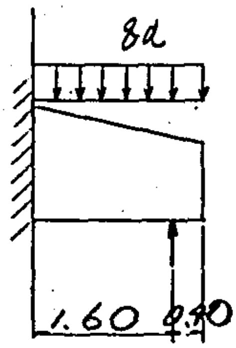
Acting point

$$x = 2.00 - 2.00 \times 1/3 \times \frac{2 \times 2.06 + 12.35}{2.06 + 12.35} = 1.24^{m}$$

$$M = (2.06 + 12.35) \times 2.00 \times 1/2 \times 1.24 = 17.87^{t \cdot m}$$

e. Stress caused by surcharge

$$q_a = 1.00^{t/m^2}$$



$$x = 2.00 \times 1/2 = 1.00^{m}$$

$$M = 1.00 \times 2.00 \times 1.00 = 2.00^{t \cdot m}$$

f. Summary of stresses

	pile reaction	own weight and surcharge	Earth pressure	Surcharge load	ΣM	Value converted to ordinary case
D+S+EP	25.35	-34.69	-17.87	-2.00	-29.21	-19.47

Minimum reinforcing is arranged since no tention acting at the upper part

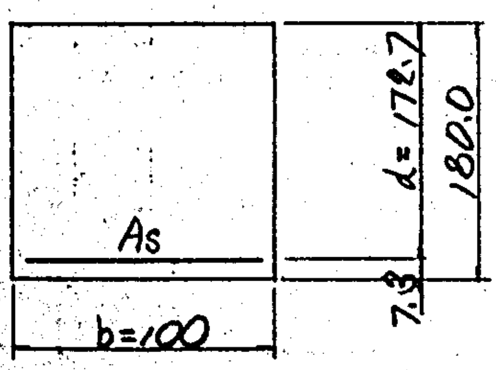
Stress calculation

$$D + T + I + E_p \begin{cases} M = 19.47 \text{ tm} \\ N = \text{———} \\ H = \text{———} \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	19.47	Q (t)	0
N (t)	0	u (cm)	82.7
b (cm)	100		
d (cm)	172.7	d′(cm)	7.3
As (cm2)	D 19 X 4 = 11.46		
As′ (cm2)			
d′/d	0.04	f (cm)	0
M′ (t·m)	19.47		
M′/bd2(kg/cm2)	0.65	Q/bd (kg/cm2)	0
nP	0.01	As′/As	0

C	S	Z
15.89	104.90	1.04

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
10.37 < 80	1,027.22 < 1800	0.00 < 4



As = D 19 - 250cm c.t.c

(2) Analysis of Shearing Stress.

a. Effective width

Calculation at rear heel

$B_0 = 1.33^{m}$, $A = 1.84^{m^2}$

b. Reaction force acting on pile.

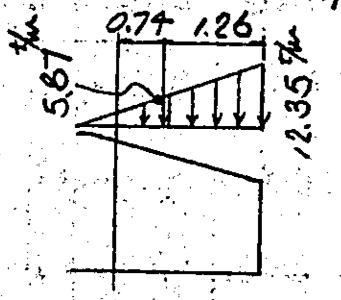
$D + S + E_p$

$P_{min} = 2.15^{t}$

c. Own weight of footing and weight of covering

$N = 34.81 \times \dfrac{1.26}{2.00} \times 1.33 = 29.17^{t}$

d. Vertical component of earth pressure.



$P_V = 1/2 \times (5.87 + 12.35) \times 1.26 \times 1.33$

$= 15.27^{t}$

e. Surcharge load

$g_d = 1.00^{t/m^2} \times 1.26 \times 1.33 = 1.68^{t}$

f. Shearing force

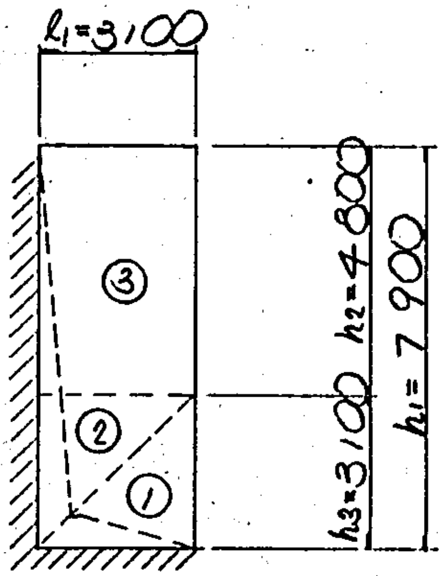
$S = -2.15 + 29.17 + 15.27 + 1.68 = 43.97^{t}$

g. Shearing stress

$\tau = \dfrac{43.97 \times 10^3}{18400} = 2.4^{kg/m^2} < 3.7 \times 1.5$

$= 5.55^{kg/m^2}$

[6] Design of wing wall



$h_3 = l_1$

D + T + I + Ep

$h' = 0.56 + 2.17$

$= 2.73^{m}$

coefficient of earth pressure

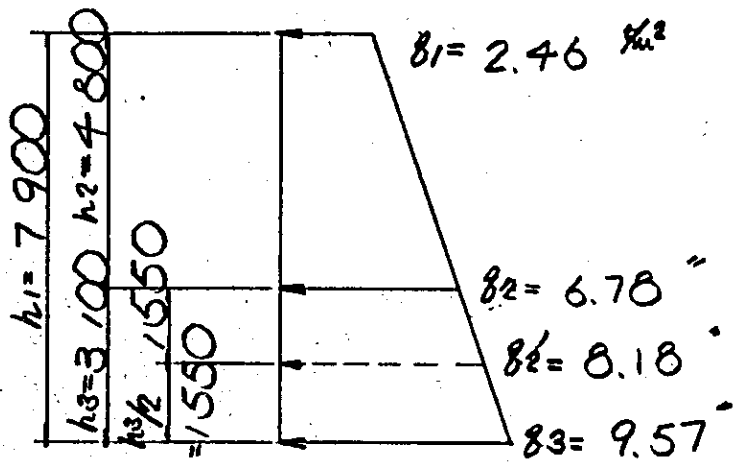
$K_0 = 0.5$

1. Earth pressure

$q_1 = 1.8 \times 0.5 \times (\overset{h'}{2.73} + 0) = 2.46$ t/m²

$q_2 = 1.8 \times 0.5 \times (\overset{h'}{2.73} + \overset{h_2}{4.80}) = 6.78$ "

$q_3 = 1.8 \times 0.5 \times (\overset{h'}{2.73} + \overset{h_1}{7.90}) = 9.57$ "



$$q_2' = \frac{\overset{q_3}{9.57} - \overset{q_2}{6.78}}{2} + \overset{q_2}{6.78} = 8.18 \text{ t/m}^2$$

2. Stress calculation on each section.

① $M_1 = \frac{1}{2} \times \overset{\delta_2'}{8.18} \times \overset{h_3/2}{1.55}^2 + \frac{1}{6} \times (\overset{\delta_3}{9.57} - \overset{\delta_2'}{8.18}) \times \overset{h_3/2}{1.55}^2$

$= 10.38$ t·m/m

$S_1 = \overset{\delta_2'}{8.18} \times \overset{h_3/2}{1.55} + \frac{1}{2} \times (\overset{\delta_3}{9.57} - \overset{\delta_2'}{8.18}) \times \overset{h_3/2}{1.55}$

$= 13.76$ t/m

② $M_2 = \frac{1}{2} \times \overset{\delta_2'}{8.18} \times \overset{\ell/2}{1.55}^2 = 9.83$ t·m/m

$S_2 = \overset{\delta_2'}{8.18} \times \overset{\ell/2}{1.55} = 12.68$ t/m

③ $M_3 = \frac{1}{2} \times (\overset{\delta_1}{2.46} + \overset{\delta_2}{6.78}) \times \frac{1}{2} \times \overset{\ell_1}{3.10}^2 = 22.20$ t·m/m

$S_3 = (\overset{\delta_1}{2.46} + \overset{\delta_2}{6.78}) \times \frac{1}{2} \times \overset{\ell_1}{3.10} = 14.32$ t/m

Stress calculation.

①

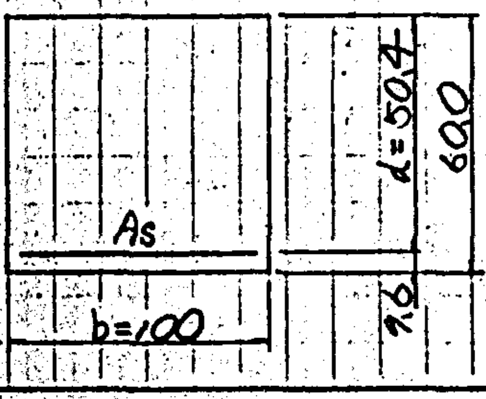
$D+T+I+Ep \left\{ \begin{array}{l} M = 10.38\ ^{t\cdot m} \\ N = \text{——} \\ H = 13.76\ ^{t} \end{array} \right.$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	10.38	Q (t)	0
N (t)	0	u (cm)	20.4
b (cm)	100		
d (cm)	50.4	d′(cm)	9.6
As (cm2)	D 22 X 4 = 15.484		
As′ (cm2)			
d′/d	0.19	f (cm)	0
M′ (t·m)	10.38		
M′/bd2(kg/cm2)	4.09	Q/bd (kg/cm2)	0
nP	0.046	As′/As	0

C	S	Z
8.39	23.79	1.09

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
34.31 < 80	1,458.71 < 1800	0.00 < 4

$$\tau = \frac{13.76\times10^3}{100\times50.4} = 2.7\ ^{kg/cm^2} < \tau_R = 3.7\ ^{kg/cm^2}$$



As = D 22 - 12.5 cm ctc

Stress calculation.

③

$D + T + I + Ep \left\{ \begin{array}{l} M = 22.20 \ ^{t\cdot m} \\ N = \text{—} \\ H = 14.32 \ ^{t} \end{array} \right.$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Τca (kg/cm2)	3.7		
M (t·m)	22.2	Q (t)	0
N (t)	0	u (cm)	22.6
b (cm)	100		
d (cm)	52.6	d'(cm)	7.4

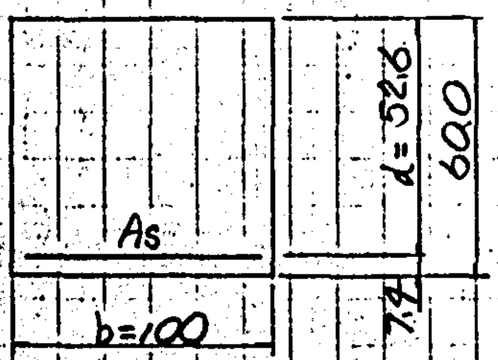
As (cm2) D 22 X 8 = 30.968

As' (cm2)			
d'/d	0.14	f (cm)	0
M' (t·m)	22.2		
M'/bd2(kg/cm2)	8.02	Q/bd (kg/cm2)	0
mP	0.088	As'/As	0

C	S	Z
6.61	12.76	1.12

Σc(kg/cm2)	Σs(kg/cm2)	Τc(kg/cm2)
53.06 < 80	1,536.49 < 1800	0.00 < 4

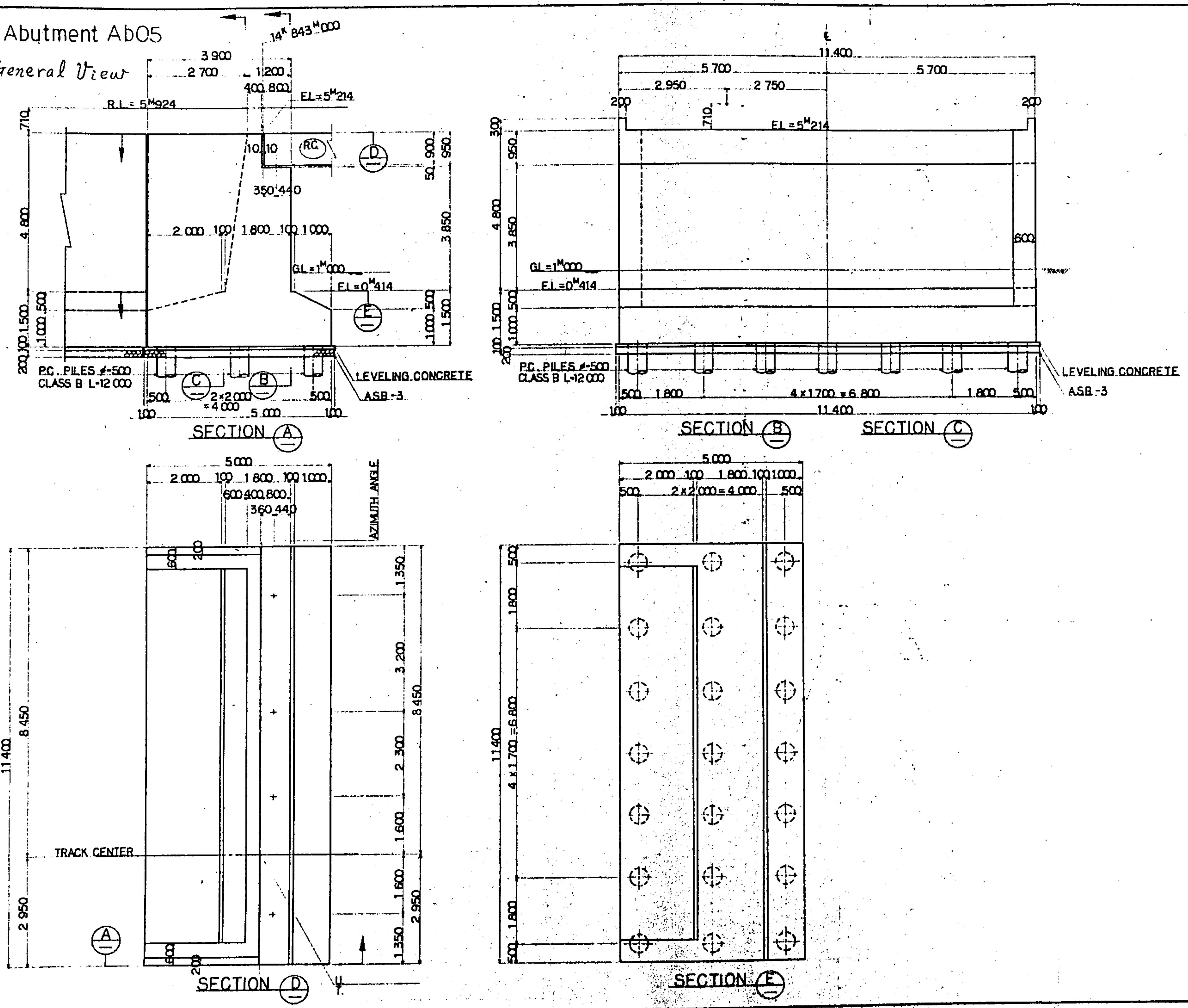
$$\tau = \frac{14.32 \times 10^3}{100 \times 52.6} = 2.7 \ ^{kg/cm^2} < \tau_R = 3.7 \ ^{kg/cm^2}$$



As = D 22 - ctc 12.5 cm

§4. Abutment Ab05

1 General View



NOTES:

1. ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE INDICATED
2. REFERENCE DRAWING FOR BAR ARRANGEMENT : CS - 142
   CS - 143
   CS - 144

2 Calculation of loads

1. Own weight of abutment and weight of coviring earth

(1) Eearth fill equivalent height of surcharge load

a. Dead load

1) distributed load of track weight	$0.45^{t/m} \times \frac{1}{11.40} \times 2$	$= 0.08^{t/m^2}$
2) ballast	$0.40^{m} \times 1.8^{t/m^3}$	$= 0.80$ "
		$0.88^{t/m^2}$

$$\therefore q \fallingdotseq 1.00^{t/m^2}$$

height of surcharge load

$$h_1 = \frac{q_1}{\gamma} = \frac{1.00}{1.80} = 0.56^{m}$$

b. Train load

For calculation of stability and design of struture body, full width of abutment is used.
For disign of wall, the width equivalent to the distance of sleeper length plus twice of ballast thickness is used.

Distributed load $(B = 1.0^{m})$

$$q_2 = \frac{P}{a \cdot b}$$

P : Train load per one driving axle $(P = 16^{t})$

a : Distance between axles $(a = 1.5^{m})$

b : Distribution width of train load in transversel direction

For the stability calculation, also for the design of structure body

$$b = 1.40^{m}$$

$$q_2 = \frac{16 \times 2}{1.5 \times 11.40} = 1.87 \ {}^{t}/_{m^2}$$

Height of surcharge load

$$h_2 = \frac{q_2}{\gamma} = \frac{1.87}{1.8} = 1.04^{m}$$

For the deign of breast wall

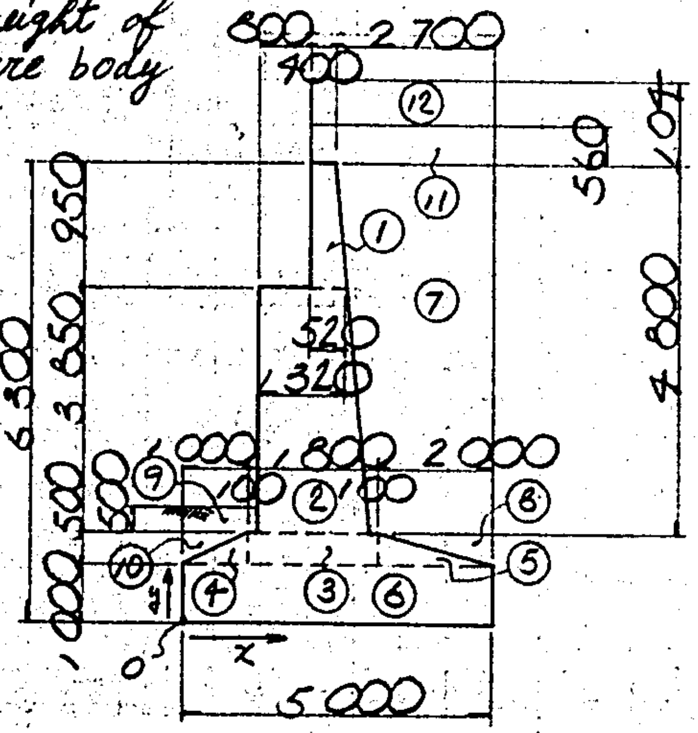
$$b = 2.00 + 2 \times 0.40 = 2.80^{m}$$

$$q_2' = \frac{16}{1.5 \times 2.80} = 3.81 \ {}^{t}/_{m^2}$$

Height of surchargn load

$$h_2' = \frac{q_2'}{\gamma} = \frac{3.81}{1.8} = 2.12 \ {}^{m}$$

2) Own weight of structure body



①	2.5 t/m³ × (0.40 + 0.52) × 0.95 × 1/2	= 1.10 t/m
②	2.5 × (1.32 + 1.80) × 3.85 × 1/2	= 15.06 "
③	2.5 × 2.00 × 0.50	= 2.50 "
④	2.5 × 1.00 × 0.50 × 1/2	= 0.63 "
⑤	2.5 × 2.00 × 0.50 × 1/2	= 1.25 "
⑥	2.5 × 5.00 × 1.00	= 12.50 "
⑦	1.8 × (2.70 + 2.10) × 4.80 × 1/2	= 20.74 "
⑧	1.8 × 2.00 × 0.50 × 1/2	= 0.90 "
⑨	1.8 × 1.10 × 0.50	= 0.99 "
⑩	1.8 × 1.00 × 0.50 × 1/2	= 0.45 "
⑪	0.56 × 2.70	= 1.51 "
⑫	1.04 × 2.70	= 2.81 "

	Vertical force (t) N	Horizontal distance (m) x	N · x (t·m)	Horizontal force (t) H	Vertical distance (m) y	H · y (t·m)
①	1.10	2.09	2.30	0.11	5.40	0.59
②	15.06	1.69	25.45	1.51	3.33	5.03
③	2.50	2.00	5.00	0.25	1.25	0.31
④	0.63	0.67	0.42	0.06	1.17	0.07
⑤	1.25	3.67	4.59	0.13	1.17	0.15
⑥	12.50	2.50	31.25	1.25	0.50	0.63
⑦	20.74	4.11	85.24	2.07	4.00	8.28
⑧	0.90	4.33	3.90	0.09	1.33	0.12
⑨	0.99	0.33	0.33	0.10	1.75	0.18
⑩	0.45	0.67	0.30	0.05	1.33	0.07
⑪	1.51	3.65	5.51	0.15	6.18	0.93
SUB TOTAL	57.63	—	164.29	5.77	—	16.51
⑫	2.81	3.65	10.26	—	—	—
TOTAL	60.44	—	174.55	—	—	—

$K_H = 0.10$

3) Loads acting on shoes.

(1) Vertical load on shoes

a. dead load.

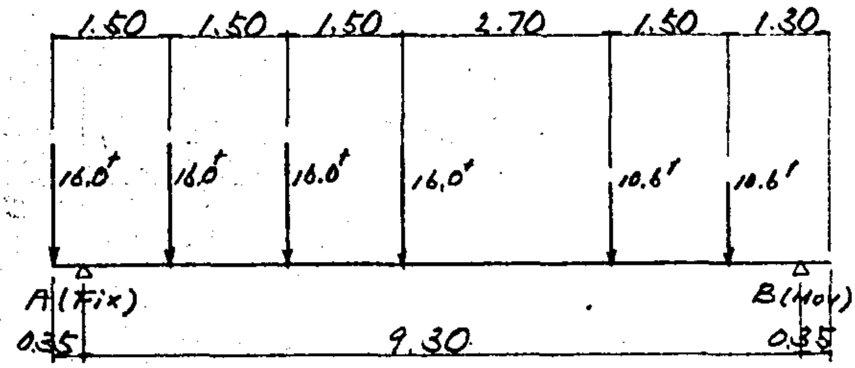
dead load of superstructure. R_d

Refer The Table, Reaction fore of superstructure)

$R_d = 138.00^{t}$ $\left(R_d = \dfrac{276.0}{11.4} = 24.21^{t/m} \right)$

b. Train Load

Reaction forces transmitted from KS Loading.



$$R_{\ell A} = \frac{1}{9.30} \times \{16.0 \times (5.15 + 6.65 + 8.15 + 9.65)$$

$$+ 10.6 \times (0.95 + 2.45)\} \qquad = 54.80^{t}$$

$$R_{\ell B} = (16.0 \times 4 + 10.6 \times 2) - 54.80 \qquad = 30.40\ ''$$

Impact Coefficient ($\ell = 9.30^{m}$)

$$i = 0.44 - \frac{9.30 - 5}{10 - 5} \times (0.44 - 0.43)$$

$$= 0.431$$

$$\therefore R_{A(\ell+i)} = 54.80 \times (1 + 0.431) \times 2$$

$$= 156.84^{t}$$

$$\left(R_{A(\ell+i)} = \frac{156.84}{11.40} = 13.76^{t/m} \right)$$

(2) Horizontal load acting at shoes

Horizontal load at the support of fixed end is assumed acting at the bottom face of shoe.

Brack load is assumed as 15% of Train load

$$T = 0.15 \times \Sigma Re$$
$$= 0.15 \times (\overset{R\ell A}{54.80} + \overset{R\ell B}{30.40}) = 12.78^t$$

Brack load acting at fixed end shoe

$$T_A = T_B = 0.55 \cdot T$$
$$= 0.55 \times 12.78$$
$$= 7.03^t$$

perone meter width of abutment

$$T_A = \frac{7.03}{11.40} = 0.62 \ ^t/_m$$

(3) Seismic load

$$KH = 0.10$$

Fixed end Support bearing.

$$H = 0.10 \times \Sigma Rd$$
$$= 0.10 \times 276.0 \times 2 \qquad = 55.20^t$$

$$HA = 55.20 - 1/2 \times 0.10 \times 276.0$$
$$= 41.40^t > H/2 = 55.20 \times 1/2 = 27.60^t$$

$$\therefore HA = \frac{41.40}{11.40} = 3.63 \ ^t/_m$$

4) Loads acting brest wall

(1) Dead load

Distributed dead load $q_1 = 1.00$ t/m²

Vertical force perone meter width of abutment. $D = 1.00 \times 0.40 = 0.40$ t/m

(2) Train load

a. stability colculation

Distributed train load. $q_2 = 1.87$ t/m²

Vestical force perone meter width of abutment $L = 1.87 \times 0.40 = 0.75$ t/m

b. desingn of breast wall

Vertical force force perone meter width of abutment

$$L = 16 \times \frac{1}{2.80} = 5.71 \text{ t/m}$$

5) Earth pressure

(1) Ordinary

$P = \frac{1}{2} \cdot \gamma \cdot (h + 2 \cdot h') \cdot h \cdot Ka$

γ; Unit weight of soil (t/m^3)

h; Height of acting point of earth pressure (m)

h'; Eearth equivalenf height of surcharge (m)

Ka; Coefficient of active earth pressure

$(\alpha = 0,\ \phi = 30,\ \delta = \phi = 30^\circ)$

$P = \frac{1}{2} \times 1.8 \times (6.30 + 2 \times 0.56) \times 6.30 \times 0.297$

$= 13.88$ t

$P_H = 13.88 \times \cos 30^\circ = 12.02$ t $(\leftarrow)$

$P_V = 13.88 \times \sin 30^\circ = 6.94$ " $(\downarrow)$

Coordinate of acting point $x = 5.00$ m

$y = 2.26$ "

(2) Ordinary + Temporary

$P = \frac{1}{2} \times 1.8 \times \{6.30 + 2 \times (1.04 + 0.56)\} \times 6.30$

$\times 0.297 = 17.78$ t

$P_H = 17.78 \times \cos 30^\circ = 15.40$ t $(\leftarrow)$

$P_V = 17.78 \times \sin 30^\circ = 8.89$ " $(\downarrow)$

Coordinate of acting point $x = 5.00$ m

$y = 2.45$ "

(3) Earth quake

Ke ; $(\alpha = 0,\ \phi = 30^\circ,\ \delta = \phi/2 = 15^\circ)$ 0.354

$$P = 1/2 \times 1.8 \times (6.30 + 2 \times 0.56) \times 6.30 \times 0.354$$

$$= 16.55^{t}$$

$P_H = 16.55 \times \cos 30^\circ = 14.33^{t}$ (←)

$P_V = 16.55 \times \sin 30^\circ = 8.28$ 〃 (↓)

Coordinate of acting point $x = 5.00^{m}$

$y = 2.26$ 〃

3 Calculation of stability

1. Dead load + Earth pressure

	Vertical force N (t)	Horizontal distance x (m)	N·x (t·m)	Horizontal force H (t)	Vertical distance y (m)	H·y (t·m)
Own weight and surcharge	57.63	—	164.29	—	—	—
Dead load acting at shoes	24.21	1.55	37.53	—	—	—
Dead load acting at breast wall	0.40	2.10	0.84	—	—	—
Active earth pressure	6.94	5.00	34.70	-12.02	2.26	-27.17
Total	89.18	—	237.36	-12.02	—	-27.17

Stress at point O

$N = 89.18^{t}$

$H = -12.02''$

$M = 237.36 - 27.17 = 210.19^{t\cdot m}$

2. Dead load + Train load + Impact + Earth pressure

	N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
Own weight and surcharge	60.44	—	174.55	—	—	—
Dead load acting at shoes	24.21	1.55	37.53	—	—	—
Train load acting at shoes	13.76	1.55	21.33	—	—	—
Dead load acting at breast wall	0.40	2.10	0.84	—	—	—
Train load acting at breast wall	0.75	2.10	1.58	—	—	—
Active earth pressure	8.89	5.00	44.45	-15.40	2.45	-37.73
Total	108.45	—	280.28	-15.40	—	-37.73

$N = 108.45^{t}$, $H = -15.40^{t}$

$M = 280.28 - 37.73 = 242.55^{t\cdot m}$

3. Dead load + Train load + Impact load + Earthe pressure + Bracke load

	$N^{(t)}$	$x^{(m)}$	$N \cdot x^{(t \cdot m)}$	$H^{(t)}$	$y^{(m)}$	$H \cdot y^{(t \cdot m)}$
D+T.I+EP	108.45	—	280.28	-15.40	—	-37.73
B	—	—	—	-0.58	5.35	-3.10
Total	108.45	—	280.28	-15.98	—	-40.83

$N = 108.45^{t}$

$H = -15.98$ ″

$M = 280.28 - 40.83 = 239.45^{t \cdot m}$

4. Dead load + Earth pressure + Seismic load.

	$N^{(t)}$	$x^{(m)}$	$N \cdot x^{(t \cdot m)}$	$H^{(t)}$	$y^{(m)}$	$H \cdot y^{(t \cdot m)}$
Own weight and surcharge	57.63	—	164.29	-5.77	—	-16.51
Dead load acting at shoes	24.21	1.55	37.53	-3.63	5.35	-19.42
Dead load acting at breast wall	0.40	2.10	0.84	-0.04	6.58	-0.26
Active earth Pressure	8.28	5.00	41.40	-14.33	2.26	-32.39
Total	90.52	—	244.06	-23.77	—	-68.58

$N = 90.52^{t}$

$H = -23.77$ ″

$M = 244.06 - 68.58 = 175.48^{t \cdot m}$

2. Design of reaction force acting on piles

(1) Moment of inertia of the group of piles with respect to the geometrical center

Namber of piles $n = 21^{pile}$

Coordinate of acting point $x = 2.50^{m}$

Moment of inertia of the group of piles with respect to the geometrical center $I = 4 \times 2.00^2 \times 2 = 56.00^{pile \cdot m^2}$

(2) Peaction force acting on pile

$$P = \frac{N}{n} \pm \frac{M}{I} \cdot x$$

P; Reaction force

N; Axial force acting at center of the btlom of footing. (t)

n; Nunber of piles. (pile)

M; Moment acting at the center of the bottom of footing (t·m)

I; Moment of inertia of the group of piles with respect to the geometrical center (pile·m²)

x; Distance from the geometrical center of the group of piles to the center of the pile to be calculated (m)

a. Dead load (D) + Eath pressure (Ep)

$$e = \frac{M}{N} = \frac{210.19}{89.18} = 2.36^{m}$$

$$e_0 = 2.50 - 2.36 = 0.14 \;''$$

$$P = \frac{89.18}{21} \pm \frac{89.18 \times 0.14}{56.00} \times 2.00 = 4.46 \pm 0.45$$

$$= \begin{cases} 4.91 & t/pile \; (P_{max} = 55.97 \; t/pile) \\ 4.01 & '' \; (P_{min} = 45.71 \; '' \;) \end{cases}$$

b. Dead load (D) + Traim load (T) + Impact load (I) + Eath pressure (EP)

$$e = \frac{242.55}{108.45} = 2.24^{m}$$

$$e_0 = 2.50 - 2.24 = 0.26^{m}$$

$$P = \frac{180.45}{21} \pm \frac{108.45 \times 0.26}{56.00} \times 2.00$$

$$= 5.16 \pm 1.01$$

$$= \begin{cases} 6.17 \ ^{t}/\text{pile} & (P_{max} = 70.34 \ ^{t}/\text{pile}) \\ 4.15 \ '' & (P_{min} = 47.31 \ '') \end{cases}$$

c. Dead load (D) + Train (T) + Impact load (I) + Eath pressure (EP) + Brake load (Br)

$$e = \frac{239.45}{108.45} = 2.21^{m}$$

$$e_0 = 2.50 - 2.21 = 0.29^{m}$$

$$P = \frac{108.45}{21} \pm \frac{108.45 \times 0.29}{56.00} \times 2.00$$

$$= 5.16 \pm 1.12$$

$$= \begin{cases} 6.28 \ ^{t}/\text{pile} & (P_{max} = 71.59 \ '') \\ 4.04 & (P_{min} = 46.06) \end{cases}$$

d. Dead load (D) + Eath pressure (EP) + Seismic load (Se)

$$e = \frac{175.48}{90.52} = 1.94^{m}$$

$$e_0 = 2.50 - 1.94 = 0.56^{m}$$

$$P = \frac{90.52}{21} \pm \frac{90.52 \times 0.56}{56.00} \times 2.00$$

$$= 4.31 \pm 1.81$$

$$= \begin{cases} 6.12 \ ^{t}/\text{pile} & (P_{max} = 69.77 \ ^{t}/\text{pile}) \\ 2.50 \ '' & (P_{min} = 28.50 \ '') \end{cases}$$

3. Stability against overturning

a. D+EP

$$\frac{P_{min}}{P_{max}} = \frac{45.71}{55.97} = 0.82 > 0.3$$

b. D+T+I+EP+Br

$$\frac{P_{min}}{P_{max}} = \frac{47.31}{70.34} = 0.67 > 0.1$$

c. D+T+I+EP+Br

$$\frac{P_{min}}{P_{max}} = \frac{46.06}{71.59} = 0.64 > 0.1$$

d. D+EP+Se

$$\frac{M}{N} = \frac{50.69}{90.52} = 0.56 < 2.00^{m}$$

4. Stability against vertical Support

a. D+EP

$P_{max} = 55.97\ ^{t}/_{Pile} < P_a = 58\ ^{t}/_{Pile}$

b. D+T+I+EP+Br

$P_{max} = 70.34\ ^{t}/_{Pile} < P_a = 88\ ^{t}/_{Pile}$

c. D+T+I+EP+Br

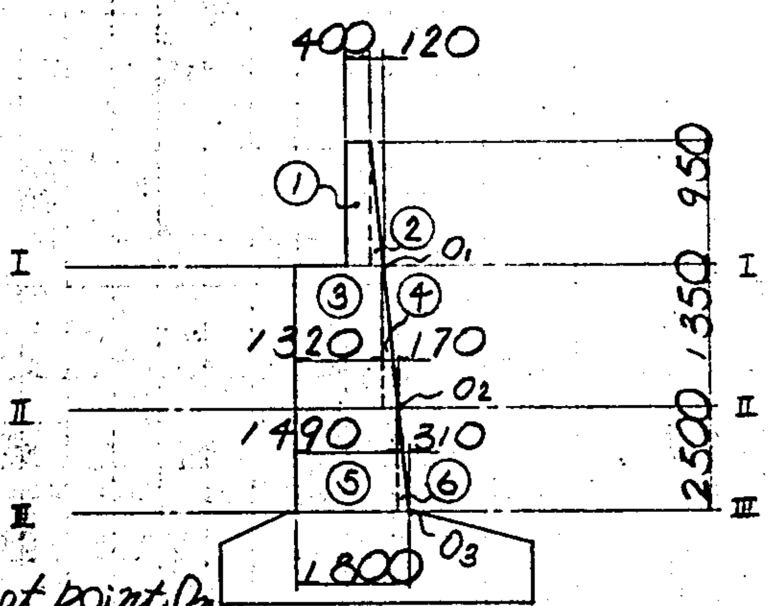
$P_{max} = 71.59\ ^{t}/_{Pile} < P_a = 88\ ^{t}/_{Pile}$

d. D+EP+Se

$P_{max} = 69.77\ ^{t}/_{Pile} < P_a = 117\ ^{t}/_{Pile}$

§5. Stress calculation of abutment body

1. Own weight of substructure and center of gravity calculated by sections



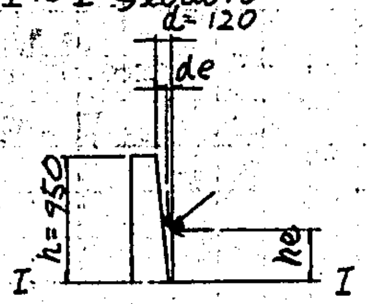
Stress at point On

			N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
I SECTION	①	2.5×0.40×0.95	0.95	0.32	0.30	0.10	0.48	0.05
	②	2.5×0.12×0.95×1/2	0.14	0.08	0.01	0.01	0.32	0.01
	Total		1.09	—	0.31	0.11	—	0.06
II SECTION	①	—	0.95	0.49	0.47	0.10	1.83	0.18
	②	—	0.14	0.25	0.04	0.01	1.67	0.02
	③	2.5×1.32×1.35	4.46	0.83	3.70	0.45	0.68	0.31
	④	2.5×0.17×1.35×1/2	0.29	0.11	0.03	0.03	0.45	0.01
	Total		5.84	—	4.24	0.59	—	0.52
III SECTION	①	—	0.95	0.80	0.76	0.10	4.33	0.43
	②	—	0.14	0.56	0.08	0.01	4.17	0.04
	③	—	4.46	1.14	5.08	0.45	3.18	1.43
	④	—	0.29	0.42	0.12	0.03	2.95	0.09
	⑤	2.5×1.49×2.50	9.31	1.06	9.87	0.93	1.25	1.16
	⑥	2.5×0.31×2.50×1/2	0.97	0.21	0.20	0.10	0.83	0.08
	Total		16.12	—	16.11	1.62	—	3.23

2. Earth pressure and its acting point

(1) Ordinary

a. I ~ I Section



$h' = 0.56^m$, $h = 0.95^m$

Ka ; ($\phi = 30°$, $\delta = 15°$, $\alpha = 2.579°$, $\beta = 0°$)

Ka = 0.320

$$P_a = 1/2 \times \gamma \times (h + 2 \cdot h') \times h \times Ka$$

$$P_a = 1/2 \times 1.8 \times (0.95 + 2 \times 0.56) \times 0.95 \times 0.320 = 0.57 \ ^{t/m}$$

$$P_H = 0.57 \times \cos(2.579° + 15°) = 0.54 \ ^{t/m} \quad (\leftarrow)$$

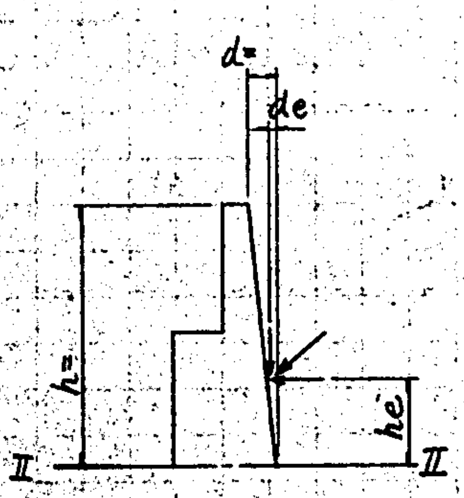
$$P_V = 0.57 \times \sin(2.579° + 15°) = 0.17 \quad (\downarrow)$$

Acting point

$$he = \frac{h}{3} \times \frac{h + 3h'}{h + 2h'} = \frac{0.95}{3} \times \frac{0.95 + 3 \times 0.56}{0.95 + 2 \times 0.56} = 0.40 \ ^m$$

$$de = \frac{d}{h} \times he = \frac{0.12}{0.95} \times 0.40 = 0.05 \ ^m$$

b. II ~ II Section



$h' = 0.56^m$, $h = 2.30^m$

$$P_a = 1/2 \times 1.8 \times (2.30 + 2 \times 0.56) \times 2.30 \times 0.320 = 2.27 \ ^{t/m}$$

$$P_H = 2.27 \times \cos(2.579° + 15°) = 2.16 \ ^{t/m} \quad (\leftarrow)$$

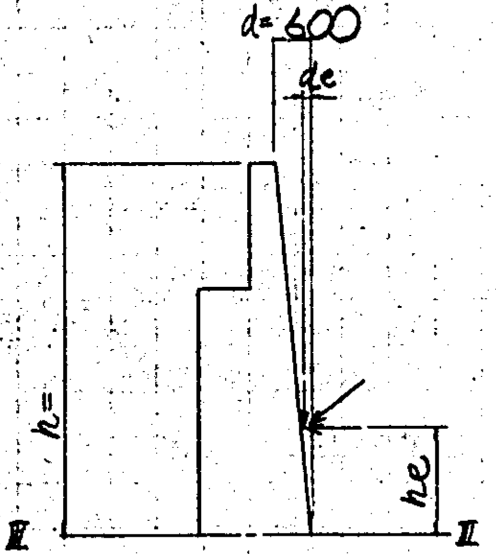
$$P_V = 2.27 \times \sin(2.579° + 15°) = 0.69 \quad (\downarrow)$$

acting point

$$he = \frac{2.30}{3} \times \frac{2.30 + 3\times 0.56}{2.30 + 2\times 0.56} = 0.89^{m}$$

$$de = \frac{0.29}{2.30} \times 0.89 = 0.11^{m}$$

C. Ⅲ ~ Ⅲ section



$h' = 0.56^{m}$

$h = 4.80^{m}$

$$Pa = 1/2 \times 1.8 \times (4.80 + 2\times 0.56) \times 4.80 \times 0.320 = 8.18^{t/m}$$

$$P_H = 8.18 \times \cos(2.579^\circ + 15^\circ) = 7.79^{t/m} \ (\leftarrow)$$

$$P_V = 8.18 \times \sin(2.579^\circ + 15^\circ) = 2.47 \ '' \ (\downarrow)$$

acting point

$$he = \frac{4.80}{3} \times \frac{4.80 + 3\times 0.56}{4.80 + 2\times 0.56} = 1.75^{m}$$

$$de = \frac{0.60}{4.80} \times 1.75 = 0.22^{m}$$

(2) Ordinary + Temporary

a. I ~ I Section

$h' = 2.12 + 0.56, \quad h = 0.95$

$K_A = 0.320$

$P_a = \frac{1}{2} \times 1.8 \times \{0.95 + 2 \times (2.12 + 0.56)\} \times 0.95$

$\times 0.320 = 1.73 \ ^{t/m}$

$P_H = 1.73 \times \cos(2.579^\circ + 15^\circ) = 1.65 \ ^{t/m} \ (\leftarrow)$

$P_V = 1.73 \times \sin(2.579^\circ + 15^\circ) = 0.52 \ ^{\prime\prime} \ (\downarrow)$

Actingpoint

$h_e = \frac{0.95}{3} \times \frac{0.95 + 3 \times 2.68}{0.95 + 2 \times 2.68} = 0.45 \ ^m$

$d_e = \frac{0.12}{0.95} \times 0.45 = 0.06 \ ^m$

b. II ~ II Section

$h' = 2.12 + 0.56, \quad h = 2.30$

$P_a = \frac{1}{2} \times 1.8 \times \{2.30 + 2 \times (2.12 + 0.56)\}$

$\times 0.320 = 5.07 \ ^{t/m}$

$P_H = 5.07 \times \cos(2.579^\circ + 15^\circ) = 4.83 \ ^{t/m} \ (\leftarrow)$

$P_V = 5.07 \times \sin(2.579^\circ + 15^\circ) = 1.53 \ ^{\prime\prime} \ (\downarrow)$

Actingpoint

$h_e = \frac{2.30}{3} \times \frac{2.30 + 3 \times 2.68}{2.30 + 2 \times 2.68} = 1.03 \ ^m$

$d_e = \frac{0.29}{2.30} \times 1.03 = 0.13 \ ^m$

C. III-III Section

$h' = 2.12 \quad , h = 4.80$

$Pa = 1/2 \times 1.8 \times \{ 4.80 + 2 \times (2.12 + 0.56) \} \times 4.80$

$\times 0.320 = 14.05 \ ^{t/m}$

$PH = 14.05 \times \cos (2.579^\circ + 15^\circ) = 13.39 \ ^{t/m} \ (\leftarrow)$

$PV = 14.05 \times \sin (2.579^\circ + 15^\circ) = 4.24 \ ^{\prime\prime} \ (\downarrow)$

Acting point

$he = \dfrac{4.80}{3} \times \dfrac{4.80 + 3 \times 2.68}{4.80 + 2 \times 2.68} = 2.02 \ ^{m}$

$de = \dfrac{0.60}{4.80} \times 2.02 = 0.25 \ ^{m}$

(3) Earth quake

a. I ~ I Section

$h' = 0.56$, $h = 0.95$

K_E ; ($\phi = 30°$, $\delta = 0°$, $\alpha = 2.579°$, $\beta = 0°$)

$K_E = 0.401$

$P_E = 1/2 \times \gamma \times (h + 2 \times h') \times h \times K_E$

$= 1/2 \times 1.8 \times (0.95 + 2 \times 0.56) \times 0.95 \times 0.401$

$= 0.71$ t/m

$P_{EH} = 0.71 \times \cos 2.579° = 0.71$ t/m ($\leftarrow$)

$P_{EV} = 0.71 \times \sin 2.579° = 0.03$ 〃 ($\downarrow$)

Acting point

$h_e = \dfrac{0.95}{3} \times \dfrac{0.95 + 3 \times 0.56}{0.95 + 2 \times 0.56} = 0.40$ m

$d_e = \dfrac{0.12}{0.95} \times 0.40 = 0.05$ m

b. II ~ II Section

$h' = 0.56$, $h = 2.30$

$P_E = 1/2 \times 1.8 \times (2.30 + 2 \times 0.56) \times 2.30 \times 0.401$

$= 2.84$ t/m

$P_{EH} = 2.84 \times \cos 2.579° = 2.84$ t/m ($\leftarrow$)

$P_{EV} = 2.84 \times \sin 2.579° = 0.13$ 〃 ($\downarrow$)

Acting point

$h_e = \dfrac{2.30}{3} \times \dfrac{2.30 + 3 \times 0.56}{2.30 + 2 \times 0.56} = 0.89$ m

$d_e = \dfrac{0.29}{2.30} \times 0.89 = 0.11$ m

c. III ~ III Section

$$h' = 0.56 \quad , \quad h = 4.80$$

$$P_E = 1/2 \times 1.8 \times (4.80 + 2 \times 0.56) \times 4.80 \times 0.401$$

$$= 10.26 \ ^{t/m}$$

$$P_{EH} = 10.26 \times \cos 2.579° = 10.26 \ ^{t/m} \ (\leftarrow)$$

$$P_{EV} = 10.26 \times \sin 2.579° = 0.46 \ '' \ (\downarrow)$$

Acting point

$$he = \frac{4.80}{3} \times \frac{4.80 + 3 \times 0.56}{4.80 + 2 \times 0.56} = 1.75 \ ^{m}$$

$$de = \frac{0.60}{4.80} \times 1.75 = 0.22 \ ^{m}$$

3. Stress calculation on each section

(1) I - I Section

		Vertical force (t) N	Horizontal distance x (m)	N·x (t·m)	Horizontal forse (t) H	Vertical distance y (m)	H·y (t·m)
①	Own weight of structure body	1.09	—	0.31	0.11	—	0.01
②	Dead load acting at breast wall	0.40	0.32	0.13	0.04	1.23	0.05
③	Train load and Impact load acting at breast wall	5.71	0.32	1.83	—	—	—
④	Dead load acting at shues	—	—	—	—	—	—
⑤	Train load and Impact load acting at shues	—	—	—	—	—	—
⑥	Brake load	—	—	—	—	—	—
⑦	Earth pressure (ordinary)	0.17	0.05	0.01	0.54	0.40	0.22
⑧	Earth pressure (ordinary + Temporary)	0.52	0.06	0.03	1.65	0.45	0.74
⑨	Earth pressure (earth quake)	0.03	0.05	0.01	0.71	0.40	0.28
Case 1	D + Ep	1.66	—	0.41	0.69	—	0.28
2	D+T+I+Ep	7.72	—	2.30	1.80	—	0.80
3	D+T+I+Ep+B	7.72	—	2.30	1.80	—	0.80
4	D+Ep (S)	1.52	—	0.45	0.86	—	0.34

Case 1 ; Dead load + Earth pressure ; ①+②+④ ⑦
$\alpha = 1.0$ (for crack analysis)

Case 2 ; Dead load + Train load + Impact load + Earth pressure
; ①~⑤, +⑧ $\alpha = 1.0$

Case 3 ; Dead load + Train load + Impact load + Earth pressure + Brake load
; ①~⑥, +⑧ $\alpha = 1.15$

Case 4 ; Dead load + Earth pressure (seismic)
; ①+②+④+⑨ $\alpha = 1.5$

(2) II - II Section

		Vertical force (t) N	Horizontal distance x (m)	N·x (t·m)	Horizontal forse (t) H	Vertical distance y (m)	H·y (t·m)
①	Own weight of structure body	5.84	—	4.24	0.59	—	0.52
②	Dead load acting at breast wall	0.40	0.49	0.20	0.04	2.58	0.10
③	Train load and Impact load acting at breast wall	5.71	0.49	2.80	—	—	—
④	Dead load acting at shues	24.21	1.04	25.18	3.63	1.35	4.90
⑤	Train load and Impact load acting at shues	13.76	1.04	14.31	—	—	—
⑥	Brake load	—	—	—	0.62	1.35	0.84
⑦	Earth prossure (ordinary)	0.69	0.11	0.08	2.16	0.89	1.92
⑧	Earth pressure (ordinary + Temporary)	1.53	0.13	0.20	4.83	1.03	4.97
⑨	Earth pressure (earth quake)	4.24	0.25	1.06	2.84	0.89	2.53
Case 1	D + Ep	31.14	—	29.70	2.16	—	1.92
2	D+T+I+Ep	51.45	—	46.93	4.83	—	4.97
3	D+T+I+Ep+B	51.45	—	46.93	6.20	—	5.81
4	D+Ep (S)	34.69	—	30.68	7.10	—	8.05

(3) III - III Section

		Vertical force (t) N	Horizontal distance x (m)	(t·m) N·x	Horizontal forse (t) H	Vertical distance y (m)	(t·m) H·y
①	Own weight of structure body	16.12	—	16.11	1.62	—	3.23
②	Dead load acting at breast wall	0.40	0.80	0.32	0.04	5.08	0.20
③	Train load and Impact load acting at breast wall	5.71	0.80	4.57	—	—	—
④	Dead load acting at shues	24.21	1.35	32.68	3.63	3.85	13.98
⑤	Train load and Impact load acting at shues	13.76	1.35	18.58	—	—	—
⑥	Brake load	—	—	—	0.62	3.85	2.39
⑦	Earth pressure (ordinary)	2.47	0.22	0.54	7.79	1.75	13.63
⑧	Earth pressure (ordinary + Temporary)	4.24	0.25	1.06	13.39	2.02	27.05
⑨	Earth pressure (earth quake)	0.46	0.22	0.10	10.25	1.75	17.94
Case 1	D + Ep	43.20	—	49.65	7.79	—	13.63
2	D+T+I+Ep	64.44	—	73.32	13.39	—	27.05
3	D+T+I+Ep+B	64.44	—	73.32	14.01	—	29.44
4	D+Ep (S)	41.19	—	49.21	15.54	—	35.35

Stress calculation I - I Section.

Dead load + train load + impact + earth pressure $\left\{ \begin{array}{l} M = 2.30 + 0.80 = 3.10 \ t.m \\ N = 7.72 \\ H = 1.80 \ t \end{array} \right.$

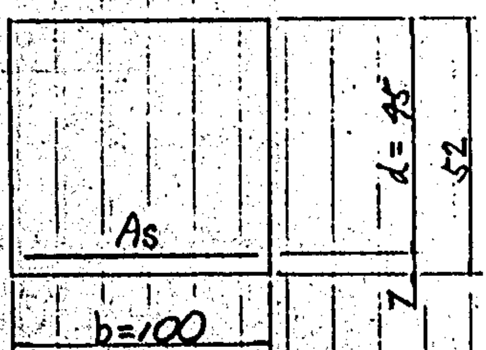
n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	3.1	Q (t)	1.8
N (t)	7.72	u (cm)	19
b (cm)	100		
d (cm)	45	d' (cm)	7
As (cm2)	D 13 X 8 = 10.136		
As' (cm2)			
d'/d	0.16	f (cm)	59.16
M' (t·m)	4.57		
M'/bd2(kg/cm2)	2.26	Q/bd (kg/cm2)	0.4
nP	0.034	As'/As	0

C	S	Z
6.29	11.12	1.02

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
14.19 < 80	376.34 < 1800	0.41 < 4

$$\tau = \frac{1.80 \times 10^3}{100 \times 45} = 0.4 \ kg/cm^2$$

$$< \tau_a = 3.7 \ kg/cm^2$$



As = D 13 - 12.5 cm ctc

Stress calculation II - II Section

Dead load + train load + impact + eath pressure
$$\left\{ \begin{array}{l} M = 46.93 + 4.97 = 51.90^{t \cdot m} \\ N = 51.45^{t} \\ H = 4.83^{t} \end{array} \right.$$

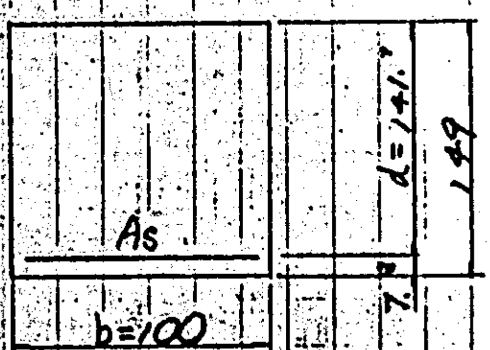
n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	51.9	Q (t)	4.8
N (t)	51.45	u (cm)	67.2
b (cm)	100		
d (cm)	141.7	d' (cm)	7.3
As (cm2)	D 19 X 4 = 11.46		
As' (cm2)			
d'/d	0.05	f (cm)	168.07
M' (t·m)	86.47		
M'/bd2(kg/cm2)	4.31	Q/bd (kg/cm2)	0.34
nP	0.012	As'/As	0

C	S	Z
8.03	21.31	0.96

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
34.61 < 80	1,377.07 < 1800	0.32 < 4

$$\tau = \frac{4.80 \times 10^3}{100 \times 141.7} = 0.3 \ ^{kg/cm^2}$$

$$< \tau_a = 3.7 \ ^{kg/cm^2}$$



As = D 19 - 25 cm ctc

Stress calculation III - III Section

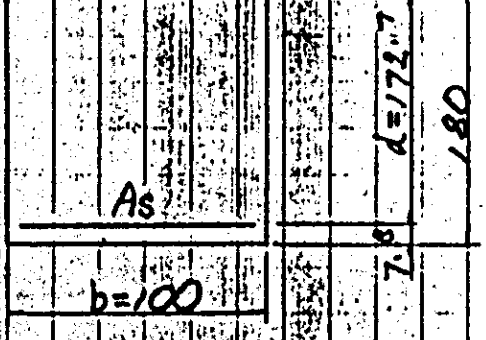
Dead load + train load + impact + eath pressure

$$\begin{cases} M = 73.32 + 27.05 = 100.37^{t \cdot m} \\ N = 73.32^{t} \\ H = 13.39^{t} \end{cases}$$

```
n                15
Σca (kg/cm2) 80                      Σsa (kg/cm2) 1800
Tca (kg/cm2) 3.7

M (t·m)         100.37               Q (t)          13.3
N (t)            73.32               u (cm)         82.7
b    (cm)       100
d    (cm)       172.7                d'(cm)          7.3

As   (cm2)        D 19 X 8 = 22.92
As'  (cm2)
d'/d            0.04                 f (cm)        219.59
M' (t·m)        161.01
M'/bd2(kg/cm2) 5.4                   Q/bd (kg/cm2) 0.78

nP              0.02                 As'/As          0

             C                 S                        Z
           7.26              16.44                     1.00

       Σc(kg/cm2)           Σs(kg/cm2)             Tc(kg/cm2)
      39.23 < 80        1,331.70 < 1800           0.77 < 4
```

$$\tau = \frac{13.30 \times 10^{3}}{100 \times 172.7} = 0.8 \ \mathrm{kg/cm^2}$$

$$< \tau_a = 3.7 \ \mathrm{kg/cm^2}$$



As = D 19 - 12.5 cm ctc

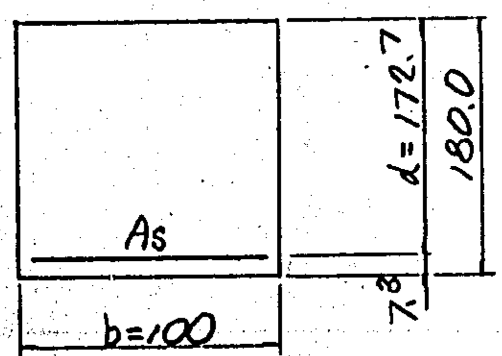
Stress calculation I-I section.

Dead load + train load + impact + earth pressure $\left\{ \begin{array}{l} M = 49.65 + 13.63 = 63.28 \ ^{tm} \\ N = 43.20 \ ^{t} \\ H = 7.79 \ ^{\prime\prime} \end{array} \right.$

n	15	Σsa (kg/cm2)	1400
Σca (kg/cm2)	80		
Tca (kg/cm2)	3.7		
M (t·m)	63.28	Q (t)	7.7
N (t)	43.2	u (cm)	82.7
b (cm)	100		
d (cm)	172.7	d′(cm)	7.3
As (cm2)	D 19 X 8 = 22.92		
As′ (cm2)			
d′/d	0.04	f (cm)	229.18
M′ (t·m)	99.01		
M′/bd2(kg/cm2)	3.32	Q/bd (kg/cm2)	0.45
nP	0.02	As′/As	0

C	S	Z
7.50	17.86	1.01

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
24.90 < 80	889.63 < 1400	0.45 < 4



As = D 19 - 12.5 cm c.t.c

5 Design of foundation

1. Reaction force Actieng on piles

Calculation of reaction force to be used for culculation of piles and foundation

$$P = \underbrace{\frac{N}{n} \pm \frac{M}{I} \cdot x}_{} \pm (\Delta N)$$

Refer the article of stability calculation

$$\Delta N = \frac{\Sigma M_0}{I} \cdot x$$

$$M_0 = \frac{H}{2\beta} \quad (M_0 ;\ \text{Moment at the pile top})$$

(1) D+ED

$$\Sigma M_0 = \frac{H}{2 \cdot \beta} = \frac{12.02 \times 11.40}{2 \times 0.177} = 387.08 \ t\cdot m$$

$$P = \begin{cases} 55.97 \\ 45.71 \end{cases} \pm \frac{387.08}{56.00} \times 2.0$$

$$= \begin{cases} 55.97 \\ 45.71 \end{cases} \pm 13.82 = \begin{cases} 69.79 \\ 31.89 \end{cases} \ t/pile$$

(2) D+T+I+Ep

$$\Sigma M_0 = \frac{15.40 \times 11.40}{2 \times 2.14} = 410.19 \ t\cdot m$$

$$P = \begin{cases} 70.34 \\ 47.31 \end{cases} \pm \frac{410.19}{56.00} \times 2.00$$

$$= \begin{cases} 70.34 \\ 47.31 \end{cases} \pm 14.65 = \begin{cases} 84.99 \\ 32.66 \end{cases} \ t/pile$$

(3) D+T+I+Ep+Br

$$\Sigma M_0 = \frac{15.98 \times 11.40}{2 \times 2.14} = 425.64 \ t\cdot m$$

$$P = \begin{cases} 71.59 \\ 46.06 \end{cases} \pm \frac{425.64}{56.00} \times 2.0$$

$$= \begin{cases} 71.59 \\ 46.06 \end{cases} \pm 15.20 = \begin{cases} 86.79 \\ 30.86 \end{cases} \ t/pile$$

(4). D + EP (seismic)

$$\Sigma M_0 = \frac{23.77 \times 11.40}{2 \times 2.14} = 633.13 \quad t\cdot m$$

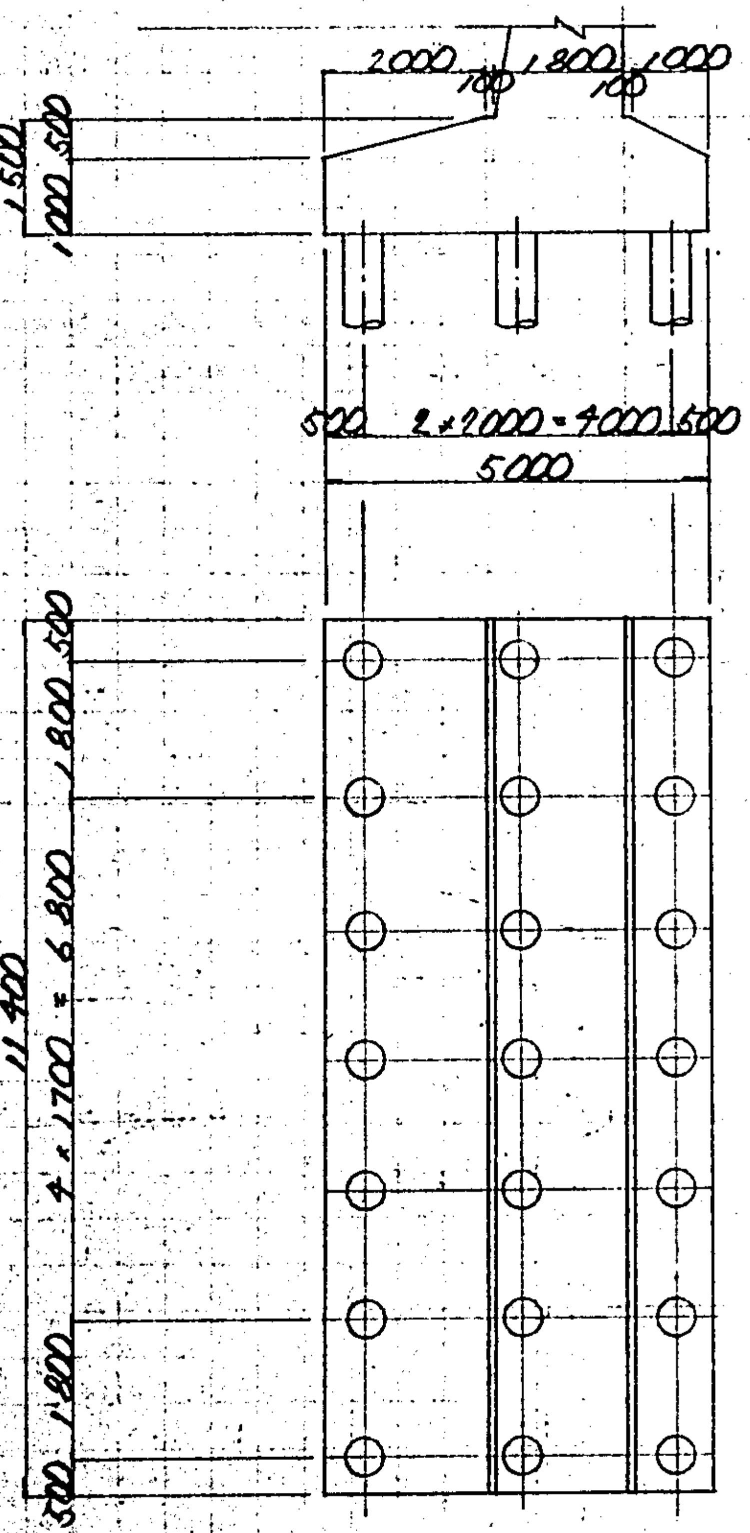
$$P = \begin{cases} 69.77 \\ 28.50 \end{cases} \pm \frac{633.13}{56.00} \times 2.00 \quad t/pile$$

$$= \begin{cases} 69.77 \\ 28.50 \end{cases} \pm 22.61 = \begin{cases} 92.38 \\ 5.89 \end{cases} \quad "$$

2. Calculation at front toe.

(1). Bending moment analysis

a. effective with of foundation



(1) Center part

$h = 1/2 \times 150 - 15 = 60$

$b_0 = D + 2h = 50 + 2 \times 60 = 170^{cm}$

$b = b_0 + 2e = 170 + 2 \times 60 = 290^{cm}$

$\nless$ distance between piles $= 170^{cm}$

$\therefore b = 170^{cm}$

(2) end part

$b_s = 50^{cm}$

$b/2 = 90^{cm}$

since,

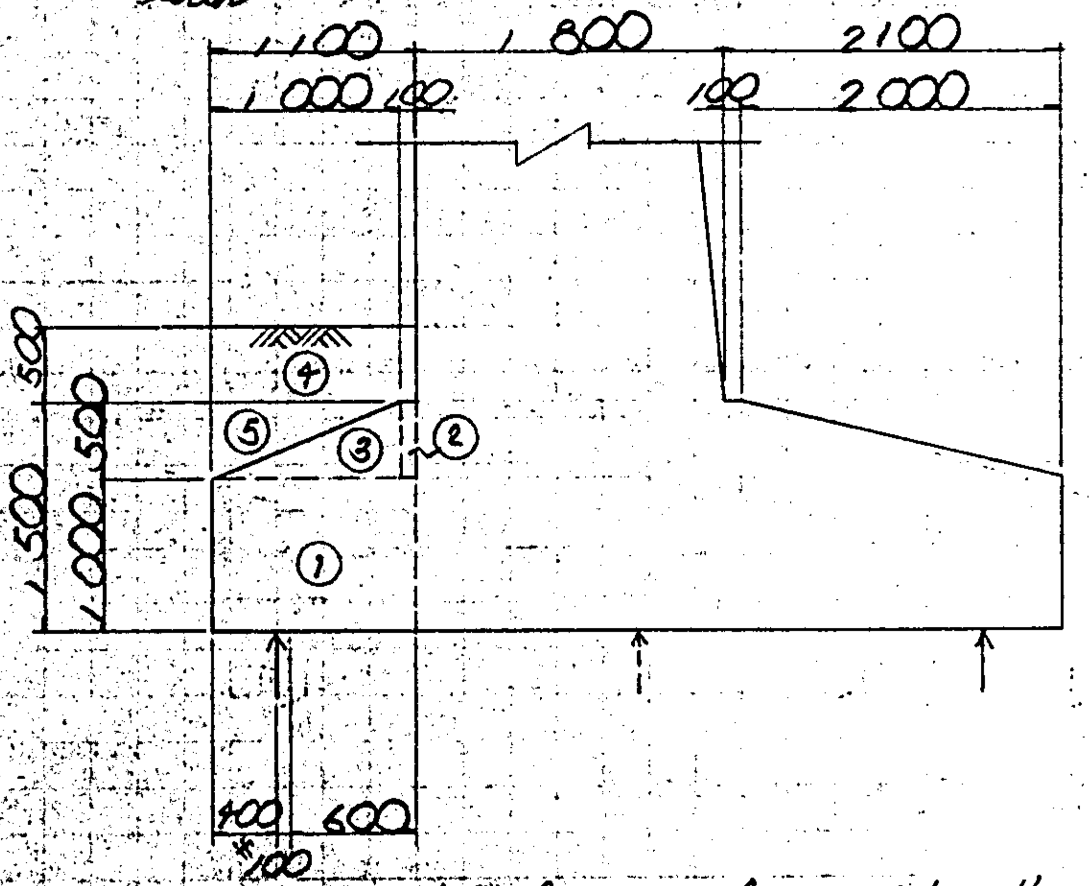
$b_s < b/2$

$b = b_s + b/2 = 50 + 90 = 140^{cm}$

b. Vertical reaction force acting on piles

Combined loads	Reaction force acting on piles P_{max} (t)
D+EP	69.79
D+T+I+EP	84.99
D+T+I+BF	86.79
D+Se+EP(se)	92.38

c. Stress of own weight of footing and weight of covering earth



* Tolerance for construction

(2) Analysis of Shearing Stress

a. Effective width.

$B_o = 1.51^{m} \qquad A = 2.04^{m^2}$

b. Reaction force acting on piles.

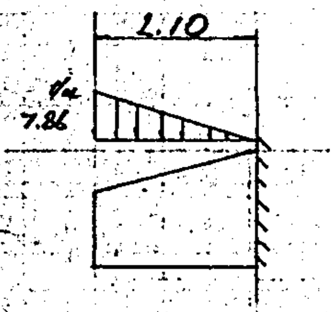
D + S + Ep

$P_{max} = 5.89^{t}$

c. Own weight of footing and weight of earth covering earth.

$N = 47.98 \times 1.51 = 72.45^{t}$

d. Vertical component of earth presser.



$P_V = \frac{1}{2} \times 7.86 \times 2.10 \times 1.51$

$= 12.46^{t}$

e. Surcharge load

$q_l = 1.00 \times 2.10 \times 1.51 = 3.17^{t}$

f. Shearing force.

$S = -5.89 + 72.45 + 12.46 + 3.17 = 82.19^{t}$

g. Shearing Stress.

$\tau = \dfrac{82.19 \times 10^3}{20400} = 4.0^{kg/cm^2} < 3.7 \times 1.5 = 5.55^{kg/cm^2}$

Own weight of footing and weight covering earth

	Calculation	N (t)	x (m)	M = N·x (t.m)
①	2.5 × 1.10 × 1.00	2.75	0.55	1.51
②	2.5 × 0.10 × 0.50	0.13	0.05	0.01
③	2.5 × 1.00 × 0.50 × 1/2	0.63	0.43	0.27
④	1.8 × 1.10 × 0.50	0.99	0.55	0.54
⑤	1.8 × 1.00 × 0.50 × 1/2	0.45	0.77	0.35
Σ	——	4.95	——	2.68

d. Calculation of section force

	per own pile			per meter				
case	$P_V \cdot l$ (t.m)	Mo (t.m)	M (t.m)	M/ef (t.m)	W (t.m)	M (t.m)	α	V (t.m)
case 1	48.85	-18.43	30.42	21.73	-2.68	19.05	1.00	19.05
〃 2	59.49	-19.53	39.96	28.54	-2.68	25.86	〃	25.86
〃 3	60.75	-20.27	40.48	28.91	-2.68	26.23	1.15	22.81
〃 4	64.67	-30.15	34.52	24.66	-2.68	21.98	1.50	14.65

ef : effectivewidth

W : Own weight of footing

V : Value converted to ordinary to ordinary case

Stress calculation. for crack analysis

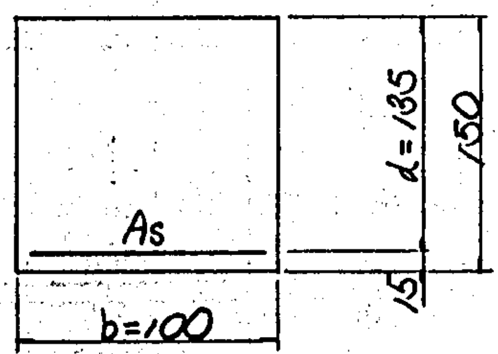
Dead load + trean load + Impact + earth pressur.

M = 25.86 t·m
N = ——
H = ——

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	25.86	Q (t)	0
N (t)	0	u (cm)	60
b (cm)	100		
d (cm)	135	d′(cm)	15
As (cm2)	D 16 X 8 = 15.888		
As′ (cm2)			
d′/d	0.11	f (cm)	0
M′ (t·m)	25.86		
M′/bd2(kg/cm2)	1.42	Q/bd (kg/cm2)	0
np	0.018	As′/As	0

C	S	Z
12.40	60.12	1.06

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
17.59 < 80	1,279.66 < 1800	0.00 < 4



As = D 16 - 12.5 cm c.t.c

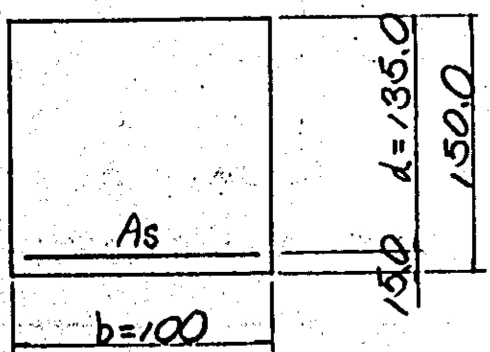
Stress calculation.

Dead load + earth pressure $\left\{ \begin{array}{l} M = 19.05\ ^{t\cdot m} \\ N = \text{——} \\ H = \text{——} \end{array} \right.$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1400
Tca (kg/cm2)	3.4		
M (t·m)	19.05	Q (t)	0
N (t)	0	u (cm)	60
b (cm)	100		
d (cm)	135	d′(cm)	15
As (cm2)	D 16 X 8 = 15.888		
As′ (cm2)			
d′/d	0.11	f (cm)	0
M′ (t·m)	19.05		
M′/bd2(kg/cm2)	1.05	Q/bd (kg/cm2)	0
np	0.018	As′/As	0

C	S	Z
12.40	60.12	1.06

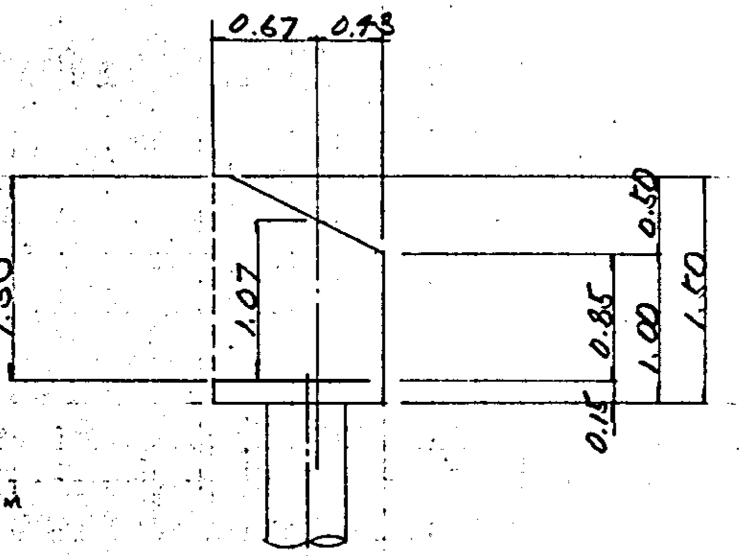
Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
12.96 < 80	942.67 < 1400	0.00 < 3



As = D 16 - 12.5cm c.t.c

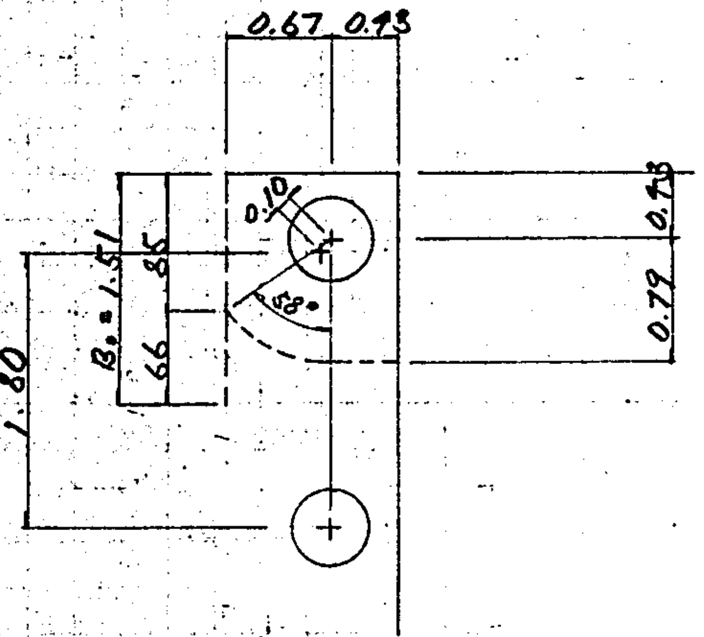
2) Calculation of shiring force

a effective length.



$$\overset{D}{0.50} + \overset{d_1}{1.07} \;\; d = 0.50 + 1.07 = 1.57^{m}$$

$$d/2 = 1.57 \times 1/2 = 0.79''$$



1, effective area of shearing stress caused by bending.

$$B_0 = l_1 + l_2$$

$$= 0.6 \times 1.80 + 0.43 = 1.51^{m}$$

$$A = B_0 \; d$$

$$= 1.51 \times 1.35 = 2.04^{m^2}$$

(ii) Effective area resisting punching shear.

$$A = \Sigma b \cdot d$$

$$= 1/2 \times (0.85 + 1.07) \times 0.67 + 1.50 \times 0.85$$

$$+ 58^\circ/180^\circ \times 3.142 \times 0.79 \times 1/2 \times (1.07 + 1.50)$$

$$= 3.95^{m^2}$$

b. Own weight of footing and weight of covering earth.

Resisting area for normal load

$$A = 0.43^2 + 0.43 \times (0.67 + 0.79) + 3.142 \times 0.79^2$$

$$\times 58/360 + 0.67 \times 0.42 \times 1/2 = 1.27^{m^2}$$ (for punching shear)

$$A = 1.51 \times 1.10 = 1.66^{m^2}$$ (for bending shear)

$$\overset{(1.27)}{1.66} \times (1.00 + 1.50) \times 1/2 \times 2.5 = \overset{(3.97)}{5.19}$$

$$\overset{(1.27)}{1.66} \times (0.50 + 1.00) \times 1/2 \times 1.8 = \overset{(1.71)}{2.24}$$

$$\Sigma = 7.43^t$$

(5.68)

(3) Calculation of Shering Stress

D+T+I+Ep is assumed

$$S = 84.99 - \underset{}{\overset{(5.68)}{7.43}} = \overset{(79.31)}{77.56}\ t$$

a. Shearing stress caused by bending

$$\tau = \frac{77.56 \times 10^3}{20400} = 3.8\ kg/cm^2 < \tau_a = 3.7\ kg/cm^2$$

b. Shearing stress caused by punching

$$\tau = \frac{79.31 \times 10^3}{39500} = \quad kg/cm^2 < \tau_a = 5.1\ kg/cm^2$$

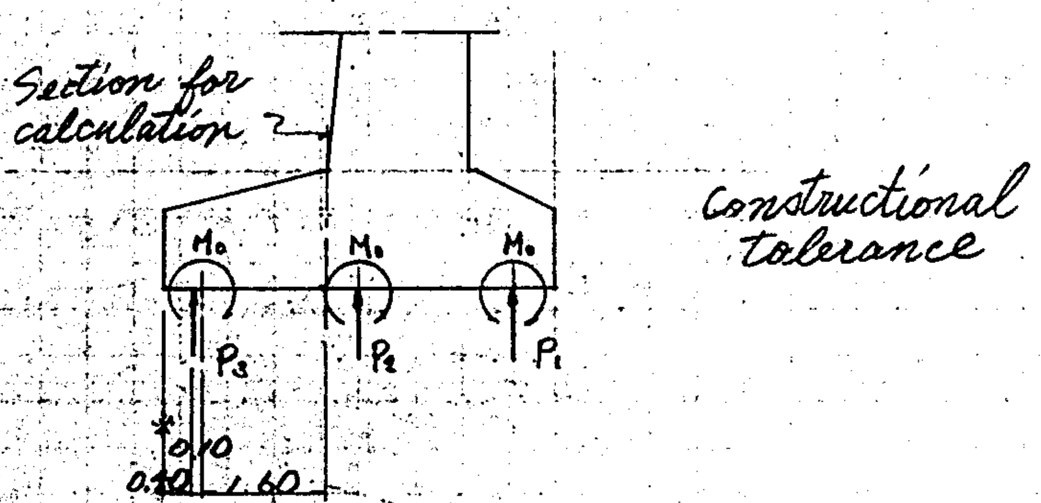
3. Calculation at rear heel.

(1) Bending moment analysis

◦ upside reinforcing is calculated on the condition of dead load + seismic load + earth pressure, referred the reaction of piles.

a. effective width of footing. Calculation at rear heel, referred 2, (1), a $b = 1.40^{m}$

b. Stress caused by reaction force of pile.



$P_3 = 5.89^{t}$

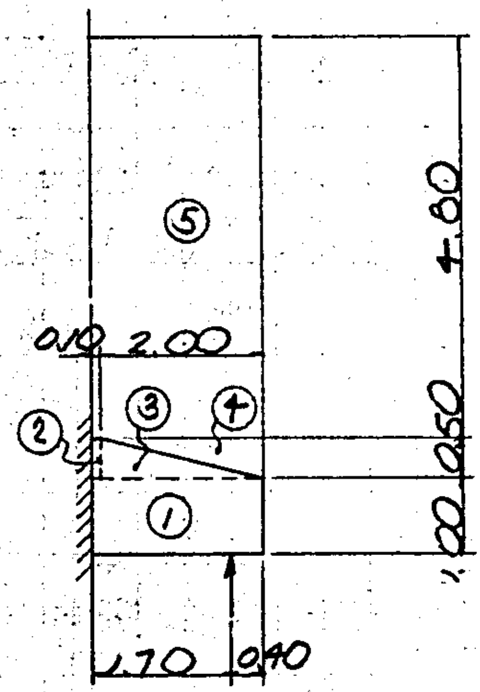
$M_0 = 30.15^{t.m}$

$M = 5.89 \times 1.70 + 30.15 = 40.16^{t.m}$

per meter moment will be,

$M = 40.16 \times 1/1.40 = 28.69^{t.m}$

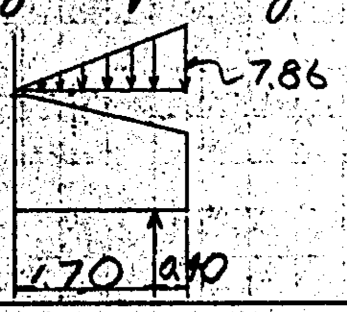
c. Own weight of footing and weight of covering earth



	Calculation	N (t)	x (m)	N·x (t·m)
①	2.5 × 2.10 × 1.00	27.56	1.05	28.94
②	2.5 × 0.10 × 0.50	0.13	0.05	0.01
③	2.5 × 2.00 × 0.50 × 1/2	1.25	0.77	0.96
④	1.8 × 2.00 × 0.50 × 1/2	0.90	1.43	1.29
⑤	1.8 × 2.10 × 4.80	18.14	1.05	19.05
Σ	———	47.98	——	50.25

d. Stress caused by vertical component of earth pressure

Vertical component of active earth pressure Pv is suppased acting at the virtual back face with the load of increasing uniformly



D + S + EP

$P_V = 8.28$ t

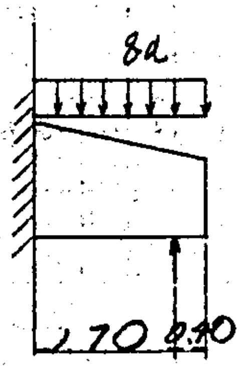
Acting point

$$x = 2.10 \times 1/3 \times 2 = 1.40^{m}$$

$$M = 7.86 \times 2.10 \times 1/2 \times 1.40 = 11.55^{t \cdot m}$$

e. Stress caused by surcharge

$$q_a = 1.00^{t/m^2}$$



$$x = 2.10 \times 1/2 = 1.05^{m}$$

$$M = 1.00 \times 2.10 \times 1.05 = 2.21^{t \cdot m}$$

f. Summary of stresses

	pile reaction	own weight and surcharge	Earth pressure	Surcharge load	Σ M	Value converted to ordinary case
D+S+EP	28.69	-50.25	-11.55	-2.21	-35.32	-23.55

Minimum reinforcing is arranged, since no tension acting at the upper part.

Stress calculation.

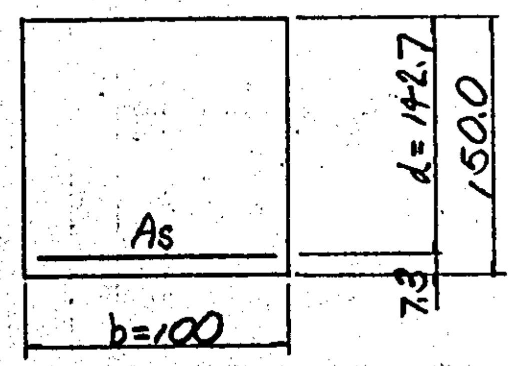
D + S + E

M = 23.55 t·m
N = ———
H = ———

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	23.55	Q (t)	0
N (t)	0	u (cm)	67.7
b (cm)	100		
d (cm)	142.7	d′(cm)	7.3
As (cm2)	D 19 X 4 = 11.46		
As′ (cm2)			
d′/d	0.05	f (cm)	0
M′ (t·m)	23.55		
M′/bd2(kg/cm2)	1.16	Q/bd (kg/cm2)	0
np	0.012	As′/As	0

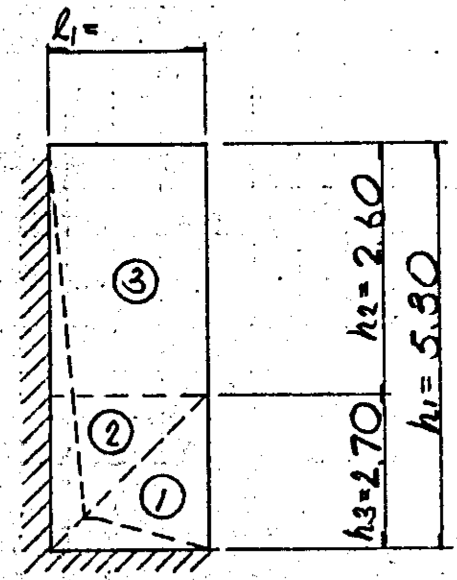
C	S	Z
14.60	86.95	1.05

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
16.89 < 80	1,508.50 < 1800	0.00 < 4



As = D 19 - 25 ctc cm

6 Design of wing wall



$h_3 = l_1$

D + T + I + Ep

$h' = 0.56 + 2.12$

$= 2.68^{m}$

Coefficient of earth pressure

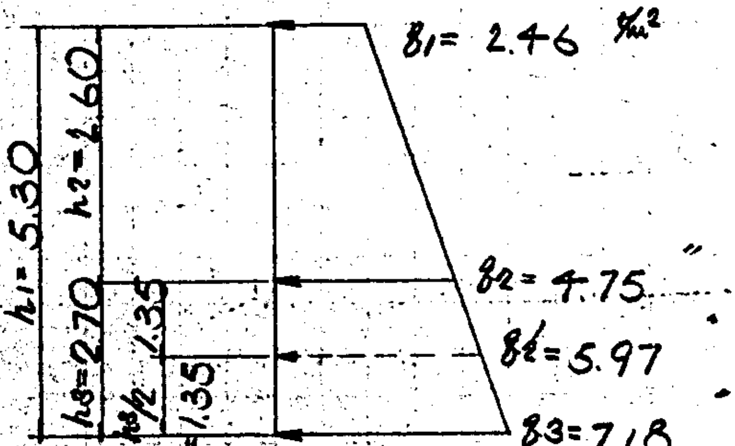
$K_0 = 0.5$

1. Earth pressure

$q_1 = 1.8 \times 0.5 \times (\overset{h'}{2.68} + 0) = 2.46\ t/m^2$

$q_2 = 1.8 \times 0.5 \times (\overset{h'}{2.68} + \overset{h_2}{2.60}) = 4.75$ 〃

$q_3 = 1.8 \times 0.5 \times (\overset{h'}{2.68} + \overset{h_1}{5.30}) = 7.18$ 〃



$$q_2' = \frac{\overset{q_3}{7.18} - \overset{q_2}{4.75}}{2} + \overset{q_2}{4.75} = 5.97\ t/m^2$$

2. Stress calculation on each section

① $M_1 = \frac{1}{2} \times \overset{\delta_2'}{5.97} \times \overset{h_3/2}{1.35}^2 + \frac{1}{6} \times (\overset{\delta_3}{7.18} - \overset{\delta_2'}{5.97}) \times \overset{h_3/2}{1.35}^2$

$= 5.81 \quad t \cdot m/m$

$S_1 = \overset{\delta_2'}{5.97} \times \overset{h_3/2}{1.35} + \frac{1}{2} \times (\overset{\delta_3}{7.18} - \overset{\delta_2'}{5.97}) \times \overset{h_3/2}{1.35}$

$= 8.88 \quad t/m$

② $M_2 = \frac{1}{2} \times \overset{\delta_2'}{5.97} \times \overset{l/2}{1.35}^2 \qquad = 5.44 \quad t \cdot m/m$

$S_2 = \overset{\delta_2'}{5.97} \times \overset{l/2}{1.35} \qquad = 8.06 \quad t/m$

③ $M_3 = \frac{1}{2} \times (\overset{\delta_1}{2.46} + \overset{\delta_2}{4.75}) \times \frac{1}{2} \times \overset{l_1}{2.70}^2 = 13.14 \quad t \cdot m/m$

$S_3 = (\overset{\delta_1}{2.46} + \overset{\delta_2}{4.75}) \times \frac{1}{2} \times \overset{l_1}{2.70} \qquad = 9.73 \quad t/m$

Stress calculation.

① D + T + I + Ep $\begin{cases} M = 5.81\ ^{t.m} \\ N = \text{—} \\ H = 8.88\ ^{t} \end{cases}$

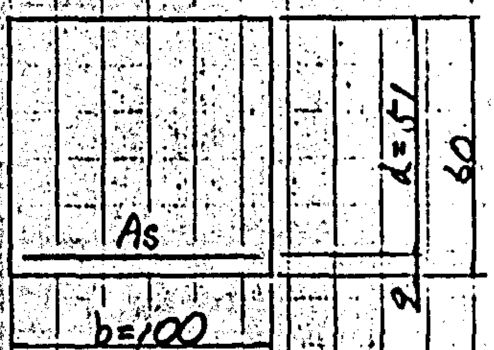
n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	5.81	Q (t)	0
N (t)	0	u (cm)	21
b (cm)	100		
d (cm)	51	d' (cm)	9
As (cm2)	D 16 X 4 = 7.944		
As' (cm2)			
d'/d	0.18	f (cm)	0
M' (t·m)	5.81		
M'/bd2(kg/cm2)	2.23	Q/bd (kg/cm2)	0
nP	0.023	As'/As	0

C	S	Z
11.01	45.70	1.06

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
24.60 < 80	1,531.53 < 1800	0.00 < 4

$\tau = \dfrac{8.88 \times 10^3}{100 \times 51} = 1.7\ ^{kg}/_{cm^2}$

$< \tau_a = 3.7\ ^{kg}/_{cm^2}$



As = D 16 - 25 cm ctc

Stress calculation

③

$D + T + I + E_p \left\{ \begin{array}{l} M = 13.14\ ^{t\cdot m} \\ N = \text{—} \\ H = 9.73\ ^{t} \end{array} \right.$

n 15
Σca (kg/cm2) 80 Σsa (kg/cm2) 1800
Tca (kg/cm2) 3.7

M (t·m)	13.14	Q (t)	0
N (t)	0	u (cm)	22.7
b (cm)	100		
d (cm)	52.7	d' (cm)	7.3

As (cm2) D 19 X 8 = 22.92
As' (cm2)

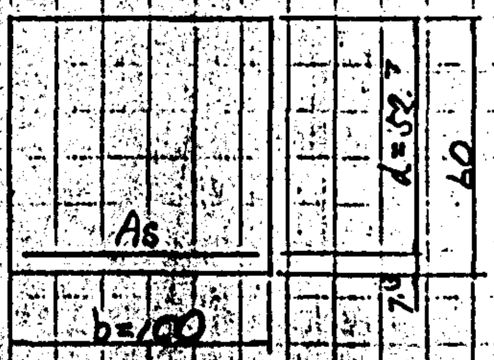
d'/d	0.14	f (cm)	0
M' (t·m)	13.14		
M'/bd2(kg/cm2)	4.73	Q/bd (kg/cm2)	0
nP	0.065	As'/As	0

C	S	Z
7.36	17.05	1.11

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
34.86 < 80	1,210.36 < 1800	0.00 < 4

$$\tau = \frac{9.73 \times 10^3}{100 \times 52.7} = 1.8\ ^{kg/cm^2}$$

$$< \tau_a = 3.7\ ^{kg/cm^2}$$



As = D 19 - 12.5 cm ctc

Referred the data of boring log.

$$\bar{N} = \frac{42 + 40 + 47}{3} = 43$$

$$AP = 1/4 \times 3.1416 \times 0.45^2 = 0.1590 \; m^2$$

$$\therefore QP = 10 \times 43 \times 0.1590$$
$$= 68.4^t$$

2) Ultimate skin priction.

Refer hapter 12, "campaction method of heading part of bored hofe pile.

$$QF = \phi . \Sigma f . h$$

QF : Untimate vertical supporling power of pile feared by skin friction.

ϕ : Circumferenfial fength of pile.

f : Ultimate skin friction on earth layer.

$$f = 0.1 . N$$
$$= 0.1 \times 25 = 2.5 \; t/m^2$$

$$\therefore QF = 1.41 \times 2.5 \times 7.00 = 24.7 \; t/pile$$

31. Safe supporting of pile.

$$Ra = \frac{1}{FS} (Qp + Q_s)$$

Ra : Safe supporting power of pile at pile top
Fs : Safety factor corresponding to each loading condition
Qp : Ultimate supporting power at pile tip.
Qs : Skin friction of pile.

a) Ordinary
$FS = 3$

$Pa = \frac{1}{3} \times (68.4 + 24.7) = 31.0$

b) Ordinary + Temporary.
$FS = 2$

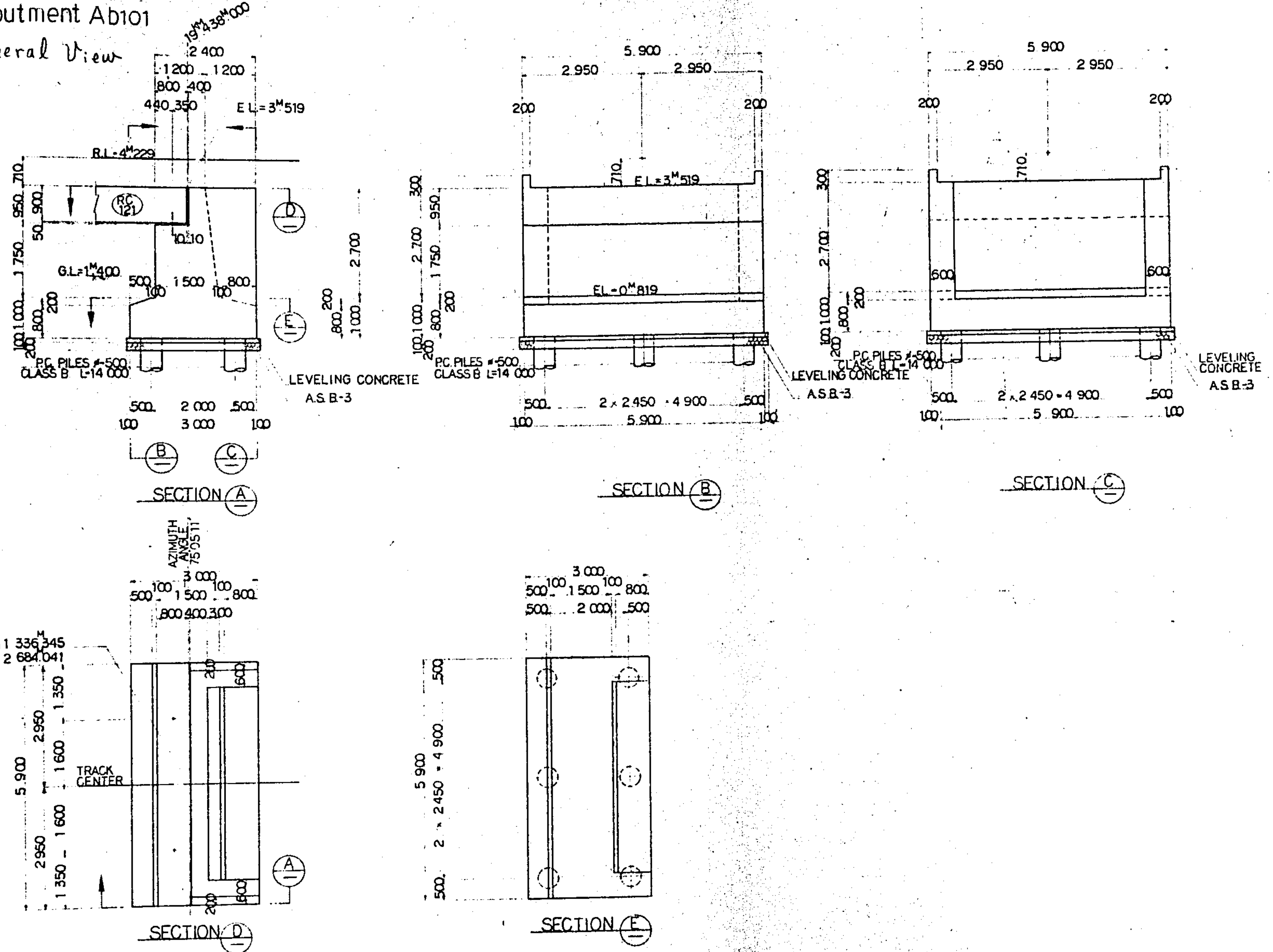
$Pa = \frac{1}{2} \times (68.4 + 24.7) = 46.6$ t/pile

c) Earthquake (Rain load. seismic)
$FS = 1.5 \quad (1.2)$

$Pa = \frac{1}{1.5} \times (68.4 + 24.7) = 62.1$ t/pile
(1.2) (77.6)

§5 Abutment Ab101

[1] General View



NOTES:

1. ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE INDICATED
2. REFERENCE DRAWING FOR BAR ARRANGEMENT CS - 142
   CS - 143
   CS - 144

2 Calculation of loads.

1. own weight of abutment and weight of covering earth

1) Eearth fill eguivapent height of surcharge load

a. dead load

distributed load of track weight $0.45^{t/m} \times \frac{1}{5.90} = 0.08^{t/m^2}$

ballast $0.40^{m} \times 1.8^{t/m^3} = 0.72$

$0.80^{t/m^2}$

$\therefore q_1 \fallingdotseq 1.00^{t/m^2}$

height of surcharge load $h_1 = \frac{q_1}{\gamma} = \frac{1.00}{1.8} = 0.56^{m}$

b. Train load

For calculation of stability and design of structure body, pull width of abutment is used.

For design of wall the width equivalent to The distance of sleeper length plus twice of ballast thickness is used.

distributed load $(B = 1.0^{M})$

$q_2 = \frac{P}{a \cdot b}$

P : train load per one driving axpe $(P = 16^{t})$

a : distance between axes $(a = 1.5^{m})$

b : distribution width of train load in transversal direction

$b = 5.90^{m}$

$q_2 = \frac{16}{1.5 \times 5.90} = 1.81^{t/m^2}$

height of surcharge load $h_2 = \frac{q_2}{\gamma} = \frac{1.81}{1.8} = 1.01^{m}$

For the design of brest wall

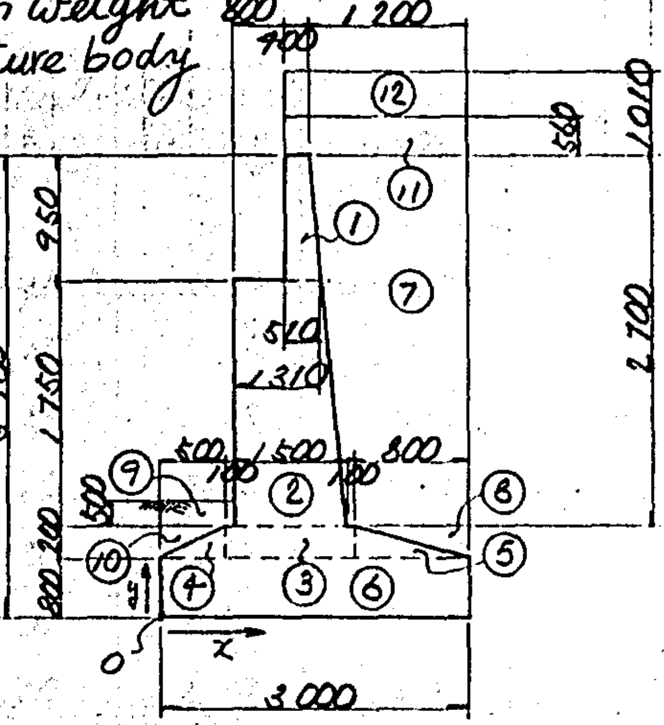
$$b = 2.00 + 2 \times 0.40 = 2.80^{m}$$

$$q_2' = \frac{16}{1.5 \times 2.80} = 3.81 \ {}^{t}/_{m^2}$$

height surcharge load

$$h_2' = \frac{q_2'}{\gamma} = \frac{3.81}{1.8} = 2.12^{m}$$

2) Own weight of structure body



①	$2.5^{t/m^3} \times (0.40 + 0.51) \times 0.95 \times 1/2 =$	$1.08^{t/m}$
②	$2.5 \times (1.31 + 1.50) \times 1.75 \times 1/2 =$	6.15 "
③	$2.5 \times 1.70 \times 0.20 =$	0.85 "
④	$2.5 \times 0.50 \times 0.20 \times 1/2 =$	0.13 "
⑤	$2.5 \times 0.80 \times 0.20 \times 1/2 =$	0.20 "
⑥	$2.5 \times 3.00 \times 0.80 =$	6.00 "
⑦	$1.8 \times (1.20 + 0.90) \times 2.70 \times 1/2 =$	5.10 "
⑧	$1.8 \times 0.80 \times 0.20 \times 1/2 =$	0.14 "
⑨	$1.8 \times 0.60 \times 0.50 =$	0.54 "
⑩	$1.8 \times 0.50 \times 0.20 \times 1/2 =$	0.09 "
⑪	$0.56 \times 1.20 =$	0.67 "
⑫	$1.01 \times 1.20 =$	1.21

	Vertical porce (t) N	Horizontal distance (m) x	N·x (t·m)	Horizontal porce (t) H	Vertical y (m)	H·y (t·m)
①	1.08	1.61	1.74	0.11	3.22	0.35
②	6.15	1.22	7.50	0.62	1.86	1.15
③	0.85	1.35	1.15	0.09	0.90	0.08
④	0.13	0.33	0.04	0.01	0.87	0.01
⑤	0.20	2.47	0.49	0.02	0.87	0.02
⑥	6.00	1.50	9.00	0.60	0.40	0.24
⑦	5.10	2.59	13.21	0.51	2.42	1.23
⑧	0.14	2.73	0.38	0.01	0.93	0.01
⑨	0.54	0.30	0.16	0.05	1.25	0.06
⑩	0.09	0.17	0.02	0.01	0.93	0.01
⑪	0.67	2.40	1.61	0.07	3.98	0.28
	0.95	—	35.30	2.10	—	3.44
⑫	1.21	2.40	2.90	—	—	—
	22.16	—	38.20	—	—	—

$K_H = 0.10$

3) Loads acting on shoes

(1) Vertical load on shoes

a. dead load

dead load of superstructure R_d

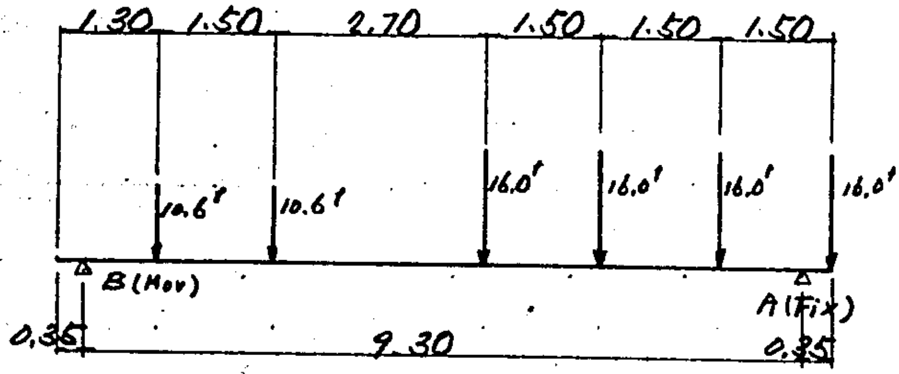
(Reper the Table, Reaction fore of Superstructure)

$R_d = 138.00^{t}$ $\quad (R_d = \frac{138.00}{5.90} = 23.39^{t/m})$

b. Train load:

reaction force of abutment caused by KS loading

② - ⑤ , ...



$$R_{lA} = \frac{1}{9.30} \times \{16 \times (5.15 + 6.65 + 8.15 + 9.65)$$

$$+ 10.6 \times (0.95 + 2.45)\} = 54.80 \ ^{t}$$

$$R_{lB} = (16 \times 4 + 10.6 \times 2) - 54.80 = 30.40 \ ''$$

Impact Coefficient ($l = 9.30 \ ^{m}$)

$$i = 0.44 - \frac{9.30 - 5}{10 - 5} \times (0.44 - 0.43)$$

$$= 0.431$$

$$\therefore R_{A(l+i)} = 54.80 \times (1 + 0.431)$$

$$= 78.42 \ ^{t}$$

$$\left(R_{A(l+i)} = \frac{78.42}{5.90} = 13.46 \ ^{t/m} \right)$$

(2) Horizontal load acting at shoes over turning moment acting at mid point

Horizontal load at the support of fixed end is assumed acting at the bottom pace of shoe.

Brake load is assumed as 15% of train load

$$T = 0.15 \times \Sigma R\ell$$

$$= 0.15 \times (\overset{R\ell A}{54.80} + \overset{R\ell B}{30.40}) = 12.78^{t}$$

Brake load acting at fixed end shoe TA = TB)

$$T_A = T_B = 0.55 \cdot T$$

$$= 0.55 \times 12.78$$

$$= 7.03^{t}$$

per one meter width of abutment

$$T_A = \frac{7.03}{5.90} = 1.19 \ \text{t/m}$$

(3) Seismic load

$$K_H = 0.10$$

Fixed end support bearing

$$H = 0.10 \times \Sigma Rd$$

$$= 0.10 \times 138.00 \times 2 = 27.60^{t}$$

$$H_A = 27.60 - 1/2 \times 0.10 \times 138.00$$

$$= 20.70^{t} > H/2 = 27.60 \times 1/2 = 13.80^{t}$$

$$\therefore H_A = \frac{20.70}{5.90} = 3.51 \ \text{t/m}$$

4) Loads acting breast wall

(1) Dead load

Distributed dead load $q_1 = 1.00$ t/m²

vertical force per one meter width of abutment $D = 1.00 \times 0.40 = 0.40$ t/m

(2) Train load

a. stability calculation and design of substructure body

Distubuted train load $q_2 = 1.81$ t/m²

vertical force per one meter width of abutment $L = 1.81 \times 0.40 = 0.72$ t/m

b. design of breast wall

Vertical force per one meter width of abutment $L = 16 \times \frac{1}{2.80} = 5.71$ t/m

5) Earth pressure

(1) Ordinary

$$P = 1/2 \cdot \gamma \cdot (h + 2 \cdot h') \cdot h \cdot Ka$$

γ; Unit weight of soil (t/m^3)

h; Height of acting point of earth pressure. (m)

h'; Eearth equivalenf height of surcharge (m)

Ka; Coefficient of active earth pressure.

$(\alpha = 0,\ \phi = 30^\circ,\ \delta = \phi = 30^\circ)$

$$P = 1/2 \times 1.8 \times (3.70 + 2 \times 0.56) \times 3.70 \times 0.297$$

$$= 4.77\ ^{t}$$

$$P_H = 4.77 \times \cos 30^\circ = 4.13\ ^{t}\ (\leftarrow)$$

$$P_V = 4.77 \times \sin 30^\circ = 2.39\ ^{''}\ (\downarrow)$$

Coordinate of acting point x = 3.00 ^{m}

y = 1.38 $^{''}$

(2) Ordinary + Temporary

$$P = 1/2 \times 1.8 \times \{3.70 + 2 \times (1.01 + 0.56)\} \times 3.70$$

$$\times 0.297 = 6.76\ ^{t}$$

$$P_H = 6.76 \times \cos 30^\circ = 5.85\ ^{t}\ (\leftarrow)$$

$$P_V = 6.70 \times \sin 30^\circ = 3.38\ ^{''}\ (\downarrow)$$

Coordinate of acting point x = 3.00 ^{m}

y = 1.51 $^{''}$

(3) Earthquake.

K_e ; ($\alpha = 0$, $\phi = 30^\circ$, $\delta = \phi/2 = 15^\circ$) 0.354

$P = 1/2 \times 1.8 \times (3.70 + 2 \times 0.56) \times 3.70 \times 0.354$
$= 5.68^{t}$

$P_H = 5.68 \times \cos 30^\circ = 4.92^{t}$ ($\leftarrow$)

$P_V = 5.68 \times \sin 30^\circ = 2.84$ 〃 ($\downarrow$)

Coordinate of acting point $x = 3.00^{m}$

$y = 1.38$ 〃

[3] Calculation of stability

1. Dead load + Earth pressure

	Vertical porce (t) N (t)	Horizontal distance x (m)	$N \cdot x$ (t·m)	Horizontal force (t) H (t)	Vertical distance y (m)	$H \cdot y$ (t·m)
Own weight and Surcharge	20.95	—	35.30	—	—	—
Dead load acting at Shoes	23.39	1.05	24.56	—	—	—
Dead load acting at breast wall	0.40	1.60	0.64	—	—	—
Active earth pressure	2.39	3.00	7.17	-4.13	1.38	-5.70
Total	47.13	—	67.67	-4.13	—	-5.70

Stress at point O

$N = 47.13^{t}$

$H = -4.13$ "

$M = 67.67 - 5.70 = 61.97^{t \cdot m}$

2. Dead load + Train load + Impact load + Earth pressure

	N (t)	x (m)	$N \cdot x$ (t·m)	H (t)	y (m)	$H \cdot y$ (t·m)
Own weight and Surcharge	22.16	—	38.20	—	—	—
Dead load acting at Shoes	23.39	1.05	24.56	—	—	—
Train load acting at Shoes	13.46	1.05	14.13	—	—	—
Dead load acting at breast wall	0.40	1.60	0.64	—	—	—
Train load acting at breast wall	0.72	1.60	1.15	—	—	—
Active earth pressure	3.38	3.00	10.14	-5.85	1.51	-8.83
Total	63.51	—	88.82	-5.85	—	-8.83

$N = 63.51^{t}$, $H = -5.85^{t}$

$M = 88.82 - 8.83 = 79.99^{t \cdot m}$

3. Dead load + Train load + Impact load + Earth pressure + Brake load

	N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
D+T.I+EP	63.51	—	88.82	-5.85	—	-8.88
B	—	—	—	1.19	2.75	3.27
Total	63.51	—	88.82	-4.66	—	-5.61

$N = 63.51$ t

$H = -4.66$ 〃

$M = 88.82 - 5.61 = 83.21$ t·m

4. Dead load + Earth pressure + Seismic load

	N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
own weight and surcharge	20.95	—	35.30	2.10	—	3.74
Dead load acting at shoes	23.39	1.05	24.56	3.51	2.75	9.65
Dead load acting at breast wall	0.40	1.60	0.64	0.04	3.98	0.16
Active earth pressure	2.84	3.00	8.52	-4.92	1.38	-6.79
Total	47.58	—	69.02	0.73	—	6.76

$N = 47.58$ t

$H = 0.73$ 〃

$M = 69.02 + 6.76 = 75.78$ t·m

2. Design of reaction force acting on piles

(1) Moment of inertia of the group of piles with respect to the geometrical center

number of piles $n = 6$ pile

Coordinate of acting point $x = 1.00$ m

Moment of inertia of the group of piles with respect to the geometrical center

$$I = 3 \times 1.00^2 \times 2 = 6.00 \text{ pile}^2$$

(2) Reaction force acting on piles

$$P = \frac{N}{n} \pm \frac{M}{I} \cdot x$$

N ; Axial force acting at the center of the bottom of footing (t)

n ; Number of piles

M ; Moment acting at the center of the bottom of footing (t.m)

I ; Moment of inertia of the group of piles with respect to the geometrical center

x ; Distance from the geometrical center (m) of the group of piles to the center of the pile to be calculated

a. Dead load + eath pressure

$$e = \frac{M}{N} = \frac{61.97}{47.13} = 1.31 \text{ m}$$

$$e_0 = 1.50 - 1.31 = 0.19 \text{ "}$$

$$P = \frac{47.13}{6} \pm \frac{47.13 \times 0.19}{6.00} \times 1.00$$

$$= 7.86 \pm 7.49$$

$$= \begin{cases} 9.35 \text{ t/pile} & (P_{max} = 55.17 \text{ t/pile}) \\ 6.37 \text{ "} & (P_{min} = 37.58 \text{ "}) \end{cases}$$

b. Dead load + Train load + Impact load + Eath pressure

$$e = \frac{79.99}{63.51} = 1.26^{m}$$

$$e_0 = 1.50 - 1.26 = 0.24^{m}$$

$$P = \frac{63.51}{6} \pm \frac{63.51 \times 0.24}{6.00} \times 1.00$$

$$= 10.59 \pm 2.54$$

$$= \begin{cases} 13.13 \ {}^{t/pile} & (P_{max} = 77.47 \ {}^{t/pile}) \\ 8.06 \ '' & (P_{min} = 47.50 \ '') \end{cases}$$

c. Dead load + Train load + Impact load + Eath pressure + Brake load

$$e = \frac{83.21}{63.51} = 1.31^{m}$$

$$e_0 = 1.50 - 1.31 = 0.19^{m}$$

$$P = \frac{63.51}{6} \pm \frac{63.51 \times 0.19}{6.00} \times 1.00$$

$$= 10.59 \pm 2.01$$

$$= \begin{cases} 12.60 \ {}^{t/pile} & (P_{max} = 74.34 \ {}^{t/pile}) \\ 8.58 \ '' & (P_{min} = 50.62 \ '') \end{cases}$$

d. Dead load + Eath pressure + Seismic load

$$e = \frac{75.48}{47.58} = 1.59^{m}$$

$$e_0 = 1.50 - 1.59 = -0.09^{m}$$

$$P = \frac{47.58}{6} \pm \frac{47.58 \times (-0.09)}{6.00} \times 1.00$$

$$= 7.93 \pm 0.71$$

$$= \begin{cases} 8.64 \ {}^{t/pile} & (P_{max} = 50.98 \ {}^{t/pile}) \\ 7.22 \ '' & (P_{min} = 42.60 \ '') \end{cases}$$

3 Stability against overturning

a. Dead load + Earth pressure

$$\frac{P_{min}}{P_{max}} = \frac{37.58}{55.17} = 0.68 > 0.3$$

b. Dead load + Train load + Impact load + Earth pressure

$$\frac{P_{min}}{P_{max}} = \frac{47.50}{77.47} = 0.61 > 0.1$$

c. Dead load + Train load + Impact load + Earth pressure + Brake load

$$\frac{P_{min}}{P_{max}} = \frac{50.62}{74.34} = 0.68 > 0.1$$

d. Dead load + Earth pressure + Seismic load

$$\frac{M}{N} = \frac{47.58 \times (-0.09)}{47.58} = 0.09 < 1.50^{m}$$

4. Stability against vertical support

a. Dead load + Earth pressure

$$P_{max} = 55.17 < P_a = 58^{t}/\text{pile}$$

b. Dead load + Train load + Impact load + Earth pressure

$$P_{max} = 77.47 < P_a = 88^{t}/\text{pile}$$

c. Dead load + Train load + Impact load + Earth pressure + Brake load

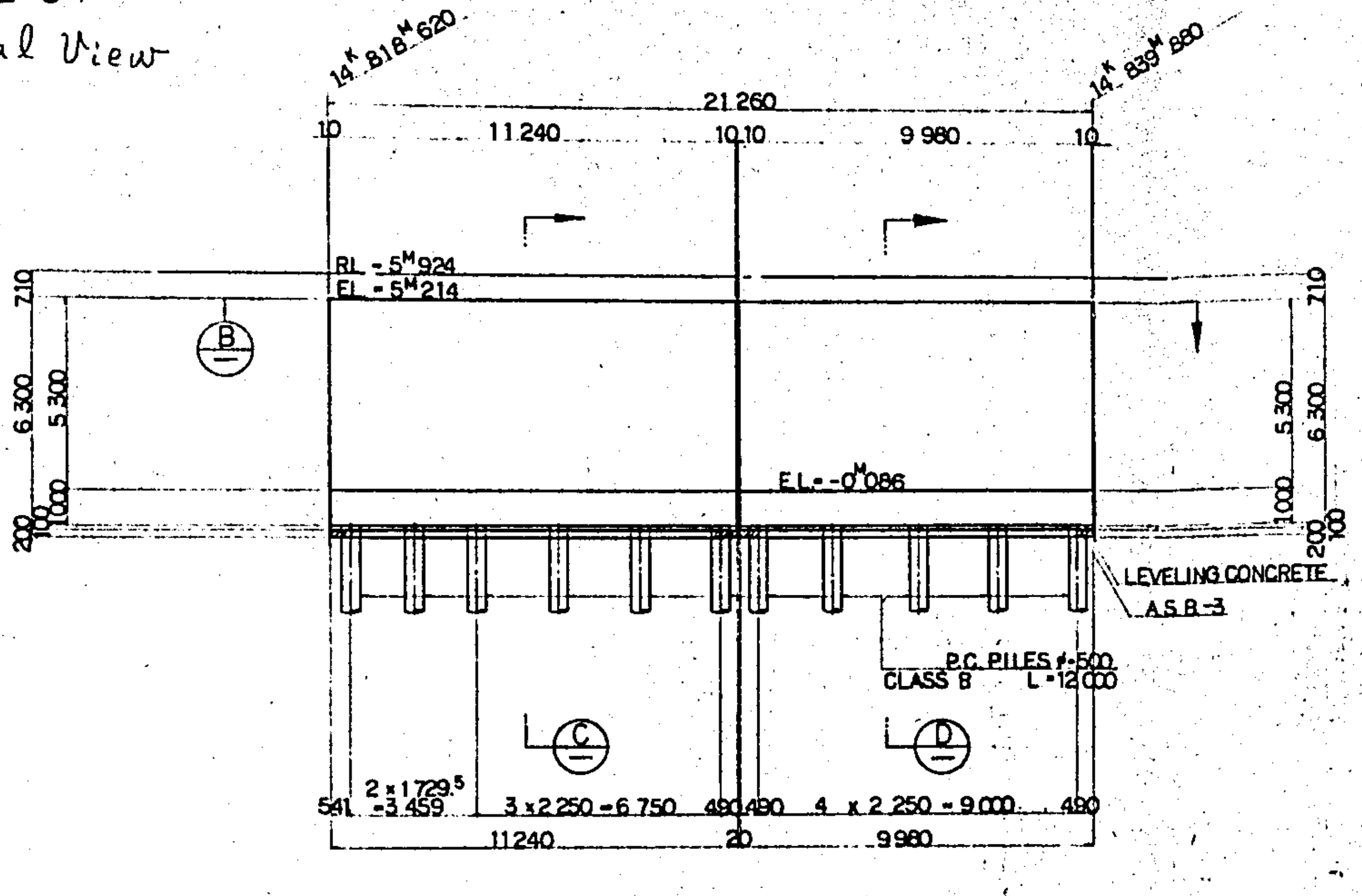
$$P_{max} = 74.34 < P_a = 88$$

d. Dead load + Earth pressure + Seismic load

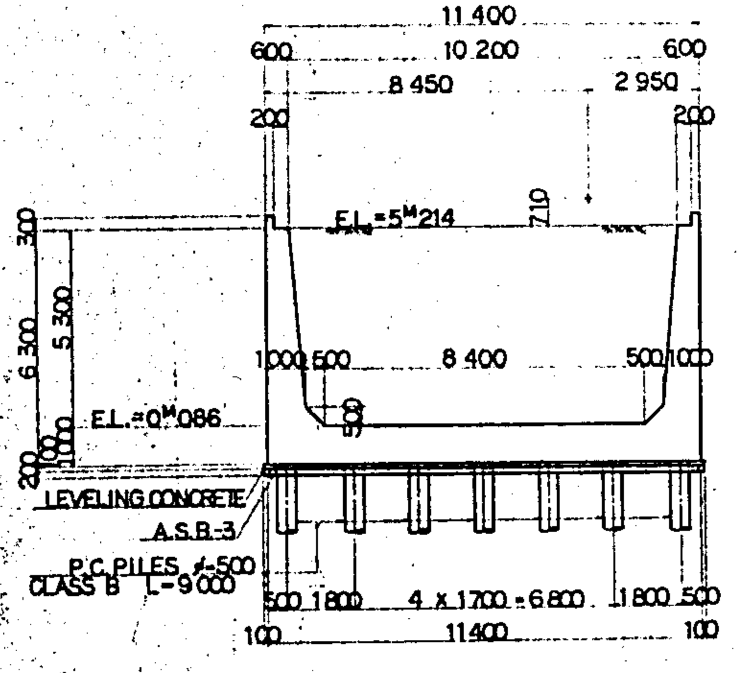
$$P_{max} = 50.98 < P_a = 117^{t}/\text{pile}$$

§6.U TYPE U1

1 General View



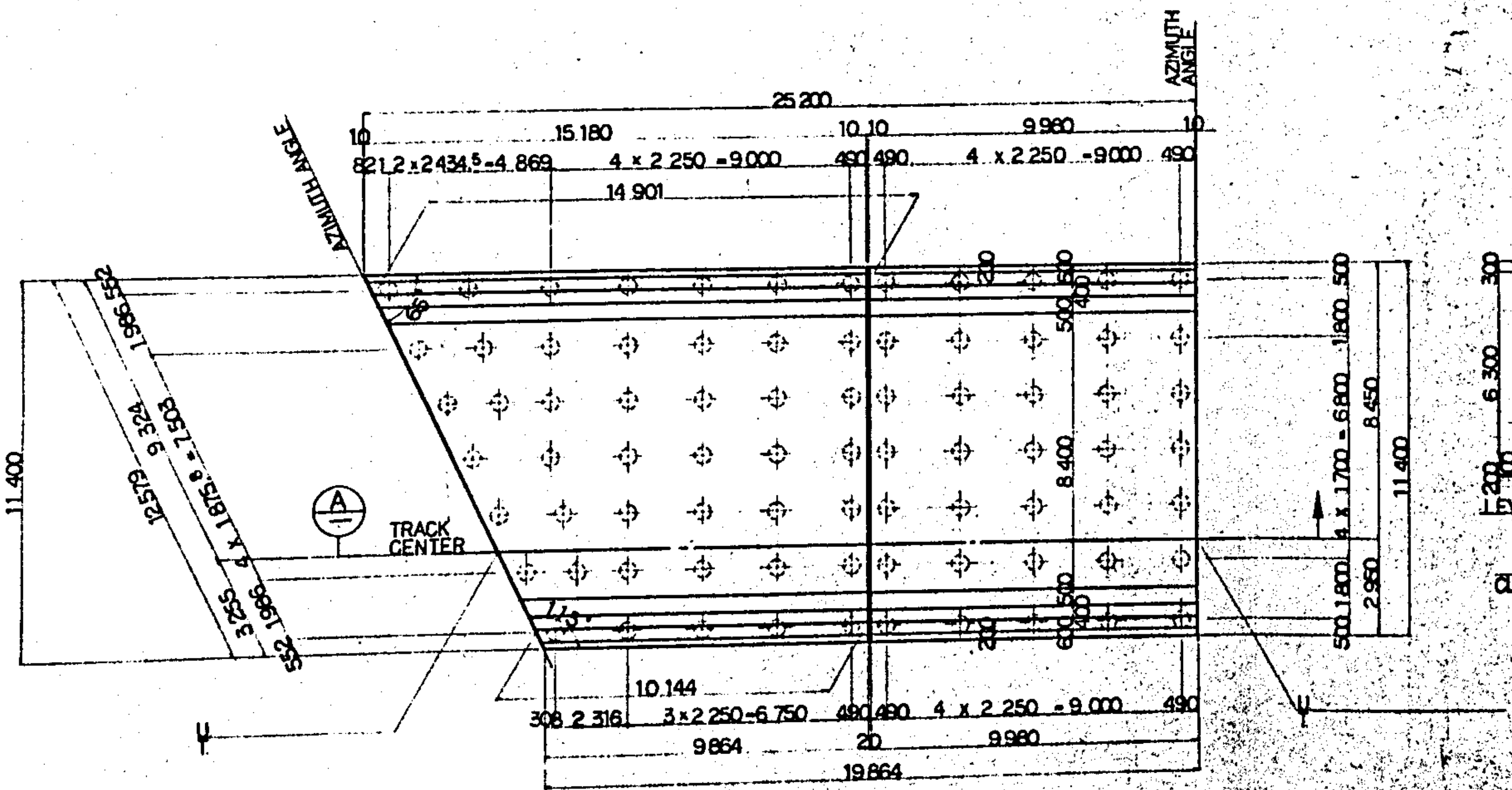
SECTION A



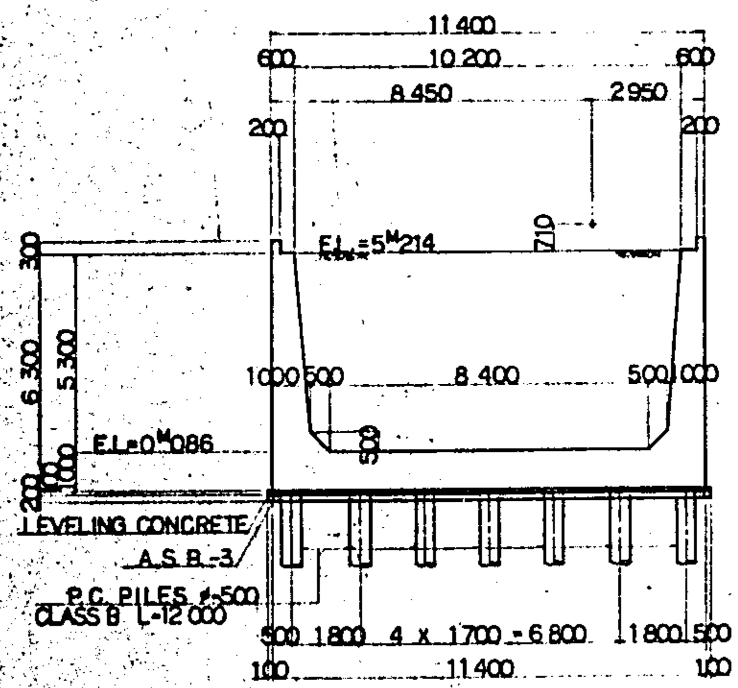
SECTION C

NOTES:

1. ALL DIMENSIONS ARE SHOW IN MILLMETERS UNLESS OTHERWISE INDICATED
2. DRAIN PIPE ∅75mm .P. .C EACH PER 2m2
3. REFERENCE DRAWING FOR BAR ARRANGEMENT : CS - 146
   CS - 147
   CS - 148



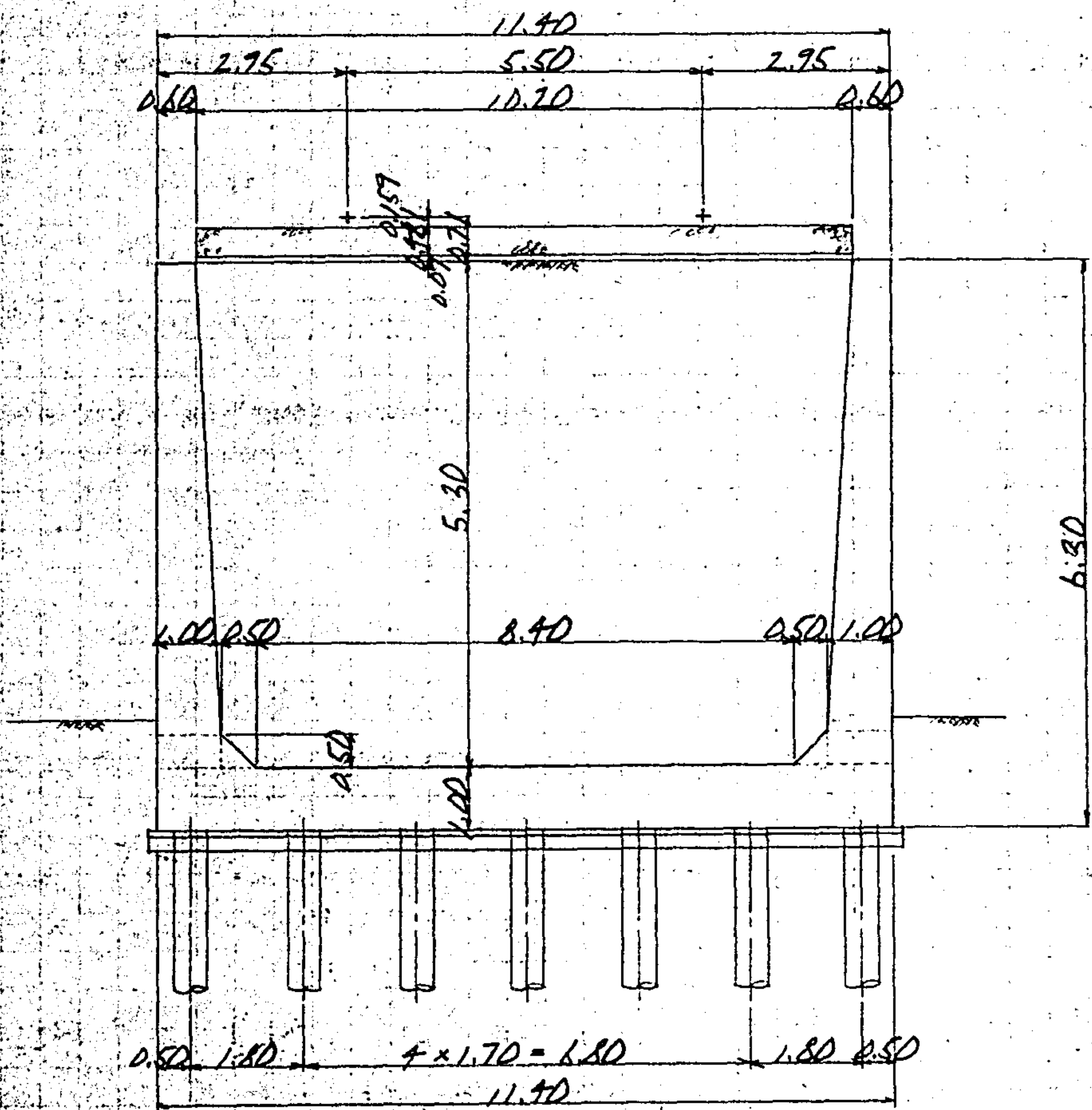
SECTION B



SECTION D

[2] Configuration and Dimensions.

(u) Retaining wall



$$h_1' = \frac{1.9 \times 0.481 + 2.35 \times 0.07}{1.8} = 0.600^{m}$$

$$h_2' = \frac{0.45 \times 2}{10.20} \times \frac{1}{1.8} = 0.050 \ ''$$

$$\therefore h' = 0.600 + 0.050 = 0.650^{m}$$

3

Calculation of Reaction Borceacting on Pile.

Dead load

Handrail	0.20×2	=	0.40^t
Curb	$2.5 \times 0.25 \times 0.30 \times 2$	=	0.38 "
Duct cover	$(0.06 + 2.5 \times 0.05 \times 0.30) \times 2$	=	0.20 "
ballast stopper	$2.5 \times 0.20 \times 0.30 \times 2$	=	0.30 "
Distributed Road of Track weight	0.45×2	=	0.90 "
ballast	$1.9 \times 10.20 \times 0.481$	=	9.32 "
Sloping concrete	$2.35 \times 10.20 \times 0.07$	=	1.68 "
Side wall	$2.5 \times (0.80 \times 4.80 + 1.00 \times 0.50) \times 2$	=	18.90 "
"	$2.5 \times 1/2 \times 0.40 \times 4.80 \times 2$	=	2.40 "
haunch	$2.5 \times 1/2 \times 0.50 \times 0.50 \times 2$	=	0.63 "
base SPab	$2.5 \times 11.40 \times 1.00$	=	28.50 "
Weight of earte covering	$1.8 \times 1/2 \times (10.20 + 9.40) \times 4.80$	=	84.67 "
"	$1.8 \times 1/2 \times (9.40 + 8.40) \times 0.50$	=	8.01 "

Per one meter = 154.29^t

Per distame between piles (2.25^m)

$$P_{Nd} = \frac{154.29 \times 2.25}{7} = 49.59^t < 58.0^t$$

F

: Train load

Train load can be calcupated by the poppowing equation, por convenience.

$$q = \frac{KS\ poading}{distance\ between\ axpes \times distributed\ width}$$

Distributed width $= 10.20^{m}$ (assumed)

$$q' = \frac{16 \times 2}{1.5 \times 10.20} = 2.10^{t/m^2}$$

per distance between piles(2.25^{m})

$$P_{Nl} = \frac{2.10 \times 10.20 \times 2.25}{7} = 6.89^{t}$$

$$P_{Nd} + P_{Nl} = 49.59 + 6.89 = 56.48^{t} < 88.0^{t}$$

Seismic load exerting
on dead weight ($K_H = 0.1$)

	N	H	y	H·y
Hand rail	0.40	0.04	6.600	0.26
Curb	0.38	0.04	6.450	0.26
Dust cover	0.20	0.02	6.300	0.13
Distributed Road of ballast stopper	0.30	0.03	6.450	0.19
Track weight	0.90	0.09	7.010	0.63
Ballast	9.32	0.93	6.610	6.15
Spoping concrete	1.68	0.17	6.335	1.08
Side wall	16.90	1.69	3.900	6.59
"	2.40	0.24	3.100	0.74
haunch	0.63	0.06	1.167	0.07
base spab	28.50	2.85	0.500	1.43
Covering earth	84.67	8.47	3.933	33.31
"	8.01	0.80	1.255	1.00
	154.29	15.43		51.84

$$y_e = \frac{51.84}{15.43} = 3.360 \text{ m}$$

Per distance between piles (2.25^m)

$$\Sigma N = 154.29 \times 2.25 = 347.15^t$$
$$\Sigma H = 15.43 \times 2.25 = 34.72\ ''$$
$$\Sigma M = 34.72 \times 3.36 = 116.66^{t.m}$$

Moment of inertia of group of piles with respect ot the geometrical center.

$$I = (1.70^2 + 3.40^2 + 5.20^2) \times 2 = 82.89^{m^2 \cdot \text{本}}$$

$$\begin{matrix} P_{max} \\ P_{min} \end{matrix} = \frac{N}{n} \pm \frac{M}{I} \cdot x$$

$$\left.\begin{matrix} P_1 \\ P_7 \end{matrix}\right\} = \frac{347.15}{7} \pm \frac{116.66}{82.89} \times 5.20 = \left\{\begin{matrix} 56.91^t \\ 42.27\ '' \end{matrix}\right. < 117.0^t$$

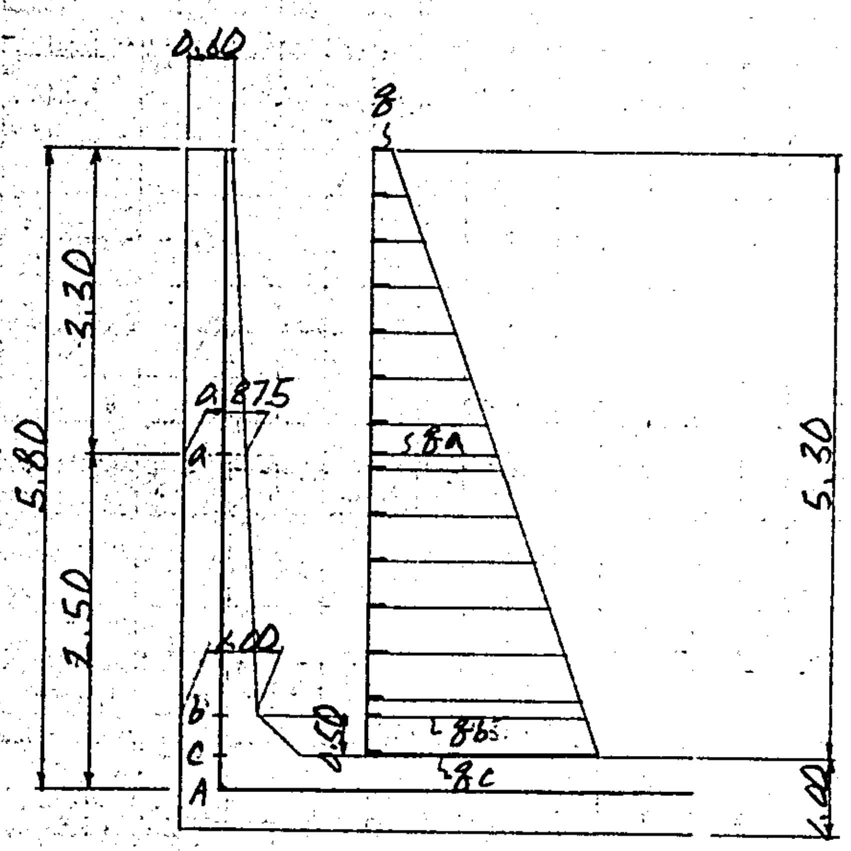
$$\left.\begin{matrix} P_2 \\ P_6 \end{matrix}\right\} = \frac{347.15}{7} \pm \frac{116.66}{82.89} \times 3.40 = \left\{\begin{matrix} 54.38^t \\ 44.81\ '' \end{matrix}\right.$$

$$\left.\begin{matrix} P_3 \\ P_5 \end{matrix}\right\} = \frac{347.15}{7} \pm \frac{116.66}{82.89} \times 1.70 = \left\{\begin{matrix} 51.99^t \\ 47.20\ '' \end{matrix}\right.$$

$$P_4 = \frac{347.15}{7} \pm \frac{116.66}{82.89} \times 0 = 49.59^t$$

4 Calculateion of wall

(1) Stress due to Dead load



$K_o = 0.50$

$q = 1.80 \times 0.650 \times 0.50 \times 2.25 = 1.31^t$

$q_a = 1.80 \times (0.65 + 3.30) \times 0.50 \times 2.25 = 8.00^t$

$q_b = 1.80 \times (0.65 + 4.80) \times 0.50 \times 2.25 = 11.04^t$

$q_c = 1.80 \times (0.65 + 5.30) \times 0.50 \times 2.25 = 12.05^t$

Stress at point a

$S_a = \frac{1}{2} \times (1.31 + 8.00 \times 3.30 = 15.36^{t}$

$M_a = \frac{1}{2} \times 1.31 \times 3.30^2 + \frac{1}{6} \times (8.00 - 1.31) \times 3.30^2$

$= 19.28^{t.m}$

Stress at point b

$S_b = \frac{1}{2} \times (1.31 + 11.04) \times 4.80 = 29.64^{t}$

$M_b = \frac{1}{2} \times 1.31 \times 4.80^2 + \frac{1}{6} \times (11.04 - 1.31) \times 4.80^2$

$= 52.45^{t.m}$

Stress at point C

$S_c = \frac{1}{2} \times (1.31 + 12.05) \times 5.30 = 35.40^{t}$

$M_c = \frac{1}{2} \times 1.31 \times 5.30^2 + \frac{1}{6} \times (12.05 - 1.31) \times 5.30^2$

$= 68.68^{t.m}$

Stress at point A

$S_A = 35.40^{t}$

$M_A = 1.31 \times 5.30 \times (\frac{1}{2} \times 5.30 + 0.50)$

$+ \frac{1}{2} \times (12.05 - 1.31) \times 5.30 \times (\frac{1}{3} \times 5.30 + 0.50)$

$= 86.38^{t.m}$

(2) Stress caused by Train load

$w_l = 2.10 \times 2.25 = 4.73 \ ^{t/m^2}$

$q_l = 4.73 \times 0.50 = 2.37 \ ^{t/m}$

Stress at point a

$S_a = 2.37 \times 3.30 = 7.82 \ ^t$

$M_a = 1/2 \times 2.37 \times 3.30^2 = 12.90 \ ^{t.m}$

Stress at point b

$S_b = 2.37 \times 4.80 = 11.38 \ ^t$

$M_b = 1/2 \times 2.37 \times 4.80^2 = 27.30 \ ^{t.m}$

Stress at point c

$S_c = 2.37 \times 5.30 = 12.56 \ ^t$

$M_c = 1/2 \times 2.37 \times 5.30^2 = 33.29 \ ^{t.m}$

Stress at point A

$S_A = 2.37 \times 5.30 = 12.56 \ ^t$

$M_A = 2.37 \times 5.30 \times (1/2 \times 5.30 + 0.50) = 39.57 \ ^{t.m}$

3) Composition of Stress

		Dead load	Train load	Total	
a, point	$S^{(t)}$	15.36	7.82	(10.30) 23.18	
	$M^{(tm)}$	(8.57) 19.28	12.90	(14.30) 32.18	
c, point	$S^{(t)}$	35.40	12.56	(21.32) 47.96	
	$M^{(tm)}$	(30.52) 68.68	33.29	(45.32) 101.97	

within parenthesis of figure in par m. (1/2.25)

Examination of Safety (wall)

NO.	1 (crock)	1	2 (crock)	2
CALCULATING PT. No.	(C)	(C)	(a)	(a)
M [T*M]	30.52	45.32	8.57	14.30
S [T]		21.32		10.30
B [CM]	100.00	100.00	100.00	100.00
H [CM]	100.00	100.00	87.50	87.50
D [CM]	92.00	92.00	79.50	79.50
D' [CM]	8.00	8.00	8.00	8.00
AS (DIA-HON)	D22- 8.0	D22- 8.0	D22- 4.0	D22- 4.0
[CM**2]	30.97	30.97	15.48	15.48
AS' (DIA-HON)				
[CM**2]				
P	0.003366	0.003366	0.001948	0.001948
P'				
K	0.2713	0.2713	0.2143	0.2143
LC	0.1234	0.1234	0.0995	0.0995
SIG-C [KG/CM**2]	29.23	43.40	13.63	22.74
SIG-S [KG/CM**2]	1177.73	1748.84	749.74	1251.03
TAU [KG/CM**2]		2.32		1.30
SIG-CA[KG/CM**2]	80.00	80.00	80.00	80.00
SIG-SA[KG/CM**2]	1400.00	1800.00	1400.00	1800.00
TAU-A [KG/CM**2]		3.90		3.90

```
            +-------B-------+
            !               !
            !               !
+---+-------****************
!   D'      *              *
!   +-------*  ..........  *
!           *     AS'      *
!           *              *
D           *              *
!           *              *
!           *              *
!           *      AS      *
+-----------*  ..........  *
            *              *
            ****************
```

5 Calculation of base slab

(1) Dead load

uniform load

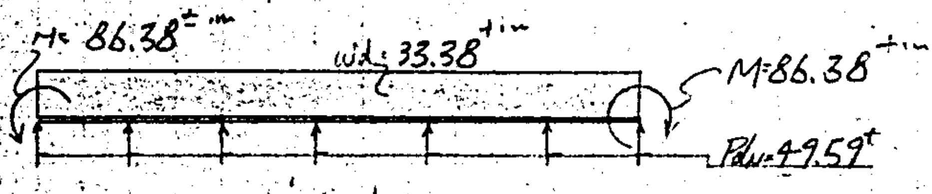
$Wd = 154.29 \times 2.25 \times 1/(11.40 - 1.00) = 33.38^{t/m}$

Reaction force acting on pile

$P_{Nd} = 49.59^{\ t/pile}$

moment of panel pint

$M_{dA} = -86.38^{\ t\cdot m}$



(2) Train load

uniform load

$W_L = 2.10 \times 2.25 \times 10.20/10.40 = 4.63^{t/m}$

Reaction force acting on pile

$P_{NL} = 6.89^{\ t/pile}$

moment of panel pint

$M_L = 39.57^{\ t\cdot m}$

```
=== INPUT DATA ===
     5   10        20        30        40        50        60        70        80    CARD NUMBER
.................................................................................
TITLE     U-TYPE                                                                        1
JOINT         1      0.0       0.0                                                      2
              2      5.20      0.0                                                      3
              3     10.40      0.0                                                      4
MEMBER        1  1 1    1      2                                                        5
              2  1 1    2      3                                                        6
SUPPORT       1  001                                                                    7
              3  001                                                                    8
SECTION       1      1.000     1.000                                                    9
PROPERTY      1         2.55E6                                                         10
POINT    10.90       1.80      2.65      3.50      4.35                                11
         20.85       1.70      2.55      3.40      4.30                                12
LOAD          1      D                                                                 13
J             1                49.59     86.38                                         14
              2                49.59                                                   15
              3                49.59    -86.38                                         16
LL            1  2 2-33.38    -33.38     0.0       0.0                                 17
SC            1    249.59      1.80     49.59      1.70                                18
              2    249.59      1.70     49.59      1.70                                19
END                                                                                    20
LOAD          2      T                                                                 21
J             1                6.89     39.57                                          22
              2                6.89                                                    23
              3                6.89    -39.57                                          24
LL            1  2 2-4.63     -4.63      0.0       0.0                                 25
SC            1     26.89      1.80      6.89      1.70                                26
              2     26.89      1.70      6.89      1.70                                27
END                                                                                    28
MT2      31.0000          1      2                                                     29
FINISH                                                                                 30
```

```
--------------
CONTROL DATA
--------------
     METHOD     STRUCTURE  J.RENUMBER  M.RENUMBER  S.F. DIS.  UNI.SPRING  STAN.STIF.    BARA     SKEW MEM.
    ---------   ---------  ----------  ----------  ---------  ----------  ----------  ---------  ---------
     *DIS*      *RAHMEN*     *OFF*       *OFF*      *OFF*       *OFF*       *OFF*      *OFF*      *OFF*

------------
LOAD TITLE
------------
     LOAD  1   D                                   LOAD  2   T

     MIX   3   CASE  3 (1+2)
```

JOINT DATA

JOINT NUMBER 3

ZAHYO(M) JOINT	X	Y
1	0.0000	0.0000
2	5.2000	0.0000
3	10.4000	0.0000

MEMBER DATA

MEMBER NUMBER 2

MEMBER	IEND	JEND	CONNECT. I END	J END	LENGTH	A	I	AES	KD(BANE)	PRO.NUM
1	1	2	FIX	FIX	5.2000	1.00000	1.0000000			1
2	2	3	FIX	FIX	5.2000	1.00000	1.0000000			1

PROPERTY DATA

PROPERTY NUMBER 1

PROPERTY	E	G	EPS
1	2.550E+06	0.	1.000E-05

SUPPORT DATA

SUPPORT NUMBER 2

SUPPORT	SUPPORT-CONDITION X	Y	THET Z	X(BANE)	Y(BANE)	THET Z(BANE)
1	FIX	FIX	FREE	0.0	0.0	0.0
3	FIX	FIX	FREE	0.0	0.0	0.0

STRUCTURAL FIGURE

```
+--------------------+--------------------+
# 1                  2                    # 3
```

MOVE DATA

MEMBER	GOIKI ITAN	GOIKI JTAN	POINT 1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	......	......	.900	1.800	2.650	3.500	4.350									
2	......	......	.850	1.700	2.550	3.400	4.300									

LOAD DATA

	M/J	NAME	D	W1	W2	L1	L2
LOAD -	1	D					
MEMBER	1	LINEAR	Y	-33.380	-33.380	0.000	0.000
		CONCENT	Y	49.590		1.800	
		CONCENT	Y	49.590		3.500	
	2	LINEAR	Y	-33.380	-33.380	0.000	0.000
		CONCENT	Y	49.590		1.700	
		CONCENT	Y	49.590		3.400	
JOINT	1	JOINTLOAD	Y	49.590			
		JOINTLOAD	Z	86.380			
	2	JOINTLOAD	Y	49.590			
	3	JOINTLOAD	Y	49.590			
		JOINTLOAD	Z	-86.380			
LOAD -	2	T					
MEMBER	1	LINEAR	Y	-4.630	-4.630	0.000	0.000
		CONCENT	Y	6.890		1.800	
		CONCENT	Y	6.890		3.500	
	2	LINEAR	Y	-4.630	-4.630	0.000	0.000
		CONCENT	Y	6.890		1.700	
		CONCENT	Y	6.890		3.400	
JOINT	1	JOINTLOAD	Y	6.890			
		JOINTLOAD	Z	39.570			
	2	JOINTLOAD	Y	6.890			
	3	JOINTLOAD	Y	6.890			
		JOINTLOAD	Z	-39.570			

TITLE.. U-TYPE

MIX DATA

MIX NUMBER : 1

LOAD	SS	N	S1	K1	S2	K2	S3	K3	S4	K4	S5	K5	S6	K6	S7	K7	S8	K8
3	1.0000	2	1.0000	1	1.0000	2												

R E A C T I O N

LOAD - 1 D

SUPPORT	X (TON)	Y (TON)	Z (TON.M)
1	0.000	.011	0.000
3	0.000	.011	0.000

MIX - 3 CASE 3 (1+2)

SUPPORT	X (TON)	Y (TON)	Z (TON.M)
1	0.000	-.028	0.000
3	0.000	-.028	0.000

LOAD - 2 T

SUPPORT	X (TON)	Y (TON)	Z (TON.M)
1	0.000	-.039	0.000
3	0.000	-.039	0.000

D E F L E C T I O N

LOAD - 1 D

JOINT	X (MM)	Y (MM)	Z (MMRAD)
1	0.000	0.000	.079
2	0.000	.155	.000
3	0.000	0.000	-.079

MIX - 3 CASE 3 (1+2)

JOINT	X (MM)	Y (MM)	Z (MMRAD)
1	0.000	0.000	.147
2	0.000	.324	.000
3	0.000	0.000	-.147

LOAD - 2 T

JOINT	X (MM)	Y (MM)	Z (MMRAD)
1	0.000	0.000	.067
2	0.000	.168	.000
3	0.000	0.000	-.067

TITLE.. U-TYPE

LOAD 1 D

	L	M	(M)	Q	N
= = MEMBER 1 (1 - 2) G = =					
I END	0.000	-86.380		49.601	0.000
1	.900	-55.258		-19.559	0.000
2	1.800	-51.174		39.107	0.000
3	2.650	-29.991		10.734	0.000
4	3.500	-32.926		31.951	0.000
5	4.350	-17.826		3.578	0.000
J END	5.200	-26.843		-24.795	0.000
MAX	4.350	-17.826		3.578	0.000
= = MEMBER 2 (2 - 3) G = =					
I END	0.000	-26.843		24.795	0.000
1	.850	-17.826		-3.578	0.000
2	1.700	-32.926		17.639	0.000
3	2.550	-29.991		-10.734	0.000
4	3.400	-51.174		10.483	0.000
5	4.300	-55.258		-19.559	0.000
J END	5.200	-86.380		-49.601	0.000
MAX	.850	-17.826		-3.578	0.000

LOAD 2 T

	L	M	(M)	Q	N
= = MEMBER 1 (1 - 2) G = =					
ITAN	0.000	-39.570		6.851	0.000
1	.900	-35.279		2.684	0.000
2	1.800	-34.739		5.407	0.000
3	2.650	-31.815		1.472	0.000
4	3.500	-32.237		4.426	0.000
5	4.350	-30.148		.491	0.000
JTAN	5.200	-31.403		-3.445	0.000
MAX	4.350	-30.148		.491	0.000
= = MEMBER 2 (2 - 3) G = =					
ITAN	0.000	-31.403		3.445	0.000
1	.850	-30.148		-.491	0.000
2	1.700	-32.237		2.464	0.000
3	2.550	-31.815		-1.472	0.000
4	3.400	-34.739		1.483	0.000
5	4.300	-35.279		-2.684	0.000
JTAN	5.200	-39.570		-6.851	0.000
MAX	.850	-30.148		-.491	0.000

TITLE.. U-TYPE

MIX 3 CASE 3 (1+2)

= = MEMBER 1 (1 - 2) G = =

	L	M	(M)	Q	N
I END	0.000	-125.950		56.452	0.000
1	.900	-90.537		22.243	0.000
2	1.800	-85.913		44.514	0.000
3	2.650	-61.807		12.206	0.000
4	3.500	-65.163		36.377	0.000
5	4.350	-47.974		4.069	0.000
J END	5.200	-58.247		-28.240	0.000
MAX	4.350	-47.974		4.069	0.000

= = MEMBER 2 (2 - 3) G = =

	L	M	(M)	Q	N
I END	0.000	-58.247		28.240	0.000
1	.850	-47.974		-4.069	0.000
2	1.700	-65.163		20.103	0.000
3	2.550	-61.807		-12.206	0.000
4	3.400	-85.913		11.966	0.000
5	4.300	-90.537		-22.243	0.000
J END	5.200	-125.950		-56.452	0.000
MAX	.850	-47.974		-4.069	0.000

Examination of safety (base slab)

NO.	1	1
CALULATING CASE	(1)	(3)
	(86.38 x 1/2.25)	(125.95 x 1/2.25)
M [T*M]	38.39	55.98
S [T]		
B [CM]	100.00	100.00
H [CM]	100.00	100.00
D [CM]	92.00	92.00
D' [CM]	8.00	8.00
AS (DIA-HON)	D25- 8.0	D25- 8.0
[CM**2]	40.54	40.54
AS' (DIA-HON)		
[CM**2]		
P	0.004406	0.004406
P'		
K	0.3034	0.3034
LC	0.1364	0.1364
SIG-C [KG/CM**2]	33.26	48.50
SIG-S [KG/CM**2]	1145.25	1669.99
TAU [KG/CM**2]		
SIG-CA[KG/CM**2]	80.00	80.00
SIG-SA[KG/CM**2]	1400.00	1800.00
TAU-A [KG/CM**2]		

```
              +-------B-------+
              !               !
              !               !
+--+----------***************
!  D'  *                   *
!  +---*  ...........      *
!      *       AS'         *
!      *                   *
D      *                   *
!      *                   *
!      *                   *
!      *       AS          *
+------*  ...........      *
       *                   *
       *********************
```