Vol. 6

P.S.C.I E-RAMP P_1 Pier

(P.S.C.I E-RAMP P_3 , F-RAMP P_1, P_3)

(P -)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.497 t/m^3

4 Allowable stresses of reinforced concrete

1.) concrete (grade 25 , BS 5400)

cube strength at 28 days ck = 255 kg/cm^2

bending stress ca = 85

direct stress ca = 64.3

shear stress a = 8.2

2.) reinforcement (hot rolled high yield bars, BS 4449)

specified characteristic strength

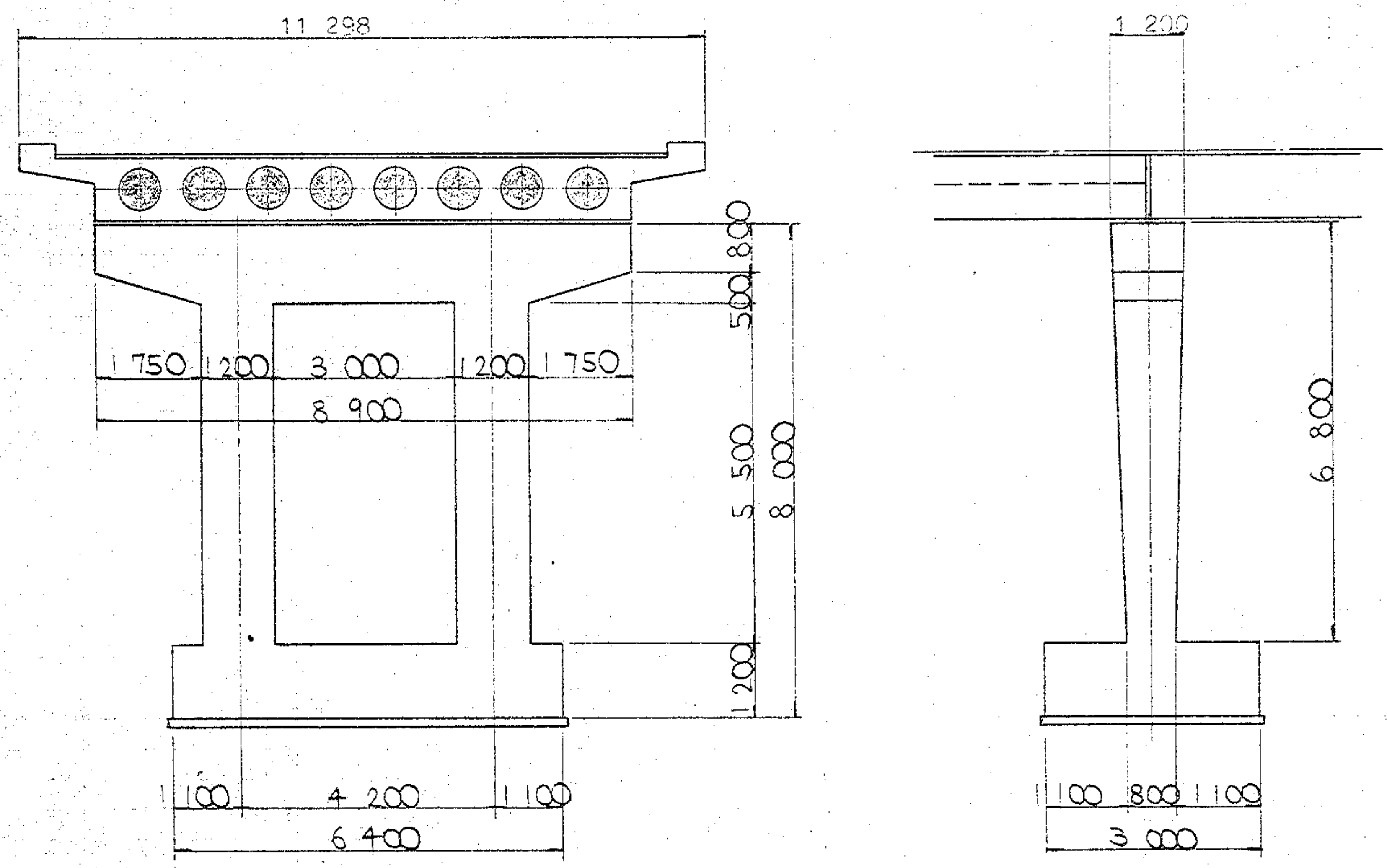
su = 4 180 kg/cm^2

permissible tensile stress

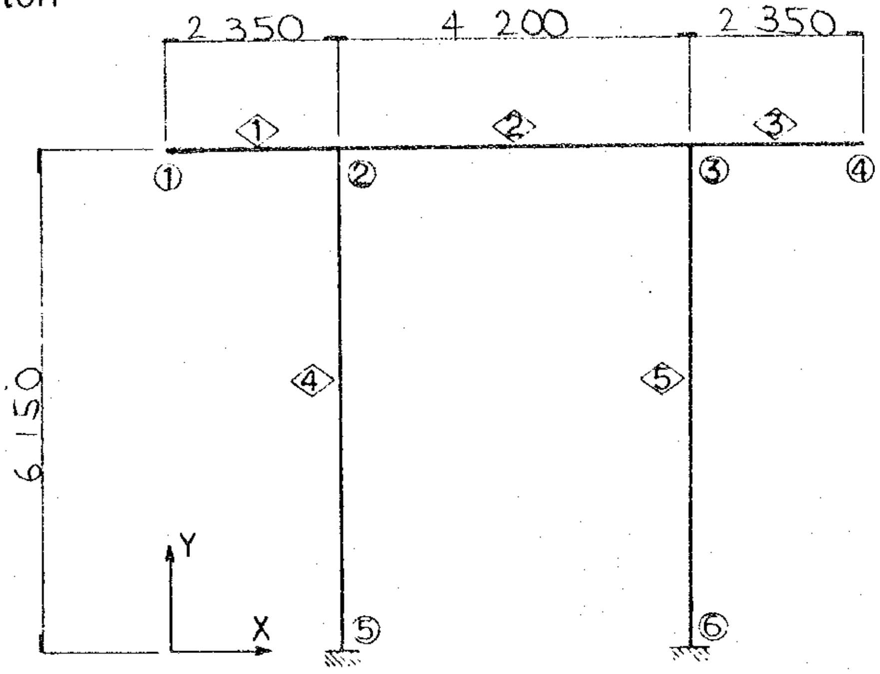
sa = 2 340 kg/cm^2

modular ratio n = 15

§ 2 GENERAL DIMENSION



Skeleton



○ : Joint number
◇ : Menber number

Coordinates

(m)

	X	Y		X	Y
①	0.0	6.15	④	8.90	6.15
②	2.35	6.15	⑤	2.35	0.0
③	6.55	6.15	⑥	6.55	0.0

Modulus of elasticity of concrete

$E_c = 26\ kN/mm^2$

$= 2.65 \times 10^6\ t/m^2$

member	section B x H	area of section A (m²)	moment of inertia I (m⁴)
⟨1⟩	1.16 x 1.05	1.22	0.1119
⟨2⟩	1.16 x 1.30	1.51	0.2124
⟨3⟩	1.16 x 1.05	1.22	0.1119
⟨4⟩	0.96 x 1.20	1.15	0.1382
⟨5⟩	0.96 x 1.20	1.15	0.1382

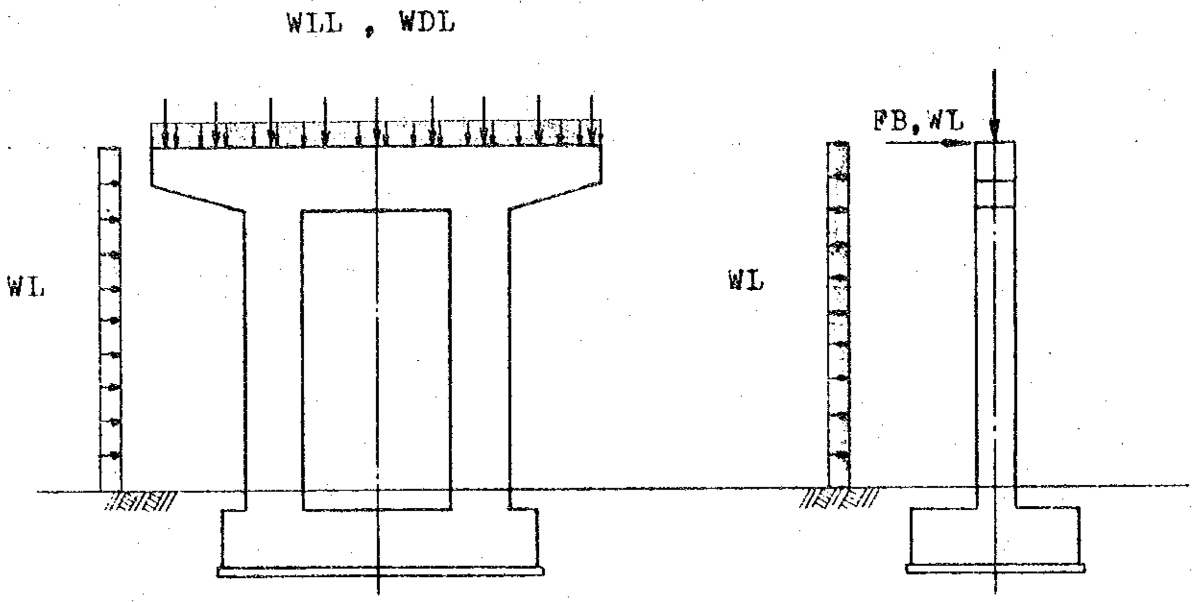
§ 3 CALCULATION OF LOADS

1.) Loading case and increase in basic stresses

No.	base load case	mark
1	dead load of deck	WDL
2	max reaction under HA	WLL(A)
3	max reaction under HB	WLL(B)
4	HA braking	FB(A)
5	HB braking	FB(B)
6	Self weight	W
7	tenperture change and drying schrinkage	FT & FS
8	wind	WL

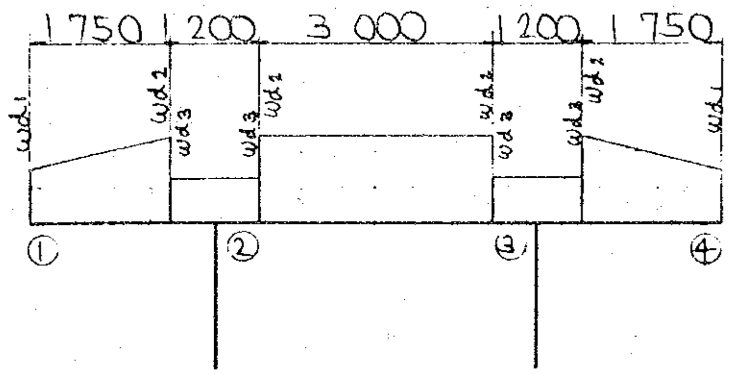
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3) Dead load

(a) beam



wd1 = 1.16 × 0.80 × 2.407 = 2.23 t/m

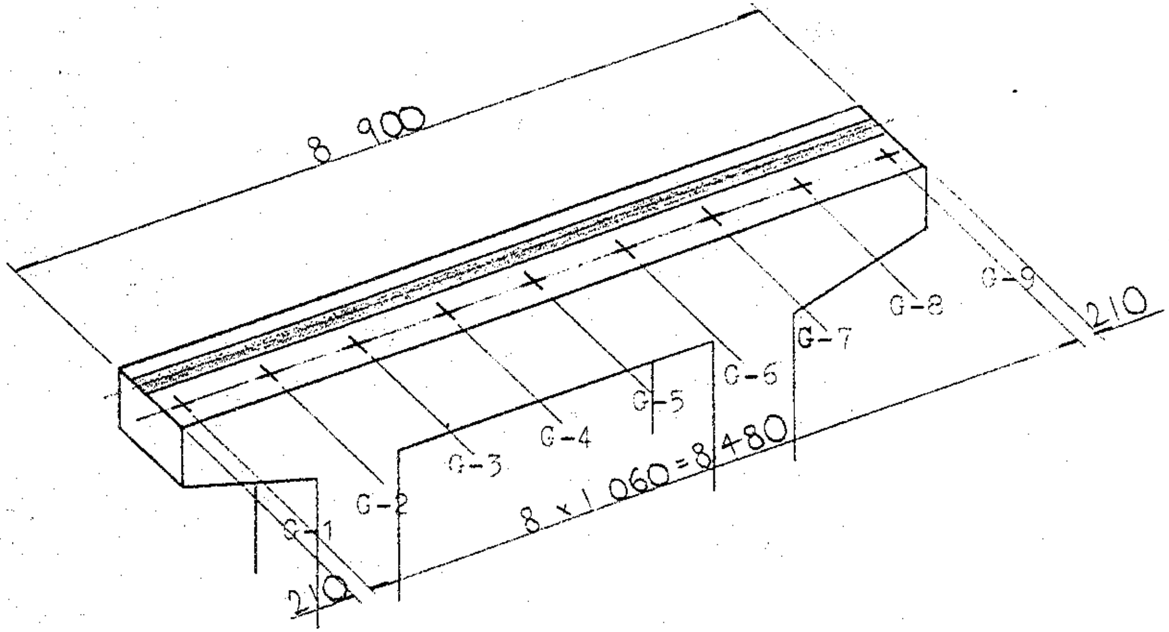
wd2 = 1.16 × 1.30 × 2.407 = 3.63 t/m

wd3 = 1.16 × 0.65 × 2.407 = 1.81 t/m

(b) column

pw = 0.96 × 1.20 × 2.407 = 2.77 t/m

4) Reaction



		G-1	G-2	G-3	G-4	G-5	G-6	G-7	G-8	G-9
Dead load		26.7	3.9	14.6	9.5	12.7	11.2	12.7	14.0	8.7
		4.24 t/m								
HA load		43.8	18.7	20.4	16.1	18.8	18.5	18.0	19.6	24.9
		2.97 t/m								
HB load		70.3	12.1	25.7	22.3	24.4	22.4	24.5	22.5	38.2
		0.43 t/m								

5.) Temperture change and Drying schrinkage

temperture change -12 +20 deg.

drying schrinkage -20 deg.

6.) Wind Load (transverse)

(a) For superstructure

Vc = 38 x 1.0 x 1.0 x 1.56 = 59.3 m/sec. > 35 m/sec.

Unloaded

Pt = q・A・Cd

q = 0.613 × 59.3^2 × 0.102 = 219.9 kg/m^2.

A = 1.23 × 10.8 = 13.3 m^2

Cd= 1.15

Pt = 219.9 ×13.3 × 1.15 = 3363 kg

= 3.4 ton

Live loaded

Pt = q・A・Cd

q = 0.613 × 35^2 × 0.102 = 76.6 kg/m^2

A = 3.58 × 10.8 = 38.7 m^2

Cd= 1.35

Pt = 76.6 × 38.7 × 1.35 = 4 002 kg

= 4.0 ton

(b) For safety fences

Vc = 38 x 1.0 x 1.0 x 1.56 = 59.3 m/sec.

Pt = q・A・Cd

q = 219.9 kg/m^2

A = $(0.150 \times 10.8 + 0.850 \times 0.05 \times \frac{10.8}{2}) \times 2$ = 3.70 m^2

Cd= 1.1

Pt = 219.9 × 3.70 × 1.1 = 895 kg

= 0.9 ton

(c) For Pier

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3$ m/sec.

Unloaded

$q = 219.9$ kg/m²

$A1 = \frac{1}{2}(1.20 + 0.83) \times 6.30 = 6.39$ m²

$A2 = \frac{1}{2}(1.124 + 0.83) \times 5.00 = 4.89$ m²

$Cd = 2.0$

$Pt = 219.9 \times (6.39 + 4.89) \times 2.0 = 4961$ kg

$= 5.0$ ton

Live loaded

$q = 76.6$ kg/m²

$A = A1 + A2 = 11.28$ m²

$Cd = 2.0$

$Pt = 76.6 \times 11.28 \times 2.0 = 1728$ kg $= 1.7$ ton

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	3.4	4.0
safety fences	0.9	——
Pier	5.0	1.7

7.) Longitudinal force

(a) due to breaking

HA ——— 25.8 ton

HB ——— 38.3 ton

(b) due to Wind

for superstructure

unloaded PLS = 0.25 Pt = 0.25 × 3.4

= 0.85 ton

live loaded PLL = 0.5 Pt

Pt = 76.6 × (2.5 × 10.8) × 1.35

= 2.8 ton

PLL = 0.5 × 2.8 = 1.4 ton

PLS = 0.25 Pt

Pt = 76.6 × (3.58 − 2.50) × 10.8 × 1.15

= 1.0 ton

PLS = 0.25 × 1.0 = 0.25 ton

for safety fences

vertical member

PL = 0.8 Pt

Pt = 219.9 × (0.85 × 0.05 × $\frac{10.8}{2}$ × 2) × 1.1

= 111 kg = 0.1 ton

PL = 0.8 × 0.1 ≒ 0.1 ton

longitudinal member

PL = 0.4 Pt

Pt = 219.9 × (0.15 × 10.8 × 2) × 1.1

= 784 kg = 0.8 ton

PL = 0.4 × 0.8 = 0.3 ton

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for Pier

Vc = 59.3 m/sec.

unloaded

q = 219.9 kg/m²

A1= 1.30×8.90−1.75×0.50 = 10.7 m²

A2= 1.20×5.00×2 = 12.0 m²

Cd1= 2.1

Cd2= 2.2

Pt1= 219.9×10.7×2.1 = 4.9 ton

Pt2= 219.9×12.0×2.2 = 5.8 ton

live loaded

q = 76.6 kg/m²

A1= 10.7 m²

A2= 12.0 m²

Cd1= 2.1

Cd2= 2.2

Pt1= 76.6×10.7×2.1 = 1.7 ton

Pt2= 76.6×12.0×2.2 = 2.0 ton

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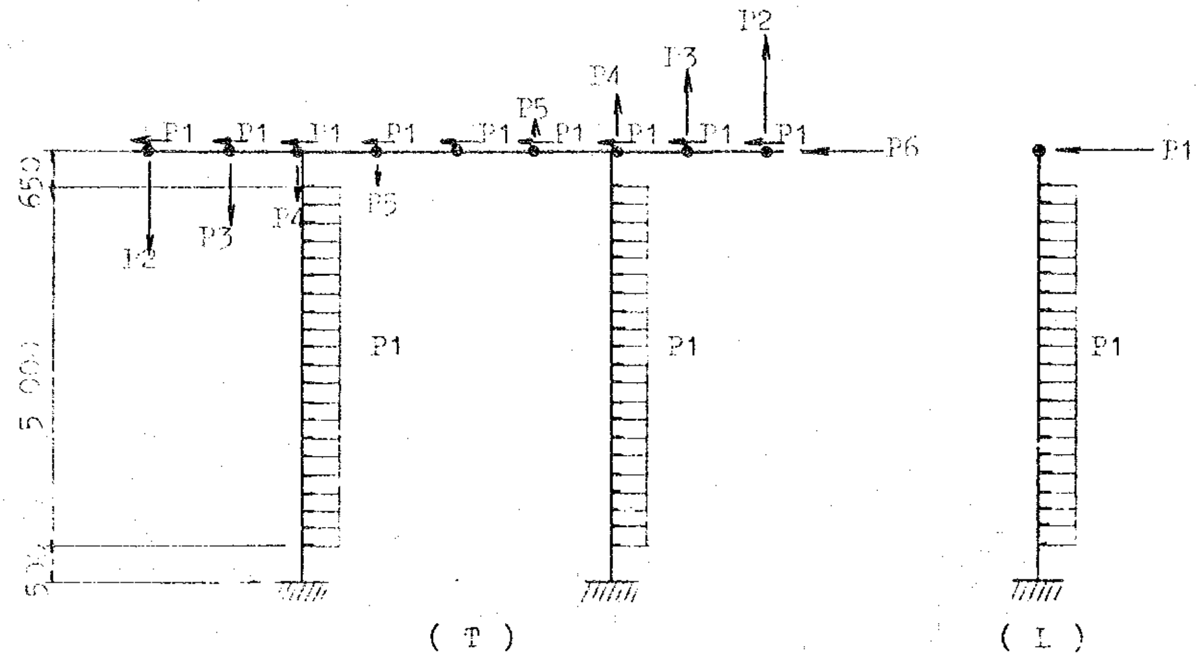
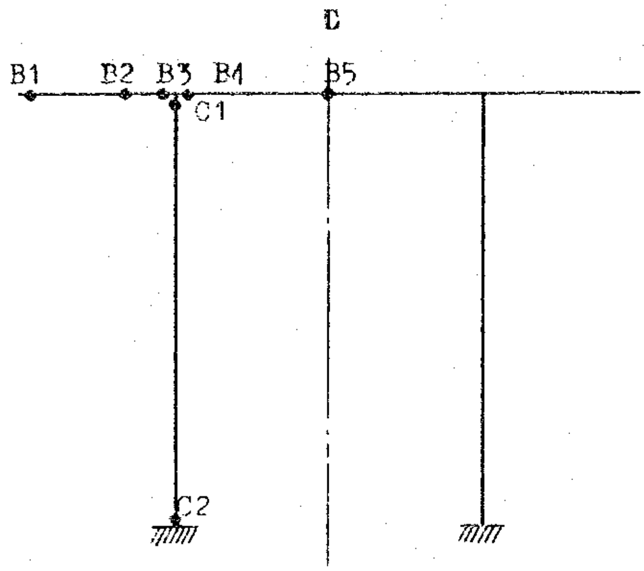
8) Wind loading diagram



		For Piace	unloaded	live loaded
(T)	P1	superstructure safety fences	0.48 ton	0.44 ton
	P2	superstructure safety fences	0.27 ton	0.61 ton
	P3	superstructure safety fences	0.20 ton	0.46 ton
	P4	superstructure safety fences	0.14 ton	0.31 ton
	P5	superstructure safety fences	0.07 ton	0.15 ton
	P6	Pier	0.70 ton	0.20 ton
	P1	Pier	0.43 t/m	0.15 t/m
(L)	P1	superstructure safety fences	3.08 ton	1.68 ton
	P1	Pier	0.58 t/m	0.20 t/m

§ 4 ACTING FORCE TABLE

1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max	+					
		-	3	-0.209	0	-72.500	1.00
	N max	+	2	-0.144	0.480	-28.346	1.00
		-					
	S max	+					
		-	4	-0.123	0	-78.173	1.25
B2	M max	+					
		-	4	-86.926	0	-97.927	1.25
	N max	+	2	-34.079	0.960	-39.932	1.00
		-					
	S max	+					
		-	3	-82.616	0	-105.734	1.00
B3	M max	+					
		-	4	-134.672	0	-101.040	1.25
	N max	+	2	-54.124	0.960	-43.617	1.00
		-					
	S max	+					
		-	3	-134.588	0	-110.845	1.00

			case	M (tm)	N (ton)	S (ton)	
E4	M max.	+					
		-	3	-175.036	2.372	95.582	1.00
	N max.	+	3	-107.501	26.193	45.854	1.00
		-	5	-83.949	-4.671	64.152	1.15
	S max.	+	3	-134.985	18.290	95.582	1.00
		-					
B5	M max.	+	3	49.204	11.306	22.632	1.00
		-	3	-76.716	10.275	8.219	1.00
	N max.	+	3	-36.665	26.193	8.219	1.00
		-	3	9.153	-4.612	-8.879	1.00
	S max.	+	6	1.384	16.185	52.987	1.30
		-					
C1	M max.	+	5	12.392	-118.850	-5.801	1.15
		-	3	-96.317	-196.966	26.173	1.00
	N max.	+					
		-	3	-68.669	-246.675	18.314	1.00
	S max.	+	3	-95.917	-197.111	26.193	1.00
		-					
C2	M max.	+	3	18.084	-146.719	4.592	1.00
		-	3	-73.363	-138.653	-26.173	1.00
	N max.	+					
		-	3	-44.835	-215.595	-16.063	1.00
	S max.	+					
		-	3	-72.921	-156.508	-26.193	1.00

These value is divided by "i".

i ; coefficient of increase

in basic stresses

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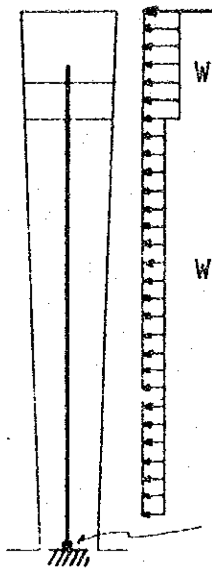
2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7				1.00
case-8	65.3	239.2	9.6	1.25
case-9	58.1	218.7	9.2	1.15
case-10	79.2	239.2	12.3	1.30
case-11				1.00

CASE 8

	N	H	y	M
WDL	75.9			
WLL (B)	133.1			
FB (B)		9.6	6.80	65.3
W	30.2			
TOTAL	239.2	9.6		65.3

M = 65.3 t·m

N = 239.2 ton

H = 9.6 ton

CASE 9

	N	H	y	M
WDL	75.9			
WLL . FB (A)	112.6	6.5	6.80	44.2
W	30.2			
WL (super)		0.8	6.80	5.4
WL (1)		0.9	6.15	5.5
WL (2)		1.0	3.00	3.0
TOTAL	218.7	9.2		58.1

M = 58.1 t·m

N = 218.7 ton

H = 9.2 ton

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CASE 10

	N	H	y	M
WDL	75.9			
WLL FB (B)	133.1	9.6	6.80	65.3
W	30.2			
WL (Super)		0.8	6.80	5.4
WL (1)		0.9	6.15	5.5
WL (2)		1.0	3.00	3.0
TOTAL	239.2	12.3		79.2

M = 79.2 t·m

N = 239.2 ton

H = 12.3 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

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	at B2	at B4	at B5
	case 4	case 3	case 3
M (tm)	86.926	175.036	49.204
N (ton)	0	2.372	11.306
Q;shear force (ton)	97.927	95.582	22.632
b (cm)	112	112	112
d (cm)	106	120	120
d' (cm)	10	10	10
As (cm^2)	80.4 (D32 10.2 cm Pitch)	80.4 (D32 10.2 cm Pitch)	31.4 (D20 10.2 cm Pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.10	0.09	0.035
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	6.91	10.85	3.05
Q/b·d (kg/cm²)	8.25	7.11	1.68
C	6.34	6.57	9.35
S	11.4	12.6	31.0
Z	1.14	1.13	1.08
σ_c (kg/cm²)	44	71	29
σ_s (kg/cm²)	1182	2051	1418
τ (kg/cm²)	9.4 ※	8.0	1.8
σ_{ca} (kg/cm²)	85	85	85
σ_{sa} (kg/cm²)	2340	2340	2340
τ_a (kg/cm²)	8.2	8.2	8.2

See page 169'

	at C_1	at C_2	
	case 3	case 3	case 10
M (tm)	96.317	73.363	79.2
N (ton)	196.966	138.653	239.2
Q;shear force (ton)	26.173	26.173	12.3
b (cm)	112	80	120
d (cm)	110	110	70
d' (cm)	10	10	10
As (cm^2)	44.2 (D25 11.5 cm Pitch)	24.6 D25 15 cm Pitch	44.2 (D25 12.5 cm Pitch)
d'/d	0.091	0.091	0.14
f=(M/N)+u (cm)	98.9	102.9	63.1
f/d	0.90	0.94	0.90
n	15	15	15
nP=n·As/b·d	0.054	0.042	0.079
M'=M+N·u (tm)	194.8	142.7	151.0
M'/(b·d^2) (kg/cm^2)	14.37	14.7	25.68
Q/b·d (kg/cm^2)	2.12	2.97	1.46
C	3.51	4.00	3.30
S	2.3	3.5	1.80
Z	0.8	0.8	0.84
σc (kg/cm^2)	50	59	85
σs (kg/cm^2)	496	772	693
τ (kg/cm^2)	1.7	2.4	1.2
σca (kg/cm^2)	85	85	111
σsa (kg/cm^2)	2 340	2 340	3 042
τa (kg/cm^2)	8.2	8.2	10.7

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Check for stirrps

$$\tau = \frac{S}{b \times d} \times Z = \frac{97\,927}{112 \times 106} \times 1.14 = 9.4 \ cm^2$$

$$> \tau_a = 8.2 \ cm^2$$

$$Req \ A_w = \frac{S \times a}{\sigma_{sa} \times d} \times Z = \frac{97\,927 \times 12.5}{1780 \times 106} \times 1.14$$

$$= 7.40 \ cm^2$$

$\Phi 16$ - ctc 125 nos = 4

Used $A_w = 2.01 \times 4 = 8.04 \ cm^2 > Req \ A_w = 7.40 \ cm^2$

§ 6 Caiculation of stability

Pier self weight

Cantilever beam	½×(0.80+1.30)×1.75×1.160×2.407×2	10.26
beam	1.30×5.40×1.160×2.407	19.60
column	1.20×5.50×0.960×2.407×2	30.50
Footing	3.00×6.40×1.20×2.407	55.46
Σ		115.82

Reaction due to superstructure

	P
Dead load	151.7
HA Live load	108.5
HB Live load	116.7
Crowd load	——
Total (HA)	260.2
Total (HB)	268.4

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Transverse direction

a) due to wind

· for superstructure

Unloaded $P_t = 3.4$ ton

Live loaded $P_t = 4.0$ ton

· for safety fences

$P_t = 0.9$ ton

· for pier

Unloaded $P_{t-1} = 0.7$ ton

$P_{t-2} = 4.3$ ton

Live loaded $P_{t-1} = 0.2$ ton

$P_{t-2} = 1.5$ ton

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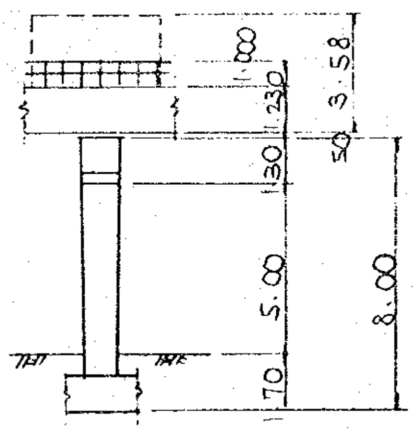

Longitudinal direction

a) due to braking

Under HA ; FB = 25.8 × ½ = 12.9 ton

" HB ; FB = 38.3 × ½ = 19.2 ton

b) due to wind

· for superstructure

Unloaded P_{LS} = 0.9 ton

Live loaded P_{LL} = 1.4 ton

P_{LS} = 0.3 ton

· for safety fences P_{L1} = 0.1 ton

P_{L2} = 0.3 ton

· for pier

Unloaded P_{L1} = 4.9 ton

P_{L2} = 5.8 ton

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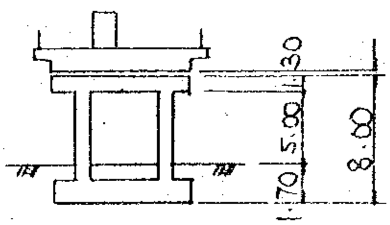
Live loaded P_{L-1} = 1.7 ton

P_{L-2} = 2.0 ton



		Vertical Force N (t)	distance x (m)	Moment N·x (t·m)	Horizontal Force H (t)	distance y (m)	Moment H·y (t·m)
	WDL	151.7					
	WLL (HA)	108.5			12.9	8.05	103.85
	WLL (HB)	116.7			19.2	8.05	154.56
	Self weight	115.8					
Transverse direction	WL (Unloaded)						
	for superstruct.				3.4	8.665	29.46
	for safety fence				0.9	9.780	8.80
	for pier (1)				0.7	7.350	5.15
	(2)				4.3	4.200	18.06
	WL (Live loaded)						
	for superstruct (1)				4.0	9.840	39.36
	for pier (1)				0.2	7.350	1.47
	" (2)				1.5	4.200	6.30
Longitudinal direction	WL (Unloaded)						
	for superstruct.				0.9	8.05	7.25
	for safety fence				0.4	8.05	3.22
	for pier (1)				4.9	7.35	36.02
	(2)				5.8	4.20	24.36
	WL (Live loaded)						
	for superstruct. (1)				1.4	8.05	11.27
	(2)				0.3	8.05	2.42
	for pier (1)				1.7	7.35	12.50
	(2)				2.0	4.20	8.40

Longitudinal direction

Case No	Loading combination	N (t)	H (t)	M (t)
1	Dead load only	267.5	0	0
2	Dead load + wind	267.5	12.0	70.85
3	HA Loading	376.0	12.9	103.85
4	HB Loading	384.2	19.2	154.56
5	HA Loading + wind	376.0	18.3	138.44
6	HB Loading + wind	384.2	24.6	189.15

Calculation of Stability

- eccentric $e = M/N$ $\qquad B/6 = 0.50$
- Soil reaction i) $e < B/6$

$$q_{\substack{max \\ min}} = \frac{N}{B \times L} \left(1 \pm \frac{6 \times e}{B} \right)$$

ii) $e > B/6$ $\qquad x = (\frac{B}{2} - e) \times 3$

$$q_{max} = \frac{2 \cdot N}{B \times L}$$

- Sliding $F = \dfrac{N \times 0.6}{H}$

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Case No.	B/6 (m)	eccentric e (m)	Soil reaction q max (t/m²)	q min (t/m²)	qa t/m²	Sliding F	Fa
1		0	13.9				
2	0.500 ↓	0.265	21.3	6.5	60.0	13	1.5 ↓
3		0.276	30.4	8.8	60.0	17	
4		0.402	36.1	3.9	75.0	12	
5		0.368	34.0	5.2	69.0	12	
6		0.492	39.7	0.3	78.0	9	

Transverse direction

Case No	Loading combination	N (t)	H (t)	M (t)
~~1~~	~~Dead load only~~			
2	Dead load + wind	267.5	9.3	61.47
~~3~~	~~HA Loading~~			
~~4~~	~~HB Loading~~			
5	HA Loading + wind	376.0	5.7	47.13
6	HB Loading + wind	384.2	5.7	47.13

Calculation of Stability

- eccentric $e = M/N$ $\quad$ $B/6 = 1.07$
- Soil reaction i) $e < B/6$

$$q_{\substack{max\\min}} = \frac{N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right)$$

ii) $e > B/6$

$$q_{max} = \frac{2 \cdot N}{B \cdot L}$$

- Sliding $F = \dfrac{N \times 0.6}{H}$

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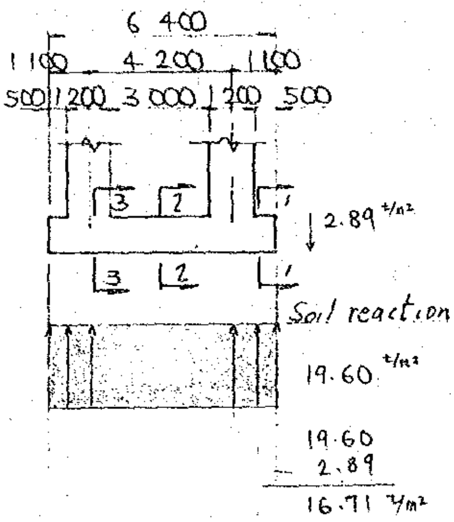
Case No.	B/6 (m)	eccentric e (m)	Soil reaction q_{max} (t/m²)	Soil reaction q_{min} (t/m²)	q_a t/m²	Sliding F	Sliding Fa
~~1~~							
2	1.07	0.230	16.9	10.9	60.0		
~~3~~							
~~4~~							
5	1.07	0.125	21.9	17.3	69.0		
6	1.07	0.123	22.3	17.7	78.0		

5

Calculation of footing section

1 Transverse direction



Sect 1.

$M = 16.71 \times 0.50^2 \times \frac{1}{2} \times 3.0 = 6.27$ t·m

$M' = 6.27 \times \frac{1}{3} = 2.09$ tm

Sect 2.

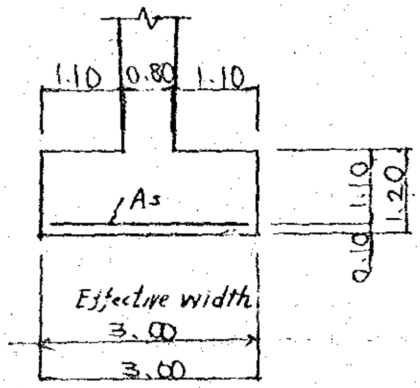
$M = 16.71 \times 3.00^2 \times \frac{1}{8} \times 3.0 = 56.40$ tm

$M' = 56.40 \times \frac{1}{3} = 18.80$ tm

Sect 3.

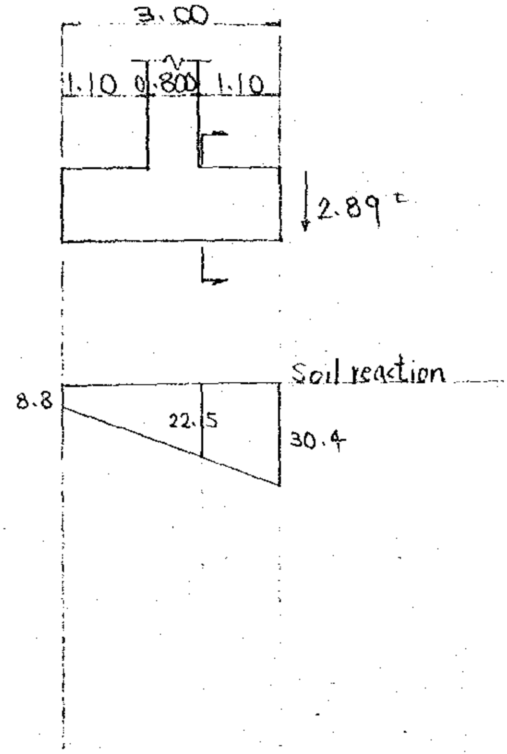
$M = 16.71 \times 4.20^2 \times \frac{1}{12} \times 3.0 = 73.69$ t·m

$M' = 73.69 \times \frac{1}{3} = 24.56$ t·m



28

2 Longitudinal direction



$$S = 2.89 \times 1.10 \times 6.40 \quad (20.35)$$

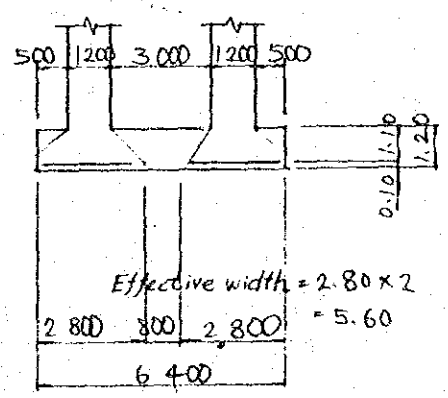
$$- \frac{1}{2} \times (22.5 + 30.4) \times 1.10 \times 6.40 \quad (186.21) = -165.86$$

$$M = 20.35 \times \frac{1}{2} \times 1.10 \quad (11.19)$$

$$-165.86 \times \frac{1}{3} \times \frac{2 \times 30.4 + 22.5}{30.4 + 22.5} \times 1.10 \quad (95.76) = -84.57$$

$$S' = -165.8 \times \frac{1}{5.60} = 29.61 \text{ ton}$$

$$M' = -84.57 \times \frac{1}{5.60} = 15.10 \text{ t.m}$$



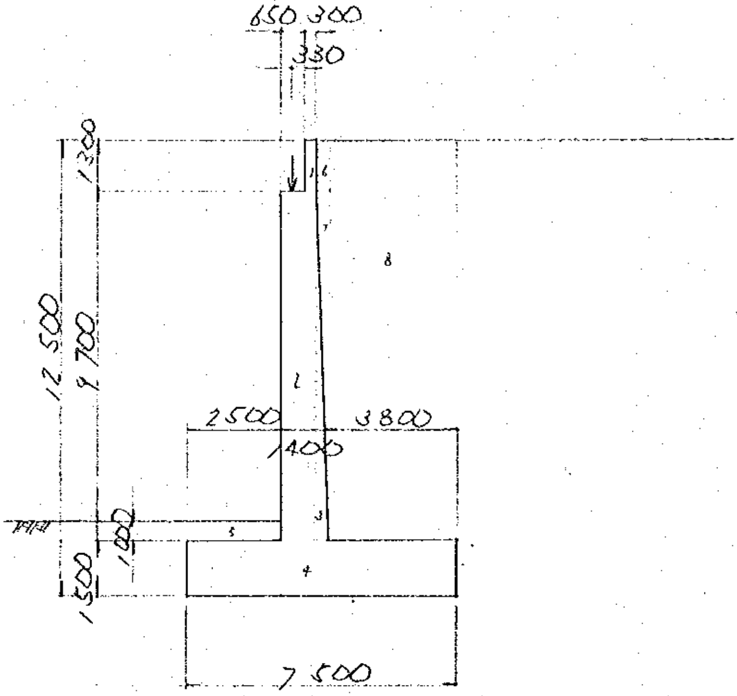
29

	Transverse direction		Longitudinal direction
	case bottom	case Top	case bottom
M (tm)	24.56	18.80	15.10
N (ton)			
Q;shear force (ton)			
b (cm)	100	100	100
d (cm)	110	110	110
d' (cm)	10	10	10
As (cm^2)	D16 - 125 =16.08	D16 - 125 =16.08	D16 - 125 =16.08
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.022	0.022	0.022
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm²)	2.03	1.55	1.25
Q/b·d (kg/cm^2)			
C	11.3	11.3	11.3
S	48.5	48.5	48.5
Z			
σ_c (kg/cm^2)	22.9	17.5	14.1
σ_s (kg/cm^2)	1477	1128	909
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)			
σ_{sa} (kg/cm^2)			
τ_a (kg/cm^2)			

Vol. 7

M.J G-RAMP BR. A_2 Abutment

§ 1 Dimension



§ 2 Calculation of Load

2-1 selfweight and weight of soil

		N (t)	x (m)	N · x (t·m)
1	0.30 × 1.30 × 12.0 × 2.407	11.26	3.30	37.17
2	0.95 × 9.70 × 12.0 × "	266.17	2.975	791.84
3	½ × 0.35 × 9.70 × 12.0 × "	49.03	3.567	174.89
4	7.50 × 1.50 × 12.0 × "	324.95	3.750	1218.54
5	2.50 × 1.00 × 12.0 × 1.9	57.00	1.25	71.25
6	0.35 × 1.30 × 12.0 × "	10.37	3.625	37.61
7	½ × 0.35 × 9.70 × 12.0 × "	38.70	3.683	142.54
8	3.7 × 11.0 × 12.0 × "	927.96	5.65	5242.97
Σ		1685.44		7716.81

2-2 Weight of Surcharge

Under HA = 1.02 × 4.35 × 12.0 = 53.24 t

HB = 1.66 × 4.35 × 12.0 = 86.65 t

2-3 Earth pressure

$P_a = \frac{1}{2} \times 0.271 \times 1.9 \times 12.5^2 \times 12.0 = 482.72$

$P_{a\,add\,(HA)} = 0.271 \times 1.02 \times 12.5 \times 12.0 = 41.46$

$P_{a\,add\,(HB)} = 0.271 \times 1.66 \times 12.5 \times 12.0 = 67.48$

2-4 Reaction due to superstructure

(unit - tons)

	HA loading	HB loading
Dead load of deck	202.7	202.7
live load	73.7	123.4
Total	276.4	326.1
Longitudinal Force (skew 45°)	18.2	27.0

2-5 Temperature load

$\delta = 0.5 \times 17.03 = 8.5^{mm} = 0.85^{cm}$

$t = 3^{cm}$

$A = 30^{cm} \times 50^{cm} = 1500^{cm^2}$ $G = 13.5$

$P_H = \frac{13.5 \times 1500 \times 0.85}{3} = 5738^{kg} = 5.74^{ton}$ / 1-shoe

$\Sigma P_H = 10 \times 5.74^{ton} = 57.4^{ton}$

3

§ 3. Calculation of Stability

Case 1. HA Loading

	N (t)	x (m)	N x (t m)	H (t)	y (m)	H y (t m)
WDL WLL	276.40	2.820	779.45			
FB				18.20	11.250	204.75
T				57.90	11.250	695.75
W Ws, WT	1685.44		7716.81			
Ws add	53.24	5.325	283.50			
Pa				482.72	4.167	2011.49
Pa add				41.46	6.250	259.13
TOTAL	2015.08		8779.76	599.78		3121.12

1) check for eccentric

$$x = \frac{\Sigma N \cdot x - \Sigma H y}{\Sigma N} = \frac{8779.76 - 3121.12}{2015.08} = 2.808$$

$$e = \frac{B}{2} - x = 3.750 - 2.808 = 0.941 < \frac{B}{6} = 1.250$$

2) Soil reaction

$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{2015.08}{7.5 \times 12.0} \times \left(1 \pm \frac{6 \times 0.941}{7.5}\right)$$

$$= \begin{matrix} 39.26 \\ 5.52 \end{matrix} \ t/m^2$$

3) check for sliding

$$F = \frac{\Sigma N \cdot \tan\phi}{H} \ (=0.6) = \frac{2015.08 \times 0.6}{599.78} = 2.0$$

Case - 2 HB Loading

	N (t)	x (m)	N x (t m)	H (t)	y (m)	H y (t m)
WDL WLL	326.10	2.820	919.60			
FB				27.00	11.250	303.75
T				57.40	11.250	645.75
W, Ws, WT	1685.44		7716.81			
Ws add	86.65	5.325	461.41			
Pa				482.72		2011.49
Pa add				67.48	6.250	421.75
TOTAL	2098.19		9097.82	634.60		3382.74

1) check for eccentric

$$x = \frac{\Sigma N \cdot x - \Sigma H y}{\Sigma N} = \frac{9097.82 - 3382.74}{2098.19} = 2.724$$

$$e = \frac{B}{2} - x = 3.750 - 2.724 = 1.026 < \frac{B}{6} = 1.25$$

2) Soil reaction

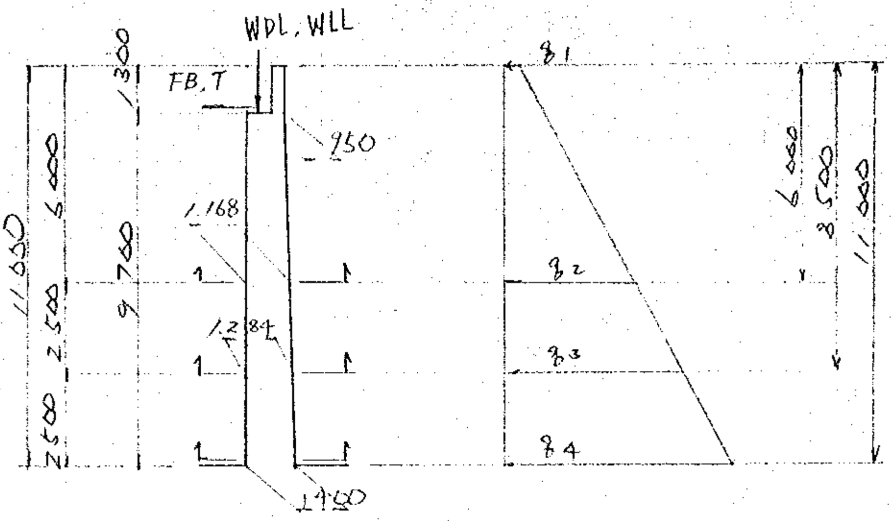
$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{2098.19}{7.5 \times 12.0} \times \left(1 \pm \frac{6 \times 1.026}{7.5}\right)$$

$$= \begin{bmatrix} 42.45 \\ 4.17 \end{bmatrix} t/m^2$$

3) check for sliding

$$F = \frac{\Sigma N \cdot \tan\phi}{H} = \frac{2098.19 \times 0.6}{634.60} = 1.98 \quad (\tan\phi = 0.6)$$

§ 4 Calculation of wall section



	N (t)	H (t)
Dead Load of Deck	16.89	
HA Live	6.14	6.30
HB Live	10.28	7.03

	q1	q2	q3	q4
HA loading	0.28	3.36	4.65	5.94
HB "	0.45	3.54	4.83	6.11

Section 1-1

	N (t)	H (t)	y (m)	M (t·m)
Reaction due to superstructure	23.03	6.30	(1.24) 4.75	5.53 29.93
self weight	11.71 0.94			
Earth pressure		10.92	2.154	23.52
Total	35.68	17.22		58.98

Section 2-2

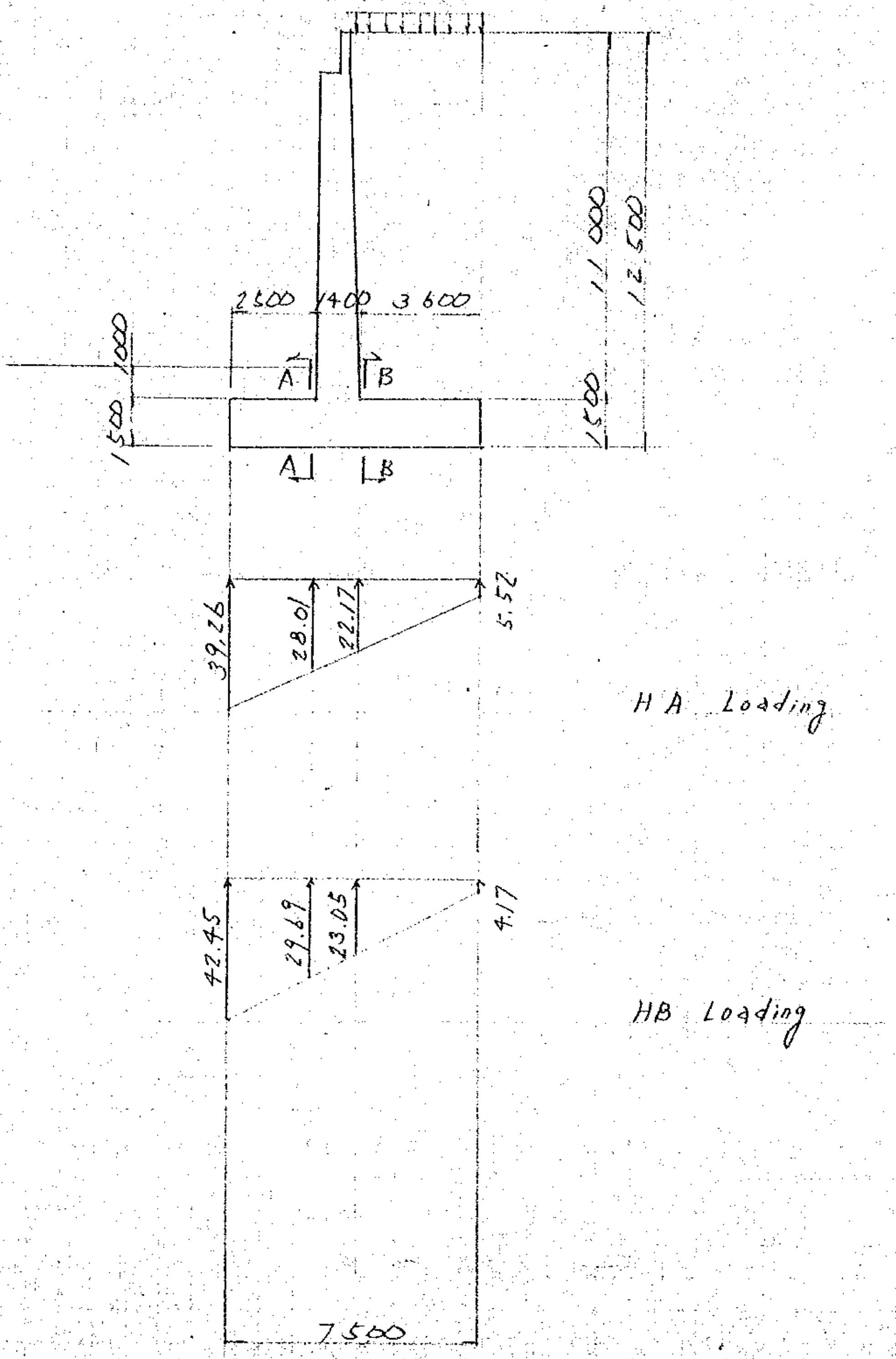
	N (t)	H (t)	y (m)	M (t·m)
Reaction due to superstructure	23.03	6.30	7.25	5.53 45.68
self weight	18.72 0.94			
Earth pressure		20.95	2.994	62.73
	42.69	27.25		113.94

Section 3-3

	N (t)	H (t)	y (m)	M (t·m)
Reaction due to superstructure	23.03	6.30	9.75	5.53 61.43
Self weight	26.27 0.94			
Earth pressure		34.21	3.832	131.08
	50.24	40.51		198.09

	1-1	2-2	3-3
M (tm)	58.98	113.94	198.09
N (ton)	35.68	42.69	50.29
Q;shear force (ton)	17.22	27.25	40.51
b (cm)	100	100	100
d (cm)	107	118	130
d' (cm)	10	10	10
As (cm^2)	Φ25 - 250ctc 19.64	Φ25 - 250 } Φ32 - 250 } (As' = 0) 51.8	Φ32 - 125ctc (As' Φ32 - 250ctc) 64.32
d'/d	0.10	0.09	0.08
f=(M/N)+u (cm)	u=48.5 cm 213.8	u=54.0 cm 320.9	u=60.0 cm 454.1
f/d	2.0	2.7	3.5
n	15	15	15
nP=n·As/b·d	0.028	0.066	0.074
M'=M+N·u (tm)	76.28	136.99	228.18
M'/(b·d^2) (kg/cm^2)	6.663	9.840	13.500
Q/b·d (kg/cm^2)	1.61	2.31	3.12
C	7.95	6.4	5.75
S	21.0	11.6	11.4
Z	1.08	1.11	1.10
σ_c (kg/cm^2)	53	63	78
σ_s (kg/cm^2)	2099	1712	2309
τ (kg/cm^2)	1.7	2.56	3.4
σ_{ca} (kg/cm^2)	85		
σ_{sa} (kg/cm^2)	2346		
τ_a (kg/cm^2)	2.35	3.47	3.47
		$\frac{100 As}{b \cdot d} \fallingdotseq 0.5$	$\frac{100 As}{b \cdot d} \fallingdotseq 0.5$

§ 5. Calculation of Footing Section



Section A-A

		V	x	M
Wt	1.0 × 2.5 × 1.9	4.75	1.250	5.94
W	1.5 × 2.5 × 2.407	9.03	1.250	11.28
g	$-\frac{1}{2}$ (39.26 + 28.01) × 2.5	-84.09	1.320	-110.97
Σ		-70.31		-93.75

Shear Force V = 70.31 ^{t}

Bending Moment M = 93.75 $^{t \cdot m}$

Section B-B

		V	x	M
Ws	11.0 × 3.70 × 1.9	77.33	1.85	143.06
W	1.5 × 3.70 × 2.407	13.36	"	24.71
Wsold	3.70 × 1.02	3.77	"	6.98
g	$-\frac{1}{2}$ (22.17 + 5.32) × 3.70	-51.23	1.479	-75.76
Σ		43.23		98.99

	Sect A-A	Sect B-B	
M (tm)	93.75	98.99	
N (ton)	0.0	0.0	
Q;shear force (ton)	70.81	45.23	
b (cm)	100	100	
d (cm)	140	140	
d' (cm)	10	10	
As (cm^2)	Φ25-125 39.28 cm^2	Φ25-125 39.28	
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.042	0.042	
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)	4.78	5.05	
Q/b·d (kg/cm^2)			
C	8.71	〃	
S	26.2	〃	
Z	1.09	〃	
σc (kg/cm^2)	42	44	
σs (kg/cm^2)	1879	1984	
τ (kg/cm^2)	5.0 *	3.1 *	
σca (kg/cm^2)	85	85	
σsa (kg/cm^2)	2346	2346	
τa (kg/cm^2)	2.35	2.35	

See next page

check for stirrups

$$\tau = \frac{S_h}{b \cdot d} \times Z$$

$$= \frac{70.31 \times 10^3}{100 \times 140} \times 1.09 = 5.47 > \tau_a = 2.35$$

$$\text{req. } A_w = \frac{S_h' \times a}{\sigma_{sa} \cdot d} \times Z \quad (cm^2)$$

$$S_h' = S_h - S_c$$

$$S_c = \tau_a \times b \times d \times \frac{1}{Z}$$

$$= 2.35 \times 100 \times 140 \times \frac{1}{1.09} = 30\,183 \text{ kg}$$

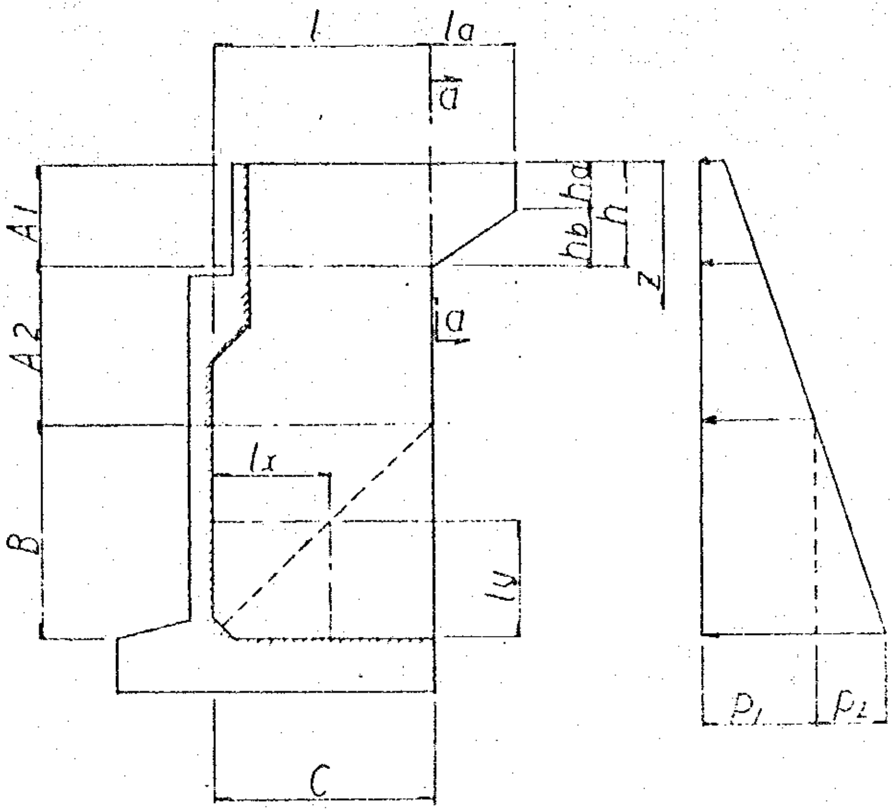
$$S_h' = (70.31 - 30.18) \times 10^3 = 40.13 \times 10^3$$

$$\text{req. } A_w = \frac{40.13 \times 10^3 \times 25}{1780 \times 140} \times 1.09 = 4.39 \text{ cm}^2$$

Φ 20 - ctc 250 $\quad n = 2$

Used $A_w = 3.14 \times 2 = 6.28 > \text{req. } A_w = 4.39$

§ 6 CALCULATION OF WING SECTION



		S (t)	M (tm)
a-a	$0 < z < h_a$	$(q+\gamma\cdot z)\cdot K\cdot l_a$	$(q+\gamma\cdot z)\cdot K\cdot \frac{l_a^2}{2}$
	$h_a < z < h$	$(q+\gamma\cdot z)\cdot K\cdot l_a\cdot \frac{h-z}{h_b}$	$(q+\gamma z)\cdot K\cdot \frac{l_a^2}{2}\cdot\left(\frac{h-z}{h_b}\right)^2$

$$M_{max}\ (h_a < z < h) \longrightarrow z = \frac{\gamma\cdot h - 2\cdot q}{3\gamma}\ (m)$$

$$q = 1.02\ t/m^2$$

$$K = 0.27$$

$$\gamma = 1.9\ t/m^3$$

	M (t)	S (tm)
A1 − 1	$\frac{1}{2} p \cdot l^2 + Ma + Sa \cdot l$	$p l + Sa$
A2 − 2	$\frac{1}{2} \cdot p \cdot l^2$	$p \cdot l$
B − B	$\frac{1}{2} \cdot p \cdot lx^2$	$p \cdot lx$
C − C	$(\frac{p_1}{2} + \frac{p_2}{6})\, ly^2$	$(p_1 + \frac{p_2}{2}) \cdot ly$

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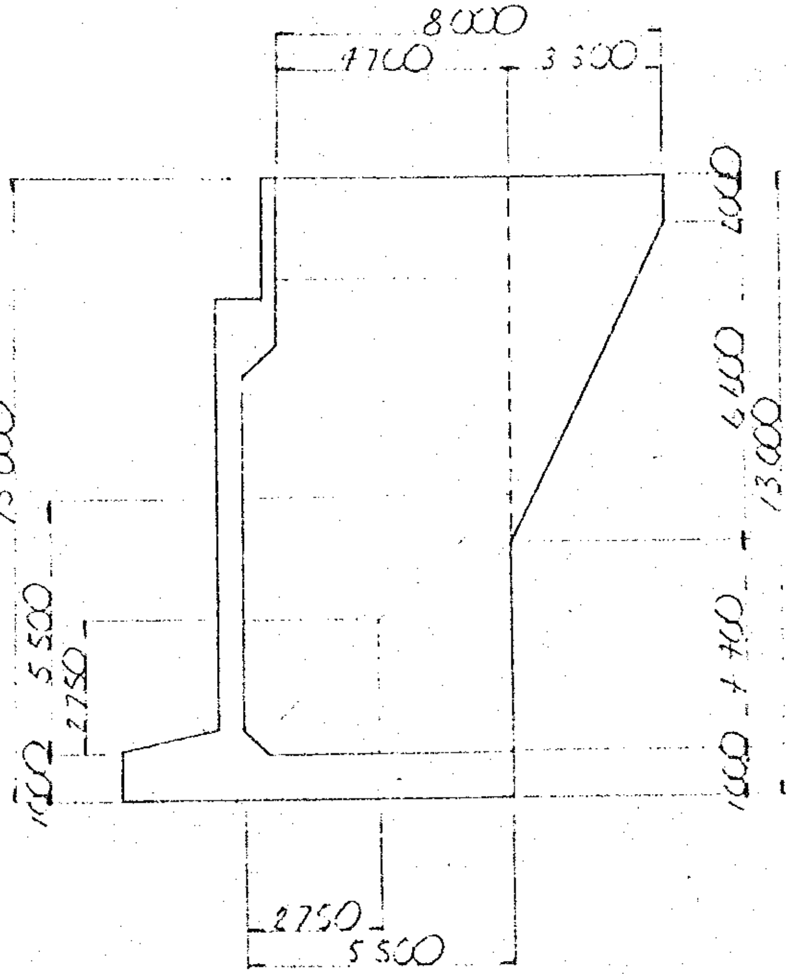
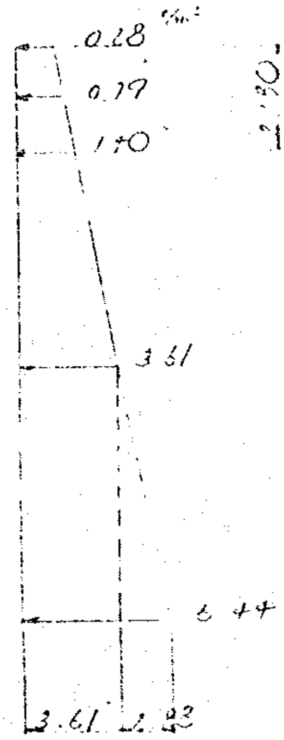
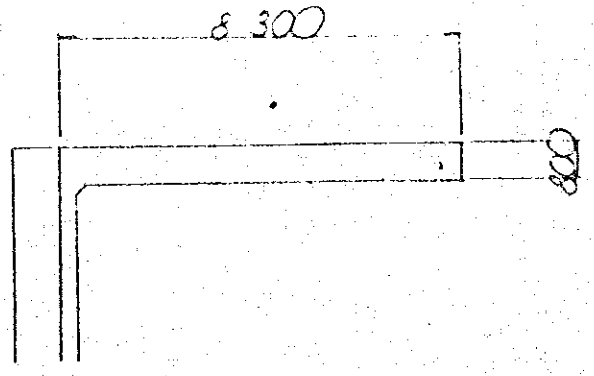
(G - RAMP , A 2 , L)

9 - 1 dimension and loading







		x (m)	M (tm)		S (t)	
a – a	1 – 1	1.00	$(1.02 + 1.9 \times 1.00) \times 0.27 \times \frac{1}{2} \times 3.30^2$	4.29	$(1.02 + 1.9 \times 1.00) \times 0.27 \times 3.30$	2.60
	2 – 2	1.00 ~ 7.60	$(1.02 + 1.9 \times 2.18) \times 0.27 \times \frac{3.30^2}{2} \times \left(\frac{7.60 - 2.18}{6.60}\right)^2$	5.12	$(1.02 + 1.9 \times 2.18) \times 0.27 \times 3.30 \times \frac{7.60 - 2.18}{6.60}$	3.78
A 1		0 ~ 7.60	$\frac{1}{2} \times 1.40 \times 5.50^2 + 5.12 + 3.78 \times 5.50$	47.09	$1.40 \times 5.50 + 3.78$	11.48
A 2	1 – 1		$\frac{1}{2} \times 3.61 \times 5.50^2$	54.60	3.61×5.50	19.86
	2 – 2	—	—	—	—	—
B – B		7.60 ~ 12.00	$\frac{1}{2} \times \frac{3.61 + 6.44}{2} \times 2.75^2$	19.00	$\frac{3.61 + 6.44}{2} \times 2.75$	13.82
C – C		12.00	$\left(\frac{3.61}{2} + \frac{2.83}{6}\right) \times 2.75^2$	16.90	$\left(3.61 + \frac{2.83}{2}\right) \times 2.75$	13.82

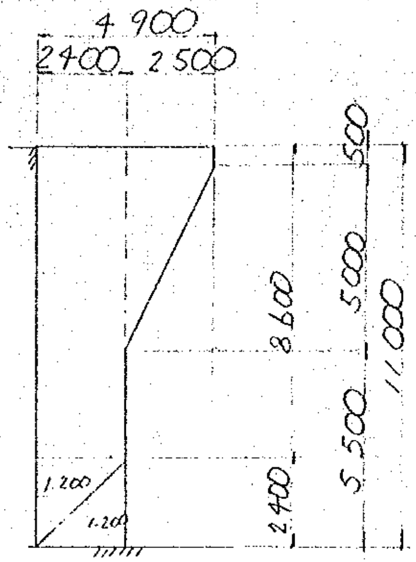
$$z = \frac{1.9 \times 7.60 - 2 \times 1.02}{3 \times 1.9} = 2.18^{m}$$

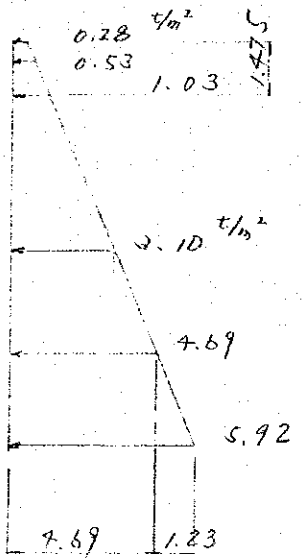
9 - 2 list of stresses σ_c, σ_s, τ : working stress.

$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

※ $A_{s\,min}$ = 100 × 73 × 0.0015 = 10.95 cm^2

	a - a	A_1	A_2	B	C	
M	5.12	47.09	54.10	19.00	16.90	
N	——	——	——	——	——	
S	3.78	11.48	19.86	13.82	13.82	
b	100	———	———	———	——→	
h	73	———	———	———	——→	
d'	7	———	———	———	——→	
As	D20@250 = 12.56	D25@125 39.18	——→	D25@250 19.64	D16@125 16.08	
As'	——	——	——	——	——	
f/d		0	0	0	0	
M'/bd²		10.64	10.15	3.57	3.17	
S/bd		2.02	2.72	1.59	1.89	
n·P		0.0807	0.0807	0.0404	0.0330	
C		6.83	6.85	8.84	9.57	
S		13.73	13.93	27.02	32.77	
Z		1.12	1.12	1.09	1.02	
σ_c		73	70	32	31	
σ_s		2222	2240	1445	1559	
τ		2.3	3.1	2.1	2.0	
σ_{ca}	83	———	———	———	——→	
σ_{sa}	2346	———	———	———	——→	
τ_a	2.35	3.47	3.47	2.35	2.35	





$$z = \frac{1.9 \times 5.5 - 2 \times 1.02}{3 \times 1.9} = 1.475^{m}$$

		Z (m)	M (t·m)		S (t)	
a	1-1	0.5	$0.53 \times \frac{1}{2} \times 2.5^2$	1.66	0.53×2.5	1.33
a	2-2	0.5 ~ 5.5	$1.03 \times \frac{2.5^2}{2} \times \left(\frac{5.5-1.475}{5.0}\right)^2$	2.09	$1.03 \times 2.5 \times \left(\frac{5.5-1.475}{5.0}\right)$	2.07
A1		0 ~ 5.5	$\frac{1}{2} \times 2.4^2 \times 3.10 + 2.09 + 2.07 \times 2.4$	15.99	$2.4 \times 3.10 + 2.07$	9.51
A2	1-1	8.6	$\frac{1}{2} \times 2.4^2 \times 4.69$	13.51	2.4×4.69	11.26
A2	2-2					
B-B		8.6 ~ 11.0	$\frac{1}{2} \times (4.69+5.92) \times 1.2^2 \times \frac{1}{2}$	3.82	$\frac{1}{2} \times (4.69+5.92) \times 1.2$	6.37
C-C		9.8	$\left(\frac{4.69}{2} + \frac{1.23}{6}\right) \times 1.2^2$	3.67	$\left(4.69 + \frac{1.23}{2}\right) \times 1.2$	6.37

Used As　　A_1 and A_2 1-1 → Φ16 @ 125 = 16.08 cm^2

others → Φ16 @ 250 = 8.09 cm^2