2-2, LONGITUDINAL DIRECTION

	AXIAL FORCE	MOMENT	HORIZONTAL FORCE
No 1	2152.00	19.10	―
No 2	2680.62	19.10 + 22.74 + 116.88 × 2 = 275.60	8.10 × 2 = 16.20
No 3	2318.94	275.60	16.20
No 4	2680.62	275.60 + 395.09 = 670.69	16.20 + 27.38 = 43.58
No 5	2318.94	670.69	43.58
No 6	2152.00	197.87 + 19.10 + (799.75 + 231.30) × 2 = 2279.07	71.33 + (68.53 + 19.82) × 2 = 248.03

3. REACTION FORCE ACTING ON PILES

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

WHERE: P REACTION FORCE

n NUMBER OF PILES

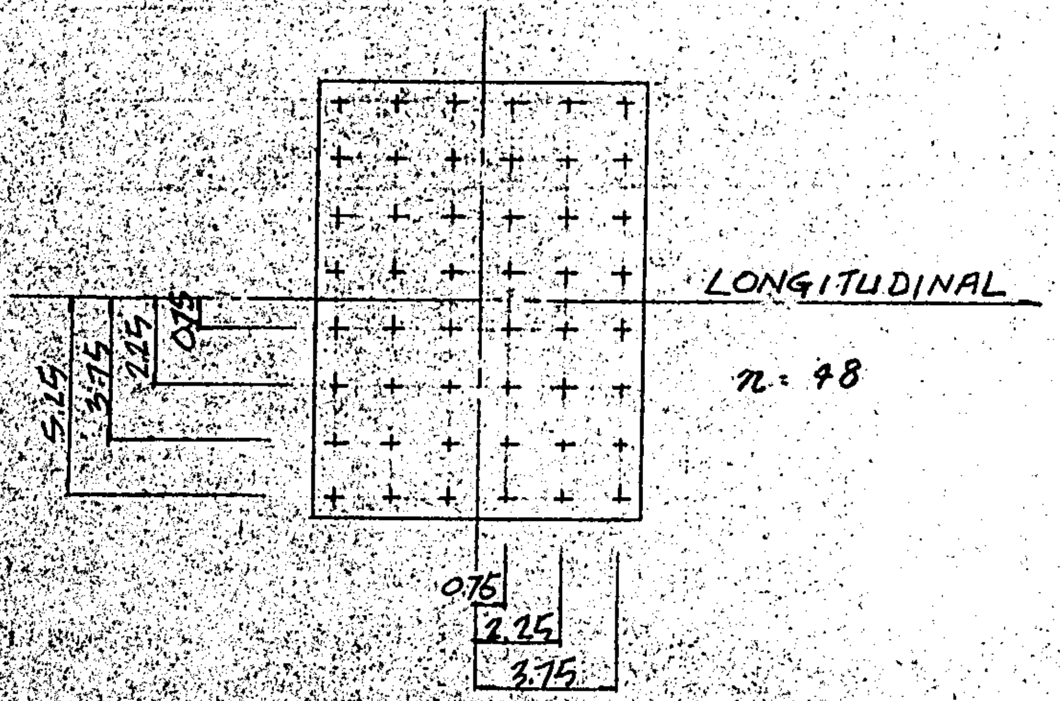
N AXIAL FORCE ACTING AT THE CENTER OF THE BOTTOM OF FOOTING

M MOMENT ACTING AT THE CENTER OF THE BOTTOM OF FOOTING

I MOMENT OF INERTIA OF THE GROUP OF PILES WITH RESPECT TO THE GEOMETRICAL CENTER

x DISTANCE FROM THE GEOMETRICAL CENTER OF THE GROUP OF PILES TO THE CENTER OF THE PILE TO BE CALCULATED

3-1. ARRANGEMENT OF PILES



3-2 REACTION FORCE ACTING ON PILES
- DUE TO TRANSVERSAL LOAD

	N (t)	M (t·m)	$\frac{N}{n}$ (t)	$\frac{M}{I}$ (t/m)	Pmax (t)	Pmin (t)
No 1	1572.26	1694.38	32.76	2.99	48.46	17.06
No 2	1836.57	2467.88	38.26	4.35	61.10	15.42
No 3	1474.89	2467.88	30.73	4.35	53.57	7.89
No 4	1836.57	2121.36	38.26	4.80	63.46	13.06
No 5	1474.89	2121.36	30.73	4.80	55.93	5.53
No 6	1572.26	2818.85	32.76	4.97	58.85	6.67
No 7	2152.00	—	44.83	—	44.83	44.83
No 8	2680.62	642.14	55.85	1.13	61.78	49.92
No 9	2318.94	642.14	48.31	1.13	54.24	42.38
No 10	2680.62	786.01	55.85	1.39	63.15	48.55
No. 11	2318.94	786.01	48.31	1.39	55.61	41.01
No. 12	2152.00	2051.07	44.83	3.62	63.84	25.83

$I = (5.25^2 + 3.75^2 + 2.25^2 + 0.75^2) \times 6 \times 2 = 567.00\ m^2$

$Z = 5.25\ m$

3-3. REACTION FORCE ACTING ON PILES DUE TO LONGITUDINAL LOAD.

	N (t)	M (t·m)	$\frac{N}{n}$ (t)	$\frac{M}{I}$ ($\frac{t}{m}$)	x (m)	P_{max} (t)	P_{min} (t)	
No1	2152.00	19.10	44.83	0.06	x_1	44.88	44.78	0.05
					x_2	44.97	44.69	0.14
					x_3	45.06	44.60	0.23
No2	2680.62	275.60	55.85	0.87	x_1	56.50	55.20	0.65
					x_2	57.81	53.89	1.96
					x_3	59.11	52.59	3.26
No3	2318.94	275.60	48.31	0.87	x_1	48.96	47.66	
					x_2	50.27	46.35	
					x_3	51.57	45.05	
No4	2680.62	670.69	55.85	2.13	x_1	57.42	54.22	1.60
					x_2	60.61	51.03	4.79
					x_3	63.81	47.83	7.99
No5	2318.94	670.69	48.31	2.13	x_1	49.91	46.71	
					x_2	53.10	43.52	
					x_3	56.30	40.32	
No6	2152.00	2219.07	44.83	7.24	x_1	50.26	39.40	5.43
					x_2	61.12	28.54	16.29
					x_3	71.98	17.68	27.15

$$I = (3.75^2 + 2.25^2 + 0.75^2) \times 8 \times 2 = 315.00\ m^2$$

$x_1 = 0.75^m$

$x_2 = 2.25^m$

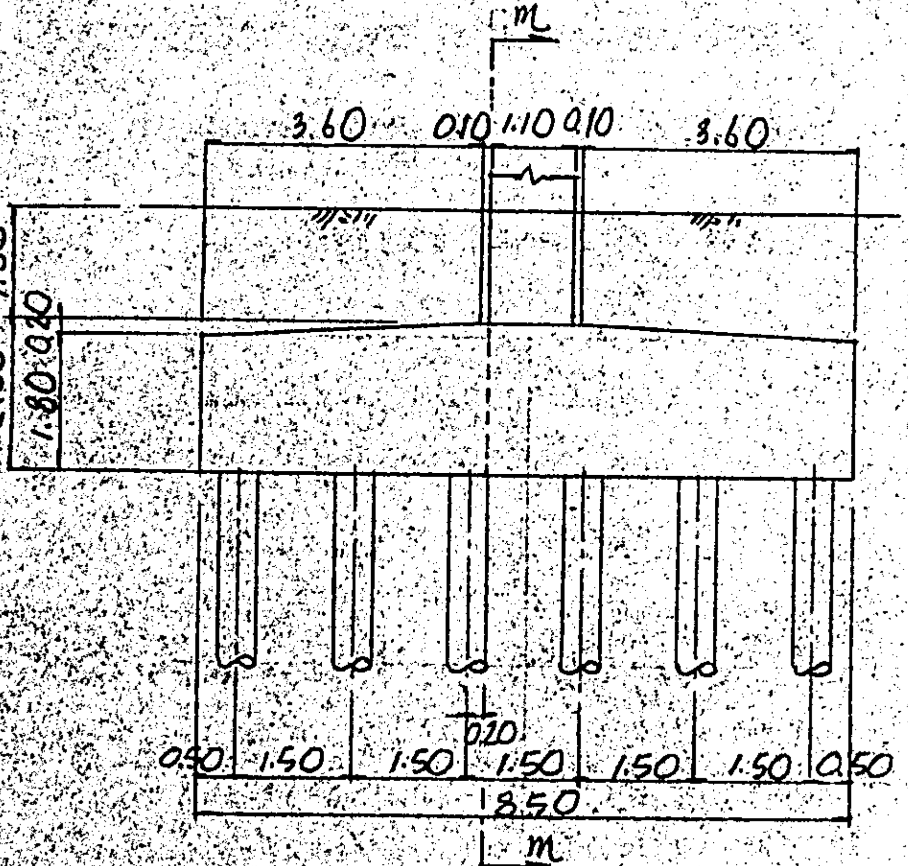
$x_3 = 3.75^m$

5 DESIGN OF FOOTING

CALCULATION IS MADE ONLY THE CASE OF LONGITUDINAL LOADING, BECAUSE NO STRESS OCCURS UNDER THE TRANSVERSAL LOADING DUE TO REACTION FORCES ON PILES WITHIN THE WALL SHAFT.

1. EXAMINATION OF SAFETY AGAINST BENDING MOMENT

1-1. BENDING MOMENT DUE TO OWN WEIGHT OF FOOTING AND WEIGHT OF EARTH COVER



WHERE: m SECTION WHERE MOMENT IS CALCULATED

Mm BENDING MOMENT AT SECTION m

BENDING MOMENT AT SECTION m

i) THE CASE NO BUOYANCY IS CONSIDERED

$M_m = (2.5^{t/m^3} \times 2.00 + 1.8^{t/m^3} \times 1.70) \times \frac{1}{2} \times 3.70^2$

$- \frac{1}{2} \times 3.60 \times 0.20 \times 2.50 \times (2.5 - 1.8) = 54.54^{tm}$

ii) THE CASE BUOYANCY IS CONSIDERED

$M_m = 54.54 - 1.00^{t/m^3} \times \frac{1}{2} \times 3.70^2 \times 3.70 = 29.21^{tm}$

1-2, STRESS DUE TO REACTION FORCE ON PILES

BENDING MOMENT AT SECTION m

No.1 $M_m = (45.06 \times 3.30 + 44.97 \times 1.80 + 44.88 \times 0.30) \times \frac{8}{11.50} = 169.12^{tm/m}$

No.2 $M_m = (59.11 \times$ 〃 $+ 57.81 \times$ 〃 $+ 56.50 \times$ 〃 $) \times$ 〃 $= 219.88$ 〃

No.3 $M_m = (51.57 \times$ 〃 $+ 50.27 \times$ 〃 $+ 48.96 \times$ 〃 $) \times$ 〃 $= 191.55$ 〃

No.4 $M_m = (63.81 \times$ 〃 $+ 60.61 \times$ 〃 $+ 57.42 \times$ 〃 $) \times$ 〃 $= 234.36$ 〃

No.5 $M_m = (56.30 \times$ 〃 $+ 53.10 \times$ 〃 $+ 49.91 \times$ 〃 $) \times$ 〃 $= 206.15$ 〃

No.6 $M_m = (71.98 \times$ 〃 $+ 61.12 \times$ 〃 $+ 50.26 \times$ 〃 $) \times$ 〃 $= 252.26$

1-3. SUMMARY OF BENDING MOMENT ON FOOTING

No1 : $M_x = (169.12 - 54.54) \times 1.8/1.4 = 147.32$ tm/m

No2 : $M_x = (219.88 - 54.54) \times 1.00 = 165.34$ "

No3 : $M_x = (191.55 - 29.21) \times 1.00 = 162.34$ "

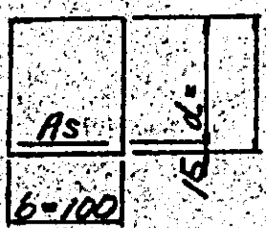
No4 : $M_x = (234.36 - 54.54) \times 1/1.15 = 156.37$ "

No5 : $M_x = (206.15 - 29.21) \times 1/1.15 = 153.86$ "

No6 : $M_x = (252.26 - 54.54) \times 1/1.50 = 131.81$ "

1-4. EXAMINATION OF SAFETY

$M_x = 165.34$ tm/m



$As = 8 - \phi 32 = 63.54$ cm²

$$p = \frac{As}{b \cdot d} = \frac{63.54}{100 \times 185} = 0.00343$$

$$\frac{1}{Lc} = 8.05 \qquad \frac{1}{Ls} = 320$$

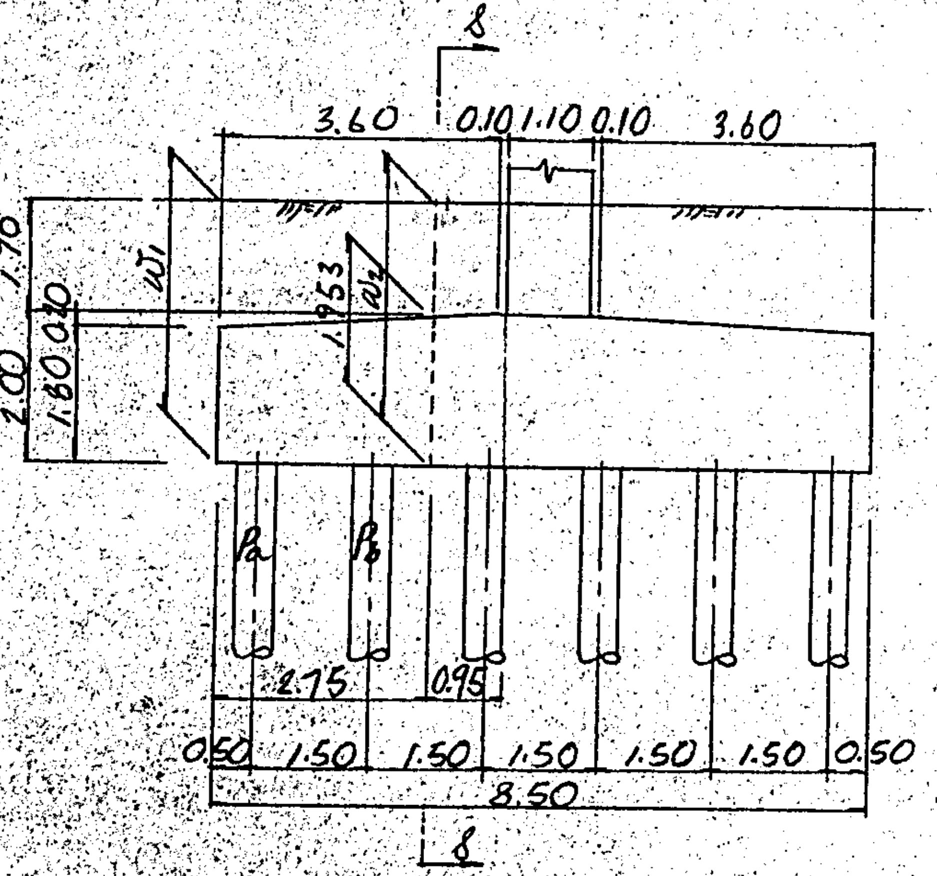
$$\sigma_c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc} = \frac{165.34 \times 10^5}{100 \times 185^2} \times 8.05 = 38.9 \text{ kg/cm}^2 < 80 \text{ kg/cm}^2$$

$$\sigma_s = \frac{M}{b \cdot d^2} \times \frac{1}{Ls} = \text{———"———} \times 320 = 1546 \text{ "} < 1800 \text{ "}$$

2. EXAMINATION OF SAFETY AGAINST SHEARING FORCE

EXAMINATION ON THE COMBINATION No.6

2-1. SHEARING FORCE DUE TO OWN WEIGHT OF FOOTING AND WEIGHT OF EARTH COVER.



WHERE: s SECTION WHERE SHEARING FORCE IS CALCULATED

S_s SHEARING FORCE AT SECTION s

SHEARING FORCE AT SECTION s

$$w_1 = 2.5^{t/m^3} \times 1.80 + 1.8^{t/m^3} \times 1.90 = 7.92^{t/m}$$

$$w_2 = (2.5'' \times 1.953 + 1.8'' \times 1.747) \times \frac{0.95}{2 \times (1.953 - 0.15)} = 2.11''$$

$$S_s = 1/2 \times (7.92 + 2.11) \times 2.75 = 13.79^t$$

2-2. STRESS DUE TO REACTION FORCE ON PILES

SHEARING FORCE AT SECTION s

$$P_a = 71.98 \times \frac{3.20}{2 \times (1.953 - 0.15)} = 63.88^t$$

$$P_b = 61.12 \times \frac{1.70}{2 \times (1.953 - 0.15)} = 28.81''$$

$$S_s = (63.88 + 28.81) \times \frac{8}{11.50} = 64.48^t$$

2-3. SUMMARY OF SHEARING FORCES ON FOOTING

$$\Sigma S_s = 64.48 - 13.79 = 50.69^t$$

2-4. EXAMINATION OF SAFETY,

$$\tau = \frac{50.69 \times 10^3}{100 \times (195.3 - 15)} = 2.81^{kg/cm^2} < 3.7^{kg/cm^2}$$

6 DESIGN OF SUBSTRUCTURE.

EXAMINATION IS MADE ONLY THE CASE OF LONGITUDINAL LOADING.

1. COMBINATION OF LOADS

NO.6 DEAD LOAD + EARTHQUAKE LOAD

2. BENDING MOMENT AND AXIAL FORCE ACTING AT THE BOTTOM OF WALL

THE FOLLOWING FIGURES ARE REFERRED TO THE CALCULATION OF THE REACTION FORCES TRANSMITTED FROM THE SUPERSTRUCTURE.

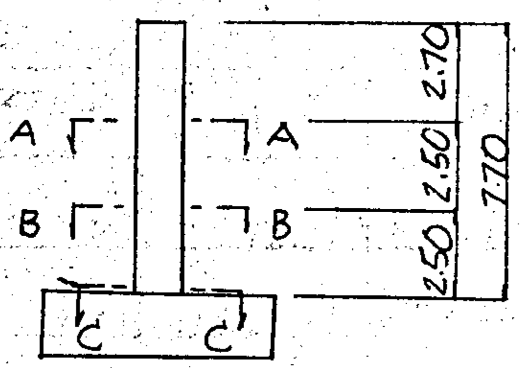
$$M = (42.84 \times 7.35 + 13.24 \times 6.862 + 189.11 \times 3.35) \times 0.10$$
$$+ (68.53 + 19.82) \times 2 \times 9.67 + 19.10 = 1831.71^{tm}$$

$$N = 42.84 + 13.24 + 189.11$$
$$+ (299.54 + 280.00) \times 2 = 1404.27^{t}$$

3 EXAMINATION OF SAFETY

M (tm)	1831.71
N (t)	1404.27
S (t)	
b (cm)	940
h (cm)	110
d' (cm)	
d (cm)	8.5
As (cm²)	77 - D32 = 611.53
p	0.00591
As' (cm²)	= As
p'	= p
u = M/N (cm)	130.4
e = M/N + u (cm)	
e = M/N − u (cm)	
e/h	1.185
d/e	
d'/h	0.077
d'/d	
Ne/bd² (kg/cm²)	13.580
k	0.376
c	0.130
j	
1/Lc	
1/Ls	
β = σs/σc	
σc (kg/cm²)	104.1
σs (kg/cm²)	2762
τ (kg/cm²)	
σsa (kg/cm²)	180
σca (kg/cm²)	2700
τa (kg/cm²)	

4. DEDUCTION OF REINFORCING BARS



4-1 BENDING MOMENT AND AXIAL FORCE ACTING AT THE SECTION A AND B

$$M_A = 19.10 + (42.84 \times 4.35 + 13.24 \times 3.862 + 2.5 \times 11.290 \times 1.70^2 \times 1/2) \times 0.1 + (68.53 + 19.82) \times 4.67 \times 2 = 872.12^{tm}$$

$$N_A = 42.84 + 13.24 + 2.5 \times 11.290 \times 1.70 + (299.54 + 280.00) \times 2 = 1263.14^{t}$$

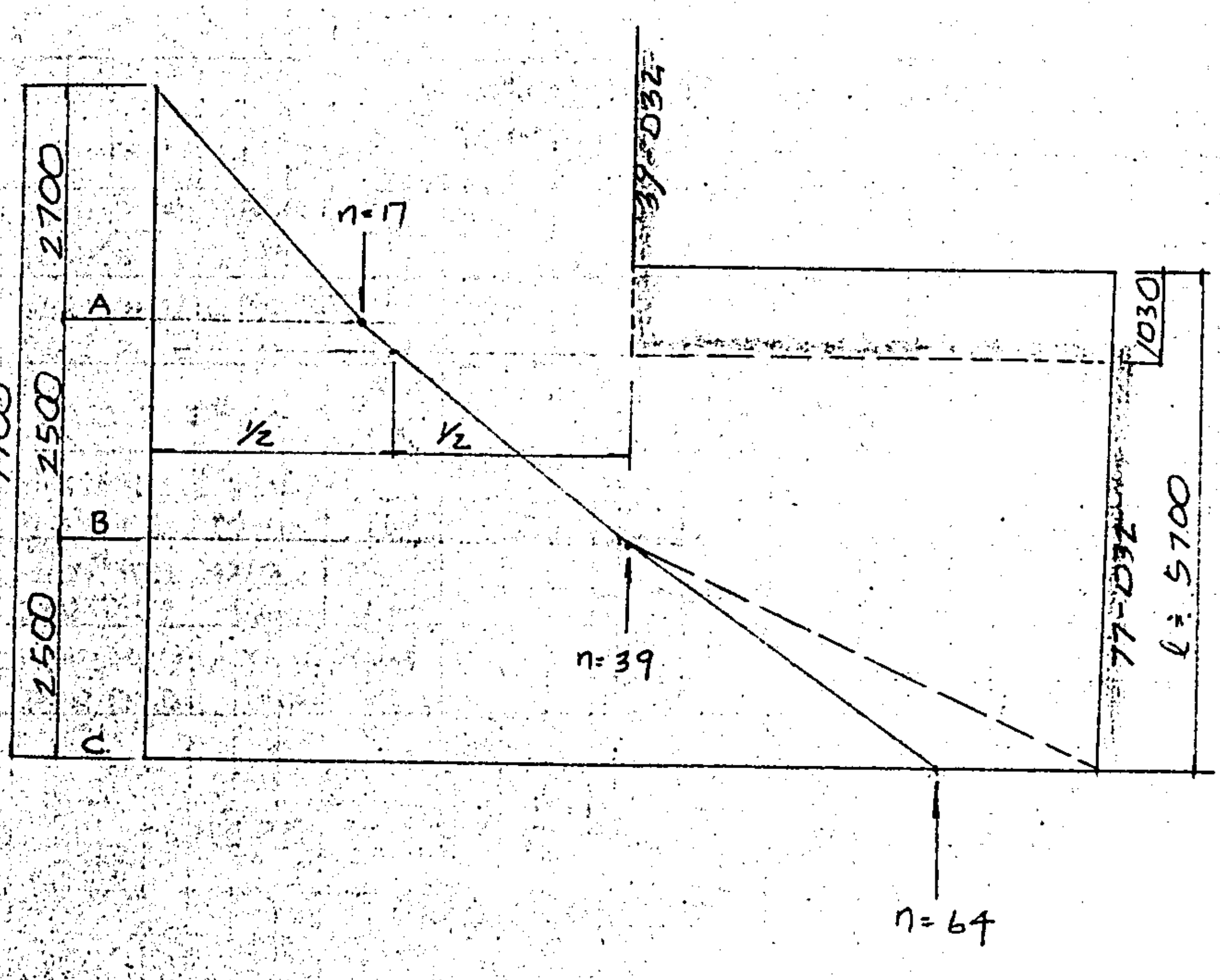
$$M_B = 19.10 + (42.84 \times 6.85 + 13.24 \times 6.362 + 2.5 \times 11.290 \times 4.20^2 \times 1/2) \times 0.1 + (68.53 + 19.82) \times 7.17 \times 2 = 1348.70^{tm}$$

$$N_B = 1263.14 + 2.5 \times 11.290 \times 2.50 = 1333.70^{t}$$

4-2. THE LEAST REINFORCING BARS REQUIRED AT THE SECTION A AND B

SECTION	A	B	C
M (tm)	872.12	1348.70	1831.71
N (t)	1263.14	1333.70	1404.27
b (cm)	940	"	"
h (")	110	"	"
d' (")	8.5	"	"
As	D32 - 17	D32 - 39	D32 - 64
σ_c (kg/cm²)	90.8	118.7	113.5
σ_s (")	2627	3630	2697

4-3 MOMENT DIAGRAM



7 DESIGN OF PILE

1. REACTION FORCES ACTING ON PILES

EXAMINATION IS MADE ONLY TO THE EARTHQUAKE LOADING.

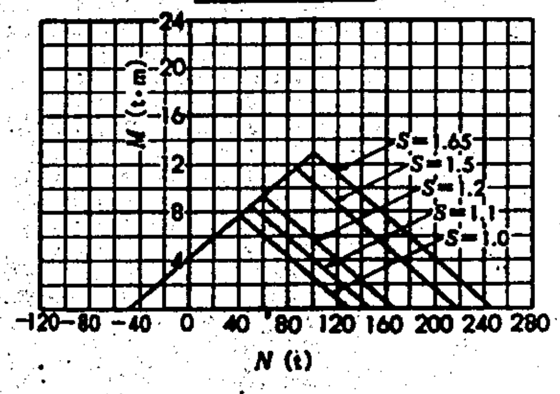
	LONGITUDINAL DIRECTION ①	TRANSVERSAL DIRECTION ② SINGLE TRACK	TRANSVERSAL DIRECTION ③ DOUBLE TRACK
WHOLE HORIZONTAL FORCE, ΣH (t)	248.03	150.73	230.13
HORIZONTAL FORCE PER PILE, H (t)	5.17	3.14	4.79
VALUE OF β (m^{-1})	0.214	〃	〃
MAXIMUM MOMENT PER PILE, M_{max} (tm)	7.78	4.72	7.21
MAXIMUM AXIAL FORCE PER PILE, P_{max} (t)	71.98	58.85	63.84
MINIMUM AXIAL FORCE PER PILE, P_{min} (t)	17.68	6.67	25.83

WHERE, $H = \Sigma H / n$, n …. NUMBER OF PILE $n = 48$

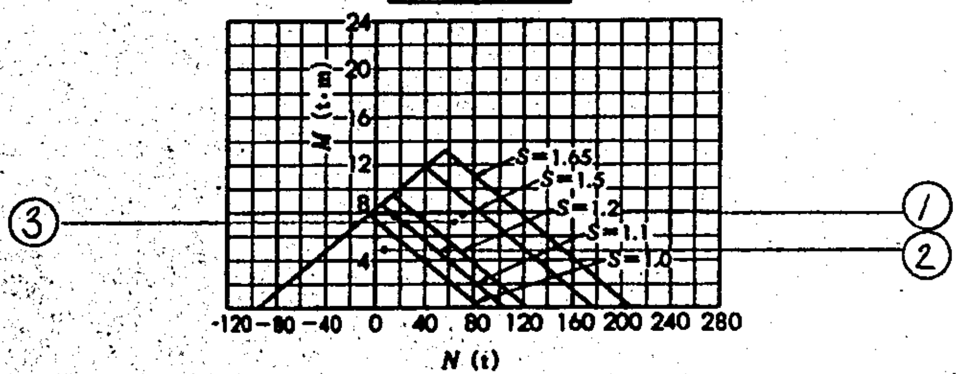
$$M_{max} = \frac{0.322 \cdot H}{\beta}$$

2. EXAMINATION OF SAFETY

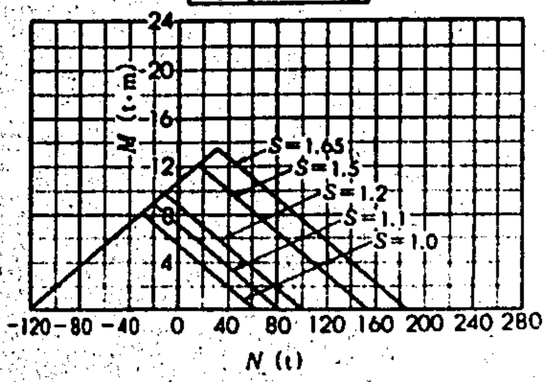
13 D500-A



14 D500-B

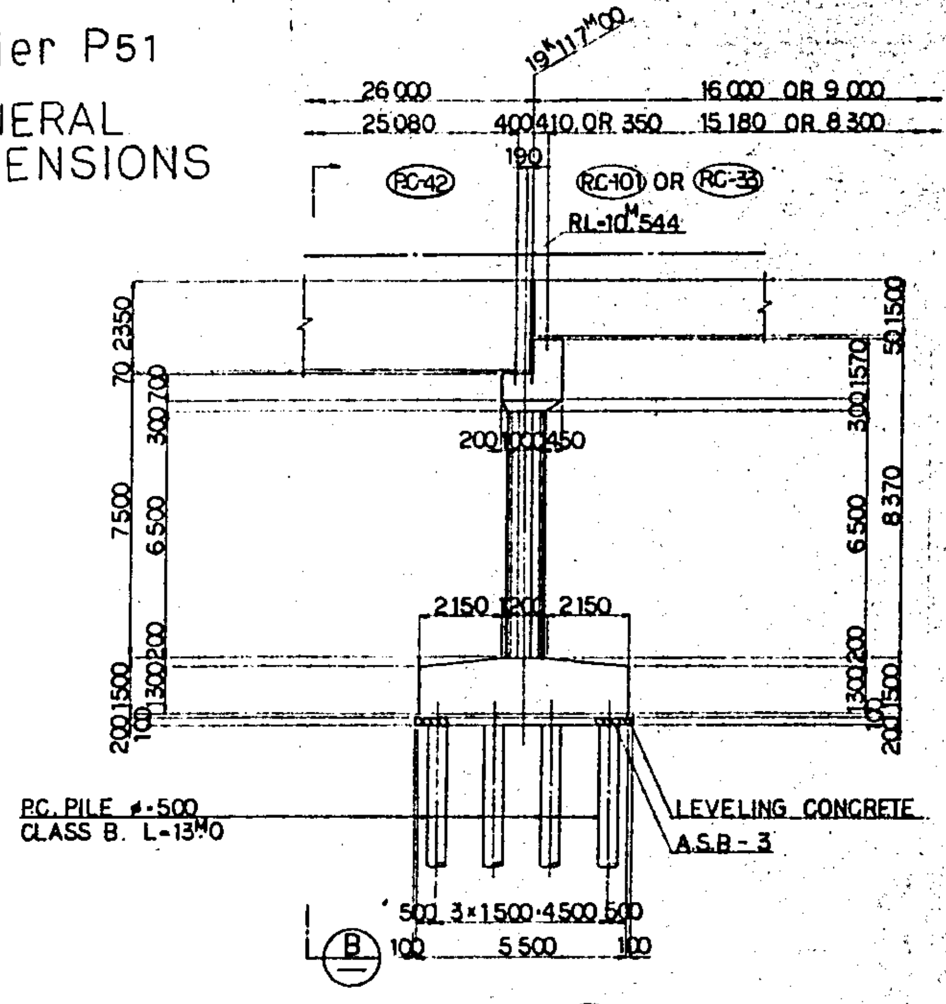


15 D500-C

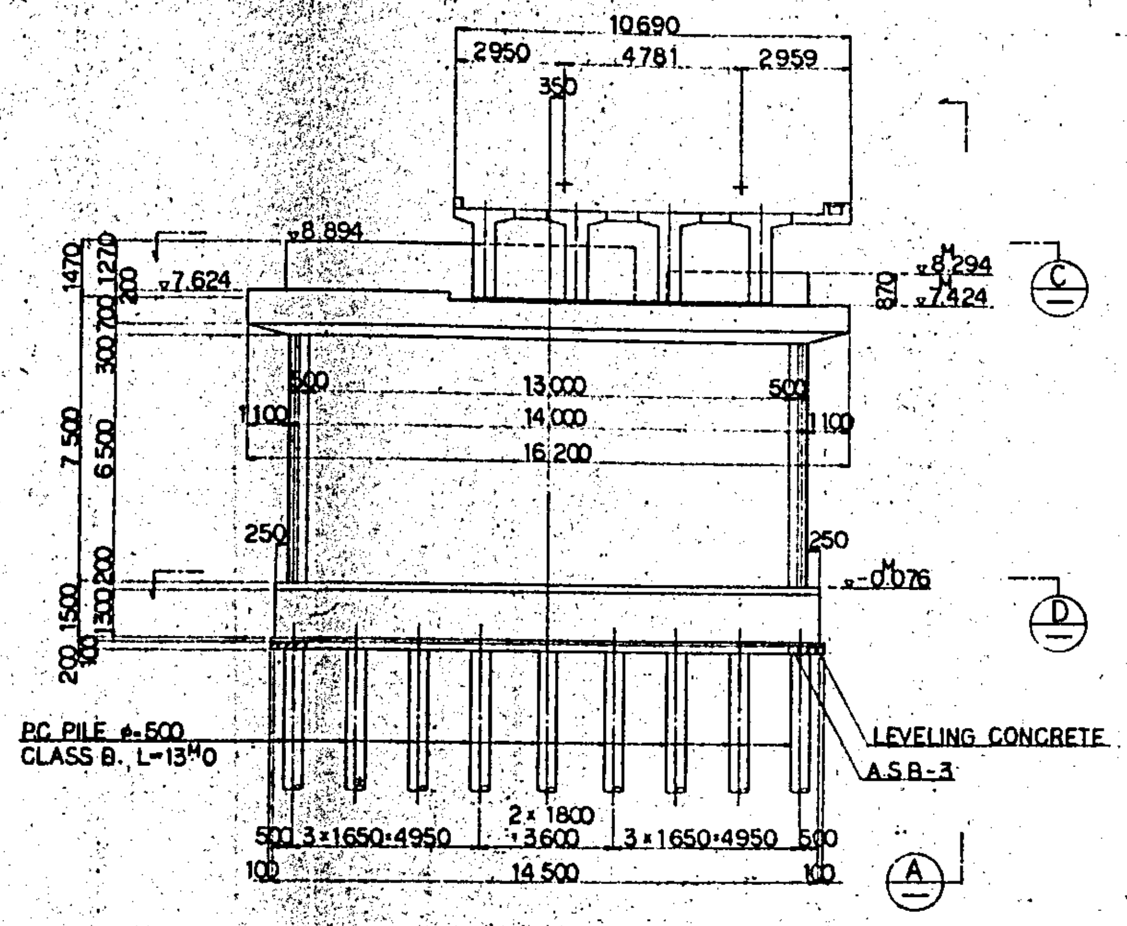


§12. Pier P51

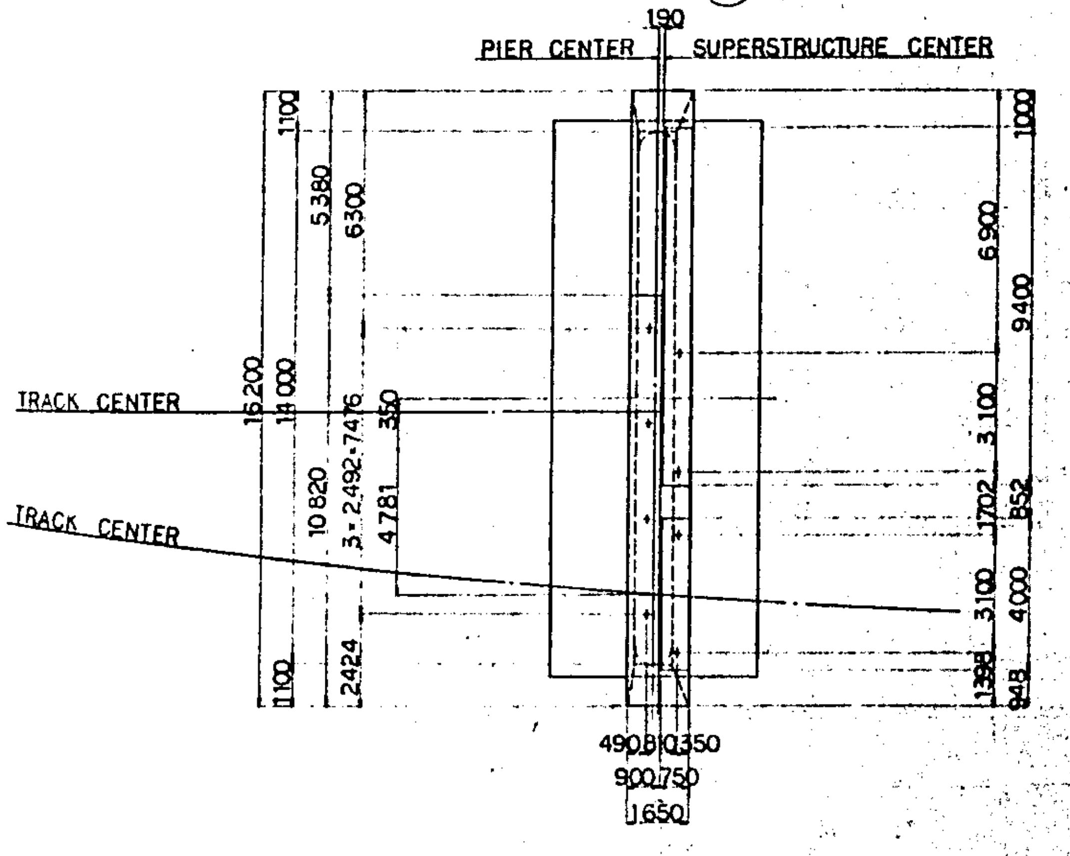
1 GENERAL DIMENSIONS



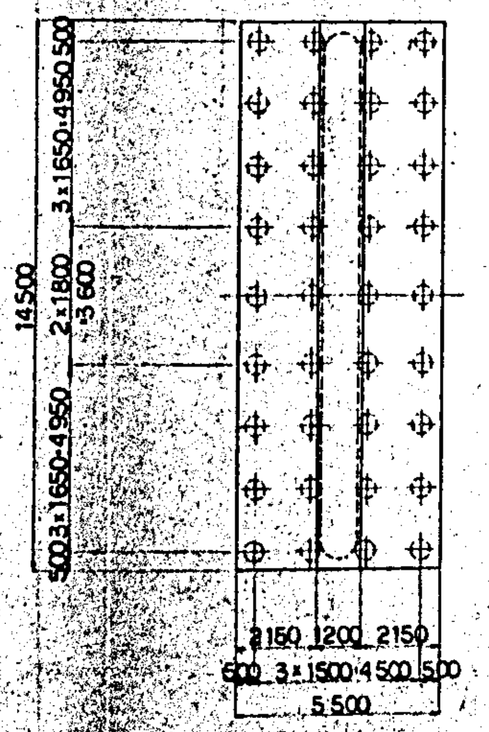
SECTION A



SECTION B

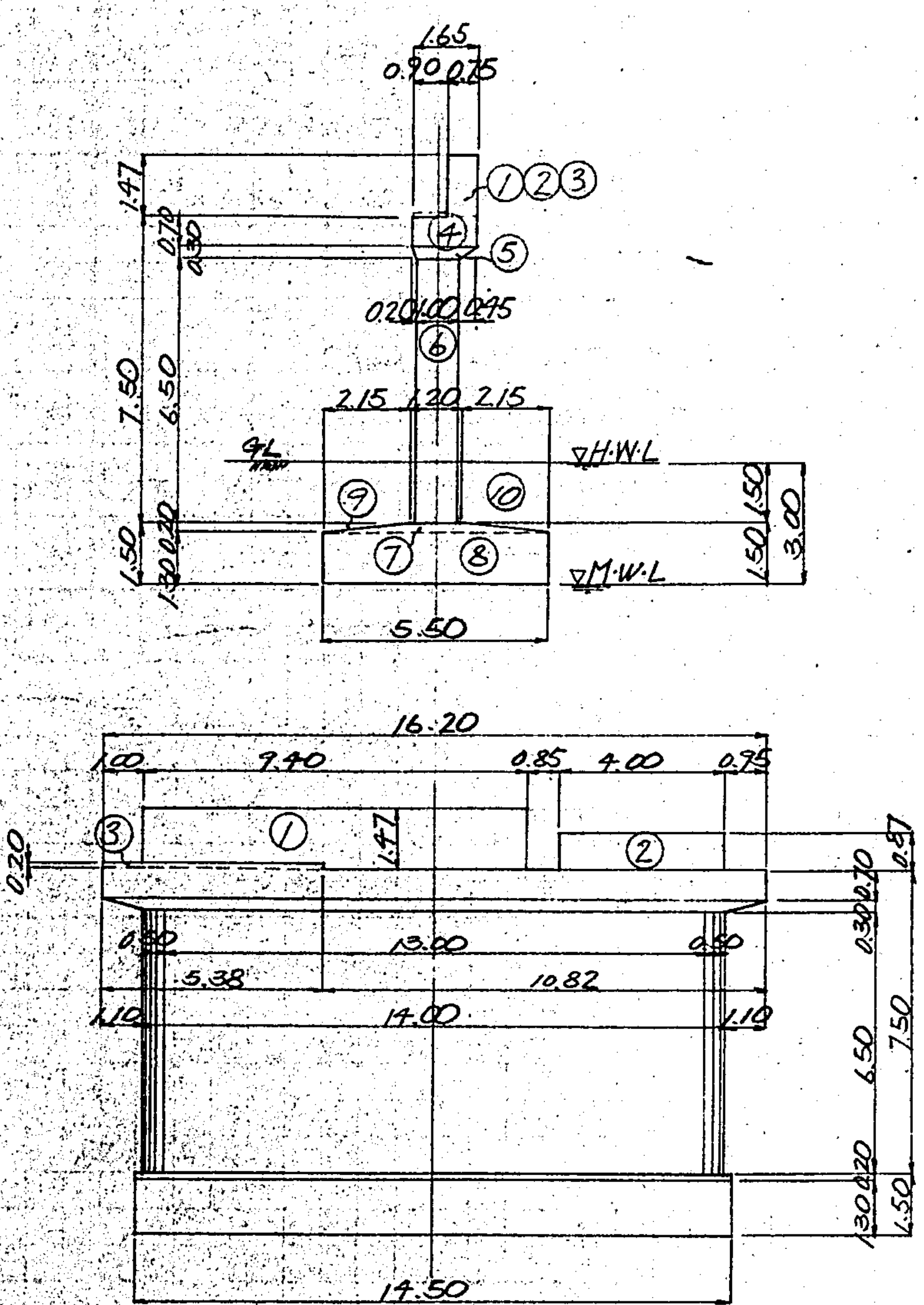


SECTION C



SECTION D

2 OWN WEIGHT OF SUBSTRUCTURE AND WEIGHT EARTH COVER



1. OWN WEIGHT OF SUBSTRUCTURE AND WEIGHT EARTH COVER

SUBSTRUCTURE

①	$2.5^{t/m^3} \times 0.75 \times 1.47 \times 9.40$	25.91^{t}
②	$2.5'' \times 0.75 \times 0.87 \times 9.40$	$15.33''$
③	$2.5'' \times 0.85 \times 0.20 \times 5.38$	$2.29''$
④	$2.5'' \times 1.65 \times 0.70 \times 16.20$	$46.78''$
⑤	$2.5'' \times \frac{0.30}{6} \times \{16.20 \times 1.65 + 14.00 \times 1.00 + (16.20 + 14.00) \times (1.65 + 1.00)\}$	$15.10''$
⑥	$2.5'' \times (\frac{\pi}{4} \times 1.00^2 + 13.00 \times 1.00) \times 6.50$ (≈ 13.785)	$224.01''$
⑦	$2.5'' \times \frac{1}{2} \times (1.20 + 5.50) \times 0.20 \times 14.50$	$24.29''$
⑧	$2.5'' \times 5.50 \times 14.50 \times 1.30$	$259.19''$
	SUB-TOTAL	612.90^{t}

EARTH COVER

⑨	$1.8^{t/m^3} \times \frac{1}{2} \times 2.15 \times 0.20 \times 14.50 \times 2$	11.22^{t}
⑩	$1.8'' \times (5.50 \times 14.50 - 13.785) \times 1.50$	$178.11''$
	SOB-TOTAL	189.33^{t}

TOTAL $612.90 + 189.33 = 802.23^{t}$

2. BUOYANCY

$1.0^{t/m^3} \times 5.50 \times 14.50 \times 3.00 = 239.25^{t}$

3. HORIZONTAL FORCES DUE TO EARTHQUAKE LOAD

	N (t)	y (m)	N×y (t·m)
①	25.91	9.735	252.23
②	15.33	9.435	144.64
③	2.29	9.10	20.84
④	46.78	8.65	404.65
⑤	15.10	8.162	123.25
⑥	224.01	4.75	1064.05
⑦	24.29	1.379	33.50
⑧	259.19	0.65	168.47
Σ	612.90		2211.63

NOTE N: AXIAL FORCE

y: DISTANCE FROM THE BOTTOM OF FOOTING TO GRAVITY CENTER

4. OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING

$$y_e = \frac{N \cdot y}{N} = \frac{2211.63}{612.90} = 3.608^{m}$$

$$P_E = 612.90 \times 0.1 = 61.29^{t}$$

$$M_o = 61.29 \times 3.608 = 221.13^{tm}$$

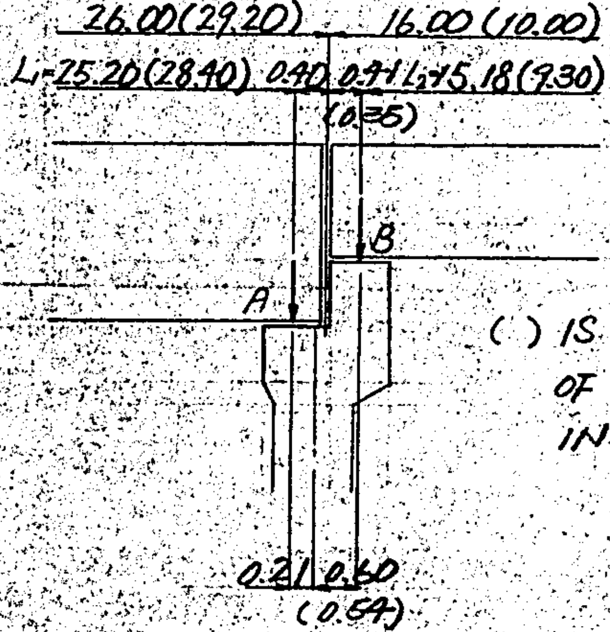
NOTE y_e : DISTANCE FROM THE BOTTOM OF FOOTING TO THE CENTER OF SUBSTRUCTURE.

P_E : HORIZONTAL FORCE DUE TO EARTHQUAKE LOAD

M_o : OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING

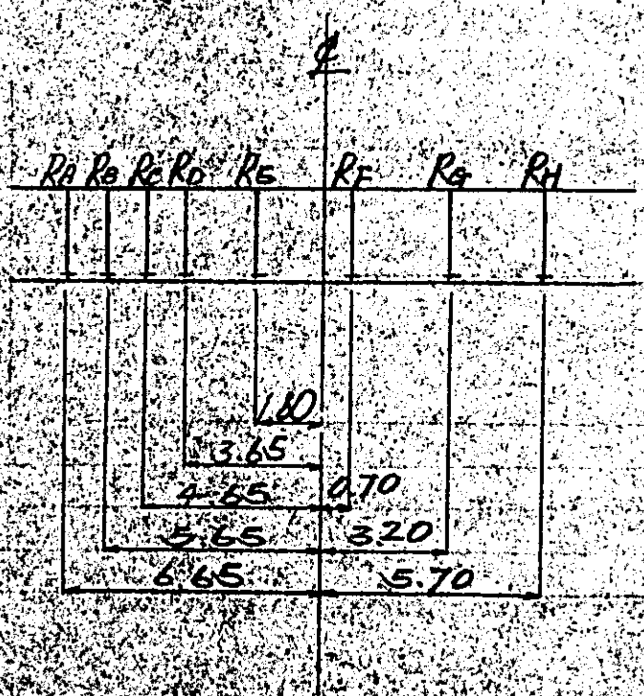
[3] REACTION FORCES TRANSMITTED FROM SUPERSTRUCTURE

1. REACTION FORCES DUE TO DEAD WEIGHT OF SUPERSTRUCTURE



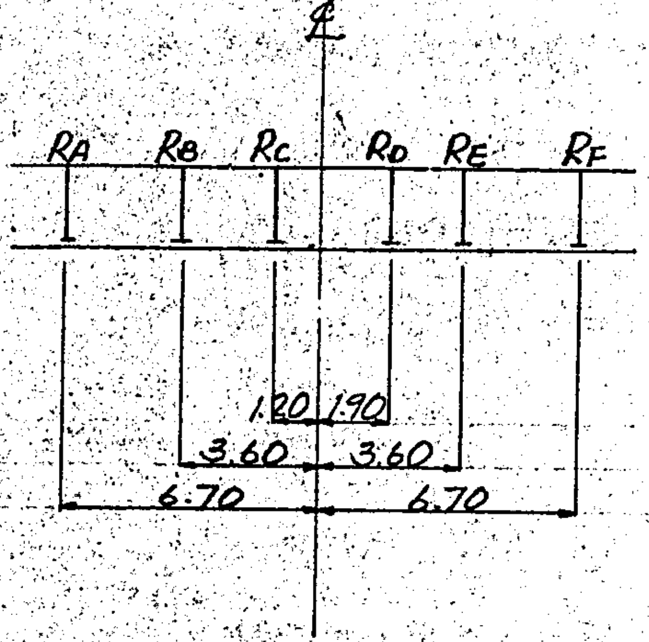
( ) IS SHOWN THE DIMENSIONS OF THE SUPERSTRUCTURE IN FUTURE

1-1. REACTION FORCES ACTING AT BEARINGS - A



FUTURE	PRESENT
RA = 67.3 t	RE = 92.5 t
RB = 64.2 "	RF = 92.5 "
RC = 64.2 "	RG = 92.5 "
RD = 64.2 "	RH = 92.5 "
Σ = 259.9 t	Σ = 370.0 t

1-2. REACTION FORCES ACTING AT BEARINGS-B



FUTURE	PRESENT
$R_A = 34.0^t$	$R_E = 62.0^t$
$R_B = 34.0$ "	$R_F = 62.0$ "
$R_C = 34.0$ "	$\Sigma = 124.0^t$
$R_D = 34.0$ "	
$\Sigma = 136.0^t$	

1-3. SECTIONAL MOMENT AT THE CENTER OF PIER SHAFT DUE TO DEAD WEIGHT OF SUPERSTRUCTURE

SINGLE-TRACK LOADING FOR CALCULATION OF TRANSVERSAL STABILITY. DOUBLE-TRACK LOADING FOR CALCULATION OF LONGITUDINAL AND TRANSVERSAL STABILITY

TRANSVERSAL DIRECTION

SINGLE-TRACK

$$M_1 = 370.0 \times 1.95 + 124.0 \times 5.15 = 1360.10^{tm}$$

DOUBLE-TRACK

$$M_2 = 1360.10 - 67.3 \times 6.65 - 64.2 \times 3 \times 4.65 - 136.0 \times 2.40 = -309.44^{tm}$$

LONGITUDINAL DIRECTION

$$M_3 = (259.9 + 370.0) \times -0.21 + 136.0 \times 0.54 + 124.0 \times 0.60 = 15.56^{tm}$$

1-4 HORIZONTAL FORCES DUE TO EARTHQUAKE ACTING ON DEAD WEIGHT OF SUPERSTRUCTURE

a) LONGITUDINAL DIRECTION

L_1 - SPAN SIDE

$$P_{E1-1} = (340.0+370.0)\times 0.1 - \frac{1}{2}\times 0.1\times 340.0 = 54.00^{t}$$

$$P_{E1-2} = 259.9\times 2\times 0.1 - \frac{1}{2}\times 0.1\times 259.9 = 38.99^{\prime\prime}$$

$$P_{E1} = 92.99^{t}$$

L_2 - SPAN SIDE

$$P_{E2-1} = 136.0\times 2\times 0.1\times 0.55 = 14.96^{t}$$

$$P_{E2-2} = 0.1\times 124.0 = 12.40^{\prime\prime}$$

$$P_{E2} = 27.36^{t}$$

$$\Sigma P_E = P_{E1} + P_{E2} = 92.99 + 27.36 = 120.35^{t}$$

OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING

$$M = 54.00\times 9.00 + 38.99\times 9.20$$

$$+ 14.96\times 10.47 + 12.40\times 9.87 = 1123.73^{tm}$$

b) TRANSVERSAL DIRECTION

SINGLE-TRACK

L1-SPAN SIDE

$P_{E1} = 370.0 \times 0.1 = 37.00^t$

L2-SPAN SIDE

$P_{E2} = 124.0 \times 0.1 = 12.40^t$

$\Sigma P_E = P_{E1} + P_{E2} = 37.00 + 12.40 = 49.40^t$

DOUBLE-TRACK

L1-SPAN SIDE

$P_{E1} = (259.9 + 370.0) \times 0.1 = 62.99^t$

L2-SPAN SIDE

$P_{E2} = (136.0 + 124.0) \times 0.1 = 26.00^t$

$\Sigma P_E = P_{E1} + P_{E2} = 62.99 + 26.00 = 88.99^t$

OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING

SINGLE-TRACK

$M = 37.00 \times 11.17 + 12.40 \times 11.42 = 554.90^t$

DOUBLE-TRACK

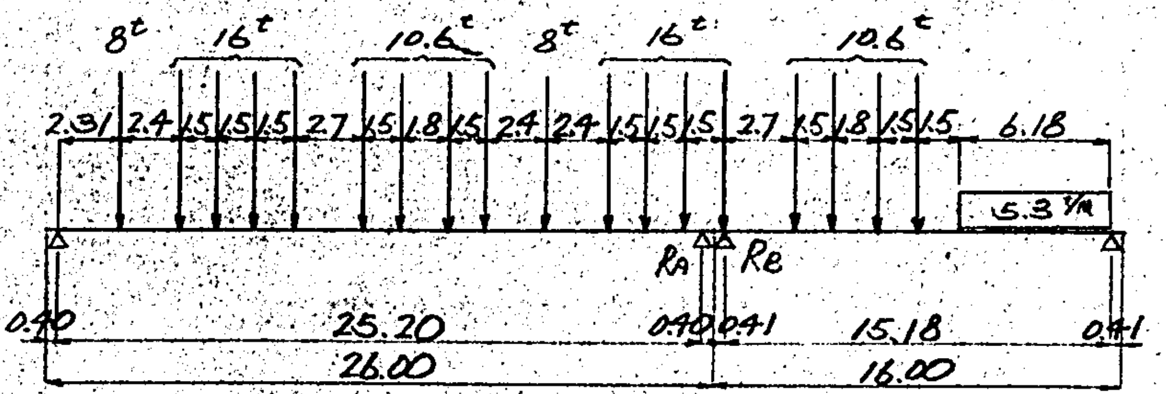
$M = 62.99 \times 11.17 + 26.00 \times 11.42 = 1000.52^t$

2. REACTION FORCES DUE TO LIVE LOAD PLUS -IMPACT LOAD

EQUIVALENT TO KS-16 LOADING

CALCULATION IS CARRIED OUT UNDER THE LOADING CONDITION OF PRODUCING MAXIMUM REACTION FORCE ON PIER

PRESENT TRACK



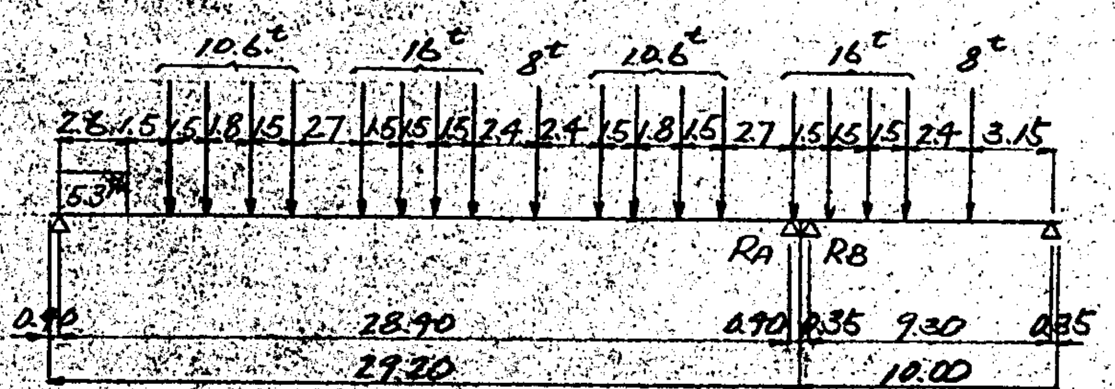
$$R_A = \frac{1}{25.20} \times (8 \times 2.31 + 16 \times 4 \times 6.96 + 10.6 \times 4 \times 14.31$$

$$+ 8 \times 19.11 + 16 \times 3 \times 23.01) = 92.38^t$$

$$R_B = \frac{1}{15.18} \times (\tfrac{1}{2} \times 5.3 \times 6.18^2 + 10.6 \times 4 \times 10.08$$

$$+ 16 \times 15.18) = 50.82^t$$

FUTURE TRACK



$$R_A = \frac{1}{28.40} \times (1/2 \times 5.3 \times 2.60^2 + 10.6 \times 4 \times 6.50$$
$$+ 16 \times 4 \times 13.85 + 8 \times 18.50 + 10.6 \times 4 \times 23.30$$
$$+ 16 \times 28.40) = 97.54^t$$

$$R_B = \frac{1}{9.30} \times (8 \times 3.15 + 16 \times 3 \times 7.05) = 39.10^t$$

IMPACT COEFFICIENT

25.20 m - SPAN SIDE ---- $i = 0.354$

15.18 " - SPAN SIDE ---- $i = 0.399$

28.40 " - SPAN SIDE ---- $i = 0.345$

9.30 " - SPAN SIDE ---- $i = 0.437$

LIVE LOAD PLUS IMPACT

SINGLE-TRACK

$R_{(A+i)} = 92.38 \times 1.354 = 125.08^t$

$R_{(B+i)} = 50.82 \times 1.399 = 71.10$ "

TOTAL = 196.18^t

DOUBLE-TRACK

$R_{(A+i)} = 97.54 \times 1.345 \times 2 = 262.38^t$

$R_{(B+i)} = 39.10 \times 1.437 \times 2 = 112.37$ "

TOTAL = 374.75^t

2-1. SECTIONAL MOMENT ACTING AT THE CENTER OF PIER SHAFT

SINGLE-TRACK LOADING FOR CALCULATION OF TRANSVERSAL STABILITY. DOUBLE-TRACK LOADING FOR CALCULATION OF LONGITUDINAL ANT TRANSVERSAL STABILITY.

TRANSVERSAL DIRECTION

SINGLE-TRACK

$$M = 125.08 \times 1.95 + 71.10 \times 5.15 = 610.07^{tm}$$

DOUBLE-TRACK

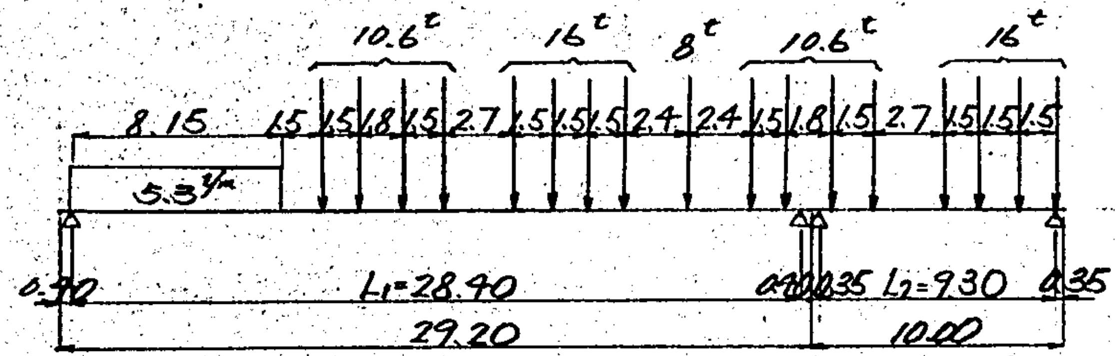
$$M = 262.38 \times \frac{1}{2} \times (1.95 - 5.15)$$
$$- 112.37 \times 2.40 = -689.50^{tm}$$

LONGITUDINAL DIRECTION

$$M = 262.38 \times -0.21 + 112.37 \times 0.54$$
$$= 5.58^{tm}$$

3. BRAKE LOAD AND TRACTION LOAD

UNDER THE LOADING CONDITION AS SHOWN BELOW



3-1. BRAKE LOAD

15% OF KS LOADING

L1 - SPAN SIDE

$$P_{B-1} = (5.3^{t/m} \times 8.15 + 10.6 \times 4 + 16 \times 4 + 8 + 10.6 \times 2) \times 0.15 \times \frac{1}{2} = 13.41^{t}$$

L2 - SPAN SIDE

$$P_{B-2} = (10.6 \times 2 + 16 \times 4) \times 0.15 \times 0.55 = 7.03^{t}$$

$$\Sigma P_B = P_{B-1} + P_{B-2} = 13.41 + 7.03 = 20.44^{t}$$

3-2. TRACTION LOAD

25% OF MOVING WHEEL AXLE LOAD OF KS LOADING

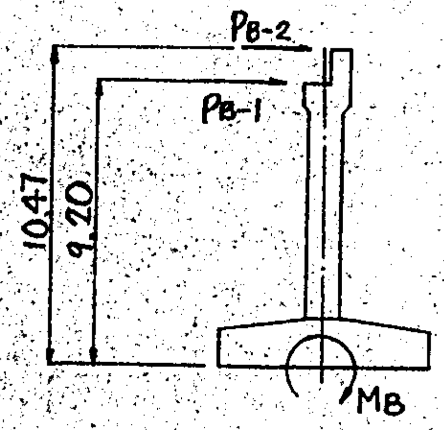
L1 - SPAN SIDE

$P_{T-1} = 16 \times 4 \times 0.25 \times \frac{1}{2} = 8.00^t$

L2 - SPAN SIDE

$P_{T-2} = 16 \times 4 \times 0.25 \times 0.55 = 8.80^t$

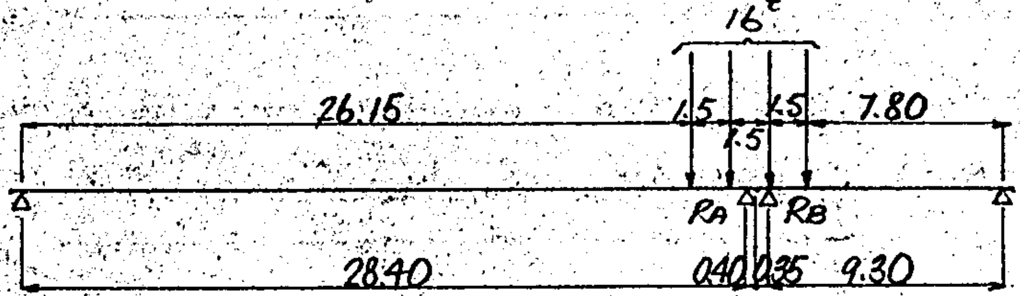
3-3. OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING



$M_B = 13.41 \times 9.20 + 7.03 \times 10.47 = 196.98^{tm}$

4. TRAIN LATERAL LOAD

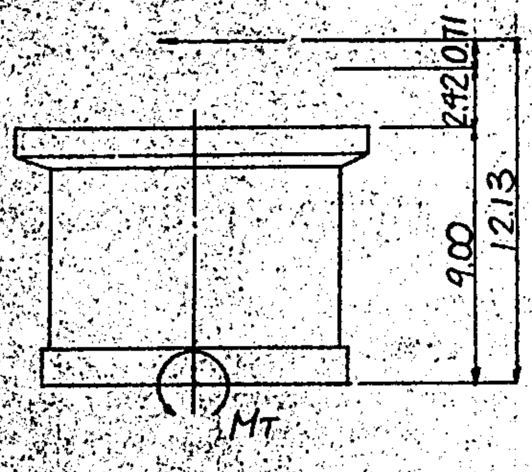
TRANSVERSAL FORCE DUE TO LIVE (TRAIN) LOAD IS ASSUMED TO BE EQUIVALENT TO THE SERIES OF CONCENTRATED LOADS CONSISTED OF 15% OF MOVING WHEEL AXLE LOAD ACTING TRANSVERSALLY AT THE TRACK LEVEL.
CALCULATION IS MADE UNDER THE SINGLE-TRACK LOADING



$$R_A + R_B = \frac{16}{28.40} \times (26.15 + 27.65) + \frac{16}{9.30} \times (7.80 + 9.30) = 59.73^t$$

$$P_T = 59.73 \times 0.15 = 8.96^t$$

5-1. OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING.



$$M_T = 8.96 \times 12.13 = 108.68^{tm}$$

4 CALCULATION ON STABILITY

1. COMBINATION OF LOADS

1-1. TRANSVERSAL DIRECTION

No1. DEAD LOAD (M.W.L)

No2. DEAD LOAD + (TRAIN + IMPACT) LOAD (M.W.L)

No3. ——— " ——— (H.W.L)

No4. DEAD LOAD + (TRAIN + IMPACT) LOAD
+ TRAIN LATERAL LOAD
+ BRAKE LOAD OR TRACTION LOAD (M.W.L)

No5. ——— " ——— (H.W.L)

No6. DEAD LOAD + EARTHQUAKE LOAD (M.W.L)

No7. DEAD LOAD (M.W.L)

No8. DEAD LOAD + (TRAIN + IMPACT) LOAD
+ CENTRIFUGAL LOAD (M.W.L)

No.9. ——— " ——— (H.W.L)

No10. DEAD LOAD + (TRAIN + IMPACT) LOAD
+ TRAIN LATERAL LOAD
+ BRAKE LOAD OR TRACTION LOAD (M.W.L)

No11 ——— " ——— (H.W.L)

No12 DEAD LOAD + EARTHQUAKE LOAD (M.W.L)

THE ABOVE LOADING COMBINATIONS ARE APPLIED NO. 1 THRU NO. 6 FOR SINGLE-TRACK LOADING AND NO. 7 THRU NO. 12 FOR DOUBLE-TRACK LOADING.

1-2. LONGITUDINAL DIRECTION.

No.1. DEAD LOAD (M.W.L)

No.2. DEAD LOAD + (TRAIN + IMPACT) LOAD (M.W.L)

No.3. ——————— " ——————— (H.W.L)

No.4. DEAD LOAD + (TRAIN + IMPACT) LOAD
+ TRAIN LATERAL LOAD
+ BRAKE LOAD or TRACTION LOAD (M.W.L)

No.5. ——————— " ——————— (H.W.L)

No.6. DEAD LOAD + EARTHQUAKE LOAD (M.W.L)

ALL OF THE ABOVE LOADING COMBINATIONS ARE APPLIED FOR DOUBLE-TRACK LOADING.

DOUBLE-TRACK LOADING

	AXIAL FORCE	MOMENT	HORIZONTAL FORCES
No7	802.23+259.9 +370.0+124.0 +136.0=1692.13	309.44	———
No8	1692.13+374.75 =2066.88	309.44+689.50 =998.94	———
No9	2066.88-239.25 =1827.63	998.94	———
No10	2066.88	998.94+108.68 =1107.62	8.96
No11	1827.63	1107.62	8.96
No12	1692.13	309.44+219.73 +1000.52 =1529.69	61.12+88.99 =150.11

2. TABLE OF FORCE AND MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING.

2-1. TRANSVERSAL DIRECTION

SINGLE-TRACK LOADING

	AXIAL FORCE	MOMENT	HORIZONTAL FORCE
No1	802.23 + 370.0 + 124.0 = 1296.23	1360.10	———
No2	1296.23 + 196.18 = 1492.41	1360.10 + 610.07 = 1970.17	———
No3	1492.41 − 239.25 = 1253.16	1970.17	———
No4	1492.41	1970.17 + 108.68 = 2078.85	8.96
No5	1253.16	2078.85	8.96
No6	1296.23	1360.10 + 219.73 + 554.90 = 2134.73	61.12 + 49.40 = 110.52

2-2, LONGITUDINAL DIRECTION

	AXIAL FORCE	MOMENT	HORIZONTAL FORCE
No 1	1692.13	15.56	———
No 2	2066.88	15.56 + 5.58 = 21.14	———
No 3	1827.63	21.14	———
No 4	2066.88	21.14 + 196.98 = 218.12	20.44
No 5	1827.63	218.12	20.44
No 6	1692.13	15.56 + 219.73 + 1123.73 = 1359.02	61.12 + 120.35 = 181.47

3. REACTION FORCE ACTING ON PILES

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

WHERE: P REACTION FORCE

n NUMBER OF PILES

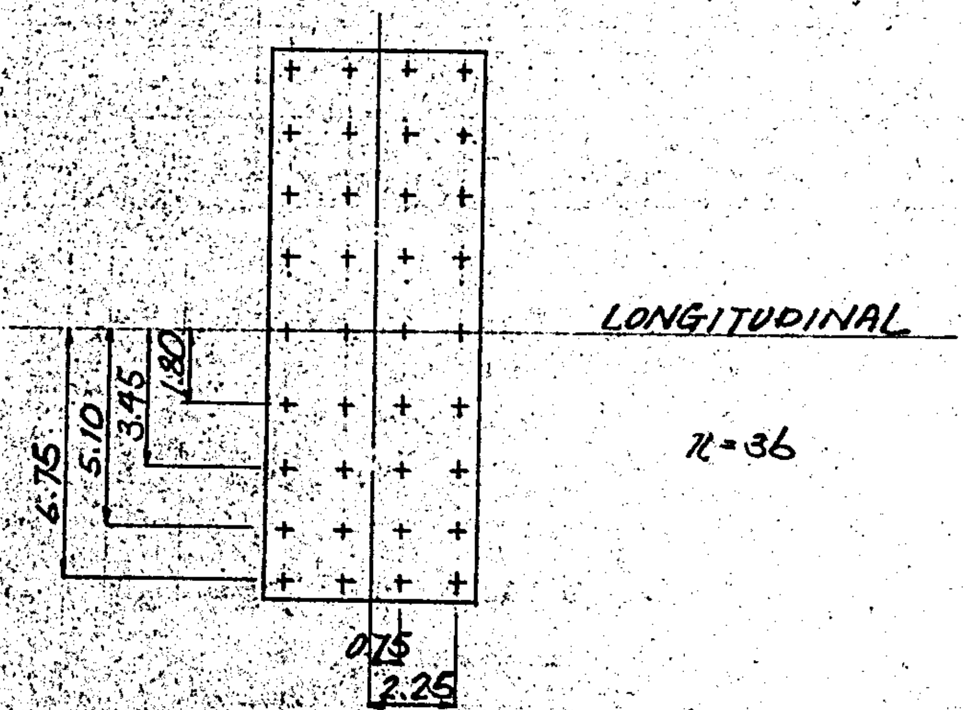
N AXIAL FORCE ACTING AT THE CENTER OF THE BOTTOM OF FOOTING

M MOMENT ACTING AT THE CENTER OF THE BOTTOM OF FOOTING

I MOMENT OF INERTIA OF THE GROUP OF PILES WITH RESPECT TO THE GEOMETRICAL CENTER

x DISTANCE FROM THE GEOMETRICAL CENTER OF THE GROUP OF PILES TO THE CENTER OF THE PILE TO BE CALCULATED

3-1. ARRANGEMENT OF PILES



3-2 REACTION FORCE ACTING ON PILES DUE TO TRANSVERSAL LOAD

	N (t)	M (t·m)	$\frac{N}{n}$ (t)	$\frac{M}{I}$ ($\frac{t}{m}$)	P_{max} (t)	P_{min} (t)
No 1	1296.23	1360.10	36.01	1.96	49.24	22.78
No 2	1492.41	1970.17	41.46	2.84	60.63	22.29
No 3	1253.16	1970.17	34.81	2.84	53.98	15.64
No 4	1492.41	2078.85	41.46	3.00	61.71	21.21
No 5	1253.16	2078.85	34.81	3.00	55.06	14.56
No 6	1296.23	2134.73	36.01	3.08	56.80	15.22
No 7	1692.13	309.44	47.00	0.45	50.04	43.96
No 8	2066.88	998.94	57.41	1.44	67.13	47.69
No 9	1827.63	998.94	50.77	1.44	60.49	41.05
No 10	2066.88	1107.62	57.41	1.60	68.21	46.61
No 11	1827.63	1107.62	50.77	1.60	61.57	39.97
No 12	1692.13	1529.69	47.00	2.21	61.92	32.08

$$I = (6.75^2 + 5.10^2 + 3.45^2 + 1.80^2) \times 4 \times 2 = 693.72 \, m^2$$

$$x = 6.75^m$$

3-3. REACTION FORCE ACTING ON PILES DUE TO LONGITUDINAL LOAD

	N (t)	M (t·m)	$\frac{N}{n}$ (t)	$\frac{M}{I}$ ($\frac{t}{m}$)	x (m)	P_{max} (t)	P_{min} (t)
No1	1692.13	15.56	47.00	0.15	x_1	47.11	46.89
					x_2	47.34	46.66
					x_3	—	—
No2	2066.88	21.14	57.41	0.21	x_1	57.57	57.25
					x_2	57.88	56.94
					x_3	—	—
No3	1827.63	21.14	50.77	0.21	x_1	50.93	50.61
					x_2	51.24	50.30
					x_3	—	—
No4	2066.88	218.12	57.41	2.15	x_1	59.02	55.80
					x_2	62.25	52.57
					x_3	—	—
No5	1827.63	218.12	50.77	2.15	x_1	52.38	49.16
					x_2	55.61	45.93
					x_3	—	—
No6	1692.13	1359.02	47.00	13.42	x_1	57.07	36.94
					x_2	77.20	16.81
					x_3	—	—

$$I = (2.25^2 + 0.75^2) \times 9 \times 2 = 101.25 \ m^2$$

$$x_1 = 0.75 \ m$$

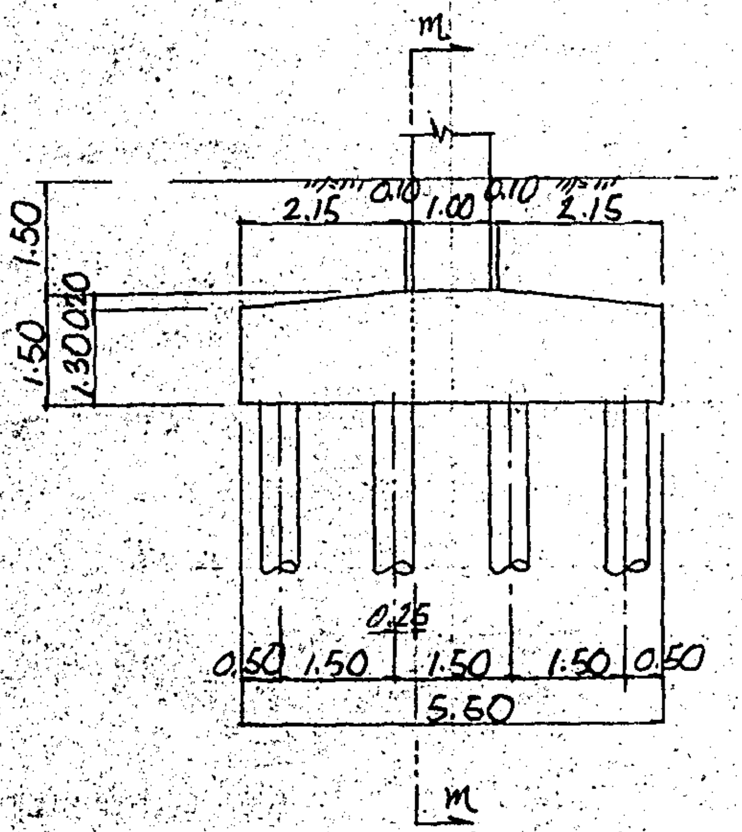
$$x_2 = 2.25 \ ''$$

5 DESIGN OF FOOTING

CALCULATION IS MADE ONLY THE CASE OF LONGITUDINAL LOADING, BECAUSE NO STRESS OCCURS UNDER THE TRANSVERSAL LOADING DUE TO REACTION FORCES ON PILES WITHIN THE WALL SHAFT

1. EXAMINATION OF SAFETY AGAINST BENDING MOMENT

1-1. BENDING MOMENT DUE TO OWN WEIGHT OF FOOTING AND WEIGHT OF EARTH COVER



WHERE: m SECTION WHERE MOMENT IS CALCULATED

M_m BENDING MOMENT AT SECTION m

BENDING MOMENT AT SECTION m

i) THE CASE NO BUOYANCY IS CONSIDERED

$$M_m = (2.5^{t/m^3} \times 1.50 + 1.8^{t/m^3} \times 1.50) \times 2.25^2 \times 1/2$$
$$- 1/2 \times 2.15 \times 0.20 \times 1.533 \times (2.5 - 1.8) = 16.10^{t\cdot m/m}$$

ii) THE CASE BUOYANCY IS CONSIDERED

$$M_m = 16.10 - 1/2 \times 2.25^2 \times 3.00 \times 1.00^{t/m^3} = 8.51^{t\cdot m/m}$$

1-2, STRESS DUE TO REACTION FORCE ON PILES

BENDING MOMENT AT SECTION m

No 1. $M_m = (47.34 \times 1.85 + 47.11 \times 0.35) \times \frac{9}{14.50} = 64.59^{t\cdot m/m}$

No 2 $M_m = (57.88 \times$ " $+ 57.57 \times$ " $) \times$ " $= 78.97$ "

No 3 $M_m = (51.24 \times$ " $+ 50.93 \times$ " $) \times$ " $= 69.90$ "

No 4 $M_m = (62.25 \times$ " $+ 59.02 \times$ " $) \times$ " $= 84.30$ "

No 5 $M_m = (55.61 \times$ " $+ 52.38 \times$ " $) \times$ " $= 75.23$ "

No 6 $M_m = (77.20 \times$ " $+ 57.07 \times$ " $) \times$ " $= 101.04$ "

1-3. SUMMARY OF BENDING MOMENT ON FOOTING

No.1: $M_x = (64.59 - 16.10) \times 1.8/1.4 = 62.34$ t·m/m

No.2: $M_x = (78.97 - 16.10) \times 1.00 = 62.87$ "

No.3: $M_x = (69.90 - 8.51) \times 1.00 = 61.39$ "

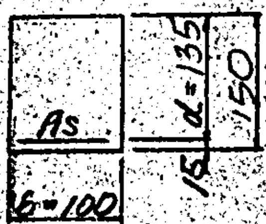
No.4: $M_x = (84.30 - 16.10) \times 1/1.15 = 59.30$ "

No.5: $M_x = (75.23 - 8.51) \times 1/1.15 = 58.02$ "

No.6: $M_x = (101.04 - 16.10) \times 1/1.50 = 56.63$ "

1-4. EXAMINATION OF SAFETY

$M_x = 62.87$ t·m/m



$A_s = 6.67 - D25 = 33.78$ cm²

$$P = \frac{A_s}{b \cdot d} = \frac{33.78}{100 \times 135} = 0.00250$$

$$\frac{1}{L_c} = 9.10 \qquad \frac{1}{L_s} = 436$$

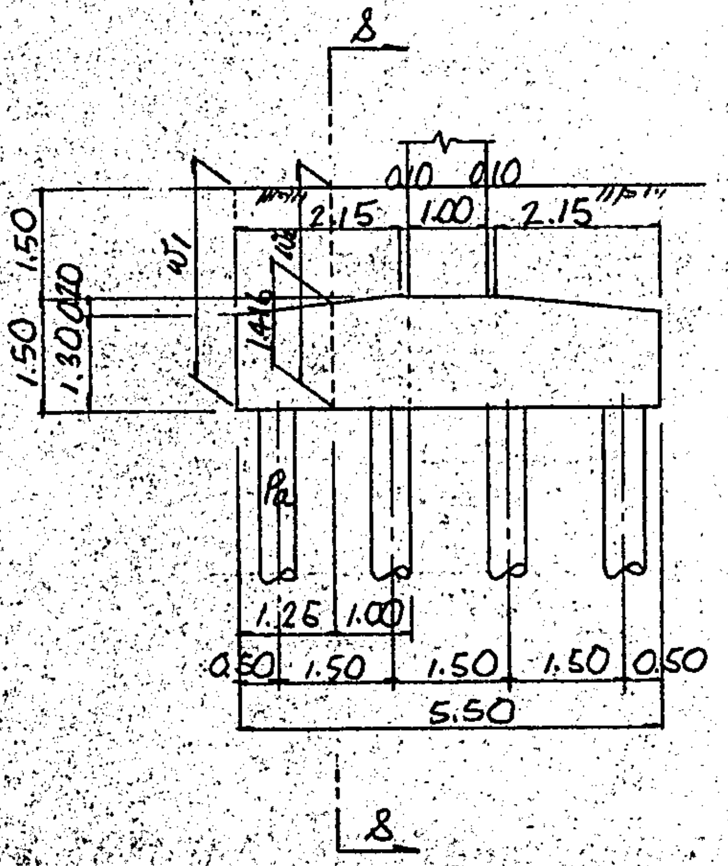
$$\sigma_c = \frac{M}{b \cdot d^2} \times \frac{1}{L_c} = \frac{62.87 \times 10^5}{100 \times 135^2} \times 9.10 = 31.4 \text{ kg/cm}^2 < 80 \text{ kg/cm}^2$$

$$\sigma_s = \frac{M}{b \cdot d^2} \times \frac{1}{L_s} = \text{———} '' \text{———} \times 436 = 1504 '' < 1800 ''$$

2. EXAMINATION OF SAFETY AGAINST SHEARING FORCE

EXAMINATION ON THE COMBINATION No.6

2-1. SHEARING FORCE : DUE TO OWN WEIGHT OF FOOTING AND WEIGHT OF EARTH COVER



WHERE: S SECTION WHERE SHEARING FORCE IS CALCULATED

S_s SHEARING FORCE AT SECTION S

SHEARING FORCE AT SECTION s

$$w_1 = 2.5^{T/m^3} \times 1.30 + 1.8^{T/m^3} \times 1.70 = 6.31^{T/m}$$

$$w_2 = (2.5 \times 1.416 + 1.8 \times 1.584) \times \frac{1.00}{2 \times (1.416 - 0.15)} = 2.52''$$

$$\therefore S_s = \tfrac{1}{2} \times (6.31 + 2.52) \times 1.25 = 5.52^t$$

2-2. STRESS DUE TO REACTION FORCE ON PILES

SHEARING FORCE AT SECTION s

$$P_a = 77.20 \times \frac{1.75}{2 \times (1.416 - 0.15)} = 53.36^t$$

$$\therefore S_s = 53.36 \times \frac{9}{14.50} = 33.12^t$$

2-3. SUMMARY OF SHEARING FORCES ON FOOTING

$$\therefore \Sigma S_s = 33.12 - 5.52 = 27.60^t$$

2-4. EXAMINATION OF SAFETY,

$$\tau = \frac{27.60 \times 10^3}{100 \times (141.6 - 15)} = 2.18^{kg/cm^2} < 3.7^{kg/cm^2}$$

6 DESIGN OF SUBSTRUCTURE.

EXAMINATION IS MADE ONLY THE CASE OF LONGITUDINAL LOADING.

1. COMBINATION OF LOADS

No.6 DEAD LOAD + EARTHQUAKE LOAD

2. BENDING MOMENT AND AXIAL FORCE ACTING AT THE BOTTOM OF WALL

THE FOLLOWING FIGURES ARE REFERRED TO THE CALCULATION OF THE REACTION FORCES TRANSMITTED FROM THE SUPERSTRUCTURE.

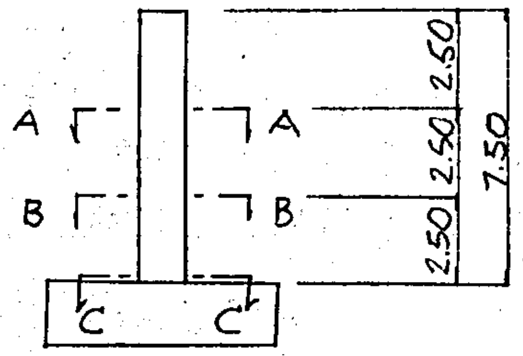
$$M = (25.91 \times 8.235 + 15.33 \times 7.935 + 2.29 \times 7.60$$
$$+ 46.78 \times 7.15 + 15.10 \times 6.662 + 224.01 \times 3.25) \times 0.1$$
$$+ 15.56 + 54.00 \times 7.50 + 38.99 \times 7.70$$
$$+ 14.96 \times 8.97 + 12.40 \times 8.37$$
$$= 1110.31 \text{ tm}$$

$$N = 25.91 + 15.33 + 2.29 + 46.78 + 15.10 + 224.01$$
$$+ 259.9 + 310.0 + 136.0 + 124.0$$
$$= 1219.32 \text{ t}$$

3. EXAMINATION OF SAFETY

M (tm)	1110.31
N (t)	1219.32
S (t)	
b (cm)	1300
h (cm)	100
d (cm)	
d' (cm)	7.5
As (cm²)	89 - D22 344.51
p	0.00265
As' (cm²)	=As
p'	=P
e=M/N (cm)	91.0
e=M/N+u (cm)	
e=M/N−u (cm)	
e/h	0.91
d/e	
d'/h	0.075
d'/d	
Ne/bd² (kg/cm²)	9.319
k	0.323
c	0.118
j	
1/Lc	
1/Ls	
β=σs/σc	
σc (kg/cm²)	79.2
σs (kg/cm²)	2208
τ (kg/cm²)	
σsa (kg/cm²)	120
σca (kg/cm²)	2700
τa (kg/cm²)	

4. DEDUCTION OF REINFORCING BARS



4-1 BENDING MOMENT AND AXIAL FORCE ACTING AT THE SECTION A AND B

$$M_A = 15.56 + (25.91\times3.235 + 15.33\times2.935 + 2.29\times2.60 + 46.78\times4.65 + 15.10\times1.662 + 2.5\times13.785\times1.50^2\times\tfrac{1}{2})\times0.1 + 54.00\times1.50 + 38.99\times2.70 + 14.96\times3.97 + 12.40\times3.37 = 398.63^{tm}$$

$$N_A = 25.91 + 15.33 + 2.29 + 46.78 + 15.10 + 2.5\times13.785\times1.50 + 259.96 + 310.00 + 136.00 + 124.00 = 1047.00^{t}$$

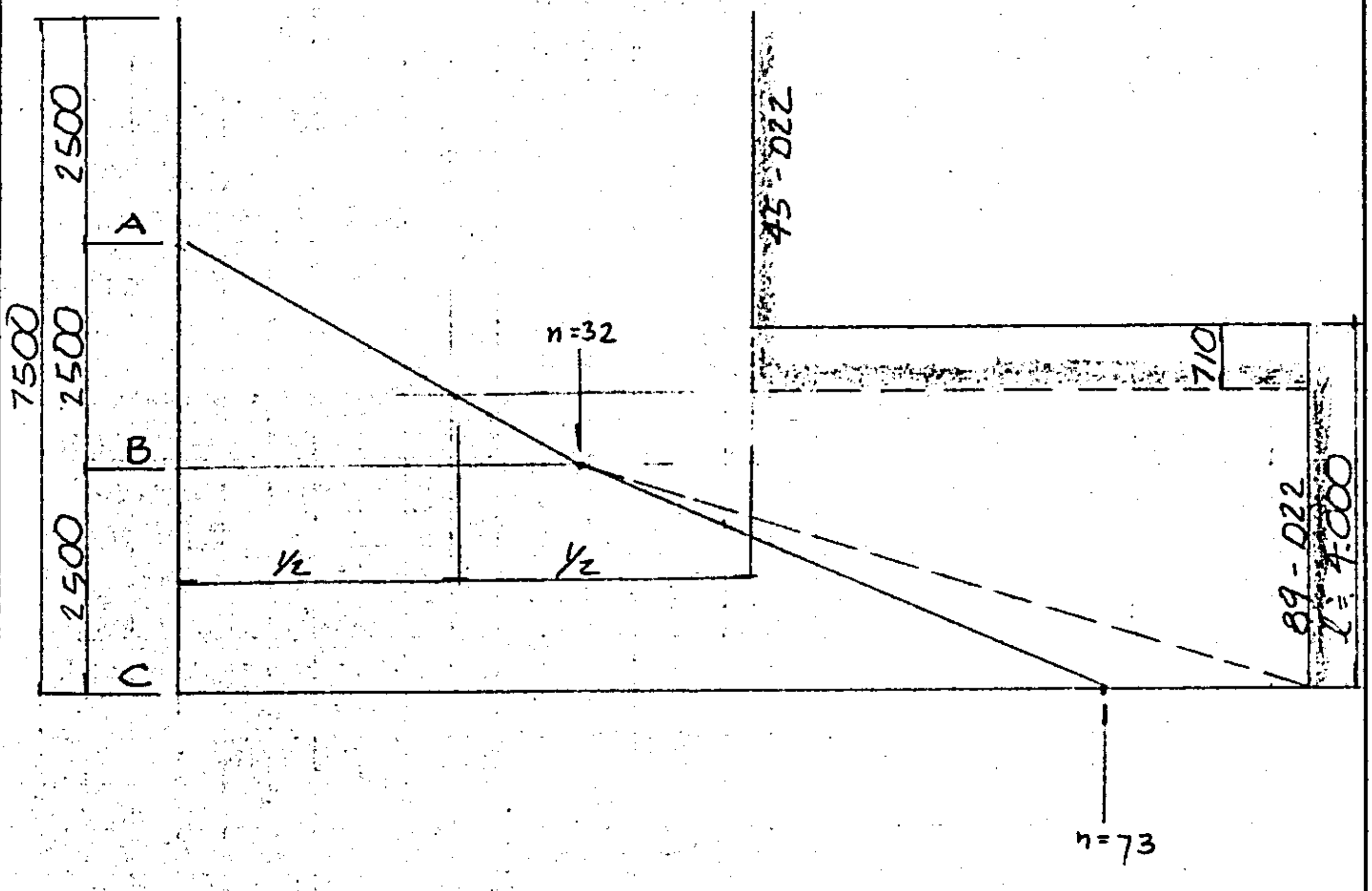
$$M_B = 15.56 + (25.91\times5.735 + 15.33\times5.435 + 2.29\times5.10 + 46.78\times4.65 + 15.10\times4.162 + 2.5\times13.785\times4.00^2\times\tfrac{1}{2})\times0.1 + 54.00\times5.00 + 38.99\times5.20 + 14.96\times6.47 + 12.40\times5.87 = 737.85^{tm}$$

$$N_B = 1047.00 + 2.5\times13.785\times2.50 = 1133.16^{t}$$

4-2. THE LEAST REINFORCING BARS REQUIRED AT THE SECTION A AND B

SECTION	A	B	C
M (tm)	398.63	737.85	1110.31
N (t)	1047.00	1133.16	1219.32
b (cm)	1300	"	"
h (")	100	"	"
d' (")	7.5	"	"
As	D22-1	D22-32	D22-73
σ_c (kg/cm²)	43.9	75.5	85.9
σ_s (")	1002	2656	2648

4-3. MOMENT DIAGRAM



7 DESIGN OF PILE

1. REACTION FORCES ACTING ON PILES

EXAMINATION IS MADE ONLY TO THE EARTHQUAKE LOADING.

	LONGITUDINAL DIRECTION ①	TRANSVERSAL DIRECTION	
		② SINGLE TRACK	③ DOUBLE TRACK
WHOLE HORIZONTAL FORCE, ΣH (t)	181.47	110.52	150.11
HORIZONTAL FORCE PER PILE, H (t)	5.04	3.07	4.17
VALUE OF β (m^{-1})	0.214	〃	〃
MAXIMUM MOMENT PER PILE, M_{max} (tm)	7.58	4.62	6.27
MAXIMUM AXIAL FORCE PER PILE, P_{max} (t)	77.20	56.80	61.92
MINIMUM AXIAL FORCE PER PILE, P_{min} (t)	16.81	15.22	32.08

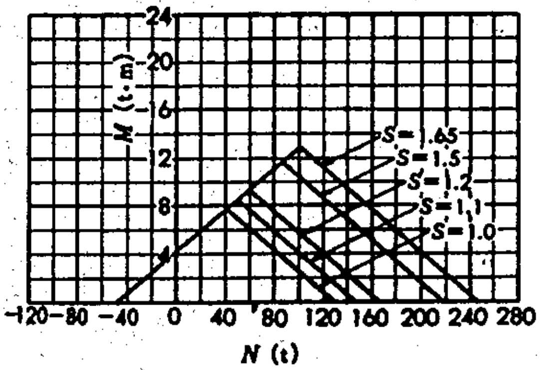
WHERE, $H = \Sigma H / n$, n …. NUMBER OF PILE

$n = 36$

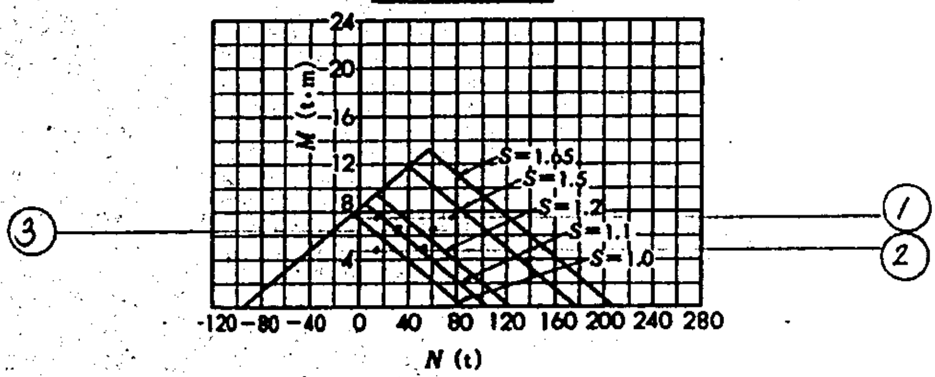
$$M_{max} = \frac{0.322 \cdot H}{\beta}$$

2. EXAMINATION OF SAFETY

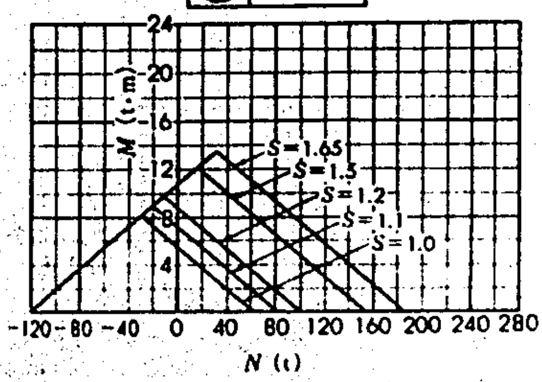
13 D500-A



14 D500-B

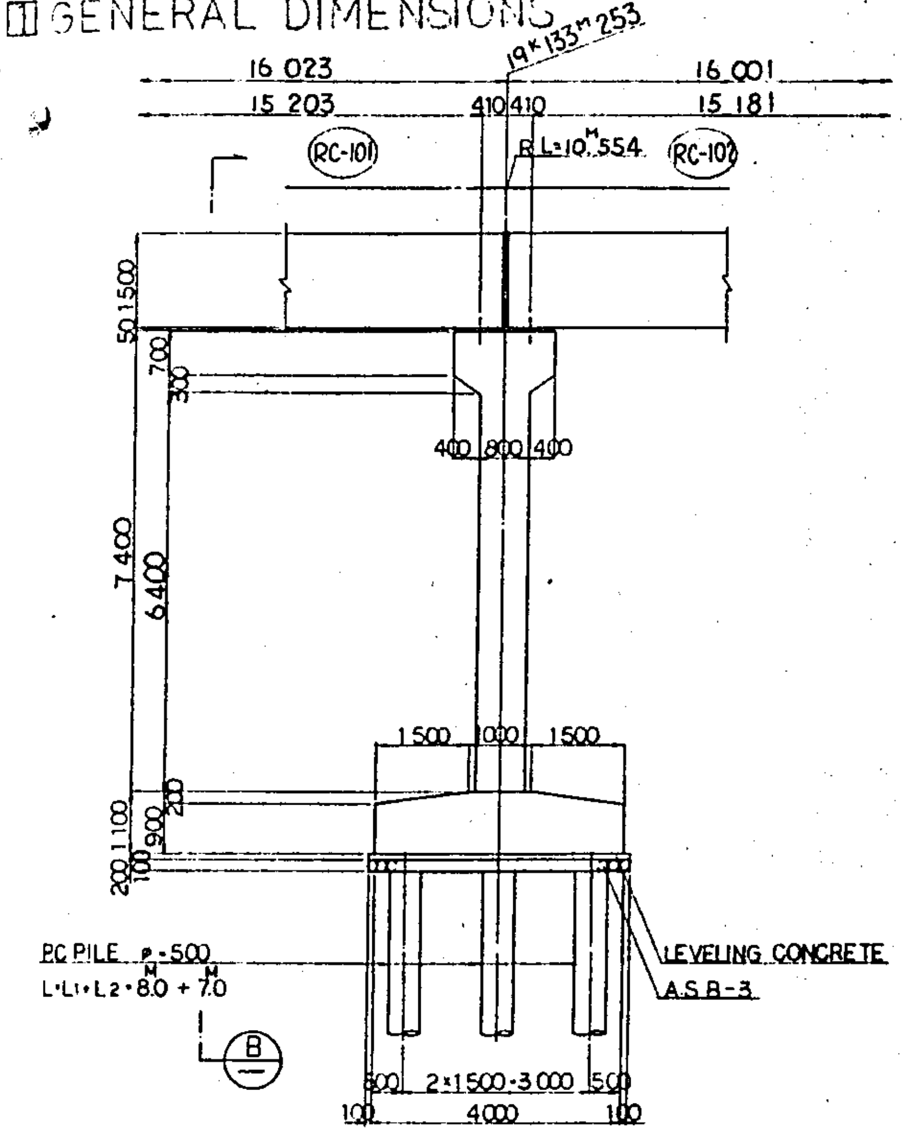


15 D500-C

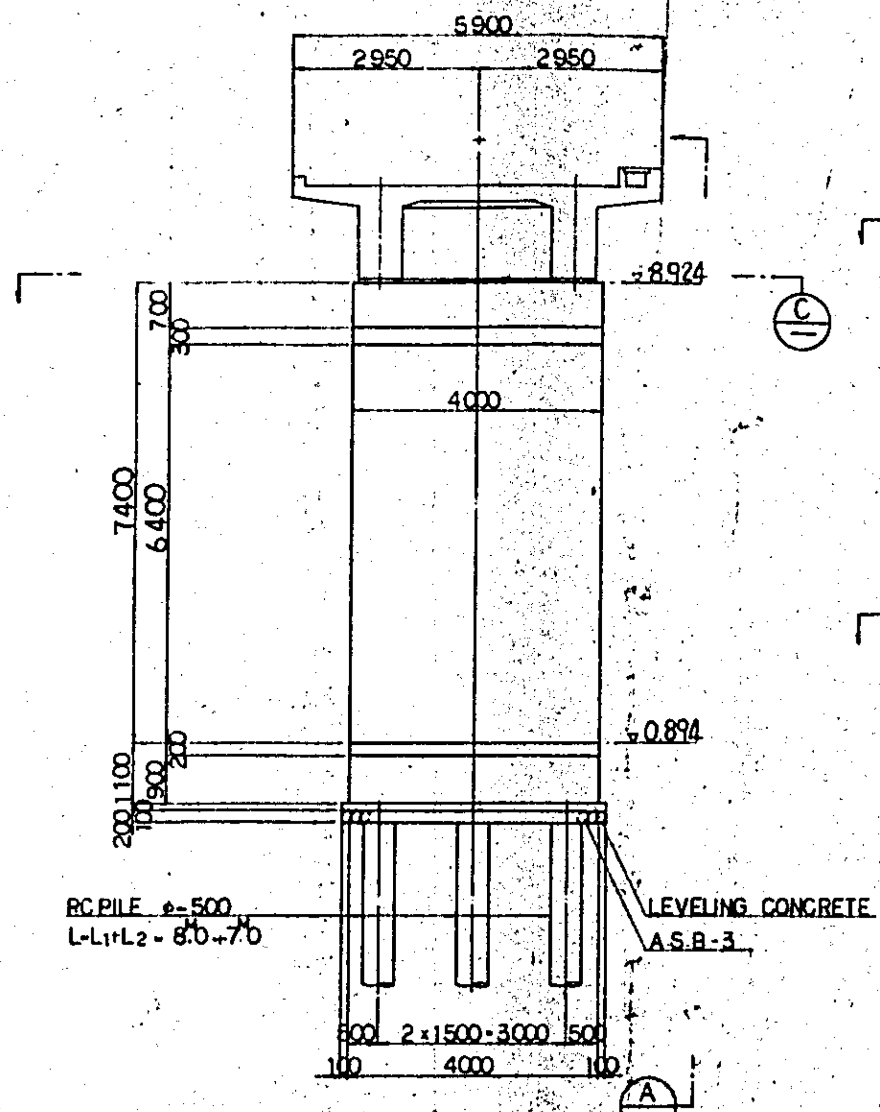


§13. Pier P101

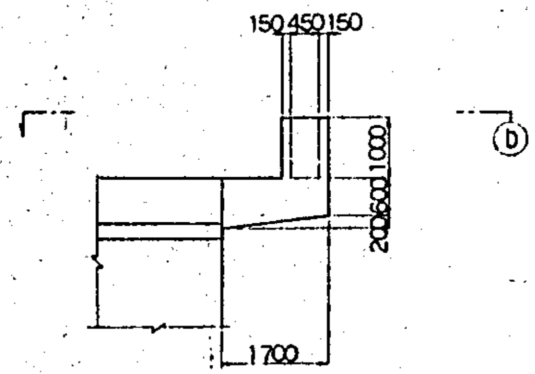
1 GENERAL DIMENSIONS



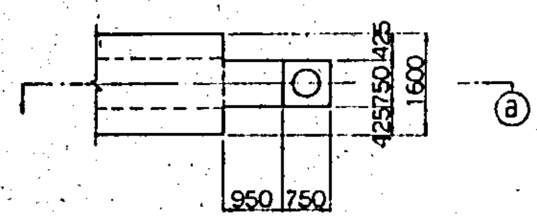
SECTION (A)



SECTION (B)



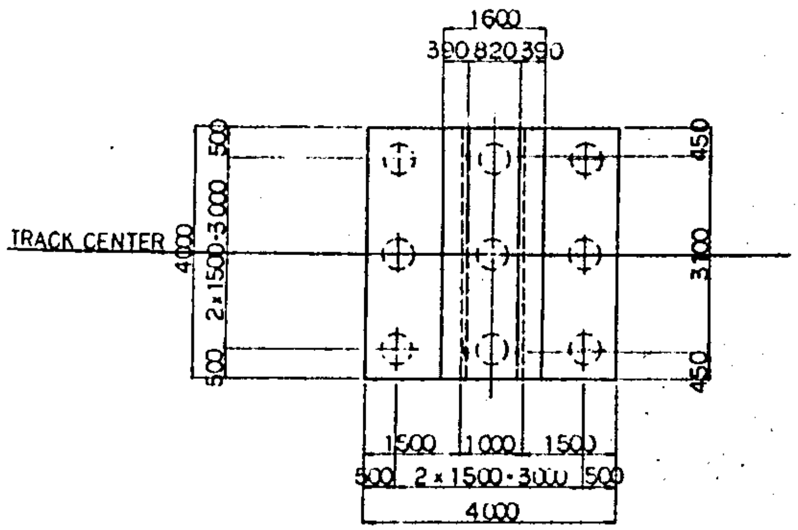
SECTION (a)-(a)



SECTION (b)-(b)

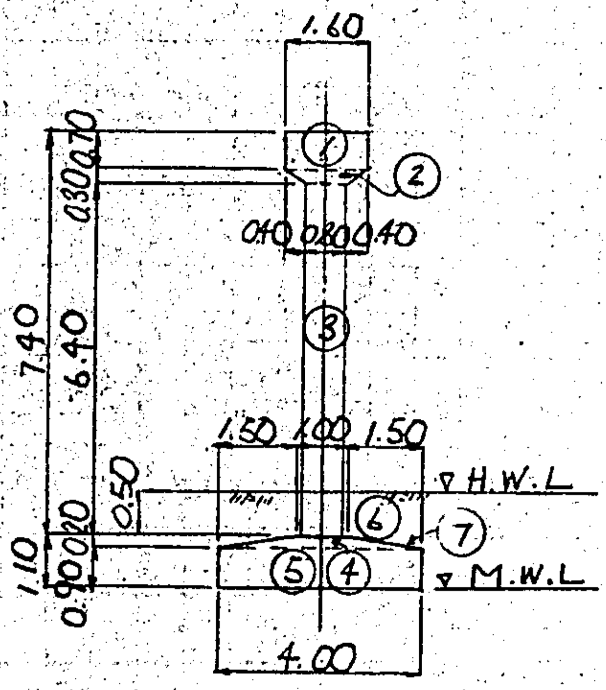
BEAM SEAT OF ELCTRIC POLE

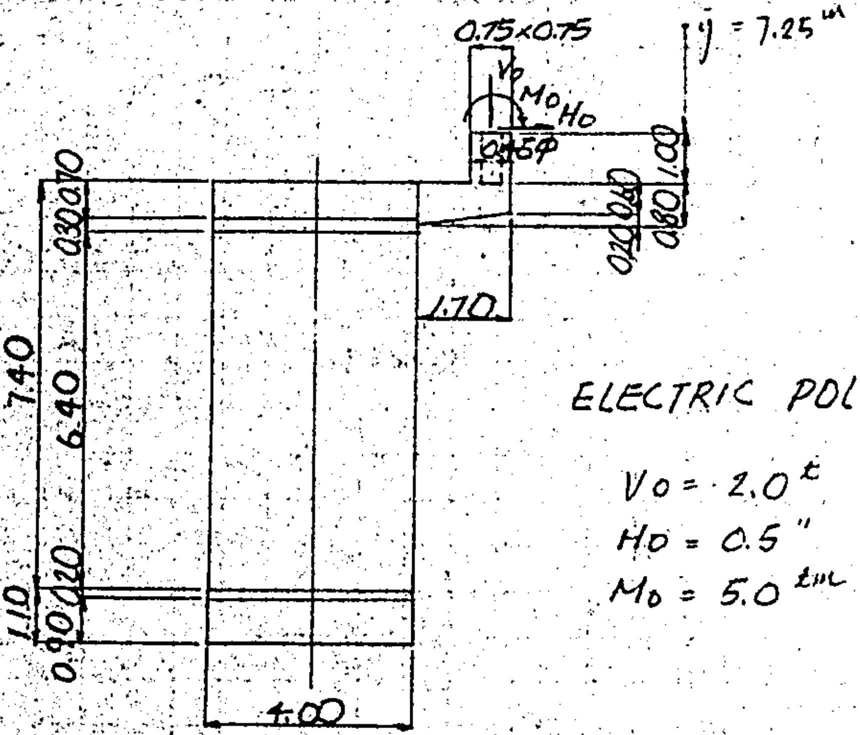
FOR P-104, P-106, P-108, P-111
P-114, P-117 ONLY



SECTION (C)

2 OWN WEIGHT OF SUBSTRUCTURE AND WEIGHT OF EARTH COVER





ELECTRIC POLE LOAD

$V_0 = 2.0^t$

$H_0 = 0.5$ "

$M_0 = 5.0^{tm}$

1. OWN WEIGHT OF SUBSTRUCTURE AND WEIGHT OF EARTH COVER

SUBSTRUCTURE

①	2.5 t/m³ × 1.60 × 0.70 × 4.00	11.20 t
②	2.5 " × ½ × (1.60 + 0.80) × 0.30 × 4.00	3.60 "
③	2.5 " × 0.80 × 4.00 × 6.40	51.20 "
④	2.5 " × ½ × (1.00 + 4.00) × 0.20 × 4.00	5.00 "
⑤	2.5 " × 4.00 × 0.90 × 4.00	36.00 "
	SUB-TOTAL	107.00 t

EARTH COVER

⑥	1.8 t/m³ × (4.00 − 0.80) × 0.50 × 4.00	11.52 t
⑦	1.8 " × ½ × 1.50 × 0.20 × 4.00 × 2	2.16 "
	SUB-TOTAL	13.68 t

TOTAL

107.00 + 13.68 = 120.68 t

2. BUOYANCY

1.0 t/m³ × 4.00 × 1.60 × 4.00 = 25.60 t

3. HORIZONTAL FORCES DUE TO EARTHQUAKE LOAD

	N (t)	y (m)	N×y (t·m)
①	11.20	8.15	91.28
②	3.60	7.667	27.60
③	51.20	4.30	220.16
④	5.00	0.98	4.90
⑤	36.00	0.45	16.20
Σ	107.00		360.14

NOTE N: AXIAL FORCE

y: DISTANCE FROM THE BOTTOM OF FOOTING TO GRAVITY CENTER

4. OVERTURNING MOMENT
AT THE MID-POINT OF THE BOTTOM OF FOOTING

$$y_e = \frac{N \cdot y}{N} = \frac{360.14}{107.00} = 3.366^{m}$$

$$P_E = 107.00 \times 0.1 = 10.70^{t}$$

$$M_o = 10.70 \times 3.366 = 36.02^{tm}$$

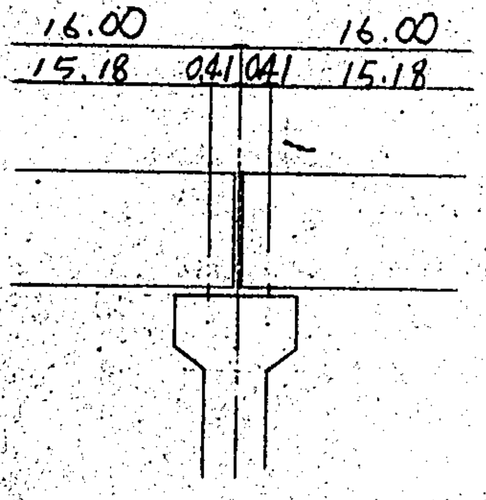
NOTE y_e : DISTANCE FROM THE BOTTOM OF FOOTING TO THE CENTER OF SUBSTRUCTURE.

P_E : HORIZONTAL FORCE DUE TO EARTHQUAKE LOAD

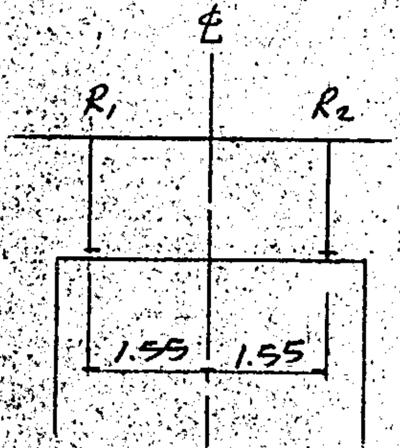
M_o : OVERTURNING MOMENT AT THE MID-POINT OF THE BOTTOM OF FOOTING

3 REACTION FORCES TRANSMITTED FROM SUPERSTRUCTURE

1. REACTION FORCES DUE TO DEAD WEIGHT OF SUPERSTRUCTURE



1-1. REACTION FORCES ACTING AT BEARINGS



$R_1 = 62.00^t$

$R_2 = 62.00$ "

TOTAL = 124.00 "

1-2. HORIZONTAL FORCES DUE TO EARTHQUAKE ACTING ON DEAD WEIGHT OF SUPERSTRUCTURE

a) LONGITUDINAL DIRECTION

L_1 - SPAN SIDE

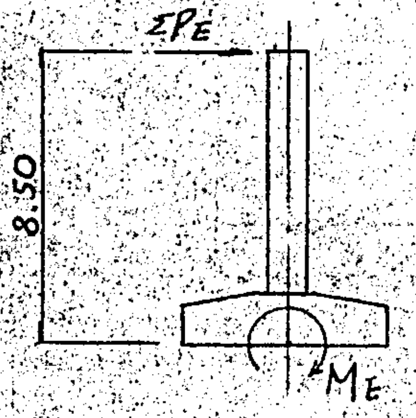
$$P_{E1} = 124.00 \times 2 \times 0.1 - \frac{1}{2} \times 124.00 \times 0.1$$
$$= 18.60^t$$

L_2 - SPAN SIDE

$$P_{E2} = 124.00 \times 0.1 = 12.40^t$$

$$\therefore \Sigma P_E = 18.60 + 12.40 = 31.00^t$$

OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING



$$M_E = 31.00 \times 8.50 = 263.50^{tm}$$

b) TRANSVERSAL DIRECTION

L_1 - SPAN SIDE

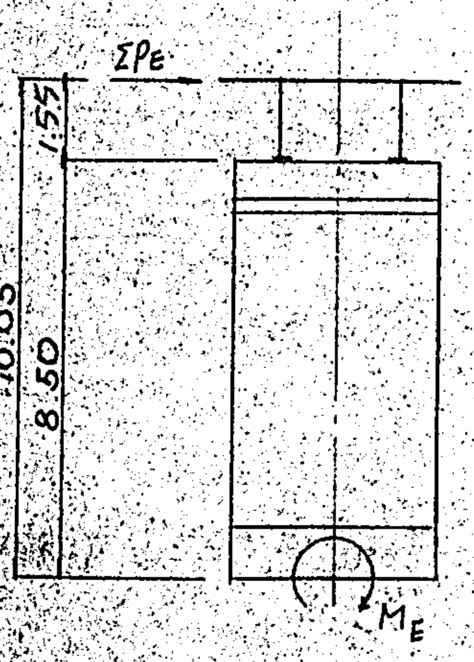
$P_{E_1} = 124.00 \times 0.1 = 12.40^t$

L_2 - SPAN SIDE

$P_{E_2} = 124.00 \times 0.1 = 12.40^t$

$\Sigma P_E = 12.40 + 12.40 = 24.80^t$

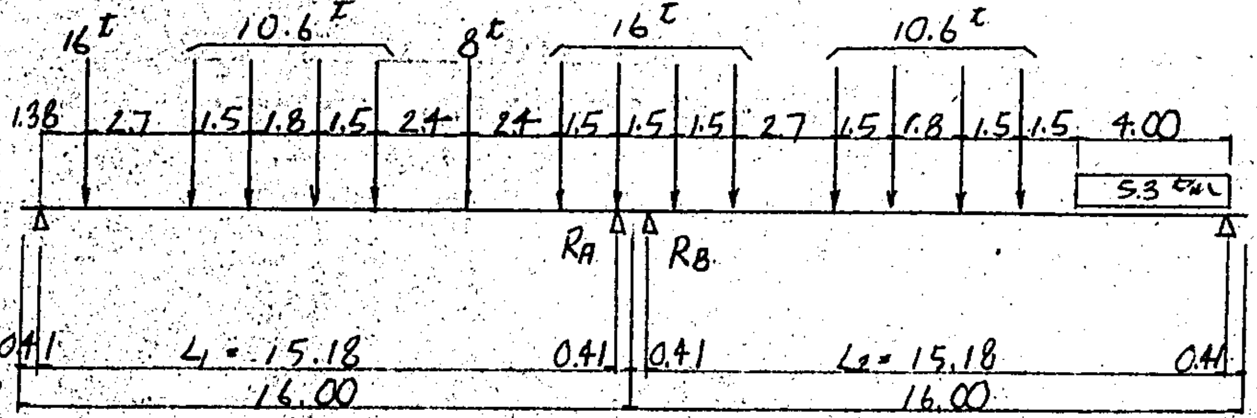
OVERTURNING MOMENT AT THE MID-POINT OF THE BOTTOM OF FOOTING



$M_E = 24.80 \times 10.05 = 249.24^{tm}$

2. REACTION FORCES DUE TO LIVE LOAD PLUS IMPACT

EQUIVALENT TO KS-16 LOADING
CALCULATION IS CARRIED OUT UNDER THE LOADING CONDITION OF PRODUCING MAXIMUM REACTION FORCE ON PIER.



$$R_A = \frac{1}{15.18} \times (16 \times 1.38 + 10.6 \times 4 \times 6.48 + 8 \times 11.28 + 16 \times 2 \times 14.43) = 55.92^t$$

$$R_B = \frac{1}{15.18} \times (16 \times 2 \times 13.75 + 10.6 \times 4 \times 7.90 + 5.3 \times 4.00^2 \times \tfrac{1}{2}) = 53.84''$$

$$\text{TOTAL} = 109.76^t$$

IMPACT COEFFICIENT

L_1 - SPAN SIDE --- $i = 0.399$

L_2 - SPAN SIDE --- $i = 0.399$

LIVE LOAD PLUS IMPACT

$R(A+L) = 55.92 \times 1.399 = 78.23^{t}$

$R(B+L) = 53.84 \times 1.399 = 75.32^{"}$

TOTAL $= 153.55^{t}$

2-1. SECTIONAL MOMENT ACTING AT THE CENTER OF PIER SHAFT

TRANSVERSAL DIRECTION

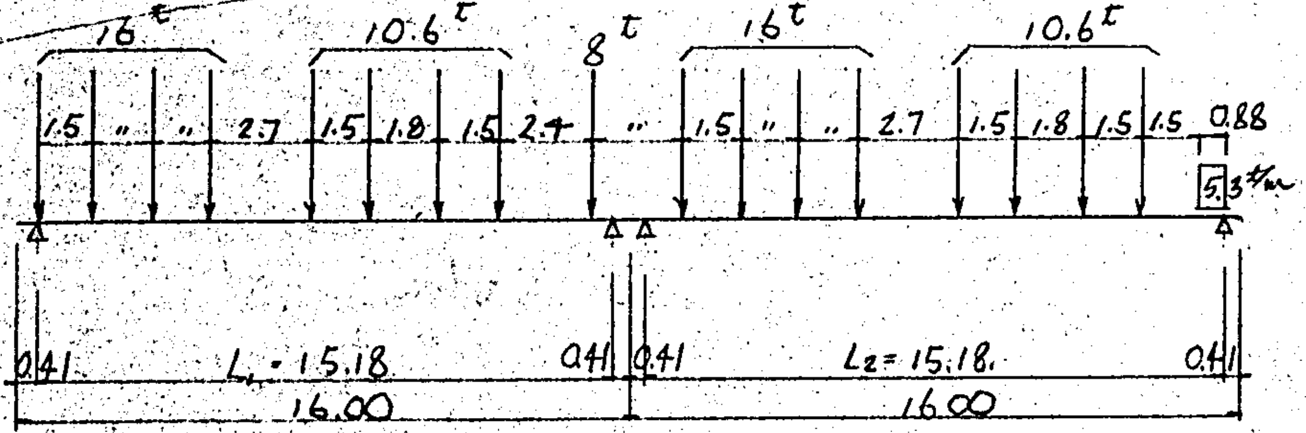
$M_1 = 0$

LONGITUDINAL DIRECTION

$M_2 = (78.23 - 75.32) \times 0.41 = 1.19^{tm}$

3. BRAKE LOAD AND TRACTION LOAD

UNDER THE LOADING CONDITION AS SHOWN BELOW



3-1. BRAKE LOAD

15% OF KS LOADING

L1 - SPAN SIDE

$$P_{B-1} = (16 \times 4 + 10.6 \times 4 + 8) \times 0.15 \times 1/2 = 8.58^t$$

L2 - SPAN SIDE

$$P_{B-2} = (16 \times 4 + 10.6 \times 4 + 5.3 \times 0.88) \times 0.15 \times 1/2 = 8.33^t$$

$$\therefore \Sigma P_B = 8.58 + 8.33 = 16.91^t$$

3-2. TRACTION LOAD

25% OF MOVING WHEEL AXLE LOAD OF KS LOADING

L1 - SPAN SIDE

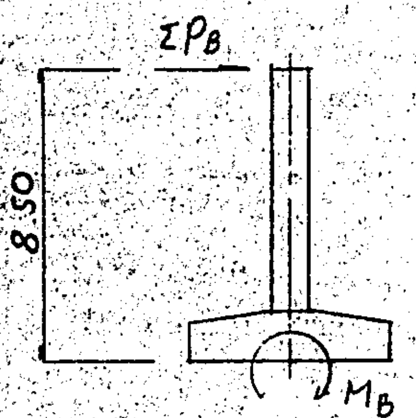
$P_{T-1} = 16 \times 4 \times 0.25 \times 1/2 = 8.00^t$

L2 - SPAN SIDE

$P_{T-2} = 16 \times 4 \times 0.25 \times 1/2 = 8.00^t$

$\therefore \Sigma P_T = 8.00 + 8.00 = 16.00^t < \Sigma P_B = 16.91^t$

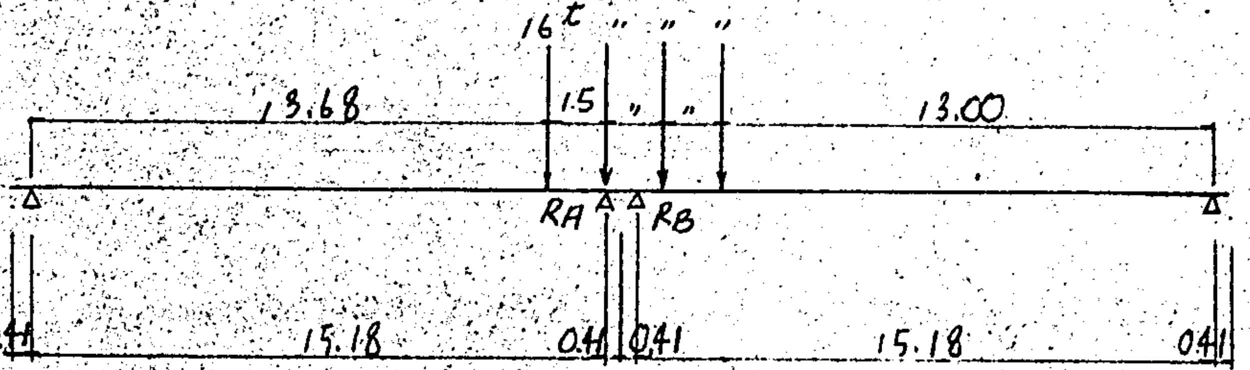
3-3. OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING



$M_B = 16.91 \times 8.50 = 143.74^{tm}$

4. TRAIN LATERAL LOAD

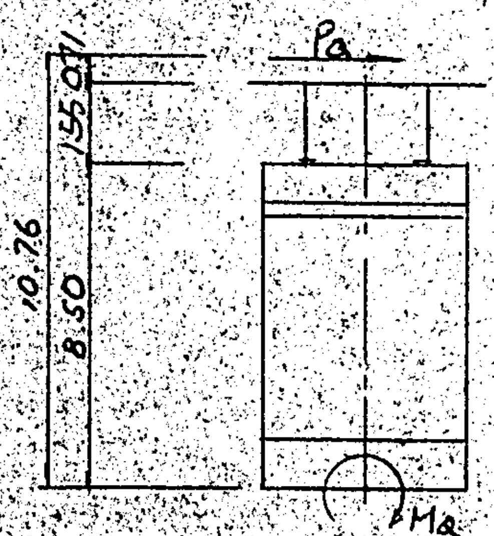
TRANSVERSAL FORCE DUE TO LIVE (TRAIN) LOAD IS ASSUMED TO BE EQUIVALENT TO THE SERIES OF CONCENTRATED LOADS CONSISTED OF 15% OF MOVING WHEEL AXLE LOAD ACTING TRANSVERSALLY AT THE TRACK LEVEL.
CALCULATION IS MADE UNDER THE SINGLE-TRACK LOADING



$$R_A + R_B = \frac{16}{15.18} \times (13.68 + 15.18 + 14.50 + 13.00) = 59.40^t$$

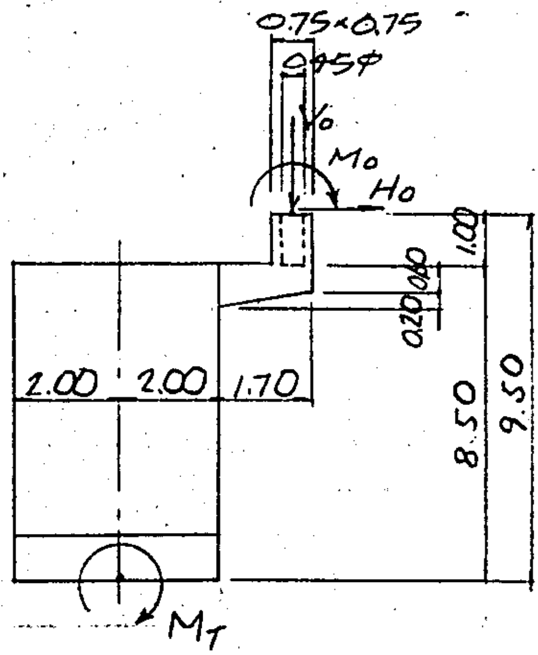
$$P_Q = 59.40 \times 0.15 = 8.91^t$$

4-1 OVERTURNING MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING.



$$M_Q = 8.91 \times 10.76 = 95.87^{tm}$$

5. ELECTRIC POLE LOAD



THE REACTION FORCES DUE TO THE ELECTRIC POLE.

$H_o = 0.5^t \quad V_o = 2.0^t \quad M_o = 5.0^{tm}$

THE DEAD LOAD OF THE BASEMENT OF ELECTRIC POLE AND THE CANTILEVER BEAM

$2.5^{t/m^3} \times (0.75^2 - \frac{1}{4} \times \pi \times 0.45^2) \times 1.00 = 1.01^t$

$2.5'' \times 1.70 \times 0.60 \times 0.75 = 1.91''$

$2.5'' \times \frac{1}{2} \times 1.70 \times 0.20 \times 0.75 = 0.32$

3.24^t

$y = \dfrac{1.01 \times 9.00 + 1.91 \times 8.20 + 0.32 \times 7.833}{3.24}$

$= 8.413^m$

1) ELECTRIC POLE LOAD IN ORDINARY CASE

a) LONITUDINAL DIRECTION

$N = 2.0 + 3.24 \qquad = 5.24^{t}$

$H = \qquad = 0.50''$

$M_L = 5.00 + 0.50 \times 9.50 = 9.75^{tm}$

b) TRANSVERSAL DIRECTION

$N = 5.24^{t}$

$H = 0.50''$

$M_T = 5.00 + 0.50 \times 9.50 + 2.00 \times 3.325$

$+ 1.01 \times 3.325 + 1.91 \times 2.65 + 0.32 \times 2.567$

$= 26.02^{tm}$

2) ELECTRIC POLE LOAD IN SEISMIC CASE

a) LONGITUDINAL DIRECTION

$N = 5.24^{t}$

$H = 0.50 + 3.24 \times 0.1 = 0.82^{t}$

$M_L = 9.75 + 2.0 \times 0.1 \times 16.75 + 3.24 \times 0.1 \times 8.413 = 15.83^{tm}$

b) TRANSVERSAL DIRECTION

$N = 5.24^{t}$

$H = 0.82^{t}$

$M_T = 26.02 + 2.0 \times 0.1 \times 16.75 + 3.24 \times 0.1 \times 8.413 = 32.10^{tm}$

$\boxed{4}$ CALCULATION ON STABILITY

1. COMBINATION OF LOADS

1-1 TRANSVERSAL DIRECTION

No. 1. DEAD LOAD + ELECTRIC POLE LOAD (M.W.L)

No. 2. DEAD LOAD + (TRAIN + IMPACT) LOAD + ELECTRIC POLE LOAD (M.W.L)

No. 3. ———— " ———— (H.W.L)

No. 4. DEAD LOAD + (TRAIN + IMPACT) LOAD + ELECTRIC POLE LOAD + TRAIN LATERAL LOAD (M.W.L)

No. 5 ———— " ———— (H.W.L)

No. 6 DEAD LOAD + ELECTRIC POLE LOAD + EARTHQUAKE LOAD (M.W.L)

1-2. LONGITUDINAL DIRECTION

No.1. DEAD LOAD + ELECTRIC POLE LOAD (M.W.L)

No.2. DEAD LOAD + (TRAIN + IMPACT) LOAD
+ ELECTRIC POLE LOAD (M.W.L)

No.3 ——————— " ——————— (H.W.L)

No.4 DEAD LOAD + (TRAIN + IMPACT) LOAD
+ ELECTRIC POLE LOAD
+ BRAKE LOAD OR TRACTION LOAD (M.W.L)

No.5 ——————— " ——————— (H.W.L)

No.6 DEAD LOAD + ELECTRIC POLE LOAD
+ EARTHQUAKE LOAD (M.W.L)

2. TABLE OF FORCE AND MOMENT ACTING AT THE MID-POINT OF THE BOTTOM OF FOOTING

2-1. TRANSVERSAL DIRECTION

SINGLE-TRACK LOADING

	AXIAL FORCE	MOMENT	HORIZONTAL FORCE
No1	120.68 + 124.00×2 +5.24 = 373.92	26.02	0.50
No2	373.92 + 153.55 = 527.47	26.02	0.50
No3	527.47 − 25.60 = 501.87	26.02	0.50
No4	527.47	26.02 + 95.87 = 121.89	0.50 + 8.91 = 9.41
No5	501.87	121.89	9.41
No6	373.92	36.02 + 32.10 + 249.24 = 317.36	10.70 + 0.82 + 24.80 = 36.32

2-2. LONGITUDINAL DIRECTION

	AXIAL FORCE	MOMENT	HORIZONTAL FORCE
No 1	373.92	9.75	0.50
No 2	527.47	9.95 + 1.19 = 10.94	0.50
No 3	501.87	10.94	0.50
No 4	527.47	10.94 + 143.74 = 154.68	0.50 + 16.91 = 17.41
No 5	501.87	154.68	17.41
No 6	373.92	36.02 + 15.83 + 263.50 = 315.35	10.70 + 0.82 + 31.00 = 42.52

3. REACTION FORCE ACTING ON PILES

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

WHERE: P REACTION FORCE

n NUMBER OF PILES

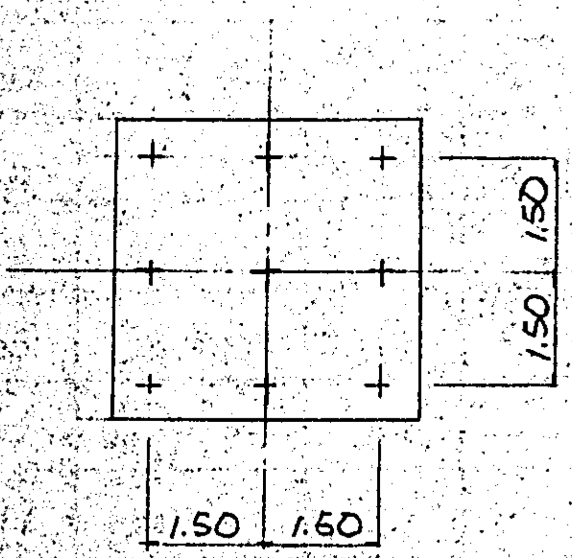
N AXIAL FORCE ACTING AT THE CENTER OF THE BOTTOM OF FOOTING

M MOMENT ACTING AT THE CENTER OF THE BOTTOM OF FOOTING

I MOMENT OF INERTIA OF THE GROUP OF PILES WITH RESPECT TO THE GEOMETRICAL CENTER

x DISTANCE FROM THE GEOMETRICAL CENTER OF THE GROUP OF PILES TO THE CENTER OF THE PILE TO BE CALCULATED

3-1. ARRANGEMENT OF PILES



3-2 REACTION FORCE ACTING ON PILES DUE TO TRANSVERSAL LOAD

	N (t)	M (t·m)	$\frac{N}{n}$ (t)	$\frac{M}{I}$ (t/m)	P_{max} (t)	P_{min} (t)
No1	373.92	26.02	41.55	1.93	44.45	38.66
No2	527.47	26.02	58.61	1.93	61.51	55.72
No3	501.87	26.02	55.76	1.93	58.66	52.87
No4	527.47	121.89	58.61	9.03	72.16	45.07
No5	501.87	121.89	55.76	9.03	69.31	42.22
No6	373.92	317.36	41.55	23.51	76.82	6.29

$$I = 1.50^2 \times 3 \times 2 = 13.50^{m^2}$$

3-3. REACTION FORCE ACTING ON PILES DUE TO LONGITUDINAL LOAD

	N (t)	M (t·m)	$\frac{N}{n}$ (t)	$\frac{M}{I}$ (t/m)	P_{max} (t)	P_{min} (t)
No1	373.92	9.75	41.55	0.72	42.63	40.47
No2	527.47	10.94	58.61	0.81	59.83	57.40
No3	501.87	10.94	55.76	0.81	56.98	54.55
No4	527.47	154.68	58.61	11.46	75.80	41.42
No5	501.87	154.68	55.76	11.46	72.95	38.57
No6	373.92	315.35	41.55	23.36	76.59	6.51

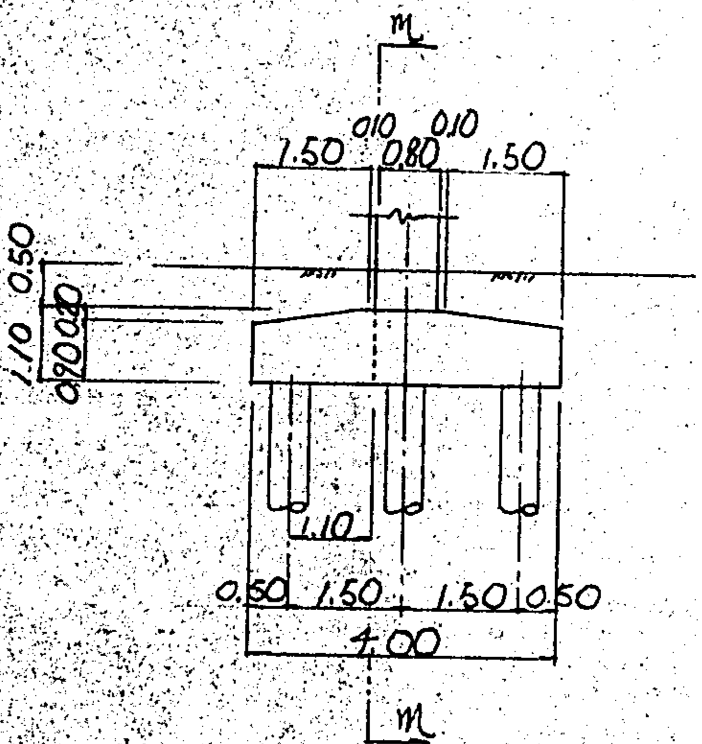
$$I = 1.50^2 \times 3 \times 2 = 13.50^{m^2}$$

5 DESIGN OF FOOTING

CALCULATION IS MADE ONLY THE CASE OF LONGITUDINAL LOADING, BECAUSE NO STRESS OCCURS UNDER THE TRANSVERSAL LOADING DUE TO REACTION FORCES ON PILES WITHIN THE WALL SHAFT.

1. EXAMINATION OF SAFETY AGAINST BENDING MOMENT

1-1. BENDING MOMENT DUE TO OWN WEIGHT OF FOOTING AND WEIGHT OF EARTH COVER.



WHERE: m SECTION WHERE MOMENT IS CALCULATED

M_m BENDING MOMENT AT SECTION m

BENDING MOMENT AT SECTION m

i) THE CASE NO BUOYANCY IS CONSIDERED

$M_m = (2.5^{t/m^3} \times 1.10 + 1.8^{t/m^3} \times 0.50) \times 1.60^2 \times 1/2$

$- 1/2 \times 1.50 \times 0.20 \times 1.10 \times (2.5 - 1.8) = 4.56^{tm/m}$

ii) THE CASE BUOYANCY IS CONSIDERED

$M_m = 4.56 - 1.00^{t/m^3} \times \overline{1.60}^2 \times 1/2 \times 1.60 = 2.51^{tm/m}$

1-2, STRESS DUE TO REACTION FORCE ON PILES

BENDING MOMENT AT SECTION m

No.1 $M_m = 42.63 \times 1.20 \times \frac{3}{4.00} = 38.37$ tm/m

No.2 $M_m = 59.83 \times$ " $\times$ " $= 53.85$ "

No.3 $M_m = 56.98 \times$ " $\times$ " $= 51.28$ "

No.4 $M_m = 75.80 \times$ " $\times$ " $= 68.22$ "

No.5 $M_m = 72.95 \times$ " $\times$ " $= 65.66$ "

No.6 $M_m = 76.59 \times$ " $\times$ " $= 68.93$ "

1-3. SUMMARY OF BENDING MOMENT ON FOOTING

No1 : $M_u = (38.37 - 4.56) \times 1.8/1.4 = 43.47$ tm/m

No2 : $M_u = (53.85 - 4.56) \times 1.00 = 49.29$ "

No3 : $M_u = (51.28 - 2.51) \times 1.00 = 48.77$ "

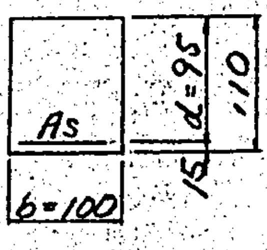
No4 : $M_u = (68.22 - 4.56) \times 1/1.15 = 55.36$ "

No5 : $M_u = (65.66 - 2.51) \times 1/1.15 = 54.91$ "

No6 : $M_u = (68.93 - 4.56) \times 1/1.50 = 42.91$ "

1-4. EXAMINATION OF SAFETY

$$M_u = 55.36 \text{ tm/m}$$



$$A_s = 6.67 - D29 = 42.85$$

$$p = \frac{A_s}{b \cdot d} = \frac{42.85}{100 \times 95} = 0.00451$$

$$\frac{1}{L_c} = 7.28 \qquad \frac{1}{L_s} = 247$$

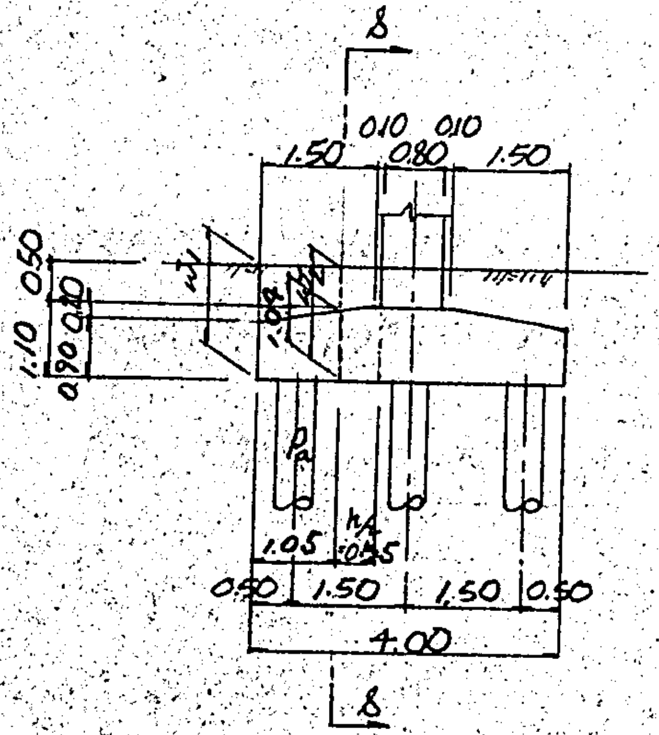
$$\sigma_c = \frac{M}{b \cdot d^2} \times \frac{1}{L_c} = \frac{55.36 \times 10^5}{100 \times 95^2} \times 7.28 = 44.7 \text{ kg/cm}^2 < 80 \text{ kg/cm}^2$$

$$\sigma_s = \frac{M}{b \cdot d^2} \times \frac{1}{L_s} = \text{———}" \text{———} \times 247 = 1515 \text{ "} < 1800 \text{ "}$$

2. EXAMINATION OF SAFETY AGAINST SHEARING FORCE

EXAMINATION ON THE COMBINATION No.6

2-1. SHEARING FORCE DUE TO OWN WEIGHT OF FOOTING AND WEIGHT OF EARTH COVER



WHERE: s SECTION WHERE SHEARING FORCE IS CALCULATED

S_s SHEARING FORCE AT SECTION s

SHEARING FORCE AT SECTION s.

$$w_1 = 2.5^{t/m^3} \times 0.90 + 1.8^{t/m^3} \times 0.70 = 3.51^{t/m}$$

$$w_2 = (2.5' \times 1.04 + 1.8' \times 0.56) \times \frac{0.50}{2 \times (1.04 - 0.15)} = 1.01'$$

$$S_s = \frac{1}{2} \times (3.51 + 1.01) \times 1.05 = 2.37^t$$

2-2. STRESS DUE TO REACTION FORCE ON PILES

SHEARING FORCE AT SECTION s

$$P_a = 76.59 \times \frac{1.10}{2 \times (1.04 - 0.15)} = 47.33^t$$

$$S_s = 47.33 \times \frac{2}{4.00} = 35.50^t$$

2-3. SUMMARY OF SHEARING FORCES ON FOOTING

$$\Sigma S_s = 35.50 - 2.37 = 33.13^t$$

2-4. EXAMINATION OF SAFETY,

$$\tau = \frac{33.13 \times 10^3}{100 \times (104.0 - 15)} = 3.72^{kg/cm^2} \doteqdot 3.7^{kg/cm^2}$$

6 DESIGN OF SUBSTRUCTURE.

EXAMINATION IS MADE ONLY THE CASE OF LONGITUDINAL LOADING.

1. COMBINATION OF LOADS

No.6 DEAD LOAD + EARTHQUAKE LOAD

2. BENDING MOMENT AND AXIAL FORCE ACTING AT THE BOTTOM OF WALL

THE FOLLOWING FIGURES ARE REFERRED TO THE CALCULATION OF THE REACTION FORCES TRANSMITTED FROM THE SUPERSTRUCTURE.

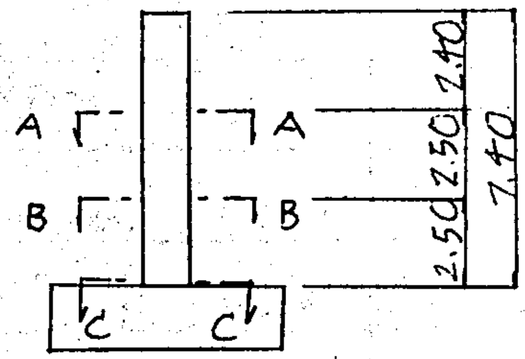
$$M = (11.20 \times 7.05 + 3.60 \times 6.567 + 51.20 \times 3.20) \times 0.1$$
$$+ 31.00 \times 7.40 + 3.24 \times 0.1 \times 7.313 + 2.00 \times 0.1 \times 15.65$$
$$+ 0.50 \times 8.10 + 5.00$$
$$= 270.59^{tm}$$

$$N = 11.20 + 3.60 + 51.20 + 124.00 \times 2$$
$$+ 2.00 + 3.24$$
$$= 319.24^{t}$$

3 EXAMINATION OF SAFETY

M (tm)	270.59
N (t)	319.24
S (t)	
b (cm)	400
h (cm)	80
d (cm)	
d' (cm)	8
As (cm²)	25-D25 = 126.67
p	0.00395
As' (cm²)	= As
p'	= p
e= M/N (cm)	84.7
e= M/N+u (cm)	
e= M/N−u (cm)	
e/h	1.059
d/e	
d'/h	0.100
d'/d	
Ne/bd² (kg/cm²)	9.976
k	0.340
c	0.114
j	
1/Lc	
1/Ls	
β= σs/σc	
σc (kg/cm²)	86.9
σs (kg/cm²)	2140
τ (kg/cm²)	
σsa (kg/cm²)	120
σca (kg/cm²)	2700
τa (kg/cm²)	

4. DEDUCTION OF REINFORCING BARS



4-1 BENDING MOMENT AND AXIAL FORCE ACTING AT THE SECTION A AND B.

$$M_A = (11.20 \times 2.05 + 3.60 \times 1.567 + 2.5 \times 0.80 \times 4.00 \times 1.40^2 \times \tfrac{1}{2}) \times 0.1 + 31.00 \times 2.40 + 3.24 \times 0.1 \times 2.313 + 2.00 \times 0.1 \times 10.65 + 0.50 \times 3.10 + 5.00 = 87.47^{tm}$$

$$N_A = 11.20 + 3.60 + 2.5 \times 0.80 \times 4.00 \times 1.40 + 124.00 \times 2 + 2.00 + 3.24 = 279.24^{t}$$

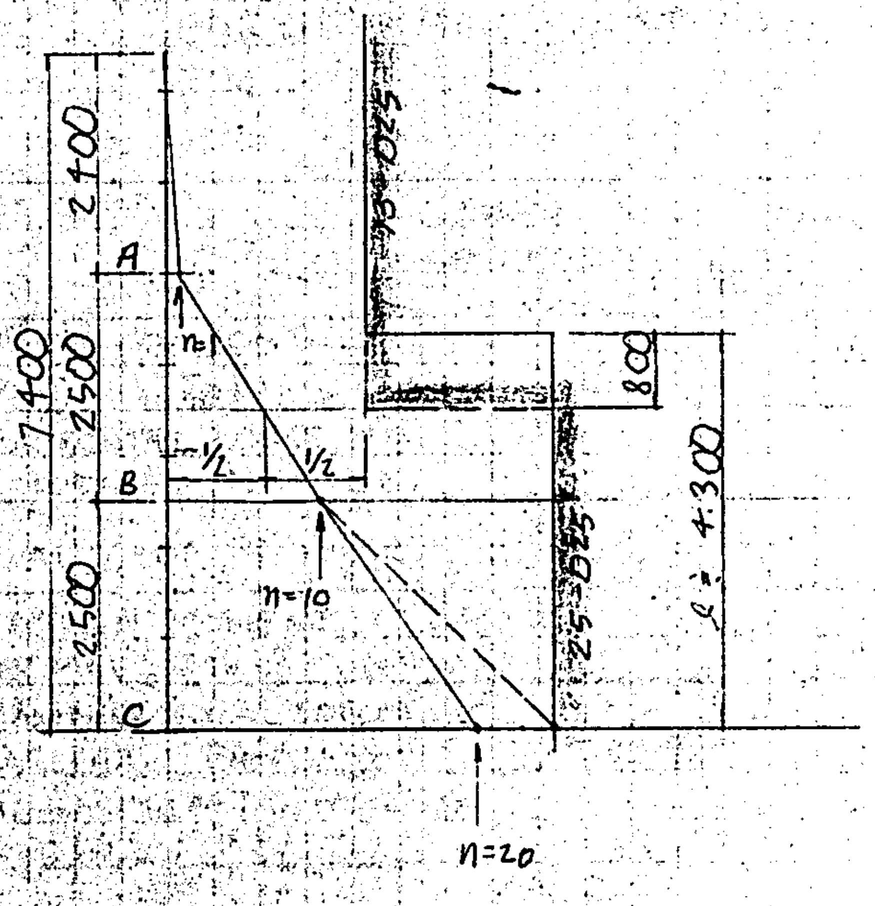
$$M_B = (11.20 \times 4.55 + 3.60 \times 4.067 + 2.5 \times 0.80 \times 4.00 \times 3.90^2 \times \tfrac{1}{2}) \times 0.1 + 31.00 \times 4.90 + 3.24 \times 0.10 \times 4.813 + 2.00 \times 0.1 \times 13.15 + 0.50 \times 5.60 + 5.00 = 176.53^{tm}$$

$$N_B = 279.24 + 2.5 \times 0.80 \times 4.00 \times 2.50 = 299.24^{t}$$

4-2. THE LEAST REINFORCING BARS REQUIRED AT THE SECTION A AND B

SECTION	A	B	C
M (tm)	87.47	176.53	270.59
N (t)	279.24	299.24	319.24
b (cm)	400	"	"
h (")	80	"	"
d' (")	8	"	"
As	D25-1	D25-10	D25-20
σ_c (kg/cm²)	47.6	81.0	95.6
σ_s (")	1027	2478	2632

4-3. MOMENT DIAGRAM



7 DESIGN OF PILE

1. REACTION FORCES ACTING ON PILES

EXAMINATION IS MADE ONLY TO THE EARTHQUAKE LOADING.

	LONGITUDINAL DIRECTION ①	TRANSVERSAL DIRECTION	
		② SINGLE TRACK	③ DOUBLE TRACK
WHOLE HORIZONTAL FORCE, ΣH (t)	42.52	36.32	
HORIZONTAL FORCE PER PILE, H (t)	4.72	4.04	
VALUE OF β (m^{-1})	0.214	〃	
MAXIMUM MOMENT PER PILE, M_{max} (tm)	7.10	6.08	
MAXIMUM AXIAL FORCE PER PILE, P_{max} (t)	76.59	76.82	
MINIMUM AXIAL FORCE PER PILE, P_{min} (t)	6.51	6.29	

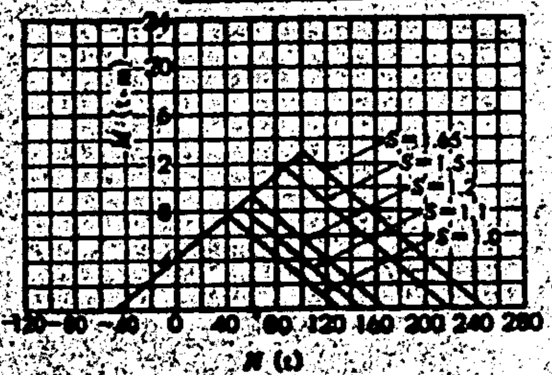
WHERE, $H = \Sigma H / n$, n .... NUMBER OF PILE

$n = 9$

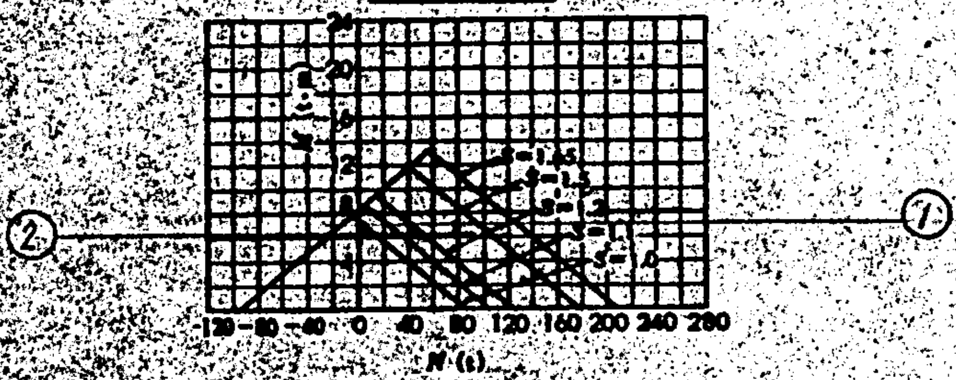
$$M_{max} = \frac{0.322 \cdot H}{\beta}$$

2. EXAMINATION OF SAFETY

13 D500-A

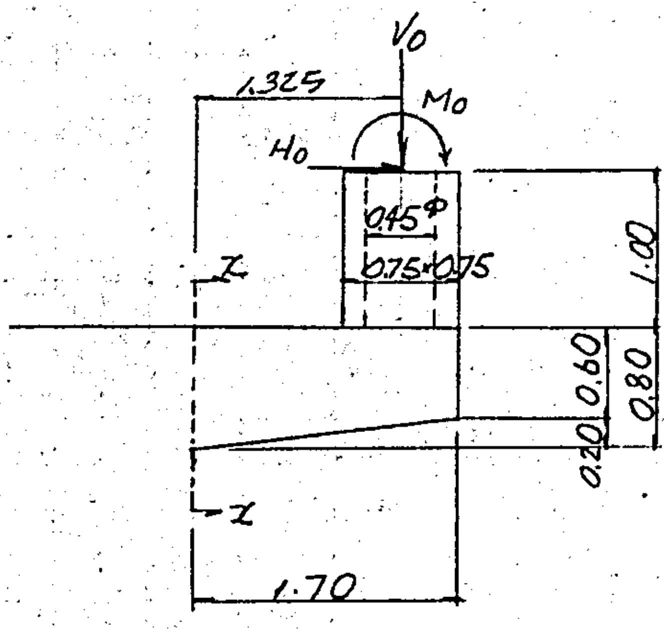


14 D500-B



8. DESIGN OF CANTILEVER BEAM FOR ELECTRIC POLE.

1. EXAMINATION OF SAFETY AGAINST BENDING MOMENT



1-1. ELECTRIC POLE LOAD

$V_o = 2.00^t$

$H_o = 0.50$ "

$M_o = 5.00^{tm}$

1-2. THE DEAD LOAD OF THE BASEMENT OF ELECTRIC POLE AND THE CANTILEVER BEAM

$W = 1.01 + 1.91 + 0.32$

$= 3.24^t$

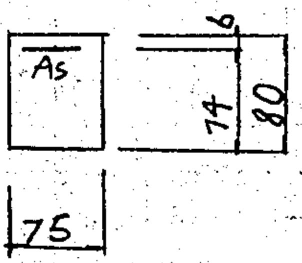
1-3. BENDING MOMENT AT SECTION X

$$M_x = \overset{M_0}{5.00} + \overset{V_0}{2.00} \times 1.325 + \overset{H_0}{0.50} \times (1.00 + 0.40)$$

$$+ 1.01 \times 1.325 + 1.91 \times 1.70 \times \tfrac{1}{2} + 0.32 \times 1.70 \times \tfrac{1}{3}$$

$$= 11.49^{\,tm}$$

1-4. EXAMINATION OF SAFETY



$$A_s = 5\text{-}D22 = 19.36^{\,cm^2}$$

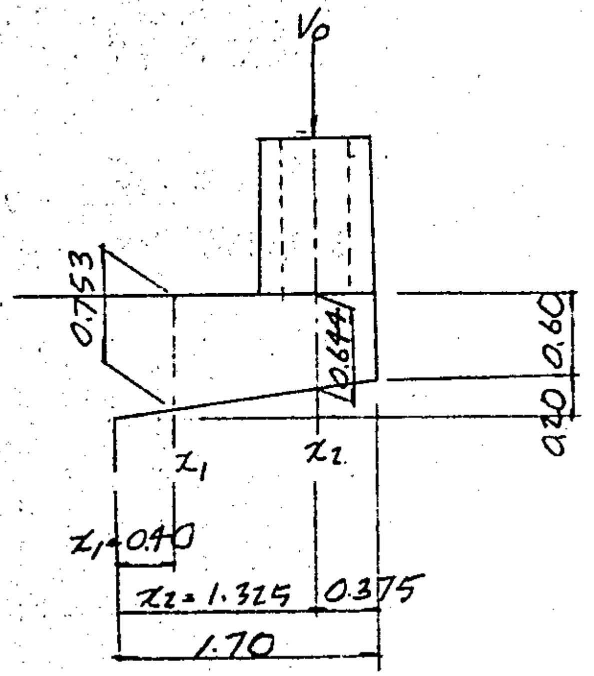
$$p = \frac{19.36}{75 \times 74} = 0.00349$$

$$1/L_c = 7.99 \qquad 1/L_s = 315$$

$$\sigma_c = \frac{M_x}{bd^2} \cdot \frac{1}{L_c} = \frac{11.49 \times 10^5}{75 \times 74^2} \times 7.99 = 22.4^{\,kg/cm^2} < 80^{\,kg/cm^2}$$

$$\sigma_s = \text{〃} \times \frac{1}{L_s} = \text{〃} \times 315 = 881\text{〃} < 1200\text{〃}$$

2. EXAMINATION OF SAFET AGAINST SHEARING FORCE



2-1. SHEARING FORCE AT SECTION x_1 AND x_2

$$S_{x_1} = 2.00 + 1.01 + 2.5^{t/m^3} \times \frac{1}{2} \times (0.60 + 0.753) \times 1.30 \times 0.75^{1.65} = 4.66^{t}$$

$$S_{x_2} = 2.00 + 1.01 + 2.5^{t/m^3} \times \frac{1}{2} \times (0.60 + 0.644) \times 0.375 \times 0.75^{0.44} = 3.45^{t}$$

2-2 EXAMINATION OF SAFETY

$$\tau_{x_1} = \frac{4.66 \times 10^3}{75 \times (75.3-6)} = 1.47\ ^{kg/cm^2} < 3.7\ ^{kg/cm^2}$$

$$\tau_{x_2} = \frac{3.45 \times 10^3}{75 \times (64.4-6)} = 0.79\ '' < \ ''$$

§§4. ABUTMENTS

CONTENTS

§ 1. Design criteria

1-1. Type of abutment structure

Wall type abutment made of R.C.

1-2. Track construction on superstructure

Ballast type track

1-3. Type of foundation

Pile foundation

Bearing stratum ; Dcs formation of its

N value $N > 30$

1-4. Design Load

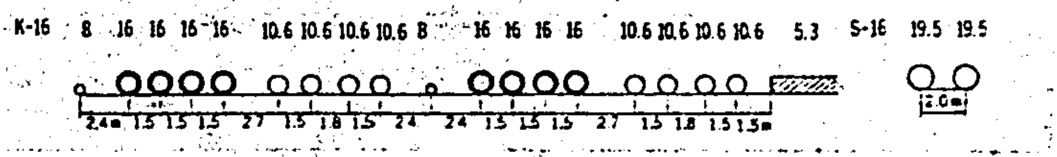
(1). Dead Load (Unit Weight)

Track assembly weight	0.45	t/m
Ballast	1.9	t/m3
Reinforced concrete	2.5	〃
Plain concrete	2.35	〃
Steel materials	7.85	〃

Material will be used actual unit weight of relevant

(2). Train Load

Train Load will be equivalent to KS-16



(K- Loading) (S-Loading)

(3). Impact Load

The impact of train load shall be the train load multipied by the following impact coefficient

Impact coefficient (KS- Loading)

span l (m)	0	5	10	20
Impact coefficient(i)	0.60	0.48	0.43	0.37

(4). Centrifugal Load

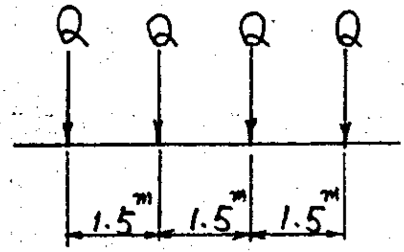
The magnitude of centrifugal load shall be the train load multiplied by the following coefficient as shown below, The working height of load is 1.8m above the rail level. The acting direction of load is horizontal and right angle to the track.

Curve Radius R (m)	Centrifugal Coefficient α
$R \leqq 700$	0.12
$700 < R \leqq 1000$	0.10
$1000 < R \leqq 1800$	0.08
$1800 < R$	0

(5). Train Lateral Load

Train lateral load under KS loading scheme shall be Q loading diagram as shown on the figure which is 15% of a driving axle load per track under K-loading scheme warking horizontal on the track at rail level in direction of right angle to the track

In the case of structure supporting the line with two or more tracks. Train lateral load is assumed as the load of only one track.



(6). Brake Load and Traction

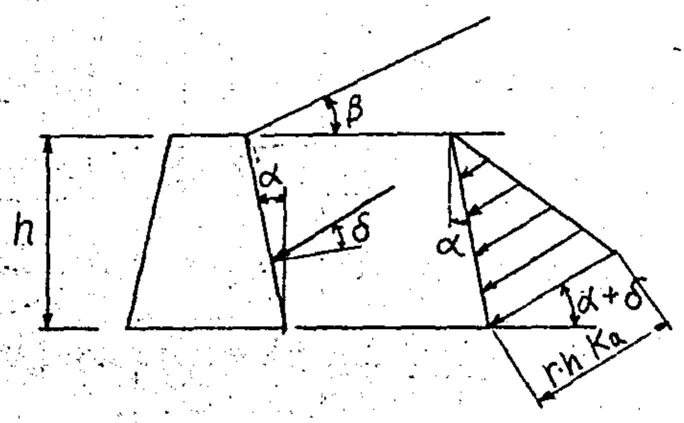
Brake load and traction load per track shall be the value as indicated below, warking parallel to the track in the track center profile at 1.8^m above rail level

Brake load	15% of the train load
Traction load	25% of the weight of the driving axle

(7). Earth Pressure

Coulomb and Rankine's coefficient of earth pressure will be used.

Coefficient of active earth pressure



◦ Under normal condition (include plus temporary)

$$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cdot\cos(\alpha+\delta)\left\{1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\alpha+\delta)\cdot\cos(\alpha-\beta)}}\right\}^2}$$

◦ Under the condition of earthquake

$$K_e = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cdot\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cdot\cos(\alpha-\beta)}}\right\}^2}$$

$$\theta = \tan^{-1}\frac{K_h}{1-K_v} = \tan^{-1}\frac{0.10}{1-0} = 5°43'$$

(8). Seismic Effect

Seismic effect of eartquake is assumed as dead load and surcharge load multiplied by seismic coefficient plus seismic earth pressure

$K_h = 0.1$ …… in horizontal direction

$K_v = 0$ …… in vertical direction

(9). Transmitted load through superstructure

--- Shoe is consisted of rubber made suport shoe and stopper of the type fitted to either fixed end or movale end.

1). Vertical force

At movale end $R_m = \frac{R}{2}$

At fixed end $R_f = \frac{R}{2}$

R : Total reaction on girder

2) Horizontal force

$$T_f = T - \frac{1}{2} \cdot \mu \cdot R_m \quad > \quad \frac{T}{2}$$

$$T_m = \mu \cdot R_m \quad < \quad \frac{T}{2}$$

μ : Coefficient of friction of rubber shoe

$\mu = 0.1$

T_f : Horizontal force acting at fixed end shoe

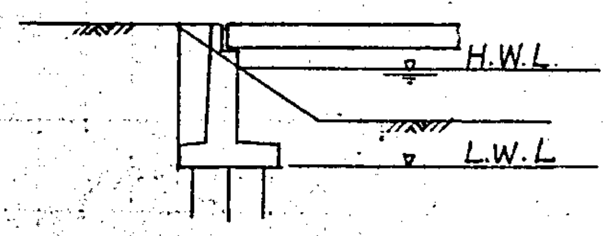
T_m : Horizontal force acting at movable end shoe

T : Total horizontal force acting at girder

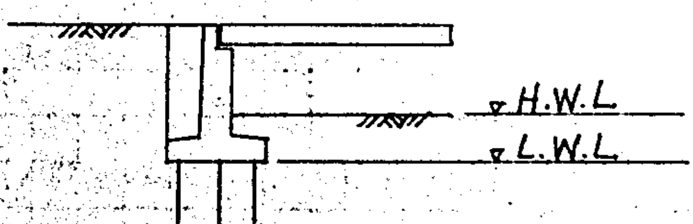
R_m : Girder reaction at movable side

(10). Water level

1). Around river area



2). Around non-river area



1-5 Concrete minimum cover

Wall, front face 50mm (net thickness)

Wall, back face 50" (")

Footing top or side face 50mm (")

Footing bottom face 150" (from bar center to concrete face)

1-6 Material and allowble stress

(1). Maleriaℓ

1). Concrete $\sigma_{ck} = 210$ kg/cm²

2). Reinforcing bar SD30

3). Max. size of coarse aggregate 25mm

(2). Allowable stress

1). Reinforced Concrete

			Design stress
Allowable compressive stress			80 kg/cm²
Allowable shearing stress	Diagonal tension member	Bending shear τ_{a1}	3.7 "
		Punching shear τ_{ap}	5.1 "
		τ_{a2}	16 "
Allowable bonding stress		Deformed reinforcing bar	16 "

2). Reinforcing bar

Type of reinforcing bar	SD30
Allowable tensile stress determined by yielding point	1800 kg/cm²

Allowable stress for analysis, against cracking

Surrounding condition	Allowable value corresponding to dead load	
	slab	beam
Permanent wet condition	1400 kg/cm²	1600 kg/cm²
Alternate dry and wet condition	1000 kg/cm²	1200 kg/cm²

1-7. Allowable stress, subjected combined load

Combination of loads	Given Extra
D + T + I + E	1.00
D + T + I + C E	1.00
D + T + I + B E	1.15
D + T + I + C + TL + E	1.25
D + S + E	1.5

D : Dead load
T : Train load
I : Impact load
C : Centrifugal load
TL : Train lateral load
B : Brake load
S : Seismic force
E : Earth pressure

1-8. Allowable Reaction of pile

$$Q = 30\bar{N} \cdot A_p$$

$\bar{N}$: Mean N value obtained from the N values measured within 4D vertical distance from pile bottom. $\bar{N} = 30$

A_p : Base area of pile $A_p = \frac{1}{4} \cdot \pi \times 0.50^2 = 0.1963^{m^2}$

Q : Ullimate reaction of pile

D : Diameter $D = 0.50^m$

$$Q = 30 \times 30 \times 0.1963 = 176^t$$

Under ordinary condition $Ra = \frac{Q}{F} = \frac{176}{3} = 58\ {}^t/_{Pile}$

Under the condition of ordinary plus temporary

$$Ra = \frac{176}{2} = 88\ {}^t/_{Pile}$$

Under the condilion of earthquake

$$Ra = \frac{176}{1.5} = 117\ {}^t/_{Pile}$$

Over-lurning analysis

Under ordinary condition

Ratio of minimum and maximum reaction of pile shall be 0.3 or more $\frac{R_{min}}{R_{max}} > 0.3$

1-9. Calculation for the pile elasticity,

Horizontal direction

$$Kh = 0.4 \times \alpha' \times \alpha \cdot E_o Bh^{-\frac{3}{4}}$$

Where

Kh : Coefficient of elasticity.
horizontal direction (kg/cm³)

α' : Correction factor for the pile sides
$\alpha' = 1.2$ (circular section)

α : For the permanent load $\alpha = 2$
For the Temporary load $\alpha = 4$

E_o : Deformation factor of the ground.
Assumed the average N value to be
$E_o = 2$ kg/cm² as observed from the
boring log.

Bh : Equivalent width of the pile
$Bh = \sqrt{D \times l_m}$
D : Diameter of pile
l_m : Depth of the first nonmove pint.

$$\beta = \sqrt[4]{\frac{Kh \cdot D}{4EI}}$$

D : Diameter of pile.

I : Moment of inertia of pile

E : Modulus of elasticity.

$$Kh = 0.32\,(\alpha E_o)^{1.103} \times D^{-0.310} \times (EI)^{-0.103}$$

$$l_m = \frac{\pi}{2\beta} \qquad D = 50^{cm}$$

$E = 3.5 \times 10^5$ kg/cm²

$E_o = 2$ kg/cm²

Under the condition	For the permanent load	For the Temporary load
$(\alpha \cdot E)^{-0.103}$	9.91	21.29
I	255 300	255 300
$D^{-0.310}$	0.297	0.297
$(E \cdot I)^{-0.103}$	0.074	0.074
Kh	0.070	0.150
β	1.77×10^{-3}	2.14×10^{-3}

1-10. P.C. Pile.

(1). Given conditions for preparing M N Diagram.

Coefficient to be multiplied to basic allowable stress	Allowable compressive stress of concrete	Allowable tensile stress of Tendon used for P.C. pile
1.0	150 kg/cm^2	90 kg/mm^2
1.1	165 "	100 "
1.2	180 "	110 "
1.5	225 "	135 "
1.65	250 "	150 "

Young's modulus of concrete.

$$E_c = 3.5 \times 10^5 \ kg/cm^2$$

Young's modulus of Tendon

$$E_s = 2.0 \times 10^6 \ kg/cm^2$$

$$n = \frac{E_s}{E_c} = 5.7$$

Design standard of concrete stress

$$\sigma_{ck} = 500 \ kg/cm^2$$

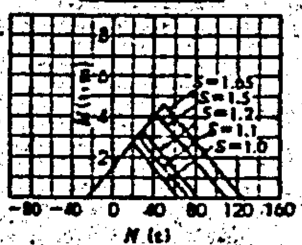
Prestressing Tendon

$$\sigma_{pa} = 0.6\,\sigma_{pu} \text{ or } 0.75\,\sigma_{py}$$
$$= 90 \ kg/mm^2 \ (\phi 7.0^{mm})$$

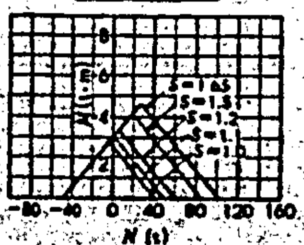
Where, concrete stress shall be $\sigma_c > 0$.

Diameter of pile (mm)	Wall thickness of pile (mm)	Category of pile prestressing	Cross sectional area of Tendon used for P.C. pile (cm²)	MN Diagram No.
350	65	A	3.1	1
		B	6.1	2
500	90	A	6.1	3
		B	12.2	4

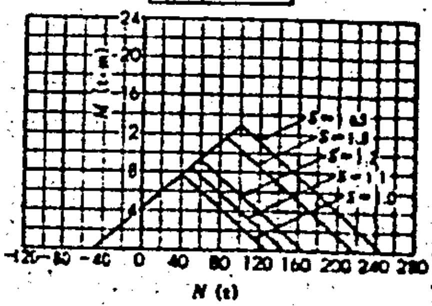
1 D350-A



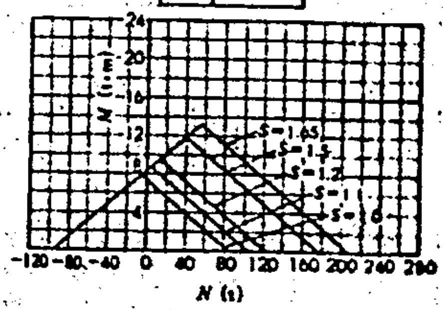
2 D350-B



3 D500-A



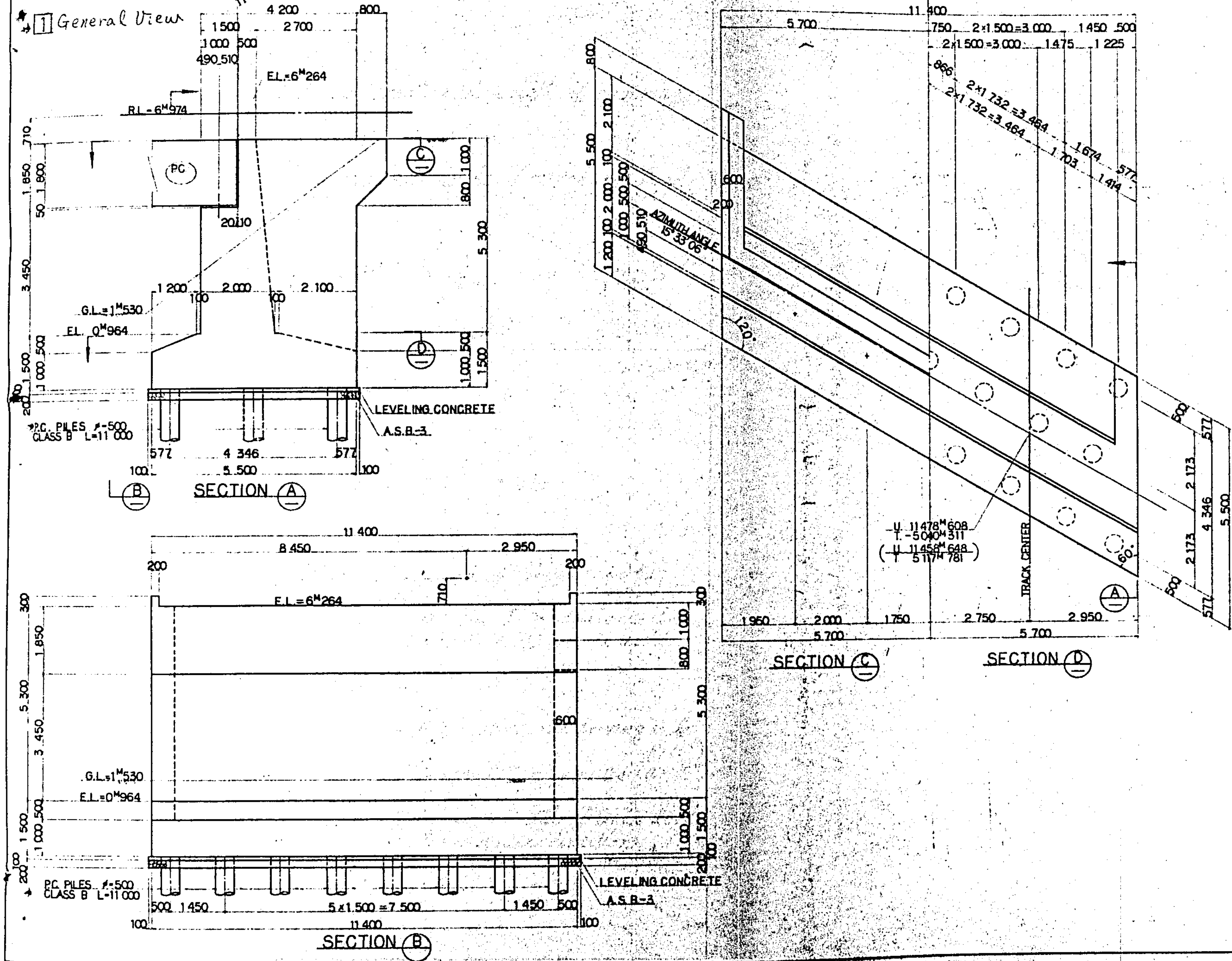
4 D500-B



MN Diagram

§2. Abutment Ab02

[1] General View



NOTES:

1. ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE INDICATED
2. REFERENCE DRAWING FOR BAR ARRANGEMENT : CS - 133
   CS - 134
   CS - 135

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	$0.45^{t/m} \times \frac{1}{5.90}$	$= 0.08^{t/m^2}$
2) ballast	$0.40^{m} \times 1.8^{t/m^3}$	$= 0.72$ 〃
		$0.80^{t/m^2}$

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

height of surcharge load

$$h_1 = \frac{q_1}{r} = \frac{1.00}{1.8} = 0.56^{m}$$

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

$$b = 5.90^{m}$$

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

Height of surcharge load

$$h_2 = \frac{q_2}{\gamma} = \frac{1.81}{1.8} = 1.01^{m}$$

For the design of breast wall

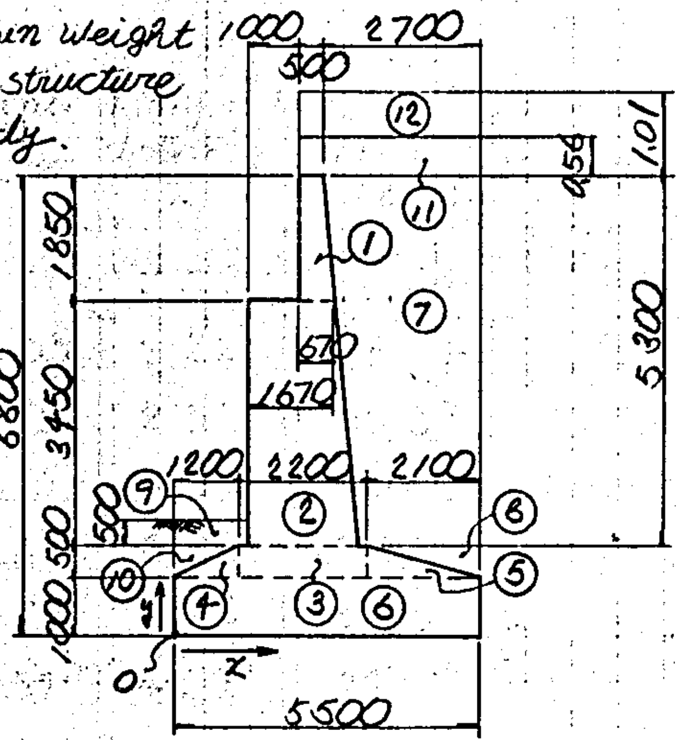
$$b = 2.00 + 2 \times 0.40 = 2.80^{m}$$

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

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	$2.5\ ^{t/m^3} \times (0.50 + 0.67) \times 1.85 \times 1/2 =$	$2.71\ ^{t/m}$
②	$2.5 \times (1.67 + 2.00) \times 3.45 \times 1/2 =$	15.83 "
③	$2.5 \times 2.20 \times 0.50 =$	2.75 "
④	$2.5 \times 1.20 \times 0.50 \times 1/2 =$	0.75 "
⑤	$2.5 \times 2.10 \times 0.50 \times 1/2 =$	1.31 "
⑥	$2.5 \times 5.50 \times 1.00 =$	13.75 "
⑦	$1.8 \times (2.70 + 2.20) \times 5.30 \times 1/2 =$	23.37 "
⑧	$1.8 \times 2.10 \times 0.50 \times 1/2 =$	0.95 "
⑨	$1.8 \times 1.30 \times 0.50 =$	1.17 "
⑩	$1.8 \times 1.20 \times 0.50 \times 1/2 =$	0.54 "
⑪	$0.56 \times 2.70 =$	1.51 "
⑫	$1.01 \times 2.70 =$	2.73 "

	Vertical force (t) N	Horizontal distance (m) x	$N \cdot x$ (t·m)	Horizontal force (t) H	Vertical distance (m) y	$H \cdot y$ (t·m)
①	2.71	2.59	7.02	0.27	5.83	1.57
②	15.83	2.22	35.14	1.58	3.17	5.01
③	2.75	2.30	6.33	0.28	1.25	0.35
④	0.75	0.80	0.60	0.08	1.17	0.09
⑤	1.31	4.10	5.37	0.13	1.17	0.15
⑥	13.75	2.75	37.81	1.38	0.50	0.69
⑦	23.37	4.27	99.79	2.34	4.24	9.92
⑧	0.95	4.80	4.56	0.10	1.33	0.13
⑨	1.17	0.65	0.76	0.12	1.75	0.21
⑩	0.54	0.40	0.22	0.05	1.33	0.07
⑪	1.51	4.15	6.27	0.15	7.08	1.06
SUB-TOTAL	64.64	—	203.87	6.48	—	19.25
⑫	2.73	4.15	11.33	—	—	—
TOTAL	67.37	—	215.20	—	—	—

$K_H = 0.10$

3) Loads acting on shoes.

(1) Vertical load on shoes.

a. dead load.

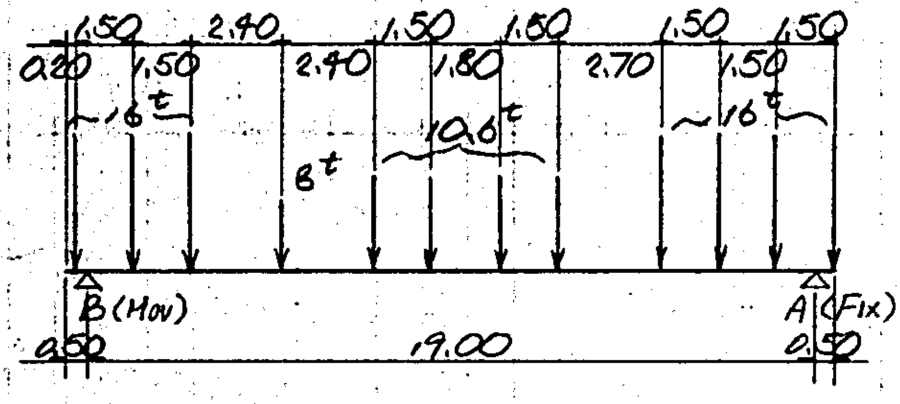
dead load of superstructure; R_d

(Refer the Table, Reaction fore of superstructure)

$R_d = 138.0^t$ $\left(R_d = \frac{138.00}{5.90} = 23.39 \ ^{t/m} \right)$

b. Train load

Reaction force of abutment caused by K.S loading



$$R_{\ell A} = \frac{1}{19.00} \times \{16 \times (1.2 + 2.7 + 15.0 + 16.5 + 18.0 + 19.5 - 0.3)$$

$$+ 10.6 \times (7.5 + 9.0 + 10.8 + 12.3) + 8 \times 5.1\} = 85.38^{t}$$

$$R_{\ell B} = (16 \times 7 + 10.6 \times 4 + 8) - 85.38 = 77.02 \ ''$$

Impact coefficient ($\ell = 19.00^{m}$)

$$i = 0.43 - \frac{19-10}{20-10} \times (0.43 - 0.37)$$

$$= 0.376$$

$$\therefore R_{A(\ell+i)} = 85.38 \times (1 + 0.376)$$

$$= 117.48^{t}$$

$$\left(R_{A(\ell+i)} = \frac{117.48}{5.90} = 19.91 \ {}^{t/m} \right)$$

(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

$$T = 0.15 \times \Sigma Re$$

$$= 0.15 \times (\overset{ReA}{85.38} + \overset{ReB}{77.02}) = 24.36^t$$

Brack load acting at fixed end shoe

$$T_A = T - \tfrac{1}{2} \cdot \mu \, (Rd + Re)$$

$$= 24.36 - \tfrac{1}{2} \times 0.10 \times (138.00 + 77.02)$$

$$= 13.61^t > T/2 = 24.36 \times \tfrac{1}{2} = 12.18^t$$

$$\therefore T_A = 13.61^t$$

perone meter width of abutment

$$T_A = \frac{13.61}{5.90} = 2.31\ {}^t/m$$

(3) Seisnic load

$$K_H = 0.10$$

Fixed end Support bearing.

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

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

$$H_A = 27.60 - \tfrac{1}{2} \times 0.10 \times 138.00$$

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

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

4) Loads acting breast wall

(1) Dead load

Distributed dead load $q_1 = 1.00\ t/m^2$

Vertical force per one meter width of abutment. $D = 1.00 \times 0.50 = 0.50\ t/m$

(2) Train load

Stability calculacion and design of substructure body.

a. Distributed train load. $q_2 = 1.81\ t/m^2$

Vertical force per one meter width of abutment. $L = 1.81 \times 0.50 = 0.91\ t/m$

b. Design of breast wall

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

5) Earth pressure

(1) Ordinary

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

r : Unit weight of soil (t/m3)

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

h' : Earth equivalent height of surcharge. (m)

Ka: Coefficient of active earth pressure.

$(\alpha = 0.\ \ \phi = 30°,\ \delta = \phi = 30°)$

$$P = \frac{1}{2} \times 1.8 \times (6.80 + 2 \times 0.56) \times 6.80 \times 0.297$$
$$= 14.40^t$$

$PH = 14.40 \times \cos 30° = 12.47^t$ $(\leftarrow)$

$PV = 14.40 \times \sin 30° = 7.20''$ $(\downarrow)$

Coordinate of acting point. $x = 5.50^m$

$y = 2.43''$

(2) Ordinary + Temporary

$$P = \frac{1}{2} \times 1.8 \times \{6.80 + 2 \times (1.01 + 0.56)\} \times 6.80$$
$$\times 0.297 = 18.07^t$$

$PH = 18.07 \times \cos 30° = 15.65^{t}$ ($\longleftarrow$)

$PV = 18.07 \times \sin 30° = 9.04$ 〃 ($\downarrow$)

Coordinate of acting point $x = 5.50^{m}$

$y = 2.62$ 〃

(3) Eorthquake

Ke : ($\alpha = 0$, $\phi = 30°$, $\delta = \phi/2 = 15°$) $\times$ 0.354

$P = 1/2 \times 1.8 \times (6.80 \times 2 \times 0.56) \times 6.80 \times 0.354$

$= 17.16^{t}$

$PH = 17.16 \times \cos 30° = 14.86^{t}$ ($\longleftarrow$)

$PV = 17.16 \times \sin 30° = 8.58$ 〃 ($\downarrow$)

Coordinate of acting point $x = 5.50^{m}$

$y = 2.43$ 〃

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	64.64	—	203.87	—	—	—
Dead load acting at shoes	23.39	1.80	42.10	—	—	—
Dead load acting at breast wall	0.50	2.55	1.28	—	—	—
Active earth pressure	7.20	5.50	39.60	-12.47	2.43	-30.30
Total	95.73	—	286.85	-12.47	—	-30.30

Stress at Point O

$N = 95.73^{t}$

$H = -12.47$ "

$M = 286.85 - 30.30 = 256.55^{t \cdot m}$

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

	N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
Own weight and surcharge	67.37	—	215.20	—	—	—
Dead load acting at shoes	23.39	1.80	42.10	—	—	—
Train load acting at shoes	19.91	1.80	35.84	—	—	—
Dead load acting at breast wall	0.50	2.55	1.28	—	—	—
Train load acting at breast wall	0.91	2.55	2.32	—	—	—
Active eath pressure	9.04	5.50	49.72	-15.65	2.62	-41.00
Total	121.12	—	346.46	-15.65	—	-41.00

$N = 121.12^{t}$, $H = -15.65^{t}$

$M = 346.46 - 41.00 = 305.46^{t \cdot m}$

3. Dead load + Train load + Impact load + Earthpressure + Brake 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	121.12	—	346.46	-15.65	—	-41.00
B	—	—	—	-2.31	4.95	-11.43
Total	121.12	—	346.46	-17.96	—	-52.43

$N = 121.12^{t}$

$H = -17.96^{"}$

$M = 346.46 - 52.43 = 294.03^{t \cdot m}$

4. Dead load + Earthpressure + 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	64.64	—	203.87	-6.48	—	-19.25
Dead load acting at shoes	23.39	1.80	42.10	-3.51	4.95	-17.37
Dead load acting at breast wall	0.50	2.55	1.28	-0.05	7.08	-0.35
Active earth Pressure	8.58	5.50	47.19	-14.86	2.43	-36.11
Total	97.11	—	294.44	-24.90	—	-73.08

$N = 97.11^{t}$

$H = -24.90^{"}$

$M = 294.44 - 73.08 = 221.36^{t \cdot m}$

2. Design of reaction force acting on piles

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

Number of piles $n = 11^{pile}$

Coordinate of acting point $x = 2.25^{m}$

Moment of inertia of the group of piles with respect to the geometrical center $I = 4 \times 2.25^2 \times 2 = 40.5^{Pile \cdot m^2}$

(2) Reaction 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 bttom of footing. (t)

n; Number of piles. (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 (piles $\cdot$ 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)

$$e = \frac{M}{N} = \frac{256.55}{95.73} = 2.68^{m}$$

$$e_0 = 2.75 - 2.68 = 0.07''$$

$$P = \frac{95.73}{11} \pm \frac{95.73 \times 0.07}{40.5} \times 2.25 = 8.70 \pm 0.37$$

$$= \begin{cases} 9.07 & t/pile \; (P_{max} = 53.51 \; t/pile) \\ 8.33 & '' \; (P_{min} = 49.15 \; '' \;) \end{cases}$$

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

$$e = \frac{305.46}{121.12} = 2.52^{m}$$

$$e_0 = 2.75 - 2.52 = 0.23^{m}$$

$$P = \frac{121.12}{11} \pm \frac{121.12 \times 0.23}{40.50} \times 2.25$$

$$= 11.01 \pm 1.55$$

$$= \begin{cases} 12.56 \ ^{t}/_{pile} & (P_{max} = 74.10 \ ^{t}/_{pile}) \\ 9.46 \ \text{"} & (P_{min} = 55.81 \ \text{"}) \end{cases}$$

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

$$e = \frac{294.03}{121.12} = 2.43$$

$$e_0 = 2.75 - 2.43 = 0.32^{m}$$

$$P = \frac{121.12}{11} \pm \frac{121.12 \times 0.32}{40.50} \times 2.25$$

$$= 11.01 \pm 2.15$$

$$= \begin{cases} 13.16 \ ^{t}/_{pile} & (P_{max} = 77.64 \ ^{t}/_{pile}) \\ 8.86 \ \text{"} & (P_{min} = 52.27 \ \text{"}) \end{cases}$$

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

$$e = \frac{221.36}{97.11} = 2.28^{m}$$

$$e_0 = 2.75 - 2.28 = 0.47^{m}$$

$$P = \frac{97.11}{11} \pm \frac{97.11 \times 0.47}{40.50} \times 2.25$$

$$= 8.83 \pm 2.54$$

$$= \begin{cases} 11.37 \ ^{t}/_{pile} & (P_{max} = 67.08 \ ^{t}/_{pile}) \\ 6.29 \ \text{"} & (P_{min} = 37.11 \ \text{"}) \end{cases}$$

3. Stability against overturning

a. D+EP

$$\frac{P_{min}}{P_{max}} = \frac{49.15}{53.51} = 0.92 > 0.3$$

b. D+T+I+EP+Br

$$\frac{P_{min}}{P_{max}} = \frac{55.81}{74.10} = 0.75 > 0.1$$

c. D+T+I+EP+Br

$$\frac{P_{min}}{P_{max}} = \frac{52.27}{77.64} = 0.67 > 0.1$$

d. D+EP+Se

$$\frac{M}{N} = \frac{45.64}{97.11} = 0.47 < 2.25^{m}$$

4. Stability against vertical support.

a. D+EP

$$P_{max} = 53.51\ ^{t}/_{Pile} < P_A = 58\ ^{t}/_{Pile}$$

b. D+T+I+EP+Br

$$P_{max} = 74.10\ ^{t}/_{Pile} < P_A = 88\ ^{t}/_{Pile}$$

c. D+T+I+EP+Br

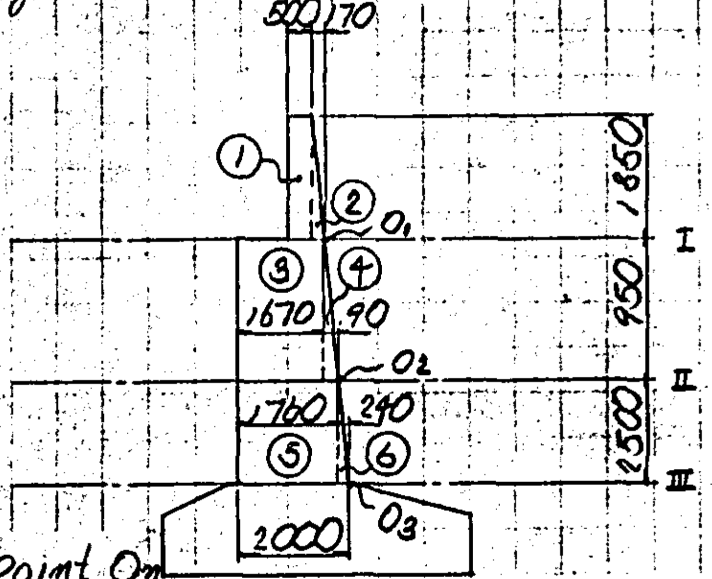
$$P_{max} = 77.64\ ^{t}/_{Pile} < P_A = 88\ ^{t}/_{Pile}$$

d. D+ED+Se

$$P_{max} = 67.08\ ^{t}/_{Pile} < P_A = 117\ ^{t}/_{Pile}$$

4 Stress calculation of abutment body

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



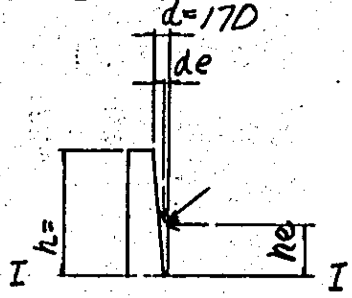
Stress at point On

			N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
I ~ I SECTION	①	2.5× 0.50 × 1.85	2.31	0.42	0.97	0.23	0.93	0.21
	②	2.5× 0.17 × 1.85× 1/2	0.39	0.11	0.04	0.04	0.62	0.02
	TOTAL		2.70	—	1.01	0.27	—	0.23
II ~ II SECTION	①	—	2.31	0.51	1.18	0.23	1.88	0.43
	②	—	0.39	0.20	0.08	0.04	1.57	0.06
	③	2.5× 1.67 × 0.95	3.97	0.93	3.69	0.40	0.48	0.19
	④	2.5× 0.09 × 0.95× 1/2	0.11	0.06	0.01	0.01	0.32	0.01
	TOTAL		6.78	—	4.96	0.68	—	0.69
III ~ III SECTION	①	—	2.31	0.75	1.73	0.23	4.38	1.01
	②	—	0.39	0.44	0.17	0.04	4.07	0.16
	③	—	3.97	1.17	4.64	0.40	2.98	1.19
	④	—	0.11	0.30	0.03	0.01	2.82	0.03
	⑤	2.5× 1.76 × 2.50	11.00	1.12	12.32	1.10	1.25	1.38
	⑥	2.5× 0.24 × 2.50× 1/2	0.75	0.16	0.12	0.08	0.83	0.07
	TOTAL		18.53	—	19.01	1.86	—	3.84

2. Earth pressure and its acting point

(1) Ordinary

a. I ~ I section



$h' = 0.56$, $h = 1.85$

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

$Ka = 0.320$

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

$P_a = 1/2 \times 1.8 \times (1.85 + 2 \times 0.56) \times 1.85 \times 0.320 = 1.58$ t/m

$P_H = 1.58 \times \cos(2.579° + 15°) = 1.50$ t/m (←)

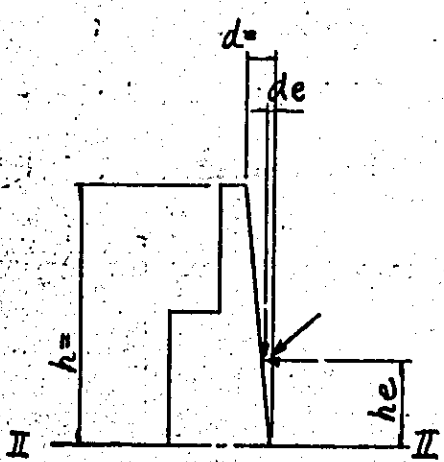
$P_V = 1.58 \times \sin(2.579° + 15°) = 0.48$ " (↓)

Acting point

$$he = \frac{h}{3} \times \frac{h + 3h'}{h + 2h} = \frac{1.85}{3} \times \frac{1.85 + 3 \times 0.56}{1.85 + 2 \times 0.56} = 0.73 \text{ m}$$

$$de = \frac{d}{h} \times he = \frac{0.17}{1.85} \times 0.73 = 0.07 \text{ m}$$

b. II ~ II Section



$h' = 0.56$, $h = 2.80$

$P_a = 1/2 \times 1.8 \times (2.80 + 2 \times 0.56)$
$\times 2.80 \times 0.320 = 3.16$ t/m

$P_H = 3.16 \times \cos(2.579° + 15°) = 3.01$ t/m (←)

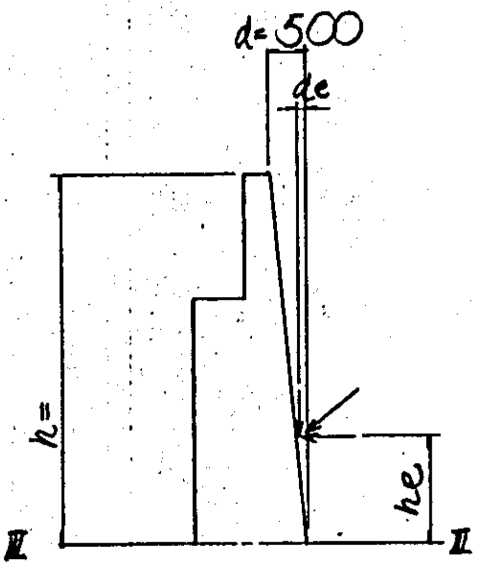
$P_V = 3.16 \times \sin(2.579° + 15°) = 0.95$ " (↓)

Acting point

$$he = \frac{2.80}{3} \times \frac{2.80 + 3 \times 0.56}{2.80 + 2 \times 0.56} = 1.07 \ ^{m}$$

$$de = \frac{0.26}{2.80} \times 1.07 = 0.10 \ ^{m}$$

c. III ~ III section



$h' = 0.56$

$h = 5.30$

$$Pa = 1/2 \times 1.8 \times (5.30 + 2 \times 0.56) \times 5.30 \times 0.320 = 9.80 \ ^{t/m}$$

$$P_H = 9.80 \times \cos(2.579^\circ + 15^\circ) = 9.34 \ ^{t/m} \ (\leftarrow)$$

$$P_V = 9.80 \times \sin(2.579^\circ + 15^\circ) = 2.96 \ ^{\prime\prime} \ (\downarrow)$$

Acting point

$$he = \frac{5.30}{3} \times \frac{5.30 + 3 \times 0.56}{5.30 + 2 \times 0.56} = 1.92 \ ^{m}$$

$$de = \frac{0.50}{5.30} \times 1.92 = 0.18 \ ^{m}$$

(2) Ordinary + Temporary

a. I ~ I section

$h' = 2.12$, $h = 1.85$

$K_A = 0.320$

$P_A = 1/2 \times 1.8 \times \{1.85 + 2 \times (2.12 + 0.56)\} \times 1.85$

$\times 0.320 = 3.84^{t/m}$

$P_H = 3.84 \times \cos(2.579° + 15°) = 3.66^{t/m} (\leftarrow)$

$P_V = 3.84 \times \sin(2.579° + 15°) = 1.16$ " $(\downarrow)$

Actingpoint

$he = \frac{1.85}{3} \times \frac{1.85 + 3 \times 2.68}{1.85 + 2 \times 2.68} = 0.85^{m}$

$de = \frac{0.17}{1.85} \times 0.85 = 0.08^{m}$

b. II ~ II section

$h' = 2.12 + 0.56$, $h = 2.80$

$P_A = 1/2 \times 1.8 \times \{2.80 + 2 \times (2.12 + 0.56)\} \times 2.80$

$\times 0.320 = 6.58^{t/m}$

$P_H = 6.58 \times \cos(2.579° + 15°) = 6.27^{t/m} (\leftarrow)$

$P_V = 6.58 \times \sin(2.579° + 15°) = 1.99$ " $(\downarrow)$

Actingpoint

$he = \frac{2.80}{3} \times \frac{2.80 + 3 \times 2.68}{2.80 + 2 \times 2.68} = 1.24^{m}$

$de = \frac{0.26}{2.80} \times 1.24 = 0.12^{m}$

c. III - III Section

$h' = 2.12 + 0.56$, $h = 5.30$

$Pa = 1/2 \times 1.8 \times \{ 5.30 + 2 \times (2.12 + 0.56) \} \times 5.30$

$\times 0.320 = 16.27$ t/m

$PH = 16.27 \times \cos(2.579° + 15°) = 15.51$ t/m (←)

$PV = 16.27 \times \sin(2.579° + 15°) = 4.91$ " (↓)

Acting point

$$he = \frac{5.30}{3} \times \frac{5.30 + 3 \times 2.68}{5.30 + 2 \times 2.68} = 2.21 \text{ m}$$

$$de = \frac{0.50}{5.30} \times 2.21 = 0.21 \text{ m}$$

(3) Earthquake

- a. I~I section

$h' = 0.56$, $h = 1.85$

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

$K_E = 0.401$

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

$= 1/2 \times 1.8 \times (1.85 + 2 \times 0.56) \times 1.85 \times 0.401$

$= 1.98$ t/m

$P_{EH} = 1.98 \times \cos 2.579° = 1.98$ t/m ($\leftarrow$)

$P_{EV} = 1.98 \times \sin 2.579° = 0.09$ " ($\downarrow$)

Acting point

$he = \frac{1.85}{3} \times \frac{1.85 + 3 \times 0.56}{1.85 + 2 \times 0.56} = 0.73$ m

$de = \frac{0.17}{1.85} \times 0.73 = 0.07$ m

b. II~II Section

$h' = 0.56$, $h = 2.80$

$P_E = 1/2 \times 1.8 \times (2.80 + 2 \times 0.56) \times 2.80 \times 0.401$

$= 3.96$ t/m

$P_{EH} = 3.96 \times \cos 2.579° = 3.96$ t/m ($\leftarrow$)

$P_{EV} = 3.96 \times \sin 2.579° = 0.18$ " ($\downarrow$)

Acting point

$he = \frac{2.80}{3} \times \frac{2.80 + 3 \times 0.56}{2.80 + 2 \times 0.56} = 1.07$ m

$de = \frac{0.26}{2.80} \times 1.07 = 0.10$ m

c. Ⅲ~Ⅲ Section

$h' = 0.56 \quad , \quad h = 5.30$

$P_E = 1/2 \times 1.8 \times (5.30 + 2 \times 0.56) \times 5.30 \times 0.401$
$= 12.28 \ ^{t/m}$

$P_{EH} = 12.28 \times \cos 2.579° = 12.27 \ ^{t/m} \quad (\leftarrow)$
$P_{EV} = 12.28 \times \sin 2.579° = 0.55 \ '' \quad (\downarrow)$

Acting point

$$he = \frac{5.30}{3} \times \frac{5.30 + 3 \times 0.56}{5.30 + 2 \times 0.56} = 1.92 \ ^{m}$$

$$de = \frac{0.50}{5.30} \times 1.92 = 0.18 \ ^{m}$$

3. Stress calculation on each section

(B= 1m)
per one meter width of abutment.

(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	2.70	—	1.01	0.27	—	0.23
②	Dead load acting at breast wall	0.50	0.42	0.21	0.05	2.13	0.11
③	Train load and Impact load acting at breast wall	5.71	0.42	2.40	—	—	—
④	Dead load acting at shues	—	—	—	—	—	—
⑤	Train load and Impact load acting at shues	—	—	—	—	—	—
⑥	Brake load	—	—	—	—	—	—
⑦	Earth pressure (ordinary)	0.48	0.07	0.03	1.50	0.73	1.10
⑧	Earth pressure (Ordinary + Temporary)	1.16	0.08	0.09	3.66	0.85	3.11
⑨	Earth pressure (earth quake)	0.09	0.07	0.01	1.98	0.73	1.45
case 1	D + Ep	3.68	—	1.25	1.50	—	1.10
2	D+T+I+Ep	10.07	—	3.71	3.66	—	3.11
3	D+T+I+Ep+B	10.07	—	3.71	3.66	—	3.11
4	D+Ep (S)	3.29	—	1.23	2.30	—	1.79

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

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

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

Case 4 ; Dead load + Earth pressure (seismic)
; ① + ② + ④ + ⑨ α = 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	6.78	—	4.96	0.68	—	0.69
②	Dead load acting at breast wall	0.50	0.51	0.26	0.05	3.08	0.15
③	Train load and Impact load acting at breast wall	5.71	0.51	2.91	—	—	—
④	Dead load acting at shues	23.39	1.26	29.47	3.51	0.95	3.33
⑤	Train load and Impact load acting at shues	19.91	1.26	25.09	—	—	—
⑥	Brake load	—	—	—	2.31	0.95	2.19
⑦	Earth pressure (ordinary)	0.95	0.10	0.10	3.01	1.07	3.22
⑧	Earth pressure (ordinary + Temporary)	1.99	0.12	0.24	6.27	1.24	7.77
⑨	Earth pressure (earth quake)	0.18	0.10	0.02	3.96	1.07	4.24
case 1	D + Ep	31.62	—	34.79	3.01	—	3.22
2	D+T+I+Ep	58.28	—	62.93	6.27	—	7.77
3	D+T+I+Ep+B	58.28	—	62.93	8.53	—	9.96
4	D+Ep (S)	30.85	—	34.71	8.20	—	8.41

(3) III - III 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	18.53	—	19.01	1.86	—	3.84
② Dead load acting at breast wall	0.50	0.75	0.38	0.05	5.58	0.28
③ Train load and Impact load acting at breast wall	5.71	0.75	4.28	—	—	—
④ Dead load acting at shues	23.39	1.50	35.09	3.51	3.45	12.11
⑤ Train load and Impact load acting at shues	19.91	1.50	29.87	—	—	—
⑥ Brake load	—	—	—	2.31	3.45	7.97
⑦ Earth pressure (ordinary)	2.96	0.18	0.53	9.34	1.92	17.93
⑧ Earth pressure (ordinary + Temporary)	4.91	0.21	1.03	15.51	2.21	34.28
⑨ Earth pressure (earth quake)	0.55	0.18	0.10	12.27	1.92	23.56
Case 1 D + Ep	45.38	—	55.01	9.34	—	17.93
2 D+T+I+Ep	72.95	—	89.66	15.51	—	34.28
3 D+T+I+Ep+B	72.95	—	89.66	17.81	—	42.25
4 D+Ep (S)	42.97	—	54.58	17.69	—	39.79

Stress calculation I-I Section

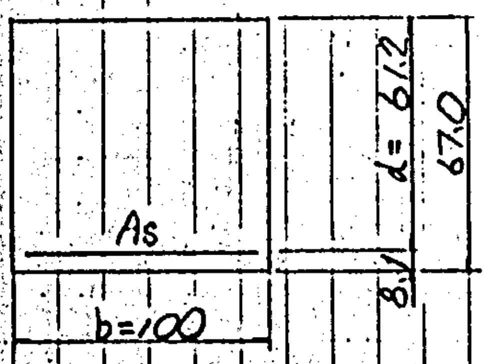
$$D + T + I + Ep \left\{ \begin{array}{l} M = 3.71 + 3.11 = 6.82^{t\cdot m} \\ N = 10.07^{t} \\ H = 3.66^{t} \end{array} \right.$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	6.82	Q (t)	3.6
N (t)	10.07	u (cm)	25.4
b (cm)	100		
d (cm)	58.9	d'(cm)	8.1
As (cm2)	D 13 X 8 = 10.136		
As' (cm2)			
d'/d	0.14	f (cm)	93.13
M' (t·m)	9.38		
M'/bd2(kg/cm2)	2.7	Q/bd (kg/cm2)	0.62
nP	0.026	As'/As	0

C	S	Z
7.58	18.38	1.04

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
20.50 < 80	745.53 < 1800	0.64 < 4

$$\tau = \frac{3.66 \times 10^3}{100 \times 58.9} = 0.6\ \mathrm{kg/cm^2} < \tau_R = 3.7\ \mathrm{kg/cm^2}$$



As = D13 - 12.5cm ctc

Stress calculation II-II Section

$$D + T + I + E_P \left\{ \begin{array}{l} M = 62.93 + 7.77 = 70.70^{t \cdot m} \\ N = 58.28^{t} \\ H = 6.27^{t} \end{array} \right.$$

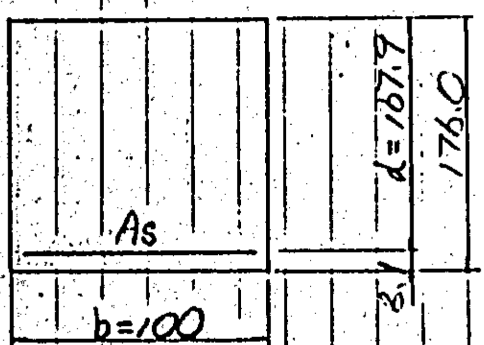
n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	70.7	Q (t)	6.2
N (t)	58.28	u (cm)	79.9
b (cm)	100		
d (cm)	167.9	d′ (cm)	8.1

As (cm2)	D 19 X 4 = 11.46		
As′ (cm2)			
d′/d	0.05	f (cm)	201.21
M′ (t·m)	117.27		
M′/bd2(kg/cm2)	4.16	Q/bd (kg/cm2)	0.37
nP	0.01	As′/As	0

C	S	Z
8.58	25.13	0.96

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
35.70 < 80	1,568.38 < 1800	0.36 < 4

$$\tau = \frac{6.27 \times 10^3}{100 \times 167.9} = 0.4 \ \mathrm{kg/cm^2} < \tau_R = 3.7 \ \mathrm{kg/cm^2}$$



As = D 19 - 25.0 cm c.t.c

Stress calculation III - III Section

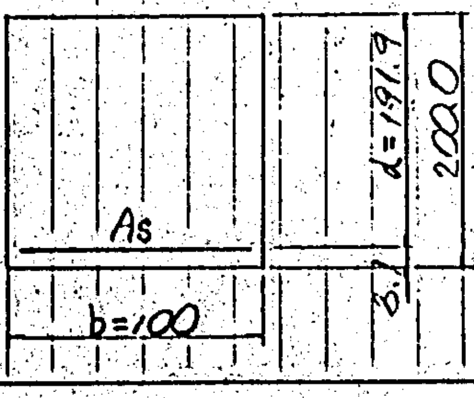
$$D + T + I + E_p \quad \begin{cases} M = 89.66 + 34.28 = 123.94^{t \cdot m} \\ N = 72.95^{t} \\ H = 15.51^{t} \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	123.94	Q (t)	15.5
N (t)	72.95	u (cm)	91.9
b (cm)	100		
d (cm)	191.9	d' (cm)	8.1
As (cm2)	D 19 X 8 = 22.92		
As' (cm2)			
d'/d	0.04	r (cm)	261.8
M' (t·m)	190.98		
M'/bd2(kg/cm2)	5.19	Q/bd (kg/cm2)	0.81
nP	0.018	As'/As	0

C	S	Z
7.93	20.60	1.01

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
41.13 < 80	1,603.03 < 1800	0.82 < 4

$$\tau = \frac{15.51 \times 10^3}{100 \times 191.9} = 0.8 \ \mathrm{kg/cm^2} < \tau_R = 3.7 \ \mathrm{kg/cm^2}$$



As = D 19 - 12.5cm ctc

Stress calculation II-II Section for crack analysis

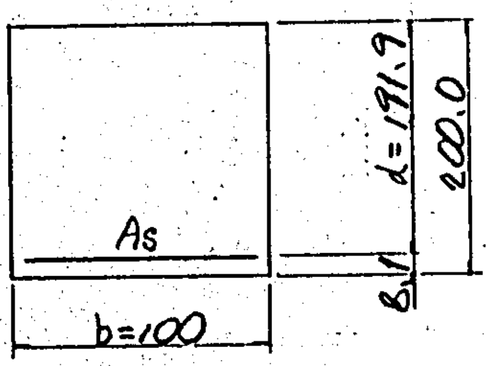
D + T

$$\begin{cases} M = 55.01 + 17.93 = 72.94^{t \cdot m} \\ N = 9.34^{t} \\ H = 45.38'' \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1400
Tca (kg/cm2)	3.7		
M (t·m)	72.94	Q (t)	9.3
N (t)	45.38	u (cm)	91.9
b (cm)	100		
d (cm)	191.9	d′(cm)	8.1
As (cm2)	D 19 X 8 = 22.92		
As′ (cm2)			
d′/d	0.04	f (cm)	252.63
M′ (t·m)	114.64		
M′/bd2(kg/cm2)	3.11	Q/bd (kg/cm2)	0.49
np	0.018	As′/As	0

C	S	Z
7.73	19.31	1.00

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
24.07 < 80	902.12 < 1400	0.49 < 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 calculation of Piles and foundation.

$$P = \underbrace{\frac{N}{n} \pm \frac{M}{I} \cdot x}_{\text{Refer The article of stability calculation}} \pm (\Delta N)$$

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

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

(1) D+Ep

$$\Sigma Mo = \frac{H}{2 \cdot \beta} = \frac{12.47 \times 5.90}{2 \times 1.77} = 207.83 \ \text{t.m}$$

$$P = \begin{cases} 53.51 \\ 49.15 \end{cases} \pm \frac{207.83}{40.5} \times 2.25$$

$$= \begin{matrix} 53.51 \\ 49.15 \end{matrix} \pm 11.55 = \begin{cases} 65.06 \\ 37.60 \end{cases} \ \text{t/pile}$$

(2) D+T+I+Ep

$$\Sigma Mo = \frac{15.65 \times 5.90}{2 \times 2.14} = 215.74 \ \text{t.m}$$

$$P = \begin{cases} 74.10 \\ 55.81 \end{cases} \pm \frac{215.74}{40.5} \times 2.25$$

$$= \begin{cases} 74.10 \\ 55.81 \end{cases} \pm 11.99 = \begin{cases} 86.09 \\ 43.82 \end{cases} \ \text{t/pile}$$

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

$$\Sigma Mo = \frac{17.96 \times 5.90}{2 \times 2.14} = 247.58 \ \text{t.m}$$

$$P = \begin{cases} 77.64 \\ 52.27 \end{cases} \pm \frac{247.58}{40.5} \times 2.25$$

$$= \begin{cases} 77.64 \\ 52.27 \end{cases} \pm 13.75 = \begin{cases} 91.39 \\ 38.52 \end{cases} \ \text{t/pile}$$

(4) Dead load + Earth pressure (seismic)

$$\Sigma M_0 = \frac{24.90 \times 5.90}{2 \times 0.214} = 343.25^{\text{t}\cdot\text{m}}$$

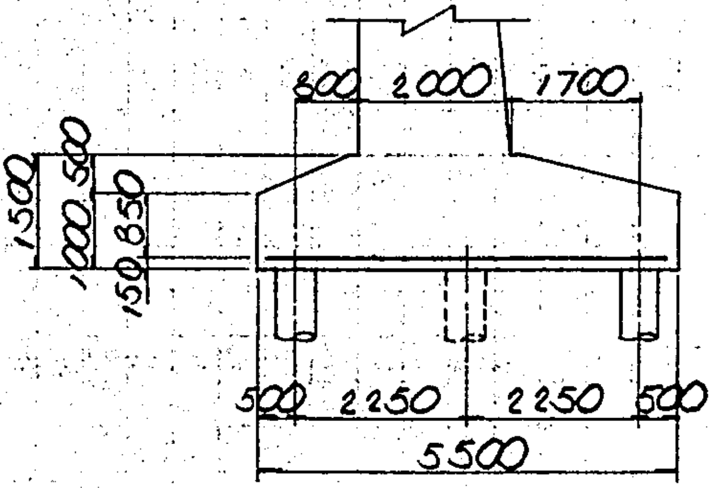
$$P = \left\{ \begin{matrix} 67.08 \\ 37.11 \end{matrix} \right. \pm \frac{343.25}{40.5} \times 2.25$$

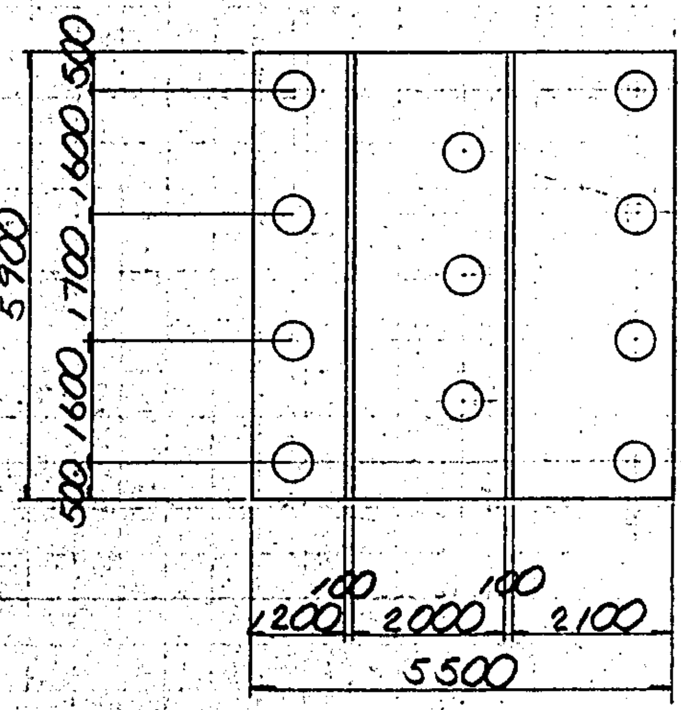
$$= \left\{ \begin{matrix} 67.08 \\ 37.11 \end{matrix} \right. \pm 19.07 = \left\{ \begin{matrix} 86.15 \\ 18.04 \end{matrix} \right. \begin{matrix} \text{t/pile} \\ " \end{matrix}$$

2. Calculation at front toe

(1) Bending moment analysis

a. effective width of footing





(i) Center part

$$h = \frac{1}{2} \times 150 - 15 = 60^{cm}$$

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

$$b = b_0 + 2e = 170 + 2 \times 80 = 330^{cm}$$

$$< \text{Distance between piles} = 160^{cm}$$

$$\therefore b = 160^{cm}$$

(ii) End part

$$b_s = 50^{cm}$$

$$b/2 = 80^{cm}$$

Since,

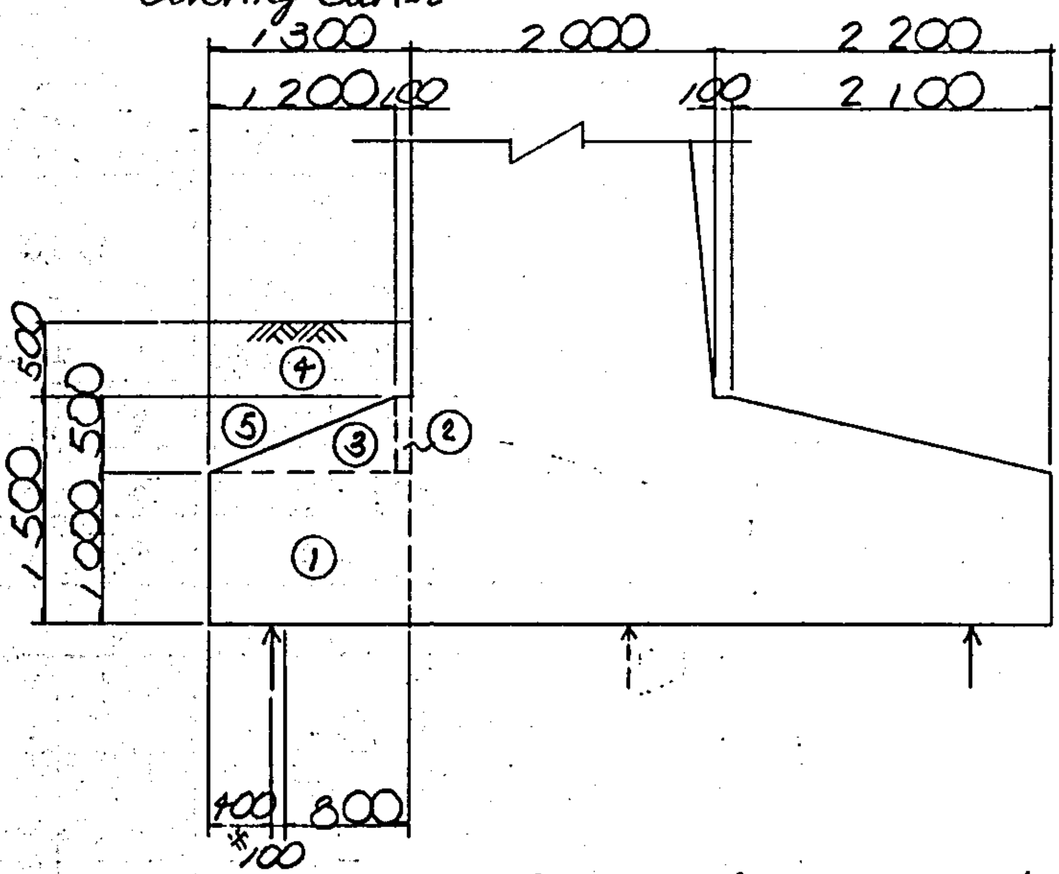
$$b_s < b/2$$

$$b = b_s + b/2 = 50 + 80 = 130^{cm}$$

b. Vertical reaction force acting on piles

Combined loads	Reaction force acting on piles Pmax (t)
D + EP	65.06
D+T+I+EP	86.09
D+T+I+Br	91.39
D+Se+EP(se)	81.15

c. Stress of own weinght of footing and weight of Covering earth



* Tolerance for construction

Own waght of footing and weight of covering earth.

	calculation	N (t)	x (m)	M = N·x (t·m)
①	2.5 × 1.30 × 1.00	3.25	0.65	2.11
②	2.5 × 0.10 × 0.50	0.13	0.05	0.01
③	2.5 × 1.20 × 0.50 × 1/2	0.75	0.50	0.38
④	1.8 × 1.30 × 0.50	1.17	0.65	0.76
⑤	1.8 × 1.20 × 0.50 × 1/2	0.54	0.90	0.49
Σ	———	5.84	———	3.75

d. Calculation of section force

	per own pile			per meter				
case	Pv·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	58.55	-18.89	39.66	30.51	-3.75	26.76	1.00	26.76
〃 2	77.48	-19.61	57.87	44.52	-3.75	40.77	〃	40.77
〃 3	82.25	-22.51	59.74	45.95	-3.75	42.20	1.15	36.70
〃 4	77.54	-31.20	46.34	35.65	-3.75	31.90	1.50	21.27

$l = 0.90^m$

ef : effectivewidth

W : Own weight of footing

V : Value Converted to ordinary to ordinary Cace

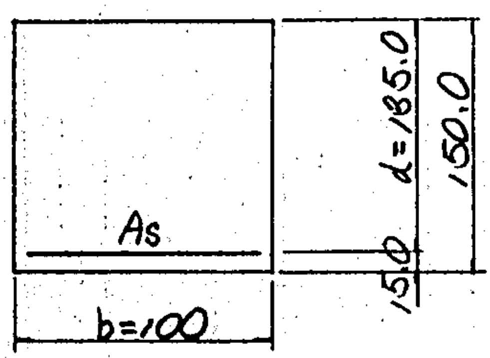
Stress calculation.

$$D + T + I + Ep \begin{cases} M = 40.77 \ ^{t\cdot m} \\ N = \text{——} \\ H = \text{——} \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	40.77	Q (t)	0
N (t)	0	u (cm)	60
b (cm)	100		
d (cm)	135	d′(cm)	15
As (cm2)	D 19 X 8 = 22.92		
As′ (cm2)			
d′/d	0.11	f (cm)	0
M′ (t·m)	40.77		
M′/bd2(kg/cm2)	2.24	Q/bd (kg/cm2)	0
nP	0.025	As′/As	0

C	S	Z
10.62	42.01	1.07

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
23.76 < 80	1,409.93 < 1800	0.00 < 4



As = D 19 - ctc 12.5cm

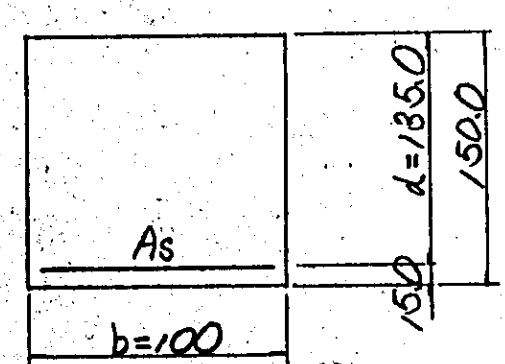
Stress calculation for crack analysis.

$$\begin{cases} M = 26.76 \ t{\cdot}m \\ N = \text{——} \\ H = \text{——} \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1400
Tca (kg/cm2)	3.7		
M (t・m)	26.76	Q (t)	0
N (t)	0	u (cm)	60
b (cm)	100		
d (cm)	135	d′(cm)	15
As (cm2)	D 19 X 8 = 22.92		
As′ (cm2)			
d′/d	0.11	f (cm)	0
M′ (t・m)	26.76		
M′/bd2(kg/cm2)	1.47	Q/bd (kg/cm2)	0
nP	0.025	As′/As	0

C	S	Z
10.62	42.01	1.07

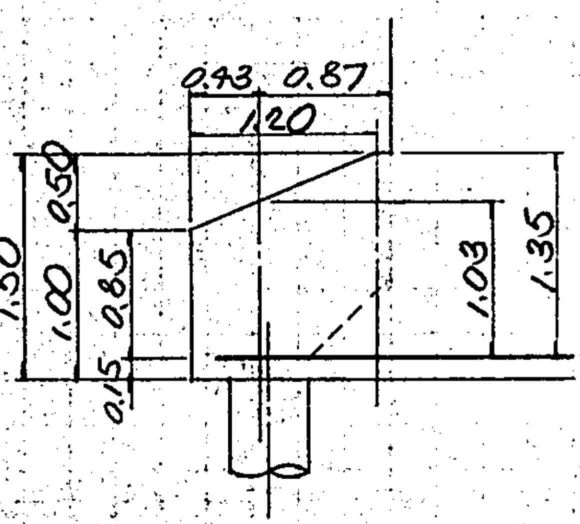
Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
15.59 < 80	925.43 < 1400	0.00 < 4

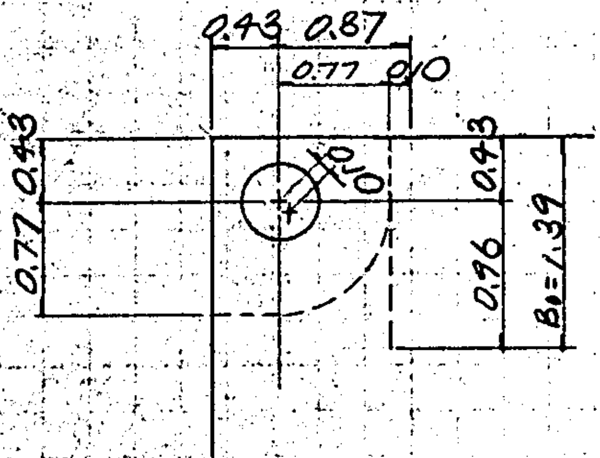


As = D 19 - 12.5 cm c.t.c

(2) Calculation of Shearing force

a. effective width (area of shearing section)





10cm constructional tolerance is assumed towards dangerous side.

$d = 0.50 + 1.03 = 1.53^{m}$

$d/2 = 1.53 \times 1/2 = 0.77^{"}$

(i) Effective area resisting bending shear

$B_0 = l_1 + l_2$

$= 0.6 \times 1.60 + 0.43 = 1.39^{m}$

$A = B_0 \cdot d$

$= 1.39 \times 1.35 = 1.88\ m^2$

(ii) Effective area resisting punching shear.

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

$$= \tfrac{1}{2} \times (0.85 + 1.03) \times 0.43 + \tfrac{1}{2} \times 3.142 \times 0.77$$

$$\times \tfrac{1}{2} \times (1.03 + 1.35) + 1.35 \times 0.43$$

$$= 2.42 \ m^2$$

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

Resisting area for normal load.

$$A = 0.43^2 + 0.43 \times 0.77 \times 2 + \tfrac{1}{4} \times 3.142 \times 0.77^2$$

$$= 1.31 \ m^2 \quad (\text{for punching shear.})$$

$$A = 1.20 \times 1.39 = 1.67 \ m^2 \quad (\text{for bending shear})$$

$$\overset{(1.31)}{1.67} \times (1.00 + 1.50) \times \tfrac{1}{2} \times 2.5 = \overset{(4.09)}{5.22} \ t$$

$$\overset{(1.31)}{1.67} \times (0.50 + 1.00) \times \tfrac{1}{2} \times 1.8 = \overset{(1.77)}{2.25} \ "$$

$$\Sigma = \underset{(5.86)}{7.47} \ t$$

(3) Calculation of shearing stress

D+T+I+EP is assumed

$$S = 86.09 - \overset{(5.86)}{7.47} = \overset{(80.23)}{78.62}\ t$$

a. Shearing stress caused by bending

$$\tau = \frac{78.62 \times 10^3}{18800} = 4.2\ kg/cm^2 < \tau_a = 3.7\ kg/cm^2$$

b. Shearing stress caused by punching

$$\tau = \frac{80.23 \times 10^3}{24\ 200} = 3.3\ kg/cm^2 < \tau_a = 5.1\ kg/cm^2$$

3. Calculation at rear heel.

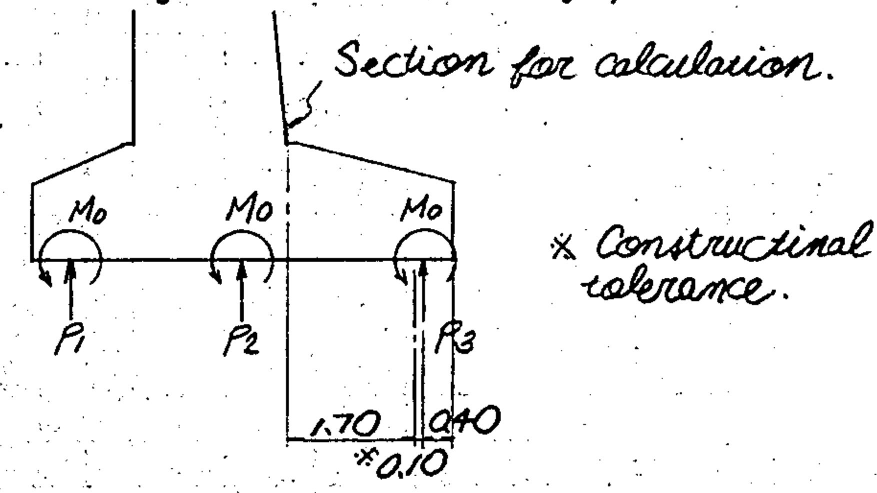
(1) Bending moment analysis.

Upside reinforcing is calculated on the condition of dead load + seismic load + earth presser, referred the reaction of pils.

a. Effective width of footing

calculation at rear heel, referred 2,(1), a $b = 1.30^{m}$

b. Stress caused by reaction force of pile.



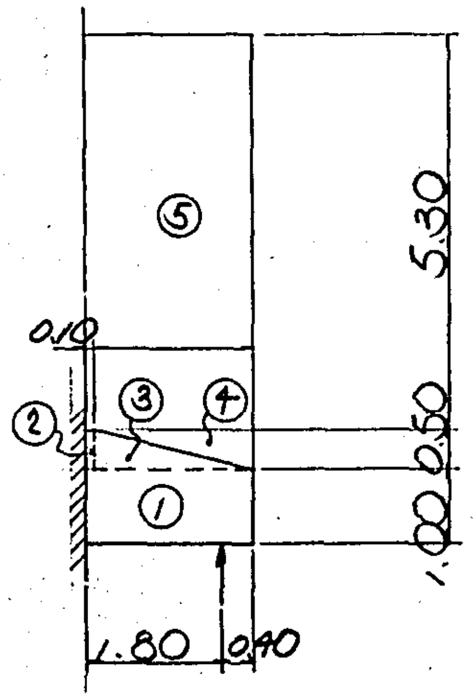
$P_3 = 18.04^{t}$, $M_o = 31.20^{t \cdot m}$

$M = 18.04 \times 1.80 + 31.20 = 63.67^{t \cdot m}$

Per meter moment will be,

$M = 63.67 \times 1/1.30 = 48.98^{t \cdot m}$

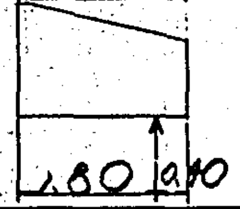
c. Own weight of footing and weight of covering earth



	Calculation	N (t)	x (m)	N·x (t·m)
1	2.5 × 2.20 × 1.00	5.50	1.10	6.05
2	2.5 × 0.10 × 0.50	0.13	0.05	0.01
3	2.5 × 2.10 × 0.50 × 1/2	1.31	0.80	1.05
4	1.8 × 2.10 × 0.50 × 1/2	0.95	1.50	1.43
5	1.8 × 2.20 × 5.30	20.99	1.10	23.09
Σ	———	28.88	——	31.63

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

$P_V = 8.58^{\ t}$

(Refer §1. 1.5, (3))

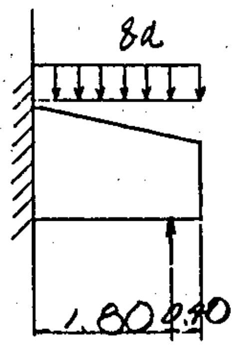
Acting point

$x = 2.20 \times 1/3 \times 2 = 1.47$ m

$M = 7.80 \times 2.20 \times 1/2 \times 1.47 = 12.61$ t·m

e. Stress caused by surcharge

$q_a = 1.00$ t/m²



$x = 2.20 \times 1/2 = 1.10$ m

$M = 1.00 \times 2.20 \times 1.10 = 2.42$ t·m

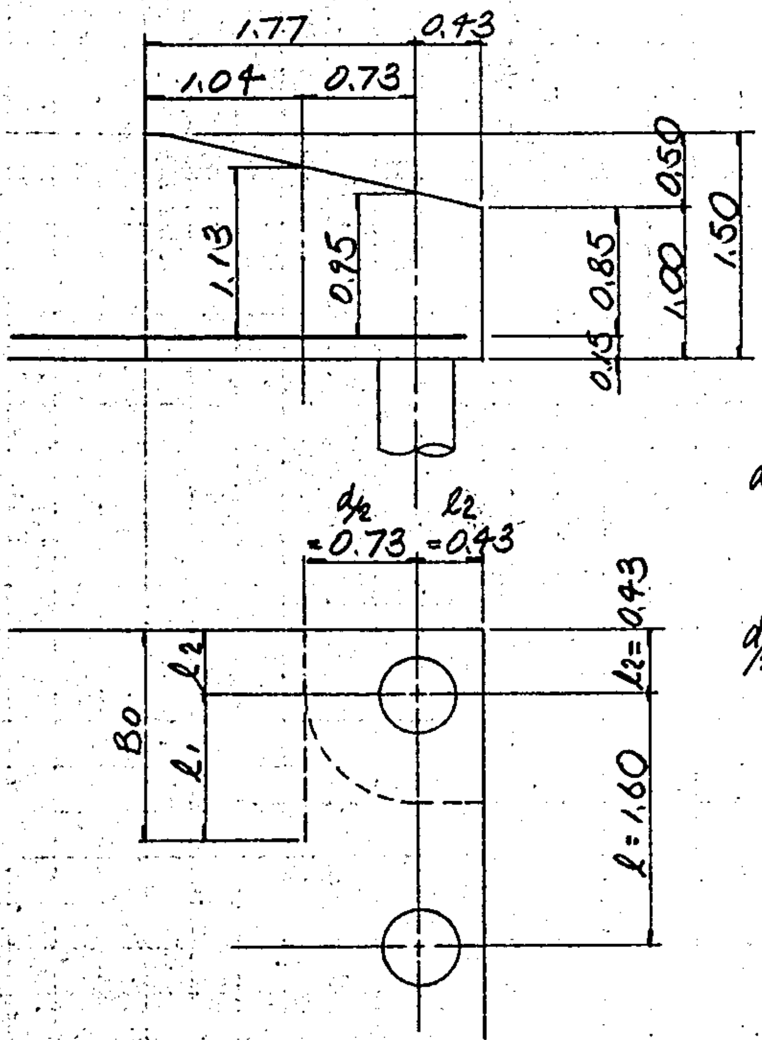
f. Summary of stresses

	pile reaction	own weight and surcharge	Earth pressure	surcharge load	ΣM	Value converted to ordinary case
D+S+EP	48.98	-31.63	-12.61	-2.42	2.32	1.55

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

(2) Analysis of shearing stress.

a. Effective width.



$d = D + d_1$

$= 0.50 + 0.95 = 1.45^m$

$d/2 = 1.45 \times 1/2 = 0.73^m$

(i) Effective area resisting bending shear.

$B_o = \ell_1 + \ell_2$

$= 0.6 \times 1.60 + 0.43 = 1.39^m$

(ii) Effective area resisting punching shear.

$b = 0.43 \times 2 + 1/2 \times 3.142 \times 0.73$

$= 2.01^m$

b. Reaction force acting on pile.

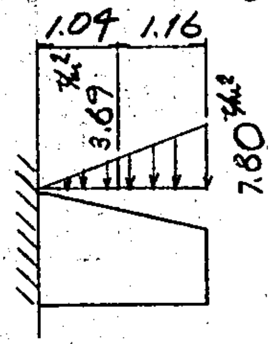
$D + S + Ep$

$P_{min} = 18.04^{t}$

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

$$N = 28.88 \times \frac{1.16}{2.20} \times 1.39 = 21.17^{t}$$

d. Vertical component of earth pressure.



$$P_v = 1/2 \times (3.69 + 7.80) \times 1.16 \times 1.39$$

$$= 9.26^{t}$$

e. Surcharge load

$$q_d = 1.00^{t/m^2} \times 1.16 \times 1.39 = 1.61^{t}$$

f. Shearing forse

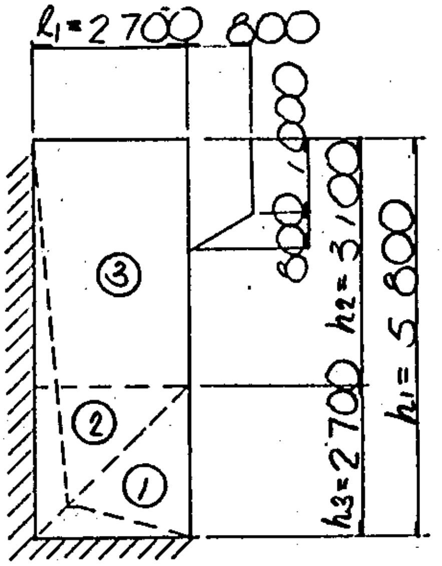
$$S = -18.04 + 21.17 + 9.26 + 1.61$$

$$= 14.00^{t}$$

g. Shearing stress.

$$\tau = \frac{14.00 \times 10^3}{139 \times 113} = 0.9^{kg/cm^2} < \tau_a = 3.7^{kg/cm^2}$$

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

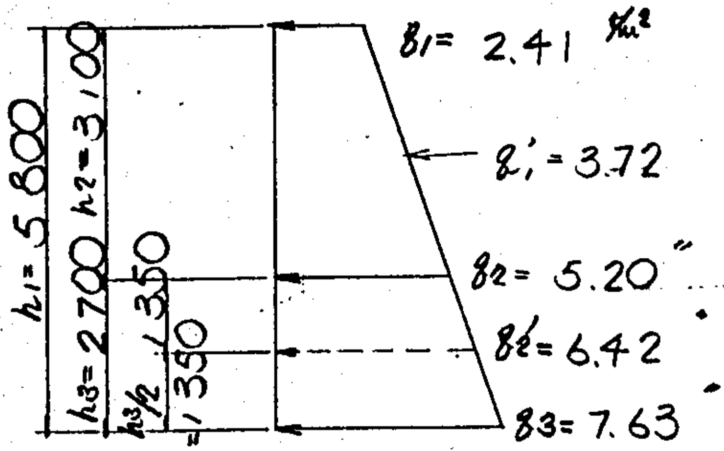
$K_0 = 0.5$

1. Earth pressure

$$q_1 = 1.8 \times 0.5 \times (\overset{h'}{2.68} + 0) = 2.41 \ t/m^2$$

$$q_2 = 1.8 \times 0.5 \times (\overset{h'}{2.68} + \overset{h_2}{3.10}) = 5.20 \ ''$$

$$q_3 = 1.8 \times 0.5 \times (\overset{h'}{2.68} + \overset{h_1}{5.80}) = 7.63 \ ''$$



$$q_2' = \frac{\overset{q_3}{7.63} - \overset{q_2}{5.20}}{2} + \overset{q_2}{5.20} = 6.42 \ t/m^2$$

$$q_1' = 1.8 \times 0.5 \times (2.68 + 1.80) = 4.03 \ ''$$

2. Stress calculation on each section.

① $M_1 = \frac{1}{2} \times \overset{\delta_2'}{6.42} \times \overset{h_3/2}{1.35}^2 + \frac{1}{6} \times (\overset{\delta_3}{7.63} - \overset{\delta_2'}{6.42}) \times \overset{h_3/2}{1.35}^2$

$= 6.22 \ \mathrm{t \cdot m/m}$

$S_1 = 6.42 \times 1.35 + \frac{1}{2} \times (7.63 - 6.42) \times 1.35$

$= 9.48 \ \mathrm{t/m}$

② $M_2 = \frac{1}{2} \times \overset{\delta_2'}{6.42} \times \overset{\ell_1/2}{1.35}^2 = 5.85 \ \mathrm{t \cdot m/m}$

$S_2 = 6.42 \times 1.35 = 8.67 \ \mathrm{t/m}$

③ $M_3 = \frac{1}{2} \times (\overset{\delta_1}{2.41} + \overset{\delta_2}{5.20}) \times \frac{1}{2} \times \overset{\ell_1}{2.70}^2$

$+ \overset{M_4}{1.03} + \overset{S_4}{2.58} \times \overset{\ell_1}{2.70} = 21.87 \ \mathrm{t \cdot m/m}$

$S_3 = (2.41 + 5.20) \times \frac{1}{2} \times 2.70 + \overset{S_4}{2.58}$

$= 12.85 \ \mathrm{t/m}$

④ $M_4 = \frac{1}{2} \times (\overset{\delta_1}{2.41} + \overset{\delta_1'}{4.03}) \times \frac{1}{2} \times 0.80^2 = 1.03 \ \mathrm{t \cdot m/m}$

$S_4 = (2.41 + 4.03) \times \frac{1}{2} \times 0.80 = 2.58 \ \mathrm{t/m}$

Stress calqulation.

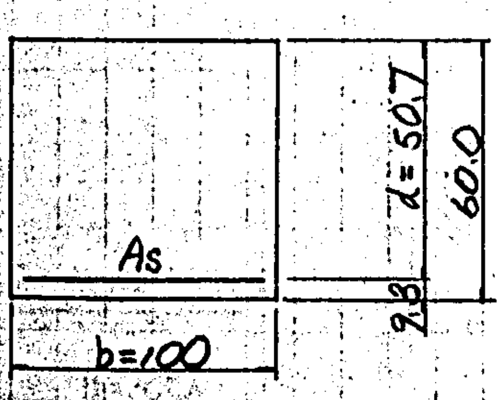
① $D + T + I + E_p \begin{cases} M = 6.22^{tm} \\ N = \text{———} \\ H = 9.48^{t} \end{cases}$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	6.22	Q (t)	0
N (t)	0	u (cm)	20.7
b (cm)	100		
d (cm)	50.7	d′(cm)	9.3
As (cm2)	D 16 X 4 = 7.944		
As′ (cm2)			
d′/d	0.18	f (cm)	0
M′ (t·m)	6.22		
M′/bd2(kg/cm2)	2.42	Q/bd (kg/cm2)	0
np	0.024	As′/As	0

C	S	Z
10.99	45.48	1.06

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
26.59 < 80	1,651.07 < 1800	0.00 < 4

$$\tau = \frac{9.48 \times 10^3}{100 \times 50.7} = 1.9^{kg/cm^2} < \tau_a = 3.7^{kg/cm^2}$$



As = D 16 - 25 cm ctc

Stress calculation.

③ $D + T + I + E_p$ $\begin{cases} M = 21.87\ t\cdot m \\ N = \text{—} \\ H = 12.85\ t \end{cases}$

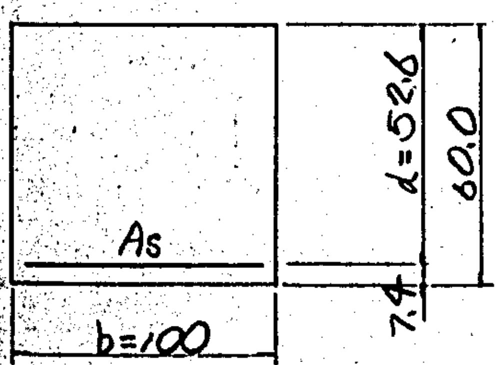
② omit

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	21.87	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)	21.87		
M'/bd2(kg/cm2)	7.9	Q/bd (kg/cm2)	0
nP	0.088	As'/As	0

C	S	Z
6.61	12.76	1.12

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
52.27 < 80	1,513.65 < 1800	0.00 < 4

$$\tau = \frac{12.85 \times 10^3}{100 \times 52.6} = 2.4\ kg/cm^2 < \tau_a = 3.7\ kg/cm^2$$



As = D 22 - 12.5 cm c.t.c

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 hole pile.

$$QF = \varphi \cdot \Sigma f \cdot h$$

QF untimate vertical supporting power of pile beared by skin friction.

φ : Circumferenfial length of pile.

f : Ultimate skin friction on earth layer.

$$f = 0.1 \cdot N$$
$$= 0.1 \times 25 \doteq 2.5\ ^t/m^2$$

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

31. Safe supporting of pile.

$$R_a = \frac{1}{F_S}(Q_p + Q_s)$$

R_a : Safe supportingpower of pile at pile top.
F_S : Safty factor corresponding to each loding condition.
Q_p : ultimate supporting power at pile tip.
Q_s : Skin friction of pile.

a) Ordinary

$F_S = 3$

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

b) Ordinary + Temporary

$F_S = 2$

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

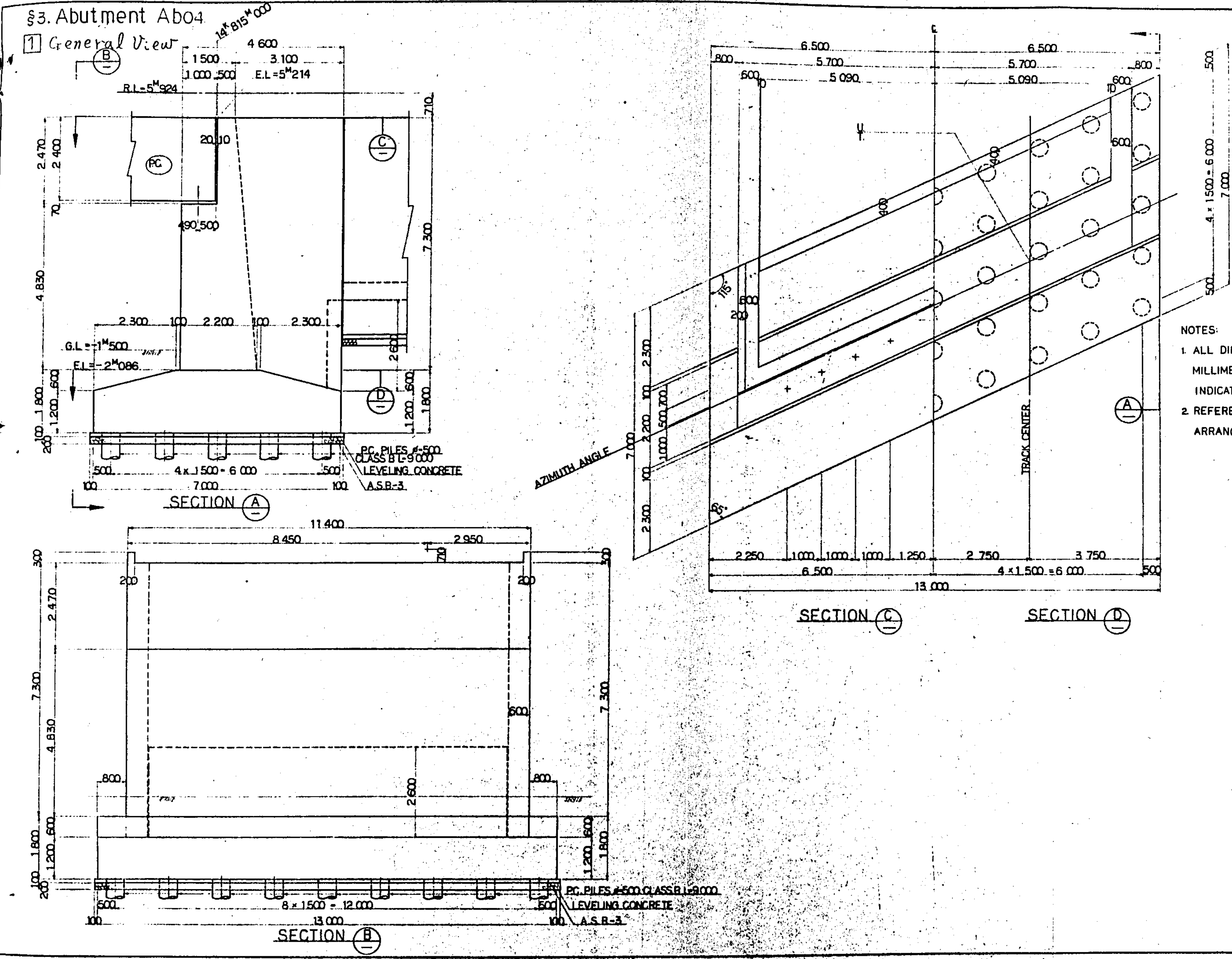
c) Earthquake (Kain load, seismic)

$F_S = 1.5 \ (1.2)$

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

§3. Abutment Ab04

[1] General View



NOTES:

1. ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE INDICATED
2. REFERENCE DRAWING FOR BAR ARRANGEMENT : CS - 138
   CS - 139
   CS - 140

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

$0.80^{t/m^2}$

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

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

$$b = 11.40^{m}$$

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

Height of surcharge load

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

For the design of breast wall

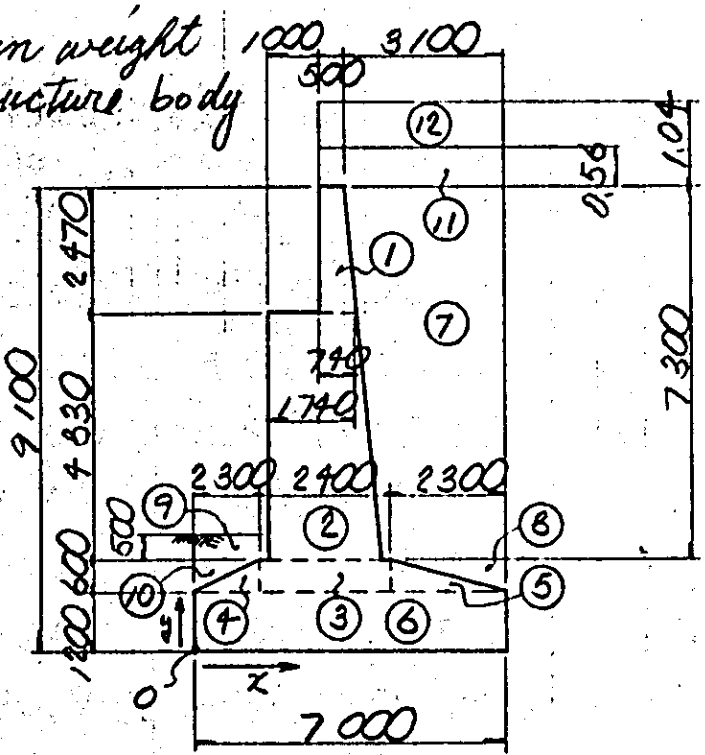
$$b = 2.00 + 2 \times 0.40 = 2.80^{m}$$

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

Height of surchargn load

$$h_2' = \frac{q_2'}{\gamma} = \frac{3.81}{1.8} = 2.17^{m}$$

2) Own weight of structure body



① $2.5\ t/m^3 \times (0.50 + 0.74) \times 2.47 \times 1/2 =$ (3.36) t/m 3.83

② $2.5 \times (1.74 + 2.20) \times 4.83 \times 1/2 =$ (20.86) 〃 23.79

③ $2.5 \times 2.40 \times 0.60 =$ 3.60 〃

④ $2.5 \times 2.30 \times 0.60 \times 1/2 =$ 1.73 〃

⑤ $2.5 \times 2.30 \times 0.60 \times 1/2 =$ 1.73 〃

⑥ $2.5 \times 7.00 \times 1.20 =$ 21.00 〃

⑦ $1.8 \times (3.10 + 2.40) \times 7.30 \times 1/2 =$ (31.69) 〃 36.14

⑧ $1.8 \times 2.30 \times 0.60 \times 1/2 =$ (1.09) 〃 1.24

⑨ $1.8 \times 2.40 \times 0.50 =$ 2.16 〃

⑩ $1.8 \times 2.30 \times 0.60 \times 1/2 =$ 1.24 〃

⑪ $0.56 \times 3.10 =$ (1.53) 〃 1.74

⑫ $1.04 \times 3.10 =$ (2.82) 〃 3.22

	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	3.36	3.71	12.47	0.34	7.79	2.65
②	20.86	3.39	70.72	2.09	4.12	8.61
③	3.60	3.50	12.60	0.36	1.50	0.54
④	1.73	1.53	2.65	0.17	1.40	0.24
⑤	1.73	5.47	9.46	0.17	1.40	0.24
⑥	21.00	3.50	73.50	2.10	0.60	1.26
⑦	31.69	4.74	150.21	3.17	5.60	17.75
⑧	1.09	6.23	6.79	0.11	1.60	0.18
⑨	2.16	1.20	2.59	0.22	2.05	0.45
⑩	1.24	0.77	0.95	0.12	1.60	1.45
⑪	1.53	5.45	8.34	0.15	9.35	1.40
小計	89.99	—	350.28	9.00	—	33.51
⑫	2.82	5.45	15.37	—	—	—
計	92.81	—	365.65	—	—	—

$K_H = 0.10$

3) Loads acting on shoes.

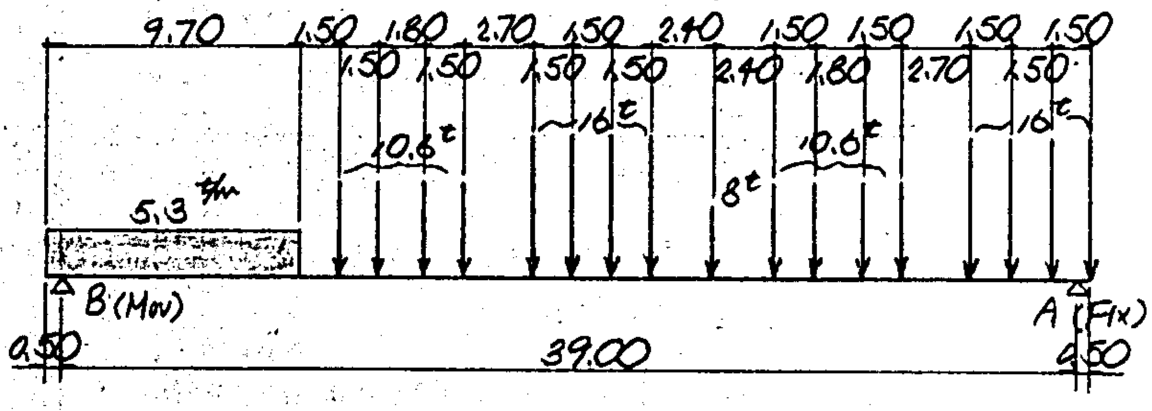
(1) Vertical load on shoes.

a. dead load.
dead load of superstructure; Rd

Refer The Table, Reaction fore of superstructure)

$$Rd = 447.0^{t} \times 2 \quad \left(Rd = \frac{894.0}{11.40} = \begin{matrix}(68.77)\\ 78.42\end{matrix} \; t/m \right)$$
$$= 894.0^{t}$$

b. Train load
reaction force of abutment caused by
K.S loading.



$$R_{\ell A}=\frac{1}{39.00}\times\{16\times(18.2+19.7+21.2+22.7+35.0+36.5+38.0+39.5)+10.6\times(10.7+12.2+14.0+15.5+27.5+29.0+30.8+32.3)+8\times25.1+5.3\times(9.2^2\times1/2-0.5^2\times1/2)\}=152.32^{t}$$

$$R_{\ell B}=(16\times8+10.6\times8+8+5.3\times9.7)-152.32=119.89''$$

Impact Coefficient. (ℓ = 39.00 m)

$$i=0.34-\frac{39.00-30}{40-30}\times(0.34-0.32)$$

$$=0.322$$

$$\therefore R_{A(\ell+i)}=152.32\times(1+0.322)\times2$$

$$=402.73^{t}$$

$$\left(R_{A(\ell+i)}=\frac{402.73}{11.40}=\begin{matrix}(30.98)\\35.33\end{matrix}\ t/m\right)$$

(2) Horizontal load acting at shoes.

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

Brake load is assumed as 15% of train load.

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

$$= 0.15 \times (\overset{R_{\ell A}}{152.32} + \overset{R_{\ell B}}{119.89}) = 40.83^t$$

Brake load acting at fixed end shoe ; TA

$$T_A = T - \tfrac{1}{2} \cdot \mu \, (R_A + R_\ell)$$

$$= 40.83 - \tfrac{1}{2} \times 0.10 \times (447.0 + 152.32)$$

$$= 10.86^t \quad T/2 = 40.83 \times 1/2 = 20.42^t$$

$$\therefore T_A = 20.42^t$$

Per one meter width of abutment.

$$T_A = \frac{20.42}{11.40} = \underset{1.79}{(1.57)}\ t/m$$

(3) Seismic load.

$$K_H = 0.10$$

Fixed end support bearing.

$$H = 0.10 \times \Sigma R_d$$

$$= 0.10 \times 894.0 \times 2 \qquad = 178.80^t$$

$$H_A = 178.80 - \tfrac{1}{2} \times 0.10 \times 894.0$$

$$= 134.10^t \quad H/2 = 178.80 \times 1/2 = 89.40^t$$

$$\therefore H_A = \frac{134.10}{11.40} = \underset{11.76}{(10.31)}\ t/m$$

4) Loads acting brest wall

(1) Dead load

Distributed dead load $q_1 = 1.00$ t/m²

Vertical force perone meter width of abutment. $D = 1.00 \times 0.50 = 0.50$ (0.44) t/m

(2) Train load

a. stability calculation

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

Vestical force perone meter width of abutment $L = 1.87 \times 0.50 = 0.94$ (0.82) t/m

b. desingn of breast wall

Vertical force force perone meter width of obutment

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

5) Earth pressure

(1) Ordinary

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

γ; Unit weght of soil (t/m^3)

h; Height of acting point of soil. (m)

h'; Eearth equivalent of surcharge. (m)

Ka; Coefficient of active earth pressure.

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

$$P = \frac{1}{2} \times 1.8 \times (9.10 + 2 \times 0.56) \times 9.10 \times 0.297$$

$$= 24.86^{t}$$

$$P_H = 24.86 \times \cos 30^\circ = \begin{matrix}(18.88)\\21.53\end{matrix}^{t} \quad (\leftarrow)$$

$$P_V = 24.86 \times \sin 30^\circ = \begin{matrix}(10.90)\\12.43\end{matrix}^{\prime\prime} \quad (\downarrow)$$

Coordinate of acting point. $x = 7.00^{m}$

$y = 3.20$ 〃

(2) Ordinary + Temporary

$$P = \frac{1}{2} \times 1.8 \times \{9.10 + 2 \times (1.04 + 0.56)\} \times 9.10$$

$$\times 0.297 = 29.92^{t}$$

$$P_H = 29.92 \times \cos 30^\circ = \begin{matrix}(22.72)\\25.91\end{matrix}^{t} \quad (\leftarrow)$$

$$P_V = 29.92 \times \sin 30^\circ = \begin{matrix}(13.12)\\14.96\end{matrix}^{\prime\prime} \quad (\downarrow)$$

Coordinate of acting point $x = 7.00^{m}$

$y = 3.42$ 〃

(3) Earthquake

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

$P = 1/2 \times 1.8 \times (9.10 + 2 \times 0.56) \times 9.10 \times 0.354$

$= 29.63^{t}$

$P_H = 29.63 \times \cos 30^\circ = \overset{(22.50)}{25.66}{}^{t}$ $(\leftarrow)$

$P_V = 29.63 \times \sin 30^\circ = \overset{(13.00)}{14.82}$ 〃 $(\downarrow)$

Coordinate of acting point $x = 7.00^{m}$

$y = 3.20$

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	89.99	—	350.28	—	—	—
Dead load acting at shoes	68.77	2.90	199.43	—	—	—
Dead load acting at breast wall	0.44	3.65	1.61	—	—	—
Active earth pressure	10.90	7.00	76.30	-18.88	3.20	-60.42
Total	170.10	—	627.62	-18.88	—	-60.42

Stress at Point O

$N = 170.10^{t}$

$H = -18.88$ "

$M = 627.62 - 60.42 = 567.20^{t\cdot m}$

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	92.81	—	365.55	—	—	—
Dead load acting at shoes	68.77	2.90	199.43	—	—	—
Train load acting at shoes	30.98	2.90	89.84	—	—	—
Dead load acting at breast wall	0.44	3.65	1.61	—	—	—
Train load acting at breast wall	0.82	3.65	2.99	—	—	—
Active earth pressure	13.12	7.00	91.84	-22.72	3.42	-77.70
Total	206.94	—	751.36	-22.72	—	-77.70

$N = 206.94^{t}$, $H = -22.72^{t}$

$M = 751.36 - 77.70 = 673.66^{t\cdot m}$

3. Dead load + Train load + Impact load + Earthpressure + Brake 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	206.94	—	751.36	-22.72	—	-77.70
B	—	—	—	-1.57	6.63	-10.41
Total	206.94	—	751.36	-24.29	—	-88.11

$N = 206.94^{t}$

$H = -24.29$ "

$M = 751.36 - 88.11 = 663.25^{t \cdot m}$

4. Dead load + Earthpressure + 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	89.99	—	350.28	-9.00	—	-33.51
Dead load acting at shoes	68.77	2.90	199.43	-10.31	6.63	-68.36
Dead load acting at breast wall	0.44	3.65	1.61	-0.04	9.38	-0.37
Active earth pressure	13.00	7.00	91.00	-22.50	3.20	-72.00
Total	172.20	—	642.32	-41.85	—	-174.24

$N = 172.20^{t}$

$H = -41.85$ "

$M = 642.32 - 174.24 = 468.08^{t \cdot m}$

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 $n = 45$ pile

Coordinate of acting point $x = 3.50$ m

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

$$I = 9 \times (3.00^2 + 1.50^2) \times 2 = 202.50 \text{ Pile}\cdot m^2$$

(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)

$$e = \frac{M}{N} = \frac{567.20}{170.10} = 3.33 \text{ m}$$

$$e_o = 3.50 - 3.33 = 0.17 \text{ ″}$$

$$P = \frac{170.10}{45} \pm \frac{170.10 \times 0.17}{202.50} \times 3.00 = 3.78 \pm 0.43$$

$$= \begin{cases} 4.21 & \text{t/pile} \ (P_{max} = 54.73 \text{ t/pile}) \\ 3.35 & \text{″} \ (P_{min} = 43.55 \text{ ″}) \end{cases}$$

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

$$e = \frac{673.66}{206.94} = 3.26^{m}$$

$$e_0 = 3.50 - 3.26 = 0.24^{m}$$

$$P = \frac{206.94}{45} \pm \frac{206.94 \times 0.24}{202.50} \times 3.00$$

$$= 4.60 \pm 0.74$$

$$= \begin{cases} 5.34 \ {}^{t}/\text{pile} & (P_{max} = 69.42 \ {}^{t}/\text{pile}) \\ 3.86 \ '' & (P_{min} = 50.18 \ '') \end{cases}$$

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

$$e = \frac{663.25}{206.94} = 3.21^{m}$$

$$e_0 = 3.50 - 3.21 = 0.29^{m}$$

$$P = \frac{206.94}{45} \pm \frac{206.94 \times 0.29}{202.50} \times 3.00$$

$$= 4.60 \pm 0.89$$

$$= \begin{cases} 5.49 \ {}^{t}/\text{pile} & (P_{max} = 71.37 \ '') \\ 3.71 \ '' & (P_{min} = 48.23 \ '') \end{cases}$$

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

$$e = \frac{468.08}{172.20} = 2.72^{m}$$

$$e_0 = 3.50 - 2.72 = 0.78^{m}$$

$$P = \frac{172.20}{45} \pm \frac{172.20 \times 0.78}{202.50} \times 3.00$$

$$= 3.83 \pm 1.99$$

$$= \begin{cases} 5.82 \ {}^{t}/\text{pile} & (P_{max} = 66.35 \ {}^{t}/\text{pile}) \\ 1.84 \ '' & (P_{min} = 20.98 \ '') \end{cases}$$

3. Stability against overturning

a. D+EP

$$\frac{P_{min}}{P_{max}} = \frac{43.55}{54.73} = 0.80 > 0.3$$

b. D+T+I+EP+Br

$$\frac{P_{min}}{P_{max}} = \frac{50.18}{69.42} = 0.72 > 0.1$$

c. D+T+I+EP+Br

$$\frac{P_{min}}{P_{max}} = \frac{48.23}{71.37} = 0.68 > 0.1$$

d. D+EP+Se

$$\frac{M}{N} = \frac{134.32}{172.20} = 0.78 < 3.00^{m}$$

4. Stability against vertical Support

a. D+EP

$$P_{max} = 54.73^{\text{Pile}} < P_a = 58^{\text{Pile}}$$

b. D+T+I+EP+Br

$$P_{max} = 69.42^{\text{Pile}} < P_a = 88^{\text{Pile}}$$

c. D+T+I+EP+Br

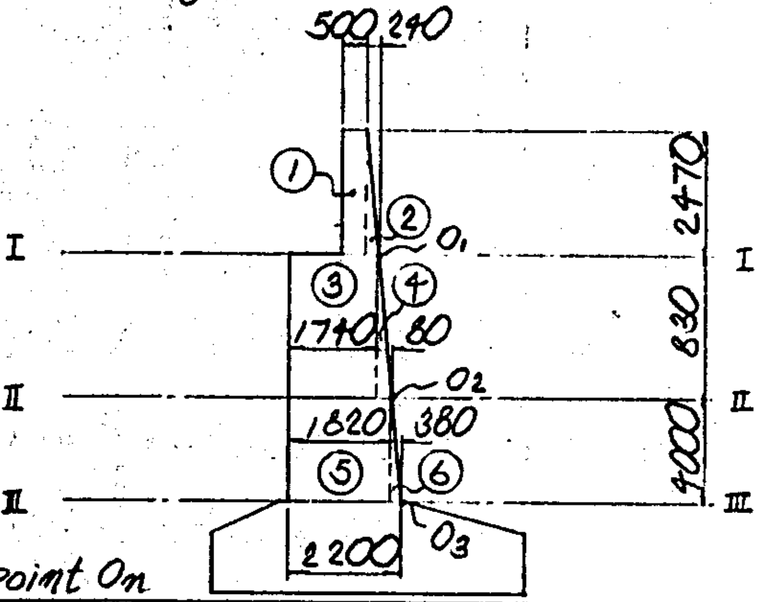
$$P_{max} = 71.37^{\text{Pile}} < P_a = 88^{\text{Pile}}$$

d. D+EP+Se

$$P_{max} = 66.35^{\text{Pile}} < P_a = 117^{\text{Pile}}$$

4 Stress calculation of abutment body

1. Own weight of sufs tructure and center of gravity calculated by sections.



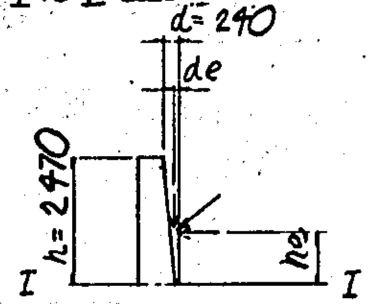
Stress at point On

			N (t)	x (m)	N·x (t·m)	H (t)	y (m)	H·y (t·m)
I ~ I SECTION	①	2.5× 0.50 × 2.47	3.09	0.49	1.51	0.31	1.24	0.38
	②	2.5× 0.24 ×2.47×1/2	0.74	0.16	0.12	0.07	0.82	0.06
	TOTAL		3.83	—	1.63	0.38	—	0.44
II ~ II SECTION	①	—	3.09	0.57	1.76	0.31	2.07	0.64
	②	—	0.74	0.24	0.18	0.07	1.65	0.12
	③	2.5 × 1.74 × 0.83	3.61	0.95	3.43	0.36	0.42	0.15
	④	2.5 × 0.08 × 0.83×1/2	0.08	0.05	0.01	0.01	0.28	0.01
	TOTAL		7.52	—	5.38	0.75	—	0.92
III ~ III SECTION	①	—	3.09	0.95	2.94	0.31	6.07	1.88
	②	—	0.74	0.62	0.46	0.07	5.65	0.40
	③	—	3.61	1.33	4.80	0.36	4.42	1.59
	④	—	0.08	0.43	0.03	0.01	4.28	0.04
	⑤	2.5× 1.82 × 4.00	18.20	1.29	23.48	1.82	2.00	3.64
	⑥	2.5× 0.38 × 4.00×1/2	1.90	0.25	0.48	0.19	1.33	0.25
	TOTAL		27.62	—	32.19	2.76	—	7.80

2. Earth presser and its acting point.

(1) Ordinary

a. I ~ I SECTION



$h' = 0.56$, $h = 2.47$

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

$Ka = 0.320$

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

$Pa = 1/2 \times 1.8 \times (2.47 + 2 \times 0.56) \times 2.47 \times 0.320 = 2.55$ t/m

$P_H = 2.55 \times \cos(2.579^\circ + 15^\circ) = 2.43$ t/m ($\leftarrow$)

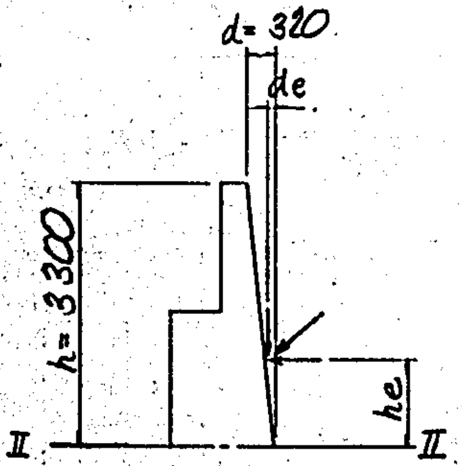
$P_V = 2.55 \times \sin(2.579^\circ + 15^\circ) = 0.77$ " ($\downarrow$)

Acting point

$$he = \frac{h}{3} \times \frac{h + 3h'}{h + 2h'} = \frac{2.47}{3} \times \frac{2.47 + 3 \times 0.56}{2.47 + 2 \times 0.56} = 0.95 \text{ m}$$

$$de = \frac{d}{h} \times he = \frac{0.24}{2.47} \times 0.95 = 0.09 \text{ m}$$

b. II ~ II section



$h' = 0.56$, $h = 3.30$

$Pa = 1/2 \times 1.8 \times (3.30 + 2 \times 0.56)$

$\times 3.30 \times 0.320 = 4.20$ t/m

$P_H = 4.20 \times \cos(2.579^\circ + 15^\circ) = 4.00$ t/m ($\leftarrow$)

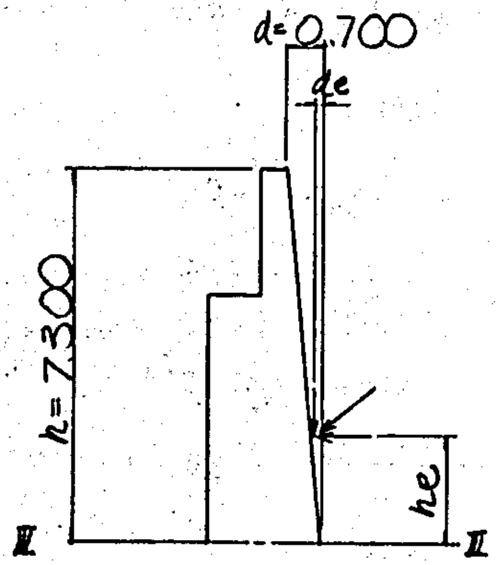
$P_V = 4.20 \times \sin(2.579^\circ + 15^\circ) = 1.27$ " ($\downarrow$)

Acting point

$$h_e = \frac{3.30}{3} \times \frac{3.30 + 3 \times 0.56}{3.30 + 2 \times 0.56} = 1.24 \ \mathrm{m}$$

$$d_e = \frac{0.32}{3.30} \times 1.24 = 0.12 \ \mathrm{m}$$

C. III ~ III Section



$h' = 0.56$

$h = 7.30$

$$P_a = \tfrac{1}{2} \times 1.8 \times (7.30 + 2 \times 0.56) \times 7.30 \times 0.320 = 17.70 \ \mathrm{t/m}$$

$$P_H = 17.70 \times \cos(2.579^\circ + 15^\circ) = 16.87 \ \mathrm{t/m} \ (\leftarrow)$$

$$P_V = 17.70 \times \sin(2.579^\circ + 15^\circ) = 5.35 \ '' \ (\downarrow)$$

Acting point

$$h_e = \frac{7.30}{3} \times \frac{7.30 + 3 \times 0.56}{7.30 + 2 \times 0.56} = 2.60 \ \mathrm{m}$$

$$d_e = \frac{0.70}{7.30} \times 2.60 = 0.25 \ \mathrm{m}$$

(2) Ordinary + Temporary

a. I ~ I Section

$h' = 2.17 + 0.56$, $h = 2.47$

$K_a = 0.320$

$P_a = 1/2 \times 1.8 \times \{2.47 + 2 \times (2.17 + 0.56)\} \times 2.47$

$\times 0.320 = 5.64$ t/m

$P_H = 5.64 \times \cos(2.579° + 15°) = 5.38$ t/m (←)

$P_V = 5.64 \times \sin(2.579° + 15°) = 1.70$ 〃 (↓)

Actingpoint

$h_e = \frac{2.47}{3} \times \frac{2.47 + 3 \times 2.73}{2.47 + 2 \times 2.73} = 1.11$ m

$d_e = \frac{0.24}{2.47} \times 1.11 = 0.11$ m

b. II ~ II Section

$h' = 2.17 + 0.56$, $h = 3.30$

$P_a = 1/2 \times 1.8 \times \{3.30 + 2 \times (2.17 + 0.56)\} \times 3.30$

$\times 0.320 = 8.33$ t/m

$P_H = 8.33 \times \cos(2.579° + 15°) = 7.94$ t/m (←)

$P_V = 8.33 \times \sin(2.579° + 15°) = 2.52$ 〃 (↓)

Actingpoint

$h_e = \frac{3.30}{3} \times \frac{3.30 + 3 \times 2.73}{3.30 + 2 \times 2.73} = 1.44$ m

$d_e = \frac{0.32}{3.30} \times 1.44 = 0.14$ m

c. III-III Section

$h' = 2.17 + 0.56 \quad h = 7.30$

$Pa = 1/2 \times 1.8 \times \{7.30 + 2\times(2.17 + 0.56)\} \times 7.30$

$\times 0.320 = 26.83 \ ^{t/m}$

$PH = 26.83 \times \cos(2.579^\circ + 15^\circ) = 25.58 \ ^{t/m} \quad (\leftarrow)$

$PV = 26.83 \times \sin(2.579^\circ + 15^\circ) = 8.10 \ '' \quad (\downarrow)$

Acting point

$he = \frac{7.30}{3} \times \frac{7.30 + 3\times 2.73}{7.30 + 2\times 2.73} = 2.95 \ ^{m}$

$de = \frac{0.70}{7.30} \times 2.95 = 0.28 \ ^{m}$

(3) Earthquake

a. I~I Section

$h' = 0.56$, $h = 2.47$

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

$K_E = 0.401$

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

$= 1/2 \times 1.8 \times (2.47 + 2 \times 0.56) \times 2.47 \times 0.401$

$= 3.20$ t/m

$P_{EH} = 3.20 \times \cos 2.579° = 3.20$ t/m (←)

$P_{EV} = 3.20 \times \sin 2.579° = 0.14$ " (↓)

Acting point

$he = \dfrac{2.47}{3} \times \dfrac{2.47 + 3 \times 0.56}{2.47 + 2 \times 0.56} = 0.95$ m

$de = \dfrac{0.24}{2.47} \times 0.95 = 0.09$ m

b. II~II Section

$h' = 0.56$, $h = 3.30$

$P_E = 1/2 \times 1.8 \times (3.30 + 2 \times 0.56) \times 3.30 \times 0.401$

$= 5.26$ t/m

$P_{EH} = 5.26 \times \cos 2.579° = 5.25$ t/m (←)

$P_{EV} = 5.26 \times \sin 2.579° = 0.24$ " (↓)

Acting point

$he = \dfrac{3.30}{3} \times \dfrac{3.30 + 3 \times 0.56}{3.30 + 2 \times 0.56} = 1.24$ m

$de = \dfrac{0.32}{3.30} \times 1.24 = 0.12$ m

c. III ~ III Section

$h' = 0.56 \quad , \quad h = 7.30$

$P_E = 1/2 \times 1.8 \times (7.30 + 2 \times 0.56) \times 7.30 \times 0.401$

$= 22.18 \ ^{t/m}$

$P_{EH} = 22.18 \times \cos 2.579° = 22.16 \ ^{t/m} \ (\leftarrow)$

$P_{EV} = 22.18 \times \sin 2.579° = 1.00 \ '' \ (\downarrow)$

Acting point

$$he = \frac{7.30}{3} \times \frac{7.30 + 3 \times 0.56}{7.30 + 2 \times 0.56} = 2.60 \ ^{m}$$

$$de = \frac{0.70}{7.30} \times 2.60 = 0.25 \ ^{m}$$

3. Stress calculation on each section

(1) I - I Section

(B = 1^M)

		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	3.83	—	1.63	0.38	—	0.44
②	Dead load acting at breast wall	0.50	0.49	0.25	0.05	2.75	0.14
③	Train load and Impact load acting at breast wall	5.71	0.49	2.80	—	—	—
④	Dead load acting at shues	—	—	—	—	—	—
⑤	Train load and Impact load acting at shues	—	—	—	—	—	—
⑥	Brake load	—	—	—	—	—	—
⑦	Earth pressure (ordinary)	0.77	0.09	0.07	2.43	0.95	2.31
⑧	Earth pressure (Ordinary + Temporary)	1.70	0.11	0.19	5.38	1.11	5.97
⑨	Earth pressure (earth quake)	0.14	0.09	0.01	3.20	0.95	3.04
case 1	D + Ep	5.10	—	1.95	2.43	—	2.31
2	D+T+I+Ep	11.74	—	4.87	5.38	—	5.97
3	D+T+I+Ep+B	11.74	—	4.87	5.38	—	5.97
4	D+Ep (S)	4.47	—	1.89	3.63	—	3.62

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

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

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

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

(2) II - II 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	7.52	—	5.38	0.75	—	0.92
②	Dead load acting at breast wall	0.50	0.57	0.29	0.05	3.58	0.18
③	Train load and Impact load acting at breast wall	5.71	0.57	3.25	—	—	—
④	Dead load acting at shues	78.42	1.32	103.51	11.76	0.83	9.76
⑤	Train load and Impact load acting at shues	35.33	1.32	46.64	—	—	—
⑥	Brake load	—	—	—	1.79	0.83	1.49
⑦	Earth pressure (ordinary)	1.27	0.12	0.15	4.00	1.24	4.96
⑧	Earth pressure (Ordinary + Temporary)	2.52	0.14	0.35	7.94	1.44	11.43
⑨	Earth pressure (earth quake)	0.24	0.12	0.03	5.25	1.24	6.51
Case 1	D + Ep	87.71	—	109.33	4.00	—	4.96
2	D+T+I+Ep	130.00	—	159.42	7.94	—	11.43
3	D+T+I+Ep+B	130.00	—	159.42	9.73	—	12.92
4	D+Ep (S)	86.68	—	109.21	17.81	—	17.37

(3) III - III Section

	Vertical force (t) N	Horizontal distance x (m)	$N \cdot x$ (t·m)	Horizontal forse (t) H	Vertical distance y (m)	$H \cdot y$ (t·m)
① Own weight of structure body	27.62	—	32.19	2.76	—	7.80
② Dead load acting at breast wall	0.50	0.95	0.48	0.05	7.58	0.38
③ Train load and Impact load acting at breast wall	5.71	0.95	5.42	—	—	—
④ Dead load acting at shues	78.42	1.70	133.31	11.76	4.83	56.80
⑤ Train load and Impact load acting at shues	35.33	1.70	60.06	—	—	—
⑥ Brake load	—	—	—	1.79	4.83	8.65
⑦ Earth pressure (ordinary)	5.35	0.25	1.34	16.87	2.60	43.86
⑧ Earth pressure (Ordinary + Temporary)	8.10	0.28	2.27	25.58	2.95	75.46
⑨ Earth pressure (earth quake)	1.00	0.25	0.25	22.16	2.60	57.62
Case 1. D + Ep	111.89	—	167.32	16.87	—	43.86
2 D+T+I+Ep	149.01	—	233.73	25.58	—	75.46
3 D+T+I+Ep+B	149.01	—	233.73	27.37	—	84.11
4 D+Ep (S)	107.54	—	166.23	36.73	—	122.60

Stress calculation I - I Section

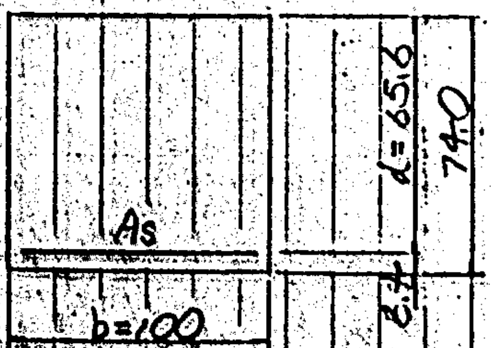
$$D + T + I + Ep \quad \begin{cases} M = 4.87 + 5.97 = 10.84 \ t\cdot m \\ N = 11.74 \ t \\ H = 5.38 \ t \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	10.84	Q (t)	5.3
N (t)	11.74	u (cm)	28.6
b (cm)	100		
d (cm)	65.6	d'(cm)	8.4
As (cm2)	D 13 % 8 = 10.136		
As' (cm2)			
d'/d	0.13	f (cm)	120.93
M' (t·m)	14.2		
M'/bd2(kg/cm2)	3.3	Q/bd (kg/cm2)	0.82
nP	0.023	As'/As	0

C	S	Z
8.40	23.87	1.05

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
27.74 < 80	1,181.55 < 1800	0.86 < 4

$$\tau = \frac{5.38 \times 10^3}{100 \times 65.6} = 0.8 \ kg/cm^2 < \tau_R = 3.7 \ kg/cm^2$$



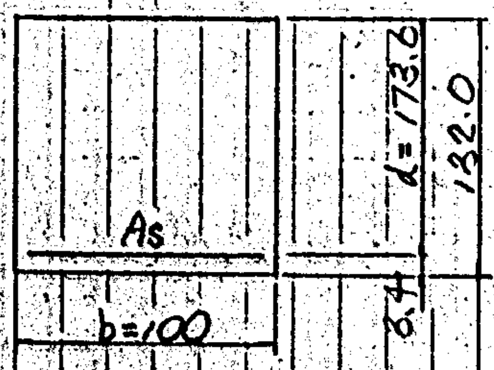
As = D 13 - 12.5 cm ctc

Stress calculation I-I Section

$$D + T + I + E_p \begin{cases} M = 159.42 + 11.43 = 170.85^{t\cdot m} \\ N = 132.00^{t} \\ H = 7.94^{t} \end{cases}$$

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

M (t·m)            170.85                       Q (t)         7.9
N (t)              130                          u (cm)        82.6
b    (cm)          100
d    (cm)          173.6                        d'(cm)        8.4

As   (cm2)         D 32 X 4 = 31.768
As'  (cm2)
d'/d               0.05                         f (cm)        214.02
M' (t·m)           278.23
M'/bd2(kg/cm2)     9.23                         Q/bd (kg/cm2) 0.46

nP                 0.027                        As'/As        0

            C                    S                     Z
          6.41                 11.75                  1.00

      Σc(kg/cm2)           Σs(kg/cm2)            Tc(kg/cm2)
      59.25 < 80       1,627.61 < 1800           0.45 < 4
```

$$\tau = \frac{7.94 \times 10^3}{100 \times 173.6} = 0.5 \ \mathrm{kg/cm^2} < \tau_R = 3.7 \ \mathrm{kg/cm^2}$$



As = D 32 - 25.0 cm ctc

Stress calculation I-I Section

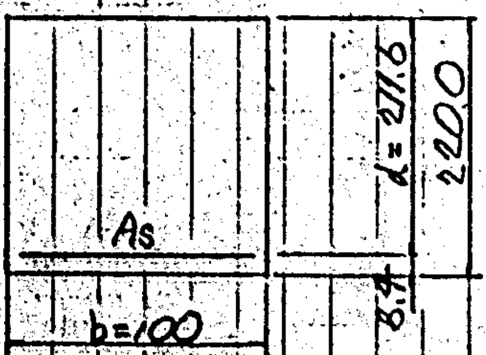
$$D + T + I + Ep \left\{ \begin{array}{l} M = 233.73 + 75.46 = 309.19^{t \cdot m} \\ N = 149.01^{t} \\ H = 25.58^{t} \end{array} \right.$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	309.19	Q (t)	25.5
N (t)	149.01	u (cm)	101.6
b (cm)	100		
d (cm)	211.6	d′(cm)	8.4
As (cm2)	D 32 X 8 = 63.536		
As′ (cm2)			
d′/d	0.04	f (cm)	309.1
M′ (t·m)	460.58		
M′/bd2(kg/cm2)	10.29	Q/bd (kg/cm2)	1.21
nP	0.045	As′/As	0

C	S	Z
6.09	10.17	1.05

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
62.73 < 80	1,570.33 < 1800	1.27 < 4

$$\tau = \frac{25.58 \times 10^3}{100 \times 211.6} = 1.2 \ kg/cm^2 < \tau_R = 3.7 \ kg/cm^2$$



As = D 32 - 12.5 cm etc

Stress calculation III-III section for crack analysis.

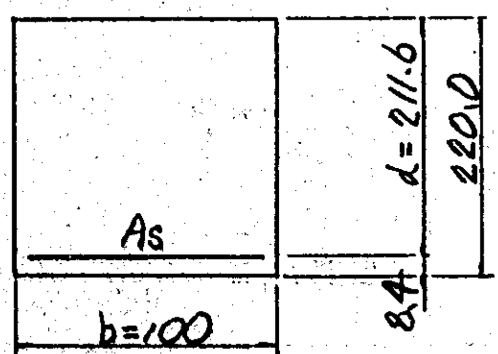
D + Ep

$$\begin{cases} M = 167.32 + 43.86 = 211.18 \ t\cdot m \\ N = 111.89 \ t \\ H = 16.87 \ '' \end{cases}$$

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1400
Tca (kg/cm2)	3.7		
M (t・m)	211.18	Q (t)	16.8
N (t)	111.89	u (cm)	101.6
b (cm)	100		
d (cm)	211.6	d′(cm)	8.4
As (cm2)	D 32 X 8 = 63.536		
As′ (cm2)			
d′/d	0.04	f (cm)	290.34
M′ (t・m)	324.86		
M′/bd2(kg/cm2)	7.26	Q/bd (kg/cm2)	0.8
nP	0.045	As′/As	0

C	S	Z
5.91	9.31	1.04

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
42.92 < 80	1,014.20 < 1400	0.83 < 4



As = D 32 - c.t.c 12.5cm

5 Design of foundation

1. Reaction force Actieng on piles

Calculation of reaction force to be used for calculation of Piles and foundation

$$P = \underbrace{\frac{N}{n} \pm \frac{M}{I} \cdot x}_{\text{Refer The article of stability calculution}} \pm (\Delta N)$$

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

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

(1) D+EP

$$\Sigma M_0 = \frac{H}{2 \cdot \beta} = \frac{18.88 \times 13.00}{2 \times 1.77} = 693.33 \ \text{t.m}$$

$$P = \begin{Bmatrix} 54.73 \\ 43.55 \end{Bmatrix} \pm \frac{693.33}{202.50} \times 3.00$$

$$= \begin{Bmatrix} 54.73 \\ 43.55 \end{Bmatrix} \pm 10.27 = \begin{Bmatrix} 65.00 \\ 33.28 \end{Bmatrix} \ \text{t/pile}$$

(2) D+T+I+Ep

$$\Sigma M_0 = \frac{22.72 \times 13.00}{2 \times 2.14} = 690.09 \ \text{t.m}$$

$$P = \begin{Bmatrix} 69.42 \\ 50.18 \end{Bmatrix} \pm \frac{690.09}{202.50} \times 3.00$$

$$= \begin{Bmatrix} 69.42 \\ 50.18 \end{Bmatrix} \pm 10.22 = \begin{Bmatrix} 79.64 \\ 39.96 \end{Bmatrix} \ \text{t/pile}$$

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

$$\Sigma M_0 = \frac{24.29 \times 13.00}{2 \times 0.214} = 737.78 \ \text{t.m}$$

$$P = \begin{Bmatrix} 71.37 \\ 48.23 \end{Bmatrix} \pm \frac{737.78}{202.50} \times 3.00$$

$$= \begin{Bmatrix} 71.37 \\ 48.23 \end{Bmatrix} \pm 10.93 = \begin{Bmatrix} 82.30 \\ 37.30 \end{Bmatrix} \ \text{t/pile}$$

(4) Dead load + Earth pressure (seismic)

$$\Sigma M_0 = \frac{41.85 \times 13.00}{2 \times 0.214} = 1271.14 \ ^{t \cdot m}$$

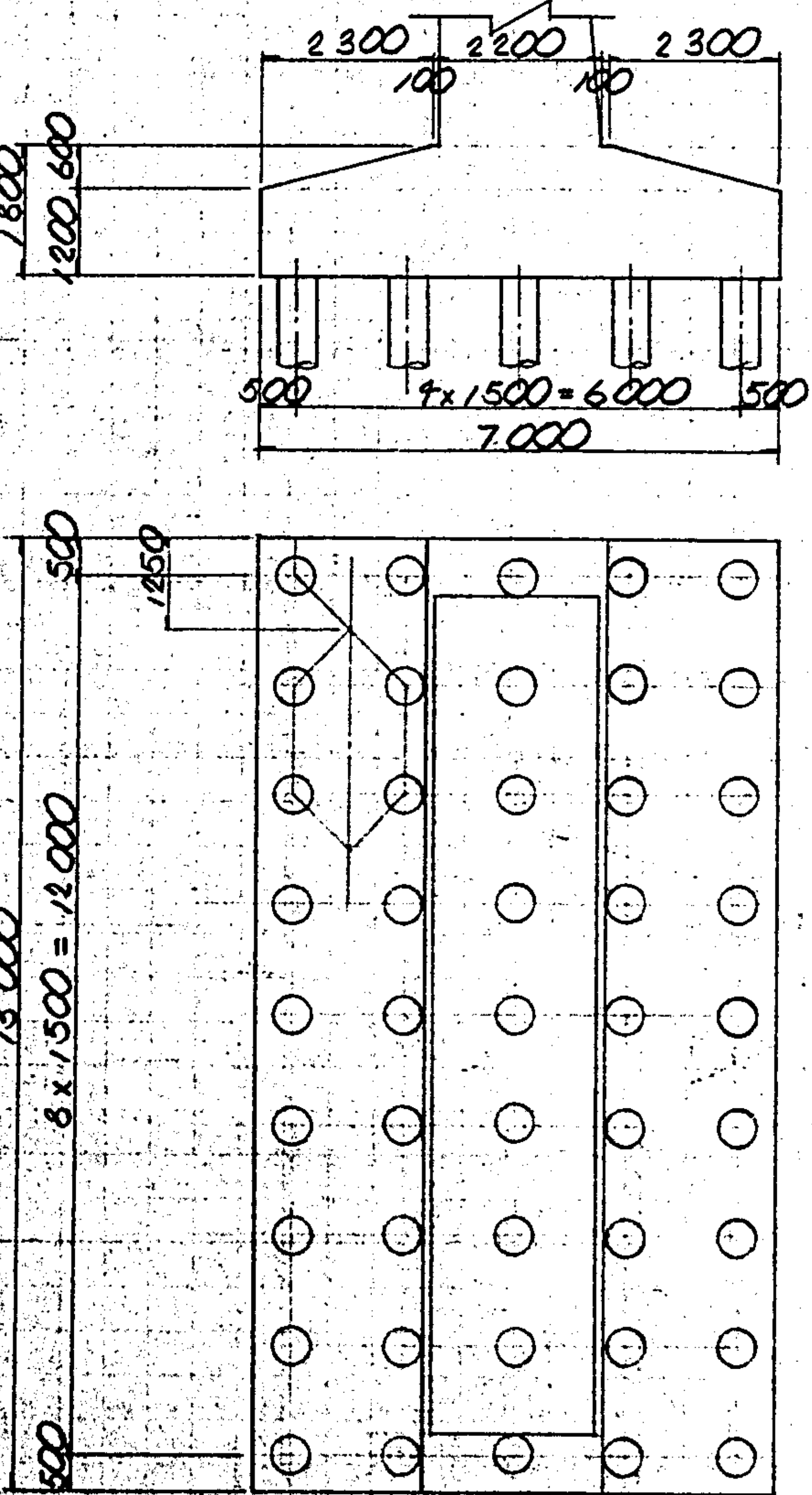
$$P = \left\{ \begin{matrix} 66.35 \\ 20.98 \end{matrix} \right. \pm \frac{1271.14}{202.50} \times 3.00 \qquad t/\text{Pile}$$

$$= \left\{ \begin{matrix} 66.35 \\ 20.98 \end{matrix} \right. \pm 18.83 = \left\{ \begin{matrix} 85.18 \\ 2.15 \end{matrix} \right. \quad "$$

2. Calculation at front toe.

(1) Bending moment analysis.

a. Effective width of footing.



(i) Center part

$h = 1/2 \times 180 - 15 = 75^{cm}$

$b_0 = D + 2h = 50 + 2 \times 75 = 200^{cm}$

$b = b_0 + 2e = 200 + 2 \times 190 = 580^{cm}$

$<$ Distance between piles $= 150^{cm}$

$\therefore b = 150^{cm}$

(ii) End part

$b_s = 50^{cm}$

$b/2 = 75^{cm}$

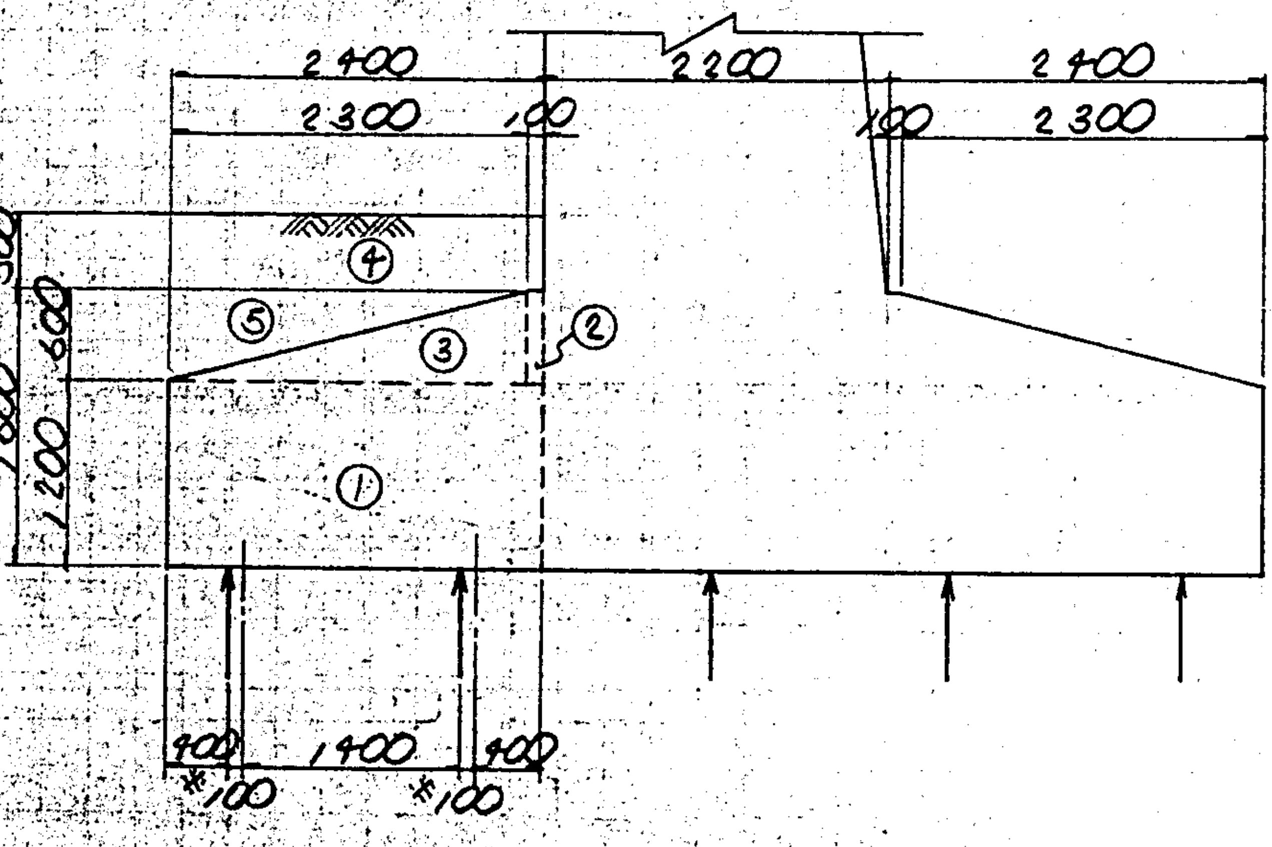
Since, $b_s < b/2$

$b = b_s + b/2 = 50 + 75 = 125^{cm}$

b. Vertical reaction force acting on pils.

Combined loads	Reaction force acting on piles.	
	P_1 (t)	P_2 (t)
D + Ep	65.00	57.05
D+T+I + Ep	79.64	69.72
D+T+I +Ep+Br	82.30	71.05
D +Se+Ep	85.18	64.42

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



* Tolerance for construcion.

Own waght of footing and weight covering earth

	Calculation	N (t)	x (m)	$M = N \cdot x$ (t·m)
①	2.5 × 2.40 × 1.20	7.20	1.20	8.64
②	2.5 × 0.10 × 0.60	0.15	0.05	0.01
③	2.5 × 2.30 × 0.60 × 1/2	1.73	0.87	1.51
④	1.8 × 2.40 × 0.50	2.16	1.20	2.59
⑤	1.8 × 2.30 × 0.60 × 1/2	1.24	1.63	2.02
Σ	———	12.48	———	14.77

d. Calculation of section force

	per own pile				per meter				
case	$P_{V_1} \cdot l_1$ (t·m)	$P_{V_2} \cdot l_2$ (t·m)	Mo (t·m)	M (t·m)	M/ef (t·m)	W (t·m)	M (t·m)	α	V (t·m)
1	130.00	28.53	-15.41×2	127.71	102.17	-14.77	87.40	1.00	87.40
2	159.28	34.86	-15.34×2	163.46	130.77	-14.77	116.00	〃	116.00
3	164.60	35.53	-16.40×2	167.33	133.86	-14.77	119.09	1.15	103.56
4	170.36	32.21	-28.25×2	146.07	116.86	-14.77	102.09	1.50	68.06

$l_1 = 2.00$
$l_2 = 0.50$

ef : effectivewidth
W : Own weight of footing
V : Value converted to ordinary to ordinary cace

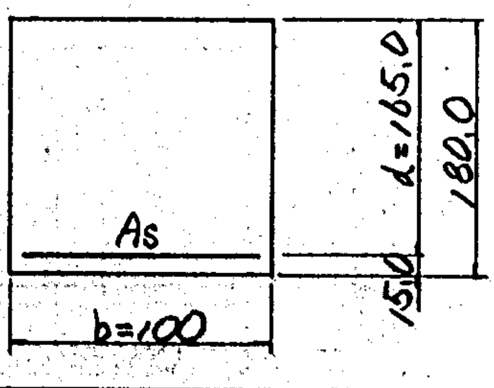
Stress calculation

$D + T + I + Ep$ { $M = 116.00$ t·m, $N =$ ——, $H =$ —— }

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1800
Tca (kg/cm2)	3.7		
M (t·m)	116	Q (t)	0
N (t)	0	u (cm)	75
b (cm)	100		
d (cm)	165	d′(cm)	15
As (cm2)	D 29 X 8 = 51.392		
As′ (cm2)			
d′/d	0.09	f (cm)	0
M′ (t·m)	116		
M′/bd2(kg/cm2)	4.26	Q/bd (kg/cm2)	0
mP	0.047	As′/As	0

C	S	Z
8.35	23.46	1.09

Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
35.57 < 80	1,499.62 < 1800	0.00 < 4



As = D 29 - ctc 12.5cm

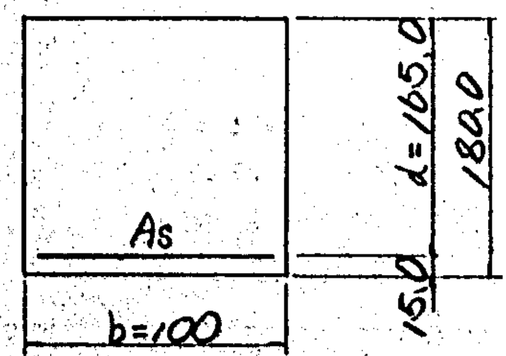
Stress calculation fore crack analysis.

D + Ep { M = 87.40 t.m
N = ———
H = ———

n	15		
Σca (kg/cm2)	80	Σsa (kg/cm2)	1400
Tca (kg/cm2)	3.7		
M (t·m)	87.4	Q (t)	0
N (t)	0	u (cm)	75
b (cm)	100		
d (cm)	165	d′(cm)	15
As (cm2)	D 29 X 8 = 51.392		
As′ (cm2)			
d′/d	0.09	f (cm)	0
M′ (t·m)	87.4		
M′/bd2(kg/cm2)	3.21	Q/bd (kg/cm2)	0
mP	0.047	As′/As	0

C	S	Z
8.35	23.46	1.09

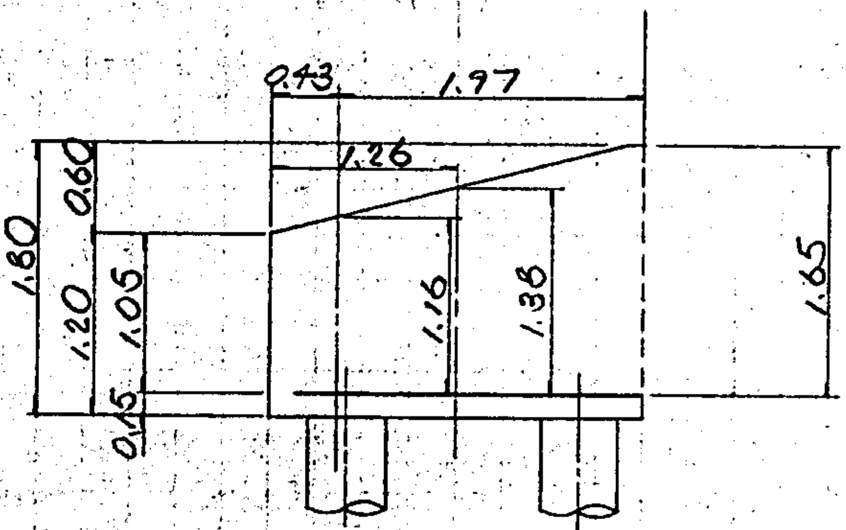
Σc(kg/cm2)	Σs(kg/cm2)	Tc(kg/cm2)
26.80 < 80	1,129.88 < 1400	0.00 < 4



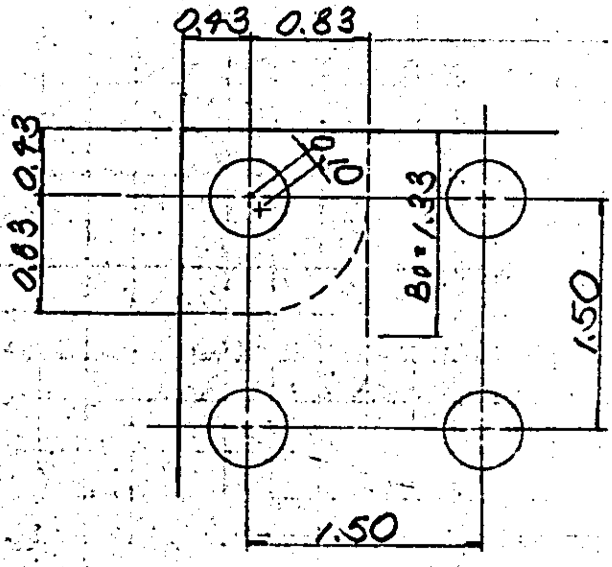
As = D 29 - 12.5cm c.t.c

(2) Calculation of Shearing force

a. Effective width (area of shearing section)



10 cm constructional tolerance is assumed towards dangerous side.



$$d = \overset{D}{0.50} + \overset{d_1}{1.16} = 1.66^{m}$$

$$d/2 = 1.66 \times 1/2 = 0.83''$$

(i) Effective area resisting bending shear.

$$B_0 = \ell_1 + \ell_2$$

$$= 0.6 \times 1.50 + 0.43 = 1.33^{m}$$

$$A = B_0 \cdot d$$

$$= 1.33 \times 1.38 = 1.84\ m^2$$

(ii) Effective area resisting punching shear

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

$$= 1/2 \times (1.05 + 1.16) \times 0.43 + 1/2 \times 3.142 \times 0.83 \times 1/2$$

$$\times (1.16 + 1.37) + 1.37 \times 0.43$$

$$= 2.71 \ m^2$$

b. Own weight of footing and weight of covering earth

Resisting area for normal load

$$A = 0.43^2 + 0.43 \times 0.83 \times 2 + 1/4 \times 3.142 \times 0.83^2 = 1.44 \ m^2$$

(for Punching shear)

$$A = 1.33 \times 1.26 = 1.68 \ m^2$$ (for bending shear)

(1.44) $1.68 \times (1.20 + 1.52) \times 1/2 \times 2.5$	=	(4.90) 5.71 t
(1.44) $1.68 \times (0.78 + 1.10) \times 1/2 \times 1.8$	=	(2.44) 2.84 "
	Σ =	(7.34) 8.55 t

(3) Cakulation of Shering Stress

D+T+I+Ep is assumed

$$S = 79.64 - \overset{(7.34)}{8.55} = \overset{(72.30)}{71.09}\ t$$

a. Shearing stress caused by bending

$$\tau = \frac{71.09 \times 10^3}{18400} = 3.9\ \mathrm{kg/cm^2} < \tau_a = 3.7\ \mathrm{kg/cm^2}$$

b. Shearing stress caused by punching

$$\tau = \frac{72.30 \times 10^3}{27100} = 2.7\ \mathrm{kg/cm^2} < \tau_a = 5.1\ \mathrm{kg/cm^2}$$

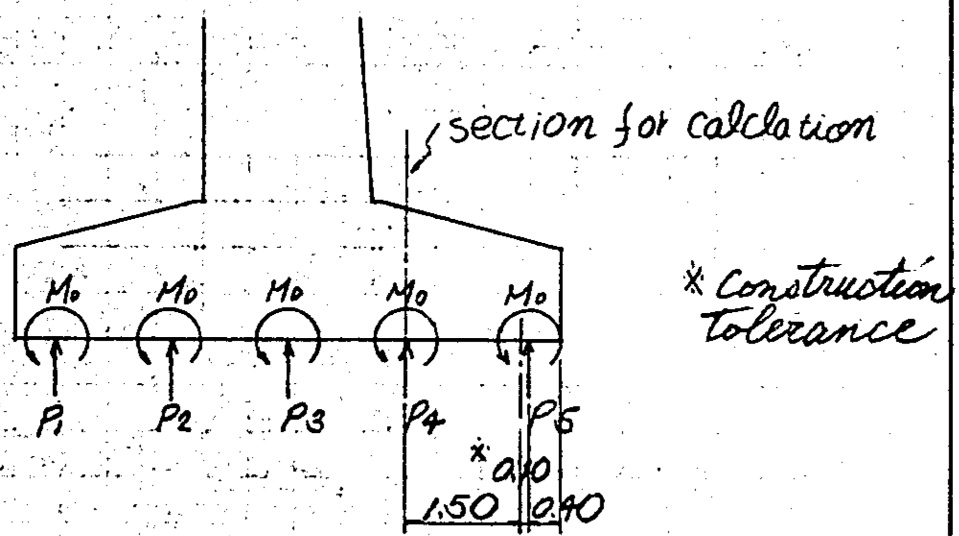
3. Calculation at rear heel

(1) Bending moment analysis

° Up side 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. $b = 1.30^{m}$

b. Stress caused by reaction force of pile.



$P_5 = 2.15^{t}$

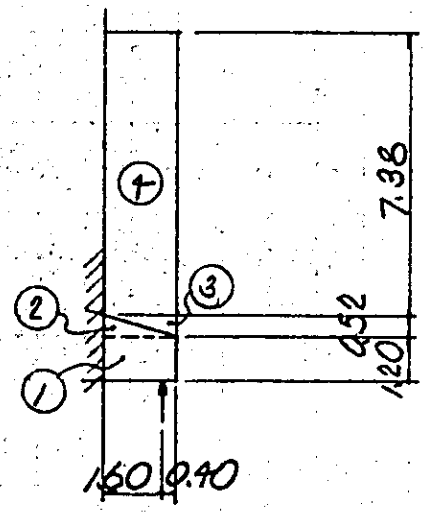
$M_0 = 28.25^{t \cdot m}$

$M = 2.15 \times 1.60 + 28.25 = 31.69^{t \cdot m}$

per meter moment will be,

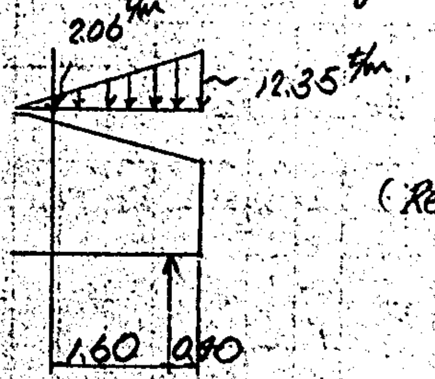
$M = 31.69 \times 1/1.25 = 25.35^{t \cdot m}$

c. Own weight of footing and weight of covaring earth



	calculation	N (t)	x (m)	N·x (t·m)
①	2.5 × 2.00 × 1.20	6.00	1.00	6.00
②	2.5 × 2.00 × 0.52 × 1/2	1.30	0.67	0.87
③	1.8 × 2.00 × 0.52 × 1/2	0.94	1.33	1.25
④	1.8 × 2.00 × 7.38	26.57	1.00	26.57
Σ	——	34.81	——	34.69

d. Stress caused by Vertical component of earth pressure vertical component of active eath pressure Pv is suppased acting at the virtual back face with the load of increasing uniformly



$D + S + Ep_t$

$Pv = 14.82$

(Refer 1, 1, 5), (3))