JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

昭和三年八月

岡山縣吉井川架橋

芦田橋設計々算書

及材料調書

CALCULATIONS FOR

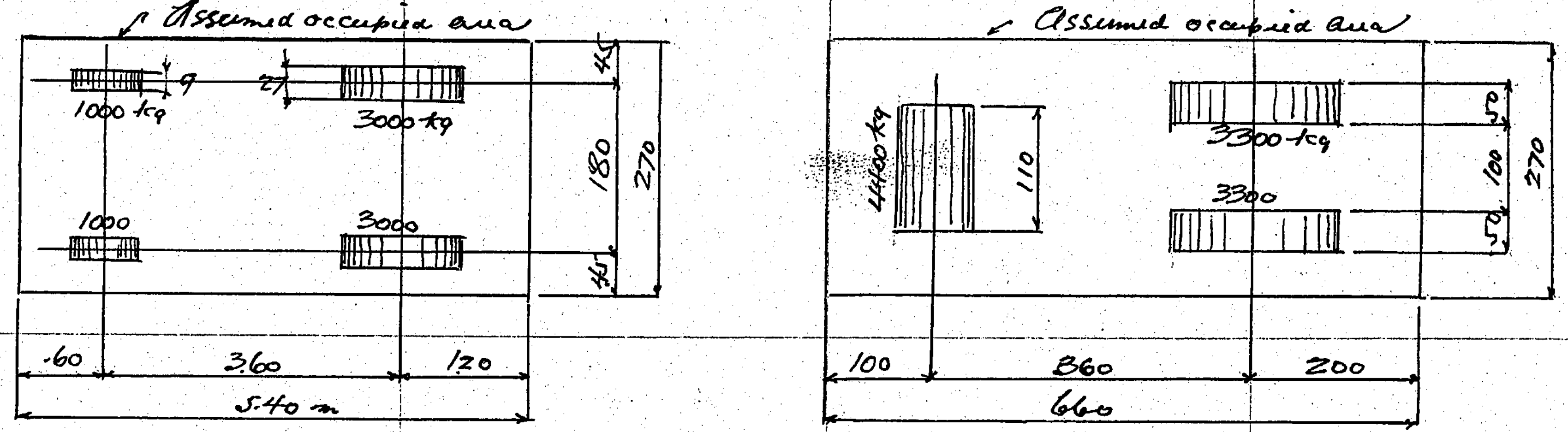
Ashida-Bashi for Okayama-Ken.

2 truss spans 10 panels @ 3.75 m = 37.5 meters
Clear Roadway 5.4 meters between curb lines pavement 2" or 5cm Soliditet or Granolithic pavement on reinforced concrete slab. Handrails ornamental cast iron design.

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

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

8 ton motor truck loading 11 ton road roller.



2 rows of motor traffic on roadway with occupied width of 270 cm each; Unoccupied space at front, rear and sides of motor truck shall be filled with uniform load specified above. One roadroller on span
Impact for motor truck loading

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

No impact for roadroller and uniform load.

Allowable Working strength
Structural Steel or Reinforcing Bars

Tension net	1200 kg/cm²
Extreme fibre stress net	1200 "
Shear of web gross section	900 "
Compression member	
$1500(1-0.0055\ l/r)$ not over	1000 "

where l = length of member in cm
r = least radius of gyration in cm²

Compression flange of girder	
$1200(1-0.012\ l/b)$ not over	1100 "

where l = Unsupported length of flange in cm
b = width of flange in cm

Shear on shop driven rivets (machine driven)	850 kg/cm²
Field " " and turned bolts (machine driven)	750 "
Shear on pin	900 "
Bearing on shop driven rivets (machine driven	1700 "
" " field " "	1500
" " pin	1800

Roller 45d kg where d = diameter in cm

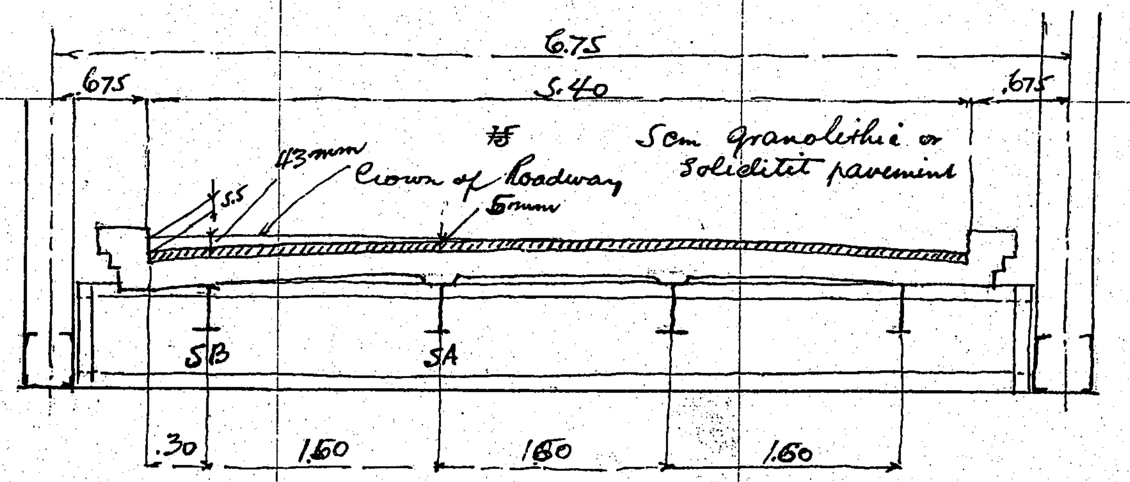
Ashida- Bashi for Okayama-Ken

Allowable Working strength
Concrete 1:2:4 mixture

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

Considering wind or temperature stress in addition to dead, live and impact stresses ~~the~~ allowable working strength shall be increased 25% in case of earthquake increase stress 80%.
Seismic accelleration 1500 mm/sec² $k = 0.15$
1000

Cross section of bridge assumed as shown on sketch



Design of floor slab. span length 1.60 meters
Dead Load

5cm solidititi or granolithic	@ 24 kg	=	120
15cm Concrete slab.	@ 24 "	=	360
filler and allowance			20
			500 kg per sq meter

Dead Load moment $= 1/10 \cdot 500 \cdot 1.60^2 = 128.$ kgm
Dead Load shear $= 1/2 \cdot 500 \cdot 1.60 = 400$ kg.

Live Load motor truck loading
Rear wheel Concentration 3000
30% impact 900
3900 kg.
Front wheel Concentration
$3900 \div 3 = 1300$ kg.

Distribution of wheel Concentration on slab.
Thickness of pavement 5cm
Longitudinal distribution a Contact between wheel and pavement 20
Distribution 2@5.0 10
30 cm

Transverse distribution $b = 27 + 10 = 37$ cm

Effective width $E = 2/3 l + a$ where l = span length
$= 2/3 \cdot 1.60 + .30 = 1.37$

Load per meter strip $3900 \div 1.37 = 2850$ kg for rear wheel

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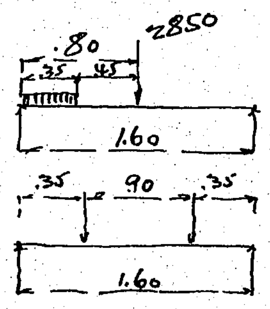
CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken



moment per meter strip $\frac{2850}{2} \times 0.80 = 1140$ kgm

For continuity of slab $1140 \times 0.8 = 911$ kgm

unif. load $500 \times \frac{.35^2}{2 \times 1.60} \times 0.80 = \frac{14}{925}$

max end shear symmetrical loading for center span of slab.

max shear = 2850 kg.

Summary for moments and shears

	moments	shear
Dead Load	128.	400
Live Load	925	2850
	1053 kgm	3250 kg.

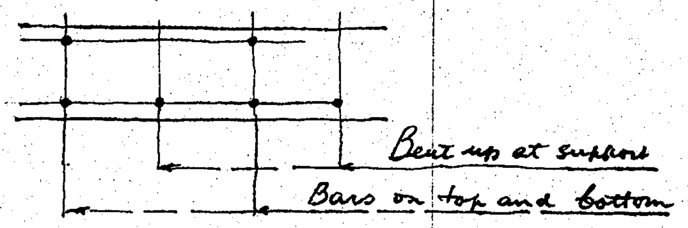
Effective depth required for slab $f_s = 1200$ kg/cm² $f_c = 45$ kg/cm²

$$R = \frac{M}{bd^2} \qquad d = \sqrt{\frac{M}{bR}} \qquad \text{where } R = 7.18$$

$$d = \sqrt{\frac{105300}{100 \times 7.18}} = 12.1\text{cm}$$ use 15 cm slab. 2.5 cm insulation at bottom of slab. depth say 12.5 cm

Steel area reqd $= \frac{105300}{7/8 \times 12.5 \times 1200} = 8.03$ □cm per meter strip

13mmφ bar 1.33 □cm spacing of bar $\frac{1.33 \times 100}{8.03} = 16.6$ cm use 15 cm spacing



stresses in Concrete and steel reinforcement as double reinforcement

$d = 12.5$ cm $d' = 2.5$ cm $\frac{d'}{d} = .20$

tensile steel $1.33 \times \frac{100}{15} = 8.87$ □cm per meter

% $\frac{8.87}{1250} = .71$ %.

Compressive steel = .355 % $k = 0.350$

$$f_c = \frac{105300}{100 \times 12.5^2 \times .173} = 38.9 \text{ kg/□cm}$$

$$f_s = \frac{105300}{100 \times 12.5^2 \times 0.0061} = 1108 \text{ kg/cm}^2$$

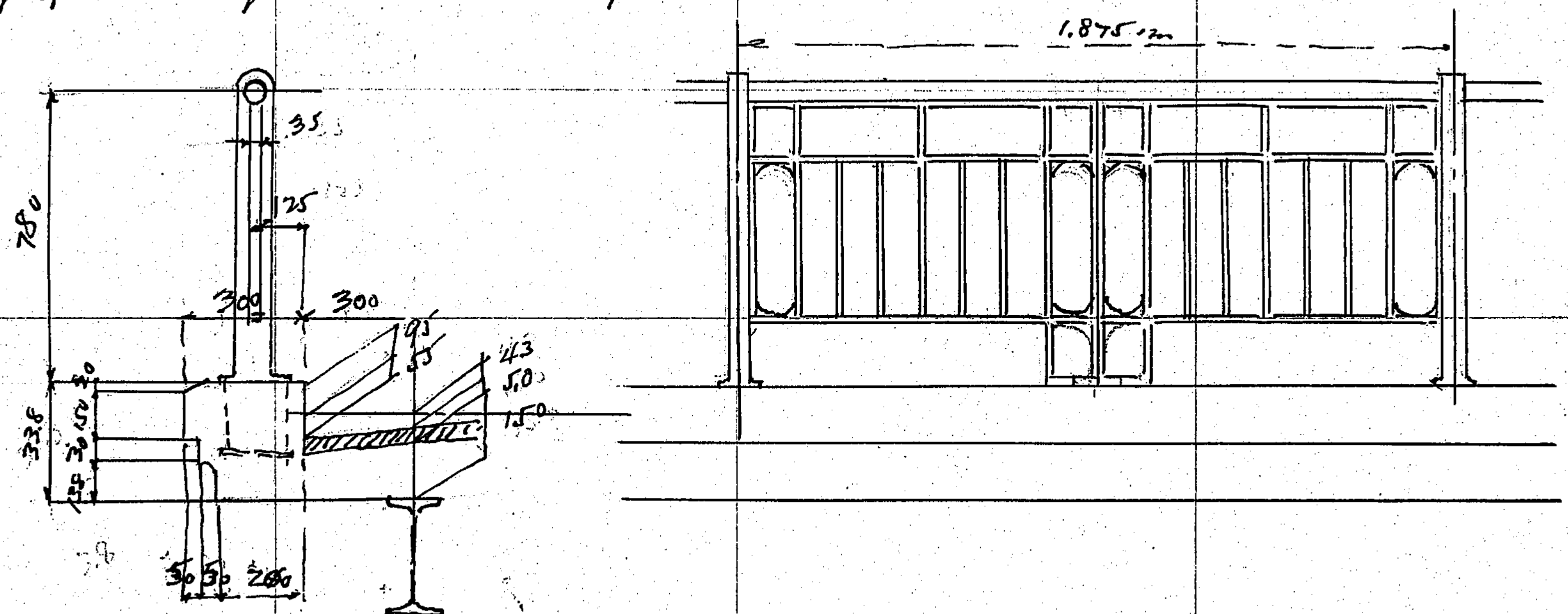
Bent up reinforcing bars shall be lapped over stringer

Bond stress	dia	circumference			for meter strip
straight bar	13mm	4.08	× 3.33	=	13.6
bent up bars lapped	"	4.08	× 6.67	=	27.2
					40.8 cm

Unit bond $= \frac{3250}{7/8 \times 12.5 \times 40.8} = 7.3$ kg/cm². < 9.

use deformed bars ~~to~~ increase bond stress for safety of structure.

Overhanging slab beyond outside stringer



增田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken

Approximate weight of Cast iron Handrail.

Grate Cross section. 1.6 × 3.5 = 5.6 @ .785 = 4.4 kg per lin. meter.

6 -	4.4 kg ×	.57	=	15.0
8 -	4.4 .. ×	.42	=	14.8
4	4.4 ×	.75	=	13.2
2	4.4 ×	.25	=	2.2
misc.				3.8
				49.0
longl. 3 -	4.4 ×	~~1.875~~ 1.85	=	24.4

73.4

Handrail Post 60 cm² @ .785 × .82 = 38.5

111.9 ÷ 1.875 = 59.6 kg per meter

2½" pipe 5.793 # per ft 8.6

68.2 " " "

Call this 70.0 kg per meter.

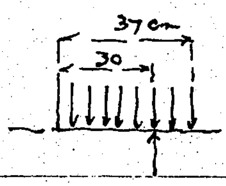
Approximate weight of Coping.

.300 × .338 = 0.101
less .015
0.086 @ 2400 = 206 kg per meter

Dead Load moment

	load		arm		
Handrail	70	×	.425	=	29.8
Coping	206	×	.44	=	90.6
Slab and pavement 500 × .3 =	150	×	.15	=	22.5
	426		.336		142.9

Live Load motor truck rear wheel at curb line assumed



Distribution on ℄ stringer assumed 2 × .30 + .20 = .80
Transverse distribution .37
Load on .30 $3900 \times \frac{30}{37} = 3160$ kg
For one meter strip 3160 ÷ 0.8 = 3950
Live Load moment = 3950 × 0.15 = 592. kgm
shear = 3950 kg.

Summary for moments and shears

	moment	shear
Dead Load	143	426
Live Load	592	3950
	735 kgm	4376 kg

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

$$d = \sqrt{\frac{735 \times 100}{100 \times 7.18}} = 10.1 \text{ cm}$$

Use 15 cm slab. Effective depth 12.5 cm

$$\text{Steel area reqd} = \frac{73500}{7/8 \times 12.5 \times 1200} = 5.60 \text{ cm}^2$$

Use 13 mm bar 15 cm spacing 1.33 × 6.67 = 8.85
9 mm bar " .64 × " = 4.26
13.11 cm² used

Circumferential area 13 mm bar 4.08 × 6.67 = 27.20
9 " bar 2.83 × 6.67 = 18.90
46.10 cm²

$$\text{Unit bond} = \frac{4376}{7/8 \times 12.5 \times 46.10} = 9.0 \text{ kg/cm}^2$$

Use deformed bars in slab to carry necessary bond stress.

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CALCULATIONS FOR

Ashida - Bashi for Okayama-Ken.

Design of I Beam stringer   span length 3.75 meters   spacing 1.6 meters
Inside stringer SA.

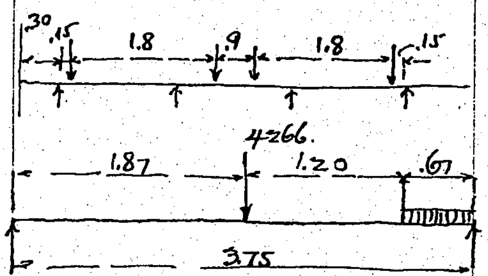
Dead Load   floor slab and pavement $500 \times 1.6 = 800$
beam assumed 50
850 kg/m

Dead Load moment $= \frac{1}{8} \cdot 850 \cdot 3.75^2 = 1495$ kgm
" " shear $= \frac{1}{2} \cdot 850 \cdot 375 = 1594$ kg.

Live Load   motor truck rear wheel concentration with impact $= 3900$ kg
front " " " $= 1300$ "

max load on stringer.

$3900 \cdot \frac{.15}{1.60} = 366$
3900
4266 kg.



Uniform live load $500 \cdot 1.60 = 800$ kg per meter.
Reaction $\frac{800 \cdot .67^2}{2 \cdot 3.75} = 48.$ kg.

moment
Due to motor truck $\frac{4266}{2} \cdot 1.87 = 3990$
" " Uniform load $48 \cdot 1.87 = 90$
4080 kg meter

Summary for moments and shears

	moments	shear
Dead Load	1495	1594
Live Load	4080	4961
	5575 kgm	6555 kg

max End shear
Unif. $\frac{800 \cdot 2.55^2}{2 \cdot 3.75} = 695$
motor truck 4266
4961

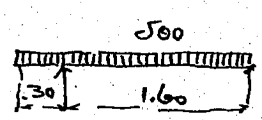
Section modulus required $= \frac{5575 \cdot 100}{1100} = 507$

Use $300 \cdot 150$ I @ 48.34 kg   Sm = 633.2
Unit stress $= \frac{5575 \cdot 100}{633.2} = 880$ kg/cm²

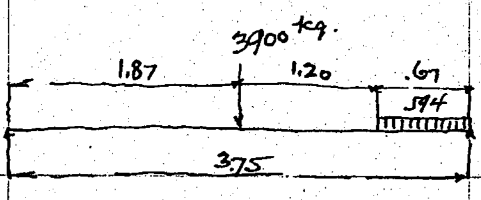
Stringer SB.   Span length 3.75 meters
Dead Load   Direct from cantilever 426
due to Cantilever $142.9 \div 1.60 = 89$
floor between stringers $500 \cdot 0.8 = 400$
915
stringer assumed 50
965 kg per lin meter

Dead Load moment $= \frac{1}{8} \cdot 965 \cdot 3.75^2 = 1695$ kgm
Dead Load shear $= \frac{1}{2} \cdot 965 \cdot 3.75 = 1810$ kg

Live Load   motor truck loading rear wheel concentration with impact $= 3900$ kg.
assumed on ℄ stringer.



Uniform live load. $500 \cdot \frac{1.90}{1.60} = 594$ kg per meter



Reaction $\frac{594 \cdot .67^2}{2 \cdot 3.75} = 35.5$ kg.

Bending moment
Due to motor truck $\frac{1950}{\cancel{1850}} \cdot 1.87 = \frac{3640}{\cancel{3460}}$
" " Unif. load $35.5 \cdot 1.87 = 67$
~~3527~~ kgm.
3707

Max End shear
Uniform load $\frac{594 \cdot 2.55^2}{2 \cdot 3.75} = 515$
motor truck 3900
4415 kg.

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CALCULATIONS FOR

Ashida-Bashi for Okayama-ken

Summary for moments and shears

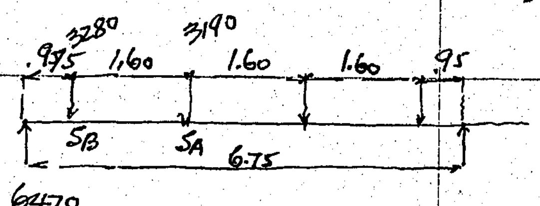
	moment	shear
Dead Load	1695	1810
Live Load	~~3527~~ 3707	4415
	~~5222~~ 5402 kgm	6225 kg.

Use 300 × 150 I @ 48.34 kg. Sm = 633.2

Unit Stress = $\frac{\cancel{522200}\ 540200}{633.2}$ = ~~825~~ 855 kg/cm²

Design of Intermediate Floor Beam

Dead Load.



6470

Beam assumed 160 kg per lin meter

Dead Load concentration without cantilever action.

On SA. 500 × 1.60 = 800
stringer = 50
850 × 3.75 = 3190

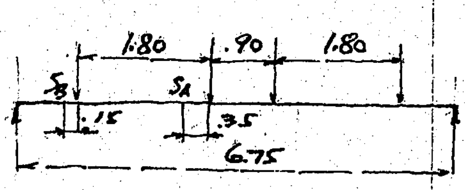
On SB 500 × 0.8 = 400
Handrail coping + slab 426
stringer 50
876 × 3.75 = 3280

Moment at SA. 6470 × ~~2.55~~ 2.575 = ~~16600~~ 16650
3190 × 1.60 = 5100
~~11400~~ 11550

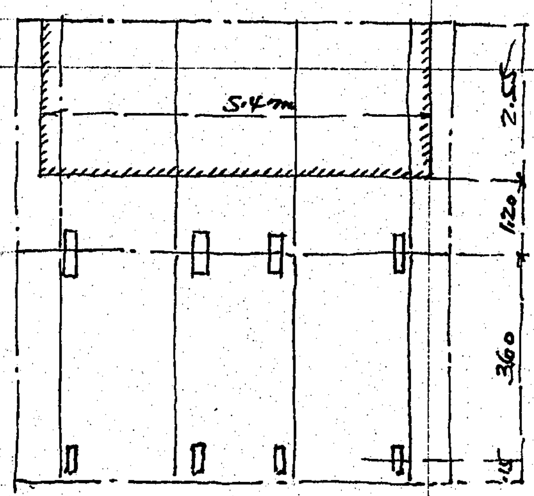
Beam $\frac{1}{8}$ × 160 × 6.75² = 910
~~12310~~ 12460 kgm

End shear 6470 + 540 = 7010 kg.

Live Load motor truck loading rear wheel concentration with impact 3900 kg
front " " " 1300 "



Rear wheel direct on floor beam
Front wheel of motor truck and uniform live load through stringers connected to floor beam assumed.



Reaction on stringer due to front wheel of motor truck

On SA. 1300 × $\frac{.15}{\cancel{3.75}\ 1.60}$ = 122
1300
1422

On SB. 1300 − 122 = 1178

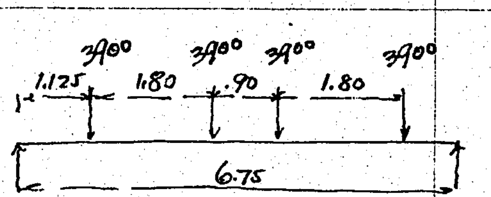
Reaction on floor beam

SA 1422 × $\frac{.15}{3.75}$ = 57 kg

SB 1178 × $\frac{.15}{3.75}$ = 47 kg.

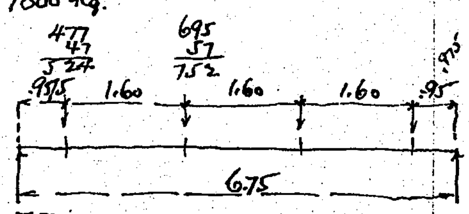
Uniform live load 500 kg/m²

On stringer SA 500 × 1.60 = 800 kg.
" " SB 500 × 1.10 = 550 kg.



Reaction on floor beam from SA 800 × $\frac{2.55^2}{2 \times 3.75}$ = 695 kg
SB 550 × $\frac{2.55^2}{2 \times 3.75}$ = 477 kg.

7800 kg.



752
524
1276

Moment due to rear wheel of motor truck

moment 7800 × 2.925 = 22800
3900 × 1.800 = −7020
15780

Moment due to front wheel of motor truck and Unif. load

moment 1276 × 2.575 = 3280
524 × 1.60 = −840
2440

Total moment 15780 + 2440 = 18220 kgm

Max End shear 7800 + 1276 = 9076 kg.

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CALCULATIONS FOR

Ashida-Bashi for Okayama Ken

Summary for moments and shears.

	moment	shear
Dead Load	12460	7010
Live Load	18220	9076
	30680 kgm	16086 kg.

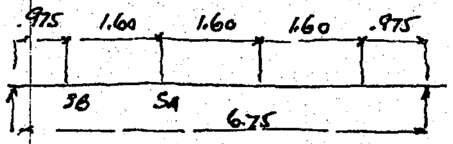
web assumed 67.0 × .8 = 53.6 □cm 1/8 web = 6.70 cm²
Back to back of Ls 68.0 cm Effective depth = 63.6
flange stress = $\frac{3068000}{63.6}$ = 48300 kg.

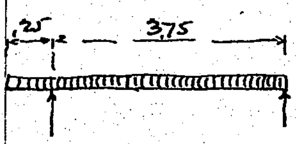
flange section reqd = $\frac{48300}{1200}$ = 40.20
− 6.70
33.50 □cm net

Use 2Ls 125 × 90 × 10 = ~~40.50~~ 41.00 − 4.4 = ~~36.10~~ 36.60 □cm net Rivet 19mm

Design of End floor beam span length 6.75 meters

Dead Load Dead Load Concentration





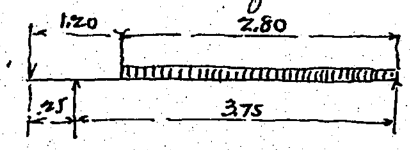
SA. 850 × $\frac{4.0^2}{2 \times 3.75}$ = 1820 kg

SB 876 × $\frac{4.0^2}{2 \times 3.75}$ = 1870
3690

Dead Load beam assumed 160 kg/m moment at SA. 3690 × 2.575 = 9500
1870 × 1.60 = −3000
6500
Beam 1/8 × 160 × 6.75² = 910
7410 kgm

End shear 3690 + 540 = 4230 kg.

Live Load motor truck loading rear wheel Concentration with impact = 3900 kg.
motor truck rear wheel carries direct by floor beam and uniform live load through stringer to floor beam assumed



motor truck loading 3900 × $\frac{4.00}{3.75}$ = 4160 kg on floor beam

Due to Uniform load
SA = 800 × $\frac{2.80^2}{2 \cdot 3.75}$ = 836 kg

SB 550 × $\frac{2.80^2}{2 \cdot 3.75}$ = 575
1411

moment due to motor truck 4160 × 2 × 2.925 = 24350
4160 × 1.800 = 7500
16850

Moment due to uniform load 1411 × 2.575 = 3620
575 × 1.60 = 920
2720

Total moment 19570 kgm

max End shear motor truck 8320
Unif. load 1411
9731 kg

Summary for moments and shears

	moment	shear
Dead Load	7410	4230
Live Load	19570	9731
	26980	13961

web assumed 67. × 0.8 = 53.6 1/8 web = 6.70 □cm
Back to back of Ls 68.0 cm Effective depth = 63.08
flange stress = $\frac{26980 \times 100}{63.08}$ = 42700

Section req'd = $\frac{42700}{1200}$ = 35.60
6.70
28.90 □cm net

Use 2Ls 100 × 90 × 10 = 36.00 − 4.4 = 31.60 □cm net 19mm Rivets

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CALCULATIONS FOR

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Approximate weight of intermediate floor beam

flange	4Ls	125 × 90 × 10	@ 16.09	× 6.43	=	414.0	
web	1Pl.	670 × 8	@ 42.10	× 6.43	=	271.0	
End Connection	4Ls	90 × 90 × 10	@ 13.34	× .66	=	35.2	
fills	4Pls.	90 × 10	@ 7.06	× .50	=	14.1	
stringer conn	8Ls	90 × 90 × 10	@ 13.34	× .66	=	70.4	
fills	8Pls.	90 × 10	@ 7.06	× .50	=	28.2	
"	8Pls.	90 × 10	@ 7.06	× .30	=	17.0	
Int. Stiffs	6Ls	75 × 75 × 9	@ 9.96	× .68	=	40.6	
		Rivet Heads and variation				31.0	
						921.5	Call this 925 kg.

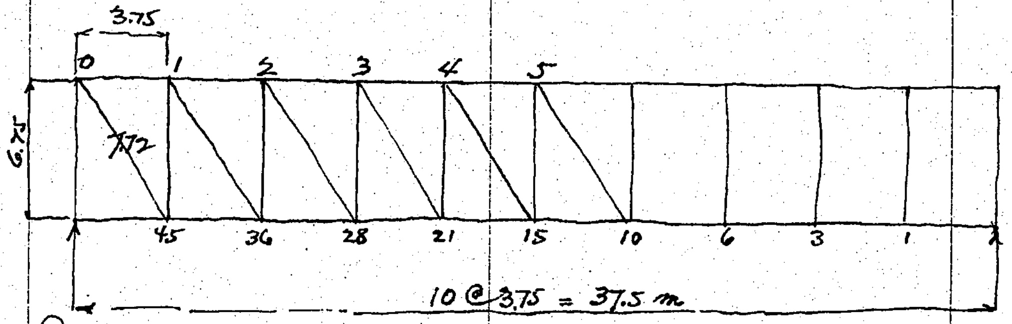
925 ÷ 6.43 = 144 kg per lin. meter.
925 ÷ 3.75 = 247 " per meter of truss.

Approximate weight of End floor beam

flange	4Ls	100 × 90 × 10	@ 14.13	× 6.43	=	363.	
		Other details same as intermediate F.B.				507.5	
						870.5	kg.

870 ÷ 6.43 = 135 kg per lin. meter.

Lower Lateral Bracings



wind load
loaded chord 400
unloaded chord 200
600 × 3.75 = 2250 kg

$\sec\theta = \frac{7.72}{6.75} = \cancel{1.134}$ 1.143

$\tan\theta = \frac{3.75}{6.75} = 0.555$

Diagonal stresses

Panel				Shear		stress	SR 1200 kg/cm²	19mm Rivet	Rivet	
0-1	$\frac{2250}{10}$ ×	4.5	=	10120	× ~~1.134~~ 1.143 =	~~11500~~ 11590	~~9.58~~ 9.65	5.4	6	1L 125×75×10 @ = 19.00 cm²
1-2		36	=	8100		~~9190~~ 9250	~~7.65~~ 7.72	4.3	5	do
2-3		28	=	6300		~~7150~~ 7200	~~5.95~~ 6.00	3.4	4	1L 75×75×9 = 12.69 cm²
3-4		21	=	4730		~~5360~~ 5410	~~4.47~~ 4.50	2.5	3	
4-5		15	=	3380		~~3840~~ 3860	~~3.20~~ 3.22	1.8	3	

Chord stresses

Panel						
0-1	2250 ×	4.5	×	0.555	=	5620 kg.
1-2		8.0	×		=	10000
2-3		10.5	×		=	13100
3-4		12.0	×		=	15000
4-5		12.5	×		=	15600

Approximate weight of bottom lateral Bracing

4Ls	125 × 75 × 10	@ 14.91 kg	× 7.5 =	448	
6Ls	75 × 75 × 9	@ 9.96	× 7.5 =	448	
				896	
Center Connection say		5 @ 25	=	125	
			2 ×	1021	= 2042 kg.

2042 ÷ 37.5 = 54.5 kg per lin. meter

Panel load 54.5 × 3.75 = 204 kg.

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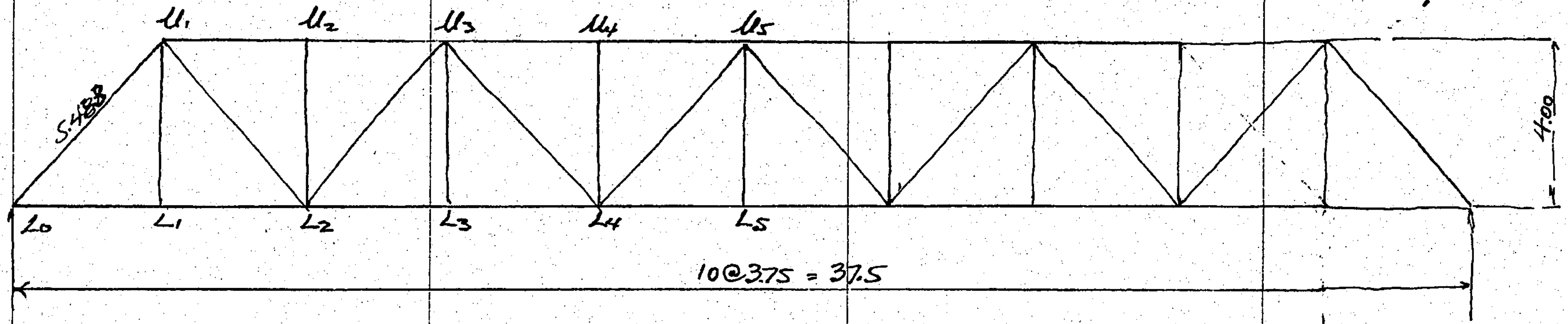
CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken

Design of truss span length 10 @ 3.75 = 37.5 metres
Dead Load Panel Concentration.
Handrail coping floor and stringer 6470
Floor beam 925 ÷ 2 462
Lateral Bracing 204 ÷ 2 102
7034 kg.

Truss assumed 500 kg per metre
Upper half 250 × 3.75 = 937
Lower half " " = 937
Lower chord panel Concentration 7034
937
7971
Upper chord panel Concentration 937
8908 kg.

Dead Load End panel
Handrail coping floor and stringer 850
876
1726 × ($\frac{3.75}{2}$ + 0.25) = 3660
Floor beam 870 ÷ 2 435
Lower lateral 51
truss assumed 937
5083 kg.



$\sec\theta = \frac{5.483}{4.00} = 1.370$ $\tan\theta = \frac{3.75}{4.00} = 0.937$

W sec θ = 8908 × 1.370 = 12200 kg
W tan θ = 8908 × 0.937 = 8350 "

Chord stresses

L0-L2	8350 × 4.5	=	37600 kg
U1-U3	8.0	=	66800 "
L2-L4	10.5	=	87700 "
U3-U5	12.0	=	100000 "
L4-L4	12.5	=	104500 "

Diagonals

L0-U1	12200 × 4.5	=	55000
U1-L2	3.5	=	42700
L2-U3	2.5	=	30500
U3-L4	1.5	=	18300
L4-U5	0.5	=	6100

Hanger U1-L1 U3-L3 and U5-L5 7971 kg
Vertical U2-L2 and U4-L4 937 "

Live Load on truss Uniform live load w = $\frac{100.000}{170 + 37.5}$ = 482 kg/m²

motor truck rear wheel 3000
impact $\frac{20}{60 + 37.5}$ = 20.5 % 615
3615 kg.
Front wheel 3615 ÷ 3 = 1205 kg.

Panel Concentration One motor truck on one truss Rear wheels 2 @ 3615 = 7230 kg
Front wheels 2 @ 1205 = 2410 "

Uniform live load 482 × 2.70 = 1300 kg per metre
U1 1300 × 2.55 = 3320 kg
U2 × 3.30 = 4290 "
U3 × 3.75 = 4870 "

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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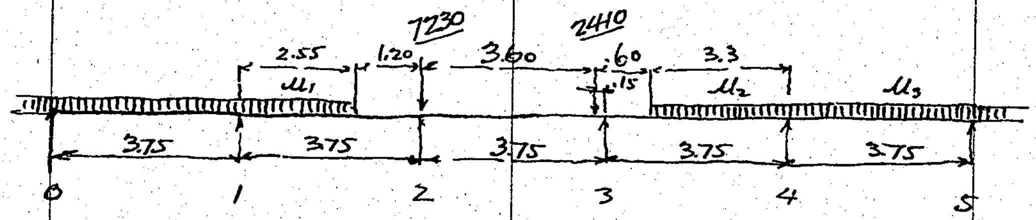
CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken.



2435
2190 ---- 1130 } 8360
7230
96 ---- 2314
1890 ---- 2400
2435 ---- 2435
2435
4870

4870   4625   8456   4204   4835   4870

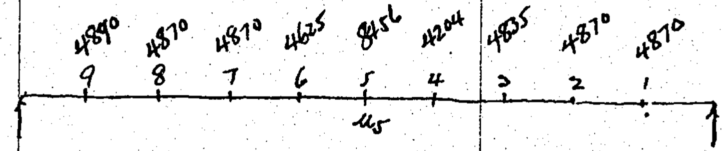
motor truck $2410 \times \frac{.15}{3.75} = 96$
$2410 - 96 = 2314$

u_1 $3320 \times \frac{1.275}{3.75} = 1130$ kg
$3320 - 1130 = 2190$ kg.

u_2 $4290 \times \frac{1.65}{3.75} = 1890$ "
$4290 - 1890 = 2400$ "

Chord stresses

L_4-L_5 moment at U_5

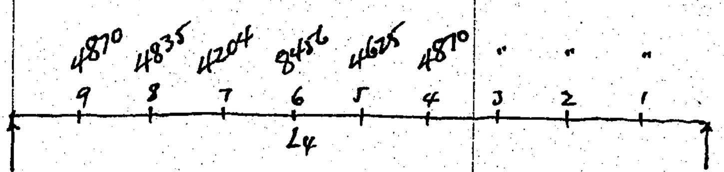


Reaction
$4835 \times 0.3 = 1450$
$4204 \times 0.4 = 1684$
$8456 \times 0.5 = 4228$
$4625 \times 0.6 = 2775$
$4870 \times 0.7 = 3410$
$2 \times 4870 = 9740$
23287 kg.

moment
$23287 \times 18.75 = 436000$
$4625 \times 1 = 4625$
$4870 \times 9 = 43830$
$48455 \times 3.75 = -181200$
254800

stress L_4-L_5 = $254800 \div 4 = 63700$ kg.

U_3-U_5 moment at L_4

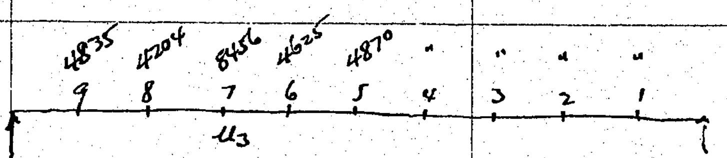


Reaction
$4870 \times 1/10 = 4870$
$4625 \times .5 = 2312$
$8456 \times 0.6 = 5075$
$4204 \times 0.7 = 2945$
$4835 \times 0.8 = 3860$
$4870 \times 0.9 = 4380$
23442 kg.

Moment
$23442 \times 15.00 = 352000$
$4204 \times 1 = 4204$
$4835 \times 2 = 9670$
$4870 \times 3 = 14610$
$28484 \times 3.75 = -107.000$
245.000

stress U_3-U_5 $245000 \div 4 = 61100$ kg.

L_2-L_4 moment at U_3



Reaction
$4870 \times 15/10 = 7300$
$4625 \times 0.6 = 2780$
$8456 \times 0.7 = 5925$
$4204 \times 0.8 = 3360$
$4835 \times 0.9 = 4350$
23715 kg.

moment
$23715 \times 11.25 = 267000$
$4204 \times 1 = 4204$
$4835 \times 2 = 9670$
$13874 \times 3.75 = -52000$
215000

stress L_2-L_4 $215000 \div 4 = 53800$ kg.

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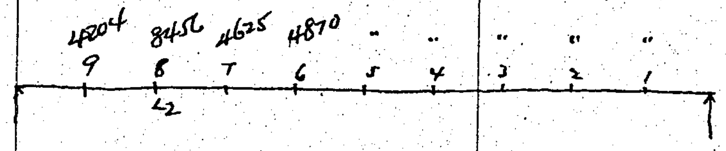
CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken

U_1-U_3 moment at L_2



Reaction
4870 × 21/10 = 10210
4625 × 0.7 = 3240
8456 × 0.8 = 6770
4204 × 0.9 = 3780
24000

moment
24000 × 7.50 = 180.000
4204 × 375 = −15750
164250

Stress U_1-U_3 164250 ÷ 4 = 41000 kg.

L_0-L_2 moment at U_1

Reaction 4870 × 28/10 = 13640
4625 × 0.8 = 3700
8456 × 0.9 = 7600
24940 × 3.75 = 93300 kgm
Stress L_0-L_2 93300 ÷ 4 = 23400 kg

Diagonal U_1-L_2
motor truck rear wheel 3000
impact $\frac{20}{60+30}$ = 22.2 % 666
3666 kg.
2@3666 = 7332.

motor truck front wheel
3666 ÷ 3 = 1222 kg.
2@1222 = 2444

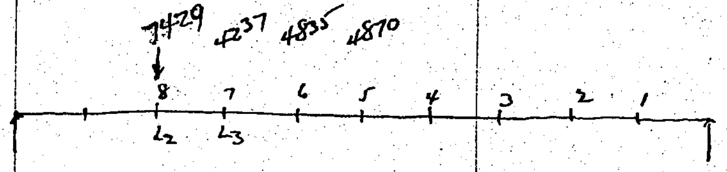
Concentration due to motor truck
Front wheel 2444 × $\frac{.15}{3.75}$ = 97   2444 − 97 = 2347
7332
7429

at L_2 motor truck 7429
~~Unif. load~~ ~~1130~~
~~8559~~

at L_3 motor truck 2347
Unif. load 1890
4237

at L_4 Uniform load 4835 kg
at L_5 -- " 4870 "

End reaction Rear wheel at L_2



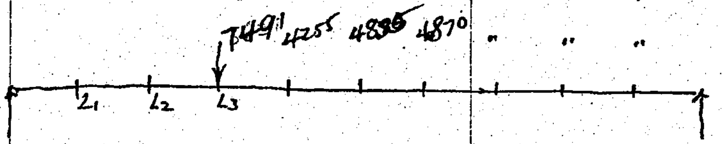
4870 × 15/10 = ~~10210~~ 7300
4835 × 0.6 = 2900
4237 × 0.7 = 2960
7429 × 0.8 = 5950
19110 kg.

Stress = 19110 × 1.37 = 26200 kg

Diagonal L_2-U_3 Rear wheel at L_3.
motor truck rear wheel 3000
impact $\frac{20}{60+26.25}$ = 23.2 % 696
3696 × 2 = 7392

2464 × $\frac{.15}{3.75}$ = 99
2464 − 99 = 2365
Unif. 1890
4255

Front wheel 3696 ÷ 3 = 1232 × 2 = 2464
motor truck { 7392
99
7491



End reaction.
4870 × 10/10 = 4870
4835 × 0.5 = 2417
4255 × 0.6 = 2560
7491 × 0.7 = 5240
15087

Stress 15087 × 1.37 = 20650 kg.

Diagonal U_3-L_4
motor truck rear wheel 3000
impact $\frac{20}{60+22.5}$ = 24.2 % 726
3726 × 2 = 7452
Front wheel 3726 ÷ 3 = 1242 × 2 = 2484

2484 × $\frac{.15}{3.75}$ = 100
7452
7552

2484 − 100 = 2384
Unif. 1890
4274

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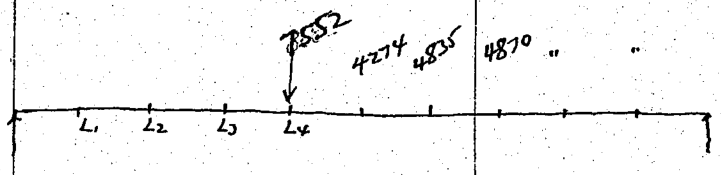


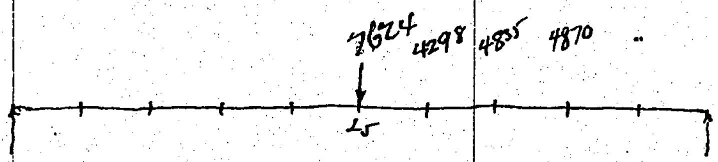
End reaction
4870 × 6/10 = 2920
4835 × 0.4 = 1930
4274 × 0.5 = 2140
7552 × 0.6 = 4540
11530

Stress 11530 × 1.37 = 15800 kg.

Diagonal L4-U5
motor truck rear wheel 3000
impact $\frac{20}{60+18.75}$ = 25.4% 762
3762 × 2 = 7524
Front wheel 3762 ÷ 3 = 1254 × 2 = 2508

2508 × $\frac{.15}{3.75}$ = 100
7524
7624

2508 − 100 = 2408
Unif. 1890
4298



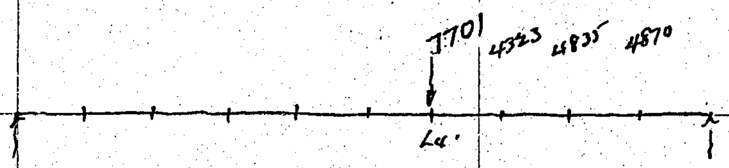
End reaction
4870 × 3/10 = 1460
4835 × 0.3 = 1450
4298 × 0.4 = 1720
7624 × 0.5 = 3812
8442

Stress 8442 × 1.37 = 11600 kg.

Diagonal U5-L4'
motor truck rear wheel 3000
impact $\frac{20}{60+15.0}$ = 26.7% 800
3800 × 2 = 7600
Front wheel 3800 ÷ 3 = 1267 × 2 = 2534

2534 × $\frac{.15}{3.75}$ = 101.
7600
7701

2534 − 101 = 2433
Unif. 1890
4323

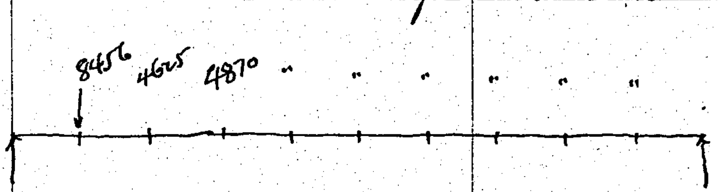


End reaction
4870 × 0.1 = 487
4835 × 0.2 = 967
4323 × 0.3 = 1300
7701 × 0.4 = 3080
5834 kg.

Stress 5834 × 1.37 = 8000 kg.

Stress in Hanger Live load floor beam reaction = 9076 kg.

End Post L0-U1 Full live load motor truck rear wheel at L1



Reaction
4870 × 28/10 = 13640
4625 × 0.8 = 3700
8456 × 0.9 = 7600
24940 kg.

Stress = 24940 × 1.37 = 34200 kg. C

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CALCULATIONS FOR

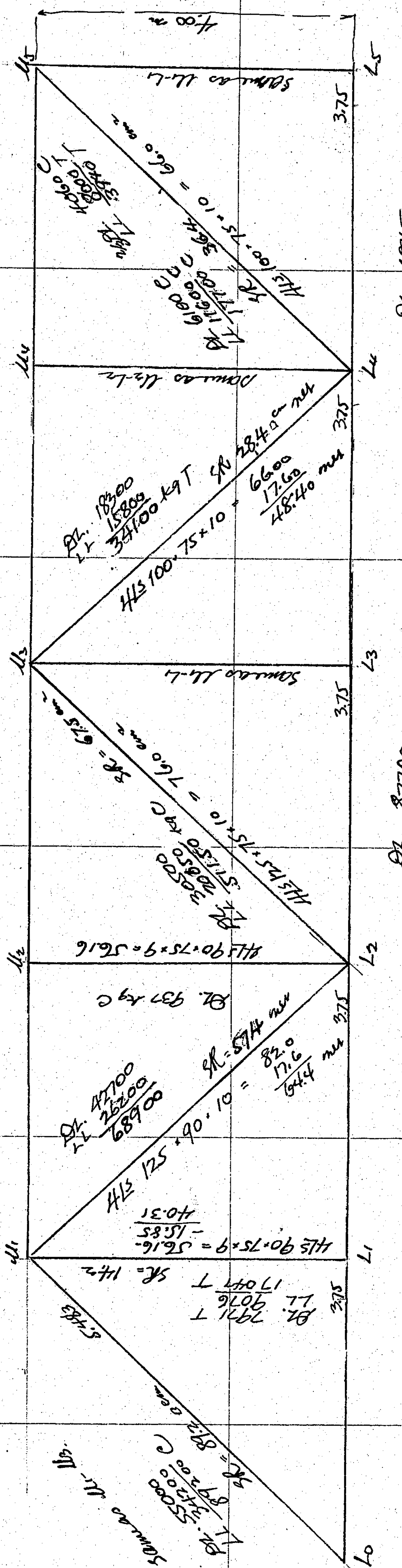
Ashida-Bashi for Okayama-Ken

1Pl. 550·9 = 49.50
2∟ 250·90·9 = 88.15
2 Pls. 190·9 = 34.20
171.85 □cm

DL. 100,000
LL 61,100
161,100 kg C
SR = 161.1 □cm

1Pl. 550·9 = 49.50
2∟ 250·90·9 = 88.15
137.65 cm²

DL. 66800
LL 41000
107800 C
SR = 107.8 □cm



DL. 104500
LL 63700
168200 kg T
SR = 140.0 □cm net

2∟ 250·90·9 = 88.15 − 19.36 = 68.79
2Pls 250·12 = 60.00 − 10.58 = 49.42
2Pls 190·9 = 34.20 − 7.2 = 27.00
182.35 145.21 cm²

68.79
49.42
118.21 cm²

DL. 87700
LL 53800
141500 kg T
SR = 118.0 □cm net

2∟ 250·90·9 = 88.15 − 19.36 = 68.79
2Pls. 250·12 = 60.00 − 10.58 = 49.42
148.15 cm²

2∟ 250·90·9 = 88.15
19.36
68.79 □cm

DL. 37600
LL 23400
61000 kg T
SR = 50.8 □cm net

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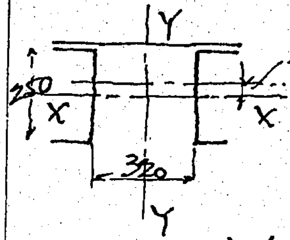
CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken.

Chord section $L_0 U_1$ and U_1-U_3.



1 Pl 550 × 9 = 49.50 × 12.95 = 642
2 [s 250 × 90 × 9 = 88.15
137.65

Ecc = 642 / 137.65 = 4.66 cm ~~9.98~~

1 Pl 550 × 9 = 49.50 × 8.29² = 3400

moment of inertia XX axis 2 [s 250 × 90 × 9 = 88.15 × 4.66² + 8362 = 10277
137.65 13677

$r = \sqrt{\frac{13677}{137.65}} = 9.98$

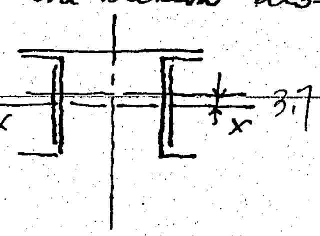
m of inertia YY axis 1 Pl 550 × 9 = 49.50 = 12500
2 [s 250 × 90 × 9 = 88.15 × 18.42² + 612 = 30512
137.65 43012

$r = \sqrt{\frac{43012}{137.65}} = 17.7$

Allowable unit stress U_1-U_3 $P = 1500\left(1 - 0.0055 \frac{375}{9.98}\right) = 1190$ kg/cm² Use 1000 kg/cm²

L_0-U_1 $P = 1500\left(1 - 0.0055 \times \frac{548}{9.98}\right) = 1050$ Use 1000 kg/cm²

Chord section U_3-U_5



1 Pl. 550 × 9 = 49.50 × 12.95 = 642
2 [s 250 × 90 × 9 = 88.15
2 Pls. 200 × 9 = 36.00
173.65

Ecc = 642 / 173.65 = 3.7 cm

1 Pl. 550 × 9 = 49.50 × 9.25² = 4240

axis XX 2 [s 250 × 90 × 9 = 88.15 × 3.70² + 8362 = 9572
2 Pls. 200 × 9 = 36.00 × 3.70² + 1200 = 1693
173.65 15505

$r = \sqrt{\frac{15505}{173.65}} = 9.45$

allowable unit stress $P = 1500\left(1 - 0.0055 \times \frac{375}{9.45}\right) = 1170$ kg/cm² Use 1000 kg/cm²

Diagonal L_2-U_3. Stress 51350 kg C

area m of I

4 L$s 125 × 75 × 10 = 76.0 × 4.62² + 1189 = 2811

$r = \sqrt{\frac{2811}{76}} = 6.08$

allowable unit stress $P = 1500\left(1 - 0.0055 \times \frac{548}{6.08}\right) = 757$ kg/cm²

Section required 51350 ÷ 757 = 67.5 □cm

Diagonal L_4-U_5

17700 kg C

½ × 3940 = 1970 kg T

19670 kg C design stress

m of I

Try 4 L$s 100 × 75 × 10 = 66.0 × 3.57² + 636 = 1476

$r = \sqrt{\frac{1476}{66}} = 4.72$

Allowable unit stress $P = 1500\left(1 - 0.0055 \times \frac{548}{4.72}\right) = 540$ kg/cm²

Section required 19670 ÷ 540 = 36.4 □cm

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CALCULATIONS FOR

Ashida-Bashi for Okayama-Ken.

Approximate weight of truss

member				
L_0-U_1	137.65	@ .785 × 5.48	=	592
U_1-U_3	137.65	7.50	=	810
U_3-U_5	171.85	7.50	=	1010
L_0-L_2	88.15	7.50	=	520
L_2-L_4	148.15	7.50	=	871
L_4-L_5	182.35	3.75	=	536
U_1-L_2	82.00	5.48	=	352
L_2-U_3	76.00	"	=	327
diag.	2 - 66.00	"	=	567
vert.	4.5 - 56×6	4.00	=	793

6378 × 2 = 12756
Details say 38.0% 4856
17612 say 17600

17600 ÷ 37.5 = 470 kg per lin. meter.

Load on shoe

Dead Load Intermediate panels 4.5 × 8908 = 40100
End panel 5080
45180 say 46000 kg.

Live Load motor truck rear wheel at end panel point.
Uniform live load 1300 kg per lin. meter (see page 10).
4625 × 0.9 = 4160
4870 × 3.6 = 17500
8360
30020 Call this 30.000

Summary
Dead Load 46000
Live Load 30.000
76000 kg.

Roller 9.0 dia 45×9 = 405 kg per cm of roller
76000 ÷ 405 = 188 cm 4 Rollers @ 47. cm = 188 net

Approximate weight of structural steel in one span

~~stringers~~	4 @ 50 × 38.0	=	7600
floor beam	9 × 925	=	8320
" "	2 × 870	=	1740
Lateral Bracings			2042
Trusses	2 @ 17600	=	35200
shoes say	4 @ 700	=	2800
			57702 kg.

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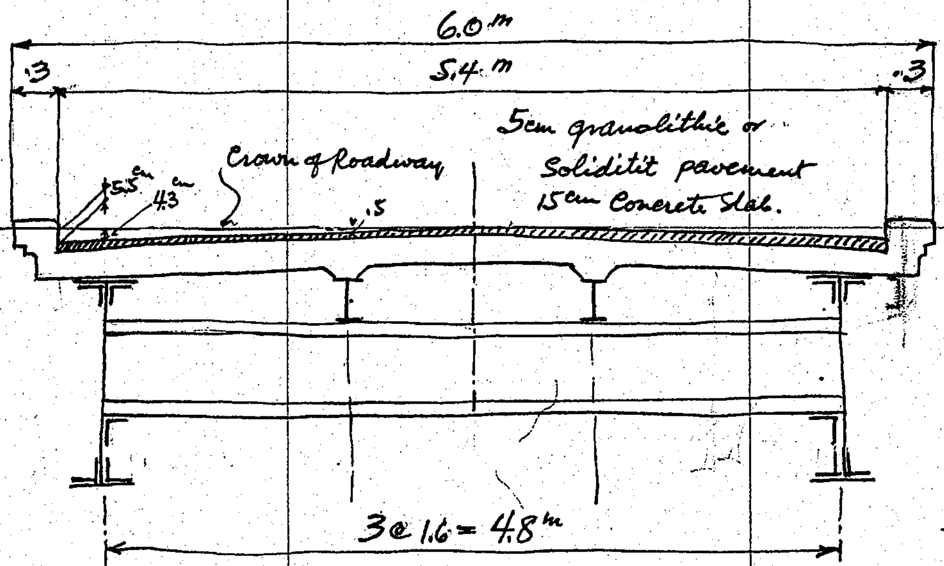
CALCULATIONS FOR

Ashida-Bashi for Okayama ken

Design of Girder span. Span length 5 panels @ 3.73m = 18.65m.
Cross section of bridge assumed as shown on sketch.



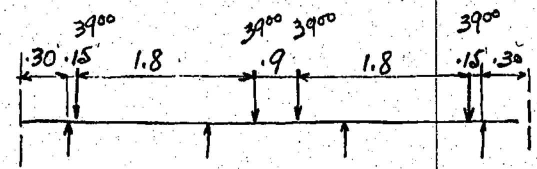
Design of Floor slab.
Use same details for floor slab and handrail as for truss span. See page 2–4

Design of I Beam Stringer. span length 3.73 meters spacing 1.6 meters
Dead Load — floor slab and pavement $500 \times 1.6 = 800$
beam assumed 50
850 kg/m.

Dead load moment $= 1/8 \times 850 \times 3.73^2 = 1{,}478$ kgm
" " shear $= 1/2 \times 850 \times 3.73 = 1{,}585$ kg.

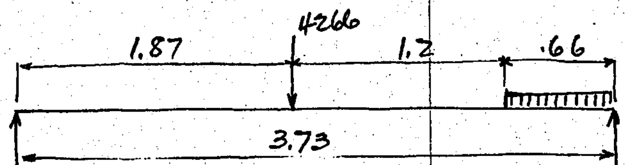
Live Load — motor truck rear wheel concentration with impact = 3900 kg
" front wheel " " " = 1,300

max load on stringer.



$3900 \times \frac{.15}{1.6} = 366$
3900
4266 kg

Uniform live load $500 \times 1.6 = 800$ kg per meter.
Reaction $\frac{800 \times .66^2}{2 \times 3.73} = 47$ kg.



moment
Due to motor truck $\frac{4266}{2} \times 1.87 = 3990$
" unif. live load $47 \times 1.87 = 88$
4078 kgmeter.

Summary for moments and shears.

	moment	Shear
Dead Load	1478	1,585
Live Load.	4078	4953
	5,556 kgm	6,538 kg.

max. end shear
Unif. $\frac{800 \times 2.53^2}{2 \times 3.73} = 687$
motor truck 4266
4953 kg.

Section modulus required $= \frac{5556 \times 100}{1100} = 505$

Use 300 × 150 I @ 48.34 kg Sm. = 633.2
Unit stress $= \frac{5556 \times 100}{633.2} = 877$ kg/cm²

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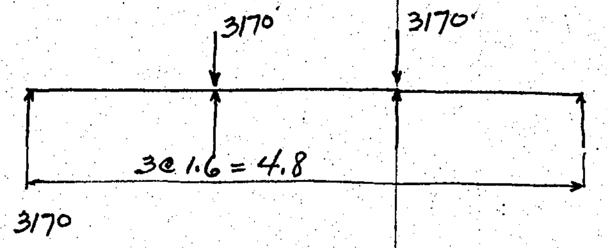
CALCULATIONS FOR

Ashida-Bashi for Okayama ken. (17)

Design of Intermediate floor beam.
Dead Load.

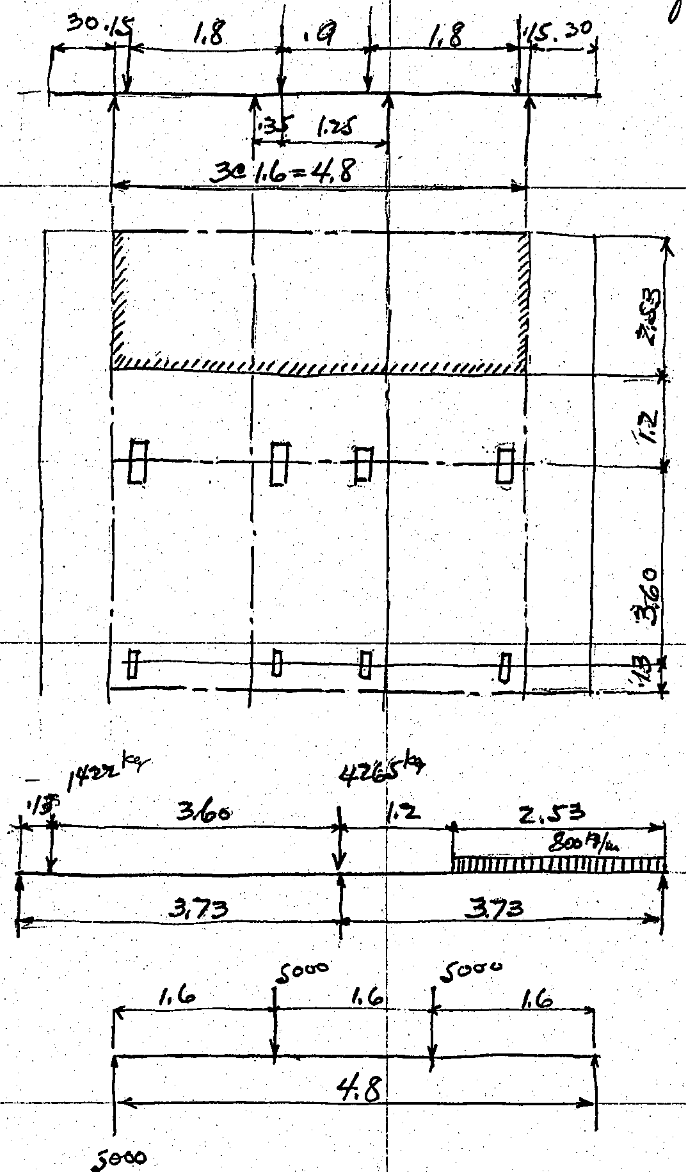


Dead Load concentration on stringer
500 × 1.6 = 800
Stringer 50
850 × 3.73 = 3170 kg.

moment due to stringer concentration
3170 × 1.6 = 5070 kgm.
Beam assumed 140 kg/m.
moment 1/8 × 140 × 4.8² = 403 kgm.
Total D.L. moment = 5473 kgm.

End shear.
Beam 140 × 2.4 = 336
Stringer concentration = 3170
3,506 kg.

Live Load. motor truck loading rear wheel concentration with impact 3900 kg
front 1300



Reaction on stringer due to rear wheel of motor truck.
3900 × .15/1.6 = 365
3900
4265 kg.

Front wheel.
4265 ÷ 3 = 1422 kg
Uniform load 500 kg/m².
on stringer = 500 × 1.6 = 800 kg/m.

Reaction on floor beam motor truck.
due to front wheel. (1422 × .13)/3.73 = 50 kg.
" rear wheel = 4265
4315 kg

Uniform live load.
(800 × 2.53²)/(2 × 3.73) = 686 kg.
total 5001 kg Call this 5000 kg.

moment = 5000 × 1.6 = 8,000 kgm.
end shear = 5,000 kg.

Summary for moment and shears.

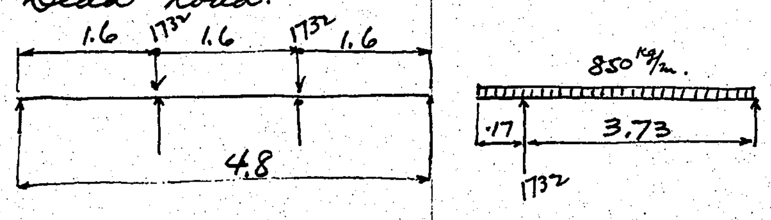
	moments	Shears.
Dead Load	5473	3,506
Live Load.	8000	5,000
	13,473 kgm	8,506 kg

web assumed 60 × .8 = ~~53.6~~ 48.0 cm² 1/8 web = 6.0 cm
Back to back of Ls = 61.0 cm effective depth 58.0 cm.
flange stress = (13473 × 100)/58 = 23,240 kg
flange area required = 23240/1200 = 19.4 cm²
6.0
13.4 cm² net

revise to 75×75×9
Use 2Ls 80 × 60 × 8 = 21.12 − 3.52 = 17.6 cm² net. rivet 19φ

allowable stress in comp. flange.
1200(1 − 0.012 × 160/16.8) = 1065 kg/cm²
SR. = 23240/1065 = 21.8 cm²
6
15.8 cm²

Design of End floor beam. span length 4.8 meters.
Dead Load.



Reaction on floor beam
(850 × 3.90²)/(2 × 3.73) = 1732 kg

moment = 1732 × 1.6 = 2770 kgm
beam 1/8 × 140 × 4.8² = 403
D.L. moment 3173 kgm.

end shear = 1732 + 336 = 2,068 kg

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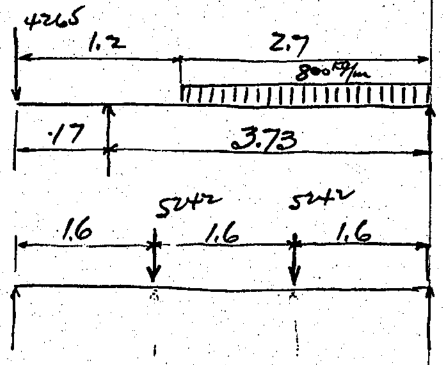
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken. (18)

Live Load. Stringer concentration due to rear wheel of motor truck. = 4265kg page 17.



motor truck loading $\frac{4265 \times 3.9}{3.73} =$ 4460kg

unif. loading $\frac{800 \times 2.7^2}{2 \times 3.73} =$ 782
5,242 kg

moment = 5242 × 1.6 = 8390 kgm
end shear = 5242 kg.

Summary for moments and shears

	moments	shears.
Dead Load.	3173	2068
Live Load.	8390	5242
	11,563 kgm	7,310 kg

web assumed 60 × .8 = 48.0 ocm $\frac{1}{8}$ web = 6.0 ocm
Back to back of Ls = 61 cm effective depth say 58 cm
flange stress = $\frac{11563 \times 100}{58}$ = 19950 kg

flange area required = $\frac{19950}{1200}$ = 16.64 ocm net
6.0
10.64 ocm net

Use 2Ls 80×60×8 (revice to 75×75×9) = 21.12 - 3.52 = 17.6 ocm 19φ rivet.

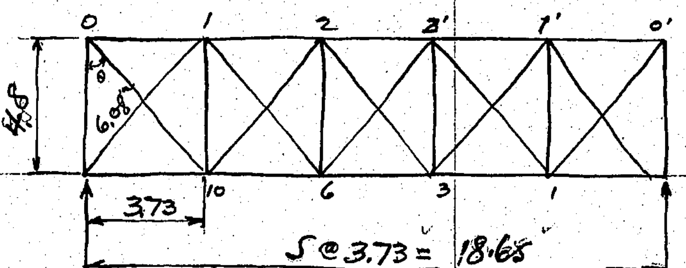
allowable unit stress on compression flange. 1200(1 - 0.012 × $\frac{160}{16.8}$) = 1065 kg/cm²
Compression flange area required = $\frac{19950}{1065}$ = 18.75
6
12.75 cm² gross. ok.

Approximate weight of Floor beam. revice 75×75×9

						revised figure
flange	4Ls	80×60×8	@ 8.28 × 4.80		159.0	191.
web	1 Pl.	600×8	@ 37.68 × 4.80		181.0	
End connection	4Ls	75×75×9	@ 9.96 × .59		23.5	
fills	2 Pls	150 × 8 revice 9	@ 9.42 × .49		9.4	10.4
Stringer conn.	2 Pls	180 × 10	@ 14.13 × .17		4.8	
intermediat stiff.	4Ls	65×65×8	@ 7.66 × .59		18.1	45.2
revised to 10	Rivet heads & variation	3%			14.4	15.-
					410. kg	~~444~~ 472.1 kg

410 ÷ 4.8 = 85.4 kg per lin meter (99)
410 ÷ 3.73 = 110.0 per meter of main girder (129)

Lower Lateral Bracing.



wind load 600 kg per meter of bridge.
or 600 × 3.73 = 2240 kg
diagonal length $\sqrt{4.8^2 + 3.73^2}$ = 6.08 m.
sec θ = $\frac{6.08}{4.8}$ = 1.267
tan θ = $\frac{3.73}{4.8}$ = .777

Diagonal Stresses.

Panel.		Shear	sec θ	Stress	Sh. 1200kg/cm²	19φ rivets	use	
0-1	$\frac{2240}{5}$ × 10	= 4480 ×	1.267 =	5670 kg	4.73	2.7	3	1L 75×75×9 = 12.69 cm²
1-2	" × 6	= 2690 ×	" =	3410	2.88	1.6	3	do
2-3'	" × 3	= 1345 ×	" =	1,704	1.42	0.8	2	do

Chord stresses

panel				
0-1	2240 × 2 × 0.777	=	3480 kg
1-2	" × 3 × "	=	5220
2-2'	" × 3 × "	=	"

Approximately weight of lateral bracing
10Ls 75×75×9 @ 9.96 × 6.0 = 598
Center conn. & flanges Say 5 @ 28.8 = 144
742 kg

742 ÷ 18.65 = 39.8 kg/m of bridge
panel load = 39.8 × 3.73 = 148.5 kg

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CALCULATIONS FOR

Ashida Bashi for Okayamaken.

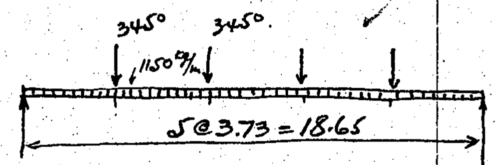
Design of main girder. span length $5 @ 3.73 = 18.65$ meters.

Dead Load. panel concentration

Due to stringer concentrations		3170
Floor beam	$410 \div 2$	205
Lateral bracing	$148.5 \div 2$ say	75
		3450 kg

Uniform load

Handrail coping + overhanging slab	426
floor between main girder + stringer	400
	826 kg/m.
main girder assumed	324
	1150 kg/m.



Dead load moment at center of span.

panel concentration $3450 \times 2 = 6900 \times 2 = 13800$
$3450 \times 1 = 3450$
$10350 \times 3.73 = 38600$ kgm

Uniform load $1/8 \times 1150 \times 18.65^2 = 50000$
88600 kgm

End shear panel concentration $3450 \times 2 = 6900$
uniform load $1150 \times \frac{18.65}{2} = ~~10860~~$ 10730
~~17760~~ 17630 kg

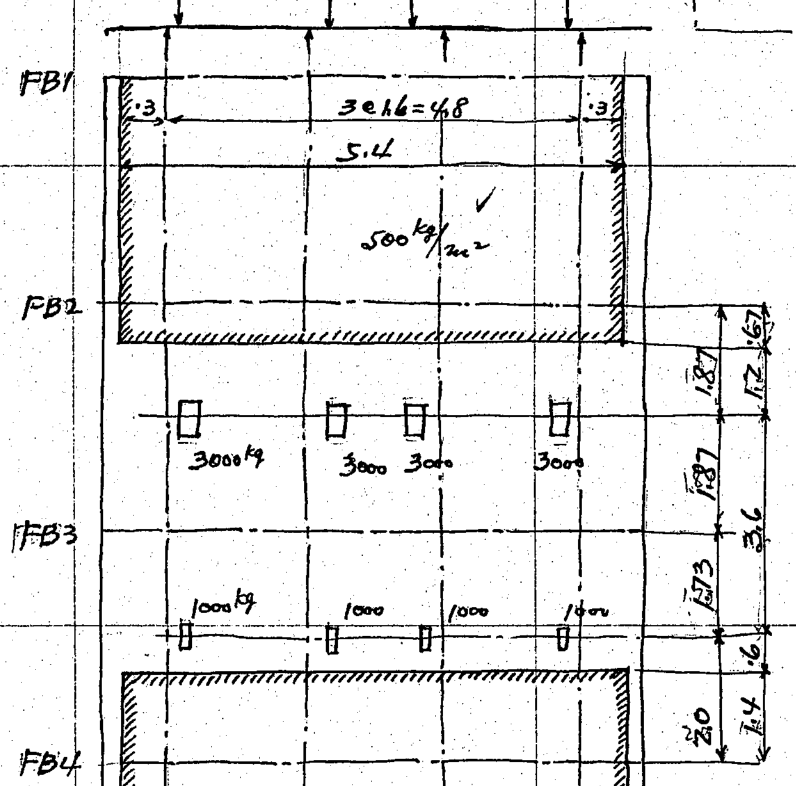
moment at panel pt. 1.
$17630 \times 3.73 = 65800$
$1150 \times \frac{3.73^2}{2} = -8000$
57800 kgm

moment at panel pt. 2.
Unif. load $\frac{1150 \times 7.46}{2}(18.65 - 7.46) = 48000$
panel concentration 38600
86600 kgm

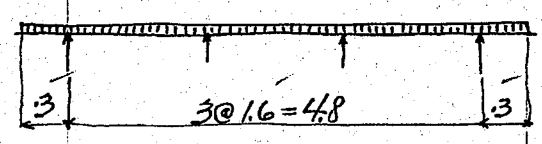
Shear at panel pt. 1.
$17630 - 1150 \times 3.73 = 13350$ kg

Shear at panel pt. No 2.
$3450 + 1150 \times 1.87 = 5600$ kg

Live Load.



Uniform Live Load 500 kg/m²
transverse distribution.



Load on main girder $\frac{500 \times 1.9^2}{2 \times 1.6} = 565$ kg/m.

Load on stringer $500 \times 1.9 = 950$
$500 \times .8 = 400$
$1350 - 565 = 785$ kg/m.

Panel concentration due to stringer for rear wheel at center of panel.

for FB1	$785 \times 3.73 =$		2925 kg
FB2	$785 \times 3.73 \div 2 =$	1465	
	$785 \times .67 \times 3.4 \div 3.73 =$	480	
			1945
FB3	$785 \times .67^2 \div 2 \div 3.73 =$	48	
	$785 \times 2.0^2 \div 2 \div 3.73 =$	422	
			470
FB4	$785 \times 3.73 \div 2 =$	1465	
	$785 \times 2 \times 2.73 \div 3.73 =$	1150	
			2615

Motor truck loading
Impact coeff. $= \frac{20}{60 + 18.65} = 25.4\%$

Rear wheel concentration 3000 kg
impact 25.4% 760
3760 kg

Front wheel $3760 \div 3 = 1250$ kg
concentration on main girder due to rear wheel.
assumed $2 \times 3760 = 7520$ kg
front wheel $2 \times 1250 = 2500$ kg.

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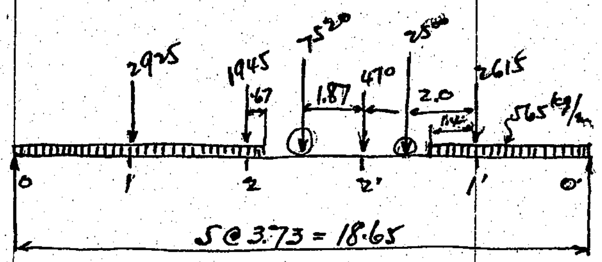
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

moment at center of span.



Panel loads (unif. load)

2615 × 1 = 2615

470 × 2 = 940

1945 × 3 = 5830

2925 × 4 = 11700

Reaction 21085 ÷ 5 = 4220 kg

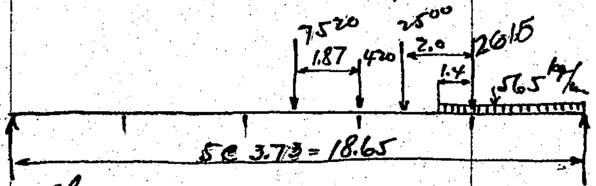
4220 × 2.5 = 10550

1945 × 0.5 = − 970

2925 × 1.5 = − 4390

moment = 5190 × 3.73 = 19350 kgm

Max shear at center of span.



motor truck

2500 × 5.73 ÷ 18.65 = 766

7520 ÷ 2 = 3760

Reaction 4526 kg

moment = 4526 × 18.65/2 = 42200 kgm

shear

panel load 2615 × 1/5 = 520

420 × 2/5 = 170

690 kg

Uniform load.

565 × 8.13 = 4600 kg

4600 × 14.58 = 67100

565 × 5.13²/2 = 7440

Reaction 74540 ÷ 18.65 = 4000 kg

4000 × 9.33 = 37300

4600 × 5.26 = 24200

13100 kgm

motor truck

7520 ÷ 2 = 3760

2500 × 5.73/18.65 = 770

4530 kg

Uniform load

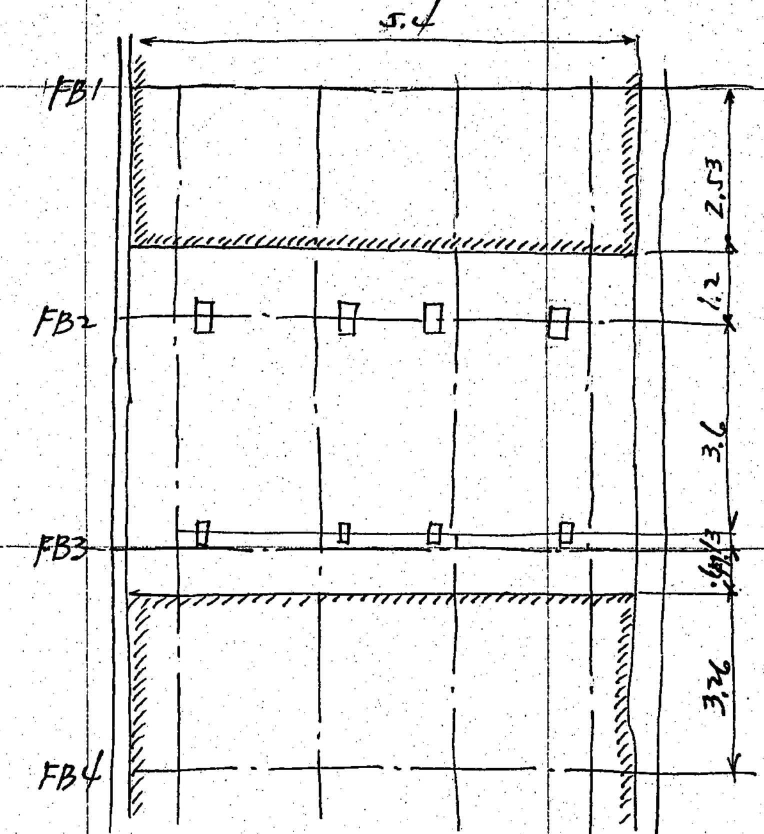
565 × 5.13² / 2 × 18.65 = 400

total live load shear = 5020 kg

Total Live load moment = 74650 kgm

Uniform load directly on main girder 565 kg/m

Panel Concentration due to stringer (Rear wheel on floor beam)



for FB1. 785 × 3.73 ÷ 2 = 1465 kg

785 × 2.53 × 2.46/3.73 = 1310

2775 kg

for FB2 785 × 2.53²/2 ÷ 3.73 = 675 kg

for FB3 785 × 3.26²/2 ÷ 3.73 = 1120 kg

for FB4 785 × 3.26 × 2.1/3.73 = 1440

785 × 3.73 ÷ 2 = 1465

2905 kg

Motor truck wheels directly on main girder assumed

Rear wheel 2@3760 = 7520 kg

front wheel 7520 ÷ 3 = 2500 kg

moments due to.

Panel loads 2775 × 4 = 11100

675 × 3 = 2025

1120 × 2 = 2240

2905 × 1 = 2905

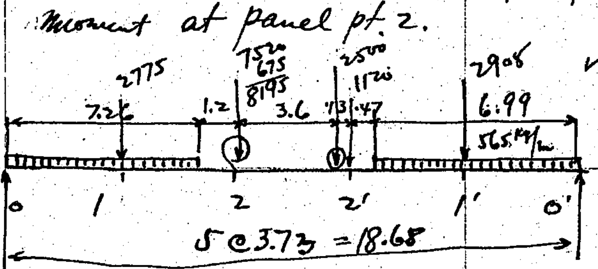
Reaction 18270 ÷ 5 = 3655 kg

Moment 3655 × 7.46 = 27300

2775 × 3.73 = 10350

16950 kgm

moment at panel pt. 2.



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Moment due to motor truck wheels.

Rear wheel $7520 \times 3/5 = 4510$

$\frac{2500 \times 7.59}{18.65} = 1020$

Reaction $= 5530$ kg

moment at panel pt. 2. $5530 \times 7.46 = 41,200$ kgm

Moment due to Uniform load.

$565 \times 7.26 \times 15.02 = 61600$

$565 \times \frac{6.99^2}{2} = 13800$

Reaction $= 75400 \div 18.65 = 4040$ kg

moment at panel pt. 2 $= 4040 \times 7.46 = 30,100$

$565 \times 7.26 \times 3.83 = 15700$

14400 kgm

Summary for moment

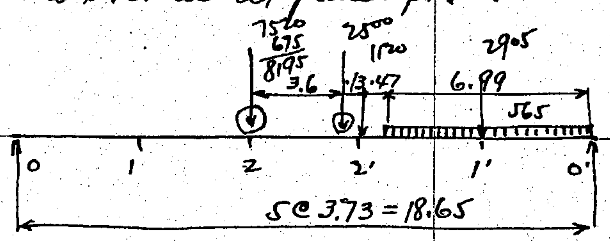
due to panel concentration $= 16,950$ kgm

" motor truck $= 41,200$

" uniform load $= 14400$

72550 kgm

Max. Shear at panel pt. 2.



Reaction panel concentration $2905 \times 1/5 = 580$

$1120 \times 2/5 = 450$

$675 \times 3/5 = 405$

1435 kg

motor truck rear wheel $7520 \times 3/5 = 4510$

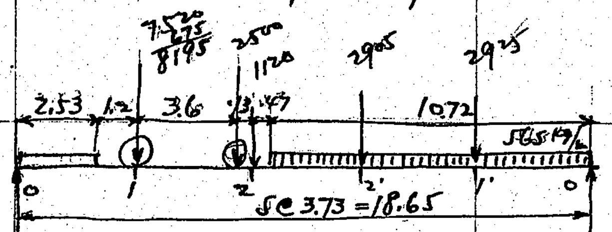
" front " $\frac{2500 \times 7.59}{18.65} = 1015$

5525 kg

Uniform load $\frac{565 \times 6.99^2}{2 \times 18.65} = 740$

7700 kg

moment at panel pt. 1



moment due to panel concentration

$675 \times 4/5 = 540$

$1120 \times 3/5 = 673$

$2905 \times 2/5 = 1162$

$2925 \times 1/5 = 585$

Reaction $= 2960$ kg

moment $= 2960 \times 3.73 = 11,050$ kgm

moment due to motor truck

rear wheel $7520 \times 4/5 = 6020$

front " $2500 \times \frac{11.32}{18.65} = 1520$

reaction $= 7540$ kg

moment $= 7540 \times 3.73 = 28,200$ kgm

moment due to uniform load

$565 \times 2.53 \times 17.38 = 24850$

$565 \times \frac{10.72^2}{2} = 32450$

57300

Reaction $\frac{57300}{18.65} = 3075$ kg

moment $3075 \times 3.73 = 11470$

$565 \times 2.53 \times 2.47 = 3530$

7940 kgm

Total moment at panel pt 1 $= 47,190$ kgm

Max. shear at panel pt No 1.

Shear due to panel concentration $= 2960$ kg

" " wheel loads $= 7540$

" " uniform load

$32450 \div 18.65 = 1740$

$12,240$ kg

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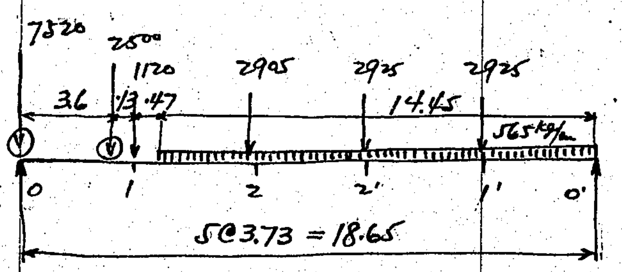
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

max. End shear

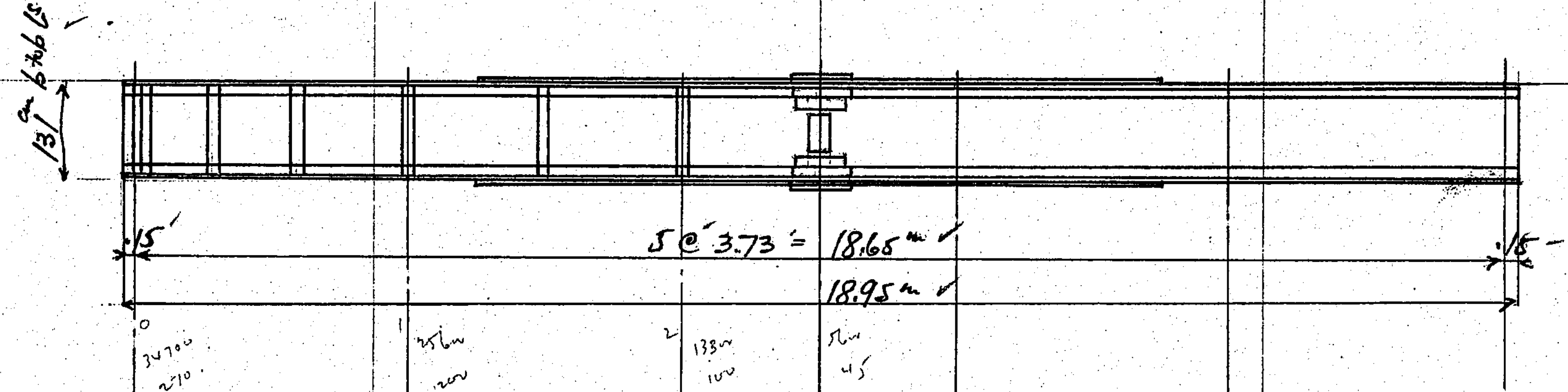


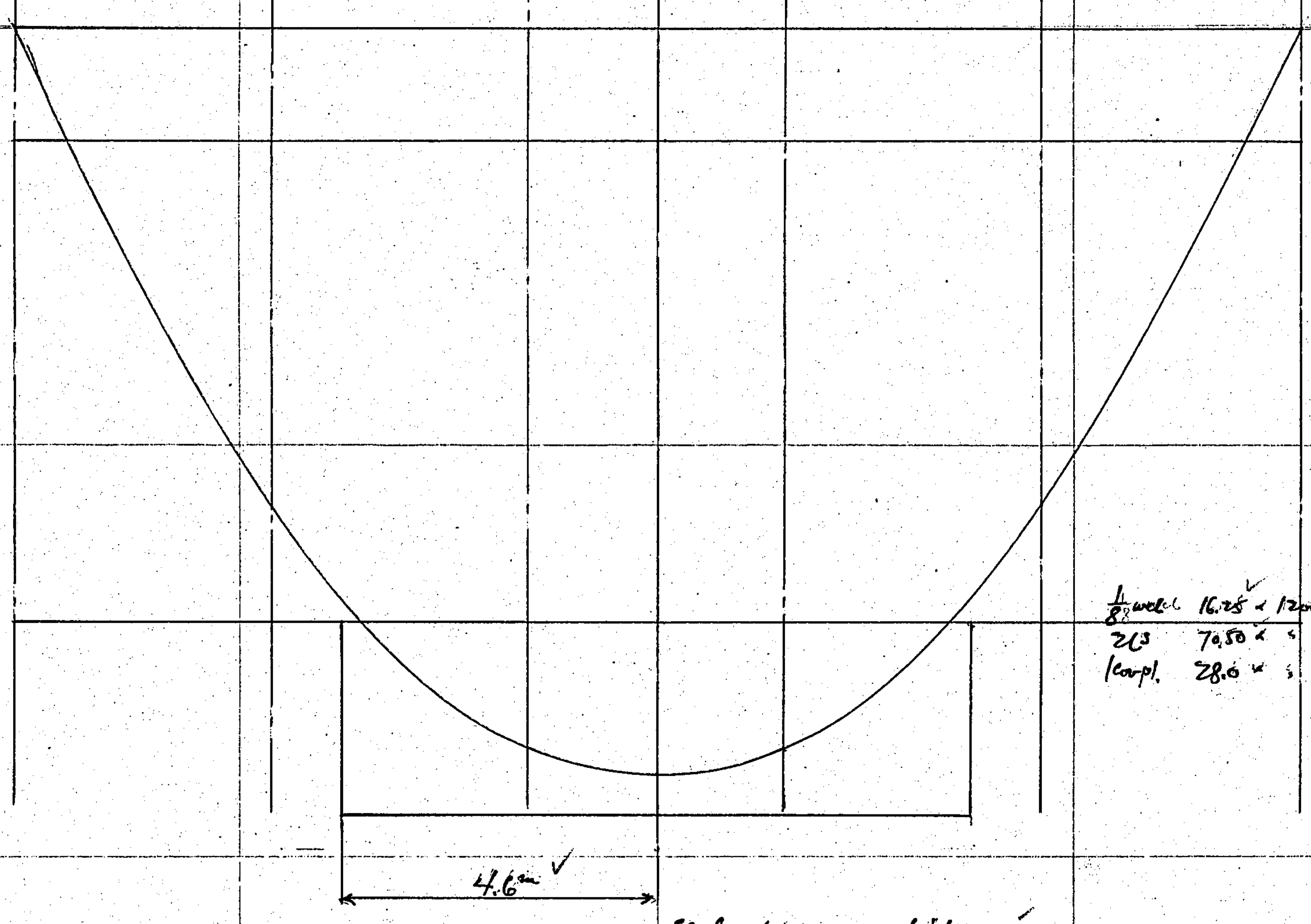
Reaction

Panel Concentration	1120 × 4/5 =	896
	2905 × 3/5 =	1745
	2925 × 3/5 =	1755
		4396 kg
Motor truck rear wheel		9520
front ×	2500 × 15.05 / 18.65 = 2020	
		9540
Uniform load	565 × 14.45² / (2 × 18.65) =	3164
		17,100 kg

Summary for moments and shears.

	Center of span moment	Center of span shear	panel pt No 2. moment	panel pt No 2. shear	panel pt. No 1. moment	panel pt. No 1. shear	End support. End shear
Dead Load	88,600	0	86,600	5600	57,800	13350	17,630
Live Load	74650	5620	72550	7700	47190	12240	17,100
	163,250 kgm	5620 kg	159,150 kgm	13300 kg	104,990 kgm	25590 kg	34,730 kg





1/8 w.d.l.	16.25 × 1200 × 1.25 =	24,400 kgm	
2L3	70.50 × " × 1.25 =	105,800	
Cov pl.	28.0 × " × 1.25 =	42,000	
		172,200 kgm	

Scale of space 1 : 100
Scale of moment 1/5 meter = 10,000 kgm.

Ashida-Bashi for Okayama ken

Section of main girder
at Center of span

Moment 163250 kg.m. Shear 5620 kg.

$130^{cm} \times 10^{cm}$ Web pl. assumed, gross area = $130^{\square cm}$, 1/8 web area = $16.25^{\square cm}$

depth back to back of Flg. Ls = 131^{cm}. effective depth say 131-6 = 125^{cm}.

Flg. stress = $\frac{163250 \times 100}{125}$ = 131000 kg.

Flange area required = $\frac{131000}{1200}$ = $109.2^{\square cm}$ net.

16.25
$92.95^{\square cm}$ net

Use 2Ls 150×150×15 = 85.50 - 15.0 = 70.5
1Cov.Pl. 330×10 = 33.0 - 5.0 = 28.0
$118.5^{\square cm}$ gross. $98.5^{\square cm}$ net

Unit Compression in Flg. = $\frac{131000}{118.5+16.25}$ = 972.0 kg/cm^2 ok.

End Shear = $\frac{34730}{130 \times 1}$ = 267 kg/cm^2

Approximate weight of main girder

Web	1Pl.	1300×10	@	102.05 kg	× 18.95 m	=	1935 kg
Flg.	4Ls.	150×150×15	@	33.55	× 18.95	=	2543
Cov.Pl.	2Pls.	330×10	@	25.91	× 9.20	=	487
Splice	2Pls.	300×11	@	23.55	× .66	=	31
"	4Ls.	150×150×19	@	41.91	× .86	=	144
"	2Pls.	330×10	@	25.91	× .86	=	45
" web	4Pls.	170×15	@	20.02	× .72	=	58
Stiff.	8Ls.	125×90×10	@	16.09	× 1.28	=	165
Fill.	4Pls.	300×15	@	35.33	× 1.005	=	142
Stiff.	28Ls.	125×90×10	@	16.09	× 1.31	=	590
Shelf.Ls	10Ls.	100×75×10	@	12.95	.17	=	22
guss.Pls.	4Pls.	300×9	@	19.78	.535	=	42
"	2Pls.	235×9	@	16.60	.30	=	10
Shoes	2Pls.	300×19	@	50.88	.53	=	108
"	2Pls.	320×35	@	87.92	.53	=	93
"	4Ls.	75×75×9	@	9.96	.21	=	8
Rivet head etc.		35%					215

Detail 28.4% including rivet head

6638 ÷ 18.65 = 356 kg/m. of span

Approximate steel for one span

Stringers	48.34 × 18.95 × 2		= 1833
Floor Beams	6 @	444	= 2664
Lateral Bracing			= 742
Main girders + shoe	2 @	6638	= 13276
			18515 kg. ton per span

Load on shoe.

Dead Load end shear 17630 — 17630

Live load, motor truck & Unif. load 17100, unif. load only $\frac{500 \times 2.7 \times 18.95}{2}$ = 12800

34730 — 30430

Considering effect of over hanging take this at 36000 kg for one shoe

Ashida-Bashi for Okayama Ken.

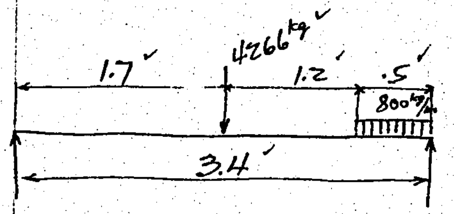
Design of Approach Girder span, Span length 4 panels @ 3.4 = 13.6 meters.
Cross section of the bridge same as for 18.65 m girder spans previously designed. see page 16
Design of Floor Slab.
Use same details floor slab and handrail as for truss span. see on page 2-4.

Design of I-beam stringer span length 3.4 meters, spacing 1.6 meters.

Dead Load
floor slab and pavement 500 × 1.6 = 800
beam assumed 50
850 kg/m

Dead Load moment = 1/8 × 850 × 3.4² = 1,228 kgm.
Dead Load end shear = 1/2 × 850 × 3.4 = 1445 kg.

Live Load
motor truck rear wheel concentration with impact = 3900 kg
" front wheel " " = 1,300
max load on stringer due to motor truck rear wheel = 4,266 kg see on page 16
" uniform load = 800 kg/m.



Reaction due to
Uniform load = $\frac{800 \times .5^2}{2 \times 3.4}$ = 29
motor truck rear wheel = $\frac{4266}{2}$ = 2133
2162 kg

Live Load moment = 2162 × 1.7 = 3,680 kgm

Summary for moments and shears.

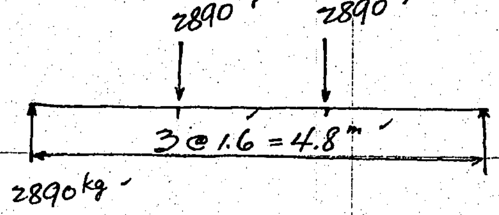
	moment	shear
Dead Load	1,228	1,445
Live Load	3680	4836
	4,908 kgm.	6,281 kg.

End shear
Unif. load $\frac{800 \times 2.2^2}{2 \times 3.4}$ = 570
motor truck 4266
4836 kg

Section modulus required = $\frac{4908 \times 100}{1100}$ = 447 cm³

Design of Intermediate floor beam. Span length 4.8 meters

Dead Load.
Dead Load concentration on stringer.
Slab and pavement. 500 × 1.6 = 800
Stringer 50
850 × 3.4 = 2890 kg



moment due to stringer concentrations.
2890 × 1.6 = 4,625 kgm.
Beam assumed 100 kg/m.
moment 1/8 × 100 × 4.8² = 288
Total D.L moment = 4,913 kgm.
End shear
Beam 100 × 2.4 = 240
Stringer concentration 2890
3,130 kg

Live Load
motor truck rear wheel concentration with impact = 3900 kg
" front wheel " " = 1300

Reaction on stringer due to rear wheel of motor truck see next figure.
3900 × $\frac{.15}{1.6}$ = 365
3900
4265 kg
do. due to front wheel.
4265 ÷ 3 = 1,422 kg

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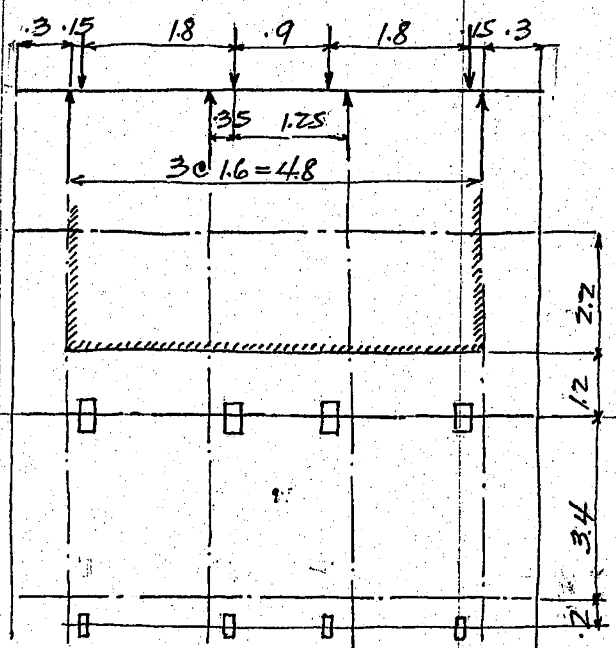
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

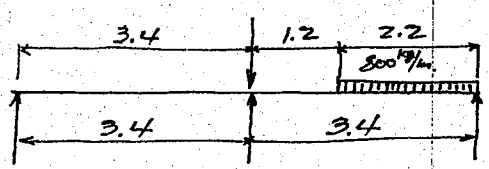


Uniform load $500^{kg/m^2}$
" on stringer $= 500 \times 1.6 = 800^{kg/m}$
Reaction on floor beam due to motor truck 4265^{kg}
unif. load. $\frac{800 \times 2.2^2}{2 \times 3.4} = \frac{570}{4835^{kg}}$

moment $= 4835 \times 1.6 = 7750^{kgm}$
end shear $= 4835^{kg}$

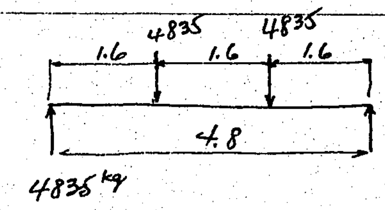
Summary for moments and shears.

	moments	shears.
Dead Load	4913	3130
Live Load	7750	4835
	$12,663^{kgm}$	$7,965^{kg}$



web assumed $60^{cm} \times .8^{cm} = 48.0^{cm^2}$ $\frac{1}{8}$ web $= 6.0^{cm^2}$
depth back to back of L's. $= 61.^{cm}$ effective depth say 58.0^{cm}
flange stress $= \frac{12663 \times 100}{58} = 22000$ kg

flange area required $= \frac{22000}{1200} = 18.3^{\square cm}$
$\underline{-6.0}$
$12.3^{\square cm}$ net.



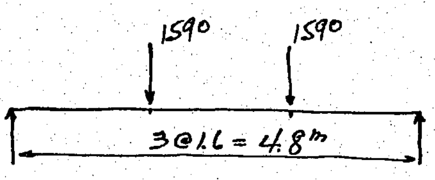
Use 2L's $75 \times 65 \times 8$ (revise $75 \times 75 \times 9$) $= 21.12 - 3.52 = 17.6^{\square cm}$ net 19^{ϕ} rivet

Allowable stress on compression flange
$1200 \times (1 - 0.012 \times \frac{160}{15.8}) = 1053^{kg/cm^2}$

~~tension flange area req'd. $= \frac{22000}{1200} =$~~

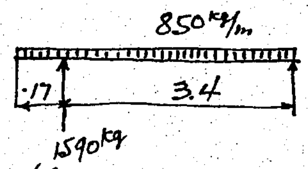
S.R. $= \frac{22000}{1053} = 20.9$
$\underline{-6.0}$
$14.9^{\square cm}$ gr ok

Design of End floor beams. Span length 4.8 meters.
Dead Load.

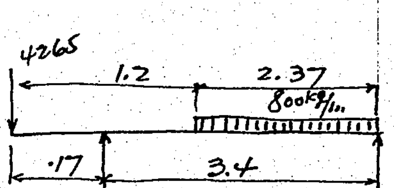


Reaction on floor beam.
$\frac{850 \times 3.57^2}{2 \times 3.4} = 1,590^{kg}$

moment $= 1590 \times 1.6 = 2,545^{kgm}$
beam $= \frac{1}{8} \times 100 \times 4.8^2 = \underline{288}$
Total D.L. moment $= 2,833$ kgm.
End shear $= 1590 + 240 = 1,830^{kg}$



Live Load. Stringer concentration due to rear wheel of motor truck $= 4265^{kg}$ see page 17.



motor truck loading $\frac{4265 \times 3.57}{3.4} = 4480^{kg}$

Uniform loading $\frac{800 \times 2.37^2}{2 \times 3.4} = \frac{660}{5,140^{kg}}$

moment $= 5140 \times 1.6 = 8,230^{kgm}$.
end shear $= 5,140^{kg}$.

Summary for moments + Shears

	moments	shears.
Dead Load	2833	1830
Live Load	8230	5140
	$11,063^{kgm}$	6970^{kg}.

Web assumed $60 \times .8 = 48.0^{\square cm}$ $\frac{1}{8}$ web $= 6.0^{\square cm}$
depth back to back of flg. L's $= 61^{cm}$ effective depth say 58^{cm}
flange stress $= \frac{11063 \times 100}{58} = 19200^{kg}$

flange area required $= \frac{19200}{1200} = 16.0^{\square cm}$
$\underline{-6.0}$
$10.0^{\square cm}$ net.

Use 2L's $75 \times 65 \times 8 = 21.12 - 3.52 = 17.6^{\square cm}$ net 19^{ϕ} rivet
Allowable unit stress on comp. flange 1065
Compression flange area req'd $= \frac{19200}{1065} = 17.95^{\square cm}$ gr
$\underline{-6}$
11.95 gr ok

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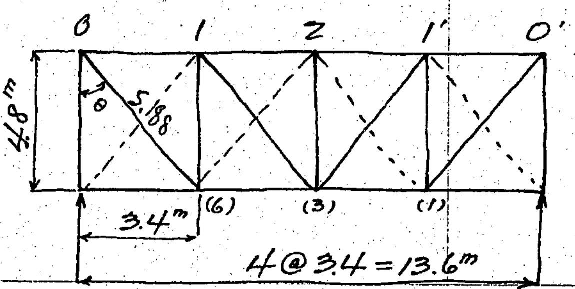
Approximate weight of floor beam.

					revise	
Flange	4Ls 75×65×8	@	8.28 × 4.8 =	159	4Ls 75×75×9	191
Web	1Pl 600×8	@	37.68 × 4.8 =	181		
End connection	4Ls 75×75×9	@	9.96 × .59 =	23.5	2Ls 75×75×9	11.8
fills	2Pls 150×8	@	9.42 × .49 =	9.4		
Stringer conn.	2Pls 180×10	@	14.13 × .17 =	4.8		
intermediate stiff	10Ls 65×65×8	@	7.66 × .59 =	45.2	8Ls 65×65×8	36
rivets heads + variation				12.7		10
				435.6 kg		444

435.6 ÷ 4.8 = 91 kg per meter
435.6 ÷ 3.4 = 128 kg per meter of main girder.

Lateral Bracing



wind load 600 kg per lin meter of bridge
panel load = 600 × 3.4 = 2040 kg
diagonal length = $\sqrt{4.8^2+3.4^2}$ = 5.88 m
$\sec\theta = \frac{5.88}{4.8} = 1.225$

$\tan\theta = \frac{3.4}{4.8} = 0.71$

Diagonal stresses.

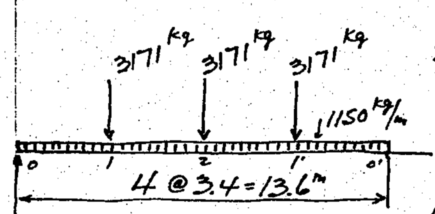
panel		shear		S.R. 1200 kg/cm²	19ϕ rivets	use	
0-1	2,040 × 6/4 =	3,060 × 1.225 =	3,750 kg	3.12 cm	1.8	3 rivets	1L 75×75×9 = 12.69 cm²
1-2	2,040 × 3/4 =	1,530 × " =	1,870	1.56	0.9	2 "	do

Approximate weight of lateral bracing.
8Ls 75×75×9 @ 9.96 × 5.5 = 438
center connections 4 @ 15 = 60
498 (gusset pl. " main girder = 7/4 × ex.)

498 ÷ 13.6 = 36.6 kg/meter of bridge
panel load = 36.6 × 3.4 = 125 kg

Design of Main Girder. Span length = 4 panels @ 3.4 = 13.6 meters.
Dead Load panel concentration due to
stringer concentrations = 2,890 kg
floor beam 436 ÷ 2 = 218
lateral bracing 125 ÷ 2 = 63
3,171 kg

Uniform load due to
Handrail, coping + over hanging slab 426
floor between main girder + stringer 400
826 kg/m



main girder assumed 324
1150 kg/m.

Dead Load moment at center of span.
panel concentration. 3171 × 2 = 6342 × 2 = 12,684
3171 × 1 = -3171
9513 × 3.4 = 32,300 kgm

Uniform load 1/8 × 1150 × 13.6² = 26,600
total = 58,900 kgm.

End shear panel concentration 3171 × 1.5 = 4760 kg
" uniform load 1150 × 3.4 × 2 = 7820
12,580 kg.

Moment at panel pt. 1. 12,580 × 3.4 = 42,700 ~~moment at panel pt 2.~~
1150 × 3.4²/2 = -6660
36,040 kgm.

Shear at panel pt. 1. 12,580 - 1150 × 3.4 = 8,670 kg

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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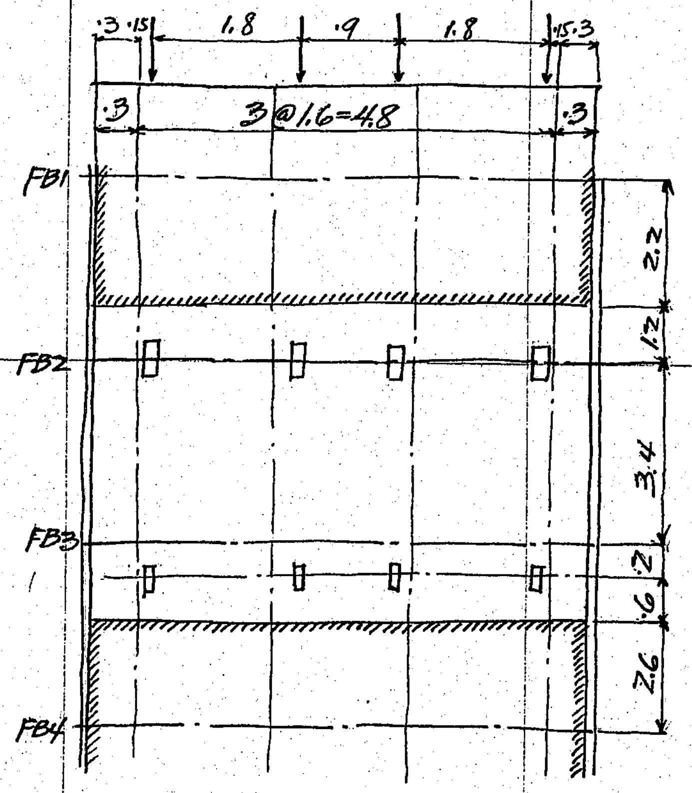
CHECKED BY M. Jinda DATE 3-10-11 PAGE NO 27

CALCULATIONS FOR

Ashida-Bashi for Okayama Ken

Live Load.



Uniform load directly on main girder. 565 kg/m. see page 19.

panel concentration due to stringer.

for FB1. 785 × 3.4 ÷ 2 = 1,335
785 × 2.2 × 2.3/3.4 = 1,168
2,503 kg

for FB2. $\frac{785 \times 2.2^2}{2 \times 3.4}$ = 559 kg

for FB3. $\frac{785 \times 2.6^2}{2 \times 3.4}$ = 780 kg

for FB4 $\frac{785 \times 2.1 \times 2.6}{3.4}$ = 1,260
785 × 3.4 ÷ 2 = 1,335
2,595 kg

motor truck wheels directly on main girder assumed.
motor truck rear wheel.

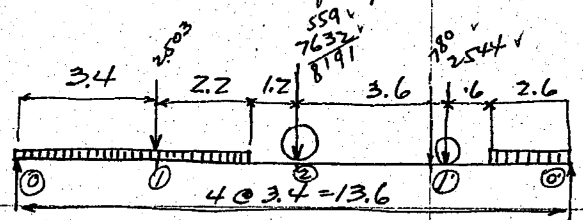
Impact coefficient = $\frac{20}{60+13.6}$ = 27.2 %

Rear wheel concentration 3,000 kg
impact 27.2 % 816
3,816 kg

front wheel 3,816 ÷ 3 = 1,272 kg

Concentration on main girder due to rear wheel.
Assumed 2 × 3816 = 7632 kg
front wheel 2 × 1272 = 2,544 kg

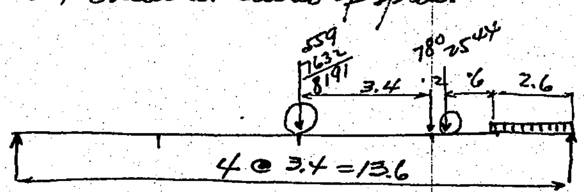
moment at center of span.



Panel load due to uniform load.
2503 × 3 = 7,510
559 × 2 = 1118
780 × 1 = 780
reaction = 9408 kg ÷ 4 = 2,350 kg
~~9408~~ 2350 × 2 = ~~18820~~ 4,700
2503 × 1 = -2,503
~~16317~~ 2,197 × 3.40 = ~~55450~~ 7,470 kgm.

Motor truck
7632 × 6.8 ÷ 13.6 = 3,830
2544 × 3.2 ÷ 13.6 = 598
4,428 kg × 6.8 = 30,100
37,570 kgm.

max shear at center of span.



panel concentration due to unif load.
780 × 1/4 = 195
559 × 2/4 = 279
474 kg

motor truck
7632 ÷ 2 = 3,816
~~780~~ 2544 × 3.2/13.6 = 598
4,414 kg

Uniform load.
$\frac{565 \times 2.6^2}{2 \times 13.6}$ = 141
5,029 kg

Uniform load direct on main girder
565 × 5.6 = 3,165 kg
3,165 × 10.8 = 34,200
565 × 2.6² ÷ 2 = 1,910
36,110 ÷ 13.6 = 2,660 kg

2,660 × 6.8 = 18,100
3165 × 4.0 = 12,660
5,440 kgm.

Total live load moment 7,470 + 37,570 + 5,440 = 50,480 kgm

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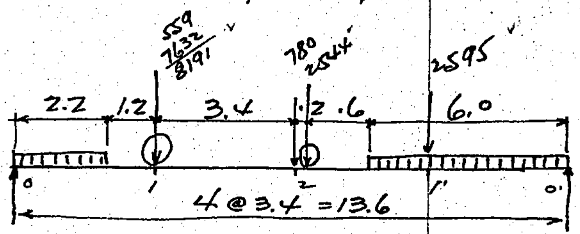
Moment at panel point No.1.



Moment due to panel concentration (unif. load).

559 × 3/4 = 420
780 × 2/4 = 390
2595 × 1/4 = 648
1458 × 3.4 = 4960 kgm.

Moment due to motor truck

7632 × 3/4 = 5720
2544 × 6.6/13.6 = 1235
6955 × 3.4 = 23650 kgm

Max shear at panel pt. No.1.

Shear due to panel concentration = 1458 kg
" " motor truck = 6955
" " unif. load 10170/13.6 = 748
9161 kg.

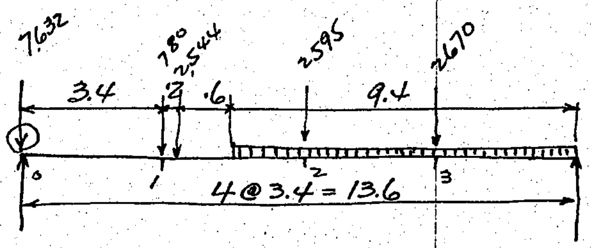
Moment due to uniform load.

565 × 2.2 × 12.5 = 15530
565 × 6.0² ÷ 2 = 10170
25700

reaction 25700/13.6 = 1890 kg
moment 1890 × 3.4 = 6420
565 × 2.2 × 2.3 = 2860
9280 kgm.

Total Live Load moment = 37890 kgm.

Max end shear.



Shear due to.

panel concentration due to unif. load.

2670 × 1/4 = 668
2595 × 2/4 = 1298
780 × 3/4 = 585
2551

motor truck

7632
2544 × 10.0/13.6 = 1870
9502

Uniform load.

565 × 9.4²/(13.6 × 2) = 1850

total end shear = 13903 kg.

Summary for moments and shears.

	Center of span (panel pt 2) moment	Center of span (panel pt 2) shear	Panel pt. No.1. moment	Panel pt. No.1. shear	End support End shear
Dead Load	58900	—	36040	8670	12580 kg
Live Load.	50480	5029	37890	9161	13903
	109380 kgm	5029 kg	73930 kgm	17831 kg	26483 kg

Section of main Girder

web pl. assumed 130 cm × 1.0 cm. gross area = 130 □cm 1/8 web = 16.25 □cm

depth back to back of flange L's = 131 cm effective depth say 126 cm.

flange stress = 109380 × 100 / 126 = 86800 kg

flange area req'd. = 86800/1200 = 72.4 □cm net
16.25
56.15 □cm net

Use 2L's 150 × 150 × 15 = 85.5 □cm − 15.0 = 70.5 □cm net OK.

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CONSULTING ENGINEER
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CALCULATIONS FOR

Ashida Bashi for Okayama ken.

Approximate weight of main girders

Web	1Pl.	1300 × 10	@	102.05 kg	× 13.9	= 1420 kg
Flg.	4Ls.	150×150×15	@	33.55	× 13.9	= 1866
Splice	2Pls.	300 × 10	@	23.55	× 0.66	= 31
〃	4Pls.	170 × 15	@	20.02	× .62	= 50
Stiff.	8Ls.	125×90×10	@	16.09	× 1.28	= 165
Fills	4Pls.	300 × 15	@	35.33	× 1.005	= 142
Stiff.	22Ls.	125×90×10	@	16.09	× 1.31	= 464
Shelf.	8Ls.	100×75×10	@	12.95	× .16	= 17
guss. lateral	3Pls.	330 × 9	@	23.32	× .51	= 36
〃	2Pls.	225 × 9	@	15.9	× .31	= 10
Shoes	2Pls.	300 × 19	@	44.75	× .53	= 47
〃	2Pls.	320 × 35	@	~~97.92~~	× .53	= ~~93~~ 101
〃	4Ls.	75×75×9	@	9.96	× .21	= 8
Rivets heads + Variation 3.5 %						= 156

4513 kg ÷ 13.6 = ~~332~~ kg/m of span

Details 32.8 % including rivet heads

Stringer 4834 × 1360 × 2 = 1315
Floor Beam 5 @ 444 = 2220
lateral Bracings = 498
Main girders 2 @ 4509 = 9020
13,053

Max. Load on shoe

Dead Load	end shear	12580	dead load	12580
Live load motor truck + unif. load		13903	unif. live load only	$\frac{500 \times 2.7 \times 13.9}{2}$ = 9380
		26483 kg.		21960 kg.

call 22 000 kg.

Considering effect of overhanging, take this at 28000 kg for one shoe.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
SEIYU BLDG, TOKIO.

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CALCULATIONS FOR

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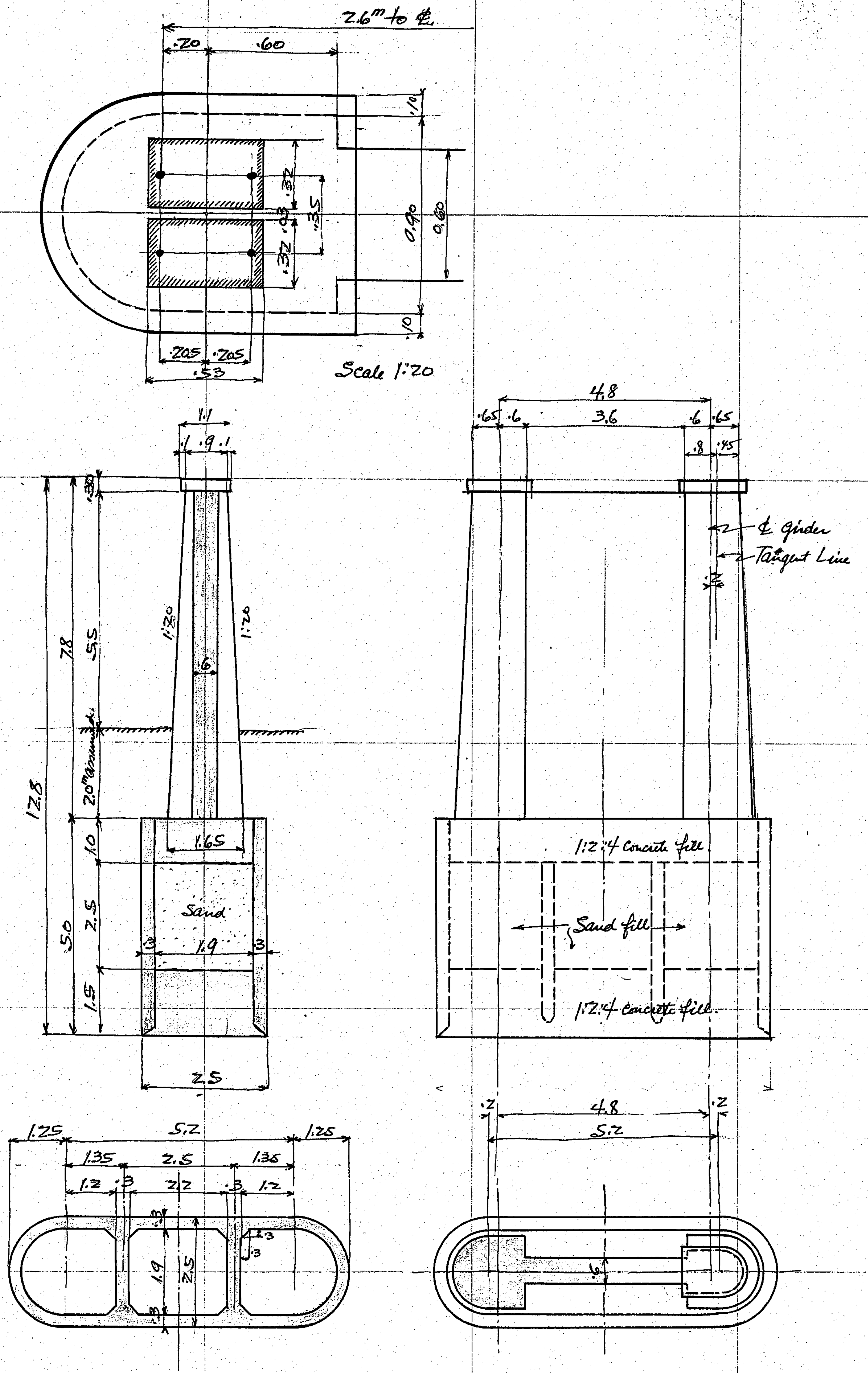
Design of Piers for girder spans.



Scale 1:100

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CALCULATIONS FOR

Ashida-Bashi for Okayama ken.

Loads on pier for 18.65 m girder spans.

Super imposed dead load say 18,000
do. live load 18,000
total = 36,000 kg. for one shoe

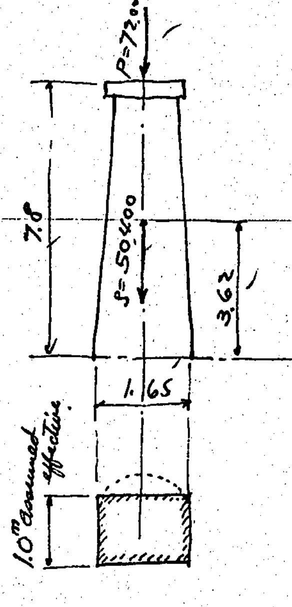
Weight and Center of gravity of shaft.

	Reqd no.	section	length	volume	weight	lever arm	moment
Coping	1	1.1φ	0.3	.285 c2400 =	684	7.65	5230
"	2	0.9×1.1	0.3	.594 @ " =	1,425	7.65	10,900
shaft	1	1.275φ	7.5	9.58 @ " =	23,000	3.13	72,000
"	2	1.275×.8	7.5	15.30 @ " =	36,700	3.44	126,300
curtain wall	1	3.6×.6	7.5	16.20 @ " =	38,900	3.75	146,000
				41.96 cub m	100,709 kg ×	3.58 =	360430 kgm

Call this 100,700 kg

Stability of shaft

Case 1. Stability at normal state.



Super imposed load say 2 @ 36000 = 72000 kg for one shaft.
dead weight of shaft. 100700 ÷ 2 = 50,400
122,400 kg.

Unit compression = $\frac{122400}{165 \times 100}$ = 7.4 kg/cm² ok.

Case 2. Stability of shaft during earthquake. k = 0.10 assumed

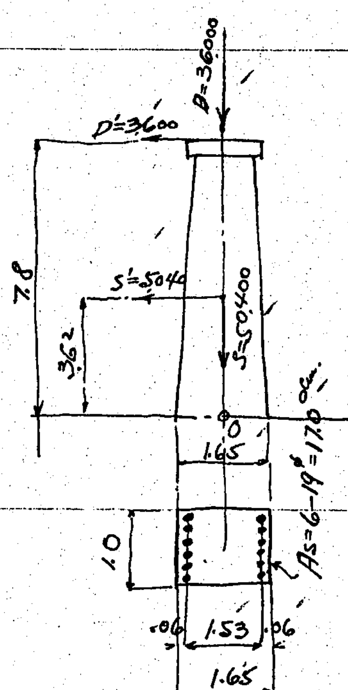
Taking moment about point o. (Seismic force due to super structure assumed to be applied at the top of pier.)

Vertical load	Hor. load	lever arm	moment
p = 36,000	p' = 3600	× 7.8 =	28100
s = 50400	s' = 5,040	× 3.58 =	18,050
86400 kg	8640 kg		46,050 kgm

Eccentricity $e = \frac{46100}{86400} = 0.536^m$

$e/h = \frac{.536}{1.65} = 0.325$, $d'/h = .06/1.65 = 0.0364$

$p = 2p = \frac{17 \times 2}{100 \times 165} = 0.00206$



k = 0.62, L = 0.102

$f_c = \frac{46100 \times 100}{0.102 \times 100 \times 165^2} = 16.7$ kg/cm² ok

$f_s = 15 \times 16.7 \left(\frac{159}{.62 \times 165} - 1\right) = 139$ kg/cm² ok.

Unit shear = $\frac{8640}{100 \times \frac{7}{8} \times 159}$ = 0.6 kg/cm² ok

Unit bond = $\frac{8640}{5.97 \times 6 \times \frac{7}{8} \times 159}$ = 1.73 kg/cm² ok.

Design of well 5.0 m deep and 2.5 m wide.

Temporary earth pressure on well during execution.

Reffer to Ketchum's, Walls, bins and Grain elevators page 120+121.

Side pressure for temporary trench works.

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

Where. L = Lateral unit pressure in kg per sq. meter at depth y meters.
v = vertical " " " " "
w = weight of earth in kg per cub. meter.
φ = angle of repose of earth
μ = tan φ coefficient of friction of earth on earth.
b = the distance in meters that the earth breaks around the well.
φ' = angle of friction of earth on the surface of the well.

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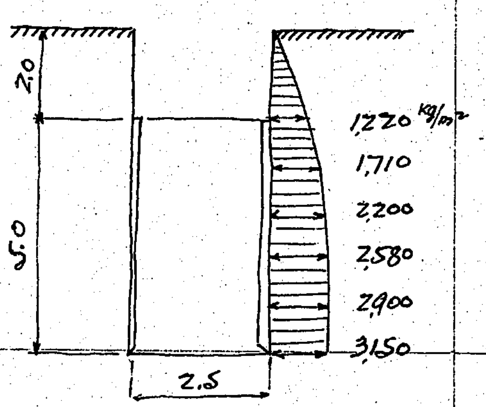
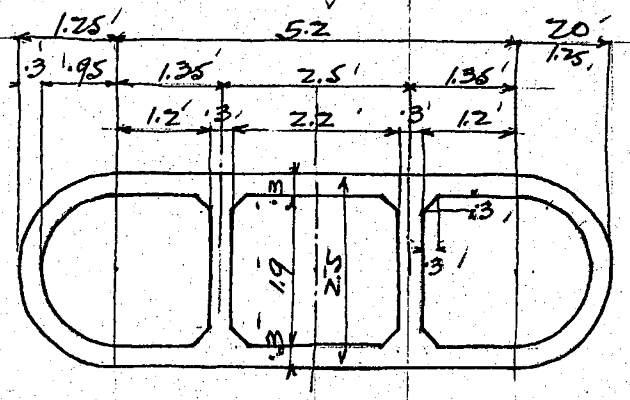


Assume that $\phi = \phi' = 30°$, $k = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1}{3}$.
b = 3 meters, w = 2000 kg per cu. meter.

Depth of earth	temporary earth pressure		
1 meter	490	× 2000/1600 =	610 kg/□m
2	980	" " =	1,220
3	1,370	" " =	1,710
4	1,760	" " =	2,200
5	2,060	" " =	2,580
6	2,320	" " =	2,900
7	2,520	" " =	3,150
8	2,740	" " =	3,420
9	2,930	" " =	3,660
10	3,070	" " =	3,840
12	3,340	" " =	4,180
14	3,520	" " =	4,400
15	3,610	" " =	4,510
16	3,660	" " =	4,580
18	3,800	" " =	4,750
20	3,900	" " =	4,870



moment on side wall. $1/12\ wl^2$ assumed. $l = 2.5^m$.

Section	side pressure	moment.	moment on circular end $1/16\ wl^2$
top.	1,220 kg/m²	635 kgm	370 kgm
1m below top.	1,710	890	520
2	2,200	1,145	666
3	2,580	1,345	780
4	2,900	1,510	880
bottom	3,150	1,640	955

Section at bottom

moment on side wall = 1,640 kgm. moment on circular ends = 955 kgm.
effective depth of wall required $= \sqrt{\frac{M}{bR}}$, $R = 7.18$

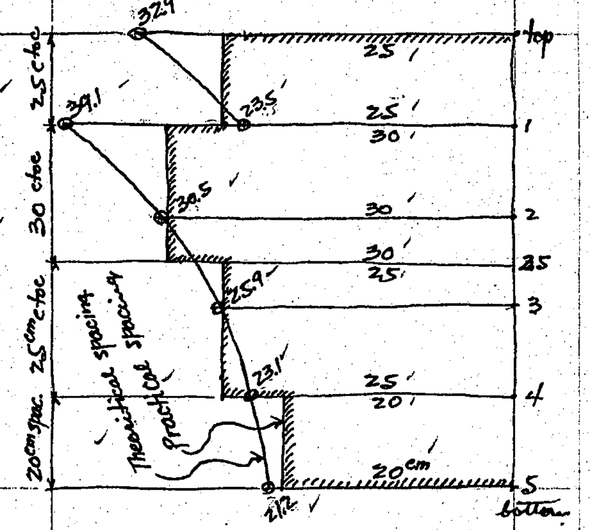
$$= \sqrt{\frac{1640 \times 100}{100 \times 7.18}} = 15.1^{cm}.$$

Use 25 cm effective depth with 5 cm insulation, total depth 30 cm.
steel area req'd. $= \frac{1640 \times 100}{1200 \times \frac{7}{8} \times 25} = 6.25$ □cm per meter strip of wall.

Use 13mm φ bars at 21.2 cm spacing = 6.25 □cm.
end shear = 3150 × 1.25 = 3,940 kg
Unit shear $= \frac{3940}{100 \times \frac{7}{8} \times 25} = 1.8$ kg/cm² ok

Unit bond $= \frac{3940}{\frac{4.08}{.212} \times \frac{7}{8} \times 25} = 4.23$ kg/cm² ok

Shear at end of chamfer = 3,150 × 0.8 = 2,520 kg
unit bond $= \frac{2520}{\frac{4.08}{.212} \times \frac{7}{8} \times 25} = 5.98$ kg/cm² ok.



Reinforcement spacing diagram

Section at 4m below top. moment on side wall = 1,510 kgm.
steel area req'd. $= \frac{1510 \times 100}{1200 \times \frac{7}{8} \times 25} = 5.75$ □cm

Use 13mm φ bars at 23.1 cm spacing = 5.75 □cm.

Section at 3m below top moment on side wall = 1345 kgm
steel area req'd $= \frac{1345 \times 100}{1200 \times \frac{7}{8} \times 25} = 5.13$ □cm

Use 13mm φ bars at 25.9 cm spacing = 5.13 □cm.

Section at 2m below top moment on side wall = 1145 kgm SR = 4.36 □cm
Use 13mm φ bars at 30.5 cm = 4.36 □cm.

JIUN MASUDA
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CALCULATIONS FOR

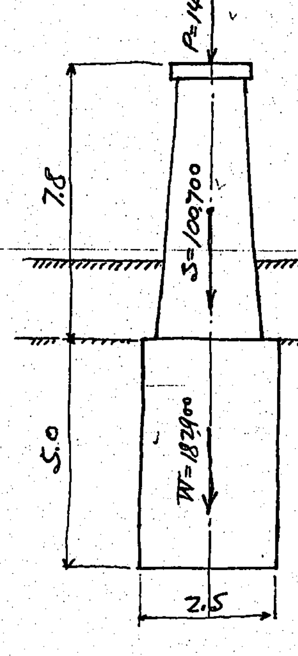
Ashida Bashi for Okayama Ken.

section at 1 meter below top moment on side wall = 890 kgm.
steel area reqd. = $\frac{890 \times 100}{1200 \times \frac{7}{8} \times 25}$ = 3.39 ocm
Use 13 mm ϕ bars at 39.1 cm = 3.39 ocm

section at 0 ~ 1 m from top moment on side wall = 890 kgm.
effective depth say 15 cm
steel area required = $\frac{890 \times 100}{1200 \times \frac{7}{8} \times 15}$ = 5.65 ocm
Use 13 mm ϕ at 23.5 cm = 5.65 ocm

for circular ends at all sections, use same reinforcements at same spacings.

Stability of Piers for 18.65 m girder spans.
Case 1. Stability at normal state.



Weight and center of gravity of well.

total area of well.		area of partition.		Circumference of well.
5.2 × 2.5 =	13.0 ⁰ᵐ	1.9 × 4.6 =	8.74	5.2 × 2 = 10.4 m
2.5ϕ =	4.9	1.9ϕ =	2.84	2.5ϕ = 7.85
	17.9 ⁰ᵐ	.3 × .3 × 4 =	− .36	18.25 m
			11.22 ⁰ᵐ	

sectional area of shell = 17.90 − 11.22 = 6.68 ⁰ᵐ

0 to 1 m from top.
vol. of concrete 17.9 × 1 = 17.9 cu m @ 2,200 = 39,400 kg ~~39,800~~

1 to 2 m from top.
vol. of concrete 6.68 × 1 = 6.68 @ 2,200 = 14,700 kg.
vol. of sand 11.22 × 1 = 11.22 @ 1700 = 19,100
33,800 kg.

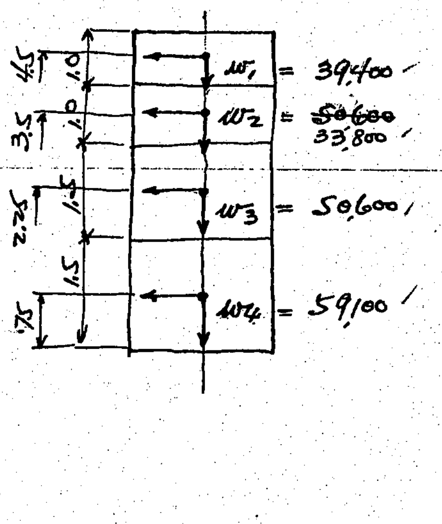
2 to 3.5 m from top.
vol. of concrete 6.68 × 1.5 = 10.01 @ 2,200 = 22,000
vol. of sand 11.22 × 1.5 = 16.85 @ 1,700 = 28,600
50,600 kg.

3.5 m to 5 m from top
Vol. of Concrete 17.90 × 1.5 = 26.85 @ 2200 = 59,100 = 59,100
~~vol. of sand~~
Total wt. of well = 182,900 kg

Loads on well.
superimposed D.L. + L.L. = 36000 × 4 = 144,000 kg
weight of shaft = 100,700
weight of well = 182,900
total = 427,600 kg

Frictional resistance of earth on well surface assumed at 1000 kg/m²
Total friction ; assuming effective penetration of well = 5.0 m
= 1000 × 18.25 × 5 = 91,300 kg
Resulting pressure on well = 427,600 − 91,300 = 336,300 kg
Bearing pressure on the bottom of well = $\frac{336,300}{17.9}$ = 18,800 kg/m² (or 1.72 tons/□') O.K.
If frictional resistance be neglected $18800 \times \frac{427600}{336300}$ = 23,900 kg/m² (or 2.19 tons/□') O.K.

Center of gravity of well.



section	weight	lev. arm	moment
0 ~ 1 m	39,400 kg ×	4.5 m =	177,500
1 ~ 2	33,800 ×	3.5 =	118,300
2 ~ 3.5	50,600 ×	2.25 =	114,000
3.5 ~ 5 m	59,100 ×	.75 =	44,300
	182,900 kg ×	2.49 =	454,100

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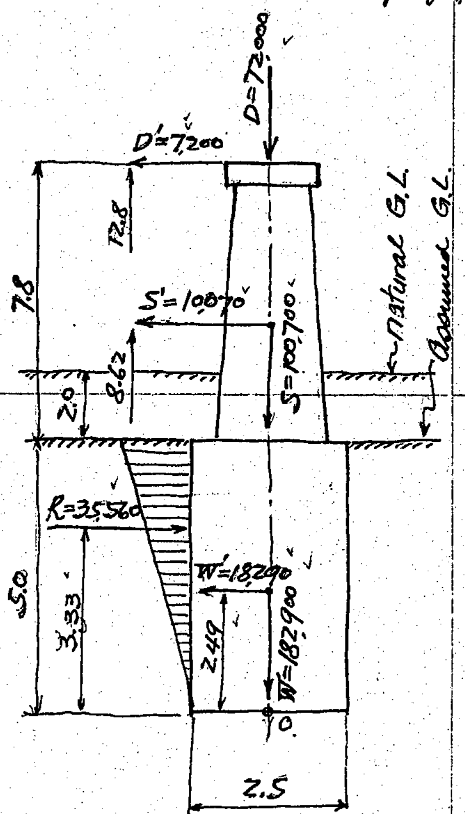
Case 2. Stability of pier during earthquake. K=0.1 assumed.



Taking moment about pt. O.

Loads	Hor. forces	Vert. forces	Lever arms	moments
D		72,000		
D'	7,200		× 12.8 =	92,200
S		100,700		
S'	10,070		× 8.62 =	86,800
W		182,900		
W'	18,290		× 2.49 =	45,500
	ΣH = ~~45,560~~ 35,560	ΣV = 355,600		ΣM = 224,500

Frictional resistance 1000 kg/m², effective penetration of well assumed 5.0 m
Frictional resistance on side wall of well. mean width of well say 7.0 m
1000 × 5 × 7 = 35,000 kg

frictional couple = 35000 × 2.5 = 87,500 kgm.
moment due to earth reaction
35,560 × 3.33 = 11,8500
206,000 kgm.

Resulting moment = 224500 − 206,000 = 18500 kgm.

Eccentricity $e = \frac{18500}{355600} = 0.052$ m. Resultant force within middle third

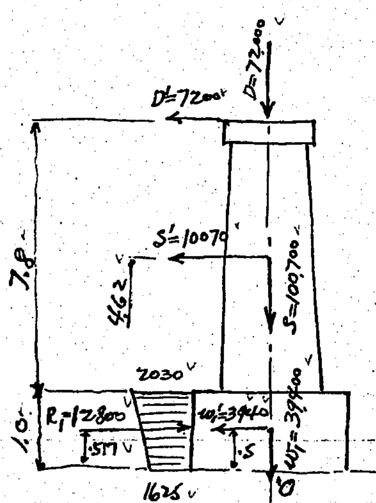
If frictional couple be neglected
moment = 87500 + 18,500 = 106000 kgm
eccentricity $e = \frac{106000}{355600} = .298$ m
max toe pres. $= \frac{355600}{2.5\times7.0}\left(1 \pm \frac{6\times.298}{2.5}\right)$
$= 34800$ kg/m² (or 3.18 tons/) or 5800

max toe pressure $= \frac{355600}{2.5\times7.0}\left(1 \pm \frac{6\times.052}{2.5}\right) = 22,900$ kg/m² (2.09 tons/) OK.
or = ~~20,000~~ 17,800

max reactional pressure on earth (2 m below ground surface)
$= \frac{35560\times2}{7\times5} = 2030$ kg/m²

Safe horizontal bearing power of earth at 2 meters below ground surface.
$= wh \times \frac{1+\sin\phi}{1-\sin\phi}$ = say $3wh = 3\times1600\times2 = 9600$ kg/m² > 2030 OK.

Vertical reinforcements for the well shell.
Moment at 1 m below top of well.



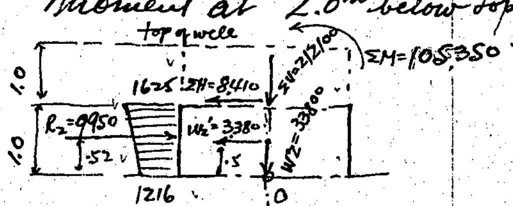
Taking moment about point O.

Loads.	Hor. forces	Vert. forces	lever arm	moment.
D		72,000 ×	0	= 0
D'	7,200	×	8.8	= 63,500
S		100,700 ×	0	= 0
S'	10,070	×	4.62	= 46,500
w1		39400 ×	0	= 0
w1'	3,940	×	.5	= 1,970
R1	−12,800	×	.517	= −6,620
	ΣH = 8,410 kg	ΣV = 212,100 kg		ΣM = 105,350 kgm

frictional couple 87500 ÷ 5 = 17,500
87,850 kgm.

eccentricity $= \frac{87850}{212,100} = .41$ m resultant force within middle third.

Moment at 2.0 m below top of well.



Moment about O.

	Hor. forces	Vert. forces	lever arm	moment
ΣV		212,100 ×	0	= 0
ΣH	8,410	×	1.0	= 8,410
ΣM.				= 105,350
W2		33,800 ×	0	= 0
W2'	3,380	×	.5	= 1,690
R2 =	−9,950	×	.52	= −5,170
	1,840	245,900		110,280

frictional couple −35,000
75,280 kgm

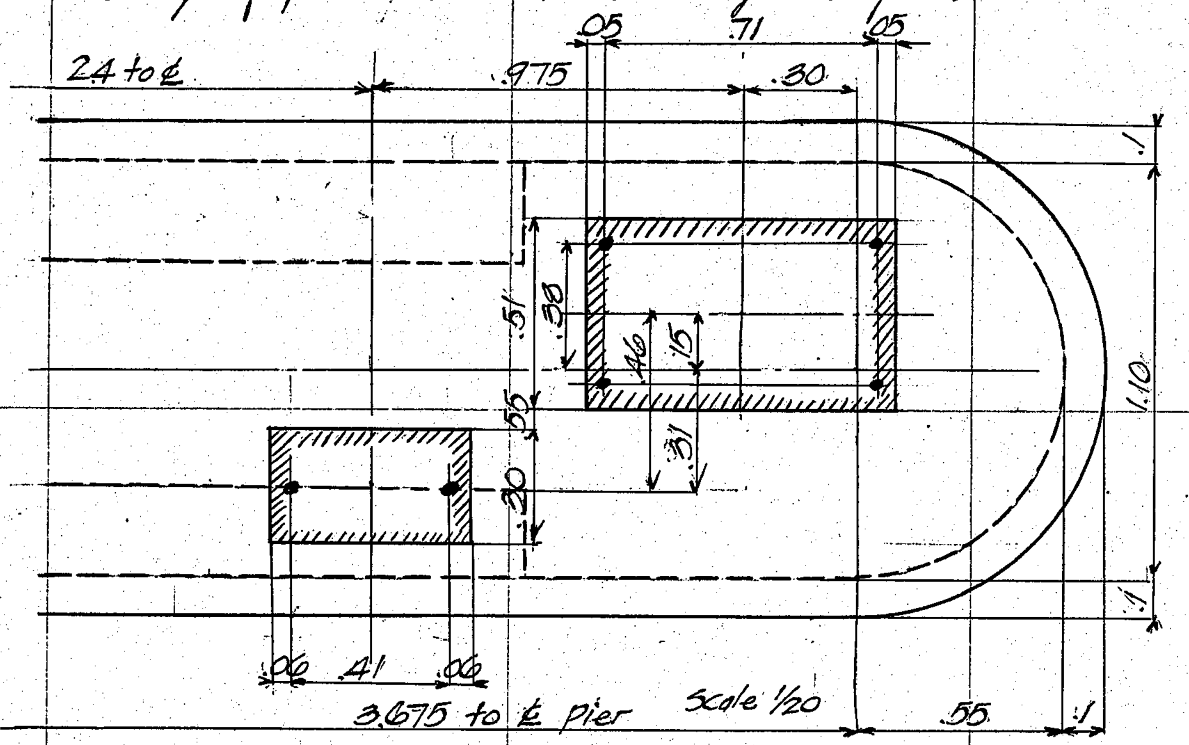
Eccentricity $= \frac{75280}{245900} = .306$ m
resultant force within middle third.

No vertical reinforcements required theoretically.
use 13 mm φ bars at 30 cm c to c on both sides

CALCULATIONS FOR

Ashida-Bashi for Okayama ken

Design of pier between Truss and Girder spans



Super imposed load on pier

Truss span

D.L. = 46,000 kg

L.L. = 30,000

76,000 kg/shoe

Girder span

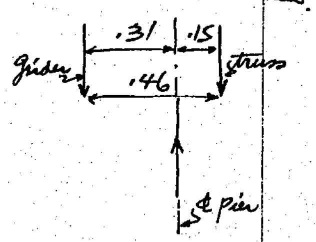
D.L. 18,000

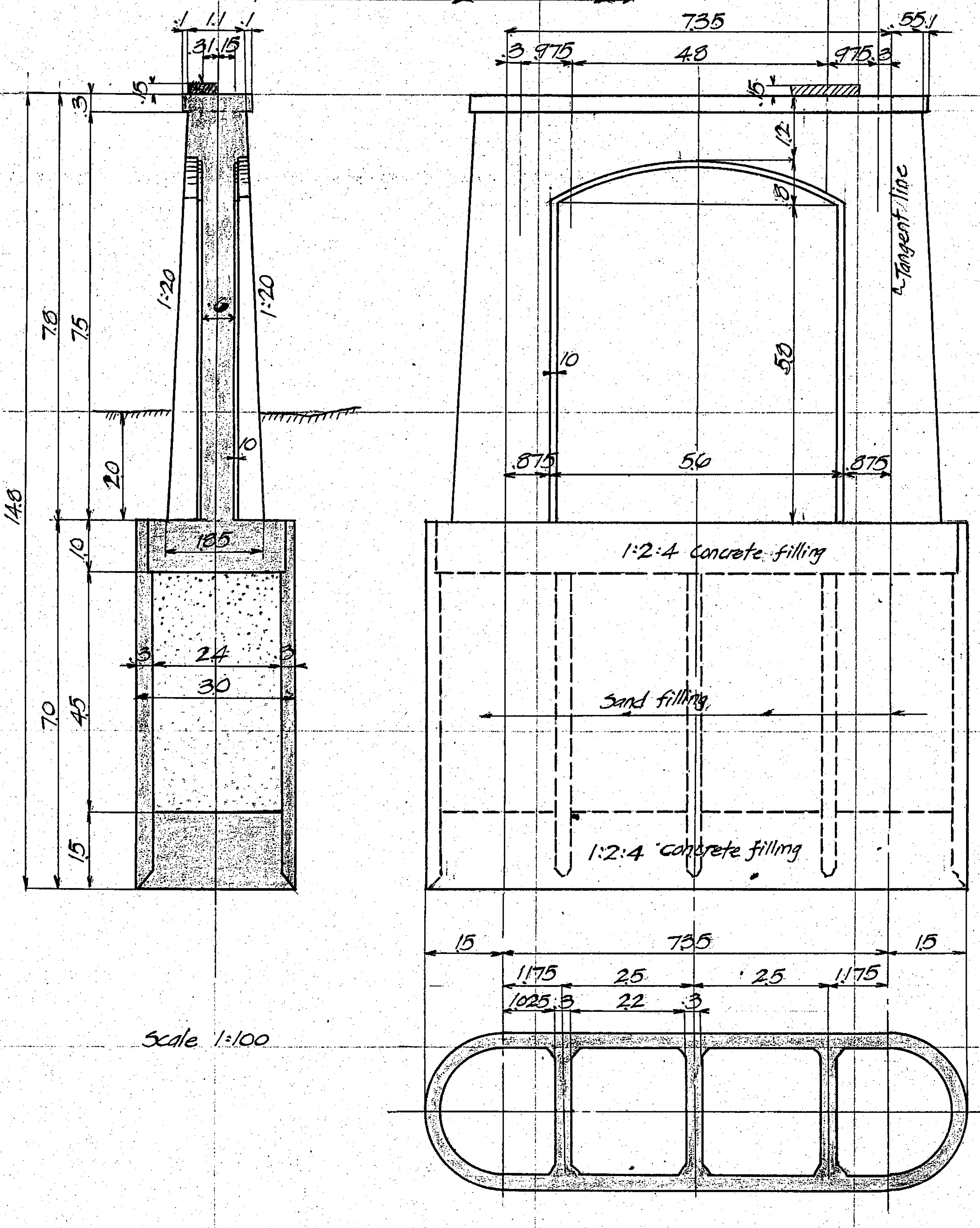
L.L. 18,000

36,000 kg/shoe

Distance c to c of bearings 0.46m Truss & Girder

Center of gravity $.46 \times \frac{36{,}000}{112{,}000} = .15^{m}$ from ℄ truss shoe.





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CALCULATIONS FOR

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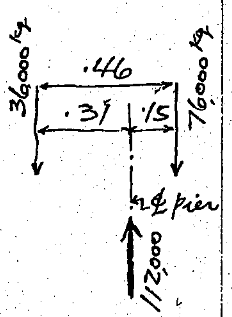
Weight and Center of gravity of shaft.

	Reqd. no	Section	Length	Volume		weight	lever arm	moment
Coping	1	1.3φ	× 0.3	.40 m³	@ 2400	1000 kg	7.65	7,700
〃	1	1.3 × .3	× 7.35	2.87	@ 〃	6900	7.65	52,800
shaft	1	1.475φ	× 7.5	12.83	@ 〃	30,800	3.15	97,000
〃	2	.875 × 1.475	× 7.5	19.35	@ 〃	46,400	3.40	158,000
top strut	1	1.3 × 1.17	× 5.6	8.51	@ 〃	20,500	6.85	140,300
curtain wall	1	.6 × 5.6	× 6.2	20.82	@ 〃	50,000	3.10	155,000
fillet	2	.1 × .1	× 17.2	.34	@ 〃	800	4.30	3,400
				65.12 cub.m.		156,400	3.93 m	614,200

Stability of shaft

Case 1. Stability at normal state.



super imposed load on pier

due to Truss span D.L. 46000 kg

L.L. 30000

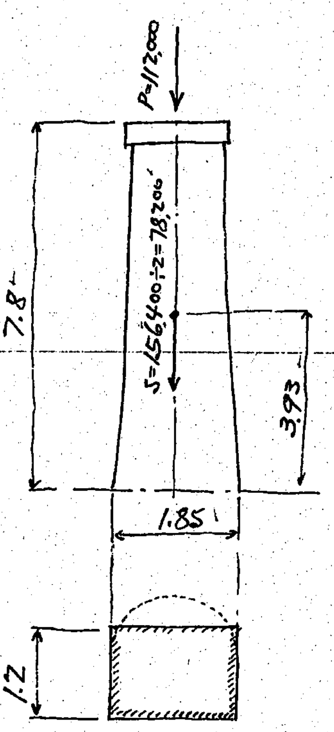
76,000 kg for one shoe.

due to Girder span D.L. 18,000

L.L. 18,000

36,000 kg for one shoe.

112,000 kg for one shaft.



Weight of shaft = 156400 kg

Effective width of shaft assumed 1.2 meters

max load on one shaft = 112000

78200

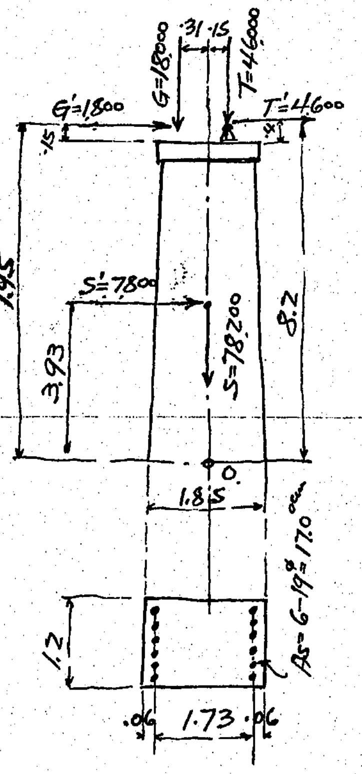
190,200 kg.

Unit bearing compression $= \frac{190200}{120 \times 185} = 8.6$ kg/cm² ok.

Case 2. Stability of shaft during earthquake $K = 0.1$ assumed.

Taking moment about pt. O.

Loads	Hor. forces	Vert. forces	lever arm	moments
G		18,000	× −.31	−5,600
G'	1,800		× 7.95	14,300
T		46,000	× .15	6,900
T'	4,600		× 8.20	37,700
S		78,200	× 0	0
S'	7800		× 3.93	30,600
	14,200 kg	142,200 kg		83,900 kgm



Eccentricity $= \frac{83900}{142200} = .59$ m

$e/h = \frac{.59}{1.85} = .32$, $d'/h = .06/1.85 = .033$

$p_0 = 2p = \frac{17 \times 2}{120 \times 185} = .00153$

From the prepared diagrams.

$K = .615$ $L = .099$

$f_c = \frac{83900 \times 100}{.099 \times 120 \times 185^2} = 207$ kg/cm² ok.

$f_s = 15 \times 207 \times \left(\frac{1.79}{.615 \times 1.85} - 1\right) = 179$ kg/cm² ok

Unit shear $= \frac{14200}{120 \times \frac{7}{8} \times 179} = .76$ kg/cm² ok

unit bond $= \frac{14200}{5.97 \times 6 \times \frac{7}{8} \times 179} = 2.5$ 〃 ok.

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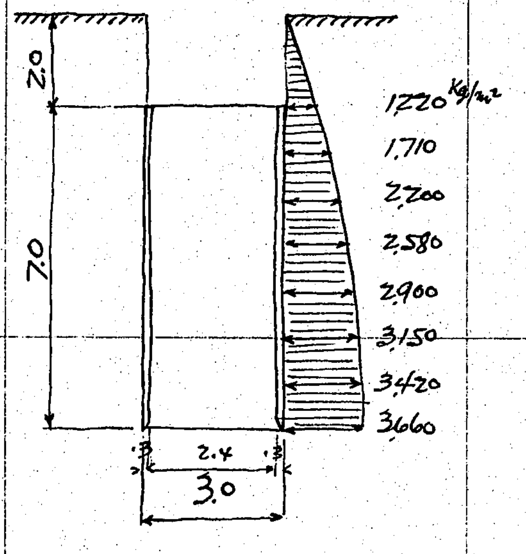
CALCULATIONS FOR

Ashida Bashi for Okayama Ken

Design of well. 7.0m long

Temporary earth pressure on well. see page 32
moment assumed $\frac{1}{12} wl^2$ on side wall, $\frac{1}{16} wl_1^2$ on circular ends.
where $l = 2.5^m$, $l_1 = 2.70^m$.
moment at several sections.



Section	side pressure	moment on side wall	moment on circular ends.
Top of well.	1,220 kg/m²	$\frac{1}{12}wl^2$ = 635 kgm	$\frac{1}{16}wl_1^2$ = 555 kgm
1m below top.	1,710	890	780
2	2,200	1,145	1,000
3	2,580	1,345	1,175
4	2,900	1,510	1,320
5	3,150	1,640	1,435
6	3,420	1,780	1,555
7 (bottom)	3,660	1,910	1,665

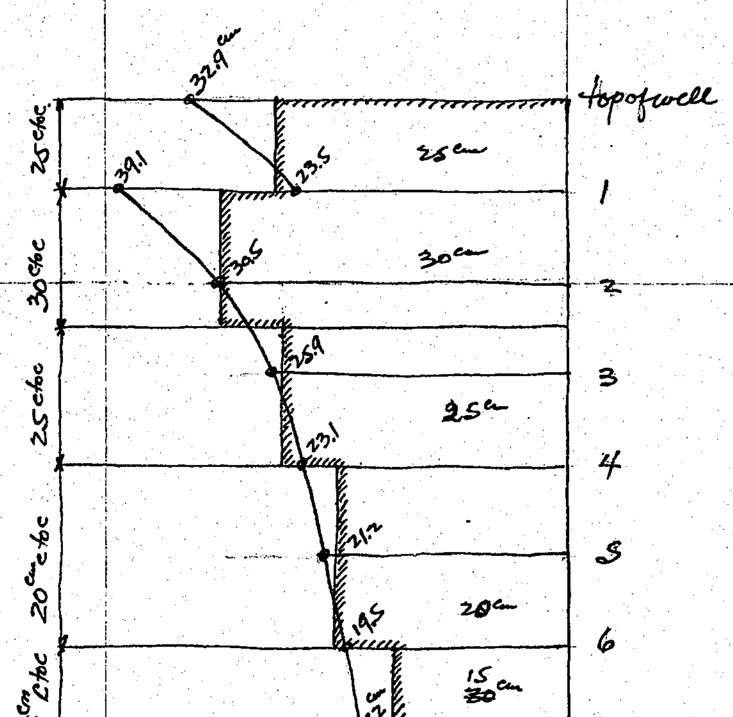
Section at bottom

moment on side wall = 1,910 kgm.
effective depth of wall $= \sqrt{\frac{M}{bR}}$, $R = 7.18$
$= \sqrt{\frac{1910 \times 100}{100 \times 7.18}} = 16.3^{cm}$.

Use 25cm effective depth with 5cm insulation, total depth 30cm.
Steel area required $= \frac{1910 \times 100}{1200 \times \frac{7}{8} \times 25} = 7.28$ ocm per meter strip of wall.
use 13mmφ bars at 18.2cm c to c. = 7.28 ocm.

Section at 6 meter below top. moment = 1780 Kgm.
Steel area req'd. $= \frac{1780 \times 100}{1200 \times \frac{7}{8} \times 25} = 6.79$ ocm
use 13mmφ bars at 19.5cm c to c.

The portion from top to 5 meter below, use same details as the pier for 18.65m girder span.
For circular ends use the same details as above for side wall.



Horizontal reinforcement spacing diagram.

7.0m × 2.5m well for Girder span (Pier P2) Pressure and moment on well same as for above well. use simillar details.

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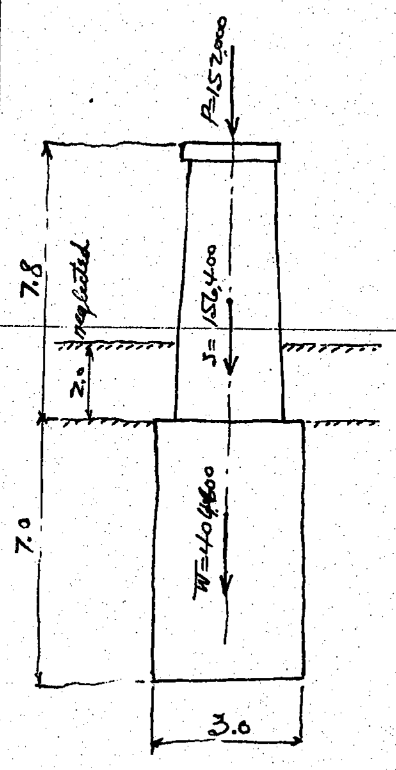
CALCULATIONS FOR

Ashida Bashi for Okayama Ken.

Stability of pier between Truss and Girder spans.

Case 1. Stability at normal state.



Weight and center of gravity of well.

total area of well.	Area of partition	Circumference of well.
7.35 × 3.0 = 22.05	2.4 × 6.45 = 15.47	7.35 × 2 = 14.70^m
3.0$^\phi$ = 7.07	2.4$^\phi$ = 4.52	3.0$^\phi$ = 9.42
29.12$^{\square m}$	.3 × .3 × 6 = − .54	24.12^m
	19.45$^{\square m}$	

Sectional area of shell = 29.12 − 19.45 = 9.67$^{\square m}$.

0 to 1^m from top.
volume of concrete = 29.12 × 1.0 = 29.12$^{m^3}$ @ 2200 = 64,100^{kg}

1^m to 3^m from top.
volume of concrete = 9.67 × 2.0 = 19.34 @ 2200 = 42,500
volume of sand = 19.45 × 2.0 = 38.90 @ 1700 = 66,100
108,600

3 to 5.5^m from top.
volume of concrete = 9.67 × 2.5 = 24.18 @ 2200 = 53,200
volume of sand = 19.45 × 2.5 = 48.60 @ 1,700 = 82,600
135,800

5.5 to 7.0^m from top.
volume of concrete = 29.12 × 1.5 = 43.70 @ 2200 = 96,100

Total weight of well. = 404,600 ^{kg}

total vol of concrete = 116.34$^{m^3}$
" " sand = 87.50 "

Loads on well.

super imposed loads D.L.+L.L. Truss span 2 @ 76000 = 152,000 kg
Girder span 2 @ 36000 = 72,000
224,000 kg

weight of shaft 156,400
weight of well 404,600
785,000 kg.

Frictional resistance of earth on well surface assumed 1000 kg/m².

Total friction on well = 1000 × 24.12 × 7.0 = 169,000 kg

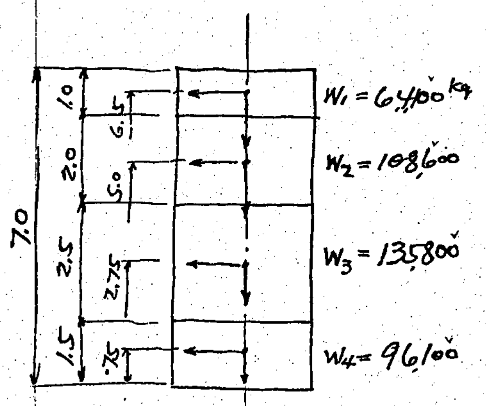
Resulting pressure on well = 785,000 − 169,000 = 616,000 kg

Bearing pressure on the bottom of well.

$= \frac{616000}{29.12} = 21,150$ kg/m² or (1.93 tons/□') ok.

If frictional resistance be neglected max pressure on bottom

$= \frac{785000}{29.12} = 26,950$ kg/m² or (2.46 tons/□') ok.



Center of gravity of well.

section	weight		lever arm		moment
0 to 1^m	64,100	×	6.5	=	416,500
1 to 3	108,600	×	5.0	=	543,000
3 to 5.5	135,800	×	2.75	=	373,500
5.5 to 7.0	96,100	×	.75	=	72,000
	404,600	×	3.47^m	=	1,405,000

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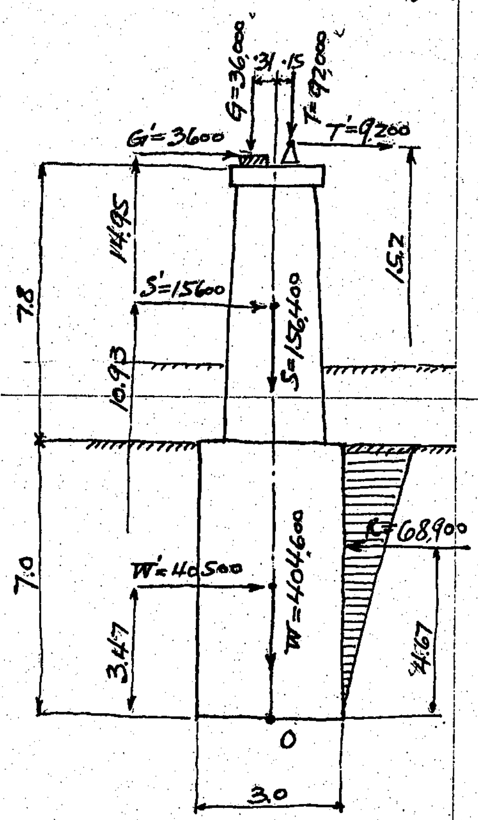
CALCULATIONS FOR

Ashida Bashi for Okayama Ken

Case 2. Stability of pier during earthquake. $k = 0.1$.



Taking moment about point O.

Loads	Horizontal forces	Vertical forces	lever arm	moments
T		92,000	× .15 =	13,800
T'	9,200		× 15.2 =	139,800
G		36,000	× −.31 =	−11,200
G'	3,600		× 14.95 =	53,800
S		156,400	× 0 =	0
S'	15,600		× 10.93 =	170,600
W		404,600	× 0 =	0
W'	40,500		× 3.47 =	140,500
	ΣH = 68,900 kg	ΣV = 689,000 kg	ΣM =	507,300 kgm.

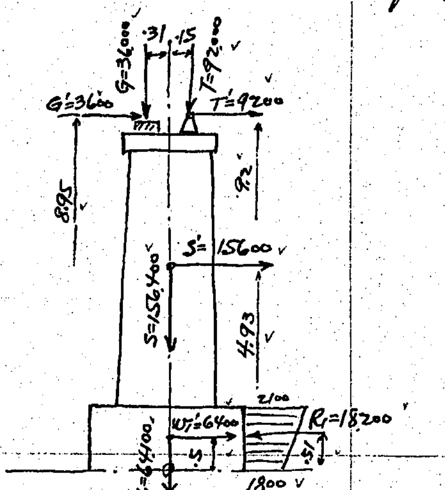
Neglecting frictional resistance of earth. Effective penetration 7.0 m assumed.
Thickness of pier 3.0 m at bottom. mean effective width of well 9.35 m say
Earth reaction 68,900 kg lever arm $\frac{2}{3} \times 7 = 4.67$ m
~~moment due to earth reaction~~
$= 68900 \times 4.67 = 322,000$ kgm.

Resulting moment $= 507,300 - 322,000 = 185,300$ kgm
eccentricity $= \frac{185300}{689000} = .269$ m

Resultant force within middle third.
max. toe pressure $= \frac{689000}{9.35 \times 3.0}\left(1 \pm \frac{6 \times .269}{3.0}\right) = 37,800$ kg/m² or (3.45 tons/□) ok
or $= 11,350$

max reactional pressure on earth (at the top of well)
$= \frac{68,900 \times 2}{7.0 \times 9.35} = 2,100$ kg/m² < 9600 see page 34

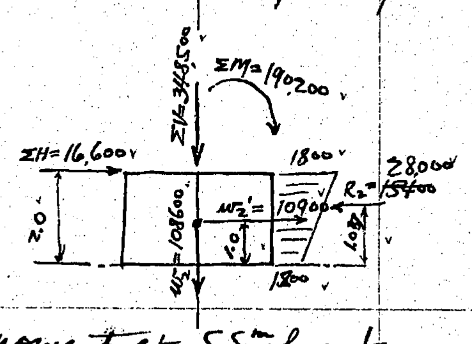
Moment at several sections of well.
moment at 1.0 m below top of well.



Taking moment about point O.

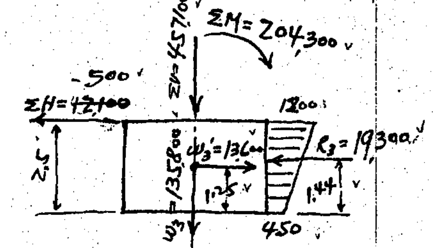
T		92000	× .15 =	13,800
T'	9200		× 9.20 =	84,600
G		36000	× −.31 =	−11,200
G'	3600		× 8.95 =	32,200
S		156,400	× 0 =	0
S'	15,600		× 4.93 =	76,900
w_1		64,100	× 0 =	0
w_1'	6,400		× .50 =	3,200
R_1	−18,200		× .51 =	−9300
	ΣH = 16,600	ΣV = 348,500	× .55 ΣM =	190,200 kgm.

Moment at 3.0 m from top.



ΣV		348,500	× 0 =	0
ΣH	16,600		× 2.0 =	33,200
ΣM			=	190,200
w_2		108,600	× 0 =	0
w_2'	10,900		× 1.00 =	10,900
R_2	~~−15400~~ −28000		× 1.037 =	~~−15,900~~ −36,000
	ΣH = ~~12100~~ −500	ΣV = 457,100	× .45 ΣM =	~~218,400~~ 204,300

moment at 5.5 m from top.



ΣV		457,100	× 0 =	0
ΣH	−500		× 2.5 =	−1,300
ΣM				204,300
w_3		135,800	× 0 =	0
w_3'	13,600		× 1.25 =	17,000
R_3	−19,300		× 1.44 =	−27,800
	ΣH = −6,200	592,900	× .325	192,200 kgm.

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CONSULTIEG ENGINEER
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CALCULATIONS FOR

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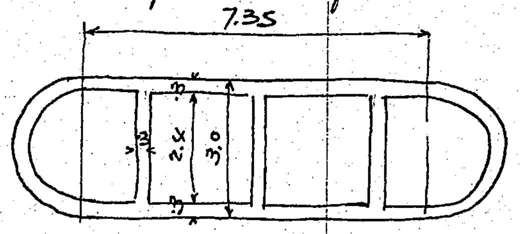
Vertical Reinforcements for the well shell.



Moment of inertia of section. effect of reinforcement neglected.

Circular ends. $0.049 \times 3.0^4 = 3.97$ m^4

Straight portion $\frac{7.35 \times 3.0^3}{12} = \underline{16.55}$

20.52 m^4

Moment of inertia of hollow

circular ends. $0.049 \times 2.4^4 = 1.63$

Straight portions $\frac{6.45 \times 2.4^3}{12} = \underline{17.40}$

$\underline{19.03}$

11.49 m^4 $= I$

Sectional area of well shell = 9.67 m^2

Section at 1 meter below top of well. Direct load = 348,500 kg moment = 190,200 kgm

moment stress $= \frac{190200 \times 1.5}{11.49} = 24,800$ kg/m² $= 2.4$ kg/cm² C or T.

direct compression $= \frac{348,500}{9.67} = 36,000$ kg/m² $= 3.6$ kg/cm² C.

Resulting max. fibre stress $= 3.6 \pm 2.4 = 6.0$ kg/cm² C or 1.2 kg/cm² C.

No vertical reinforcement required theoritically, but for the practice use 13 mm φ bars 30 cm c to c both sides of well.

Section at 5.5 m below top of well. Direct load = 592,900 kg, moment = 192,200 kgm.

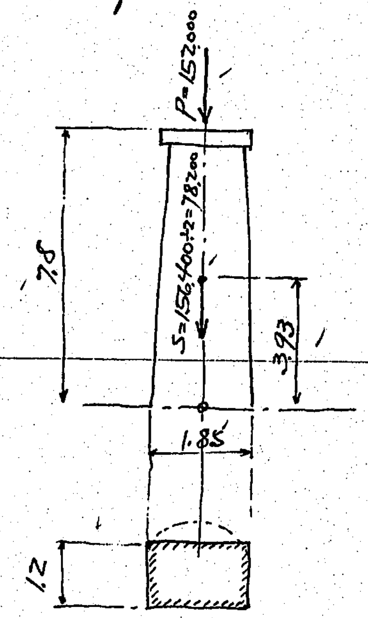
moment stress $= \frac{192,200 \times 1.5}{11.49} = 25,100$ kg/m² or 2.5 kg/cm² C or T

direct stress $= \frac{592900}{9.67} = 61,500$ " or 6.2 kg/cm² C

Resulting max. fibre stress $= 6.2 \pm 2.5 = 8.7$ kg/cm² C or 3.7 kg/cm² C.

Use same detail as for the previous section.

Ashida Bashi for Okayama ken.
Design of Pier for Truss spans.
Shaft. Use same details as for pier between Truss & Girder span.

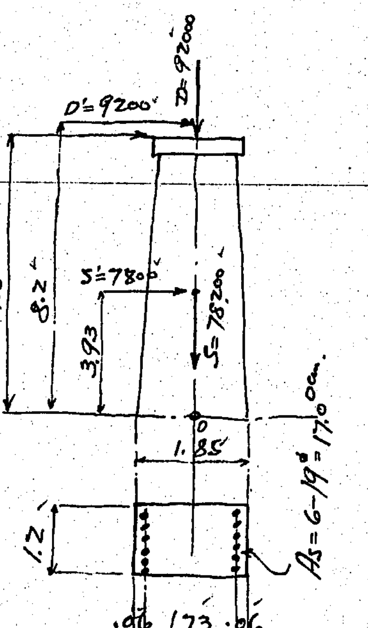


Weight of shaft. 156,400 kg Center of gravity 3.93 m above bottom of shaft.
Super imposed load on pier

Dead Load.	46000 × 2 =	92,000		184,000	
Live Load	30,000 × 2 =	60,000		120,000	
	76,000 kg	152,000 kg.	for one shaft.	304,000	for one pier.

Case 1. Stability of shaft at normal state.
effective width of shaft assumed 1.2 meters.
Max. load on one shaft = 152,000 kg
weight of shaft. (½) = 78,200
230,200 kg.

Unit bearing compression $= \frac{230200}{120 \times 185} = 10.4$ kg/cm² ok.



Case 2. Stability of shaft during Earthquake K=0.1 assumed.
Dead Load on one shaft = 92,000 kg
Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lever arm	Moment.
D		92,000	0	0
D'	9200		8.2	75400
S		78,200	0	0
S'	7800		3.93	30,650
	ΣH= 17000 kg	ΣV= 170,200 kg		ΣM= 106,050 kgm.

Eccentricity $e = \frac{106050}{170200} = .62$ m.

$e/h = \frac{.62}{1.85} = .335$, $d'/h = .06/1.85 = .032$

$p_0 = 2p = \frac{17 \times 2}{120 \times 185} = .00153$

From the prepared diagrams.
$k = .58$, $L = .098$

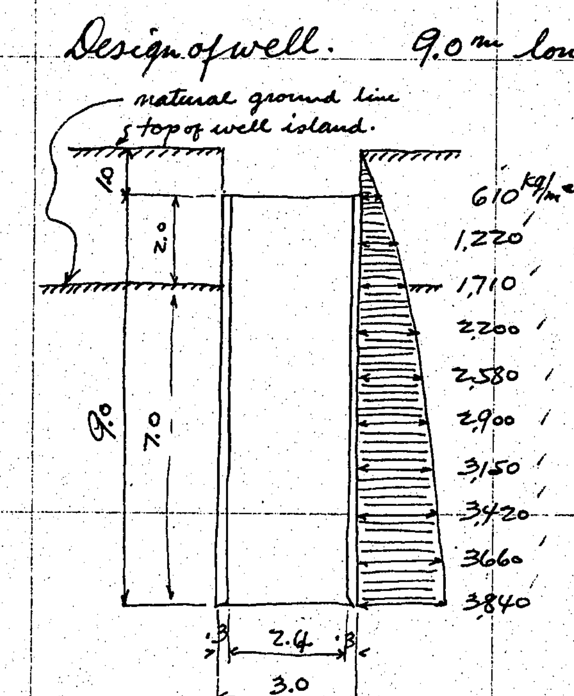
$f_c = \frac{106050 \times 100}{.098 \times 120 \times 185^2} = 26.4$ kg/cm² ok.

$f_s = 15 \times 26.4 \left(\frac{1.79}{1.85 \times .58} - 1\right) = 265$ kg/cm² ok

Unit shear $= \frac{17000}{120 \times \frac{7}{8} \times 179} = 0.9$ " ok

Unit bond $= \frac{17000}{5.97 \times 6 \times \frac{7}{8} \times 179} = 3.0$ " ok.

Design of well. 9.0 m long. Section same as for 7.0 m well previously discussed.



Temporary earth pressure on well see page 32
moment assumed $\frac{1}{12} wl^2$ on side wall, $\frac{1}{16} wl_1^2$ on circular ends.
where $l = 2.5$ m, $l_1 = 2.7$ m.
moment at several sections.

Section	Side pressure.	moment on side wall	moment on circular ends.
Top of well.	610 kg/m²	$\frac{1}{12}wl^2$ = 320 kgm	$\frac{1}{16}wl_1^2$ = 280 kgm.
1 m below top.	1,220	635	555
2	1,710	890	780
3	2,200	1,145	1,000
4	2,580	1,345	1,175
5	2,900	1,510	1,320
6	3,150	1,640	1,435
7	3,420	1,780	1,555
8	3,660	1,910	1,665
9. (bottom)	3,840	2,000	1,750

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CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

Horizontal Reinforcements of the well.

section at bottom.

moment on sidewall. = 2000 kgm

effective depth required $= \sqrt{\frac{M}{bR}}$ $R = 7.18$

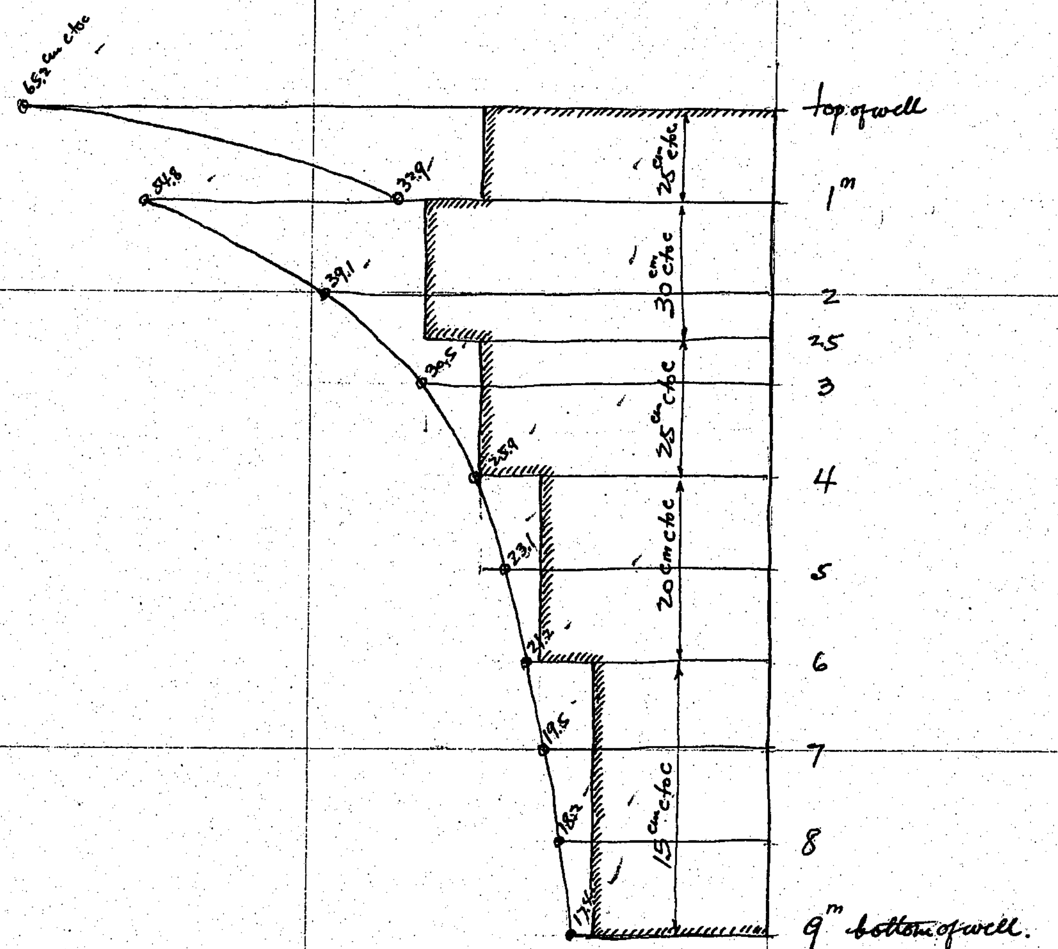
$= \sqrt{\frac{2000 \times 100}{100 \times 7.18}} = 16.7$ cm

Use 25 cm effective depth with 5 cm insulation, total depth of 30 cm.

steel area required $= \frac{2000 \times 100}{1200 \times \frac{7}{8} \times 25} = 7.62$ □cm per meter strip of sidewall.

Use 13 mm ϕ bars. spacing $= \frac{1.327}{7.62} = 17.4$ cm c to c.

Section between top to 8.0 m below, the moments same as for 7.0 m well.
use same details as for 7 m well previously discussed.

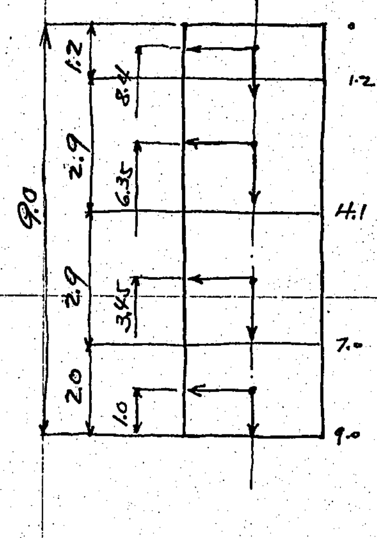


Spacing diagram for
Horizontal reinforcements of well.

For Details of Circular ends, use same reinforcements as for sidewall.

Stability of pier for Truss span.

Weight and center of gravity of 9.0 meter well.



total area of well. 29.12 □m

area of partition 19.45

area of shell net. 9.67 □m

Circumference of well = 24.12 m.

0 to 1.2 m from top. volume of concrete 29.12 × 1.2 = 34.95 m³ @ 2200 = 77,000 kg

1.2 to 4.1 m " " " 9.67 × 2.9 = 28.05 @ 2200 = 61,700

" sand 19.45 × 2.9 = 56.40 @ 1700 = 95800

157,500 kg

4.1 m to 7.0 m " same as above section 28.05 @ 2200 = 61,700

56.40 @ 1700 = 95800

157,500 kg

7.0 m to 9.0 m from top volume of concrete 29.12 × 2.0 = 58.24 @ 2200 = 128,000 kg

Total weight of well = 520,000 kg.

Total volume of concrete = 149.29 m³

" " " sand = 112.8 m³

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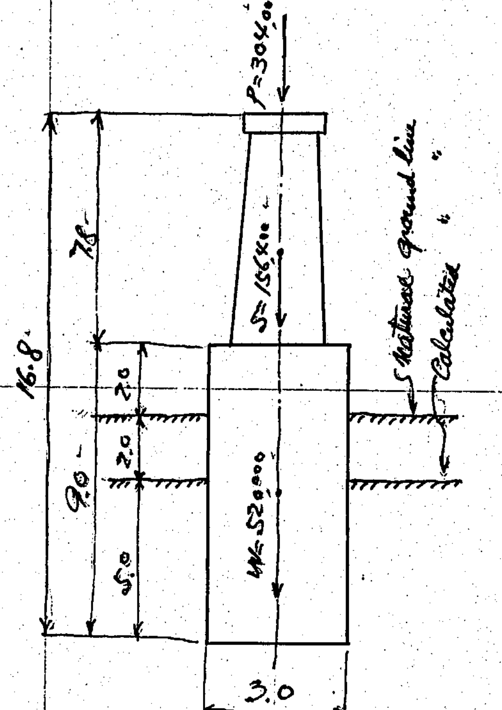
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken

Design of Pier for ℄ Truss span continued.
Center of gravity of well.

section	weight		lever arm		moment
0 – 1.2m from top.	77,000 kg	×	8.4 m	=	647,000
1.2 – 4.1	157,500	×	6.35	=	1,000,000
4.1 – 7.0	157,500	×	3.45	=	543,000
7.0 – 9.0	128,000	×	1.00	=	128,000
	520,000 kg	×	4.45 m	=	2318,000

Stability of Pier. ~~at~~
Case 1. Stability at normal state.

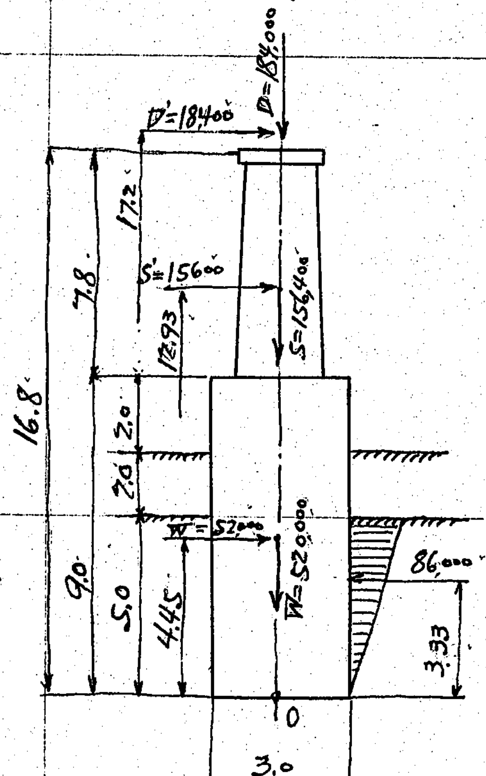


Effective penetration assumed 5.0m
Loads on well
Super imposed D.L.+L.L. = 304,000 kg
weight of shaft 156,400
" " well 520,000
980,400 kg

max. bearing pressure $= \frac{980400}{29.12} = 33750$ kg/m² or (3.08 tons/□′) ok

If frictional resistance of well surface be considered
frictional resistance 1000 kg/□m assumed.
Circumference of well = 24.12m effective penetration of well = 5.0m assumed.
Total frictional resistance = 1000 × 24.12 × 5 = 120,600 kg
Resulting bearing pressure = 980,400 − 120,600 = 859800
Unit bearing press. on foundation $= \frac{859800}{29.12} = 29500$ kg/m² or (2.7 tons/□′) ok

Case 2 Stability during earthquake, k = 0.1 assumed.



Taking moment ~~abut~~ about point 0

Loads	Hor. forces	Vert. forces		Lever arm		Moments
D		184,000	×	0	=	0
D′	18400		×	17.2	=	316,500
S		156,400	×	0	=	0
S′	15,600		×	12.93	=	201,700
W		520,000	×	0	=	0
W′	52,000		×	4.45	=	231,500
	ΣH = 86,000 kg	ΣV = 860,400				ΣM = 749,700 kgm

Earth reaction 86,000 — Call this 860,000
its moment = 86,000 × 3.33 = −286,500
463,200 kgm

Eccentricity $e = \frac{463200}{860000} = .538$ m

Resultant force out of middle third
Pressure area = 3(1.5 − .538) = 2.89 m × 9.35
max toe pressure $= \frac{860000 \times 2}{2.89 \times 9.35} = 63600$ kg/m² or (5.81 tons/□′) ok

If frictional couple be considered
frictional resistance (one side) = 1000 × 5 × 9.35 = 46,750 kg
frictional couple = 46750 × 3.0 = 140,300 kgm
Resulting moment = 463200 − 140300 = 322900 kgm
Eccentricity $e = \frac{322900}{860000} = .375$ m

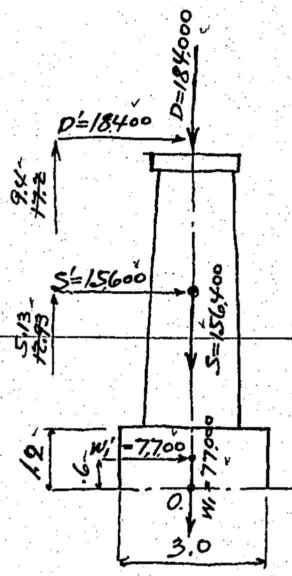
Resultant force within middle third.
max toe pressure $= \frac{860000}{9.35 \times 3.0}\left(1 \pm \frac{6 \times .375}{3.0}\right) = 53700$ kg/m² or (4.90 tons/□′)

max reactional pressure on earth 2m below ground surface
$= \frac{86000 \times 2}{9.35 \times 5} = 3680$ kg/m² < 9600 ok see page 34

Ashida Bashi for Okayama Ken.

Moment at several sections of well.

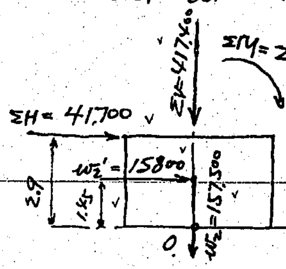
Moment at 1.2m below top of well.



Taking moment about point O.

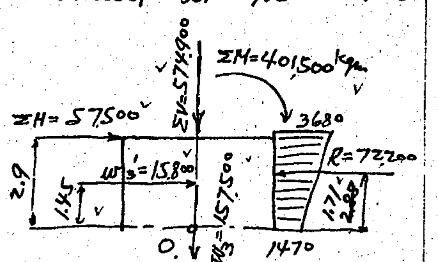
Loads	Horizontal forces	Vertical forces	lever arm	moment
D		184,000	× 0 =	0
D'	18,400		× 9.40 =	173,000
S		156,400	× 0 =	0
S'	15,600		× 5.13 =	80,000
w_1		77,000	× 0 =	0
w_1'	7,700		× .60 =	4,600
	ΣH = 41,700 kg	ΣV = 417,400 kg	× .617	ΣM = 257,600 kgm

Moment at 4.1m below top of well.



Loads	Horizontal forces	Vertical forces	lever arm	moment
ΣV		417,400	× 0 =	0
ΣH	41,700		× 2.9 =	121,000
ΣM			=	257,600
w_2		157,500	× 0 =	0
w_2'	15,800		× 1.45 =	22,900
	ΣH = 57,500 kg	ΣV = 574,900 kg	× .71m	ΣM = 401,500 kgm

Moment at 7.0m below top of well.



Loads	Horizontal forces	Vertical forces	lever arm	moment
ΣV		574,900	× 0 =	0
ΣH	57,500		× 2.9 =	166,700
ΣM			=	401,500
w_3		157,500	× 0 =	0
w_3'	15,800		× 1.45 =	22,900
R	−72,200		× 1.71 ~~2.85~~ =	−123,500
	ΣH = 1,100	ΣV = 732,400	× .64m	ΣM = 467,600 kgm

Moment of inertia of well shell = 11.49 m^4

Sectional area of 〃 〃 = 9.67 m^2

Section at 1.2m below top of well. Direct load = 417,400 ~~574,900~~ kg, moment = 257,600 ~~401,500~~ kgm.

moment stress = $\frac{257,600 \times 1.5}{11.49}$ = 33,650 kg/m² or 3.4 kg/cm² C or T.

direct stress = $\frac{417,400}{9.67}$ = 43,200 〃 or 4.3 〃 C

Total max fibre stress = 4.3 ± 3.4 = 7.7 kg/cm² C or 0.9 kg/cm² C.

No vertical reinforcement required theoritically but for the practice use 13mm bars 30cm c to c both sides.

Section at 4.1m below top of well. Direct load = 574,900 kg moment = 401,500 kgm

moment stress = $\frac{401,500 \times 1.5}{11.49 \times 10,000}$ = 5.2 ~~3.5~~ kg/cm² C or T

direct stress = $\frac{574,900}{9.67 \times 10,000}$ = 5.9 〃 C

Total max. fibre stress = 5.9 ± 5.2 ~~3.5~~ = 11.1 ~~9.4~~ kg/cm² C or 0.7 ~~2.4~~ kg/cm² C.

Use same vertical reinforcements as above.

Section at 7.0m below top of well. Direct load = 732,400 kg, moment = 467,600 kgm

moment stress = $\frac{467,600 \times 1.5}{11.49 \times 10,000}$ = 6.1 kg/cm² C or T.

direct stress = $\frac{732,400}{9.67 \times 10,000}$ = 7.6 〃 C.

Total max. fibre stress = 7.6 ± 6.1 = 13.7 kg/cm² C or 1.5 kg/cm² C.

Use same vertical reinforcements as above section. 13mm × 30cm c to c both sides.

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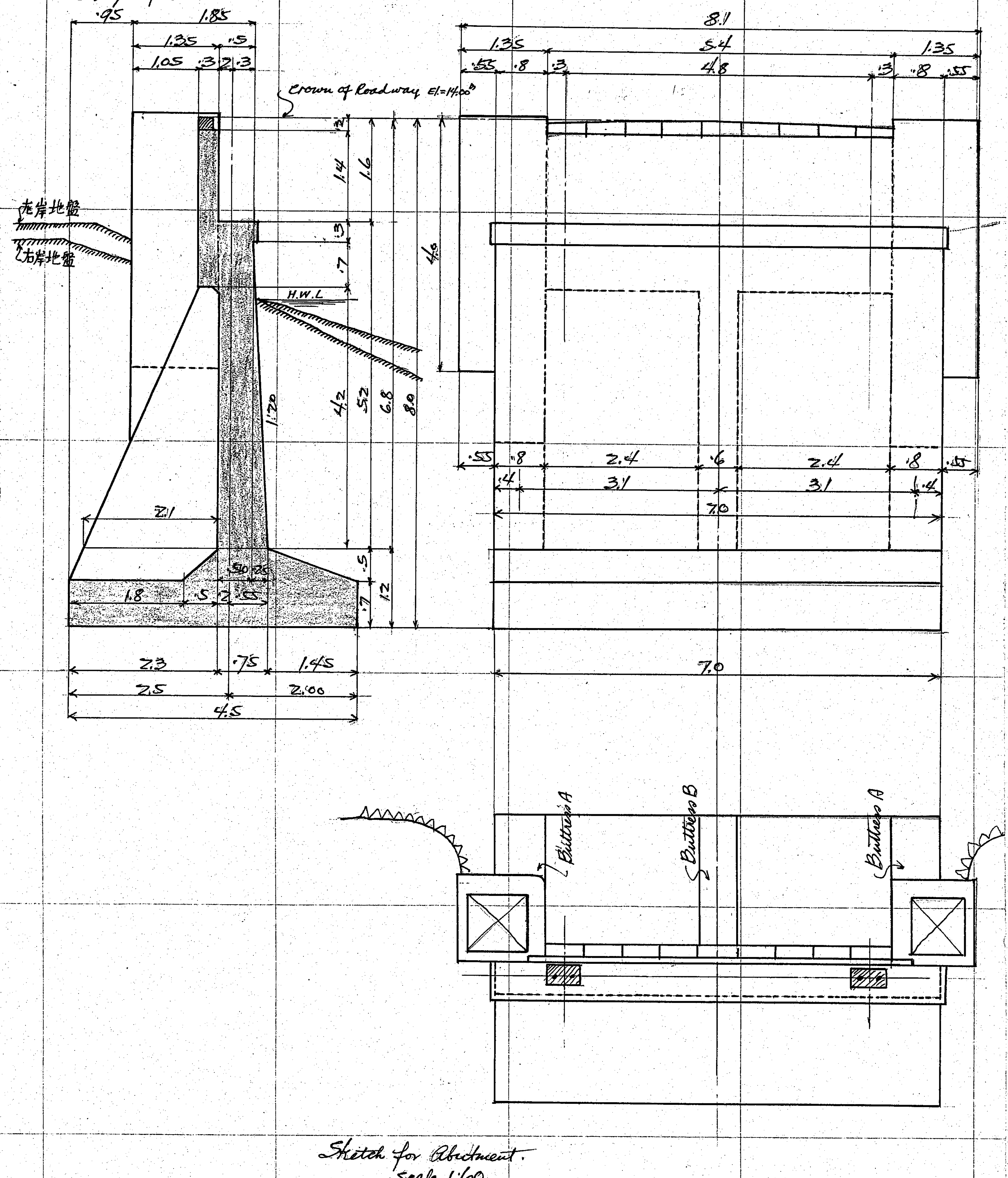
CALCULATIONS FOR

Ashida-Bashi for Okayama ken
Design of Abutment



Sketch for Abutment.
Scale 1:60.

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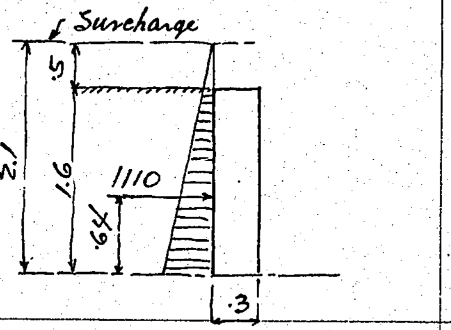
CALCULATIONS FOR

Ashida-Bashi for Okayamaken

Parapet wall of Abutment.

Case 1. Stability at normal state.



Surcharge due to live load = 0.5 m assumed.

Earth pressure on parapet wall = $\frac{1}{3} \times 1600 \times 0.5 = 267$

$\frac{1}{3} \times 1600 \times 2.1 = 1120$

$1387 \div 2 = 694$ kg/m²

Total pressure = $96 + 694 \times 1.6 = 1,110$ kg per lin meter of wall.

moment on parapet wall = $1110 \times .64 = 710$ kgm.

effective depth required = $\sqrt{\frac{710 \times 100}{100 \times 7.18}} = 10$ cm.

Use 27 cm effective depth with 3 cm insulation, total depth 30 cm.

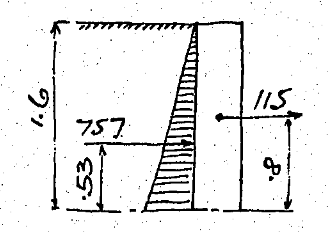
Steel area required = $\frac{710 \times 100}{1200 \times \frac{7}{8} \times 27} = 2.5$ □cm per meter strip of wall.

Use 13 mm φ bars at 30 cm c to c both sides = 4.42 □cm

Unit shear = $\frac{1110}{100 \times \frac{7}{8} \times 27} = .47$ kg/cm² ok

Unit bond = $\frac{1110}{4.08 \times 3.33 \times \frac{7}{8} \times 27} = 3.5$, ok.

Case 2. Stability of parapet wall during earthquake. k = 0.1 assumed.



Earth pressure during earthquake = $0.185\, wh^2 = .185 \times 1600 \times 1.6^2 = 757$

weight of parapet wall $.3 \times 1.6 = .48$ @ 2400 = 1,150 kg seismic force = 115 kg

moment $757 \times .53 = 400$

$115 \times .80 = 92$

872 kg — 492 kgm.

Assumed section is ample.

Design of Shaft.

Weight and center of gravity of shaft.

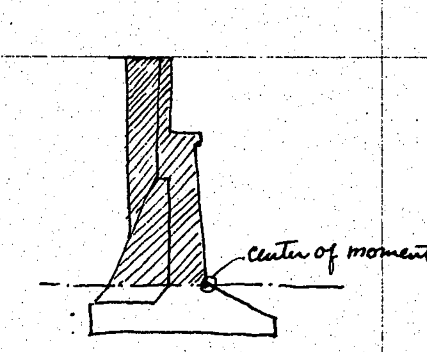
Buttress A. (both sides)

	section	length	reqd no	vol.	weight	hor. lev. arm	M_H	vert. lev. arm	M_V
Light pedestals.	.9 × .9	1.5	1	1.215 m³	@2400 = 2920 kg	1.5	4380	7.6	22,150
〃	.6 × .6	2.5	1	0.900	2,160	1.5	3,240	9.6	20,730
parapet wall.	.3 × 2.6	1.2	1	0.934	2,250	.9	2,030	5.5	12,370
Column.	1.35 × 1.35	2.65	1	4.830	11,600	1.43	16,600	5.5	63,800
〃	1.05 × .8	1.2	1	1.008	2,420	1.75	4,230	3.4	8,230
〃	1.35 × .55	1.4	1	1.040	2,490	1.43	3,560	3.5	8,720
front wall.	.625 × 2.0	5.2	1	6.500	15,600	.44	6,860	2.43	37,900
Coping projection	.3 × .1	2.6	1	0.078	190	.21	40	5.05	960
buttress	1.2 × .8	4.2	1	4.030	9,670	1.46	14,120	1.57	15,700
				20.535 cubm	49,300 kg	1.17	55,060 kgm	3.87	190,560 kgm
			for 2 buttresses	41.07 m³	98,600				

Buttress B. (center)

	section	length	reqd no	vol.	weight	hor. lev. arm	M_H	vert. lev. arm	M_V
parapet wall	.3 × 2.6	3.0	1	2.340	@2400 = 5,610	.9	5,050	5.5	30,900
front wall	.625 × 3.0	5.2	1	9.750	23,400	.44	10,300	2.43	56,800
Coping, projection	.3 × .1	3.0	1	.090	220	.20	40	5.05	1,110
buttress	1.2 × .6	4.2	1	3.020	7,250	1.46	10,580	1.57	11,390
				15.20 m³	36,480 kg	.71 m	25,970	2.75 m	100,200
				Call this	36,500 kg				

Total concrete for shaft = 41.07 + 15.20 = 56.27 m³ — weight 98,600 + 36,500 = 135,100 kg.



Center of gravity of whole shaft

		weight	hor. lev. arm	M_H	vert. lev. arm	M_V
Buttress	A	98,600 kg		110,120		381,120
〃	B	36,500		25,970		100,200
		135,100 kg	1.01 m	136,090	3.56 m	481,320

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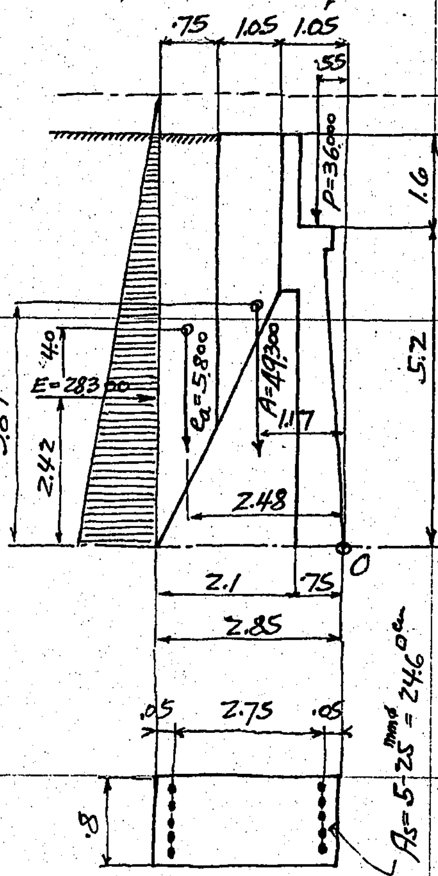
CALCULATIONS FOR

Ashida Bashi for Okayama Ken.

Stability of Shaft.

Buttress A. (End buttresses).

Case 1. Stability at normal state.



Super imposed load on abutment. D.L. = 18000
L.L. = 18000
36000 kg for one buttress A.

weight of earth on buttress. .75×.8×6 @1600 = 5,800 kg

surcharge due to live load assumed 0.5 m

earth pressure on wall. top 1/3×1600×.5 = 267
bottom 1/3×1600×7.3 = 3890
4157÷2 = 2079 kg/m² average.

Total pressure on buttress A 2079×6.8×2.0 = 28,300 kg.

Taking moment about toe O.

Loads	Hor. forces	Vert. forces		lever arm		moment
P.		36000	×	.55	=	19800
A.		49300	×	1.17	=	57,700
e_a		5800	×	2.48	=	14,400
E	28,300		×	−2.42	=	−68,500
	ΣH = 28,300 kg	ΣV = 91,100 kg		.23 m		ΣM = 20,900 kgm

eccentricity $e = \frac{2.85}{2} - .23 = 1.195^m$ right

Design buttress as a rectangular wall of .8 × 2.85 m at bottom.

moment on buttress = 91,100×1.195 = 109,000 kgm.

$e/h = 1.195/2.85 = .42$, $d'/h = .05/2.85 = .0175$

$p_0 = 2p = \frac{2 \times 24.6}{80 \times 285} = .00215$

$K = .50$, $L = .097$

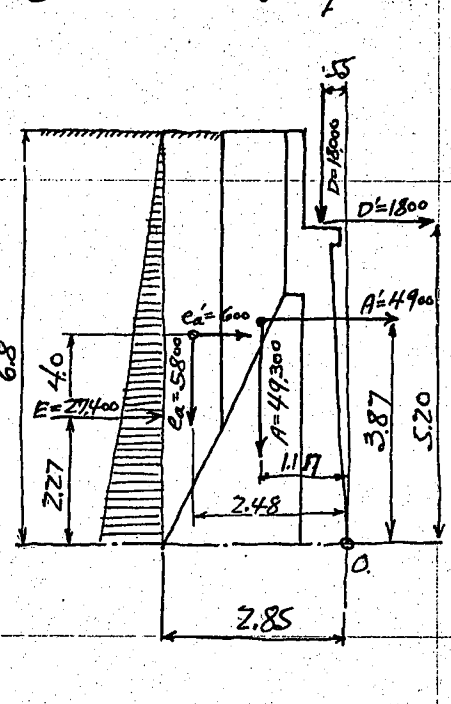
$f_c = \frac{109,000 \times 100}{.097 \times 80 \times 285^2} = 17.3\ kg/cm^2$ ok

$f_s = 15 \times 17.3 \left(\frac{280}{.5 \times 285} - 1\right) = 250\ kg/cm^2$ ok

Unit shear $= \frac{28,300}{80 \times \frac{7}{8} \times 280} = 1.45$ ok

Unit bond $= \frac{28,300}{7.85 \times 5 \times \frac{7}{8} \times 280} = 2.95$ ok.

Case 2. Stability during earthquake. K = 0.1 assumed.



Earth pressure during earthquake = .185×1600×6.8²×2.0 = 27,400 kg

Loads	Hor. forces	Vert. forces		lever arm		moment
D		18,000	×	.55	=	−9900
D'	1,800		×	5.20	=	9360
A		49,300	×	1.17	=	−57,700
A'	4,900		×	3.87	=	19,000
e_a		5800	×	2.48	=	−14,400
e_a'	600		×	4.00	=	2,400
E	27,400		×	2.27	=	62,200
	ΣH = 34,700 kg	ΣV = 73,100 kg		0.15		ΣM = 10,960 kgm

Eccentricity $e = \frac{2.85}{2} + 0.15 = 1.28^m$ left.

Moment on buttress = 73100×1.28 = 93600 kgm.

$e/h = 1.28/2.85 = .45$, $d'/h = .0175$, $p_0 = .00215$

$K = .43$, $L = .092$

$f_c = \frac{93600 \times 100}{.092 \times 80 \times 285^2} = 15.7\ kg/cm^2$ ok

$f_s = 15 \times 15.7 \left(\frac{280}{.43 \times 285} - 1\right) = 303\ kg/cm^2$ ok

Unit shear $= \frac{34700}{80 \times \frac{7}{8} \times 280} = 1.8\ kg/cm^2$ ok

Unit bond $= \frac{34,700}{7.85 \times 5 \times \frac{7}{8} \times 280} = 3.6$ ok.

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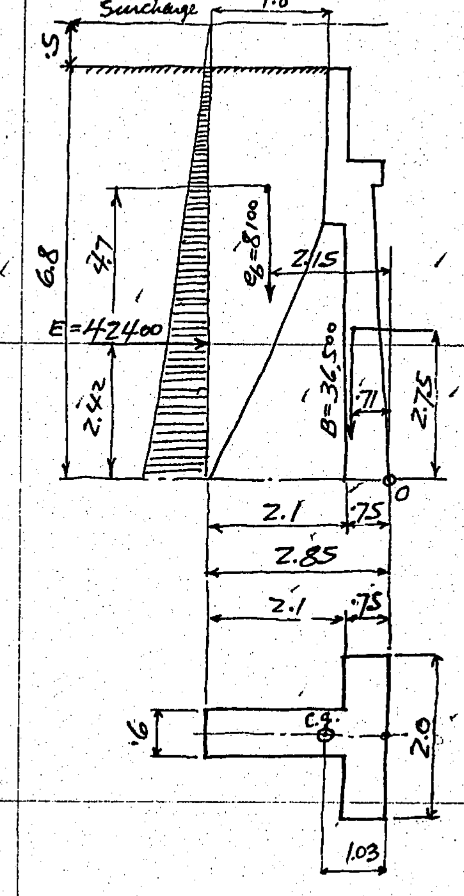
CALCULATIONS FOR

Ashida Bashi for Okayama Ken.

Buttress B (center).

Case 1. Stability at normal state.



Weight of earth on buttress $1.8 \times 6 \times 4.7 @ 1600 = 8100^{kg}$

Surcharge due to live load assumed 0.5^m.

Earth pressure on wall. top $\frac{1}{3} \times 1600 \times .5 = 267$

bottom $\frac{1}{3} \times 1600 \times 7.3 = 3890$

$4157 \div 2 = 2079^{kg/m^2}$ average.

Total pressure on buttress B $= 2079 \times 6.8 \times 3.0 = 42400$ kg

Taking moment about toe O.

Loads	Hor. forces	Vert. forces		lever arm		moment
B		36,500	×	.71	=	25,900
e_b		8,100	×	2.15	=	17,400
E	42,400		×	2.42	=	−102,600
	ΣH = 42,400 kg	ΣV = 44,600 kg	×	−1.33		ΣM = −59,300 kgm

Design Buttress as a wall of T-section.

Center of gravity of bottom section.

flange $.75 \times 2.0 = 1.5 \times .375 = .562$

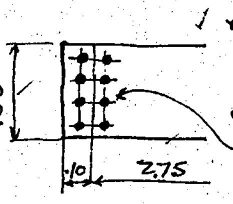
web. $.60 \times 2.1 = 1.26 \times 1.80 = 2.270$

$2.76^{\square m} \times 1.03 \quad 2.832$

Eccentricity $= 1.03 + 1.33 = 2.36^m$

moment on Buttress $= 44{,}600 \times 2.36 = 105{,}300$ kgm.

direct compression $= \dfrac{44600}{2.76 \times 10{,}000} = 1.6$ kg/cm²



Stress due to moment.

Steel area req'd. $= \dfrac{105300 \times 100}{1200 \times \frac{7}{8} \times 275} = 36.5^{\square cm}$.

Use 8-25 mmφ bars $= 39.3\ \square$cm.

Steel ratio $p = \dfrac{39.3}{200 \times 275} = 0.00072$

$t/d = \dfrac{75}{275} = 0.27$, neutral axis in the flange.

$k = .145$, $j = .943$

$f_s = \dfrac{M}{A_s j d} = \dfrac{105300 \times 100}{39.3 \times .943 \times 275} = 1030$ kg/cm²

$f_c = \dfrac{f_s k}{n(1-k)} = \dfrac{1030 \times .145}{15(1-.145)} = 11.65$ kg/cm²

unit shear $= \dfrac{42400}{60 \times .943 \times 275} = 2.7$ kg/cm² ok

Unit bond $= \dfrac{42400}{7.85 \times 8 \times .943 \times 275} = 2.6$ " ok

Combined stress.

direct compression on concrete $= 1.6$ kg/cm²

bending " $= 11.65$

Total $f_c = 13.25$ kg/cm² ok

direct compression on steel $= 1.6 \times 15 = 24.0$ kg/cm²

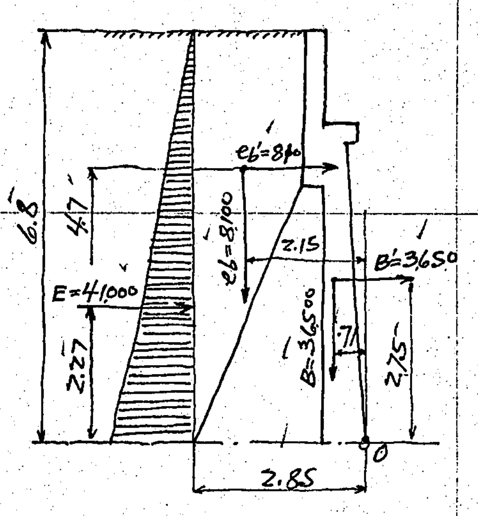
bending stress, tension " $= 1{,}030.0$

$1{,}006$ kg/cm² ok

Case 2. Stability during earthquake $k = 0.1$ assumed.

Earth pressure during earthquake. $= .185 \times 1600 \times 6.8^2 \times 3.0 = 41{,}000$ kg

Taking moment about point O.



Loads	Hor. forces	Vert. forces		Lever arm		Moments
B		36,500	×	.71	=	−25,900
B'	3,650		×	2.75	=	10,000
e_b		8100	×	2.15	=	−17,400
e_b'	810		×	4.70	=	3,800
E	41000		×	2.27	=	93,200
	ΣH = 45,460 kg	ΣV = 44,600 kg	×	1.42		ΣM = 63,700 kgm

Eccentricity $e = 1.03 + 1.42 = 2.45^m$.

moment on buttress $= 44{,}600 \times 2.45 = 109{,}000$ kgm

Assumed section is ample.

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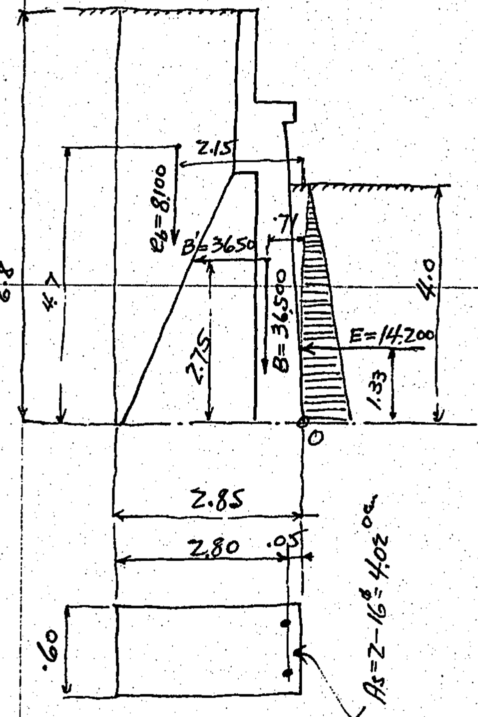
Case 3. Stability during earthquake. k=0.1 assumed.



Earth pressure during earthquake. = ·185 × 1600 × 4.0² ÷ 3.0 = 14,200 kg

Taking moment about point O.

Loads	Hor. forces	Vert. forces	Lever arm	moments.
B		36,500	.71	25,900
B'	3,650		2.75	10,000
e_b		8,100	2.15	17,400
E	14,200		1.33	18,900
	ΣH = 17,850 kg	ΣV = 44,600 kg	1.61 m	ΣM = 72,200 kgm

Eccentricity $e = -\frac{2.85}{2} + 1.61 = .18$ m

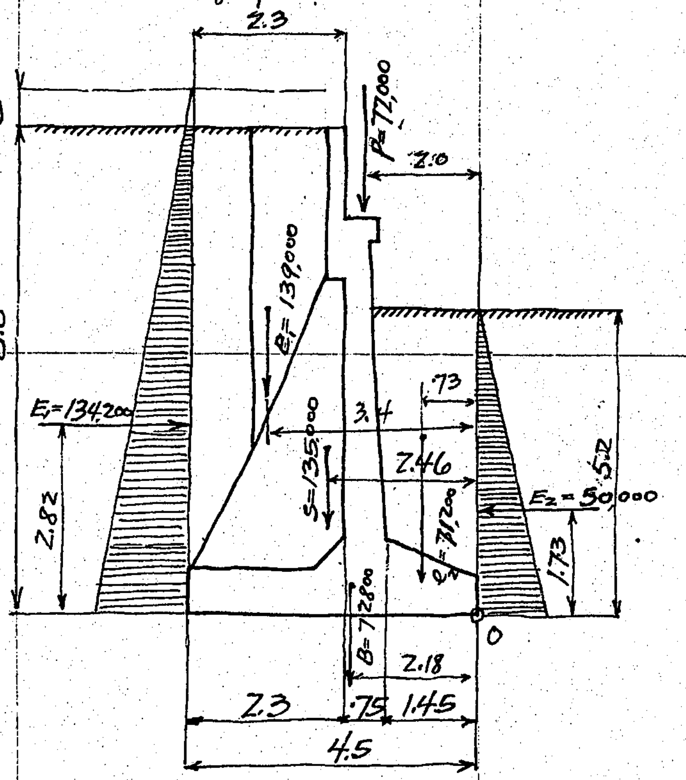
Moment on buttress = 44,600 × .18 = 8,050 kgm.

steel area req'd for moment $= \frac{8050 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 280} = 1.52$ □cm

Use 2-16 mmφ bars = 2.01 × 2 = 4.02 □cm.

Case 1.

Stability of Abutment at normal state. Weight of base.



				MN		MH
1.73 · .5 × 7.0 = 6.05 cu m @ 2400 =	14,500 kg	.9	13,000	1.5	21,800
0.7 · 4.5 × 7.0 = 22.05 @ " =	53,000	.35	18,500	2.25	119,000
2.2 · .5 · 2.0 = 2.20 @ " =	5,300	.95	5,040	3.4	18,000
30.30 m³	72,800 kg	.50	36,540	2.18	158,800

Earth pressure rear side top. ⅓ · 1600 · .5 = 267

bottom ⅓ · 1600 · 8.5 = 4,533

4,800 ÷ 2 = 2400 kg/m² average.

total earth pressure E₁ = 2400 · 8 · 7 = 134,200 kg

Earth pressure front side $E_2 = \frac{1}{6} \cdot 1600 \cdot 5.2^2 \cdot 7 = 50,000$ kg

Earth fill, rear side e_1 = 2.2 · 7.3 · 5.4 @ ~~2400~~ 1600 = ~~8700~~ 139,000 kg

" front side e_2 = 1.5 · 4.25 · 7.0 @ 1600 = 71,200 kg

Taking moment about O.

Loads	Hor. forces	Vert. forces	Lever arm	Moments.
P		72,000	2.00	144,000
S		135,000	2.46	332,000
B		72,800	2.18	159,000
e_1		139,000	3.40	473,000
e_2		71,200	.73	52,000
E₁	−134,200		2.82	−379,000
E₂	50,000		1.73	86,500
	ΣH = −84,200 kg	ΣV = 490,000 kg	1.77 m	ΣM = 867,500

Eccentricity = 2.25 − 1.77 = .48 m right.

Resultant force within middle third.

max. toe pressure $= \frac{490000}{7.0 \times 4.5}\left(1 \pm \frac{6 \times .48}{4.5}\right) = 25500$ kg/m² c (2.32 tons/□') ok

or = 5,600 " c

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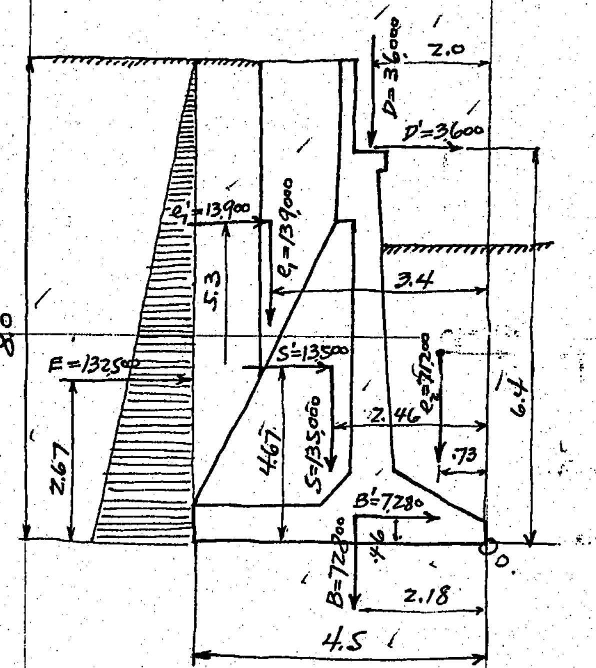
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

Case 2. Stability of Abutment ~~at~~ during earthquake. $K = 0.1$ assumed.

Earth pressure during earthquake. $0.185 \times 1600 \times 8.0^2 \times 7.0 = 132{,}500$ kg

Taking moment about point O.



Loads.	Hor. forces.	Vert. forces		Lever arm		moment.
D		36,000	×	2.00	=	72,000
D'	3,600		×	6.40	=	− 23,000
S		135,000	×	2.46	=	332,000
S'	13,500		×	4.67	=	− 63,000
B		72,800	×	2.18	=	159,000
B'	7,280		×	0.46	=	− 3,400
e_1		139,000	×	3.40	=	473,000
e_1'	13,900		×	5.30	=	− 73,700
e_2		71,200	×	.73	=	52,000
E	132,500		×	2.67	=	− 353,800
	ΣH = 170,780 kg	ΣV = 454,000 kg	×	1.26		ΣM = 571,100 kgm.

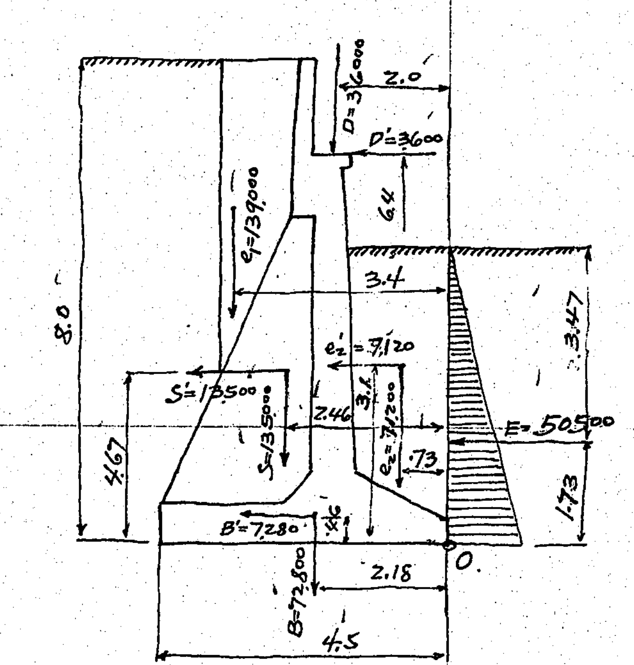
Eccentricity = 2.25 − 1.26 = 0.99 m right.

Resultant force out of middle third. pressure area = $1.26 \times 3 \times 7 = 26.5$ ^{cm}

max. toe pressure = $\frac{454000 \times 2}{26.5}$ = 34,200 kg/m² or (3.13 tons/○) OK.

Case 3. Stability of Abutment during earthquake seismic forces reversal to case 2.

Taking moment about point O.



Loads	Hor. forces	Vert. forces		Lever arm		moment
D		36,000	×	2.00	=	72,000
D'	3,600		×	6.40	=	23,000
S		135,000	×	2.46	=	332,000
S'	13,500		×	4.67	=	63,000
B		72800	×	2.18	=	159,000
B'	7,280		×	.46	=	3,400
e_1		139,000	×	3.40	=	473,000
e_2		71,200	×	.73	=	52,000
e_2'	7,120		×	3.10	=	22,100
E	50,500		×	1.73	=	87,400
	ΣH = 87,000	ΣV = 454,000		2.84 m		ΣM = 1,286,900 kgm.

Eccentricity = 2.84 − 2.25 = 0.59 m left.

Resultant force within middle third

max. toe pressure = $\frac{454000}{7.0 \times 4.5}\left(1 \pm \frac{6 \times .59}{4.5}\right)$ = 25800 kg/m² or (2.36 tons/○) OK
3180 " c

Earth pressure during earthquake
= $\frac{1}{6} \times 1600 \times 5.2^2 \times 7 = 50{,}500$ kg

Design of Cantilever footing.

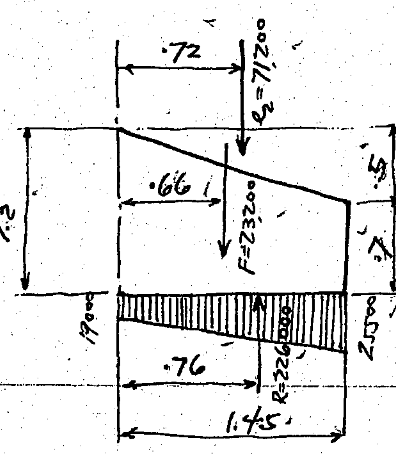
Cantilever footing at toe.

Upward pressure (Case 1) 25,500 kg/m² at toe.

19000 at fixed point

44,500 ÷ 2 = 22,250

22,250 × 1.45 × 7.0 = 226,000 kg = R



downward pressure.

weight of footing ~~.85~~ $.95 \times 1.45 \times 7.0 \times 2400$ = 23,200 kg = F

weight of earth fill 71,200 kg = e_2

moment

226,000 × .76 = + 171,800 kgm.

− 23,200 × .66 = − 15,300

− 71,200 × .72 = − 51,200

131,600 kg ÷ 7 = 18800 kg/m 105,300 kgm ÷ 7 = 15,050 kgm / meter strip.

Steel area required = $\frac{15050 \times 100}{1200 \times \frac{7}{8} \times 115}$ = 12.5 cm / meter strip.

Use 22 mm ϕ bars 20 cm ctoc = 19.0 cm

unit shear = $\frac{18800}{100 \times \frac{7}{8} \times 115}$ = 1.87 kg/cm² OK

unit bond = $\frac{18800}{5 \times 6.91 \times \frac{7}{8} \times 115}$ = 5.4 kg/cm² OK. max spacing $20 \times \frac{6}{5.4} = 22.2$ cm ctoc.

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CONSULTING ENGINEER
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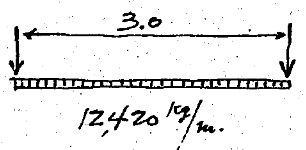
CALCULATIONS FOR

Ashida-Bashi for Okayama ken.

Footing at heel.



Span length = 3.0m say

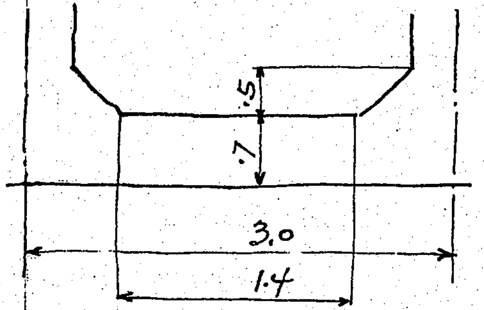
Upward pressure Case 3. during earthquake 25800 kg/m²
downward wt. of earth fill 7.3 × 1600 = −11,700
wt. of footing 0.7 × 2400 = − 1,680
12420 kg/m² upward.

moment = $\frac{12420 \times 3.0^2}{10}$ = 11180 kgm / meter strip.

steel area required = $\frac{11,180 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 65}$ = 9.1 □cm.

Use 19φ bars at 30cm c to c = 9.45 □cm / meter strip

$p = \frac{9.1}{100 \times 65} = 0.0014$, k = .181, j = .94



$f_s = \frac{11180 \times 100}{9.45 \times .94 \times 65}$ = 1940 kg/cm² < 2160 ok

$f_c = \frac{1940 \times .181}{15(1-.181)}$ = 28.6 kg/cm² ok

End shear = 12420 × 1.5 = 18,600 kg

Unit shear = $\frac{18600}{100 \times .94 \times 115}$ = 1.72 kg/cm² ok

~~Unit bond = $\frac{18,600}{5.97 \times 3.33 \times .94 \times 115}$ = 8.65 kg/cm² < 10.8 ok~~

Shear at end of chamfer = 12,420 × 0.7 = 8,700 kg

Unit shear = $\frac{8700}{100 \times .94 \times 65}$ = 1.43 kg/cm² ok

Unit bond = $\frac{8700}{5.97 \times 3.33 \times .94 \times 65}$ = 7.16 kg/cm² ok.

Design of curtain wall. Span length = 3.0m say
Earth pressure on wall. top of wall. $\frac{1}{3}$ × 1600 × 2.6 = 1390 kg/m²
bottom $\frac{1}{3}$ × 1600 × 6.8 = 3620

moment at bottom = $\frac{3620 \times 3.0^2}{10}$ = 3260 kgm per meter strip.
end shear = 3620 × 1.5 = 5430 kg

Steel req'd = $\frac{3260 \times 100}{1200 \times \frac{7}{8} \times 70}$ = 4.43 □cm / meter strip

Use 16mmφ bars at 30cm c to c = 6.7 □cm at bottom

unit shear = $\frac{5430}{100 \times \frac{7}{8} \times 70}$ = .9 kg/cm² ok

unit bond = $\frac{5430}{5.03 \times 3.33 \times \frac{7}{8} \times 70}$ = 5.3 " ok.

moment at top = $\frac{1930 \times 3.0^2}{10}$ = 1740 kgm / meter strip

end shear = 1930 × 1.5 = 2900 kg

Steel req'd = $\frac{1740 \times 100}{1200 \times \frac{7}{8} \times 50}$ = 3.3 □cm / meter strip

use 16mmφ bars at 50cm c to c = 4.02 □cm

unit shear = $\frac{2900}{100 \times \frac{7}{8} \times 50}$ = .7 kg/cm² ok

unit bond = $\frac{2900}{5.03 \times 2 \times \frac{7}{8} \times 50}$ = 6.58 kg/cm² over

spacing for unit bond of 6 kg/cm² = 30 × $\frac{6}{6.58}$ = 45.5 cm c to c use 45cm.

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CALCULATIONS FOR

Material list of Ashida-Bashi for Okayama-Ken.

NO.	Materials	Length	Weight kg/m.	Total wt.	Remarks
		End POST Lo-U1	4-Req'd.		
1	Cov. Pl. 550 ×	9 × 5,085	@ 38.86	197.6	
2	[. 250 × 90	× 5,340	@ 34.59	369.4	
4	Ls. 90 × 75 ×	9 × 230	" 11.02	10.1	
1	Tie Pl. 550 ×	9 × 595	" 38.86	23.1	
1	" 520 ×	9 × 550	" 36.74	20.2	
12	Lac. bars 60 ×	10 × 723	" 4.71	40.9	
1	bent Pl. 490 ×	9 × 550	" 34.62	19.0	
2	" 85 ×	10 × 300	" 6.67	4.0	
2	Pls. 300 ×	9 × 570	" 21.20	24.2	
2	" 700 ×	9 × 1,200	" 49.46	118.7	
2	Ls 100 × 100 ×	10 × 390	" 14.91	11.6	
2	" "	× 910	" "	27.1	
2	fillers 95 ×	9 × 510	" 6.71	6.8	
				872.7 × 4 = 3,490.8	
		Top CHORD U1 - U3	4-Req'd.		
1	Cov. Pl. 550 ×	9 × 7,550	@ 38.86	293.4	
2	[. 250 × 90	× 7,545	" 34.59	522.0	
1	Tie Pl. 520 ×	9 × 550	" 36.74	20.2	
2	" 370 ×	9 × 550	" 26.14	28.8	
1	" 550 ×	9 × 595	" 38.86	23.1	
16	Lac. bars 60 ×	10 × 723	" 4.71	54.5	
1	Pl. 550 ×	9 × 750	" 38.86	29.1	
2	fillers. 190 ×	9 × 310	" 13.42	8.3	
2	Pls. 85 ×	13 × 750	" 8.67	13.0	
2	" 190 ×	9 × 475	" 13.42	12.7	
2	" 625 ×	9 × 1,095	" 44.16	96.7	
2	Ls. 100 × 100 ×	10 × 390	" 14.91	11.6	
				1113.4 × 4 = 4453.6	
		U3-U5 & U3-U5A	4-Req'd.		
1	Cov. Pl. 550 ×	9 × 7,510	@ 38.86	291.8	
2	[. 250 × 90	× 7,510	" 34.59	519.5	
2	Tie Pls. 520 ×	9 × 550	" 36.74	40.4	
2	" 295 ×	9 × 550	" 20.84	22.9	
20	Lac. bars. 60 ×	10 × 723	" 4.71	68.1	
2	pls 190 ×	9 × 7,510	" 13.42	201.6	
				1,144.3 × 4 = 4,577.2	
		at U5	2-Req'd.		
1	Pl. 550 ×	9 × 750	@ 38.86	29.1	
2	Pls. 85 ×	13 × 750	" 8.67	13.0	
2	" 190 ×	9 × 470	" 13.42	12.6	
2	" 550 ×	9 × 995	" 38.86	77.3	
				132.0 × 2 = 264.0	
		Summary of Top Chords.		12,785.6	

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CALCULATIONS FOR

Material list of Ashida-Bashi for Okayama-Ken.

NO.	Materials	Length.	Weight/m.	Total weight
		BOTTOM CHORD L0-L2	4-Req'd.	
2	[. 250 × 90	× 6,380	@ 34.59	441.4
1	L. 90 × 75 × 9	× 470	" 11.02	5.2
1	" "	× 250	" "	2.8
1	Bent Pl. 550 × 9	× 677	" 38.86	26.2
2	fillers 190 × 9	× 295	" 13.42	7.9
1	" 265 × 9	× 475	" 18.72	8.9
4	Ls. 90 × 75 × 9	× 760	" 11.02	33.5
1	L. "	× 300	" "	3.3
1	Pl. 310 × 8	× 760	" 19.47	14.8
1	Washer 60$^{\phi}$ × 9	×	" 0.20	0.2
2	Pls. 945 × 9	× 950	" 66.76	126.8
2	Pin Pls. 480 × 9	× 650	" 33.91	44.1
2	" 340 × 9	× 345	" 24.02	16.6
1	L. 90 × 90 × 10	× 275	" 13.34	3.7
1	Pl. 420 × 9	× 555	" 29.67	16.5
2	Tie Pls. 330 × 8	× 330	" 20.72	13.7
10	" 200 × 8	× 330	" 12.56	41.4
2	Pls. 340 × 9	× 675	" 24.02	32.4
4	Ls. 90 × 60 × 9 ($3\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{3}{8}$)	× 190	" 9.96	7.6
1	Pl. 190 × 8	× 310	" 11.93	3.7
2	Ls. 100 × 100 × 10	× 150	" 14.91	4.5
1	Pl. 720 × 9	× 780	" 50.87	39.7
2	" 250 × 9	× 505	" 17.66	17.8
2	" 300 × 9	× 445	" 21.20	18.9
				931.6 × 4 = 3726.4
		L2 - L4	4-Req'd.	
2	[. 250 × 90	× 7410	@ 34.59	512.6
2	Pls. 250 × 12	× 7,410	" 23.55	349.0
2	" 690 × 9	× 1,090	" 48.75	106.3
8	Ls. 90 × 60 × 9 ($3\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{3}{8}$)	190	" 9.96	15.1
2	Pls. 190 × 8	× 290	" 11.93	6.9
4	Ls. 100 × 100 × 10	× 130	" 14.91	7.8
1	Tie Pl. 305 × 8	× 390	" 19.15	7.5
10	" 200 × 8	× 305	" 12.56	38.3
1	Pl. 705 × 9	× 770	" 49.81	38.4
1	" 620 × 9	× 690	" 43.80	30.2
2	Pls. 250 × 12	× 865	" 23.55	40.7
2	" 190 × 16	× 865	" 23.86	41.3
2	" 310 × 9	× 445	" 21.90	19.5
2	" 460 × 9	× 675	" 32.50	43.9
				1257.5 × 4 = 5030.0
		L4 - L4'	2-Req'd.	
2.	[. 250 × 90	× 9270	@ 34.59	641.3
2	Pls. 190 × 9	× 9,270	" 13.42	248.8
2	Pls. 250 × 12	× 9,270	" 23.55	436.6
4	" 670 × 9	× 900	" 47.34	170.4
12	Ls. 90 × 60 × 9 ($3\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{3}{8}$)	190.	" 9.96	22.7
3	Pls. 190 × 8	270	" 11.93	9.7
6	Ls. 100 × 100 × 10	130	" 14.91	11.6

CALCULATIONS FOR

Material list of Ashida-Bashi for Okayama-Ken.

No.	Materials				Length	Weight kg/m	Total Weight
12	Tie Pls.	200	×	8	305	@ 12.56	46.0 ✓
3	Pl.	620	×	9	690	〃 43.80	90.7 ✓
2	〃	460	×	9	675	〃 32.50	43.9 ✓
							1721.7 ✓ × 2 = 3443.4 ✓
						Summary of Bottom chords	12199.8 ✓
					VERTICALS L_1-U_1	4-Req'd.	
2	Ls.	90 × 75	×	9	3,530	@ 11.02	77.8 ✓
2	〃		〃		3,620	〃 〃	79.8 ✓
1	Pl.	325	×	8	470	〃 20.41	9.6 ✓
1	〃		〃		460	〃 〃	9.4 ✓
12	Lac. bars	55	×	8	348	〃 3.45	14.4 ✓
							191.0 ✓ × 4 = 764.0 ✓
					L_2-U_2 & L_4-U_4	8-Req'd.	
4	Ls.	90 × 75	×	9	3,913	@ 11.02	172.5 ✓
1	Pl.	325	×	8	470	〃 20.41	9.6 ✓
1	〃		〃		313	〃 〃	6.4 ✓
14	Lac. bars	55	×	8	348	〃 3.45	16.8 ✓
2	Pls.	190	×	9	313	〃 13.42	8.4 ✓
							213.7 ✓ × 8 = 1709.6 ✓
					L_3-U_3 & L_5-U_5	6-Req'd.	
4	Ls.	90 × 75	×	9	3,913	@ 11.02	172.5 ✓
1	Pl.	325	×	8	470	〃 20.41	9.6 ✓
1	〃		〃		313	〃 〃	6.4 ✓
14	Lac. bars	55	×	8	348	〃 3.45	16.8 ✓
							205.3 ✓ × 6 = 1,231.8 ✓
						Summary of Verticals	3705.4 ✓
					DIAGONALS U_1-L_2	4-Req'd.	
2	Ls.	125 × 90	×	10	4,910	@ 16.09	158.0 ✓
2	Ls.		〃		5,010	〃 〃	161.2 ✓
2	Pls.	325	×	8	370	〃 20.41	15.1 ✓
19	Lac. bars	55	×	8	326	〃 3.45	21.4 ✓
							355.7 ✓ × 4 = 1422.8 ✓
					L_2-U_3	4-Req'd.	
2	Ls.	125 × 75	×	10	4,910	@ 14.91	146.4 ✓
2	〃		〃		5,010	〃 〃	149.4 ✓
2	Pl.	325	×	8	370	〃 20.41	15.1 ✓
18	Lac. bars	55	×	8	348	〃 3.45	21.6 ✓
							332.5 ✓ × 4 = 1330.0

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CALCULATIONS FOR

Material list of Ashida Bashi for Okayama-Ken.

NO.	Materials	Length	Weight Kg/m.	Total weight.
		U3-L4 & L4-U5	8-Req'd.	
2	Ls. 100 × 75 × 10	4.910	@ 12.95	127.2
2	〃 〃	4.960	〃 〃	128.5
2	Ps 310 × 8	325	〃 19.47	12.7
19	Lacters 55 × 8	348	〃 3.45	22.8
				291.2 × 8 = 2329.6
		Summary of Diagonals		5082.4
		FLOOR BEAM	FB1 & FB1A 2 Req'd.	
4	Flg. Ls. 100 × 90 × 10	6.375	@ 14.13	360.3
1	P. 670 × 8	6.375	〃 42.08	268.3
4	Ps. 210 × 10	230	〃 16.49	15.2
4	Ls. 125 × 75 × 10	660	〃 14.91	39.4
4	filler 125 × 10	495	〃 9.81	19.4
14	Ls. 75 × 75 × 9	680	〃 9.96	94.8
				797.4 × 2 = 1594.8
		FB2, FB3, FB4 & FB5	9-Req'd.	
4	Flg. Ls 125 × 90 × 10	6.390	@ 16.09	411.3
1	P. 670 × 8	6.390	〃 42.08	268.9
4	〃 230 × 10	260	〃 18.06	18.8
4	Ls. 125 × 75 × 10	660	〃 14.91	39.4
4	fillers 125 × 10	495	〃 9.81	19.4
14	Ls. 75 × 75 × 9	680	〃 9.96	94.8
				852.6 × 9 = 7,673.4
		BRACKET BT1	10-Req'd.	
2	Ls. 90 × 75 × 9	1.370	@ 11.02	30.2
1	P. 195 × 8	1.440	〃 12.25	17.6
2	Ls. 75 × 75 × 9	650	〃 9.96	12.9
1	P. 330 × 8	470	〃 20.72	9.7
2	Ps. 145 × 8	215	〃 9.11	3.9
2	Ls. 100 × 75 × 10	670	〃 12.95	17.4
				91.7 × 10 = 917.0
		Summary of floor beams		10,185.2
		BOTTOM LATERAL BRACINGS	BL1, BL2 & BL3 4-Req'd.	
1	L. 125 × 75 × 10	6.990	@ 14.91	104.2
1	〃 〃	3.370	〃 〃	50.2
1	〃 〃	3.410	〃 〃	50.8
1	P. 305 × 9	860	〃 21.55	18.5
2	Ls. 75 × 75 × 9	655	〃 9.96	13.0
2	Ps. 140 × 9	140	〃 9.89	2.8
				239.5 × 4 = 958.0

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CALCULATIONS FOR

Material list of Ashida Bashi for Okayama-Ken.

NO.	Materials.	Length.	Weight kg/m.	Total wt.	
		BL4 & BL5	6-Reqd.		
1	L. 75 × 75 × 9	7.010	@ 9.96	69.8	
2	〃 〃	3.430	〃 〃	68.3	
1	Pl. 270 × 9	645	〃 19.08	12.3	
2	Ls. 75 × 75 × 9	655	〃 9.96	13.0	
2	Pls. 140 × 9	140	〃 9.89	2.8	
				166.2 × 6 =	997.2
				166.2	
	Summary of Bottom lateral Bracing				1955.2
	STRINGERS				
8	Is. 300 × 150	4.005	@ 48.34	1548.8	
32	〃 〃	3.740	〃 〃	5785.3	
72	Pls. 230 × 8	3.000	〃 14.44	311.9	
20	Ls. 150 × 90 × 9	173	〃 16.32	56.5	
				7702.5	
	SHOES				
2	Cast steel shoes RS1		@ 124.7	249.4	1067.0 (Casting)
4	〃 〃 Dust guards DG1		〃 16.4	65.6	
2	〃 〃 Bed Pls. BP1		〃 137.5	275.0	
2	〃 〃 fixed Shoe FS1		〃 238.5	477.0	
4	Pins 115φ × 530		〃 81.53	172.8	
8	nuts for 115φ Pin		〃 2.09	16.7	
8	Rollers 90φ × 550		〃 49.93	219.7	
4	Side Pls. 70 × 13 × 370		〃 7.14	10.6	
16	Pins 25φ × 50		〃 0.017	0.3	
4	dust guard Pls. 115 × 6 × 580		〃 5.42	12.6	
20	Tapped bolts 6φ × 20		〃 0.007	0.1	
16	Anchor bolts 32φ × 700		〃 5.20	83.2	
16	Anch. Pls. 150 × 10 × 150		〃 11.78	28.3	
				1611.3	
	RIVET HEADS.				
21.790	Shop rivet heads 19φ @ 0.0646			1408.0	
11.680	Field 〃 〃 〃 〃 〃			755.0	
				2163.0	

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CALCULATIONS FOR

Material list of Ashida-Bashi for Okayama-Ken.

Total summary of weight for Truss Span

	Kg
Top chords.	12785.6
Bottom "	12,199.8
Vertical members	3705.4
Diagonals	5,082.4
Floor beams	10,185.2
Bottom lateral bracings	1,955.2
Stringers	7,702.5
Shoes	1,611.3 (内 Cast steel 1067.0 ヲ含ム)
Rivet heads.	2,163.0
	57390.4

or 57.390 Tons

× 2

114.780

Caststeel 1.067 × 2 = (−)2.134

112.646

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CALCULATIONS FOR

Material list of Ashida-bashi for Okayama-Ken

No.	Description		Length	Weight Kg/m	Total Weight	Remarks
	Girder Span (18.m650 span)			16 span - Required		
	MAIN GIRDER G3 R/L & G4 R/L 2-Reqd.					
8	Flg. Ls.	150×150×15	× 9.475	33.55	2,543.1	
2	Web Pls.	1300 × 10	× 9.475	102.05	1,933.8	
4	Cov. Pls.	330 × 10	× 4.619	25.905	478.6	
8	Ls.	125×90×10	× 1,280	16.09	164.8	
28	〃	〃	× 1,310	〃	590.0	
4	Fillers	300 × 15	× 1,005	35.325	142.0	
2	Pls.	230 × 10	× 325	18.055	11.7	
10	Ls.	100×75×10	× 170	12.95	22.0	
2	Pls.	330 × 10	× 860	25.905	44.6	
4	Ls.	150×150×19	× 860	41.91	144.2	
4	Pls.	170 × 15	× 720	20.018	57.7	
2	〃	300 × 10	× 660	23.55	31.1	
2	Sole Pls.	300 × 19	× 530	44.745	47.4	
					6211.0 × 2 = 12,422.	
	STRINGER S5, S6, S7 & S8					
4	Is	300 × 150	× 3,875	48.34	749.3	
4	Pls.	230 × 9	× 240	16.25	15.6	
4	Ls.	150×90×9	× 182	16.32	11.9	
6	Is	300 × 150	× 3,720	48.34	1078.9	
12	Pls.	230 × 9	× 240	16.25	46.8	
6	Ls.	150×90×9	× 184	16.32	18.0	
					1,920.5 × 1 = 1,920.5	
	FLOOR BEAMS FB1 & FB2					
12	Flg. Ls.	75×75×9	× 4,690	9.96	560.5	
12	〃	〃	× 4,500	〃	537.8	
6	Pls.	750 × 8	× 3,910	47.100	1,104.7	
48	Ls.	75×75×9	× 760	9.96	363.3	
24	Pls.	145 × 9	× 605	10.244	148.7	
12	〃	160 × 10	× 225	12.560	33.9	
4	〃	390 × 8	× 1,260	24.492	123.4	
8	〃	〃	× 1,025	〃	200.8	
					3073.1 × 1 = 3,073.1	
	BOTTOM LATERAL BRACINGS LB5, 6, 7 & 8					
4	Ls.	75×75×9	× 5,585	9.96	222.5	
2	Pls.	75 × 9	× 200	5.299	2.1	
6	Ls.	75×75×9	× 5,670	9.96	338.8	
3	Pls.	75 × 9	× 200	5.299	3.2	
10	Ls.	75×75×9	× 735	9.96	73.2	
10	Pls.	140 × 9	× 140	9.891	13.8	
4	〃	235 × 9	× 300	16.603	19.9	
8	〃	300 × 9	× 535	21.195	90.7	
					764.2 × 1 = 764.2	

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CALCULATIONS FOR

Material list of Ashida-bashi for Okayama-Ken

No.	Description		Length	Weight kg/m	Total Weight	Remarks
	BED PLATE BP1			4-Required		
1	Pls.	320 × 35	× 530	87.92	46.6	
2	Ls.	75 × 75 × 9	× 210	9.96	4.2	
					50.8	× 4 = 203.2
	ANCHOR BOLTS AB1					
8	bolts	35φ	× 700	@ 6.5	52.0	
8	Washers	150 × 9	× 150	10.598	12.7	
					64.7	× 1 = 64.7
	RIVET HEADS					
3,696	Shop Rivets heads	22φ		@ 0.096	354.8	
672	field " "	"		"	64.5	
952	" " "	19φ		@ 0.065	61.9	
2,053	Shop " "	"		"	133.4	
					614.6	× 1 = 614.6
	Summary for 1 span			19,062.3		
	Girder Span (13.600 m span)			1-span-Required		
	MAINGIRDER G1L & G2L			2-Reqd.		
4	Flg. Ls.	150 × 150 × 15	6,330	33.55	849.5	
4	"	"	7,570	"	1015.9	
1	Web Pl.	1,300 × 10	6,330	102.05	646.0	
1	"	"	7,570	"	772.5	
8	Ls.	125 × 90 × 10	1,280	16.09	164.8	
4	filler	300 × 15	1,005	35.325	142.0	
22	Ls.	125 × 90 × 10	1,310	16.09	463.7	
2	Ls.	100 × 75 × 10	160	12.95	4.1	
3	"	"	150	"	5.8	
3	"	"	170	"	6.7	6
4	"	150 × 150 × 19	860	41.91	144.2	
2	Pls.	260 × 10	310	20.41	12.7	
4	"	170 × 15	720	20.018	57.7	
2	"	300 × 10	660	23.55	31.1	
2	Sole Pls.	300 × 19	530	44.745	47.4	
1	Pl.	230 × 10	310	18.055	5.6	
					4,369.7	× 2 = 8,739.4
	FLOOR BEAMS FB1 & FB2					
10	Ls.	75 × 75 × 9	4,500	9.96	448.2	
10	"	"	4,690	"	467.1	
5	Web Pls.	750 × 8	3,910	47.10	920.8	
4	Pls.	390 × 8	1,260	24.492	123.4	
6	"	"	1,025	"	150.6	
20	Pls.	145 × 9	605	10.244	124.0	
10	"	160 × 10	225	12.560	28.3	
40	Ls.	75 × 75 × 9	760	9.96	302.8	
					2,565.2	× 1 = 2,565.2

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CALCULATIONS FOR

Material list of Ashida-bashi for Okayama-Ken

No.		Description	Length	Weight kg/m	Total weight	Remarks
		STRINGERS	S1, S2, S3 & S4			
4	Is.	300 × 150	3,545	48.34	685.5	
4	"	"	3,390	"	655.5	
4	Ls.	150 × 90 × 9	178	16.32	11.6	
4	"	"	180	"	11.8	
12	Pls.	230 × 9	240	16.25	46.8	
					1,411.2	×1 = 1,411.2
		BOTTOM LATERAL BRACINGS	LB1, 2, 3 & 4			
4	Ls.	75 × 75 × 9	5,365	9.96	213.7	
4	"	"	5,450	"	217.1	
4	Pls.	75 × 9	200	5.299	4.2	
8	Ls.	75 × 75 × 9	735	9.96	58.6	
8	Pls.	140 × 9	140	9.891	11.1	
4	Pls.	225 × 9	310	15.896	19.8	
4	"	330 × 9	510	23.315	47.6	
2	"	315 × 9	510	22.255	22.7	
					594.8	×1 = 594.8
		BED PLATES	BP1	4-Reqd.		
1	Pls.	320 × 35	530	87.92	46.6	
2	Ls.	75 × 75 × 9	210	9.96	4.2	
					50.8	×4 = 203.2
		ANCHOR BOLTS	AB1			
8	bolts	35φ	700	@ 6.5	52.0	
8	Washers	150 × 9	150	10.598	12.7	
					64.7	×1 = 64.7
		RIVET HEADS				
1,908	Shop rivet heads		22φ	@ 0.096	183.2	
672	field " "		"	"	64.5	
1,692	Shop " "		19φ	@ 0.065	110.0	
768	field " "		"	"	49.9	
					407.6	×1 = 407.6
		Summary for 1 span		13,986.1		
		BRACKET HB1 for lamp post		14-Reqd (for 16 span)		
2	Ls.	75 × 75 × 9	325	9.96	6.5	
1	Pl.	305 × 8	420	19.154	8.0	
					14.5	×14 = 203.0

Total Summary for Girder Span

18.650m span	16	@ 19,062.3	=	304,996.8
13.600m "	1	@ 13,986.1	=	13,986.1
Brackets				203.0
				319,185.9 kg. or 319.186 Tons

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CALCULATIONS FOR 芦田橋

Material list of Ashida-bashi for Okayama-Ken.

No.	Description	Length Hand	Unit weight Rails	Weight	Remarks
324	Cast Iron		@ 68.8	22,291.2 kg	HR1
76			" 69.2	5,259.2	HR2
14			" 65.5	917.0	HR3
2			" 34.0	68.0	HR4
18			" 18.7	336.6	HR5
2			" 22.6	45.2	HR6
14			" 22.1	309.4	HR7
2			" 19.4	38.8	HR8
2			" 21.8	43.6	HR9
324	Gas pipe 63φ.	1.785	" 8.5	4,915.9	
76	" " "	1.805	"	1,166.0	
14	" " "	1.661	"	197.7	
2	" " "	965	"	16.4	
18	" " "	400	"	61.2	
2	" " "	544	"	9.2	
14	" " "	490	"	58.3	
2	" " "	440	"	7.5	
2	" " "	470	"	8.0	
5,090	Screws 8φ		@ 0.015	76.4	
2,610	" "		@ 0.02	52.2	
398	Cast Iron post.		@ 29.00	11542.0	PT1
36	" " "		" 27.00	972.0	PT2
2	" " "		" 28.3	56.6	PT3
2	" " "		" 27.0	54.0	PT4.
876	Anchor bolts	16φ. 225	@ 0.45	394.2	
438	P/s 60 × 9	200	4.24	371.4	
	Lamp posts.				
14	Cast Iron		@ 106.8	1,495.2	LP1
4	" "		@ 122.2	488.8	LP2
18	Gas pipe	89φ 1.150	@ 13.2	273.2	
18	" "	76φ 1.550	@ 11.0	306.9	
72	Anchor bolts	19φ 70	@ 0.33	23.8	
216	Screws	8φ	@ 0.012	2.6	
				51,858.5	

Total weight. 51,858.5 kg or 51.859 Tons

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CALCULATIONS FOR

Material list of Ashida-bashi for Okayama-Ken.

No.	materials	Length	Weight kg/m	Total wt.	Remarks
		Expansion joint	EJ1	17 Required	
1	Bar 30 × 10	× 6.310 m	@ 2.355	14.9	
1	L 100 × 75 × 10	× 6.310	〃 12.95	81.7	
2	pls 320 × 9	× 560	〃 22.608	25.3	
2	〃 160 × 9	× 210	〃 11.304	4.7	
2	Ls 90 × 75 × 9	× 490	〃 11.02	10.8	
2	〃 〃	× 160	〃 11.02	3.5	
42	Rivet heads 10φ		〃 0.01	0.4	
60	〃 〃 19φ		〃 0.065	3.9	
				145.2	× 17 = 2,468.4
		EJ2		17 Required	
1	Checkered pl. 230 × 9	× 6.310	@ 18.2	114.8	
1	L 65 × 65 × 8	× 6.310	〃 7.66	48.3	
2	pls 320 × 9	× 560	〃 22.608	25.3	
2	〃 160 × 9	× 215	〃 11.304	4.9	
2	Ls 90 × 75 × 9	× 490	〃 11.02	10.8	
2	〃 〃	× 160	〃 11.02	3.5	
42	Rivet heads 10φ		〃 0.01	0.4	
60	〃 〃 19φ		〃 0.065	3.9	
				211.9	× 17 = 3,602.3
		EJ3		2 Required	
1	Bar 30 × 10	× 6.310	@ 2.355	14.9	
1	L 100 × 75 × 10	× 6.310	〃 12.95	81.7	
2	pls 320 × 9	× 635	〃 22.608	28.7	
2	〃 160 × 9	× 215	〃 11.304	4.9	
2	Ls 65 × 65 × 8	× 570	〃 7.66	8.7	
2	〃 〃	× 160	〃 7.66	2.4	
42	Rivet heads 10φ		〃 0.01	0.4	
60	〃 〃 19φ		〃 0.065	3.9	
				145.6	× 2 = 291.2
		EJ4		2 Required	
1	Checkered pl. 230 × 9	× 6.310	@ 18.2	114.8	
1	L 65 × 65 × 8	× 6.310	〃 7.66	48.3	
2	pls 320 × 9	× 635	〃 22.608	28.7	
2	〃 160 × 9	× 225	〃 11.304	5.1	
2	Ls 65 × 65 × 8	× 570	〃 7.66	8.7	
2	〃 〃	× 160	〃 7.66	2.4	
42	Rivet heads 10φ		〃 0.01	0.4	
60	〃 〃 19φ		〃 0.065	3.9	
				212.3	× 2 = 424.6

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CALCULATIONS FOR

Material list of Ashida-bashi for Okayama-Ken

Connection Bolts for Expansion joint

148	Bolts	19φ × 60	@ 0.31	45.9	
4	Anchor bolts	16φ × 300	@ 0.568	2.3	
				48.2 × 1	= 48.2

Total weight 6,834.7

Total summary of weight

2 -	Truss spans	112.646
17 -	Girder "	319.186
	Expansion joint	6.835
		438.667
	Handrails	51.859
	Cast steel	2.134
		492.660

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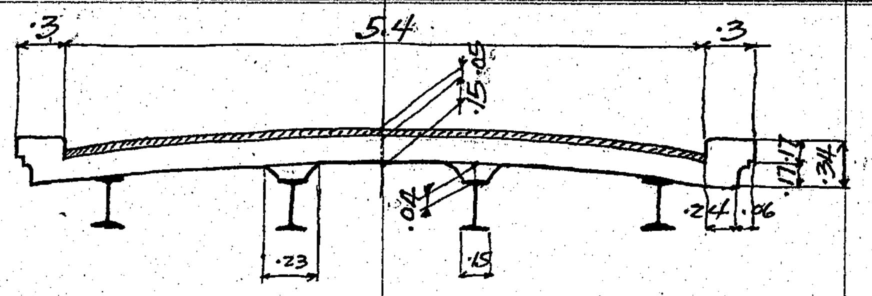
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

Materials for Bridge Floor.



Total length of Bridge out to out of slab.

distance between parapet walls 394.130 m.

+ .090

− .085

Out to out of slab = 394.135 m

Exp. jt. Girder & girder 15 @ .12 = 1.80

" & truss 2 @ .125 = .25

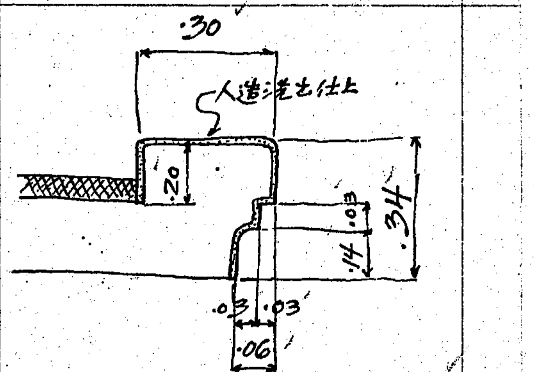
truss & truss 1 @ .12 = .12

2.170

Total net length of slab = 391.965 m. Call this 391.97 m

Total " " " pavement & Artificial granite finish = 391.97 m.

(Construction jts. neglected.)



Concrete for Floor slab. 1:2:4 mixture.

Sectional area of slab.

Slab.	0.15 × 5.40 =	.810
Coping	0.30 × 0.34 × 2 =	.204
" less	0.06 × 0.15 × 2 = (−)	.018
fillets on stringers	0.19 × 0.04 × 2 =	.015
		1.011 □m.

total width of finish = .90 m

" " " form for coping = 0.6 m

total volume of slab concrete = 1.011 × 391.97 = 396.282 cub. meters.

Total area of pavement 5.4 × 391.97 = 2116.64 □m.

Less area of drains 158 @ .0693 = (−) 10.95

2105.69 □m.

area of 1 drain .315 × .22 = .0693 □m

drains

2 truss spans @ 12 = 24

16 girder " @ 8 = 128

1 " " @ 6 = 6

Total no. of drains = 158 drains

Total area of artificial granite finish (人造洗石仕上)

0.9 × 391.97 × 2 = 705.55 □m

Less handrail & lampposts area (−) 12.56

692.99 □m.

Handrail post 434 posts.

@ 0.20 × 0.14 = 0.028 □m × 434 = 12.15 □m

Lamppost 18 posts.

@ .15 × .15 = .0225 × 18 = 0.41

12.56 □m

Forms for Slab.

Bottom width 5.4 + 2 × .24 = 5.88

Coping area 0.6 × 2 = 1.20

7.08 m.

7.08 × 391.97 = 2,775.15 □m

Top area of stringers for truss spans.

4 × .15 × 37.94 × 2 = 45.53 □m.

Top area of stringers, +girders for Girder spans.

total length of Stringers = 391.97 − 75.88 = 316.09 m.

Stringers 2 × .15 × 316.09 = 94.83

Girders 2 × .33 × 316.09 = 208.62

303.45 □m.

Total area of form. 2,775.15

truss span stringers (−) 45.53

girder " " + main girders (−) 303.45

2,426.17 □m

Reinforcements for slab. ~~plain~~ Deformed bars See drawing.

Girder spans 38648 Kg tons

Truss spans 9346

Total slab reinf. = 47.994 Kg tons.

Drains 158 drains @ 18.65 Kg = 2946.7 Kg = 2.947 Kg tons. (786 貫)

(4.98 貫)

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

MADE BY K. Inaba DATE 3-10-8 FILE NO

CHECKED BY I. Matano DATE 3-10-9 PAGE NO 65

CALCULATIONS FOR

Ashida-Bashi for Okayamaken.

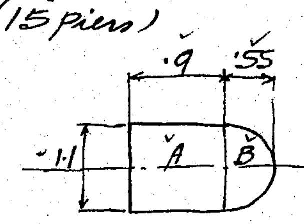
Material List for Piers P1, P2, P6, P7, P8, P9, P10, P11, P12, P13, P14, P15, P16, P17 & P18. (15 piers)

Concrete for Shaft. 1:2:4 mixture

Coping.

	Section	Length	Reqd. no.	Volume	Remarks.
Coping rectangle	1.1 × .3	.9	2	0.594	A
" Circular ends.	1.1φ	.3	1	0.285	B.

total = 0.879 m^3 for all piers



Shaft.

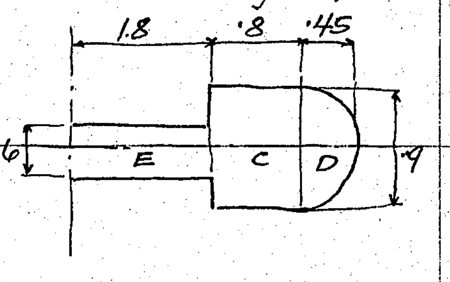
Top section

Shaft. rectangle C .8 × .9 × 2 = 1.440

Circular ends D .9φ = .636

Curtain wall E .6 × 3.6 = 2.160

4.236 m^2 for all piers

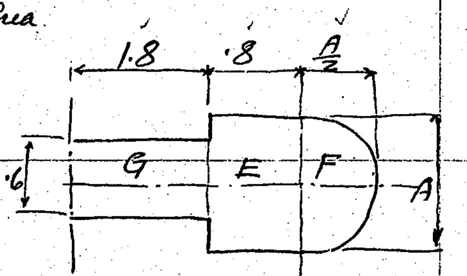


Bottom sections.

Curtain wall G .6 × 3.6 = 2.160 m^2 for all piers.

Shaft: (Dimension A

Piers	~~Ht~~	Area E		Circular ends F		Curtain wall G		Total Bottom Area
P1	1.66	× .8 × 2 = 2.656	+	2.164 m^2	+	2.160 m^2	=	6.980
P2	1.67	" = 2.672	+	2.190	+	"	=	7.022
P6 & P11	1.69	" = 2.704	+	2.243	+	"	=	7.107
P7 & P10	1.69	" = "	+	"	+	"	=	7.107
P8 & P9	1.69	" = "	+	"	+	"	=	7.107
P12	1.68	" = 2.688	+	2.217	+	"	=	7.065
P13	1.68	" = "	+	"	+	"	=	7.065
P14	1.68	" = "	+	"	+	"	=	7.065
P15	1.68	" = "	+	"	+	"	=	7.065
P16	1.67	" = 2.672	+	2.190	+	"	=	7.022
P17	1.67	" = "	+	"	+	"	=	7.022
P18	1.66	" = 2.656	+	2.164	+	"	=	6.980



Total volume of shaft.

Piers	Top area	Bottom area	Mean area		Height		Volume of shaft		Coping		Total volume of Shaft.
P1	4.236	6.980	5.608	×	7.612 m	=	42.688	+	0.879	=	43.567 cub. m.
P2	"	7.022	5.629	×	7.663	=	43.135	+	"	=	44.014
P6 & P11	"	7.107	5.672	×	7.853	=	44.542	+	"	=	45.421
P7 & P10	"	7.107	5.672	×	7.864	=	44.605	+	"	=	45.484
P8 & P9	"	7.107	5.672	×	7.869	=	44.633	+	"	=	45.512
P12	"	7.065	5.651	×	7.837	=	44.287	+	"	=	45.166
P13	"	7.065	5.651	×	7.815	=	44.163	+	"	=	45.042
P14	"	7.065	5.651	×	7.788	=	44.010	+	"	=	44.889
P15	"	7.065	5.651	×	7.755	=	43.824	+	"	=	44.703
P16	"	7.022	5.629	×	7.718	=	43.445	+	"	=	44.324
P17	"	7.022	5.629	×	7.673	=	43.191	+	"	=	44.070
P18	"	6.980	5.608	×	7.624	=	42.755	+	"	=	43.634

Reinforcements for Shaft. Plain Bars.

1.684 Kg. tons for each pier. see drawing

JIUN MASUDA
CONSULTING ENGINEER.
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

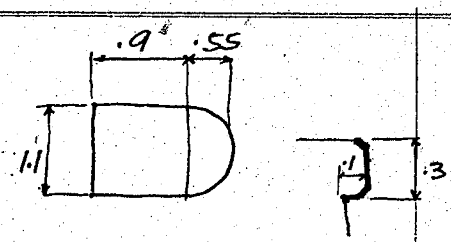
Ashida-Bashi for Okayama Ken.

Forms for Shaft.

Coping

Sides	$\cdot 9 \times 4 =$	3.68^{m}
ends	$1.1 \times 2 =$	2.20
Circular ends	$1.1\pi =$	3.46

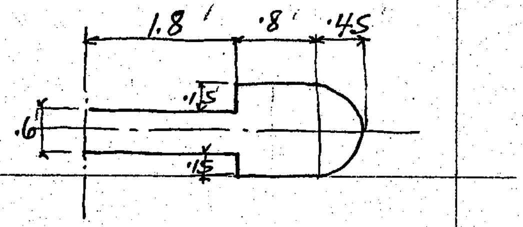
$9.26^{m} \times \cdot 4 = 3.70^{\square m}$ for all piers.



Shaft. Total length of perimeter at top of shaft.

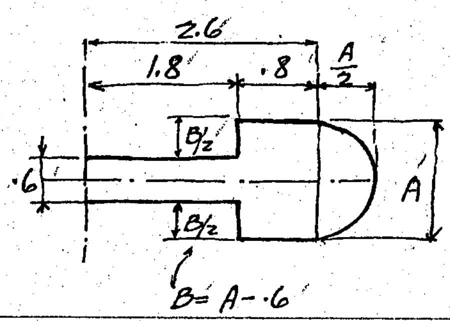
Shaft	$\cdot 8 \times 4 =$	3.20^{m}
"	$\cdot 15 \times 4 =$	·60
"	$\cdot 9\pi =$	2.83
Curtain wall	$3.6 \times 2 =$	7.20

13.83^{m} for all piers



Total length of perimeter at bottom of shaft.

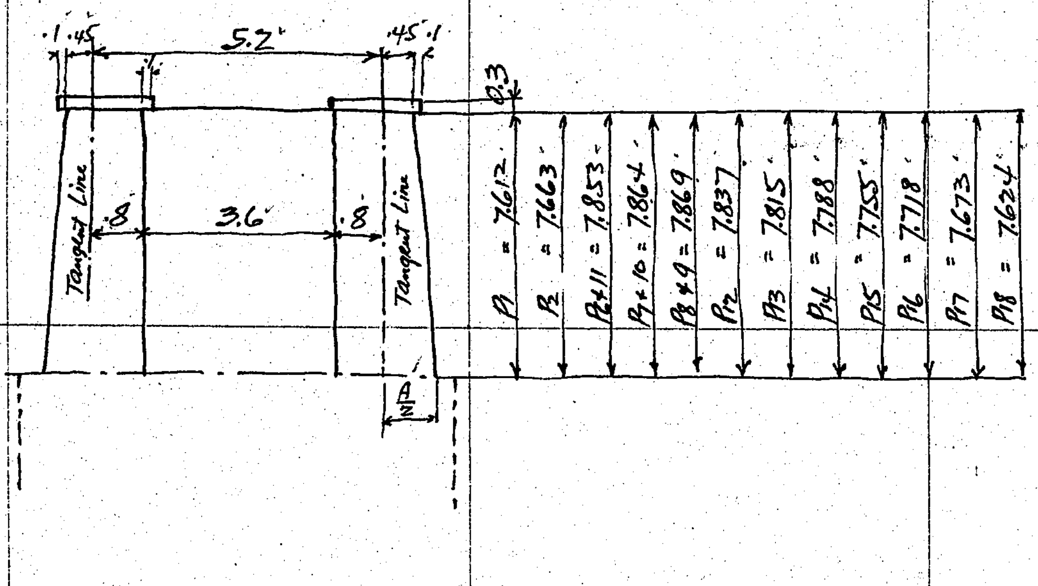
Piers	Dimension A.	Circular ends		Dimension B		Straight portions		Total Length.
P1	1.66	5.22	+	2.12	+	10.4	=	17.74^{m}
P2	1.67	5.25	+	2.14	+	"		17.79
P6+P11	1.69	5.31	+	2.18	+	"		17.89
P7+P10	1.69	"	+	"	+	"		17.89
P8+P9	1.69	"	+	"	+	"		17.89
P12	1.68	5.28	+	2.16	+	"		17.84
P13	1.68	"	+	"	+	"		17.84
P14	1.68	"	+	"	+	"		17.84
P15	1.68	"	+	"	+	"		17.84
P16	1.67	5.25	+	2.14	+	"		17.79
P17	1.67	"	+	"	+	"		17.79
P18	1.66	5.22	+	2.12	+	"		17.74



Straight portion $5.2 \times 2 = 10.4^{m}$ for all piers

Total Form for Shaft.

Piers	Top length	Bottom length	Mean length	Height		Area for shaft		Coping		Total Form for shaft.
P1	13.83^{m}	17.74	15.79^{m}	× 7.612	=	120.19	+	3.70	=	$123.89^{\square m}$
P2	"	17.79	15.81	× 7.663	=	121.15	+	"	=	124.85
P6+P11	"	17.89	15.86	× 7.853	=	124.55	+	"	=	128.25
P7+P10	"	"	"	× 7.864	=	124.72	+	"	=	128.42
P8+P9	"	"	"	× 7.869	=	124.80	+	"	=	128.50
P12	"	17.84	15.84	× 7.837	=	124.14	+	"	=	127.84
P13	"	"	"	× 7.815	=	123.79	+	"	=	127.49
P14	"	"	"	× 7.788	=	123.36	+	"	=	127.06
P15	"	"	"	× 7.755	=	122.84	+	"	=	126.54
P16	"	17.79	15.81	× 7.718	=	122.02	+	"	=	125.72
P17	"	"	"	× 7.673	=	121.31	+	"	=	125.01
P18	"	17.74	15.79	× 7.624	=	120.38	+	"	=	124.08



JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

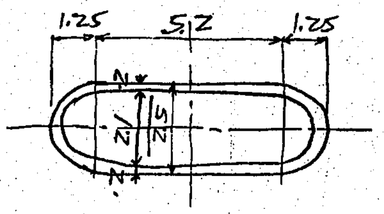
Ashida-Bashi for Okayama Ken.

Material List for well.

$2.5^m \times 5.0^m$ well. for Piers P1, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17 & 18 (14 piers)

Concrete 1:2:4 mixture.

Shell 1^m from top.



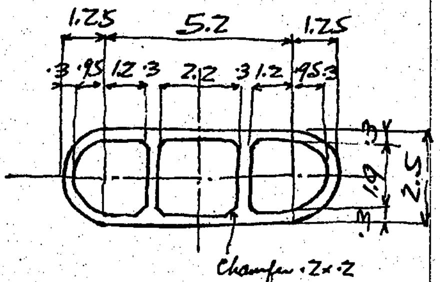
Total area of well $5.2 \times 2.5 = 13.000$
$2.5^{\phi} = 4.909$
$17.909^{\square m}$

Area of hollow space $5.2 \times 2.1 = 10.920$
$2.1^{\phi} = 3.464$
$14.384^{\square m}$

net area of shell = $3.525^{\square m}$

Volume of shell 1^m from top. $= 3.525\ m^3$

Shell 4^m from bottom.



Total area of well. $5.2 \times 2.5 = 13.000$
$2.5^{\phi} = 4.909$
$17.909^{\square m}$

Area of hollow spaces. $2.2 \times 1.9 = 4.180$
$1.2 \times 1.9 \times 2 = 4.560$
$1.9^{\phi} = 2.835$
less chamfers $.2 \times .2 \times 4 = (-)\ .160$
$11.415^{\square m}$

net area of shell. $= 6.494^{\square m}$

mean length of shell $= 3.82^m$.

Volume of shell 4^m from bottom $= 6.494 \times 3.82 = 24.807\ m^3$
" " " 1^m " top $= 3.525$
Total vol. of shell $= \underline{\underline{28.332}}\ m^3$

Top filling 1:2:4 mixture.
Area of hollow space $= 14.384^{\square m} \times 1 = 14.384$ cub. m.

Bottom filling 1:2:4 mixture
Area of hollow spaces $= 11.415^{\square m} \times 1.32 = 15.068$
total area of well $= 17.909^{\square m} \times 0.18 = 3.224$
18.292 cub. m

Total volume of Top & bottom fillings $= \underline{\underline{32.676}}\ m^3$

total concrete for well $61.008\ m^3$

Sand filling $11.415 \times 2.5 = \underline{\underline{28.538}}\ m^3$

$2.5^m \times 7.0^m$ well. for piers P2.

Concrete 1:2:4 mixture

Shell 1^m from top $3.525\ m^3$ see above calculations.
6^m from bottom (mean length) $5.82^m \times 6.494 = 37.795$
Total volume of shell $= \underline{\underline{41.320}}\ m^3$

Filling concrete 1:2:4 mix
Top filling $14.384\ m^3$
Bottom filling 18.292
Total volume of concrete filling $= \underline{\underline{32.676}}\ m^3$

total concrete for well $= 73.996\ m^3$

Sand filling $11.415 \times 4.5 = \underline{\underline{51.368}}\ m^3$

—— · ——

Summary	$5.0^m \times 2.5^m$ well	$7.0^m \times 2.5^m$ well.
Concrete for shell	$28.332\ m^3$	$41.320\ m^3$
do for fillings	32.676	32.676
total concrete for well	$61.008\ m^3$	$73.996\ m^3$
Sand filling	$28.538\ m^3$	$51.368\ m^3$
total volume of well (excavation for well sinking)	$89.546\ m^3$	$125.364\ m^3$

—— · ——

CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

Forms for well.

$2.5^m \times 5.0^m$ well.

Total length of perimeter out side $5.2 \times 2 = 10.40^m$

$2.5^\phi = 7.85$

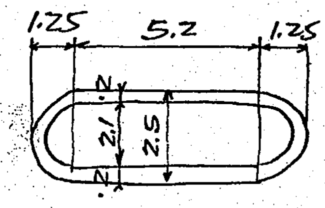
18.25^m

inside $5.2 \times 2 = 10.40^m$

$2.1^\phi = 6.60$

17.00^m

Forms for upper 1^m = 35.25^{m^2}



Total length of perimeter out side 18.25^m

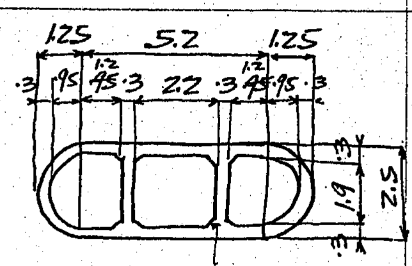
inside $(1.8 + 1.0 + 1.0) \times 2 = 7.60$

$1.9^\phi = 5.97$

$1.5 \times 4 = 6.00$

$.2 \times 1.41 \times 8 = 2.26$

21.83^m



.2x.2 chamfer.

Forms for lower 4^m. mean length say 3.82^m (inside)

outside area = $18.25 \times \cancel{3.82}$ 3.70 = 67.53^{m^2}

inside area = 21.83×3.82 = 83.39

$150.92^{\square m}$

Form for bottom of top filling hollow area = $11.415^{\square m}$ see last page.

~~11.415~~

Total area of forms = $35.25 + 150.92 + 11.415 = 197.59^{\square m}$.

2.5×7.0^m well.

Add area for 2 meters to above well.

outside $18.25 \times 2.0 = 36.50^{\square m}$

inside $21.83 \times 2.0 = 43.66$

$80.16^{\square m}$

area for 5^m well = 197.59

Total forms for 2.5×7.0^m well = $277.75^{\square m}$

Reinforcements for well.

for 2.5×5.0^m well 1.585 kg tons. See drawings.

for 2.5×7.0^m well 2.472 "

Steel for curb shoe

for both wells @ 0.922 kg tons.

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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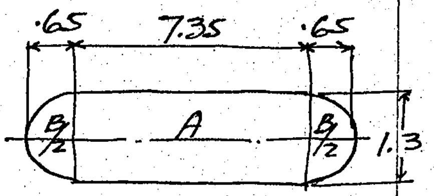
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken

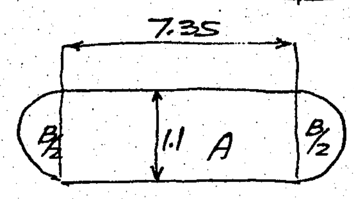
Material List for Piers P3 & P5.

Concrete for Shaft (1:2:4 mixture) for Pier P3.

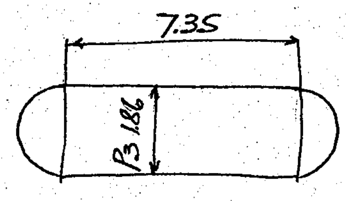


Coping	Section	Length	Reqd. no.	volume	Remarks.
Rectangle A	1.3 × .3	× 7.35	× 1 =	2.867	
Circular ends B.	1.3φ	× 0.3.	× 1 =	.398	

total vol. of coping = (+) 3.265 m³

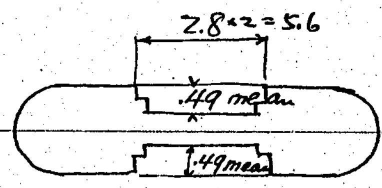


Shaft. Sectional area at top of shaft.
rectangle A 1.1 × 7.35 = 8.085
Circular ends B 1.1φ = 0.950
total 9.035 □m



Sectional area at bottom of shaft (gross area)
rectangle A 1.86 × 7.35 = 13.671
Circular ends B 1.86φ = 2.717
total = 16.388 □m
mean area = 25.423 ÷ 2 = 12.712 □m

Volume of shaft = 12.712 × 7.607 = (+) 96.700 m³



Less depression on bothsides mean height 6.44 m mean depression = .98 m say
5.6 × 6.44 = 36.064 □m × .98 = (−) 35.343 m³
add moulding = .1 × .1 × 17.8 × 2 = (+) 0.356 m³

Summary for Shaft concrete (P3)

Bearing for girder.
.48 × .8 × .14 × 2 = .108 m³ for one pier

Coping	3.265
Shaft	96.700
depression less (−)	35.343
moulding	0.356
	64.978 m³ + .108 (under girder shoe) = 65.086 m³

Concrete for Shaft 1:2:4 mix. for Pier P5.
Coping same as for P3 = 3.265 m³
Shaft sectional area at top = 9.035 □m
sectional " " bottom
1.87 × 7.35 = 13.745 □m
1.87φ = 2.746
16.491 □m
mean 25.526 ÷ 2 = 12.763 □m

Volume of Shaft (gross) = 12.763 × 7.695 = 98.211 m³
Depression mean height = 6.528, mean depression = .99
Volume of depression 5.6 × 6.528 × .99 = (−) 36.191 m³
moulding say 0.400

Summary for Shaft Concrete (P5)

Coping	3.265
Shaft	98.211
depression less	(−) 36.191
moulding	0.400
	65.685 m³ + .108 (under girder shoe) = 65.793

Reinforcements for Shaft. Plain bars.
Piers P3 2.064 Kg tons See drawing
Pier P5 2.064 "

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY K. Inaba DATE 3-10-11 FILE NO

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CALCULATIONS FOR

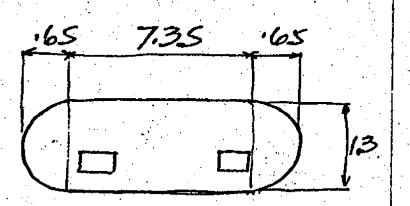
Ashida-Bashi for Okayama Ken

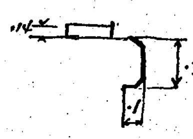
Forms for Piers P3 & P5

Coping. total length of perimeter of coping 2×7.35 = 14.70 m
1.3φ = 4.08
18.78 m

Form for coping = 18.78 × 0.4 = 7.51 □m for each pier

Bearing under girder shoes (.48+.8)×2 = 2.56 × .14×2 = .72
8.23 □m

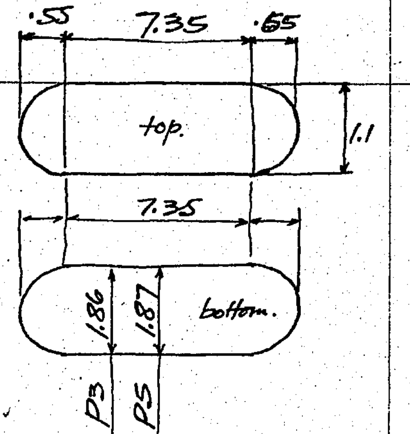




Shaft. total length of perimeter at top of shaft. 2×7.35 = 14.70 m
1.1φ = 3.46
18.16 m for both piers

do at bottom of shaft. 2×7.35 = 14.70
1.86φ = 5.84
20.54 m for P3 mean 19.35 m

do " - 2×7.35 = 14.70
1.87φ = 5.87
20.57 m for P5 mean 19.37



Forms for Shaft. P3 ~~19.35~~ × 7.607 = ~~147.20~~ □m
P5 ~~19.37~~ × 7.695 = ~~149.05~~ □m

Depression mean width .49 m × 17.8 × 2 = 17.44 □m for each pier assumed.

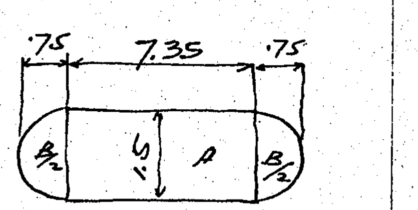
Summary of forms for shaft.

	Pier P3	Pier P5
Coping	8.23 □m	8.23 □m
Shaft	~~147.20~~	~~149.05~~
depression (side)	17.44	17.44
	~~172.87~~ □m	~~174.72~~ □m

Material List for Pier P4.

Concrete for Shaft. 1:2:4 mixture.

Coping rectangle A 1.5 × .3 × 7.35 = 3.308
circular ends B 1.5φ × .3 = .530
volume of coping = 3.838 cu.m

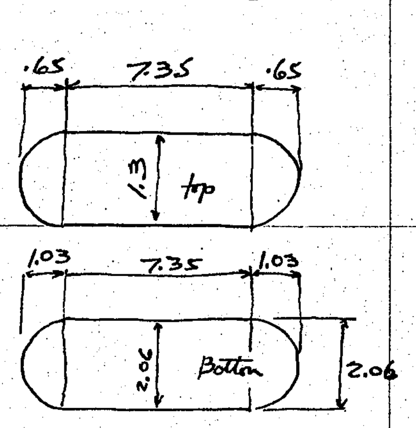


Shaft.

Sectional area of shaft at top. 7.35 × 1.3 = 9.555
1.3φ = 1.327
10.882 □m

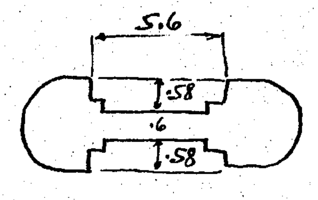
do at bottom 7.35 × 2.06 = 15.141
2.06φ = 3.333
18.474 □m
29.356 ÷ 2 = 14.678 □m mean

volume for shaft = 14.678 × 7.643 = 112.184 m³



depression of shaft mean height = 6.480 mean depression 2 × .58 = 1.16
5.6 × 6.48 = 36.288 □m × 1.16 = (−) 42.094 m³

moulding .1 × .1 × 17.70 m × 2 = (+) 0.354 m³



Summary for Shaft concrete.

Coping	3.838 m³
Shaft	112.184
depression, less	(−) 42.094
moulding add.	.354
	74.282 m³

Reinforcements for Shaft Plain bars 2.042 kg tons see drawing.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY K. Inaba. DATE 3-10-12 **FILE NO**

CHECKED BY M. Jinda DATE 3-10-12 **PAGE NO** 71

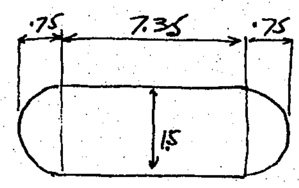
CALCULATIONS FOR

Ashida-Bashi for Okayama Ken.

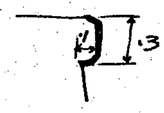
Forms for Pier P4.

Coping total length of perimeter of coping $2 \times 7.35 = 14.70^m$
$1.5^\phi = 4.71$
19.41^m

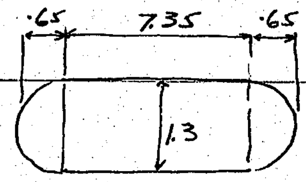
Form for coping $= 19.41 \times 0.4 = 7.76^{\square m}$



Shaft total length of perimeter at top of shaft. $2 \times 7.35 = 14.70$
$1.3^\phi = 4.08$
18.78^m



do at bottom $7.35 \times 2 = 14.70$
$2.06^\phi = 6.47$
21.17^m



mean perimeter $= \frac{18.78 + 21.17}{2} = 19.98^m$

Forms for shaft $= 19.98 \times 7.643 = 152.71^{\square m}$

Depression mean width $1.165^m \times 17.75 = 20.68^{\square m}$

Summary for forms for shaft.

coping	$7.76^{\square m}$
Shaft (front)	152.71
depression (side)	20.68
	$181.15^{\square m}$

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

"Ashida-Bashi" for Okayama Ken.

Material List for well.

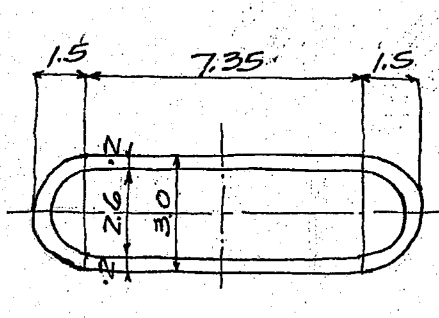
3.0m × 7.0m well for Piers P3 & P5.

Concrete 1:2:4 mixture.

Shell 1m from top.

Total area of well 7.35 × 3.0 = 22.05 sq.m
3.0φ = 7.07
29.12 sq.m

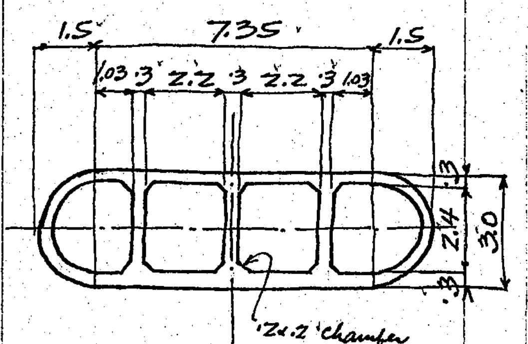
Area of hollow space 7.35 × 2.6 = 19.11
2.6φ = 5.31
24.42 sq.m



Net area of shell = 4.70 sq.m
Volume of upper 1m of well = 4.700 m³

Shell 6m from bottom

Total area of well = 29.12 sq.m
Area of hollow spaces. 2.2 × 2.4 × 2 = 10.560
1.03 × 2.4 × 2 = 4.944
2.4φ = 4.524
less chamfers. .2 × .2 × 6 = (−) .240
19.788 sq.m



Net area of shell = 9.332 sq.m
mean length of shell = 5.82m say

Volume of shell for lower ~~7m~~ 6m = 9.332 × 5.82 = 54.312 m³
" " " upper 1m = 4.700
Total volume of shell = 59.012 cub.m + 1.228 = 60.240 m³

Top filling 1:2:4 concrete.
Area of hollow space = 24.42 × 1.0 = 24.420 m³

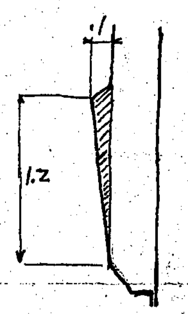
Bottom filling 1:2:4 concrete
Area of hollow space = 19.788 × 1.32 = 26.120
total area of well. = 29.120 × .18 = 5.242
31.362 m³

Total volume of fillings = 55.782 m³ − 1.228 = 54.554 m³

Sand filling hollow area = 19.788 × 4.5 = 89.046 m³

see

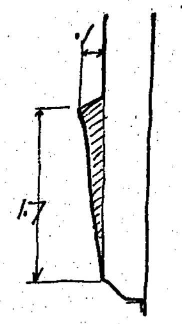
Projection of shell at Bottom
total length = 2.2 × ~~2~~ 4 + 1.03 × ~~2~~ 4 + 2.4φ = ~~17.444m~~ 20.460m
Area of projection = .1 × 1.2 ÷ 2 = .06 sq.m
Volume of projection = ~~17.444~~ 20.460 × .06 = 1.228 m³
Add this volume ~~on~~ to shell concrete and deduct from filling concrete for correction.



3.0 × 9.0m well. for Pier P4.

Concrete 1:2:4 mixture.

Shell upper 1.2m area = 4.700 × 1.2 = 5.640 m³
lower 7.8m area = 9.332 × 7.62 = 71.110 "
projection at bottom .085 × 20.46 = +1.739 "
total volume of shell = 78.489 m³



Top filling hollow area = 24.420 × 1.2 = 29.304 m³
Bottom filling " = 19.788 × 1.82 = 36.014
total area = 29.120 × .18 = 5.242
less projection of shell = = (−) 1.739
total concrete for fillings = 68.821 m³

Area of projection = .1 × 1.7 / 2 = .085 sq.m
total length = 20.46m

Sand filling 19.788 × 5.80 = 114.770 m³

Summary.

	3.0 × 7.0 well.	3.0 × 9.0 well.
Concrete for Shell.	60.240 m³	78.489
do for fillings	54.554	68.821
total concrete for well	114.794	147.310
Sand filling	89.046	114.770
total volume of well or excavation for well sinking	203.840 m³	262.080 m³

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CALCULATIONS FOR

Ashida-Bashi for Okayama Ken

Forms for well.

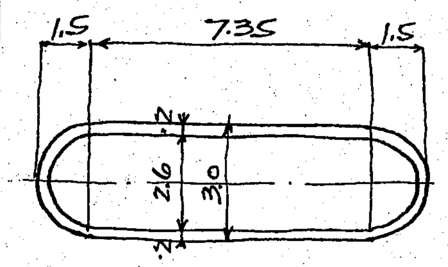
3.0ᵐ × 7.0ᵐ well

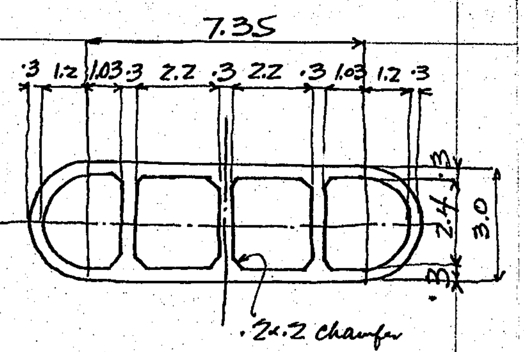
Total length of perimeter

outside $7.35 \times 2 = 14.70^m$
$3.0^\phi = \underline{9.425}$
24.125^m

inside $7.35 \times 2 = 14.70^m$
$2.6^\phi = \underline{8.17}$
22.870^m

Form for upper 1.0ᵐ = $\underline{47.00}^{\square m}$





Total length of perimeter

outside $24.125^{\#m}$

inside $(1.8+0.83)4 = 10.52$
$2.4^\phi = 7.54$
$2.0 \times 6 = 12.00$
$2 \times 1.41 \times 12 = \underline{3.38}$
$33.44^{\#m}$

Forms for lower 6ᵐ mean length of form say 5.82ᵐ (inside)

Out side area = $24.125 \times 5.70 = 137.51^{\square m}$

inside " = $33.44 \times 5.82 = \underline{194.62}$

Total for lower 6ᵐ shell. = $\underline{332.13}^{\square m}$ projection at bottom neglected.

Forms for bottom of top filling = hollow area = $\underline{19.788}^{\square m}$

Total area of forms. = $47.00 + 332.13 + 19.788 = \underline{398.92}^{\square m}$

3.0 × 9.0ᵐ well.

upper 1.2ᵐ Shell. = $47.0 \times 1.2 = 56.40^{m^2}$

lower 7.8ᵐ shell.

Outside area = $24.125 \times 7.5 = 180.94^{m^2}$

inside area = $33.44 \times 7.62 = 254.81$

Bottom of top filling = $\underline{19.79}$

511.94^{m^2}

Reinforcements for well

for 3.0 × 7.0ᵐ well 3,503 kg tons see drawings.
for 3.0 × 9.0ᵐ well 4,888 "

Steel for Curb shoe.

for both wells. c 1,258 kg tons

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY K. Inaba DATE 3-10-11 FILE NO

CHECKED BY M. Jinda DATE 3-10-12 PAGE NO 74

CALCULATIONS FOR

"Ashida-Bashi for Okayama Ken."

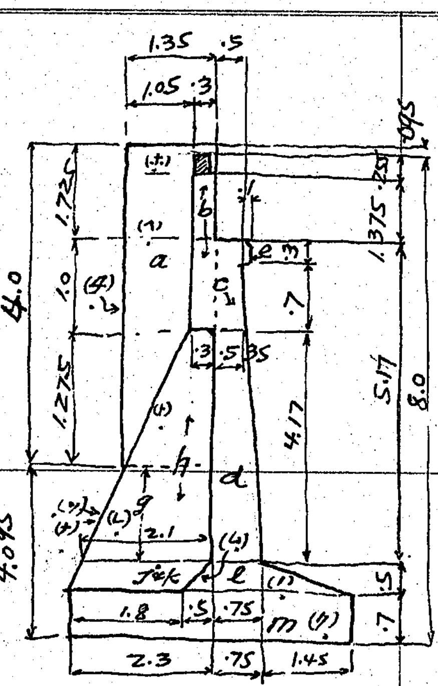
Material List for Abutments. A1 and A2.

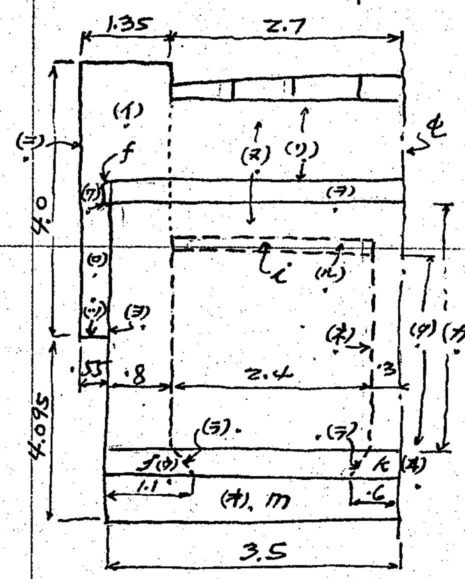
Concrete 1:2:4 mixture.

	Section	Length	Req'd. no	Total volume	Remarks
Column	1.35×1.35	4.0	2.	14.580	a.
parapet wall	.3×2.375	5.4	1	3.848	b
Shaft top beam.	.518×1.0	7.0	1	3.626	c
Curtain wall	.643×4.17	7.0	1.	18.769	d
Coping front.	.1×.3	7.2	1	.216	e
〃 Sides	.1×.3	.5	2.	.030	f
Buttress A	.8×1.725	2.895	2.	7.990	g
〃 B	.6×1.20	4.17	1	3.002	h
Chamfer, top of wall.	.1×.1÷2	2.40	2	.024	i
Base, under buttress A	.95×.5	1.95	2.	1.853	j
〃 〃 〃 B	.9×.5	1.95	1	.878	k
〃 〃 front wall	.5×1.725	7.0	1	6.038	l
〃 bottom layer	.7×4.5	7.0	1	22.050	m
				82.904 cub.m.	

Forms.

	Width	Length	Req'd no	Total area	Remarks
Column front side	1.35	1.725	2	4.658 m²	(イ)
〃 〃	.55	2.275	2	2.503	(ロ)
〃 Outside	1.35	4.00	2	10.800	(ハ)
〃 inside	1.35	.35	2	.945	(ニ)
〃 〃	1.05	2.375	2	4.988	(ホ)
〃 〃	1.35	1.275	2	3.443	(ヘ)
〃 rear side	1.35	4.00	2	10.800	(ト)
parapet wall front.	1.375	5.40	1	7.425	(チ)
〃 〃 rear	2.375	5.40	1	12.825	(リ)
〃 〃 under side	.3	2.40	2	1.440	(ヌ)
Coping front.	.4	7.20	1	2.880	(ル)
〃 sides	.4	.60	2	.480	(ヲ)
front wall. front.	4.87	7.00	1	34.090	(ワ)
〃 Sides	4.87	.625	2	6.088	(カ)
〃 rear	4.17	2.40	2	20.016	(ヨ)
Buttress A, sides	1.725	2.895	4	19.976	(タ)
〃 〃 rear	.80	2.895	2	4.632	(レ)
Column bottom	1.35	.55	2	1.485	(ソ)
Buttress B, Sides	1.20	4.17	2	10.008	(ツ)
〃 〃 rear	.60	4.17	1	2.502	(ネ)
Base. under buttresses	.58	1.95	4	4.524	(ナ)
〃 〃 wall	.71	2.10	2	2.982	(ラ)
〃 〃 buttresses	.95	.50	2	.950	(ム)
〃 〃 〃	.90	.50	1	.450	(ウ)
〃 〃 wall, sides	.50	1.725	2	1.725	(ヰ)
〃 front & rear	.70	7.00	2	9.800	(ノ)
〃 Sides	.70	4.50	2	6.300	(オ)
				188.715 m²	





Reinforcements for Abutment Plain bars 2.743 kg tons for each abutment

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

CALCULATIONS FOR

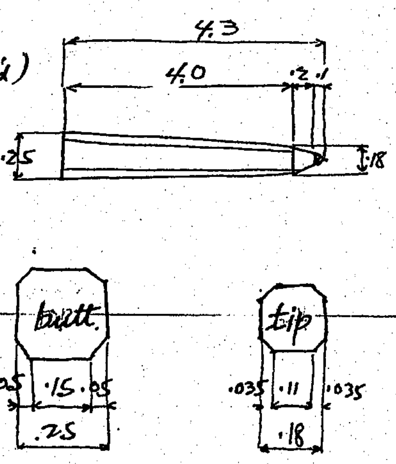
Ashida-Bashi for Okayama Ken.

Materials for Abutment.

蹴込石 花崗石.

$.25 \times 2.0 \times .6 = .030 \times 9 = 0.270\ m^3$ for one abutment

Foundation piles. Reinforced concrete piles $18^{cm\phi}$ at tip 4.3^m long. (28 req'd)



Concrete for pile. Sectional area at butt. $.25 \times .25 = .0625^{\square m}$

less chamfer $.05 \times .05 \times 2 = -.0050$

$.0575^{\square m}$

Sectional area at tip $.18 \times .18 = .0324^{\square m}$

less chamfer $.035 \times .035 \times 2 = -.0025$

$.0299^{\square m}$

mean area $= \dfrac{.0575 + .0299}{2} = .0437^{\square m}$

volume $= 0.0437 \times 4 = 0.175\ m^3$

$.14 \times .14 \times .2 = 0.004$

$\underline{0.179}\ m^3$ for one pile.

Reinforcements for pile plain bars 43.2^{kg} say $\underline{0.043}$ kg tons/pile See drawing.

Form for pile top perimeter $= .25 \times 4 = 1.00^m$ say

bottom " $= .18 \times 4 = .72$

$1.72 \div 2 = .86 \times 4.0 = \underline{3.44}^{\square m}$ for one pile tip neglected.

Cast iron shoe. $\underline{4.7}^{kg}$ per pile. see drawing.

人造洗出仕上.

	Width	Length	req'd. no.	Area	Remarks.
Column front.	1.35	1.725	2	4.66 □m	
parapet wall "	0.4	1.375	2	1.10	
Column front	0.55	2.00	2	2.20	
" Outside	1.35	3.725	2	10.06	
" rearside	1.35	.50	2	1.35	
" top	1.35	1.35	2	3.65	
less light pedestal area	.9	.9	2	1.62	

total area of finish $= \underline{24.64}^{\square m}$ for one abutment

Materials for Light Pedestals. 4 pedestals req'd.

Concrete 1:2:4 mix

	Section	Length	req'd. no.	volume	remarks
bottom	0.9 × 0.9	1.2	1	.972	
intermediate	.78 × .78	.4	1	.243	
top	.58 × .58	2.355	1	.792	
piramid	.38 × .38	.05	1	.007	
"	.38 × .38	.15 ÷ 3	1	.007	
				2.021 m³ for one pedestal.	

Forms.	width	length	req'd. no.	area
bottom	.9 × 4	1.2	1	4.32 □m
intermediate	.78 × 4	.4	1	1.25
top	.58 × 4	2.355	1	5.46
piramid	.38 × 4	.05	1	.08
"	.38 × 4	.15 ÷ 2	1	.11
				11.22 □m for one pedestals.

人造洗出仕上 Say $11.22^{\square m}$

電燈 青銅 一揃 4- 100 watt lamps.

Reinforcements plain bars 0.071 kg ton/pedestal.

Bronze name plate 1枚

Gas pipe 1— $5^{cm\phi} \times 3.85^m$ with branches and accessories.

" 1— $6.3'' \times 1.0^m$

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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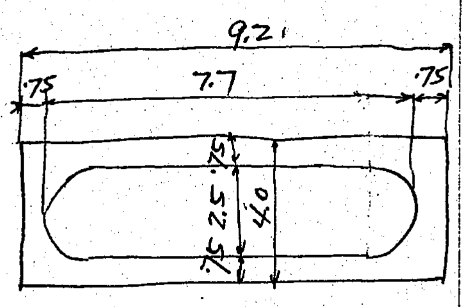
CHECKED BY M. Jinda DATE 3-10-11 PAGE NO 76

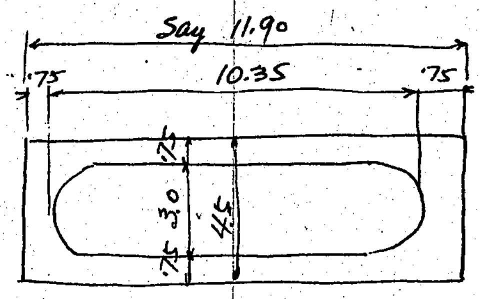
CALCULATIONS FOR

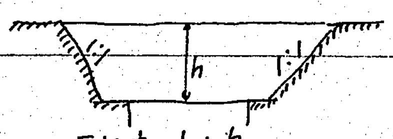
Ashida-Bashi for Okayama Ken.

Excavation for Piers.

Piers	Bottom area of excavation Length	width	Depth	Mean area Length	width	Excavation in cub. meters.	
P1	9.2	4.0	4.70m	13.9	8.7	568.4 m3	
P6	"	4.0	~~4.70~~ 2.50m	11.7	6.5	190.1	} mean 190.1 m3
P11	"	"	2.50	11.7	6.5	190.1	
P7	"	"	3.9	13.1	7.9	403.6	} " 329.9
P10	"	"	3.0	12.2	7.0	256.2	
P8	"	"	3.6	12.8	7.6	350.2	} " 350.2
P9	"	"	3.6	12.8	7.6	350.2	
P12	"	"	2.4	11.6	6.4	178.2	
P13	"	"	1.7	10.9	5.7	105.6	
P14	"	"	2.2	11.4	6.2	155.5	
P15	"	"	0.6	9.8	4.6	27.0	
P16	"	"	0	—	—	—	
P17	"	"	2.4	11.6	6.4	178.2	
P18	"	"	3.5	12.7	7.5	333.4	
P2	"	"	1.0	10.2	5.0	51.0	
P3	11.9	4.5	3.0	14.9	7.5	335.3	} ~~mean 293.7~~ m3
P5	"	"	2.5	14.4	7.0	252.0	
P4	"	"	—	—	—	—	







平均巾 b+h
平均長 l+h
for 2.5m well

for 3.0m well.

Excavation for Abutments. A1 and A2.

Bottom area = 4.5 × 7.0 = 31.50 □m.

mean depth of excavation = 4.9 ~~for both abutments~~.

Excavation = 31.5 × 4.9 = 154.35 cubmeters. ~~for each abutments~~

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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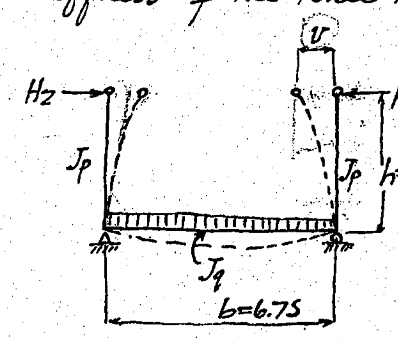
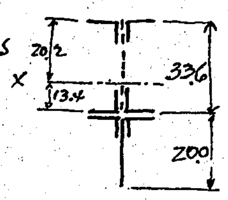
CHECKED BY______ DATE______ **PAGE NO** 1

CALCULATIONS FOR

岡山縣芦田橋

Stiffness of the Knee Brace. Taschenbuch Für Bauingenieure Von M. Foerster. page 397. 参照



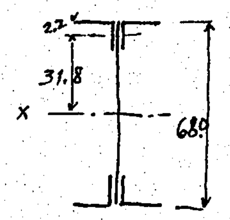


Moment of inertia of Knee Brace with vertical member.

2Ls	90×75×9 = 28.08 cm² × 3.56 =	100.00	
4Ls	〃 = 56.16 × 16.8 =	945.0	
1 Pl.	195×8 = 15.60 × 43.6 =	680	
	99.84 · 20.2 cm	2013.0	

Moment of inertia about x axis

2Ls	$28.08 \times 18.2^2 + 2 \times 68.16 =$	$9330 + 136 = 9466$
2Ls	$28.08 \times 17.4^2 + 2 \times 68.16 =$	$3650 + 136 = 3786$
2Ls	$28.08 \times 15.4^2 + 2 \times 68.16 =$	$6650 + 136 = 6786$
1 Pl.	$15.6 \times 43.6^2 + 495$	$29700 + 495 = 30195$
	99.84	$J_p = 50233$ cm⁴



Moment of inertia of Floor Beam.

4Ls 125×90×10 = 82.0 × 31.8² + 138×4 = 83450

1 Pl. 670×8 = 53.6 × 20000

135.6

$J_q = 103450$ cm⁴

$$\mathcal{M} = \frac{b}{h} \cdot \frac{J_p}{J_q} = \frac{6.75}{3.445} \times \frac{50233}{103450} = 0.95$$

$$v_0 = \frac{h^3}{3EJ_p} = \frac{344.5^3}{3 \times 2100000 \times 50233} = 0.00013$$

$$v_1 = H_1\left(\frac{h^3}{3EJ_p} + \frac{h^2 b}{3EJ_q}\right) = H_1 \cdot \frac{h^3}{3EJ_p}\left(1 + \frac{b}{h}\frac{J_p}{J_q}\right) = H_1 \cdot v_0 \cdot (1 + \mathcal{M}) = H_1 \times 1.95 \times 0.00013 = 0.000123\,H_1.$$

$$v_2 = H_2 \frac{h^2 b}{6EJ_q} = H_2 \cdot v_0 \frac{\mathcal{M}}{2} = H_2 \times 0.00013 \times \frac{0.95}{2} = 0.000062\,H_2$$

$$v_3 = \frac{M\,bh}{3EJ_q} = \frac{M}{h}\frac{h^3}{3EJ_q} \cdot \frac{b}{h} \cdot \frac{J_p}{J_q} = \frac{M}{h} \cdot v_0 \cdot \mathcal{M} = \frac{M}{344.5} \times 0.00013 \times 0.95 = 0.000036\,M. \quad M \text{ in kgm.}$$

$$v = v_0\left[H_1 + \mathcal{M}\left(H_1 + \frac{H_2}{2} + \frac{M}{h}\right)\right] = (v_1 + v_2 + v_3) = 0.000123\,H_1 + 0.000062\,H_2 + 0.000036\,M.$$

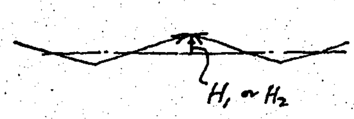
Case 1. Wind Load — 200 kg per lin meter of unloaded chord.

panel load $H_1 = 200 \times 3.75 = 750$ kg

$v_1 = 0.000123\,H_1 = 0.000123 \times 750 = 0.092$ cm. ——— (1)

approx. stress on knee brace. $= \dfrac{750 \times 344.5 \times 33.4}{50233} = 172$ kg/cm².

Case 2. Eccentric loading



Max. live load stress on top chord member = 61100 kg = S

assume $H_1 = H_2 = \frac{1}{200} S = 305$ kg.

$v_1 = 0.000123\,H_1 = 0.000123 \times 305 = 0.038$ cm ——— (2)

$v_2 = 0.000062\,H_2 = 0.000062 \times 305 = 0.019$ cm ——— (3)

fibre stress = $172 \times \frac{305}{750} = 70$ kg/cm.

Case 3. Moment of Floor Beam.

Max Live Load moment on Floor Beam = 18220 kgm.

$v_3 = 0.000036\,M = 0.000036 \times 18220 = 0.655$ cm. ——— (4)

Summary.

		Deformation	Fibre stress of Knee brace.
Case 1	wind load.	0.092	172
Case 2	Eccentric load.	{0.038, 0.019}	70
Case 3.	moment on Floor Beam.	0.655	
Total		0.804 cm.	242 kg/cm²

or 0.317" 0.317"

or 5/16 inch — or 3,440 %.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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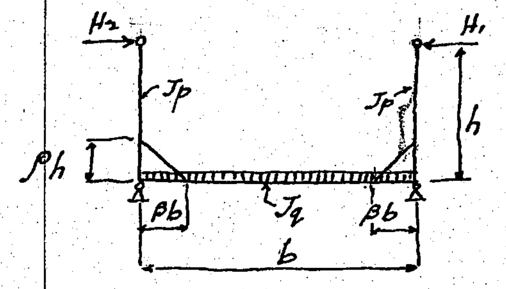
CALCULATIONS FOR

岡山縣芦田橋

Knee Brace with Bracket



$\rho h = 0.40^{m}$ or $\rho = \frac{0.40}{3.445} = 0.116$ $\quad 1-\rho = 0.884 \quad (1-\rho)^2 = .782$

$\beta b = 0.82^{m}$ or $\beta = \frac{0.82}{6.75} = 0.122$ $\quad 1-\beta = 0.878 \quad (1-\beta)^2 = .770$

$$v = v_0\left[H_1(1-\rho)^2 + \mu\left\{H_1(1-\beta)^2 + \frac{H_2}{2}(1-2\beta^2) + \frac{M}{h}(1-2\beta^2+\beta^3)\right\}\right]$$

$H_1 = 305 + 750 = 1{,}055^{kg}$ $\quad \mu = 0.95$

$H_2 = 305$ " $\quad v_0 = 0.00013$

$M = 18220^{kgm}$

$$v = 0.00013\left[1055 \times 0.782 + 0.95\left\{0.77 \times 1055 + \frac{305}{2}(1-2\times0.122^2) + \frac{18220}{3.445}(1-2\times.122^2+.122^3)\right\}\right]$$

$= 0.718^{cm}$

or $9/32'$ inch