AFRICAN DEVELOPMENT BANK

GOVERNMENT OF MAURITIUS

BEAU BASSIN - PORT LOUIS LINK ROAD

CALCULATION NOTE
FOR
SUBSTRUCTURES

1

SEPTEMBER 1980

Japan International Cooperation Agency

CONTENTS

1. C.R.N.W. Bridge and St. Louis River Bridge

CONTENT

SS 1. DESIGN CONDITIONS

§ 1 DESIGN CONDITIONS

1 structural type

abutments	· counterfort type	Height	10.0	meters.
	· cantilever type	Height	7.5	meters.
piers	· Wall type	Height	26.0	meters
	· T type	Height	14.7	meters

2 foundation type

Spread footing

3 unit weight of reinforced concrete and soil

reinforced concrete	2.41	ton/m^3
soil	1.90	ton/m^3

4 bearing capacity

permissible bearing capacity $qa = 50\ ton/m^2$

6 permissible stresses of reinforced concrete

1) Concrete grade 25

specified works cube strength at 28 days $25\ N/mm^2$

$= 255\ kg/cm^2$

Permissible compressive stress $\sigma ca = 83\ kg/cm^2$

permissible shear stress

$$\tau_o^{*} = 0.81\ N/mm^2$$
$$= 8.2\ kg/cm^2$$

* permissible shear in solid slab without shear reinforcement.

Percentage of flexural tensile steel $100\ A_s/bd$	0.25 or less	0.5	1.0	2.0	3.0 or more
Permissible shear stress N/mm^2	0.23	0.34	0.46	0.63	0.70
kg/cm^2	2.35	3.47	4.69	6.43	7.14

2) Reinforcement

Hot rolled high yield bars

specified characteristic strength $\sigma_{sy} = 410\ N/mm^2$
$= 4180\ kg/cm^2$

permissible tensile stress $\sigma_{sa} = 230\ N/mm^2$
$= 2340\ kg/cm^2$

permissible tensile stress in shear reinforcement $\sigma_{sa} = 175\ N/mm^2$
$= 1780\ kg/cm^2$

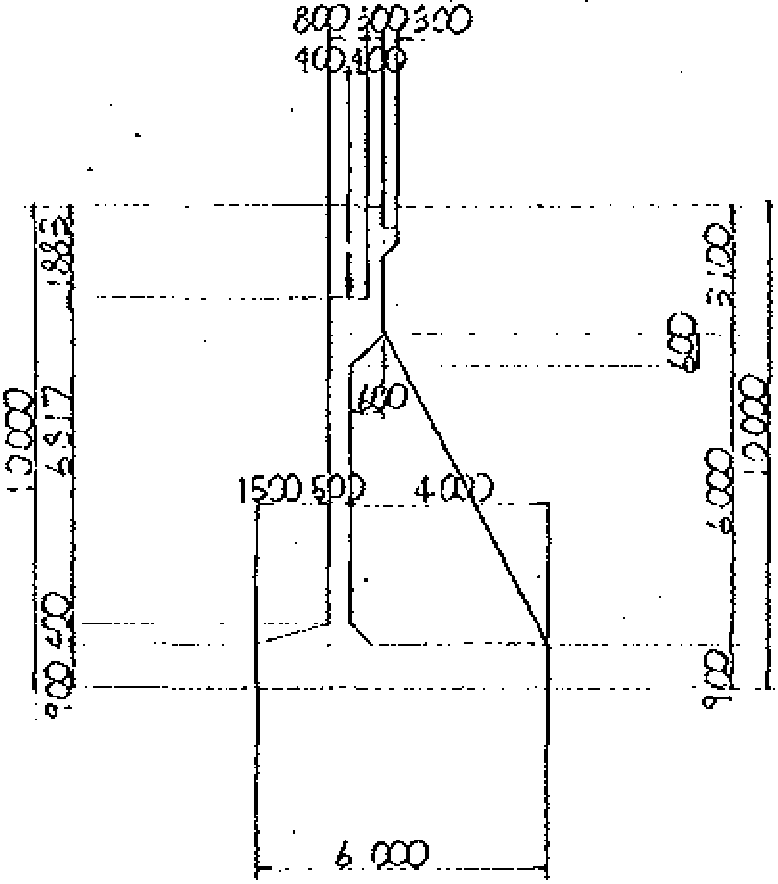
7. permissible increase in basic working stresse

Load combination	Increase in basic permissible stresses (per cent)
Dead Load (+ Temperature effect)	0
Dead Load + HA Load	0
Dead Load + H B Load	25
Dead Load + wind Load	15
Dead Load + H A Load + wind Load	15
Dead Load + H B Load + wind Load	30

§§ 2 ABUTMENT

$H = 10.00^m$

§ 1 DIMENSIONS

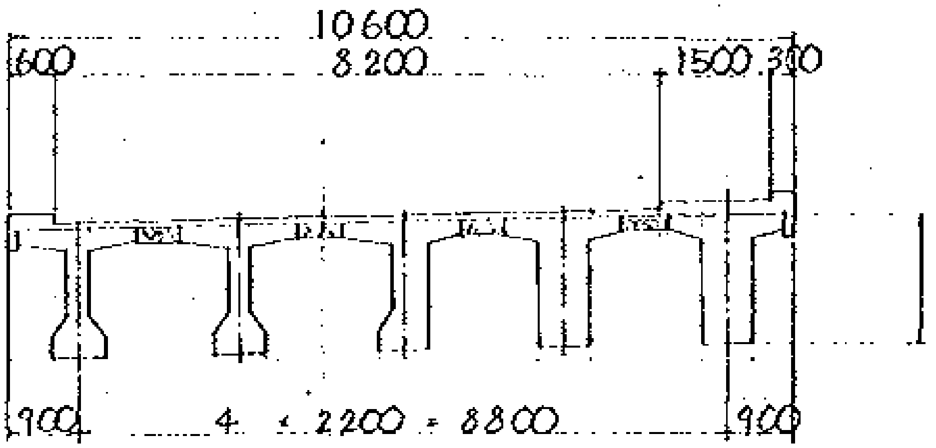


counterforts span $l = 3.400^{m}$

(ST LOUIS A1 . A2 .)
(G.R.N.W A2)

§ 2 REACTION OF SUPERSTRUCTURE

2 - 1 dimensions

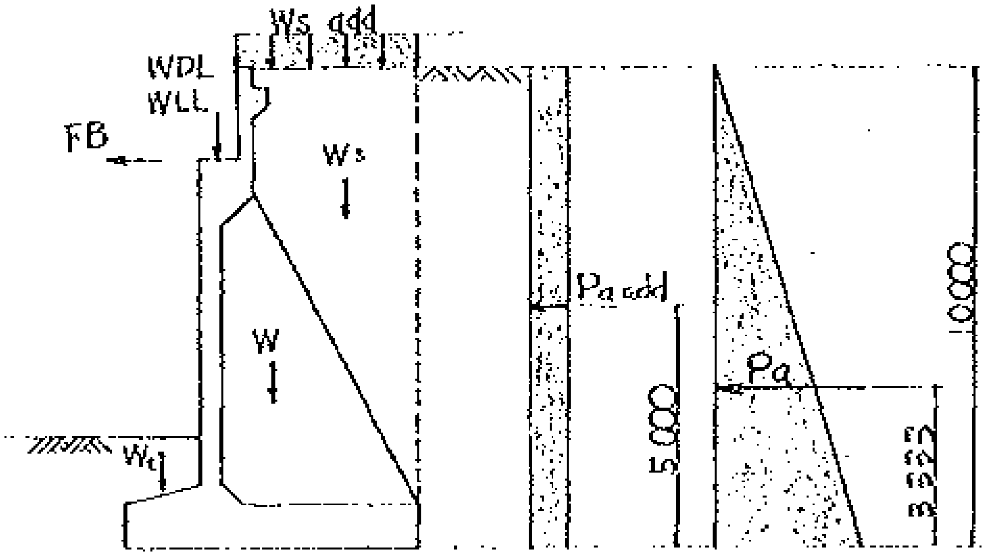


2 - 2 whole reaction due to superstructure

		HA loading		HB loading	
		N (t)	H (t)	N (t)	H (t)
dead load of deck	I	190.7	------	190.7	------
	II	44.5	------	44.5	------
live load		113.4	------	141.5	------
crowd load		10.3	------	10.3	------
longitudinal force		------	25.8	------	38.2
TOTAL		358.9	25.8	387.0	38.2

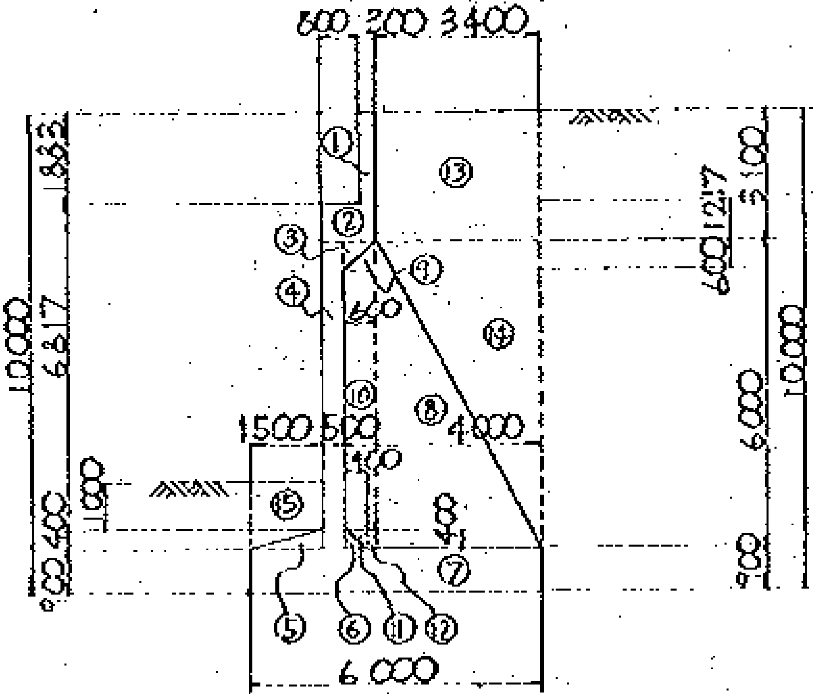
§ 3 CALCULATION OF LOAD

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
PA : active pressure
PP : passive pressure
PA add : surcharge
T : temperature load

3-2 self weight and weight of soil



$$\bar{\gamma} = \frac{2.41 \times 0.60 \times 4 + 1.9 \times 8.20}{10.60} = 2.02 \ \mathrm{t/m^3}$$

		N (t)	x (m)	N x (t m)
①	0.30 × 1.883 × 10.60 × 2.41	14.43	2.45	35.35
②	1.10 × 1.217 × 10.60 × 2.41	34.20	2.05	70.11
③	½ × 0.60 × 0.60 × 10.60 × 2.41	4.60	2.20	10.12
④	0.50 × 6.00 × 10.60 × 2.41	76.64	1.75	134.12
⑤	½ × 1.50 × 0.40 × 10.60 × 2.41	7.66	1.00	7.66
⑥	½ × 0.40 × 0.40 × 10.60 × 2.41	2.04	2.133	4.35
⑦	0.90 × 6.00 × 10.60 × 2.41	137.90	3.00	413.70
⑧	½ × 6.00 × 3.40 × 10.60 × 2.02	218.40	3.733	819.25
⑨	½ × 0.60 × 0.60 × 10.60 × 2.02	3.85	2.40	9.24
⑩	0.60 × 5.00 × 10.60 × 2.02	64.24	2.30	147.75
⑪	½ × 0.40 × 0.40 × 10.60 × 2.02	1.71	2.267	3.88
⑫	0.20 × 0.40 × 10.60 × 2.02	1.71	2.50	4.28
⑬	3.40 × 3.10 × 10.60 × 1.9	212.28	4.30	912.80
⑭	½ × 3.40 × 6.00 × 10.60 × 1.9	205.42	4.867	999.78
⑮	½ × (1.00 + 1.40) × 1.50 × 10.60 × 1.9	36.25	0.708	25.67
Σ		1,021.33		3,598.06

3-3 weight of surcharge

under H.A = $1.02 \times 3.70 \times 10.60 = 40.0^t$

under H.B = $1.66 \times 3.70 \times 10.60 = 65.11^t$

3-4 earth pressure

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

angle of internal friction $\phi = 35°$

(1) active pressure

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

$$= 1/2 \times \frac{1 - \sin 35°}{1 + \sin 35°} \times 1.9 \times 10.00^2 \times 10.60 = 272.89^t$$

(2) active pressure due to surcharge

under H.A surcharge

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

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

$= 0.27 \times 1.02 \times 10.00 \times 10.60 = 29.19^t$

under H.B surcharge

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

$Pa\ add = K q H L$

$= 0.27 \times 1.66 \times 10.00 \times 10.60 = 47.51^t$

3 - 5 temperature load

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

$$\delta = \mathcal{I} \cdot l \qquad \mathcal{I} = \left| \begin{array}{l} P.C \longrightarrow 0.7 \\ R.C \longrightarrow 0.5 \end{array} \right.$$

$$\left[\begin{array}{l} \delta = 0.7 \times 26.80 = 18.76^{mm} \\ R(d+l) = 358.9 \times 1/5 \times 1.4 = 100.49^{t} \end{array} \right]$$

δ ; elongation of girder

RING SHOE
- 110 TON
- $D\phi$: 54^{cm} (Diameter)
- $d\phi$: 32^{cm}
- A : 2289^{cm^2} (Area)
- t : 7.1^{cm} (thickness of bearing)
- G_0 : $13.5^{kg/cm^2}$ (modulus of rigidity)

$$P_H = \frac{13.5 \times 2289 \times 1.88}{7.1} = 8182^{kg} = 8.18^{t}$$

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

$$= 5 \times 8.18 \times 1/2 = 20.45^{t}$$

§ 4 CALCULATION OF STABILITY

case 1 HA loading

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	H·y (tm)
WDL. WLL	358.9	1.90	681.91	——	——	——
F B	——	——	——	25.80	8.117	209.42
T	——	——	——	20.45	8.117	166.00
W. Ws. Wt	1021.33		3598.06	——	——	——
Ws add	40.00	4.30	172.0	——	——	——
P a	——	——	——	272.89	3.333	909.54
Pa add	——	——	——	29.19	5.00	145.95
TOTAL	1420.23		4451.97	348.33		1430.91

1) check for eccentric

$$x = \frac{\Sigma Nx - \Sigma HY}{\Sigma N} = \frac{4451.97 - 1430.91}{1420.23} = 2.13^{m}$$

$$e = \frac{B}{2} - x = \frac{6.00}{2} - 2.13 = 0.87^{m} < \frac{B}{6} = 1.0^{m}$$

2) soil reaction

$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{1420.23}{6.00 \times 10.60} \times \left(1 \pm \frac{6 \times 0.87}{6.00}\right)$$

$$= \left\{ \begin{array}{l} 41.76 \ {}^{t}/m^2 \\ 2.90 \end{array} \right. < 50 \ {}^{t}/m^2$$

3) check for sliding

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

$$F = \frac{Hu}{\Sigma H} = \frac{1420.23 \times 0.6}{348.33} = 2.45 > Fa = 1.5$$

case 2 HB loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL.WLL	387.00	1.90	735.3	—	—	—
F B	—	—	—	38.2	8.117	310.07
T	—	—	—	20.45	8.117	165.99
W, Ws, WI	1021.33		3598.06	—	—	—
Ws add	65.11	4.15	270.21	—	—	—
Pa	—	—	—	272.89	3.333	909.54
Pa add	—	—	—	47.51	5.00	237.55
TOTAL	1473.44		4603.57	379.05		1623.15

1) check for eccentric

$$x = \frac{\Sigma Nx - \Sigma H{\cdot}y}{\Sigma N} = \frac{4603.57 - 1623.15}{1473.44} = 2.02^{m}$$

$$e = \frac{B}{2} - x = \frac{6.00}{2} - 2.02 = 0.98^{m} < \frac{B}{3} = \quad {}^{m}$$

2) soil reaction

$$q = \frac{1473.44}{6.00 \times 10.60} \times \left(1 \pm \frac{6 \times 0.98}{6.00}\right) = \begin{cases} 45.87 \ {}^{t}/_{m^2} \\ 0.46 \end{cases} , < 62.5 \ {}^{t}/_{m^2}$$

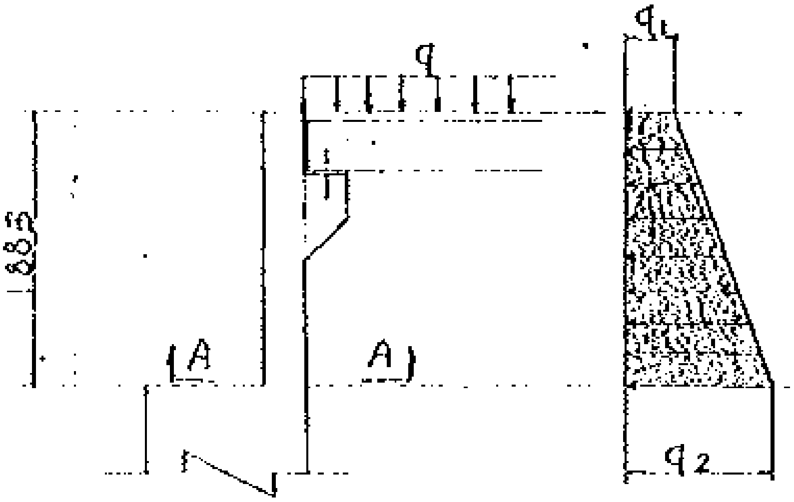
3) check for sliding

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

$$F = \frac{Hu}{\Sigma H} = \frac{0.6 \times 1473.44}{379.05} = 2.33 \quad > Fa = 1.2$$

§ 5 CALCULATION OF PARAPET SECTION

5-1 dimension and loading



	q_1	q_2
H.A loading	0.28	1.24
H.B loading	0.45	1.41

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

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

5 - 2 sectional force of parapet

CASE 1 (HA)

$S = 1/2 \times (0.28 + 1.24) \times 1.883 = 1.43^{t}$

$M = 1.43 \times 1/3 \times 1.883 \times \dfrac{2 \times 0.28 + 1.24}{0.28 + 1.24} = 1.06^{tm}$

CASE 2 (HB)

$S = 1/2 \times (0.45 + 1.41) \times 1.883 = 1.75^{t}$

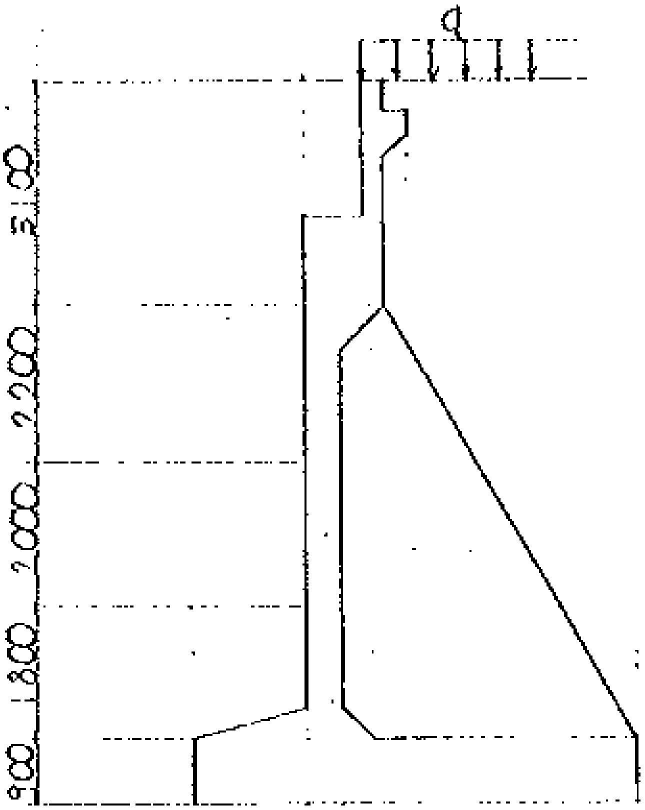
$M = 1.75 \times 1/3 \times 1.883 \times \dfrac{2 \times 0.45 + 1.41}{0.45 + 1.41} = 1.36^{tm}$

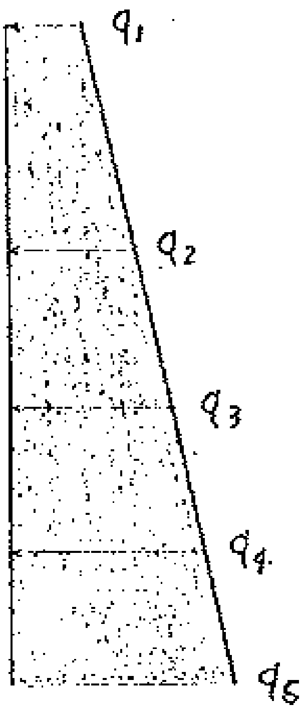
5 - 3. list of stresses σ_c, σ_s, τ : working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

	* A_s min = b·d·0.15% = 3.45 $^{in^2}$					
	case 1	case 2				
M	1.06	1.36				
N	——	——				
S	1.43	1.75				
b	100	———→				
h	23	———→				
d'	7	———→				
As	D16 @125 16.08	———→		$\frac{100 A_s}{b \cdot d}$ = 0.7	~ τ_a = 0.34 [illegible]	
As'	-----	-----				
f/d	0	0				
M'/bd^2	2.00	2.57				
S/bd	0.62	0.76				
nP	0.1049	———→				
C	6.24	———→				
S	10.86	———→				
Z	1.14	———→				
σ_c	13	16				
σ_s	326	419				
τ	0.7	0.9				
σ_{ca}	83	103				
σ_{sa}	2346	2933				
τ_a	3.47	4.34				

§ 6 . CALCULATION OF WALL SECTION

6-1 dimension and loading.



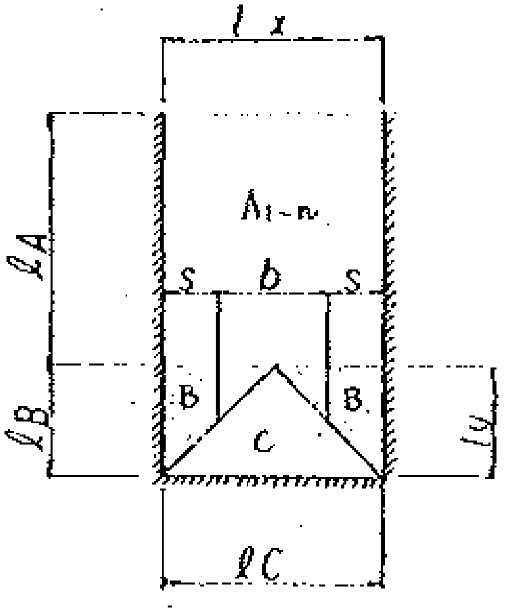


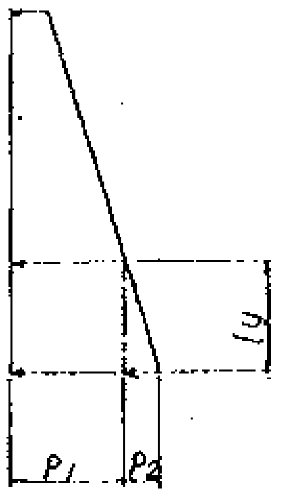
	q_1	q_2	q_3	q_4	q_5	q_6	q_7
HA loading	0.28	1.87	2.99	4.02	4.94	—	—
HB loading	0.45	2.04	3.17	4.19	5.12	—	—

$q_1 = q \cdot K = 0.27 \cdot q$ t/m²

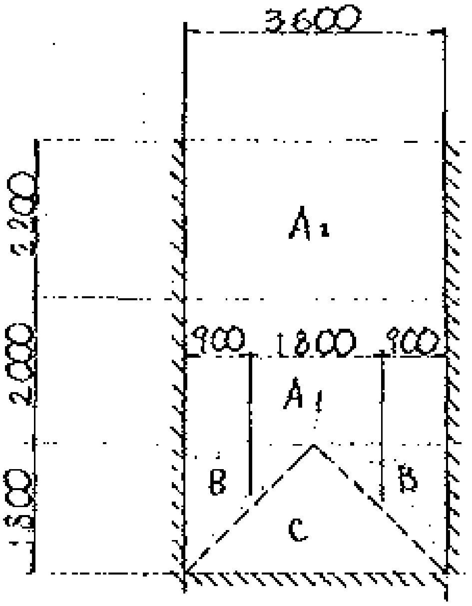
$q_x = K \cdot \gamma_s \cdot H_x + q_1 = 0.513 \cdot H_x + 0.27 \cdot q$

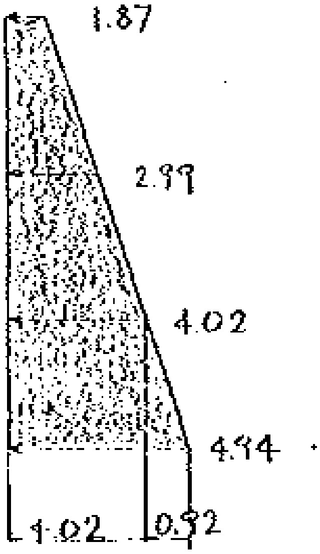
6-2 sectional force of wall





	A	B	C
(tm) M	$\frac{p\cdot lx^2}{10}$	$\frac{p\cdot s^2}{6\cdot lx}(2\cdot lx + b)$	$\frac{1}{2}(\frac{p_1}{2}+\frac{p_2}{6})ly^2$
(t) S	$\frac{p\cdot lx}{2}$	$p\cdot s$	$(p_1+\frac{p_2}{2})\,ly$





	M	(tm)	S	(t)
C－C	$\frac{1}{2}\cdot\left(\frac{4.02}{2}+\frac{0.92}{6}\right)\times1.80^2$	3.50	$\left(4.02+\frac{0.92}{2}\right)\times1.80$	8.06
B－B	$\frac{4.48\times0.90^2}{6\times3.60}\times(2\times3.60+1.80)$	1.51	4.48×0.90	4.03
A1－1	$\frac{4.02\times3.60^2}{10}$	5.21	$\frac{4.02\times3.60}{2}$	7.24
A2－2	$\frac{2.99\times3.60^2}{10}$	3.88	$\frac{2.99\times3.60}{2}$	5.38
A3－3	—	—	—	—
A4－4	—	—	—	—

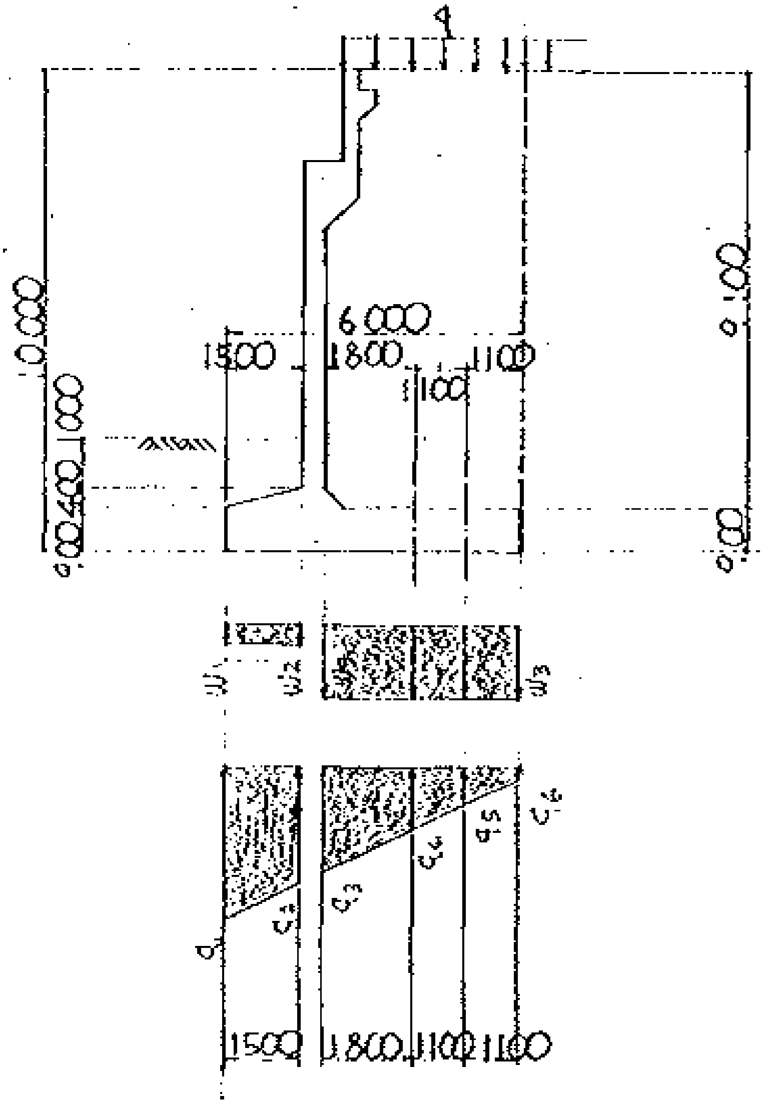
6 - 3 list of stresses σ_c, σ_s, τ : working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

	* $A_{s\,min} = b \cdot d \cdot 0.15\% = 6.95\ cm^2$					
	C - C	B - B	$A_1 - 1$	$A_2 - 2$		
M	3.50	1.51	5.21	3.88		
N	——	——	——	——		
S	8.06	4.03	7.24	5.38		
b	1.00	------	------	------→		
h	43	------	------	------→		
d'	7	------	------	------→		
As	* D16 @ 250 8.04	* ------	------	* ——→		
As'	——	——	——	——		
f/d	│	│	0	│		
M'/bd^2	│	│	2.82	│		
S/bd	│	│	1.68	│		
n·P	│	│	00280	│		
C	│	│	10.22	│		
S	│	│	3836	│		
Z	│	│	1.08	│		
σ_c	│	│	29	│		
σ_s	│	│	1622	│		
τ	│	│	1.8	│		
σ_{ca}	83	------	------	------→		
σ_{sa}	2346	------	------	------→		
τ_a	2.35	------	------	------→		

$\frac{100A_s}{b \cdot d} = 0.2$

§ 7 CALCULATION OF FOOTING SECTION

7-1 dimension and loading



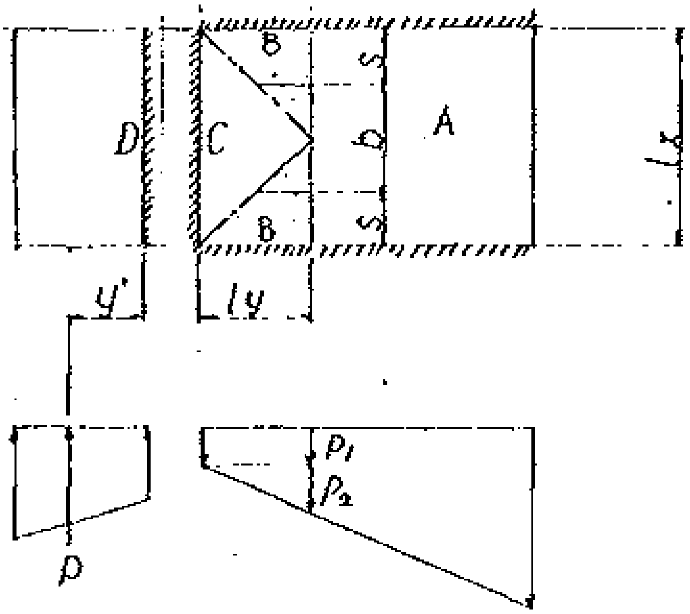
$w_1 = 0.90 \times 2.41 + 1.40 \times 1.9 = 4.83$ t/m²

$w_2 = 1.30 \times 〃 + 1.00 \times 〃 = 5.03$ 〃

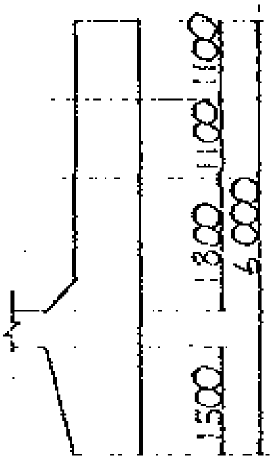
$w_3 = 0.90 \times 〃 + 9.10 \times 〃 + q = 19.46 + q$ 〃

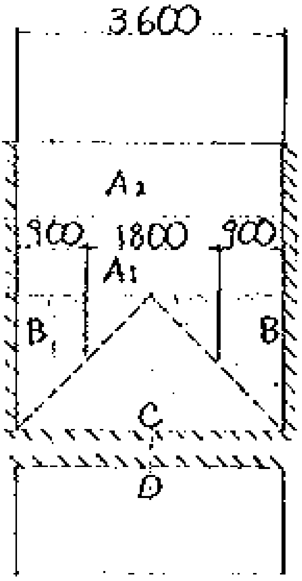
	q_1	q_2	q_3	q_4	q_5	q_6
HA loading	41.76	32.06	26.83	17.19	10.07	2.96
HB loading	45.87	34.52	30.73	17.11	8.79	0.46

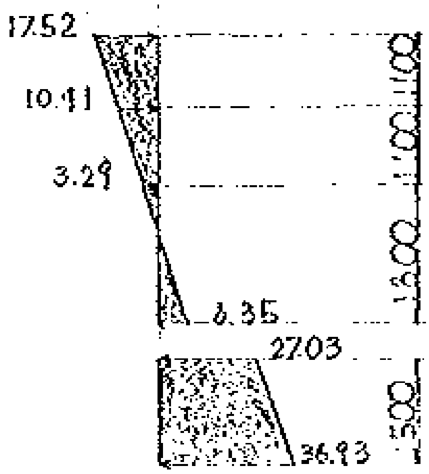
7 - 2 sectional force of footing



	A	B	C	D
(tm) M	$\frac{p \cdot lx^2}{10}$	$\frac{p \cdot s}{6 \cdot lx}(2 \cdot lx + b)$	$\frac{1}{2}(\frac{p_1}{2} + \frac{p_2}{6}) ly^2$	$p \cdot y'$
(t) S	$\frac{p \cdot lx}{2}$	$p \cdot s$	$(p_1 + \frac{p_2}{2}) \cdot ly$	p







	M	(tm)	S	(t)
D − D	$63.96 \times \frac{1.50}{3} \times \left(\frac{27.03+2\times36.93}{27.03+36.93}\right)$	50.45	$\frac{1}{2}\times(27.03+36.93)\times1.50$	47.97
C − C	$\frac{1}{2}\times\left(\frac{0}{2}+\frac{8.35}{6}\right)\times1.80^2$	2.25	$\left(0+\frac{8.35}{2}\right)\times1.80$	7.52
B − B	$\frac{4.18\times0.90^2}{6\times3.60}\times(2\times3.60+180)$	1.41	4.18×0.90	3.76
A1 − 1	$\frac{10.41\times3.60^2}{10}$	13.49	$\frac{10.41\times3.60}{2}$	18.74
A2 − 2	$\frac{17.52\times3.60^2}{10}$	22.71	$\frac{17.52\times3.60}{2}$	31.54
A3 − 3	———	——	———	——

7 - 3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.

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

	※ As min = b·d·0.15% = 12.00 cm^2					
	D - D	C - C	B - B	A1 - 1	A2 - 2	
M	50.45	2.25	1.41	13.49	22.71	
N	----	----	----	----	----	
S	47.97	7.52	3.76	18.74	31.54	
b	100	----	----	----	----→	
h	130	80	----	----	----→	
d'	10	10	----	----	----→	
As	D20@125 25.12	D16@125 ※ 16.08	※	※	D16@125 16.08	
As'	----	----	----	----	----	
f/d	0				0	
M'/bd²	2.99				1.34	
S/bd	3.69				2.43	
n·P	0.0290				0.0186	
C	10.09				12.14	
S'	37.15				57.29	
Z	1.08				1.06	
σ_c	30				16	
σ_s	1664				1155	
τ	4.0				2.6	
σ_{ca}	83	----	----	----	----→	
σ_{sa}	2346	----	----	----	----→	
τ_a	2.35	----	----	----	----→	

Check for stirrups

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

$$= \frac{47.77 \times 10^3}{100 \times 130} \times 1.08 = 4.00 > \tau_c = 2.35$$

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

$$S_h' = S_h - S_c$$

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

$$= 2.35 \times 100 \times 130 \times \frac{1}{1.08} = 28\ 290^{kg}$$

$$S_h' = (47.77 - 28.29) \times 10^3 = 19.68 \times 10^3\ ^{kg}$$

$$\text{req. } A_w = \frac{19.68 \times 10^3 \times 25^{cm}}{1780 \times 130} \times 1.08 = 2.30^{cm^2}$$

$\Phi 16$ - c/c 250^{mm} $n = 2$

Used $A_w = 2.01 \times 2 = 4.02^{cm^2} > \text{req. } A_w = 2.30^{cm^2}$

Sect $A_2 - c$

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

$$= \frac{31.54 \times 10^3}{100 \times 80} \times 1.06 = 4.18\ {}^{kg}/_{cm^2} > \tau_a = 2.35\ {}^{kg}/_{cm^2}$$

$$req\ A_v = \frac{S_h' \times a}{\sigma_{sa} \cdot d} \times Z$$

$$S_h' = S_h - S_c$$

$$S_c = \tau_a \cdot b \cdot d \cdot {}^1/_Z$$

$$= 2.35 \times 100 \times 80 \times {}^1/_{1.06} = 17.74 \times 10^3\ kg$$

$$S_h' = (31.54 - 17.74) \times 10^3 = 13.80 \times 10^3\ kg$$

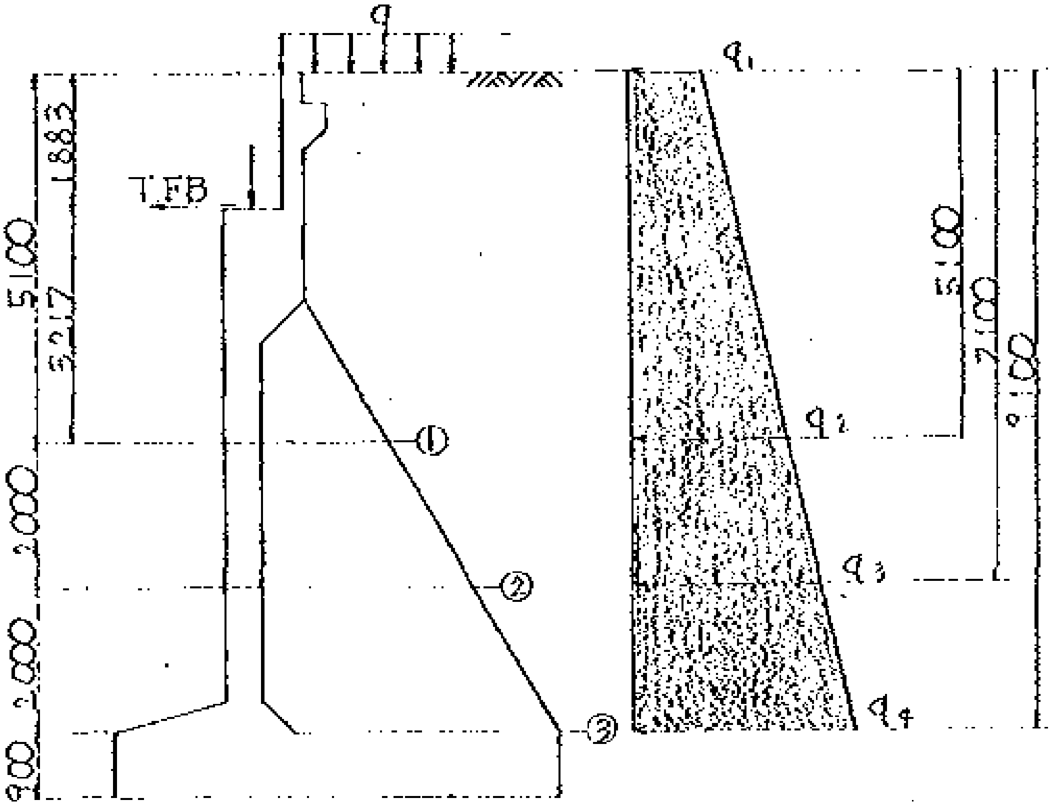
$$req\ A_v = \frac{13.80 \times 10^3 \times 25}{1780 \times 80} \times 1.06 = 2.57\ cm^2$$

$\phi 16$ - ctc 250 $n = 2$

$$A_s = 2.01 \times 2 = 4.02\ cm^2 > req\ A_v = 2.57\ cm^2$$

§ 8 CALCULATION OF COUNTERFORT SECTION

8 - 1 dimenston and loading



$$q_x = (K \cdot \gamma_s \cdot H + q \cdot K) \cdot l$$

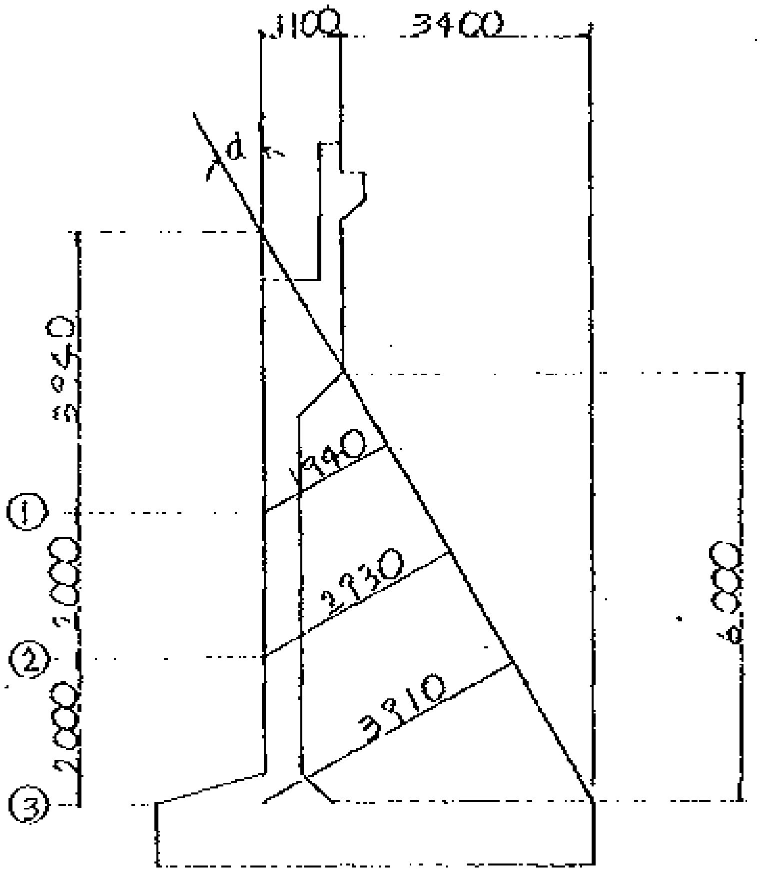
	(t) F B	(t) T	(t) FB + T	(t) (FB+T) l / B
HA loading	25.80	20.58	46.38	15.75
HA loading	38.20	20.58	58.78	19.96

	q_1	q_2	q_3	q_4	q_5	q_6
HA loading	0.99	10.41	14.10	17.80	——	——
HB loading	1.61	11.03	14.73	18.42	——	——

8 - 2 sectional force of counterfort

		HA loading			HB loading		
		H (t)	y (m)	H·y (tm)	H (t)	y (m)	H·y (tm)
1 - 1	FB · T	15.75	3.217	50.67	19.96	3.217	64.21
	Pa	29.07	1.848	53.72	32.23	1.917	61.78
	Σ	44.82		104.39	52.19		125.99
2 - 2	FB · T	15.75	5.217	82.17	19.96	5.217	104.13
	Pa	53.57	2.522	135.10	58.01	2.600	150.83
	Σ	69.32		217.27	77.97		254.96
3 - 3	FB · T	15.75	7.217	113.67	19.96	7.217	144.05
	Pa	85.49	3.193	272.97	91.14	3.277	298.67
	Σ	101.24		386.64	111.10		442.72
4 - 4	FB · T						
	Pa						
	Σ						
5 - 5	FB · T						
	Pa						
	Σ						

8 - 3 calculation of members

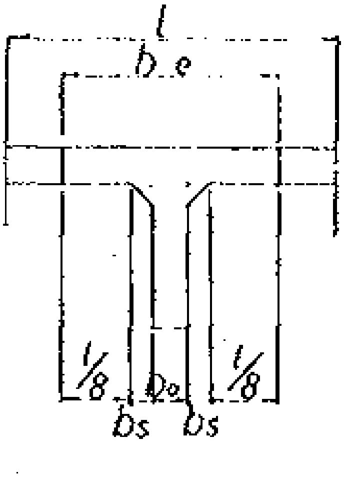


$$\tan\alpha = \frac{340}{600} = 0.567$$

$$\alpha = 29°32'$$

$$H = h \cdot \sin\alpha$$

$$= 0.493 \cdot h$$



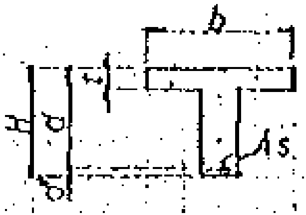
$$b_e = b_0 + 2\left(b_s + \frac{l}{8}\right) = 60 + 2 \times (30 + 45)$$

$$= 210^{cm}$$

8.-4 list of stresses

σ_c, σ_s, τ : working stress.
σ_{ca}, σ_{sa}, τ_a : Permissible stress.

		※·$(A_s)_{min} = b\cdot d\cdot 0.15\%$				
		1 − 1	2 − 2	3 − 3		
M	tm	104.39	217.27	386.64		
S	t	44.82	69.32	101.24		
b	cm	210	———	———→		
t	〃	50	———	———→		
d	〃	184	283	376		
A s	cm²	4 - D32 ※ (16.56) 3216	5 - D32 × (25.47) 40.20	$\frac{5}{2}$ > D32 ※(33.84) 56.28		
P		0.0008	0.0007	0.0007		
t/d		0.27	0.18	0.13		
K		0.172	0.139	0.135		
j		1.033	0.962	0.956		
σ_s	kg/cm²	1708	1984	1912		
σ_c	〃	24	21	20		
τ	〃	2.4	3.7	5.5		
σ_{sa}	〃	2346	———	———→		
σ_{ca}	〃	83	———	———→		
τ_a	〃	8.2	———	———→		



8 - 5 check for tie bers

1) wall and counterfort

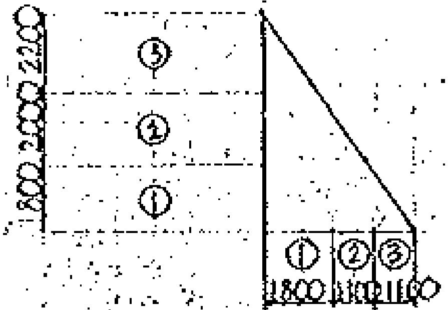
$$A_s = \frac{S}{\sigma_{sa}} \quad (cm^2)$$

		S (t)	As (cm²)	As' (cm²)	
section	1 -- 1	4.03	1.72	D16 ctc 250	8.04
	2 -- 2	7.24	3.09	↓	↓
	3 -- 3	5.38	2.29	↓	↓
	4 -- 4	—	—	—	—
	5 -- 5	—	—	—	—

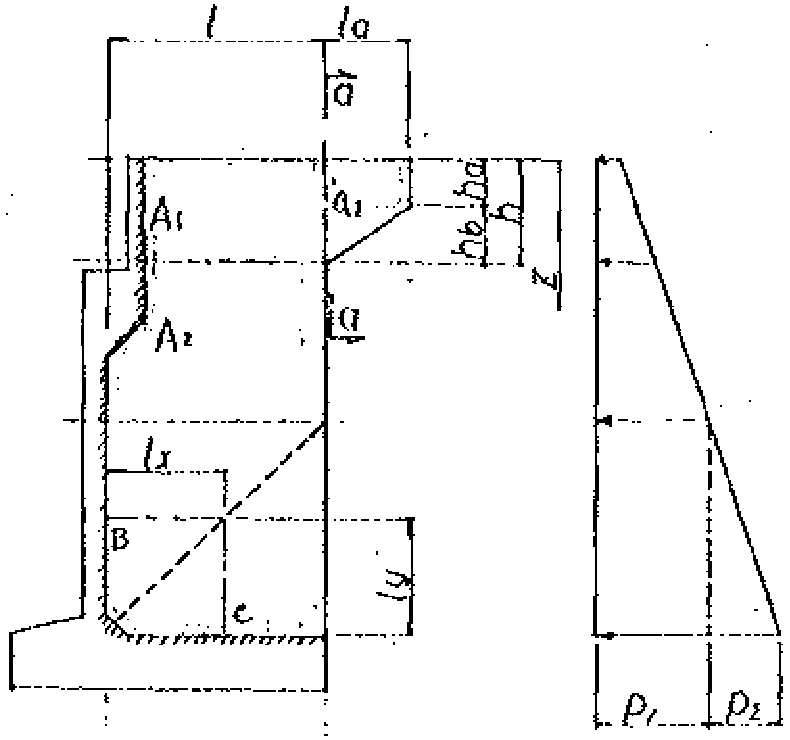
2) footing and counterfort

$$A_s = \frac{S}{\sigma_{sa}} \quad (cm^2)$$

		S (t)	As (cm²)	As' (cm²)	
section 1	1 1	3.76	1.60	D16 ctc 250	8.04
	2 2	18.74	7.99	D16 ctc 250	8.04
	3 3	31.54	13.44	D16 ctc 125	16.08



§ 9 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/m²

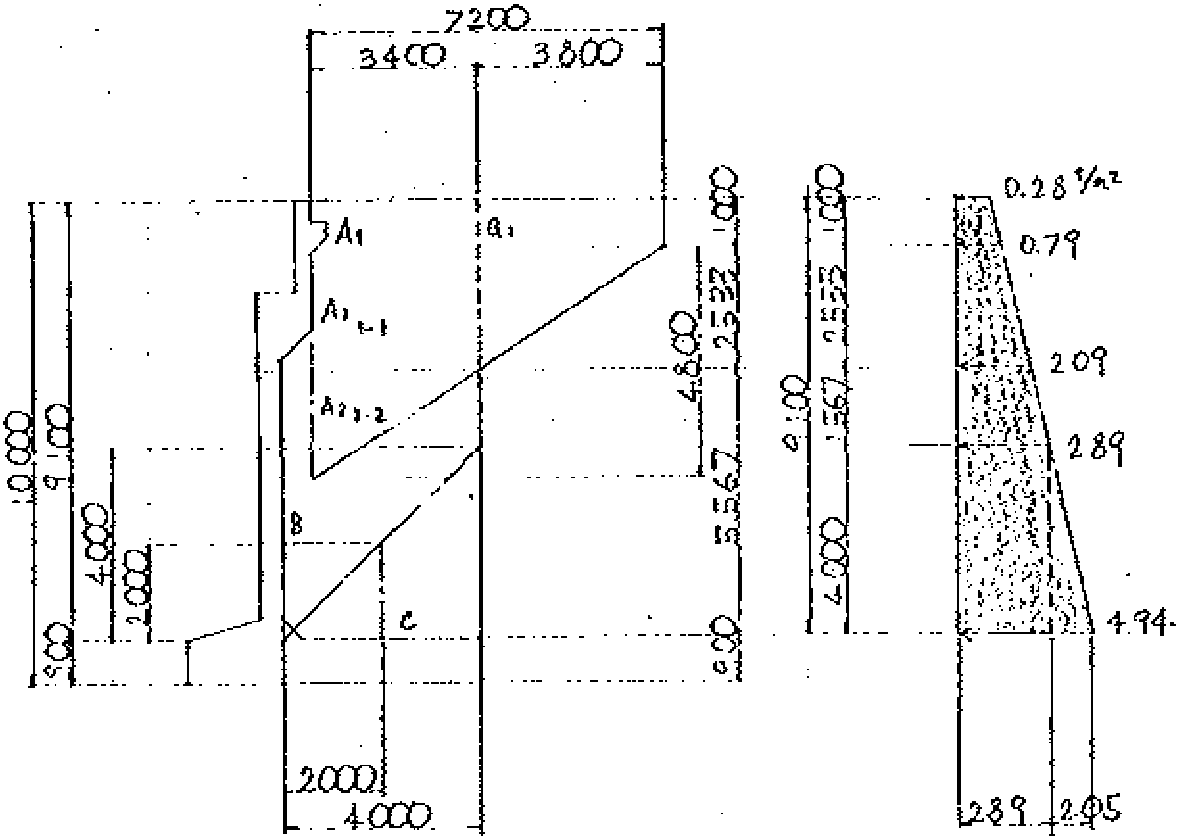
$K = 0.27$

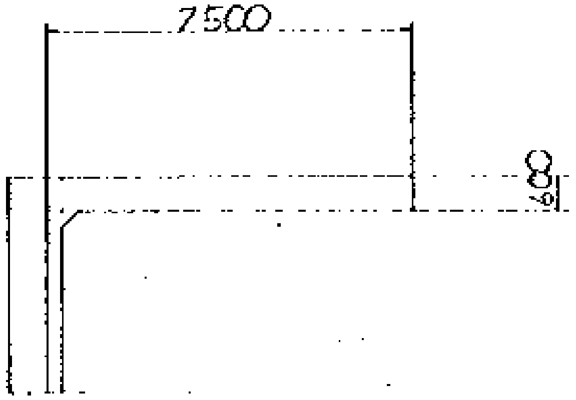
$\gamma = 1.9$ t/m³

	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	$\left(\frac{p_1}{2} + \frac{p_2}{6}\right) ly^2$	$\left(p_1 + \frac{p_2}{2}\right) \cdot ly$

(ST LOUIS A2 L)

9 – 1 dimension and loading





		z (m)	M	(t m)	S	(t)
a – a	1 – 1	1.00	$(1.02+1.9\times1.00)\times0.27\times3.80^2\times\frac{1}{2}$	5.69	$(1.02+1.9\times1.00)\times0.27\times3.80$	3.00
	2 – 2	---	---	---	---	---
A 1		0 ~3.533	$(1.02+1.9\times1.58)\times0.27\times\frac{7.20^2}{2}\times\left(\frac{5.80-1.58}{4.80}\right)^2$	21.76	$(1.02+1.9\times1.58)\times0.27\times7.20\times\left(\frac{5.80-1.58}{4.80}\right)$	6.87
A 2	1 – 1	3.533	$\frac{1}{2}\times2.09\times4.00^2$	16.72	2.09×4.00	8.36
	2 – 2	5.10	$\frac{1}{2}\times2.89\times4.00^2$	23.12	2.89×4.00	11.56
B – B		5.10 ~9.10	$\frac{1}{2}\times3.92\times2.00^2$	7.84	3.92×2.00	7.84
C – C		9.10	$\left(\frac{2.89}{2}+\frac{2.05}{6}\right)\times2.00$	3.57	$\left(2.89+\frac{2.05}{2}\right)\times2.00$	7.83

$$z^{*}=\frac{1.9\times3.533-2\times1.02}{3\times1.9}=0.82^{m}<1.00$$

$$z=\frac{1.9\times5.80-2\times1.02}{3\times1.9}=1.58^{m}$$

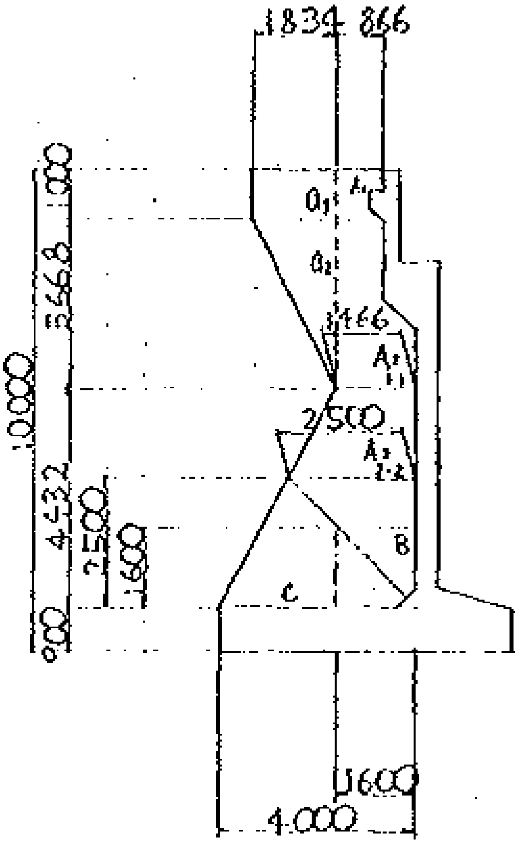
9 − 2 list of stresses σ_c, σ_s, τ : working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: Permissible stress.

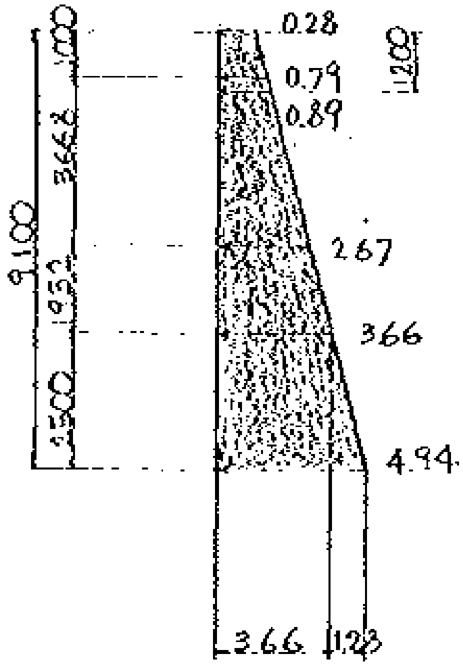
	* $A_{s\,min} = b \cdot d$ 0.15% = 7.95 cm²					
	a − a	A_1	A_2 1−1	A_2 2−2	B	C
M	5.69	21.76	16.72	23.12	7.84	3.57
N	—	—	—	—	—	—
S	3.00	6.87	8.36	11.56	7.84	7.83
b	100	—	—	—	—	→
h	53	—	—	—	—	→
d'	7	—	—	—	—	→
As	D16 @ 250 8.04	D20, D16 @125 20.60	D16 @ 125 16.08	D20 @ 125 25.12	D16 @ 250 8.04	→
As'	—	—	—	—	—	—
f/d	0	0	0	0	0	
M'/bd^2	2.03	7.75	5.95	8.23	2.79	
S/bd	0.57	1.30	1.58	2.18	1.48	
n·P	0.0226	0.0583	0.0455	0.0711	0.0228	
C	11.15	7.68	8.44	7.14	11.15	
S	46.99	18.99	24.08	15.71	46.99	
Z	1.07	1.11	1.09	1.12	1.07	
σ_c	23	60	50	59	31	
σ_s	1428	2207	2150	1939	1967	
τ	0.6	1.4	1.7	2.4	1.6	
σ_{ca}	83	—	—	—	—	→
σ_{sa}	2346	—	—	—	—	→
τ_a	2.35	—	→	3.47	2.35	→

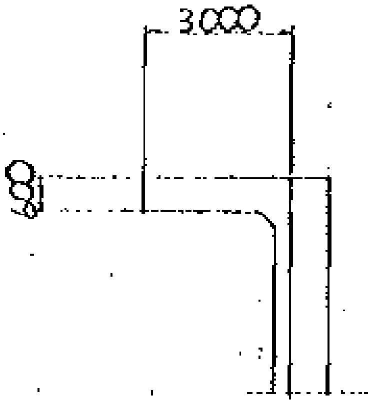
$\frac{100A_s}{bd} = 0.2$ $\frac{100A_s}{b \cdot d} = 0.5$

(ST. LOUIS A1 R)

9 – 3 dimension and loading







		z (m)	M (t·m)		S (t)	
a – a	1 – 1	1.00	$(1.02+1.9\times1.00)\times0.27\times1.834^2\times\frac{1}{2}$	1.33	$(1.02+1.9\times1.00)\times0.27\times1.834$	1.45
	2 – 2	1.00 ~4.668	$(1.02+1.9\times\overset{*}{1.20})\times0.27\times\frac{1.834^2}{2}\times\left(\frac{4.668-1.20}{3.668}\right)^2$	1.34	$(1.02+1.9\times1.20)\times0.27\times1.834\times\frac{4.668-1.20}{3.668}$	1.54
A 1		0. ~4.668	$\frac{1}{2}\times0.89\times0.866^2+1.34+1.54\times0.866$	3.01	$0.89\times0.866+1.54$	2.31
A 2	1 – 1	4.668	$\frac{1}{2}\times2.67\times1.466^2$	2.87	2.67×1.468	3.92
	2 – 2	6.60	$\frac{1}{2}\times3.66\times2.50^2$	11.44	3.66×2.50	9.15
B – B		6.60 ~9.10	$\frac{1}{2}\times4.30\times1.60^2$	5.50	4.30×1.60	6.88
C – C			$\left(\frac{3.66}{2}+\frac{1.28}{6}\right)\times1.60^2$	5.23	$\left(3.66+\frac{1.28}{2}\right)\times1.60$	6.88

$$z^* = \frac{1.9\times4.668-2\times1.02}{3\times1.9} = 1.20^m$$

9-4 list of stresses σ_c, σ_s, τ : working stress.

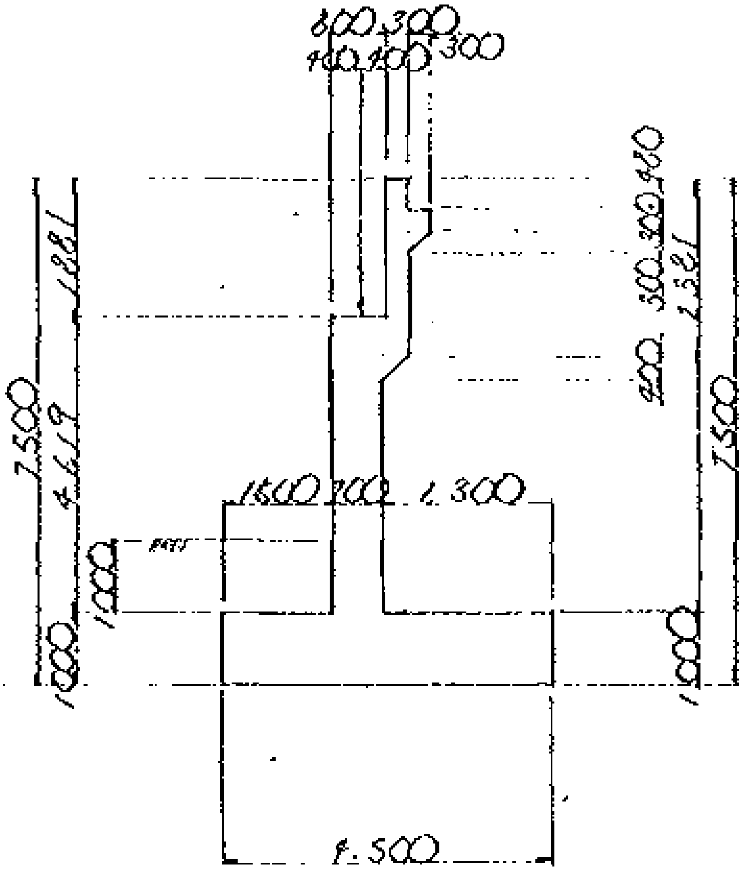
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

	※ As min = b·d·0.15% = 7.75 cm²					
	a − a	A 1	A2 1-1	A2 2-2	B	C
M	1.34	3.01	2.87	11.44	5.50	5.23
N	——	——	——	——	——	——
S	1.45	2.31	3.92	9.15	6.88	6.88
b	100	——	——	——	——	——→
h	53	——	——	——	——	——→
d'	7	——	——	——	——	——→
As	D16 @ 250 ※ 8.04	※ ——	※ ——→	D20 @ 250 12.56	D16 @ 250 8.04	——→
As'	——	——	——	——	——	——
f/d	0	0	0	0	0	0
M'/bd^2	│	│	│	4.07	1.96	│
S/bd	│	│	│	1.73	1.30	│
n·P	│	│	│	0.0355	0.0228	│
C	│	│	│	9.29	11.15	│
S	│	│	│	30.52	46.99	│
Z	│	│	│	1.08	1.07	│
σ_c	│	│	│	38	22	│
σ_s	│	│	│	1864	1380	│
τ	│	│	│	1.9	1.4	│
σ_{ca}	83	——	——	——	——	——→
σ_{sa}	2346	——	——	——	——	——→
τ_a	2.35	——	——	——	——	——→

§§ 3 ABUTMENT

$H = 7.50^{m}$

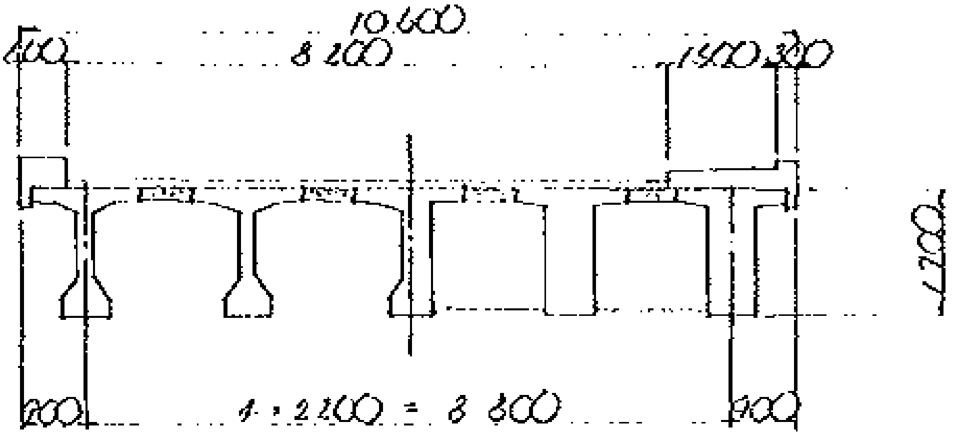
§ 1 DIMENSIONS



L = 10.600 ml

§ 2 REACTION OF SUPERSTRUCTURE

2 - 1 dimensions

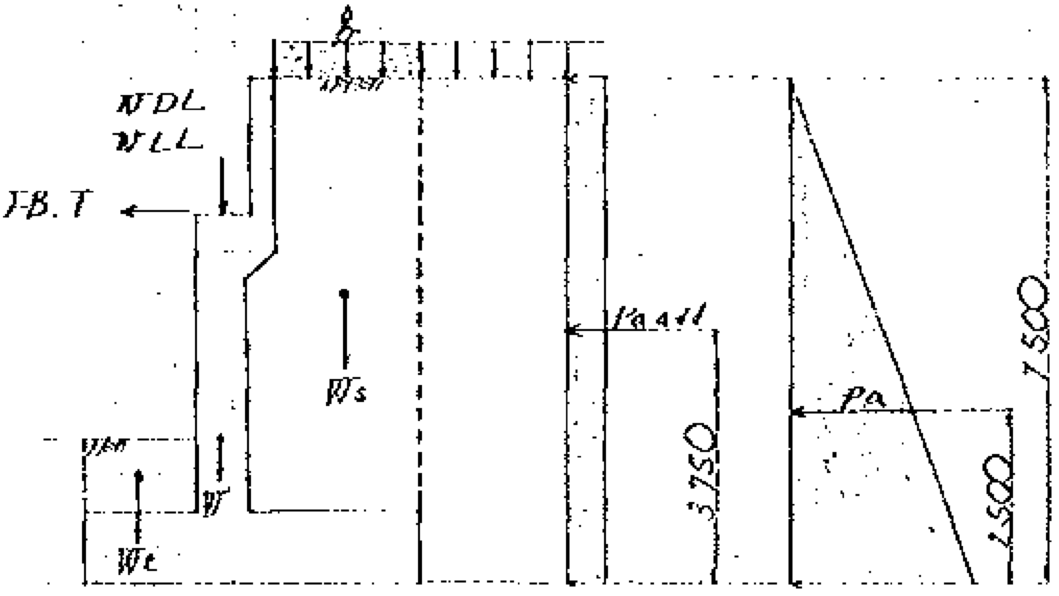


2 - 2 whole reaction due to superstructure

		HA loading		HB loading	
		N (t)	H (t)	N (t)	H (t)
dead load of deck	I	190.7	------	190.7	--- -
	II	44.5	------	44.5	------
live load		113.9	------	141.5	-- -
crowd load		------	------	------	- - -
longitudinal force		------	25.8	------	38.2
TOTAL		348.6	25.8	376.7	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

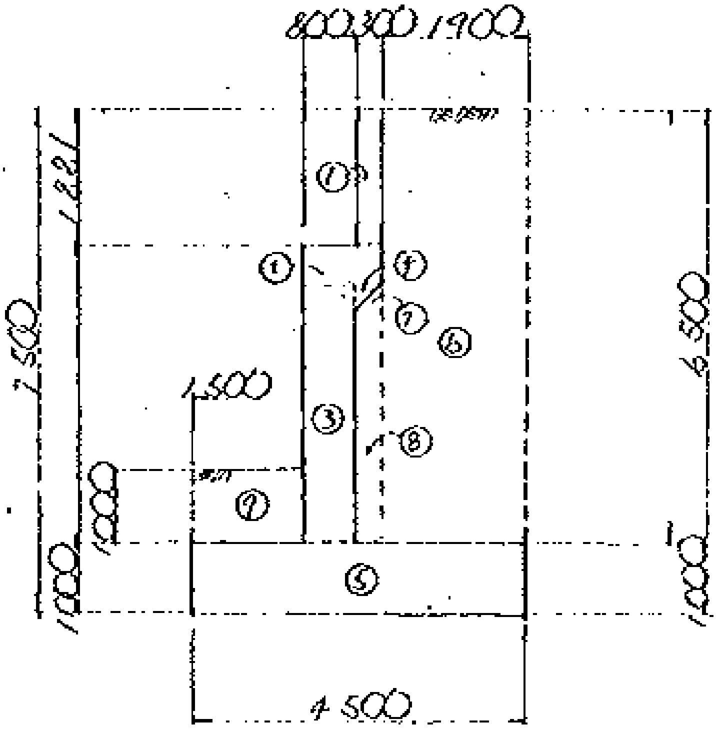
PA : active pressure

PP : passive pressure

PA add : surcharge

T : temperature load

3-2 self weight and weight of soil



		N (t)	x (m)	N·x (tm)
①	0.30×1.881×10.60×2.41	13.57	2.450	33.25
②	1.10×1.00×10.60×2.41	28.10	2.050	57.61
③	½×0.10²×10.60×2.41	0.13	2.533	0.33
④	1.00×3.619×10.60×2.41	92.45	2.000	184.90
⑤	4.50×1.00×10.60×2.41	114.96	2.250	258.66
⑥	1.90×6.50×10.60×1.9	248.73	3.550	882.99
⑦	½×0.10²×10.60×1.9	0.10	2.567	0.25
⑧	0.10×3.519×10.60×1.9	7.09	2.550	18.08
⑨	1.50×1.00×10.60×1.9	30.21	0.750	22.66
Σ		535.34		1456.71

3 - 3 weight of surcharge

under H.A $= 1.02 \times 1.90 \times 10.60 = 20.54^{t}$

under H.B $= 1.66 \times 1.90 \times 10.60 = 33.43^{t}$

3 - 4 earth pressure

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

angle of internal friction $\phi = 35°$

(1) active pressure

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

$$= 1/2 \times \frac{1 - \sin 35°}{1 + \sin 35°} \times 1.9 \times 7.50^2 \times 10.60 = 152.94^{t}$$

(2) active pressure due to surcharge

under H.A surcharge

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

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

$= 0.27 \times 1.02 \times 7.50 \times 10.60 = 21.89^{t}$

under H B surcharge

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

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

$= 0.27 \times 1.66 \times 7.50 \times 10.60 = 35.63^{t}$

3 - 5 temperature load

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

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

$$\left[\begin{array}{l} \delta = 0.7 \times 26.80 = 18.76^{mm} \\ R(d\cdot l) = 376.7 \times 1/5 \times 1.4 = 105.48^{t} \end{array}\right]$$

δ: elongation of a girder

RING SHOE

110 TON

$D\phi$: 54^{cm} (diameter)

$d\phi$: 32^{cm}

A : 2289^{cm^2} (Area)

t : 7.1^{cm} (thickness)

G_0 : $13.5^{kg/cm^2}$ (modulus of rigidity)

$$P_H = \frac{13.5 \times 2289 \times 1.87}{7.1} = 8139^{kg} = 8.13^{t}$$

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

$$= 5 \times 8.13 \times 1/2 = 20.33^{t}$$

§ 4 CALCULATION OF STABILITY

case 1 HA loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL. WLL	348.60	1.900	662.34	-------	-----	-- ---
F. B	-----	-----	-------	25.80	5.619	144.97
T	------	-----	------	20.33	5.619	114.23
W. Ws. Wt	535.34	----	1458.71	-------	----	------
Ws add	20.54	3.550	72.92	------	----	-----
P a	------	----	-------	154.94	2.500	387.35
Pa add	----	-----	----	21.89	3.750	82.09
TOTAL	904.48		2193.97	222.96		728.64

1) check for eccentric

$$x = \frac{\Sigma Nx - \Sigma HY}{\Sigma N} = \frac{2193.97 - 728.64}{904.48} = 1.62^{m}$$

$$e = \frac{B}{2} - x = \frac{4.500}{2} - 1.62 = 0.63^{m} < \frac{B}{6} = 0.75^{m}$$

2) soil reaction

$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{904.48}{4.50 \times 10.60} \times \left(1 \pm \frac{6 \times 0.63}{4.50}\right)$$

$$= \begin{cases} 34.89 \ {}^{t}/m^{2} \\ 3.03 \end{cases} < 50 \ {}^{t}/m^{2}$$

3) check for sliding

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

$$F = \frac{Hu}{\Sigma H} = \frac{904.48 \times 0.6}{222.96} = 2.43 > Fa = 1.5$$

case 2 HB loading

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL.WLL	376.70	1.900	715.73	----	----	----
F B	----	----	----	38.20	5.619	214.65
T	----	----	----	20.33	5.619	114.23
W.Ws.Wt	535.34	----	1458.71	----	----	----
Ws add	33.43	3.550	118.68	----	----	----
Pa	----	----	----	154.94	2.500	387.35
Pa add	----	----	----	35.63	3.750	133.61
TOTAL	945.47		2293.12	249.10		849.84

1) check for eccentric

$$x = \frac{\Sigma N x - \Sigma H y}{\Sigma N} = \frac{2293.12 - 849.84}{945.47} = 1.53^{m}$$

$$e = \frac{B}{2} - x = \frac{4.50}{2} - 1.53 = 0.72^{m} < \frac{B}{3} = 1.50^{m}$$

2) soil reaction

$$q = \frac{945.47}{4.50 \times 10.60} \times \left(1 \pm \frac{6 \times 0.72}{4.50}\right) = \begin{cases} 38.85 \ t/m^2 \\ 0.79 \end{cases} , < 62 \ t/m^2$$

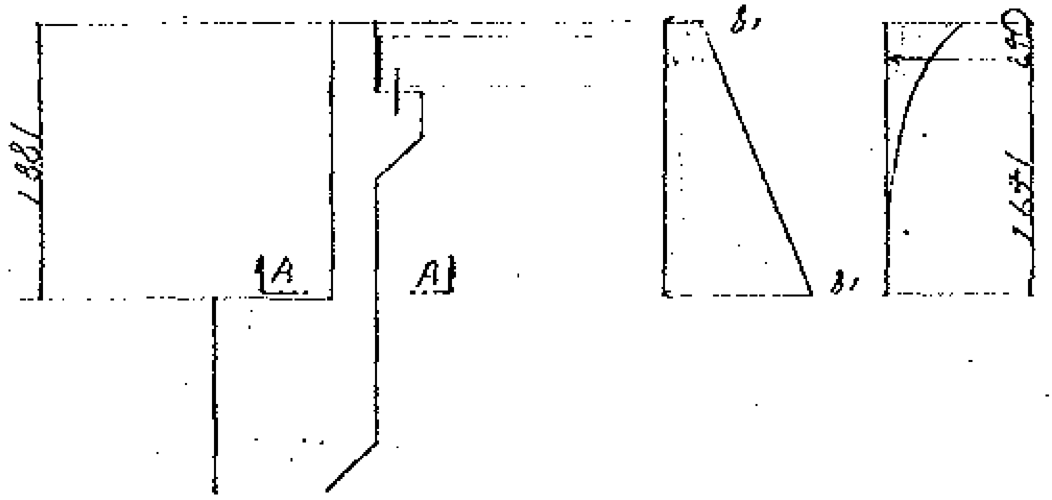
3) check for sliding

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

$$F = \frac{Hu}{\Sigma H} = \frac{0.6 \times 945.47}{249.10} = 2.28 \quad > Fa = 1.2$$

§ 5 CALCULATION OF PARAPET SECTION

5-1 dimension and loading



	q_1	q_2
HA loading	0.18	1.24
HB loading	0.45	1.41

$q_1 = q \cdot K \quad = 0.27 \cdot q \quad {}^{t}/_{m^2}$

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

5 - 2 sectional force of parapet

CASE. 1 (HA)

$S = 1/2 \times (0.28 + 1.24) \times 1.881 = 1.43^{t}$

$M = 1.43 \times 1/3 \times 1.881 \times \frac{2 \times 0.28 + 1.24}{0.28 + 1.24} = 1.06^{tm}$

CASE 2 (HB)

$S = 1/2 \times (0.45 + 1.41) \times 1.881 = 1.75^{t}$

$M = 1.75 \times 1/3 \times 1.881 \times \frac{2 \times 0.45 + 1.41}{0.45 + 1.41} = 1.36^{tm}$

CASE 3

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

$M = 2.96 \times 1.641 = 4.86^{tm}$

5 - 3 list of stresses σ_c, σ_s, τ : working stress.

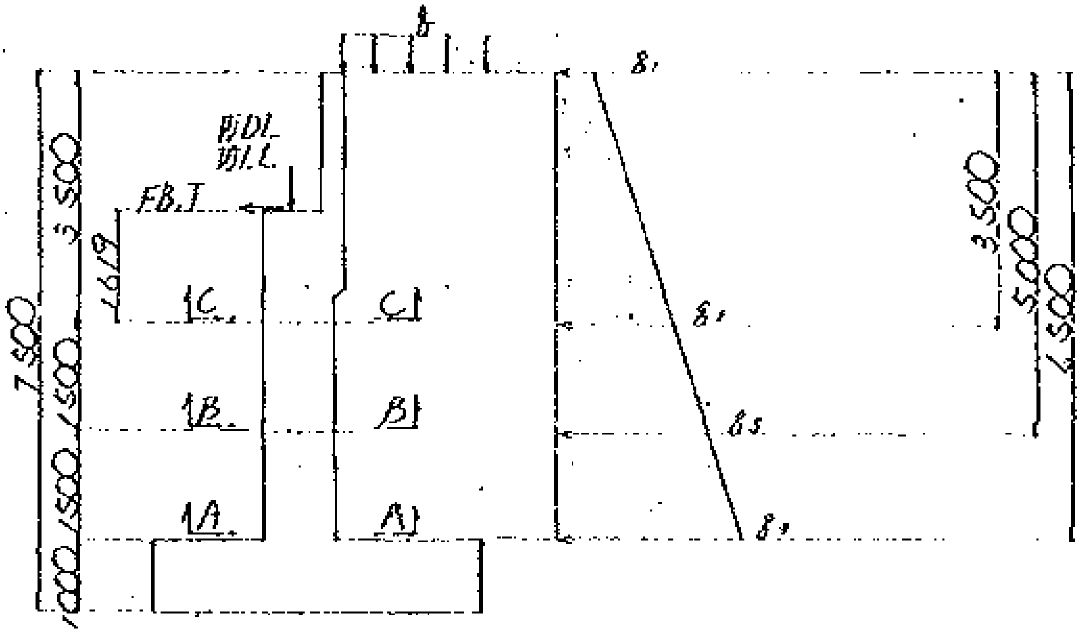
$\sigma_{ca}, \sigma_{sa}, \tau_a$: Permissible stress.

※ $A_{s\,min} = 100 \times 23 \times 0.0015 = 3.95\ cm^2$

	case 1	case 2	case 3			
M	1.06	1.36	4.86			
N	----	----	—			
S	1.43	1.75	2.56			
b	100	100	100			
h	23	23	23			
d'	7	7	7			
As	D16@250 8.04	--------	-------→			
As'	----	----	----			
f/d	|	|	0			
M'/bd^2	|	|	9.19			
S/bd	|	|	1.29			
n·P	|	|	0.1049			
C	|	|	6.24			
S	|	|	10.86			
Z	|	|	1.14			
σ_c	|	|	57			
σ_s	|	|	1497			
τ	|	|	1.5			
σ_{ca}	83	--------	-----→			
σ_{sa}	2346	2933	2346			
τ_a	2.35	2.94	2.35			

§ 6 CALCULATION OF WALL SECTION

6 – 1 dimension and loading



	N (t)	H (t)
dead load of deck	22.19	
HA live load	10.70	4.35
HB live load	13.35	5.52

	q1	q2	q3	q4
HA loading	0.18	1.08	1.85	3.61
HB loading	0.95	1.15	3.01	3.78

6 - 2 sectional force of wall

section 1 - 1

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	22.19	4.35	1.619	7.04
self weight	5.51	----	----	----
earth pressure	----	4.13	1.305	5.39
TOTAL	27.70	8.48		12.43

section 2 - 2

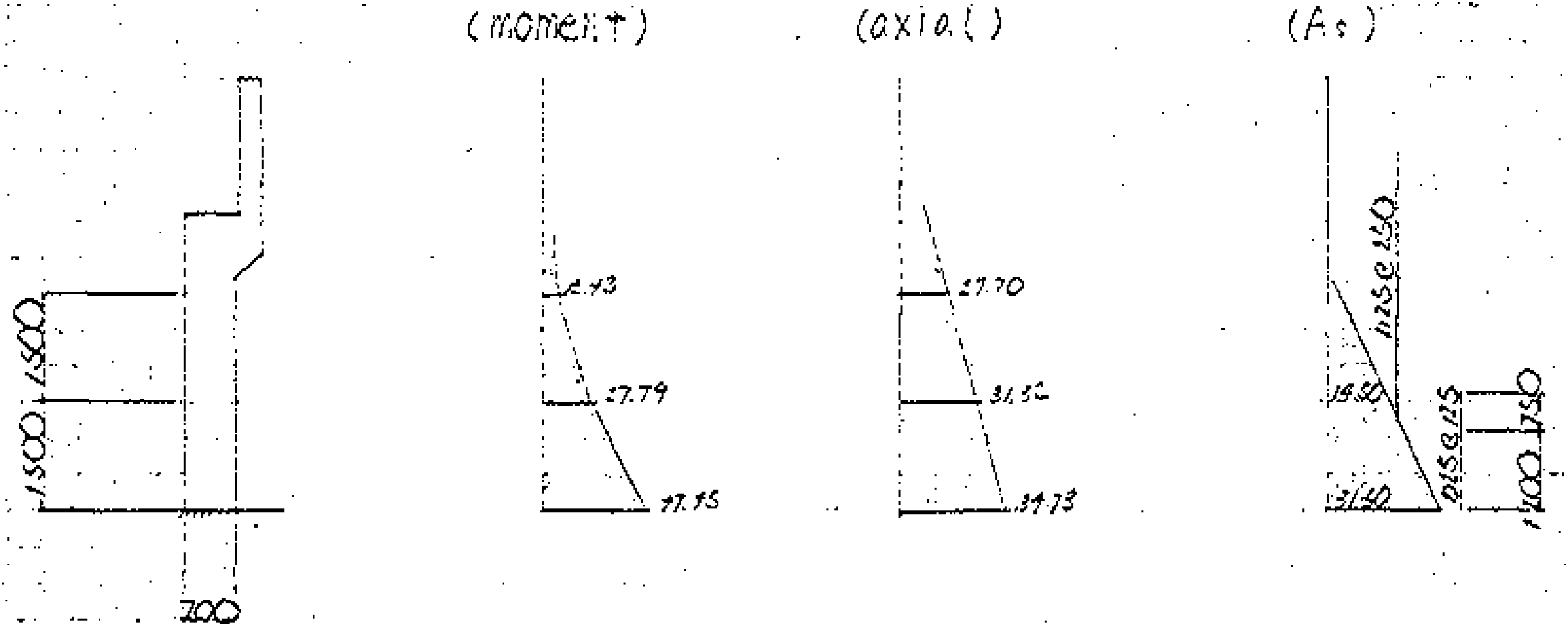
	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	22.19	4.35	3.119	13.57
self weight	9.13	----	----	----
earth pressure	----	7.83	1.816	14.22
TOTAL	31.32	12.18		27.79

section 3 - 3

	N (t)	H (t)	y (m)	H·y (tm)
reaction of superstructure	22.19	4.35	4.619	20.09
self weight	12.74	----	----	----
earth pressure	----	12.64	1.323	19.36
TOTAL	34.93	16.99		49.45

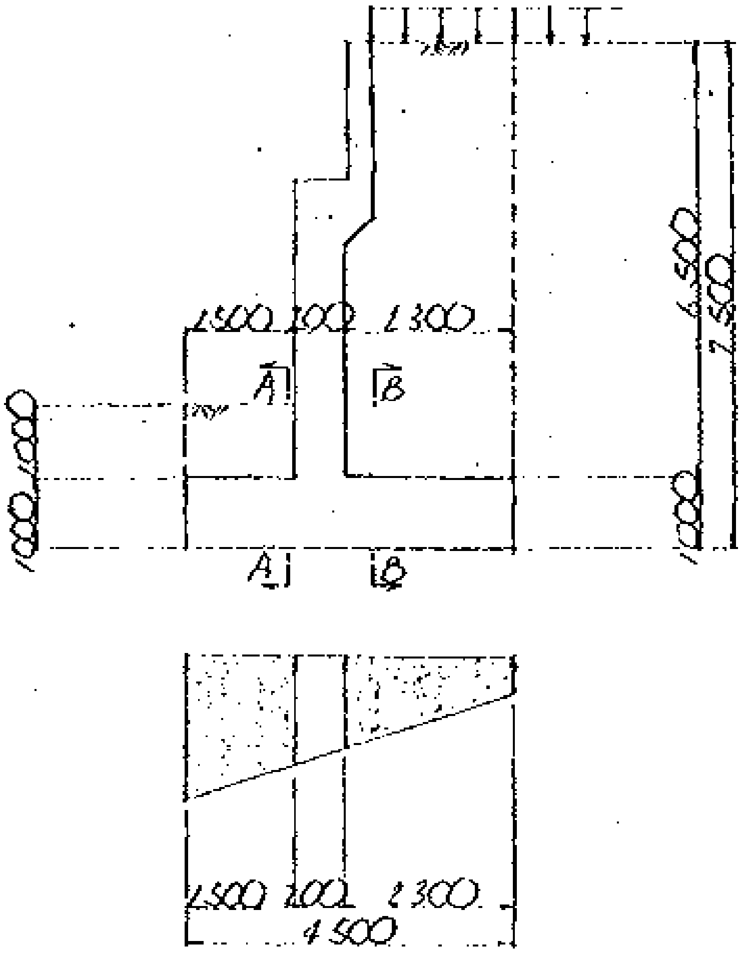
6-3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

	A-A	B-B	C-C			
M	47.15	27.19	12.93			
N	34.93	31.32	22.70			
S	16.99	12.18	6.48			
b	100	——	——→			
h	63	——	——→			
d'	7	——	——→			
As	D25@125 39.28	D25@250 19.64	——→			
As'	——	——	——			
f/d	2.69	1.85				
M'/bd^2	19.92	9.21				
S/bd	2.70	1.93				
np	0.0935	0.0468				
C	5.70	6.68				
S	8.39	12.61				
Z	1.1	2.02				
σ_c	82	61				
σ_s	1878	1742				
τ	3.0	2.1				
σ_{ca}	83	——→	——→			
σ_{sa}	2346	——→	——→			
τ_a	3.47	2.35				


1500
1500
200
(moment)
2.43
27.79
77.95
(axial)
27.70
31.52
34.73
(As)
D25@250
19.50
31.40
D15@125
1400
750

§ 7 CALCULATION OF FOOTING SECTION

7 - 1 dimension and loading



	q 1	q 2	q 3	q 4
HA loading	34.89	24.22	17.19	3.03
HB loading	38.65	26.16	17.71	0.79

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 × (34.89 + 24.27) × 1.50	- 44.37	0.795	- 35.27
Σ		37.90		30.42

$$\begin{cases} M = 30.42^{tm} \\ S = 37.90^{t} \end{cases}$$

CASE 2

		S	x	M
Wt	———	2.85	———	2.14
W	———	3.62	———	2.71
q (HA)	-½ × (38.85 + 26.16) × 1.50	- 48.76	0.799	- 38.96
Σ		42.29		34.11

$$\begin{cases} M = 34.11 \times 1/1.25 = 27.29^{tm} \\ S = 42.29 \times \ " \ = 33.83^{t} \end{cases}$$

section B - B

CASE 1

		S	x	M
Ws	2.00 × 6.50 × 1.9	24.70	1.000	24.70
W	2.00 × 1.00 × 2.41	4.82	1.000	4.82
Ws add	2.00 × 1.02	2.04	1.000	2.04
q(HA)	½ × (17.19 + 3.03) × 2.00	- 20.22	0.767	- 15.09
Σ		11.34		16.47

$$\left\{ \begin{array}{l} M = 16.47 \ ^{tm} \\ S = 11.34 \ ^{t} \end{array} \right.$$

CASE 2

		S	x	M
Ws	-------	24.70	------	24.70
W	-------	4.82	------	4.82
Ws add	2.00 × 1.66	3.32	1.000	3.32
q(HA)	½ × (12.21 + 6.29) × 2.00	- 18.50	0.695	- 12.86
Σ		14.34		19.98

$$\left\{ \begin{array}{l} M = 19.98 \times 1/1.25 = 15.98 \ ^{tm} \\ S = 14.32 \times \quad = 11.46 \ ^{t} \end{array} \right.$$

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

	A - A	B - B				
M	30.92	16.47				
N	——	——				
S	37.90	11.46				
b	100	100				
h	90	90				
d'	10	10				
As	D16 ct 125 16.08	D16 ct 125 ※ 16.08	As min =	bd · 0.15%	= 13.50 cm²	
As'	——	——				
f/d	0					
M'/bd²	3.76					
S/bd	4.21					
n·P	0.0268					
C	10.92					
S	40.10					
Z	1.07					
σc	39					
σs	2259					
τ	4.5					
σca	83	→				
σsa	2396	→				
τa	2.35	→				

check for stirrups

$$\tau = \frac{Sh}{b \cdot d} \times k$$

$$= \frac{37.90 \times 10^3}{100 \times 90} \times 1.07 = 4.51 \ ^{kg/cm^2} > \tau_a = 2.35$$

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

$$Sh' = S \cdot h - S_c$$

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

$$= 2.35 \times 100 \times 90 \times \frac{1}{1.07} = 19.770$$

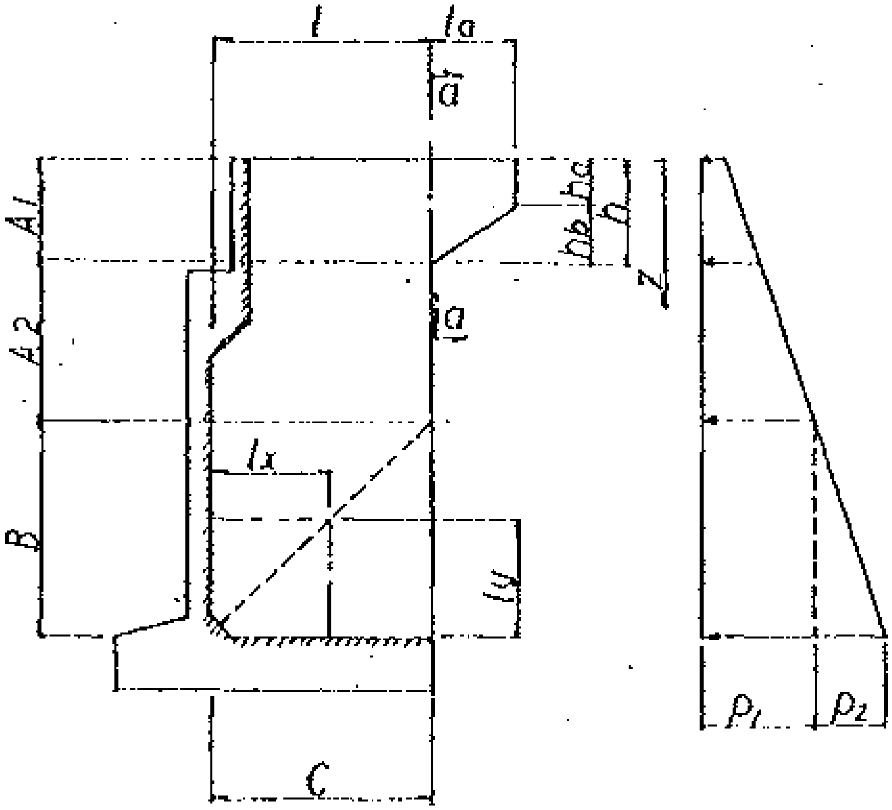
$$Sh' = (37.90 - 19.77) \times 10^3 = 18.13 \times 10^3$$

$$req.\ A_w = \frac{18.13 \times 10^3 \times 25}{1780 \times 90} \times 1.07 = 3.03 \ ^{cm^2}$$

ϕ 16 - ctc 250 $n = 2$

Used $A_w = 2.01 \times 2 = 4.02 \ ^{cm^2} > req.\ A_w = 3.03$

§ 8 CALCULATION OF WING SECTION



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

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

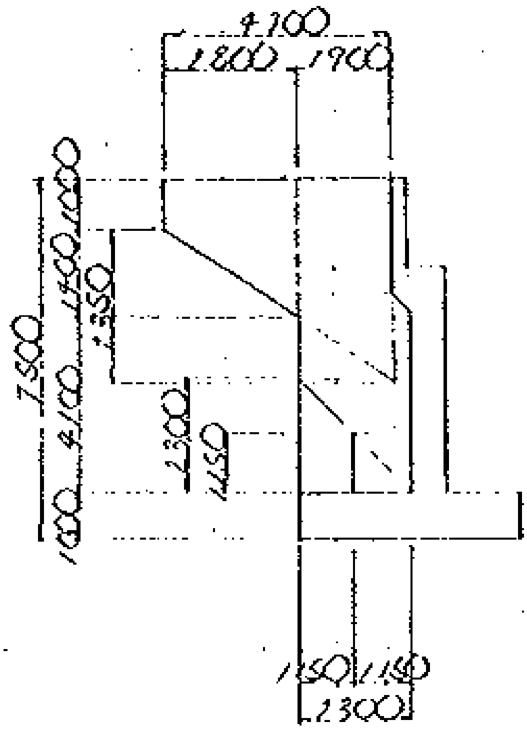
$q = 1.02$ t/m²

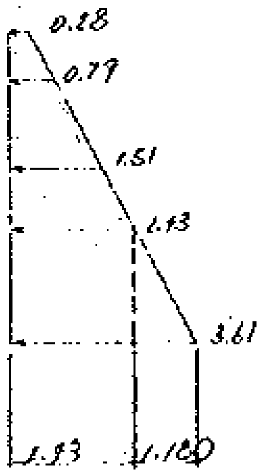
$K = 0.27$

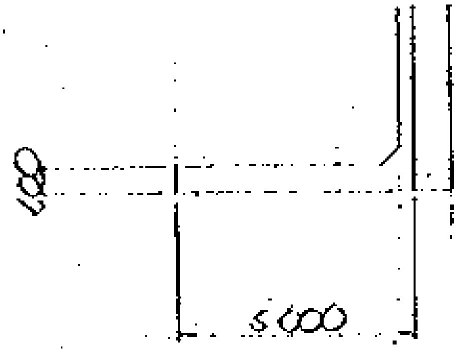
$\gamma = 1.9$ t/m³

	(t) M	(tm) S
A1 − 1	$\frac{1}{2} p \cdot l^2 + M_a + S_o \cdot l$	$p \cdot l + S_a$
A2 − 2	$\frac{1}{2} \cdot p \cdot l^2$	$p \cdot l$
B − B	$\frac{1}{2} \cdot p \cdot l_x^2$	$p \cdot l_x$
C − C	$(\frac{p_1}{2} + \frac{p_2}{6}) l_y^2$	$(p_1 + \frac{p_2}{2}) \cdot l_y$

8 - 1 dimension and loading







		Z (m)	M (tm)		S (t)	
a-a	1 − 1	1.00	$\frac{1}{2}\times 0.79\times 2.80^2$	3.10	0.79×2.80	2.21
	2 − 2	1.00 ~2.40	―――	――	―――	――
A 1		0 ~2.40	$\frac{1}{2}\times 0.79\times 4.70^2$	8.73	0.79×4.70	3.71
A 2	1 − 1	2.40	$\frac{1}{2}\times 1.51\times 2.30^2$	3.99	1.51×2.30	3.47
	2 − 2	4.20	$\frac{1}{2}\times 2.43\times 2.30^2$	6.43	2.43×2.30	5.58
B − B		4.20 ~6.50	$\frac{1}{2}\times\frac{2.43+3.61}{2}\times 1.15^2$	2.00	$\frac{2.43+3.61}{2}\times 1.15$	3.47
C − C		6.50	$(\frac{2.43}{2}+\frac{1.18}{6})\times 1.15^2$	1.87	$(2.43+\frac{1.18}{2})\times 1.15$	3.47

$$Z_1 = \frac{1.9\times 2.40-2\times 1.02}{3\times 1.9} = 0.44 < 1.00^{*1}$$

$$Z_2 = \frac{1.9\times 3.35-2\times 1.02}{3\times 1.9} = 0.76 < 1.00^{*1}$$

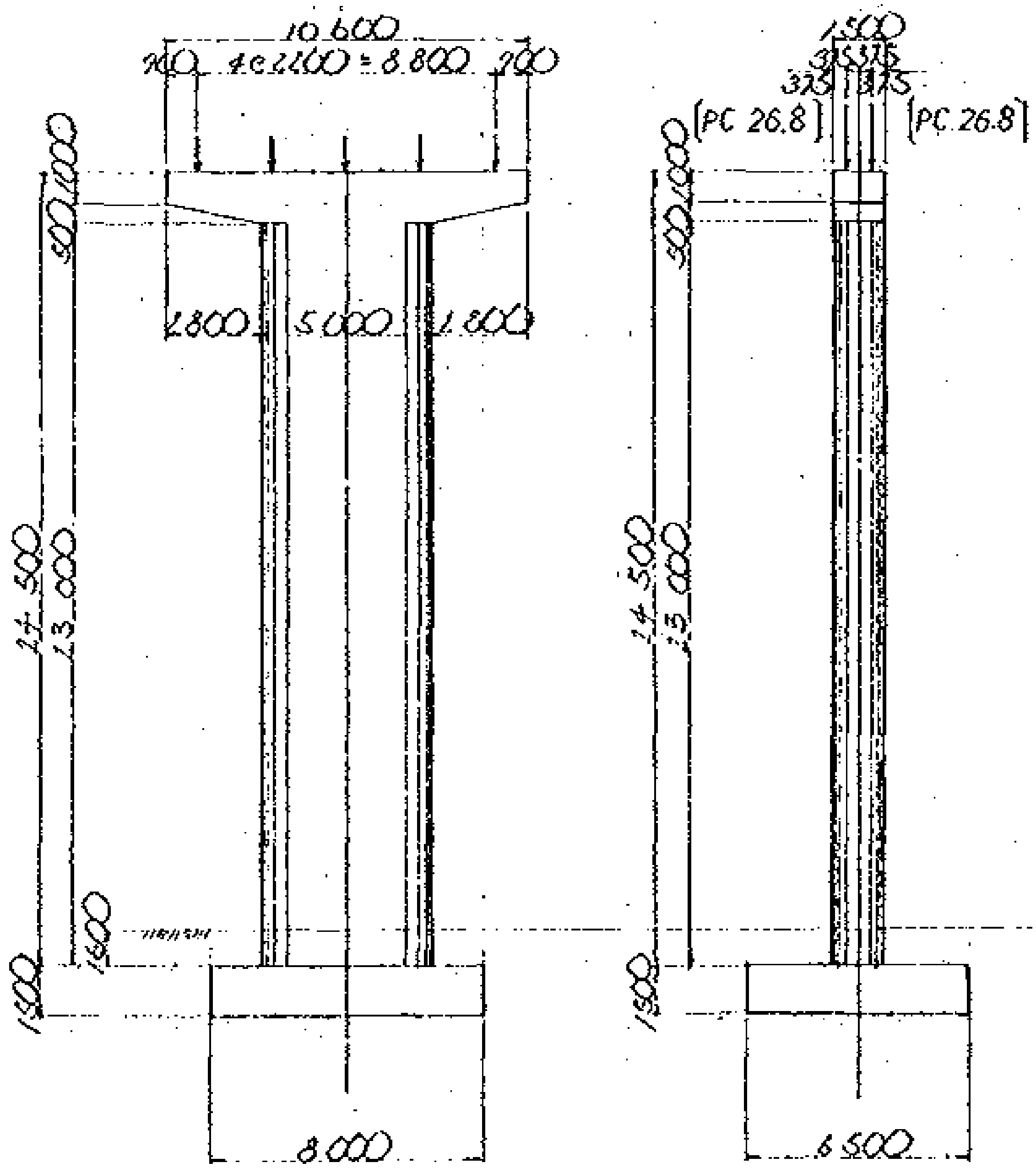
8 - 2 list of stresses $\sigma c, \sigma s, \tau$: working stress.
$\sigma ca, \sigma sa, \tau a$: Permissible stress.

	※As min = 100×53×0.0015 = 7.95 cm²					
	a - a	A_1	A_2	B	C	
M	3.10	8.73	6.43	6.00	1.87	
N	—	—	—	—	—	
S	2.21	3.71	5.58	3.97	3.97	
b	100	→				
h	53	→				
d'	7	→				
As	D16@150 ※ 8.04	→	※	※	※ →	
As'		—				
f/d		0				
M'/bd^2		3.11				
S/bd		0.70				
n·P		0.0228				
C		11.15				
S		16.97				
Z		1.07				
σc		35				
σs		2190				
τ		0.7				
σca	80	→				
σsa	2346	→				
τa	2.35	→				

§§ 4 PIER

H = 26.00 m

§ 1 DIMENSIONS



§ 2 REACTION DUE TO SUPERSTRUCTURE

2-1 whole reaction due to superstructure

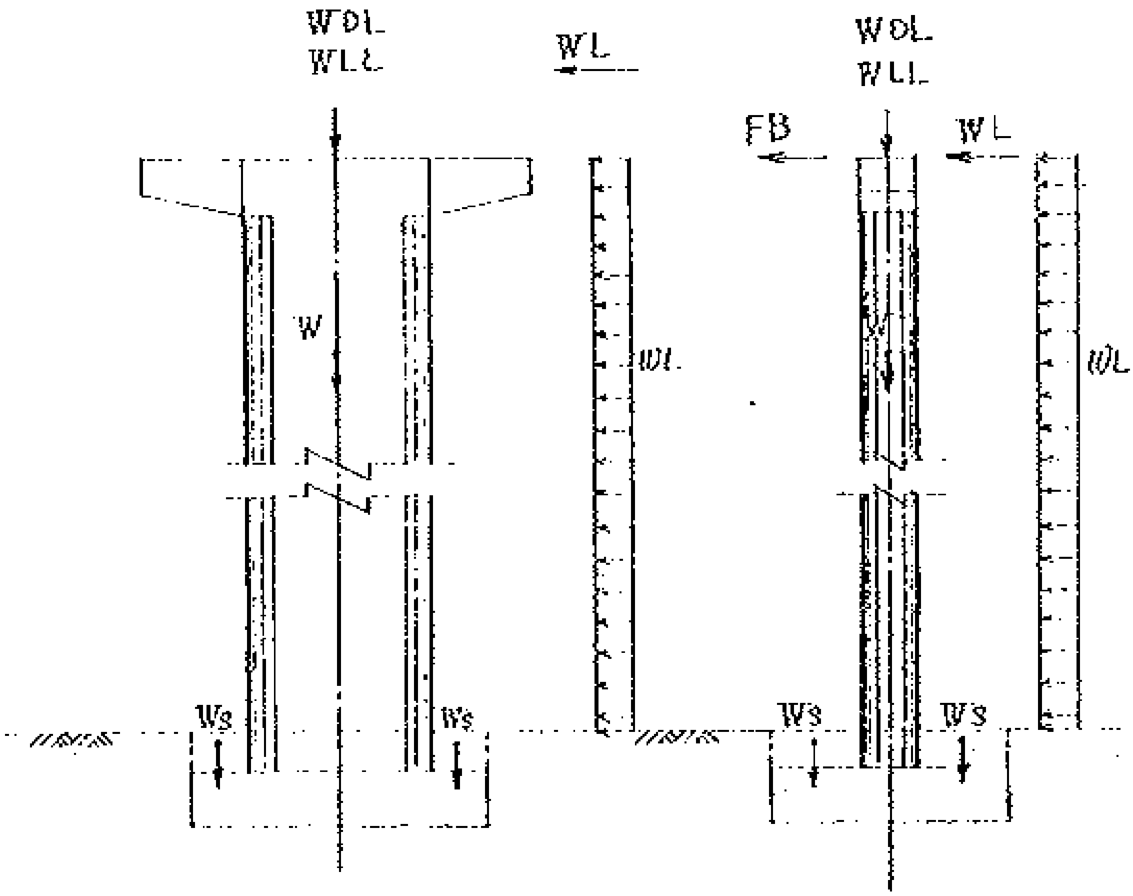
dead load of deck		235.2	235.2
H. A Live load		113.4	82.4
H.B Live load		82.5	78.9
crowd load		10.3	10.3
longitudinal forces	under H.A	12.9	
	under H.B	19.1	

2-2 reaction per each girder

		G 1	G 2	G 3	G 4	G 5
dead load of deck		83.8	92.4	95.2	96.2	103.0
live load	H A	59.2	58.6	55.8	57.1	48.6
	H B	63.4	58.7	57.0	62.7	33.4
TOTAL	H A	143.0	151.0	151.0	153.3	151.6
	H B	147.2	152.1	155.0	163.9	151.0

§ 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 or traction

W : self weight

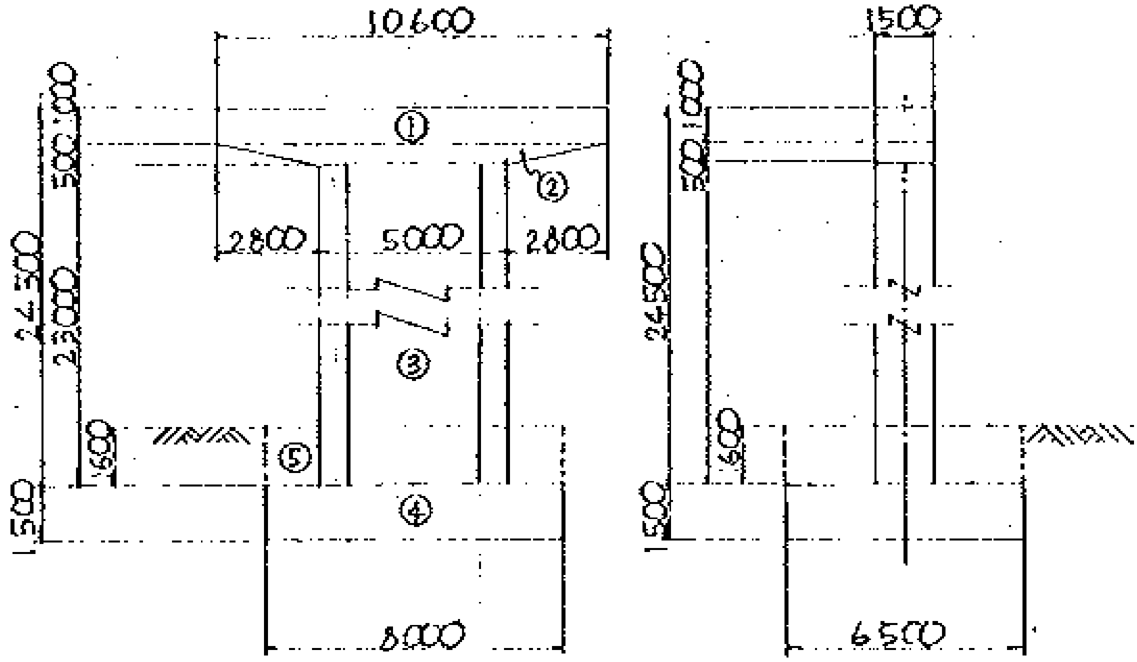
Ws : weight of soil

WL : wind load on the superstructure

wL : wind load on the pier

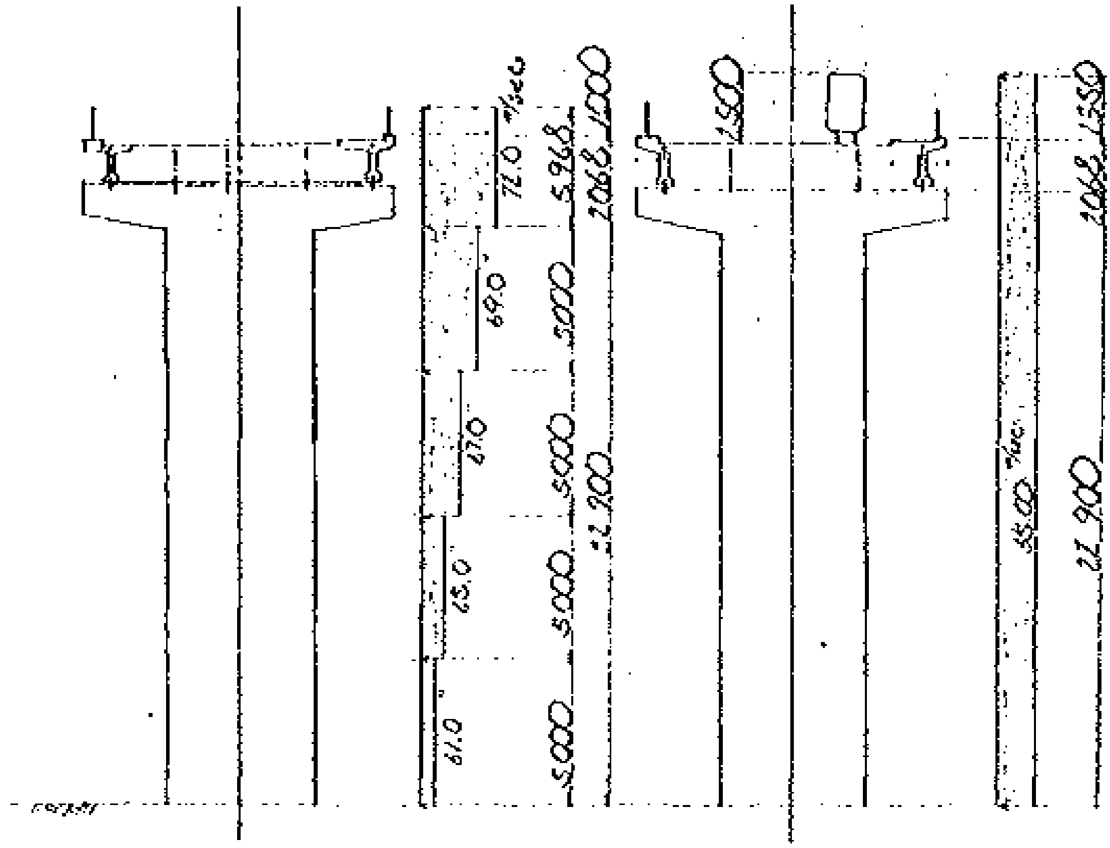
B : buoyancy

3 - 2 self weight and weight of soil



		N (t)
①	$10.60 \times 1.00 \times 1.50 \times 2.41$	38.32
②	$\frac{1}{2} \times (5.00 + 10.60) \times 0.50 \times 1.50 \times 2.41$	14.10
③	$(\pi/4 \times 1.50^2 + 3.50 \times 1.50) \times 23.00 \times 2.41$	388.96
④	$8.00 \times 6.50 \times 1.50 \times 2.41$	187.98
⑤	$\{8.00 \times 6.50 - (\pi/4 \times 1.50^2 + 1.50 \times 3.50)\} \times 1.60 \times 1.9$	136.75
		766.11

3-3 wind pressure



wind gust speed

$$V_c = v \cdot k_1 \cdot S_1 \; S_2$$

case 1 (without live load)

$$V_c = 38 \times 1.0 \times 1.1 \times S_1 = 41.8 \cdot S_1 \; {}^{m}/_{sec}$$

case 2 (with live load)

$$V_c = 35.00 \; {}^{m}/_{sec}$$

(A) transverse wind load

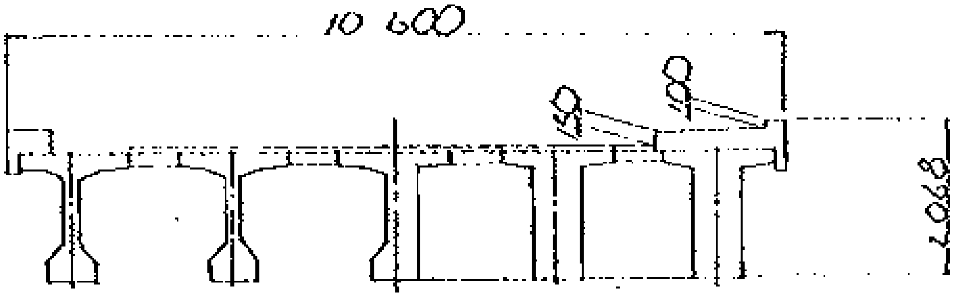
(A-1) for superstructures

$P_t = q \cdot A \cdot C_d$ (t)

$q = 0.613 \cdot V_c^2 \times 0.102$ (kg/m^2)

C_d : drag coefficient

A : loading area (m^2)



case 1

$q = 0.613 \times 72^2 \times 0.102 \times 10^{-3} = 0.32\ t/m^2$

$A = 2.068 \times 27.55 = 56.97\ m^2$

$C_d = 1.38 \quad (b/d = 10.60/2.068 = 5.13)$

$P_t = 0.32 \times 56.97 \times 1.38 = 25.16\ t$

case 2

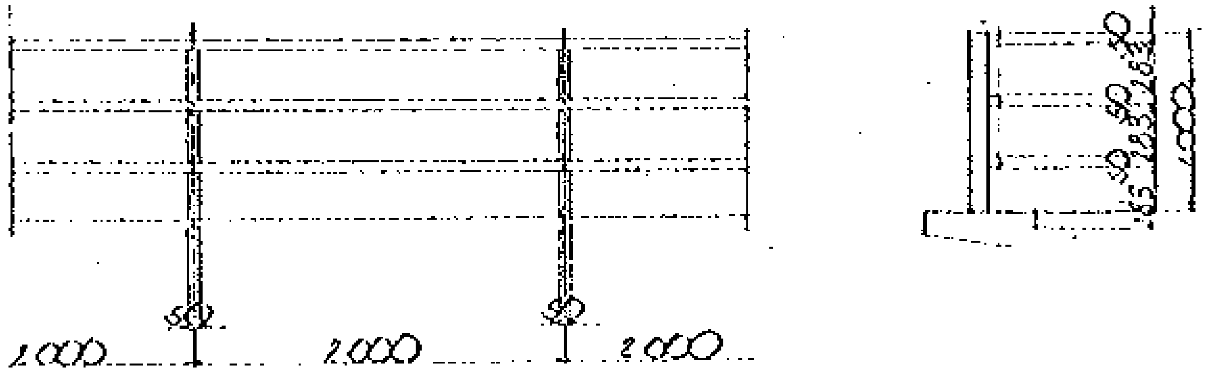
$q = 0.613 \times 35^2 \times 0.102 \times 10^{-3} = 0.077\ t/m^2$

$A = 27.55 \times 4.418 = 121.72$

$C_d = 1.39 \quad (b/d = 10.60/2.50 = 4.24)$

$P_t = 0.077 \times 121.72 \times 1.39 = 13.03\ t$

(A - 2) for parapet



case 1

$$A = 0.05 \times 3 \times 27.50 + (1.00 - 0.05 \times 3) \times \frac{27.50}{2.00}$$

$$= 15.81 \ ^{m^2}$$

$$q = 0.613 \times 72.0^2 \times 0.102 \times 10^{-3} = 0.32 \ ^{t}/_{m^2}$$

$C_d = 1.1$ (from table 8)

$$P_t = 0.32 \times 15.81 \times 1.1 = 5.57^{t}$$

(A-3) for substructure

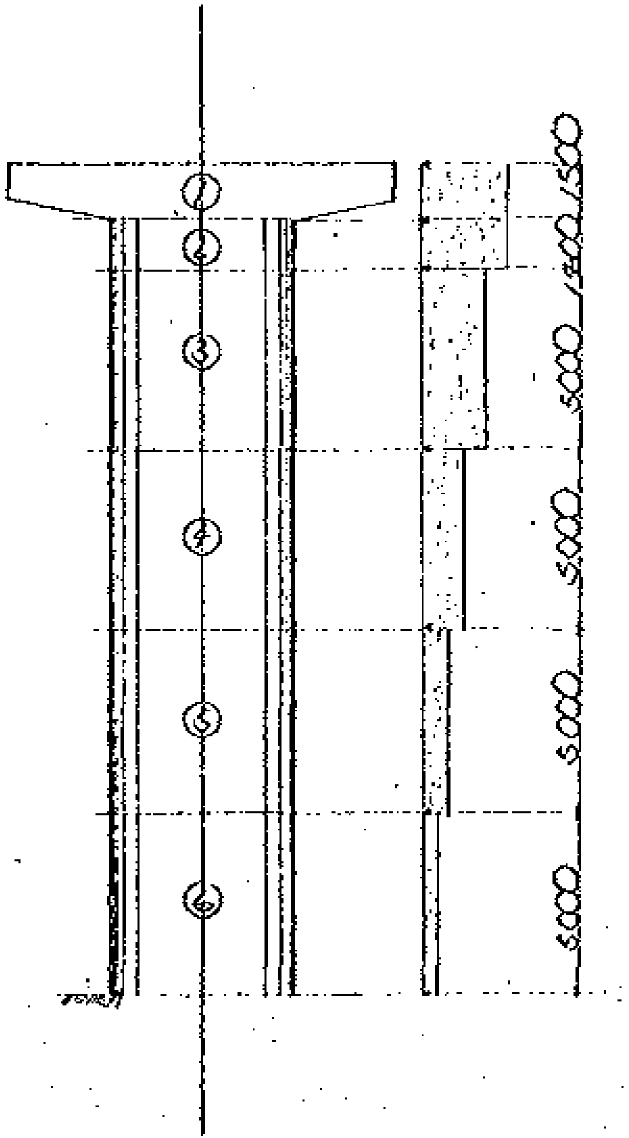
case 1

$$V_c = v \cdot k_1 \cdot s_1 \cdot s_2$$

$$= 38 \times 1.0 \times 1.1 \times s_2 = 41.8 \cdot s_2 \quad \text{m/sec}$$

$$q = 0.613 \cdot V_c^2 \times 0.102 \times 10^{-3} = 0.1092 \cdot (s_2)^2 \quad \text{t/m}^2$$

$$P_t = q \cdot A \cdot C_d$$



	V_c (m/sec)	q (t/m²)	t/b	Cd
①	72.0	0.32	>4	1.1
②	72.0	0.32	3	1.2
③	67.0	0.30	↓	↓
④	67.0	0.28		
⑤	65.0	0.26		
⑥	61.0	0.23		

	A (m^2)		P_t (t)	y (m)	$P_t y$ (tm)
①	1.50 × 1.50	2.25	0.79	25.250	19.95
②	1.50 × 1.40	2.10	0.81	23.800	19.28
③	1.50 × 5.00	7.50	2.70	20.600	55.62
④	〃	7.50	2.52	15.600	39.31
⑤	〃	7.50	2.34	10.600	24.80
⑥	〃	7.50	2.07	5.600	11.59
Σ		34.35	11.23		170.55

case – 2

$V_c = 35\ {}^{m}/_{sec}$

$q = 0.613 \times 35^2 \times 0.102 \times 10^{-3} = 0.077\ {}^{t}/_{m^2}$

$A = 34.35^{m^2}$

$C_d = (2.25 \times 1.1 + 32.10 \times 1.2) \times 1/34.35 = 1.19$

$P_t = 0.077 \times 34.35 \times 1.19 = 3.15^{t}$

$y = 13.110^{m}$

(B) longitudinal wind load

(B - 1) for superstructure

case 1

$$P_{LS} = 0.25 \cdot P_t$$

$$= 0.25 \times 25.16 \quad = 6.29^{t}$$

case 2

$$P_{LS} = 0.25 \cdot P_t$$

$$P_t = q \cdot A \cdot C_d \,, \quad A = l \cdot d$$

$$P_{LS} = 0.25 \times 0.077 \times (27.50 \times 2.068) \times 1.38$$

$$= 1.51^{t}$$

$$P_{LL} = 0.5 \cdot P_t$$

$$P_t = q \cdot A \cdot C_d \qquad A = 2.50 \cdot l \qquad C_d \geqq 1.45$$

$$P_{LL} = 0.5 \times 0.077 \times (2.50 \times 27.50) \times 1.45$$

$$= 3.84^{t}$$

(B - 2) for palapet

for vertical members

$PL1 = 0.8 \cdot Pt$

$Pt = q \cdot A \cdot Cd \qquad Cd = 1.1$

$A = 0.05 \times (1.00 - 0.05 \times 3) \times 27.50/2.00 = 0.58^{m^2}$

$PL1 = 0.8 \times 0.32 \times 0.58 \times 1.1 = 0.16^{t}$

for horizontal members

$PL2 = 0.4 \cdot Pt$

$Pt = q \cdot A \cdot Cd \qquad Cd = 1.1$

$A = 27.50 \times 0.05 \times 3 = 4.13^{m^2}$

$PL2 = 0.4 \times 0.32 \times 4.13 \times 1.1 = 0.58^{t}$

(B - 3) for substructure

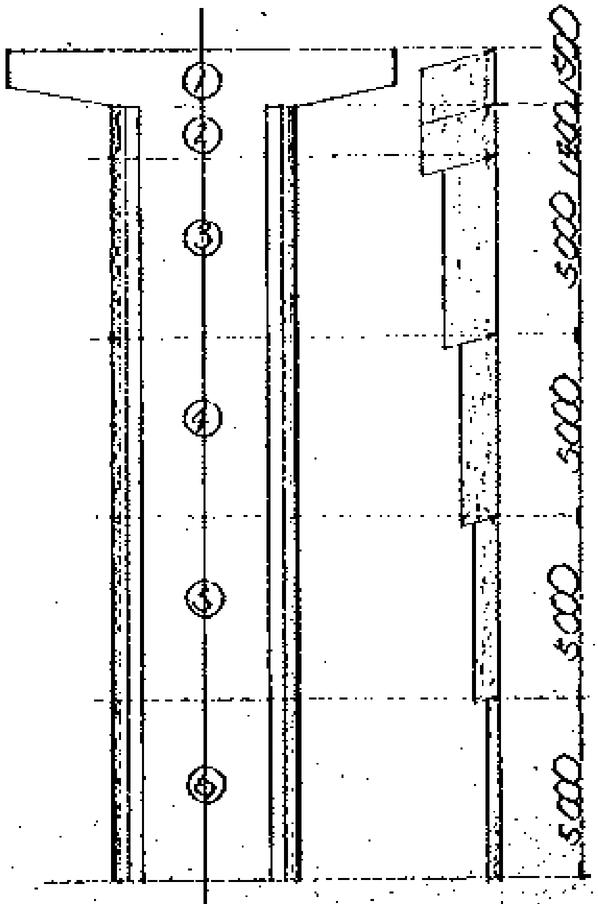
case 1

$$V_c = v \cdot k_1 \cdot S_1 \cdot S_2$$

$$= 38 \times 1.0 \times 1.10 \times S_2 = 41.8 \cdot S_2 \quad {}^{m}/_{sec}$$

$$q = 0.613 \cdot V_c^2 \times 0.102 \times 10^{-3} = 0.1092 \cdot (S_2)^2 \quad {}^{t}/_{m^2}$$

$$P_t = q \cdot A \cdot C_d$$



	V_c (m/sec)	q (t/m²)	t/b	C_d
①	72.0	0.32	$< \frac{1}{4}$	2.1
②	72.0	0.32	$< \frac{1}{3}$	2.2
③	69.0	0.30	↓	↓
④	67.0	0.28		
⑤	65.0	0.26		
⑥	61.0	0.23		

		A (m^2)	Pt (t)	y (m)	Pt·y (tm)
①	$10.60 \times 1.00 + \frac{1}{2} \times (5.00 + 10.60) \times 0.50$	14.5	9.74	25.25	245.94
②	5.00×1.40	7.00	4.93	23.80	117.33
③	5.00×5.00	25.00	16.50	20.60	339.90
④	〃	25.00	15.40	15.60	240.24
⑤	〃	25.00	14.30	10.60	151.58
⑥	〃	25.00	12.65	5.60	70.84
		121.5	73.52		1165.83

case 2

$$V_c = 35 \ ^{m}/_{sec}$$

$$q = 0.613 \times 35^2 \times 0.102 \times 10^{-3} = 0.077 \ ^{t}/_{m^2}$$

$$A = 121.5 \ ^{m^2}$$

$$C_d = (2.1 \times 14.5 + 2.2 \times 107.0) \times {}^{1}/_{121.5} = 2.19$$

$$P_t = 0.077 \times 121.5 \times 2.19 = 20.49^{t}$$

$$y = \frac{14.5 \times 25.25 + 7.0 \times 23.80 + 100.0 \times 13.10}{121.5} = 15.17^{m}$$

(C) vertical wind load

case 1

$P_V = q \cdot A \cdot C_L$

$A = 10.60 \times 27.50 = 291.5^{m^2}$

$C_L = 0.40 \quad (b/d = 10.60 / 2.068 = 5.13)$

$P_V = 0.32 \times 291.5 \times 0.4 = 37.31^{t}$

case 2

$P_V = q \cdot A \cdot C_L$

$A = 291.5^{m^2}$

$C_L = 0.4$

$P_V = 0077 \times 291.5 \times 0.40 = 8.98^{t}$

§ 4. CALCULATION OF STABILITY

4 - 1 longitudinal direction

case 1 (HA loading)

	N (t)	x (m)	$N \cdot x$ (tm)	H (t)	y (m)	$H y$ (tm)
WDL, WLL	671.2	—	9.75	—	—	—
F B	—	—	—	12.90	26.00	335.4
W, Ws	766.11	—	—	—	—	—
TOTAL	1437.31		9.75	12.90		335.4

1) check for eccentricity

$$e = \frac{\Sigma N x + \Sigma H y}{\Sigma N} = \frac{9.75 + 335.4}{1437.31} = 0.24^{m}$$

$$< \frac{B}{6} = 1.08^{m}$$

2) soil reaction

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

$$= \frac{1437.31}{6.50 \times 6.00} \times \left(1 \pm \frac{6 \times 0.24}{6.50}\right) = \begin{cases} 33.76\ t/m^2 \\ 21.52 \end{cases} < 50\ t/m^2$$

3) check for sliding

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

$$F = \frac{0.6 \times 1437.31}{12.90} = 67 \qquad > F_a = 1.5$$

case 2 (wind loading)

		N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL		470.4	—	—	—	—	—
V, Ws		766.11	—	—	—	—	—
wind Pressure	B-1	—	—	—	6.29	26.00	163.54
	B-2	—	—	—	(0.16+0.58) 0.74	26.00	19.24
	B-3	—	—	—	73.52	—	1165.83
TOTAL		1236.51		—	80.55		1348.61

1) check for eccentricity

$$e = \frac{0 \quad + 1348.61}{1236.51} = 1.09^{\,m} < \frac{B}{6} \cdot 1.15 = 1.25^{\,m}$$

2) soil reaction

$$q = \frac{1236.51}{800 \times 6.50} \times \left(1 \pm \frac{6 \times 1.09}{6.50}\right) = \begin{cases} 47.7 \ \mathrm{t/m^2} \\ 0.15 \end{cases} < 68 \ \mathrm{t/m^2}$$

3) check for sliding

$$F = \frac{0.6 \times 1236.51}{8050} = 9 \quad > Fa = 1.5 \cdot \frac{1}{1.15} = 1.3$$

case 3 (HA loading + wind B)

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL.WLL	671.2	——	9.75	——	——	——
F.B	——	——	——	12.90	26.00	335.4
W . Ws	766.11	——	——	——	——	——
wind pressure B-1	——	——	——	(1.51 + 3.84) 5.35	26.00	139.1
wind pressure B-3	——	——	——	20.49	15.17	310.83
TOTAL	1437.31		9.75	38.74		785.33

1) check for eccentricity

$$e = \frac{9.75 + 785.33}{1437.31} = 0.55^{m} < \frac{B}{6} \times 1.15 = 1.25^{m}$$

2) soil reaction

$$q = \frac{1437.31}{8.00 \times 6.50} \times \left(1 \pm \frac{6 \times 0.55}{6.50}\right) = \begin{cases} 41.67\ \text{t/m}^2 \\ 13.61 \end{cases} < 58\ \text{t/m}^2$$

3) check for sliding

$$F = \frac{0.6 \times 1437.31}{38.74} = 22 \qquad > F_a = 1.5 \times 1/1.15 = 1.3$$

case 4 (HA loading + wind (1/2 A + B + C))

		N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL, WLL		671.2	—	9.75	—	—	—
F B		—	—	—	12.90	26.00	335.4
W , Ws		766.11	—	—	—	—	—
wind pressure (B)	B-1	—	—	—	5.35	26.00	139.1
	B-3	—	—	—	20.49	15.17	310.83
	C	8.98	—	—	—	—	—
wind pressure (A)	A-1	—	—	—	(6.52)	28.209	183.92
	A-3	—	—	—	(1.58)	13.11	20.71
TOTAL		1446.29		9.75	38.74 (8.10)		785.33 (204.63)

1) check for eccentricity

$$e = \frac{9.75 + 785.32}{1446.29} = 0.55^{m} < \frac{B}{6} \times 1.15 = 1.25^{m}$$

2) soil reaction

$$q = \frac{1446.29}{8.00 \times 6.50} \times \left(1 \pm \frac{6 \times 0.55}{6.50}\right) \pm \frac{6 \times 204.62}{6.50 \times 8.00^2} = \begin{cases} 44.88 \ t/m^2 \\ 10.74 \ '' \end{cases}$$

$$< 58 \ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1446.29}{38.74} = 22 < F_a = 1.3$$

4 - 2 transverse direction

case 1 (HA loading)

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL	470.4	—	—	—	—	—
WLL	200.8	—	—	—	—	—
W . Ws	766.11	—	—	—	—	—
TOTAL	1437.31		—	—		—

1) check for eccentricity

$$e = \frac{0 + 0}{1437.31} = 0^{m} < \frac{B}{6} = 1.33^{m}$$

2) soil reaction

$$q = \frac{1437.31}{6.50 \times 8.00} = 2764\ ^{t/m^2} < 50\ ^{t/m^2}$$

3) check for sliding

$$F = \frac{0.6 \times 1437.31}{0} = \infty > F_a = 1.5$$

case 2 (wind loadig)

		N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL		470.4	—	—	—	—	—
W · Ws		766.11	—	—	—	—	—
wind pressure	A-1	—	—	—	25.16	27.034	680.18
	A-2	—	—	—	5.57	28.568	159.12
	A-3	—	—	—	11.23	—	170.55
TOTAL		1236.51		—	41.96		1009.85

1) check for eccentricity

$$e = \frac{0 + 1009.85}{1236.51} = 0.82^{m} < \frac{B}{6} \times 1.15 = 1.53^{m}$$

2) soil reaction

$$q = \frac{1236.51}{6.50 \times 8.00} \times \left(1 \pm \frac{6 \times 0.82}{8.00}\right) = \left\{ \begin{array}{l} 38.40\ t/m^2 \\ 9.15\ " \end{array} \right. < 58\ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1236.51}{41.96} = 29. \quad > F_0 = 1.5 \times 1/1.15 = 1.3$$

case 3 (HA loading + wind A)

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	HY (tm)
WDL	470.4	—	—	—	—	—
WLL	200.8	—	—	—	—	—
W, Ws'	766.11	—	—	—	—	—
wind pressure A-1	—	—	—	13.03	28.209	367.56
wind pressure A-3	—	—	—	3.15	13.11	41.30
TOTAL	1437.31		—	16.18		408.86

1) check for eccentricity

$$e = \frac{0 + 408.86}{1437.31} = 0.28^{m} < \frac{B}{6} \cdot 1.15 = 153^{m}$$

2) soil reaction

$$q = \frac{1437.31}{6.50 \times 8.00} \cdot \left(1 \pm \frac{6 \times 0.28}{8.00}\right) = \begin{cases} 33.45 \ t/m^2 \\ 21.84 \ '' \end{cases} < 58 \ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1437.31}{16.18} = 53 \quad > F_a = 1.3$$

case 1 (HA loading + wind (A + C))

		N (t)	x (m)	N x (t m)	H (t)	y (m)	H y (t m)
WDL		470.4	—	—	—	—	—
WLL		200.8	—	—	—	—	—
W , Ws		766.11	—	—	—	—	—
wind pressure	A-1	—	—	—	13.03	—	367.56
	A-2	—	—	—	3.15	—	41.30
	C	8.98		—	—	—	—
TOTAL		1446.29		—	16.18		408.86

1) check for eccentricity

$$e = \frac{0 + 408.86}{1446.29} = 0.28\ ^{m} < \frac{8}{6} \times 1.15 = 1.53\ ^{m}$$

2) soil reaction

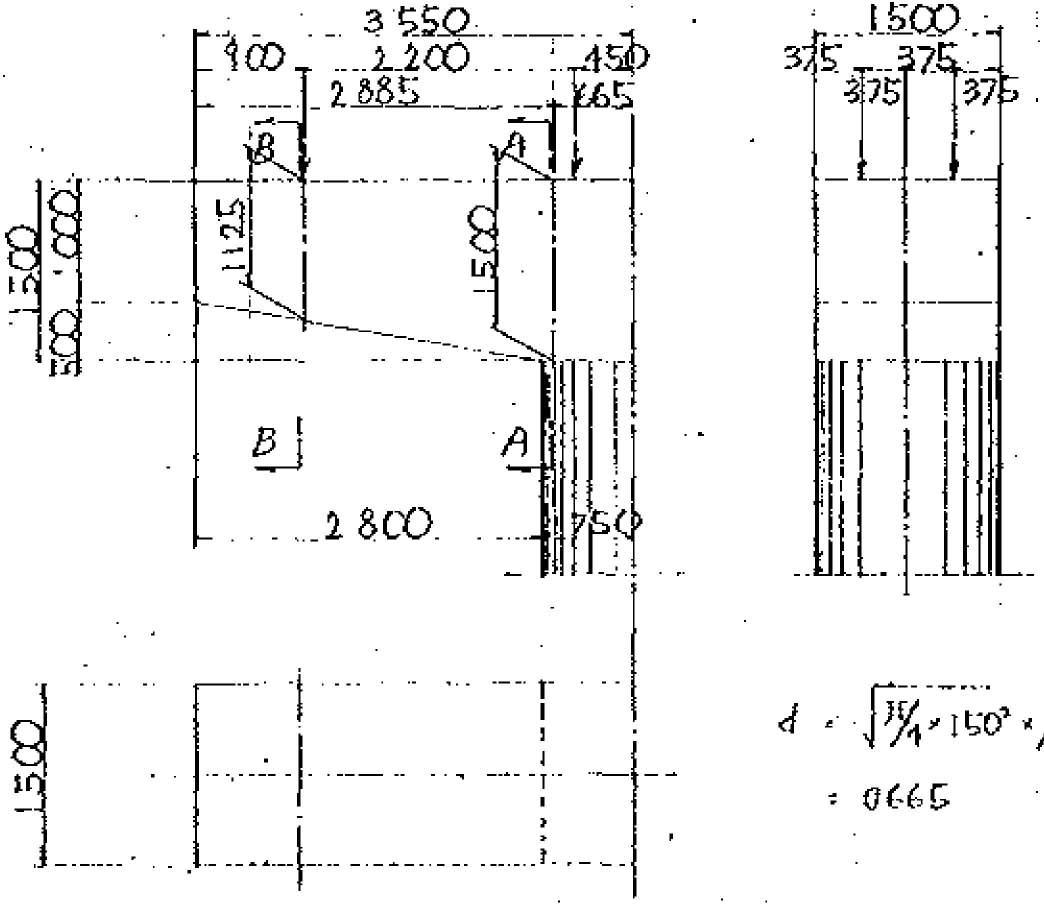
$$q = \frac{1446.29}{6.50 \times 8.00} \times \left(1 \pm \frac{6 \times 0.28}{8.00}\right) = \begin{cases} 33.65\ t/m^2 \\ 21.97\ '' \end{cases} < 58\ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1446.29}{16.18} = 54. \quad > F_a = 1.3$$

§ 5 CALCULATION OF BEAM SECTION

5 - 1 dimension of beam and load



$d = \sqrt{\pi/4 \times 150^2 \times 1/2}$

$\fallingdotseq 0665$

		G 1	G 2	G 3	G 4	G 5
VDL		83.8	92.4	95.4	96.2	103.0
WLL	H A	59.2	58.6	55.8	57.1	48.6
	H B	63.4	58.7	57.0	62.7	33.4
TOTAL	H A	143.0	151.0	151.0	153.3	151.6
	H B	147.2	152.1	155.0	163.9	151.0

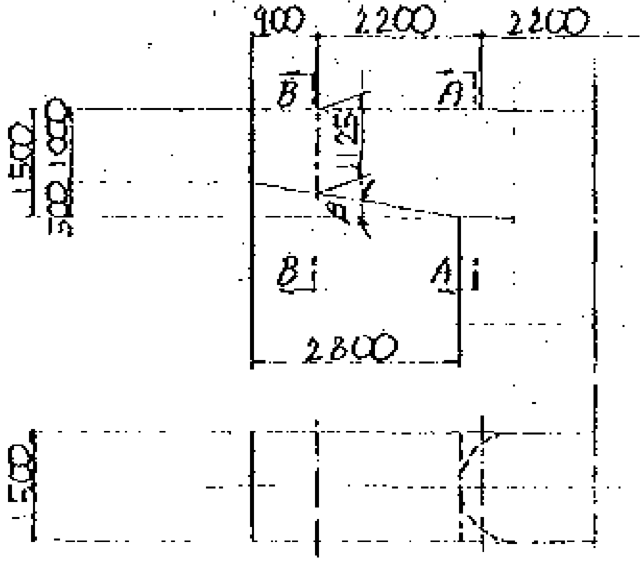
5-2 sectional force of beam (HA loding)

			S (t)	X (m)	M (tm)
A - A	W 1	0.085 × 1.50 × 1.50 × 2.41	0.46	0.043	0.02
	W 2	2.80 × 1.00 × 1.50 × 2.41	10.12	1.40	14.17
	W 3	½ × 2.80 × 0.50 × 1.50 × 2.41	7.59	0.93	7.06
	R 1		151.6	1.985	300.93
	R 2				
	Σ		169.77		322.18
B - B	W 1	1.00 × 0.90 × 1.50 × 2.41	3.25	0.45	1.46
	W 2	½ × 0.90 × 0.125 × 1.50 × 2.41	0.20	0.30	0.06
	R 1		151.6	0	0
	R 2				
	Σ		155.05 (3.45)		1.52
C - C	W 1				
	W 2				
	R 1				
	Σ				

5 - 3 list of stresses σ_c, σ_s, τ : working stress.
σ_{ca}, σ_{sa}, τ_a : Permissible stress.

	A - A	B - B				
M	322.18	1.52				
N	—	—				
S	169.77	155.05				
b	150	150				
h	140	103				
d'	10	10				
AS	14 - D32 112.56	8 - D32 80.4				
AS'	—	—				
f/d	0	0				
M'/bd^2	10.96					
S/bd	8.08					
$n \cdot p$	0.0804					
C	6.84					
S	14.00					
Z	1.12					
σ_c	75					
σ_s	2297					
τ	9					
σ_{ca}	83	83				
σ_{sa}	2346	2346				
τ_a	8.2	8.2				

5-1 check for stirrups



$\tan\beta = \frac{0.50}{2.80}$

$= 0.179$

	M (tm)	S (t)	d (m)	$\frac{M}{d}(\tan\beta)$ (t)	Sh (t)
section A-A	322.18	169.77	1.350	42.62	127.15
〃 B-B	1.52	155.05	1.030	0.26	154.79
〃 C-C					

$$S_h = S - \frac{M}{d}(\tan\beta) \quad (t)$$

	b (cm)	d (cm)	τ_a (kg/cm²)	Sc (t)	Sh' (t)
section A-A					
〃 B-B					
〃 C-C					

$$S_c = \tau_a \cdot b \cdot d \cdot 10^{-3} \quad (t)$$

$$S_h' = S_h - S_c \quad (t)$$

check for stirrups

$$\tau = \frac{Sh}{b \cdot d} \times \gamma$$

$$= \frac{154.79 \times 10^3}{150 \times 103} \times 1.15 = 11.52 > \tau_a = 8.2 \ \text{kg/cm}^2$$

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

$$Sh' = Sh - S_c$$

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

$$= 0.0 \times 150 \times 103 \times \frac{1}{1.15} = 0$$

$$Sh' = 154.79 \times 10^3$$

$$\text{req. } A_w = \frac{154.79 \times 10^3 \times 15}{1780 \times 103} \times 1.15 = 19.56 \ \text{cm}^2$$

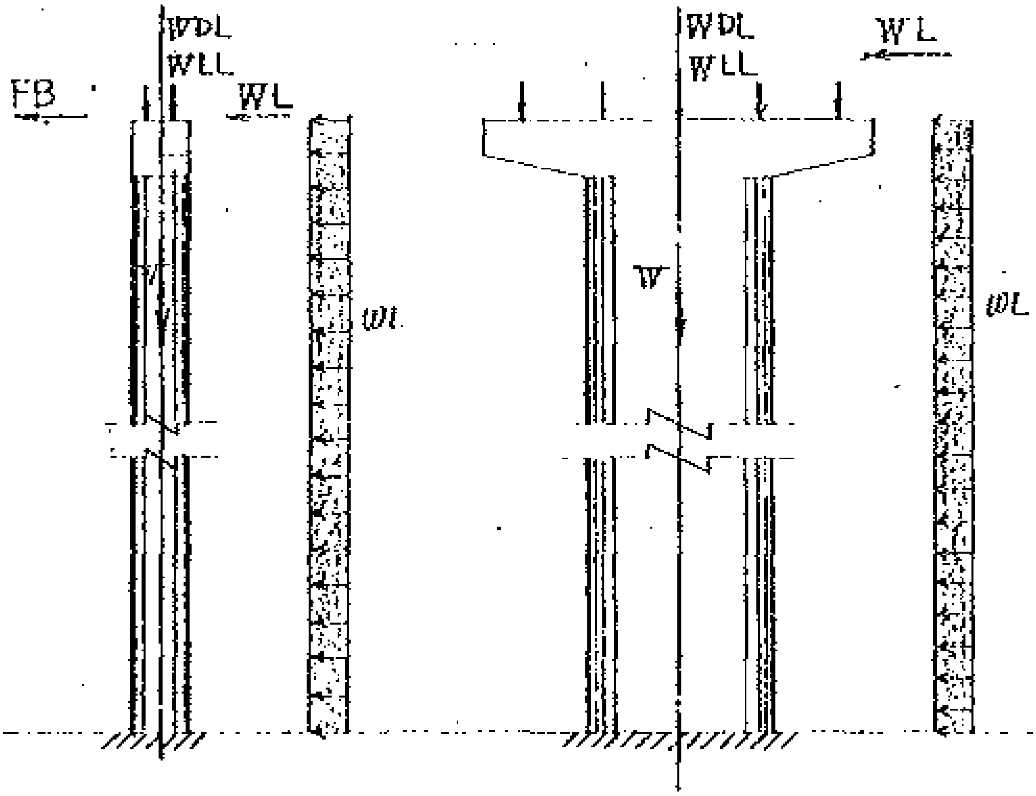
Φ 20 - ctc 150 $n = 6$

Used $A_w = 6 \times 3.14 = 18.84 > \text{req. } A_w = 19.15 \ \text{cm}^2$

§ 6 CALCULATION OF COLUMN SECTION

6 - 1 dimension of column and load

longitudinal direction transverse direction

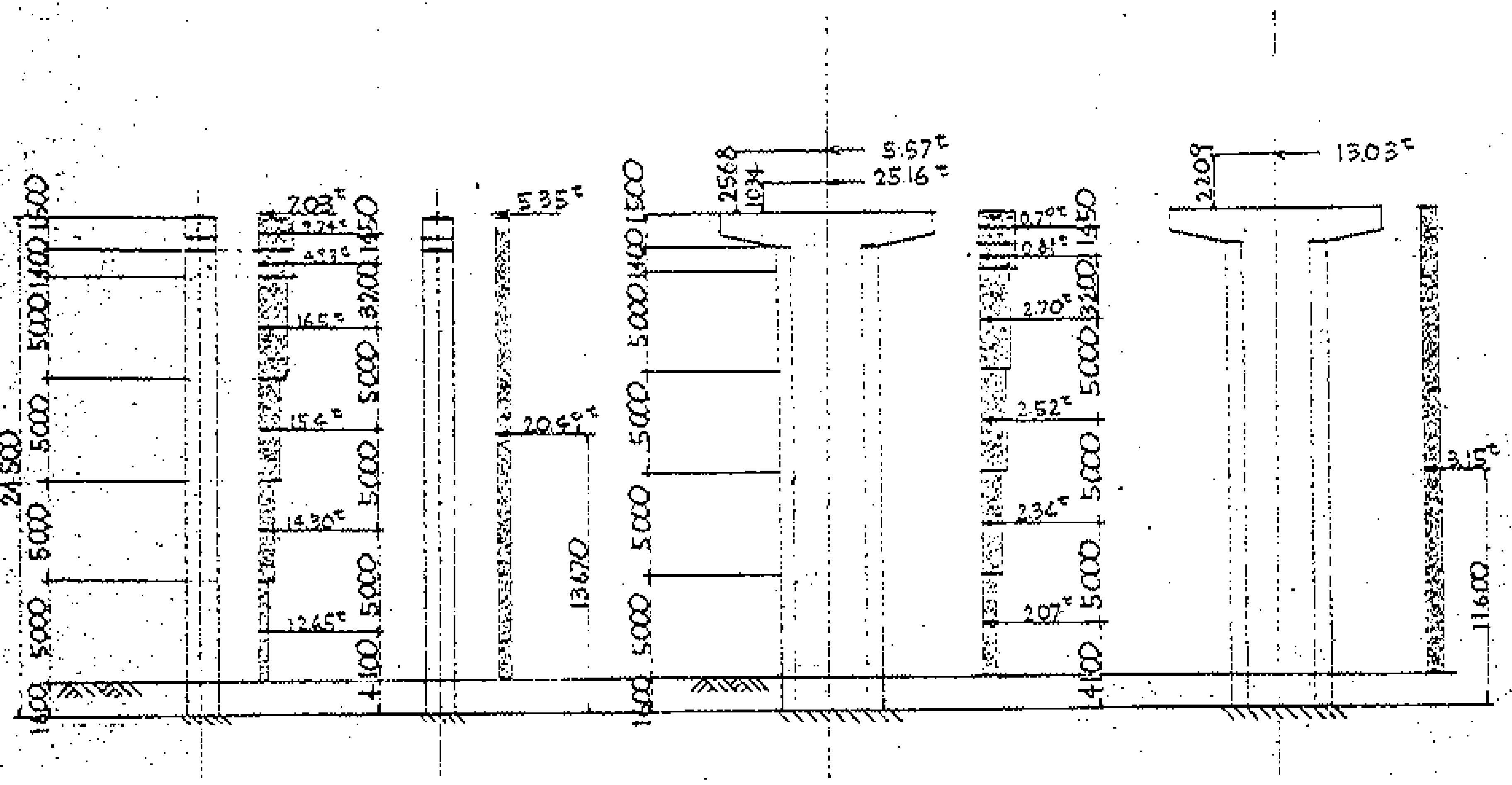


	HA loading		HB loading	
dead load of deck	235.2	235.2	235.2	235.2
live load	113.4	87.4	82.5	78.9
longitudinal forces	12.90		19.10	

wind load

longitudinal direction | transverse direction

(without live load) (with live load) | (without live load) | (with live load)



6 - 2 sectional force of column

1) longitudinal direction

			N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm
case 1 (HA loading)	WDL		470.4	—	—	—	—	—
	WLL		200.8	—	9.75	—	—	—
	F B		—	—	—	12.90	24.50	316.05
	W		441.38	—	—	—	—	—
			1112.58		9.75	12.90		316.05
case 2 (wind)	WDL		470.4	—	—	—	—	—
	W		441.38	—	—	—	—	—
	wind	B-1,2	—	—	—	7.03	24.50	172.24
		B-3	—	—	—	73.52	—	1055.55
			911.78		—	80.55		1227.79
	(α = 1.15)		(792.85)			(70.04)		(1067.64)
case 3 (HA loading + wind)	WDL		470.4	—	—	—	—	—
	WLL		200.8	—	9.75	—	—	—
	F B		—	—	—	12.90	24.50	316.05
	W		441.38	—	—	—	—	—
	wind	B-1	—	—	—	5.35	24.50	131.08
		B-2	—	—	—	20.49	13.67	280.10
			1112.58		9.75	38.74		727.23
	(α = 1.15)		(967.46)		(8.48)	(33.69)		(632.37)

2) transverse direction

			N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
case 1 (wind)	WDL		470.4	—	—	—	—	—
	W		441.38	—	—	—	—	—
	wind	A-1	—	—	—	25.16	25.534	642.44
		A-2	—	—	—	5.57	27.068	150.77
		A-3	—	—	—	11.23	—	153.71
			911.78			41.96		946.92
	(α = 1.15)		(792.85)			(36.49)		(823.41)
case 2 (HA loading +wind)	WDL		470.4	—	—	—	—	—
	WLL		200.8	—	—	—	—	—
	W		441.38	—	—	—	—	—
	wind	A-1	—	—	—	13.03	26.709	348.02
		A-3	—	—	—	3.15	11.60	36.54
			1112.58			16.18		384.56
	(α = 1.15)		(967.46)			(14.07)		(334.4)

6 - 3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.

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

	longitudinal direction			transverse direction		
	case 1	case 2	case 3	case 1	case 2	
M	325.8	1067.64	640.85	823.41	334.4	
N	1112.58	792.85	967.46	792.85	967.46	
S	12.90	70.04	33.69	36.49	14.07	
b	483	→	→	150	150	
h	140	→	→	473	473	
d'	10	→	→	→	→	
As	D32 ctc125 297.48	→	→	D32 ctc125 80.4	→	
As'	•	→	→	〃	→	
f/d		1.43	0.94	0.71	0.56	
M'/bd^2		16.72	13.41	7.92	7.67	
S/bd		1.04	0.498	0.51	0.198	
nP		0.066	0.066	0.017	0.017	
C		4.45	3.41	3.00	2.74	
s		6.61	2.29	0.296	-0.249	
z		1.07	0.844	0.13	0.065	
σ_c		74	46	24	21	
σ_s		1658	460	35	-29	
τ		1.1	0.4	0.1	0.01	
σ_{ca}	83	→	→	→	→	
σ_{sa}	2346	→	→	→	→	
τ_a	8.2	→	→	→	→	

6- 4 check for buckling of column

$$Pa = \frac{1}{3} \times (0.85 \cdot \sigma_{ck} \cdot A_c + \sigma_{sy}' \cdot A_s)$$

$$\alpha = 1.45 - 0.03 \cdot \frac{h_e}{d}$$

$$h_e = (24.50 + 1.50) \times 2 = 52.0^{m}$$

$$Pa = \frac{1}{3} \times \{0.85 \times 250 \times (\pi/4 \times 150^2 + 350 \times 150) + 2500 \times 377.88\}$$
$$= 5290000^{kg} \qquad 5290^{t}$$

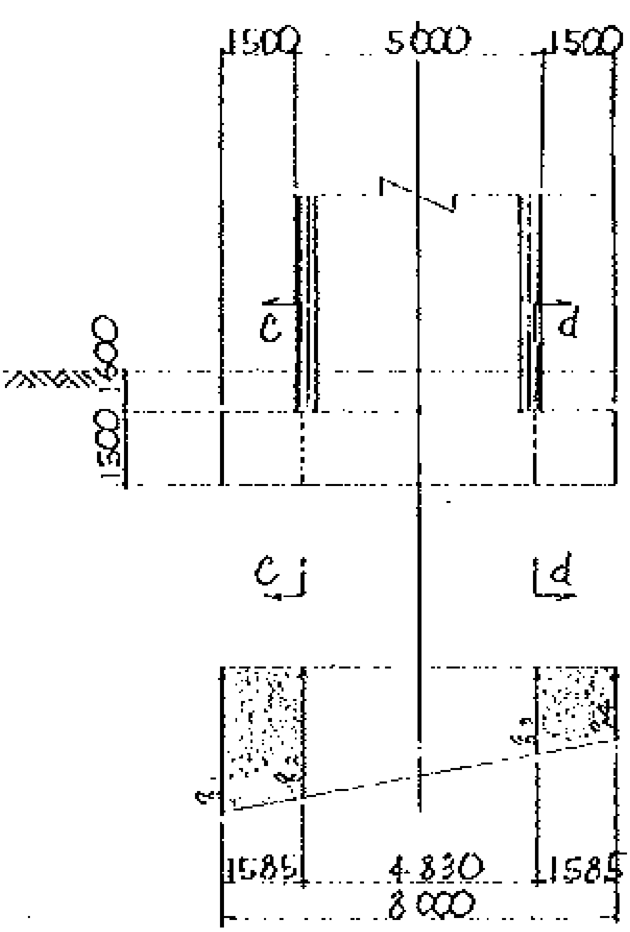
$$\alpha = 1.45 - 0.03 \times 52.0/1.50$$
$$= 0.41$$

$$\therefore Paa = 5290 \times 0.41$$
$$= 2169^{t} \qquad > P_N = 1112.58^{t}$$

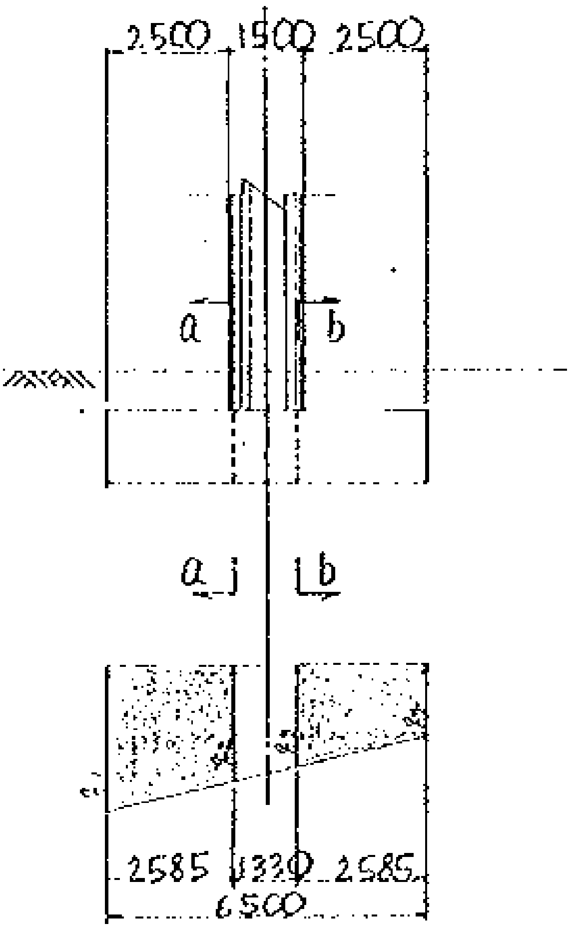
§ 7 CALCULATION OF FOOTING SECTION

7-1 dimension and soil reaction

transverse direction



longitudinal direction



case	longitudinal direction				transverse direction			
	q_1	q_2	q_3	q_4	q_1	q_2	q_3	q_4
1	33.76	28.89	26.39	21.52	27.64	27.64	27.64	27.64
2	47.7	28.67	18.88	-0.15	38.4	32.60	14.95	9.15
3	41.67	30.51	24.77	13.61	33.45	31.15	24.14	21.84
4	44.88	31.30	24.32	10.74	33.65	31.34	24.28	21.97

section A – A

			S (t)	x (m)	Sx (tm)
case 1	Wd	(1.50×2.41+1.60×1.90) ×2.585×8.00	137.63	1.293	177.96
	q	½·(33.76+28.89)×2.585×8.00	-647.80	1.326	-858.98
			510.17		681.02
case 2	Wd	———	137.63	———	177.96
	q	½·(47.7+28.67)×2585×8.00	-789.67	1.400	-1105.54
(α=1.15)			652.04 (566.99)		927.58 (806.59)
case 3	Wd	———	137.63	———	177.96
	q	½·(41.67+30.51)×2.585×8.00	-746.34	1.359	-1014.28
(α=1.15)			608.71 (529.31)		836.32 (727.23)
case 4	Wd	———	137.63	———	177.96
	q	½·(44.88+31.30)×2.585×8.00	-787.70	1.369	-1078.36
(α=1.15)			650.07 (565.28)		900.40 (782.96)

section B – B

M upper = 0

section c − c

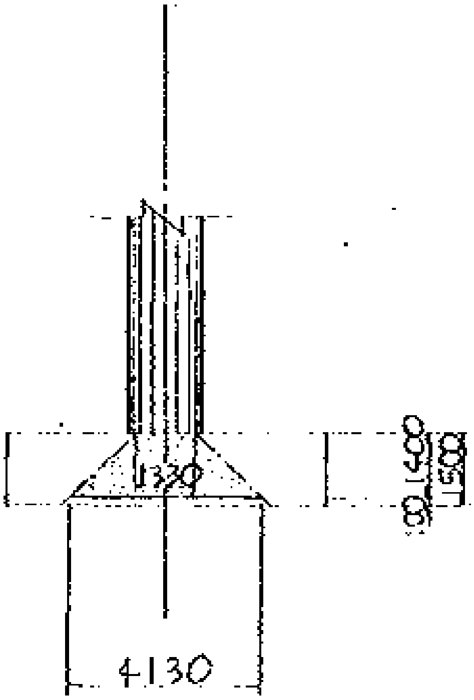
			S (t)	x (m)	Sx (tm)
case 1	wd	(150×241+160×1.9) ×1.585×6.50	68.56	0.793	54.37
	q	27.64×1.585×6.50	−284.76	0.793	−225.81
			216.2		171.44
case 2 (α=1.15)	wd	————	68.56	——	54.37
	q	½×(384+326)×1.585×6.50	−365.74	0.814	−297.71
			297.18 (258.42)		243.34 (211.6)
case 3 (α=1.15)	wd	————	68.56	——	54.37
	q	½×(33.45+31.15)×1.585×6.50	−332.77	0.802	−266.88
			264.21 (229.75)		212.51 (184.79)
case 4 (α=1.15)	wd	————	68.56	——	54.37
	q	½×(3365+31.34)×1585×6.50	−334.78	0.802	−268.49
			266.22 (231.50)		214.12 (186.19)

section d−d

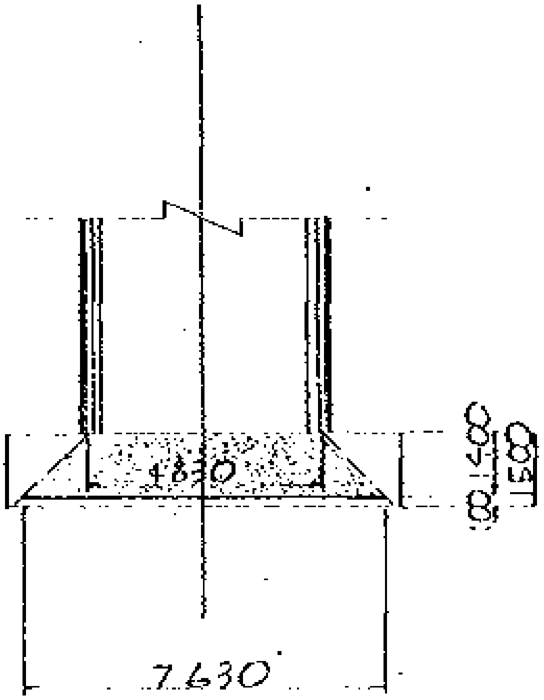
$M_{upper} = 0$

7 - 2 calculation of members

longitudinal direction



transverse direction



$$B_1 = 1.33 + 1.40 \times 2 = 4.13^{m}$$

$$B_2 = 4.83 + 1.40 \times 2 = 7.63^{m}$$

7 - 3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.

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

	a − a	b − b		c − c	d − d	
M	80659			211.6		
N	------			------		
S	566.99			258.42		
b	763	〃		413	〃	
h	140			140		
d′	10			10		
AS	D25 @125 299.51			D25 @250 28.56		
AS′	------			------		
f/d	0			0		
M'/bd^2	5.39			2.61		
S/bd	5.31			4.47		
n.P	0.0421			0.0204		
C	8.70			11.67		
S	25.95			52.27		
Z	109			1.06		
σ_c	47			31		
σ_s	2099			2050		
τ	5.8			4.8		
σ_{ca}	83	→				
σ_{sa}	2346	→				
τ_a	2.35	→				

check for stirrups

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

$$= \frac{566.99 \times 10^3}{763 \times 140} \times 1.09 = 5.79^{kg/cm^2} > \tau_a = 2.35$$

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

$$S_h' = S_h - S_c$$

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

$$= 2.35 \times 763 \times 140 \times \frac{1}{1.09} = 230\,300^{kg}$$

$$S_h' = (566.99 - 230.30) \times 10^3 = 336.69 \times 10^3 \ ^{kg}$$

$$\text{req. } A_w = \frac{336.69 \times 10^3 \times 15}{1780 \times 140} \times 1.09 = 36.82^{cm^2}$$

effective width 7.63 m ∴ req. $A_s = {}^{36.82}/_{7.63} = 4.83^{cm^2}$

Φ 20 - c.t.c 250 n = 2

Used $A_w = 2 \times 3.14 = 6.28^{cm^2} > \text{req. } A_w = 4.83^{cm^2}$

check for stirrups'

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

$$= \frac{258.42 \times 10^3}{413 \times 140} \times 1.06 = 4.74^{kg/cm^2} > \tau_a = 2.35$$

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

$$Sh' = Sh - Sc$$

$$Sc = \tau a \times b \times d \times \frac{1}{Z}$$

$$= 2.35 \times 413 \times 140 \times \frac{1}{1.06} = 128\,190^{kg}$$

$$Sh' = (258.42 - 128.19) \times 10^3 = 130.23 \times 10^3 \ kg$$

$$\text{req. } A_w = \frac{130.23 \times 10^3 \times 25}{1780 \times 140} \times 1.06 = 13.85^{cm^2}$$

effective width 4.13^m req $A_w = 13.85/4.13 = 3.35^{cm^2}$

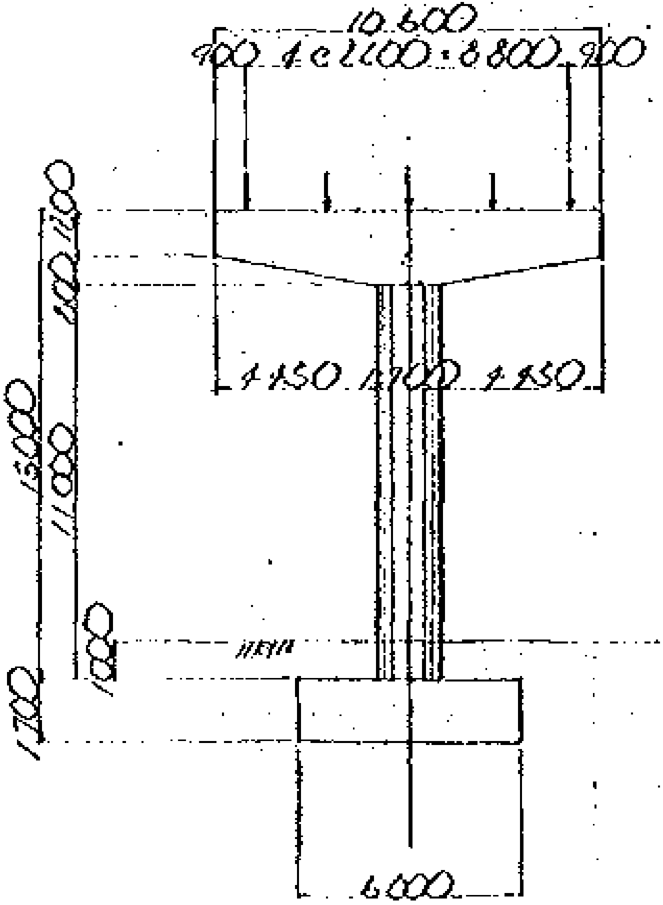
ϕ 20 - ctc 250 $n = 2$

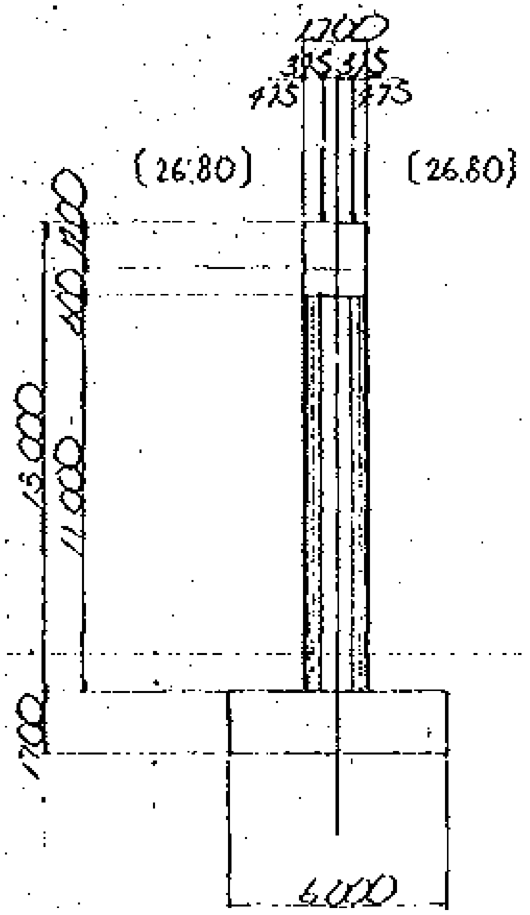
Used $A_w = 2 \times 3.14 = 6.28^{cm^2} > \text{req. } A_w = 3.35$

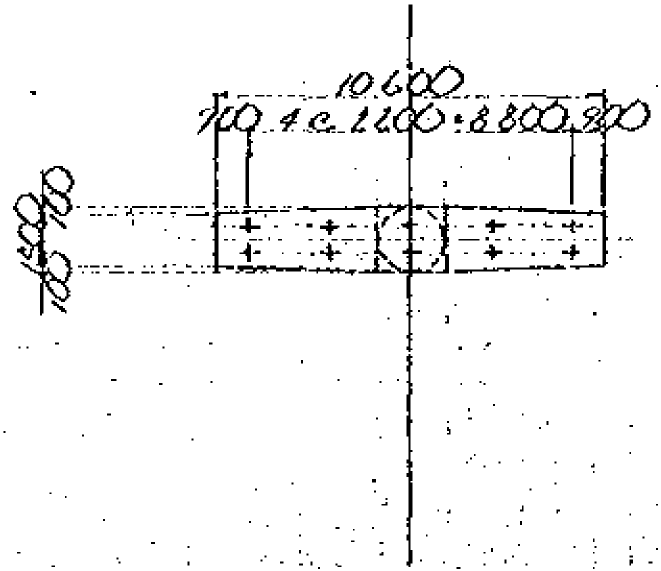
§§ 5 PIER

$H = 14.70\ ^{m}$

§ 1 DIMENSIONS







§ 2 REACTION due to SUPERSTRUCTURE

2-1 whole reaction due to superstructure

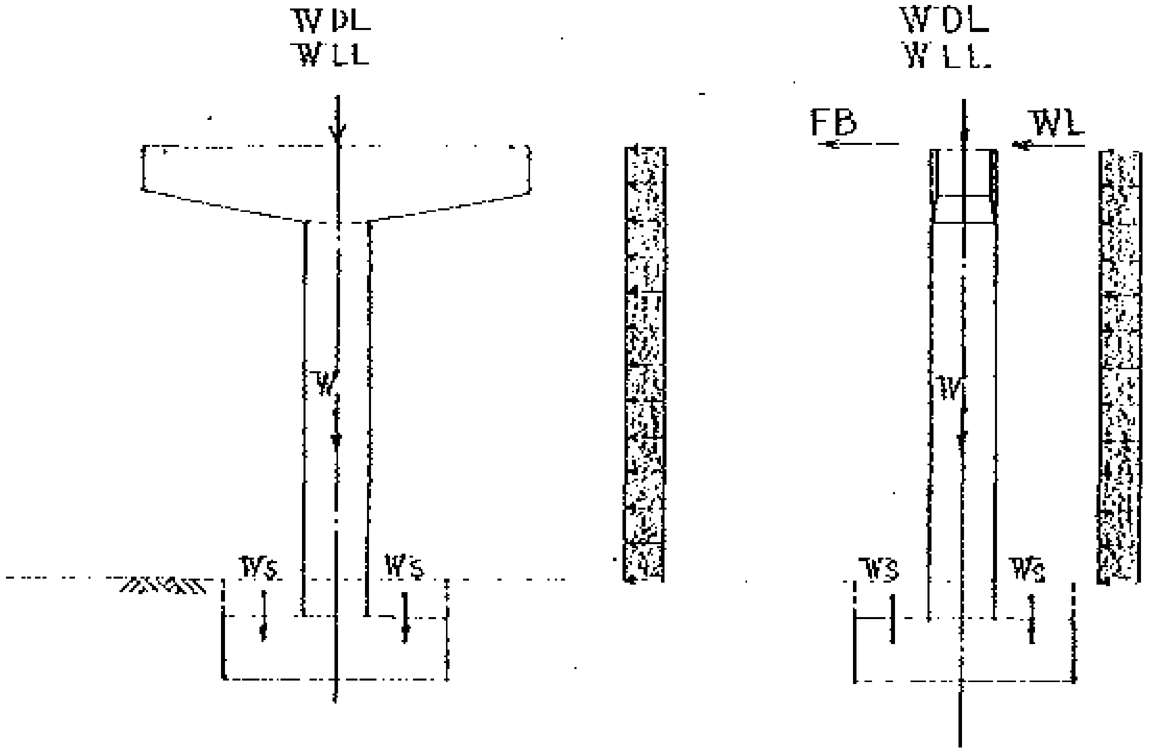
dead load of deck		235.2	235.2
H. A Live load		113.4	87.4
H. B Live load		82.55	78.85
crowd load		10.3	10.3
longitudinal forces	under H. A	12.9	
	under H. B	19.1	

2-2 reaction per each girder

		G 1	G 2	G 3	G 4	G 5
dead load of deck		83.8	92.4	95.2	96.2	103.0
live load	H A	59.2	58.6	55.8	57.1	48.6
	H B	63.4	59.7	59.8	67.7	48.0
TOTAL	H A	143.0	151.0	151.0	153.3	151.6
	H B	147.2	152.1	155.0	163.9	151.0

§ 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 or traction

W : self weight

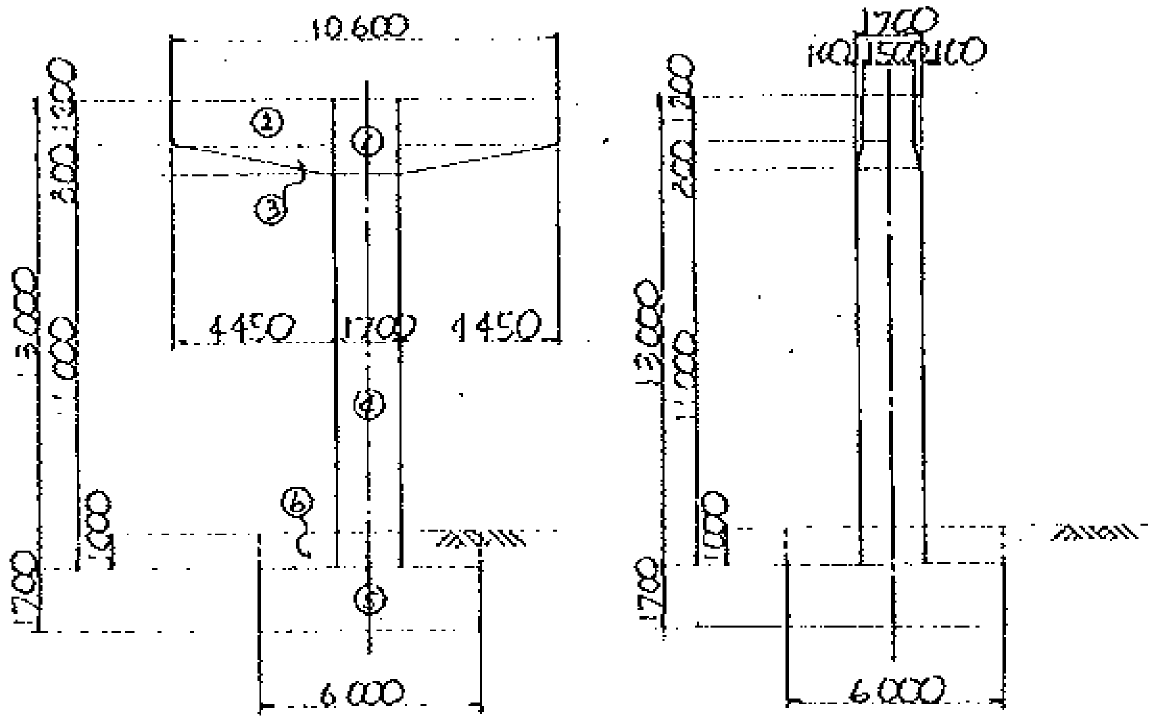
Ws : weight of soil

WL : wind load on the superstructure

wL : wind load on the pier

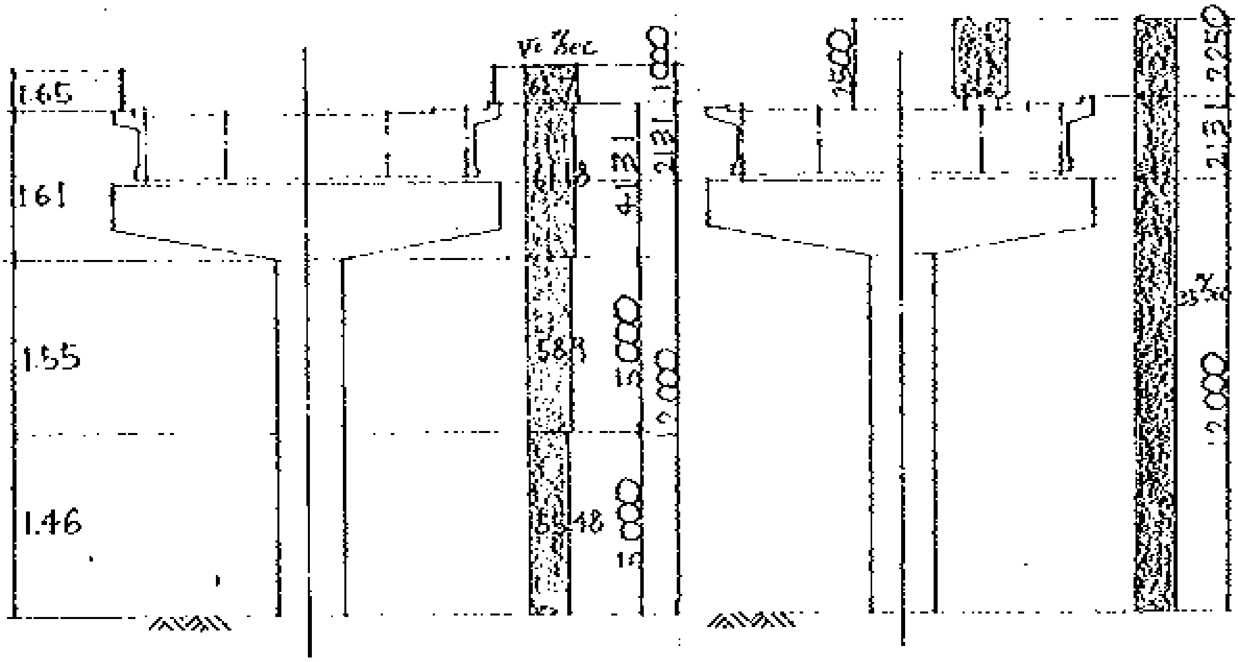
B : buoyancy

3 - 2 self weight and weight of soil



		N (t)
①	$1.70 \times 2.00 \times 1.70 \times 2.41$	13.93
②	$\{1/2 \times (1.50+1.70) \times 4.45 \times 1.20\} \times 2 \times 2.41$	41.18
③	$\{1/6 \times 0.80 \times 4.45 \times (2 \times 1.70 + 1.50)\} \times 2 \times 2.41$	14.01
④	$\pi/4 \times 1.70^2 \times 11.00 \times 2.41$	60.17
⑤	$6.00 \times 6.00 \times 1.70 \times 2.41$	147.49
⑥	$(6.00 \times 6.00 - \pi/4 \times 1.70^2) \times 1.00 \times 1.9$	64.09
		340.87

3-3 wind pressure



wind gust speed

$$V_c = v \cdot k_1 \cdot s_1 \cdot s_2$$

case 1 (without live load)

$$V_c = 38 \times 1.0 \times 1.0 \times S_2 = 38 \cdot S_2 \ {}^{m}/_{sec}$$

case 2 (with live load)

$$V_c = 35.00 \ {}^{m}/_{sec}$$

(A) transverse wind load

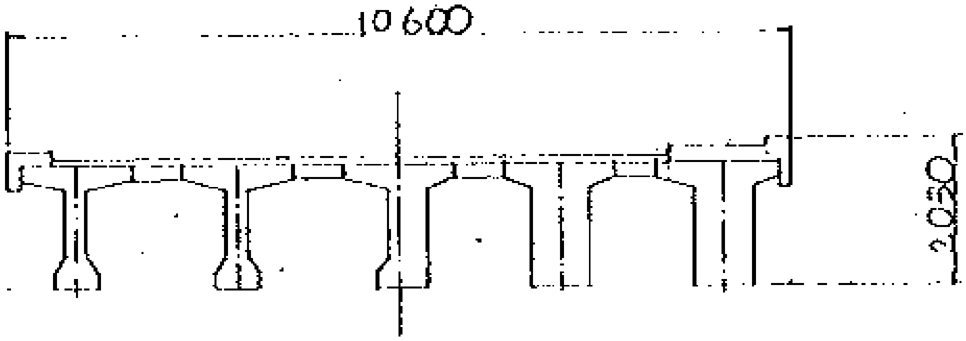
(A - 1) for superstructures

$$P_t = q \cdot A \cdot C_d \quad (t)$$

$$q = 0.613 \cdot V_c^2 \times 0.102 \quad (kg/m^2)$$

C_d : drag coefficient

A : loading area (m^2)



case 1

$$Q = 0.613 \times 61.18^2 \times 0.102 \times 10^{-3} = 0.23 \ t/m^2$$

$$A = 2.03 \times 1/2 \times (27.50 + 27.50) = 55.83 \ m^2$$

$$C_d = 1.35 \quad (b/d = 10.60/2.03 = 5.22)$$

$$P_t = 0.23 \times 55.83 \times 1.35 = 17.34 \ t$$

case 2

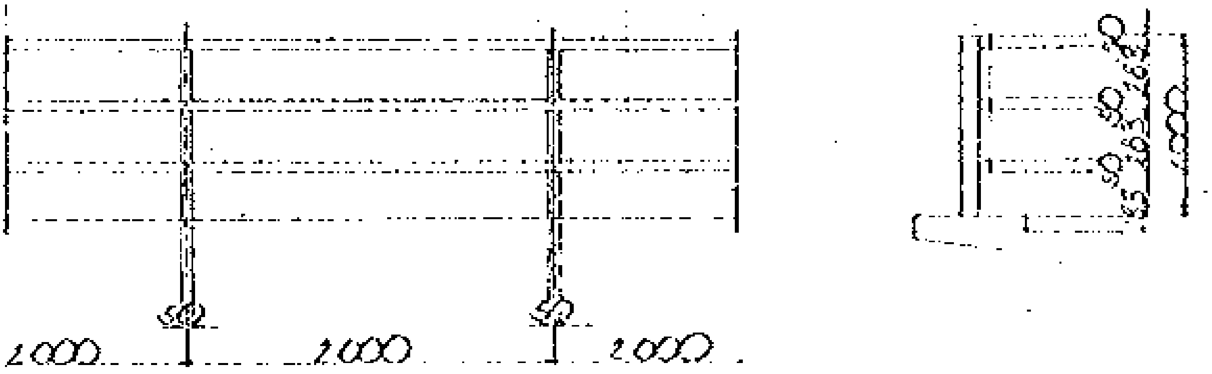
$$Q = 0.613 \times 35^2 \times 0.102 \times 10^{-3} = 0.077 \ t/m^2$$

$$A = 4.28 \times 1/2 \times (27.50 + 27.50) = 117.7$$

$$C_d = 1.38 \quad (b/d = 10.60/2.50 = 4.24)$$

$$P_t = 0.077 \times 117.7 \times 1.38 = 12.51 \ t$$

(A − 2) for parapet



case 1

$$A = 0.05 \times 3 \times 27.50 + (1.00 - 0.05 \times 3) \times \frac{27.50}{2.00}$$

$$= 15.81 \ m^2$$

$$Q = 0.613 \times 62.7^2 \times 0.102 \times 10^{-3} = 0.25 \ t/m^2$$

$$Cd = 1.1 \quad (\text{from table } 8)$$

$$Pt = 0.25 \times 15.81 \times 1.1 = 4.35 \ t$$

(A-3) for substructure

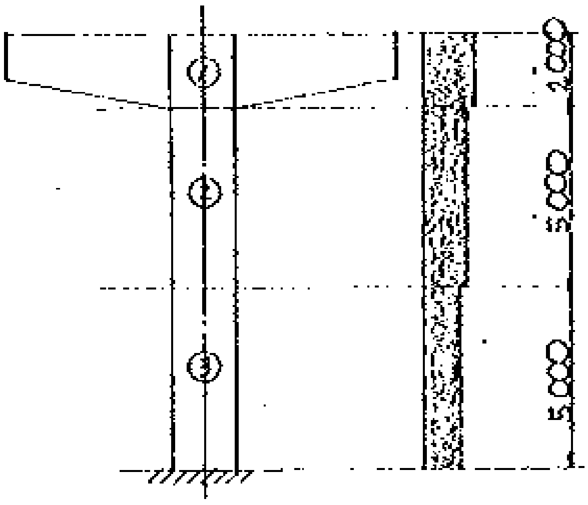
case 1

$$V_c = v \cdot k_1 \cdot s_1 \cdot s_2$$

$$= 38 \times 1.0 \times 1.0 \times S_2 = 38 \cdot S_2 \quad {}^{m}/_{sec}$$

$$q = 0.613 \cdot V_c^2 \times 0.102 \times 10^{-3} = 0.0903 \cdot (S_2)^2 \quad {}^{t}/_{m^2}$$

$$P_t = q \cdot A \cdot C_d$$



	V_c (m/sec)	q (t/m²)	t/b	C_d
①	61.18	0.23	>4	1.1
②	58.9	0.22	○	1.2
③	55.48	0.19	○	1.2

	A (m^2)		P_t (t)	y (m)	$P_t y$ (tm)
	1.70×2.00	3.40	0.86	13.70	11.78
	1.70×5.00	8.50	2.24	10.20	22.85
	"	8.50	1.94	5.20	10.09
		20.40	5.04		44.72

case - 2

$$V_c = 35 \ ^{m}/_{sec}$$

$$q = 0.613 \times 35^2 \times 0.102 \times 10^{-3} = 0.077 \ ^{t}/_{m^2}$$

$$A = 20.40^{m^2}$$

$$C_d = (1.10 \times 3.40 + 1.2 \times 17.00) \times {}^{1}/_{20.4} = 1.18$$

$$P_t = 0.077 \times 20.40 \times 1.18 = 1.85^{t}$$

$$y = \frac{3.40 \times 13.70 + 17.00 \times 7.70}{20.40} = 8.70^{m}$$

(B) longitudinal wind load

(B - 1) for superstructure

case 1

$P_{LS} = 0.25 \cdot P_t$

$= 0.25 \times 17.34 \qquad = 4.34^{t}$

case 2

$P_{LS} = 0.25 \cdot P_t$

$P_t = q \cdot A \cdot C_d \quad , \quad A = l \cdot d$

$P_{LS} = 0.25 \times 0.077 \times (27.50 \times 2.03) \times 1.35$

$= 1.45^{t}$

$P_{LL} = 0.5 \cdot P_t$

$P_t = q \cdot A \cdot C_d \qquad A = 2.50 \cdot l \qquad C_d \geqq 1.45$

$P_{LL} = 0.5 \times 0.077 \times (2.50 \times 27.50) \times 1.45$

$= 3.84^{t}$

(B − 2) for palapet

for vertical members

$$P_{L1} = 0.8 \cdot P_t$$

$$P_t = q \cdot A \cdot C_d \qquad C_d = 1.1$$

$$A = 0.05 \times (1.00 - 3 \times 0.05) \times 27.50/2.00 = 0.58^{m^2}$$

$$P_{L1} = 0.8 \times 0.25 \times 0.58 \times 1.1 = 0.13^{t}$$

for horizontal members

$$P_{L2} = 0.4 \cdot P_t$$

$$P_t = q \cdot A \cdot C_d \qquad C_d = 1.1$$

$$A = 27.50 \times 0.05 \times 3 = 4.125^{m^2}$$

$$P_{L2} = 0.4 \times 0.25 \times 4.125 \times 1.1 = 0.45^{t}$$

(B - 3) for substructure

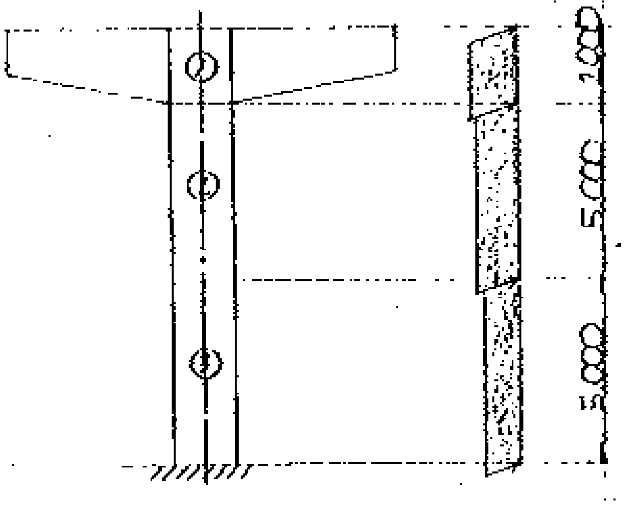
case 1

$V_c = U \cdot k_1 \cdot S_1 \cdot S_2$

$= 38 \times 1.0 \times 1.0 \times S_2 = 38 \cdot S_2$ m/sec

$q = 0.613 \cdot V_c^2 \times 0.102 \times 10^{-3} = 0.0903 \cdot (S_2)^2$ t/m²

$P_t = q \cdot A \cdot C_d$



	Vc (m/sec)	q (t/m²)	l/b	Cd
①	61.18	0.23	< 1/4	2.1
②	58.9	0.22	○	1.2
③	55.48	0.19	○	1.2

		A (m^2)	Pt (t)	y (m)	Pt·y (tm)
①	$10.60 \times 1.20 + \frac{1}{2} \times (1.70 + 10.60) \times 0.80$	17.64	8.52	13.70	116.72
②	1.70×5.00	8.50	2.24	10.20	22.85
③	1.70×5.00	8.50	1.94	5.20	10.09
		34.64	12.7		149.66

case 2

$$V_c = 35 \ {}^{m}/_{sec}$$

$$q = 0.613 \times 35^2 \times 0.102 \times 10^{-3} = 0.077 \ {}^{t}/_{m^2}$$

$$A = 34.64^{m^2}$$

$$C_d = (2.1 \times 17.64 + 1.2 \times 17.00) \times {}^{1}/_{34.64} = 1.66$$

$$P_t = 0.077 \times 34.64 \times 1.66 = 4.43^{t}$$

$$y = \frac{17.64 \times 13.70 + 17.00 \times 7.70}{34.64} = 10.755^{m}$$

(C) vertical wind load

case 1

$$P_V = q \cdot A \cdot C_L$$

$$A = 10.60 \times 27.50 = 291.50^{m^2}$$

$$C_L = 0.4 \quad (b/d = 10.60/2.03 = 5.22)$$

$$P_V = 0.25 \times 291.50 \times 0.4 = 29.15^{t}$$

case 2

$$P_V = q \cdot A \cdot C_L$$

$$A = 291.50^{m^2}$$

$$C_L = 0.4$$

$$P_V = 0.077 \times 291.50 \times 0.4 = 8.98^{t}$$

§ 4 CALCULATION OF STABILITY

4 - 1 longitudinal direction

case 1 (HA loading)

	N (t)	x (m)	N·x (tm)	H (t)	y (m)	H·y (tm)
WDL, WLL	671.2	———	9.75	———	———	———
E.B	———	———	———	12.90	14.70	189.63
W, Ws	340.87	———	———	———	———	———
TOTAL	1012.07		9.75	12.90		189.63

1) check for eccentricity

$$e = \frac{\Sigma Nx + \Sigma Hy}{\Sigma N} = \frac{9.75 + 189.63}{1012.07} = 0.20^{m}$$

$$< \frac{B}{6} = 1.00^{m}$$

2) soil reaction

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

$$= \frac{1012.07}{6.00 \times 6.00} \times \left(1 \pm \frac{6 \times 0.20}{6.00}\right) = \begin{cases} 33.74\ t/m^2 \\ 22.49 \end{cases}, < 50\ t/m^2$$

3) check for sliding

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

$$F = \frac{0.6 \times 1012.07}{12.90} = 47. \qquad > Fa = 1.5$$

case 2 (wind loading)

		N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL		470.4	—	—	—	—	—
V, Ws		340.87	—	—	—	—	—
wind Pressure	B-1	—	—	—	4.34	14.70	63.80
	B-2	—	—	—	(0.13+0.45) 0.58	16.831	9.76
	B-3	—	—	—	34.64	—	149.66
TOTAL		811.27		—	39.56		223.22

1) check for eccentricity

$$e = \frac{0 + 223.22}{811.27} = 0.28^{m} < \frac{B}{6} \cdot 1.15 = 1.00^{m}$$

2) soil reaction

$$q = \frac{811.27}{6.00 \times 6.00} \times \left(1 \pm \frac{6 \times 0.28}{6.00}\right) = \begin{cases} 28.85 \ t/m^2 \\ 16.23 \ " \end{cases} < 58 \ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 811.27}{39.56} = 12 > F_a = 1.5 \cdot \frac{1}{1.15} = 1.3$$

case 3 (HA loading + wind B)

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL.WLL	671.2	——	9.75	——	——	——
F B	——	——	——	12.90	14.70	189.63
W , Ws	340.87	——	——	——	——	——
wind pressure B-1	——	——	——	(1.45 + 3.84) 5.29	14.70	77.76
wind pressure B-3	——	——	——	4.43	10.755	47.64
TOTAL	1012.07		9.75	22.62		315.03

1) check for eccentricity

$$e = \frac{9.75 + 315.03}{1012.07} = 0.32\ ^{m} < \frac{B}{6} - 4.15 = 1.15\ ^{m}$$

2) soil reaction

$$q = \frac{1012.07}{6.00 \times 6.00} \cdot \left(1 \pm \frac{6 \times 0.32}{6.00}\right) = \begin{cases} 37.11\ t/m^2 \\ 19.12\ \ " \end{cases} < 58\ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1012.07}{22.62} = 27 > Fa = 1.5 \times 1/1.15 = 1.3$$

case 4 (HA loading + wind (1/2 A + B + C))

		N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL, WLL		671.2	—	9.75	—	—	—
F B		—	—	—	12.90	14.70	189.63
W, Ws		340.87	—	—	—	—	—
wind pressure (B)	B-1	—	—	—	5.29	14.70	77.76
	B-3	—	—	—	4.43	—	47.64
	C	8.98	—	—	—	—	—
wind pressure (A)	A-1	—	—	—	(6.26)	16.94	(106.04)
	A-3	—	—	—	(0.93)	8.70	(8.09)
TOTAL		1021.05		9.75	22.62 (7.19)		315.03 (114.13)

1) check for eccentricity

$$e = \frac{9.75 + 315.03}{1021.05} = 0.32\ \text{m} < \frac{B}{6} = 1.15 = \text{m}$$

2) soil reaction

$$q = \frac{1021.05}{6.00 \times 6.00} \times \left(1 \pm \frac{6 \times 0.32}{6.00}\right) \pm \frac{6 \times 114.13}{6.00 \times 6.00^2} = \begin{cases} 40.61\ \text{t/m}^2 \\ 16.12\ \text{"} \end{cases}$$

$$< 380\ \text{t/m}^2$$

3) check for sliding

$$F = \frac{0.6 \times 1021.05}{9.75} = 63 < F_a = 1.3$$

4 - 2 transverse direction

case 1 (HA loading)

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL	470.4	----	----	----	----	----
WLL	200.8	----	----	----	----	----
W . Ws	340.87	----	----	----	----	----
TOTAL	1012.07		----	----		----

1) check for eccentricity

$$e = \frac{0 + 0}{1012.07} = 0^{m} < \frac{B}{6} = 1.00^{m}$$

2) soil reaction

$$q = \frac{1012.07}{6.00 \times 6.00} = 28.11 \ t/m^2 < 50 \ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1012.07}{0} = \infty > F_a = 1.5$$

case 2 (wind loadig)

		N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
WDL		470.4	——	———	———	———	———
W . Ws		340.87	——	———	———	———	———
wind pressure	A – 1	———	——	———	17.34	15.816	274.25
	A – 2	———	——	———	4.35	17.331	75.39
	A – 3	———	—	———	5.04	———	44.72
TOTAL		811.27		———	26.73		394.36

1) check for eccentricity

$$e = \frac{0 + 394.36}{811.27} = 0.49^{m} < \frac{B}{6} \times 1.15 = \quad ^{m}$$

2) soil reaction

$$q = \frac{811.27}{6.00 \times 6.00} \times \left(1 \pm \frac{6 \times 0.49}{6.00}\right) = \begin{cases} 23.64 \ ^{t}/_{m^2} \\ 21.43 \ " \end{cases} < 58 \ ^{t}/_{m^2}$$

3) check for sliding

$$F = \frac{0.6 \times 811.27}{6.00 \times 6.00} = 14. \quad > F_a = 1.5 \times {}^{1}/_{1.15} = 1.3$$

case 3 (HA loading + wind A)

	N (t)	x (m)	Nx (tm)	H (t)	y (m)	HY (tm)
WDL	470.4	—	—	—	—	—
WLL	200.8	—	9.75	—	—	—
W . Ws	340.87	—	—	—	—	—
wind pressure A-1	—	—	—	12.51	16.941	211.93
wind pressure A-3	—	—	—	1.85	8.70	16.10
TOTAL	1012.07		9.75	14.36		228.03

1) check for eccentricity

$$e = \frac{9.75 + 228.03}{1012.07} = 0.23^{m} < \frac{B}{6} = 1.15 = 1.15^{m}$$

2) soil reaction

$$q = \frac{1012.07}{6.00 \times 6.00} \times \left(1 \pm \frac{6 \times 0.23}{6.00}\right) = \begin{cases} 34.58 \ t/m^2 \\ 21.65 \end{cases} < 58 \ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1012.07}{14.36} = 42 > Fa = 1.3$$

case 1 (HA loading + wind (A + C))

		N (t)	x (m)	N x (tm)	H (t)	y (m)	H y (tm)
WDL		4704	----	----	----	----	----
WLL		200.8	----	9.75	----	----	----
W . Ws		340.87	----	----	----	----	----
wind pressure	A-1	----	----	----	12.51	----	211.93
	A-2	----	----	----	1.85	----	16.10
	C	8.98	----	----	----	----	----
TOTAL		1021.05		9.75	14.36		228.03

1) check for eccentricity

$$e = \frac{9.75 + 228.03}{1021.05} = 0.23^{m} < \frac{B}{6} = 1.15 = 1.15^{m}$$

2) soil reaction

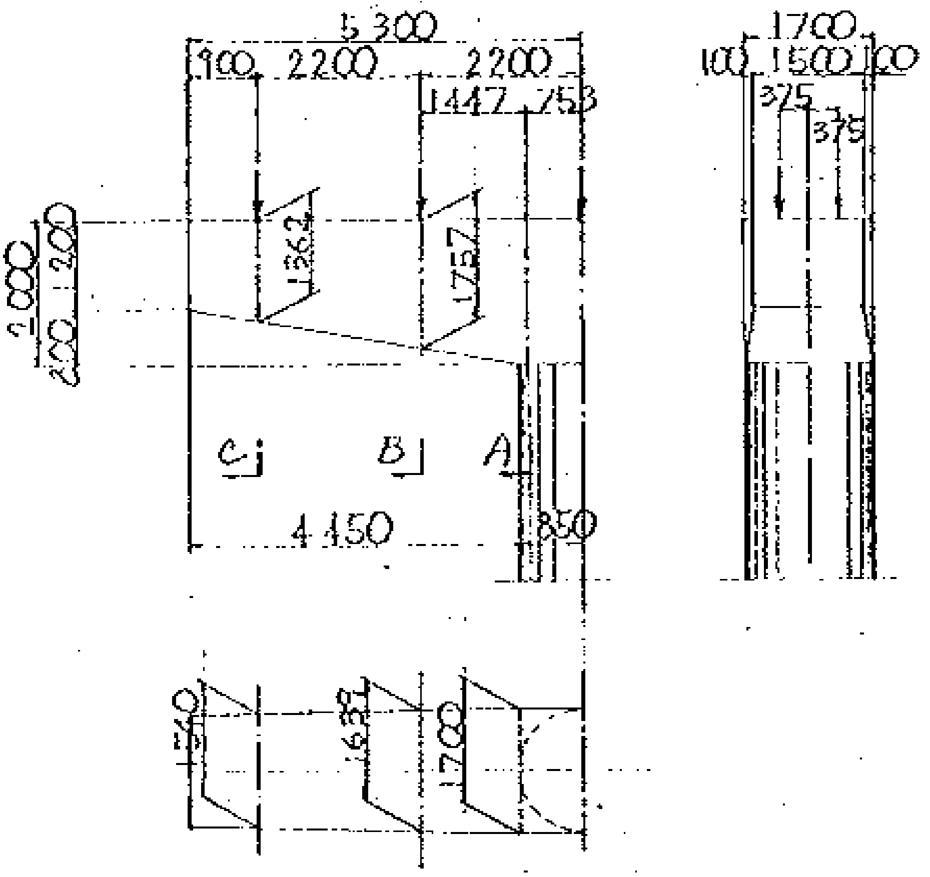
$$q = \frac{1021.05}{6.00 \times 6.00} \times \left(1 \pm \frac{6 \times 0.23}{6.00}\right) = \begin{cases} 34.89 \ t/m^2 \\ 21.84 \ " \end{cases} < 58 \ t/m^2$$

3) check for sliding

$$F = \frac{0.6 \times 1021.05}{14.36} = 43. \quad > Fa = 1.3$$

§ 5 CALCULATION OF BEAM SECTION

5 - 1 dimension of beam and load



		G 1	G 2	G 3	G 4	G 5
WDL		83.8	92.4	95.2	96.2	103.0
WLL	U A	59.2	58.6	55.8	57.1	48.6
	U B	63.4	59.7	59.8	67.7	48.0
TOTAL	U A	143.0	151.0	151.0	153.3	151.6
	U B	147.2	152.1	155.0	163.9	151.0

5-2 sectional force of beam (HA loding)

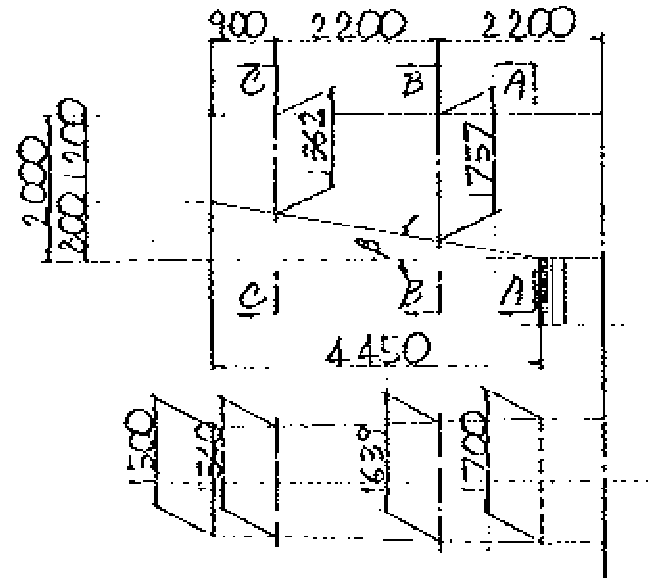
			S (t)	X (m)	M (tm)
A - A	W 1	0.097×1.70×2.00×2.41	0.79	0.0485	0.04
	W 2	½×(1.50+1.70)×4.45×1.20×2.41	20.59	2.368	48.76
	W 3	⅙×0.80×4.45×(2×1.70+1.50)×2.41	7.01	1.55	10.87
	R 1	------	151.0	1.447	218.50
	R 2	------	143.0	3.647	521.52
	Σ		322.39		799.69
B - B	W 1	½×(1.50+1.639)×3.10×1.20×2.41	14.07	1.573	22.13
	W 2	⅙×0.557×3.10×(2×1.639+1.50)×2.41	3.31	0.798	2.64
	R 1	------	(151.0)	0	0
	R 2	------	143.0	2.200	314.60
	Σ		311.38 (160.38)		339.37
C - C	W 1	½×(1.50+1.54)×0.90×1.20×2.41	3.96	0.452	1.79
	W 2	⅙×0.162×0.90×(2×1.54+1.50)×2.41	0.27	0.299	0.08
	R 1	------	143.0	0	0
	Σ		147.23 (4.23)		1.87

5 - 2 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.

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

	A - A	B - B	C - C			
M	799.69	339.37	1.87			
N	——	——	——			
S	322.39	311.38	147.23			
b	200	164	154			
h	185	166	126			
d'	15	15	10			
As	$^{14}_{13}$ > D32 217.08	14 - D32 112.56	12 - D32 96.48			
As'	——	——	——			
f/d	0	0				
M'/bd^2	11.68	7.51				
S/bd	8.71	11.44				
n·P	0.088	0.062				
C	6.62	7.51				
S	12.82	17.90				
Z	1.13	1.11				
σ_c	77	56				
σ_s	2247	2016				
τ	9.8	13				
σ_{ca}	83	→	→			
σ_{sa}	2346	→	→			
τ_a	8.2	→	→			

5 - 4 check for stirrup



$\tan\beta = \dfrac{0.80}{4.45}$

$= 0.1798$

	M (t·m)	S (t)	d (m)	$\frac{M}{d}(\tan\beta)$ (t)	S_h (t)
section A-A	799.69	322.39	1.850	77.72	244.67
B-B	339.37	311.38	1.660	36.76	274.62
C-C	1.87	147.23	1.260	0.27	146.96

$$S_h = S - \frac{M}{d}(\tan\beta) \quad (t)$$

	b (cm)	d (cm)	τ_a (kg/cm²)	S_c (t)	S_h' (t)
section A-A					
B-B					
C-C					

~~$S_c = \tau_a \cdot b \cdot d \cdot 10^{-3}$~~ (t)

~~$S_h' = S_h - S_c$ (t)~~

check for stirrups (Section B-B)

$$\tau = \frac{Sh}{b \cdot d} \cdot Z$$

$$= \frac{274.62 \times 10^3}{164 \times 166} \times 1.11 = 11.20^{\,kg/cm^2} > \tau_0 = 8.2$$

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

$$Sh' = Sh - Sc$$

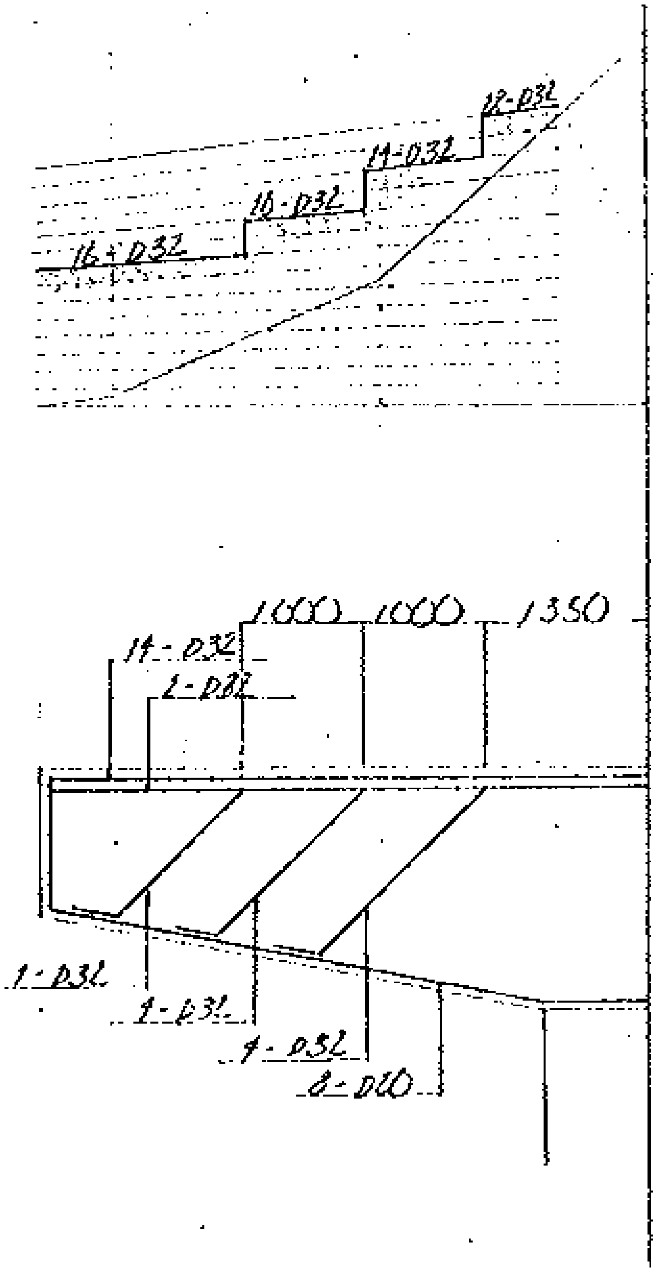
$$Sc = \tau a \times b \times d \times \frac{1}{Z}$$

$$= 0.0 \times 164 \times 166 \times \frac{1}{1.11} = 0$$

$$\text{req. } A_w = \frac{274.62 \times 10^3 \times 15}{1780 \times 166} \times 1.11 = 15.47^{\,cm^2}$$

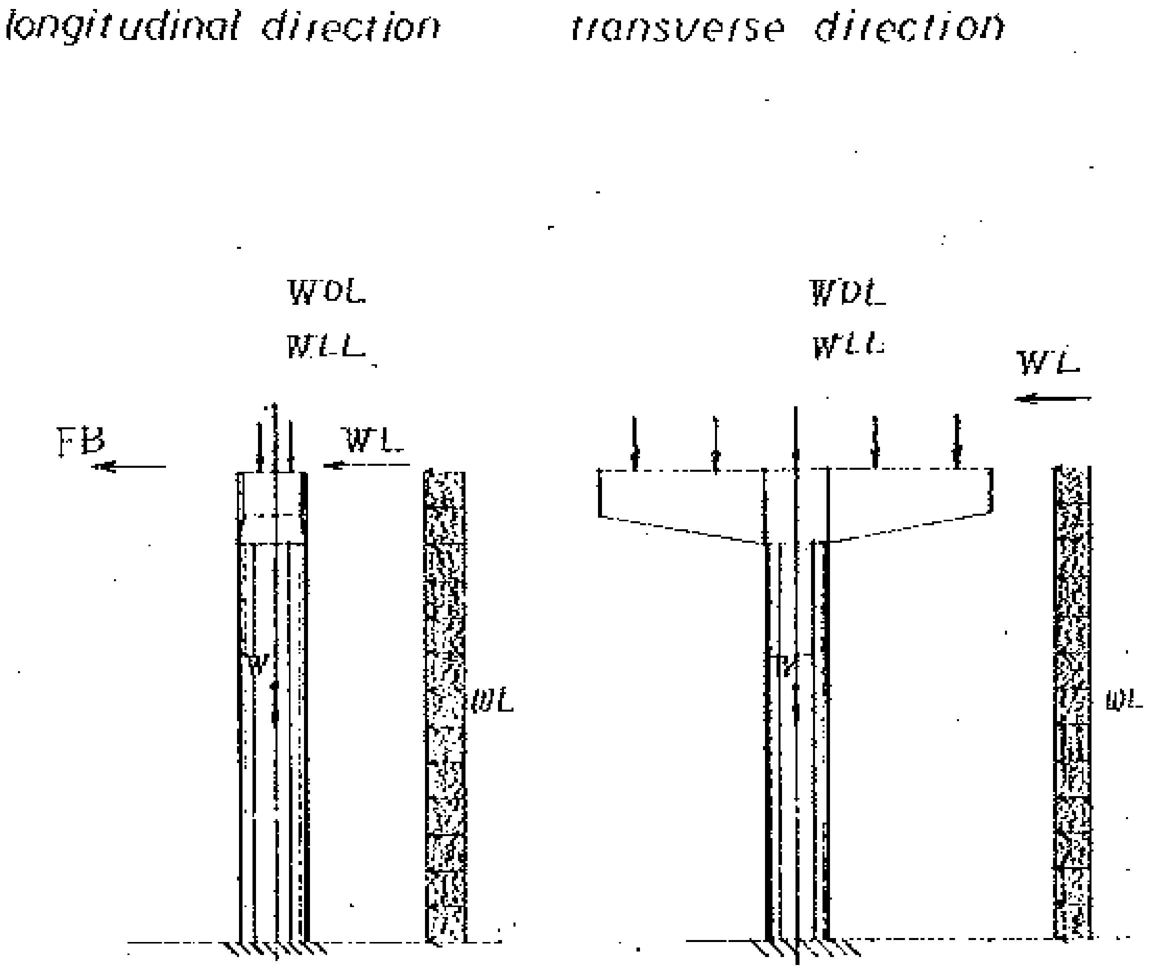
$$\phi\ 20 - \text{c/c } 150 \qquad n = 6$$

$$\text{Used } A_w = 6 \times 3.14 = 18.84^{\,cm^2} > \text{req. } A_w = 15.47^{\,cm^2}$$


12-D32
14-D32
16-D32
16-D32
1000
1000
1350
14-D32
2-D32
1-D32
4-D32
4-D32
8-D20

§ 6 CALCULATION OF COLUMN SECTION

6 - 1 dimension of column and load

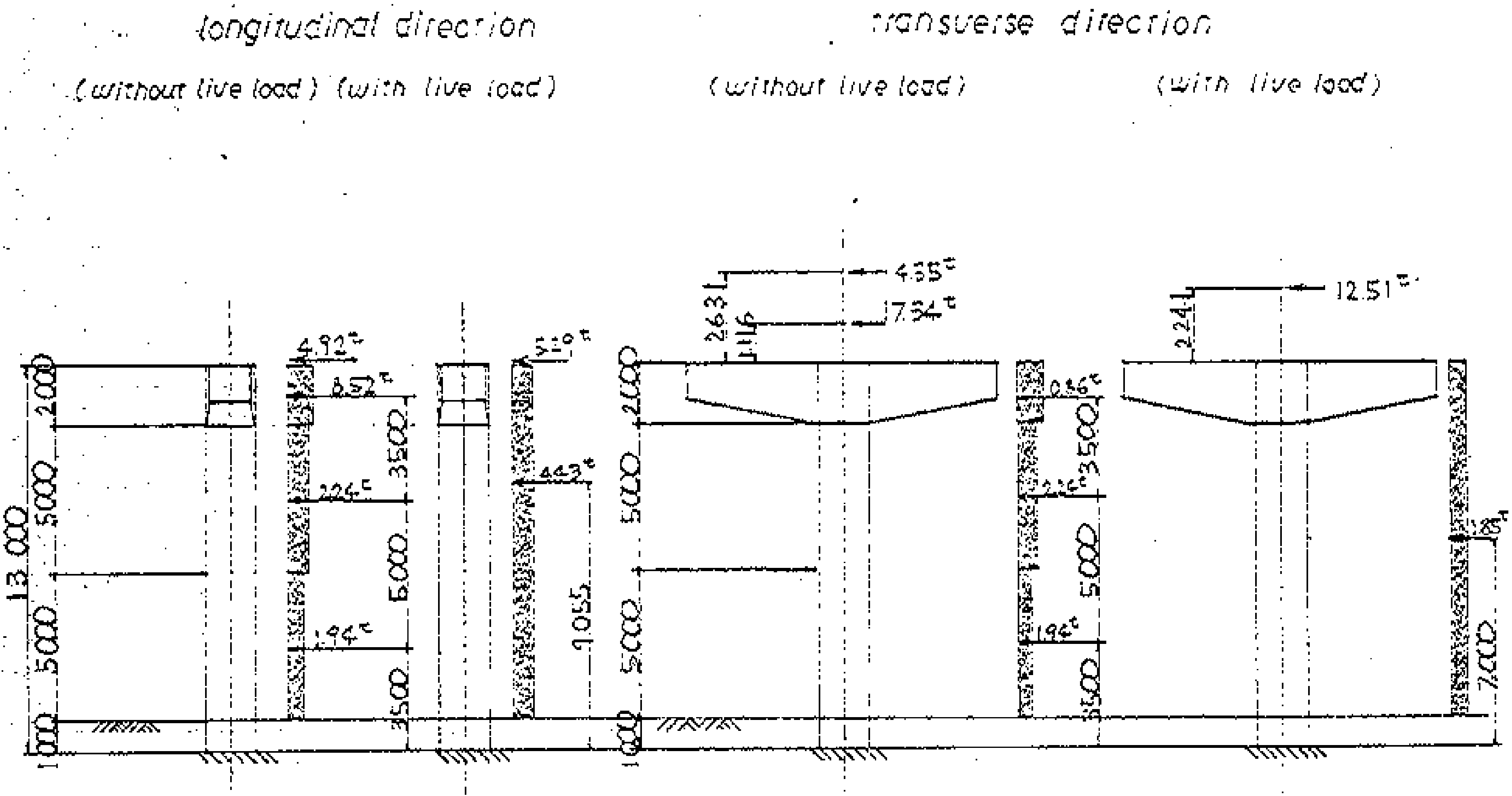


longitudinal direction

transverse direction

	HA loading		HB loading	
dead load of deck	235.2	235.2	235.2	235.2
live load	113.4	87.4	82.55	78.85
longitudinal forces	12.9		19.1	

wind load


longitudinal direction
(without live load) (with live load)
transverse direction
(without live load)
(with live load)
13 000
1000
5000
5000
2000
4.92t
6.52t
2.26t
1.94t
3500
6000
3500
5.10t
4.43t
9055
1000
5000
5000
2000
2.631
1.116
4.35t
7.54t
0.56t
2.26t
1.94t
3500
5000
3500
2.241
12.51t
1.85t
7000

6 - 2 sectional force of column

1) longitudinal direction

			N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
case 1 (HA loading)	WDL		470.4	—	—	—	—	—
	WLL		200.8	—	9.75	—	—	—
	F B		—	—	—	12.90	13.00	167.70
	W		129.29	—	—	—	—	—
			800.49		9.75	12.90		167.70
case 2 (wind)	WDL		470.4	—	—	—	—	—
	W		129.29	—	—	—	—	—
	wind	B-1, 2	—	—	—	4.92	13.00	63.96
		B-3	—	—	—	12.70	—	128.07
	(α = 1.15)		59969 (521.47)		—	17.62 (15.32)		192.03 (166.98)
case 3 (HA loading + wind)	WDL		470.4	—	—	—	—	—
	WLL		200.8	—	—	—	—	—
	F B		—	—	9.75	12.90	13.00	167.70
	W		129.29	—	—	—	—	—
	wind	B-1	—	—	—	5.29	13.00	68.77
		B-2	—	—	—	4.43	9.055	40.11
	(α = 1.15)		800.49 (696.08)		9.79 (8.51)	22.62 (19.67)		27658 (24050)

2) transverse direction

			N (t)	x (m)	Nx (tm)	H (t)	y (m)	Hy (tm)
case 1 (wind)	WDL		470.4	——	——	——	——	——
	W		129.29	——	——	——	——	——
	wind	A-1	——	——	——	17.34	14.116	244.77
		A-2	——	——	——	4.35	15.631	67.99
		A-3	——	——	——	5.04	——	36.15
	(J = 1.15)		599.69 (521.47)		——	26.73 (23.24)		348.91 (303.40)
case 2 (HA loading, +wind	WDL		470.4	——	——	——	——	——
	WLL		200.8	——	9.75	——	——	——
	W		129.29	——	——	——	——	——
	wind	A-1	——	——	——	12.51	15.241	190.66
		A-3	——	——	——	1.85	7.00	12.95
	(J = 1.15)		800.49 (696.08)		9.75 (8.48)	14.36 (12.49)		203.61 (177.05)

6 - 3 List of stresses σ_c . σ_s : working stress
σ_{ca} , σ_{sa} : permissible stress

	longitudinal direction			transverse direction	
	case 1	〃 2	〃 3	〃 1	2
M	177.45	166.98	249.01	303.40	185.53
N	800.49	521.47	696.08	521.47	696.08
r	85	———	———	———	——→
rs	75	———	———	———	——→
As	30-D32 241.2	———	———	———	——→
e		32.02	35.77	58.18	26.65
e/r		0.38	0.42	0.68	0.31
M'		610.23	840.68	746.65	777.20
M'/r'		99.37	136.89	121.58	126.55
n p		0.1594	0.1594	0.1594	0.1954
rs/r		0.90	0.90	0.90	0.90
[C]		0.486	0.511	0.6803	0.4535
[S]		0.0601	0.1021	0.4307	0.0075
σ_c		48	70	83	57
σ_s		90	210	786	14
σ_{ca}	83	———	———	———	——→
σ_{sa}	2346	———	———	———	——→

6-4 check for buckling of column

$$P_a = \frac{1}{3} \times (0.85 \cdot \sigma_{ck} \cdot A_c + \sigma_{sy'} \cdot A_s)$$

$$\alpha = 1.45 - 0.03 \cdot \frac{h_e}{d}$$

$$h_e = (13.00 + 1.70) \times 2 = 29.40^{m}$$

$$P_a = 1/3 \times (0.85 \times 250 \times \pi/4 \times 170^2 + 2500 \times 241.2)$$

$$= 1809{,}000^{kg} \quad 1809^{t}$$

$$\alpha = 1.45 - 0.03 \times 29.40/1.70$$

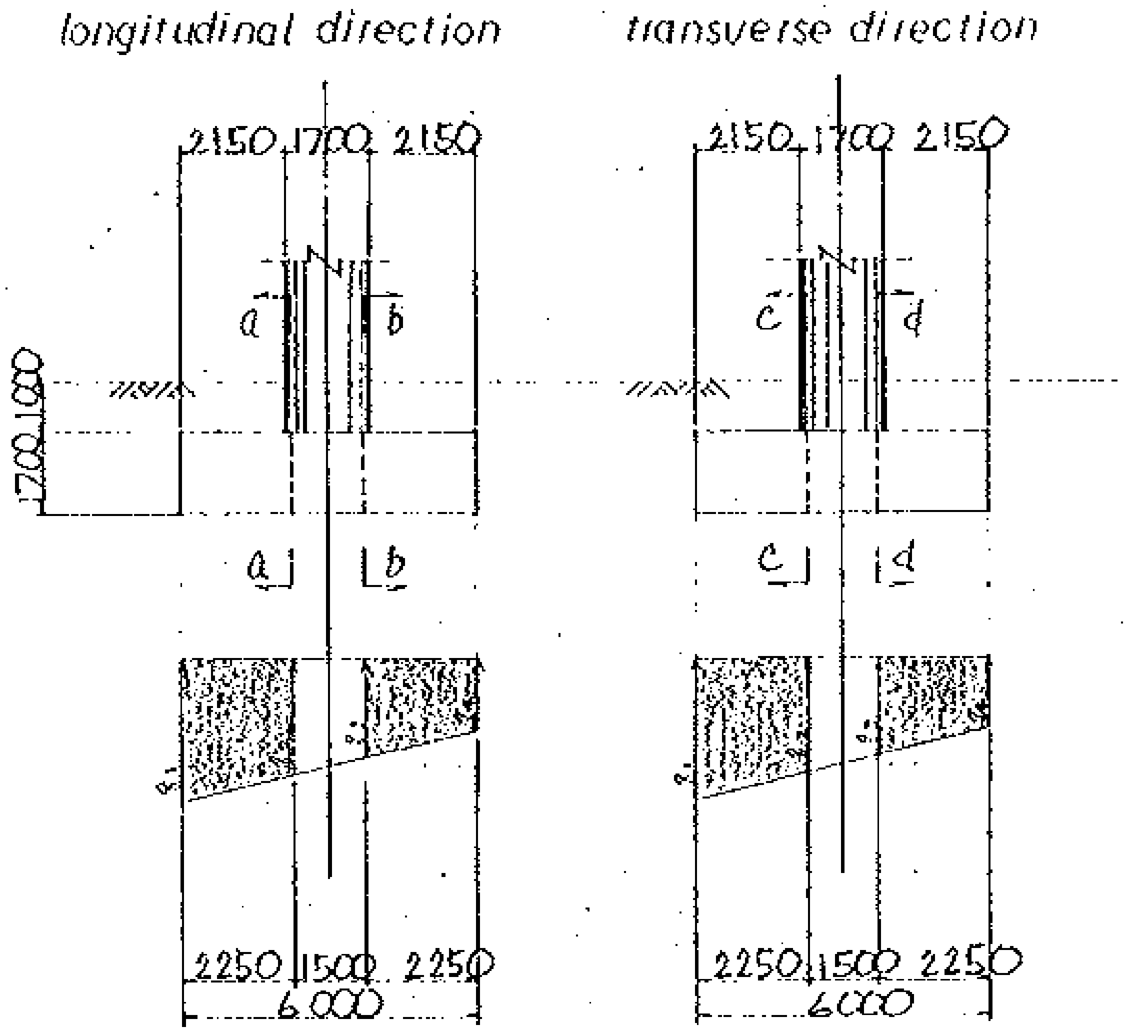
$$= 0.97$$

$$\therefore P_{aa} = 1809 \times 0.97$$

$$= 1755^{t} \qquad > P_N = 800.49^{t}$$

§ 7 CALCULATION OF FOOTING SECTION

7-1 dimension and soil reaction


longitudinal direction

transverse direction

case	longitudinal direction				transverse direction			
	q_1	q_2	q_3	q_4	q_1	q_2	q_3	q_4
1	33.74	29.52	26.71	22.49	28.11	28.11	28.11	28.11
2	28.85	24.12	20.96	16.23	23.64	22.81	22.26	21.43
3	37.11	30.36	25.87	19.12	34.58	29.75	26.50	21.65
4	37.44	30.63	26.10	19.29	34.89	30.00	26.73	21.84

section A – A

			S (t)	x (m)	Sx (tm)
case 1	wd	(1.70×241 + 1.00×1.9) 2.25 × 6.00	80.96	1.125	91.08
	q	½×(33.74 + 29.52)×2.25×6.00	-427.01	1.150	-491.06
			346.05		399.98
case 2	wd	------	80.96	—	91.08
	q	½×(28.85 + 24.12)×2.25×6.00	-357.55	1.158	-414.04
(α = 1.15)			276.59 (240.51)		322.96 (280.83)
case 3	wd	------	80.96	—	91.08
	q	½×(37.11 + 20.36)×2.25×6.00	-455.42	1.163	-529.65
(α = 1.15)			374.46 (325.62)		438.57 (381.37)
case 4	wd	------	80.96	—	91.08
	q	½×(37.44 + 30.63)×2.25×6.00	-459.47	1.163	-534.36
(α = 1.15)			378.51 (329.14)		443.28 (385.46)

section B – B

M upper … 0

section c – c

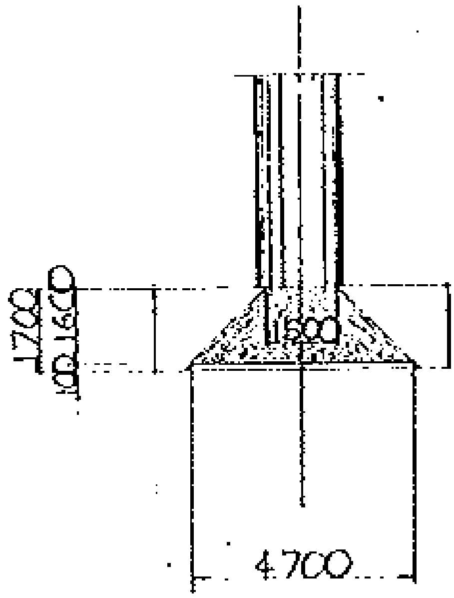
			S (t)	x (m)	Sx (tm)
case 1	wd	————	80.96	——	91.08
	q	28.11 × 2.25 × 6.00	-379.49	1.125	-426.93
			298.53		335.85
case 2 ($\alpha = 1.15$)	wd	————	80.96	——	91.08
	q	½ × (23.64 + 22.81) × 2.25 × 6.00	-313.54	1.132	-354.93
			232.58 (202.24)		263.85 (229.43)
case 3 ($\alpha = 1.15$)	wd	————	80.96	——	91.08
	q	½ × (34.58 + 29.75) × 2.25 × 6.00	-434.23	1.153	-500.67
			353.27 (307.19)		409.59 (356.17)
case 4 ($\alpha = 1.15$)	wd	————	80.96	——	91.08
	q	½ × (34.89 + 30.0) × 2.25 × 6.00	-438.01	1.153	-505.03
			357.05 (310.48)		413.95 (359.96)

section d – d

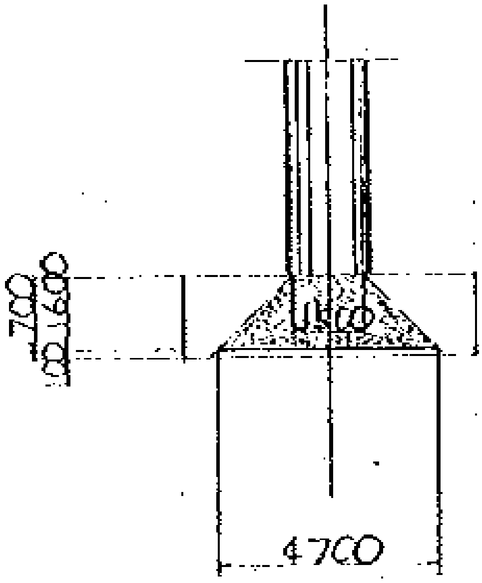
Muppor = 0

7 - 2 calculation of members

longitudinal direction



transverse direction



$$B_1 = 1.50 + 1.60 \times 2 = 4.70^{m}$$

$$B_2 = 1.50 + 1.60 \times 2 = 4.70^{m}$$

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

	a - a	b - b		c - c	d - d	
M	399.98			359.96		
N	———			———		
S	346.05			310.48		
b	470			470		
h	160			160		
d'	10			10		
As	D20@125 119.32			D20@125 119.32		
As'	———			———		
t/d	0			0		
M'/bd^2	3.32			2.99		
S/bd	4.60			4.13		
n·P	0.0238			0.0238		
C	10.94			10.94		
S	44.98			44.98		
Z	1.07			1.07		
σ_c	36			33		
σ_s	2243			2019		
τ	4.9			4.4		
σ_{ca}	83					
σ_{sa}	2346					
τ_a	2.35					

check for stirrups

$$\tau = \frac{Sh}{b \cdot d} \times z$$

$$= \frac{346.05 \times 10^3}{470 \times 160} \times 1.07 = 4.92^{kg/cm^2} > \tau_o = 2.35$$

$$req.\ A_w = \frac{Sh' \times a}{\sigma_{so} \cdot d} \times z \quad (cm^2)$$

$$Sh' = Sh - S_c$$

$$S_c = \tau_o \times b \times d \times \frac{1}{z}$$

$$= 2.35 \times 470 \times 160 \times \frac{1}{1.07} = 165\,160^{kg}$$

$$Sh' = (346.05 - 165.16) \times 10^3 = 180.89 \times 10^3$$

$$req.\ A_w = \frac{180.89 \times 10^3 \times 25}{1780 \times 160} \times 1.07 = 16.99^{cm^2}$$

effective width 4.7ᵐ $req.\ A_w = 16.99/4.7 = 3.61^{cm^2}$

$\Phi 16$ - c.c. 250 $n = 2$

$$Used\ A_w = 2 \times 2.01 = 4.02^{cm^2} > req.\ A_w = 3.61^{cm^2}$$

check for stirrups

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

$$= \frac{310.48 \times 10^3}{470 \times 160} \times 1.07 = 4.42^{\,kg/cm^2} > \tau_a = 2.35$$

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

$$Sh' = Sh - Sc$$

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

$$= 2.35 \times 470 \times 160 \times \frac{1}{1.07} = 165160^{\,kg}$$

$$Sh' = (310.48 - 165.16) \times 10^3 = 145.32 \times 10^3$$

$$\text{req. } A_w = \frac{145.32 \times 10^3 \times 25}{1780 \times 160} \times 1.07 = 13.65^{\,cm^2}$$

effective width 4.7m $\quad$ $\text{req } A_w = 13.65 / 4.7 = 2.90^{\,cm^2}$

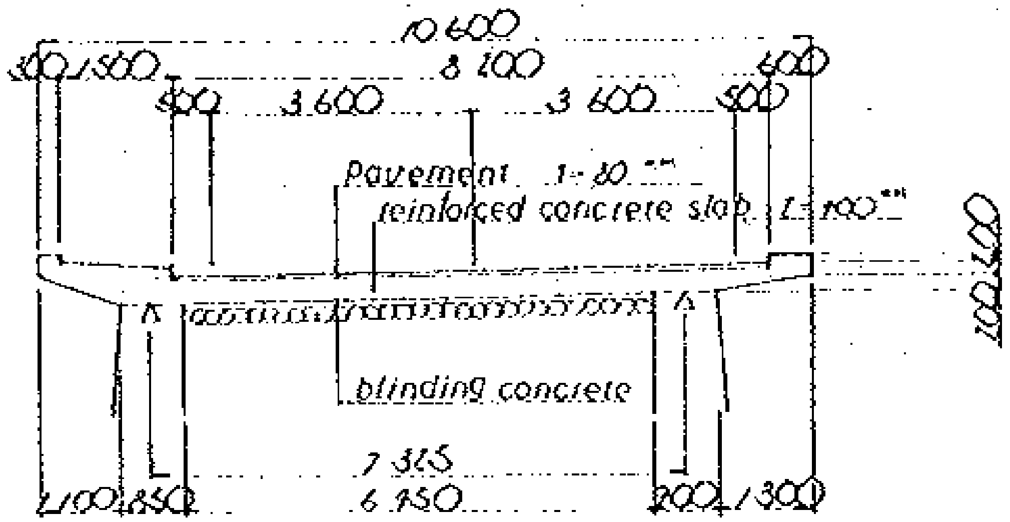
$\Phi 16$ - ctc 250 $\quad n = 2$

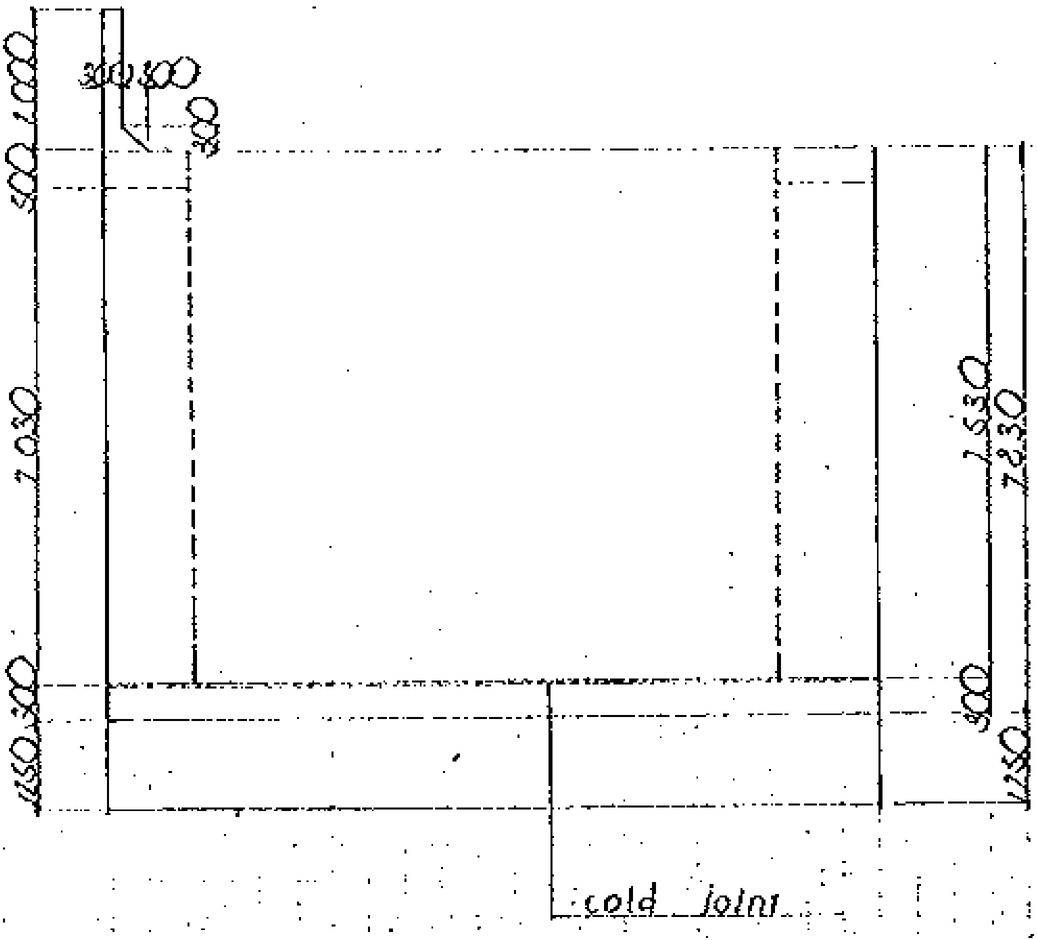
$$\text{Used } A_w = 2 \times 2.01 = 4.02^{\,cm^2} > \text{req. } A_w = 2.90$$

§§ 6. ABUTMENT

[S.T LOUIS A-LINE
G.R.N.W A-LINE]

§ 1 DIMENSIONS





§ 2 CALCULATION OF SLAB SECTION

2 - 1 intermediate slab

transverse bending moment

wearing surface $M_{d1} = 1/8 \times 0.184 \times 7.325^2 = 1.23^{tm}$

slab concrete $M_{d2} = 1/8 \times 0.964 \times 7.325^2 = 6.46$

$\Sigma M_d = 7.69^{tm}$

live load

$$M = \left(\frac{S}{2.32 \cdot S + 8 \cdot C} + \frac{0.4 \cdot S}{2 \cdot a + b} - 0.14 \right) P$$

S : span. 7.425^{m}

c : diameter of equivalent circular contract area

$0.19 + 2 \times 0.08 = 0.35^{m}$

a : distance of vehicle 0.90^{m}

b : 1.80^{m}

P : $112.5^{kN} = 11.47^{t}$

$$M = \left(\frac{7.425}{2.32 \times 7.425 + 8 \times 0.35} + \frac{0.4 \times 7.425}{2 \times 0.90 + 1.80} - 0.14 \right) \times 11.47$$

$\fallingdotseq 11.98^{tm}$

$\Sigma M = M_d + M_l$

$= 7.69 + 11.98 = 19.67^{tm}$

2-2 cantilever slab

1) bending moment due to dead load

rail $0.112 \times 1.30 = 0.15^{tm}$

kerb $0.60 \times 0.23 \times 2.41 \times 1.30 = 0.43$

wearing $1.00 \times 0.08 \times 2.30 \times 0.50 = 0.09$

slab 1 $\frac{1}{2} \times (0.20 + 0.40) \times 1.30 \times 2.41 \times 0.578 = 0.54$

〃 2 $0.30 \times 0.40 \times 2.41 \times 0.15 = 0.04$

$\Sigma Md = 1.25^{tm}$

2) bending moment due to collision load

$$Mc = \frac{6.429}{3} \times (0.11 + 0.995 + 1.280) \times \frac{1}{2} = 3.30^{tm}$$

3) bending moment due to live load

$$Mx = \frac{P}{\pi} \cdot \frac{1}{1 + \left(\frac{y}{u}\right)^2}$$

$y = 0 \qquad u = 0.75^{m} \qquad P = 11.47^{t}$

$$Mx = \frac{11.47}{\pi} \times 1.00 = 3.65^{tm}$$

$$\Sigma M = Md + Mc + Mx$$

$$= 1.25 + 3.30 + 3.65 = 8.20^{tm}$$

2 - 3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.

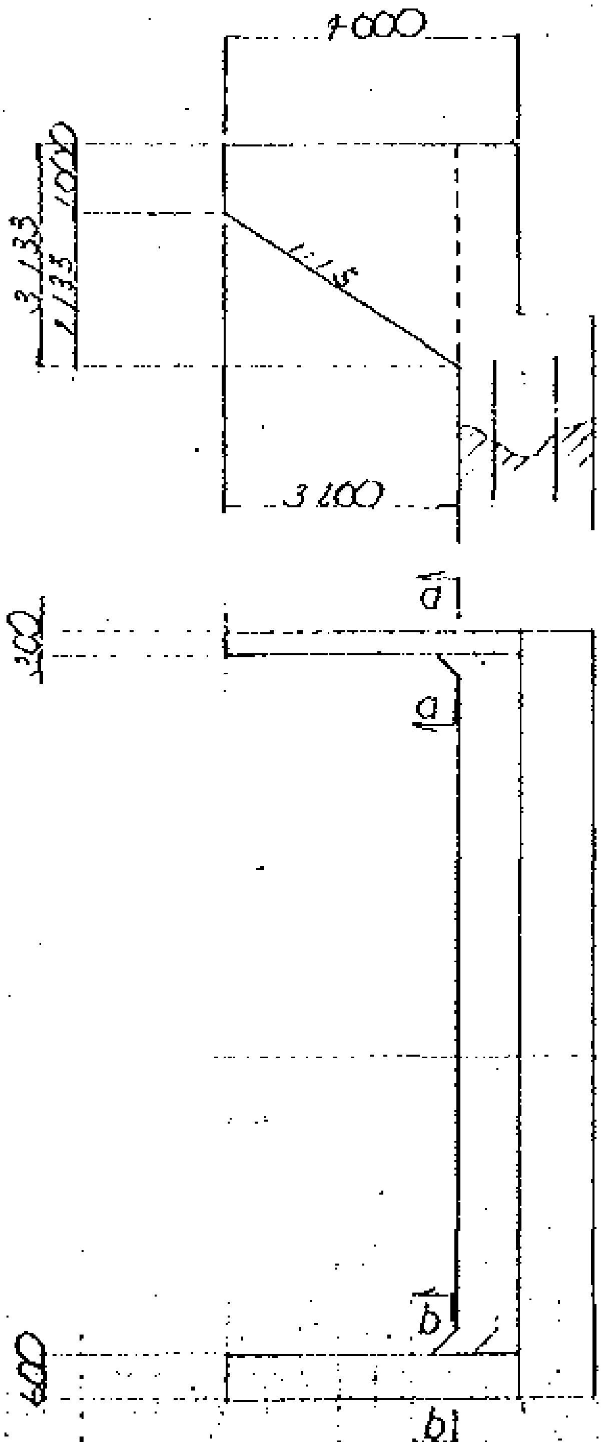
$\sigma_{ca}, \sigma_{sa}, \tau_a$: permissible stress.

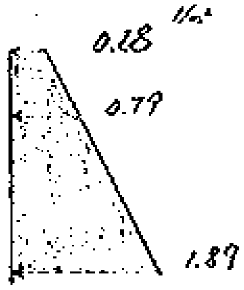
	intermediate slab			cantilever slab		
M	12.67			8.10		
N	——			——		
S	——			——		
b	100			100		
h	33			33		
d'	7			7		
As	D15 @ 150 32.75			D16 @ 150 13.41		
As'	——			——		
f/d	0			0		
M'/bd^2	18.06			7.53		
S/bd	0			0		
n·P	0.1489			0.0610		
C	5.57			1.36		
S	1.80			18.20		
Z	1.16			1.10		
σ_c	100			57		
σ_s	2113			2055		
τ	0			0		
σ_{ca}	103			83		
σ_{sa}	2933			2346		
τ_a	4.69	$\frac{As\times100}{bd}=1.6$		2.35		

§ 3 CALCULATION OF WING SECTION

G.R.N.W (A1. A2)

3 - 1 dimension and loading





3 - 2 sectional force of wing

$$(M_{max})\,y = \frac{\gamma \cdot h - 2 \cdot a}{3 \cdot \gamma} \quad (m)$$

$$= \frac{1.9 \times 3.133 - 2 \times 1.02}{3 \times 1.9}$$

$$= 0.686^{m} \longrightarrow 1.00^{m}$$

$$M = \frac{1}{2} \times 0.79 \times 3.20^2 = 4.04^{tm}$$

$$S = 0.79 \times 3.20 = 2.53^{t}$$

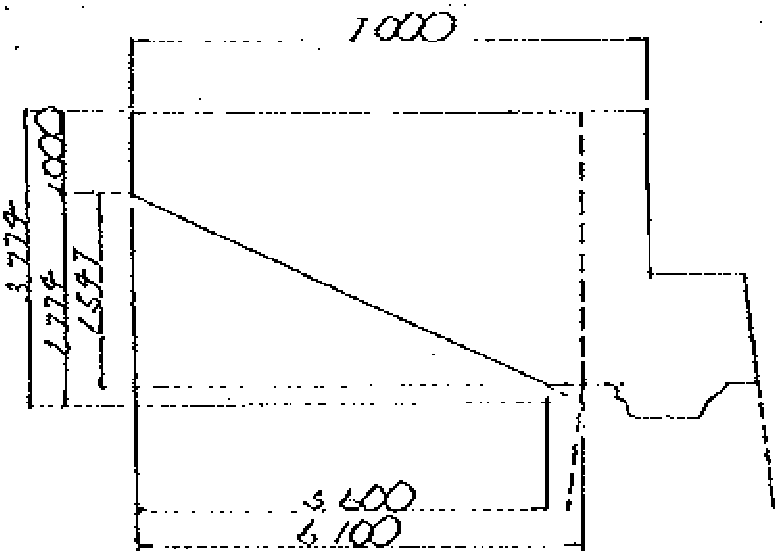
3-3 list of stresses $\sigma c . \sigma s . \tau$: working stress

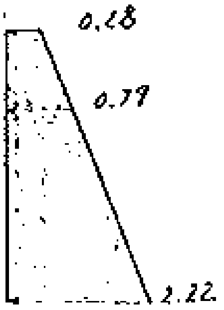
$\sigma ca, \sigma sa, \tau a$: Permissible stress.

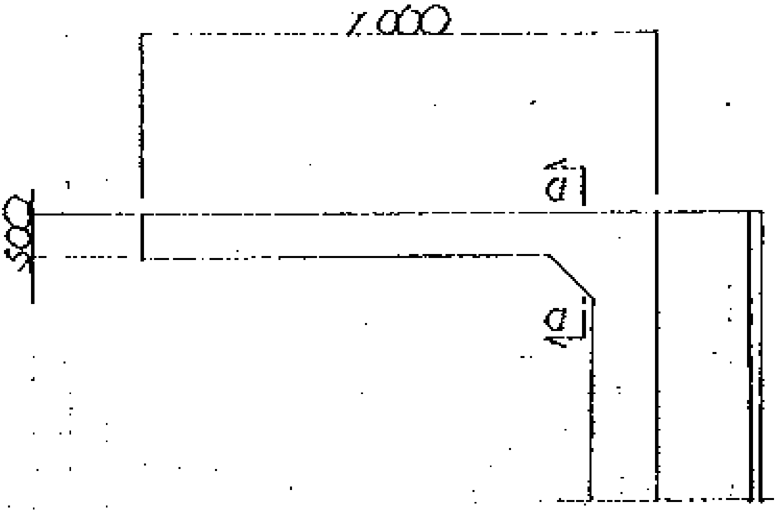
	a-a	b-b			
M	4.04	4.04			
N	----	----			
S	1.53	2.53			
b	100	100			
h	23	53			
d'	7	7			
As	D22 ctc150 11.56	D16 ctc150 8.04			
As'	----	----			
f/d	0	0			
M'/bd²	2.64	1.44			
S/bd	0.55	0.64			
n·P	0.0524	0.0228			
C	7.99	11.15			
S	21.00	16.03			
Z	1.10	1.06			
σc	61	16			
σs	1572	1013			
τ	0.6	0.3			
σca	83	83			
σsa	2346	2346			
τa	3.47	2.35			

ST LOUIS (A1)

3 - 1 dimension and loading







3 - 2 sectional force of wing

$$(M_{max})\, y = \frac{\gamma h - 2q}{3 \cdot \gamma} \quad (m)$$

$$= \frac{1.9 \times 3.77 - 2 \times 1.02}{3 \times 1.9}$$

$$= 0.89 \dashrightarrow 1.00^{m}$$

$$M = \frac{1}{2} \times 0.79 \times 6.10^{2} = 14.70^{t\cdot m}$$

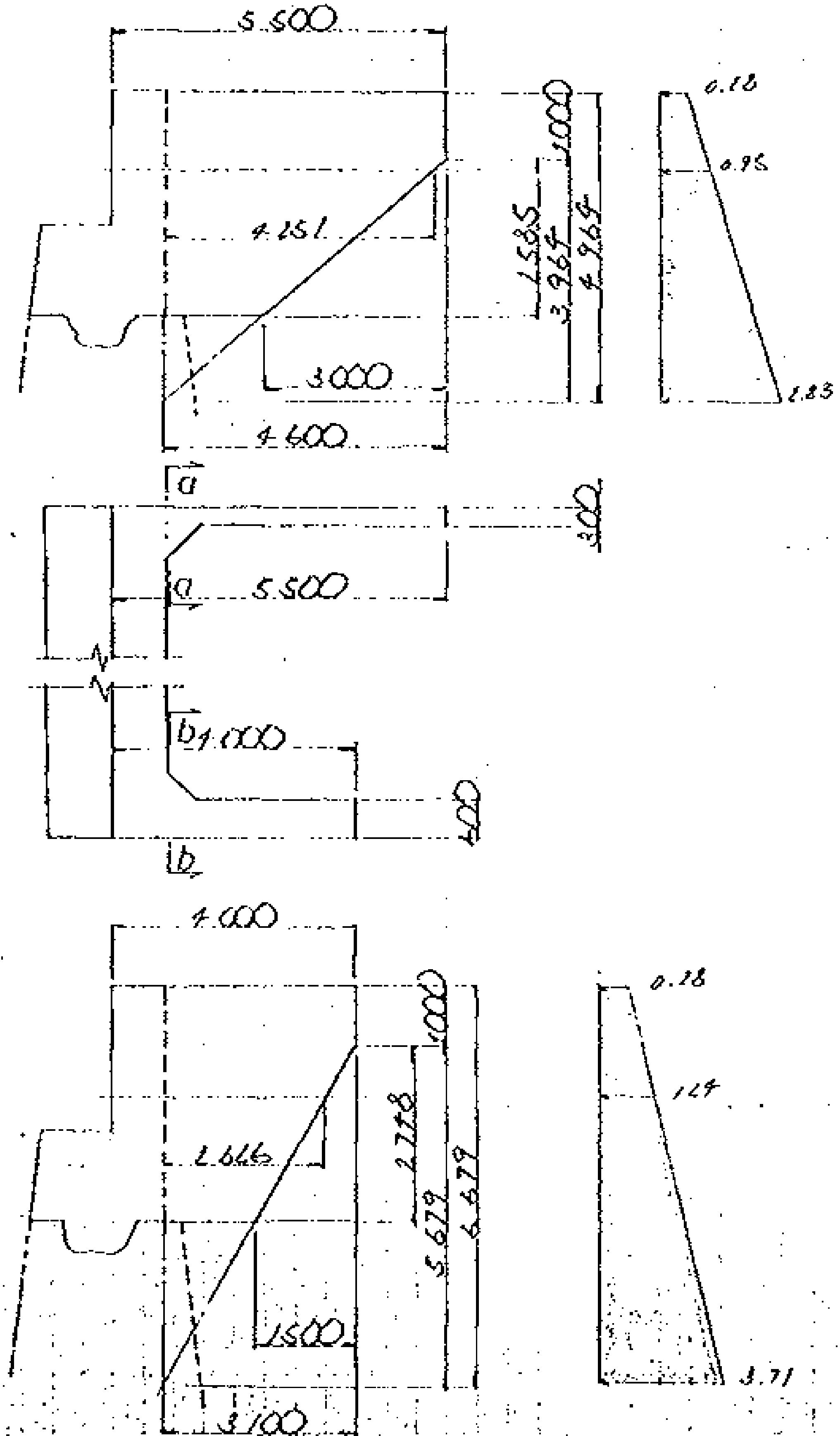
$$S = 0.79 \times 6.10 = 4.82^{t}$$

3 - 3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.

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

	a - a					
M	14.70					
N	——					
S	4.82					
b	100					
h	43					
d'	7					
As	D16 @ 125 16.08					
As'	——					
f/d	0					
M'/bd^2	1.95					
S/bd	1.12					
n·P	0.0561					
C	7.79					
S	17.69					
Z	1.10					
σ_c	62					
σ_s	1318					
τ	12					
σ_{ca}	63					
σ_{sa}	2346					
τ_a	2.35					

S.T LOUIS (A 2)


5.500
1000
1585
3.969
4.969
0.18
0.95
2.83
4.251
3000
4.600
a
300
a
5.500
b
1.000
400
b
4.000
1000
0.18
124
2.666
2778
5.679
6.679
1500
3.71
3.100

3 − 2 sectional force of wing

(a − a)

$$(M_{max})\, y = \frac{\gamma h - 2q}{3 \cdot \gamma} \quad (m)$$

$$= \frac{1.9 \times 4.264 - 2 \times 1.02}{3 \times 1.9}$$

$$= 1.30$$

$$M = \frac{1}{2} \times 0.95 \times 4.251^2 = 8.58^{tm}$$

$$S = 0.95 \times 4.251 = 4.04^{t}$$

(b − b)

$$y = \frac{1.9 \times 6.679 - 2 \times 1.02}{3 \times 1.9}$$

$$= 1.868$$

$$M = \frac{1}{2} \times 1.24 \times 2.626^2 = 4.27^{tm}$$

$$S = 1.24 \times 2.626 = 3.26^{t}$$

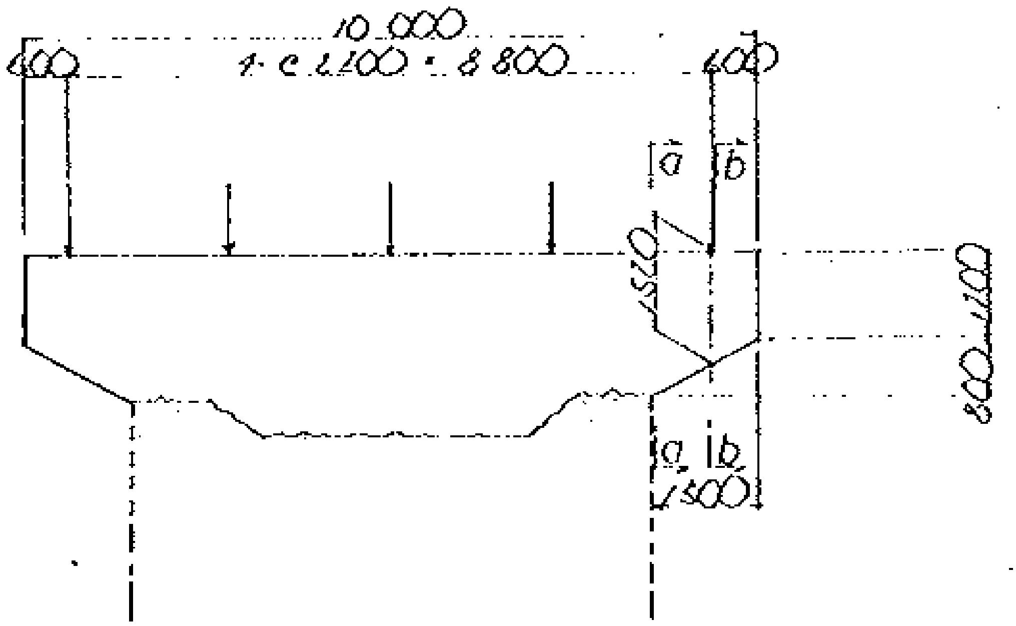
3 - 3 list of stresses $\sigma_c . \sigma_s . \tau$: working stress.
$\sigma_{ca}, \sigma_{sa}, \tau_a$: Permissible stress.

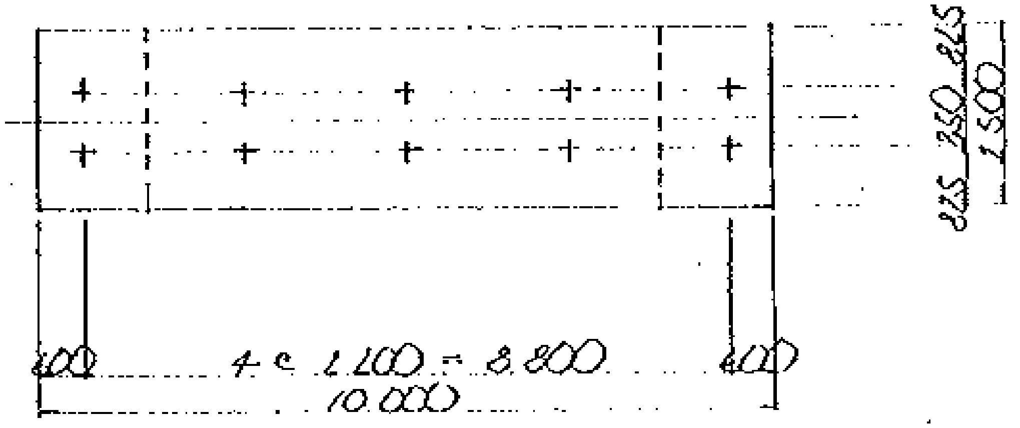
	a - a	b - b				
M	8.58	4.27				
N	———	———				
S	4.04	3.26				
b	100	100				
h	13	53				
d'	7	7				
As	D16, D20 ctc 125 10.60	D16 c 150 8.04				
As'	———	———				
f/d	0	0				
M'/bd^2	16.22	1.86				
S/bd	1.76	0.61				
$n \cdot P$	0.1343	0.0228				
C	4.71	11.15				
S	8.22	46.99				
Z	1.10	1.06				
σ_c	76	17				
σ_s	1779	1072				
τ	1.9	0.6				
σ_{ca}	83	83				
σ_{sa}	2346	2346				
τ_a	2.35	2.35				

§§ 7 PIER

G.R.N.W A-LINE

§ 1 DIMENSIONS





§ 2 REACTION OF SUPERSTRUCTURE

2-1 whole reaction of superstructure

dead load of deck		235.2	235.2
H. A Live load		113.4	87.4
H. B Live load		82.5	78.9
crowd load		10.3	10.3
longitudinal forces	under H. A	12.9	
	under H. B	19.1	

2-2 reaction per each girder

		G 1	G 2	G 3	G 4	G 5
dead load of deck		83.8	92.4	95.2	96.2	103.0
live load	H A	59.2	58.6	55.8	57.1	48.6
	H B	63.4	58.7	52.0	62.7	33.4
TOTAL	H A	143.0	151.0	151.0	153.3	151.6
	H B	147.2	152.1	155.0	163.9	151.0

§ 3 CALCULATION OF BEAM SECTION

3 - 1 sectional force of beam

$$M_A = 1/2 \times (1.20 + 2.00) \times 1.50 \times 2.50 \times 2.41 \times 0.688$$
$$+ 151.60 \times 0.900$$
$$= 146.39^{tm}$$

$$S_a = 1/2 \times (1.20 + 2.00) \times 1.50 \times 2.50 \times 2.41$$
$$+ 151.60$$
$$= 166.06^{t}$$

$$S_b = 1/2 \times (1.20 + 1.52) \times 0.60 \times 2.50 \times 2.41$$
$$+ 151.60$$
$$= 156.52^{t}$$

3 − 2 list of stresses

$\sigma_c . \sigma_s . \tau$: working stress.

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

	a − a	b − b				
M	146.39	----				
N	----	----				
S	166.06	156.52				
b	150	150				
h	185	137				
d'	15	15				
As	10 - D25 49.10	10 - D25 49.10				
As'	----	----				
f/d	0	0				
M'/bd^2	1.71	----				
S/bd	1.59	0				
n·P	0.0159	0.0215				
C	12.97	11.41				
S	66.63	19.65				
Z	1.06	1.07				
σ_c	22	0				
σ_s	1709	0				
τ	3.6	4.6				
σ_{ca}	83	83				
σ_{sa}	2346	2346				Used As
τ_a	8.2	8.2	Min. Stirrup	As = 0.0012×b = 0.0012×150	×S ×20 = 6 cm^2	Φ12 ctc200 1:6 1.13×6 = 6.78 cm^2

2. Over Bridges

Contents

Vol. 1 PAILLES OV. BR. P_1 , P_2 , P_3 PIERS

PIER IN PAILLES O.V.

(P - 1)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight $H = 8.71$ m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

$2.407\ t/m^3$

4 Allowable stresses of reinforced concrete

1.) concrete (grade 25 , BS 5400)

cube strength at 28 days $\sigma_{ck} = 255\ kg/cm^2$

bending stress $\sigma_{ca} = 85$

direct stress $\sigma_{ca} = 64.3$

shear stress $\tau_a = 8.2$

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

specified characteristic strength

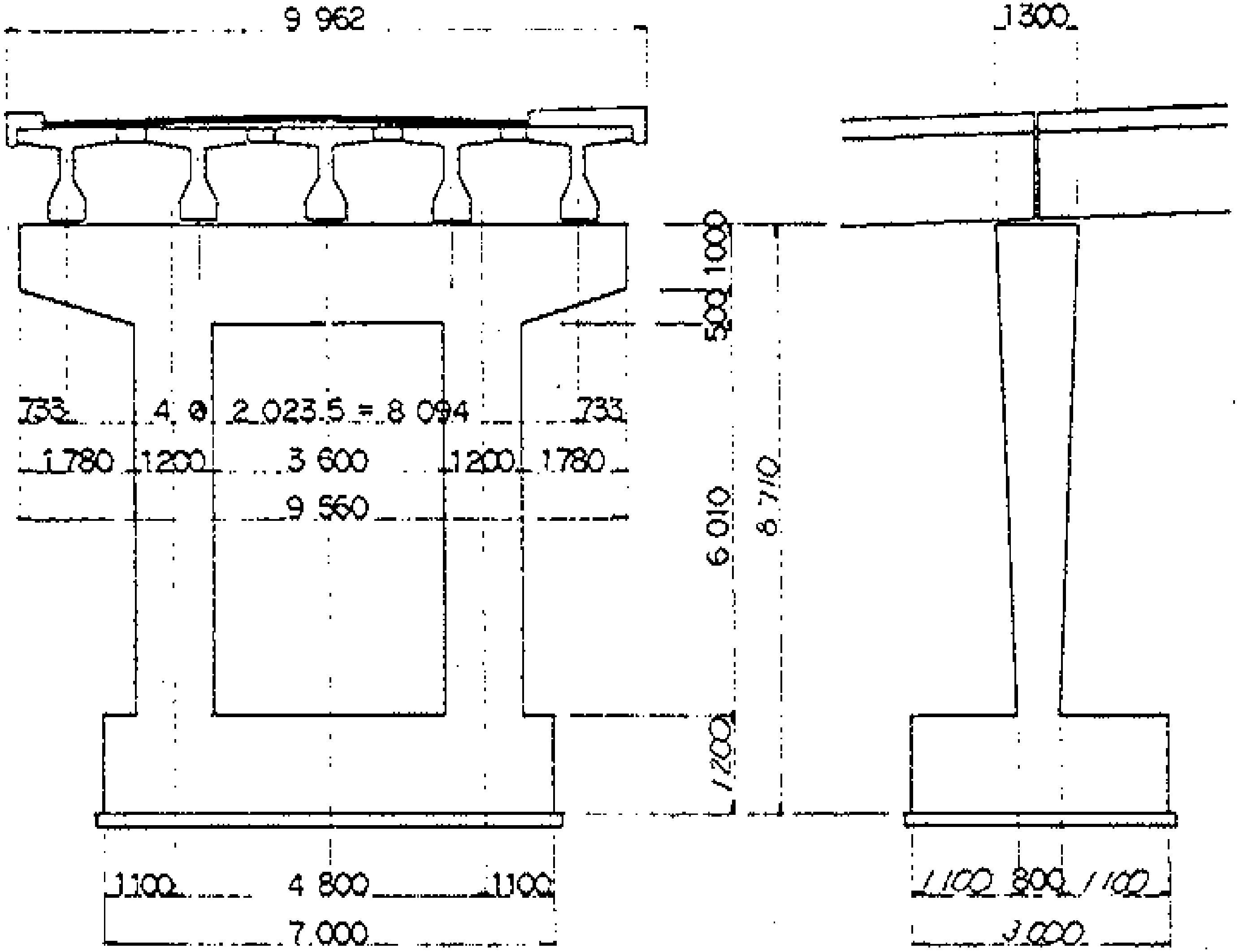
$\sigma_{su} = 4\ 180\ kg/cm^2$

permissible tensile stress

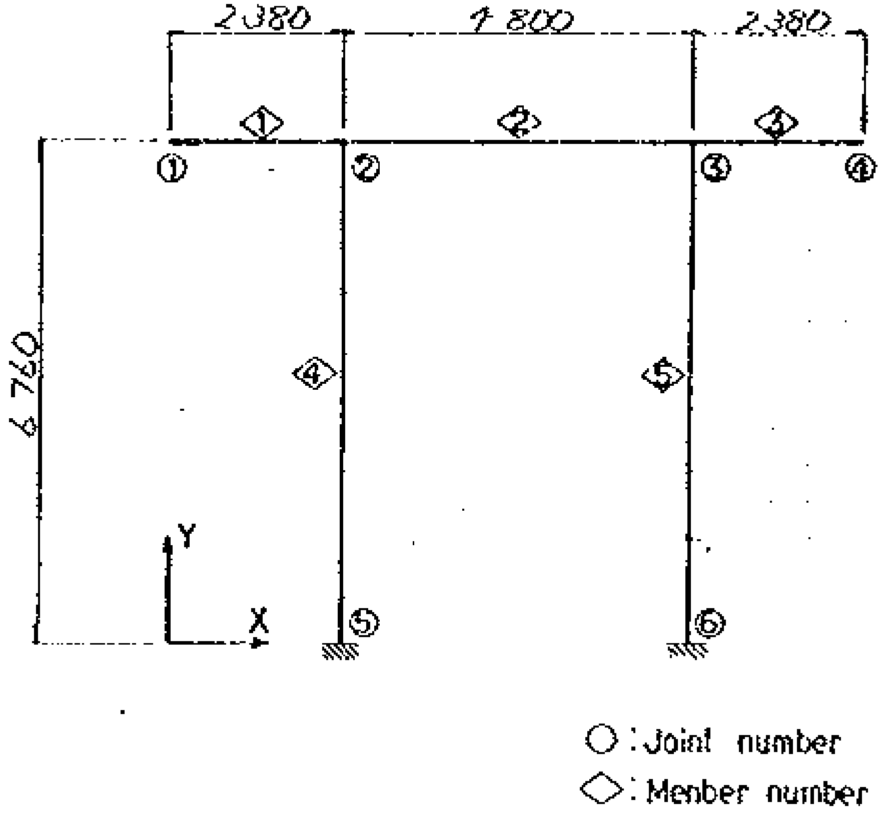
$\sigma_{sa} = 2\ 340\ kg/cm^2$

modular ratio $n = 15$

GENERAL DIMENSION



1) Skeleton



Coordinates

(m)

	X	Y		X	Y
①	0,0	6.76	④	9.56	6.76
②	2.38	6.76	⑤	2.38	0.0
③	7.18	6.76	⑥	7.18	0.0

2) Modulus of elasticity of concrete

$Ec = 26\ kN/mm^2$

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

member	section B x H	area of section A (m²)	moment of inertia I (m⁴)
◇1	1.25 x 1.25	1.5625	0.2035
◇2	1.25 x 1.50	1.875	0.3516
◇3	1.25 x 1.25	1.5625	0.2035
◇4	1.025 x 1.20	1.23	0.1476
◇5	1.025 x 1.20	1.23	0.1476

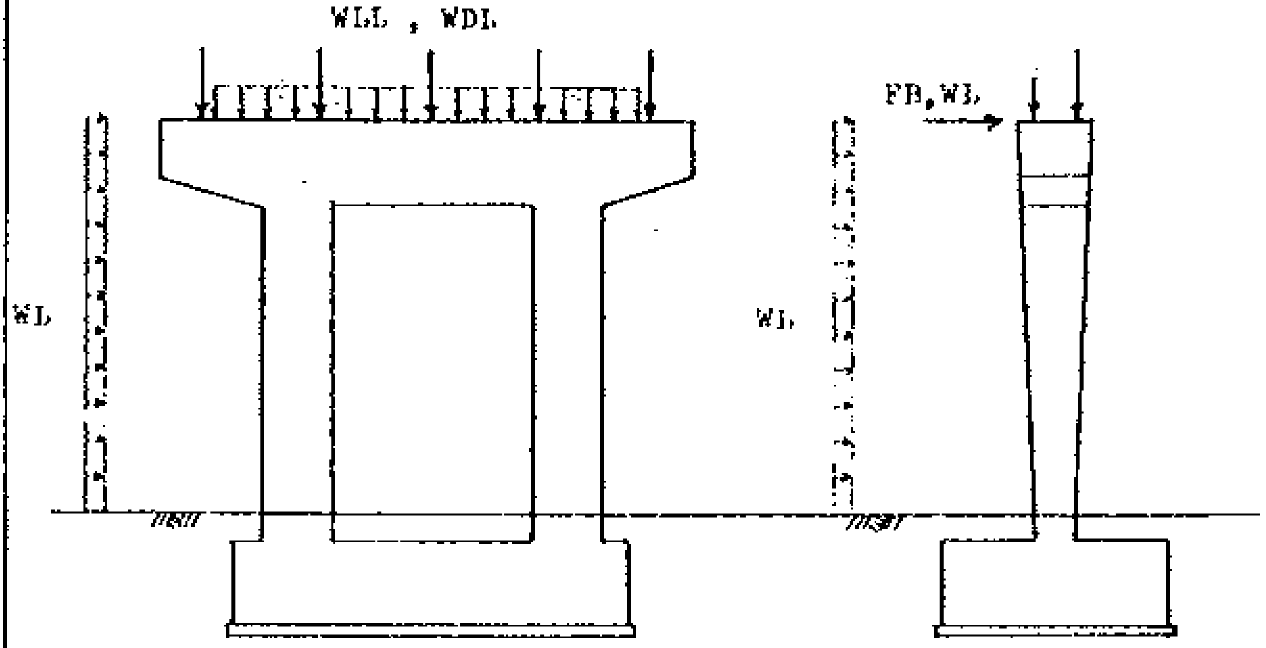
§ 3 CALCULATION OF LOADS

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

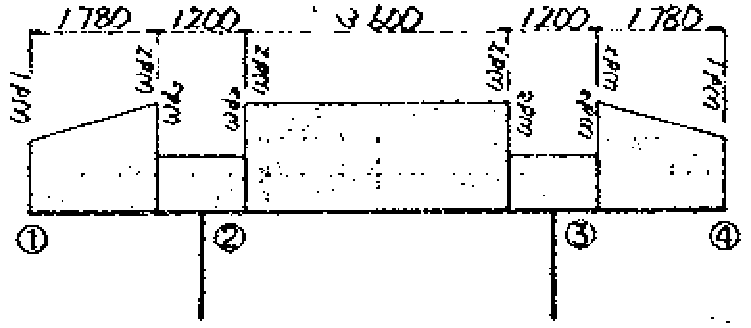
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3) Dead load

(a) beam



$Wd_1 = 1.25 \times 1.0 \times 2.407 = 3.01$ t/m

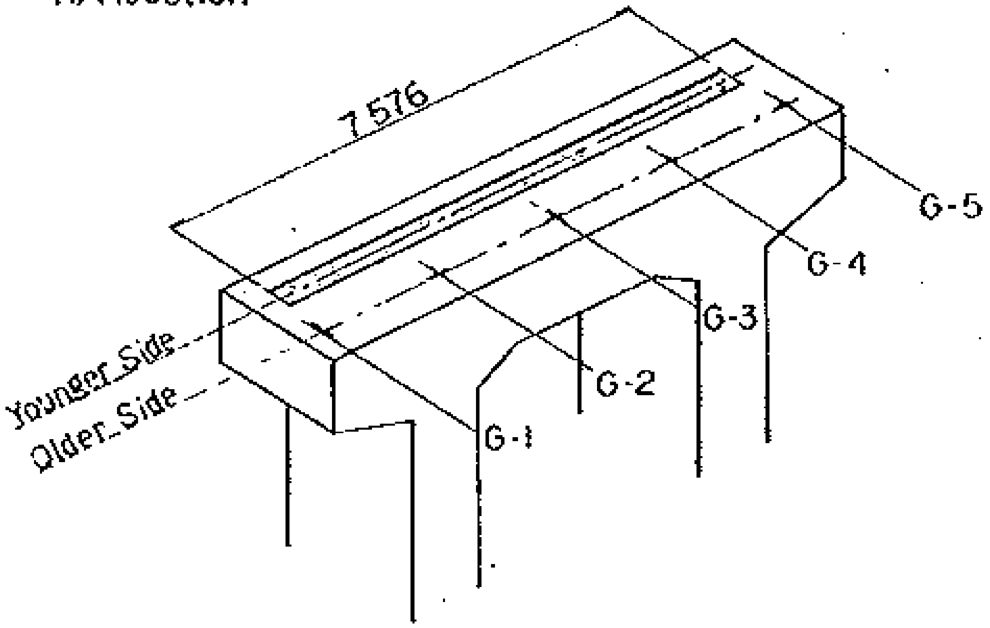
$Wd_2 = 1.25 \times 1.5 \times 2.407 = 4.51$ t/m

$Wd_3 = 1.25 \times 0.75 \times 2.407 = 2.26$ t/m

(b) Column

$P_w = 1.025 \times 1.2 \times 2.407 = 2.96$ t/m

4.) Reaction



(ton)

		G-1	G-2	G-3	G-4	G-5
Dead load	Younger	39.8/7.576 = 5.25 t/m				
	Older	32.8	30.6	31.2	30.4	29.4
HA load uniform	Younger	23.1/7.576 = 3.05 t/m				
	Older	8.6	14.1	14.7	16.2	14.7
HA load	line	2.5	7.1	6.7	7.4	4.9
Crowd load	Younger	2.1/7.576 = 0.28 t/m				
	Older	6.0	1.4	1.2		
H B load	(H B)	27.1	45.7	40.5	52.4	50.4
	H B	79.2	49.3	43.7	56.5	54.4
$\frac{1}{3}$ HA uniform	Younger	3.8/7.576 = 0.50 t/m				
	Older		1.3	1.9	1.2	0.5
$\frac{1}{3}$ HA	line		0.4	0.6	0.4	0.2
Total (H A)		(25.2/7.576 = 3.33 t/m) 65/7.576 = 8.58 t/m				
		(17.1) 49.9	(22.6) 53.2	(22.6) 53.8	(23.6) 54.0	(19.6) 49.0
Total (H B)		(5.9/7.576 = 0.78 t/m) 45.7/7.576 = 6.03 t/m				
		(35.2) 68.0	(52.4) 89.0	(47.4) 78.6	(58.1) 88.6	(55.1) 84.5

5) Temperture change and Drying schrinkage

temperture change $-12 \sim +20$ deg.

drying schrinkage -20 deg.

6) Wind Load (transverse)

(a) For Superstructure

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3 \text{ m/sec} > 35 \text{ m/sec}$

unloaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9 \text{ kg/m}^2$

$A = 1.785 \times 13.325 = 23.8 \text{ m}^2$

$C_D = 1.3$

$\therefore P_t = 219.9 \times 23.8 \times 1.3 = 6804 \text{ kg}$

$\fallingdotseq 6.8 \text{ ton}$

live loaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 35^2 \times 0.102 = 76.6 \text{ kg/m}^2$

$A = 4.07 \times 13.325 = 54.2 \text{ m}^2$

$C_D = 1.45$

$\therefore P_t = 76.6 \times 54.2 \times 1.45 = 6024 \text{ kg}$

$\fallingdotseq 6.0 \text{ ton}$

(b) For safety fences

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3\ {}^{m}/sec$

$P_t = q \cdot A \cdot C_D$

$q = 219.9\ {}^{kg}/m^2$

$A = (0.150^{m} \times 13.3^{m} + 0.850^{m} \times 0.05^{m} \times \frac{13.3}{2}) \times 2 = 4.56\ m^2$

$C_D = 1.1$

$\therefore P_t = 219.9 \times 4.56 \times 1.1 = 1103^{kg}$

$= 1.1^{ton}$

(c) For pier

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3\ {}^{m}/sec > 35\ {}^{m}/sec$

Unloaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9\ {}^{kg}/m^2$

$A_1 = (1.30 + 0.83) \times 7.01 \times 1/2 = 7.5\ m^2$

$A_2 = (1.20 + 0.83) \times 5.51 \times 1/2 = 5.6\ m^2$

$C_D = 1.7$

$\therefore P_t = 219.9 \times (7.5 + 5.6) \times 1.7 = 4\,897^{kg}$

$= 4.9^{ton}$

Live loaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ {}^{kg}/m^2$

$A = A_1 + A_2 = 13.1\ m^2$

$C_D = 1.7$

$\therefore P_t = 76.6 \times 13.1 \times 1.7 = 1680^{kg} = 1.7^{ton}$

(d) Table of Wind load (transverse)

(ton)

	unloaded	live loaded
super-structure	6.8	6.0
safety fences	1.1	—
pier	4.9	1.7

7). Longitudinal forth

(a) due to braking

HA ---- 25.8 ton

HB ---- 38.8 ton

(b) due to Wind

-for Superstructur

unloaded $P_{LS} = 0.25\,Pt = 0.25 \times 6.8$

$= 1.7$ ton

live loaded $P_{LL} = 0.5\,Pt.$

$Pt = 76.6 \times (7.5 \times 13.325) \times 1.45$

$= 3.7$ ton

$\therefore P_{LL} = 0.5 \times 3.7 = 1.85$ ton

$P_{LS} = 0.25\,Pt.$

$Pt = 76.6 \times (4.07 - 2.5) \times 13.325 \times 1.3$

$\fallingdotseq 2.1$ ton

$\therefore P_{LL} = 0.25 \times 2.1 = 0.5$ ton

for safety fences

vertical member

$P_L = 0.8\, P_t$

$P_t = 219.9 \times (0.85 \times 0.05 \times \frac{13.325}{2} \times 2) \times 1.1$

$= 137^{kg} = 0.137^{ton}$

$P_L = 0.8 \times 0.137^{ton} = 0.1^{ton}$

Longitudinal member

$P_L = 0.4\, P_t$

$P_t = 219.9 \times (0.15 \times 13.325 \times 2) \times 1.1$

$= 967^{kg} = 1.0^{ton}$

$P_l = 0.4 \times 1.0 = 0.4^{ton}$

for pier

$V_c = 59.3\ m/sec$

unloaded

$q = 219.9\ kg/m^2$

$A1 = (6.0 + 9.56) \times 1.5 \times 1/2 = 11.7\ m^2$

$A2 = 1.2 \times 5.51 \times 2 = 13.2\ m^2$

$C_{D1} = 2.1$

$C_{D2} = 2.0$

$P_{t1} = 219.9 \times 11.7 \times 2.1 = 5.4^{ton}$

$P_{t2} = 219.9 \times 13.2 \times 2.0 = 5.8^{ton}$

live loaded

$q = 76.6\ ^{kg/m^2}$

$A_1 = 11.7\ m^2$

$A_2 = 13.2\ m^2$

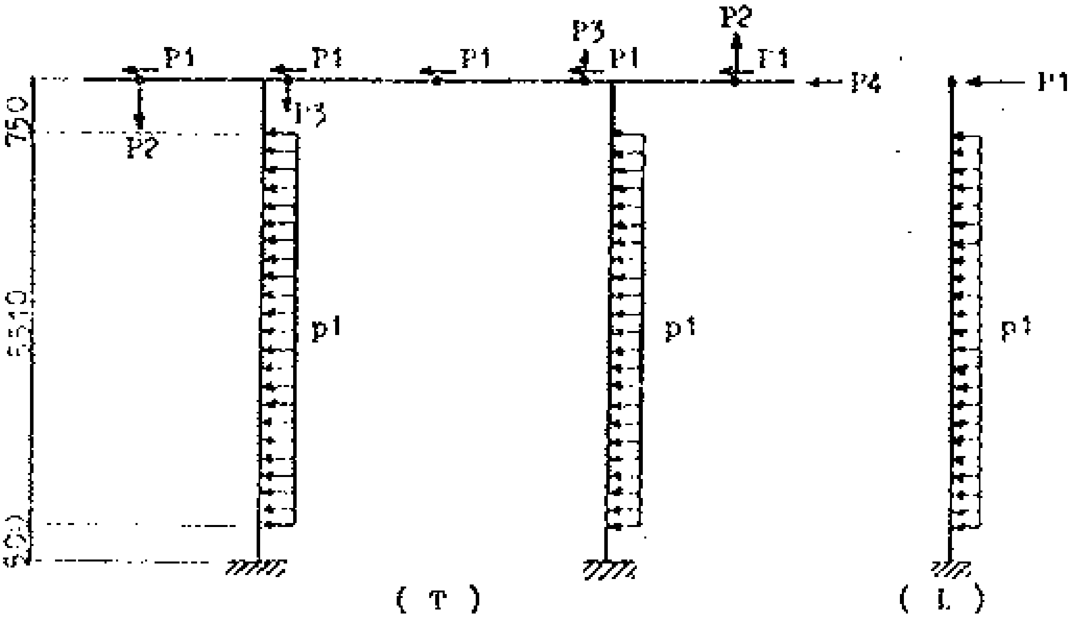
$C_{D1} = 2.1$

$C_{D2} = 2.0$

$P_{t1} = 76.6 \times 11.7 \times 2.1 = 1.9\ ^{ton}$

$P_{t2} = 76.6 \times 13.2 \times 2.0 = 2.0\ ^{ton}$

8) Wind loading diagram

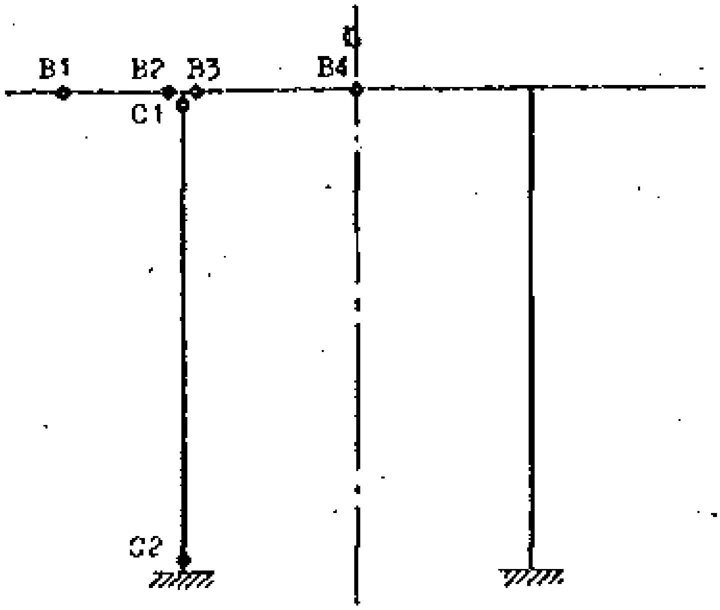


		For place	unloaded	live loaded
(T)	P1	superstructure safety fences	1.58 ton	1.20 ton
	P2	superstructure safety fences	1.39 ton	1.68 ton
	P3	superstructure safety fences	0.70 ton	0.84 ton
	P4	pier	0.71 ton	0.247 ton
	p1	pier	0.38 t/m	0.132 t/m
(L)	P1	superstructure safety fences	9.80 ton	2.18 ton
	p1	pier	0.326 t/m	0.181 t/m

p- 1

§ 4 ACTING FORCE TABLE

1.) due to transeverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.864	0.000	-52.333	1.00
	N max.	+	2	-0.864	1.580	-36.623	1.0[illegible]
		-	2	-0.864	-0.710	2.433	1.00
	S max.	+	3	-0.864	0.000	2.433	1.00
		-	4	-0.691	0.000	-56.346	1.25
B2	M max.	+					
		-	4	-123.970	0.000	80.735	1.25
	N max.	+	2	-71.351	1.580	-49.526	1.00
		-	2	-61.173	-2.290	43.346	1.00
	S max.	+	4	-123.970	0.000	80.735	1.25
		-	3	-100.433	0.000	-69.858	1.[illegible]

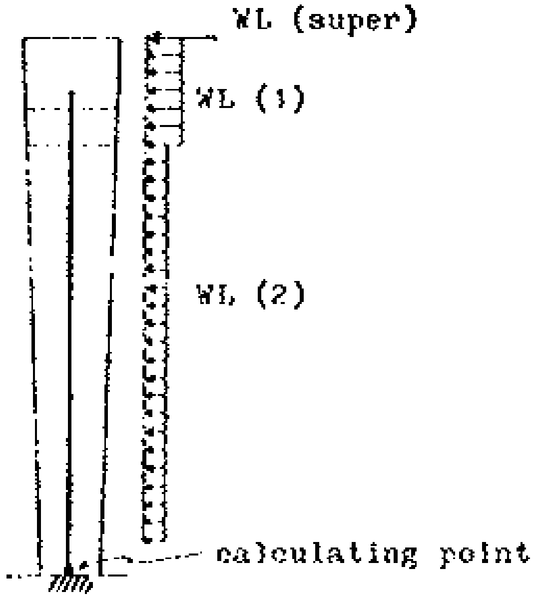
R 1

B3	M max.	+					
		-	4	-130.876	0.000	80.735	1.25
	N max.	+	6	-58.694	14.946	-51.609	1.30
		-	6	-85.898	-8.235	111.378	1.30
	S max.	+	4	-76.597	7.946	127.071	1.25
		-	4	-92.531	9.027	-133.991	1.25
B4	M max.	+	4	86.745	6.009	-30.226	1.25
		-	4	-38.644	-1.166	-16.596	1.25
	N max.	+	3	25.795	14.316	-15.337	1.00
		-	3	33.847	-7.040	-25.776	1.00
	S max.	+	6	27.335	10.482	27.335	1.30
		-	4	58.019	9.691	-47.930	1.25
C1	M max.	+	6	58.380	-127.946	-16.059	1.30
		-	4	-50.478	-118.038	14.202	1.25
	N max.	+					
		-	4	31.440	-214.726	-9.027	1.25
	S max.	+	3	-48.932	-139.395	14.316	1.00
		-	6	58.380	-127.946	-16.059	1.30
C2	M max.	+	3	47.845	-158.404	14.316	1.00
		-	4	-50.478	-118.038	14.202	1.25
	N max.	+					
		-	4	-29.580	-230.733	-9.027	1.25
	S max.	+	3	47.845	-158.404	14.316	1.00
		-	2	-62.663	-122.488	-17.961	1.30

These value is divided by "i".

i ; coefficient of increase
in basic stresses.

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	48.4	194.6	6.45	1.00
case-8	71.9	257.6	9.58	1.25
case-9	66.7	194.6	4.58	1.15
case-10	90.2	257.6	12.71	1.30
case-11	95.0	191.9	6.70	1.00

CASE 7

	N	H	y	M
WDL	97.1			
WLL (A)	60.0			
FB (A)		6.45	7.51	48.4
W	37.5			
TOTAL	194.6	6.45		48.4

M = 48.4 t·m

N = 194.6 ton

H = 6.45 ton

CASE 8

	N	H	y	M
WDL	97.1			
WLL (B)	123.0			
FB (B)		9.58	7.51	71.9
W	37.5			
TOTAL	257.6	9.58		71.9

M = 71.9 t·m

N = 257.6 ton

H = 9.58 ton

CASE 9

	N	H	y	M
WDL	97.1			
WLL, FB(A)	60.0	6.45	7.51	48.4
W	37.5			
WL (super)		1.18	7.51	8.9
WL (1)		0.95	6.76	6.4
WL (2)		1.00	3.01	3.0
TOTAL	194.6	9.58		66.7

M = 66.7 t·m

N = 194.6 ton

H = 9.58 ton

CASE 10

	N	H	y	M
WDL	97.1			
WLL, FB(A)	123.0	9.58	7.51	71.9
W	37.5			
WL (super)		1.18	7.51	8.9
WL (1)		0.95	6.76	6.4
WL (2)		1.00	3.01	3.0
TOTAL	257.6	12.71		90.2

M = 90.2 t·m

N = 257.6 ton

H = 12.71 ton

CASE 11

	N	H	y	M
WDL	154.4			
W	37.5			
WL (Super)		1.10	7.51	8.3
WL (1)		2.7	6.76	18.3
WL (2)		2.9	3.01	8.7
TOTAL	191.9	6.7		35.3

M = 35.3 t·m

N = 191.9 ton

H = 6.7 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

P1

§ 5 CALCULATION OF SECTION

1) Beam section

	at B1 case 4	at B2 case 4	at B3 case 4
M (tm)	-0.864	-154.963	-163.595
N (ton)	0.000	0.000	0.000
Q: shear force (ton)	-70.493	100.919	-167.489
b (cm)	120	120	120
d (cm)	110.6	140	140
d' (cm)	10	10	10
As (cm^2)	25.1 (D20 15cm pitch)	49.1 (10-D25 17.5cm pitch)	49.1 (10-D25 12.5cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.026	0.042	0.042
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)	0.059	6.59	6.96
Q/b·d (kg/cm^2)	5.31	6.01	9.97
C	10.5	8.71	8.71
S	41.3	26.2	26.2
	1.07	1.09	1.09
σ_c (kg/cm^2)	0.6	57	61
σ_s (kg/cm^2)	37	2590	2735
τ (kg/cm^2)	5.68	6.55	10.87 *
σ_{ca} (kg/cm^2)	106	106	106
σ_{sa} (kg/cm^2)	2925	2925	2925
τ_a (kg/cm^2)	10.3	10.3	10.3

stirrups Req Av = $\frac{\tau \cdot b \cdot a}{\sigma \cdot a} = \frac{10.87 \times 120 \times 12.5}{1980 \times 1.75} = 4.32$ cm^2

a: spacing of stirrups

Used Av = ϕ16 [] × 2 = 4 × 2.01 = 8.04 cm^2 > Req Av O.K.

P1

	st. B4 case 1	case	case
M (tm)	108.431		
N (ton)	7.511		
shear force (ton)	-37.783		
b (cm)	120		
D (cm)	140		
d' (cm)	10		
As (cm^2)	39.3 (8-D25 15cm pitch)		
d'/d			
e=(M/N)+ (cm)			
e/d			
n	15		
np=n·As/b·d	0.035		
M'=M+N·a (tm)			
M'/(b·d²) (kg/cm^2)	4.61		
S/b·j (kg/cm^2)	2.25		
C	4.35		
S	31.0		
[illegible]	1.08		
σ_c (kg/cm^2)	43		
σ_s (kg/cm^2)	2144		
τ (kg/cm^2)	2.42		
σ_{ca} (kg/cm^2)	106		
σ_{sa} (kg/cm^2)	2925		
τ_a (kg/cm^2)	10.3		

2) Column section

	case 6	case 10	case
M (tm)	75.894	90.2	
N (ton)	-166.330	-237.6	
Q;shear force (ton)	-20.872	12.71	
b (cm)	80	120	
d (cm)	110	70	
d' (cm)	10	10	
As (cm^2)	39.3 8-D25	39.3 8-D25	
d'/d	0.091	0.14	
f=(M/N)+u (cm)	95.6	65.0	
f/d	0.87	0.93	
n	15	15	15
nF=n·As/b·d	0.067	0.070	
M'=M+N·u (tm)	159.059	167.48	
M'/(b·d^2) (t/m^2)	16.43	28.48	
Q/b·d (kg/cm^2)	2.37	1.32	
C	3.2	3.25	
S	1.7	1.6	
σ_c (kg/cm^2)	53	93	
σ_s (kg/cm^2)	919	684	
τ (kg/cm^2)	2.4	1.3	
σ_{ca} (kg/cm^2)	111	111	
σ_{sa} (kg/cm^2)	3042	3042	
τ_a (kg/cm^2)	10.7	10.7	

PIER IN PAILLES O. V.

(P - 2)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight $H = 8.71^{m}$

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

$2.407\ t/m^3$

4 Allowable stresses of reinforced concrete

1.) concrete (grade 25 , BS 5400)

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

bending stress $\sigma ca = 85$

direct stress $\sigma ca = 64.3$

shear stress $\tau a = 8.2$

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

specified characteristic strength

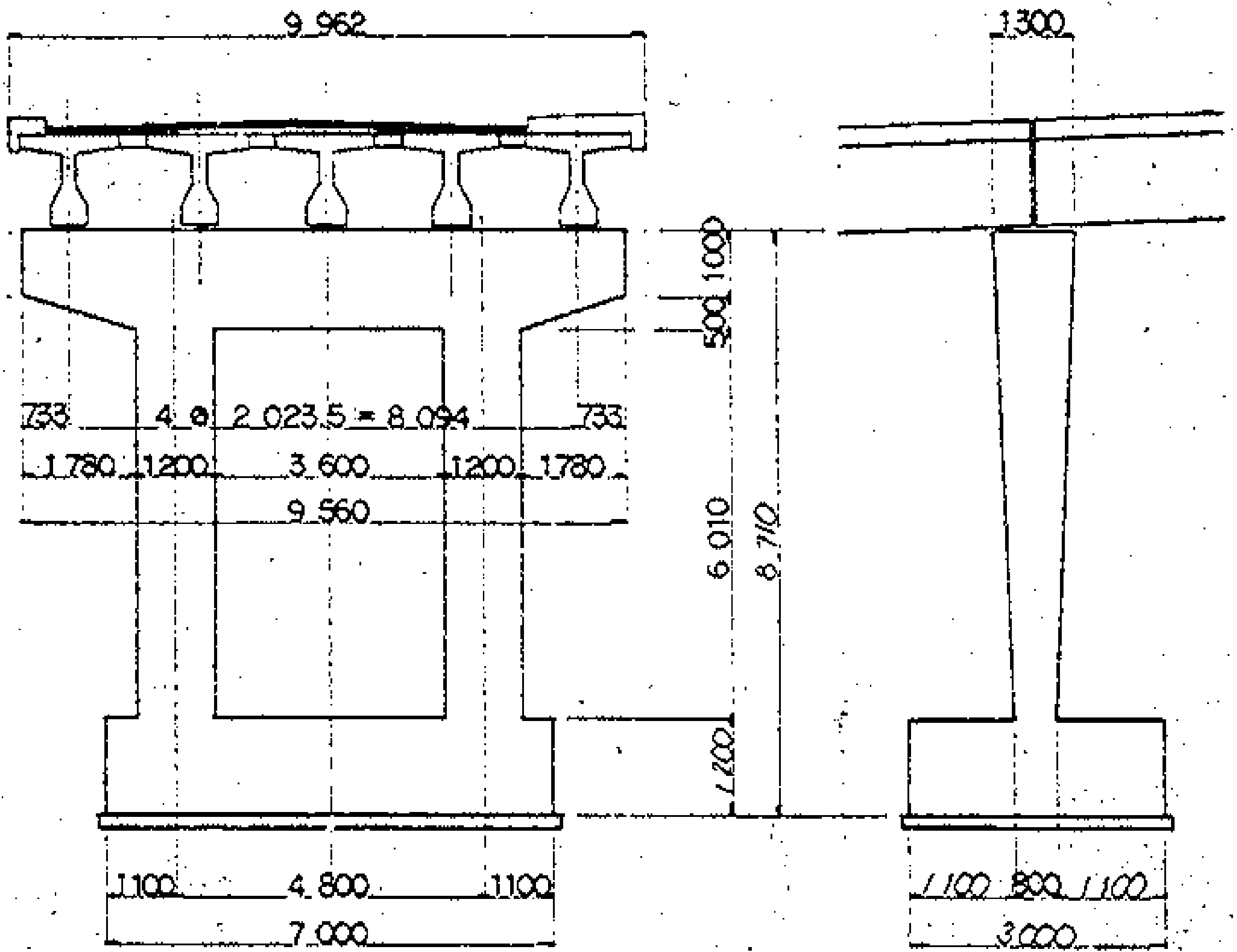
$\sigma su = 4\ 180\ kg/cm^2$

permissible tensile stress

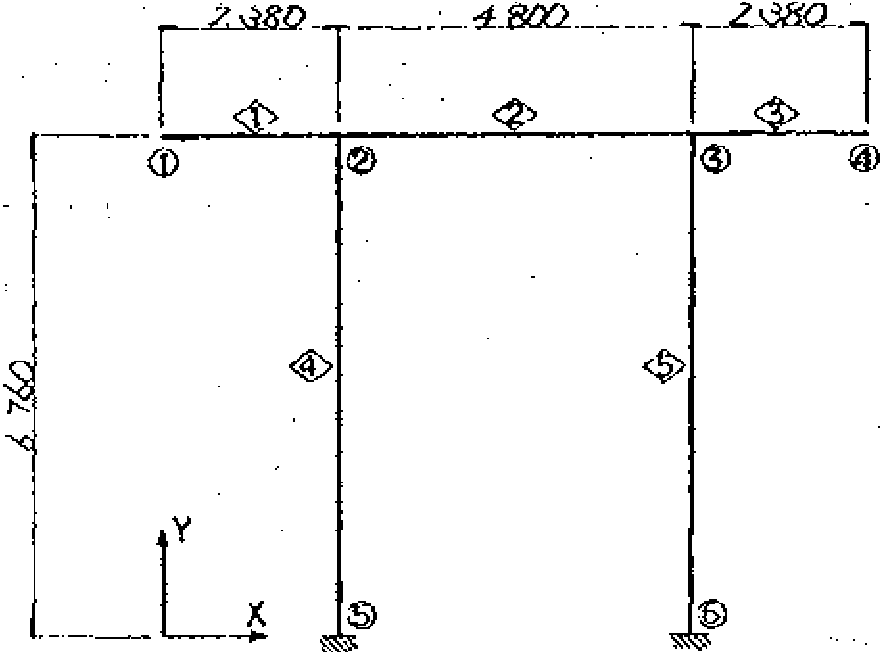
$\sigma sa = 2\ 340\ kg/cm^2$

modular ratio $n = 15$

§ 2 GENERAL DIMENSION



1) Skeleton



○ : Joint number
◇ : Menber number

Coordinates

(m)

	X	Y		X	Y
①	0.0	6.76	④	9.56	6.76
②	2.38	6.76	⑤	2.38	0.0
③	7.18	6.76	⑥	7.18	0.0

2) 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 (m^2)	moment of inertia I (m^4)
1	1.25 x 1.25	1.5625	0.2035
2	1.25 x 1.50	1.875	0.3516
3	1.25 x 1.25	1.5625	0.2035
4	1.025 x 1.20	1.23	0.1476
5	1.025 x 1.20	1.23	0.1476

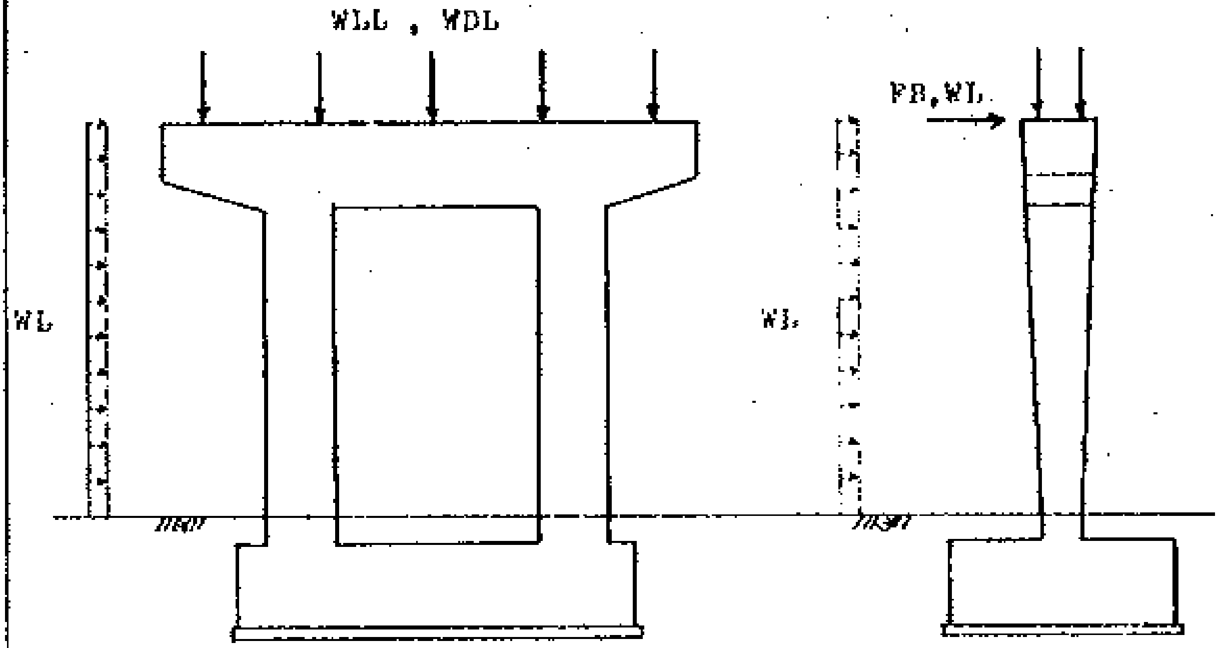
§ 3 CALCUIATION OF LOADS

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	temperture change and drying schrinkage	FT & FS
8	wind	WL

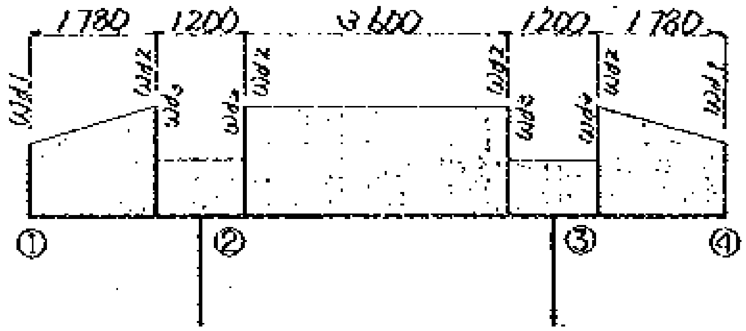
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3) Dead load

(a) beam



$W_{d1} = 1.25 \times 1.0 \times 2.407 = 3.01\ t/m$

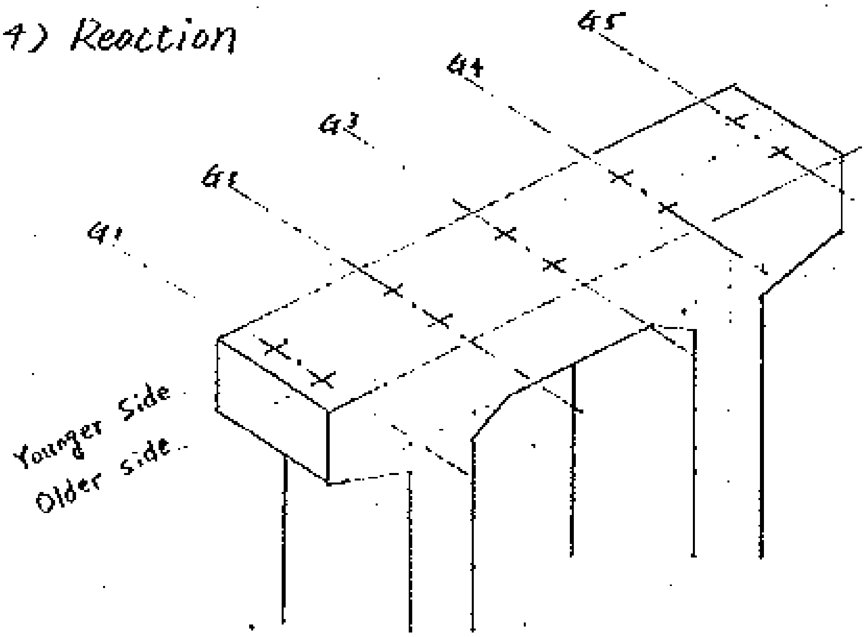
$W_{d2} = 1.25 \times 1.5 \times 2.407 = 4.51\ t/m$

$W_{d3} = 1.25 \times 0.75 \times 2.407 = 2.26\ t/m$

(b) column

$P_w = 1.025 \times 1.2 \times 2.407 = 2.96\ t/m$

4) Reaction

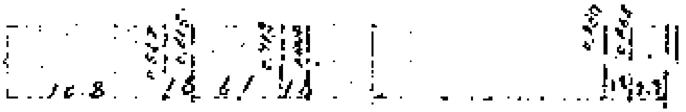


Tablation of Reaction

Unit : ton

			G-1	G-2	G-3	G-4	G-5
Dead load		Younger	32.8	30.6	31.2	30.4	29.4
		Older	32.8	30.6	31.2	30.4	29.4
HA load	Uniform	Younger	8.6	19.1	19.7	16.2	14.7
		Older	8.6	19.1	19.7	16.2	14.7
	line		2.5	7.1	6.7	7.4	4.9
Crowd load		Younger	6.0	1.9	1.2		
		Older	6.0	1.9	1.2		
HB load (1/3 HA)	(HB)		27.1	45.7	40.5	52.4	50.4
	HB		29.2	49.3	43.7	56.5	54.4
	Uniform	Younger		1.3	1.9	1.2	0.5
		Older		1.3	1.9	1.2	0.5
	line			0.9	0.6	0.4	0.2
Total (HA)			(31.7) 97.3	(38.1) 99.3	(38.5) 100.9	(39.8) 100.6	(39.3) 93.1
Total (HB)			(41.2) 106.8	(55.1) 116.3	(50.5) 112.3	(59.3) 120.1	(55.6) 114.9

α = 1.079



5) Temperture change and Drying schrinkage

temperture change $-12 \sim +20$ deg.

drying schrinkage -20 deg.

6) Wind Lood (transverse)

(a) For Superstructure

$V_c = 38 \times 1.0 \times 1.0 \times 1.53 = 58.1\ \mathrm{m/sec} > 35\ \mathrm{m/sec}.$

unloaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 58.1^2 \times 0.102 = 211.1\ \mathrm{kg/m^2}$

$A = 1.785 \times 21.0 = 37.5\ \mathrm{m^2}$

$C_D = 1.3$

$P_t = 211.1 \times 37.5 \times 1.3 = 10291\ \mathrm{kg}$

$= 10.3\ \mathrm{ton}$

live loaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ \mathrm{kg/m^2}$

$A = 4.07 \times 21.0 = 85.5\ \mathrm{m^2}$

$C_D = 1.45$

$\therefore P_t = 76.6 \times 85.5 \times 1.45 = 9496\ \mathrm{kg}$

$= 9.5\ \mathrm{ton}$

(b) For safety fences

$V_c = 38 \times 1.0 \times 1.0 \times 1.53 = 58.1\ ^{m}/_{sec}$

$P_t = q \cdot A \cdot C_D$

$q = 211.1\ ^{kg}/_{m^2}$

$A = (0.150^m \times 21^m + 0.850^m \times 0.05^m \times \frac{21}{2}) \times 2 = 7.19\ m^2$

$C_D = 1.1$

$\therefore P_t = 211.1 \times 7.19 \times 1.1 = 1670^{kg}$

$= 1.7^{ton}$

(c) For pier.

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3\ ^{m}/_{sec} > 35\ ^{m}/_{sec}$

Unloaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9\ ^{kg}/_{m^2}$

$A_1 = (1.30 + 0.83) \times 7.01 \times 1/2 = 7.5\ m^2$

$A_2 = (1.20 + 0.83) \times 5.51 \times 1/2 = 5.6\ m^2$

$C_D = 1.7$

$\therefore P_t = 219.9 \times (7.5 + 5.6) \times 1.7 = 4\,897^{kg}$

$= 4.9^{ton}$

live loaded

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ ^{kg}/_{m^2}$

$A = A_1 + A_2 = 13.1\ m^2$

$C_D = 1.7$

$\therefore P_t = 76.6 \times 13.1 \times 1.7 = 1680^{kg} \fallingdotseq 1.7^{ton}$

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	10.3	9.5
safety fences	1.7	—
pier	4.9	1.7

7). Longitudinal forth

(a) due to braking.

HA --- 25.8 ton

HB --- 08.3 ton

(b) due to Wind

for Superstructur

unloaded $P_{LS} = 0.25\,P_t = 0.25 \times 10.3$

$= 2.6^{ton}$

live loaded $P_{LL} = 0.5\,P_t$

$P_t = 76.6 \times (2.5 \times 21) \times 1.45$

$= 5.8^{ton}$

$\therefore P_{LL} = 0.5 \times 5.8 = 2.9^{ton}$

$P_{LS} = 0.25\,P_t$

$P_t = 76.6 \times (4.07 - 2.5) \times 21 \times 1.3$

$= 3.3^{ton}$

$\therefore P_{LL} = 0.25 \times 3.3 = 0.8^{ton}$

for safety fences

vertical member

$P_L = 0.8\,P_t$

$P_t = 211.1 \times (0.85 \times 0.05 \times \frac{21}{2} \times 2) \times 1.1$

$= 207^{kg} = 0.2^{ton}$

$P_L = 0.8 \times 0.2 = 0.16^{ton}$

Longitudinal member

$P_L = 0.4\,P_t$

$P_t = 211.1 \times (0.15 \times 21 \times 2) \times 1.1$

$= 1463^{kg} = 1.5^{ton}$

$P_L = 0.4 \times 1.5 = 0.6^{ton}$

for pier

$V_c = 39.3\ ^{m}/sec.$

unloaded

$q = 219.9\ ^{kg}/m^2$

$A_1 = (6.0 + 9.56) \times 1.5 \times 1/2 = 11.7\ m^2$

$A_2 = 1.2 \times 5.51 \times 2 = 13.2\ m^2$

$C_{D1} = 2.1$

$C_{D2} = 2.0$

$P_{t1} = 219.9 \times 11.7 \times 2.1 = 5.4^{ton}$

$P_{t2} = 219.9 \times 13.2 \times 2.0 = 5.8^{ton}$

live loaded

$q = 76.6^{\mathrm{kg/m^2}}$

$A_1 = 11.7\ \mathrm{m^2}$

$A_2 = 13.2\ \mathrm{m^2}$

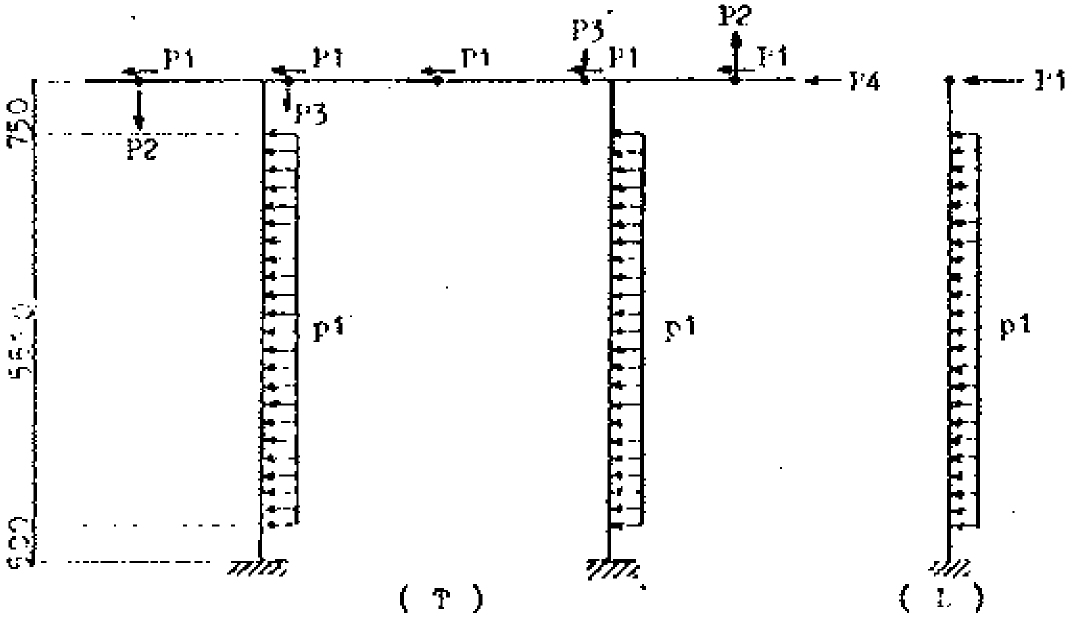
$C_{D1} = 2.1$

$C_{D2} = 2.0$

$P_{t1} = 76.6 \times 11.7 \times 2.1 = 1.9^{\mathrm{ton}}$

$P_{t2} = 76.6 \times 13.2 \times 2.0 = 2.0^{\mathrm{ton}}$

8) Wind loading diagram

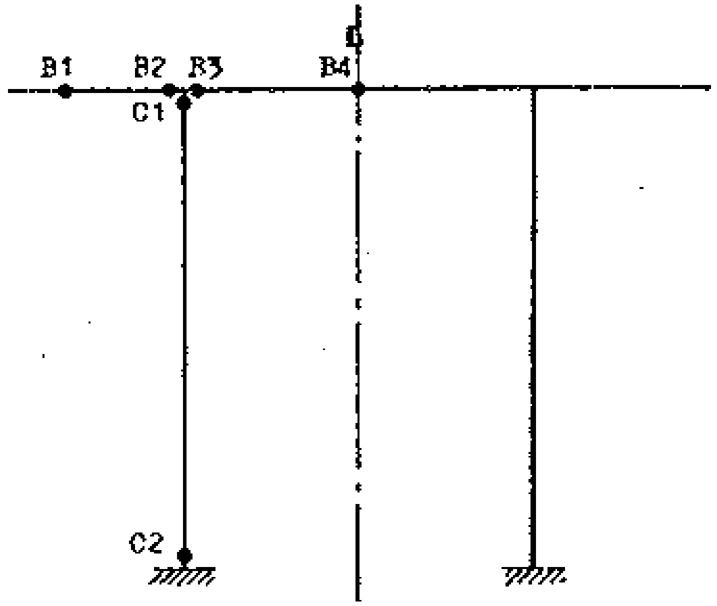


		For place	unloaded	live loaded
(T)	P1	superstructure safety fences	2.40 ton	1.90 ton
	P2	superstructure safety fences	2.12 ton	2.61 ton
	P3	superstructure safety fences	1.06 ton	1.31 ton
	P4	pier	0.71 ton	0.247 ton
	p1	pier	0.38 t/m	0.132 t/m
(L)	P1	superstructure safety fences	4.40 ton	2.90 ton
	p1	pier	0.526 t/m	0.181 t/m

§ 4 ACTING FORCE TABLE

1.) due to transverse force

P2



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.864	-0.000	-99.733	1.00
	N max.	+	2	-0.864	2.400	-70.153	1.00
		-	2	-0.693	-0.710	1.996	1.00
	S max.	+	2	-0.690	-0.710	1.996	1.00
		-	3	-0.864	-0.000	-99.733	1.00
B2	M max.	+					
		-	3	-170.236	-0.000	-105.349	1.00
	N max.	+	2	-121.518	2.400	-75.769	1.00
		-	2	-102.142	-3.110	64.061	1.00
	S max.	+	3	-162.126	0.000	100.481	1.00
		-	3	-0.864	-0.000	-99.733	1.00

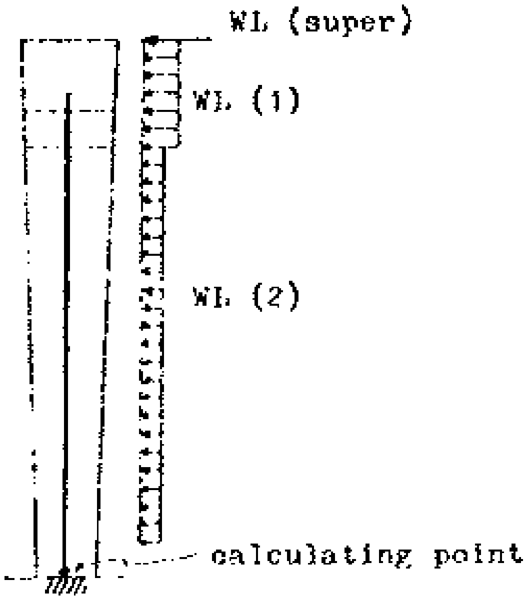
P2

B3	M max.	+					
		-	3	-172.829	-3.428	171.524	1.00
	N max.	+	3	-103.396	18.213	103.371	1.00
		-	3	-121.536	-5.581	-158.076	1.00
	S max.	+	3	-124.900	12.866	171.524	1.00
		-	3	-116.923	13.042	-168.736	1.00
B4	M max.	+	3	92.058	10.713	-48.002	1.00
		-	4	-42.519	3.208	-26.696	1.25
	N max.	+	3	10.703	18.213	-29.703	1.00
		-	3	44.128	-5.581	-48.002	1.00
	S max.	+					
		-	3	69.404	13.611	-61.998	1.00
C1	M max.	+	5	71.020	-163.871	-20.076	1.15
		-	3	-66.840	-208.720	18.213	1.00
	N max.	+	3				
		-	3	-45.336	-276.873	12.866	1.00
	S max.	+	3	-66.840	-208.720	18.213	1.00
		-	2	67.229	-151.886	-20.089	1.00
C2	M max.	+	3	56.280	-228.730	18.213	1.00
		-	2	-75.297	-171.896	-22.144	1.00
	N max.	+					
		-	3	41.635	-296.882	12.866	1.00
	S max.	+	3	56.280	-228.730	18.213	1.00
		-	2	-75.297	-171.896	-22.144	1.00

These value is divided by "i".

i ; coefficient of increase
in basic stresses.

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	48.4	238.1	6.5	1.00
case-8	71.9	322.8	9.6	1.25
case-9	72.5	283.1	10.4	1.15
case-10	96.0	322.8	13.5	1.30
case-11	39.8	191.9	7.9	[illegible]

CASE 7

	N	H	y	M
WDL	154.4			
WLL (A)	91.2			
EB (A)		6.45	7.51	48.4
W	37.5			
TOTAL	283.1	6.45		48.4

M = 48.4 tm

N = 283.1 ton

H = 6.5 ton

CASE 8

	N	H	y	M
WDL	154.4			
WLL (B)	130.9			
EB (B)		9.58	7.51	71.9
W	37.5			
TOTAL	322.8	9.58		71.9

M = 71.9 tm

N = 322.8 ton

H = 9.6 ton

CASE 9

	N	H	Y	M
WDL	154.4			
WLL,FB(A)	91.2	6.45	7.51	48.4
W	37.5			
WL.(Super)		1.9	7.51	14.3
WL (1)		1.0	6.76	6.8
WL (2)		1.0	3.01	3.0
TOTAL	283.1	10.4		72.5

M = 72.5 t·m

N = 283.1 ton

H = 10.4 ton

CASE 10

	N	H	Y	M
WDL	154.4			
WLL, FB(B)	130.9	9.58	7.51	71.9
W	37.5			
WL (Super)		1.9	7.51	14.3
WL (1)		1.0	6.76	6.8
WL (2)		1.0	3.01	3.0
TOTAL	322.8	13.5		96.0

M = 96.0 t·m

N = 322.8 ton

H = 13.5 ton

CASE 11

	N	H	y	M
WDL	154.4			
N	37.5			
WL (supper)		1.7	7.51	12.8
WL (1)		2.7	6.76	18.3
WL (2)		2.9	3.01	8.7
TOTAL	191.9	7.3		39.8

M = 39.8 t·m

N = 191.9 ton

H = 7.3 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

P2.

§ 5 CALCULATION OF SECTION

1) Beam section

	at B1 case ③	at B2 case ③	at B3 case 3
M (tm)	-0.864	-170.236	-172.879
N (ton)	-0.000	-0.000	-3.428
shear force (ton)	-99.733	-105.349	171.524
b (cm)	120	120	120
d (cm)	110.6	140	140
d' (cm)	10	10	10
As (cm^2)	25.1 (D20 15cm pitch)	64.3 (8-D32 15cm pitch)	64.3 (8-D32 15cm pitch)
d'/d			
e=(M/N)+c (cm)			
e/d			
n	15	15	15
nP=n·As/b·d	0.026	0.057	0.057
M'=M+N·g (tm)			
M'/(b·d^2) (kg)	0.059	7.24	7.35
S/b·d (kg/cm^2)	7.51	6.27	10.1
C	10.5	7.75	7.75
S	41.3	19.4	19.4
	1.07	1.10	1.10
σ_c (kg/cm^2)	0.6	36	37
σ_s (kg/cm^2)	37	2107	2139
τ (kg/cm^2)	8.0	6.5	11.22
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

※ Stirrups $A_v = \frac{\tau \cdot b \cdot a}{\sigma_{sa}} = \frac{11.22 \times 110 \times 13.5}{1780} = 9.46\,cm^2$

Used A_r = 4D20 × 2 = 4 × 3.14 ≒ 12.56 cm^2

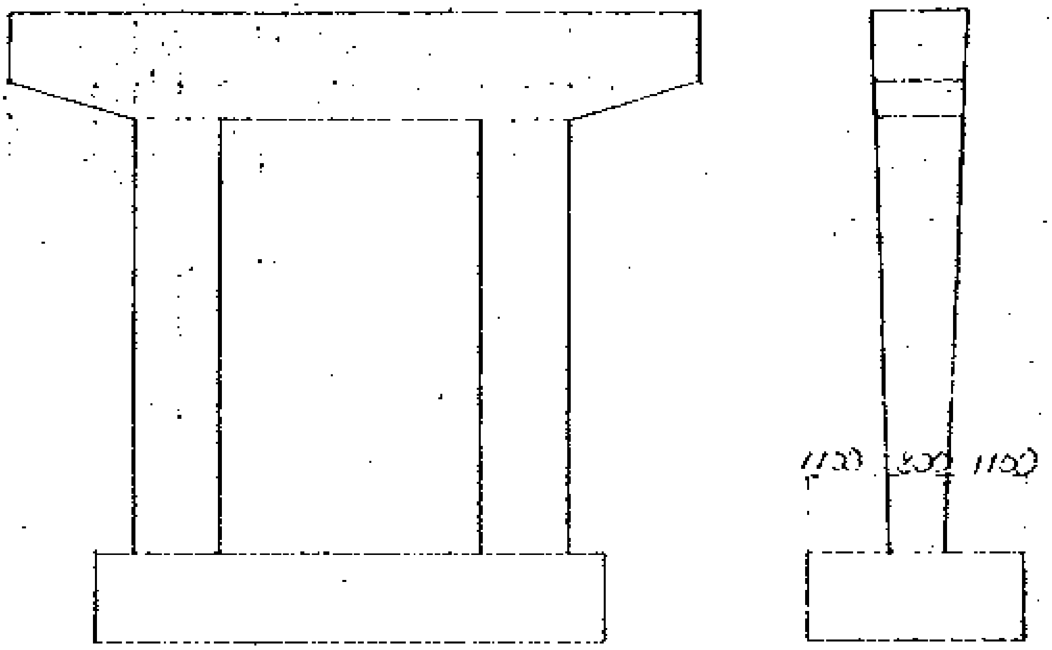
P2

	at B4 case ③	case	case
M (tm)	92.058		
N (ton)	10.713		
S:shear force (ton)	-48.002		
b (cm)	120		
d (cm)	140		
d' (cm)	10		
As (cm^2)	39.3 (8-D25 13cm pitch)		
d'/d			
e=(M/N)+u (cm)			
e/d			
n	15	15	15
np=n·As/b·d	0.035		
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)	3.91		
S/b·d (kg/cm^2)	2.85		
C	9.35		
S	31.0		
	1.08		
σ_c (kg/cm^2)	37		
σ_s (kg/cm^2)	1818		
τ (kg/cm^2)	3.1		
σ_{ca} (kg/cm^2)	85		
σ_{sa} (kg/cm^2)	2370		
τ_a (kg/cm^2)	8.2		

2) Column section

	case 2	case 10	case
M (tm)	-75.297	96.0	
N (ton)	-171.896	322.8	
Q;shear force (ton)	-22.144	13.5	
b (cm)	80	120	
d (cm)	110	70	
d' (cm)	10	10	
As (cm^2)	39.3 8-D25	39.3 8-D25	
d'/d	0.091	0.14	
f=(M/N)+u (cm)	93.8	59.7	
f/d	0.85	0.85	
n	15	15	15
nP=n·As/b·d	0.067	0.070	
M'=M+N·u (tm)	161.245	192.8	
M'/($b \cdot d^2$) (kg/cm^2)	16.66	32.79	
Q/b·d (kg/cm^2)	2.31	1.41	
C	3.14	3.25	
S	1.4	1.6	
z			
σ_c (kg/cm^2)	52	107	
σ_s (kg/cm^2)	350	787	
τ (kg/cm^2)	2.3	1.4	
σ_{ca} (kg/cm^2)	85	111	
σ_{sa} (kg/cm^2)	2340	3042	
τ_a (kg/cm^2)	8.2	10.7	

§ 6. Calculation of Stability



6-1) Calculation of Load

Self weight

1.250 × ½ × (1.0 + 1.5) × 1.78 × 2.407 × 2 = 13.37 t

1.250 × 1.5 × 6.0 × 2.457 = 27.08

1.025 × 1.2 × 6.01 × 2.407 × 2 = 35.69

76.06 t

3.0 × 7.0 × 1.2 × 2.407 = 60.66

136.72 t

Reaction due to superstructure

Dead Load	154.4 × 2 =	308.8 t	HA Total
HA Live		156.3	480.7
HB Live		152.5	HB Total
Crowd Load		15.6	476.9

6;2). Longitudinal Force

a). due to braking.

$HA = 25.8 \times 1/2 = 12.9^{ton}$

$HB = 38.3 \times 1/2 = 19.2^{ton}$

b). due to wind

for superstructure

unloaded $P_{LS} = 2.6^{ton}$

live load $P_{LL} = 2.9^{ton}$

$P_{LS} = 0.8^{ton}$

for safety fences

$P_L = 0.16^{ton}$

$P_L = 0.6^{ton}$

for pier

unloaded $P_{L-1} = 5.9^{t}$

$P_{L-2} = 11.6^{t}$

loaded $P_{L-1} = 1.9^{t}$

$P_{L-2} = 7.0^{t}$

Item	Virtical Force N (t)	distence z (m)	N z	Horizontal force H (t)	distence y (m)	H·y
Pier self weight ( ) ; Dead load	135.72 (308.80)					
HA Live	430.70					
HB Live	(303.80) 476.70					
FB (A)				12.9	8.71	112.36
FB (B)				19.2	8.71	167.23
W.L. Unloaded				2.6	8.71	22.65
				0.76	8.71	6.62
				5.4	7.96	42.18
				11.6	4.455	51.68
W.L. live load				3.7	8.71	32.23
				1.9	7.96	15.12
				4.0	4.455	17.82
				1.6		15.17
Dead load only	445.52					
Dead load + wind	445.52			26.9		123.93
HA Live	617.42	29.1 - 0.35	8.6	12.9		112.36
HB Live	613.62			19.2		167.23
HA + wind	617.42			22.5		177.63
HB + wind	613.62			28.8		232.40

Calculation of stability

Case 1. Dead Load only

$$\sigma = \frac{445.52}{7.0 \times 3.0} = 21.2 \ t/m^2$$

Case 2 Dead + wind

eccentricity $e = \frac{M}{N} = \frac{123.93}{445.52} = 0.278 < 0.5 = \frac{B}{6}$

$$\sigma = \frac{445.52}{7.0 \times 3.0} \left(1 \pm \frac{6 \times 0.278}{3.0}\right) = \begin{cases} 33.0 \\ 9.4 \end{cases} t/m^2$$

Case 3 HA Live

eccentricity $e = \frac{112.36}{617.42} = 0.182$

$$\sigma = \frac{617.42}{7.0 \times 3.0} \left(1 \pm \frac{6 \times 0.182}{3.0}\right) = \begin{cases} 40.1 \\ 18.7 \end{cases} t/m^2$$

Case 4 HB Live

$$e = \frac{167.23}{613.62} = 0.273$$

$$\sigma = \frac{613.62}{7.0 \times 3.0} \left(1 \pm \frac{6 \times 0.273}{3.0}\right) = \begin{cases} 45.2 \\ 13.3 \end{cases}$$

Case 5 HA Live + wind

$$e = \frac{177.53}{617.42} = 0.288$$

$$\sigma = \frac{617.42}{7.0 \times 3.0} \times \left(1 \pm \frac{6 \times 0.288}{3.0}\right) = \begin{cases} 46.3 \\ 12.5 \end{cases}$$

Case 6 HB Live + wind

$$e = \frac{232.40}{613.62} = 0.379$$

$$\sigma = \frac{613.62}{7.0 \times 3.0} \times \left(1 \pm \frac{6 \times 0.379}{3.0}\right) = \begin{cases} 51.4 \\ 7.1 \end{cases} t/m^2$$

6-3) Transverse Force

due to wind

for superstructure

unloaded $P_t = 10.3^{t}$

live loaded $P_t = 9.5^{t}$

for safety fence

$P_t = 1.7^{t}$

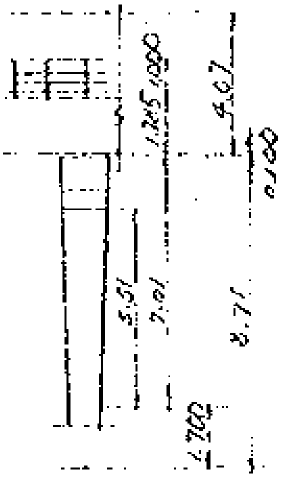
for pier

unloaded $P_{t-1} = 2.8^{t}$

$P_{t-2} = 2.1^{t}$

live loaded $P_{t-1} = 1.0^{t}$

$P_{t-2} = 0.7^{t}$



Acting Force due to wind load at center of bottom

a) Unloaded

	H (t)	y (m)	H·y
for superstructure	10.3	9.702^{5}	99.9
for safety fence	1.7	11.075	18.9
for pier	2.8	5.205	14.6
	2.1	4.455	9.4
	16.9^{t}		142.8^{t}

b). with live load

	H (t)	y (m)	H·y
for superstructure	9.5	10.845	103.0
for pier	1.0	5.205	5.2
	0.7	4.455	3.1
	11.2^{ton}		111.3$^{t \cdot m}$

Calculation of stability

Case 2 Dead + wind

eccentricity $e = \frac{142.8}{445.52} = 0.321$

$$q = \frac{445.52}{3.0 \times 7.0} \left(1 \pm \frac{6 \times 0.321}{7.0}\right) = 27.0 \text{ or } 15.4 \ t/m^2$$

Case 5 HA live + wind

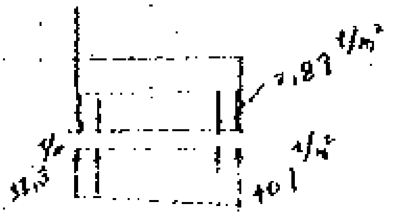
$e = \frac{111.3}{617.92} = 0.180$

$$q = \frac{617.92}{3.0 \times 7.0} \left(1 \pm \frac{6 \times 0.180}{7.0}\right) = 33.9 \text{ or } 24.9 \ t/m^2$$

29.4 t/m²

§ 7. Calculation of Footing Section

Longitudinal direction



37.3 − 7.89 = 29.41

40.1 − 2.89 = 37.21

$P = \frac{1}{2} \times (37.21 + 29.41) \times 1.1^{m} \times 3.5 = 128.2^{t}$

$M = 128.2 \times \frac{1}{3} \times 1.1 \times \frac{2 \times 37.21 + 29.41}{37.21 + 29.41} = 73.3^{t\cdot m}$

effective width 2.8^{m}

$M' = 73.3/2.8 = 26.2^{t\cdot m}/m$

$S' = 128.2/2.8 = 45.8^{t}/m$

$D20 - 150^{mm}\ c.t.c = 3.142 \times 6.667 = 20.95^{cm^2}$

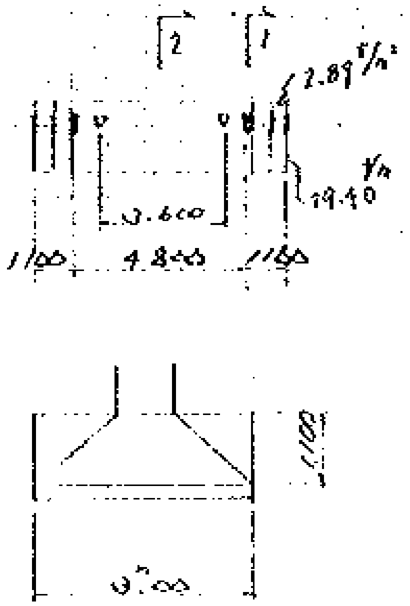
$(\text{Minimum}\ 100^{cm} \times 110^{cm} \times 0.0015 = 16.5^{cm^2})$

$np = 15 \times 20.95/_{100 \times 110} = 0.029 \qquad c = 10.1 \qquad s = 37.1$

$\sigma_c = \frac{2620000}{100 \times 110^2} \times 10.1 \doteqdot 22 \quad kg/cm^2$

$\sigma_s = \quad 〃 \quad \times 15 \times 37.1 = 1289 \quad 〃$

Transverse direction



Sect. 1).

$P = (29.40 - 2.89) \times 1.10 \times 3.0 = 87.48^{t}$

$M = 87.48 \times 1/2 \times 1.10 = 48.12^{t\cdot m}$

effective width 3.0^{m}

$M' = 16.04^{t\cdot m}$

$S' = 29.16^{t}$

Sect. 2)

$M = 1/8 \times (29.40 - 2.89) \times 3.6^{2} \times 3.0 = 128.84^{t\cdot m}$

Used $A_s = \overset{\phi 16 - 250}{\phi 20 - 250}$ c to c $= 20.6\ cm^{2} \times 3$

$np = 15 \times 20.6 \times 3 \times \frac{1}{300 \times 110} = 0.028$ $c = 10.2$ $s = 38.4$

$\sigma_c = \frac{128.84 \times 10^{5}}{300 \times 110^{2}} \times 10.2 = 36\ kg/cm^{2}$

$\sigma_s = 3.35 \times 15 \times 38.4 = 2045$ "

Sect. 2') $M = 1/12 \times (29.40 - 2.89) \times 4.8^{2} \times 3.0 = 152.70^{t\cdot m}$

Used $A_s = \phi 20 - 125$ c to c $= 25.12\ cm^{2} \times 3$

$\sigma_c = \frac{152.70 \times 10^{5}}{300 \times 110^{2}} \times 9.46 = 40.$

$\sigma_s =$ " $\times 37.0 \times 15 = 2020$

PIER IN PAILLES O.V.

(P - 3)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight $H = 10.7$ 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 $\sigma_{ck} = 255\ kg/cm^2$

bending stress $\sigma_{ca} = 85$

direct stress $\sigma_{ca} = 64.3$

shear stress $\tau_a = 8.2$

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

specified characteristic strength

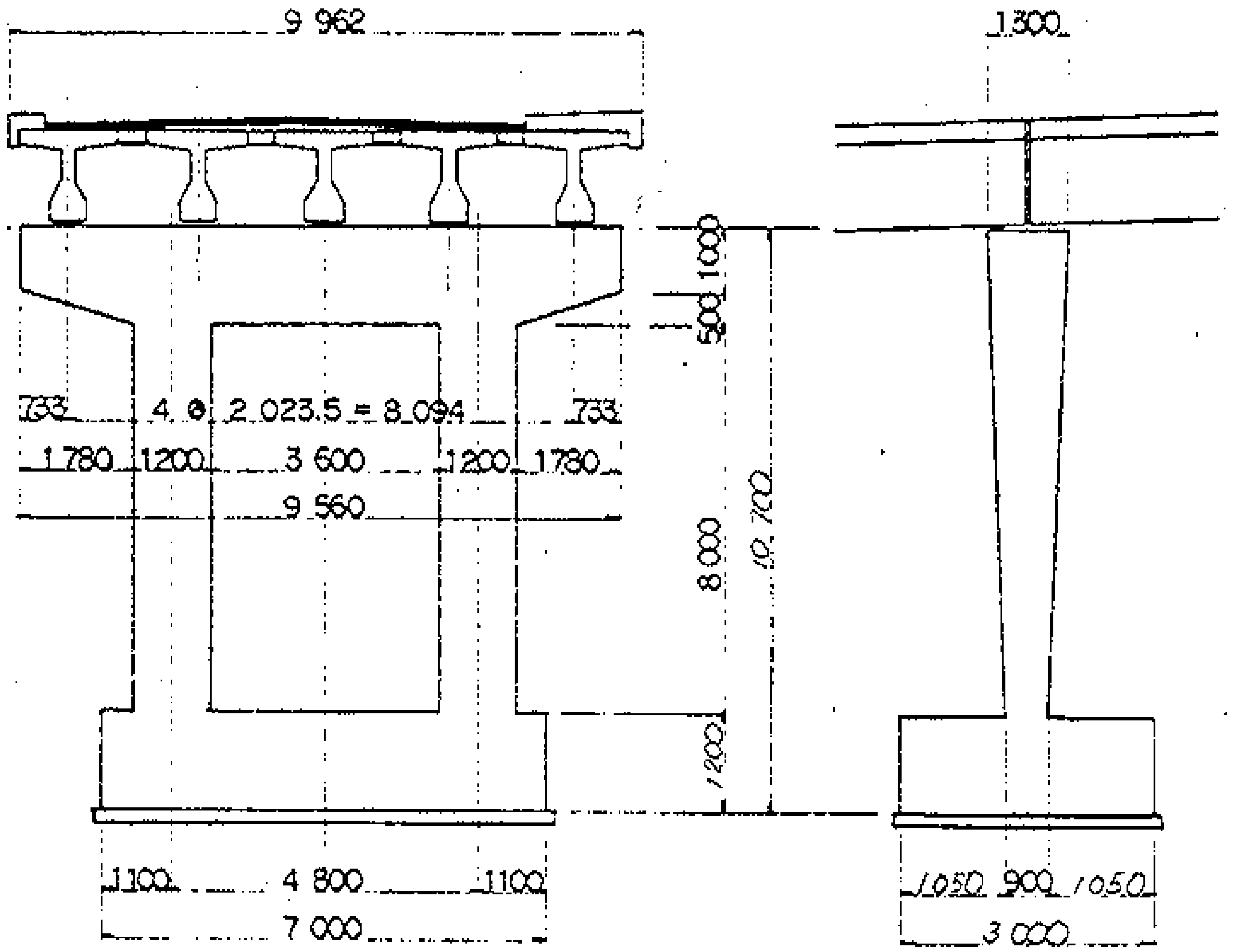
$\sigma_{su} = 4\ 180\ kg/cm^2$

permissible tensile stress

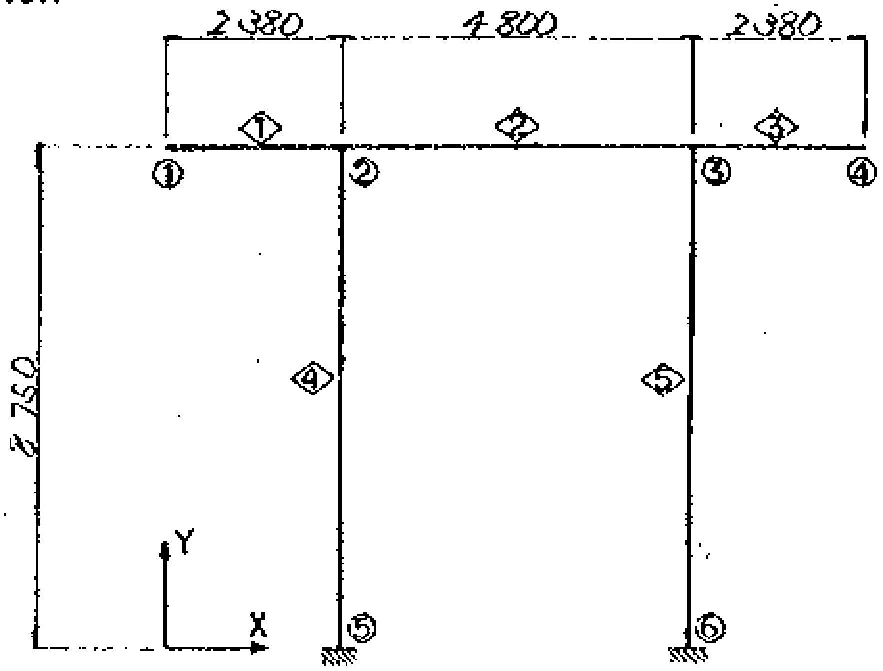
$\sigma_{sa} = 2\ 340\ kg/cm^2$

modular ratio $n = 15$

§ 2 GENERAL DIMENSION



1) Skeleton



○ : Joint number
◇ : Menber number

Coordinates

(m)

	X	Y		X	Y
①	0.0	8.75	④	9.56	8.75
②	2.38	8.75	⑤	2.38	0.0
③	7.18	8.75	⑥	7.18	0.0

2) Modulus of elasticity of concrete

$$E_c = 26\ \mathrm{kN/mm^2}$$

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

member	section B x H	area of section A (m²)	moment of inertia I (m⁴)
◇1	1.27 x 1.25	1.5875	0.2067
◇2	1.27 x 1.50	1.9050	0.3572
◇3	1.27 x 1.25	1.5875	0.2067
◇4	1.085 x 1.20	1.302	0.1562
◇5	1.085 x 1.20	1.302	0.1562

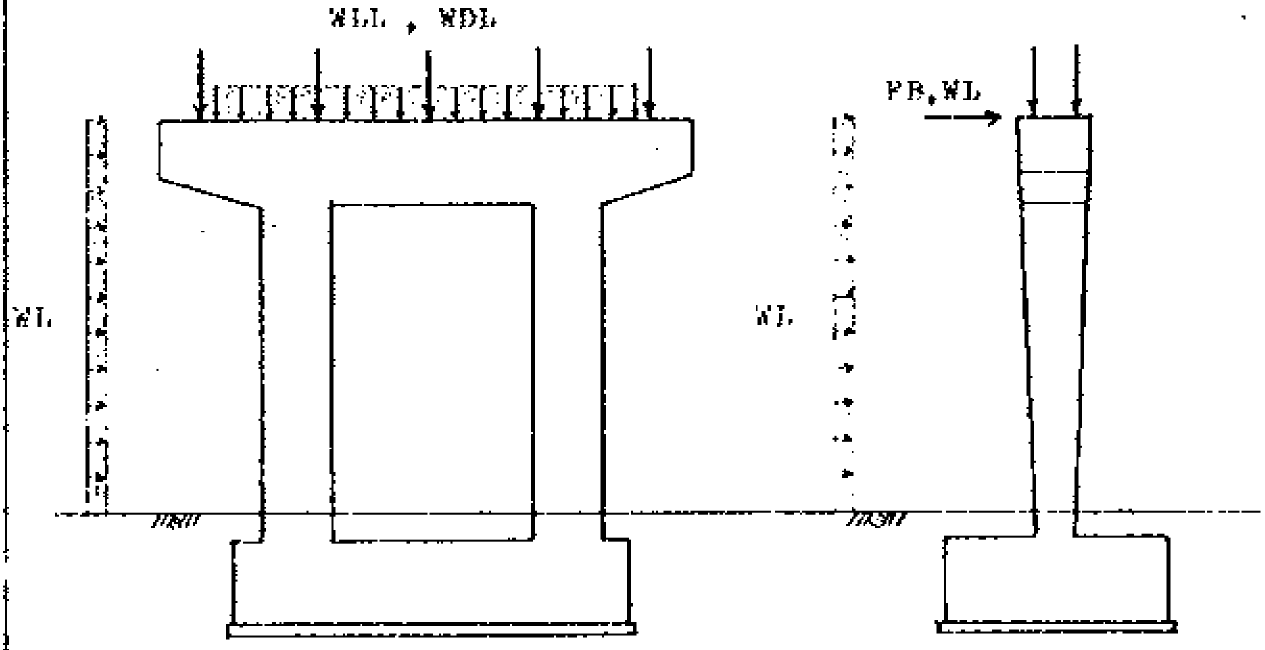
§ 3 CALCULATION OF LOADS

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	temperture change and drying schrinkage	FT & FS
8	wind	WL

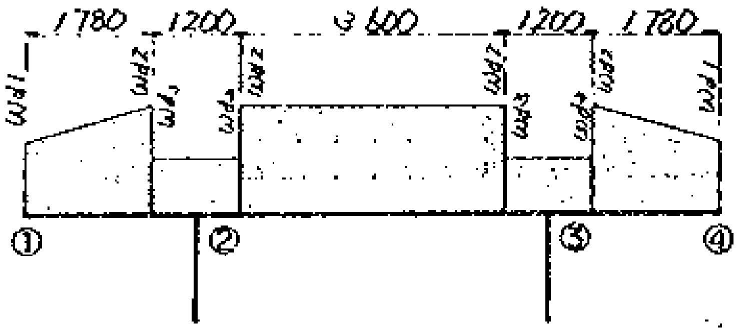
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8,	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3) Dead load

(a) beam



$W_{d1} = 1.27 \times 1.0 \times 2.407 = 3.06$ t/m

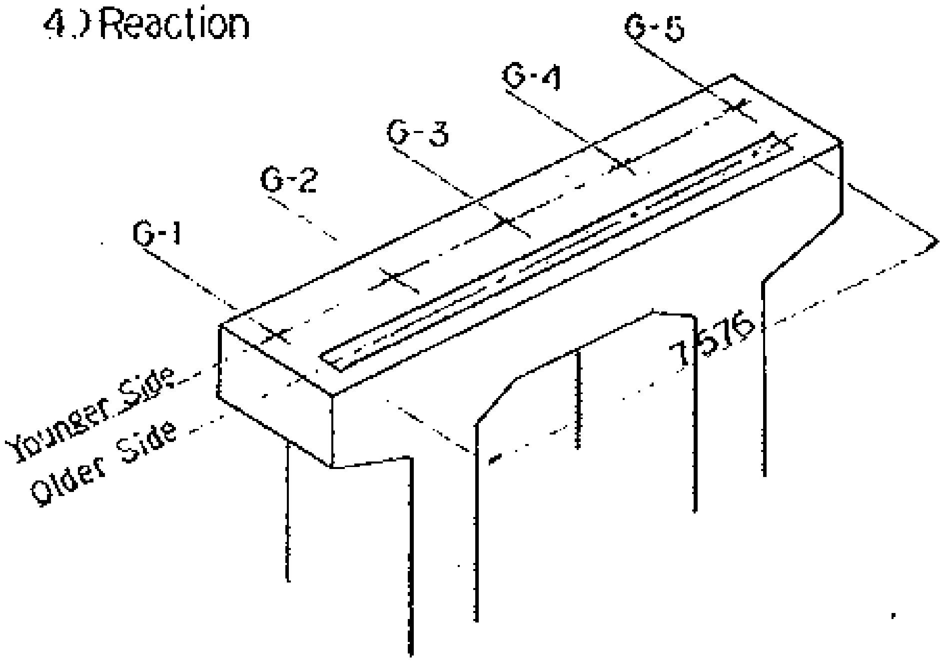
$W_{d2} = 1.27 \times 1.5 \times 2.407 = 4.59$ t/m

$W_{d3} = 1.27 \times 0.75 \times 2.407 = 2.29$ t/m

(b) column

$P_w = 1.085 \times 1.2 \times 2.407 = 3.13$ t/m

4.) Reaction



(ton)

			G-1	G-2	G-3	G-4	G-5
Dead load		Younger	32.8	30.6	31.2	30.4	29.4
		Older	57.3/7.576 = 7.56 t/m				
HA load	Uniform	Younger	8.6	14.1	14.7	16.2	14.7
		Older	22.5/7.576 = 2.97 t/m				
	line		2.5	7.1	6.7	7.4	4.9
Crowd load		Younger	6.0	1.4	1.2		
		Older	2.7/7.576 = 0.36 t/m				
HB load	(H B)		27.1	45.7	40.5	52.4	50.4
	H B		29.2	47.9	43.7	56.5	54.4
$\frac{1}{3}$HA	Uniform	Younger		1.3	1.9	1.2	0.5
		Older	3.8/7.576 = 0.50 t/m				
	line			0.4	0.6	0.4	0.2
Total (H A)			(17.1) 49.9	(22.6) 53.2	(22.6) 63.8	(23.6) 54.0	(19.6) 49.0
			(25.2/7.576 = 3.33) 82.5/7.576 = 10.89 t/m				
Total (H B)			(35.2) 68.0	(52.4) 83.0	(47.4) 78.6	(58.1) 88.5	(55.1) 84.5
			(6.5/7.576 = 0.86) 63.8/7.576 = 8.42 t/m				

5) Temperture change and Dry schrinkage

temperture change	-12 ~ +20 deg.
dry schrinkage	-20 deg.

6) Wind Load (transverse)

(a) For Superstructure

- Unloaded

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.28$ m/sec

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 59.28^2 \times 0.102 = 219.7$ kg/m²

$A = 1.785 \times 14.3 = 25.5$ m²

$C_D = 1.3$

$\therefore P_t = 219.7 \times 25.5 \times 1.3 = 7283$ kg

$\fallingdotseq 7.3$ ton

- live loaded

$V_c = 38 \times 1.0 \times 1.0 \times 1.62 = 61.56$ m/sec > 35 m/sec

$P_t = q \cdot A \cdot C_D$

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

$A = 4.07 \times 14.3 = 58.2$ m²

$C_D = 1.45$

$P_t = 76.6 \times 58.2 \times 1.45 = 6464$ kg

$\fallingdotseq 6.5$ ton

(b) For safety fences

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.28$ m/sec

$q = 219.7$ kg/m²

$A = (0.150 \times 14.3 + 0.850 \times 0.05 \times 14.3/2) \times 2 = 4.90$ m²

$C_D = 1.1$

$\therefore P_t = 219.7 \times 4.90 \times 1.1 = 1184$ kg $\fallingdotseq 1.2$ ton

(c) For Pier

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3 \ m/sec > 35 \ m/sec$

- Unlooded

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

$A_1 = (1.30 + 0.99) \times 7.3 \times 1/2 = 8.4 \ m^2$

$A_2 = (1.24 + 0.99) \times 5.8 \times 1/2 = 6.5 \ m^2$

$C_D = 1.7$

$P_t = 219.9 \times (8.4 + 6.5) \times 1.7 = 5570 \ kg$

$= 5.6^{ton}$

- live looded

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

$A = A_1 + A_2 = 14.9 \ m^2$

$C_D = 1.7$

$P_t = 76.6 \times 14.9 \times 1.7 = 1940 \ kg$

$= 1.9^{ton}$

(d) Table of Wind lord (transverce)

(ton)

	unlooded	live looded
Superstructure	7.3	6.5
safety fences	1.2	0.0
pier	5.6	1.9

7) Longitudinal forch

(a) due to braking

HA ---- 25.8 ton

HB ---- 38.8 ton

(b) due to Wind

for Superstructure

unloaded $P_{LS} = 0.25\ P_t = 0.25 \times 7.3 = 1.8$ ton

live loaded $P_{LL} = 0.5\ P_t$

$P_t = 76.6 \times (2.5 \times 14.3) \times 1.45 = 4.0$ ton

$\therefore P_{LL} = 2.0$ ton

$P_{LS} = 0.25\ P_t$

$P_t = 76.6 \times (4.07 - 2.5) \times 14.3 \times 1.3 = 2.2$ ton

$\therefore P_{LS} = 0.6$ ton

for Safety fences

vertical member

$P_L = 0.8\ P_t$

$P_t = 219.7 \times (0.85 \times 0.05 \times \frac{14.3}{2} \times 2) \times 1.1 = 0.15$ ton

$\therefore P_L = 0.1$ ton

Longitudinal member

$P_L = 0.4\ P_t$

$P_t = 219.7 \times (0.15 \times 14.3 \times 2) \times 1.1 = 1.0$ ton

$\therefore P_L = 0.4$ ton

for Pier

$V_c = 59.3 \ m/sec$

- Unlooded

$q = 219.9 \ kg/m^2$

$A_1 = (6.0 + 9.56) \times 1.5 \times 1/2 = 11.7 \ m^2$

$A_2 = 1.2 \times 5.8 \times 2 = 13.9 \ m^2$

$C_{D1} = 2.1$

$C_{D2} = 2.0$

$\therefore P_{t1} = 219.9 \times 11.7 \times 2.1 = 5.4^{ton}$

$P_{t2} = 219.9 \times 13.9 \times 2.0 = 6.1^{ton}$

- live loaded

$q = 76.6 \ kg/m^2$

$A_1 = 11.7 \ m^2$

$A_2 = 13.9 \ m^2$

$C_{D1} = 2.1$

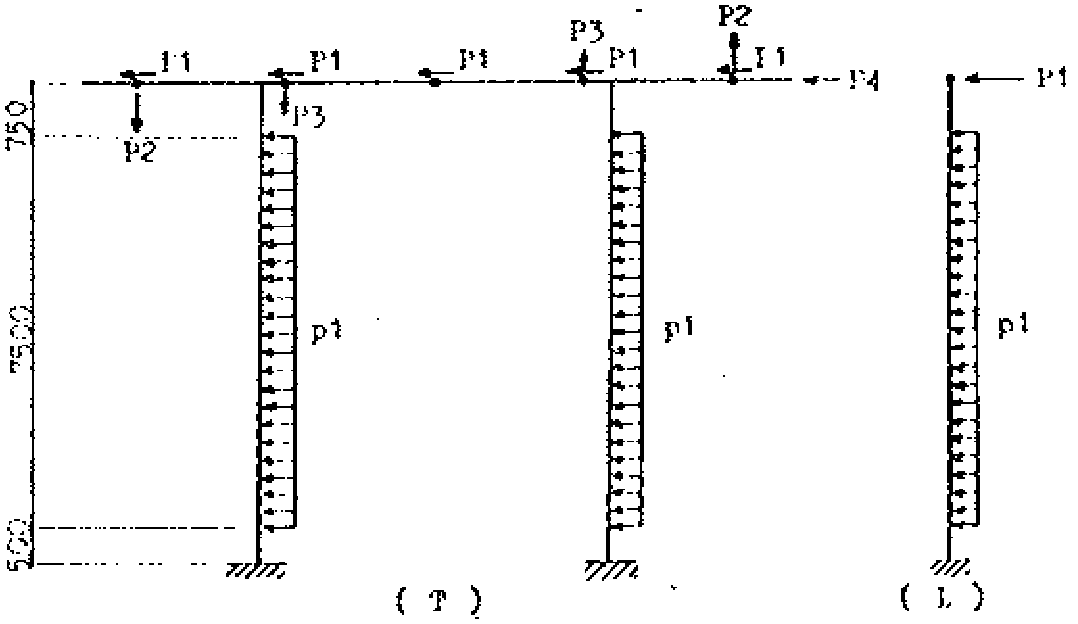
$C_{D2} = 2.0$

$\therefore P_{t1} = 76.6 \times 11.7 \times 2.1 = 1.9^{ton}$

$P_{t2} = 76.6 \times 13.9 \times 2.0 = 2.1^{ton}$

P3

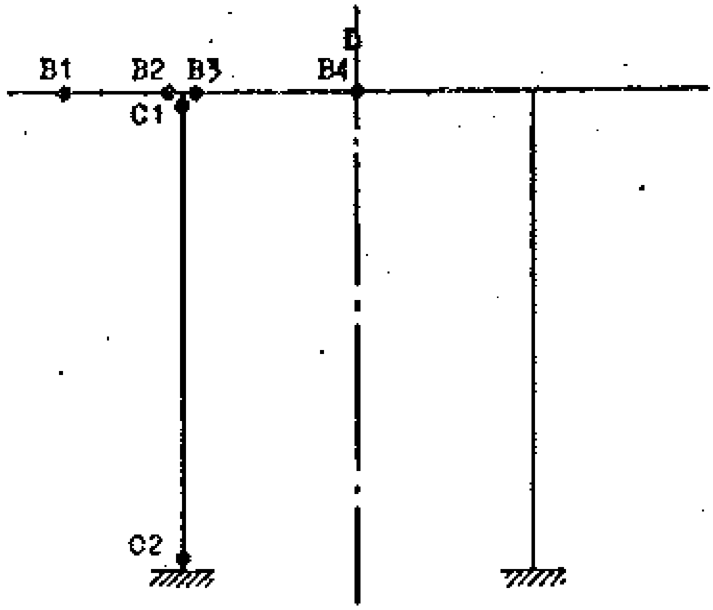
8) Wind loading diagram



		For place	unloaded	live loaded
(T)	P1	superstructure safety fences	1.70 ^{ton}	1.30 ^{ton}
	P2	superstructure ~~safety fences~~	1.50 ^{ton}	1.82 ^{ton}
	P3	superstructure safety fences	0.75 ^{ton}	0.91 ^{ton}
	P4	pier	0.71 ^{ton}	0.247 ^{ton}
	p1	pier	0.924 t/m	0.113 t/m
(L)	P1	superstructure safety fences	0.90 ^{ton}	2.30 ^{ton}
	p1	pier	0.413 t/m	0.147 t/m

§ 4 ACTING FORCE TABLE

1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.878	-0.000	-52.374	1.00
	N max.	+	2	-0.878	1.700	-36.774	1.00
		-	2	-0.878	-0.710	2.474	1.00
	S max.	+	3	-0.878	-0.000	2.474	1.00
		-	4	-0.703	-0.000	-56.379	1.25
B2	M max.	+					
		-	4	-125.948	-0.000	83.496	1.25
	N max.	+	2	-73.927	1.700	-52.976	1.00
		-	2	-63.387	-2.410	46.576	1.00
	S max.	+	4	-125.948	-0.000	83.496	1.25
		-	3	-102.828	-0.000	-73.198	1.00

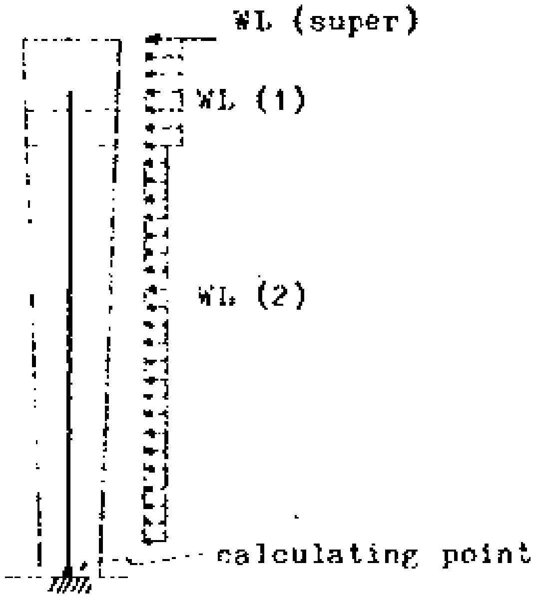
P-3

B3	M max.	+					
		-	4	-129.642	-2.005	-138.837	1.25
	N max.	+	6	-68.208	9.279	-54.450	1.30
		-	6	-86.405	-5.475	124.598	1.30
	S max.	+	6	-97.176	2.226	133.117	1.30
		-	4	-103.449	4.700	-138.837	1.25
B4	M max.	+	4	83.615	2.669	-30.215	1.25
		-	4	-34.636	1.483	-16.634	1.25
	N max.	+	6	-8.117	8.279	-10.397	1.30
		-	4	57.421	-4.036	-30.215	1.25
	S max.	+	6	21.610	5.896	6.786	1.30
		-	4	52.699	5.147	-48.085	1.25
C1	M max.	+	6	50.590	-133.335	-10.469	1.30
		-	4	-38.903	-125.326	8.188	1.25
	N max.	+					
		-	4	22.499	-222.332	-4.700	1.25
	S max.	+	4	-38.903	-125.326	8.188	1.25
		-	2	47.367	-108.416	-10.673	1.00
C2	M max.	+	3	32.959	-174.831	7.920	1.00
		-	2	-57.505	-135.803	-13.103	1.00
	N max.	+					
		-	4	-18.624	-244.242	-4.700	1.25
	S max.	+	4	32.744	-147.236	8.188	1.25
		-	2	-57.505	-135.803	-13.103	1.00

These value is divided by "i".

i ; coefficient of increase
in basic stresses.

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	61.3	210.8	6.5	1.00
case-8	91.0	274.1	9.6	1.25
case-9	86.9	210.8	9.9	1.15
case-10	116.6	274.1	13.0	1.30
case-11	47.4	151.1	7.0	1.05

CASE 7

	N	H	y	M
WDL	105.9			
WLL (A)	59.7			
FB (A)		6.45	9.5	61.3
W	45.2			
TOTAL	210.8	6.45		61.3

M = 61.3 tm

N = 210.8 ton

H = 6.5 ton

CASE 8

	N	H	y	M
WDL	105.9			
WLL (B)	123.0			
FB (B)		9.58	9.5	91.0
W	45.2			
TOTAL	274.1	9.58		91.0

M = 91.0 tm

N = 274.1 ton

H = 9.6 ton

CASE 9

	N	H	y	M
WDL	105.9			
WLL.FB (A)	59.7	6.45	9.5	61.3
W	45.2			
WL (super)		1.3	9.5	12.4
WL (1)		1.0	8.75	8.8
WL (2)		1.1	4.0	4.4
TOTAL	210.8	9.85		86.9

M = 86.9 t·m

N = 210.8 ton

H = 9.9 ton

CASE 10

	N	H	y	M
WDL	105.9			
WLL.FB (B)	123.0	9.58	9.5	91.0
W	45.2			
WL (super)		1.3	9.5	12.4
WL (1)		1.0	8.75	8.8
WL (2)		1.1	4.0	4.4
TOTAL	274.1	13.0		116.6

M = 116.6 t·m

N = 274.1 ton

H = 13.0 ton

CASE 11

	N	H	y	M
WDL	105.9			
W	45.2			
WL (Super)		1.2	9.5	11.4
WL (1)		2.7	8.75	23.6
WL (2)		3.1	4.0	12.4
TOTAL	151.1	7.0		47.4

M = 47.4 t·m

N = 151.1 ton

H = 7.0 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

§ 5 CALCULATION OF SECTION

1) Beam section

	at B1 case 1	at B2 case 4	at B3 case 4
M (tm)	-0.878	-157.435	-162.053
N (ton)	-0.000	-0.000	-2.506
Q:shear force (ton)	-70.474	104.370	-173.546
b (cm)	120	120	120
d (cm)	110.6	140	140
d' (cm)	10	10	10
As (cm²)	26.1 (D20 15cm pitch)	49.1 (10-D25 12.5cm pitch)	49.1 (10-D25 12.5cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.026	0.042	0.042
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	0.060	6.69	6.87
Q/b·d (kg/cm²)	5.31	6.21	10.33
c	10.5	8.71	8.71
s	41.3	26.2	26.2
	1.07	1.09	1.07
σc (kg/cm²)	0.6	58	60
σs (kg/cm²)	37	2629	2708
τ (kg/cm²)	5.68	6.77	11.26*
σca (kg/cm²)	106	106	106
σsa (kg/cm²)	2925	2925	2925
τa (kg/cm²)	10.3	10.3	10.3

Stirrups $\text{Req. } A_v = \frac{\tau \cdot b \cdot a}{\sigma_{sa}} = \frac{11.26 \times 120 \times 12.5}{1780 \times 1.25} = 7.59^{cm^2}$

$\text{Used } A_v = 4D16 \cdot 2 = 4 \times 2.01 = 8.04^{cm^2} > \text{Req } A_v$

	at B4		
	case 4	case	case
M (tm)	109.519		
N (ton)	3.336		
Q;shear force (ton)	- 37.769		
b (cm)	120		
d (cm)	140		
d' (cm)	10		
As (cm^2)	39.3 (8-D25 13ar pitch)		
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.035		
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	4.44		
Q/b·d (kg/cm²)	2.24		
c	9.35		
s	31.0		
	1.08		
σc (kg/cm²)	42		
σs (kg/cm²)	2065		
τ (kg/cm²)	2.42		
σca (kg/cm²)	85		
σsa (kg/cm²)	2340		
τa (kg/cm²)	8.2		

2) Column section

	case 2	case 10	case
M (tm)	-57.505	116.6	
N (ton)	-135.803	274.1	
Q;shear force (ton)	-13.103	13.0	
b (cm)	90	120	
d (cm)	110	80	
d' (cm)	10	10	
As (cm^2)	39.3 8-D25	39.3 8-D25	
d'/d	0.091	0.125	
f=(M/N)+u (cm)	92.3	77.5	
f/d	0.84	0.97	
n	15	15	15
nP=n·As/b·d	0.060	0.061	
M'=M+N·u (tm)	125.407	212.5	
$M'/(b \cdot d^2)$ (kg/cm^2)	11.52	27.67	
Q/b·d (kg/cm^2)	1.3	1.2	
C	3.2	3.75	
S	1.4	3.0	
Z			
σ_c (kg/cm^2)	37	104	
σ_s (kg/cm^2)	342	1245	
τ (kg/cm^2)	1.3	1.2	
σ_{ca} (kg/cm^2)	85	111	
σ_{sa} (kg/cm^2)	2340	3042	
τ_a (kg/cm^2)	8.2	10.7	

§6. Calculation of stability

Self weight

$1.27 \times 1/2 \times (1.0 + 1.5) \times 1.78 \times 2.407 \times 2 = 13.60^{t}$

$1.27 \times 1.5 \times 6.0 \times 2.407 = 27.51$

$1.085 \times 1.2 \times 8.0 \times 2.407 \times 2 = 50.14$

91.25

$3.0 \times 7.0 \times 1.2 \times 2.407 = 60.66$

151.91^{t}

Reaction due to superstructure

Dead load	154.9 + 57.3	=	211.7
HA Live	65.9 + 22.5 + 24.5	=	112.9
HB Live	126.9 + 11.0 + 3.8 + 4.1	=	145.3
Crowd load	7.8 + 2.7	=	10.5
	HA (total)	=	335.1 t
	HB "	=	367.5 t

Longitudinal. Force

a) due to braking

HA = 12.9 ton × 10.7 = 138.0

HB = 19.2 ton × 10.7 = 205.9

b). due to wind

for superstructure

unloaded PL = 1.8 t × 10.7 = 19.3

loaded PL = 2.0 × 10.7 = 21.4

PLo = 0.6 × 10.7 = 6.4

for safety fence

PL = 0.1 + 0.4 = 0.5 ton × 10.7 = 5.4

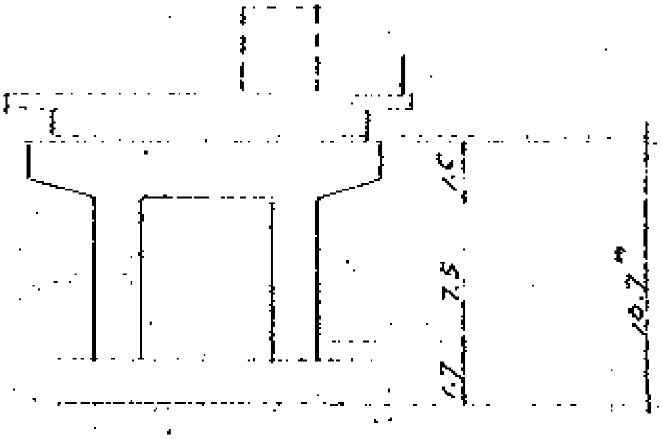
for pier

unloaded PL-1 = 3.4 t × 9.95 = 33.7

PL-2 = 6.1 t × 5.45 = 33.2

live loaded PL-1 = 1.9 × 9.95 = 18.9

PL-2 = 2.1 × 5.55 = 11.4



Acting Force at the center of bottom

Item		Virtical Force N(t)	distance x (m)	N x (t·m)	Horizontal force H(t)	distance y (m)	H y (t·m)
Pier self weight	Ws	151.9					
Dead load of deck	WDL	211.7					
HA Live	WLL(A)	123.4					
HB Live	WLL(B)	155.8					
Braking	FB (A)				11.9		138.0
"	FB (B)				19.2		205.4
Wind load	WL (unload)				13.8		111.6
	WL (load)				6.6		58.1
Dead load only		363.6					
Dead load + wind		363.6			13.8		111.6
HA Live		487.0			11.9		138.0
HB Live		519.4			19.2		205.4
HA + wind		487.0			19.5		196.1
HB + wind		519.4			25.8		263.5

Calculation of Stability.

Case 1. Dead load only.

$$\sigma = \frac{363.6}{7.0 \times 3.0} = 17.3 \ ^{t}/_{m^2}$$

Case 2. Dead load + Wind.

eccentricity $e = \frac{111.6}{363.6} = 0.307$

Reaction $\sigma = \frac{363.6}{7.0 \times 3.0} \times \left(1 \pm \frac{6 \times 0.307}{3.0}\right) = \left\{ \begin{matrix} 27.9 \\ 6.7 \end{matrix} \right. \ ^{t}/_{m^2}$

Case 3. HA Live.

$$e = \frac{138.0}{487.0} = 0.283$$

$$\sigma = \frac{487.0}{7.0 \times 3.0} \left(1 \pm \frac{6 \times 0.283}{3.0}\right) = \left\{ \begin{matrix} 36.3 \\ 10.0 \end{matrix} \right. \ ^{t}/_{m^2}$$

Case 4. H.B Live

$$e = \frac{205.4}{519.4} = 0.395$$

$$\sigma = \frac{519.4}{7.0 \times 3.0} \left(1 \pm \frac{6 \times 0.395}{3.0}\right) = \left\{ \begin{matrix} 44.3 \\ 5.2 \end{matrix} \right. \ ^{t}/_{m^2}$$

Case 5. HA + wind

$$e = \frac{196.1}{487.0} = 0.403$$

$$\sigma = \frac{487.0}{7.0 \times 3.0} \left(1 \pm \frac{6 \times 0.403}{3.0}\right) = \left\{ \begin{matrix} 41.9 \\ 4.5 \end{matrix} \right.$$

Case 6

$e = \frac{262.5}{519.4} = 0.507$ $x = 2.979 = \left\{\left(\frac{B}{2} - e\right) \times 3\right\}$

$$\sigma = \frac{519.4 \times 2}{7.0 \times 2.979} = 49.8 \ ^{t}/_{m^2}$$

Vol. 2 A_1 - ROAD OV. BR. P_1 . P_2 . P_3 PIERS

A1 - ROAD O.V.

(P - 3)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = 11.1 ^{m}

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2,407 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

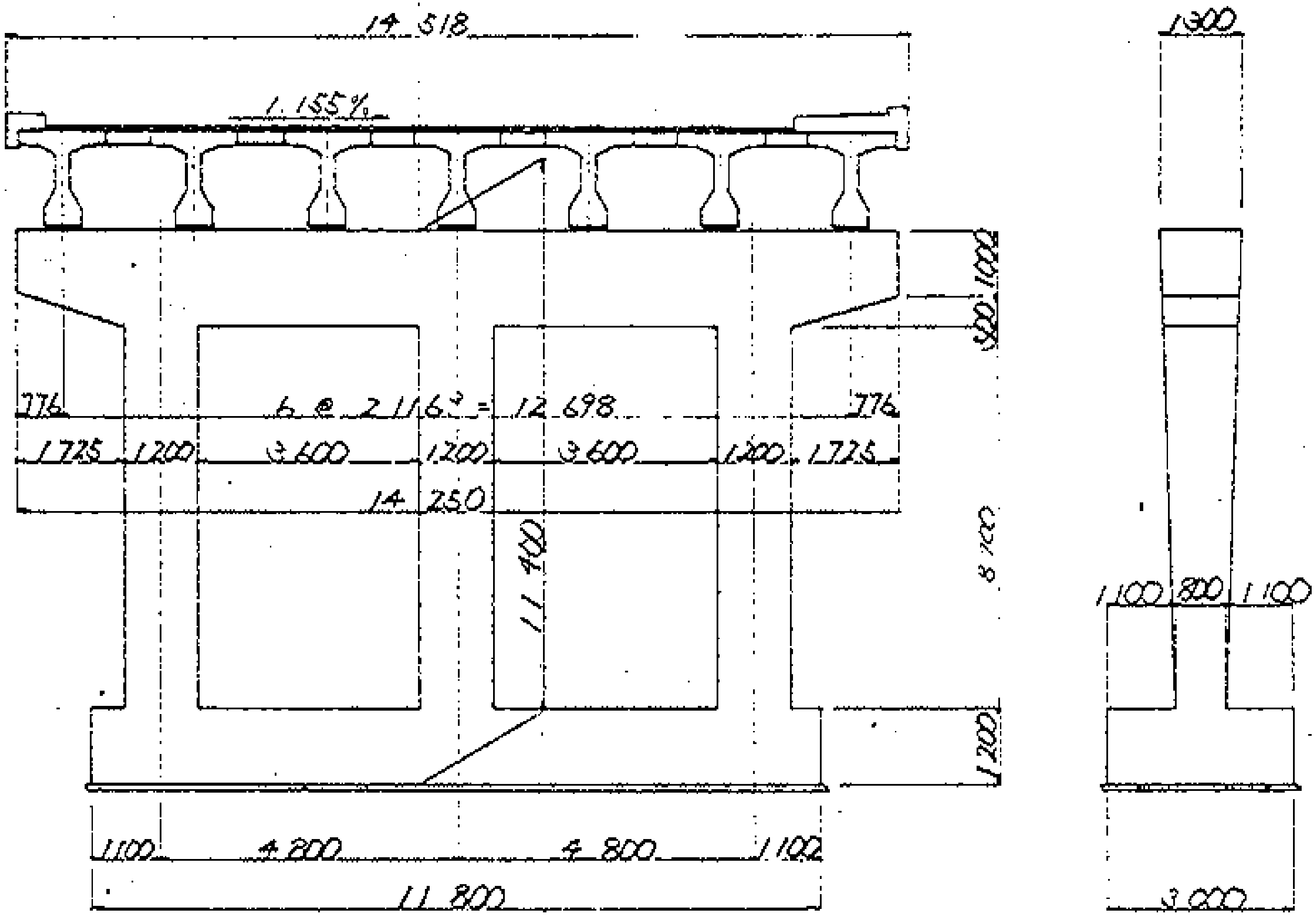
su = 4 180 kg/cm^2

permissible tensile stress

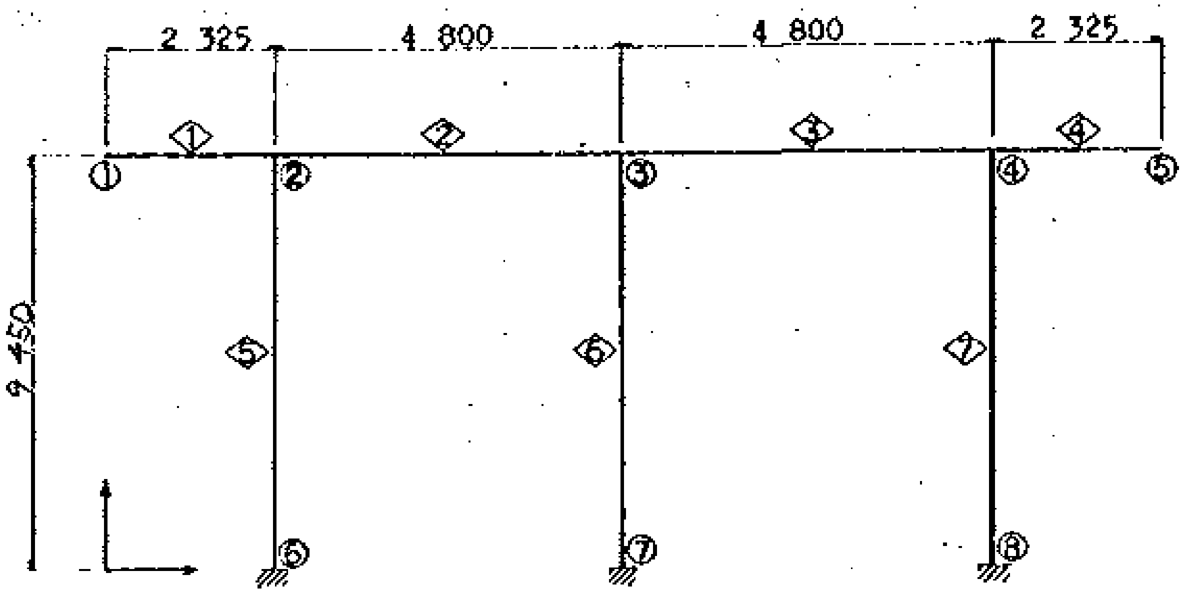
sa = 2 340 kg/cm^2

modular ratio n = 15

GENERAL DIMNSION



Skeleton



○ ; joint number

◇ ; member number

Coordinates

(m)

	X	Y		X	Y
①	0.0	9.450	⑤	14.250	9.450
②	2.325	9.450	⑥	2.325	0.0
③	7.125	9.450	⑦	7.125	0.0
④	11.925	9.450	⑧	11.925	0.0

2.) Modulus of elasticity of concrete

$Eo = 26\ kN/mm^2$

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

member	section B x H	area of section A (m^2)	moment of inertia I (m^4)
◇1	1.265 x 1.25	1.5813	0.20589
◇2	1.265 x 1.50	1.8975	0.35578
◇3	1.265 x 1.50	1.8975	0.35578
◇4	1.265 x 1.25	1.5813	0.20589
◇5	1.033 x 1.20	1.2396	0.14875
◇6	1.033 x 1.20	1.2396	0.14875
◇7	1.033 x 1.20	1.2396	0.14875

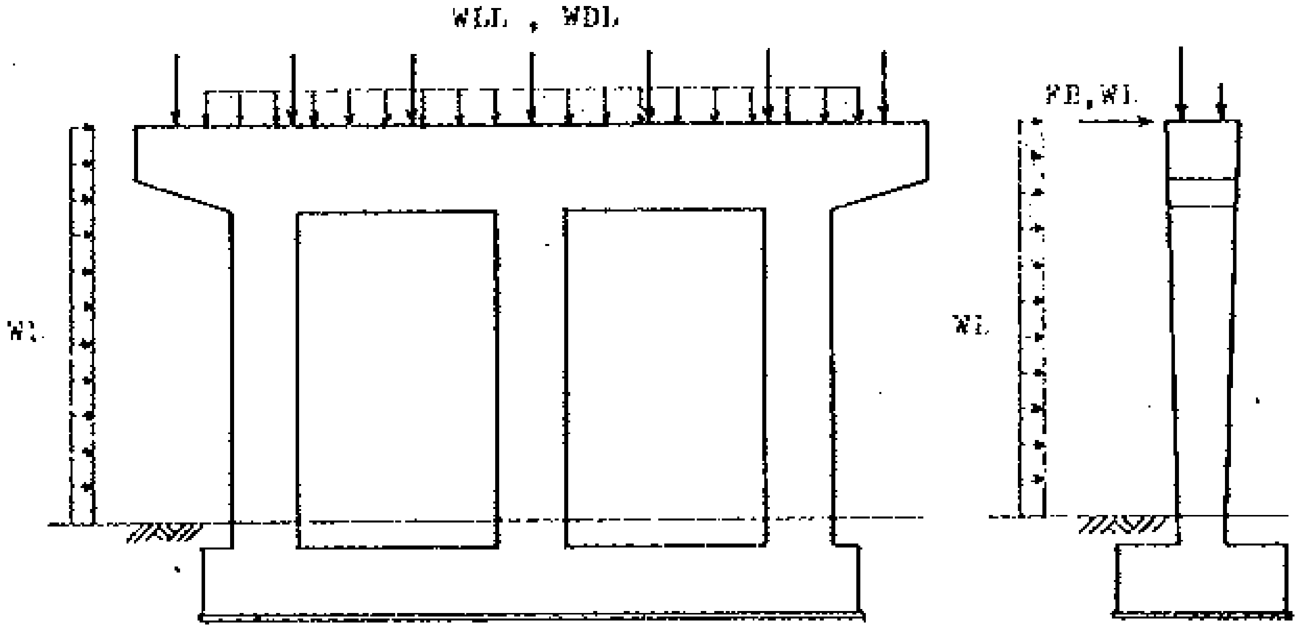
§ 3 CALCULATION OF LOADS

1.) Loading case and increase in basic stresses

No.	case 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	temperture charge and drying schrinkage	FT & FS
8	wind	WL

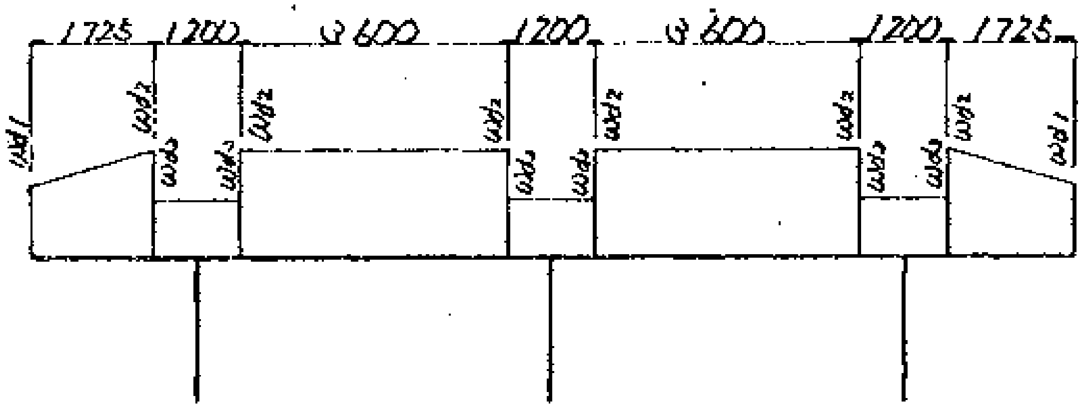
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3) Dead load

(a) beam



$Wd_1 = 1.265 \times 1.0 \times 2.407 = 3.04$ t/m

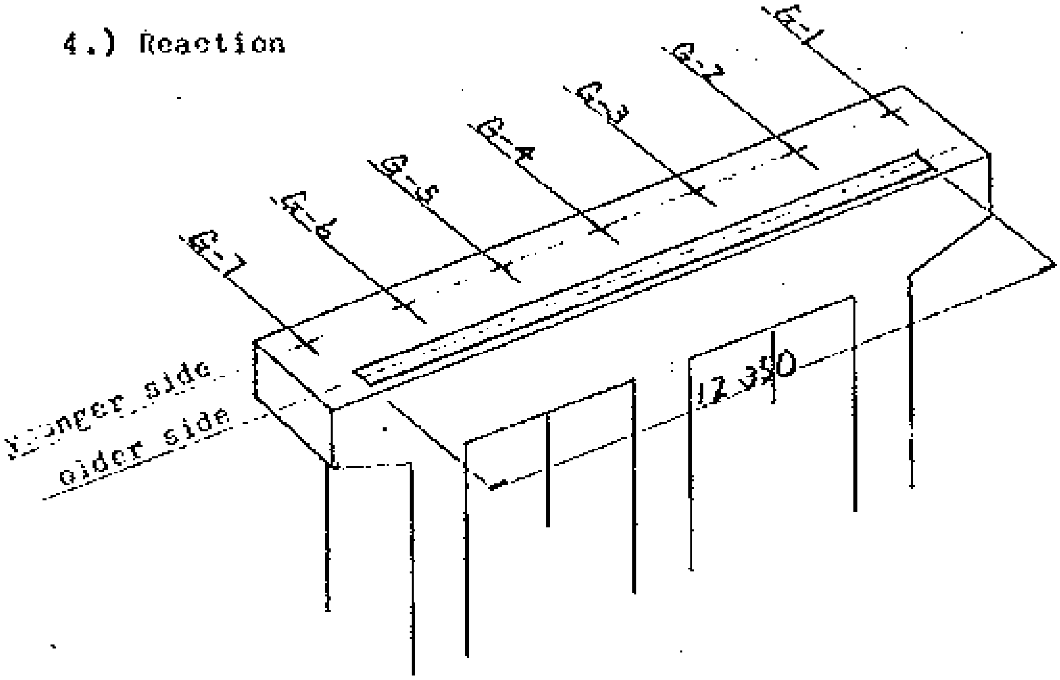
$Wd_2 = 1.265 \times 1.5 \times 2.407 = 4.57$ t/m

$Wd_3 = 1.283 \times 0.75 \times 2.407 = 2.32$ t/m

(b) column

$Pw = 1.033 \times 1.2 \times 2.407 = 2.98$ t/m

4.) Reaction



(unit ; ton)

			G-1	G-2	G-3	G-4	G-5	G-6	G-7
Dead load		younger	36.0	36.9	37.1	37.2	37.6	37.0	38.6
		older	8.640 t/m						
HA load	unit	younger	19.0	19.8	17.6	17.0	17.2	16.9	12.3
		older	2.866 t/m						
	line		7.0	8.6	9.9	9.1	8.9	9.3	3.6
Crowd load		younger			0.1	0.5	1.2	1.6	5.4
		older	0.259 t/m						
HB load	(HF)		50.3	48.0	47.0	41.8	39.8	49.1	26.9
	HF		54.3	51.8	50.7	45.1	42.9	53.0	29.0
(HA/3)	unit	younger	0.4	0.9	2.3	2.9	2.7	0.8	
		older	0.713 t/m						
	line		0.1	0.5	0.7	1.0	1.2	0.3	
Total (HA)			(26.0) 62.0	(28.4) 65.3	(27.6) 64.7	(26.6) 63.8	(27.3) 64.9	(27.8) 64.8	(21.3) 59.9
			(3.125 t/m) 11.765 t/m						
Total (HF)			(54.8) 90.8	(53.2) 90.1	(53.8) 90.9	(49.5) 86.7	(48.0) 85.6	(55.7) 92.7	(34.4) 73.0
			(0.972 t/m) 9.612 t/m						

5.) Temperture change and Drying schrinkage

temperture change -12 ~ +20 deg.

drying schrinkage -20 deg.

P3

6.) Wind Load (transeverse)

(a) For superstructure

Vc = 38 x 1.0 x 1.0 x 1.62 = 61.6 m/sec. 35 m/sec.

Unloaded

Pt = q·A·Cd

$q = 0.613 \times 61.6^2 \times 0.102 = 237.0\ \mathrm{kg/m^2}$

$A = 1.875 \times 15.05 = 28.2\ \mathrm{m^2}$

Cd = 1.2

Pt = 237.0 × 28.2 × 1.2

$= 8.0^{\mathrm{ton}}$

Live loaded

Pt = q·A·Cd

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ \mathrm{kg/m^2}$

$A = 4.080 \times 15.05 = 61.4\ \mathrm{m^2}$

Cd = 1.4

Pt = 76.6 × 61.4 × 1.4

= 6.6 ton

(b) For safety fences

Vo = 38 x 1.0 x 1.0 x 1.62 = 61.6 m/sec.

Pt = q·A·Cd

$q = 237.0\ \mathrm{kg/m^2}$

$A = (0.15 \times 15.05 + 0.85 \times 0.05 \times \frac{15.05}{2}) \times 2 = 5.15\ \mathrm{m^2}$

Cd = 1.1

$Pt = 237.0 \times 5.15 \times 1.1 = 1.3^{\mathrm{ton}}$

(c) For Pier

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

Unloaded

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9\ \mathrm{kg/m^2}$

$A1 = (1.30 + 1.23) \times 1.5 \times 1/2 = 1.90\ \mathrm{m^2}$

$A2 = (1.23 + 0.87) \times 8.2 \times 1/2 \times 3 = 25.22\ \mathrm{m^2}$

$Cd = 1.7$

$Pt = 219.9 \times (1.90 + 25.22) \times 1.7$

$= 10.1$ ton

Live loaded

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ \mathrm{kg/m^2}$

$A = A1 + A2 = 27.12\ \mathrm{m^2}$

$Cd = 1.7$

$Pt = 76.6 \times 27.12 \times 1.7$

$= 3.5$ ton

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	8.0	6.6
safety fences	1.3	
Pier	10.1	3.5

7.) Longitudinal force

(a) due to breaking

HA ——— 25.8 ton

HB ——— 38.3 ton

(b) due to Wind

for superstructure

unloaded $PLS = 0.25\ Pt = 0.25 \times 8.0$

$= 2.0^{ton}$

live loaded $PLL = 0.5\ Pt$

$Pt = 76.6 \times (2.5 \times 15.05) \times 1.4$

$= 4.0$

$PLL = 0.5 \times 4.0 = 2.0^{ton}$

$PLS = 0.25\ Pt$

$Pt = 76.6 \times (4.08 - 2.5) \times 15.05 \times 1.2$

$= 2.2$

$PLS = 0.25 \times 2.2 = 0.5^{ton}$

for safety fences

vertical member

$PL = 0.8\ Pt$

$Pt = 237.0 \times (0.85 \times 0.05 \times 15.05 \times \frac{1}{2} \times 2) \times 1.1$

$= 0.17$

$PL = 0.8 \times 0.17 = 0.1^{ton}$

longitudinal member

$PL = 0.4\ Pt$

$Pt = 237.0 \times (0.15 \times 15.05 \times 2) \times 1.1$

$= 1.18$

$PL = 0.4 \times 1.18 = 0.5^{ton}$

for Pier

Vc = 59.3 m/sec.

unloaded

$q = 219.9\ kg/m^2$

$A1 = (10.8 + 14.25) \times 1.5 \times 1/2 = 18.8\ m^2$

$A2 = 1.2 \times 8.2 \times 3 = 29.5\ m^2$

$Cd1 = 2.1$

$Cd2 = 2.0$

$Pt1 = 219.9 \times 18.8 \times 2.1 = 8.7\ ton$

$Pt2 = 219.9 \times 29.5 \times 2.0 = 13.0\ ton$

live loaded

$q = 76.6\ kg/m^2$

$A1 = 18.8\ m^2$

$A2 = 29.5\ m^2$

$Cd1 = 2.1$

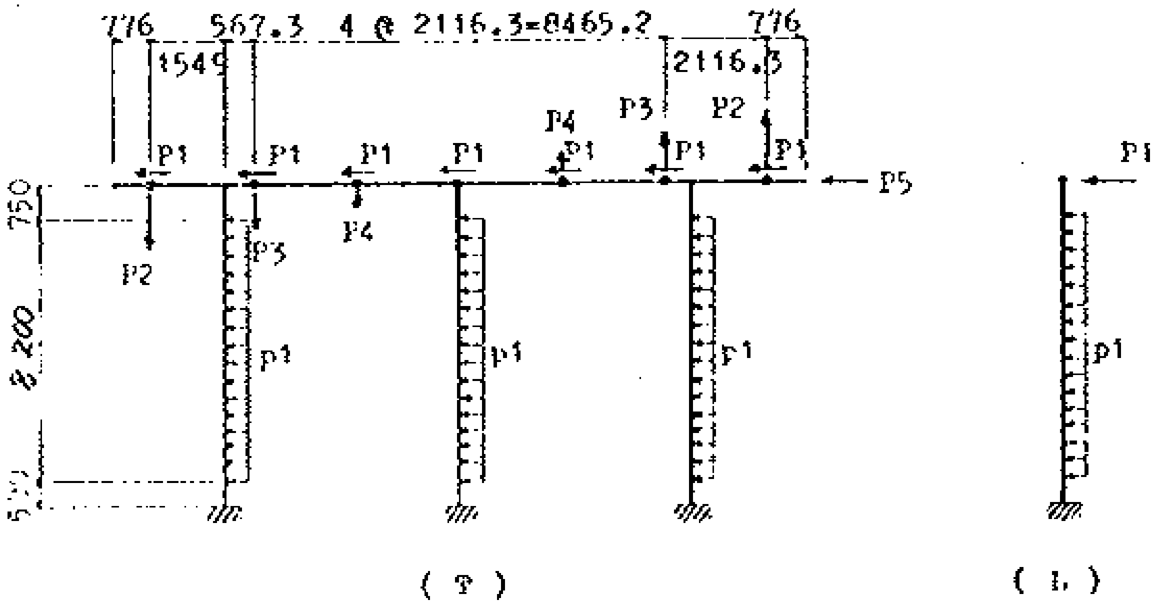
$Cd2 = 2.0$

$Pt1 = 76.6 \times 18.8 \times 2.1 = 3.0\ ton$

$Pt2 = 76.6 \times 29.5 \times 2.0 = 4.5\ ton$

P 3

8.) Wind loading diagram

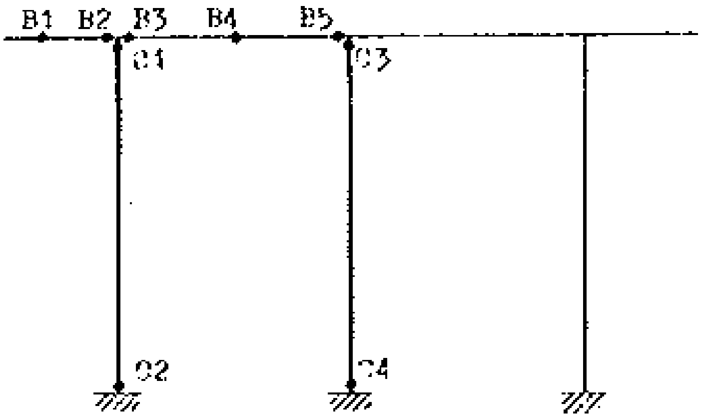


		for place	unloaded	live loaded
(T)	P1	superstructure safety fences	1.33	0.94
	P2	superstructure safety fences	0.54	0.68
	P3	superstructure safety fences	0.36	0.45
	P4	superstructure safety fences	0.18	0.23
	P5	pier	0.71	0.25
	p1	pier	0.37	0.13
(L)	P1	superstructure safety fences pier	3.77	1.83
	p1	pier	0.53	0.18

(P ; ton , p ; t/m)

§ 4 ACTING FORCE TABLE

1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.984	-0.000	-44.626	1.00
	N max.	+	2	-0.984	1.330	-39.166	1.00
		-	2	-0.984	-0.710	-2.626	1.00
	S max.	+					
		-	4	-0.788	-0.000	-74.741	1.25
B2	M max.	+					
		-	4	-127.498	-0.000	-89.578	1.25
	N max.	+	2	-74.405	1.330	-56.376	1.00
		-	2	-76.760	-2.040	57.896	1.00
	S max.	+					
		-	4	-127.498	-0.000	-89.678	1.25

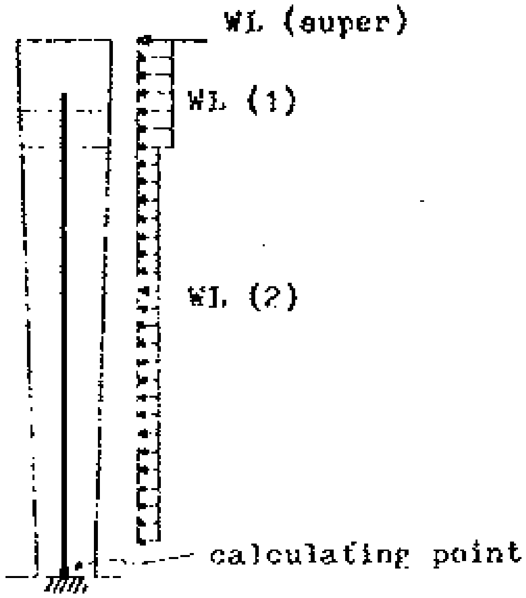
B3	N max.	+					
		-	6	-145.646	-4.643	138.421	1.30
	V max.	+	3	-64.229	10.917	79.654	1.00
		-	6	-94.980	-6.808	127.512	1.30
	M max.	+	4	-140.445	-3.625	141.795	1.25
		-					
B4	N max.	+	4	81.673	5.691	18.894	1.25
		-					
	V max.	+	3	74.103	10.917	12.400	1.00
		+	3	51.450	-6.570	30.250	1.00
	M max.	+	6	26.459	-3.370	51.497	1.30
		-					
B5	N max.	+					
		-	6	-89.811	5.757	83.857	1.30
	V max.	+	3	-62.292	10.917	-35.054	1.00
		-	3	-59.514	-6.570	-72.304	1.00
	M max.	+					
		-	3	-83.580	8.687	-91.894	1.00
	N max.	+					
		-					
	V max.	+					
		-					
	M max.	+					
		-					

C1	M max.	+	2	58.090	-127.125	-12.063	1.00
		-	3	-52.568	-165.786	10.917	1.00
	N max.	+					
		-	4	12.946	-231.373	-3.625	1.25
	S max.	+					
		-	2	58.090	-127.175	-12.063	1.00
C2	M max.	+	3	30.399	-193.947	10.917	1.00
		-	2	-69.857	-155.286	-15.097	1.00
	N max.	+					
		-	3	-26.147	-255.919	-4.923	1.00
	S max.	+					
		-	2	-69.857	-155.286	-15.097	1.00
C3	M max.	+	6	22.948	-176.560	-4.196	1.30
		-	4	-12.175	-156.679	2.102	1.25
	N max.	+					
		-	3	-0.334	-210.292	0.069	1.00
	S max.	+					
		-	6	22.948	-155.560	-4.196	1.30
C4	M max.	+	4	7.689	-179.208	2.102	1.25
		-	2	-27.105	-184.837	-6.668	1.00
	N max.	+					
		-	3	0.317	-268.453	0.069	1.00
	S max.	+					
		-	2	-27.105	-184.837	-6.668	1.00

These value is divided by "i".

i ; coefficient of increase
in basic stresses.

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7				1.00
case-8				1.25
case-9				1.15
case-10	89.6	227.3	9.71	1.30
case-11				1.00

CASE 10

	N	H	y	M
WDL	122.4			
WLL, FB(B)	58.6	6.38	10.20	65.1
W	46.3			
WL (Super)		0.83	10.20	8.5
WL (1)		1.00	9.45	9.5
WL (2)		1.50	4.35	6.5
TOTAL	227.3	9.71		89.6

M = 89.6 tm

N = 227.3 ton

H = 9.71 ton

CASE

	N	H	y	M
TOTAL				

M = tm

N = ton

H = ton

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1 case	B2, B3 case 6.	B4 case 4
M (tm)	-0.985	-189.940	101.966
N (ton)	-0.000	-6.036	7.114
Q;shear force (ton)	-93.426	179.947	23.618
b (cm)	120	120	120
d (cm)	107	140	140
d' (cm)	10	10	10
As (cm²)	25.1 D20 (15cm pitch)	64.3 8-D32 (15cm pitch)	39.3 8-D25 (15cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.079	0.057	0.035
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	0.07	8.05	4.94
Q/b·d (kg/cm²)	7.28	10.71	1.41
C	10.1	7.75	9.35
S	37.1	19.4	31.0
j	1.07	1.10	1.08
σ_c (kg/cm²)	0.7	62	41
σ_s (kg/cm²)	39	2343	2018
τ (kg/cm²)	7.19	11.78*	1.52
σ_{ca} (kg/cm²)	106	111	106
σ_{sa} (kg/cm²)	2925	3042	2925
τ_a (kg/cm²)	10.3	10.7	10.3

* stirrups Reg. Av = $\frac{\tau \times b \times a}{\sigma_{sa}} = \frac{11.78 \times 120 \times 12.5}{1780 \times 1.30} = 7.64$ cm²

Used Av = Φ16 ⊏ × 2 = 4 × 2.01 = 8.04 cm² > Reg.Av = 7.64 o.k.

P3

	BS case 6	case	case
M (tm)	-126.754		
N (ton)	7.484		
Q;shear force (ton)	109.014		
b (cm)	120		
d (cm)	140		
d' (cm)	10		
As' (cm^2)	39.3 (D25 15cm pitch)		
d'/d			
f=(M/N)+[illegible] (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.035		
M'=M+N·u (tm)			
M'/(b·d²) [illegible]	4.96		
Q/b·d (kg/cm^2)	6.49		
C	9.35		
S	31.0		
	1.08		
σc (kg/cm^2)	46		
σs (kg/cm^2)	2306		
τ (kg/cm^2)	7.00		
σca (kg/cm^2)	111		
σsa (kg/cm^2)	3.042		
τa (kg/cm^2)	10.7		

2.) Column section

	case 2	case 10	case
M (tm)	-69.857	89.6	
N (ton)	-155.286	227.3	
Q;shear force (ton)	-15.097	9.71	
b (cm)	80	120	
d (cm)	110	70	
d' (cm)	10	10	
As (cm^2)	39.3 8-D25	39.3 cm² 8-D25	
d'/d	0.09	0.14	
f=(M/N)+u (cm)	94.99	69.4	
f/d	0.86	0.99	
n	15	15	15
nP=n·As/b·d	0.067	0.07	
M'=M+N·u (tm)	147.5	157.8	
M'/($b \cdot d^2$) (kg/cm^2)	15.24	26.84	
Q/b·d (kg/cm^2)	1.57	1.01	
C	3.20	3.73	
S	1.50	3.00	
σ_c (kg/cm^2)	49	100	
σ_s (kg/cm^2)	732	1,208	
τ (kg/cm^2)	1.6	1.0	
σ_{ca} (kg/cm^2)	85	111	
σ_{sa} (kg/cm^2)	2340	3042	
τ_a (kg/cm^2)	8.2	10.7	

A,1 - ROAD O. V.

(P - 2)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight $H = 8.700$ m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.407 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

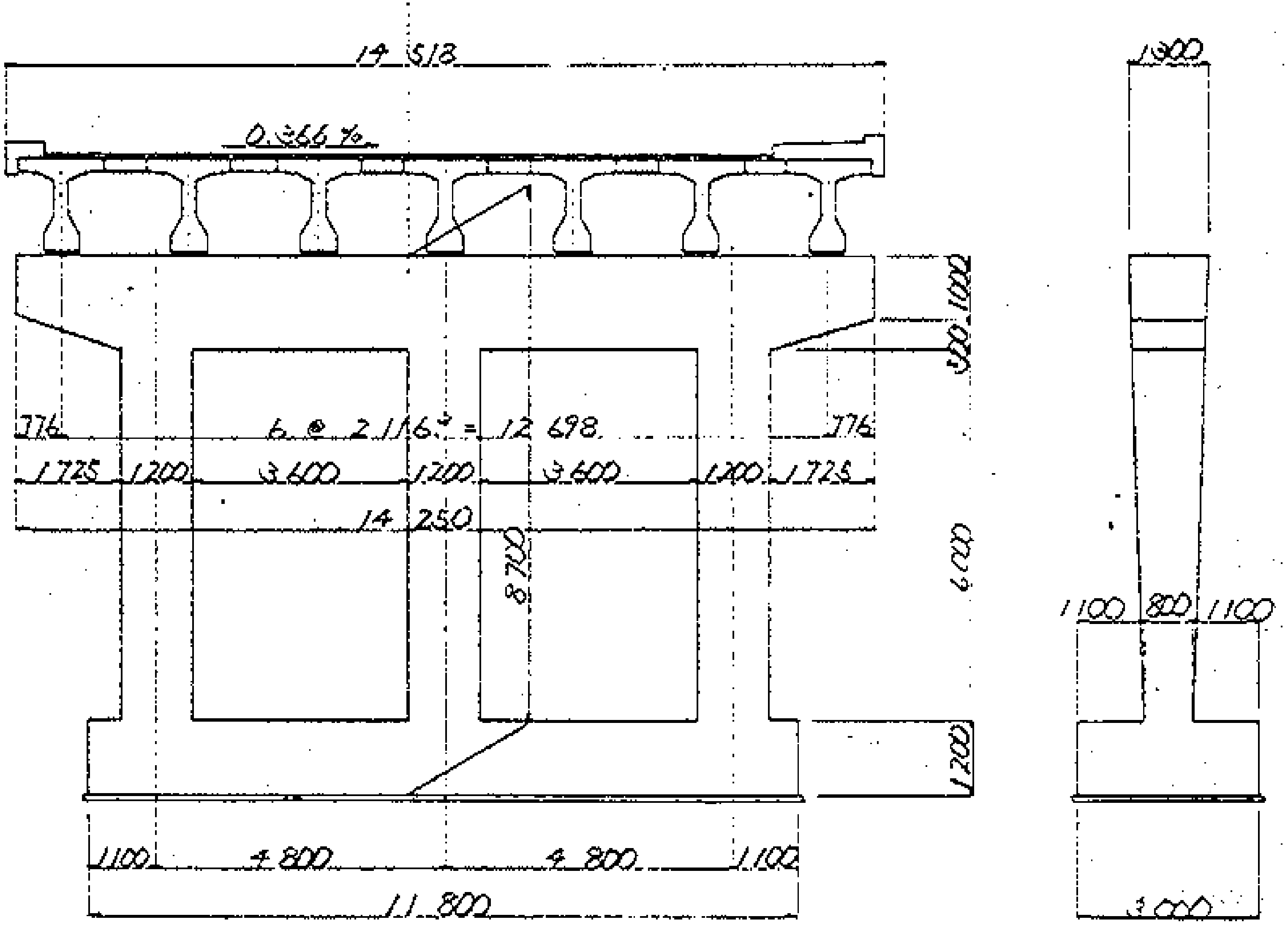
su = 4 180 kg/cm^2

permissible tensile stress

sa = 2 340 kg/cm^2

modular ratio n = 15

GENERAL DIMNSION



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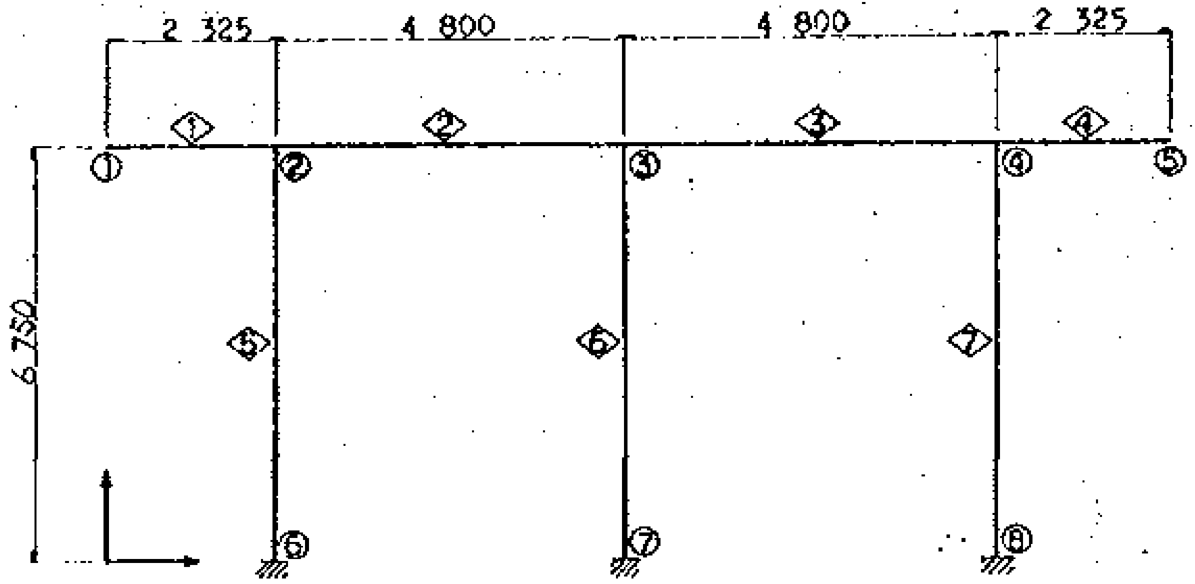
P2

Skeleton



○ ; joint number

◇ ; menber number

Coordinates

(m)

	X	Y		X	Y
①	0.0	6.750	⑤	14.250	6.750
②	2.325	6.750	⑥	2.325	0.0
③	7.125	6.750	⑦	7.125	0.0
④	11.925	6.750	⑧	11.925	0.0

2.) 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 (m^2)	moment of inertia I (m^4)
<1>	1.25 x 1.25	1.5625	0.20345
<2>	1.25 x 1.50	1.8750	0.35156
<3>	1.25 x 1.50	1.8750	0.35156
<4>	1.25 x 1.25	1.5625	0.20345
<5>	1.025 x 1.20	1.2300	0.14760
<6>	1.025 x 1.20	1.2300	0.14760
<7>	1.025 x 1.20	1.2300	0.14760

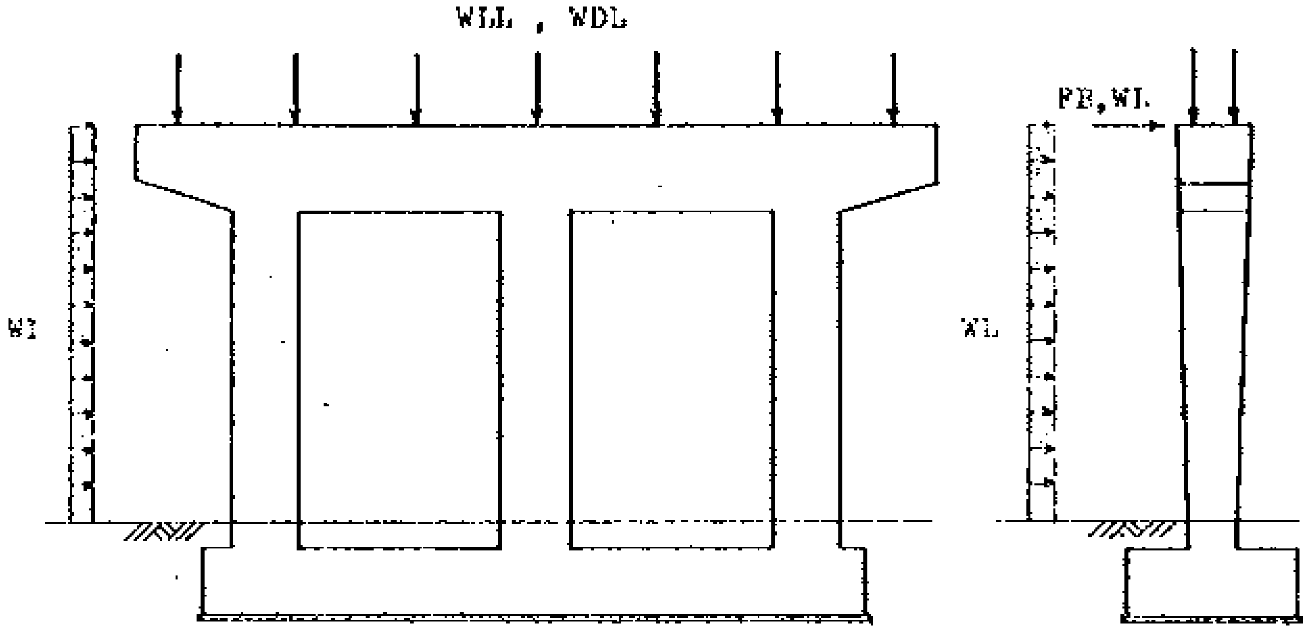
§ 3 CALCULATION OF LOADS

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	temperture change and drying schrinkage	FT & FS
8	wind	WL

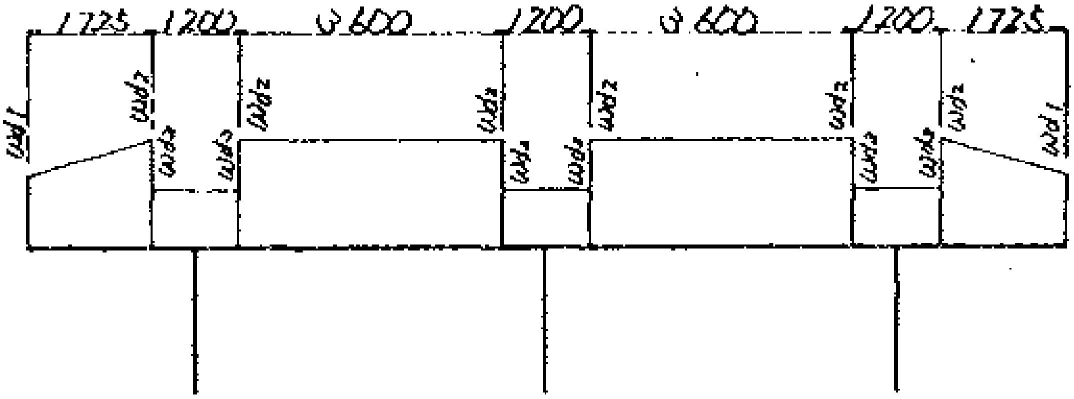
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



(3) Dead load

(a) beam



$Wd_1 = 1.25 \times 1.0 \times 2.407 = 3.01$ t/m

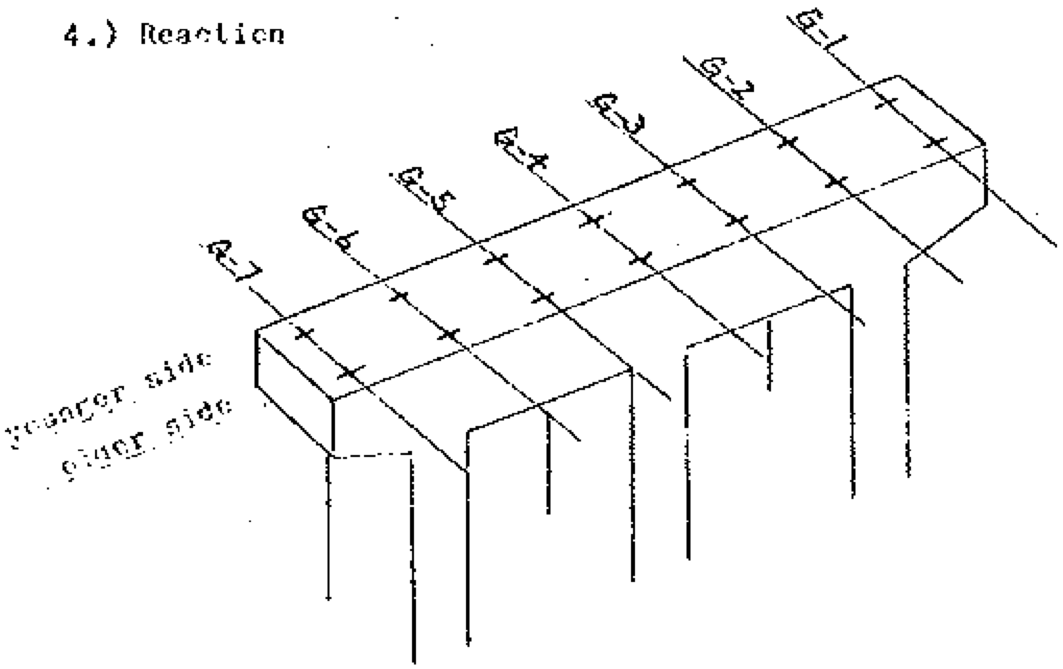
$Wd_2 = 1.25 \times 1.5 \times 2.407 = 4.51$ t/m

$Wd_3 = 1.275 \times 0.75 \times 2.407 = 2.30$ t/m

(b) Column

$P_w = 1.025 \times 1.2 \times 2.407 = 2.96$ t/m

4.) Reaction



(unit ; ton)

		G-1	G-2	G-3	G-4	G-5	G-6	G-7
[illegible]	younger	36.0	36.9	37.1	37.2	37.6	37.0	38.6
	older	36.0	36.9	37.1	37.2	37.6	37.0	38.6
[illegible] load	[illegible] younger	19.0	19.8	17.6	17.0	17.2	16.9	12.3
	[illegible] older	19.0	19.8	17.6	17.0	17.2	16.9	12.3
	line	7.0	8.6	9.9	9.1	8.9	9.3	3.6
Crowd load	younger			0.1	0.5	1.2	1.6	5.4
	older			0.1	0.5	1.2	1.6	5.4
[illegible] load (SE/[illegible])	(SE)	50.8	48.0	47.0	41.8	39.8	49.1	26.9
	SE	54.3	51.8	50.7	45.1	42.9	53.0	29.0
	[illegible] younger	0.4	0.9	2.3	2.9	2.7	0.8	
	[illegible] older	0.4	0.9	2.3	2.9	2.7	0.8	
	line	0.1	0.5	0.7	1.0	1.2	0.3	
Total (NA)		(45.0) 117.0	(48.2) 122.0	(45.3) 119.5	(44.1) 118.5	(45.7) 120.9	(46.3) 120.3	(39.0) 116.2
Total (SE)		(55.2) 127.2	(54.1) 127.9	(56.2) 130.4	(52.9) 127.3	(51.9) 127.1	(58.1) 132.1	(39.8) 117.0

5.) Temperture change and Drying schrinkage

temperture change $-12 \sim +20$ deg.

drying schrinkage -20 deg.

6.) Wind Load (transverse)

(a) For superstructure

$Vc = 38 \times 1.0 \times 1.0 \times 1.53 = 58.1$ m/sec. > 35 m/sec.

Unloaded

$Pt = q \cdot A \cdot Cd$

$q = 0.613 \times 58.1^2 \times 0.102 = 211.1$ kg/m²

$A = 1.875 \times 21.1 = 39.6$ m²

$Cd = 1.2$

$Pt = 211.1 \times 39.6 \times 1.2$

$= 10.0$ ton

Live loaded

$Pt = q \cdot A \cdot Cd$

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

$A = 4.08 \times 21.1 = 86.1$ m²

$Cd = 1.4$

$Pt = 76.6 \times 86.1 \times 1.4$

$= 9.2$ ton

(b) For safety fences

$Vc = 38 \times 1.0 \times 1.0 \times 1.53 = 58.1$ m/sec.

$Pt = q \cdot A \cdot Cd$

$q = 211.1$ kg/m²

$A = (0.15 \times 21.1 + 0.85 \times 0.05 \times \frac{21.1}{2}) \times 2 = 7.23$ m²

$Cd = 1.1$

$Pt = 211.1 \times 7.23 \times 1.1 = 1.7$ ton

(c) For Pier

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

Unloaded

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

A1 = $(1.30 + 1.20) \times 1.5 \times 1/2 = 1.88$ m²

A2 = $(1.20 + 0.83) \times 5.5 \times 1/2 \times 3 = 16.75$ m²

Cd = 1.7

Pt = $219.9 \times (1.88 + 16.75) \times 1.7$

= 7.0 ton

Live loaded

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

A = A1 + A2 = 18.63 m²

Cd = 1.7

Pt = $76.6 \times 18.63 \times 1.7$

= 2.4 ton

(d) Table of Wind load (transeverce)

(ton)

	unloaded	live loaded
super-structure	10.0	9.2
safety fences	1.7	
Pier	7.0	2.4

7.) Longitudinal force

(a) due to breaking

HA $\longrightarrow$ 25.8 ton

HB $\longrightarrow$ 38.3 ton

(b) due to Wind

for superstructure

unloaded PLS = 0.25 Pt = 0.25 × 10.0

= 2.5 ton

live loaded PLL = 0.5 Pt

Pt = 76.6 × (2.5 × 21.1) × 1.4

= 5.7 ton

PLL = 0.5 × 5.7 = 2.8 ton

PLS = 0.25 Pt

Pt = 76.6 × (4.08 − 2.5) × 21.1 × 1.2

= 3.06 ton

PLS = 0.25 × 3.06 = 0.8 ton

for safety fences

vertical member

PL = 0.8 Pt

Pt = 211.1 × (0.85 × 0.05 × 21.1 × ½ × 2) × 1.1

= 0.21

PL = 0.8 × 0.21 ≒ 0.2 ton

longitudinal member

PL = 0.4 Pt

Pt = 211.1 × (0.15 × 21.1 × 2) × 1.1

= 1.47

PL = 0.4 × 1.47 = 0.6 ton

P_2

for Pier

Vc = 59.3 m/sec.

unloaded

$q = 219.9\ kg/m^2$

$A1 = (10.8 + 14.25) \times 1.5 \times 1/2 = 18.8\ m^2$

$A2 = 1.2 \times 5.5 \times 3 = 19.8\ m^2$

$Cd1 = 2.1$

$Cd2 = 2.0$

$Pt1 = 219.9 \times 18.8 \times 2.1 = 8.7^{ton}$

$Pt2 = 219.9 \times 19.8 \times 2.0 = 8.7^{ton}$

live loaded

$q = 76.6\ kg/m^2$

$A1 = 18.8\ m^2$

$A2 = 19.8\ m^2$

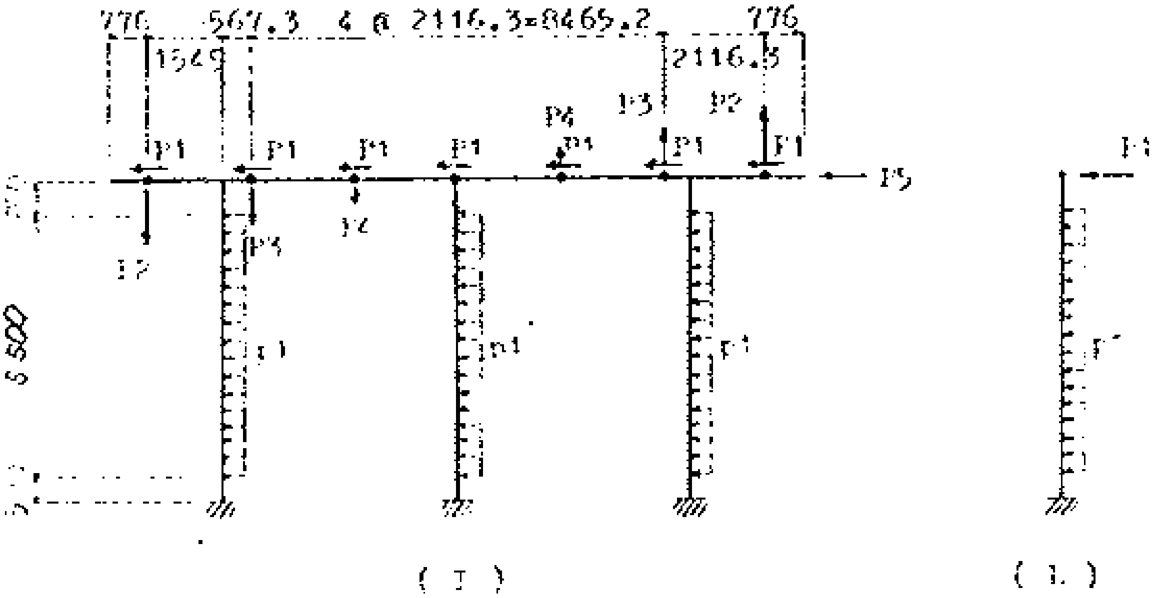
$Cd1 = 2.1$

$Cd2 = 2.0$

$Pt1 = 76.6 \times 18.8 \times 2.1 = 3.0^{ton}$

$Pt2 = 76.6 \times 19.8 \times 2.0 = 3.0^{ton}$

8.) Wind loading diagram

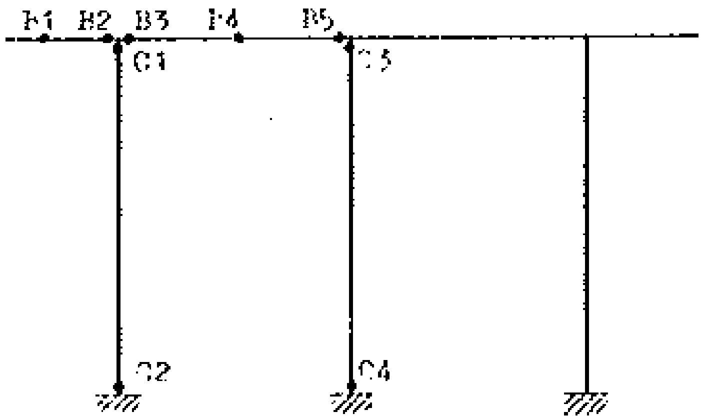


		for place	unloaded	live loaded
(T)	P1	superstructure safety fences	1.67	1.31
	P2	superstructure safety fences	0.68	0.95
	P3	superstructure safety fences	0.45	0.63
	P4	superstructure safety fences	0.23	0.32
	P5	pier	0.70	0.24
	p1	pier	0.38	0.13
(L)	P1	superstructure safety fences pier	4.00	2.20
	p1	pier	0.53	0.18

(P ; ton , p ; t/m)

§ 4 ACTING FORCE TABLE

1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.974	0.000	-119.598	1.00
	N max.	+	2	-0.974	1.610	-75.278	1.00
		-	2	-0.974	-0.700	2.598	1.00
	S max.	+					
		-	3	-0.974	0.000	-119.598	1.00
B2	M max.	+					
		-	3	-190.761	0.000	-124.866	1.00
	N max.	+	2	-122.109	1.670	-80.546	1.00
		-	2	-128.657	-2.370	84.986	1.00
	S max.	+					
		-	3	-190.761	0.000	-124.866	1.00

B3	M max.	+					
		-	3	-216.608	-10.334	201.608	1.00
	N max.	+	3	-87.230	29.939	109.393	1.00
		-	3	-166.088	-14.316	190.042	1.00
	S max.	+	3	-215.497	-10.259	203.420	1.00
		-					
B4	M max.	+	3	100.888	22.557	29.987	1.00
		-	6	-18.009	-4.930	46.434	1.30
	N max.	+	3	29.855	29.939	26.095	1.00
		-	3	56.220	-14.316	58.544	1.00
	S max.	+					
		-	3	33.574	-9.930	-29.859	1.00
B5	M max.	+					
		-	3	-119.846	23.279	-105.360	1.00
	N max.	+	3	-77.138	29.939	-57.603	1.00
		-	3	-68.762	-14.316	70.454	1.00
	S max.	+					
		-	3	-119.846	23.279	-105.360	1.00
	M max.	+					
		-					
	N max.	+					
		-					
	S max.	+					
		-					

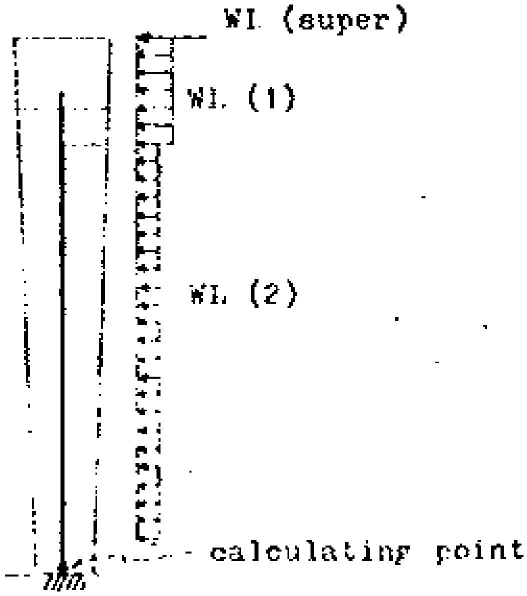
C1	N max.	+	3	103.115	-233.781	-29.865	1.00
		-	3	-103.531	-234.259	29.939	1.00
	M max.	+					
		-	3	24.736	-328.356	-10.259	1.00
	S max.	+	3	-103.531	-234.259	29.939	1.00
		-					
C2	N max.	+	3	98.556	-254.239	29.939	1.00
		-	2	-105.868	-201.611	-30.788	1.00
	M max.	+					
		-	3	-44.512	-348.336	-10.259	1.00
	S max.	+					
		-	2	-105.868	-201.611	-30.788	1.00
C3	N max.	+	5	26.111	-236.264	-6.849	1.15
		-	3	-18.135	-235.200	4.499	1.00
	M max.	+					
		-	3	-0.758	-319.495	0.221	1.00
	S max.	+					
		-	5	26.111	-236.264	-6.849	1.15
C4	N max.	+	3	12.233	-255.180	4.499	1.00
		-	5	-22.140	-253.637	-7.471	1.15
	M max.	+					
		-	3	0.733	-339.475	0.221	1.00
	S max.	+					
		-	5	-22.140	-253.637	-7.471	1.15

These value is divided by "i".

i ; coefficient of increase

in basic stresses.

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7				1.00
case-8				1.25
case-9				1.15
case-10	66.7	275.9	9.58	1.30
case-11				1.00

CASE

	N	H	y	M
WDL	173.6			
WLL, FB (B)	64.4	6.38	7.50	47.9
W	37.9			
WL (super)		1.20	7.50	9.0
WL (1)		1.00	6.75	6.8
WL (2)		1.00	3.00	3.0
TOTAL	275.9	9.58		66.7

M = 66.7 t·m

N = 275.9 ton

H = 9.58 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

P.2

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1 case 3	B2, B3 case 3	B4 case
M (tm)	-0.974	-216.608	100.838
N (ton)	0.000	-10.534	22.557
Q; shear force (ton)	-119.578	201.608	29.987
b (cm)	120	120	120
d (cm)	107	140	140
d' (cm)	10	10	10
As (cm²)	19.3 (D16 12.5cm pitch)	80.4 (10-D32 12.5cm pitch)	39.25 (8-D25 12.5cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.023	0.069	0.035
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	0.07	9.21	4.29
Q/b·d (kg/cm²)	9.31	12.00	1.78
C	11.1	7.22	9.35
S	46.5	16.1	31.0
	1.07	1.11	1.08
σ_c (kg/cm²)	0.8	66	40
σ_s (kg/cm²)	49	2224	1997
τ (kg/cm²)	9.97 *	13.32 *	1.92
σ_{ca} (kg/cm²)	85	85	85
σ_{sa} (kg/cm²)	2340	2340	2340
τ_a (kg/cm²)	8.2	8.2	8.2

$A_v = \frac{\tau \cdot b \cdot a}{\sigma_{sa}} = \frac{13.32 \times 120 \times 12.5}{1780} = 11.22^{cm^2}$

a: 12.5^{cm}

Used stirrups

D20⊓ × 2 = 4 × 3.14 = 12.56^{cm^2} > Req Av = 11.22^{cm^2}

P2

	B5 case 3	case	case
M (tm)	-119.846		
N (ton)	23.779		
S;shear force (ton)	-105.360		
b (cm)	120		
h (cm)	140		
d' (cm)	10		
As (cm²)	47.1 D25 (12.5cm pitch)		
d'/h			
e=(M/N)+u (cm)			
e/h			
n	15	15	15
p [illegible] As/b·d	0.042		
[illegible] (tm)			
M'/(b·d²) (kg/cm²)	5.10		
S/b·d (kg/cm²)	6.27		
C	8.71		
S	76.2		
	1.09		
σc (kg/cm²)	44		
σs (kg/cm²)	2.004		
τ (kg/cm²)	6.83		
σca (kg/cm²)	85		
σsa (kg/cm²)	2340		
τa (kg/cm²)	8.2		

P2

2.) Column section

	case 2	case 10	case
M (tm)	-105.868	66.7	
N (ton)	-201.611	275.9	
Q: shear force (ton)	-30,788	9.58	
b (cm)	80	120	
d (cm)	110	70	
d' (cm)	10	10	
As (cm^2)	39.3 8-D25	39.3 8-D25	
d'/d	0.09	0.14	
f=(M/N)+u (cm)	102.5	54.2	
f/d	0.93	0.77	
n	15	15	15
pt=n·As/b·d	0.067	0.07	
M'=M+N·u (tm)	206.7	149.3	
$M'/(b\cdot d^2)$ (%)	21.35	25.42	
Q/b·d (kg/cm^2)	3.50	1.0	
C	3.45	2.85	
S	2.30	0.70	
σ_c (kg/cm^2)	74	72	
σ_s (kg/cm^2)	737	267	
τ (kg/cm^2)	3.5	1.0	
σ_{ca} (kg/cm^2)	85	111	
σ_{sa} (kg/cm^2)	2340	3042	
τ_a (kg/cm^2)	8.2	10.7	

§ 6. Calculation of stability

Pier self weight

Cantilever beam	1/2 × (1.00 + 1.500) × 1.725 × 1.250 × 2.407 × 2	12.98
beam	1.50 × 10.80 × 1.250 × 2.407	48.74
column	6.0 × 1.2 × 1.025 × 2.407 × 3	53.29
Footing	1.2 × 11.80 × 3.0 × 2.407	102.25
Σ		217.26

Reaction due to superstructure

	P1	P2	P3
Dead load	53.2 260.9	520.8	260.9 106.1
HA Live load	30.2 + 86.9 132.1	209.9	86.9 + 35.9 132.1
HB Live load	126.9 + 7.6 131.6 + 8.0	171.6	176.9 + 11.6 188 + 8.0
Crowd load	7.1 7.8	15.6	7.8 3.2
Total (HA)	472.3	741.3	532.0
Total (HB)	487.2	719.0	512.9

Transverse direction

a) due to wind

- for superstructure

Unloaded Pt = 10.0^t

Live loaded Pt = 7.2^t

- for safety fences

Pt = 1.7^t

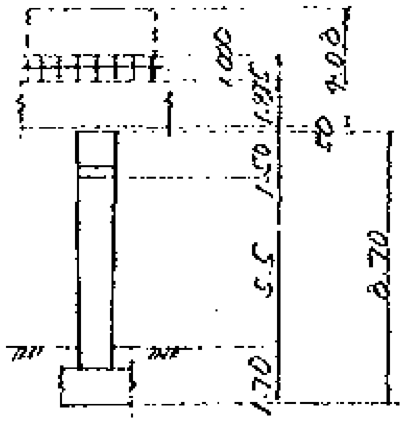
- for pier

Unloaded P_{t-1} = 0.7^t

P_{t-2} = 6.3^t

Live loaded P_{t-1} = 0.2^t

P_{t-2} = 2.2^t



Longitudinal direction

a) due to braking

Under HA ; FB = 25.8 × 1/2 = 12.9 ton

" HB ; FB = 38.3 × 1/2 = 19.2 "

b) due to wind

- for superstructure

Unloaded P_{LS} = 2.5 ton

Live loaded P_{LL} = 2.8

P_{LS} = 0.8

- for safety fences P_{L1} = 0.2

P_{L2} = 0.6

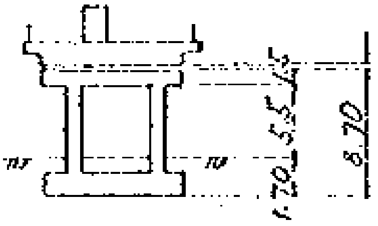
- for pier

Unloaded P_{L1} = 8.7

P_{L2} = 8.7

Live loaded P_{L-1} = 3.0

P_{L-2} = 3.0



		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	520.8					
	WLL (HA)	220.5			12.9	8.75	112.88
	WLL (HB)	193.2			19.2	8.75	168.00
	Self weight	217.3					
Transverse direction	WL (Unloaded)						
	for superstruct.				10.0	7.688	76.88
	for safety fence				1.7	11.125	18.91
	for pier (1)				0.7	7.950	5.57
	(2)				6.3	4.450	28.04
	WL (live loaded)						
	for superstruct (1)				9.2	10.790	99.27
	〃 (2)						
	for pier (1)				0.2	7.950	1.59
	〃 (2)				2.2	4.450	9.79
Longitudinal direction	WL (Unloaded)						
	for superstruct.				2.5	8.75	21.88
	for safety fence				0.8	8.75	7.00
	for pier (1)				8.7	7.95	69.17
	(2)				8.7	4.45	38.72
	WL (live loaded)						
	for superstruct. (1)				2.8	8.75	24.50
	〃 (2)				0.8	8.75	7.00
	for pier (1)				3.0	7.95	23.85
	(2)				3.0	4.45	13.35

Longitudinal direction

Case No	Loading combination	N (t)	H (t)	M (t)
1	Dead load only	738.1	0	0
2	Dead load + wind	738.1	20.7	136.77
3	HA Loading	958.6	12.9	112.88
4	HB Loading	931.3	19.2	168.00
5	HA Loading + wind	958.6	27.5	181.58
6	HB loading + wind	931.3	28.8	236.70

Calculation of Stability

- eccentric $e = M/N$ $B/6 = 0.5$
- 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) $e > B/6$ $x = (\frac{B}{2} - e) \times 3$

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

- Sliding $f = \frac{N \times 0.6}{H}$

Case No.	B/6 (m)	eccentric e (m)	Soil reaction q max (t/m²)	Soil reaction q min (t/m²)	qa t/m²	Sliding F	Sliding Fa
1		0	20.9				
2	0.500	0.185	28.6	13.1	60.0	21	1.5
3	|	0.118	33.5	20.7	60.0	45	|
4	|	0.180	35.8	16.8	75.0	29	|
5	|	0.189	37.3	16.8	69.0	26	|
6	|	0.254	39.7	12.9	78.0	19	|

Transverse direction.

Case No	Loading combination	N (t)	H (t)	M (t)
~~1~~	~~Dead load only~~			
2	Dead load + wind	738.1	18.7	149.40
~~3~~	~~HA loading~~			
~~4~~	~~HB loading~~			
5	HA loading + wind	958.6	11.6	110.65
6	HB loading + wind	931.3	11.6	110.65

Calculation of Stability

- eccentric $e = M/N$

$B/6 = 11.8/6 = 1.97$

- 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) $e > B/6$

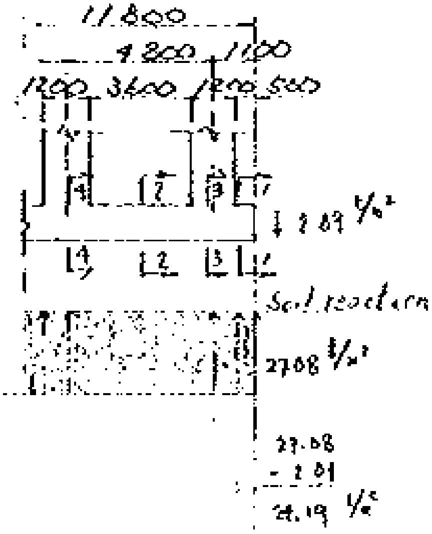
$$q_{max} = \frac{2 \cdot N}{B \cdot L}$$

- Sliding $F = \frac{N \cdot 0.6}{H}$

Case No.	B/6 (m)	eccentric e (m)	Soil reaction q max (t/m²)	q min (t/m²)	q_a t/m²	Sliding F	Fa
~~1~~							
2	1.970	0.202	13.0	18.7	60.0		
~~3~~							
~~4~~							
5	1.970	0.115	28.7	25.5	69.0		
6	1.970	0.119	27.9	24.7	78.0		

§7. Calculation of footing section

1 Transverse direction



Sect 1.

$$M = 24.19 \times 0.5^2 \times \frac{1}{2} \times 3.0 = 9.07^{\,t\cdot m}$$

$$M' = 9.07 \times \frac{1}{3} = 3.02^{\,t\cdot m}$$

Sect 2.

$$M = 24.19 \times 3.6^2 \times \frac{1}{8} \times 3.0 = 117.56^{\,t\cdot m}$$

$$M' = 117.56 \times \frac{1}{3} = 39.19^{\,t\cdot m}$$

Sect 3.

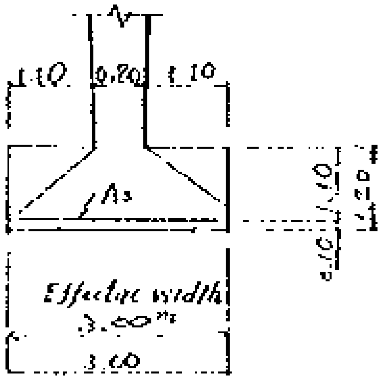
$$M = 24.19 \times 4.8^2 \times \frac{1}{12} \times 3.0 = 139.33^{\,t\cdot m}$$

$$M' = 139.33 \times \frac{1}{3} = 46.44^{\,t\cdot m}$$

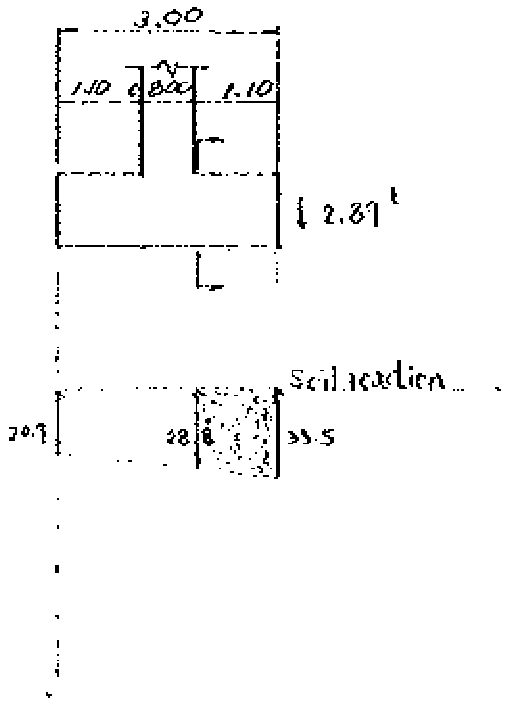
Sect 4.

$$M = 24.19 \times 4.2^2 \times \frac{1}{8} \times 3.0 = 160.02^{\,t\cdot m}$$

$$M' = 160.02 \times \frac{1}{3} = 53.34^{\,t\cdot m}$$



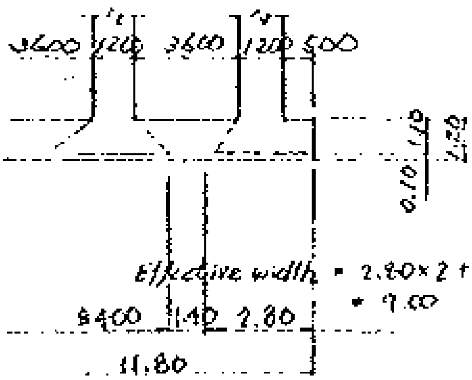
2 Longitudinal direction



$$S = 2.89 \times 1.10 \times 11.86 \overset{37.51}{}$$
$$- \tfrac{1}{2}(33.5 + 28.8) \times 1.10 \times 11.80 \overset{404.33}{} = -366.82$$

$$M = 37.51 \times \tfrac{1}{2} \times 1.10 \overset{20.63}{}$$
$$- 404.33 \times \tfrac{1}{3} \times \frac{2 \times 33.5 + 28.8}{33.5 + 28.8} \times 1.10 \overset{-227.97}{} = -207.34$$

$$S' = -366.82 \times 1/9.0 = 40.76^{\,t/m}$$
$$M' = -207.34 \times 1/9.0 = 23.04^{\,t\cdot m}$$



Effective width = 2.80×2 + 3.40
= 9.00

	Transverse direction		longitudinal directio
	case bottom	case Top	case bottom
M (tm)	53.34	39.19	23.04
N (ton)			
Q;shear force (ton)			
b (cm)	100	100	100
d (cm)	110	110	110
d' (cm)	10	10	10
As (cm^2)	Φ20 - 125 = 25.12	Φ20 - 125 = 25.12	Φ16 - 125 = 16.08
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.034	0.034	0.022
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)	4.41	3.24	1.90
Q/b·d (kg/cm^2)			
C	9.46	9.46	11.3
S	32.0	32.0	48.5
σ_c (kg/cm^2)	41.7	30.7	21.5
σ_a (kg/cm^2)	2 117	1 555	1 382
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)	85	85	85
σ_{sa} (kg/cm^2)	2 340	2 340	2 340
τ_a (kg/cm^2)	8.2	8.2	8.2

20

A 1 - ROAD O. V.

(P - 1)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = 7.95 m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.407 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

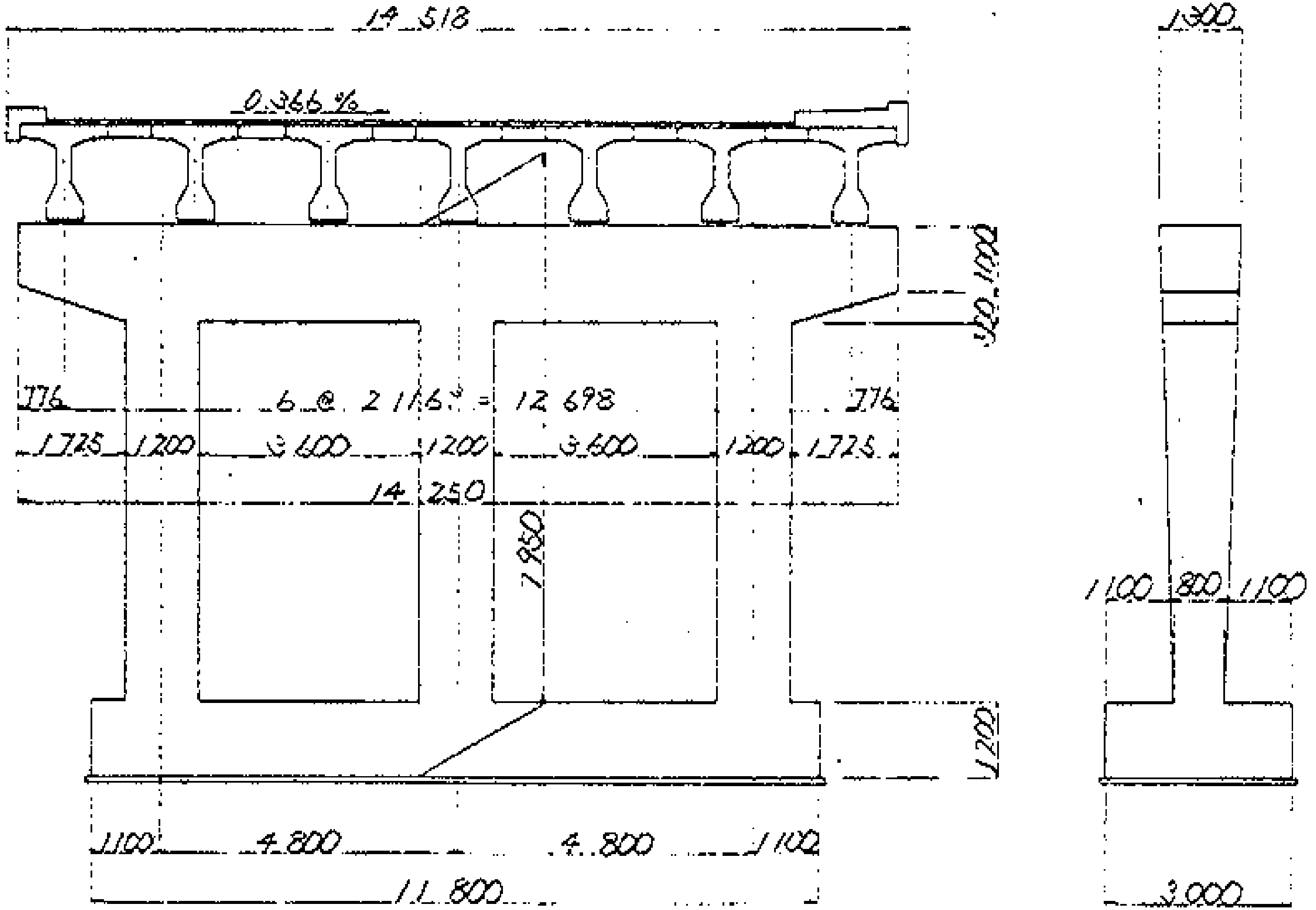
su = 4 180 kg/cm^2

permissible tensile stress

sa = 2 340 kg/cm^2

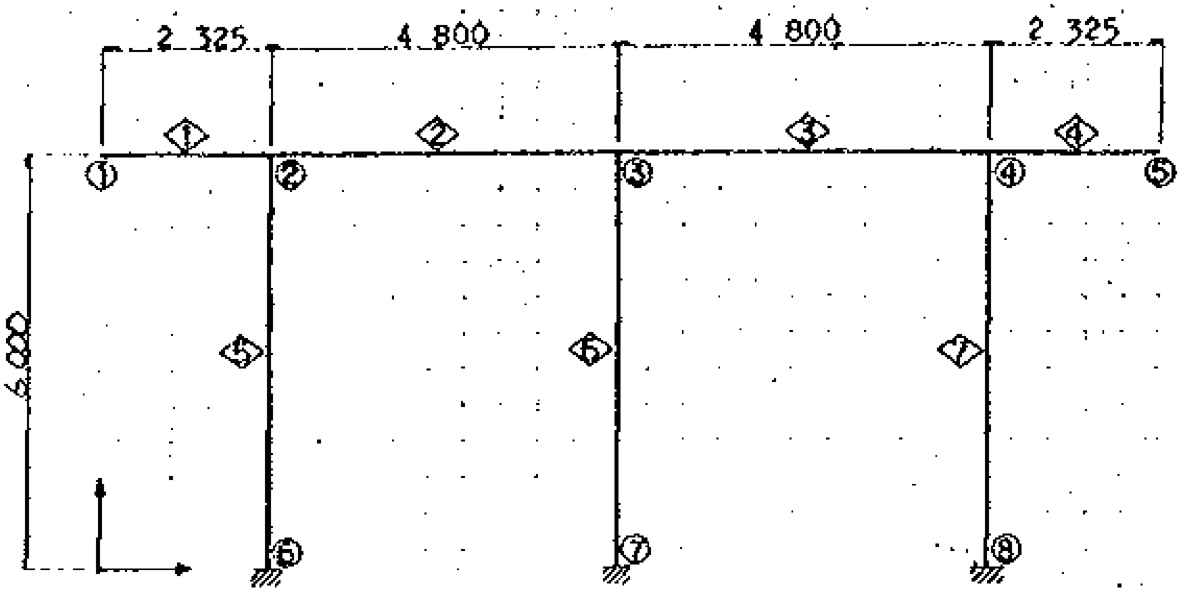
modular ratio n = 15

GENERAL DIMNSION



P1

Skeleton



○ ; joint number

◇ ; member number

Coordinates

(m)

	X	Y		X	Y
①	0.0	6.000	⑤	14.250	6.000
②	2.325	6.000	⑥	2.325	0.0
③	7.125	6.000	⑦	7.125	0.0
④	11.925	6.000	⑧	11.925	0.0

2.) 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 (m^2)	moment of inertia I (m^4)
<1>	1.245 x 1.25	1.5563	0.20264
<2>	1.245 x 1.50	1.8675	0.35016
<3>	1.245 x 1.50	1.8675	0.35016
<4>	1.245 x 1.25	1.5563	0.20264
<5>	1.023 x 1.20	1.2270	0.14724
<6>	1.023 x 1.20	1.2270	0.14724
<7>	1.023 x 1.20	1.2270	0.14724

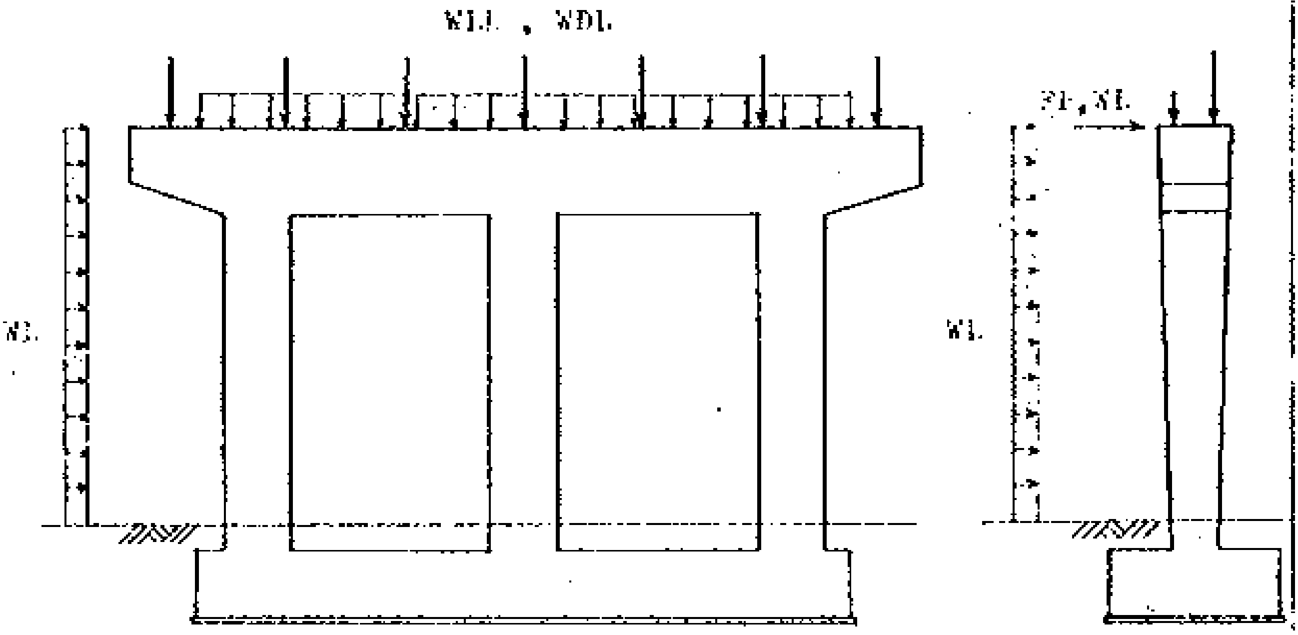
§ 3 CALCULATION OF LOADS

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	temperture change and drying schrinkage	FT & FS
8	wind	WL

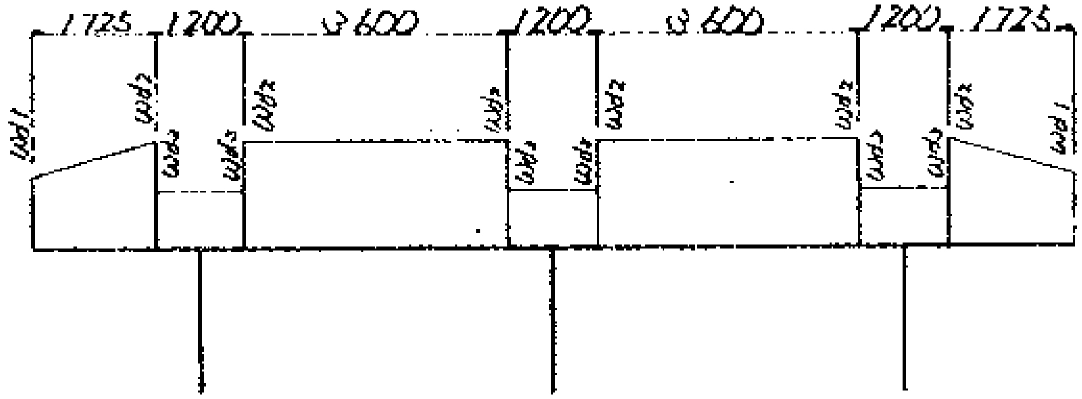
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



(3) Dead load

(a) beam



$Wd_1 = 1.245 \times 1.0 \times 2.407 = 3.00$ t/m

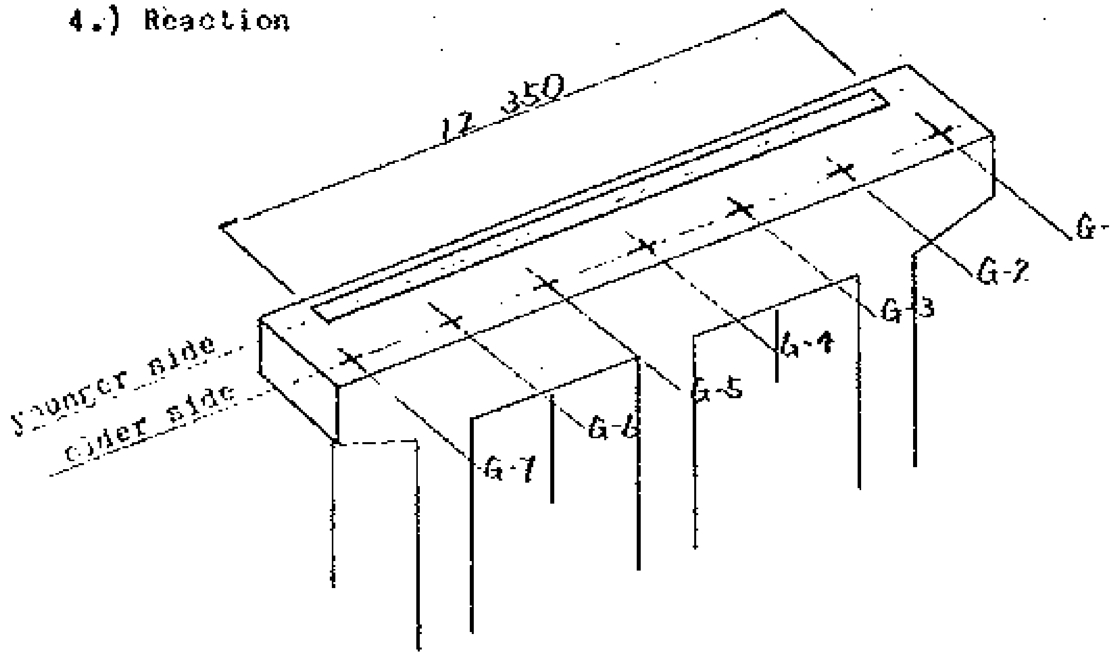
$Wd_2 = 1.245 \times 1.5 \times 2.407 = 4.50$ t/m

$Wd_3 = 1.273 \times 0.75 \times 2.407 = 2.30$ t/m

(b) column

$P_w = 1.023 \times 1.2 \times 2.407 = 2.95$ t/m

4.) Reaction



(unit ; ton)

			G-1	G-2	G-3	G-4	G-5	G-6	G-7
Dead load		younger	4.316 t/m						
		older	36.0	36.9	37.1	37.2	37.6	37.0	38.6
HA load	unif.	younger	2.445 t/m						
		older	19.0	19.8	17.6	17.0	17.2	16.7	12.3
		line	7.0	8.6	9.9	9.1	8.9	9.3	3.6
Crowd load		younger	0.170 t/m						
		older			0.1	0.5	1.2	1.6	5.4
HB load		(HB)	50.3	48.0	47.0	41.8	39.8	44.1	26.9
		HB	54.3	51.8	50.7	45.1	42.9	53.0	29.0
(HA/3)	unif.	younger	0.615 t/m						
		older	0.4	0.9	2.3	2.9	2.7	0.8	
		line	0.1	0.5	0.7	1.0	1.2	0.3	
Total (HA)			(2.615 t/m) 6.931 t/m						
			(26.0) 62.0	(28.4) 65.3	(27.6) 64.7	(26.6) 63.8	(27.3) 64.9	(27.8) 64.8	(21.3) 59.9
Total (HB)			(0.785 t/m) 5.101 t/m						
			(54.8) 90.8	(53.2) 90.1	(53.8) 90.9)	(49.5) 84.7	(48.0) 85.6	(55.7) 92.7	(54.4) 73.0

5.) Temperture change and Drying schrinkage

temperture change +12 ~ +20 deg.

drying schrinkage -20 deg.

6.) Wind Load (transverse)

(a) For superstructure

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.3$ m/sec. > 35 m/sec.

Unloaded

$P_t = q \cdot A \cdot C_d$

$q = 0.613 \times 59.3^2 \times 0.102 = 219.7$ kg/m^2

$A = 1.875 \times 13.6 = 25.5$ m^2

$C_d = 1.2$

$P_t = 219.7 \times 25.5 \times 1.2$

$= 6.7$ ton

Live loaded

$P_t = q \cdot A \cdot C_d$

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

$A = 4.080 \times 13.6 = 55.5$ m^2

$C_d = 1.4$

$P_t = 76.6 \times 55.5 \times 1.4$

$= 6.0$ ton

(b) For safety fences

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

$P_t = q \cdot A \cdot C_d$

$q = 219.7$ kg/m^2

$A = (0.15 \times 13.6 + 0.85 \times 0.05 \times \frac{13.6}{2}) \times 2 = 4.66$ m^2

$C_d = 1.1$

$P_t = 219.7 \times 4.66 \times 1.1 = 1.1$ ton

(c) For Pier

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

Unloaded

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9\ \mathrm{kg/m^2}$

$A1 = (1.30 + 1.19) \times 1.5 \times 1/2 = 1.87\ \mathrm{m^2}$

$A2 = (1.19 + 0.84) \times 4.75 \times 1/2 \times 3 = 14.46\ \mathrm{m^2}$

$Cd = 1.7$

$Pt = 219.9 \times (1.87 + 14.46) \times 1.7$

$= 6.1\ \mathrm{ton}$

Live loaded

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ \mathrm{kg/m^2}$

$A = A1 + A2 = 16.33\ \mathrm{m^2}$

$Cd = 1.7$

$Pt = 76.6 \times 16.33 \times 1.7 = 2.1\ \mathrm{ton}$

(d) Table of Wind load (transaverce)

(ton)

	unloaded	live loaded
super-structure	6.7	6.0
safety fences	1.1	–
Pier	6.1	2.1

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 × 6.7

= 1.7 ton

live loaded PLL = 0.5 Pt

Pt = 76.6 × (2.5 × 13.6) × 1.4

= 3.6 ton

PLL = 0.5 × 3.6 = 1.8 ton

PLS = 0.25 Pt

Pt = 76.6 × (4.08 − 2.5) × 13.6 × 1.2

= 2.0 ton

PLS = 0.25 × 2.0 = 0.5 ton

for safety fences

vertical member

PL = 0.8 Pt

Pt = 219.7 × (0.85 × 0.05 × 13.6 × 1/2 × 2) × 1.1

= 0.14 ton

PL = 0.8 × 0.14 ≑ 0.1 ton

longitudinal member

PL = 0.4 Pt

Pt = 219.7 × (0.15 × 13.6 × 2) × 1.1

= 0.99 ton

PL = 0.4 × 0.99 = 0.4 ton

for Pier

Vc = 39.3 m/sec.

unloaded

q = 219.9 kg/m^2

A1 = $(10.8 + 14.25) \times 1.5 \times 1/2 = 18.8\ m^2$

A2 = $1.2 \times 4.75 \times 3 = 17.1\ m^2$

Cd1 = 2.1

Cd2 = 2.0

Pt1 = $219.9 \times 18.8 \times 2.1 = 8.7^{ton}$

Pt2 = $219.9 \times 17.1 \times 2.0 = 7.5^{ton}$

live loaded

q = 76.6 kg/m^2

A1 = 18.8 m^2

A2 = 17.1 m^2

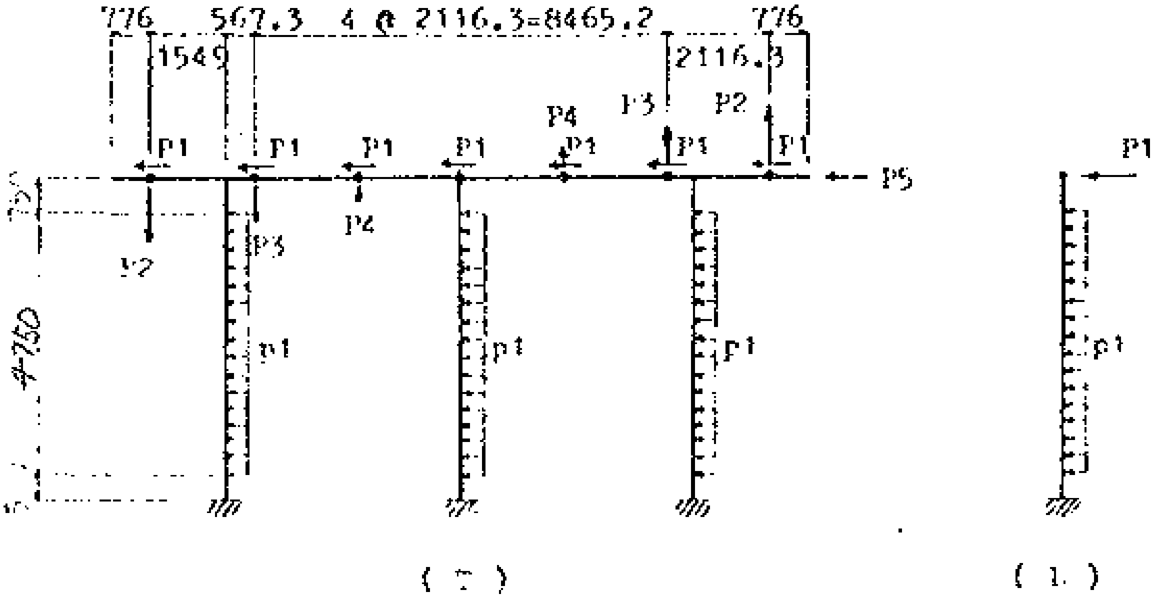
Cd1 = 2.1

Cd2 = 2.0

Pt1 = $76.6 \times 18.8 \times 2.1 = 3.0^{ton}$

Pt2 = $76.6 \times 17.1 \times 2.0 = 2.6^{ton}$

8.) Wind loading diagram



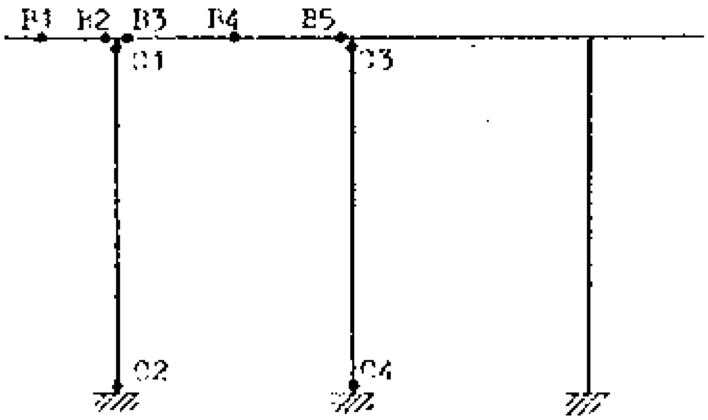
(T) (L)

		for place	unloaded	live loaded
(T)	P1	superstructure safety fences	1.11	0.86
	P2	superstructure safety fences	0.45	0.62
	P3	superstructure safety fences	0.30	0.41
	P4	superstructure safety fences	0.15	0.21
	P5	pier	0.70	0.24
	p1	pier	0.27	0.09
(L)	P1	superstructure safety fences pier	3.63	1.77
	p1	pier	0.37	0.13

(P ; ton , p ; t/m)

§ 4 ACTING FORCE TABLE

1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.971	-0.000	-64.590	1.00
	N max.	+	2	-0.971	1.110	-39.040	1.00
		-	2	-0.971	-0.700	2.590	1.00
	S max.	+					
		-	4	-0.777	-0.000	-74.712	1.25
B2	M max.	+					
		-	4	-123.979	-0.000	-84.530	1.25
	N max.	+	2	-70.044	1.110	-50.233	1.00
		-	2	-72.677	-1.810	51.933	1.00
	S max.	+					
		-	4	-123.979	-0.000	-84.530	1.25

[illegible]	[illegible] max.	+	3	19.901	32.460	50.115	1.00
		-	3	-157.410	-17.803	136.525	1.00
	[illegible] max.	+	3	-10.265	35.240	57.196	1.00
		-	3	-127.244	-20.583	129.444	1.00
	[illegible] max.	+	4	-155.954	-13.416	139.757	1.25
		-					
M	[illegible] max.	+	4	84.372	22.365	8.657	1.25
		-	4	-73.857	-9.615	33.287	1.25
	[illegible] max.	+	3	36.762	35.240	0.458	1.00
		-	3	33.597	-20.583	38.029	1.00
	[illegible] max.	+	4	18.998	-12.586	56.211	1.25
		-					
B5	[illegible] max.	+					
		-	3	-96.917	30.049	-91.836	1.00
	[illegible] max.	+	3	-65.648	35.240	-56.480	1.00
		-	3	-44.583	-20.583	-52.785	1.00
	[illegible] max.	+					
		-	3	-96.914	27.868	-92.177	1.00
	[illegible] max.	+					
		-					
	[illegible] max.	+					
		-					
	[illegible] max.	+					
		-					

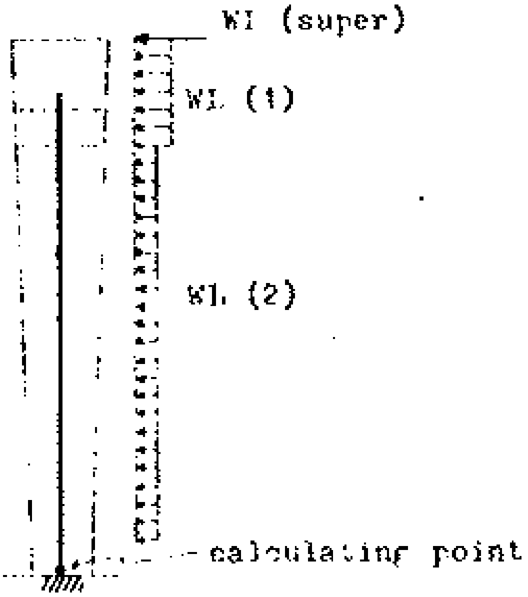
C1	N max.	+	3	57.897	-179.227	-20.583	1.00
		-	3	-101.828	-136.575	35.240	1.00
	M max.	+					
		-	1	31.975	-223.789	-13.416	1.25
	S max.	+	3	-101.828	-136.575	35.240	1.00
		-					
C2	N max.	+	3	107.613	-157.275	35.240	1.00
		-	2	-70.546	-158.980	-21.315	1.00
	M max.	+					
		-	1	-48.523	-237.949	-13.416	1.25
	S max.	+					
		-	2.	-113.624	-119.169	-35.454	1.00
C3	N max.	+	6	23.290	-146.677	-6.767	1.30
		-					
	M max.	+					
		-	3	-0.474	-242.395	0.151	1.00
	S max.	+					
		-	6	23.290	-146.677	-6.767	1.30
C4	N max.	+					
		-	6	-18.258	-160.292	-7.096	1.30
	M max.	+					
		-	3	0.430	-260.095	0.151	1.00
	S max.	+					
		-	6	-18.258	-160.292	-7.096	1.30

These value is divided by "i".

i ; coefficient of increase

in basic stresses.

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	29.0	193.1	4.30	1.00
case-8	43.1	198.0	6.38	1.25
case-9	42.5	193.1	6.73	1.15
case-10	36.6	198.0	9.01	1.30
case-11	28.9	140.2	6.13	1.05

CASE 7

	N	H	y	M
WDL	104.6			
WLL(A)	52.9			
FB (A)		4.30	6.75	29.0
W	35.6			
TOTAL	193.1	4.30		29.0

M = 29.0 t·m

N = 193.1 ton

H = 4.30 ton

CASE 8

	N	H	y	M
WDL	104.6			
WLL (B)	57.8			
FB (B)		6.38	6.75	43.1
W	35.6			
TOTAL	198.0	6.38		43.1

M = 43.1 t·m

N = 198.0 ton

H = 6.38 ton

CASE 9

	N	H	y	M
WDL	104.6			
WLL, EB(A)	52.9	4.30	6.75	29.0
W	35.6			
WL (super)		0.77	6.75	5.2
WL (1)		1.00	6.00	6.0
WL (2)		0.86	2.63	2.3
TOTAL	193.1	6.93		42.5

M = 42.5 t·m

N = 193.1 ton

H = 6.93 ton

CASE 10

	N	H	y	M
WDL	104.6			
WLL, EB(B)	57.8	6.38	6.75	43.1
W	35.6			
WL (super)		0.77	6.75	5.2
WL (1)		1.00	6.00	6.0
WL (2)		0.86	2.63	2.3
TOTAL	198.0	9.01		56.6

M = 56.6 t·m

N = 198.0 ton

H = 9.01 ton

CASE 11

	N	H	y	M
WDL	104.6			
W	35.6			
WL (super)		0.73	6.75	4.9
WL (1)		2.90	6.00	17.4
WL (2)		2.50	2.63	6.6
TOTAL	140.2	6.13		28.9

M = 28.9 t·m

N = 140.2 ton

H = 6.13 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1 case 4	B2, B3 case 3	B4 case 4
M (tm)	-0.971	-157.410	105.465
N (ton)	-0.000	-17.803	27.956
Q:shear force (ton)	-93.390	136.526	10.821
b (cm)	120	120	120
d (cm)	107	140	140
d' (cm)	10	10	10
As (cm²)	25.1 (D20 15cm pitch)	64.3 (D32 15cm Pitch)	39.3 (D25 15cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.029	0.057	0.035
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)	0.07	6.69	4.48
Q/b·d (kg/cm²)	7.27	8.13	0.69
C	10.1	7.25	9.35
S	37.1	19.4	31.0
	1.07	1.10	1.08
σ_c (kg/cm²)	0.7	52	42
σ_s (kg/cm²)	39	1,947	2,083
τ (kg/cm²)	7.78	8.94 ※	0.7
σ_{ca} (kg/cm²)	106	85	106
σ_{sa} (kg/cm²)	2,925	2,340	2,925
τ_a (kg/cm²)	10.3	8.2	10.3

※ stirrups Req. Av = $\frac{\tau \times b \times a}{\sigma_{sa}} = \frac{8.94 \times 120 \times 12.5}{1780} = 7.53$ cm²

Used As = φ16コ×2 × 4 × 2.01 = 8.04 cm² > Req Av = 7.53 cm²

	B 5		
	case 3	case	case
M (tm)	-96.917		
N (ton)	30.049		
S;shear force (ton)	-91.836		
b (cm)	120		
d (cm)	140		
d' (cm)	10		
As (cm^2)	39.3 (D25 13 piece)		
d'/d			
e=(M/N)+u (cm)			
e/d			
n	15	15	15
np=n·As/b·d			
M'=M+N·u (tm)			
M'/(b·d^2) ([illegible])	9.12		
S/b·d (kg/cm^2)	5.47		
C	9.35		
S	31.0		
	1.08		
σc (kg/cm^2)	39		
σs (kg/cm^2)	1916		
τ (kg/cm^2)	5.90		
σca (kg/cm^2)	85		
σsa (kg/cm^2)	2340		
τa (kg/cm^2)	8.2		

P 1

2.) Column section

	case 2	case 10	case
M (tm)	-113.524	56.6	
N (ton)	-119.169	198.0	
Q;shear force (ton)	35.457	9.01	
b (cm)	80	120	
d (cm)	110	70	
d' (cm)	10	10	
As (cm^2)	39.3 cm^2 8-D25	39.3 cm^2 8-D25	
d'/d	0.09	0.14	
f=(M/N)+u (cm)	145.3	58.6	
f/d	1.32	0.84	
n	15	15	15
nP=n·As/b·d	0.067	0.07	
M'=M+N·u (tm)	173.1	116.0	
$M'/(b \cdot d^2)$ $(^t/_{m^2})$	17.88	19.73	
Q/b·d $(^{kg}/cm^2)$	3.69	0.94	
C	4.38	3.28	
S	6.00	1.40	
σ_c $(^{kg}/cm^2)$	78	65	
σ_s $(^{kg}/cm^2)$	1609	414	
τ $(^{kg}/cm^2)$	3.7	0.9	
σ_{ca} $(^{kg}/cm^2)$	85	111	
σ_{sa} $(^{kg}/cm^2)$	2340	3042	
τ_a $(^{kg}/cm^2)$	8.2	10.7	

Vol. 3 CORONANDEL OV. BR. P_1, P_2, P_3 PIERS

PIER IN COROMANDEL O.V.

(P - 2)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = 8.28 ^{m}

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.407 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

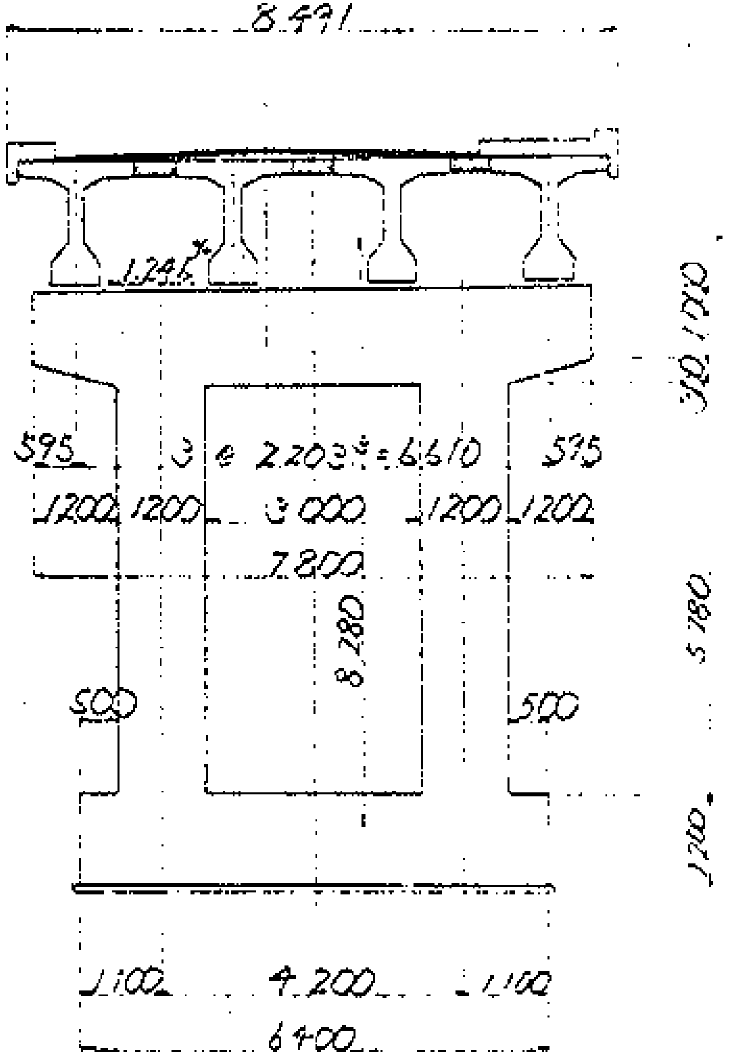
su = 4 180 kg/cm^2

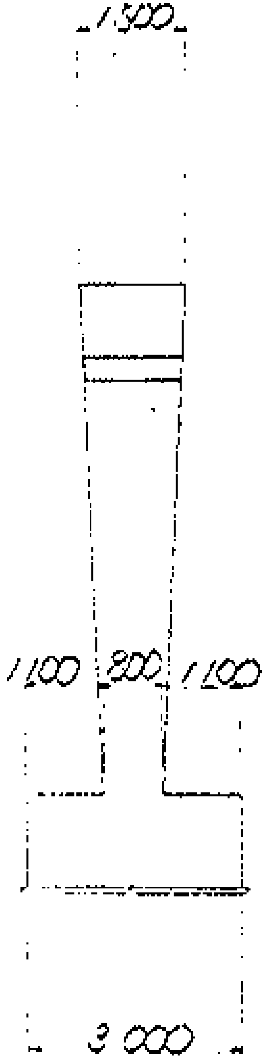
permissible tensile stress

sa = 2 340 kg/cm^2

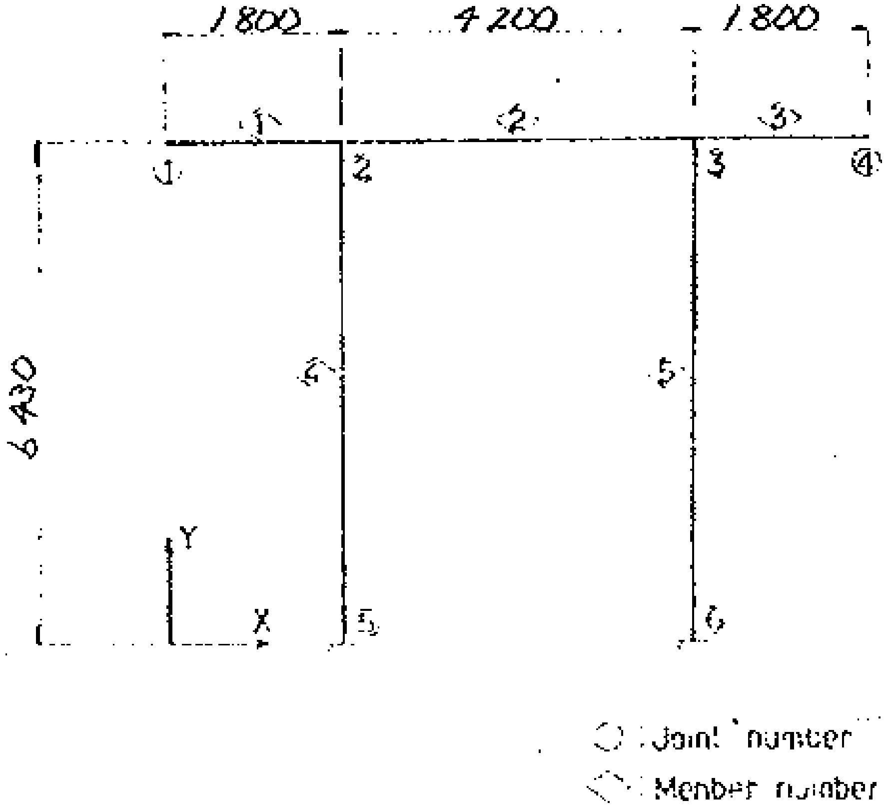
modular ratio n = 15

32 GENERAL DIMENCION





Skeleton



Coordinates

(m)

	X	Y		X	Y
①	0.0	6.430	4	7.800	6.430
2	1.800	6.430	5	1.800	0.0
3	6.000	6.430	6	6.000	0.0

Modulus of elasticity of concrete

$E_c = 26 \text{ kN/mm}^2$

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

member	section B x H	area of section A (m^2)	moment of inertia I (m^4)
<1>	1.45 × 1.15	1.668	0.18377
<2>	1.45 × 1.50	2.175	0.40781
<3>	1.45 × 1.15	1.668	0.18377
<4>	1.12 × 1.20	1.344	0.16128
<5>	1.12 × 1.20	1.344	0.16128

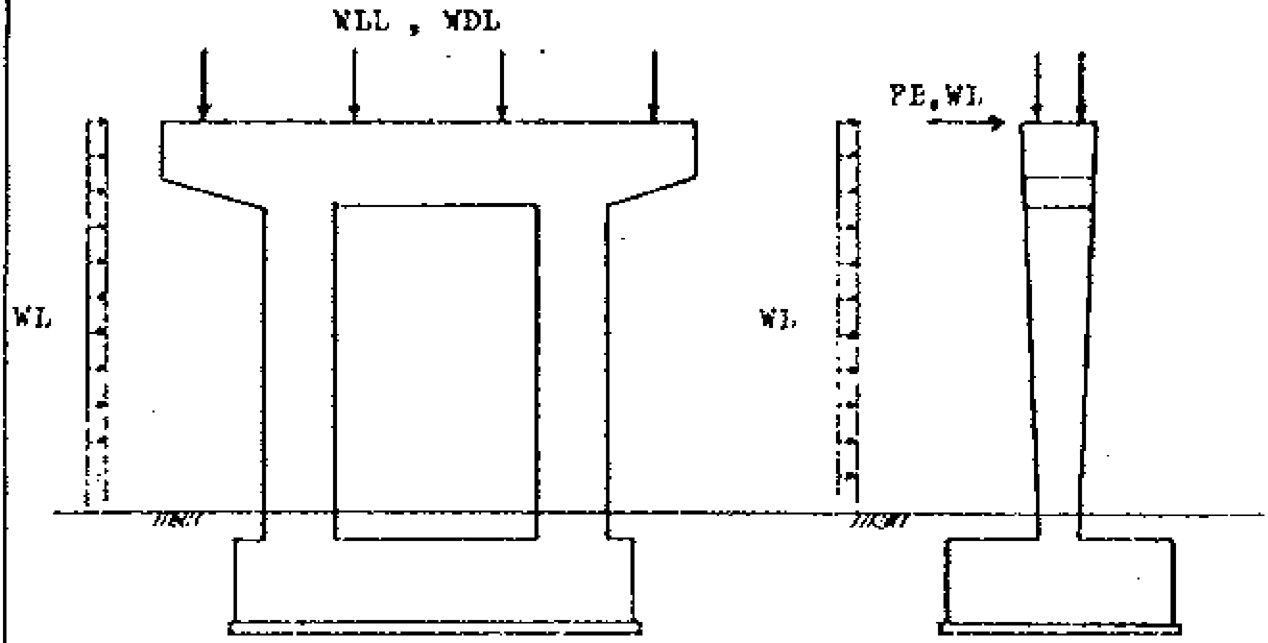
§ 3 CALCULATION OF LOADS

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	temperture charge and drying schrinkage	FT & FS
8	wind	WL

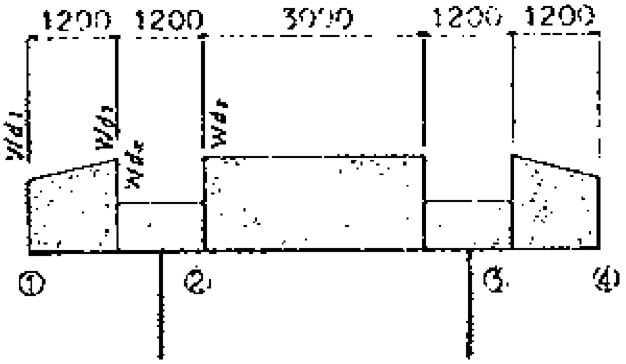
case	load combination	increase in basic stresses
1	1, 6, 7,	1.00
2	1, 6, 7, 8,	1.00
3	1, 2, 6, 7,	1.00
4	1, 3, 6, 7,	1.25
5	1, 2, 6, 7, 8,	1.15
6	1, 3, 6, 7, 8,	1.30
7	1, 2, 4, 6, 7,	1.00
8	1, 3, 5, 6, 7,	1.25
9	1, 2, 4, 6, 7, 8,	1.15
10	1, 3, 5, 6, 7, 8,	1.30
11	1, 6, 7, 8,	1.00

2.) Loading diagram



3.) Dead load

(a) beam



Wd1 = 1.45 x 1.0 x 2.407 = 3.49 t/m

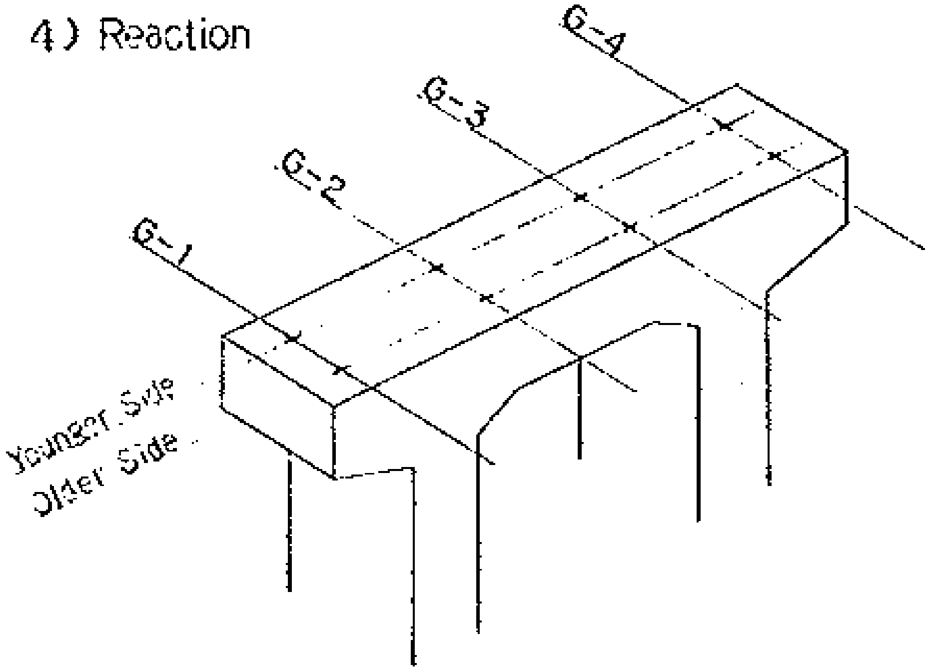
Wd2 = 1.44 x 1.3 x 2.407 = 4.51 t/m

Wd3 = 1.47 x 0.65 x 2.407 = 2.30 t/m

(b) column

Pw = 1.12 x 1.2 x 2.407 = 3.24 t/m

4) Reaction



(ton)

		G-1	G-2	G-3	G-4	G-5
Dead load	Younger	46.2	49.0	48.5	51.8	
	Older	46.2	49.0	48.5	51.8	
HA load uniform	Younger	22.1	23.9	22.6	12.1	
	Older	22.1	23.9	22.6	12.1	
	line	6.4	8.3	9.3	2.9	
Crowd load	Younger		0.9	1.5	9.0	
	Older		0.9	1.5	9.0	
HB load	(HB)					
	HB					
$\frac{1}{3}$HA) uniform	Younger					
	Older					
	line					
Total (HA)		(30.6) 143.0	(37.9) 155.9	(37.5) 134.5	(43.1) 148.7	
Total (HB)						

5.) Temperture change and Drying schrinkage

temperture change -12 ~ +20 deg.

drying schrinkage -20 deg.

6.) Wind Load (transverse)

(a) For superstructure

$V_c = 38 \times 1.0 \times 1.0 \times 1.62 = 61.6$ m/sec. > 35 m/sec.

Unloaded

$P_t = q \cdot A \cdot C_d$

$q = 0.613 \times 61.6^2 \times 0.102 = 237.0$ kg/m²

$A = 2.075 \times 28.5 = 59.1$ m²

$C_d = 1.4$

$P_t = 237.0 \times 59.1 \times 1.4$

$= 19.6$ ton

Live loaded

$P_t = q \cdot A \cdot C_d$

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

$A = 4.350 \times 28.5 = 124.0$ m²

$C_d = 1.6$

$P_t = 76.6 \times 124 \times 1.6$

$= 15.2$ ton

(b) For safety fences

$V_c = 38 \times 1.0 \times 1.0 \times 1.62 = 61.6$ m/sec.

$P_t = q \cdot A \cdot C_d$

$q = 237.0$ kg/m²

$A = (0.150 \times 28.5 + 0.850 \times 0.05 \times 28.5 \times 1/2) \times 2 = 9.76$ m²

$C_d = 1.1$

$P_t = 237.0 \times 9.76 \times 1.1 = 2.5$ ton

(c) For Pier

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

Unloaded

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

A1 = (1.50 + 1.37) × 1.3 × 1/2 ≒ 1.9 m^2

A2 = (1.37 + 0.80) × 5.78 × 1/2 × 2 ≒ 12.5 m^2

Cd = 1.7

Pt = 219.9 × (1.9 + 12.5) × 1.7

= 5.4 ton

Live loaded

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

A = A1 + A2 = 14.4 m^2

Cd = 1.7

Pt = 76.6 × 14.4 × 1.7 ≒ 1.9 ton

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	12.6	13.2
safety fences	2.5	
Pier	5.4	1.9

7.) Longitudinal force

(a) due to breaking

HA ⟶ 25.8 ton

RB ⟶ 38.3 ton

(b) due to Wind

for superstructure

unloaded PLS = 0.25 Pt = 0.25×19.6

$= 4.9^{ton}$

live loaded PLL = 0.5 Pt

Pt = $76.6 \times 2.5 \times 28.5 \times 1.42$

$= 7.75^{ton}$

PLL = 0.88^{ton}

PLS = 0.25 Pt

Pt = $76.6 \times (4.35 - 2.5) \times 28.5 \times 1.40$

$= 3.7^{ton}$

PLS = 1.4^{ton}

for safety fences

vertical member

PL = 0.8 Pt

Pt = $237.0 \times (0.85 \times 0.05 \times \frac{28.5}{2} \times 2) \times 1.1$

$= 0.3^{ton}$

PL = 0.3^{ton}

longitudinal member

PL = 0.4 Pt

Pt = $237.0 \times (0.13 \times 28.5 \times 2) \times 1.1$

$= 2.2^{ton}$

PL = 0.9^{ton}

for Pier

Vc = 59.3 m/sec.

unloaded

$q = 219.9 \ \text{kg/m}^2$

$A1 = (5.4 + 7.8) \times 1.3 \times 1/2 = 8.6 \ \text{m}^2$

$A2 = 1.2 \times 5.78 \times 2 = 13.9 \ \text{m}^2$

$Cd1 = 2.1$

$Cd2 = 2.0$

$Pt1 = 219.9 \times 8.6 \times 2.1 = 4.0^{\text{ton}}$

$Pt2 = 219.9 \times 13.9 \times 2.0 = 6.1^{\text{ton}}$

live loaded

$q = 76.6 \ \text{kg/m}^2$

$A1 = 8.6 \ \text{m}^2$

$A2 = 13.9 \ \text{m}^2$

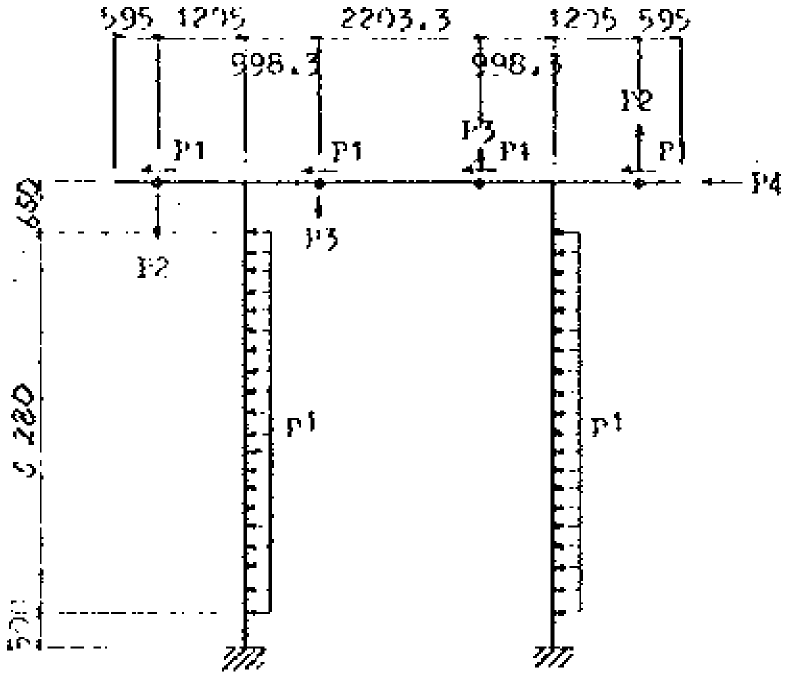
$Cd1 = 2.1$

$Cd2 = 2.0$

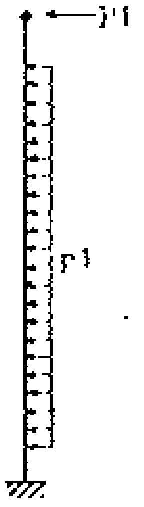
$Pt1 = 76.6 \times 8.6 \times 2.1 = 1.4^{\text{ton}}$

$Pt2 = 76.6 \times 13.9 \times 2.0 = 2.1^{\text{ton}}$

8.) Wind loading diagram



(T)



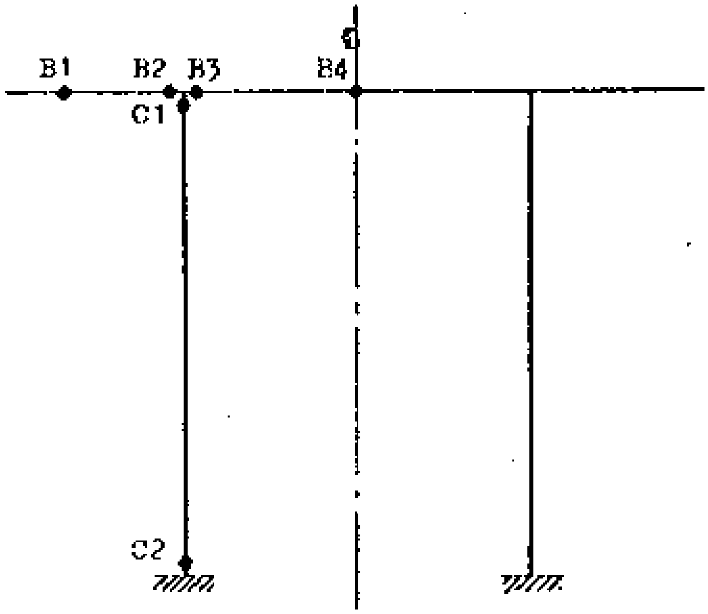
(L)

		for place	unloaded	live loaded
(T)	P1	superstructure safety fences	5.53	3.80
	P2	superstructure safety fences	5.60	5.85
	P3	superstructure safety fences	1.87	1.95
	P4	pier	0.71	0.25
	p1	pier	0.44	0.15
(L)	P1	superstructure safety fences pier	5.05	3.34
	p1	pier	0.58	0.20

(P;ton , p;t/m)

§ 4 ACTING FORCE TABLE

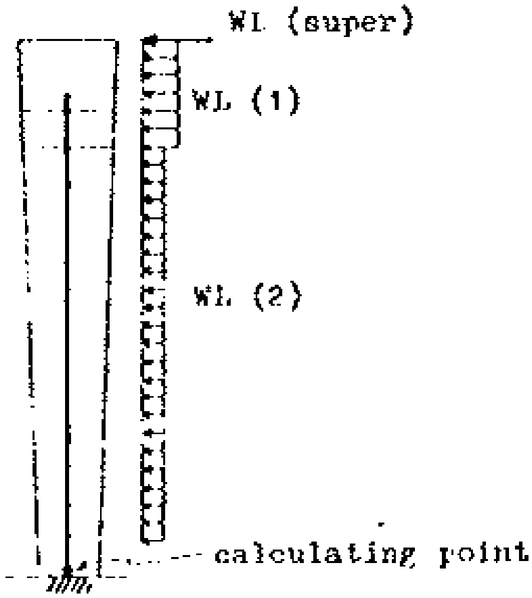
1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.648	-0.000	-145.227	1.0
	N max.	+	2	-0.648	1.530	-100.227	1.0
		-	2	-0.648	-0.710	2.227	1.0
	S max.	+					
		-	3	0.648	-0.000	-145.227	1.0
B2	M max.	+					
		-	3	-185.235	-0.000	154.880	1.0
	N max.	+	2	-124.142	3.530	-104.180	1.0
		-	2	-124.142	-6.240	104.180	1.0
	S max.	+	3	-185.235	-0.000	154.880	1.0
		-					

B3	M max.	+					
		-	3	-192.874	-3.072	-177.358	1.0
	N max.	+	5	76.086	20.181	-79.528	1.15
		-	5	-146.841	-9.691	153.248	1.15
	S max.	+					
		-	3	-139.564	13.651	-177.358	1.0
B4	M max.	+	3	23.067	11.503	-3.197	1.0
		-	3	-61.024	0.809	-1.696	1.0
	N max.	+	3	-7.714	19.333	-1.696	1.0
		-	3	19.737	-7.220	-3.197	1.0
	S max.	+	5	27.400	13.503	32.248	1.15
		-					
C1	M max.	+	2	83.935	-191.287	-26.391	1.0
		-					
	N max.	+					
		-	3	45.671	-332.238	-13.651	1.0
	S max.	+					
		-	2	83.935	-191.287	-26.391	1.0
C2	M max.	+					
		-	2	-93.037	-212.121	-28.715	1.0
	N max.	+					
		-	3	-42.107	-353.072	-13.651	1.0
	S max.	+					
		-	2	-93.037	-212.121	-28.715	1.0

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	45.7	336.3	6.45	1.00
case-8				~~1.25~~
case-9	71.9	336.3	10.84	1.15
case-10				~~1.30~~
case-11				1.00

CASE 7

	N	H	Y	M
WDL	195.5			
WLL(A)	105.6			
EB (A)		6.45	7.08	45.7
W	35.2			
TOTAL	336.3	6.45		45.7

M = 43.7 t·m

N = 336.3 ton

H = 6.45 ton

CASE 9

	N	H	Y	M
WDL	195.5			
WLL, EB (A)	105.6	6.45	7.08	43.7
W	35.2			
WL (Super)		2.69	7.08	18.7
WL (1)		0.70	6.43	4.5
WL (2)		1.05	2.89	3.0
TOTAL	336.3	10.84		71.9

M' = 71.9 t·m

N = 336.3 ton

H = 10.84 ton

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1	B2, B3	B4
	case 3	case 3	case 3
M (tm)	-0.648	-192.874	73.067
N (ton)	-0.000	-5.072	11.503
Q; shear force (ton)	-145.227	-177.358	-3.197
b (cm)	144	145	145
d (cm)	105	120	120
d' (cm)	10	10	10
As (cm^2)		80.4 cm^2 (10-D32 13cm pitch)	44.2 cm^2 (9-D25 15cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d		0.069	0.038
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)		9.24	3.50
Q/b·d (kg/cm^2)	9.60	10.19	0.18
C		7.22	9.03
S		16.1	28.5
	1.11	1.11	1.08
σ_c (kg/cm^2)		67	32
σ_s (kg/cm^2)		2231	1476
τ (kg/cm^2)	10.66	11.31 ※	0.2
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

※ Req. $A_v = \frac{\tau \cdot b \cdot a}{\sigma_{sa}} = \frac{11.31 \times 145 \times 12.5}{1780} = 11.52^{cm^2}$

$a = 12.5^{cm}$ Used A_s = Φ10 ▢ × 2 = 4 × 3.14 = 12.56 $^{cm^2}$ > 11.52$^{cm^2}$

2.) Column section

	at C_1 case	at C_2 case 2	at C_2 case 9
M (tm)		93.037	71.9
N (ton)		212.121	336.3
Q;shear force (ton)		28.715	10.84
b (cm)		80	120
d (cm)		110	70
d' (cm)		10	10
As (cm^2)	38.4 cm^2 6-D32 } Alternate 5-D16 }	40.2 cm^2 5-D32	45.6 cm^2 2-D32 (edge) 6-D25 (inter)
d'/d		0.091	0.14
$f=(M/N)+u$ (cm)		93.9	51.4
f/d		0.85	0.73
n	15	15	15
$nP=n\cdot As/b\cdot d$		0.069	0.081
$M'=M+N\cdot u$ (tm)		199.1	172.8
$M'/(b\cdot d^2)$ (Kd)		20.6	22.4
$Q/b\cdot d$ (kg/cm^2)		2.99	1.13
C		3.15	2.8
S		1.5	0.3
σ_c (kg/cm^2)		65	82
σ_s (kg/cm^2)		464	132
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)		85	98
σ_{sa} (kg/cm^2)		2340	2691
τ_a (kg/cm^2)		8.2	9.4

§6. Calculation of stability

Pier self weight

Cantilever beam	½×(1.0+1.3)×1.20 × 1.45 × 2.407 × 2	9.63
beam	1.3 × 5.4 × 1.45 × 2.407	24.50
column	5.78 × 1.12 × 1.2 × 2.407 × 2	37.40
Footing	6.4 × 3.0 × 1.2 × 2.407	55.45
Σ		126.98 ton

Reaction due to superstructure

	P 1	P 2	P 3
Dead load	31.4 175.5	391.0	175.5 42.0
HA Live load	21.1 + 77.5 + 22.9	177.4	77.5 + 17.1 + 22.9
HB Live load	----	----	----
Crowd load	2.1 10.6	21.2	10.6 2.5
Total (HA)	360.5	589.6	369.6
Total (HB)	----	----	----

Longitudinal direction

a) due to braking

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

" HB ; $FB =$

b) due to wind

- for superstructure

Unloaded $P_{LS} = 4.7$

Live loaded $P_{LL} = 2.9$

$P_{LS} = 1.9$

- for safety fences $P_{L1} = 0.3$

$P_{L2} = 0.9$

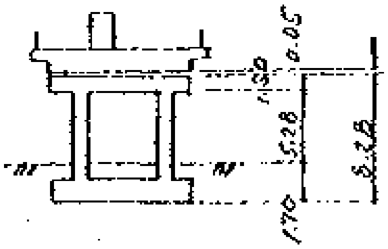
- for pier

Unloaded $P_{L1} = 4.0$

$P_{L2} = 6.1$

Live loaded $P_{L-1} = 1.9$

$P_{L-2} = 2.1$



Transverse direction

a) due to wind

- for superstructure

Unloaded Pt = 17.6

Live loaded Pt = 15.2

- for safety fences

Pt = 2.5

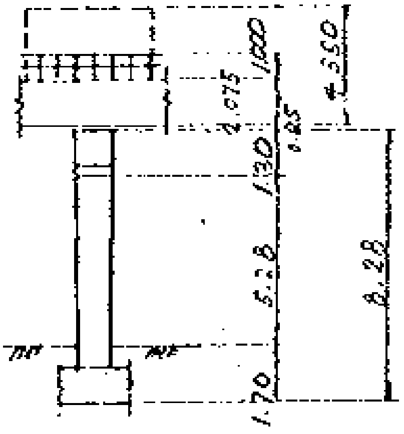
- for pier

Unloaded Pt-1 = 0.7

Pt-2 = 4.7

Live loaded Pt-1 = 6.2

Pt-2 = 1.6



		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	872.0					—
	WLL (HA)	198.6			12.9	8.33	107.46
	WLL (HB)						—
	Self weight	127.0					
Transverse direction	WL (Unloaded)						
	for superstruct.				19.6	9.368	183.61
	for safety fence				2.5	10.905	27.26
	for pier (1)				0.7	7.63	5.34
	(2)				4.7	4.34	20.40
					Σ 27.5		Σ 236.61
	WL (live loaded)						
	for superstruct (1)				15.2	10.505	157.68
	〃 (2)						
	for pier (1)				0.2	7.63	1.53
	〃 (2)				1.6	4.34	6.94
					Σ 17.5		Σ 166.15
Longitudinal direction	WL (Unloaded)						
	for superstruct.				4.9	8.33	40.82
	for safety fence				1.2	8.33	10.00
	for pier (1)				4.0	7.63	30.52
	(2)				6.1	4.34	26.47
					Σ 16.2		Σ 107.81
	WL (live loaded)						
	for superstruct. (1)				3.9	8.33	32.49
	〃 (2)				1.4	8.33	11.66
	for pier (1)				1.4	7.63	10.68
	(2)				2.1	4.34	9.11
					Σ 8.8		Σ 63.94

Longitudinal direction

Case No	Loading combination	N (t)	H (t)	M (t)
1	Dead load only	518.0		
2	Dead load + wind	518.0	16.2	107.81
3	HA Loading	716.6	12.9	107.26
4	HB Loading			
5	HA Loading + wind	716.6	21.7	171.90
6	HB Loading + wind			

Calculation of Stability

- eccentric $e = M/N$ $B/6 = 0.5$
- 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 $F = \frac{N \cdot 0.6}{H}$

Case No.	B/6 (m)	eccentric e (m)	Soil reaction q max (t/m²)	Soil reaction q min (t/m²)	qa t/m²	Sliding F	Sliding Fa
1		0	27.0	27.0	60		
2	0.500	0.208	38.2	15.7	60		
3	0.500	0.150	48.5	26.1	60		
4							
5	0.500	0.239	55.2	19.5	69	19	1.5
6							

Transverse direction.

Case No	loading combination	N (t)	H (t)	M (t)
~~1~~	~~Dead load only~~			
2	Dead load + wind	518.0	27.5	236.61
~~3~~	~~HA loading~~			
~~4~~	~~HB loading~~			
5	HA loading + wind	716.6	17.0	168.15
6	HB loading + wind			

Calculation of Stability

- eccentric $e = M/N$ $B/6 =$
- 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) $e > B/6$

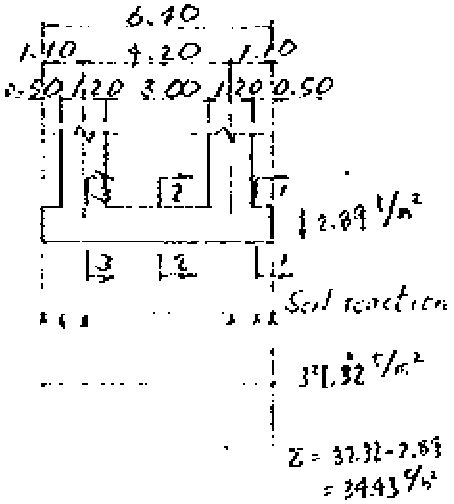
$$q_{max} = \frac{2 \cdot N}{B \cdot L}$$

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

Case No.	B/6 (m)	eccentric	Soil reaction		q_a t/m²	Sliding	
		e (m)	q_{max} (t/m²)	q_{min} (t/m²)		F	Fa
~~1~~							
2	1.067	0.457	28.5	15.4	60	11.3	1.5
~~3~~							
~~4~~							
5	1.067	0.235	45.5	29.1	69		
6							

§ 7 Calculation of footing section

1 Transverse direction



Sect 1.

$$M = 34.42 \times 0.5^2 \times \frac{1}{2} \times 3.0 = 12.91^{\,t\cdot m}$$

$$M' = 12.91 \times \frac{1}{3.0} = 4.30^{\,t\cdot m}$$

Sect 2.

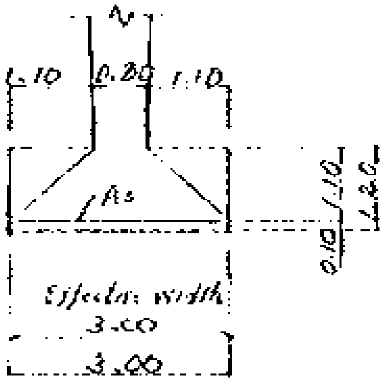
$$M = 34.43 \times 3.0^2 \times \frac{1}{8} \times 3.0 = 116.20^{\,t\cdot m}$$

$$M' = 116.20 \times \frac{1}{3.0} = 38.73^{\,t\cdot m}$$

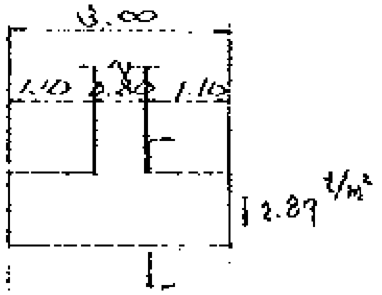
Sect. 3

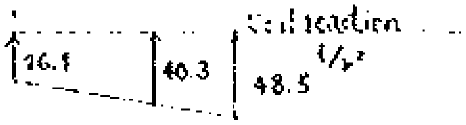
$$M = 34.43 \times 4.2^2 \times \frac{1}{12} \times 3.0 = 151.84^{\,t\cdot m}$$

$$M' = 151.84 \times \frac{1}{3.0} = 50.61^{\,t\cdot m}$$



2 Longitudinal direction





$$S = 2.89 \times 1.10 \times \overset{20.35}{6.40}$$

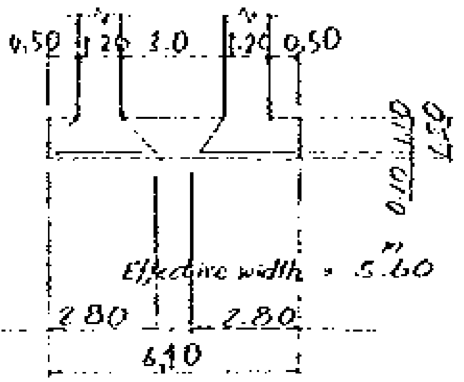
$$- \frac{1}{2} \times (48.5 + 40.3) \times 1.10 \times \overset{-312.58}{6.40} = -292.23^{t}$$

$$H = 20.35 \times \frac{1}{2} \times \overset{11.19}{1.10}$$

$$-312.58 \times \frac{1}{3} \times 1.10 \times \overset{-177.21}{\frac{48.5 \times 2 + 40.3}{48.5 + 40.3}} = -166.02^{t\cdot m}$$

$$S' = -292.23 \times {}^{1}/_{5.6} = -52.18^{t}$$

$$H' = -161.76 \times {}^{1}/_{5.6} = -29.65^{t\cdot m}$$



	Transverse direction		Longitudinal direction
	case Top	case bottom	case bottom
M (tm)	38.73	50.61	29.65
N (ton)			
Q; shear force (ton)			
b (cm)	100	100	100
d (cm)	110	110	110
d' (cm)	10	10	10
As (cm^2)	Φ16 250 [illegible] Φ20 250 [illegible] 20.6	Φ20 -125 25.12	[illegible] = 16.5 cm Φ16 - 125 = 16.08
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.028	0.034	0.022
M'=M+N·u (tm)			
M'/($b \cdot d^2$) (kg/cm²)	3.20	4.18	2.45
Q/b·d (kg/cm^2)			
C	10.2	9.16	11.8
S	38.4	32.0	48.5
z			
σ_c (kg/cm^2)	32.6	39.6	27.7
σ_s (kg/cm^2)	1849	2008	1783
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)	85	85	85
σ_{sa} (kg/cm^2)	2340	2340	1340
τ_a (kg/cm^2)			

PIER IN COROMANDEL O/V.

(P - 1)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = 8.07 m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.407 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

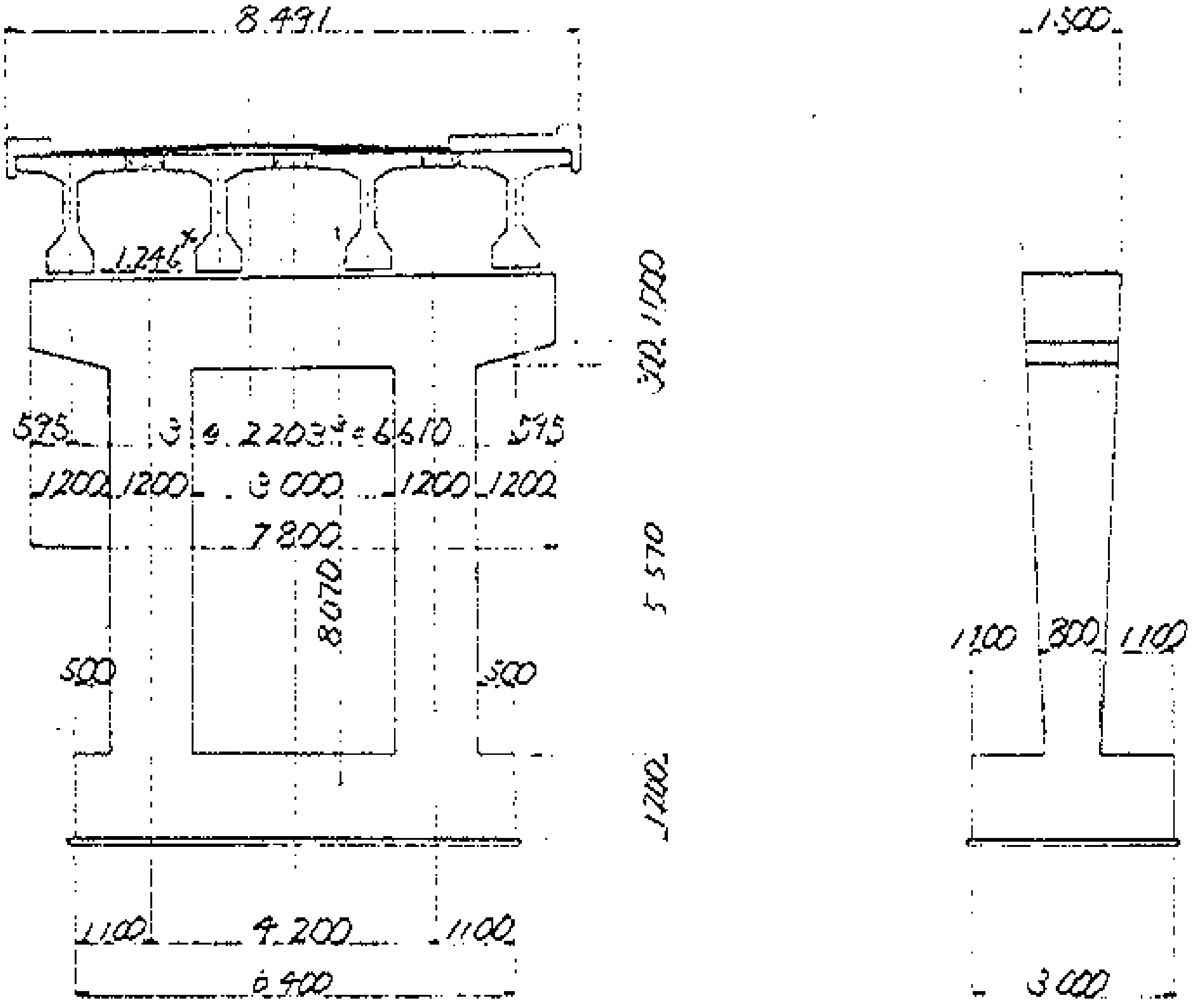
su = 4 180 kg/cm^2

permissible tensile stress

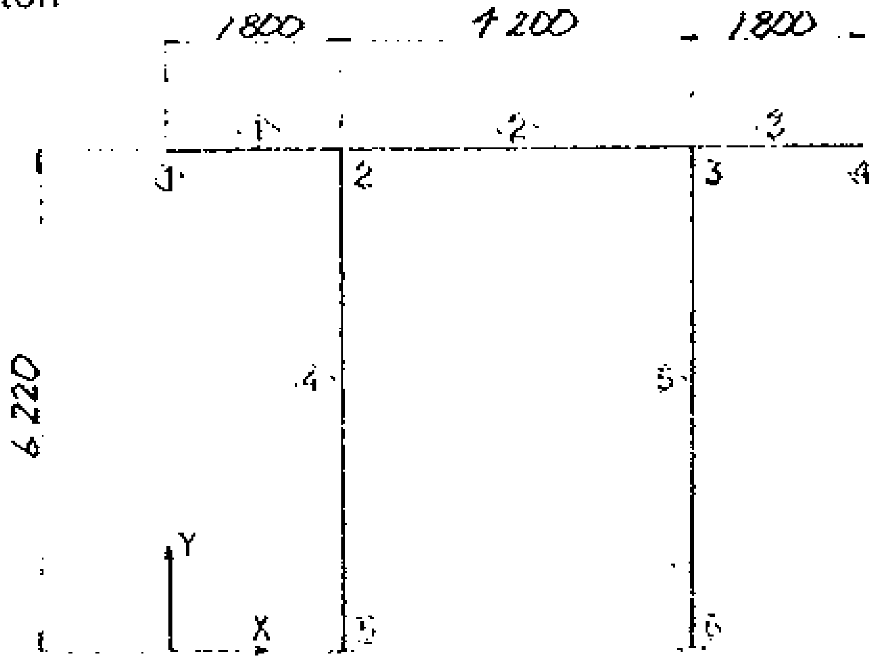
sa = 2 340 kg/cm^2

modular ratio n = 15

§2 GENERAL DIMENSION



Skeleton



○ : Joint number
◇ : Member number

Coordinates

(m)

	X	Y		X	Y
1	0.0	6.220	4	7.800	6.220
2	1.800	6.220	5	1.800	0.0
3	6.000	6.220	6	6.000	0.0

Modulus of elasticity of concrete

$E_c = 26 \ kN/mm^2$

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

member	section B × H	area of section A (m²)	moment of inertia I (m⁴)
<1>	1.45 × 1.15	1.668	0.18377
<2>	1.44 × 1.50	2.160	0.40500
<3>	1.45 1.15	1.668	0.18377
<4>	1.12 × 1.20	1.344	0.16128
5	1.12 × 1.20	1.344	0.16128

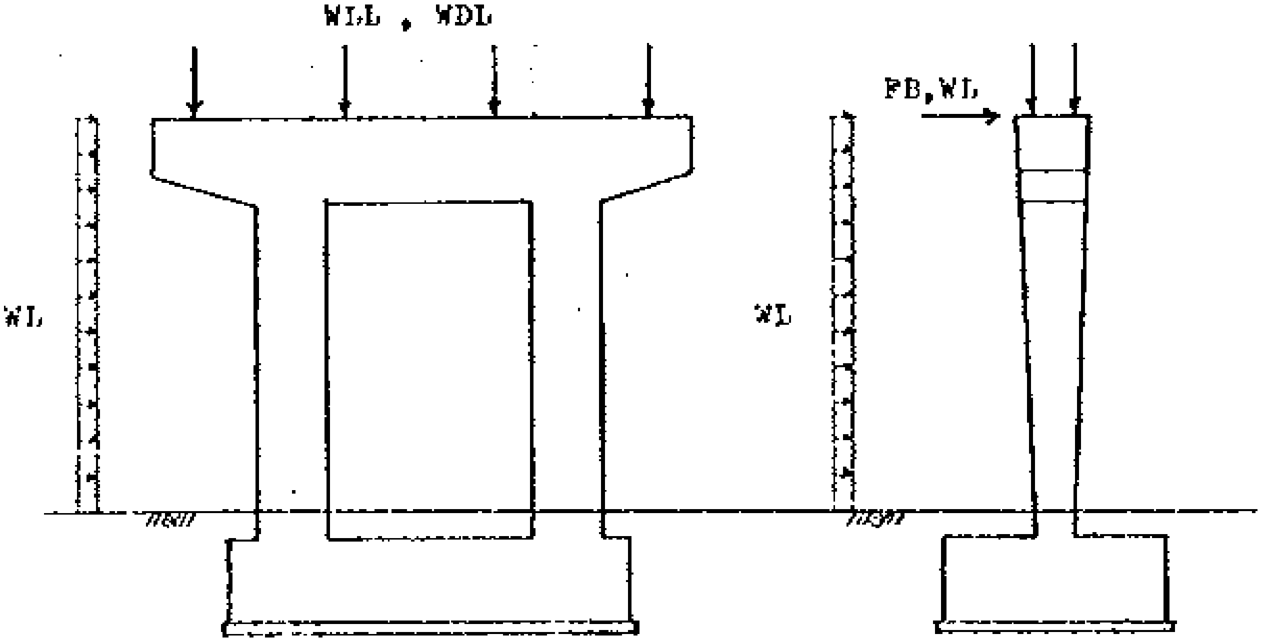
§ 3 CALCULATION OF LOADS

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 charge and drying schrinkage	FT & FS
8	wind	WL

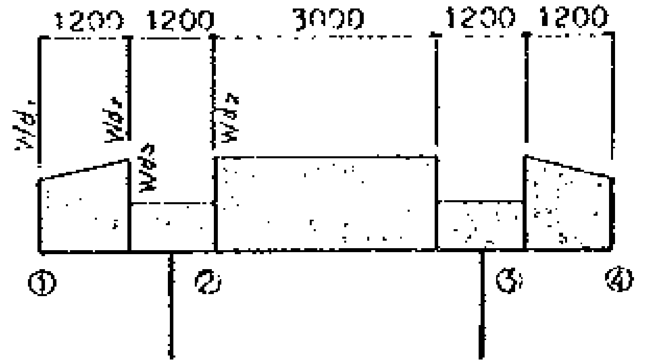
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3.) Dead load

(a) beam



Wd1 = 1.45 x 1.0 x 2.407 = 3.49 t/m

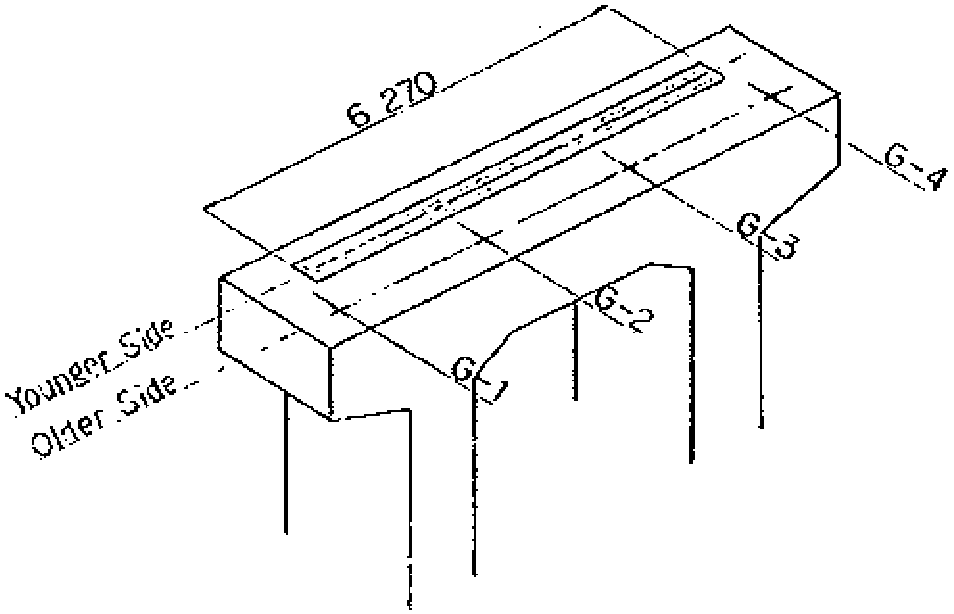
Wd2 = 1.43 x 1.3 x 2.407 = 4.47 t/m

Wd3 = 1.47 x 0.65 x 2.407 = 2.30 t/m

(b) column

Pw = 1.115 x 1.2 x 2.407 = 3.22 t/m

4) Reaction



(ton)

			G-1	G-2	G-3	G-4	G-5
Dead load		Younger	$31.4/6.27 = 5.01$ t/m				
		Older	46.2	49.0	48.5	51.8	
HA load	uniform	Younger	$21.1/6.27 = 3.37$ t/m				
		Older	22.1	23.9	22.6	12.1	
	line		6.4	8.3	9.3	2.9	
Crowd load		Younger	$2.1/6.27 = 0.33$ t/m				
		Older		0.9	1.5	9.0	
H B load	(H B)						
	H B						
$\frac{1}{3}$HA	uniform	Younger					
		Older					
	line						
Total (H A)			($23.2/6.27 = 3.70$ t/m) $54.6/6.27 = 8.71$ t/m				
			(28.5) 74.7	(33.7) 82.1	(35.4) 81.9	(24.0) 75.8	
Total (H B)							

5.) Temperture change and Drying schrinkage

temperture change -12 ~ +20 deg.

drying schrinkage -20 deg.

6.) Wind Load (transverse)

(a) For superstructure

$Vc = 38 \times 1.0 \times 1.0 \times 1.62 = 61.6$ m/sec. > 35 m/sec.

Unloaded

$Pt = q \cdot A \cdot Cd$

$q = 0.613 \times 61.6^2 \times 0.102 = 237.0$ kg/m²

$A = 2.075 \times 17.3 = 35.9$ m²

$Cd = 1.4$

$Pt = 237.0 \times 35.9 \times 1.4$

$= 11.9$ ton

Live loaded

$Pt = q \cdot A \cdot Cd$

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

$A = 4.360 \times 17.3 = 75.2$ m²

$Cd = 1.6$

$Pt = 76.6 \times 75.2 \times 1.6$

$= 9.2$ ton

(b) For safety fences

$Vc = 38 \times 1.0 \times 1.0 \times 1.62 = 61.6$ m/sec.

$Pt = q \cdot A \cdot Cd$

$q = 237.0$ kg/m²

$A = (0.150 \times 17.3 + 0.850 \times 0.05 \times 17.3/2) \times 2 = 5.93$ m²

$Cd = 1.1$

$Pt = 237.0 \times 5.93 \times 1.1 = 1.5$ ton

(c) For Pier

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

Unloaded

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

$A1 = (1.50 + 1.37) \times 1.3 \times 1/2 = 1.9$ m²

$A2 = (1.37 + 0.80) \times 5.57 \times 1/2 \times 2 = 12.1$ m²

$Cd = 1.7$

$Pt = 219.9 \times (1.9 + 12.1) \times 1.7$

$= 5.2$ ton

Live loaded

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

$A = A1 + A2 = 14.0$ m²

$Cd = 1.7$

$Pt = 76.6 \times 14.0 \times 1.7 = 1.8$ ton

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	11.9	9.2
safety fences	1.5	
Pier	5.2	1.8

7.) Longitudinal force

(a) due to breaking

$HA \longrightarrow 25.8$ ton

$HB \longrightarrow 38.3$ ton

(b) due to Wind

for superstructure

unloaded $PLS = 0.25\ Pt = 0.25 \times 11.9$

$= 3.0^{ton}$

live loaded $PLL = 0.5\ Pt$

$Pt = 76.6 \times (2.5 \times 17.3) \times 1.42$

$= 4.7^{ton}$

$PLL = 0.5 \times 4.7 = 2.4^{ton}$

$PLS = 0.25\ Pt$

$Pt = 76.6 \times (4.35 - 2.5) \times 17.3 \times 1.40.$

$= 3.4^{ton}$

$PLS = 0.25 \times 3.4 = 0.9^{ton}$

for safety fences

vertical member

$PL = 0.8\ Pt$

$Pt = 237.0 \times (0.85 \times 0.05 \times \frac{17.3}{2} \times 2) \times 1.1$

$= 0.2^{ton}$

$PL = 0.2^{ton}$

longitudinal member

$PL = 0.4\ Pt$

$Pt = 237.0 \times (0.15 \times 17.3 \times 2) \times 1.1$

$= 1.4^{ton}$

$PL = 0.5^{ton}$

for Pier

Vc = 59.3 m/sec.

unloaded

Q = 219.9 kg/m^2

A1 = (5.4 + 7.8) × 1.3 × 1/2 = 8.6 m^2

A2 = 1.2 × 5.57 × 2 = 13.4 m^2

Cd1 = 2.1

Cd2 = 2.0

Pt1 = 219.9 × 8.6 × 2.1 = 4.0 ton

Pt2 = 219.9 × 13.4 × 2.0 = 5.9 ton

live loaded

Q = 76.6 kg/m^2

A1 = 8.6 m^2

A2 = 13.4 m^2

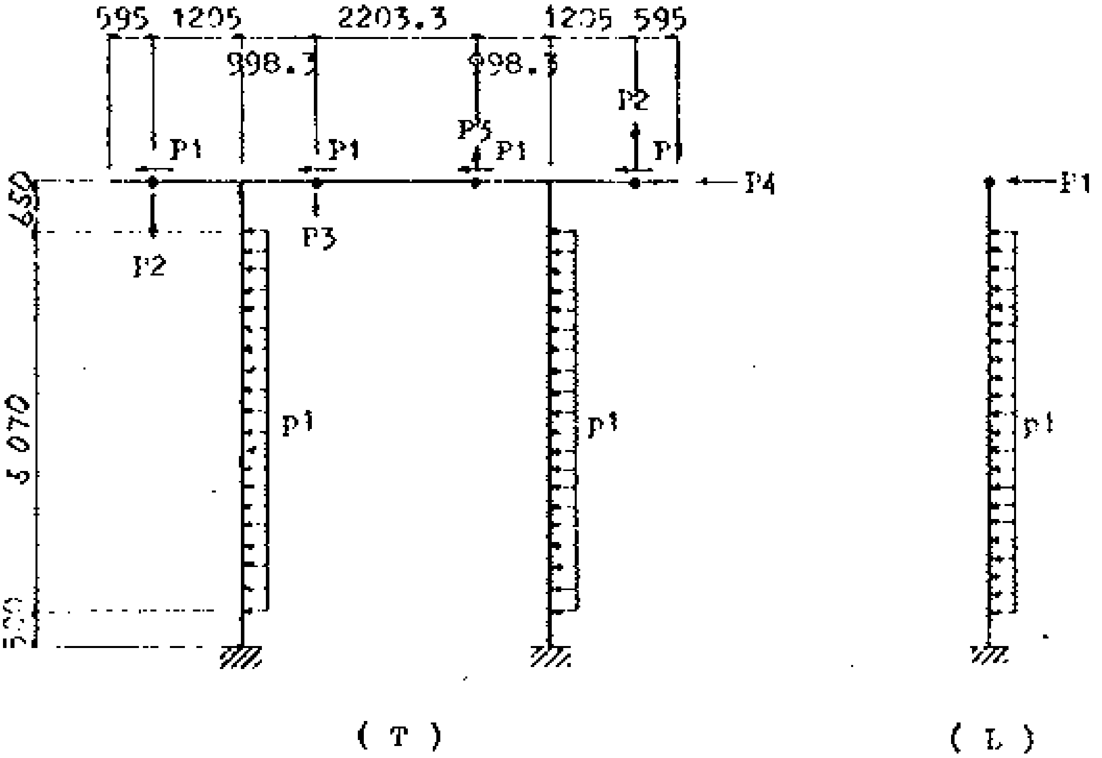
Cd1 = 2.1

Cd2 = 2.0

Pt1 = 76.6 × 8.6 × 2.1 = 1.4 ton

Pt2 = 76.6 × 13.4 × 2.0 = 2.1 ton

8.) Wind loading diagram

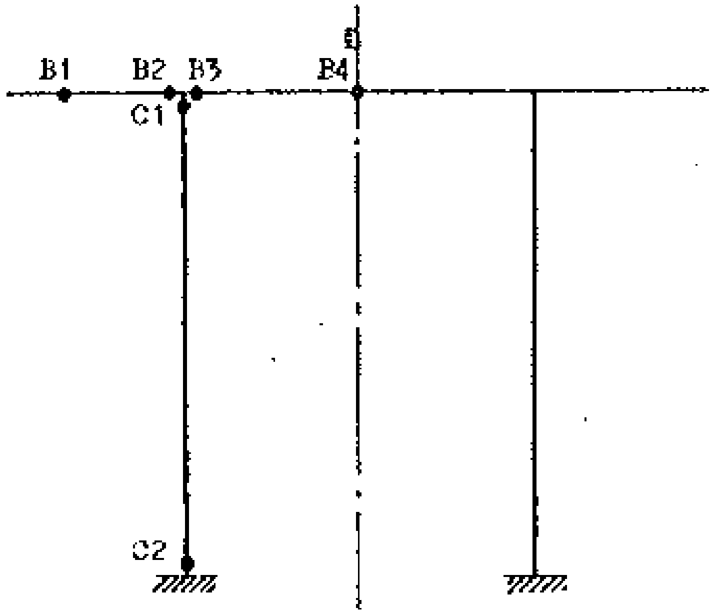


		for place	unloaded	live loaded
(T)	P1	superstructure safety fences	3.35	2.30
	P2	superstructure safety fences	0.39	0.34
	P3	superstructure safety fences	1.13	1.18
	P4	pier	0.71	0.25
	p1	pier	0.45	0.16
(L)	P1	superstructure safety fences pier	3.85	2.35
	p1	pier	0.58	0.21

(P;ton , p;t/m)

§ 4 ACTING FORCE TABLE

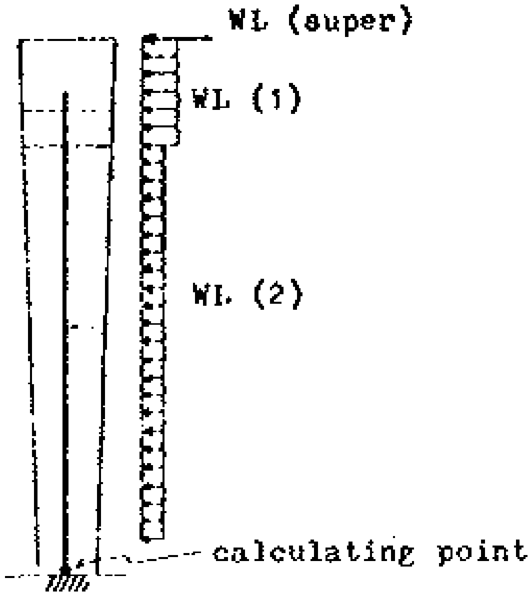
1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.646	-0.000	-76.921	1.0
	N max.	+	2	-0.646	3.350	-51.811	1.0
		-	2	-0.646	-0.710	2.221	1.0
	S max.	+					
		-	3	-0.646	-0.000	-76.921	1.0
B2	M max.	+					
		-	3	-102.032	0.000	90.971	1.0
	N max.	+	2	-68.467	3.350	-60.931	1.0
		-	2	-67.045	-4.060	59.731	1.0
	S max.	+	3	-102.032	0.000	90.971	1.0
		-					

B3	M max.	+					
		-	3	-120.855	-7.929	-116.262	1.0
	N max.	+	2	-4.218	17.035	-55.668	1.0
		-	2	-106.426	-10.150	79.044	1.0
	S max.	+					
		-	3	-64.698	12.755	-116.262	1.0
B4	M max.	+	3	72.572	11.460	-1.476	1.0
		-	3	-36.129	-3.628	-0.491	1.0
	N max.	+	3	20.078	16.836	-0.401	1.0
		-	3	16.415	-7.025	-1.476	1.0
	S max.	+	3	36.819	17.496	19.332	1.15
		-					
C1	M max.	+	2	62.827	-113.419	-21.095	1.0
		-	3	-52.146	-137.076	16.836	1.0
	N max.	+					
		-	3	37.334	-207.233	-12.755	1.0
	S max.	+					
		-	2	62.827	-113.419	-21.095	1.0
C2	M max.	+	3	32.700	-177.105	16.856	1.0
		-	2	-75.307	-135.447	-23.376	1.0
	N max.	+					
		-	3	-47.605	-227.261	-12.755	1.0
	S max.	+					
		-	2	-75.307	-135.447	-23.376	1.0

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	44.3	218.9	6.45	1.00
case-8	—	—	—	1.25
case-9	62.9	218.9	9.85	1.15
case-10	—	—	—	1.30
case-11	33.3	147.8	6.80	1.00

CASE 7

	N	H	y	M
WDL	113.5			
WLL (A)	71.1			
FB (A)		6.45	6.87	44.3
W	34.3			
TOTAL	218.9	6.45		44.3

M = 44.3 t·m

N = 218.9 ton

H = 6.45 ton

CASE 9

	N	H	y	M
WDL	113.5			
WLL, FB(A)	71.1	6.45	6.87	44.3
W	34.3			
WL (Super)		1.65	6.87	11.3
WL (1)		0.70	6.22	4.4
WL (2)		1.05	2.79	2.9
TOTAL	218.9	9.85		62.9

M = 62.9 t·m

N = 218.9 ton

H = 9.85 ton

CASE 11

	N	H	y	M
WDL	113.5			
W	34.3			
WL (super)		1.85	6.87	12.7
WL (1)		2.00	6.22	12.4
WL (2)		2.95	2.79	8.2
TOTAL	147.8	6.80		33.3

M = 33.3 t·m

N = 147.8 ton

H = 6.80 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1	B2, B3	B4
	case ③	case ③	case ③
M (tm)	0.646	120.855	72.572
N (ton)	0.000	7.729	11.460
Q; shear force (ton)	76.921	116.262	1.476
b (cm)	144	144	144
d (cm)	105	120	120
d' (cm)	10	10	10
As (cm^2)		49.1 cm² (10-D25 15cm pitch)	28.3 cm² (9-D20 15cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d		0.043	0.075
M'=M+N·u (tm)			
M'/(b·d²) (%d)		5.83	3.30
Q/b·d (kg/cm^2)	5.09	6.73	0.09
C		8.63	10.7
S		25.6	42.9
	1.09	1.09	1.07
σ_c (kg/cm^2)		50	37
σ_s (kg/cm^2)		2239	2252
τ (kg/cm^2)	5.55	7.3	0.1
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

stirrups Min. As = 0.0012 × 144 × 12.5 = 2.16

S = 12.5 cm used As = Φ16 □ × 2 = 4 × 2.01 = 8.04 cm²

2.) Column section

	of C1	of C2	of C2
	case	case 2	case 9
M (tm)		73.307	62.9
N (ton)		135.447	218.9
Q;shear force (ton)		16.856	9.85
b (cm)		80	120
d (cm)		110	70
d' (cm)		10	10
As (cm^2)	38.4 cm^2 6-D32 } alternate 5-D16	40.2 cm^2 5-D32	45.6 cm^2 2-D32 (edge) 6-D25 (inter)
d'/d		0.091	0.14
f=(M/N)+u (cm)		105.6	58.7
f/d		0.96	0.84
n	15	15	15
nP=n·As/b·d		0.068	0.081
M'=M+N·u (tm)		143.0	128.6
M'/(b·d^2) ([illegible])		14.8	21.9
Q/b·d (kg/cm^2)		1.76	1.03
C		3.6	0.05
S		2.7	1.00
σ_c (kg/cm^2)		53	67
σ_s (kg/cm^2)		599	427
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)		85	98
σ_{sa} (kg/cm^2)		2340	2691
τ_a (kg/cm^2)		8.2	9.4

PIER IN COROMANDEL O.V.

(P - 3)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight $H = 8.87$ m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.407 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

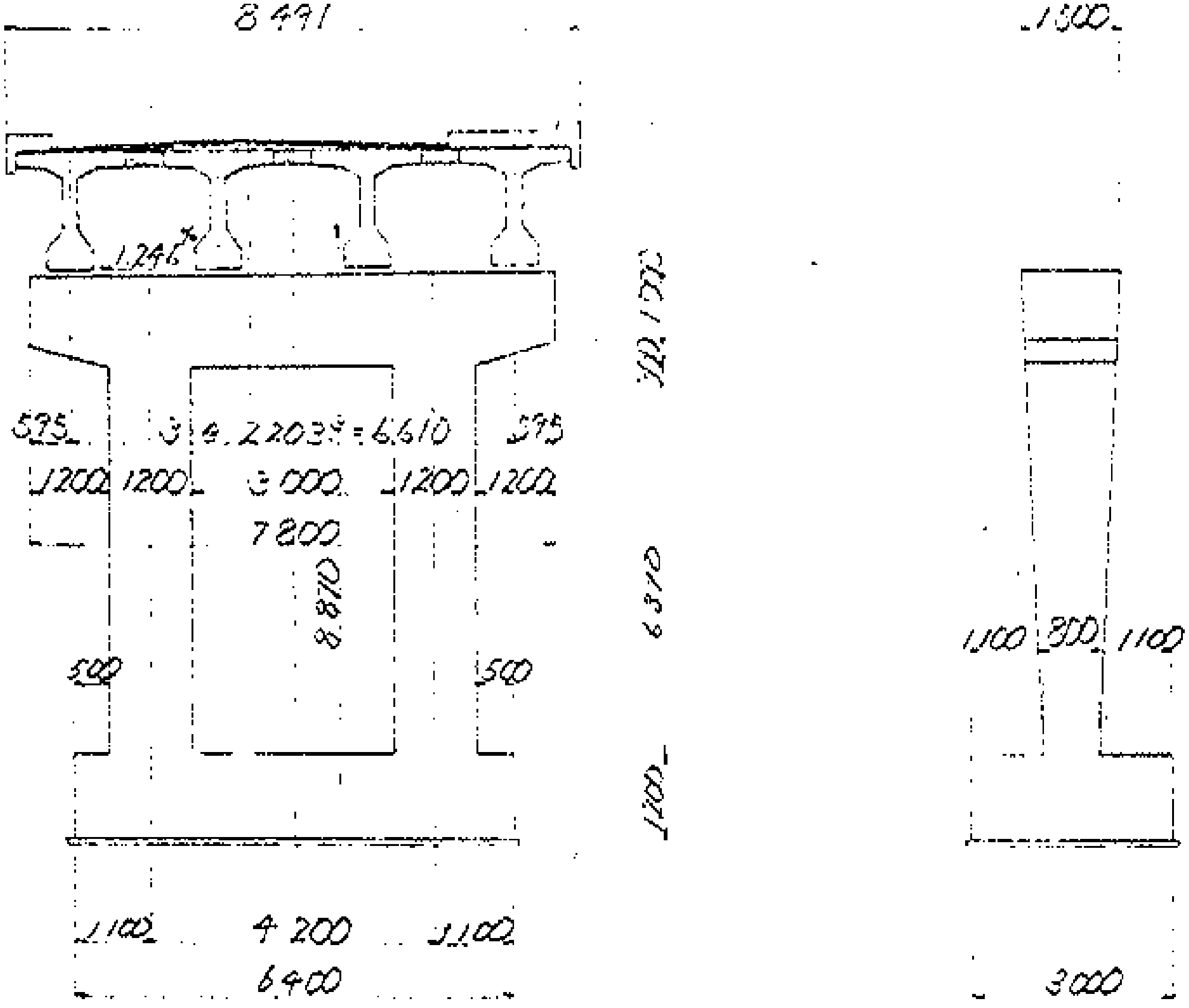
su = 4 180 kg/cm^2

permissible tensile stress

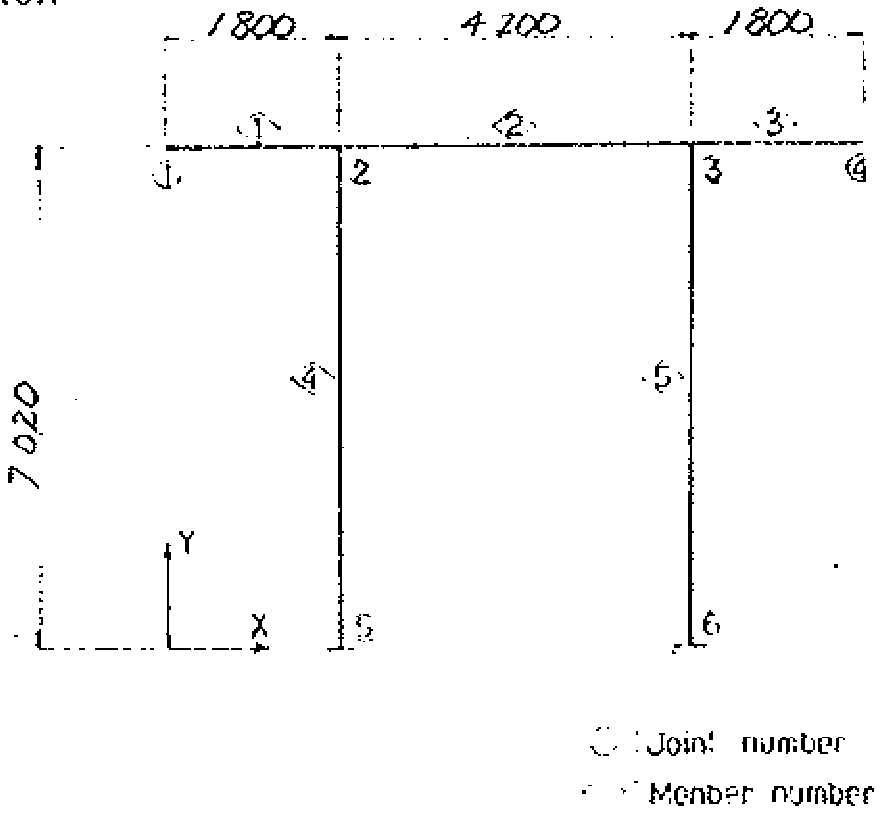
sa = 2 340 kg/cm^2

modular ratio $n = 15$

62 GENERAL DIMENCION



Skeleton



Coordinates

(m)

	X	Y		X	Y
①	0.0	7.020	4	7.800	7.020
2	1.800	7.020	5	1.800	0.0
3	6.000	7.020	6	6.000	0.0

Modulus of elasticity of concrete

$E_c = 26 \text{ kN/mm}^2$

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

member	section B x H	area of section A (m^2)	moment of inertia I (m^4)
①	1.45 x 1.15	1.668	0.18377
②	1.45 x 1.50	2.175	0.40781
③	1.45 x 1.15	1.668	0.18377
④	1.12 x 1.20	1.344	0.16128
⑤	1.12 x 1.20	1.344	0.16128

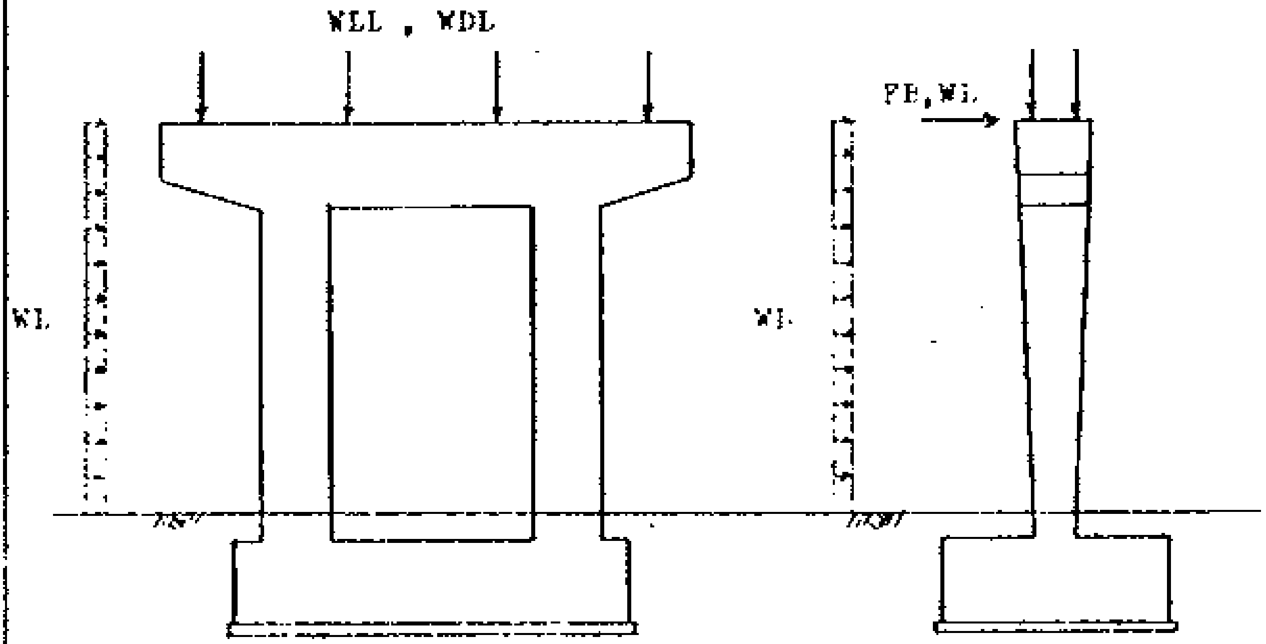
§ 3 CALCULATION OF LOADS

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

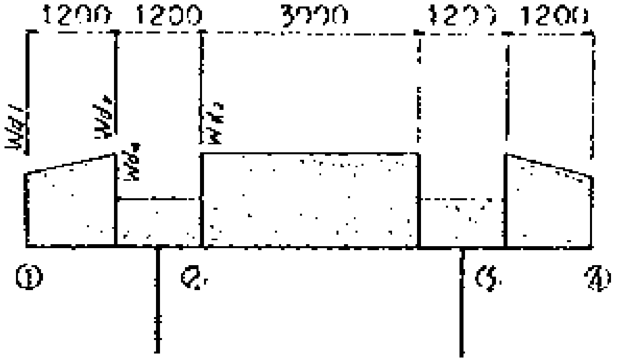
case	load combination	increase in basic stresses
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3.) Dead load

(a) beam



Wd1 = 1.45 x 1.0 x 2.407 = 3.49 t/m

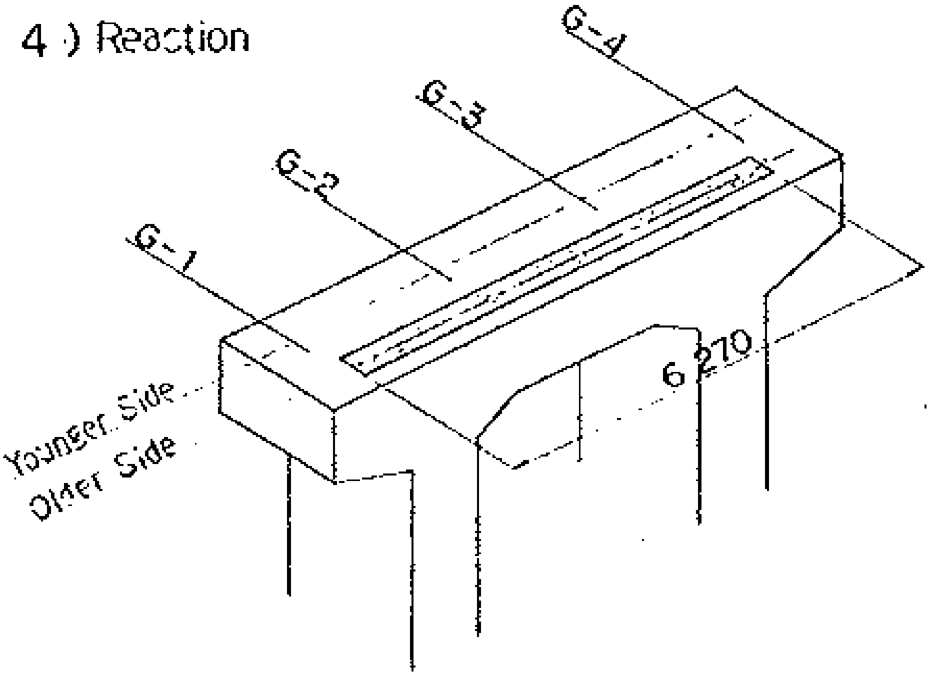
Wd2 = 1.44 x 1.3 x 2.407 = 4.51 t/m

Wd3 = 1.47 x 0.65 x 2.407 = 2.30 t/m

(b) column

Pw = 1.12 x 1.2 x 2.407 = 3.24 t/m

4) Reaction



(ton)

			G-1	G-2	G-3	G-4	G-5
Dead load		Younger	46.2	49.0	48.5	51.8	
		Older	42.0/6.27 = 6.70 t/m				
HA load	uniform	Younger	22.1	23.9	22.6	12.1	
		Older	19.1/6.27 = 3.05 t/m				
	line		6.4	8.3	9.3	2.9	
Crowd load		Younger		0.9	1.5	9.0	
		Older	2.5/6.27 = 0.40 t/m				
HB load ($\frac{1}{3}$HA)	(HB)						
	HB						
	uniform	Younger					
		Older					
	line						
Total (HA)			(28.5) 74.7	(33.1) 82.1	(33.4) 81.9	(24.0) 75.8	
			(21.6/6.27 = 3.44 t/m) 63.6/6.27 = 10.14 t/m				
Total (HB)							

5.) Temperture change and Drying schrinkage

temperture change -12 +20 deg.

drying schrinkage -20 deg.

6.) Wind Load (transverse)

(a) For superstructure

$V_c = 38 \times 1.0 \times 1.0 \times 1.62 = 61.6$ m/sec. > 35 m/sec.

Unloaded

$P_t = q \cdot A \cdot C_d$

$q = 0.613 \times 61.6^2 \times 0.102 = 237.0$ kg/m²

$A = 2.075 \times 17.8 = 36.9$ m²

$C_d = 1.4$

$P_t = 237.0 \times 36.9 \times 1.4$

$= 12.2$ ton

Live loaded

$P_t = q \cdot A \cdot C_d$

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

$A = 4.350 \times 17.8 = 77.4$ m²

$C_d = 1.6$

$P_t = 76.6 \times 77.4 \times 1.6$

$= 9.5$ ton

(b) For safety fences

$V_c = 38 \times 1.0 \times 1.0 \times 1.62 = 61.6$ m/sec.

$P_t = q \cdot A \cdot C_d$

$q = 237.0$ kg/m²

$A = (0.150 \times 17.8 + 0.850 \times 0.05 \times 17.8 \times \frac{1}{2}) \times 2 = 6.10$ m²

$C_d = 1.1$

$P_t = 237.0 \times 6.10 \times 1.1 = 1.6$ ton

(c) For Pier

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

Unloaded

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9 \ \mathrm{kg/m^2}$

$A1 = (1.50 + 1.38) \times 1.3 \times 1/2 = 1.9 \ \mathrm{m^2}$

$A2 = (1.38 + 0.80) \times 6.37 \times 1/2 \times 2 = 13.9 \ \mathrm{m^2}$

$Cd = 1.7$

$Pt = 219.9 \times (1.9 + 13.9) \times 1.7$

$= 5.9^{\mathrm{ton}}$

Live loaded

$q = 0.613 \times 35.0^2 \times 0.102 = 76.6 \ \mathrm{kg/m^2}$

$A = A1 + A2 = 15.8 \ \mathrm{m^2}$

$Cd = 1.7$

$Pt = 76.6 \times 15.8 \times 1.7 = 2.1^{\mathrm{ton}}$

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	12.2	9.5
safety fences	1.6	
Pier	5.9	2.1

7.) Longitudinal force

(a) due to breaking

HA ———→ 25.8 ton

HB ———→ 38.3 ton

(b) due to Wind

for superstructure

unloaded $PLS = 0.25\ Pt = 0.25 \times 12.2$

$= 3.1^{ton}$

live loaded $PLL = 0.5\ Pt$

$Pt = 76.6 \times (2.5 \times 17.8) \times 1.42$

$= 4.8^{ton}$

$PLL = 2.4^{ton}$

$PLS = 0.25\ Pt$

$Pt = 76.6 \times (4.35 - 2.5) \times 17.8 \times 1.40$

$= 3.5^{ton}$

$PLS = 0.9^{ton}$

for safety fences

vertical member

$PL = 0.8\ Pt$

$Pt = 237.0 \times (0.85 \times 0.05 \times \frac{17.8}{2} \times 2) \times 1.1$

$= 0.2^{ton}$

$PL = 0.2^{ton}$

longitudinal member

$PL = 0.4\ Pt$

$Pt = 237.0 \times (0.15 \times 17.8 \times 2) \times 1.1$

$= 1.4^{ton}$

$PL = 0.6^{ton}$

for Pier

Vc = 59.3 m/sec.

unloaded

q = 219.9 kg/m^2

A1 = (5.4 + 7.8) × 1.3 × 1/2 = 8.6 m^2

A2 = 1.2 × 6.37 × 2 = 15.3 m^2

Cd1 = 2.1

Cd2 = 2.0

Pt1 = 219.9 × 8.6 × 2.1 = 4.0 ton

Pt2 = 219.9 × 15.3 × 2.0 = 6.7 ton

live loaded

q = 76.6 kg/m^2

A1 = 8.6 m^2

A2 = 15.3 m^2

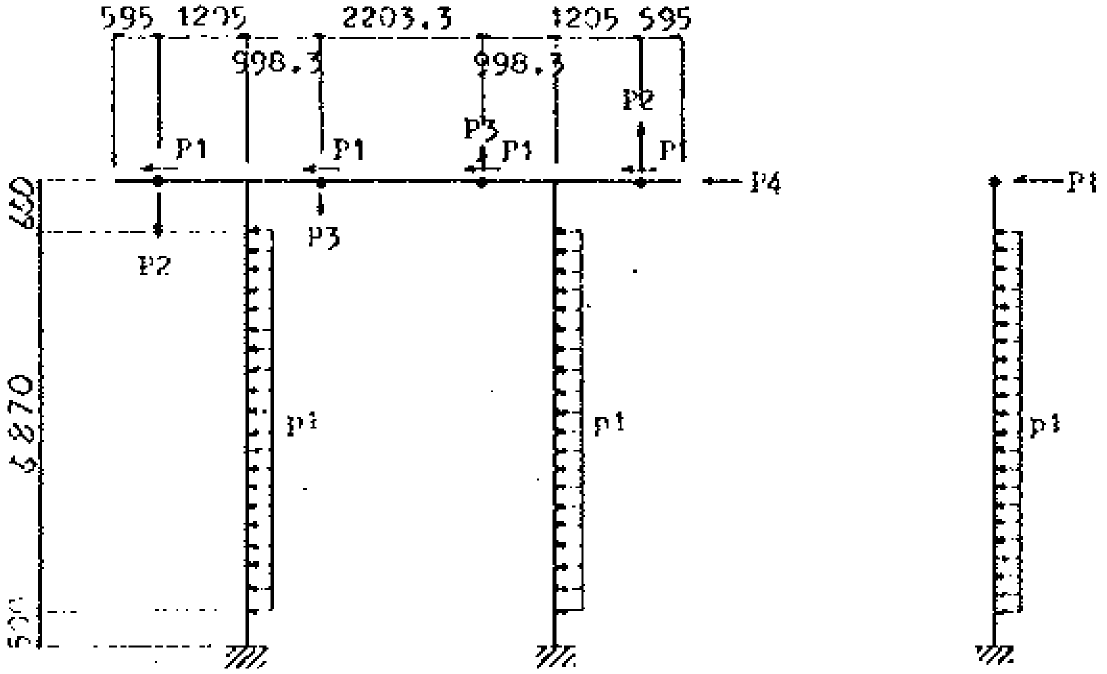
Cd1 = 2.1

Cd2 = 2.0

Pt1 = 76.6 × 8.6 × 2.1 = 1.4 ton

Pt2 = 76.6 × 15.3 × 2.0 = 2.3 ton

8.) Wind loading diagram



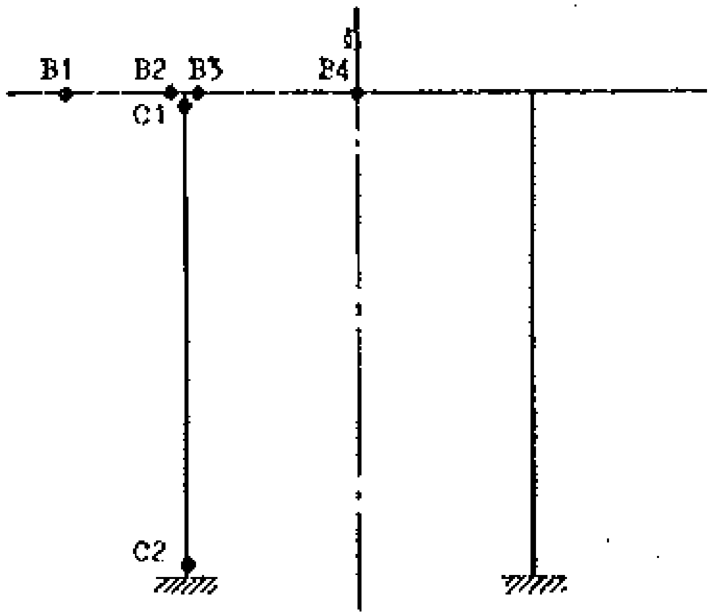
(T) (L)

		for place	unloaded	live loaded
(T)	P1	superstructure safety fences	0.45	2.38
	P2	superstructure safety fences	0.51	0.65
	P3	superstructure safety fences	1.17	1.22
	P4	pier	0.71	0.25
	p1	pier	0.44	0.15
(L)	P1	superstructure safety fences pier	0.95	2.35
	p1	pier	0.57	0.20

(P;ton , p;t/m)

§ 4 ACTING FORCE TABLE

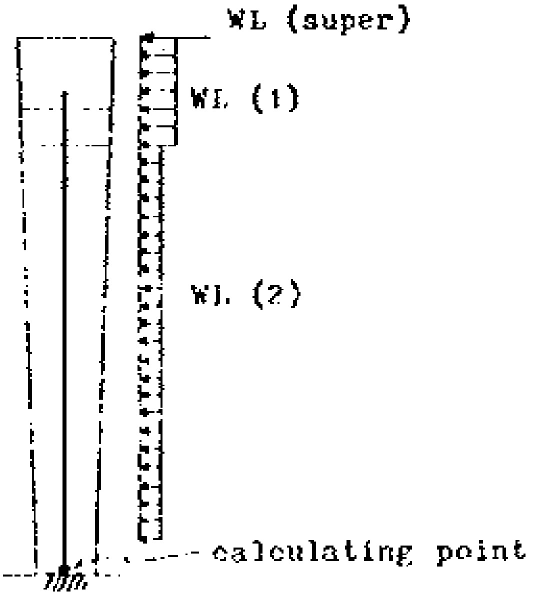
1.) due to transverse force



			case	M (tm)	N (ton)	S (ton)	
B1	M max.	+					
		-	3	-0.648	-0.000	76.927	1.0
	N max.	+	2	-0.648	3.450	-31.937	1.0
		-	2	-0.648	-0.710	2.227	1.0
	S max.	+					
		-	3	-0.648	0.000	-76.927	1.0
B2	M max.	+					
		-	3	-107.872	-0.000	92.475	1.0
	N max.	+	2	69.341	3.450	-62.824	1.0
		-	2	67.830	-1.160	61.404	1.0
	S max.	+	3	-107.872	-0.000	92.475	1.0
		-					

B3	M max.	+					
		-	5	-119.494	-6.900	111.359	1.15
	N max.	+	2	-8.210	13.191	-57.232	1.0
		-	2	-108.604	-8.395	84.638	1.0
	S max.	+					
		-	3	-72.240	9.071	-119.337	1.0
B4	M max.	+	3	68.677	8.020	-1.483	1.0
		-	3	-31.223	-2.303	-0.402	1.0
	N max.	+	3	14.873	12.383	-0.402	1.0
		-	3	22.381	-6.667	-1.483	1.0
	S max.	+	5	32.982	8.990	20.440	1.15
		-					
C1	M max.	+	2	37.620	-118.696	-17.351	1.0
		-					
	N max.	+					
		-	3	30.582	-211.812	-9.071	1.0
	S max.	+					
		-	2	37.620	-118.696	-17.351	1.0
C2	M max.	+					
		-	2	-71.060	-141.441	-19.934	1.0
	N max.	+					
		-	3	-33.098	-234.557	-9.071	1.0
	S max.	+					
		-	2	-71.060	-141.441	-19.934	1.0

2) due to longitudinal force



	M (tm)	V (ton)	H (ton)	
case-7	49.5	226.7	6.45	1.00
case-8				1.25
case-9	74.4	226.2	9.95	1.15
case-10				1.30
case-11				1.00

CASE 7

	N	H	y	M
WDL	118.8			
WLL (A)	70.3			
FB (A)		6.45	7.67	49.5
W	37.1			
TOTAL	226.2	6.45		49.5

M = 49.5 t·m

N = 226.2 ton

H = 6.45 ton

CASE 9

	N	H	y	M
WDL	118.8			
WLL, FB (A)	70.3	6.45	7.67	49.5
W	37.1			
WL (Super)		1.65	7.67	12.7
WL (1)		0.70	7.02	4.9
WL (2)		1.15	6.37	7.3
TOTAL	226.2	9.95		74.4

M = 74.4 t·m

N = 226.2 ton

H = 9.95 ton

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1 case 3	B2, B3 case 5	B4 case 3
M (tm)	-0.648	-197.418	68.677
N (ton)	-6.000	-7.935	8.020
Q; shear force (ton)	-76.727	128.293	-1.483
b (cm)	145	145	145
d (cm)	105	120	120
d' (cm)	10	10	10
As (cm²)		47.1 cm² (10-D25 15cm pitch)	28.3 cm² (9-D20 15cm pitch)
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nF=n·As/b·d		0.042	0.024
M'=M+N·u (tm)			
M'/(b·d²) (kg/cm²)		6.58	3.27
Q/b·d (kg/cm²)	5.05	7.37	0.09
C		8.71	10.9
S		26.2	44.6
z	1.09	1.09	1.07
σ_c (kg/cm²)		57	36
σ_s (kg/cm²)		2586	2201
τ (kg/cm²)	5.51	8.03	0.1
σ_{ca} (kg/cm²)	85	98	85
σ_{sa} (kg/cm²)	2340	2691	2340
τ_a (kg/cm²)	8.2	9.4	8.2

Stirrups As = ϕ16 ⊏×2 = 4 × 2.01

= 8.04 cm²

2.) Column section

	at C1 case	at C2 case 2	at C2 case 9
M (tm)		71.060	74.4
N (ton)		141.441	226.2
Q;shear force (ton)		19.934	9.95
b (cm)		80	120
d (cm)		110	70
d' (cm)		10	10
As (cm^2)	38.4 cm² 6-D32 } ultimate 5-D16	4 . cm² - D32	45.6 cm² 2-D32 (edge) 6-D25 (inter)
d'/d		0.091	0.14
f=(M/N)+u (cm)		100.2	62.9
f/d		0.91	0.90
n	15	15	15
nP=n·As/b·d		0.069	0.081
M'=M+N·u (tm)		141.8	147.3
M'/(b·d^2) ([illegible])		14.6	24.2
Q/b·d (kg/cm^2)		2.08	1.04
C		3.25	3.26
S		1.9	1.8
Z			
σ_c (kg/cm^2)		47	79
σ_s (kg/cm^2)		416	658
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)		85	98
σ_{sa} (kg/cm^2)		2340	2691
τ_a (kg/cm^2)		8.2	9.4

Vol. 4 STA. 22 OV. BR. P_2 , P_1 , P_3 PIERS

PIER IN No.22 OV.

(P - 2)

§ 1 DESIGN CONDITIONS

1 Pier type

Rigid-frame pier

hight H = 9.0 m

2 Foundation type

footing foundation

3 Unit weight of reinforced concrete

2.4[illegible] t/m^3

4 Allowable stresses of reinforced concrete

1.) concrete (grade 2[illegible] , BS 54[illegible])

cube strength at 28 days ck = 2[illegible] kg/cm^2

bending stress ca = 85

direct stress ca = [illegible]

shear stress a = [illegible]

2.) reinforcement (hot rolled high yield bars, BS 444[illegible])

specified characteristic strength

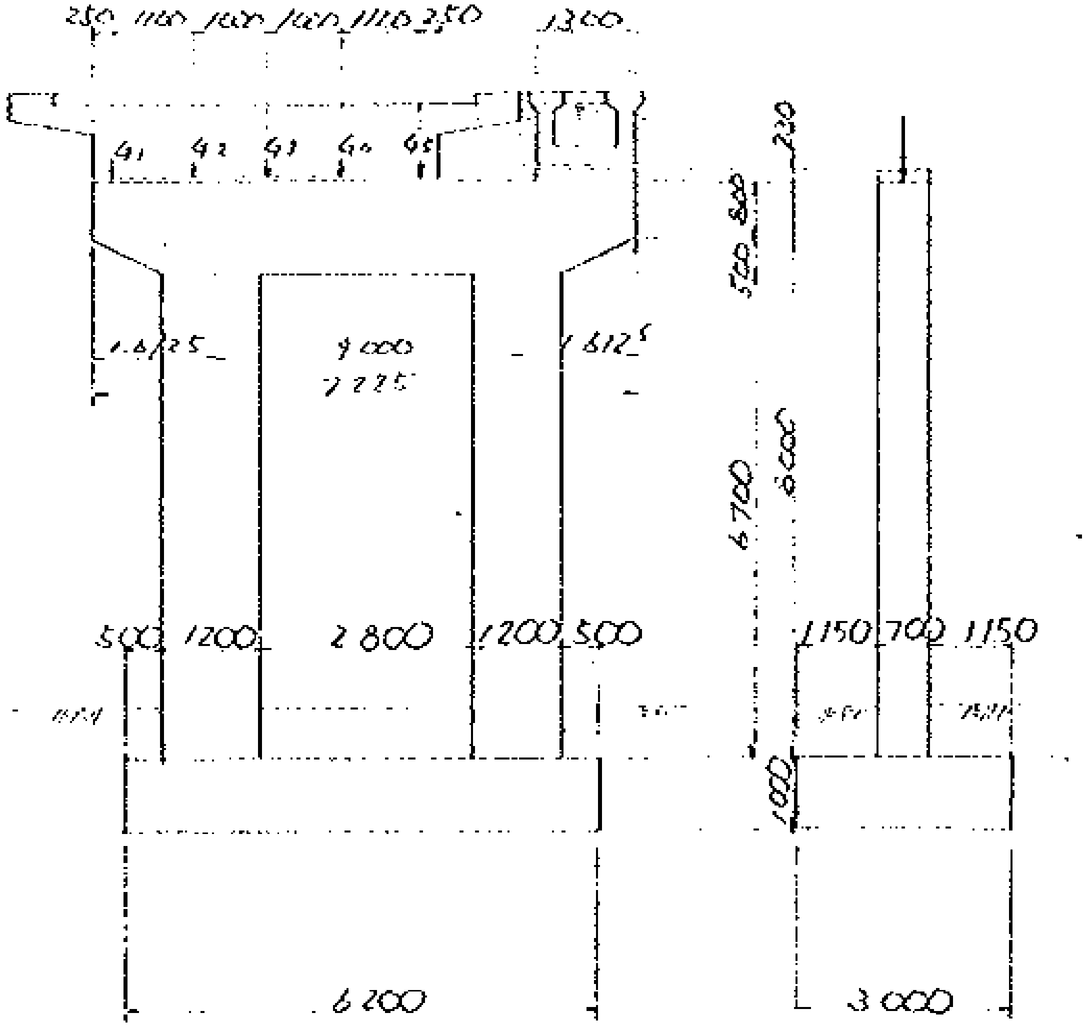
su = 4 10[illegible] kg/cm^2

permissible tensile stress

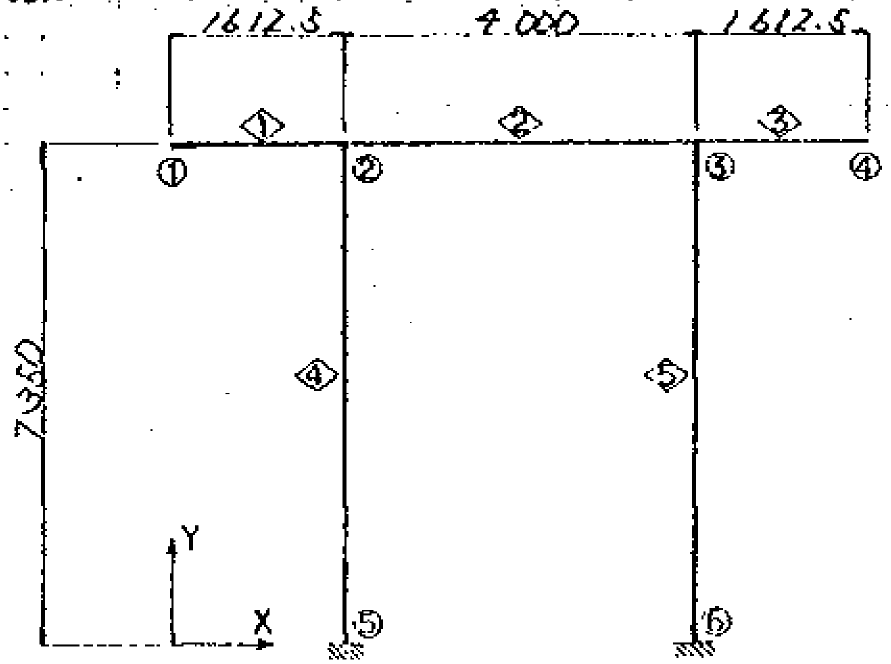
sa = [illegible] 34[illegible] kg/cm^2

modular ratio n = 15

§2. GENERAL DIMENSION



1.) Skeleton



○ : Joint number
◇ : Member number

Coordinates

(m)

	X	Y		X	Y
①	0.0	7.35	④	7.225	7.35
②	1.6125	7.35	⑤	1.6125	0.0
③	5.6125	7.35	⑥	5.6125	0.0

2.) Modulus of elasticity of concrete

$Ec = 26\ kN/mm^2$

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

member	section B x H	area of section A (m^2)	moment of inertia I (m^4)
①	0.7 × 1.05	0.735	0.06753
②	0.7 × 1.30	0.910	0.12816
③	0.7 × 1.05	0.735	0.06753
④	0.7 × 1.20	0.840	0.10080
⑤	0.7 × 1.20	0.840	0.10080

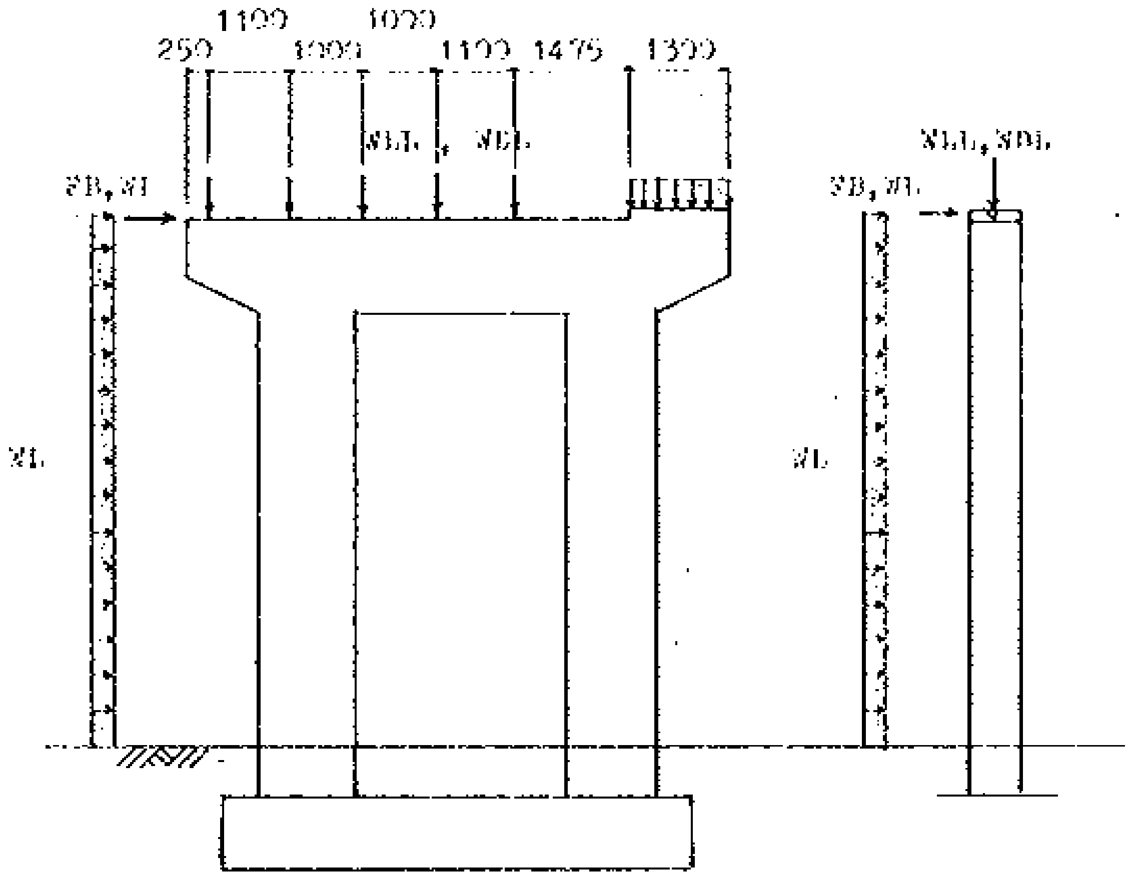
§ 3 CALCULATION OF LOADS

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

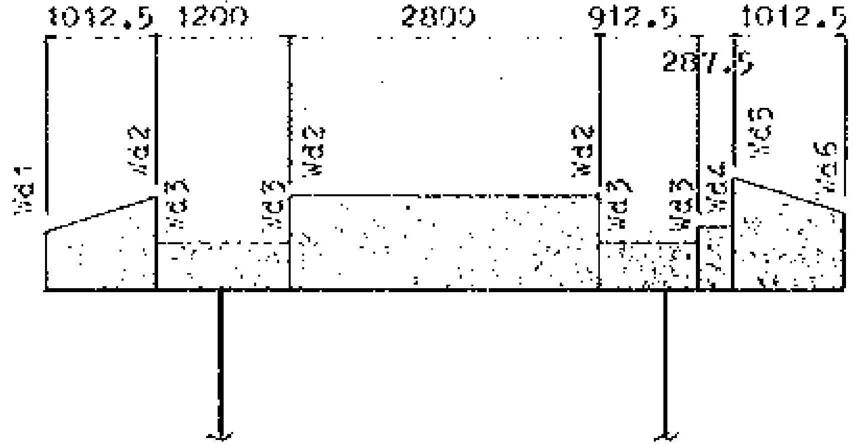
case	load combination	increase in basic stress
1	1, 6, 7	1.00
2	1, 6, 7, 8	1.00
3	1, 2, 6, 7	1.00
4	1, 3, 6, 7	1.25
5	1, 2, 6, 7, 8	1.15
6	1, 3, 6, 7, 8	1.30
7	1, 2, 4, 6, 7	1.00
8	1, 3, 5, 6, 7	1.25
9	1, 2, 4, 6, 7, 8	1.15
10	1, 3, 5, 6, 7, 8	1.30
11	1, 6, 7, 8	1.00

2.) Loading diagram



3.) Dead load

(a) beam



Wd1 = 0.7 x 0.8 x 2.407 = 1.35 t/m

Wd2 = 0.7 x 1.3 x 2.407 = 2.19 t/m

Wd3 = 0.7 x 0.65 x 2.407 = 1.10 t/m

Wd4 = 0.7 x 0.88 x 2.407 = 1.48 t/m

Wd5 = 0.7 x 1.53 x 2.407 = 2.58 t/m

Wd6 = 0.7 x 1.05 x 2.407 = 1.74 t/m

(b) column

Pw = 0.7 x 1.2 x 2.407 = 2.02 t/m

4) Reaction Table

a) Whole reaction

	Highway br.	chanel br.
Dead Load	220.35 t	55.99
HA Live	125.71 t	
footway live		7.26
Water		11.10 t

b) Each girder reaction

	G1	G2	G3	G4	G5	chanel
Dead	51.76	37.28	47.27	37.28	51.76	43.03 t/m
HALive	47.30	37.50	33.4	37.50	47.30	14.12 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

$Pt = q \cdot A \cdot Cd$

$q = 0.613 \times 59.3^2 \times 0.102 = 219.9\ \mathrm{kg/m^2}$

$A = 1.230 \times 15.8 = 19.4\ \mathrm{m^2}$

$Cd = 1.32$

$Pt = 219.9 \times 19.4 \times 1.32$

$= 5.6\ \mathrm{ton}$

Live loaded

$Pt = q \cdot A \cdot Cd$

$q = 0.613 \times 35.0^2 \times 0.102 = 76.6\ \mathrm{kg/m^2}$

$A = 3.580 \times 15.8 = 56.6\ \mathrm{m^2}$

$Cd = 1.55$

$Pt = 76.6 \times 56.6 \times 1.55$

$= 6.7\ \mathrm{ton}$

(b) For safety fences

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

$Pt = q \cdot A \cdot Cd$

$q = 219.9\ \mathrm{kg/m^2}$

$A = (0.15 \times 15.8 + 0.850 \times 0.05 \times 15.8 \times \frac{1}{2}) \times 2 = 5.4\ \mathrm{m^2}$

$Cd = 1.1$

$Pt = 219.9 \times 5.4 \times 1.1 = 1.3\ \mathrm{ton}$

(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

$A1 = 1.3 \times 0.7 = 0.91\ m^2$

$A2 = 6.2 \times 0.7 \times 2 = 8.68\ m^2$

$Cd = 2.2$

$Pt = 219.9 \times (0.91 + 8.68) \times 2.2$

$= 4.6$ ton

Live loaded

$q = 76.6$ kg/m²

$A = A1 + A2 = 9.59\ m^2$

$Cd = 2.2$

$Pt = 76.6 \times 9.59 \times 2.2 = 1.6$ ton

(d) Table of Wind load (transverce)

(ton)

	unloaded	live loaded
super-structure	5.6	6.7
safety fences	1.3	—
Pier	4.6	1.6

7.) Longitudinal force

(a) due to breaking

HA ——→ 25.8 ton

HB ——→ 38.3 ton

(b) due to Wind

for superstructure

unloaded $P_{LS} = 0.25\ P_t = 0.25 \times 5.6$

$= 1.4^{ton}$

live loaded $P_{LL} = 0.5\ P_t$

$P_t = 76.6 \times 2.5 \times 15.8 \times 1.45$

$= 4.4$

$P_{LL} = 2.2^{ton}$

$P_{LS} = 0.25\ P_t$

$P_t = 76.6 \times 1.08 \times 15.8 \times 1.32$

$= 1.7$

$P_{LS} = 0.4^{ton}$

for safety fences

vertical member

$P_L = 0.8\ P_t$

$P_t = 219.9 \times (0.85 \times 0.05 \times 15.8 \times 1/2 \times 2) \times 1.1$

$= 0.162$

$P_L = 0.1^{ton}$

longitudinal member

$P_L = 0.4\ P_t$

$P_t = 219.9 \times (0.15 \times 15.8 \times 2) \times 1.1$

$= 1.147$

$P_L = 0.5^{ton}$

for Pier

$V_c = 59.3$ m/sec.

unloaded

$q = 219.9\ \mathrm{kg/m^2}$

$A1 = 7.225 \times 1.3 - 1.6125 \times 0.5 + 1.3 \times 0.23 = 8.9\ \mathrm{m^2}$

$A2 = 1.2 \times 6.2 \times 2 = 14.9\ \mathrm{m^2}$

$Cd1 = 2.1$

$Cd2 = 1.7$

$Pt1 = 219.9 \times 8.9 \times 2.1 = 4.1^{\mathrm{ton}}$

$Pt2 = 219.9 \times 14.9 \times 1.7 = 5.6^{\mathrm{ton}}$

live loaded

$q = 76.6\ \mathrm{kg/m^2}$

$A1 = 8.9\ \mathrm{m^2}$

$A2 = 14.9\ \mathrm{m^2}$

$Cd1 = 2.1$

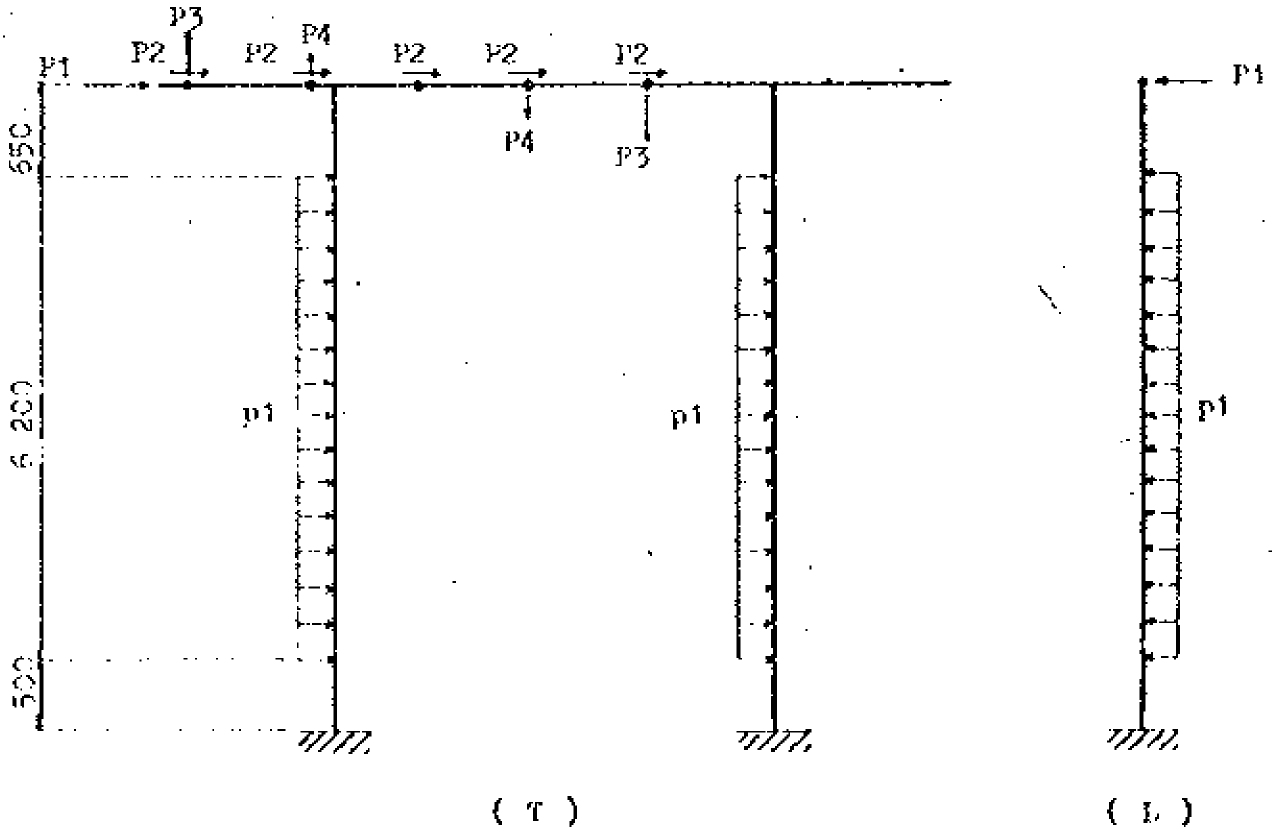
$Cd2 = 1.7$

$Pt1 = 76.6 \times 8.9 \times 2.1 = 1.4^{\mathrm{ton}}$

$Pt2 = 76.6 \times 14.9 \times 1.7 = 1.9^{\mathrm{ton}}$

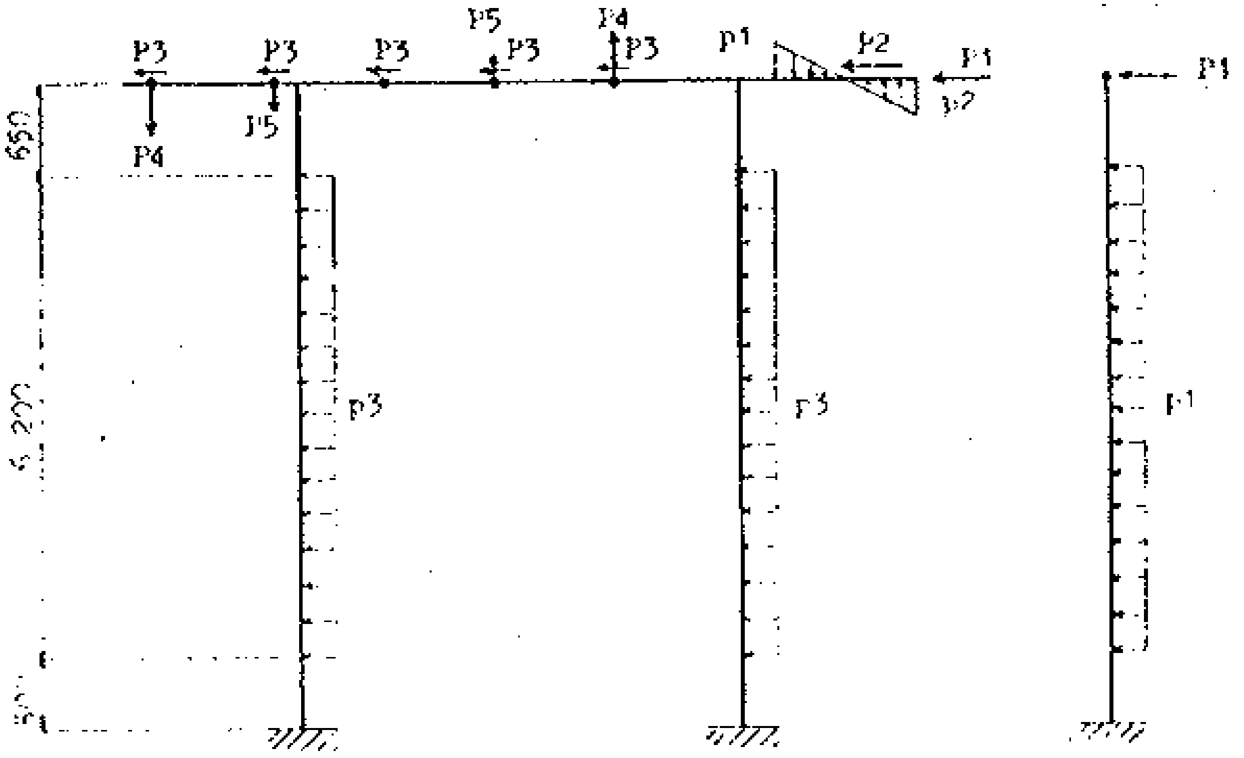
8.) Wind loading diagram

(a) case 1



		unloaded	live loaded
(T)	P1	0.44 t	0.15 t
	P2	1.08 t	1.04 t
	P3	1.98 t	3.17 t
	P4	0.94 t	1.31 t
	p1	0.34 t/m	0.12 t/m
(L)	P1	3.03 t	2.00 t
	p1	0.45 t/m	0.15 t/m

(b) case 2

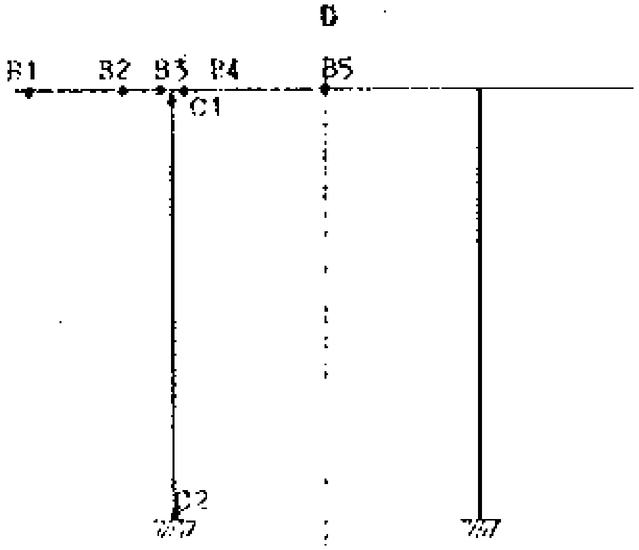


(ア)　　　　(イ)

		unloaded	live loaded
(ア)	P1	0.44 ton	0.15 ton
	P2	5.60 ton	1.96 ton
	P3	0.76 ton	0.94 ton
	P4	0.60 ton	0.91 ton
	P5	0.29 ton	0.43 ton
	p1	23.15 t/m	8.80 t/m
	p2	23.15 t/m	8.80 t/m
	p3	0.34 t/m	0.12 t/m
(イ)	P1	3.05 ton	2.00 ton
	p1	0.45 t/m	0.15 t/m

§ 4 ACTING FORCE TABLE

1.) due to transverse force



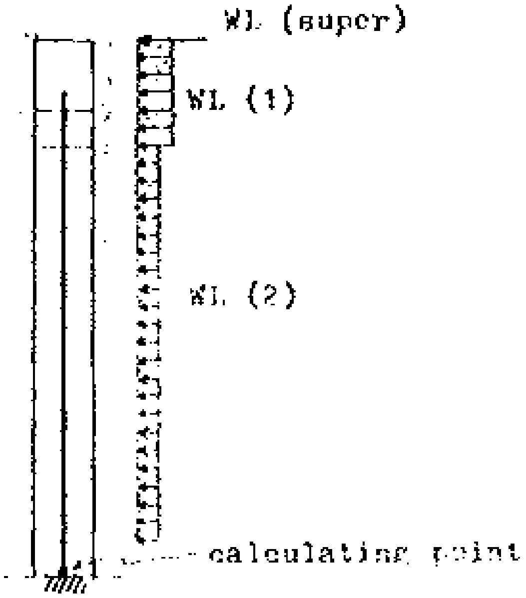
			case	M (tm)	N (ton)	S (ton)	
B1	M max	+					
		-	3	-1.842	-0.000	14.748	1.0
	N max	+					
		-	2	-0.044	-1.820	-50.143	1.0
	S max	+					
		-	3	-0.044	0.000	-101.428	1.0
B2	M max	+					
		-	3	-112.659	-0.000	-175.003	1.0
	N max	+					
		-	2	-31.002	-6.013	38.351	1.0
	S max	+					
		-	3	-112.659	-0.000	-175.003	1.0
B3	M max	+					
		-	3	-158.645	-0.000	-175.292	1.0
	N max	+					
		-	2	-49.353	-6.043	38.895	1.0
	S max	+					
		-	3	-158.645	-0.000	-175.292	1.0

B4	M max.	+					
		-	3	-135.762	-2.046	156.879	1.0
	N max.	+	3	-111.313	9.778	92.100	1.0
		-	5	-83 027	-8.216	-108.906	1.15
	S max.	+	3	-135.900	4.515	156.877	1.0
		-					
B5	M max.	+	3	80.795	0.943	-10.459	1.0
		-					
	N max.	+	3	6.268	9.778	8.824	1.0
		-	5	56.035	-7.031	5.031	1.15
	S max.	+					
		-	5	57.371	1.031	-30.765	1.15
[illegible]	M max.	+					
		-	5	-49.324	-223.823	11.128	1.15
	N max.	+					
		-	3	-22.745	-332.171	4.515	1.0
	S max.	+					
		-	5	37.679	-101.021	-11.467	1.15
C2	M max.	+					
		-	5	-46.982	-113.932	-12.114	1.15
	N max.	+					
		-	3	10.443	-347.018	4.515	1.0
	S max.	+					
		-	5	-46.982	-113.932	-12.114	1.15

These value is divided by "i".

i ; coefficient of increase
in basic stresses

2) due to longitudinal force



	M (tm)	N (ton)	H (ton)	
case-7	92.3	260.3	4.3	1.00
case-8	—	—	—	~~1.25~~
case-9	50.2	260.5	7.3	1.15
case-10	—	—	—	~~1.30~~
case-11	91.6	163.0	5.9	1.00

21.28

CASE 7

	N	H	y	M
WDL	131.69			
WLL (A)	107.36			
FB (A)		4.30	7.50	32.25
W	21.28			
TOTAL	260.33	4.30		32.25

M = 32.3 tm

N = 260.3 ton

H = 4.3 ton

CASE 9

	N	H	y	M
WDL	131.69			
WLL. FB (A)	107.36	4.30	7.50	32.25
W	21.28			
WL (super)		1.30	7.50	9.75
WL (1)		0.70	6.85	4.80
WL (2)		0.95	3.60	3.42
TOTAL	260.33	7.25		50.22

M = 50.2 tm

N = 260.3 ton

H = 7.3 ton

CASE 11

	N	H	y	M
WDL	131.69			
W	21.28			
WL (Super)		1.00	7.50	7.50
WL (1)		2.05	6.85	14.04
WL (2)		2.80	3.60	10.08
TOTAL	152.97	5.85		31.62

M = 31.6 t·m

N = 153.0 ton

H = 5.9 ton

CASE

	N	H	y	M
TOTAL				

M = t·m

N = ton

H = ton

§ 5 CALCULATION OF SECTION

1.) Beam section

	B1 case ③	B2, 3, 4 case ③	B5 case ③
M (tm)	-0.044	-158.645	80.795
N (ton)	-0.000	-0.000	0.943
Q; shear force (ton)	-101.423	-175.292	-10.459
b (cm)	70	70	70
d (cm)	82	115	170
d' (cm)	10	15	10
As (cm^2)		80.4 cm² (5-D32 5-D32 12.5cm pitch)	40.2 cm² (5-D32 12.5cm pitch)
d'/d		0.13	
f=(M/N)+u (cm)		∞	
f/d		∞	
n	15	15	15
nP=n·As/b·d		0.15	0.072
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm²)		12.1	8.0
Q/b·d (kg/cm²)	17.67	21.77	1.25
C		4.82	7.1
S		7.70	15.5
z	1.17	1.17	1.12
σc (kg/cm²)		84	37
σa (kg/cm²)		1975	1860
τ (kg/cm²)	20.7	25.5 ※	1.39
σca (kg/cm²)	85	85	85
σsa (kg/cm²)	2340	2340	2340
τa (kg/cm²)	8.2	8.2	8.2

※ Stirrups $A_v = \frac{\tau \cdot b \cdot a}{\sigma_{sa}} = \frac{25.5 \times 70 \times 12.5}{1480} = 12.54\ cm^2$

$a = 12.5^{cm}$ Used $A_s = \phi 20 \square \times 2 = 4 \times 3.14 = 12.56^{cm^2}$

$> Req\ A_v = 12.56^{cm^2}$

2.) Column section

	case 5	case 9	case
M (tm)	49.524	50.2	
N (ton)	223.873	260.5	
Q;shear force (ton)	11.128	7.3	
b (cm)	70	120	
d (cm)	110	60	
d' (cm)	10	10	
As (cm^2)	24.6cm² (5-D25 12.5cm pitch)	31.8cm² (2-D25 (edge) 7-D20 (inter) 12.5cm pitch)	
d'/d	0.09	0.17	
f=(M/N)+u (cm)	72.1	44.3	
f/d	0.65	0.74	
n	15	15	15
nP=n·As/b·d	0.048	0.066	
M'=M+N·u (tm)	161.4	115.3	
M'/(b·d^2) (%f)	19.1	26.7	
Q/b·d (kg/cm^2)	1.32	0.87	
C	7.5	3.0	
S			
Z			
σc (kg/cm^2)	48	80	
σs (kg/cm^2)			
τ (kg/cm^2)	1.3	0.9	
σca (kg/cm^2)	98	98	
σsa (kg/cm^2)	2691	2691	
τa (kg/cm^2)	9.4	9.4	

§ 6. Calculation of stability

Pier self weight

(a) beam

$$\tfrac{1}{2} \times (1.35 + 2.19) \times 1.0125 = 1.79$$

$$1.10 \times 1.2 = 1.32$$

$$2.19 \times 2.8 = 6.13$$

$$1.10 \times 0.9125 = 1.00$$

$$1.48 \times 0.2875 = 1.42$$

$$\tfrac{1}{2} \times (2.58 + 1.74) \times 1.0125 = 2.19 \qquad \Sigma = 12.86^{\text{ton}}$$

(b) column

$$2.02 \times 7.350 \times 2 = 29.69$$

(c) Footing

$$6.2 \times 3.0 \times 10 \times 2.407 = 44.77^{\text{ton}}$$

$$\Sigma = 12.85 + 29.69 + 44.77 = 87.31^{\text{ton}}$$

Reaction due to superstructure

Dead Load $220.35 + 55.94 = 276.29^{\text{ton}}$

Live load $125.71 + 7.26 + 11\,10 = 144.07^{\text{ton}}$

Longitudinal direction

a) due to braking

Under HA ; $F_B = 25.8 \times 1/3 = 8.6^{ton}$

" HB ; $F_B =$

b) due to wind

- for superstructure

Unloaded $P_{LS} = 1.4$

Live loaded $P_{LL} = 1.2$

$P_{LS} = 0.4$

- for safety fences $P_{L1} = 0.1$

$P_{L2} = 0.5$

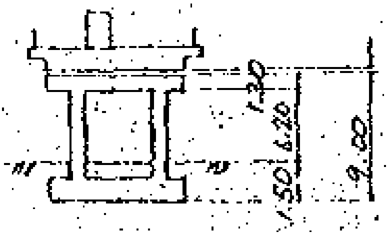
- for pier

Unloaded $P_{L1} = 4.1$

$P_{L2} = 5.6$

Live loaded $P_{L1} = 1.9$

$P_{L2} = 1.9$



Transverse direction

a) due to wind

- for superstructure

Unloaded Pt = 5.6

Live loaded Pt = 6.7

- for safety fences

Pt = 1.3

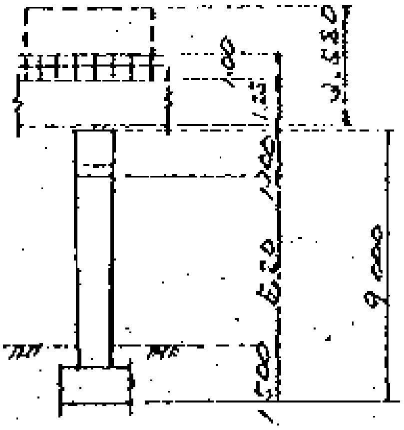
- for pier

Unloaded Pt-1 = 0.7

Pt-2 = 4.2

Live loaded Pt-1 = 0.2

Pt-2 = 1.5



		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	276.29					
	WLL (HA)	149.07			8.6	9.05	77.83
	WLL (HB)						
	Self weight	87.31					
Transverse direction	WL (Unloaded)						
	for superstruct.				5.6	9.665	54.12
	for safety fence				1.3	10.780	14.01
	for pier (1)				0.4	8.350	3.34
	(2)				4.2	4.300	18.06
					Σ 11.5		Σ 89.53
	WL (live loaded)						
	for superstruct (1)				6.7	10.890	72.63
	〃 (2)						
	for pier (1)				0.2	8.350	1.67
	〃 (2)				1.5	4.300	6.45
					Σ 8.40		Σ 80.75
Longitudinal direction	WL (Unloaded)						
	for superstruct.				1.4	9.05	12.67
	for safety fence				0.6	9.05	5.43
	for pier (1)				4.1	8.35	34.24
	(2)				5.6	4.30	24.08
					Σ = 11.7		Σ 76.42
	WL (live loaded)						
	for superstruct. (1)				2.2	9.05	19.91
	〃 (2)				0.4	9.05	3.62
	for pier (1)				1.4	8.35	11.69
	(2)				1.9	4.30	8.17
					Σ = 5.9		Σ 43.39

Longitudinal direction

Case No	Loading combination	N (t)	H (t)	M (t)
1	Dead load only	363.60	0	0
2	Dead load + wind	363.60	11.7	76.42
3	HA Loading	507.67	8.6	77.83
4	~~HB Loading~~			
5	HA loading + wind	507.67	14.5	121.22
6	~~HB Loading + wind~~			

Calculation of Stability

- eccentric $e = M/N$ $B/6 = 0.500$
- 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 $F = \dfrac{N \times 0.6}{H}$

Case No.	B/6 (m)	eccentric	Soil reaction		q_a t/m²	Sliding	
		e (m)	q max (t/m²)	q min (t/m²)		F	Fa
1		0.	19.5	19.5			
2	0.500	0.210	27.8	11.3	40.		
3	0.500	0.153	35.7	18.9	40.		
4							
5	0.500	0.239	40.3	14.3	46	21	1.5
6							

Transverse direction.

Case No	Loading combination	N (t)	H (t)	M (t)
~~1~~	~~Dead load only~~			
2	Dead load + wind	363.60	11.5	89.53
~~3~~	~~HA Loading~~			
~~4~~	~~HB Loading~~			
5	HA loading + wind	507.67	8.4	80.75
~~6~~	~~HB loading + wind~~			

Calculation of Stability

- eccentric $e = M/N$ $B/6 = 1.033$
- 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) $e > B/6$

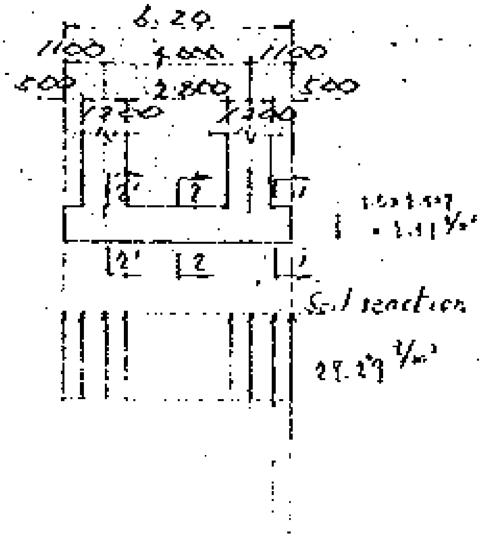
$$q_{max} = \frac{2 \cdot N}{B \times L}$$

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

Case No.	B/6 (m)	eccentric	Soil reaction		q_a t/m²	Sliding	
		e (m)	q max (t/m²)	q min (t/m²)		F	Fa
~~1~~							
2	1.033	0.246	24.2	19.2	90	19	1.5
~~3~~							
~~4~~							
5		0.159	31.5	23.1	46.		
~~6~~							

§7. Calculation of footing section

1 Transverse direction



Sect 1.

$S = (27.29 - 2.41) \times 0.5 \times 3.0 = 37.32^{t}$

$M = 37.32 \times 1/2 \times 0.5 = 9.33^{t\cdot m}$

effective width 2.5^{m}

$S' = 37.32 / 2.5 = 14.93^{ton}$

$M' = 9.33 / 2.5 = 3.73^{t\cdot m}$

Sect. 2

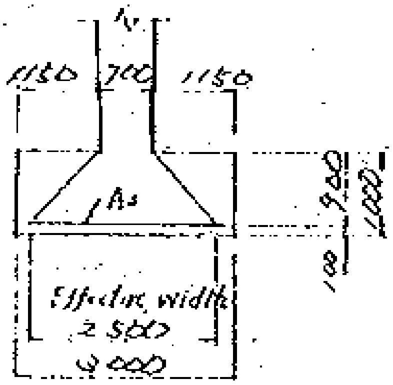
$M = (27.29 - 2.41) \times \frac{1}{8} \times 2.8^{2} \times 3.0 = 73.15^{t\cdot m}$

$M' = 73.15 / 2.5 = 29.26^{t\cdot m}$

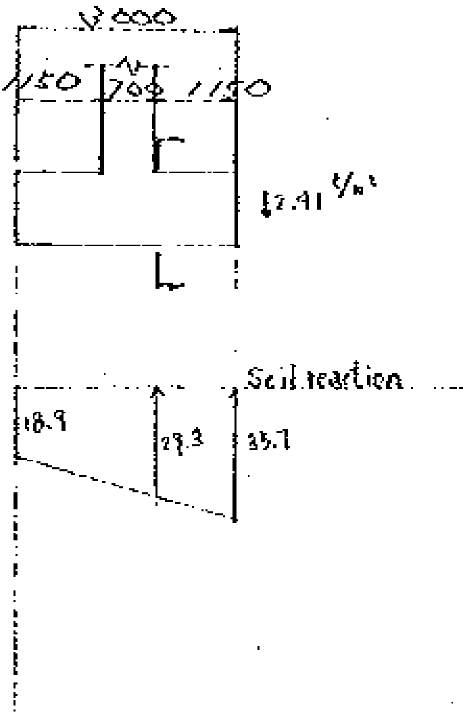
Sect. 2'

$M = (27.29 - 2.41) \times \frac{1}{12} \times 4.0^{2} \times 3.0 = 99.52^{t\cdot m}$

$M' = 99.52 / 2.5 = 39.81^{t\cdot m}$



2. Longitudinal direction



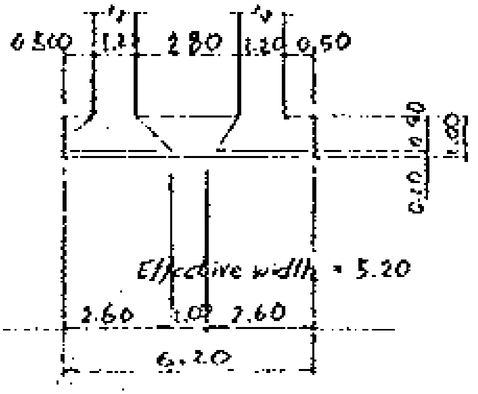
$$S = 2.41 \times 1.15 \times 6.2 \overset{17.18}{} - \tfrac{1}{2} \times (35.7 + 29.3) \times 1.15 \times 6.2 \overset{231.73}{} = -214.55^{t}$$

$$M = 17.18 \times \tfrac{1}{2} \times 1.15 \overset{9.88}{} - 231.73 \times \tfrac{1}{3} \times 1.15 \times \frac{2 \times 35.7 + 29.3}{35.7 + 29.3} \overset{137.62}{} = -127.74^{t \cdot m}$$

effective width = 5.2ᵐ

$$S' = -214.55 \times 1/5.2 = -41.26^{t}$$

$$M' = -127.74 \times 1/5.2 = -24.57^{t \cdot m}$$

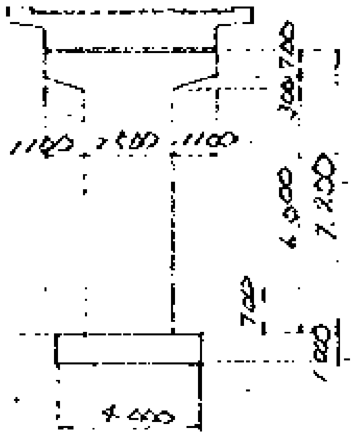


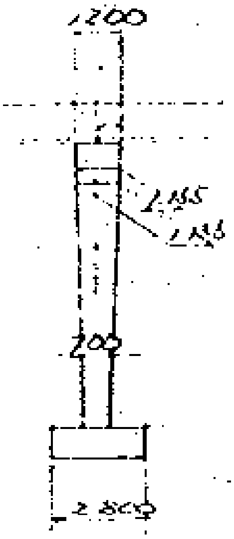
	Transverse direction		Longitudinal direction
	case bottom	case Top	case bottom
M (tm)	39.81	29.26	24.57
N (ton)			
Q:shear force (ton)			
b (cm)	100	100	
d (cm)	90	90	
d' (cm)	10	10	
As (cm^2)	Φ20 - 125 = 25.12	Φ16 - 125 = 16.08	Φ16 - 125 = 16.08
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
np=n·As/b·d	0.042	0.027	0.027
M'=M+N·u (tm)			
$M'/(b \cdot d^2)$ (kg/cm^2)	4.91	3.61	3.03
Q/b·d (kg/cm^2)			
C	8.71	10.3	10.3
S	26.2	39.8	39.8
z			
σ_c (kg/cm^2)	42.8	37.2	31
σ_s (kg/cm^2)	1931	2157	1811
τ (kg/cm^2)			
σ_{ca} (kg/cm^2)	85	85	85
σ_{sa} (kg/cm^2)	2340	2340	2340
τ_a (kg/cm^2)			

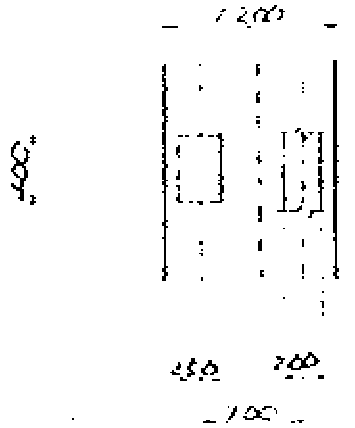
PIER IN No.22 OY.

(P - 1)

§ 1) General dimension







Concrete weight

$4.7 \times 0.7 \times \frac{1}{2}(1.2+1.155) \times 2.407$ $\overset{1.178}{}$ $= 9.33^t$

$\frac{1}{2} \times (4.7+2.5) \times 0.3 \times \frac{1}{2}(1.155+1.136) \times 2.407$ $\overset{1.146}{}$ $= 2.98$

$2.5 \times 6.8 \times \frac{1}{2}(1.136+0.3) \times 2.407$ $\overset{0.718}{}$ $= 17.56$

$3.9 \times 0.6 \times 0.575 \times 2.407$ $= 3.24$

33.11^t

§ 2 Calculation of Load

1) Reaction due to superstructure

	span 15.5	span 7.9	span 5.5
Dead load	69.2	41.6	23.3
HA live (U)	39.9	23.3	21.1
HA live (K)	22.9^t		

2) Transverse Force

2-1) Wind load

(a) For superstructure

- Unloaded

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.28\ ^{m}/_{sec}$

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \times 59.28^2 \times 0.102 = 219.7\ ^{kg}/_{m^2}$

$A = (\overset{1.368}{1.0 + 0.08 + 0.138 + 0.150}) \times (\overset{12.15}{15.8 + 8.5}) \times \frac{1}{2} = 16.62\ ^{m^2}$

$C_D = 1.35 \qquad b/d = 6.7/1.368 =$

$\therefore P_t = 0.2197 \times 16.62 \times 1.35 = 4.93\ ^{t}$

- Live Loaded

$V_c = 35\ ^{m}/_{sec}$

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ ^{kg}/_{m^2}$

$A = (1.368 + 2.5) \times 12.15 = 47.00\ ^{m^2}$

$C_D = 1.45 \qquad b/d = 6.7/2.5 = 2.68$

$P_t = 0.0766 \times 47.00 \times 1.45 = 5.22\ ^{t}$

(b) for safety fence

$V_c = 59.28\ ^{m}/_{sec}$

$q = 0.2197\ ^{t}/_{m^2}$

$A = (0.150 \times 12.15 + 1.15 \times 0.05 \times \frac{12.15}{2}) \times 2 = 4.16\ ^{m^2}$

$C_D = 1.1$

$P_t = 0.2197 \times 4.16 \times 1.1 = 1.01\ ^{ton}$

for pier

- Unloaded

$\gamma = 0.2197\ ^{t}/_{m^2}$

$A_1 = \frac{1}{2} \times (1.200 + \overset{1.168}{1.136}) \times 1.00 = 1.17\ m^2$

$A_2 = \frac{1}{2} \times (1.136 \overset{0.918}{\times 1.10}) \times 6.1 = 5.60\ m^2$

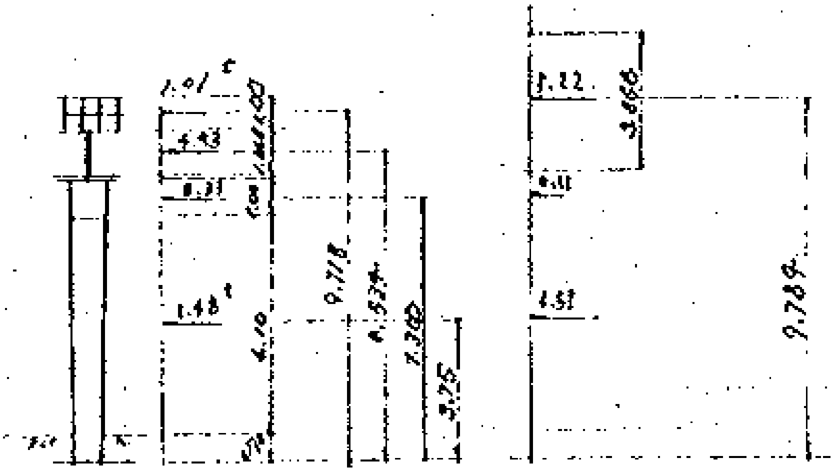
$C_d = 1.2$ $\qquad {}^{t}/_{b} = {}^{2.5}/_{0.918} = 2.7$

$P_k = 0.2197 \times \overset{1.264}{1.2} \times (1.17 + 5.60) = 1.79^{t}$

- live loaded

$\gamma = 0.0766\ ^{t}/_{m^2}$

$P_k = 0.0766 \times 1.2 \times (1.17 + 5.60) = 0.62^{t}$



3) Longitudinal Force

3-1) Braking load

$FB = 100^{kN} + 17^{kN}(\ell - 3.0)$

$= 100 + 17(15.5 - 3.0) = 312.5^{kN} > 253^{kN}$

$\therefore FB' = 1/2 \times 253^{kN} = 126.5^{kN} = 12.9^{ton}$

2) Wind load

Refer to Transeverse wind load.

a) For superstructure

- Unloaded $P_L = 0.25\,P_X = 0.25 \times 4.93 = 1.23^{t}$
- Live loaded $P_{LL} = 0.5\,P_X$

$= \{0.0766 \times (2.5 \times 12.15) \times 1.45\} \times 0.5 = 1.67$

$P_{LS} = 0.25\,P_X$

$= \{0.0766 \times (1.369 \times 12.15) \times 1.35\} \times 0.25 = 0.34$

b) For safety fence

Vertical member

$P_{L1} = 0.8\,P_X$

$= 0.8 \times \{0.2197 \times (0.85 \times 0.05 \times \frac{12.15}{2} \times 2) \times 1.1\} = 0.10^{t}$

Longitudinal member

$P_{L2} = 0.4\,P_X$

$= 0.4 \times \{0.2197 \times (0.160 \times 12.15 \times 2) \times 1.1\} = 0.35^{t}$

c) For pier

- Unloaded

$q = 0.2197 \ {}^{t}/_{m^2}$

$A_1 = 4.7 \times 1.0 = 4.70 \ m^2$

$A_2 = 2.5 \times 6.1 = 15.25 \ m^2$

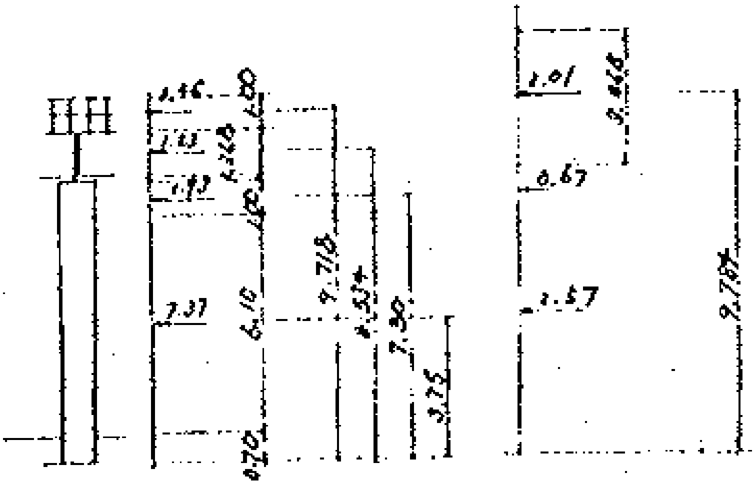
$C_D = 2.2$

$P_L = 0.2197 \times 2.2 \times (4.0 + 15.25) = 9.30^{t}$

- Live load.

$q = 0.0766 \ {}^{t}/_{m^2}$

$P_L = 0.0766 \times 2.2 \times (4.0 + 15.25) = 3.24^{t}$

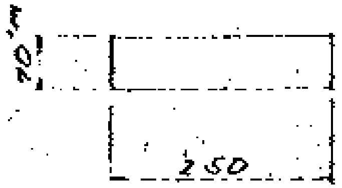


§3. Calculation of Wall Section

1). Longitudinal direction

	N	x	N·x	H	y	H·y
W.L.L.	33.40					
W.D.L.	92.50					
Ws	53.11					
Wind (unload)				0.45	7.8	3.51
				1.23	7.8	9.59
				1.93	7.30	14.09
				7.37	3.75	27.64
Wind (loaded)				2.01	7.8	15.68
				0.67	7.30	4.89
				2.57	3.75	9.64
FB				12.90	7.8	100.62
Dead + wind	145.61			10.98		69.93
Dead + Live	229.01			12.90		100.62
Dead + Live + W	229.01			23.15		130.33

$e = \frac{M}{N} = \frac{100.62}{229.01} = 0.439$ $\qquad \frac{l}{t} = \frac{229}{60} = 1.15$



$f = 0.439 + 0.250 = 0.689^{m}$

$M' = 100.62 + 229.01 \times 0.25 = 157.87^{t\cdot m}$

$A_s = D25 - 150^{ctc} = 16 \times 4.91 = 78.56^{cm^2}$

$np = 15 \times 152.76 \times \frac{1}{250 \times 60} = 0.079$ $\qquad c \to 3.95$, $s \to 80$

$\sigma_c = \frac{15787000}{250 \times 60^2} \times 3.95 = 69^{kg/cm^2}$

$\sigma_s = \text{———} \times 1.0 \times 15 = 1052^{kg/cm^2}$

2.) Transverse direction

	N	x	N·X	H	Y	H·Y
WDL	92.50					
WLL	83.40					
WE	53.11					
Wind (unloaded)				1.01	9718	9.82
				4.93	8.534	42.07
				0.31	7.30	2.26
				1.48	3.75	5.55
Wind (loaded)				5.22	9.784	51.07
				0.11	7.30	0.80
				0.61	3.75	1.91
Dead+wind	115.61			7.73		59.70
Dead+Live	229.01					
Dead+Live+W	229.01			5.89		53.78

§4. Calculation of Stability

Footing Weight

$$W_{of} = 4.0 \times 2.8 \times 1.0 \times 2.407 = 26.96^{t}$$

Dead + Wind.

$$N = 145.61 + 26.96 = 172.57^{t}$$

$$H = 10.98$$

$$M = 10.98 \times 1.0 + 54.83 = 65.81^{t \cdot m}$$

$$e = M/N = 0.381 < 0.467 = B/6$$

$$\sigma = \frac{172.57}{4.0 \times 2.8} \times \left(1 \pm \frac{6 \times 0.381}{2.8}\right) = \begin{cases} 28.0 \\ 2.8 \end{cases} t/m^2$$

Dead + Live

$$N = 229.01 + 26.96 = 255.97^{t}$$

$$H = 12.90$$

$$M = 12.90 \times 1.0 + 100.62 = 113.52^{t}$$

$$e = 0.443 < 0.467$$

$$\sigma = \frac{255.97}{4.0 \times 2.8} \times \left(1 \pm \frac{6 \times 0.443}{2.8}\right) = \begin{cases} 44.6 \\ 1.1 \end{cases} t/m^2$$

Dead + Live + Wind

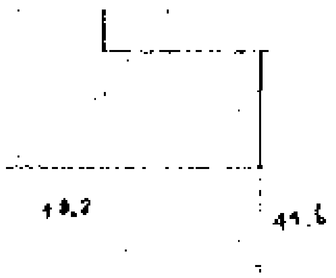
$$N = 255.97 \qquad H = 18.15$$

$$M = 130.83 + 18.15 \times 1.0 = 148.98^{t \cdot m}$$

$$e = 0.582 \qquad x = \left(\frac{B}{2} - e\right)3 = 2.455$$

$$\sigma = \frac{2 \times 255.97}{4.0 \times 2.455} = 52.1 \ t/m^2$$

§5. Calculation of Footing Section



$P_1 = \frac{1}{2} \times (44.6 + 28.3) \times 1.05 \times 4.0 = 153.09^t$

$P_2 = 1.05 \times 1.0 \times 4.0 \times 2.407 = 10.11^t$

142.98^t

$M_1 = 153.09 \times \left(\overset{0.516}{\frac{44.6 \times 2 + 28.3}{44.6 + 28.3}} \times \frac{1}{3} \times 1.05 \right) = 86.36^{t \cdot m}$

$M_2 = \frac{1}{2} \times 1.05 \times 10.11 = 5.31^{t \cdot m}$

81.05^t

effective width 4.0^m

$S = 142.98 / 4.0 = 35.75^t$

$M = 81.05 / 4.0 = 20.26^{t \cdot m}$

$A_s = \Phi 16 - 125^{ctoc} = 2.01 \times 8 = 16.08$

$np = 0.077 \qquad x = 10.3 \quad d = 29.8$

$\sigma_c = \frac{20.26 \times 10^5}{100 \times 98^{??}} \times 10.3 = 26 \ {}^{kg}/{}_{cm^2}$

$\sigma_s = \quad " \quad \times 15 \cdot 29.8 = 1493$

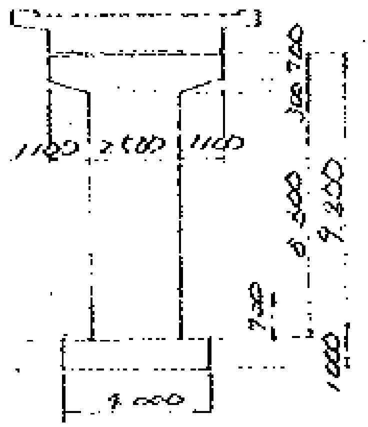
Transverse direction

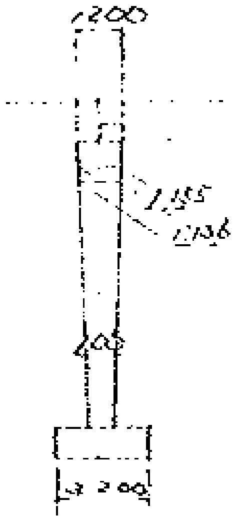
Minimum $A_s = \Phi 16^{mm} - ctc\ 125^{mm}$

PIER IN No.22 OV.

(P - 3)

§1. General dimension





Concrete weight

$\overset{1.177}{}$

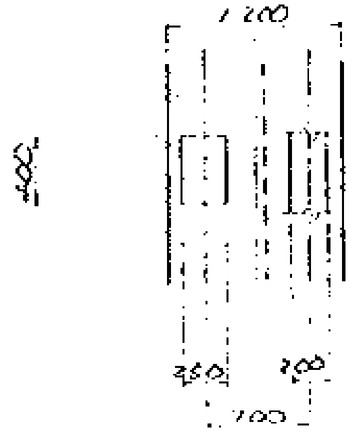
$4.7 \times 0.7 \times \frac{1}{2}(1.2 + 1.155) \times 2.407 = 9.33^{t}$

$\frac{1}{2} \times \overset{3.6}{(4.7+2.5)} \times 0.3 \times \frac{1}{2}\overset{1.146}{(1.155+1.136)} \times 2.407 = 2.98$

$2.5 \times 6.5 \times \frac{1}{2}\overset{0.918}{(1.136+0.7)} \times 2.407 = 33.61$

$3.9 \times 0.6 \times 0.575 \times 2.407 = 3.24$

49.16^{t}



§2. Calculation of Load

1) Reaction due to superstructure

	span 15.5	span 7.9	span 5.5
Dead load	69.2	41.6	13.3
HA Live (U)	39.9	23.3	21.1
HA live (K)		22.9 ^{t}	

2) Transverse Force

2-1) Wind load

(a) For superstructure

- Unloaded

$V_c = 38 \times 1.0 \times 1.0 \times 1.56 = 59.28\ ^{m}/_{sec}$

$P_L = q \cdot A \cdot C_D$

$q = 0.613 \times 59.28^2 \times 0.102 = 219.7\ ^{kg}/_{m^2}$

$A = (\overset{1.368}{1.0 + 0.08 + 0.138 + 0.150}) \times (\overset{12.15}{15.8 + 8.5}) \times \frac{1}{2} = 16.62\ m^2$

$C_D = 1.35$ $\qquad b/d = 6.7/1.368 =$

$\therefore P_L = 0.2197 \times 16.62 \times 1.35 = 4.93\ ^{t}$

- live Loaded

$V_c = 35\ ^{m}/_{sec}$

$q = 0.613 \times 35^2 \times 0.102 = 76.6\ ^{kg}/_{m^2}$

$A = (1.368 + 2.5) \times 12.15 = 47.00\ m^2$

$C_D = 1.45$ $\qquad b/d = 6.7/2.5 = 2.68$

$P_L = 0.0766 \times 47.00 \times 1.45 = 5.22\ ^{t}$

(b) for safety fence

$V_c = 59.28\ ^{m}/_{sec}$

$q = 0.2197\ ^{t}/_{m^2}$

$A = (0.150 \times 12.15 + 0.85 \times 0.05 \times \frac{12.15}{2}) \times 2 = 4.16\ m^2$

$C_D = 1.1$

$P_L = 0.2197 \times 4.16 \times 1.1 = 1.01\ ^{ton}$

for pier

- Unloaded

$\gamma = 0.2197 \ t/m^2$

$A_1 = \frac{1}{2} \times (1.200 + \overset{1.163}{1.136}) \times 1.00 = 1.17 \ m^2$

$A_2 = \frac{1}{2} \times (1.136 \overset{0.918}{\times 0.70}) \times 8.1 = 7.44 \ m^2$

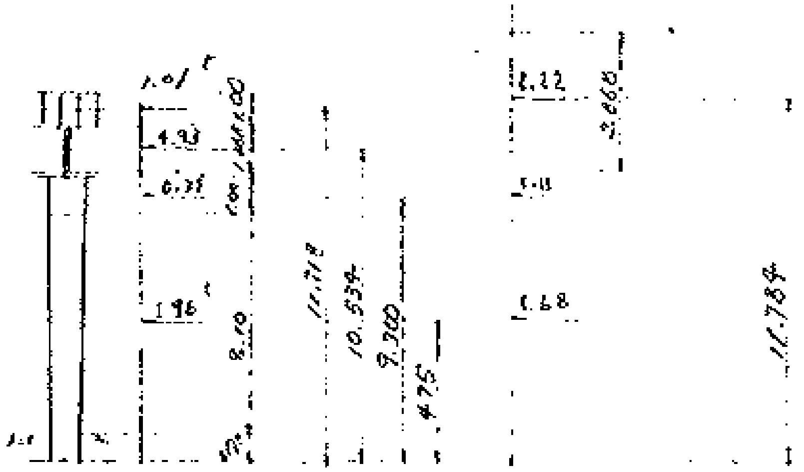
$C_D = 1.2$ $\qquad t/d = 2.5/0.918 = 1.7$

$P_t = 0.2197 \overset{2.264}{\times 1.2} \times (1.17 + 7.44) = 2.27 \ t$

- Live loaded

$\gamma = 0.0766 \ t/m^2$

$P_t = 0.0766 \times 1.2 \times (1.17 + 7.44) = 0.79 \ t$



3) Longitudinal Force.

3-1) Braking load

$$FB = 100^{kN} + 17^{kN}(\ell - 3.0)$$

$$= 100 + 17\,(15.5 - 3.0) \qquad = 312.5^{kN} > 253^{kN}$$

$$\therefore FB' = 1/2 \times 253^{kN} = 126.5^{kN} = 12.9^{ton}$$

2) Wind load

Refer to Transeverse wind load

a) for superstructure

- Unloaded $\quad P_L = 0.25\,P_X = 0.25 \times 4.93 = 1.23^{t}$
- Live loaded $\quad P_{L2} = 0.5\,P_X$

$$= \{0.0766 \times (2.5 \times 12.15) \times 1.15\} \times 0.5 = 1.67$$

$$P_{L3} = 0.25\,P_X$$

$$= \{0.0766 \times (1.369 \times 12.15) \times 1.35\} \times 0.25 = 0.34$$

b) For safety fence

Vertical member

$$P_{L1} = 0.8\,P_X$$

$$= 0.8 \times \{0.2197 \times (0.85 \times 0.05 \times \frac{12.15}{2} \times 2) \times 1.1\} = 0.10^{t}$$

Longitudinal member

$$P_{L2} = 0.4\,P_X$$

$$= 0.4 \times \{0.2197 \times (0.150 \times 12.15 \times 2) \times 1.1\} = 0.35^{t}$$

c) For pier

- Unloaded

$g = 0.2197 \ t/m^2$

$A_1 = 4.7 \times 1.0 = 4.70 \ m^2$

$A_2 = 2.5 \times 8.1 = 20.25 \ m^2$

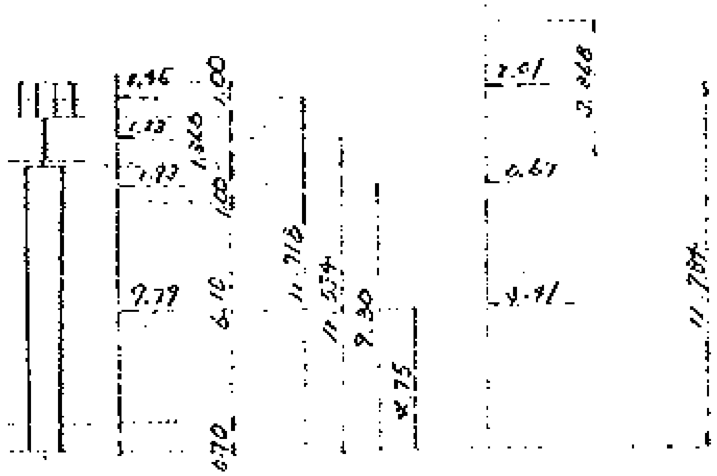
$C_D = 2.2$

$P_r = 0.2197 \times 2.2 \times (4.0 + 20.25) = 11.72 \ t$

- Live load

$g = 0.0766 \ t/m^2$

$P_r = 0.0766 \times 2.2 \times (4.0 + 20.25) = 4.08 \ t$

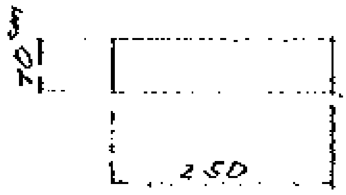


33. Calculation of Wall Section

1) Longitudinal direction

	N	x	N·x	H	y	H·y
W.L.L	83.60					
W.D.L	110.80					
Ws	64.16					
Wind (unload)				0.45	9.8	4.41
				1.23	9.8	12.05
				1.93	9.30	17.95
				9.79	4.75	46.50
Wind (loaded)				2.01	9.8	19.70
				0.67	9.30	6.23
				3.41	4.75	16.20
FB				12.90	9.8	126.42
Dead + wind	174.96			13.40		80.91
Dead + Live	260.56			12.90		126.42
Dead + Live + W	260.56			18.99		168.55

$e = \frac{M}{N} = \frac{168.55}{260.56} = 0.647 \qquad \frac{f}{t} = \frac{0.897}{0.60} = 1.5$



$f = 0.647 + 0.250 = 0.897^{m}$

$M' = 168.55 + 260.56 \times 0.25 = 233.69^{t\,m}$

$A_s = D25 - 150^{ctc} = 16 \times 8.04 = 128.64^{cm^2}$

$np = 15 \times 128.64 / 250 \times 60 = 0.129 \qquad c \to 3.6,\; s \to 3.9$

$\sigma_c = \frac{23369000}{250 \times 60^2} \times 3.60 = 93^{kg/cm^2} < 83 \times 1.15 = 95^{kg/cm^2}$

$\sigma_s = \cdots \times 3.9 \times 15 = 1519^{kg/cm^2}$

2.) Tranceverse direction

	N	x	N·X	H	y	H·Y
W.DL	85.60					
W.LL	110.80					
Ws	64.16					
Wind (unloaded)				1.01	11.718	11.84
				4.93	10.534	51.93
				0.31	9.32	2.88
				1.96	4.75	9.31
Wind (loaded)				5.22	11.729	61.31
				0.11	9.10	1.02
				0.68	4.75	3.23
Dead + wind	149.76			8.21		75.96
Dead + Live	260.56					
Dead + Live + W	260.56			6.01		65.16

34. Calculation of Stability

Footing weight

$W_{sf} = 4.0 \times 3.2 \times 1.0 \times 2.407 = 30.81^{t}$

Dead + Wind.

$N = 174.96 + 30.81 = 205.77^{t}$ $\quad H = 13.90^{t}$

$M = 13.90 \times 1.0 + 80.91 = 94.81^{t\cdot m}$

$e = 0.458 < 0.533$

$\sigma = \dfrac{205.77}{4.0 \times 3.2} \times \left(1 \pm \dfrac{6 \times 0.458}{3.2}\right) = \begin{cases} 29.9 \\ 2.3 \end{cases} {}^{t}/_{m^2}$

Dead + Live.

$N = 260.56 + 30.81 = 291.37$ $\quad H = 12.90$

$M = 12.90 \times 1.0 + 126.42 = 139.32^{t\cdot m}$

$e = 0.478 < 0.533$

$\sigma = \dfrac{291.37}{4.0 \times 3.2} \times 1 \pm \dfrac{6 \times 0.478}{3.2} = \begin{cases} 43.2 \\ 2.4 \end{cases} {}^{t}/_{m^2}$

Dead + Live + Wind

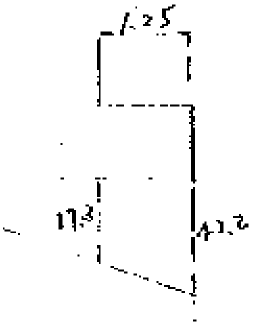
$N = 291.37$ $\quad H = 18.99$

$M = 18.99 \times 1.0 + 168.55 = 187.54^{t\cdot m}$

$e = 0.644^{m} > 0.533$ $\quad x = \left(\frac{B}{2} - e\right)3 = 2.869^{m}$

$\sigma = \dfrac{2 \times 291.37}{2.869 \times 4.0} = 50.8 \; {}^{t}/_{m^2}$

§5. Calculation of Footing Section



$P_1 = \frac{1}{2} \times (43.2 + 27.3) \times 1.25 \times 4.0 = 176.25^{t}$

$P_2 = 1.25 \times 1.0 \times 4.0 \times 2.407 = 12.04^{t}$

164.21^{t}

$M_1 = 176.25 \times \frac{2 \times 43.2 + 27.3}{43.2 + 27.3} \times \frac{1}{3} \times 1.25 = 118.44^{t \cdot m}$

$M_2 = 12.04 \times \frac{1}{2} \times 1.25 = 7.53^{t \cdot m}$

$110.91^{t \cdot m}$

Effective width

$S' = 164.21 / 4.0 = 41.1^{t}/_{m}$

$M' = 110.91 / 4.0 = 27.73^{t \cdot m}/_{m}$

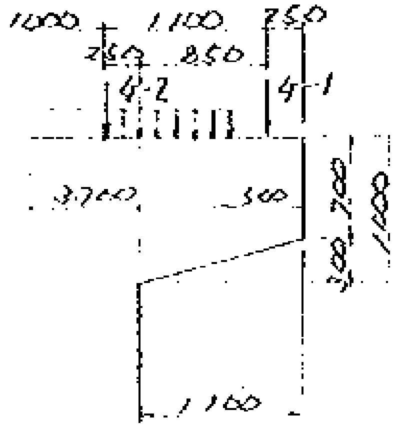
$A_s = 4 \cdot 16 - 125^{c/c} = 2.01 \times 8 = 16.08$

$np = 0.027 \qquad C = 10.3 \qquad S = 39.8$

$\sigma = \frac{2773000}{100 \times 90^2} \times 10.3 = 35 \ kg/cm^2$

$\sigma_s = \text{-----}''\text{-----} \times 39.8 \times 15 = 2044 \ kg/cm^2$

§6. Calculation of Cantilever beam



Reaction due to superstructure

		4-1	4-2
Voided slab	Dead load	17.8	10.4
	live Load	18.5	11.0
		10.9	8.9
	Total	47.2	30.3

Solid slab

Dead load $41.6 / 3.7 = 11.24 \ ^{t}/_{m}$

live load $23.3 / 3.7 = 6.30 \ ^{t}/_{m}$

$17.54 \ ^{t}/_{m}$

Concrete Weight

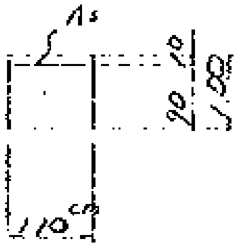
$$W = 1/2 \times (0.7+1.0) \times 1.1 \times \overset{\text{width}}{1.1} \times 2.407 = 2.48^{t}$$

Sect.-1

$$S = 47.2 + \overset{10.52}{17.54 \times 0.6} + 2.48^{t} = 60.2^{T}$$

$$M = \overset{40.1}{47.2 \times 0.85} + \overset{3.16}{10.52 \times 0.3} + \overset{1.28}{2.48 \times \frac{1}{3} \times 1.1 \times \frac{1.0+2\times0.7}{1.0+0.7}}$$

$$= 44.56^{t}$$



$A_s = \phi 25 \cdot 125^{ctc} = 8 \times 4.91 = 39.28^{cm^2}$

$np = 0.060 \quad c = 7.60 \quad s = 18.5$

$\sigma_c = \frac{44.56 \times 10^5}{110 \times 90^2} \times 7.60 = 42 \ ^{kg}/_{cm^2}$

$\sigma_s = \text{〃} \times 16.5 \times 15 = 1526 \ ^{kg}/_{cm^2}$

$\tau = \frac{60200 \times 1.11}{110 \times 90} = 6.7 \ ^{kg}/_{cm^2} < \tau_0 = 8.2 \ ^{kg}/_{cm^2}$

Min $A_v = 0.0012 \times b \times a = 0.0012 \times 110 \times 12.5 = 2.55^{cm^2}$

Used stirrup $\phi 16$ ctc 125 $= 4 \times 2.01 = 8.04^{cm^2}$

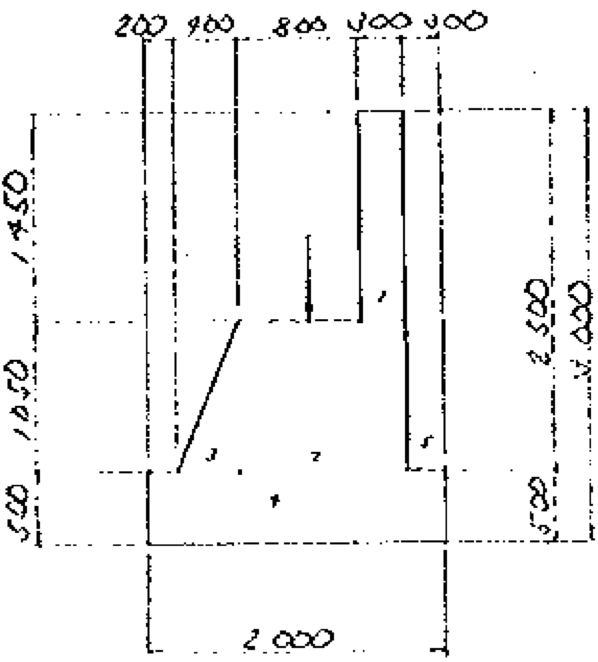
Vol. 5 Gravity type abutment

§ 1 Abutment Height $H = 3.0^{m}$

§ 2 " $H = 3.5^{m}$

§ 3. " $H = 4.0^{m}$

§ 1 Abutment height 3.0^m



			N	x	
1	0.300 × 1.450	× 2.407	1,047	1.550	1.623
2	1,100 × 1,050	× 2.305	2.662	1.150	3.062
3	½ × 0.400 × 1.050	× 2.305	0.484	0.467	0.226
4	2.00 × 0.500	× 2.305	2.305	1.000	2.305
5	0.30 × 2.50	× 1.900	1.425	1.850	2.636
			7.923	1.243	9.852

Surcharge

Under HA $= 1.02 \times 0.6 = 0.612^{t}$

HB $= 1.66 \times 0.6 = 0.996^{t}$

Earth pressure

$Pa = \frac{1}{2} \times 0.271 \times 1.9 \times 3.0^{2} = 1.317^{t}$

Earth pressure due To surcharge

Under HA $Pa\,add = 1.02 \times 0.271 \times 3.0 = 0.829^{t}$

" HB $Pa\,add = 1.66 \times 0.271 \times 3.0 = 1.350^{t}$

Longitudinal force

Under HA $L.F = 100^{kN} + 17\,(L - 3.0)$

$= 100^{kN} + 17 \times (8.0 - 3.0) = 185^{kN} = 18.87^{t}$

Under HB $L.F = 375^{kN} = 38.25^{t}$

Therefor

Under HA $L.F = 1/9.6 \times 18.87 = 1.97$

HB $L.F = 1/9.6 \times 38.25 = 3.98$

	N	x	N x	H	y	H·y
WDL + WLL	11.150	1.050	11.708			
L.F				1.97	1.60	3.152
W + Ws	7.923		9.852			
Ws add	0.612	1.700	1.040			
Pa				2.317	1.00	2.317
Pa add				0.829	1.50	1.244
	19.685		22.600	5.116		6.713

1). Check for eccentricity

$$x = \frac{22.600 - 6.713}{19.685} = 0.807$$

$$e = B/2 - x = 1.000 - 0.807 = 0.193 < B/6 = 0.333.$$

2). Soil Reaction

$$q = \frac{19.685}{2.00} \times \left(1 \pm \frac{6 \times 0.193}{2.00}\right) = \begin{cases} 15.5 \\ 4.1 \end{cases} \ t/m^2$$

3) Check for sliding

$$F = \frac{0.6 \times 19.685}{5.116} = 2.3 > 1.5$$

	N (t)	x (m)	N x (t·m)	H (t)	y (m)	H·y (t·m)
WDL + WLL	14.01	1.050	14.71			
F.B				3.98	1.60	6.368
N + Ws	7.923		9.852			
Ws add	0.996	1.700	1.693			
Pa				2.317	1.000	2.317
Pa add				1.350	0.829	1.119
total	22.929		26.256	7.647		9.804

1) Check for eccentricity

$$x = \frac{26.256 - 9.804}{22.929} = 0.717$$

$$e = B/2 - x = 1.000 - 0.717 = 0.282 \quad < \frac{B}{6} = 0.333$$

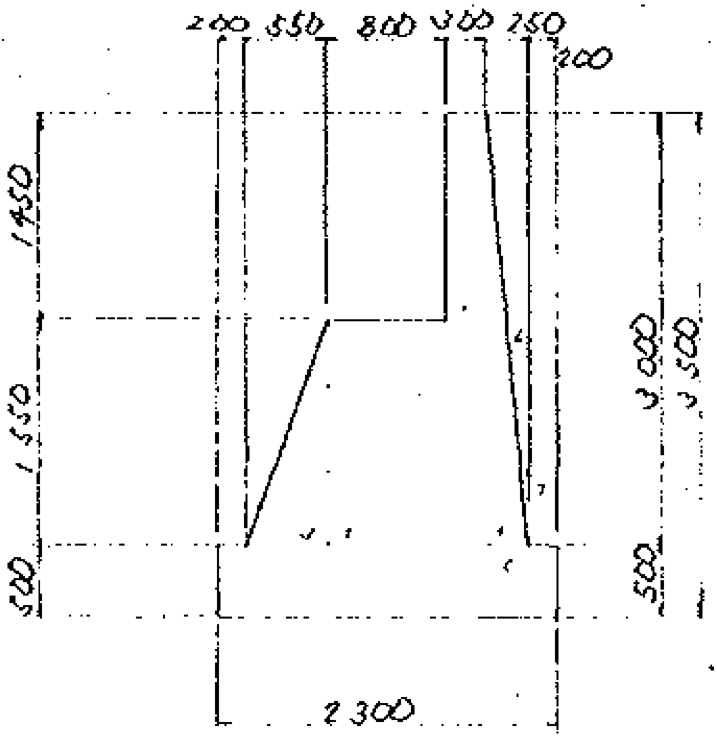
2) Soil Reaction

$$q = \frac{22.929}{2.0} \times \left(1 \pm \frac{6 \times 0.282}{2.0}\right) = \begin{cases} 21.2 \\ 1.2 \end{cases} \ t/m^2$$

3) Check for sliding

$$f = \frac{0.6 \times 22.929}{7.647} = 1.8$$

§ 2. Abutment height 3.5 ^{m}



		N	x	N·x
1	0.300 × 1.450 × 2.407	1.047	1.700	1.780
2	1.100 × 1.550 × 2.305	3.930	1.300	5.109
3	1/2 × 0.55 × 1.55 × 2.305	0.983	0.567	0.557
4	1/2 × 0.25 × 3.00 × 2.305	0.864	1.933	1.671
5	2.300 × 0.500 × 2.305	2.651	1.150	3.048
6	1/2 × 0.25 × 3.00 × 1.900	0.713	2.017	1.437
7	0.200 × 3.00 × 1.900	1.140	2.200	2.508
		11.378	1.422	16.110

Surcharge

Under HA = 1.02 × 0.750 = 0.765^{t}

HB = 1.66 × 0.750 = 1.245

Earth pressure

$P_a = 1/2 \times 0.271 \times 1.9 \times 3.5^2 = 3.154^{t}$

Earth pressure due to surcharge

Under HA $P_{a\,add} = 1.02 \times 0.271 \times 3.5 = 0.967^{t}$

HB $P_{a\,add} = 1.66 \times 0.271 \times 3.5 = 1.575^{t}$

Longitudinal force

Under HA L.F = 1.97^{t}

HB L.F = 3.78^{t}

	N (t)	x (m)	N x (t·m)	H (t)	y (m)	H y (t·m)
WDL + WLL	11.150	1.200	13.380			
F.B				1.97	2.1	4.137
N + WS	11.328		16.110			
Ws add	0.765	1.925	1.473			
PA				3.154	1.167	3.680
Pa add				0.967	1.750	1.692
total	23.243		30.963	6.091		9.509

1) Check for eccentricity

$$x = \frac{30.963 - 9.509}{23.243} = 0.923$$

$$e = B/2 - x = 1.150 - 0.923 = 0.227^{m} < \frac{B}{6} = 0.383$$

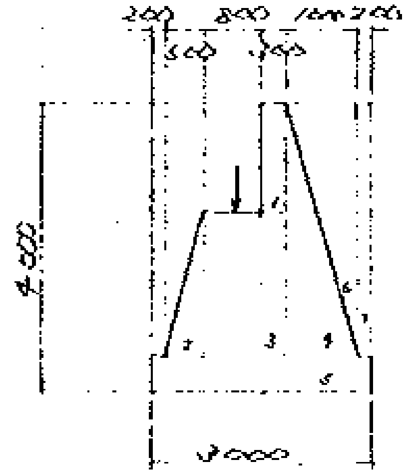
2) Soil Reaction

$$q = \frac{23.243}{2.3}\left(1 \pm \frac{6 \times 0.227}{2.3}\right) = \left\{\begin{matrix} 16.1 \\ 4.1 \end{matrix}\right. \quad t/m^2$$

3) Check for sliding

$$F = \frac{0.6 \times 23.243}{6.091} = 2.3 > 1.5$$

§ 3. Abutment height 4.0 m



①	0.3 × 1.5 × 2.407	=	1.083 ×	1.650	1.787
	½ × 0.5 × 2.0 × 2.305	=	1.153	0.533	0.614
	1.1 × 2.0 × 2.305	=	5.071	1.250	6.339
	½ × 3.5 × 1.0 × 2.305	=	4.034	2.133	8.604
	0.5 × 3.0 × 2.305	=	3.458	1.500	5.186
	½ × 3.5 × 1.0 × 1.9	=	3.325	2.467	8.202
	0.2 × 3.5 × 1.9	=	1.330	2.900	3.857
			19.454		34.589

Reaction due to superstructure

Dead Load $103.4 \ / 6.7^{m} = 15.43 \ ^{t/m}$

Live Load $82.8 \ / 6.7 = 12.36 \ ^{t/m}$

Longitudinal Force

$LF = 100 + 17\,(20.5 - 3.0) = 397.5 > 253^{kN} = 25.806^{t}$

$L.F = 25.806 \times \frac{1}{2} \times 1/6.7 = 1.926^{t}$

Temperature and shrinkage

$\delta = 20.5 \times 0.7 \times 1/2 = 7.175^{mm}$

$P_H = \dfrac{G \cdot A \cdot \delta}{\Sigma \cdot t_e} = \dfrac{13.5 \times 1452.2 \times 0.72}{2.5^{cm}} = 5646^{kg} = 5.6^{t}$

$\Sigma P_H = 5.6 \times 4 = 22.6^{t}$

$P_H = 22.6 / 6.7 = 3.37^{t}$

Surcharge

$W_{surch.} = 1.5^{m} \times 1.02^{t/m^2} = 1.53^{t}$

Earth pressure

$P_a = 1/2 \times 0.271 \times 4.0^2 \times 1.9 = 4.119$

$P_{a\,add} = 1.02 \times 4.0 \times 0.271 = 1.106$

	N	x	N·x	H	y	H·y
WDL WLL	27.79	1.15	31.959			
T.B				1.726	2.50	4.315
Temp.				3.370	2.50	8.425
N.Ws	19.454		34.589			
Ws add	1.530	2.25	3.443			
Pa				4.479	1.333	5.971
Pa add				1.106	2.000	2.212
	48.774		69.991			20.923

$$x = \frac{69.991 - 20.923}{48.774} = 1.006$$

$$e = 1.5 - 1.006 = 0.494 < B/6 = 0.5$$

$$\lambda = \frac{48.774}{3.0} \times \left(1 \pm \frac{6 \times 0.494}{3.0}\right) = \begin{cases} 32.3 \\ 0.2 \end{cases}$$

Contents

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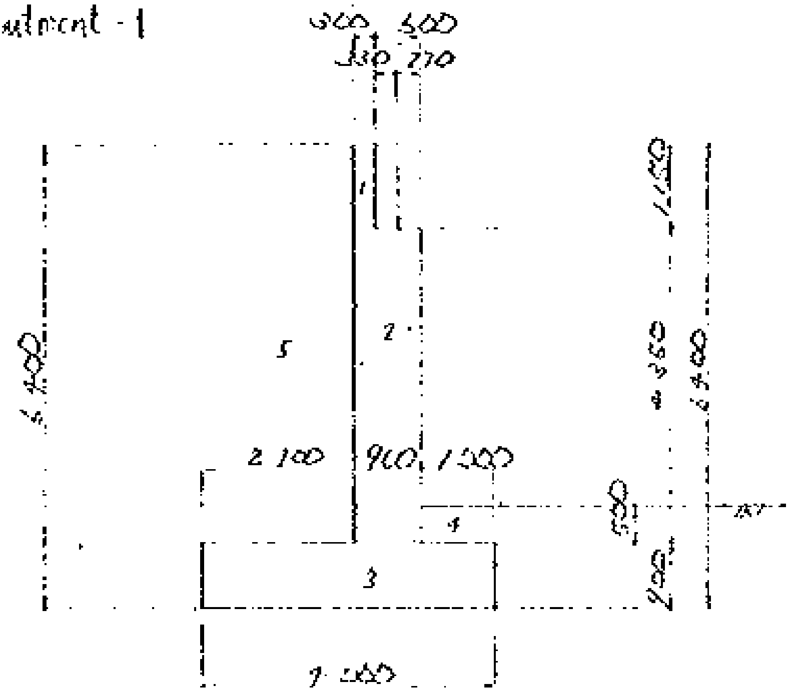
§1. Abutment - 1



		N (t)	x (m)	N · x
1	0.30 × 1.15 × 3.00 × 2.407	2.49	1.750	4.36
2	0.90 × 4.35 × 3.00 × 2.407	28.27	1.450	40.99
3	4.0 × 0.90 × 3.00 × 2.407	26.00	2.000	52.00
4	1.00 × 0.50 × 3.00 × 1.9	2.85	0.500	1.43
5	2.10 × 5.5 × 3.00 × 1.9	65.84	2.950	194.23
	Σ	125.45		293.01

Calculation of Stability

	N (t)	x (m)	N·x (t m)	H (t)	y (m)	H·y (t m)
Body	30.64	1.270	38.91			
Temp				5.64	5.250	29.61
W+Ws	125.45		292.01			
Ws add	3.67	2.800	10.28			
Pa				31.64	2.133	67.49
Pa add				2.65	3.200	8.48
	159.76		342.20	39.93		105.58

1) check for eccentric

$$x = \frac{\Sigma N\cdot x - \Sigma H\cdot y}{\Sigma N} = \frac{342.20 - 105.58}{159.76} = 1.481$$

$$e = \frac{B}{2} - x = \frac{4.00}{2} - 1.481 = 0.519 < \frac{B}{6} = 0.667$$

2) Soil reaction

$$q = \frac{\Sigma N}{B\cdot L}\left(1 \pm \frac{6\cdot e}{B}\right) = \frac{159.76}{3.0 \times 4.0}\left(1 \pm \frac{6 \times 0.519}{4.0}\right)$$

$$= 23.7 \text{ or } 3.0 \ t/m^2$$

3) Check for sliding

$$F = \frac{159.76 \times 0.6}{39.93} = 2.4 > 1.5$$

ok

Weight of surcharge

$W_{s\,add} = 0.51^{t/m^2} \times 3.0^{m} \times 2.4^{m} = 3.67^{t}$

Earth pressure

$P_a = 1/2 \times 0.271 \times 1.9 \times 6.4^2 \times 3.0 = 31.64^{t}$

$P_{a\,add} = 0.271 \times 0.51 \times 6.4 \times 3.0 = 2.65^{t}$

Temperature load

$\delta = 0.5 \times 18.7 = 9.4^{mm}$

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

$t_e = 20^{mm} \quad A = 300^{mm} \times 200^{mm}$

$G_0 = 10^{kg/cm^2}$

$$P_H = \frac{10^{kg/cm^2} \times 30^{cm} \times 20^{cm} \times 0.94^{cm}}{2^{cm}} = 2820^{kg} = 2.82^{t}$$

$P_H' = 2.82 \times 2 = 5.64^{t}$

Reaction due to superstructure

$R_{dl} = 36.04^{t}$

Calculation of wall section

load of deck $N_1 = 36.04/3.0 = 12.01^{t}$

Concrete self weight $N_2 = (0.3 \times 1.150 + 1.9 \times 4.35) \times 2.907 = 10.75^{t}$

Temp load $H_1 = 5.64/3.0 = 1.88^{t}$

Earth pressure $P_a = 0.271 \times 1/2 \times 1.9 \times 5.5^2 = 7.79^{t}$

$P_{a\,add} = 0.271 \times 1.51 \times 5.5 = 0.76^{t}$

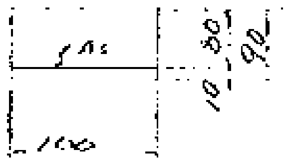
Sectional Force

$N = 12.01 + 10.25 = 22.26^{t}$

$H = 1.88 + 7.79 + 0.76 = 10.43^{t}$

$M = \overset{2.16}{12.01 \times 0.18} + \overset{8.18}{1.88 \times 4.35} + \overset{14.28}{7.79 \times 1.833} + \overset{2.09}{0.76 \times 2.75}$

$= 26.71^{t \cdot m}$



Minimum $A_s = 0.0015 \times 100 \times 80 = 12.0^{cm^2}$

$\phi 16 - 150^{ctc} = 2.01 \times 6.667 = 13.40^{cm^2}$

$np = 0.025 \quad d'/d = 0.125$

$f = \frac{M}{N} + u = 1.20 + 0.35 = 1.55^{m}$

$M' = 26.71 + 22.26 \times 0.35 = 35.50^{t \cdot m}$

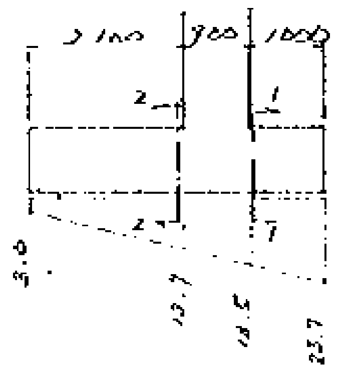
$c = 8.15 \quad s' = 24$

$\sigma_c = \frac{35.50 \times 10^5}{100 \times 80^2} \times 8.15 = 44^{kg/cm^2}$

$\sigma_s = \quad " \quad \times 23 \times 15 = 1860^{kg/cm^2}$

for parapet → $\phi 16 - 150^{ctc}$

Calculation of Footing Section



Sect. 1

Load $0.9 \times 1.0 \times 2.707 = 2.17$ t/m

$0.5 \times 1.0 \times 1.900 = 0.95$

$1/2 \times (23.7 + 18.5) \times 1.0 = -21.10$

-17.98 t/m

Moment $M_1 = (2.17 + 0.95) \times \frac{1}{2} \times 1.0 = 1.56$ t·m /m

$M_1 = -21.10 \times \frac{1}{3} \times 1.0 \times \frac{2 \times 23.7 + 18.5}{23.7 + 18.5} = -10.96$ t·m /m

Sectional Force $S = -17.98$ t

$M = -10.96$ t·m

Minimum $As = 0.0015 \times 100 \times 80 = 12$ cm²

Used $As = \phi 16 - 15$ c.t.c $= 13.40$ cm²

$np = 0.025$ $c = 10.7$ $s = 42.9$

$\sigma_c = 18$ kg/cm²

$\sigma_s = 1102$ kg/cm²

$\tau = \frac{17980}{100 \times 80} = 2.2$ kg/cm²

Sect. 2

Load $5.5 \times 2.1 \times 1.9 = 21.95$

$2.1 \times 0.51 = 1.07$

$0.9 \times 5.5 \times 2.407 = 11.91$

$1/2 \times (11.9 + 3.0) \times 2.1 = -17.75$

$17.18 \ t/m$

Moment

$M_1 = (21.95 + 1.07 + 11.91) \times \frac{1}{2} \times 2.1 = 36.68 \ ^{t \cdot m}$

$M_2 = -17.75 \times \frac{1}{3} \times 2.1 \times \frac{11.9 + 3.0 \times 2}{11.9 + 3.0} = -14.63 \ ^{t \cdot m}$

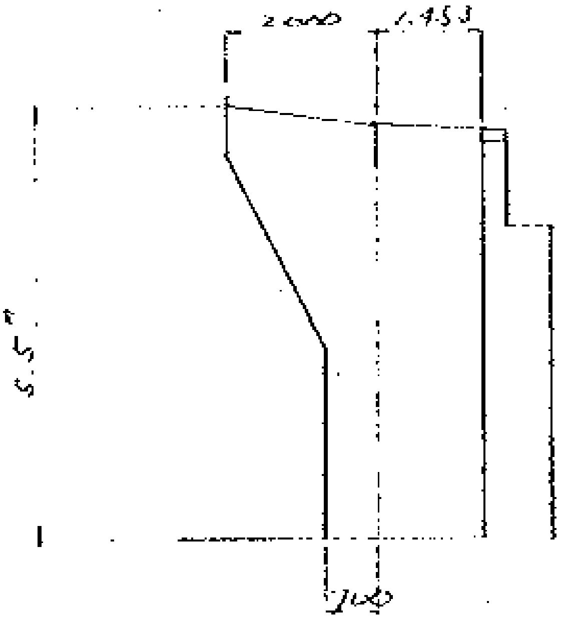
$22.05 \ ^{t \cdot m}/m$

Used $A_s = \Phi 16 - 150^{c/c} = 13.40^{cm^2}$

$\sigma_c = 37 \ ^{kg}/_{cm^2}$

$\sigma_s = 2217 \ ^{kg}/_{cm^2}$

$\tau = \frac{17130}{100 \times 20} = 21 \ ^{kg}/_{cm^2}$ o.k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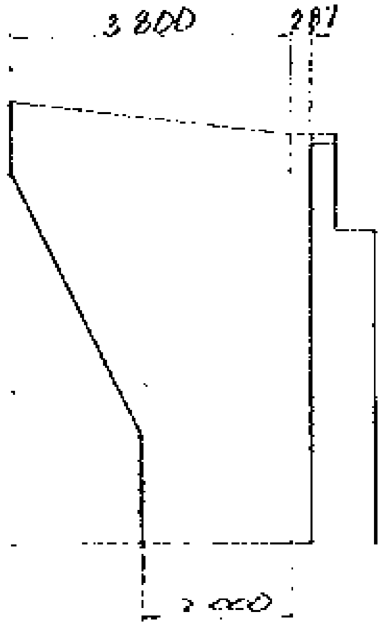
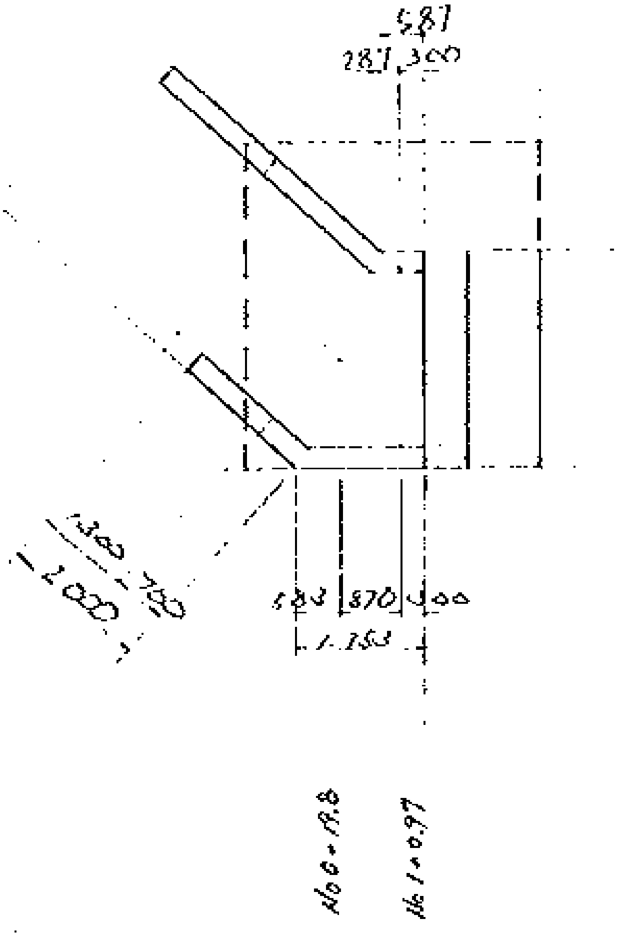
5.5°
700


3 800
287
2 000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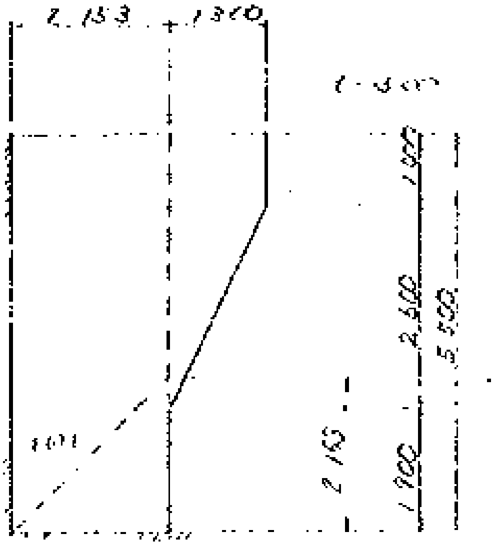
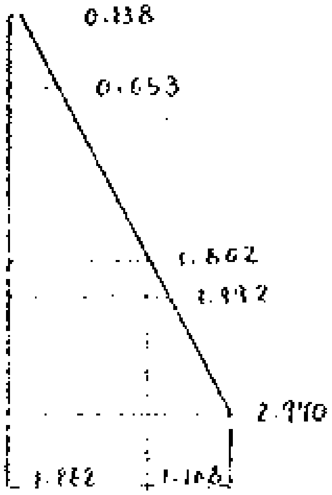
287
300
300
780
1 000
583
870
300
1.753
H00+19.8
Hc1+0.97





Earth pressure

$P_0 = (q + \gamma h_0) K_0$

$= (0.51^{t/m^2} + 1.9 \times 0) \times 0.271 = 0.138\ t/m^2$

$P_1 = (0.51 + 1.9 \times 1.0) \times 0.271 = 0.653\ t/m^2$

$P_3 = (0.51 + 1.9 \times 3.6) \times 0.271 = 1.992$ 〃

$P_4 = (0.51 + 1.9 \times 5.5) \times 0.271 = 2.970$ 〃

$P_5 = (0.51 + 1.9 \times 3.197) \times 0.271 = 1.862$ 〃

a-a 1-1 $Z = 1.0^{m}$

$M = 0.653 \times 1.30^2 \times \frac{1}{2} = 0.55^{t\cdot m}$

$S = 0.653 \times 1.30 = 0.85^{t}$

a-a 2-2 $Z = 1.0 \sim 3.6$

$Z = \frac{1.9 \times 3.60 - 2 \times 0.51}{3 \times 1.9} \fallingdotseq 1.021^{m}$

$M = (0.51 + 1.9 \times 1.021) \times 0.271 \times \frac{1}{2} \times \underset{1.69}{1.30^2} \times \left(\frac{3.6 - 1.021}{2.60}\right)^2 = 0.55^{t\cdot m}$

$S = (0.51 + 1.9 \times 1.021) \times 0.271 \times 1.30 \times \frac{3.60 - 1.021}{2.60} = 0.87^{t}$

A 1-1 $Z = 3.397^{m}$

$M = \frac{1}{2} \times 1.862 \times \overset{4.63}{2.153^2} + 0.55 + 0.87 \times 2.153 = 6.74^{t\cdot m}$

$S = 1.862 \times 2.153 + 0.87 = 4.88^{t}$

B-B $P = \frac{1}{2} \times (1.862 + 2.970) = 2.416^{t/m^2}$

$M = \frac{1}{2} \times 2.416 \times 1.077^2 = 1.40^{t\cdot m}$

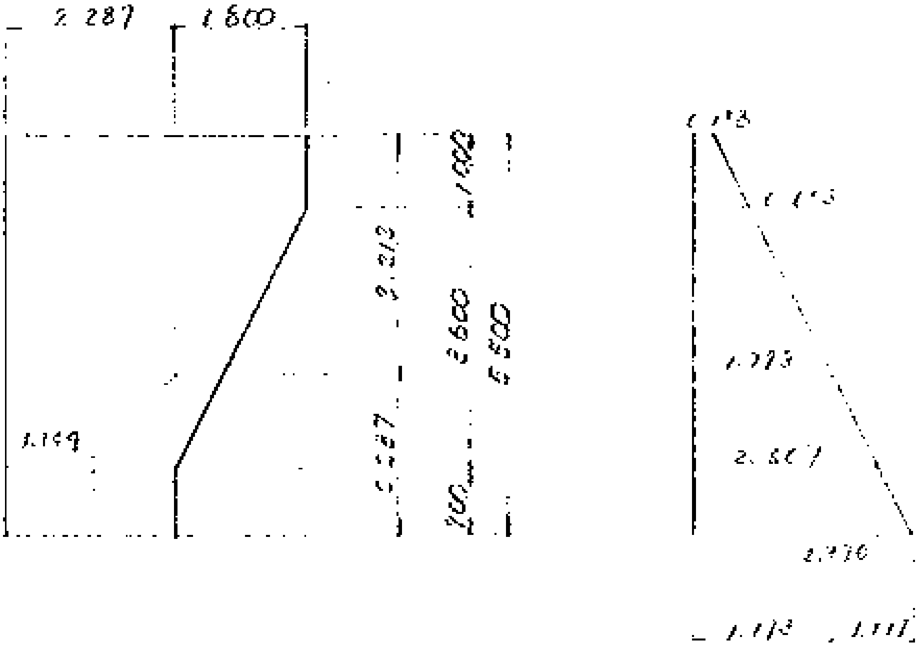
$S = 2.416 \times 1.077 = 2.60^{t}$

C-C

$M = (\frac{1}{2} \times 1.862 + \frac{1}{6} \times 1.108) \times 1.077^2 = 1.29^{t\cdot m}$

$S = (1.862 + \frac{1}{2} \times 1.108) \times 1.077 = 2.60^{t}$

	case	case	case
M (tm)	6.24	1.10	
N (ton)			
Q;shear force (ton)	4.22		
b (cm)	100		
d (cm)	23		
d' (cm)	7		
As (cm²)	φ 20 - c/c 150 = 20.93	Min As = 3.45 φ12 - c/c 150 = 7.53	
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.136		
M'=M+N·u (tm)			
M'/(b·d²) (%)	11.79		
Q/b·d (kg/cm²)			
C	5.13		
S	8.52		
Z			
σc (kg/cm²)	73		
σs (kg/cm²)	1628		
τ (kg/cm²)	2.1		
σca (kg/cm²)			
σsa (kg/cm²)			
τa (kg/cm²)			



Earth pressure

$p_0 = (0.51 + 1.9 \times 0.0) \times 0.271 = 0.135$ t/m²

$p_1 = (\;\;" \;\; + 1.9 \times 1.0) \;\;" \;\; = 0.163$

$p_2 = (\;\;" \;\; + 1.9 \times 3.213) \;\;" \;\; = 1.773$

$p_3 = (\;\;" \;\; + 1.9 \times 4.600) \;\;" \;\; = 2.567$

$p_4 = (\;\;" \;\; + 1.9 \times 5.5) \;\;" \;\; = 2.776$

a-a 1-1 $Z = 1.0^{m}$

$M = 0.653 \times 1.80^2 \times 1/2 = 1.06^{tm}$

$S = 0.653 \times 1.80 = 1.18^{t}$

a-a 2-2 $Z = 1.0 \sim 4.6$

$Z = \frac{1.9 \times 4.6 - 2 \times 0.51}{3 \times 1.9} = 1.354^{m}$

$M = (0.51 + 1.9 \times 1.354) \times 0.271 \times \frac{1}{2} \times 1.6^2 \times \left(\frac{4.60 - 1.354}{3.60}\right)^2 = 1.10^{tm}$

$S = (0.51 + 1.9 \times 1.354) \times \overset{0.835}{0.271} \times \overset{1.12}{1.6} \times \overset{1.512}{\left(\frac{4.60 - 1.354}{3.60}\right)} = 1.36^{t}$

A 1-1 $Z = 3.213^{m}$ $P = (0.51 + 1.9 \times 3.213) \times 0.271 = 1.793^{t/m^2}$

$M = \frac{1}{2} \times 2.287^2 \times \overset{4.69}{1.793} + 1.10 + \overset{3.11}{1.36 \times 2.287} = 8.90^{tm}$

$S = 2.287 \times 1.793 + 1.36 = 5.46^{t}$

B-B $p = 1/2 \times (1.793 + 2.970) = 2.382^{t/m^2}$

$M = 1/2 \times 2.382 \times 1.149^2 = 1.58^{tm}$

$S = 2.382 \times 1.149 = 2.73^{t}$

C-C

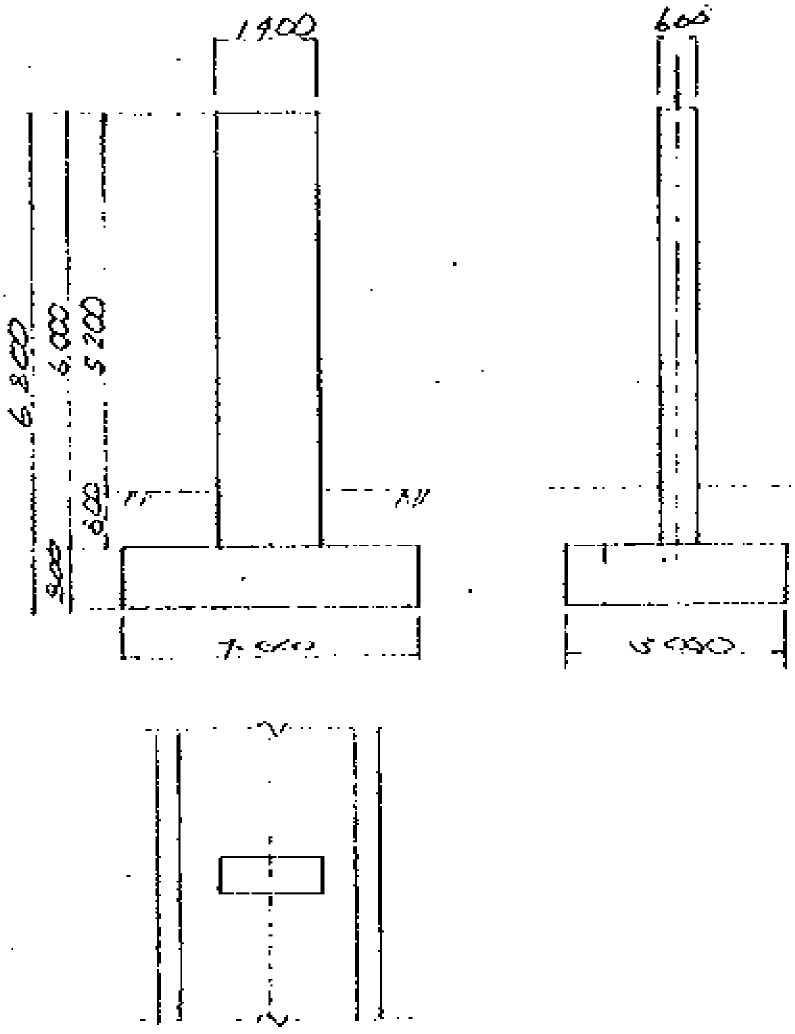
$M = (1/2 \times 1.793 + 1/6 \times 1.177) \times 1.149^2 = 1.43^{tm}$

$S = (1.793 + 1/2 \times 1.177) \times 1.147 = 2.72^{t}$

	case 1-1	case	case
M (tm)	8.90	1.5	
N (ton)			
Q;shear force (ton)	5.16		
b (cm)	100		
d (cm)	33		
d' (cm)	7		
As (cm^2)	#16-150. = 13.40	Min As = 7.95 #12-150 = 7.53	
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.061		
M'=M+N·u (tm)			
M'/($b\cdot d^2$) (kg/cm²)	8.17		
Q/b·d (kg/cm^2)			
C	1.55		
S	18.20		
Z			
σ_c (kg/cm^2)	62		
σ_s (kg/cm^2)	1.231		
τ (kg/cm^2)	1.7		
σ_{ca} (kg/cm^2)			
σ_{sa} (kg/cm^2)			
τ_a (kg/cm^2)			

§ 2 . Pier

1) Pier dimension



2) calculation of load

2-1) self weight

$$W_p = 1.4 \times 0.6 \times 6.0 \times 2.41 = 12.15^t$$

2-2) Reaction due to superstructure

Dead Load of Deck 91.64^t

Live load of Deck 28.50^t

2-3) Wind pressure

a) wind gust speed

$V_G = V \cdot K_1 \cdot S_1 \cdot S_2$

$V = 38\ m/sec$ $K_1 = 1.0$ $S_1 = 1.0$

$S_2 = 1.56$

$\therefore V = 38\ m/sec$ $V_G = 38 \times 1.0 \times 1.0 \times 1.56 = 59.28\ m/sec$

($V_G =$ $= 35.0\ m/sec$ with live load)

b) Nominal transverse wind load

$P_t = q \cdot A \cdot C_D$

$q = 0.613 \cdot V_G^2$ $\begin{cases} = 0.613 \times 59.28^2 = 2154\ N/m^2 = 220\ kgf/m^2 \\ = 0.613 \times 35.0^2 = 751\ N/m^2 = 77\ kgf/m^2 \end{cases}$

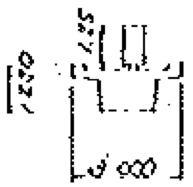
Therefore without live load $q = 220\ kgf/m^2$

with live load $q = 77\ kgf/m^2$

b-1) for superstructure

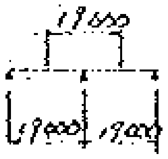
$C_D = 1.45$ $b/d = 3.00/1.23 = 2.4$ without live

$C_D = 1.45$ $b/d = 3.00/2.38 = 1.3$ with live



$A = 1.23 \times 19.0 = 23.37\ m^2$ $\therefore P_t = 23.37 \times 0.22 \times 1.45 = 7.46^t$

$A = (1.13 + 1.25) \times 19.0 = 45.22\ m^2$ $P_t = 45.22 \times 0.077 \times 1.45 = 5.05^t$



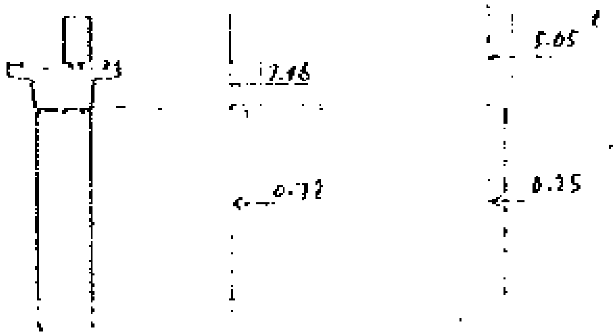
b-2) for substructure

$C_D = 1.25$ $t/d = 1.4/0.5 = 2.8$

$A = 5.2 \times 0.6 = 3.12^{m^2}$

$P_1 = 0.22 \times 3.12 \times 1.25 = 0.86^{t}$

$P_2 = 0.077 \times 3.12 \times 1.25 = 0.30^{t}$



c) Nominal longitudinal load.

c-1) for superstructure

$P_L = 0.25 \times 7.46 = 1.87^{t}$

$P_{LS} = 0.25 \times 0.077 \times (1.18 \times 17.0) \times 1.55 = 0.60^{t}$

$P_{LL} = 0.5 \times 0.077 \times (1.25 \times 17.0) \times 1.55 = 1.33^{t}$

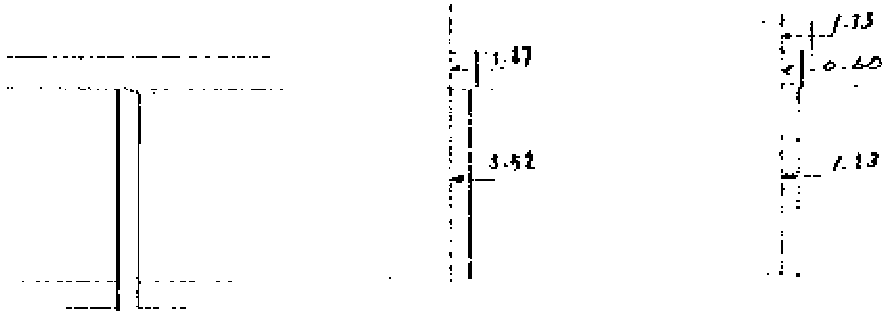
c-2) for substructure

$C_D = 2.2$ $t/d = 0.5/1.4 = 1/2.8$

$A = 1.4 \times 5.2 = 7.28^{m^2}$

$P_1 = 0.22 \times 7.28 \times 2.2 = 3.52^{t}$

$P_2 = 0.077 \times 7.28 \times 2.2 = 1.23^{t}$



d) Seismic vertical load.

$$P_V = g \cdot A_s \cdot C_c$$

$$= 0.22 \times (1.9 + 19.0) \times 1.1 = 2.34^{t}$$

$$= 0.077 \times (\quad " \quad) \times 0.9 = 1.82^{t}$$

e) load combination

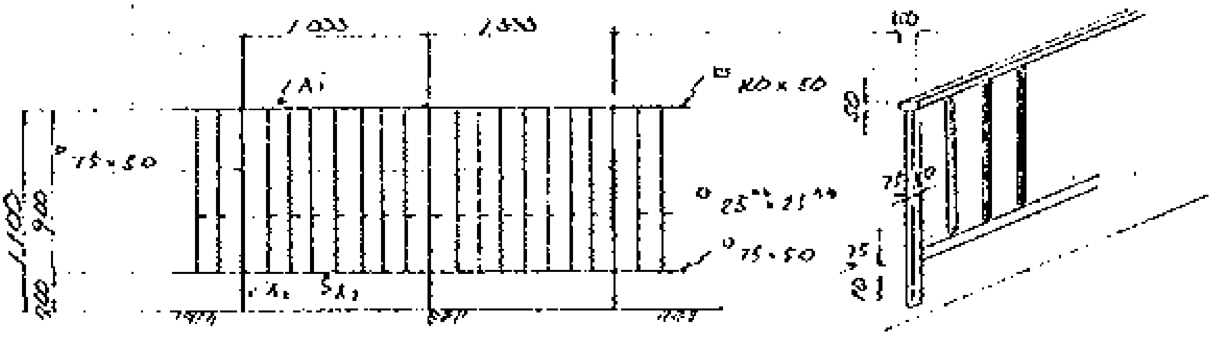
e-1) P_d

e-2) $P_d \pm P_v$

e-3) P_e

e-4) $1.5\,P_d + P_e \pm P_v$

Parapet.



$A_1 = 1.0 \times 0.05 = 0.05\ m^2$ $\quad C_D = 1.1$

$A_2 = 1.1 \times 0.05 = 0.055\ m^2$ $\quad C_D = 1.3$

$A_3 = 1.0 \times 0.05 = 0.055$ $\quad C_D = 1.3$

$A_4 = 0.9 \times 0.05 \times 7 = 0.315\ m^2$ $\quad C_D = 1.5$

$q = 220\ kg/m^2$

$P_1 = 0.22 \times 0.05 \times 1.1 \quad 0.012$

$+ \quad " \quad \times 0.055 \times 1.3 \quad 0.016$

$+ \quad " \quad \times 0.055 \times 1.3 \quad 0.016$

$+ \quad " \quad \times 0.315 \times 1.5 \quad 0.104 \quad = 0.15\ ton/m$

$P_L = 0.8 P_{L1} + 0.4 P_{L2}$

$= 0.8\ (0.012 + 0.016) + 0.4\ (0.016 + 0.104)$

$= 0.022 + 0.048 \quad = 0.07\ ton/m$

Calculation of Sectional Force

1) Transverse direction

	N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
W D.L	91.67					
W L.L	28.50					
W/s	10.12					
Wind (unloaded)	± 2.39			7.46	6.665	49.72
				0.80	3.400	2.72
0.15×18.7 m				2.81	7.830	22.00
Wind (loaded)	± 0.82			5.05	7.240	36.56
				0.30	3.400	1.02
Dead + Wind	(± 1.34) 101.79			11.07		74.44
Dead+Live+Wind	(± 0.82) 132.29			5.35		37.58

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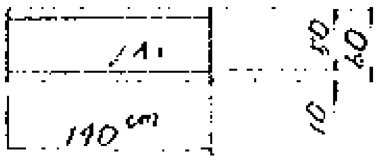
Calculation of Sectional Force

2) Longitudinal direction

	$N^{(t)}$	$x^{(m)}$	$N \cdot x^{(t \cdot m)}$	$H^{(t)}$	$y^{(m)}$	$H \cdot y^{(t \cdot m)}$
WDL	91.69					
WLL	28.50					
Ws	12.15					
Wind (unloaded)	± 2.39			1.87	6.05	11.31
				3.52	3.40	11.97
0.07 × 18.7				1.31	6.05	7.93
Wind (loaded)	± 0.82			1.33	6.05	8.05
				0.60	6.05	3.63
				1.23	3.40	4.18
Dead + wind	(± 2.39) 103.79			6.70		31.21
Dead + Live + Wind	(± 0.82) 132.29			3.16		15.85

$N = 103.79^{t}$ $M = 31.21^{t\,m}$

Used As D25 - 150^ctc = 9 × 4.91 = 44.19



$np = 0.095$ $d'/d = \frac{10}{50} = 0.2$

$f = \frac{M}{N} + u = 0.307^{m} + 0.20^{m} = 0.507$

$M' = 31.21 + 103.79 \times 0.2 = 51.968^{t \cdot m}$

$f/d = \frac{0.507}{0.50} = 0.10$ $c = 3.5$ $s = ?.?$

$\sigma_c = 51\ ^{kg/cm^2}$

$\sigma_s = 552\ ^{kg/cm^2}$

Calculation of stability

Footing Weight $W_{sf} = 4.0 \times 3.0 \times 0.8 \times 2.41 = 23.14^{t}$

1). Transverse direction

Case-1) Dead + wind

$N = 103.79 + 23.14 = 126.93^{t}$ $H = 11.07^{t}$

$M = 11.07 \times 0.8 + 74.44 = 83.30^{t \cdot m}$ $e = 0.656 \leq \frac{B}{6} = 0.666$

Soil reaction

$$q = \frac{126.93}{4.0 \times 3.0} \left(1 \pm \frac{6 \times 0.656}{4.0}\right) = \left\{\begin{matrix} 20.8 \\ 0.2 \end{matrix}\right. t/m^2$$

Case-2). Dead + Live + wind

$N = 132.29 + 23.14 = 155.43^{t}$ $H = 5.35$

$M = 5.35 \times 0.8 + 37.58 = 41.86^{t m}$ $e = 0.273 < 0.666$.

Soil reaction

$$q = \frac{155.43}{3.0 \times 4.0} \left(1 \pm \frac{6 \times 0.273}{4.0}\right) = \left(\begin{matrix} 18.0 \\ 7.5 \end{matrix}\right. t/m^2$$

2) Longitudinal direction

Case-1). Dead + wind

$N = 103.79 + 23.14 = 126.93^{t}$ $H = 6.70^{t}$

$M = 6.70 \times 0.8 + 31.21 = 36.57^{t m}$ $e = 0.293 < 0.500 = \frac{B}{6}$

Soil reaction $q = \frac{126.93}{4.0 \times 3.0} \left(1 \pm \frac{6 \times 0.293}{3.0}\right) = \left\{\begin{matrix} 16.5 \\ 4.3 \end{matrix}\right. t/m^2$

Case-2) Dead + Live + Wind

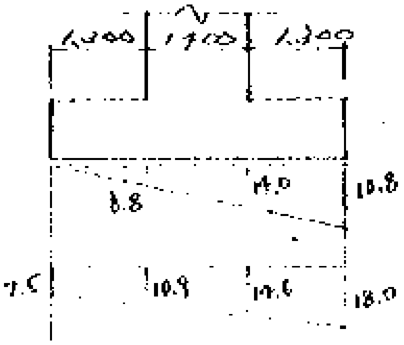
$N = 155.43$ $H = 3.16^{t}$

$M = 3.16 \times 0.8 + 15.86 = 18.39^{t m}$ $e = 0.120$

Soil reaction $q = \frac{155.43}{4.0 \times 3.0} \left(1 \pm \frac{6 \times 0.120}{3.0}\right) = \left\{\begin{matrix} 15.8 \\ 9.7 \end{matrix}\right. t/m^2$

Calculation of Footing Section

1). Transverse direction



$S_1 = 0.8 \times 1.3 \times 2.41 = 2.51^{t}$

$S_2 = \frac{1}{2} \times (20.8 + 14.0) \times 1.3 = -22.62^{t}$

-20.11^{t}

$M_1 = 2.51 \times \frac{1}{2} \times 1.3 = 1.63^{t\cdot m}$

$M_2 = -22.62 \times \frac{1}{3} \times 1.3 \times \frac{2\times 20.8 + 14.0}{20.8 + 14.0} = -15.66^{t\cdot m}$

$-14.03^{t\cdot m}$

$A_s = \phi 16 - 125^{ctc} = 2.01 \times 8. = 16.08^{cm^2}$

$np = 0.037 \quad k = 9.46 \quad j = 33.0$

$S' = -20.11 \times \frac{3.0}{2.0} = -30.17^{t}$

$M' = -14.03 \times \frac{3.0}{2.0} = -21.05$

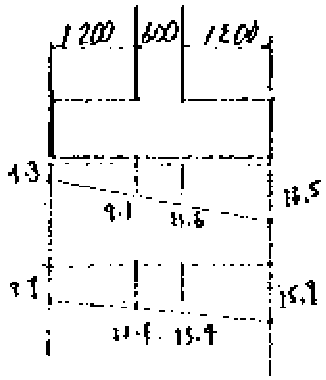
Footing width = 3.0
effective width = 2.0
(= 0.6 + (2×0.7))

$\sigma_c = 41 \ kg/cm^2$

$\sigma_s = 2061 \ kg/cm^2$

$\tau = \frac{30170}{100 \times 70} = 4.31 \ kg/cm^2$

2) Longitudinal direction



$S_1 = 0.8 \times 1.2 \times 2.41 = 2.31^{t}$

$S_2 = \frac{1}{2} \times (15.9 + 13.4) \times 1.2 = -17.58^{t}$

-15.27^{t}

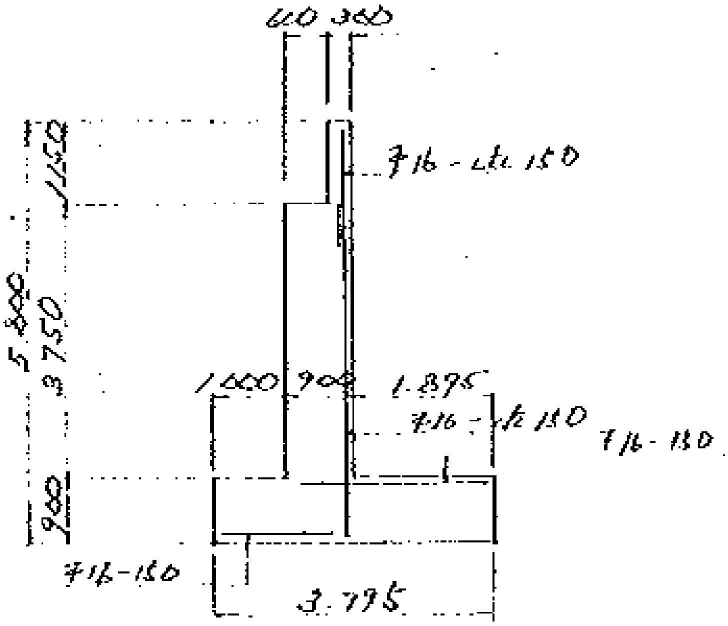
$M_1 = 2.31 \times \frac{1}{2} \times 1.20 = 1.39$

$M_2 = -17.58 \times \frac{1}{3} \times 1.20 \times \frac{2\times 15.9 + 13.4}{15.9 + 13.4} = -10.85$

$-9.46^{t\cdot m}$

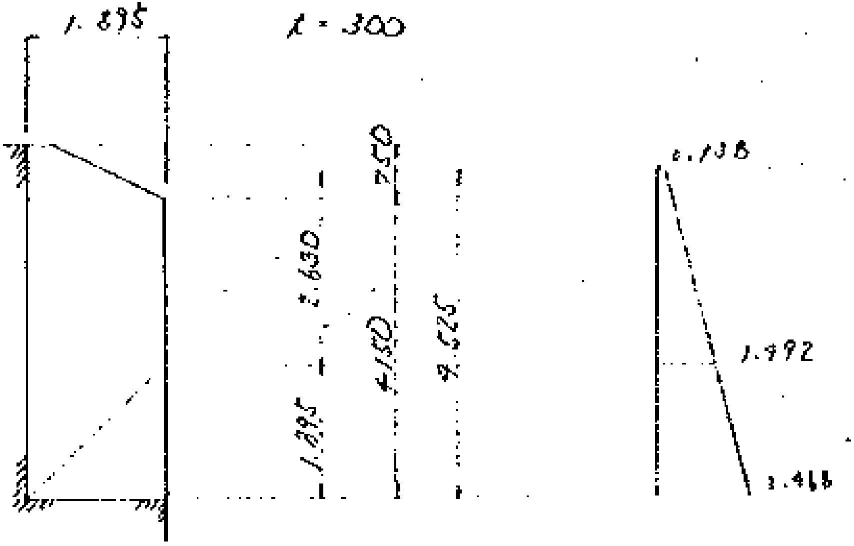
Used $A_s = \phi 16 - ctc\ 150^{mm}$

§3. Abutment -2



Refer to Abutment -1

- wing wall



Depth 2.63^{m}

$M = \frac{1}{2} \times 1.895^{2} \times 1.992 = 2.68^{t \cdot m}$

$S = 1.895 \times 1.992 = 2.83^{t}$

Depth $2.63 + \frac{1}{2} \times \overset{1.5625}{1.895} = 3.578$

$M = \frac{1}{2} \times (1.992 + \overset{1.96}{2.968}) \times 0.948^{2} \times \frac{1}{2} = 0.89^{t \cdot m}$

$S = 1.980 \times 0.948 = 1.88^{t}$

Depth 4.525

$M = \{\frac{1}{2} \times 1.992 + \frac{1}{6} \times (0.976)\} \times 0.948^{2} = 0.82^{t \cdot m}$

$S = (1.992 + \frac{1}{2} \times 0.976) \times 0.948 = 1.88^{t}$

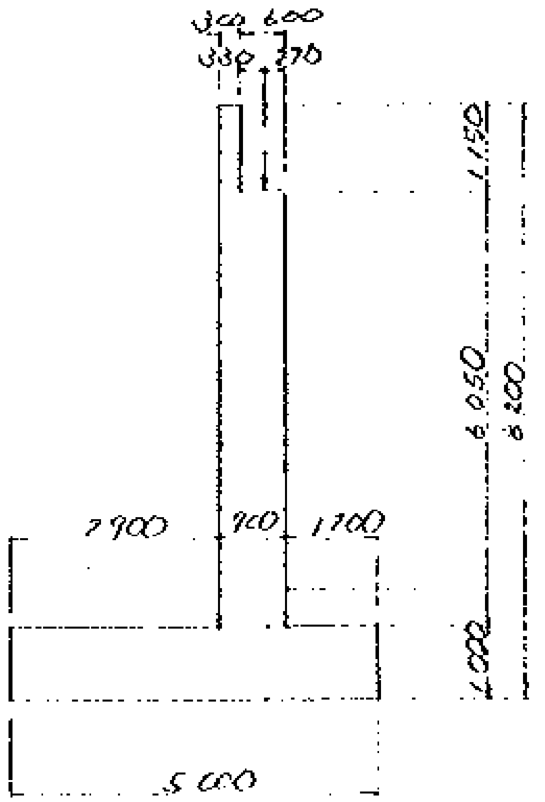
$\text{Min } A_s = 100^{cm} \times 23^{cm} \times 0.0015 = 3.45^{cm^2}$

$\text{Used } A_s = \phi 12 - 150^{mm} \text{ ctc} = 7.53^{cm^2}$

$\text{Req. } A_s = \frac{M}{f_s j d} = \frac{2.68 \times 10^{5}}{2200 \times 0.875 \times 23} = 5.79^{cm^2} < \text{Used } A_s$
o.k

Vol. 2 Beau Bassin BR.

§ 1. Abutment - 1



		N (t)	x (m)	Nx (t·m)
1	0.30 × 1.15 × 3.0 × 2.407	2.49	1.95	4.86
2	0.90 × 6.05 × 3.0 × 2.407	39.32	1.65	64.88
3	5.00 × 1.00 × 3.0 × 2.407	36.11	2.50	90.26
4	1.200 × 0.50 × 3.0 × 1.9	3.42	0.60	2.05
5	2.90 × 7.20 × 3.0 × 1.9	119.02	3.55	422.51
		200.36		584.56

Weight of surcharge

W_s add = $0.51 \times 3.2 \times 3.0 = 4.90^{t}$

Earth pressure

$P_a = 1/2 \times 0.271 \times 1.9 \times 8.2^2 \times 3.0 = 51.93^{t}$

P_a add = $0.271 \times 0.51 \times 8.2 \times 3.0 = 3.40^{t}$

Temperature load

$\delta = 0.5 \times 18.4 \times 1/2 = 4.6^{mm}$

$$H = \frac{G_a \cdot A \cdot \delta}{\Sigma t_e}$$

$$= \frac{10 \times 30 \times 30 \times 0.46}{2^{cm}} = 1380^{kg} = 1.38^{t}$$

$H' = 1.38 \times 2 = 2.76^{t}$

Reaction due to superstructure

$R_{d+l} = 98.21^{t}$

Calculation of Stability

	N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
W1.2	48.21	1.470	70.87			
Temp				2.76	7.10	19.60
Wl + Ws	200.36		589.56			
Ws add	4.90	3.40	16.66			
Pa				51.93	2.733	141.92
Pa add				3.40	4.100	13.94
	253.47		677.09	58.09		175.46

1) check for eccentric

$$x = \frac{\Sigma N x - \Sigma H y}{\Sigma N} = \frac{677.09 - 175.46}{253.47} = 1.959$$

$$e = \frac{B}{2} - x = \frac{5.0}{2} - 1.959 = 0.541 < \frac{B}{6} = 0.833$$

2) Soil reaction

$$q = \frac{\Sigma N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{253.47}{5.0 \times 3.0}\left(1 \pm \frac{6 \times 0.541}{5.0}\right)$$

$$= \begin{matrix} 27.86 \ t/m^2 \\ 5.93 \ " \end{matrix}$$

3) Check for sliding

$$F = \frac{253.47 \times 0.6}{58.09} = 2.6$$

Calculation of wall section.

load of deck $N_1 = 48.21/3.0 = 16.07^{t}$

Concrete weight $N_2 = (0.3 \times 1.150 + 0.9 \times 6.05) \times 2.407 = 13.94^{t}$

Temp load $H_1 = 2.76/3.0 = 0.92^{t}$

Earth pressure $P_A = 0.271 \times \frac{1}{2} \times 1.9 \times 7.2^2 = 13.35^{t}$

" $P_A \, add = 0.271 \times 0.51 \times 7.2 = 1.00^{t}$

Sectional Force

$N = 16.07 + 13.94 = 30.01^{t}$

$H = 0.92 + 13.35 + 1.00 = 15.27^{t}$

$M = 0.92 \times \overset{5.61}{6.10} + 13.35 \times \overset{11.09}{\frac{1}{3}} \times 7.2 + 1.00 \times \overset{3.60}{\frac{1}{2}} \times 7.2$

$+ 16.07 \times \overset{2.33}{0.18} = 44.14^{t\,m}$

Used $A_s = \phi 20 - 150^{ctoc} \cdots 3.14 \times 6.667 = 20.93^{cm^2}$

$np = 0.39$, $d'/d = 0.125$

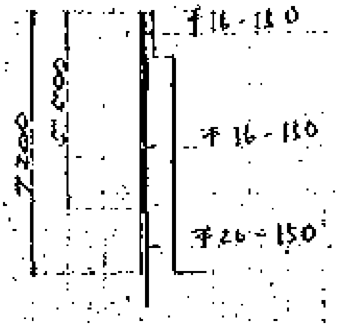
$f = M/N + u = 1.471 + 0.350 = 1.821$

$M' = 44.14 + 30.01 \times 0.35 = 54.69$ $\sigma = 8.538$

$f/d = 1.821/0.8 = 2.3$ $c = 7.05$ $s = 17$

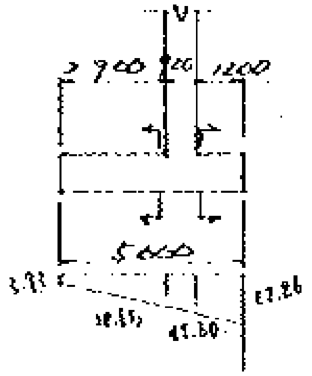
$\sigma_c = 8.518 \times 7.05 = 60 \ kg/cm^2$

$\sigma_s = 17.0 \times 8.538 \times 15 = 2177$



Calculation of Footing Section

Sect. 1



$1.0 \times 1.2 \times 2.407 = 2.89$

$0.5 \times 1.2 \times 1.9 = 1.14$

$\frac{1}{2} \times (22.60 + 27.86) \times 1.2 = -30.28$

-26.25

Moment

$M_1 = (2.89 + 1.14) \times \frac{1}{2} \times 1.20 = 2.42$

$M_2 = -30.28 \times \frac{1}{3} \times 1.2 \times \frac{27.86 \times 2 + 22.60}{27.86 + 22.60} = -18.80$

-16.38 t·m

Used As = φ16 - 150 ctc = 13.40

Sect - 2

$1.0 \times 2.90 \times 2.407 = 6.98 \times 1.45 = 10.12$

$7.2 \times 2.90 \times 1.9 = 39.67 \times 1.45 = 57.52$

$0.51 \times 2.9 = 1.48 \times 1.45 = 2.15$

$\frac{1}{2} \times (5.93 + 18.65) \times 2.9 = -35.64 \times \frac{1}{3} \times 2.9 \times \frac{5.93 \times 2 + 18.65}{5.93 + 18.65} = -42.73$

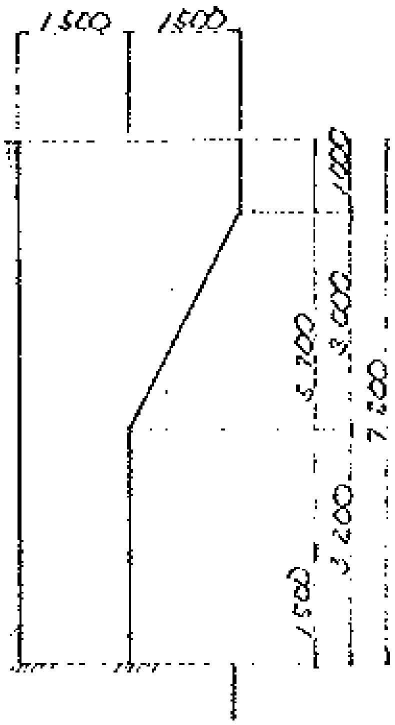
$12.49 \qquad -27.06$

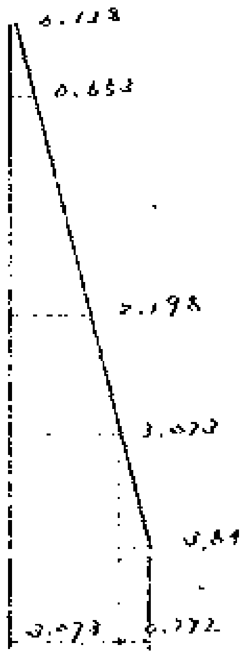
Used As = $\frac{\phi 16}{\phi 20}$ (300 ctc Alternate) = $(2.01 + 3.14) \times \frac{1}{2} \times 6.667$
= 17.17 cm²

$np = 0.029 \qquad 0.161 \qquad 0.371$

$\sigma_c = 39$ kg/cm²

$\sigma_s = 1859$ kg/cm²





Earth pressure

t/m^2

$P_0 = (0.51 + 1.9 \times 0.0) \times 0.271 = 0.138$

$P_1 = (0.51 + 1.9 \times 1.0) \times \text{〃} = 0.653$

$P_2 = (0.51 + 1.9 \times 4.0) \times \text{〃} = 2.198$

$P_3 = (0.51 + 1.9 \times 5.7) \times \text{〃} = 3.073$

$P_4 = (0.51 + 1.9 \times 7.2) \times \text{〃} = 3.845$

a-a 1-1

$$M = 0.653 \times 1.5^2 \times 1/2 = 0.73^{t\cdot m}$$

$$S = 0.653 \times 1.5 = 0.98^{t}$$

a-a 2-2 $h = 1.0 \sim 4.0$

$$z = \frac{1.9 \times 4.0 - 2 \times 0.31}{3 \times 1.9} = 1.154^{m}$$

$$M = (0.31 + 1.9 \times 1.154) \times \underset{0.737}{0.271} \times \frac{1}{2} \times \underset{1.125}{1.5^2} \times \underset{0.9}{\left(\frac{4.0-1.154}{3.0}\right)^2} = 0.74^{t\cdot m}$$

$$S = (0.31 + 1.9 \times 1.154) \times 0.271 \times 1.5 \times \left(\frac{4.0-1.154}{3.0}\right) = 1.04^{t}$$

A 1 1 $Z = 4.0$

$$M = \frac{1}{2} \times 2.198 \times 1.5^2 + 0.74 + 1.04 \times 1.5 = 4.77^{t\cdot m}$$

$$S = 2.198 \times 1.5 + 1.04 = 4.34^{t}$$

A-A 2 $Z = 5.7^{m}$

$$M = 1/2 \times 3.073 \times 1.5^2 = 3.46^{t\cdot m}$$

$$S = 3.073 \times 1.5 = 4.61^{t}$$

B-B $P = 3.073 + 1/2 \times 0.772 = 3.459^{t/m^2}$

$$M = 1/2 \times 3.459 \times 0.75^2 = 0.97^{t\cdot m}$$

$$S = 3.459 \times 0.75 = 2.59^{t}$$

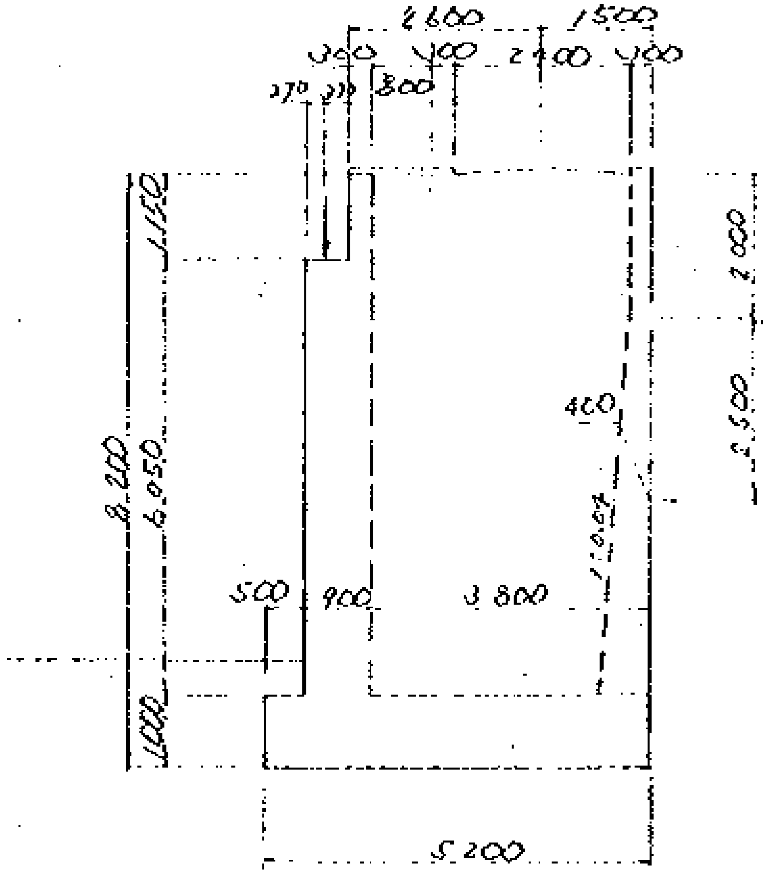
C-C

$$M = (1/2 \times 3.073 + 1/6 \times 0.772) \times 0.75^2 = 0.94^{t\cdot m}$$

$$S = (3.073 + 1/2 \times 0.772) \times 0.75 = 2.59^{t\cdot m}$$

	case A 1-1	case A 2-2	case
M (tm)	4.17	3.46	
N (ton)			
Q;shear force (ton)	4.34	4.61	
b (cm)	100	100	
d (cm)	23	23	
d' (cm)	7	7	
As (cm^2)	φ16 - ctc150 = 13.40	φ12 - ctc150 = 7.53	
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.087	0.049	
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm²)	4.02	6.59	
Q/b·d (kg/cm^2)			
C	6.64	8.20	
S	12.4	22.5	
Z			
σ_c (kg/cm^2)	60	54	
σ_s (kg/cm^2)	1745	2207	
τ (kg/cm^2)	1.7	2.0	
σ_{ca} (kg/cm^2)			
σ_{sa} (kg/cm^2)			
τ_a (kg/cm^2)			

§ 2. Abutment 2.


2600
1500
300
400
2100
400
270
270
300
1150
8200
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2000
2500
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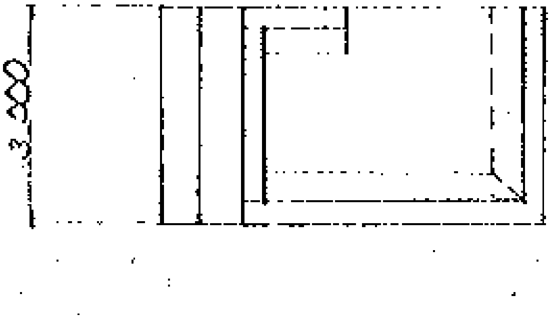
1:0.04
500
900
3800
1000
5200


3300

Concrete and soil weight

		N	x	$N \cdot x$
1	0.3 × 1.15 × 3.0 × 2.407	2.49	1.250	3.11
2	0.9 × 6.05 × 3.0 × 2.407	39.32	0.950	37.35
3	5.2 × 0.5 × 3.0 × 2.407	18.77	2.600	48.81
4	0.5 × 0.5 × 3.0 × 1.9	1.43	0.250	0.36
5	3.8 × 7.2 × 3.0 × 1.95	155.84	3.300	514.29
Σ.		217.85		603.92

Surcharge

$Ws\ add = 0.51 \times 3.8 \times 3.0 = 5.81^{t}$

Temperature

$PH = 5.64^{t}$

Reaction due to superstructure

$Rdl = 36.04^{t}$

Calculation of Stability

	N (t)	x (m)	N·x (t m)	H (t)	y (m)	H·y (t m)
Rd+Q	36.09	0.77	27.75			
Temp				5.69	7.10	± 40.09
W + Ws	217.85		603.92			
Ws add	5.81	3.00	17.43			
Σ	259.70		649.10	5.69		± 40.09

1) check for eccentric

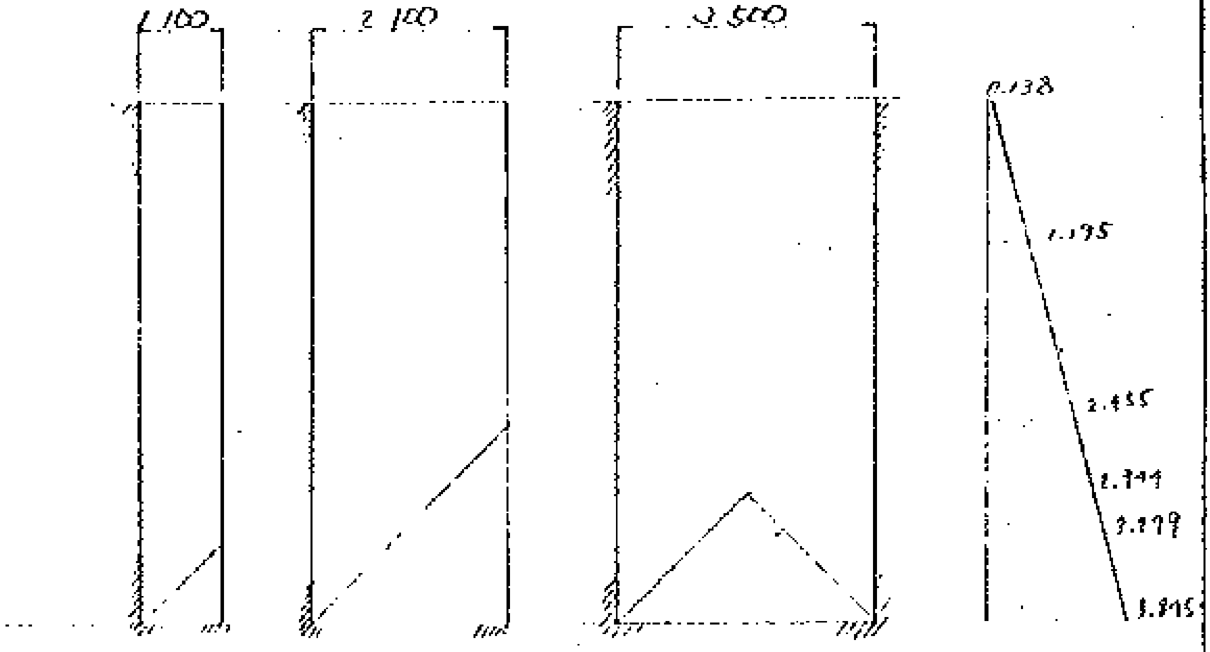
$$x = \frac{\Sigma N\cdot x \mp \Sigma H\cdot y}{\Sigma N} = \frac{649.10 \mp 40.09}{259.70} = \begin{matrix} 2.345 \\ 2.654 \end{matrix}$$

$$e = \frac{B}{2} - x = \frac{5.2}{2} - 2.345 = \begin{matrix} 0.255 \\ -0.054 \end{matrix} < \frac{B}{6} = 0.867$$

2) Soil reaction

$$q = \frac{\Sigma N}{B\cdot L}\left(1 \pm \frac{6\cdot e}{B}\right) = \frac{259.70}{3.0\times 5.2}\left(1 \pm \frac{6\times 0.255}{5.2}\right)$$

$$= \begin{cases} 21.5 \ \text{t/m}^2 \\ 11.7 \end{cases}$$



$P_0 = (0.51 + 19 \times 0.0) \times 0.271 = 0.138$ t/m²

$P_1 = (0.51 + 1.9 \times 6.1) \times 0.271 = 3.279$ t/m²

$P_2 = (0.51 + 1.9 \times 4.5) \times 0.271 = 2.455$ 〃

$P_3 = (0.51 + 1.9 \times 5.45) \times 0.271 = 2.944$ 〃

$P_4 = (0.51 + 1.9 \times 7.2) \times 0.271 = 3.845$ 〃

$P_5 = (0.51 + 1.9 \times 2.0) \times 0.271 = 1.195$ 〃

1) Width 1.1 m

- Depth 6.1 m

  $M = \frac{1}{2} \times 1.1^2 \times 3.279 = 1.98^{t \cdot m}$

  $S = 1.1 \times 3.279 = 3.61^{t}$

- Depth 6.65

  $M = \frac{1}{2} \times (3.279 + 3.895) \times 0.55^2 \times \frac{1}{2} = 0.54^{t \cdot m}$

  $S = 〃 \times 0.55 = 1.96^{t}$

2) Width 2.7 m

- Depth 2.0 m

  $M = \frac{1}{2} \times 2.7^2 \times 1.195 = 4.36^{t \cdot m}$

  $S = 2.7 \times 1.195 = 3.23^{t}$

- Depth 4.5 m

  $M = \frac{1}{2} \times 2.7^2 \times 2.455 = 8.95^{t \cdot m}$

  $S = 2.7 \times 2.455 = 6.63^{t}$

- Depth 4.5 + 1.35 = 5.85 m

  $M = \frac{1}{2} \times (1.35)^2 \times \frac{1}{2} \times (2.455 + 3.895) = 2.87^{t \cdot m}$

  $S = 1.55 \times 〃 = 4.25^{t}$

3) Width 3.5 m

- Depth 5.45 m

  $M = \frac{1}{10} \times 2.944 \times 3.5^2 = 3.61^{t \cdot m}$

  $S = 2.944 \times 3.5 = 10.30^{t}$

- Depth 5.45 ~ 7.2 m

  $M = \{\frac{1}{2} \times 2.944 + \frac{1}{6} \times (3.845 - 2.944)\} \times 1.75^2 = 4.97^{t \cdot m}$

  $S = \{2.944 + \frac{1}{2} \times (3.845 - 2.944)\} \times 1.75 = 5.94^{t}$

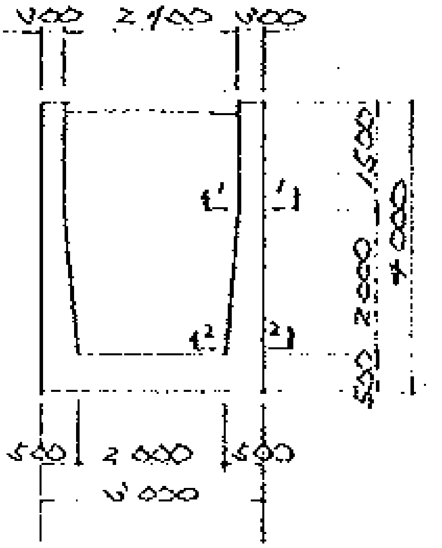
	case $W=1.7$ $D=6.1$	case $W=2.7$ $D=2.0$	case $W=17$ $D=4.5$
M (tm)	1.98	9.36	8.75
N (ton)			
Q;shear force (ton)	3.61	3.23	6.63
b (cm)	100	100	100
d (cm)	23	23	33
d' (cm)	7	7	7
As (cm^2)	Φ12 - ctc 150 = 7.53 cm^2	Φ16 - ctc -150 = 13.40	Φ16 - ctc 150 = 13.40
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.049	0.087	0.061
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)	3.79	2.39	8.22
Q/b·d (kg/cm^2)			
C	8.20	6.69	7.55
S	42.5	17.9	18.2
z			
σ_c (kg/cm^2)	31.	55	62
σ_s (kg/cm^2)	1263	1595	2243
τ (kg/cm^2)	1.6	1.9	2.0
σ_{ca} (kg/cm^2)	83		
σ_{sa} (kg/cm^2)	2346		
τ_a (kg/cm^2)			

	case $\psi=2.7$ $p=3.85$	case $\psi=3.5$ $p=5.45$	case $\psi=7.5$ $p=7.2$
M (tm)	2.87	3.61	4.97
N (ton)			
Q: shear force (ton)	9.25	10.30	5.99
b (cm)	100	100	100
d (cm)	21.6 43.8	23	23
d' (cm)	7.0	7	7
As (cm^2)	Min. As = 6.57 φ12 - ctc - 150 = 7.53	φ12 - ctc 150 = 7.53	φ16 - ctc 150 = 13.40
d'/d			
f=(M/N)+u (cm)			
f/d			
n	15	15	15
nP=n·As/b·d	0.026	0.049	0.087
M'=M+N·u (tm)			
M'/(b·d^2) (kg/cm^2)	1.50	6.82	9.90
Q/b·d (kg/cm^2)			
C	10.5	8.2	6.64
S	91.3	22.5	12.9
Z			
σ_c (kg/cm^2)	16	56	62
σ_s (kg/cm^2)	927	2303	1819
τ (kg/cm^2)	6.0	4.5	2.6
σ_{ca} (kg/cm^2)			
σ_{sa} (kg/cm^2)			
τ_a (kg/cm^2)			

U-Type
Retaining wall

Vol. 3 U-Type Retaining wall

§1. Height 4.0^m



Calculation of Load

1) Concret weight and soil weight

$0.30 \times 1.50 \times 2.407 \times 2 = 2.17^t$

$1/2 \times (0.3 + 0.5) \times 2.00 \times 2.407 \times 2 = 3.85$

$3.00 \times 0.5 \times 2.407 = 3.61$

$2.40 \times 1.50 \times 1.9 = 6.84$

$1/2 \times (2.40 + 2.00) \times 2.00 \times 1.9 = 8.36$

24.83^t

2) live load

$0.51^{t/m^2} \times 2.4 = 1.22^{t/m}$

Nominal Horizontally load $1.4^{kN/m} = 0.143^{t/m}$

Soil reaction

$$q = \frac{24.83 + 1.22}{3.0} = 8.7^{t/m^2}$$

Calculation of Section

Sect - 1

earth pressure

$P_a = 0.6 \times \gamma \cdot h^2 \times 1/2$

$= 0.6 \times 1.9 \times 1.5^2 \times 1/2 = 1.283^{t}$

earth pressure due to surcharge

$P_{a\,add} = 0.6 \times 0.51 \times 1.5 = 0.559^{t}$

Bending Moment

$M = \overset{0.642}{1.283 \times 1/3 \times 1.5} + \overset{0.419}{0.559 \times \frac{1}{2} \times 1.5} + \overset{0.358}{0.193 \times (1.5 + 1.0)}$

$= 1.419^{t \cdot m}$

$\phi 12 - 150^{ctc} = 1.13 \times 6.667 = 7.53^{cm^2}$ $d = 23^{cm}$, $d' = 7^{cm}$

$np = 0.019$ $c = 8.2$ $s = 22.5$

$\sigma_c = 22^{kg/cm^2}$

$\sigma_s = 70.5^{kg/cm^2}$

Sect - 2

$P_a = 0.6 \times 1.9 \times 3.5^2 \times \frac{1}{2} = 6.983^{t}$

$P_{a\,add} = 0.6 \times 0.51 \times 3.5 = 1.071^{t}$

Bending Moment

$M = \overset{8.147}{6.983 \times 1/3 \times 3.5} + \overset{1.874}{1.071 \times \frac{1}{2} \times 3.5} + \overset{0.869}{0.193 \times (3.5 + 1.0)}$

$= 10.665^{t \cdot m}$

$\phi 16 - 150^{ctc} = 13.40^{cm^2}$ $np = 0.047$ $c = [illegible]$, $s = 23.5$

$\sigma_c = 48^{kg/cm^2}$

$\sigma_c = 2.033^{kg/cm^2}$