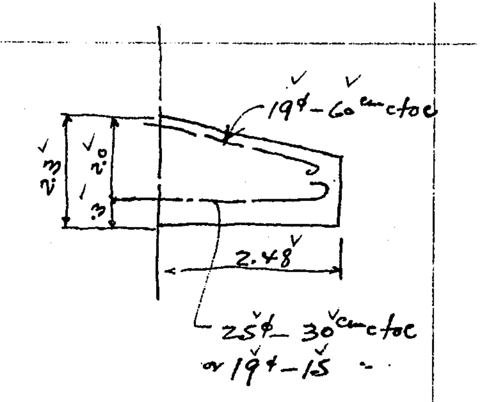
Loads	Hor. forces	Vert. forces		Lev. arm		Moment
D_L		147,600	×	−.34	=	− 50,200
D_R		282,400	×	+.34	=	+ 96,000
D′	129,000		×	+8.52	=	+ 1,099,000
S		462,000	×	+ 0	=	0
S′	138,600		×	+5.08	=	+ 704,000
W		428,000	×	0	=	0
B		669,000	×	0	=	0
B′	200,700		×	+1.01	=	+ 202,800
E		203000	×	+1.50	=	+ 304,500
	468,300	2192,000	×	1.075 m	=	+ 2356,100

Resultant force within middle third.

max. toe pressure = $\frac{2192,000}{7.2 \times 19.8}\left(1 \pm \frac{6 \times 1.075}{7.2}\right)$ = 29,180 kg/m² or (2.67 tons/□′)
.896
or 1,600

Max. load on one pile = 27.44 × .9 × .9 = 22.4 kg tons

Design of Cantilever footing



	at end		at fixed point.
Earthquake			
Upward pressure	29180		19670
wt. of footing	− 3120		− 5,520
wt of earth 3.2 @ 1600 =	− 5,120	2.2 @ 1600 =	− 3,520
	20,940		10,630

moment = $\frac{20940+10630}{2}$ × 2.48 × 1.35 = 52800 kgm/m strip, Shear = 39150 kg/m

At normal state		
upward pressure	16200	15,400
wt of footing + earth	− 8,240	− 9,040
	7,960	6,360

moment = $\frac{7960+6360}{2}$ × 2.48 × 1.28 = 22750 kgm/m strip, Shear = 17750 kg/m.

Moment + Shear during Earthquake controls the section.
52800 ÷ 1.8 = 29300 < 31450 for Pier B.
Use Same details as for Pier B + C. for Base reinforcements.

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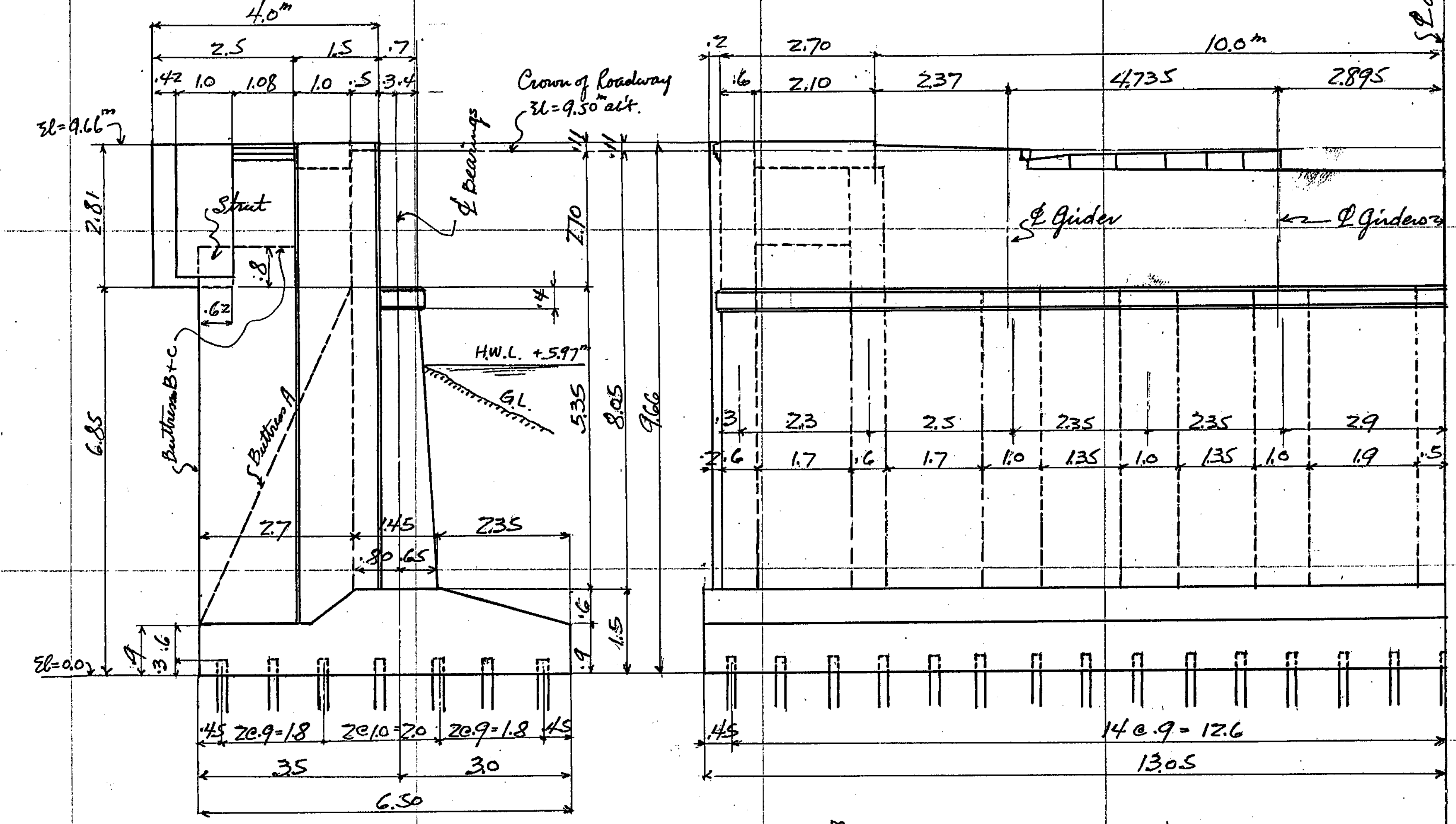
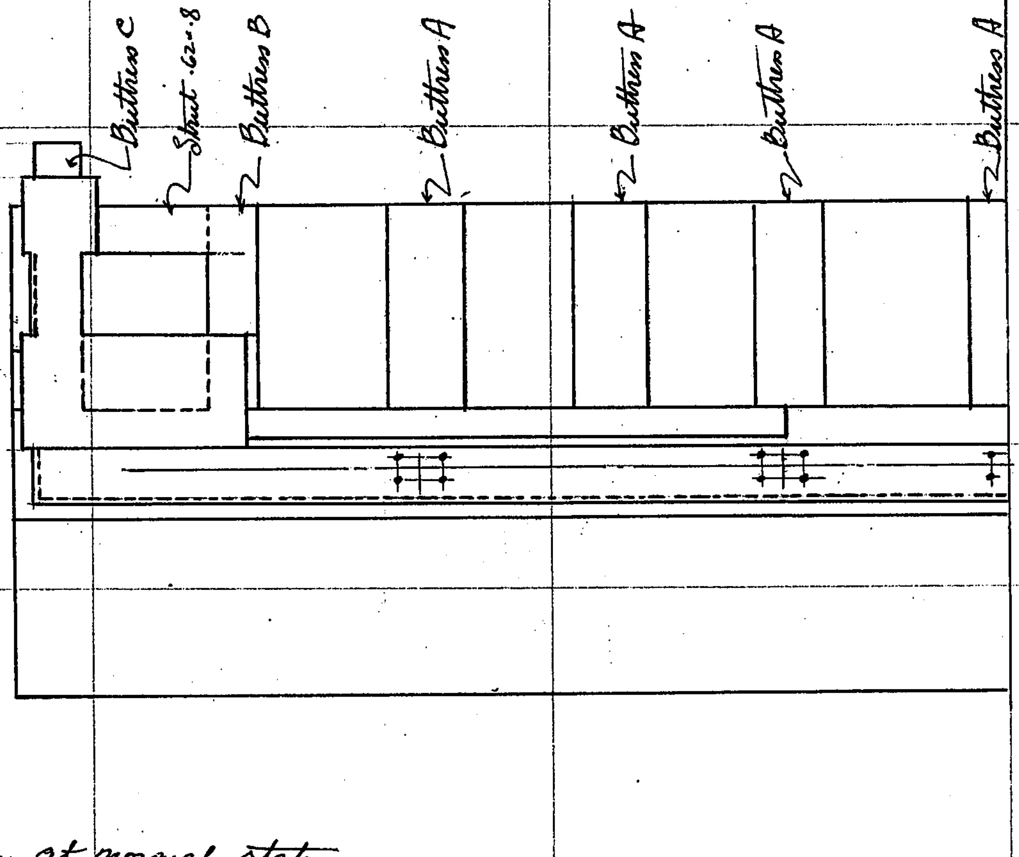
CALCULATIONS FOR

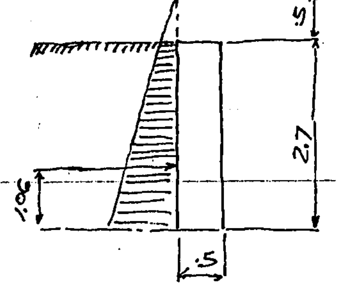
Design of Jiuso Bashi for Osaka prefecture.
Design of Abutment for River Spans. (abutments A3 and A4).





Sketch of Abutments A3 and A4.
Scale 1:100.
Note. Abutment A4 has cantilevers under handrails on both wings.

Parapet wall.



Moment and shear at normal state.

Earth pressure $= \frac{1}{3} \cdot 1600 \cdot 0.5 = 267$

$\frac{1}{3} \cdot 1600 \cdot 3.2 = \underline{1706}$

$1973 \div 2 =$ ~~1487~~ 987 kg/m² average.

Earth pressure for one meter strip of wall $= 987 \cdot 2.7 = 2665$ kg

moment $= 2665 \cdot 1.06 = 2830$ kgm

Shear $= 2665$ kg

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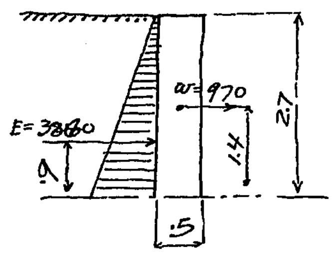
CALCULATIONS FOR

Design of Jinso-Bashi for Osaka prefecture.



moment and shears for Earthquake Seismic coef. $k=0.3$ assumed.

Earth pressure $= 0.662\ \frac{wh^2}{2} = .331 \times 1600 \times 2.7^2 = 3860$ kg per lin m.

wt. of wall $= .5 \times 2.7 \times 2400 = 3240$ kg

Seismic force $= 3240 \times .3 = \underline{970}$ kg

~~4210~~

moment $3860 \times .9 = 3475$

$970 \times 1.4 = \underline{1358}$

4833 kgm

Shear $= 3860 + 970 = 4830$ kg

moment at normal state governs.

Effective depth required $= \sqrt{\frac{2830 \times 100}{100 \times 7.18}} = 19.8$ cm

Use 45 cm effective depth with 5 cm insulation or 50 cm overall.

Steel area required $= \frac{2830 \times 100}{1200 \times \frac{7}{8} \times 45} = 5.99$ cm² per meter strip.

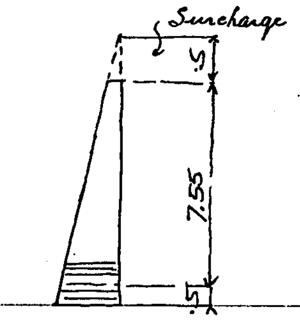
Use 16 mm ϕ bars at 30 cm c to c $= 6.71$ cm² on rear side

13 " " " " " $= 4.43$ " " front "

Unit Shear $= \frac{4830}{100 \times \frac{7}{8} \times 45} = 1.2$ kg/cm² during earthquake. ok

Unit bond $= \frac{4830}{5.03 \times 3.33 \times \frac{7}{8} \times 45} = 7.3$ kg/cm² during earthquake ok.

Curtain wall at center. Span length $= 2.9$ m c to c of buttresses. clear span $= 1.9$ m.



At normal state.

Average earthpressure on lowest 1 meter strip.

$= \frac{1}{3} \times 1600 \times 8.05 = 4290$ kg/m²

moment $= \frac{1}{10} \times 4290 \times 2.9^2 = 3610$ kgm.

Shear $= \frac{1}{2} \times 4290 \times 1.9 = 4075$ kg

During Earthquake.

Average earth pressure on lowest 1 meter strip.

$= \frac{1}{3}\ 0.662 \times 1600 \times 7.55 = 7990$ kg/m²

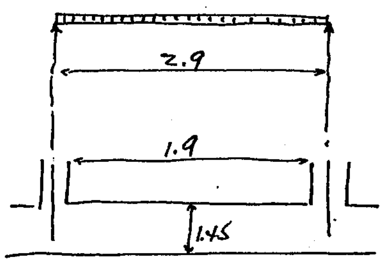
wt. of wall $= 1.45 \times 1.0 \times 2400 = 3480$ kg

Seismic force $= 3480 \times .3 = 1045$ kg.

Total load on wall $= 7990 + 1045 = 9035$ kg per lin meter

moment $= \frac{1}{10} \times 9035 \times 2.9^2 = \underline{7600}$ kgm.

Shear $= \frac{1}{2} \times 9035 \times 1.9 = \underline{8590}$ kg.



moment and shear during earthquake governs the section.

Effective depth req'd $= \sqrt{\frac{7600 \times 100}{100 \times 7.18}} = 32.6$ cm

Use 140 eff. depth with 5 cm insulation. total depth $= 145$ cm

Steel area req'd. $= \frac{7600 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 140} = 2.87$ cm²

Unit shear $= \frac{8590}{100 \times \frac{7}{8} \times 140} = 0.7$ kg/cm² ok.

Use 19 mm ϕ bars at 50 cm c to c $= 5.67$ cm²

Unit bond $= \frac{8590}{5.97 \times 2 \times \frac{7}{8} \times 140} = 5.87$ kg/cm² ok.

For other curtain walls, use the same details as for center curtain wall discussed above.

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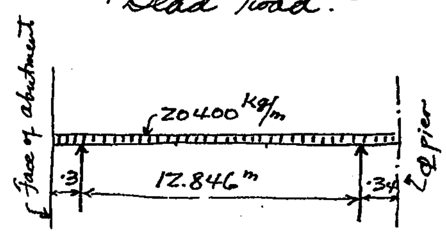
CALCULATIONS FOR

Design of Jiuso Bashi for Osaka Prefecture.

Superimposed Loads on Abutment.

Dead Load:–



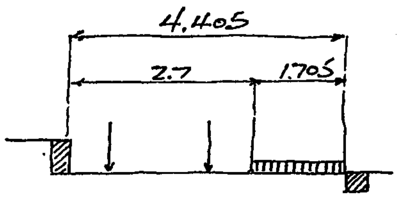
Dead load of Bridge = 20,400 kg/lin m.
D.L. on abutment = 20,400 × 12.846 ÷ 2 = 131,000 kg
outside of Bearing = 20400 × ·3 say = 6,100
weight of Shoes say 5 @ 700 = 3,500
140,600 kg for one abutment.
Call this 140,000 kg

Live Load:

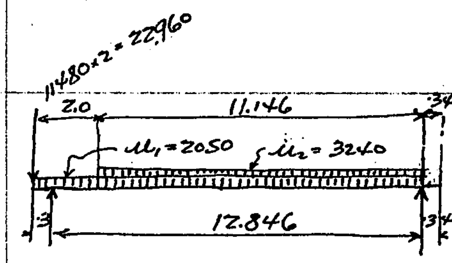
Uniform load on sidewalk. 500 kg/m²
500 × 2.75 × 2 = 2750 kg/lin meter.

½ × 12,846 = 6.423
·300
6.723 m

Load on abutment = 2750 × 6.723 = 18,500 kg for one abutment.
Uniform load on roadway = 600 kg/m²
Motor truck wheels on roadway
Rear wheel concentration = 4,500 kg
Imp. coef. = $\frac{20}{60+12.846}$ = 27.5 % = 1,240
5,740 kg × 2 = 11,480 kg
Front wheel conc. with impact = $\frac{5740}{3}$ × 2 = 3,830 kg

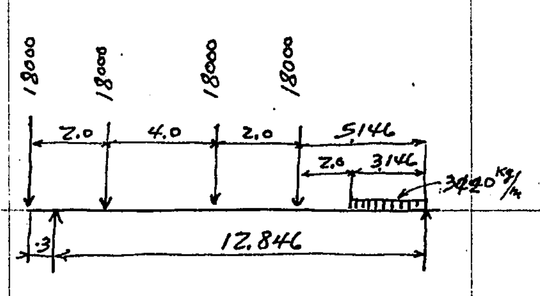


Uniform load on roadway throughout the span = 600 × 1.705 × 2 = 2050 kg/m = U_1
Uniform load on roadway on front + rear of trucks = 600 × 2.7 × 2 = 3240 = U_2
Load on abutment.



Rear wheels.	22,960 × 13,146 ÷ 12,846	=	23,550
Unif. load U_1	$\frac{2,050 \times 13,146^2}{2 \times 12,846}$	=	13,800
Unif. load U_2	$\frac{3240 \times 11,146^2}{2 \times 12.846}$	=	15,650
			53,000 kg for one abutment.

Loads on Electric Railway.
water sprinklers, double lines @ 2 cars.
axle load = 9,000 kg. 9000 × 2 = 18000 kg for double line.
Unif. load on electric railway 600 × 5.69 = 3420 kg/lin meter.
Load on abutment.



wheel loads. = $\frac{18000\,(\overset{36.584}{5.146+7.146+11.146+13.146})}{12.846}$ = 51280

Unif. load = $\frac{3\overset{420}{\cancel{240}} \times 3.146^2}{2 \times 12.846}$ = 1,320
52,600 kg for one abutment.

Summary for Live load on abutment

Uniform load on sidewalk	=	18,500
Loads on roadway	=	53,000
" " electric railway	=	52,600
		124,100 misc. variations say 900 kg

Call this 125,000 kg

Summary of Superimposed Loads on abutment.

	For one abutment		average for one girder.
Dead Load.	140,000 ÷ 5	=	28,000
Live Load.	125,000 ÷ 5	=	25,000
	265,000 kg		53,000 kg

Seismic force 30% of D.L. = say 80,000 kg. say 18,400 kg

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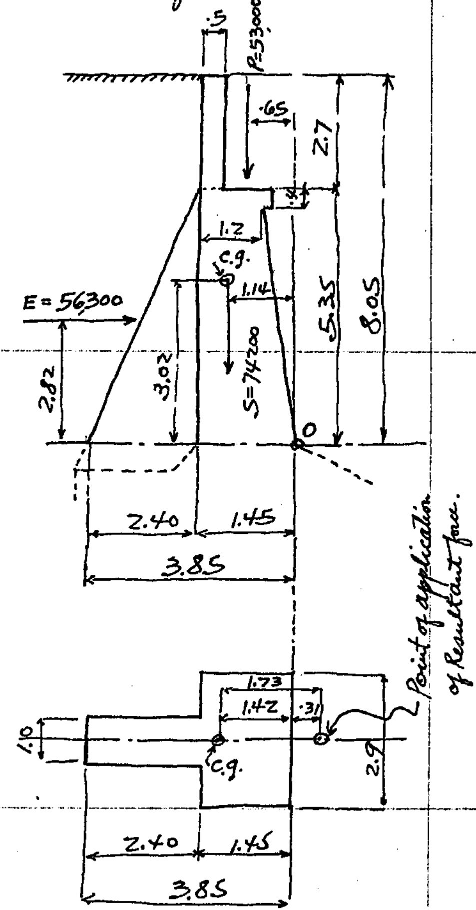
CALCULATIONS FOR

Design of Jiuso Bashi for Osaka prefecture.
Design of Counterfort walls.
Counterfort wall at center A. 2.9 meters c to c.



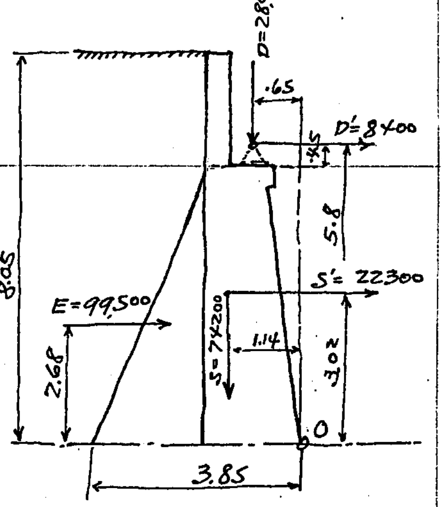
Weight of concrete	for 2.9ᵐ width·		hor. arm		vert. arm	
parapet wall	·5×2.7×2.9 @2400 =	9400	1.20	11,280	6.70	68,000
Coping	·4×1.3×2.9 @ " =	3600	·80	2880	5.15	18,550
Curtain wall	1.33×4.95×2.9 @ " =	45800	~~2.25~~ ·78	35,700	2.40	110,000
Counterfort wall.	1.2×5.35×1.0 @ " =	15,400	2.25	34,650	1.78	27,400
	30.94 m³	74,200 kg	1.14 m	84,510	3.02 m	223,950

Center of gravity of Botton section
Front wall. $2.9 \times 1.45 = 4.21 \times \cdot 725 = 3.05$
Counterfort wall $2.4 \times 1.00 = 2.40 \times 2.65 = 6.36$
$6.61^{m} \times 1.42^{m} = 9.41$

Earthpressure at normal state, assuming surcharge due to L.L. = 0.5ᵐ
$\frac{1}{3} \times 1600 \times 0.5 = 267$
$\frac{1}{3} \times 1600 \times 8.55 = 4557$
$4824 \div 2 \times 8.05 = 19,420$ kg/meter strip
for 2.9 m strip, total earth pressure $E = 19420 \times 2.9 = 56,300$ kg
Superimposed Dead + Live load for one girder = 53,000 kg page A14
Taking moment about point O.

Loads	Horizontal forces	Vertical forces		Lever arm		Moment
P	$	53,000	×	− 0.65	=	− 34,450
S		74,200	×	− 1.14	=	− 84,500
E	56300		×	2.82	=	158750
	56,300 kg	127,200 kg	×	0.31 m	=	39,800 kgm

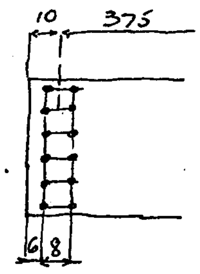
Eccentricity $\varepsilon = 0.31 + 1.42 = 1.73^{m}$.
moment at bottom section $= 127,200 \times 1.73 = 220,000$ kgm.
Shear at " " $= 56,300$ kg



Earthpressure during earthquake k=0.3 Assumed.
$0.662 \frac{wh^2}{2} = 0.331 \times 1,600 \times 8.05^2 = 34,300$ kg per meter strip.
for 2.9ᵐ width of wall $E = 34300 \times 2.9 = 99,500$ kg
Superimposed Dead load = 28,000 kg
Seismic force $= 28000 \times .3 = 8,400$ "
Taking moments about point O.

Loads	Hor. forces	Vert. forces		Lev. arm		Moment
D		28,000	×	−.65	= −	18,200
D'	8,400		×	5.8	= +	48,700
S		74,200	×	− 1.14	= −	84,600
S'	22,300		×	3.02	= +	67,300
E	99,500		×	2.68	= +	266,500
	130,200 kg	102,200 kg	×	2.74 m	= +	279,700

Eccentricity $\varepsilon = 2.74 + 1.42 = 4.16^{m}$
moment at bottom $= 102,200 \times 4.16 = 425,000$ kgm $> 220,000 \times 1.8$
Shear " " $= 130,200$ kg $> 56,300 \times 1.8$

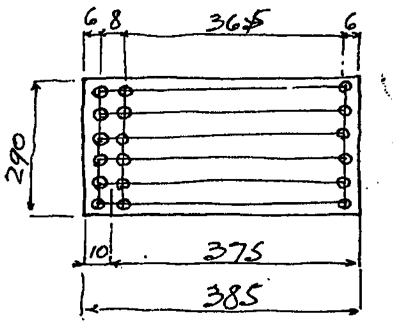


Moment + Shear for Earthquake case controls the section.
approx. steel area required. $\frac{425000 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 375} = 60.0$ cm² for moment only.

Try reinforcements. 12 - 25 mmφ bars = 58.9 cm²
Steel ratio $p = \frac{58.9}{290 \times 375} = .00054$

$t/d = 145/375 = 0.39$
Neutral axis in the flange. Design as a rectangular section of double reinforcements.

Design of Jinsō-Bashi for Ōsaka prefecture.



$b = 290$, $d = 375$, $d' = 6$, $A_s = 12-25^\phi = 58.9^{cm^2}$ $A_s' = \frac{A_s}{2} = 6-25^\phi = 29.5^{cm^2}$

$p = \frac{58.9}{290 \times 375} = .00054$, $p' = \frac{.00054}{2} = .00027$, $n = 15$. $p+p' = .00081$ $d'/d = \frac{6}{375} = .016$

$$k = \sqrt{2n(p+p'\frac{d'}{d}) + n^2(p+p')^2} - n(p+p')$$

$$= \sqrt{2\times15(.00054+.00027\times\frac{6}{375}) + 15^2(.00081)^2} - 15\times.00081 = \sqrt{.0178} - .0122 = 0.121$$

$.01405$ $.000037$

$$j = \frac{k^2(1-\frac{1}{3}k) + 2p'n(k-\frac{d'}{d})(1-\frac{d'}{d})}{k^2 + 2p'n(k-\frac{d'}{d})} = \frac{.121^2 \times .96 + 30\times.00027\times.105\times.984}{.121^2 + 30\times.00027\times.105} = 0.96$$

$.01464$ $.00085$

$f_s = \frac{425000 \times 100}{58.9 \times .96 \times 375} = 2005$ kg/cm² $< 1200 \times 1.8 = 2160$ ok. For safe side, effect of direct compression neglected.

$f_c = \frac{2005 \times .121}{15(1-.121)} = 18.4$ " due to bending moment.

" $= \frac{102200}{66100} = 1.6$ " due to direct compression.

20.0 kg/cm² $< 35 \times 1.8 = 63$ ok.

width of web $= 100^{cm}$.

Unit shear $= \frac{130200}{100 \times .96 \times 375} = 3.6$ kg/cm² $< 4 \times 1.8$ ok.

Unit bond $= \frac{130200}{12 \times 7.85 \times .96 \times 375} = 3.8$ " ok.

Case of Seismic forces backward.

Taking moment about point O. refer to the sketch for above case page A15.

Earth pressure E'
$E' = .331 \times 1600 \times 3^2 \times 2.9$
$= 13,800$ kg

Loads	Hor. forces	Vert. forces		Lev. arms		moments.
D		28,000	×	.65	=	18,200
D'	8,400		×	5.8	=	48,700
S		74,200	×	1.14	=	84,600
S'	22,300		×	3.02	=	67,300
E'	13,800		×	1.00	=	13,800
	44,500 kg	102,200 kg	×	2.28	=	232,600 kgm

Eccentricity $\varepsilon = 2.28 - 1.42 = 0.86^m$

moment on bottom section $= 102200 \times .86 = 87,900$ kgm.

Shear " " $=$ 44,500 kg

try reinforcements 6-25mmφ bars $= 29.5$ cm² eff. depth $= 385 - 6 = 379^{cm}$

$p = \frac{29.5}{100 \times 379} = .00078$, $j = .95$, $k = .147$

$f_s = \frac{87900 \times 100}{29.5 \times .95 \times 379} = 827$ kg/cm² ok. Effect of direct comp. neglected.

$f_c = \frac{827 \times .147}{15(1-.147)} = 9.5$ " due to bending moment.

" $= \frac{102200}{66100} = 1.6$ due to direct compression.

11.1 kg/cm² ok.

Shear + bond stress ok

For all intermediate counterforts A, use same details as above.

Counterfort walls B. at end. Following calculations are figured for counterfort wall under curb line.

Volume of concrete and weight of wall. (2.4m wide).

						hor arm		vert arm.	
Parapet wall	2.4×2.7×.5 =	3.24	@ 2,400	=	7,780	1.20	9,330	6.70	52,100
Coping	0.4×1.3×2.4 =	1.25	@ "	=	3,000	.80	2,400	5.15	15,450
Curtain wall	1.33×4.95×2.4 =	15.80	@ "	=	37,900	.78	25,650	2.40	91,000
Counterfort wall.	2.7×.6×6.75 =	10.93	@ "	=	26,220	2.80	73,400	2.68	70,300
" "	1.0×.6×2.32 =	1.39	@ "	=	3,340	1.95	6,500	5.61	18,750
Slab under pedestal	1.5×1.25×.5 =	.94	@ "	=	2,260	1.70	3,840	7.91	17,870
Strut between walls.	.62×.8×1.45 =	.72	@ "	=	1,730	3.84	6,640	5.75	9,940
Pedestal etc say					7,770	1.70	13,200	9.50	73,800
		34.27m³			90,000 kg	1.57m	140,960	3.88m	349,210

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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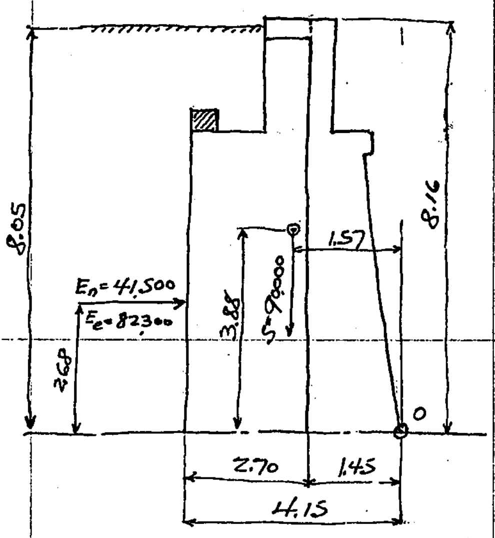
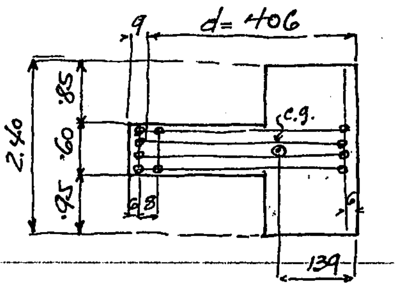
CALCULATIONS FOR

Design of Jinsō-Bashi for Osaka-Prefecture.





Center of gravity of Bottom area.

Curtain wall 145 × 240 = 34,800 × 72.5 = 2,521,000
Counterfort 60 × 270 = 16200 × 280. = 4,539,000
51,000 cm² : 139 cm = 7,060,000

Earthpressure at normal state. Surcharge due to L.L. is very small and neglected.
$H = \frac{1}{3} \times 1600 \times \frac{8.05^2}{2}$ = 17,300 kg/m strip.
E_n = 17300 × 2.4 = 41,500 kg.
Taking moment about O.
weight wall S = 90,000 × 1.57 = − 141,300
Earth pressure E_n = 41,500 × 2.68 = 111,300
− 30,000 kgm ÷ 90,000 = −0.33 m

Eccentricity ε = 1.39 − 0.33 = 1.06 m
moment at bottom section = 90,000 × 1.06 = 95,400 kgm
Shear 〃 〃 〃 = 41,500 kg

Earthpressure during earthquake.
E_e = 34,300 × 2.4 = 82,300 kg page A 15.
moment about O.
Weight of wall 90,000 × 1.57 = − 141,300
earth pressure 82,300 × 2.68 = 220,500
79,200 ÷ 90,000 = .88
Eccentricity ε = 1.39 + .88 = 2.27 m
moment at bottom section = 90,000 × 2.27 = 204,300 kgm. > 95400 × 1.8
Shear 〃 〃 = 82,300 kg > 41500 × 1.8
moment and shear during earthquake governs.
Steel area required for moment = $\frac{204300 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 406}$ = 26.7 cm²

Use 6−25 mmφ bars = 29.5 cm²
Steel ratio = $\frac{29.5}{240 \times 406}$ = .00030 = p. p′=0.00015 p+p′ = .00045. $\frac{d'}{d} = \frac{6}{406}$ = .0147
t/d = 145/406 = 0.36

neutral axis in the flange. Design as a rectangular section.
$k = \sqrt{30(.0003 + .00015 \times .0147) + .00675^2} - .00675 = .089$
.00768 .0000329
$j = \frac{.00792 \times .9703 + .0045 \times .0743 \times .9853}{.00792 + .0045 \times .0743} = \frac{.00801}{.00826} = 0.970$
.000334
$f_s = \frac{204300 \times 100}{29.5 \times .97 \times 406}$ = ~~1550~~ 1760 kg/cm² < 1200 × 1.8 = 216. ok. Effect of direct comp. neg'td.
$f_c = \frac{1760 \times .089}{15(1 - .089)}$ = 11.5 due to bending moment
〃 = $\frac{90,000}{51,000}$ = 1.8 due to direct compression
13.3 kg/cm² ok.
Unit Shear = $\frac{82300}{60 \times .97 \times 406}$ = 3.5 kg/cm² ok.
Unit bond = $\frac{82300}{7.85 \times 6 \times .97 \times 406}$ = 4.4 〃 ok.

Case of Seismic forces backward. moment about O.
wt. of wall 90,000 × 1.57 = − 141,300
Earth pressure E_e = .331 × 1600 × 3² × 2.4 = 11,500 kg
earth pressure 11,500 × 1.0 = − 11,500
− 152,800 ÷ 90,000 = −1.7 m
Ecc. ε = 1.39 − 1.7 = −0.31 m = 90000 × .31 = 27,900 kgm. Shear = 11,500 kg
Steel req'd (approx) = $\frac{27900 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 409}$ = 3.6 cm²
Use 4−25 mmφ bars = 19.36 cm²

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CALCULATIONS FOR

Design of Jinso-Bashi for Osaka prefecture.

Weight and center of gravity of whole Shaft of abutment A3.

For 3 intermediate counterforts. A.

					hor. arm		vert. arm.	
parapet wall.	.5×2.7×7.40 =	10.00	@2400 =	24000	1.20	28800	6.70	160800
Coping	.4×1.3×7.40 =	3.85	@ 〃 =	9240	.80	7400	5.15	47600
Curtain wall	1.33×4.95×7.40	48.70	@ 〃 =	116800	.78	91,100	2.40	280,500
Counterforts.	1.2×5.35×1.0×3 =	19.28	@ 〃 =	46260	2.25	104,000	1.78	82,400
		81.83 m³		196300	1.18	231,300	2.91	571,300

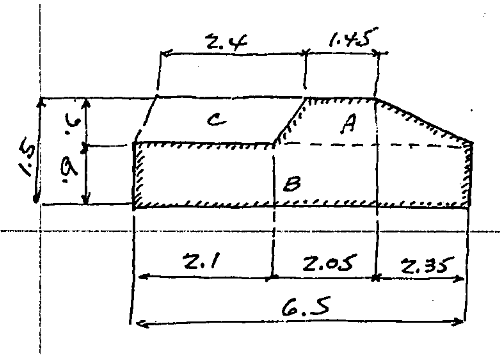
For End counterfort C.

part same as B.		34.27		90,000	1.57	140,960	3.88	349,210
light pedestal less.				−7770	1.70	−13,200	9.50	−73,800
wing wall. add.	2.5×.6×2.5 say	= 3.75	@ 〃 =	9,000	3.70	33,300	~~8.~~ 7.10	63,900
End col. projection 〃	.4×1.0×2.5	= 1.00	@ 〃 =	2400	4.03	9,670	6.91	16,600
Column 〃	.2×1.5×8.16	= 2.45	@ 〃 =	5870	1.70	9970	4.08	23950
Handrail etc say			=	1,200	3.50	4,200	8.60	10,320
Strut	~~add.~~ less .62×.8×.8	= .40	@ 〃 =	960	3.84	3690	5.75	5,520
		41.87		101,660	1.86	188,590	3.89	395700

Summary.	vol. of concrete	weight	hor. arm		vert. arm		
Center counterfort 1-A.	30.94	74,200	1.14	84,510	3.02	223,950	Page A15
intermediate 〃 6-A	163.66	392,600	1.18	462,600	2.91	1,142,600	Page A18
〃 〃 2-B	68.54	180000	1.57	281920	3.88	698420	〃 A16
end 〃 2-C	83.74	203320	1.86	377,180	3.89	791,400	
	346.88 m³	850,120 kg	1.42 m	1206,210	3.36 m	2,856,370	

Call this 850,000 kg = S

weight and center of gravity of Base.



A	2.93×.6×26.1 =	45.90	@2400 =	110100	2.55	281,000	1.15	126,500
B	6.5×.9×26.1 =	152.70	@ 〃 =	366200	3.25	1,190000	0.45	164800
C	~~9.~~ 12.8 ×.6×2.4 =	18.43	@ 〃 =	44200	5.0	221000	1.15	50800
		217.03 m³	(B) =	520,500 kg	3.25	1692000	0.66	342,100

Total concrete for abutment 346.88
563.91 m³

Superimposed loads on abutment D.L. 140,000 = D.
L.L. 125,000
265000 kg = P.

Earth on Rear footing 2.7×26.1×8.65 = 610 − 55 (vol. of counterforts.) = 555 @ 1600 (11.00) = 888,000 kg = W₁

Earth on Front 〃 2.35×26.1×3.0 = 184 @ 〃 = 294,000 〃 = W₂

Earth pressure on rear $\frac{1}{6}\times1600\times10.05^2$ = 27100×26.1 = 706000 kg at normal state = E_n

〃 〃 〃 front. $\frac{1}{6}\times1600\times4.5^2$ = 5400×26.1 = 141000 kg 〃 〃 〃 = E_n'

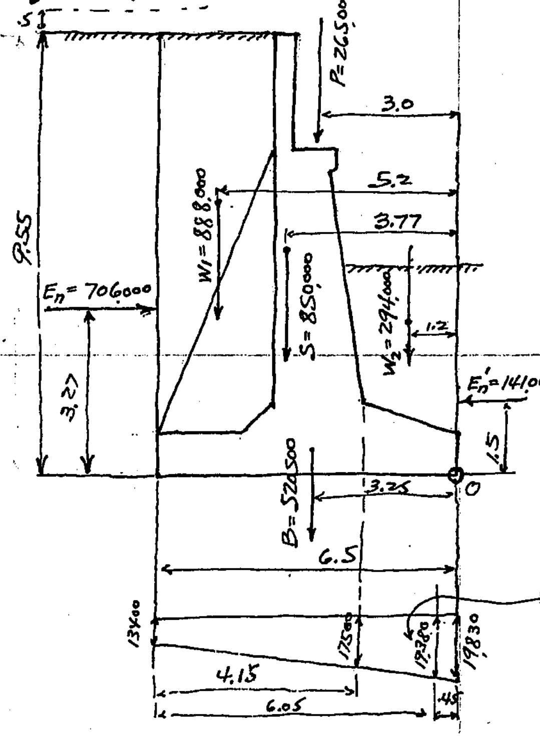
〃 〃 〃 rear .331×1600×~~10.05~~ 9.55² = ~~53500~~ 48300 ×26.1 = ~~1348000~~ 1260000 kg during earthquake = E_e

〃 〃 〃 front .331×1600×4.5² = 10,700×26.1 = 279,000 〃 〃 〃 = E_e'

Stability of abutment as a whole.

Case 1. Stability at normal state.

Taking moment about point O (toe).



Loads	Hor. forces	Vert. forces		Hor. arms		moments.
P		265000	×	−3.0	=	−795000
S		850,000	×	−3.77	=	−3205000
B		520,500	×	−3.25	=	−1692000
W₁		888000	×	−5.20	=	−4620000
W₂		294000	×	−1.20	=	−353000
Eₙ	706000		×	+3.27	=	+2310000
Eₙ'	141000		×	−1.50	=	−211,500
	847000 kg	2817500 kg		−3.04 m	=	−8,566500

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

max. toe pressure = $\frac{2,817,500}{6.5\times26.1}\left(1\pm\frac{6\times.21}{6.5}\right)$ = $\frac{19830}{13400}$ kg/m² ≈ (1.81 tons/□') ≈ (1.23 〃)

max. load on one pile = 19380×.9×.9 = 15.7 kg tons. at extreme row. (杭ノミニテ支持スルモノトシテ)

piles 10"φ at tip 60 ft. long

Surface of friction 1.1'φ×60 = 207.5□' load 15.7×2206# = 34650#

average skin friction = $\frac{34650}{207.5}$ = 167 #/□' ok.

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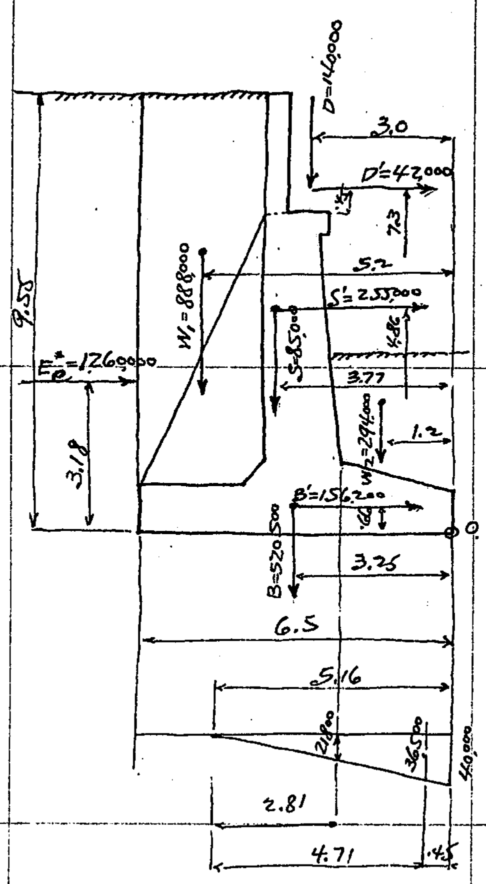
CALCULATIONS FOR

Design of Jūsō-Bashi for Ōsaka-prefecture.

Case 2. Stability of Abutment during Earthquake (Seismic forces forward). K = 0.3 assumed.

Taking moment about toe O.



Load	Hor. forces	Vert. forces		Lev. arms		Moment
D		140,000	×	− 3.00	=	− 420,000
D'	42,000		×	+ 7.30	=	+ 306,500
S		850,000	×	− 3.77	=	− 3,205,000
S'	255,000		×	+ 4.86	=	+ 1,240,000
W₁		888,000	×	− 5.20	=	− 4,620,000
W₂		294,000	×	− 1.20	=	− 352,500
B		520,500	×	− 3.25	=	− 1,692,000
B'	156,200		×	+ 0.66	=	+ 103,000
Ee	1260,000	~~1260,000~~	×	+ 3.18	=	+ 4,005,000
	1,713,200 kg	2,692,500	×	− 1.72 m	=	− 4,635,000

Eccentricity ε = 3.25 − 1.72 = 1.53 m Resultant force outside of middle third.

Pressure area = 1.72 × 3 × 26.1 = 134.7 □m

max. toe pressure = $\frac{2692500 \times 2}{134.7}$ = 40,000 kg/m² or (3.66 tons/□')

max. pressure on one pile = 36,500 × .9 × .9 = 29.5 kg tons. (杭ニテ支持スルモノトシテ)

average skin friction = $\frac{29.5 \times 2206}{207.5}$ = 313 #/□' ok.

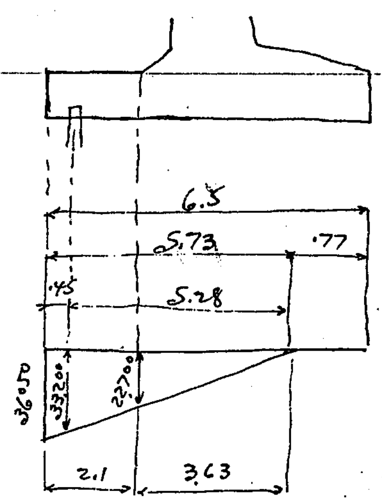
If 10,000 kg/m² (or 0.94 tons/□') be allowed for safe bearing of earth

max. load on one pile = 29.5 − ~~10~~ 8.1 = ~~19.5~~ 21.4 kg tons. (地盤ノ支持力 10,000 kg/m² (0.94 tons/□') ヲ許シタル場合杭ニカカル荷重)

average skin friction = $\frac{21.4 \times 2206}{207.5}$ = 228 #/□' ok.

Case 3. Stability during Earthquake. (Seismic forces backward) k = 0.3 assumed.

Loads	Hor. forces	Vert. forces		Lev. arm		moment
D		140,000	×	+ 3.00	=	420,000
D'	42,000		×	+ ~~3.77~~ 7.30	=	306,500
S		850,000	×	+ ~~3.25~~ 3.77	=	3,205,000
S'	255,000		×	+ 4.86	=	1,240,000
W₁		888,000	×	+ 5.20	=	4,620,000
W₂		294,000	×	+ 1.20	=	352,500
B		520,500	×	+ 3.25	=	1,692,000
B'	156,200		×	+ 0.66	=	103,000
E'e	279,000		×	+ 1.50	=	418,500
	732,200	2,692,500 kg	×	4.59 m	=	12,357,500



Eccentricity ε = 4.59 − 3.25 = 1.34 m Resultant force outside of middle third.

pressure area = 3.25 − 1.34 = 1.91 × 3 × 26.1 = 149.5 m²

max. heel pressure = $\frac{2692500 \times 2}{149.5}$ = ~~18~~ 36,050 kg/m² (or 3.3 tons/□')

max. load on one pile = 33,200 × .9 × .9 = 26.9 kg tons (杭ニテ支持スルモノトシテ)

If. 10,000 kg/m² (or 0.94 tons/□') be allowed for safe bearing of earth.

max. load on one pile = 26.9 − 10 × .9 × .9 = 18.8 kg tons. (地盤ノ支持力 10000 kg/m² (or 0.94 tons/□') ヲ許シタル場合ノ杭ニカカル荷重)

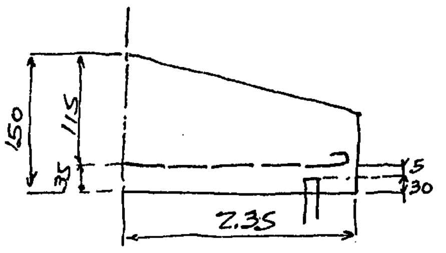
Average skin friction of pile

The former case 26.9 × 2206 ÷ 207.5 = 286 #/□'

The latter case 18.8 × 2206 ÷ 207.5 = 200 "

Design of Jiusō-Bashi for Osaka-prefecture.

Design of Cantilever footing at toe.



At. normal state.	at face of wall.		at toe
Upward pressure	17,500 kg/m²		19,830 kg/m²
wt. of footing	-3,600		-2,160
wt. of earth on footing	-4,800		-5,760
	9,100	+	11,910 = 21010÷2 = 10,500 kg/m² average.

upward pressure = $10500 \times 2.35 = 24,700$ kg
moment = $24,700 \times 1.23 = 30,400$ kgm per meter strip of footing.

During Earthquake.			
Upward pressure	21,800		40,000
weight of footing	-3,600		-2,160
wt. of earth on footing	-4,800		-5,760
	13,400	+	32,080 = $\frac{45480}{2}$ = 22,740 kg/m² average.

Upward force = $22740 \times 2.35 = 53,400$ kg per meter strip.
moment = $53,400 \times 1.34 = 71,600$ kgm
Shear = 53400 kg
moment and shear during earthquake govern.
Effective depth required = $\sqrt{\frac{71,600 \times 100}{100 \times 7.18 \times 18}} = 74.4$ cm

Use 115 cm eff. depth
Steel area req'd = $\frac{71600 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 115} = 32.9$ cm²
use 25 mmφ bars at 15 cm ctoc = 32.8 cm²
Steel ratio $p = \frac{32.8}{100 \times 115} = .00285$ $k = 0.25$, $j = 0.916$

$f_s = \frac{71600 \times 100}{32.8 \times .916 \times 115} = 2070$ kg/cm² $< 1200 \times 1.8 = 2160$ ok.

$f_c = \frac{2070 \times .25}{15(1-.25)} = 46$ kg/cm² $< 45 \times 1.8 = 81$ ok.

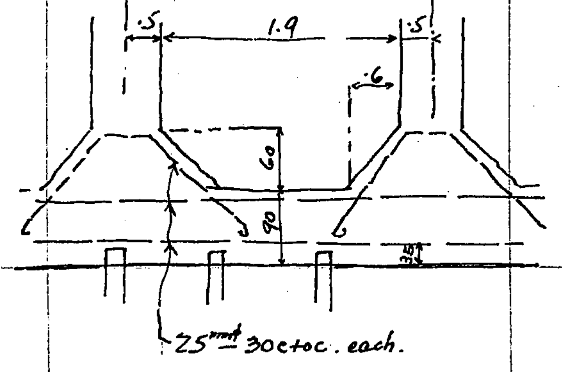
Unit Shear = $\frac{53400}{100 \times .916 \times 115} = 5.1$ kg/cm² $< 4 \times 1.8 = 7.2$ ok

Unit bond = $\frac{53400}{7.85 \times 6.67 \times .916 \times 115} = 9.7$ " $< 6 \times 1.8 = 10.8$ ok.

Footing at heel.

max. span length = 2.9 m near center
During Earthquake Case 2.
no upward pressure. downward pressure = Earth $8.65 \times 1600 = 13,830$
footing $.9 \times 2400 = 2160$
15,990 kg/m²

moment = $\frac{1}{10} \times 15,990 \times 2.9^2 = 13,450$ kgm per meter strip.
Effective depth reqd = $\sqrt{\frac{13450 \times 100}{100 \times 7.18 \times 1.8}}$ = ~~43.3~~ cm 32.3 cm



use 55 cm effective depth.
Steel area reqd = $\frac{13450 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 55} = 12.94$ cm²
Use 25 mmφ bars at 30 cm ctoc = 16.4 cm².
Steel ratio $p = \frac{12.94}{100 \times 55} = .00235$, $k = .229$ $j = .924$

Shear = $15990 \times 1.9 \div 2 = 15,200$ kg
use 60 cm fillet at end. effective depth = 115 cm
unit shear = $\frac{15200}{100 \times .924 \times 115} = 1.4$ kg/cm² ok

unit bond = $\frac{15200}{7.85 \times 3.33 \times .924 \times 115} = 5.5$ kg/cm² ok.

Use same details throughout the abutment base.

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CALCULATIONS FOR

Design of Jiuso-Bashi for Osaka prefecture.

Design of wing wall.

unbalance of earth fill = 3 meters assumed.

earth pressure at normal state = $\frac{1}{3} \times 1600 \times 3 = 1600$ kg/m²

" " during earthquake = $.662 \times 1600 \times 3 = 3180$ "

latter governs the section.

effective depth.

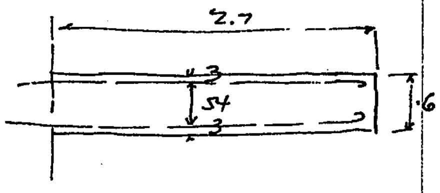
moment = $3180 \times 2.7^2 \times \frac{1}{2} = 11600$ kgm.

Shear = $3180 \times 2.7 = 8580$ kg

effective depth reqd. = $\sqrt{\frac{11600 \times 100}{100 \times 7.18 \times 1.8}} = 30.0$ cm

Use 57cm effective depth with 3cm insulation or 60cm over all.

Steel area reqd = $\frac{11600 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 57} = 10.77$ cm² per meter strip.



use 19mmφ bars at 25cm ctoc = 11.35 cm²

Unit Shear = $\frac{8580}{100 \times \frac{7}{8} \times 57} = 1.8$ kg/cm² ok

Unit bond = $\frac{8580}{5.03 \times 6.67 \times \frac{7}{8} \times 55} = 5.3$ " ok.

use same details for lower 5m

For upper 3 meters of wall, use 19φ bars at 40cm ctoc.

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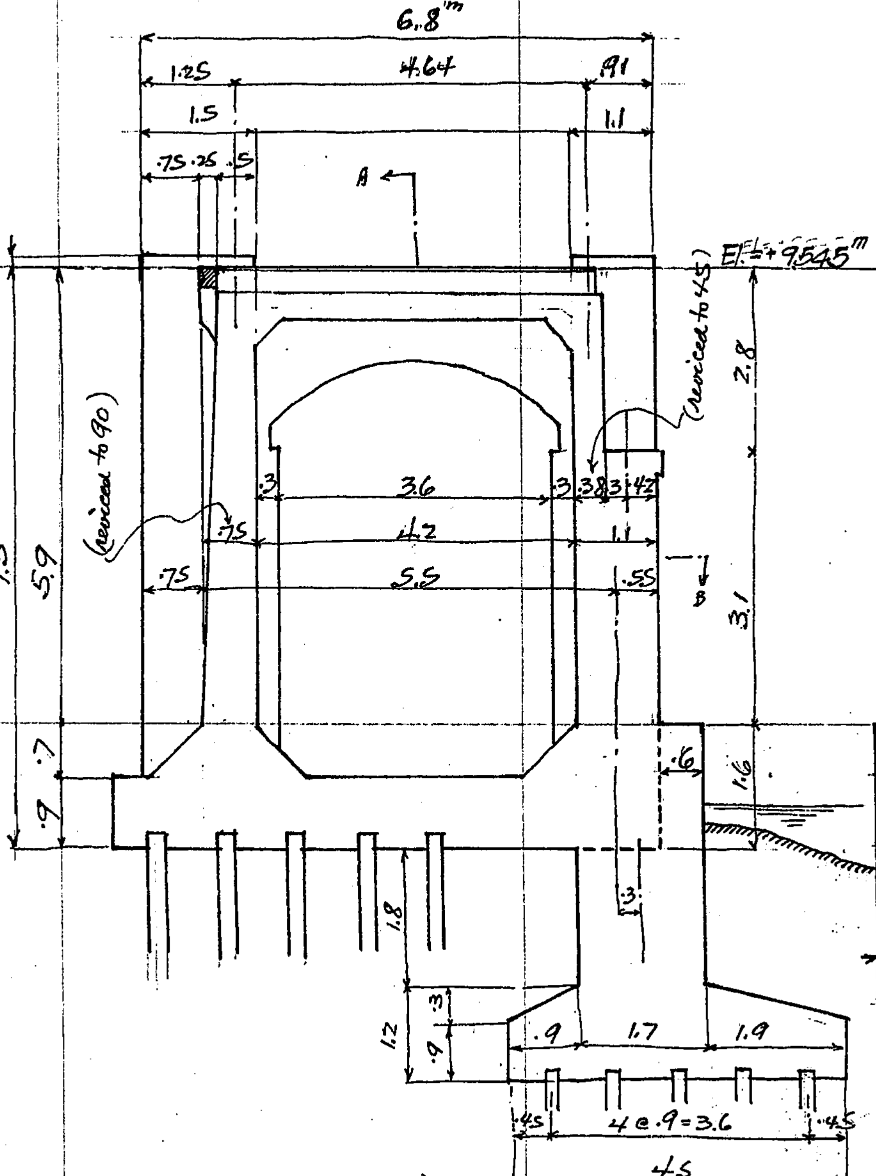
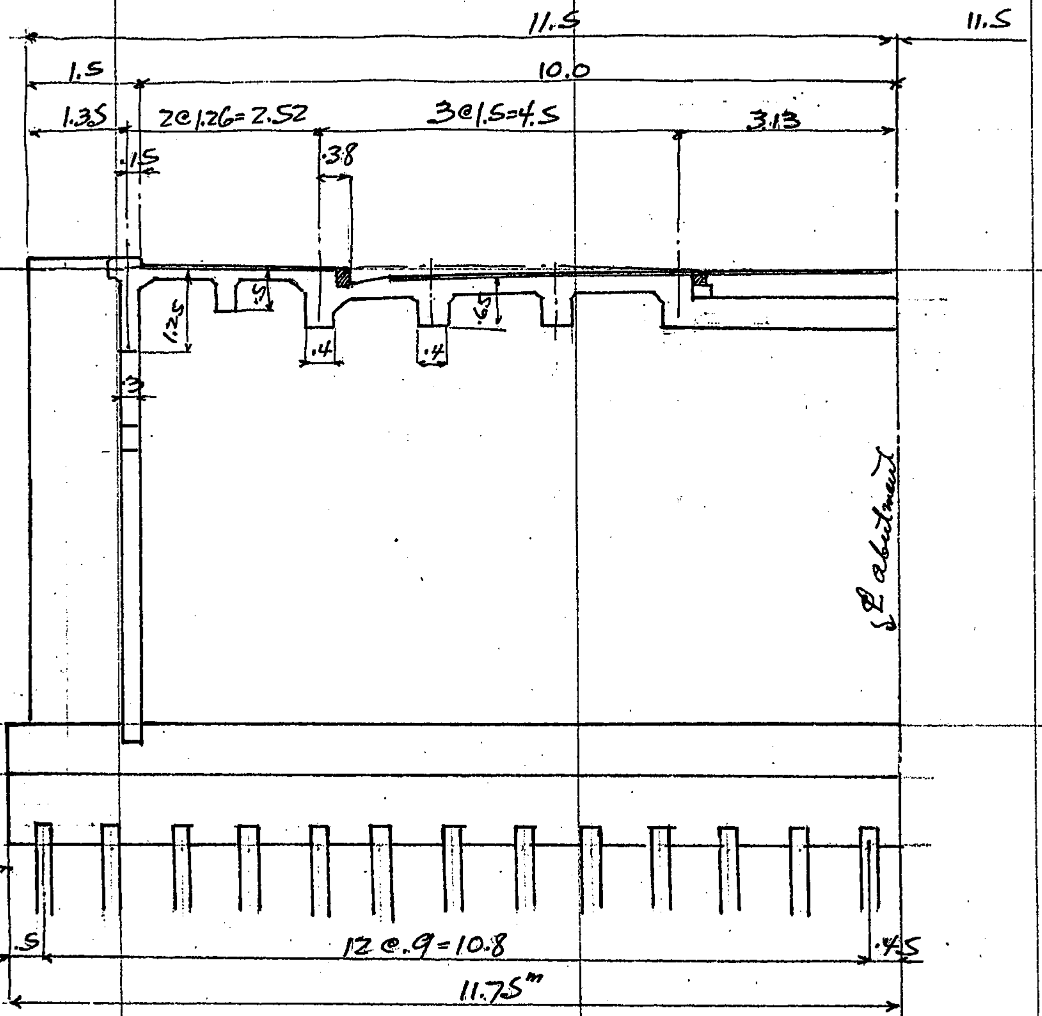
CALCULATIONS FOR

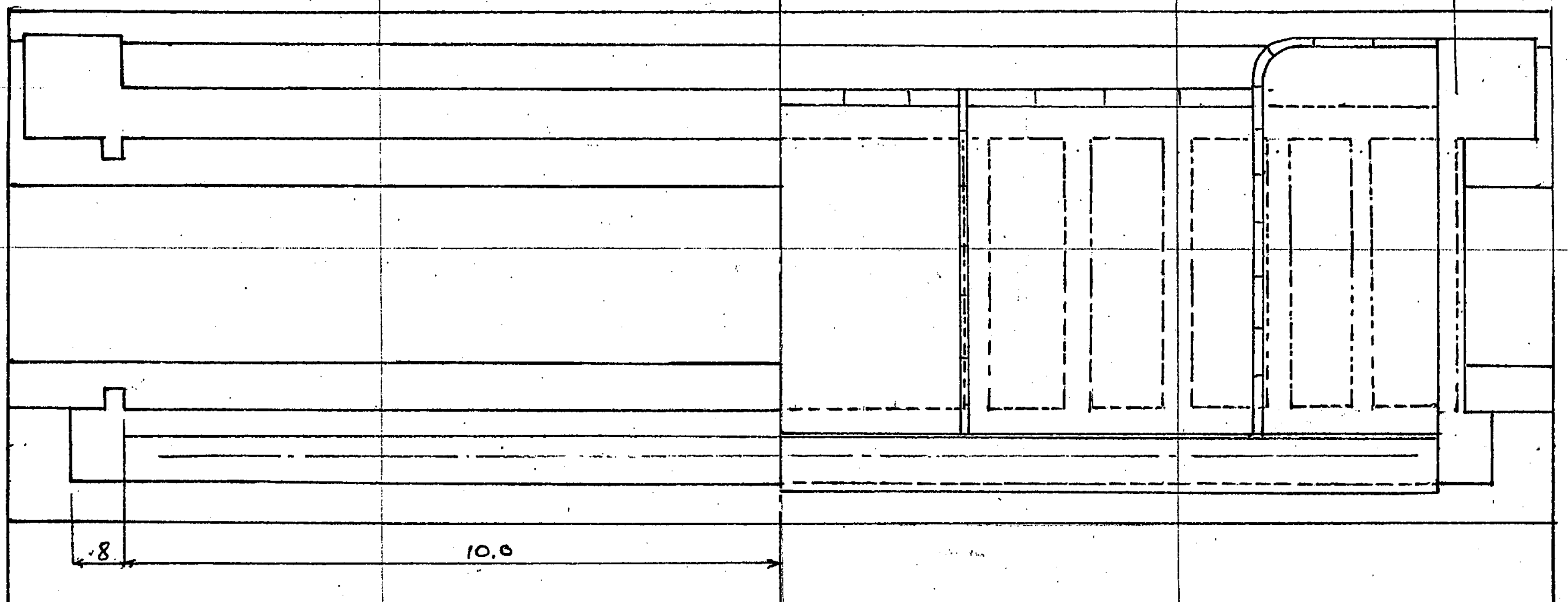
Design of Juso-ko-Bashi for Osaka prefecture
Design of Abutment.
General construction of abutment as shown on sketch below:



Section at ℄



Sectional Elev. A-A



Sectional Plan B-B

Plan.

Scale 1:100

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CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture.

Design of Floor.

Electric Railway slab. Span length 4.64 meters

Dead Load: Floor slab 38 cm @ 24 = 910
wt. of track complete assumed 815
1725 kg per sq. meter

Dead Load moment = $\frac{1}{8} \cdot 1725 \cdot 4.64^2 = 4650$ kgm per meter strip.
for continuity of slab, moment = $0.8 \cdot 4650 = 3720$ kgm per meter strip.
Dead Load shear = $\frac{1}{2} \cdot 1725 \cdot 4.64 = 4000$ kg

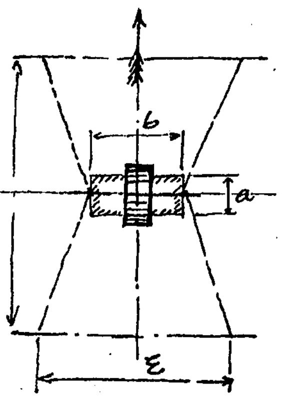
Live Load. Motor truck loading.

Rear wheel concentration	4500 kg	Front wheel concentration	1500
impact 30%	1350	impact 30%	450
	5850 kg.		1950 kg.

Distribution of wheel concentration on slab.

Longitudinal distribution a
contact between wheel and pavement = .20
distribution 38 × 2 = .76
a = .96

Transverse distribution b
width of wheel .39
distribution 38 × 2 = .76
b = 1.15

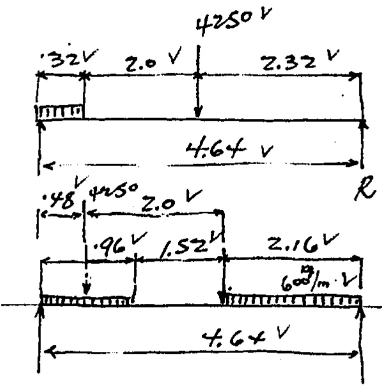


Effective width of slab E = ⅔ l + b = ⅔ × 4.64 + 1.15 = 4.24 m.
Use 2.0 meters as effective width.

Load per meter strip.
Rear wheel concentration with impact = 5850 ÷ 2 = 2925 kg for single wheel.

For 2 wheels 0.9 m between centers.
Effective width say 2.0 + 0.9 = 2.9 m.
Rear wheel concentration with impact = 5850 × 2 ÷ 2.9 = 4010 kg per meter strip.

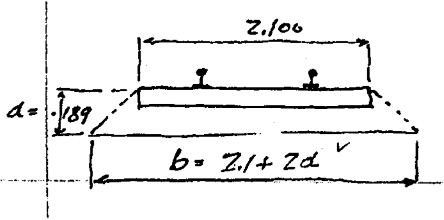
For 2 rows of motor trucks.
Effective width say total width = 5.5 m assumed.
Rear wheel concentration with impact = 5850 × 4 ÷ 5.5 = 4250 kg per meter strip
The last case gives the max moment & shear due to motor truck wheels.

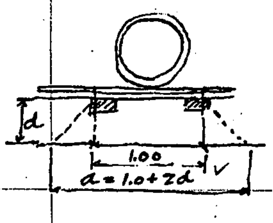


Reaction
Rear wheel. $4250 \times 2.32 = 9850$
Unif. load. $600 \times \frac{.32^2}{2} = 30$
$9880 \div 4.64 = 2130$ kg

Live Load moment = $2130 \times 2.32 = 4940$ kgm per ~~[illegible]~~ meter strip.
for continuity of slab m = $0.8 \times 4940 = 3950$ kgm
End shear due to
Rear wheel $4250 \times 4.16 = 17680$
Unif. load $600 \times \frac{2.16^2}{2} = 1400$
$19080 \div 4.64 = 4120$ kg per meter strip.

Electric car Loading. water splinker. axle load 9000 kg.
Distribution for single track.
Transverse distribution b = 2.1 + 2 × .189 = say 2.48 m
Longitudinal " a = 1.0 + 2 × .189 = 1.38 m





Car loading axle concentration	9000
impact 30%	2700
	11700 kg

Load per meter strip = 11700 ÷ 2.48 = 4720 kg

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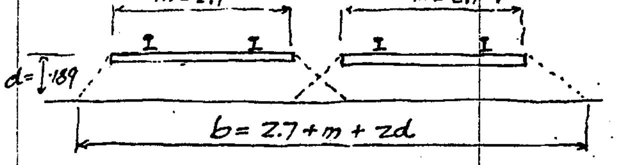
CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.

Distribution for double track.

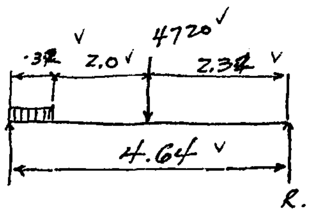


Transverse distribution = b = 2.7+2.1+2×.189 say = 5.18 m
Longitudinal " a = 1.0+2×.189 " = 1.38 m

Car Loading with 30% imp. = 11,700
for double line. 2×11700 = 23400 kg
Load per meter strip = $\frac{23400}{5.18}$ = 4,520 kg. < 4720

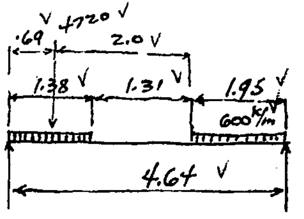
Single track loading governs.

Live Load moment for Single track loading for One axle load.



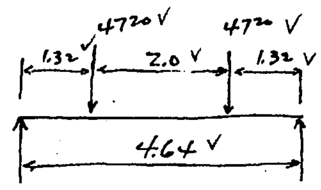
Reaction R. 4720×2.37 = 10,950
600×$\frac{.37^2}{2}$ = 30
10980÷4.64 = 2,370 kg

moment = 2370×2.32 = 5500 kgm.
for continuity of slab m = .8×5500 = 4400 kgm per meter strip.

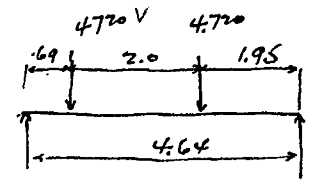


End Shear.
Rear wheel. 4720×3.95 = 18,650
Unif. load 600×$\frac{1.95^2}{2}$ = 1,140
19,790÷4.64 = 4,260 kg per meter strip.

Live Load moment for single track 2 axle loading.



Reaction = 4720
moment = 4720×1.32 = 6230
for continuity of slab m = 6230×.8 = 4,980 kgm per meter strip.



End shear.
4720×3.95 = 18,650
4720×1.95 = 9,200
27,850÷4.64 = 6000 kg per meter strip.

Summary for

	2 rows of motor truck		Electric car loading			
			Single track one axle load.		Single track 2 axle loads.	
	moment	end shear	moment	end shear	moment	end shear.
Dead Load	3720	4000	3720	4000	3720	4000
Live Load.	3950	4120	4400	4260	4980	6000
	7,670 kgm	8,120 kg	8,120 kgm	8,260 kg	8,700 kgm	10,000 kg

Effective depth required for f_s = 1200 kg/cm², f_c = 45 kg/cm², R = 7.18

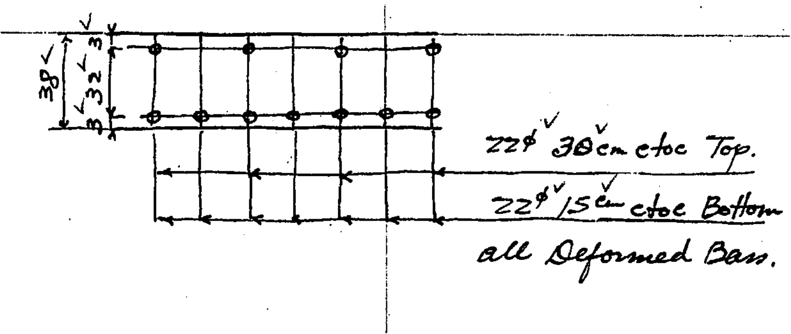
$\sqrt{\frac{M}{bR}} = \sqrt{\frac{8700\times100}{100\times7.18}}$ = 34.8 cm use 35 cm effective depth with 3 cm insulation or totatal depth of 38 cm.

Steel area req'd. for moment = $\frac{8700\times100}{1200\times\frac{7}{8}\times35}$ = 23.7 □cm per meter strip.

Use 22φ bars at 15 cm ctoc = 25.4 □cm per meter strip.
unit shear = $\frac{10,000}{100\times\frac{7}{8}\times35}$ = 3.37 kg/cm² ok

unit bond. $\frac{10000}{6.91\times6.67\times\frac{7}{8}\times35}$ = 7.98 Use Deformed bars. ok.

See page A35 for end construction.



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CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.

Floor Slab under Roadway. Span length 1.5 meters.

Dead Load.

5 cm asphalt block pavement	@ 21 kg	= 105 kg
2 cm mortar cushion	@ 22	= 44
17 cm concrete slab	@ 24	= 408
miscellaneous concrete say		13
		570 kg per sq. meter.

Dead Load moment = $\frac{1}{10} \times 570 \times 1.5^2 = 128$ kgm.
Dead Load shear = $\frac{1}{2} \times 570 \times 1.5 = 427$ kg.

Live Load. motor truck loading.

Rear wheel concentration with 30% imp. = 5850 kg
Front - - - - - = 1950 kg

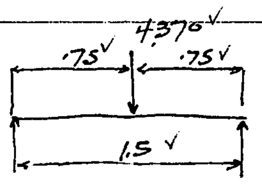
Distribution of wheel concentration on slab. Thickness of pavement and cushion = 7 cm.

Longitudinal distribution a. Contact between wheel + pavement = 20 cm
distribution 2 @ 7 = 14
a = 34 cm

Transverse distribution b. 39 + 2 × 7 = b = 53 cm.

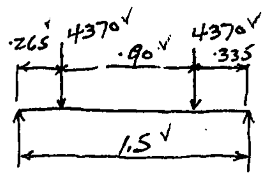
Effective width of slab. $E = \frac{2}{3} \cdot l + a = \frac{2}{3} \times 1.5 + .34 = 1.34$ meters.

Load per meter strip = 5850 ÷ 1.34 = 4370 kg for rear wheel concentration
1950 ÷ 1.34 = 1455 " " front " "



moment $\frac{4,370}{2} \times \frac{1.5}{2} = 1640$ kgm

For Continuity of slab moment = 0.8 × 1640 = 1,310 kgm per meter strip.



End shear
4370 × .335 = 1,450
4,370 × 1.235 = 5,390
6,840 ÷ 1.5 = 4,560 kg. per meter strip.

Summary for moment and End shear.

	moment	End Shear
Dead Load	128	427
Live Load.	1310	4,560
	1,438 kgm	4,987 kg

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

$d = \sqrt{\frac{M}{bR}} = \sqrt{\frac{1438 \times 100}{100 \times 7.18}} = 14.15$ cm

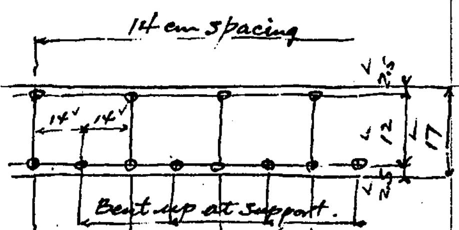
Use effective depth 14.5 cm with 2.5 cm insulation total depth being 17.0 cm.

Steel area required for moment.

$= \frac{1438 \times 100}{1200 \times \frac{7}{8} \times 14.5} = 9.46$ □cm per meter strip Use 13 mmφ bars at 14 cm ctoc = 9.51

Steel ratio = $\frac{9.\overset{51}{46}}{100 \times 14.5} = .0066$ $k = .356$, $j = .881$.

$f_s = \frac{1438 \times 100}{9.51 \times .881 \times 14.5} = 1187$ kg/cm² ok



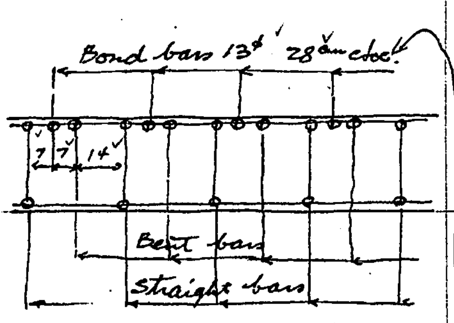
Section at center of span.

$f_c = \frac{1187 \times .356}{15(1 - .356)} = 43.8$. ok

Unit Shear = $\frac{4987}{100 \times .881 \times 14.5} = 3.90$ kg/cm² ok.

Bond stress for one 13 mmφ bars.
$= \frac{4987}{4.08 \times .881 \times 14.5} = 95.7$ kg/cm²

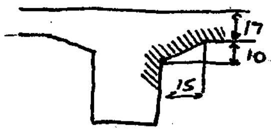
No. of Deformed 13 mmφ bars for bond stress
$= \frac{95.7}{9.0} = 10.63$ bars per meter strip.



Section at support.

Use 1-13φ bar in every other spaces of main bars, or total no of bars per meter strip of slab = $\frac{100}{14} \times 1.5 = 10.71$

Use deformed bars for all reinforcements.
Note: If 10 cm × 15 cm fillets at both ends of slab are used, bond bars are not necessary.



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CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture.

Slab under Sidewalk. Span length 1.26 meters.

Dead Load.	Concrete 10 cm @ 24 =	240
	wearing course 2.5 cm @ 21 =	53
	miscellaneous say	7
		300
Live Load		500
		800 kg per sq. meter.

Dead + Live Load moment $= \frac{1}{10} \times 800 \times 1.26^2 = 127$ kgm per meter strip.

" " shear $= \frac{5}{8} \times 800 \times 1.26 = 630$ kg " " "

Effective depth required for steel stress of 1200 kg/cm² & Concrete stress of 45 kg/cm²

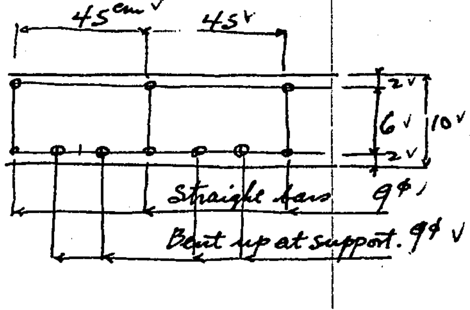
$d = \sqrt{\frac{M}{bR}} = \sqrt{\frac{127 \times 100}{100 \times 7.18}} = 4.2$ cm   use 8 cm effective depth with 2 cm insulation a total depth of 10 cm.

Steel area required for moment $= \frac{12700}{1200 \times \frac{7}{8} \times 8} = 1.51$ cm² per meter strip

use 9 mm φ bars at 15 cm c to c = 4.25 cm² per meter strip.

Steel ratio $= \frac{4.25}{100 \times 8} = 0.0053$, $k = .331$, $j = .890$

$f_s = \frac{12700}{4.25 \times .89 \times 8} = 420$ kg/cm², $f_c = \frac{420 \times .331}{15(1-.331)} = 13.9$ kg/cm²

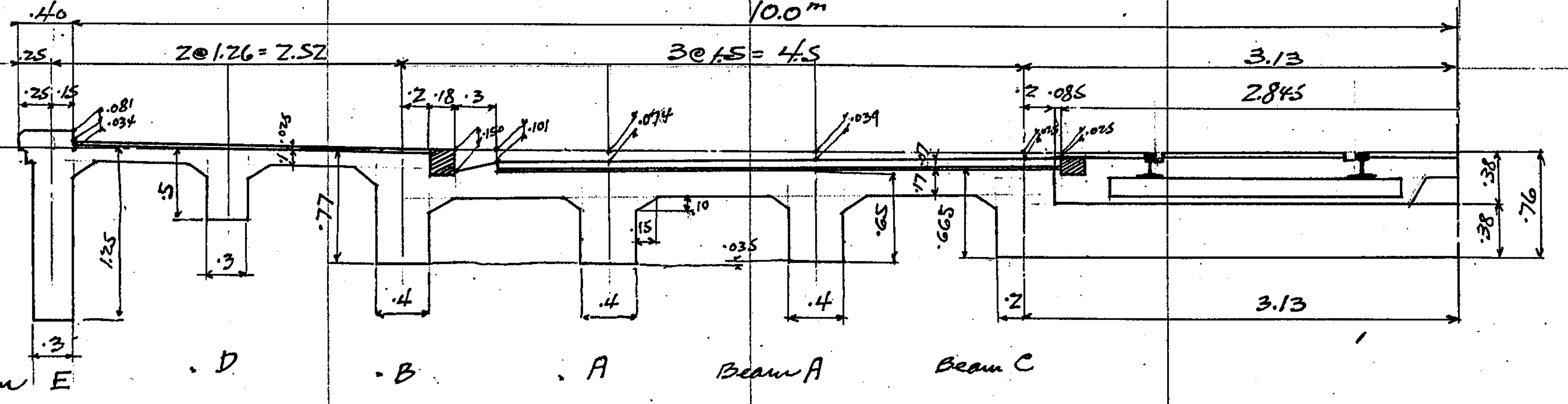


Unit Shear $= \frac{630}{100 \times .89 \times 8} = 0.885$ kg/cm² ok.

Unit bond $= \frac{630}{\frac{2.83}{.15} \times .89 \times 8} = 4.67$ kg/cm² ok.

Use deformed bars.

Design of T-Beams for Roadway and Sidewalk.



Design of Beam A. Span length 4.64 meters, spacing 1.5 meters.

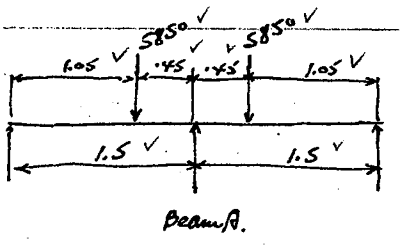
Dead Load	Slab and pavement	1.5 @ 570 =	855
	Stem of Beam	.4 × .48 @ 2400 =	461
	Fillets	.1 × $\frac{.15}{2}$ × 2 @ " =	36
	Miscellaneous concrete	say	8
			1360 kg per lin meter of span

Dead Load moment $= \frac{1}{10} \times 1360 \times 4.64^2 = 2930$ kgm. per meter strip.

Dead Load shear $= \frac{1}{2} \times 1360 \times 4.64 = 3155$ kg " "

Live Load. Motor truck rear wheel concentration with impact = 5850 kg.

" " front " " " " = 1950 "



Transver distribution

Load on Beam A. $\frac{5850 \times 1.05}{1.5} \times 2 = 8190$ kg rear wheel.

$8190 \div 3 = 2730$ kg front wheel.

Uniform Live load $= 600 \times 1.5 = 900$ kg per lin meter of beam.

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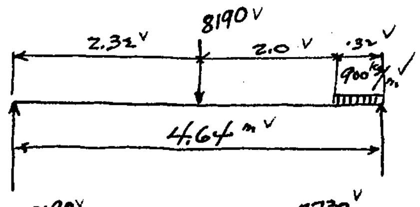
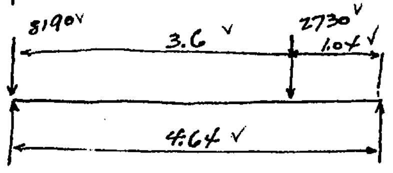
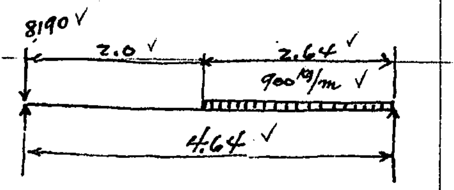
CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka prefecture.







Reaction $8190 \times 2.32 = 19000$
$\frac{900 \times .32^2}{2} = \underline{46}$
$19046 \div 4.64 = 4100$ kg

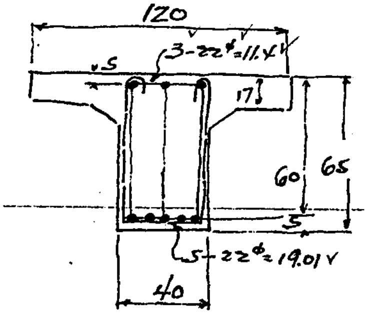
moment $= 4100 \times 2.32 = 9510$ kgm
For continuity of slab, moment $= 9510 \times .8 = 7610$ kgm
End shear for Rear and front wheel.
$\frac{2730 \times 1.04}{4.64} = 612$
$\underline{8190}$
8802 kg

End shear for Rear wheel and uniform load
$\frac{900 \times 2.64^2}{2 \times 4.64} = 676$
$\underline{8190}$
8866 kg

Summary for moments & end shears.

	moment	Shear
Dead Load	2930	3155
Live Load.	7610	8866
	10540 kgm	12021 kg

Section at center of span



Steel area required $= \frac{10540 \times 100}{1200 \times \frac{7}{8} \times 60} = 16.75$ cm²

Try 5-22φ bars on bottom and 3-22φ on top.

$p = \frac{19.01}{120 \times 60} = 0.00264$, $p' = \frac{11.4}{120 \times 60} = 0.00159$

$\Delta = \frac{t}{d} = \frac{17}{60} = .283$, $d'/d = \frac{5.0}{60} = .083$ $\frac{\Delta}{n} = .283 \div 15 = .019$, $\frac{\Delta^2}{2n} = .0027$

$k = \frac{p + p'(\frac{d'}{d}) + \frac{\Delta^2}{2n}}{p + p' + \frac{\Delta}{n}} = \frac{.00264 + .00159 \times .083 + .0027}{.00264 + .00159 + .0190} = .235 < \Delta$

neutral axis in the flange.
From the prepared diagrams of double reinforced rectangular beam.
$k = .24$, $j = .92$

$f_s = \frac{10540 \times 100}{19.01 \times .92 \times 60} = 1005$ kg/cm² ok.

$f_c = \frac{f_s k}{n(1-k)} = \frac{1005 \times .24}{15 \times .76} = 21.2$ kg/cm² ok.

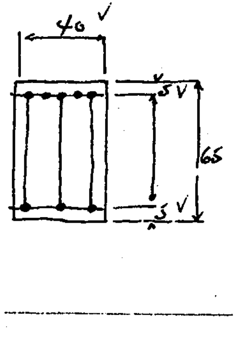
Unit shear $= \frac{12021}{40 \times .92 \times 60} = 5.44$ use stirrups.

Unit bond $= \frac{12021}{6.91 \times .92 \times 60} = 31.5$ kg/cm² for one bar

no. of bars at end for bond stress $= \frac{31.5}{9} = 3.5$ bars. use ~~4 or~~ 5 on top.

Stirrups U-shape 13φ. area $= 2 \times 1.33 = 2.66$ cm² $= A_s$
Spacing $S = \frac{3}{2} \cdot \frac{A_s . f_s . jd}{V} = \frac{3}{2} \cdot \frac{2.66 \times 900 \times .92 \times 60}{12021} = 16.5$ cm at end.

Section at end.



Moment at end assumed same value as for center.
Steel. 5-22φ on top $= 19.01$ cm²
3-22φ on bottom $= 11.4$ "

Steel ratio $p = \frac{19.01}{40 \times 60} = .0079$, $p' = \frac{11.4}{40 \times 60} = .0048$ $\frac{d'}{d} = .083$

$k = .353$, $j = .895$

$f_s = \frac{10540 \times 100}{19.01 \times .895 \times 60} = 1032$ kg/cm² ok.

$f_c = \frac{1032 \times .353}{15(1-.353)} = 37.6$ kg/cm² ok.

Assumed section is ample.

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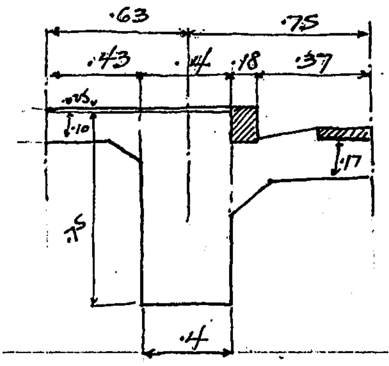
CHECKED BY M. Jinda DATE 4-6-19 PAGE NO A28

CALCULATIONS FOR

Design of Juso-Bashi for Osaka prefecture

Design of Beam B.
Dead Load.



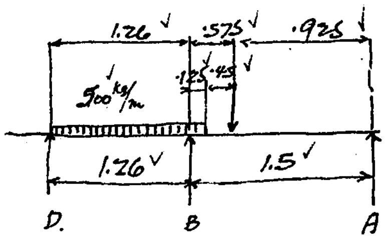
Span length 4.64 meters

Slab and pavement		say .37 @ 570 =	211.
Slab end.	.22 × .18 =	.04 @ 2400 =	96
Stem of beam	.4 × .75 =	.30 @ " =	720
Sidewalk slab	.1 × .43 =	.043 @ " =	103
Fillet "	.15 × .15 ÷ 2 =	.011 @ " =	26
Sidewalk pavement	.83 × .025 =	.021 @ 2100 =	44
Curb stone, granite	.18 × .16 =	.029 @ 2600 =	75
			1275 kg per lin meter.

Dead Load moment $= \frac{1}{10} \times 1275 \times 4.64^2 =$ 2,750 kgm

Dead Load shear $= \frac{1}{2} \times 1275 \times 4.64 =$ 2960 kg

Live Load: Transverse distribution. motor truck wheels close contact to curb line assumed.



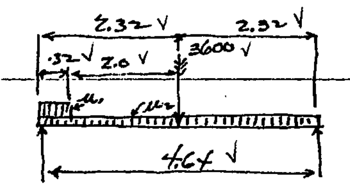
Load on Beam B.

motor truck rear wheel concentration with impact $= 5850\text{ kg} \times \frac{.925}{1.5} = 3600\text{ kg}$

" " front " " " " say $= 1950 \times \frac{.925}{1.5} = 1200\text{ kg}$

Uniform live load on sidewalk. $\frac{500 \times 1.385^2}{2 \times 1.26} =$ 380 kg/m of beam (M_2)

Unif. load on carriageway on front + rear of truck $\frac{600 \times 1.12^2}{2 \times 1.5} =$ 250 kg/m of beam. (M_1)



moment due to.

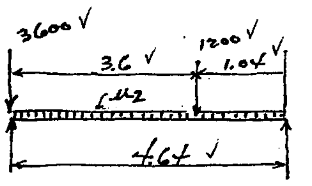
Rear wheel $\frac{3600 \times 4.64}{4} =$ 4175

M_1 $\frac{250 \times .32^2}{2 \times 4.64} = 3$

M_2 $380 \times \frac{4.64}{2} = 882$

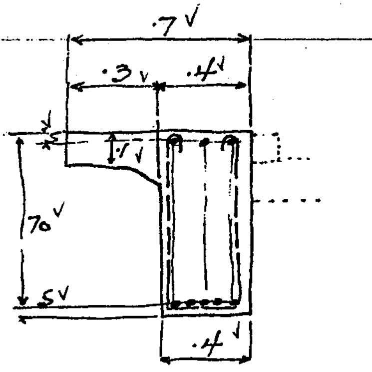
$\Big\} 885 \times 2.32 =$ 2005

6,180 kgm. for continuity of beam
$m = 0.8 \times 6180 = 4950$ kgm



End shear.

Rear wheel	–	3600
front wheel	$1200 \times \frac{1.04}{4.64} =$	270
M_2	$380 \times 2.32 =$	880
		4,750 kg.



Assumed section of Beam B.

Summary for moments and shears.

	~~Dead Load~~ moment	End shear
Live Load	4950	4750
Dead Load.	2750	2960
	7700 kgm	7,710 kg.

Steel area required $= \frac{7700 \times 100}{1200 \times \frac{7}{8} \times 70} = 10.48\text{ cm}^2$

Try $5\text{-}19^{\phi} = 14.18\text{ cm}^2$ on ~~bott.~~ + $3\text{-}19^{\phi} = 8.5\text{ cm}^2$ on top

Steel ratio $p = \frac{14.18}{70 \times 70} = .0029$, $p' = \frac{8.5}{70 \times 70} = .0017$

$\Delta = \frac{t}{d} = \frac{10}{70} = .143$, $d'/d = 5/70 = .0715$, $\frac{\Delta}{n} = \frac{.143}{15} = .0096$, $\frac{\Delta^2}{2n} = \frac{.143^2}{30} = .0007$

$k = \frac{.0029 + .0017 \times .0715 + .0007}{.0029 + .0017 + .0096} = 0.26 > \Delta = .143$

Neutral axis in the web. approx. value of $j = .93$

$f_s = \frac{7700 \times 100}{14.18 \times .93 \times 70} = 835\text{ kg/cm}^2$

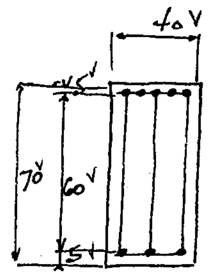
$f_c = \frac{835 \times .26}{15(1 - .26)} = 19.6\text{ kg/cm}^2$

Unit Shear $= \frac{7710}{40 \times .93 \times 70} = 2.96\text{ kg/cm}^2$ Ok. Use some stirrups 13^{ϕ} w-shape

Unit bond st for one bar $= \frac{7710}{5.97 \times .93 \times 70} = 19.8$

No. of bars req'd for bond stress at end $= \frac{19.8}{9} = 2.2$ Use more than 3 on top at ends

Design of Juso-Bashi for Osaka Prefecture.



Negative moment for Beam B at end. = assumed same value as for pos. m = 7,700 kgm

Steel Try 5-19ϕ = 14.18 cm² on top
3-19ϕ = 8.5 " " bottom.

Steel ratio $p = \frac{14.18}{40 \times 70} = .0051$, $p' = \frac{8.5}{40 \times 70} = .0030$

$k = .30$, $j = .91$

$f_s = \frac{7700 \times 100}{14.18 \times .91 \times 70} = 852$ kg/cm² ok.

$f_c = \frac{852 \times .30}{15(1-.30)} = 24.3$ " ok

Design of Beam C

Use Same reinforcements as for Beam A.
5-22ϕ bars on bottom
3 " " " top.
13ϕ U-stirrup 16.5 cm c to c at end.

Design of Beam D. under sidewalk. Span length 4.64 meters.

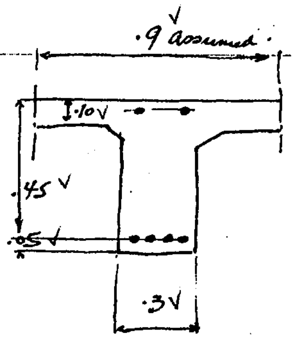
Dead Load. Slab and pavement 300 kg × 1.26 = 378
~~Lo~~ Stem of Beam .3 × .4 = .12
fillet .1 × .1 = .01
.13 × 2400 = 312
690 kg.

Dead Load moment = $\frac{1}{10} \times 690 \times 4.64^2 = 1,485$ kgm
" " End shear = $\frac{1}{2} \times 690 \times 4.64 = 1,600$ kg.

Live Load 500 kg/m² 500 × 1.26 = 630 kg/m
Live Load moment = $\frac{1}{10} \times 630 \times 4.64^2 = 1,355$ kgm
" " end shear = $\frac{1}{2} \times 630 \times 4.64 = 1,460$ kg

Summary for moment and shears

	moment	End shear
Dead Load.	1,485	1,600
Live Load.	1355	1460
	2,840 kgm.	3,060 kg



Approx. steel area required $= \frac{2840 \times 100}{1200 \times \frac{7}{8} \times 45} = 6.0$ cm²

Use 4-19ϕ = 11.33 cm² on bottom
& 2-19ϕ = 5.67 " on top.

Steel ratio $p = \frac{11.33}{90 \times 45} = .0028$, $p' = .0014$

$\Delta = t/d = 10/45 = .222$, $d'/d = 5/45 = .11$, $\frac{\Delta}{n} = \frac{.222}{15} = .0148$, $\frac{\Delta^2}{2n} = \frac{.222^2}{30} = .00165$

$k = \frac{.0028 + .0014 \times .11 + .00165}{.0028 + .0014 + .0148} = .247$

neutral axis in the web approx. $j = .92$

$f_s = \frac{2840 \times 100}{11.33 \times .92 \times 45} = 605$ kg/cm²

$f_c = \frac{605 \times .24}{15(1-.24)} = 12.7$ kg/cm²

Unit shear $= \frac{3060}{30 \times \frac{7}{8} \times 45} =$ ~~2.46~~ 2.6 kg/cm² OK. Use some stirrups at end (9ϕ U)

Bond stress for one bar $= \frac{3060}{5.97 \times \frac{7}{8} \times 45} = 13.0$

req'd. no of bars $= \frac{13.0}{9.0} = 1.45$ Use more than 2. on top at end.

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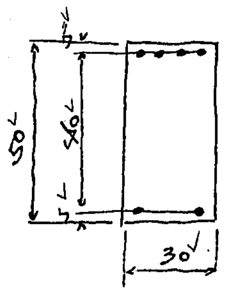
CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka Prefecture.

Neg. moment at end of Beam B assumed -2840 kgm.



Reinforcement 4-19φ = 11.33 cm² on top.
2-19φ = 5.67 " on bottom.

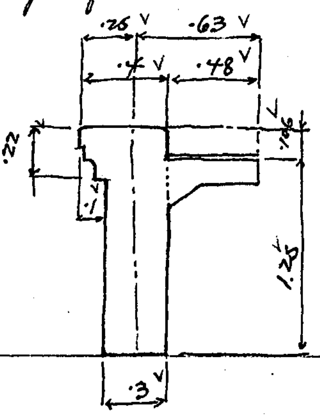
Steel ratio $p = \frac{11.33}{30 \times 45} = .0084$, $p' = 0.0042$, $d'/d = 5/45 = .11$

$K = .36$, $j = .885$

$f_s = \frac{2840 \times 100}{11.33 \times .885 \times 45} = 628$ kg/cm² ok.

$f_c = \frac{628 \times .36}{15(1-.36)} = 23.5$ " ok.

Design of Beam E.



Dead Load.

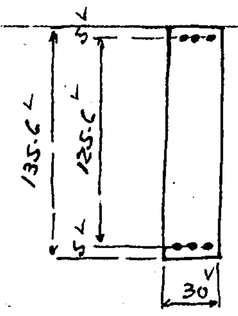
Slab and pavement		0.48 × 300 =	144
Fillet	$\frac{.1 \times .15}{2}$ =	.0075 × 2400 =	18
Beam	1.356 × .3 =	.407 × " =	977
Coping proj.	.22 × .1 =	.022 × " =	53
Handrail		=	90
Miscellaneous conc. &c		say =	18
			1,300 kg per lin meter.

Dead Load moment = $\frac{1}{10} \times 1300 \times 4.64^2 = 2,800$ kgm.
" " End shear = $\frac{1}{2} \times 1300 \times 4.64 = 3,020$ kg

Live Load. .48 × 500 = 240 kg per lin meter.
Live Load moment = $\frac{1}{10} \times 240 \times 4.64^2 = 520$ kgm.
" " End shear = $\frac{1}{2} \times 240 \times 4.64 = 560$ kg.

Summary for moment & shears.

	moment	End shear.
Dead Load.	2800	3020
Live Load.	520	560
	3320 kgm	3,580 kg



Steel req'd = $\frac{3320 \times 100}{1200 \times \frac{7}{8} \times 130.6} = 2.42$ cm²

Use
3-19φ = ~~11.33~~ 8.50 cm² on bottom. } ... at center of span.
2-19φ = 5.67 " " top

3-19φ on top } ----- at end of span.
2-19φ on bottom

Reinforcements, Revice to 3-19φ on top and bottom. See page A48

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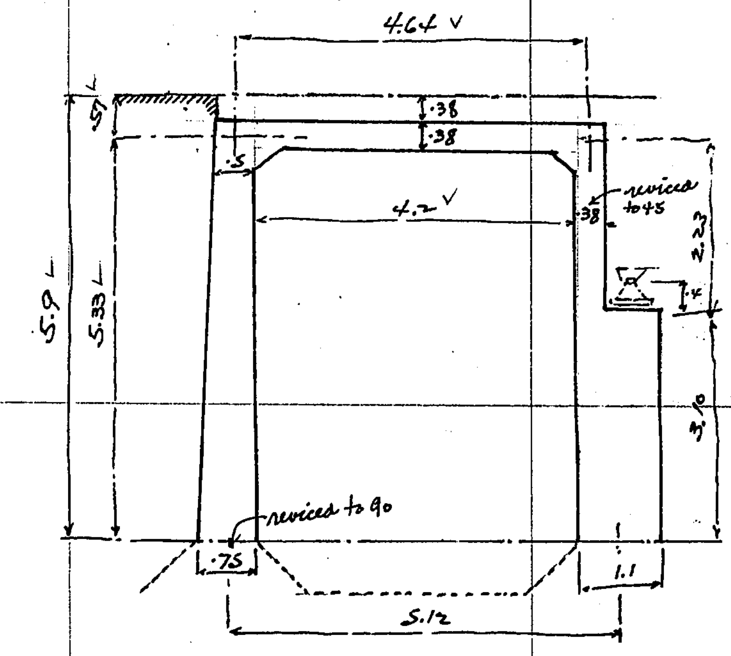
CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture.

Stresses on Frame under Electric Railway.

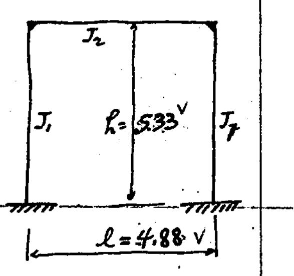


assumptions: Span length top 4.64

bottom 5.12

$9.76 \div 2 = 4.88^m = l$

height $h = 5.33^m$



moment of inertia J_2. 1 meter strip.

$J_2 = \frac{1.0 \times .38^3}{12} = .00458$

moment of inertia J_1.

rear wall. $\frac{100 \times \overset{.75}{\cancel{.65}}{}^3}{12} = .0351$

$\frac{100 \times .5^3}{12} = .0104$

$.0455 \div 2 = .02275$

front wall. $\frac{1.0 \times .38^3}{12} = .00458$

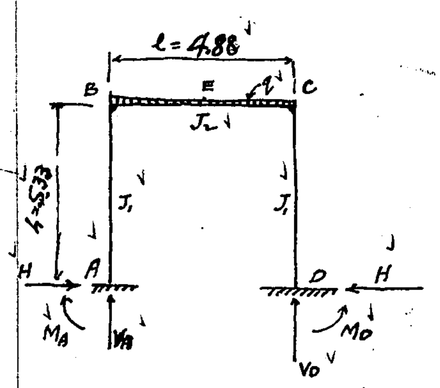
$\frac{1.0 \times 1.1^3}{12} = .10950$

$.11408 \div 2 = .05704$

$J_1 = .07979 \div 2 = .0400$

$K = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{.00458}{.0400} \cdot \frac{5.33}{4.88} = 0.125$

Dead Load.



$1725 \, kg/m. = q$ See page A23

$V = \frac{ql}{2} = \frac{1725 \times 4.88}{2} \quad 4210 \, kg$

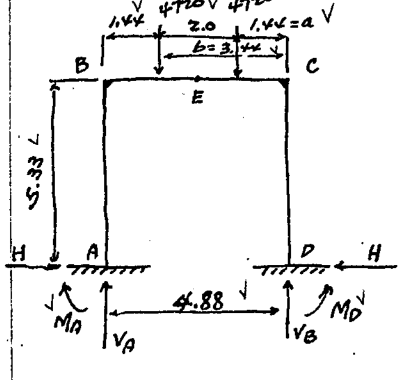
$H = \frac{ql^2}{4h(k+2)} = \frac{1725 \times 4.88^2}{4 \times 5.33 \times 2.125} = 907 \, kg$

$M_B = M_C = -\frac{ql^2}{6(k+2)} = -\frac{1725 \times 4.88^2}{6 \times 2.125} = -3220 \, kgm$

$M_A = M_D = +\frac{ql^2}{12(k+2)} = +1610 \, kgm$

$M_E = +\frac{ql^2}{8} - \frac{ql^2}{6(k+2)} = +\frac{1725 \times 4.88^2}{8} - 3220 = +1910 \, kgm.$

Live Load



Water splinker single track 2 axle loading see page A24

Concentration on one meter strip of slab = 4,720 kg 2.0 m c to c. longitudinary

$V = p = 4,720 \, kg$

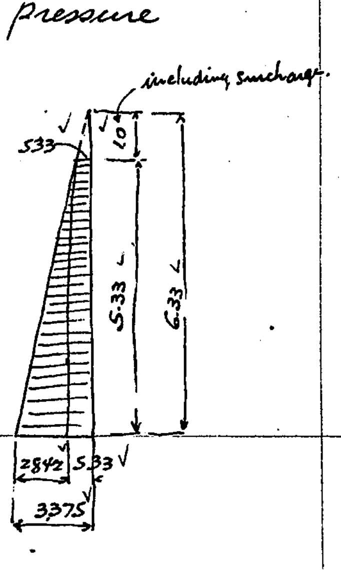
$H = \frac{3Pab}{hl(k+2)} = \frac{3 \times 4720 \times 1.44 \times 3.44}{5.33 \times 4.88 \times 2.125} = 1,270 \, kg$

$M_A = M_D = +H \cdot \frac{h}{3} = 1270 \times \frac{5.33}{3} = +2,260 \, kgm$

$M_B = M_C = -H \cdot \frac{2h}{3} = -2260 \times 2 = -4,520 \, kgm$

$M_E = Pa - M_B = 4,720 \times 1.44 - 4,520 = +2,280$

Earth pressure



Earth pressure at top. $\frac{1600 \times 1.0}{3} = 533 \, kg/m^2$

" " bottom $\frac{1600 \times 6.33}{3} = 3375$

$3375 - 533 = 2842 \, kg/m^2$

Constants.

$K = .125$
$k + 2 = 2.125$
$2K + 3 = 3.25$
$3k + 4 = 4.375$
$5k + 9 = 9.625$
$6k + 1 = 1.75$
$7k + 12 = 12.875$
$12K = 1.500$
$15k = 1.875$

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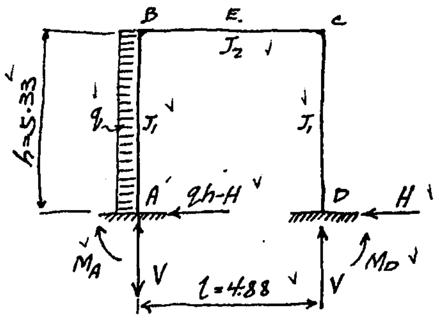
CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka Prefecture.



$V = \frac{qh^2k}{l(6k+1)} = \frac{533 \times 5.33^2 \times .125}{4.88 \times 1.75} = 222$ kg

$H = \frac{qh}{8} \cdot \frac{2k+3}{k+2} = \frac{533 \times 5.33}{8} \cdot \frac{3.25}{2.125} = 543$ kg , $qh - H = 533 \times 5.33 - 543 = 2300$ kg

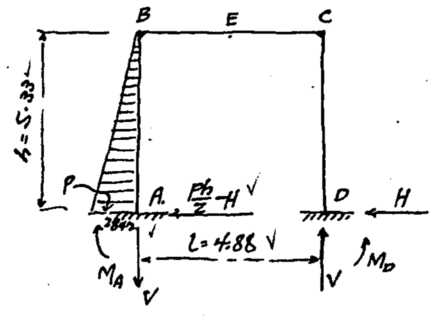
$M_A = -\frac{qh^2}{24}\left(12 - \frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = -\frac{533 \times 5.33^2}{24}\left(12 - \frac{9.625}{2.125} - \frac{1.5}{1.75}\right) = -4,180$ kgm

$M_D = +\frac{qh^2}{24}\left(+\frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = 632(4.53 - \cdot 858) = +2,320$ kgm

$M_B = M_A - Hh + \frac{qh^2}{2} = -4180 - 543 \times 5.33 + \frac{533 \times 5.33^2}{2} = +510$ kgm

~~$M_C = M_D - Hh = +2320 - 543 \times 5.33$~~ $= -580$

M_E = by propotion = −36 kgm



$V = \frac{Pkh^2}{4l(6k+1)} = \frac{2842 \times .125 \times 5.33^2}{4 \times 4.88 \times 1.75} = 295$ kg

$H = \frac{Ph}{40} \cdot \frac{3k+4}{k+2} = \frac{2842 \times 5.33}{40} \cdot \frac{4.375}{2.125} = 780$ kg

$\frac{Ph}{2} - H = \frac{2842 \times 5.33}{2} - 780 = 6,800$ kg

$M_A = -\frac{Ph^2}{120}\left(20 - \frac{7k+12}{k+2} - \frac{15k}{6k+1}\right) = -\frac{2842 \times 5.33^2}{120}\left(20 - \frac{12.875}{2.125} - \frac{1.875}{1.75}\right) = -8,660$ kgm

$M_D = +\frac{Ph^2}{120}\left(\frac{7k+12}{k+2} - \frac{15k}{6k+1}\right) = +673(6.06 - 1.072) = +3,360$ kgm

$M_B = M_A - Hh + \frac{Ph^2}{6} = -8660 - 780 \times 5.33 + \frac{2842 \times 5.33^2}{6} = +650$ kgm

$M_C = M_D - Hh = +3360 - 780 \times 5.33 = -800$ kgm.

M_E by propotion = −77 kgm.

Dead Load due to weight of wall on A + D.

Load on A. $\frac{.5 + .75}{2} = .625 \times 5.33 = 3.33 @ 2400 = 8,000$ kg

" " D $.38 \times 2.33 = .885$
$1.10 \times 3.10 = 3.410$
$4.295 @ 2400 = 10,300$ kg

Load on wall D. from Girder span

Girder		Dead Load		Live Load
G1	2@63500 =	127000	2@43000 =	86,000
G2	2@51900 =	103800	2@46000 =	92000
G3	1@60,000 =	60000	1@46000 =	46000
		290,800 kg		224,000 kg

Length of front wall = 21.6 m

Dead Load, average = 290800 ÷ 21.6 = 13460 kg per lin meter of wall.
Live Load, " = 224000 ÷ 21.6 = 10370 kg " " " "
DL + LL. = 23830 kg " " " "

Eccentricity = .13 m
moment on D. Dead Load moment $M_D = +13,460 \times .13 = +1,750$ kgm/m. (M_A M_B M_C etc neglected)
Live Load " $M_D = +10,370 \times .13 = +1,350$ " "

Summary for moments + Reactions

	M_A	M_D	M_B	M_C	M_E	H_A	H_D	V_A	V_D
						→+	+←	↓↑ −+	
Dead Load, abutment floor	+1610	+1,610	−3,220	−3220	+1910	+907	+907	+4210	+4210
" " + wt. of wall								+8000	+10300
" " girder span		+1750							+13460
	+1,610	+3,360	−3220	−3320	+1910	+907	+907	+12210	+27,970
Earth pressure	−4180	+2,320	+510	−580	−36	−2300	+543	−222	+222
" "	−8660	+3360	+650	−800	−77	−6800	+780	−295	+295
Live Load abutment	+2260	+2260	−4520	−4520	+2280	+1270	+1270	+4720	+4,720
" girder span		+1350							+10370
Total max.	−8970	+12,650	−2,060	−4,700	+4,077	−6923	+3,500	+16,413	+43,577
Total min.	−11,230	+9,040	−6,580	−9,220	+1,797	−8,193	+2,230	+11,693	+28,487

CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.

Stresses on frame under Electric Railway during Earthquake. (Seismic force ~~fora~~ forward)

Seismic forces on frame.

Dead Load Floor slab + Track $1{,}725 \times 4.\overset{88}{\cancel{64}} = \overset{8{,}420}{\cancel{8000}}$ kg per meter strip of frame.

Seismic force $= 0.3 \times \overset{420}{8\cancel{000}} = \overset{2{,}525}{\cancel{2400}}$ kg $\times \frac{5.52}{5.33} = 2620$ kg " " $= P_1$.

lev. arm

Weight of rear wall. $8{,}000$ kg $\times 0.3 = 2{,}400$ kg $\times 2.5 = 6{,}000$

" " front " $10300 \times 0.3 = \underline{3090}$ $\times 2.1 = \underline{6{,}490}$

$5{,}490$ kg $\times 2.28 = 12{,}490$

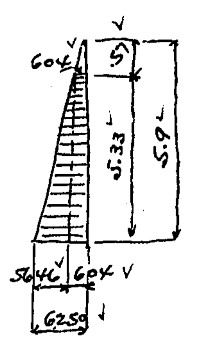
Equivalent uniform hor. load. $= \frac{5490}{5.33} \times \frac{2.28}{2.665} = \underline{\underline{880}}$ kg per lin meter of height $= q_1$

Earth pressure for 3000 mm earthquake $= 0.662\, wh$ kg/m²

top. $0.662 \times 1600 \times .57 = 604$ kg/m² ------------ q_2

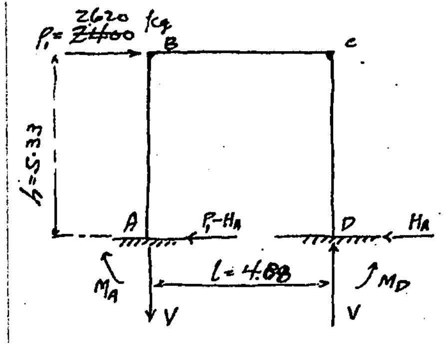
bottom. $0.662 \times 1600 \times 5.90 = \underline{6{,}250}$ "

$5{,}646$ " ------------ q_3



Dead Load due to Girder span. average $13{,}460$ kg $\times 0.3 = 4{,}040$ kg per lin meter of wall. --- P_2

Stresses due to P_1. Floor slab + track.



$$V = \frac{3Phk}{l(6k+1)} = \frac{3 \times 2620 \times 5.33 \times .125}{4.88 \times 1.75} = 613 \text{ kg}$$

$$H_A = \frac{P_1}{2} = \frac{2620}{2} = 1{,}310 \text{ kg} = H_D$$

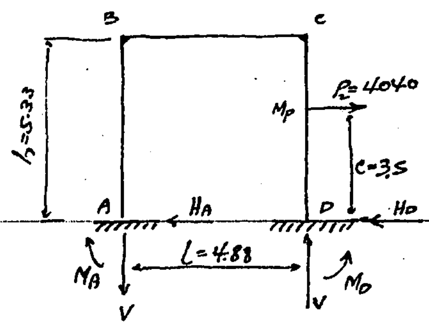
$$M_A = -\frac{P_1 h}{2} \cdot \frac{3k+1}{6k+1} = -\frac{2620 \times 5.33}{2} \cdot \frac{1.375}{1.75} = -5480 \text{ kgm}$$

$$M_D = +\frac{P_1 h}{2} \cdot \frac{3k+1}{6k+1} = \qquad +5480 \text{ "}$$

$$M_B = +\frac{P_1 h}{2} \cdot \frac{3k}{6k+1} = +\frac{2620 \times 5.33}{2} \times \frac{.375}{1.75} = +1{,}495 \text{ kgm}$$

$$M_C = -\frac{P_1 h}{2} \cdot \frac{3k}{6k+1} = \qquad -1{,}495 \text{ "}$$

Stresses due to P_2. D.L. of main girder. $\delta = c/h = 3.5/5.33 = .657$



$$V = \frac{3P_2 c\delta k}{l(6k+1)} = \frac{3 \times 4040 \times 3.5 \times .657 \times .125}{4.88 \times 1.75} = 375 \text{ kg}$$ (Vノ値ニ少シ誤差アルモ軽微ニ付 訂正セズ)

$$H_A = \frac{P_2\delta^2}{2(k+2)}\left[3(k+1) - \delta(2k+1)\right] = \frac{4040 \times .657^2}{2 \times 2.125}\left[3 \times 1.125 - .657 \times 1.25\right] = 1{,}050 \text{ kg} = H_D$$

$$H_D = P_2 - H_A = 4040 - 1050 = 2990 \text{ kg}$$

$$M_A = -\frac{P_2 c\delta}{2}\left[\frac{3+2k-\delta(k+1)}{k+2} - \frac{3k}{6k+1}\right] = -\frac{4040 \times 3.5 \times .657}{2}\left[\frac{3+.25-.657 \times 1.125}{2.125} - \frac{.375}{1.75}\right] = -4480 \text{ kgm}$$

$$M_D = +\frac{P_2 c\delta}{2}\left[\frac{2}{\delta} - \frac{3+2k-\delta(k+1)}{k+2} - \frac{3k}{6k+1}\right] = 4{,}640\left[\frac{2}{.657} - 1.180 - .214\right] = +7660 \text{ kgm}$$

$$M_B = M_A + H_A h = -4480 + 1050 \times 5.33 = +1{,}120 \text{ kgm}$$

$$M_C = M_D + H_A h - P_2 c = +7{,}660 + 1050 \times 5.33 - 4040 \times 3.5 = -890 \text{ kgm}$$

$$M_P = M_D - H_D c = +7660 - 2990 \times 3.5 = -2810 \text{ kgm}$$

Stresses due to $q_1 + q_2$ weight of walls + Earth pressure (a part 604)

$q_1 = 880$

$q_2 = \underline{604}$

$q = 1484$ kg per meter of height.

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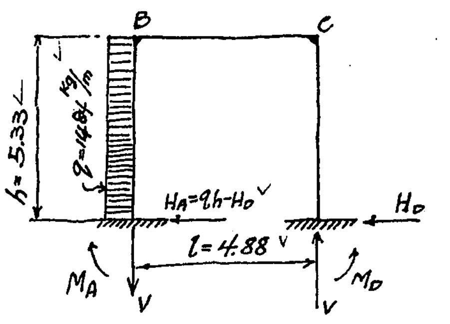
CHECKED BY M. Jinda DATE 4-6-23 PAGE NO 934

CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.



$$V=\frac{qh^2k}{l(6k+1)}=\frac{1484\times5.33^2\times.125}{4.88\times1.75}=618\text{ kg}$$

$$H_D=\frac{qh}{8}\cdot\frac{2k+3}{k+2}=\frac{1484\times5.33}{8}\cdot\frac{3.25}{2.125}=1515\text{ kg}$$

$$H_A=qh-H_D=1484\times5.33-1430=6490$$

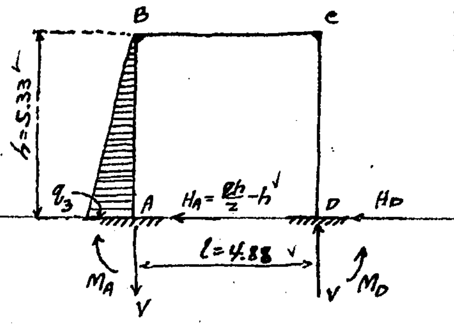
$$M_A=-\frac{qh^2}{24}\left(12-\frac{5k+9}{k+2}-\frac{12k}{6k+1}\right)=-\frac{1484\times5.33^2}{24}\left(12-\frac{9.625}{2.125}-\frac{1.50}{1.75}\right)=-11630\text{ kgm}$$

$$M_D=+\frac{qh^2}{24}\left(+\frac{5k+9}{k+2}-\frac{12k}{6k+1}\right)=+1760\,(4.53-.858)=\quad+6450\text{ kgm.}$$

$$M_B=M_A-H_Dh+\frac{qh^2}{2}=-11630-1515\times5.33+\frac{1484\times5.33^2}{2}=+1390\text{ kgm}$$

$$M_C=M_D-H_Dh=+6450-1515\times5.33\quad=-1640$$

Stresses due to q_3 = 5646 kg/m². (a part of earth pressure)



$$V=\frac{q_3kh^2}{4l(6k+1)}=\frac{5646\times.125\times5.33^2}{4\times4.88\times1.75}=587\text{ kg}$$

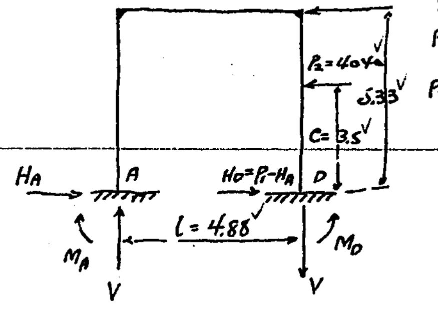
Propotion to the simillar case on page 13 $\frac{q_3}{p}=\frac{5646}{2842}=1.99$

H_D = 780 × 1.99 = 1,550 kg
H_A = 6,800 × 〃 = 13,520 〃
M_A = −8,660 × 〃 = −17,230 kgm
M_D = +3360 × 〃 = +6690 〃
M_B = +650 × 〃 = +1,290
M_C = −800 × 〃 = −1,590

Summary for moments + Reactions during Earthquake. (one meter strip).

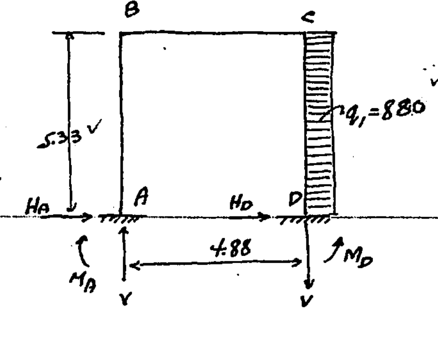
	M_A	M_D	M_B	M_C	H_A	H_D	V_A	V_D
					+→	←+	↓ −	↑ +
Dead Load. Summary	+1,610	+3,360	−3,220	−3,220	+907	+907	+12,210	+27,970
Seismic force, floor.	−5480	+5,480	+1,495	−1,495	−1,310	+1310	−613	+613
〃 〃 Girder span	−4480	+7660	+1,120	−890	−1050	+2990	−375	+375
Earth pressure + wt. of walls.	−11,630	+6450	+1,390	−1,640	−6490	+1,515	−618	+618
Earth pressure	−17,230	+6690	+1290	−1590	−13,520	+1,550	−587	+587
Total max. stress.	−37,210 kgm	+29,640 kgm	+2,076 kgm	−8835 kgm	−21,463 kg	+8272 kg	+10,017 kg	+30,163 kg

Stresses on frame under Electric Railway during Earthquake (Seismic force backward).
Stresses due to P1 + P2 Floor slab + track + Girder (from the last page.)



	M_A	M_D	M_B	M_C	H_A	H_D	V_A	V_D
					+→	←+	↓ −	↑ +
P_1	+5480	−5480	−1495	+1,495	+1,310	−1,310	+613	−613
P_2	+4480	−7660	−1,120	+890	+1,050	−2990	+375	−375

Stresses due to q_1, wall weight. Propotion 880/1484 = .593.



V = 618 × .593 = 367 kg
H_A = +1515 × 〃 = + 900 〃
H_D = −6490 × 〃 = − 3,840 〃
M_A = +6450 × 〃 = + 3820 kgm
M_D = −11630 × 〃 = − 6900 〃
M_B = −1640 × 〃 = − 972 〃
M_C = +1390 × 〃 = + 825 〃

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CHECKED BY M. Jinda DATE 4-6-23 PAGE NO A35

CALCULATIONS FOR

Design of Juso-ko-Bashi for Osaka prefecture.

Summary for stresses during Earthquake. (Seismic force backward.) →+ +← ⇇+ ↑+↓

		M_A	M_D	M_B	M_C	H_A	H_D	V_A	V_D
Dead Load Summary		+1,610	+3,360	-3220	-3220	+907	+907	+12210	+27970
Seismic force, floor	P_1	+5480	-5480	-1495	+1,495	+1,310	-1310	+ 613	- 613
" " girder	P_2	+4480	-7,660	-1120	+1890	+1,050	-2990	+ 375	- 375
" " walls	q_1	+3820	-6900	- 972	+ 825	+ 909	-3840	+ 367	- 367
Total stress.		+15390	-16680	-6807	-10	+4,167	-7,233	+13,565	+26615

Summary of Stresses for 3 cases calculated above.

		M_A	M_D	M_B	M_C	M_E	H_A	H_D	V_A	V_D
Normal state.	max.	-8970	+12650	-2060	-4,700	+4077	-6923	+3,500	+16,413	+43577
"	min.	-11,230	+9,040	-6580	-9220	+1,797	-8193	+2230	+11,693	+28,487
Earthquake.	forward.	-37,210	+29,640	+2,075	-8835		-2,463	+8272	+10,015	+30,163
"	backward.	+15390	-16680	-6,807	+ 10		+4,167	-7,233	13,565	+26,615

Check for stresses of slab. refere to page A24

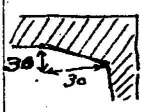
Effective depth of slab = 35 cm. Steel area = 25.4 cm^2 per meter strip.

~~moment at end near parapet = -9220 kg in normal state.~~

Steel ratio $p = \frac{25.4}{100 \cdot 35} = .00726$ $K = .370$, $j = .877$

$f_s = \frac{9220 \times 100}{25.4 \cdot .877 \cdot 35} = 1182\ kg/cm^2$

$f_c = \frac{1182 \cdot .370}{15(1 - .370)} = 46.2\ kg/cm^2$ use proper fillet to increase effective depth at end.



For rear end of slab, use same detail as for center of span. (main reinforcement on top.)

Parapet wall. moment -9220 kgm in normal state at top of wall.

Use 38 cm total depth ; 35 cm effective depth with 3 cm insulation.

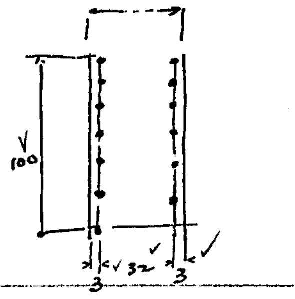
Reinforcements 22φ bars at 15 cm ctoc = 25.4 cm^2/meter strip on front side.
{ 16φ " " 15 " " = 13.4 " " on rear side
{ or 22φ " " 30 " " = 12.7 " " " "

Vert. load on wall.

D.L. abutment floor	4210
" parapet wall wt. .885×2400 =	2120
Earth pressure, reaction	222
" "	295
L.L. abutment	4720
	11,567 kg

call this 11570 kg/meter strip.

Eccentricity $\varepsilon = 9220 \div 11570 = .80\ m$



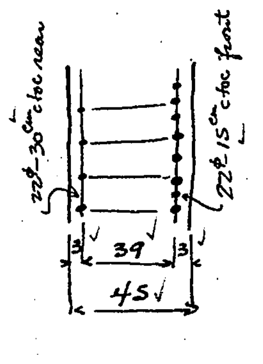
$p_0 = p + p' = \frac{25.4 + 12.7}{100 \times 38} = 0.010$, $\varepsilon/h = 80/38 = 2.1$, $d'/h = 3/38 = 0.079$

From the prepared diagrams of combined stresses. (direct + bending)

$K = .317$, $L = .147$

$f_c = \frac{M}{Lbh^2} = \frac{9220 \times 100}{.147 \times 100 \times 38^2} = 43.5\ kg/cm^2$ over stress.

Revise the thickness of parapet to 45 cm effective depth being 42 cm.



$p_0 = \frac{25.4 + 12.7}{100 \times 45} = .0085$, $\varepsilon/h = 80/45 = 1.78$, $d'/h = 3/45 = .067$

$K = .31$, $L = .14$

$f_c = \frac{9220 \times 100}{.14 \times 100 \times 45^2} = 32.5\ kg/cm^2$ ok

$f_s = nf_c\left(\frac{d}{Kh} - 1\right) = 15 \times 32.5\left(\frac{42}{.31 \times 45} - 1\right) = 981\ kg/cm^2$ ok.

unit shear $= \frac{3500}{100 \times \frac{7}{8} \times 42} = 1.0\ kg/cm^2$ ok unit bond $= \frac{3500}{6.91 \times 6.67 \times \frac{7}{8} \times 42} = 2.1\ kg/cm^2$ ok.

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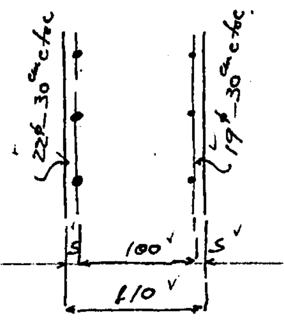
CHECKED BY M. Jinda DATE 4-6-23 **PAGE NO** A36

CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka Prefecture.

Design of Front wall.
Section at Bottom.



$M_D = +29640$ kgm., $V_D = 30163$ kg, $H_D = 8272$ kg during earthquake
-16680 $\quad 26615$ $\quad 7233$ "

Eccentricity $\varepsilon = \frac{29640}{30163} = 0.99^m$ $\quad \varepsilon/h = 99/110 = 0.90$ $\quad d'/h = 5/110 = .046$

Steel ratio $22^\phi - 30^{cm}$ ctoc $= 12.70^{cm^2}$ on rear side
$19^\phi - 30$ " " $= 9.45$ " - front side.

$p_0 = p+p' = \frac{22.15}{110 \times 100} = .002$

$K = .23$, $L = .076$

$f_c = \frac{29640 \times 100}{.076 \times 100 \times 110^2} = 32.3$ kg/cm² ok

$f_s = 15 \times 32.3 \left(\frac{105}{.23 \times 110} - 1\right) = 1525$ " $< 1200 \times 1.8 = 2160$ ok

unit shear $= \frac{8272}{100 \times \frac{7}{8} \times 105} = 0.9$ kg/cm² ok

Unit bond $= \frac{8272}{3.33 \times 6.91 \times \frac{7}{8} \times 105} = 3.9$ " ok.

Design of Rear wall.
Bottom section

$M_A = -3,727$ kgm, $V_A = 10,017$ kg, $H_D = 21,463$ kg during earthquake
$+15,390$ $\quad 13,565$ $\quad 7,233$ "

$\varepsilon = \frac{37210}{10017} = 3.6$ $\quad \varepsilon/h = 360/75 = 4.8$ $\quad d'/h = 5/75 = .067$

Steel $22^\phi - 15^{cm}$ ctoc $= 25.4^{cm^2}$ on rear.
$22^\phi - 30$ " $= \frac{12.7}{38.1}$ " on front.

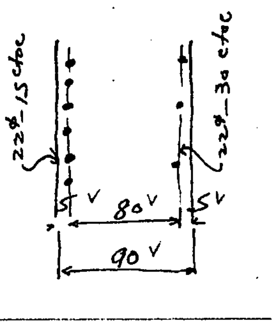
$p_0 = \frac{38.1}{75 \times 100} = .0051$

$K = .22$, $L = .113$

$f_c = \frac{37,210 \times 100}{.113 \times 100 \times 75^2} = 57$ kg/cm² $< 35 \times 1.8 = 63$ ok

$f_s = 15 \times 57 \left(\frac{70}{.22 \times 75} - 1\right) = 2770$ over stress.

Revice the depth of wall to 90cm.



$\varepsilon/h = 360/90 = 4.0$, $d'/h = 5/90 = ~~.555~~$

$p_0 = \frac{38.1}{90 \times 100} = .00423$

$k = .23$ $\quad L = .104$

$f_c = \frac{37,210 \times 100}{.104 \times 100 \times 90^2} = 44.2$ kg/cm² < 63 ok

$f_s = 15 \times 44.2 \left(\frac{85}{.23 \times 90} - 1\right) = 2060$ kg/cm² $< 1200 \times 1.8 = 2160$ ok

unit shear $= \frac{21463}{100 \times \frac{7}{8} \times 85} = 2.8$ " ok

unit bond $= \frac{21463}{6.91 \times 6.67 \times \frac{7}{8} \times 85} = 6.1$ " ok.

Top section

$M_B = -6580$ kgm, $V_B = 11,693$ kg $\quad$ Ecc. $\varepsilon = .56^m$ $\quad \varepsilon/h = 56/50 = 1.12$, $d'/d = 5/50 = .1$

Steel. $19^\phi - 15^{cm}$ ctoc ~~out side~~ $= 18.9$
$19^\phi - 30$ " " in side $= 9.45$

$p_0 = \frac{28.35}{100 \times 50} = .0057$

$K = .30$, $L = .106$

$f_c = \frac{6580 \times 100}{.106 \times 100 \times 50^2} = 24.8$ kg/cm² ok

$f_s = 15 \times 24.8 \left(\frac{45}{.3 \times 50} - 1\right) = 744$ kg/cm² ok

MADE BY K. Inaba DATE 4-5-31 FILE NO

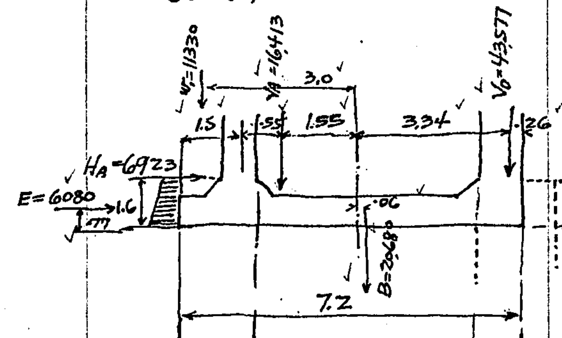
CHECKED BY M. Jinda DATE 4-6-24 PAGE NO 137

CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture.

Design of Base.

Case 1. Normal State.



$M_A = -8970$ kgm $V_A = 16413$ kg eccentricity = .546 m $H_A = -6923$

$M_D = +12650$ " $V_D = 43577$ " " = .29 m $H_D = +3500$

weight of Base. hor. lev.arm vert arm.

1.6 × .7 × 2400	= 2690	− 2.1	= 5650	1.2	=	3230
1.45 × .7 × "	= 2440	2.8	= 6830	1.32	=	2980
.9 × 7.2 × "	= 15550	0	= 0	.45	=	7000
	20680 kg	× .06	= 1180	.64	=	13210

Earth on Rear footing.

1.2 × 1.0 × 5.9 = 7.08 × 1600 = 11330 kg

Earth pres. $\frac{1600 \times 6.33}{3} = 3370$

$\frac{1600 \times 7.93}{3} = 4230$

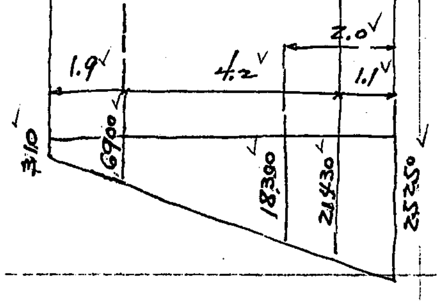
7600

7600 ÷ 2 = 3800 average.

3800 × 1.6 = 6080 kg

Taking Moment at center.

Loads.	Hor. forces	Vert. forces		lever arm		moment.
VA		16413	×	−1.55	=	− 25450
W₁		11330	×	−3.00	=	− 34000
VD		43577	×	+3.34	=	+ 145500
B		20680	×	+.06	=	+ 1240
E	6080		×	+1.77	=	+ 4680
HA	6923	℄	×	+1.60	=	+ 11080
HD.	3500		×	+1.60	=	+ 5600
	16503 kg	92000 kg	×	1.17	=	+ 108650 kgm



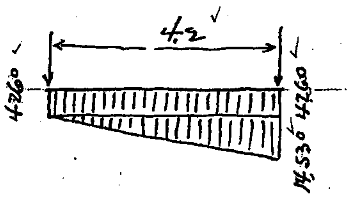
Resultant force within middle third.

max. toe pressure = $\frac{92000}{1.0 \times 7.2}\left(1 \pm \frac{6 \times 1.17}{7.2}\right) = 25250$ kg/m²

or 310

Load for 2 meters of toe be transmitted to lower base under toe assumed.

$\frac{25250 + 18300}{2} \times 2 = 43550$ kg



upward pressure		6900	21430
wt. of base	0.9 × 2400 =	− 2160	− 2160
" roadway	.3 × 1600 =	− 480	− 480
		4260 kg/m² of rear end	18790 kg/m² front end

moment

unif. load. $\frac{4260 \times 4.2^2}{8} = 9390$

triangular load. $\frac{14530 \times 4.2^2}{15.6}$ 16450

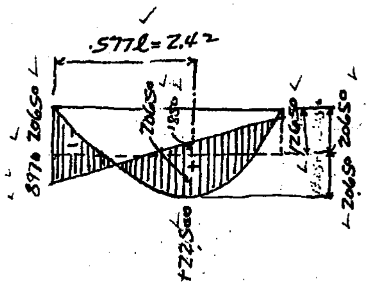
25840

Shear

$\frac{1}{2} \times 4260 \times 4.2 = 8950$

$\frac{2}{3} \times 14530 \times 4.2 = 40750$

49700 kg on front end.



for continuity of base moment = ~~16550~~ 25840 × .8 = 20650 kgm.

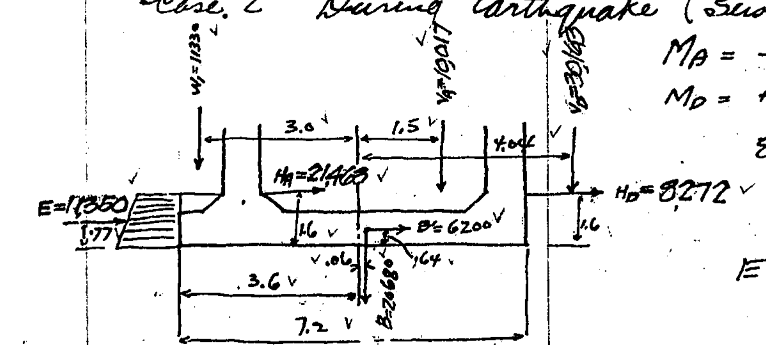
front end. +20650 rear end. −20650

M_D +12650 M_A − 8970

− 8000 kgm. − 29620 kgm.

max pos. moment 20650 + 1850 = +22500 kgm

Case 2 During Earthquake (Seismic force forward.)



$M_A = -37210$ kgm $V_A = +10017$ kg Ecc. = 3.6 $H_A = -21463$ kg

$M_D = +29640$ " $V_D = +30163$ " " = .99 $H_D = +8272$ "

Earth pressure. .662 × 1600 × 5.9 = 6250

" .662 × 1600 × 7.5 = 7950

14200 ÷ 2 = 7100 kg/m² average.

E = 7100 × 1.6 = 11350 kg

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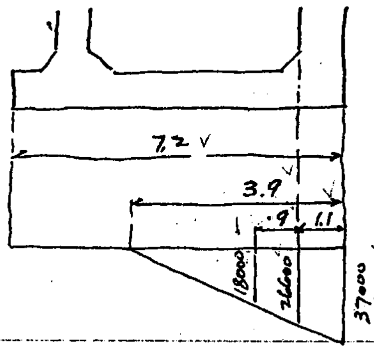
CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture.

Taking moment about center of bottom area.



Loads	Horizontal force	Vert. force	Lev. arm	moment.
W1.		11,330 ×	− 3.0 =	− 34,000
VA		10,017 ×	+ 1.5 =	+ 15,770
VD		30,163 ×	+ 4.04 =	+ 121,700
B		20680 ×	+ .06 =	+ 1,240
HA	21,463	21,433 ×	+ 1.6 =	+ 34,300
HD	8272	8,187 ×	+ 1.6 =	+ 13,100
B'	6,200	6,200 ×	+ .64 =	+ 3,960
E	11,350	13,350 ×	+ .77 =	+ ~~8,765~~ 8,750
	47,285 kg	72,190 kg ×	2.30 =	+ 164,820 kgm

Resultant force out of middle third.

Neglecting tension at heel. pressure area $3(3.6-2.3) = 3.9^m$

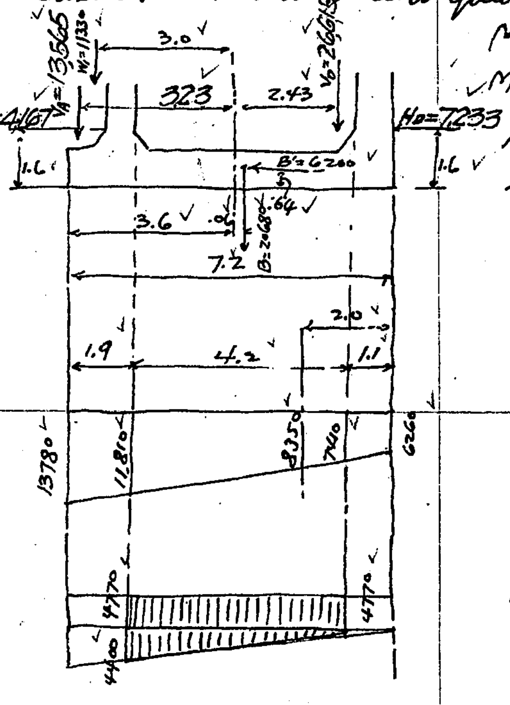
max. toe pressure $= \frac{72190 \times 2}{1.0 \times 3.9} = 37000$ kg/m²

Loads for 2 meters of toe be transmitted to lower base under toe assumed.

$\frac{37000+18000}{2} \times 2 = 55000$ kg.

Moment on base smaller than for case 1.

Case 3. During Earthquake (seismic force backward).



MA = + 15,390 kgm   VA = + 13,565   Eccentricity ε = 1.13 m   HA = + 4,167
MD = − 16,680   VD = + 26,615   " = .62   HD = − 7,233

Taking moment about center of base.

Loads	Hor. forces	Vert. forces.	Leverarm	moment.
W1		11,330 ×	+ 3.00 =	+ 34,000
VA		13,565 ×	+ 3.23 =	+ 43,800
HA	4,167	×	+ 1.60 =	+ 6,670
HD	7,233	×	+ 1.60 =	+ 11,550
VD.		26,615 ×	− 2.43 =	− 64,700
B		20,680 ×	− 0.06 =	− 1,240
B'	6,200	×	+ 0.64 =	+ 3,960
	17,600 kg	72,190 kg ×	+ .47 =	+ 34,040 kgm.

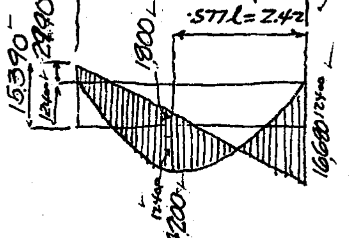
Resultant force within middle third.

max. toe pressure $= \frac{72190}{1.0 \times 7.2}\left(1 \pm \frac{6 \times .47}{7.2}\right) = 13,780$ kg/m²
6260

max. load on one pile $= 13780 \times .9 \times .9 = 11,500$ kg or 11.5 ~~kg~~ tons.

upward pressure.	11,810	7410
wt. of base.	− 2160	− 2160
" roadway fill	− 480	− 480
	+ 9170 rear end	+ 4,770 front end.
	4770	
	4,400	

Load on lower base
$\frac{8350+6260}{2} \times 2 = 14,610$ kg



Moment.

Unif load. $\frac{4770 \times 4.2^2}{8} = 10,500$ kgm

Trianglar load $\frac{4400 \times 4.2^2}{15.6} = 4980$
15,480 kgm

Shear
$\frac{1}{2} \times 4770 \times 4.2 = 10000$
$\frac{2}{3} \times 4400 \times 4.2 = 12,300$
22,300 kg
on rear end.

for continuity of base moment $= 15480 \times .8 = 12400$ kgm

rear end − 12,400
MA + 15390
+ 2,990 kgm

front end. − 12,400
MD − 16680
− 29,080 kgm.

Pos. moment = 12400 + 1410 = + 13,810 kgm.

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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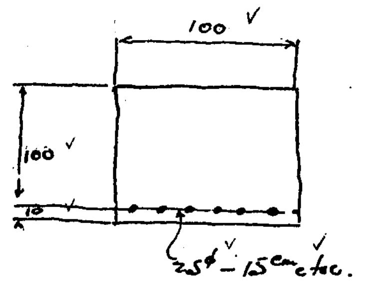
CALCULATIONS FOR

Design of Juso-ko-Bashi for Osaka Prefecture.

Section of Base.



	moment		Shear	
front end.	−29,080	seismic.	49,700	normal
rear end.	−29,620	normal.	22,300	seismic.
max. pos. m.	+22,500	"		

Rear End.

Effective depth required $= \sqrt{\frac{29620 \times 100}{100 \times 7.18}} = 64.2^{cm}$.

Use 100cm effective depth with 10cm insulation. or total depth of 110cm including fillet.

Steel area req'd $= \frac{29620 \times 100}{1200 \times \frac{7}{8} \times 100} = 28.2^{cm^2}$ per meter strip.

Use 25φ bars at 15cm ctoc = 32.7cm²

Steel ratio $p = \frac{32.7}{100 \times 100} = 0.0033$

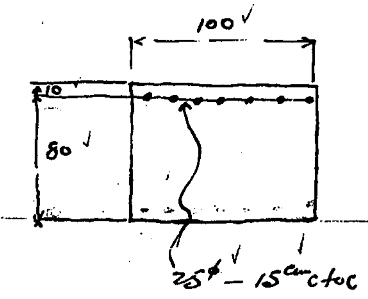
$k = .267$, $j = .911$

$f_s = \frac{29620 \times 100}{32.7 \times .911 \times 100} = 9950$ kg/cm² ok. $f_c = \frac{9950 \times .267}{15(1-.267)} = 24.2$ kg/cm² ok.

Unit shear $= \frac{22300}{100 \times .911 \times 100} = 2.45$ kg/cm² ok

Unit bond $= \frac{22300}{7.854 \times 6.67 \times .911 \times 100} = 4.67$ kg/cm² ok.

Section at Center.



total depth 90cm insulation 10 effective depth 80cm assumed.

Steel area req'd $= \frac{22500 \times 100}{1200 \times \frac{7}{8} \times 80} = 26.8^{cm^2}$

Use 25φ bars at 15cm ctoc = 32.7cm² on top.

Steel ratio $p = \frac{32.7}{100 \times 80} = .0041$

$k = .295$ $j = .902$

$f_s = \frac{22500 \times 100}{32.7 \times .902 \times 80} = 954$ kg/cm² ok.

$f_c = \frac{954 \times .295}{15(1-.295)} = 26.6$ kg/cm² ok.

Section at front end.

total depth 110, insulation 10, effective depth 100cm assumed.

Steel area req'd $= \frac{29080 \times 100}{1200 \times \frac{7}{8} \times 100 \times 1.8} = 15.3^{cm^2}$

Use 25φ bars at 15cm ctoc = 32.7cm²

Steel ratio $p = 0.0033$ $k = .267$, $j = .911$

$f_s = \frac{29080 \times 100}{32.7 \times .911 \times 100} = 971$ kg/cm² ok.

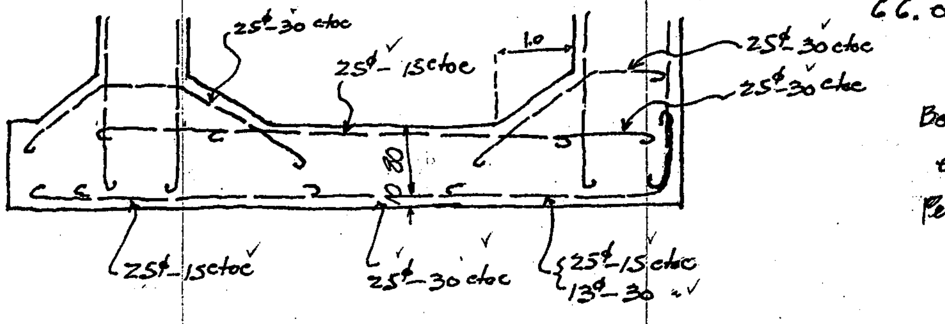
$f_c = \frac{971 \times .267}{15(1-.267)} = 23.6$ " ok.

Unit shear $= \frac{49700}{100 \times .911 \times 100} = 5.45$ kg/cm² use stirrups.

Perimeter of bars req'd for bond stress $= \frac{49700}{9.0 \times .911 \times 100} = 60.6^{cm}$

25φ bars 15cm ctoc = 7.854 × 6.67 = 52.4
13φ " 30 " = 4.08 × 3.33 = 13.6
66.0cm.

Deformed bars.



Bond Bars, reviced.
effective depth = 120cm
Perimeter req'd for bond $= \frac{49700}{9.0 \times .911 \times 120} = 50.5^{cm}$

use 25φ bars 15cm ctoc. Perimeter = 52.4cm ok.

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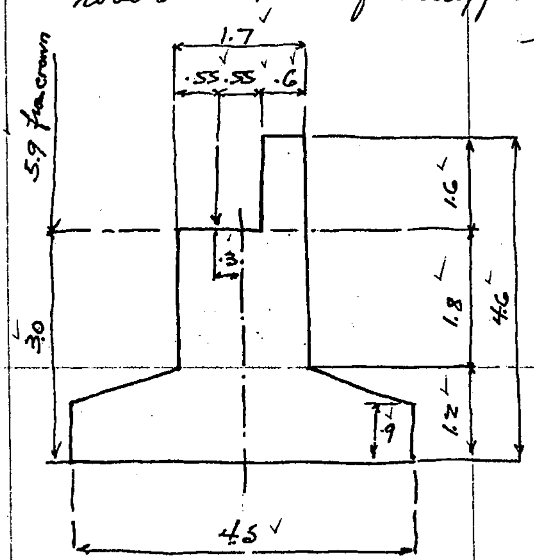
CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture

Design of Tower Base under toe. Electric Railway.

Loads on Base from upper frame.

		Vert. load.	hor. load. ½ΣH.
Case 1	normal	43550 kg	8250 kg →
Case 2	Seismic →	55,000	23,640 →
Case 3.	" ←	14,610	8,800 ←



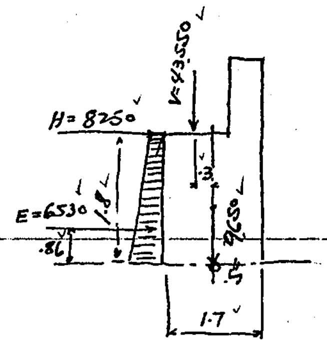
weight of base.

$.6 \times 1.6 = .96$ }
$1.7 \times 1.8 = 3.06$ } $4.02 @ 2400 = 9650$ kg
$3.1 \times .3 = .93$ }
$4.5 \times .9 = 4.05$ } ~~4.98 @ 2400 = 11,950~~
$9.00 @ 2400 = 21,600$ kg per meter strip.

Stresses on shaft.

Case 1. normal state.

Earth pressure. $5.9 \times 1600 \div 3 = 3150$
$7.7 \times$ " $\times$ " $= \underline{4110}$
$7260 \div 2 = 3630 \times 1.8 = 6530$ kg



Taking moment at center.

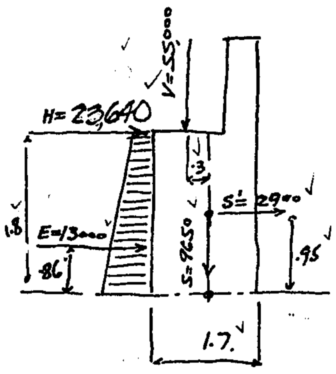
Load	Hor. forces	Vert. forces	Lev. arm	moments
V		43,550	× −.3 =	−13,070
S		9,650	× 0 =	0
H	8250		× 1.8 =	+ 14,840
E	6530		× .86 =	+ 5,610
	14,780 kg	53,200 kg	× .14 =	7,380 kgm

Try 19φ bars at 30 cm c to c = 9.45 cm² per meter strip · both sides.

$p_o = 2p = \frac{9.45 \times 2}{170 \times 100} = .0011$ $\quad e/h = .14/1.7 = .082$, $\quad d'/h = 5/170 = .03$

$k = 1.46 \quad \neq \quad$ whole section compression. .146

$f_c = \frac{Nk}{bh} = \frac{53,200 \times 1.46}{100 \times 170} = 4.6$ kg/cm² ok.



Case 2. During Earthquake.

Loads	Hor. forces	Vert. forces	Lever arm	moments.
V		55,000	× −.3	= − 16,500
S		9650	× 0	= 0
S'	2900		× .95	= + 2750
H	23,640		× 1.8	= + 42,300
E	13,000		× .86	= + 11,200
	39,540 kg	64,650 kg	× .62	= + 39,750 kgm

Earth pressure
$5.9 \times 1600 \times .662 = 6250$
$7.7 \times$ " $\times$ " $= \underline{8150}$
14400

$\frac{14400}{2} = 7200 \times 1.8 = 13,000$ kg

$e/h = 62/170 = .365$, $\quad d'/h = 5/170 = .03$

$p_o = .0011$

$k = .51 \qquad L = .091$

$f_c = \frac{39750 \times 100}{.091 \times 100 \times 170^2} = 15.1$ kg/cm² ok

$f_s = 15 \times 15.1 \left(\frac{165}{.51 \times 170} - 1 \right) = 204$ kg/cm² ok

unit shear $= \frac{39540}{100 \times \frac{7}{8} \times 165} = 2.7$ kg/cm² ok

unit bond $= \frac{39540}{5.97 \times 3.33 \times \frac{7}{8} \times 165} = 13.7$ kg/cm² $< 9 \times 1.8 = 16.2$

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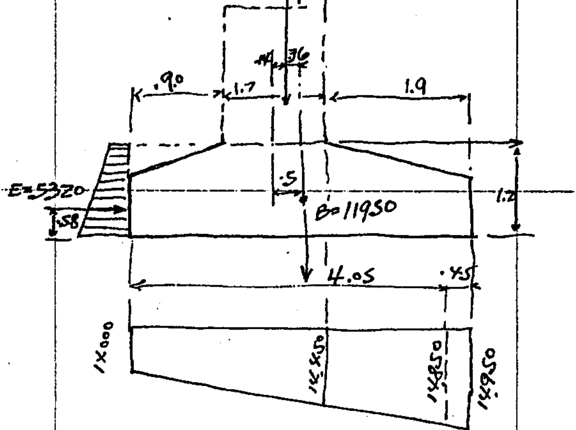
CALCULATIONS FOR

Design of Jinso-Ko-Bashi for Osaka Prefecture.

Stability of Base at Bottom Section

Case 1. Stability at normal state.



$V = 53,200^{kg}$ with 0.14^{m} eccentricity with respect to shaft.

$H = 14780^{kg}$ at top of Base.

Earth pressure $1600 \times 7.7 \div 3 = 4110$

$1600 \times 8.9 \div 3 = \underline{4750}$

$8860 \div 2 = 4430 \times 1.2 = 5320^{kg}$

Loads	Hor. forces	Vert. forces		Lev. arm		Moment
V		53,200	×	−.36	=	−19,150
H	14,780		×	1.20	=	17730
B		11,950	×	0	=	0
E	5320		×	0.58	=	3080
	20,100 kg	65,150 kg	×	.025	=	+1,660 kgm

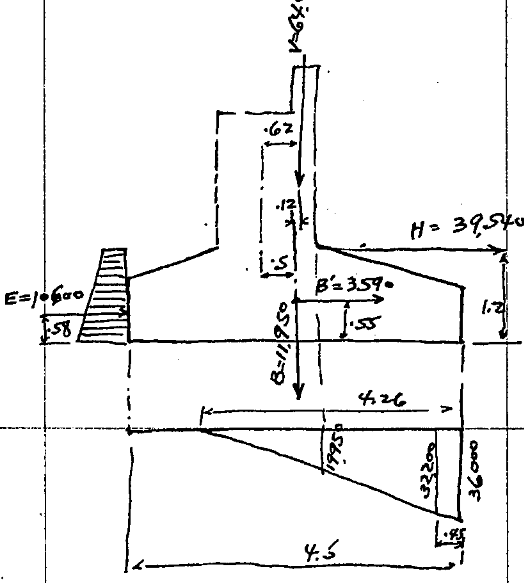
Resultant force within middle third.

max. toe pressure $= \frac{65150}{1.0 \times 4.5}\left(1 \pm \frac{6 \times .025}{4.5}\right) = 14,950^{kg/m^2}$ (or $1.37^{tons/□'}$) $^{.033}$

or $14000^{kg/m^2}$

max. load on one pile $= 14.85 \times .9 \times .9 = \underline{\underline{12.0}}^{kg\ tons}$.

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



$V = 64650^{kg}$ with .62 eccentricity with respect to shaft

$H = 39540^{kg}$ at top of base.

Earth pressure $1600 \times 7.7 \times .662 = 8,160$

$1600 \times 8.9 \times .662 = \underline{9440}$

$17,600 \div 2 = 8,800 \times 1.2 = 10,600^{kg}$

Loads	Hor forces	Vert. forces		Lev. arm		moment
V		64,650	×	.12	=	7,750
H	39540		×	1.20	=	47,450
B		11950	×	0	=	0
B'	3,590		×	.55	=	2000
E	10,600		×	.58	=	6150
	51,730 kg	76,600 kg	×	.83	=	63,350 kgm

Resultant force outside of middle third neglecting tension at heel,

Pressure area $= (2.25 - .83) \times 3 = 4.26$

max. toe pressure $= \frac{76600 \times 2}{1.0 \times 4.26} = 36,000^{kg/m^2}$ or $(3.3^{tons/□'})$

max. load on one pile $= 32.200 \times .9 \times .9 = \underline{\underline{26.1}}^{kg\ tons}$

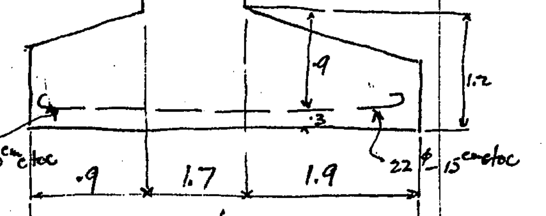
Case 3. Stability during Earthquake (Seismic force backward)

By a rough approximation

max. toe pressure $= 12,800^{kg/m^2}$ Ecc. $= .64^{m}$ alt.

max. load on one pile = 10.0 alt.

Design of Cantilever footing at toe.



Normal state. upward pressure $= \frac{14450 + 14950}{2} = 14700 \times 1.9 = 27900 \times .95 = 26,500^{kgm}$

wt. of footing $1.2 \times 2400 = 2880$ } 6080×1.9 $= \underline{11,550} \times .95 = \underline{-11,000}$

" " earth fill $2.0 \times 1600 = 3200$ }

$16,350$ $\quad 15,500^{kgm}$

During Earthquake upward pres. $= \frac{36000 + 19950}{2} = 27980 \times 1.9 = 53,200 \times 1.04 = 55,300^{kgm}$

wt. of footing & fill $\underline{-11550}$ $\quad \underline{-11,000}$

$41,650^{kg}$ $\quad 44,300^{kgm}$

Effective depth req'd $= \sqrt{\frac{44300 \times 100}{100 \times 7.18}} = 78.6^{cm}$ Use 90^{cm} effective depth.

~~Steel area req'd $= \frac{44300 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 90}$ 26.0^{cm^2} per meter strip of base.~~

Use $22^{\phi} - 15^{cm}$ c to c $= 25.4\ ^{cm^2/m}$ strip.

$p = \frac{25.4}{100 \times 90} = .0028$, $k = .248$, $j = .917$

$f_s = \frac{44300 \times 100}{25.4 \times .917 \times 90} = 2115^{kg/cm^2} < 1200 \times 1.8 = 2160$ ok

$f_c = \frac{2175 \times .248}{15(1 - .248)} = 46.5^{kg/cm^2} < 45 \times 1.8$ ok

unit shear $= \frac{41650}{100 \times .917 \times 90} = 5.05 < 4 \times 1.8$ ok

unit bond $= \frac{41650}{69 \times 6.67 \times .917 \times 90} = 10.96 < 9 \times 1.8$ ok

Deformed Bars. used.

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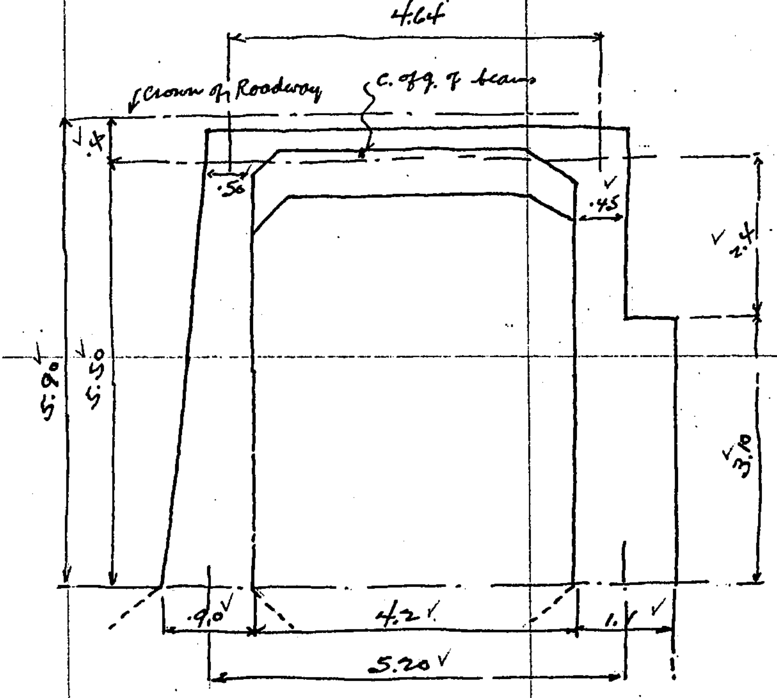
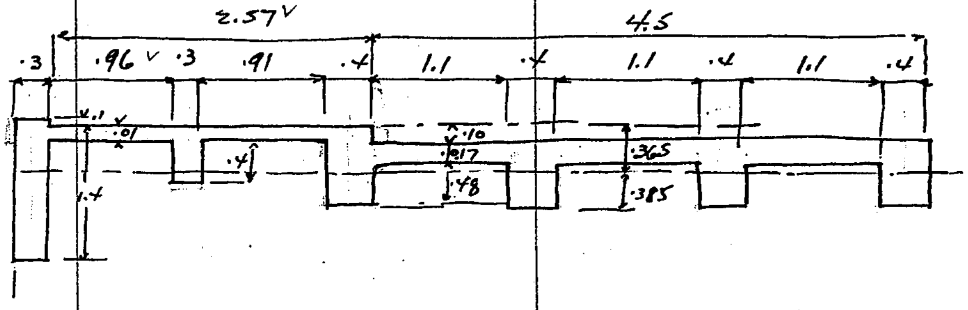
CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka Prefecture.
Stresses on Frame under Roadway and Sidewalk.





Center of gravity, taking moment about a hor. line through the crown of Roadway.

Roadway slab	$.17 \times 4.5 = .765 \times .185 = .1415$		
" stem	$.4 \times .48 \times 4 = .768 \times .510 = .3915$		
Sidewalk slab	$.1 \times 2.57 = .257 \times .050 = .0129$		
" stem	$.4 \times .17 = .068 \times .185 = .0126$		
" "	$.4 \times .3 = .120 \times .300 = .0360$		
" "	$.3 \times 1.5 = .450 \times .650 = .2925$		
	$2.428 \times .365 = .8870$		

Call this .40 m for calculating h.

Moment of inertia J_2. (Beams)

Slab of Roadway	$\frac{4.5 \times .17^3}{12} + .765 \times .18^2 =$	$.00184 + .0248 =$	$.02664$
Stem "	$4 \cdot \frac{.4 \times .48^3}{12} + .768 \times .145^2 =$	$.01475 + .01615 =$	$.03090$
" "			
" "	$\frac{.4 \times .17^3}{12} + .068 \times .18^2 =$	$.00016 + .0022 =$	$.00236$
Sidewalk slab	$\frac{2.57 \times .1^3}{12} + .257 \times .315^2 =$	$.00021 + .0255 =$	$.02571$
stem "	$\frac{.3 \times .4^3}{12} + .120 \times .065^2 =$	$.0016 + .00051 =$	$.00211$
" "	$\frac{.3 \times 1.5^3}{12} + .45 \times .285^2 =$	$.0844 + .03655 =$	$.12095$
		$J_2 =$	$0.20867 \ m^3$

Moment of inertia J_1. (walls.)

Rear wall.	Bottom.	$\frac{8.57 \times .9^3}{12} = .5200$	
	Top.	$\frac{8.57 \times .5^3}{12} = .0892$	$.6892 \div 2 = .3046$
Front wall.	Top.	$\frac{8.57 \times .45^3}{12} = .0651$	
	Bottom	$\frac{8.57 \times 1.1^3}{12} = .9510$	$1.0161 \div 2 = .5081$

$J_1 = .8127 \div 2 = .4064 \ m^3$ average.

$h = 5.9 - .4 = 5.50^m$, $l = \frac{5.2 + 4.64}{2} = 4.92^m$

$K = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{.20867}{.4064} \times \frac{5.50}{4.92} = 0.574$

Constants including K

$K = 0.574$
$K+2 = 2.574$ $K+1 = 1.574$
$2K+3 = 4.148$ $2K+1 = 2.148$
$3K+4 = 5.722$
$5K+9 = 11.870$ $5K-1 = 1.870$
$6K+1 = 4.444$
$7K+12 = 16.018$ $3+7K = 7.018$
$12K = 6.888$
$15K = 8.610$

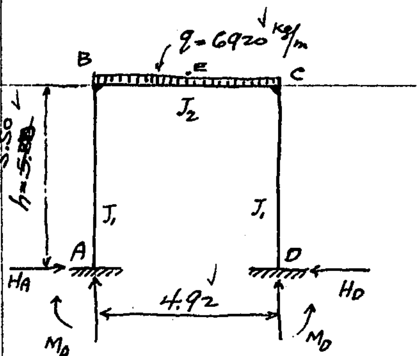
Design of Jusō-ko-Bashi for Osaka prefecture.

Dead Load Stresses on Frame.

Weight of floor.

Concrete	2.428	@ 2400	=	5820 Kg.
Roadway pavement	4.32	@ 149	=	644
Sidewalk	" 2.57	@ 53	=	136
Concrete fillets	$\frac{.1\times.15}{2}\times 10$	@ 2400	=	180
coping	.2×.1	@ 2400	=	48
curb stone	.18×.16	@ 2600	=	85
				6913 Kg per lin meter of span.

call this 6920



$$V = \frac{ql}{2} = \frac{6920\times4.92}{2} = 17{,}050\ \text{kg}$$

$$H = \frac{ql^2}{4h(K+2)} = \frac{6920\times4.92^2}{4\times5.5\times2.574} = 2{,}960\ \text{kg}$$

$$M_B = M_C = -\frac{ql^2}{6(K+2)} = -\frac{6920\times4.92^2}{6\times2.574} = -10{,}850\ \text{kgm}$$

$$M_A = M_D = +\frac{ql^2}{12(K+2)} = \qquad +5{,}420\ \text{kgm.}$$

$$M_E = +\frac{ql^2}{8} - \frac{ql^2}{6(K+2)} = \frac{6920\times4.92^2}{8} - 10{,}850 = +10{,}100\ \text{kgm.}$$

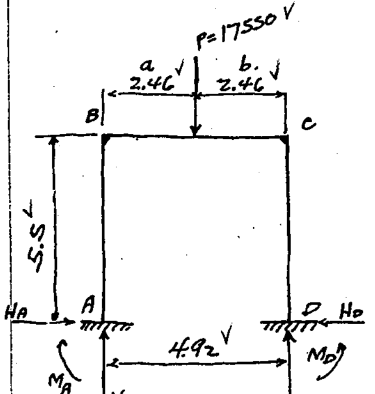
Live Load

Roadway Rear wheel concentration with 30% imp. = 5850 kg × 3 = 17,550 kg

front " " " " = 2730 × 3 = 8,190 "

Uniform load on front & rear of trucks = say 4.5 @ 600 = 2,700 kg/meter

Sidewalk uniform load. = 2.75 @ 500 = 1,380 "



Stress due to motor truck rear wheels.

$a = b = \frac{l}{2}$, $\quad \delta = a/l = \frac{1}{2}/l = \frac{1}{2}$

$$V = \frac{P}{2} = \frac{17550}{2} = +8{,}780\ \text{kg}$$

$$H = \frac{3Pl}{8h(K+2)} = \frac{3\times17550\times4.92}{8\times5.5\times2.574} = +2{,}290\ \text{kg}$$

$$M_A = +\frac{Pl}{8}\cdot\frac{(5k-1)+(K+2)}{(K+2)(6k+1)} = +\frac{17550\times4.92}{8}\cdot\frac{1.87+2.574}{2.574\times4.444} = +4{,}200\ \text{kgm}$$

$$M_D = +\frac{Pl}{8}\cdot\frac{(3+7k)-(K+2)}{(K+2)(6K+1)} = +\frac{17550\times4.92}{8}\cdot\frac{7.018-2.574}{2.574\times4.444} = +4{,}200\ \text{kgm.}$$

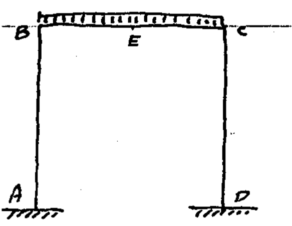
$$M_B = M_A - Hh = 4200 - 2290\times5.5 = -8400\ \text{Kgm}$$

$$M_C = M_D - Hh = \quad " \quad - \quad " \quad = -8400 \ "$$

$$M_E = M_A - Hh + V_A\frac{l}{2} = -8400 + 8780\times\frac{4.92}{2} = +13{,}200\ \text{kgm}$$

Uniform load on rear of motor truck is loaded only a length of 0.46 and neglected.

Stresses due to uniform load on sidewalk. q = 1380 kg/m.



$$V = \frac{ql}{2} = \frac{1380\times4.92}{2} = +3{,}400\ \text{kg.}$$

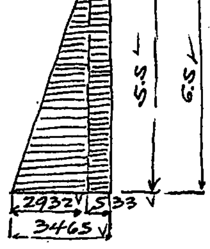
$$H = \frac{ql^2}{4h(k+2)} = \frac{1380\times4.92^2}{4\times5.5\times2.574} = +590\ \text{kg}$$

$$M_B = M_C = -\frac{ql^2}{6(k+2)} = -\frac{1380\times4.92^2}{6\times2.574} = -2{,}160\ \text{Kgm}$$

$$M_A = M_D = +\frac{ql^2}{12(K+2)} = \qquad +1{,}080\ \text{kgm}$$

$$M_E = +\frac{ql^2}{8} - \frac{ql^2}{6(k+2)} = +\frac{1380\times4.92^2}{8} - \frac{1380\times4.92^2}{6\times2.574} = +2{,}010\ \text{kgm}$$

Earth pressure at normal state.



Earthpressure at top. $\frac{1600\times1.0}{3} = 533\ \text{kg/m}^2$

at bottom $\frac{1600\times6.5}{3} = 3465$

$3465 - 533 = 2932\ \text{kg/m}^2$

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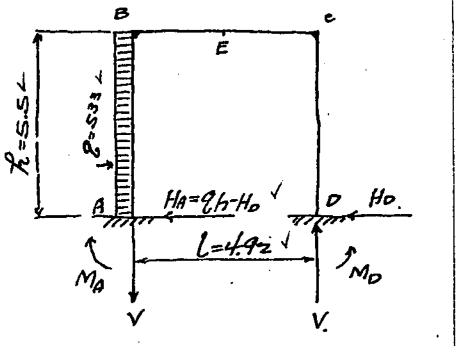
CALCULATIONS FOR

Design of Juso-ko-Bashi for Osaka prefecture.



$V = \frac{qh^2k}{l(6k+1)} = \frac{533 \times 5.5^2 \times .574}{4.92 \times 4.444} = \pm 420$ kg/meter strip for 8.57m ---- 3,600 kg

$H_D = \frac{qh}{8} \cdot \frac{2k+3}{k+2} = \frac{533 \times 5.5}{8} \cdot \frac{4.148}{2.574} = 590$ kg/meter strip 5,060 kg

$H_A = qh - H_D = 533 \times 5.5 - 590 = 2340$ kg/m strip 20,050 kgm

$M_A = -\frac{qh^2}{24}\left(12 - \frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = -\frac{533 \times 5.5^2}{24}\left(12 - \frac{11.87}{2.574} - \frac{6.888}{4.444}\right) = -3920$ kgm/m −33,600 kgm

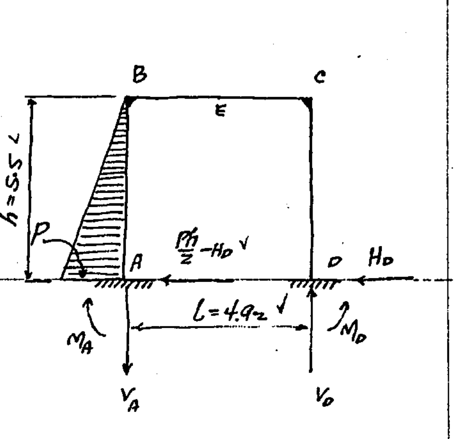
$M_D = +\frac{qh^2}{24}\left(+\frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = +672(+4.61 - 1.55) = +2,060$ "/m +17,650 "

$M_B = M_A - H_D h + \frac{qh^2}{2} = -3920 - 590 \times 5.5 + \frac{533 \times 5.5^2}{2} = +910$ kgm/m +7,800 "

$M_C = M_D - H_D h = +2060 - 3250 = -1190$ kgm/m −10,200 "

M_E by propotion $= -140$ kgm/m −1,200 "

— o —



$V = \frac{Pkh^2}{4l(6k+1)} = \frac{2932 \times .574 \times 5.5^2}{4 \times 4.92 \times 4.444} = 580$ kg/m strip of wall for 8.57m ------ 4,970 kg

$H_D = \frac{Ph}{40} \cdot \frac{3k+4}{k+2} = \frac{2932 \times 5.5}{40} \cdot \frac{5.722}{2.574} = 900$ kg/m 7710 "

$H_A = \frac{Ph}{2} - H_D = \frac{2932 \times 5.5}{2} - 900 = 7170$ kg/m 61,500 "

$M_A = -\frac{Ph^2}{120}\left(20 - \frac{7k+12}{k+2} - \frac{15k}{6k+1}\right) = -\frac{2938 \times 5.5^2}{120}\left(20 - \frac{16.018}{2.574} - \frac{8.61}{4.444}\right) = -8770$ kgm/m −75100 kgm

$M_D = +\frac{Ph^2}{120}\left(\frac{7k+12}{k+2} - \frac{15k}{6k+1}\right) = +740(6.22 - 1.94) = +3170$ kgm/m +27150 "

$M_B = M_A - H_D h + \frac{Ph^2}{6} = -8770 - 900 \times 5.5 + \frac{2932 \times 5.5^2}{6} = +1080$ "/m +9250 "

$M_C = M_D - H_D h = +3170 - 4950 = -1780$ " −15250 "

M_E by propotion $= -350$ kgm/m −3000 "

Dead Load due to weight of walls on A & D.

Load on A $\frac{.5+.9}{2} = .7 \times 5.5 = 3.85 @ 2400 = 9240$ kg $\times 8.57^m = 79,100$ kg

" " D. $.45 \times 2.4 = 1.08$
$1.1 \times 3.1 = 3.41$
$4.49 @ 2400 = 10780 \times 8.57^m = 92,400$ kg

Load from main girder spans. D.L. $13460 \times 8.57 = 115,500$ kg See page A32
L.L. $10370 \times 8.57 = 88800$ kg

Eccentricity = .13m

moment on D. D.L. $M_D = +115,500 \times .13 = +15000$ kgm (M_A, M_B, M_B &c neglected)
L.L. $M_D = +88800 \times .13 = +11,550$ "

Summary for moments and reactions.

	M_A	M_D	M_B	M_C	M_E	$\overset{\rightarrow}{+}$ H_A	$\overset{\leftarrow}{+}$ H_D	↓↑ V_A	V_D
Dead Load, abutment floor	+5420	+5420	−10850	−10850	+10100	+2960	+2960	+17050	+17050
" " wt. of walls								+79,100	+92,400
" " Girder span		+15000							+115500
Total	+5420	+20420	−10850	−10850	+10100	+2960	+2960	+96150	+224950
Earth pressure, uniform.	−33600	+17650	+7800	−10200	−1200	−20050	+5060	−3600	+3600
" , triangular	−75,100	+27,150	+9250	−15250	−3000	−61,300	+7710	−4970	+4970
Live Load on abutment Roadway	+4200	+4,200	−8400	−8400	+13200	+2290	+2290	+8780	+8780
" " , Sidewalk	+1080	+1,080	−2160	−2160	+2010	+590	+590	+3400	+3,400
" " " Girder span		+11,550							+88,800
Total DL + Earth pressure + L.L	−98000 kgm	+82,050 kgm	−4,360 kgm	−46860 kgm	+21,210 kgm	−75,710 kg	+18,610 kg	+99760 kg	+334,500 kg
" DL + Earth pressure	−103,280	+65,220	~~+6020~~ 6,200	−36,300	+5,900	−78,590	+15,730	+87,580	+233,520

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CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.

Stresses on Frame under Roadway and Sidewalk during Earthquake. (Seismic forces forward).

Seismic forces on frame.

Dead Load:— Floor $6920 \times 4.92 = 34000$ kg

seismic force $= 34000 \times 0.3 = 10,200$ kg on top. ———— $= P_1$.

Weight of Rearwall. $79,100 \times .3 = 23,740 \times 2.49 = 59,100$

" " front " $92,400 \times .3 = 27,700 \times 2.20 = 60,900$

$171,500$ kg $51,440$ kg 2.33 $120,000$

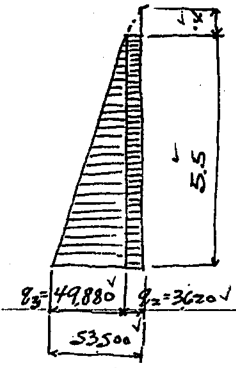
Equivalent uniform horizontal load $= \frac{51,440}{5.5} \times \frac{2.33}{2.75} = 7900$ kg per lin m. of height $= q_1$.

Earth pressure for 3000 mm earthquake. $= 0.662\, wh$ kg/m²

top. $0.662 \times 1600 \times .4 = 423$ kg/m² $\times 8.57$ m $= 3620$ kg $= q_2$

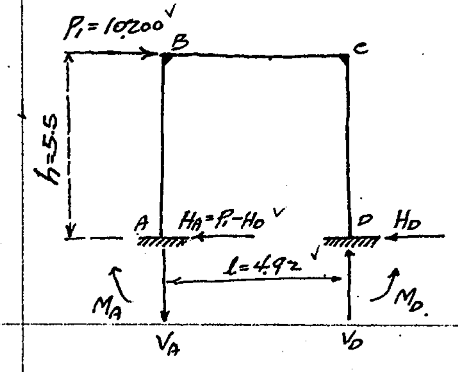
bottom $0.662 \times 1600 \times 5.9 = 6250$ " $\times 8.57 = 53,500$ "

$49,880$ kg $= q_3$.



Dead Load due to girder span, average $1346 \times .3 = 4040 \times 8.57 = 34,600$ kg -- P_2

Stresses due to P_1. (Floor, seismic force).



$V = \frac{3Phk}{l(6k+1)} = \frac{3 \times 10,200 \times 5.5 \times .574}{4.92 \times 4.444} = 4430$ kg

$H_A = H_D = \frac{1}{2} P_1 = \frac{10,200}{2} = 5,100$ kg

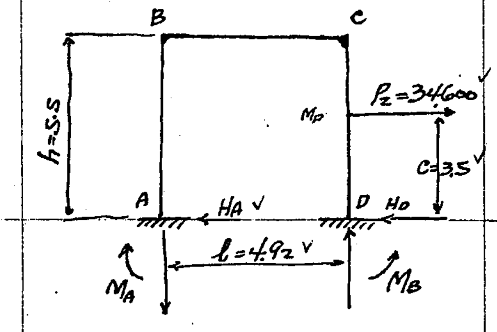
$M_A = -\frac{P_1 h}{2} \cdot \frac{3k+1}{6k+1} = -\frac{10200 \times 5.5}{2} \times \frac{2.722}{4.444} = -17,200$ kgm

$M_D = +$ " " " $= +$ " " " $= +17,200$ "

$M_B = +\frac{P_1 h}{2} \frac{3k}{6k+1} = +\frac{10200 \times 5.5}{2} \times \frac{1.722}{4.444} = +10,900$ "

$M_C = -$ " " " $=$ $= -10,900$ "

Stresses due to D.L. of main girder span. $\delta = c/h = 3.5/5.5 = .637$



$V = \frac{3P_2 c\delta k}{l(6k+1)} = \frac{3 \times 34,600 \times .637 \times 3.5 \times .574}{4.92 \times 4.444} = 6080$ kg

$H_A = \frac{P_2 \delta^2}{2(k+2)}\left[3(k+1) - \delta(2k+1)\right] = \frac{34600 \times .637^2}{2 \times 2.574}(3 \times 1.574 - .637 \times 2.148) = 9150$ kg

$H_D = P_2 - H_A = 34600 - 9150 = 25450$ kg

$M_A = -\frac{P_2 \delta c}{2}\left[\frac{3+2k-\delta(k+1)}{k+2} \frac{3k}{6k+1}\right] = -\frac{34600 \times .637 \times 3.5}{2}\left(\frac{4.148 - .637 \times 1.574}{2.574} \frac{1.722}{4.444}\right) = -32,200$ kgm

$M_D = +\frac{P_2 \delta c}{2}\left[\frac{2}{\delta} - \frac{3+2k-\delta(k+1)}{k+2} - \frac{3k}{6k+1}\right] = +38,560(3.14 - 1.223 - .3875) = +59,000$ kgm.

$M_B = M_A + H_A h = -32,200 + 9150 \times 5.5 = +18,100$ kgm.

$M_C = M_D + H_A h - P_2 c = +59000 + 9150 \times 5.5 - 34600 \times 3.5 = -11,806$ kgm

$M_P = M_D - H_D c = +59000 - 25450 \times 3.5 = -30,100$ "

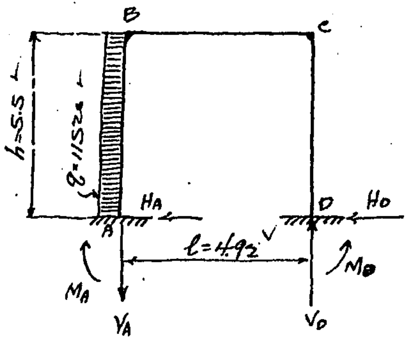
Stresses due to $q_1 + q_2$, weight of walls and Earth pressure.

$q_1 = 7900$

$q_2 = 3620$

$11,520$ kg per meter of height.

Design of Juso-Ko-Bashi for Osaka Prefecture



$V = \frac{qh^2k}{l(6k+1)} = \frac{11520 \times 5.5^2 \times .574}{4.92 \times 4.444} = 9150$ kg.

$H_D = \frac{qh}{8} \cdot \frac{2k+3}{k+2} = \frac{11520 \times 5.5}{8} \times \frac{4.148}{2.574} = 12750$ kg

$H_A = qh - H_D = 11520 \times 5.5 - 12750 = 50650$ kg

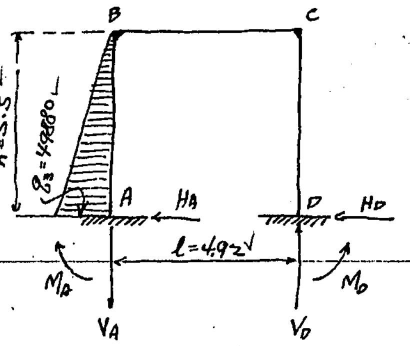
$M_A = -\frac{qh^2}{24}\left(12 - \frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = -\frac{11520 \times 5.5^2}{24}\left(12 - \frac{11.870}{2.574} - \frac{6.888}{4.444}\right) = -84900$ kgm.

$M_D = +\frac{qh^2}{24}\left(+\frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = +14520(4.61 - 1.55) = +44450$ "

$M_B = M_A - H_D h + \frac{qh^2}{2} = -84900 - 12750 \times 5.5 + \frac{11520 \times 5.5^2}{2} = +19300$ "

$M_C = M_D - H_D h = +44450 - 12750 \times 5.5 = -25650$ "

Stresses due to q_3 = 49880 kg/m² at bottom. (triangular part of earth pressure)



$V = \frac{q_3 K h^2}{4l(6k+1)} = \frac{49880 \times .574 \times 5.5^2}{4 \times 4.92 \times 4.444} = 9900$ kg

$H_D = \frac{q_3 h}{40} \cdot \frac{3K+4}{K+2} = \frac{49880 \times 5.5}{40} \cdot \frac{5.722}{2.574} = 15280$ "

$H_A = \frac{q_3 h}{2} - H_D = \frac{49880 \times 5.5}{2} - 15280 = 121920$ "

$M_A = -\frac{q_3 h^2}{120}\left(20 - \frac{7k+12}{k+2} - \frac{15k}{6k+1}\right) = -149000$ kgm

$M_D = +\frac{q_3 h^2}{120}\left(\frac{7k+12}{k+2} - \frac{15k}{6k+1}\right) = +53900$ "

$M_B = M_A - H_D h + \frac{q_3 h^2}{6} = +18,350$ "

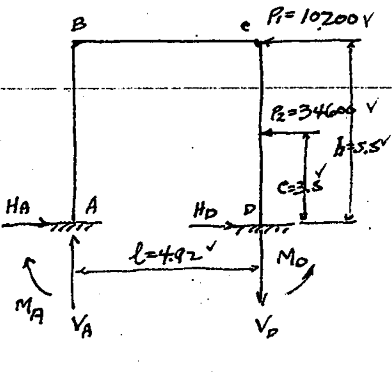
$M_C = M_D - H_D h = -30,250$ "

$M_E = -5,950$ "

Summary for moments and reactions during earthquake (forward).

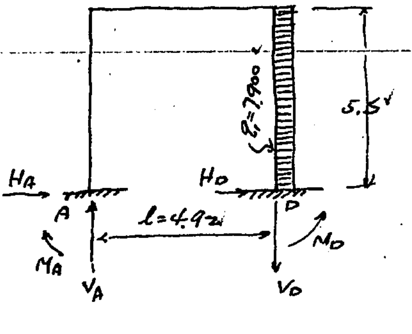
	M_A	M_D	M_B	M_C	H_A $+\rightarrow$	H_D $\leftarrow+$	V_A $\downarrow\uparrow$	V_D
Dead Load summary	+9420	+20420	−10850	−10850	+2960	+2960	+96150	+224950
Seismic force, floor	−17200	+17200	+10900	−10900	−5100	+5100	−4430	+4430
" girder span	−32200	+59000	+18100	−11800	~~−25450~~ +9150	+25450	−6080	+6080
Earth pressure + wt. of wall	−84900	+44450	+19300	−25650	−50650	+12750	−9150	+9150
Earth pressure (triangular part)	−149000	+53900	+18350	−30250	−121920	+15280	−9900	+9900
	−277880 kgm	+194970 kgm	+55800 kgm	−89450 kgm	−183860	+61540 kg	+66590 kg	+254510 kg

Stresses on frame during earthquake (backward)



	M_A	M_D	M_B	M_C	H_A	H_D	V_A	V_D
P1	+17200	−17200	−10900	+10900	+5100	−5100	+4430	−4430
P2	+32200	−59000	−18100	+11800	+9150	−25450	+6080	−6080

Stresses due to q_1 = 7900 or wt. of walls. proportion $\frac{q_2+q_1}{q_1} = \frac{11520}{7900} = 1.46$



$V = 9150 \div 1.46 = 6,260$ kg

$H_A = +12750 \div$ " $= +8,740$ "

$H_D = -50650 \div$ " $= -34700$ "

$M_A = +44450 \div$ " $= +30450$ kgm

$M_D = -84900 \div$ " $= -58,200$ "

$M_B = -25650 \div$ " $= -17,580$ "

$M_C = +19300 \div$ " $= +13220$

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CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka prefecture

Summary for Stresses during earthquake (backward).

	M_A	M_D	M_B	M_C	H_A	H_D	V_A	V_D
Dead Load. Summary	+5420 ✓	+20420 ✓	−10850 ✓	−10850 ✓	+2960 ✓	+2960 ✓	+96150 ✓	+224950
Seismic force, floor P_1	+17200 ✓	−17200 ✓	−10900 ✓	+10900 ✓	+5100 ✓	−5100 ✓	+4430	−4430
〃 〃 Girder P_2	+32200	−59000 ✓	−18100 ✓	+11800 ✓	+9150 ✓	−25450 ✓	+6080 ✓	−6080 ✓
〃 〃 walls Q_1	+30450 ✓	−58200 ✓	−17580 ✓	+13220 ✓	+8740 ✓	−34700 ✓	+6260 ✓	−6260 ✓
Total stresses.	+85270 ✓	−113980 ✓	−57430 ✓	+25070 ✓	+25950 ✓	−62290 ✓	+112920	+208180

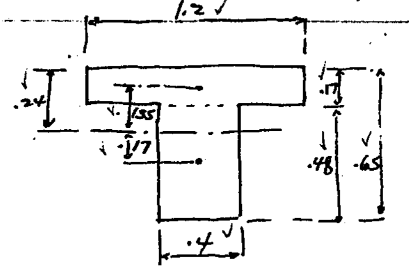
Summary for Stresses in 3 cases calculated above.

		M_A	M_D	M_B	M_C	H_A	H_D	V_A	V_D
Normal state.	D.L.+Earth pres.+L.L.	−98000 ✓	+82050 ✓	−4360 ✓	−46860 ✓	−75,710 ✓	+18,610 ✓	+99760 ✓	+334500 ✓
〃	D.L.+Earth pres.	−103280 ✓	+65220 ✓	+6200 ✓	−36300 ✓	−78590 ✓	+15730 ✓	+87580 ✓	+233520 ✓
Earthquake.	forward.	−277880 ✓	+194970 ✓	+55800 ✓	−89450 ✓	−183860 ✓	+01540	+66590 ✓	+254510 ✓
〃	backward.	+85270 ✓	−113980 ✓	−57430 ✓	+25070 ✓	+25950 ✓	−62290 ✓	+112920 ✓	+208180 ✓

Check for stresses on Beam end.

Moments $M_B + M_C$ are assumed to be distributed on several beams propotionally to their moment of inertias respectively.

moment of inertia of Beam A.



Center of gravity.

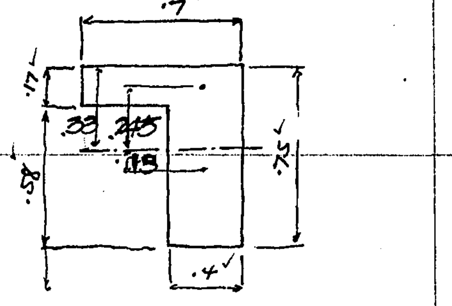
$.17 \times 1.2 = .204 \times .085 = .0171 =$
$.48 \times .4 = \underline{.192} \times .410 = \underline{.0787} =$
$.396 \times .24^m = .0958$

moment of inertia

$\frac{1.2 \times .17^3}{12} + .204 \times .155^2 = .00049 + .0049 = .00539$
$\frac{.4 \times .48^3}{12} + .192 \times .17^2 = .003685 + .00554 = \overset{.00923}{\underline{\cancel{.04239}}}$
$J_A = \overset{}{\cancel{.04778}}\ m^4$ .01462

Beam C same moment of inertia as for A assumed.

moment of inertia of Beam B.

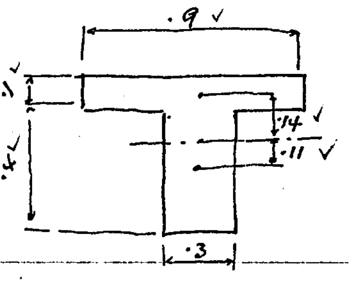


Center of gravity

$.17 \times .7 = .119 \times .085 = .0101$
$.40 \times .58 = \underline{.232} \times .46 = \underline{.1067}$
$.351 \times .33^m = .1168$

$\frac{.7 \times .17^3}{12} + .119 \times .245^2 = .00029 + .0071 = .00939$
$\frac{.4 \times .58^3}{12} + .232 \times .13^2 = .0065 + .0049 = \underline{.0114}$
$J_B = .01879$

moment of inertia of Beam D.

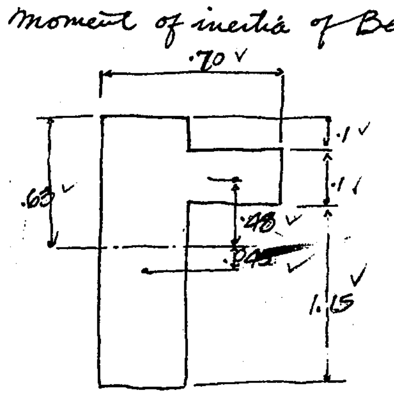


Center of gravity

$.1 \times .9 = .09 \times .05 = .0045$
$.3 \times .4 = \underline{.12} \times .30 = \underline{.0360}$
$.21 \times .19^m = .0405$

$\frac{.9 \times .1^3}{12} + .09 \times .14^2 = .00008 + .00177 = .00185$
$\frac{.3 \times .4^3}{12} + .12 \times .11^2 = .0016 + .00145 = \underline{.00305}$
$J_D = .00490\ m^4$

moment of inertia of Beam E center of gravity



$.3 \times 1.35 = .405 \times .675 = .273$
$.1 \times .4 = \underline{.04} \times .15 = \underline{.006}$
$.445 \times .63^m = .279$

$\frac{.3 \times 1.35^3}{12} + .405 \times .043 = .0614 + .00082 = .06222$
$\frac{.4 \times .1^3}{12} + .04 \times .48^2 = .00003 + .0092 = \underline{.00923}$
$J_E = .07145\ m^4$

Summary for moment of inertias of beams.

Beam	A	$2 \times .01462 =$	.02924 ✓	$\frac{.01462}{.20867} = .07$ ✓
〃	C	assumed	= .01462 ✓	〃 = .07 ✓
〃	B		= .01879 ✓	$\frac{.01879}{.20687} = .091$ ✓
〃	D		= .00490 ✓	$\frac{.0049}{.20687} = .024$ ✓
〃	E		= .07145 ✓	$\frac{.07144}{.20867} = .0338$ ✓
			.13900 m^4	

Moment of inertia of floor = .20867 m^4

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CALCULATIONS FOR

Design of Jusō-ko-Bashi for Osaka prefecture.

Max. moment	M_B	M_C.
Earthquake	+ 55800	− 89450
〃	− 57430	+ 25070

Moment on beam end.

	Beam A + C	Beam B	Beam D	Beam E
M_B	+ 3900	+5070	+1340	+18800
〃	− 4020	−5230	−1380	−19400
M_C	− 6260	−8130	−2150	−30200
〃	+ 1750	+2280	+ 600	+8470

Beams A + C. Moment at end of beams. + 3900 kgm, − 6260 kgm in earthquake.
Moment for these beams design = 10,540 kgm at normal state.
Assumed section is ample.

Beam B. moment = +5070, or − 8,130 kgm during earthquake.
moment for this beam design was. 7700 kgm at normal state 7700×1.8 = 13900 kgm ok.
Assumed section is ample.

Beam C. Same as for Beam A.

Beam D. Assumed section is ample.

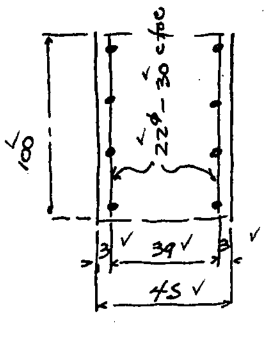
Beam E. moment = +18,800 kgm or −30,200 kgm during earthquake. (at ends of beam)
depth of beam at end = 2.0 m effective depth = 195 cm
Steel area req'd. at end = $\frac{30200\times100}{1200\times1.8\times\frac{7}{8}\times195} = 8.24\ \text{cm}^2$

use 3 − 19φ = 8.50 cm²
Steel ratio = $\frac{8.5}{30\times195} = .00145$ $k = .183$, $j = .939$

$f_s = \frac{30200\times100}{8.5\times.939\times195} = 1940\ \text{kg/cm}^2 < 1200\times1.8 = 2160$ ok

$f_c = \frac{1940\times.183}{15(1-.183)} = 29.0\ \text{kg/cm}^2$ ok.

Parapet wall. moment + 25070 kgm or − 89450 kgm total width of wall = 7.87 m for Roadway + Sidewalk
moment for one meter strip = + 3,190 kgm or − 11,370 kgm
vert. load on wall for these moments.
D.L. abutment floor = + 17050 ~~+ 4250~~
Seismic force floor. = + 4250
〃 〃 girder = + 6080
Earth pres. + wall. = + 9150
〃 = + 9900
+ 46430 kg ÷ 7.87 = 5900 kg / meter strip of wall.



Eccentricity $\varepsilon = \frac{-11,370}{5900} = 1.93\ \text{m}$, $\varepsilon/h = \frac{1.93}{.45} = 4.3$. $d''/h = 3/45 = .067$

Reinforcement 22φ − 30 cm ctoc = 12.7 cm² on both side
Steel ratio $p_0 = 2p = \frac{12.7\times2}{100\times45} = .00564$

$k = .24$, $L = .116$

$f_c = \frac{11370\times100}{.116\times100\times45^2} = 48.4\ \text{kg/cm}^2 < 35\times1.8 = 63$ ok

$f_s = 15\times48.4\left(\frac{42}{.24\times45}-1\right) = 2105\ \text{kg/cm}^2 < 1200\times1.8 = 2160$ ok.

Top of rear wall, use same reinforcements as parapet wall above calculated.

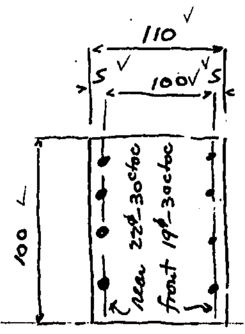
MADE BY K. Inaba DATE 4-6-4 FILE NO

CHECKED BY M. Jinda DATE 4-6-28 PAGE NO A49

CALCULATIONS FOR

Design of Jusō-Ko-Bashi for Osaka Prefecture.

Design of Front wall.
Section at Bottom



moment $M_D = +82050^{kgm}$, $V_D = +334500^{kg}$, $H_D = +18610^{kg}$ at normal state.
$+194970$, $+254510$, $+61540$ during earthquake.

Length of front wall under Roadway + Sidewalk = 7.87^m

Stresses per meter strip,
$M_D = +10440^{kgm}$, $V_D = +42500^{kg}$, $H_D = +2360^{kg}$ at normal state
$+24750$, $+32300$, $+7830$ during earthquake.

Reinforcements, try,
22φ — 30^{cm} ctoc $= 12.70^{cm^2}$ on rear side
19φ — 30 " " $= 9.45$ " front side.
22.15

$p_0 = \frac{22.15}{100 \times 110} = .002$, $d'/h = 5/110 = .046$

Stresses at normal state.
Eccentricity $\varepsilon = \frac{10440}{42500} = 0.246$
$\varepsilon/h = 24.6/110 = .224$
$k = .858$, $L = .099$
$f_c = \frac{10440 \times 100}{.099 \times 100 \times 110^2} = 8.7^{kg/cm^2}$ o/c
$f_s = 15 \times 8.7 \times \left(\frac{105}{.858 \times 110} - 1\right) = 15^{kg/cm^2}$ o/c

Stresses during earthquake. forward.
Eccentricity $\varepsilon = \frac{24750}{32300} = 0.766$
$\varepsilon/h = 76.6/110 = .696$
$k = .27$, $L = .0785$
$f_c = \frac{24750 \times 100}{.0785 \times 100 \times 110^2} = 26.1^{kg/cm^2}$ o/c
$f_s = 15 \times 26.1 \times \left(\frac{105}{.27 \times 110} - 1\right) = 992^{kg/cm^2}$ ok.
Unit Shear $= \frac{7830}{100 \times \frac{7}{8} \times 105} = 0.85^{kg/cm^2}$ ok
Unit bond $= \frac{7830}{3.33 \times 6.91 \times \frac{7}{8} \times 105} = 3.7$ " ok.

Design of Rear wall.
Section at Bottom.

$M_A = -277880^{kgm}$, $V_A = +66590^{kg}$, $H_A = \mp 183860^{kg}$ during earthquake

Total length of rear wall = 8.57^m

Stresses for one meter strip of wall.
$M_D = -32400^{kgm}$, $V_A = +7780^{kg}$, $H_A = +21400^{kg}$ during earthquake.
eccentricity $\varepsilon = \frac{32400}{7780} = 4.17^m$
$\varepsilon/h = 4.17/.9 = 4.63$, $d'/h = 5/90 = .555$
steel, try 22φ — 15^{cm} ctoc $= 25.4^{cm^2}$ on rear
22φ — 30 " " $= 12.7$ " on front
38.1

$p_0 = \frac{38.1}{90 \times 100} = .00423$

$k = .22$, $L = ~~.095~~ .104$
$f_c = \frac{32400 \times 100}{.104 \times 100 \times 90^2} = 38.5^{kg/cm^2} < 35 \times 1.8 = 63$ o/c
$f_s = 15 \times 38.5 \left(\frac{85}{.22 \times 90} - 1\right) = 1900^{kg/cm^2} < 1200 \times 1.8 = 2160$ o/c
Unit shear $= \frac{21400}{100 \times \frac{7}{8} \times 85} = 2.88^{kg/cm^2}$ ok
Unit bond $= \frac{21400}{6.91 \times 6.67 \times \frac{7}{8} \times 85} = 6.2$ " ok Deformed Bars.

for top section of rear wall see parapet wall.

JIUN MASUDA
CONSULTIEG ENGINEEER
JIJI BLDG, TOKYO

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CHECKED BY M. Jinda DATE 4-6-28 PAGE NO A50

CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture

Design of Base.

Case 1. Normal State.

$M_A = -98,000$ kgm, $V_A = +99,760$ kg, $H_A = -75710$ kg

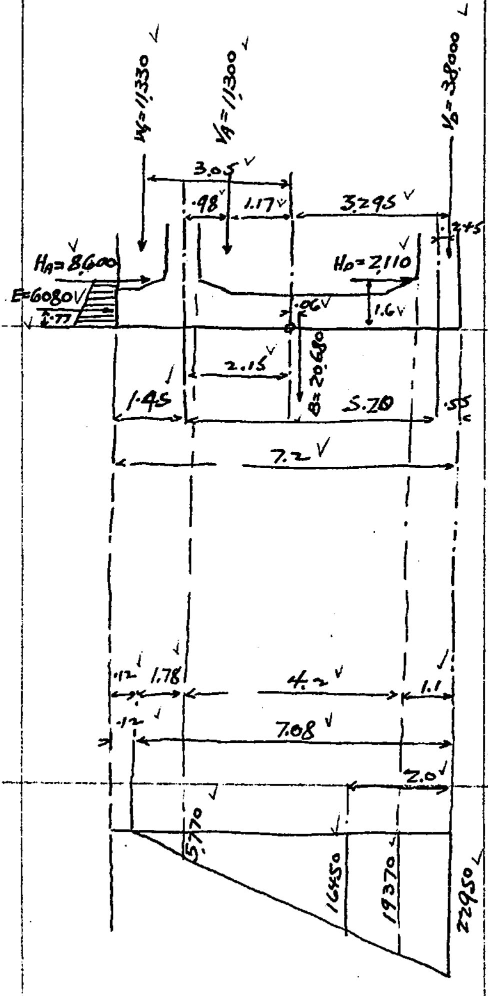
$M_D = +82,050$, $V_D = +334,500$, $H_D = +18,610$

Length of Base for Roadway + Sidewalk = 8.82 m

Stresses for one meter of base average.

$M_A = -11,100$, $V_A = +11,300$, Ecc = .98 m, $H_A = -8,600$ kg

$M_D = +9,300$, $V_D = +38,000$, " = .245, $H_D = +2,110$



Weight of Base 20,680 kg /m strip C.G. vert lev. arm .64, hor. arm 0.06 See page A37

Earth pressure 6,080 " " .77

Earth on rear footing 11,330 kg

Taking moments at center of base.

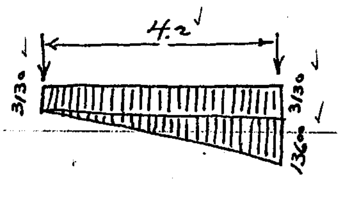
Loads	Hor. forces	Vert. forces		Lev. arm		moment.
V_A		11,300	× -	1.17	=	-13,280
W_1		11,330	× -	3.05	=	-34,600
V_D		38,000	×	3.295	=	125,200
B		20,680	×	.06	=	1,200
E	6080		×	.77	=	4,700
H_A	8600		×	1.60	=	13,860
H_D	2110		×	1.60	=	3,400
	16,790 kg	81,310	×	1.24	=	100,500

Resultant force out of middle third neglecting tension, pressure area 3(3.6-1.24) = 7.08

max. toe pressure = $\frac{81310 \times 2}{1.0 \times 7.08}$ = 22950 kg/m² or (2.1 tons/□')

Load for 2 meters wide of toe, be assumed to be transmitted to lower base. $\frac{22950+16450}{2} \times 2 = 39,400$ kg.

Moment on base Beam.



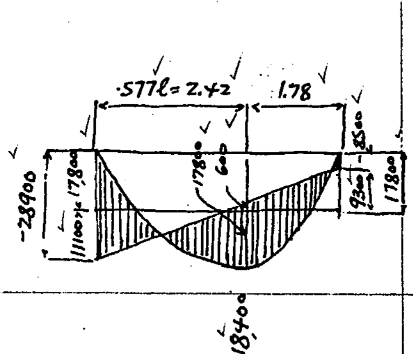
	Rear end.	front end.
upward pres.	5,770	19,370
wt. of base	-2160	-2160
" roadway fill	-480	-480
	3,130 kg/m²	16,730 kg/m²
		3130
		13,600

Moment.

unif. load. $\frac{3130 \times 4.2^2}{8}$ = 6,900

triangular. $\frac{13600 \times 4.2^2}{15.6}$ = 15,380

22,280 kgm



For continuity of base, moment = 22280 × 0.8 = 17,800 kgm at pt. .577 × 4.2 = 2.42 from left.

neg. m assumed same as pos. m.

moment at

front end	rear end.	Shear at front end.
$M_D = +9,300$	$M_A = -11,100$	$\frac{1}{2} \times 3130 \times 4.2 = 6570$
-17,800	-17,800	$\frac{2}{3} \times 13600 \times 4.2 = 38100$
-8,500 kgm	-28,900 kgm	44,670 kg

max. pos. moment = +17,800 + 600 = +18,400 kgm

Case 2. During Earthquake (Seismic force forward).

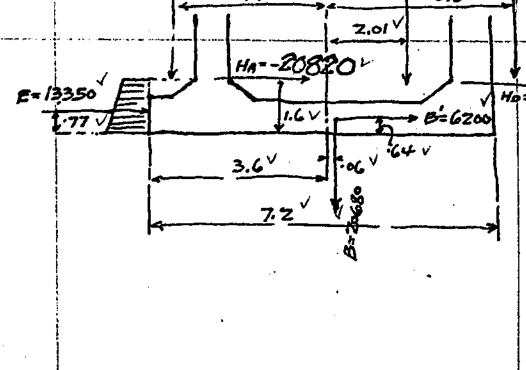
$M_A = -277,880$, $V_A = +66,590$ $H_A = -183,860$

$M_D = +194,970$, $V_D = +254,510$ $H_D = +61,540$

Stresses for one meter of base, average. (8.82 m long)

$M_A = -31,500$ kgm $V_A = +7,570$ kg Ecc. = 4.16 $H_A = -20,820$ kg

$M_D = +22,100$ $V_D = +28,850$ Ecc. = .77 $H_D = +6,980$



Earth pressure. 7100 × 1.6 = 13,350 kg same as for electric railway frame.

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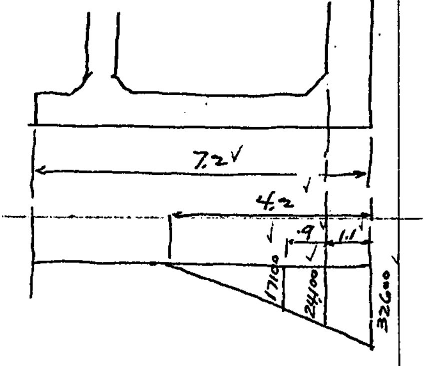
CHECKED BY M. Jinda DATE 4-6-29 **PAGE NO** A51

CALCULATIONS FOR

Design of Juso-Ko-Bashi for Osaka prefecture.

Taking moment about center of base.

Loads	Horizontal forces	Vertical forces		Lev. arm		Moments
W1		11,330	×	-3.05	=	-34,600
VA		7,570	"	2.01	=	15,200
VD		28,850	"	3.82	=	110,200
B		20,680	"	.06	=	1,200
HA	20,820		"	1.60	=	33,300
HD	6,980		"	1.60	=	11,200
B'	6,200		"	.64	=	3,960
E	13,350		"	.77	=	10,280
	47,350 kg	68430 kg	×	2.20 m	=	150,740 kgm



Resultant force outside of middle third. pressure area = (3.6-2.2)3 = 4.2 m

max. toe pressure = $\frac{68430 \times 2}{1.0 \times 4.2}$ = 32600 kg/m²

Loads for 2 meter width of toe be transmitted to lower base assumed.

$\frac{32600 + 17100}{2} \times 2$ = 49,700 kg

moment & shear for base smaller than for case 1.

Case 3 During Earthquake (seismic force backward.)

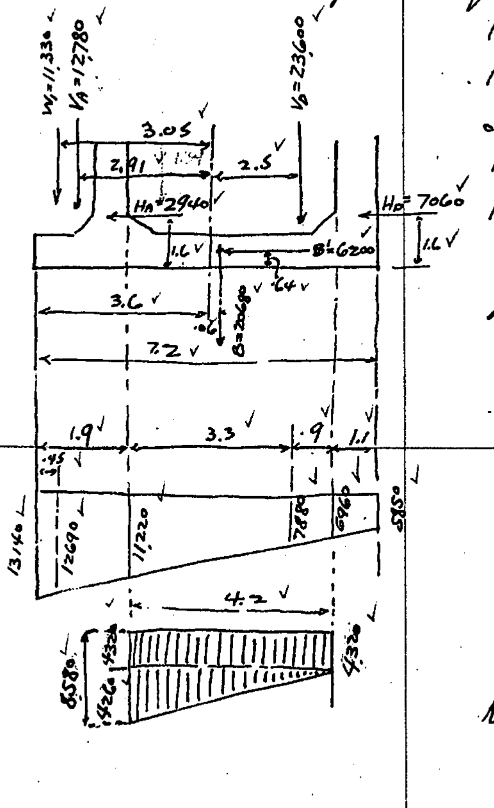
MA = +85,270 kgm, VA = +112,920 kg, HA = +75,950

MD = -113,980, VD = +208,180, HD = -62,290

Stresses for one meter strip of base. (8.82 m long)

MA = + 9,670 kgm, VA = + 12,780, Ecc = .76, HA = + 2,940

MD = - 12,920, VD = + 23,600, " = .55, HD = - 7,060



Taking moment about center of base.

Loads	Hor. forces	Vert. forces		Lev. arm		moment
W1		11,330	×	3.05	=	34,600
VA		12,780	"	2.91	=	37,200
VD		23,600	"	-2.50	=	-59,000
B		20,680	"	-.06	=	-1,200
HA	2,940		"	1.60	=	4,700
HD	7,060		×	1.60	=	11,300
B'	6,200		"	.64	=	3,960
	16,200	68,390	×	.46 m	=	31,560

Resultant force within middle third.

max. toe pressure = $\frac{68390}{1.0 \times 7.2}\left(1 \pm \frac{6 \times .46}{7.2}\right)$ = 13140 kg/m², 5850

Max load on one pile = 12690 × .9 × .9 = 10.3 kg tons

upward pressure	rear end	front end
	11220	6960
wt. of base	-2160	-2160
wt. of roadway fill.	- 480	- 480
	8580	4320
	4320	
	4260	

Shear at rear end.

$\frac{1}{2} \times 4320 \times 4.2$ = 9070

$\frac{2}{3} \times 4260 \times 4.2$ = 11930

21,000 kg

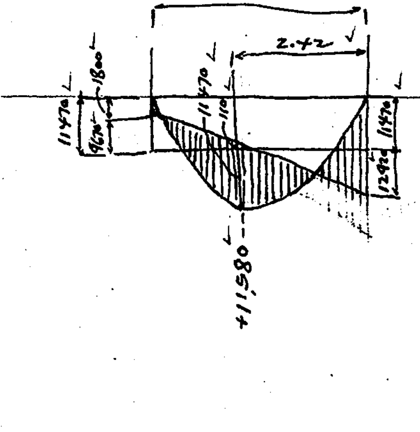
moment

uniform load. $\frac{4320 \times 4.2^2}{8}$ = 9520

triangular " $\frac{4260 \times 4.2^2}{15.6}$ = 4820

14340

for continuity of base moment = 14340 × .8 = 11,470 kgm



Rear end		front end
- 11470		- 11,470
MA + 9,670		MD - 12920
- 1,800 kgm		- 24,390 kgm

pos. moment = 11470 + 110 = +11,580 kgm

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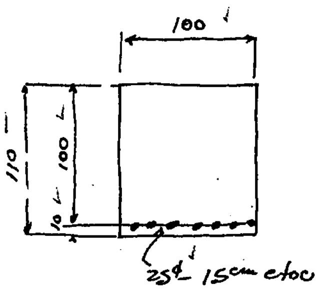
CHECKED BY M. Ineda DATE 4-6-29 PAGE NO A52

CALCULATIONS FOR

Design of Juso-ko-Bashi for Osaka Prefecture.

Section of Base.



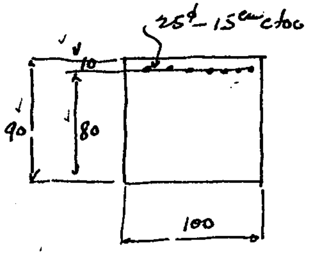
	moment		Shear.	
Front end.	-24390 kgm	seismic	44,670 kg	normal
Rear end.	-28900	normal	21,000 kg	Seismic.
max. pos. m.	+18400			

Rear End.

Effective depth required $= \sqrt{\frac{28900 \times 100}{100 \times 7.18}} = 63.4^{cm}$

use 100cm effective depth with 10cm insulation or total depth of 110cm including fillet.

Steel area req'd $= \frac{28900 \times 100}{1200 \times \frac{7}{8} \times 100} = 275^{cm^2}$ per meter strip.

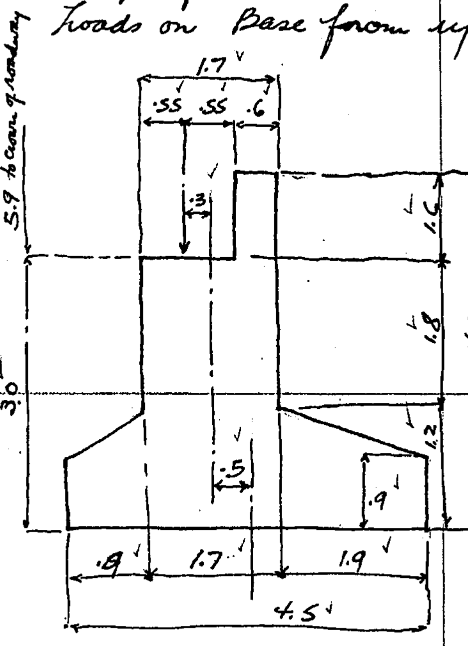


Section at center.

Use same section as for base under Electric railway.
For Front end & at center of span moments & Shears are ~~&~~ slightly smaller than for Electric railway frame base, use the same details.
See page A39

Design of Tower Base under toe for Roadway and Sidewalk.

Loads on Base from upper frame.



		vert. load.	hor. load ($\frac{1}{2} \Sigma H$).
Case 1	normal	39400 kg	8,400 kg →
Case 2	Seismic →	49,700 "	23,680 →
Case 3	" ←	13,730	8100 ←

weight of base

1.9 × 1.05 = 1.995 × .996 = 1.986 .53 = 1.057
1.2 × 1.7 = 2.040 × 2.75 = 5.610 .60 = 1.225
.9 × 1.05 = .945 × 4.03 = 3.810 .53 = .500
4.98 × 2.30 = 11.406 .56 = 2.782

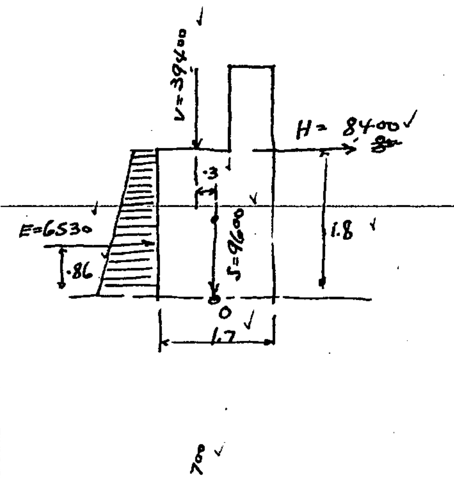
2.25
.05

wt. = 4.98 @ 2400 = 12000 kg base.
Shaft. 4.02 @ " = 9680. Shaft.
21,680 kg

Stresses on Shaft.

-Case 1. at normal state.

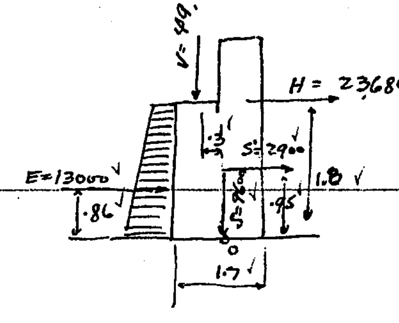
Taking moment about Center of bottom area.



Loads	Hor forces	Vert. forces		Lev. arm		moments.
V.		39400	×	-.3	=	-11,820
S		9600	×	0	=	0
H	8400		×	1.8	=	15120
E	6530		×	.86	=	5620
	14,930	49,000	×	.18m	=	8920 kgm

whole section compression.
Use 19φ bars at 30cm ctoc on both sides.

Case 2. During earthquake. (forward)



V		49700	×	-.3	=	-14,900
S		9600	×	0	=	0
H	23680		×	1.8	=	42600
E	13000		×	.86	=	11,200
S'	2900		×	.95	=	2800
	39580	59300	×	.70	=	41700

$e/h = 70/170 = .41$, $d'/h = 5/170 = .03$

$p_0 = .0011$

$K = .40$ $L = .0825$

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CHECKED BY M. Jinda DATE 4-6-29 PAGE NO A53

CALCULATIONS FOR

Design of Juso-Ko-Boshi for Osaka prefecture.

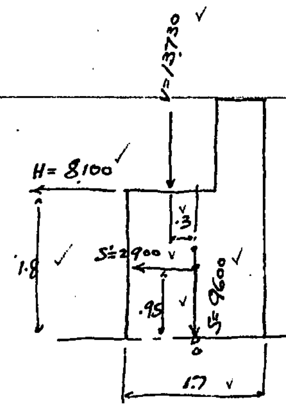
$$f_c = \frac{41700 \times 100}{.0825 \times 100 \times 170^2} = 17.5\ \text{kg/cm}^2 \text{ ok}$$

$$f_s = 15 \times 17.5\left(\frac{165}{.4 \times 170} - 1\right) = 372 \quad \text{ok.}$$

$$\text{unit Shear} = \frac{39580}{100 \times \frac{7}{8} \times 165} = 2.7\ \text{kg/cm}^2 \text{ ok}$$

$$\text{unit bond} = \frac{39560}{5.97 \times 3.33 \times \frac{7}{8} \times 165} = 13.8\ \text{kg/cm}^2 < 9 \times 1.8 \text{ ok..}$$

Case 3 During earthquake (backward.)



Taking moment about center of base.

Loads	Hor forces	Vert forces	Lev. arm	moment
V		13,730	× +.3 =	4120
S		9,600	× 0 =	0
S'	2,900		× +.95 =	2,750
H	8100		× +1.8 =	14,600
	11,000 kg	23,330 kg	÷ .92 =	21,470 kgm

$e/h = 92/170 = .54$, $d'/h = .03$, $p_o = .0011$

$K = .28$, $L = .071$

$$f_c = \frac{21470 \times 100}{.071 \times 100 \times 170^2} = 10.5\ \text{kg/cm}^2 \text{ ok}$$

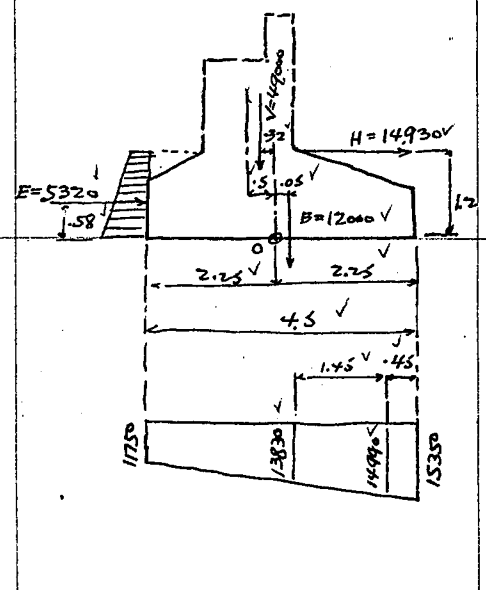
$$f_s = 15 \times 10.5\left(\frac{165}{.28 \times 170} - 1\right) = 390\ \text{kg/cm}^2 \text{ ok.}$$

Stability of Base at Bottom section.

Case 1. At normal state. V = 49,000 kg with .18m right.

H = 14,930 at top of ~~shaft~~ base

E = Same as before = 5,320 kg.



Taking moment about center of base O.

Loads	Hor. forces	Vert. forces	Lev. arm	moment
V		49,000	× -.32 =	-15,700
H	14,930		× 1.2 =	17,930
B		12,000	× .05 =	600
E	5,320		× .58 =	3,090
	20,250 kg	61,000	× .10 =	5,920

Resultant force within middle third.

$$\text{max. toe pressure} = \frac{61000}{1.00 \times 4.5}\left(1 \pm \frac{6 \times .1}{4.5}\right) = 15350\ \text{kg/m}^2 \quad (1.40\ \text{tons/}\square')$$

~~18000~~

or 11,750 (1.08 ")

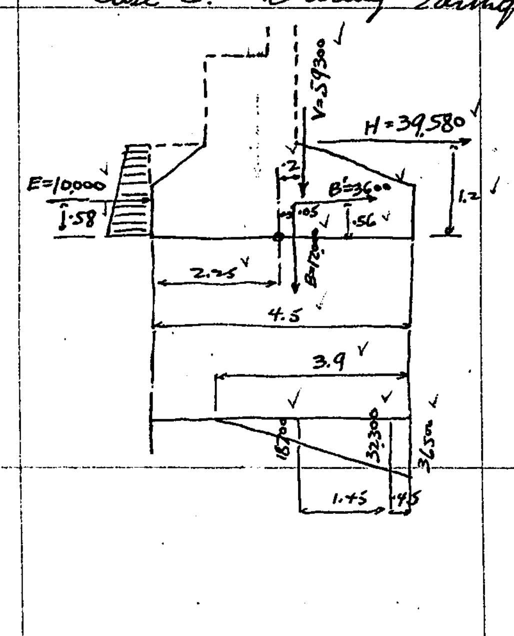
max. load on one pile = 14990 × .9 × .9 = $\underline{\underline{12.1}}$ kg tons.

Case 2. During Earthquake (forward)

V = 59,300 kg with .70m eccentricity

H = 39,580 at top of base.

E = Same as before = 10,000 kg



V		59,300	× .2 =	11,860
H	39,580		× 1.2 =	47,500
B		12000	× .05 =	600
B'	3,600		× .56 =	2,020
E	10,000		× .58 =	5,800
	53,180 kg	71,300 kg	× .95 =	67,780 kgm

Resultant force outside of middle third. pressure area = 3(2.25-.95) = 3.9

$$\text{max. toe pressure} = \frac{71300 \times 2}{1.0 \times 3.9} = 36500\ \text{kg/m}^2 \quad (\text{or } 3.3\ \text{tons/}\square')$$

max. load on one pile = 32,300 × .9 × .9 = 26.2 kg tons.

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CHECKED BY M. Jinda DATE 4-6-30 **PAGE NO** A54

CALCULATIONS FOR

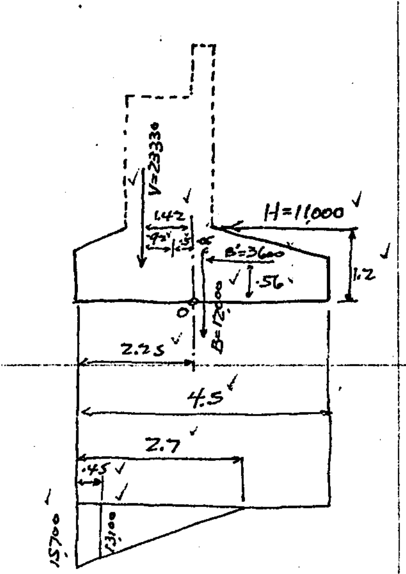
Design of Juso-Ko-Bashi for Osaka Prefecture.

Case 3. During Earthquake (Backward)

$V = 23330^{kg}$ with 0.92^m eccentricity, left.

$H = 11000$ " on top of base.

Taking moment about center of base -o.



Loads.	Hor. forces	Vert. forces		Lev. arm		moment.
V		23330	×	1.42	=	33100
H	11000		×	1.2	=	13200
B		12000	×	-.05	=	-600
B'	3600		×	.56	=	2020
	14600	35330	×	1.35	=	47720

Resultant force outside of middle third. pressure area $3(2.25-1.35)=2.7$

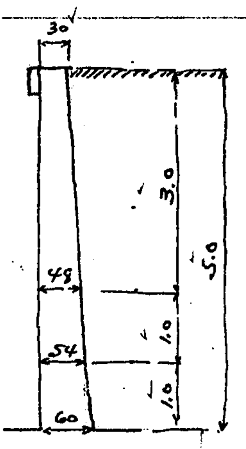
max. pressure at heel $= \frac{35330\times2}{4.5\times1.0} = 15700$ kg/m² or (1.44 tons/o')

Max. load on one pile at heel $= 13100^{kg}\times.9\times.9 = \underline{\underline{10.6}}$ kg tons.

Cantilever footing. Use same details as for Base under Electric Railway frame.

Design of wing wall. Earth pressure at normal state

$\frac{1600\times5.0^2}{6} = 6670^{kg}\times\frac{5.0}{3} = 11100$ kgm per meter strip.



Earth pressure during earthquake.

$\frac{.662\times1600\times5.0^2}{2} = 13250^{kg}\times\frac{5.0}{3} = 22100$ kgm per meter strip.

Latter controls the section.

effective depth req'd $= \sqrt{\frac{22100\times100}{100\times1.8\times7.18}} = 41.4^{cm}$

Use 55^{cm} effective depth with 5^{cm} insulation, total depth 60^{cm}.

Steel area req'd at bottom $= \frac{22100\times100}{1200\times1.8\times\frac{7}{8}\times55} = 21.3$ cm²

unit shear $= \frac{13250}{100\times\frac{7}{8}\times55} = 2.8$ kg/cm² ok

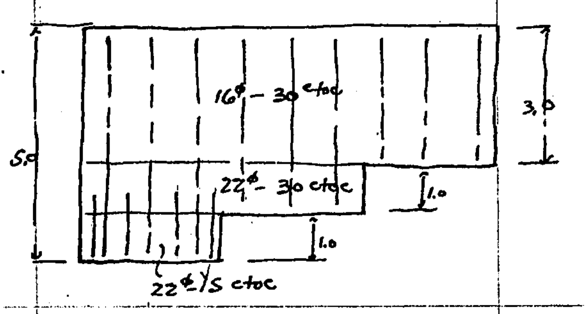
Use 22^ϕ bars — 15^{cm} ctoc $= \underline{\underline{25.4}}$ cm²

unit bond $= \frac{13250}{6.91\times6.67\times\frac{7}{8}\times55} = 6.0$ ok

At 4.0^m from top.

Earth pressure at normal state.

$\frac{1600\times4.0^2}{6} = 4270^{kg}\times\frac{4.0}{3} = 5690$ kgm.



Earth pressure during earthquake.

$\frac{.662\times1600\times4.0^2}{2} = 8480^{kg}\times\frac{4.0}{3} = 11300$ kgm

Latter controls.

Effective depth $= 49^{cm}$.

Steel area req'd $= \frac{11300\times100}{1200\times1.8\times\frac{7}{8}\times49} = 12.2$ cm²

Use 22^ϕ bars — 30^{cm} ctoc. $= \underline{\underline{12.67}}$ cm²

At 3.0^m from top.

Earth pressure during earthquake.

$\frac{.662\times1600\times3.0^2}{2} = 4770\times\frac{3.0}{3} = 4770$ kgm

Effective depth $= 43^{cm}$

Steel area req'd $= \frac{4770\times100}{1200\times1.8\times\frac{7}{8}\times43} = 5.9$ cm²

Use 16^ϕ — 30^{cm} ctoc $= \underline{\underline{6.7}}$ cm²

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MADE BY K. Inaba. DATE 4-6-6 **FILE NO**

CHECKED BY M. Jinda DATE 4-6-30 **PAGE NO** A55

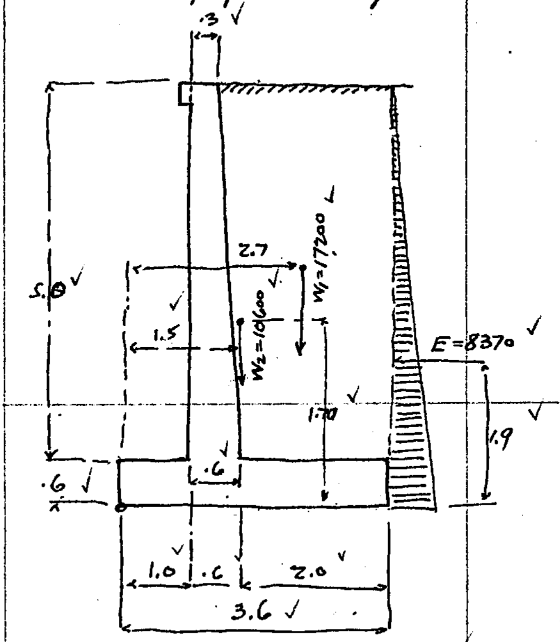
CALCULATIONS FOR

Design of Juso-ko-Bashi for Osaka prefecture.

Stability of Retaining wall 5.6m heigh. Case 1. at normal state

weight of wall.

			hor. arm		vert. arm	
wall.	.45√ × 5.0√ =	2.25 √	× 1.23√ =	2.77 √	× 3.05√ =	6.86 √
base	.6√ × 3.6√ =	2.160 √	× 1.8√ =	3.89 √	× .30√ =	.65 √
		4.41 √	× 1.5 √ =	6.66 √	× 1.70 √ =	7.51 √



Earth fill on rear footing.
2.15√ × 5.6√ = 10.75√ @ 1600√ = 17,200√kg. = W1
wt. of wall. 4.41√ @ 2400√ = 10600√ kg. = W2
Earth pressure at normal state.
$\frac{1600 \times 5.6^2}{6}$√ = 8,370√ kg

Taking moment about toe.

Loads	Hor. force	Vert. force	Lev. arm	moment.
W1		17,200√	× -2.7 √ =	-46,400 √
W2		10,600√	× -1.5√ =	-15,900 √
E	8370 √		× 1.9 √ =	15,900 √
	8370 kg √	27,800√kg	× -1.67 √ =	-46400 √

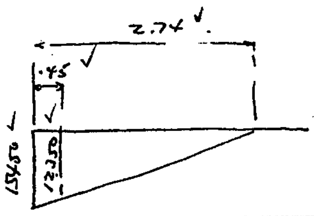
Eccentricity ε = 1.8√ - 1.67√ = .13m √ left.
max. toe pressure = $\frac{27,800}{1.0 \times 3.6}$√ × $(1 \pm \frac{6 \times .13}{3.6})$√ = 9400√ kg/m² √
6040

max load on one pile = 9400√ × .9√ × .9√ = 7.6√ tons.

Case 2. Stability during earthquake. (forward).

Earth pressure = $\frac{.662 \times 1600 \times 5.6^2}{2}$√ = 16,600√ kg.

W1		17200 √	× -2.7√ =	-46400 √
W2		10600 √	× -1.5√ =	-15,900 √
W2'	3180 √		× +1.70√ =	5400 √
E	16,600 √		× +1.9 √ =	31,500 √
	19780 kg √	27,800 kg	× -.914√ =	-25,400 √

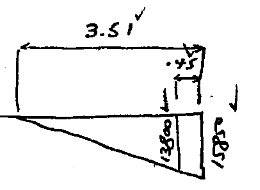


Ecc. = 1.8√ - .914√ = .886√
Resultant force outside of middle third. pressure area = .914√ × 3√ = 2.74√
max. toe pressure = $\frac{27800 \times 2}{1.0 \times 3.6}$√ = 15450√ kg/m²
max. load on one pile = 12350√ × .9√ × .9 = 10.0√ kg tons.

Case 3. Stability during earthquake (backward)

moment about toe.

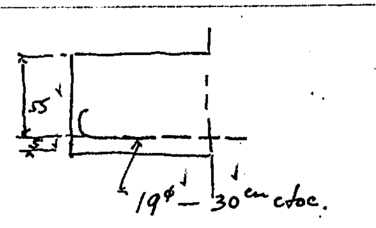
W1		17200√	× -2.7 √ =	- 46,400 √
W2		10600√	× -1.5 √ =	- 15,900 √
W2'	3180 √		× -1.7√ =	- 5,400 √
	3180 √	27800√	× -2.43√ =	- 67,700 √



Eccentricity ε = 2.43√ - 1.8√ = .63√ right.
Resultant outside of middle third pressure area (1.8√ - .63√) × 3√ = 3.51√
max. pressure at heel. = $\frac{27800 \times 2}{1.0 \times 3.51}$√ = 15850√ kg/m²
max. load on one pile at heel = 13800√ × .9√ × .9√ = 11.2√ kg tons.

- Cantilever footing at toe.

upward pressure. 9,400√ kg/m²
downward = 2400√ × .6√ = -1440√
7,960√



moment = $\frac{7960 \times 1^2}{2}$√ = 3980√ kgm
Steel req'd = $\frac{3980 \times 100}{1200 \times \frac{7}{8} \times 55}$√ = 6.90√ cm²
use 19φ√ bars at 30cm√ ctoc = 9.4√ cm²
unit shear = $\frac{7960}{100 \times \frac{7}{8} \times 55}$√ = 1.7√ kg/cm² o/c
unit bond = $\frac{7960}{3.33 \times 5.97 \times \frac{7}{8} \times 55}$√ = 8.3√ kg/cm² o/c

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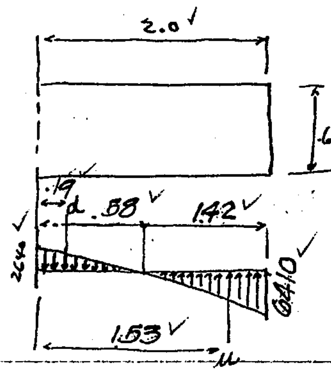
CALCULATIONS FOR

Design of Jusō-ko-Bashi for Osaka prefecture.

Cantilever footing at heel.



case 3.

upward force 15850 during earthquake.

downward force conc. -1440

" earth $1600 \times 5 = -8000$

6410 kg/m² at heel.

$m = \frac{6410}{2} \times 1.42 = \overset{4550}{\cancel{6410}}$ kg $\times 1.53 = 6960$

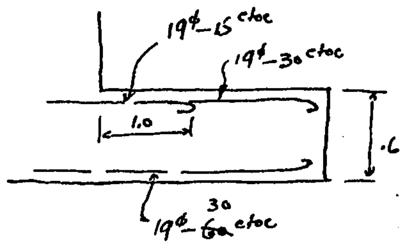
$d = \frac{2640}{2} \times .58 = 765$ kg $\times .19 = -145$

6815 kgm

6800
-1440
-8000
-2640 kg/m² at fixture

for case 2.

downward force earth 8000
concrete 1440
9440 kg/m²



upward force very small and neglected.

moment $= \frac{9440 \times 2.0^2}{2} = -18880$ kgm

effective depth required $= \sqrt{\frac{18880 \times 100}{100 \times 1.8 \times 7.18}} = 38.2$ cm

use 55 cm effective depth with 5 cm insulation

Steel req'd $= \frac{18880 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 55} = 18.2$ cm²

use 19φ bars at 15 cm ctoc $= 18.9$ cm².

unit shear $= \frac{9440}{100 \times \frac{7}{8} \times 55} = 1.96$ kg/cm² o/c

unit bond $= \frac{9440}{5.97 \times 6.67 \times \frac{7}{8} \times 55} = 4.9$ o/c.

Footing for 4.6 meter wall.

use same width + depth as above.

Cantilever footing at toe same as for 5.6 m wall.

Rear footing. approx. calculation.

downward force earth fill $1600 \times 4.0 = 6400$ kg
concrete 1440
7840 kg/m²

moment $= \frac{7840 \times 2^2}{2} = 15680$ kgm.

Steel req'd $= \frac{15680 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 55} = 15.05$ cm²

use 19φ bars. at 15 cm ctoc $= 18.9$ cm².

MADE BY J Masuda DATE 4-6-17 FILE NO P1
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CALCULATIONS FOR

Design of Jiusō Boshi for Osaka Prefecture.

Design of Piers no 7-8-9-10
Superimposed load
Dead Load Floor load complete 14300 kg per lin. meter.
approximate weight of structural Steel.
stringers 1552 • 65.4 = 101.400
intermediate Floor Beams 13 @ 8590 = 111600
End floor Beams 2 @ 4200 = 8400
Lower Lateral Bracing complete = 29500
Top Lateral Sway and portals complete = 65000.
2 trusses assumed 2 @ 215.700 = 431400
Shoes complete 13.000
misc. steel say 1.500
761,800 kg.

structural Steel in one shoe 761.800 ÷ 4 = 190.500 kg.
Floor load 14300 • $\frac{65.4}{4}$ = 234.000
424.500 kg.

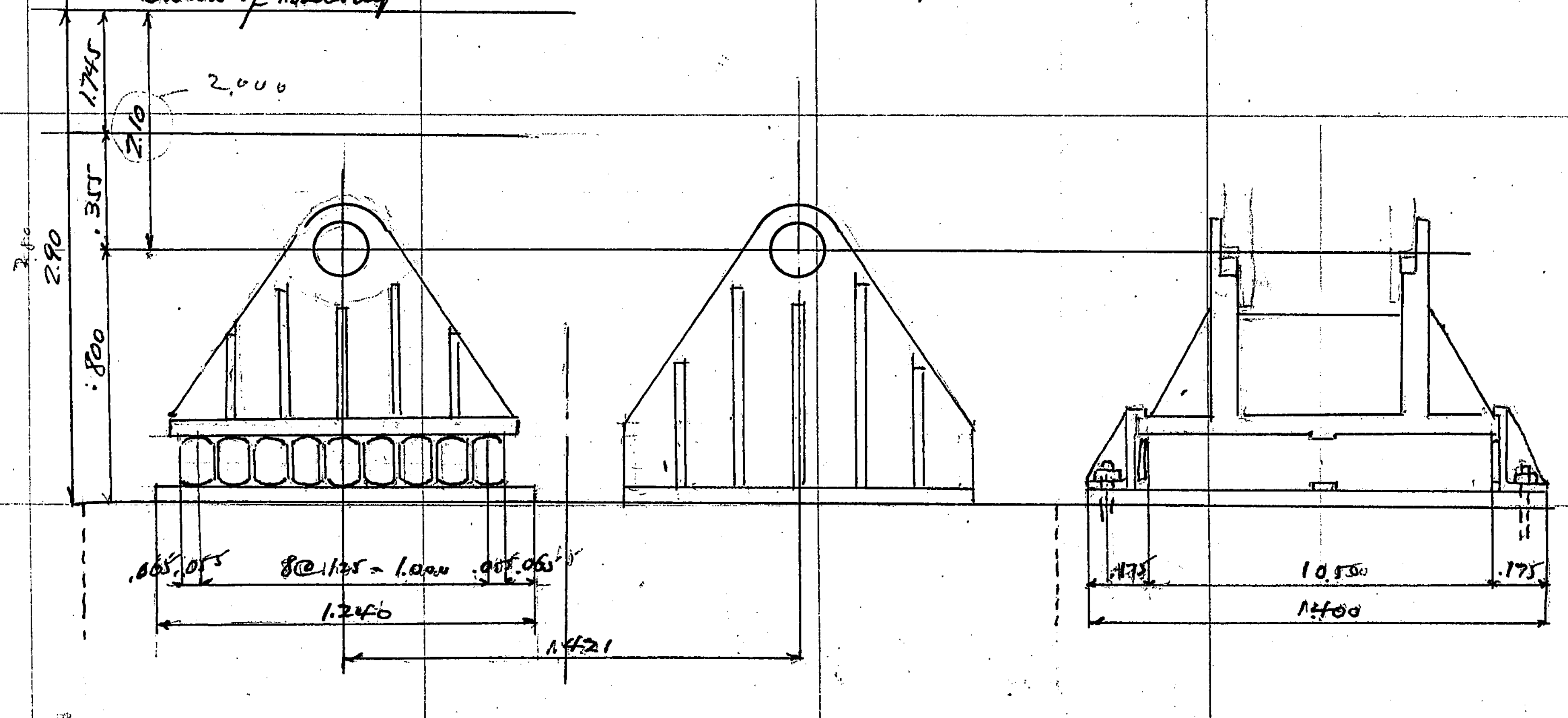
Live Load. On sidewalk. 428 • 2.75 = 1180
On roadway 513 • 4.355 = 2230
3410

Electric Car Loading 5 cars @ 28.000 = 140.000
140.000 ÷ 64. = 2190
impact $\frac{20}{60+64}$ = 16.1% = 352
2542
5950

Extra loading for motor truck assumed 550
for one truss 6500 kg. per lin meter.
Live Load on shoe 6500 • $\frac{65.4}{2}$ = 213.000 kg.

Summary load on one shoe Dead Load 424500
Live Load 213000
637,500 kg.

Length of roller. 6½" = 16.5 cm roller 45d = 45 × 16.5 = 742 kg per cm
637.500 ÷ 742 = 860 cm 10 rollers @ 86 cm
9 rollers @ 95.5 cm



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MADE BY J. Masuda DATE 4-6-18 **FILE NO** P2
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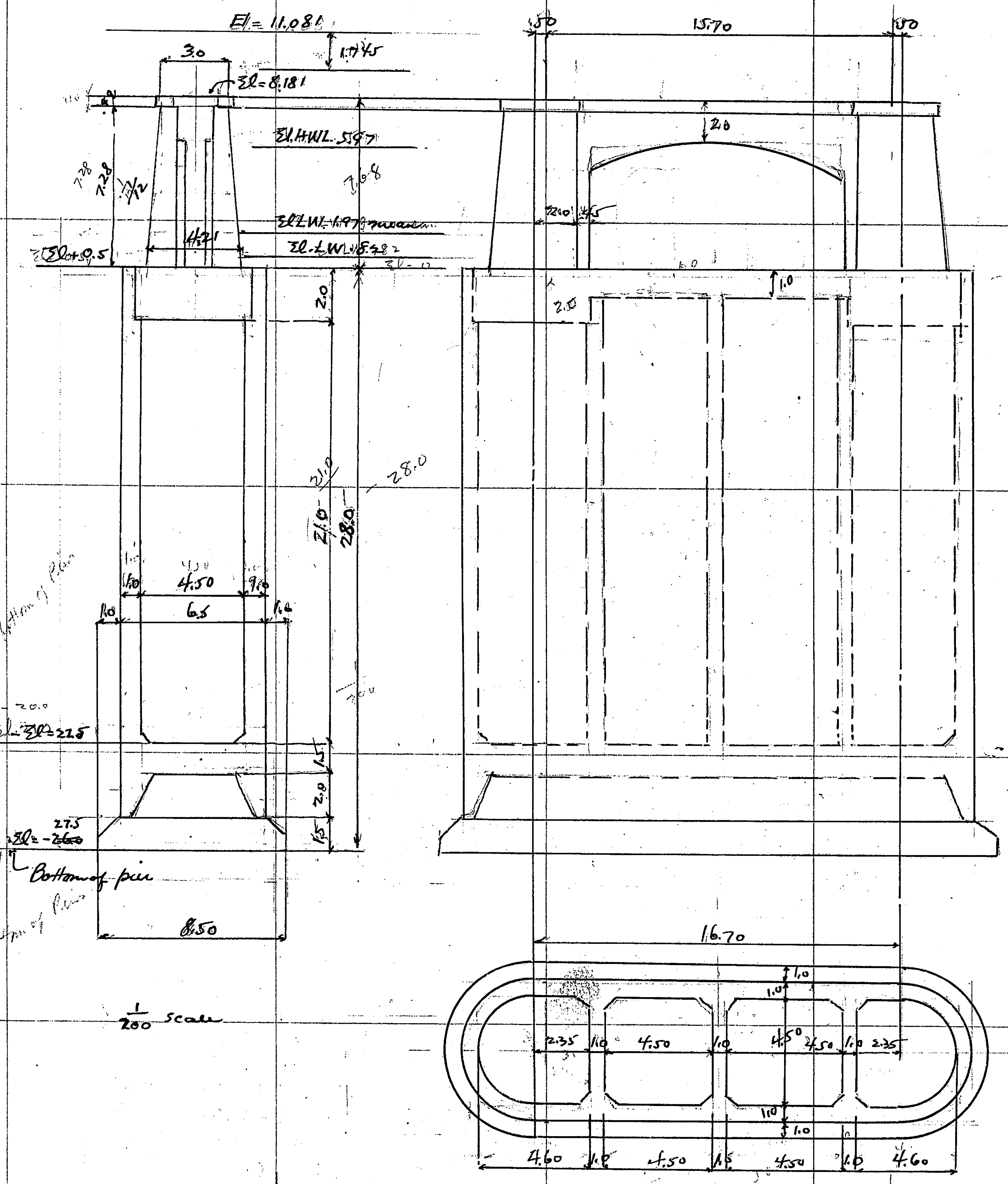
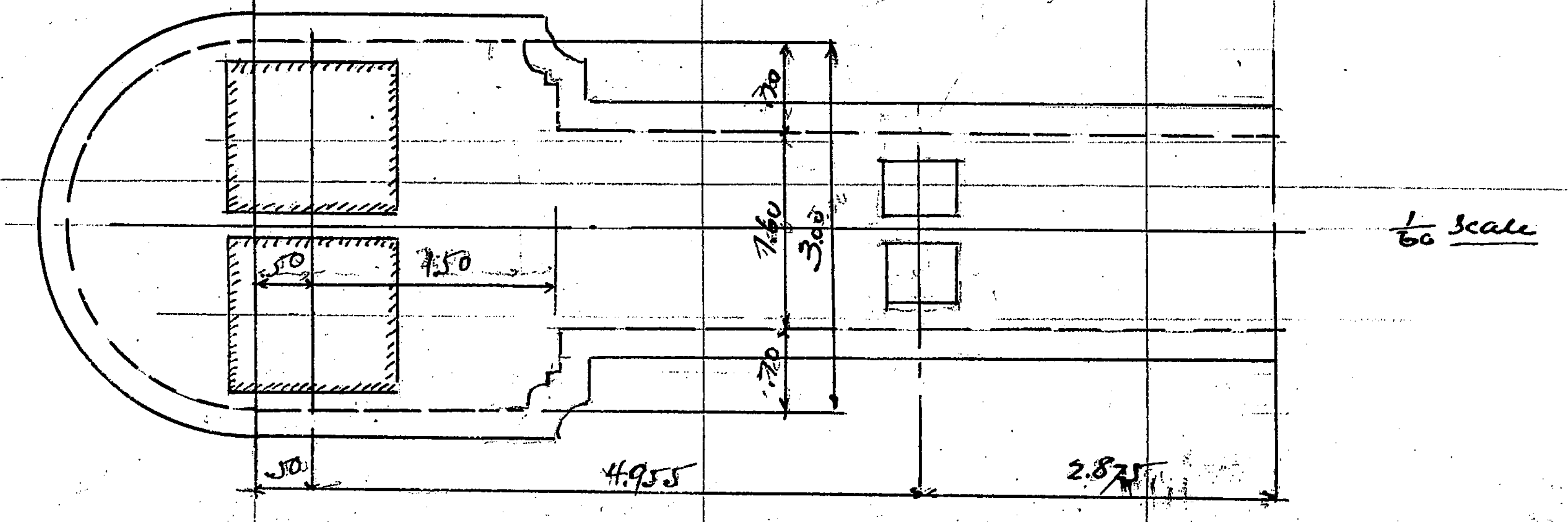
CALCULATIONS FOR

Design of Jūsō Bashi for Ōsaka Prefecture.

Superimposed load on pier. 653,500 × 4 = 2614,000 kg.



1/200 scale



1/60 scale

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CALCULATIONS FOR

Design of Jūsō-Bashi for Ōsaka Prefecture.

Volume of Concrete in Pier.
Coping. ½ 3.4^{ϕ} = 4.54
3.4 × 2.4 = 8.15
12.69 × 2 × .40 = 10.1
between shafts $\overset{2.0}{2.4}$ × 11.9 × 0.4 say = 11.4
21.5 cubic meters
Shaft ½. 3.0^{ϕ} = 3.53 ½ 4.21^{ϕ} = 6.96
3. × 2 = 6.00 4.21 × 2 = 8.42
9.53 15.38
15.38

24.91 ÷ 2 = 12.46 × 7.28 = 90.60 cubic meters
For 2 shafts 2@ 90.6 = 181.2 cubic meters
Strut between shafts 1.6 × 2.0^{abt} × 11.8 = 37.7
Col. 2 - .45 × 1.6 × 7.28 = 10.5
web. 11.8 × 1.00 × 5.28 = 62.3
110.5
Coping 21.5
2 shafts 181.2
strut & web 110.5
313.2 @ 2400 = 751.000 kg.

Concrete Caisson Outside area = 6.5^{ϕ} = 33.2
6.5 × 16.70 = 108.5
141.7
Space between partition walls. at middle Both ends.
4.5 × 4.5 = 20.25 4.5^{ϕ} 15.90
2 × .60 . .60 = .72 2 × 2.35 × 4.5 = 21.20
19.53 × 2 = 39.06 sq meters 37.10
36.38 − .72
75.44 36.38
area of Cross section of caisson 141.70
− 75.44
66.26 sq meters
Volume of Concrete in top slab.
141.7 × 2.0 = 283.4
− 39.06
244.34 cubic meters
Summary of Concrete
Top slab 2 × 141.7 − 39.06 = 244.34 @ 2400 = 587.000
well shell 66.26 × 21.0 = 1392.00 @ 2400 = ~~3340.000~~ 3344.000
bottom fill & shell 141.7 × 3.5 = 495.00 @ 2200 = 1090.000
base say 198.75 × 1.5 = 298.00 @ 2200 = 655.000
2329.34 cubic meters 5672.000 kg.
5672000

Volume of filling
both ends 36.38 × 21.0 = 763 ~~water~~ sand fill @ 1900 = 763.000
middle 39.06 × 22.0 = 860 water fill @ 1000 = 860.000
1623.000
1623000
Total load of pier.
Shaft and coping 751.000 751.000
Caisson 5672.000 5672.000
filling say 1623.000 1623000
8046000
8046.000 kg.
Superimposed dead and live loads 637500 × 4 = 2550.000
10596.000 kg.

Bottom area = 198.75 Unit Pressure = $\frac{10596000}{198.75}$ = 53300 kg/m2
4.95 tons/□'

Design of Jiuso-Bashi for Osaka Prefecture

Assuming skin friction 250#/□' or say 1400 kg per sq meter and the depth of caisson below firm ground assumed 24.5 meters

circumference 6.5 ф = 20.4
2 × 16.7 = 33.4
53.8 × 1400 × 24.5 = 1845000 kg.

Total load 10596000
− 1845000
8751000 ÷ 198.75 = 44000 kg/m²
4.09 tons/□'

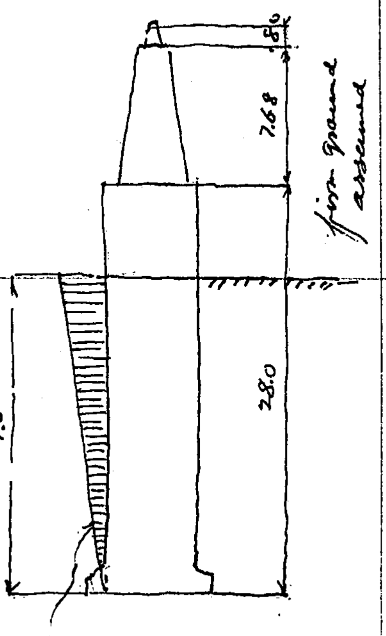
Assuming skin friction 350 #/□' or say 1710 kg/m²
Total load 10596000
friction 53.8 × 1710 × 24.5 = − 2255000
8341000 ÷ 198.75 = 42000 kg/m²
3.90 tons/□'

Stability of Pier during Earthquake
Horizontal force k = 0.30
Superimposed dead load 424500 × 4 = 1700.000 kg.

Centre of gravity for shafts etc | Top slab of well

	vol	arm from top				
Coping	21.50 ×	0.20 =	4.30	283.4 ×	1.0 =	283.4
2 shafts	181.20 ×	3.50 abt =	634.00	− 39.06 ×	1.5 =	− 58.5
Strut & web	110.20		371.80	244.34	.92 m	224.9
	313.20	3.23 m	1010.10			

		Hor. Force	arm	moment
Dead Load superstructure	1700.000 × 0.3 =	510.000 ×	36.48 =	18600.000
shaft and coping	751.000	225.300	32.45 =	7300.000
top slab	587.000	176.100	27.08 =	4940.000
well shell and partition	3340.000	1002.000	15.50 =	15550.000
bottom	1090.000	327.000	3.28 =	1062.000
base	655.000	196500	0.75 =	147.000
water fills sides	763.000	228900	15.50 =	3545.000
" " Center section	860.000	258000	16.00 =	4125.000
	9746.000	2923800		55269000



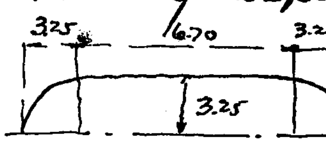
Reaction due to Hor. force assumed thus

Moment due to Hor. Reaction 2923800 × 2/3 × 24.5 = 47700.000
7569000

max intensity $\frac{2923800 \times 2}{24.5}$ = 23900 kg

For whole width of well
Counting ½ skin friction effective for resistance of overturning
Load on base 9746.000
Skin friction $\frac{2255000}{2}$ = 1127500
8618500 kg.

Resisting couple



16.70 × 3.25 = 54.2
10.21 × 2.07 = 21.2
26.91 2.8 75.4
arm 2.8 × 2 = 5.4 meters

Resisting Couple $\frac{1127.5000}{2}$ × 5.4 = 3040.000 kgm
Net moment = 7.829.000 − 3040.000 = 4529.000 kgm

Ecc = 4529.000 ÷ 8618500 = 0.525 meter

Bottom area of base 8.5 ф 56.75
8.5 × 16.70 = 142.00
198.75 sq meters

Design of Jiusō-Bashi for Osaka-Prefecture.

moment of inertia of base area 8.5^{ϕ} $0.0491 \times 8.5^4 = 256.0$

$\frac{1}{2} \times 16.70 \times 8.5^3 = 854.0$

$1110.0 \ m^4$

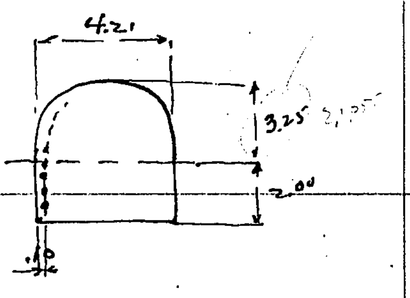
Unit bearing direct pressure $= \frac{8618500}{198.75} = 43500 \ kg/m^2$

bearing due to bending moment $4529.000 \times \frac{4.25}{1110} = \pm 17300 \ kg/m^2$

Summary for bearing		
direct load	43500	43500
bending	17300	-17300
	60800	24400
	5.65 tons/□'	2.27 tons/□'

Reinforcement in the shaft during Earth quake.

				Hor. Force	arm	
Dead Load superstructure	1700.000 ÷ 2 =	850.000	× 0.3 =	255000	× 8.48 =	2.160.000
shaft with strut web neglected	say	301.000	" =	90300	× 4.90 abt =	443.000
		1151.000		For one shaft.		2603.000



moment of inertia of bottom section.

Circular End $I = \frac{0.0491}{2} \times 4.21^4 = 7.70$

Equivalent square $b = \frac{7.70 \times 12}{4.21^3} = 1.24$ meters

2.00

Total width —— 3.24 meters.

assumed reinforcing bar 25^{ϕ} bars

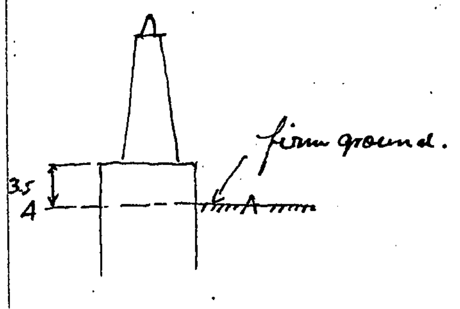
$2 \times 30 \times 4.91 = 294.5 \ \square cm$ $\frac{294.5}{136000} = .216 \%$

area of Concrete $324 \times 421 = 136000 \ \square cm$

Value of $k = 0.342$ Coef $= 0.0882$ stress in concrete $f_c = \frac{260300000}{324 \times 421^2 \times 0.0882} = 51.5 \ kg/cm^2$

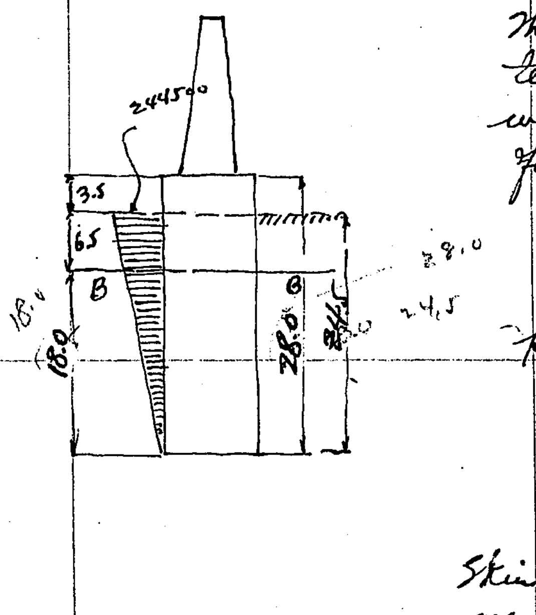
Steel Stress $f_s = 15 \times 51.5 \left(\frac{411}{421 \times 0.342} - 1 \right) = 1440 \ kg/cm^2$

Bending moment of well. section AA



				Hor. Force	arm	
Dead Load superstructure		1700.000	× 0.3 =	510.000	× 11.98 =	6100.000
shaft +		751.000		225.300	7.95 =	1.790.000
Top slab		587.000		176100	2.58 =	455.000
shell and partition		238 000		71400	0.75 =	535.000
filling	129000 × 1.5 36380 × 1.5	54500		16000	0.75 =	12.000
"	39060 × 2.5	92700		27800	1.25 =	35.000
		3423200		1026600		8927.000

Bending moment Section BB.



Dead Load superstructure		1700.000	× 0.3 =	510.000	× 18.48 =	9410.000
Shaft		751.000		225.300	14.45 =	3250.000
Top Slab.		587.000		176100	9.08 =	1600.000
well shell	159000 × 8 =	1272.000		382.000	4.00 =	1530.000
Filling	36380 × 8 =	291.000		87300	4.00 =	350.000
"	39060 × 9 =	352.000		105500	4.50 =	475.000
		4310.000 643 000		1486200		16.615.000

Horizontal reaction $239000 \times \frac{18.0}{24.5} = \frac{239000}{\frac{176000}{53000}}$

moment about BB $176000 \times 6.5 \times 3.25 =$ ~~3.720.000~~

$53000 \times \frac{6.5^2}{3} =$ 745.000

4.465 000

Skin friction $1710 \times 5.38 \times 6.5 = 598000$

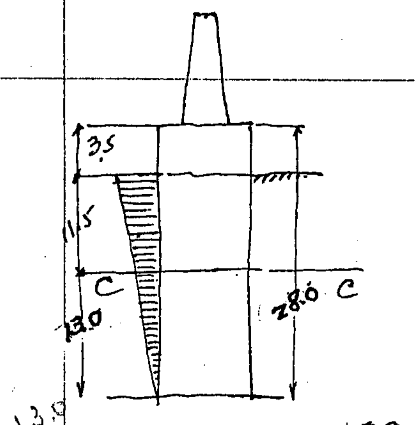
assuming ½ friction as effective $\frac{598000}{2} = 299000$ kg.

Design of Jiuso-Bashi for Osaka-Prefecture.

Resisting moment = $\frac{299000}{2}$ × 5.4 = $\frac{1615.000}{2}$ = 807.000 kgm.

Direct load	4.310.000	moment	16.615.000
	− 299.000		− 4465.000
	4011.000 kg.		− 807.000
			11,343,000 kgm.

Bending moment Section CC. 15 meters from top.



Dead Load superstructure		1700.000	× 0.3 =	510.000	× 23.48	=	12000.000
Shaft		751.000		225.300	× 19.45	=	4375.000
top slab		587.000		176100	× 14.08	=	2470.000
well shell	159000 × 13 =	2065.000		619500	× 6.50	=	4025.000
filling	36380 × 13 =	472.000		141600	× 6.50	=	920.000
"	39060 × 14 =	546.000		163800	× 7.00	=	1142.000
		5103000		1836300			24.932.000
		10.18000					
		6121000					

Horizontal reaction m = 127000 × 11.5 × 5.75 = 8400.000

$239000 \times \frac{13.0}{24.5} = \frac{\overset{239000}{127000}}{112000}$ ~~112000~~ × $\frac{11.5^2}{3}$ = 4950.000

− 13.350.000
11.582.000

frictional couple 1710 × 53.8 × 11.5 = 1060.000

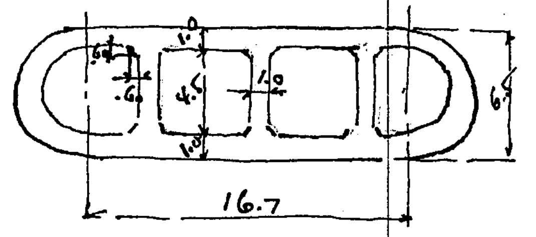
½ skin friction assumed 1060.000 ÷ 2 = 530.000 kg.

moment = $\frac{530.000}{2}$ × 5.4 = 1430.000

−1.430.000
10.152.000 kgm

Direct load on ring.
5103.000
− 530.000
4573,000 kg.

Section of well.



cross sectional area = 66.26 □m

moment of inertia.

both circular Ends	I = 0.0491 ($6.5^4 - 4.5^4$)	= 67.5
straight portion	16.7 × 2 × 2.75^2 + 2 × $\frac{1}{12}$ × 16.7 × 1^3	= 255.28
Rib	3 × $\frac{1}{12}$ × 4.5^3	= 22.80
fillet	12 × $\frac{.6 \times .6}{2}$ × 2.05^2 + 12 × $\frac{.60 \times .60^3}{36}$	= 9.10
		354.68

max moment at Section BB. $\overset{11.343.000}{11.343.000}$ kgm

direct load 4.011.000 kg. ✓

unit stress due to direct load 4.011.000 ÷ 662600 = 6.05 kg/cm² ✓

fibre stress $\overset{343}{11.343.000.00}$ × $\frac{3.25}{354.68}$ = $\overset{104}{104.000}$ kg/m² or $\overset{10.4}{10.4}$ kg/cm²

Summary stress	direct load	6.05 ✓	6.05 ✓
	due to bending	10.40	10.40
		16.45	4.35 tension.
		16.45	4.35

Center of gravity of half section.

	area					
	16.7 × 1.0 =	16.70	×	2.75	=	45.90
Ring.		8.62	×	1.775	=	15.30
Rib.	3 × 2.25 =	6.75	×	1.125	=	7.60
	6 × $\frac{.36}{2}$ =	1.08	×	2.050	=	2.21
		33.15		2.14		71.01

2.14 × 2 = 4.28 m

stress due to bending $\overset{343}{11343.000}$ ÷ 4.28 = $\overset{50}{2650.000}$ kg T or C.

4011000 ÷ 2 = − 2005500

644500 kg. tension.

Steel Section reqd = $\frac{\overset{644500}{644.500}}{1200 \times 1.8}$ = $\overset{299.0}{299.0}$ □cm in one half section of well.

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CHECKED BY________ DATE________ PAGE NO A63

CALCULATIONS FOR

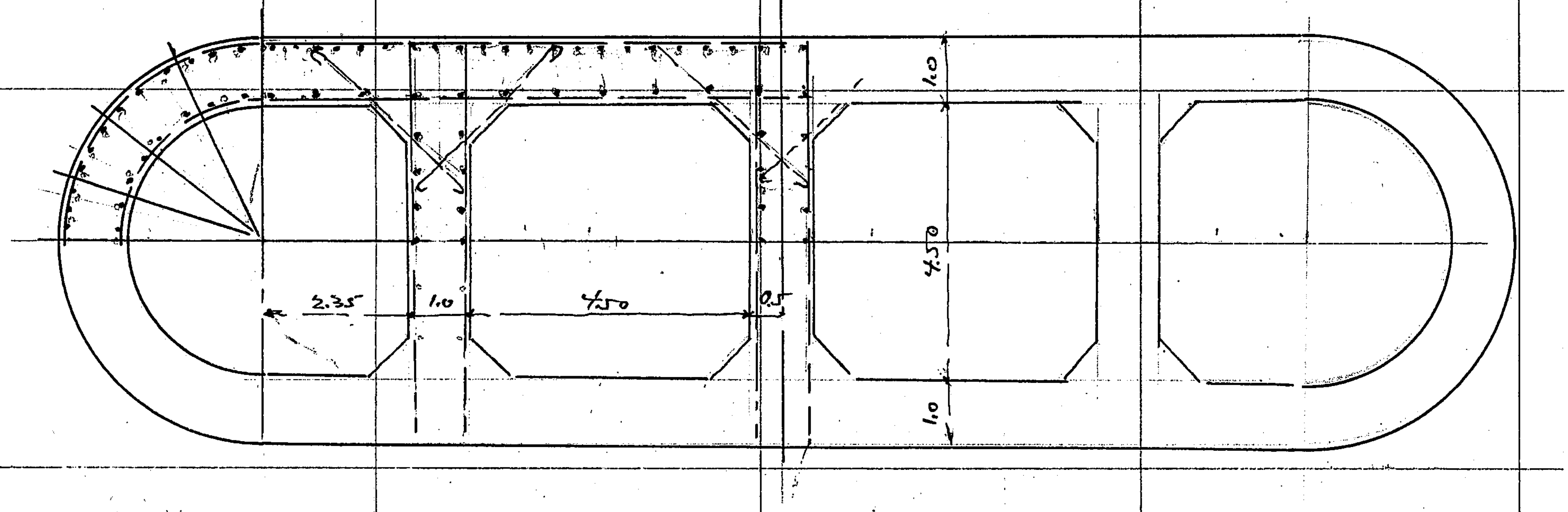
Design of Jūsō-Bashi for Osaka-Prefecture

vertical reinforcement required for temperature stress.

$\frac{1}{2}$ cross section = 33.15 □m

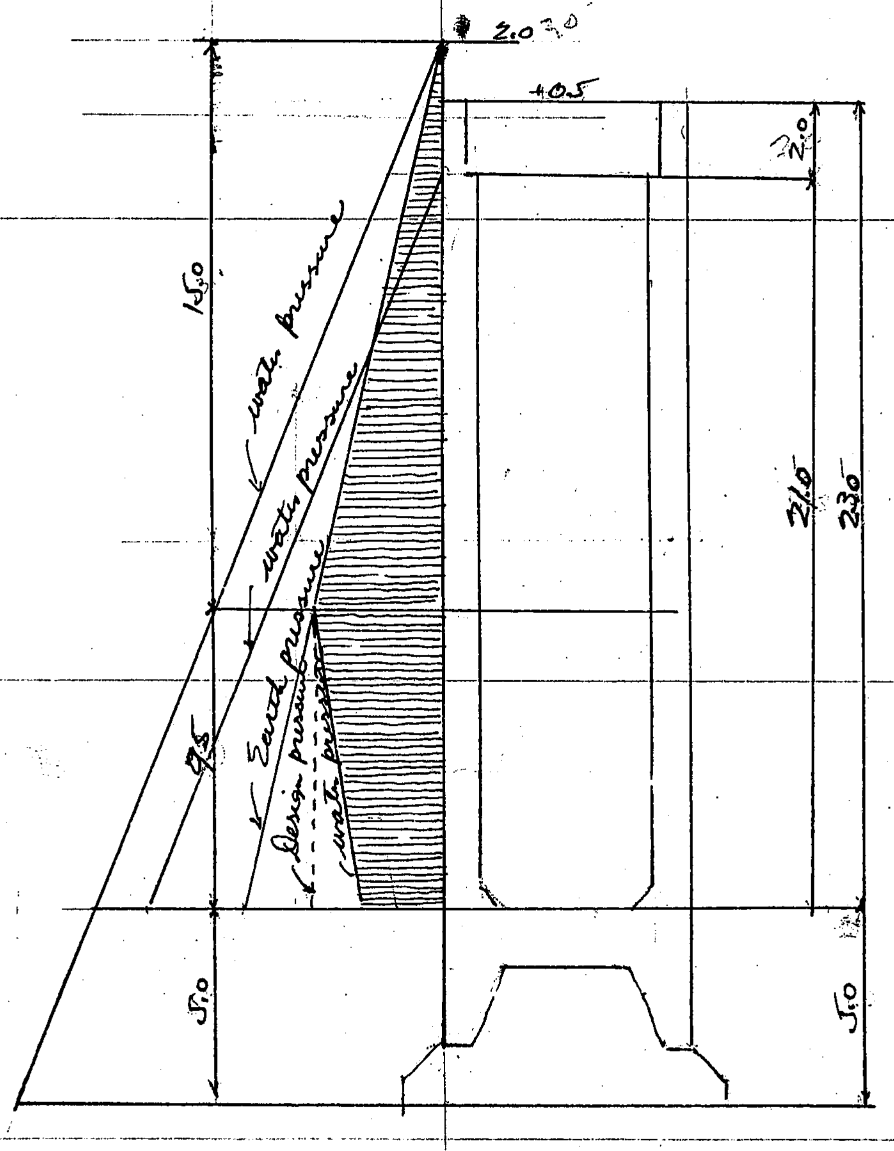
= 331500 × $\frac{3}{1000}$ = 993.0 □cm

Use 25 mm φ area = 4.9 cm² 126 × 4.9 = 617.0 □cm used.



Reinforcement in the ring.

Earth pressure = $\frac{1}{3}$ × 1700 × h = 567 h.
water pressure = 1000 × h



Depth	Earth P		water P	Design P
1	567			
2	1134			
3	1700			1700
4	2270	.5	500	2270
5	2835	1.5	1500	2835
6	3400	2.5	2500	3400
7	3970	3.5	3500	3970
8	4530	4.5	4500	4530
9	5100	5.5	5500	5700
10	5670	6.5	6500	5670
11	6250	7.5	7500	6250
12	6810	8.5	8500	6810
13	7380	9.5	9500	7380
14	7950	10.5	10500	7950
15	8510	11.5	11500	8510
16	9080	12.5	12500	"
17	9650	13.5	13500	"
18	10200	14.5	14500	"
19	10790	15.5	15500	"
20	11340	16.5	16500	"
21	11900	17.5	17500	"
22	12500	18.5	18500	"
23	13050	19.5	19500	"
24	13620	205	20500	"

moment at Circular Ends assumed $\frac{1}{16}wd^2$ where d = diameter of ring
moment at partition wall " $\frac{1}{10}wl^2$ " l = length between ℄ partitions
moment at center of wall . $\frac{1}{12}wl^2$ " " " " " "

Reinforcement in the ring figured for $\frac{1}{2}$ moment throughout; depth of beam at partition wall greater than depth between walls. will be ok using same reinforcement as for center of beam

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CALCULATIONS FOR

Design of Jiusō Bashi for Osaka-Prefecture

Depth of Earth	depth from top of well		moment $\frac{1}{12}w \times 5.5^2$	moment $\frac{1}{16}w \times 5.5^2$	$\frac{1}{12}$ m between partitions section Rqd	spacing 19 bar 2.840
1						
2	0.5					
3	1.5	1700	4290	3220	4.29	
4	2.5	2270	5720	4290	5.72	49.6
5	3.5	2835	7150	5350	7.15	39.7
6	4.5	3400	8570	6420	8.57	33.1
7	5.5	3970	10000	7500	10.00	28.4
8	6.5	4530	11420	8570	11.40	25.0
9	7.5	5100	12870	9650	12.87	22.0
10	8.5	5670	14300	10720	14.30	19.8
11	9.5	6250	15750	11800	15.75	18.0
12	10.5	6810	17200	12850	17.20	16.5
13	11.5	7380	18600	13930	18.60	15.2
14	12.5	7950	20020	15030	20.10	14.1
15	13.5	8510	21450	16080	21.5	13.2
16	14.5	"	"	"		
17	15.5	"	"	"		
18	16.5	"	"	"		
19	17.5	"	"	"		
20	18.5	"	"	"		
21	19.5	"	"	"		
22	20.5	"	"	"		
23	21.5	"	"	"		
24	22.5	"	"	"		

Pressure in working chamber
Total weight of well.

well shell with partitions = 66.26 × 21.0 @ 2400 = 3340.000
Top well shell only say 50.00 × 2.0 @ 2400 = 240.000
Ceiling of working chamber 141.7 × 1.5 @ 2400 = 510.000
wall of " " 66.26 abt @ 2.0 × 2400 = 318.000
4408.000 kg.

water filling say 75.44 ✓ × 21.5 ✓ × 1000 = 1,620.000 ✓ kg.

Skin friction

			circumference	depth	
350 #/□'	$\frac{350}{2.2} \times 3.28^2$ = 1710 kg/m²	×	53.8	× 26.5	= 2440.000
400 "	1960 "		"	"	= 2790.000
450 "	2200 "		"	"	= 3140.000

Theoretical water pressure depth of water 29.5 meter p = 29500 kg/m²
upward pressure = 141.7 × 29.500 = 4175.000 kg.

	350 #/□'	400 #/□'	450 #/□'
Total downward load	4408.000	4408000	4408 000
Skin friction	2440 - 2440.000	2790 - 2790.000	3140 - 3140.000
net load at bottom	~~+968.000~~	1618.000	1268.000
water filling	1620.000	1620.000	1620.000
	3588.000 kg	3238 000 kg	2888.000 kg.
	$\frac{3.588}{4175} = 86\%$	$\frac{3238}{4175} = 77.5\%$	$\frac{2888}{4175} = 68.9\%$

Pressure in working chamber 29.5 × .86 = 25.4 kg/cm² 36.1 #/□" ; 29.5 × .775 = 2.29 kg/cm² 32.6 #/□" ; 2.95 × .689 = 2.03 kg/cm² 28.8 #/□"

Design of Juso- Bashi for Osaka Prefecture

Design of working chamber

Theoretical water pressure depth 29.5 meters $P = 29500$ kg/m²
2.95 kg/cm² = 41.9 #/□"

Outside Earth pressure on walls of working chamber

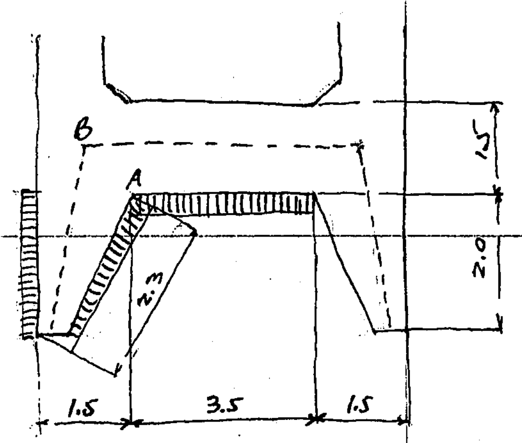
h = 28.0 assumed $P = \frac{1}{3} \times 1700 \times 28.0 = 15900$ kg. unif. on wall assumed

Ballanced pressure Pressure in working chamber 29500
Earth pressure −15900
13600 kg/m²

75% Theoretical pressure = 29500 × .75 = 22100 kg/m² (31.4 #/□").

Actual pressure in working chamber will be less than 31.4 #/□". However design the part of working chamber for 2.8 kg/cm² or 39.7 #/□" pressure.

walls of working chamber



Design approximate only.

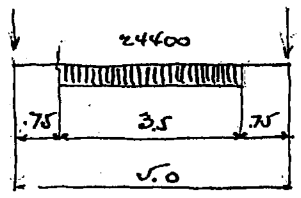
Pressure 28000
Concrete 1.5 × 2400 = −3600
24400 kg/m²

moment at A Pressure = 24400 × 2.3 = 56100 kg
at A m = 56100 × 1.15 = 64500 kgm
at B m = 56100 × 1.75 = 98200

Full Earth pressure from outside $\frac{1}{3} \times 1700 \times 26.5 = 15000$ kg/m²
net pressure from inside 24400
−15000
9400 kg.

Total pressure = 9400 × 2.3 = 21600 kg.
m at A = 21600 × 1.15 = 24800 kgm
at B = 21600 × 1.75 = 37800 "

Ceiling of working chamber



Reaction $24400 \times \frac{3.5}{2} = 42700$ kg.
moment at center 42700 × 2.5 = 109.200
less $\frac{42700}{2} \times \frac{1.75}{2}$ = −18700
90500
− moment cantilever arm −37800
52700 kgm

Tension on top of slab.

This stress carried by on 4 supports reduce $m = \frac{52700}{2}$ to 26350 kgm

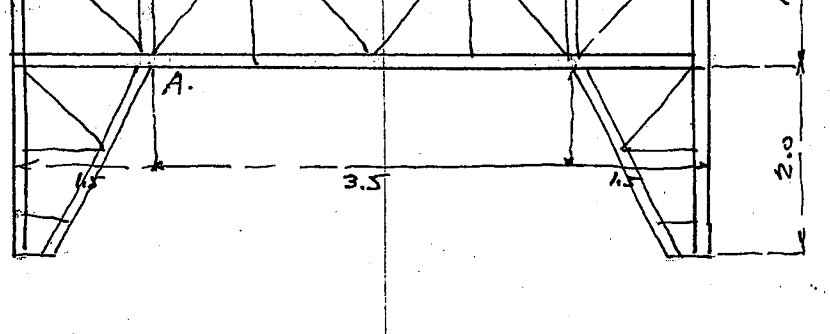
moment due to downward load span length assumed 4.5 meters

no pressure in working chamber

water fill 22.0 × 1000 = 22000
Concrete slab 1.5 × 2400 = 3600
25600

$m = \frac{1}{10} \times 25600 \times 4.5^2 = 51900$ kgm at center and ends.

Being supported on 4 sides $m = \frac{51900}{2} = 25950$ kgm



Cantilever wall design moment at A
m = 24800 × 2.25 = 55900 kgm
Effective depth say 1.500 − 0.08 = 1.42

Flangestress = $\frac{55900}{1.42}$ = 39400 kg
section reqd = 39400 ÷ 1200 = 32.8 cm²

use 2Ls 125 × 90 × 10 = 41.0 − 4.4 = 36.6 cm² net

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CHECKED BY________ DATE________ PAGE NO A66

CALCULATIONS FOR

Design of Jiusō-Bashi for Osaka-Prefecture.

Slab at Ceiling
$m = 25950 \times 2.25 = 56200$
Flange stress $= \frac{56200}{1.35} = 41600$ kg.
Section reg'd $= 41600 \div 1200 = 34.7$ cm² net
use 2Ls 125 × 90 × 10 = 41.0 − 4.4 = 36.6 cm² net

Transverse direction
top of slab. moment carried by reinforcing bars
bott " " " " " steel plate.

Structural Steel in cutting Edge and top of Slab.

Top section	2Ls 130 × 130 × 9	=	45.18	×	− 3.51	=	− 159.0
	1L 130 × 130 × 9	=	22.59	×	+ 3.51	=	79.5
	1Pl. 630 × 9	=	56.70	×	+ 18.50	=	1050.0
			124.47		9.05 cm		1129.5
bottom section	2Ls 130 × 130 × 10	=	38.0	×	~~2.81~~ 2.90	=	110.0
	1Pl 550 × 9	=	49.50		2.80 say	=	1387.0
	1Pl. 500 × 9	=	45.00		2.59	=	1165.0
	1Pl. 220 × 9	=	19.80		0.45	=	8.9
			152.30		17.5		2670.9

9.05
17.5
26.55

Effective depth 3.43 − .26 = 3.16 meters
Load on cutting Edge. .87 × 2.0 × 2400 = 4180
Slab − 3.25 × 1.5 × 2400 = 11700
15880 kg per lin. meter

Unsupported length assumed 16.0 meters
moment say $\frac{1}{8} \times 15880 \times 16.0^2 = 509000$ kgm
Flange stress $= \frac{509000}{3.16} = 16100$ kg
Section reqd $= 16100 \div 1200 = 134.0$ □cm net

Unit stresses of steel can be raised somewhat as it is temporary frame to prevent cracking of concrete during execution.

Top Reinforcement in the top Slab. $m = 26350$ kgm
Section reg'd $= \frac{26350}{\frac{7}{8} \times 1.4 \times 1200} = \cancel{15.65}\ 17.85$ cm²
spacing of 19 mm bars $\frac{2.83 \times 100}{\cancel{15.65}\ 17.85} = \cancel{48.10}\ 15.90$ cm use 15 cm spacing

Horizontal Reinforcement in side wall of working chamber
$m = \frac{1}{10} \times 9400 \times 2.25^2 = 4760$ kgm
section reg'd $= \frac{4760}{\frac{7}{8} \times .55 \times 1200} = 7.2$ □cm
say
Spacing 19 mm bars $\frac{2.83 \times 100}{7.2} = 25.5$ cm near bottom

Vertical Reinforcement between steel frame use 25 cm bars − 35 cm spacing above.
Reinforcement in the base projection 1.0 meter p = assumed 42900 kg/m²
$m = 42900 \times \frac{1^2}{2} = 21450$ kgm
Effective depth of slab 1.3 meters
Steel area reg'd $= \frac{21450}{\frac{7}{8} \times 1.3 \times 1200} = 15.7$ □cm
25 mm bars $\frac{4.91 \times 100}{15.7} = 31.3$ cm

Downward pressure
Earth 1700 × 28.0 = 47500
2400 × 1.5 = 3600
51100 kg
min pressure upward during Earth quake − 24400
26700 $m = 13350$ kgm
Steel area $= \frac{1335000}{\frac{7}{8} \times 1.30 \times 1200 \times 1.8} = 5.44$ cm² use 19 mm bars same spacing with bottom reinforcement

11

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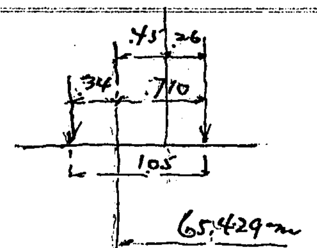
CALCULATIONS FOR

Design of Jiusō - Bashi for Osaka-Prefecture.

Design of piers P6 and P11

truss span   Dead Load on pier   424500 × 2 = 849000
Live Load on pier   213000 × 2 = 426000
637500   1.275.000 kg.



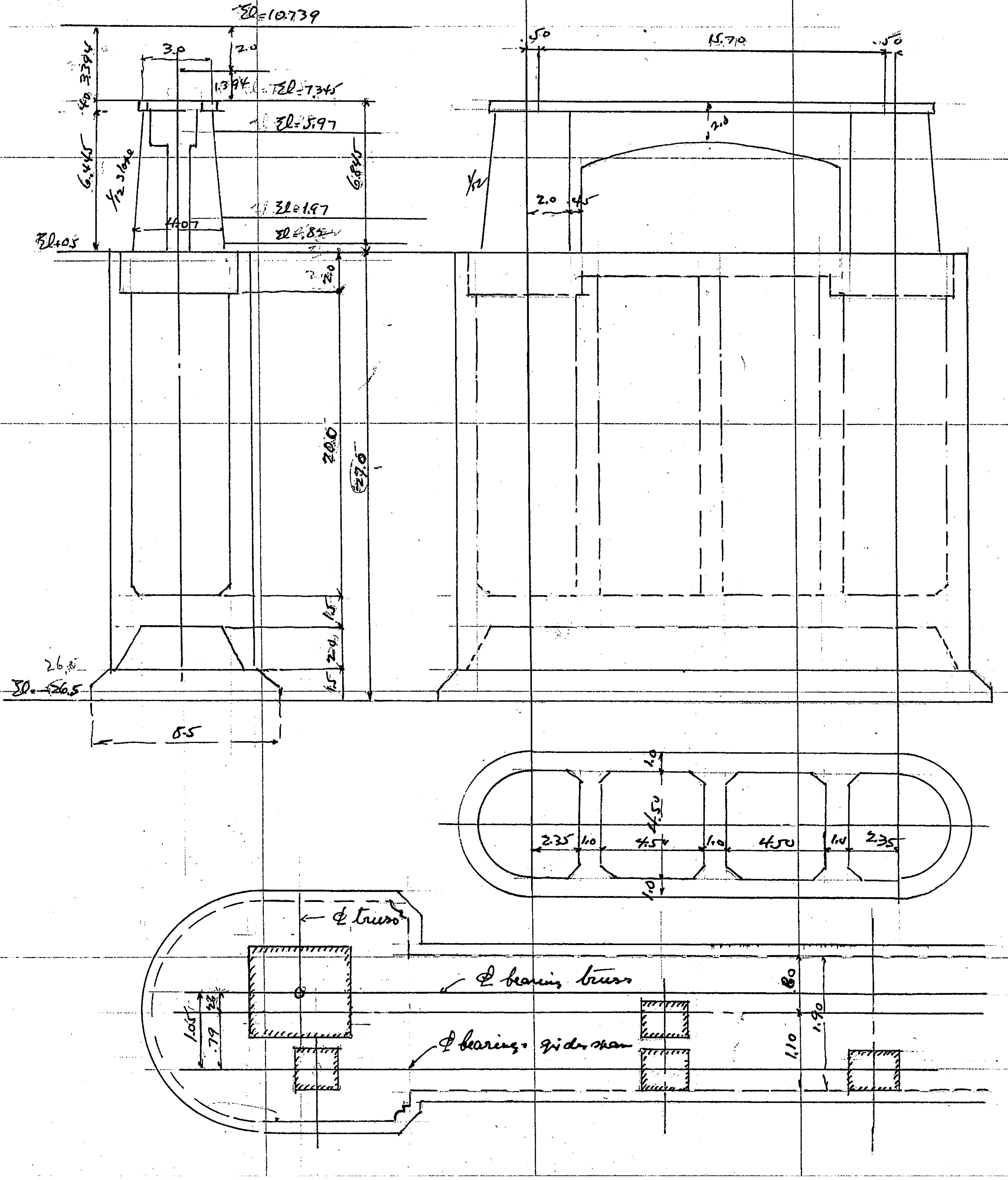
Girder span
Dead Load floor   15300
structural steel   6900
22200 × $\frac{24.909}{2}$ = 276000

Live Load say same as truss span
2 × 6500 × $\frac{24.909}{2}$ = 162.000
438.000

℄ gravity Dead Load
849000 × 0
276000 × 1.05 = 290.000
1125000 × .26 = 290.000

℄ gravity Dead and Live loads
1275.000
438.000 × 1.05 = 460.000
1713.000   .27   460.000

assume eccentricity .45 meter.



Design of Jiusō Bashi for Osaka- Prefecture

Volume of Concrete in pier
Coping. 1/2 3.4^{ϕ} = 4.54
3.4 × 2.4 = 8.15
12.69 × 2 × .40 = 10.1
between shafts say 11.4
21.5 cubic meters

Shaft 1/2 3.0^{ϕ} = 3.53 1/2 4.07^{ϕ} - 6.50
3 × 2 = 6.00 4.07 × 2 = 8.14
9.53 14.64
14.64
24.17 ÷ 2 = 12.09 @ 6.445 = 77.8 cubic meters

Strut between shafts 1.9 × 2.0^{abt} × 11.8 = 44.9
Pol. 2 - .45 × 1.9 × 6.45 = 11.0
web. 11.8 × 1.0 × 4.05^{abt} = 47.7
103.6 cubic meters

Summary Coping - 21.5
2 shafts 2 × 77.8 = 155.6 Center of gravity say 3.4 about from top
strut & web. 103.6
280.7 @ 2400 = 684.000 kg.

Concrete Caisson Outside area 141.7 sq meters
space between walls at middle 39.06 75.44
Both ends 36.38 66.26 square meters
75.44

Volume of Concrete in top Slab. 244.34 cubic meters

Summary of Concrete
Top slab 2 × 141.7 - 39.06 = 244.34 @ 2400 = 587 000
well shell 66.26 × 20.0 = 1328.00 @ 2400 = 3190 000
bottom fill and shell 141.7 × 3.5 = 495.00 @ 2200 = 1090.000
base say 198.75 × 1.5 = 298.00 @ 2200 = 655.000
5.522,000 kg.

Summary for water fill
both ends 36.38 × 20.0 = 727.6 @ 1000 = 727.600
middle 39.06 × 21.0 = 820.0 @ " = 820.000
1547.600 kg.

Total weight of pier
Shaft and coping 684.000 ✓
Caisson 5522.000
filling 1547.600
7753.600
Superimposed dead & Live Loads 1713.000
9466600 kg.

Base area - 198.75 Unit pressure = $\frac{9466600}{198.75}$ = 47700 kg/m2
4.44 tons/□'

assuming skin friction 250 #/□' = 1400 kg per sq meters
depth of caisson below firm ground 24.5 meters
Total load 9466.600
Skin friction 53.8 × 1400 × 24.5 = 1845.000
7621.600

Unit pressure 7621600.0 ÷ 198.75 = 38400 kg/m2
3.58 tons/□'

Design of Jiuso-Bashi for Osaka Prefecture

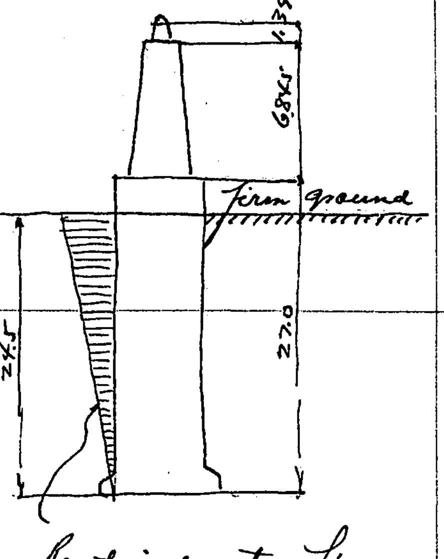
Assuming skin friction 350 #/□' or 1710 kg/m²

Total load 9466600
Skin friction 53.8 × 1710 × 24.5 = 2255000
7211600 kg.

Unit Pressure 7211600 ÷ 198.75 = 36400 kg/m²
3.39 tons/□'

Stability of Pier during Earthquake
Horizontal force k = 0.3
Superimposed Dead Load 1125.000 kg.

			Hor. Force	arm		
Dead Load superstructure	1125.000	× 0.3 =	337500	× 35.24	=	11900.000
Shaft and coping	684.000		205200	31.45	=	6450.000
top slab	587.000		176100	26.08	=	4595.000
well shell and partitions	3190.000		955000	15.00	=	14340.000
bottom working chamber	1090.000		327000	3.25	=	1062.000
base	655.000		196500	0.75	=	147.000
water fill both ends	727600		218500	15.00	=	3280.000
" " middle	820.000		246000	15.50	=	3810.000
	8873600		2661800			45584.000



Reaction due to Hor. Force assumed thus

moment due to hor. reaction 2661800 × 2/3 × 24.5 = 43400.000
2184.000

Counting ½ skin friction effective to ballance for overturning
skin friction 350 #/□' = 1710 kg/m²

53.8 × 1710 × 24.5 = 2255000

Resisting couple $\frac{1127500}{2}$ × 5.4 = 6090.000 kgm

required 2184.000 kgm to ballance for overturning.

Load on base 8873600
Less friction say − 1500000
7373600

Unit pressure $\frac{7373600}{19875}$ = 37200 kg/m²
3.45 tons/□'

Reinforcement in the shaft during Earthquake

Dead Load superstructure 1125 000 ÷ 2 = 562500
shaft with strut web neglected say 280.000 280.000
842500 kg.

load
562500 × 0.3 = 168750 × 8.24 = 1.390.000
280.000 84000 × 3.5 say = 294.000
1.684.000 kgm

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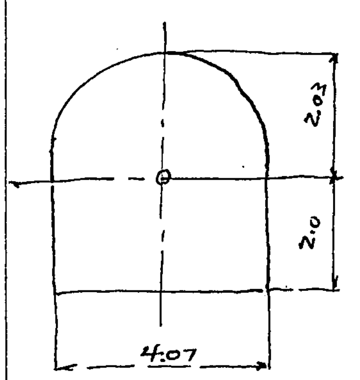
CALCULATIONS FOR

Design of Jiuso-Bashi for Osaka-Prefecture



moment of inertia of bottom section

circular End $I = \frac{0.0491}{2} \cdot 4.07^4 = 67.0$

Equivalent square $b = \frac{67.0 \times 12}{4.07^3} = 1.19$

2.00

3.19 meter wide

Reinforcing bars $25^{\text{cm dia}}$ $2 \times 15 @ 4.91 = 147.0^{\text{cm}}$

concrete area $3.19 \times 4.07 = 1.30$.. steel $\% = \frac{147}{13000} = .113\ \%$.

$\frac{d'}{h} = \frac{5}{407} = 0.012$ $e = \frac{1684.000}{842500} = 2.0$ $e/h = \frac{2.0}{4.07} = .492$

value of $k = .34$ coef $= .0772$

stress in concrete $f_c = \frac{1684.00000}{319 \times 407^2 \times 0.0772} = 41.3\ ^{kg}/_{cm^2}$

Steel Stress $f_s = 15 \times 41.3 \left(\frac{402}{407 \times .34} - 1 \right) = 1180\ ^{kg}/_{cm^2}$

Pressure in working chamber

Total weight of well

well shell with partitions		66.26 ×	20.0	@ 2400	=	3.180.000
Top well shell only	say	50.0 ×	2.0	@ 2400	=	240.000
Ceiling of working chamber		141.7 ×	1.5	@ 2400	=	510.000
wall of " "	say	66.26^{abt} ×	2.0	@ 2400	=	318.000
						3248.000 kg.

water fill say 75.44 × 20.5 × 1000 = 1.550.000 kg.

Skin friction assumed

350 #/□'	or 1710 kg/m²	× 538	× 24.5	=	2260.000 kg.
400	1960				2585.000
450	2200				2900.000

Theoritical water pressure depth of water 28.5 p = 28500 kg/m² [illegible] #/□"

upward pressure = 141.7 × 28500 = 4040.000 kg.

	350 #/□'	400	450
Downward load	4248.000	4248 000	4248.000
Skin friction	− 2260.000	2585 000	2900.000
net load at bottom	1988 000	1663 000	1348.000
water fill	1550 000	1550 000	1550 000
	3538 000	3213 000	2898.000

$\frac{3538}{4040} = 87.5\ \%$ $\frac{3213}{4040} = 79.5\ \%$ $\frac{2898}{4040} = 71.5\ \%$

Pressure in working chamber

$2.85 \times .875 = 2.50\ ^{kg}/_{cm^2}$ $2.85 \times .795 = 2.26\ ^{kg}/_{cm^2}$ $2.85 \times .715 = 2.04\ ^{kg}/_{cm^2}$

35.51 #/□" 32.11 #/□" 29.0 #/□"

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SEIYU BLDG, TOKIO

MADE BY K. Inaba DATE 4-7-7 **FILE NO**

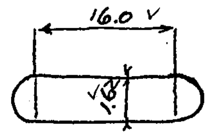
CHECKED BY M. Jinda DATE 4-7-10 **PAGE NO** A71

CALCULATIONS FOR

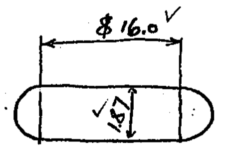
Materials for Juiso Bashi Osaka Prefecture.

Materials for P1 and P16
Concrete 1:2:4 mixture.

Coping Rectangle 1.62 × 0.4 × 16.0 = 10.38
Circular ends. 1.62$^{\phi}$ × 0.4 = .82
11.20 cub. meter



Shaft. Rectangle 1.87 × 16.0 × 5.56 = 166.42
Circular ends. 1.87$^{\phi}$ × 5.56 = 15.28
181.70 cub meter



Base. Area. 1.8 × 2.48 × 2 = 8.92
2.24 × 2.3 = 5.15
14.07 × 19.8 = 278.50 cub. meter

Total concrete for one pier = 471.40 cub. meter.

Rubbles for foundation
7.2 × 19.8 × 0.5 = 71.3 cub meter

.4+.06=.46m

Forms Coping Straight 0.46 × 16.0 × 2 = 14.71
Circular ends. 1.62$^{\phi}$ × .46 = 2.34
17.05 m²

Shaft. Straight 16.0 × 5.56 × 2 = 177.90
Circular ends. 1.87$^{\phi}$ × 5.56 = 32.65
210.55 m²

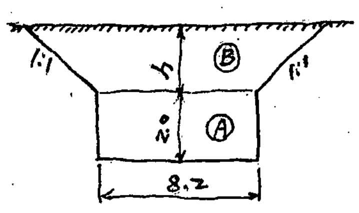
Base. Front + rear 1.3 × 19.8 × 2 = 51.43
Both ends. 1.3 × 7.2 × 2 = 18.72
" 2.48 × 0.5 × 2 = 2.48
" 2.24 × 1.0 = 2.24
74.87 m²

Total form for one pier = 302.47 sq. meter.

Reinforcements ~~Plain~~ Deformed Bars, See drawing, = 6.982 kg tons.

Foundation piles 末お 末12 10" 長 50'-0" -- 176 本.

Excavation Volume A = 8.2 × 20.8 × 2.0 = 341 m³
Pier no. P1. 341.0 + 11.0 × 23.6 × 2.8 = 1068.
" P16 341.0 + 11.2 × 23.8 × 3.0 = 1141.
2209. m³



Average excavation for one pier 2209. ÷ 2 = 1105 m³

Materials for Piers P2, P3, P4, P5, P12, P13, P14, & P15
Concrete 1:2:4 mixture.
Coping Same as for P1. and P16 = 11.20 cub. meter.
Shaft.

Pier nos.	mean width × length	Rectangle	Circular ends	Mean area	Height	Volume of Shaft.
P2 + P15	1.761 m × 16.0 =	28.18	1.761$^{\phi}$ = 2.44	30.62 m² ×	3.910 =	119.7 cub. meter
P3 + P14	1.777 × " =	28.41	1.777$^{\phi}$ = 2.48	30.89 ×	4.152 =	128.2 "
P4 + P13	1.791 × " =	28.65	1.791$^{\phi}$ = 2.52	31.17 ×	4.365 =	136.0 "
P5 + P12	1.804 × " =	28.85	1.804$^{\phi}$ = 2.55	31.40 ×	4.550 =	142.8 "

Base. upper layer

Pier nos.	mean width	height	length	volume.
P2 + P15	3.01 ×	1.5 ×	19.0 =	85.8 cub. meter
P3 + P14	3.027 ×	1.5 ×	19.0 =	86.2 "
P4 + P13	3.041 ×	1.5 ×	19.0 =	86.6 "
P5 + P12	3.053 ×	1.5 ×	19.0 =	87.0 "

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CHECKED BY *M. Jinda* DATE *4-7-10* **PAGE NO** *A72*

CALCULATIONS FOR

Materials of Jinso Bashi for Osaka Prefecture.

Base concrete. area. 1.8 × 2.5 × 2 = 9.0
2.3 × 4.0 = 9.2
18.2 × 19.8 = 360.4 cub. meter.

Summary of concrete for one pier.

Pier no.s	Coping	Shaft	Base	Base.	Total conc. for one pier
P2 + P15	11.2 +	119.7 +	85.8 +	360.4 =	577.1 cub. meter
P3 + P14	11.2 +	128.2 +	86.2 +	360.4 =	586.0 〃
P4 + P13	11.2 +	136.0 +	86.6 +	360.4 =	594.2 〃
P5 + P12	11.2 +	142.8 +	87.0 +	360.4 =	601.4 〃

Forms.

Coping same as for Piers P1 + P16 = 17.05 □m

Shaft.	Circular ends.	straight potion	total area	Height	Area
Pier nos.	1.761φ = 5.53	16 × 2 = 32.0	37.53 ×	3.91 =	146.7 □m
P2 + P15	1.777φ = 5.58	32.0	37.58 ×	4.152 =	156.0 〃
P3 + P14	1.791φ = 5.62	32.0	37.62 ×	4.365 =	164.3
P4 + P13	1.804 = 5.67	32.0	37.67 ×	4.550 =	171.5 〃
P5 + P12					

Base, upper layer

Pier nos.	front and rear	Bothsides	Total area.
P2 + P15	1.80 × 19 × 2 = 68.4	3.01 × 1.5 × 2 = 9.03	77.4 □m
P3 + P14	1.79 × 19 × 2 = 68.0	3.027 × 1.5 × 2 = 9.08	77.1 〃
P4 + P13	1.78 × 19 × 2 = 67.6	3.041 × 1.5 × 2 = 9.12	76.7 〃
P5 + P12	1.77 × 19 × 2 = 67.2	3.053 × 1.5 × 2 = 9.16	76.4 〃

Base. lower layer.

front and rear = 1.3 × 19.8 × 2 = 51.5
both ends = 6.5 × 1.0 × 2 = 13.0
〃 = 9.0 × 1.3 × 2 = 23.4
87.9 □m.

Summary of Forms for one pier.

Pier nos.	Coping	Shaft.	Base	Base.	Total □m
P2 + P15	17.1	146.7	77.4	87.9	329.1
P3 + P14	17.1	156.0	77.1	87.9	338.1
P4 + P13	17.1	164.3	76.7	87.9	346.0
P5 + P12	17.1	171.5	76.4	87.9	352.9

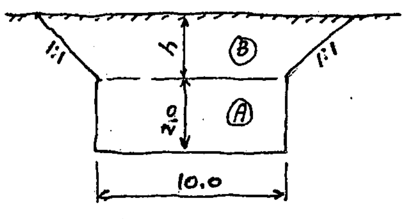
Reinforcements, ~~Plain~~ Deformed Bars. See drawing.

Pier P2 + P15 -- 7.996 kg. tons. for one pier
〃 P3 + P14 -- 8.106
〃 P4 + P13 -- 8.276
〃 P5 + P12 -- 8.350

Foundation piles 末松末口 10" 長 50尺 — 220 本

Rubbles for foundation
9.0 × 19.8 × 0.5 = 89.1 cub. meter

Excavation.



底部四方0.5m宛余裕視込ム

Volume A. 10.0 × 20.8 × 2.0 = 416.0 m³

Pier no.	Volume A	Volume B.			h.	根入
P2 + P15	416.0 +	12.9 × 23.7 × 2.9 =	1302.	Cub. meters	2.9m	4.9m
P3 + P14	416.0 +	13.0 × 23.8 × 3.0 =	1347	〃	3.0	5.0
P4 + P13	416.0 +	13.2 × 24.0 × 3.2 =	1,43.0	〃	3.2	5.2
P5 + P12	416.0 +	=	1430	〃	3.2	5.2

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MADE BY K. Inaba DATE 4-7-22 FILE NO

CHECKED BY M. Jinda DATE 4-7-26 PAGE NO A73

CALCULATIONS FOR

Materials of Jiuso Bashi for Osaka prefecture

Abutment for Canal Span. (A1 and A2)

Floor

Concrete 1:2:4 mixture.

Sectional area.

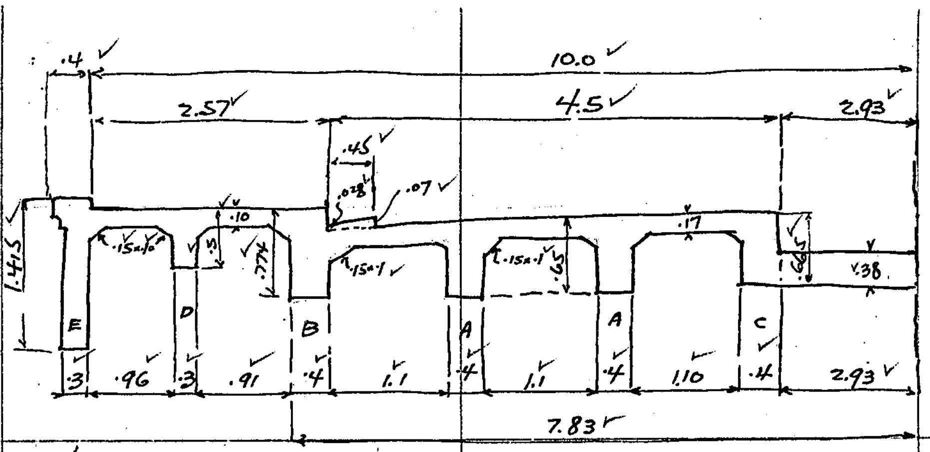
Electric Ry. slab	2.93 × .38 =	1.114
Roadway slab	4.5 × .17 =	.765
Gutter	.049 × .45 =	.022
Sidewalk slab	2.57 × .10 =	.257
Stem of Beam C.	.4 × .495 =	.197
〃 〃 A	.4 × .48 × 2 =	.384
〃 〃 B.	.4 × .674 =	.270
〃 〃 D	.3 × .40 =	.120
〃 〃 E	.3 × say 1.80 =	.540
Coping 〃 E	.22 × .07 =	.015
Fillets of Beams.	.1 × .15 ÷ 2 × 10 =	.075

3.759 × 2 = 7.518 $^{\square m}$ × 4.10 ^{m} = 30.82 cub. meter

Fillets at both ends of slab + beams.	$\frac{.4 \times .4}{2}$ × 2 × 7.83 =	1.252
	$\frac{.3 \times .3}{2}$ × 2 × 1.87 =	.168
	$\frac{.4 \times .4}{2}$ × 2 × .3 =	.048

1.468 × 2 = 2.94 cub. meter

Total concrete for floor = 33.76



Concrete for abutment structure. 1:2:4 mixture.

parapet wall. Elect. Ry.	.45 × 2.425 × 5.86 =	6.40
〃 〃 Roadway	.45 × 2.68 × 4.5 × 2 =	10.85
〃 〃 Sidewalk.	.45 × 2.80 × 2.57 × 2 =	6.48
front wall between columns.	1.2 × 3.49 × 20.0 =	83.76
Coping	.1 × .4 × 20.0 =	.80
Columns.	1.2 × .9 × 6.41 × 2 =	13.85
〃 under fascia beams	.3 × .2 × ~~3.49~~ 3.15 × 2 =	.38
Rear wall. Elect. Ry.	.7 × 5.92 × 5.86 =	24.29
〃 〃 Roadway	.7 × 6.17 × 4.5 × 2 =	38.87
〃 〃 Sidewalk.	.7 × 6.29 × 2.57 × 2 =	22.62
Columns.	1.5 × 1.5 × 6.41 × 2 =	28.85
〃 under fascia beams.	.3 × .2 × 3.15 × 2 =	.38
projection, Roadway, top.	.25 × .8 × 4.5 × 2 =	1.80
〃 Sidewalk 〃 slab.	.1 × .82 × 2.57 × 2 =	.42
〃 〃 beam.	.4 × ~~.9~~ .55 × .65 × 2 =	.29
〃 〃 〃	.3 × .35 × .65 × 2 =	.14
〃 〃 〃	.3 × .3 × 2.17 × 2 =	.39
Base.	1.25 × 7.2 × 23.5 =	211.50
Fillets of base	$\frac{.5 \times .35}{2}$ × 3 × 23.5 =	6.17

458.24 cub meter

Concrete for Lower Base. 1:2:4

wall.	.6 × 1.6 = .96 $^{\square m}$
〃	1.8 × 1.6 = 2.88
base	3.15 × .5 = 1.575
〃	4.5 × .9 = 4.05

9.465 × 23.5 = 222.50 = 222.50 cub. meter.

Total concrete for abutment A2 = 714.50 〃

Extra concrete under girder bearing for A1 only
.19 × .75 × 20.0 2.85 $^{m^3}$

Total concrete for abutment A1 = 717.35 cub. meter

Average = 715.93 cub meter.

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CHECKED BY M Jinda DATE 4-7-26 PAGE NO A74

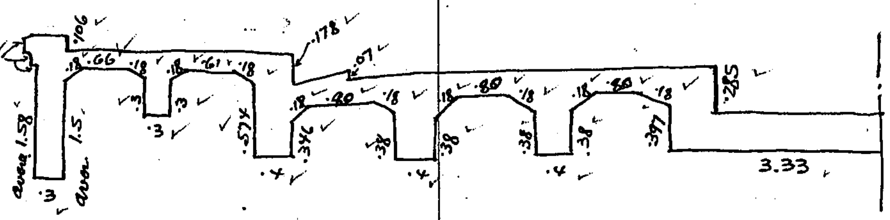
CALCULATIONS FOR

Materials of Juiso Bashi for Osaka Prefecture

Forms for Abutment

Total width of Form for floor.

bottom.		3.330
〃	3×.4	= 1.200
〃	2×.3	= 0.600
〃	3×.8	= 2.400
〃		0.610
〃		0.660
Fillets	10×.18	= 1.800
Coping		0.320
〃		0.106
Curb		0.178
gutter		0.070
between Roadway + Elec. Ry.		0.285
Stem of beams.		0.397
〃	4×.38	= 1.520
〃		0.346
〃		0.574
〃	2×.3	= 0.600
〃 faxcia beam		= 1.50
〃 〃		= 1.58
		18.076 m



Between walls.	18.076 × 4.1	=	74.1	
Extra length for fillets, say	2×.165 × ~~18.076~~ 10.00	=	3.3	
Outside of faces of wall.	.639 × 0.83	=	0.53	(285+07+.178+.106 = .639) 0.5+0.33 = 0.83
Both ends of slab.	.3 × 7.07	=	2.1	
〃	.2 × 7.07	=	1.4	
Total form for floor		=	81.43×2 = 162.9 sq.m.	

Form for Abutment structure.

Parapet wall.	front.	Elec. Ry.	2.425 × 5.86	=	14.22
〃	〃	Roadway.	~~2.68~~ 〃 × 4.5×2	=	21.82
〃	〃	Sidewalk.	~~2.80~~ 〃 × 2.57×2	=	12.47
〃	rear.	elec. Ry.	1.645 × 5.86	=	9.64
〃	〃	Roadway	say. 2.0 × 4.5×2	=	18.00
〃	〃	Sidewalk	〃 2.3 × 2.57×2	=	11.82
Front wall.	front		~~4.79~~ 3.14 × 20.0	=	62.80
〃	coping bott.		.1 × 20.0	=	2.00
〃	rear.		3.14 × 20.0	=	62.80
〃	fillet bott.		0.61 × 23.5	=	14.34
Column	front.		0.9×2× ~~7.66~~ 6.06	=	10.91
〃	outside		1.2×2× ~~7.66~~ 6.06	=	14.55
〃	inside		.75×2×2.92	=	4.38
〃	rear		.60×2×2.92	=	3.50
〃	rear.		.90×2×3.14	=	5.65
〃			.2×2×2×3.14	=	2.51
Rear wall.	front		4.79 × 20.0	=	95.80
〃	~~rear~~		0.4×2×6.67	=	5.31
〃	rear.		.557 × 20.0	=	111.40
Fillets bott.	front		0.61 × ~~#~~ × 23.5	=	14.33
〃	rear		0.61 × 20.0	=	12.20
Column.	outside + rear		1.5×2×2×6.40	=	38.40
〃	inside		0.7×2×6.40	=	8.96
〃	front.		say 1.9×2×6.40	=	24.30
Sidewalk Cantilever		slab.	0.75×2×2.75	=	4.13
〃		beam	0.55×2×2×.65	=	1.43
〃		〃	.35×2×2×.65	=	0.91
〃	edge.	〃	.3×3 2.0×2	=	3.60
Base	Side + rear		37.9×1.25	=	47.38
〃	sides.		.35 × 1.45×2	=	1.02
					640.58 sq. 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 K. Inaba DATE 4-7-22 FILE NO

CHECKED BY M. Jinda DATE 4-7-26 PAGE NO A75

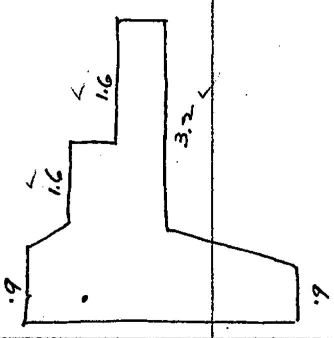
CALCULATIONS FOR

5

Materials of Jiuso Bashi for Osaka Prefecture

Forms for Lower Base of Abutment · A1 + A2.

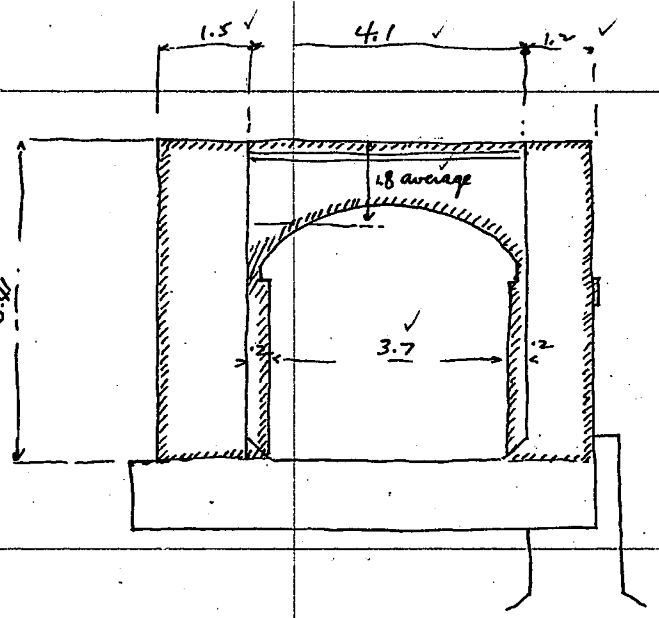
front side	3.2 + .9 =	4.1 × 23.5 =	96.4
rear "	1.6 + 1.6 + .9 =	4.1 × 23.5 =	96.4
Both ends.		.6 × 1.6 × 2 =	1.92
"		1.8 × 1.6 × 2 =	5.76
"		3.15 × .5 × 2 =	3.15
"		0.9 × 4.5 × 2 =	8.10
			211.73 sq.m

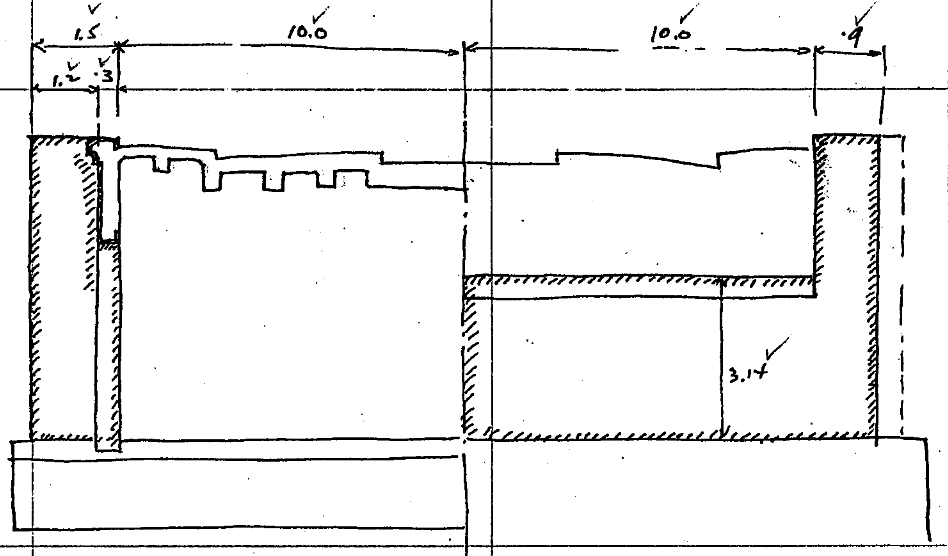


Total form for one abutment

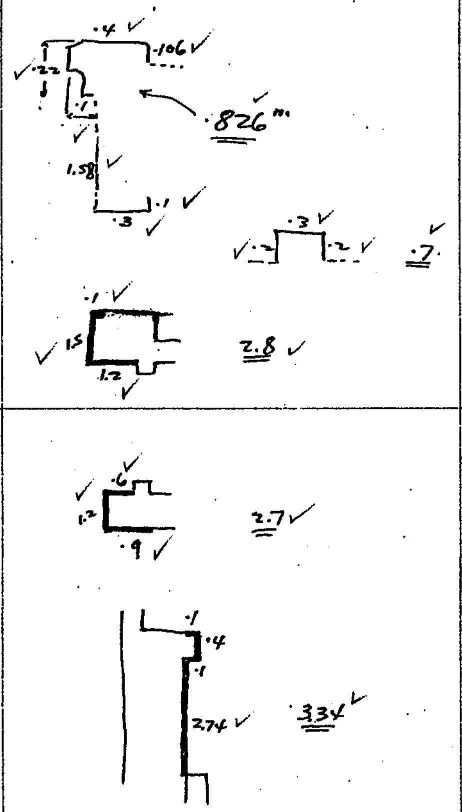
Forms for floor.	=	162.9
" " Structure.	=	640.6
Lower base		211.7
		1015.2 sq. meter.

人造洗出仕上.





Coping	.826 × 4.10 × 2	=		6.77
less handrail bott.	.2 × 4.10 × 2	=	(−)	1.64
Fascia beam outside	1.58 × 4.1 × 2	=		12.95
" bott. + inside	.4 × 5.4 × 2	=		4.32
projection under fas. beam	.7 × 3.2 × 4	=		8.96
Rear Column top.	1.5 × 1.5 × 2	=		4.50
less pedestal area	1.2 × 1.2 × 2	=	(−)	2.88
Rear column	2.8 × 6.41 × 2	=		35.90
" curl	.115 × 1.5 × 2	=		.35
Front column top.	1.2 × 0.9 × 2	=		2.16
less post area	1.0 × .7 × 2	=	(−)	1.40
Front column.	2.7 × 6.41 × 2	=		34.60
" inside	.75 × 2.92 × 2	=		4.38
" curl	.45 × .115 × 2.	=		.10
Front wall. front side	3.34 × 20.0	=		66.80
				175.87 sq. meter.



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CALCULATIONS FOR

6)

Materials of Jiuso Bashi for Osaka prefecture.

Reinforcing Bars. Deformed Bars. Abutment A1 38.627 ✓ kg tons see drawings.
A2 38.163 ✓ 〃
average 38.395 ✓ kg tons.

Pavements. Roadway. Asphalt block pavement 5 cm thick with 1 cm cement mortar cushion (1:2)
width 4.405 × .45 = 3.955 m
length 4.10 + .5 + .33 = 4.93 m
paved area = 3.955 × 4.93 × 2 = 39.00 sq. meter for one abutment.

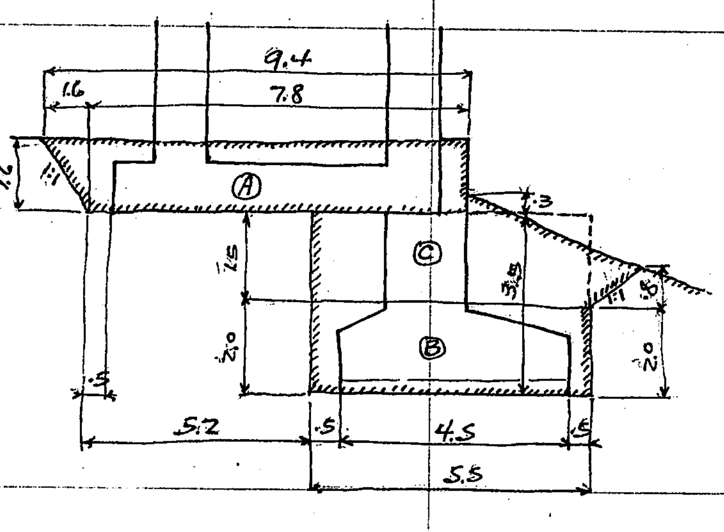
Sidewalk. Cement mortar finishing 2.5 cm thick 1:2 mixture.
paved area = 2.57 × 5.75 × 2 = 29.56 sq. meter for one abutment.

Granite Curb stone. 横 [illegible]
歩車道
縁 2 × .18 × .17 × .75 = 0.0458
縦 6 × .18 × .17 × 1.72 = 0.3158
0.3616
踏掛石 10 × .25 × .2 × .87 = 0.4350 ✓
0.7966 cub meter. for one abutment.
軌車道境界石 6 × .18 × .17 × 1.72 = 0.3158 ✓
1.1124 cub meter for one abutment

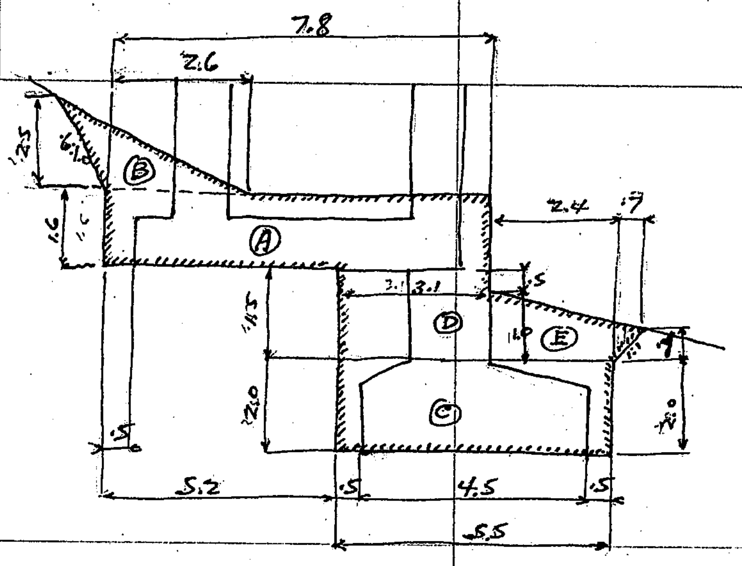
Foundation Piles 米松 末口 9吋 長 40呎
Upper base. 5 × 26 = 130 本 ✓
Lower 〃 5 × 26 = 130 ✓
260 ✓ for one abutment.
割栗石. Lower base. 4.5 × 23.5 × 0.5 = 52.8 cub meter for one abutment.

Excavation for Abutment A1 and A2.

Abutment A1.



Abutment A2.



Volume A 1.6 × 8.6 × 26.1 = 359
〃 B 5.5 × 2.0 × 24.5 = 270
〃 C say 1.5 × 5.5 × 25.3 = 209
838 m³

Volume A 7.8 × 1.6 × 24.5 = 306
B 2.6 × 2.5 ÷ 2 × 25.3 = 82
C 5.5 × 2.0 × 24.5 = 270
D 3.1 × 1.5 × 24.5 = 114
E say 0.9 × 2.4 × 25.4 = 55
827 m³

average $\frac{838+827}{2}$ = 832.5 m³

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CHECKED BY DATE PAGE NO A77

CALCULATIONS FOR

Materials of Juiso-Bashi for Osaka Prefecture.

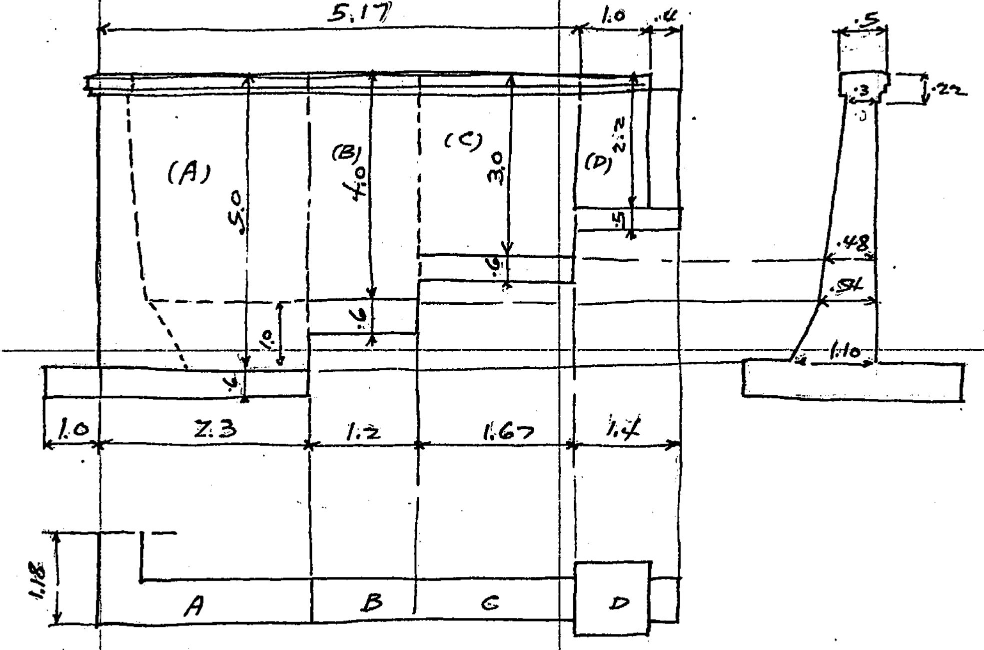
Approach Retaining walls neighbouring to abatment Az.
1. Concrete 1:2:4 mixture.

wall A.

Coping .5 × .22 = .11
wall .42 × 3.78 = 1.59
" .82 × 1.0 = .82
2.52 × 2.85 = 7.14

base 3.6 × .6 × 1.9 = 4.11
" 1.4 × .6 × 2.18 = 1.83
5.94

13.08 $^{m^3}$



wall B.

Coping .5 × .22 = .11
wall .42 × 3.78 = 1.59
base .6 × 3.2 = 1.92
3.62 × 1.2 = 4.34

wall C.

Coping .5 × .22 = .11
wall .39 × 2.78 = 1.09
base .6 × 2.5 = 1.50
2.70 × 1.67 = 4.51

wall D.

Column 1.0 × 1.0 × 2.2 = 2.20
wall .4 × .36 × 2.05 = 0.30
base .5 × 2.5 × 1.4 = 1.75
4.25

Summary 26.18 m^2 for one side

Total concrete for 2 walls = 2 × 26.18 = 52.36 $^{m^3}$

Reinforcements Deformed Bars.
1.439 kg tons for one wall. See drawings.
2.878 " for 2 walls.

Forms.

wall A. 5.1 × 3.48 = 17.75
" " 5.1 × 2.64 = 13.46
" B 4.1 × 2 × 1.2 = 9.84
" C 3.1 × 2 × 1.67 = 10.35
" D. 4.4 × 2.2 = 9.68
End of base A .6 × 10.98 = 6.59
" " 1.0 × .4 = .40
" B .6 × 7.8 = 4.68
" " 1.0 × .4 = .40
" C .6 × 7.8 = 4.68
" " .54 × .4 = .22
" D. .5 × 7.$\overset{3}{8}$2 = 3.66
" " .48 × .3 = .14
Total wall for one wall. 81.85 call this 81.9 m^2
for 2 walls. 163.8 "

Foundation piles ~~赤杉~~ 末口 18 ^{cm} 長 5.5 ^{m} long 内地産赤杉杭.
30 piles for one wall.
60 " " 2 walls.

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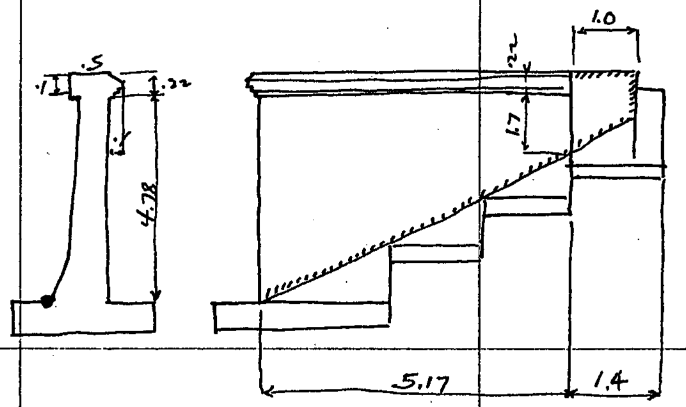
CHECKED BY ______ DATE ______ PAGE NO A78

CALCULATIONS FOR

Materials of Juisō-Bashi for Ōsaka prefecture.

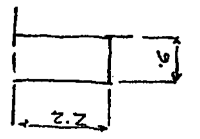
人造洗出仕上

Coping .92 × 6.35 = 5.84
less handrail .25 × 5.75 = (−) 1.44
wall 3.24 × 5.17 = 16.75
" 4.78 × 1.18 = 5.64
Column 1.56 × 1.0 = 1.56
" 1.9 × .35 = .66
" 1.0 × .15 = .15
" top. 1.0 × 1.0 = 1.00
" less .6 × .6 = (−) .36
Total finish for one wall = 29.80 m²
for 2 walls = 59.60 m²

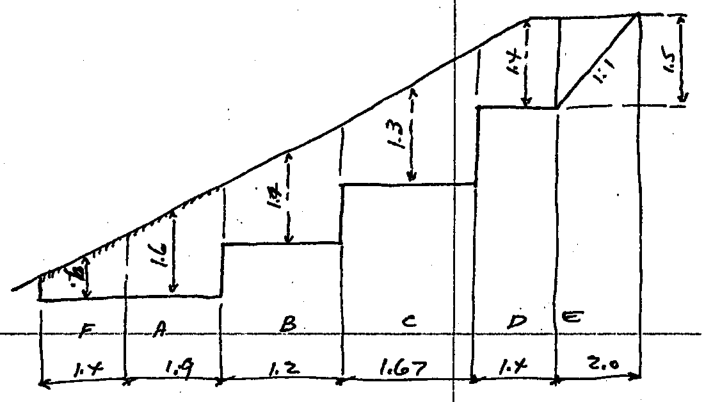


Excavation.
Cross-Sectional area.

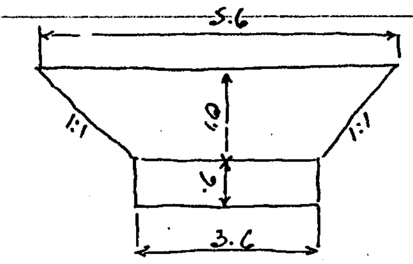
F



2.2 × 0.6 = 1.32 □m.

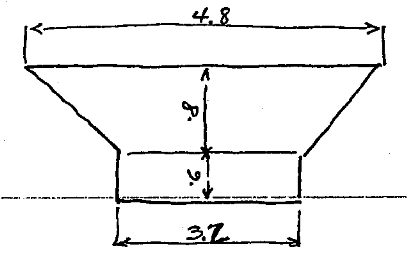


A.



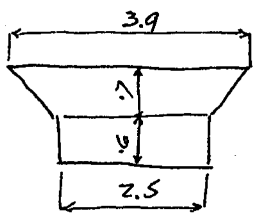
3.6 × .6 = 2.16
4.6 × 1.0 = 4.60
6.76 □m

B



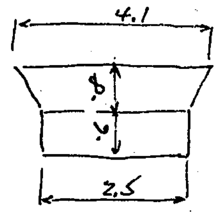
3.2 × .6 = 1.92
4.0 × .8 = 3.20
5.12 □m

C



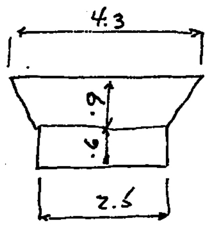
2.5 × .6 = 1.50
3.2 × .7 = 2.24
3.74 □m

D



2.5 × .6 = 1.50
3.3 × .8 = 2.64
4.14 □m

E



2.5 × .6 = 1.50
3.4 × .9 = 3.06
4.56 □m

Excavation.

F 1.32 × 1.4 = 1.85
A 6.76 × 1.9 = 12.85
B 5.12 × 1.2 = 6.14
C 3.74 × 1.67 = 6.24
D 4.14 × 1.4 = 5.80
E 4.56 × $\frac{2.0}{2}$ = 4.56
37.44 m³ for one wall
74.88 m³ for 2 walls.

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CALCULATIONS FOR

Materials of Jiuso-Bashi for Osaka prefecture

Abutment for River Spans. A3 and A4.

Concrete. 1:2:4 mixture. for A3.

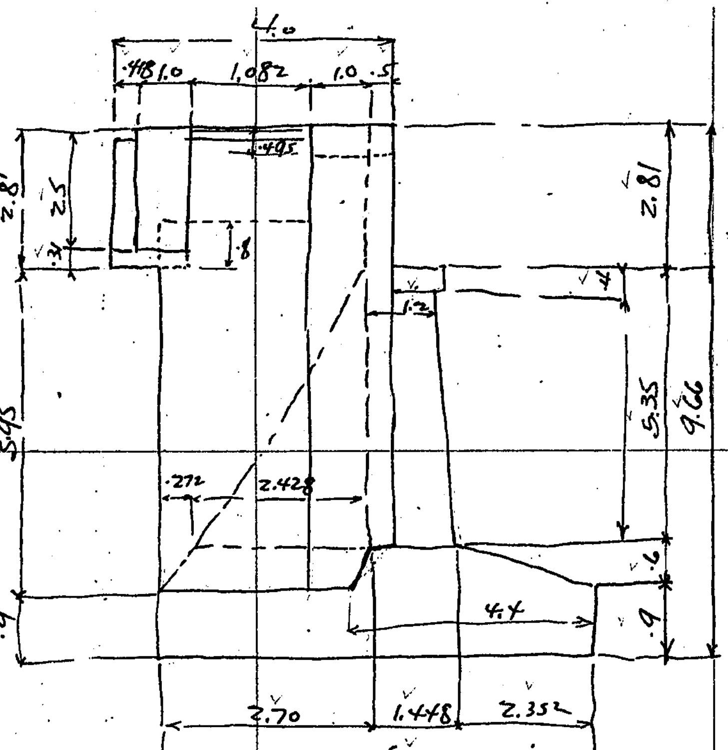
parapet wall.	0.5 × 2.315 × 20	= 23.15 ✓
" " top.	.35 × .111 × 4.405 × 2	= 0.34 ✓
" " "	.35 × .272 × 2.75 × 2	= 0.52 ✓
" "	0.5 × 2.81 × 2.1 × 2	= 5.90 ✓
Coping	0.4 × 1.3 × 25.6	= 13.32 ✓
front wall	1.324 × 4.95 × 25.4	= 166.50 ✓
Buttress A.	1.214 × 1.0 × 7 × 5.35	= 45.45 ✓
" " bottom	.6 × 1.0 × 2.264 × 7	= 9.51 ✓
" " fillets	.5 × .6 × 2.264 × 2	= 1.36 ✓
" " "	.3 × .6 × 2.264 × 5	= 2.04 ✓
Base	2.924 × .6 × 26.1	= 45.80 ✓
"	6.5 × .9 × 26.1	= 152.70 ✓
		466.59
Buttress B.	6.75 × .6 × (2.7 × 2	= 21.89 ✓
" "	1.0 × .6 × 1.515 × 2	= 1.82 ✓
" " fillet.	.3 × .6 × ~~2.264~~ 2.4 × 2	= .86 ✓
		24.57 ✓
Buttress C.	~~2.7~~ 3.2 × 5.95 × .6 × 2	= 22.85 ✓
" "	~~3.2~~ 2.81 × 4.0 × .6 × 2	13.49 ✓
Column projection	1.0 × .4 × 2.5 × 2	= 2.00 ✓
" "	1.5 × .2 × 8.76 × 2	= 5.25 ✓
Slab under pedestal.	1.0 × 2.1 × .495 × 2	= 2.07 ✓
Strut	.62 × .8 × 1.5 × 2	= 1.49 ✓
fillet bottom	.3 × .6 × 2.4 × 1	= .43 ✓
		47.58 ✓
	Total concrete for A3	= 538.74 m³ ✓

Concrete for A.4.

Parapet, front wall, base, Buttress A &c.		466.59 ✓
buttress B	2 × 24.57	= 49.14 ✓
" less fillets	.30 × .6 × 2.4 × 1	= (−) .43 ✓
parapet and	.5 × 2.81 × 1.537 × 2	= 4.32 ✓
" bottom	.5 × .937 × 1.25 × 2	= 1.17 ✓
wing wall.	.6 × 1.082 × 3.1 × 2	= 4.02 ✓
"	.6 × 1.418 × 2.81 × 2	= 4.78
Column projection	.4 × 1.0 × 2.5 × 2	= 2.00 ✓
" under pedestal.	.8 × 1.5 × 3.8 × 2	= 9.12 ✓
Slab " "	1.0 × .495 × 3.637 × 2	= 3.60 ✓
Strut	.62 × 1.20 × 3.817 × 2	= 5.68 ✓
" less	.6 × .8 × .62 × 4	= (−) 1.19 ✓
	Total Concrete for A4.	= 548.80 m³ ✓

Forms for abutment A3.

Parapet wall. front + rear	2.31 × 20.0 × 2	= 92.40 ✓
" top. front + rear	.101 × 4.405 × 4	= 1.78 ✓
" " "	.4 × 2.75 × 4	= 4.40 ✓
" front	2.9 × 2.81 × 2	= 16.30 ✓
" rear.	1.7 × 2.32 × 2	= 7.89 ✓
Coping	.5 × 27.2	= 13.60 ✓
front wall. front	4.95 × 25.4	= 125.70 ✓
" Side	1.824 × 4.95 × 2	= 18.16 ✓
" rear	5.35 × 16.0	= 85.60 ✓



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CALCULATIONS FOR

Materials of Jiusō-Bashi for Osaka prefecture.

Base front + rear	0.9 × 26.1 × 2	= 46.95 ✓	
〃 side	0.9 × 6.5 × 2	= 11.70 ✓	
〃 〃	0.6 × 2.921 × 2	= 3.51 ✓	
〃 rear fillet	0.85 × 16.0	= 13.60 ✓	
Buttress A. sides	1.214 × 5.35 × 2 × 7	= 90.80 ✓	
〃 〃 rear	6.55 × 1.0 × 7	= 45.85 ✓	
〃 〃 bott. fillet	0.78 × 2.264 × 4	= 7.06 ✓	
〃 〃 〃 〃	0.67 × 2.264 × 10	= 15.17 ✓	
		590.47 m² ✓	
Buttress B. sides	2.7 × 6.15 × 4	= 66.40 ✓	
〃 〃 rear	6.75 × .6 × 2	= 8.10 ✓	
〃 〃 bott. fillet	0.67 × 2.4 × 4	= 6.43 ✓	
〃 〃 top, side	1.0 × 1.515 × 4	= 6.06 ✓	
〃 〃 〃, rear	.6 × 1.515 × 2	= 1.82	
		88.81 m² ✓	
Buttress C. inside	2.7 × 8.16 × 2	= 44.10 ✓	
less slab area	.495 × 1.0 × 2	= (−) .99 ✓	
〃	.8 × 2.81 × 2	= 4.49 ✓	
〃 projection	.2 × 2.5 × 4	= 2.00 ✓	
〃 〃 bott.	.2 × 1.0 × 2	= .40 ✓	
〃 ~~Outside~~ fillet	.67 × 2.4 × 2	= 3.21 ✓	
〃 Outside	3.2 × 8.16 × 2	= 52.21 ✓	
〃 〃 front.	0.2 × 5.35 × 2	= 2.14 ✓	
〃 〃	.8 × 2.81 × 2	= 4.50 ✓	
〃 projection	.2 × 8.76 × 2	= 3.51 ✓	
〃 〃	.2 × 2.5 × 4	= 2.00 ✓	
〃 〃 bott.	.2 × 1.0 × 2	= .40 ✓	
〃 coping	.08 × 1.082 × 2	= .17 ✓	
〃 rear	8.76 × .6 × 2	= 10.51 ✓	
〃 〃 bott.	.8 × .6 × 2	= .96 ✓	
〃 slab. bott. 〃 〃 rear	1.7 × 1.0 × 2 .495 × 2.1 × 2	= 3.40 ✓ = 2.08 ✓	
〃 strut.	2.22 × 1.5 × 2	= 6.66 ✓	
〃 less strut area	.62 × .8 × 4	= (−) 1.99 ✓	
fillet rear face ◭ ◮	.5 × .6 × 2	= .60 ✓	
〃 〃	.3 × .6 × 8	= 1.44 ✓	
		141.80 m² ✓	
	Total forms for abutment A3 =	821.08 m² ✓	

Forms for Abutment A4.

parapet, front wall, buttress A, base &c		= 592.94 m² ✓	Same as for A3.
Buttress B.	~~Z0~~ inside buttresses.	= 88.81 ✓	
〃 〃 outside	$88.81 - \frac{6.43}{2} + .6 \times 2.4 \times 2$	= 88.47 ✓	± fillet outside.
wing front	2.81 × 1.537 × 2	= 8.63 ✓	
〃 〃	1.737 × 1.25 × 2	= 4.34 ✓	
〃 rear	.937 × 3.565 × 2	= 6.68 ✓	
〃 outside	4.00 × 2.81 × 2	= 22.50 ✓	
〃 〃	2.582 ~~1.29~~ × 0.625 × 2	= 3.23 ✓	
〃 inside	3.50 × 2.81 × 2	= 19.67 ✓	
〃 〃 〃 less slab area	2.082 × 0.625 × ~~2~~ 1.0 × .495 × 2	2.60 = (−) .99 ✓	
〃 projection	.2 × 3.45 × 2	= 1.38 ✓	
〃 〃	.2 × 2.5 × 8	= 4.00 ✓	
〃 〃 bott.	.2 × 1.0 × 4	= .80 ✓	
〃 rear	.6 × 2.81 × 2	= 3.37 ✓	
〃 bottom	.6 × 4.32 × 2	= 5.18 ✓	
〃 〃 (col.)	.2 × 1.5 × 2	= .60 ✓	
〃 coping	.08 × 1.082 × 2	= .17 ✓	

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CHECKED BY DATE PAGE NO A81

CALCULATIONS FOR

Materials of Jinso-Bashi for Osaka prefecture.

Slab under pedestal	1.7×1.0 × 2	=		3.40 ✓
〃 〃 〃	.495 × 3.64 × 2	=		3.61 ✓
〃 〃 〃	.937 × 1.0 × 2	=		1.87 ✓
Strutts	~~3.636~~ 3.02 × 2.64 × 2	=		15.95 ✓
〃	2.22 × 2 × .6 × 2	=		5.33 ✓
〃 end.	.4 × .618 × 2	=		.50 ✓
〃 less endarea	.618 × 1.2 × 2	=	(−)	1.48 ✓
〃 〃	.618 × .8 × 2	=	(−)	.99 ✓
fillet rear faces.	.5×.6× 2	=		.60 ✓
〃 〃	.3×.6× 8	=		1.44 ✓

Total forms for abutment A4 = 887.61 m² ✓

Reinforcements Deformed Bars.
Abutment A3. -- 20.493 ✓ kg.tons See drawings.
Abutment A4 -- 21.356 ✓ 〃 〃

人造洗出仕上.
Abutment A3.

parapet wall.	2.31 × 5.3 × 2	=		24.49 ✓
〃 〃	0.5 × 2.9 × 2	=		2.90 ✓
Wing	2.95 × 3.582 × 2	=		21.15 ✓
〃 col. front.	.2 × 1.14 × 2	=		.46 ✓
〃 〃	.2 × 3.1 × 2	=		1.24 ✓
〃 〃	.2 × 2.5 × 2	=		1.00 ✓
〃 〃	.2 × 2.0 × 2	=		.80 ✓
〃 coping	.08 × 1.082 × 2	=		.17 ✓
〃 capling	.10 × 6.88 × 2	=		1.38 ✓
〃 coping top.	1.5 × 2.9 × 2	=		8.70 ✓
〃 〃 〃	.68 × 1.08 × 2	=		1.47 ✓
〃 〃 〃	1.0 × 1.0 × 2	=		2.00 ✓
less handrail area.	1.2 × 1.2 × 2	=	(−)	2.88 ✓
〃 〃	.6 × .6 × 2	=	(−)	.72 ✓
〃 〃	.4 × ~~1.13~~ 2.98 × 2	=	(−)	~~1.13~~ 2.38 ✓

〃 Total area of Finish for A3 = 59.78 sq.m. ✓

Abutment A4. Total area of Finish for A4. 59.78 ✓

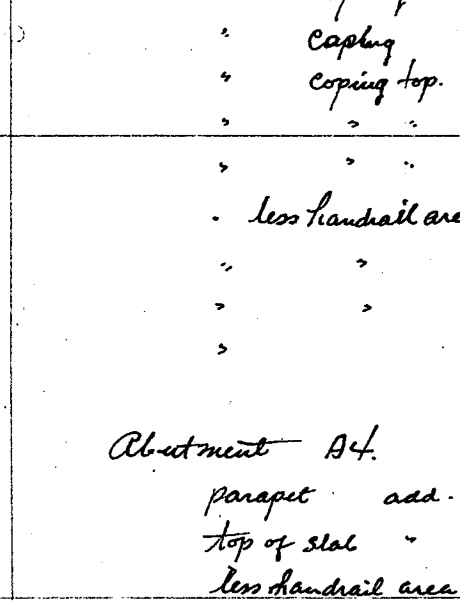
parapet add.	1.537 × 3.95 × 2	=		12.14 ✓
top of slab 〃	1.537 × 1.5 × 2	=		4.61 ✓
less handrail area	.4 × 1.537 × 2	=	(−)	1.23 ✓
coping	.1 × 1.537 × 2	=		.31 ✓
				15.83 ✓

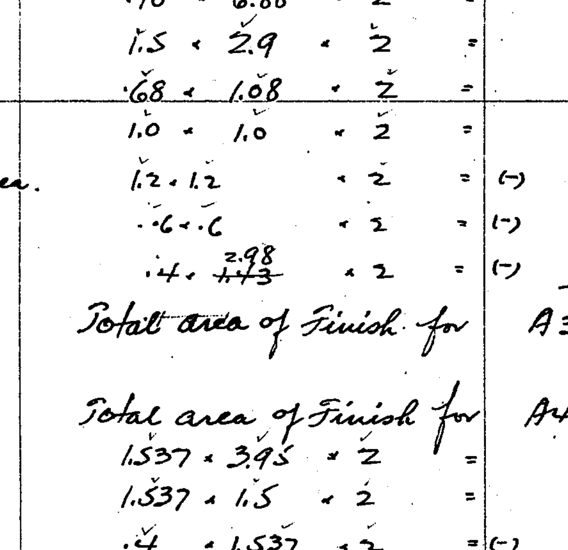
Total area of finish for A4 = 75.61 sq.m. ✓

踏掛石. 花崗石. 10ヶ — .2×.25×.87 = 0.435 m³ for abutment A3 + A4 each.

Foundation piles. 米松 末口 10吋 長 60呎 — 7×29 = 203本 for abutment A3 + A4 each.

Excavation.





Center of gravity

△A	3.6×6.9÷2 =	12.42 × 4.6 =	57.1 ✓
△B	8.6×6.5÷2 =	27.95 × 3.1 =	86.6 ✓
△C	8.6×2.2÷2 =	9.46 × .8 =	7.6 ✓
△D	3.3×1.2÷2 =	1.98 × 1.55 =	3.1 ✓
		51.81 m² × 2.98 m =	154.4

Excavation 51.81 × 33.06 = 1715. ✓
7.5×27.1×2.0 = 406. ✓
2121 cub.meter
for A3 + A4 each.

割栗地形 6.5×26.1×0.5 = 84.8 m³ for one abutment of A3 or A4.

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CALCULATIONS FOR

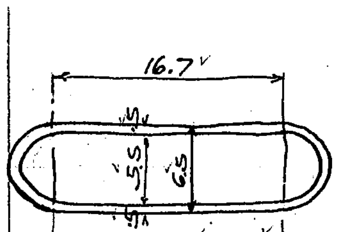
Materials of Jiuso Bashi for Osaka prefecture.

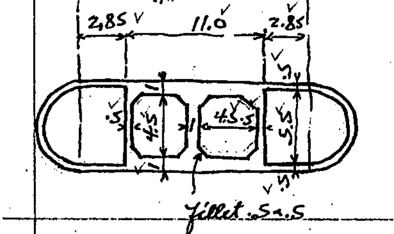
Pneumatic Caisson for Piers P7 + P8. $6.5^m \times 23.2^m$. height 28^m meters.

Concrete 1:2:4 mixture.

Section 0 to 1.0^m from top.

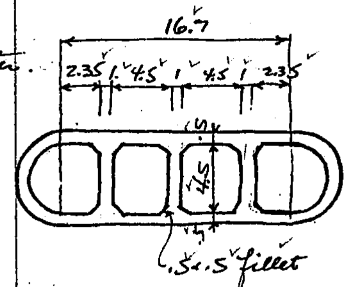
Sectional area.
$16.7 \times 0.5 \times 2 = 16.70$
$6.5^{\phi} - 5.5^{\phi} = 9.42$
$26.12^{\square m} \times 1.0 = 26.12$ cub.m.





Section 1.0^m to 2.0^m from top.

Sectional area.
$2.85 \times .5 \times 4 = 5.70$
$6.5^{\phi} - 5.5^{\phi} = 9.42$
$11.0 \times 1.0 \times 2 = 22.00$
$4.5 \times 1.0 \times 1 = 4.50$
$4.5 \times 0.5 \times 2 = 4.50$
$\frac{0.5 \times 0.5}{2} \times 8 = 1.00$
$47.12^{\square m} \times 1.0 = 47.12$ cub. meter.

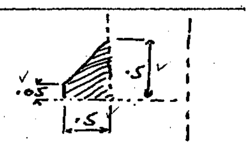


Section 2.0^m to 23.0^m from top.

Sectional area.
$16.7 \times 1.0 \times 2 = 33.40$
$6.5^{\phi} - 4.5^{\phi} = 17.28$
$4.5 \times 1.0 \times 3 = 13.50$
$\frac{0.5 \times 0.5}{2} \times 12 = 1.50$
$65.68^{\square m} \times 21.0 = 1379.28$ cub meter

Fillet at bottom area $0.5 \times 0.275 = 0.138^{\square m}$

length
$4.5 \times 2 \times 2 = 18.0$
$2.35 \times 2 \times 2 = 9.4$
$4.17^{\phi} = 13.1$
$(4.5 - .33) \times 6 = 25.0$
$65.5^m \times .138 = 9.04$ cub meter



Total Concrete for Shell = <u>1461.56</u> cub meter.

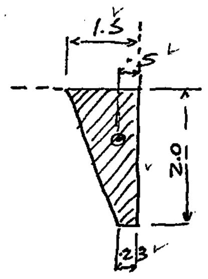
Working chamber.

Top floor.
$16.7 \times 6.5 = 108.55$
$6.5^{\phi} = 33.18$
holes less $1.22^{\phi} \times 2 = (-)2.34$
$139.39^{\square m} \times 1.5 = 209.09$ cub meter.

Cutting edge.

Sectional area $.865 \times 2.0 = 1.73^{\square m}$

Length
$16.7 \times 2 = 33.40^m$
$5.5^{\phi} = 17.28$
$50.68^m \times 1.73 = 87.68$ cub meter



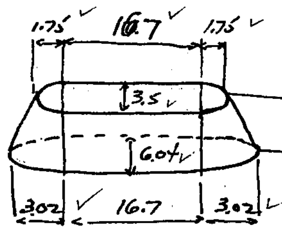
Total Concrete for working chamber = <u>296.77</u> cub meter.

Base Concrete.

working chamber filling.

Top area
$16.7 \times 3.5 = 58.45$
$3.50^{\phi} = 9.62$
$68.07^{\square m}$

Bottom area
$16.7 \times 6.04 = 100.87$
$6.04^{\phi} = 28.65$
$129.52^{\square m}$



Average area $= 68.07 + 129.52 = 197.59 \div 2 = 98.8^{\square m}$

volume of filling $98.80 \times 2.0 = 197.60$ cub m.

Base. area.
$16.7 \times 8.5 = 141.95$
$8.5^{\phi} = 56.75$
$198.70^{\square m} \times 1.5 = 298.05$

less $.35 \times 1.0 \times 16.7 \times 2 = 11.69$
" $.35 \times 1.0 \times 7.84^{\phi} = 8.62$
$(-)20.31$
277.74 cub m

Total Base concrete = <u>475.34</u> cub meter

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CALCULATIONS FOR

Materials of Jiuso Bashi for Osaka prefecture.

Summary of concrete for Pneumatic Caisson. Piers P7 × P8

Concrete for Shell	1,461.56	} 1,758.33 m³
〃 working chamber	296.77	
Base concrete	475.34	
	2,233.67 cub. meters.	

Forms.

Section 0.0m to 1.0m from top.

Straight	16.7 × 4 =	66.80		
Or end	6.5φ =	20.42		
〃	5.5φ =	17.28		
		104.5 0m	× 1.0 =	104.5 om.

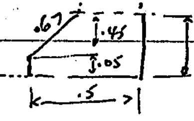
Section 1.0m to 2.0m from top.

Straight outside	16.7 × 2 =	33.40		
Or end 〃	6.5φ =	20.42		
Straight inside	2.85 × 4 =	11.40		
〃 〃	3.5 × 8 =	28.00		
〃 〃	5.5 × 2 =	11.00		
Or end 〃	5.5φ =	17.28		
fillet	0.71 × 8 =	5.68		
		~~109.90~~ 127.18	× 1.0 =	~~109.90~~ om 127.18

Section 2.0m to 22.5

Straight outside	16.7 × 2 =	33.40		
Or end 〃	6.5φ =	20.42		
Straight inside	1.85 × 4 =	7.40		
〃 〃	3.5 × 10 =	35.00		
fillet 〃	0.71 × 12 =	8.52		
Or end 〃	4.5φ =	14.14		
		118.88	× 20.5 =	2,437.04

Section 22.5m to 23.0m



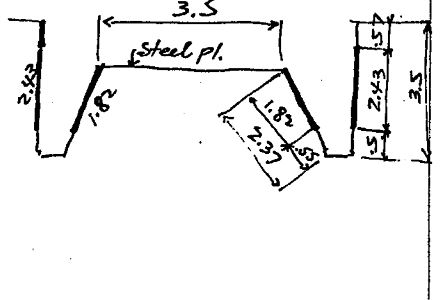
Straight outside	16.7 × 2 =	33.40		
Or end 〃	6.5φ =	20.42		
			53.82 × 0.5 =	26.91
Straight inside	1.85 × 4 =	7.40		
〃 〃	4.5 × 4 =	18.00		
〃 partition	4.17 × 6 =	25.02		
Circular end 〃	4.5φ =	14.14		
			64.56 × .67 =	43.26

Total form for Shell = 2,738.89

Call this 2738.9 sq. meter.

Working chamber

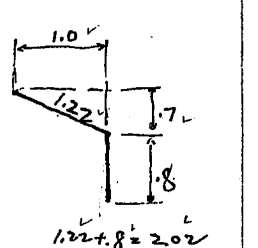
Bottom of slab : no form required.



Cutting edge inside	16.7 × 2 =	33.40		
〃 〃	4.4φ =	13.82		
		47.22 × 1.82 =	85.94	
Out side	16.7 × 2 =	33.40		
〃	6.5φ =	20.42		
		53.82		
less plate	1.2 × 6 =	− 7.20		
		46.62 × 2.43 =	113.29	

Total form for working chamber = 199.23 sq. meters

Footing.



16.7 × 2 = 33.4 × 2.02 =	67.47	
8.5φ × 0.8 =	21.36	
7.5φ × 1.22 =	28.74	

Total form for footing 117.57 sq. meter

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CALCULATIONS FOR

Materials for Jiuso Bashi for Osaka prefecture

Summary of Forms for Pneumatic caisson. Piers P7 + P8

Form for Shell = 2,738.9 ✓
working chamber = 199.2 ✓
2,938.1 ✓ ――――― 損料.
Footing of Base = 117.6 ✓ ―――――― 材料埋殺し
3,055.7 ✓ sq. meter.

Reinforcements for Pneumatic caisson. P7 + P8
Deformed Bars 88,538 kg. tons.

Structural steel for working chamber. 43,~~493~~ 599 ✓ kg. tons.

Pneumatic Caisson for Piers P9 + P10. P11 + P6 Total height 27.0 ✓ meters.
Concrete 1:2:4 mixture.
Shell concrete for 28m caisson = 1,461.56 ✓
less 1.0m of shell. $65.68^{\square m} \times 1.0$ = (−) 65.68 ✓
Total concrete for shell = 1,395.88 cub. meter } 1,692.65
" " working chamber = 296.77 ✓
" " base + fill = 475.34 ✓
2,167.99 ✓ Cub. meter.

Forms.
Shell forms for 28m caisson = 2,738.89 ✓
less 1.0m ✓ of shell $118.88^{\square m} \times 1.0$ ✓ = (−) 118.88 ✓
Total form for shell = 2,620.01 ✓
" " working chamber = 199.23 ✓
2,819.24 ✓ ――――― 損料
" " base 117.60 ✓ ――――― 材料埋殺し.
2,936.84 ✓ sq. meter.

Reinforcements for Pneumatic caisson
Deformed Bars. 84,873 ✓ kg. ✓ tons.

Structural steel for working chamber 43,~~493~~ 599 ✓ kg. tons.

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CALCULATIONS FOR

Materials for Jinso Bashi for Osaka prefecture.

Excavation for Pneumatic Caissons.

Sectional area of Caisson

$16.7 \times 6.5 = 108.55$ √
$6.5^{\phi} = 33.18$ √
141.73 □m √

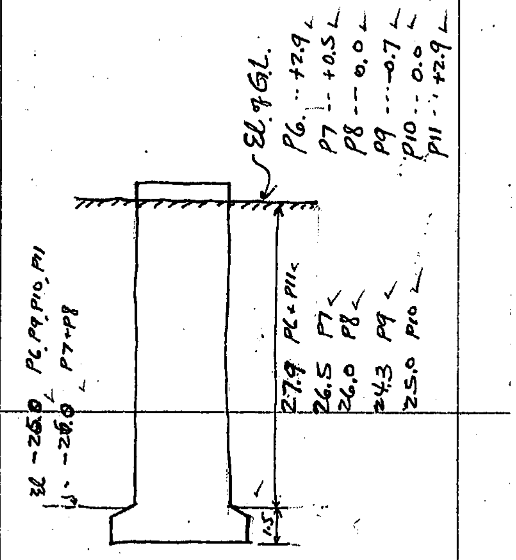
volume of Base.

$16.7 \times 8.5 = 141.95$ √
$8.5^{\phi} = 56.75$ √
198.70 □m $\times 1.5 = 298.05$ √

less. Chamfer $.35 \times 1.0 \times 16.7 \times 2 = 11.69$ √
" " $.35 \times 1.0 \times 7.84^{\phi} = 8.62$ √
(−)20.31 √
277.74 √ cub meter.

Excavation for Several Piers.

Pier nos.	Caisson		Base		Total Excav. under G.L.
P6	141.73 × ~~29.3~~ 27.9 =	3955. √	+ 277.74 √	=	4232.7 m³ √
P7	" × 26.5 √ =	3755 √	+ "	=	4032.7 √
P8	" × 26.0 √ =	3685 √	+ "	=	3962.7 √
P9	" × 24.3 √ =	3445 √	+ "	=	3722.7 √
P10	" × 25.0 √ =	3545 √	+ "	=	3822.7 √
P11.	" × 27.9 √ =	3955 √	+ "	=	4232.7 √



average Excavation for P6 & P11 √	4232.7 √
" " " P7 & P8 √	3997.7 √
" " " P9 & P10 √	3772.7 √

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CHECKED BY M. Jinda DATE 4-8-6 PAGE NO A86

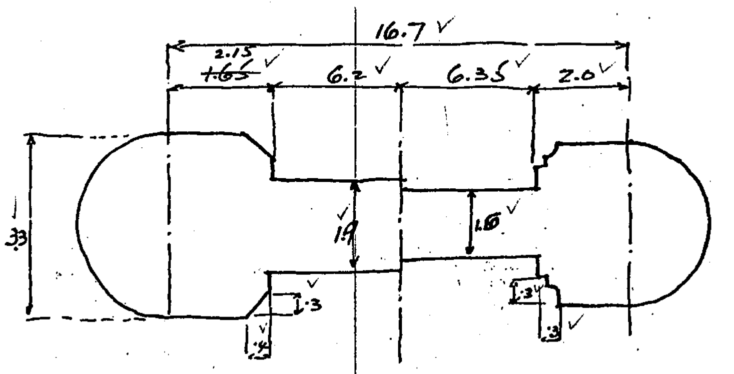
CALCULATIONS FOR

Materials for Jiuso Bashi for Osaka prefecture

Materials for Pier Shaft. Piers P7, P8, P9 + P10

Concrete 1:2:4 mixture.

Coping 2.15 × 3.3 × 2 = 14.19
" 3.3ϕ = 8.55
" 1.9 × 12.4 = 23.56
Chamfer less. .3 × .4 × 2 = (−) 0.24
46.06 × 0.5 = 23.03 m³



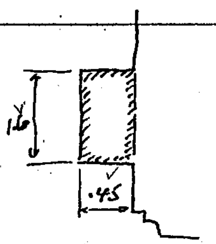
Shaft.

Pier P7 + P10 Top section		Pier P7 + P10 Bottom section		Pier P8 + P9 Top Section	Pier P8 + P9 Bottom section	
3.0 × 2.0 × 2	12.00	4.188 × 2.0 × 2	= 16.75		4.207 × 2.0 × 2 =	16.83
3.0ϕ =	7.07	4.188ϕ	= 13.75		4.207ϕ =	13.90
less .3 × .3 × 2 = (−)	0.18	less	−0.18		less	−0.18
	18.89 ㎡		30.32 ㎡	18.89 ㎡		30.55 ㎡

average area = 24.61 ㎡   Average area = 24.72 ㎡

Concrete for Shaft Piers P7 + P10 24.61 × 7.13 = 175.47 Cub meter.
Piers P8 + P9 24.72 × 7.24 = 178.97 "

Inside projection shaft. Piers P7 + P10 0.72 × 2 × 7.13 = 10.27 Cub meter
Piers P8 + P9 0.72 × 2 × 7.24 = 10.43 "

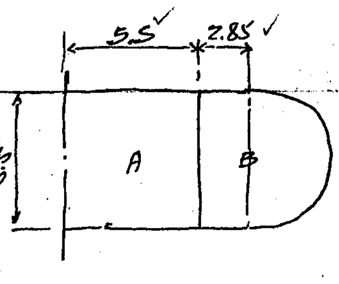


Top Strut. 1.6 × $\overset{\text{mean}}{2.0}$ × 11.8 = 37.76 Cub. meter

Curtain wall. section 11.8 × 1.0 = 11.8 ㎡
Piers P7 + P10 11.8 × 5.13 = 60.53 cub meter.
Piers P8 + P9 11.8 × 5.24 = 61.83 "

.45 × 1.6 = .72 ㎡

Base of shaft. (top fill of caisson).
Area A. 5.5 × 5.5 = 30.25
30.25 × 1.0 × 2 ~~30~~ = 60.50
Area B. 5.5 × 2.85 = 15.68
5.5ϕ ÷ 2 = 11.88
27.56
27.56 × 2.0 × 2 = 110.24
170.74 cub. meter.



Summary of Concrete for Shaft.

	Piers P7 + P10	Piers P8 + P9.
Coping	23.03	23.03
Shaft.	175.47	178.97
Inside projection of shaft.	10.27	10.43
Top Strut	37.76	37.76
Curtain wall.	60.53	61.83
Base of shaft.	170.74	170.74
	477.80 cub. meter	482.76 cub meter.
Chairs for stringers.	2.44	2.44
	480.24 cub. meter	485.20 cub meter for one pier.

Average 482.72 cub meter

Chairs for Stringer Bearings.
1.5 × 0.8 × 1.02 = 1.22 cub. meter
for 2 chairs 1.22 × 2 = 2.44 cub meter.

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CHECKED BY M. Jinda DATE 4-8-6 PAGE NO A87

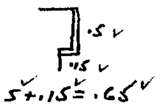
CALCULATIONS FOR

Materials of Jiuso-Bashi for Osaka Prefecture

Forms for Piers P7, P8, P9, & P10

Coping Total length of perimeter.

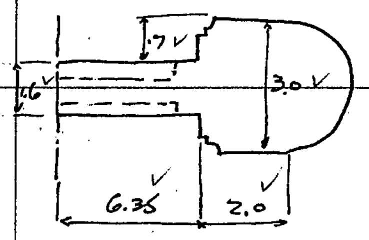
2.15 × 4 = 8.60
0.7 × 4 = 2.80
12.4 × 2 = 24.80
3.3φ = 10.37
46.57 × 0.65 = 30.27 sq. meter



Shaft, strut + walls.

Total perimeter of top section

6.35 × 4 = 25.40
2.0 × 4 = 8.00
0.7 × 4 = 2.80
3.0φ = 9.42
45.62 m for all piers



Total length of perimeter at bottom section.

Piers P7 + P10		Piers P8 + P9
	6.35 × 4 = 25.40	6.35 × 4 = 25.40
	2.0 × 4 = 8.00	2.0 × 4 = 8.00
	1.299 × 4 = 5.18	1.304 × 4 = 5.22
	4.188φ = 13.16	4.207φ = 13.23
Bottom	51.74	51.85
top.	45.62	45.62
	97.36	97.47
average	48.68 m	48.74 m

Form for P7 + P10 48.68 × 7.13 = 347.09 sq. meter
" " P8 + P9 48.74 × 7.24 = 352.88 "

Depression between strut + wall.

Total length 11.8 + 4.63 = 16.43 for P7 + 10
11.8 + 4.74 = 16.54 for P8 + P9.

width 1.6 − 1.0 = 0.6 m

Form for P7 + P10 16.43 × 0.6 = 9.86 sq. meter
" P8 + P9 16.54 × 0.6 = 9.92 "

Stringer bearing chair.

4.6 × 1.02 × 2 = 9.38 sq. meter

Summary of Forms for Piers shafts. P7, P8, P9, + P10 (模料 1部)

	Piers P7 + P10	Piers P8 + P9
Coping	30.27	30.27
Shaft, strut, + wall	347.09	352.88
Depression, wall.	9.86	9.92
Stringer bearing chair	9.38	9.38
	396.60 sq. meter	402.45 sq. meter for one pier

average 399.53 sq. meter.

Bottom of Base of shaft. (埋殺之 1部)

4.5 × 4.5 × 2 = 40.5
4.5 × 2.35 × 2 = 21.15
4.5φ = 15.90
less .5 × .5 × 6 = (−) 1.50
76.05 sq. meter.

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CALCULATIONS FOR

Materials for Juiso Bashi for Osaka prefecture

Reinforcements for Shaft. Piers P7 P8 P9 + P10

Deformed Bars.

For Piers P7 and P10 17.122 ✓ kg tons. See drawing of pier shaft.

" " P8 and P9 17.122 ✓ " " "

Materials for Piers P6 + P11. Shaft.

Shaft. Concrete 1:2:4 mixture.

Coping for P7 +c = 23.03 ✓ cub. meters.

.3 × .5 × 12.4 = 1.86 ✓

Shaft. Top section

Same as for P7 +c

Bottom section. 24.89 ✓

4.055 × 2.0 × 2 = 16.22 ✓

4.055^{ϕ} = 12.91 ✓

less corner .3 × .3 × 2 = (−) .18 ✓

$28.95^{\square m}$ ✓

$18.89^{\square m}$ ✓

average area = $23.92^{\square m}$ ✓

Concrete for Shaft = 23.92 × 6.33 ✓ = 151.41 ✓ cub. meters.

Inside projection of shaft.

1.9 × 0.45 = $0.855^{\square m}$ × 2 × 6.33 ✓ = 10.82 ✓ cub. meters

Top Strut. 1.9 × $\overset{mean}{2.0}$ × 11.8 ✓ = 44.84 ✓ cub. meter.

Curtain wall section 1.0 × 11.8 = $11.8^{\square m}$ ✓

11.8 × 4.33 ✓ = 51.09 ✓ cub. meters.

Base of shaft same as for P7 +c. = 170.74 ✓ cub. meters

Chairs for truss shoes. 1.5 × 1.8 × .57 × 2 ✓ = 3.08 ✓ cub. meter.

Chairs for stringer bearings. .8 × .8 × 1.59 × 2 ✓ = 2.04 ✓ cub. meter

Summary of Concrete for Pier Shaft P6 + P11.

Coping	24.89 ✓
Shaft.	151.41 ✓
Inside projection of shaft.	10.82 ✓
Top Strut.	44.84 ✓
Curtain wall	51.09 ✓
Base of shaft.	170.74 ✓
Chair for truss shoes	3.08 ✓
" " Stringer bearing	2.04 ✓
	458.91 ✓ Cub. meter.

Reinforcements for Shaft P6 and P11.

Deformed Bars. 15.208 ✓ kg tons. See drawing of pier shaft.

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CALCULATIONS FOR

Materials of Jiuso Bashi for Osaka prefecture.

Forms for Pier Shaft. P6 × P11

Coping Total length of perimeter

2.15 × 4 = 8.60
0.7 × 2 = 1.40
0.4 × 2 = 0.80
12.4 × 2 = 24.80
3.3φ = 10.37
45.97 × 0.65 = 29.88 square meter.

Shaft strut and wall.

Total length of perimeter at top section.

6.35 × 4 = 25.40
2.0 × 4 = 8.00
0.7 × 2 = 1.40
0.4 × 2 = 0.80
3.0φ = 9.42
45.02 m

Total length of perimeter at bottom section.

6.35 × 4 = 25.40
2.0 × 4 = 8.00
1.228 × 2 = 2.46
0.928 × 2 = 1.86
4.055φ = 12.73
50.45

average = 47.74 m
Form = 47.74 × 6.33 = 302.19 Square meter.

Depression between strut and wall.

Total length 11.8 + 3.83 × 2 = 19.46 m
width = 1.9 − 1.0 = 0.9 m
form = 19.46 × 0.9 = 17.51 Square meter.

Chairs for truss shoe.

0.57 × 6.6 m × 2 = 7.52 square meter.

Chairs for stringers 8.4 × 1.60 × 2 = 10.24 square meter.

Bottom of Base of shaft. same as for P7 ~ c = 76.05 square meter.

Summary of Forms for Pier shaft P6 + P11

Coping	29.88	357.10
Shaft wall, + strut	302.19	
Depression, wall	17.51	
Chair, truss shoes	7.52	
Chains stringers	10.24	
~~So~~	367.34 □m	------(損料ノ部)
Bottom of base for shaft.	76.05 □m	-----(埋殺ノ部)
Total form for one pier =	443.39 "	

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3

Materials of Jiuso Bashi for Osaka Prefecture.

Pneumatic Caisson. 6.5m × 23.2m – 25.2m height. Refer to page A82.

– Concrete 1:2:4 mixture

Section 0 to 1.0 meter from top.
26.12 m² × 1.0 = 26.12 cub meters.

Section 1.0m to 2.0m from top
47.12 × 1.0 = 47.12 "

Section 2.0m to 20.2m " 65.68 □m × 18.2 = 1195.38
fillet at bottom 9.04
1204.42 "

working chamber 296.77 "

Total concrete 1574.43 cub. meter.

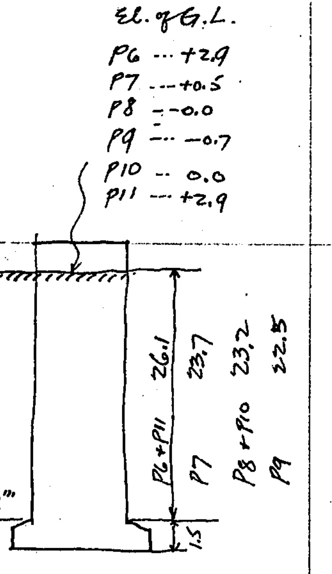
Base concrete 475.34
2049.77 "

Forms.

Section 0m to 1.0m from top 104.50 × 1.0 = 104.50 m²
Section 1.0 to 2.0m " 127.18 × 1.0 = 127.18
Section 2.0 to 19.7 " 118.88 × 17.7 = 2104.18
Section 19.7 to 20.2 " 70.17
2406.03 } ~~2605.26 call this 2605.3 m² 材料~~
working chamber 199.23
footing 117.57 call this 117.6 m² --- 材料埋殺
2722.83

Reinforcement for pneumatic caisson Deformed bars. 81.391 kg tons.

Structural steel for working chamber 43.599 kg. tons.



Excavation of several piers.

Sectional area of pneumatic caisson = 141.73 □m
volume of base = 277.74 cub. meter
call this 278. "

Excavation.

Pier no.	Caisson			Base		Total excav.	average.
P6	141.73	× 26.1	= 3,700.	+ 278	=	3,978	}---- 3,978 m³
P11	"	× 26.1	= 3,700	+ 278	=	3,978	
P7	"	× 23.7	= $3\overset{3}{\cancel{4}}58$	+ 278	=	3,636	}---- 3,601
P10	"	× 23.2	= 3287	+ 278	=	3,565	
P8	"	× 23.2	= 3287	+ 278	=	3,565	}---- 3,515
P9	"	× 22.5	= 3187	+ 278	=	3,465	

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CALCULATIONS FOR

Materials of Juiso Bashi for Osaka prefecture.

Materials of Abutment for River span A3 and A4. Refer to page A79.

Concrete for abutment A3 1:2:4 mixture.

Ornaments
0.2 × 0.48 × 3 × 2.81 = .81
0.37 × .54 × .38 × 2 = .15
.12 × .09 × .48 × 4 = .02
.05 × .4 × 2.42 × = .05
.120 × .85 × 2.42 = .25
1.28 × 2 = 2.56 cub meter
Abutment body 538.74
541.30 cub meters.

Concrete for abutment A4. 1:2:4 mixture
concrete for ornaments 1.28 × 2 = 2.56
" " abutment body 548.80
551.36 cub meters.

Forms for abutment A3.
Ornaments projection.
0.2 × 2.81 × 2 = 1.12
0.2 × 2.27 × 4 = 1.82
0.19 × 0.54 × 4 = .41
3.35 × 2 = 6.70
0.125 × 1.25 × 2 = 0.31 × 2 = 0.62
~~0.1 × 2.42~~ =
7.32
Abutment body 821.08
828.40 sq. meters.

Forms for abutment A4.
Ornaments 7.32
abutment body 882.61
889.93 call this 889.9 square meters.

人造洗と仕上.
Ornaments.
2.18 × 2.81 = 6.12
0.2 × 2.81 × 2 = 1.12
0.2 × 2.27 × 4 = 1.82
0.19 × 0.54 × 4 = 0.41
0.2 × 0.48 × 3 = 0.29
0.37 × 0.38 × 2 = 0.28
0.60 × 2.42 = 1.45
2.42 × 1.25 = 3.02
0.85 × 1.25 = 1.06
15.57 × 2 = 31.14 sq. meters
abutment A3. 59.78
less 6.12 × 2 = -12.24
78.68 sq. meters.

abutment A4
ornament 31.14
abutment body 75.61
less -12.24
94.51 sq. meters