H. TASHIMA

CALCULATIONS FOR

MADE BY H. DATE FILE NO.
CHECKED BY DATE PAGE NO.

昭和八年二月

上塚橋計算書

愛知縣

H. TASHIMA

MADE BY H. Tashima DATE 8-1-17 **FILE NO.**

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CALCULATIONS FOR

Design for Kamitsuka-Bashi, Aichi-ken

The Present bridge is made of wooden beam span having 450. meter between abutments. The proposed new-bridge is located at 55. meters up stream at right abutment. The total length of the new bridge is 450.62 meters between faces of parapet walls of both abutments.
The total width of roadway is 5.50 meters wide between curb lines, paved with soliditit or granolithic concrete on reinforced concrete slab.
The grade of the bridge shall be parabolic curve throughout the structure giving the total camber of 75 cm at the centers of bridge

final layout we adopted the follows

span length next to abutment	2 @	20.46 =	40.92^{m}
suspended spans	7 @	14.390 =	100.73^{m}
cantilever arms	14 @	6.445 =	90.23
intermediate spans	8 @	27.280 =	218.16
		total length ct.c. of bearings =	450.12 meters

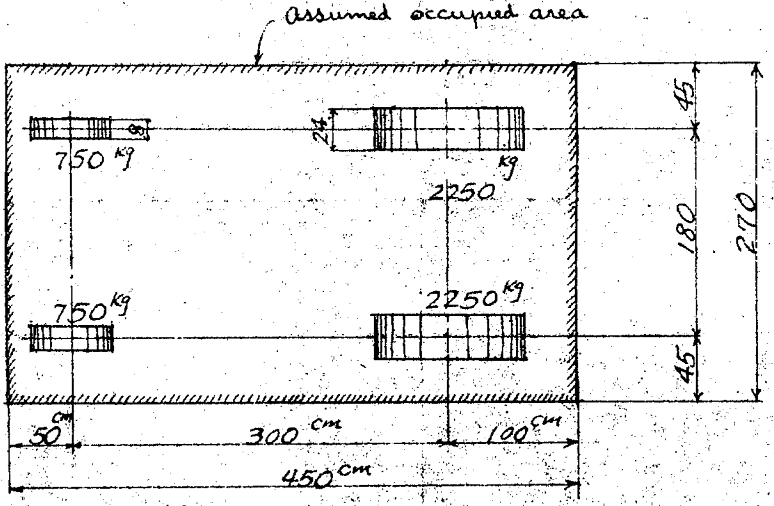
{ 2 suspended span @ 14.015
2 cantilever arms @ 6.445

Assumed Loadings

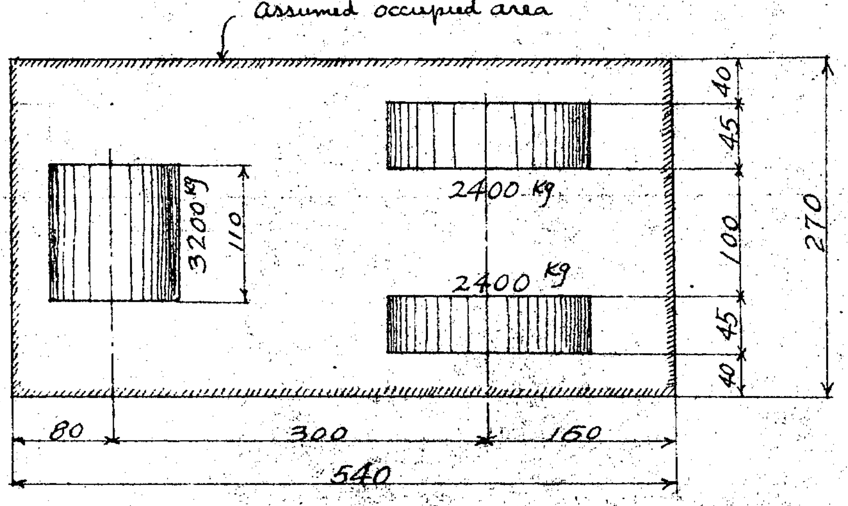
uniform load on Roadway $w = \frac{100.000}{170+l} \leq 500$ kg/m²

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

6 Ton motor truck loading



8 Ton Road Roller loading



2 rows of motor traffic on roadway with occupied width of 270 cm each, unoccupied space shall be filled with uniform load specified above.
One Road roller on one span.
Impact for motor truck loading Coef = $\frac{20}{60+l}$ where l = loaded length in meter
max. impact 30%

No impact considered for road roller and uniform load.

Allowable working strength

concrete 1:2:4 mixture

Direct Compression	35 kg/cm²
Fibre stress due to bending	45
Combined stresses due to direct and bending, compression member	35
punching shear of concrete	9
Shear of plain concrete	4
Bearing value	45

Reinforcing bars

Tension or Compression	1200
shearing strength	900

H. TASHIMA

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CALCULATIONS FOR

Design of Kamitsuka-Bashi for Aichi-Ken

structural steel

Tension net ------------------------------ $1200\ ^{kg}/_{cm^2}$

Extreme fibre stress, net ------------------------------ 1200 "

shear of web, gross section 900 "

Compression member

$1500\,(1-0.0055\ l/r)$ not over 1.000 "

where l = length of member in cm

r = least radius of gyration in cm

Compression flange of girder

$1200\,(1-0.012\ l/b)$ not over 1.100 "

where l = Unsupported length of flange in cm

b = width of flange in cm

Shear on shop driven rivets (machine driven) 850

field " , and turned bolts (machine driven) 750 "

Shear on pin ------------------------------ 900 "

Bearing on shop driven rivets (machine driven) ---------- 1700 "

" " field " " 1500 "

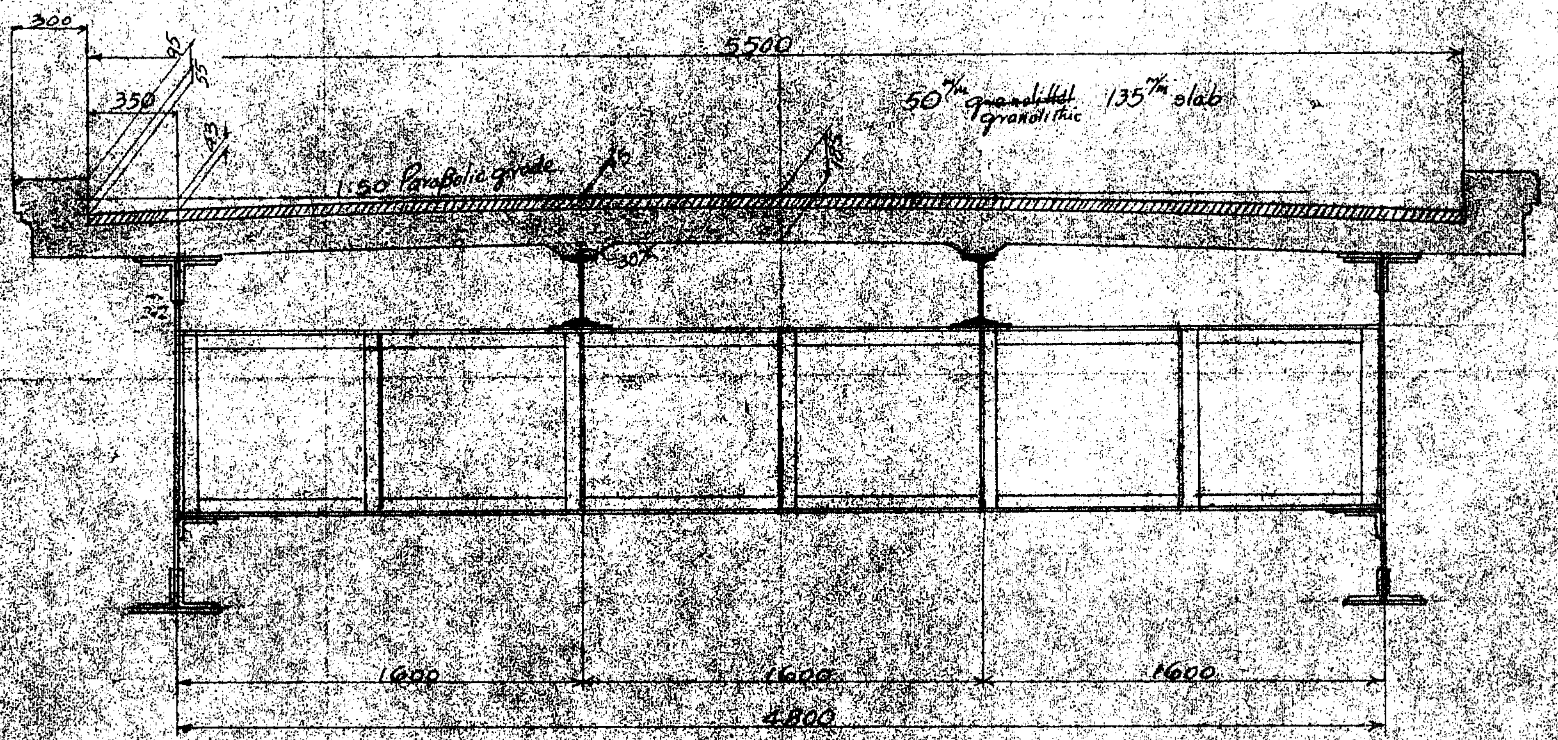
pin ------------------------------ 1800 "

Roller (steel) $45d\ ^{kg}/_{cm}$ where d = diameter of roller in cm

Considering wind or temperature stress in addition to dead, live and impact stresses, the working stresses shall be increased 25% ; in case of earthquake increase working strength 60%

Seismic acceleration $2000\ ^{mm}/_{sec^2}$ or $k = 0.20$

Cross section of Bridge assumed as shown on sketch below.



Design of Kamitsu Ka-Bashi for Aichi-Ken

Design of floor slab span length 1.60 meters

Dead Load

$50^{m/m}$ granolithic or soliditit	@ 220^{kg}	=	110^{kg}
13.5^{cm} slab concrete	@ 240	=	324
fillet say			16
			$450^{kg/m^2}$

Dead load moment $= 1/10 \times 450 \times 1.60^2 = 115^{kgm}$

Dead load shear $= 1/2 \times 450 \times 1.60 = 360^{kg}$

live load motor truck loading

Rear wheel load 2250^{kg}

impact 30% 675

2925^{kg}

Front wheel load $= 2925 \div 3$

$= 975^{kg}$

Distribution of wheel concentration on slab

longitudinal distribution a = contact between wheel and pavement $= 20^{cm}$

distribution 2@50 = 10

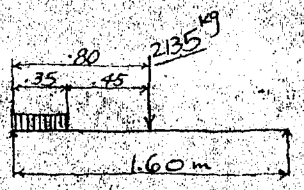
30^{cm}

Transverse distribution b = 24 + 10 = 34cm

Effective width $\mathcal{E} = 2/3\,l + a$

$= 2/3 \times 1.60 + .30 = 1.37$ meters

load per meter strip $2925 \div 1.37 = 2135^{kg}$

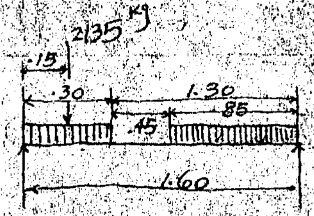


moment per meter strip $= \frac{2135}{2} \times 0.80 = 854^{kgm}$

for continuity of slab $854 \times 0.80 = 683^{kgm}$

uniform load $\frac{500 \times 0.35^2}{2 \times 1.60} \times 0.80 = 15$

698 kgm for safe side



max end shear $\frac{2135 \times 1.45}{1.60} = 1935$

uniform load $\frac{500 \times 0.85^2}{2 \times 1.60} = 113$

2048^{kg}

summary for moments and shears

	moment	shear
Dead load	115	360
live load	698	2048
	813 kgm	2408 kg

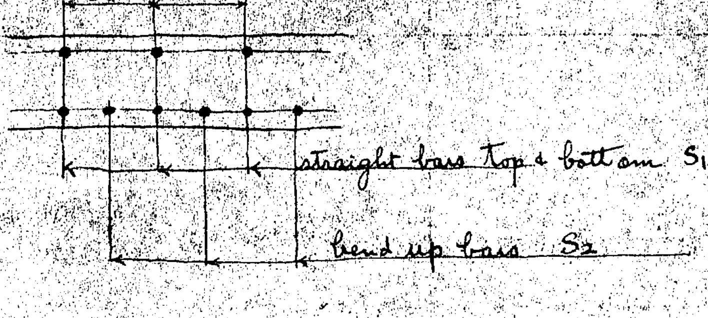
Effective depth required for $f_s = 1200^{kg/cm^2}$ & $f_c = 45^{kg/cm^2}$

$R = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{b.d}}$ where R = 7.18

$d = \sqrt{\frac{813 \times 100}{100 \times 7.18}} = 10.6cm$ use 13.5 cm slab with insulation at bottom of 2.5cm

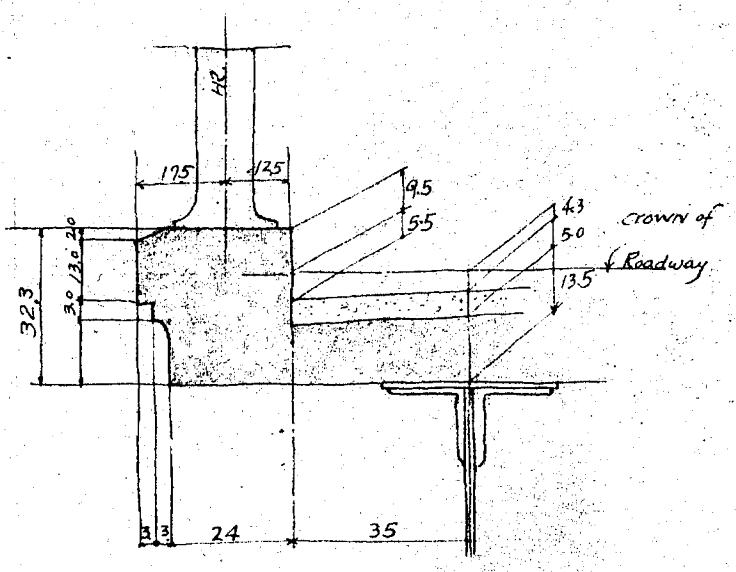
steel area req'd $= \frac{813 \times 100}{7/8 \times 11.0 \times 1200} = 7.04cm^2$ per meter strip

spacing of $12^{m/m}$ bars $\frac{1.13 \times 100}{7.04} = 16.^{cm}$ Use 15 cm spacing



Design of Kamitsuka-Bashi for Aichi-Ken.

Over hanging slab beyond outside girder



approximate weight of Handrails 70^{kg} per lin meters.
" " " Coping 192 " " "

Dead load. load
Handrails .... 70 × 0.475 = 33.3
Copings .... 192 × 0.490 = 94.1
slab & pavement 450 × 0.35 = 157.5 × 0.175 = 27.6
419.5 0.37 = 155.0

live load. motor truck rear wheel at curb line assumed
Distribution or effective width on ℄ stringer assumed thus
2 × 0.35 + 0.20 = 0.90
Transverse distribution 24 + 10.0 = 34 cm
load on 30 cm $2925 \times \frac{30}{34} = 2580^{kg}$.
For one meter strip $2580 \div \overset{0.90}{\cancel{0.80}} = \overset{2870}{\cancel{3225}}$ kg

live load moment = $\overset{2870}{\cancel{3225}} \times 0.175 = 502$ Kgm
" " shear = 2870 kg.

Summary for moments and shears

	moment	shear
Dead load	155	420
Live load	502	2870
	657 Kgm	3290 Kg.

Effective depth required $d = \sqrt{\frac{657 \times 100}{100 \times 7.18}} = 9.5$ cm

use 13.5 cm slab effective depth say = 10.5 cm

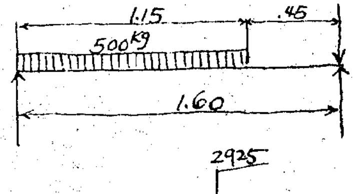
unit shear = $\frac{3290}{7/8 \times 100 \times 10.5} = 3.58$ kg/cm² O.K.

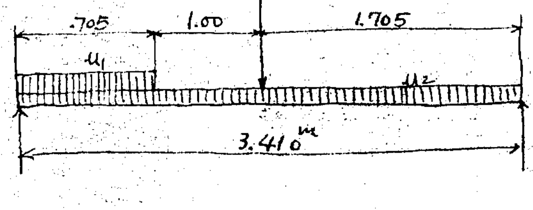
Design of I beam stringer span length 3.410 meter

floor slab and pavement 450 × 1.60 = 720
Stringer assumed 45
765 kg

Dead load moment = $1/8 \times \overset{765}{\cancel{770}} \times 3.41^2 = 1110$ kgm
" " shear = $1/2 \times 765 \times 3.41 = 1300$ kg

live load.
motor truck rear wheel with impact = 2925^{kg}
uniform live load 500 kg/m²





load on stringer = $\frac{500 \times 1.15^2}{2 \times 1.60} = 207^{kg}$ ---- u_2
full load 500 × 1.60 = 800
less − 207
593 kg ---- u_1.

moment
Due to motor truck $\frac{2925}{2} \times 1.705 = 2494$
Uniform load u_1 $\frac{593 \times 0.705^2}{2 \times 3.410} \times 1.705 = 72$
Unif. load u_2 $1/8 \times 207 \times 3.410^2 = 300$
2866 Kgm

End shear.
unif. load u_1 $\frac{593 \times 2.41^2}{2 \times 3.410} = 505$
" " u_2 $207 \times 1.705 = 353$
motor truck loading 2925
3783 kg.

Design of Kamitsuka-Bashi for Aichi-Ken

summary for moments and shears

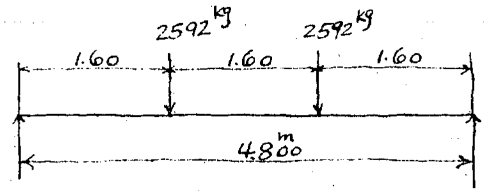
	moment	shear
Dead load	1110	1300
live load.	2866	3783
	3976 Kgm	5083 Kg.

section modulus req'd = $\frac{397600}{.1100}$ = 361.4 cm³

Use 250×125 I @ 38.3 kg/m S.m = 414.9 cm³

Design of intermediate floor beams. span length 4.80 meter

Dead load concentration on stringer 760 × 3.410 = 2592 kg



moment = 2592 × 1.600 = 4150

floor beam say 100 kg/meter assumed

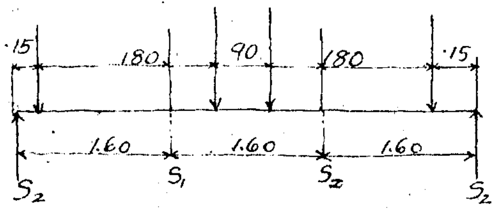
$m = \frac{1}{8} \times 4.80^2 \times 100$ = <u>288</u>

4438 kg m

End shear = 2592 + ½ × 100 × 4.80 = 2832 kg

live load. motor truck loading rear wheel concentration with impact = 2925 kg

front wheel " " " = 975

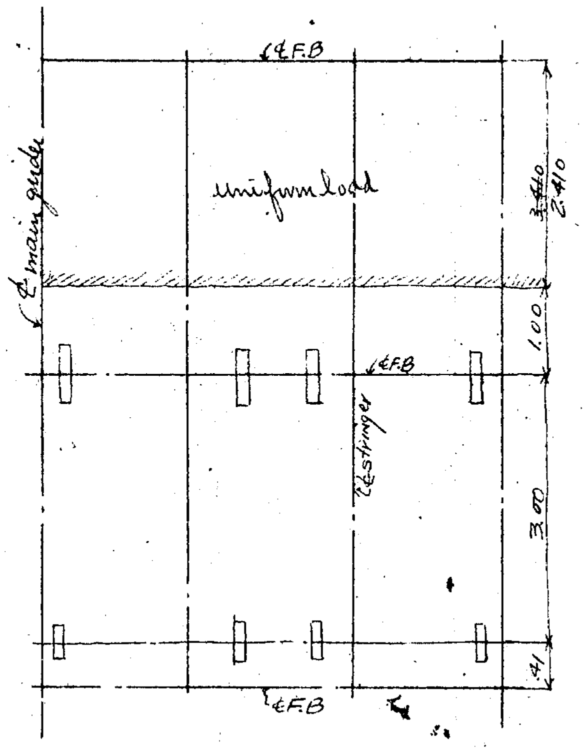


load on S₁ $\frac{0.15}{1.60}$ = 0.093

$\frac{1.000}{1.093}$ × 2925 = 3200 kg

" × 975 = 1065 "

uniform live load 500 × 1.60 = 800 kg on stringer



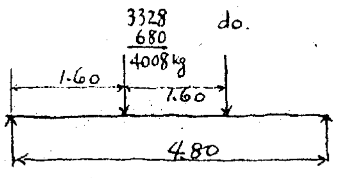
Concentration on floor beam

motor truck loading rear wheel 3200

front wheel 1065 × $\frac{0.41}{3.41}$ = <u>128</u>

3328 kg.

Uniform load. $\frac{800 \times 2.41^2}{2 \times 3.41}$ = <u>680</u> kg.



moment = 4008 × 1.600 = 6410 kgm

shear = 4008 kg.

summary for moments and shears

	moment	shear
Dead load	4438	2832
live load.	6410	4008
	10,848 kgm	6840 kg.

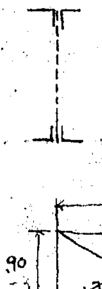
depth assumed 70 cm chord stress = $\frac{10,848}{0.90}$ = ~~15,500~~ 12,050 kg

section required = ~~15,500~~ 12,050 ÷ 1200 = ~~12.92~~ 10.04 cm²

Try 2Ls 75×75×9 = 25.38 cm²

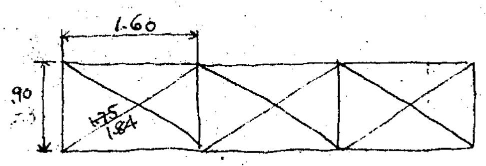
− <u>2×1.98</u>

21.42 cm²



Diagonals shear 6840 kg

stress in diagonal 6840 × $\frac{1.85}{0.90}$ = ~~17,176~~ 14,090 kg



S.R = $\frac{~~17.17~~ \; 14.09}{2}$ = ~~8.6~~ 7.05 cm² use 1-L 75×75×9 = 12.69 cm²

no of rivet 14,090 ÷ 2126 = 6.6 4 Rivets for one angle, field.

1.60² = 2.56

0.90² = ~~0.49~~ 0.81

~~3.05~~ = ~~1.75~~

3.37 = 1.84

Design of Kamitsuka-Bashi for Aichi-Ken.

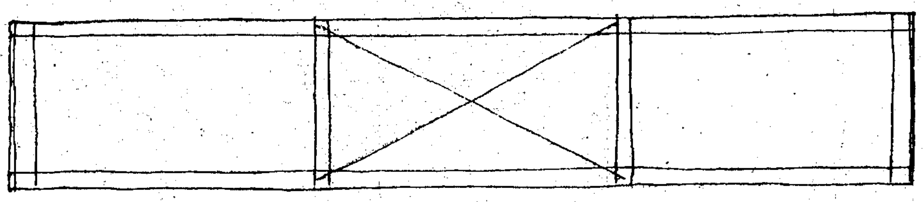
approximate weight of intermediate F.B.

Top and Bottom angles	4Ls	75×75×9	@9.96^{kg}	× 4.80	=	191^{kg}
Verticals	4Ls	75×75×9	@ 〃	× 0.90	=	36
end stiffs.	4Ls	90×90×10	@13.3	× 0.90	=	48
Diagonals	6Ls	75×75×9	@9.96	× 1.80	=	108
connection pls	4Pls		@ 15^{kg}		=	60
END Connection Plate	2Pls	800×8	@ 50.2 × 0.90		=	90
	misc					20
						553^{kg}.

553 ÷ 4.80 = 115 kg per lin meter of cross beam

553 ÷ 3.41 = 162 kg per 〃 〃 〃 span.

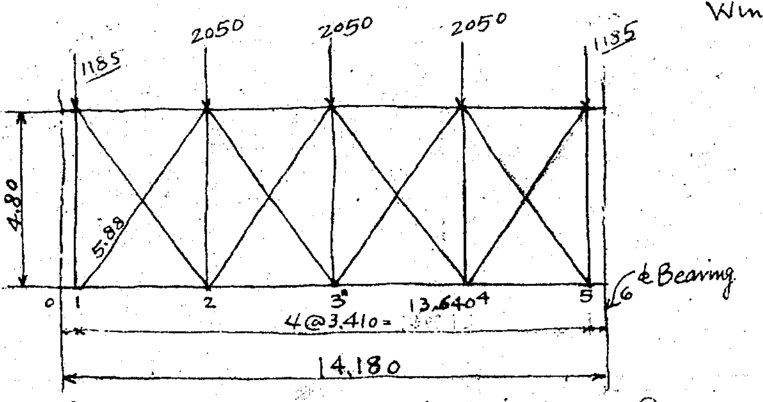
End floor beam



approximate weight

4Ls (Top & Bottom chords)	75×75×9	@ 9.96×4.80	=	191.
END stiffs. 4Ls	90×90×10	@ 0.90×13.3	=	48
Verticals 4Ls	75×75×9	@ 9.96×0.9	=	36
2 web pls	1900×8	@ 119.3 × 0.9	=	215
diagonals 2Ls	75×75×9	@9.96×1.80	=	36
Center Connections	1 @			–
	misc			18
				544. kg.

lateral Bracing for suspended span.



Wind load loaded chord 400

unloaded 〃 200

panel load 600^{kg} × 3.41 = 2046^{kg} say 2050^{kg}

3.41^2 = 11.628

4.80^2 = 23.040

34.668 = 5.88 --- diagonal length

sec θ = 5.88 ÷ 4.80 = 1.225

R = 1185 + 1.5×2050 = 4260 Kg.

Diagonal stresses

panel	shear		stress		R. area (Net)	Rivet No (19φ)	use	adopted section
0-1.	4260	× 1.225 =	5.220^{kg}	÷1200 =	4.35$^{cm^2}$	2.46	3	1-L 75×75×9 = 10.71$^{cm^2}$ Net
1-2.	3075	× 〃 =	3.770	〃	3.14 〃	1.77	3	do
2-3	1580	× 〃	1.940	〃	1.61 〃	0.91	3	do.

approximate weight

8Ls 75×75×9 @ 9.96 × 5.50	=	438
Center connections @ $\overset{25}{15}$ Kg×4	=	100
		538
misc. say		18
		556 Kgs

556 ÷ 14.180 = 39.2^{kg}/meter of span

panel load = 39.2 × 3.41 = 133.7 Kg.

H. TASHIMA

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CALCULATIONS FOR

Design of Kamitsu Ka-Bashi for Aichi-Ken

Main Gerder for suspended span.

span length 14.180 meters between Bearings

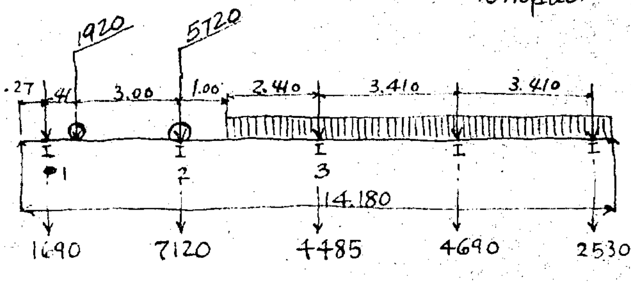
Dead load.
floor slab & pavement 450 × 5.50 = 2475
Coping, ~~Handrails & over~~ 2@ 192 = 384
concrete under coping —
Handrails 2@ 70 = 140
2999 say 3000 kg

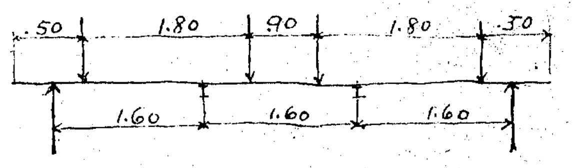
structural steel
stringers 2@ 40 kg/m = 80
floor Beam ------ = 162
lateral bracing ------ = 40
girders say 2@ 380 = 760
1042 ---------- 1042
4042 ÷ 2 = 2021 kg
Call this 2020 kg

dead L. moment = $1/8 \times 2020 \times 14.180^2$ = 50,770 kgm at center
Dead load shear = $1/2 \times 2020 \times 14.180$ = 14,320 kg at end.

live load motor truck loading
rear wheel load --- 2250 kg
impact $\frac{20}{60+14.18} \doteq 27\%$ = 610
2860 kg.

Front wheel 2860 ÷ 3 = 960 kg.





Reaction on main girder due to rear wheel = 2@ 2860 = 5720 kg
" " " " front " = 2@ 960 = 1920 "

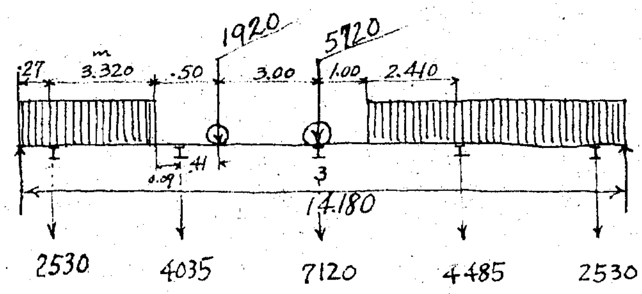
uniform live load on front or rear side of truck = $\frac{5.50}{2} \times 500 = 1375$ kg/m

$\frac{2.41^2 \times 1375}{2 \times 3.41} = 1170$, 2.41 × 1375 = 3310.

3.410 × 1375 = 4690

moment at ② Reaction
1690 × 13.91 = 23.500
7120 × 10.50 = 74.760
4485 × 7.09 = 31.800
4690 × 3.68 = 17.260
2530 × 0.27 = 680
148,000 ÷ 14.180
R = 10,440 kg.

moment = 10.440 × 3.68 = 38,420
less 1690 × 3.41 = − 5,760
32,660 kgm



Reaction
4035 × 10.50 = 42,370
2535 × 13.91 = 35,260
7120 × 7.09 = 50,480
4485 × 3.68 = 16,500
2530 × 0.27 = 680
R. 145,290 ÷ 14.180 = 10,245 kg

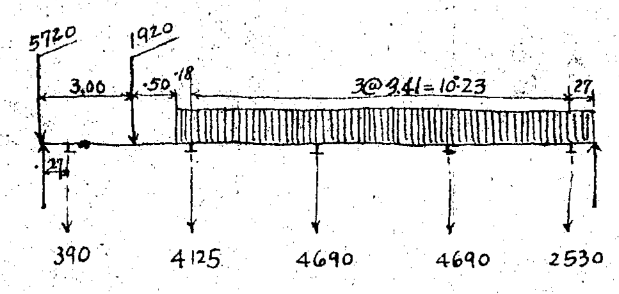
moment at 3 (center) = 10,245 × 7.09 = 72,640
less 2530 × 6.82 = 17,250
4035 × 3.41 = 13,760
− 31,010
M = 41,630 kgm

summary of moment = 41,630 + 50,770 = 92,400 kgm at center

at 2 pt. = 32,660 + 39,020 = 71,680 "

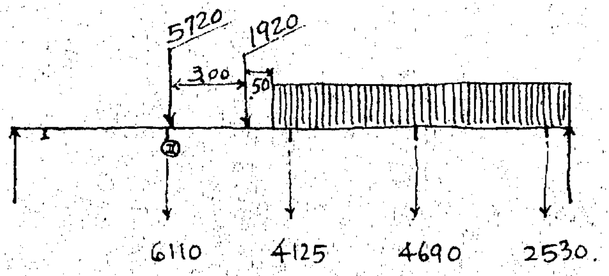
② Design of Kamitsuka-Bashi for Aichi-Ken

End max shear



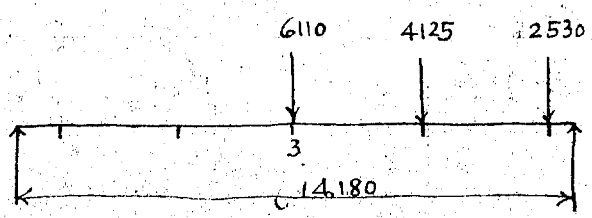
Reaction.
390× 13.91 = 5,420
4125× 10.50 = 43,310
4690× 10.77 = 50,710
2530× 0.27 = 680
R= 100,120 ÷ 14.18 = ~~70,600~~ 7,060 Kg.
Direct Load = 5,720
12,780 kg.

max shear at II pt.



Reaction
6110 × ~~13~~10.50 = 64,160
4125 × 7.09 = 29,250
4690 × 3.68 = 17,260
2530 × .27 = 680
R= 111,350 ÷ 14.18 = 7,880 kg

max shear at center

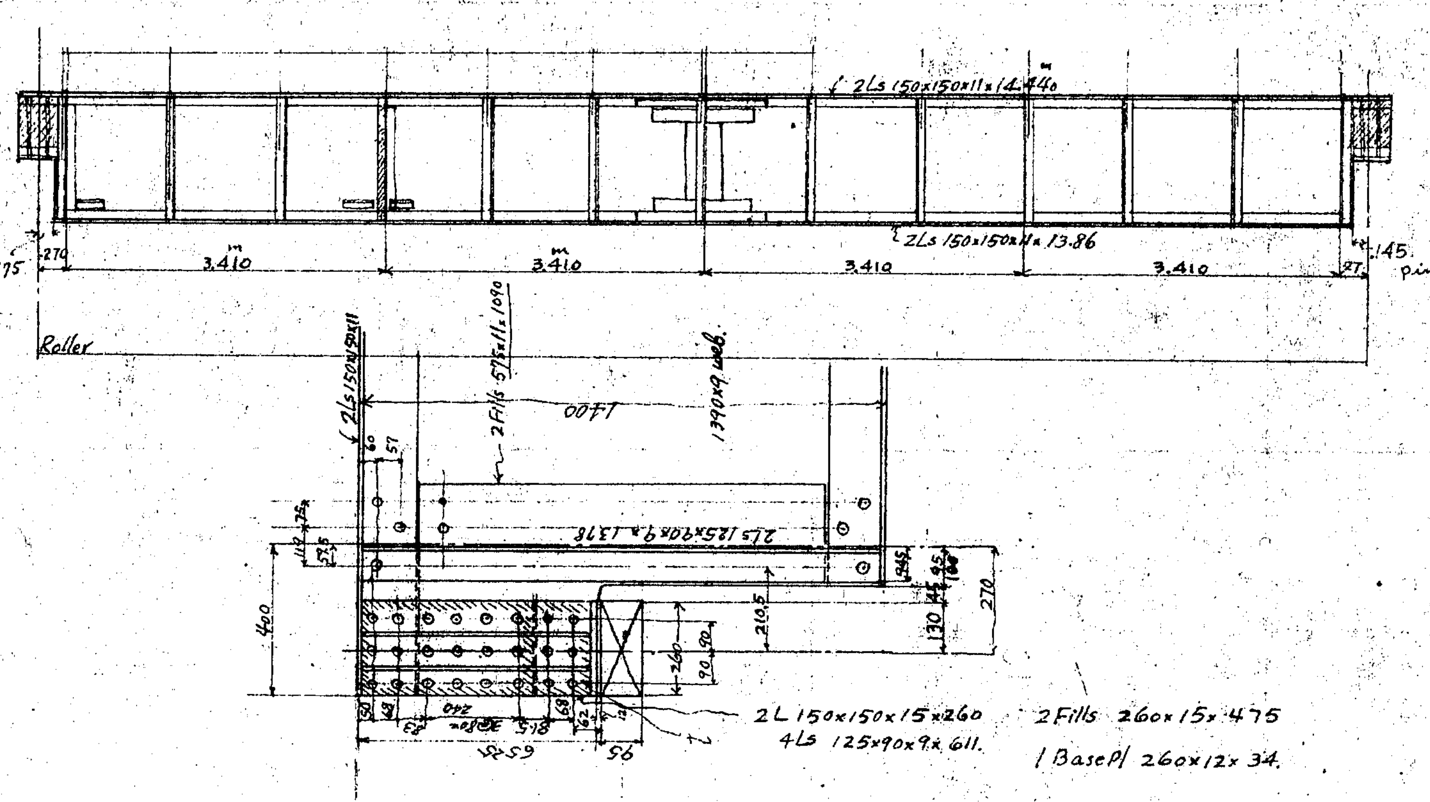


Reaction
6110 × 7.09 = 43,320
4125 × 3.68 = 15,180
2530 × 0.27 = 680
R= 59,180 ÷ 14.18 = 4,170 kg.

summary of shear.

	dead load		live load		sum
at end	14,320	+	12,780	=	27,100 kg.
at II pt	6,890	+	7,880	=	14,770 "
at center (III)	0	+	4,170	=	4,170 "

Section of Main Girder:- web assumed 1400 × 9. = 126 cm² 1/8 web area = 15.7 cm²
back to back of Ls = 1,410 meters effective depth = 132.86 cm
flange stress = 92,400 ÷ 1.328 = 70,000 Kg
section required = 70,000 ÷ 1200 = 58.330
− 15.700
42.63 cm² Net
use 2Ls 150×150×11 = 63.56 − 5.5 = 58.06 cm² Net. throughout the span.



Approximate weight of Main girder

2 Flg Ls	150 × 150 × 11	@	31.78 kg	× 14.440 m	=	918
2 "	" " "		"	× 13.86	=	881
26 stiffners	125 × 90 × 9	@	14.60	× 1.380	=	524
4 Fills Pls	200 × 11	@	17.27	× 1.090	=	75
18 Fill Pls	90 × 15	@	10.60	× 1.090	=	208
4- End Fill Pls	260 × 15	@	30.61	× 0.475	=	[illegible] 58
End 2Ls × 2 = 4Ls -	150 × 150 × 15	@	33.60	× 0.260	=	35
do 4Ls × 2 = 8Ls ...	125 × 90 × 9	@	14.60	× 0.611	=	71
~~2 Fill Pls~~ II Pls	260 × 12	@	24.49	× 0.034	=	33
4- End Con. Pls	575 × 11	@	49.65	× 1.090	=	216
4- splice Ls	150 × 150 × 15	@	33.60	× 1.200	=	161
4 splice pls	160 × 11	@	13.81	× 0.950	=	52
2 splice pls	330 × 11	@	28.50	× 0.770	=	44
8- self Ls	~~100~~ 125 × 90 × ~~10~~ 9	@	14.60	× 0.310	=	36
3- gusset pls			30 kg		=	90
2 " "			20 "		=	60
Rivet head and variation						165
1- web Pl	1390 × 9	@	98.2	× 14.44	=	1418
						5,045 kg.
misc say						100
						5,145 kg

5,145 ÷ 14.18 = 363 kg/meter of span.

Summary for structural steel in suspended span

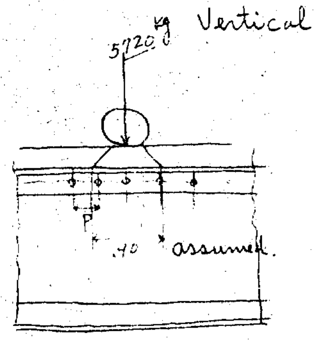
stringers	2 @ 40 = 80 kg × 14.40	=	1152
floor beam	5 @ 553	=	2765
lateral bracings			556
main girders	2 @ 5,145	=	10,290
(Expansion joint)			
			14,763 kg.

14,763 ÷ 14.18 = 1,040 kg per meter

Rivet spacing for suspended main girders.

Horizontal increment of flange stress $= H.I = \frac{\nabla}{h} \times \frac{A_f}{A_f + 1/8\,web}$ (where ∇ = end shear, h = effective depth)

at end $H.I. = \frac{27.100}{132.8} \times \frac{63.56}{63.56+15.7} = \frac{1722,480}{10,526} = 163$ kg per lin cm



Vertical load per linier meter per girder is as follows.

Dead load. 2020 kg/meter

live load. $5720 \div \overset{0.90}{\cancel{0.57}} = \overset{6,360}{\cancel{10,035}}$ kg/meter } 8380 kg/m

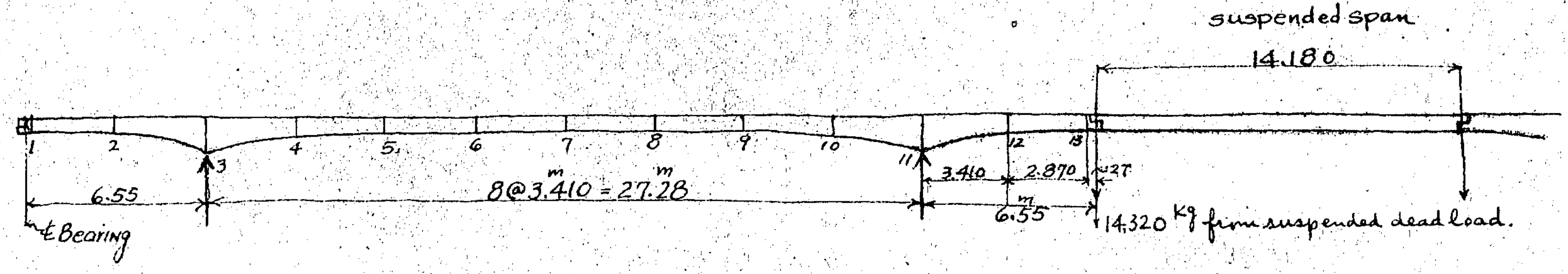
Vertical load per lin c.m = 83.8 kg say 84 kg

The resultant increment $= \sqrt{\overline{83.8}^2 + \overline{163}^2} = 183$ kg per lin cm.

max allowable rivet pitch at end of girder $= \frac{3360}{183} = 18.3$ cm (22 φ bearing)

Design of intermediate anchor span.

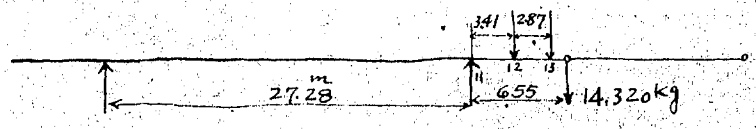
Dead load.



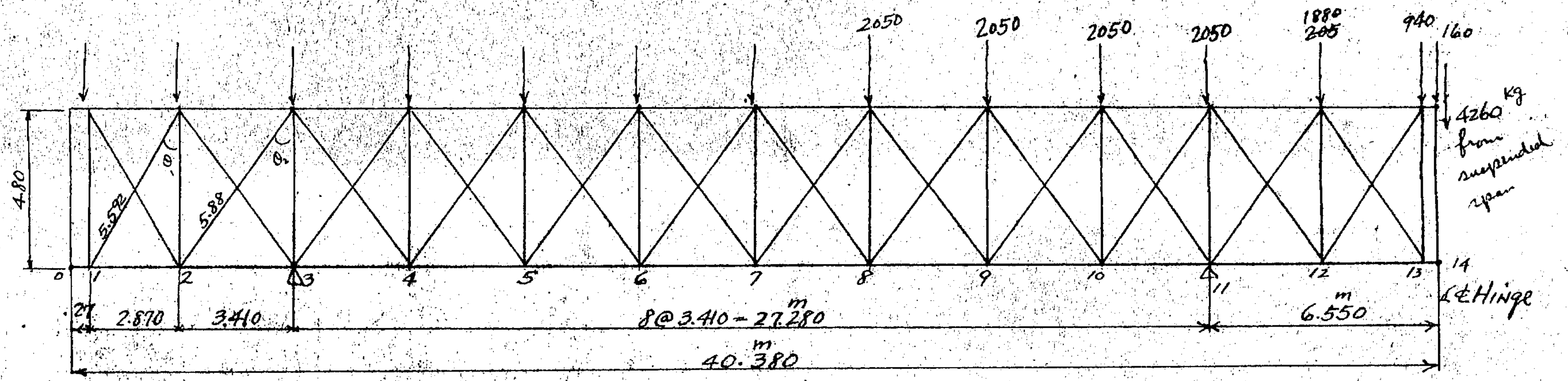
Cantilever arm :- length of arm = 6.55 meters

Dead load of main girder assumed as 520 kg/meter mean

floor slab & pavement, Coping, H.R = 1500 kg/m for one side



Bottom Lateral Bracing for anchor span



Wind load. loaded chord 400

unloaded " 200

600 kg.

Panel wind load 600×3.41 = 2046 say 2050 kg.

" " " for 2pt 600× 3.14 = 1880 Kg

" " on ⊕1 600 × 1.57 = 940 "

$3.41^2 = 11.628$
$4.80^2 = 23.040$ } $34.668 = 5.88^2$

~~$3.41^2 = 11$~~ $2.870^2 = 8.2369$
$4.80^2 = 23.0400$ } $31.2769 =$

$\sec\theta_1 = 5.592 \div 4.80 = 1.165$

$\sec\theta_2 = 5.88 \div 4.80 = 1.225$.

stress in diagonals

12 - 14	(4260+160) × 1.165	= 5,150
	940 × "	= 1.095
	5360	6,245 kg ÷ 1200 = 5.20 cm² Net req'd area. Rivet 19 m/m ϕ 3.

11 - 12. 7240 × 1.225 = 8,870 ÷ 1200 = 7.39 cm² Net req'd area. Rivet · 4.17.

Positive Shear at 11 pt

Reaction at 11.

Due to suspended span		4260
cantilever action	$\overset{6.55\times}{4260} \div 27.28$	1020
	160 × 6.55 = 1050	
	940 × 6.28 = 5.900	
	1880 × 3.41 = 6.400	
	13,350 ÷ 27.28 = 490	
Cantilever load		2980
anchor span 3.5@2050 =		7175
	R_{11} =	15,925 kg

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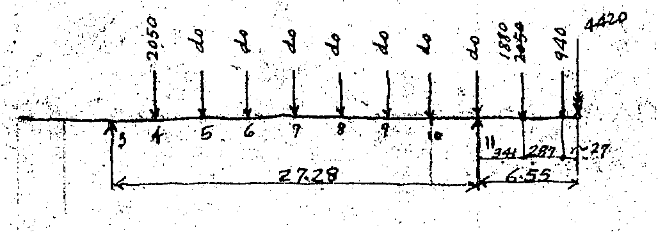
CALCULATIONS FOR

shear at 10 pt.



Reaction at 11.

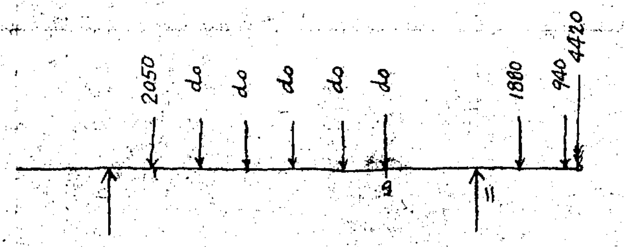
due to cantilever effect 8750

due to anchor span 7175

R_{11}= 15,925

shear = 15,925 - 7240 = 8685 kg.

shear at 9 pt



Reaction

cantilever effect 8750

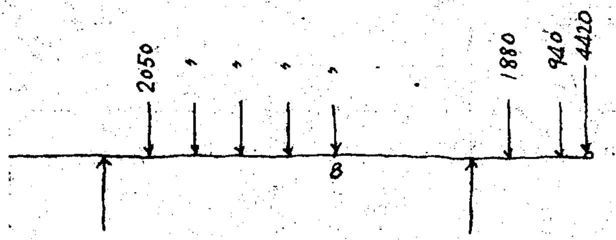
anchor span

2025 x 71.61 = 145,000 ÷ 27.28 = 5320

R= 14,070 kg

shear= 14090 - 7240 = 6,830 kg.

shear at 8 pt



Reaction

cantilever effect 8750

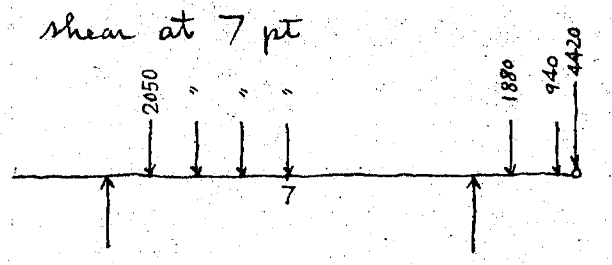
anchor span

2050 x 51.15 = 104,860 ÷ 27.28 = 3840

R= 12,590

shear = 12,590 - 7240 = 5,350 kg

shear at 7 pt



Reaction

cantilever effect ---- 8750

anchor span

$\frac{2050 \times 34.10}{27.28}$ = ---- 2560

R= 11,310 kg.

shear = 11,310 - 7240 = 4070 kg.

Diagonal stresses:-

panels	shear	secθ	stresses			R. Area □cm	Req'd Rivets (19m/mφ)	Used No.	adopted sections
Hinge ~ 13	4420	—							
13 ~ 12	5360	1.165	6245	÷1200	=	5.20	3.0	3	1-L 125x75x10 = 19.0 - 2.2 = 16.8 cm²
12 ~ 11	7,240	1.225	8870	"	=	7.39	4.17	5	do
11 ~ 10	8,685	"	10,600	"	=	8.83	~~4.00~~ 5.00	5	do
10 ~ 9	6,830	"	8,370	"	=	6.98	3.94	4	1-L 75x75x9 = 10.71 cm² Net
9 ~ 8	5,350	"	6550	"	=	5.46	3.07	4	do
8 ~ 7.	4,070	"	4990	"	=	4.16	2.35	3.	do.

approximate weight of lower lateral bracings

12Ls 125x75x10	@ 14.91 x 5.50	=	984
12Ls 75x75x9	@ 9.96 x 5.50	=	896 657
Center connections	12 @ 18 kg	=	216
Rivet heads		=	65
			1922 kg.

1922 ÷ 40.38 = 47.6 kg per lin meters

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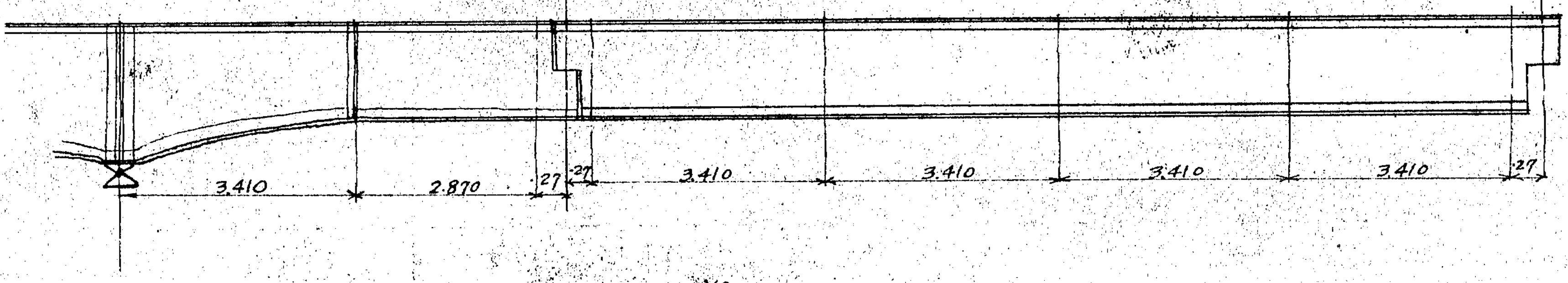
Cantilever arm

Excluding main girder dead load concentration assumed same as for suspended span.



floor slab, pavement, Coping & H.R = 3000 kg per meter of bridge (see page 7)

structural steel

stringers 2@40 = 80

floor beam 162

lateral Bracing 47.6

main girders say 2@520 = 1040

call this 1330

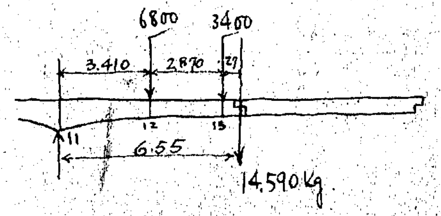
sum = 3000 + 1330 = 4330 kg

for one girder = 4330÷2 = 2165 kg/meter

Panel load on 12 = 2165 × 3.14 = 6,800 kg

panel load on 13 = 2165 × 1.57 abt = 3400 abt

load on hinge from suspended span = 14,320 + 270 = 14,590 kg.



moment at 11 pt.

14.590 × 6.55 = 95,560

3400 × 6.28 = 21,350

6800 × 3.410 = 23,190

140,100 kg. ---- as concentrated load

~~Considered~~ Assumed the dead load on girder as follows.

Concentration

at 12. slab & pavement 450 × 1.60 × 3.14 = 2230

load of structural steel

stringers, lateral $\overset{127.6}{\cancel{120}}$ × 3.14 = $\overset{400}{\cancel{377}}$

Flow beam = 553

$\overset{3183}{\cancel{3160}}$ kg

at 13. slab & pavement 450 × 1.60 × 1.57 = 1115

wt of steel

stringers, laterals $\overset{127.6}{\cancel{120}}$ × 1.57 = 200

Flow beam 553

1868 kg

uniform load

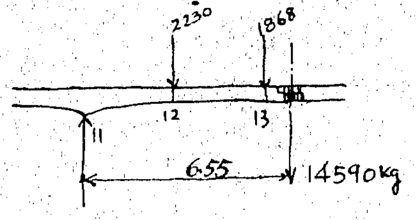
overhanging wt = 419.5 kg/m (see page 4)

flooring on girder = 450 × 0.8 = 360 kg/m

} 779.5 kg/m

girder assumed 520 kg/mete

total uniform load = 520 + 779.5 = 1299.5 call this 1300 kg/m



moment at 11 pt

due to concentration 14590 × 6.55 = 95,560

1868 × 6.28 = 11,730

2230 × 3.41 = 7,600

due to uniform load $\frac{1}{2} \times 1300 \times 6.55^2$ = 27,900

142,790 kgm --- design mt

shear at 11~12.

due to Concentration = 18,688

due to uniform l. = 8,515

27,200 kg.

moment at 12.

due to concentration 14,590 × 3.14 = 45,810

1868 × 2.87 = 5,360

Uniform load $\frac{1}{2} \times 1300 \times 3.14^2$ = 6,410

57,580 kgm

shear at 12-13.

Concentration = 16,460

Unif. load = 4,080

20,540 kg.

shear at 13-hinge = 14,940 kg.

moment at 13.

due to uniform load $\frac{1}{2} \times 1300 \times 0.27^2$ = 50

due to concentration 14590 × 0.27 = 3,940

3990 kgm

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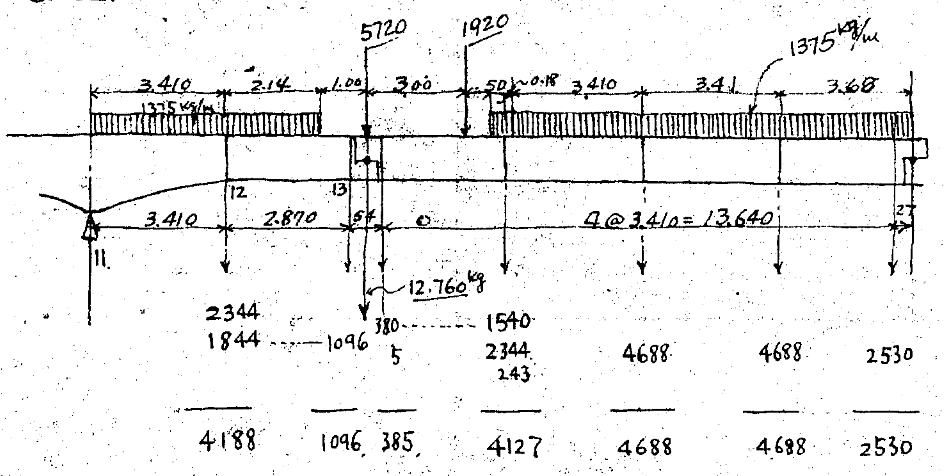
CALCULATIONS FOR

Live load.

Case I.



Concentration at hinge from suspended span

385 × 13.91 = 5360
4127 × 10.50 = 43300
4688 × 7.09 = 33200
4688 × 3.68 = 17250
2530 × 0.27 = 680

99,790 ÷ 14.18 = 7.040

Direct load at hinge 5.720

12.760 kg

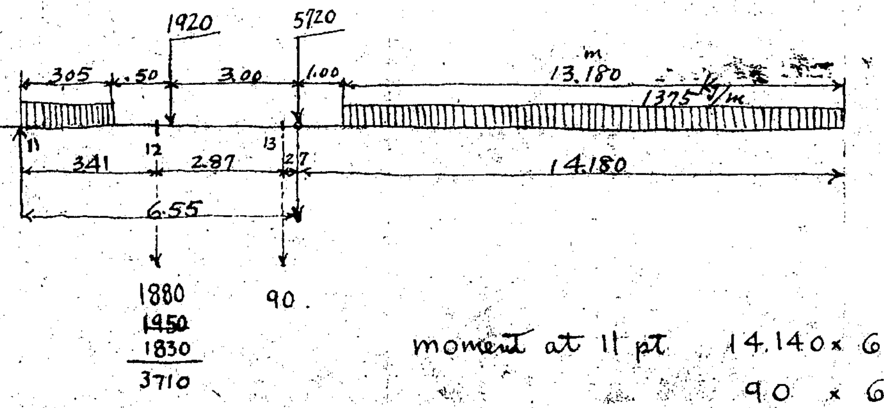
Moment at 11 pt Concentration at hinge 12.760 × 6.55 = 83,580

Cantilever load 1096 × 6.28 = 6,880

" 4188 × 3.41 = 14.280

104.740 Kgm

Case II loading



load on hinge from suspended span

$$\frac{13.18^2 \times 1375}{2 \times 14.18} = 8420 \text{ kg}$$

5720

14,140 kg.

moment at 11 pt 14.140 × 6.55 = 92,750

90 × 6.28 = 565

3710 × 3.41 = 12, 650 ✓

105, 965 Kgm ✓ ---- max.

moment at 12 pt 14140 × $\overset{3.14}{\cancel{2.87}}$ = ~~44.400~~

90 × 2.87 = 260 ✓

44, 660 Kgm

moment at 13 pt 14.140 × 0.27 = 3,820 Kgm

shear at 11 pt ~ 12 pt 18,000 Kg ✓

shear at 12 ~ 13. 14,230 Kg.

shear at 13 ~ hinge 14,140 Kg ✓

Summary for moments and shears

		Moment	Shear	
at 11 pt	Dead load	142,790	27,200	11~12
	live load	105,965	18,000	
		248,755 Kgm	45,200 Kg.	
at 12 pt	Dead load	57,580	20,540	12~13.
	live load	44,660	14,230	
		102,240 Kgm	34,770 Kg	
at 13 pt	Dead load	3,990	14,940	13~hinge
	live load	3,820	14,140	
		7,810 Kgm	29,080 Kg.	

web assumed 1850 × 9 = 166.5 cm² 1/8 web = 20.8 cm²

effective depth say 1.80 meter

Back to Back of Ls 1860 m/m

flange stress = 248,755 ÷ 1.80 = 138,200 Kg

section req'd ≑ 138,200 ÷ 1200 = 115.2

− 20.8

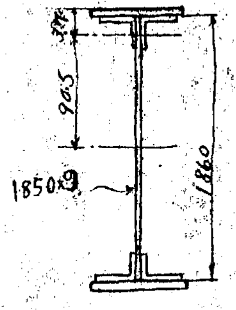
94.40 cm² Net area

Use 2Ls 150 × 150 × 15 = 85.48 − 15.0 = 70.48

1 Pl. 350 × 1.2 = 42.00 − 6.0 = 36.00

106.48 cm²

or.

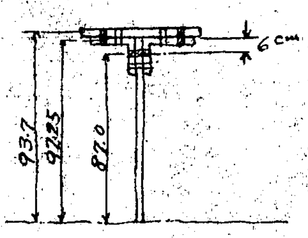


effective depth $= \left(\frac{85.48 \times 88.78 + 42 \times 93.6}{127.48}\right)^{\times 2} = 181$ cm

Moment of Inertia :

Flange Ls	150×150×15	------	4×889 = 3556
2 Coverpls	350×12	------	2×5.0 = 10
	$2 \times \overset{134.48}{\cancel{138.7}} \times 90.5^2$		= 2202849

2206415 cm⁴ (gross) ---- 2206,415



Rivet holes. 4-22 m/m φ

$4 \times 2.5 \times 1.5 \times 87^2 = 113,535$

" $4 \times 2.5 \times 1.5 \times 92.25^2 = 127,650$

" $4 \times 2.5 \times 1.2 \times 93.7^2 = 105.356$

346,541 cm⁴ ------ 346,541

Net I. = 1859.074 cm⁴

web Pl. 1850×10 ------ 474 072 cm⁴ gross

reduce 15% for rivet holes 403,641 cm⁴ Net.

summary of I $J_c = 2206,415 + 474,072 = 2681.207$ Cm⁴ gross

$J_t = 1.859,074 + 403,641 = 2263.515$ cm⁴ Net.

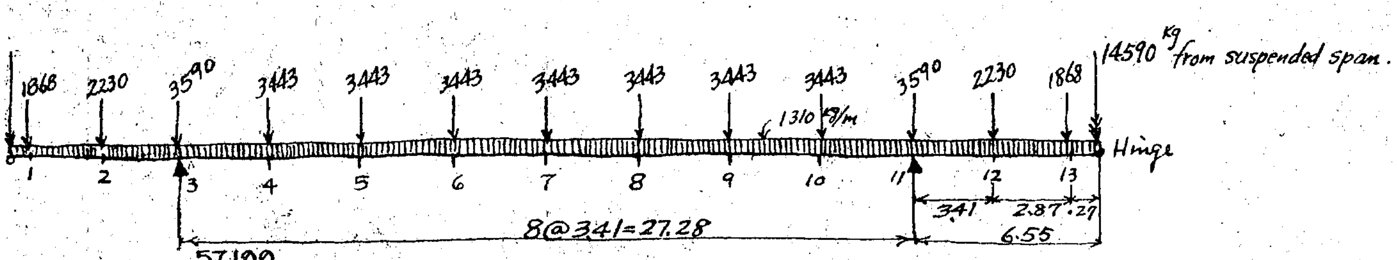
tensile fibre stress $= \frac{248,760 \times 94.4}{2263.515} = 1037$ kg/cm² < 1200 kg/cm²

Compression fibre stress $= \frac{248760 \times 94.4}{2681.207} = 880$ kg/cm² < 1058 kg/cm²

allowable Compression $= 1200\left(1 - 0.012 \times \frac{340}{35}\right) = 1058$ kg/cm²

Intermediate Anchor span

Dead load.



concentrated dead load at intermediate panel point between 3 ~ 11.

slab, pavement	450×1.60×3.410 =		2455 kg
stringers	80×3.410	=	273
lateral Bracing	47.6×3.41	=	162
Flow Beam	say	----	553
	panel load	=	3443 kg.

Uniform dead load

flooring	779.5 kg/m
girder say 530 kg ------	530.0
	1309.5

Call this 1310 kg/meter

load on 3 & 11. $= 2890 + \overset{F.B.}{700} = 3590$ kg

load on pier ;

14590+1868+2230 =	18,688 kg	--- Cantilever load
3.5 @ 3443 =	12,050	--- anchor span
½×40.38×1310 =	26,450	--- Uniform load
	57,188	
direct load	3,590	
	60,778 Kg	on pier or shoe Call this 61,000 kg

moment at pier side from cantilever and suspended load = 142,790 kgm (see page 12).

moment at centr as simple span = $\frac{1}{8} \times 1310 \times 27.28^2$ = 121,860 kgm --- Uniform load

(12.050× 13.64)-(3443×20.46)= 93920 } --- Concentration

+ 215,780 Kgm

moment at center (7 pt) = + 215,780
less − 142,790
call + 73,000 Kgm

Moment at several points — for Uniform load $m = \frac{wx}{2}(l-x)$. — x is measured from left support

Uniform dead load.

Point	x	w	(l−x)	m	due to Concentrated load		sum
4	3.41	1310 Kg/m	23.86	+ ~~106600~~ 53,300	12.050× 3.410 =	41,090	94,390 Kgm
5	6.82	"	20.46	~~182,800~~ 91,400	(12,050×6.82)−(3443×3.41)	= 70,440	161,840 "
6	10.23	"	17.05	114,250	(12050×~~6.82~~ 10.23)−(3443× 10.23)	= 88,050	202,300 "
(center) 7	13.64	"	13.64	121,860	(12.050×13.64)−(3443×20.46)	= 93,920	215,780 "

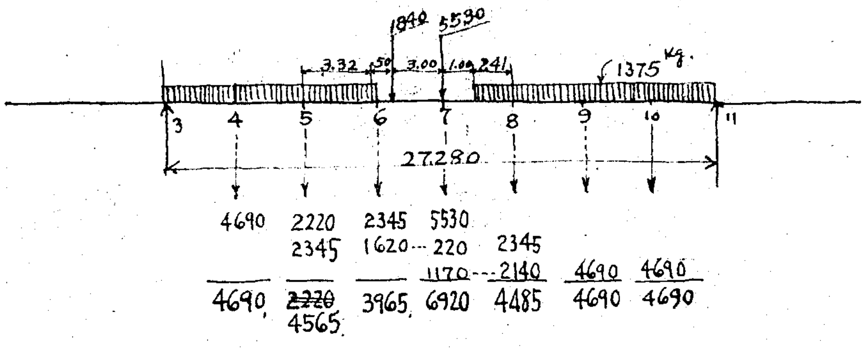
actual moment

Point	Positive Moment	Neg. Mt		sum
4	+ 94 390	− 142790	=	−48400 Kgm
5	+ 161,840	"	=	+19,050 "
6	+ 202,300	"	=	+ 59,510 "
at center				+ 73,000 "

Live load.

Positive moment

at 7 pt



motor truck ~~2295~~ 2250

impact $\frac{20}{60+27.280}$ = 22.8% --- 513

rear wheel. 2763 kg.

front " 921 kg.

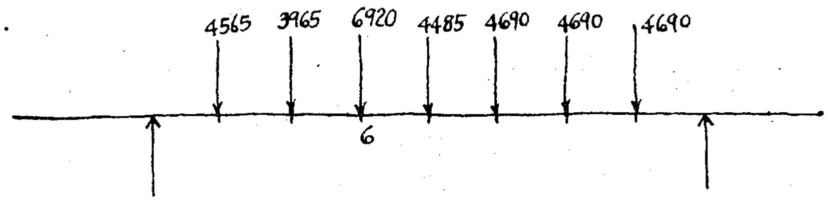
Reaction

4690 × 34.10 = 159,930
4565 × 20.46 = 93,400
3965 × 17.05 = 67,600
6920 × 13.64 = 94,390
4485 × 10.23 = 45,880
R = 461,200 ÷ 27.28 = 16,900 Kg

moment at 7 pt

16,900 × 13.64 = 230,520
4690 × 10.23 = 47,980
4565 × 6.82 = 31,130
3965 × 3.41 = 13,520
− 92,630
m = 137,890 Kgm

At 6



Reaction

4565 × 23.87 = 109,070
3965 × 20.46 = 81,120
6920 × 17.05 = 117,990
4485 × 13.64 = 61,180
4690 × 10.23×2 = ~~47,980~~ 95,960
R = ~~417,340~~ 465,320 ÷ 27.28 = ~~15,290~~ 17,060 kg

moment at 6

~~15,290~~ 17,060 × 10.23 = ~~156,420~~ 174,520
4565 × 6.82 = 31,130
3965 × 3.41 = 13,520
− 44,650
m = ~~111,770~~ 129,870 Kgm

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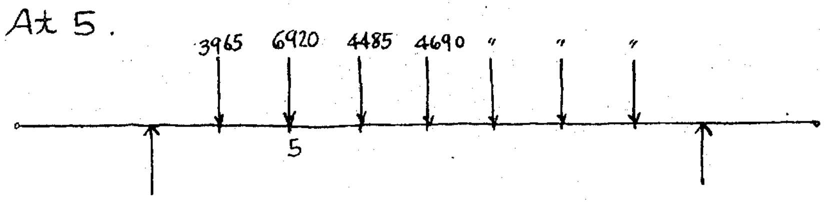
CALCULATIONS FOR

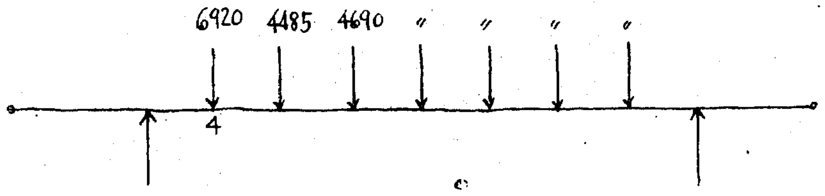
At 5.



Reaction
3965 × 23.87 = 94,640
6920 × 20.46 = 141,580
4485 × 17.05 = 76,470
4690 × 34.1 0 = 159,930
R = 472,620 ÷ 27.28
= 17,320 kg.

moment
17.320 × 6.82 = 118,120
3,965 × 3.41 = − 13,520
104,600 kgm.

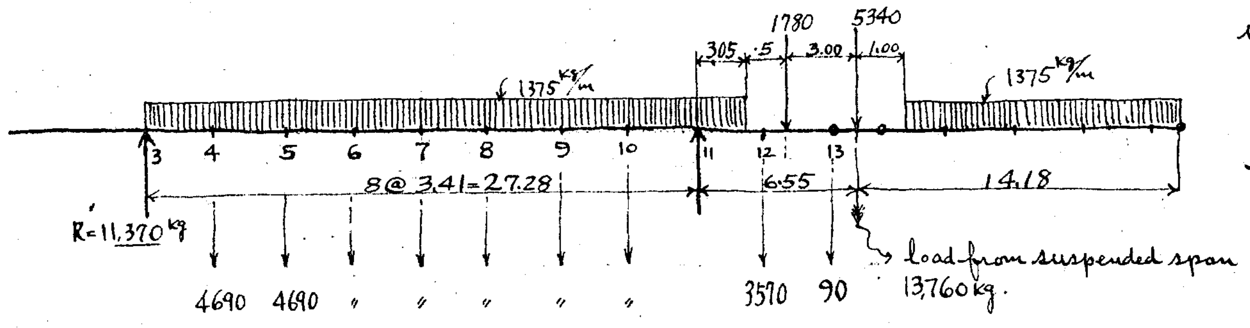
at 4.



Reaction
6,920 × 23.87 = 165,180
4,485 × 20.46 = 91,760
4690 × 51.15 = 239,890
R = 496,830 ÷ 27.28
= 18,210 kg.

moment 18,210 × 3.41 = 62,000 kgm.

Max. Negative Moment. at support. (此項不要.)



Rear wheel 2250
impact $\frac{20}{60+48.}$ = 18.6%
420
2670 kg
front wheel = 890 〃
load on girder = 2×2670 = 5340 kg
〃 1780 〃

Reaction on 11.
(13,760+5340) × 6.55 = 125,100
90 × 6.28 = 565
3,570 × 3.41 = 12,170
mt = 137,835 — ÷ 27.28 = 5,050 kg
Cantilever load. — 22,760
anchor span 3.5 @ 4,690 = 16,420
Reaction on R11. — 44,230 kg

~~moment 10~~
~~− 44,230 × 3.41 = −150,820~~
~~Cantilever action = +137, 835~~
~~− 12,985 kgm~~

~~moment at 9.~~
~~− 44,230 × 6.82 = − 301,650~~
~~4,690 × 3.41 = + 15,990~~
~~Cantilever action + 137,835~~

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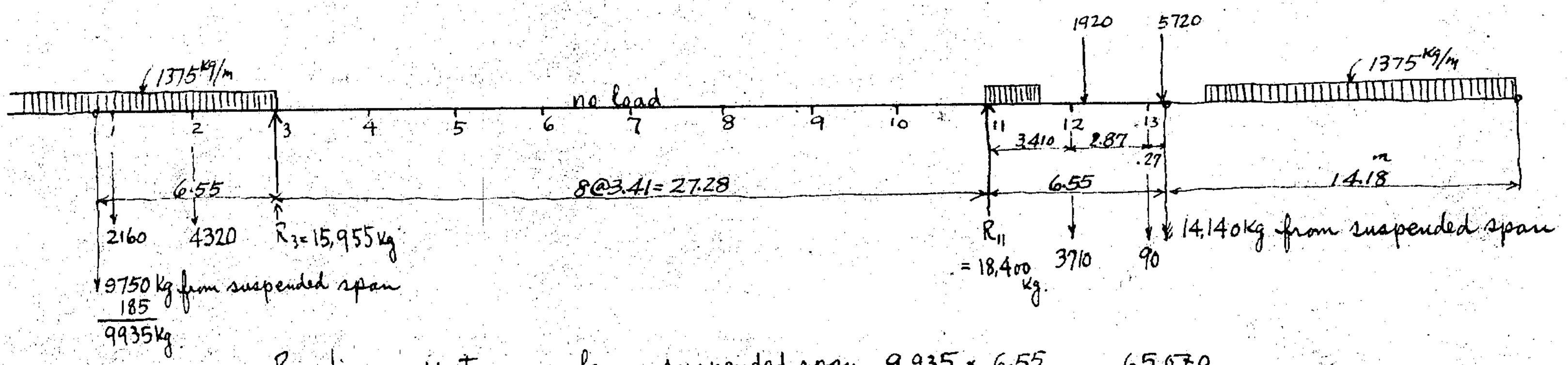
Negative moment for intermediate anchor span

–Max. moment occurs with both suspended span and both cantilever arms loaded and intermediate span unloaded.



Reaction on 11 pt for right side
from suspended span 9.935 × 6.55 = 65.070
from Cantilever arm
2160 × 6.28 = 13,565
4320 × 3.41 = 14,730
16,415 kg
93,365 ÷ 27.28 = 3420 kg.

from Cantilever load
90 + 14.140 + 3710 = 17.940

Reaction on R3 from right suspended load
from suspended span 14140 × 6.55 = 92.620
from Cantilever load. 90 × 2.87 = 565
3710 × 3.41 = 12.650
17940. kg.
105,835 ÷ 27.28 = 3880 kg

Reaction on R 3.
Due to left cantilever load = +16.415
Due to left cantilever action = – 3 880
Due to right cantilever action = + 3420
R3 = 15,955 kg.

Reaction on R11.
Due to right cantilever load = 17.940
Due to right cantilever action = 3 880
Due to left cantilever effect = – 3,420
R11 = 18,400 kg

Moment at 10. omitt
~~(–16,415 + 15,955) × 3.41 = –1569~~
~~93,365~~
(+18400 – 17,940) = –460 × 3.41 = +1,570
–105,835
– 90,135 Kgm

moment at 9
–460 × 6.82 =

moment at 4.
(16,415 – 15,955) × 3.41 = 460 × 3.41 = 1569
93,365
94,934 kgm

moment at 7. 460 × 13.64 = 6 274
93,365
99,639 kgm

moment at 8 460 × 17.05 = 7 844
93,365
101,209 Kgm

moment at 9 460 × 20.46 = 9.412
93.365
102,777 Kgm

moment at 10 460 × 23.87 = 10,980
93,365
104,345 Kgm

moment at 11 460 × 27.28 = 12,550
93,365
105,915 Kgm

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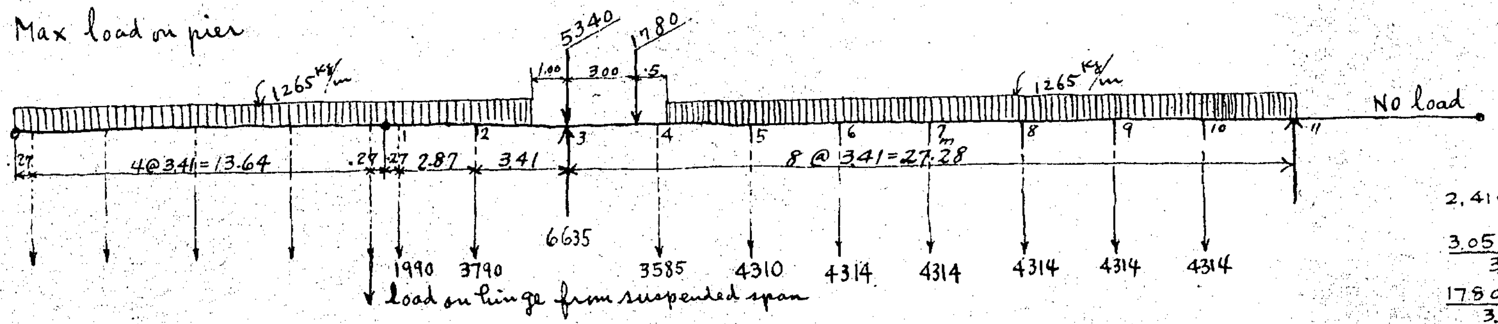
CALCULATIONS FOR

Positive shear

Max load on pier



2.41×1265 = 3,050

$\frac{3.050\times1.205}{3.41}$ = 1077.

$\frac{1780\times0.4}{3.41}$ = 209.

8970
170
9140 kg

impact $\frac{20}{60+48}$ = 18.6% motor truck rear wheel 2250
impact 420
2670 kg front wheel load = 2670÷3 = 890 kg.

load on girder for rear wheel = 2×2670 = 5340 kg
" " front " = 2×890 = 1780 "

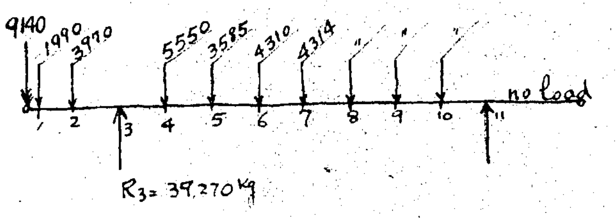
uniform live load = $\frac{100.000}{170+48}$ ≑ 460 kg/m² Uniform load on girder = 2.75 m × 460 = 1265 kg/meter run

Reaction on pier R 3:-
load of left cantilever arm & on hinge = 9140+1990+3790 = 14,920 kg
cantilever action
9140 × 6.55 = 59.870
1990 × 6.28 = 12.500.
3790 × 3.41 = 12.920
14920 kg 85,290 ÷ 27.28 = 3,130 kg.

From anchor span
3585 × 23.87 = 85.570
4310 × 20.46 = 88.180
4314 × 51.15 = 220.660
394.410 ÷ 27.28 = 14.460
32.510 kg

load on shoe or bearing = 32.510 + 6635 = 39.145 kg

shear at 4 point

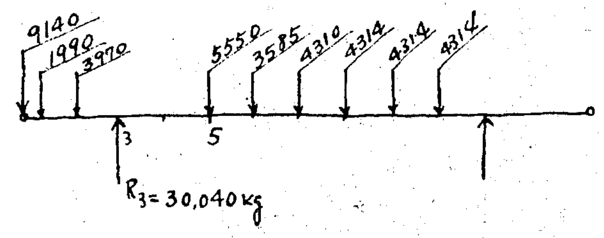


Reaction on R3.
Cantilever action
9140 × 6.55 = 59,870
1990 × 6.28 = 12,500
3970 × 3.41 = 13,540
15,100 kg. 85,910 ÷ 27.28 = 3150 kg
15,100

Cantilever load
from anchor span
5550 × 23.87 = 132,480
3585 × 20.46 = 73,350
4310 × 17.05 = 73,490
4314 × 34.10 = 147,110
426,430 ÷ 27.28 = 15,630
R₃ = 33,880 kg

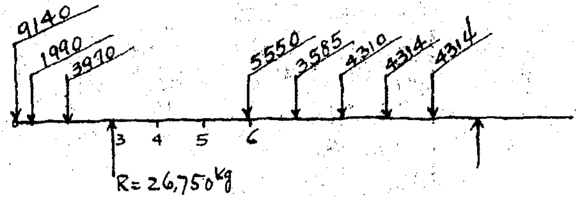
shear at 4. = 33880 − 15,100 = 18780 kg

shear at 5 pt



Reaction R₃: Cantilever action = 3,150
Cantilever load = 15,100
from anchor span
5550 × 20.46 = 113,550
3585 × 17.05 = 61,120
4310 × 13.64 = 58,790
4314 × 20.46 = 88,260
321,720 ÷ 27.28 = 11,790
sum = 3,150 + 15,100 + 11,790 = 30,040 kg

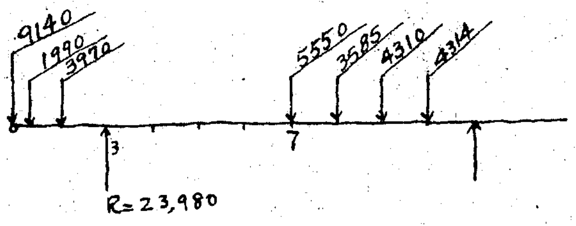
shear at 5 = 30,040 − 15,100 = 14,940 kg positive shear at 6 point



Reaction R₃. cantilever effect -- 3.150 kg
cantilever load — 15.100
anchor span
5550 × 17.05 = 94.630
3585 × 13.64 = 48.900
4310 × 10.23 = 44.100
4314 × 10.23 = 44.130
231.760 ÷ 27.28 = 8,500
R₃ = 26,750 kg.

shear at 6 = 26.750 - 15.100 = 11.650 kg

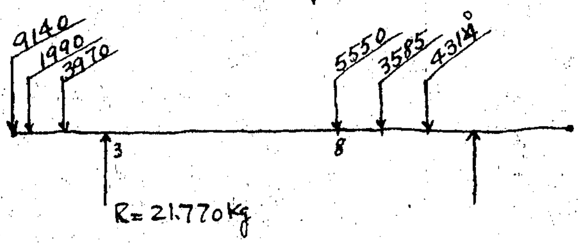
Positive shear at 7 pt



Reaction at R₃ Canti lever effect -------- 3150
cantilever load ------ 15.100
anchor span
5550 × 13.64 = 75.700
3585 × 10.23 = 36.670
4310 × 6.82 = 29.390
4314 × 3.41 = 14.700
156.460 ÷ 27.28 --- = 5.730
R₃ = 23.980 kg

shear at 7. = 23.980 - 15.100 = 8.880 kg

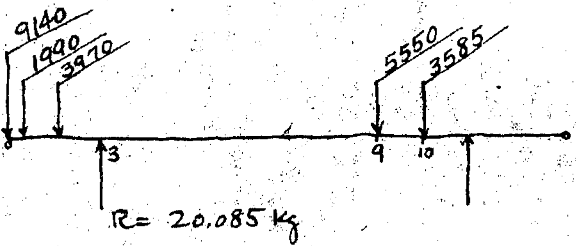
Positive shear at 8 pt



Reaction at R₃. Cantilever effect 3150
Cantilever load 15.100
anchor span
5550 × 10.23 = 56.780
3585 × 6.82 = 24.450
4310 × 3.41 = 14,700
95,930 ÷ 27.28 = 3,520
R₃ = 21,770 kg.

shear at 8 = 21.770 - 15.100 = 6.670 kg

shear at 9 point



Reaction at R₃ cantilever effect 3,150
Cantilever load 15.100
anchor span
5550 × 6.82 = 37,850
3585 × 3.41 = 12,220
50,070 ÷ 27.28 = 1835
R₃ = 20,085 kg

shear = 20.085 - 15.100 = 4,985 kg.

shear at 10 point

Reaction at R₃ Cantilever effect = 3150
Cantilever load = 15.100.
anchor span 5550 × 3.41. = 18,920 ÷ 27.28 = 690
R₃ = 18.940 kg.
shear at 10 = 18.940 - 15.100 = 3,840 kg

shear at right pier side $S_{11} = (3150 + 15.100) - 15100 = 3150$ kg

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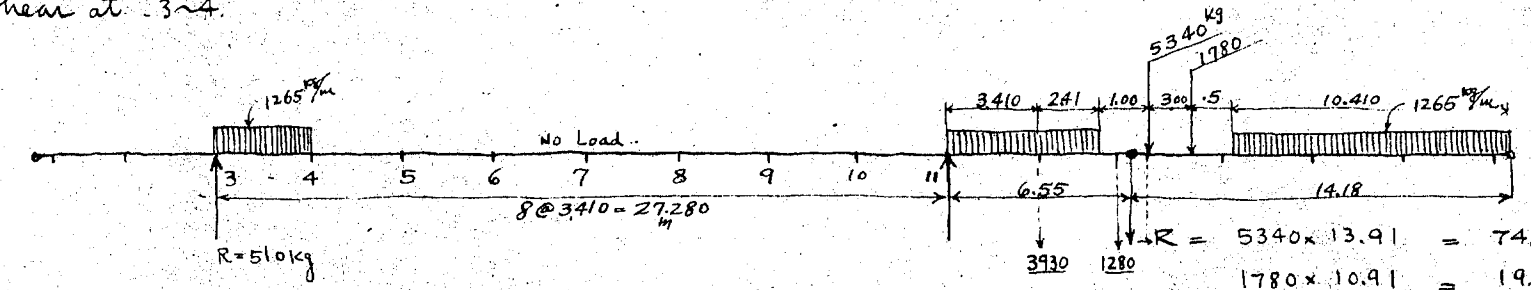
CALCULATIONS FOR

Max. Negative Shear.

shear at 3~4.



R = 5340 × 13.91 = 74,280

1780 × 10.91 = 19,420 } ÷ 14.18 = 6610

$\frac{1265 \times 10.41^2}{2 \times 1418}$ = 4,830

11,440 kg

Reaction R_3. Due to anchor span = $wx(1-\frac{x}{2l})$

= $1265 \times 3.41(1-\frac{3.41}{54.56}) = 4040^{kg}$

due to cantilever load

11,440 × 6.55 = 74,930

1280 × 6.28 = 8,040

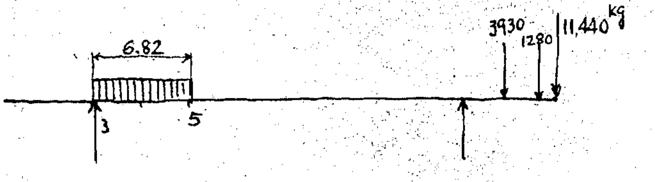
3930 × 3.41 = 13,400

96,370 ÷ 27.28 = -3530

R_3 = + 510 kg.

shear at 4 pt   510 - 1265 × 3.41 = -3800 kg.

shear at 5 point



Cantilever effect -3530

anchor span $1265 \times 6.82(1-\frac{6.82}{54.56})$ = 7.550

R_3 = + 4020 kg.

shear at 5 = 4.020 - 1265 × 6.82 = -4610 kg.

shear at 6 pt

Cantilever effect -3,530

anchor span $1265(1-\frac{10.23}{54.56}) \times 10.23$ = +10.410

R_3 = +6,880 kg.

shear at 6 = 6.880 - 1265 × 10.23 = -6.060 kg

shear at 7 pt

Cantilever action -3530

anchor span $1265 \times 13.64(1-\frac{13.64}{54.56})$ = +12,940

R_3 = 9.410 kg.

shear at 7 = 9410 - 1265 × 13.64 = -7.840 kg

shear at 8 point

cantilever action -3530

anchor span $1265(1-\frac{17.05}{54.56}) \times 17.05$ = 14,830

R_3 = 11.300 kg

shear = 11.300 - 1265 × 17.05 = -10,270 kg.

shear at 9 point

cantilever action -3530

anchor span $1265 \times 20.46(1-\frac{20.46}{54.56})$ = 16,180

R_3 = 12.650 kg

shear at 9 = 12.650 - 1265 × 20.46 = -13,230 kg

shear at 9~10.

Cantilever effect -3530

anchor span $1265(1-\frac{23.87}{54.56}) \times 23.87$ = 16.980

R_3 = 13,450

shear = 13,450 - 1265 × 23.87 = -16,750 kg

shear 10~11.

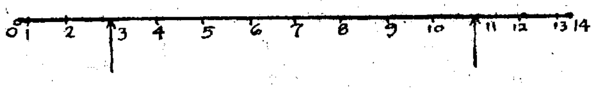
Cantilever effect -3530

anchor span 1265 × 13.64 = 17.250

R_3 = 13.720 kg.

shear = 13.720 - 1265 × 27.28 = -20,780 kg

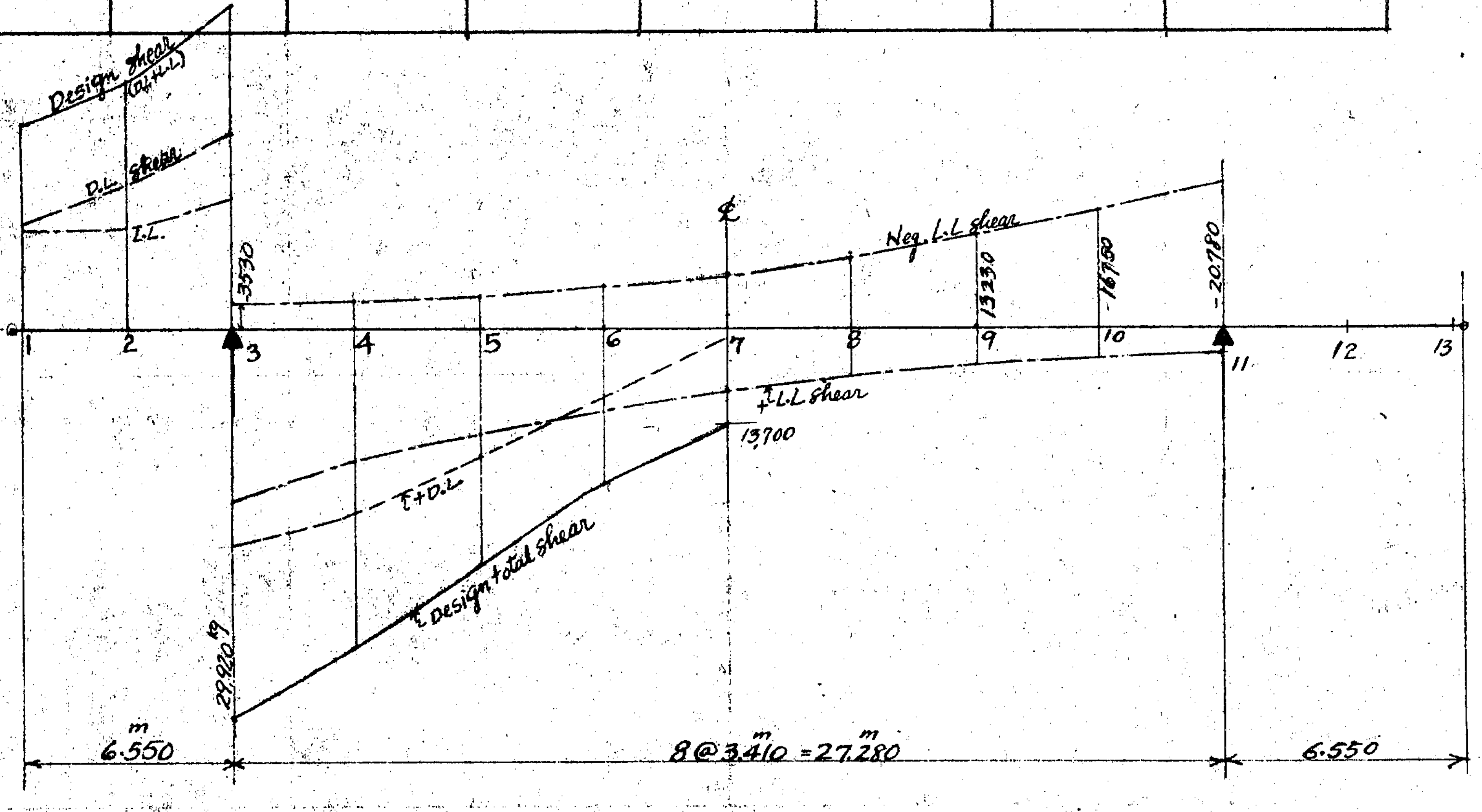
Summary for moments



Panel Point	Dead Load Moment		Live Load Moment		Resultant Moment		Design Moment (Kgm)
	Positive	Negative	Positive	Negative	(+)	(−)	
	Kgm	Kgm	Kgm	Kgm		Kgm	
0		0		0		0	0
1		3.990		3820		7.810	− 7.810
2		57.580		44.660		102.240	− 102.240
3 (at END)		142.790	0	105,960		248,750	− 248.750
4		48.400	62,000	104,350	13,600	152,750	− 159.550
5	19,050		104.600	102,780	123,650	83,730	+ 165,520
6	59.510		129.870	101.210	189.380	41.700	+ 210.230
7 (center)	73,000		137,890	99.640	210,890	26,640	+ 224.210

Summary for shears

Panel Point	Dead Load Shear		Live Load Shear		Resultant Shears		Design Shear (kg)
	~~Positive~~(−)	~~Negative~~(+)	~~Positive~~(−)	~~Negative~~(+)	Pos.	Neg.	
	Kg	Kg	Kg	Kg	Kg	Kg	
0							
1	14.940		14.140			29.080	− 29.100
2	20.540		14.230			34.770	− 34.800
3 (support)	27.200	29.920	{18.000 3.530	24,200	54,120	45,200	− {45,200 + 54.100
4		25,450	3,800	18,780	44,230		+ 44.230
5		17,540	4.610	14,940	32,480		+ 32,500
6		9,630	6,060	11.650	21.280		+ 21,300
7 (center)	1,722	1,722	7.840	8,880	10,600	6.120	+ 13,700



shear diagram.

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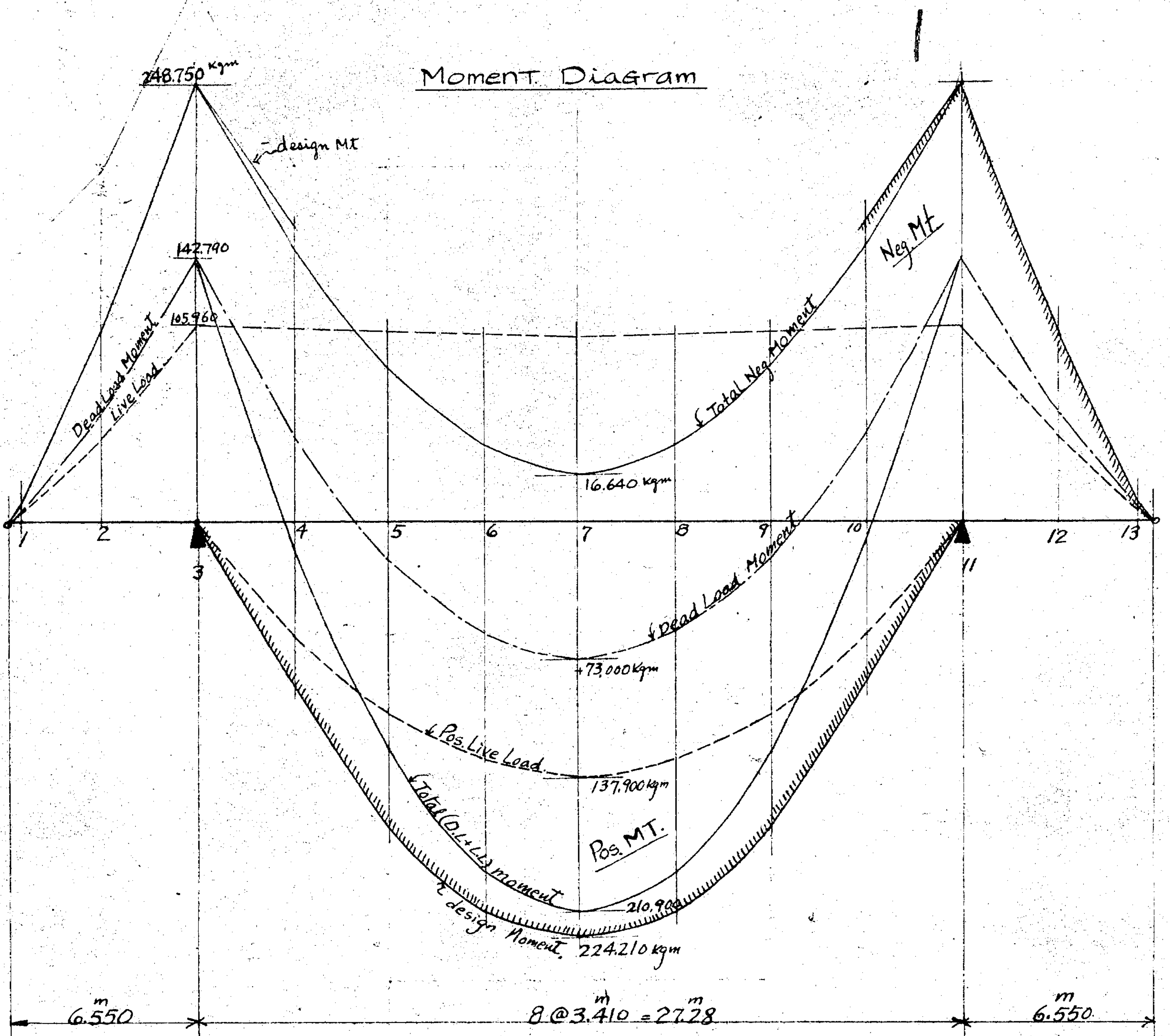
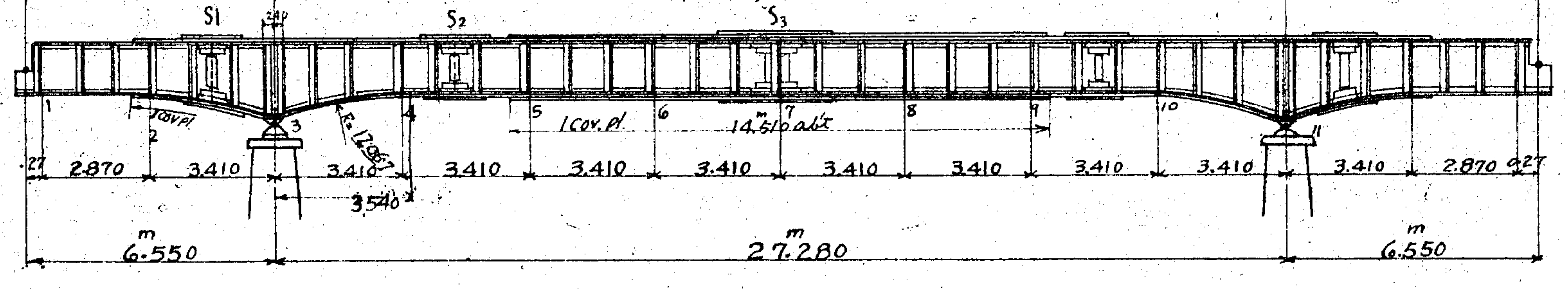
CALCULATIONS FOR

Moment Diagram



scale, dimension 1/200, moment force 33.3 m/m = 100,000 kgm.



sketch of Anchor span.

Scale 1/200

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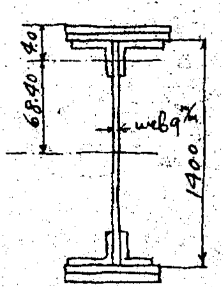
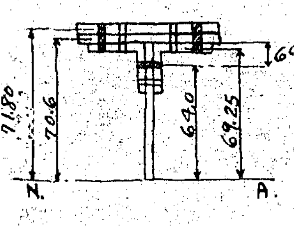
CALCULATIONS FOR

Design of Kamitsuka-Bashi For Aichi-Ken

Section of main Girder





Moment = + 224,210 kgm   shear = 13,700 kg

effective depth 136.8 cm   web 1390 × 9 = 125.1 cm²   ⅛ web = 15.6 cm²

flange stress = $\frac{224.210}{1.368}$ = 164.000 kg   req'd flange area = $\frac{164.000}{1200}$ = 136.7

− 15.6

Net area = 121.1 cm²

Use 2 Ls 150 × 150 × 15 = 85.5 − 15.0 = 70.50

2 Cov Pls 350 × 12 = 84.0 − 12.0 = 72.00

142.50 cm² Net area.

Moment of inertia   Flange Ls 150 × 150 × 15 ------ 4 × 889 = 3556

4 Cov Pls. 4 × 350 × 12 ------- 4 × 5.04 = 20

do 2 × 169.5 × 68.4² = 1586,030

1589.606 cm⁴ gross

Rivet holes 4-22 m/m φ $4 \times 2.5 \times 1.5 \times 64^2$ = 61,440

〃 $4 \times 2.5 \times 1.5 \times 69.3^2$ = 72,040

$4 \times 2.5 \times 1.2 \times 70.6^2$ = 59.912

$4 \times 2.5 \times 1.2 \times 71.8^2$ = 61,863

255,255 cm⁴ --- 255,255

J_t = 1334,351 cm⁴ Net.

Web 1390 × 9 ------ 201 421 cm⁴ gross

reduce 15% for rivet hole 171,210 〃 Net.

J_c = 1589,606 + 201421 = 1791,027 cm⁴ gross.

J_t = 1334,351 + 171,210 = 1505,552 〃 Net.

Comp. ~~tensile~~ fibre stress = $\frac{224210 \times 72.4}{1791,027}$ = 910 kg/cm². O.K. < 1058 kg/cm²

tensile ~~Compressive~~ fibre stress = $\frac{224210 \times 72.4}{1505,552}$ = 1080 kg/cm² < 1200 kg/cm²

allowable Compression = $1200\left(1 - 0.012 \times \frac{340}{35}\right)$ = 1058 kg/cm²

approximate weight of intermediate anchor span.

2 web Pls	1550 abt × 9	@	109.5 kg/m	× 4.70 m	=	1029 kg.
2 web Pls	1840 × 9	@	~~165.6~~ 130.0	× ~~5.10~~ 6.80	=	~~1689~~ ~~1326~~ 1768
2 web Pls	1390 × 9	@	98.2	× 8.54	=	1677.
2 Flang Ls	150 × 150 × 15	@	33.55	× 40.06	=	2688
2 do.	〃 〃 〃	〃	〃	× 41.00	=	2751.
2 Cov Pls	350 × 12	@	32.97	× 35.0	=	2308
2 Cov. Pls	350 × 12	@	〃	× 1451	=	957
splices	2	@	360 kg		=	720
〃	2	@	400		=	800
〃	1	@	450		=	450
8 stiffers Ls	130 × 130 × 12	@	23.36	× 1.82	=	340
70 intermediate stiffs Ls	125 × 90 × 10	@	16.09	× mean 1.40	=	1577.
fillers Pls	48 − 90 × 15	@	10.60	× 1.10	=	560
fillers Pls	9 − 200 × 15	@	23.55	× 1.10	=	233
〃	9 − 90 × 15	@	10.60	× 1.10	=	105
〃	4 − 240 × 15	@	28.26	× ~~1.82~~ 1.55	=	175
END Connection details				say	=	420
self Ls	24 − 125 × 90 × 9	@	14.60	× 0.310	=	109
lateral Connection Pls	13	@	25 say		=	325
4-stiffning Pl under the support	300 × 15	@	35.3	× 2.00 m	=	282
shoes	say					1200
rockers						600
Rivet heads						696
						21,770 kg

weight per meter of span = 21.770 ÷ 40.38^{m} = <u>539</u> kg/meter run

Approximate weight of intermediate span for structural steel.

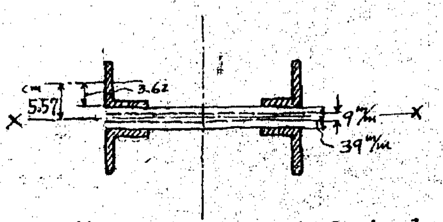
stringers	80^{kg}	@ 40.10^{m}	=	3208^{kg}
floor beams	11	@ 553	=	6083
do	2	@ 680	=	1360
lateral bracings	-------		=	1922
Girders with details	2	@ 21.770	=	43.540
				56.113 kg for one span

Total structural steel weight for whole Bridge length (approximatly)

Suspended spans	9 spans	@ 14.763	=	132.867^{kg}
Anchor spans with Cantilever arms	8	@ 56.113	=	$448,904^{kg}$
Expansion joint	11 pices	@ 570	=	6.270
misc		say		1.000
				588.041 kg
Bracket for Handrail Post	16 pices	@ 25 kg	=	400
				588,440 kg. ✓

Details :-

Stiffner on Pier



4Ls 130×130×12 = 119.04 cm²

Max. load on support.

Dead load.

structural steel $\left(\frac{14.763+56.113}{2}\right)\div 2$ = 17.720^{kg}

flooring 450×2.75 = 1237.5

Copings & H.R 262.0

Call $1500^{kg}/m$ × 27.28 = <u>40,920</u>

58.640^{kg}

live load. (See page 18) -------- <u>39.145</u>

97.785

Call this <u>98,000</u> kg ✓

$I_x = 4\times 466.3 + 119.04\times 5.57^2 = 5557\,cm^4$

least radius of gyration, $r = \sqrt{\frac{5557}{119.04}} = 6.83^{cm}$

$f_c = 1500\left(1-0.0055\times\frac{92.5}{6.83}\right) = 1380\,kg/cm^2$ call $1000\,kg/cm^2$

section req'd = $\frac{98.000}{1000}$ = 98 cm² < adopted area = 4Ls-130×130×12 = 119.04

Flange rivet spacing

r = allowable bearing for 9 m/m web plate = 3366^{kg}

Portion	(Shear) ∇	r	h_o=Effective depth	Horizontal increment	Verticalload/cm	Resultant Increment	Pich	adopted Pitch
3 (at Pier)	54.100^{kg}	3366	181.0^{cm}	256	84	270 kg/cm	12.4^{cm}	
4	44.230	"	say 135.0	292	"	304 "	11.0	
5	32.500	"	136.8	217	"	234 "	14.3	
6	21.300	"	"	143	"	179 "	18.7	
7.	13.700	"	"	92	"	125 "	26.8	

or another method:— (参考)

rivet spacing connected flange vertical angle and web plate

allowable max Pitch $P = \dfrac{r}{\sqrt{(\nabla \times \frac{G}{J})^2 + w}}$ where

∇ = max shear considered pt

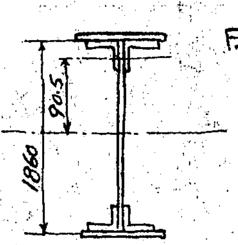
G = statical moment of flange section about its Neutral axis

J = moment of inertia about Neutral axis

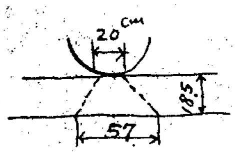
r = rivet value kg (9 m/m bearing = 3366)

w = direct vertical load on flange kg per cm

at 3 pt (support) ∇ = 54.120 kg



Flange area = 106.48 $^{cm^2}$ Net.



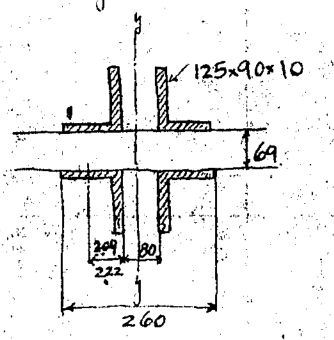
w = flooring between flanges ---- 450×0.80 = 360 kg/m
overhang load. (see page 4) = 420
main girder say = 540
1320 kg/m
wheel load 2925 ÷ .57 = 5130 kg/m
6450 kg/m or 65 kg/cm

∇ = 54.120 kg
J_t = 2263.515 cm^4
r = 3366 kg
G = 106.48 × 90.5 = 9.636 cm^3
w = 65 kg/cm

$$P = \frac{3366}{\sqrt{(54.120 \times \frac{9.636}{2263.515})^2 + 65^2}} = 14\text{cm}$$

---

stiffner at hinge



load on hinge = 30,000 kg say
4Ls 125×90×10 ---- 82.00 cm^2

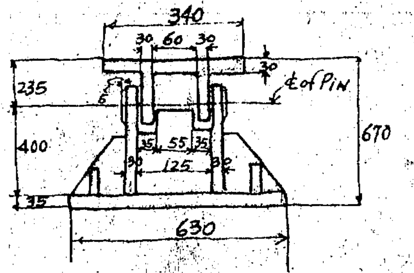
J_y 4Ls 125×90×10 ---- 4×138.1 = 552.4
$4 \times 20.5 \times 6.22^2$ = 3172.4
J = 3724.8 cm^4

least radius gyration, $r = \sqrt{\dfrac{3724.8}{82}} = 6.74$ cm^2
l = 640 ÷ 2 = 32 cm
allowable compression = $1500 \times (1 - 0.055 \times \frac{32}{6.74})$ = 1460 kg/cm^2 use 1000 kg/cm^2
required area = $\dfrac{30,000}{1000}$ = 30 cm^2 ample.

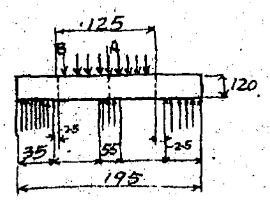
Design of shoe for anchor span

max load on Pier 98,000 kg Design load call this 100,000 kg

Fix shoe



Pin use dia 120 m/m φ grip = 196 m/m



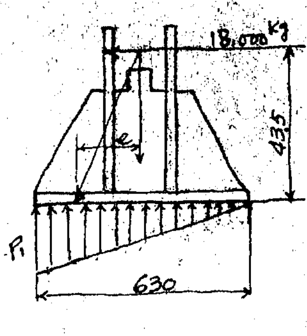
Bending moment
section A-A
$$M = \frac{100,000}{12.5}\left[3.5 \times (1.75 + 0.25 + 6) + \frac{2.75^2}{2}\right] - \frac{100,000}{12.5} \times \frac{6^2}{2}$$
= 110,240 kg cm

moment of inertia of pin = $\dfrac{\pi d^4}{64} = \dfrac{3.14 \times 12^4}{64}$ = 1015 cm^4

unit bearing of base = $\dfrac{100,000}{63 \times 56}$ = 28 kg/cm^2 < 45 kg/cm^2

fibre stress = $\dfrac{110,240}{1015} \times \dfrac{12}{2}$ = 650 kg/cm^2 < 1800 kg/cm^2

Considering earthquake



load on shoe of D.L = ~~40,000~~ 60,000 kg say.
seismic force = ~~40,000~~ 60,000 × 0.3 = 18,000 kg.
eccentricity, e = $\dfrac{18,000}{60,000}$ × 43.5 = 13.05 cm

$P_1 = \dfrac{60,000}{3528} \times (1 + \dfrac{6 \times 13.05}{63})$ = 38.2 kg/cm^2 < 45×1.6

$P_2 = \dfrac{60,000}{3528} \times (1 - \dfrac{6 \times 13.05}{63})$ = − 4 kg/cm^2

section B-B. shear = $\dfrac{100,000}{12.5}$ × 3.5 = 28,000 kg

unit shear = $\dfrac{28,000}{113.04}$ = 250 kg/cm^2 < 900 kg/cm^2

Bearing = $\dfrac{100,000}{1.285 \times 19.5 \times 6}$ = 660 kg/cm < 1800 kg/cm^2

H. TASHIMA

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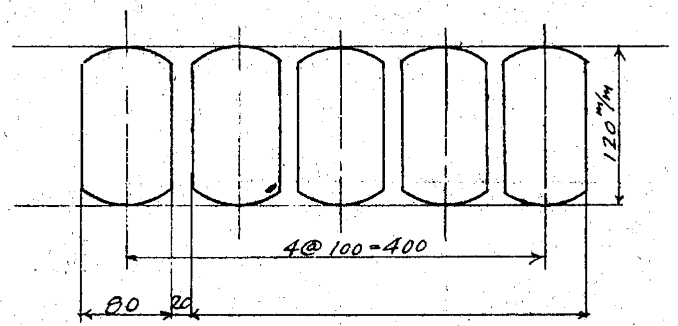
CALCULATIONS FOR

CHECKED BY ________ DATE ________ **PAGE NO.** 26

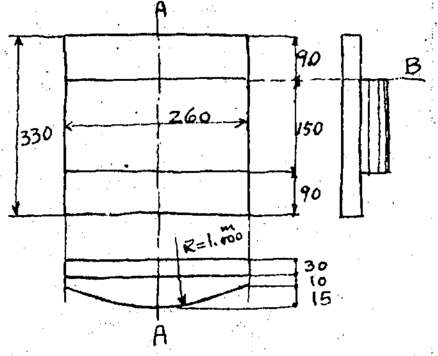
Roller shoe for anchor span.



Use segmental roller, dia of Roller = 120 m/m
effective length of roller = 420 m/m
bearing = 45×12×42.0×5 = 113.4,00 kg > 100,000 kg.

Rocker on the support of suspended span.
load on rocker = 30,000 kg (D.L+L.L)

Top shoe

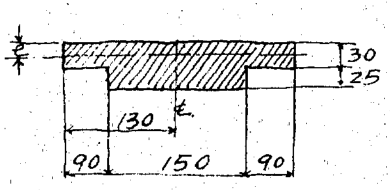


bearing area = 26×33 = 858 cm² $f = \frac{30,000}{858} = 35\text{ kg/cm}^2 < 45\text{ kg/cm}^2$

bearing for curvature radia of roller = 100 cm
bearing = 45dl. = 45×200×15 = 135000 kg/ > 30,000 kg.

max bearing of contact surface
by Helze's formula $P_0 = 0.42\sqrt{\frac{PE}{lr}} = 0.42\sqrt{\frac{30,000 \times 2100,000}{15 \times 100}}$
$= 2,720\text{ kg/cm}^2 < 6500$
(cast steel)

section A-A

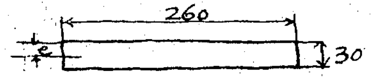


e = 2.26 cm, moment of inertia = J = 329.7 cm⁴

Moment at center = $\frac{30,000}{26} \times \frac{13^2}{2} = 97,500$ kgcm

fibre stress = $\frac{97,500}{329.7} \times (5.5 - 2.26) = 960\text{ kg/cm}^2 < 1200\text{ kg/cm}^2$.

section B-B



e = 1.5 cm $J = \frac{26}{12} \times 3^3 = 58.5\text{ cm}^4$

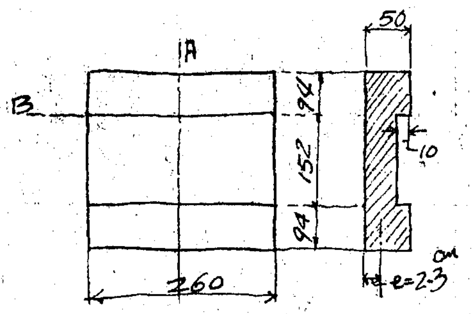
moment = $\frac{30,000}{33} \times \frac{9^2}{2} = 36,800$ kgcm

fibre stress = $\frac{36,800 \times 1.5}{58.5} = 940\text{ kg/cm}^2 < 1200\text{ kg/cm}^2$.

Connecting rivets use 22 m/m φ field rivet No 4.
gross area = $3.14 \times \left(\frac{2.2}{2}\right)^2 \times 4 = 15.3\text{ cm}^2$

shear = Deadload say 15,000 ×0.3 = 4500 kg — seismic force
unit shear = $4500 \div 15.3 = 295\text{ kg/cm}^2 < 750 \times 1.6$

Bottom shoe



J = 286.6 cm⁴

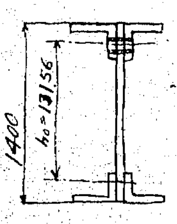
bearing area = 26×34 = 884 cm²
bearing = $30,000 \div 884 = 34\text{ kg/cm}^2 < 45\text{ kg/cm}^2$

section A. $M = \frac{30,000}{26} \times \frac{13^2}{2} = 97,500$ kgcm

fibre stress = $\frac{97,500}{286.6} \times (5 - 2.3) = 920\text{ kg/cm} < 1200\text{ kg/cm}$

Resisting Moment at several section.

at ① section (end of Cantilever arm).



Flange 2Ls 150×150×15 = 85.5 cm²
web 1390×9.
effective depth = 140 − 2×4.22 = 131.56 cm

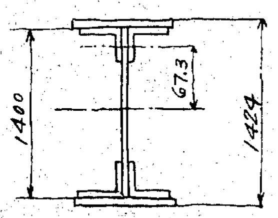
moment of inertia
4Ls 150×150×15 ---- 4×889 = 3556
$2\times85.5\times65.8^2$ = 740,374
743,930 cm⁴ gross
rivet holes 4-22ᵐ/ₘφ --- $4\times2.5\times1.5\times64^2$ = 61,440
682,490 Net

Web. 1390×9 ------- 201,421 cm⁴ gross
reduce 15% for rivets 171,210 " Net

J_t = 171,210 + 682,490 = 853,700 cm⁴
J_c = 201,421 + 743,930 = 945,351 "

Resisting Moment = $\frac{853700}{70}\times1200$ = 14635000 kgcm
= 14.6350 kgm

at 2 section



2Ls 150×150×15 = 85.5 cm² gross
1 Cov Pl 350×12 = 42.0 "
A_c = 127.5 cm² gross.
web Pl 1390×9.
effective depth = 134.6 cm

4Ls 150×150×15 ---- = 3556 cm⁴
$2\times127.5\times67.3^2$ ---- = 1154969
2 Cov. Pls ---- = 10
1158,535 cm⁴ gross

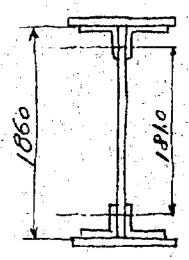
rivet holes
4 -22ᵐ/ₘφ --- $4\times2.5\times1.5\times64^2$ = 61,440
4 " $4\times2.5\times1.5\times69.3^2$ = 72,040
" " $4\times2.5\times1.5\times70.6^2$ = 59,912
} − 193,392
Net --- 965,142 cm⁴

Web. 139×9 ----- 201,421. cm⁴
15% reduced ---- 171,210 "
J_t = 171,210 + 965,142 = 1136,352 cm⁴

J_c = 201,421 + 1158,535 = 1359,956

R.M = $\frac{1136,352}{71.2}\times1200$ = 19152000 kgcm
= 191,520 Kgm

at 3 section



2Ls 150×150×15 = 85.5 cm²
1 Cov Pl. 350×12 = 42.0 "
A_c = 127.5 " gross
Web 1850×9.
effective depth = 181 cm

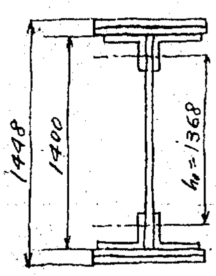
J_t = 22 63,515 cm⁴ J_c = 2681,287 cm⁴

R.M = $\frac{2263.515}{94.2}\times1200$ = 28834600 Kgcm
or 288,350 Kgm > 248,750 kgm

at 4 section same as for ②

R.M = 191,520 kgm > 159,550 kgm

at 5 section & 6, 7 section



2Ls 150×150×15 = 85.5
2 CovPls 350×12 = 84.0
169.50 cm gross
Web 1390×9.

J_t = 1505,552 cm⁴

R.M = $\frac{1505,552}{72.4}\times1200$ = 249.500 Kgm > 224,210 kgm

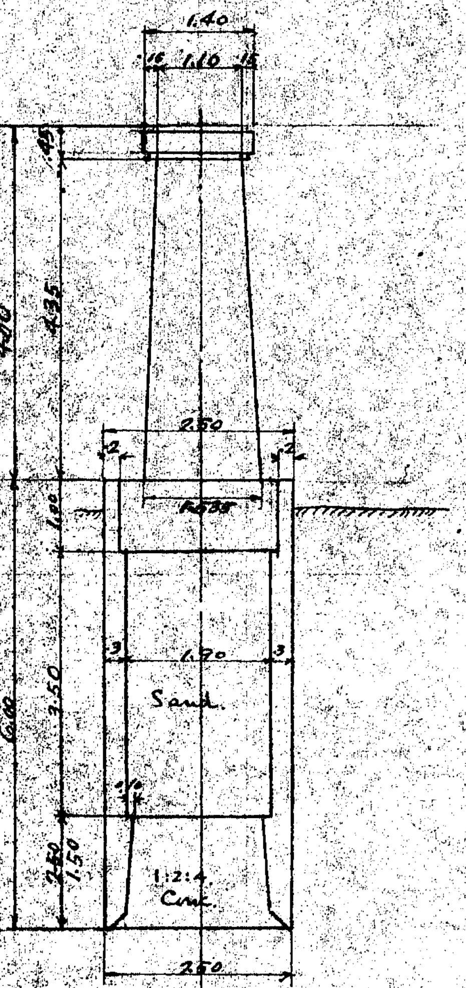
H. TASHIMA

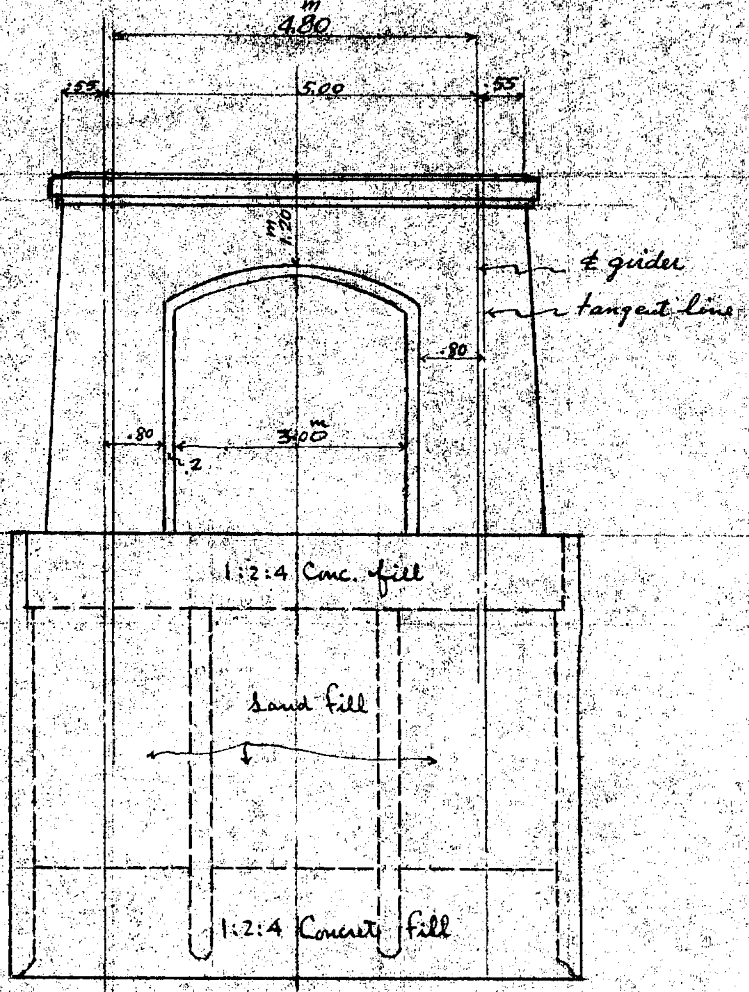
MADE BY H. Tashima DATE 8-2-15 **FILE NO.** 28

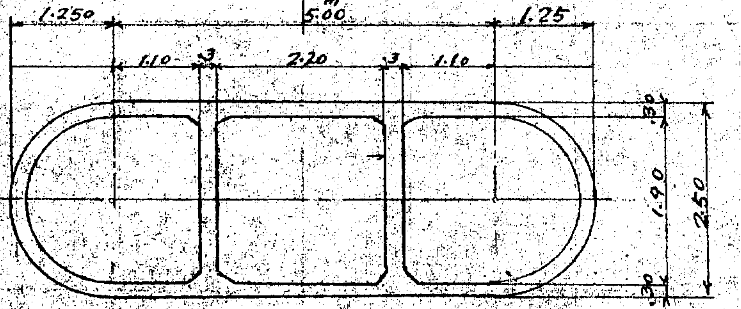
CHECKED BY DATE **PAGE NO.**

CALCULATIONS FOR

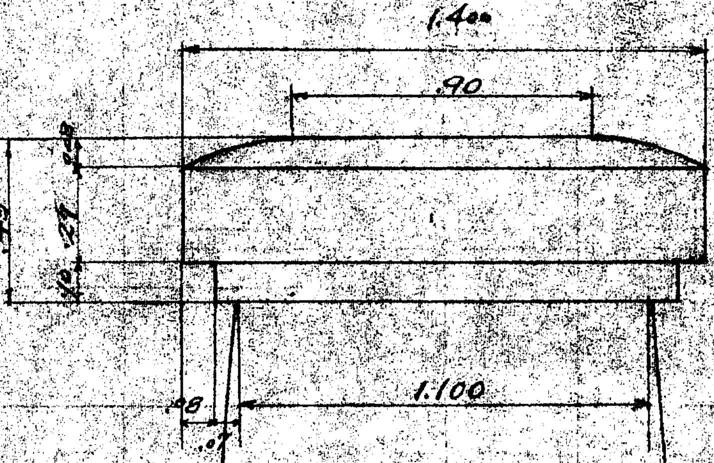
Design of piers







Scale 1:100



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Design of Pier

Load on pier for intermediate span.

Super imposed dead load say 61.000 kg

live load 39.000 "

total = 100.000 Kg for one shoe

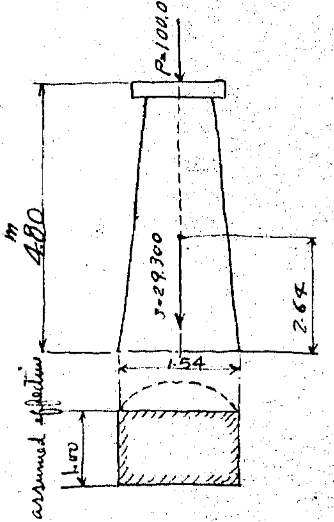
weight and Center of gravity of soft.

	Req. No	section	length	Volume		weight	lever arm	moment
Coping	1	1.4φ	0.35	0.539	@2400 =	1294	4.53	5,860
"	1	1.4×5.0	0.35	2.45	" =	5,880	4.53	26,636
"	1	1.24φ	0.10	0,121	" =	290	4.40	1.276
"	1	1.24×5.0	0.10	0.620	" =	1.488	4.40	6,547
shaft	1	1.318 ~~1.40~~φ	4.35	5.932 ~~4.132~~	" =	14.237 ~~9.917~~	1.79	25.484
"	2	1.318×0.8	4.35	9.173	" =	22,015	1.82 ~~2.53~~	40,070
top strut	1	mean 1.4×1.17	3.40	5.57	" =	13,370	3.65	48,800
				24.405 m³		58,574 Kg ·	2.64	= 154,673 kgm

Call this 24.400 cub.m

Stability of shaft.

Case 1.



stability at normal state

superimposed load say 2@100.000 = 200,000 kg for one pier

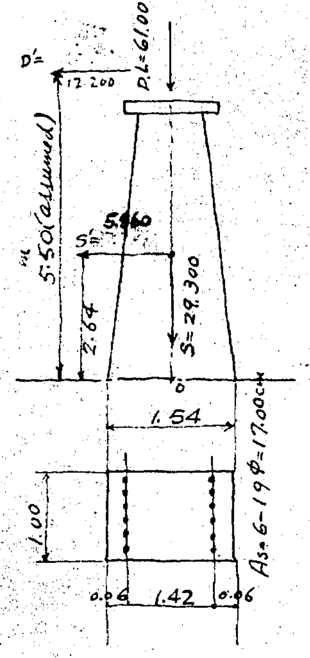
or 100,000 " for one shaft

dead weight of shaft 58.574÷2 = 29,300

129,300 Kg " " "

$$\text{Unit Compression} = \frac{129.300}{1.54\times100\times100} = 8.40\ \text{Kg/cm}^2 \quad \text{O.K.}$$

Case 2. Stability of shaft during earthquake K = 0.20 ~~0.30~~ assumed.



Vertical load		Hor. load.	lever arm	moment.
P =	61.000	p' = 12.200 ~~18.300~~ ×	5.50 =	$\boxed{100,650}$ 67.100
S =	29.300	S' = 5.860 ~~8.800~~ ×	2.64 =	$\boxed{23,230}$ 15,470
	90.300 kg	~~27.100~~ kg 18.060		$\boxed{123,880}$ Kgm 82.570

$$\text{Eccentricity} \quad e = \frac{\overset{82,570}{\boxed{123,880}}}{90.300} = \boxed{1.37m} \; 0.91m$$

$$e/h = \frac{0.910}{1.54} = 0.591 \qquad d'/h = {}^{0.06}/1.54 = 0.039.$$

$$p_0 = 2p = \frac{17\times2}{100\times153} = 0.0022. \qquad K = 0.32. \qquad L = 0.086$$

$$f_c = \frac{82.570\times100}{0.086\times100\times154^2} = 40.3\ \text{kg/cm}^2 \quad \text{O.K}$$

$$f_s = 15\times40.3\times\left(\frac{1.48\times100}{0.32\times154}-1\right) = 1209\ \text{kg/cm}^2 < 1200\times1.6$$

$$\text{unit shear} = \frac{18.060}{100\times7/8\times148} = 1.4\ \text{kg/cm}^2$$

$$\text{unit Bond.} = \frac{18.060}{5.97\times6\times7/8\times148} = 3.9\ \text{kg/cm}^2 \quad \text{O.K}$$

Design of well 6.0 m deep and 2.5 m wide

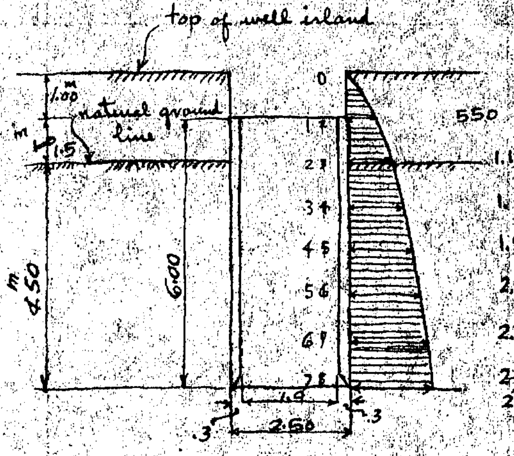
Temporary earth pressure on well during execution

Reffer to Ketchum's walls, bins and grain elevators, page 120 & 121.

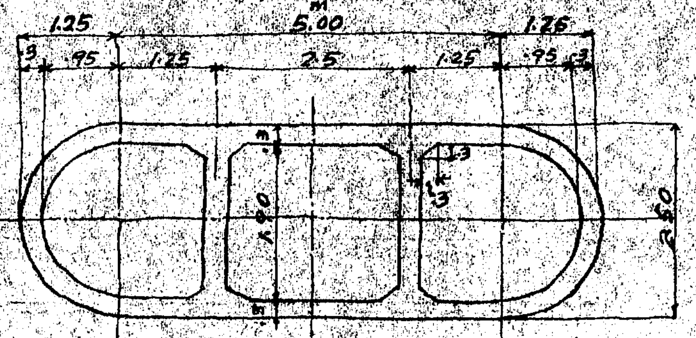
side pressure for temporary trench work

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

where L = lateral unit pressure in kg per sq meter at depth of meters
V = vertical " " " "
w = weight of earth in kg per cub. meter
ϕ = angle of repose of earth
r = $\tan\phi$ coeff. of friction of earth on earth
b = the distance in meter that the earth breaks around the well
ϕ' = angle of friction of earth on the surface of the well
assumed that $\phi = \phi' = 30°$, $K = \frac{1-\sin\phi}{1+\sin\phi} = 1/3$
b = 3 meters, $w = \overset{18000}{\cancel{2000}}$ kg per cub. meter



Depth of earth	temporary earth pressure		
1 meter	490 × $\frac{18000}{16000}$	=	550 kg/m²
2	980 ×	" =	1,100
3	1,370 ×	" =	1,540
4	1760 ×	" =	1,980
5	2060 ×	" =	2,320
6	2320 ×	" =	2,610
7	2520 ×	" =	$\boxed{2590}$ 2840
8	2740 ×	" =	3,080
9	2930 ×	" =	3,300
10	3070 ×	" =	3,450
12	3340 ×	" =	3,760
14	3520 ×	" =	3,960



Section at bottom.

moment on side wall $\frac{1}{12} wl^2$ assumed $l = 2.5$ m

	($\frac{1}{12} l^2 = 0.5208$)		($\frac{1}{16} \times l^2 = 0.3906$)
Section	side pressure	moment	moment on circular end $\frac{1}{16} wl^2$
top	550 kg/m²	290 kgm	215 kgm
1m below top	1,100 "	570 "	430
2	1,540	800	600
3	1,980	1030	770
4	2,320	1210	910
5	2,610	1360	1020
6	2,840	1480	1110

Section at bottom

moment on side wall = 1480 kgm moment on circular ends = 1110 kgm

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

$= \sqrt{\frac{148000}{100 \times 7.18}} = 14.35$ cm

use 25 cm effective depth with 5 cm insulation, total depth 30 cm

steel area req'd $= \frac{148000}{1200 \times 7/8 \times 25} = 5.65$ □cm per meter strip of wall

use 12 m/m ϕ bars at 20 cm spacing

end shear = $2840 \times 1.25 = 3.550$ kg

unit shear = $\frac{3550}{100 \times 7/8 \times 25} = 1.62$ kg/cm² O.K.

unit Bond = $\frac{3550}{\frac{377}{.20} \times 7/8 \times 55} = \overset{3.9}{\cancel{8.6}}$ kg/cm² < 6.0 kg O.K.

~~try again 12 m/m φ × 18 cm spacing used.~~

~~unit Bond~~ = $\cancel{\frac{3550}{\frac{3550}{0.18} \times 7/8 \times 25}}$ =

shear at end of chamfer $= 2840 \times 0.80 = 2270$ kg.

unit bond $= \dfrac{2270}{\frac{3.77}{2} \times \frac{7}{8} \times 25} = 5.5$ kg/cm² < 6.0 kg/cm² O.K.

section at 5.0 meter below top moment on side wall = 1360

steel area req'd $= \dfrac{1360 \times 100}{1200 \times \frac{7}{8} \times 25} = 5.18$ cm²

use 12 m/m φ bars at 21.8 cm spacing = 5.18 cm²

section at 4.0 meter below top moment on side wall = 1210 kgm

steel area req'd $= \dfrac{1210 \times 100}{1200 \times \frac{7}{8} \times 25} = 4.60$ cm²

use 12 m/m φ bars at 24.5 cm spacing

section at 3.0 meter below top moment on side wall = 1030 kgm

steel area req'd $= \dfrac{1030 \times 100}{1200 \times \frac{7}{8} \times 25} = 3.92$ cm² ~~sp~~

spacing req'd = 28.7 cm

section at 2.0 meter below top moment on side wall = 800 kgm

steel area req'd $= \dfrac{800 \times 100}{1200 \times \frac{7}{8} \times 25} = 3.05$ cm²

req'd spacing = 37 cm

section at 1.0 meter below top moment on side wall = 570 kgm

steel area req'd $= \dfrac{57000}{1200 \times \frac{7}{8} \times 25} = 2.17$ cm²

spacing req'd = 52 cm spacing 12 m/m φ

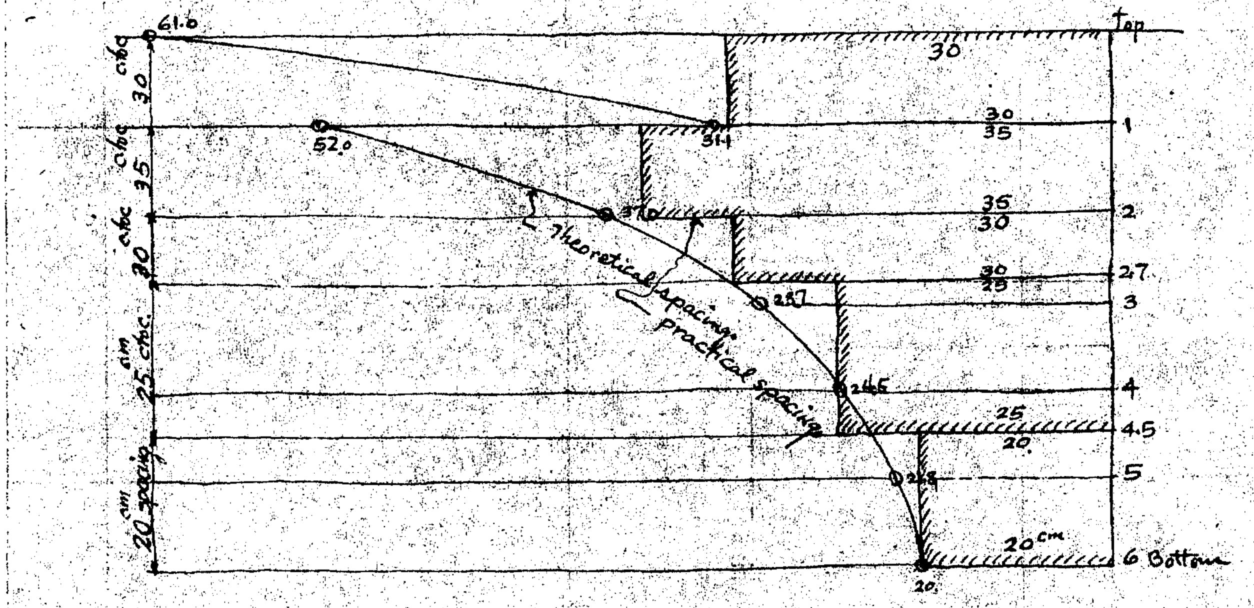
section at 0~1m from top moment on side wall = 570 kgm

effective depth say 15 cm

steel area req'd $= \dfrac{57000}{1200 \times \frac{7}{8} \times 15} =$ ~~2.16~~ 3.62 cm²

spacing req'd = 31.1 cm 12 m/m bars.

for circular ends of all sections, use same reinforcement at same spacings



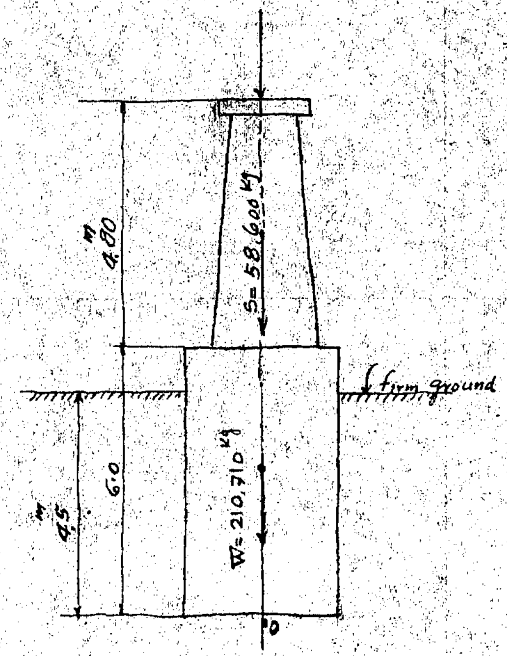
Reinforcement spacing diagram

Stability of pier

Case 1. Stability at normal state

weight of and center of gravity of well

total area of well	area of partition	circumference of well
5.00×2.5 = 12.50 $^{\square m}$	1.9×4.4 = 8.36	5.0×2 = 10^{m}
2.5φ = 4.90	1.9φ = 2.84	2.5φ = 7.85
17.40m²	0.3×3×4 = -0.36	17.85^{m}
	10.84$^{\square m}$	



sectional area of shell = 17.40 − 10.84 = 6.56 $^{\square m}$

0 to 1.0^{m} from top
Vol. of Concrete 17.4×1 = 17.4$^{m^3}$ @ 2.200 = 38,280^{kg} ... 38,280

1m to 2m from top
Vol of concrete 6.56×1 = 6.56 @ 2.200 = 14,430
Vol of sand 10.84×1 = 10.84 @ 1.700 = 18,430
32,860 kg

2m to 3.5 from top
vol of concrete 6.56×1.5 = 9.84 @ 2,200 = 21,650
vol of sand. 10.84×1.5 = 16.26 @ 1.700 = 27,640
49,290$^{kgs.}$

3.5m to 4.50 from top
Vol of concrete 6.56×1.0 = 6.56 @ 2.200 = 14,430
Vol of sand 10.84×1.0 = 10.84 @ 1.700 = 18,430
32,860

4.5m to 6.0^{m} from top 17.40×1.5 = 26.1 @ 2200 = 57,420 ... 57,420

total weight of well = 210,710 Kgs.

Load on well
superimposed D.L + L.L = 2×100,000 = 200,000^{kgs}
weight of shaft = 58,600
weight of well = 210,710
total = 469,310 Kgs

Frictional resistance of earth on well surface assumed at 1000 $^{kg}/_{m^2}$

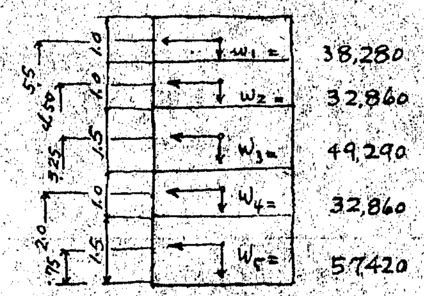
Total friction ; assuming effective penetration of well = 4.5.0 meters
= 1000 × 17.85 × 4.5 = 80,330 Kg.

Resulting pressure on well = 469.310 − 80,330 = 388,980 Kg.

Bearing pressure on ~~well~~ the bottom of well = $\frac{388,980}{17.40}$ = 22,360 $^{kg}/_{m^2}$ (or 2.05 $^{kg\,ton}/_{\square'}$)

If frictional resistance be neglected 22,360 × $\frac{469,310}{388,980}$ = 27,000 $^{kg}/_{m^2}$ (or 2.46 $^{ton}/_{\square'}$)

Center of gravity of well

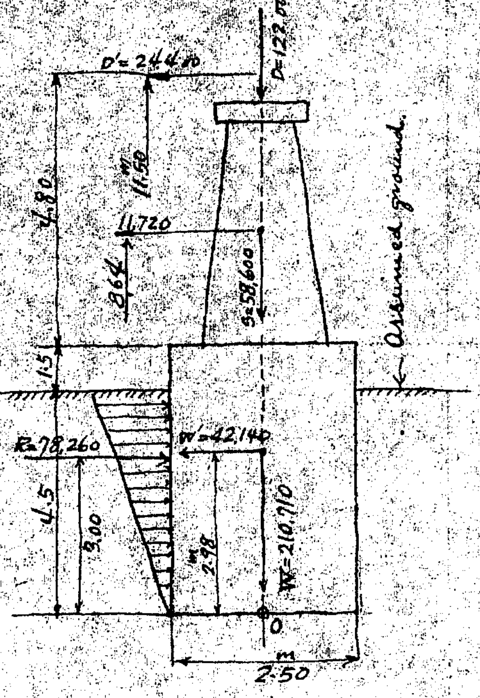


section	weight		lev. arm	moment
0~1^{m}	38,280	×	5.50	210,540
1−2	32,860	×	4.50	147,870
2−3.5	49,290	×	3.25	160,200
3.5−4.5	32,860	×	2.00	65,720
4.5−6.0	57,420	×	0.75	43,070
	210,710 Kg	×	2.98 m	627.400 Kgm

MADE BY H. Tashima **DATE** 8-2-19 **FILE NO.** 6

CHECKED BY ______ **DATE** ______ **PAGE NO.** 33

Case 2. Stability of pier during earthquake K = ~~ad~~ 0.2 assumed.



Taking moment about pt 0.

Loads	Hor. forces	Vert. forces		lever arm		moment
D		122,000				
D'	24400		×	11.50	=	280.600
S		58.600				
S'	11.720		×	8.64	=	101.260
W		210,710				
W'	42,140	–	×	2.98	=	125,580
	ΣH=78,260	ΣV=391,310			ΣM=	507,440 Kgm

Frictional resistance on side wall of well, mean width of well say 7.0 m

$1.000 \times 4.5 \times 7.0 = 31,500$ kg

friction Couple $= 31.500 \times 2.5 = 78,750$ kgm

moment due to earth reaction

$78,260 \times 3.00 = \underline{234,780}$

313,530 kgm

Resisting moment = $507.440 - 313530 = 193,910$ kgm

Eccentricity, $e = \frac{193.910}{\cancel{25\times7.0}\ 391.310} = 0.495$ meter Resultant force out of middle third

pressure area = $3(1.25-0.495) = 2.265^{m} \times 7.0^{m}$

max toe pressure = $\frac{391.310\times2}{2.265\times7.0} = 49.300\ kg/m^2$ or $(4.5\ t/\square')$

max reactional pressure on earth (~~1.0~~ 1.5 m below ground surface)

$\boxed{= wh \times \frac{1+\sin\phi}{1+\sin\phi} = \text{say } 3wh = 3\times1600\times1 = 4800\ kg/m^2}$

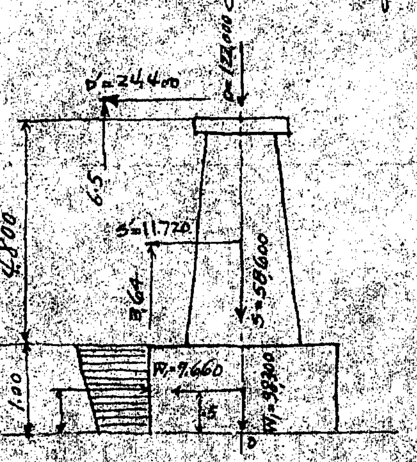
$= \frac{78,260\times\cancel{1.0}\ 1.5}{7\times4.5} = \cancel{2480}\ 3720\ kg/m^2$

safe horizontal bearing power of earth at ~~1.0~~ 1.5 meter below ground surface

$= wh \times \frac{1+\sin\theta}{1-\sin\theta} = \text{say } 3h = 3\times1600\times1.5 = 7200\ kg/m^2$

$> 3720\ kg/m^2$

Vertical reinforcement for the well shell.



moment at 1 meter below of ~~tob~~ well

taking moment about pt 0.

loads	Hor. forces	Vert. forces		arm		moment
D		122,000				
D'	24,400		×	6.5	=	158,600
S		58,600				
S'	11,720		×	3.64	=	42,660
W1		38,300				
W1'	7,660		×	0.50	=	3,830
	ΣH=43,780 kg	ΣV=218,900				ΣM=205,090 kgm

Eccentricity $= \frac{205,090}{218,900} = 0.940$ meter resultant force out of middle third

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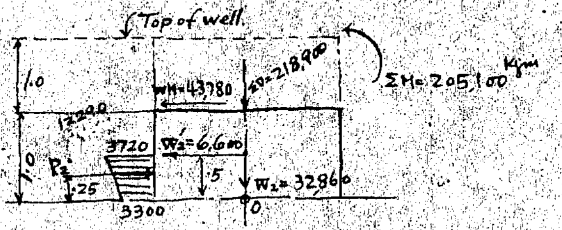
CALCULATIONS FOR

moment at 2.0 meter below top of well.



	Hor. forces	Vert. forces	arm	moment
ΣV		218,900		
ΣH	43,780		× 1.0 =	43780
W_2		32,860		
W_2'	6,600		× 0.5 =	3,300
ΣM				205,100
R_2	− 12,290 ~~− 24,570~~		× 0.25 =	− 3,070
	ΣH= 38,090	ΣV=251,760		ΣM= 249,110 Kgm

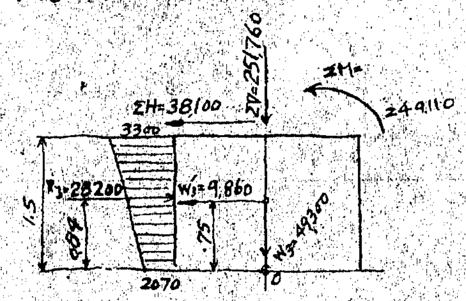
friction couple $78,750 \times \frac{0.5}{4.5}$ = − 8,750

240,360 Kgm

Eccentricity $\varepsilon = \frac{240.360}{251.760} = 0.95$ meter

resultant force out of middle third.

moment at 3.5 meter below top of well



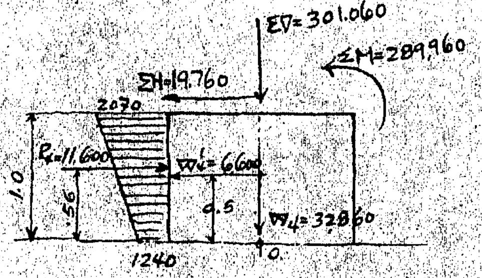
ΣV		251.760		0
ΣH	38,100		× 1.5 =	57,150
ΣM			=	249.110
W_3		49,300		0
W_3'	9,860		× 0.75 =	7.400
R_3	− 28,200		× 0.84 =	− 23,700
	ΣH= 19.760	ΣV=301,060		ΣM= 289.960 Kgm

friction Couple $\frac{78,750 \times 2.0}{4.5}$ = − 35,000

254,960.

Eccentricity $\varepsilon = \frac{254960}{301.060} = 0.847$ meter resultant force out of middle third

moment at 4.5 meter below top of well



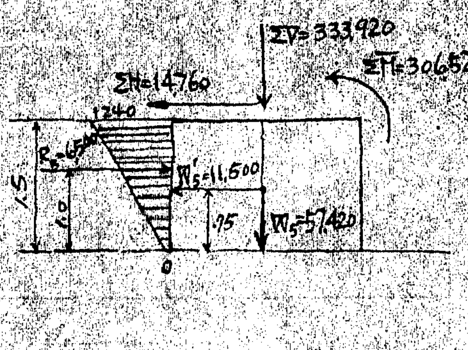
ΣV		301.060	· 0	
ΣH	19,760		× 1.0 =	19,760
$\Sigma \overline{M}$			=	289.960
W_4		32,860	× 0	
W_4'	6,600		× 0.5 =	3,300
R_4	− 11,600		× 0.56 =	− 6,500
	ΣH= 14760 kg	ΣV=333,920		ΣM= 306,520

friction Couple 78750 × 3.0 ÷ 4.5 = − 52.500

254.020 Kgm

Eccentricity ε = 254,020 ÷ 333920 = 0.76 meter

moment at 6.0 meter below top of well (bottom of well)

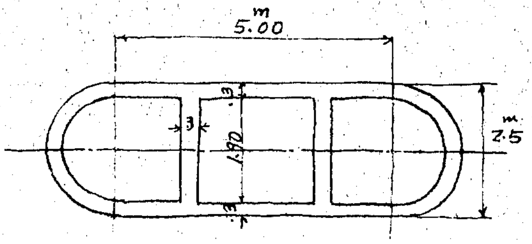


ΣV		333,920	× 0 =	0
ΣH	14760		× 1.5 =	22,140
ΣM			=	306,520
W_5		57420	× 0 =	0
W'_5	11,500		× .75 =	8,630
R_5	− 6,500		× 1.0 =	− 6,500
	ΣH 19,760	ΣV= 391.340		ΣM= 330.790 kgm

friction couple − 78,750

252,040 Kgm

Eccentricity ε = 252,040 ÷ 391.340 = 0.77 meters
Resultant force out of middle third



moment of inertia of section, effect of reinforcement neglected

Circular ends $0.049 \times 2.5^4 = 1.914^{m^4}$

straight portion $\frac{5.00 \times 2.5^3}{12} = \underline{6.51}$

8.424

moment of inertia of hollow

circular ends $0.049 \times 1.9^4 = 0.638^{m^4}$

straight portion $\frac{4.40 \times 1.9^3}{12} = \underline{2.515}$

$-\underline{3.153}$

moment of inertia of shell ---- $I = 5.271\ m^4$

sectional area of well shell = $6.56\,m^2$.

section at 2 meter below top of well. Direct load = 251,760 Kg moment = 240,360 kgm

moment stress $= \frac{240.360 \times 1.25}{5.271} = 57,000\ \text{kg}/\text{m}^2$ C or T. $= 5.7\ \text{kg}/\text{cm}^2$

direct stress (Compression) $= \frac{251.760}{6.56} = 38,380\ \text{kg}/\text{m}^2$ C $= 3.84\ \text{kg}/\text{cm}^2$

Resultant max fibre stress $= 3.84 \pm 5.7 = \underline{9.54}\ \text{kg}/\text{cm}^2$ C or $\underline{1.86}\ \text{kg}/\text{cm}^2$ T.

section at 4.5 meter below top of well

Direct load = 333,920 kg moment = 254,020 kgm

moment stress $= \frac{254.020 \times 1.25}{5.271} = 60,300\ \text{kg}/\text{m}^2 = 6.03\ \text{kg}/\text{cm}^2$ C or T.

Direct stress (Compression) $= \frac{333.920}{6.56} = 50.900\ \text{kg}/\text{m}^2 = 5.09\ \text{kg}/\text{cm}^2$ C

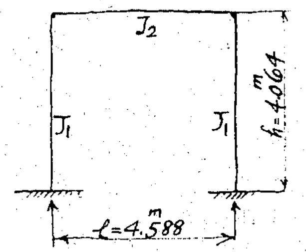
Resulting max fibre stress $= 5.09 \pm 6.03 = 11.12\ \text{kg}/\text{cm}^2$ C

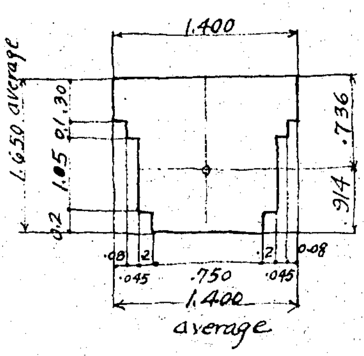
or $0.94\ \text{kg}/\text{cm}^2$ T.

steel area required for tension $= \frac{18,620}{1200 \times 1.6} = 9.7\,\text{cm}^2$ per m of side wall.

use ^{12} m/m φ bars at ^{20} cm ctoc on both sides $= 1.13 \times 10 = 11.3\ \text{cm}^2$ ~~per meter of~~ — OK.

Transverse moment

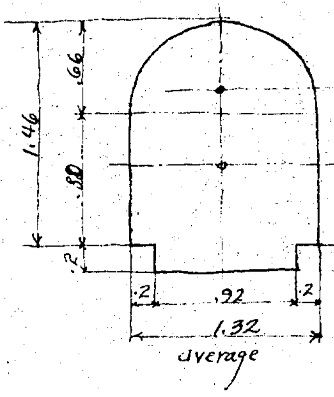




moment of Inertia of Top strut J_2

area	0.750 × 0.20	=	0.150 × 0.010	=	0.0150
	1.050 × 1.15	=	1.207 × 0.725	=	0.8751
	1.240 × 0.10	=	0.124 × 1.750	=	0.2170
	1.400 × 0.30	=	0.420 × 1.500	=	0.6300
			1.901 × 0.914	=	1.7371

$J_2 = 0.4496\ m^4$

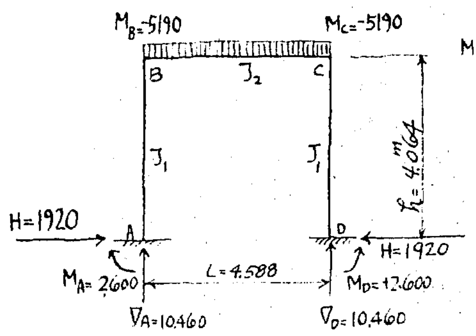


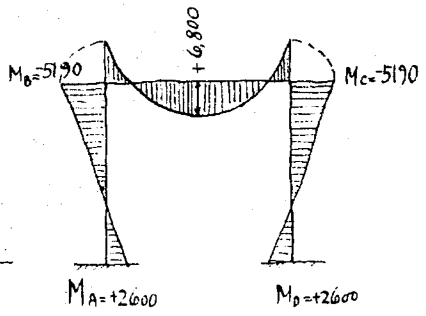
moment of inertia of shaft J_1:—

area	0.92 × 0.20	=	0.184 × 0.10	=	0.0184
	1.32 × 0.80	=	1.056 × 0.60	=	0.6336
	1.32φ ÷ 2	=	0.684 × 1.28	=	0.8755
			1.924 × 0.794	=	1.5275

$$\frac{0.92\times0.2^3}{12}+0.184\times.694^2+\frac{0.80^3\times1.32}{12}+1.056\times0.194^2+0.007\times1.32^4+0.684\times0.486^2=0.368\,m^4 \text{ -- } J_1$$

Dead Load moment





weight of top strut 1.901 × 2400 = 4560 kg/meter = q
See Kleinlogel's Rahmenformeln page 89.

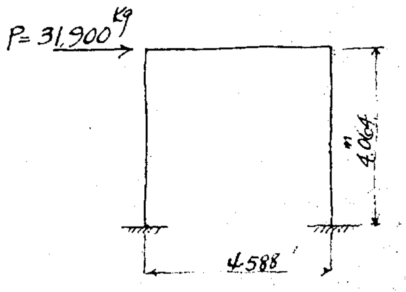
$$K=\frac{J_2}{J_1}\cdot\frac{h}{l}=\frac{0.4486}{0.3680}\times\frac{4.064}{4.588}=1.08$$

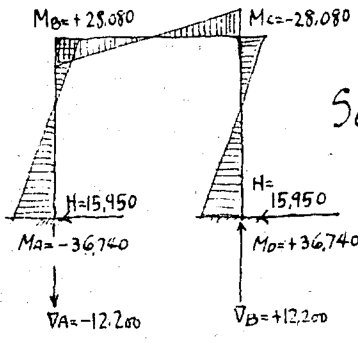
$$V=\frac{ql}{2}=\frac{4560\times4.588}{2}=10.460^{kg}$$

$$H=\frac{ql^2}{4h(K+2)}=\frac{4560\times4.588^2}{4\times4.064\times3.08}=1.920^{kg}$$

$$M_B=M_C=\frac{-ql^2}{6(K+2)}=+2.600^{kgm}$$

$$+M_{max}=+\frac{ql^2}{24}\times\frac{2+3K}{K+2}=+6,800^{kgm}$$





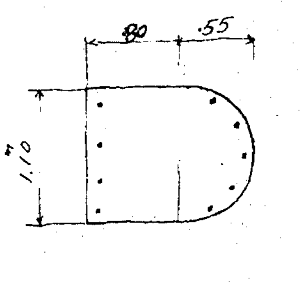
Seismic force due to

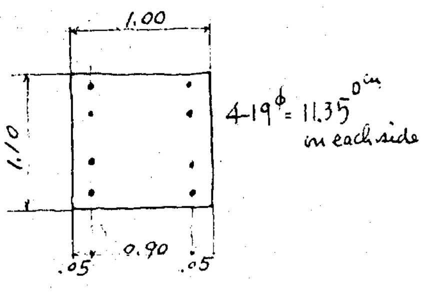
			Top	Bottom
Top strut	= 4560 × 3.00 × 0.20	=	2,735 kg	—
Shaft say	45,000 × 0.2 × $\frac{2.15}{4.064}$	=	4,760	4230
D.L on shoes	61.000 × 2 × 0.2	=	24,400	—
	180,680 kg		31,895	4.230

Vertical load on one shaft = 90,340 kgs   Call this 31.900 kg = P

K = 1.08   3K = 3.24   6K+1 = 7.48   3K+1 = 4.24

section at C.





$$V=\frac{3Phk}{l(6K+1)}=\frac{3\times31,900\times4.064\times1.08}{4.588\times7.48}=12,200^{kg}$$

$$H=P\div2=15,950^{kg}\quad M_A=-\frac{Ph}{2}\cdot\frac{(3K+1)}{(6K+1)}=-\frac{31.900\times4.064\times4.24}{2\times7.48}=-36,740\,kgm$$

$$M_D=+\frac{Ph}{2}\times\frac{3K+1}{6K+1}=+36,740\,kgm$$

$$M_B=+\frac{Ph}{2}\cdot\frac{3K}{6K+1}=\frac{31.900\times4.064}{2}\times\frac{3.24}{7.48}=+28,080^{kgm}$$

$$M_C=-\ ''\quad ''\quad=-28.080''$$

Summary of moments, Reactions & Hor. thrusts

	M_A	M_B	M_C	M_D	M_{center}	V.	H.
Dead Load	+2.600	-5190	-5190	+2600	+6,800	+10.460	± 1920
Seismic forces	-36,740	+28,080	-28,080	+36,740	0	± 12,200	+ 15,950
	-34,140 kgm	+22,890 kgm	-33270	+39340	+6,800	+22,660 kg - 1.740	+17.870 kg +14.030

~~Transverse~~ Transformed rectangle of equal moment of inertia assumed 1.10 × 1.0 = 1.10 m²

Vertical load = D.L on shoes = 61.000
Top. strut = 6.840
67,840 kg

moment = $M_C = -33,270^{kgm}$

$$\text{Eccentricity}=\frac{33.270}{67.840}=0.490^{m}$$

$e/h = 0.490/1.00 = 0.490$ $P_0 = 2p = \frac{11.35\times2}{1.10\times1.0\times10000} = 0.002$ $K = .375$

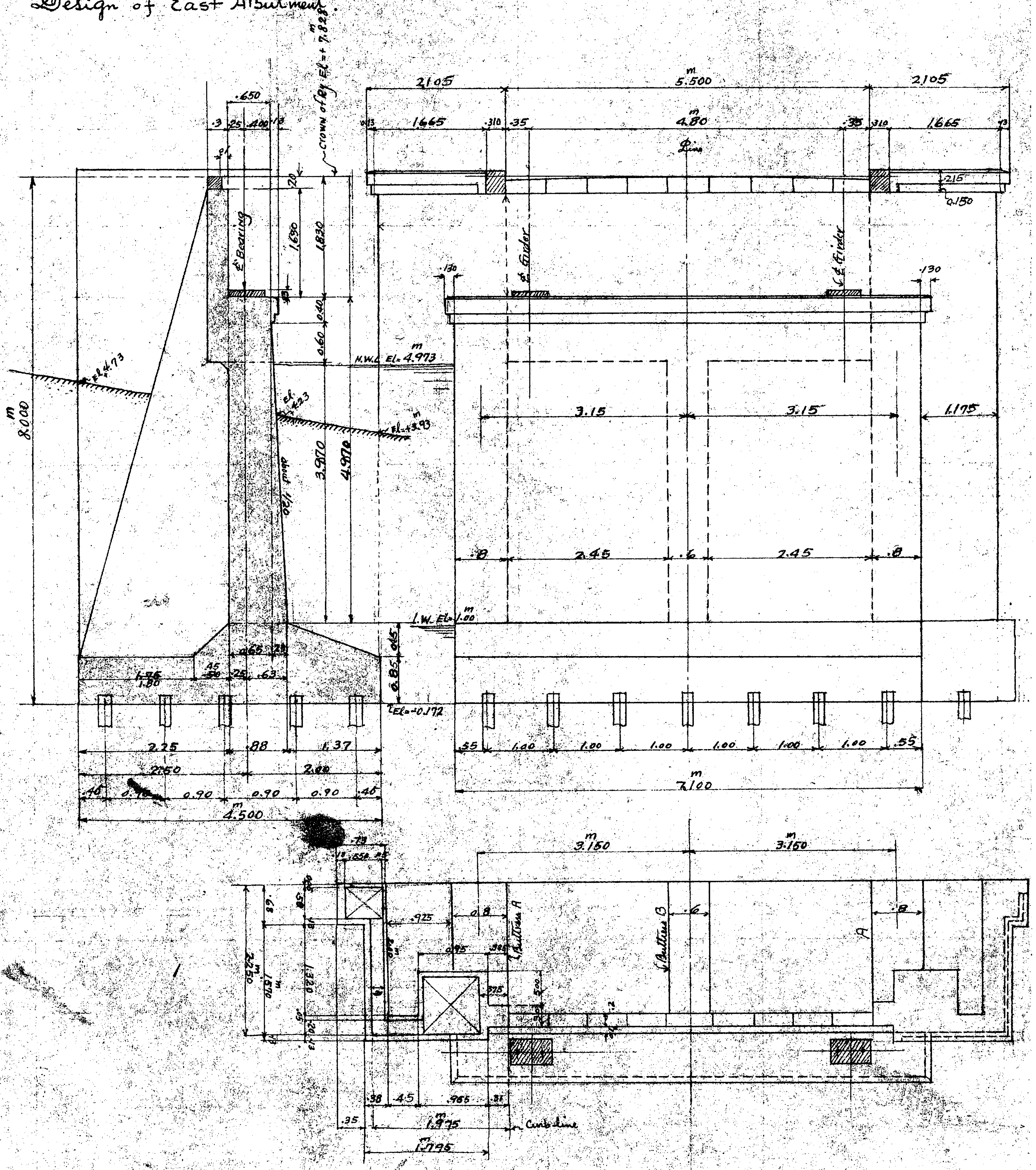
$d'/h = 0.05/1.00 = 0.05$ $L = 0.087.$

$$f_c=\frac{33270\times100}{0.087\times1\,10\times100^2}=34.8\ kg/cm^2\quad o.K$$

$$f_s=15\times34.8\left(\frac{95}{.375\times100}-1\right)=800\ kg/cm^2\quad o.K.$$

Kamitsu Ka Bashi

Design of East Abutment.



sketch of East Abutment scale 1:60

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

H. TASHIMA

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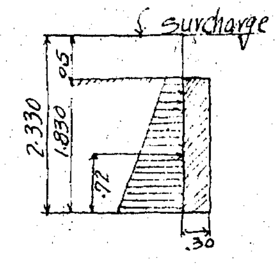
CALCULATIONS FOR

Kamitsuka-Bashi
parapet wall of abutment

Case 1. Stability at Normal state



Surcharge due to live load = 0.50 meter assumed

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

$\frac{1}{3} \times 1600 \times 2.33 = 1243$

average $= 1510 \div 2 = 755\ ^{kg}/_{m^2}$

Total pressure $= 755 \times 1.83 = 1380$ kg lin meter of wall

moment on parapet wall $= 1380 \times .72 = 994$ kgm

effective depth required $= \sqrt{\frac{99400}{100 \times 7.18}} = 11.8$ cm

Use 27 cm effective depth with 3 cm insulation, total depth 30^{cm}

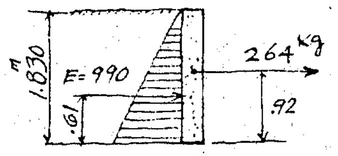
steel area required $= \frac{99400}{1200 \times 7/8 \times 27} = 3.5\,cm^2$ per meter strip of wall

Use $12^{m/m}$ φ bars at 30 cm c. to c both sides = 3.76 cm

unit shear $= \frac{1380}{100 \times 7/8 \times 27} = 0.58\ ^{kg}/_{cm^2}$ O.K.

unit bond $= 1380 \div (12.55 \times 7/8 \times 27) = 4.67\ ^{kg}/_{cm^2}$ O.K

Case 2. stability of parapet wall during earthquakes K=0.20 assumed



earth pressure during earthquake $= 0.185wh^2 = .185 \times 1600 \times 1.83^2 = 990^{kg}$

weight of parapet wall per meter $= 0.30 \times 1.83$ @2400 = 1,320 kg

seismic force $= 1,320 \times 0.20 = 264$ kg/meter

moment $= 990 \times 0.61 = 604$

$264 \times 0.92 = 243$

847 kgm

assumed section is ample.

Design of shaft.

weight and Center of gravity of shaft.

Buttress A (Both sides)

Name	section	length	Req'd no.	Vol. m^3	wt. kg	hor. Arm m	M_H	Vert. Arm m	M_V kgm
light pedestals	.85 × .85	1.10	1	.795	@2400 = 1,910	1.31	2,500	7.35	14,040
〃 〃	.65 × .65	1.50	1	.634	〃 = 1,520	1.31	1,990	8.65	13,150
parapet wall	.30 × 2.59	1.23	1	.956	〃 = 2,290	1.03	2,360	5.67	12,980
stone on wall	.20 × .20	1.23	1	.050	@2600 = 130	1.08	140	6.60	860
Column	.90 × .475	½ × 2.07	1	.442	@2400 = 1,060	1.48	1,570	6.10	6,470
〃	.60 × .475	2.07	1	.590	〃 = 1,420	1.48	2,100	5.73	8,140
Wing wall	.65 × 6.80	.46	1	2.033	〃 = 4,880	1.36	6,640	2.80	13,660
Coping projections	.11 × .36	.46	1	.018	〃 = 40	.83	30	6.60	260
front wall	.765 × 2.025	4.87	1	7.544	〃 = 18,110	.49	8,870	2.32	42,000
Coping projections	.40 × .120 平均	2.80	1	.134	〃 = 320	.22	70	4.67	1,490
Coping	.325 × .40	.36	1	.047	〃 = 110	1.08	120	6.60	730
Buttress	.34 × .80	2.33	1	.634	〃 = 1,520	1.41	2,140	4.65	7,070
〃	.98 × .80	3.87	1	3.034	〃 = 7,280	1.37	9,970	1.935	14,080
〃	½ × 1.12 × .80	3.87	1	1.734	〃 = 4,160	2.23	9,280	1.290	5,370
			Σ =		44,750 kg	1.07 m	47,780 kg	3.135 m	140,300 kgm

H. TASHIMA

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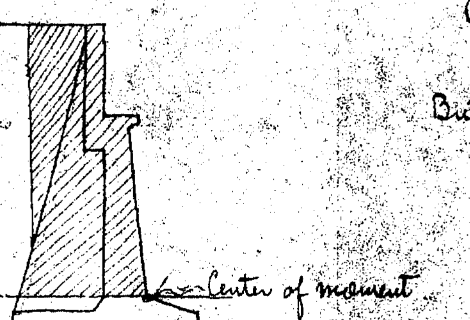
CALCULATIONS FOR ______ CHECKED BY ______ DATE ______ **PAGE NO.** 39

Buttress B (at center)	section	length	req'd no.	vol. m³	wt kg	hor. arm	MH	Vert. arm	Mv.
parapet wall	.3 × 2.6	3.05	1	2.379	@ 2400 = 5,710	1.03	5,880	5.67	32,380
stone on parapet wall	.2 × .20	3.05	1	1.220	@ 2600 = 3,170	1.08	3,420	6.60	20,920
front wall	.765 × 3.05	4.87	1	11.364	@ 2400 = 27,270	0.49	13,360	2.32	63,270
Coping projections	0.40 × .12	3.05	1	0.146	@ 2400 = 350	0.22	80	4.67	1,630
buttress	0.34 × .60	2.33	1	0.475	〃 = 1,140	1.41	1,610	4.65	5,300
〃	0.98 × .60	3.87	1	2.276	〃 = 5,460	1.37	7,480	1.935	10,570
〃	½ × 1.12 × .60	3.87	1	1.300	〃 = 3,120	2.23	6,960	1.290	4,020
				19.16 m³	46,220 kg	0.839	38,790	2.978 m	138,090



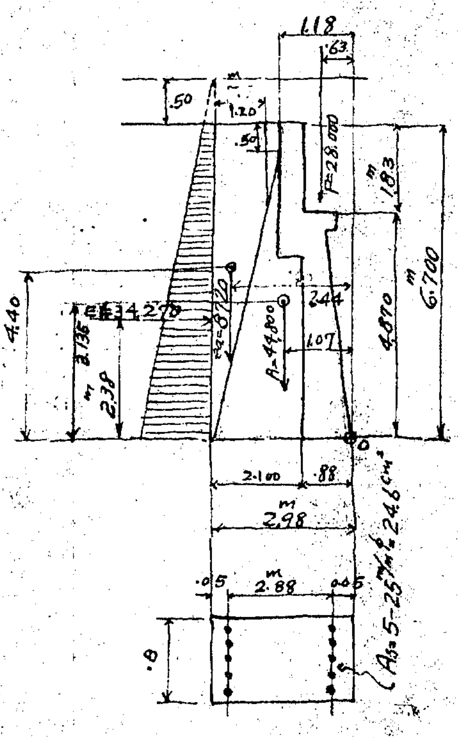
Center of gravity of whole shaft

		kg	hor. arm	MH	Vert arm	Mv
Buttress A	2 @ 44,750 =	89,500		95,560		280,600
〃 B		46,220		38,790		138,090
		135,720 kg	0.990 m	134,350	3.09 m	418,690

Stability of shaft

Buttress A (End buttress)

Case 1. Stability at normal state



superimposed load on abutment D.L = 15,000 kg say

L.L = 13,000 〃

28,000 kg for one buttress A

weight of earth on buttress 4.39 × $\overset{0.475 \times 1.20}{\cancel{0.8} \times \cancel{1.35}}$ @ 1600 = $\overset{4500}{\cancel{7,590}}$ kg 4000 kg

〃 〃 3.65 × $\overset{1.70}{\cancel{1.85}}$ × 0.475 @ 〃 = ~~5,130~~ 〃 4720

~~9,630 kg.~~ 8720 kg

Surcharge due to live load assumed 0.5 meter

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

Bottom $\frac{1}{3} \times 1600 \times 7.2$ = 3840

4,107 ÷ 2 = 2054 kg/m² average

total pressure on buttress A 2054 × 6.7 × 2.49 m = 34,270 kg.

Taking moment about toe O

loads	Hor. forces	Vert. forces		lever arm		moment
P		28,000	×	.63	=	17,640
A		44,800	×	1.07	=	47,940
ea		8,720	×	2.44	=	21,280
E	34,270		×	− 2.38	=	− 81,560
	ΣH = 34,270 kg.	ΣV = 81,520 kg.		0.065 m		ΣM = 5,300 kgm

Eccentricity $e = \frac{2.980}{2} - 0.065 = 1.425^{m}$ right

Design buttress as a rectangular wall of 0.8 × 2.98 m at bottom

moment on buttress = 81.520 × 1.425 = 116,170 kgm

$e/h = 1.425/2.98 = 0.478$ $d'/h = 0.05 \div 2.98 = 0.0168$

$R_0 = 2p = \frac{2 \times 24.6}{80 \times 298} = 0.00206$

from prepared diagram we have $K = 0.40$ $L = 0.092$

$f_c = \frac{116,170 \times 100}{0.092 \times 80 \times 298^2} = 17.7$ kg/cm² o.K

$f_s = 15 \times 17.7 \left(\frac{293}{.4 \times 298} - 1\right) = 390$ kg/cm² o.K

unit shear = $\frac{34,270}{80 \times 7/8 \times 293} = 1.67$ kg/cm² o.K

unit bond = $\frac{34,270}{7.85 \times 7/8 \times 293 \times 5} = 3.4$ kg/cm² ok.

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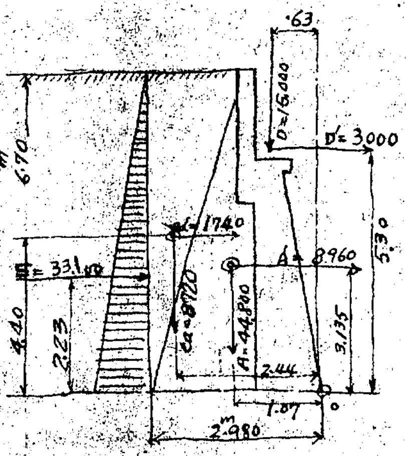
CALCULATIONS FOR

Case 2. Stability during earthquake K=0.2 assumed.

Earthquake during earthquake $= 0.185 \times 1600 \times 6.7^2 \times \overset{m}{2.49} = 33{,}100^{kg}$



loads	Hov. forces	Vert. forces		arm		moment
D		15.000	×	.63	=	- 9.450
D'	3.000		×	5.30	=	15.900
A		44.800	×	1.07	=	- 47.940
A'	8.960		×	3.135	=	24.090
ea		8.720	×	2.44	=	- 21,280
ea'	1.740		×	4.40	=	7.660
E	33,100		×	2.23	=	73,810
	ΣH= 46,800 kg	ΣV= 68,520 kg	·	0.624 m		ΣM=+42,790 kgm

Eccentricity $\varepsilon = \frac{2.98}{2} + 0.624 = \overset{m}{2.114}$ right.

moment on buttress $= 68.520 \times 2.114 = 144{,}850^{kgm}$

$e/h = 2.114/2.98 = 0.709$ $d'/h = 0.0168$ $p_o = 0.00206$

$K = 0.285$ $L = 0.082$

$f_c = \frac{144{,}850 \times 100}{0.082 \times 80 \times 298^2} = 24.7\ ^{kg}/cm^2$ ok

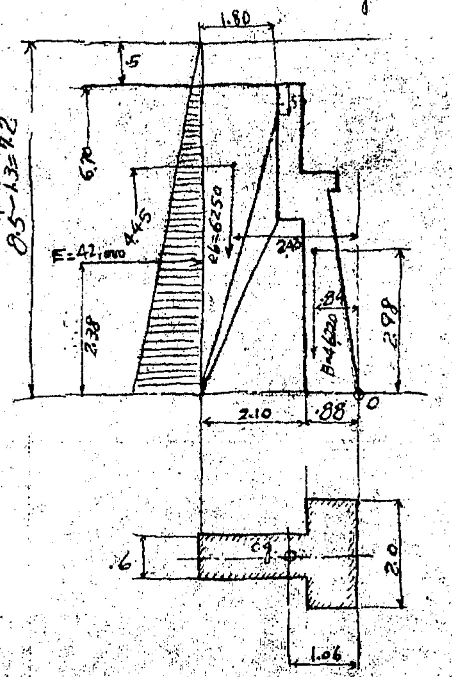
$f_s = 15 \times 24.7 \left(\frac{293}{.285 \times 298} - 1\right) = 910\ ^{kg}/cm^2$ ok

unit shear $= \frac{46{,}800}{80 \times 7/8 \times 293} = 2.3\ ^{kg}/cm^2$ o.k

unit bond $= \frac{46{,}800}{7.85 \times 5 \times 7/8 \times 293} = 4.6\ ^{kg}/cm^2$ ok.

Buttress B (center)

Case 1. stability at normal state



weight of earth on buttress $3.615 \times 0.6 \times 1.8$ @1600 = $6{,}250^{kg}$

surcharge due to live load assumed 0.5 m

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

bottom $\frac{1}{3} \times 1600 \times 7.2 = 3840$ kg/m²

$4107 \div 2 = 2054$ average

Total pressure on buttress B $= 2054 \times 6.7 \times 3.05 = 42{,}000^{kg}$

Taking moment about toe 0.

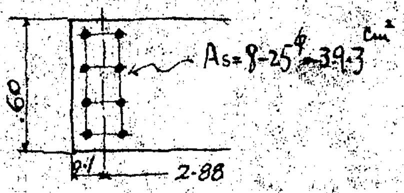
loads	Hor. forces	Vert. forces		lever arm		moment
B		46,220	×	.84	=	38,820
eb		6,250	×	2.43	=	15,190
E	42,000		×	2.38	=	- 100,000
	ΣH= 42,000 kg	ΣV= 52,470	·	0.896 m		ΣM=-45,990 kgm

Design of buttress as a wall of T-section

flange $.88 \times 2.0 = 1.76 \times 0.44 = 0.774$

web $.60 \times 2.1 = 1.26 \times 1.93 = 2.432$

$3.02^{\square cm} \times 1.06 = 3.206$



Eccentricity $= 1.06 + 0.876 = 1.936^m$

moment on buttress $= 52{,}470 \times 1.936 = 101{,}600^{kgm}$

direct compression $= \frac{52{,}470}{3.02 \times 10{,}000} = 1.74\ kg/cm^2$

Stress due to moment

steel area req'd $= \frac{101{,}600 \times 100}{1200 \times 7/8 \times 288} = 33.6\ cm^2$ Use $\overset{mm\phi}{8\text{-}25}$ bars $= 39.3^{\square cm}$

steel ratio $p = \frac{39.3}{200 \times 288} = 0.00068$ $t/d = 88/288 = 0.305$ Neutral axis in the flange

$K = 0.18$ $j = .94$ $f_s = \frac{M}{A_s j d} = \frac{101{,}600}{39.3 \times .94 \times 288} = 950\ ^{kg}/cm^2$

$f_c = \frac{f_s K}{n(1-K)} = \frac{950 \times 0.180}{15(1-0.18)} = 13.9\ ^{kg}/cm^2$ o.K

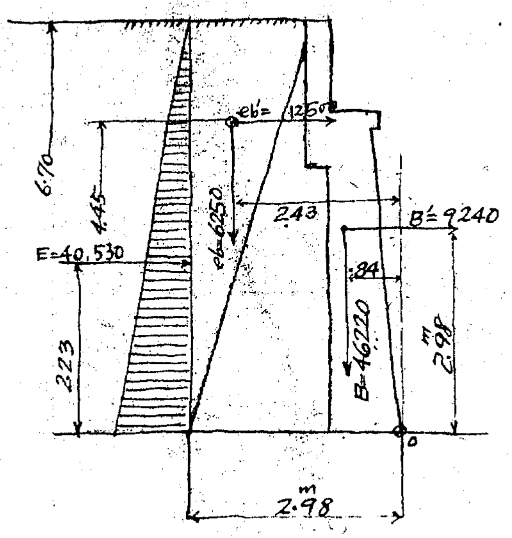
unit shear $= \frac{42{,}000}{60 \times 0.94 \times 288} = 2.6\ ^{kg}/cm^2$

unit bond $= \frac{42{,}000}{7.85 \times 8 \times .94 \times 288} = 2.0\ ^{kg}/cm^2$

o.K

Case 2. Stability during earthquake K=.2 assumed

Earth pressure during earthquake = .185 × 1600 × 6.7² × 3.05 = 40,530 kg



loads	Hor. forces	Vert. forces		Lever arm		moment
B		46,220	×	0.84	=	– 38,820
B'	9240		×	2.98	=	27,540
eb		6250	×	2.43	=	– 15,190
eb'	1250		×	4.45	=	5,560
E	40,530		×	2.23	=	90,380
	ΣH = 51,020 kg	ΣV = 52,470 kg	×	1.32 m		ΣM = 69,470 kgm

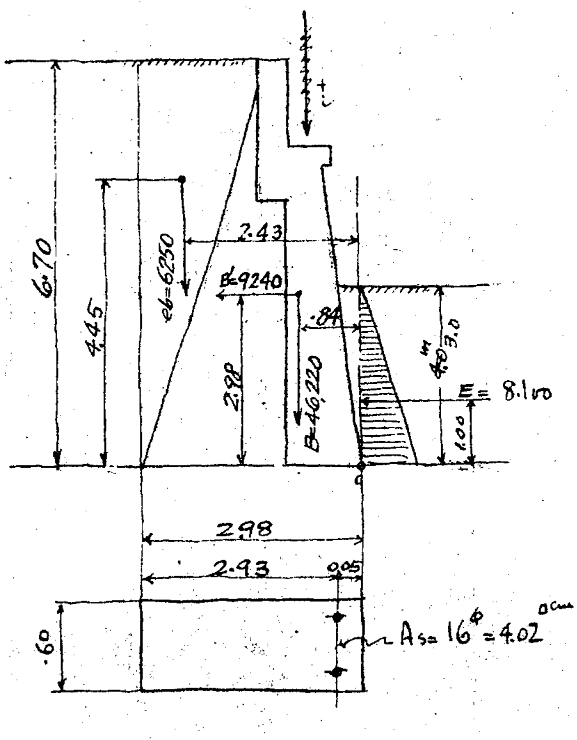
Eccentricity ε = 1.06 + 1.32 = 2.38 meter

moment on buttress = 52,470 × 2.38 = 124,880 kgm

assumed section is ample.

Case 3. Stability during earthquake K=.2 assumed.

earth pressure during earthquake = .185 × 1600 × 3.0² × 3.05 = 8,100 kg



loads	Hor. forces	Vert. forces		lever arm		moments
B		46,220	×	.84	=	38,820
B'	9240		×	2.98	=	27,540
eb		6,250	×	2.43	=	15,190
E	8,100		×	1.00	=	8,100
	ΣH = 17,340 kg	ΣV = 52,470	×	1.71 m	=	89,650

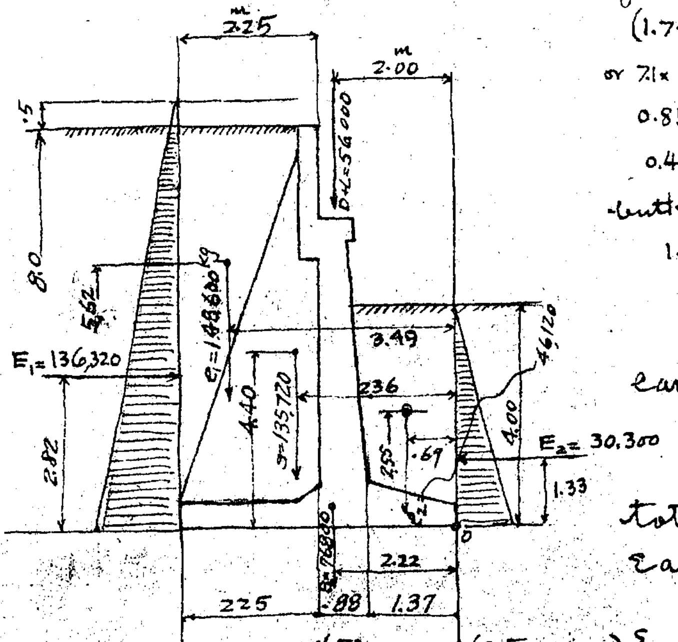
Eccentricity $= -\frac{2.98}{2} + 1.71 = 0.22^{m}$

moment on buttress = 52,470 × 0.22 = 11,540 kgm

steel area req'd for moment $= \frac{11,540 \times 100}{1200 \times 1.6 \times 7/8 \times 293} = 2.34$ □cm

Use 2-16 m/mφ bars = 2.01 × 2 = 4.020 cm

Case 1. Stability of abutment at normal state



weight of the base.

					M_H		M_V
(1.79 × 0.45 × 7.10 =	5.72 cubm	@2400 =	~~13,730~~ kg)				
or 7.1 × 1.37 × 0.45 × ½ =	~~0.308~~ 2.19	@ 〃 =	~~740~~ 5260	0.9	4,730	1.00	5,260
0.85 × 4.5 × 7.1 =	27.16	@ 〃 =	65,180	2.25	146,660	0.425	27,700
0.45 × 0.225 × 7.1 =	0.72	@ 〃 =	1,730	2.40	4,150	1.00	1,730
buttress (bottom part)							
1.95 × 0.45 × 2.2 =	1.93	@ 〃 =	4,630	3.20	14,820	1.07	4,950
	~~3.772~~ m³ 32.00		76,800 kg	× 2.22	170,360	0.516 m	39,640

earth pressure rear side top ⅓ × 1600 × 0.5 = 267

bottom ⅓ × 1600 × 8.5 = 4533

4800 ÷ 2 = 2400 kg/m²

total earth pressure E₁ = 2400 × 8 × 7.1 = 136,320 kg

Earth pressure front side $E_2 = \frac{1}{6} \times 1600 \times 4.0^2 \times 7.1 = 30,300$ kg

(Between Buttresses) Earth fill, rear side $e_1 = (1.95 \times 7.15 \times 4.9) + (4.1 \times 0.3 \times 4.9)$ @1600 = 124,910 kg

〃 〃 front side $e_2 = 2.9 \times 1.4 \times 7.1$ @1600 = 46,120 〃

Center of gravity of whole earth fill on rear side.

		Hor. arm	MH	Vert. arm	MV
Earth on Buttress A	2 × 8720 = 17,440 kg	3.81	66,450	5.70	99,410
〃 〃 〃 B	6,250	3.80	23,750	5.75	35,940
〃 〃 between Buttresses	124,910	3.43	428,440	5.60	699,500
	Σ 148,600 kg	3.49	518,640	5.62	834,850

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Taking moment about o pt

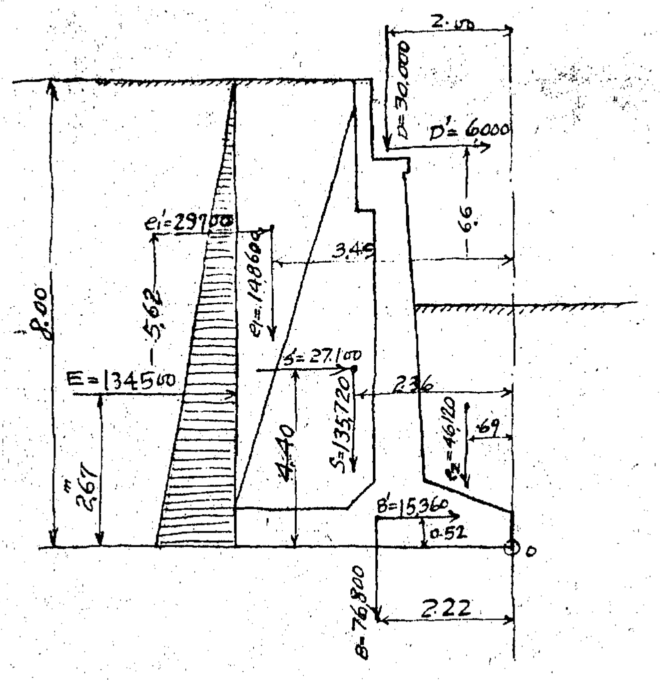
Loads	Hor. forces	Vert. forces		lever arm		moments
P		56,000	×	2.00	=	112,000
S		135,720	×	2.36	=	320,300
B		76,800	×	2.22	=	170,500
e'		148,600	×	3.49	=	518,610
e_2		46,120	×	0.69	=	31,820
E_1	− 136,320		×	2.82	=	− 384,420
E_2	30,300		×	1.33	=	40,300
	ΣH= -106,020 Kg	ΣV= 463,240 Kg	×	1.75^m		ΣM= 809,110 Kgm

Eccentricity $e = 2.25 - 1.75 = 0.50$ meter

Resultant force within middle third

max toe pressure $= \frac{463240}{7.1\times 4.5}\left(1 \pm \frac{6\times 0.50}{4.5}\right) = 24{,}200\ \mathrm{kg/m^2}$ C (2.2 ton/□′) O.K
or 4800 C.

Case 2. Stability of abutment during earthquake K=0.2 assumed.



earth pressure during earthquake $0.185\times 1600\times 8.0^2\times 7.1 = 134.500^{kg}$

taking moment about point o

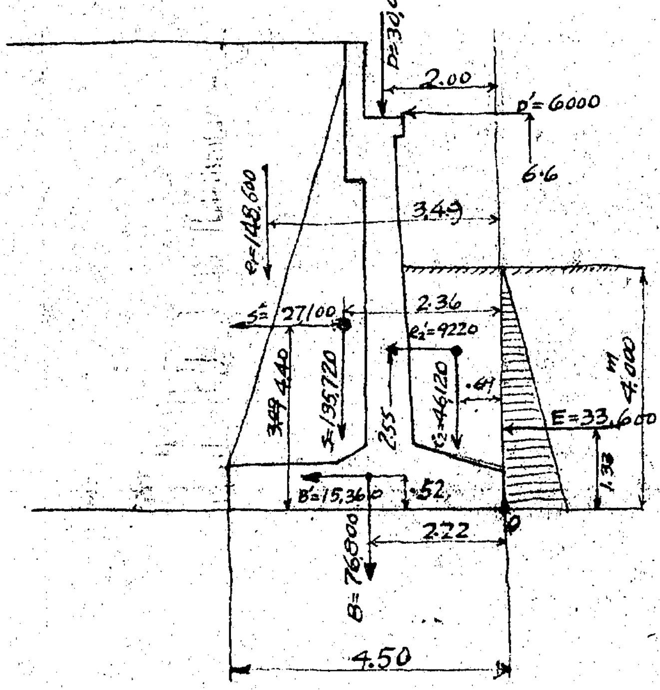
Loads	Hor. forces	Vertical forces		lever arm		moments
D		30,000	×	2.00	=	60,000
D′	6,000		×	6.60	=	− 39,600
S		135,720	×	2.36	=	320,300
S′	27,100		×	4.40	=	− 119,240
B		76,800	×	2.22	=	170,500
B′	15,360		×	0.52	=	− 8,000
e_1		148,600	×	3.49	=	518,610
e_1'	29,700		×	5.62	=	− 166,900
e_2		46,120	×	0.69	=	31,820
E	134,500		×	2.67	=	− 359,100
	ΣH = 212,660	ΣV= 437,240	×	0.93		ΣM= 408,390 Kgm

Eccentricity $= 2.25 - .93 = 1.32^m$ right.

Resultant force out of middle third pressure area $= 0.93\times 3\times 7.1 = 19.8\ \square^m$

max toe pressure $= \frac{437240\times 2}{19.8} = 44{,}200\ \mathrm{kg/m^2}$ or (4.04 tons/□′) o.k

Case 3. Stability of abutment during earthquake (seismic force's reversal to case 2.)



taking moment about point o

D		30,000	×	2.00	=	60,000
D′	6,000		×	6.60	=	39,600
S		135,720	×	2.36	=	320,300
S′	27,100		×	4.40	=	119,240
B		76,800	×	2.22	=	170,500
B′	15,360		×	0.52	=	8,000
e_1		148,600	×	3.49	=	518,610
e_2		46,120	×	0.69	=	31,820
e_2'	9,220		×	2.55	=	23,510
E	33,600		×	1.33	=	44,700
	ΣH= 91,280 kg	ΣV= 437,240 kg	×	3.05^m		ΣM= 1336,280 kgm

Eccentricity $= 3.05 - 2.25 = 0.800$ left

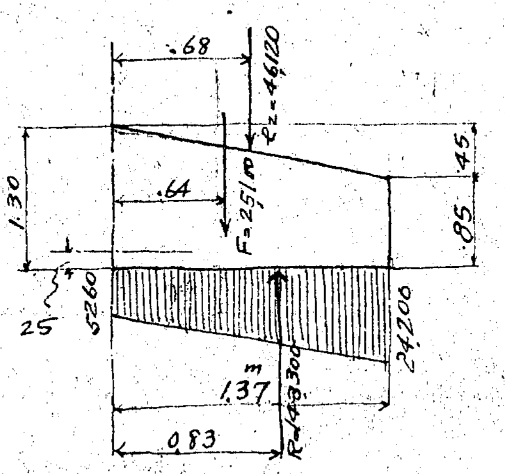
Resultant force out of middle third a little

pressure area $= 1.45\times 3\times 7.1 = 30.890^m$

max toe pressure $= \frac{437.240\times 2}{30.89} = 28{,}300\ \mathrm{Kg/cm^{m^2}}$ (or 2.58 Tons/□′)

Design of Cantilever footing

Cantilever footing at toe



Upward pressure (Case 2) 44,200 kg/m^2 at toe

" " (Case 1) 24,200 " } at toe

5,260 } at fixed point

29,460 ÷ 2 = 14,730 kg

14,730 × 1.37 × 7.1 = 143,300 kg = R

downward pressure

weight of footing 1.37 × 1.075 × 7.1 @ 2400 = 25,100 kg = F

weight of earth fill 46,120 " = e_2

moment

143,300 × .83 = + 118,940 kgm

− 25,100 × .64 = − 16,060 "

− 46,120 × .68 = − 31,360

72,080 kg ÷ 7.1 = 10,150 kg/m 71,520 kgm ÷ 7.1 = 10,073 kgm/m strip.

steel area required = $\frac{10{,}073\ 00}{1200 \times 7/8 \times 105}$ = 9.15 cm^2 per meter strip

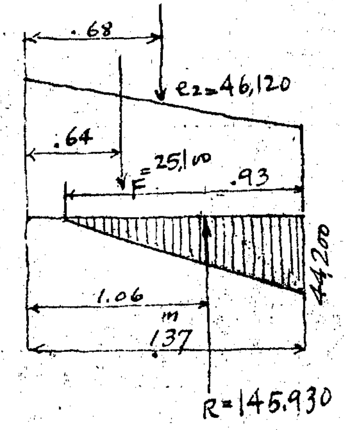
use 19φ bars @ 2.84 spacing = $\frac{2.84 \times 100}{9.15}$ = 31.0 cm

use 19φ bars 20 c to c = 14.2 cm^2

unit shear = $\frac{10{,}150}{100 \times 7/8 \times 105}$ = 1.1 kg/cm^2 O.K

unit bond = $\frac{10{,}150}{5 \times 5.97 \times 7/8 \times 105}$ = 3.7 kg/cm^2 O.K

Case 2.



upward pressure = $\frac{.93 \times 44{,}200}{2}$ × 7.1 = 145,930 kg

moment

145,930 × 1.06 = + 154,690

− 25,100 × .64 = − 16,060

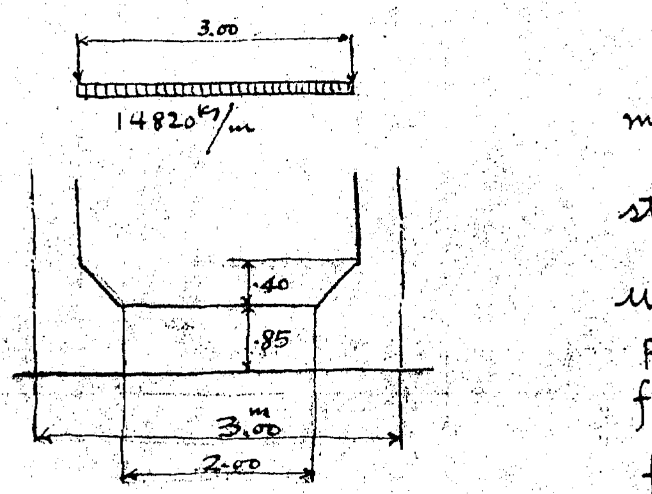
− 46,120 × .68 = − 31,360

74,710 kg ÷ 7.1 = 1052 kg/m 107,270 kgm ÷ 7.1 = 15,110 kg/m per meter

unit shear = $\frac{\cancel{15{,}110}\ 1052}{100 \times 7/8 \times 105}$ = 1.14 kg/cm^2 O.K

unit bond = O.K. $f_s = \frac{15110 \times 100}{14.2 \times 7/8 \times 105}$ = 1160 kg/cm^2 O.K

Footing at heel.



span length = 3.00 meter say

upward pressure Case 3. during earthquake 28,300 kg/m^2

downward pressure wt of earth fill 7.15 × 1600 = −11,440

wt of footing 0.85 × 2400 = −2,040

14,820 kg/m^2 upward.

moment = $\frac{14{,}820 \times 3.0^2}{10}$ = 13,340 kgm/meter strip.

steel area required = $\frac{13{,}340}{1200 \times 1.6 \times 7/8 \times 60}$ = 13.2 □cm

use 19φ bars at 20 cm c to c = 14.2 cm^2 /meter strip.

p = 14.2 ÷ (100 × 60) = 0.0024 k = 0.235 j = 0.922.

$f_s = \frac{13340 \times 100}{14.2 \times .922 \times 60}$ = 1710 kg/cm^2 < 1920 kg/cm^2 O.K

$f_c = \frac{1920 \times 0.235}{15 \times (1 - 0.235)}$ = 39.3 kg/cm^2 O.K

shear at end of chamber = 14820 × 1.00 = 14,820

unit shear = $\frac{14{,}820}{100 \times .922 \times 60}$ = 2.7 kg/cm^2

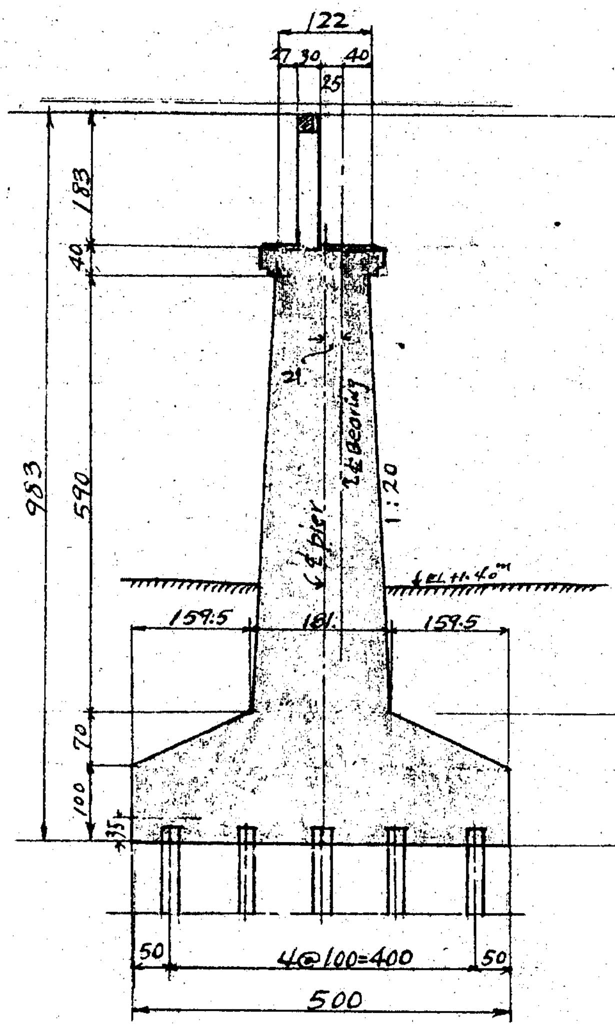
unit bond = $\frac{14{,}820}{5.97 \times 5 \times .922 \times 60}$ = 9.0 kg O.K.

end shear = 14820 × 1.5 = 22,230 kg

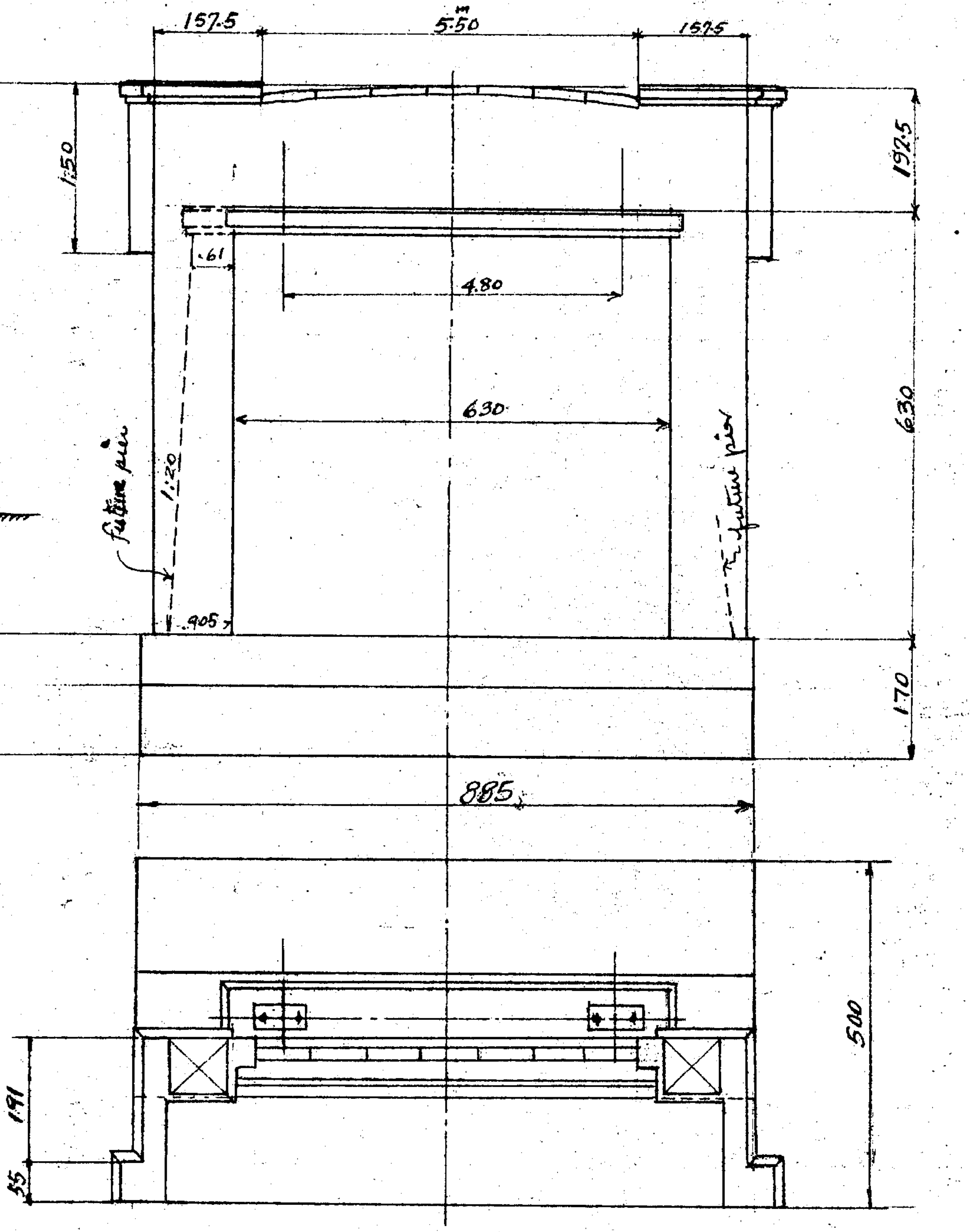
unit shear = $\frac{22{,}230}{100 \times .922 \times 100}$ = 2.4 kg/cm^2 ok.

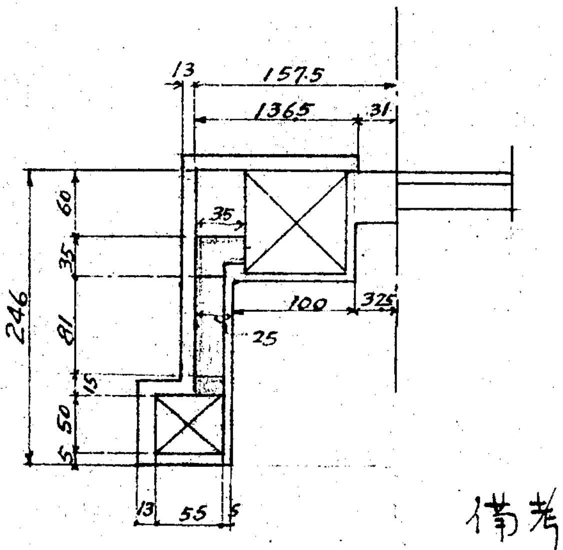
unit bond = $\frac{22{,}230}{5.97 \times 5 \times .922 \times 100}$ = 8.1 kg/cm^2 < 9.6 kg/cm^2 O.K.

West abutment



Section at center





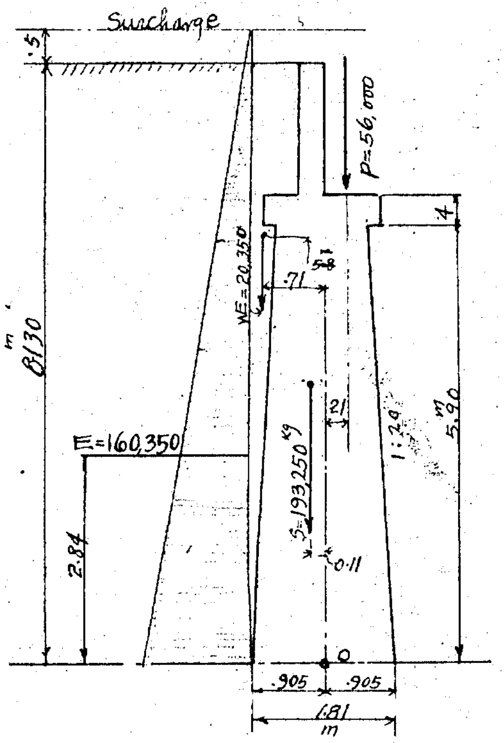
sketch for abutment
scale 1:100

備考. 本橋臺ハ将来ニ於ケル河川改修ノ結果橋梁ノ延長ヲ要スルヿアルヲ豫想シ此ノ場合必要ニ應シ橋脚ニ改造シ得ベキ様設計セルモノナリ。

Volume of Concrete and center of gravity of abutment

	section (m)		vol (m³)		wt (kg)	Vert arm (m)	Mv	H arm	MH
parapet wall	0·30 × 1.6	× 6.15	= 2.95	@2400	= 7080	9.030	63,930	0.40	2,830
踏掛石	0.20 × 0.2	× 5.50	= 0.22	@2600	= 580	9.70	5,630	0.45	260
Coping	1.48 × 0.4	× 6.55	= 3.88	@2400	= 9.310	7.80	72,620	0.21	1,960
post under pedestals	2@1.25 × 0.9	× 8.22	= 18.50	@ 〃	= 44.400	5.81	258,000	0.70	31,100
less	−0.57 × 0.30	× 6.30	= 1.08	@ 〃	= −2,590	4.81	−12,560	0.54	− 1,400
less	−0.63 × 0.148	× 0.30	= 0.28	@ 〃	= − 670	3.80	− 2,550	0.92	− 620
shaft	1.515 × 5.9	× 6.30	= 56.31	@ 〃	= 135,140	4.46	602,720	0.21	28,380
					193,250 kg	5.11 m	987,790 kgm	0.32 m	62,510 kgm

Base concrete	1.00 × 5.0	× 8.85	= 44.25	@ 2400	= 106,200	× ·50	=	53,100
	3.405 × 0.7	× 〃	= 21.09	〃	= 50,620	× 1.30	=	65,810
			65.34 m³		156,820 kg	× 0.76 m	=	118,910



Shaft. Case 1.

stability of shaft at normal state.

L.L+D.L ≒ 2@28000 = 56,000 kg for the whole abutment = P

weight of shaft = 193,250 kg = S

Earth fill on rear = 0.57 × 1.83 × 6.2 = 6.47 m³ @1600 = 10,950 kg

3.15 × 0.30 × 6.2 = 5.86 @ 〃 = 9,400

20,350 kg … W.

Earth pressure = $\frac{1600}{3} \times \cdot 5 = 267$ kg

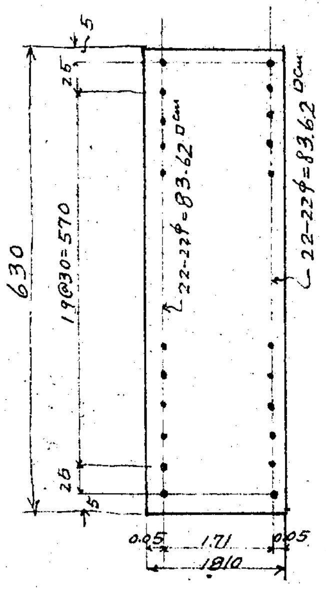
$\frac{1600}{3} \times 8.63 = 4603$

E = 4870 ÷ 2 = 2435 … × 8.1 × 8.13 = 160,350 kg

備考　橋台前面土砂ハ將来行ハル可キ改修工事ノ結果低水位附近迄切取ラルベキヲ豫想シ其土圧ヲ無視セリ。

Taking moment about point O (center of base)

	Loads		Hor. lever arm		moment
P	56,000 kg	×	+ 0.21	=	11,760
S	193,250	×	− 0.11	=	−21,260
W	20,350	×	− 0.71	=	− 14,450
E	160,350	×	+ 2.84	=	455,400
	ΣV = 269,600 kg		1.60 m ≒ eccentricity		ΣM = +431,450 kgm
	ΣH = 160,350 kg				



Try reinforcement 22-22 m/m φ bars on both sides = 83.62 × 2 = 167.24 cm²

steel ratio $p_0 = 2p = 167.24 \div 630 \times 181 = 0.0015$

$\varepsilon/h = 1.60/181 = 0.884$　　$d'/h = 5/181 = 0.028$

$K = 0.22$　　$L = 0.071$　　$j = 0.93$

$$f_c = \overset{\text{omitt}}{\boxed{\frac{301,400}{0.078 \times 630 \times 181^2} = 18.7\ \text{kg/cm}^2\ \text{O.K}}}$$

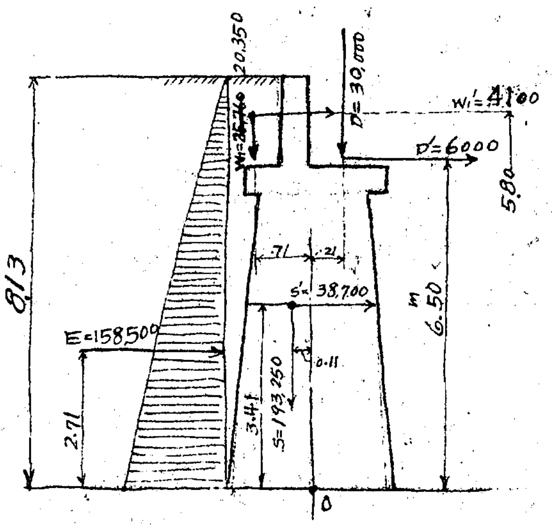
$$f_c = \frac{431,450}{0.071 \times 630 \times 181^2} = 29.7\ \text{kg/cm}^2\ \text{O.K}$$

$$f_s = 15 \times 29.7 \left(\frac{176}{0.22 \times 181} - 1\right) = 1520\ \text{kg/cm}^2 \quad > 1200\ \text{kg/cm}^2$$

$$\text{unit bond} = \frac{160,350}{6.91 \times 22 \times 0.9 \times 176} = 6.6\ \text{kg/cm}^2\ \text{O.K}$$

Case 2. Stability of shaft during Earthquake K=0.2 assumed.

earth pressure during Earthquakes = $\frac{0.37}{2} \times 1600 \times 8.13^2 \times 8.1 = 158,500^{kg}$



Taking moment about o pt

load	Hor. forces	Vert. forces	lever arm	moments
D		30,000	× + 0.21	6,300
D'	6000		× + 6.50	39,000
S		193,250	× − 0.11	− 21,260
S'	38,700		× + 3.41	132,000
W1		20,350	× − 0.71	− 14,450
W1'	4100		× + 5.80	23,780
E	158,500		× + 2.71	429,540
	ΣH= $\boxed{261,300}$ Kgs 207,300	ΣV=243,600 kg	× 2.44 m	ΣM= + $\boxed{614,540}$ Kgm 594,910

e/h = 2.44/181 = 1.35 d'/h = 0.028 po = 2p = 0.0015

K=0.183 L = 0.0692

$f_c = \frac{594,910}{0.0692 \times 630 \times 181^2} = 41^{kg}/cm^2 < 35 \times 1.6 = 56^{kg}/cm^2$ O.K

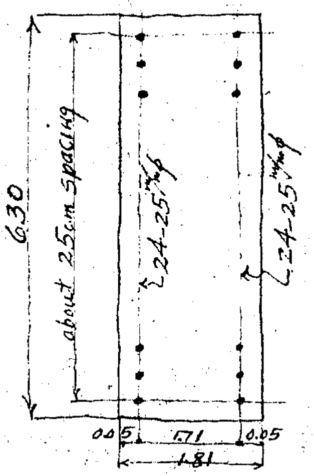
$f_s = 15 \times 41 \left(\frac{176}{0.183 \times 181} - 1\right) = 2780^{kg}/cm^2 > 1.6 \times 1200 = 1920^{kg}/cm^2$

try again ~~25 m/mφ~~ ~~22~~ 25 m/mφ 21- 21×4.909 = 103.09 cm² As. on both sides

$p_o = 2p = \frac{2 \times 103.09}{630 \times 181} = 0.0018$ e/h = 1.34 d'/h = 0.028

K = 0.192 L = 0.0732

$f_c = \frac{594910}{0.0732 \times 630 \times 181^2} = 39.4^{kg}/cm^2$ $f_s = 15 \times 39.4 \times \left(\frac{176}{0.192 \times 181} - 1\right) = 2400^{kg}/cm^2$



then 25 m/mφ 24 - 4.91 cm² = 117.84 cm² As. p = 0.00103.

e/h = 1.35 d'/h = 0.028 K = 0.20 L = 0.078

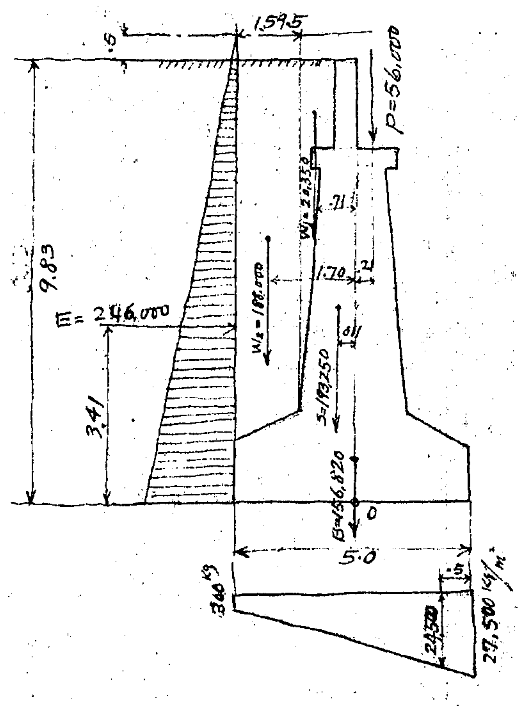
$f_c = \frac{594,910}{0.078 \times 630 \times 181^2} = 37^{kg}/cm^2$ $f_s = 15 \times 37 \left(\frac{176}{0.20 \times 181} - 1\right) = 2140^{kg}/cm^2$

許容応力ヨリ11% overスルモ実際橋台ノ両翼壁ガ後方ニ延長セルヲ以テ shaft 全体ノ重心ハ計算上ヨリモ遙ニ後方ニ位スルモノト推定サルルガ故ニ安全ナリトス。

unit Bond = $\frac{207,300}{7.85 \times 24 \times 0.94 \times 176} = 6.7^{kg}/cm^2 < 1.6 \times 6$ O.K.

stability of abutment as a whole

Case 1. stability at normal state



earth pressure = $\frac{1600 \times 0.5}{3} = 267$

$\frac{1600 \times \cancel{9.83}\ 10.33}{3} = \cancel{5243}\ 5510$

~~5510~~ 5777 ÷ 2 = ~~2755~~ 2889 × 9.83 × 8.65 = 246,000 kg

earth fill on rear 1.595 × 8.65 × 8.5 × 1600 = 188,000 kg W2

load	Hor. forces	Vert. forces	lever arm		moment
P		56,000	× +.21	=	11,760
S		193,250	× −0.11	=	− 21,260
W1		20,350	× − 0.71	=	−14,450
B		156,820	× 0	=	0
W2		188,000	× − 1.70	=	− 319,600
E	246,000		× 3.41	=	839,000
	ΣH= 246,000 kg	ΣV= 614420 kg	× 0.81 m		ΣM= + $\boxed{500,450}$ Kgm 495,450

Resultant force within middle third

max toe pressure = $\frac{614420}{5 \times 8.85}\left(1 \pm \frac{6 \times 0.81}{5.0}\right) = 27,500^{kg}/m^2$ or (2.5 Tons/□') or .300' or (0.03 T/□")

load on pile = 24500 × 1.0 × 1.0 = 24500 kg on one pile

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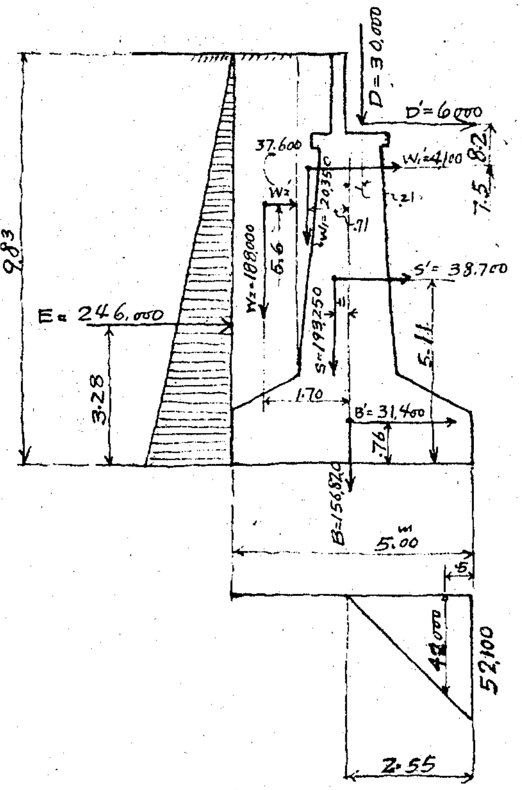
CALCULATIONS FOR

Case 2. Stability during Earthquake

Earth pressure = $\frac{0.37 \times 1600 \times 9.83^2}{2} \times 8.65 = 246.000^{kg}$



loads	Hor. forces	Vert. forces		lever arm		moment
D		30,000	×	+0.21	=	6,300
D'	6000		×	+8.20	=	49,200
S		193,250	×	-0.11	=	-21,260
S'	38.700		×	+5.11	=	197,800
B		156,820	×	0	=	0
B'	31,400		×	+0.76		23,860
W1		20,350	×	-0.71	=	-14,450
W1'	4100		×	7.5	=	30,750
W2		188,000	×	-1.70	=	-319,600
W2'	37,600		×	5.6	=	210,600
E	246,000		×	3.28	=	806,900
	ΣH= 363,800 kg	ΣV= 588,400 kg	×	1.65 m		ΣM= 970,100 kgm

Resultant force out of middle third. (a of pressure area= 3(2.5-1.65)= 2.55 m

pressure area= 3ab = 2.55× 8.85= 22.57 □m

max toe pressure = $\frac{588,400 \times 2}{22.57} = 52,100^{kg}/m^2$ or $(4.8^{ton}/□')$

load on pile = 42,000×1.0×1.0= 42,000 kg or 42 kg tons per on one pile

Case 3. Stability during earthquake seismic force backward.

Earth press front side

$\frac{1600 \times 3^2}{6} \times 8.65 = 20,760^{kg}$

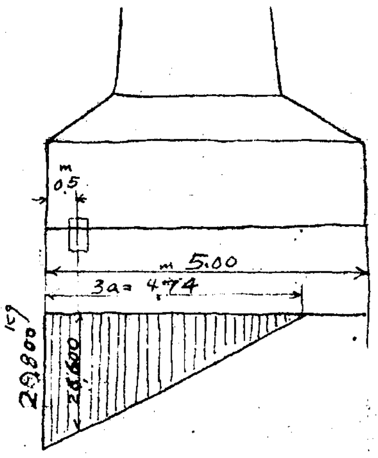
earth on base = 1.60×1.7×8.65= 23.55 m³
×1600= 37,700 kg

	Hor. forces	Vert. forces		lever arm		moment
	76,100 kg	588,400 kg	×	1.053	=	619,870
earth press. front 20,760			×	1.00	=	20,760
earth on base		37,700	×	-1.7	=	-64,100
	ΣH=96,860 kg	ΣV=626,100	×	0.92 m		ΣM= 576,530 kgm

press. area = 3× 1.58 × 8.85 = 41.95 m²

max toe pressure = $\frac{626,100 \times 2}{41.95} = 29,800\ kg/m^2\ (2.7^{ton}/□')$

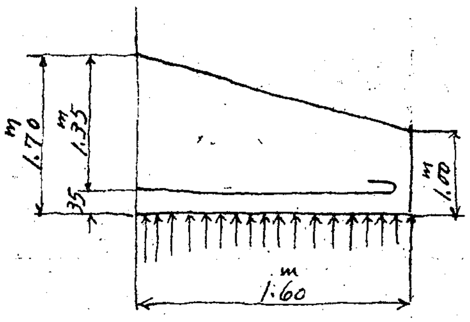
load on one pile = 26,600×1.0×1.0 = 26.6 kg tons



Case 1. at Normal state

upward pressure		=	26,600 kg/m²
downward "	1.35 m³ @2400	=	- 3,240 "
earth say	1.00@ 1600		- 1600
	upward		21,760 kg/m²

Cantilever footing at toe



moment = $\frac{21,760 \times 1.6^2}{2}$ = 27,900 kgm per meter strip

shear = 21,760×1.6 = 34,800 kg

steel area required = $\frac{28000 \times 100}{1200 \times 7/8 \times 1.35}$ = 20.55 cm²

use 25 m/m φ bars at 20cm spacing As= 24.55 cm²

unit shear = $\frac{34.800}{100 \times 0.931 \times 135}$ = 2.8 kg/cm² o.K

unit bond = $\frac{34.800}{7.85 \times 5 \times 0.931 \times 135}$ = 7.0 kg/cm² > 6 kg/cm²

$p = \frac{24.55}{100 \times 135} = 0.0018$

$j = 0.931$.

Try effective depth 1.6 meter unit bond = $\frac{34.800}{7.85 \times 5 \times 0.93 \times 160}$ = 6.0 kg/cm² o.K

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Design of west abutment

Case 2. Considering earthquake

upward pressure $= 52.100^{kg/m^2}$
downward " $= -4.840$
upward ——— 47.360^{kg}/meter strip

moment at face of footing $= \frac{47.360 \times 1.6^2}{2} = 60.620^{kgm}$/meter

shear $= 47.360 \times 1.6 = 75.800^{kg}$

steel area reqd $= \frac{60.620 \times 100}{1200 \times 1.6 \times 0.93 \times 160} = 21.2^{cm^2} < 24.55^{cm^2}$ O.K

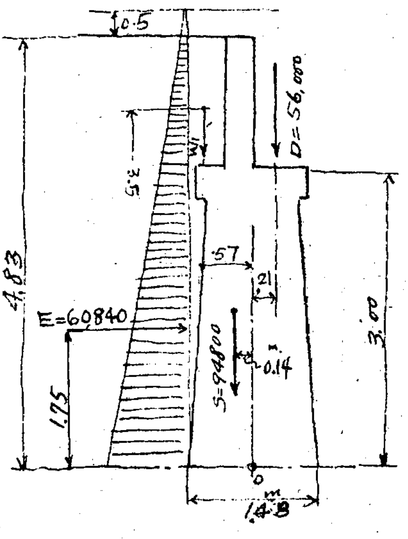
unit bond $= \frac{75.800}{39.25 \times 160 \times 0.93} = 13.0^{kg/cm^2} > 1.6 \times 6$

Some Extra bond bars will be used.

unit shear $= \frac{75.800}{100 \times 0.93 \times 160} = 5.1^{kg/cm^2} < 1.6 \times 4 = 6.4^{kg/cm^2}$
O.K

for footing at heel use same details as for above.

Stability at section 3.00 metr from top of Coping



Case 1. at normal state

Center of gravity

		m³	kg	Vert arm m	Mv	Hor. arm	MH
parapet	0.30 × 1.80 × 6.15 =	3.32 @ 2400	= 7990	3.93	31,320	0.19	1510
Coping	1.48 × 0.4 × 6.55 =	3.88 @ "	= 9,310	2.80	26,100	0	0
post under pedestal							
	2@ 1.25 × 0.9 × 4.93 =	11.10 @ "	= 26,640	2.47	65,800	.45	11,990
less.	−0.57 × 0.30 × 3.0 =	0.51 @ "	= −1.220	1.50	−1.830	0.29	−350
less	−0.068 × 2.6 × 0.3 =	0.05 @ "	= −120	0.87	−100	0.61	−70
shaft	1.35 × 2.60 × 6.2 =	21.76 @ "	= 52,220	1.26	65,800	0	0
			94800 kg	1.97 m (y)	187,090 kgm	0.138 m	13,080

e'/h =

earth fill on rear $= 0.40 \times 1.83 \times 6.2$ @1600 = 7.260 kg
$\frac{1}{2} \times 2.6 \times 0.13 \times 6.2$ @ " = 1.680
8,940 kg —— w_1

earth pressure $\frac{1600}{3} \times 0.5 = 267$
$\frac{1600}{3} \times 5.33 = 2,843$
$3.110 \div 2 = 1555 . 4.83 \times 8.1 = 60,840^{kg}$ E

Taking moment about o (center of section)

loads		arm		moment
P	56,000	+0.21	=	11,760
S	94800	−0.14	=	−13,270
W	8,940	−0.57	=	−5,100
E	60,840	+1.75	=	106,500
$\Sigma V =$	159,540 kg	0.338 m e eccentricity		$\Sigma M = +54,000$ kgm
$\Sigma H =$	60,840 "			

use 19 mφ bar on rear face & 16 mφ bar on front side as shown