CV-6 STRUCTURAL CALCULATION OF DISCHARGE TUNNEL

Contents of this calculation note is shown as below.

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1. Structural Calculation of Discharge Tunnel

1.1 Soil Condition

Boring data around the construction area is shown in Fig 1.
Now the average N−value above the foundation level is caluculated as follows.

$$\bar{N} = \frac{\{(15+24)+(24+5)+(5+23)+(23+13)+(13+4)+(4+5)+(5+17)\}}{2 \times 7}$$

$$\fallingdotseq 12$$

According to the above calculation, the angle of the internal friction is calculated by the following equation.

$$\phi = (\sqrt{15\,\bar{N}} + 15)^\circ = (\sqrt{15 \times 12} + 15)^\circ \fallingdotseq 28^\circ$$

the bulk density of soil above the ground water r : $r = 1.9\ t/m^3$

the bulk density of soil under the ground water r' : $r' = 1.0\ t/m^3$

Other basic condition data are descripted in " Civil Design Condition " (vid.No EWC−1001).

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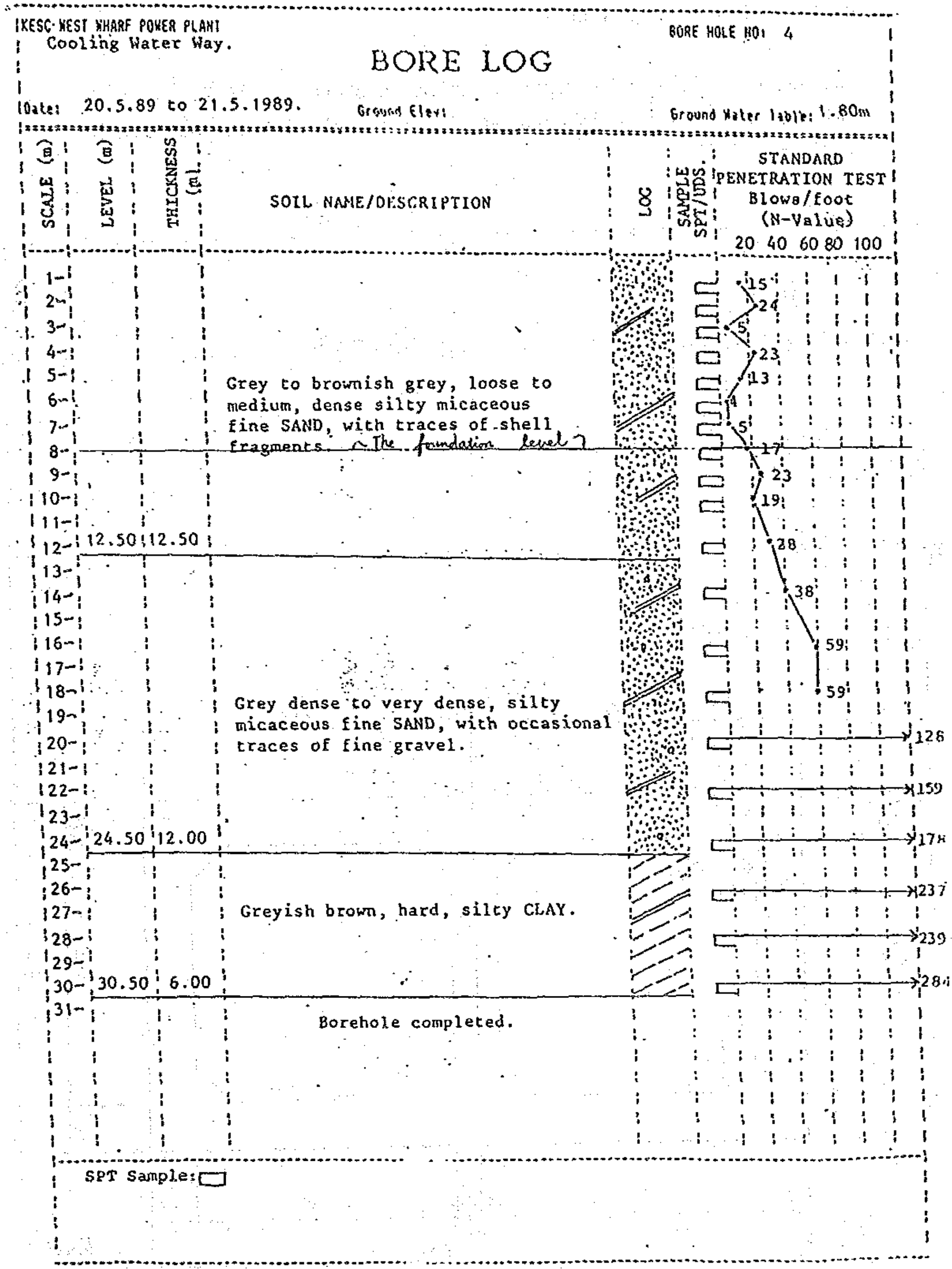

Fig 1. Soil column diagram

FORM 04

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1.2 Outline of Discharge Tunnel

The design section of Discharge Tunnel is shown in Fig 2.
This design structure is considered for the rigid frame structure.

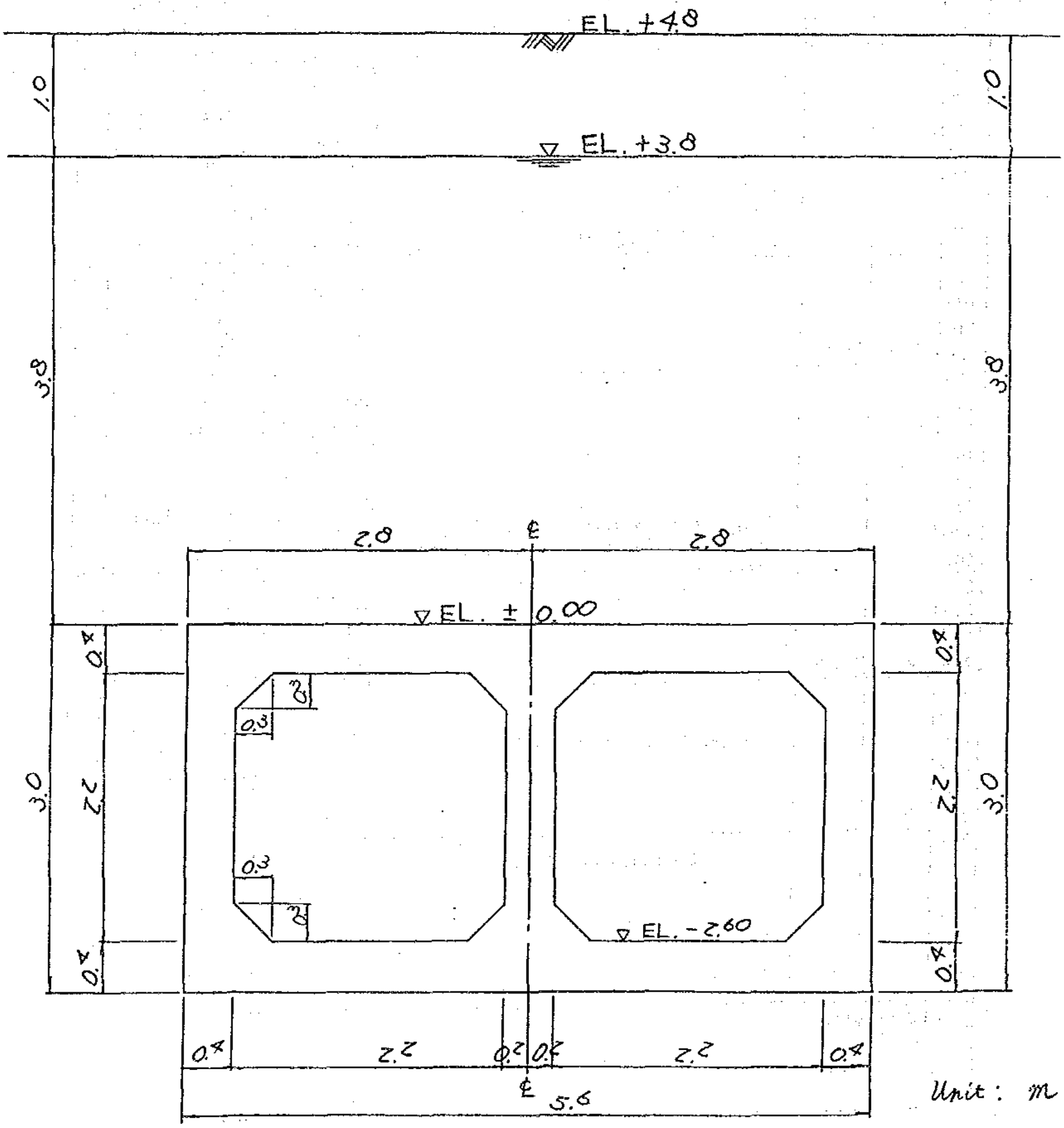

Fig 2. The design section of Discharge Tunnel

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1.3 Stability Calculatioin

1) Vertical forces (per 1m unit length)

a) Base slab

$W_{c1} = 5.6 \times 0.4 \times 2.45 = 5.5$ t

b) Side wall (both sides)

$W_{c2} = 2\times(2.2\times0.4+0.3\times0.3)\times2.45 = 4.8$ t

c) Partition wall

$W_{c3} = (2.2\times0.4+2\times0.3\times0.3)\times2.45 = 2.6$ t

d) Upper slab

$W_{c4} = 5.6 \times 0.4 \times 2.45 = 5.5$ t

e) Soil weight

$W_s = 5.6\times(1.9\times1.0+1.0\times3.8) = 31.9$ t

f) Water weight

Water weigth is calculated by dividing between the inside and the outside of Discharge Tunnel.

i) at the inside

$W_{w1} = 2\times(2.2\times2.2-2\times0.3\times0.3)\times1.0 = 9.3$ t

ii) at the outside

$W_{w2} = 3.8 \times 1.0 \times 5.6 = 21.3$ t

g) Buoyancy

$V_b = 1.0\times5.6\times6.8 = 38.1$ t

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Accordingly the vertical forces are summarized as shown in Table 1.

Table1. The summary of vertical forces

Species	Vertical forces [t]	Remarks
W_{c1}	5.5	Base slab
W_{c2}	4.8	Side wall
W_{c3}	2.6	Partition wall
W_{c4}	5.5	Upper slab
W_s	31.9	Soil weight
W_{w1}	9.3	Water weight (inside)
W_{w2}	21.3	Water weight (outside)
V_b	−38.1	Buoyancy
TOTAL	V= 42.8 t	

2) The ground reaction q_r

$$q_r = \frac{V}{B \cdot L} \cdot \left(1 \pm \frac{6e}{B}\right)$$ where e : the eccentric distance

e = 0

$$= \frac{42.8}{5.6 \times 1.0}$$

$$= 7.6\ t/m^2$$

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3) Study of the bearing capacity q_u

The ultimate bearing capacity is calculated by the following equation.

$$q_u = \alpha KCNc + KqN_q + \frac{1}{2} r_1 \beta B' N_r$$

where C : soil cohesion $= 0\ t/m^2$

q : the surcharge load

$q = 1.9 \times 1.0 + 1.0 \times 6.8 = 8.7\ t/m^2$

r_1: the bulk density of the bearing soil (t/m^3)

$r_1 = 1.0\ t/m^3$

(r_1 is the bulk density under the ground water.)

B' : the effective width considered for the eccentric distance

$B' = 5.6\ m$

α, β : the coefficient of basic form $\alpha = \beta = 1.0$

K : the extra coefficient for the embedded effect

$K = 1.0$

N_q, N_r : the bearing coefficients are adopted from the following graphs.

$N_q = 15$

$N_r = 11$

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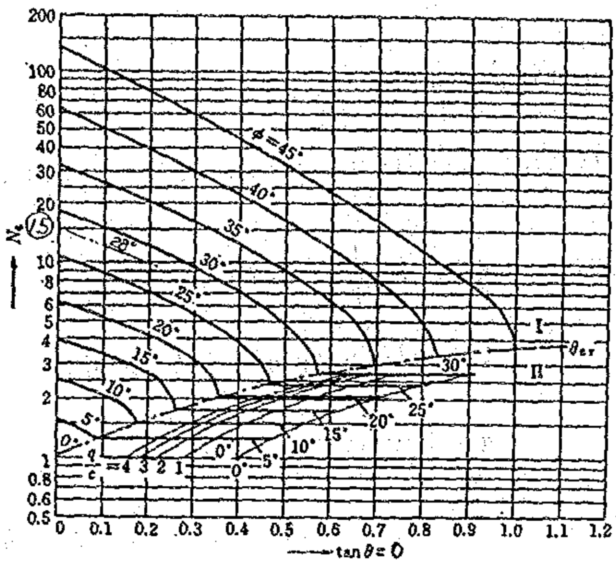

Fig 3. Graph of the bearing coefficient N_q

where tan θ : tan $\theta = \frac{H}{V} = 0$

Because the horizontal forces are equilibriumed at both sides.

ϕ : the angle of the internal friction $\phi = 28°$

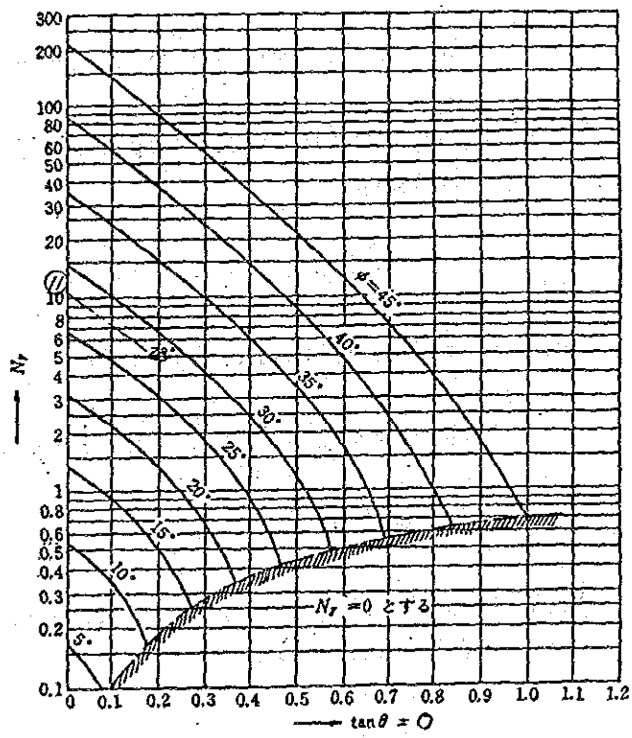

Fig 4. Graph of the bearing coefficient N_r

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Accordingly the ultimate bearing capacity q_u is calculated as below.

$$q_u = 1.0\times 8.7\times 15+\frac{1}{2}\times 1.0\times 5.6\times 11 = 161\ t$$

4) The allowable bearing capacity q_a

The allowable bearing capacity is calculated as below.

$$q_a = \frac{1}{Fs}\cdot q_u = \frac{1}{3}\times 161 \fallingdotseq 53\ t/m^2 \underset{O.K}{>} q_r = 7.6\ t/m^2$$

where F_s : the factor of safety $F_s = 3$

Therefore the spread foundation is adopted for the foundation of Discharge Tunnel.

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5) Study of floating

The calculation of floating is executed at normal and at construction, so this calculation is as follows.

a) Total vertical force

i) at normal

$$V_1 = W_{c1} + W_{c2} + W_{c3} + W_{c4} + W_s + W_{w1} + W_{w2} = 80.9\ \text{t}$$

ii) at construction (The internal water is no considered.)

$$V_2 = W_{c1} + W_{c2} + W_{c3} + W_{c4} + W_s + W_{w2} = 71.6\ \text{t}$$

b) Up lift U

Up lift U is calculated as below.

$$U = 1.0 \times 6.8 \times 5.6 = 38.1\ \text{t}$$

c) Checking on the factor of safety of floating F_l

The factor of safety of floating is checked by the following two cases.

i) at normal

$$F_{l1} = \frac{V_1}{U} = \frac{80.9}{38.1} = 2.1 \underset{\text{O.K}}{>} 1.1$$

ii) at construction

$$F_{l2} = \frac{V_2}{U} = \frac{71.6}{38.1} = 1.88 \underset{\text{O.K}}{>} 1.0$$

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1.4 The Structural Design Case

The following three cases are considered for the structural design cases.

Table 2. Summary of the design cases

CASE	1	2	3
CONDITION	at Normal	at Construction	at Inspection
The internal water conditions	Full	Empty	Empty(at oneside)
The surcharge load	1.0 t/m²	1.0 t/m²	1.0 t/m²
The period	Long term	Short term	Short term
The incremental coefficient of the allowable stress	1.0	1.25	1.25

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1.5 The Structural Design

1.5.1 Load Case-1 (at Normal)

1) Frame of the design structure

Frame of the design structure is shown in Fig 5.

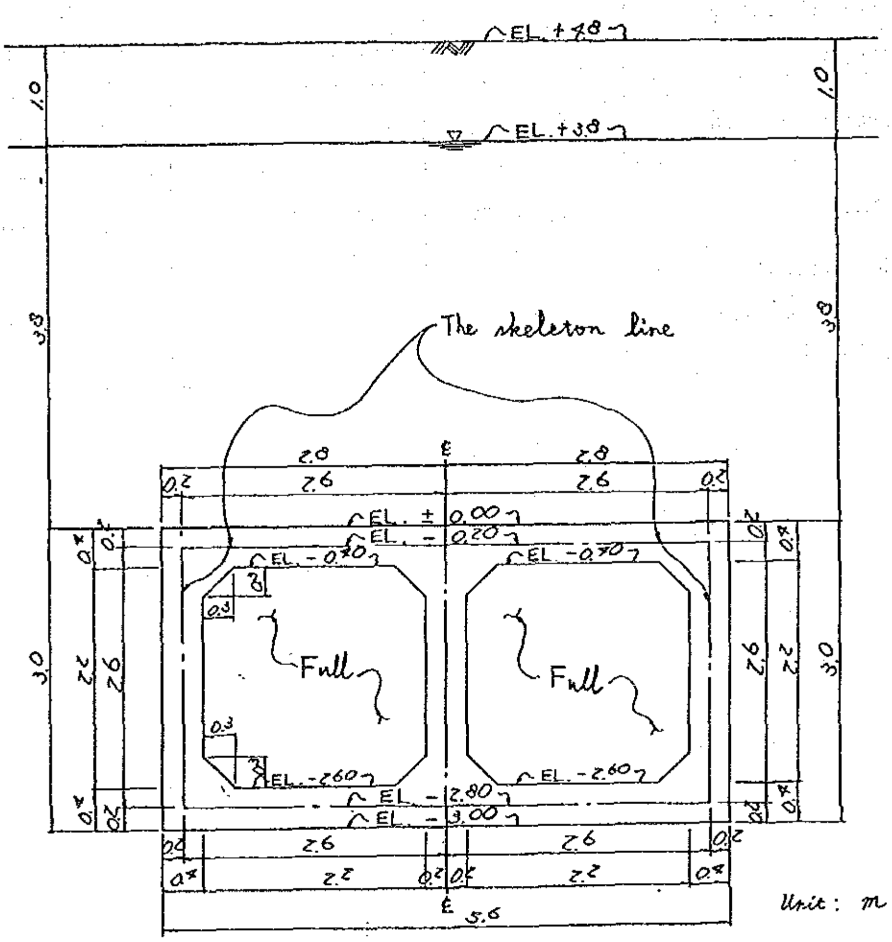

Fig 5. Frame of the design structure

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2) Load calculation (per 1m unit length)

a) The surcharge load at the ground surface q $q = 1.0\ t/m^2$

b) Self weight

i) Base slab

$$W_{c1} = 0.4 \times 2.45 = 0.98\ t/m^2$$

ii) Side wall (at one side)

$$W_{c2} = (0.4\times2.2+0.3\times0.3)\times2.45\div2.2 = 1.08\ t/m^2$$

iii) Partition wall

$$W_{c3} = (0.4\times2.2+2\times0.3\times0.3)\times2.45\div2.2 = 1.18\ t/m^2$$

iv) Upper slab

$$W_{c4} = 0.4 \times 2.45 = 0.98\ t/m^2$$

v) Soil weight

$$W_s = 1.9 \times 1.0 + 1.0 \times 3.8 = 5.7\ t/m^2$$

vi) Water weight

(at outside)

$$W_{w1} = 1.0 \times 3.8 = 3.8\ t/m^2$$

(at inside)

$$W_{w2} = 1.0 \times 2.2 = 2.2\ t/m^2$$

c) The earth pressure

$$P_{e1} = 0.5\times(1.0+1.9\times1.0+1.0\times4.0) = 3.45\ t/m^2$$

$$P_{e2} = 0.5\times(1.0+1.9\times1.0+1.0\times6.6) = 4.75\ t/m^2$$

d) The water pressure

i) at inside

$$P_{wi} = 1.0 \times 2.2 = 2.2\ t/m^2$$

ii) at outside

$$P_{w1} = 1.0 \times 4.0 = 4.0\ t/m^2$$

$$P_{w2} = 1.0 \times 6.6 = 6.6\ t/m^2$$

iii) Up lift

$$P_u = 1.0 \times 6.8 = 6.8\ t/m^2$$

e) The ground reaction

$$q_r = 7.6\ t/m^2$$

According to the above calculations, the results of load calculations are shown in Fig 6.

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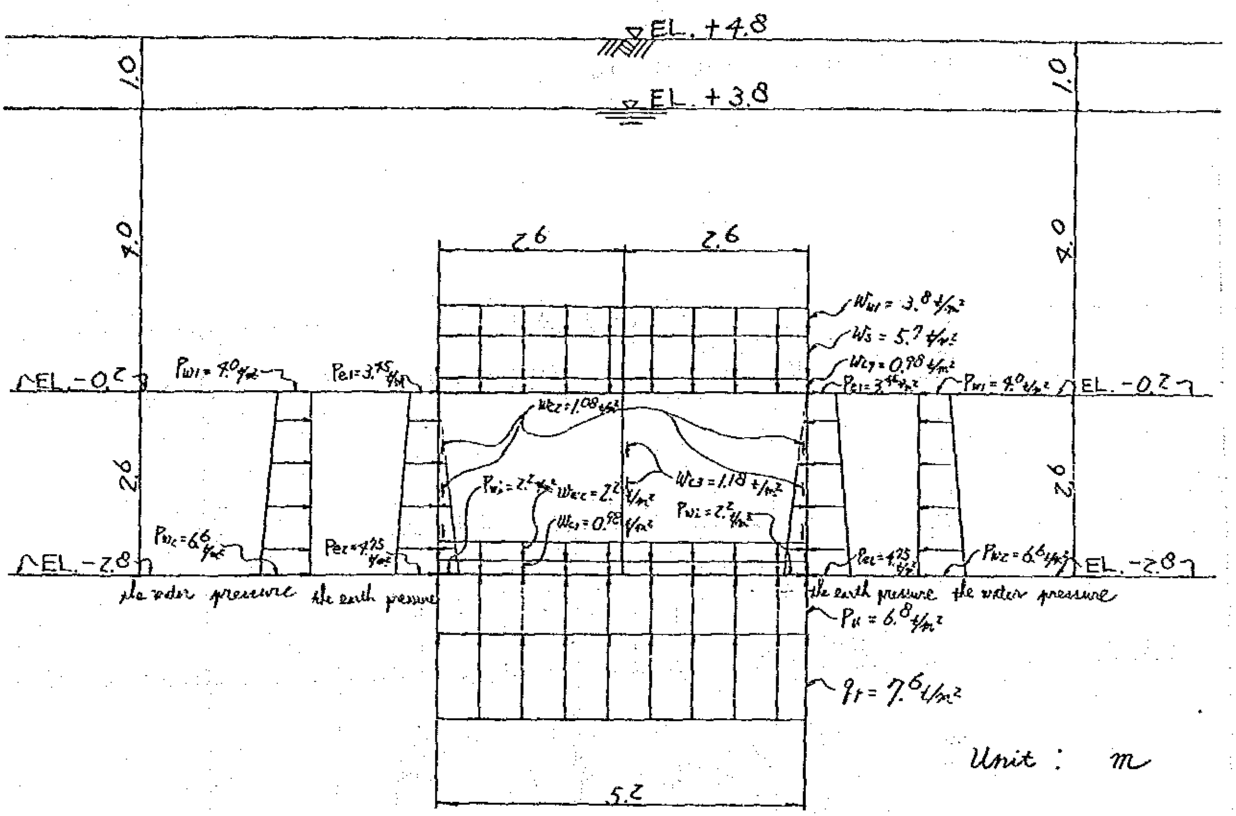

Fig 6. The results of load calculations

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3) The load diagram

The load diagram is shown in Fig 7.

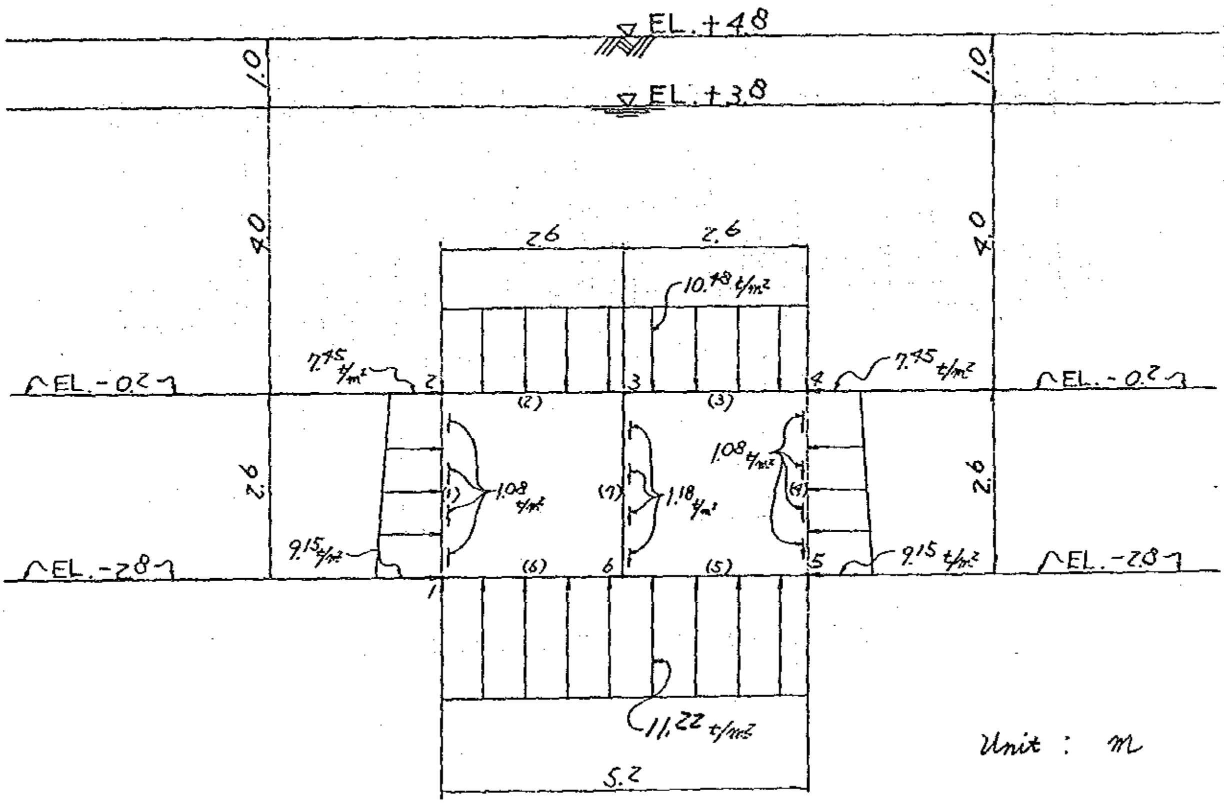

Fig 7. The load diagram

4) Input data for the sectional dimensions

The sectional forces are calculated by computer, so input data for the sectional dimensions are indicated in Table 3.

Table 3. Table of the sectional dimensions (per 1m unit length)

Member's number	Section area A [m²]	Geometrical moment of inertia I [m⁴]	Remarks
(1)	0.4	0.0053	Side wall
(2),(3)	0.4	0.0053	Upper slab
(4)	0.4	0.0053	Side wall
(5),(6)	0.4	0.0053	Base slab
(7)	0.4	0.0053	Partition wall

5) The computer calculation results

The computer calculation results are shown in the follwing figures and table (Fig 8 −10 and Table 4).

WEST WHARF DISCHARGE TUNNEL CASE-1 (NORMAL)

BENDING MOMENT

(TON*M)

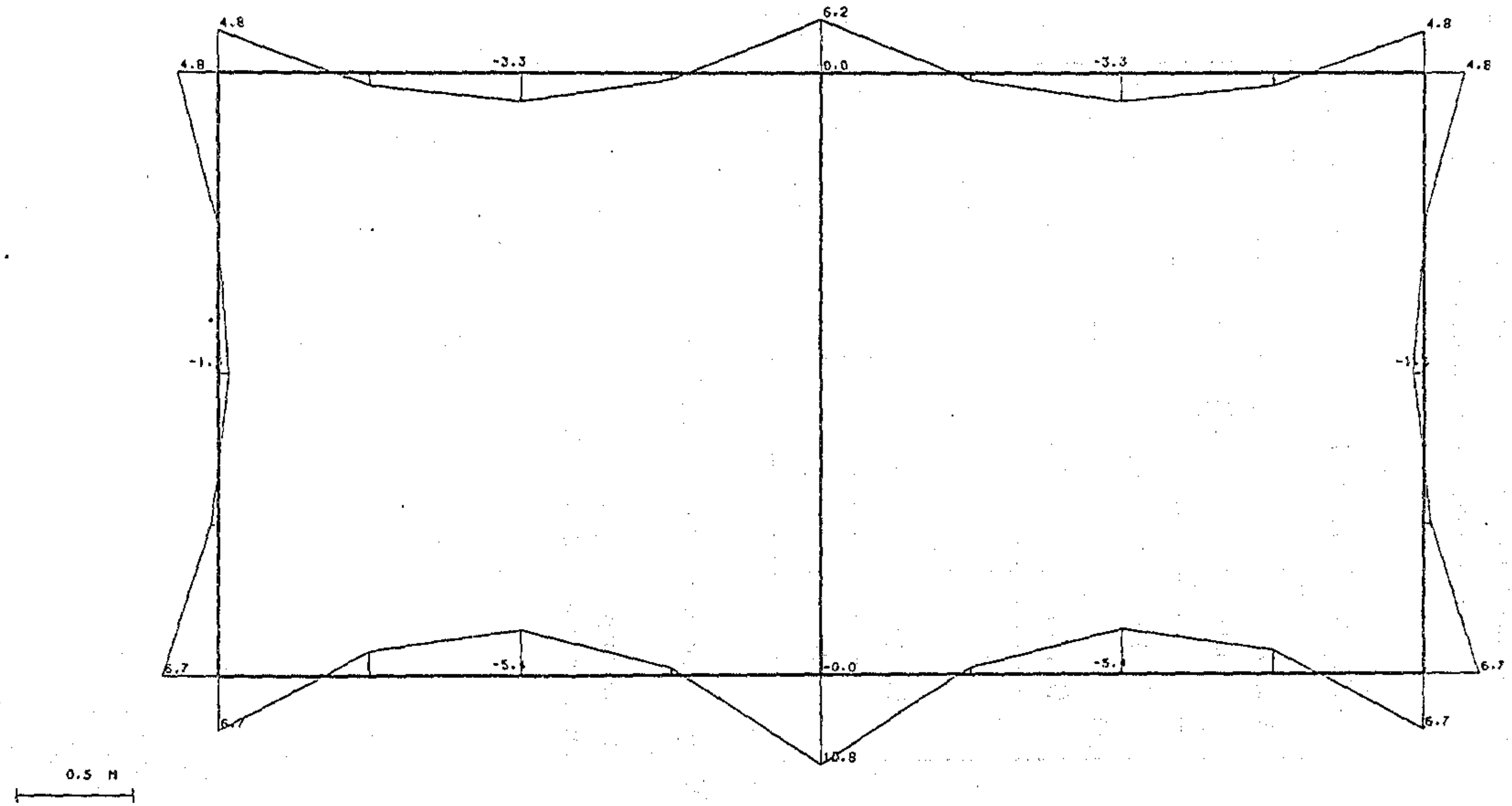

Fig 8. The bending moment diagram

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WEST WHARF DISCHARGE TUNNEL CASE-1 (NORMAL)

SHEARING FORCE

(TON)

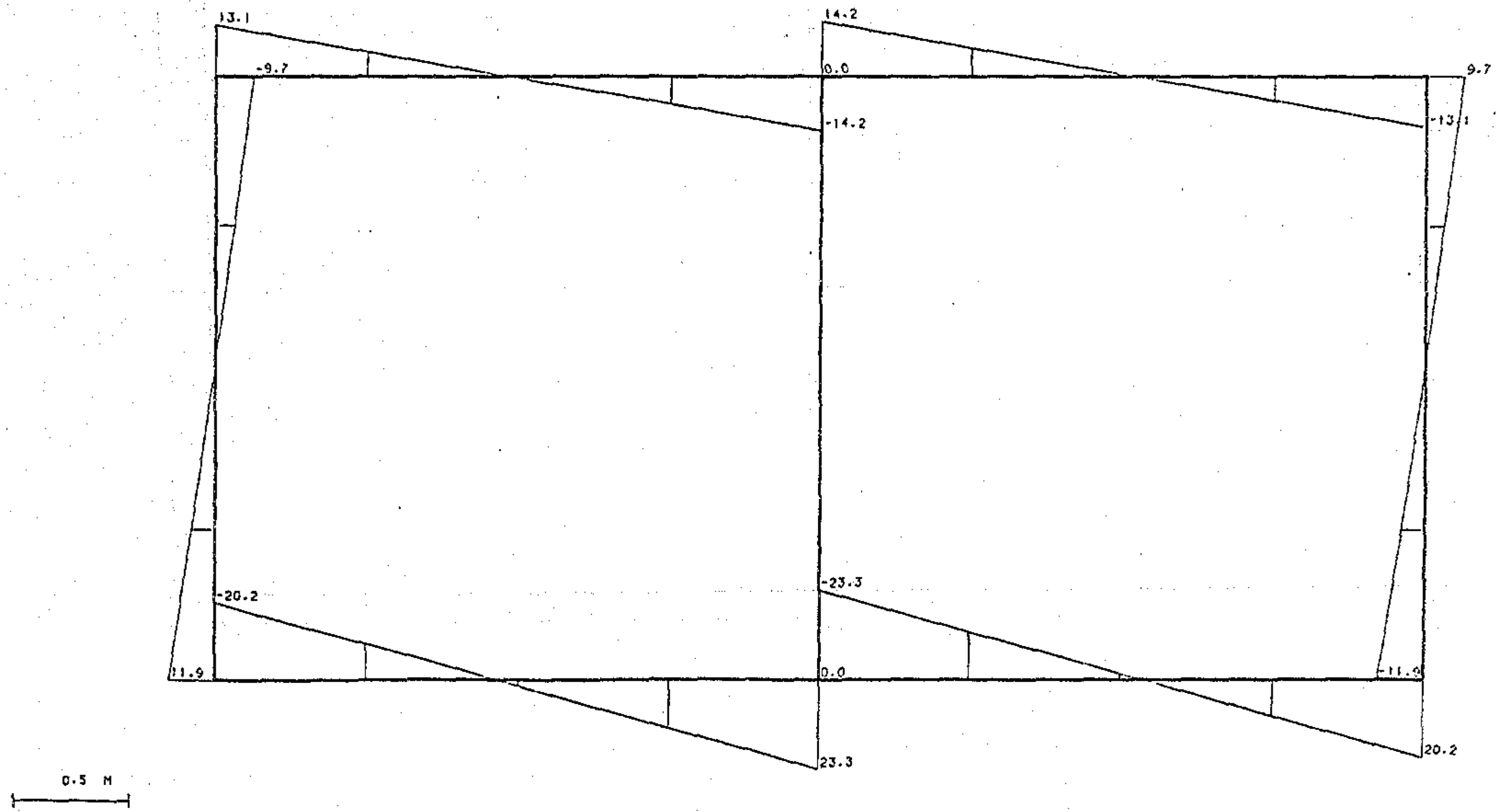
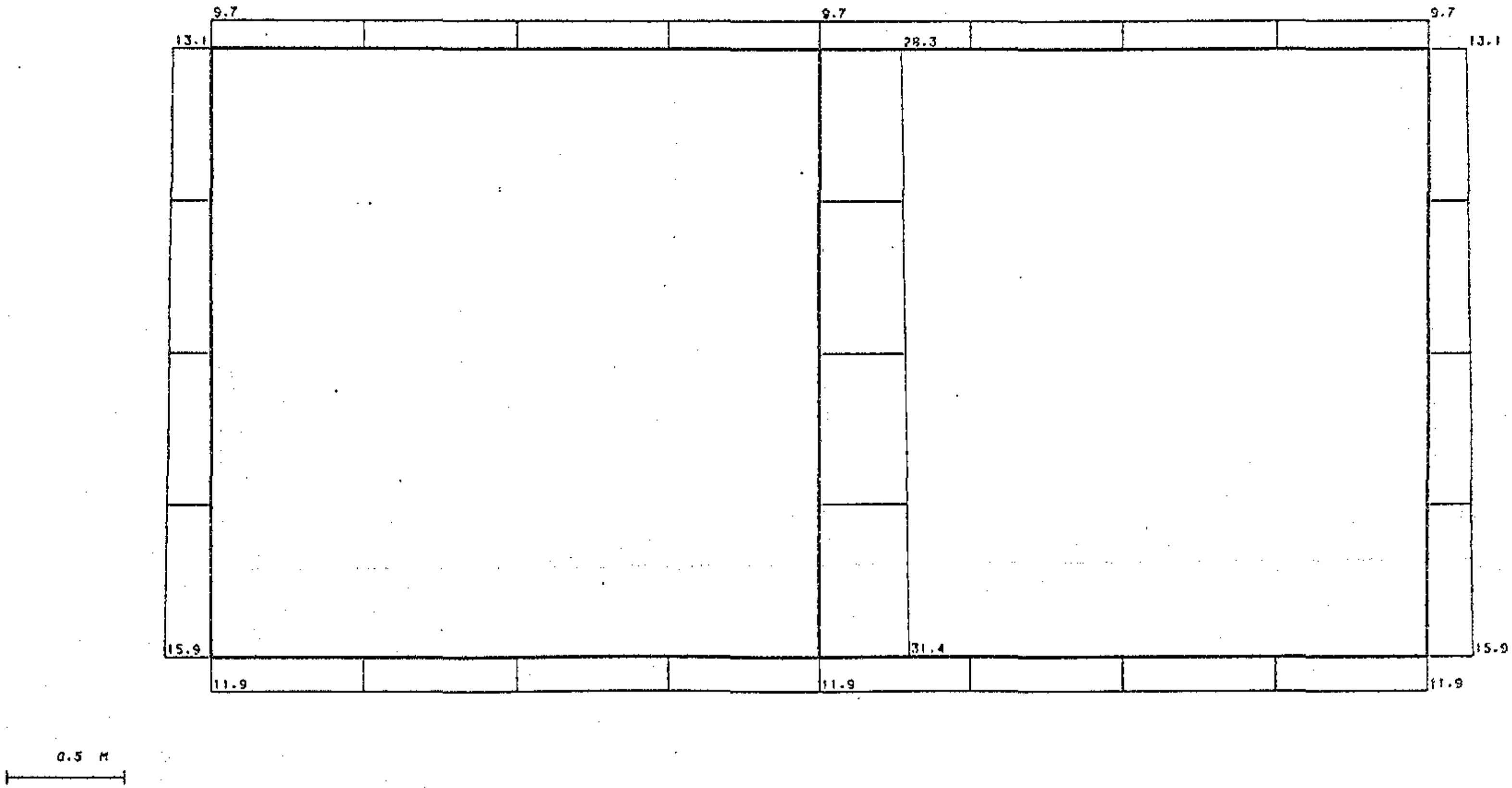


Fig 9. The shearing force diagram

WEST WHARF DISCHARGE TUNNEL CASE-1 (NORMAL)

AXIAL FORCE



(TON)

Fig 10. The axial force diagram

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Table 4. The calculation results of the sectional forces (at Normal)

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	1.5899E+01	1.1870E+01	6.6747E+00	7	1.5197E+01	6.0610E+00	8.6502E-01
2	7	1.5197E+01	6.0610E+00	8.6203E-01	8	1.4495E+01	5.2784E-01	-1.2614E+00
3	8	1.4495E+01	5.2784E-01	-1.2644E+00	9	1.3793E+01	-4.7290E+00	1.1897E-01
4	9	1.3793E+01	-4.7290E+00	1.1598E-01	2	1.3091E+01	-9.7097E+00	4.8265E+00
5	2	9.7097E+00	1.3091E+01	4.8235E+00	10	9.7097E+00	6.2791E+00	-1.4718E+00
6	10	9.7097E+00	6.2791E+00	-1.4718E+00	11	9.7097E+00	-5.3292E-01	-3.3393E+00
7	11	9.7097E+00	-5.3292E-01	-3.3393E+00	12	9.7097E+00	-7.3449E+00	-7.7898E-01
8	12	9.7097E+00	-7.3449E+00	-7.7898E-01	3	9.7097E+00	-1.4157E+01	6.2091E+00
9	3	9.7097E+00	1.4157E+01	6.2091E+00	13	9.7097E+00	7.3449E+00	-7.7898E-01
10	13	9.7097E+00	7.3449E+00	-7.7898E-01	14	9.7097E+00	5.3292E-01	-3.3393E+00
11	14	9.7097E+00	5.3292E-01	-3.3393E+00	15	9.7097E+00	-6.2791E+00	-1.4718E+00
12	15	9.7097E+00	-6.2791E+00	-1.4718E+00	4	9.7097E+00	-1.3091E+01	4.8235E+00
13	4	1.3091E+01	9.7097E+00	4.8235E+00	16	1.3793E+01	4.7290E+00	1.1299E-01
14	16	1.3793E+01	4.7290E+00	1.1598E-01	17	1.4495E+01	-5.2784E-01	-1.2674E+00
15	17	1.4495E+01	-5.2784E-01	-1.2644E+00	18	1.5197E+01	-6.0610E+00	8.5903E-01
16	18	1.5197E+01	-6.0610E+00	8.6203E-01	5	1.5899E+01	-1.1870E+01	6.6717E+00
17	5	1.1870E+01	2.0153E+01	6.6747E+00	19	1.1870E+01	9.2848E+00	-2.8925E+00
18	19	1.1870E+01	9.2848E+00	-2.8925E+00	20	1.1870E+01	-1.5832E+00	-5.3955E+00
19	20	1.1870E+01	-1.5832E+00	-5.3955E+00	21	1.1870E+01	-1.2451E+01	-8.3434E-01
20	21	1.1870E+01	-1.2451E+01	-8.3434E-01	6	1.1870E+01	-2.3319E+01	1.0791E+01
21	6	1.1870E+01	2.3319E+01	1.0791E+01	22	1.1870E+01	1.2451E+01	-8.3434E-01
22	22	1.1870E+01	1.2451E+01	-8.3434E-01	23	1.1870E+01	1.5832E+00	-5.3955E+00
23	23	1.1870E+01	1.5832E+00	-5.3955E+00	24	1.1870E+01	-9.2848E+00	-2.8925E+00
24	24	1.1870E+01	-9.2848E+00	-2.8925E+00	1	1.1870E+01	-2.0153E+01	6.6747E+00
25	3	2.8314E+01	2.0566E-15	4.8417E-15	25	2.9081E+01	2.0566E-15	3.5049E-15
26	25	2.9081E+01	2.0566E-15	3.5049E-15	26	2.9848E+01	2.0566E-15	2.1682E-15
27	26	2.9848E+01	2.0566E-15	2.1682E-15	27	3.0615E+01	2.0566E-15	8.3138E-16
28	27	3.0615E+01	2.0566E-15	8.3138E-16	6	3.1382E+01	2.0566E-15	-5.0541E-16

1.5.2 Load Case−2 (at Construction)

1) Frame of the design structure

Frame of the design structure is the same structure as that of Case−1 excluding a part that the internal water condition is empty.

2) The load calculation

The load calculations are the same calculations as those of Case−1 excluding a part that the internal water loads are nothing.

3) The load diagram

The load diagram is shown in Fig 11.

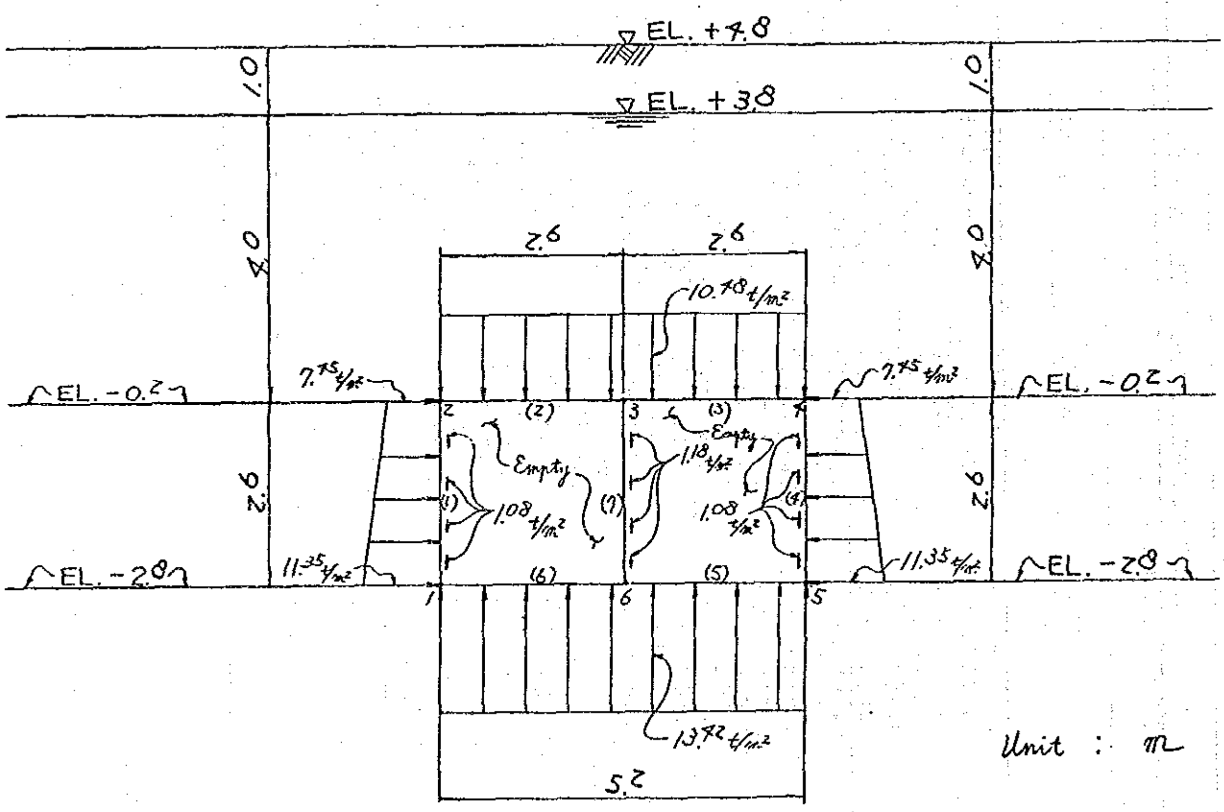

Fig 11. The load diagram

4) Input data for the sectional dimensions

Input data for the sectional dimensions are all the same values as those of Case−1 (vid.Table 3, P17).

5) The computer calculataion results

The computer calculation results are shown in the following figures and table (Fig 12−14 and Table 5).

WWTPP DISCHARGE TUNNEL CASE-2 (CONSTRUCTION)

BENDING MOMENT

(TON*M)

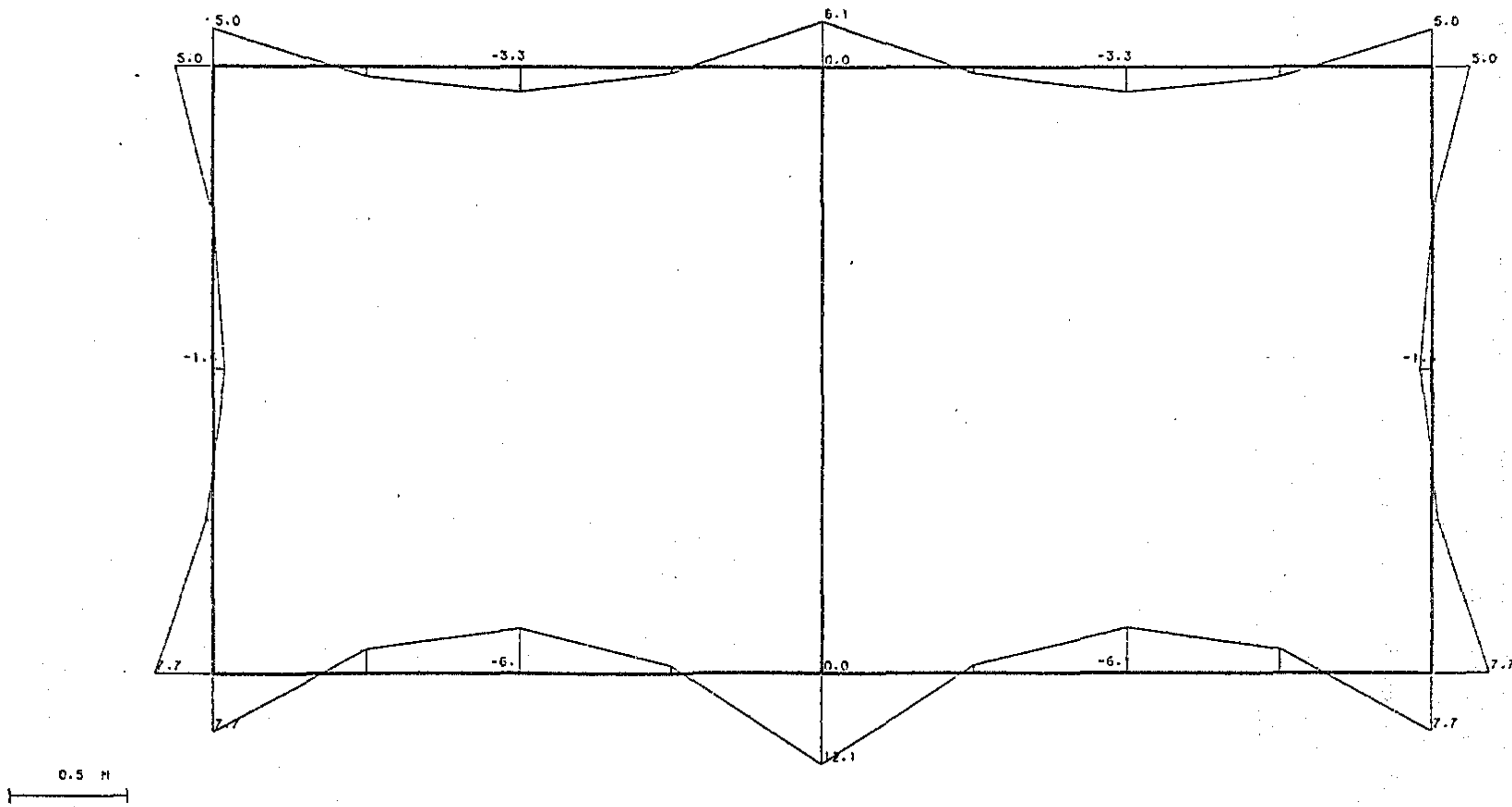

Fig 12. The bending moment diagram

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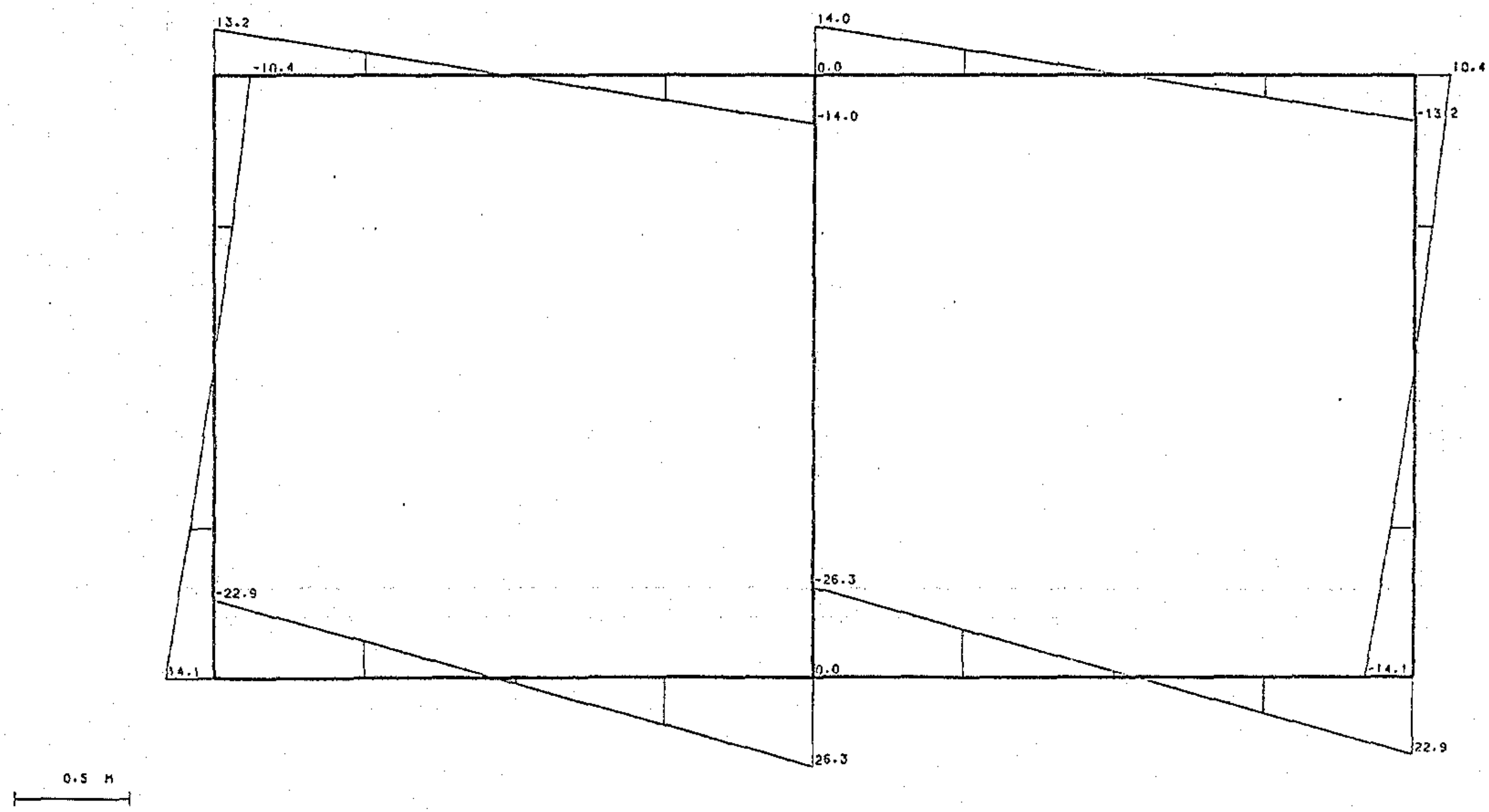
WWTPP DISCHARGE TUNNEL CASE-2 (CONSTRUCTION) SHEARING FORCE

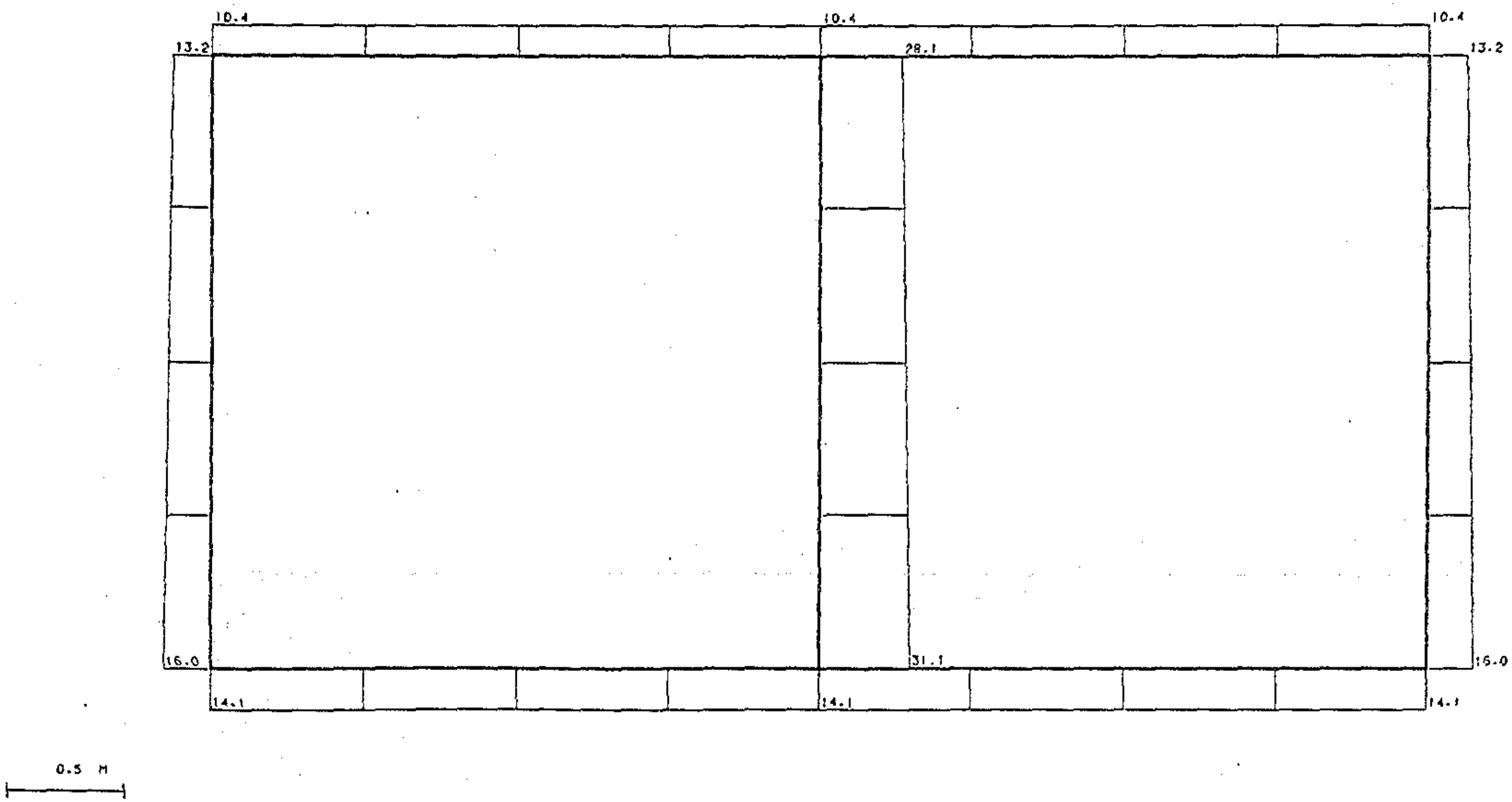


(TON)

Fig 13. The shearing force diagram

WWTPP DISCHARGE TUNNEL CASE-2 (CONSTRUCTION)

AXIAL FORCE



(TON)

Fig 14. The axial force diagram

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Table 5. The calculation results of the sectional forces (at Construction)

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	1.6020E+01	1.4089E+01	7.6980E+00	7	1.5318E+01	7.0282E+00	8.7610E-01
2	7	1.5318E+01	7.0282E+00	8.6924E-01	8	1.4616E+01	6.0135E-01	-1.5692E+00
3	8	1.4616E+01	6.0135E-01	-1.5760E+00	9	1.3914E+01	-5.1918E+00	-4.2965E-02
4	9	1.3914E+01	-5.1918E+00	-4.9830E-02	2	1.3212E+01	-1.0351E+01	5.0428E+00
5	2	1.0351E+01	1.3212E+01	5.0359E+00	10	1.0351E+01	6.3995E+00	-1.3376E+00
6	10	1.0351E+01	6.3995E+00	-1.3376E+00	11	1.0351E+01	-4.1249E-01	-3.2834E+00
7	11	1.0351E+01	-4.1249E-01	-3.2834E+00	12	1.0351E+01	-7.2245E+00	-8.0139E-01
8	12	1.0351E+01	-7.2245E+00	-8.0139E-01	3	1.0351E+01	-1.4036E+01	6.1084E+00
9	3	1.0351E+01	1.4036E+01	6.1084E+00	13	1.0351E+01	7.2245E+00	-8.0139E-01
10	13	1.0351E+01	7.2245E+00	-8.0139E-01	14	1.0351E+01	4.1249E-01	-3.2834E+00
11	14	1.0351E+01	4.1249E-01	-3.2834E+00	15	1.0351E+01	-6.3995E+00	-1.3376E+00
12	15	1.0351E+01	-6.3995E+00	-1.3376E+00	4	1.0351E+01	-1.3212E+01	5.0359E+00
13	4	1.3212E+01	1.0351E+01	5.0359E+00	16	1.3914E+01	5.1918E+00	-5.6696E-02
14	16	1.3914E+01	5.1918E+00	-4.9830E-02	17	1.4616E+01	-6.0135E-01	-1.5829E+00
15	17	1.4616E+01	-6.0135E-01	-1.5760E+00	18	1.5318E+01	-7.0282E+00	8.6237E-01
16	18	1.5318E+01	-7.0282E+00	8.6924E-01	5	1.6020E+01	-1.4089E+01	7.6911E+00
17	5	1.4089E+01	2.2888E+01	7.6980E+00	19	1.4089E+01	1.0590E+01	-3.1825E+00
18	19	1.4089E+01	1.0590E+01	-3.1825E+00	20	1.4089E+01	-1.7079E+00	-6.0692E+00
19	20	1.4089E+01	-1.7079E+00	-6.0692E+00	21	1.4089E+01	-1.4006E+01	-9.6225E-01
20	21	1.4089E+01	-1.4006E+01	-9.6225E-01	6	1.4089E+01	-2.6304E+01	1.2138E+01
21	6	1.4089E+01	2.6304E+01	1.2138E+01	22	1.4089E+01	1.4006E+01	-9.6225E-01
22	22	1.4089E+01	1.4006E+01	-9.6225E-01	23	1.4089E+01	1.7079E+00	-6.0692E+00
23	23	1.4089E+01	1.7079E+00	-6.0692E+00	24	1.4089E+01	-1.0590E+01	-3.1825E+00
24	24	1.4089E+01	-1.0590E+01	-3.1825E+00	1	1.4089E+01	-2.2888E+01	7.6980E+00
25	3	2.8073E+01	1.4972E-15	4.2153E-15	25	2.8840E+01	1.4972E-15	3.2421E-15
26	25	2.8840E+01	1.4972E-15	3.2421E-15	26	2.9607E+01	1.4972E-15	2.2689E-15
27	26	2.9607E+01	1.4972E-15	2.2689E-15	27	3.0374E+01	1.4972E-15	1.2957E-15
28	27	3.0374E+01	1.4972E-15	1.2957E-15	6	3.1141E+01	1.4972E-15	3.2249E-16

1.5.3 Load Case-3 (at Inspection)

1) Frame of the design structure

Frame of the design structure is the same structure as that of Case-1 excluding a part that the internal water condition is empty at oneside.

2) The load calculation

The load calculations are the same calculations as those of Case-1 excluding a part that the internal water loads are nothing at oneside.

3) The load diagram

The load diagram is shown in Fig 15.

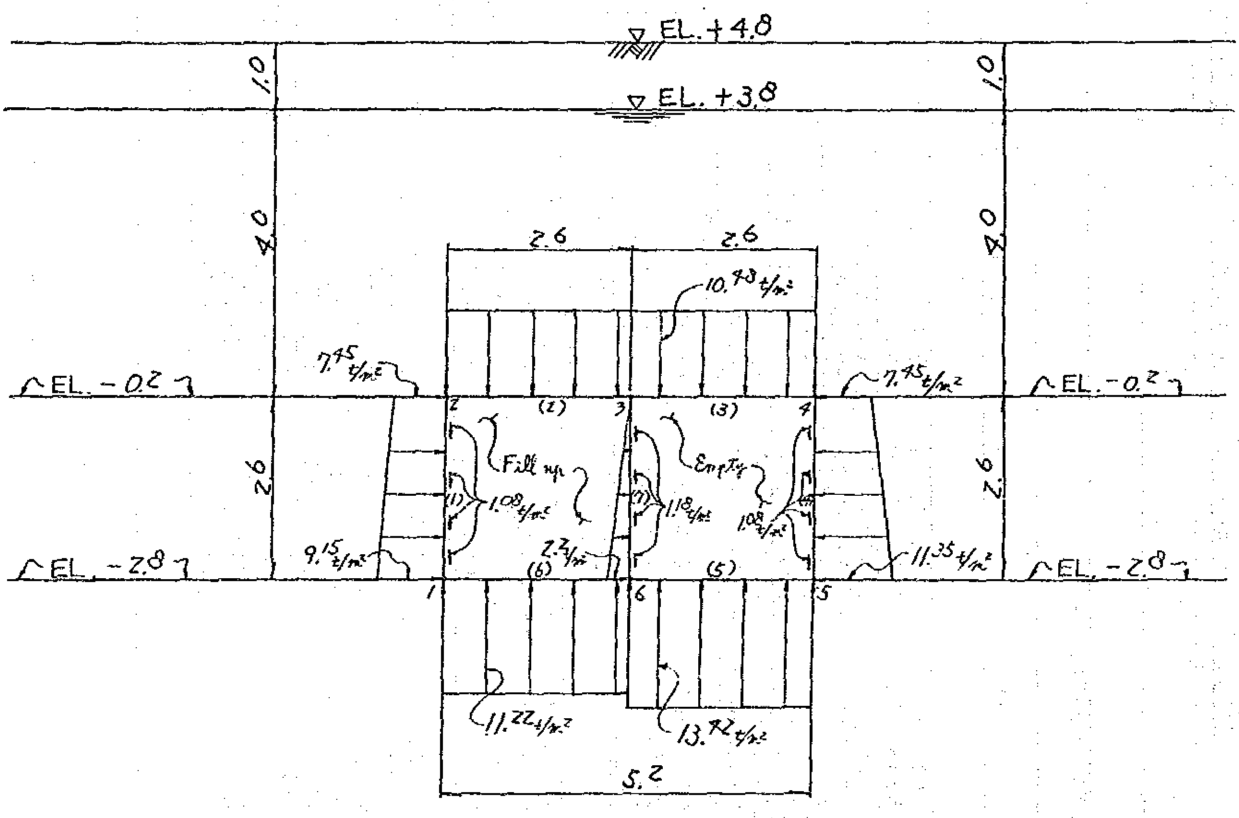

Fig 15. The load diagram

4) Input data for the sectional dimensions

Input data for the sectional dimensions are all the same values as those of Case−1 (vid.Table 3, P17).

5) The computer calculataion results

The computer calculation results are shown in the following figures and table (Fig 16−18 and Table 6).

WWTPP DISCHARGE TUNNEL CASE-3(INSPECTION)

BENDING MOMENT

SHEET 30 OF

(TON*M)

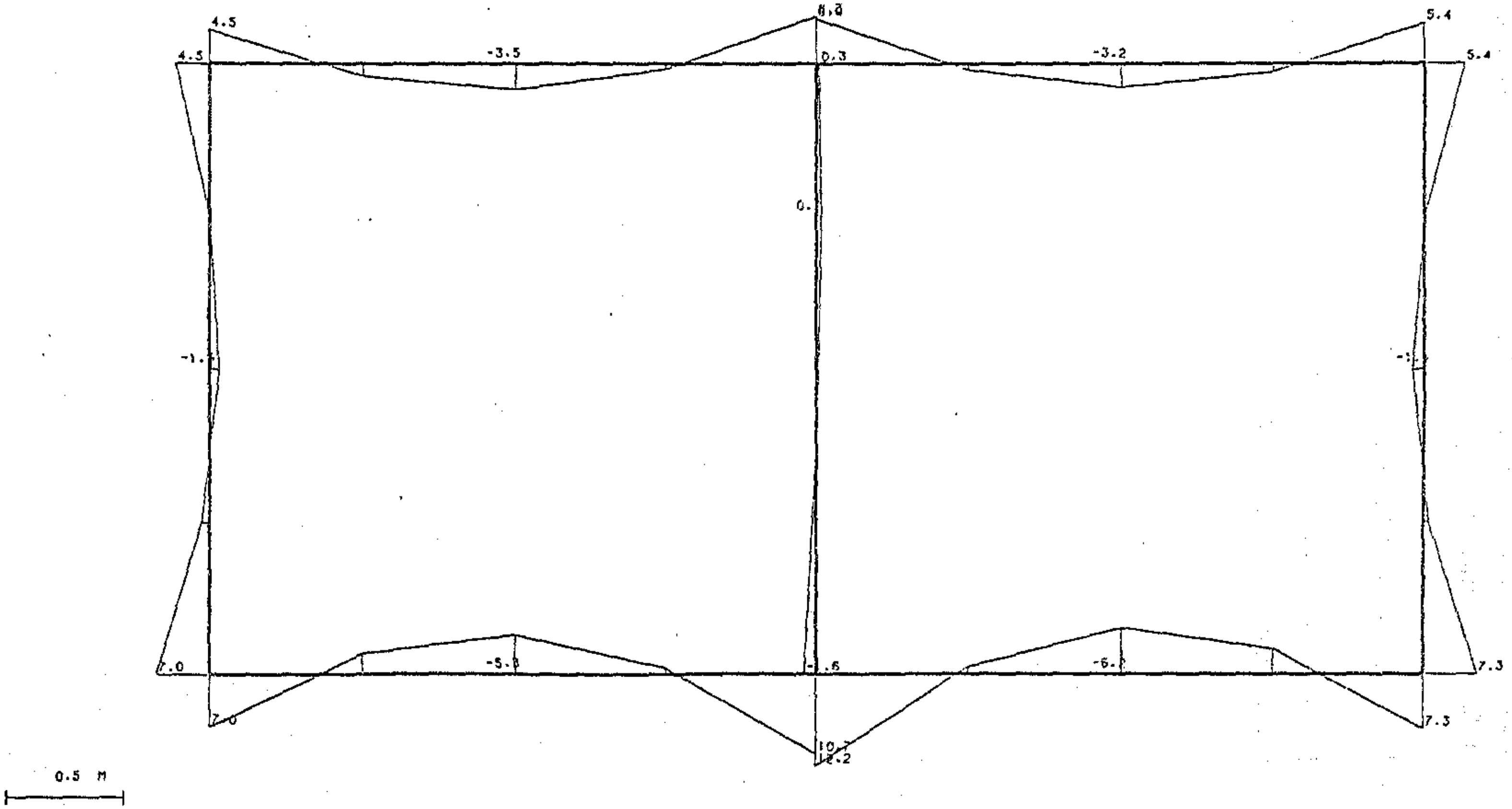

Fig 16. The bending moment diagram

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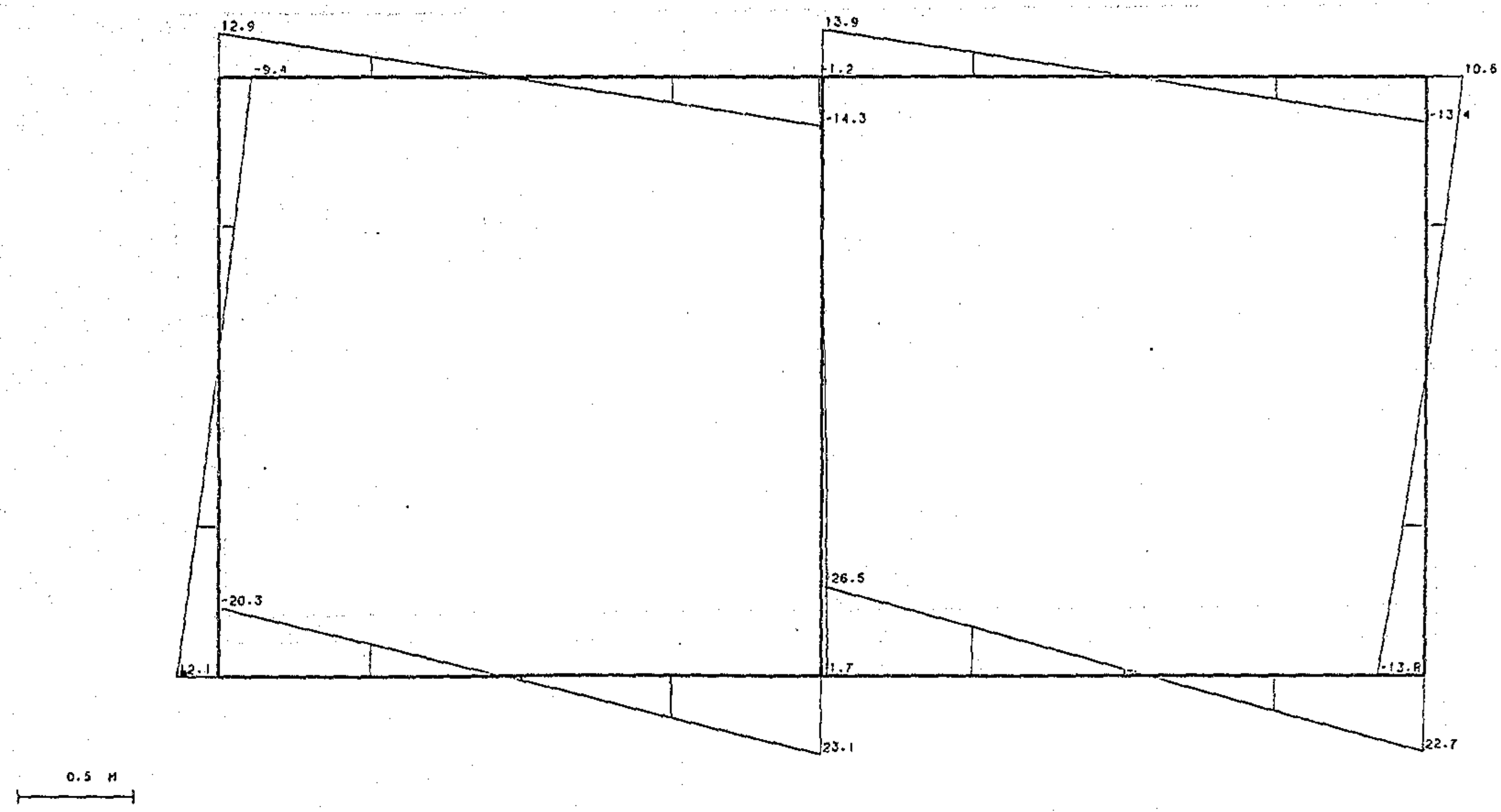
WWTPP DISCHARGE TUNNEL CASE-3 (INSPECTION)

SHEARING FORCE

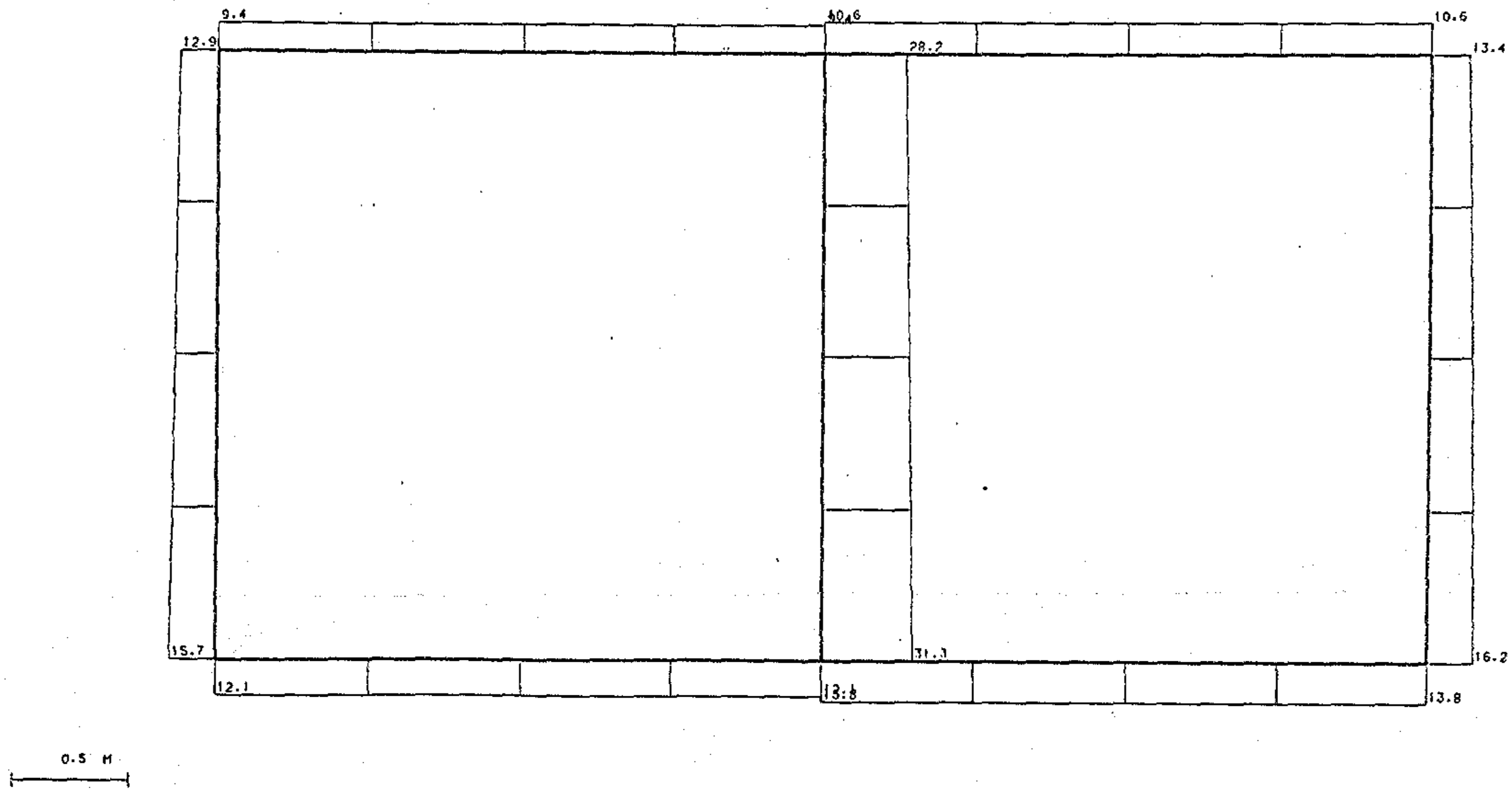


(TON)

Fig 17. The shearing force diagram

WWTPP DISCHARGE TUNNEL CASE-3(INSPECTION)

AXIAL FORCE



(TON)

Fig 18. The axial force diagram

1338

Table 6. The calculation results of the sectional forces (at Inspection)

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	1.5721E+01	1.2138E+01	7.0228E+00	7	1.5019E+01	6.3288E+00	1.0390E+00
2	7	1.5019E+01	6.3288E+00	1.0360E+00	8	1.4317E+01	7.9565E-01	-1.2615E+00
3	8	1.4317E+01	7.9565E-01	-1.2645E+00	9	1.3615E+01	-4.4612E+00	-5.5181E-02
4	9	1.3615E+01	-4.4612E+00	-5.8174E-02	2	1.2913E+01	-9.4419E+00	4.4783E+00
5	2	9.4419E+00	1.2913E+01	4.4753E+00	10	9.4419E+00	6.1015E+00	-1.7046E+00
6	10	9.4419E+00	6.1015E+00	-1.7046E+00	11	9.4419E+00	-7.1050E-01	-3.4567E+00
7	11	9.4419E+00	-7.1050E-01	-3.4567E+00	12	9.4419E+00	-7.5225E+00	-7.8093E-01
8	12	9.4419E+00	-7.5225E+00	-7.8093E-01	3	9.4419E+00	-1.4335E+01	6.3226E+00
9	3	1.0619E+01	1.3859E+01	5.9949E+00	13	1.0619E+01	7.0469E+00	-7.9944E-01
10	13	1.0619E+01	7.0469E+00	-7.9944E-01	14	1.0619E+01	2.3491E-01	-3.1660E+00
11	14	1.0619E+01	2.3491E-01	-3.1660E+00	15	1.0619E+01	-6.5771E+00	-1.1048E+00
12	15	1.0619E+01	-6.5771E+00	-1.1048E+00	4	1.0619E+01	-1.3389E+01	5.3842E+00
13	4	1.3389E+01	1.0619E+01	5.3842E+00	16	1.4091E+01	5.4596E+00	1.1746E-01
14	16	1.4091E+01	5.4596E+00	1.2433E-01	17	1.4793E+01	-3.3355E-01	-1.5828E+00
15	17	1.4793E+01	-3.3355E-01	-1.5760E+00	18	1.5495E+01	-6.7604E+00	6.8838E-01
16	18	1.5495E+01	-6.7604E+00	6.9525E-01	5	1.6197E+01	-1.3821E+01	7.3430E+00
17	5	1.3821E+01	2.2712E+01	7.3499E+00	19	1.3821E+01	1.0414E+01	-3.4158E+00
18	19	1.3821E+01	1.0414E+01	-3.4158E+00	20	1.3821E+01	-1.8844E+00	-6.1878E+00
19	20	1.3821E+01	-1.8844E+00	-6.1878E+00	21	1.3821E+01	-1.4182E+01	-9.6607E-01
20	21	1.3821E+01	-1.4182E+01	-9.6607E-01	6	1.3821E+01	-2.6480E+01	1.2249E+01
21	6	1.2138E+01	2.3143E+01	1.0680E+01	22	1.2138E+01	1.2275E+01	-8.3051E-01
22	22	1.2138E+01	1.2275E+01	-8.3051E-01	23	1.2138E+01	1.4067E+00	-5.2769E+00
23	23	1.2138E+01	1.4067E+00	-5.2769E+00	24	1.2138E+01	-9.4613E+00	-2.6592E+00
24	24	1.2138E+01	-9.4613E+00	-2.6592E+00	1	1.2138E+01	-2.0329E+01	7.0228E+00
25	3	2.8193E+01	-1.1771E+00	3.2765E-01	25	2.8960E+01	7.4150E-02	6.6287E-01
26	25	2.8960E+01	7.4150E-02	6.6675E-01	26	2.9727E+01	9.6790E-01	3.0484E-01
27	26	2.9727E+01	9.6790E-01	3.0871E-01	27	3.0494E+01	1.5042E+00	-5.1794E-01
28	27	3.0494E+01	1.5042E+00	-5.1407E-01	6	3.1261E+01	1.6829E+00	-1.5731E+00

MESSAGE SUMMARY: MESSAGE NUMBER - COUNT

215 1

1.5.4 The Stress Calculation

Before calculating the stress, the sectional force for the structural design is determined by selecting one case among three design cases from a viewpoint of the safety design, and after the stress calculations executed, the stress calculation results are indicated in Table 7 and the arrangement of the reinforcing bars is shown in Fig 19.

[Discharge Tunnel]

Table. 7-1 The Calculation Results of The Stress

Member	Point	The Sectional Force M [kg·cm]	N [kg]	S [kg]	The Sectional Dimensions B [cm]	H [cm]	d [cm]	d' [cm]	The Arrangement of Reinforcing Bars D [mm]	Pitch [mm]	As [cm²] / A's [cm²]	The strees (kg/cm²) σ_b	σ_c	τ	Remarks
(1)	1	(Case-1) 670 000	(Case-1) 15 900	(Case-1) 11 900	100	50	40	20	D19 D19	150 300	19.1 9.5	555	30.1	3.0	As + A's ≧ 0.004B·H = 16 cm²
	Center	(Case-1) -130 000	(Case-1) 14 500	0	100	40	30	10	D19 D19	300 150	9.5 19.1	81	7.8	0	DITTO
	2	(Case-1) 480 000	(Case-1) 13 100	(Case-1) 9 700	100	50	40	20	D19 D19	150 300	19.1 9.5	361	21.4	2.4	DITTO
(2)	2	(Case-1) 480 000	(Case-1) 9 700	(Case-1) 13 100	100	50	40	20	D19 D19	150 300	19.1 9.5	436	21.7	3.3	DITTO
	Center	(Case-1) -330 000	(Case-1) 9 700	(Case-1) 0	100	40	30	10	D19 D19	300 150	9.5 19.1	710	28.0	0	DITTO
	3	(Case-1) 620 000	(Case-1) 9 700	(Case-1) 14 200	100	50	40	20	D19 D19	150 300	9.5 19.1	628	28.2	3.6	DITTO
(3)	3	(Case-1) 620 000	(Case-1) 9 700	(Case-1) 14 200	100	50	40	20	D19 D19	150 300	19.1 9.5	628	28.2	3.6	DITTO
	Center	(Case-1) -330 000	(Case-1) 9 700	0	100	40	30	10	D19 D19	300 150	9.5 19.1	710	28.0	0	DITTO
	4	(Case-1) 480 000	(Case-1) 9 700	(Case-1) 13 100	100	50	40	20	D19 D19	150 300	19.1 9.5	436	21.7	3.3	DITTO
(4)	4	(Case-1) 480 000	(Case-1) 13 100	(Case-1) 9 700	100	50	40	20	D19 D19	150 300	19.1 9.5	361	21.4	2.4	DITTO
	Center	(Case-1) -130 000	(Case-1) 14 500	0	100	40	30	10	D19 D19	300 150	9.5 19.1	81	7.8	0	DITTO
	5	(Case-1) 670 000	(Case-1) 15 900	(Case-1) 11 900	100	50	40	20	D19 D19	150 300	19.1 9.5	555	30.1	3.0	DITTO

Where
Mb : Bending moment
N : Axial force
S : Shearing force
B : The Width
H : The Height
d : The effective height
d' : The covering of compression bar
D : Diameter of bars
As : The area of tension bars
A's : The area of compression bars
σ_b : The bending stress
σ_c' : The compressive stress
τ : The shearing stress

[Discharge Tunnel]

Table. 7-2 The Calculation Results of The Stress

Member	Point	The Sectional Force M [kg·cm]	N [kg]	S [kg]	The Sectional Dimensions B [cm]	H [cm]	d [cm]	d' [cm]	The Arrangement of Reinforcing Bars D [mm]	Pitch [mm]	As [cm²] A's [cm²]	The strees [kg/cm²] σ_b	σ_c	τ	Remarks
(5)	5	(Case-1) 670 000	(Case-1) 11 900	(Case-1) 20 200	100	50	40	20	Φ22 Φ22	150 300	25.8 12.9	500	27.8	5.1	$A_s + A_s' \geqq 0.004 B \cdot H = 16 cm^2$
	Center	(Case-1) -540 000	(Case-1) 11 900	(Case-1) 0	100	40	30	10	Φ22 Φ22	300 150	12.9 25.8	1 057	42.2	0	DITTO
	6	(Case-1) 1 080 000	(Case-1) 11 900	(Case-1) 23 300	100	50	40	20	Φ22 Φ22	150 300	25.8 12.9	929	44.8	5.8	DITTO
(6)	6	(Case-1) 1 080 000	(Case-1) 11 900	(Case-1) 23 300	100	50	40	20	Φ22 Φ22	150 300	25.8 12.9	929	44.8	5.8	DITTO
	Center	(Case-1) -540 000	(Case-1) 11 900	(Case-1) 0	100	40	30	10	Φ22 Φ22	300 150	12.9 25.8	1 057	42.2	0	DITTO
	1	(Case-1) 670 000	(Case-1) 11 900	(Case-1) 20 200	100	50	40	20	Φ22 Φ22	150 300	25.8 12.9	500	27.8	5.1	DITTO
(7)	3	(Case-3) 30 000	(Case-3) 28 200	(Case-3) 1 200	100	60	40	20	Φ19 Φ19	300 300	9.5 9.5	70	5.0	0.3	$\sigma_{ca} = 87.5 kg/cm^2$ $\sigma_{sa} = 2000 kg/cm^2$ $\tau_a = 10.6 kg/cm^2$
	Center	(Case-3) 70 000	(Case-3) 29 700	(Case-3) 0	100	40	30	10	Φ19 Φ19	300 300	9.5 9.5	123	9.4	0	DITTO
	6	(Case-3) -160 000	(Case-3) 31 300	(Case-3) 1 700	100	60	40	20	Φ19 Φ19	300 300	9.5 9.5	88	7.6	0.4	DITTO

Where
Mb : Bending moment
N : Axial force
S : Shearing force
B : The Width
H : The Height
d : The effective height
d' : The covering of compression bar
D : Diameter of bars
As : The area of tension bars
A's : The area of compression bars
σ_b : The bending stress
σ_c' : The compressive stress
τ : The shearing stress

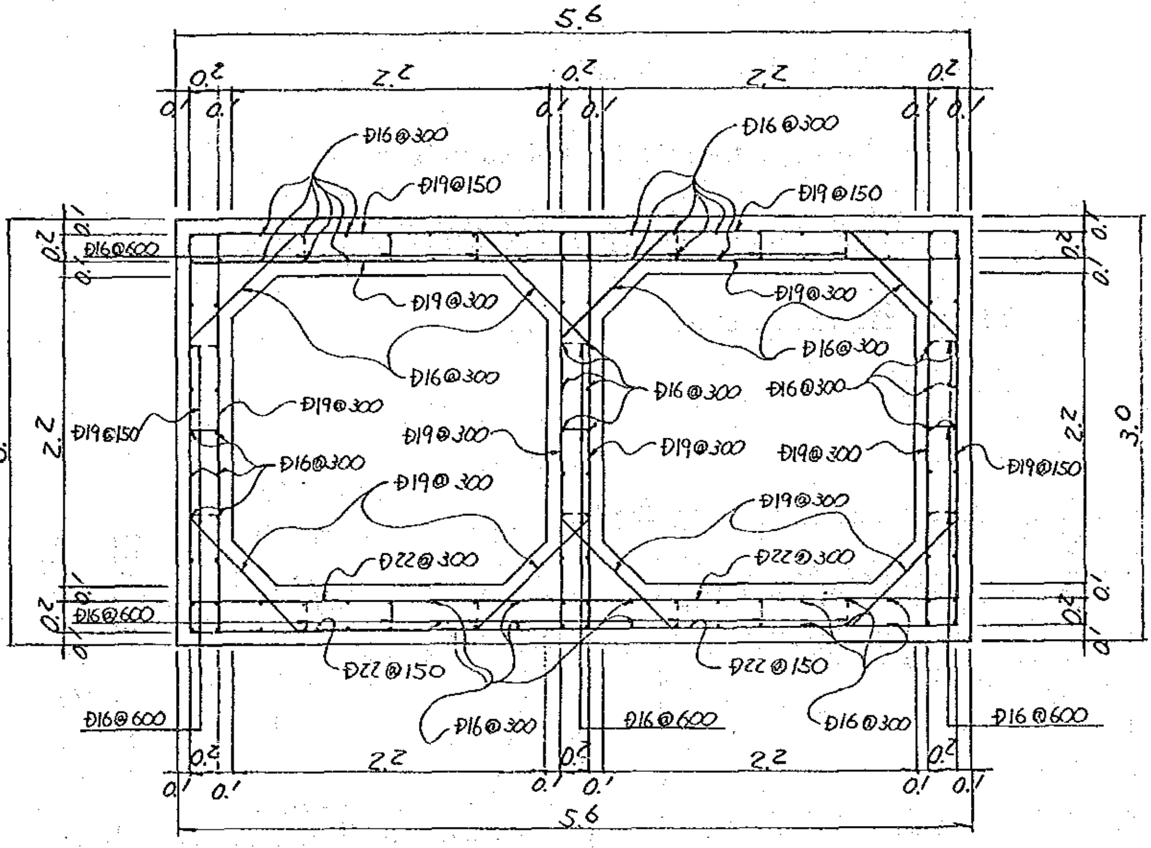


Fig 19. The arrangement of the reinforcing bars

1.5.5 Study of Open Pit

1) The load calculation (per 1m unit length)

a) The water pressure

$$P_w = 1.0 \times 2.37 = 2.4\ t/m^2$$

b) The earth pressure

$$P_{e0} = 0.5 \times 1.0 = 0.5\ t/m^2$$

$$P_{e1} = 0.5 \times (1.0 + 1.9 \times 1.0) = 1.45\ t/m^2$$

$$P_{e2} = 0.5 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 2.37) = 2.64\ t/m^2$$

The load diagram is shown in Fig 20.

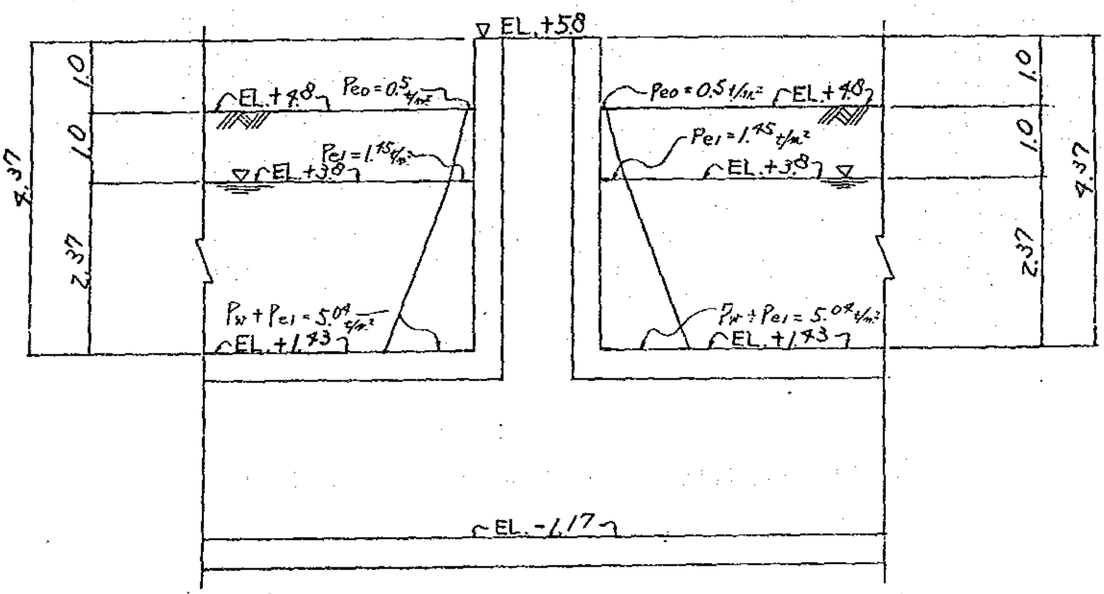


Fig 20. The load diagram

2) The structural design calculation

The design structure of the wall of Open Pit is considered for the two dimensional plate with three sides fixed and one side free, so the structural design calculation is executed as follows.

a) The bending moments

$$M_{x1} = 0.0096 \times 5.04 \times 4.8^2 = 1.1 \text{ t·m}$$

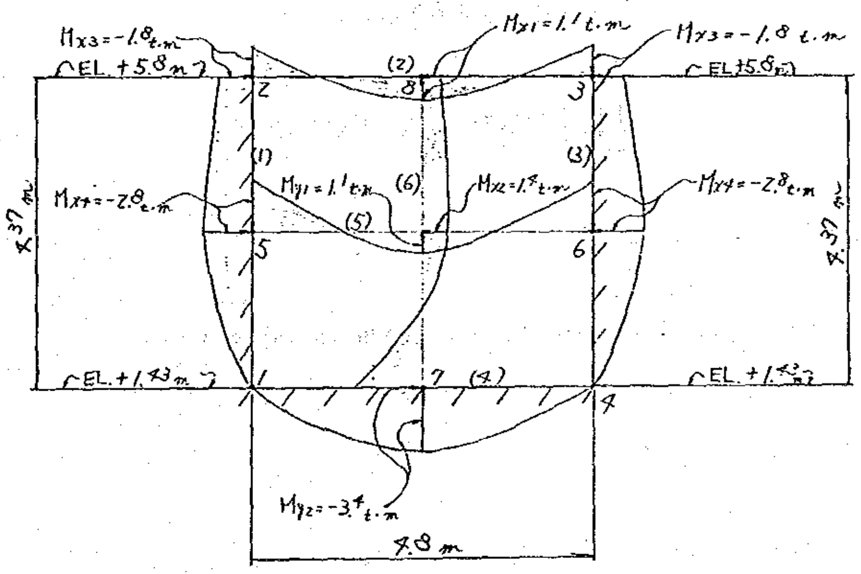
$$M_{x2} = 0.0118 \times 5.04 \times 4.8^2 = 1.4 \text{ t·m}$$

$$M_{y1} = 0.009 \times 5.04 \times 4.8^2 = 1.1 \text{ t·m}$$

$$M_{x3} = -0.0156 \times 5.04 \times 4.8^2 = -1.8 \text{ t·m}$$

$$M_{x4} = -0.0239 \times 5.04 \times 4.8^2 = -2.8 \text{ t·m}$$

$$M_{y2} = -0.029 \times 5.04 \times 4.8^2 = -3.4 \text{ t·m}$$



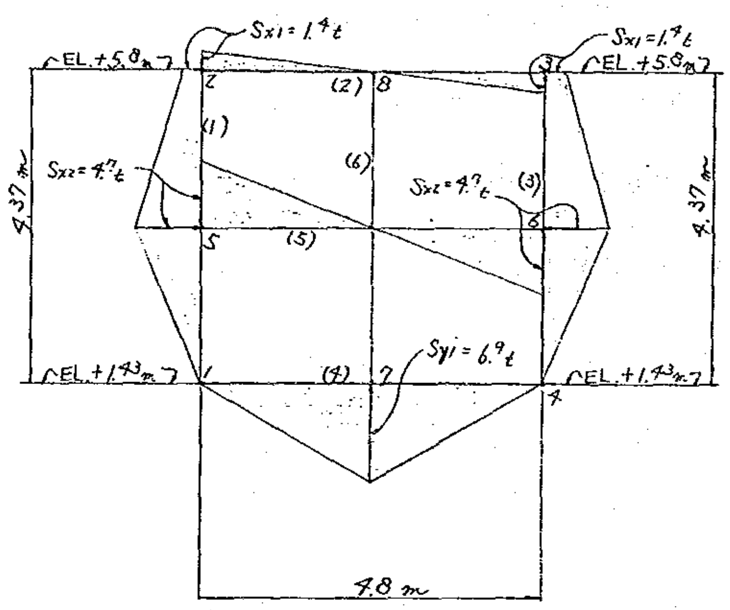
The bending moment diagram

b) The shearing forces

$S_{x1} = 0.057 \times 5.0^4 \times 4.8 = 1.4$ t

$S_{x2} = 0.194 \times 5.0^4 \times 4.8 = 4.7$ t

$S_{x3} = 0.286 \times 5.0^4 \times 4.8 = 6.9$ t



The shearing force diagram

3) The stress calculation

The stress calculation results are shown in Table 8.

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(Discharge Tunnel)

The wall of Open Pit

Table. 8-1 The Calculation Results of The Stress

Member	Point	The Sectional Force M [kg·cm]	N [kg]	S [kg]	The Sectional Dimensions B [cm]	H [cm]	d [cm]	d' [cm]	The Arrangement of Reinforcing Bars D [mm]	Pitch [mm]	As [cm²] / A's [cm²]	The strees (kg/cm²) σ_b	σ_c	τ	Remarks
(1)	1	0	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	0	0	0	As+As' ≧0.004B·H = 16cm²
	Center	-280 000	0	4 700	100	40	30	10	D19 D19	300 300	9.5 9.5	1 058	26.5	1.6	DITTO
	2	-180 000	0	1 400	100	40	30	10	D19 D19	300 300	9.5 9.5	680	17.1	0.5	DITTO
(2)	2	-180 000	0	1 400	100	40	30	10	D19 D19	300 300	9.5 9.5	680	17.1	0.5	DITTO
	Center	110 000	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	416	10.4	0	DITTO
	3	-180 000	0	1 400	100	40	30	10	D19 D19	300 300	9.5 9.5	680	17.1	0.5	DITTO
(3)	3	-180 000	0	1 400	100	40	30	10	D19 D19	300 300	9.5 9.5	680	17.1	0.5	DITTO
	Center	-280 000	0	4 700	100	40	30	10	D19 D19	300 300	9.5 9.5	1 058	26.5	1.6	DITTO
	4	0	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	0	0	0	DITTO
(4)	4	0	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	0	0	0	DITTO
	Center	-340 000	0	6 900	100	40	30	10	D19 D19	300 300	9.5 9.5	1 284	32.2	2.3	DITTO
	1	0	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	0	0	0	DITTO

Where
- Mb : Bending moment
- N : Axial force
- S : Shearing force
- B : The Width
- H : The Height
- d : The effective height
- d' : The covering of compression bar
- D : Diameter of bars
- As : The area of tension bars
- A's : The area of compression bars
- σ_b : The bending stress
- σ_c' : The compressive stress
- τ : The shearing stress

(Discharge Tunnel)

The wall of Open Pit

Table. 8-2 The Calculation Results of The Stress

Member	Point	The Sectional Force M [kg·cm]	N [kg]	S [kg]	The Sectional Dimensions B [cm]	H [cm]	d [cm]	d' [cm]	The Arrangement of Reinforcing Bars D [mm]	Pitch [mm]	As [cm²] A's [cm²]	The strees (kg/cm²) σ_b	σ_c	τ	Remarks
(5)	5	-280 000	0	4 700	100	40	30	10	D19 D19	300 300	9.5 9.5	1 058	26.5	1.6	As+As' ≧0.004B·H =16 cm²
	Center	110 000	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	416	10.4	0	DITTO
	6	-280 000	0	4 700	100	40	30	10	D19 D19	300 300	9.5 9.5	1 058	26.5	1.6	DITTO
(6)	7	-340 000	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	1 284	32.2	0	DITTO
	Center	140 000	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	529	13.3	0	DITTO
	8	110 000	0	0	100	40	30	10	D19 D19	300 300	9.5 9.5	416	10.4	0	DITTO

Where Mb : Bending moment
N : Axial force
S : Shearing force

B : The Width
H : The Height
d : The effective height
d' : The covering of compression bar

D : Diameter of bars
As : The area of tension bars
A's : The area of compression bars

σ_b : The bending stress
σ_c' : The compressive stress
τ : The shearing stress

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CV-7 STRUCTURAL CALCULATION OF OUTLET

Contents of this calculation note is shown as below.

1348

1.1 Soil Condition

Boring data around the construction area is shown in Fig 1.
Now the average N−value above the foundation level is calculated as follows.

$$\overline{N} = \frac{\frac{1}{2}\times 1.0\times\{(0+18)+(18+5)+(5+11)+(11+11)+(11+5)+(5+6)+0.5\times(6+10)\}}{6.5}$$

$$\fallingdotseq 8$$

According to the above calculation, the angle of the internal friction is calculated by the following equation.

$$\phi = (\sqrt{15\cdot\overline{N}} + 15)^{\circ} = (\sqrt{15\times 8} + 15)^{\circ} \fallingdotseq 26^{\circ}$$

The bulk density of soil above the ground water $r = 1.9\ t/m^3$

The bulk density of soil under the ground water $r' = 1.0\ t/m^3$

Moreover other design condition data are already indicated in "Civil Condition" (vid.ENGINEERING SHEET No. EWC−1001).

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IKESC WEST WHARF POWER PLANT
COOLING WATER WAY.

BORE LOG

BORE HOLE NO: 5

Date: 16.5.89 to 17.5.1989. Ground Elev: EL. + 3.5 m Ground Water Table: 1.90m

SCALE (m)	LEVEL (m)	THICKNESS (m)	SOIL NAME/DESCRIPTION
1–9	9.45	9.45	Grey to dark grey loose to medium dense fine to medium SAND with shell fragments. ⌐The Foundation level¬
10	10.45	1.00	Greyish brown, stiff silty CLAY.
11–20	20.45	10.00	Greyish brown dense fine to medium micaceous SAND.
21–22	22.00	1.55	Brown very dense fine to medium SAND with gravel.
23	23.00	1.00	Brown very dense fine SAND
24–30	30.50	7.50	Greyish brown hard Silty CLAY.
31			Borehole Completed.

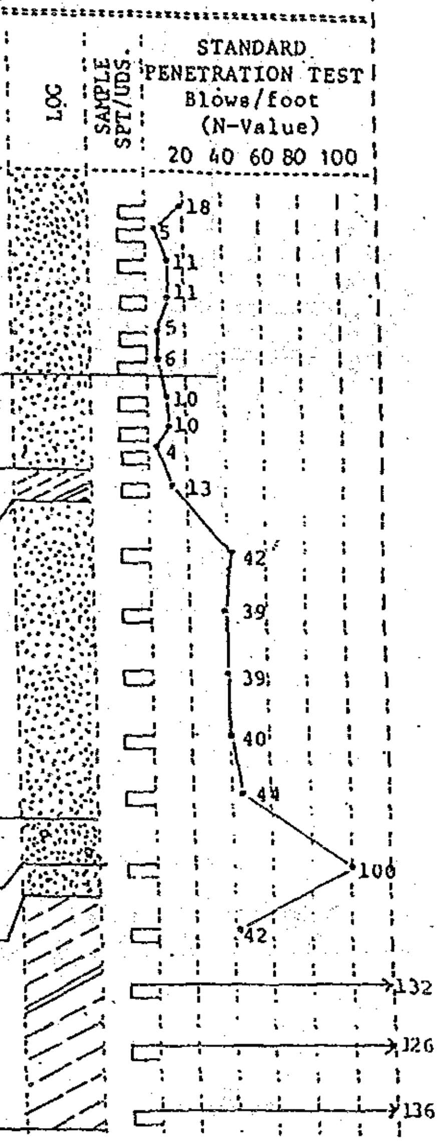

Fig 1. The soil column diagram

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1.2 Outline of Outlet

Plan of Outlet is shown in Fig 2, the design sections are Fig 3, Fig 4 and Fig 5.

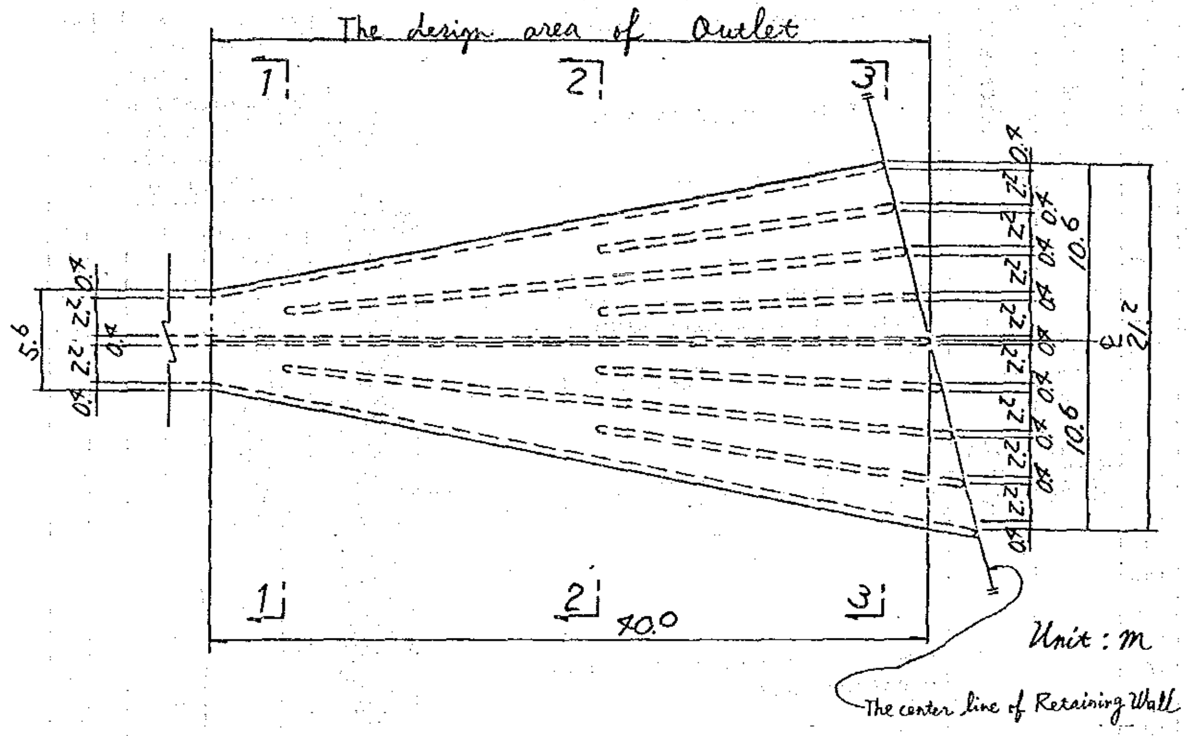

Fig 2. Plan of Outlet

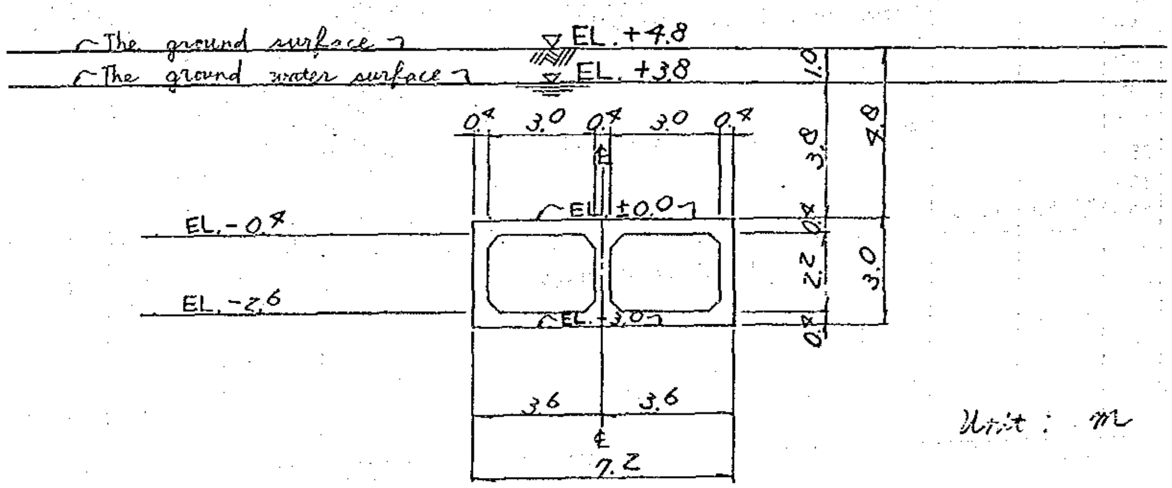

Fig 3. Section 1 - 1

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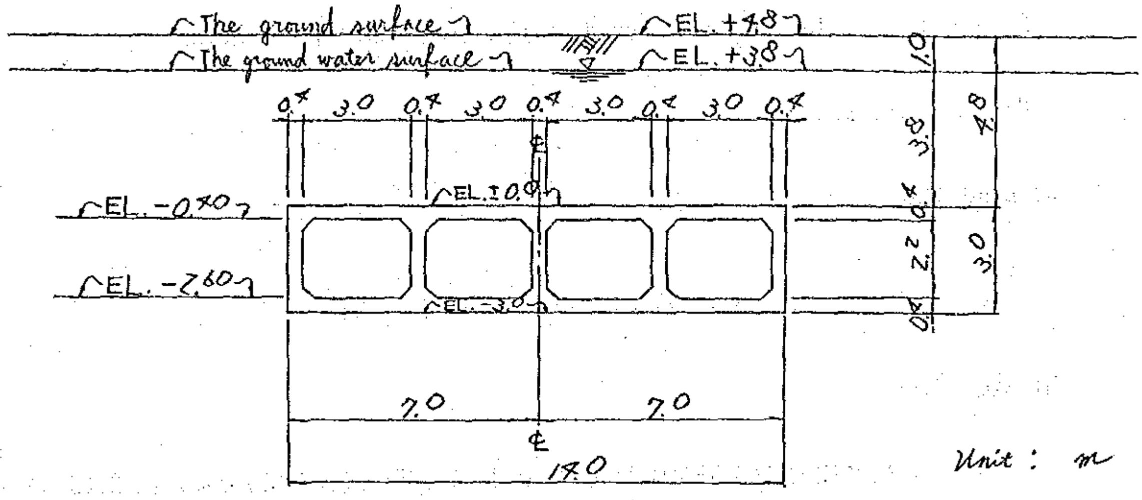

Fig 4. Section 2 − 2

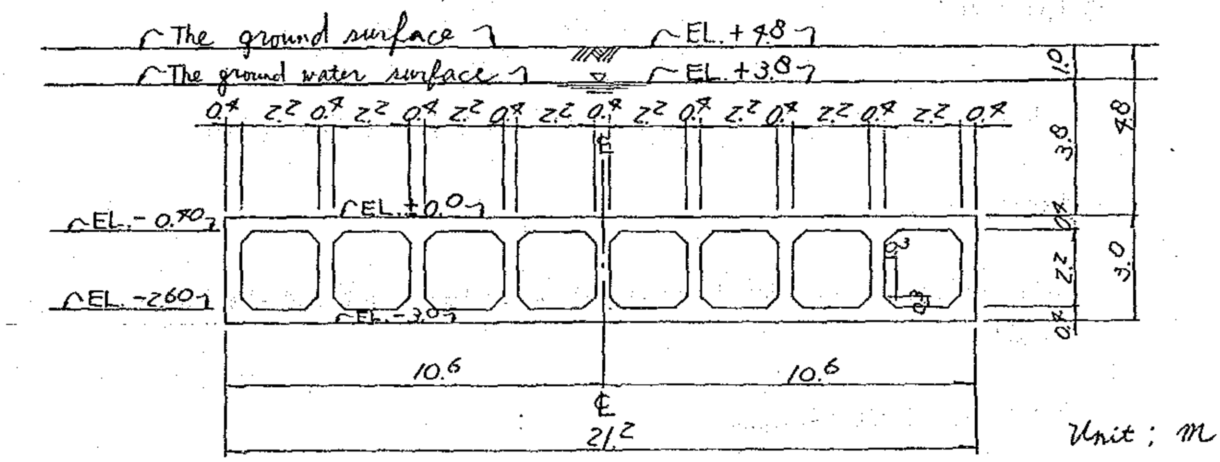

Fig 5. Section 3 − 3

Accordingly Section 1 is adopted for the design section of the structural design calculation of Outlet.

1.3 Stability Calculation

Stability calculation is executed at the longitudinal direction.

1) Vertical forces

a) Base slab

$$W_{c1} = \frac{1}{2} \times (5.6+21.2) \times 40.0 \times 0.4 \times 2.45 = 525 \text{ t}$$

b) Side wall

$$W_{c2} = \{2\times(2.2\times40.75\times0.4+0.3\times0.3\times40.75)+(2.2\times40.0\times0.4+2\times0.3\times0.3$$

$$\times40.0)+2\times(2.2\times35.9\times0.4+2\times0.3\times0.3\times35.9)+4\times(2.2\times18.5\times0.4$$

$$+2\times0.3\times0.3\times18.5)\}\times2.45$$

$$= 676 \text{ t}$$

c) Upper slab

$$W_{c3} = \frac{1}{2} \times (5.6 + 21.2) \times 40.0 \times 0.4 \times 2.45 = 525 \text{ t}$$

d) Water weight

Water weight is considered for the ground water weight and the internal water weight.

i) the ground water weight

$$W_{w1} = \frac{1}{2} \times (5.6 + 21.2) \times 40.0 \times 3.8 \doteqdot 2\,037 \text{ t}$$

ii) ther internal water weight

$$W_{w2} = \frac{1}{2} \times (5.6 + 21.2) \times 40 \times 2.2 - (676 \div 2.45)$$

$$= 903 \text{ t}$$

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e) Soil weight

Soil weight W_s is calculated for including the surcharage load $q = 1.0\ t/m^2$.

$$W_s = \frac{1}{2} \times (5.6+21.2) \times 40.0 \times (1.0+1.9 \times 1.0+1.0 \times 3.8)$$

$$= 3\,591\ t$$

f) Buoyancy

$$U_b = \frac{1}{2} \times (5.6 + 21.2) \times 40.0 \times 6.8 = 3\,645\ t$$

2) Horizontal forces

a) The external water pressure P_w

As the external water pressure P_w is working to both side walls, so the water pressure P_w is calculated as follows.

$$P_w = \frac{1}{2} \times 1.0 \times 6.8^2 \times (21.2 - 5.6)$$

$$= 361\ t$$

b) The earth pressure P_e

$$P_e = \frac{1}{2} \times \{(0.5+1.45) \times 1.0+(1.45+4.85) \times 6.8\} \times (21.2-5.6)$$

$$= 349\ t$$

Accordingly the calculation results of the external forces arae shown in Fig 6.

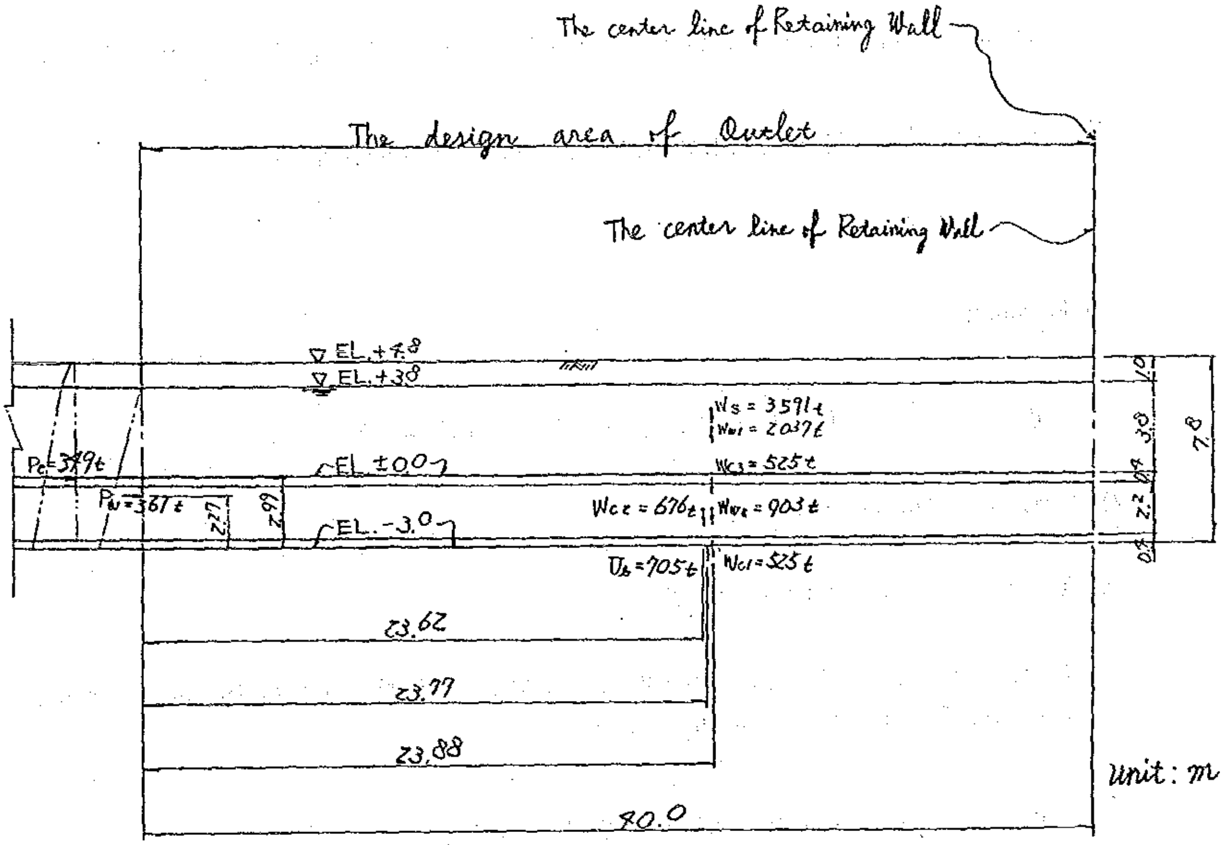

Fig 6. The calculation results of external forces

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3) The calculation of the ground reaction

a) The calculation of the eccentric distance

The eccentric distance is determined by the external moment calculations, then the summarized table of the external moments is shown in Table 1.

Table 1. The summarized table of the external moments

Species	Vertical force Vi[t]	Arm Xi [m]	Moment Mi [t·m]	Horizontal force Hi[t]	Arm Yi [m]	Moment Mi [t·m]
W_{c1}	525	23.88	12 537			
W_{c2}	676	23.62	15 967			
W_{c3}	525	23.88	12 537			
W_{w1}	2 037	23.88	48 644			
W_{w2}	903	13.88	21 564			
W_s	3 591	23.88	85 753			
V_b	−3 645	23.77	−86 642			
P_e				349	2.99	1 044
P_w				361	2.27	820
TOTAL	4 612		110 360	710		1 864

According to Table 1, the eccentric distance "e" is calculated as follows.

$$e = \frac{\Sigma M_i}{\Sigma V_i} - 23.88 = \frac{110\ 360 + 1\ 864}{4\ 612} - 23.88$$

$$= 24.33 - 23.88$$

$$\fallingdotseq 0.45\ \mathrm{m} < \frac{L}{6} = \frac{40}{6} = 6.67\ \mathrm{m}$$

Therefore working point pf the composite force at the basement is within the middle−third.

b) The calculation of the ground reaction q_{max}, q_{min}

$$\left.\begin{matrix} q_{max} \\ q_{min} \end{matrix}\right\} = \frac{\Sigma V_i}{B \cdot L} \quad \left(1 \pm \frac{6e}{L}\right)$$

$$= \frac{4\,612}{\frac{1}{2} \times (5.6+21.2) \times 40} \quad \left(1 \pm \frac{6 \times 0.45}{40}\right)$$

$$= \left\{ \begin{matrix} q_{max} = 10.34 \text{ t/m}^2 \\ q_{min} = 9.04 \text{ t/m}^2 \end{matrix} \right.$$

4) Study of the bearing capacity

a) The ultimate bearing capacity

The ultimate bearing capacity q_u is calculated as follows.

$$q_u = \alpha K C N_c + K q N_q + \frac{1}{2} r_i \beta B' N_r$$

where C : cohesion $C = 0$

q : the surcharge load

$$q = 1.9 \times 1.0 + 1.0 \times 6.8 = 8.7 \text{ t/m}^2$$

r_i : the bulk density of the bearing soil

$$r_i = 1.0 \text{ t/m}^3$$

B′ : the effective width considered for the eccentric distance

$$B' = 5.6 \text{ m}$$

α, β : the coefficient of the basic form

$$\alpha = 1 + 0.3 \times \frac{B'}{L'} = 1 + 0.3 \times \frac{5.6}{40 - 2 \times 0.23}$$

$$= 1.04$$

$$\beta = 1 - 0.4 \times \frac{B'}{L'} = 1 - 0.4 \times \frac{5.6}{40 - 2 \times 0.23}$$

$$= 0.94$$

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K : the extra coefficient for the embedded effect
$K = 1.0$

N_q, N_r : the bearing coefficients considered for the load inclination, and these coefficients are adopted from the following graphs.

$N_q = 8.3$ (from Fig 7.)

$N_r = 4.3$ (from Fig 8.)

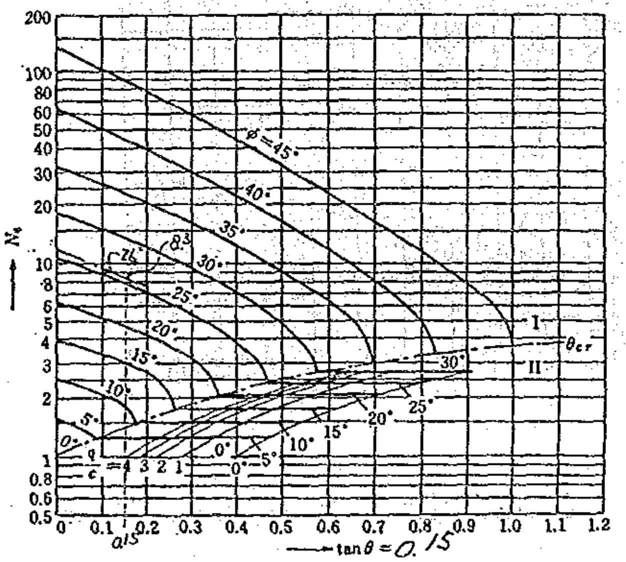

Fig 7. Graph of the bearing coefficient N_q

where $\tan\theta$: $\tan\theta = \frac{H}{V} = \frac{710}{4\,612} \fallingdotseq 0.15$

V : total vertical force at the basement
$V = 4\,612$ t

H : total horizontal force
$H = 710$ t

ϕ : the angle of the internal friction
$\phi = 26°$

FORM 04

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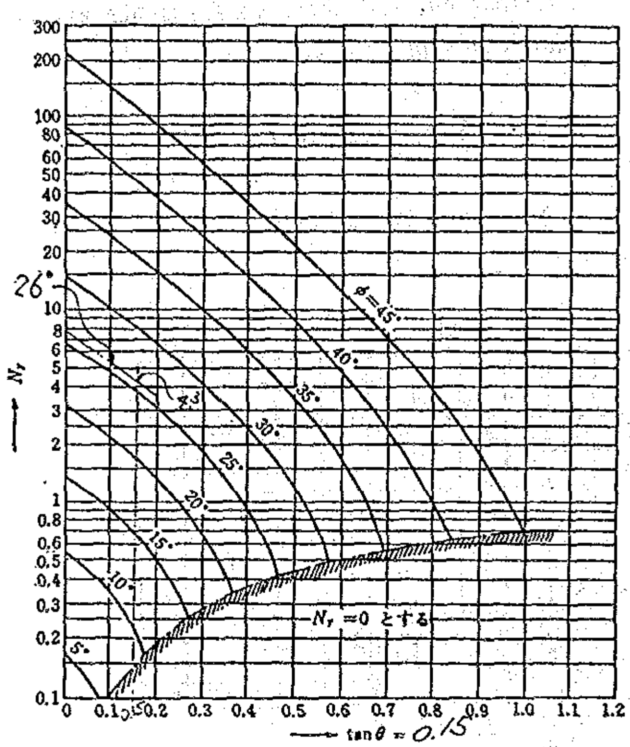

Fig 8. Graph of the bearing coefficient N_r

Accordingly the ultimate bearing capacity q_u is calculated as follows.

$$q_u = 1.0 \times 8.7 \times 8.3 + \frac{1}{2} \times 1.0 \times 0.94 \times 5.6 \times 4.3$$

$$= 83\ t/m^2$$

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b) The allowable bearing capacity q_a

The allowable bearing capacity q_a is calculated by the following equation.

$$q_a = \frac{1}{Fs} \cdot q_u$$

$$= \frac{1}{3} \times 83$$

$$= 27 \ t/m^2 > q_{max} = 10.34 \ t/m^2$$

O.K

Therefore the spread foundation is adopted for the foundation of Outlet.

5) Study of floating

The calculation of floating is executed at Normal and at Construction, so this calculation is as follows.

a) Total vertical force

i) at normal (L.L.W.L)

$$V_1 = W_{c1} + W_{c2} + W_{c3} + W_{w1} + W_{w2} + W_s = 8\ 257\ \text{t}$$

ii) at construction (Empty)

$$V_2 = W_{c1} + W_{c2} + W_{c3} + W_{w1} + W_s = 7\ 354\ \text{t}$$

b) Up lift U

Up lift U is calculated as below.

$$U = r_1 \cdot h_w \cdot A = 1.0 \times 6.8 \times \frac{1}{2} \times (5.6 + 21.2) \times 40.0$$

$$= 3\ 645\ \text{t}$$

c) Checking on the safety factor of Floating F_1

The safety factor of floating is checked by the following two cases.

i) at normal

$$F_{11} = \frac{V_1}{U} = \frac{8\ 257}{3\ 645} = 2.27 \underset{\text{O.K}}{>} 1.1$$

ii) at construction

$$F_{12} = \frac{V_2}{U} = \frac{7\ 354}{3\ 645} = 2.02 \underset{\text{O.K}}{>} 1.0$$

6) Study of sliding

The calculation of sliding is executed in the longitudinal direction at normal and at earthquake, so this calculation is as follows.

a) Total vertical force

i) at normal

$$V_1' = V_1 - U = 8\ 257 - 3\ 645 = 4\ 612\ t$$

ii) at earthquake

$$V_2' = V_1' = 4\ 612\ t$$

b) Total horizontal force

i) at normal

$$H_1 = \Sigma H_i = 710\ t$$

ii) at earthquake

① the water pressure

(the static water pressure)

$$P_w = 361\ t$$

② the active earth pressure

At first the coefficient of the seismic active earth pressure is calculated as below.

$$K_{ea} = \frac{\cos^2(\phi - \theta_0 - \theta)}{\cos\theta_0 \cdot \cos^2\theta \cdot \cos(\theta + \theta_0 + \delta)\left\{1 + \sqrt{\dfrac{\sin(\phi + \delta)\cdot\sin(\phi - \alpha - \theta_0)}{\cos(\theta + \theta_0 + \delta)\cdot\cos(\theta - \alpha)}}\right\}^2}$$

where ϕ : the angle of the internal friction $= 26°$

α : the angle between the ground surface and the horizontal plane $\alpha = 0°$

θ : the angle between the back wall surface and the vertical plane $\theta = 0°$

δ : the angle of the wall surface friction $\delta = 15°$

θ_0 : the composite angle of earthquake

$$\theta_0 = \tan^{-1} k_h = \tan^{-1} 0.1 \fallingdotseq 5.7°$$

Accordingly

$$K_{ea} = \frac{\cos^2(26° - 5.7° - 0°)}{\cos 5.7° \times \cos^2 0° \times \cos(5.7° + 15°) \times \left\{1 + \sqrt{\dfrac{\sin(26° + 15°) \times \sin(26° - 5.7°)}{\cos(5.7° + 15°) \times \cos 0°}}\right\}^2}$$

$= 0.426$

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Therefore the seismic active earth pressure is calculated as follows.

$$P_{EA} = \left(\frac{1}{2}\times0.426\times(1.0+1.0+1.9)\times1.0+\frac{1}{2}\times0.426\right.$$
$$\left.\times(1.0+1.9+1.0+1.9+1.0\times6.8)\times6.8\right)$$
$$\times(21.2-5.6)$$
$$= 298\ \text{t}$$

③ the force of inertia

$$H_i = K_h\cdot V_1 = 0.1\times(2\times525+676+3\ 591) = 532\ \text{t}$$

Accordingly total horizontal force at earthquake H_2 is calculated as below.

$$H_2 = P_w + P_{ew} + P_{EA} + H_i$$
$$= 361 + 42 + 298 + 532$$
$$= 1\ 233\ \text{t}$$

c) Checking on the safety factor of sliding F_s

i) at normal

$$F_{s1} = \frac{V_1'\cdot\tan\phi}{H_1} = \frac{4\ 612\times\tan26^\circ}{710}$$
$$= \frac{2\ 249}{710}$$
$$= 3.17 > 1.5$$

O.K

ii) at Earthquake

$$F_{s2} = \frac{V_1'\cdot\tan\phi}{H_2} = \frac{2\ 249}{1\ 233} = 1.82 \underset{\text{O.K}}{>} 1.2$$

The source : " Specification for Highway Bridges "

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1.4 Design Case

The following three cases are considered for the structural design cases at each design section.

Table 2. The summary of the design cases

Case	1	2	3
Condition	at Normal	at Construction	at Inspection
Period	Long term	Short term	Short term
Water Content	Full	Empty	Empty(oneside)
Self weight	Considered	Considered	Considered
The distributed surcharge load	1.0 t/m²	1.0 t/m²	1.0 t/m²
The incremental coefficient for the allowable stress	1.0	1.25	1.25

1.5 The Structural Design Calculation (at Section 1)

1.5.1 Case−1 (at Normal)

1) Frame of the design structure

Frame of the design structure is shown in Fig 9.

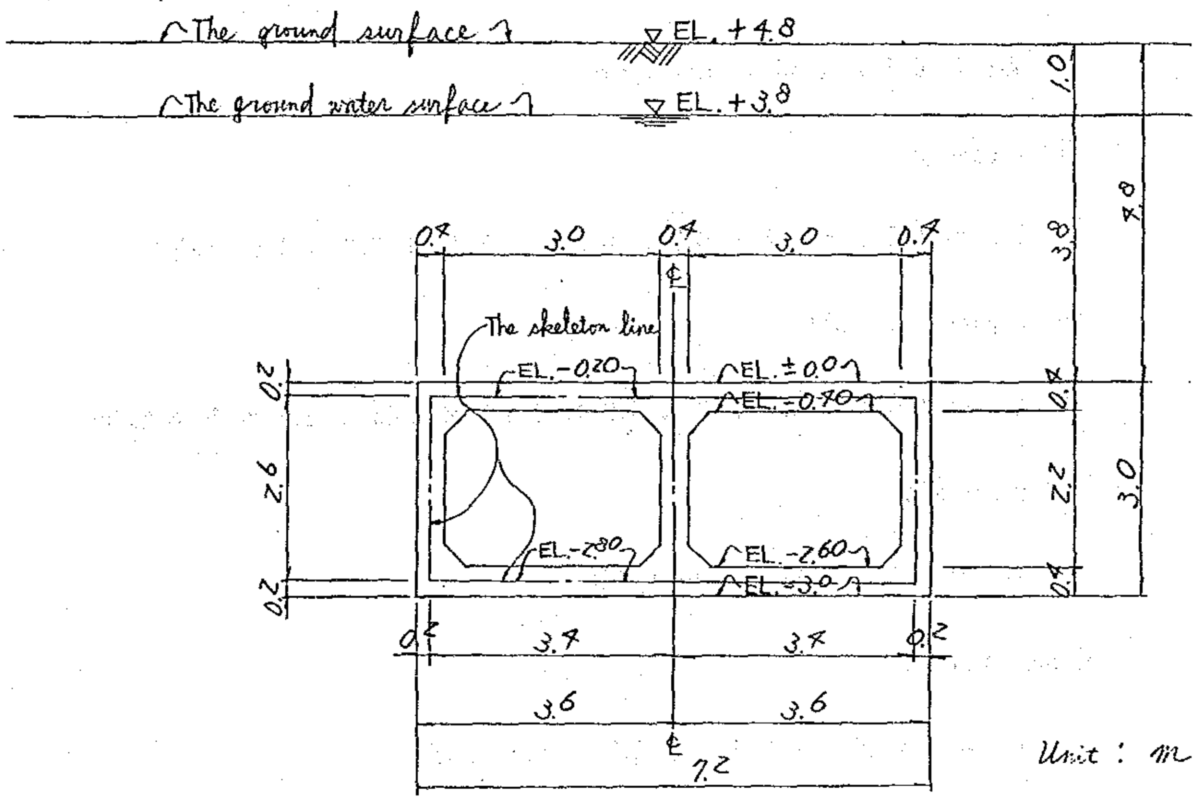

Fig 9. Frame of the design structure

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2) The load calculation (per 1m unit length)

a) The ground reaction q_r

$$q_r = 9.04 + \frac{35.9}{40} \times (14.58 - 13.60) = 9.9 \text{ t/m}^2$$

b) Concrete weight

i) base slab

$$W_{c1} = 0.4 \times 2.45 = 0.98 \text{ t/m}^2$$

ii) a side wall

$$W_{c2} = (0.4 \times 2.2 + 0.3 \times 0.3) \times 2.45 \div 2.2 = 1.08 \text{ t/m}^2$$

iii) a partition wall

$$W_{c3} = (0.4 \times 2.2 + 2 \times 0.3 \times 0.3) \times 2.45 \div 2.2 = 1.18 \text{ t/m}^2$$

iv) Upper slab

$$W_{c4} = 0.4 \times 2.45 = 0.98 \text{ t/m}^2$$

c) Soil weight

$$W_s = 1.9 \times 1.0 + 1.0 \times 3.8 = 5.7 \text{ t/m}^2$$

d) Water weight

i) the ground water weight

$$W_{w1} = 1.0 \times 3.8 = 3.8 \text{ t/m}^2$$

ii) the internal water weight

$$W_{w2} = 1.0 \times 2.2 = 2.2 \text{ t/m}^2$$

e) Up lift

$U = 1.0 \times 6.8 = 6.8\ t/m^2$

f) The water pressure

i) at outside

$P_{wo1} = 1.0 \times 4.0 = 4.0\ t/m^2$

$P_{wo2} = 1.0 \times 6.6 = 6.6\ t/m^2$

ii) at inside

$P_{wi} = 1.0 \times 2.2 = 2.2\ t/m^2$

g) The earth pressure

The ground surface surcharge load is considered for $q = 1.0\ t/m^2$

$P_{e1} = 0.5 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 4.0) = 3.45\ t/m^2$

$P_{e2} = 0.5 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 6.6) = 4.75\ t/m^2$

According to the above calculations, the results of the load calculations are shown in Fig 10.

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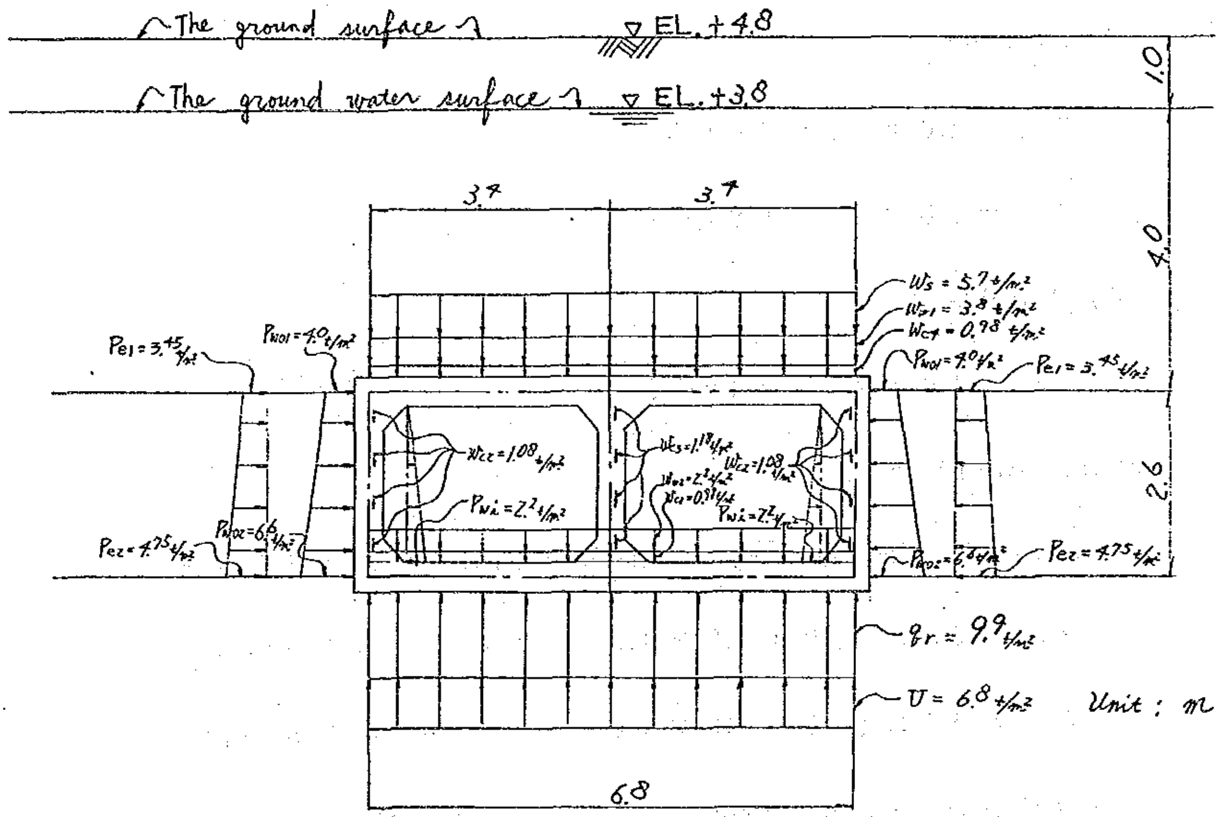

Fig 10. The results of the load calculations

3) The load diagram

The load diagram is shown in Fig 11.

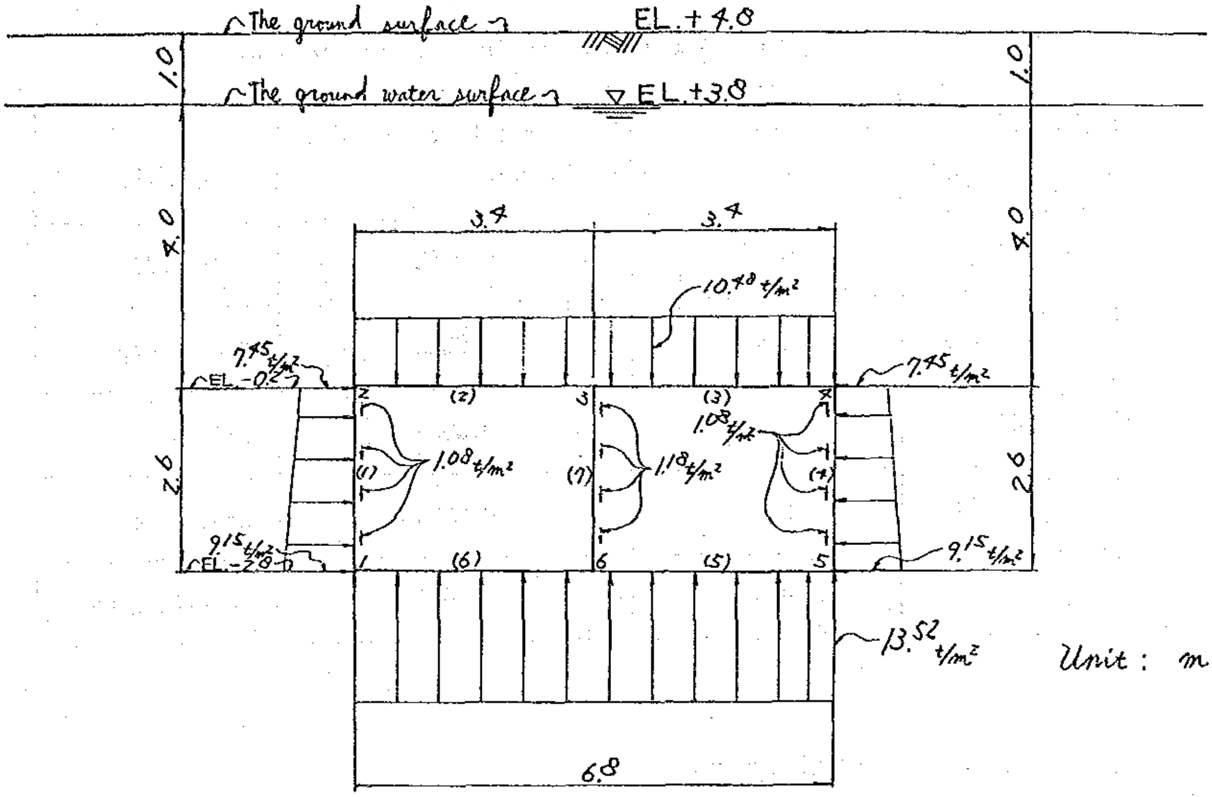

Fig 11. The load diagram

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4) Input data for the sectional dimensions

The sectional forces are calculated by computer, so input data for the sectional dimensions are summarized in Table 3.

Table 3. The sectional dimensions (per 1m unit length)

Member's number	The section area A [m^2]	The geomentrical moment of inertia I [m^4]	Remarks
(1)	0.4	0.0053	Side wall
(2) − (3)	0.4	0.0053	Upper slab
(4)	0.4	0.0053	Side wall
(5) − (6)	0.4	0.0053	Base slab
(7)	0.4	0.0053	Partition wall

5) The computer calculation results

The computer calculation results are shown in the following figures and table (Fig 12−14 and Table 4).

WEST WHARF DISCHARGE OUTLET CASE-1 (NORMAL)

BENDING MOMENT

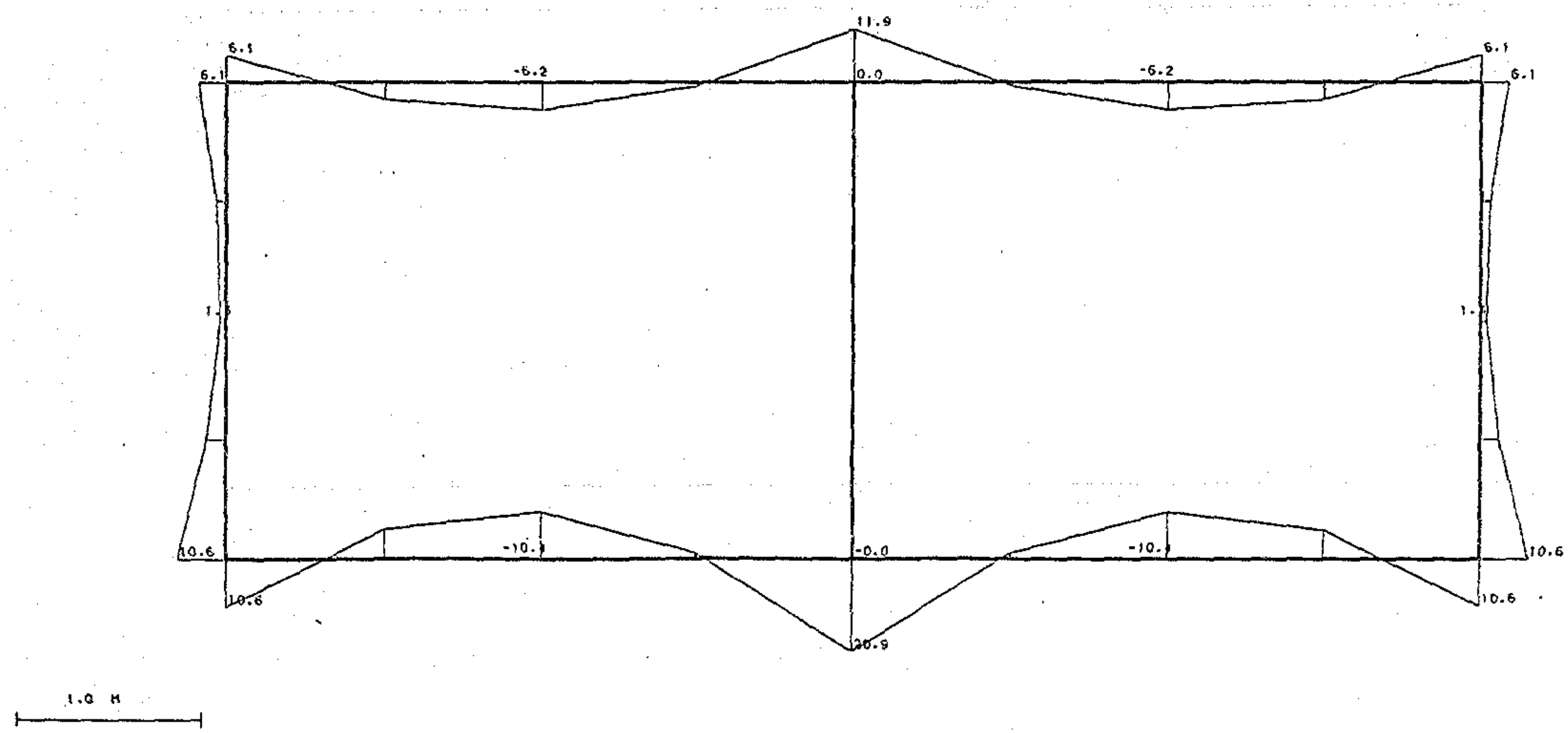

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Fig 12. The bending moment diagram

WEST WHARF DISCHARGE OUTLET CASE-1 (NORMAL)

SHEARING FORCE

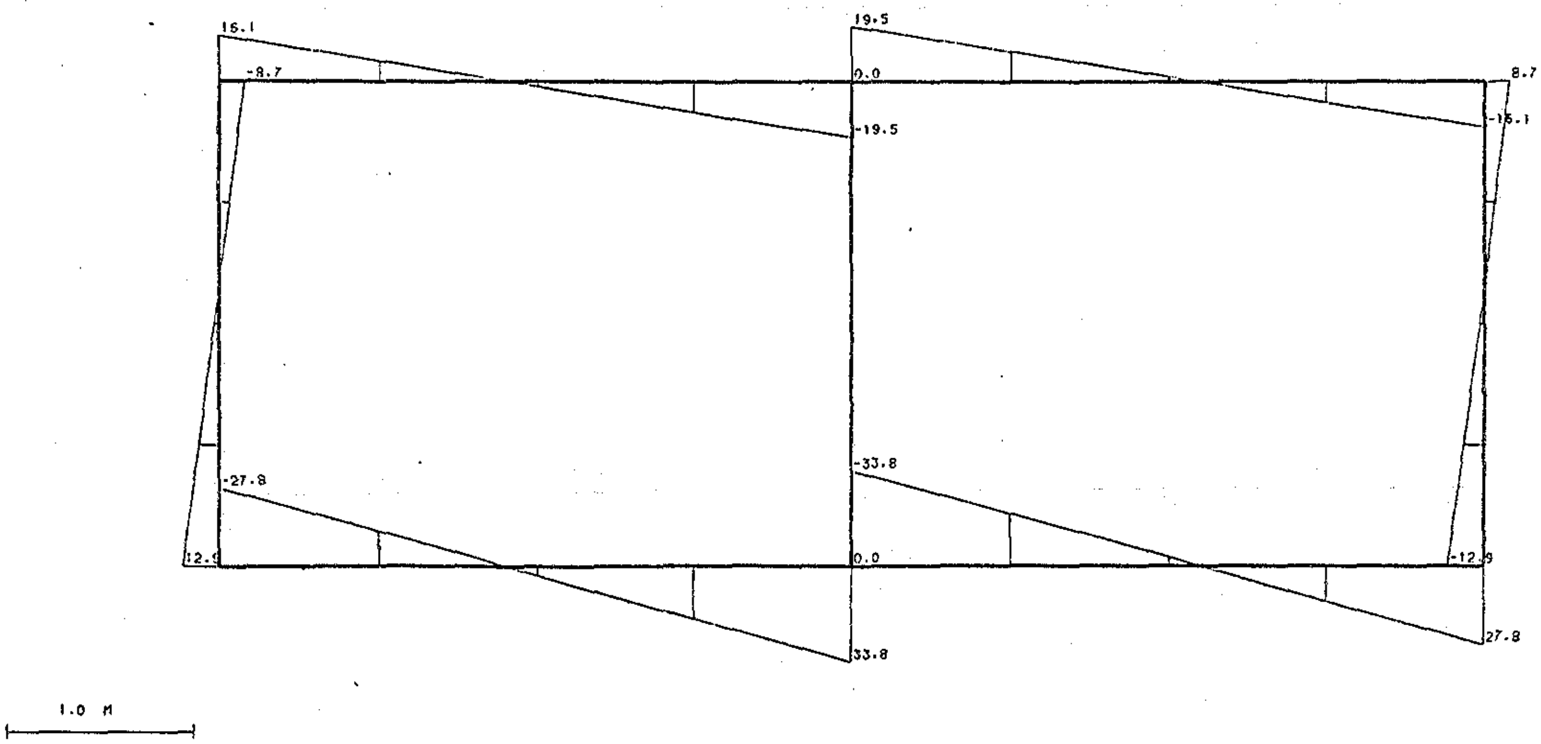

Fig 13. The shearing force diagram

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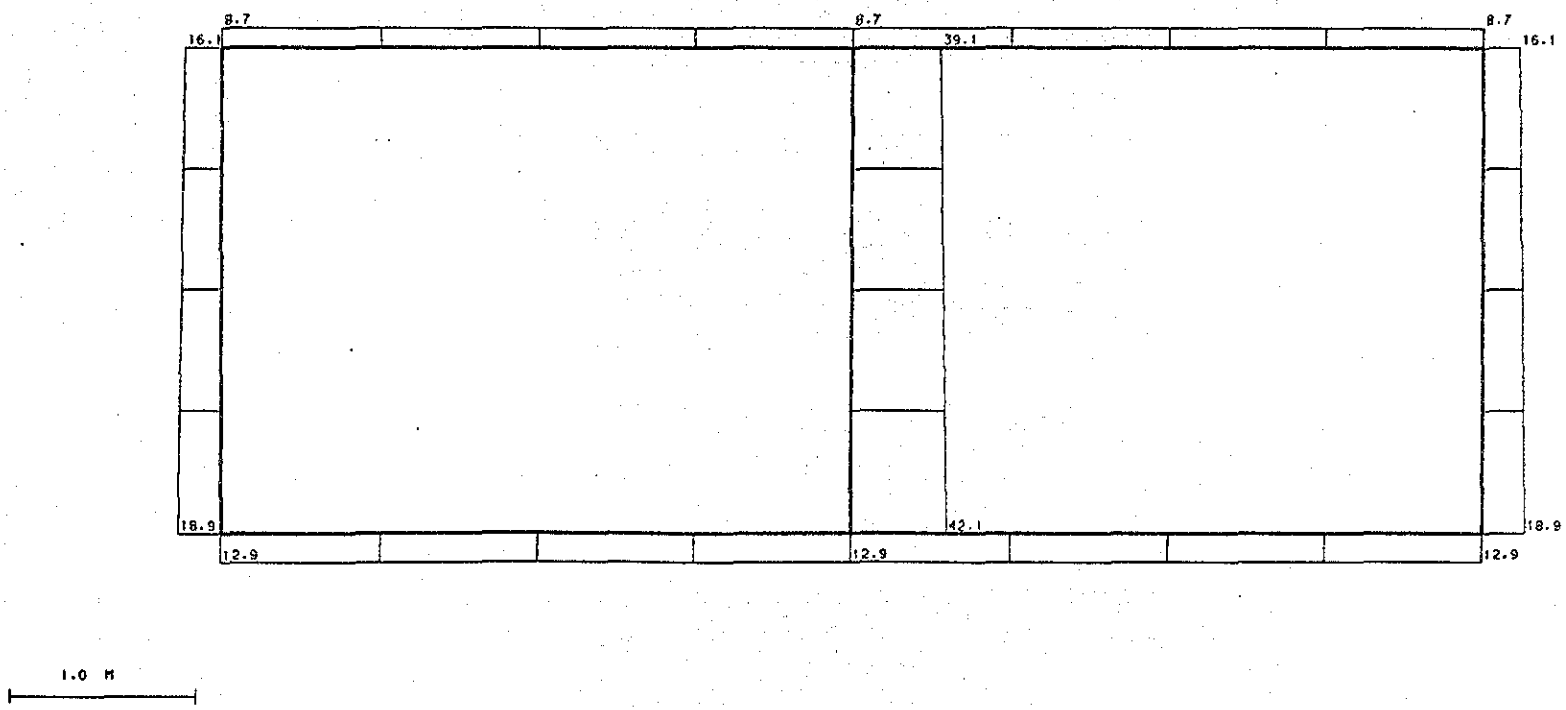
1375

WEST WHARF DISCHARGE OUTLET CASE-1 (NORMAL)

AXIAL FORCE



(TON)

Fig 14. The axial force diagram

Table 4. The calculation results of the rectional forces

** ELEPENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	1.8913E+01	1.2909E+01	1.0631E+01	7	1.8211E+01	7.0996E+00	4.1459E+00
2	7	1.8211E+01	7.0996E+00	4.1429E+00	8	1.7509E+01	1.5665E+00	1.3444E+00
3	8	1.7509E+01	1.5665E+00	1.3414E+00	9	1.6807E+01	-3.6904E+00	2.0497E+00
4	9	1.6807E+01	-3.6904E+00	2.0467E+00	2	1.6105E+01	-8.6710E+00	6.0821E+00
5	2	8.6710E+00	1.6105E+01	6.0791E+00	10	8.6710E+00	7.1966E+00	-3.8239E+00
6	10	8.6710E+00	7.1966E+00	-3.8239E+00	11	8.6710E+00	-1.7114E+00	-6.1552E+00
7	11	8.6710E+00	-1.7114E+00	-6.1552E+00	12	8.6710E+00	-1.0619E+01	-9.1465E-01
8	12	8.6710E+00	-1.0619E+01	-9.1465E-01	3	8.6710E+00	-1.9527E+01	1.1898E+01
9	3	8.6710E+00	1.9527E+01	1.1898E+01	13	8.6710E+00	1.0619E+01	-9.1465E-01
10	13	8.6710E+00	1.0619E+01	-9.1465E-01	14	8.6710E+00	1.7114E+00	-6.1552E+00
11	14	8.6710E+00	1.7114E+00	-6.1552E+00	15	8.6710E+00	-7.1966E+00	-3.8239E+00
12	15	8.6710E+00	-7.1966E+00	-3.8239E+00	4	8.6710E+00	-1.6105E+01	6.0791E+00
13	4	1.6105E+01	8.6710E+00	6.0791E+00	16	1.6807E+01	3.6904E+00	2.0437E+00
14	16	1.6807E+01	3.6904E+00	2.0467E+00	17	1.7509E+01	-1.5665E+00	1.3384E+00
15	17	1.7509E+01	-1.5665E+00	1.3414E+00	18	1.8211E+01	-7.0996E+00	4.1399E+00
16	18	1.8211E+01	-7.0996E+00	4.1429E+00	5	1.8913E+01	-1.2909E+01	1.0628E+01
17	5	1.2909E+01	2.7793E+01	1.0631E+01	19	1.2909E+01	1.2391E+01	-6.4475E+00
18	19	1.2909E+01	1.2391E+01	-6.4475E+00	20	1.2909E+01	-3.0110E+00	-1.0434E+01
19	20	1.2909E+01	-3.0110E+00	-1.0434E+01	21	1.2909E+01	-1.8413E+01	-1.3288E+00
20	21	1.2909E+01	-1.8413E+01	-1.3288E+00	6	1.2909E+01	-3.3815E+01	2.0868E+01
21	6	1.2909E+01	3.3815E+01	2.0863E+01	22	1.2909E+01	1.8413E+01	-1.3288E+00
22	22	1.2909E+01	1.8413E+01	-1.3288E+00	23	1.2909E+01	3.0110E+00	-1.0434E+01
23	23	1.2909E+01	3.0110E+00	-1.0434E+01	24	1.2909E+01	-1.2391E+01	-6.4475E+00
24	24	1.2909E+01	-1.2391E+01	-6.4475E+00	1	1.2909E+01	-2.7793E+01	1.0631E+01
25	3	3.9055E+01	5.7296E-15	4.2839E-15	25	3.9822E+01	5.7296E-15	5.5963E-16
26	25	3.9822E+01	5.7296E-15	5.5963E-16	26	4.0589E+01	5.7296E-15	-3.1646E-15
27	26	4.0589E+01	5.7296E-15	-3.1646E-15	27	4.1356E+01	5.7296E-15	-6.8889E-15
28	27	4.1356E+01	5.7296E-15	-6.8889E-15	5	4.2123E+01	5.7296E-15	-1.0613E-14

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1.5.2 Case – 2 (at Construction)

1) Frame of the design structure

Frame of the design structure is the same structure as that of Case–1 excluding a part that the internal water condition is empty.

2) The load calculation

The load calculations are the same calculations as those of case–1 excluding a part that the internal water loads are no considered (=0).

3) The load diagram

The load diagram is shown in Fig 15.

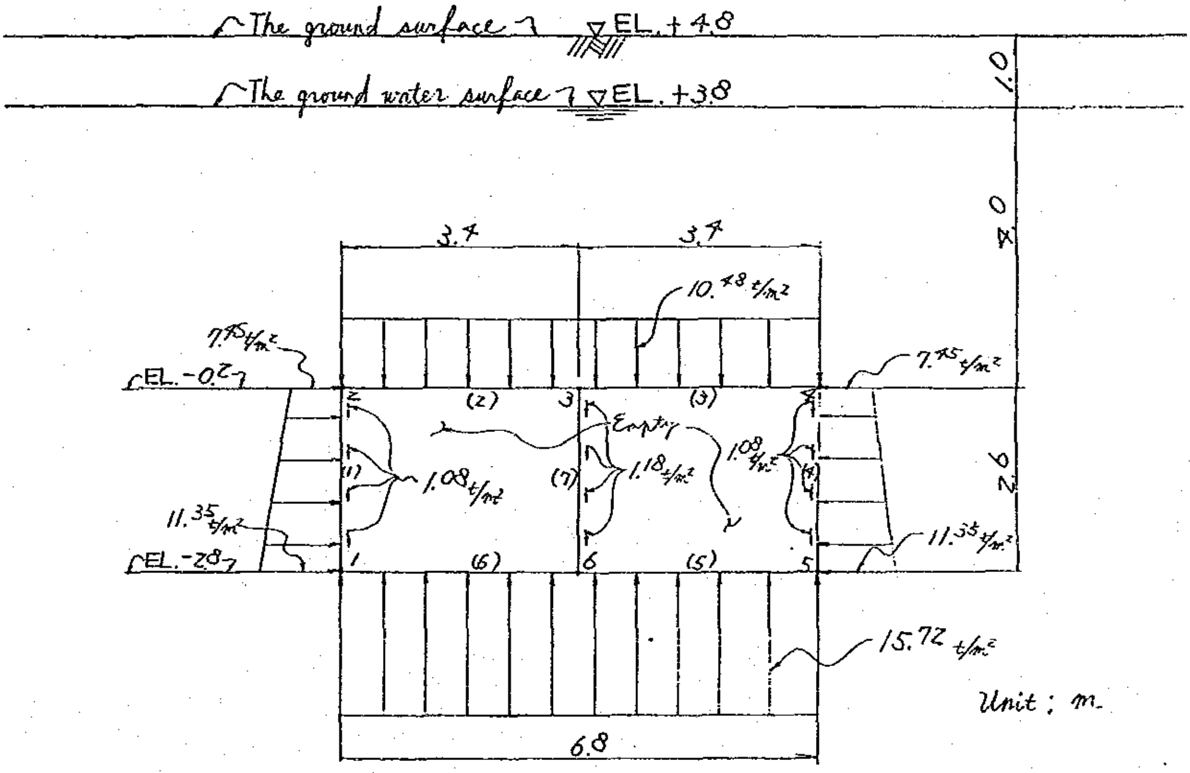

Fig 15. The load diagram

4) Input data for the sectional dimensions

Input data for the sectional dimensions are all the same values as those of Case−1.

5) The computer calculation results

The computer calculation results are shown in the following figures and table (Fig 16−18, Table 5)

WWTPP DISCHARGE OUTLET CASE-2 (AT CONSTRUCTION) BENDING MOMENT

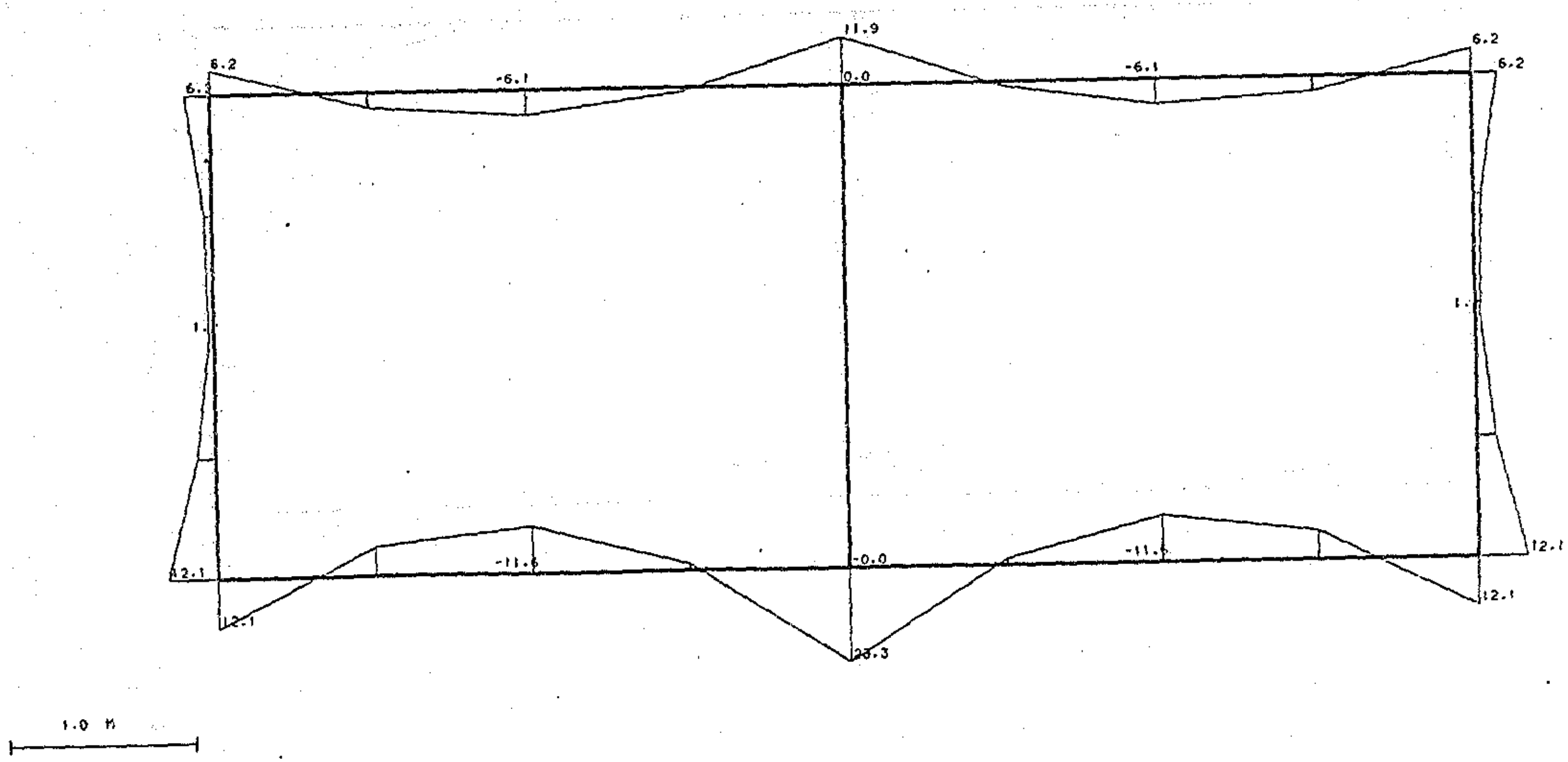

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Fig 16. The bending moment diagram

WWTPP DISCHARGE OUTLET CASE-2 (AT CONSTRUCTION) SHEARING FORCE

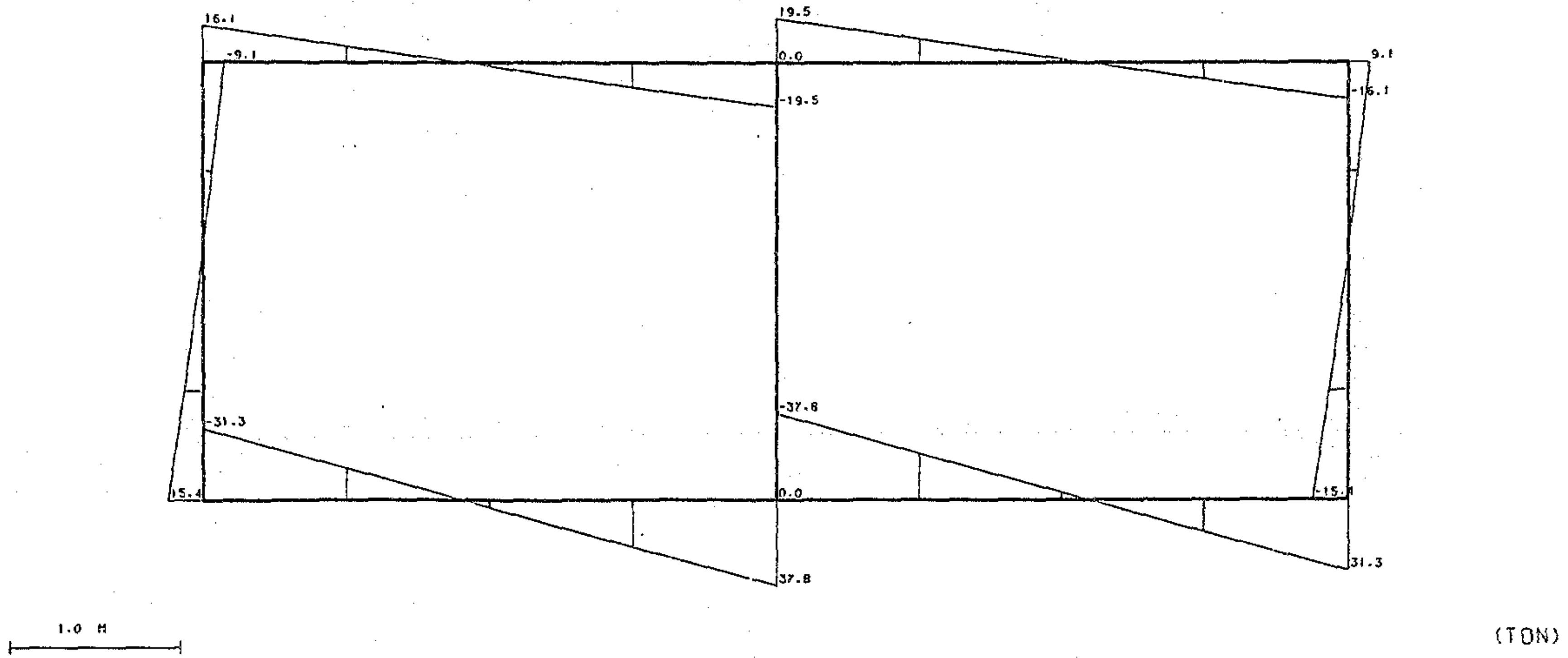

Fig 17. The shearing force diagram

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WWTPP DISCHARGE OUTLET CASE-2 (AT CONSTRUCTION) AXIAL FORCE

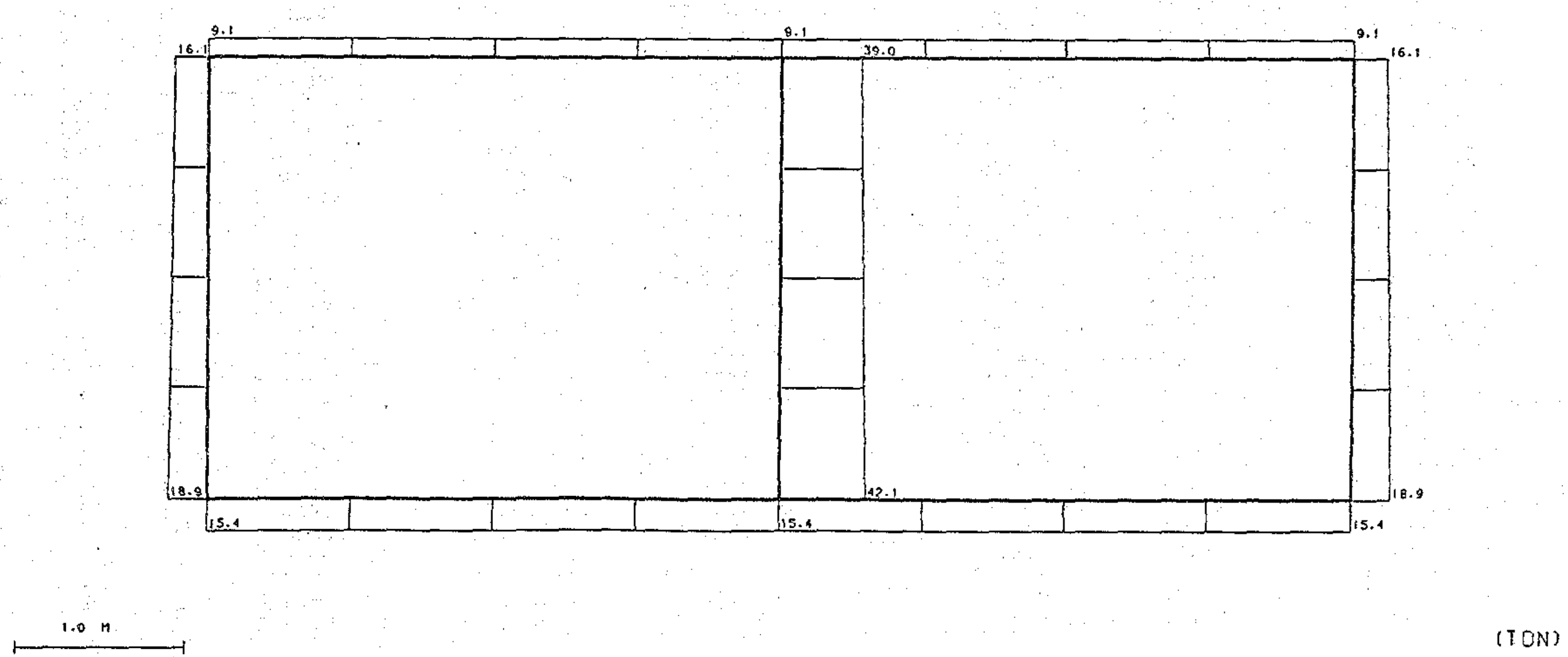

Fig 18. The axial force diagram

Table 5. The calculation results of the rectional forces

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	1.8946E+01	1.5367E+01	1.2141E+01	7	1.8244E+01	8.3062E+00	4.4880E+00
2	7	1.8244E+01	8.3062E+00	4.4811E+00	8	1.7542E+01	1.8793E+00	1.2120E+00
3	8	1.7542E+01	1.8793E+00	1.2052E+00	9	1.6840E+01	-3.9138E+00	1.9076E+00
4	9	1.6840E+01	-3.9138E+00	1.9007E+00	2	1.6138E+01	-9.0732E+00	6.1627E+00
5	2	9.0732E+00	1.6138E+01	6.1558E+00	10	9.0732E+00	7.2302E+00	-3.7758E+00
6	10	9.0732E+00	7.2302E+00	-3.7758E+00	11	9.0732E+00	-1.6778E+00	-6.1356E+00
7	11	9.0732E+00	-1.6778E+00	-6.1356E+00	12	9.0732E+00	-1.0586E+01	-9.2356E-01
8	12	9.0732E+00	-1.0586E+01	-9.2356E-01	3	9.0732E+00	-1.9494E+01	1.1860E+01
9	3	9.0732E+00	1.9494E+01	1.1860E+01	13	9.0732E+00	1.0586E+01	-9.2356E-01
10	13	9.0732E+00	1.0586E+01	-9.2356E-01	14	9.0732E+00	1.6778E+00	-6.1356E+00
11	14	9.0732E+00	1.6778E+00	-6.1356E+00	15	9.0732E+00	-7.2302E+00	-3.7758E+00
12	15	9.0732E+00	-7.2302E+00	-3.7758E+00	4	9.0732E+00	-1.6138E+01	6.1558E+00
13	4	1.6138E+01	9.0732E+00	6.1558E+00	16	1.6840E+01	3.9138E+00	1.8939E+00
14	16	1.6840E+01	3.9138E+00	1.9007E+00	17	1.7542E+01	-1.8793E+00	1.1983E+00
15	17	1.7542E+01	-1.8793E+00	1.2052E+00	18	1.8244E+01	-8.3062E+00	4.4743E+00
16	18	1.8244E+01	-8.3062E+00	4.4811E+00	5	1.8946E+01	-1.5367E+01	1.2134E+01
17	5	1.5367E+01	3.1264E+01	1.2141E+01	19	1.5367E+01	1.3992E+01	-7.0934E+00
18	19	1.5367E+01	1.3992E+01	-7.0934E+00	20	1.5367E+01	-3.2799E+00	-1.1646E+01
19	20	1.5367E+01	-3.2799E+00	-1.1646E+01	21	1.5367E+01	-2.0552E+01	-1.5176E+00
20	21	1.5367E+01	-2.0552E+01	-1.5176E+00	6	1.5367E+01	-3.7824E+01	2.3292E+01
21	6	1.5367E+01	3.7824E+01	2.3292E+01	22	1.5367E+01	2.0552E+01	-1.5176E+00
22	22	1.5367E+01	2.0552E+01	-1.5176E+00	23	1.5367E+01	3.2799E+00	-1.1646E+01
23	23	1.5367E+01	3.2799E+00	-1.1646E+01	24	1.5367E+01	-1.3992E+01	-7.0934E+00
24	24	1.5367E+01	-1.3992E+01	-7.0934E+00	1	1.5367E+01	-3.1264E+01	1.2141E+01
25	3	3.8988E+01	6.5155E-15	5.3791E-15	25	3.9755E+01	6.5155E-15	1.1441E-15
26	25	3.9755E+01	6.5155E-15	1.1441E-15	26	4.0522E+01	6.5155E-15	-3.0910E-15
27	26	4.0522E+01	6.5155E-15	-3.0910E-15	27	4.1289E+01	6.5155E-15	-7.3261E-15
28	27	4.1289E+01	6.5155E-15	-7.3261E-15	6	4.2056E+01	6.5155E-15	-1.1561E-14

1.5.3 Case − 3 (at Inspection)

1) Frame of the design structure

Frame of the design structure is the same structure as that of Case−1 excluding a part that the internal water condition is empty at oneside.

2) The load calculation

The load calculations are the same calculations as those of case−1 excluding a part that the internal water loads are no considered (=0) at oneside.

3) The load diagram

The load diagram is shown in Fig 19.

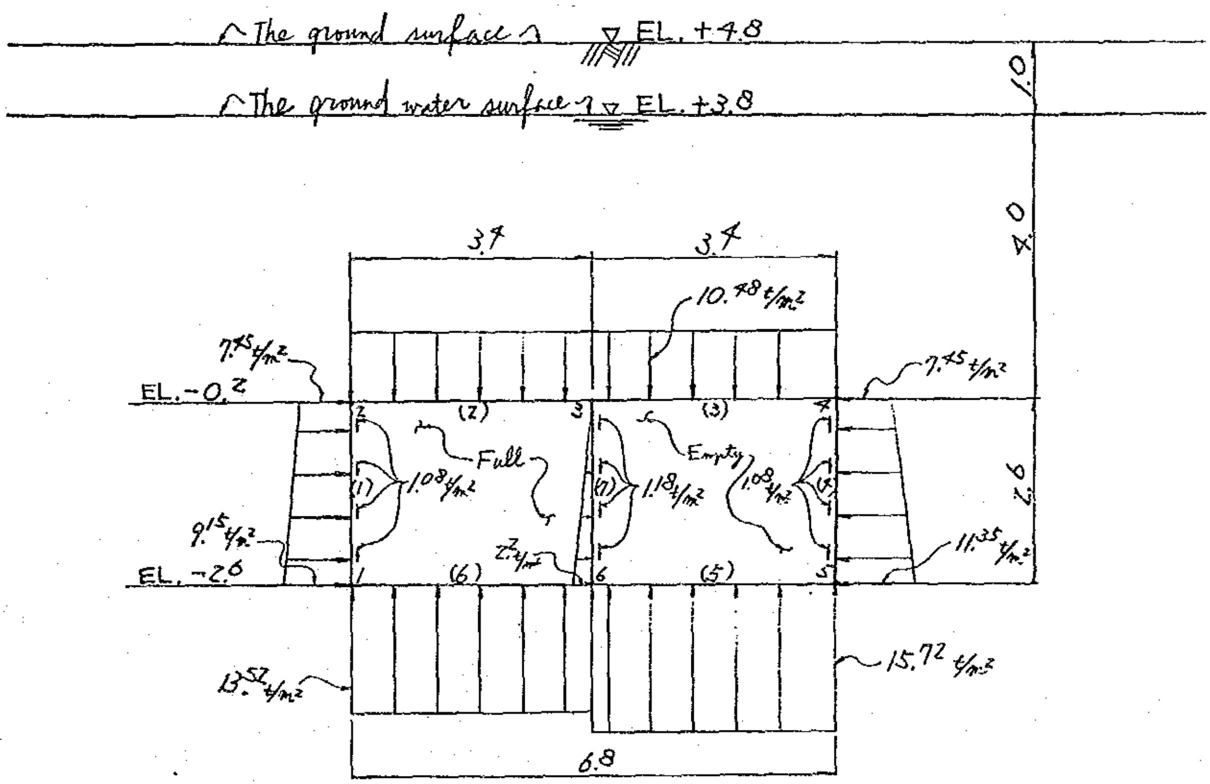

Fig 19. The load diagram

4) Input data for the sectional dimensions

Input data for the sectional dimensions are the all same values as those of Case−1.

5) The computer calculation results

The computer calculation results are shown in the following figures and table (Fig 20−22, Table 6)

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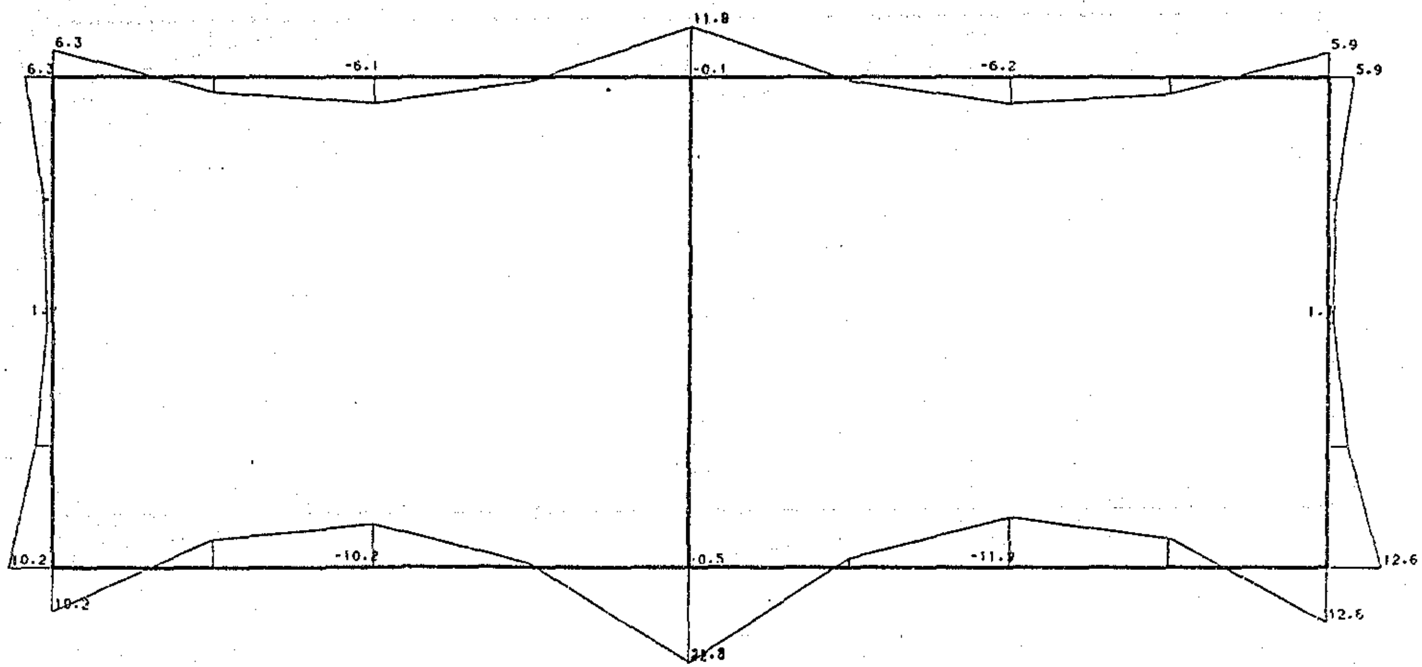
WWTPP DISCHARGE OUTLET CASE-3 (AT INSPECTION)

BENDING MOMENT

(TON*M)



1.0 M

Fig 20. The bending moment diagram

WWTPP DISCHARGE OUTLET CASE-3 (AT INSPECTION) SHEARING FORCE

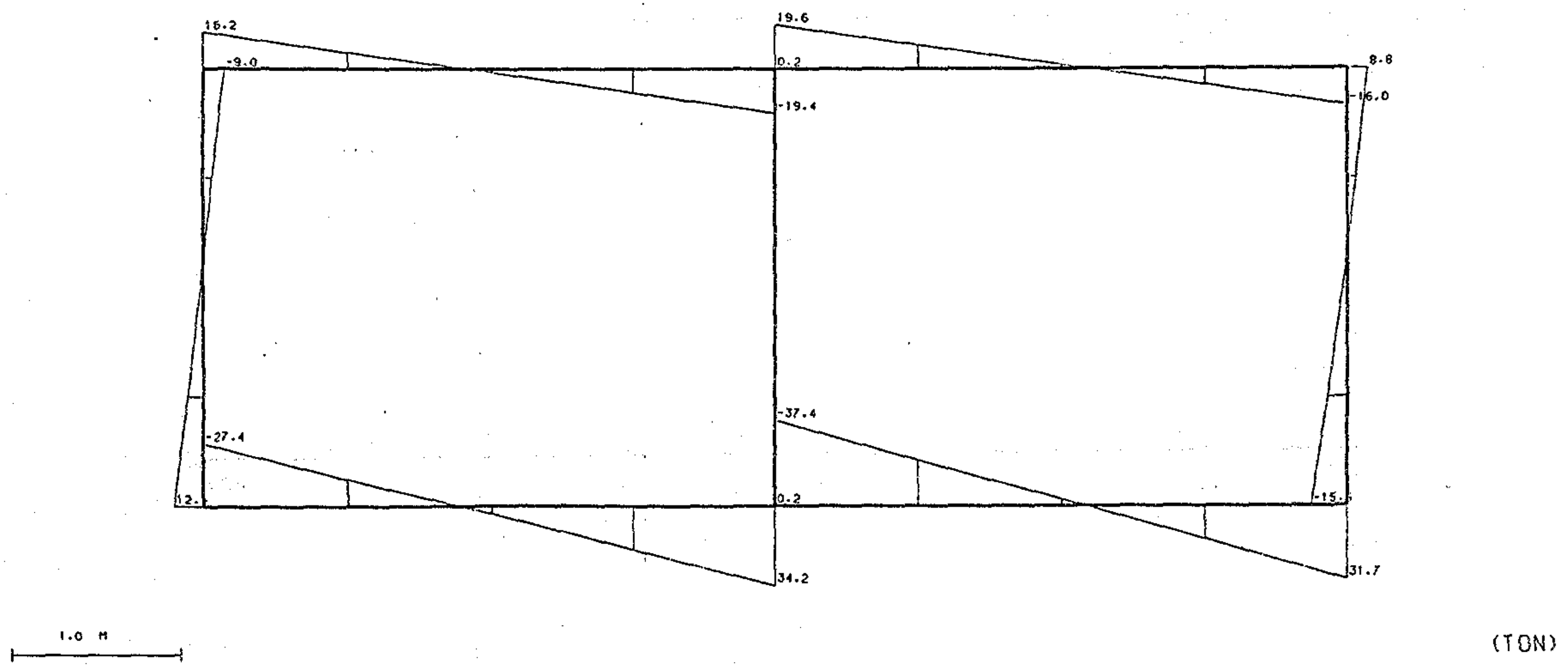


Fig 21. The shearing force diagram

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WWTPP DISCHARGE OUTLET CASE-3 (AT INSPECTION)

AXIAL FORCE

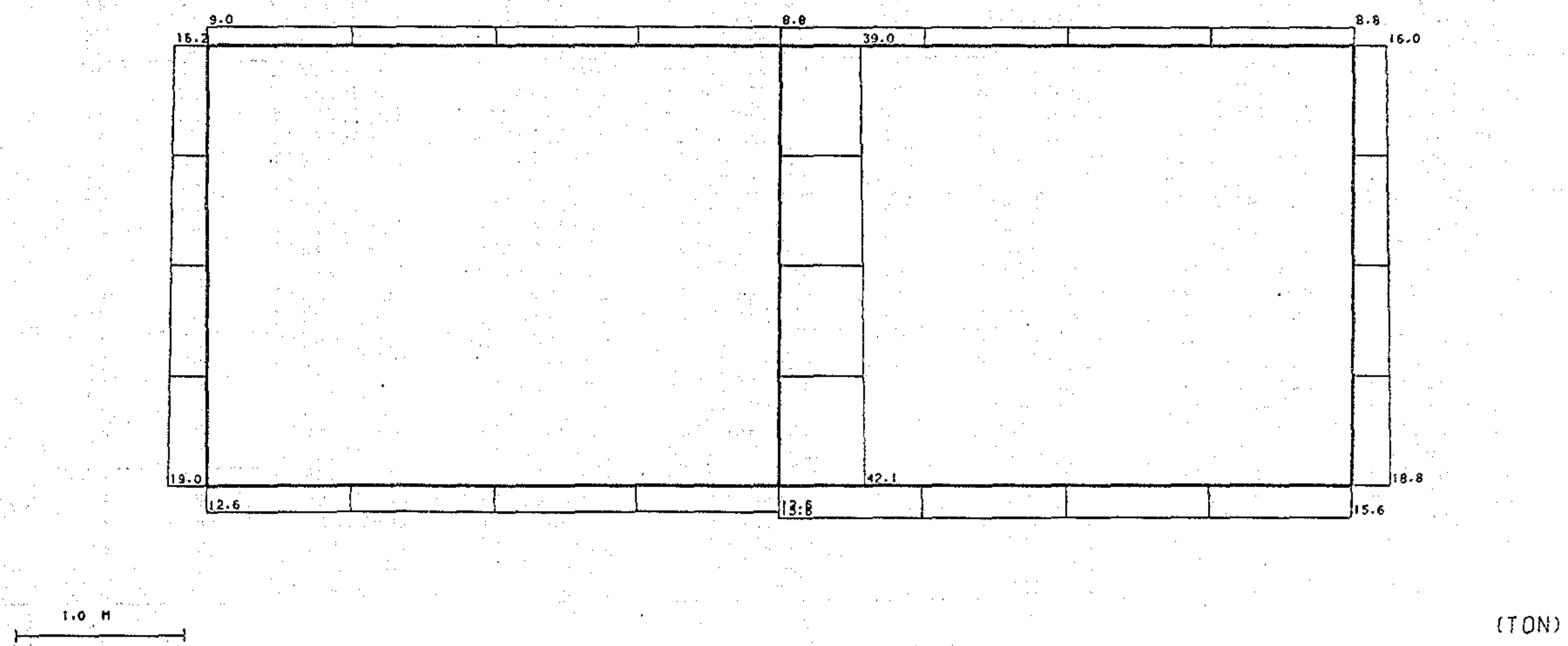

Fig 22. The axial force diagram

Table 6. The calculation results of the rectional forces

** ELEMENTAL FORCES **

FLEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	1.9014E+01	1.2630E+01	1.0167E+01	7	1.8312E+01	6.8201E+00	3.8636E+00
2	7	1.8312E+01	6.8201E+00	3.8606E+00	8	1.7610E+01	1.2870E+00	1.2438E+00
3	8	1.7610E+01	1.2870E+00	1.2408E+00	9	1.6908E+01	-3.9699E+00	2.1307E+00
4	9	1.6908E+01	-3.9699E+00	2.1277E+00	2	1.6206E+01	-8.9505E+00	6.3447E+00
5	2	8.9505E+00	1.6206E+01	6.3417E+00	10	8.9505E+00	7.2982E+00	-3.6476E+00
6	10	8.9505E+00	7.2982E+00	-3.6476E+00	11	8.9505E+00	-1.6098E+00	-6.0651E+00
7	11	8.9505E+00	-1.6098E+00	-6.0651E+00	12	8.9505E+00	-1.0518E+01	-9.1088E-01
8	12	8.9505E+00	-1.0518E+01	-9.1088E-01	3	8.9505E+00	-1.9426E+01	1.1815E+01
9	3	8.7937E+00	1.9595E+01	1.1943E+01	13	8.7937E+00	1.0687E+01	-9.2733E-01
10	13	8.7937E+00	1.0687E+01	-9.2733E-01	14	8.7937E+00	1.7793E+00	-6.2256E+00
11	14	8.7937E+00	1.7793E+00	-6.2256E+00	15	8.7937E+00	-7.1287E+00	-3.9521E+00
12	15	8.7937E+00	-7.1287E+00	-3.9521E+00	4	8.7937E+00	-1.6037E+01	5.8932E+00
13	4	1.6037E+01	8.7937E+00	5.8932E+00	16	1.6739E+01	3.6344E+00	1.8129E+00
14	16	1.6739E+01	3.6344E+00	1.8197E+00	17	1.7441E+01	-2.1588E+00	1.2990E+00
15	17	1.7441E+01	-2.1588E+00	1.3058E+00	18	1.8143E+01	-8.5856E+00	4.7566E+00
16	18	1.8143E+01	-8.5856E+00	4.7634E+00	5	1.8845E+01	-1.5646E+01	1.2598E+01
17	5	1.5646E+01	3.1678E+01	1.2604E+01	19	1.5646E+01	1.4406E+01	-6.9815E+00
18	19	1.5646E+01	1.4406E+01	-6.9915E+00	20	1.5646E+01	-2.8656E+00	-1.1886E+01
19	20	1.5646E+01	-2.8656E+00	-1.1886E+01	21	1.5646E+01	-2.0138E+01	-2.1100E+00
20	21	1.5646E+01	-2.0138E+01	-2.1100E+00	6	1.5646E+01	-3.7410E+01	2.2348E+01
21	6	1.2630E+01	3.4229E+01	2.1813E+01	22	1.2630E+01	1.8827E+01	-7.3645E-01
22	22	1.2630E+01	1.8827E+01	-7.3645E-01	23	1.2630E+01	3.4252E+00	-1.0194E+01
23	23	1.2630E+01	3.4252E+00	-1.0194E+01	24	1.2630E+01	-1.1977E+01	-6.5593E+00
24	24	1.2630E+01	-1.1977E+01	-6.5593E+00	1	1.2630E+01	-2.7379E+01	1.0167E+01
25	3	3.9021E+01	1.5674E-01	-1.2758E-01	25	3.9788E+01	1.5674E-01	-2.2946E-01
26	25	3.9788E+01	1.5674E-01	-2.2946E-01	26	4.0555E+01	1.5674E-01	-3.3134E-01
27	26	4.0555E+01	1.5674E-01	-3.3134E-01	27	4.1322E+01	1.5674E-01	-4.3322E-01
28	27	4.1322E+01	1.5674E-01	-4.3322E-01	6	4.2089E+01	1.5674E-01	-5.3510E-01

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1.5.4 The stress calculation

Before calculating the stress, the sectional force for the structural design is determined by selecting one case among three design cases from a view point of the safety design, and the stress calculation results are indicated in Table 7 and the arrangement of reinforcing bars is shown in Fig 23.
As the arrangements of reinforcing bars for other sections (Section 2 and 3) are applied to that of Section 1 and that of Discharge Tunnel, these are shown in Fig 24 and 25.

[Outlet]

Section 1

Table. 7-1 The Calculation Results of The Stress

Member	Point	The Sectional Force: M [kg·cm]	N [kg]	S [kg]	The Sectional Dimensions: B [cm]	H [cm]	d [cm]	d' [cm]	The Arrangement of Reinforcing Bars: D [mm]	Pitch [mm]	As [cm²] / A's [cm²]	The strees (kg/cm²): σ_b	σ_c	τ	Remarks
(1)	1	[Case-1] 1 060 000	[Case-1] 18 900	[Case-1] 12 900	100	50	40	10	D19 D16	150 150	19.1 13.3	1 058	45.6	3.2	As+As' ≧0.004 B·H = 20 cm²
	Center	[Case-1] 150 000	[Case-1] 17 500	0	100	40	30	10	D16 D19	150 150	13.3 19.1	96	9.1	0	
	2	[Case-1] 610 000	[Case-1] 16 100	[Case-1] 8 700	100	50	40	10	D19 D16	150 150	19.1 13.3	480	25.8	2.2	
(2)	2	[Case-1] 610 000	[Case-1] 8 700	[Case-1] 16 100	100	50	40	10	D19 D16	150 150	19.1 13.3	664	26.3	4.0	
	Center	[Case-1] -620 000	[Case-1] 8 700	[Case-1] 0	100	40	30	10	D16 D19	150 150	13.3 19.1	1 367	49.5	0	
	3	[Case-1] 1 190 000	[Case-1] 8 700	[Case-1] 19 500	100	50	40	10	D22 D16	150 150	25.8 13.3	1 144	46.4	4.9	
(3)	3	[Case-1] 1 190 000	[Case-1] 8 700	[Case-1] 19 500	100	50	40	10	D22 D16	150 150	25.8 13.3	1 144	46.4	4.9	
	Center	[Case-1] -620 000	[Case-1] 8 700	0	100	40	30	10	D16 D19	150 150	13.3 19.1	1 367	49.5	0	
	4	[Case-1] 610 000	[Case-1] 8 700	[Case-1] 16 100	100	50	40	10	D19 D16	150 150	19.1 13.3	664	26.3	4.0	
(4)	4	[Case-1] 610 000	[Case-1] 16 100	[Case-1] 8 700	100	50	40	10	D19 D16	150 150	19.1 13.3	480	25.8	2.2	
	Center	[Case-1] 150 000	[Case-1] 17 500	0	100	40	30	10	D16 D19	150 150	13.3 19.1	96	9.1	0	
	5	[Case-1] 1060 000	[Case-1] 18 900	[Case-1] 12 900	100	50	40	10	D19 D16	150 150	19.1 13.3	1 058	45.6	3.2	

Where
- Mb : Bending moment
- N : Axial force
- S : Shearing force
- B : The Width
- H : The Height
- d : The effective height
- d' : The covering of compression bar
- D : Diameter of bars
- As : The area of tension bars
- A's : The area of compression bars
- σ_b : The bending stress
- σ_c' : The compressive stress
- τ : The shearing stress

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[Outlet]

Section 1

Table. 7-2 The Calculation Results of The Stress

Member	Point	The Sectional Force			The Sectional Dimensions				The Arrangement of Reinforcing Bars			The strees (kg/cm²)			Remarks
		M [kg·cm]	N [kg]	S [kg]	B [cm]	H [cm]	d [cm]	d' [cm]	D [mm]	Pitch [mm]	As [cm²] A's [cm²]	σ_b	σ_c	τ	
(5)	5	1 060 000	12 900	27 800	100	50	40	10	D22 D22	150 150	25.8 25.8	927	39.8	7.0	As+As' ≧0.004B·H =20cm²
	Center	-1 040 000	12 900	0	100	40	30	10	D22 D22	150 150	25.8 25.8	1307	64.5	0	
	6	2 090 000	12 900	33 800	100	50	40	10	D29 D22	150 150	42.9 25.8	1286	65.9	8.5	
(6)	6	2090000	12 900	33 800	100	50	40	10	D29 D22	150 150	42.9 25.8	1286	65.9	8.5	
	Center	-1 040 000	12 900	0	100	40	30	10	D22 D22	150 150	25.8 25.8	1307	64.5	0	
	7	1060 000	12 900	27 800	100	50	40	10	D22 D22	150 150	25.8 25.8	927	39.8	7.0	
(7)	7	0	39 000	200	100	60	40	10	D19 D19	150 150	19.1 19.1	90	5.7	0.1	σsa=2000kg/cm² σca=87.5kg/cm² τa=10.6kg/cm²
	Center	0	40 600	0	100	40	30	10	D19 D19	150 150	19.1 19.1	133	8.9	0	DITTO
	8	-50 000	42 100	200	100	60	40	10	D19 D19	150 150	19.1 19.1	101	6.9	0.1	DITTO

Where
Mb : Bending moment
N : Axial force
S : Shearing force

B : The Width
H : The Height
d : The effective height
d' : The covering of compression bar

D : Diameter of bars
As : The area of tension bars
A's : The area of compression bars

σ_b : The bending stress
σ_c' : The compressive stress
τ : The shearing stress

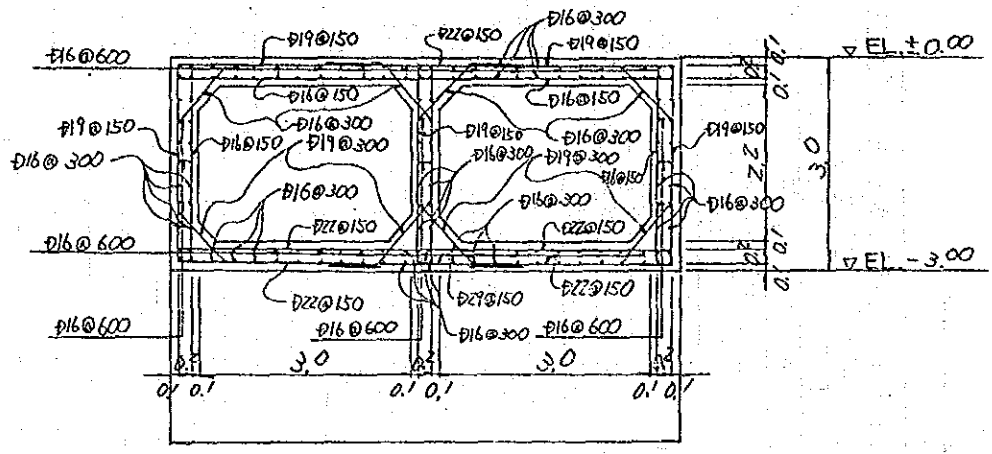

Fig 23. The arrangement of reinforcing bars at Section 1

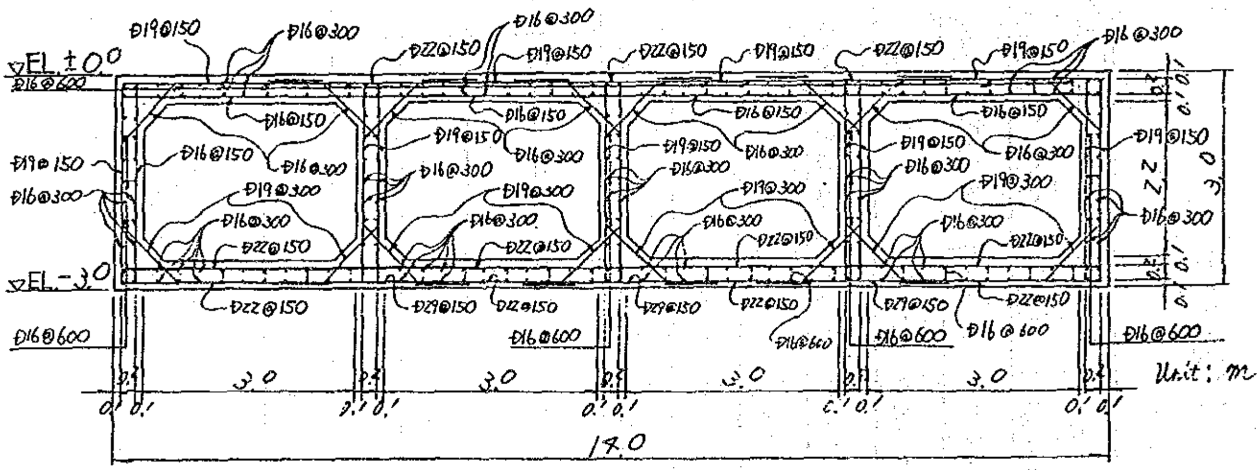

Fig 24. The arrangement of reinforcing bars at Section 2

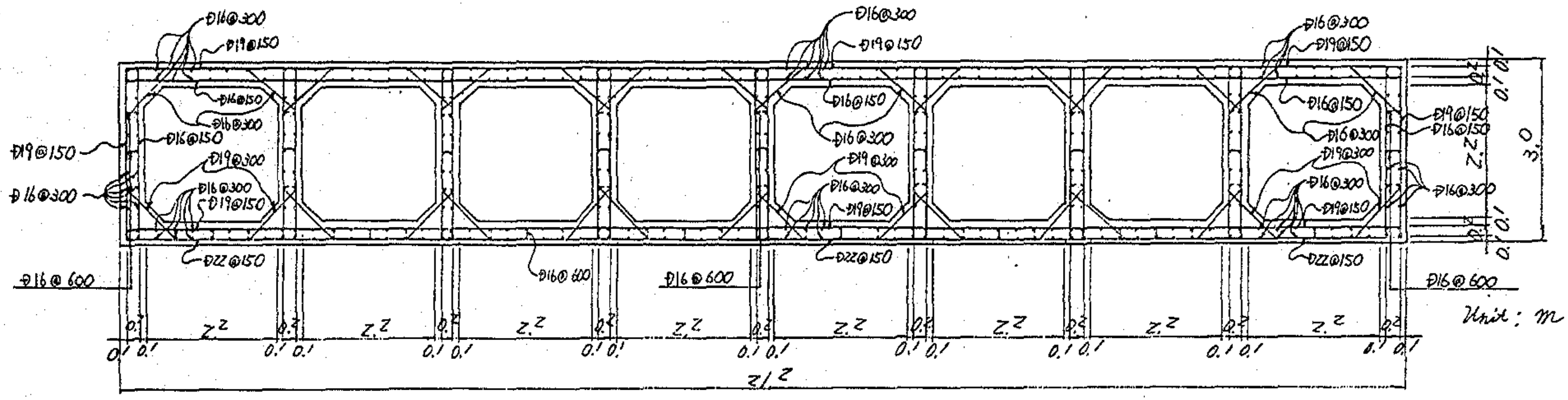

Fig 25. The arrangement of reinforcing bars at Section 3

FORM 04

CV-8 STRUCTURAL CALCULATION OF RETAINING WALL

Contents of this calculation note is shown as below.

Contents

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1. Outline of Revetment

The side view of Revetment is shown in Fig 1 and Plan of Revetment is shown in Fig 2.

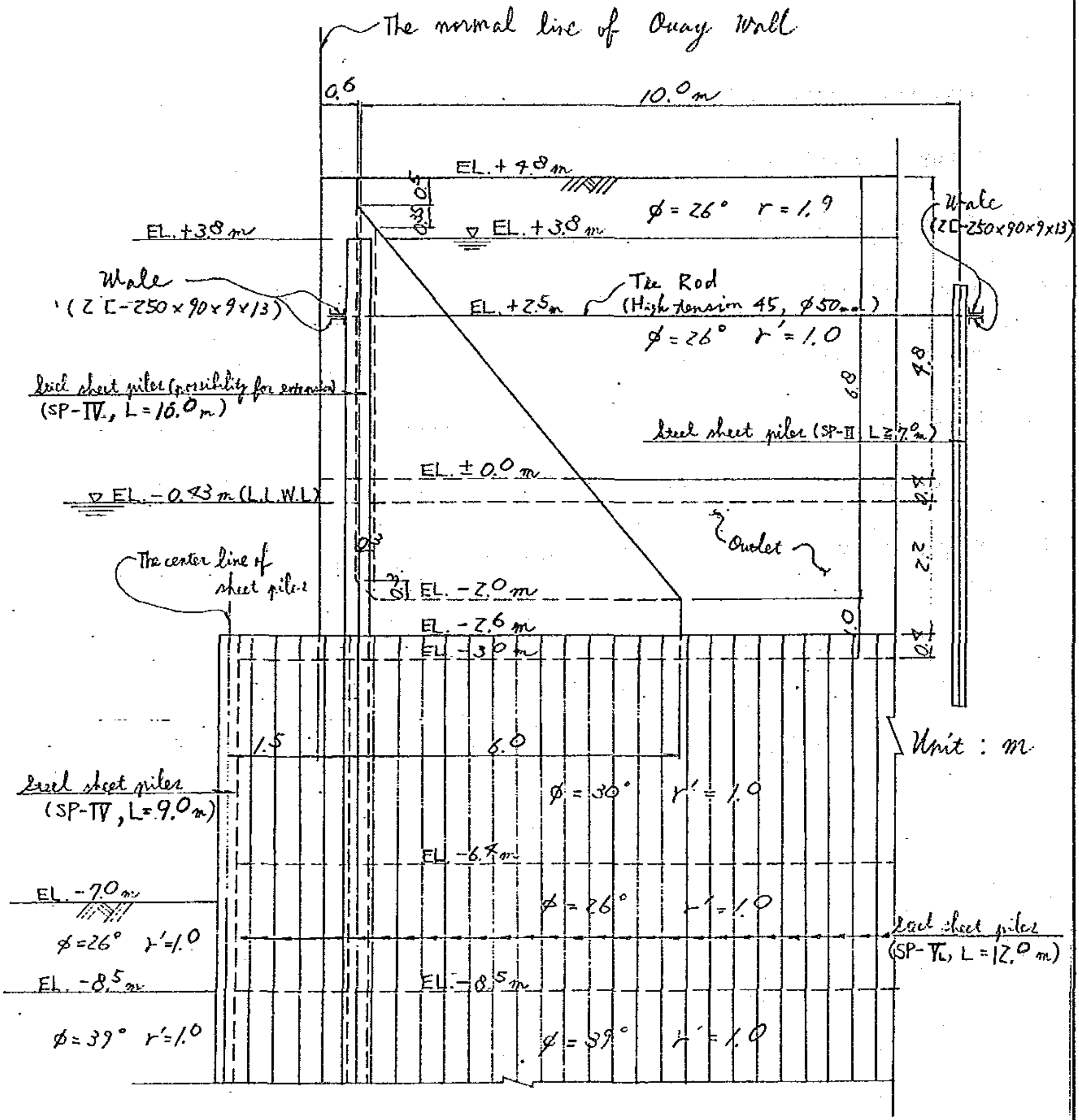

Fig 1. The side view of Revetment

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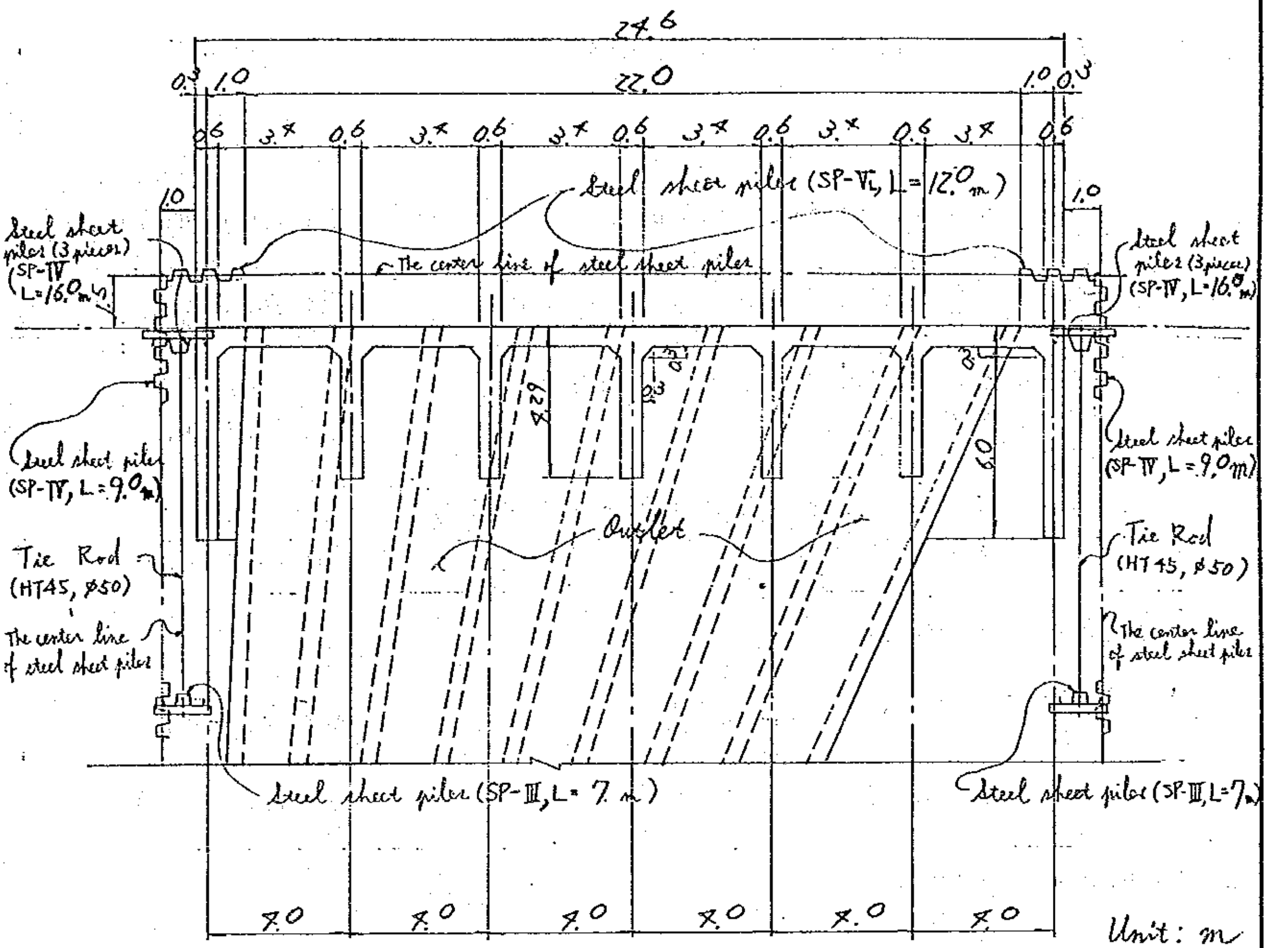

Fig 2. Plan of Revetment

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2. Study of Vertical Wall

1) The load calculation (per 1m unit length)

i) The earth pressure

$P_{e0} = 0.5 \times 1.0\ t/m^2 = 0.5\ t/m^2$ (at the surface)

$P_{e1} = 0.5 \times (1.0 + 1.9 + 1.0) = 1.45\ t/m^2$ (at EL. 3.8 m)

$P_{e2} = 0.5 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 3.8)$
$= 3.35\ t/m^2$ (at EL. ± 0 m)

$P_{e3} = 0.5 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 5.8)$
$= 4.35\ t/m^2$ (at EL. −2.0 m)

ii) The water pressure

$P_{w1} = 1.0 \times 3.8 = 3.8\ t/m^2$ (at EL. ± 0m)

$P_{w2} = 1.0 \times 4.23 = 4.23\ t/m^2$ (at EL. −0.43m)

2) The load diagram

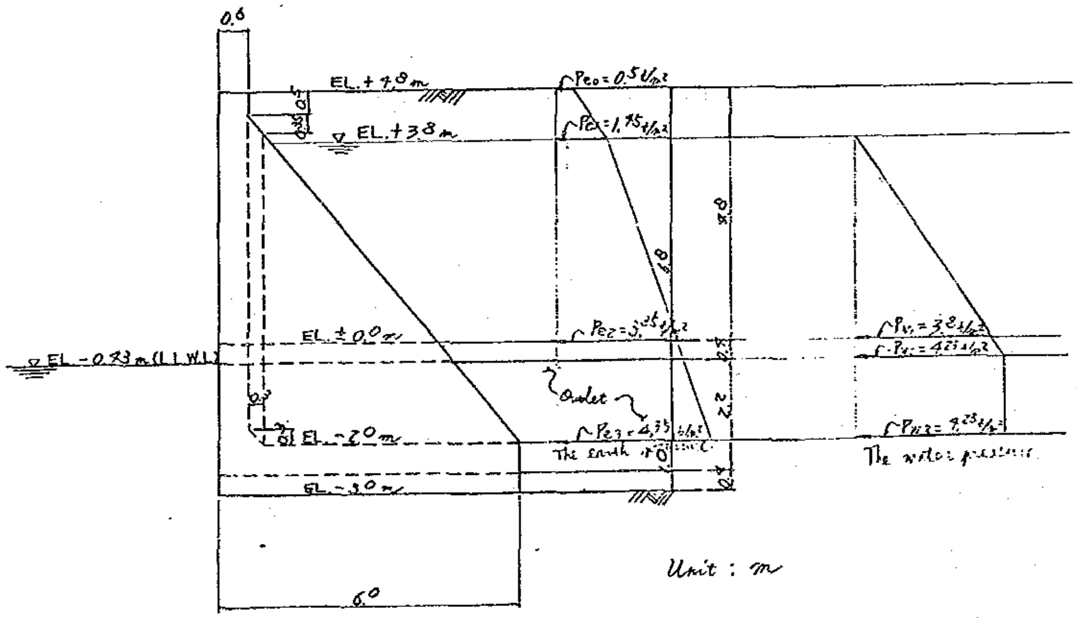

Fig 3. The load diagram

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3) The Structural calculation

i) at the middle span

The structure of vertical wall is considered for the continuous beam at the middle span, so the calculation is as follows.

$$M_1 = \frac{\omega \cdot l^2}{10} = \frac{(3.35 + 3.8) \times 4^2}{10} = 11.5 \text{ t·m}$$

$$S_1 = \frac{\omega \cdot l}{2} = \frac{(3.35 + 3.8) \times 4}{2} = 14.3 \text{ t}$$

ii) at the end span

The structure of vertical wall is considered for a fixed beam at the end span, so the calculation is as follows.

$$M_2 = \frac{\omega \cdot l^2}{12} + M_1 = \frac{(4.35 + 4.23) \times 1.2^2}{12} + 11.5 = 12.5 \text{ t·m}$$

$$S_2 = \frac{\omega \cdot l}{2} + S_1 = \frac{(4.35 + 4.23) \times 1.2}{2} + 14.3 = 19.5 \text{ t}$$

iii) the stress calculation

The stress calculation results are indicated in Table 1.

Revetment

Table. / The Calculation Results of The Stress

Member	Point	The Sectional Force M [kg·cm]	N [kg]	S [kg]	The Sectional Dimensions B [cm]	H [cm]	d [cm]	d' [cm]	The Arrangement of Reinforcing Bars D [mm]	Pitch [mm]	As [cm²] / A's [cm²]	The strees (kg/cm²) σ_b	σ_c	τ	Remarks
Vertical Wall		1250 000	0	19 500	100	60	50	10	19 / 19	150 / 150	19.1 / 19.1	1458	36.6	3.9	
Base Slab		1630 000	0	27 400	100	100	90	10	16 / 16	150 / 150	13.3 / 13.3	1456	21.6	2.7	

Where
Mb : Bending moment
N : Axial force
S : Shearing force
B : The Width
H : The Height
d : The effective height
d' : The covering of compression bar
D : Diameter of bars
As : The area of tension bars
A's : The area of compression bars
σ_b : The bending stress
σ_c' : The compressive stress
τ : The shearing stress

3. Study of Base Slab (at the end span)

1) The load calculation (per 1m unit length)

i) The soil weight W_s

$$W_s = 1.0 + 1.9 \times 1.0 + 1.0 \times 6.8 = 9.7\ t/m^2$$

ii) The water weight W_w

$$W_w = 1.0 \times 6.8 = 6.8\ t/m^2$$

iii) The concrete weight

$$W_c = 1.0 \times 2.45 = 2.5\ t/m^2$$

iv) Up lift

$$P_u = 1.0 \times 6.8 = 6.8\ t/m^2$$

Accordingly working load W to the base slab is calculated as follows.

$$\begin{aligned} W &= W_s + W_w + W_c - P_u \\ &= 9.7 + 6.8 + 2.5 - 6.8 \\ &= 12.2\ t/m^2 \end{aligned}$$

2) The structural calculation

The Structure of base slab is considered for a fixed beam, so the calculation is as follows.

$$M_3 = \frac{W \cdot lm^2}{12} = \frac{12.2 \times 4.0^2}{12}$$ where lm : the maximum span length lm = 4.0 m

$$= 16.3\ t{\cdot}m$$

$$S_3 = \frac{W \cdot lm}{2} = \frac{12.2 \times 4.0}{2} = 24.4\ t$$

3) The stress calculation

The stress calculation results are indicated in Table 1.

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4. Study of Buttress

The design structure of buttress is considered for T-formed beam that have vertical wall as a flange and buttress as a web, so the design calculation is as follows.

1) The load calculation

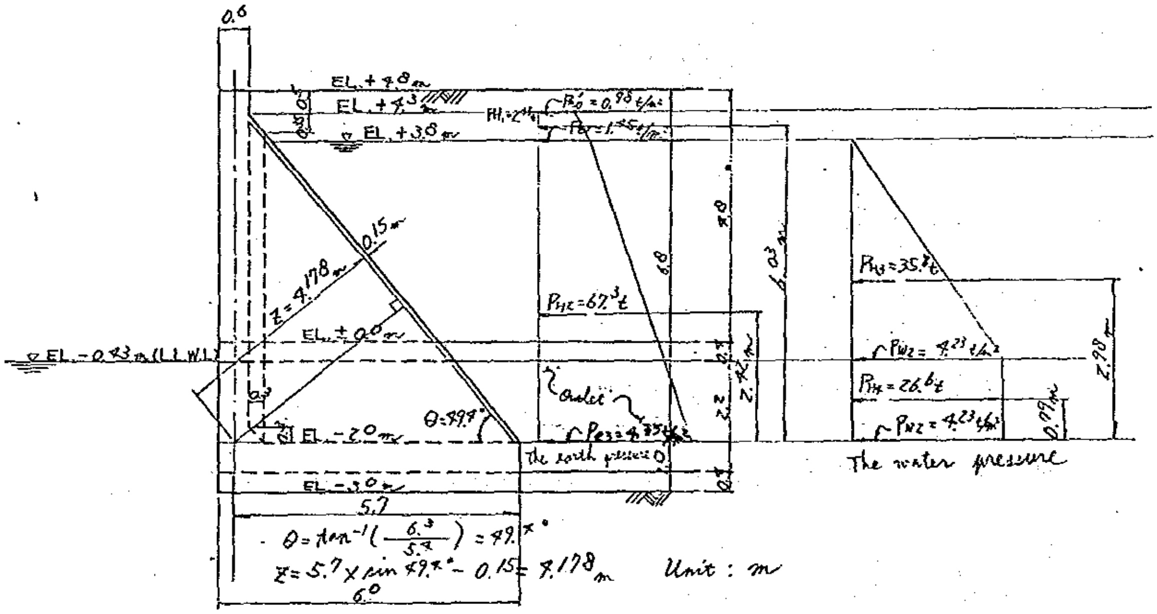

Fig 4. The load diagram

The earth pressure P_{Hi} working to the interval of buttressed wall and the working points are as follows.

$$P_{H1} = \frac{1}{2} \times (0.98 + 1.45) \times 0.5 \times 4.0 = 2.43 \text{ t}$$

$$H_1 = \frac{1}{3} \times \frac{2 \times 0.98 + 1.45}{0.98 + 1.45} \times 0.5 + 5.8 = 6.03 \text{ m}$$

$$P_{H2} = \frac{1}{2} \times (1.45 + 4.35) \times 5.8 \times 4.0 = 67.3 \text{ t}$$

$$H_2 = \frac{1}{3} \times \frac{2 \times 1.45 + 4.35}{1.45 + 4.35} \times 5.8 = 2.42 \text{ m}$$

$$P_{H3} = \frac{1}{2} \times 4.23 \times 4.23 \times 4.0 = 35.8 \text{ t}$$

$$H_3 = \frac{1}{3} \times 4.23 + 1.57 = 2.98 \text{ m}$$

$$P_{H}4 = 4.23 \times 1.57 \times 4.0 = 26.6 \text{ t}$$

$$H_4 = \frac{1}{2} \times 1.57 = 0.79 \text{ m}$$

2) The structural calculation

The bending moment M_b and the shearing force S are as follows.

$$M_b = P_{H1} \cdot H_1 + PH_2 \cdot H_2 + PH_3 \cdot H_3 + PH_4 \cdot H_4$$

$$= 2.43 \times 6.03 + 67.3 \times 2.42 + 35.8 \times 2.98 + 26.6 \times 0.79$$

$$= 305.2 \text{ t} \cdot \text{m}$$

$$S = P_{H1} + PH_2 + PH_3 + PH_4$$

$$= 132.1 \text{ t}$$

Accordingly the required sectional area A_{so} for the tension bars is calculated as follows.

$$A_{so} = \frac{M}{\delta_{sa} \cdot Z} = \frac{30520000}{1600 \times 417.8} = 45.66 \text{ cm}^2$$

Now using the reinforcing bars D25($A = 5.067 \text{ cm}^2$) 10 pieces,

$$A_s = 5.067 \times 10 = 50.67 \text{ cm}^2 > A_{so} = 45.66 \text{ cm}^2$$

1402

The effective width b_e of flange (vertical wall) at the end span is calculated as follows.

$$b_e = b_o + \left(b_s + \frac{l}{8}\right)$$

$$= 0.6 + 0.3 + \frac{2.4}{8}$$

$$= 1.2 \text{ m} < 4.0 \text{ m}$$

b_o : the width of web
$b_o = 0.6$ m

b_s : the width of flange
$b_s = 0.3$ m

l : the span length of reflection
$l = 0.6 \times 4 = 2.4$ m

Therefore $b = be = 1.2 \text{ m} = 120 \text{ cm}$

Calculating the distance X to the neutral axis,

$$X = -\frac{t\,(b - b_o) + n \cdot A_s}{b_o} + \sqrt{\frac{t\,(b - b_o) + n \cdot A_s}{b_o} + \frac{t^2\,(b - b_o) + 2n \cdot A_s \cdot d}{b_o}}$$

$$= -\frac{60 \times (120 - 60) + 15 \times 50.67}{60}$$

$$+ \frac{60 \times (120 - 60) + 15 \times 50.67}{60} + \sqrt{\frac{60^2 \times (120 - 60) + 2 \times 15 \times 50.67 \times 457.3}{60}}$$

$$= -72.67 + 143.06$$

$$= 70.39 \text{ cm} > 60 \text{ cm}$$

Outline of T-formed beam (at the end span) in shows in Fig 5.

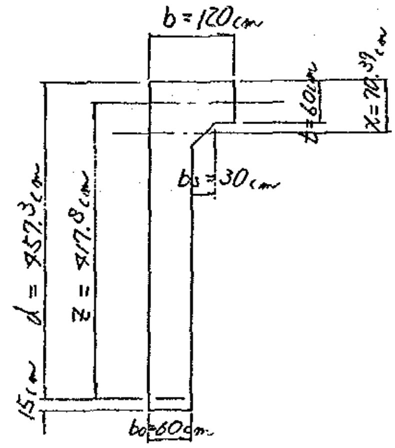

Fig 5. Outline of T-formed bar

The inertia moment I related with the neutral axis

$$I = \frac{1}{3}\ \{b \cdot x^3 - (b - bo)\ (x - t)^3\} + n \cdot As \cdot (d - x)^2$$

$$= \frac{1}{3} \times \{120 \times 70.39^3 - (120 - 60) \times (70.39 - 60)^3\} + 15 \times 50.67 \times (457.3 - 70.39)^2$$

$$= 12.771 \times 10^7\ cm^4$$

Accordingly

$$\delta_c = \frac{M}{I} \cdot x = \frac{305.2 \times 10^5}{12.771 \times 10^7} \times 70.39$$

$$= 16.8\ kg/cm^2 \underset{ok}{<} \delta ca = 70\ kg/cm^2$$

$$\delta_c = \frac{n \cdot M}{I} \cdot (d\text{-}x) = \frac{15 \times 305.2 \times 10^5 \times (457.3 - 70.39)}{12.771 \times 10^7}$$

$$= 1387\ kg/cm^2 \underset{ok}{<} \delta_{sa} = 1600\ kg/cm^2$$

$$\iota = \frac{S}{b_o \cdot Z} = \frac{132100}{60 \times 417.8} = 5.3\ kg/cm^2 \underset{ok}{<} \iota a = 8.5\ kg/cm^2$$

And the required sectional area of the connecting bars at buttress is calculated as follows.
(at the connecting portion with vertical wall)

$$A_{s1} = \frac{S_1}{\delta_{sa}} = \frac{19500}{1600} = 12.2\ cm^2$$

$(D16 @ 150 \rightarrow A_s'{}_1 = 13.3\ cm^2)$

(at the connecting portion with base slab)

$$A_{s2} = \frac{S_2}{\delta_{sa}} = \frac{24400}{1600} = 15.3\ cm^2$$

$(D19 @ 150 \rightarrow A_s'{}_2 = 19.1\ cm^2)$

1404

5. Study of the Front Sheet Piles

5.1 The Calculation of The External Forces

1) The Coefficient of the earth pressure

a) The Coefficient of the active earth pressure

$K_{A1} = \tan^2(45° - \frac{30°}{2}) = 0.333$ [EL.-6.4 m ~ EL.-3.0 m]

$K_{A2} = \tan^2(45° - \frac{26°}{2}) = 0.390$ [EL.-8.5 m ~ EL.-6.4 m]

$K_{A3} = \tan^2(45° - \frac{39°}{2}) = 0.228$ [below EL.-8.5 m]

b) The coefficient of the passive earth pressure

$K_{P1} = \tan^2(45° + \frac{26°}{2}) = 2.561$ [EL.-8.5 m ~ EL.-6.4 m]

$K_{P2} = \tan^2(45° + \frac{39°}{2}) = 4.395$ [below EL.-8.5 m]

1405

2) The load calculation

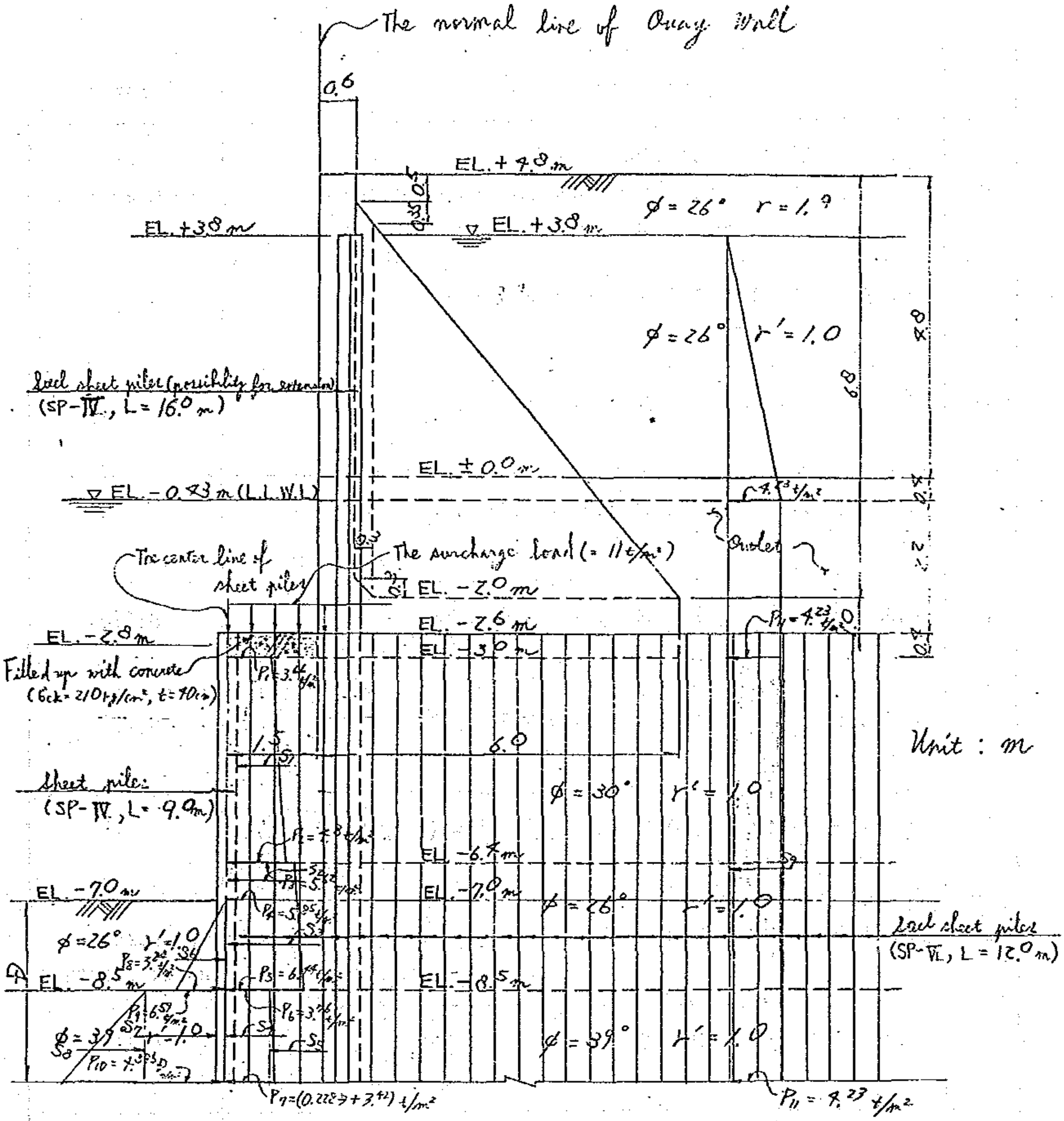

Fig 6. The load diagram

1406

a) The horizontal forces

Table 1. The summary of horizontal forces

	The Calculation	Remarks
P_1	$0.333 \times \underline{11.0} = 3.66$ t/m² * This surcharge load is taken for adding the ground reaction of Outlet and concrete weight	The active earth pressure
P_2	$0.333 \times (11.0 + 1.0 \times 3.4) = 4.8$ t/m²	DITTO
P_3	$0.390 \times (11.0 + 1.0 \times 3.4) = 5.62$ t/m²	DITTO
P_4	$0.390 \times (11.0 + 1.0 \times 4.0) = 5.85$ t/m²	DITTO
P_5	$0.390 \times (11.0 + 1.0 \times 5.5) = 6.44$ t/m²	DITTO
P_6	$0.228 \times (11.0 + 1.0 \times 5.5) = 3.76$ t/m²	DITTO
P_7	$0.228 \times \{11.0 + 1.0 \times (4.0 + D)\} = (0.228D + 3.42)$ t/m²	DITTO
P_8	$2.561 \times 1.5 = 3.84$ t/m²	The passive earth pressure
P_9	$4.395 \times 1.5 = 6.59$ t/m²	DITTO
P_{10}	$4.395 \times D = 4.395D$ t/m²	DITTO
P_{11}	$1.0 \times 4.23 = 4.23$ t/m²	The water pressure

1471

3) The external moments

Table 2. The Summary of the external moments

No	The Forces S_i (t/m²)	Arms A_i (m)	Moments M_i (t.m/m)
1	$\frac{3.66+4.8}{2}\times3.4=14.38$	$\frac{2\times4.8+3.66}{3.66+4.8}\times\frac{3.4}{3}=1.777$	25.54
2	$\frac{5.62+5.85}{2}\times0.6=3.44$	$\frac{2\times5.85+5.62}{5.62+5.85}\times\frac{0.6}{3}+3.4=3.702$	12.73
3	$\frac{5.85+6.44}{2}\times1.5=9.22$	$\frac{2\times6.44+5.85}{5.85+6.44}\times\frac{1.5}{3}+4.0=4.762$	43.91
4	$3.76\times(D-1.5)$ $=3.76D-5.64$	$0.5\times(D-1.5)+5.5=0.5D+4.75$	$1.88D^2+15.04D$ -26.79
5	$0.5\times(0.228D-0.34)$ $\times(D-1.5)=0.114D^2$ $-0.342D+0.257$	$0.667\times(D-1.5)+5.5$ $=0.667D+4.5$	$0.076D^3+0.285D^2$ $-1.368D+1.157$
6	$0.5\times3.84\times1.5=2.88$	$0.667\times1.5+4.0=5.0$	14.4
7	$6.59\times(D-1.5)$ $=6.59D-9.885$	$0.5\times(D-1.5)+5.5=0.5D+4.75$	$3.295D^2-26.36D$ -46.954
8	$0.5\times(4.395D-6.59)$ $\times(D-1.5)=2.198D^2$ $-6.593D+4.944$	$0.667\times(D-1.5)+5.5$ $=0.667D+4.5$	$1.466D^3+5.493D^2$ $-26.371D+22.248$
9	$4.23D\times(D+4.0)$ $=4.23D+16.92$	$0.5\times(D+4.0)=0.5D+2.0$	$2.115D^2+16.92D$ $+33.84$

Accordingly the turning moment M_t and the resistant moment M_r are calculated as follows.

i) The turning moment

$$
\begin{aligned}
M_t &= M_1 + M_2 + M_3 + M_4 + M_5 + M_9 \\
&= 25.54 + 12.73 + 43.91 + 1.88D^2 + 15.04D - 26.79 \\
&\quad + 0.076D^3 + 0.285D^2 - 1.368D + 1.157 + 2.115D^2 \\
&\quad + 16.92D + 33.84 \\
&= 0.076D^3 + 4.28D^2 + 30.592D + 90.387
\end{aligned}
$$

ii) The resistant moment

$$
\begin{aligned}
M_r &= M_6 + M_7 + M_8 \\
&= 14.4 + 3.295D^2 - 26.36D - 46.954 + 1.466D^3 + 5.493D^2 \\
&\quad - 26.371D + 22.248 \\
&= 1.466D^3 + 8.788D^2 - 52.731D - 10.306
\end{aligned}
$$

1408

5.2 The Calculation of The Enbedded Length

The embedded length is calculated by making the balance of the turning moment and the resistant moment. This calculation is as follows.

$M_r = F_s \cdot M_t$ where F_s : the factor of safety = 1.5

$$1.466D^3 + 8.788D^2 - 52.731D - 10.306 = 1.5\times(0.076D^3 + 4.28D^2 + 30.592D + 90.387)$$
$$1.352D^3 + 2.368D^2 - 6.843D - 145.887 = 0$$

Accordingly solving the above equation

$D = 4.6$ m

Therefore all length of a front sheet pile is calculated as follows.

$L \geqq 4.4 + 4.6 = 9.0$ m

According to the above calculation, all length of a front sheet pile shall be determined L = 9.0 m in consideration of making use of a temporary sheet pile of the cofferdam.

FORM 04

1409

5.3 Design of The Front Sheet Piles

1) The Reactions at the supports

The Reactions at the supports are calculated for a simple beam that has the upper end as reinforced concrete placed and the lower end as the sea bottom, and the formal equation is as follows.

(at the lower end)

$$R_l = \frac{M_o}{l_o}$$

where M_o : the total external moment [t·m/m]
l_o : the span length = 4.0 [m]
S_o : the total horizontal force

(at the upper end)

$$R_u = S_o - R_l$$

Accordingly this calculation is as follows.

$$M_o = 25.54 + 12.73 + 33.84 = 72.1 \text{ t·m}$$
$$S_o = 14.38 + 3.44 + 16.92 = 34.8 \text{ t}$$
$$R_l = \frac{72.1}{4.0} = 18.0 \text{ t/m}$$
$$R_u = 34.8 - 18.0 = 16.8 \text{ t/m}$$

2) The maximum bending moment

The formal equation for the bending moment is as follows.

$$M_x = R_l \cdot X - \frac{}{2} P_l X^2 + \frac{}{6} (KX) \cdot X^2 \quad [\text{t·m/m}]$$
$$S_x = R_l - P_l X + \frac{1}{2} KX^2 \quad [\text{t/m}]$$

Where $K = \dfrac{P_l - P_u}{l_o}$ $\quad P_l$: $P_l = P_4 + P_{11} = 5.85 + 4.23 = 10.08$ t/m²
$\quad P_u$: $P_u = P_1 + P_{11} = 3.66 + 4.23 = 7.89$ t/m²

Provided that $S_x = 0$

$$X = \frac{P_l - \sqrt{P_l^2 - 2K \cdot R_l}}{K} \quad [\text{m}]$$

Accordingly the real calculation is as follows.

$$K = \frac{10.08 - 7.89}{4.0} = 0.5475$$

$$X = \frac{10.08 - \sqrt{10.08^2 - 2 \times 0.5475 \times 18.0}}{0.5475}$$
$$= 1.882 \text{ m}$$

FORM 04

14/0

Therefore M_{max} is calculated as follows.

$$M_{max} = 18.0 \times 1.882 - \frac{1}{2} \times 10.08 \times 1.882^2 + \frac{1}{6} \times (0.5475 \times 1.882) \times 1.882^2$$

$$= 16.7 \text{ t·m/m}$$

Fig 7. The load diagram

3) The Determination for the section of sheet piles

Now using the sheet pile type SP-IV(the section modulus: Z = 2270 cm^3/m, the material: SY-30), the stress calculations are as follows.

$$\sigma_b = \frac{M_{max}}{Z} = \frac{1\ 670\ 000}{2270} = 736 \text{ kg/cm}^2 < \sigma_{ba} = 1800 \text{ kg/cm}^2$$

O.K

4) The required sectional area of the connecting bars
(between the front sheet piles and the base slab of Outlet)

The required sectional area of the connecting bars A_s are calculated as follows.

$$A_s = \frac{S}{\sigma_{sa}} = \frac{18\ 000}{945} = 19.05\ \text{cm}^2$$

[Double alignment of D19 @ 300 → 19.1 cm²]

Where σ_{sa}: the allowable shearing stress for the field fillet weld

$$\sigma_{sa} = 0.9 \times 1\ 050 = 945\ \text{kg/cm}^2$$

The arrangement of reinforcing bars is shown in Fig 8.

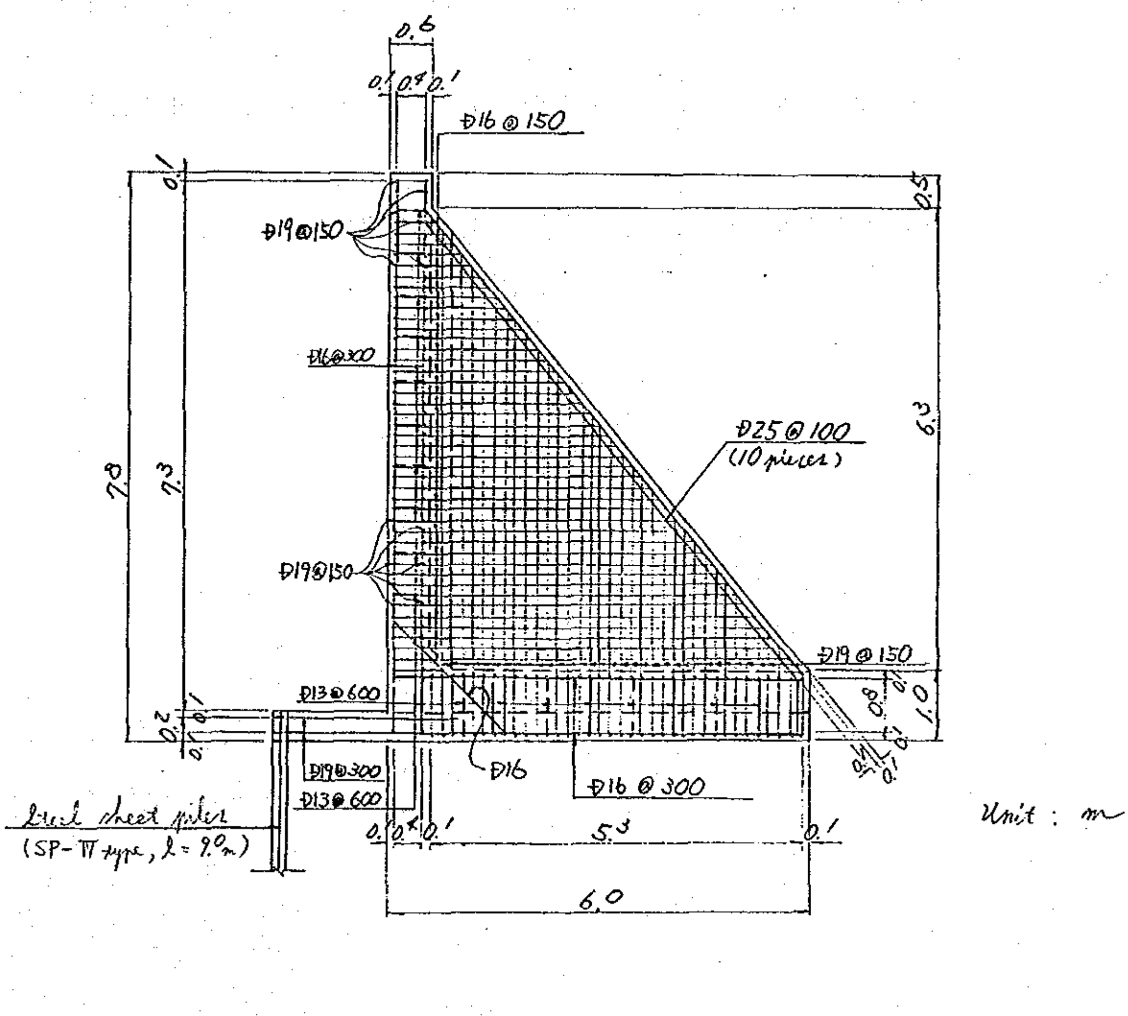

Fig 8. The general arrangement of reinforcing bars

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6. Study of Seawall

The design structure of seawall is the sheet pile counterforted by anchoring sheet piles.

6.1 The Calculation of The External Force

1) The Coefficient of the earth pressure

a) The Coefficient of the active earth pressure

$$K_{A1}=\tan^2\left(45^\circ - \frac{26^\circ}{2}\right) = 0.390 \quad [\text{ EL.- 8.5 m} \sim \text{EL.+ 3.8 m }]$$

$$K_{A2}=\tan^2\left(45^\circ - \frac{39^\circ}{2}\right) = 0.228 \quad [\text{ below EL.- 8.5 m }]$$

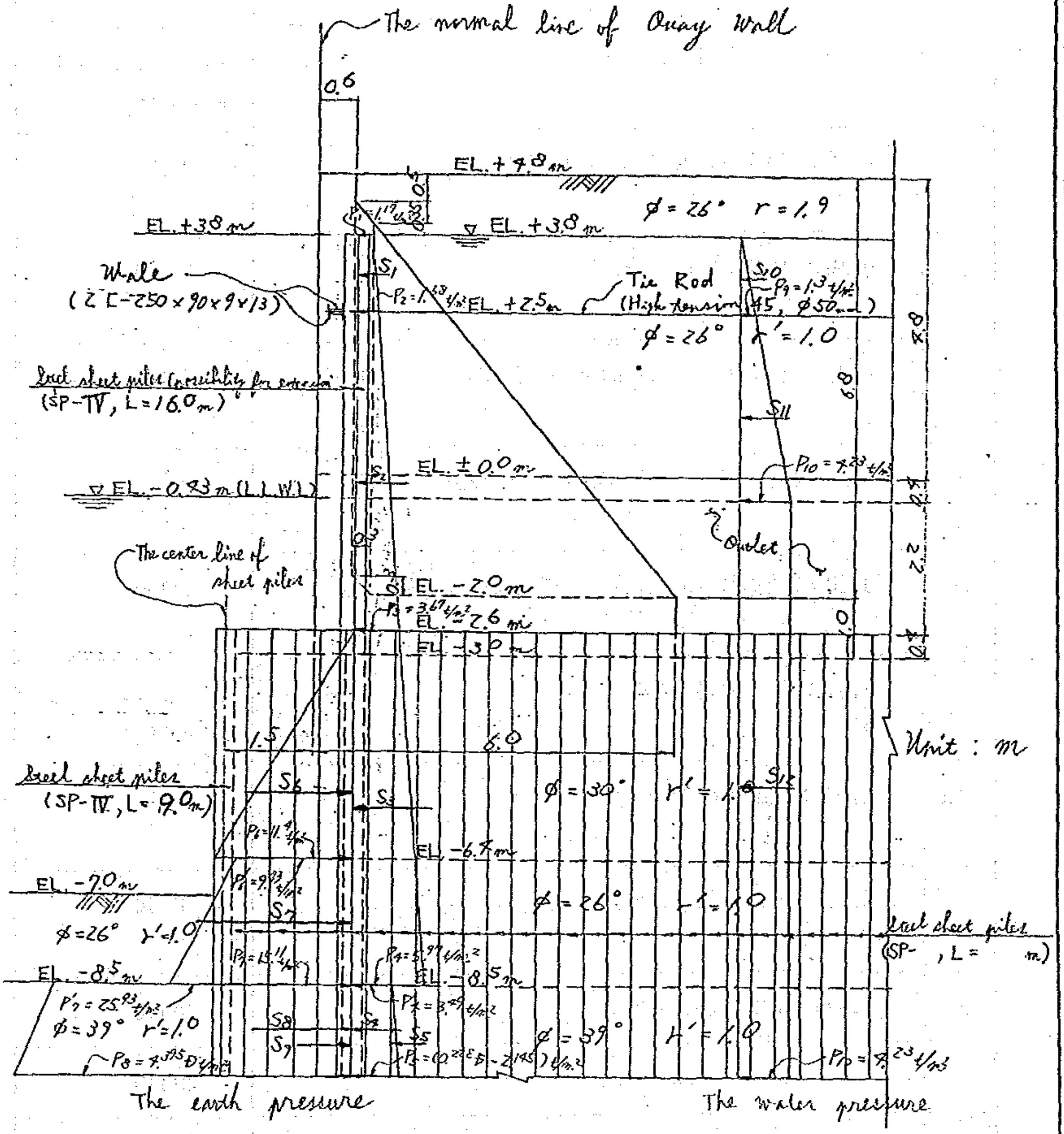
b) The Coefficient of the passive earth pressure

$$K_{P1}=\tan^2\left(45^\circ + \frac{30^\circ}{2}\right) = 3.0 \quad [\text{ EL.- 6.4 m} \sim \text{EL.- 2.6 m }]$$

$$K_{P2}=\tan^2\left(45^\circ + \frac{26^\circ}{2}\right) = 2.561 \quad [\text{ EL.- 8.5 m} \sim \text{EL.- 6.4 m }]$$

$$K_{P3}=\tan^2\left(45^\circ + \frac{39^\circ}{2}\right) = 4.395 \quad [\text{ below EL.- 8.5 m }]$$

2) The load calculation



<u>Fig 9. The load diagram</u>

1414

a) The horizontal forces

Table 3. The Summary of the horizontal forces

No.	The Calculations	Remarks
P_1	$0.39 \times 3.0 = 1.17$ t/m²	The active earth pressure
P_2	$0.39\times(3.0+1.0\times1.3) = 1.68$ t/m²	DITTO
P_3	$0.39\times(3.0+6.4) = 3.67$ t/m²	DITTO
P_4	$0.39\times(3.0+1.0\times12.3) = 5.97$ t/m²	DITTO
P'_4	$0.228\times(3.0+1.0\times12.3) = 3.49$ t/m²	DITTO
P_5	$3.49+0.228\times(D-5.9) = (0.228D-2.145)$ t/m²	DITTO
P_6	$3.0\times3.8 = 11.4$ t/m²	The passive earth pressure
P'_6	$2.561\times3.8 = 9.73$ t/m²	DITTO
P_7	$2.561\times5.9 = 15.11$ t/m²	DITTO
P'_7	$4.395\times5.9 = 25.93$ t/m²	DITTO
P_8	$4.395D$ t/m²	DITTO
P_9	1.3 t/m²	The water pressure
P_{10}	4.23 t/m²	DITTO

FORM 04

1415

3) The external moment calculation

Table 4. The Summary of the external moments

No	The Forces S_i (t/m²)	Arms A_i (m)	Moments M_i (t.m/m)
1	$\frac{1.17+1.68}{2}$ x1.3=1.85	$-(\frac{2x1.17+1.68}{1.17+1.68}) \times \frac{1.3}{3}$ =-0.611	- 1.13
2	$\frac{1.68+3.67}{2}$ x5.1=13.64	$\frac{2x3.67+1.68}{3.67+1.68} \times \frac{5.1}{3}$ = 2.866	39.09
3	$\frac{3.67+5.97}{2}$ x5.9=28.44	$\frac{2x5.97+3.67}{3.67+5.97} \times \frac{5.9}{3}$ + 5.1 = 8.285	235.63
4	3.49x(D-5.9) =3.49D - 20.591	11.0 + $\frac{1}{2}$ x(D-5.9)= 0.5D + 8.05	$1.745D^2$ +17.799D -165.758
5	$0.5x0.228x(D-5.9)^2$ $=0.114D^2$-1.345D+3.968	11.0 + $\frac{2}{3}$ x(D-5.9)=0.667D + 7.067	$0.076D^3-0.09D^2$ -6.858D+28.042
6	0.5 x 11.4 x 3.8 = 21.66	5.1 + $\frac{2}{3}$ x 3.8 = 7.633	165.33
7	0.5 x (9.73+15.11) x 2.1 = 26.08	8.9 + $\frac{2x15.11+9.73}{9.73+15.11} \times \frac{2.1}{3}$ = 10.026	261.48
8	25.93 x (D - 5.9) = 25.93D - 152.987	11.0 + $\frac{1}{2}$ x(D-5.9)= 0.5D + 8.05	$12.965D^2$ + 132.243D - 1231.545
9	$0.5x4.395x(D-5.9)^2$ $=2.198D^2$-25.93D+76.495	11.0 + $\frac{2}{3}$ x(D-5.9)= 0.667D + 7.067	$1.466D^3-1.76D^2$ -132.232D +540.59
10	0.5 x 1.3 x 1.3 = 0.85	$-(\frac{1}{3}$ x 1.3) = - 0.433	- 0.37
11	0.5x(1.3+4.23)x2.93 = 8.10	$\frac{2x4.23+1.3}{1.3+4.23} \times \frac{2.93}{3}$ = 1.724	13.964
12	4.23 x (D + 2.17) = 4.23D + 9.179	0.5 x (D + 2.17) = 0.5D + 1.085	$2.115D^2$ + 9.179D + 9.959

Accordingly the turning moment M_t and the resistant moment M_r are calculated as follows.

i) The turning moment

$$M_t = M_1 + M_2 + M_3 + M_4 + M_5 + M_{11} + M_{12}$$
$$= -1.13 + 39.09 + 235.63 + 1.745D^2 + 17.799D - 165.758 + 0.076D^3$$
$$-0.091D^2 - 6.858D + 28.042 + 13.964 + 2.115D^2 + 9.179D + 9.959 - 0.37$$
$$= 0.076D^3 + 3.769D^2 + 20.12D + 159.427$$

FORM 04

ii) The resistant moment

$$
\begin{aligned}
M_r &= M_6 + M_7 + M_8 + M_9 \\
&= 165.33 + 261.48 + 12.965D^2 + 132.243D - 1231.545 + 1.466D^3 \\
&\quad - 1.763D^2 - 132.232D + 540.59 \\
&= 1.466D^3 + 11.202D^2 - 264.145
\end{aligned}
$$

6.2 The Calculation of The Embedded Length

The embedded length is calculated by making the balance of the turning moment and the resistant moment, so this calculation is as follows.

$M_r = F_s \cdot M_t$ where F_s : the factor of safety = 1.5

$1.466D^3+11.202D^2-264.145 = 1.5\times(0.076D^3+3.769D^2+20.12D+159.427)$

$1.352D^3+5.549D^2-30.18D-503.286 = 0$

Accordingly solving the above equation

$$D = 7.0\ m$$

Therefore the required length L'of steel sheet piles for seawall is colculated as follows.

$L' = 6.4 + 7.0 = 13.4\ m$

Now the length L of steel sheet piles for seawall is determined in accordance with the point level of the front sheet piles(= EL.- 11.6 m), so this calculation is as follows.

$L = 6.4 + 9.0 = 15.4\ m \rightarrow \underline{\underline{16.0\ m}} > L' = 13.4\ m$

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6.3 Design of The Steel Sheet Piles

1) The reaction at the support

The reaction at the support is calculated for a sinple beam that have the upper end as Tie Rod setting up and the lover end as the sea bottom, so the calculations are as follows.

$$M_o = -1.13 + 39.09 - 0.37 + 13.964 + 4.23 \times 2.17 \times (0.5 \times 2.17 + 2.93)$$
$$= 88.41 \text{ t·m}$$
$$S_o = 1.85 + 13.64 + 0.85 + 8.1 + 4.23 \times 2.17 = 33.62 \text{ t}$$

$$R_l = \frac{M_o}{l_o} = \frac{88.41}{5.1} = 17.34 \text{ t/m}$$

$$R_u = S_o - R_l = 33.62 - 17.34 = 16.28 \text{ t/m}$$

2) The maximum bending moment

The calculations for the maximum bending moment are as follows.

$$K = \frac{P_l - P_o}{l_o} = \frac{10.07 - 2.98}{5.1} = 1.39$$

$$X = \frac{10.07 - \sqrt{10.07^2 - 2 \times 1.39 \times 17.34}}{1.39} = 1.997 \text{ m}$$

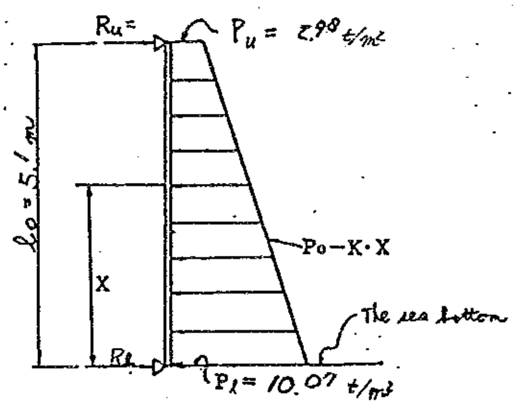

Fig 10. The load diagram

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Therefore M_{max} is calculated as follows.

$$M_{max} = 17.34 \times 1.997 - \frac{1}{2} \times 10.07 \times 1.997^2 + \frac{1}{6} \times 1.39 \times 1.997^3$$
$$= 16.4 \text{ t·m/m}$$

Now using the sheet pile type SP-V_L (the section modulus: $Z = 2270$ cm³/m, the material: SY-30), the stress calculation is as follows.

$$\sigma_t = \frac{M_{max}}{Z} = \frac{1\,640\,000}{2270} = 723 \text{ kg/cm}^2 < \sigma_{ta} = 1800 \text{ kg/cm}^2$$

o.k

1422

7. Design of Other Equipments

1) Design of tie rod

Setting the span length of tie rod is $l = 1.25$ m, the tension T of tie rod is as follows.

$$T = R_u \times l = 16.28 \times 1.25 = 20.35 \text{ t}$$

Then setting up tie rod processed by the following material and size, the stress calculation is as follows.

$$\sigma_t = \frac{T}{A} = \frac{20\,350}{19.63} = 1\,037 \text{ kg/cm}^2 < \sigma_{ta} = 1800 \text{ kg/cm}^2$$

o.k

where the material : high tension steel 45(σ_y=45kg/mm^2)

the diameter : ϕ = 50 mm

the section area : A = 19.63 cm^2

2) Design of wale

Calculating the maximum bending moment working to wale assumed to be a simple beam,

$$M_b = \frac{T \times l}{8} = \frac{20.35 \times 1.25}{8} = 3.18 \text{ t·m}$$

Now using the two chanel type steels(the size: [- 250 x 90 x 9 x 13, the section modulus: Z = 335cm^3,the material SS-41) as wale, the stress calculation for wale is as follows.

$$\sigma_t = \frac{M_b}{2Z} = \frac{318\,000}{2 \times 335} = 475 \text{ kg/cm}^2 < \sigma_{ta} = 1400 \text{ kg/cm}^2$$

o.k

8. Design of Anchor Sheet Piles

1) The determination of the size of anchor sheet piles

The process of this calculation is applied to the design of a vertical anchor pile, so the calculation is as follows.

Tension of tie rod : $R_u = 16.28$ t/m

The ground reaction coefficient K is calculated as follows.

$$K = \alpha \cdot E_o \cdot D^{-0.75} \cdot y^{-0.5}$$

$$= 0.2 \times 168 \times 100^{-0.75} \times 1.0^{-0.5}$$
$$= 1.06 \ \text{kg/cm}^3 = 1.06 \times 10^3 \ \text{t/m}^3$$

where. E_o : the transformed coefficient of the ground (kg/cm²)

$E_o = 28N = 28 \times 6 = 168$ kg/cm²

α : the coefficient applied to E_o = 0.2

D : the unit width of sheet piles = 100 cm = 1 m

y : the basic displacement = 1.0 cm

Now using SP-Ⅲ type sheet piles(the section modulus: Z = 1 340 cm³/m), the bending stiffness and the maximum bending moment is calculated as follows.

$$EI = 21\ 000\ 000 \times 0.000168 = 3\ 528 \ \text{t}\cdot\text{m}^2$$

Therefore calculating the specific value of sheet piles(per 1 m unit width),

$$\beta = \sqrt[4]{\frac{k \cdot B}{4EI}} = \sqrt[4]{\frac{1.06 \times 1000 \times 1.0}{4 \times 3\ 528}} = 0.524 \ \text{m}^{-1}$$

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$$M_{max} = \frac{T}{\beta} \cdot \exp\left(-\frac{\pi}{4}\right) \cdot \sin\frac{\pi}{4}$$

$$= 0.322 \times \frac{16.28}{0.524}$$

$$= 10.0 \ \mathrm{t \cdot m/m}$$

Accordingly the stress calculation for sheet piles is as follows.

$$\sigma_t = \frac{M_{max}}{Z} = \frac{1\,000\,000}{1\,340} = 747 \ \mathrm{kg/cm^2} < \underset{ok}{\sigma_{ta}} = 1800 \ \mathrm{kg/cm^2}$$

Then calculating the displacement δ at the top of sheeet piles,

$$\delta = \frac{R_u}{2EI\beta^3} = \frac{16\,280}{2 \times 3\,528 \times 10^7 \times (0.00524)^{-3}} = 1.6 \ \mathrm{cm}$$

And calculating the length of sheet piles

$$L = \frac{\pi}{\beta} + 0.5 = \frac{\pi}{0.524} + 0.5 = 6.495 \rightarrow \underline{7 \ \mathrm{m}}$$

According to the previous calculations, the size of an anchor sheet pile is determined as follows.

Type : SP-Ⅲ
The length : $L \geqq 7.0$ m

* The length pf the anchor sheet piles will be determined by the ground surface elevation at the site.

2) The calculation for the distance between seawall and anchor sheet piles
The angle of the active failure and the angle of the passive failure are shown in Table 5.

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Table 5. The summary of the angle of failure

Items	The Angle
The angle of the avtive failure	53°
The angle of the passive failure	22°

According to the above table, the distance L' between seawall and the anchor sheet piles is calculated as follows.

$$L' = 5.1 \times \cot 53^\circ + \frac{\pi}{3\beta} \times \cot 22^\circ$$

$$= 8.8\ \mathrm{m}$$

Therefore the distance between seawall and the anchor sheet piles is determined as flollws.

$\therefore$ $\underline{\underline{L = 10.0\ \mathrm{m}}}$