JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

MADE BY ______ DATE ______ FILE NO ______
CHECKED BY ______ DATE ______ PAGE NO ______

昭和四年八月

國道第拾八號

鳥取縣

千代橋架設工事設計々算書

附 工事用材料計算表

Design of Sendai Bashi for Tottori Ken.

General Layout of the Bridge.

16 spans	@ 21.84	c to c bearings	=	349.44
15 spaces	@ 0.55	" "	=	8.25
2 spaces	@ 0.30	at both ends	=	0.60
Total Length between faces of Parapet walls.			=	358.29 meters

Each span being a plate girder of 4 panels @ 5.46 meter long and will have a total length of 22.34 meters out to out. Clear Roadway 7.5 meters between curb lines. Granolithic pavement on reinforced concrete floor slab.

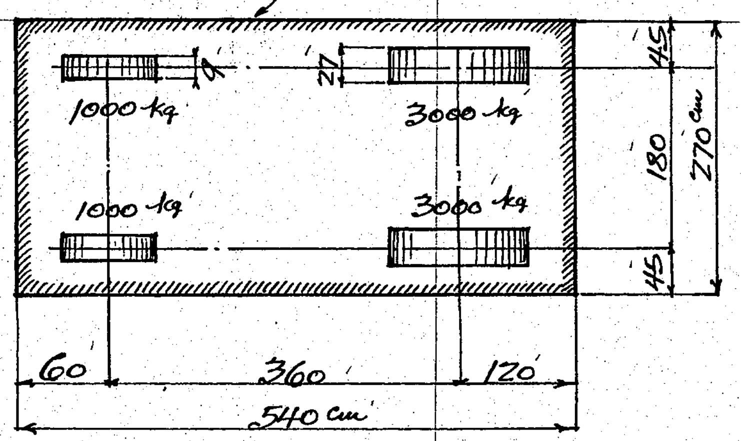
Assumed Loadings.

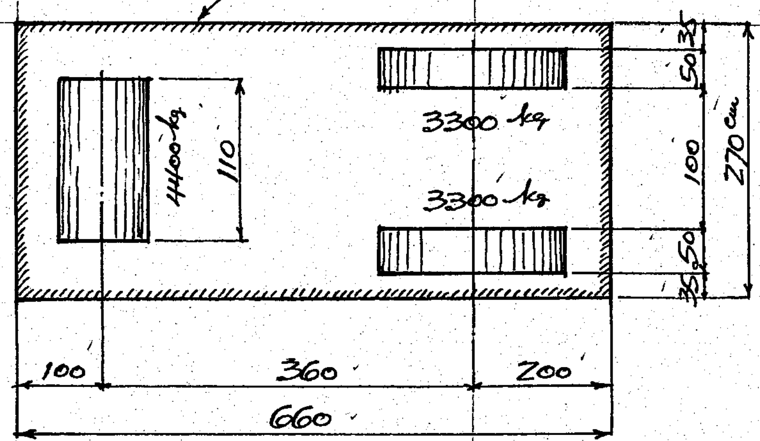
Uniform load on roadway $w = \frac{100000}{170+l} \leq 500$ kg/m²

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

8 ton Motor truck loading.

11 ton road roller loading.





2 rows of motor traffics on roadway with occupied width of 270 cm each, unoccupied space around the motor truck shall be filled with uniform load specified above.

One road roller on one span.

Impact for motor truck loading. Coefficient $= \frac{20}{60+l}$ where l = loaded length in meters.
max. impact limitted 30%.

No impact considered for road roller and uniform live load.

Allowable working strength of materials

Concrete 1:2:4 mixture.

Direct compression	35 kg/cm²
fibre stress due to bending	45 "
Combined stress due to direct and bending, compression members	35 "
do , arch ring	45 "
Punching shear of concrete	9 "
Shear of plain concrete	4 "
bearing value	45 "
bond stress for plain bars	6 "
do for deformed bars.	9 "

Structural steel.

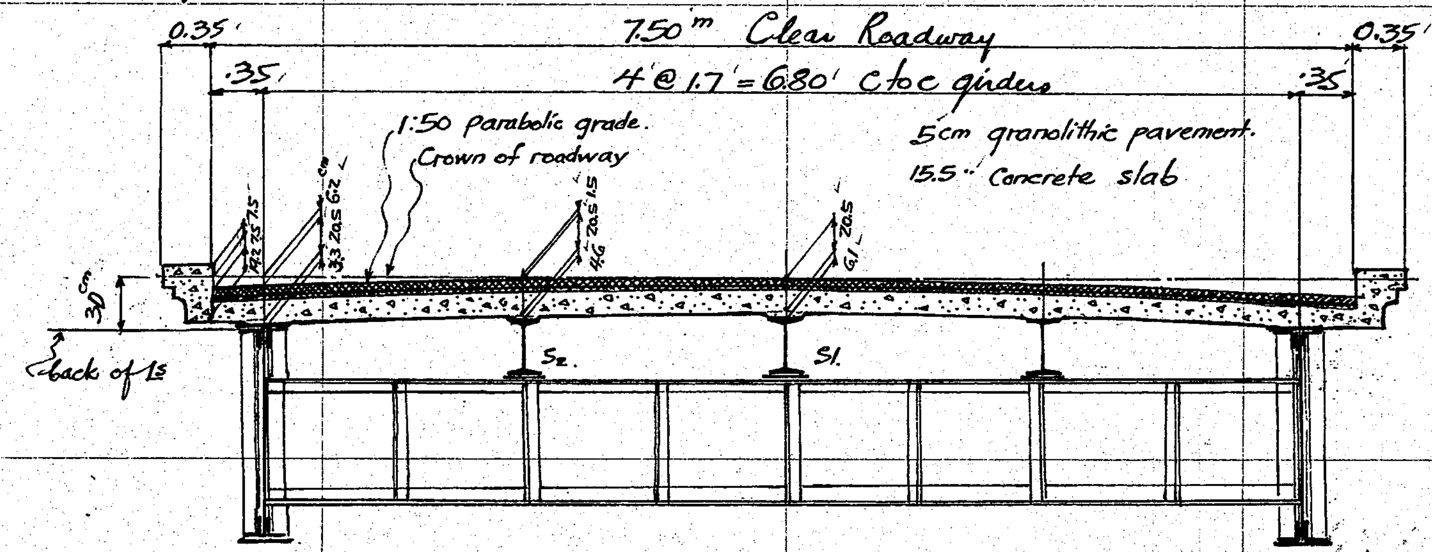
Tension, net	1,200 kg/cm²
extreme fibre stress net	1,200 "
Compression members $1500\,(1-0.0055\,l/r)$ not over	1,000 "
Shear of web, gross section	900 "

Design of Sendai Bashi for Tottori Ken.

Compression flange of girder
$1200(1-0.012\ l/b)$ not over ———— 1,100 kg/cm²
where l = unsupported length of flange in cm
b = width of flange in cm.
Shear on shop driven rivets (machine driven) ———— 850 "
" " field driven rivets and turned bolts (machine driven) ———— 750 "
Shear on pins ———— 900 "
bearing on shop driven rivets (machine driven) ———— 1,700 "
" " field driven rivets " ———— 1,500 "
" " pins ———— 1,800 "
Steel roller $45d$ kg/cm where d = diameter of roller in cm.

Considering wind or temperature stresses in addition to dead, live and impact stresses, the allowable working strength shall be increased 25% ; in case of earthquake increase working strength 60%.
Seismic acceleration 2,000 mm/sec² or $k = 0.20$ assumed.

Cross section of Bridge assumed as shown on sketch below.



Scale 1:50.

Design of Floor slab. Span length = 1.70 meters
Dead Load.

5.0 cm	granolithic pavement	@ 22 kg	=	110.
15.5 "	Concrete slab	@ 24	=	372
	miscellaneous concrete say		=	8
				490 kg/sq. meter.

Dead Load moment = $1/10 \times 490 \times 1.70^2 = 142$ kgm
Dead Load shear = $1/2 \times 490 \times 1.70 = 416$ kg

Live Load. Motor truck rear wheel concentration = 3,000 kg
30% impact = 900
3,900 kg

Distribution of wheel concentration on slab.
Contact between wheel and pavement = 20.0 cm
distribution 2×5.0 = 10.0
Longitudinal distribution = $a = 30.0$ cm
Transverse distribution $b = 27 + 10 = 37.0$ cm

Effective width $\varepsilon = 2/3\ l + a$ where l = span length in meters.
$= 2/3 \times 1.70 + 0.30 = 1.43$ meters.

Load per meter strip of slab = $3900 \div 1.43 = 2730$ kg

Uniform live load = 500 kg per sq. meter.

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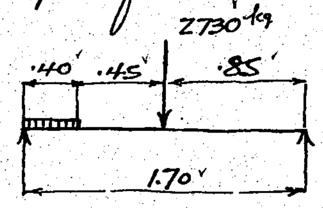
Design of Sendai Bashi for Tottori ken.



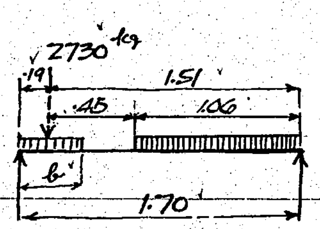
Moment on slab.

Reaction due to uniform load $= \frac{500 \times 0.40^2}{2 \times 1.70} =$ 24

due to motor truck $= 2730 =$ 1365 / 1389 kg

moment at center $= 1389 \times .85 = 1180$ kgm.

for continuity of slab, moment $= 0.80 \times 1180 =$ 945 kgm



End shear on slab. $2730 \times \frac{1.51}{1.70} =$ 2425 kg

uniform load $\frac{500 \times 1.06^2}{2 \times 1.70} =$ 166 / 2591 kgs

Summary for moments and shears on slab.

	moments	End shears.
Dead Load	142	416
Live Load	945	2591
	1087 kgm	3007 kg

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

$R = \frac{M}{bd^2}$, $d = \sqrt{\frac{M}{bR}}$ where $R = 7.18$ ∴ $d = \sqrt{\frac{1087 \times 100}{100 \times 7.18}} = 12.3$ cm

Use 13 cm effective depth with 2.5 cm insulation or total depth 15.5 cm

Steel area required $= \frac{1087 \times 100}{1200 \times 7/8 \times 13} = 7.97$ cm² per meter strip.

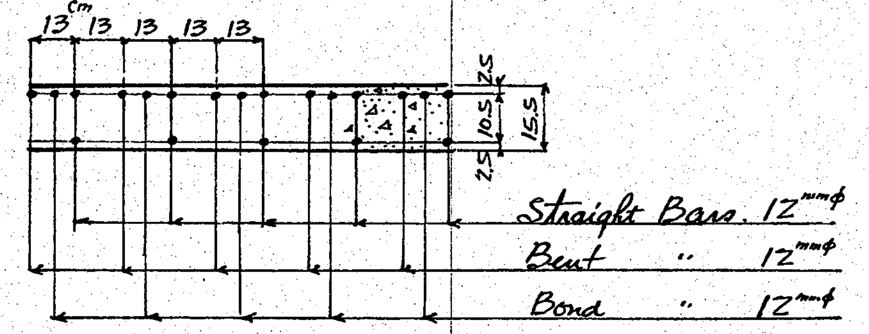
Use 12 mmϕ bars at 13 cm c to c $= 8.70$ " "

Steel ratio $= \frac{8.7}{100 \times 13} = .0067$ $k = .360$; $j = .880$

$f_s = \frac{1087 \times 100}{8.7 \times .88 \times 13} =$ 1093 kg/cm²

$f_c = 1093 \times .36 \times \frac{1}{(1.0 - .36)15} =$ 41 kg/cm²

unit shear $= \frac{3007}{100 \times .88 \times 13} =$ 2.6 kg/cm²



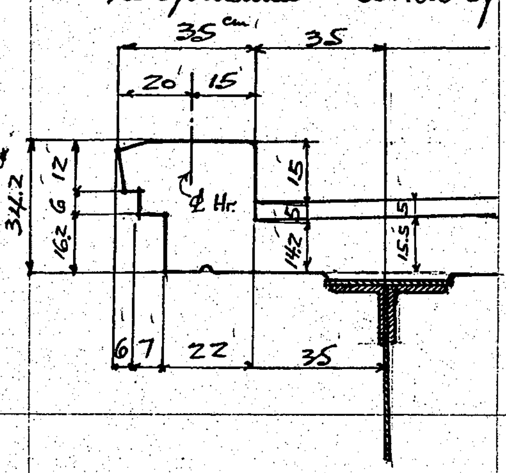
Total perimeter of bars for bond.

main bars $7.70 \times 3.77 =$ 29.0

bond " $3.85 \times 3.77 =$ 14.5 / 43.5 cm

Unit bond $= \frac{3007}{43.5 \times .88 \times 13} =$ 6.0 kg/cm²

Longitudinal section of slab at support.



Overhanging slab beyond main girder.

Dead Load

Coping concrete	.28 × .342	× 2400	= 230	× .49	= 113
slab and pavement.	.35	× 490	= 172	× .175	= 30
Handrail say			= 170	× .50	= 85
miscellaneous, say			= 8	× .40	= 3
			580 kg		231 kgm

Dead Load moment = 231 kgm

Dead Load shear = 580 kg

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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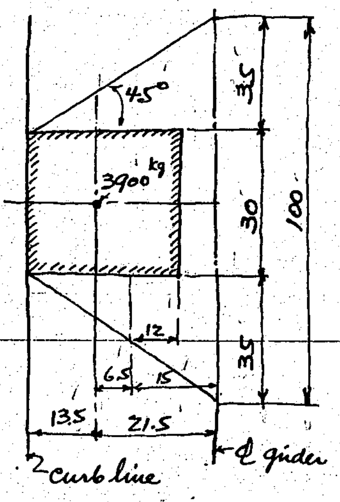
CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.

Live Load.



Motor truck rear wheel concentration with impact = 3900 kg
Outside of type in contact with curb line assumed, distribution assumed as follows.
Contact on pav. = 20 + distribution 2×5 = 30
distribution of slab say 2×35 = 70
100 cm effective width.

Load per meter strip = 3900 × 100/100 = 3900 kg.

Moment = 3900 × 0.215 = 838 kgm
End shear = 3900 × (37−12)/37 = 2635 kg

Summary of moments and shears.

	moments	End shears.
Dead Load	231	580
Live Load	838	2635
	1,069 kgm	3215 kg

Unit stresses by propotion.

$f_s = 1093 \times \frac{1069}{1087} = 1,076$ kg/cm²

$f_c = 41 \times \frac{1069}{1087} = 40.3$ 〃

unit shear = $2.6 \times \frac{\cancel{2841}\ 3215}{3007} = 2.8$ 〃

unit bond = $6.0 \times \frac{3215}{3007} = 6.4$ 〃

普通ノ場合ハ車輪中心ガ Curb line ヨリ 45cm 内方ニアリテ moment 及 shear ヲ起サズ此場合ハ突発事故ノ場合例ヘハ自働車ガ高欄ニ衝突破壊シタル如キ事故ニ対スル所謂 emergency stress ノ計算ナル故 Bond stress ガ僅ニ超過スレド相当 allowable working stress ヲ増加スルモ差支ナシ

Design of I-Beam Stringers.

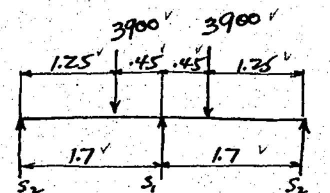
Center stringer S1. Span length 5.46 meters, Spacing 1.70 meter c to c.

Dead load:—

Floor slab and pavement	1.70 @ 490	=	833
beam assumed			75
			908 kg per lin meter.
		Call this	910 〃 〃

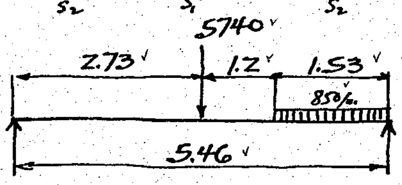
Dead Load moment = 1/8 × 910 × 5.46² = 3390 kgm
Dead Load shear = 1/2 × 910 × 5.46 = 2490 kg

Live Load



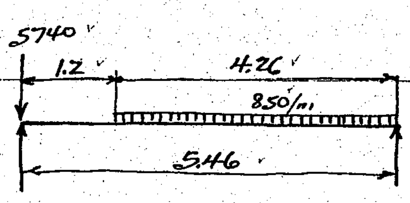
Max. wheel load on stringer 2 @ 3900 × 1.25/1.70 = 5740 kg.
Uniform load on stringer 1.7 @ 500 = 850 kg per lin m.

Moment on stringer



Reaction due to wheel = 5740 ÷ 2 = 2870
〃 〃 unif. load = $\frac{1.53^2}{2} \times \frac{850}{5.46}$ = 182
3052 kg.

Moment = 3052 × 2.73 = 8330 kgm



Max. end shear
Uniform load $\frac{850 \times 4.26^2}{2 \times 5.46}$ = 1410
wheel load = 5740
7150 kg.

Summary for moments and shears.

	Moments	End shears.
Dead Load	3390	2490
Live Load	8330	7150
	11,720 kgm	9640 kg

Design of Sendai Bashi for Tottori Ken.

Section modulus required = $\frac{11720 \times 100}{1100}$ = 1065 cm³

Use I-beam 400 × 150 @ 72 kg whose s.m. = 1199 cm³

Extreme fibre stress = $\frac{11720 \times 100}{1199}$ = 977 kg/cm²

Shear on web. = $\frac{9640}{1 \times 40}$ = 241 kg/cm²

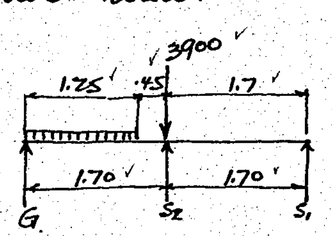
Outside Stringer S_2. Span length 5.46 meters, Spacing 1.70 m c to c.

Dead Load Dead Load moment and shear same as for S_1.

Dead Load moment = 3390 kgm

Dead Load shear = 2490 kg

Live Load.



max. load on stringer

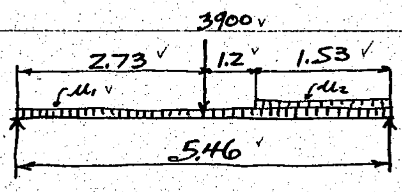
Uniform load $\frac{500 \times 1.25^2}{2 \times 1.70}$ = 230 kg/m. full load. = U_1

wheel load = 3900 kg at center of span.

Uniform load on rear of motor truck = 850 kg/m. = $U_1 + U_2$

230

620 = U_2



moment.

Reaction U_2 $\frac{620 \times 1.53^2}{2 \times 5.46}$ = 133

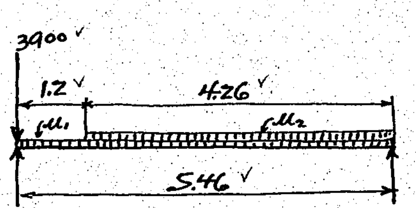
wheel $3900 \div 2$ = 1950

2083 kg.

moment U_2 + wheel 2083 × 2.73 = 5680

U_1 $1/8 \times 230 \times 5.46^2$ 857

6537 kgm



Shear.

U_2 $\frac{620 \times 4.26^2}{2 \times 5.46}$ = 1032

U_1 $230 \times 5.46 \div 2$ = 628

Wheel 3900 = 3900

5560 kg.

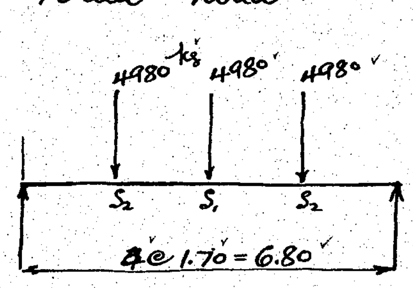
Summary for moments and shears.

	Moments	End shears
Dead Load	3390	2490
Live Load	6537	5560
	9927 kgm	8050 kg

Use same section as for stringer S_1.
or
1 I-beam 400 × 150 @ 72.0 kg.

Design of Intermediate floor beam. Span length 6.80 meters, spacing 5.46 meter c to c.

Dead Load



Stringer Concentration on floor beam. = 2 × 2490 = 4980 kg

Overhanging effect of coping, handrail &c on S_2 neglected.

Reaction

4980 × 1.5 = 7470 kg

moment. 7470 × 3.40 = 25400

4980 × 1.70 = − 8460

16940 kgm

weight of floor beam assumed 150 kg per lin m.

moment = $\frac{1}{8} \times 150 \times 6.80^2$ = 866 kgm.

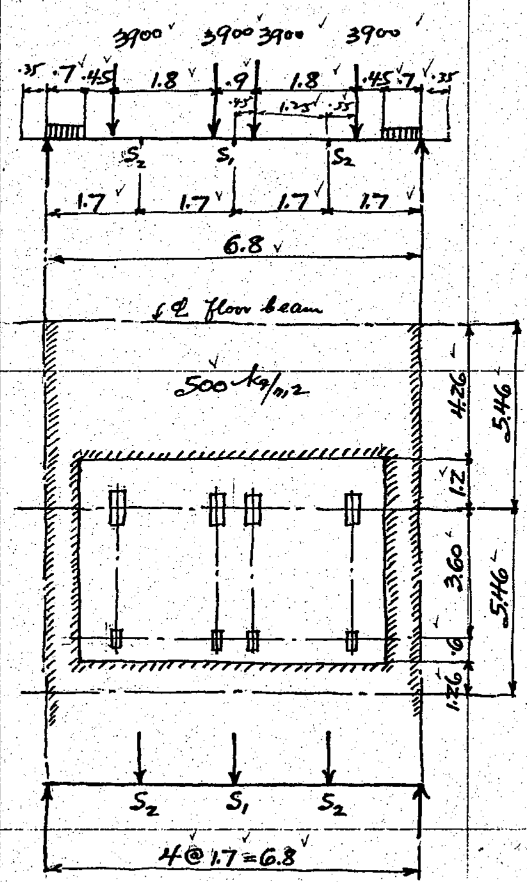
shear = $\frac{1}{2} \times 150 \times 6.8$ = 510 kg.

Summary of Dead Load moments and shears.

	moment	end shear
Stringer Concentrations	16940	7470
floor beam weight	866	510
	17806 kgm	7980 kg

Design of Sendai Bashi for Tottori ken.

Live Load:-



motor truck rear wheel concentration = 3,000
Impact coef. = $\frac{20}{60+6.8}$ = 30% = 900
3900 kg

front wheel concentration with imp. say 3900 ÷ 3 = 1,300 kg
Uniform live load 500 kg per sq. meter.

Wheel load concentration on Stringers.
on S_1 rear wheel $2 \times 3900 \times \frac{1.25}{1.70} = 5740$ kg front wheel 5740 ÷ 3 = 1910 kg.
on S_2 " $3900 \times \frac{1.6}{1.7} = 3670$ " " 3670 ÷ 3 = 1220 "

Concentration on floor beam.
at S_1 front wheel $1910 \times \frac{1.86}{5.46} = 650$
rear " 5740
6390 kg

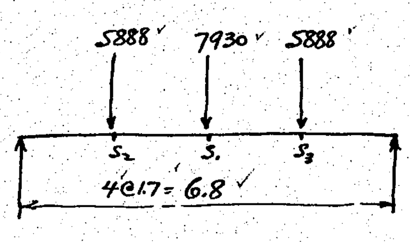
at S_2 front wheel $1220 \times \frac{1.86}{5.46} = 415$
rear " 3670
4085

uniform load on front + rear of truck
load on stringers $S_1 + S_2$ $500 \times 1.7 = 850$ kg/m.
Concentration on floor beam
on $S_1 + S_2$ $\frac{850 \times 4.26^2}{2} = 7720$
$\frac{850 \times 1.26^2}{2} = 675$
8395 ÷ 5.46 = 1540 kg.

Uniform load on sides of truck.
load on stringer S_2 $\frac{500 \times .7^2}{2 \times 1.7} = 72$ kg/m.
Concentration on floor beam at S_2
$72 \times 1.2 \times \frac{4.86}{5.46} = 77$
$72 \times 4.2 \times \frac{3.36}{5.46} = 186$
263 kg.

Summary of stringer concentration on floor beam.

	at S_1	at S_2
motor truck wheels.	6390	4085
Unif. load	1540	1540
	—	263
	7930 kg	5888 kg



Live Load moments + shears.
End shear = 7930 ÷ 2 = 3965
5888
9853 kg

moment at center $9853 \times 3.4 = 33500$
$5888 \times 1.7 = 10{,}000$
23500 kgm

Summary of moments and End shears.

	Moments	Shears.
Dead Load	17806	7980
Live Load	23500	9853
	41306 kgm	17833 kg

Try web plate 800×9 $\frac{1}{8}$ web area = 9.0 cm²
Depth of beam b to b Ls = 81 cm
effective depth = 81 − 4.4 = 76.6 cm
flange stress = $\frac{41306}{0.766}$ = 54000 kg T or C

Bottom flange area required = $\frac{54000}{1200}$ = 45.0
1/8 web area = − 9.0
36.0 cm² net.

Use 2Ls 125×90×10 = 41.0 − 4.4 = 36.6 " "

Top flange area required = $\frac{54000}{1100}$ = 49.1
1/8 web area = − 9.0
40.0 cm² gross.

Shearing stress on web = 17833 ÷ 72 = 248 kg/cm² Assumed section is ample.

Design of Sendai Bashi for Tottori Ken.

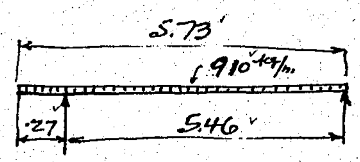
Approximate weight of Intermediate floor beam.

1 web Pl.	800 · 9	@ 56.52 · 6.80	=	385	
4 flange Ls	125 · 90 · 10	@ 16.10 · 6.66	=	429	
2 Conn. Ls	125 · 90 · 10	@ 16.10 · 0.62	=	20	
14 Stiff. Ls	90 · 75 · 9	@ 11.00 · 0.81	=	125	
3 Stringer bed pls.	235 · 9	@ 16.60 · 0.26	=	13	
Rivet heads and variation say		3½ %	=	34	
				1006 kg	Call this 1010 kg

weight of beam per lin meter = 1010 ÷ 6.8 = 149 kg.

Design of End Floor Beam. Span length = 6.80 meters. Spacing 5.46 meters c to c.

Dead Load.

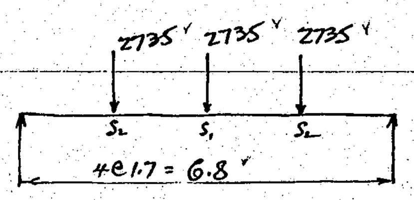


Stringer concentration on end floor beam

$\frac{910 \times 5.73^2}{2 \times 5.46} = 2735$ kg at $S_1 + S_2$

moment due to stringer concentration.

2735 · 1.5 = 4103



4103 · 3.40	=	13950	
2735 · 1.70	=	− 4650	
		9,300	kgm

weight of beam assumed 140 kg per lin meter.

moment = 1/8 · 140 · 6.8² = 810 kgm

Shear = 1/2 · 140 · 6.8 = 475 kg.

Summary of Dead Load Moments and Shears.

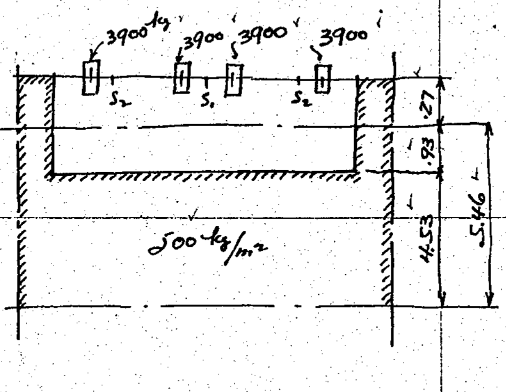
	moments	End shears.
Stringer concentration	9300	4103
weight of beam	810	475
	10110 kgm	4578 kg.

Live Load :-

Rear wheel concentration on floor beam

at S_1 5740 · $\frac{5.73}{5.46}$ = 6020

at S_2 3670 · " = 3850



Unif. load concentration at $S_1 + S_2$

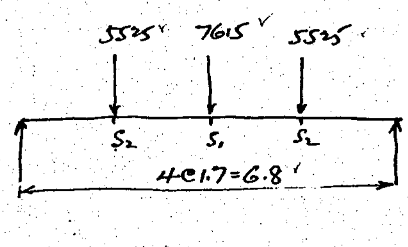
$\frac{850 \times 4.53^2}{2 \times 5.46}$ = 1,595 kg

do, at S_2 only.

72 · 1.20 · $\frac{5.13}{5.46}$ = 80 kg.

Summary of concentration on floor beam

	at S_1	at S_2
Wheel load	6020	3850
Unif. load	1595	1595
	—	80
	7615 kg	5525 kg



Shear = 9333 kg.

moment =	9333 · 3.40 =	31,750	
	5525 · 1.70 = −	9385	
		22,365	kgm

Summary for moments and end shears.

	moments	end shears.
Dead Load	10,110	4578
Live Load	22365	9333
	32475 kgm	13911 kg

Design of Sendai Bashi for Tottori Ken

Try web 800 × 9 = 72.0 cm^2 1/8 web area = 9.0 cm^2 depth 81.0 cm b to b Ls.
Effective depth 81.0 − 3.4 = 77.6 cm
Flange stress = $\frac{32475 \times 100}{77.6}$ = 41,900 kg T or C

Bottom flange area required = $\frac{41900}{1200}$ = 34.90 cm^2 net
1/8 web area = − 9.00
25.90 net

Use 2Ls 125 × 75 × 9 = 34.38 − 3.96 = 30.42 cm^2 net.

Top flange area required = $\frac{41900}{1100}$ = 38.10
1/8 web area = − 9.00
29.10 cm^2 gross.

Approximate weight of end floor beam.

1	Web pl.	800 × 9	@ 56.52	× 6.80	=	385
4	flange Ls	125 × 75 × 9	@ 13.50	× 6.66	=	360
2	Conn Ls	125 × 90 × 10	@ 16.10	× 0.62	=	20
14	stiff. Ls	90 × 75 × 9	@ 11.00	× 0.81	=	125
3	Stringer bed pls.	235 × 9	@ 16.60	× 0.26	=	13
	Rivet heads + variations	say	3½ %		=	32
						935 kg.

weight per lin meter of floor beam = 935 ÷ 6.8 = 138 kg.

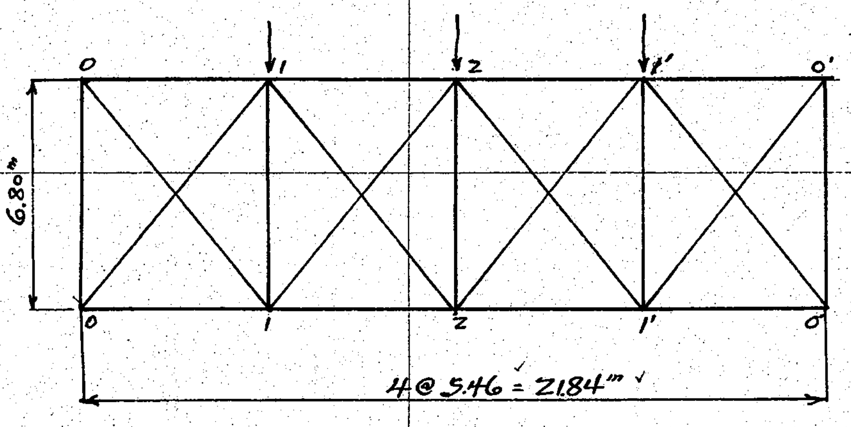
Design of Bottom Lateral bracings.
Dead Load
metal assumed.

Stringers	3 @ 75 =	225
floor beam	1010 ÷ 5.46 =	185
lateral bracing assumed		55
main girders	"	900
		1,365

Floor slab + pavement between girders 4.90 × 6.8 = 3330
overhanging slab, coping + handrails. 2 @ 580 = 1160
4490
5855 kg per lin m.

Panel concentration = 5855 × 5.46 = 32,000 kg
Seismic acceleration assumed 2000 mm/sec² or k = 0.200
Seismic panel load = 32000 × 0.20 = 6,400 kg

Wind load assumed 600 kg per lin meter of bridge
wind panel load = 600 × 5.46 = 3280 kg.



Diagonal Length
6.80^2 = 46.2400
5.46^2 = 29.8116
76.0516
$\sqrt{76.0516}$ = 8.721 m
Coefficient = $\frac{8.721}{6.80}$ = 1.283

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Design of Sendai Bashi for Tottori Ken.

Seismic stresses on lateral bracings
Shear in panel 0-1 = 6400 × 1.5 = 9600 kg   Stress on member 0-1 = 9600 × 1.283 = <u>12330</u> kg T
Shear " " 1-2 = 6400 × 0.5 = 3200 "   " " " 1-2 = 3200 × " = 4110 "

Wind stresses on lateral bracings (as a moving load)
Shear in panel 0-1 = 3280 × 1.5 = 4920 kg   Stress in member 0-1 = 4920 × 1.283 = 6310 kg T
" " " 1-2 = 3280 × 3/4 = 2460 "   " " " 1-2 = 2460 × " = <u>3160</u> "

Note:- Figures underlined are the governing stresses of corresponding members.

Sectional area required for member 0-1 = $\frac{12330}{1200 \times 1.60}$ = 6.43 cm² net

Use 1L 100 × 100 × 10 = 19.00 − 2.50 = 16.50 cm² net.

22 mmφ rivet no. required = $\frac{12330}{2851 \times 1.60}$ = 2.70 use 3 rivets

Use same section for member 1-2 as 0-1.

All intersection points of diagonal members shall be suspended from stringers to increase their vertical stiffness in order to avoid vibration.

Approximate weight of lateral bracings.

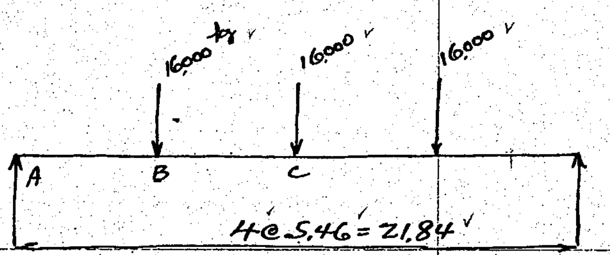
Diagonals.	8 Ls	100 × 100 × 10	@ 14.90	× 8.15	= 972
Center conn.	4 @		25.00		= 100
Side "	10 @		15.00		= 150
Rivet heads + variation say 3½ %					43
					1265 kg.

weight per meter = 1265 ÷ 21.84 = 58 kg.

Design of Main Girders   Span length 4 panels @ 5.46 = 21.84 meters, Spacing 6.8 m c to c.
For simplicity sake, let us assume all dead loads to be concentrated at panel points
Dead Load.

Floor slab + pavement between main girders	6.8 @ 490 =	3330	
Overhanging slab, coping and handrail	2 @ 580 =	1160	
			4490
Stringers	3 @ 75 =	225	
floor beams	1010 ÷ 5.46 =	185	
lateral bracings		58	
main girder assumed		900	
			1368
			5858

panel concentration = 5858 × $\frac{5.46}{2}$ = 16000 kg on one girder.



End panel load say 5858 × $\frac{3.0}{2}$ = 8800 kg on one girder

End shear = 16000 × 1.5 = 24000 kg
load direct on main girder 1240 × 2.73 = 3400 ---- main girder, handrail, coping etc
27400 kg.

moment at B = 24000 × 5.46 = 131000 kgm
moment at C
24000 × 10.92 = 262000
16000 × 5.46 = − 87400
174600 kgm

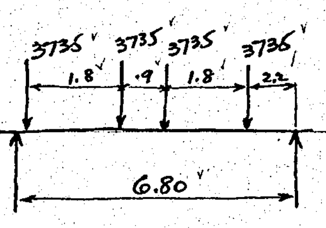
Dead load on shoe = 24000 + 8800 = 32800 kg

Design of Sendai Bashi for Tottori Ken.

Live load :—

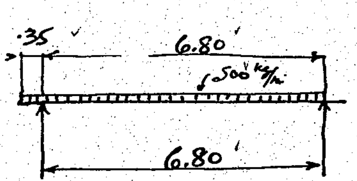
Uniform live load = 500 kg/m²
motor truck rear wheel concentration = 3000
Impact coef. = $\frac{20}{60+21.84}$ = 24.5% = 735
3735 kg
front wheel with impact = 3735 ÷ 3 = 1245 "

All live loads directly on main girder assumed.



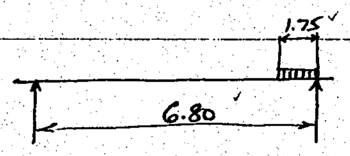
wheel load concentration on main girder.
rear wheels. 3735 × 2.20 = 8250
" × 4.00 = 14950
" × 4.90 = 18300
" × 6.70 = 25000
66500 ÷ 6.8 = 9800 kg.

front wheels. 9800 ÷ 3 = 3270 kg.



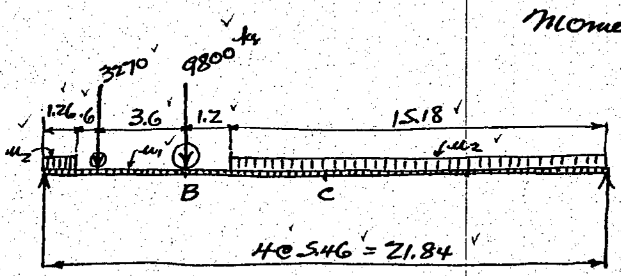
Uniform load on front and rear of truck
$\frac{500 \times 7.15^2}{2 \times 6.80}$ = 1880 kg per lin m of girder.

uniform load on sides of truck.



$\frac{500 \times 1.75^2}{2 \times 6.8}$ = 110 kg " " " = u_1
1880 − 110 = 1770 " " " " = u_2

moment at B.



Reaction due to wheel loads.
9800 × 16.38 = 160500
3270 × 19.98 = 65400
225900 ÷ 21.84 = 10340 kg.

moment =
10340 × 5.46 = 56500
3270 × 3.60 = −11,800
44,700 kgm

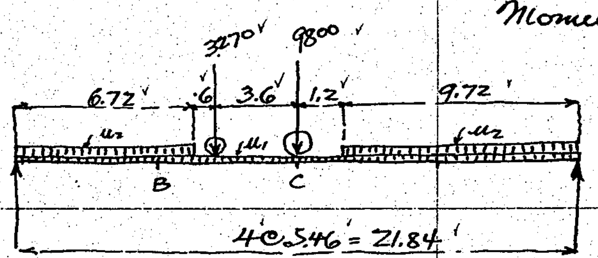
Reaction due to unif. load u_2
$\frac{1770 \times 15.18^2}{2 \times 21.84}$ = 9350
$\frac{1770 \times 1.26 \times 21.21}{21.84}$ = 2170
11,520 kg.

moment
11,520 × 5.46 = 62900
1770 × 1.26 × 4.83 = − 10,770
52,130 kgm

Reaction due to unif. load u_1
110 × 10.92 = 1200 kg   moment 1200 × 5.46 = 6550
$110 \times 5.46^2 \div 2$ = − 1640
4,910 kgm

Total live load moment at B = 101,740 kgm

Moment at C.



Reaction due to wheel load.
9800 ÷ 2 = 4900
3270 × 7.32 ÷ 21.84 = +1100
6,000 kg.

moment = 6000 × 10.92 = 65,500 kgm

Reaction due to unif. load u_2
$\frac{1770 \times 9.72^2}{2 \times 21.84}$ = 3830
$\frac{1770 \times 6.72 \times 18.48}{21.84}$ = 10050
13,880 kg.

moment = 13880 × 10.92 = 152,000
1770 × 6.72 × 7.56 = − 89,900
62,100 kgm

moment due to u_1. $\frac{110 \times 21.84^2}{8}$ = 6560
Total live load moment at C = 134,160 kgm

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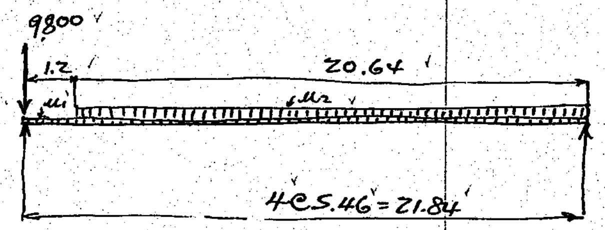
Design of Sendai Bashi for Tottori Ken.

Max. End shear.



Rear wheel		=	9800
unif. load	w_2 $\frac{1770 \times 20.64^2}{2 \times 21.84}$	=	17,250
unif. load	w_1 110×10.92	=	1200
			28250 kg

Summary for Moments and Shears.

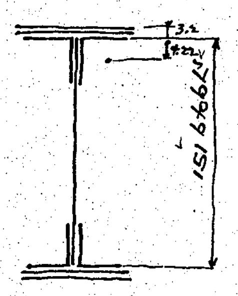
	moment at B	moment at C	end shear	load on shoe
Dead Load	131000	174600	27400	32800
Live Load	101,740	134,160	28,250	28,900
	232,740 kgm	308,760 kgm	55,650 kg	61,700 kg

Live Load on Shoe.

rear wheel	$9800 \times \frac{22.11}{21.84}$ =	9930	
w_2	$\frac{1770 \times 20.91^2}{2 \times 21.84}$ =	17,730	
w_1	$\frac{110 \times 22.11^2}{2 \times 21.84}$ =	1240	
		28,900 kg	

Assumed section at C

web pl. 1500 × 9 = 135 cm² 1/8 web area = 16.9 cm²



2 Ls	150 × 150 × 15 =	85.48 −	15.00 =	70.48
2 Pls.	340 × 16 =	108.80 −	16.00 =	92.80
		194.28 cm² gr.		163.28 cm² net

Effective depth = 151.0 − 0.96 × 2 = 149.08 cm

Flange stress = $\frac{308,760}{1.4908}$ = 207,500 kg T or C.

85.48 × 4.22 = −360.5
108.8 × 1.60 = +174.2
194.28 × .96 = −186.3

Bottom flange area required = $\frac{207,500}{1200}$ = 173.00
1/8 web area = − 16.90
150.10 cm² net.

unit tension $= 1200 \times \frac{173.00}{163.28 + 16.9} = 1150$ kg/cm²

Top flange area required =
$\frac{207,500}{1100}$ = 188.50
1/8 web area = − 16.90
171.60 cm² gross.

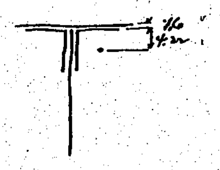
unit compression $= 1100 \times \frac{188.50}{194.28 + 16.9} = 980$ kg/cm²

max. unit shear = $\frac{55650}{135}$ = 413 kg/cm².

Assumed section is ample.

Section at end of 2nd cover plate. See next page. moment = 232740 kgm

Effective depth = 151.0 − 2.27 × 2 = 146.46 cm



Section

2 Ls	150 × 150 × 15 =	85.48 −	15.00 =	70.48
1 Pl.	340 × 16 =	54.40 −	8.00 =	46.40
		139.88 gr.		116.88 cm² net.

85.48 × 4.22 = −360.5
54.40 × .8 = 43.5
139.88 × 2.27 = −317.0

Flange stress = $\frac{232740}{1.4646}$ = 159000 kg T or C

Unit tension = $\frac{159000}{116.88 + 16.9}$ = 1190 kg/cm² < 1200

Unit compression = $\frac{159000}{139.88 + 16.9}$ = 1015 " < 1100

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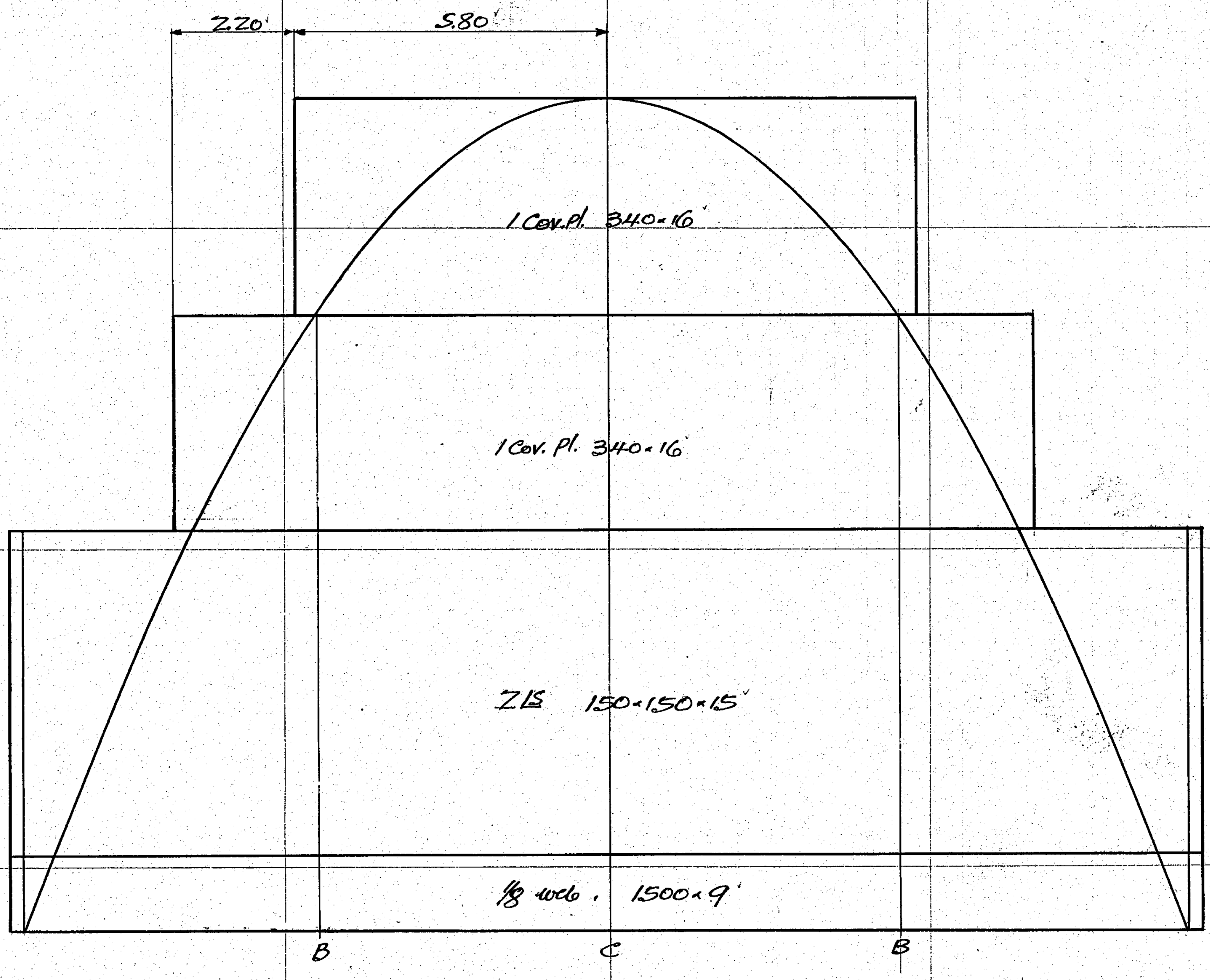
MADE BY K. Inaba DATE 6-8-10 FILE NO
CHECKED BY M. Jinda DATE 6-8-25 PAGE NO. 12

CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.
Bending moment diagram of main girder



Scale of space 1:100
Scale of moment 1/20" = 100,000 kgm.

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Design of Sendai Bashi for Tottori Ken.

Approximate weight of main Girder.

Flange	4Ls	150 × 150 × 15	@ 33.60	× 22.34	= 3,005
web pl.	1Pl.	1500 × 9	@ 105.98	× 22.34	= 2,365
Cov. pl.	2Pls.	340 × 16	@ 42.70	× 16.00	= 1,366
"	2Pls	340 × 16	@ "	" 11.60	= 991
end stiff Ls	8Ls	125 × 90 × 13	@ 20.60	× 1.48	= 244
fills	4Pls	190 × 15	@ 22.37	× 1.20	= 107
int. stiffs	24Ls	125 × 90 × 9	@ 14.60	× 1.51	= 529
" "	6Ls	125 × 90 × 9	@ 14.60	× 1.48	= 130
fills	6Pls	90 × 15	@ 10.60	× 1.20	= 76
Spl. Ls	8Ls	150 × 150 × 19	@ 41.90	× 1.17	= 392
Spl. Pls	8Pls	235 × 14	@ 25.83	× 0.63	= 131
"	4Pls	330 × 9	@ 23.32	× 0.73	= 68
fills	4Pls	90 × 5	@ 3.53	× 0.73	= 10
Cov. spl. pl.	4Pls	340 × 16	@ 42.70	× 2.25	= 384
Conn. Ls	16Ls	100 × 75 × 10	@ 13.00	× 0.50	= 104
Sole pl.	2Pls	465 × 28	@ 102.21	× 0.485	= 99
		Rivet heads and variation say		3½ %	= 354
					10,355 kg or 10.36 kg tons.

weight per lin meter = 10,355 ÷ 22.34 = 463 kg

Summary of structural steel in one span.

Stringers	3 @ 74 × 22.34	= 4955
intermediate floor beams	3 @ 1010	= 3030
end " "	2 @ 935	= 1,870
lateral bracings complete		= 1265
main girders	2 @ 10,355	= 20700
Shoes + anchor bolts	4 @ 200	= 800
		32620 kg or 32.62 kg tons.

Total structural steel for the whole bridge

16 spans	@ 32.620	= 522.00
16 Exp. Jts.	@ 0.620	= 9.90
		531.90 kg tons

Call this 532 tons

Design of Sendai Bashi for Tottori Ken.

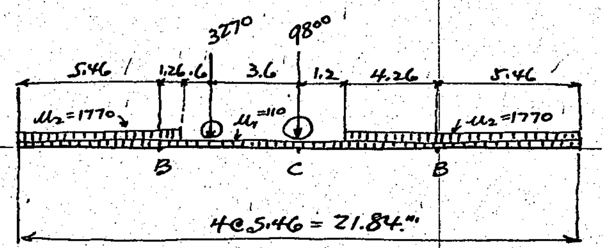
Deflection of main girder.

For simplicity sake, let us assume all the loads to be concentrated at panel points.

Dead load panel concentration

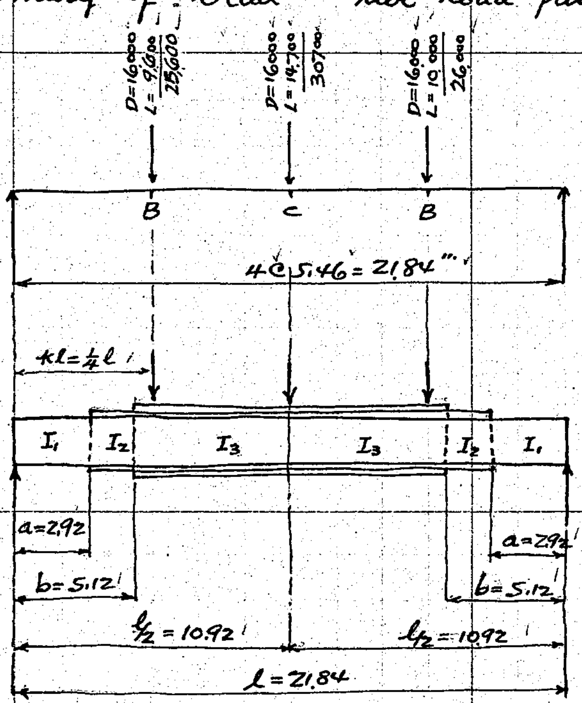
panel load on B.+C = 16000 kg each. See page 9.

Live load panel concentration. see page 10.



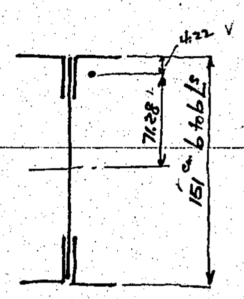
M_1	300	600	600	600	300
M_2	4835	4835		4835	4835
"		1973	257		
"			2940	4600	
wheel.			9800		
"		2157	1113		
	5135	9565	14710	10035	5135
Call this		9,600 kg	14,700	10,000	

Summary of Dead + Live load panel concentration



Moments of inertia at several sections.

for no. cover plate. I_1.



4Ls	150×150×15	889×4 + 85.48×71.28²×2	=	871,500
1Pl.	1500×9	$\frac{0.9 \times 150^3}{12}$	=	253,000
			I_1 =	1,124,500 cm⁴

for one cover plate I_2.

4Ls	150×150×15			871,500
1Pl.	1500×9			253,000
2Pls	340×16	108.80 × 76.3²	=	634,500
			I_2 =	1,759,000 cm⁴

for Two cover plates I_3.

		I_2 =	1,759,000
add 2Pls. 340×16	108.80 × 77.9²		660,000
		I_3 =	2,419,000 cm⁴

Deflection at center of span C due to a load P at any point on the second cover plate is

$$\Delta_C = \frac{P}{6E}\left(\frac{a^3}{I_1} - \frac{a^3}{I_2} + \frac{b^3}{I_2} - \frac{b^3}{I_3}\right) - \frac{Pk^3l^3}{12EI_3} + \frac{Pkl^3}{16EI_3}$$

$$= \frac{P}{6E}\left(\frac{a^3}{I_1} - \frac{a^3}{I_2} + \frac{b^3}{I_2} - \frac{b^3}{I_3}\right) + \frac{Pkl^3}{48EI_3}(3-4k^2)$$

$$= \frac{P}{6 \times 2{,}100{,}000}\left(22.16 - 14.16 + 76.30 - 55.50\right) + Pk(3-4k^2) \times 0.000043$$

(28.80)

$$= 0.00000229P + 0.000043\,Pk(3-4k^2)$$

$\frac{a^3}{I_1} = \frac{292^3}{1{,}124{,}500} = 22.16$

$\frac{a^3}{I_2} = \frac{292^3}{1{,}759{,}000} = 14.16$

$\frac{b^3}{I_2} = \frac{512^3}{1{,}759{,}000} = 76.30$

$\frac{b^3}{I_3} = \frac{512^3}{2{,}419{,}000} = 55.50$

$\frac{l^3}{48EI_3} = \frac{2184^3}{48 \times 2{,}100{,}000 \times 2{,}419{,}000} = 0.000043$

Design of Sendai Bashi for Tottori Ken.

For a concentration of 10000 kg at B. $k = 1/4$

$$\Delta_c = 0.0229 + 0.43 \times \frac{1}{4}\left(3-\frac{1}{4}\right) = 0.0229 + 0.296 = 0.3189 \text{ cm}$$

For a concentration of 10000 kg at C $k = \frac{1}{2}$

$$\Delta_c = 0.0229 + 0.43 \times \frac{1}{2}(3-1) = 0.0229 + 0.430 = 0.4529 \text{ cm}$$

Dead Load Deflection. Panel load 16000 kg on B+C.

$$\Delta_c = 0.3189 \times \frac{16000}{10000} \times 2 + 0.4529 \times \frac{16000}{10000} = 1.020 + 0.725 = 1.745 \text{ cm}$$

Live Load Deflection

Panel load on B = 9600 × 10000, 14700 on C

$$\Delta_c = 0.3189 \times 1.96 + 0.4529 \times \cancel{2.02}\ 1.47 = 0.625 + 0.666 = 1.291 \text{ cm}$$

Summary of Deflection at center of span.

Dead Load Deflection	=	1.745
Live Load "	=	1.291
		3.036 cm

$$\text{Deflection ratio} = \frac{\Delta_c}{l} = \frac{3.036}{2184} = \frac{1}{720}$$

Average moment of inertia

I_1	1124,500 ×	5.84	=	6570,000
I_2	1759,000 ×	4.40	=	7740,000
I_3	2419,000 ×	11.60	=	28050,000
		21.84		42360,000

$$\text{average } I = \frac{42360000}{21.84} = 1940000 \text{ cm}^4$$

Unif load

D.L. 5858 ÷ 2	=	2930
L.L.		1880
		4810 kg/m
Single con.		4300 kg at center

$$\text{Unif load } \Delta = \frac{5wl^4}{384EI} = \frac{5 \times 2184^4 w}{384 \times 2100000 \times 1940000} = 0.0726w \text{ cm}$$

$$\text{Con. } \Delta = \frac{Wl^3}{48EI} = \frac{2184^3 W}{48 \times 2100000 \times 1940000} = 0.0000532W$$

D.L. Deflection	= 0.0726 ×	29.30	=		2.127 cm
L.L. unif.	= 0.0726 ×	18.80	=	1.365	
" con.	= 0.0000532 ×	4300.	=	0.229	
					1.594
					3.721

average $I = 0.98 I_3 = 2370,000$ [illegible] Δ [illegible]

Design of Sendai Bashi for Tottori Ken.

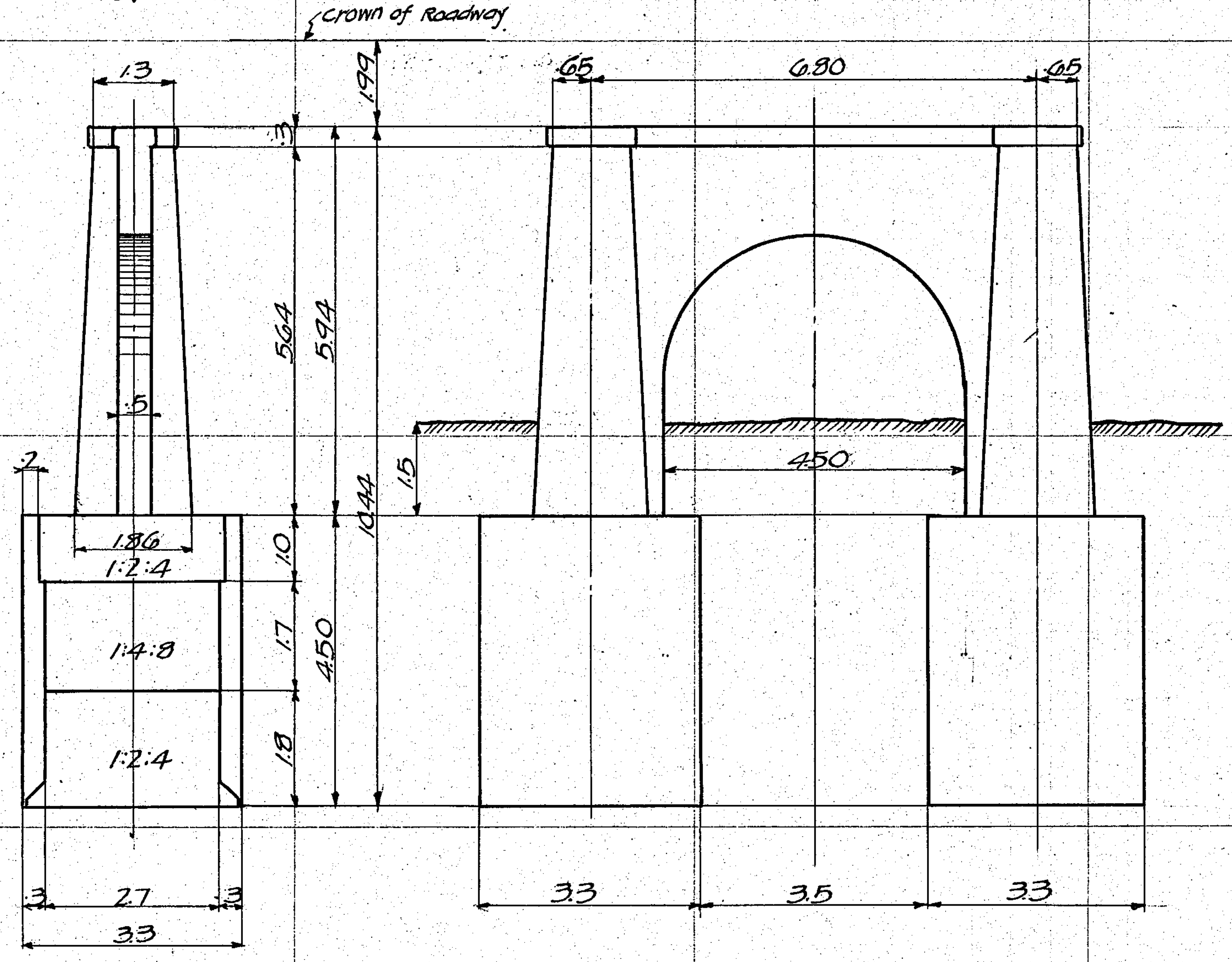
Design of Piers.

Superimposed loads on pier for one girder.

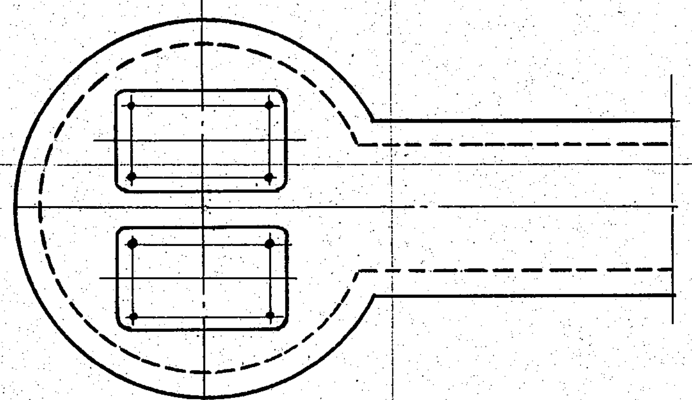
Dead Load		2 @ 32800	=	65600 kg
Live Load	say	2 @ 28900	=	57800 "
				123400 kg

Seismic force 0.2 × 65600 = 13120 kg 15 cm above top of pier

Pier no 13.



General sketch of Pier no. 13.
Scale 1:100



Detail of Top of Pier.
Scale 1:30

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Design of Sendai Bashi for Tottori Ken.

Weight and center of gravity of Pier

Pier shaft.

Coping	1.50φ × 0.30 × 2	= 1.06	× 5.79m =	6.14
〃	0.70 × 0.30 × 5.60	= 1.18	× 5.79 =	6.83
shaft	1.58φ × 5.64 × 2	= 22.11	× 2.50 =	55.30
wall	5.50 × 2.00 × 0.50	= 5.50	× 4.80 =	26.40
		29.85m3	× 3.17m	94.67

weight of $\frac{1}{2}$ of shaft = 14.93 @ 2400 = 35900 kg

for which seismic force = 35900 × 0.20 = 7,180 〃 3.17m above top of caisson

Caisson 3.30 × 3.30 × 4.50 = 49.00m3 @ 2200 = 107,800 kg

seismic force on caisson = 107800 × .20 = 21,560 〃 2.25m above bottom.

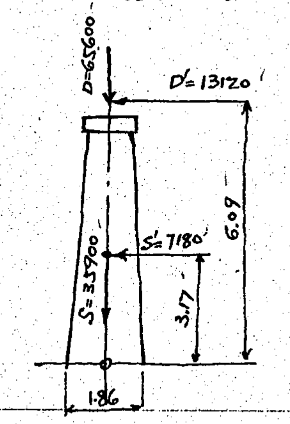
Stresses on shaft.

max. load on shaft = 123,400 kg on one column. see page 14.

unit compression at top $= \frac{123400}{130^\phi} = \frac{123400}{13273} = 9.3$ kg/cm² C.

〃 〃 〃 bottom $= \frac{123400+35900}{186^\phi} = \frac{159300}{27172} = 5.9$ 〃 〃

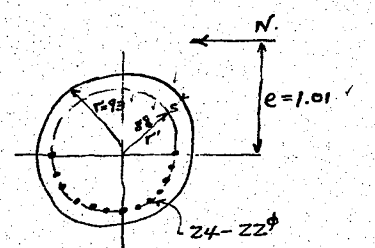
Seismic stresses on shaft. Taking moment about center of bottom area



Loads.	Hor forces	Vert. forces	lev. arms	moments.
D		65,600		
D'	13,120		× 6.09 =	79,800
S		35,900		
S'	7,180		× 3.17 =	22,800
	20,300 kg	101,500 kg	1.01m	102,600

$A_s = 24 @ 3.801 = 91.3$ cm²

$p = \frac{91.3}{93^2\pi} = 0.0034$



$N = 101{,}500$ kg

$r = 93^{cm}$, $r' = 88^{cm}$, $r'/r = \frac{88}{93} = .946$, $\frac{e}{r} = 101/93 = 1.087$

$k = 0.72$, $C = 0.360$

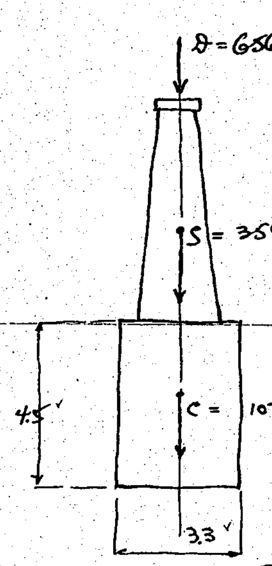
備考
柴田直光著
鐵筋混凝土の計算 ノモグラムに依る
参照

$f_c = \frac{N}{r^2C} = \frac{101500}{93^2 \cdot .360} = 32.6$ kg/cm²

$f_s = nf_c \frac{1+\frac{r'}{r}-k}{k} = 15 \cdot 32.6 \frac{1+.946-.72}{.72} = 834$ kg/cm²

Stability of Pier:

Case 1. Dead load only.



Superimposed Dead load = 65,600

weight of shaft. = 35,900

weight of caisson = 107,800

209,300 kg

friction assumed. 1200 kg/m²

= 3.3 × 4 × 4.5 @ 1200 = − 71,300

138,000 kg.

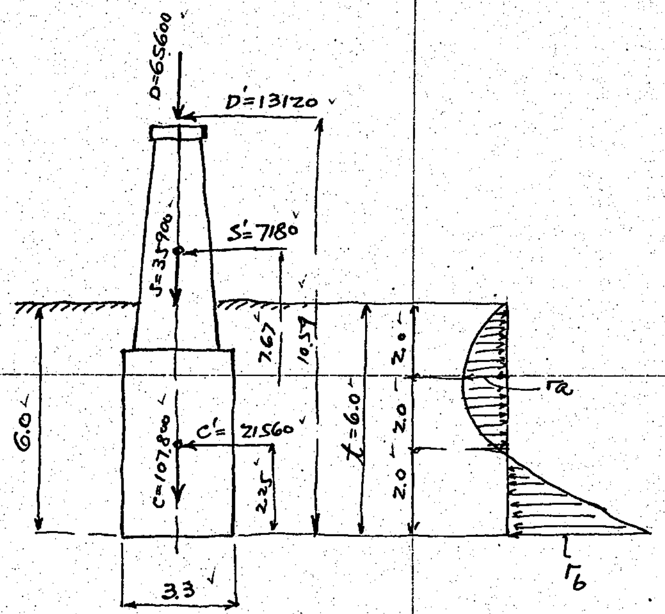
unit bearing pressure $= \frac{138000}{3.3 \times 3.3} = 12700$ kg/m² or (1.16 tons/□′)

Case 2 Full Loaded Total net load = 138000 + 57800 = 195800 kg.

unit bearing pressure $= \frac{195800}{3.3 \times 3.3} = 18000$ kg/m² or (1.65 tons/□′)

Design of Sendai Bashi for Tottori Ken.

Case 3. Stability during Earthquake, k assumed 0.20



moment due to seismic forces about center of base.

D'	13120	×	10.59 =	139000
S'	7180	×	7.67 =	55000
C'	21560	×	2.25 =	48500
	41860			242500 kgm

friction between earth and well assumed 1200 kg/m2

frictional couple = 1200 × 3.3 × 4.5 × 3.3 = 58800 kgm

net moment = 242500 − 58800 = 183700 kgm

unit pressure $r_b = \frac{12M}{bh^2} = \frac{12 \times 183700}{3.3 \times 36} = 18550$ kg/m2

" " $r_a = \frac{r_b}{3} = \frac{18550}{3} = 6180$ "

Passive earth pressure $p = wx \cdot \frac{1+\sin\phi'}{1-\sin\phi'}$, where $\phi' = \phi - \tan^{-1} k$,

ϕ = angle of repose at normal state

捨石ニ入レ $\phi = 45°$ ト假定ス

$\phi' = 45° - \tan^{-1} 0.20 = 33°-40'$ $\sin\phi' = .5544$

$\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.5544}{.4456} = 3.49$

Passive earth pressure at r_b = 1800 × 6.0 × 3.49 = 37700 kg/m2

" " " r_a = 1800 × 2.0 × 3.49 = 12570 "

Design of Caisson shell.

Temporary earth pressure on caisson

Reffered to Ketchum's Walls, Bins and Grain Elevators page 120 + 121

$$L = \frac{wb}{2\mu}\left(1 - e^{\frac{-2k\mu y}{b}}\right), \quad V = \frac{wb}{2k\mu}\left(1 - e^{\frac{-2k\mu y}{b}}\right)$$

where V = vertical unit pressure in kg/m2 at depth y
L = lateral " " " " "
w = weight of earth in kg/m3
$\mu = \tan\phi$ = coefficient of friction of earth on earth.
b = the distance in meter, that the earth breaks around the well.
k = a constant $\doteqdot \frac{1-\sin\phi}{1+\sin\phi}$
ϕ = angle of repose of earth.

For w = 1600, b = 3m, k = $\frac{1}{3}$

y	L	h	water pressure	Difference
1	490			490
2	970	0		970
3	1370	1	1000	370
4	1750	2	2000	−250
5	2050	3	3000	−950
6	2320	4	4000	−1680
7	2540	5	5000	−2460
8	2730	6	6000	−3270
9	2930	7	7000	−4070
10	3070	8	8000	−4930

max pressure on well.

Internal pressure of 1680 kg/m2 at bottom.

External " 1810 " at 1.8m above bottom.

moment on wall = $\frac{1}{10} \times 1810 \times 3.0^2 = 1630$ kgm

Effective depth required = $\sqrt{\frac{1630 \times 100}{100 \times 7.18}} = 15.1$ cm

use eff. depth 27cm with 3cm insulation

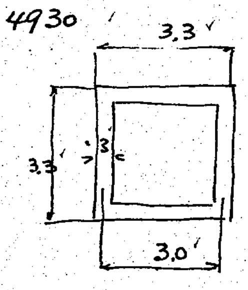
Steel area required = $\frac{1630 \times 100}{1200 \times \frac{7}{8} \times 27} = 5.75$ cm2

use 12mm φ bars at 15cm c to c = 7.54 cm2

spacing

2.0m from bottom 15cm spacing

2.5m " top 20~25 "



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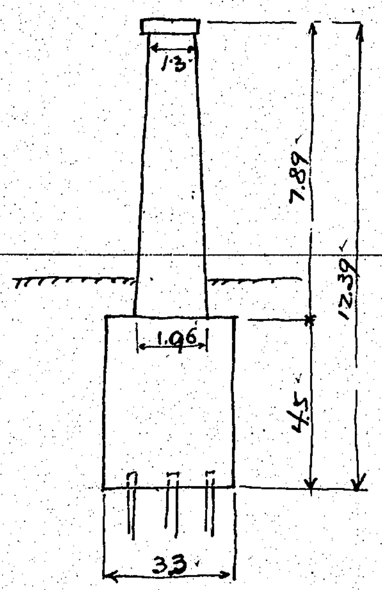
Design of Sendai Bashi for Tottori Ken.

Pier no. 8. Simillar details as for P13.
Height of shaft = 7.89 meters.



weight and center of gravity of pier
Pier shaft.

Coping	$1.5^{\phi} \times 0.30 \times 2$ =	1.06	×	7.74	=	8.20
"	$0.70 \times 0.30 \times 5.60$ =	1.18	×	7.74	=	9.14
shaft.	$\cancel{1.58}\ 1.63^{\phi} \times 7.59 \times 2$ =	31.66	×	3.19	=	101.00
wall	$5.50 \times 2.00 \times 0.5$ =	5.50	×	6.75	=	37.10
		39.40^{m^3}		3.94		155.44

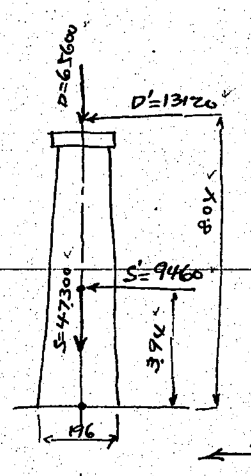
weight of $\frac{1}{2}$ of shaft = 19.70 @ 2400 = 47,300 kg
for which seismic force = 47,300 × 0.20 = 9460 "

Caisson same as for P13. 107,800 kg
seismic force = 21,560 "

Stresses on shaft.
max. load on shaft. 123,400 kg
Unit comp at top = $\frac{123400}{130^{\phi}}$ = 9.3 kg/cm²

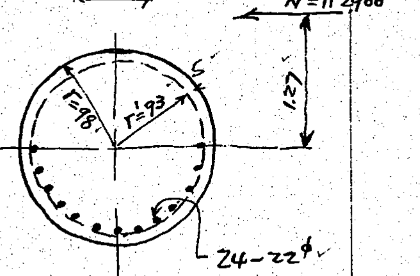
" " " bottom = $\frac{123400+47300}{196^{\phi}} = \frac{170700}{30172} = 5.7$ kg/cm²

Seismic stresses on shaft.



Taking moment about center of base.

Loads	Hor. forces	Vert. forces		lev. arms		moments.
D		65600	×	~~8.04~~		
D'	13120		×	8.04	=	105500
S		47300	×			
S'	9460		×	3.94	=	37300
	22580 kg	112900 kg		1.27 m		142800



$A_s = 24 - 22^{mm\phi} = 91.3^{cm^2}$

$p = \frac{91.3}{\pi \times 98^2} = .0030$ $N = 112900$ kg

$r = 98^{cm}$, $r' = 93^{cm}$ $r'/r = 93/98 = 0.95$, $e/r = 127/98 = 1.30$

from the prepared diagrams, we get.
$k = 0.640$, $C = 0.280$

$f_c = \frac{112900}{98^2 \times .280} = 42.0$ kg/cm² $< 35 \times 1.6 = 56$ kg/cm²

$f_s = 15 \times 42 \times \frac{1+.95-.64}{.64} = 1289$ kg/cm² $< 1200 \times 1.6 = 1920$ "

Stability of Pier
Case 1. Dead load only.

Superimposed Dead load =	65600	
weight of shaft	47,300	
weight of Caisson	107,800	
		220,700
friction 3.3 × 4 × 4.5 @1200 =		− 71,300
		149,400 kg

Dead load on pile
= 149400 ÷ 12 = 12.45 tons.

unit bearing pressure = $\frac{149400}{3.3 \times 3.3}$ = 13720 kg/m² or (1.255 tons/□')

Case 2. Full loaded. Total net load = 149400 + 57800 = 207200 kg.
unit bearing pressure = $\frac{207200}{3.3^2}$ = 18850 kg/m² or (1.723 tons/□')

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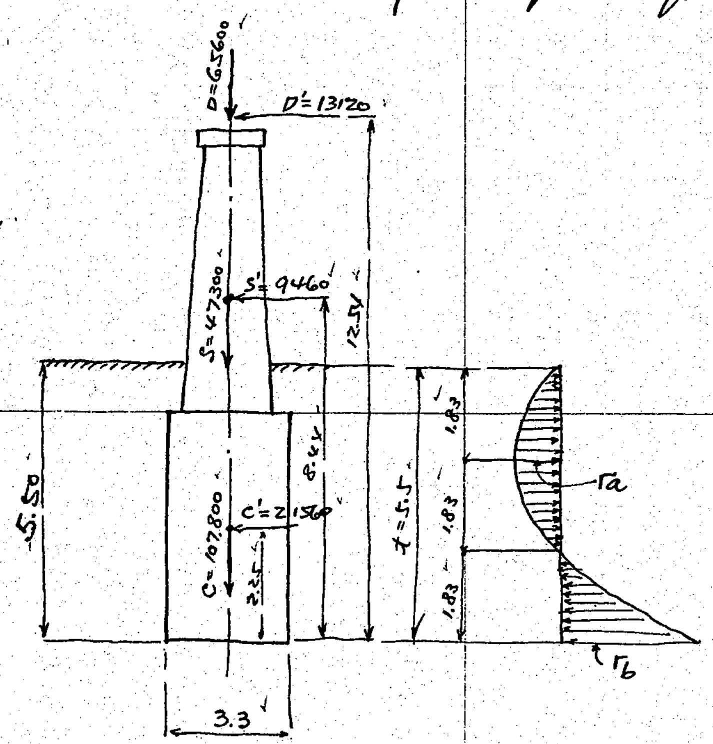
Design of Sendai Bashi for Tottori Ken.

Case 3. Stability during Earthquake.



moment due to seismic forces about centre of base.

D'	13120	×	12.54	= 164500
S'	9460	×	8.44	= 79800
C'	21560	×	2.25	= 48500
	44140 kg			292800

frictional couple − 58800

234000 kgm

unit pressure $r_b = \dfrac{12 \times 234000}{3.3 \times 5.5^2} = 28100$ kg/m²

" $r_a = 28100 \div 3 = 9370$ "

passive earth pressure at r_a = 1800 × 1.83 × 3.49 = 11500 kg/m²

" " " r_b = 1800 × 5.5 × 3.49 = 34550 "

Design of Caisson shell.

Temporary side pressure on well.

y	L	water pressure	Difference
1	490	1000	−510
2	970	2000	−1030
3	1370	3000	−1630
4	1750	4000	−2250
5	2050	5000	−2950
5.5	2185	5500	−3315

max. pressure on well.

Internal pressure at bottom = 3315 kg/m²

External " at 1.8m above bott. = 1635 "

~~Eff. depth required = $\sqrt{3315 \times 1}$~~

moment on wall = $\frac{1}{10} \times 3315 \times 3.0^2 = 2985$ kgm

Eff. depth required = $\sqrt{\dfrac{2985 \times 100}{100 \times 7.18}} = 20.4$ cm

use eff. depth of 27 cm with 3 cm insulation

Steel area required = $\dfrac{2985 \times 100}{1200 \times \frac{7}{8} \times 27} = 10.53$ cm²

use 12 mm φ bars at 10 cm c.to c. = 11.30 "

At section 1.5 m from bottom

moment = $\frac{1}{10} \times 2250 \times 3.0^2 = 2025$ kgm

Steel area required = $\dfrac{2025 \times 100}{1200 \times \frac{7}{8} \times 27} = 7.15$ cm²

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

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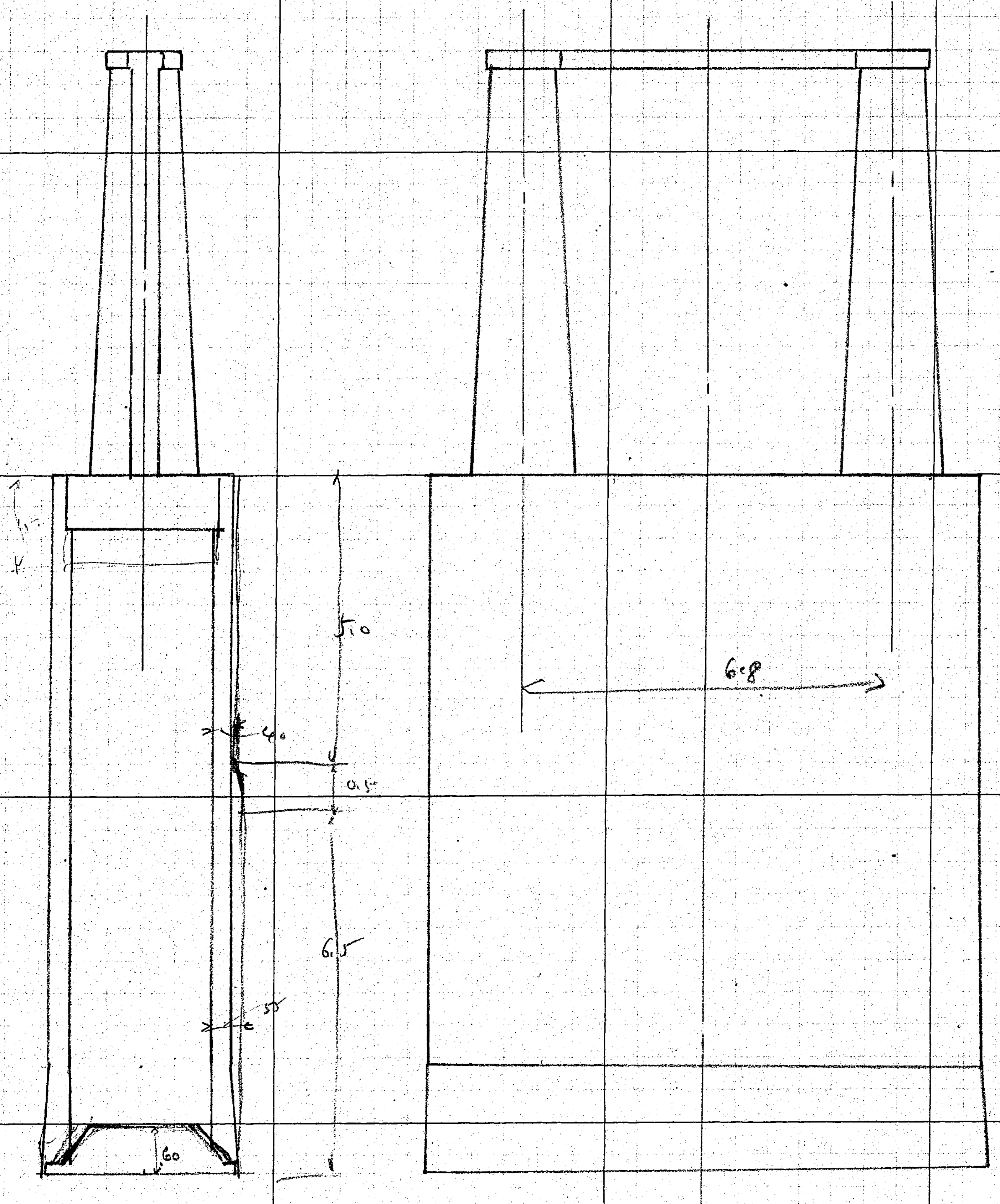

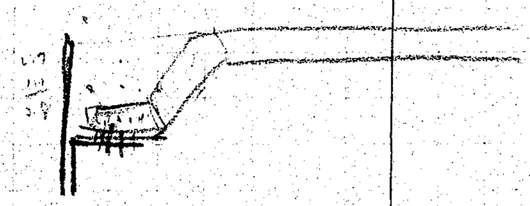

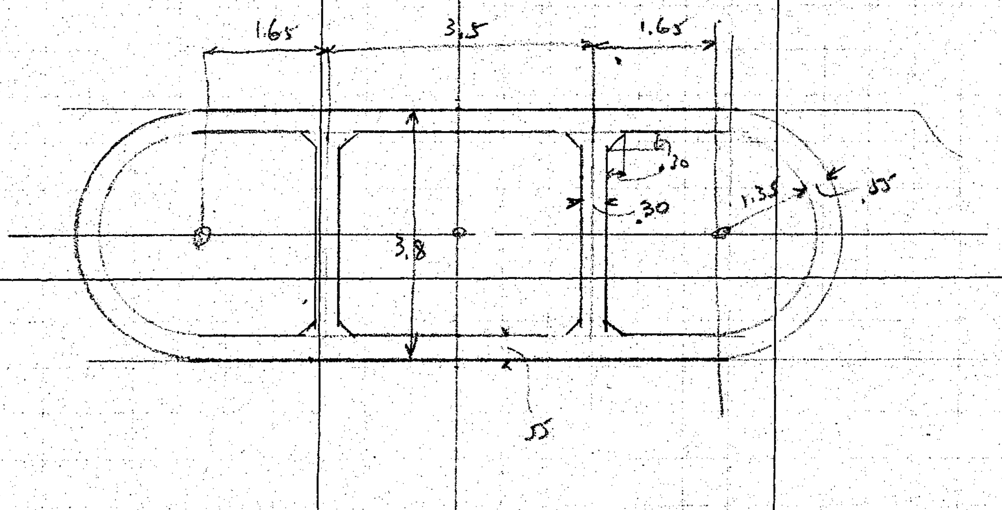

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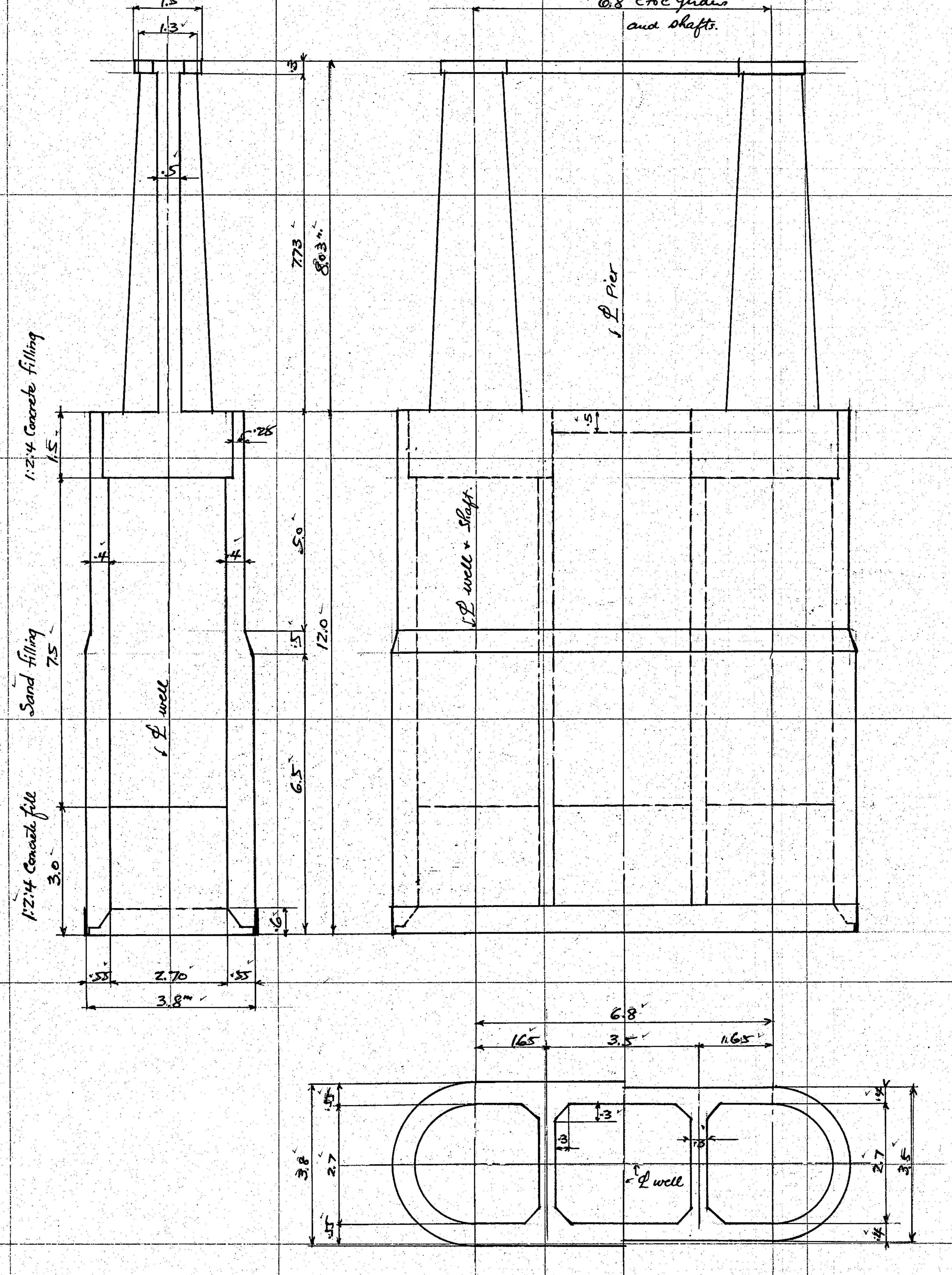
Design of Sendai Bashi for Tottori Ken.

Design of Pier no 2.

General type and dimensions as shown on the following sketch.



General sketch of Pier P.2.
scale 1:100.

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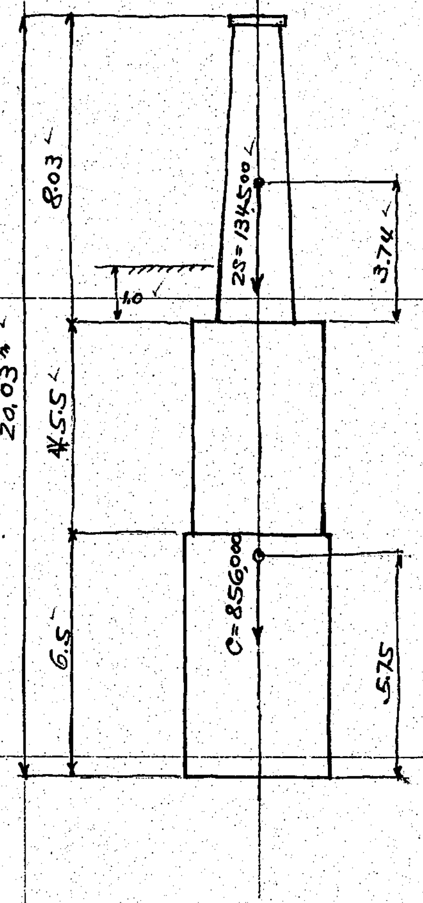
CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.

Pier no 2.



weight and center of gravity of pier.

Pier Shaft.

Coping	1.5⌀ × 0.30 × 2	= 1.06	× 7.88	= 8.35
"	0.70 × 0.30 × 5.60	= 1.18	× 7.88	= 9.30
Shaft.	1.64⌀ × 7.73 × 2	= 32.60	× 3.35	= 109.20
wall	5.50 × 7.73 × 0.50	= 21.25	× 3.90	= 83.00
		56.09	3.74m	209.85

weight of shaft = 56.09 @ 2400 = 134500 kg

for $\frac{1}{2}$ of shaft = 67300 = S

Seismic force 67300 × 0.20 = 13,500 = S'

Caisson.

top 1.5m.

Cross section 6.80 × 3.50 × ~~1.5~~ = 23.80

3.50⌀ = 9.60

33.40 ⌀m × 1.5 = 50.10 m³

next 4.0m

33.40

6.20 × 2.70 = −16.75

2.70⌀ = −5.72 } −22.11

fillet .3 × .3 × 4 = + .36

11.29 ⌀m × 4.0 = 45.20 m³

Sand fill. 22.11 × 4.0 = 88.4 m³

next 3.5m

6.80 × 3.80 = 25.85

3.80⌀ = 11.34

37.19

− 22.11

15.08 × 3.5 = 52.80 m³

Sand fill 22.11 × 3.5 = 77.40

bottom 3.0m

37.19 × 3.00 = 111.50

weight and center of gravity of Caisson.

	Concrete	Sand					
top 1.5m	50.10		@ 2200 =	110,200	× 11.25	=	1,240,000
next 4.0m	45.20		@ 2400 =	108,500	× 8.50	=	922,000
"		88.40	@ 1600 =	141,500	× "	=	1,203,000
next 3.5	52.80		@ 2400 =	126,800	× 4.75	=	602,000
"		77.40	@ 1600 =	123,800	× "	=	588,000
bottom 3.0	111.50		@ 2200 =	245,200	× 1.50	=	368,000
	259.60 m³	165.80 m³		856,000 kg.	5.75m		4,923,000

Seismic Stresses on shaft.

Loads	Hor. forces.	Vert. forces	Lever. arms	Moments.
D.		65,600		
D'	13120		× 8.18 =	107,200
S		67300		
S'	13500		× 3.74 =	50,500
	26620 kg	132900 kg.	1.19m	157700 kgm.

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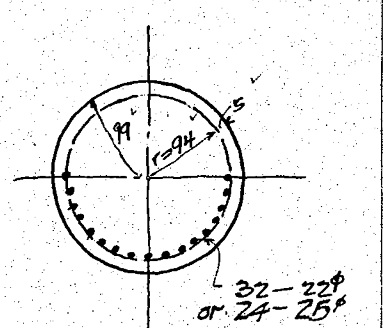
CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.



$A_s = 32\text{-}22^{mm\phi} = 121.7\ cm^2$ or $24\text{-}25^{mm\phi} = 117.8^{cm^2}$

$P = \frac{117.8}{\pi \times 99^2} = .0038$ $N = 132900\ kg$

$r = 99^{cm}$, $r' = 94^{cm}$, $r'/r = .95$ $e/r = 119/99 = 1.20$

from the prepared diagrams.

$k = 0.700$, $C = 0.33$

$f_c = \frac{N}{Cr^2} = \frac{132900}{0.33 \times 99^2} = 41.1\ kg/cm^2 < 35 \times 1.6 = 56.0\ kg/cm^2$

$f_s = nf_c \frac{1 + \frac{r'}{r} - k}{k} = 15 \times 41.1 \frac{1 + .95 - .70}{.70} = 1100\ kg/cm^2$

Assumed section is ample.

Stability of Pier

Case 1. Dead Load only.

Superimposed Dead Load	= 2 @ 65600 =	131200
weight of shaft	=	134500
weight of caisson	=	856000
		1121700
friction 25.55 m × 12.0 m @ 1200	=	− 369000
		752700 kg

bottom area of caisson = $37.19\ m^2$

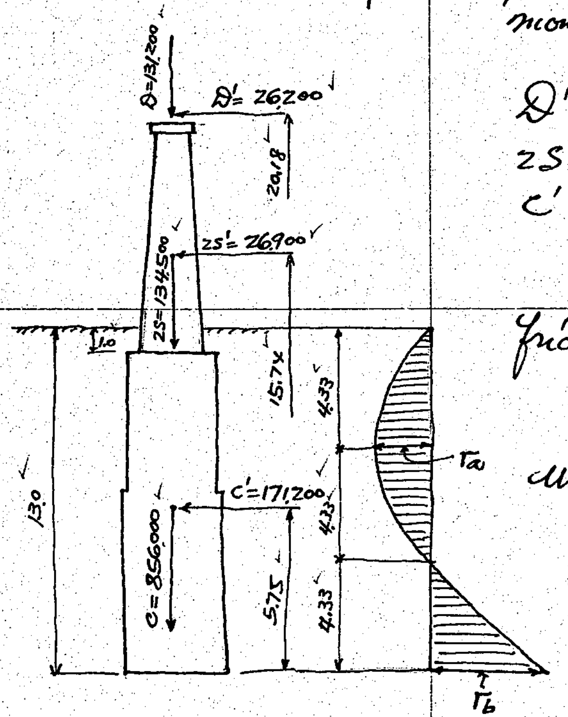
Unit bearing pressure = $\frac{752700}{37.19} = 20250\ kg/m^2$ or (1.85 tons/□')

Case 2. Full loaded.

Total net D.L.	=	752700
L.L.	=	115600
		868300 kg.

Unit bearing pressure = $\frac{868300}{37.19} = 23350\ kg/m^2$ or (2.14 tons/□')

Case 3. Stability during Earthquake.



moment due to seismic forces about center of base area.

D'	26200	×	20.18	=	528000
2S'	26900	×	15.74	=	423000
C'	171200	×	5.75	=	985000
	224300				1936000

frictional couple 9.4 × 12.0 × 1200 × 3.65 = 494000

1442000 kgm = M.

Unit pressure $r_b = \frac{12M}{bt^2} = \frac{12 \times 1442000}{9.4 \times 13^2} = 10900\ kg/m^2$

" " $r_a = \frac{1}{3} r_b = \frac{10900}{3} = 3640$ "

passive earthpressure at r_a

$\phi' = \phi - \tan^{-1} k = 30° - \tan^{-1} 0.200 = 18° - 40'$ $\frac{1 + \sin\phi'}{1 - \sin\phi'} = 1.94$

$1800 \times 4.33 \times 1.94 = 13450\ kg/m^2$ at r_a. ✓

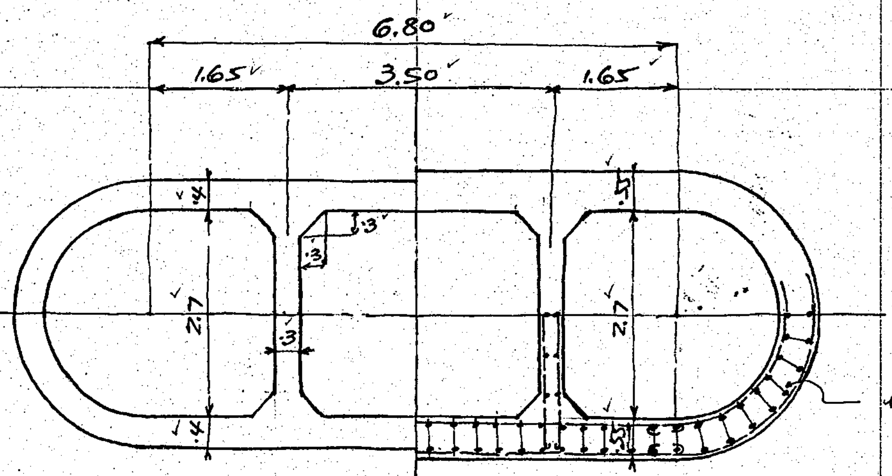
$1600 \times 13.0 \times 1.94 = 40300$ " at r_b. ✓ all right.

Design of Sendai Bashi for Tottori Ken.

Design of Caisson for Piers P_1 + P_2.

Temporary side pressure on wall of caisson.

Depth of earth + water	Side pressure	Inside water pressure	Outside water pressure. 25% say.	Total pressure on wall in cases of water filled	empty
1 m	+ 490	− 1000	+ 250	− 260	+ 740 kg/m²
2	+ 970	− 2000	+ 500	− 530	+ 1470
3	+ 1370	− 3000	+ 750	− 880	+ 2120
4	+ 1750	− 4000	+ 1000	− 1250	+ 2750
5	+ 2050	− 5000	+ 1250	− 1700	+ 3300
6	+ 2320	− 6000	+ 1500	− 2180	+ 3820
7	+ 2540	− 7000	+ 1750	− 2710	+ 4290
8	+ 2730	− 8000	+ 2000	− 3270	+ 4730
9	+ 2930	− 9000	+ 2250	− 3820	+ 5180
10	+ 3070	− 10000	+ 2500	− 4430	+ 5570
11	+ 3210	− 11000	+ 2750	− 5040	+ 5960
12	+ 3350	− 12000	+ 3000	− 5650	+ 6350
13	+ 3440	− 13000	+ 3250	− 6310	+ 6690



vert bars in sidewall 156 − 12φ
" " Partition wall 20 − " "
176 − 12φ.

Upper portion (5.5 m), Lower portion (6.5 m)

note. l = 3.5 m D = 3.10 m or 3.25 m.

Moment on 1 m horizontal strip of wall. and required steel area for ~~that~~ strip.

Depth	max. pressure	moments: Straight wall $wl^2 \div 12$	Circular end $wD^2 \div 16$	Effective depth req'd. straight wall. Eff. depth	Total depth	Steel area req'd. Straight wall	use Spacing 12 mmφ	16 mmφ
1	740 kg/m²	755 kgm	440 kgm	10.3 cm	use 25	3.1	30 cm.	
2	1470	1500	880	14.5	40	3.6	"	
3	2120	2160	1270	17.4	"	5.2	20	
4	2750	2800	1650	19.8	"	6.8	"	
5	3300	3370	1980	21.7	"	8.2	15	
6	3820	3900	2290	23.3	"	9.4	12	
7	4290	4380	2830	24.6	55	7.5	12	25 cm
8	4730	4820	3120	26.0	"	8.3		25
9	5180	5280	3420	27.3	"	9.1		20
10	5570	5680	3680	28.2	"	9.8		"
11	5960	6080	3930	29.1	"	10.5		"
12	6350	6480	4190	30.0	"	11.2		17
13	6690	6820	4410	30.8	"	11.8		17

note
average steel ratio 0.00195 for which j = .930

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Design of Sendai Bashi for Tottori Ken.

Vertical reinforcements of the caisson.

Lower 1/3 of shell is assumed to be suspended freely during sinking work.

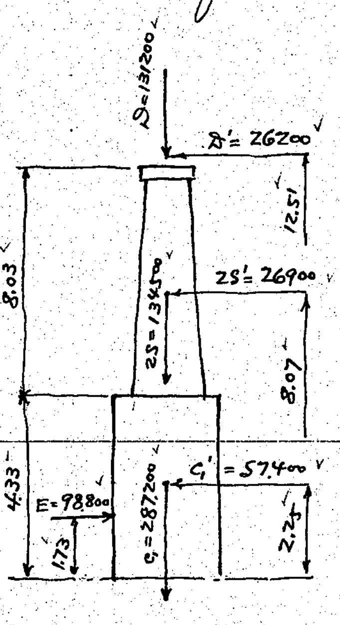
Weight of Suspended portion.

weight $15.08^{m^2} \times 4.0^{m}$ @2400 = 144,800 kg.

Steel area required = $\frac{144,800}{1200} = 120.6^{cm^2}$

Use more than 107 − $12^{mm\phi}$ bars. = 120.6^{cm^2}

Vertical reinforcements required for Bending moment on the shell.



moment on caisson at ra.

weight of Caisson 4.33m.

Top 1.5m		110,200	× 3.58	=	395,000
next 2.83m	$250000 \times \frac{2.83}{4}$ =	177,000	× 1.42	=	251,000
		287,200	2.25		646,000

resisting earth pressure of ra.

$3640 \times 9.4 \times 4.33 \times \frac{2}{3}$ = 98,800 kg. arm. $4.33 \times \frac{4}{10} = 1.73^{m}$

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
D		131,200		
D'	26,200		× 12.51 =	328,000
2S		134,500		
2S'	26,900		× 8.07 =	217,000
G		287,200		
G'	57,400		× 2.25 =	129,000
E	−98800		× 1.73 =	−171,000
	~~209300~~ −11,700 kg	552,900 kg		503,000 kg-m

Direct compression = $\frac{552900}{11.29 \times 100^2} = 4.9$ kg/cm²

Bending stress $= \pm \frac{503,000}{3.10 \times 9.8 \times 100^2 \times .4} = \pm 4.1$ "

9.0 kg/cm² C

or 0.8 " C

No. reinforcement required theoritically.

Use $12^{mm\phi}$ bars at 30cm c to c about on both sides of wall.

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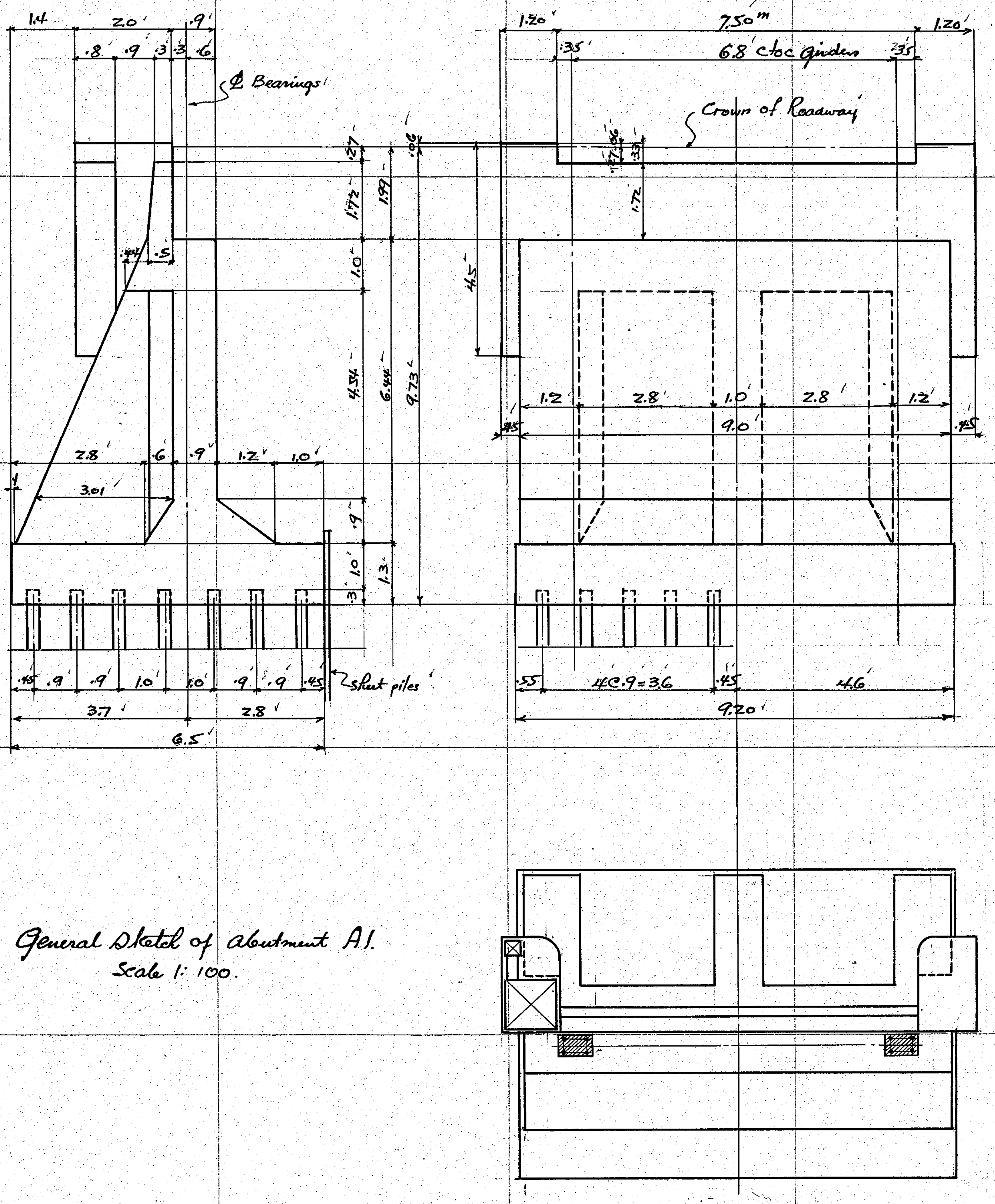
Design of Sendai Bashi for Tottori Ken.

Design of West abutment A1.

General dimensions of abutment as shown on sketch below.



General Sketch of abutment A1.
Scale 1: 100.

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CHECKED BY M. Iinda DATE 6-8-31 PAGE NO 22.

CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken

Superimposed Loads on abutment.

	on one shoe	on 2 shoes.
Dead Load	32800	65600
Live Load.	28900	57800
	61,700 kg.	123,400 kg
Seismic force	6,560 kg	13,120 kg

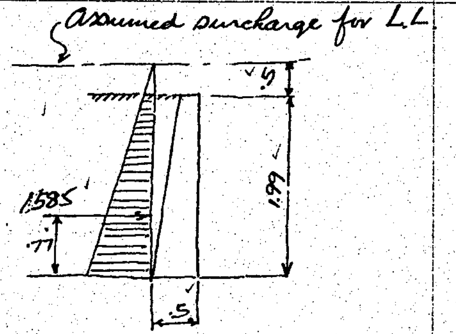
Design of Parapet wall.

Case 1 Normal state. Earth pressure on wall.

Pressure at top $1/3 \times 1600 \times .5 = 267$

" " bottom $1/3 \times 1600 \times 2.49 = 1327$

$1594 \times \frac{1.99}{2} = 1585$ kg. per meter strip.



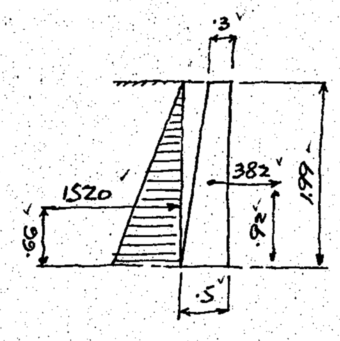
moment on wall = $1585 \times .77 = 1,220$ kgm.

Shear on wall = 1585 kg.

Case 2. During Earthquake.

Earth pressure during earthquake. k assumed 0.200

$= 0.480 \times \frac{1600 \times 1.99^2}{2} = 1520$ kg per meter strip



Seismic force due to weight of wall.

$0.40 \times 1.99 \times 2400 \times 0.2 = 382$ kg per meter strip.

moment on wall.

$1520 \times .660$	=	1004
$382 \times .92$	=	352
1902 kg		1356 kgm

Shear on wall = 1902 kg.

Stresses at normal state governs the section.

Effective depth required $= \sqrt{\frac{1220 \times 100}{100 \times 7.18}} = 13.0$ cm

use 47 cm effective depth with 3 cm insulation

Steel area required $= \frac{1220 \times 100}{1200 \times \frac{7}{8} \times 47} = 2.47$ cm² per meter strip.

use 12 mm φ bars at 30 cm ctoc = 3.78 cm²

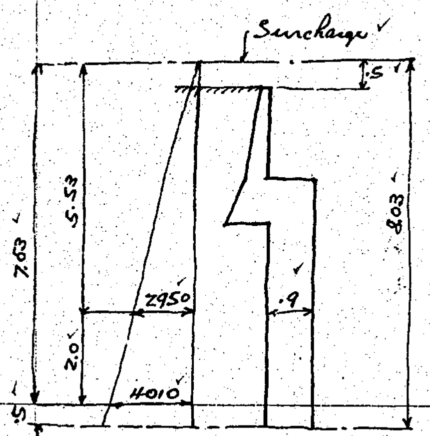
Design of Front wall.

Case 1. Normal state. Earth pressure on bottom 1 meter strip Depth = 7.03 + .5 = 7.53 m

$1/3 \times 1600 \times 7.53 = 4,010$ kg/m²

moment $= 1/10 \times 4010 \times 3.90^2 = 6100$ kgm.

shear $= 1/2 \times 4010 \times 2.80 = 5620$ kg



Section at 2.5 m above bottom.

Earth pressure Depth = 5.53

$1/3 \times 1600 \times 5.53 = 2,950$ kg/m²

moment $= 1/10 \times 2950 \times 3.90^2 = 4490$ kgm

shear $= 1/2 \times 2950 \times 2.80 = 4130$ kg

Case 2. During Earthquake Earth pressure & on bottom 1 m strip Depth = 7.03 m

$0.48 \times 1600 \times 7.03 = 5400$ kg/m²

Seismic force $0.90 \times 2400 \times 0.2 = 432$

5,832 kg/m²

moment $= 1/10 \times 5832 \times 3.90^2 = 8,870$ kgm per meter strip.

Shear $= 1/2 \times 5832 \times 2.80 = 8,160$ kg

Design of Sendai Bashi for Tottori Ken.

at section 2.5 meter above bottom. Depth = 5.03 m

Earth pressure

0.48 × 1600 × 5.03 = 3870

Seismic force 0.90 @ 2400 × 0.2 = 432

4302 kg/m²

moment = 1/10 × 4302 × 3.90² = 6540 kgm per meter strip.

Shear = 1/2 × 4302 × 2.80 = 6020 kg " "

normal state stresses for Both sections govern the wall section.

Effective depth required = $\sqrt{\frac{6100 \times 100}{100 \times 7.18}}$ = 29.2 cm

use 85 cm eff. depth with 5 cm insulation.

Steel area required at bottom 1 m strip = $\frac{6100 \times 100}{1200 \times \frac{7}{8} \times 85}$ = 6.84 cm²

use 19 mm bars at 35 cm ctoc = 8.10 cm²

unit shear = $\frac{5620}{100 \times \frac{7}{8} \times 85}$ = 0.76 kg/cm²

Steel area required at section 2.5 m above bottom = $\frac{4490 \times 100}{1200 \times \frac{7}{8} \times 85}$ = 5.04 cm².

Use 19 mm bars at 44 cm ctoc = 6.45 cm²

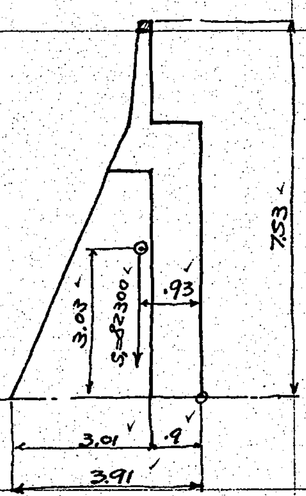
max. bond stress = $\frac{4130}{\frac{5.97}{.35} \times \frac{7}{8} \times 85}$ = 3.20 kg/cm²

Design of Buttress walls.

Center Buttress wall:

Weight and center of gravity of wall.

			hor. arm	hor. moment	Vert. arm	vert. moment
Parapet wall	0.40 × 1.95 × 3.90 =	3.04	1.10	3.34	6.43	19.55
front wall top	1.00 × 1.62 × 3.90 =	6.32	.81	5.12	5.02	31.70
" "	0.90 × 4.54 × 3.90 =	15.94	.45	7.17	2.27	36.15
buttress wall.	1.00 × 1.98 × 4.54 =	9.00	1.81	16.28	1.86	16.73
		34.30 m³	0.93 m	31.91	3.03 m	104.13

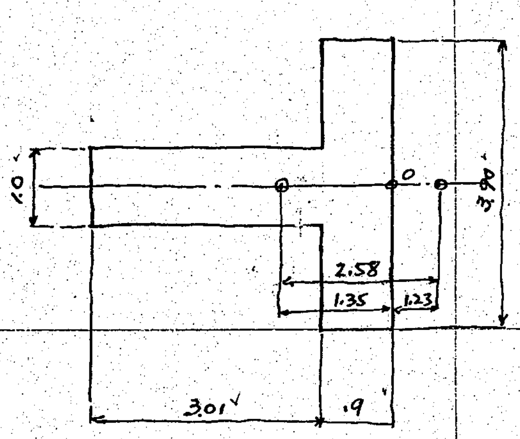


weight of wall = 34.30 @ 2400 = 82300 kg

seismic force = 82300 × .2 = 16460 kg

Case 1.

Stresses at normal state.



Earth pressure = 1/3 × 1600 × .5 = 267

1/3 × 1600 × 8.03 = 4283

4550 ÷ 2 × 7.53 × 3.90 = 66900 kg = E_n

moment on wall = 66900 × 2.65 = 177300 kgm

82300 × 0.93 = − 76500

100800

100800 ÷ 82300 = 1.23 m from O.

Center of gravity of bottom section.

flange .9 × 3.90 = 3.51 × .45 = 1.58

web. 1.0 × 3.01 = 3.01 × 2.41 = 7.25

6.52 1.35 m 8.83

Eccentricity = 1.35 + 1.23 = 2.58 m

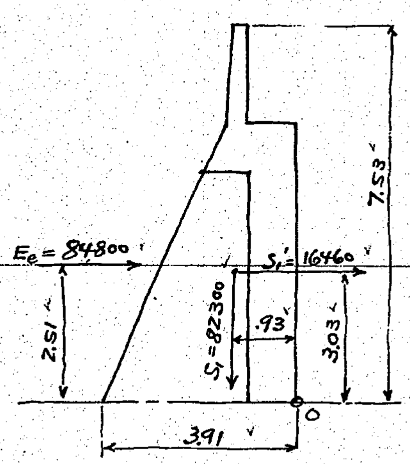
Bending moment at bottom = 82300 × 2.58 = 212500 kgm.

Design of Sendai Bashi for Tottori Ken

Case 2. Stresses during Earthquake (forward).

Earth pressure during earthquake

$0.48 \times 1600 \times \frac{7.53^2}{2} \times 3.90 = 84800$ kg. $= E_e$.



moment about toe O.

84800 × 2.51	=	213000	
16460 × 3.03	=	49900	
82300 × 0.93	=	− 76500	
		186400	

$186400 \div 82300 = 2.27^m$

Eccentricity $= 2.27 + 1.35 = 3.62^m$

Bending moment at bottom $= 82300 \times 3.62 = 298000$ kgm

Case 3 Stresses during Earthquake (backward)

moment about Point O.

82300 × 0.93	=	76500
16460 × 3.03	=	49900
		126400

$126400 \div 82300 = 1.54^m$

Eccentricity $= -1.54 + 1.35 = -0.19^m$

Bending moment at bottom $= 82300 \times 0.19 = 15650$ kgm.

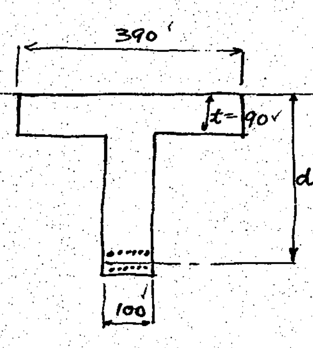
front side earth pressure assumed.

$0.48 \times 1600 \times \frac{4.0^2}{2} \times 3.9 = 24000$ kg.

front side earth pressure $24000 \times 1.33 = 31900$

47550 kgm

Stresses due to positive moment. 212500 kgm



approximate steel area required $= \frac{212500 \times 100}{1200 \times \frac{7}{8} \times 380} = 532$ cm²

use 12 ~ 25 mm ϕ bars $= 58.85$ cm²

steel ratio $p = \frac{58.85}{390 \times 380} = .000398$ $t/d = 90/380 = .237$

neutral axis in flange. Design as rectangular beam.

		Direct stress	Summary
$j = 0.958$, $k = .12$			
$f_s = \frac{212500 \times 100}{58.85 \times .958 \times 380} = 992$ kg/cm²		$-\frac{82300}{65200} \times 15 = -19$	973 kg/cm²
$f_c = \frac{992 \times .12}{15(1-.12)} = 9$ "		$\frac{82300}{65200} = 1.3$	10.3 "

Stresses due to negative moment. − 47550 kgm.

steel area required $= \frac{47550 \times 100}{1.6 \times 1200 \times \frac{7}{8} \times 385} = 7.35$ cm²

use 5 − 19ϕ bars $= 14.18$ cm²

$p = \frac{14.18}{100 \times 385} = .000368$

$j = .96$, $k = .12$ Direct stress

$f_s = \frac{47550 \times 100}{14.18 \times .96 \times 385} = 9080$ kg/cm² $-19 = 889$ kg/cm²

$f_c = \frac{9080 \times .12}{15(1-.12)} = 8.3$ " $+1.3 = 9.6$ kg/cm²

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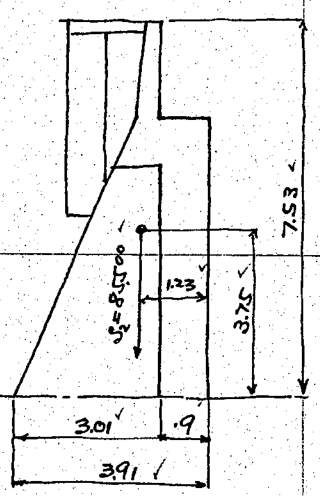
CHECKED BY M. Iida DATE 6-9-1 **PAGE NO** 25

CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.

Buttress walls on Both sides.

Weight and center of gravity of wall.

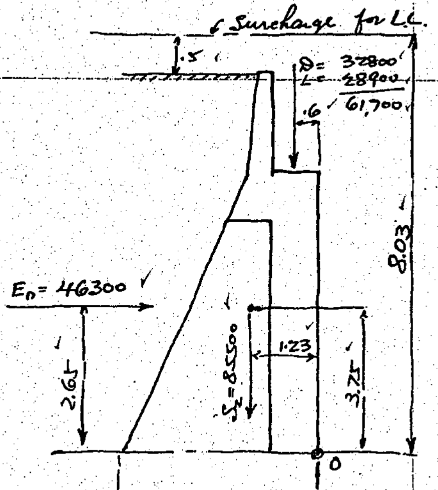


						hor arm	hor. m	vert arm	vert. m.
parapet wall.	0.40 ×	1.95 ×	1.80 =	1.40	×	1.10	1.54	6.43	9.00
front wall top	1.00 ×	1.62 ×	2.55 =	4.13		.81	3.35	5.02	20.75
" "	0.90 ×	4.54 ×	2.55 =	10.43		.45	4.70	2.27	23.70
buttress wall.	1.20 ×	1.98 ×	4.54 =	10.78		1.81	19.50	1.86	20.05
Column	1.20 ×	1.20 ×	2.05 =	2.95		1.50	4.42	6.56	19.35
"	1.20 ×	.45 ×	2.45 =	1.32		1.50	1.98	4.31	5.69
wing wall.	0.80 ×	4.50 ×	.45 =	1.62		2.50	4.05	5.34	8.65
slab	0.30 ×	.80 ×	.75 =	.18		2.50	.45	7.44	1.34
light pedestals	1.05 ×	1.10 ×	2.30 =	2.65		1.50	3.98	8.95	23.70
wing handrail.	.25 ×	.85 ×	.80 =	.17		2.50	.43	8.02	1.36
				35.63		1.23^m	44.40	3.75^m	133.59

Weight of wall = 35.63 @ 2400 = 85500 kg

Seismic force = 85580 × .20 = 17100 kg

Case 1. Stresses at Normal state



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

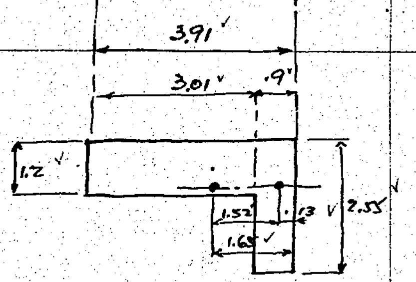
$\frac{1}{3}$ × 1600 × 8.03 = 4283

4550 ÷ 2 × 7.53 × 2.70 = 46300 kg = E_n

moment about 0.

	61700 ×	0.60 =	37000
	85500 ×	1.23 =	105100
46300		× 2.65 =	− 122800
	147200	0.13^m	19300

Center of gravity of bottom area.



flange .9 × 2.55 = 2.30 × .45 = 1.03

web 1.2 × 3.01 = 3.61 × 2.41 = 8.70

5.91 1.65^m 9.73

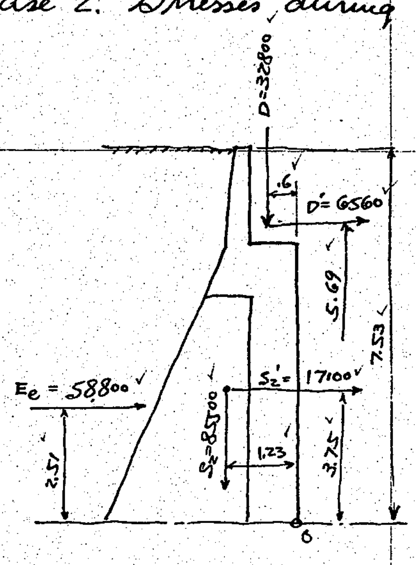
Eccentricity = 1.65 − .13 = 1.52^m

Bending moment at bottom = 147200 × 1.52 = 224000 kgm

Case 2. Stresses during Earthquake (forward).

Earth pressure during earthquake.

0.48 × 1600 × $\frac{7.53^2}{2}$ × 2.70 = 58800 kg = Ee.



Moment about 0.

D		32800 ×	0.60 =	19700
D'	6560		× − 5.69 =	− 37300
S_2		85500 ×	1.23 =	105100
S_2'	17100		× − 3.75 =	− 64100
E_e	58800		× − 2.51 =	− 147600
	82460	118300	1.05	− 124200

Eccentricity = 1.65 + 1.05 = 2.70^m

Bending moment = 118300 × 2.70 = 319500 kgm

Case 3. Stresses during Earthquake (Backward).

Earth pressure on front side.

0.48 × 1600 × $\frac{4.0^2}{2}$ × 2.70 = 16580 kg = E_e.

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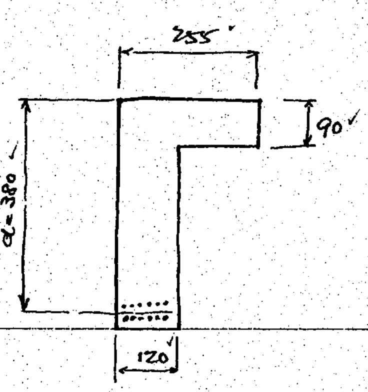
Design of Sendai Bashi for Tottori Ken.

moment about point O.

D		32800	× 0.60	=	19,700
D'	6560		× 5.69	=	37,300
S_2		85500	× 1.23	=	105,100
S_2'	17,100		× 3.75	=	64,100
E_e	16580		× 1.33	=	22,100
	40240	118300	2.10		248,300

Eccentricity = −2.10 + 1.65 = −0.45 m.
Bending moment = −118300 × .45 = −53300 kgm.

Stresses due to max. positive moment m = +224000 kgm



Steel area required $= \frac{224000 \times 100}{1200 \times \frac{7}{8} \times 380} = 56.2$ cm²

use 12 − 25 mm φ bars = 58.85 cm²

$p = \frac{58.85}{255 \times 380} = 0.00061$ $t/d = 90/380 = .237$

neutral axis in flange. Design as a rectangular beam.

$j = .954$ $k = .135$

$f_s = \frac{224000 \times 100}{58.85 \times .954 \times 380} = 1050$ kg/cm²

$f_c = \frac{1050 \times .135}{15(1-.135)} = 10.9$ 〃

Direct compressive stress

$f_s = -\frac{147200}{59100} \times 15 = -37$ kg/cm²

$f_c = \frac{147200}{59100} = 2.5$ 〃

Summary of stresses

	f_s	f_c
Bending stress	1050	10.9
direct 〃	−37	2.5
	1013 kg/cm²	13.4 kg/cm²

Stresses due to negative moment m = −53300 kgm.

Steel area required $= \frac{53300 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 385} = 8.24$ cm²

use 5 − 19φ bars = 14.18 cm²

$p = \frac{14.18}{120 \times 385} = .00031$

$j = .96$, $k = .12$

$f_s = \frac{\cancel{118300}\ 53300 \times 100}{14.18 \times .96 \times 385} = 1015$ kg/cm²

$f_c = \frac{1015 \times .12}{15 \times .88} = 9.2$ 〃

Direct compression

$f_c = \frac{118300}{59100} = 2.0$ kg/cm²

$f_s = 2.0 \times 15 = -30$ 〃

Summary for stresses

	f_s	f_c
Bending stress	1015	9.2
direct 〃	−30	2.0
	985 kg/cm²	11.2 kg/cm²

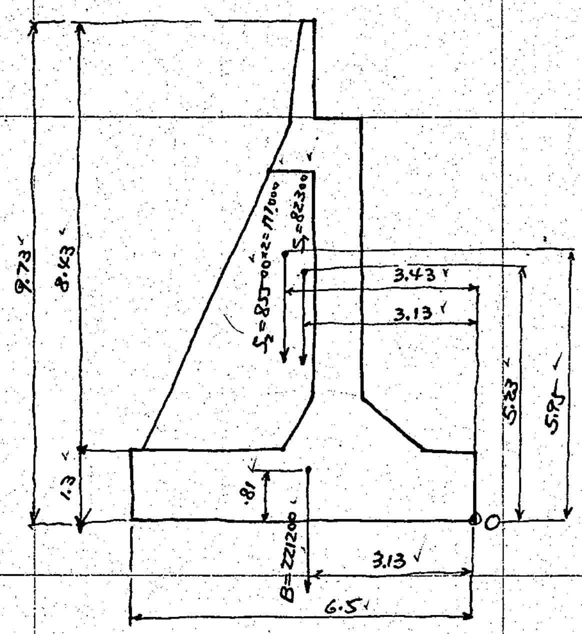
Design of Sendai Bashi for Tottori Ken.

Stability of Abutment as a whole.

Weight and center of gravity of Base.

		hor. arm	hor. m.	vert. arm	vert. m.
1.8 × .90 × 9.00 =	14.58	2.47	36.00	1.67	24.35
1.3 × 6.50 × 9.20 =	77.62	3.25	252.50	.65	50.50
	92.20 m³	3.13	288.50	.81	74.85

weight of base 92.2 @ 2400 = 221,200 kg.



Weight and center of gravity of abutment.

S_1	82300	× 3.13	257500	5.23	430000
S_2	171000	4.43	758000	5.95	1018000
B	221200	3.13	693000	.81	179000
	474500 kg	3.60 m	1708500	3.43 m	1627000

Seismic force = 474,500 × .2 = 94900 kg.

Earth on rear footing, depth = 8.43 m

weight ≒ 3.0 × 8.43 × 7.70 @ 1600 = 308,000 kg. = w_1

Earth on front footing depth 5.0 m

2.2 × 5 × 9.2 @ 1600 = 162000 kg = w_2

Earth pressure on rear.

normal state, say $\frac{1600 \times 9.73^2}{6} \times 9.2$ = 232000 kg = E_{nr}

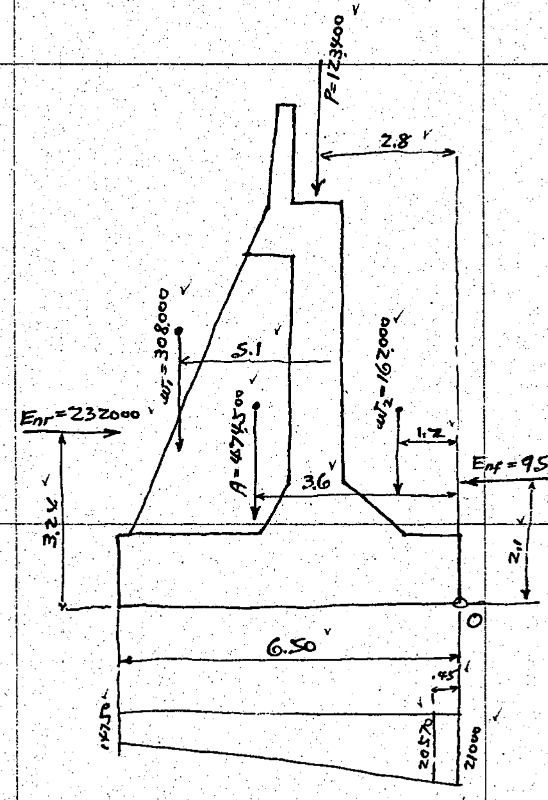
Earthquake $0.48 \times 1600 \times \frac{9.73^2}{2} \times 9.2$ = 334,000 " = E_{er}

Earth pressure on front.

normal state say $\frac{1600 \times 6.3^2}{6} \times 9.0$ = 95000 " = E_{nf}

earthquake $0.48 \times 1600 \times \frac{6.3^2}{2} \times 9$ = 137500 " = E_{ef}

Case 1. Stability at normal state.



moment about toe O.

Loads	Hor. forces	Vert. forces		Lev. arms		Moment about O
P		123400	×	2.80	=	346000
A		474500	×	3.60	=	1708000
w_1		308000	×	5.10	=	1571000
w_2		162000	×	1.20	=	194300
E_{nr}	232000		×	3.24	=	− 752000
E_{nf}	95000		×	2.10	=	199500
	327000 kg	1067900 kg		3.06 m		3266800

Eccentricity = 3.25 − 3.06 = 0.19

Resultant force within middle third

max. toe pressure $= \frac{1067900}{6.5 \times 9.2}\left(1 \pm \frac{6 \times .19}{6.5}\right) \doteqdot$ 21000 kg/m² (or 1.92 tons/□)

or = 14750 "

Load on one pile = 20570 × .9 × .9 = 16.65 kg tons.

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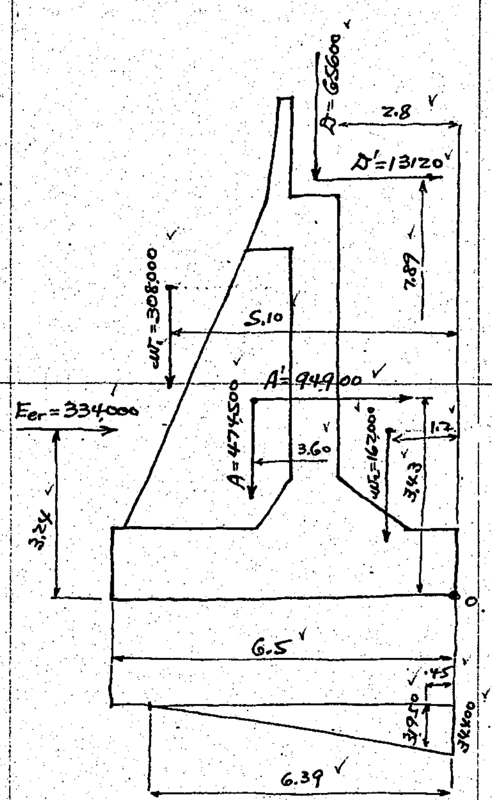
Design of Sendai Bashi for Tottori Ken.

Case 2. Stability during Earthquake. (forward)

moment about toe O.



Loads	Hor. forces	Vert. forces		Lev. arms		Moment about O.
D		65,600	×	2.80	=	183,800
D'	13120		×	7.89	=	− 103,500
A		474,500	×	3.60	=	1,708,000
A'	94,900		×	3.43	=	− 325,500
w_1		308,000	×	5.10	=	1,571,000
w_2		162,000	×	1.20	=	194,300
Eer	334,000		×	3.24	=	− 1,087,000
	442,020 kg	1,010,100 kg		2.13 m		2,146,100

Eccentricity = 3.25 − 2.13 = 1.12 m

– Resultant force outside of middle third, neglecting tension.

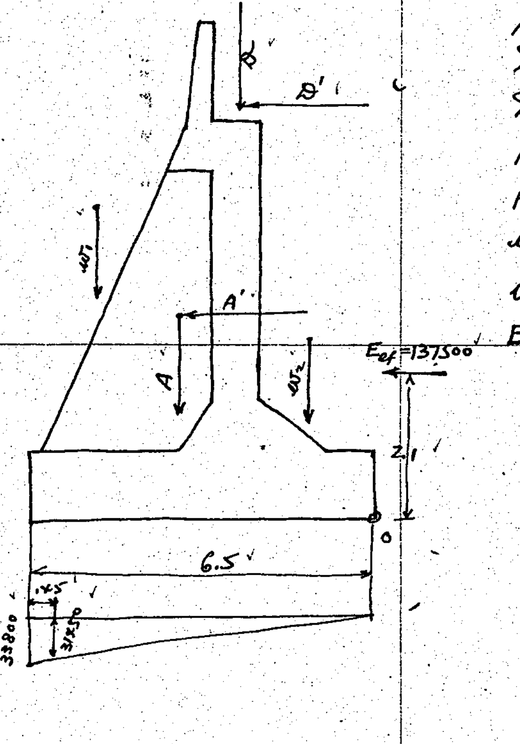
pressure area = 2.13 × 3 × 9.2 = 58.8 m²

max. bearing pressure = $\frac{2 \times 1,010,100}{58.8}$ = 34400 kg/m² or (3.14 tons/□')

max. load on one pile = 31950 × .9 × .9 = 25.9 tons

Case 3. Stability during Earthquake. (Backward).

moment about toe O.



Loads	Hor. forces	Vert. forces		Lev. arms		Moments.
D		65600	×	2.80	=	183,800
D'	13120		×	7.89	=	103,500
A		474,500	×	3.60	=	1,708,000
A'	94900		×	3.43	=	325,500
w_1		308,000	×	5.10	=	1,571,000
w_2		162,000	×	1.20	=	194,300
Eef	137,500		×	2.10	=	289,000
	245,520 kg	1010,100 kg		4.33 m		4,375,100

Eccentricity = 3.25 − 4.33 = −1.08 m

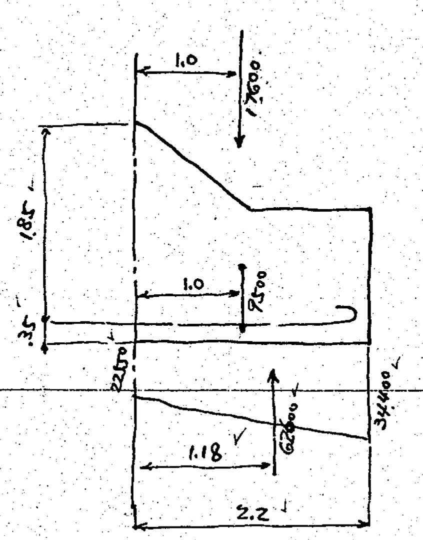
Resultant force just on middle third point.

max. bearing pressure = $\frac{1010,100 \times 2}{6.5 \times 9.2}$ = 33800 kg/m² or (3.09 tons/□')

max. load on one pile = 31450 × .9 × .9 = 25.5 kg tons.

Design of Cantilever footing at toe.

Case 2.



upward pressure = 28475 × 2.2 = 62,600 kg. × 1.18 = 73,900

downward – earth 2.2 × 5.0 @ 1600 = −17600 kg × 1.11 = −19,500

– Concrete 1.8 × 2.2 @ 2400 = −9,500 × 1.0 = − 9,500

shear = 35,500 kg   moment = 44,900 kgm

Eff. depth required = $\sqrt{\frac{44900 \times 100}{100 \times 1.6 \times 7.18}}$ = 62.5 cm

use eff. depth of 185 cm with 5 cm insulation (between & steel + pile head)

Steel area required = $\frac{44900 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 185}$ = 14.42 cm²

use 22 mm ø bars at 25 cm ctoc = 15.21 cm²

unit shear = $\frac{35,500}{100 \times \frac{7}{8} \times 185}$ = 2.2 kg/cm²

unit bond = $\frac{35,500}{6.91 \times 4 \times \frac{7}{8} \times 185}$ = 7.95 " < 6 × 1.6 = 9.6

Design of Sendai Bashi for Tottori Ken.

Design of Rear footing

Let us assume the propotion of loads carried by longitudinal continuous beam and transverse cantilever beam as follows.

span length of longitudinal beam = 3.90 m = l_1 $\frac{l_1}{l_2} = \frac{3.90}{5.60} = 0.7$

" " " transverse beam = 2.8, 2.8×2 = 5.60 = l_2

Load on l_1 span = $1.5 - \frac{l_1}{l_2} = 1.5 - .7 = 0.80$ of total load.

Load on l_2 span = $\frac{l_1}{l_2} - 0.5 = 0.7 - 0.5 = 0.20$

Case 2. Upward pressure neglected.

downward pressure. earth 8.43 @ 1600 = 13480

concrete 1.3 @ 2400 = 3120

16600 kg/m²

Longitudinal beam. load = 16600 × .8 = 13270 kg/m²

moment = 1/10 × 3.9² × 13270 = 20150 kgm per meter strip

Shear = 1/2 × 3.9 × 13270 = 25900 kg " " "

Eff. depth required = $\sqrt{\frac{20150 \times 100}{100 \times 1.6 \times 7.18}}$ = 42 cm Use 95 cm eff. depth with 5 cm insulation.

Steel area required = $\frac{20150 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 95}$ = 12.61 cm² per meter strip

use 22 mm ϕ bars at 30 cm ctoc = 12.68 cm²

unit shear = $\frac{25900}{100 \times \frac{7}{8} \times 95}$ = 3.1 kg/cm²

unit bond = $\frac{25900}{6.91 \times 3.33 \times \frac{7}{8} \times 95}$ = 13.5 "

use 90 × 60 cm fillets.

unit bond = $\frac{25900}{6.91 \times 3.33 \times \frac{7}{8} \times 185}$ = 6.96 < 6.0 × 1.6 = 9.6

shear at end of fillet = 1/2 × 1.6 × 13270 = 10620 kg

unit bond = $\frac{10620}{6.91 \times 3.33 \times \frac{7}{8} \times 95}$ = 5.55 kg/cm².

Transverse cantilever beam load = 16600 × .2 = 3320 kg/m²

moment = 1/2 × 3320 × 2.8² = 13020 kgm.

Steel area req'd = $\frac{13020 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 95}$ = 8.15 cm²

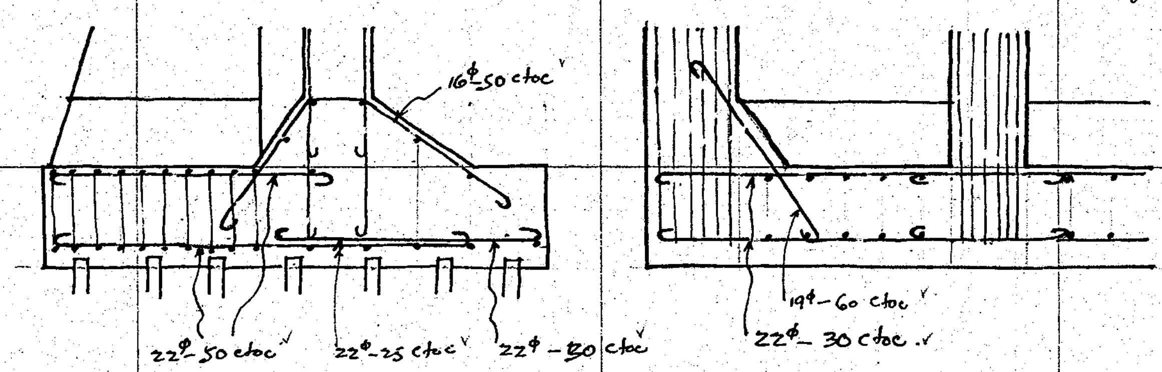
use { 22ϕ bars at 50 cm ctoc = 7.60
  16ϕ " " 50 " = 4.02
  11.62 cm²

Case 3. Upward pressure average 26500 = 26500

downward " earth 8.43 @ 1600 = − 13480

" " concrete 1.3 @ 2400 = − 3120

9900 kg/m² safe.



Reinforcements for footings.

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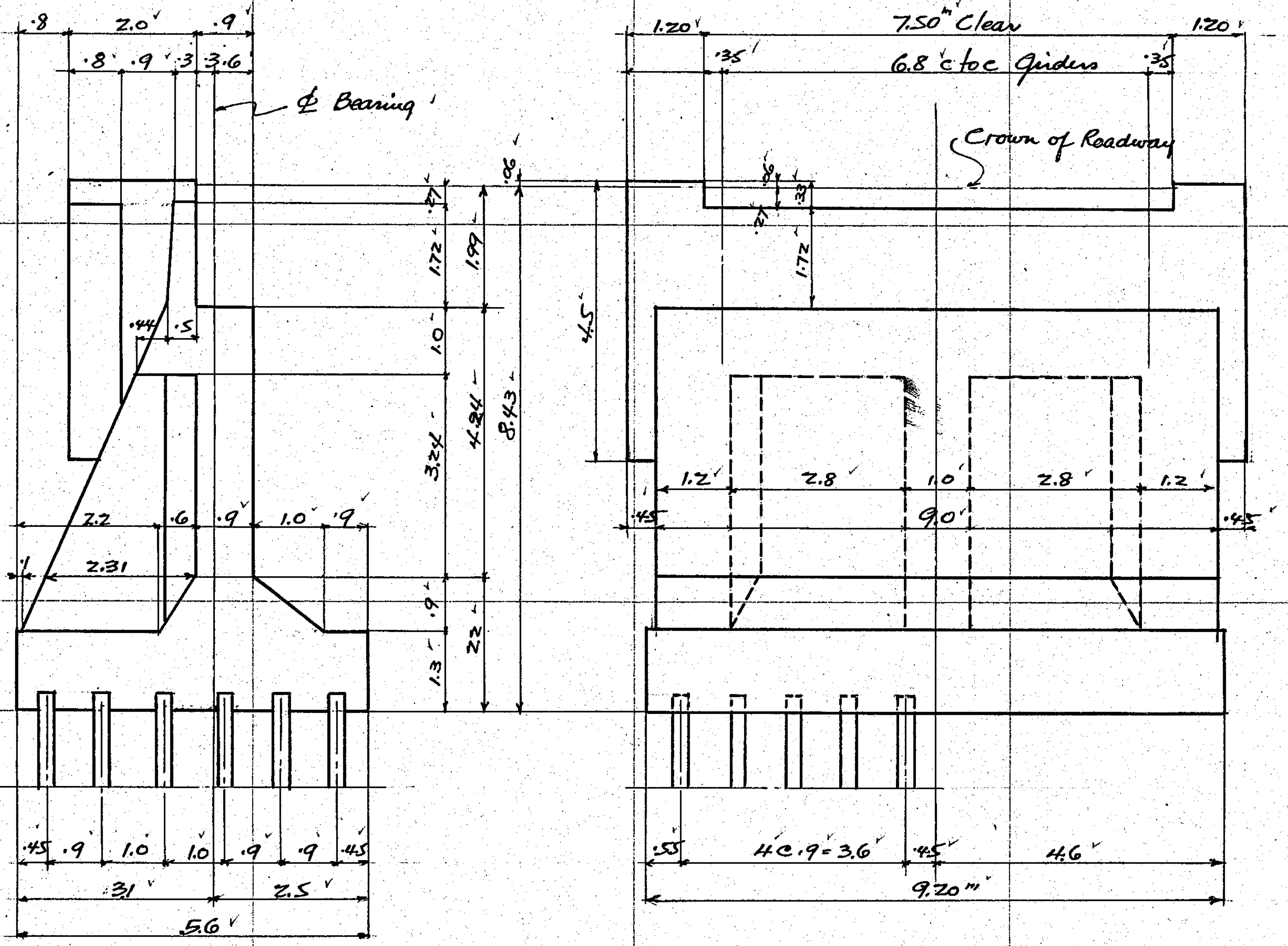
CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.

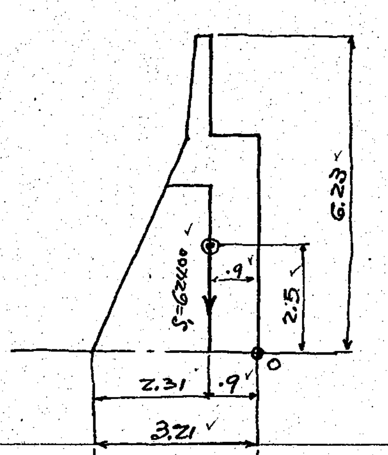
Design of East abutment A2.
General dimensions are as shown on sketch below.



General sketch of abutment A2.
Scale 1:100.

For Parapet and front walls, use same details as for abutment A1 see page 22 & 23.

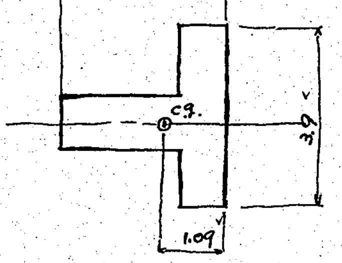
Design of Buttress walls.
Buttress wall at center.
Weight and center of gravity of wall.



		hor. arm	hor m.	vert. arm	vert m.
parapet wall	0.40 × 1.95 × 3.90 = 3.04	1.10	3.34	5.13	15.60
front wall top	1.00 × 1.62 × 3.90 = 6.32	.81	5.12	3.72	23.50
"	0.90 × 3.24 × 3.90 = 11.37	.45	5.11	1.62	18.40
buttress wall	1.00 × 3.24 × 1.63 = 5.28	1.76	9.30	1.39	7.34
	26.01	.9	22.87	2.5 m	64.84

weight of wall = 26.01 × 2400 = 62400 kg.
seismic force = 62400 × .2 = 12,500

Center of gravity of Bottom area.



flange	3.90 × .9 =	3.51	× 0.45 =	1.58
web.	1.00 × 2.31 =	2.31	× 2.06 =	4.76
		5.82 cm	1.09 m	6.34

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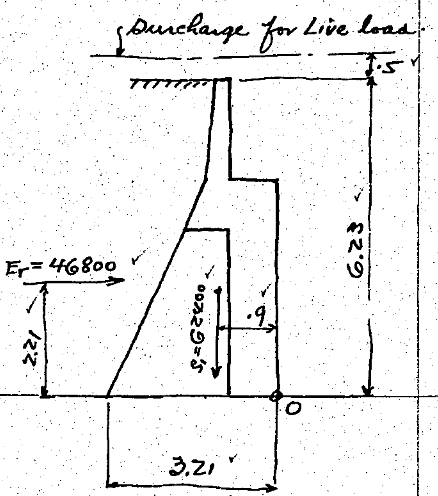
CALCULATIONS FOR

Design of Sendai Bashi for Tottori Area

Case 1. Stresses at normal state.



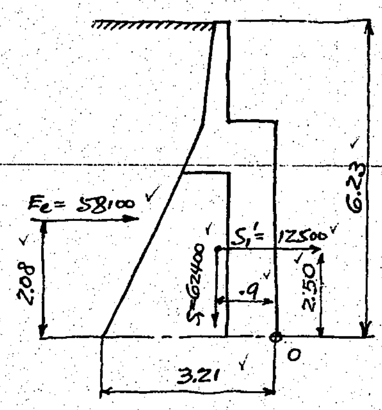
Earthpressure = 1/3 × 1600 × .5 = 267
1/3 × 1600 × $\frac{6.73}{8.03}$ = 3587
3854 ÷ 2 = 1927 × 6.23 × 3.90 = 46800 kg = E_n

moment on wall. 46800 × 2.21 = 103500
62400 × 0.90 = −56200
~~159700~~
47,300 kgm

47300 ÷ 62400 = 0.76 m from O.
Eccentricity = 1.09 + .76 = 1.85 m
Bending moment at bottom = 62400 × 1.85 = 115,500 kgm

Case 2. Stresses during Earthquake. (foward).

Earthpressure during earthquake
0.48 × 1600 × $\frac{6.23^2}{2}$ × 3.90 = 58100 kg = E_e



moment about toe O.
58100 × 2.08 = 121000
62400 × 0.90 = −56200
12500 × 2.50 = 31300
96,100 kgm.

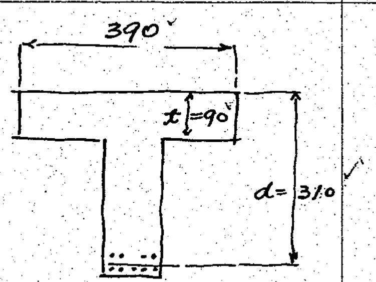
96,100 ÷ 62400 = 1.54 m
Eccentricity = 1.09 + 1.54 = 2.63 m
Bending moment at bottom = 62400 × 2.63 = 164,000 kgm.

Case 3. Stresses during Earthquake (Backward).

Earthpressure during earthquake.
0.48 × 1600 × $\frac{3.0^2}{2}$ × 3.9 = 13500 kg. = E_e
moment about point O
62400 × 0.90 = 56200
12500 × 2.50 = 31,300
13500 × 1.00 = 13500
101,000

101,000 ÷ 62400 = 1.62 m
Eccentricity = 1.62 − 1.09 = .53 m
Bending moment at botton = −62400 × 0.53 = −33100 kgm.

Stresses due to positive moment. m = 115,500 kgm at normal state.



steel area required = $\frac{115500 \times 100}{1200 \times \frac{7}{8} \times 310}$ = 35,50 cm²

try: 9 − 22 mmφ bars = 34.24 cm²
steel ratio p = $\frac{34.24}{390 \times 310}$ = .0028, t/d = 90/310 = .29

neutral axis in the flange.
j = .96 k = 0.11

		Direct stress	Summary
f_s = $\frac{115500 \times 100}{34.24 \times .96 \times 310}$ =	1.135 kg/cm²	$\frac{62400}{58200}$ × 15 = −16	1,119 kg/cm²
f_c = $\frac{1.135 \times .11}{15 \times (1-.11)}$ =	9.3 "	$\frac{62400}{58200}$ = 1.1	10.4 "

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CALCULATIONS FOR

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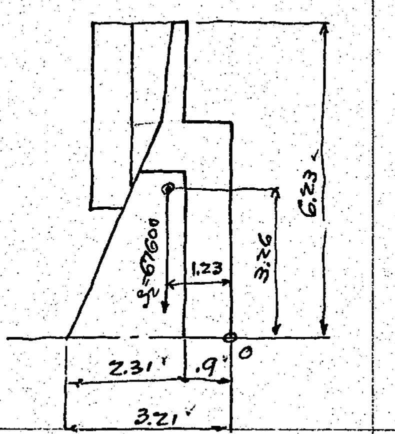
Design of Sendai Bashi for Tottori Ken.

Stresses due to negative moment. $m = -33{,}100$ kgm

Steel area required $= \dfrac{33100 \times 100}{1.6 \times 1200 \times \frac{7}{8} \times 31.5} = 6.3$ cm²

Use 5-16φ bars $= 10.06$ cm²

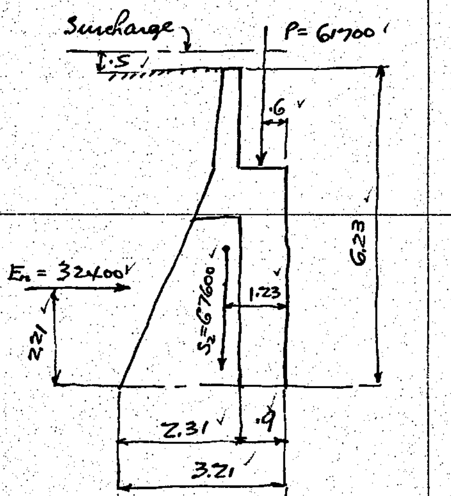
Design of Buttress on Both sides.
Weight and center of gravity of wall.



			hor. arm	hor. m.	vert. arm	vert m.
Parapet wall	0.40 × 1.95 × 1.80	= 1.40	1.13	1.58	5.13	7.18
front wall, top	1.00 × 1.62 × 2.55	= 4.13	.81	3.35	3.72	15.37
" "	0.90 × 3.24 × 2.55	= 7.43	.45	3.34	1.62	12.04
buttress wall	1.20 × 3.24 × 1.63	= 6.33	1.76	11.14	1.39	8.80
column	1.20 × 1.20 × 2.05	= 2.95	1.50	4.42	5.26	15.52
"	1.20 × .45 × 2.45	= 1.32	1.50	1.98	3.01	3.97
wing wall	0.80 × 4.50 × .45	= 1.62	2.50	4.05	4.04	6.55
slab	0.30 × 0.80 × .75	= .18	2.50	0.45	6.14	1.11
light pedestal	1.05 × 1.10 × 2.30	= 2.65	1.50	3.98	7.65	20.28
wing handrail	.25 × .85 × .80	= .17	2.50	0.43	6.72	1.14
		28.18 m³	1.23 m	34.72	3.26 m	91.96

Weight of wall $= 28.18 \times 2400 = 67600$ kg.
Seismic force $= 67600 \times .2 = 13500$

Case 1 Stresses at normal state



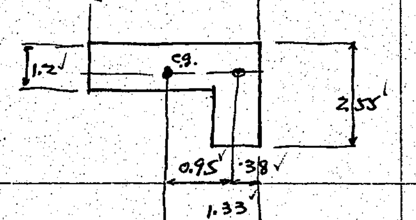
Earthpressure $1927 \times 6.23 \times 2.70 = 32400$ kg $= E_n$

moment on wall

P.		61700	× −0.60	= −37000
S_2		67600	× −1.23	= −83200
E_n	32400		× 2.21	= 71600
	32400	129300	−0.38 m	−48600

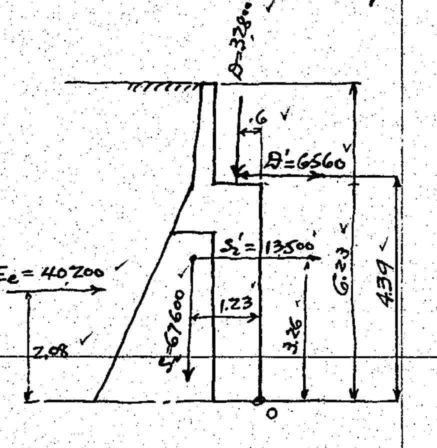
Center of gravity of bottom area

flange	.9 × 2.55	= 2.30	× .45	= 1.03
web	1.2 × 2.31	= 2.77	× 2.06	= 5.70
		5.07 m	1.33	6.73



Eccentricity $= 1.33 - 0.38 = 0.95$ m
Bending moment at bottom $= 129300 \times .95 = 122800$ kgm.

Case 2. Stresses during Earthquake. (forward).



Earthpressure during earthquake.
$0.48 \times 1600 \times \frac{6.23^2}{2} \times 2.70 = 40200$ kg $= E_e$

moment about point O.

D		32800	× −.60	= −19700
D'	6560		× 4.39	= 28800
S_2		67600	× −1.23	= −83100
S_2'	13500		× 3.26	= 44000
E_e	40200		× 2.08	= 83650
	60260	100400	0.54 m	53650

Eccentricity $= 1.33 + 0.54 = 1.87$ m
Bending moment at bottom $= 100400 \times 1.87 = 187900$ kgm.

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CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.

Case 3. Stresses during Earthquake (Backward).

Earth pressure on front side

$0.48 \times 1600 \times \frac{3.0^2}{2} \times 2.70 = 9330$ kg $= E_e$

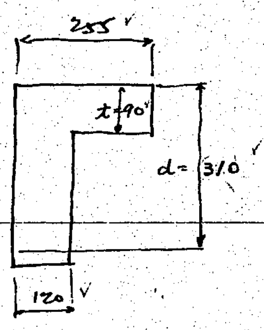
moment about O.

D		32800	× -.60	= -19700
D'	6560		× -4.39	= -28800
S_2		67600	× -1.23	= -83100
S_2'	13500		× -3.26	= -44000
E_e	9330		× -1.00	= -9330
	29390	100400	-1.84m	-184930

Eccentricity $= -1.84 + 1.33 = -0.51$

Bending moment at bottom $= -100400 \times .51 = -51200$ kgm

Stresses due to positive moment. $m = 122800$ kgm normal state.



Steel area required $= \frac{122800 \times 100}{1200 \times \frac{7}{8} \times 310} = 37.75$ cm

Try 9-22mmφ bars. $= 34.24$ cm²

steel ratio $p = \frac{34.24}{255 \times 310} = .00043$

$k = .122$, $j = .959$

$f_s = \frac{122800 \times 100}{34.24 \times .959 \times 310} = 1206$ kg/cm²

$f_c = \frac{1206 \times .122}{15 \times .878} = 11.2$

Direct stresses.

$f_c = \frac{129300}{50700} = 2.6$ kg/cm²

$f_s = -2.6 \times 15 = -39$

Summary of stresses.

		Bending stress	Direct stress	Total
Steel stress	f_s	1206 +	-39 =	1167 kg/cm²
Concrete "	f_c	11.2 +	2.6 =	13.8 "

Stresses due to negative moment. $m = -51200$ kgm Earthquake.

Steel area required $= \frac{51200 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 310} = 9.84$ cm²

use 6-16φ bars $= 12.07$ cm²

Stability of Abutment as a whole.

weight and center of gravity of Base.

		hor arm	hor m.	vert. arm	vert. m.
$1.7 \times 0.9 \times 9.0 =$	13.78	2.23	30.74	1.68	23.17
$1.3 \times 5.6 \times 9.2 =$	67.00	2.80	187.70	.65	43.55
	80.78 m³	2.71	218.44	.82	66.72

$80.78 \times 2400 = 194000$ kg

Weight and center of gravity of Abutment

		hor arm	hor m.	vert arm	vert m.
S_1	62400	2.80	174700	4.70	293000
S_2	2 × 67600	3.13	423000	5.46	739000
B.	194000	2.71	526000	0.82	159000
	391600 kg	2.87m	1123700	3.04 m	1191000

Design of Sendai Bashi for Tottori Ken.

Earth on rear footing depth = 7.13^m
weight ≒ 2.6 × 7.13 × 7.7 @ 1600 = 228,500 kg = w_1

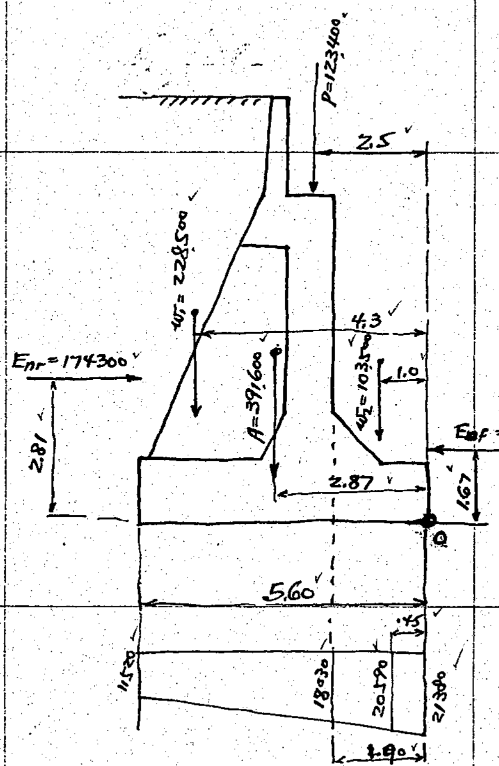
Earth on front footing. depth. 3.75
weight ≒ 3.7 × 1.9 × 9.2 @ 1600 = 103,500 kg. = w_2

Earth pressure on rear
normal state say $\frac{1600 \times 8.43^2}{6} \times 9.2 = 174300$ kg = E_{nr}

Earthquake $0.48 \times 1600 \times \frac{8.43^2}{2} \times 9.2 = 261,000$ kg = E_{er}

Earth pressure on front.
normal state $\frac{1600 \times 5.00^2}{6} \times 9.2 = 61300$ kg = E_{nf}
Earthquake $0.48 \times 1600 \times \frac{5.0^2}{2} \times 9.2 = 88400$ " = E_{ef}

Case 1. Stability at normal state.



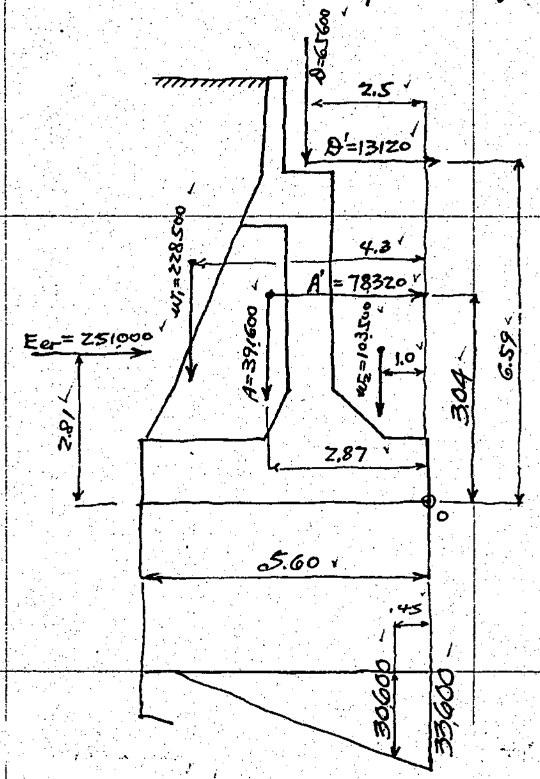
moment about toe O.

Loads	Hor. forces	Vert. forces	Lev. arms	moment about O.
P.		123,400	× 2.50 =	− 307,500
A		391,600	× 2.87 =	− 1125,000
w_1		228,500	× 4.30 =	− 982,000
w_2		103,500	× 1.00 =	− 103,500
E_{nr}	174300		× 2.81 =	490,000
E_{nf}	61,300		× 1.67 =	− 102,300
	235,600	847,000	− 2.52^m	− 2130,300

Eccentricity = 2.80 − 2.52 = 0.28^m
Resultant force within middle third
max. toe pressure = $\frac{847000}{5.60 \times 9.20}\left(1 \pm \frac{6 \times .28}{5.60}\right) = 21380$ kg/m² or (1.95 tons/□')
11520 "

max. load on one pile = 20590 × .9 × .9 = 16.7 kg tons.

Case 2. Stability during Earthquake. (forward).



moment about O.

Loads	Hor. forces	Vert. forces	Lev. arm	moment.
D		65600	× − 2.50 =	− 164,000
D'	13120		× 6.59 =	86400
A		391600	× − 2.87 =	− 1123,000
A'	78320		× 3.04 =	238,000
w_1		228,500	× − 4.30 =	− 982,500
w_2		103,500	× − 1.00 =	− 103500
E_{er}	251,000		× 2.81 =	705000
	342,440	789,200	− 1.70^m	− 1343,600

Eccentricity = 2.80 − 1.70 = 1.10^m
Resultant force outside of middle third, neglecting tension,
pressure area = 1.70 × 3 × 9.2 ≒ 46.9^{m2}
max. bearing pressure = $\frac{789200 \times 2}{46.9}$ = 33,600 kg/m² (3.07 tons/□')

max. load on one pile = 3060 × .9 × .9 = 24.9 kg tons.

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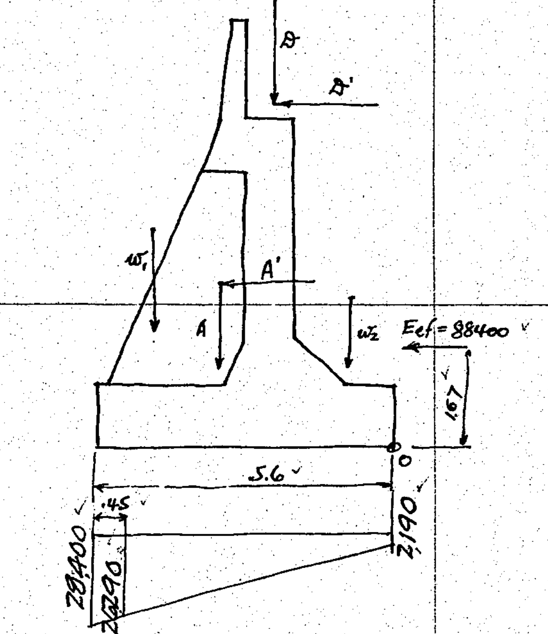
CALCULATIONS FOR

Design of Sendai Bashi for Tottori ken.

Case 3. Stability during Earthquake (Backward).



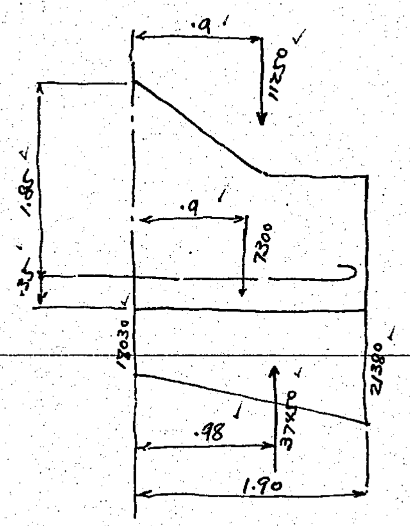
Loads	Hor. forces	Vert. forces	Lev. arms	moments
D		65,600	× − 2.50	= − 164,000
D'	13,120		× − 6.59	= − 86,400
A		391,600	× − 2.87	= − 1123,000
A'	78,320		× − 3.04	= − 238,000
w_1		228,500	× − 4.30	= − 982,500
w_2		103,500	× − 1.00	= − 103,500
Eef	88,400		× − 1.67	= − 147,500
	179840	789,200	− 3.60 m	− 2,844,900

Eccentricity = 2.80 − 3.60 = −0.80 m

max. bearing pressure = $\frac{789200}{5.6 \times 9.2}\left(1 \pm \frac{6 \times .80}{5.6}\right)$ = 28400 kg/m² or (2.56 tons/ft²)
2,190

max. load on one pile = 26.29 × .9 × .9 = 21.8 kg tons.

Design of Cantilever footing at toe.
Case 1.



upward pressure	= 19710 × 1.90 =	37450 kg ×	.98 =	36,700
downward " earth	= 1.9 × 3.7 @ 1600 =	− 11,250 ×	0.90 =	− 10,130
" " concrete	= 1.6 × 1.9 @ 2400 =	− 7300 ×	0.90 =	− 6,570
	Shear =	18900 kg	moment =	20000 kgm

Effective depth req'd = $\sqrt{\frac{20000 \times 100}{100 \times 7.18}}$ = 52.8 cm

use eff. depth of 185 cm with 5 cm insulation

Steel area required = $\frac{20000 \times 100}{1200 \times \frac{7}{8} \times 185}$ = 10.3 cm²

use 19 mm φ bars at 25 cm ctoc = 11.34 cm²

Unit shear = $\frac{18900}{100 \times \frac{7}{8} \times 185}$ = 1.17 kg/cm²

Unit bond = $\frac{18900}{5.97 \times 4 \times \frac{7}{8} \times 185}$ = 4.80 "

Design of Rear footing

Span length of longitudinal beam = 3.90 = l_1 $\frac{l_1}{l_2} = \frac{3.90}{4.40} = .89$

Transverse beam span length = 2.2 × 2 = 4.40 = l_2

Load on l_1 span = $1.5 - \frac{l_1}{l_2}$ = 1.5 − .89 = 0.61 of total load

" " l_2 − = $\frac{l_1}{l_2} - 0.5$ = .89 − 0.5 = 0.39 "

Case 2. Upward pressure neglected.

downward pressure, earth 7.13 @ 1600 = 11400
concrete 1.3 @ 2400 = 3120
14520 kg/m²

Longitudinal beam. load = 14520 × .61 = 8850 kg/m²

moment = 1/10 × 3.9² × 8850 = 13450 kgm per meter strip

Shear = 1/2 × 3.9 × 8850 = 17250 kg "

Eff. depth required = $\sqrt{\frac{13450 \times 100}{100 \times 7.18 \times 1.6}}$ = 34.2 cm use 95 cm eff. depth with 5 cm insulation.

Steel area required = $\frac{13450 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 95}$ = 9.44 cm² per meter strip.

use 19 mm φ bars at 30 ctoc = 9.46 cm² "

Design of Sendai Bashi for Tottori Ken.

unit shear $= \dfrac{17250}{100 \times \frac{7}{8} \times 95} = 2.075$ kg/cm²

unit bond $= \dfrac{17250}{5.97 \times 3.33 \times \frac{7}{8} \times 95} = 10.45$ "

use 12φ bond bars at every other spacing on both sides, lap the bars at center.

total perimeter $5.97 \times 3.33 = 19.90$
$3.77 \times 1.67 = \underline{6.30}$
26.20 cm

unit bond $= \dfrac{17250}{26.20 \times \frac{7}{8} \times 95} = 7.8$ kg/cm² $< 6 \times 1.6 = 9.6$ on both sides

or $10.45 \div 2 = 5.23$ " " at center.

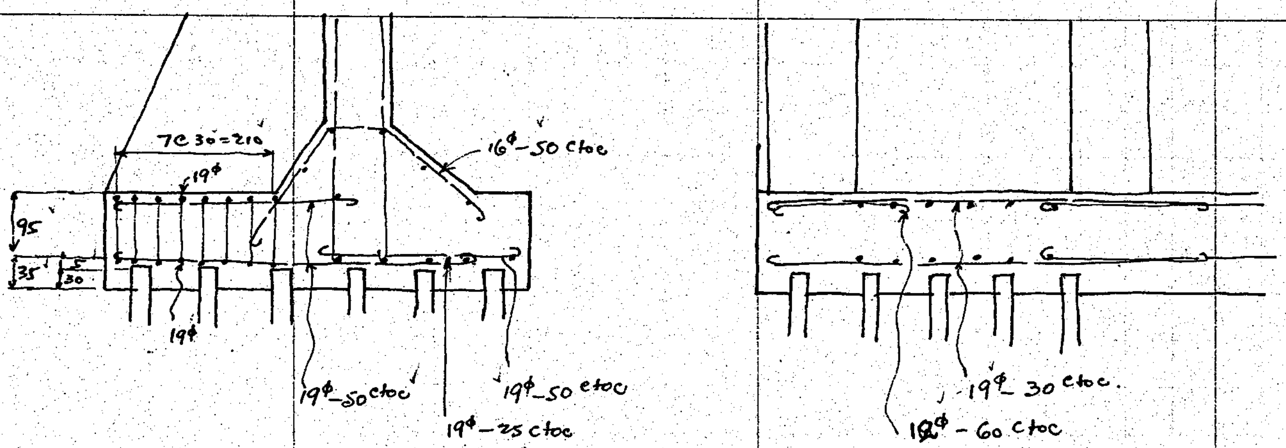
Transverse cantilever beam. load $= 14520 \times 0.39 = 5660$ kg/m

moment $= \frac{1}{2} \times 5660 \times 2.2^2 = 13700$ kgm.

Shear $= \cancel{13700}\ 5660 \times 2.2 = 12450$ kg

Steel area required $= \dfrac{13700 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 95} = 8.59$ cm²

use 19φ bars at 50 cm ctoc $= 5.67$
16φ " " 50 " " $= \underline{4.02}$
9.69 cm²



Reinforcements for footings

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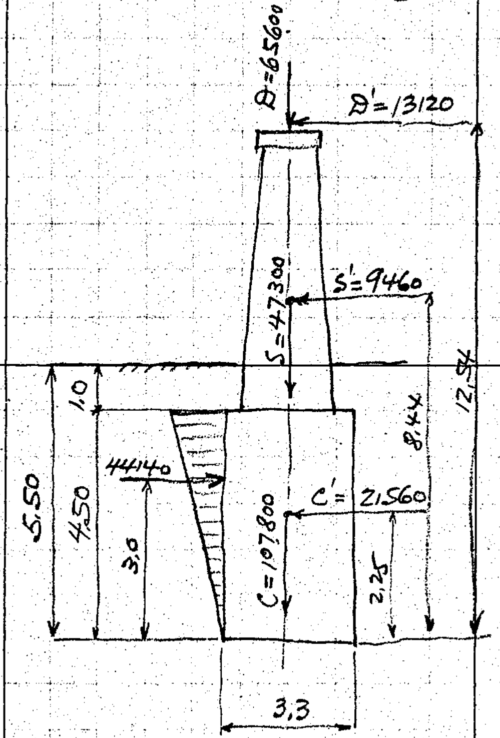
CALCULATIONS FOR

Design of Sendai Bashi for Tottori Ken.

Page 18

Case 3. Seismic Stability of Pier P8. k assumed 0.20



Moment due to seismic forces about center of base area.

D'	13,120	× 12.54	=	164,500
S'	9460	× 8.44	=	79,800
C'	21,560	× 2.25	=	48,500
	44,140 kg			292,800

frictional couple, less −58800

234,000

Resisting Earth pressure moment

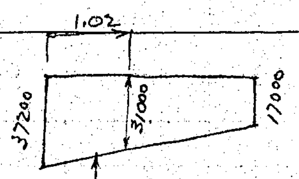
44,140 × 3.0 = −132,500

101,500 kgm

Eccentricity $= \frac{101500}{220700} = 0.46^{m}$

Resultant force within middle third

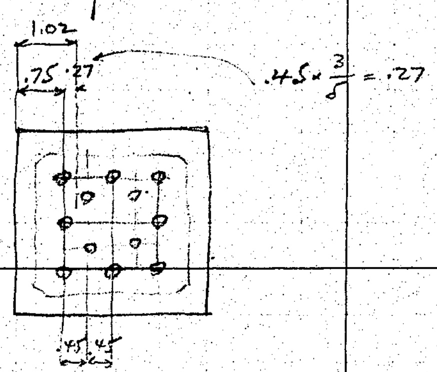
max. toe pressure $= \frac{220700}{3.3 \times 3.3}\left(1 \pm \frac{6 \times 0.46}{3.3}\right) = 37200^{kg/m^2}$ (3.4 tons/□') or 17000 "



Load on one pile

37200
31000
68200 ÷ 2 = 34100

$34100^{kg} \times 1.02 \times \frac{3.3}{3} =$ 38.200 kg tons



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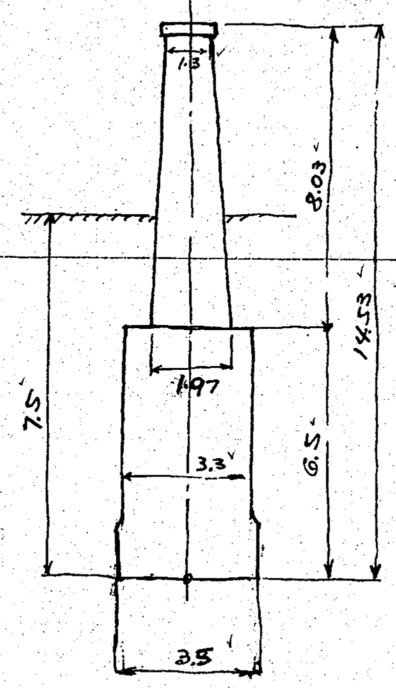
Design of Sendai Bashi for Tottori Ken.

Pier no. 2.



Weight and center of gravity of Pier

Pier shaft.

Coping	1.5φ × 0.30 × 2	= 1.06	× 7.88	=	8.35
"	0.70 × 0.30 × 5.60	= 1.18	× 7.88	=	9.30
Shaft.	1.64φ × 7.73 × 2	= 32.60	× 3.35	=	109.20
wall	5.5 × 2.00 × 0.5	= 5.50	× 6.90	=	38.00
		40.34 m³	4.08 m		164.85

weight of ½ of Shaft = 20.17 @ 2400 = 48500 kg

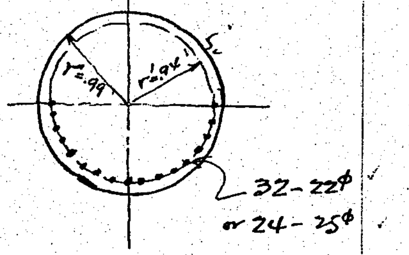
Seismic force = 48500 × .20 = 9,700 kg.

Caisson 3.3 × 3.3 × 6.5 = 70.8 @ 2200 = 155800 kg

Seismic force 155800 × 0.20 = 31,160 "

Seismic stress on shaft.

Loads	Hor. forces	Vert. forces	lev. arms	Moments.
D		65600		
D'	13120		× 8.18 =	107200
S		48500		
S'	9700		× 4.08 =	39600
	22820 kg	114100 kg	1.29 m	146800



A_s = 32 - 22 mmφ = 121.7 cm²

or 24 - 25 mmφ = 117.8 "

$P = \frac{117.8}{\pi \times 99^2} = .0038$ N = 114100 kg

r = 99 cm r' = 94 $r'/r = 94/99 = .95$ $e/r = 129/99 = 1.30$

from the prepared diagrams we get.

k = 0.670, C = .290

$f_c = \frac{114100}{.99^2 \times .29} = 40.1$ kg/cm²

$f_s = 15 \times 40.1 \, \frac{1 + .95 - .67}{.67} = 1148$ "

Stability of Pier:

Case 1. Dead load only.

Superimposed Dead load	=	65,600
weight of shaft	=	48,500
weight of Caisson	=	155,800
		269900 kg
friction 3.3 × 4 × 6.5 @ 1200	=	−103,000
		166,900 kg

Unit bearing pressure = $\frac{166900}{3.5^2}$ = 13600 kg/m² or (1.24 tons/□')

use 12 piles load on pile = 166900 ÷ 12 = 13.9 tons

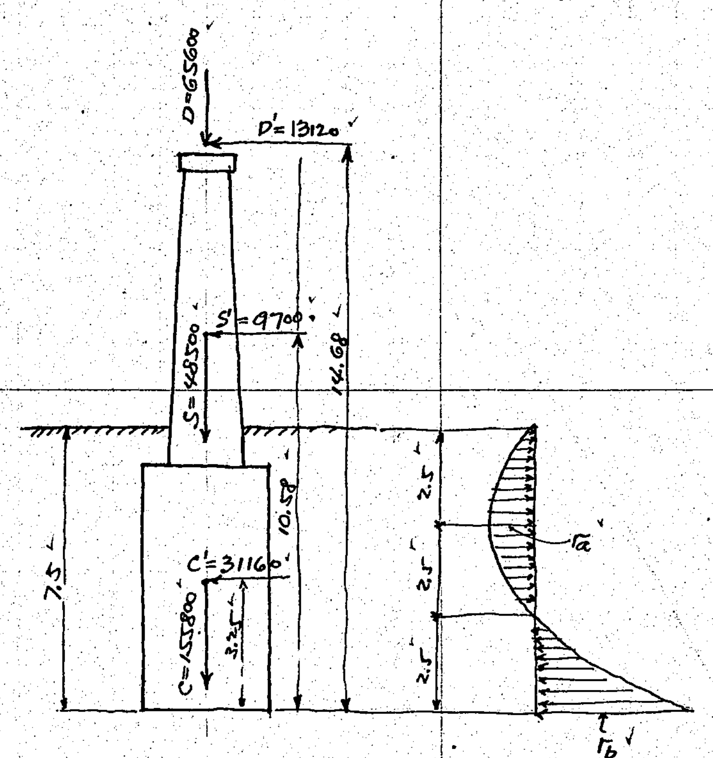
Case 2. Full loaded.

Total net load. 166,900
57800
224,700 kg

Unit bearing pressure = $\frac{224700}{3.5^2}$ = 18300 kg/m² or (1.68 tons/□')

Design of Sendai Bashi for Tottori Ken.

Case 3 Stability during Earthquake



moment due to seismic forces about center of bottom area

D'	13120	×	14.68 =	192500
S'	9700	×	10.58 =	102600
C'	31160	×	3.25 =	101400
	53980 kg			396500 kgm

frictional couple 3.3×6.5×3.3×1200 = 85000; 311500 kgm

unit pressure $r_b = \frac{12 \times 311500}{3.3 \times 7.5^2} = 20150$ kg/m²

" " $r_a = 20150 \div 3 = 6710$ "

Passive earth pressure at r_a = 1800 × 2.5 × 3.49 = 15700 kg/m²

for $\phi = 30°$ $\phi' = 18°-40'$ $\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.320}{0.680} = 1.94$

Passive pressure at r_b = 1800×7.5×1.94 = 26200 kg/m²

安全ノタメ捨石根固工ヲ施ス7

Design of Caisson shell

Temporary side pressure on well

y	L	Depth of water	water pressure	difference
1	490	1.4	1400	910
2	970	2.4	2400	1430
3	1370	3.4	3400	2030
4	1750	4.4	4400	2650
5	2050	5.4	5400	3350
6	2320	6.4	6400	4080
7	2540	7.4	7400	4860
7.5	2630	7.9	7900	5270

max. pressure on wall.
Internal pressure at bottom = 5270 kg/m²
External " 1.8m above " = 2240 "

at bottom.
moment = 1/10 × 5270 × 3.0² = 4740 kgm

Eff. depth required = $\sqrt{\frac{4740 \times 100}{100 \times 7.18}}$ = 25.7 cm

Use 37 cm eff. depth with 3 cm insulation

Steel area required = $\frac{4740 \times 100}{1200 \times \frac{7}{8} \times 37}$ = 12.2 cm²

use 16 mm bars at 15 cm ctoc = 13.42

Shear = 5270 × 1.5 = 7900 kg

unit shear = $\frac{7900}{100 \times \frac{7}{8} \times 37}$ = 2.44 kg/cm²

at section 1.0 m above bottom.

pressure on wall = 6900 − 2430 = 4470 kg/m²

moment = 1/10 × 4470 × 3² = 4020 kgm

Eff. depth ~~Steel area~~ required = $\sqrt{\frac{4020 \times 100}{100 \times 7.18}}$ = 23.7 cm

use 37 cm eff. depth with 3 cm insulation.

Steel area required = $\frac{4020 \times 100}{1200 \times \frac{7}{8} \times 37}$ = 10.35 cm²

use 16φ bars at 19.4 cm ctoc = 10.35

at section 2.0 m above bottom

pressure on wall = 5900 − 2180 = 3720 kg/m²

moment = 1/10 × 3720 × 3² = 3350 kgm

use eff. depth of 27 cm

Steel area required = $\frac{3350 \times 100}{1200 \times \frac{7}{8} \times 27}$ = 11.80 cm²

use 16φ bars at 17.0 cm ctoc = 11.84 "

Section at 3.5 m above bottom

pressure on wall = 2650 kg/m²

moment = 1/10 × 2650 × 9 = 2385 kgm

steel area req'd. = $\frac{2385 \times 100}{1200 \times \frac{7}{8} \times 27}$ = 8.42 cm²

use 12φ bars at 13 cm ctoc = 8.69 "

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CALCULATIONS FOR

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昭和六年七月

鳥取縣千代橋豫算設計

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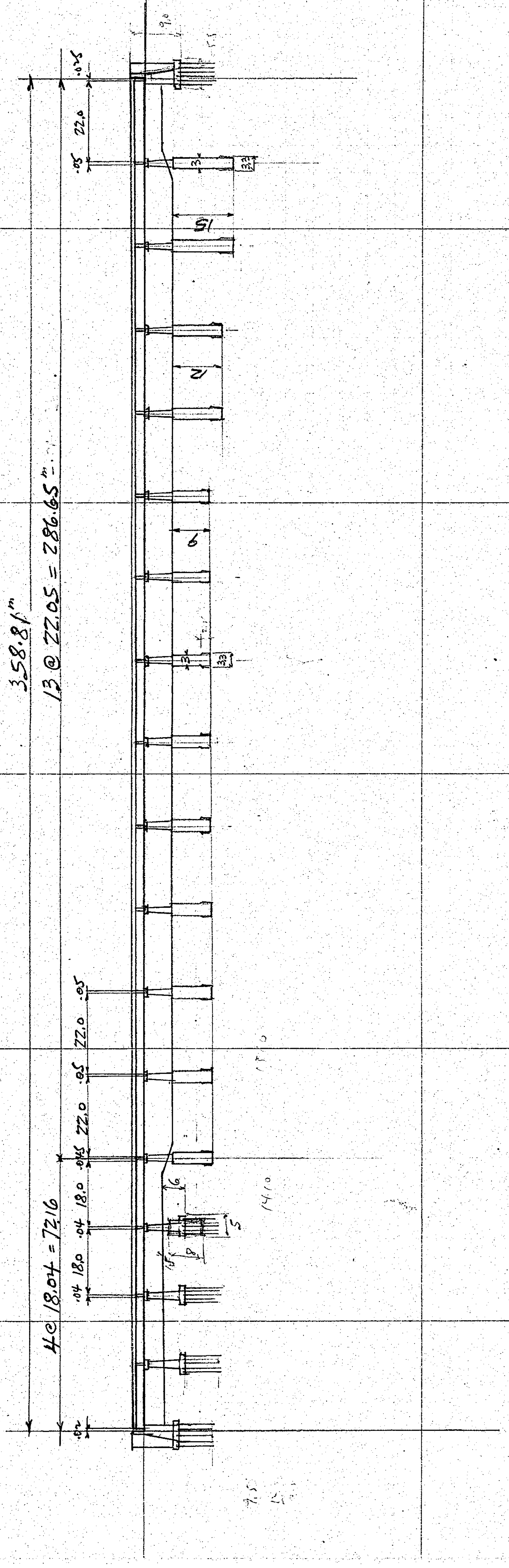
CALCULATIONS FOR
鳥取縣 千代橋



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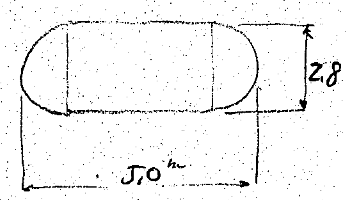
CALCULATIONS FOR

千代橋

鉄道省橋梁

well



Bottom area 2.8 × 4.1 = 11.5 m²

11.5 × 13.5 = 155. m³
2 × 4 × 6 = 48
203 @ 2,000 = 400,000

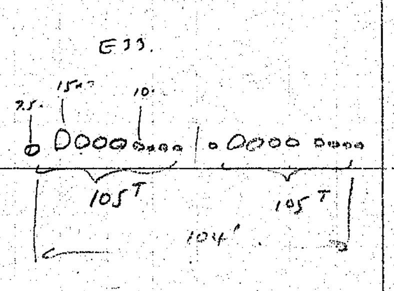
Loads on pier

Dead Load

superstructure 40.0 ton

pier 400.0

440



Live Load.

locomotive say 120

impact 75% 90

210.0

650.0 tons

Friction 周 13.0m × 高 13.0m = 169.0m² @ 1.2 sq ton = -200 tons.

650

Total Load on pier = 450 tons net.

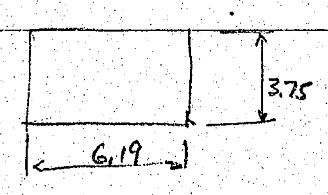
unit bearing pressure at bottom of well = $\frac{450}{11.5}$ = 39.1 tons/□m

or = 3.58 tons/□'

Spread Base

Bottom area = 6.19 × 3.75 = 23.2 □m

Dead Load



Superstructure 40.0

pier 2 × 11 × 4 = 88 @ 2.2 190.0

230

Live Load 210

440 tons

Unit bearing pressure = $\frac{440}{23.2}$ = 19.0 tons/m² or 1.75 tons/□'

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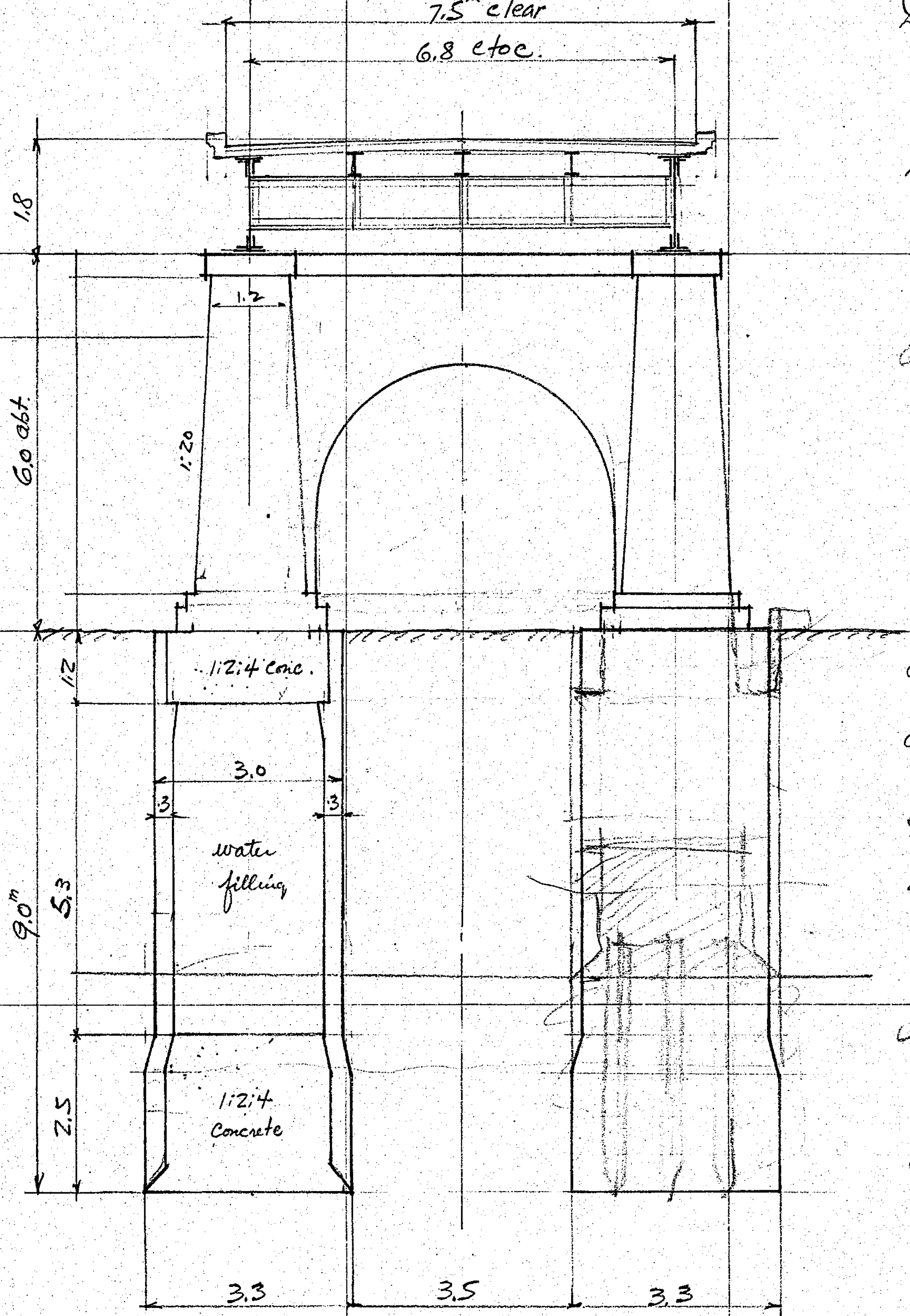
CALCULATIONS FOR

千代橋

Pier.



Superimposed Loads on pier.

Dead Load of Super structure.

22.0 @ 5740 kg = 126.0 tons.

Live Load.

22.0 @ 3750 = 83.0

Extra 6.0

215 tons

Weight of pier

Shaft 22.

Strut 7

(1)

30.

well 2@61.5 = 127

157 @ 2400 = 330

Total load on bottom area = 545 tons

Friction

10m × 15 @ 1200 = −180

Total load on one pier net = 365 tons

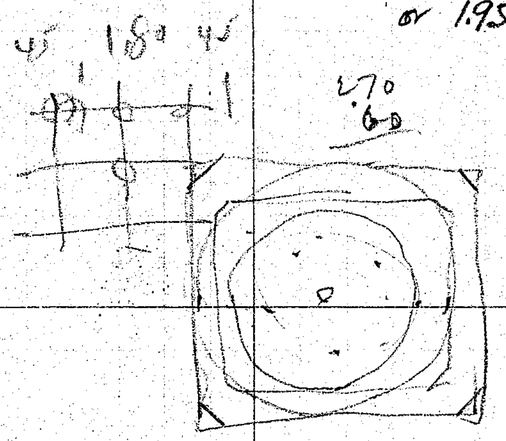
bottom area

2 − 3.3Φ = 17.1 m²

Unit Bearing pressure

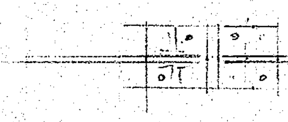
$= \frac{365}{17.1} = 21.4$ tons/m²

or 1.95 tons/□'



Estimate of cost for one pier (9m well)

Concrete	110 m³	@	15 =	1650
Reinforcements	5 tons	@	85 =	425
Forms	470 m²	@	1.5 =	705
well sinking	18 m	@	40 =	720
misc				100
				3600 円



Pier having 12m wells.

Concrete	130 m³	@	15 =	1950
Reinforcements	6.5	@	85 =	550
Forms	600	@	1.5 =	900
well sinking	24	@	50 =	1200
misc				100
				4700 円

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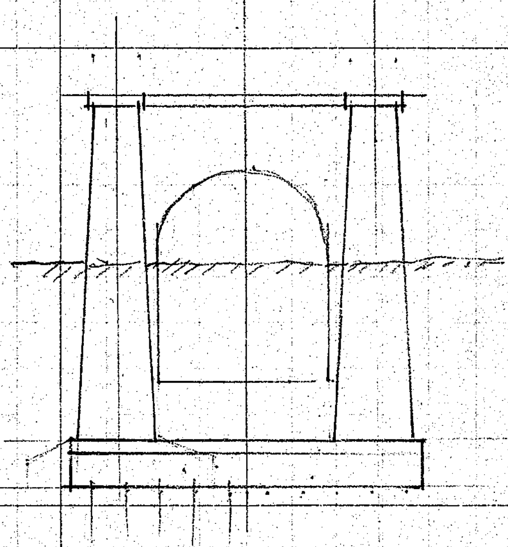
千代橋

Pier 15m well.

Concrete	150 m³ @ 15	=	2250
Reinforcements	8.0 ton @ 85	=	680
Forms.	730 m² @ 1.5	=	1,100
Well sinking	30 m @ 5	=	1,650
Well island	1 @ 3000	=	3000
misc			120
			8800 円

Pier of Spread Base



Concrete.	100 m³ @ 15	=	1500
Reinforcements.	3.5 ton @ 85	=	300
Forms	180 m² @ 1.5	=	270
piles	50 本 @ 11	=	550
excavation	360 m³ @ 1	=	360
Sheet piles + pumping			500
misc			70
			3650 円

これを 6m well にすれば 2700 円

Abutment.

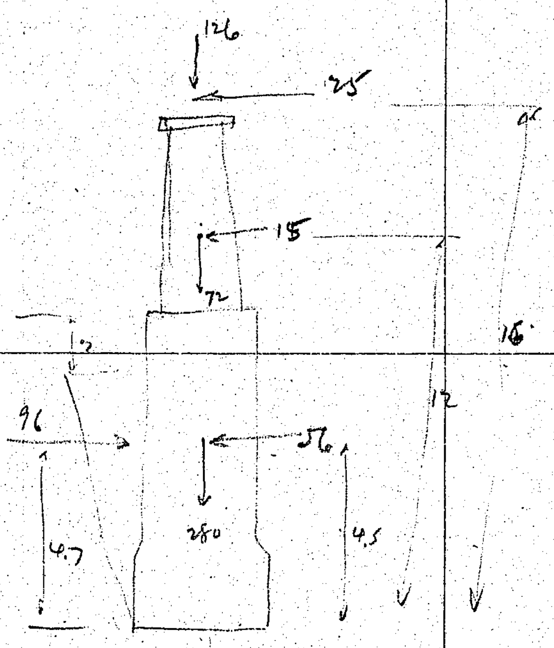
Concrete	250 m³ @ 15	=	3750
Reinforcement	6.0 ton @ 85	=	510
forms	350 m² @ 2.	=	700
割栗	20 @ 3	=	60
piles	60 本 @ 11	=	660
人造石及踏掛石等			200
Excavation 水かえ共	800 @ 1.5	=	1200
misc			130
			7250 円

Summary of Cost for substructures

piers	15m wells	2 @ 8800	=	17,600	
	12m "	2 @ 4700	=	9400	
	9m "	9 @ 3600	=	32,400	
	6m "	3 @ 2700	=	8,100	
					67500
Abutments		2 @ 7250	=		14500
					82000 円

千代橋

Stability of pier



Hor	vert			
	126	× 0	= 0	
25		× 16	=	400
	72		= 0	
15		× 12	=	180
	280		= 0	
56		× 4.5	=	250
96	478			830 ton m.

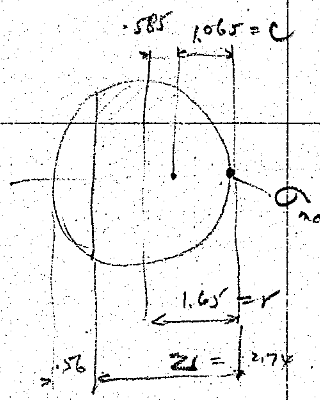
$96 \times 4.7 = 450$ → 380

fr. couple $3 \times 7 \times 120 \cdots \times 35 \times 2 \times 2 = 100$ → 280 ton m.

$$\Sigma e c = \frac{280}{478} = 0.585 \text{ m}$$

$$\frac{96 \times r}{3 \times 9 \times 2} = 4.58 \text{ ton/m}$$

$1.6 \times 2 \times 3 = 9.6$



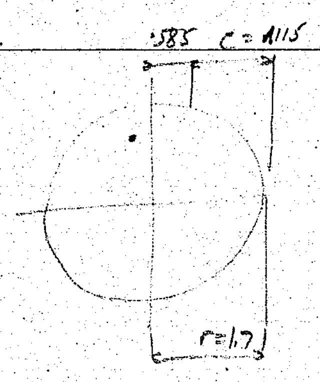
$$\frac{Z}{c} = 2.33 + 0.58\left(\frac{1.065}{1.65}\right)^2 = 2.57$$

$$Z = 2.57c = 2.57 \times 1.065 = 2.74 \text{ m}$$

$$\sigma_{max} = \left(0.372 + 0.056\frac{1.065}{1.65}\right)\frac{239}{1.065\sqrt{1.65 \times 1.065}} = .408 \times 239$$

$$= 69 \text{ tons/□m or } 6.3 \text{ ton/□'}$$

for 3.4φ base.



$$\sigma_{max} = \left(.372 + .056 \times \frac{1.115}{1.7}\right)\frac{239}{1.115\sqrt{1.7 \times 1.115}} = 63 \text{ ton/□m or } 5.75 \text{ ton/□'}$$

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CALCULATIONS FOR

千代橋

Estimate of Cost for Super Structure.

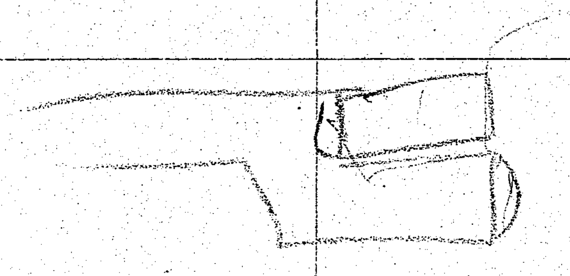
Structural steel	22m spans	13 @ 30.4 = 395.2	
	18m "	4 @ 22.5 = 90.0	
			485.2
	Metals for Exp. jts. 17 @ 0.81 =		13.8
			499.0
			Call this 500 kg tons.
		500 tons @ 120 円	= 60,000

Deck.
Area. 7.5 × 358.8m = 2690 sq. meter

Concrete slab	2690 × 0.155 =	418		
coping	358.8 × 0.1 × 2 =	72		
		490 m³ @ 15 =	7350	
Reinforcements	490 m³ @ 130 kg =	64 kg tons @ 85 =	5440	
Forms	8.3m × 358.8 =	2980 m² @ 1.5 =	4480	
人造仕上	2.1 × 358.8 =	753 m² @ 3.6 =	2700	
Granolithic pav.	7.5 × 358.8 =	2690 m² @ 2.5 =	6720	
Handrails	717.6m @ 85 kg =	61. kg tons @ 200 =	12200	
Light poles	2 × 8 =	16 sets. @ 72 =	1160	
Light pedestals	=	4 @ 500 =	2000	
Electric wiring	=	358.8m @ 6 =	2150	
Wing Handrails		4 @ 100 =	400	
				44600
				104600 円

Summary of cost

Super structure			
	Structural steel	60000	
	Deck construction	44600	
			104600
Sub structure			
	Piers	67500	
	abutments	14500	
			82000
			186600 円. (17 spans 1 系)



千代橋 7.

全長を 16 span に割あてたる場合.

358.8 ÷ 16 = 22.42
Length of girder 22.42 − 0.7 = 22.35 m

Structural steel say $30.4 \times \frac{22.35^2}{22.0^2} = 31.5$

16 spans @ 31.5 = 504
Expansion jt's. 16 @ .81 = 13
517 tons @ 120 = 62000
Deck construction complete = 44600
106600

Sub structures.

Piers 15m wells. — 2 @ 8800 = 17,600
12m " — 2 @ 4700 = 9,400
9m " — 9 @ 3600 = 32,400
6m " — 2 @ 2700 = 5400
64,800

Abutments 2 @ 7250 = 14500
79,300

Summary of costs
Super-structure complete = 106600
Sub-structure " = 79300
185900 円 (16 equal spans, 1 号)