CV-1 CIVIL DESIGN CONDITION

1. Design Condition

   1) Sea water level

      a) The Highest High Water Level (The Highest Astronomical Tide is adopted as design H.H.W.L)

         H.H.W.L = E.L + 3.23 m

      b) Mean Sea Water Level

         M.S.L = E.L + 1.64 m

      c) The Lowest Low Water Level (The lowest Astronomical Tide is adopted as design L.L.W.L)

         L.L.W.L = E.L - 0.43 m

   2) Wind load

      The design wind loads are indicated in the design concept of architectural/structural section, if necessary.

   3) Ground level (G.L)

      G.L. = E.L + 4.80 m

9501

4) Soil condition

The soil physical values shall be determined at the normal cases as follows.

a) The N – Value

The N – Value shall be adopted from the site boring test.

b) The angle of the internal friction ϕ

In the case of sandy soil, the angle of the internal friction ϕ shall be calculated by the following equation.

$$\phi = 15 + \sqrt{15N} \leqq 45^\circ \quad \text{provided that} \quad N > 5$$

2. Design load

1) Composite vertical load	The increment ratio
a) Long term (D.L + L.L)	1.0
b) Short term (D.L + L.L + W.L)	1.25
c) Short term (D.L + S.L)	1.50

Where D.L : Dead load
L.L : Live load
W.L : Wind load
S.L : Seismic load

2) Seismic load (S.L)

$F_w = K_h \cdot W$ Where K_h : Seismic coeficient = 0.1
W : Weight of structure included dead load.

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3) Live load (Wheel load)

a) Truck load

Truck load (T - 20)

The gross weight	:	20 ton
A front wheel weight	:	2 ton
A rear wheel weight	:	8 ton

b) The impact coefficient (i)

$$i = 0.3$$

4) Cohesion C

Cohesion C shall be calculated by the following equation.

$$C = \frac{q_u}{2}$$ q_u : the uniaxial compressive stress

q_u can be obtained from Table 1, if N - values can be measured.

Table 1. Relation of the N values vs consistency q_u of cohesive soil

N value	Consistency of cohesive soil	q_u (kg/cm²)
0 to 1	Very soft	0.25 to below
2 to 4	Soft	0.25 to 0.5
5 to 8	Normal(medium)	0.5 to 1.0
9 to 15	Hard	1.0 to 2.0
16 to 30	Very hard	2.0 to 4.0
30 to above	Consolidated	4.0 to above

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3. Hydrographic calculation

1) The Summary of the water head loss

Table 2. Example of the loss in water channel

Equipment	Loss
Intake	① Loss due to inflow curtain. ② Loss due to friction between intake.
Intake Channel	① Loss due to inflow at intake channel ② Loss due to friction of water channel wall ③ Loss due to water channel's cross sectional change (Gradual contractin and gradual expansion) and bend, etc.) ④ Loss due to water channel's accessory machineries and structures.
Screen Pump Pit	① Loss due to inflow at Screen Room ② Loss due to Screens ③ Loss due to inflow at Pump Room ④ Loss due to room wall friction
Discharge Channel	① Loss due to outlet conduit and discharge culvert ② Loss due to the wall friction of channel ③ Loss due to water channel's sectional change (gradual contraction, gradual expansion) ④ Loss due to water channel's accessory (machineries and structures)
Outlet	① Loss due to outlet's exit ② Loss due to the friction of water channel wall ③ Loss due to water channel's sectional change (gradual expansion, etc) ④ Loss due to water channel's necessory structures, such as hiden wier, pier, etc.

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2) The coefficient of roughness

The values given in Table 2 are used as the satandard coefficients of the roughness used for hydrographic calculation of a water channel.

Table 2. The coefficient of the roughness used for water channel

Wall shape	n	Wall shape	n
Welded steel pipe	0.012	Ordinary concrete with shells stuck	0.02
Good concrete surface such as centrifugal force steel reinforced concerte pipe	0.013	Steel plate	0.02
		Other especially rough surface	0.03
Ordinary concrete such as cast-in-field concrete	0.015		
Concrete block	0.02		

0060

4. The Allowable Unit Stress

1) The allowable unit stress of reinforced concrete (kg/cm²)

Concretes \ Stresses		Permanent Stresses			
		Compression	Shear (1)	Shear (2)	Bond
Concrete for (above) C-400	plain bar	130	5.5	24	10
	Deformed bar		(slab) 11		20
Concrete for C-300	Plain bar	100	5	22	9
	Deformed bar		(slab) 10		18
Concrete for C-240	Plain bar	80	4.5	20	8
	Deformed bar		(slab) 9.0		16
Concrete for C-210	Plain bar	70	4.25	19	7.5
	Deformed bar		(slab) 8.5		15
Concrete for C-180	Plain bar	60	4	18	7
	Deformed bar		(slab) 8		14

*Remarks (1):No calculating with diagonal tention bar.
(2):Calculating with diagonal tention bar.

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2) The allowable unit stress of reinforcing bars (kg/cm²)

Stresses / Materials	Permanent stresses	
	Tension	Compression
SD 30	1,800	1,800
SD 35	1,800	2,000
SD 30 for Pile	1,600	1,800

3) The allowable unit stress of steel (kg/cm²)

	Permanent Stresses		
Type of Steel	Tension	Compression	Shear
SS 41 SM 41	1,400	1,400	800

4) The allowable unit stress of welded joint (kg/cm²)

Appli- cation	Stresses	Permanent Stresses			
	Welding Positions	Groove Weld			Fillet
	Materials	Tension	Compress	Shear	Shear
	(SS41,SM41)	1,400	1,400	800	800

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5) The young's modulus of concrete

a) At the case of the displacement calculation

CK(kg/cm²)	180	240	270	300
Ec(kg/cm²)	2.2×10^5	2.5×10^5	2.65×10^5	2.8×10^5

b) At the case of the unit stress calculation

$E_c = 1.4 \times 10^5$ (kg/cm²)

5. Applicable Standards

1) Japan Society of Civil Engineers

a) Standards for Calculation of Reinforced Concrete Structures

b) Design Standards for Steel Structures

c) Standards for Structural Design of Building Foundations

2) Japan Road Association

Specifications for Highway Bridges

3) Japan Port and Harbour Research Institute

Technical Standards for Port and Harbour Facilities

4) Standards of Architectural Institute of Japan

Standard Specification for Concrete

5) Japanese Industrial Standards (JIS)

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CV-2 THE BASIC DATA FROM THE STUDY OF HYDRAULICS AT THE COOING WATER WAY

Contents of this note is shown as below.

Contents

1. Intake Open Channel 2

2. Pump Pit 7

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1. Intake Open Channel

Outline of Intake Open Channel is shown in Fig 1, Fig 2.
The bases of the dimensional data are explained as follows.

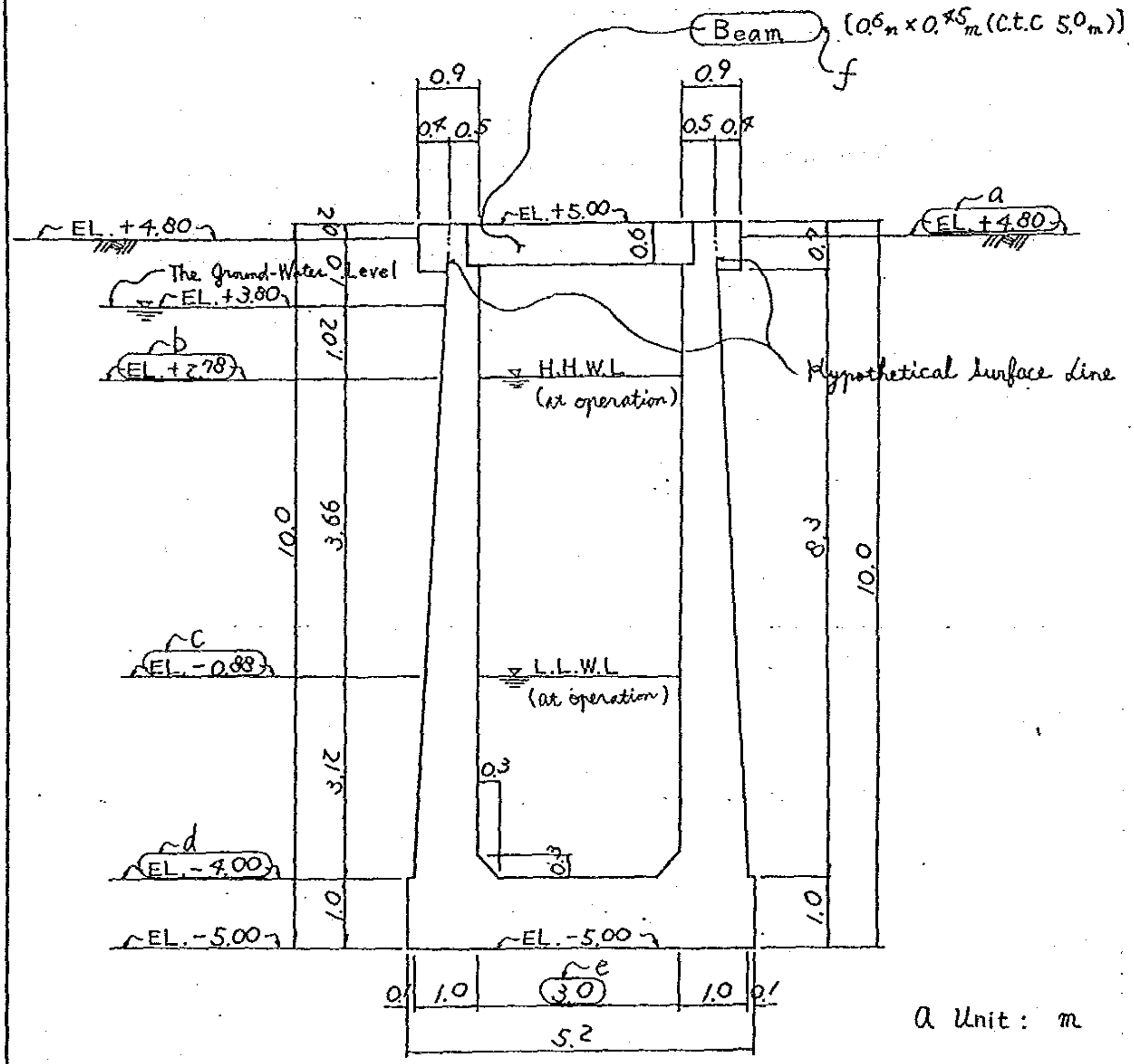

Fig 1. Typical section of Intake Open Channel

1046

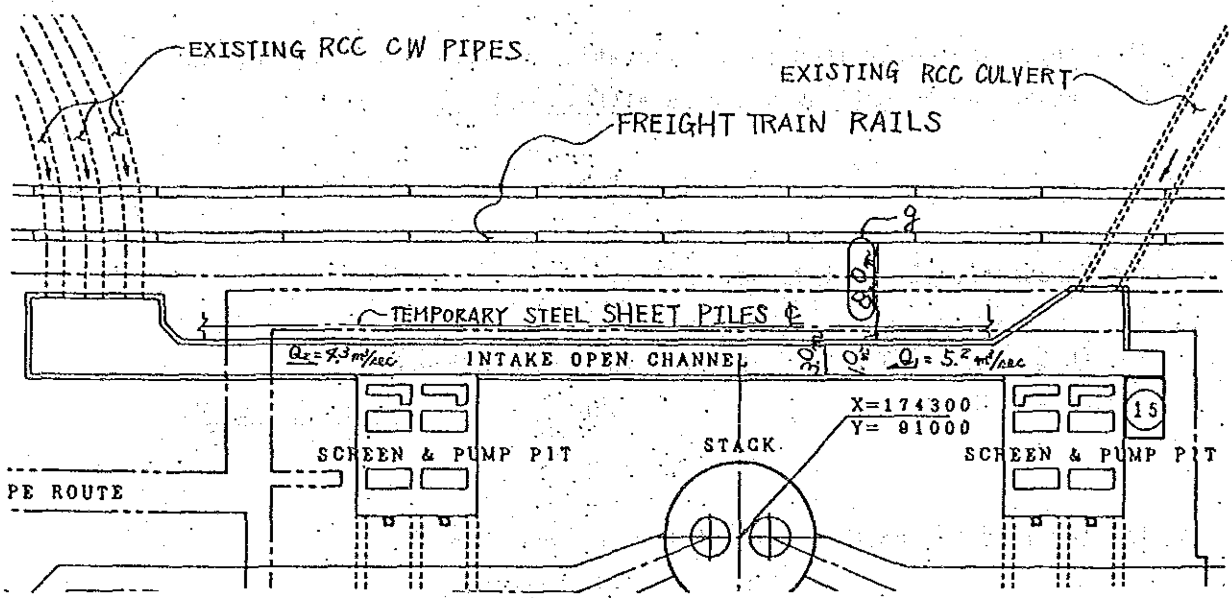

Fig 2. Plan of Intake Open Channel

a. The ground level(EL.+4.8m)

The ground level was determined in "Civil Design Conditon"(vid.1.3,P1, ENGINEERING SHEET No.EWC-1001), so the site ground shall be leveled at EL.+4.8m.

b. The highest high water level(EL.+2.78m)

This level was determined in consideration of the water head loss due to Intake Culvert(vid.3.1).a),P11,ENGINEERING SHEET No.EWC-1003) at the highest high sea water level(EL.+3.23m), so the calculation is as follows.

↓

(vid.1.1).a,P1,EWC-1001).

[H.H.W.L]		(The head loss due to culvert)	
EL.+3.23m	−	0.45m	= EL.+2.78m

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c. The lowest low water level(EL. − 0.88m)

This level was determined in consideration of the water head loss due to Intake Culvert at the lowest low water level(EL. − 0.43m).

↓

(vid.1.1).c,EWC-1001)

The Calculation is as follows.

[L.L.W.L] (The head loss due to Intake Culvert)

EL. − 0.43m − 0.45m = EL. − 0.88m

d. The level of the bottom floor of Intake Open Channel

(EL. − 4.00m)

The level of bottom floor was determined in accordance with the bottom floor level of existing Intake Culvert (EL. − 1.3feet≒EL. − 4.0m) as shown in Fig 3.

e. The width of Intake Open Channel(= 3m)

The width of Intake Open Channel was determined to limit the water velocity less than V = 1.0 m/sec at L.L.W.L.

The calculation is considered for the shells stuck 10cm.

(vid.Fig 4,P10,EWC-1003)

$$V = \frac{Q_1}{A} = \frac{5.2m^3}{(3.0-2\times0.1)\times(3.12-0.1)} = 0.61 \text{ m/sec} < 1.0 \text{ m/sec}$$

f. The distance between the rails of the freight train and the center line of temporary retaining wall(steel sheet pile).

The distance was determined to avoid the influence area occured by the relaxation of the back ground of retaining wall.

FORM 04

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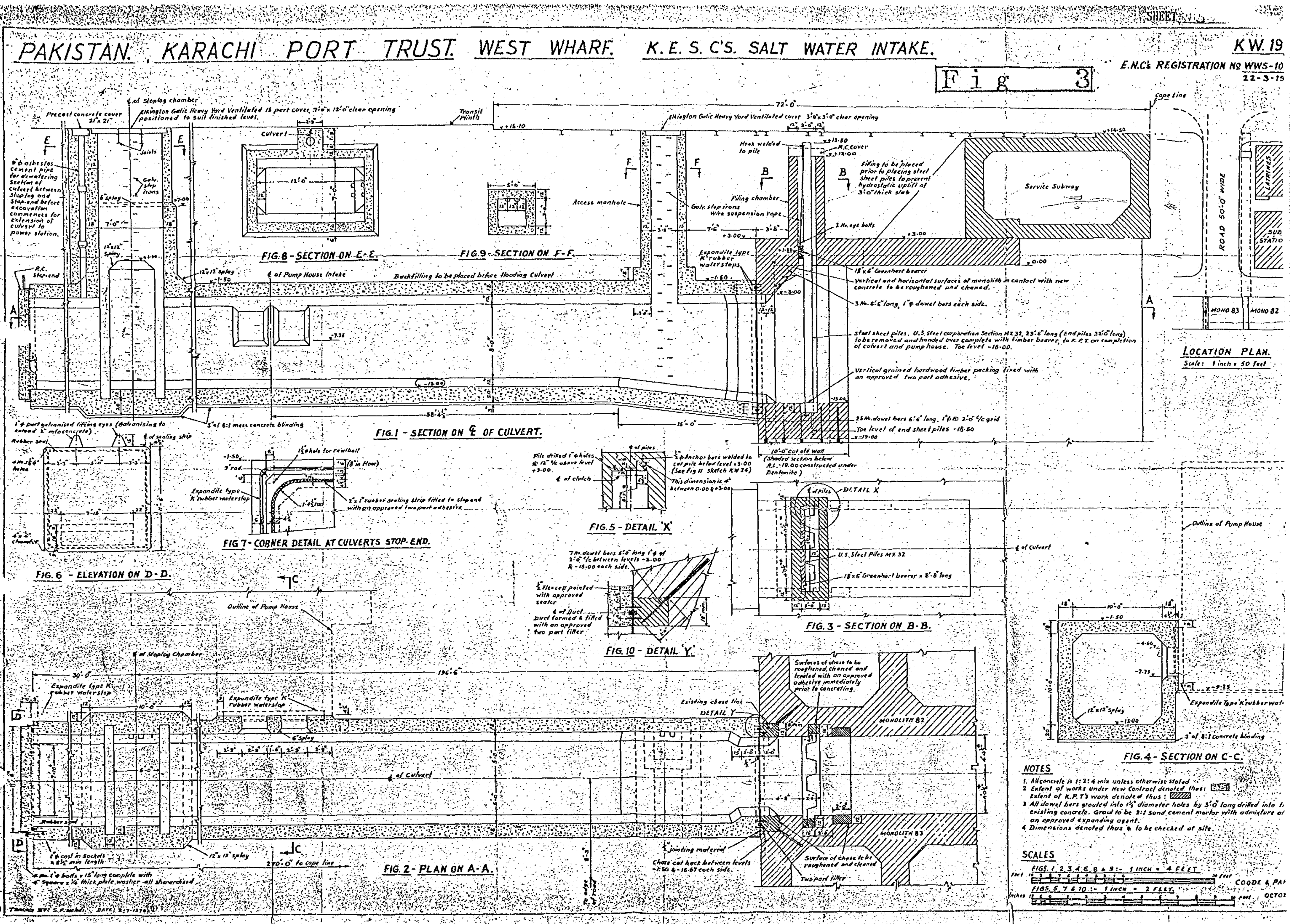
SHEET
PAKISTAN. KARACHI PORT TRUST. WEST WHARF. K. E. S. C'S. SALT WATER INTAKE.
KW. 19
E.N.C's REGISTRATION No WWS-10
22-3-75
Fig 3
FIG. 8 - SECTION ON E-E.
FIG. 9 - SECTION ON F-F.
FIG. 1 - SECTION ON ℄ OF CULVERT.
FIG. 6 - ELEVATION ON D-D.
FIG. 7 - CORNER DETAIL AT CULVERTS STOP-END.
FIG. 5 - DETAIL 'X'
FIG. 10 - DETAIL 'Y'
FIG. 3 - SECTION ON B-B.
FIG. 4 - SECTION ON C-C.
FIG. 2 - PLAN ON A-A.
LOCATION PLAN.
Scale: 1 inch = 50 feet
Service Subway
ROAD 50'-0" WIDE
MONO 83
MONO 82
MONOLITH 82
MONOLITH 83
Outline of Pump House
℄ of Culvert
℄ of Stoplog chamber
Precast concrete cover
Access manhole
Piling chamber
Steel sheet piles, U.S. Steel corporation Section MZ 32, 23'-6" long (End piles 32'-0" long) to be removed and handed over complete with timber bearer, to K.P.T. on completion of culvert and pump house. Toe level -16.00.
Vertical grained hardwood timber packing fixed with an approved two part adhesive.
Vertical and horizontal surfaces of monolith in contact with new concrete to be roughened and cleaned.
10'-0" Cut off Wall (Shaded section below R.L. -19.00 constructed under Bentonite)
Backfilling to be placed before flooding Culvert
Expandite type 'K' rubber waterstop
Surface of chase to be roughened and cleaned
Two part filler
NOTES
1. All concrete is 1:2:4 mix unless otherwise stated.
2. Extent of works under New Contract denoted thus:
Extent of K.P.T's work denoted thus:
3. All dowel bars grouted into 1½" diameter holes by 3'-0" long drilled into the existing concrete. Grout to be 2:1 sand cement mortar with admixture of an approved expanding agent.
4. Dimensions denoted thus * to be checked at site.
SCALES
FIGS. 1, 2, 3, 4, 6, 8 & 9 :- 1 INCH = 4 FEET
FIGS. 5, 7 & 10 :- 1 INCH = 2 FEET
COODE & PA

The section of the influence area occured by the relaxation of the back ground is shown in Fig 4.

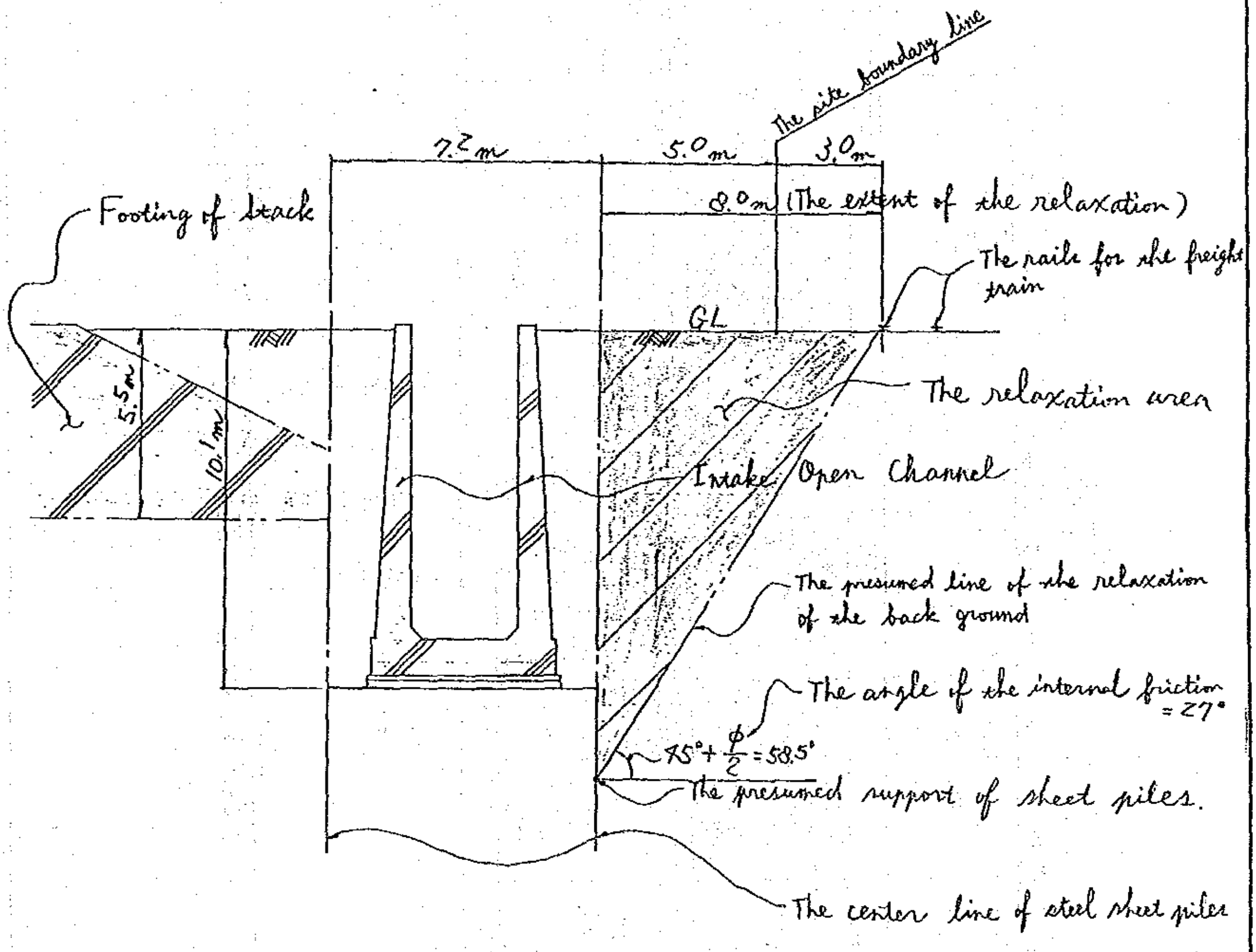

Fig 4. The influence area of the relaxation of the back ground

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2. Pump pit

Outline of Pump Pit is shown in Fig 5, Fig 6.
The bases of the dimensional data are explained as follows.

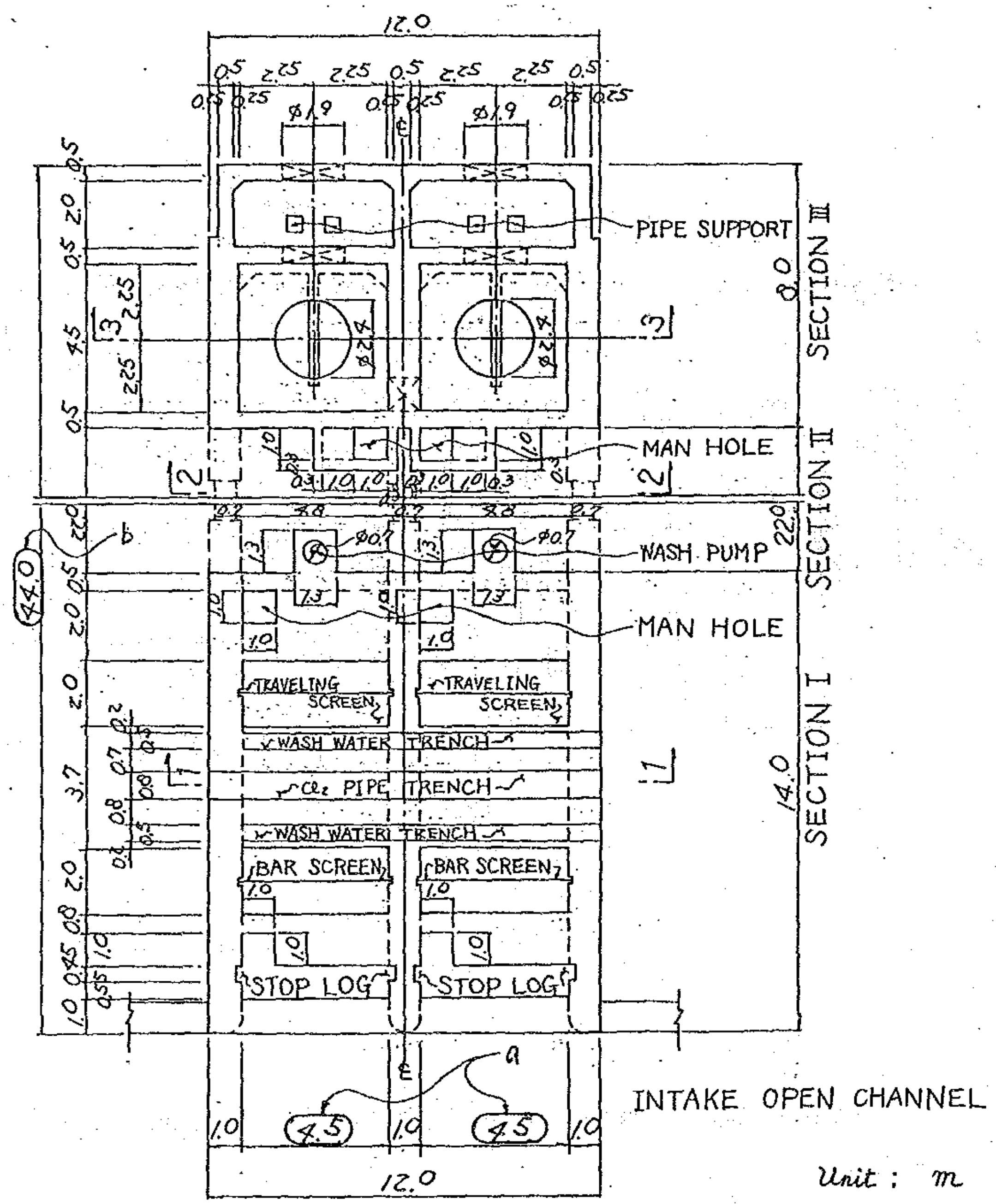

Fig 5. Plan of Pump Pit

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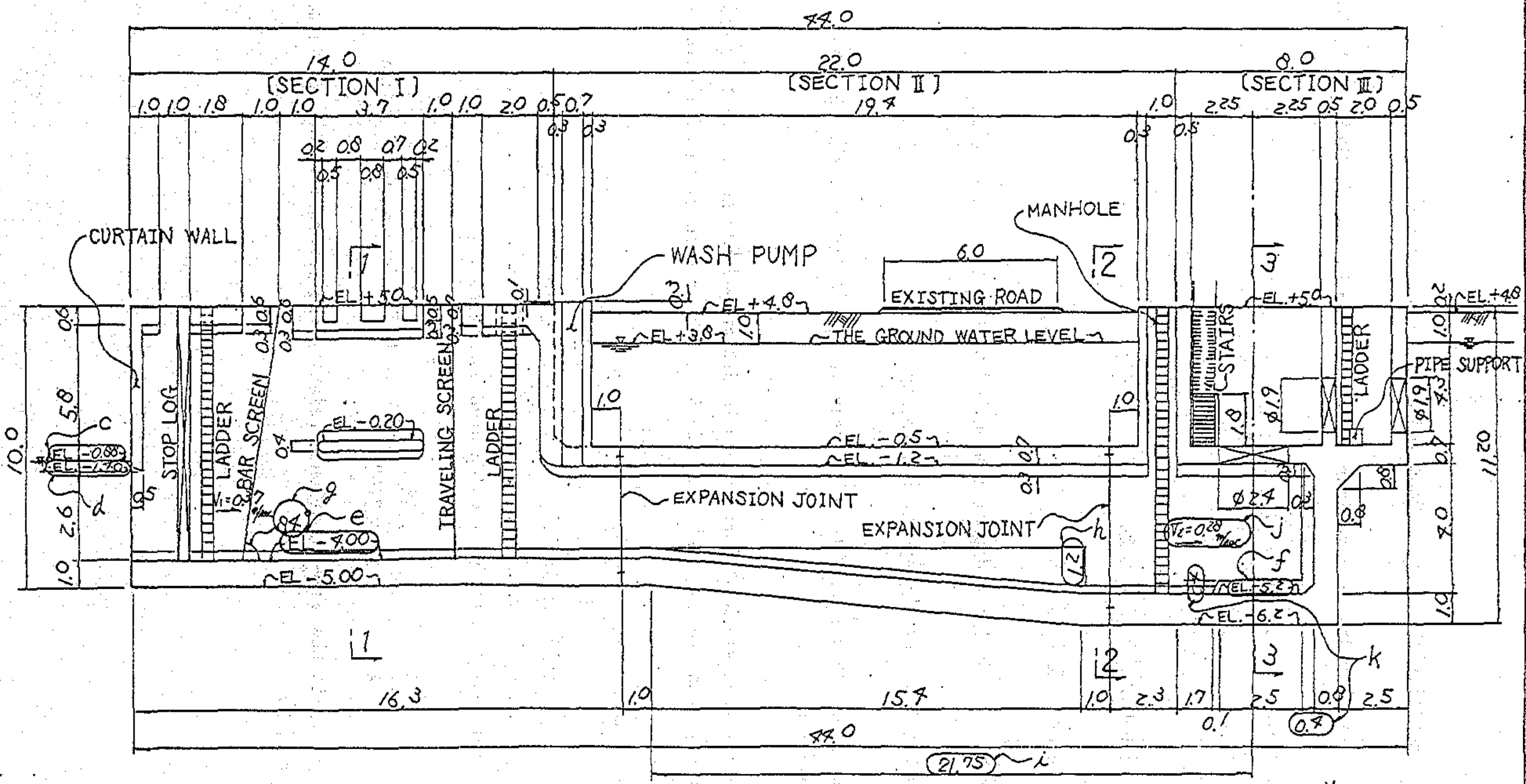

Unit: m

Fig 6. Profile of Pump Pit

a. The internal width of Pump Room and Screen Room.

The internal width of Pump Room was determined by the following relations.

i) by the data from Table 1.

Let the width is B,

$$2.0\ Do \leqq B \leqq 2.5\ Do$$

$$3.6m \leqq B \leqq 4.5m$$

Where Do : the diameter of C.W Pump's bellmouth = 1.8m

ii) by the limit of the approrch velocity

The approach velocity to bar screen V_1 is usually controled by the following relation.

The internal width of Screen Room is determined by the approach velocity to bar screen, then the approach velocity V_1 is restricted by the following relation.

$$V_1 \leqq \underline{0.375\ m/sec}$$

↓

(vid. Table 1)

In this design case V_1 is calculated as follows.

$$V_1 = \frac{Q}{A} = \frac{4.75}{4.3 \times 3.02} = 0.37\ m/sec < 0.375\ m/sec$$

According to the above mentioned relation,the internal width was dertermined $B_1 = 4.5m$

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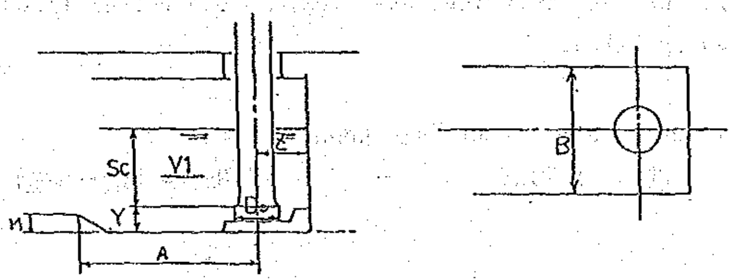
Table 1. The compared table with the usual and the recommended dimensional data of Pump Pit

No.	Items	The usual value	The recommended value	Remarks
1	The diameter of Bellmouth :Do		Do=1800mm	
2	The width of Pump Room :B	B=2.0~2.5Do	B =4500mm	
3	The distance between Bellmouth and the bottom floor :Y	B=0.5~0.75Do	Y =900mm	Y = 0.5Do
4	The distance between the center of Pump and the back wall :Z	Z=0.8~1.0Do	Z =1800mm	The preventive wall for vortex was designded
5	The submarged depth:Sc	Sc≧ 1.5Do	Sc=2700mm	
6	The approach velocity to Pump and Screen :V_2	V_2≦0.375m/sec		In this design case V_2 = 0.28 m/sec
7	The distance between the Center of Pump and the mound :A	A≧6.0Do	A≧10.8m	
8	The height of mound:Hm	Hm≦ 0.8Do	Hm≦1.44m	



The reference drawing

b. The length of Pump Pit

The length of Pump Pit is determined by the site layout plan.
(vid. DWG No. WGTS-1002)

c. The lowest low water level(EL.−0.88m)

This water level was the same level as Intake Open Channel.

d. The lowest side level of curtain wall

The lowest side level of curtain wall is calculated as follows.

[L.L.W.L]		(the allowance)	
EL.−0.88m	−	0.52m	= EL.−1.40 m

e. The bottom floor level of Screen Room

The bottom floor level of Screen Room was determined in accordance with the bottom floor level of Intake Open Channel(vid. Fig 1), accordingly this level was setted up at EL.−4.00m.

f. The bottom floor level of Pump Room

The bottom floor level of Pump Room was determined by the height of the back wall of Pump Room, and the height of the back wall of Pump Room H_b was determined by the following relation.

$$H_b = S_c + Y \geqq 1.5D_o = 1.5 \times 1.8 + 0.5 \times 1.8 = \underline{3.6m}$$

accordingly H_b was determined $\underline{H_b = 4.0m}$, so the bottom floor level was calculated as below.

(The lower side level of Pump Room)		[H_b]
EL. −1.20m	−	4.0m = EL.−5.2m

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g. The setting angle between Bar Screen and the bottom floor.

The setting angle of Bar Screen was determined by the reference of "Design for the Civil Structures of Thermal Powew Plant and Nuclear Power Plant" published by The Society of Electric Power Civil Technology, Japan.(vid.12-10-3,P435)

h. A difference in level between the bottom of Screen Room and that of Pump Room.

This difference in level H_m was determined by the following relation. (vid.Item No. 8 in Table 1)

$$\underline{H_m = 1.2m} \leqq 0.8Do = 0.8 \times 1.8m = 1.44m$$

i. The horizontal length of the slope

The horizontal length of the slope A was determined by the following relation to the distance between the vertical center of Pump and mound. (vid.Item No. 8 in Table 1)

$$\underline{A = 21.75m} \geqq 6.00Do = 10.8m$$

j. The approach velocity to Pump

The approach velocity to Pump V_2 is calculated by the following equation.

$$V_2 = \frac{Q}{A} = \frac{4.5}{4.3 \times 3.8} \fallingdotseq 0.28 \text{ m/sec} \leqq \underline{0.375\text{m/sec}}$$

(vid.Item No. 6 in Table 1)

k. The dimensional data of the protection wall for vortex

The dimensional data of the protection wall for vortex was determined by the reference data from the mechanical design structure.

CV-3 HYDROGRAPHIC CALCULATION

Contents of this calculation note is shown as below.

Contents

1. The Results of Hydrographic Calculation 2

2. Design Condition 5

3. The Calculation of The Water Head Loss 11

   1) Intake Tunnel 11

   2) Intake Open Channel 14

   3) Pump Pit 15

   4) Discharge Tunnel 18

   5) Outlet 21

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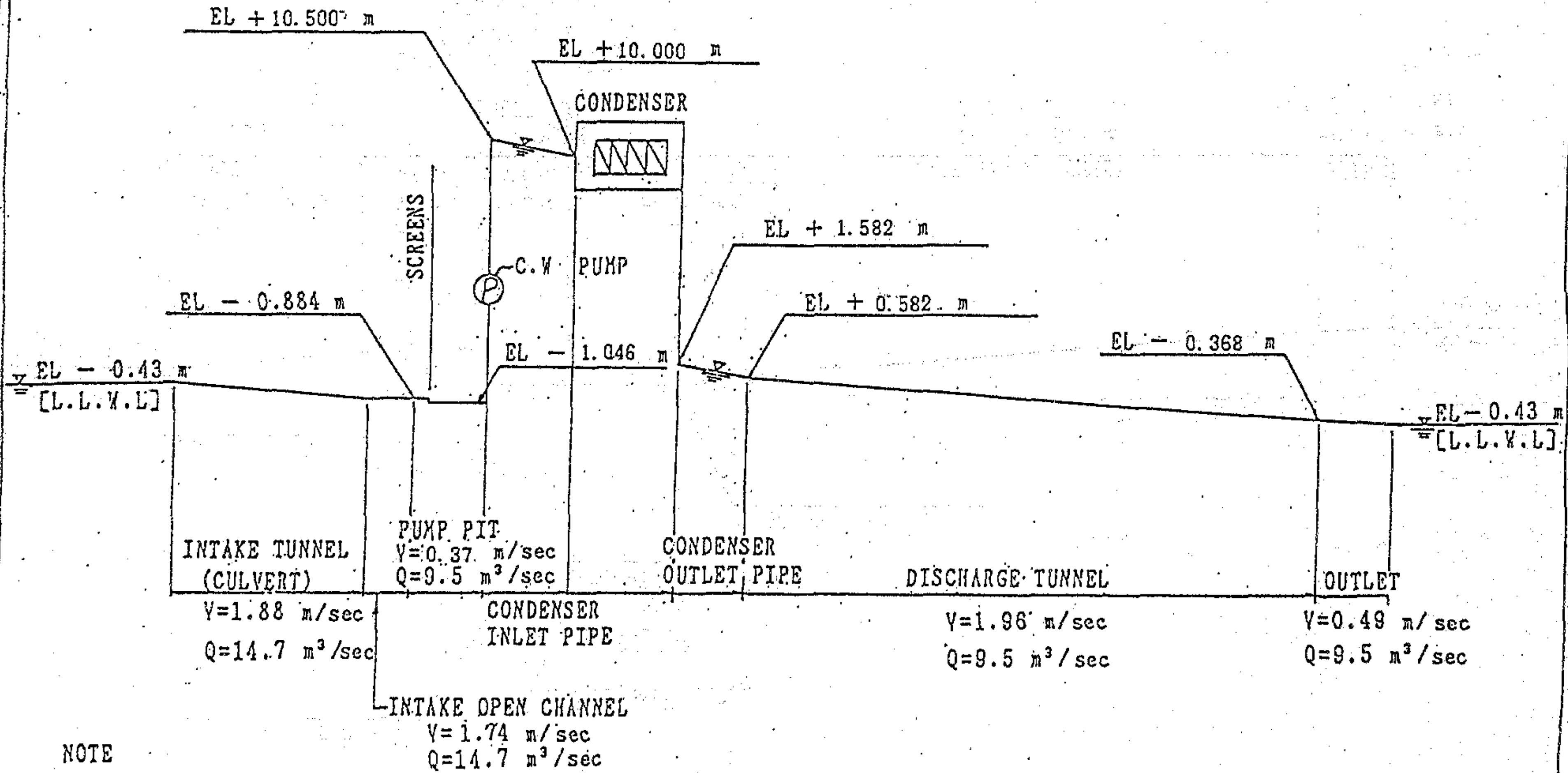
1. Thw Results of Hydrographic Calculation

Hydrographic calculation is executed to determine the water head of circulating water pump and the water head loss of condenser cooling water way.
The results of hydrographic calculation are shown in Fig 1. and Fig 2.

SHEET 3 OF

Fig 1. The Results of Hydrographic Calculaton

(UNIT I)



NOTE

V: Water velocity (m/sec)

Q: Quantity of water (m^3/sec)

FORM 04

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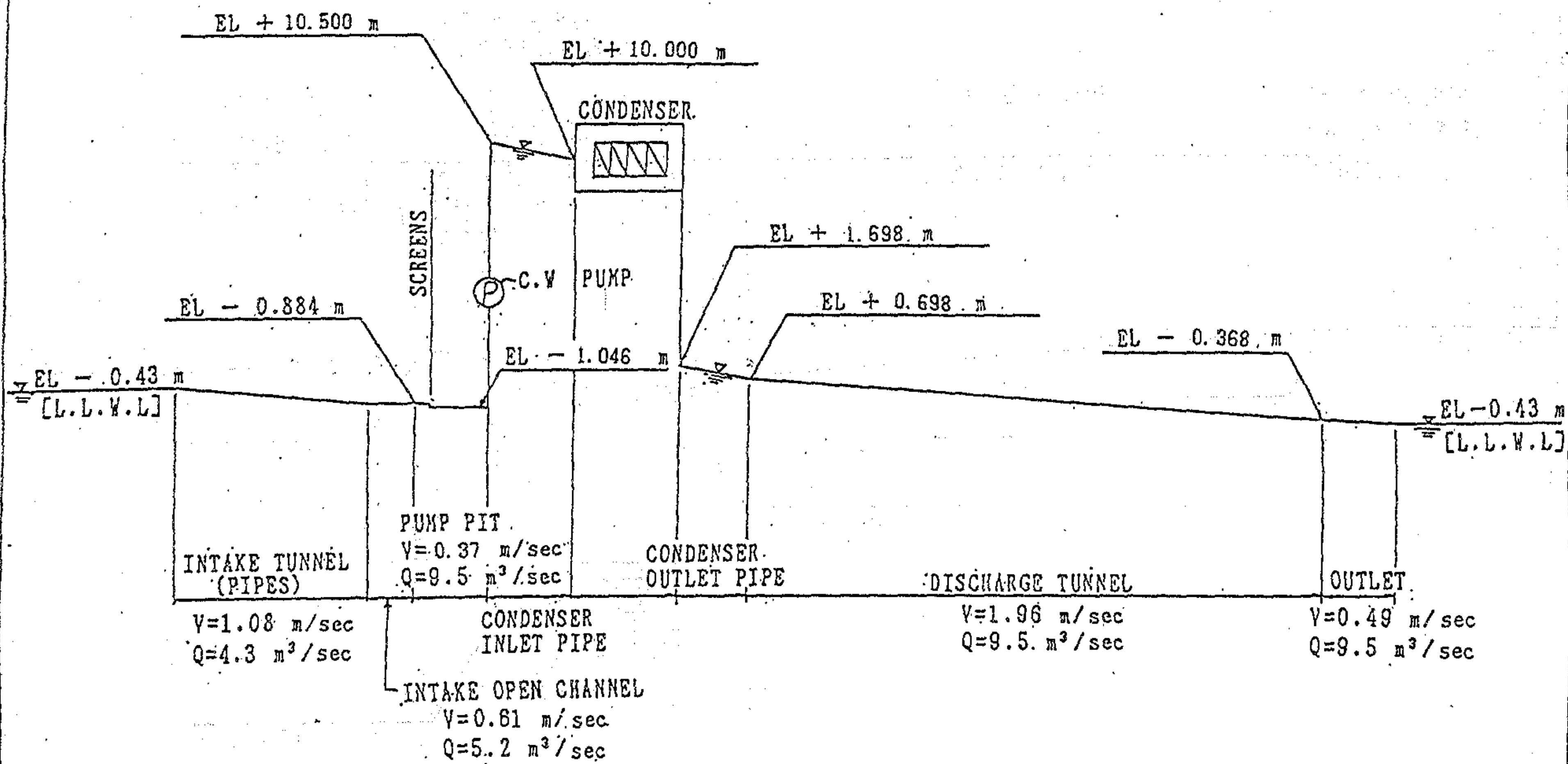
Fig 2: The Results of Hydrographic Calculaton

(UNIT II)



NOTE

V: Water velocity (m/sec)

Q: Quantity of water (m^3/sec)

2. Design Condition

1) Water level

All calculations are executed at the lowest low water level.

L.L.W.L : E.L. - 0.43 m

2) The cofficient of roughness

a) Ordinary concrete such as cast-in-field concrete

the roughness : n = 0.015

b) Ordinary concrete with shells stuck

the roughness : n = 0.02

3) The quantity calculation of Intake structures

The water flow chart of Intake structures are shown in Fig 3.

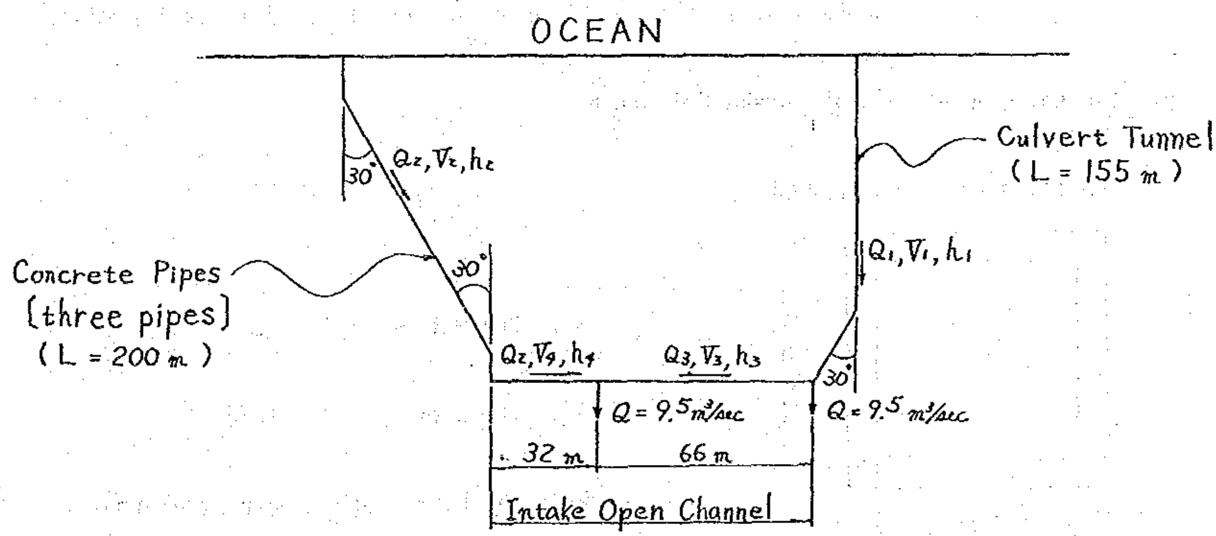


Fig 3. The water flow chart of Intake structures

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Quantities of Intake structures Q_1 are determined to solve the following equations.

$$2Q = Q_1 + Q_2 = 19.0\ m^3 \quad \text{——————} \quad ①$$
$$Q = Q_2 + Q_3 = 9.5\ m^3 \quad \text{——————} \quad ②$$
$$Q_3 = Q_1 - Q = Q_1 - 9.5\ m^3 \quad \text{——————} \quad ③$$
$$h_2 + h_4 = h_1 + h_3 \quad \text{——————} \quad ④$$

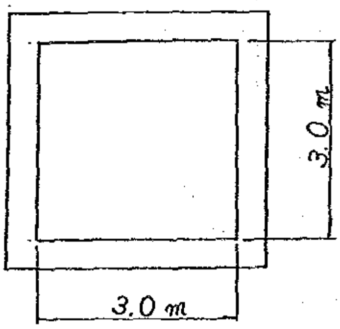
provided that $Q_1 \leqq 2Q$

$Q_2,\ Q_3 \leqq Q$

Where
Q1 : Quantity of culvert tunnel [m³/sec]
Q2 : Quantity of concrete pipes [m³/sec]
[Three pipes located]
Q3 : Quantity of Intake Open Channel (at culvert side) [m³/sec]
Q4 : Quantity of Intake Open Channel (at concrete pipe side) [m³/sec]

a) The water head loss of Intake Culvert h_1

typical section of culvert



S : the wet perimeter [m]

A : the area [m²]

R : the hydrographic mean level [m]

Shell deposits 10cm is considered for calculation.

$$S = 4 \times (3.0 - 0.2) = 11.2\ m$$
$$A = 2.8 \times 2.8 = 7.84\ m^2$$
$$R = \frac{A}{S} = \frac{7.84}{11.2} = 0.7\ m$$

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i) The head loss due to inflow h_{e1},

$$V = \frac{Q_1}{A} = \frac{Q_1}{7.84} = 0.1276\ Q_1$$

$$h_{e1} = fe \cdot \frac{v^2}{2g} = 0.5 \times \frac{(0.1276 Q_1)^2}{2 \times 9.8} = 0.0004\ Q_1^2$$

Where f_e : the coefficient of the inflow loss = 0.5

ii) The head loss due to the wall surface friction h_{f1}

$$h_{f1} = \frac{Q_1^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{Q_1^2 \cdot 0.02^2 \cdot 155}{7.842 \times 0.7^{4/3}} = 0.0016\ Q_1^2$$

iii) The head loss due to bend h_{b1}

$$h_{b1} = f_{b1} \cdot \frac{v^2}{2g} = 0.073 \times \frac{(0.1276\ Q_1)^2}{2 \times 9.8} \fallingdotseq 0.0001\ Q_1^2$$

Where f_{b1} : the coefficient of the bend loss = 0.073

Accordingly the head loss due to Intake Culvert is calculated as hereinafter.

$$h_1 = h_{e1} + h_{f1} + h_{b1} = (0.0004 + 0.0016 + 0.0001) Q_1^2 = 0.0021\ Q_1^2$$

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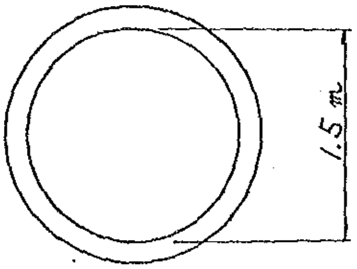
b) The water head loss due to Intake Concrete Pipes h_2

typical section of concrete pipe



Shell deposits 10cm is considered for the calculation.

$$S = \pi \times (1.5 - 2 \times 0.1) = 4.08 \text{ m}$$

$$A = \frac{1}{4} \times \pi \times 1.3^2 = 1.33 \text{ m}^2$$

$$R = \frac{A}{S} = \frac{1.33}{4.08} = 0.326 \text{ m}$$

i) The head loss due to inflow h_{e2}

$$V = \frac{Q_2}{3A} = \frac{Q_2}{1.33 \times 3} = 0.2506\ Q_2$$

$$h_{e2} = f_e \cdot \frac{v^2}{2g} = 0.5 \times \frac{(0.2506\ Q_2)^2}{2 \times 9.8} = 0.0016\ Q_2^2$$

Where f_e : the coefficient of the inflow loss = 0.5

ii) The head loss due to the wall surface friction h_{f2}

$$h_{f2} = \frac{Q_2^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{(Q_2/3)^2 \cdot 0.02^2 \cdot 200}{1.33^2 \cdot 0.326^{4/3}} = 0.0224\ Q_2^2$$

iii) The head loss due to the bend loss h_{b2}

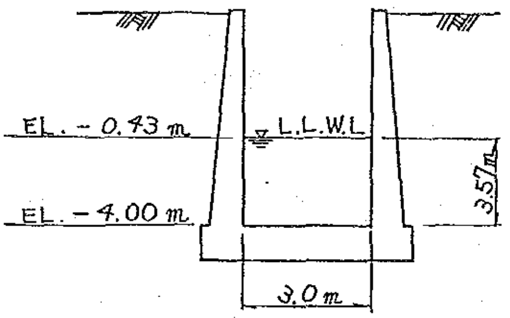
$$h_{b2} = 2 \cdot f_b \cdot \frac{v^2}{2g} = 2 \times 0.073 \times \frac{(0.2506\ Q_2)^2}{2 \times 9.8} = 0.0005\ Q_2^2$$

Accordingly the head loss of Intake Pipe Tunnel is calcurated as hereinafter.

$$h_2 = h_{e2} + h_{f2} + h_{b2} = 0.0016Q_2^2 + 0.0224Q_2^2 + 0.0005Q_2^2 = 0.0245Q_2^2$$
$$= 0.0245(19.0 - Q_1)^2$$

c) The water head loss due to Intake Open Channel [at culvert side]

the section of Intake Open Channel



Shall deposits 10cm are considered for the calculation

$$S = 2 \times 3.47 + (3 - 2 \times 0.1) = 9.74 \text{ m}$$

$$A = 2.8 \times 3.47 = 9.72 \text{ m}^2$$

$$R = \frac{A}{S} = \frac{9.72}{9.74} = 1.00 \text{ m}$$

The head loss h_3 is the same as the head loss due to the wall surface friction.

$$h_3 = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{Q_3{}^2 \times 0.02^2 \times 66}{9.72^2 \times 1.0^{4/3}} = 0.0003Q_3{}^2 = 0.0003(\ Q_1 - 9.5\ )^2$$

d) The water head loss due to Intake Open Channel h_4 [at concrete pipe side]

The section and the process of calculation are the same as C), therefore

$$h_4 = \frac{Q_2{}^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{Q_2{}^2 \times 0.02^2 \times 32}{9.72^2 \times 1.0^{4/3}} = 0.0001\ Q_2{}^2 = 0.0001\ (19.0 - Q_1)^2$$

Above calculated the head losses are substituted into the previous equation ④ here.

$$(0.0245 + 0.0001)\ (19.0 - Q_1)^2 = 0.0021\ Q_1{}^2 + 0.0003\ (Q_1 - 9.5)^2$$

$$0.0246(Q_1{}^2 - 38Q_1 + 361) = 0.0024Q_1{}^2 - 0.0057Q_1 + 0.0271$$

$$0.0222\ Q_1{}^2 - 0.9291\ Q_1 + 8.8535 = 0$$

Q_1 is determined by solving the above equation.

$\underline{Q_1 = 14.7\ \text{m}^3/\text{sec}}$

5801

Accordingly other quantities are calculated as hereinafter.

$$Q_2 = 19.0 - Q_1$$
$$= 19.0 - 14.7$$
$$= \underline{4.3 \ m^3/sec}$$

$$Q_3 = 9.5 - Q_2$$
$$= 9.5 - 4.3$$
$$= \underline{5.2 \ m^3/sec}$$

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3. The Calculation of The Water Head Loss

1) Intake Tunnel

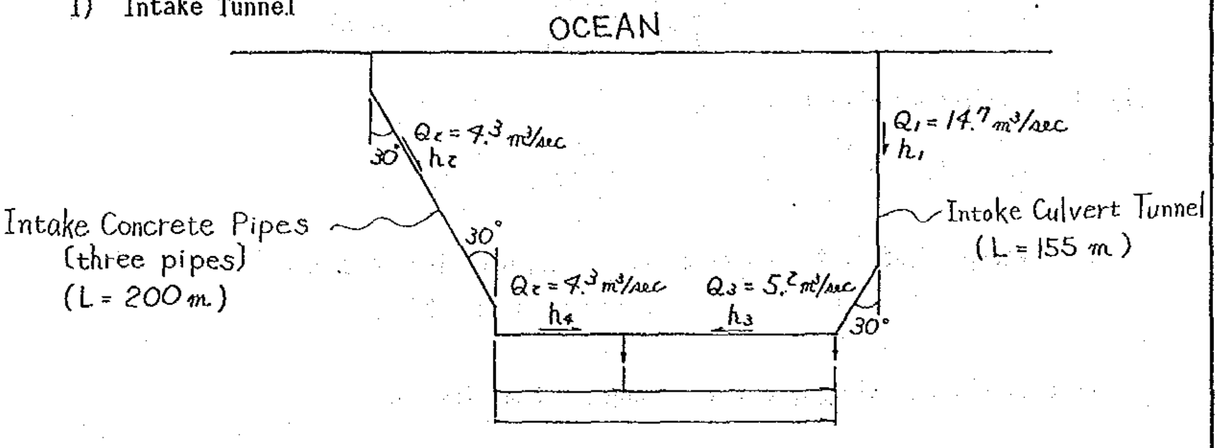


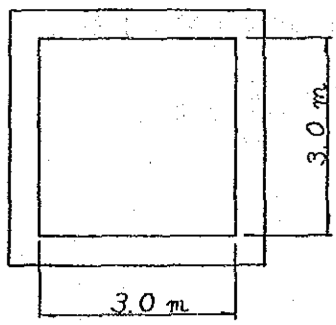
Fig 4. The water flow diagram

a) The head loss due to existing culvert h_1

The sectional dimensions S, A and R are calculated as hereinafter

Shell deposits 10cm is considered for calculations.

typical section of Culvert



S : the wet perimeter

$S = 4 \times (3.0 - 0.2) = 11.2$ m

A : the area

$A = 2.8 \times 2.8 = 7.84$ m^2

R : the hydrographic mean depth

$$R = \frac{A}{S} = \frac{7.84}{11.2} = 0.7 \text{ m}$$

i) The head loss due to inflow h_e

$$V = \frac{Q}{A} = \frac{14.7}{7.84} = 1.88 \text{ m/sec}$$

The head loss h_e

$$h_e = f_e \cdot \frac{v^2}{2g} = 0.5 \times \frac{1.88^2}{2 \times 9.8} = 0.09 \text{ m}$$

Where f_e : the coefficient of the inflow loss = 0.5

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ii) The head loss due to the wall surface friction h_f

$$h_f = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{14.7^2 \times 0.02^2 \times 155}{7.84^2 \times 0.7^{4/3}} = \underline{0.351 \text{ m}}$$

iii) The head loss due to bend h_{be}

$$h_{be} = f_{be} \cdot \frac{v^2}{2g} = 0.073 \times \frac{1.88^2}{2 \times 9.8} = \underline{0.013 \text{ m}}$$

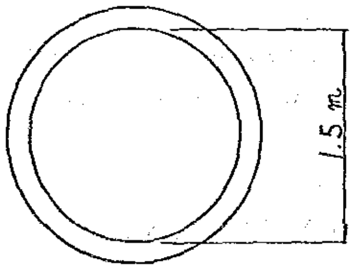
Where f_b : the coefficient of the bend loss = 0.073
(the angle of bend = 30°)

Accordingly calculating culvert tunnel's head loss h_1

$$h_1 = h_e + h_f + h_{be} = 0.09 + 0.351 + 0.013 = \underline{0.454 \text{ m}}$$

b) The head loss due to Intake pipes h_2
The sectional dimensions S, A and R are calculated as hereinafter.

the section of concrete pipe



Shell deposits 10cm is considered for calculation.

$$S = \pi \times (1.5 - 2 \times 0.1) = 4.08 \text{ m}$$

$$A = \frac{1}{4} \times \pi \times 1.3^2 = 1.33 \text{ m}^2$$

$$R = \frac{A}{S} = \frac{1.33}{4.08} = 0.326 \text{ m}$$

i) The head loss due to inflow h_e

$$V = \frac{Q}{A} = \frac{1/3 \times 4.3}{1.33} = 1.08 \text{ m/sec}$$

$$h_e = f_e \cdot \frac{v^2}{2g} = 0.5 \times \frac{1.08^2}{2 \times 9.8} = \underline{0.030 \text{ m}}$$

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ii) The head loss due to the wall surface friction h_f

$$h_f = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{1.43^2 \times 0.02^2 \times 200}{1.33^2 \times 0.326^{4/3}} = \underline{0.412\ m}$$

iii) The head loss due to bend h_{be}

$$h_{be} = f_{be} \cdot \frac{v^2}{2g} = 0.073 \times \frac{1.08^2}{2 \times 9.8} = \underline{0.004\ m}$$

Accordingly calculating the head loss h_2

$$h_2 = h_e + h_f + h_{be} = 0.03 + 0.412 + 0.004 = \underline{0.446\ m}$$

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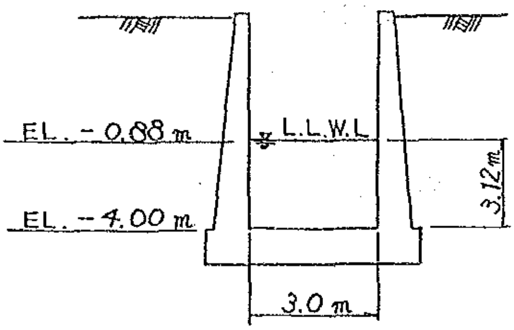
2) Intake Open Channel

a) The water head loss (at culvert side) h_3

the section of Intake Open Channel



Shell deposits 10 cm is considered for the calculations.

$S = 2 \times 3.02 + (3 - 2 \times 0.1) = 8.84$ m

$A = 2.8 \times 3.02 = 8.46$ m²

$$R = \frac{A}{S} = \frac{8.46}{8.84} = 0.96 \text{ m}$$

The head loss h_3 is the same as the head loss due to the wall surface friction.

$$h_3 = \frac{Q_3^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{5.2^2 \times 0.02^2 \times 66}{8.46^2 \times 0.96^{4/3}} = \underline{0.010 \text{ m}}$$

b) The head loss (at pipe side) h_4

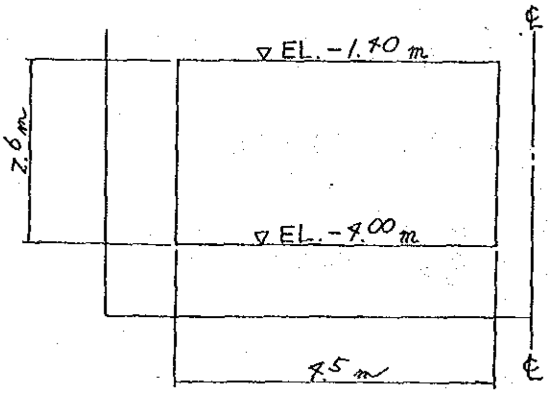
The process of calculation is the same as a).

$$h_4 = \frac{Q_3^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{4.3^2 \times 0.02^2 \times 32}{8.46^2 \times 0.96^{4/3}} = \underline{0.003 \text{ m}}$$

3) Pump Pit

a) The head loss due to curtain wall h_c

the section of the entrance



Shell deposits 10cm are considered for the calculation.

$$A = 4.3 \times 2.4 = 10.32 \text{ m}^2$$

$$Q = \frac{1}{2} \times 9.5 = 4.75 \text{ m}^3/\text{sec}$$

$$V = \frac{Q}{A} = \frac{4.75}{10.32} = 0.46 \text{ m/sec}$$

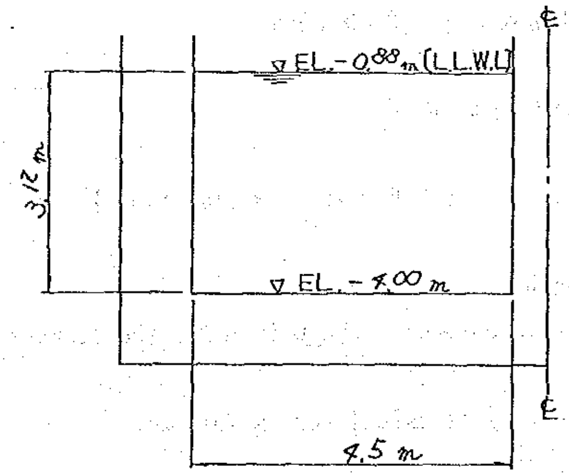
$$h_c = \frac{Q^2}{C^2 \cdot A^2 \cdot 2g} - \frac{V^2}{2g} = \frac{4.75^2}{0.5^2 \times 10.32^2 \times 2 \times 9.8} - \frac{0.46^2}{2 \times 9.8}$$

$$= 0.03 \text{ m}$$

where C : the inflow coefficient = 0.5

b) The head loss due to the wall surface friction h_f

the section of Screen Room



Shell deposits 10cm are considered for the calculation.

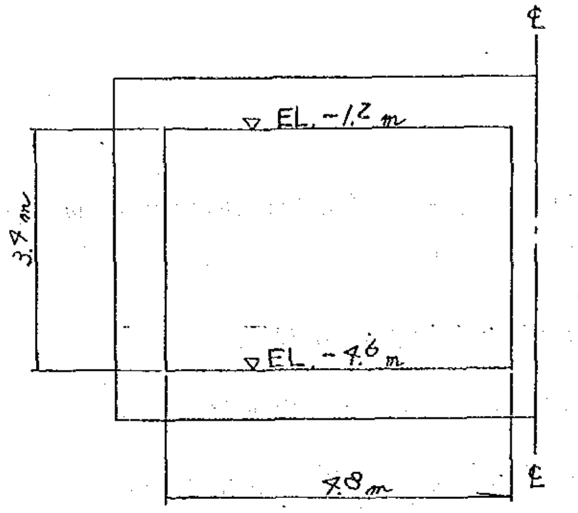
$$A = 4.3 \times 3.02 = 12.99 \text{ m}^2$$

$$S = 2 \times 3.02 + 4.3 = 10.34 \text{ m}$$

$$R = \frac{A}{S} = \frac{12.99}{10.34} = 1.26 \text{ m}$$

$$h_{f1} = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{4.75^2 \times 0.02^2 \times 13}{12.99^2 \times 1.26^{4/3}} = 0.0005 \text{ m}$$

the mean section of Connected Culvert



Shell deposits 10cm are considered for the calculation.

$$A = 4.3 \times 3.2 = 14.72 \text{ m}^2$$

$$S = 2 \times (3.2 + 4.6) = 15.6 \text{ m}$$

$$R = \frac{A}{S} = \frac{14.72}{15.6} = 0.94 \text{ m}$$

$$h_{f2} = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{4.75^2 \times 0.02^2 \times 23}{14.72^2 \times 0.94^{4/3}} = 0.001 \text{ m}$$

Accordingly h_f is calculated as follows.

$$h_f = h_{f1} + h_{f2} = 0.0005 + 0.001 = 0.0015$$

c) The head loss due to bar screen h_{ba}

The approach velocity to bar screen V_a

$$V_a = \frac{Q}{A} = \frac{4.75}{12.99} = 0.37\text{m/sec} < 0.40 \text{ m/sec}$$

Therefore the head loss h_{ba} is assumed to be 0.03 m.

(The above value 0.03 m/sec is regulated by U.S Bureau of Reclamation.)

d) The head loss due to traveling screen h_{tr}
According to Fig 5, the head loss h_{tr} is assumed to be 0.10 m for the cleaness factor 60% of screen.
Therefore the head loss due to Pump Pit h_p is calculated as follows.

$$h_p = h_c + h_f + h_{ba} + h_{tr} = 0.03 + 0.0015 + 0.03 + 0.1 = 0.162 \text{ m}$$

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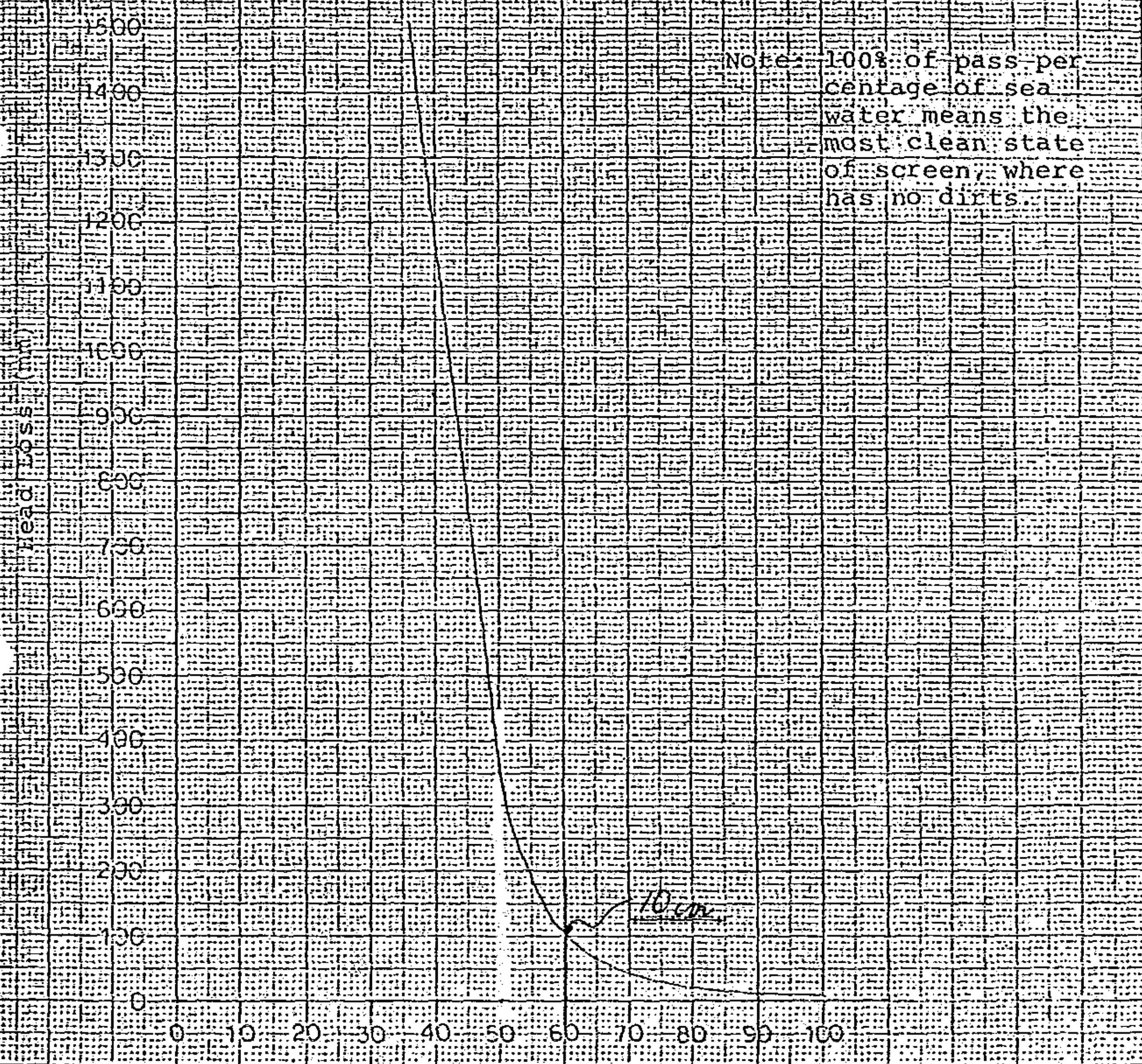
Fig.5 Travelling Screen

Pass per centage of sea water and the head loss



4) Discharge Tunnel (culvert)

The head losses due to culvert tunnels are calculated as hereinunder excluding the head loss of connected steel pipe for condenser.

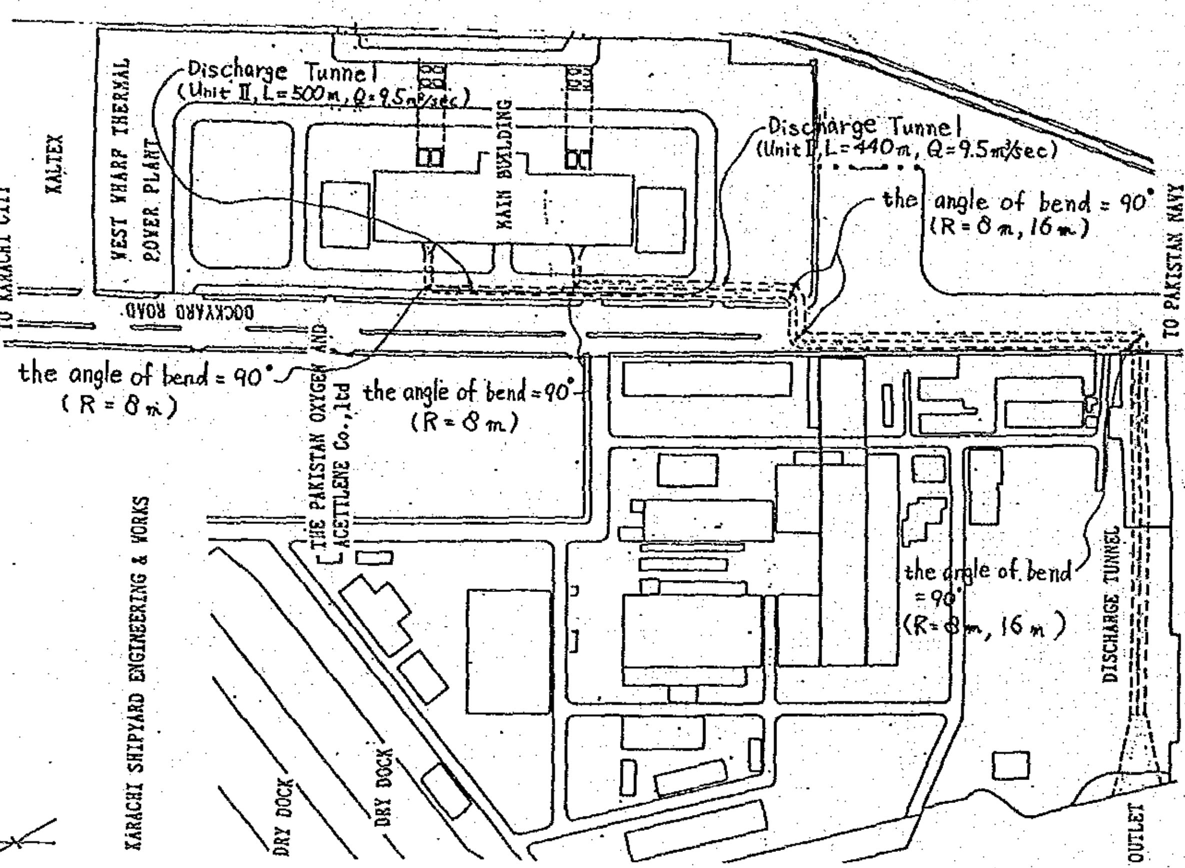
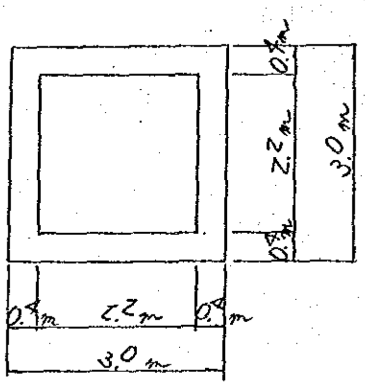


Fig 6. Plan of Discharge Tunnels

a) Discharge tunnel for Unit I

the section of discharge tunnel



$$S = 4 \times 2.2 = 8.8 \text{ m}$$

$$A = 2.2 \times 2.2 = 4.84 \text{ m}^2$$

$$R = \frac{A}{S} = \frac{4.84}{8.8} = 0.55 \text{ m}$$

$$V = \frac{9.5}{4.84} = 1.96 \text{ m/sec}$$

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i) The head loss due to the water surface friction h_f

$$h_f = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{9.5^2 \times 0.015^2 \times 440}{4.84^2 \times 0.55^{4/3}} = \underline{0.846\ \mathrm{m}}$$

ii) The head loss due to double curves of the same type h_{cu}

$$h_{cu} = 2 \times \left[\left\{0.131 + 0.1632 \left(\frac{D}{\rho_1} \right)^{3.5} \right\} \left(\frac{90°}{90°} \right) + \left\{ 0.131 + 0.1632 \left(\frac{D}{\rho_2} \right)^{3.5} \right\} \left(\frac{90°}{90°} \right) \right] \cdot \frac{V^2}{2g}$$

$$= 2 \times \left[\left\{0.131 + 0.1632 \left(\frac{2.48}{8} \right)^{3.5} \right\} \left(\frac{90°}{90°} \right) + \left\{ 0.131 + 0.1632 \left(\frac{2.48}{10.6} \right)^{3.5} \right\} \left(\frac{90°}{90°} \right) \right] \times \frac{1.96^2}{2 \times 9.8}$$

$$= \underline{0.104\ \mathrm{m}}$$

Where ρ : the radius of curvature $\rho_1 = 8$ m $\rho_2 = 10.6$ m

D : the diameter of culvent

$$D = \sqrt{\frac{4.84 \times 4}{\pi}} = 2.48\ \mathrm{m}$$

Accordingly the head loss due to discharge tunnel for unit I h_{c1} is calculated as hereinafter.

$$h_{c1} = h_f + h_{cu} = 0.846 + 0.104 = \underline{0.950\ \mathrm{m}}$$

b) Discharge tunnel for unit II

The section of culvert and the process of calculations are the same as culvert I.

i) The head loss due to the water surface friction h_f

$$h_f = \frac{9.5^2 \times 0.015^2 \times 500}{4.84^2 \times 0.55^{4/3}} = \underline{0.962\ \mathrm{m}}$$

ii) The head loss due to curve h'_{cu}

h'_{cu} is calculated as hereinafter.

$$h'_{cu} = \left[3 \times \left\{ 0.131 + 0.1632 \times \left(\frac{2.48}{8} \right)^{3.5} \times \left(\frac{90°}{90°} \right) \right. \right.$$

$$\left. \left. + \left\{ 0.131 + 0.1632 \times \left(\frac{2.48}{10.6} \right)^{3.5} \times \left(\frac{90°}{90°} \right) \right\} \right] \times \frac{1.96^2}{2 \times 9.8}$$

$$= \underline{0.104\ \mathrm{m}}$$

Accordingly the head loss due to culvert II h_{c2} is calculated as hereinafter.

$$h_{c2} = h_f + h_{cu} = 0.962 + 0.104 = \underline{1.066\ \mathrm{m}}$$

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5) Outlet

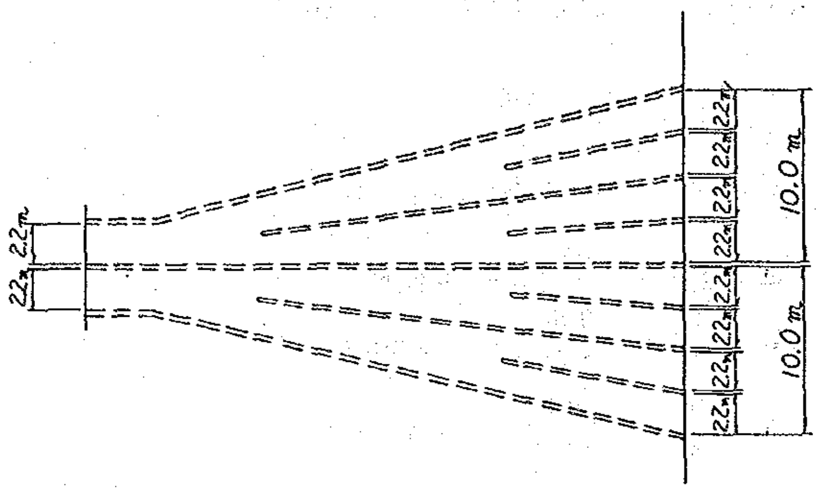
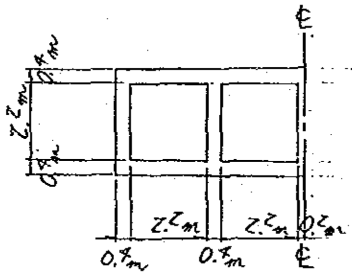


Fig 7. Plan of outlet

Both outlets of unit I and unit II are the same structures, so the head loss calculation is all the same.

a) The head loss due to outlet h_o (per one culvert box)
typical section of outlet (one unit)



$S = 4 \times 2.2 = 8.8$ m (per one box)

$A = 2.2 \times 2.2 = 4.84$ m² (per one box)

$$R = \frac{A}{S} = \frac{4.84}{8.8} = 0.55 \text{ m} \quad \text{(per one box)}$$

$$V = \frac{Q}{A} = \frac{1/2 \times 9.5}{4.84} = 0.98 \text{ m/sec} \quad \text{(per one box)}$$

i) The head loss due to the water surface friction h_f

$$h_f = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}} = \frac{4.75^2 \times 0.015^2 \times 40}{4.84^2 \times 0.55^{4/3}} = \underline{0.019 \text{ m}}$$

ii) The head loss due to the gradual expansion h_g

The entrance area of outlet A_1 (one unit)

$$A_1 = 2.2 \times 2.2 = 4.84 \text{ m}^2$$

The exit area of outlet A_2 (one unit)

$$A_2 = 4 \times 2.2 \times 2.2 = 19.36 \text{ m}^2$$

$$V_1 = \frac{Q_1}{A_1} = \frac{9.5}{4.84} = 1.96 \text{ m/sec}$$

$$V_2 = \frac{9.5}{A_2} = \frac{9.5}{19.36} = 0.49 \text{ m/sec}$$

$$h_g = f_g \cdot f_s \cdot \frac{(V_1 - V_2)^2}{2g} = 0.7 \times 0.563 \times \frac{(1.96 - 0.49)^2}{2 \times 9.8} = \underline{0.043 \text{ m}}$$

Where f_g : the coefficent of the gradual expansion loss = 0.7

f_s : the coefficent of the sudden expansion loss

$$f_s = \left(1 - \frac{A_1}{A_2}\right)^2 = \left(1 - \frac{4.84}{19.36}\right)^2 = 0.563$$

Accordingly the head loss due to outlet h_o is calculated as hereinafter.

$$h_o = h_f + h_g = 0.019 + 0.043 = \underline{0.062 \text{ m}}$$

CV-4 STRUCTURAL CALCULATION OF INTAKE OPEN CHANNEL

Contents of this calculation note is shown as below.

Contents

1. Soil Condition 2

2. Outline of The Structural Design 4

3. Stability Calculation 5

4. Design Case 11

5. Structural Design 12

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1. Structural Calculation of Intake Open Channel

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 conjectured by the following calculation.

$$N = \frac{(10+12)\times 2.0+(12+11)\times 2.0+(11+13)\times 2.0+(13+6)\times 2.0+(6+9)\times 1.54}{2\times 9.54}$$

$$\fallingdotseq 10$$

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

$$\phi = (\sqrt{15N}+15)^\circ = (\sqrt{15\times 10}+15)^\circ \fallingdotseq 27^\circ$$

the bulk density of soil above the ground water γ : $\gamma = 1.9\ t/m^3$
the bulk density of soil under the ground water γ^- : $\gamma^- = 1.0\ t/m^3$.

Moreover other basic condition data are published in "Civil Design Condition" (vid.No EWC-1001).

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SITE : WEST WHARF THERMAL POWER STATION, KARACHI.

CLIENT : KARACHI ELECTRIC SUPPLY CORPORATION.

BORE CHART OF BORING No.4

Date	Ground Water Level	Reduced Levels M/s	Depth METERS	Thickness of Layers in (Meter)	STRATAENCOUNTERED	Soil Symbols	Diameter of Boring	Penetration Test Blow No. (20 40 60 80 10)
	EL +4.538		0.00 τ =1.9		The ground water surface			
20/12/87	1.0 (m)	+3.538 +0.838	r'=1.0 3.70	3.70	filling material of brown medium to coarse sand with gravels, pebbles and clay.			12 blows @ 2m.
		-1.262	τ'=1.0 5.80	2.10	Brown medium silty medium to coarse SAND.			11 blows @ 4m.
		-5.000 -5.462	τ'=1.0 10.00	4.20	Dark grey loose to medium silty fine to coarse SAND, traces GRAVELS. Foundation Level		5"ø	13 blows @ 6m. 6 blows @8m.
27/12/87		-11.462	τ'=1.0 16.00	6.00	Grey loose to medium silty fine to medium SAND.			9 blows @ 10m. 22 blows @ 12m. 24 blows @ 14m. 27 blows@ 16m.
			τ'=1.0					

cont. on sheet 2.

Remarks : GROUNDWATER TABLE AT 1.00 METER.

Date :- 27/12/87.

Fig 1. Soil column diagram

FORM 04

1.2 Outline of The Structural Design

Structure of Intake Open Channel is shown in Fig.2.
This structure is considered as the rigid frame excluding the beam.

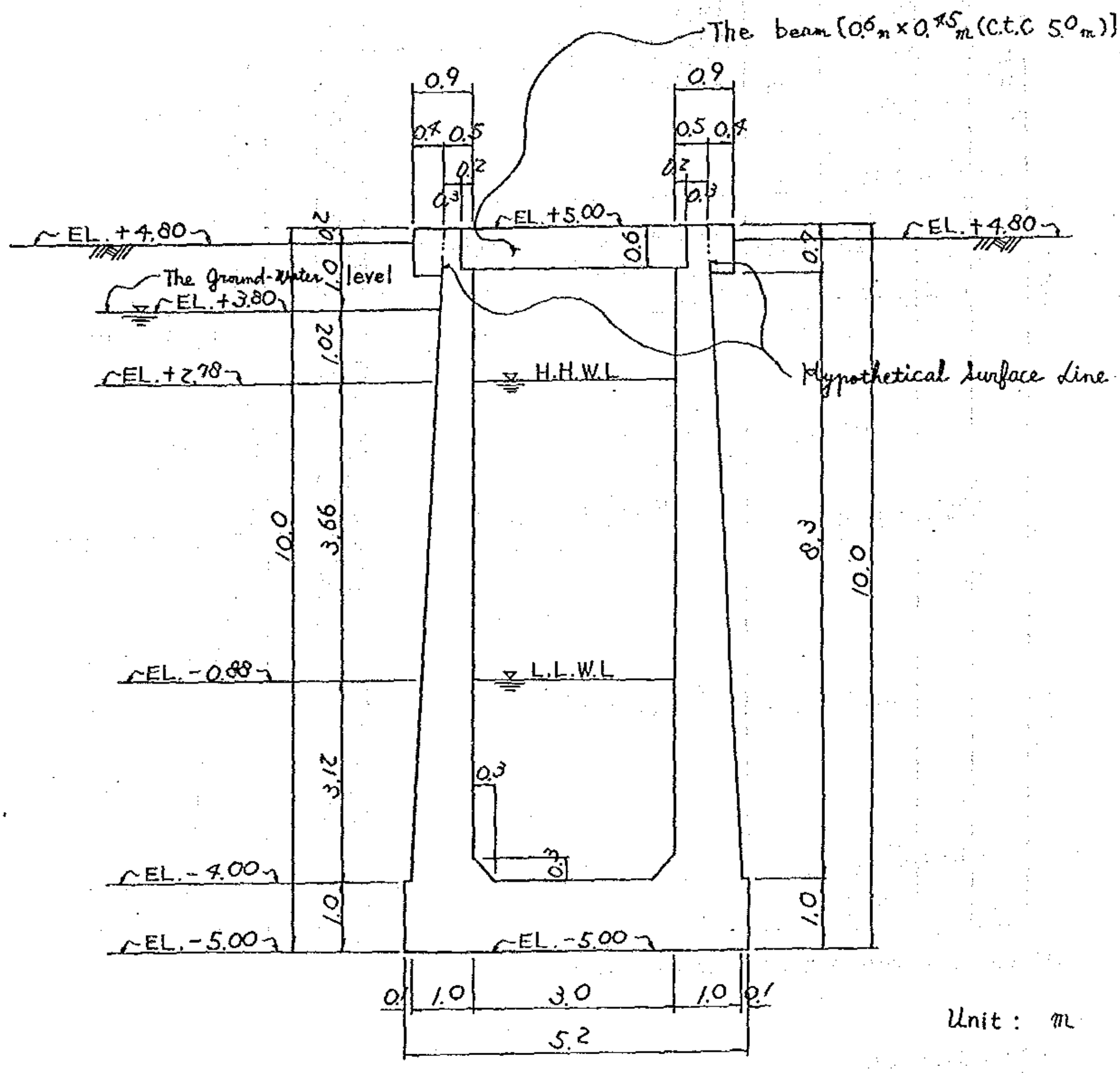

Fig.2 Typical section of Intake Open Channel

The structural design calculation is executed at the lowest low water level.
(EL − 0.88 m) (considered for the water head loss due to Intake Tunnel).
↓
(vid.1.c,P4,No. EWC-1004)

1.3 Stability Calculation

1) Vertical forces (per 1 m unit length)

a) Base slab

$$W_{c1} = 5.2 \times 1.0 \times 2.45 = 12.7 \text{ t}$$

b) Side wall (at one side)

$$W_{c2} = \{0.7 \times 0.7 + \frac{1}{2} \times (0.54+1.0) \times 8.3 + \frac{1}{2} \times 0.3 \times 0.3\} \times 2.45$$

$$= 17.0 \text{ t}$$

c) Beam

$$W_{c3} = 3.0 \times 0.6 \times 0.45 \times 2.45 \div 5.0 = 0.4 \text{ t}$$

d) Soil weight W_s (at one side)

$$W_s = \{0.2 \times 0.5 + \frac{1}{2} \times (0.56 + 0.1) \times 8.3\} \times 1.9 = 5.4 \text{ t}$$

e) Water weight

Water weight W_w is calculated at the highest high water level (EL + 2.78 m)
(considered for the water head loss due to Intake Tunnel.) ↓
(vid.1.b,P3,No EWC-1004)

$$W_w = (6.78 \times 3.0 - 0.3 \times 0.3) \times 1.0 = 20.3 \text{ t}$$

f) Buoyancy

Buoyancy U_b is working to the portion of concrete structure under H.H.W.L, so U_b is calculated as follows.

$$U_b = \{ 5.2 \times 1.0 + 0.3 \times 0.3 + (0.59 + 1.0) \times 6.78 \} \times 1.0$$

$$= 16.1 \text{ t}$$

FORM 04

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According to the above calculation, the summarized vertical forces are shown in Table 1.

Table 1. Summarized table of vertical forces (per 1 m unit length)

Species	Vertical force	Remarks
W_{c1}	12.7 t	Base slab
W_{c2}	34.6 t	Side wall
W_{c3}	0.4 t	Connected beam
W_s	11.0 t	Soil weight
W_w	20.3 t	Water weight
U_b	− 16.1 t	Buoyancy
TOTAL	V = 62.9 t	

2) The ground reaction q_1

$$q_1 = \frac{V}{B.L} \cdot \left(1 \pm \frac{6e}{B}\right)$$

Where e : the eccentric distance
e = 0

$$= \frac{62.9}{5.2 \times 1.0}$$

$$= 12.1 \ t/m^2$$

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3) Study of the bearing capacity q_u

The ultimate bearing capacity is calculated by the following equation in consideration of the unbalanced inclination.

$$q_u = \alpha K C N_c + K q N_q + \frac{1}{2} \gamma_1 \beta B' N_r$$

Where C : soil cohesion = 0 t/m²

q : the surcharge load

$q = 1.9 \times 1.0 + 1.0 \times 8.8 = 10.7$ t/m²

A' : the effective load area (m²)

$A' = 5.2 \times 1.0 = 5.2$ m²

γ_1 : the bulk density of the bearing soil(t/m³)

$\gamma_1 = 1.0$ t/m³

(γ_1 is the bulk density under the ground water.)

B' : the effective width considered for the eccentric distance.

$B' = B - 2e_b = 5.2 - 2 \times 0 = 5.2$ m

where B : the basic width

C_b : the eccentric distance

$\alpha . \beta$: the coefficient of basic form $\alpha = \beta = 1.0$

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

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

$N_c = 25$ (from Fig 3.)
$N_q = 14$ (from Fig 4.)
$N_r = 9$ (from Fig 5.)

FORM 04

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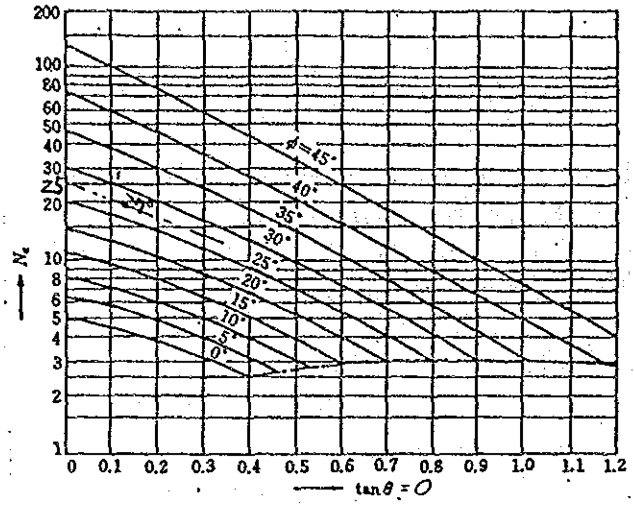

Fig.3 Graph of the bearing coefficient N_c

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

V : Vertical Force at the foundation V = 60 t

H : Horizontal Force at the foundation H = 0 t

because horizontal forces are equilibriumed at both sides.

ϕ : The angle of the internal friction

$\phi = 27°$

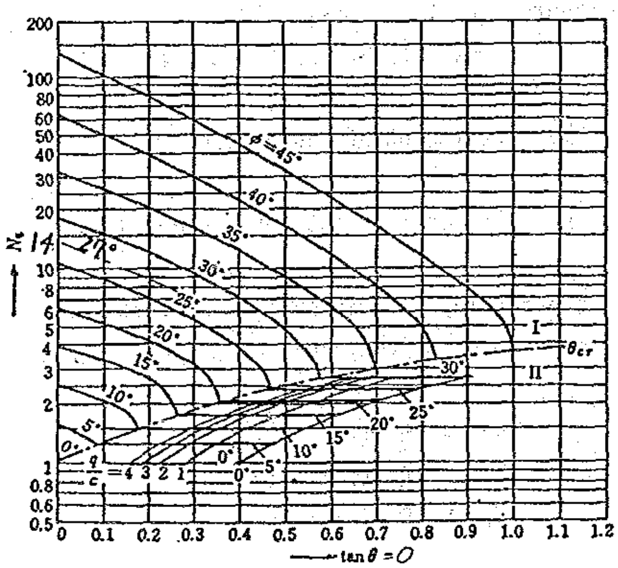

Fig.4 Graph of the bearing coefficient N_q

1106

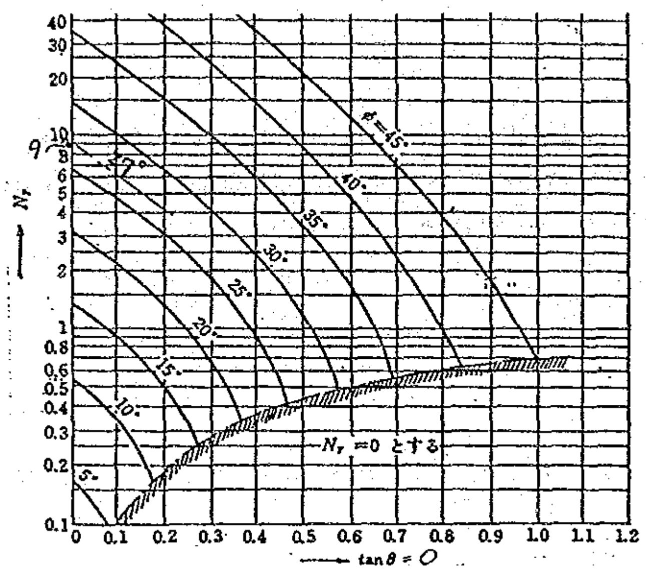

Fig.5 Graph of the bearing coefficient N_r

Accordingly the ultimate bearing capacity q_u is calculated as below.

$$q_u = 1.0 \times 10.7 \times 14 + \frac{1}{2} \times 1.0 \times 1.0 \times 5.2 \times 9 = 173\ t$$

4) The allowable bearing capacity q_a

The allowable bearing capacity is calculated as below.

$$q_a = \frac{1}{F_s} \cdot q_u = \frac{1}{3} \times 173 \fallingdotseq 57\ t/m^2 \underset{O.K}{>} q_1 = 11.5\ t/m^2$$

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

Therefore the spread foundation is adopted for the foundation of Intake Open Channel.

5) Study of floating

The calculation of floating is executed at normal (The internal water condition is at L.L.W.L) and at construction (The internal water condition is empty), so this calculation is as follows.

a) Total vertical force

i) at normal

$$V_1 = 12.7 + 34.6 + 0.4 + 11.0 + (3.12 \times 3.0 - 0.3 \times 0.3) \times 1.0 = 68.0 \text{ t}$$

ii) at construction

$$V_2 = 12.7 + 34.6 + 0.4 + 11.0 = 58.7 \text{ t}$$

b) Up lift U

Up lift U is calculated as below.

$$U = \gamma_w \cdot h_w \cdot A = 1.0 \times 8.8 \times 5.2 \times 1.0 = 45.8 \text{ t}$$

c) Checking on the safety factor of floating F_1

The safety factor of floating F_1 is checked by the follwing two cases.

i) at normal

$$F_{11} = \frac{V_1}{U} = \frac{68.0}{45.8} = 1.48 \underset{O.K}{>} 1.1$$

ii) at construction

$$F_{12} = \frac{V_2}{U} = \frac{58.7}{45.8} = 1.28 \underset{O.K}{>} 1.0$$

1.4 Design Case

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

Table 2. Summary of design cases

CASE	1	2	3
CONDITION	at Normal	at Earthquake	at Construction
Earthquake	None	Considered	None
Water Content	L.L.W.L	L.L.W.L	Empty
Self weight	Considered	Considered	Considered
The surcharge load	Considered	Considered	Considered
The increment ratio of the allowable stress	1.0	1.5	1.25

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1.5 Structural Design

1.5.1 Load Case-1 (Normal, Long term)

1) Frame of the design structure

Frame of the design structure is shown in Fig 6, and the design structure is considered for the rigid frame structure.

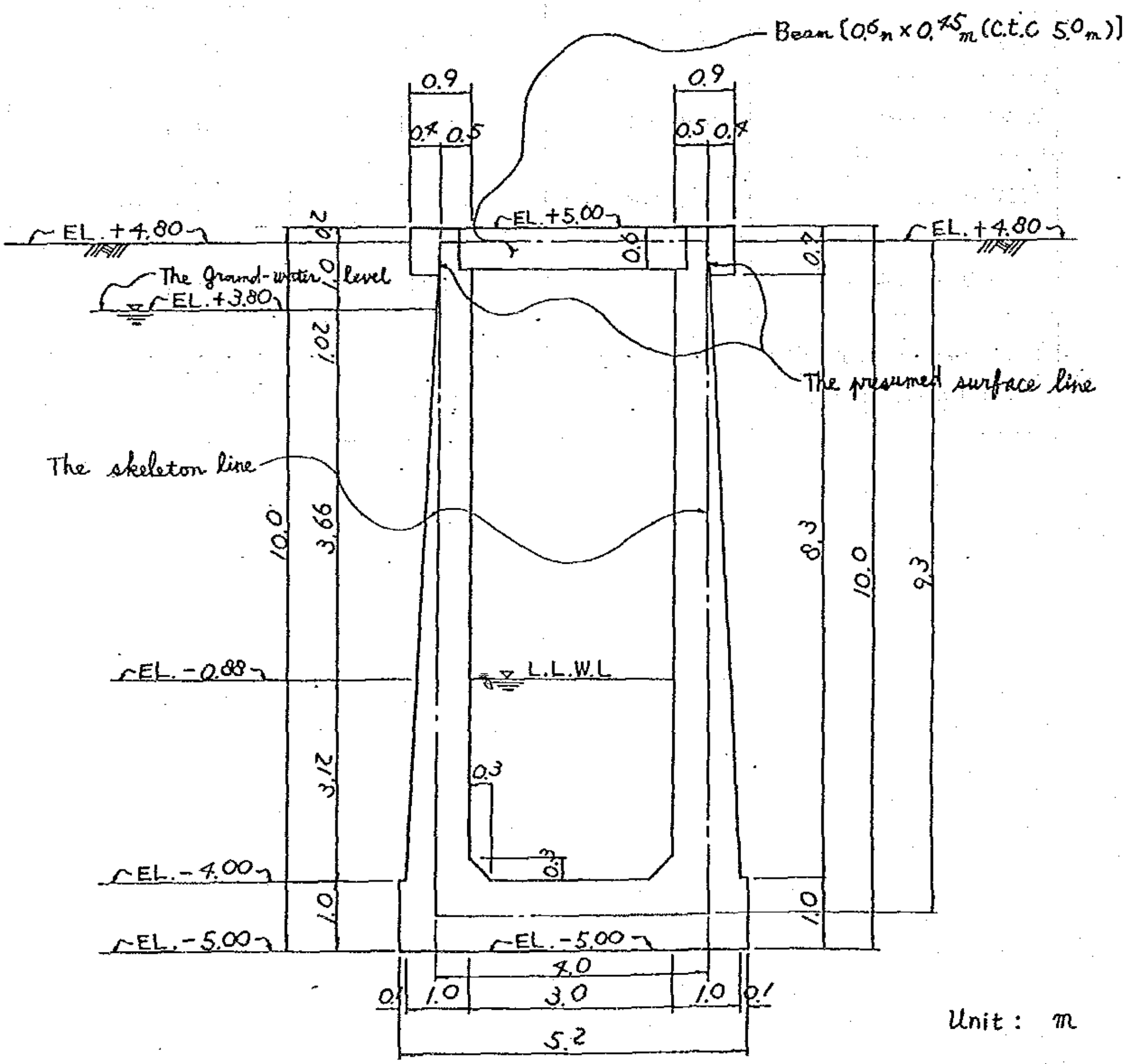

Fig 6. Frame of the design structure

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2) Load calculation (per 1m unit length)

a) Surcharge load at the ground level q $\qquad q = 1.0\ t/m^2$

b) Self weight

i) Side wall W_{c1} (at one side)

$$W_{c1} = \frac{1}{2} \times (0.5 + 1.0) \times 2.45 = 1.84\ t/m^2$$

ii) Base slab W_{c2} (at one side)

$$W_{c2} = 1.0 \times 2.45 = 2.45\ t/m^2$$

iii) Beam

$$W_{c3} = 0.6 \times 0.45 \times 2.45 \div 5.0 = 0.13\ t/m^2$$

c) The earth pressure and the water pressure

i) the earth pressures

$$P_0 = k \cdot q = 0.5 \times 1.0 = 0.5\ t/m^2$$

$$P_1 = k(\gamma h_1 + q) = 0.5 \times (1.9 \times 1.0 + 1.0) = 1.45\ t/m^2$$

$$P_2 = 1.45 + k \cdot \gamma \cdot h_2 = 1.45 + 0.5 \times 1.0 \times 8.3 = 5.60\ t/m^2$$

ii) the water pressure (outside)

$$P_{w0} = \gamma_w \cdot h_{w0} = 1.0 \times 8.3 = 8.3\ t.m^2$$

iii) the water pressure (inside)

$$P_{wi} = \gamma_w \cdot h_{wi} = 1.0 \times 3.12 = 3.12\ t/m^2$$

iv) up lift

$$P_u = \gamma_w \cdot H_w = 1.0 \times 8.8 = 8.8 \text{ t/m}^2$$

d) The ground reaction

$$q_r = 11.5 \text{ t/m}^2$$

According to the above calculations, the results of load calculations are shown in Fig 7.

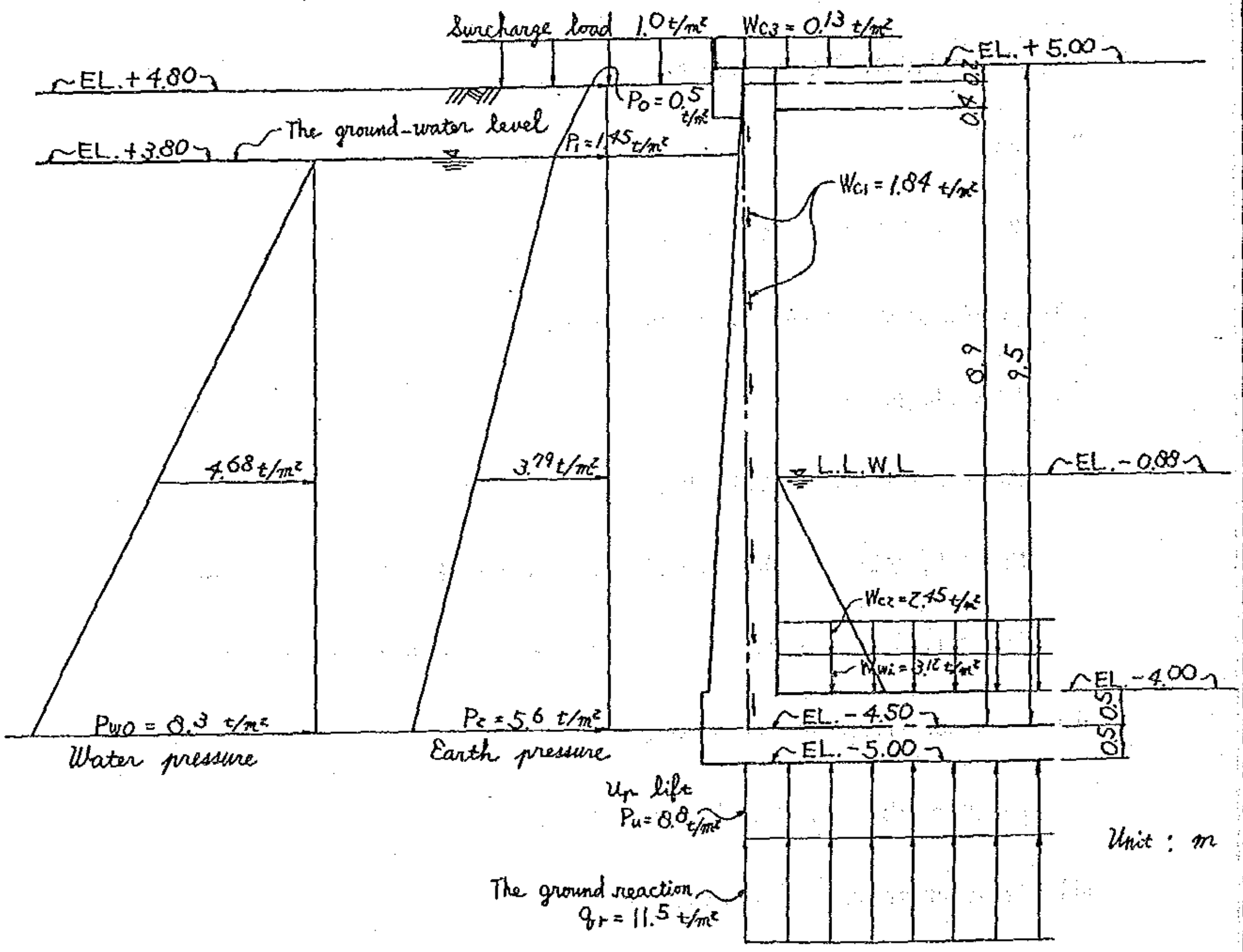

Fig.7 The results of load calculations

3) The load diagram

The load diagram is shown in Fig 8.

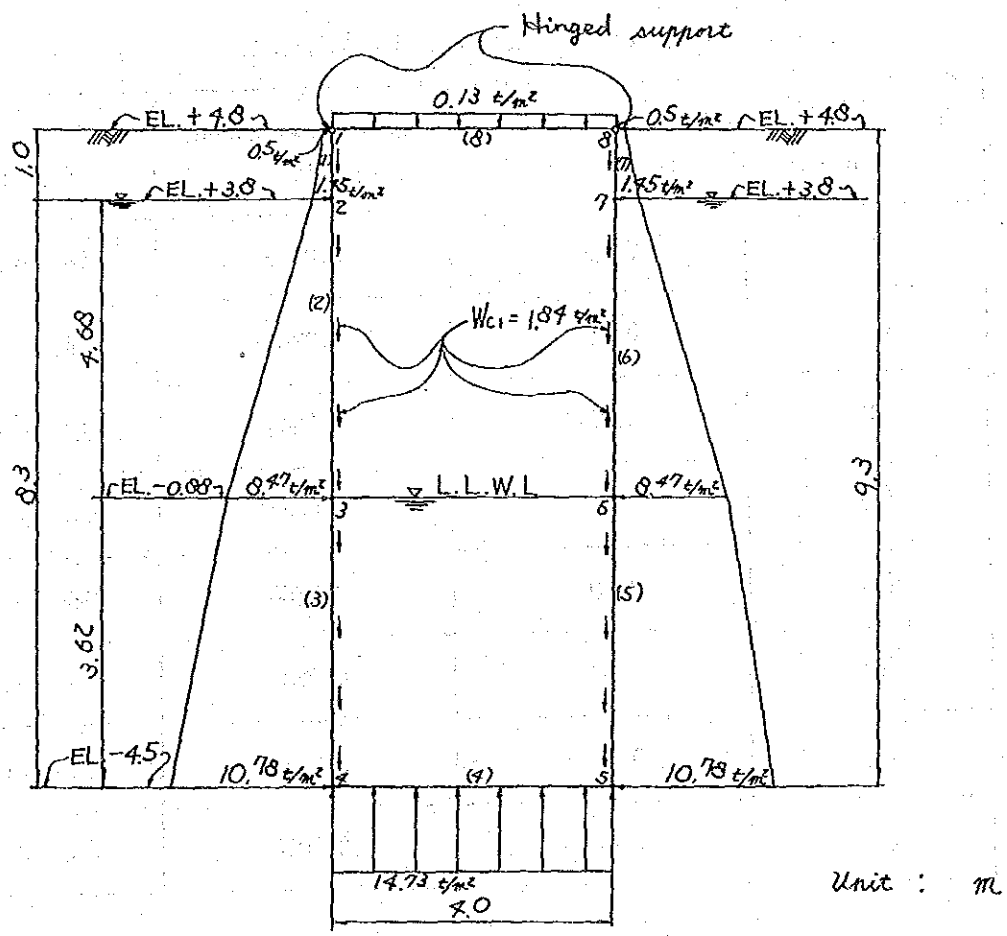

Fig.8 The load diagram

4) Imput data for the sectional dimensions.

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

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

Member's number	Section area A (m^2)	Geometrical moment of inertia I(m^4)	Remarks
(1)	0.53	0.0129	Side wall
(2)	0.68	0.0262	Side wall
(3)	0.90	0.0608	Side wall
(4)	1.0	0.0833	Footing
(5)	0.90	0.0608	Side wall
(6)	0.68	0.0262	Side wall
(7)	0.53	0.0129	Side wall
(8)	0.054	0.0016	beam @ 5.0 m

1.5.2 Load Case-2 (Earthquake, Short term)

1) Design structure

Design structure is shown in Fig 9.

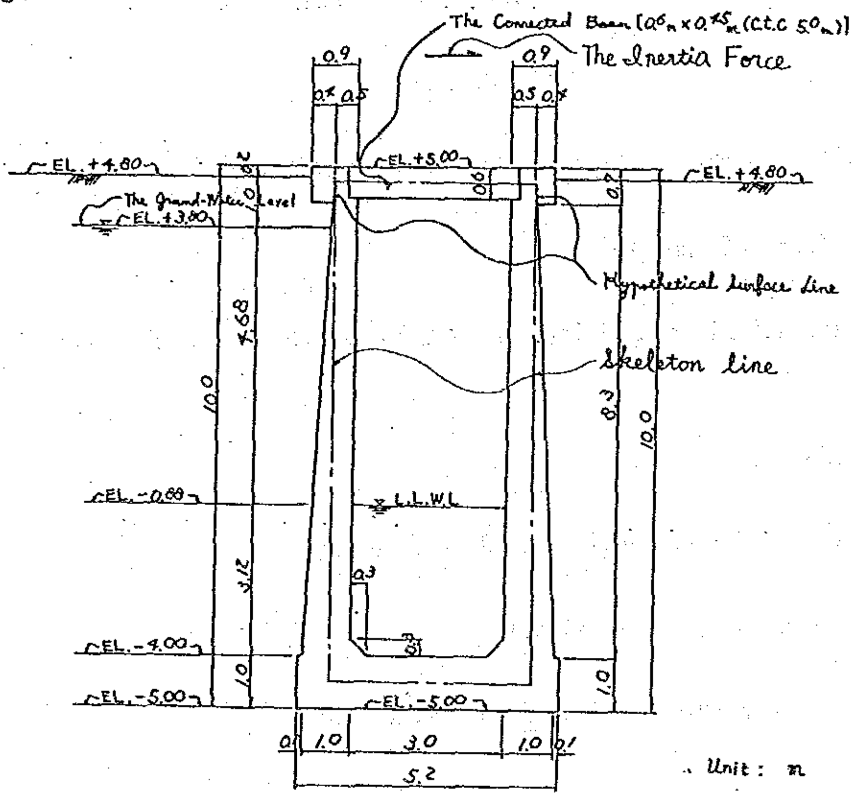

Fig.9 Design structure

2) Load calculation (per 1 m unit length)

a) The ground surface surcharge

$q = 1.0\ t/m^2$

b) Self weight

i) side wall $W_{c1} = 1.84\ t/m^2$ (at one side)

ii) base slab $W_{c2} = 2.45\ t/m^2$ (at one side)

iii) beam $W_{c3} = 0.13\ t/m^2$

iv) water weight $W_w = 1.0 \times 3.12 = 3.12\ t/m^2$

c) The force of inertia

Design seismic coefficient $k_h = 0.10$, so the force of inertia is calculated as below.

i) side wall (at one side)

$$W_{ch1} = k_h \cdot W_{c1} = 0.1 \times 1.84 = 0.18\ t/m^2$$

ii) base slab (at one side)

$$W_{ch2} = k_h \cdot W_{c2} = 0.1 \times 2.45 = 0.25\ t/m^2$$

iii) beam

$$W_{ch3} = k_h \cdot W_{c3} = 0.1 \times 0.13 = 0.01\ t/m^2$$

d) The seismic earth pressures and the water pressures

i) the seismic earth pressures

The coefficient of seismic active pressure is calculated by the following equations.

$$Kea \fallingdotseq \frac{\cos^2(\phi - \theta_0 - \theta)}{\cos\theta_0 \cdot \cos^2\theta \cdot \cos(\theta + \theta_0 + \delta)\left\{1 + \sqrt{\frac{\sin(\phi+\delta)\ \sin(\phi-\delta-\theta_0)}{\cos(\theta+\theta_0+\delta)\ \cos(\theta-\alpha)}}\right\}^2}$$

Where ϕ : the angle of the internal friction = 27°

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

$$\theta = \tan^{-1}\frac{0.5}{8.97} \fallingdotseq 3°$$

δ : the angle of the wall surface friction between the back wall surface and back-fill $\delta = 0°$

θ_0 : the composite angle of earthquake

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

Therefore

$$Kea = \frac{\cos^2(27° - 5.7° - 3°)}{\cos5.7° \times \cos^2 3° \times \cos(3° + 5.7°) \times \left\{1 + \sqrt{\frac{\sin27° \cdot \sin(27° - 5.7°)}{\cos(3° + 5.7°) \cdot \cos3°}}\right\}^2}$$

$$= 0.463$$

The seismic passive earth pressure is determined by balancing the force which is figured out by the inertia forces and the seismic active earth pressures.
Accordingly the seismic earth pressures are calculated as follows.

ii) the active earth pressures (Horizontal)

(Horizontal)

$P_{EAh0} = K_{EA} \cdot \cos 3° \cdot q = 0.463 \times 0.999 \times 1.0 = 0.46\ t/m^2$

$P_{EAh1} = 0.463 \times 0.999 \times (1.0 + 1.9 \times 1.0) = 1.34\ t/m^2$

$P_{EAh2} = 0.463 \times 0.999 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 8.3) = 5.18\ t/m^2$

(Vertical)

$P_{EAv0} = K_{EA} \cdot \sin 3° \cdot q = 0.463 \times 0.052 \times 1.0 = 0.02\ t/m^2$

$P_{EAv1} = 0.463 \times 0.052 \times (1.0 + 1.9 \times 1.0) = 0.07\ t/m^2$

$P_{EAv2} = 0.463 \times 0.052 \times (1.0 + 1.9 \times 1.0 + 1.0 \times 8.3) = 0.27\ t/m^2$

iii) the passive earth pressures

Total passive pressures are balanced to total horizontal forces, so they are calculated as below.

(Horizontal)	(Vertical)
$P_{EPh0} = 0.95$ t/m2	$P_{EPv0} = P_{EPh0} \cdot \tan 3° = 0.95 \times 0.052 = 0.05\ t/m^2$
$P_{EPh1} = 1.70\ t/m^2$	$P_{EPv1} = P_{EPv1} \cdot \tan 3° = 1.70 \times 0.052 = 0.09\ t/m^2$
$P_{EPh2} = 5.78\ t/m^2$	$P_{EPv2} = P_{EPv2} \cdot \tan 3° = 5.78 \times 0.052 = 0.30\ t/m^2$

iv) the static water pressure

The static water pressures are calculated by dividing between inside and outside.

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(Inside)

$$P_{wi} = \gamma_w \cdot h_i = 1.0 \times 3.12 = 3.12\ t/m^2$$

(Outside)

$$P_{we} = \gamma_w \cdot h_e = 1.0 \times 8.3 = 8.3\ t/m^2$$

v) the dynamic water pressure

The dynamic water pressure is considered for working to side walls, therefore dynamic water pressure is calculated as below.

$$P_{ew} = \frac{7}{12} \cdot K_h \cdot \gamma_w \cdot b \cdot h^2 = \frac{7}{12} \times 0.1 \times 1.0 \times 1.0 \times 3.12^2 = 0.57\ t$$

Working point of Pew (h_{ew})

$$h_{ew} = \frac{h}{2} + 0.5 = \frac{3.12}{2} + 0.5 = 2.06\ m$$

vi) up lift

$$P_u = \gamma w \cdot H_w = 1.0 \times (8.3 + 0.5) = 8.8\ t/m^2$$

e) The ground reaction

$$q_r = 11.5\ t/m_2$$

f) The results of load calculations

The results of load calculations are shown in Fig 10.

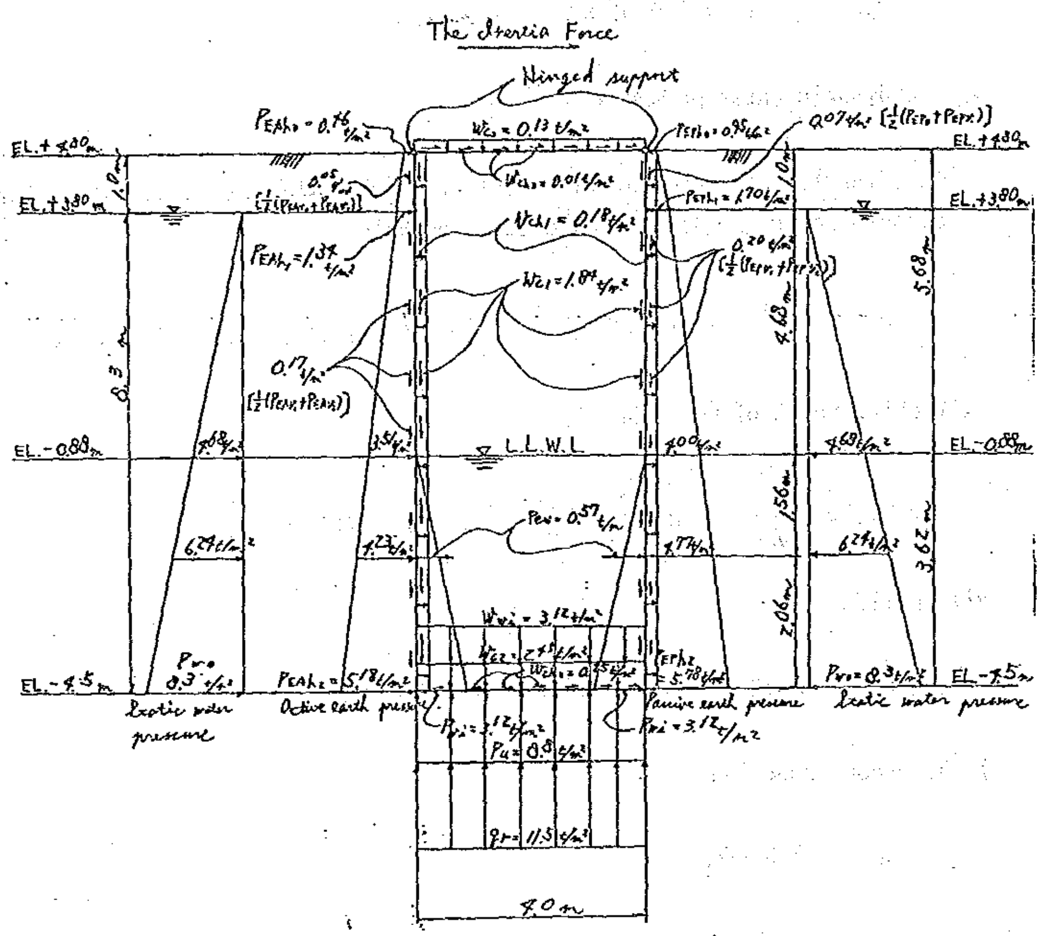

Fig.10 The results of load calculations

3) The load diagram

The load diagram is shown in Fig 11.

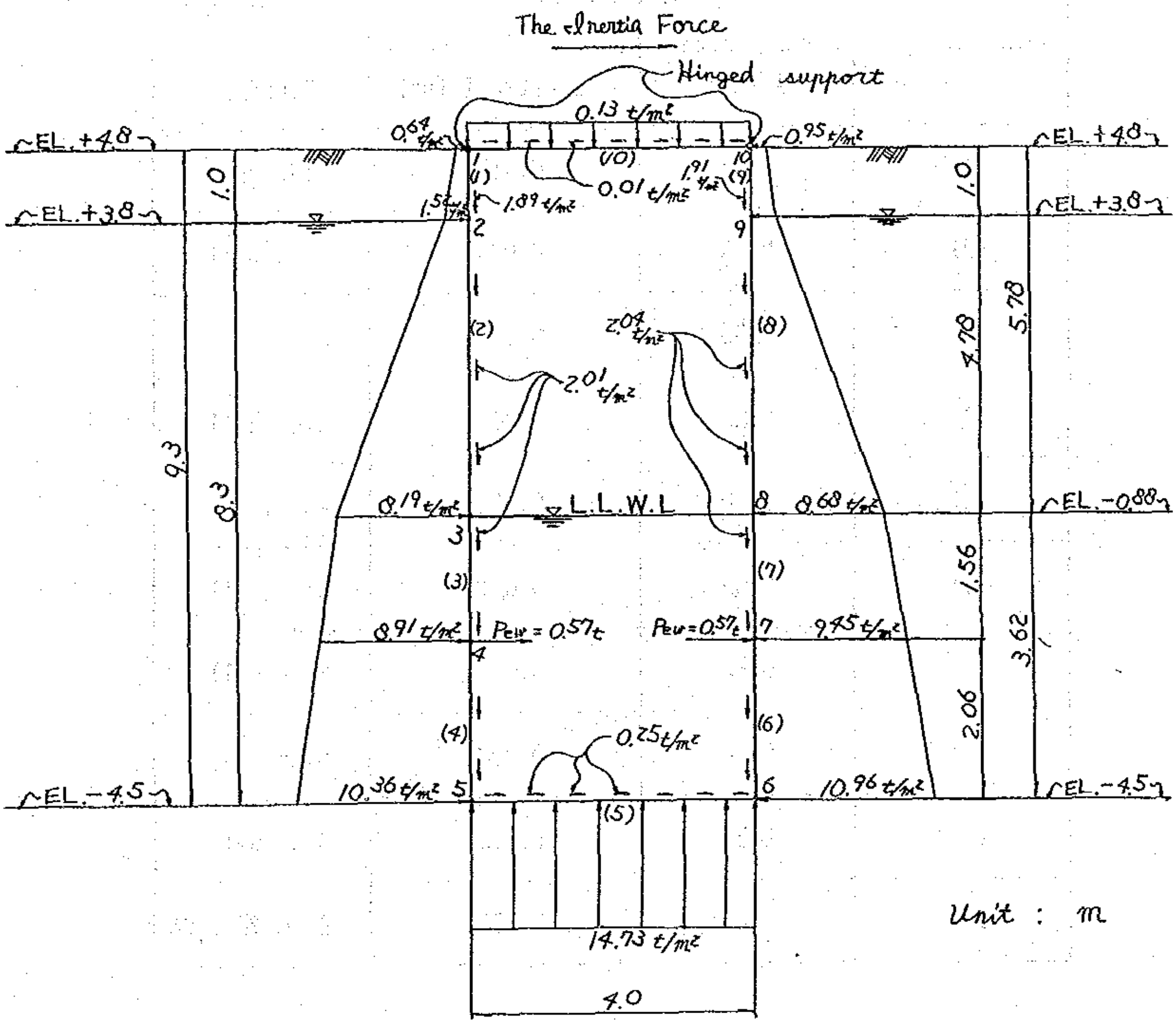

Fig.11 The load diagram

4) Input data for the sectional dimensions

a) The sectional dimensions

Table 4. Table of the sectional dimensions (per 1 m unit length)

Member's number	Section Area A (m^2)	Geometrical moment of inertia I (m^4)	Remarks
(1)	0.53	0.013	Side wall
(2)	0.68	0.026	Side wall
(3)	0.85	0.051	Side wall
(4)	0.95	0.071	Side wall
(5)	1.00	0.083	Footing
(6)	0.95	0.071	Side wall
(7)	0.85	0.051	Side wall
(8)	0.68	0.026	Side wall
(9)	0.53	0.013	Side wall
(10)	0.054	0.0016	Beam @ 5.0m

1.5.3 Load Case (at construction, Short term)

1) Design structure

Design structure is shown in Fig 12.

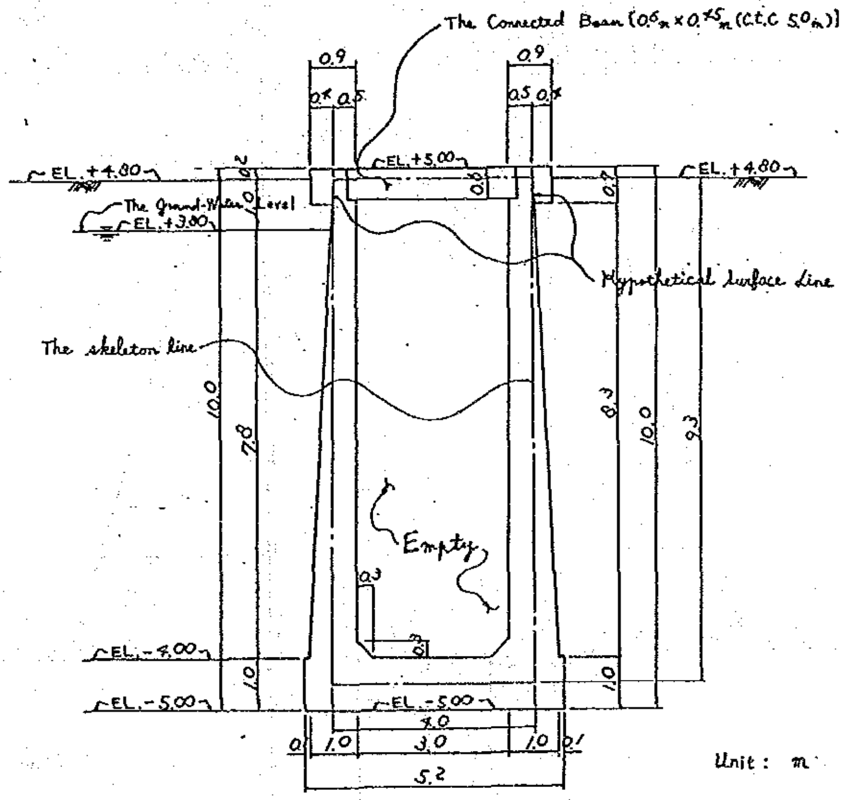

Fig 12. Design structure

2) Load calculation (per 1 m unit length)

The process and the results of load calculations are the same as Case−1 excluding a calculation of inside water load.
In this design case the inside water pressure and the inside water weight is no considered(= 0).

3) The load diagram

The load diagram is shown in Fig 13.

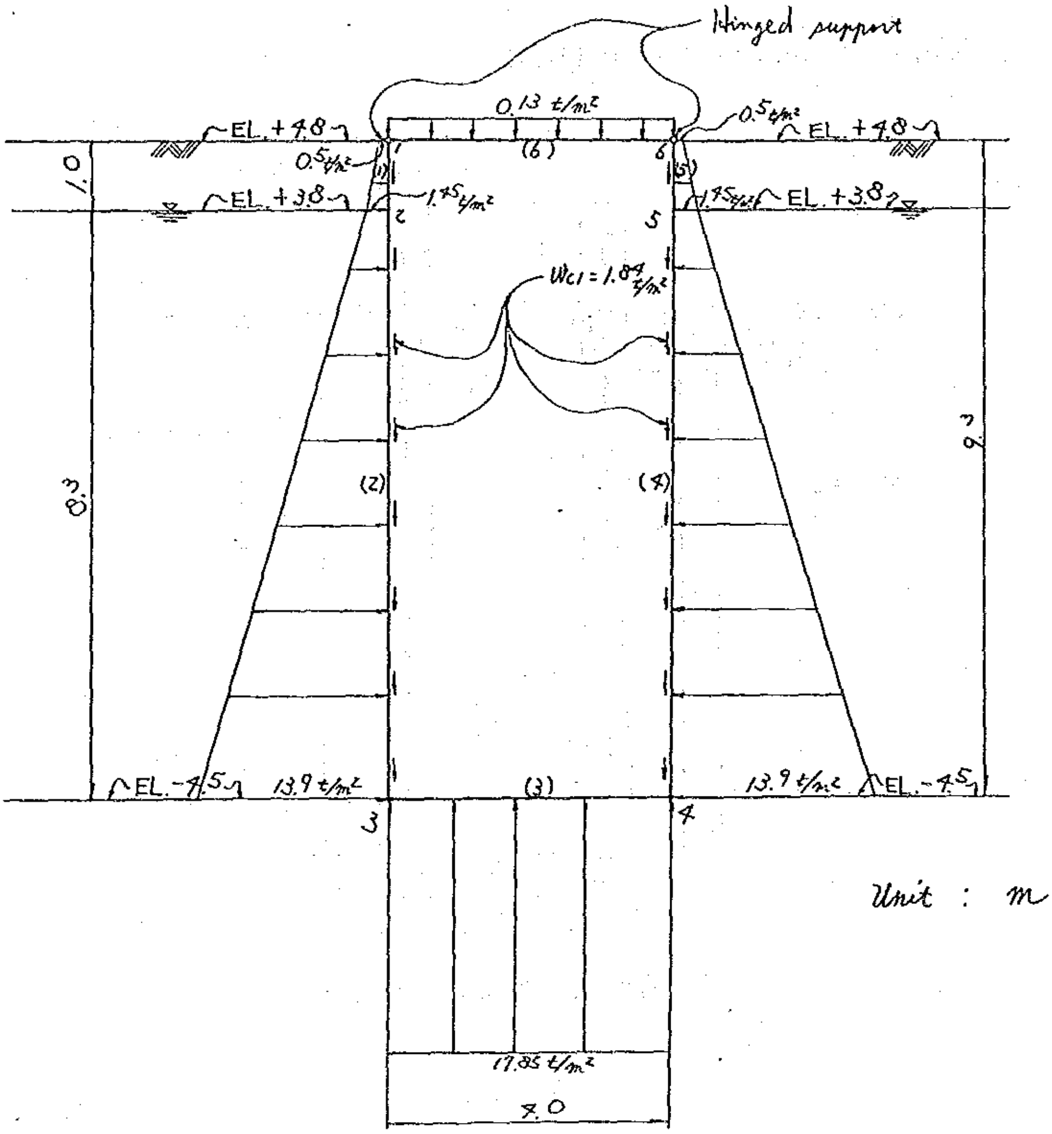

Fig 13. The load diagram

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4) Imput data for the sectional dimensions.

The sectional forces are calculated by computer, so imput data for the sectional dimensions are indicated in Table 5.

Table 5. Table of the sectional dimensions (per 1 m unit length)

Member's number	Section area A (m^2)	Geometrical moment of inertia I (m^4)	Remarks
(1)	0.53	0.0129	Side wall
(2)	0.79	0.0411	Side wall
(3)	1.0	0.0833	Footing
(4)	0.79	0.0411	Side wall
(5)	0.53	0.0129	Side wall
(6)	0.054	0.0016	Beam @ 5.0 m

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Case - 1 (at Normal)

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1.5.4 The Computer Calculation Results

The computer calculation results are bending moment, shearing force and axial force, so they are shown in the following figures. Now comparing with three load cases, the sectional forces are almost same at both cases, so the sectional forces at nomal case are mainly adopted to calculate the sectional stresses and the arrangement of reinforcing bars from the view point of safety design.

INTAKE OPEN CHANNEL CASE-1 (NORMAL, L.L.W.L)

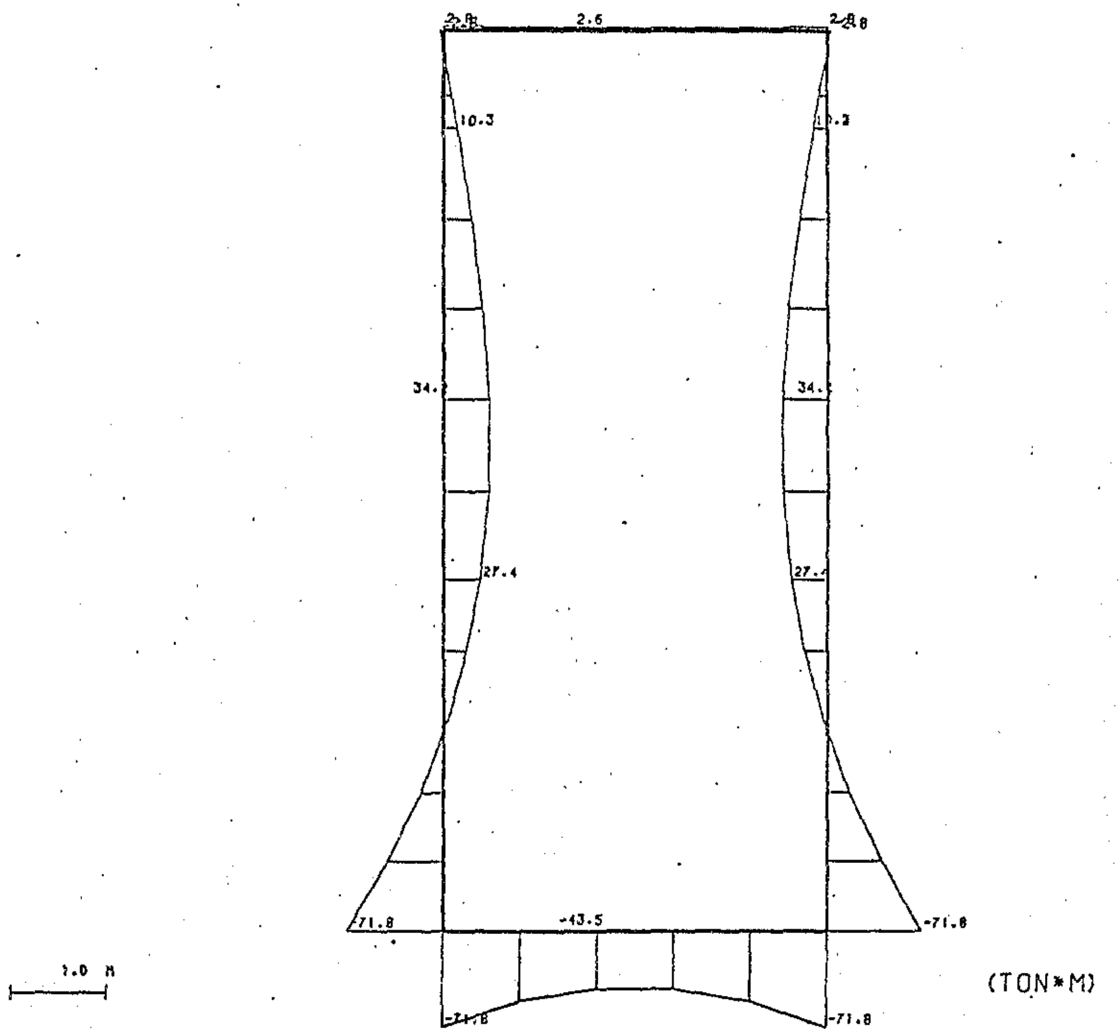

Fig 14. The bending moment diagram

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Case - 2 (at Earthquake)

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INTAKE OPEN CHANNEL CASE-1 (NORMAL, L.L.W.L)

SHEARING FORCE

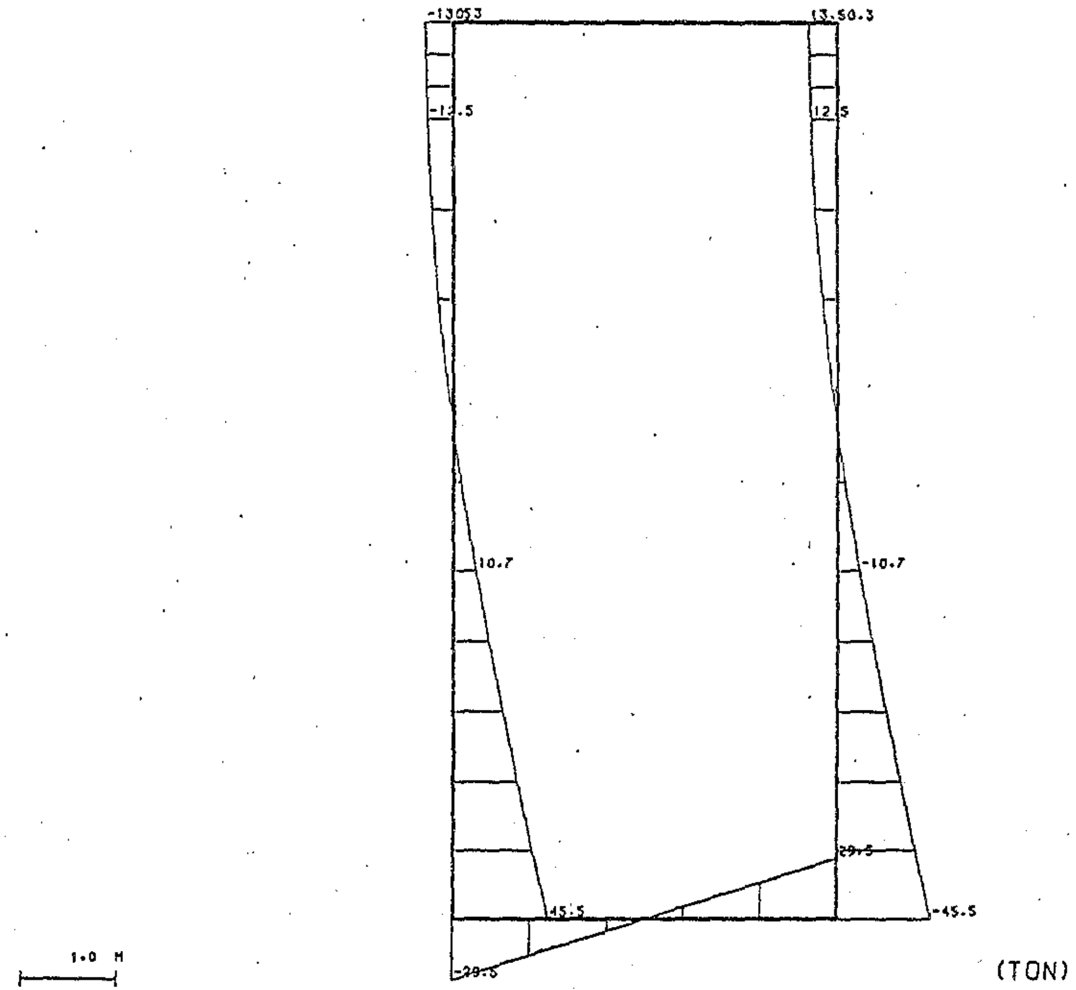

Fig 15. The shearing force diagram

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INTAKE OPEN CHANNEL CASE-1 (NORMAL, L.L.W.L)

AXIAL FORCE

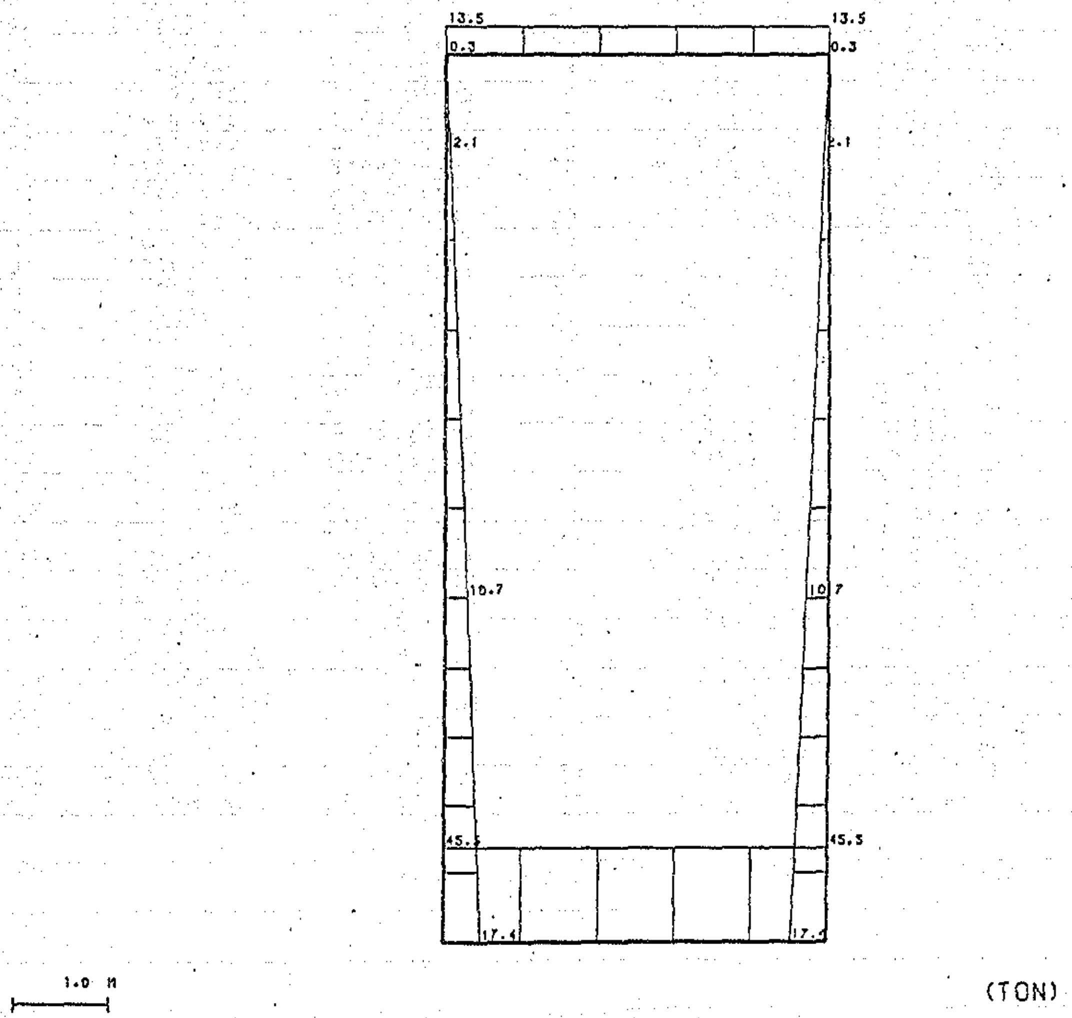

Fig 16. The axial force diagram

Table 6. Summary of the sectional forces (Normal)

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	2.6000E-01	-1.3504E+01	-2.8329E+00	11	8.7327E-01	-1.3284E+01	1.6349E+00
2	11	8.7327E-01	-1.3284E+01	1.6343E+00	12	1.4867E+00	-1.2959E+01	6.0126E+00
3	12	1.4867E+00	-1.2959E+01	6.0120E+00	2	2.1000E+00	-1.2529E+01	1.0263E+01
4	2	2.1000E+00	-1.2529E+01	1.0263E+01	13	3.8222E+00	-1.0514E+01	2.1170E+01
5	13	3.8222E+00	-1.0514E+01	2.1149E+01	14	5.5445E+00	-7.1860E+00	2.9556E+01
6	14	5.5445E+00	-7.1860E+00	2.9536E+01	15	7.2667E+00	-2.5435E+00	3.4212E+01
7	15	7.2667E+00	-2.5435E+00	3.4191E+01	16	8.9890E+00	3.4132E+00	3.3907E+01
8	16	8.9890E+00	3.4132E+00	3.3887E+01	3	1.0711E+01	1.0684E+01	2.7412E+01
9	3	1.0711E+01	1.0684E+01	2.7392E+01	17	1.2043E+01	1.6984E+01	1.7400E+01
10	17	1.2043E+01	1.6984E+01	1.7396E+01	18	1.3376E+01	2.3618E+01	2.7229E+00
11	18	1.3376E+01	2.3618E+01	2.7189E+00	19	1.4708E+01	3.0586E+01	-1.6879E+01
12	19	1.4708E+01	3.0586E+01	-1.6883E+01	20	1.6040E+01	3.7889E+01	-4.1646E+01
13	20	1.6040E+01	3.7889E+01	-4.1651E+01	4	1.7372E+01	4.5527E+01	-7.1823E+01
14	4	4.5527E+01	-2.9460E+01	-7.1827E+01	21	4.5527E+01	-1.7676E+01	-5.2972E+01
15	21	4.5527E+01	-1.7676E+01	-5.2972E+01	22	4.5527E+01	-5.8920E+00	-4.3545E+01
16	22	4.5527E+01	-5.8920E+00	-4.3545E+01	23	4.5527E+01	5.8920E+00	-4.3545E+01
17	23	4.5527E+01	5.8920E+00	-4.3545E+01	24	4.5527E+01	1.7676E+01	-5.2972E+01
18	24	4.5527E+01	1.7676E+01	-5.2972E+01	5	4.5527E+01	2.9460E+01	-7.1827E+01
19	5	1.7372E+01	-4.5527E+01	-7.1827E+01	25	1.6040E+01	-3.7889E+01	-4.1655E+01
20	25	1.6040E+01	-3.7889E+01	-4.1651E+01	26	1.4708E+01	-3.0586E+01	-1.6887E+01
21	26	1.4708E+01	-3.0586E+01	-1.6883E+01	27	1.3376E+01	-2.3618E+01	2.7149E+00
22	27	1.3376E+01	-2.3618E+01	2.7189E+00	28	1.2043E+01	-1.6984E+01	1.7392E+01
23	28	1.2043E+01	-1.6984E+01	1.7396E+01	6	1.0711E+01	-1.0684E+01	2.7388E+01
24	6	1.0711E+01	-1.0684E+01	2.7392E+01	29	8.9890E+00	-3.4132E+00	3.3866E+01
25	29	8.9890E+00	-3.4132E+00	3.3887E+01	30	7.2667E+00	2.5435E+00	3.4171E+01
26	30	7.2667E+00	2.5435E+00	3.4191E+01	31	5.5445E+00	7.1860E+00	2.9515E+01
27	31	5.5445E+00	7.1860E+00	2.9536E+01	32	3.8222E+00	1.0514E+01	2.1129E+01
28	32	3.8222E+00	1.0514E+01	2.1149E+01	7	2.1000E+00	1.2529E+01	1.0242E+01
29	7	2.1000E+00	1.2529E+01	1.0263E+01	33	1.4867E+00	1.2959E+01	6.0114E+00
30	33	1.4867E+00	1.2959E+01	6.0120E+00	34	8.7327E-01	1.3284E+01	1.6337E+00
31	34	8.7327E-01	1.3284E+01	1.6343E+00	8	2.6000E-01	1.3504E+01	-2.8334E+00
32	1	1.3504E+01	2.6000E-01	2.8329E+00	35	1.3504E+01	1.5600E-01	2.6665E+00
33	35	1.3504E+01	1.5600E-01	2.6665E+00	36	1.3504E+01	5.2000E-02	2.5833E+00
34	36	1.3504E+01	5.2000E-02	2.5833E+00	37	1.3504E+01	-5.2000E-02	2.5833E+00
35	37	1.3504E+01	-5.2000E-02	2.5833E+00	38	1.3504E+01	-1.5600E-01	2.6665E+00
36	38	1.3504E+01	-1.5600E-01	2.6665E+00	8	1.3504E+01	-2.6000E-01	2.8329E+00

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INTAKE OPEN CHANNELCASE-2 (EARTHQUAKE, L. L. W. L)

BENDING MOMENT

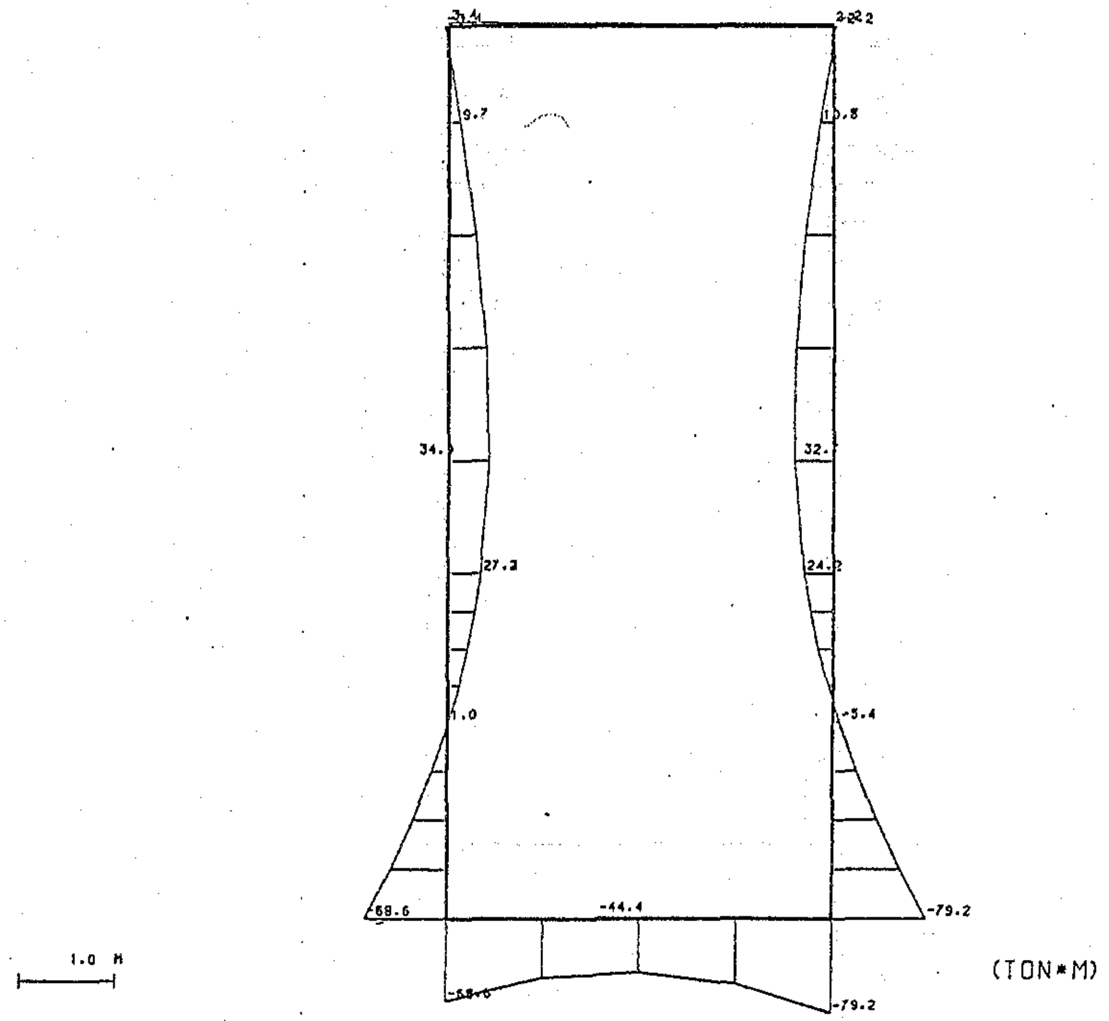

Fig 17. The bending moment diagram

INTAKE OPEN CHANNELCASE-2 (EARTHQUAKE, L. L. W. L)

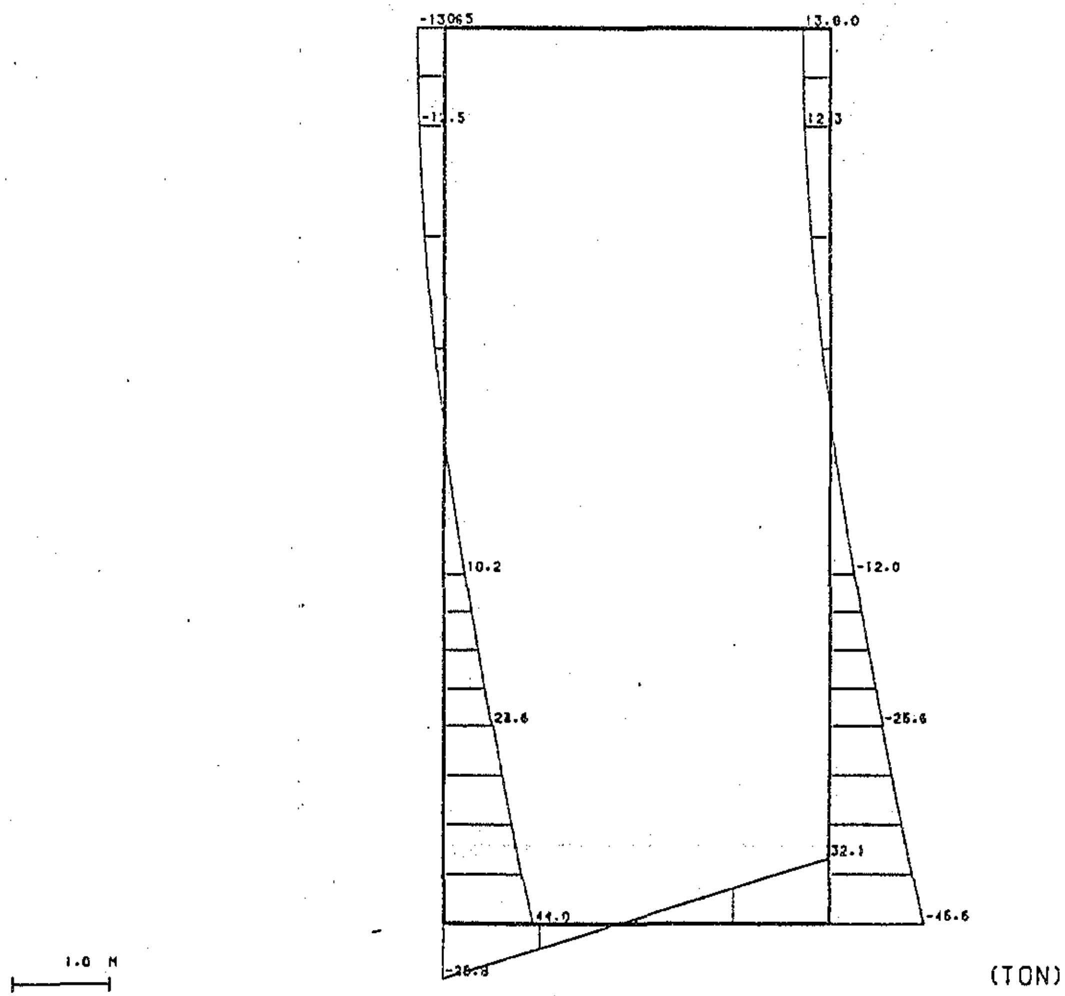

Fig 18. The shearing force diagram

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INTAKE OPEN CHANNELCASE-2 (EARTHQUAKE, L. L. W. L)

AXIAL FORCE

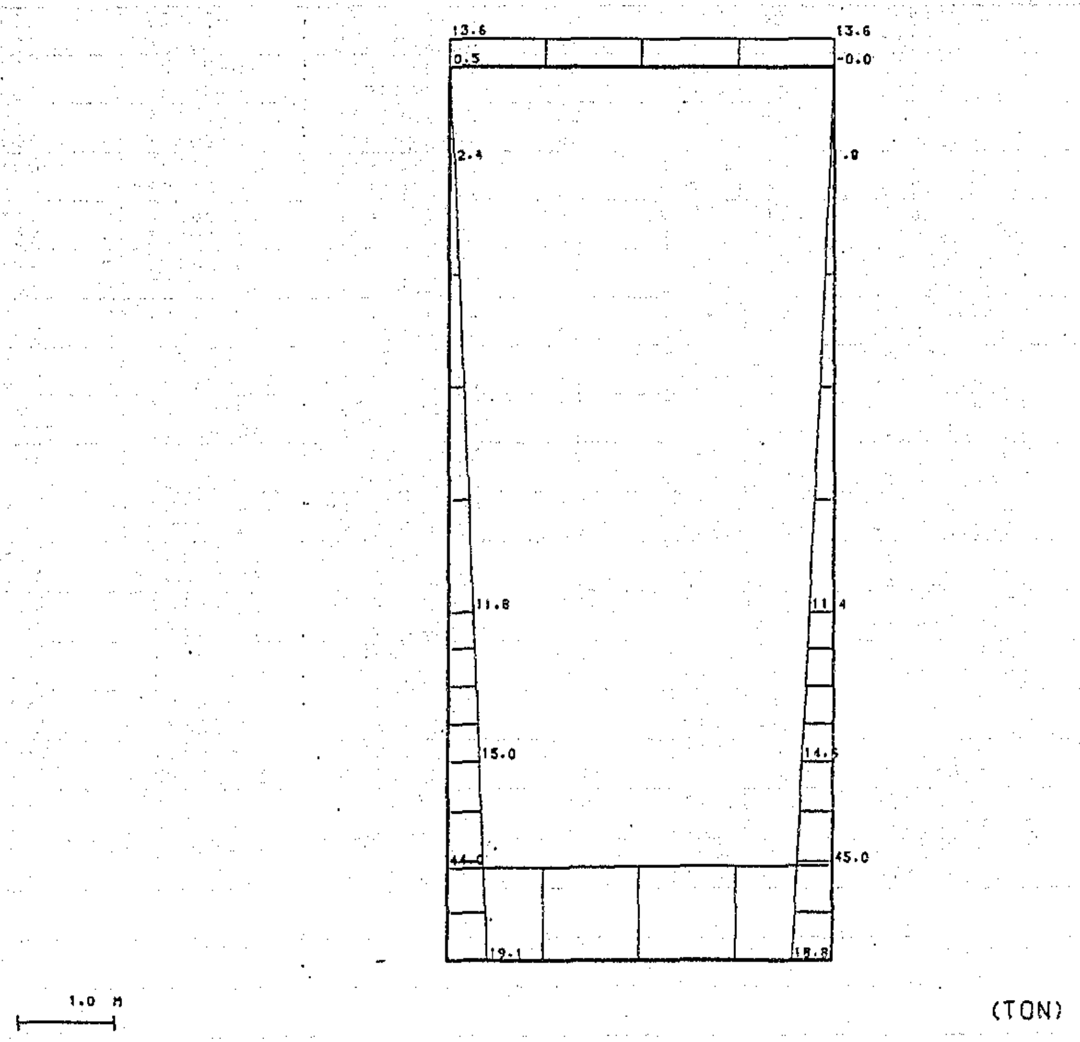

Fig 19. The axial force diagram

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Table 7. Summary of the sectional forces (Earthquake)

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	5.4210E-01	-1.3579E+01	-3.3665E+00	13	1.4871E+00	-1.3149E+01	3.3263E+00
2	13	1.4871E+00	-1.3149E+01	3.3244E+00	2	2.4321E+00	-1.2499E+01	9.7472E+00
3	2	2.4321E+00	-1.2499E+01	9.7454E+00	14	4.7838E+00	-9.7447E+00	2.2986E+01
4	14	4.7838E+00	-9.7447E+00	2.2948E+01	15	7.1355E+00	-5.0399E+00	3.1825E+01
5	15	7.1355E+00	-5.0399E+00	3.1787E+01	16	9.4872E+00	1.6160E+00	3.4018E+01
6	16	9.4872E+00	1.6160E+00	3.3980E+01	3	1.1839E+01	1.0223E+01	2.7283E+01
7	3	1.1839E+01	1.0223E+01	2.7245E+01	17	1.2623E+01	1.3452E+01	2.2631E+01
8	17	1.2623E+01	1.3452E+01	2.2631E+01	18	1.3407E+01	1.6751E+01	1.6744E+01
9	18	1.3407E+01	1.6751E+01	1.6743E+01	19	1.4191E+01	2.0121E+01	9.5558E+00
10	19	1.4191E+01	2.0121E+01	9.5554E+00	4	1.4975E+01	2.3561E+01	1.0402E+00
11	4	1.4975E+01	2.4131E+01	1.0397E+00	20	1.6010E+01	2.8813E+01	-1.2584E+01
12	20	1.6010E+01	2.8813E+01	-1.2585E+01	21	1.7045E+01	3.3681E+01	-2.8668E+01
13	21	1.7045E+01	3.3681E+01	-2.8670E+01	22	1.8080E+01	3.8737E+01	-4.7308E+01
14	22	1.8080E+01	3.8737E+01	-4.7309E+01	5	1.9115E+01	4.3979E+01	-6.8599E+01
15	5	4.3979E+01	-2.6821E+01	-6.8601E+01	23	4.4229E+01	-1.2091E+01	-4.9145E+01
16	23	4.4229E+01	-1.2091E+01	-4.9145E+01	24	4.4479E+01	2.6390E+00	-4.4419E+01
17	24	4.4479E+01	2.6390E+00	-4.4419E+01	25	4.4729E+01	1.7369E+01	-5.4423E+01
18	25	4.4729E+01	1.7369E+01	-5.4423E+01	6	4.4979E+01	3.2099E+01	-7.9157E+01
19	6	1.8820E+01	-4.6589E+01	-7.9157E+01	26	1.7769E+01	-4.1042E+01	-5.6601E+01
20	26	1.7769E+01	-4.1042E+01	-5.6600E+01	27	1.6719E+01	-3.5689E+01	-3.6851E+01
21	27	1.6719E+01	-3.5689E+01	-3.6850E+01	28	1.5668E+01	-3.0531E+01	-1.9808E+01
22	28	1.5668E+01	-3.0531E+01	-1.9806E+01	7	1.4617E+01	-2.5567E+01	-5.3712E+00
23	7	1.4617E+01	-2.6137E+01	-5.3695E+00	29	1.3822E+01	-2.2489E+01	4.1096E+00
24	29	1.3822E+01	-2.2489E+01	4.1101E+00	30	1.3026E+01	-1.8916E+01	1.2181E+01
25	30	1.3026E+01	-1.8916E+01	1.2182E+01	31	1.2231E+01	-1.5418E+01	1.8874E+01
26	31	1.2231E+01	-1.5418E+01	1.8874E+01	8	1.1435E+01	-1.1996E+01	2.4217E+01
27	8	1.1435E+01	-1.1996E+01	2.4218E+01	32	9.0483E+00	-2.8608E+00	3.2670E+01
28	32	9.0483E+00	-2.8608E+00	3.2710E+01	33	6.6615E+00	4.2323E+00	3.1668E+01
29	33	6.6615E+00	4.2323E+00	3.1708E+01	34	4.2747E+00	9.2838E+00	2.3563E+01
30	34	4.2747E+00	9.2838E+00	2.3602E+01	9	1.8879E+00	1.2294E+01	1.0741E+01
31	9	1.8879E+00	1.2294E+01	1.0780E+01	35	9.3290E-01	1.3050E+01	4.4352E+00
32	35	9.3290E-01	1.3050E+01	4.4368E+00	10	-2.2103E-02	1.3619E+01	-2.2397E+00
33	1	1.3579E+01	5.4210E-01	3.3665E+00	36	1.3589E+01	4.1210E-01	2.8894E+00
34	36	1.3589E+01	4.1210E-01	2.8894E+00	37	1.3599E+01	2.8210E-01	2.5423E+00
35	37	1.3599E+01	2.8210E-01	2.5423E+00	38	1.3609E+01	1.5210E-01	2.3252E+00
36	38	1.3609E+01	1.5210E-01	2.3252E+00	10	1.3619E+01	2.2103E-02	2.2381E+00

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Case - 3 (at Construction)

FORM 04

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INTAKE OPEN CHANNEL CASE-3 (CONSTRUCTION, EMPTY)

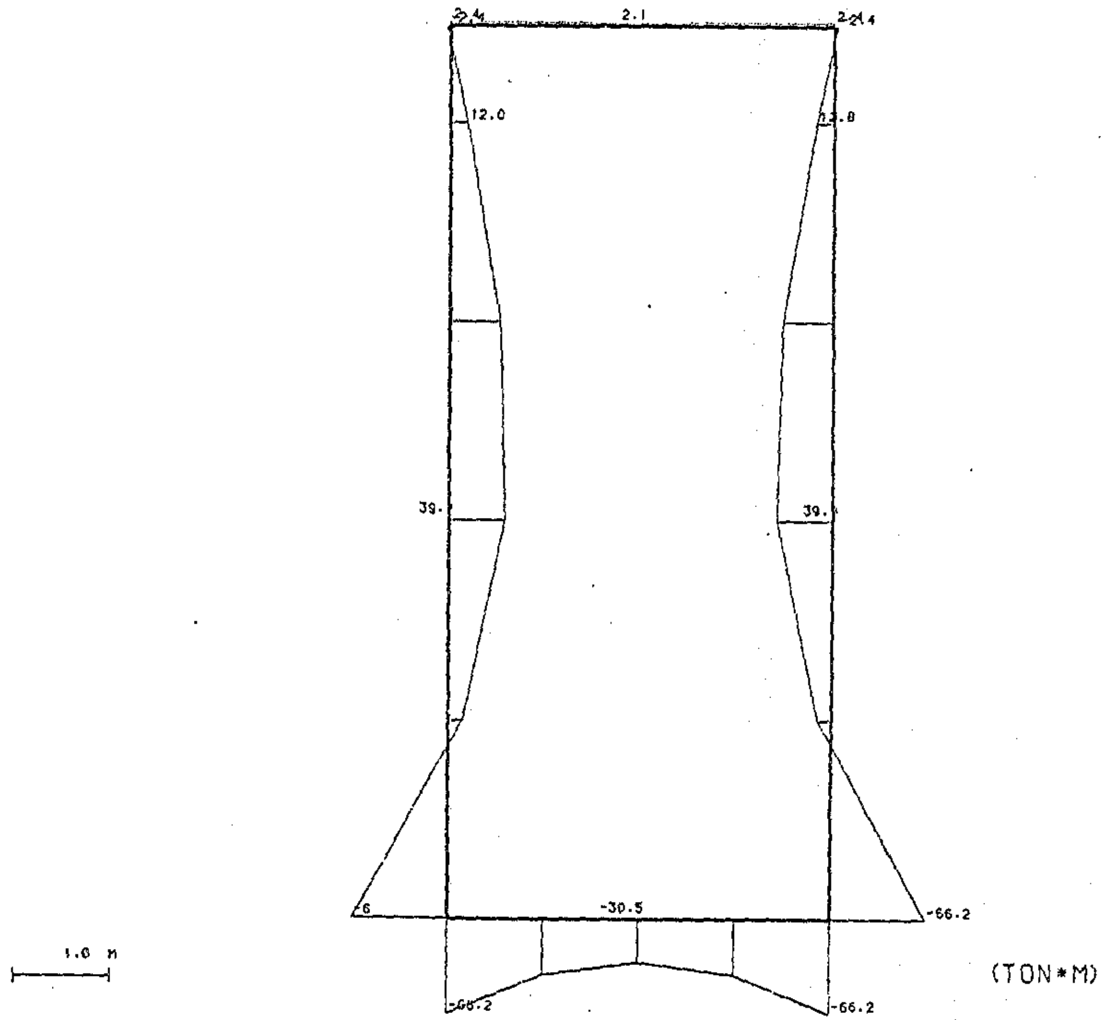

Fig 20. The bending moment diagram

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INTAKE OPEN CHANNEL CASE-3 (CONSTRUCTION, EMPTY)

SHEARING FORCE

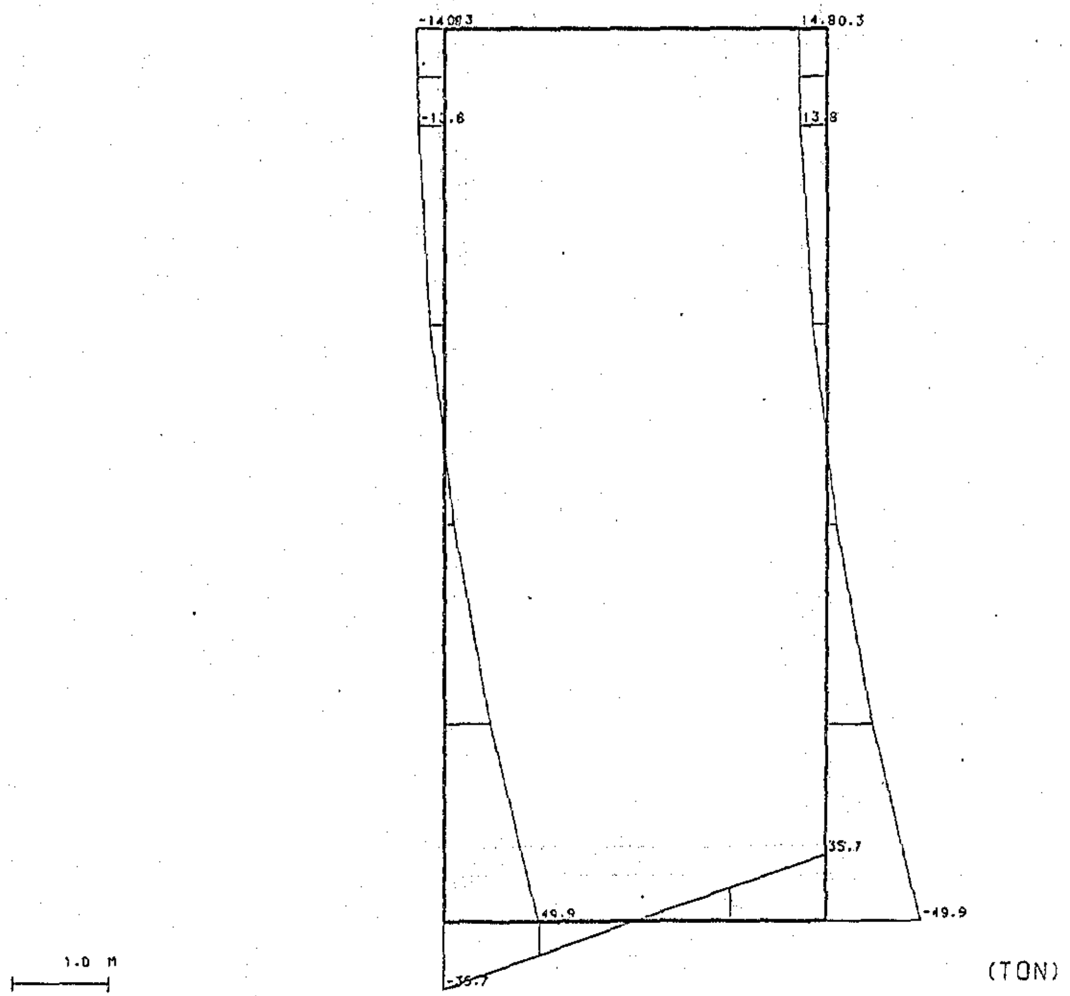

Fig 21. The shearing force diagram

INTAKE OPEN CHANNEL CASE-3 (CONSTRUCTION, EMPTY)

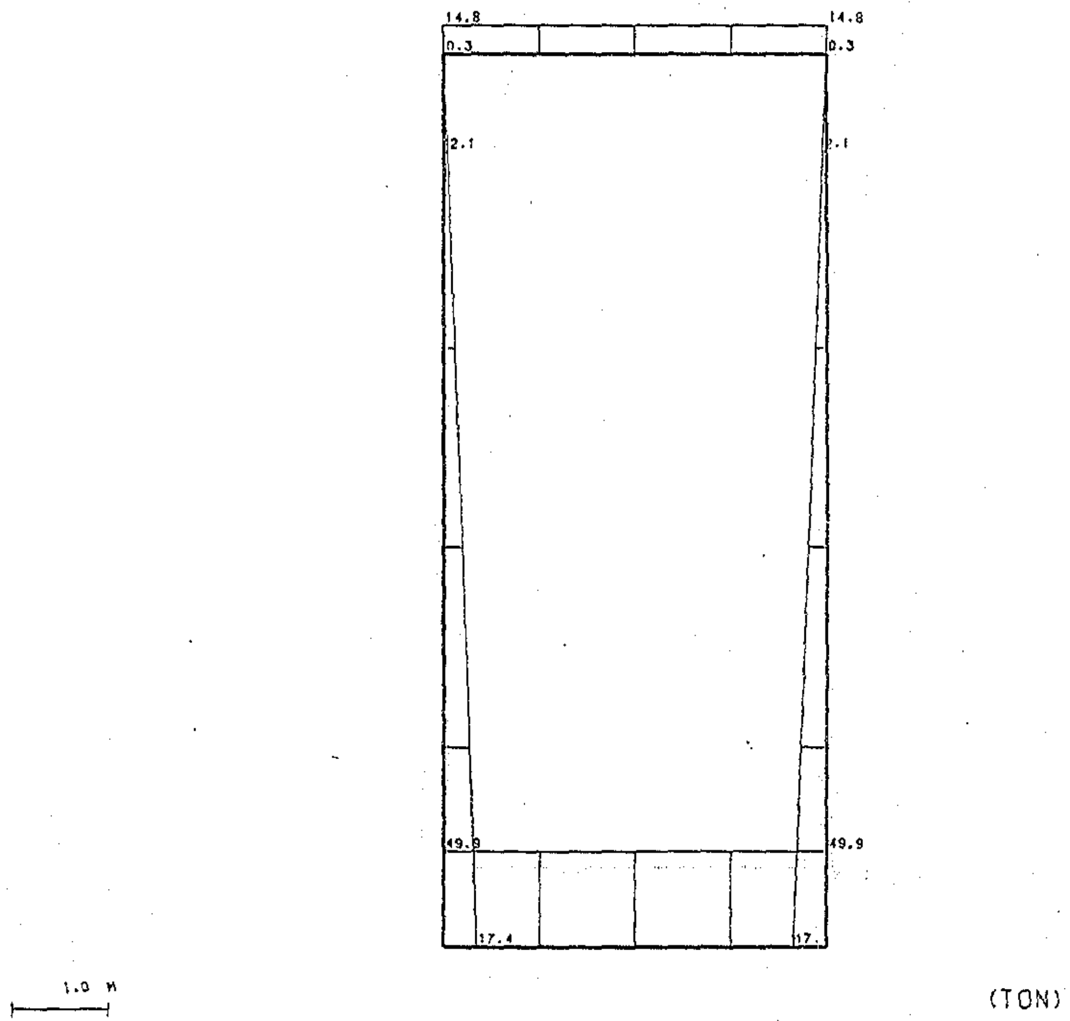

Fig 22. The axial force diagram

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Table 8. Summary of sectional forces (Construction)

** ELEMENTAL FORCES **

ELEM	I-END	AXIAL	SHEAR	MOMENT	J-END	AXIAL	SHEAR	MOMENT
1	1	2.6000E-01	-1.4799E+01	-2.4035E+00	9	1.1800E+00	-1.4430E+01	4.9156E+00
2	9	1.1800E+00	-1.4430E+01	4.9136E+00	2	2.1000E+00	-1.3824E+01	1.1969E+01
3	2	2.1000E+00	-1.3824E+01	1.1987E+01	10	5.9180E+00	-7.5858E+00	3.5540E+01
4	10	5.9180E+00	-7.5858E+00	3.5316E+01	11	9.7360E+00	5.1106E+00	3.9224E+01
5	11	9.7360E+00	5.1106E+00	3.9001E+01	12	1.3554E+01	2.4265E+01	9.8636E+00
6	12	1.3554E+01	2.4265E+01	9.6402E+00	3	1.7372E+01	4.9879E+01	-6.5944E+01
7	3	4.9879E+01	-3.5700E+01	-6.6168E+01	13	4.9879E+01	-1.7850E+01	-3.9393E+01
8	13	4.9879E+01	-1.7850E+01	-3.9393E+01	14	4.9879E+01	3.5283E-13	-3.0468E+01
9	14	4.9879E+01	4.8406E-13	-3.0468E+01	15	4.9879E+01	1.7850E+01	-3.9393E+01
10	15	4.9879E+01	1.7850E+01	-3.9393E+01	4	4.9879E+01	3.5700E+01	-6.6168E+01
11	4	1.7372E+01	-4.9879E+01	-6.6168E+01	16	1.3554E+01	-2.4265E+01	9.4169E+00
12	16	1.3554E+01	-2.4265E+01	9.6402E+00	17	9.7360E+00	-5.1106E+00	3.8778E+01
13	17	9.7360E+00	-5.1106E+00	3.9001E+01	18	5.9180E+00	7.5858E+00	3.5093E+01
14	18	5.9180E+00	7.5858E+00	3.5316E+01	5	2.1000E+00	1.3824E+01	1.1764E+01
15	5	2.1000E+00	1.3824E+01	1.1987E+01	19	1.1800E+00	1.4430E+01	4.9116E+00
16	19	1.1800E+00	1.4430E+01	4.9136E+00	6	2.6000E-01	1.4799E+01	-2.4055E+00
17	1	1.4799E+01	2.6000E-01	2.4035E+00	20	1.4799E+01	1.3000E-01	2.2085E+00
18	20	1.4799E+01	1.3000E-01	2.2085E+00	21	1.4799E+01	4.4520E-14	2.1435E+00
19	21	1.4799E+01	4.4673E-14	2.1435E+00	22	1.4799E+01	-1.3000E-01	2.2085E+00
20	22	1.4799E+01	-1.3000E-01	2.2085E+00	6	1.4799E+01	-2.6000E-01	2.4035E+00

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1.5.5 The Calculation Results of the Stress

The stress calculations are shown in the following Tables, and the general arrangement of reinforcing bars are figured in Fig.26, now the allowable stress of reinforced concrete is stated again as below.

1) The allowable bending stress of the reinforcing bars σ_{ba}
   σ_{ba} = 1800 kg/cm2

2) The allowable compressive stress σ_{ca}
   σ_{ca} = 70 kg/cm2

3) The allowable shearing stress τ_a

Table 9. The allowable shearing stress τ_a

Classification		τ_a (kg/cm^2)
No considered for the diagonal tension bars	Beam	4.25
	Slab	8.5
Considered for the diagonal tension bars	As working force is the shearing force only	19.0

In this design case the design structure is considerd for the slab structure, so the allowable shearing stress is determined τ_a= 8.5 kg/cm^2.

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4) Study of coping

Coping is considered for the continuous beam, so the bending moment M_b and the shearing force S are calculated as follows, then the uniformly distributed load is $q = 13.5$ t/m.

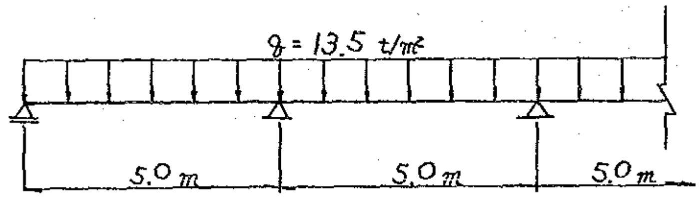

Fig.23 The load diagram

a) The bending moment

i) at the support

$$M_{be} = -\frac{1}{10} q \cdot \ell^2 = -\frac{1}{10} \times 13.5 \times 5^2 = -33.8 \text{ t/m}$$

ii) at the middle point of beam

The maximum bending moment at the middle of beam is occured at both end spans, then the bending moment is calculated as below.

$$M_{bm} = \frac{8}{100} q \cdot \ell^2 = \frac{8}{100} \times 13.5 \times 5^2 = 27.0 \text{ t/m}$$

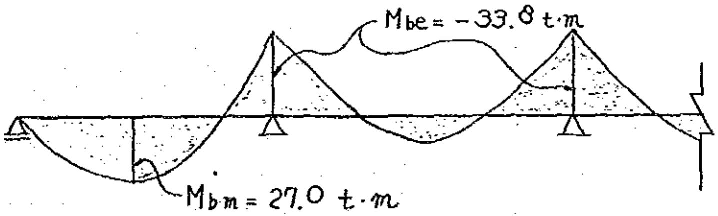

Fig. 24 The bending moment diagram

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b) The shearing stress

The shearing stress is equal to the reaction force at the support.

$$S = \frac{1}{2} q \cdot \ell = \frac{1}{2} \times 13.5 \times 5.0 = 33.8 \text{ t}$$

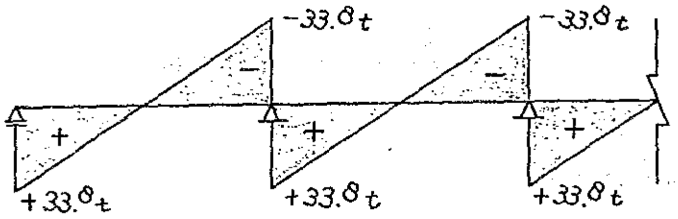

Fig.25 The shearing force diagram

[Intake Open Channel]

Table. 10-1 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	300	14 000	100	50	40	10	D25 D22	125 250	40.5 15.5	-1	0.1	3.5	Calculated on hypothetical section.
	Center	600 000	1 500	13 000	100	54	44	10	D25 D22	125 250	40.5 15.5	386	17.6	3.0	
	2	1 030 000	2 100	12 600	100	56	46	10	D25 D22	125 250	40.5 15.5	633	27.9	2.8	
(2)	2	1 030 000	2 100	12 600	100	56	46	10	D25 D22	125 250	40.5 15.5	633	27.9	2.8	
	Center	3 420 000	7 300	2 500	100	69	59	10	D25 D22	125 250	40.5 15.5	1 543	58.5	0.7	
	3	2 740 000	10 700	10 700	100	78	68	10	D25 D22	125 250	40.5 15.5	999	37.5	1.6	
(3)	3	2 740 000	10 700	10 700	100	78	68	10	D25 D22	125 250	40.5 15.5	999	37.5	1.6	
	Center	-1 690 000	14 700	30 600	100	91	81	10	D29 D25	125 125	51.4 40.5	327	15.1	1.3	
	4	-7 180 000	17 400	45 500	100	110	100	10	D29 D25	125 125	51.4 40.5	1 394	44.0	4.6	Considered for the diagonal tension bars.
(4)	4	-7 180 000	45 500	29 500	100	110	100	10	D29 D22	125 250	51.4 15.5	1 172	49.9	3.0	
	Center	-4 350 000	45 500	5 900	100	100	90	10	D29 D22	125 250	51.4 15.5	1 002	34.2	0.0	
	5	-7 180 000	45 500	29 500	100	110	100	10	D29 D22	125 250	51.4 15.5	1 172	49.9	3.0	

Where
M : 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

[Intake Open Channel]

Table.10-2 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	-7 180 000	17 400	45 500	100	110	100	10	D29 D25	125 125	51.4 40.5	1 394	44.0	4.6	Considered for the diagonal tension bars.
	Center	-1 690 000	14 700	30 600	100	91	81	10	D29 D25	125 125	51.4 40.5	327	15.1	1.3	
	6	2 740 000	10 700	10 700	100	78	68	10	D25 D22	125 250	40.5 15.5	999	37.5	1.6	
(6)	6	2 740 000	10 700	10 700	100	78	68	10	D25 D22	125 250	40.5 15.5	999	37.5	1.6	
	Center	3 420 000	7 300	2 500	100	69	59	10	D25 D22	125 250	40.5 15.5	1 543	58.5	0.4	
	7	1 030 000	2 100	12 600	100	56	46	10	D25 D22	125 250	40.5 15.5	633	27.9	2.8	
(7)	7	1 030 000	2 100	12 600	100	56	46	10	D25 D22	125 250	40.5 15.5	633	27.9	2.8	
	Center	600 000	1 500	13 000	100	54	44	10	D25 D22	125 250	40.5 15.5	386	17.6	3.0	
	8	0	300	14 000	100	50	40	10	D25 D22	125 250	40.5 15.5	-1	0.1	3.5	Calculated on hypothetical section.
(8)	1	0	67 500	1 300	45	60	52.5	7.5	D22 D22	3 pieces 3 pieces	11.6 11.6	0	66.6	0.6	
	Center	1 290 000	67 500	300	45	60	52.5	7.5	D22 D22	3 pieces 3 pieces	11.6 11.6	283	66.6	0.1	
	8	0	67 500	1 300	45	60	52.5	7.5	D22 D22	3 pieces 3 pieces	11.6 11.6	0	66.6	0.6	

Where
M : 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

[Intake Open Channel]

Coping

Table. 10-3 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	
Coping	end	-3 380000	0	33.8	70	75	60	15	D25 D25	8 pieces 8 pieces	40.5 40.5	1639	66.4	8.0	Considered for the diagonal tension force
	middle	2 700000	0	0	70	90	75	15	D25 D25	8 pieces 8 pieces	40.5 40.5	1026	35.9	0.0	

Where
M : 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

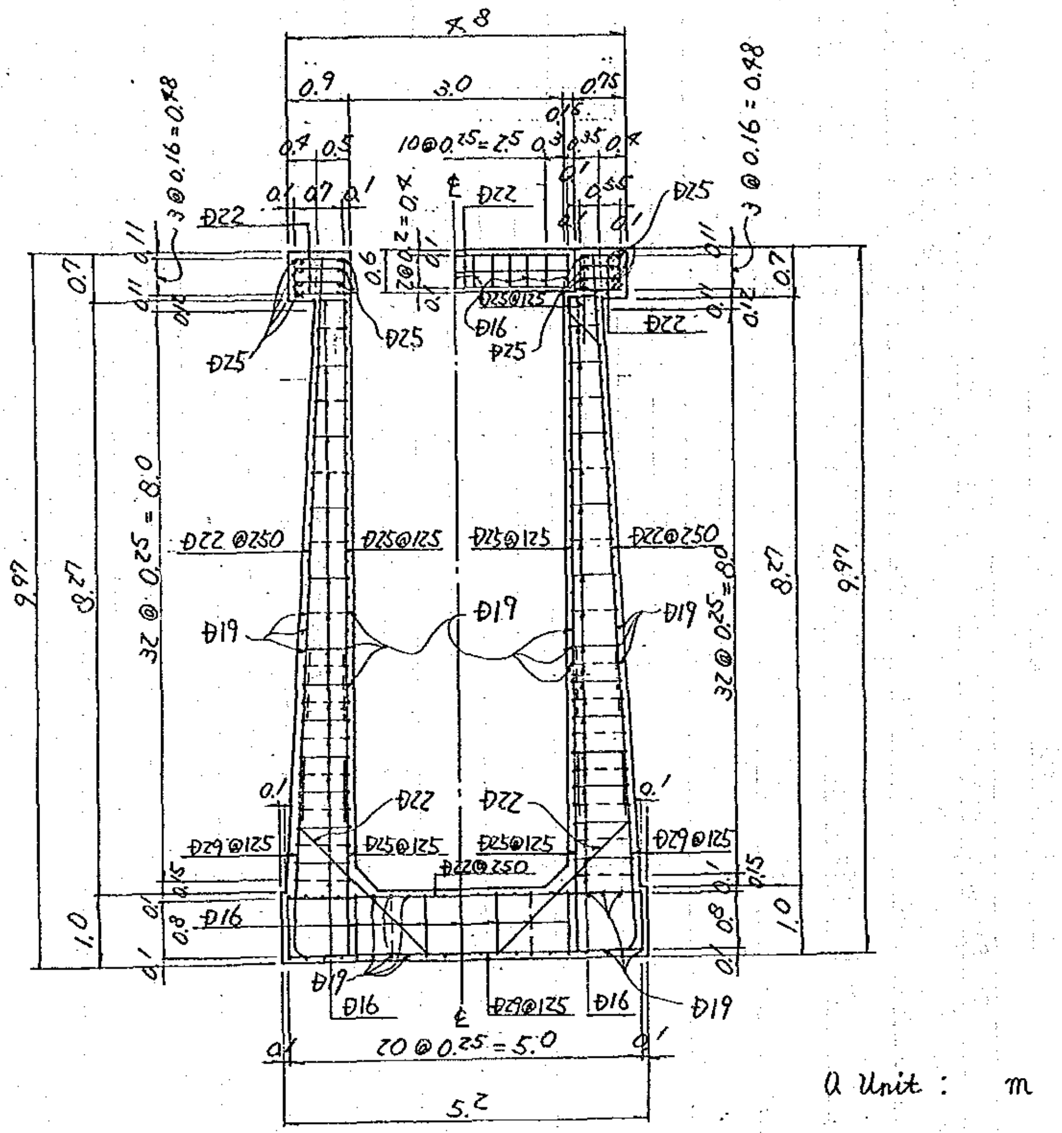

Fig. 26 The genenral arrangement of reinforcing bars

1.5.6 Study of End Wall

1) Outline of design structure

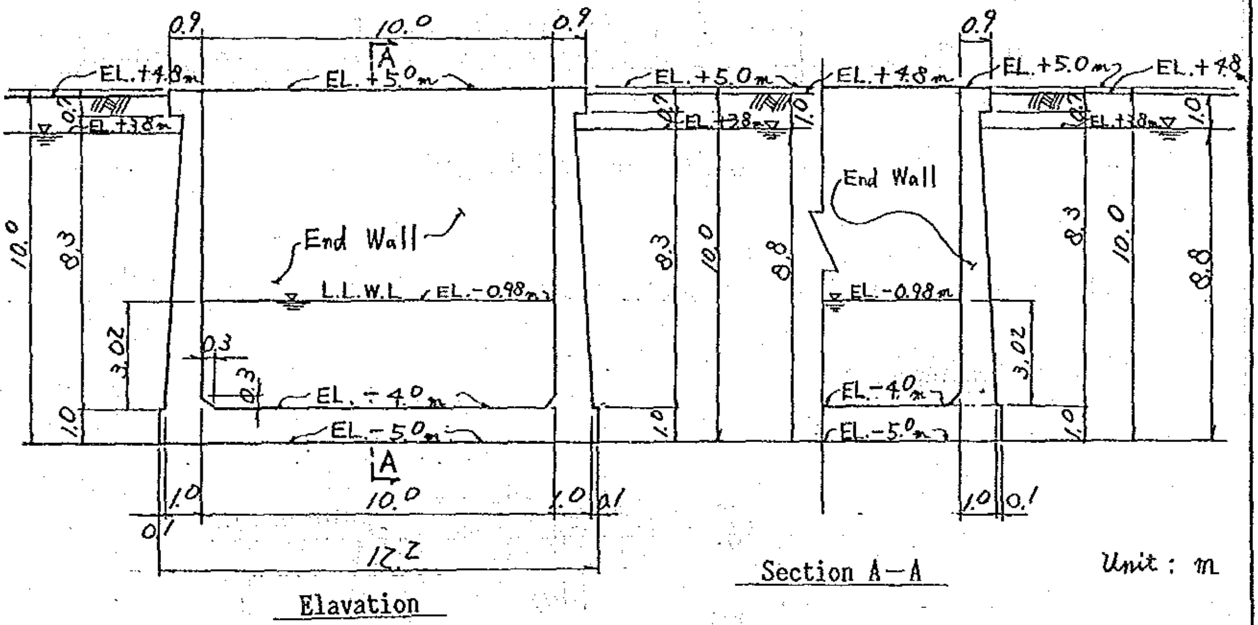

Fig 26. Outline of end wall

2) Strucrural design

End wall is considered for a two dimensional plate with three edges fixed and one edge free, so the structural design calculation is executed at nomal case as follows.

a) The load diagram (Normal)

Working load to end wall is the same load as the channel, then the uniformly distributed load of end wall is transformed to the uniformly varying load that is shown in Fig 23.

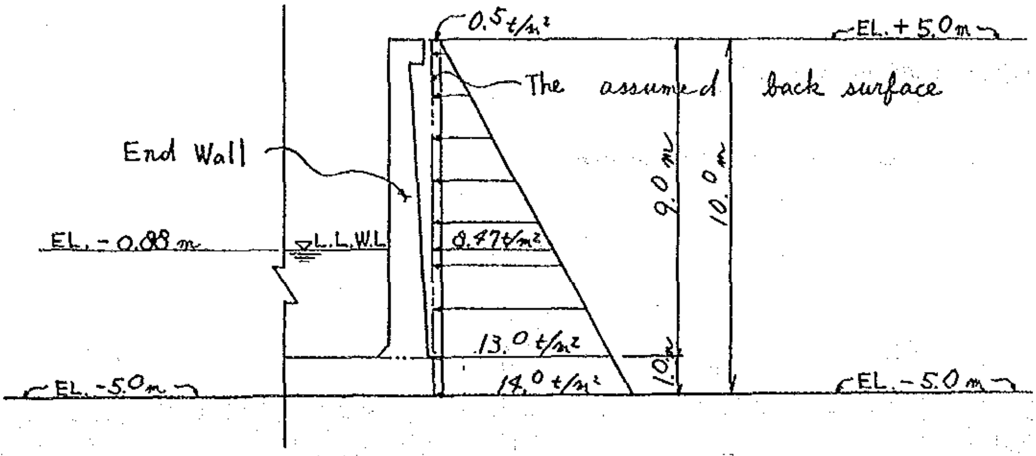

Fig 27. The load diagram

b) The calculation of the sectional force

i) the bending moment

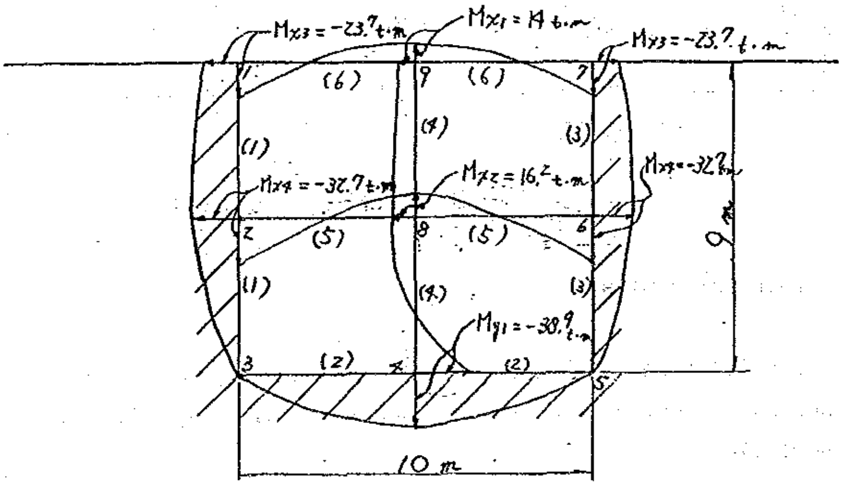

Fig 28. The bending moment diagram

M_{x1} = (0.0425 × 0.5 + 0.0095 × 12.5) × 10^2 = 14 t·m
M_{x2} = (0.0287 × 0.5 + 0.0118 × 12.5) × 10^2 = 16.2 t·m
M_{x3} = (−0.0836 × 0.5 − 0.0156 × 12.5) × 10^2 = −23.7 t·m
M_{x4} = (−0.0563 × 0.5 − 0.0239 × 12.5) × 10^2 = −32.7 t·m
M_{y1} = (−0.0523 × 0.5 − 0.029 × 12.5) × 10^2 = −38.9 t·m

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ii) the shearing force

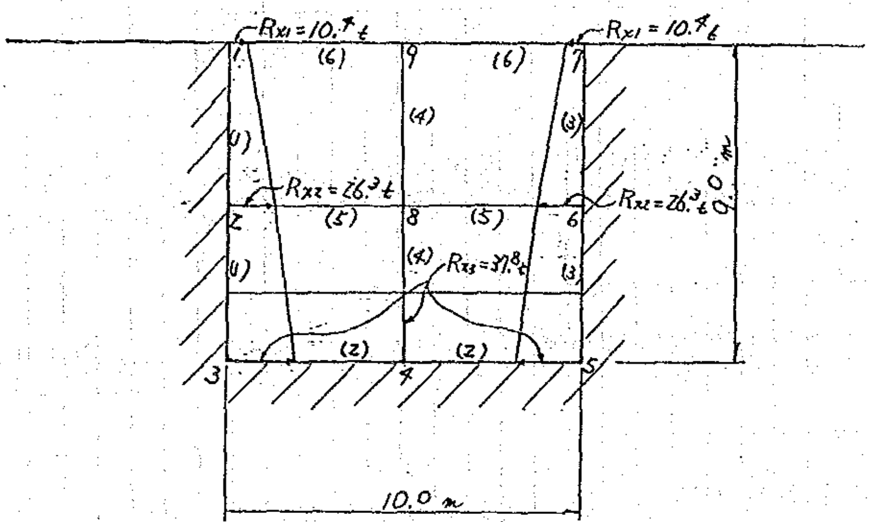

Fig 29. The reaction force diagram

$$S_{x1} = (0.656 \times 0.5 + 0.057 \times 12.5) \times 10.0 = 10.4 \text{ t}$$

$$S_{x2} = (0.414 \times 0.5 + 0.194 \times 12.5) \times 10.0 = 26.3 \text{ t}$$

$$S_{x3} = (0.406 \times 0.5 + 0.286 \times 12.5) \times 10.0 = 37.8 \text{ t}$$

Accodinig to the above calculation, the results of the stress calculations are shown in the following tables and the general arrangement of reinforcing bars is shown in Fig 30.

〔End Wall〕

Intake Open Channel

Table. 11-1 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	-2370000	0	10400	100	90	80	10	D25	125	40.5	815	22.0	1.5	
									D25	125	40.5				
	2	-3270000	0	26300	100	75	65	10	D25	125	40.5	1405	42.8	4.0	
									D25	125	40.5				
	3	0	0	37800	100	110	100	10	D22	125	31.0	0	0	3.8	
									D22	125	31.0				
(2)	3	0	0	37800	100	110	100	10	D22	125	31.0	0	0	3.8	
									D22	125	31.0				
	4	-3890000	0	37800	100	110	100	10	D22	125	31.0	1369	28.8	3.8	
									D22	125	31.0				
	5	0	0	37800	100	110	100	10	D22	125	31.0	0	0	3.8	
									D22	125	31.0				
(3)	5	0	0	37800	100	110	100	10	D22	125	31.0	0	0	3.8	
									D22	125	31.0				
	6	-3270000	0	26300	100	75	65	10	D25	125	40.5	1405	42.8	4.0	
									D25	125	40.5				
	7	-2370000	0	10400	100	90	80	10	D25	125	40.5	815	22.0	1.5	
									D25	125	40.5				
(4)	4	-3890000	0	37800	100	110	100	10	D22	125	31.0	1369	28.8	3.8	
									D22	125	31.0				
	8	1620000	0	0	100	75	65	10	D22	125	31.0	900	24.2	0	
									D22	125	31.0				
	9	1400000	0	0	100	90	80	10	D25	125	40.5	482	13.0	0	
									D25	125	40.5				

Where
M : 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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〔End Wall〕

Intake Open Channel

Table. 11-2 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)	2	-3270000	0	0	100	75	65	10	D25 / D25	125 / 125	40.5 / 40.5	1405	42.8	0	
	8	1620000	0	0	100	75	65	10	D22 / D22	125 / 125	31.0 / 31.0	900	24.2	0	
	6	-3270000	0	0	100	75	65	10	D25 / D25	125 / 125	40.5 / 40.5	1405	42.8	0	
(6)	1	-2370000	0	0	100	90	80	10	D25 / D25	125 / 125	40.5 / 40.5	815	22.0	0	
	9	1400000	0	0	100	90	80	10	D25 / D25	125 / 125	40.5 / 40.5	482	13.0	0	
	7	-2370000	0	0	100	90	80	10	D25 / D25	125 / 125	40.5 / 40.5	815	22.0	0	

Where
M : 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

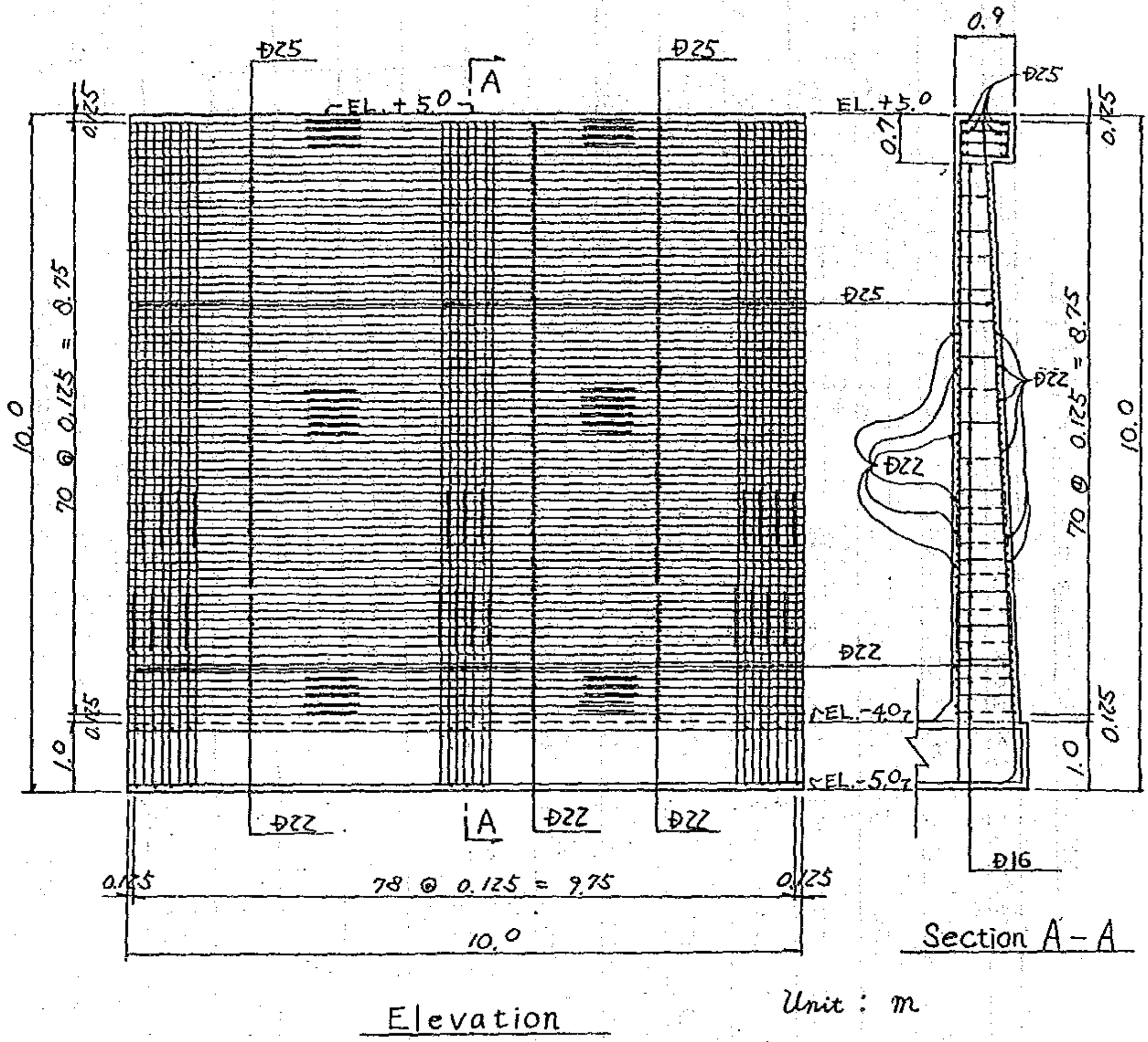

Fig 30. The general arrangement of reinforcing bars

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