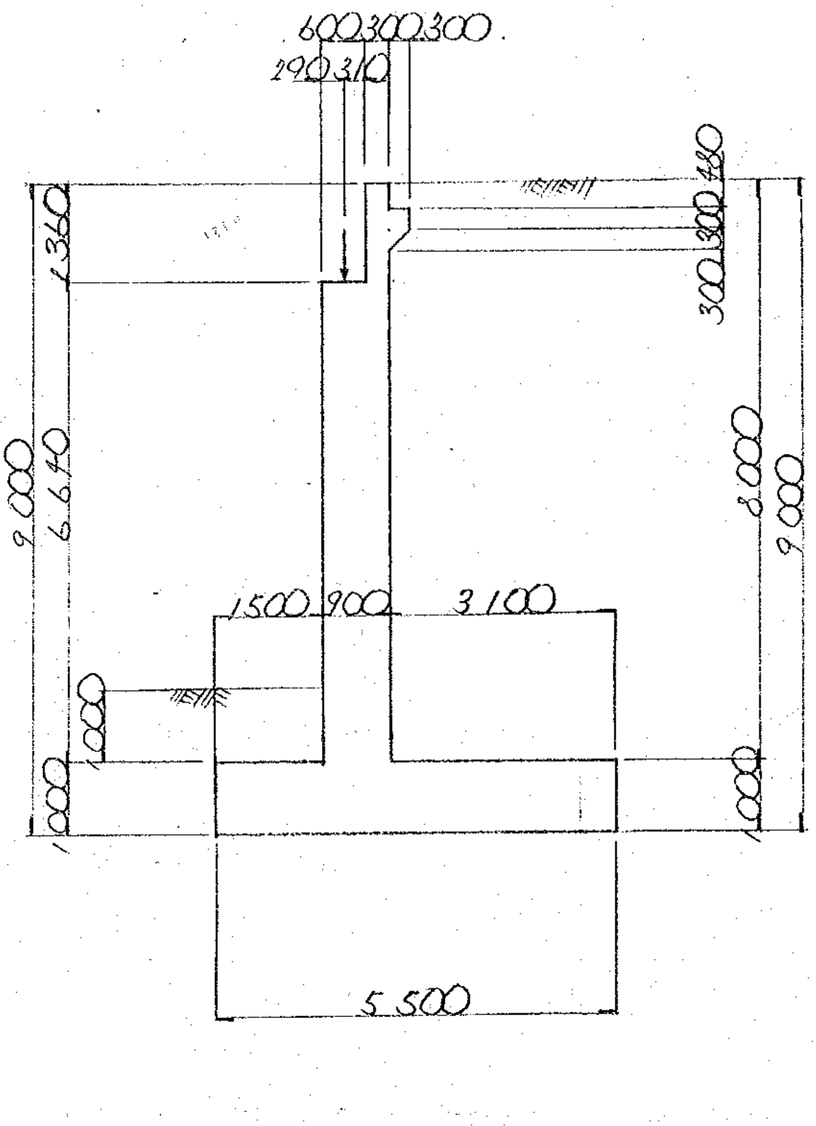
§§ 7 H = 9.00 m

B - L - 3 A 2

§ 1 STRUCTURAL FIGURE

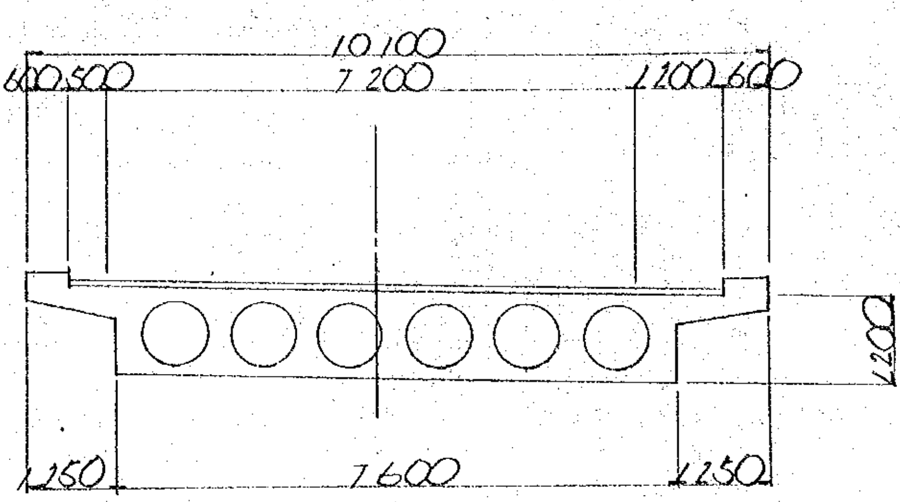


$\theta = 62°10'19''$

$L = 11.420^{m}$

§ 2 REACTION OF SUPERSTRUCTURE

2 - 1 structural figure



2 - 2 whole reaction of superstructure

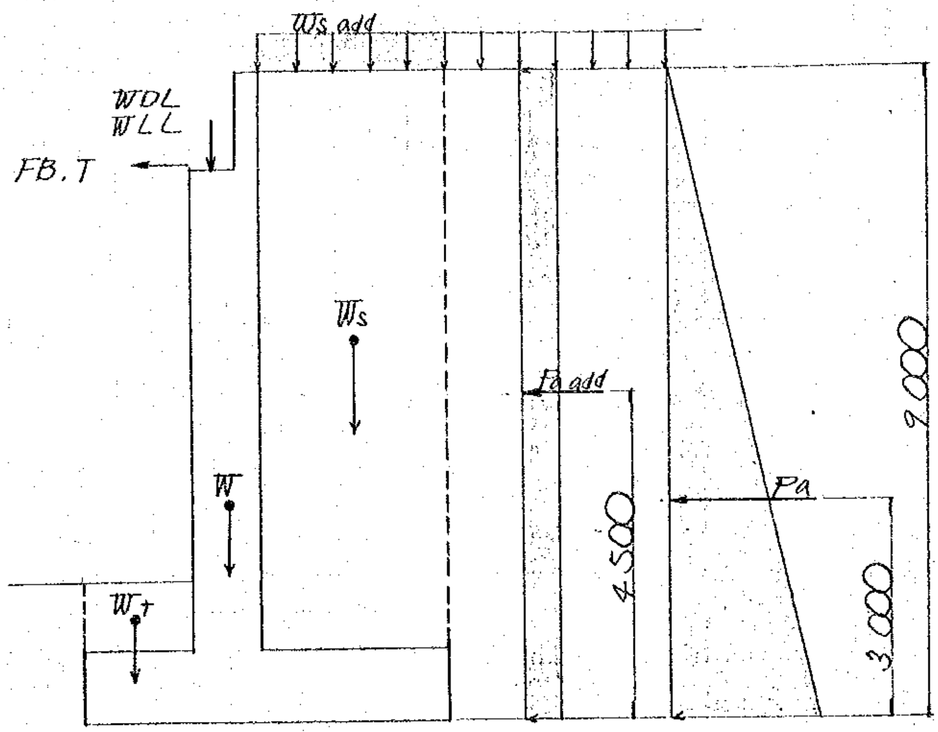
		HA loading		HB loading	
		N (t)	H (t)	N (t)	H (t)
dead load of deck	I	214.10	——	214.10	——
	II	32.20	——	32.20	——
live load		115.70	——	146.30	——
crowd load		——	——	——	——
longitudinal forse		——	25.80	——	38.20
TOTAL		362.00	25.80	392.60	38.20

$$H'_{HA} = 25.80 \times \sin 62°10'19'' = 22.82^{t}$$

$$H'_{HB} = 38.20 \times \sin 62°10'19'' = 33.78^{t}$$

§ 3 CALCULATION OF LOAD

3-1 loading diagram



WDL : dead load of deck

WLL : max LL reaction under HA & HB

F B : HA & HB braking

W : self weight

Ws : weight of soil

Wt : fill on toe

Ws add : weight of surcharge

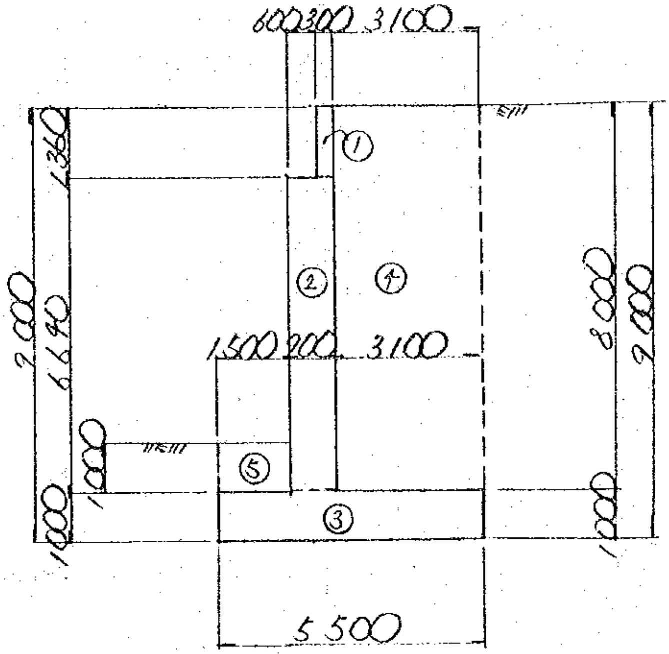
P_A : active pressure

P_P : passive pressure

P_A add : surcharge

T : temperatuwe load

3 - 2 self weight and weight of soil



		N (t)	x (m)	N·x (tm)
①	0.30 × 1.36 × 11.42 × 2.41	11.23	2.250	25.27
②	0.90 × 6.64 × 11.42 × 2.41	164.47	1.950	320.72
③	5.50 × 1.00 × 11.42 × 2.41	151.37	2.750	416.27
④	3.10 × 8.00 × 11.42 × 1.9	538.11	3.950	2125.53
⑤	1.50 × 1.00 × 11.42 × 1.9	32.55	0.750	24.41
Σ		897.73		2912.20

3-3 weight of surcharge

under H.A $= 1.02 \times 3.10 \times 11.42 = 36.11^t$

under H.B $= 1.66 \times 3.10 \times 11.42 = 58.77^t$

3-4 earth pressure

unit weight of soil $\gamma_s = 1.9$ t/m³

angle of internal friction $\phi = 35°$

(1) active pressure

$$Pa = \frac{1}{2} \cdot K \cdot \gamma_s \cdot H^2 \cdot L$$

$$= \frac{1}{2} \times \frac{1 - \sin 35°}{1 + \sin 35°} \times 1.9 \times 9.00^2 \times 11.42 = 237.27^t$$

(2) active pressure due to surcharge

under H.A surcharge

$q = 1.02$ t/m²

$$Pa\ add = K \cdot q \cdot H \cdot L$$

$$= 0.27 \times 1.02 \times 9.00 \times 11.42 = 28.31^t$$

under H.B surcharge

$q = 1.66$ t/m²

$$Pa\ add = K q H \cdot L$$

$$= 0.27 \times 1.66 \times 9.00 \times 11.42 = 46.07^t$$

3-5 temperature load

$$P_H = \frac{G_0 \cdot A \cdot \delta}{\Sigma \cdot te}$$

$$\delta = \alpha \cdot l \qquad \alpha = \begin{cases} P.C \longrightarrow 0.7 \\ R.C \longrightarrow 0.5 \end{cases}$$

$$\begin{bmatrix} \delta = 0.5 \times 16.40 = 8.2^{mm} \\ R(d+l) = 392.60 \times 1/6 \times 1.4 = 91.61^{t} \end{bmatrix}$$

RING SHOE

- 100 TON
- $D\phi$: — ^{cm}
- $d\phi$: — ^{cm}
- A : 25800^{cm^2} (860 × 30)
- t : 5^{cm}
- G_0 : $13.5^{kg/cm^2}$ (modulus of rigidity)

$$P_H = \frac{13.5 \times 25800 \times 0.82}{5} = 57121^{kg} = 57.12^{t}$$

$$\Sigma P_H = n \cdot P_H \cdot 1/2$$

$$= 1 \times 57.12 \times 1/2 = 28.56^{t}$$

$$\Sigma P_H' = 28.56 \times \sin 62°10'19'' = 25.26^{t}$$

§ 4 CALCULATION OF STABILITY

case 1 HA loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL, WLL	362.00	1.790	647.98	——	——	——
F B	——	——	——	22.82	7.640	174.34
T	——	——	——	25.26	7.640	192.99
W. Ws. W_T	897.73	——	2912.20	——	——	——
Ws add	36.11	3.950	142.63	——	——	——
P a	——	——	——	237.27	3.000	711.81
Pa add	——	——	——	28.31	4.500	127.40
TOTAL	1295.84		3702.81	313.66		1206.54

1) check for eccentric

$$x = \frac{\Sigma Nx - \Sigma HY}{\Sigma N} = \frac{3702.81 - 1206.54}{1295.84} = 1.926^{m}$$

$$e = \frac{B}{2} - x = \frac{5.500}{2} - 1.926 = 0.824^{m} < \frac{B}{6} = 0.92^{m}$$

2) soil reaction

$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{1295.84}{5.50 \times 11.42} \times \left(1 \pm \frac{6 \times 0.824}{5.50}\right)$$

$$= \begin{cases} 39.18 \ t/m^2 \\ 2.09 \end{cases} < 60 \ t/m^2$$

3) check for sliding

$$Hu = C \cdot A' + N \cdot \tan\phi' \qquad c = 0 \qquad \tan\phi' = 0.6$$

$$F = \frac{Hu}{\Sigma H} = \frac{1295.84 \times 0.6}{313.66} = 2.48 > Fa = 1.5$$

case 2 HB loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL. WLL	392.60	1.790	702.75	—	—	—
F B	—	—	—	38.20	7.640	291.85
T	—	—	—	25.26	7.640	192.99
W. Ws. WT	897.73		2912.20	—	—	—
Ws add	58.77	3.950	232.14	—	—	—
P a	—	—	—	237.27	3.000	711.81
Pa add	—	—	—	46.07	4.500	207.32
TOTAL	1349.10		3847.09	346.80		1403.97

1) check for eccentric

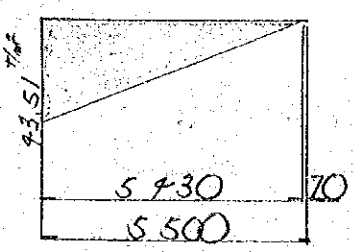
$$x = \frac{\Sigma N x + \Sigma H \cdot y}{\Sigma N} = \frac{3847.09 - 1403.97}{1349.10} = 1.81^{m}$$

$$e = \frac{B}{2} - x = \frac{5.50}{2} - 1.81 = 0.94^{m} < \frac{B}{3} = 1.83^{m}$$

2) soil reaction

$$q = \frac{2 \cdot N}{3 \cdot x \cdot L}$$

$$= \frac{2 \times 1349.10}{3 \times 1.810 \times 11.42} = 43.51 \ t/m^2$$



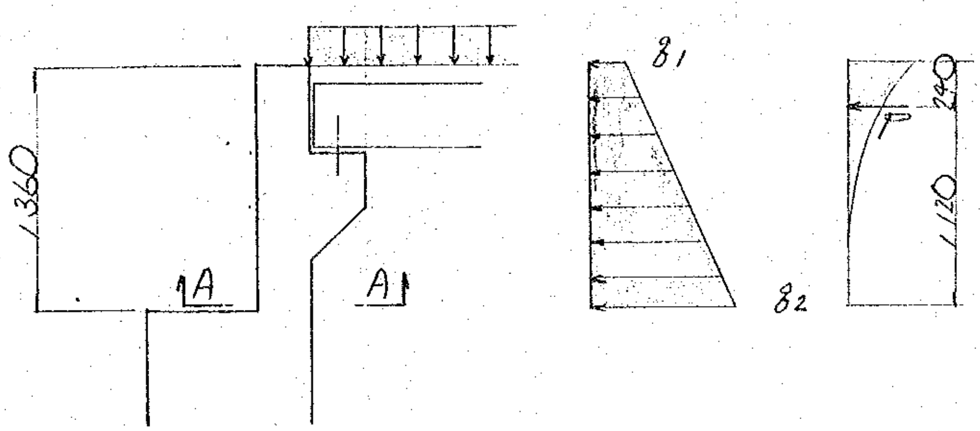
3) check for sliding

$$Hu = c \cdot A' + N \cdot \tan\phi' \qquad c = 0 \qquad \tan\phi' = 0.6$$

$$F = \frac{Hu}{\Sigma H} = \frac{0.6 \times 1349.10}{346.80} = 2.33 \quad > Fa = 1.2$$

§ 5 CALCULATION OF PARAPET SECTION

5-1 dimenston and loading



	q_1	q_2
HA loading	0.28	0.98
HB loading	0.45	1.15

$q_1 = q \cdot K \qquad = 0.27 \cdot q \ \ t/m^2$

$q_2 = K \cdot \gamma_s \cdot H + q_1 = 0.513 \cdot H + 0.27 \cdot q$

5 - 2 sectional force of parapet

CASE 1 (HA)

$$S = 1/2 \times (0.28 + 0.98) \times 1.36 = 0.86 \ t$$

$$M = 0.86 \times 1/3 \times 1.36 \times \frac{2 \times 0.28 + 0.98}{0.28 + 0.98} = 0.97 \ tm$$

CASE 2 (HB)

$$S = 1/2 \times (0.95 + 1.15) \times 1.36 = 1.09 \ t$$

$$M = 1.09 \times 1/3 \times 1.36 \times \frac{2 \times 0.95 + 1.15}{0.95 + 1.15} = 0.63 \ tm$$

CASE 3

$$S = 10.97 \times 0.27 = 2.96 \ t$$

$$M = 2.96 \times 1.12 = 3.32 \ tm$$

5-3 list of stresses

σ_c, σ_s, τ : working stress.

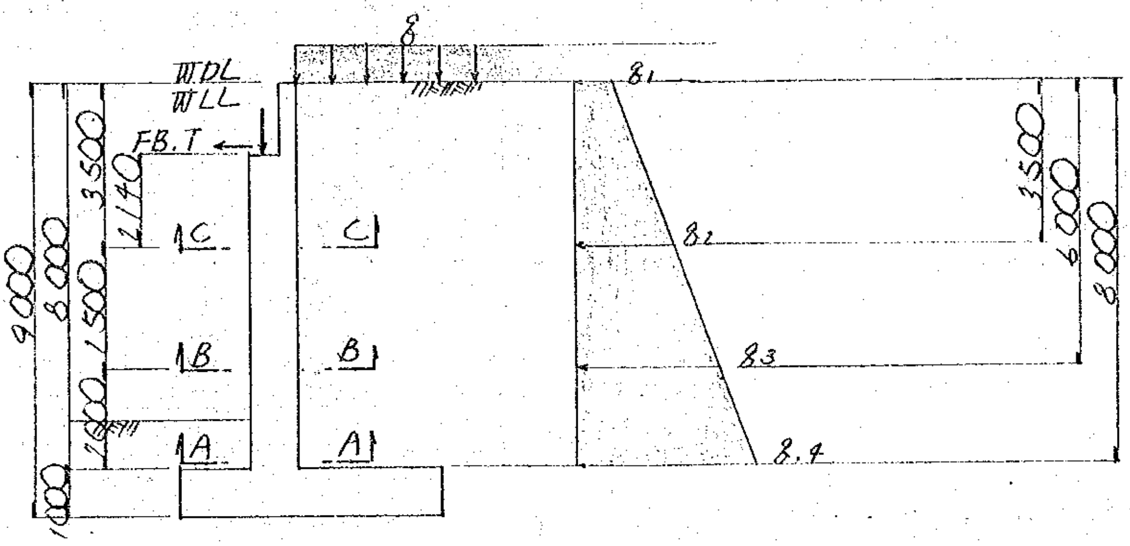
$\sigma_{ca}, \sigma_{sa}, \tau_a$: Permissible stress.

$As_{min} = 100 \times 23 \times 0.0015 = 3.45\ cm^2$

	case 1	case 2				
M	0.47	0.63				
N	—	—				
S	0.86	1.09				
b	100	100				
h	23	23				
d'	7	7				
As	D16@250 =8.04	→				
As'	—	—				
f/d	0	0				
M'/bd^2						
S/bd						
n·P						
C						
S						
Z						
σ_c						
σ_s						
τ						
σ_{ca}	83	103				
σ_{sa}	2346	2933				
τ_a	2.35	2.94				

§ 6 CALCULATION OF WALL SECTION

6 – 1 dimension and loading



	N (t)	H (t)
dead load of deck	21.57	—
HA live load	10.13	4.21
HB live load	12.81	5.56

	q_1	q_2	q_3	q_4
H A loading	0.28	2.08	3.36	4.38
H B loading	0.45	2.45	3.53	4.55

6 − 2 sectional force of wall

section 1 − 1

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	21.57	4.21	2.140	9.01
self weight	5.62	——	——	——
earth pressure	——	4.13	1.305	5.39
TOTAL	27.19	8.34		14.40

section 2 − 2

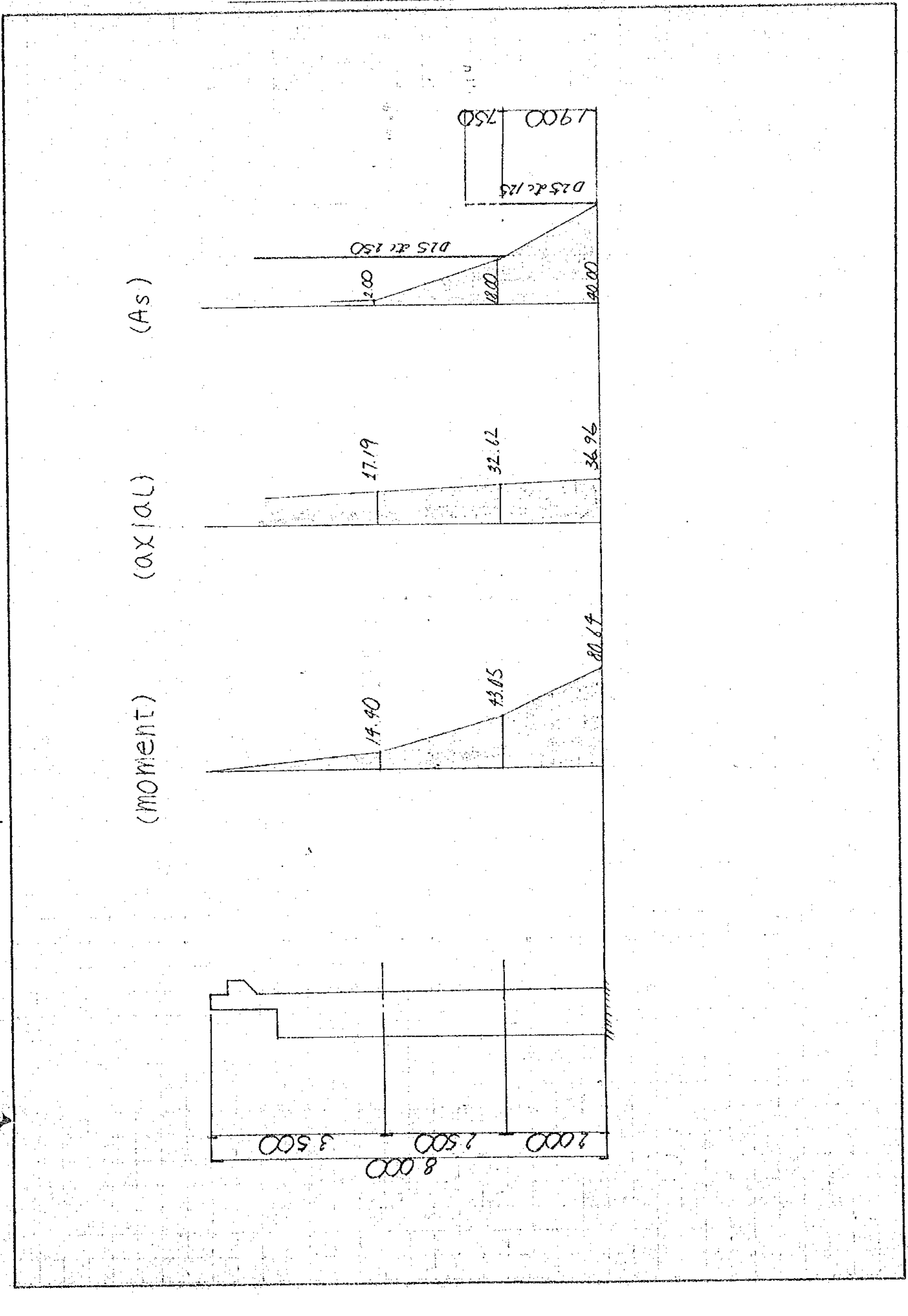
	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	21.57	4.21	4.640	19.53
self weight	11.05	——	——	——
earth pressure	——	10.92	2.154	23.52
TOTAL	32.62	15.13		43.05

section 3 − 3

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	21.57	4.21	6.640	27.95
self weight	15.39	——	——	——
earth pressure	——	18.64	2.827	52.69
TOTAL	36.96	22.85		80.64

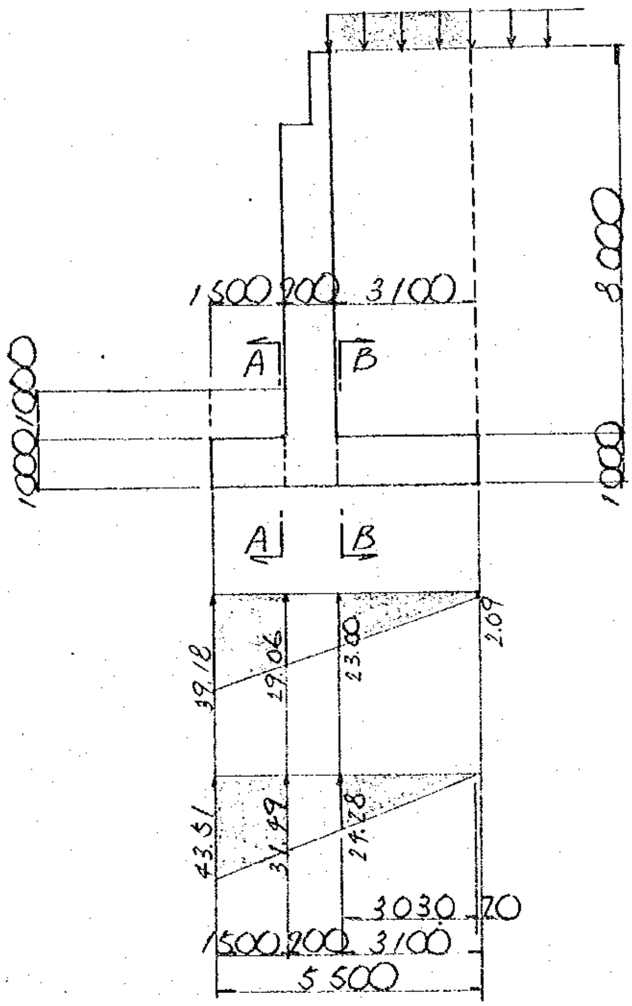
6 - 3 list of stresses σ_c, σ_s, τ : working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: Permissible stress.

	A - A	B - B	C - C			
M	80.64	43.05	14.40			
N	36.96	32.62	27.19			
S	22.85	15.13	8.34			
b	100	→	→			
h	83	→	→			
d'	7	→	→			
As	D25 ctc 125 39.28	D25 ctc 250 19.64	→	As min = b·d·0.15% =		12.45 cm^2
As'	D16 ctc 250 8.04	→	→			
f/d	3.09	2.05	1.09			
M'/bd²	13.74	8.05	3.59			
S/bd	2.75	1.82	1.00			
n·P	0.071	0.0355	0.0355			
C	6.05	7.06	5.15			
S	11.31	17.35	6.95			
Z	1.11	1.07	0.96			
σ_c	83	57	18			
σ_s	2331	2094	374			
τ	3.0	2.0	1.0			
σ_{ca}	83	→	→			
σ_{sa}	2346	→	→			
τ_a	3.47	2.35	2.35			


(moment)
14.90
43.05
80.64
(axial)
17.19
32.62
36.96
(As)
2.00
18.00
40.00
D15 ctc 150
D25 ctc 125
1900
750
3500
2500
2000
8000

§ 7 CALCULATION OF FOOTING SECTION

7 - 1 dimenston and loading



	q 1	q 2	q 3	q 4
HA loading	39.18	29.06	23.00	2.09
HB loading	43.51	31.49	24.28	0

7 - 2 sectional force of footing

section A - A

CASE 1

		S	x	M
Wt	1.50 × 1.00 × 1.9	2.85	0.750	2.14
W	1.50 × 1.00 × 2.41	3.62	0.750	2.71
q (HA)	-½ × (39.18 + 29.06) × 1.50	- 51.18	0.787	- 40.28
Σ		44.71		35.43

$$\begin{cases} M = 35.43 \ tm \\ S = 44.71 \ t \end{cases}$$

CASE 2

		S	x	M
Wt	1.50 × 1.00 × 1.9	2.85		2.14
W	1.50 × 1.00 × 2.41	3.62		2.71
q (HA)	-½ × (43.51 + 31.49) × 1.50	- 56.25	0.790	- 44.44
Σ		49.78		39.59

$$\begin{cases} M = 39.59 \times 1/1.25 = 31.67 \ tm \\ S = 49.78 \times \text{〃} = 39.83 \ t \end{cases}$$

section B – B

CASE 1

		S	x	M
Ws	3.10 × 8.00 × 1.9	47.12	1.550	73.04
W	3.10 × 1.00 × 2.41	7.47	1.550	11.58
Ws add	3.10 × 1.02	3.16	1.550	4.90
q(HA)	½ × (2.09 + 23.00) × 3.10	− 38.89	1.119	− 43.52
Σ		18.86		46.00

$$\begin{cases} M = 46.00 \ ^{tm} \\ S = 18.86 \ ^{t} \end{cases}$$

CASE 2

		S	x	M
Ws	3.10 × 8.00 × 1.9	47.12		73.04
W	3.10 × 1.00 × 2.41	7.47		11.58
Ws add	3.10 × 1.66	5.15	1.550	7.98
q(HA)	− ½ × 24.28 × 3.03	− 36.78	1.010	− 37.15
Σ		22.96		55.45

$$\begin{cases} M = 55.45 \times 1/1.25 = 44.36 \ ^{tm} \\ S = 22.96 \times \quad = 18.37 \ ^{t} \end{cases}$$

7 - 3 list of stresses σc.σs.τ : working stress.
σca,σsa,τa: Permissible stress.

	A - A	B - B				
M	35.43	46.00				
N	——	——				
S	44.71	18.86				
b	100	100				
h	90	90				
d'	10	10				
As	D20 D16 > ctc 125 20.60	D25 D16 > ctc 125 25.12	As min =	b · d · 0.15 %	= 13.50 cm²	
As'	——	——				
f/d	0	0				
M'/bd²	4.37	5.68				
S/bd	4.97	2.10				
n·P	0.0343	0.0419				
C	8.96	8.27				
S	31.58	26.06				
Z	4.10	1.11				
σc	39	47				
σs	2072	2219				
τ	4.9	2.3				
σca	83	83				
σsa	2346	2346				
τa	3.5	3.5				

Check for stirrups

Sect A – A

$$\tau = \frac{S}{b \cdot d} \times Z = \frac{44.71 \times 10^3}{100 \times 90} \times 1.10 = 5.46 \ kg/cm^2 > \tau_a = 2.35 \ kg/cm^2$$

$$S' = S - S_c$$

$$S_c = \tau_a \cdot b \cdot d \cdot 1/Z = 2.35 \times 100 \times 90 \times 1/1.10 = 19.23 \times 10^3 \ kg$$

$$S' = (44.71 - 19.23) \times 10^3 = 25.48 \times 10^3 \ kg$$

$$Req\ A_v = \frac{S' \cdot a}{\sigma_{sa} \cdot d} \times Z = \frac{25.48 \times 10^3 \times 25}{1780 \times 90} \times 1.10 = 4.4 \ cm^2$$

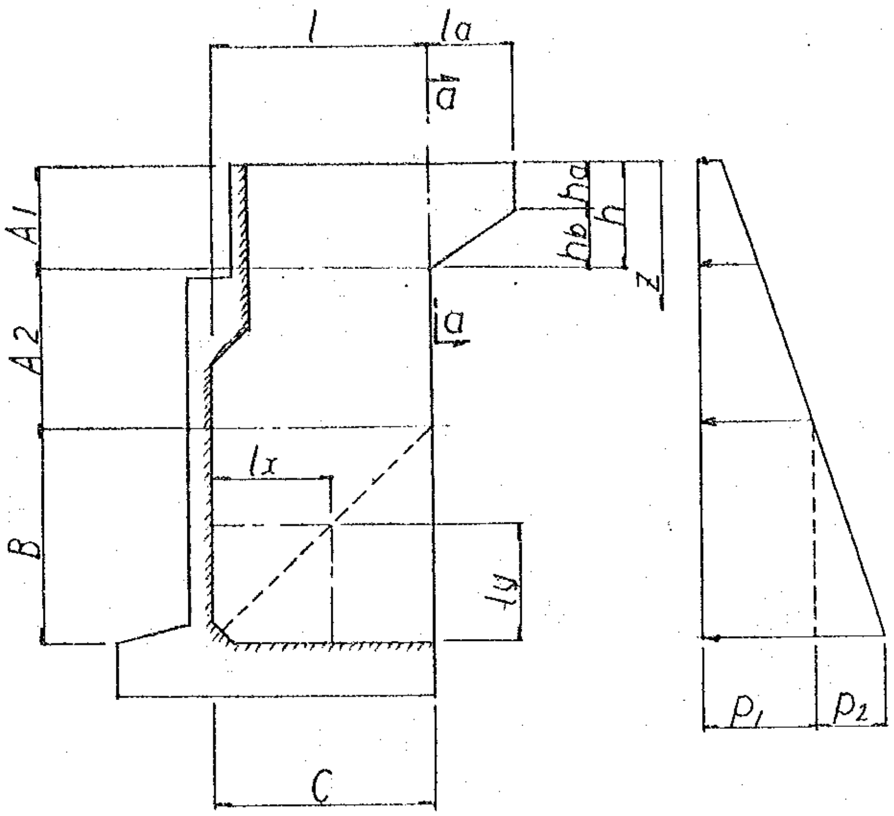
Φ 20 – ctc 250 $n = 2$

$$A_v = 3.14 \times 2 = 6.48 \ cm^2 > Req.\ A_v = 4.4 \ cm^2$$

Sect B – B

Φ 16 – ctc 500 $n = 2$.

§ 8 CALCULATION OF WING SECTION



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

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

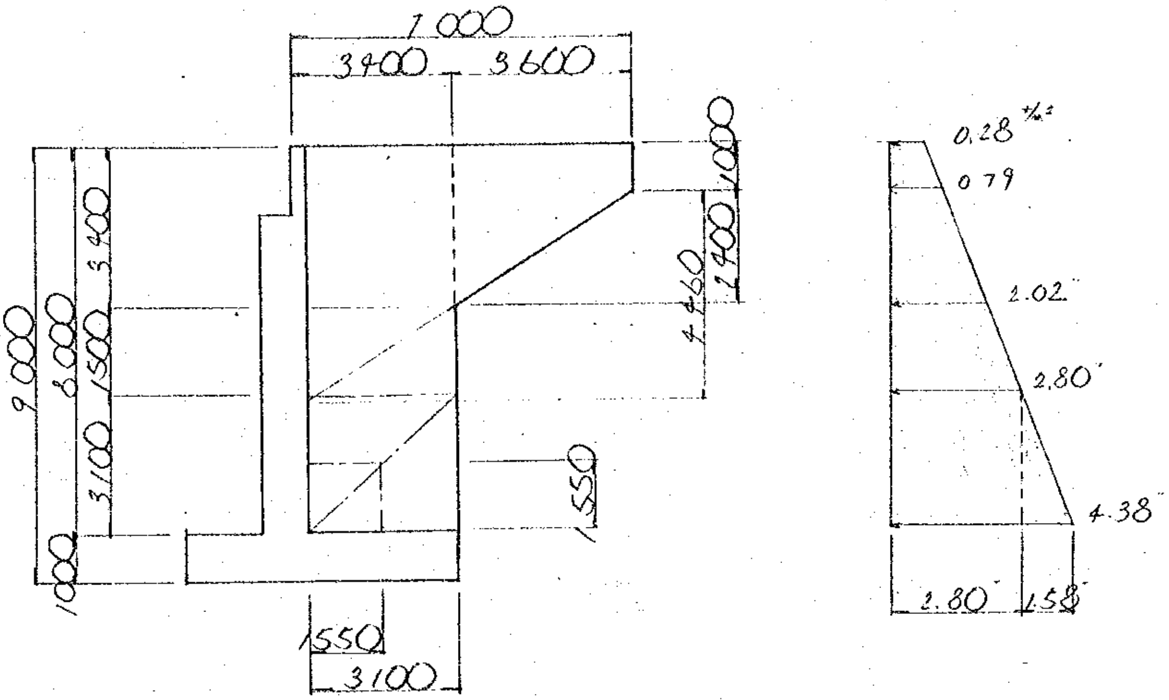
$$q = 1.02 \quad t/m^2$$

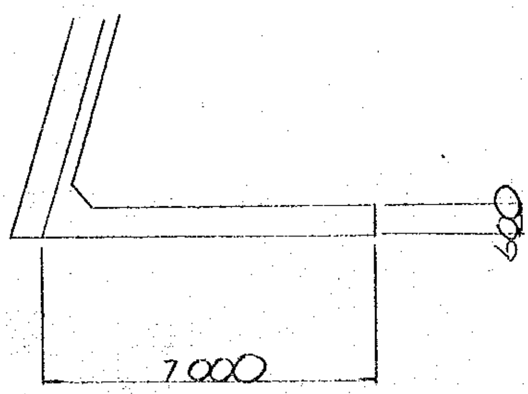
$$K = 0.27$$

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

	M (t)	S (tm)
A1 — 1	$\frac{1}{2} p \cdot l^2 + Ma + Sa \cdot l$	$p \cdot l + Sa$
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	$\left(\frac{p_1}{2} + \frac{p_2}{6}\right) ly^2$	$\left(p_1 + \frac{p_2}{2}\right) \cdot ly$

8 − 1 dimension and loading





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		$\bar{x}$ (m)	M (tm)		S (t)	
a – a	1 – 1	1.00	$(1.02+1.9\times1.00)\times0.27\times3.60^2\times\frac{1}{2}$	5.11	$(1.02+1.9\times1.00)\times0.27\times3.60$	2.84
	2 – 2	1.00 ~3.40	—	—	—	—
A 1		1.00 ~3.40	$(1.02+1.9\times1.46)\times0.27\times\frac{6.70^2}{2}\times\left(\frac{5.46-1.46}{4.46}\right)^2$	18.49	$(1.02+1.9\times1.46)\times0.27\times6.70\times\left(\frac{5.46-1.46}{4.46}\right)$	6.16
A 2	1 – 1	3.40	$\frac{1}{2}\times2.02\times3.10^2$	9.70	2.02×3.10	6.26
	2 – 2	4.90	$\frac{1}{2}\times2.80\times3.10^2$	13.45	2.80×3.10	8.68
B – B		4.90 ~8.00	$\frac{1}{2}\times\frac{2.80+4.38}{2}\times1.55^2$	4.31	$\frac{2.80+4.38}{2}\times1.55$	5.56
C – C		8.00	$\left(\frac{2.80}{2}+\frac{1.58}{6}\right)\times1.55^2$	4.00	$\left(2.80+\frac{1.58}{2}\right)\times1.55$	5.56

$$※\ \bar{x} = \frac{1.9\times3.40-2\times1.02}{3\times1.9} = 0.78 < 1.00^{m}$$

$$\bar{x}' = \frac{1.9\times5.46-2\times1.02}{3\times1.9} = 1.46^{m}$$

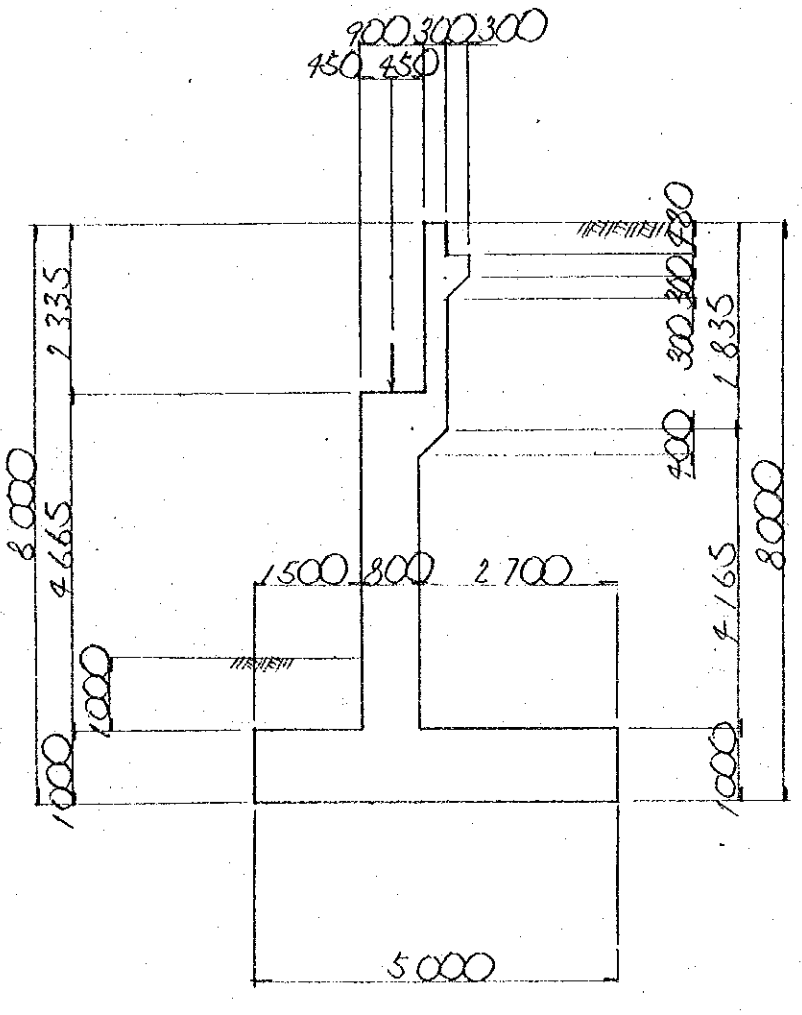
8 - 2 list of stresses   σc. σs. τ : working stress.
σca, σsa, τa : permissible stress.

	※As min = b·d·0.15% = 7.95 cm²					
	a-a	A1	A2 1-1	A2 2-2	B	C
M	5.11	18.49	9.70	13.45	4.31	4.00
N	———	———	———	———	———	———
S	2.89	6.16	6.26	8.68	5.56	5.56
b	100	——→				
h	53	——→				
d'	7	——→				
As	D16 @ 250 ※ 8.04	D16 @ 125 16.08	——→		D16 @ 250 ※ 8.04	——→ ※ 8.04
As'	———	———	———	———	———	———
f/d		0	0	0		
M'/bd²		6.21	3.45	4.79		
S/bd		1.24	1.18	1.64		
n·P		0.0455	0.0455	0.0455		
C		8.44	8.44	8.44		
S		24.08	24.08	24.08		
Z		1.09	1.09	1.09		
σc		52	29	40		
σs		2241	1247	1729		
τ		1.4	1.3	1.8		
σca	83	——→				
σsa	2346	——→				
τa	2.35	——→				

§§ 8 $H = 8.00^{m}$

E - RAMP A1

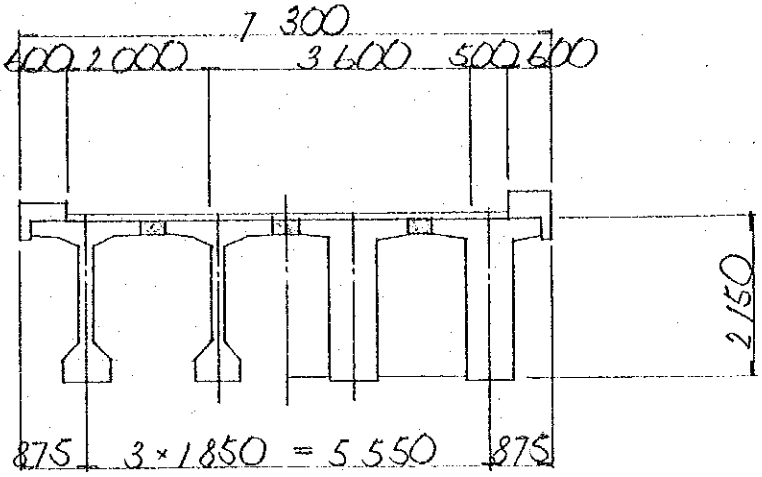
§ 1 STRUCTURAL FIGURE



$L = 7.300^{m}$

§ 2 REACTION OF SUPERSTRUCTURE

2-1 structural figure

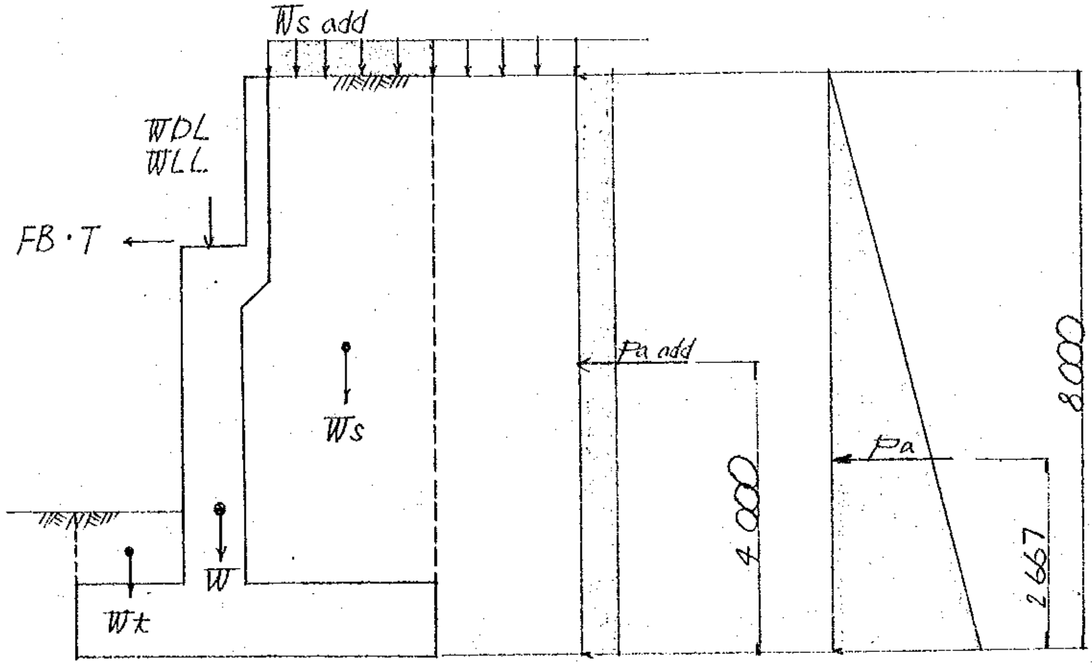


2-2 whole reaction of superstructure

		HA loading		HB loading	
		N (t)	H (t)	N (t)	H (t)
dead load of deck	I	194.7	——	194.7	——
	II	33.0	——	33.0	——
live load		119.7	——	130.6	——
crowd load		——	——	——	——
longitudinal forse		——	25.8	——	38.2
TOTAL		347.4	25.8	358.3	38.2

§ 3 CALCULATION OF LOAD

3-1 loading diagram



WDL : dead load of deck

WLL : max LL reaction under HA & HB

F B : HA & HB braking

W : self weight

Ws : weight of soil

Wt : fill on toe

Ws add : weight of surcharge

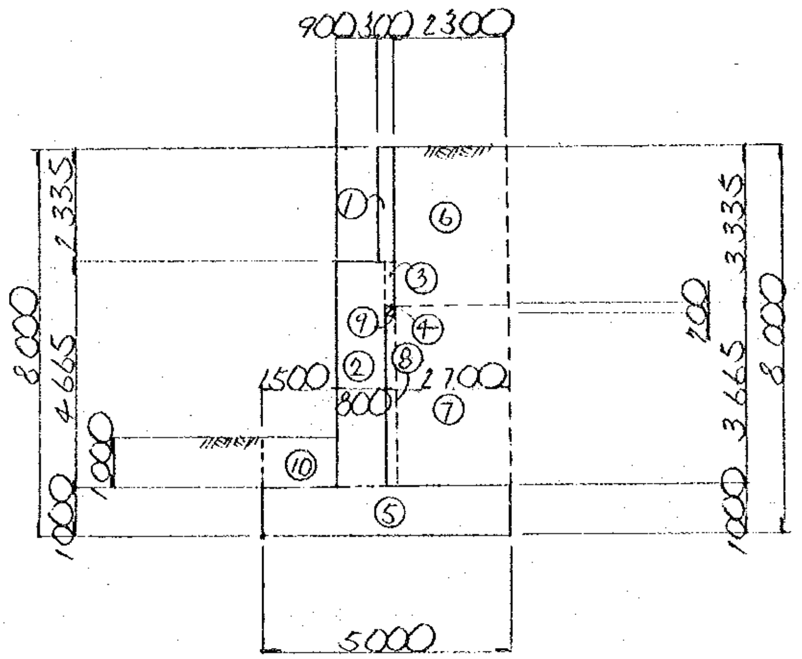
P_A : active pressure

P_P : passive pressure

P_A add : surcharge

T : temperature load

3 − 2 self weight and weight of soil



		N (t)	x (m)	N·x (tm)
①	0.30 × 2.335 × 7.30 × 2.41	12.32	2.550	31.42
②	1.00 × 4.665 × 7.30 × 2.41	82.07	2.000	164.14
③	0.20 × 1.00 × 7.30 × 2.41	3.52	2.600	9.15
④	1/2 × 0.20 × 0.20 × 7.30 × 2.41	0.35	2.567	0.90
⑤	5.00 × 1.00 × 7.30 × 2.41	87.97	2.500	219.93
⑥	2.30 × 3.335 × 7.30 × 1.9	106.39	3.850	409.60
⑦	2.30 × 3.665 × 7.30 × 1.9	116.92	3.850	450.14
⑧	0.20 × 3.465 × 7.30 × 1.9	9.61	2.600	24.99
⑨	1/2 × 0.20 × 0.20 × 7.30 × 1.9	0.28	2.633	0.74
⑩	1.50 × 1.00 × 7.30 × 1.9	20.81	0.750	15.61
Σ		440.24		1,326.62

3-3 weight of surcharge

under H.A $= 1.02 \times 2.30 \times 7.30 = 17.13^{t}$

under H.B $= 1.66 \times 2.30 \times 7.30 = 27.87^{t}$

3-4 earth pressure

unit weight of soil $\gamma_s = 1.9 \ ^{t}/_{m^3}$

angle of internal friction $\phi = 35°$

(1) active pressure

$$Pa = \frac{1}{2} \cdot K \cdot \gamma_s \cdot H^2 \cdot L$$

$$= 1/2 \times \frac{1 - \sin 35°}{1 + \sin 35°} \times 1.9 \times 8.00^2 \times 7.30 = 119.84^{t}$$

(2) active pressure due to surcharge

under H.A surcharge

$q = 1.02 \ ^{t}/_{m^2}$

$Pa\ add = K \cdot q \cdot H \cdot L$

$= 0.27 \times 1.02 \times 8.00 \times 7.30 = 16.08^{t}$

under H B surcharge

$q = 1.66 \ ^{t}/_{m^2}$

$Pa\ add = K \cdot q \cdot H \cdot L$

$= 0.27 \times 1.66 \times 8.00 \times 7.30 = 26.17^{t}$

3-5 temperature load

$$P_H = \frac{G_o \cdot A \cdot \delta}{\Sigma \cdot te}$$

$$\delta = \alpha \cdot l \qquad \alpha = \begin{cases} P.C \longrightarrow 0.7 \\ R.C \longrightarrow 0.5 \end{cases}$$

$$\left[\begin{array}{ll} \delta = 0.7 \times 33.20 & = 23.24^{mm} \\ R(d+l) = 358.30 \times 1/5 \times 1.4 & = 125.41^{t} \end{array} \right]$$

RING SHOE

130 TON		
$D\phi$	:	58^{cm}
$d\phi$	:	34^{cm}
A	:	2642^{cm^2}
t	:	7.5^{cm}
G_o	:	$13.5^{kg/cm^2}$ (modulus of rigidity)

$$P_H = \frac{13.5 \times 2642 \times 2.32}{7.5} = 11032^{kg} = 11.03^{t}$$

$$\Sigma P_H = n \cdot P_H \cdot 1/2$$

$$= 4 \times 11.03 \times 1/2 = 22.06^{t}$$

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§ 4 CALCULATION OF STABILITY

case 1 HA loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL, WLL	347.40	1.950	677.43	—	—	—
F B	—	—	—	25.80	5.665	146.16
T	—	—	—	22.06	5.665	124.97
W. Ws. W_T	440.24	—	1326.62	—	—	—
Ws add	17.13	3.850	65.95	—	—	—
P a	—	—	—	119.84	2.667	319.57
Pa add	—	—	—	16.08	4.000	64.32
TOTAL	804.77		2070.00	183.78		655.02

1) check for eccentric

$$x = \frac{\Sigma Nx - \Sigma HY}{\Sigma N} = \frac{2070.00 - 655.02}{804.77} = 1.75 \text{ m}$$

$$e = \frac{B}{2} - x = \frac{5.00}{2} - 1.75 = 0.75 \text{ m} < \frac{B}{6} = 0.83 \text{ m}$$

2) soil reaction

$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{804.77}{5.00 \times 7.30} \times \left(1 \pm \frac{6 \times 0.75}{5.00}\right)$$

$$= \begin{cases} 41.89 \ t/m^2 \\ 2.20 \end{cases} < 60 \ t/m^2$$

3) check for sliding

$$H_u = C \cdot A' + N \cdot \tan\phi' \qquad c = 0 \qquad \tan\phi' = 0.6$$

$$F = \frac{H_u}{\Sigma H} = \frac{804.77 \times 0.6}{183.78} = 2.62 > Fa = 1.5$$

case 2 HB loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL. WLL	358.30	1.950	698.69	——	——	——
F B	——	——	——	38.20	5.665	216.40
T	——	——	——	22.06	5.665	124.97
W. Ws. Wr	440.24	——	1326.62	——	——	——
Ws add	27.87	3.850	107.30	——	——	——
P a	——	——	——	119.84	2.667	319.57
Pa add	——	——	——	26.17	4.000	104.68
TOTAL	826.41		2132.61	206.27		765.62

1) check for eccentric

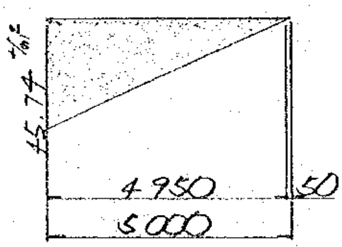
$$x = \frac{\Sigma N\cdot x + \Sigma H\cdot y}{\Sigma N} = \frac{2132.61 - 765.62}{826.41} = 1.65\ ^{m}$$

$$e = \frac{B}{2} - x = \frac{5.00}{2} - 1.65 = 0.85\ ^{m} < \frac{B}{3} = 1.66\ ^{m}$$

2) soil reaction

$$q = \frac{2\cdot N}{3\cdot X\cdot L}$$

$$= \frac{2\times 826.41}{3\times 1.65\times 7.30} = 45.74\ t/m^2$$



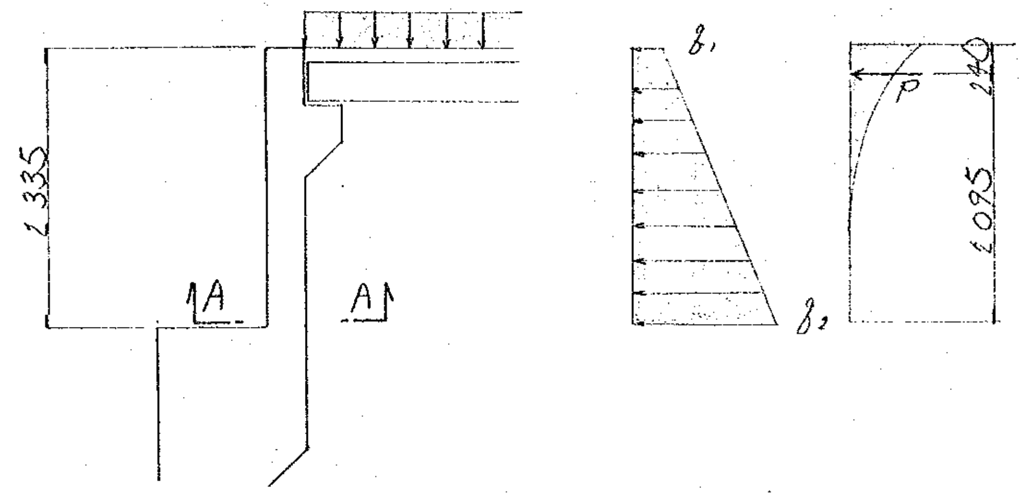
3) check for sliding

$$Hu = c\cdot A' + N\cdot \tan\phi' \qquad c = 0 \qquad \tan\phi' = 0\ 6$$

$$F = \frac{Hu}{\Sigma H} = \frac{0.6\times}{\quad} = \qquad > Fa = 1.2$$

§ 5 CALCULATION OF PARAPET SECTION

5-1 dimenston and loading



	q_1	q_2
HA loading	0.28	1.49
HB loading	0.45	1.66

$$q_1 = q \cdot K \qquad = 0.27 \cdot q \quad t/m^2$$

$$q_2 = K \cdot \gamma_s \cdot H + q_1 = 0.513 \cdot H + 0.27 \cdot q \quad "$$

5 - 2 sectional force of parapet

CASE 1 (HA)

$$S = 1/2 \times (0.28 + 1.49) \times 2.335 = 2.07^{\,t}$$

$$M = 2.07 \times 1/3 \times 2.335 \times \frac{2 \times 0.28 + 1.49}{0.28 + 1.49} = 1.87^{\,tm}$$

CASE 2 (HB)

$$S = 1/2 \times (0.45 + 1.66) \times 2.335 = 2.46^{\,t}$$

$$M = 2.46 \times 1/3 \times 2.335 \times \frac{2 \times 0.45 + 1.66}{0.45 + 1.66} = 2.32^{\,tm}$$

CASE 3

$$S = 10.97 \times 0.27 = 2.96^{\,t}$$

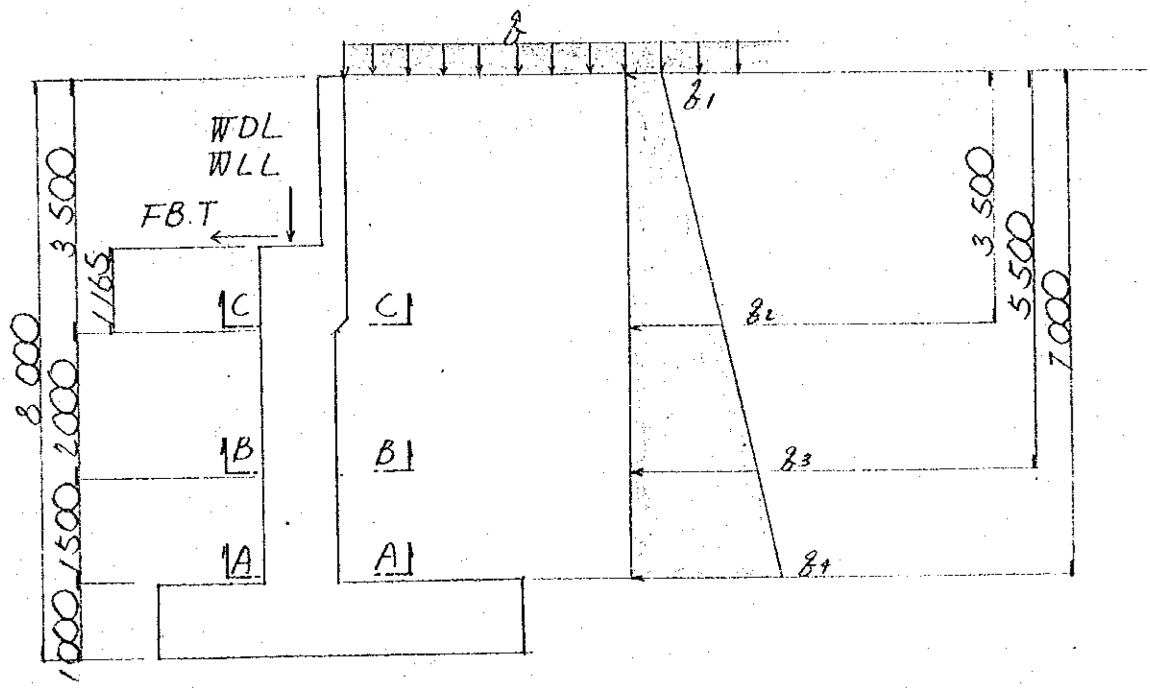
$$M = 2.96 \times 2.095 = 6.20^{\,tm}$$

5 - 3 list of stresses σ_c, σ_s, τ : working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

	case 1	case 2				
M	1.87	2.32				
N	——	——				
S	2.07	2.46				
b	100	100				
h	23	23				
d'	7	7				
As	D16@125 16.08	→				
As'	——	——				
f/d						
M'/bd^2						
S/bd						
n·P						
C						
S						
Z						
σ_c						
σ_s						
τ						
σ_{ca}	83	103				
σ_{sa}	2346	2932				
τ_a	2.35	3.94				

§ 6 CALCULATION OF WALL SECTION

6-1 dimension and loading



	N (t)	H (t)
dead load of deck	31.19	—
HA live load	16.40	6.55
HB live load	17.89	8.25

	q1	q2	q3	q4
HA loading	0.28	2.08	3.10	3.87
HB loading	0.45	1.25	3.27	4.04

6 - 2 sectional force of wall

section 1 - 1

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	31.19	6.55	1.165	7.63
self weight	5.03	——	——	——
earth pressure	——	4.13	0.783	3.23
TOTAL	36.22	10.68		10.86

section 2 - 2

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	31.19	6.55	3.165	20.73
self weight	9.85	——	——	——
earth pressure	——	9.38	1.985	18.62
TOTAL	41.04	15.93		39.35

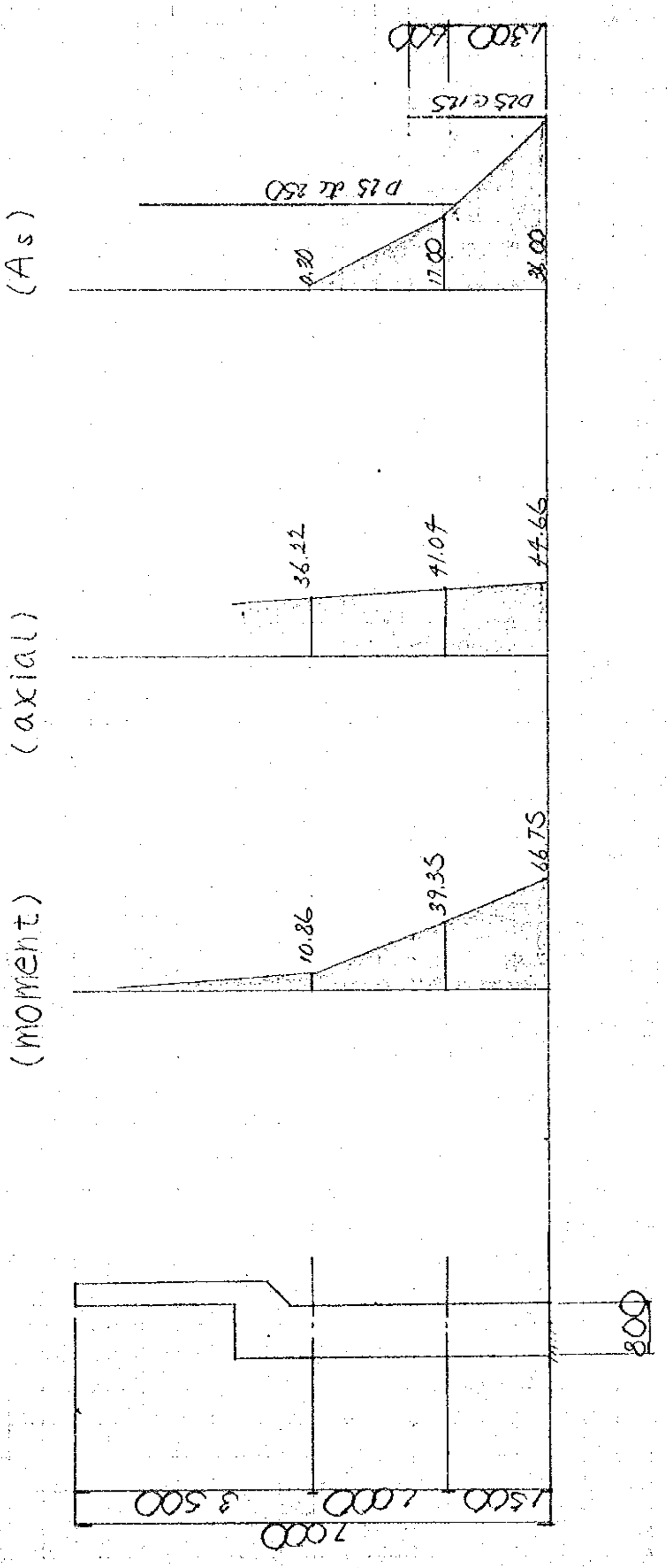
section 3 - 3

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	31.19	6.55	4.665	30.56
self weight	13.47	——	——	——
earth pressure	——	14.53	2.491	36.19
TOTAL	44.66	21.08		66.75

6 − 3 list of stresses σ_c, σ_s, τ : working stress.
σ_{ca}, σ_{sa}, τ_a : Permissible stress.

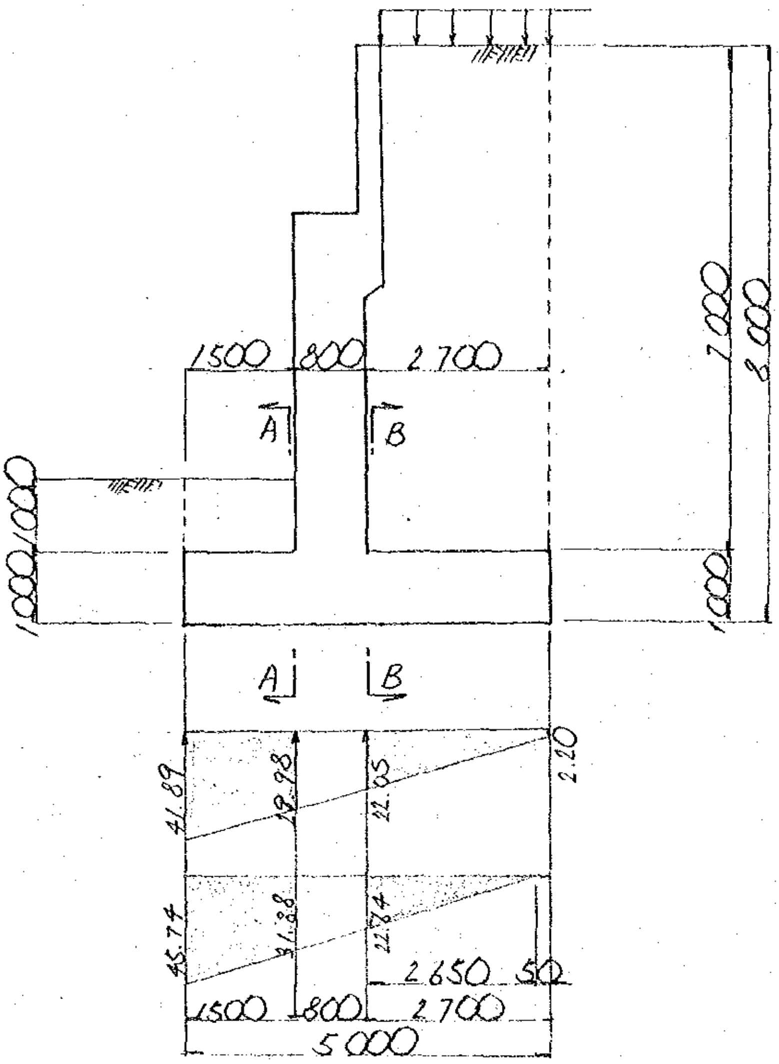
※ $As_{min} = 100 \times 73 \times 0.0015 = 10.95\ cm^2$

	A − A	B − B	C − C			
M	66.75	39.35	10.86			
N	44.66	41.04	36.22			
S	21.08	15.93	10.68			
b	100	→	→			
h	73	→	→			
d'	7	→	→			
As	D25 C 125 39.28	D25 C 250 19.64	→			
As'	—	—	—			
f/d	2.50	1.77				
M'/bd^2	15.29	9.93				
S/bd	2.89	2.18				
n·P	0.0807	0.0404				
C	5.61	6.80				
S	9.17	13.82				
Z	1.12	1.07				
σ_c	82	68				
σ_s	2102	2056				
τ	3.2	2.3				
σ_{ca}	83	→	→			
σ_{sa}	2346	→	→			
τ_a	3.47	2.35	→			


(As)
0.30
17.00
34.00
D25 ctc 250
D25 @125
1300
400
(axial)
36.22
41.07
44.66
(moment)
10.86
39.35
66.75
800
1500
1000
3500
7000

§ 7 CALCULATION OF FOOTING SECTION

7 - 1 dimenston and loading



	q 1	q 2	q 3	q 4
HA loading	41.89	29.98	22.05	2.20
HB. loading	45.74	31.88	22.64	0

7 - 2 sectional force of footing

section A - A

CASE 1

		S	x	M
Wt	1.50 × 1.00 × 1.9	2.85	0.750	2.14
W	1.50 × 1.00 × 2.41	3.62	0.750	2.72
q (HA)	1/2 × (41.89 + 29.98) × 1.50	- 53.90	0.791	- 42.63
Σ		47.43		37.77

$$\begin{cases} M = 37.77 \ ^{tm} \\ S = 47.43 \ ^{t} \end{cases}$$

CASE 2

		S	x	M
Wt	——	2.85		2.14
W	——	3.62		2.72
q (HA)	-1/2 × (45.74 + 31.88) × 1.50	- 58.22	0.795	- 46.28
Σ		51.75		41.42

$$\begin{cases} M = 41.42 \times 1/1.25 = 33.14 \ ^{tm} \\ S = 51.75 \times \ '' = 41.40 \ ^{t} \end{cases}$$

section B - B

CASE 1

		S	x	M
Ws	2.50 × 7.00 × 1.9	33.25	1.250	41.56
W	2.50 × 1.00 × 2.41	6.03	1.250	7.53
Ws add	2.50 × 1.02	2.55	1.250	3.19
q (HA)	-½ × (22.05 + 2.20) × 2.50	- 30.31	0.909	- 27.55
Σ		11.52		24.73

$$\begin{cases} M = 24.73 \ \ \text{tm} \\ S = 11.52 \ \ \text{t} \end{cases}$$

CASE 2

		S	x	M
Ws	——	33.25	——	41.56
W	——	6.03	——	7.53
Ws add	2.50 × 1.66	4.15	1.250	5.19
q (HA)	-½ × 2.45 × 22.64	- 27.73	0.816	- 22.63
Σ		15.70		31.65

$$\begin{cases} M = 31.65 \times 1/1.25 = 25.32 \ \ \text{tm} \\ S = 15.70 \times \quad = 12.56 \ \ \text{t} \end{cases}$$

7-3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: Permissible stress.

	A - A	B - B				
M	37.77	25.32				
N	———	———				
S	47.43	12.56				
b	100	100				
h	90	90				
d′	10	10				
As	D16, D20 ctc 125 20.60	D16 ctc 125 16.08	As min =	b·d·0.15%	= 13.50 cm^2	
As′	———	———				
f/d	0	0				
M'/bd^2	4.66	3.13				
S/bd	5.27	1.40				
n·P	0.0343	0.0268				
C	9.42	10.41				
S	31.56	40.10				
Z	1.08	1.07				
σ_c	44	33				
σ_s	2208	1880				
τ	5.3	1.5				
σ_{ca}	83	83				
σ_{sa}	2346	2346				
τ_a	3.5	3.5				

Check for stirrups.

Sect A - A

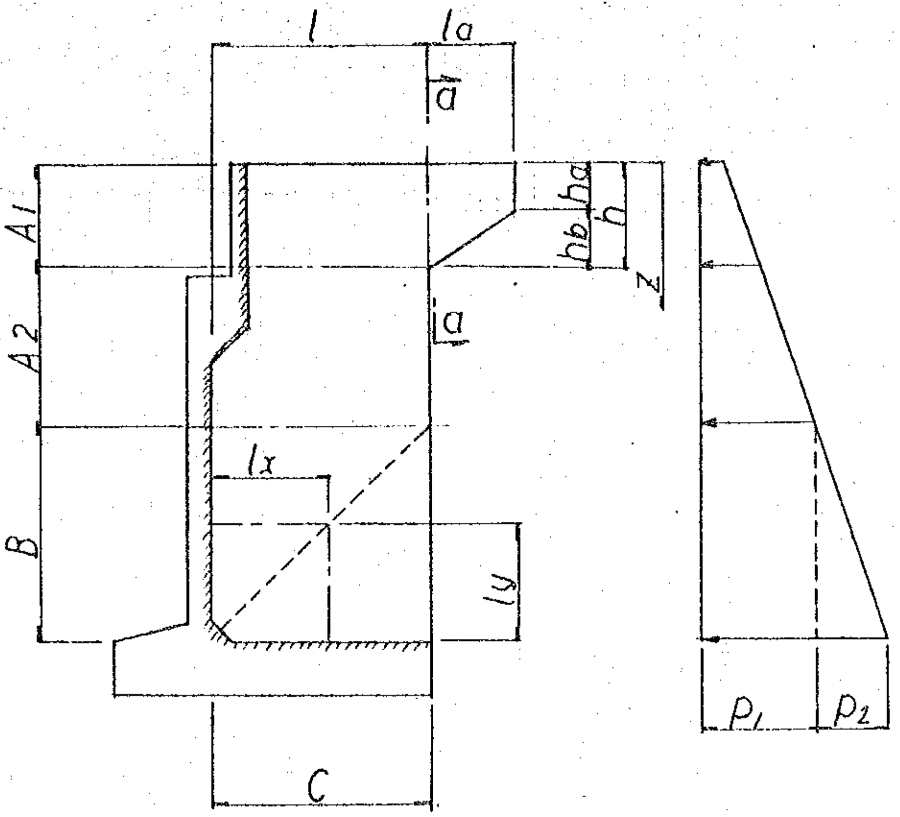
$$\tau = \frac{S}{b \cdot d} Z = \frac{47.43 \times 10^3}{100 \times 90} \times 1.08 = 5.69 \ kg/cm^2$$

Φ 20 - ctc 250 $n = 2$

Sect B - B

Φ 16 - ctc 500 $n = 2$

§ 8 CALCULATION OF WING SECTION



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

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

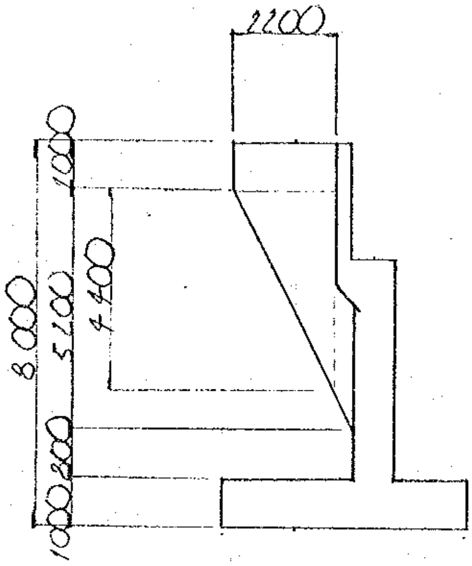
$q = 1.02$ t/m²

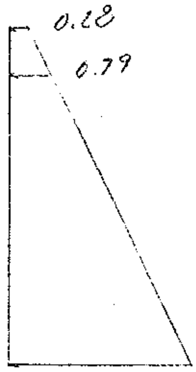
$K = 0.27$

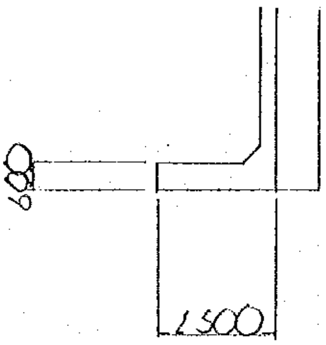
$\gamma = 1.9$ t/m³

	M (t)	S (tm)
A1 — 1	$\frac{1}{2} p \cdot l^2 + Ma + Sa \cdot l$	$p \cdot l + Sa$
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	$\left(\frac{p_1}{2} + \frac{p_2}{6}\right) ly^2$	$\left(p_1 + \frac{p_2}{2}\right) \cdot ly$

8 − 1 dimension and loading







		z (m)	M (tm)		S (t)	
a－a	1－1	1.00	$\frac{1}{2} \times 0.79 \times 2.20^2$	1.91	0.79×2.20	1.74
	2－2	1.00 ~6.20	$(1.02 + 1.9 \times 1.44) \times 0.27 \times \frac{2.20^2}{2} \times \left(\frac{5.40 - 1.44}{4.40}\right)^2$	1.99	$(1.02 + 1.9 \times 1.44) \times 0.27 \times 2.20 \times \left(\frac{5.40 - 1.44}{4.40}\right)$	2.01
A 1		—	—	—	—	—
A 2	1－1	—	—	—	—	—
	2－2	—	—	—	—	—
B－B		—	—	—	—	—
C－C		—	—	—	—	—

$$z = \frac{1.9 \times 5.40 - 2 \times 1.02}{3 \times 1.9} = 1.44^{m}$$

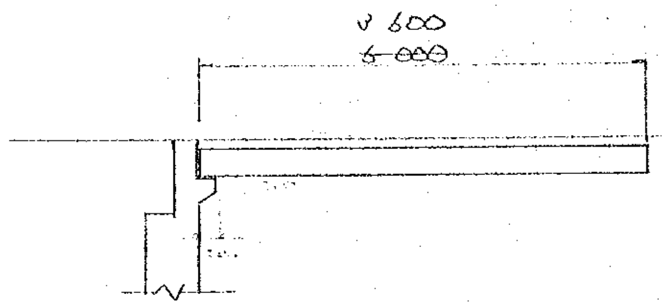
8 – 2 list of stresses σ_c, σ_s, τ : working stress.

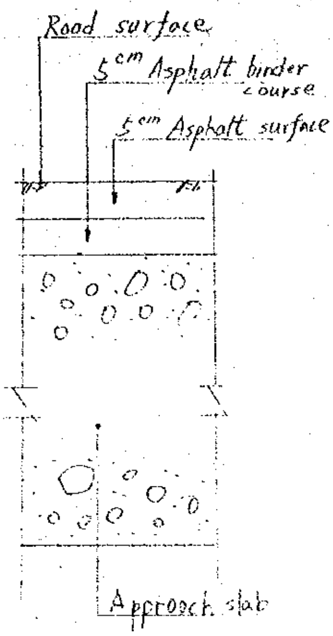
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

	※ $A_{s\,min} = 100 \times 53 \times 0.0015 = 7.95$ cm^2					
	a – a					
M	1.99					
N	——					
S	2.01					
b	100					
h	53					
d′	7					
As	D16@250 ※ 8.04					
As′	——					
f/d						
M'/bd^2						
S/bd						
n·P						
C						
S						
Z						
σ_c						
σ_s						
τ						
σ_{ca}	83					
σ_{sa}	2346					
τ_a	1.35					

§§ 9 Approach slab

Design of Approach slab.



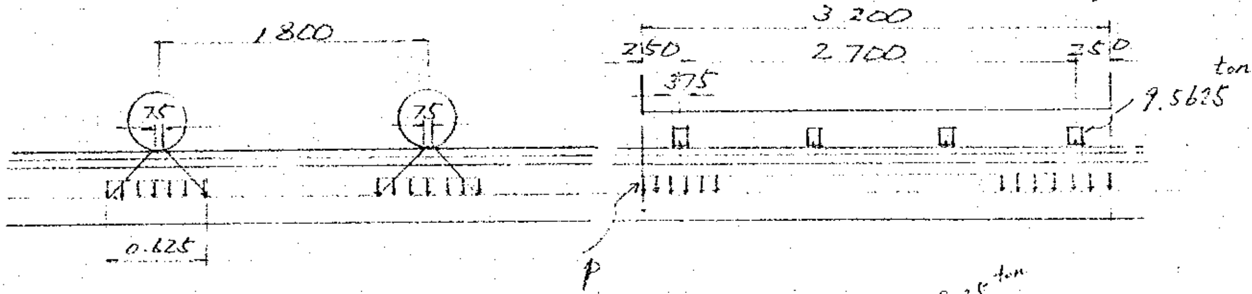


1) Dead load

pavement $\gamma_p = 2.3 \ t/m^3$

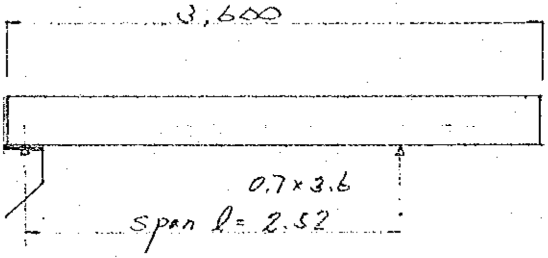
Reinforced concrete $\gamma_c = 2.41 \ t/m^3$

2) Live load H·B 37.5 unit



$$\frac{4 \times 9.5625}{3.2 \times (2D + 0.075)} = \frac{4 \times 9.5625 \ (= 38.25 \ ton)}{3.2 \times (2 \times 0.275 + 0.075)} = 19.125 \ t/m^2$$

$$D = 0.10 + \frac{1}{2} \times (0.35) = 0.275$$



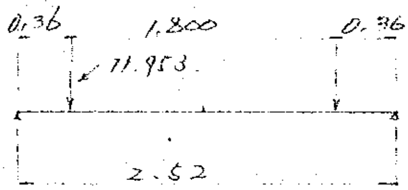
3) Calculation of load

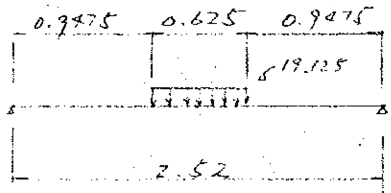
1) Dead load $0.1^{m} \times 2.3 = 0.23\ t/m$

2) self weight $0.35 \times 2.41 = 0.84\ t/m$

$\quad 1.07$

3) live load — or





4) Calculation of Sectional force

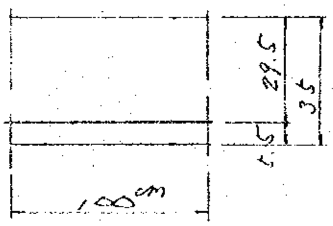
1) $M_{max} = 1/8 \times 1.07 \times 2.52^2 = 0.85\ t{\cdot}m$

2) $M_{max} = 11.953 \times 0.36 = 4.30\ t{\cdot}m$

or $M_{max} = \frac{1}{2} \times 11.953 \times 1.26 - \frac{1}{2} \times 11.953 \times \frac{1}{4} \times 0.625 = 6.60\ t{\cdot}m$

5) Section

$M_{max} = 6.60 + 0.85 = 7.45\ t{\cdot}m$



$req\ A_s = \frac{M}{\sigma_{sa} \cdot j \cdot d} = \frac{7.45 \times 10^5}{2300 \times 1.25 \times 0.875 \times 29.5}$

$= 10.04\ cm^2$

$\Phi 16^{mm} - 150^{mm}$ c-to-c $= 13.4\ cm^2$

$np = \frac{13.4 \times 15}{100 \times 29.5} = 0.068 \rightarrow c = 7.26,\ s = 16.3$

$\sigma_c = 7.26 \times 7.45 \times 10^5 / 100 \times 29.5^2\ (8.56) = 62\ kg/cm^2 < \sigma_{ca}$ OK

$\sigma_s = 16.3 \times 8.56 \times 15 = 2093\ kg/cm^2 < \sigma_{sa}$

6. Ramp Way Bridge (Piers and Abutment)

Contents

CALCULATION SHEETS
FOR SUBSTRUCTURE

M J B-RAMP BR.

Rigid frame pier

Vol. 1

M.J B-RAMP BR. P_1 Pier

(P -   )

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = 7.2 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

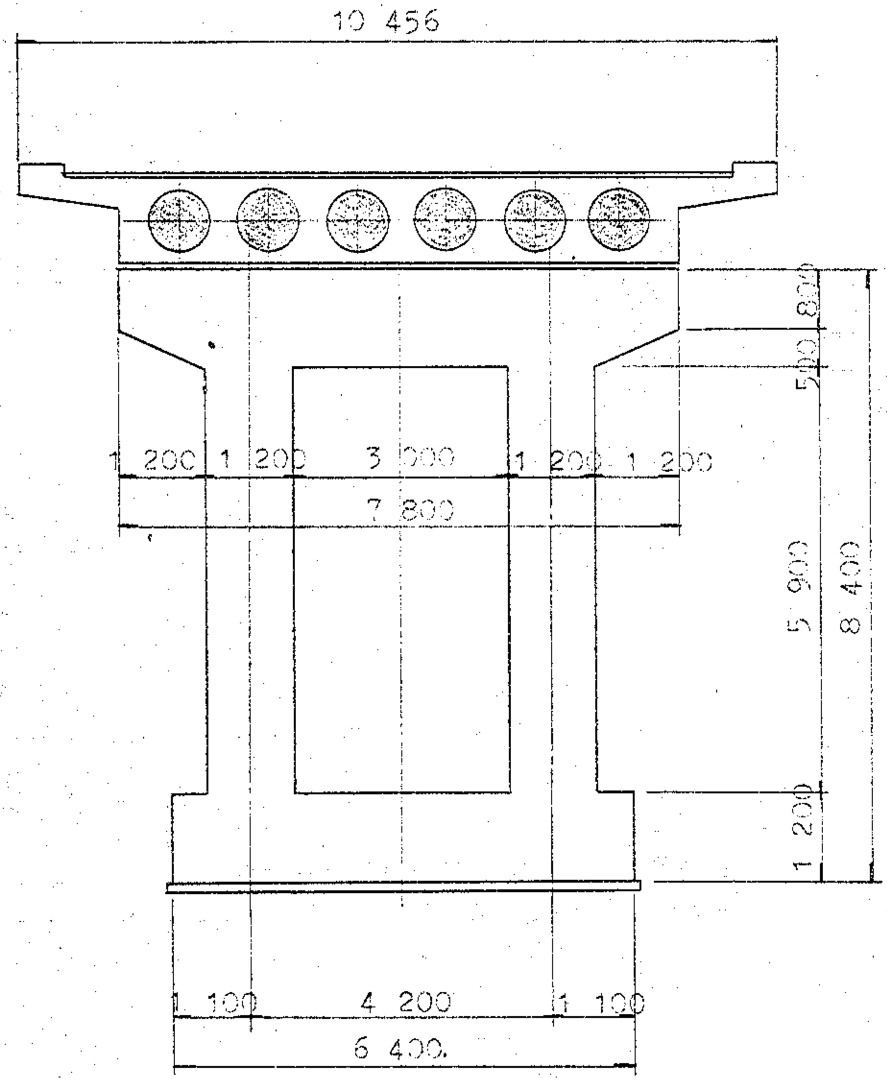
su = 4 180 kg/cm^2

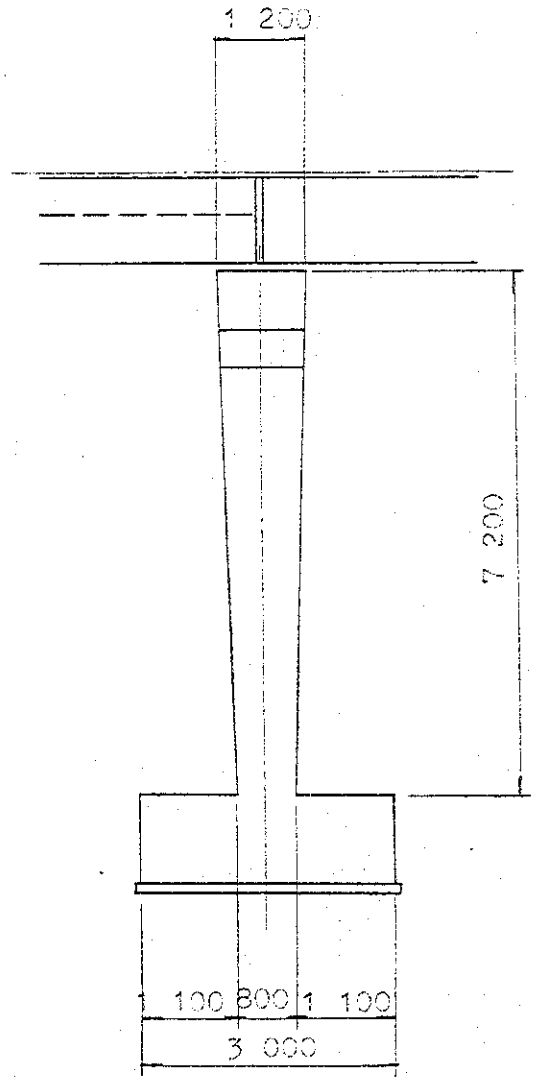
permissible tensile stress

sa = 2 340 kg/cm^2

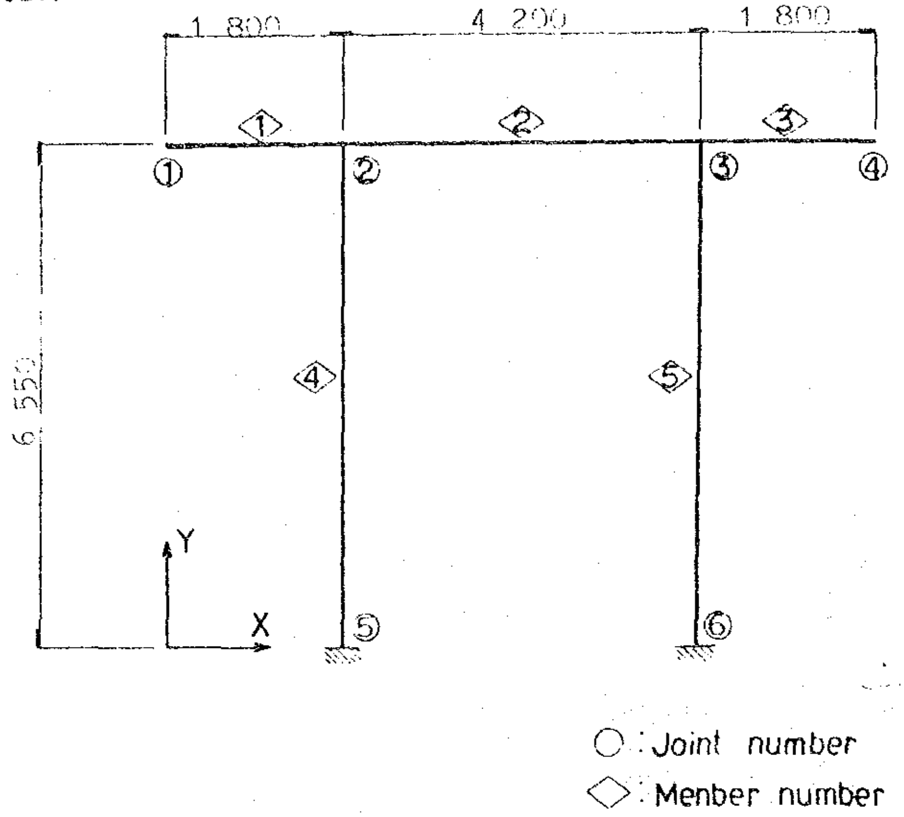
modular ratio n = 15

§ 2 GENERAL DIMENSION





Skeleton



Coordinates

(m)

	X	Y		X	Y
①	0.0	6.55	④	7.80	6.55
②	1.80	6.55	⑤	1.80	0.0
③	6.00	6.55	⑥	6.00	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 × 1.05	1.22	0.1119
②	1.16 × 1.30	1.51	0.2124
③	1.16 × 1.05	1.22	0.1119
④	0.98 × 1.20	1.18	0.1411
⑤	0.98 × 1.20	1.18	0.1411

§ 3 CALCULATION OF LOADS

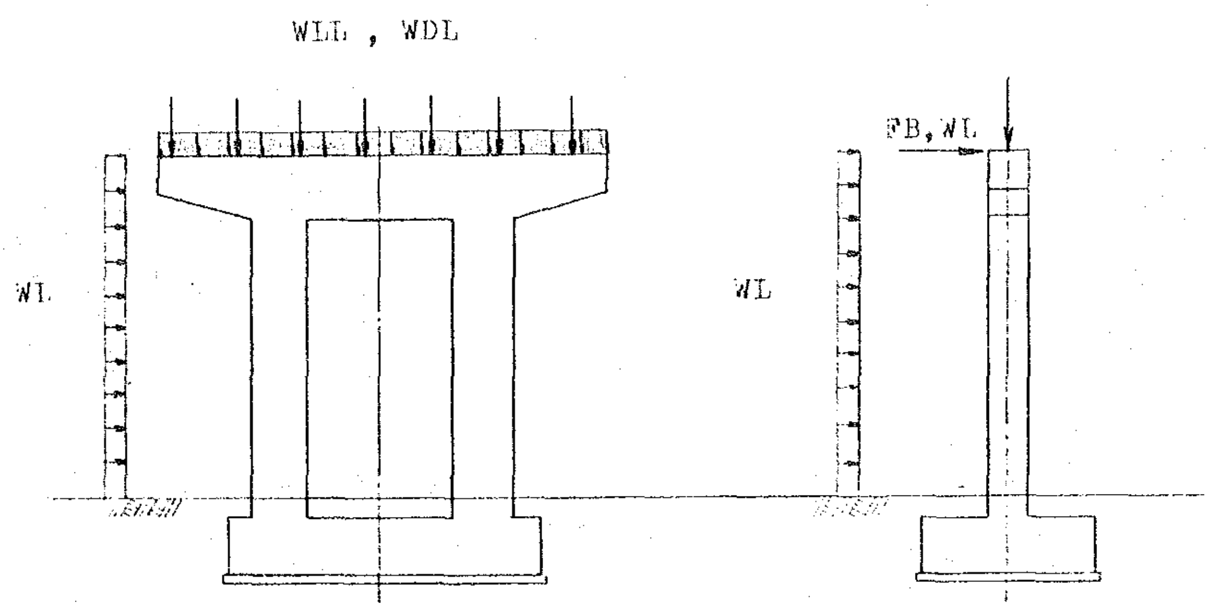
1.) Loading case and increase in basic stresses

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	temperature change and drying shrinkage	FT & FS
8	wind	WL

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

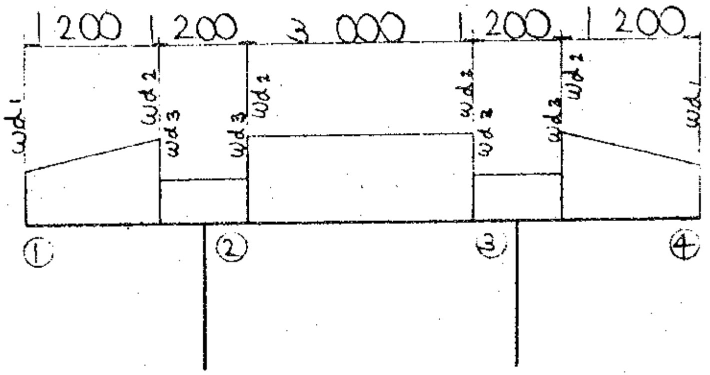
6

2.) Loading diagram



3) Dead load

(a) beam



wd1 = 1.16 × 0.80 × 2.407 = 2.23 t/m

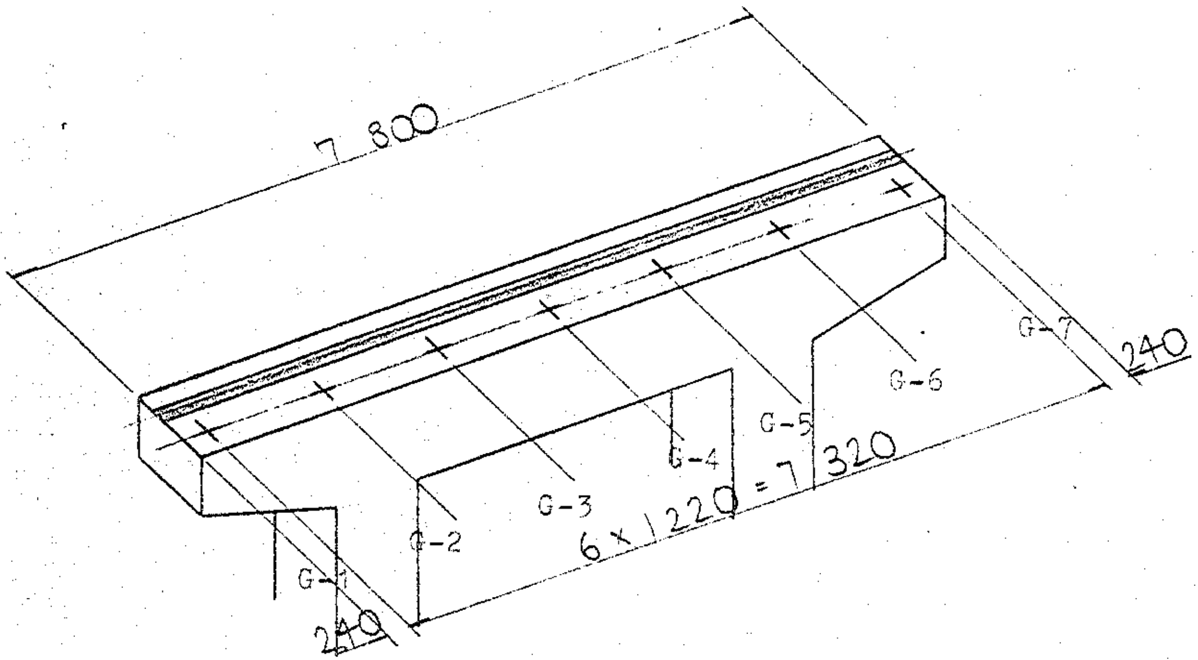
wd2 = 1.16 × 1.30 × 2.407 = 3.63 t/m

wd3 = 1.16 × 0.65 × 2.407 = 1.81 t/m

(b) column

pw = 0.98 × 1.20 × 2.407 = 2.83 t/m

4) Reaction



		G-1	G-2	G-3	G-4	G-5	G-6	G-7		
Dead load		18.5	13.5	15.8	14.2	16.3	10.4	25.2		
		5.87 t/m								
HA load		33.4	20.9	22.0	19.9	22.2	21.9	52.9		
		3.08 t/m								
HB load		50.3	21.7	26.0	26.3	29.1	16.6	82.9		
		0.44 t/m								

5.) Temperature change and Drying shrinkage

temperature change -12 +20 deg.

drying shrinkage -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

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9$ kg/m²

A = 1.38 × 11.2 = 15.5 m²

Cd = 1.2

Pt = 219.9 × 15.5 × 1.2 = 4090 kg

= 4.1 ton

Live loaded

Pt = q·A·Cd

$q = 0.613 \times 35^2 \times 0.102 = 76.6$ kg/m²

A = 3.73 × 11.2 = 41.8 m²

Cd = 1.40

Pt = 76.6 × 41.8 × 1.40 = 4483 kg

= 4.5 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/m²

$A = (0.150 \times 11.2 + 0.850 \times 0.05 \times \frac{11.2}{2}) \times 2 = 3.84$ m²

Cd = 1.1

Pt = 219.9 × 3.84 × 1.1 = 929 kg

= 1.0 ton

(c) For Pier

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

Unloaded

$q = 219.9$ kg/m²

$A1 = 0.98 \times 6.70 = 6.57$ m²

$A2 = 0.98 \times 5.40 = 5.29$ m²

$Cd = 2.0$

$Pt = 219.9 \times (6.57 + 5.29) \times 2.0 = 5216$ kg

$= 5.2$ ton

Live loaded

$q = 0.613 \times 35^2 \times 0.102 = 76.6$ kg/m²

$A = A1 + A2 = 11.86$ m²

$Cd = 2.0$

$Pt = 76.6 \times 11.86 \times 2.0 = 1817$ kg $= 1.8$ ton

(d) Table of Wind load (transverse)

(ton)

	unloaded	live loaded
super-structure	4.1	4.5
safety fences	1.0	——
Pier	5.2	1.8

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 × 4.1

= 1.0 ton

live loaded PLL = 0.5 Pt

Pt = 76.6 × (2.50 × 11.2) × 1.40

= 3.0 ton

PLL = 0.5 × 3.0 = 1.5 ton

PLS = 0.25 Pt

Pt = 76.6 × (3.73 − 2.5) × 11.2 × 1.2

= 1.3 ton

PLS = 0.25 × 1.3 = 0.3 ton

for safety fences

vertical member

PL = 0.8 Pt

Pt = $219.9 \times (0.85 \times 0.05 \times \frac{11.2}{2} \times 2) \times 1.1$

= 0.12 ton

PL = 0.8 × 0.12 = 0.1 ton

longitudinal member

PL = 0.4 Pt

Pt = 219.9 × (0.15 × 11.2 × 2) × 1.1

= 0.8 ton

PL = 0.4 × 0.8 = 0.3 ton

for Pier

Vc = 59.3m/sec.

unloaded

$q = 219.9\ kg/m^2$

$A1 = 7.80 \times 1.30 - 1.20 \times 0.50 = 9.5\ m^2$

$A2 = 1.20 \times 5.40 \times 2 = 13.0\ m^2$

$Cd1 = 2.1$

$Cd2 = 2.2$

$Pt1 = 219.9 \times 9.5 \times 2.1 = 4.4\ ton$

$Pt2 = 219.9 \times 13.0 \times 2.2 = 6.3\ ton$

live loaded

$q = 76.6\ kg/m^2$

$A1 = 9.5\ m^2$

$A2 = 13.0\ m^2$

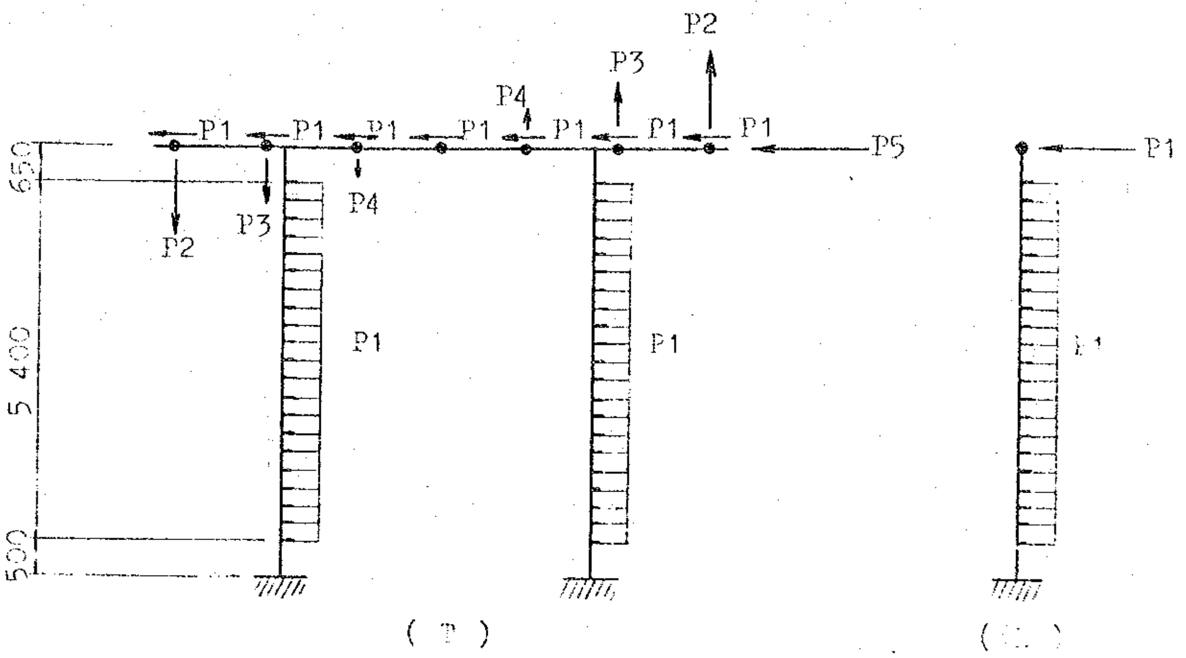
$Cd1 = 2.1$

$Cd2 = 2.2$

$Pt1 = 76.6 \times 9.5 \times 2.1 = 1.5\ ton$

$Pt2 = 76.6 \times 13.0 \times 2.2 = 2.2\ ton$

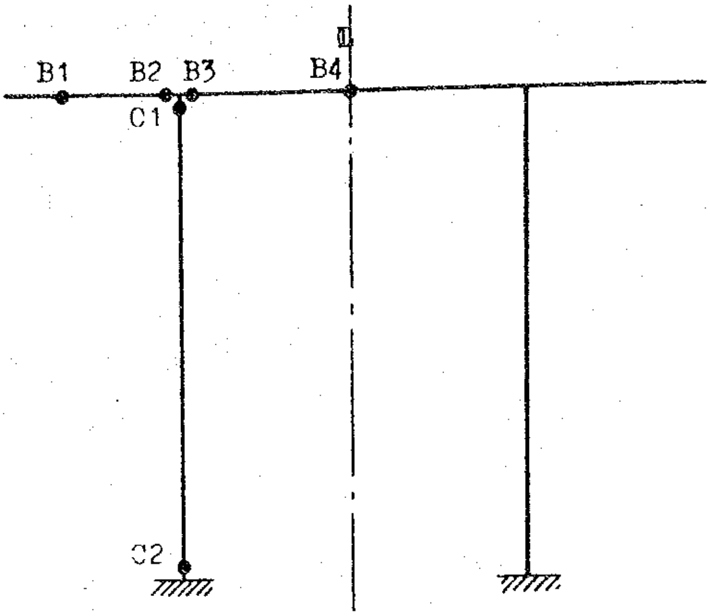
8) Wind loading diagram



		For Place	unloaded	live loaded
(T)	P1	superstructure safety fences	0.73 ton	0.64 ton
	P2	superstructure safety fences	0.48 ton	1.00 ton
	P3	superstructure safety fences	0.32 ton	0.66 ton
	P4	superstructure safety fences	0.16 ton	0.33 ton
	P5	Pier	0.56 ton	0.18 ton
	P1	Pier	0.43 t/m	0.15 t/m
(L)	P1	superstructure safety fences	2.90 ton	1.65 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	-0.325	0	80.828	1.00
	N max.	+					
		-	2	-0.236	-0.560	26.706	1.00
	S max.	+	4	-0.287	0	88.154	1.25
		-					
B2	M max.	+					
		-	4	-88.209	0	95.350	1.25
	N max.	+					
		-	2	-29.899	-1.290	35.280	1.00
	S max.	+	4	-88.209	0	95.350	1.25
		-					

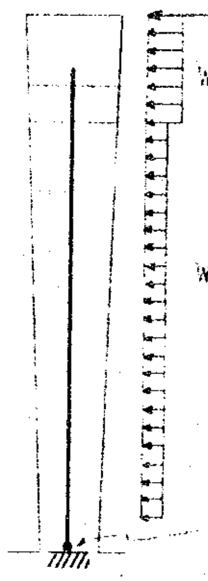
			case	M (tm)	N (ton)	S (ton)	
B3	M max.	+					
		-	3	-142.100	-1.378	-102.319	1.00
	N max.	+	3	-81.090	18.794	-50.562	1.00
		-	3	-75.234	-6.353	-82.722	1.00
	S max.	+					
		-	3	-105.301	12.251	-102.319	1.00
B4	M max.	+	3	62.538	7.277	-18.896	1.00
		-	4	-54.125	6.775	-15.439	1.25
	N max.	+	3	-13.777	18.794	-15.404	1.00
		-	3	25.739	-6.353	-18.896	1.00
	S max.	+					
		-	4	17.466	12.082	-44.633	1.25
C1	M max.	+	6	71.910	-153.296	-18.792	1.30
		-	3	-18.499	-133.490	6.353	1.00
	N max.	+					
		-	3	46.393	-233.431	-12.251	1.00
	S max.	+					
		-	3	70.604	-181.674	-18.794	1.00
C2	M max.	+	3	57.389	-136.819	18.422	1.00
		-	2	-30.782	-111.819	-8.329	1.00
	N max.	+					
		-	3	33.285	-217.060	10.640	1.00
	S max.	+	3	57.339	-159.002	18.794	1.00
		-					

These value is divided by "i".

i ; coefficient of increase

in basic stresses

2) due to longitudinal force



	M (tm)	V (ton)	H (ton)	
case-7				1.00
case-8	69.1	238.0	9.6	1.25
case-9	62.0	218.4	9.3	1.15
case-10	84.3	238.0	12.4	1.30
case-11				1.00

CASE 8

	N	H	y	M
WDL	79.8			
WLL (B)	128.2			
FB (B)		9.6	7.20	69.1
W	30.0			
TOTAL	238.0	9.6		69.1

M = 69.1 t·m

N = 238.0 ton

H = 9.6 ton

CASE 9

	N	H	y	M
WDL	79.8			
WLL . FB (4)	108.6	6.5	7.20	46.8
W	30.0			
WL (Super)		0.9	7.20	6.5
WL (1)		0.8	6.55	5.2
WL (2)		1.1	3.20	3.5
TOTAL	218.4	9.3		62.0

M = 62.0 t·m

N = 218.4 ton

H = 9.3 ton

CASE 10

	N	H	y	M
WDL	79.8			
WLL, FB(B)	128.2	9.6	7.20	69.1
W	30.0			
WL (Super)		0.9	7.20	6.5
WL (1)		0.8	6.55	5.2
WL (2)		1.1	3.20	3.5
TOTAL	238.0	12.4		84.3

M = 84.3 t·m

N = 238.0 ton

H = 12.4 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

	at B3 case 3	at B4 case 3	at C1 case 6
M (tm)	142.100	62.538	71.910
N (ton)	1.378	− 7.277	153.296
Q; shear force (ton)	102.319	18.896	18.792
b (cm)	113	113	113
d (cm)	120	120	110
d' (cm)	10	10	10
As (cm^2)	72.36 (D32 11.6cm Pitch)	28.26 (D20 11.6cm Pitch)	44.2 (D25 11.6cm Pitch)
d'/d			0.091
f=(M/N)+u (cm)			96.9
f/d			0.88
n	15	15	15
np=n·As/b·d	0.080	0.031	0.053
M'=M+N·x (tm)			148.6
M'/(b·d^2) (kg/cm^2)	8.73	3.84	10.87
Q/b·d (kg/cm^2)	7.55	1.39	1.51
C	6.85	9.82	3.45
S	14.0	35.0	2.0
Z	1.12	1.08	0.80
σ_c (kg/cm^2)	59.8	37.7	37.5
σ_s (kg/cm^2)	1833	2016	326
τ (kg/cm^2)	8.5 *	1.5	1.2
σ_{ca} (kg/cm^2)	85	85	85
σ_{sa} (kg/cm^2)	2340	2340	2340
τ_a (kg/cm^2)	8.2	8.2	8.2

*: see page 22'

	at C2 case 3	case 10	case
M (tm)	57.389	84.3	
N (ton)	136.819	238.0	
Q; shear force (ton)	18.422	12.4	
b (cm)	80	120	
d (cm)	110	70	
d' (cm)	10	10	
As (cm^2)	24.6 (D25 15cm pitch)	44.2 (D25 12.5cm pitch)	
d'/d	0.091	0.14	
e=(M/N)+u (cm)	91.9	65.4	
e/d	0.84	0.93	
n	15	15	15
np=n·As/b·d	0.042	0.079	
M'=M+N·u (tm)	125.8	155.7	
M'/($b \cdot d^2$) (kg/cm²)	13.0	26.5	
Q/b·d (kg/cm²)	2.09	1.48	
C	4.00	3.30	
S	3.5	1.8	
Z	0.8	0.84	
σ_c (kg/cm²)	52	87	
σ_s (kg/cm²)	683	716	
τ (kg/cm²)	1.7	1.2	
σ_{ca} (kg/cm²)	85	111	
σ_{sa} (kg/cm²)	2340	3042	
τ_a (kg/cm²)	8.2	10.7	

21

check for stirrups

$$\tau = \frac{Sh}{b \cdot d} \times Z = \frac{102.319 \times 10^3}{113 \times 120} \times 1.12 = 8.45 \ kg/cm^2$$

$$< \tau_a = 8.2 \ kg/cm^2$$

$$Req\ Aw = \frac{Sh \times a}{\sigma_{sa} \cdot d} \times Z$$

$$= \frac{102.319 \times 10^3 \times 12.5}{1780 \times 120} \times 1.12 = 6.71 \ cm^2$$

Φ 16 - ctc 125 $n = 4$

Used $Aw = 4 \times 2.01 = 8.04 \ cm^2 > Req\ Aw = 6.71 \ cm^2$

§ 6 Caiculation of stability

Pier self weight

Cantilever beam	$\frac{1}{2} \times (0.80 + 1.30) \times 1.20 \times 1.164 \times 2.407 \times 2$	7.06
beam	$5.40 \times 1.30 \times 1.164 \times 2.407$	19.67
column	$1.20 \times 5.90 \times 0.978 \times 2.407 \times 2$	33.33
Footing	$3.00 \times 6.40 \times 1.20 \times 2.407$	55.46
Σ		115.52

Reaction due to superstructure

	P
Dead load	159.7
HA Live load	105.6
HB Live load	115.1
Crowd load	——
Total (HA)	265.3
Total (HB)	274.8

Transverse direction

a) due to wind

· for superstructure

Unloaded $Pt = 4.1$ ton

Live loaded $Pt = 4.5$ ton

· for safety fences

$Pt = 1.0$ ton

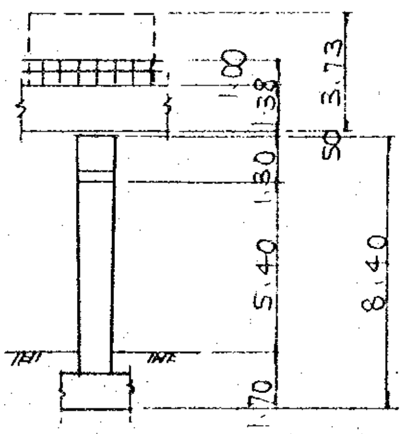
· for pier

Unloaded $Pt\text{-}1 = 0.7$ ton

$Pt\text{-}2 = 4.5$ ton

Live loaded $Pt\text{-}1 = 0.2$ ton

$Pt\text{-}2 = 1.6$ ton



Longitudinal direction

a) due to braking

Under HA ; $FB = 25.8 \times \frac{1}{2} = 12.9$ ton

" HB ; $FB = 38.3 \times \frac{1}{2} = 19.2$ ton

b) due to wind

· for superstructure

Unloaded $P_{LS} = 1.0$ ton

Live loaded $P_{LL} = 1.5$ ton

$P_{LS} = 0.3$ ton

· for safety fences $P_{L1} = 0.1$ ton

$P_{L2} = 0.3$ ton

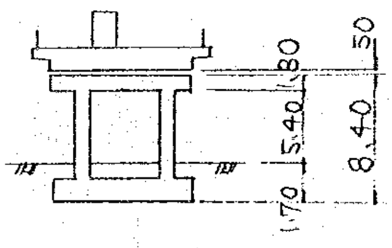
· for pier

Unloaded $P_{L1} = 4.4$ ton

$P_{L2} = 6.3$ ton

Live loaded $P_{L-1} = 1.5$ ton

$P_{L-2} = 2.2$ 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	159.7					
	WLL (HA)	105.6			12.9	8.45	109.01
	WLL (HB)	115.1			19.2	8.45	162.24
	Self weight	115.5					
Transverse direction	WL (Unloaded)						
	for superstruct.				4.1	9.14	37.47
	for safety fence				1.0	10.33	10.33
	for pier (1)				0.7	7.75	5.43
	(2)				4.5	4.40	19.80
	WL (live loaded)						
	for superstruct (1)				4.5	10.32	46.44
	for pier (1)				0.2	7.75	1.55
	〃 (2)				1.6	4.40	7.04
Longitudinal direction	WL (Unloaded)						
	for superstruct.				1.0	8.45	8.45
	for safety fence				0.4	8.45	3.38
	for pier (1)				4.4	7.75	34.10
	(2)				6.3	4.40	27.72
	WL (Live loaded)						
	for superstruct. (1)				1.5	8.45	12.68
	(2)				0.3	8.45	2.54
	for pier (1)				1.5	7.75	11.63
	(2)				2.2	4.40	9.68

Longitudinal direction

Case No	Loading combination	N (t)	H (t)	M (t)
1	Dead load only	275.2	0	0
2	Dead load + wind	275.2	12.1	73.65
3	HA Loading	380.8	12.9	109.01
4	HB Loading	390.3	19.2	162.24
5	HA Loading + wind	380.8	18.4	145.54
6	HB Loading + wind	390.3	24.7	198.77

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

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

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

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		0	14.3		60.0		
2	0.500	0.268	22.0	6.7	60.0	14	1.5
3	↓	0.286	31.2	8.5	60.0	18	↓
4	↓	0.416	37.2	3.4	75.0	12	↓
5	↓	0.382	35.0	4.7	69.0	12	↓
6	↓	0.509	41.0	—	78.0	9	↓

Transverse direction

Case No	Loading combination	N (t)	H (t)	M (t)
1	~~Dead load only~~			
2	Dead load + wind	275.2	10.3	73.03
3	~~HA Loading~~			
4	~~HB Loading~~			
5	HA Loading + wind	380.8	6.3	55.03
6	HB Loading + wind	390.3	6.3	55.03

Calculation of Stability

- eccentric $e = M/N$ $\qquad B/6 = 1.07$
- 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 $\quad F = \dfrac{N \times 0.6}{H}$

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	0.265	17.9	10.8	60.0		
~~3~~							
~~4~~							
5	1.07	0.145	22.5	17.1	69.0		
6	1.07	0.141	23.0	17.6	78.0		

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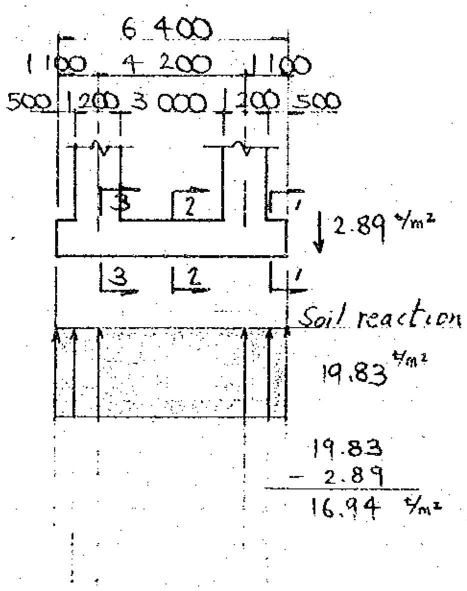
3 7. Calculation of footing section

I Transverse direction



Sect 1.

$M = 16.94 \times 0.5^2 \times \frac{1}{2} \times 3.0 = 6.35$ t·m

$M' = 6.35 \times \frac{1}{3} = 2.12$ t·m

Sect 2.

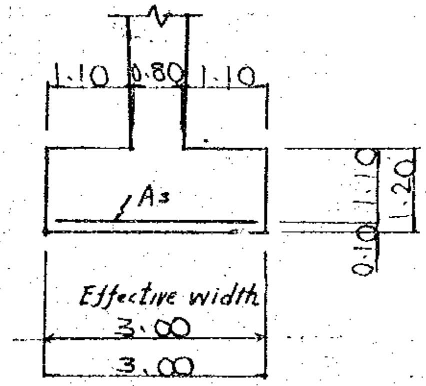
$M = 16.94 \times 3.0^2 \times \frac{1}{8} \times 3.0 = 57.17$ t·m

$M' = 57.17 \times \frac{1}{3} = 19.06$ t·m

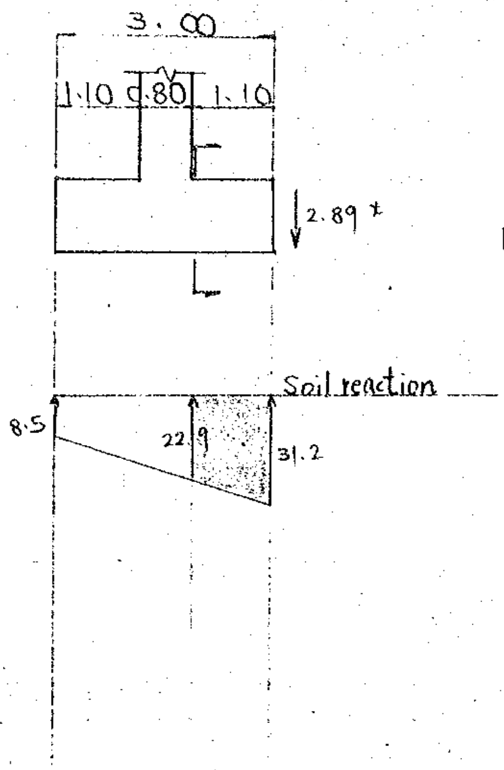
Sect 3.

$M = 16.94 \times 4.20^2 \times \frac{1}{12} \times 3.0 = 74.71$ t·m

$M' = 74.71 \times \frac{1}{3} = 24.90$ t·m



2 Longitudinal direction



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

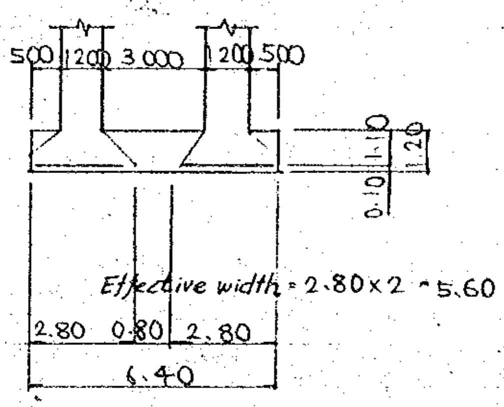
$$- \frac{1}{2} \times (31.2 + 22.9) \times 1.10 \times 6.40 \quad (190.43) = -170.08$$

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

$$- 190.43 \times \frac{1}{3} \times \frac{2 \times 31.2 + 22.9}{31.2 + 22.9} \times 1.10 \quad (110.09) = -98.90$$

$$S' = -170.08 \times \frac{1}{5.60} = 30.37 \text{ ton}$$

$$M' = -98.90 \times \frac{1}{5.60} = 17.66 \text{ t·m}$$



	Transverse direction		Longitudinal direction
	case bottom	case Top	case bottom
M (tm)	24.90	19.06	17.66
N (ton)			
S; 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			
e=(M/N)+u (cm)			
e/d			
n	15	15	15
np=n·As/b·d	0.022	0.022	0.022
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	2.06	1.58	1.46
S/b·d (kg/cm²)			
C	11.3	11.3	11.3
S	48.5	48.5	48.5
Z			
σ_c (kg/cm²)	23.3	17.9	16.5
σ_s (kg/cm²)	1499	1149	1062
τ (kg/cm²)			
σ_{ca} (kg/cm²)			
σ_{sa} (kg/cm²)			
τ_a (kg/cm²)			