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ROYAL IRRIGATION DEPARTMENT
MINISTRY OF AGRICULTURE AND COOPERATIVE
GOVERNMENT OF THE KINGDOM OF THAILAND

FINAL DESIGN REPORT
FOR
DOK KRAI – MAB TA PUD WATER PIPELINE PROJECT
IN
THE EAST COAST AREA

(SUPPORTING REPORT)

AUGUST 1982

JAPAN INTERNATIONAL COOPERATION AGENCY

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CHAPTER I - RESULTS OF DETAILED DESIGN

1.1 DESIGN CONDITIONS

1.1.1 Conditions Common to All Items

Unit Weight of Materials

Water	1.00 t/m^3
Plain concrete	2.30 t/m^3
Reinforced concrete	Unit weight of reinforced concrete is determined at each structure.
Prestressed concrete	- do -
Soil	1.80 t/m^3
Steel	7.85 t/m^3

Allowable Stress

1) Reinforcing bar

Deformed bar is used
Allowable tensile stress .... 1,400 kg/cm^2

2) Reinforced concrete

Strength of 28 days age $fc' = 210$ kg/cm^2

Allowable compressive stress : fc

a)	Bending compressive stress	94.5 kg/cm^2
b)	Axial compressive stress	52.2 kg/cm^2
c)	Shear stress without diagonal reinforcement	6.3 kg/cm^2
d)	Shear stress with diagonal reinforcement	25.2 kg/cm^2
e)	Bond stress of deformed bar	8.4 kg/cm^2

Seismic Coefficient : K

K_H = 0.05 (Horizontal)
K_V = 0 (Vertical)

1.1.2 Intake Tower

Water Level of Dok Krai Reservoir

P.H.W.L. = + 54.10m
P.N.W.L. = + 52.10m
N.W.L. = + 50.60m
L.W.L. = + 42.00mm (As pump suction level)

(P.H.W.L.: Planning High Water Level)
(P.N.W.L.: Planning Normal Water Level)
(N.W.L. : Normal Water Level)
(L.W.L. : Low Water Level)
These are explained more precisely in 3.2.4 Intake Tower of Main Report

External Force

1) Wind force

wind velocity	V = 40 m/s
wind pressure	$q = 100\ kg/m^2$

2) Wave pressure

wave height	Hw = 2.5 m
wave length	L = 31.0 m
period	T = 4.5 sec

Unit Weight of Reinforced Concrete

$2.5\ t/m^3$

Concrete and Reinforcing Bar

Refer to 1.1.1

1.1.3 Intake Bridge

Bridge Type

Simple girder type shall be employed and the material to be used and span length shall be determined after economical comparison.

Bridge Length

L = 160 m

It may slightly be changed according to the span length.

Cross-Section

The cross-section of the bridge consists of two portions. One is for pedestrians and vehicles for administration only, and the other is for a pipeline.

The effective width for the former portion is 3.5 m, and the pipeline shall be supported by same girders as the former.

Design Load

Live Load	TL-14 loadings
Coefficient of Impact (L: Span Length in metric)	$i = 10/(25 + L)$
Wind Pressure	$q = 100\ kg/m^2$

Unit Weight — Prestressed concrete = 2.4 t/m^3
Reinforced concrete = 2.4 t/m^3

Range of Open Air Temperature — 10°C - 50°C

Alignment

Plan Alignment — R = 0

Longitudinal Alignment — 0.2187%

Angle of Skew — 90°00'00"

Prestressed Concrete

1) Allowable stress for concrete

Compressive strength after 28 days — $fc' = 350 \ kg/cm^2$

Modulous of elasticity — $E = 325{,}000 \ kg/cm^2$

Allowable bending compressive stress

a) Temporary-stress before loss by creep and shrinkage — $f_{ct} = 160 \ kg/cm^2$

b) Stress at design load after loss — $f_{ca} = 125 \ kg/cm^2$

Allowable bending tensile stress

a) Temporary stress before loss by creep and shrinkage — $f_{tt} = 13.5 \ kg/cm^2$

b) Stress at design load after loss — $f_{ta} = 13.5 \ kg/cm^2$

c) Stress at dead load after loss — $f'_{ta} = 0 \ kg/cm^2$

Allowable shear stress

a) Stress at design load — $f'_{sa} = 5 \ kg/cm^2$

b) Stress at ultimate load — $f_{su} = 46.5 \ kg/cm^2$

Allowable diagonal tensile stress — $f_{ia} = 9 \ kg/cm^2$

2) Creep coefficient — $\phi = 3.0$

3) Shrinkage coefficient — $\varepsilon = 20 \times 10^{-5}$

4) Prestressing steel

T 9.3 strands are employed.

Ultimate strength — $f'_s = 175 \ kg/mm^2$

Yield point f_y = 150 kg/mm^2

Allowable stress

a) At a time of initial prestress f_{sia} = 135 kg/mm^2

b) Temporary stress before loss by creep and shrinkage f_{sta} = 122.5 kg/mm^2

c) Stress at design load after loss f_{sa} = 105 kg/mm^2

Reduction coefficient by relaxation K = 0.09

Steel Girder

1) Allowable axial tensile stress fta = 1,400 kg/cm^2

2) Allowable axial compressive stress

$\ell/r \leq 20$ fca = 1,400 kg/cm^2

$20 < l/r < 93$ $fca = 1,400-8.4\ (l/r-20)\ kg/cm^2$

$l/r \geq 93$ $fca = 1.2 \times 10^7/[6,700+(l/r)^2] kg/cm^2$

l : member length (cm)

r : radius of gyration of area (cm)

3) Allowable bending stress fba = 1,400 kg/cm^2

4) Allowable shear stress fsa = 800 kg/cm^2

Substructures

1) Abutment

Reinforced concrete abutment with piles

2) Pier

Pile bent type

3) Pile

Prestressed concrete pile and steel pipe pile shall be compared

4) Allowable stress

For prestressed concrete pile

Refer to Prestressed Concrete

For steel pile

Bending tensile and compressive stress $f_{sa} = f_{ca}$ = 1,400 kg/cm^2

5) Allowable horizontal displacement of footing — a = 50 mm

6) Non-considerable thickness for corrosion of steel pipe pile (outside only) — t = 2 mm

Concrete and Reinforcing Bar

Refer to 1.1.1.

Pipeline

Diameter	D = 1,350 mm
Wall Thickness	t = 11.9 mm

1.1.4 Air Chamber

Location

From the intake tower wall to the center of the air chamber : 201 m

Study Case

At the time of pump trip caused by power suspension

Water Level

Dok Krai reservoir	L.W.L. + 42.00m
Air chamber	N.W.L. + 60.00m
Head tank	L.W.L. + 95.55m

Design Flow Rate

$Q = 2.62\ m^3/sec$

Foundation Work

Reinforced concrete piles are employed

Allowable Stress of Concrete and Reinforcing Bar

Refer to 1.1.1

Dead Load

Unit weight of reinforced concrete — 2.5 t/m^3

Other loads except airchamber vessels — 1.0 t/m^2

Coefficient of Earth Pressure against Wall

0.5 (earth pressure at rest)

Angle of Internal Friction of Bank

$\phi = 30°$

Ground Water Level

Same as the water level of the Dok Krai Reservoir.

N.W.L. = + 50.60 m
P.N.W.L = + 52.10 m
N.W.L. , P.N.W.L.: Refer to 1.1.2

1.1.5 Head Tank

Prestressed Concrete

1) Allowable stress for concrete

Compressive stress after 28 days $f'c = 350\ kg/cm^2$

Modulus of elasticity $E = 325{,}000\ kg/cm^2$

Allowable bending compressive stress

a) Temporary stress before loss by creep and shrinkage $f_{ct} = 170\ kg/cm^2$

b) Stress at design load after loss $f_{Ca} = 135\ kg/cm^2$

Allowable axial compressive stress

a) Temporary stress before loss by creep and shrinkage $f_{ct} = 132.5\ kg/cm^2$

b) Stress at design load after loss $f_{Ca} = 105\ kg/cm^2$

Allowable bending tensile stress

a) Temporary stress before loss by creep and shrinkage $f_{tt} = 13.5\ kg/cm^2$

b) Stress at deisgn load after loss $f_{ta} = 0\ kg/cm^2$

Allowable axial tensile stress

a) Temporary stress before loss by creep and shrinkage $f_{tt} = 0\ kg/cm^2$

b) Stress at design load after loss $f_{ta} = 0\ kg/cm^2$

2) Allowable stress for steel material

	strand ϕ 21.8	steel bar ϕ 32
Nominal cross-section area (cm^2)	3.129	8.042

Ultimate tensile stress (kg/cm^2)	18,660	11,000
Yield point (kg/cm^2)	16,140	9,500
Temporary stress before loss by creep and shrinkage (kg/cm^2)	13,060	7,700
Stress at design load after loss (kg/cm^2)	11,200	5,700

Concrete and Reinforcing Bar

Refer to 1.1.1

Structural Design Water Level

EL 103.40 m

Spill Way

1) Design flow rate $Q = 2.62 \ m^3/s$

2) Stream for diversion

Around 460 m upstream of the proposed pipeline from the head tank

3) Coefficient of roughness

For steel pipe $n = 0.014$

For reinforced concrete pipe $n = 0.017$

1.1.6 Pipeline

Pipe Material

Arc welded carbon steel pipes (JIS G 3457)

Allowable Stress

At the formula of pipe wall thickness in WSP 030-80*

Pipe wall	fa	= 1,400 kg/cm^2
Welded wall	fa'	= 1,400 kg/cm^2

* WSP: Japan Water Steel Pipe Association

Allowable Displacement Ratio

= 5% of pipe diameter
(5% x 135 cm = 6.75 cm)
(in case of tar-epoxy lining)

Allowable Stress of Concrete and Reinforcing Bar

Refer to 1.1.1

1.1.7 Receiving Facilities

Unit Weight of Material

Reinforced concrete	2.50 t/m^3
Plain concrete	2.40 t/m^3
Mortar	2.10 t/m^3

Dead Load

Surcharge load on the surrounding ground surface of the structures is 1.0 t/m^2

Coefficient of Earth Pressure

Earth pressure at rest	0.5
Active earth pressure	0.3

Allowable Stress of Concrete and Reinforcing Bar

Refer to 1.1.1

1.1.8 Architecture

Basic Code : AIJ Standard
(AIJ: Architectural Institute of Japan)

Design Method : Elastic Stress Analysis

: AIJ Standard for Structural Design of Steel Structure

: AIJ Standard for Structural Calculation of Reinforced Concrete Structure

: AIJ Standard for Structural Design of Building Foundation

Load Condition :

1) Seismic load: Negligible

2) Wind load : Wind Velocity Pressure 45 kg/m^2

: Wind force coefficient shall be in accordance with AIJ standards.

Basic Material Properties

1) Concrete : Allowable stress: refer to 1.1.1

2) Reinforcing : Round bar Allowable stress, 1,400 kg/cm^2

3) Structural steel : SS41 for normal weight steel
STK41 for light gauge steel

4) High tension bolt

: F10T

5) Normal bolt : SS41

Soil Bearing Capacity

10 ton/m^2 for permanent load

1.2 INTAKE FACILITIES

1.2.1 Intake Tower

In designing the intake tower, the prerequisite is to take water from existing Dok Krai reservoir. This intake tower has two main functions, one is to take surface water from the reservoir and the other is to pump up the water to obtain required head. Many types of intake tower are compared and two types mentioned below are studied closely before a particular type is adopted.

- Jacket type	Vertical shaft mixed flow pumps installed on a platform supported by steel pipe frame called "Jacket."
- Box caisson type	Vertical shaft centrifugal type pumps installed in a water proof underwater box concrete caisson tagged to destination.

It is concluded that jacket type is less advantageous to box caisson type. The details are quoted in Section 1.2.3, "Comparative Study on Jacket Type Intake Tower". Then, steel fabricated box caisson plan is studied as modified plan of concrete caisson. The comparison shows advantage of concrete caisson over steel caisson as mentioned in (4) Comparative Study on Steel Caisson.

Fig. 1-2-1 Type of Caisson

Box Caisson Type Jacket Type

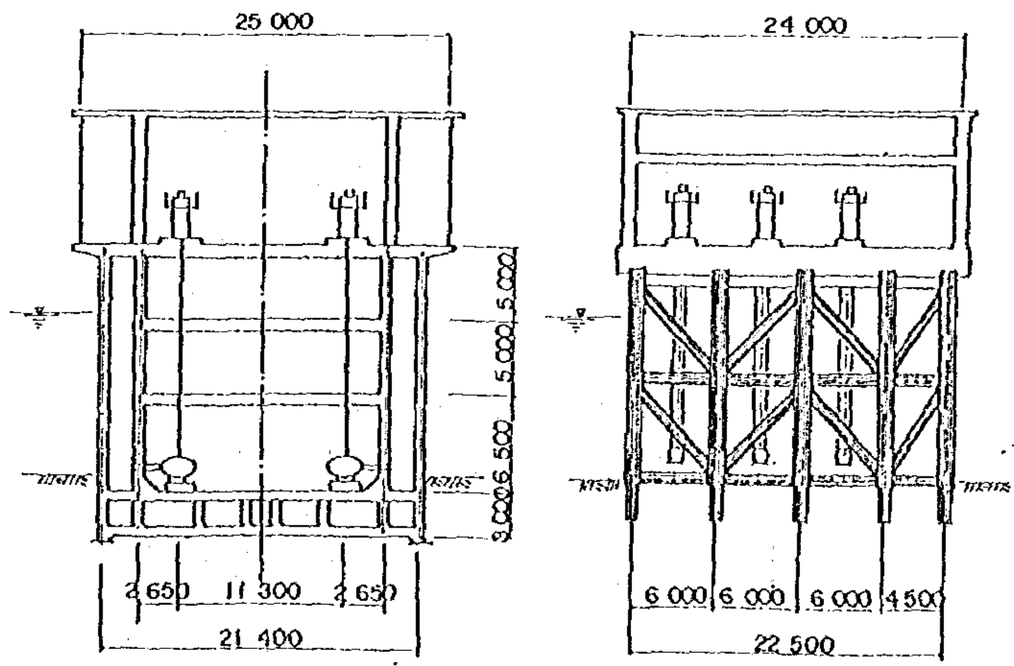
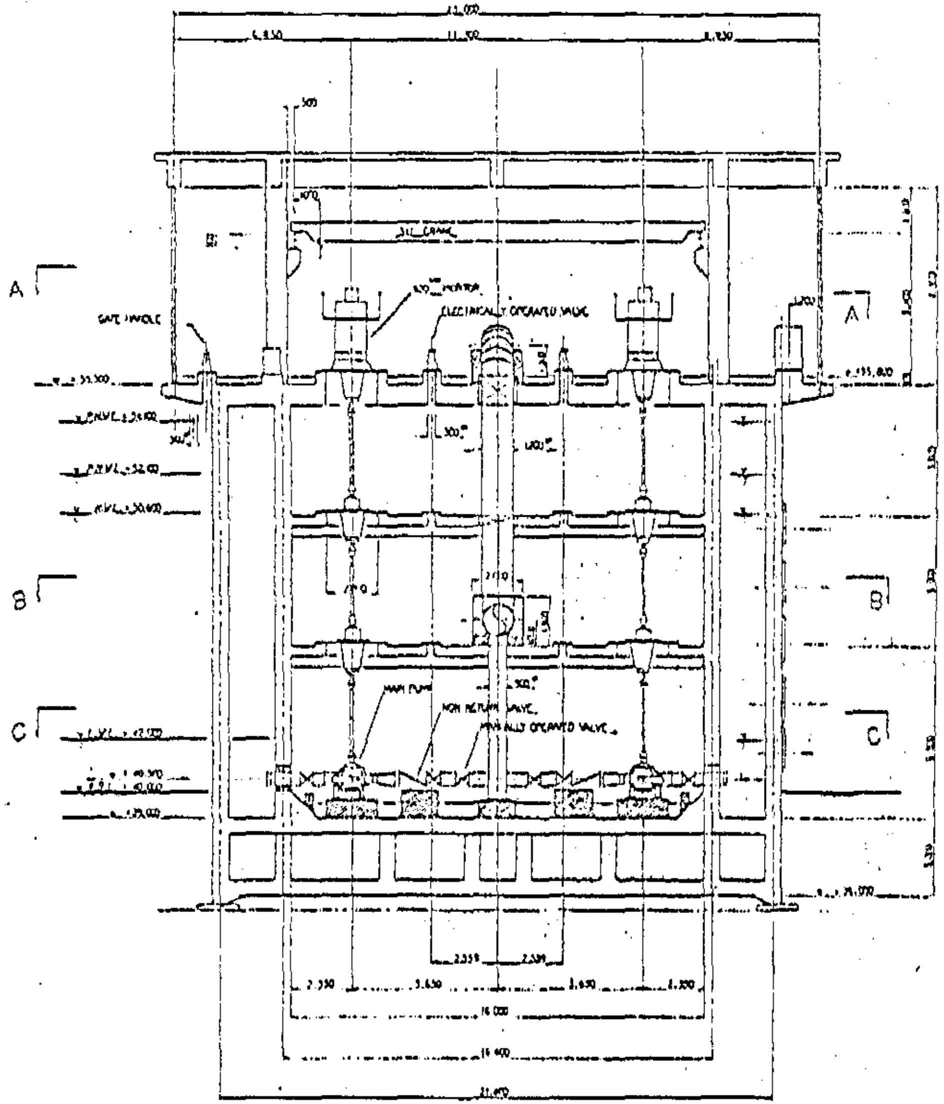


Fig. 1-2-2 Concrete Box Caisson Type



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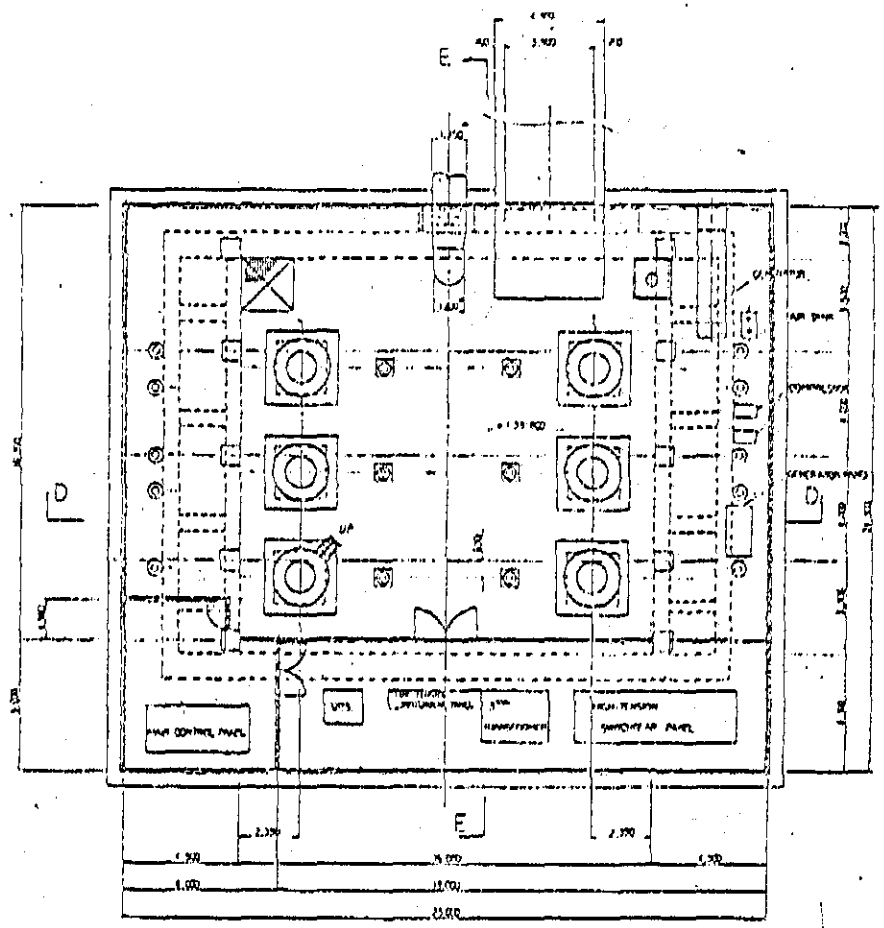


Plan

Preliminary Study on Caisson Type Intake Tower

As described in 1.1 "DESIGN CONDITIONS", design conditions to be taken into this design are complicated. Here, roughty estimated dimensions shall be determined by stability analysis of "Box Caisson" after completion without any calculations of structural details.

1) Loads

Dead loads

a) Box Caisson

Lower slab	=	1,870^t
Outer wall-1	=	775^t
Outer wall-2	=	1,492^t
Inner wall	=	1,085^t
Partition wall	=	349^t
Side wall in suction well	=	100t
(2nd+3rd)floor	=	600^t
Upper Slab	=	1,463^t
Total		7,734^t

b) Main installed machinery

Pumps and motors	=	140^t
Piping in the station	=	60^t
Electrical facilities	=	20^t
Miscellaneous	=	100t
Total		320t

c) Platform shed

shed	=	200
cinder concrete + base concrete	=	600 (400+200)
Total		800^t

b) Bridge reaction

Bridge reaction	=	50^t

Total 7,734 + 320 + 800 + 50 = $9,354^t$ $9,350^t$

Buoyancy

a) At time of storm

U_1	(Lower slab)	=	748^t
U_2	(Outer wall)	=	302^t
U_3	(Main body)	=	$4,621^t$
Total			$5,670^t$

b) At time of earthquake

U_1	(Lower slab)	=	748^t
U_2	(Outer wall)	=	262^t
U_3	(Main body)	=	$4,009^t$
Total			$5,020^t$

c) At time of draught (L.W.L.)

U_1	(Lower slab)	=	748^t
U_2	(Outer wall)	=	72^t
U_3	(Main body)	=	918^t
Total			$1,740^t$

Wind load and wave pressure

a) At the time of storm

- Wind load

Wind load acting on the structure is calculated by the following formula.

$$Hw = Hw \cdot A$$

Where ; Ww : Wind pressure (t/m^2)
$Hs \leq 15^m$ $Ww = 0.1\ t/m^2$
$Hs > 15^m$ $Ws = 0.15\ t/m^2$
A : Projected area (m^2)
Hs : Height of the structure (m)
Hw : Wind load (t)

$Hw = 0.1 \times 26.0 \times 10.0 = 26^t$
$Mw = 26.0 \times (59.5-37.0) = 585^t$

- Wave pressure

The force of wave acting on solitaly structure in the water is calculated by Morrison's formula. The caisson shall be assumed as cilindorical structure and maximum wave pressure (Fm max) and maximum moment (Mm max) shall be as follows.

$Fm\ max = Wo.C_D.D^2.H.K_D$
$Mm\ max = Fm\ max.\ (Sm+d)$

Where; Wo : Specific weight of water (=1.0)
C_D : Coefficient of mass
D : Diameter of cilinder (m)
H : Height of wave (H=2.5M)
K_D : Coefficient obtained by model test

Sm : Height of Fm max acting above reservoir bed (m)
d : Embedded depth of foundation (d=3.0m)

Here, Wave conditions are

H = 2.5 m/sec, L = 31 m, T = 4.5 m

and converted diameter of this structure is:

$$d = \sqrt{\frac{4}{\pi} \times 23.0 \times 17.0} = 22.3^m$$

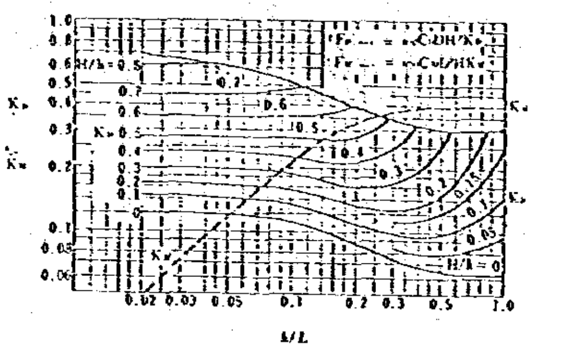
from the chart shown below:

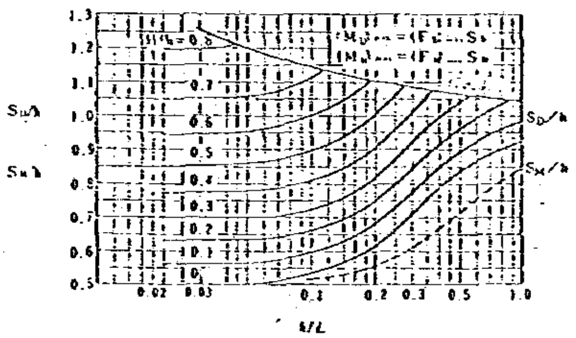
CM = 0.5
KM = 0.4
SM =11.3

Where:

$h/L = 14.1/31.0 = 0.455$
$D/L = 22.3/31.0 = 0.714$

Fig. 1.2-3 Design Chart for maximum drag and inertia force and moment





The wave pressure is calculated using abovementioned formula.

That is;

$$Fm\ max = Wo \cdot Cm . D^2 \cdot H \cdot Km$$
$$= 1.0 \times 0.5 \times 22.3^2 \times 2.5 \times 0.4$$
$$= 248.6 \fallingdotseq 250^t$$

$$Mn\ max = Fm\ max. \ (SM+d)$$
$$= 250 \times (11.3+3.0) = 3,580^{t \cdot m}$$

b) At time of earthquake

- Hydrodynamic pressure acting on columnar structure

- The hydrodynamic pressure on columnar structure caused by earthquake is to be calculated by the following formula.

$b/h \leq 2.0$
$Pd = 3/4 \cdot kh \cdot Wo \cdot b^2 \cdot h \cdot (1-b/4h)$
$hg = \frac{1}{2} h$

where: b : Width (=23^m)
h : Water depth (=12.1^m)
Wo : Specific weight of water (=1.0)
kh : horizontal seismic intensity (=0.05)
Pd : hydrodynamic pressure (t/m^2)
hg : height of pressure center acting above reservoir bed (m)

From the above formula, hydrodynamic pressure is

$$Pd = 3/4 \times 0.05 \times 1.0 \times 23.0^2 \times 12.1 \times (1-23.0/4\times12.1) = 126^t$$
$$hg = 1/2 \times 12.1 = 6.05 \text{ m}$$

therefore

$$Md = Pd \cdot (hg+d)$$
$$= 126 \times (6.05+3.0) = 1,140^{t \cdot m}$$

- Hydrodynamic pressure acting on the wall of suction well

Hydrodynamic pressure on wall is to be calculated by the following formula.

$$Pr = 7/12 \cdot kh \cdot Wo \cdot b \cdot h_2$$
$$hg = 1/2 \cdot (h+d)$$

where:
- kh : horizontal seismic intensity (=0.05)
- Wo : specific weight of water (=1.0)
- b : width of wall (2.0^m)
- h : water depth (=13.1^m)
- Pd : hydrodynamic pressure (t/m^2)
- hg : Height of pressure center acting above bed (m)

From the above formula, hydrodynamic pressure acting on the wall of suction well is

$$Pd = 4 \times 7/12 \times 0.05 \times 1.0 \times 2.0 \times 13.1_2$$
$$= 40^t$$
$$hg = 13.1/2 = 6.55^m$$

therefore ;

$$md = Pd \, . \, (hg+d)$$
$$= 40.0 \times (6.55+2.0) = 342^{t \cdot m}$$

Hydrodynamic pressure on paratition wall in the suction wells are estimated by the same way as above, here, height of the partition wall is assumed as of 2/3 h.

$$Pd = 8 \times \frac{2}{3} \times 10 = 53^t$$

$$Md = 53.0 \times (6.55+2.0) = 453^{t \cdot m}$$

c) At time of draught (L.W.L)

- Wind load

$$Hw = 0.1 \times 23.0 \times 15.0 + 0.15 \times 25.0 \times 7.5$$
$$= 63^t$$

$$Mw = 34.5 \times 12.5 + 28.1 \times 23.8$$
$$= 1,100^{t \cdot m}$$

Seismic loads

As for the hydrodynamic pressure, above mentioned formula shall be applied. Inertia force is calculated as below.

$$H = kh \cdot M$$

Where : kh : horizontal seismic intensity (=0.05)

M : Mass of structure (t)

From the above formula, inertia force is :

$$H = 0.05 \times 9,350 \doteqdot 468^{t}$$

Here, height of mass center acting above reservoir bed is assumed as hm = 14.0^{m}, then moment of inertia force is

$$M = H \times hm$$
$$= 468 \times 14.0 = 6,552^{t \cdot m}$$

2) Bearing capacity of foundation bed

As it is anticipated to be difficult to treat the surface of foundation bed, upper stratum of this decomposed granite must be replaced by underwater concrete to ensure close contact with the bottom surface of the lower slab.

Allowable bearing capacity of the foundation bed

Regarding the method of execution of construction, estimation of soil constant required in design must be so conservative as it is very difficult to make sufficient serveys and tests. Ultimate bearing capacity of the foundation bed considering the eccentricity and inclination of the load is to be obtained by the following formula.

$$Qn = A' \, (\alpha \cdot K_c \cdot Nc + K.q.N.q + \frac{1}{2} \cdot \gamma 1 . \beta \cdot B' . N \,)$$

Where : Qn : Ultimate bearing capacity of caisson bottom surface (t)

c : cohesion of the bed below the bottom surface of caisson (t/m^2)

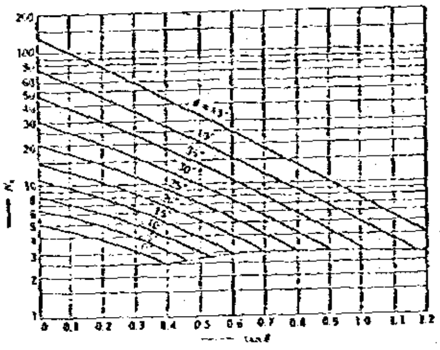
q : ground surface surcharge (t/m^2), $q = \gamma 2 \cdot Df$

A' : effective loading area of caisson bottom (t/m^2)

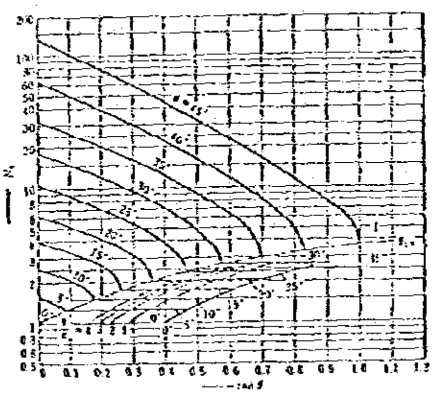
$\gamma 1, \gamma 2$: bulk density of the ground below the bottom surface of caisson (t/m^3)

B' : effective loading width of caisson bottom (m), B' = B-2eB

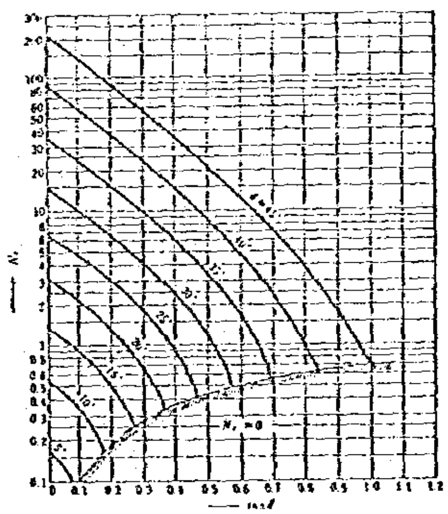
Fig.I-2-4 Design Chart for Coefficent of Bearing Capacity



Nc~tanθ



Nq~tanθ



Nr~tanθ

B : width of caisson bottom (m)

e : amount of load eccentricity (m)

Df : effective embedded depth (m)

α β : shape factors of the bottom surface of caisson $\alpha = 1.3$, $\beta = 0.6$

K : compensating coefficient for embedded portion $K = 1+0.3 \cdot Df/B'$

N_c, N_q, N_γ : coefficient of bearing capacity taking the inclination of the load into consideration given in Fig. 1-2-4.
here $\tan\theta = H_B/V$

V : vertical load acting on caisson bottom (t)

H_B : horizontal load acting on caisson bottom (t)

a) At time of storm

- External force

Vertical load ; V

$V = 9,350 - 5,670 = 3,680^t$

Horizontal load ; H_B

$H_B = 26 + 250 = 280^t$

Moment load at the center of caisson bottom ; M

$M = 585 + 1,580 = 4,170^{t \cdot m}$

- Ultimate bearing capacity

eB, B', K, A', $\tan\theta$, q

$eB = M/V = 4,170/3,680 = 1.1^m$

$B' = B-2eB = 17.0 - 2 \times 1.1 = 14.8^m$

$K = 1 + 0.3 \times Dt/B' = 1 + 0.3 \times 3.0/14.8 = 1.06$

$A' = B' \times W = 14.8 \times 23.0 = 340\ m^2$

$\tan\theta = H_B/V = 280/3,680 = 0.076$

$q = \gamma_2 Df = 1.0 \times 3.0 = 3.0\ t/m^2$

Bearing resistance of the foundation bed is obtained assuming bed cohesion (c) and internal friction angle (ϕ) of the foundation is assumed as

$$C = 0.0 \ t/m^2$$
$$\phi = 30$$

Ultimate bearing capacity is

$$Qu = A' (\alpha k \ c \ Nc + k \ q \ Nq + \frac{1}{2}\gamma_1 \ \beta \ B' \ N\gamma)$$

$$= 14.8 \times 12 \ (1.06 \times 3.0 \times 16 + \frac{1}{2} \times$$
$$1.0 \times 0.6 \times 14.8 \times 12)$$

$$= 35.400^t$$

Allowable bearing capacity is

$$Qa = 1/3 \ Qu$$
$$= 11,800^t \quad > \quad 3,680^t \qquad \text{O.K.}$$

b) At time of earthquake

- External force

Vertical load ; V

$$V = 9,350 - 5,020 = 4,330^t$$

Horizontal load ; H_B

$$H_B = 126 + 40 + 53 + 468 = 690^t$$

Moment load at the center of caisson botton ; M

$$M = 1,130 + 342 + 453 + 6,552 = 8,490^t$$

- Allowable bearing capacity

B, B', X, A', tanΘ , q

$$B = 8,490/4,330 = 2.0 \ m$$

$$B' = 17.0 - 2 \times 2.0 = 13.0 \ m$$

$$x = 1 + 0.3 \times 3.0/13.0 = 1.07$$

$$A' = 13.0 \times 23.0 = 299 \ m^2$$

$$\tan \Theta = 690/4,330 = 0.16$$

$$q = 1.0 \times 3.0 = 3.0 \ t/m^2$$

- Ultimate bearing capacity

$$Qu = 299 \times (1.07 \times 3.0 \times 13 + \frac{1}{2} \times 1.0 \times 0.6 \times 13.0 \times 8)$$
$$= 21,800^t$$

- Allowable bearing capacity

$Qa = 1/2\ Qu$

$= 10,900^t > 4,330^t$

c) At time of draught (L.W.L.)

- External force

Vertical load ; V

$V = 9,350 - 1,740 = 7,610^t$

Horizontal load; H_B

$H_B = 63^t$

Moment load at the center of caisson bottom ; M

$M = 1,100^{t.m}$

- Allowable bearing capacity

eB, B', K, A', tan Θ , q

$eB = 1,100/7,610 = 0.14$

$B' = 17.0 - 2 \times 0.14 = 16.7^m$

$K = 1 + 0.3 \times 3.0/16.7 = 1.05$

$A' = 16.7 \times 23.0 = 384\ m^2$

$\tan\Theta = 63/7,610 = 0.008$

$q = 1.0 \times 3.0 = 3.0\ t/m^2$

Ultimate bearing capacity

$$Qu = 384 \times (1.05 \times 3.0 \times 18 + \frac{1}{2} \times 1.0 \times 0.6 \times 16.7 \times 16.7 \times 14) = 48,700^t$$

Allowable bearing capcity

$Qa = 1/3 \cdot Qu$

$16,200^t > 7,610^t$

3) Sliding Resistance

The sliding resistance on the bottom surface of the caisson is calculated by the following formula

$Hu = C_B A' + V \tan\Theta\ B$

Where : C_B : cohesion between bottom surface of the caisson and foundation bed (t/m^2)

ϕ_B : frictional angle between bottom surface of the caisson and foundation bed (deg)

A' : effective load area (m^2)

V : vertical load (t)

Allowable sliding resistance is obtained by dividing above sliding resistance by safety factors.
Here, frictional angle is estimated at $\phi B = 2/3\phi$, and so

$C_B = 0.0\ t/m^2$

$\phi_B = 20$

At time of storm

- Ultimate sliding resistance

$Hu = V.\tan \theta\ B$

$= 3,680 \times \tan 20^\circ$

$= 1,339^t$

- Allowable sliding resistance

$Ha = 1/1.5 \times 1,339 = 890^t > 280^t$ O.K.

At time of earthquake

- Ultimate sliding resistance

$Hu = 4,330 \times \tan 20^\circ$

$= 1,575^t$

- Allowable sliding resistance

$Ha = 1/1.2 \times 1,575 = 1,310^t > 690^t$ O.K.

4) Displacement of Caisson

Displacement of caisson is calculated by the following formula;

Elastic settlement of caisson bottom center

$V_0 = 1/kv \cdot V/A$

where : V_0 : Elastic settlemnt of caisson bottom center (cm)

V : Vertical load acting on caisson bottom (kg)

A : Area of caisson bottom (cm^2)

kv : Coefficient of subgrade reaction (kg/cm^3)

Angle of swing of caisson

$\theta = M_B/I.kv$

where: θ : Angle of swing of caisson (rad)

M_B : Moment of external force at the center of caisson bottom (kg.cm)

I : Geometrical moment of inertia of caisson bottom (cm^4)

Lateral displacement

$\delta H = 1/ks \cdot H_B/x/L$

where: δH : Lateral displacement of caisson bottom (cm)

H_B : Horizontal load acting on caisson bottom (kg)

x : Effective reaction length (cm)

L : Effective width of caisson bottom (m)

ks : Coefficient of subgrade shear reaction (kg/cm^3)

Here, coefficient of subgrade reaction is assumed as below. Its blow count of the foundation mound is N = 20

$$kvo = \frac{1}{30} \times Eo$$
$$= \frac{1}{30} \times 1 \times 28 \times 20 = 18.7 \ kg/cm^3$$
$$kv = 12.8 \ kvo \cdot Bv^{-3/4}$$
$$= 12.8 \times 18.7 \times (17.0 \times 23.0)^{-3/4}$$
$$= 25.5 \ kg/cm^3$$
$$ks = 1/4 \cdot kv$$
$$= 6.4 \ kg/cm^3$$

Displacement of caisson is calculated by the abovementioned formula.

At time of storm

$$\delta V = 1/25.5 \times 3,680 \times 10^3/23.0 \times 10^2/17.0 \times 10^2$$
$$= 0.04 \ cm$$

$$I = bh^3/12 = 23.0 \times 17.0^3 \times 10^8/12 = 9.42 \times 10^{11} cm^4$$

$$\theta = 4{,}170 \times 10^5/9.42 \times 10^{11}/25.5$$
$$= 1.74 \times 10^{-5}$$

$$\delta H = 1/6.4 \times 28 \times 10^3/17.0 \times 10^2/23.0 \times 10^2$$
$$= 0.01 \text{ cm}$$

At time of earthquake

$$\delta V = 1/51.0 \times 4{,}330 \times 10^3/23.0 \times 10^2/17.0 \times 10^2$$
$$= 0.02 \text{ cm}$$

$$\theta = 8{,}490 \times 10^5/9.42 \times 10^{11}/51.0$$
$$= 1.77 \times 10^{-5}$$

$$\delta H = 1/12.8 \times 690 \times 10^3/17.0 \times 10^2/23.0 \times 10^2$$
$$= 0.01 \text{ cm}$$

At time of drought

$$\delta V = 1/25.5 \times 7{,}610 \times 10^3/23.0 \times 10^2/17.0 \times 10^2$$
$$= 0.08 \text{ cm}$$

5) Summary of Preliminary Study on Caisson Type Intake Tower

From brief study on stationery caisson type intake tower, the following are concluded.

- This caisson is stable against bearing, overturning and sliding at the time of storm, earthquake and drought, without any piles and rock anchors.
- As for the amount of settlement or displacement, it is permissible small.
- More detailed study is needed for determining structural details.
- More detailed stability analysis is needed including the one for construction stage.

Comparative Study on Jacket Type Intake Tower

1) General

Jacket type foundation is selected as an alternative to box caisson type foundation. Comparative study of intake tower is carried out and it is concluded that jacket type is inferior to box caisson type from the reason mentioned below.

- As for pumps to be installed in the station there are;

  Box-caisson type ------ Vertical shaft double suction volute pump

  Jacket type ----------- Vertical shaft mixed flow pump.
  Comparative study concluded advantage of centrifugal pump; details are shown in 1.7.

- Generally speaking, steel structure like jacket type structure can be used for 30 years, on the other hand, concrete caisson can bear more than 50 years.

- Short piles foundation may cause some problems on axial bearing capacity, lateral bearing capacity and axial pull-out capacity.

- Impact uplift pressure caused by wave during storm is very difficult to take into design calculation. It affects much on stability of jacket foundation, it is unavoidable load though.

- As for the construction schedule, there is no much difference of construction periods between Caisson type and Jacket type.

- As for the construction cost, jacket type plan is approximately the same as the caisson type plan. In case of caisson type plan, caisson yard is usefal for other purpose such as landing pier for temporaly use. From this point of view, jacket type plan is inferior to caisson type in construction cost too.

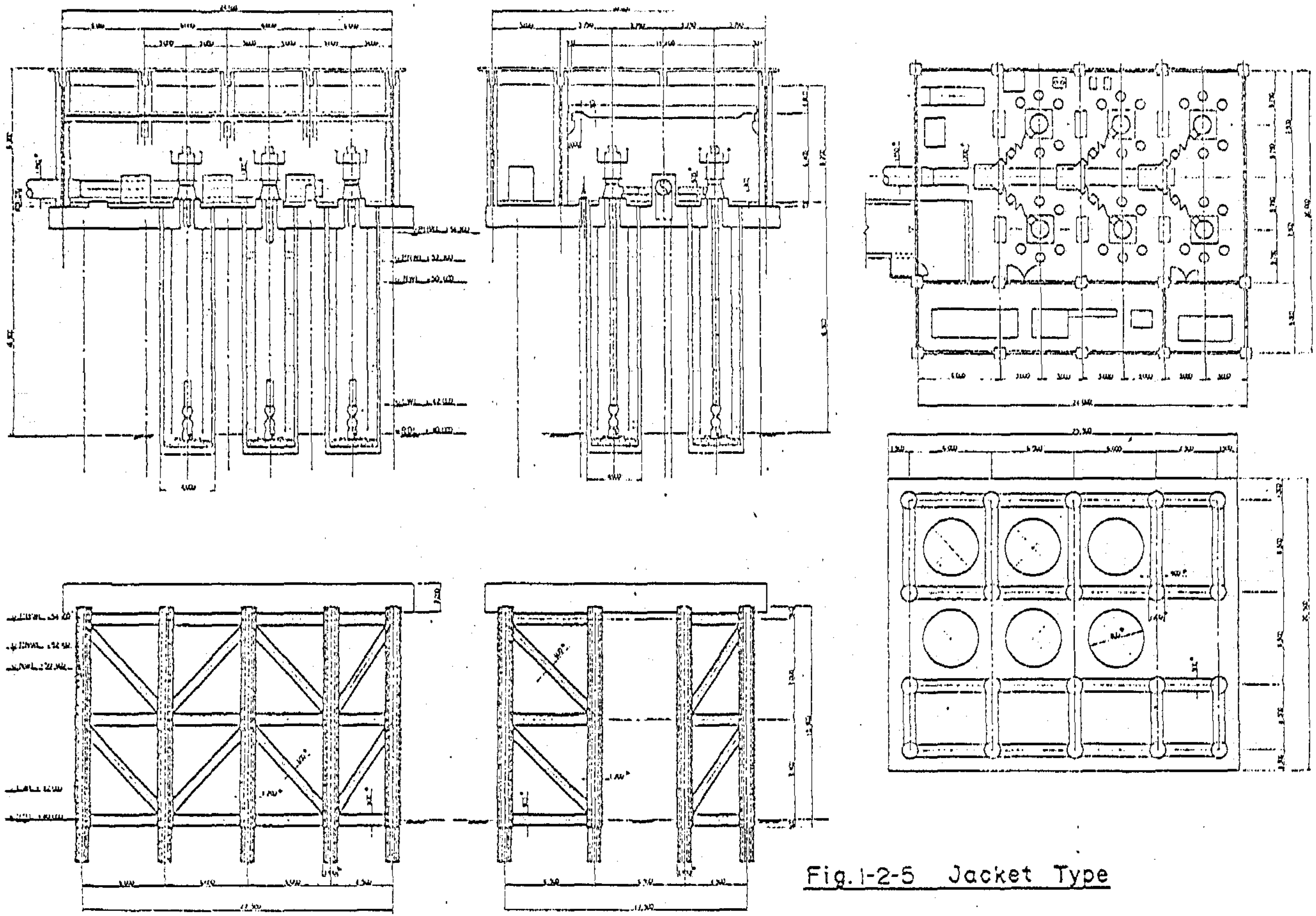


Fig.1-2-5 Jacket Type

2) Preliminary structural analysis

Load

a) Dead load

Pump	6×20	=	120 t
Floor slab $(25.5 \times 20.5 - 6 \times \frac{\pi}{4} \times 4.0^2) \times 2.0 \times 2.5$		=	2,240 t
Pipe, valve		=	30 t
Miscellaneous ($q = 1.0\ t/m^2$)		=	510 t
			2,900 t

Casing $\phi = 4{,}000 \quad t = 250$

$$w = \frac{\pi}{4} \times (6.0^2 - 3.5^2) \times 16.0 \times 2.5 = 118t/No.$$

$$u = \frac{\pi}{4} \times (6.0^2 - 3.5^2) \times 13.5 \times 1.0 = -40t/No.$$

b) Wind load and wave pressure

- At time of storm

Wind load

$$Hw = 25.5 \times 11.0 \times 0.1 = 28\ t$$

Wave pressure on casing

$$Fo = 1.0 \times 1.0 \times 2.5 \times 0.125 \times 4.0 = 3.1\ t$$

$$Fm = 1.0 \times 2.0 \times 4.0^2 \times 2.5 \times 0.4 = 32.0\ t$$

35.1t/No.

Wave pressure on Jacket

$$Fo = 1.0 \times 1.0 \times 2.5^2 \times 0.125 \times 1.2 = 0.9\ t$$

$$Fm = 1.0 \times 2.0 \times 1.2^2 \times 2.5 \times 0.4 = 2.9\ t$$

3.8t/No.

- At time of earthquake

Hydrodynamic pressure on casing

$$Pc = 2 \times 1.0 \times 0.05 \times 1.0 \times 48.4 \times 1.0 \times$$

$$(1 - 4.0/4 \times 12.1)/12.1^{1/3} \cdot [h]^{1/3}$$

$$= 1.934 \times \int^{12.1} [h]^{1/3}\, dh$$

$$= 17.9 \qquad\qquad = 18t/No.$$

Hydrodynamic pressure on jacket leg

$$Pj = 1.0 \times 0.05 \times 1.0 \times 12.1 \times 1.2 \times 1.0 \times$$

$$(1 - 1.2/4 \times 12.1)/12.1^{1/3} \cdot h^{1/3}$$

$$= 0.708 \int^{12.1} [h]^{1/3}\, dh$$

$$= 6.6 \times 1.5 \text{ (Bracing)} \qquad\qquad = 9.8t/No.$$

c) Inertia force

- Body force

$$H_B = 1900 \times 0.05 \qquad\qquad = 145\ t$$

Casing

$$H_C = 118 \times 0.05 \qquad\qquad = 6.0t/No.$$

Jacket

$$H_J = (\overline{W}_J + \overline{W}_W) \times 0.05$$

here; $\overline{W}_J$: Unit weight of jacket member
$\overline{W}_W$: Unit weight of water in member

Bearing Capacity of Pile

a) Allowable bearing capacity of pile

Allowable bearing capacity of pile is obtained by the following formula.

$$Ra = 1/n \cdot Ru$$

n; safety factor
Ru; ultinate bearing capcity

$$Ru = qd \cdot A + U \cdot li \cdot fi$$

A; cross sectional area of pile tip (m^2)

qd; ultimate bearing capacity per unit area of foundation bed at the pile tip (t/m^2)

U; Perimeter of pile (m)

li; Thickness of soil stratum (m)
fi; Maximum skin friction of soil (t/m^2)

Here, Pile dimensions and soil conditions are as follows,

- Pile
  - D = 1100 mm
  - A = 0.95 m^2
  - U = 3.46 m

- Soil
  - li = 3.0 m
  - fi = 5 t/m^2
  - $\overline{N}$ = 40 (Maximum blow count value for design)

And, assumed ratio of penetration depth by pile diameter is 2.0 m. From the chart shown below

$$qd = 1 \quad x \ \overline{N}$$

here;

$\overline{N}$; Mean blow count for design

Ultimate bearing capacity is

$$Ru = 18 \times 40 \times 0.95 + 3.46 \times 3.0 \times 5.0$$

$$= 684 + 52 = 736 \text{ t}$$

$$Ra = 1/3 \times 736 = 245 \text{ t} \quad \text{(at normal case)}$$

$$Ra = 1/2 \times 736 = 368 \text{ t} \quad \text{(at time of earthquake)}$$

b) Allowable lateral bearing capacity

The pile foundation is composed of short piles because the bed rock of granite foundation is very shallow. Upper granite layer is stiff but it is fragile because granite is vulnerable to weathering. Lateral bearing capacity is estimated 60 t by calculating passive earth pressure using trial and error method.

Design of jacket by frame structure

Two dimensional frame structural analysis is useful for the jacket type foundation. Here, A matrix approach for obtainning stress in each member is considered.

a) Displacement and force

Displacement and Force

	Unit	STORM		EARTHQUAKE	
		Node	Amount	Node	Amount
Displacement (X)	cm	5,6,15,16	1.70	5,6,15,16	2.25
Max - Reaction (Y)	t	20	241	20	279
Min - Reaction (Y)	t	1	72	1	53
Max - Reaction (X)	t	20	54	20	70

Refering to above table, conclusions are as follows;

- Displacement is small enough, so it has no harmful effect on pump shaft if it is installed correctly. ($1.70/1500 = 1.13 \times 10^{-3} < 5.0 \times 10^{-3}$)

- Bearing capacity is in the state of the highest possible amount, and stablity of the pumping station depend much on construction technique.

- As for lateral bearing capacity, it seems very diffecult to get enough bearing capacity for the amount shown above because of short-pile foundation.

b) Member stress

Comparing 2 cases, stress level of at the time of storm is more strict than that of at the time of earthquake. Refering to the table shown below, stress in each member is smaller than the allowable stress.

Member stress

	Member	Bending Moment (t·m)	Stress (kg/cm2)
Pile (ø = 1100)	11	64.1	958
Jacket leg (ø = 1200)	26	57.0	715
Jacket brace (ø = 900)	22	39.1	1,024
Jacket brace (ø = 700)	10	5.3	285
Slab	5	269.2	O.K.

Fig. 1-2-7 Frame Model

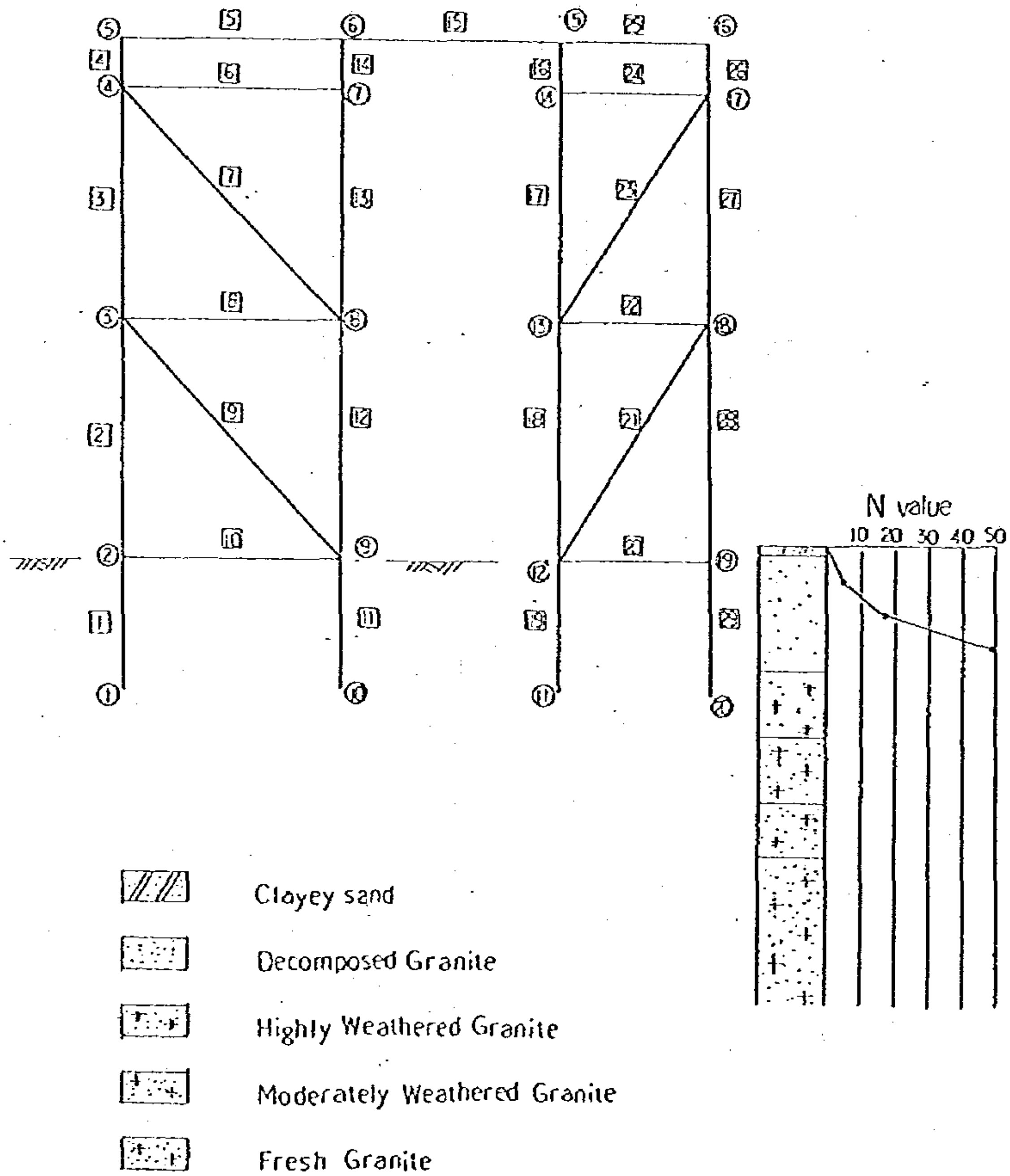
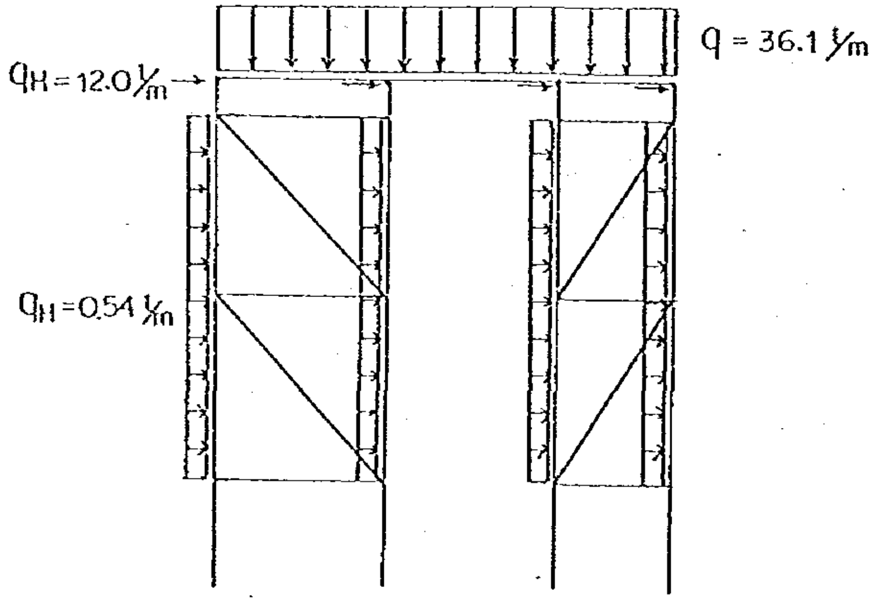


Fig. 1-2-8 Load Diagram

At the time of Storm



At the time of Earthquake

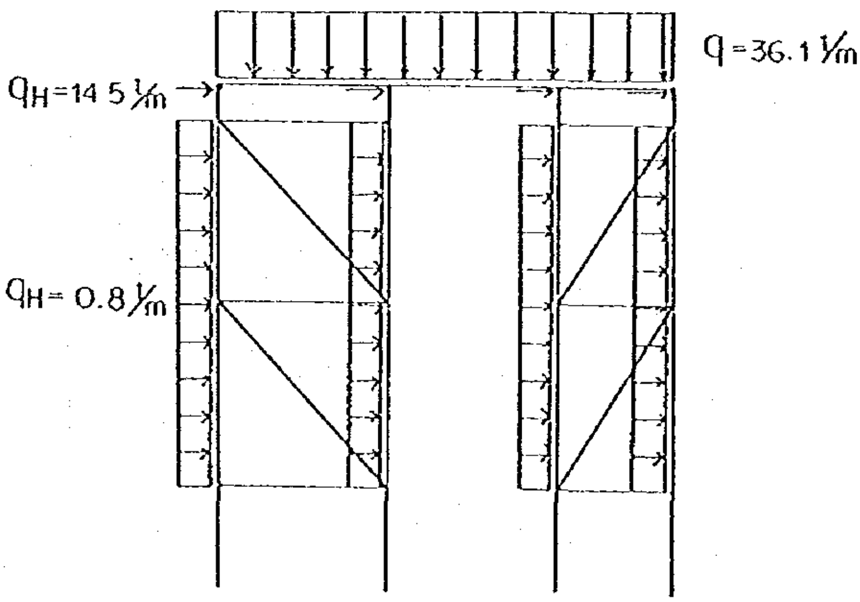
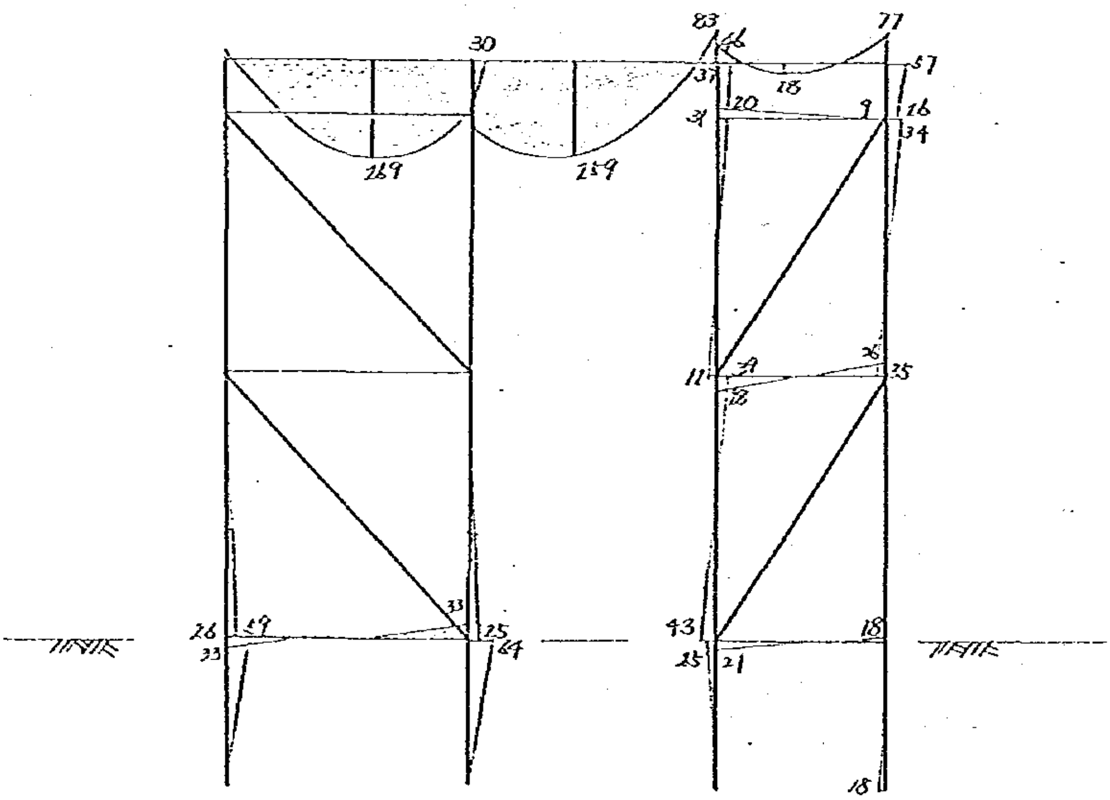


Fig. 1-2-9 Bending Moment Diagram (t·m)

(At the time of Storm)



3) Bill of Quantity

Jacket type foundation

Item	Quantity	Amount
Steel Pile ∅ = 1,100 t = 9	380 m	93.5 t
Jacket Leg ∅ = 1,200 t = 9	310 m	83.4 t
H. Brace ∅ = 900 t = 8	660 m	118.1 t
D. Brace ∅ = 700 t = 7	450 m	54.9 t
Base Floor D = 2.0 m	900 m^3	900 m^3
R.C.Casing ∅ = 4,000 t = 25	6 units	290 m^3

- Case-1 Steel pile foundation (∅ = 1,100)

  Steel Pipe W = 350^t

- Case-2 Reverse pile foundation (∅ = 1,100)

  Steel Pipe W = 255^t

  Reverse Pile L = 20^m

4) Construction Cost

Direct Construction Cost

x 10^6 ฿

	Jacket Type	Caisson Type
Caisson Yard	5	10
Intake Tower	38*	36**
Pump and Machinery	45	40
Electric Facilities	26	26
Total	114	112

* Jacket W = 350^t
** Caisson

Comparative Study on Steel Caisson

1) General

Steel fabricated caisson is considerable for one of the alternative plan of concrete box caisson. Generally, steel caisson is a caisson composed of steel on the whole. Here, a kind of steel form (refer to Figs. 1-2-10, 1-2-11 and 1-2-12) named steel caisson is studied because of the reasons mentioned below;

- Excavation depth of caisson yard will be shallow, so sheet-pilling work will be easy.
- Because of it's light weight, it is easy to tug the Caisson and settle it in the destination.
- As the steel caisson is able to fabricate at factory, less fabrication work at the site may shorten construction period of pumping station.

With the mentioned reasoning, the cassion body is the same as the original plan except the little change of base floor.

2) Design of steel caisson

Design condition

a) Load

Base plate: $q = 14.0\ t/m^2$ (At the time of settling)

Wall plate: $q = 2.5\ t/m^2$ (At the time of tugging)

b) Allowable stress

$\sigma a = 2100\ kg/cm^2$

Design of base plate

a) Main girder H-900 x 300 x 10 x 28 @ 2.0^{m}

- Bending moment (simple beam)

$M = ql^2/8 = 2.0 \times 14.0 \times 7.0^2/8 = 172^{\ t.m}$

- Stress

$\sigma = M/Z = 172 \times 10^5/9140 = 1882\ kg/cm^2$

b) Transverse stiffener H - 450 x 200 x 9 x 14 @ 2.0^{m}

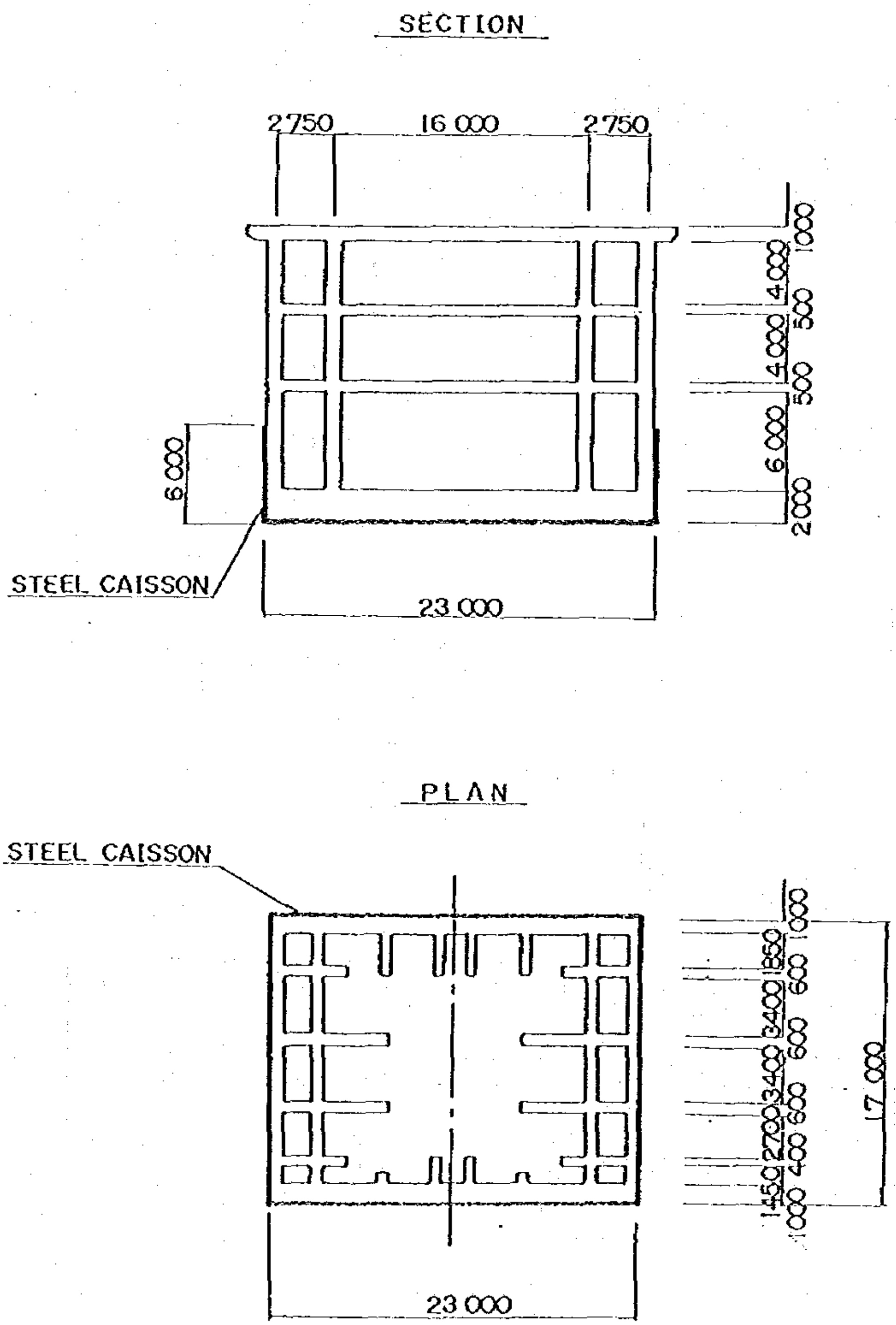
- Bending moment (simple beam)

$M = ql^2/8 = 2.0 \times 14.0 \times 2.0^2/8 = 14^{\ t.m}$

- Stress

$\sigma = M/Z = 14.0 \times 10^5/1490 = 940\ kg/cm^2$

Fig. 1-2-10 General Plan of Steel Caisson



c) Stiffener L - 250 x 90 x 10 x 15 @ 1.0 m

- Bending moment (simple beam)

$M = ql^2/8 = 14.0 \times 2.0^2/8 = 7.0$ t.m

- Stress

$\sigma = M/Z = 7.0 \times 10^5/360 = 1994$ kg/cm^2

Design of wall plate

a) Main girder I - 300 x 150 x 8 x 13 @ 4.0 m

- Bending moment (cantilever)

$M = ql^2/6 = 4.0 \times 2.5^2/6 = 10.4$ t.m

- Stress

$\sigma = 10.4 \times 10^5/633 = 1646$ kg/cm^2

b) Stiffner L - 200 x 90 x 9 x 4 @ 1.0 m

- Bending moment (simple beam)

$M = ql^2/8 = 2.5 \times 4.0^2 = 5.0$ t.m

- Stress

$\sigma = 5.0 \times 10^5/260 = 1923$ kg/cm^2

Skin plate

Thickness of the skin plte will be t=9.0 mm.

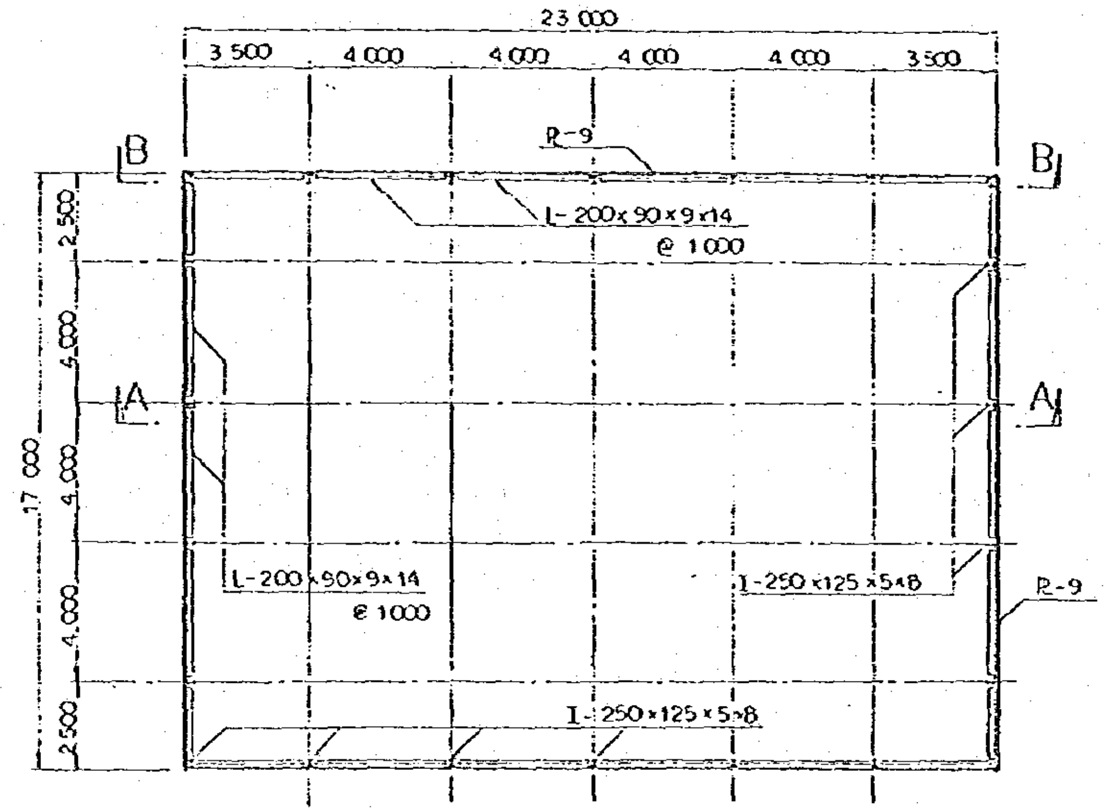
3) Bill of quantity

Steel

Steel	Quantity	Unit weight	Amount (t)
PL - 9	871 m^2	70.65 kg/m^2	61.5
H - 900 x 300 x 16 x 28	102 m	243 kg/m	24.8
H - 450 x 200 x 9 x 14	90 m	76 kg/m	6.8
I - 300 x 150 x 8 x 13	132 m	48.3 kg/m	6.4
L - 250 x 90 x 10 x 15	228 m	29.4 kg/m	6.7
L - 200 x 90 x 9 x 14	480 m	23.3 kg/m	11.2
Total			117.4

Fig. 1-2-11 Structural Details of Steel Caisson – I

PLAN



A — A

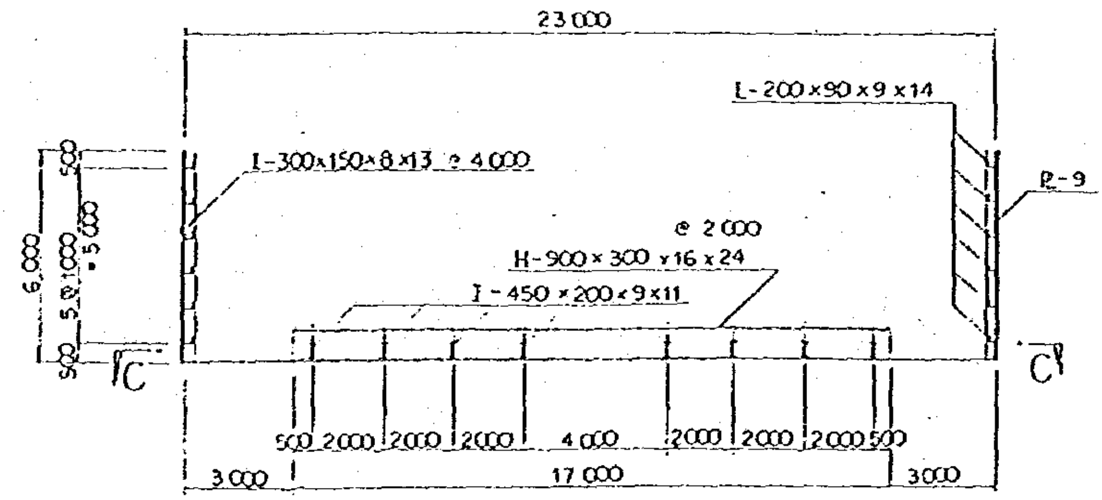
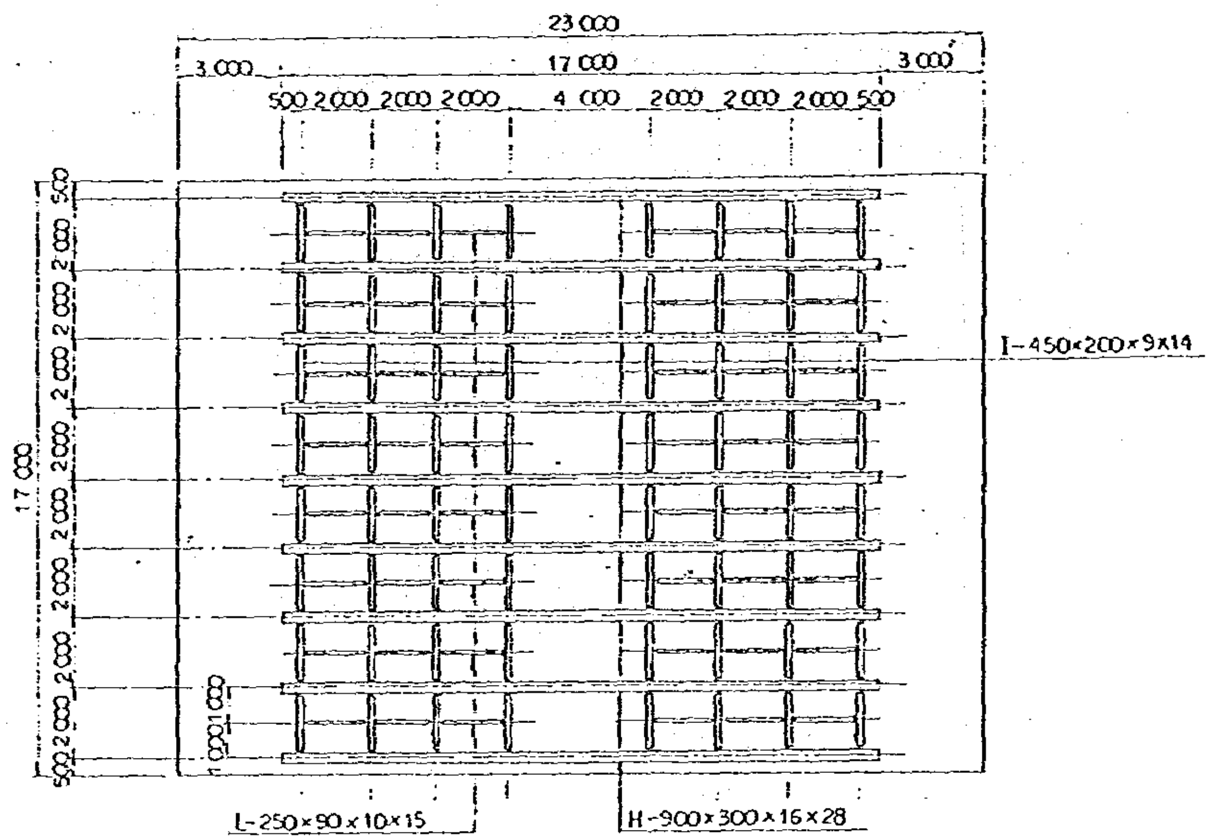
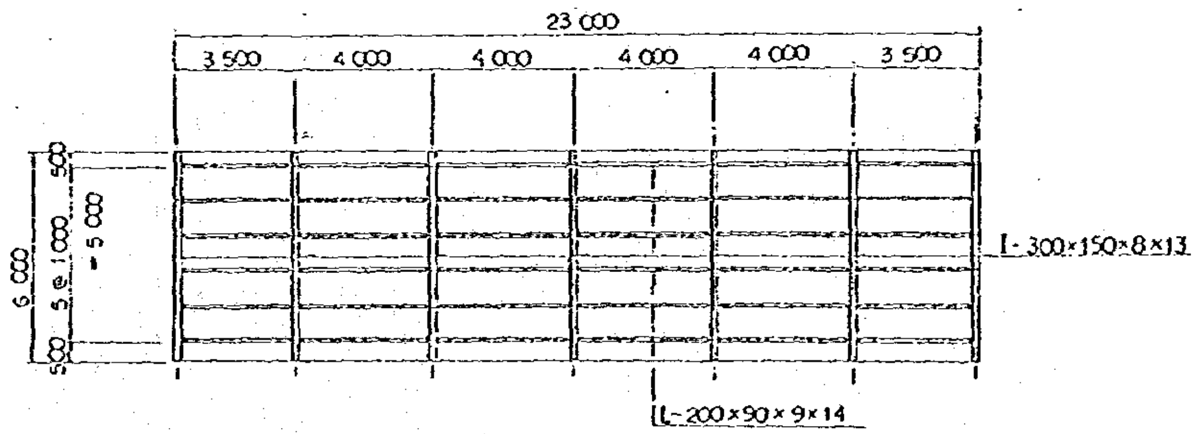


Fig.1-2-12 Structural Details of Steel Caisson - 2

PLAN



B — B



4) Comparative study of steel caisson and concrete caisson

Construction cost

As the body of the caisson is the same as the concrete one, limitted comparative study is enough to know the difference between them. Here, comparative study of direct construction cost of steel caisson and caisson yard is carried out.

Construction cost

Steel Caisson			Concrete Caisson		
Item	Quantity	Cost ($\times 10^6$ ฿)	Item	Quantity	Cost ($\times 10^6$ ฿)
Steel Material	118 t	1.2	Caisson Yard		10.0
Fabrication	118 t	4.1	Earth W.	6,000	(2.6)
Caisson Yard		4.6	Sheetpile	l=12 m	(7.4)
Earth W.		(0.9)			
Sheetpile	l=6 m	(3.7)			
Total		9.9	Total		10.0

From above table it is concluded that the construction cost of each plan is almost the same.

Result of the study

As for the type of caisson, due to the reasons below, steel caisson is inferior to concrete caisson.

- construciton cost of each plan is almost the same because steel form is used only for temporary structure.

- Use of steel as permanent members like reinforcing bar is very difficult because the construciton work is under limitted condition.

- As for the construciton period, concrete placing work on the reservoir will drag the period, so there may not be much difference between the two type.

- Because of it's enough depth, caisson yard for concrete caisson is useful for a landing pier for barges and boats during construction of pumping station and brige.

Stability Analysis on Caisson Type Intake Tower

1) General

In the design of this type stationary box caisson, two kinds of analysis is needed. One is the calculation during the construction, and the other is the calculation after completion. This paper is prepared for the stability analysis of caisson after completion.

It is concluded that the caisson is stable with all study cases taken into consideration.

2) Design condition

Study case

Study Case

CASE	CASE NO.	WATER LEVEL	LOAD COMBINATION
In normal case	1-1	P.N.W.L.	P
	1-2	L.W.L.	P
At the time of earthquake	2	P.N.W.L.	$P + E + W_D$
At the time of storm	3	P.H.W.L.	$P + W + W_p$
	4	L.W.L.	$P + W_p$

Here;

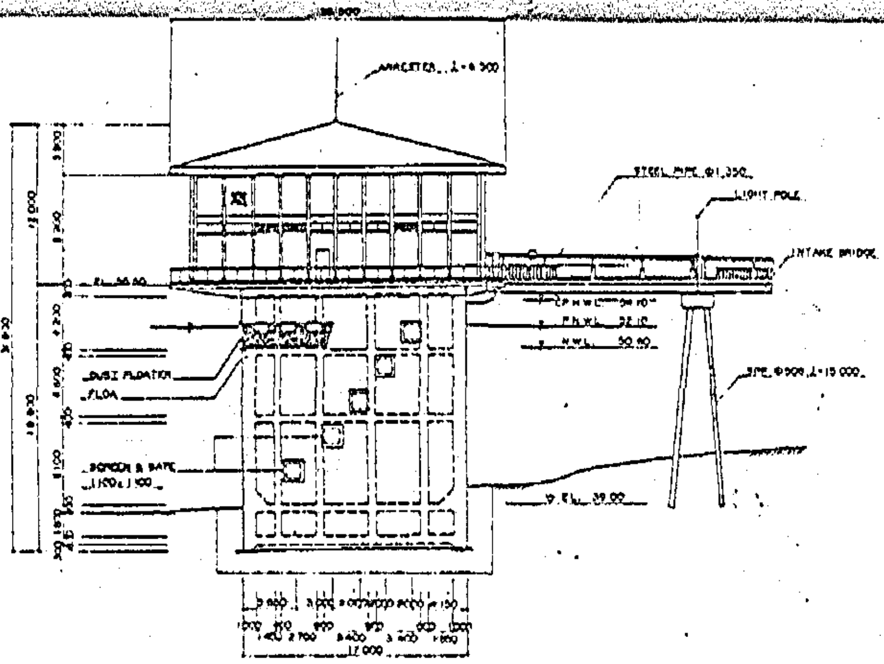
P;		Principal Load
E;		Earthquake force
W_D;		Dybanic wave pressure
W;		Wave pressure caused by wind
W_p;		Wind pressure
P.H.W.L.	=	+ 54.100 m
P.N.W.L.	=	+ 52.100 m
L.W.L.	=	+ 42.000 m

Allowable bearing capacity of foundation bed

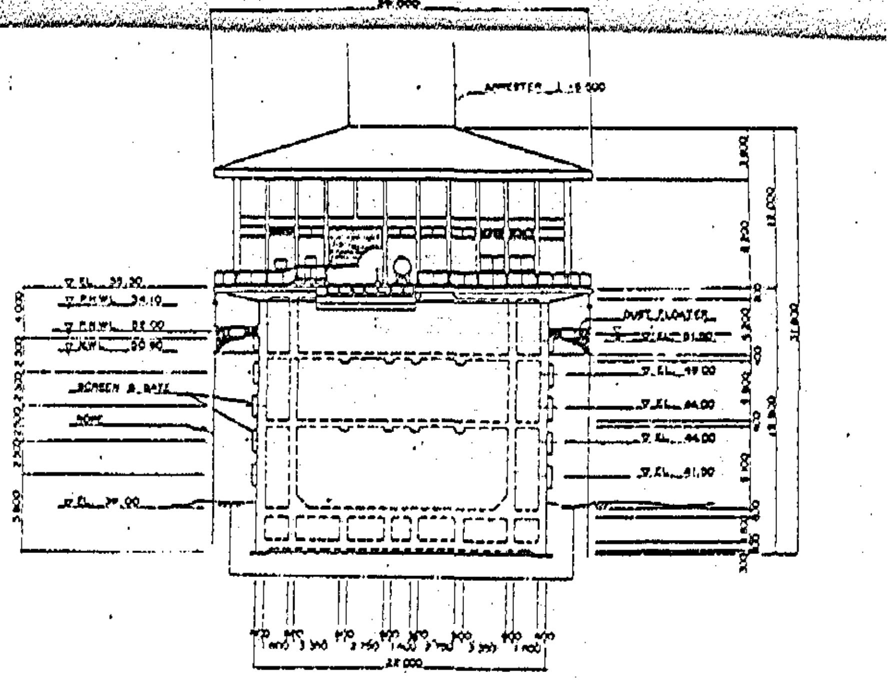
In normal case $\quad Qa = 60\ t/m^2$

At Time of earthquake $\quad Q'a = 90\ t/m^2$

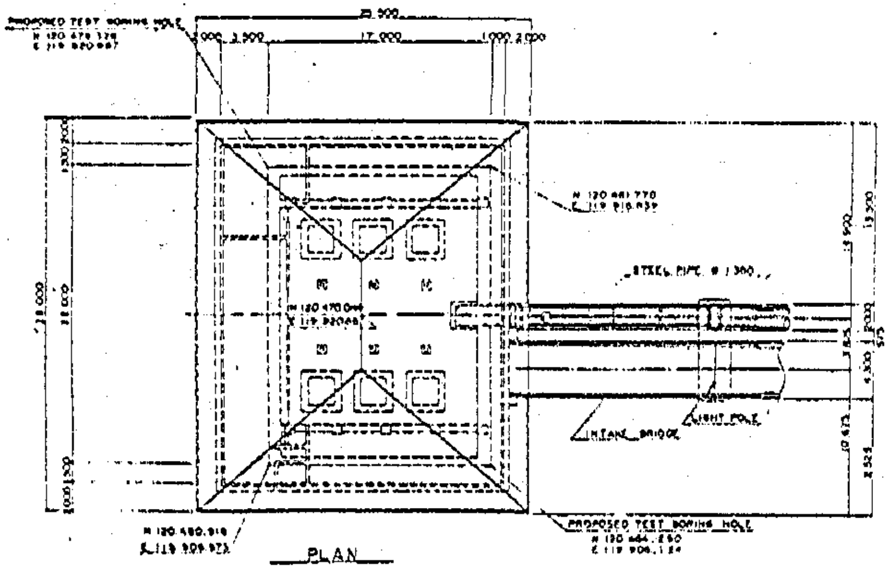
At Time of storm $\quad Q''a = 75\ t/m^2$



WEST ELEVATION



SOUTH ELEVATION



PLAN

P.H.W.L. : PLANNING HIGH WATER LEVEL
P.N.W.L. : PLANNING NORMAL WATER LEVEL
N.W.L. : NORMAL WATER LEVEL

Fig.1-2-13 General Plan of Intake Tower

Safety factor

Safety Factor

	Normal	Eearthquake	Storm
Sliding	1.2	1.0	1.1
Overturning	e<B/6	e<B/6	e<B/6

Here; e; Eccentricity of Caisson Bed
B; Width of Caisson Bed

3) Result of study

Uplift

Safety Factor

CASE	VERTICAL LOAD (t)	UPLIFT (t)	V-U (t)	S.F.
1-1	11,185	6,022	5,163	1.9
1-2	10,757	2,244	8,513	4.8
2	11,185	6,022	5,163	1.9
3	11,277	6,770	4,507	1.7
4	10,757	2,244	8,513	4.8

Overturning and sliding

Stability against Overturning and sliding

CASE		OVERTURNING	S.F. FOR SLIDING
2	L.W.	1.36m < 3.70m	3.8
	S.W.	1.99m < 2.83m	3.8
3	L.W.	1.90m < 3.70m	8.5
	S.W.	1.56m < 2.83m	8.3
4	L.W.	0.12m < 3.70m	8.3
	S.W.	0.48m < 2.83m	6.8

Here; L.W.; Longer Width of Foundation
S.W.; Shorter Width of Foundation

Ground reaction

Ground Reaction

(t/m^2)

		Ground Reaction		Allowable Ground Reaction
		q max	q min	
1-1	L.W.	13.8		60
	S.W.	16.6	11.1	
1-2	L.W.	22.8		60
	S.W.	25.5	20.0	
2	L.W.	18.9	8.7	90
	S.W.	23.5	4.1	
3	L.W.	15.0	9.1	75
	S.W.	18.9	5.2	
4	L.W.	23.4	22.0	75
	S.W.	26.6	18.9	

4) Stability analysis on caisson

Calculation of load

a) Body force

Body force of caisson is as shown below.

Body force of Caisson

	Weight (t)
Footing concrete	2,181
Outer and inner wall	3,609
Beam and slab 2F	292
3F	282
4F	1,303
Column 2F	302
3F	45
4F	45
	912
Total	8,971

b) Machinery load

	Weight(t)	Remarks
1F	67.8	Pumps, valves, pipes, etc.
2F	39.1	Bearings, pipes
3F	19.8	Bearings, pipes
4F	166.4	Motors, Crane, Gear panels, etc.
Total	294	

c) Pump house load

$W = 1{,}000t$

d) Reaction of bridge

$R = 50t$

e) Water load in the suction sump

	Water load (t)
P.H.W.L	659
P.N.W.L	567
L.W.L	139

f) Uplift

	Uplift (t)
P.H.W.L	6770
P.N.W.L	6022
L.W.L	2244

g) Wind load

Wind load acting on the structure is calculated by the following formula.

$Hw = Ww \cdot A$

Where ; Ww ; Wind pressure (t/m^2)

$Hs < 15m \quad Ww = 0.1\ t/m^2$

$Hs > 15m \quad Ww = 0.15\ t/m^2$

A ; Projected area (m^2)

Hs ; High of the sturcture

Hw ; Wind load (t)

From the above formula, wind load acting on shorter width of pumping house is

$Hw = 0.1x(9.0x21.5+1.4x17.0)x(0.9+0.3) = 26.0\ t$

h) Wave pressure

The force of wave acting on the solitaly structure in the water is calculated by Morrison's formula
The caisson shall be assumed as cylindrical structure and maximum wave pressure (Fm max) and maximum moment (Mm max) shall be as follows.

$Fm\ max = Wo.C_D.D^2.H.K_D$

$Mm\ max = Fm\ max.(Sm+d)$

Here W_o ; Specific weight of water (= 1.0 t/m^3)
C_D ; Coefficient of mass
D ; Diameter of cilinder (m)
H ; Height of wave (H = 2.5m)
K_D ; Coefficient obtained by model test
Sm ; Height of Fm max acting above reservoir bed (m)
d ; Embedded depth of foundation (d=3.0m)

Putting wave conditions,

H = 2.5 m/sec, L = 31 m, T = 4.5 sec

and converted diameter of this structure is ;

$$d = \sqrt{4 / \pi \times 22.0 \times 17.0} = 21.8 \text{ m}$$

from the chart shown below ;

CM = 0.5
KM = 0.4
SM = 11.3

Where ;

h/L = 14.1/31.0 = 0.455
D/L = 21.8/31.0 = 0.703

The wave pressure is calculated using abovementiond formula. That is

$$\begin{aligned} Fm\ max &= Wo.Cm.D^2.H.Km \\ &= 1.0 \times 0.5 \times 21.8^2 \times 2.5 \times 0.4 \\ &= 237.6 \text{ ---- } 240 \text{ t} \\ Mm\ max &= Fm\ max.(SM+d) \\ &= 240 \times (9.9 + 4.3) \\ &= 3408 \text{ t.m} \end{aligned}$$

i) Hydrodynamic pressure acting on columnar structure

The hydrodynamic pressure on columnar structure caused by earthquake is to be calculated by the following formula.

$$b/h = 2.0$$
$$Pd = 3/4.kh.Wo.b^2.h.(1-b/4h)$$
$$hg = 1/2\ h$$

Where ;
b ; Width of box caisson
Wo ; Specific weight of water (=1.0)
kh ; Horizontal seismic intensity(=0.05)
Pd ; Hydrodynamic pressure (=t/m^2)
hg ; Height of pressure center acting above reservoir bed (m)

From the above fomula, hydrodynamic pressure acting on longer width is

$$Pdl = 3/4 \times 0.05 \times 1.0 \times 22.0^2 \times 12.1 \times (1- 22.0/4 \times 12.1)$$
$$= 120\ t$$
$$Pds = 3/4 \times 0.05 \times 1.0 \times 17.0^2 \times 12.1 \times (1- 12.0/4 \times 12.1)$$
$$= 85.1t$$
$$hg = 1/2 \times 12.1 = 6.05\ m$$

Safety factor against uplift

Safety factor against uplift is calculated by following formula.

$$Fv = \frac{\Sigma W}{U}$$

Fv ; Safety factor against uplift
ΣW ; Total load of caisson
U ; Uplift acting on caisson

Result of calculation is shown in table in 3) "Result of Study, Uplift".

Stability against overturning and sliding

a) Overturning

The center of load acting on the bottom surface of the foundation is so designed as not less than 1/6 of the bottom surface width from outer edges of the foundataion to inward. Result of the calculation is shown in table in 3) "Result of Study, Overturning and Sliding".

b) Sliding

Safety factor against sliding is calculated by following formula.

$$Fs = \frac{u \Sigma W}{\Sigma H}$$

Here ;
Fs ; Safety factor against sliding
ΣW ; Total weight of caisson
ΣH ; Horizontal load acting on caisson
u ; Friction coefficient between caisson bottom and ground (u=0.5)

c) Ground reaction

Ground reaction on each edge of the caisson bottom is calculated by following formula.

$$q = \frac{\Sigma W}{A} \pm \frac{e \Sigma W}{W}$$

Here ;

ΣW ; Total weight of caisson
A ; Sectional area of caisson bottom
e ; Distance from the centroid of caisson bottom to the point where W acts
W ; Section modulus in relation to the centroid of caisson bottom
q ; Fiber ground reaction intensity

Stability Analysis on Caisson Type Intake Tower during Construction

1) General

As the caisson is setteled to the destination by its own weight, it is required to excute the work in making the dead weight of the caisson body and the buoyancy well in balance during construction. Bisides, it must be stable against the external forces during tugging. Needless to say, all the members consist of caisson body must be so designed as they are in safe during all construction stages. Here, two studies mentioned below were carried out.

a) Calculation of draft in five construction stages
b) Calculation of member stress during construction

2) Stability analysis

Caluculation of draft in five construction stages

As for the construction method of the caisson, there may be some different methods available. Here in this study, proposed method has 5 different stages as shown below and in Figs. 1.2.13 and 1.2.14.

Construction Stage	Brief description of stage
1.	construction in the dock and tugging
2.	placing in 1st floor and Outerwall of 1st story
3.	placing in pertition wall of 1st story and 2nd floor.
4.	placing filling concrete in floor chamber
5.	placing in Outerwall and pertition wall of 2nd story

Fig.I-2-15 Construction Stage — Shorter Width —

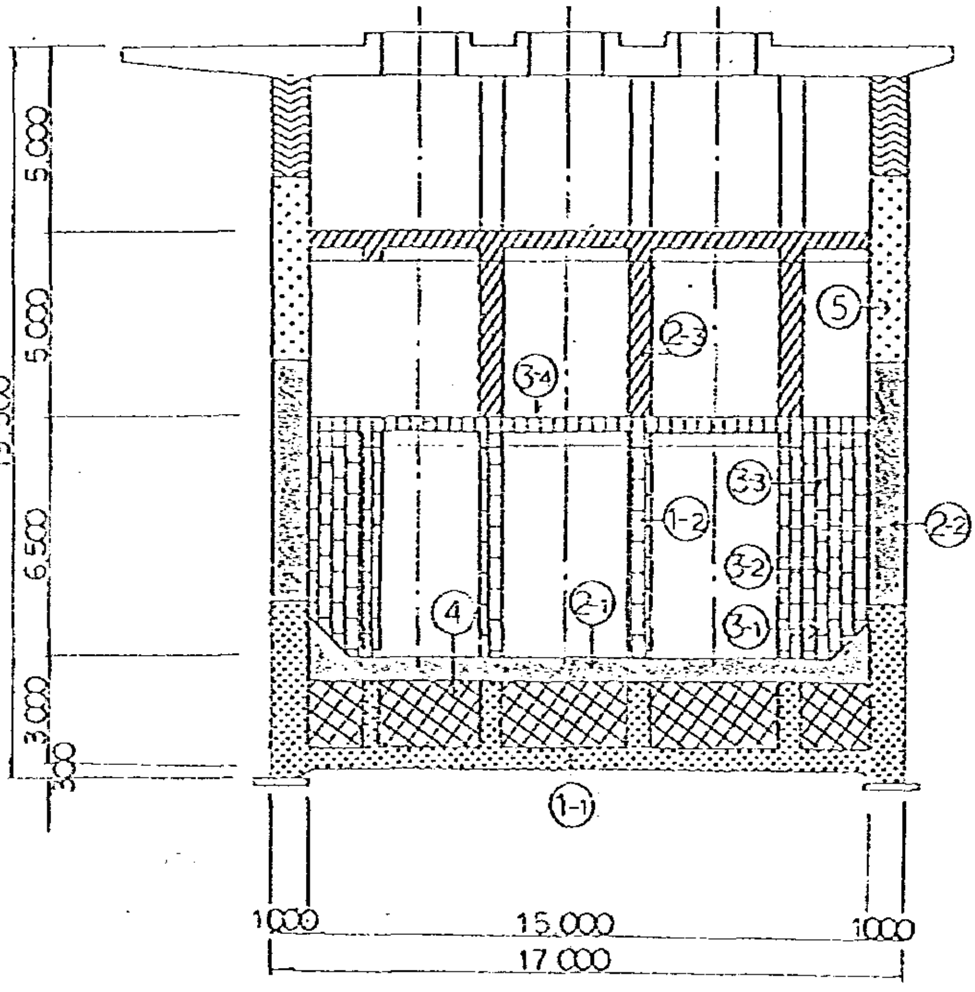
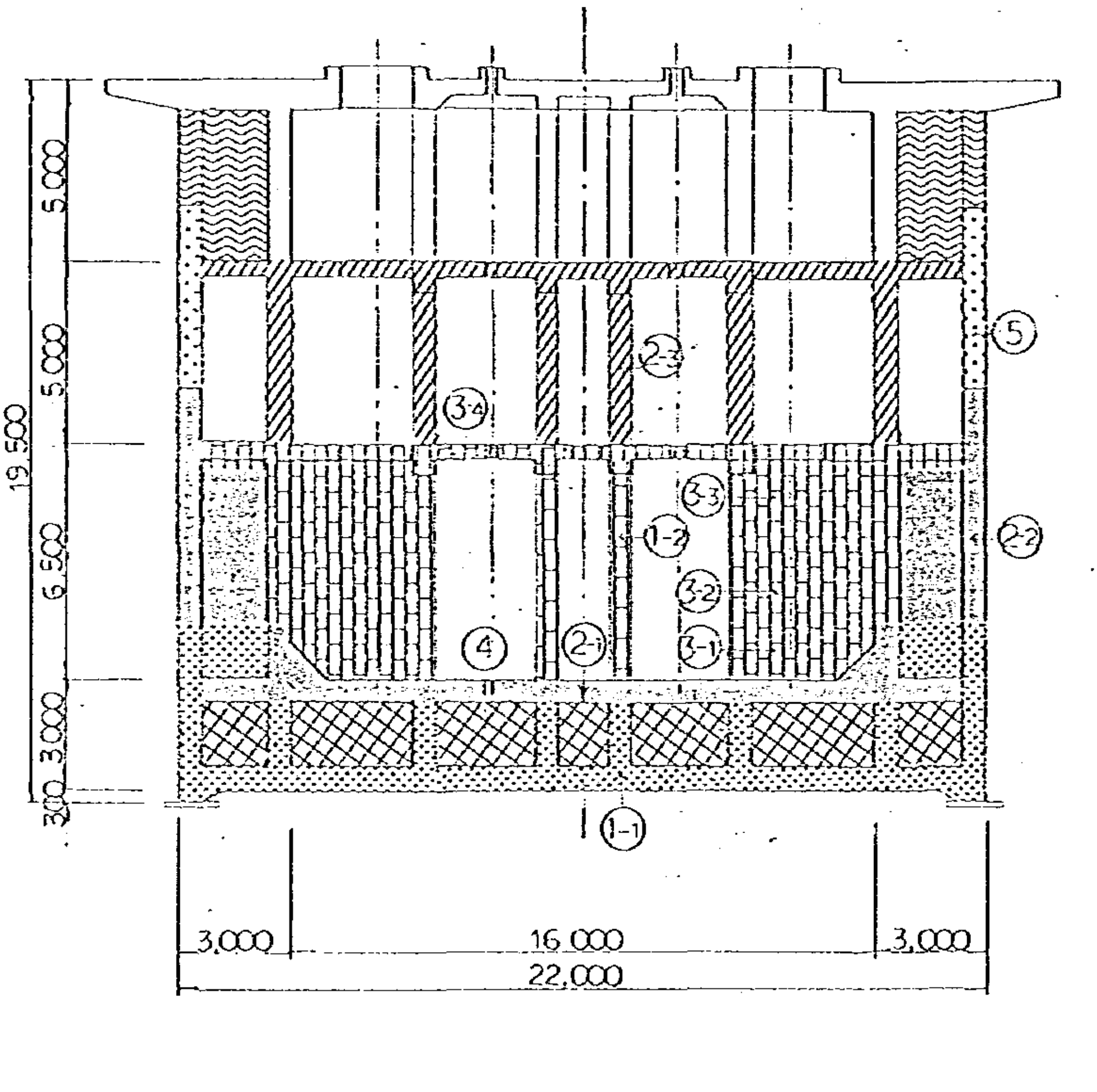


Fig.1-2-16 Construction Stage - Longer Width -



a) Calculation of self weight in each stage

(1st stage)

- Concrete

a. shoe	Va	= 2x1/2x(1.2+0.6)x0.3 x(22.0+17.0)	= 21.1m^3
b. Lower base slab	Vb	= 22.0x17.0x0.6	=224.4m^3
c. Outerwall	Vc	= 2x3.9x(0.6x15.0+1.0x22.0)	=241.8m^3
d. pertition wall	Vd	= 6x1.8x0.6x2.1+2x1.8x0.4x2.1	= 16.7m^3
e. Grill wall	Ve	= 20.8x1.8x(3x0.6+1x0.4)+6x [15.0-(3x0.6+0.4)]x0.6x1.8	=165.4m^3
			669.4m^3
	W	= 2.5x669.4	=1673.5t

- Scaffold H - 400x400x13x21

a. Column	Wa	= 24x0.172x7.0	= 28.9t
b. Beam	Wb	= 0.172x(4x22.0+6x17.0)	= 32.8t
c. Waling	Wc	= 2x2x0.172x(22.0+17.0)	= 26.9t
d. Attachment	Wd	= (28.9+32.8+26.9)x0.1	= 8.9t
			97.5t

- Waterproof form	W	= 200 kg/m^2	
	Wf	= 2x(22.0+17.0)x7.50x0.20	=117.0t

(2nd-stage)

- 2-1 step concrete

a. Upper base slab	Va	= 17.2x15.0x0.6	=154.8m^3
b. Upper base slab in suction sump	Vb	= 2x[15.0-(3x0.6+0.4)] x1.8x0.6	= 27.7m^3
c. Partition wall	Vc	= 2x15.0x1.5x0.6	= 27.0m^3
d. Haunch	Vd	= 2x1/2x1.0x1.0x(16.0+15.0)	= 31.0m^3
	W_{2-1}	= 240.5x2.5	=601.3t

- 2-2 step concrete

a. Outerwall	Va	= 2x6.5x(0.6x15.0+1.0x22.0)	=403.0m^3
b. Pertition wall in suction sump	Vb	= 1.8x4.2x(6x0.6+2x0.4)	= 33.4m^3
			436.4m^3
	W_{2-2}	= 436.4x2.5	=1091.0t

- 2-3 step scaffold H - 300x300x10x15

a. Column	Wa = 24x0.094x5.0	= 11.3t
b. Beam	Wb = 0.094x(4x22.0+6x17.0)	= 17.9t
c. Waling	Wc = 2x2x0.094x(22.0+17.0)	= 7.4t
d. Attachment	Wd = (11.3+17.9+7.4)x0.1	= 3.7t
		40.3t

- Removing of waterproof form

Wf = -117t

- Refixing of waterproof form

Wf = 117t

(3rd stage)

- 3-1 step concrete

a. Pertition wall in 1st story	Va = 6x0.6x(1.0x1.5-1/2x1.0x1.0) +4x0.6x(3.95x1.5-1/2x1.0x1.0) +2x0.4x(1.0x1.5-1/2x1.0x1.0) +2x0.6x(2.45x1.5-1/2x1.0x1.0) +2x0.6x(1.85x1.5-1/2x1.0x1.0)	= 23.9m^3
b. Column	Vb = 6x0.6x0.6x1.5	= 3.2m^3
		27.1m^3
	W_{3-1} = 27.1x2.5	= 67.8t

- 3-2 step concrete

a. Pertition wall in 1st story	Va = 6x0.6x1.0x2.0 +4x0.6x3.95x2.0 +2x0.4x1.0x2.0 +2x0.6x2.45x2.0 +2x0.6x1.85x2.0	= 38.2m^3
b. Column	Vb = 6x0.6x0.6x2.0	= 4.3m^3
c. Inner wall	Vc = 2x15.0x2.0x0.6	= 36.0m^3
		78.5m^3
	W_{3-2} = 78.5x2.5	=196.3t

- 3-3 step concrete

a. Pertition wall in 1st story	Va = 6x0.6x1.0x2.2 +4x0.6x3.95x2.2 +2x0.4x1.0x2.2 +2x0.6x2.45x2.2 +2x0.6x1.85x2.2	= 42.0m^3
b. Column	Vb = 6x0.6x0.6x2.2	= 4.8m^3
c. Inner wall	Vc = 2x15.0x0.6x2.2	= 39.6m^3
		86.4m^3
	W_{3-3} = 86.4x2.5	=216.0t

- 3-4 step concrete

a. 2nd floor slab $Va = 20.8x15.0-0.4x(6x2.0x2.0 +3x1.0x1.0)$ $=114.0m^3$

b. Beam $Vb =(20.8-6x0.6)x(3x0.6+0.4) x0.4+6x15.0x0.6x0.4$ $= 36.7m^3$

$150.7m^3$

$W_{3-4} = 150.7x2.5$ $=376.8t$

(4th stage)

- 4-1 step concrete

a. Filling conc. $Va = [(20.8-6x0.6)x(15.0x2.2)]x1.8=396.3m^3$

$W_{4-1} = 396.3x2.35$ $=931.3t$

(5-th stage)

- 5-1 step concrete

a. Outerwall $Va = 2x5.0x(0.6x15.0+1.0x22.0)$ $=310.0m^3$

b. Partition wall in suction sump $Vb = 1.8x4.2x(6x0.6+2x0.4)$ $= 33.4m^3$

$=343.4m^3$

$W_{5-1} = 343.4x2.5$ $=858.5t$

b) Caluculation of draft in each stage

Draft of the caisson body is calculated by following formula.

$$D = \frac{\Sigma W - Vs \times 1.0}{A} + Hs$$

Here; D ; Draft of the caisson(m)

ΣW ; Total weight of the caisson in each stage(t)

A ; Sectional area (=22.0x17.0m)

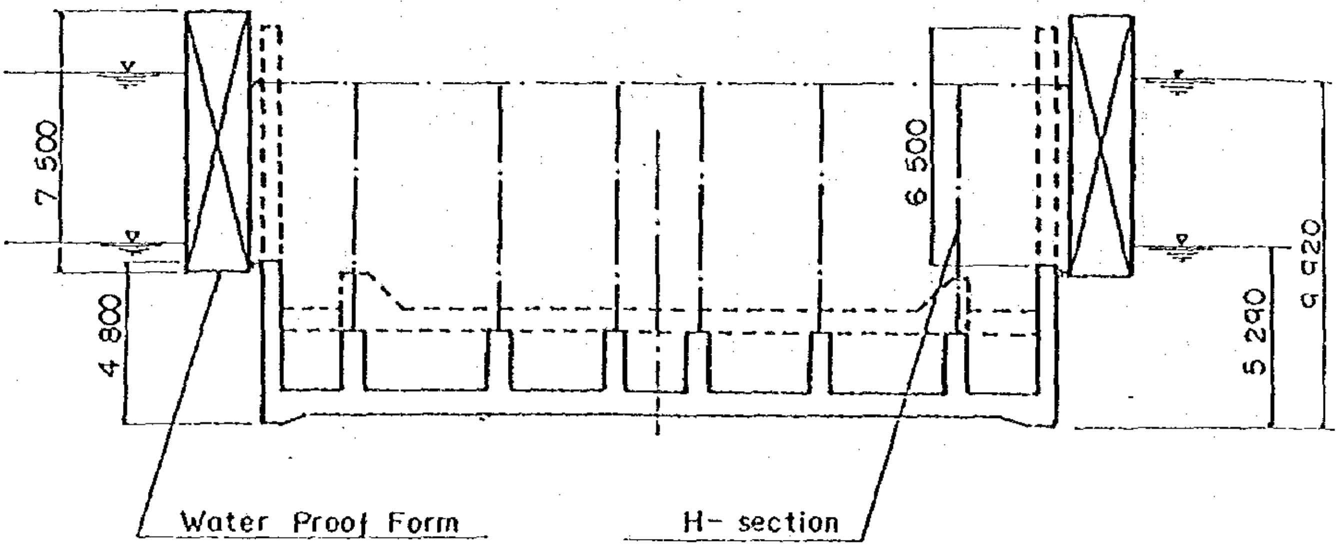
Vs, Hs ; Volume and hight of shoe ($Vs=21.1m^3$, $Hs=0.3^m$)

Draft in each construction stage

Const. stage		V(m3)	W(t)	ΣW(t)	D(m)
1	1 - 1	669.4	1673.5		
	1 - 2		97.5		
	1 - 3		117.0	1888.0	5.29
2	2 - 1	240.5	601.3	2489.3	6.90
	2 - 2	436.4	1091.0	3580.3	9.81
	2 - 3		40.3	3620.6	9.92
	2 - 4		-117.0	3503.6	9.67
	2 - 5		117.0	3620.6	9.92
3	3 - 1	27.1	67.8	3688.4	10.10
	3 - 2	78.5	196.3	3884.7	10.62
	3 - 3	86.4	216.0	4100.7	11.21
	3 - 4	150.7	376.8	4477.5	12.21
4	4 - 1	396.3	931.3	5408.8	14.70
5	5 - 1	343.4	858.5	6267.3	17.00

Fig.1-2-17 Draft of Caisson during Construction

1st stage (At the time of tugging)



3rd stage

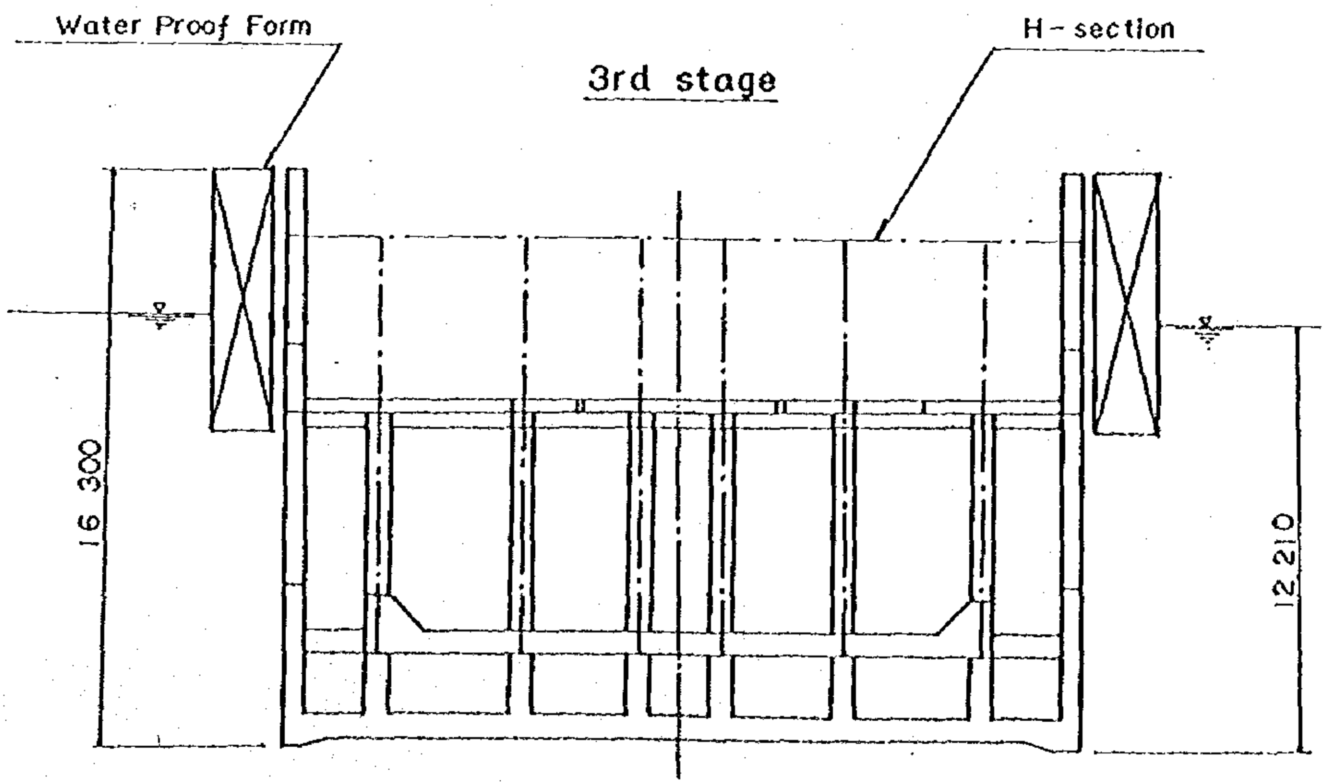
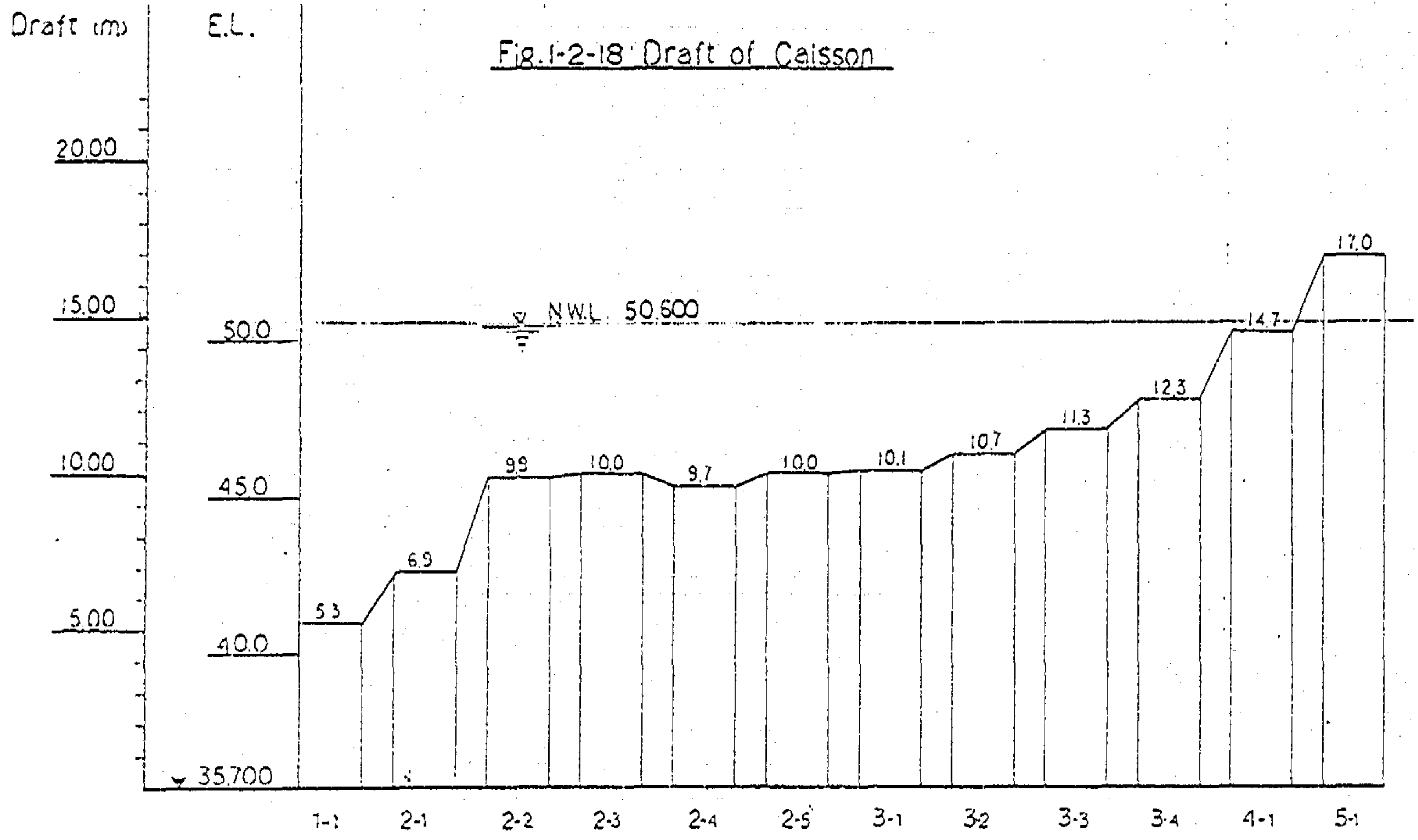


Fig.1-2-18 Draft of Caisson



Stress Analysis of Members

Since there are many members and cases to be studied in the Intake tower structure, it is necessary to mention it briefly.

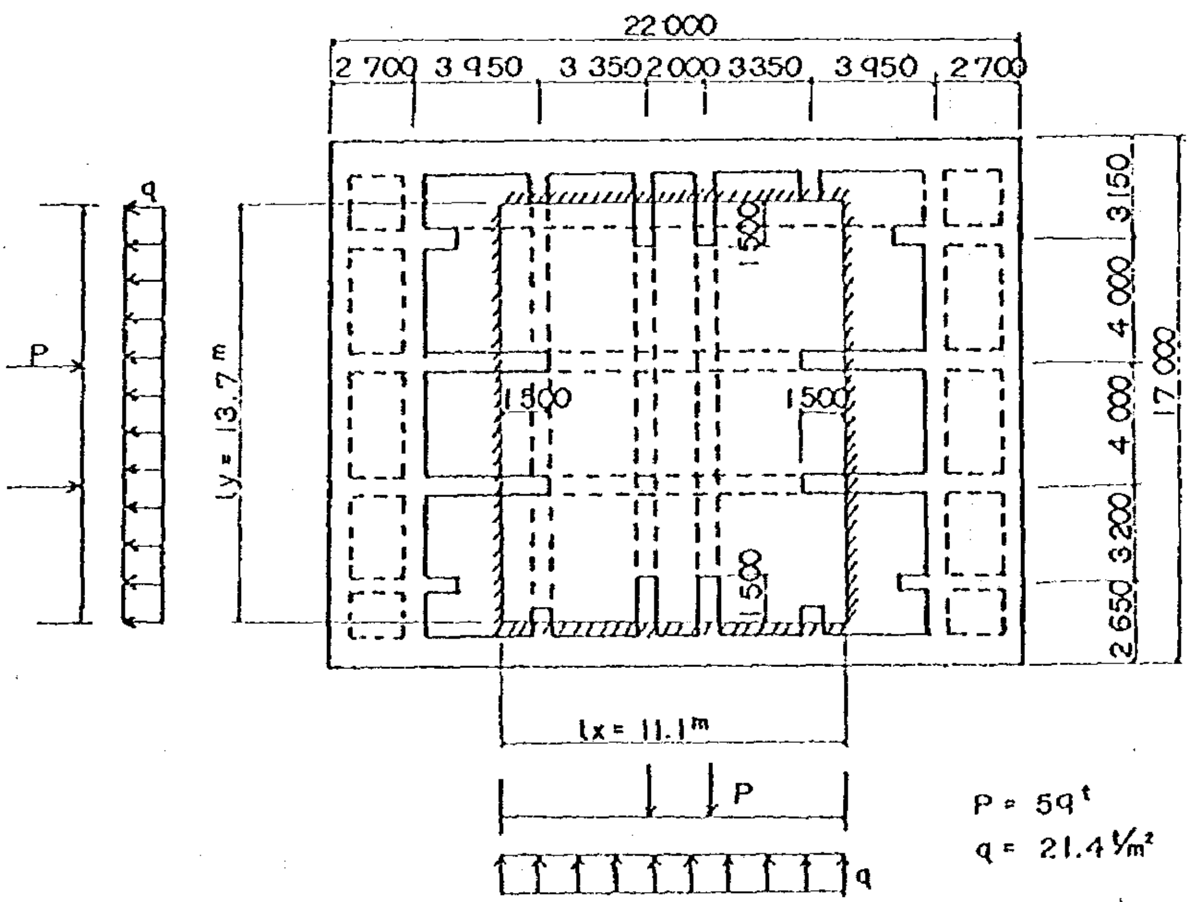
1) Base slab

Grille beam

a) Model and load

Virtual loading surface is taken into consideration as below.

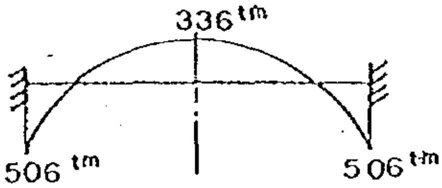
Base Slab



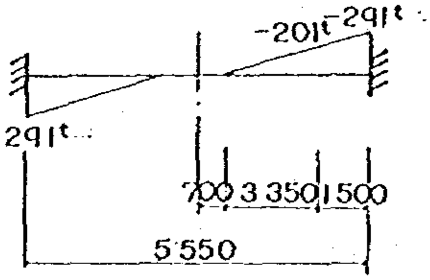
b) Bending moment and shear force

(shorter width)

B.M.D.

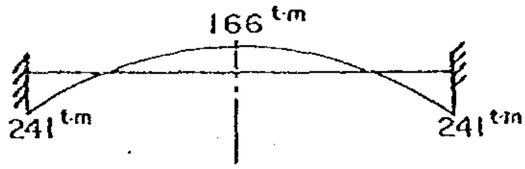


S.F.D.

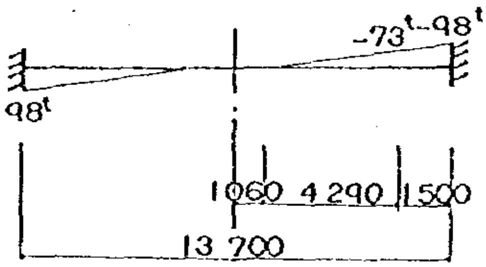


(longer width)

B.M.D.



S.F.D.



c) Calculation of reinforcing bar arrangement

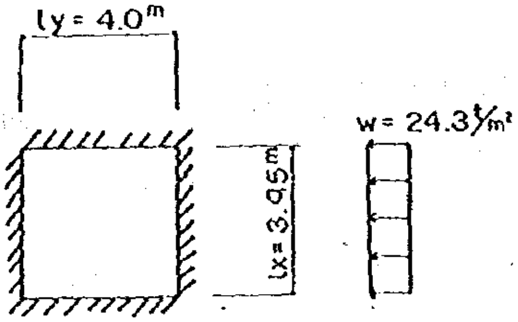
	member	D (cm)	M (t·m)	S (t)	Reinforcing bar
Shorter width	lower	300	506	201	D-25 36 pcs
	upper	300	336		D-25 24 pcs
Longer width	lower	300	241	73	D-25 24 pcs
	upper	300	166		D-25 24 pcs

Here; D : Thickness
M : Bending moment
S : Shear force

Base slab

a) Model and load

The slab is assumed as a rectangular plate with all edges built in.



b) Claculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	Reinforcing bar
Slab	60	16.0	24.3	D-25 ctc 15 cm

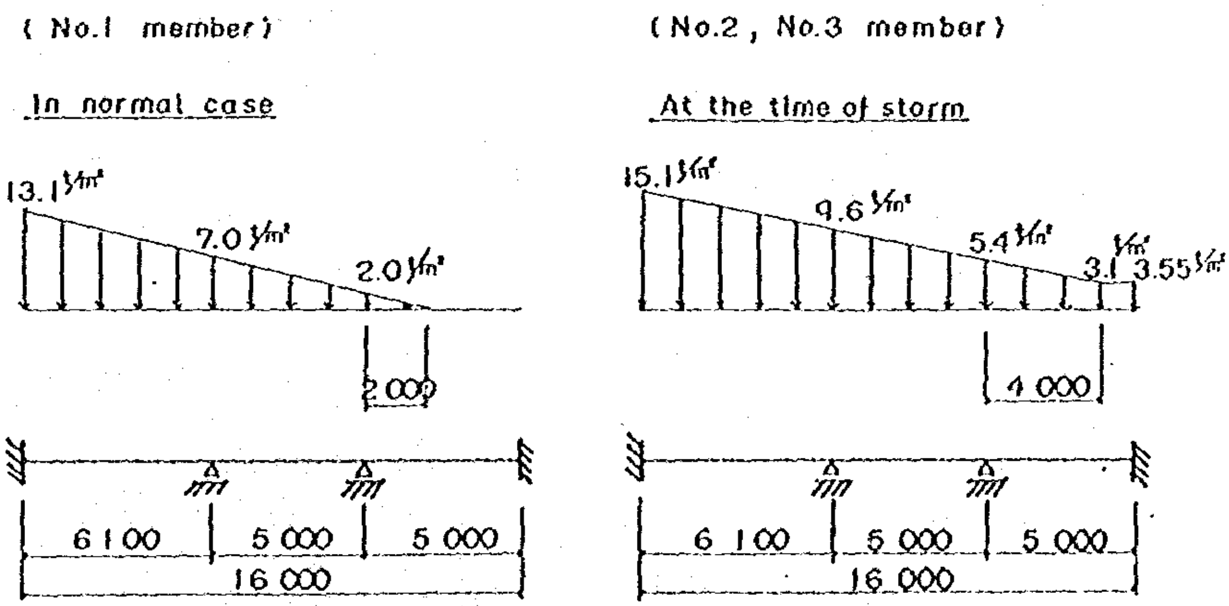
Here; ctc; Center-to-center spacing between the reinforcing bar

2) Side wall

Double wall of suction sump

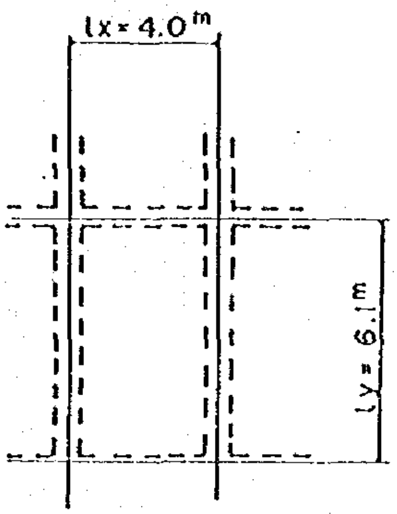
a) Model and load of grill beam

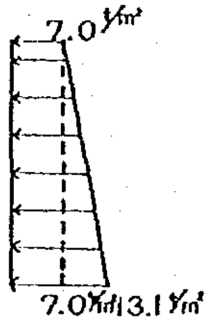
Grill beam is assumed as 3 spaned continuous beam.



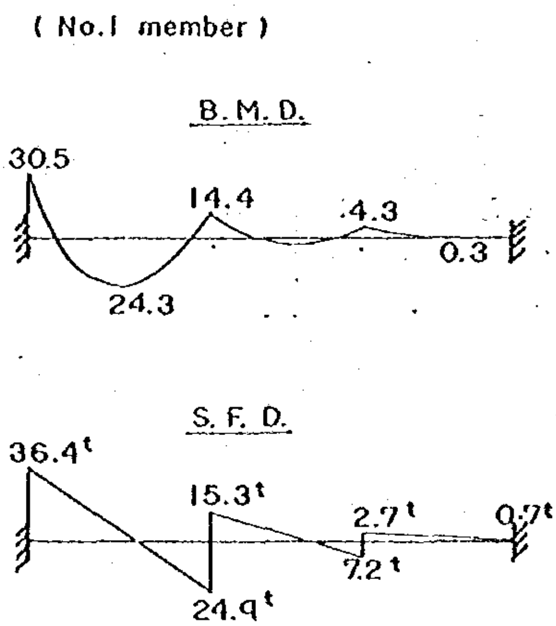
b) Model and load of wall slab

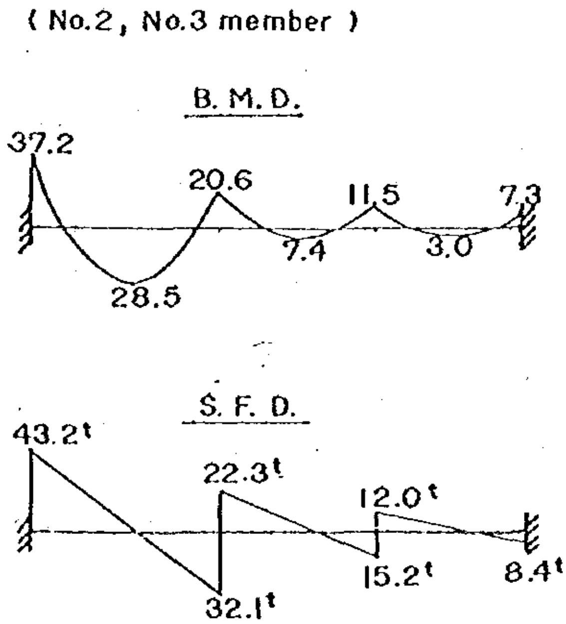
The wall slab is assumed as a rectangular plate with all edges built in.



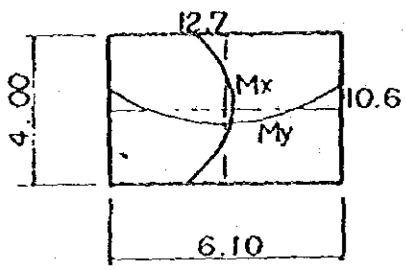


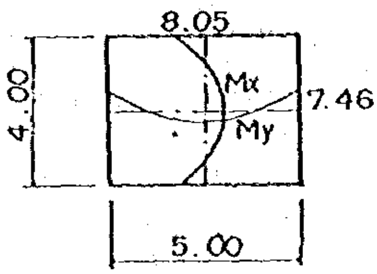
c) Bending moment and shear force





(No.1 Slab) (No.2, No.3 Slab)





d) Calculation of reinforcing bar arrangement

	member	D (cm)	M (t·m)	S (t)	reinforcing bar
Grille beam	No. 1 beam	300	30.5	36.4	D-22 12 pcs D-20 12 pcs
	No.2,3 beam		20.6	22.3	D-20 12 pcs D-16 12 pcs
Wall slab	No. 1 slab	60	2.7 10.6	23.9	D-22 ctc 10 cm D-22 ctc 15 cm
	No. 2 slab No. 3	60	8.05 7.46	17.5	D-22 ctc 20 cm D-20 ctc 15 cm

Single outer wall

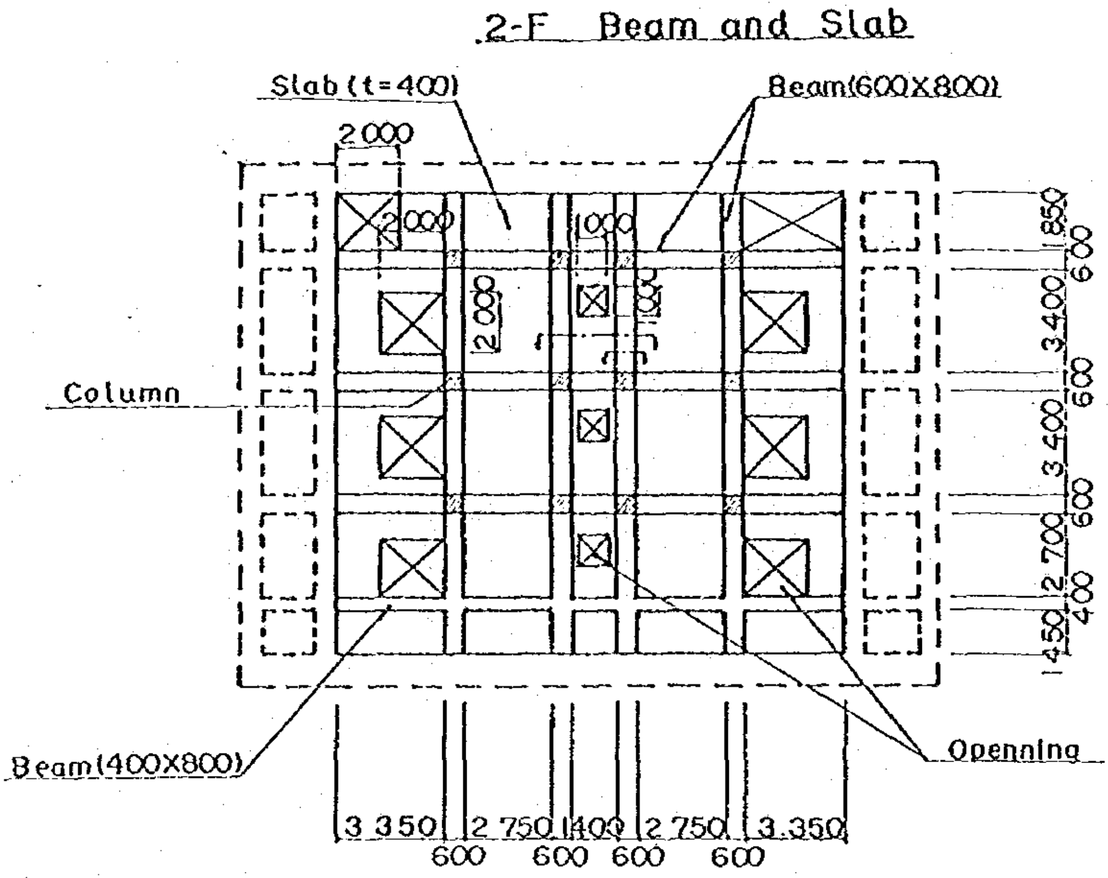
a) Model and load

The wall is assumed as the same as grilled beam mentioned above.

b) Calculation of reinfocing bar arrangement

	D (cm)	M (t.m)	S (t)	reinforcing bar
No. 1 beam	100	30.5	36.4	D-25 ctc 15 cm
No.2,3 beam	100	20.6	22.3	D-20 ctc 15 cm

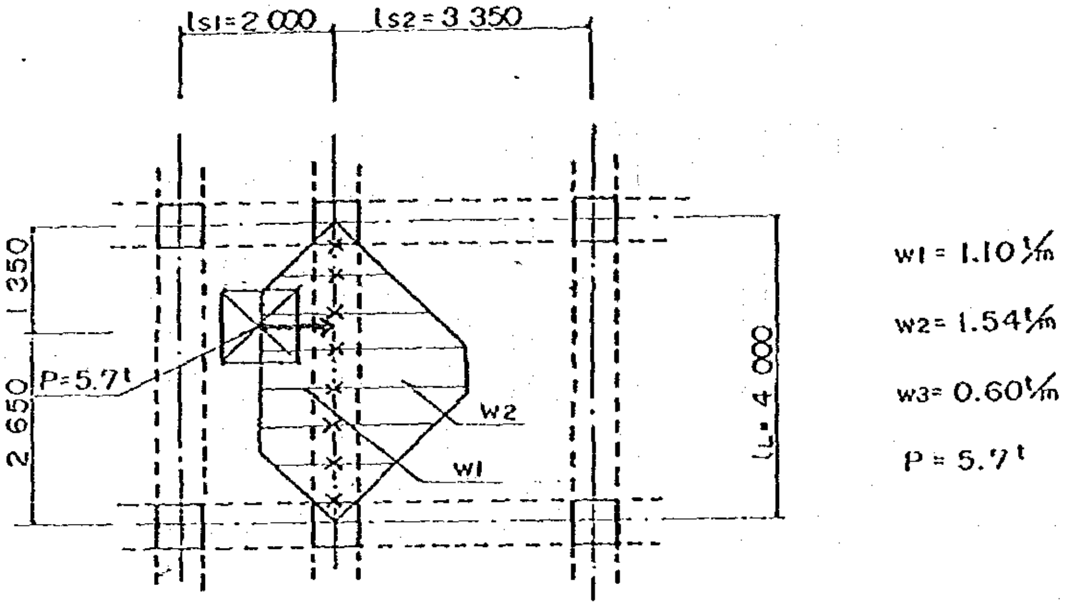
3) Beam and slab on 2nd floor



Beam

a) Model and load

Beam in central area is taken as for example. It is assumed as a simple beam.



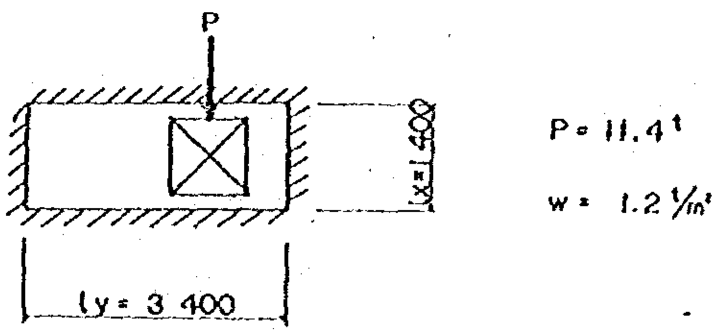
b) Calculation of reinforcing bar arrangment

	D (cm)	M (t·m)	S (t)	reinforcing bar
Beam	80	11.5	9.2	D-22 4 pcs.

Slab

a) Model and load

The slab in central area is assumed as a rectangular plate with all edges built in.

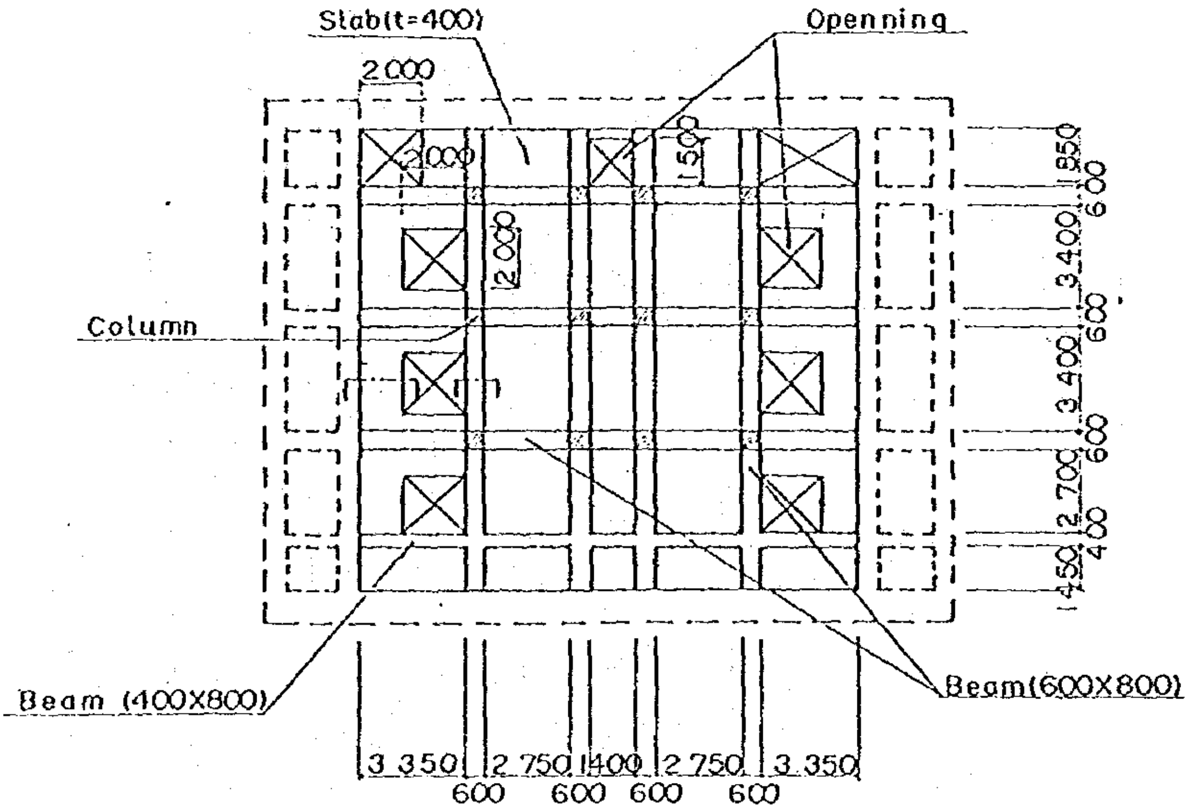


b) Calculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	reinforcing bar
Slab	40	2.60	-	D-16 ctc 15 cm

4) Beam and slab on 3rd floor

3-F Beam and Slab

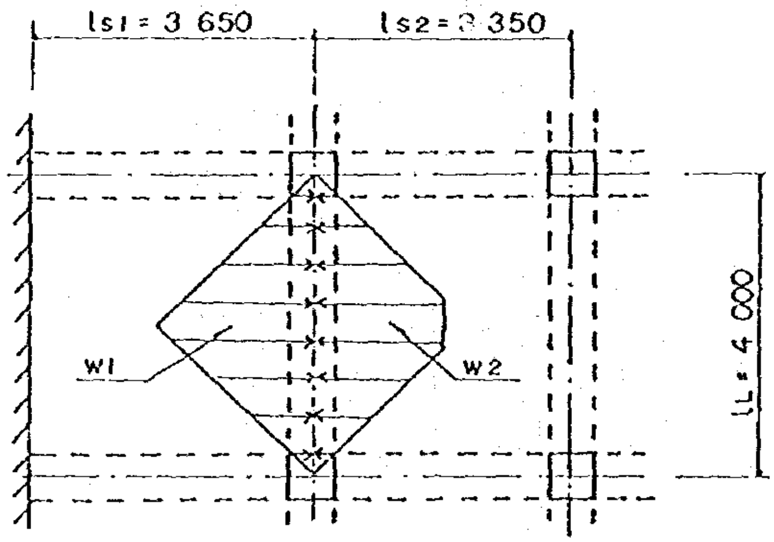


Beam

a) Model and load

Beam touching opening for pumpshaft is taken as for example.

It is assumed as a simple beam.



w = 3.7 t/m

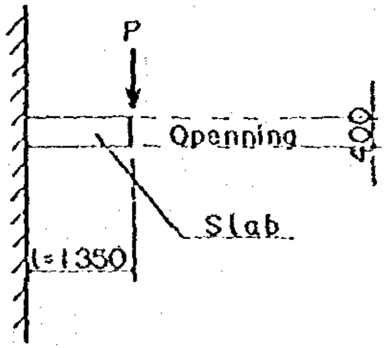
b) Calculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	reinforcing bar
Beam	80	7.4	7.4	D-22 4 pcs.

Slab

a) Model and load

As the opening is big, the rest of slab is assumed as a cantilever beam.



$P = 1.0^t$

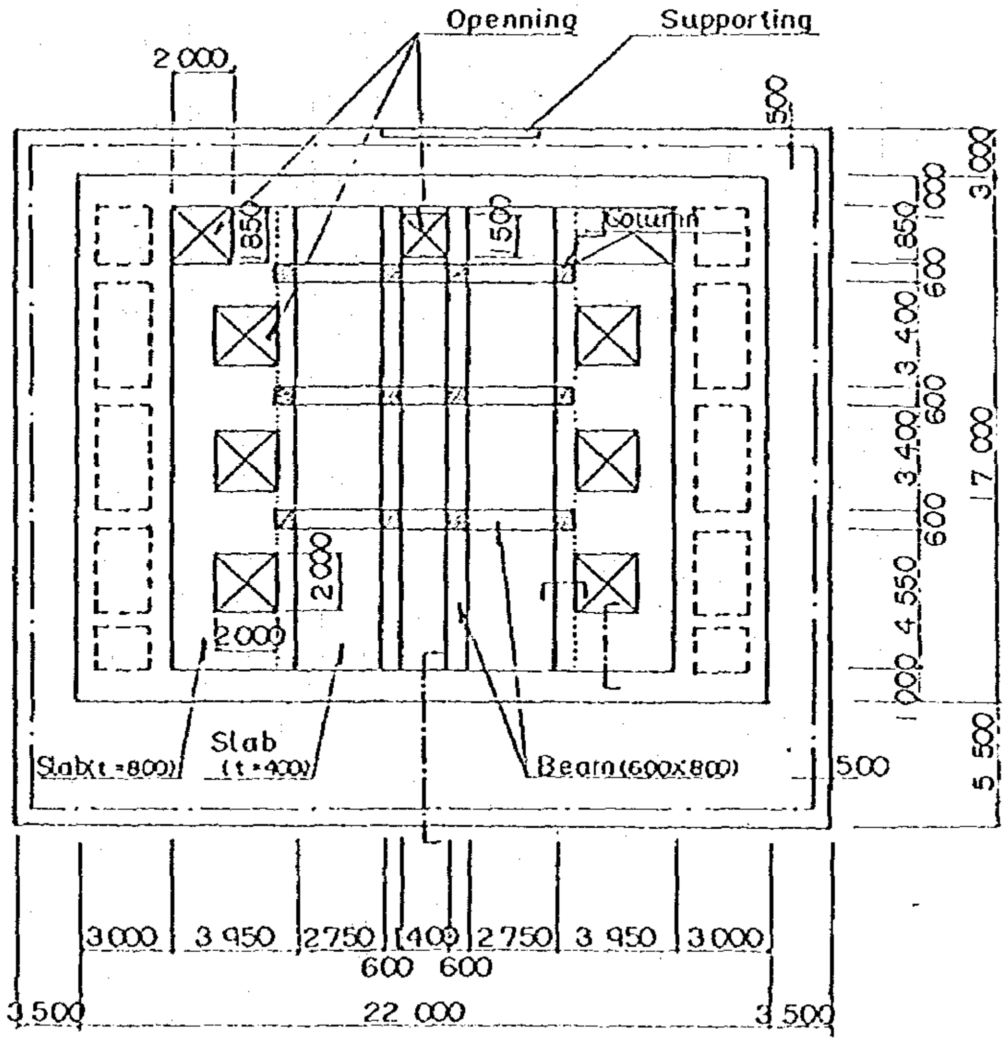
$w = 1.2\ t/m^2$

b) Calculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	reinforcing bar
Slab	40	2.4	2.6	D-16 ctc 15 cm

5) Beam and Slab on 4th Floor

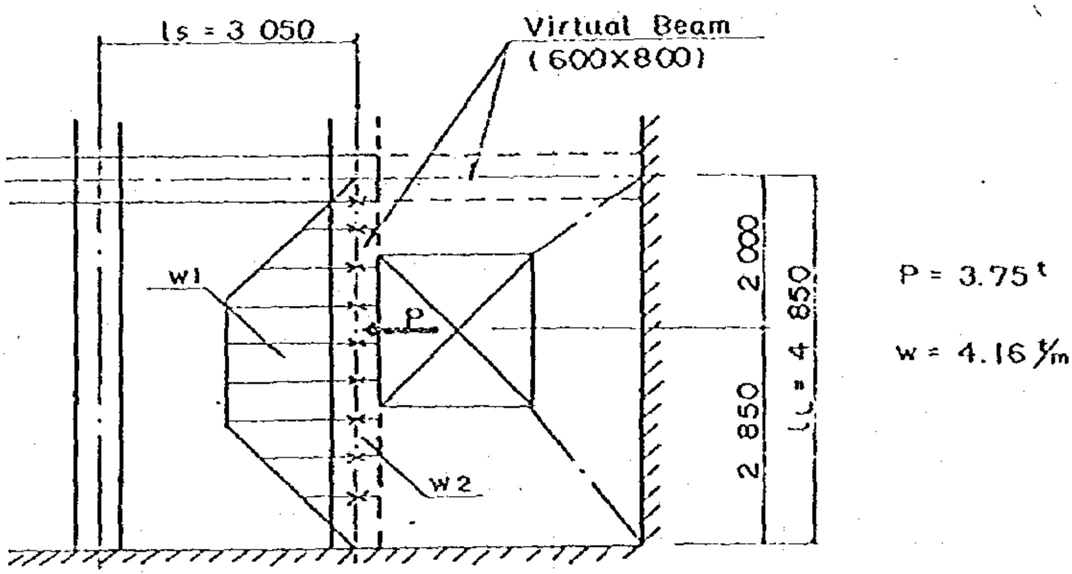
4-F Beam and Slab



Beam

a) Model and load

The longest beam touching opening for motor base is taken as for example. It is assumed as a proped beam.



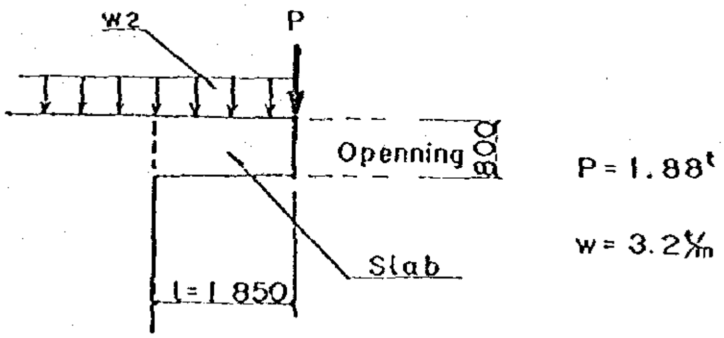
b) Calculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	reinforcing bar
Beam	80	15.1	14.8	D-25 4 pcs

Slab

a) Model and load

As the opening is big, the rest of slab is assumed as cantilever beam.



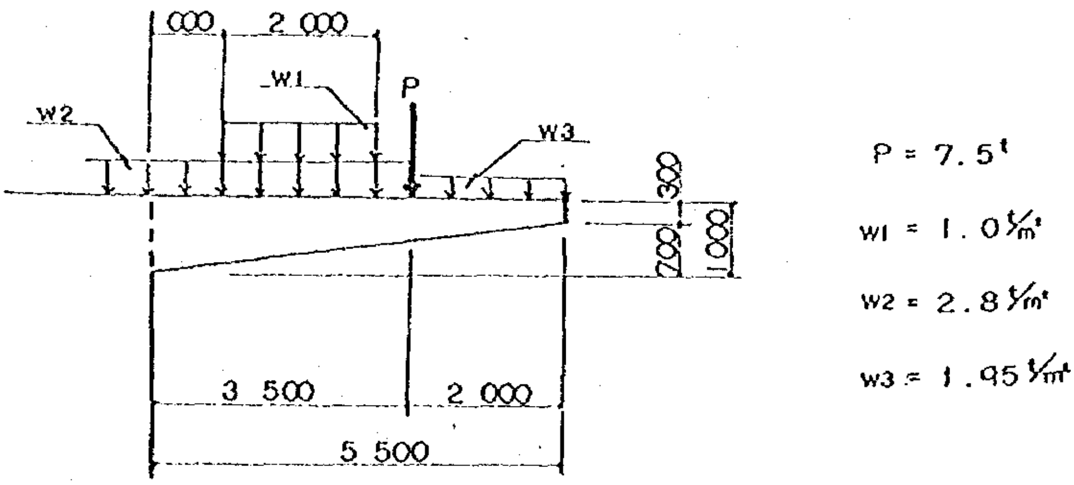
b) Calculation of reinforcing bar arrrangement

	D (cm)	M (t·m)	S (t)	reinforcing bar
Slab	80	8.95	7.8	D-16 ctc 15 cm

Overhanging slab

a) Model and load

Overhanging slab is assumed as a cantilever beam.



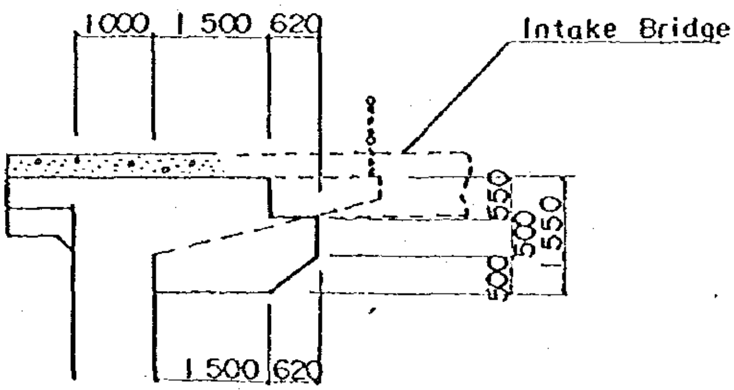
b) Calculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	rèinforcing bar
Slab	100	65.1	23.2	D-25 ctc 15 cm Double

Support for intake bridge

a) Model and load

Bearing rest for bridge is assumed as a cantilever beam.



b) Calculation of reinforcing bar arrangement

	D (cm)	M (t·m)	S (t)	reinforcing bar
Support	155	41.32	36	D-16 D-20 ctc 15 cm

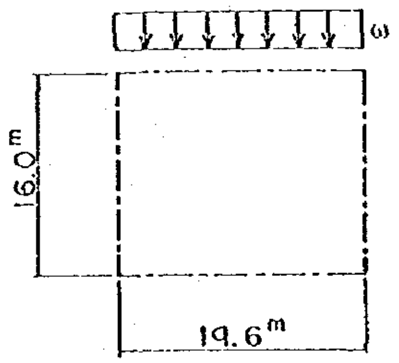
Stress Analysis of Members during Construction

Stresses of members due to external loads very much depend on the construction technique. It is anticipated that the stress level during construction is higher than that of after completion in some portions of some members. In this case, members must be strengthened with reinforcement or like.

1) Grill beam on base slab

a) Model and load

Virtual loading surface may be illustrated as below, and it is assumed that only the grill beam bears the stress.



At 2-1 Stage : $\omega = 3.3\ ^{t/m^2}$

At 3-1 Stage : $\omega = 6.5\ ^{t/m^2}$

b) Calculation of reinforcing bar arrangement

At 2-1 Stage (B = 60cm)

	D (cm)	M (t·m)	S (t)	Reinforcing bar	Stirrup
Shorter width	240	169.7	94.5	D-25 12 pcs	D-19 2 x ctc15
Longer width	240	114.8	105.6	D-25 10 pcs	D-19 2 x ctc15

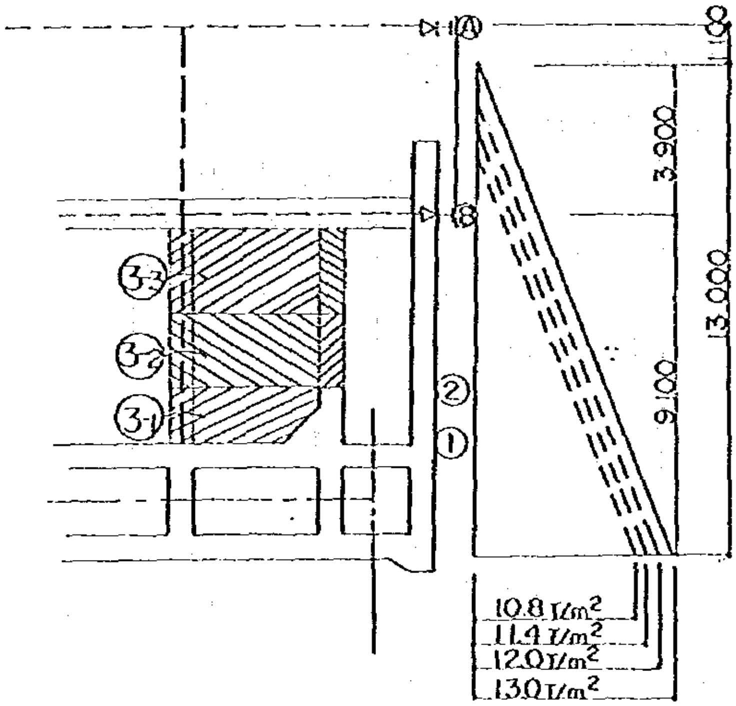
At 3-1 Stage (B = 60 cm)

	D (cm)	M (t·m)	S (t)	Reinforcing bar	Stirrup
Shorter width	300	321.9	136.4	D-25 24 pcs	D-22 2 x ctc15
Longer width	300	232.8	208.0	D-25 24 pcs	D-19 2 x ctc15

2) Side wall

Double wall of suction pump

a) Model and load



b) Moment and shear force

Stage	R_A (t/m)	R_B (t/m)	M1 (t·m/m)	M2 (t·m/m)	S1 (t/m)	S2 (t/m)
3-1	0.2	1.3	72.7	36.6	29.5	18.9
3-2	0.5	2.2	89.3	47.1	34.1	22.6
3-3	0.8	3.4	111.0	61.4	39.5	27.1
3-4	2.0	5.6	142.7	81.9	47.7	33.8

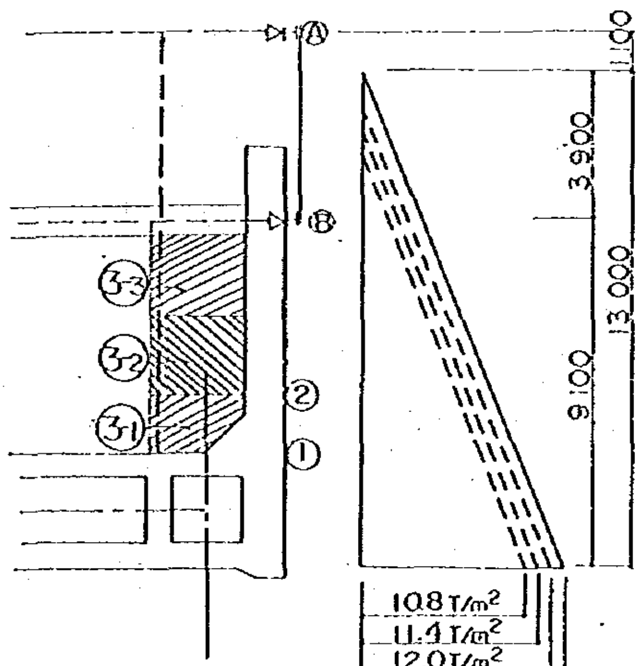
c) Calculation of reinforcing bar arrangement

Stage	B (cm)	D (cm)	M (t·m)	S (t)	Reinforcing bar	Stirrup
3-1	100	100	46.2	25.2	D-25 2 x ctc15	-
3-4	60	285	345.5	121.5	D-25 2 x ctc15	D-19 4 x ctc20
3-4*	60	285	472.3	166.1	D-25 2 x ctc15	4 x ctc2

* : Outside wall

Single outer wall

a) Model and load



b) Moment and shear force

Stage	R_A (t/m)	R_B (t/m)	M1 (t·m/m)	M2 (t·m/m)	S1 (t/m)	S2 (t/m)
3-1	0.2	1.3	46.2	16.7	25.2	14.6
3-2	0.5	2.2	72.3	34.3	31.3	19.8
3-3	0.8	3.4	99.2	52.5	37.5	25.1
3-4	2.0	5.6	129.4	71.9	45.5	31.6

c) Calculation of reinforcing bar arrangement

Stage	B (cm)	D (cm)	M (t·m)	S (t)	Reinforcing bar	Stirrup
3-1	60	300	290.8	118.0	D-25 2 x ctc15	D-19 4 x ctc20
3-2	60	240	188.4	90.4	D-25 2 x ctc15	D-19 4 x ctc20
3-4	60	695	570.8	190.8	D-25 2 x ctc15	-
3-4*	60	400	442.4	147.9	D-25 2 x ctc15	D-19 4 x ctc20
3-4**	40	400	403.8	135.0	D-25 2 x ctc15	D-19 4 x ctc20

* : Outside wall (thick)
** : Outside wall (thin)

1.2.2 Intake Bridge

Design of intake bridge consists of four parts. They are design of girder for pedestrians and vehicles and design of girder for a pipeline and design of pipeline itself and design of sub structure.

Design of Girder for Pedestrians and Vehicles

1) Design load

Dead loads factor

Girder self weight	900 kg/m
Curb	264 kg/m
Slab	240 kg/m^2
Shear key concrete	58 kg/m
Hand rail	50 kg/m
Cable box	200 kg/m

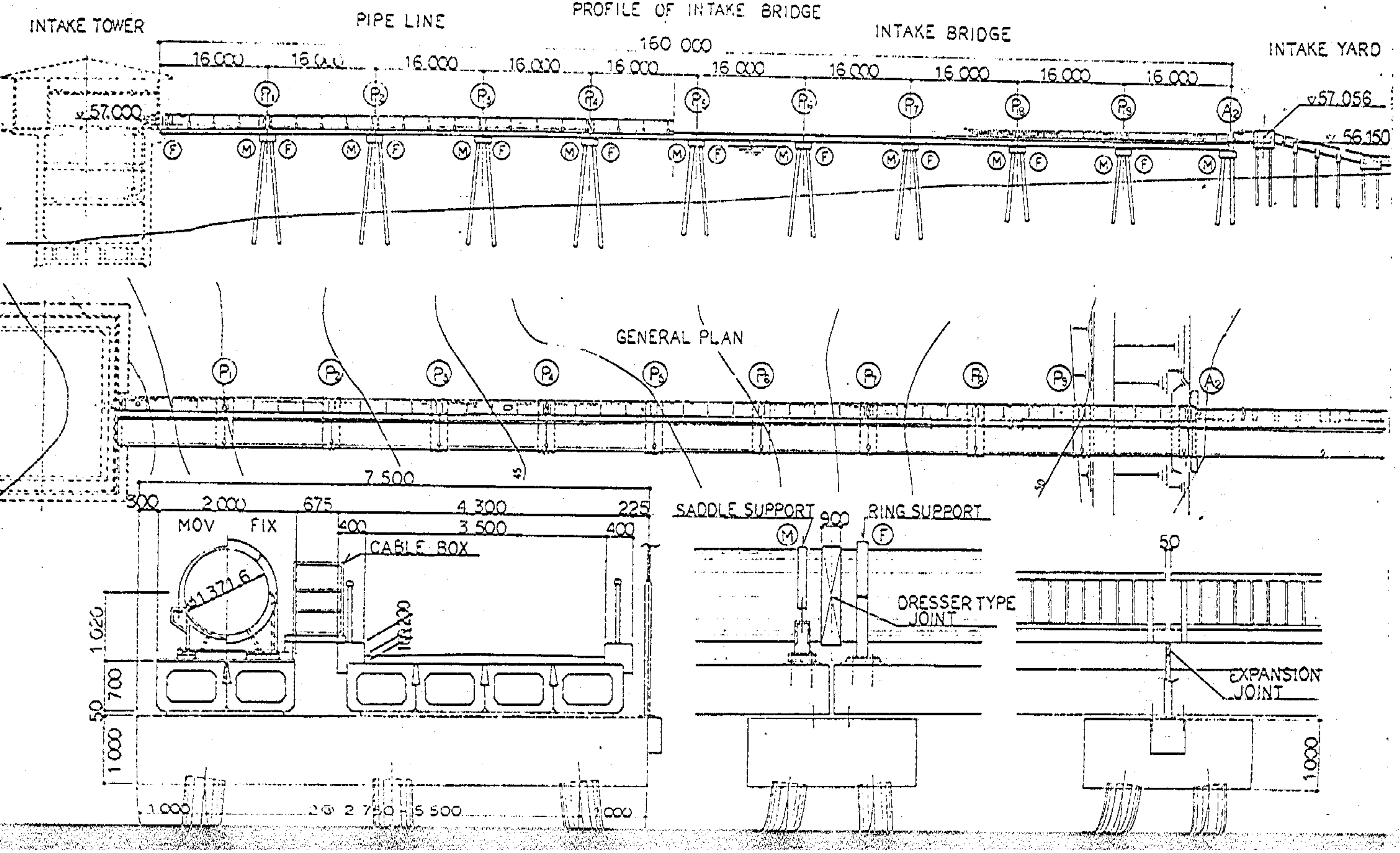
Live loads

Line load	3,500 kg/m
Uniform load	245 kg/m^2
Impact coefficient	1.25

2) Section properties of P.C box girder

Item	Unit		Ordinary Section		Prestressing steel included
Sectional area	cm^2	A	3,642	Ael	3,722
Moment of inertia	cm^4	I	2,248,078	Iel	2,308,807
Distance between gravity center and bottom surface	cm	Y'	33.68	Y'el	33.08
Distance between gravity center and upper face	cm	Y	36.32	Yel	36.92
Distance between gravity center and prestressing steel	cm	E	28.21	Eel	27.61
Section modulus (bottom)	cm^3	W'	66,748	W'el	69,795
Section modulus (top)	cm^3	W	61,896	Wel	62,535
Section modulus (prestressing steel)	cm^3	Wg	79,691	Welg	83,622

FIG 1-2-19 GENERAL VIEW OF INTAKE BRIDGE



3) Bending stress of concrete at center of girder

Bending stresses caused by dead loads and live loads are expressed by the formula as below.

$f = M/W$

where f : Bending stress (kg/cm^2)
M : Bending moment (kg·cm)
W : Section modulus of girder (cm^3)

Item	Bending moment (kg·cm)	Section modulus (cm^3)	Bending stress (kg/cm^2)		
			Top fiber	Bottom fiber	Center of prestressing steel
Girder self weight	27.2×10^5	Wel = 62,535	43.5		
		W'el= 69,795		- 39.0	
		Welg= 83,622			- 32.5
Other dead loads	33.3×10^5	Wel = 62,535	53.3		
		W'el= 69,795		- 47.7	
		Welg= 83,622			- 39.8
Live loads	20.4×10^5	Wel = 62,535	32.6		
		W'el= 69,795		- 29.2	
		Welg= 83,622			- 24.4
Total			129.4	-115.9	- 96.7

Note: Minus means tensile stress

4) Prestress

Initial prestress

$f'si = 126.0$ kg/mm^2 $<$ $fsia = 135$ kg/mm^2

Temporary stress before loss by creep and shrinkage

$fst = 112.9$ kg/mm^2 $<$ $fsta = 122.5$ kg/mm^2

Effective stress after loss by creep and shrinkage

$fse = 97.8$ kg/mm^2 $<$ $fsa = 105$ kg/mm^2

5) Prestressing stress of concrete

Prestressing stress of concrete is expressed by the formula as below.

$$fc = \frac{P}{A} \pm \frac{P \times E}{W}$$

where
P : Prestressing force (kg)
A : Sectional area = 3,642 cm^2
E : Distance between gravity center of girder and prestressing steel = 28.21 cm
W : Section modulus (cm^3)

- Temporary stress before loss by creep and shrinkage

P (kg)	W (cm^3)		Prestressing stress of concrete Top fiber	Bottom fiber	Center of prestressing steel
174,803	W	61,896	- 31.7		
	W'	66,748		121.9	
	Wg	79,691			109.9

- Stress after losses have occurred

P (kg)	W (cm^3)		Prestressing stress of concrete Top fiber	Bottom fiber	Center of prestressing steel
151,424	W	61,896	- 27.5		
	W'	66,748		105.6	
	Wg	79,691			95.2

6) Stress resultant of concrete

		Bending Stress caused by dead load and live load (kg/cm^2)	Prestressing (kg/cm^2)	Total stress (kg/cm^2)	Allowable stress (kg/cm^2)
At transfer stage	Top fiber	43.5	- 31.7	11.8	>ftt = -13.5
	Bottom fiber	- 39.0	121.9	82.9	<fct = 160
At working stage (only dead load)	Top fiber	96.8	-27.5	69.3	<fca = 125
	Bottom fiber	- 86.7	105.6	18.9	>f'ta= 0
At working stage	Top fiber	129.4	-27.5	101.9	<fca = 125
	Bottom fiber	-115.9	105.6	-10.3	>fta = -13.5

Note: Minus means tensile stress

7) Stress resultant of prestressing steel at working stage

Stress of the lowest prestressing steel is maximum value

$$f_{se} = 104.1 \text{ kg/mm}^2 < f_{sa} = 105 \text{ kg/mm}^2$$

<u>Design of Girder for Pipeline</u>

1) Design load

Pipe self weight	400 kg/m
Water self weight	1,425 kg/m
Girder self weight	1,064 kg/m
Shear key concrete	58 kg/m
Cable box	200 kg/m

2) Section properties of girder

Item	Unit		Ordinary Section		Prestresssing steel included
Sectional area	cm^2	A	3,642	Ael	3,706
Moment of inertia	cm^4	I	2,248,078	Iel	2,298,507
Distance between gravity center and bottom	cm	Y'	33.68	Y'el	33.19
Distance between gravity center and top	cm	Y	36.32	Yel	36.81
Distance between gravity center and prestressing steel	cm	E	28.21	Eel	28.19
Section modulus (bottom)	cm^3	W'	66,748	We'l	69,253
Section modulus (top)	cm^3	W	61,896	Wel	62,442
Section modulus (prestressing steel)	cm^3	Wg	79,691	Welg	81,536

3) Bending stress of concrete at center of girder

Item	Bending moment (kg·cm)	Section modulus (cm^3)	Bending stress (kg/cm^2)		
			Top fiber	Bottom fiber	Center of prestressing steel
Girder self weight	31.7×10^5	Wel = 62,442	50.8		
		W'el= 69,253		- 45.8	
		Welg= 81,536			- 38.9
Pipe (drained)	36.9×10^5	Wel = 62,442	59.1		
		W'el= 69,253		- 53.3	
		Welg= 81,536			- 45.3
Pipe (filled)	55.4×10^5	Wel = 62,442	88.7		
		W'el= 69,253		- 80.0	
		Welg= 81,536			- 67.9

Note: Minus means tensile stress

4) Prestress

Initial prestress

$f'si = 126.0\ kg/mm^2 < fsia = 135\ kg/mm^2$

Temporary stress before loss by creep and shrinkage

$fst = 114.5\ kg/mm^2 < fsta = 122.5\ kg/mm^2$

Effective stress after losses have occured

$fse = 100.7\ kg/mm^2 < fsa = 105\ kg/mm^2$

5) Prestressing stress of concrete

Prestressing stress of concrete is expressed by the formula

$$fc = \frac{P}{A} \pm \frac{P \times E}{W}$$

where
P : Prestressing force (kg)
A : Sectional area = 3,642 cm^2
E : Distance between gravity center of girder and prestressing steel = 28.21 cm
W : Section modulus (cm^3)

- Temporary stress before loss by creep and shrinkage

P (kg)	W (cm^3)		Prestress of concrete		
			Top fiber	Bottom fiber	Center of prestressing steel
141,820	W'	61,896	-25.7		
	W	66,748		98.8	
	Wg	79,691			89.1

- Stress after losses have occurred

P (kg)	W (cm^3)		Prestress of concrete		
			Top fiber	Bottom fiber	Center of prestressing steel
124,727	W'	61,896	-22.6		
	W	66,748		86.8	
	Wg	79,691			78.3

6) Stress resultant of concrete

		Bending Stress caused by design load (kg/cm^2)	Prestressing (kg/cm^2)	Total stress (kg/cm^2)	Allowable stress (kg/cm^2)
At transfer stage	Top fiber	50.8	-25.7	25.1	>ftt = -13.5
	Bottom fiber	-45.8	98.8	53.0	<fct = 160
At working stage (pipe drained)	Top fiber	59.1	-22.6	36.5	<fca = 125
	Bottom fiber	-53.3	86.8	33.5	>f'ta= 0
At working stage (pipe filled)	Top fiber	88.7	-22.6	66.1	<fca = 125
	Bottom fiber	-80.0	86.8	6.8	>f'ta = 0

Note: Minus means tensile stress

7) Stress resultant of prestressing steel at working stage

Stress of the lowest prestressing steel is maximum value

$$fse = 105.0\ kg/mm^2 \leq fsa = 105.0\ kg/mm^2$$

Design of Pipeline

1) Section properties

Interior diameter	Do	=	1,347.8 mm
Exterior diameter	Do'	=	1,371.6 mm
Effective interior diameter	D	=	1,349.8 mm
Effective exterior diameter	D'	=	1,369.6 mm
Average diameter	Dm	=	1,359.7 mm
Thickness	to	=	11.9 mm
Effective thickness	t	=	9.9 mm
Sectional area	Ap	=	508.3 cm^2
Effective sectional area	Ape	=	422.9 cm^2
Effective section modulus	Zp	=	7,136.0 cm^3
Effective radius of gyration	γ	=	48.1 cm

2) Tensile stress caused by internal pressure

$fh = Po \cdot D/2t$

where Po : Internal pressure = 7.9 kg/cm^2

$fh = 7.9 \times 134.98/2 \times 0.99$

$= 539\ kg/cm^2 < 1.5\ fsa = 2,100\ kg/cm^2$

3) Stress caused by self weight and external load

Vertical load factor

Pipe self weight		400 kg/m
Water self weight		1,425 kg/m
Total	Wd =	1,825 kg/m

Horizontal load factor

Wind load	Wh1=	137 kg/m
Seismic load	Wh2=	91 kg/m

Therefore, windload is employed.

Bending moment as continuous beam

$M = 0.11 \times Wd \times l^2$

Where l : Span length = 377.5 cm

$M = 0.11 \times 1,825 \times 10^{-2} \times 377.5^2 = 286,081$ kgcm

Bending stress

$fb = M/Zp$

$= 286081/7136.0$

$= 40\ kg/cm^2 < 0.9\ fba = 0.9 \times 1,400 = 1,260\ kg/cm^2$

4) Stress caused by axial force

Axial force caused by support at time of temperature change

$P1 = Wd \times l' \times f$

where l' : Influential length of pipe = 4,800 cm
f : Coefficient of friction = 0.25

$P1 = 1{,}825 \times 4{,}800 \times 10^{-2} \times 0.25 = 21{,}900$ kg

Axial force caused by dresser type joint at time of temperature change

$P2 = \pi \times Do' \times f'$

where f' : Friction intensity = 7.0 kg/m

$P2 = 3.14 \times 137.16 \times 7.0$
$= 3{,}015$ kg

Axial force caused by internal pressure at dresser type joint

$P3 = Po \times \pi \times Dm \times t$
$= 7.9 \times 3.14 \times 135.97 \times 0.99$
$= 3{,}339$ kg

Total axial force

$P = P1 + P2 + P3$
$= 21{,}900 + 3{,}015 + 3{,}339$
$= 28{,}254$ kg

Axial stress

$fp = P/Ape$
$= 28{,}254/422.9$

$= 67 \text{ kg/cm}^2 < fca = 736 \text{ kg/cm}^2$

because $\ell/r = 98$

$fca = 12 \times 10^6/(6{,}700 + (\ell/r)^2) = 736 \text{ kg/cm}^2$

5) Bending stress caused by restraint at ring support

$$fpr = 1.82 \times \left(\frac{Ar - Bo \times t}{Ar}\right) \times \frac{Po \times D}{2t}$$

where Ar : Sectional area of ring support = 72.2 cm^2
Bo : Effective width = 29.3 cm

$$fpr = 1.82 \times \left(\frac{72.2 - 29.3 \times 0.99}{72.2}\right) \times \frac{7.9 \times 134.98}{2 \times 0.99}$$

$= 586 \text{ kg/cm}^2 < fba = 1{,}400 \text{ kg/cm}^2$

6) Total longitudinal stress

$$\frac{fp}{fca} + \frac{fpr}{fba}$$

$$= \frac{67}{736} + \frac{586}{1400}$$

$$= 0.51 < 1.00$$

Design of Substructure

1) Design loads at top of pile

In normal case

a) Vertical force

Dead load of bridge	96.6 ton
Dead load of pipeline	64.0 ton
Live load	32.5 ton
Self weight of footing	54.0 ton
Total	247.1 ton

b) Horizontal force (parallel to bridge axis)

Axial force caused by internal Pressure at drersser type joint	3.3 ton

c) Horizontal force (perpendicular to bridge axis)

No forces act

At time of temperature change

a) Vertical force

Same as "In normal case"	247.1 ton

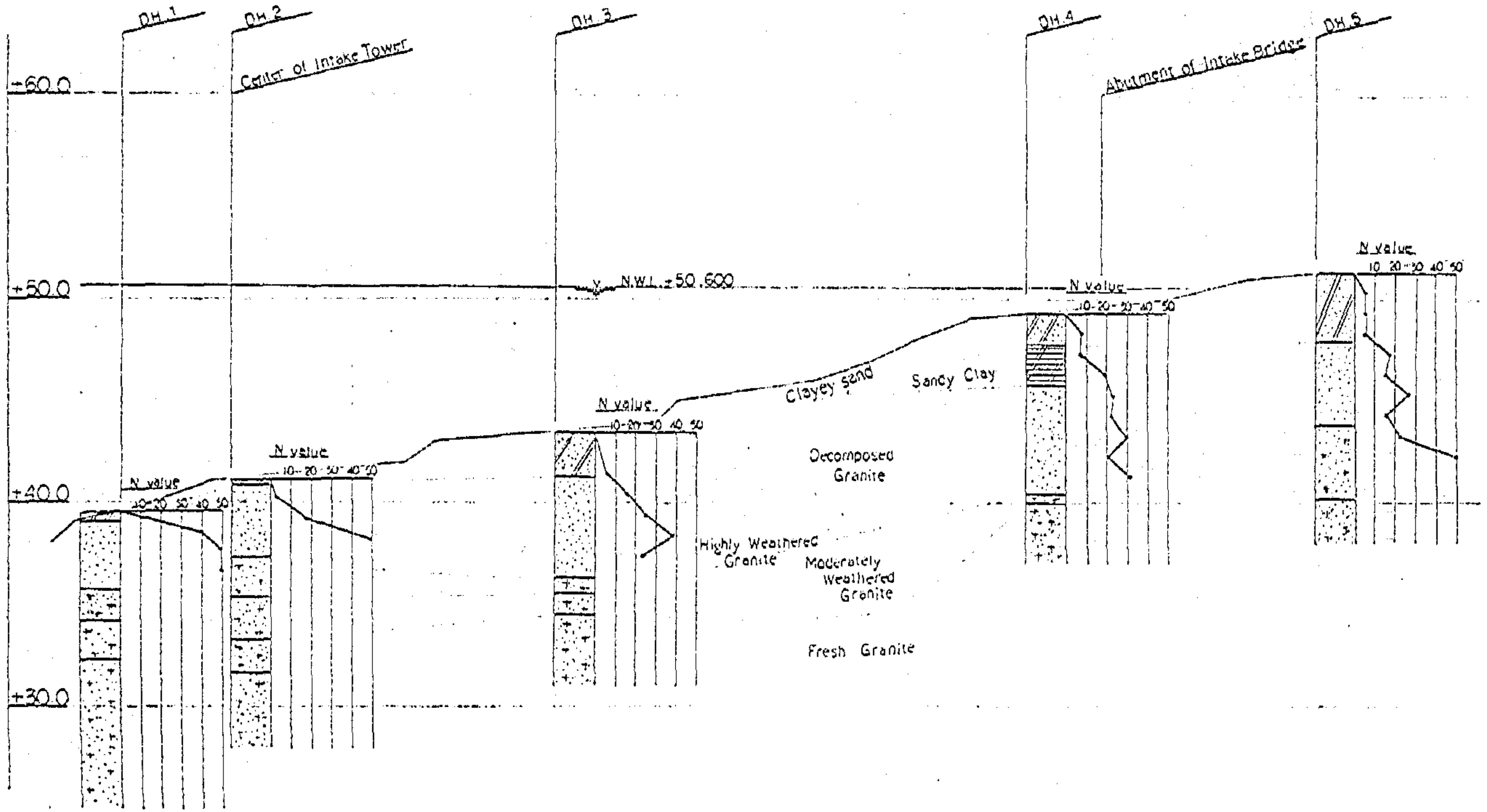
b) Horizontal force (parallel to bridge axis)

Axial force caused by friction of dresser type joint	3.0 ton
Axial force caused by internal pressure at dresser type joint	3.3 ton
Total horizontal force	6.6 ton

c) Horizontal (perpendicular to bridge axis)

No forces act

Fig.1-2-20 Geological Profile along the Intake Bridge



At time of earthquake

a) Vertical force

Dead load of bridge	96.6 ton
Dead load of pipeline	64.0 ton
Self weight of footing	54.0 ton
Total	214.6 ton

b) Horizontal force (parallel to bridge axis)

214.6 x 0.05 = 10.7 ton

c) Horizontal force (perpendicular to bridge axis)

214.6 x 0.05 = 10.7 ton

At time of storm

a) Vertical force

Same as "At time of earthquake" 214.6 ton

b) Horizontal force (parallel to bridge axis)

No forces act

c) Horizontal force (perpendicular to bridge axis)

Pipe (upstream)	3.8 ton
Bridge (downstream)	0.8 ton
Total	4.6 ton

2) Stability of substructure (P_1 pier)

In normal case

Horizontal force H = 3.3 ton is small enough to neglect compared with vertical force V = 247.1 ton.

Footing displacement

dh = 0
dv = 3.1 mm
θ = 0

Maximum pile reaction

Pni = 41.2 ton (=247.1/6) < Ra = 50 ton

At time of earthquake

a) Stability parallel to bridge axis

Vertical force	214.6 ton
Horizontal force	10.7 ton
Moment	12.9 ton (= 10.7 x 1.2)

Footing displacement

dh = 24.0 mm < dha = 50 mm
dv = 2.6 mm
θ = 1.3356×10^{-3} rad.

Maximum pile reaction

Pni = 49.0 ton < Ra' = 50 x 1.5 = 75 ton

b) Stability perpendicular to bridge axis

Vertical force	214.6 ton
Horizontal force	10.7 ton
Moment	23.6 ton (=10.7 x 2.2)

Footing displacement

dh = 36.0 mm < dha = 50 mm
dv = 2.5 mm
θ = 0.9607×10^{-3} rad

Maximum pile reaction

PNi = 42.0 ton < R'a = 50 x 1.5 = 75 ton

Other cases

"At time of temperature change" and "At time of storm" horizontal forces are smaller than that of "At time of earthquake".

3) Stress of pile

Maximum axial force occurs "In normal case"

$$P = PNi = 40.7 \text{ ton} < \frac{Pcr}{n} = 258 \text{ ton}$$

where Pcr : Euler's backling load

$$Pcr = \frac{\pi^2 \times EI}{2}$$

$$= \frac{3.14^2 \times 2.1 \times 10^7 \times 0.000287}{10.0^2}$$

= 593.4 ton

n = Safety factor 2.3

Maximum bending moment occurs "At time of earthquake", maximum bending stress is expressed as below.

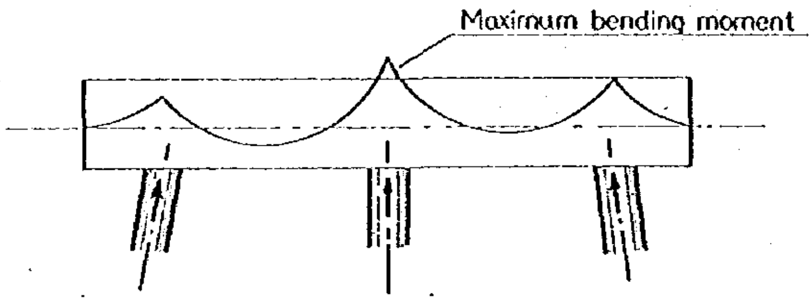
$$f_{ta} = \frac{P_{Ni}}{A} + \frac{M_{ti}}{Z}$$

$$= \frac{49.0 \times 10_3}{94.0} + \frac{9.08 \times 10_5}{1146}$$

$$= 1,314 \text{ kg/cm}^2 < f_{ta} = 1.5 \times 1400 = 2,100 \text{ kg/cm}^2$$

4) Stress of footing

Maximum bending moment occurs at center of footing in normal case.



$$M = (40.7 \times 2 - 3,699 \times 16.0) \times \frac{5,500}{2} = 63.9 \text{ ton} \cdot \text{m}$$

Main reinforcement

D25 @ 200 $\quad$ As = 113 cm^2/m

Concrete compressive stress

$f_c = 40 \text{ kg/cm}^2 < f_{ca} = 94.5 \text{ kg/cm}^2$

Reinforcing bar tensile stress

$f_s = 1,313 \text{ kg/cm}^2 < f_{sa} = 1,400 \text{ kg/cm}^2$

1.2.3 Air Chamber

General

The purpose of this section is to design substructure of the air chamber. Proposed site of the air chamber is on the embankment, called intake yard, of projecting shore close to intake tower. R.C. pile foundation is designed taking geographical, geological, economical, construction technical and weight of the structure into consideration.

Design Conditions

1) Dead load

Weight of air chamber with appertenant facilities

Air chamber	3 x 15.0	= 45.0
Water	3 x 30.0	= 90.0
Appurtenant facilities	3 x 5.0	= 15.0
		150 t

Weight of foundation

Base slab	15.5 x 8.8 x 0.6 x 2.5	=204.6
Wall	2x(15.25+8.85)x3.0x0.25x2.5	= 89.3
Cinder concretre	15.0 x 8.3 x 0.1 x 2.3	= 28.6
		327.5
	Total	480 t

2) Seismic coefficient

kh = 0.05

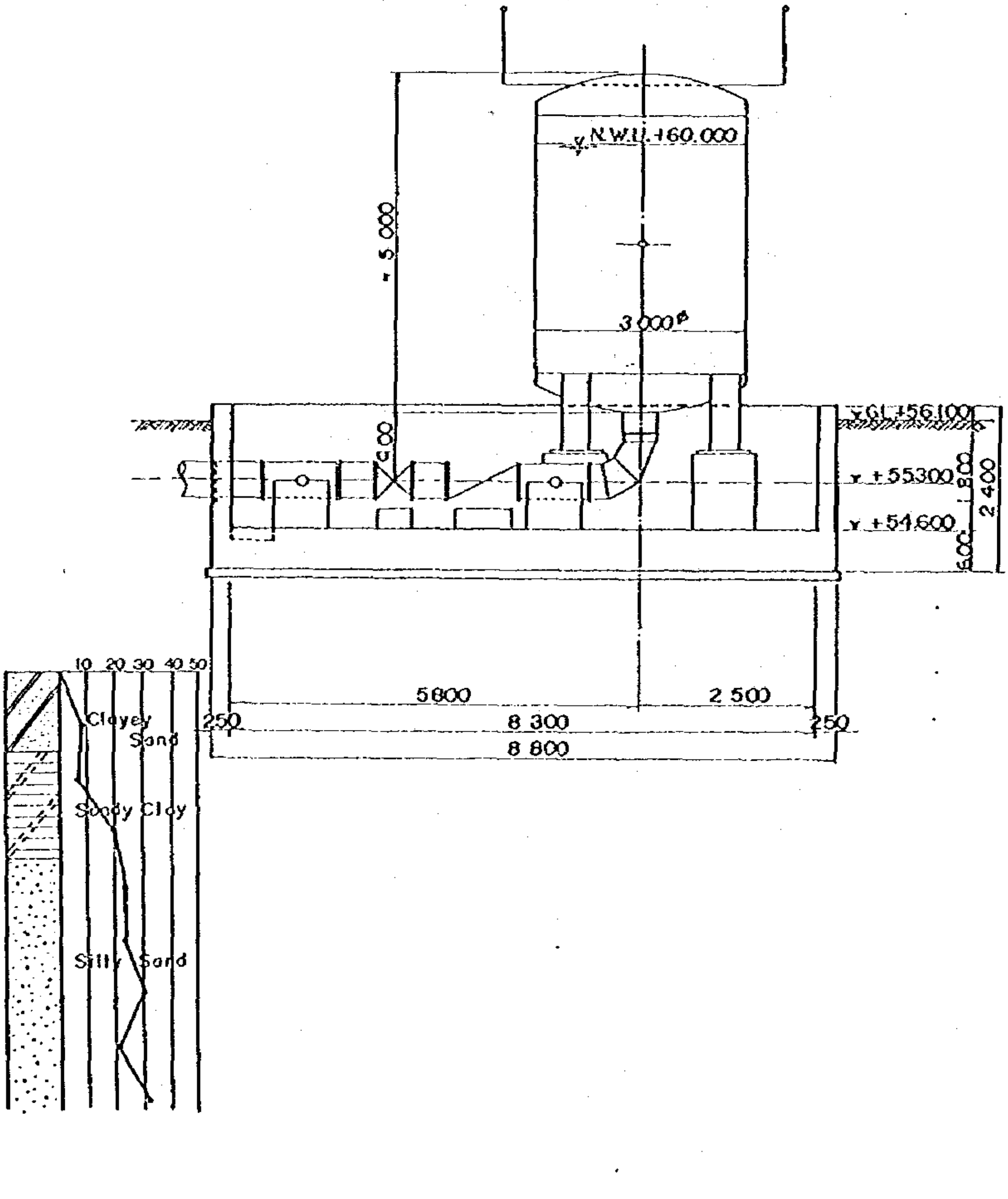
Design of R.C. Pile Foundation

Demension of the pile

Length L = 7.0 m

Section $D_1 \times D_2$ = 30.0 x 30.0 cm

Fig.I-2-21 Typical Profile of Air Chamber



Vertical bearing capacity of pile

a) Calculating formula of vertical bearing capacity

Allowable bearing capacity of pile is obtained by the following formula.

$$Ra = \frac{1}{n} Ru$$

Here
- Ra: Allowable bearing capacity of pile (t)
- n : Safety factor
- Ru: Ultimate bearing capacity of pile (t)

where;

$$Ru = qd \cdot A + U \cdot lifi$$

Here;
- qd: Ultimate bearing stress at the pile tip (t/m^2)
- A : Area of pile tip (m^2)
- U : Circumferential length of pile (m)
- li: Stratum depth (m)
- fi: Maximum skin friction of stratum (t/m^2)

b) Calculation of vertical bearing capacity

qd is calculated from mean N-value at pile tip and penetration ratio

$\bar{N} = 20$

$qd = 20 \times 22 = 440\ t/m^2$

$qD/\bar{N} = 20$

Skin friction is calculated from

$f = 0.2 \times N$

where putting, $l_1 = 3.2$ m $N_1 = 10$, $f_1 = 2.0\ t/m^2$,

$l_2 = 5.8$ m $N_2 = 15$ $f_2 = 3.0\ t/m^2$

Ultimate bearing capacity is

$$Ru = qd \cdot A + U \cdot lifi$$

$$= 440 \times 0.30^2 + 4 \times 0.3 \times (3.2 \times 2.0 + 5.8 \times 3.0)$$

$$= 40 + 28 = 68\ t/pcs.$$

Allowable bearing capacity is

$$Ra = 1/3 \times Ru = 22 \text{ t/pcs.}$$

If the arrangement of piles are $4 \times 6 = 24$ pcs, total capacity is

$$24 \times 22.0 = 528 \text{ t} \quad > 480 \text{ t} \; ..... \; \text{O.K.}$$

Lateral bearing capcity of pile

a) Calculating formula of horizontal bearing capacity

Allowable lateral bearing capacity is obtained by the following formula.

$$Ha = kD/\beta \, \delta a$$

Here;

Ha; Allowable lateral bearing capacity (kg)

k ; Coefficient of lateral subgrade reaction (kg/cm^3)

D ; Pile diameter (cm)

β ; $= 4 \sqrt{kD/4EI}$

δa; Fundamental displacement (cm)

b) Calculation of horizontal bearing capacity

It is enough to calculate only the case of earthquake.

Coefficient of lateral subgrade reaction (k-value) is estimated by

$$k = 0.2 \times 28 \times 10 \times 30^{-3/4} = 4.4 \text{ kg/cm}^3$$

Therefore,

$$\beta = 4 \sqrt{k.D/4EI}$$

$$= 6.4 \times 10^{-3} \text{ cm}^{-1}$$

Here;

$D = 30$ cm
$I = 67500 \text{ cm}^4$
$E = 2.9 \times 10^5 \text{ kg/cm}^2$

Lateral bearing capacity is estimated by

$$Ha = 4.4 \times 30.0/6.4 \times 10^{-3} \times 1.0$$

$$= 20.6 \text{ t/pcs} \quad > 480 \times 0.05/24 = 1.0 \text{ t/pcs.}$$

c) Design against moment at pile head

Bending moment at pile head is calculated by

$Mo = H/2 \cdot \beta$

Here; Mo; Bending Moment at pile head (t.m)

H ; Sear force at pile head (t)

and putting

$H = 1.0$ t, $\beta = 6.4 \times 10^{-1}$m

$Mo = 1.0/(2 \times 6.4 \times 10^{-1})$

$= 0.78$ t·m

And it is easily concluded that D-12, 8 pcs is enough.

Design of Footing Slab

1) Design condition

According to the pile pitch, unit width of 2.5 m of shorter width is taken into calculation as a beam member.

Dimension of member

Area of the section	$A = 1.5$ m^2
Moment of inertia	$I = 0.045$ m^4
Young's modulus	$E = 2.9 \times 10^6$ t/m^2

Spring constant of pile

$K_v = a \cdot A \cdot E/l = 25600$ t/m

$K_1 = 4EI\beta^3 = 2050$ t/m

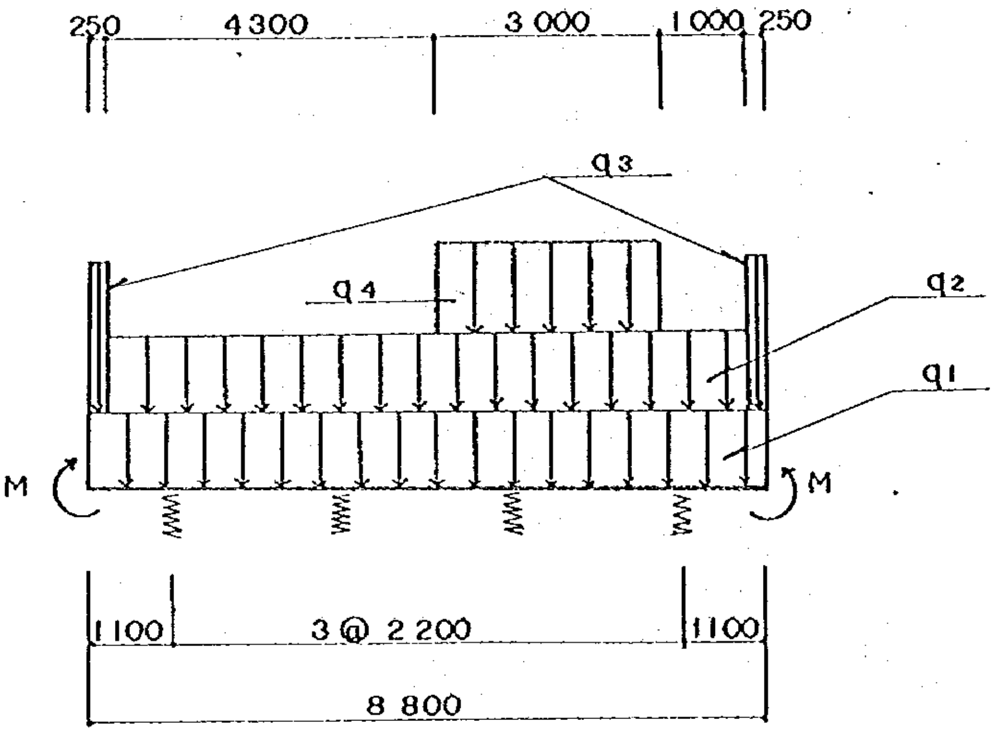
$K_2 = K_3 + 2EI\beta^2 = 1600$ t.m/m

$K_4 = 2EI\beta = 2500$ t.m/rad

Load

Dead load	$q_1 = 2.5 \times 0.6 \times 2.5$	= 3.75 t/2.5 m
Particular load	$q_2 = 1.0 \times 2.5$	= 2.50 t/2.5 m
Wall weight	$q_3 = 2.5 \times 1.8 \times 2.5$	= 11.25t/2.5 m
Air chamber	$q_4 = 150.0/(3.0 \times 10.0) \times 2.5$	= 15.0 t/2.5 m
Moment by earth pressure	$M = 1.68 \times 2.5$	= 4.2 t.m/2.5 m

Load Diagram (In normal case)

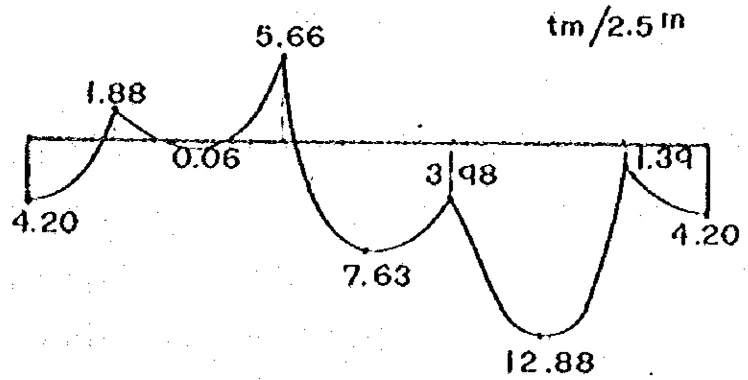


2) Stress analysis of beam on pile foundation

The beam on pile is analyzed by displacement method. As it is expected that stress level at the time of earthquake is not greater than 1.5 times that of in normal case, it is enough to calculate only the normal case.

Bending moment

Bending moment diagram



1.3 PIPELINE

1.3.1 Wall Thickness of Pipe

Formula

1) Circumferential stress by internal pressure

The following equation shall be used in calculating the circumferential stress.

$$\sigma ct = \frac{PD}{2t}$$

σct : stress, kg/cm^2
P : internal pressure, kg/cm^2
D : average diameter of pipe, cm
t : thickness of pipe wall, cm

2) Deflection and bending stress

The deflection and bending stress of steel pipe under the earth pressure is calculated by the following formula:

Deflection

$$Dx = \frac{2kx \cdot (Wv + Wt)R^4}{EI + 0.061\ E'R^3}$$

Bending stress

$$\sigma b = \frac{2}{fZ}(Wv+Wt)\ \frac{Kb \cdot R^2EI + (0.061\ Kb - 0.083\ Kx)E'R^5}{EI + 0.061\ E'R^3}$$

Dx : horizontal deflection, cm
σb : bending stress at pipe bottom, kg/cm^2
f : coefficient by shape, 1.5
Z : section modulus, cm^3
Wv : vertical earth pressure, kg/cm^2
Wt : load by truck, kg/cm^2
R : average radius of pipe, cm
E : modulus of elasticity of steel, 2,100,000 kg/cm^2
I : geometrical moment of inertia, cm^4
E' : coefficient of passive earth pressure, kg/cm^2
Kb : coefficient of bending moment at pipe bottom, 0.157
Kx : coefficient of horizontal displacement, 0.096

3) Result of calculation

a) Pipe dia. : D = 1,350 mm

b) Wall thickness : t = 11.9 mm

c) Circumferential stress

Internal pressure : $P = 9\ kg/cm^2$
$\sigma_t = 510\ kg/cm^2 < 1{,}400\ kg/cm^2$

d) Deflection and bending stress

Live load : 20 tons truck (T - 20)
Cover depth of pipe : $H = 3.50\ m$
Coefficient of passive earth pressure: $E' = 14\ kg/cm^2$

Deflection $D_x = 3.87\ cm < 6.75\ cm$
Bending stress $= 1{,}332\ kg/cm^2 < 1{,}400\ kg/cm^2$

1.3.2 Appurtenant Facilities

Air Valve

Rapid-type air valves are to be employed so as to facilitate quick filling and release of water from the pipeline, which have relatively large diameter and extention. In addition to this, three air valves are placed near receiving facilities to increse the operational reliability of air valves in presence of water hammer.

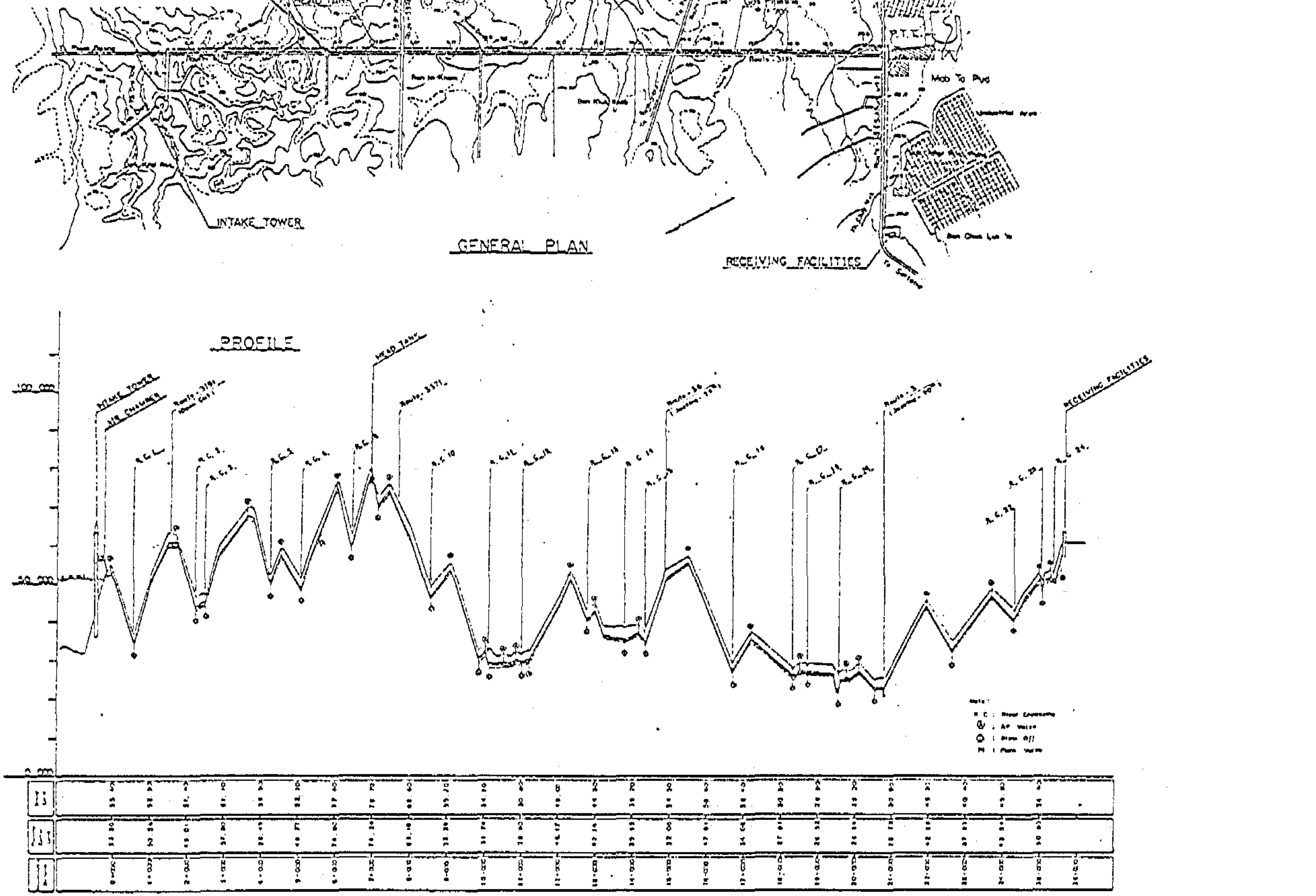
Blow-Off

Blow-offs are placed where the pipeline crosses a river, to both sides of crossing. This is to ease, as is the case for air valves, the drain of water from the pipeline for operation and maintenance purpose.

Main Valve

Main valves are installed to the down-stream of blow-offs for the purpose of optimizing the function of blow-offs as well as for other operation and maintenance works. The main valves are placed in 1 to 3 km intervals, i.e., when the pipeline crosses a river, so that they can serve the two purposes mentioned above.

Fig. 1-3-1 General Plan and Profile

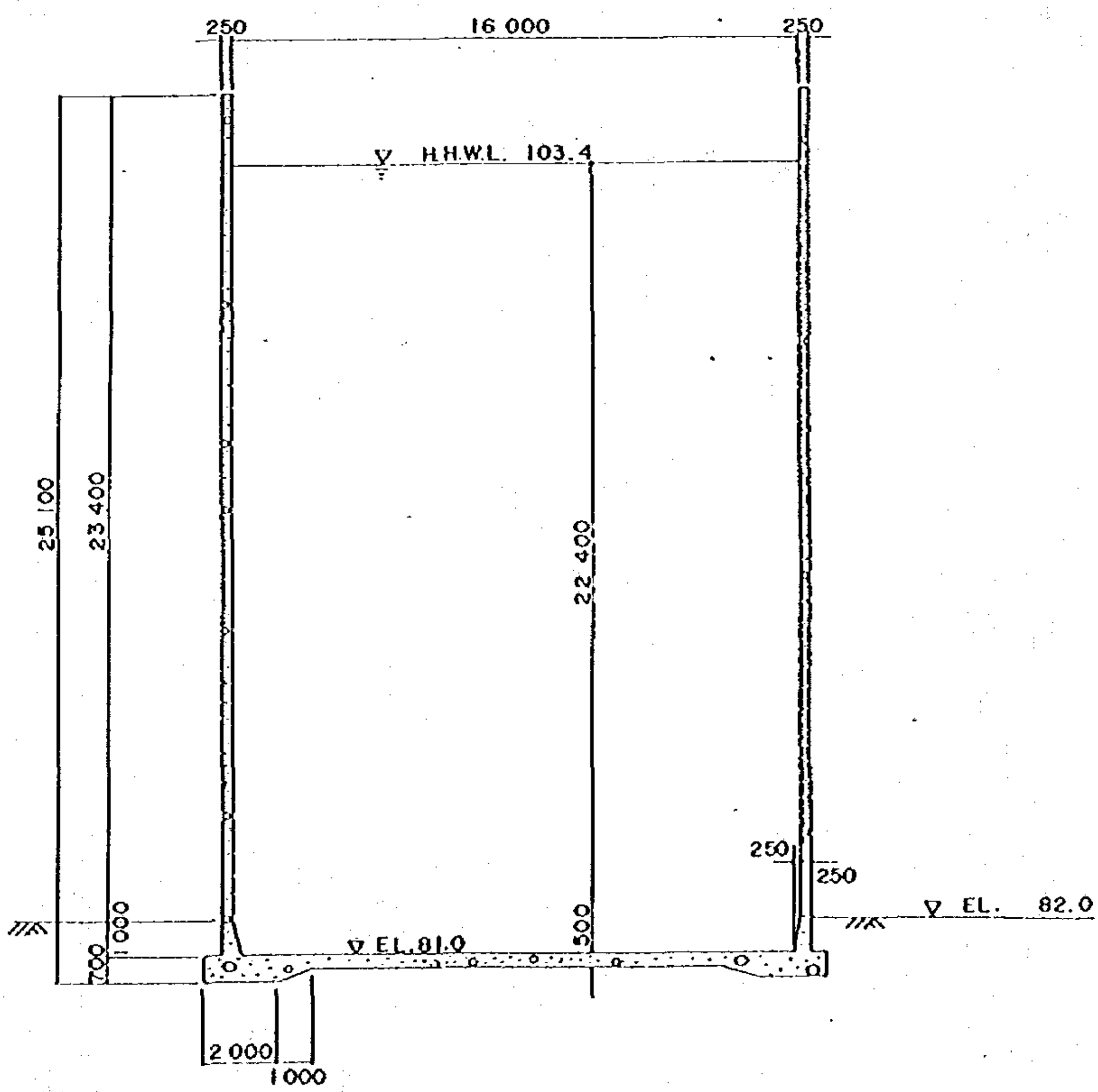


1.4 HEAD TANK

1.4.1 Assumed Dimensions

Main dimensions of the head tank are D = 16 m, H = 24.4 m as shown below.

Fig.1-4-1 Sectional Dimension of Head Tank

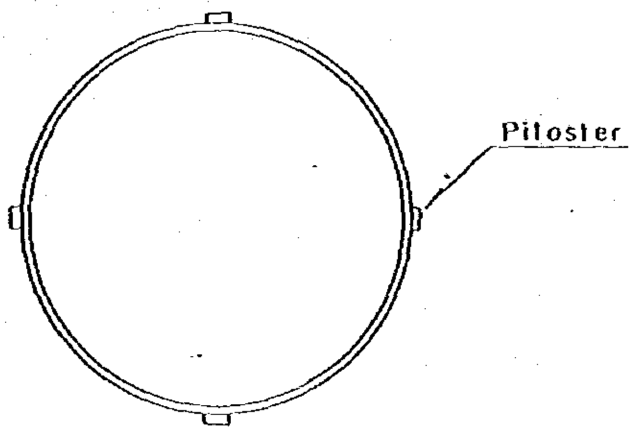


1.4.2 Calculation for Tank Wall

Basic Data

1) Prestress shall be introduced to remain 5 kg/cm^2 compressive hoop stress in concrete when the water level is EL 103.4 m (Design Water Level).

2) The pilasters are installed at 4 places as shown below.

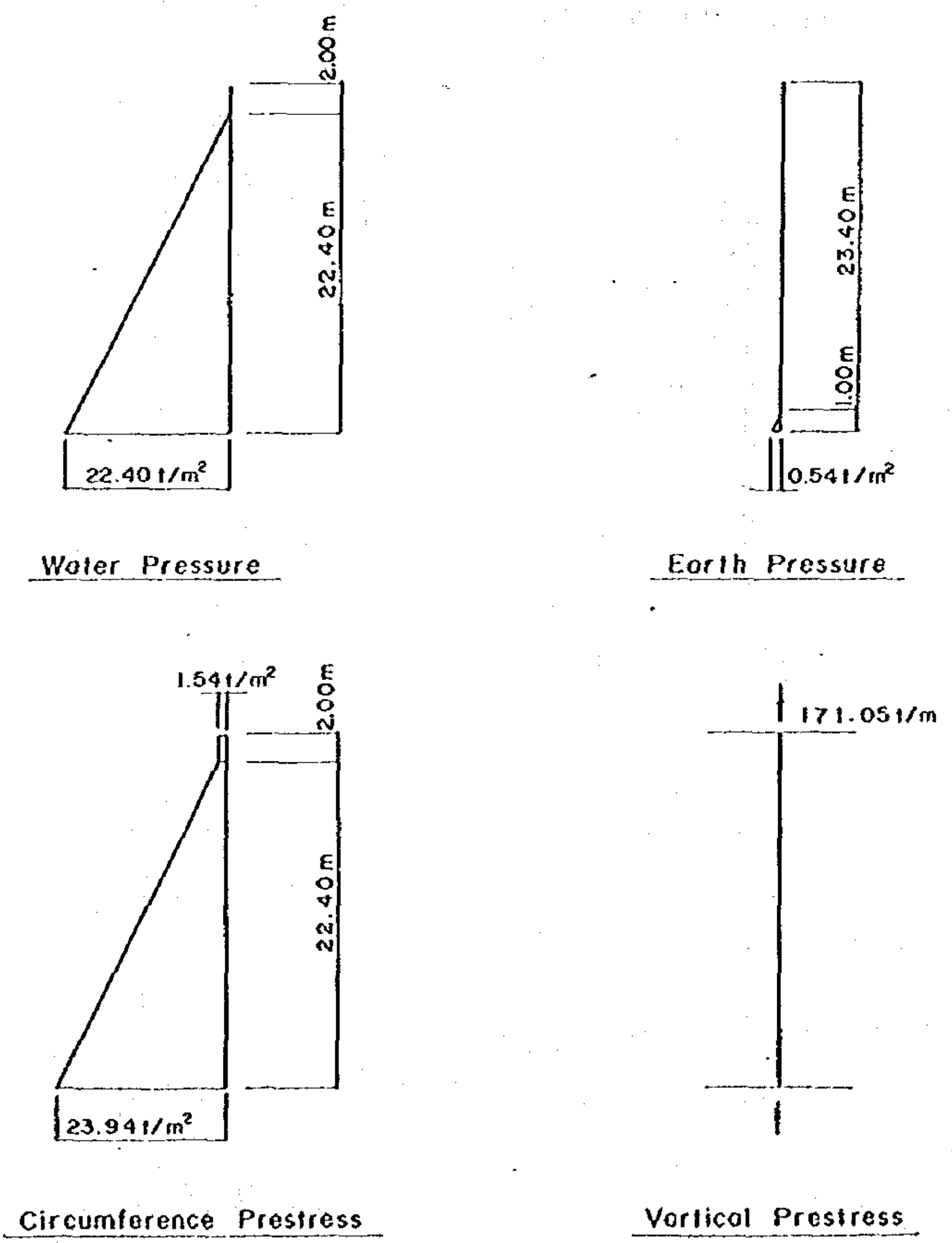


3) Coefficient of Effective Prestress 0.85

Load Diagram (Normal Case)

Loads to be considered are water pressure, earth pressure, circumference prestress and vertical prestress in a normal case.

Fig. 1-4-2 Load Diagram



Alignment of Circumference PC Strand

1) Mean Prestress of Each PC Steel Bar

$$P_e = \frac{1}{\alpha}\int_0^{\alpha} 0.85 \times P_t \times e^{(-\mu x - \lambda R x)}\, dx$$

P_e = Mean prestress (t)
P_t = Introduced prestress (t)
α = Range of Degree of PC stress (radian)
μ = Coefficient of friction for range of degree
λ = Coefficient of friction for undulation of sheath
R = Radius of the Head Tank (m)
P_e = 27.08 (t) (Strand ϕ 21.8 is used)

2) Interval of strand

$$H(I) = -\sqrt{(L-H(I-1))^2 - \frac{2\cdot Pe}{R'} + \frac{Pa\cdot T}{R'}\{2\cdot L + \frac{Pa\cdot T}{R} - 2H(I-1)\}} + L + \frac{Pa\cdot T}{R'}$$

$$Y(I) = \frac{H(I) + H(I-1)}{2} \quad (I \geq 2)$$

$$H(1) = 2 \cdot Y(1)$$

$$Y(1) = \frac{(L\cdot R' + Pa\cdot T) - \sqrt{(L\cdot R' + Pa\cdot T)^2 - R'\cdot Pe}}{R}$$

$Y(I)$	:	Height of PC Strand from Bottom of the Tank Wall	(m)
L	:	Design Water Depth	(m)
Pa	:	Residual Prestress	(t/m^2)
R'	:	R + 1/2 x Wall Thickness	(m)
P_e	:	Mean Prestress	(t)
T	:	Wall Thickness	(m)

Interval of Strand

I	Y(I) (M)	Interval (M)	I	Y(I) (M)	Interval (M)	I	Y(I) (M)	Interval (M)
1	.07	.07	2	.21	.14	3	.35	.14
4	.49	.14	5	.64	.14	6	.78	.14
7	.92	.14	8	1.07	.15	9	1.21	.15
10	1.36	.15	11	1.51	.15	12	1.66	.15
13	1.81	.15	14	1.96	.15	15	2.11	.15
16	2.27	.15	17	2.42	.15	18	2.58	.16
19	2.73	.16	20	2.89	.16	21	3.05	.16
22	3.21	.16	23	3.37	.16	24	3.53	.16
25	3.70	.16	26	3.86	.17	27	4.03	.17
28	4.20	.17	29	4.37	.17	30	4.54	.17
31	4.71	.17	32	4.88	.17	33	5.06	.18
34	5.24	.18	35	5.42	.18	36	5.60	.18
37	5.78	.18	38	5.96	.18	39	6.15	.19
40	6.34	.19	41	6.53	.19	42	6.72	.19
43	6.92	.19	44	7.11	.20	45	7.31	.20
46	7.51	.20	47	7.72	.20	48	7.93	.21
49	8.14	.21	50	8.35	.21	51	8.56	.22
52	8.78	.22	53	9.00	.22	54	9.23	.22
55	9.46	.23	56	9.69	.23	57	9.92	.24
58	10.16	.24	59	10.41	.24	60	10.66	.25
61	10.91	.25	62	11.17	.26	63	11.43	.26
64	11.70	.27	65	11.98	.28	66	12.26	.28
67	12.55	.29	68	12.84	.30	69	13.15	.30
70	13.46	.31	71	13.79	.32	72	14.12	.33
73	14.47	.35	74	14.82	.36	75	15.20	.37
76	15.59	.39	77	16.00	.41	78	16.43	.43
79	16.89	.46	80	17.38	.49	81	17.91	.53
82	18.49	.58	83	19.14	.65	84	19.90	.76
85	20.86	.95	86	22.32	1.46	87	24.17	1.85

Alignment of Vertical PC Steel Bar

1) PC steel bar ϕ32mm is used.

2) Interval of bar

Maximum moment caused by circumference prestress in the case the tank is vacant = 26.22 t·m

The interval due to full prestress at the maximum mentioned above = 0.363 m is calculated.

The interval due to allowable compressive stress of concrete at the maximum moment = 0.341 m is calculated.

The interval of the Bar may be determine between 0.362 m and 0.341 m, and so it shall be 0.362 m here.

1.4.3 Analysis of Slab

Structural Analysis

Circular slab of the head tank is a kind of spread foundation, and it is designed as reinforced concrete structure. From stress analysis, it is assumed as a bending circular plate on elastic foundation.

Here, the slab is analyzed by finite element method.

Rigidity of Slab

Flexural rigidity of the slab is defined by

$$D = \frac{Eh^3}{12\,(1-\nu^2)}$$

Here; D; Flexural rigidity of the slab
E; Modulus of elasticity
ν; Poisson's ratio
h; Thickness of the slab

Putting

ν = 1/6, E = 2.6 x 10^6 t/m2 , and h = 0.5 m or 0.7 m

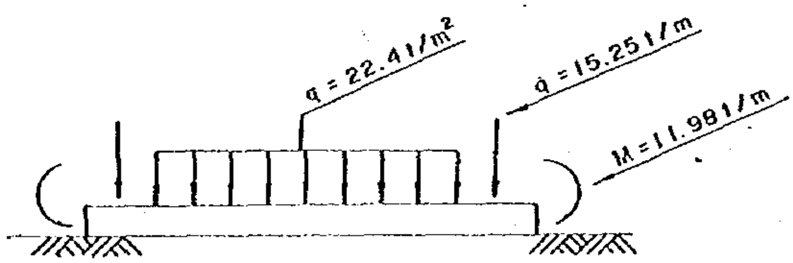
D_1 = 10,290 t·m (for center area of slab)

D_2 = 3,750 t·m (for periphery area of slab)

Load

In normal case

Loads acting on the base plate are dead load and weight of water.



Stability Analysis at Time of Earthquake

1) Analysis method

The analysis at time of earthquake is made by Housner's theory.

2) Seismic coefficient $K_H = 0.05$

3) Result of the analysis

Bending Moment at the bottom surface of the slab

$M_W = 2,684.0$ tm

Reaction at the bottom

$R_1 = 28.4$ t/m^2

$R_2 = 18.2$ t/m^2

4) Bearing capacity of the ground

From the result of geological survey at the site, minimum N-value of the standard penetration test of the existing ground is N = 35.

Bearing capacity at time of earthquake is calculated as 190 t/m^2.

Stability Analysis in Normal Case

Reaction at the bottom surface of the Slab

$R = 23.33$ t/m^2

Bearing capacity of the Ground is calculated as 110 t/cm^2.

At time of earthquake

Additional loads are moment loads acting on lower end of wall caused by dynamic water pressure and inertia force.

Total Moment: $Mo = 787.3$ t·m
Distributed Moment acting on lower end of wall; M

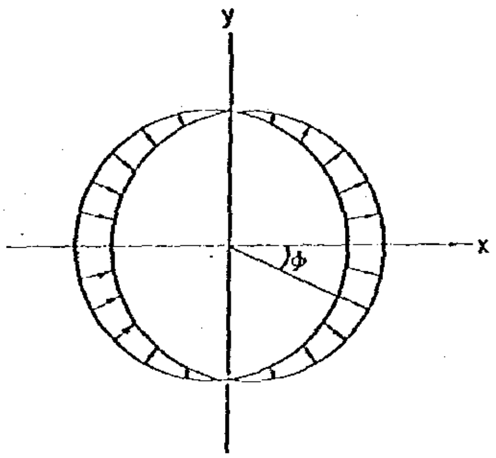
$$M = 787.3/(\phi_o \times 16.25)$$

$$= 15.42 \text{ t·m/m}$$

Then, distributed moment load on peripheral area is

$$Mx = 15.42 \times \cos^2 \phi$$

$$My = 15.42 \times \cos \phi \times \sin \phi$$



Result of the Analysis

From the analysis maximum moment are,

In normal case	$M\ max_1 = 0.55$ t·m/m
	$M\ max_2 = 0.61$ t·m/m
At time of earthquake	$M\ max_1 = 1.81$ t·m/m
	$M\ max_2 = 6.45$ t·m/m

Reinforcing bar arrangement is fixed based on these figures, they are,

Central area of slab	D-16 ctc 20.0 cm
Peripheral area of slab	D-16 ctc 15.0 cm

Fig.1-4-3 Model of Footing Slab

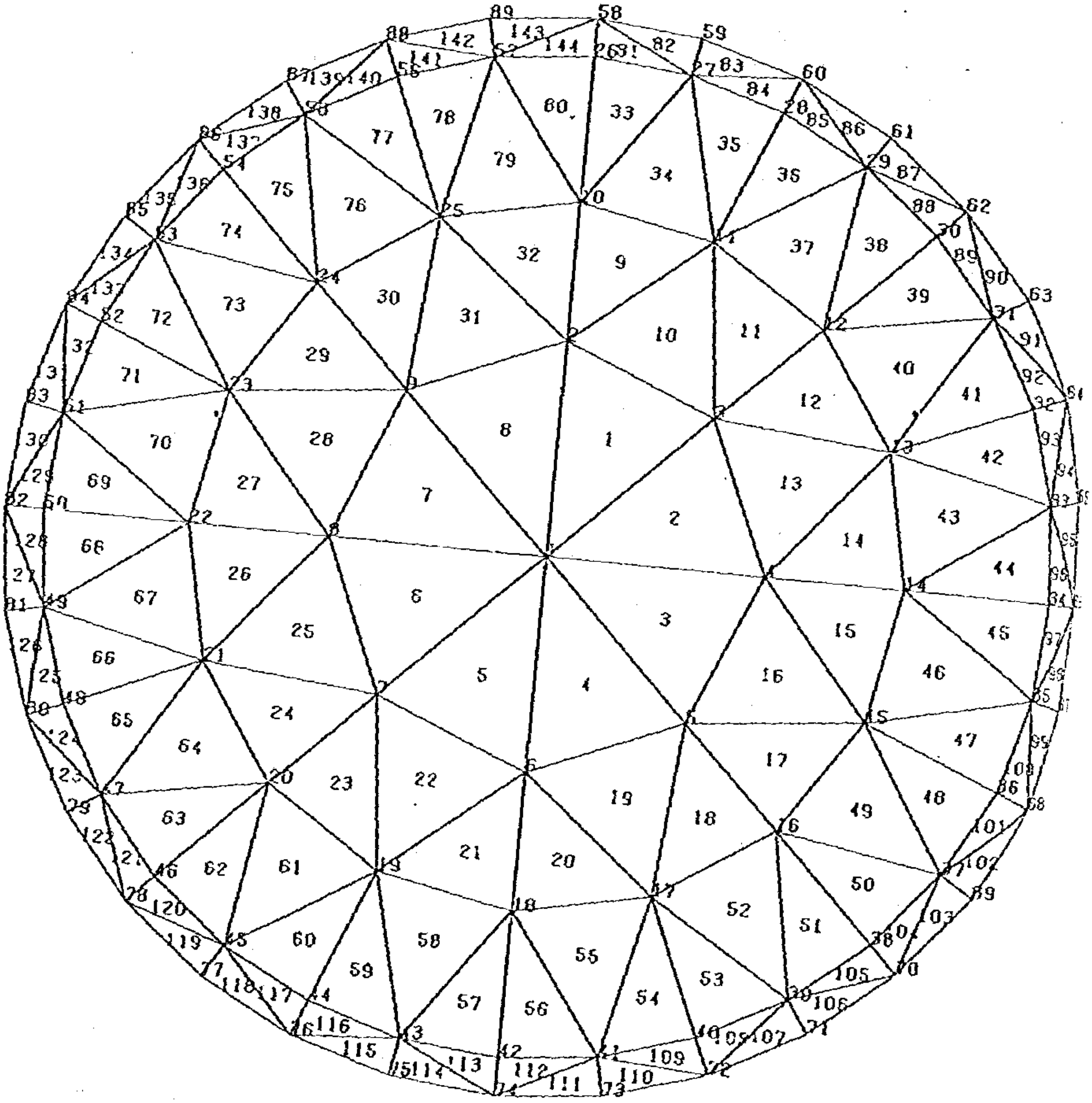
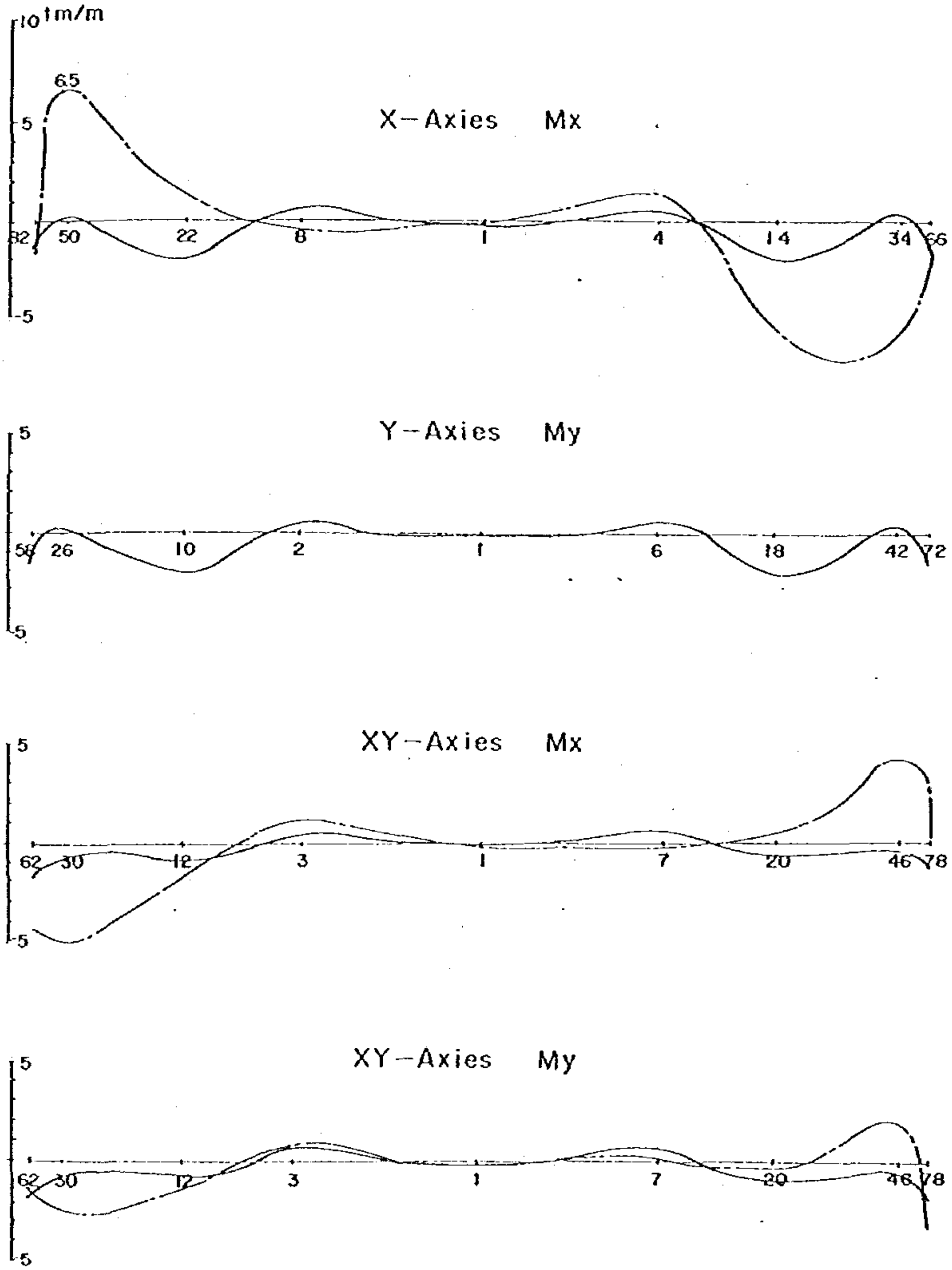


Fig. 1-4-4 Moment Diagra m



1.5 RECEIVING FACILITIES

1.5.1 Sketch of Receving Well

The dimensions of the receiving well such as wall thickness and size of reinforcing bar were determined by structural analysis. Fig. 1.5.1 through 1.5.4 show a plan and typical sections of the receiving well.

Fig. 1-5-1 Plan of Receiving Well

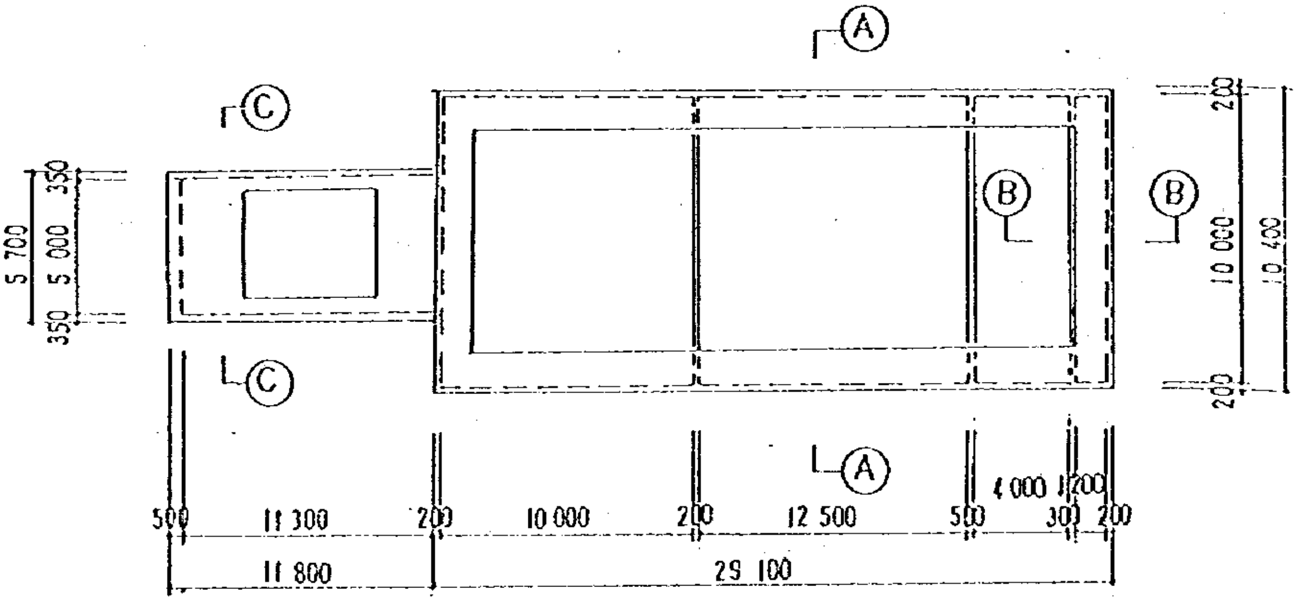


Fig. 1-5-2 A-A Section of Receiving Well

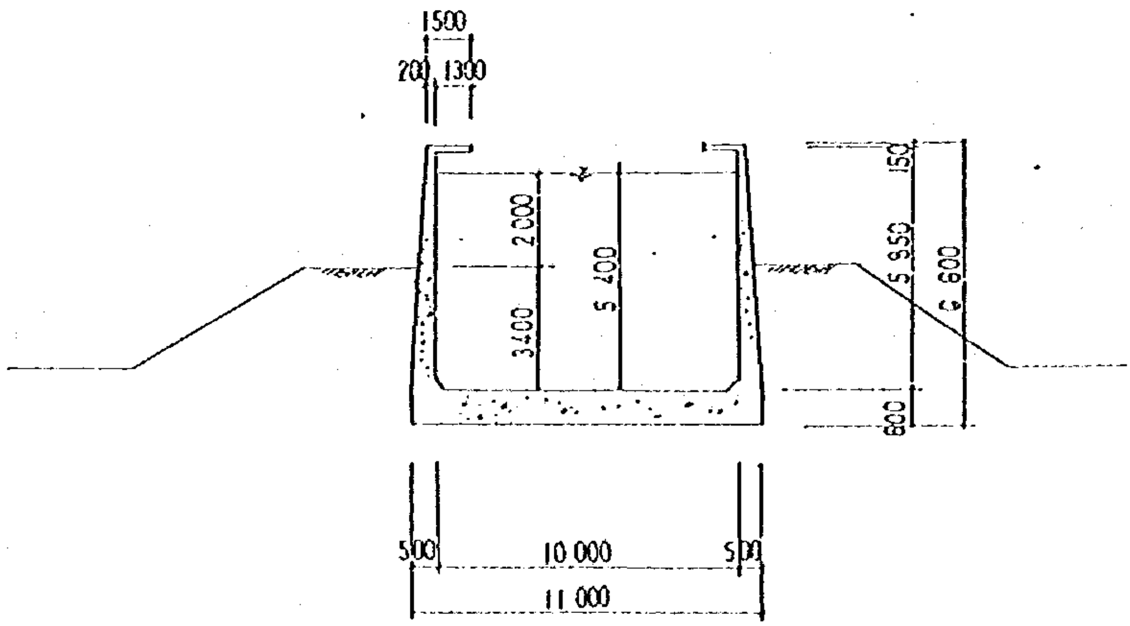


Fig. 1-5-3 B-B Section of Receiving Well

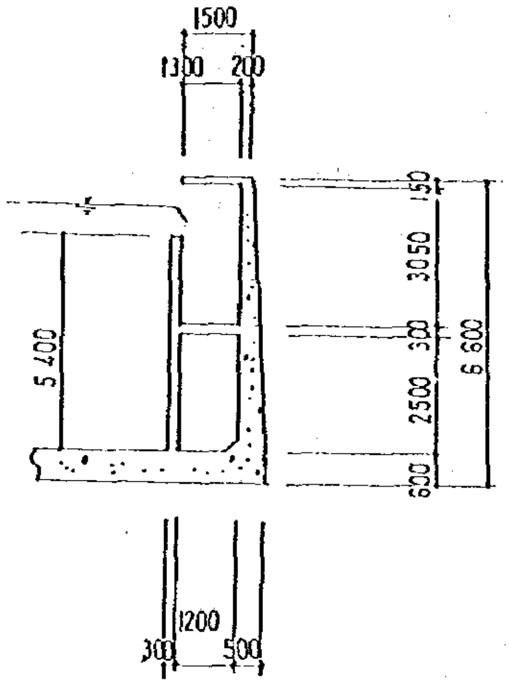
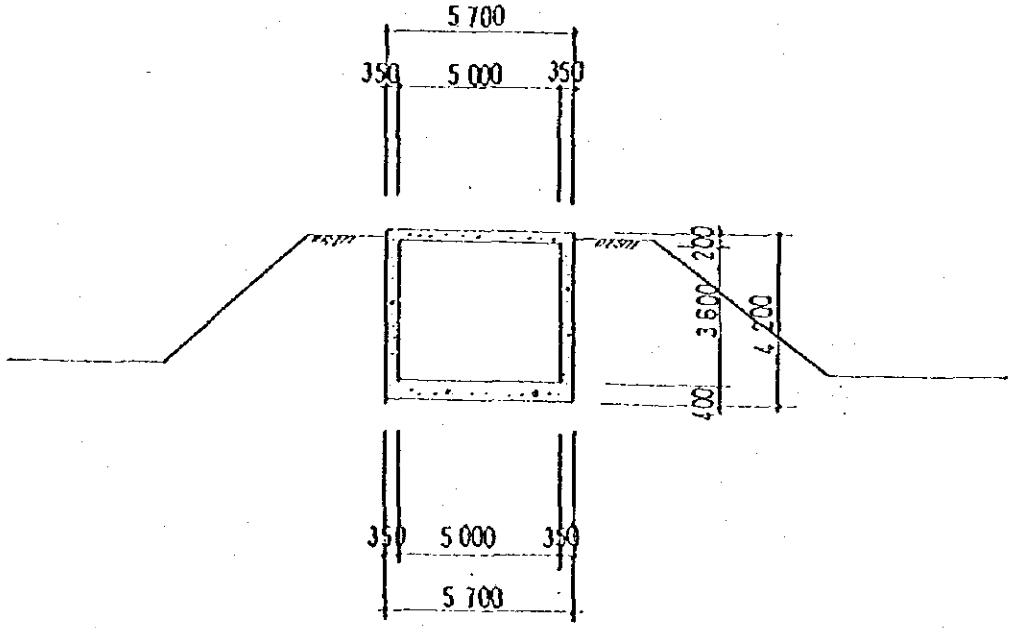


Fig. 1-5-4 C-C Section of Receiving Well

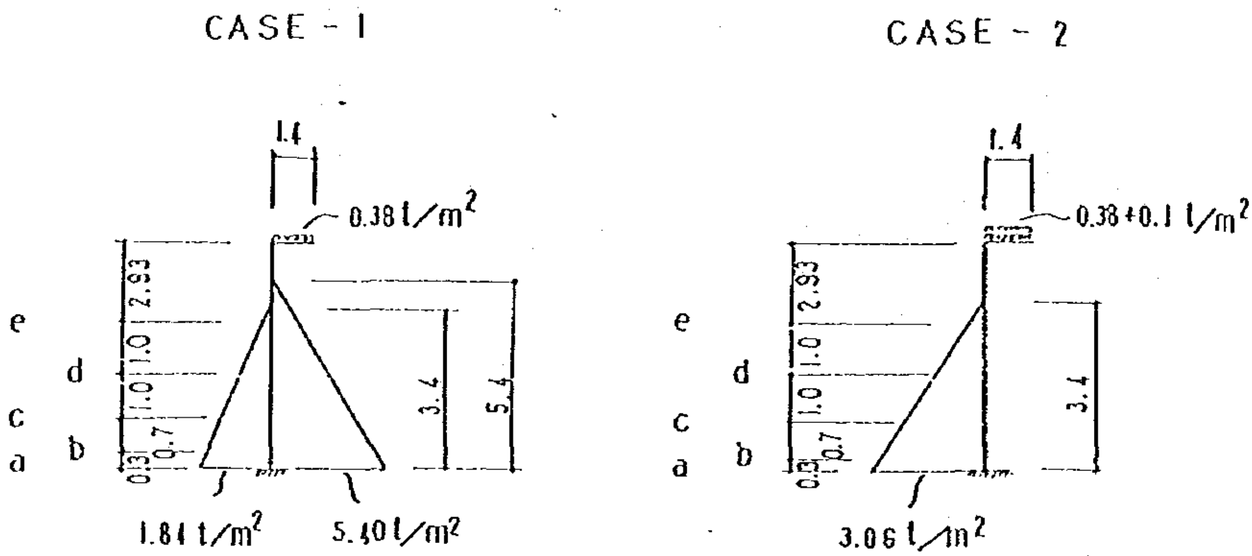


1.5.2 Structural Analysis

A-A Section

1) Loading diagram

Two cases were assumed for the analysis of A-A section: the first is the normal case (case 1) and the second is the case in which there is no water in the receiving well (case 2) as shown below.



2) Result of structural analysis

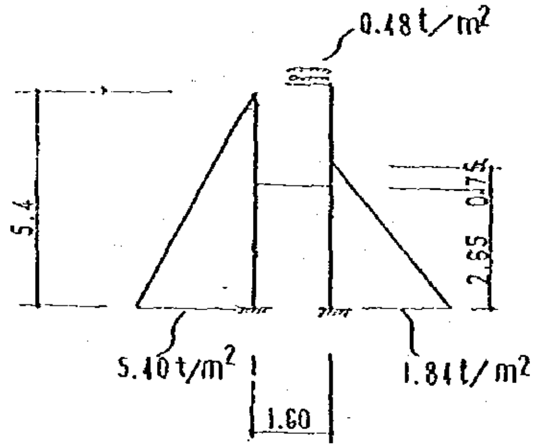
From the results of structural analysis, the following reinforcing bar shall be employed for each point of the well.

Point a	Inside of well	D25 @100 mm
	Outside of well	D19 @200 mm
Point b	Inside of well	D25 @100 mm
	Outside of well	D16 @200 mm
Point c	Inside of well	(D25 + DS16) @100 mm
	Outside of well	D16 @200 mm
Point d	Inside of well	D16 @200 mm
	Outside of well	D12 @200 mm
Point e	Inside of well	D16 @200 mm
	Outside of well	D12 @200 mm
Point f	Upside of slab	D12 @200 mm

B-B Section

1) Loading diagram

Loading diagram to be applied for B-B section is illustrated as below.



2) Result of structural analysis

Member a - b

Point a	Inside of well	(D16+D19) @100 mm
	Outside of well	D19 @200 mm
Point b	Inside of well	D19 @200 mm
	Outside of well	D19 @200 mm

Member a - a'

Point b & b'	Inside of well	D12 @200 mm
	Outside of well	D19 @200 mm

Member b - c'

Point b	Upside of slab	D19 @200 mm
	Down side of slab	D19 @200 mm
Point c	Upside of slab	(D19+D16) @100 mm
	Down side of slab	D19 @200 mm

Member c - d

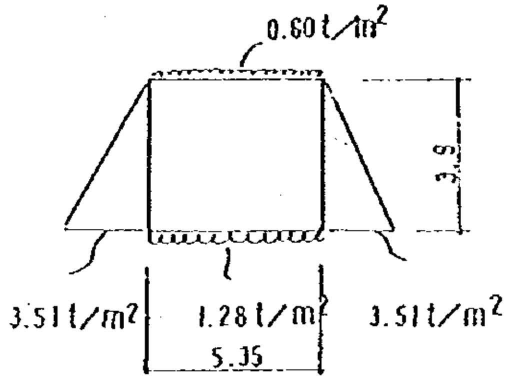
Point d	Inside of well	D16 @100 mm
	Outside of well	D16 @100 mm
Point c	Inside of well	D16 @200 mm
	Outside of well	D12 @200 mm

Member c - c' Same as section A-A.

C-C Section

1) Loading diagram

Loading diagram to be applied for C-C Section is illustrated as below.



2) Result of structural analysis

Member a - b	Upside of slab	D16 @200 mm
	Downside of slab	D16 @200 mm
Member a - d, b - c	Inside of chamber	D16 @200 mm
	Outside of chamber	D16 @200 mm
Member c - d	Upside of slab	D16 @200 mm
	Downside of slab	D16 @200 mm

1.5.3 Study on Bearing Capacity

The bearing capacity is studied at A-A section in Fig. 1-5-1 because the largest reaction at the well is present in this section.

The bearing capacity is estimated by the result of the geological survey.

The elevation of the receiving well slab is EL 57.0 m. Around this depth the N-value is N = 9 and the soil is composed of fine to medium sand. From this figure, the angle of internal friction is estimated at ϕ = 25°.

The bearing capacity of the ground for a long term load

$$qa = 14.0/m^2$$

On the contrary, the reaction of receiving well is estimated at R = 7.5 t/m^2 < qa = 14.0 t/m^2.

1.6 ARCHITECTURE

In this clause among the many buildings included in the project, the result of calculation for the control house is shown here as an example since it is the biggest and the most important building.

About other buildings, the calculations are not shown here becuase they are simple.

1.6.1 Structural Calculation of Control House

The major data are presented for structural calculation of the control house.

Above the caisson structure (22.0 m (W) x 17.0 (m) x 19.8 (m), the control house (rigid frame structure) with steel skelton roof in height of 3.75 m is constructed to house the electrical and mechanical control equipment.

In the control house, motors and other auxiliary equipment and control boards are installed. A 5-ton overhead travelling crane, span: 16 meters, is also installed.

Design Load

1) Roof

D.L.	Asbestos corrugated cement board	25 kg/m^2
	Steel base (purlin, etc.)	55
L.L.	(For Beam, Girder & Column)	20
		100 kg/m^2

2) Self Weight

MARK		B x D cm		t/m	I x 10^5 cm^4	STIFFNESS cm^3
G	1	40 x 90		0.96	24.3	3015 6030
	2	40 x 90		0.96	24.3	1507
	3	40 x 120		1.27	57.6	3573
	4	40 x 60		0.65	7.2	1920
B	1	30 x 40		0.34		
	2	40 x 90	40	0.70	9.15	2441 (AVE.)
	3	40 x 40		0.44	2.13	569
CB	1	40 x 90	40	0.70		
	2	40 x 40		0.44		

SB 1	40 x 65	0.70			
C 1	75 x 75	1.46	X26.4	X1-4	3196
			26.4	X5	3255
2	50 x 50	0.67	26.4	Y	3139
			X 5.21	X	631
			X 5.21	Y	631

3) Crane Load

Vertical $10.2^t \times 1.4 = 14.3^t$

Horizontal X-Direct $10.2^t \times 0.10 = 1.02^t$
Y-Direct $10.2^t \times 0.15 = 1.53^t$

4) Wind

X-Direct $0.05\ t/m^2 \times (5.0+3.75/2) \times 8.7m = 2.99\ t$
$2.99^t \times (0.8 + 0.4) = 3.59^t$ (per frame)

Y-Direct $0.05\ t/m^2 \times 25\ m/2 \times 8.7m = 5.44\ ^t$
$5.44\ ^t \times (0.8+0.4) = 6.53\ ^t$ (per frame)

Design of Beam

		End	Center	Stirrup
B_1	Top	3-ϕ19	2-ϕ19	ϕ9-@200
	Bottom	2-ϕ19	3-ϕ19	
CB_1	"	4-ϕ25	4-ϕ19	ϕ12-@200
	"	4-ϕ25	2-ϕ25	
CB_2	"	4-ϕ25	2-ϕ25	ϕ12-@200
	"	4-ϕ25	2-ϕ25	
B_2	"	4-ϕ25+4-o19	4-ϕ25	ϕ12-@200
	"	4-ϕ25	2-ϕ25	
B_3	All	4-ϕ25		ϕ9-@150
		2-ϕ25		
CB_3	Top	4-ϕ25	4-ϕ25	ϕ12-@100
	Bottom	2-ϕ25	2-ϕ25	

Design of Girder

		End	Center	Stirrup
G_1	Top	4-ϕ25	4-ϕ25	ϕ12-@250
	Bottom	4-ϕ25	6-ϕ25	
2	"	8-ϕ25	4-ϕ25	ϕ12-@250
	"	8-ϕ25	6-ϕ25	
G3	"	8-ϕ25	4-ϕ25	ϕ12-@250
	"	4-ϕ25	8-ϕ25	
G4	"	4-ϕ25	4-ϕ25	ϕ12-@250
	"	4-ϕ25	2-ϕ25	

1.6.2 Air Conditioner

Calculation of cooling load has been performed in accordance with Heating, Air Conditioning and Sanitary Standards of Japan (HASS). Based on the result of calculation the required air-conditioner was selected from commercialy available standard type products.

1.6.3 Design of Lighting

Design of lighting is based on Interior Lighting Standard of Rooms (JIS)

Location	Lighting, Lx by JIS
Office	500
Control House	200
Control Room	500
Electric Room	200
Sub Station	150
Reparing Shop	100
Ware house & garage	50

1.6.4 Selection of Exhaust Fan for Intake Tower

Room Air Volume:

3rd Floor		1,088 cu m
2nd Floor		1,224 "
1st Floor		1,550 "
	Total	3,862 cu m

Ventilation Cycle:

5 times per hour
3,862 cu m x 5 = 19,310 cu m/hr.

From this figure an exhaust fan of capacity shown below are employed.

320 cu m/min (Flow Rate)
30 mm Aq (Pressure)
5.5 kW

1.6.5 Selection of Submerged Sewage Pump for Control House

Sewage of the control house is pumped to the sewage tank of the administration office located near the shore through the sewage pipeline laid along the intake bridge. A total head of 10 meters is selected, taking possible friction loss into consideration.

The standard volume of sewage is 200 litter/day/person which is usually applied to a standard size office. So in the case of 2 maintenance personnel normally stationed there, the sewage is estimated as below.

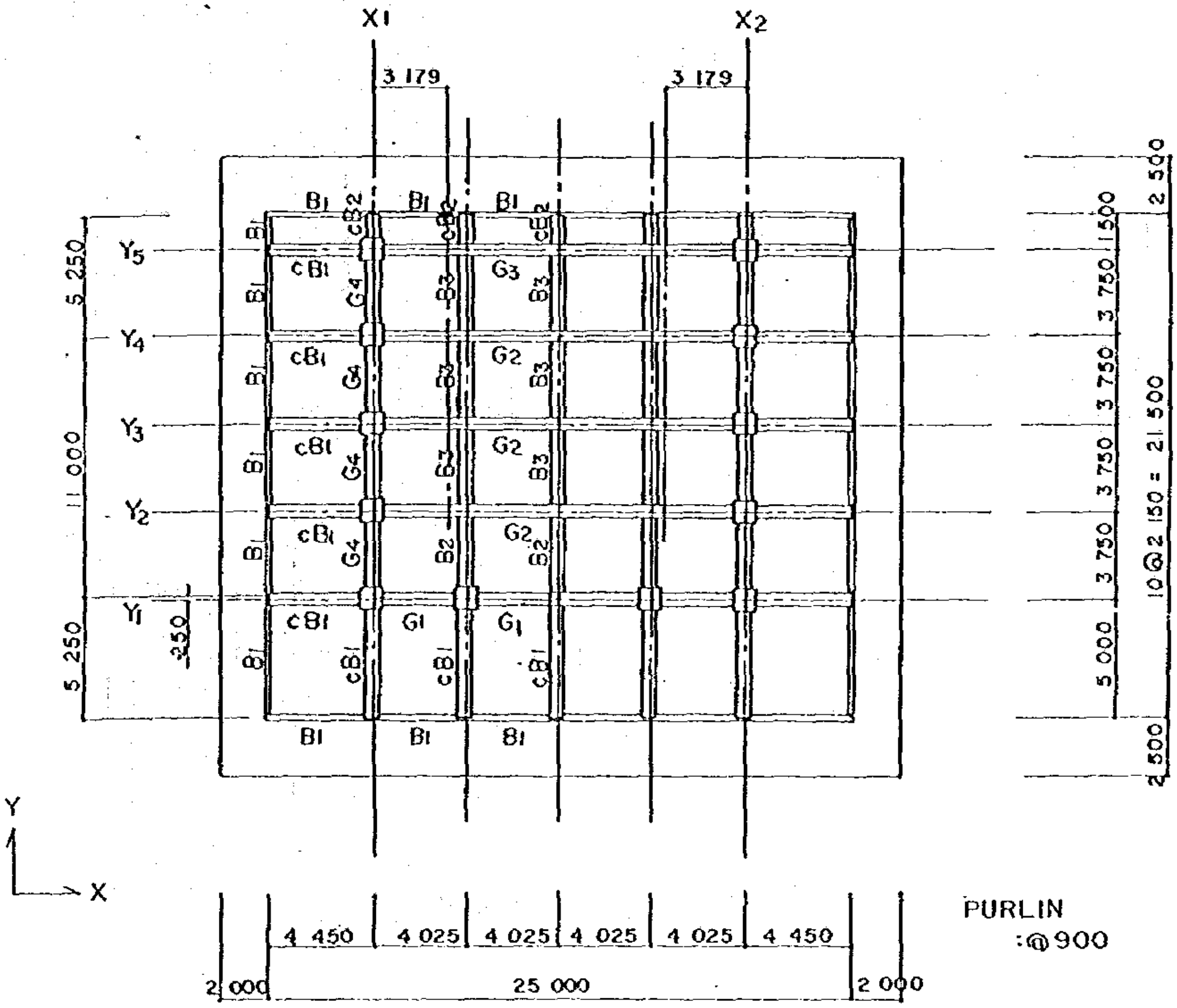
200 liter x 2 person x 1 week = 2,800 liter/week

The level which is the starting head of submerged pump decides the volume of sewage tank. The volume of sewage tank with 2.3 m long, 1.6 m wide and starting head at 60 cm, is estimated at 2,000 liter. Pump capacity is selected so as to start once in 5 days and convey sewage of 2,000 liter in one run.

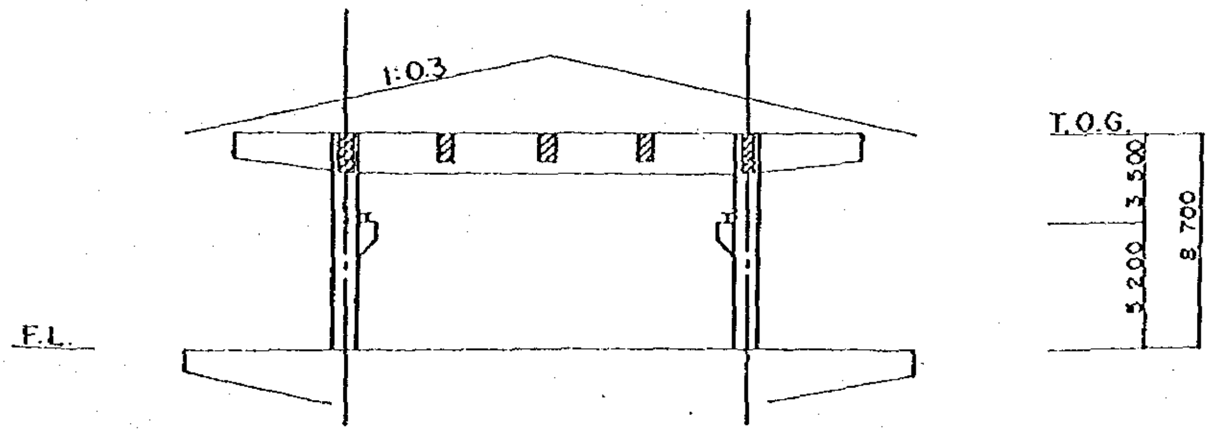
The following is the particulars of the pump adopted as suitable.

Water Head	13 m
Diameter	65 mm
Output	1.5 kW
Discharge	0.2 m^3/min.

Fig. 1-6-1 Main Members in Control House

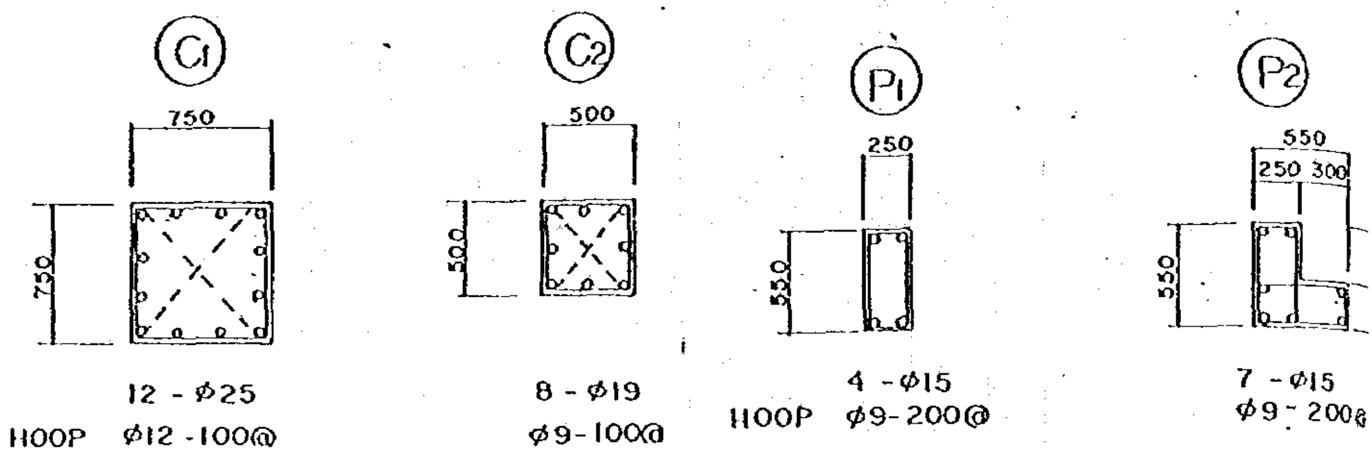


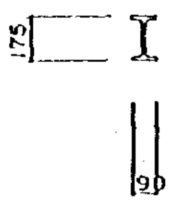
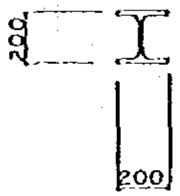
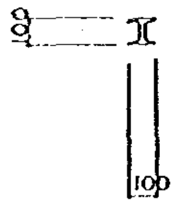
PLAN

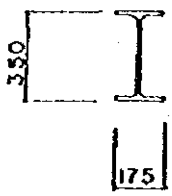
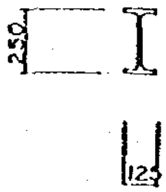
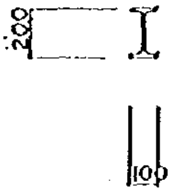


SECTION

Fig.1-6-2 Sections of Main Members in Control House



B4	P1	P2
		
H - 175x90x5x8	H - 200x200x8x12	H - 100x100x6x8
G.PL - 6, HTB2- M15		

B1	B2	B3
		
H - 350x175x7x11	H - 250x125x6x9	H - 200x100x5.5x8
G.PL - 6, HTB3-M20	G.PL - 6, HTB3-M20	G.PL - 6, HTB2-M15

1.7 MECHANICAL EQUIPMENT

1.7.1 Characteristics of Pump

The characteristics of pump are calculated by the following equations.

Head

$H = H_a + h_1 + h_2$

H : total head (m)

H_a: actual head (m)

h_1: loss head and velocity head (m) within the intake

h_2: loss head and velocity head (m) from the intake to the head tank

Capacity

$Q_T = Q \times N$

Q : capacity per unit (m^3/min)

Q_T: max. flow rate (m^3/min)

N : number of pump in operation

Motor Output

$$P_M = \frac{0.163 \times Q \times H}{\eta} \times 1.15$$

P_M: motor output (kw)

γ : specific weight of pumped liquid (kg/l)

Q : capacity (m^3/min)

H : total head (m)

η : pump efficiency (%)

Pump Suction Diameter and Pipe Diameter

$$D = \frac{4\ Q}{v}$$

D : diameter (m)

Q : flow rate (m^3/sec)

v : flow velocity

1.7.2 Type of Pump

In selecting the type of main pump, the conclusion given in Main Report Section 3.4 is based on the following table.

Table 1-7-1

Item	Vertical shaft mixed flow pump	Duble suction centrifugal pump
(1) Characteristic	- Need min. space Pump efficiency is lower than DSCP /2	- Need larger space than VSMFP /1 High pump efficiency
(2) Operation	- Possible to adopt operation	- ditto -
(3) Maintenance	- Difficult for inspection and repairing, because the impeller and shaft sit under water	- Easy for inspection and repairing because most of all equipment sit in the dry area.
(4) Installation	- More difficult than DSCP	- Easier than VSMFP
(5) Initial and running cost	- More expensive than DSCP	- Less expensive

/1 VSMFP : Vertical Shaft Mixed Flow Pump
/2 DSCP : Duble Suction Centrifugal Pump

1.7.3 Number of Pump

In selecting the number of main pump, the conclusion given in Main Report Section 3.4 is based on the following table and figures.

Item	Case A (6 sets)	Case B (4 sets)
(1) Space	- Larger space	- Smaller space
(2) Pump efficiency	- Lower	- Higher
(3) Maintenance	- Difficult for inspection, because the number of pump are larger than case B.	- Easy for inspection the number of pump are less than case A.
(4) Cost	- More expensive	- Less expensive
(5) Capacity of max. flow rate in case of 2 pump sets break	- Case A is able to discharge demanded max. flow rate.	- Case B is not able to discharge demanded max. flow rate.
(6) Capacity of max. flow rate in case of 3 pump sets breakdown /2	- Case A is not able to discharge demanded max. flow rate. - In case A, max. flow rate is 93% par demanded max. flow rate.	- In case B, max. flow rate is 50% par demanded max. flow rate.

/1, /2 ... These are shown in Fig. 1-7-1 (Case A), and Fig. 1-7-2 (Case B).

Fig 1-7-1'

Pump System Curve (5 Pumps Operating in Parallel)

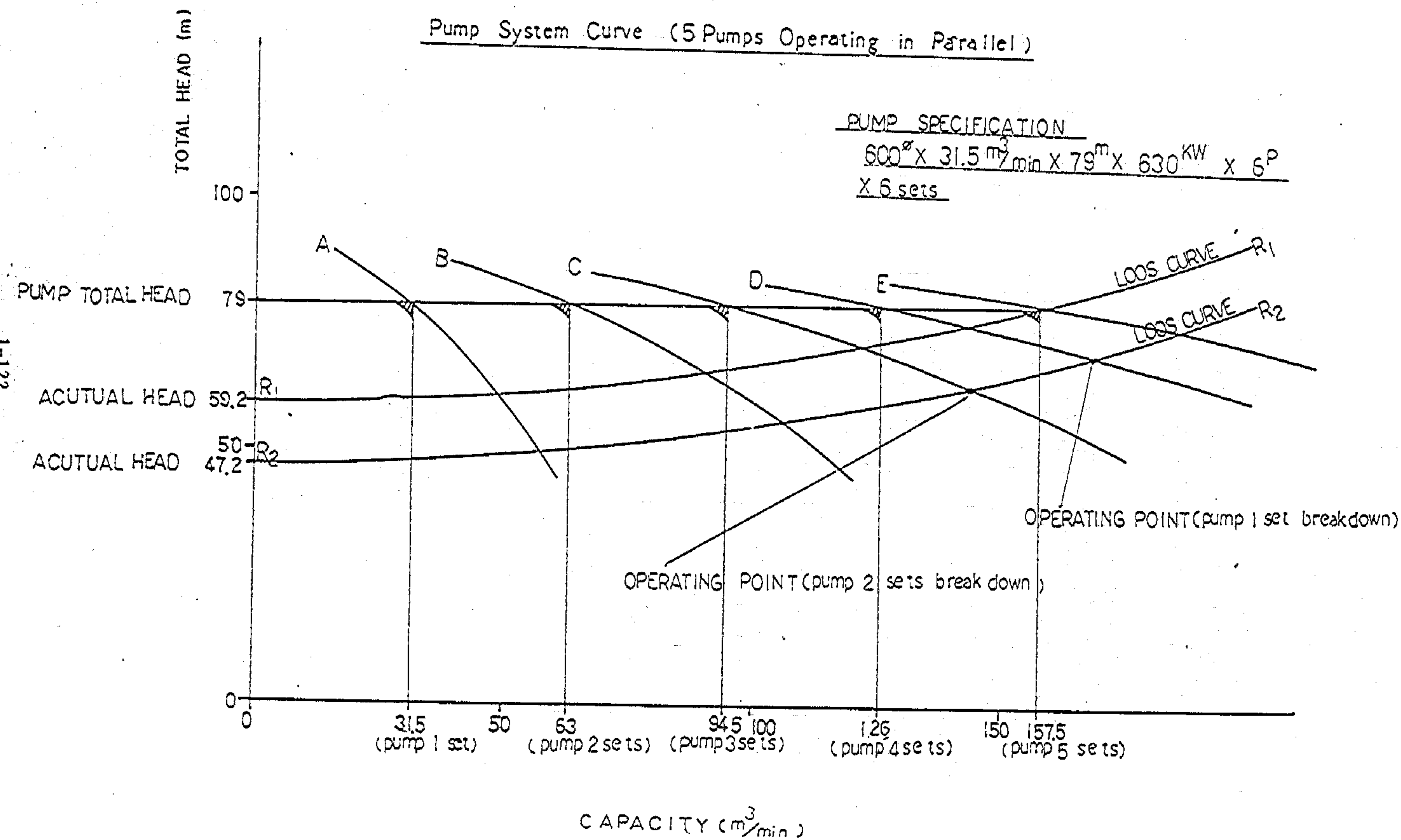
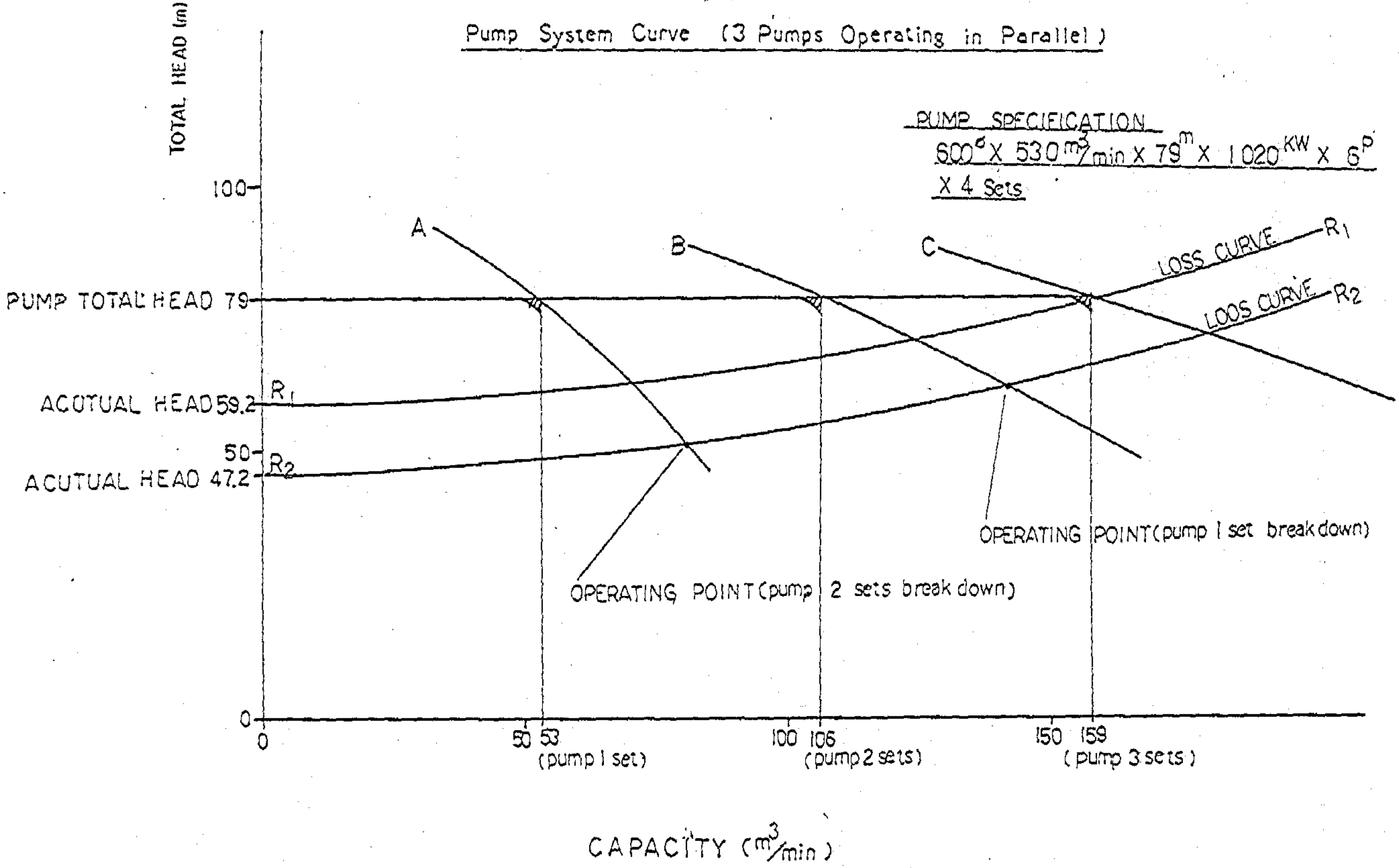


Fig 1-7-2

Pump System Curve (3 Pumps Operating in Parallel)



1.7.4 Water Hammer Prevention Equipment

The requirements of equipments for preventing water hammer are computed using the following conditions, with an electronic computer.

Pump

Type:	vertical shaft, double suction centrifugal
Capacity:	31.5 m^3/min x 79 m x 1,000 rpm x 630 kW
Number:	6 including 1 stand by
Motor:	Vertical shaft, squirrel cage type, 3,000 V x 630 kW x 1,000 rpm

Valves at Delivery Side of Pump

Type:	Swing type check valve
Diameter:	500 mm

Pipeline

Diameter:	1,350 mm
Material:	Carbon steel, SS 41 of JIS
Thickness:	11.9 mm
Max. flow velocity:	1.8 m/sec
Max. flow rate:	2.62 m^3/sec
Velocity of pressure wave propagation:	975 m/sec
Length:	7.46 km, distance of pump station to head tank

Water Level

Pump pit:	low 42.00 m, normal 52.10 m, high 54.10 m
Head tank:	low 99.55 m, high 102.10 m, extra high 102.85 m

Head Tank

Diameter:	16 m

Flow Sheet

The above mentioned conditions are shown on the flow sheet.

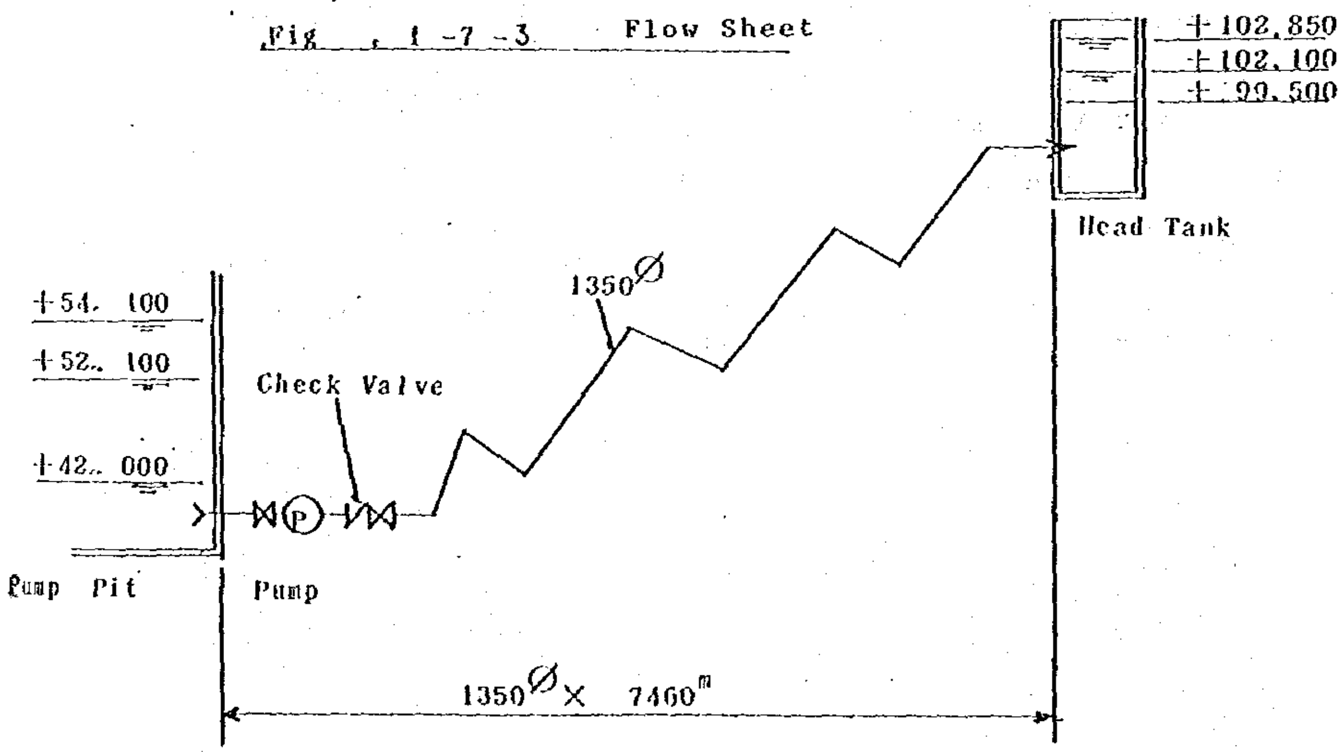
Inertia Effect of Rotating Parts

Pump:	80 $kg.m^2$
Motor:	80 "
Sum:	160 "

Resolution of Analysis

It is shown in Fig. 1-7-3.

Fig. 1-7-3 Flow Sheet



1.8 ELECTRICAL EQUIPMENT

1.8.1 Power Supply and Demand

Dok Krai

A 22 kV transmission line from PEA runs close to Dok Krai Reservoir. The power demand of 3,350 kV shall be supplied from 22 kV transmission line.

Head Tank

A low voltage line runs along Route 3191 and it can feed the head tank's consumption of 30 kW.

Receiving Well and Reservoir

A low voltage line runs along Route 3 and it can feed the receiving well and reservoir's consumption of 52 kW.

During Construction

The locations (1) to (3) needs 20-30 kW supply during construction stage.

1.8.2 Reception and Transformation at Dok Krai

Demand

The maximum power demand, being assumed that individual loads occur simaltaneously, is shown below:

Table 1-8-1 Table of Load

Item	Capacity (kW)
Main pumps 5 running, 1 stand-by	630 x 5
Delivery valves 5 no.s	0.75 x 5
Drainage pump	1.5
Exhaust fans	10
Other auxiliaries	42.05
Lightings, air conditioner	65
Total	3272.05

Voltage

The size of demand shall be fed from the 3ɸ3W 22 kV, 50 Hz transmission line.

Single System Reception

When plural supply systems coming from different substations exist and where the supply is of importance, the plural supply lines must be utilized. Though the pumps at Dok Krai are apparently important, the area is covered by only one supply system of Ao-Phai Substation. Inevitably here, the reception is on single system.

Transformation

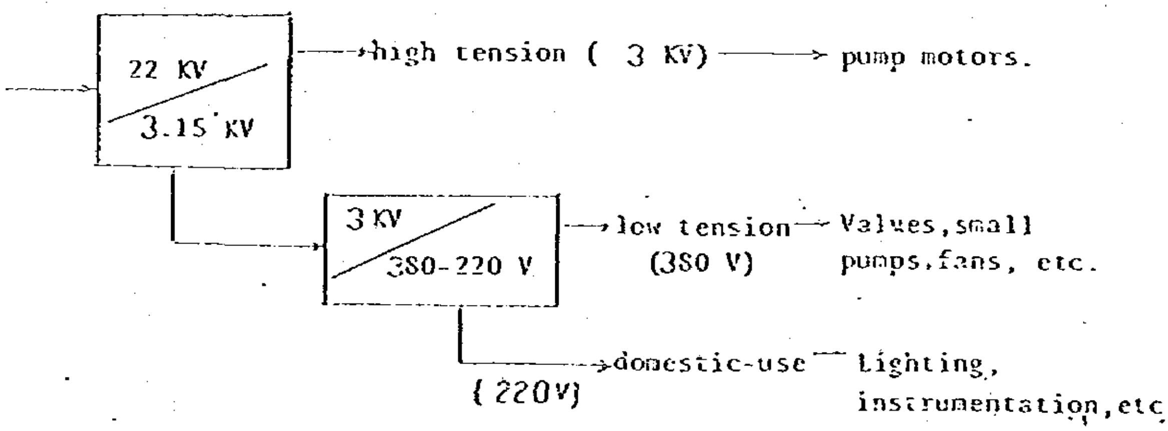
1) Main transformer

Nowadays a transformer is reliable and practically free from failure. However, to make the maintenance easier two transformers in parallel use are decided.

2) Transformation of voltage

The use at Dok Krai can be classified into the high tension for pump motors, the low tension for pump's valves, flow control valves, floor drainage pumps, exaust fans and other power loads, the domestic-use voltage for lighting sockets, instrumentation, etc. It is done as on Fig. 1-8-1.

Fig. 1-8-1 Transformation and Uses.



The voltages are 3 kV, 380 V and 220 V in accordance with Thai standards. The former two are 3 phases and the last is single phase.

3) Type of transformer station (Substation)

Three types are concidered, Outdoor and Open, Outdoor and Closed, Indoor and Closed.

The open type is the one which is constructed by using outdoor type gadgets and steel material at the site and the closed type is the one enclosed in a steel cubicle (cabinet) with indoor type gadget and manufactured at the factory. Comparing the three, the Indoor and closed type is chosen on the safety, easiness of operation and maintenance, although it is most expensive of the three as it needs housing.

4) Use of two transformers, 22 kV/3.15 kV

The requirement assumed here is that a transformer can stand the load consisting of three pump motors, low tention uses and lightening etc. When five pumps are run, two transformers shall be used, and if a transformer should fail, three pumps can be run. Each transformer's capacity is 3,000 kVA with some allowance.

5) Capacity of low voltage transformer, 3,000 V/380-220 V

With the transformer, both 380 V x 3 phase and 220 V x single phase power can be made use of. With some allowance, the capacity is 200 kVA.

1.8.3 Emergency-Use Engine-Driven Generator

Background

The situation of electricity supply in Dok Krai area is relatively good, with only 2 to 3 times' failure in a year. For the power failure, a receiving reservoir for 3 hrs' (at the design flow rate) storage capacity is already prepared.

If it is to take further precautions in operation of the pumps, the generator's capacity must be 1,000 to 2,000 kW, which is obviously unacceptable for reasons of the cost and difficulties in operation and maintenance.

An engine-driven generator should be provided for the limited uses on demand in Dok Krai and Mab Ta Pud.

Requirements and Conditions

Of the demands shown on the table of load in 1.8.2, the whole of lighting, drainage pump and exhaust fans and air conditioner and the half of other auxiliaries shall be supplied.

From the requirements, a generator of 125 kVA with an engine of 150 PS (French horse-power) shall be used in Dok Krai, and another generator of 65 kVA with an engine shall be used in Mab Ta Pud. The estimated fuel consumption is 38.4 liter per hour and a 490 liter fuel tank will meet about 12 hours' run of 125 kVA generator whose running time seems satisfactory for the purpose. The same size of fuel tank shall be supplied for 24 hour running of 65 kVA generator where consumption is 20 liter per hour.

Generated Voltage and Engine Cooling

The generator must be designed in relation with the load to be connected, and the loads are 380 V $3\phi 3W$ and 220 V 1ϕ.

Of the two alternatives in cooling the engine, the air cooling shall be better than the water cooling. The former is noisier than the latter and it can be tolerated at Dok Krai. Also the former needs no care about the quality of cooling water which is rather important with the latter.

Starting-up of Engine and Restoration of Power Supply

Two methods, using compressed air and battery, can be conceived.

Generally the air method is easier in maintenance and may be preferable where the frequency of using generator is low in a case like this.

However in Thailand, the battery starting is widely used in 100-150 kVA range, therefore the battery starting system will be preferably used for this project.

Usually the engine starts automatically when the public supply fails and after the power is restored, the operator stops the engine and switches to the public supply manually. Needless to say, the automatic operation can be also worked by the manual one.

When the public supply is restored, it shall be announced by the signal lamp on the control panel.

1.8.4 Uninterruptable Power Source (U.P.S.)

Role of U.P.S.

In case of the power failure, the diesel engine-driven generator automatically starts up. However, it normally takes 1-2 minutes till the generator reaches the full capacity and it may take more time in case of unexpected malfunctions. U.P.S. is to supply the electricity without interruption for both the normal and abnormal start-up operation of generator.

Components and Function

U.P.S. is composed of battery, battery charger, inverter, sensor and automatic switches.

The battery is automatically charged by the public supply and by the generator when it is working. The inverter is to invert direct current of the battery to alternative current for the use of wireless communication, instrumentation and control panel. The sensors and automatic switches are for sensing the power failure, generators' start of supply, power restoration and responding the conditions, switching to the necessary actions.

Loads of U.P.S.

1) Dok Krai

Alternative current

The sum of consumption by alternative current is estimated about 2,000 VA and it leads to the input of inverter as 27 A.

Direct current

The sum of direct current, including operating the high voltage circuit breaker, the inverter input and emergency lighting will amount about 117 A.

2) Mab Ta Pud

Alternative current

The sum of consumption by alternative current is estimated at about 1000 VA and it leads to the input of inverter as 14 A.

Direct current

The sum of direct current, including the inverter input and emergency lighting will amount about 25 A.

Battery and Rectifier

Comparing the characteristics of battery and application for the requirements here, the alkaline battery is better than the lead battery.

The choice is the alkaline battery with Nickel and Cadmium electrodes. The rectifier is of 130 V - 70 A.

1.8.5 Flow Meter

Three types of flow meter are widely used in the waterworks facilities. They are the differential pressure, electromagnetic and supersonic, classified by the physical they employ in measuring.

After comparing the three on various factors, the decision is on the supersonic type because of the following reasons:

1) It is not affected by the changes of water supply and turbidity.

2) As for the arrangement, it does not need a by-pass line.

3) Easily maintained

4) The cost is relatively low.

5) The accuracy is relatively high.

1.8.6 Level Meter

Measuring Points and Purpose

The depth of water at the intake tower must be referred for the operation of pumps. As the depth can be converted to the reservior's water level, it can be used as an information for wide use also.

The head tank's level shall be used for the pump operation. The receiving well's level is for reading the quantity of stored water there and it can be used for the operation of pipeline system.

Selection of Type

1) Intake tower

If the sensing part is installed outdoors, the maintenance becomes apparently difficult. It must be indoors. The diaphragm type of relatively low cost and high accuracy fits the condition.

2) Head tank

Due to the operation of pump, the water level in the head tank repeats ups and downs periodically.

Other requirements are:

- Relatively high accuracy and low cost

- Less energy consumption

As the sensing of level is electrically connected to the pump's switch, the power for the level meter shall be transmitted from Dok Krai.

The pressure type is most suitable here.

3) Receiving reservoir

Approximately measuring is satisfactory and a simple float and reversely graduated gauge seems the choice.

1.8.7 Instrumentation and Control

Flow Sheet

The flow sheet of whole pipeline system instrumentation is shown on Fig. 1-8-2 of next page. The control system and operation is discussed in other reports.

Emergency

1) Communication

In case of emergency, the communication between stations is most urgent and important. The public telephone must be installed at the three stations, Dok Krai, head tank and the receiving well. Anticipating that emergency will jam the telephone and the wireless communication between Dok Krai and the receiving well becomes a requirement.

2) U.P.S. at receiving well

U.P.S. shall be prepared at the receiving well to supply currency to the wireless transmitter-receiver and the emergency lighting. Duration of the failure of power supply may be 48 hrs. at most and U.P.S must be designed accordingly.

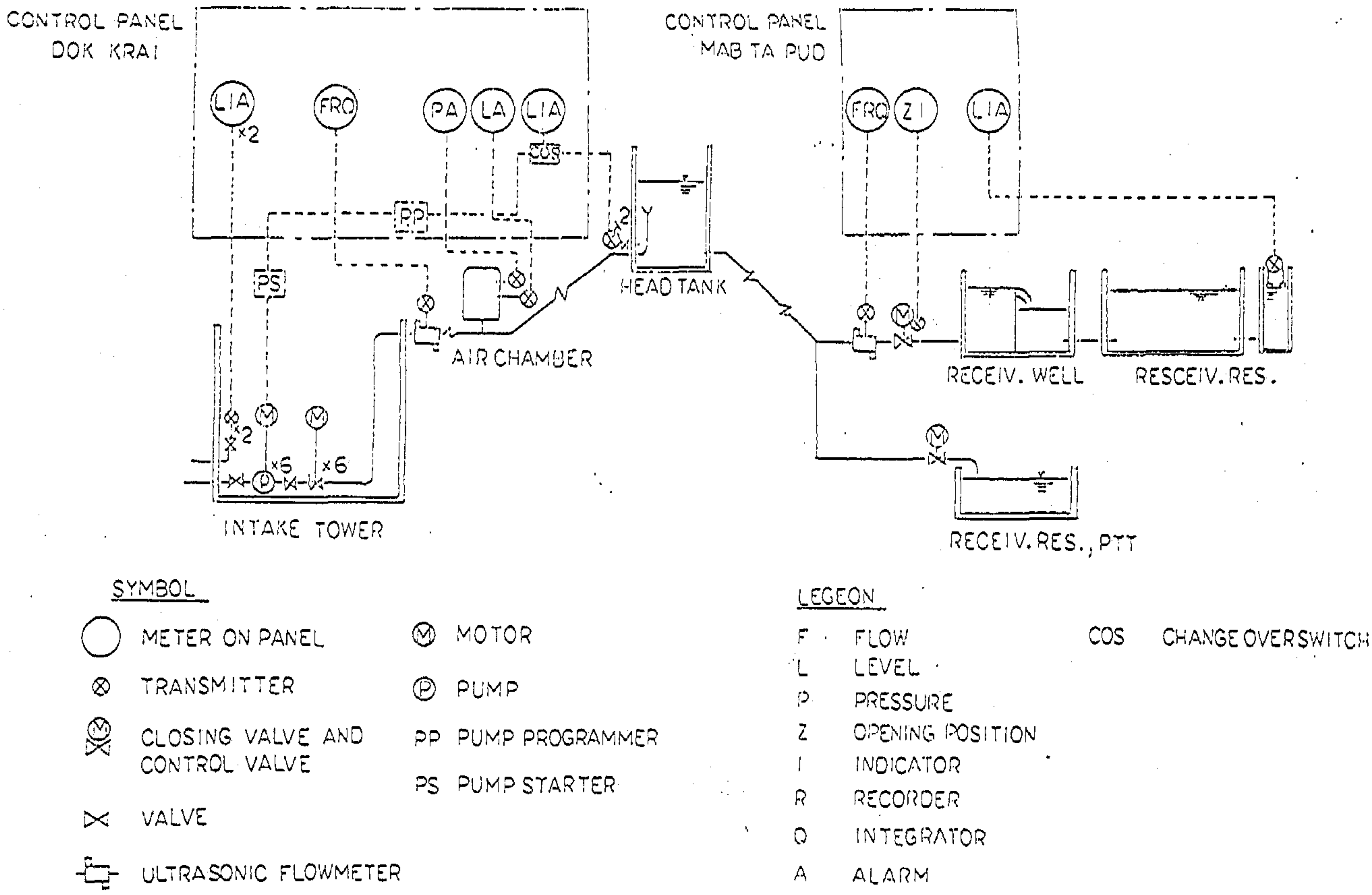
3) Fast-closing valve at receiving well

The fast closing valve, in case of the power failure, shall be the one that can be closed manually. The closing time is limited within 20 minutes due to the head tank's capacity. Consequently, U.P.S. capacity does not cover for closing the valve.

4) Walkie-talkie

As is explained in other report, the walkie-talkie shall be prepared.

Fig. 1-8-2 Flow Sheet

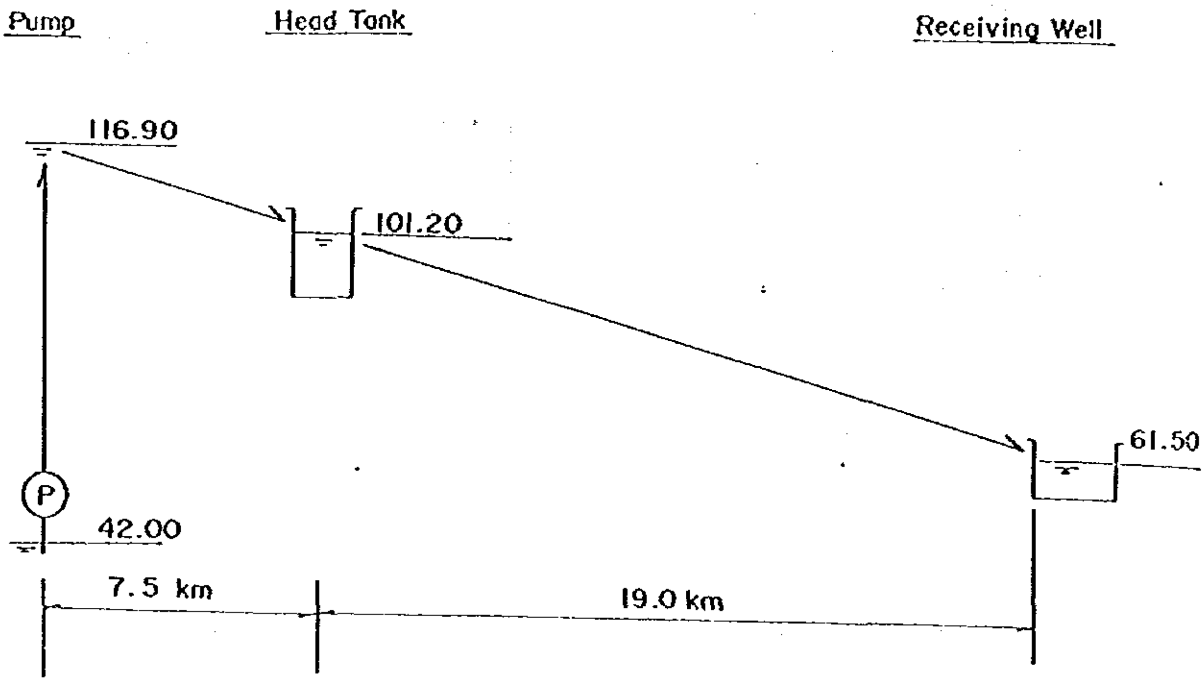


1.9 HYDRAULICS OF PIPELINE

1.9.1 Calculation by Hezene-Williams Formula

Hezene-Williams formula was used in the feasibility study and in the stage of preliminary design because of the formula's easiness in calculating the hydraulic conditions.

As the result a hydraulic conditions was pictured as shown below, under C = 120, D = 1,350 mm, Q = 2.62 cu. m/sec conditions.



As seen on the figure, the decision was:

Level of Receiving Well	EL 61.50 m
Mean Level of Head Tank	EL 101.20 m
Total Head of Pump	79.0 m including 4.1 m loss around pumps when the sump's level is EL 42.00 m the lowest.

1.9.2 Rechecking of Loss after Deciding Details

Hezene-Williams formula regards C, the roughness coefficient, as if it contains every kind of pipeline losses. When a long pipeline is under calculation, the formula can work rather accurately as the most part of the pipeline loss comes from friction loss. However, when other losses than the friction loss are relatively large, they must be calculated separately and then added to the friction loss.

The foregoing calculation of head loss between the pump and the head tank assumes 4.1 m loss "around pump" and add it to the loss calculated by Hezene-Williams formula, following the above-mentioned reasoning. In the detailed design works, the total loss shall be computed by summing up each loss, one by one, after every detail is decided and then it shall be compared with the same which is approximately assumed by calculation using Hezene-Williams formula.

1.9.3 Particular Conditions of Pipeline

Raising Water Level of Receiving Well

In the final stage of detailed design works, the level of receiving well was raised to EL 63.00 m by 1.5 m, in order to increase the supply pressure to industries in the area.

Control Valve

Till some future stage of operation when the flow is increased to a relatively high rate, a large part of loss must be killed at the control valve of the receiving well. It may possibly cause hazardous phenomenon called cavitation and to prevent it, a special type of valve must be selected. However, the type has 2.8 m loss, noticably high value, even in fully opened stage and the influence of 2.8 m over the presumed hydraulic condition must be rechecked, with 1.5 m raise of level mentioned before.

1.9.4 Various losses, Nature and Formula

Friction

The coefficient of friction is n here proposed by Manning and Ganguillet-Kutter.

The formulae are:

$$= \frac{8\ g\ n^2}{R^{1/3}} = \frac{12.7\ g\ n^2}{D^{1/3}}$$

$$h_f = \frac{L}{D} \cdot \frac{V^2}{2\ g}$$

where, $n = 0.011$ is applied for the pipe of tar-epoxy lining inside.

Inlet

$$h_e = f_e \cdot \frac{L}{D} \cdot \frac{V^2}{2\ g}$$

where, $f_e = 0.1$ is applicable for the bell-mouthed inlet.

Outlet

$$h_o = f_o \cdot \frac{V^2}{2\ g}$$

as the velocity head is completely dissipated.

Bend

$$h_b = f_{b1} \cdot f_{b2} \cdot \frac{V^2}{2g}$$

where, f_{b1} and f_{b2} are the functions of radius of bend and pipe diameter, and of angle of bend respectively.

Reducer

* Enlargement

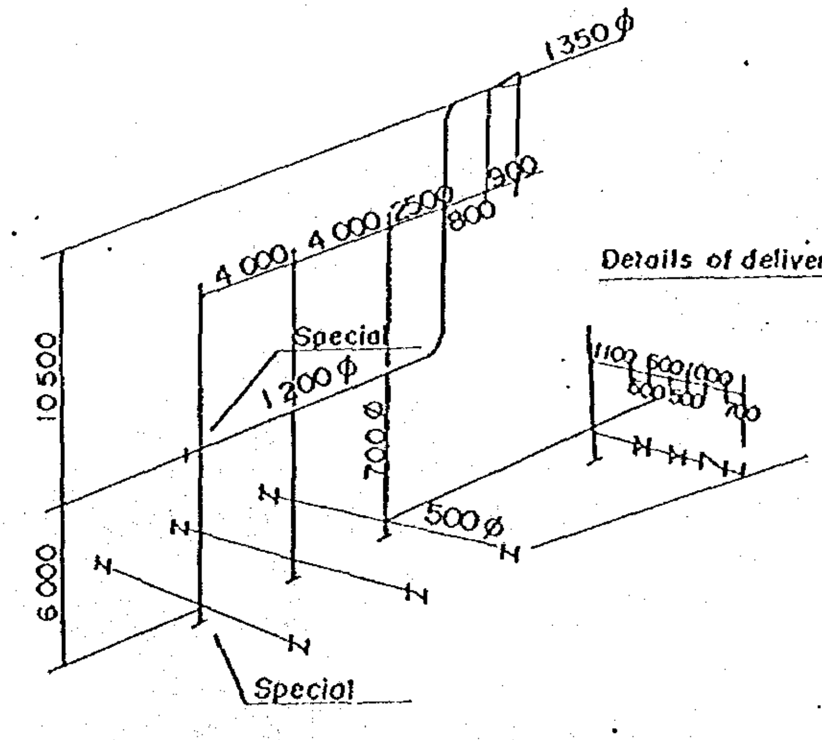
$$h_{ge} = f_{ge} \cdot f_{se} \cdot \frac{V^2}{2g}$$

where, f_{ge} and f_{se} are the functions of enlarging angle and of ratio of two pipes' sectional area respectively.

Specials

The delivery ends of pumps run horizontally and are connected with the vertical pipe and the vertical pipes are connected with the horizontal header. The connection is to be treated as "special" here.

The speciality is in that a small pipe meet a larger one in the form of vertical penetration and the flow is from the smaller to the larger. Usually it can be considered as confluence but at the points shown below it should be considered as the sudden enlargement and sharp bending, combined together.



The formula for sudden enlargement is:

$$h_{se} = f_{se} \cdot \frac{V^2}{2g}$$

f_{se} is the coefficient depending on the ratio of two pipes' sectional area and V is the velocity before the enlargement.

The formula for sharp bending is:

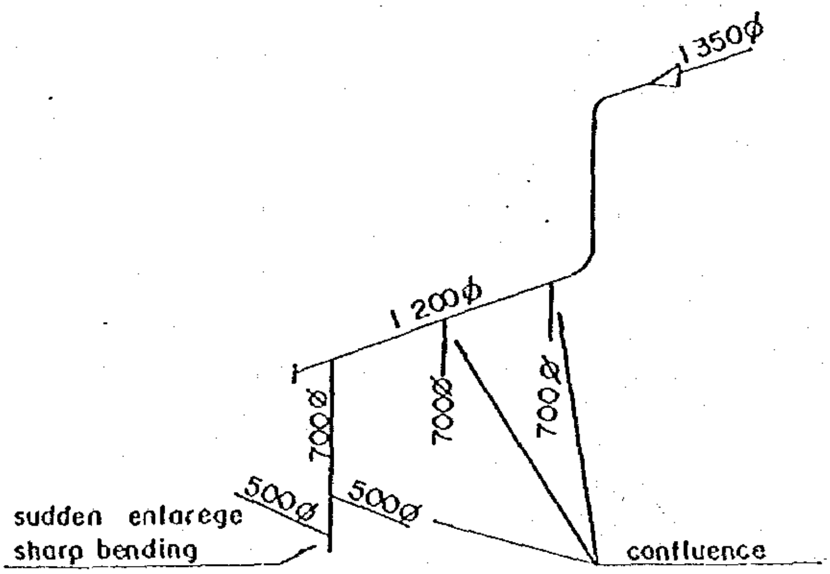
$$h_{se} = f_{se} \cdot \frac{V^2}{2g}$$

where, f_{se} is on the angle of bend.

The two losses are assumed as independent and are to summed.

Confluence

Although two 500 mm delivery pipes meet 700 mm at one point, the calculation is made as if the two inlets take place at two points. The first is treated as the special in "specials" and the second as confluence. In calculating the pipe losses, the points of confluence are the ones shown below:



The loss of confluence is expressed as follows:

$$H_b - H_a = f_b \cdot \frac{V^2}{2g}$$

$$H_c - H_a = f_c \cdot \frac{V^2}{2g}$$

$$H_c - H_b = (f_c - f_b) \cdot \frac{V^2}{2g}$$

where, f_a and f_b are the functions of the ratio of confluent/main flow rate and the ratio of two pipes' sectional area.

Check Valve

Due to the shape and mechanism the loss is:

$$h_v = f_v \cdot \frac{V^2}{2g}$$

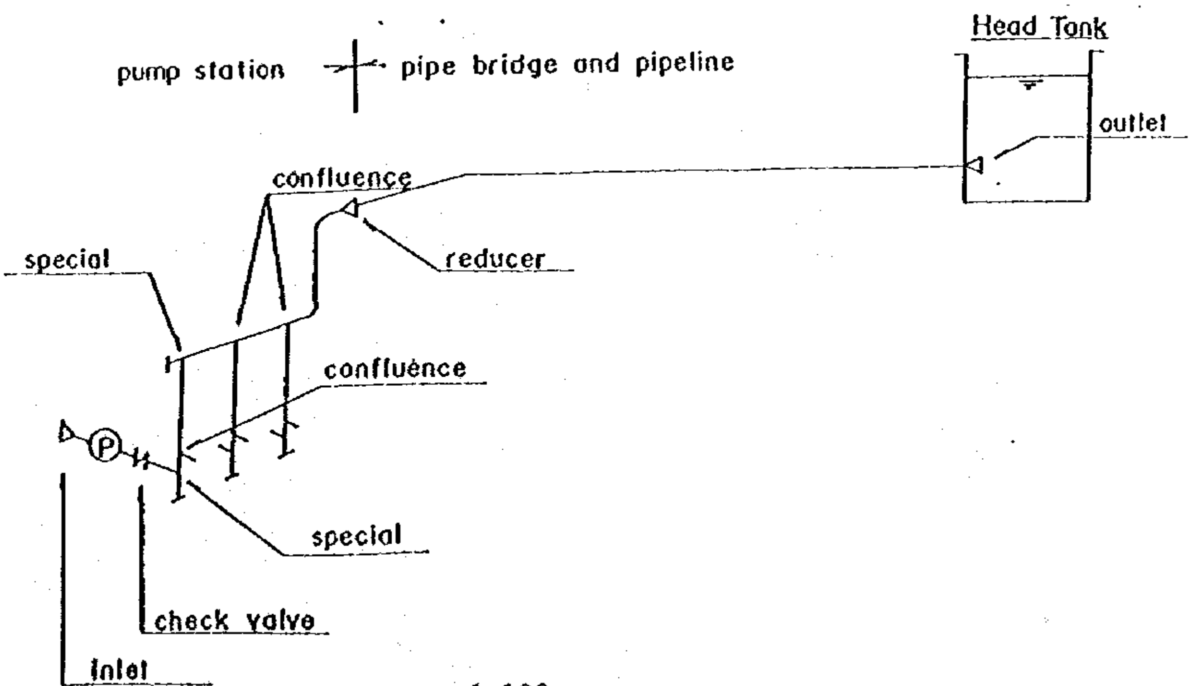
where, f_v = 1.5 is taken for the case.

Control Valve

As described before, the loss is 2.8 m when the design flow rate of 2.62 cu.m/sec runs.

1.9.5 Result of Calculation

From Pump to Head Tank



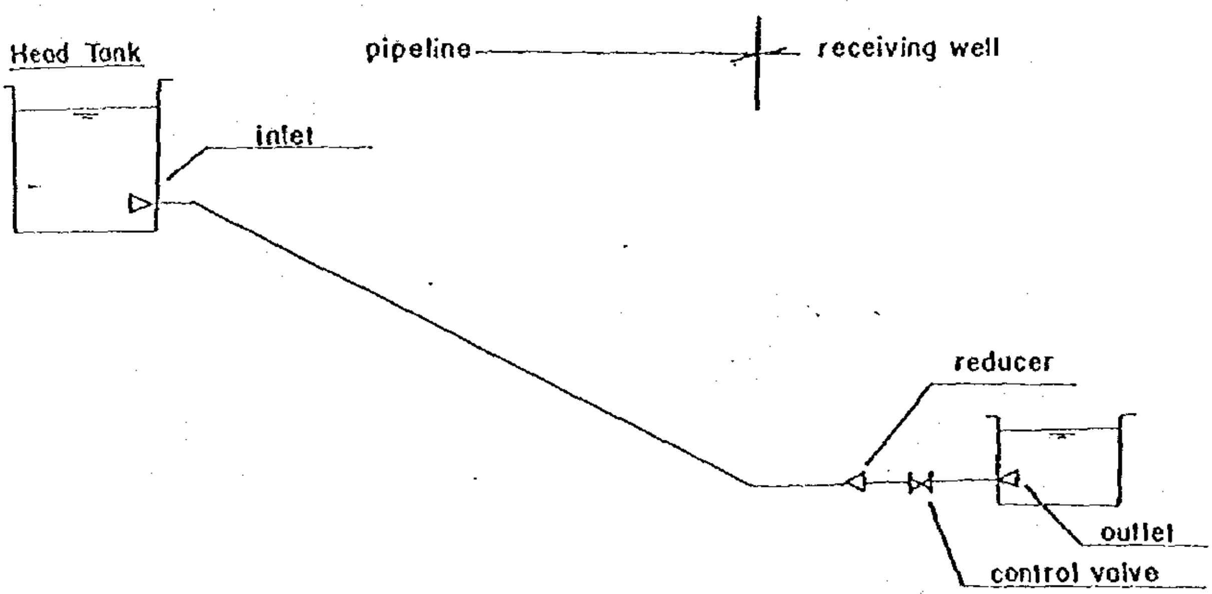
Shown above are various losses in the pipeline from the pump station to the head tank, excluding the friction and bend loss.

They are :

(1)	Friction	13.09 m
(2)	Inlet	0.04
(3)	Outlet	0.27
(4)	Bend	0.35
(5)	Reducer	0
(6)	Special	0.18
		0.14
(7)	Confluence	0.13
		0.02
		0.07
(8)	Check valve	0.55
	Total	14.84 m

From Head Tank to Reeiving Well

The same as (1) are:



(1)	Friction	32.86 m
(2)	Inlet	0.02
(3)	Outlet	0.17
(4)	Bend	1.85
(5)	Reducer	0
(6)	Special	-
(7)	Confluence	-
(8)	Check valve	-
(9)	Control valve	2.80
	Total	37.70 m

1.9.6 Existing Conditions and Conclusion

Changes

Without changing the pump's head and the level of the head tank, only the level of the receiving well was raised.

Available Head

Delivery head at Pump	42.00 + 79.00 = EL 121.00 m
Level in Head Tank	EL 101.20 m
Available Head	121.00 - 101.20 = 19.80 m between Pump and Head Tank
Level in Head Tank	EL 101.20 m
Level in Receiving Well	EL 63.00 m
Available Head	101.20 - 63.00 = 38.20 m between Head Tank and Receiving Well

Actual Head Loss and Comparison

Between the pump and the head tank, the loss is found to be 14.84 m and 4.96 m less than 19.80 m which is available.

The margin will be useful in later years when the pumps efficiency decreases.

Between the head tank and the receiving well, the loss is found to be 37.7 m and the margin is only 0.5 m.

Conclusion

Although the change of receiving well's level and the control valve of rather high loss will affect the presumed hydraulics conditions, the pipeline can be operated practically under the designed flow rate.

CHAPTER II. CONSTRUCTION PLANNING

2.1. GENERAL

2.1.1. Implementation of the Construction Works

The construction works involve many complicated structures requiring high construction technologies, and the construction period is very limited. Therefore, the internationally well-qualified contractors are required for successful execution of the Project. The Contractors must have abundant experiences in the fields of caisson work, P.C.-tank work, large-scaled steel pipeline laying and so forth.

The contract should be packaged into one tendering including the civil works, supplying of steel pipes, pumping equipment and other apparatus. Because the construction works are extremely tight in the schedule, the prime contractor should control not only the work progress but also the manufacturers and suppliers of steel pipes, pumping equipment and other apparatus. When these works are divided into several contracts, the coordination among these individual contract is very difficult under tight construction schedule.

2.1.2. Manufacturing and/or Arrangement of Construction Materials

The indigenous construction materials should be used as much as possible for encouraging the local industries and economy. The steel pipes could be supplied by local manufacturers, some of whom are equipped with modernized pipe fabrication facilities in their plants, employing competent engineers, technicians and skilled workers. Even those factories which are not sufficiently equipped with facilities such as bending roller, shot blasting equipment, etc., if may be eligible they have an intention to install such facilities so required in the specifications. But any single factory is too insufficient in its productivity to meet the requirement.

Under such situation, the plural number of the factories may be considered as pipe suppliers. No problems will exist in procuring indigenous materials of steel piles, R.C. piles and P.C. beams. The local suppliers are expected to have experience enough in production, transportation and installation for such materials.

Other construction materials of structural steels and reinforcing steel bars are available in the local market, except high tension steels and steel sheet piles or other particular materials that are to be imported.

The cement and other ordinary locally construction materials including building construction materials are available.

2.1.3. Pump Manufacturers

The vertical shaft double suction volute type pumps are to be used in the Project. The pumps of this type are not commonly used and should be supplied on the order-made basis. The pump manufacturers should have sufficient experience in manufacturing the pumps of the said type. Moreover, the suppliers are required to have abundant experiences in installation of the pumps in the similar natured physical conditions.

The shop inspection should be undertaken by the manufacturers in the presence of the Engineer. Therefore, the manufacturers should provide appropriate inspection instruments and/or testing facilities in their factories.

2.1.4. Construction Equipment

Ordinary construction equipment to be employed in the Project are bulldozers, excavators, shovels, cranes, pile drivers, trucks, concreting equipment and many others which are all available in the country. Some particular equipment for caisson works, prestressing works and pipe jacking works may be brought from abroad.

Such particular equipment as barges and pontoons which are to be used in the reservoir should be of prefabricated type, taking into consideration the inland transportation. Tugboats should be of small size transportable by trailer-trucks. The said equipment will also be provided in Thailand.

2.1.5. Laborers

Most of laborers should be locally employed, but some skilled laborers in the specific works should be expatriate workers. They may be special workers of caisson work, prestressed concrete work, pipe jacking work, and installation of steel pipes and pumping equipment.

2.1.6. Ordinary Temporary Works

All temporary works for the Project implementation should be undertaken by the Contractor but the Employer or the Engineers should reserve the right to control the works of scale, capacity, and/or quality. If these works or facilities are poor in capacity and/or quality, the Project will not be completed within the period or the performance of the works will not be acceptable. Since the construction works are tight, requiring sophisticated technologies, these temporary works should have a reserve in capacity, and the cost increase will be unavoidable to a certain extent.

The temporary works comprise those works for access roads to working sites, land preparation and final treatment of stock yards, temporary deposit places, spoil bank and borrow pits, and detours, river diversions, temporary bridges, dewatering and compensation or substitution for existing facilities.

2.1.7. Concrete Materials and Plants

The local-made standard portland cement is used for all structures except a small quantity of non-shrinking mortal for bridge. No particular type of cement is used for the caisson or P.C. tank concretes.

The concrete sand can be taken at the Rayong quarry sites. Since high quality concrete is required, the sand quality control, especially fineness modulas, is very important. The control of the fineness modulas will be made at the batching plant site by blending with various grades of sand. There is no natural gravel found for the concreting in the area, but crushed aggregate is available. Crushing plants are located at Sattahip and Rayong and these aggregates classified into three sizes of gravel, 3/4", 1/2" and 2 1/2".

The central batching plant should be provided by the Contractor, covering the whole concrete works in the Project. The simple type dry batching plant is suitable to minimize the cost for meeting considerable fluctuation of daily concreting output from 50 cu.m to 400 cu.m. The concrete mixing can be done by transit mixer during the distribution of the raw concrete.

The batching plant should have a capacity of 20 cu.m/hour and the transit mixers should provide at least three units with 5.0 cu.m capacity and three units with 3.0 cu.m capacity.

The location of the plant should be decided on the basis of concrete delivery time. It should take less than a half hour to any concreting sites by transit mixers.

2.1.8. Electrical Facilities for Construction Works

The existing power lines near the construction sites are 22-KV lines running along the Highways, Route-3, -36, -3191 and RID road to the Dok Krai Dam.

The Contractor should negotiate with Provincial Electricity Authority to receive the power in taking into account the location and capacity of three expected delivery points at Dok Krai Dam site, Head Tank site and Receiving Facilities site. The power capacity of these will vary from one temporary facility to another therein but the capacity required is estimated roughly at 100 KW, 50 KW and 150 KW respectively, including employer's camping facilities.

2.2. INTAKE FACILITIES

2.2.1. Construction Method of Intake Tower

The intake tower is to be constructed at about 15-meter deep place in water in the reservoir, and an open pump room will be located about 10 meters below the normal water surface. The construction should be conducted by the particular method in which the concrete caisson will be used as the most suitable type of the structure resulting from careful studies and considerations.

The highest priority in determining the construction method was given to the construction time factor and the second to its economy. The caisson method was formed to satisfy above conditions.

In general, the caisson method can be classified into following three methods;

a. Floating-dock method
b. Slip way method
c. Dry-dock method

The floating-dock method is the most reliable as the construction method, but requires a huge amount of cost. This method is applicable only when the dock is used repeatedly to produce many caissons. In this case, the amortization rate of the floating-dock can be reduced. In the case of producing one caisson only, this method is not applicable from the economical viewpoint.

The slip-way method is a name given to the caisson transportation method with a inclined slip-way. The caisson is made in the caisson yard on the land, and the slip-way is extended from the caisson yard to the water way in order to launch the caisson into the water.

The caisson mounted on a cradle or a trolly can move down on the slip-way toward the water by gravity when supporting jacks are removed at the caisson yard.

The original ground surface and the bottom of the reservoir along the proposed slip-way should have a very gentle slope of 1:15 to 1:20. Therefore, the slip-way must be provided for about 100 meters and the most parts are to be under the water. Such slip-way construction in reservoir is very difficult and costly. Moreover, it is dangerous to control the slipping down of the caisson having about 1,800 tons in the weight.

The dry-dock method is the most appropriate caisson construction method for this project and for ease of replacement the dry-dock is to be provided at the reservoir shore. The lowest part of the caisson concrete (4.8 meters high in the first stage concrete) will be made at the dry-dock and is towed into the reservoir by the opening of the dock. Then, the succeeding upper parts of caisson concrete are placed at the final settlement site under floating status.

The dry-dock method is more economical than the floating-dock and more reliable than the slip-way.

2.2.2. Dry-Dock (Caisson Yard)

The dry-dock should be provided just on the shore line at the reservoir for the convenience of the caisson launching. The proposed size of the dry-dock is about 30-meter long, 24 meter wide and 6.6-meter deep, and the dock is surrounded with steel sheet piles so as to cut off the seepage water. The dock bottom is covered with leveling concrete at EL. 44.60 meters, and dewatering system, ditches and pump sumps should be provided. Since the dry-dock should be provided by the Contractor as the temporary work, the Contractor should determine the detail structure. The attached Fig. 2-2-1, however, gives a general idea of the proposed dry-dock.

2.2.3. Transportation System for Construction Materials in Reservoir

Since the major part of the intake tower construction works will be executed in the reservoir, construction materials of steel piles, steel bars, concrete, formworks and other materials should be transported to the reservoir. Provision of efficient transportation system is very important so as to undertake the construction works based on the tight schedule. Especially the raw concrete should be supplied in the volume ranging from about 150 to 200 cubic meters per day without interruption, because the caisson concrete should be of water-proof and be not allowed to make a cold joint due to intermittent concrete placing.

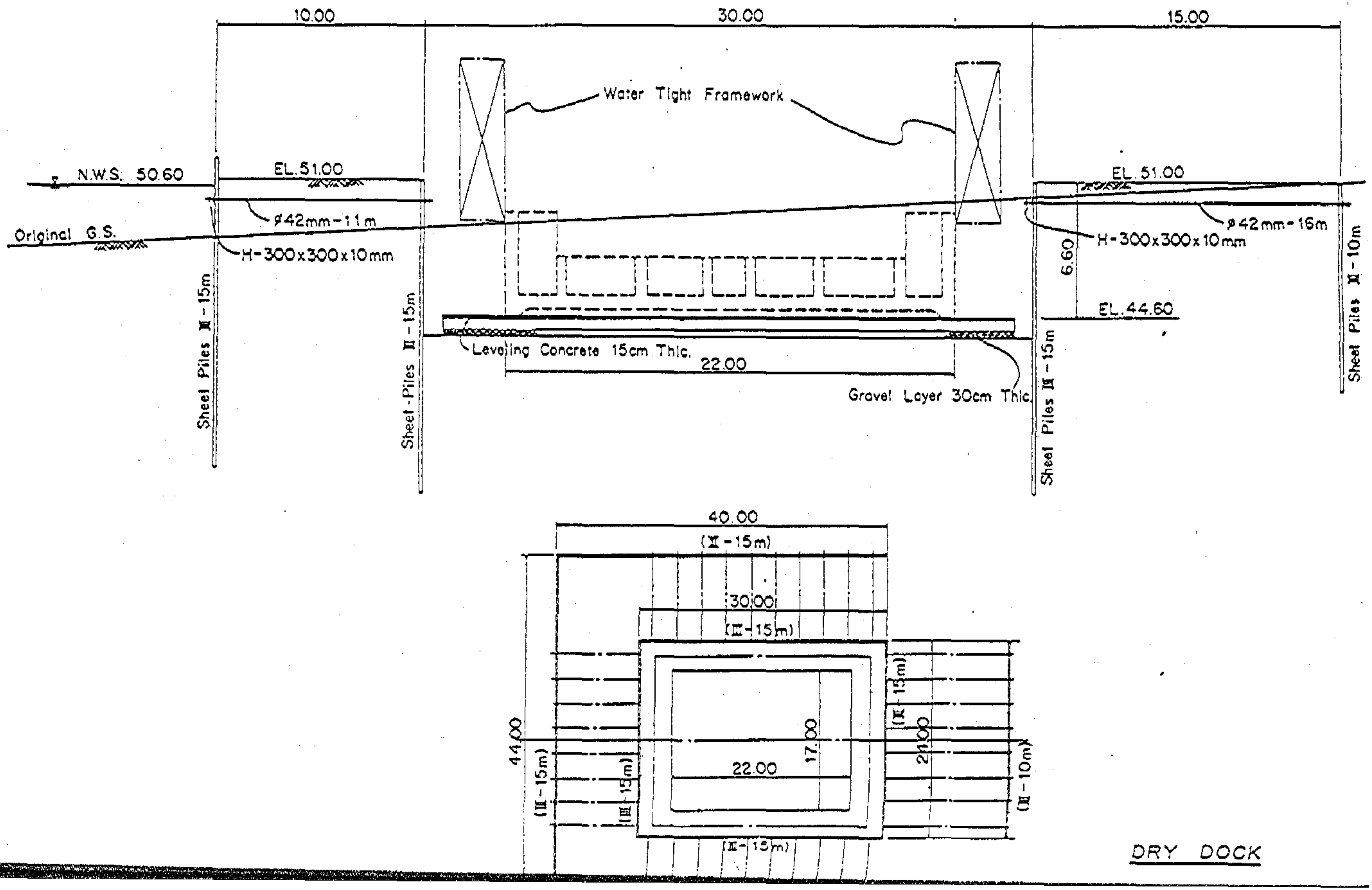
For the transportation method of temporary bridge, barge and pontoon, and concrete pump and pipeline are considered available. The temporary bridge method is deemed most reliable to meet the concrete transportation requirements, but it will be costly. About 130-meter long and 6-meter wide bridge shall be installed from the shore to the intake tower site. The bridge may be constructed with H-Steel Piers, beams and logging, taking into consideration the re-use of materials to other projects. The loading capacity may be 20 tons. The layout of the bridge is shown in the attached Fig. 2-2-2.

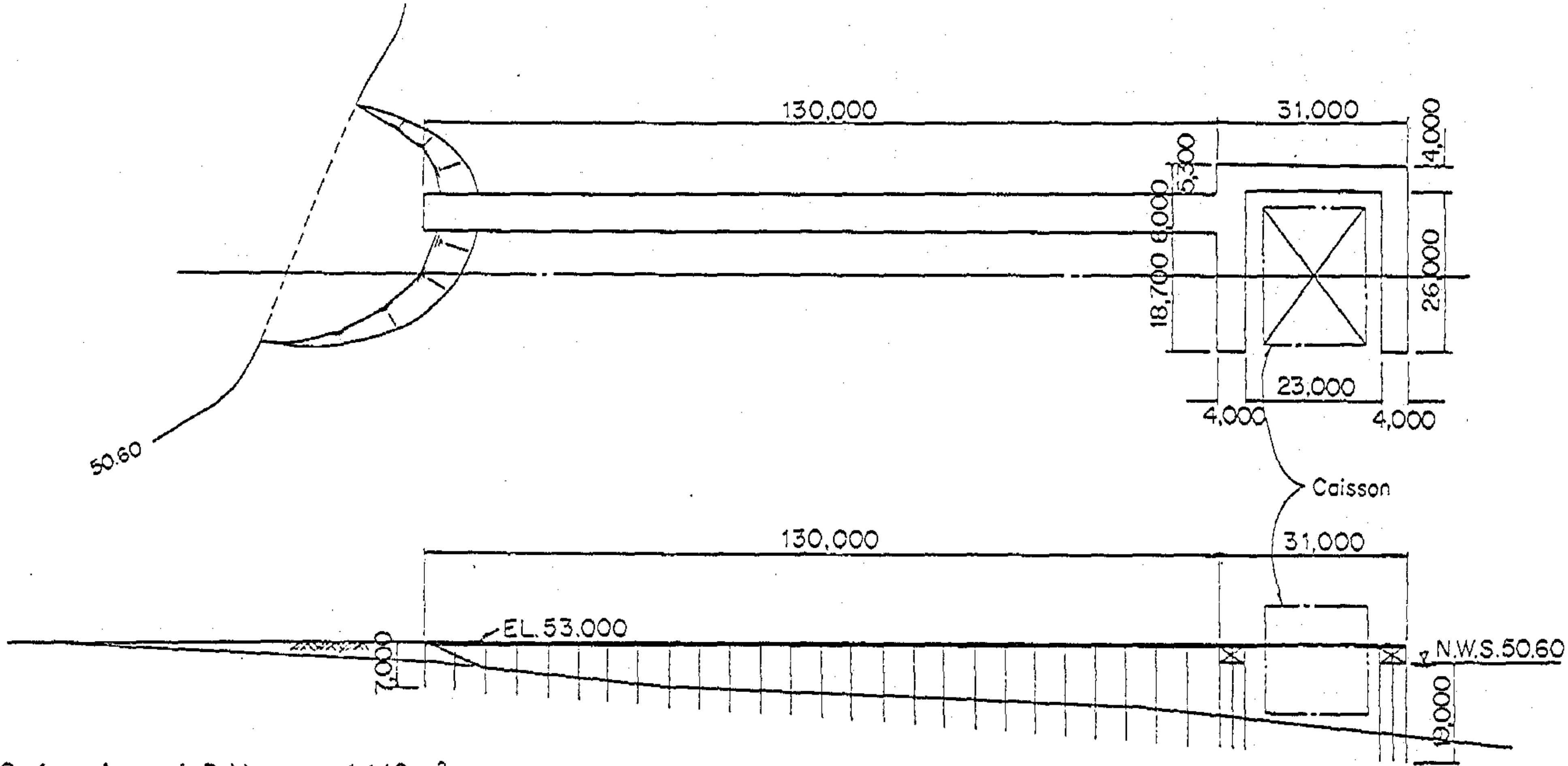
The temporary bridge is utilized not only for material transportation but for many other purposes as follows;

a. Towing and anchoring of the caisson
b. Caisson formwork and other caisson works
c. Erection of the intake bridge, piling and P.C beam installation
d. Site control and supervision

The barge and pontoon method is a common way of transportation but the heavy equipment cannot be transported by this due to their overloading to the facilities. Specially

Fig. 2-2-1 General Idea of Dry Dock (Caisson Yard)





Surface Area of Bridge : 1,112 m^2

Total Steel Weight : 1,112 m^2 x 0.48 t/m^2 = 534 t

FIG. 2-2-2. PLAN & PROFILE OF TEMPORARY BRIDGE fabricated type of equipment should be provided. This method of barge and pontoon is cheaper in construction cost than that of the temporary bridge, but the transportation of raw concrete by barge and pontoon should request many complicated procedures such as mixing and transportation to the shore, unloading and loading to pontoon, unloading at casting place, remixing of concrete, etc., and would not meet the requirements.

The concrete transportation method by concrete pump and pipeline mounted on floaters or rafts is most simple. But the concrete pump and 200 meters floating pipeline will not be able to maintain the continuous operation without trouble. The concrete placing should be continued without interruption. The concreting work usually takes five to ten hours and even 15 hours sometimes.

2.2.4. Cassion Concreting

The caisson concreting is planned in six stages and 17 times of placing as illustrated in the attached Fig. 2.2-3. The first stage concreting by three times placing will be made at the dry-dock. After placing the first stage concrete, watertight formworks should be rigged and the caisson should be launched to be towed to the intake site. From second stage to fourth stage, concreting will be carried out at the intake tower site in a floating status.

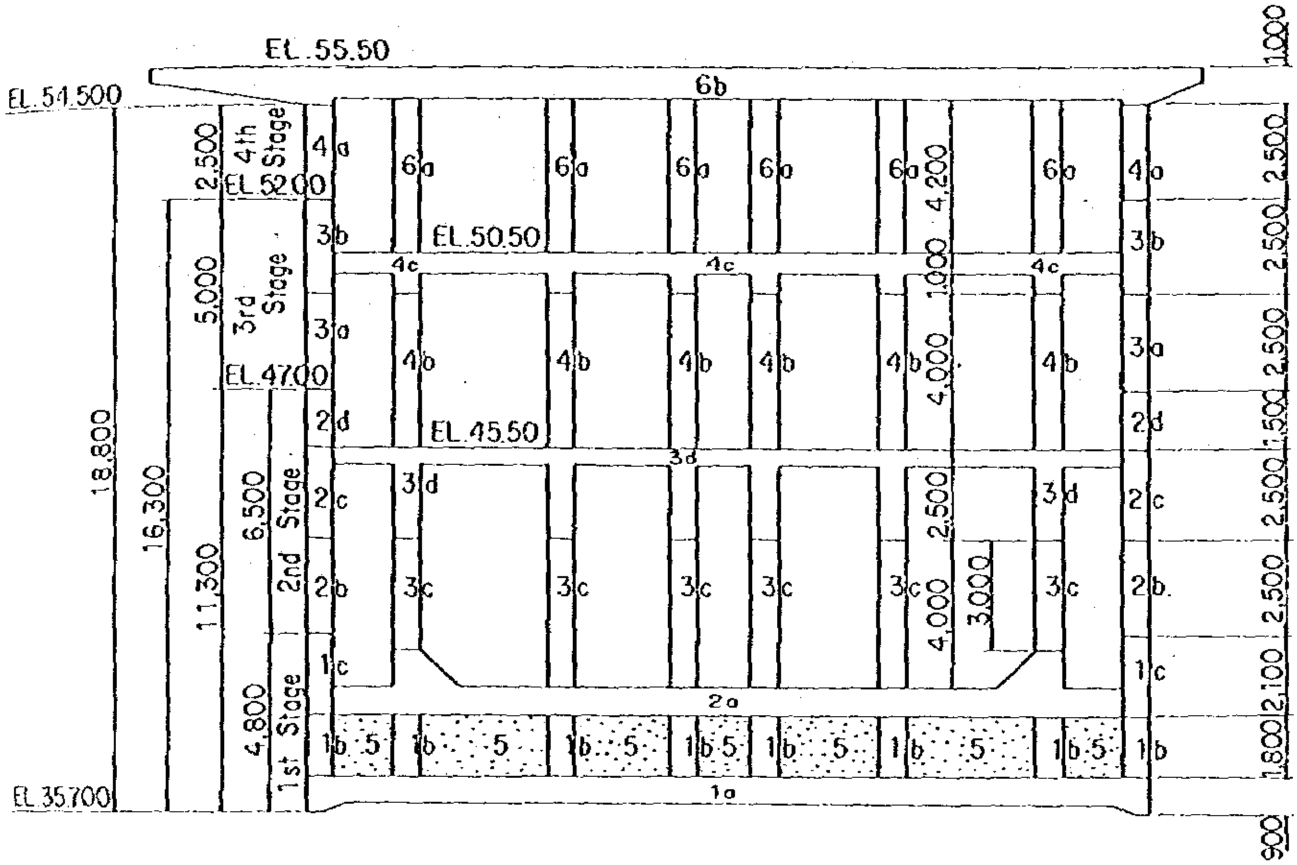
The watertight formworks should be used outside the ordinary caisson formworks in order to increase the caisson buoyancy and keep off the external water pressure against the green caisson concrete, and they should be replaced with the third stage concreting position when the second stage concreting finished. The watertight formworks should be reinforced with internal shorings, struts and bracings against lateral water pressure. These reinforced members my be steel frames, and they will be embeded into the caisson concrete as the temporary works.

The fifth stage concreting is a filling concrete into the bottom chamber of the caisson in order to settle the caisson on the foundation. After settlement of the caisson and placing the remained foundation concrete surrounding the caisson bottom, the grouting work should be made for filling the void below the bottom slab of the caisson. After that, the sixth stage concreting will be placed.

2.2.5. Foundation Excavation

The foundation excavation should be carried out in the reservoir with 10 to 15 meters of water depth. The soil to be excavated consist of decomposed or weathered granite and the excavation volume was estimated at 3,500 cubic meters. The excavation equipment planned to be used is one 0.6 cu.m. class clamshell with special crab for hard soil excavation and one 200-ton capacity of pontoon with spuds and anchors. It may be necessary to use particular chisels for weathered rock excavation.

FIG. 2-2-3. Caisson Concreting Process



Concreting Sequence

1st Stage Concreting	1a, 1b, 1c	....	3 times at caisson yard
2nd Stage Concreting	2a, 2b, 2c, 2d	....	4 times with watertight formwork
3rd Stage Concreting	3a, 3b, 3c, 3d	....	4 times -do-
4th Stage Concreting	4a, 4b, 4c	....	3 times -do-
5th Stage Concreting	5	....	filling concrete for settlement
6th Stage Concreting	6a, 6b	....	2 times
Total			17 times concreting

The excavated materials should be hauled to the spoil bank near the shore by barges.

When the foundation excavation finished the equipment should be transfered to the towing route excavation. All excavations in the reservoir should be finished before caisson launching.

2.2.6. Foundation Concrete and Sole Plates

The foundation concreting for the caisson is to be made underwater. A concrete pump or tremy pipes should be used for the concreting. This foundation concreting should divided into three layers in concreting. After placing first layer concrete, four sole plates should be installed. Then second layer concrete should be placed up to top of the plate hight.

For the installation of the sole plates, some particular measuring devices or instruments should be provided to control the exact position and elevation of the plates.

The final foundation concrete should be placed after settlement of the caisson, and this concrete should be poured not only the surrounded portion of the caisson bottom but also into the beneath of the caisson bottom as much as possible.

2.2.7. Intake Bridge

The construction works of the intake bridge can be carried out in parallel with the caisson work and should be completed at the same time.

The temporary bridge is useful for the intake bridge construction in piling pier works, P.C. beam erection works and other concrete works, so that most of the bridge construction works in the reservoir can be done by using ordinary construction equipment through the temporary bridge. Otherwise, pontoons, barges and tug boats are needed and are the cause to reduce their output.

2.2.8. Other Works

Construction works of other structures at the intake site such as the control house, the air chamber, pipeline installation and pumping equipment installation will be carried out after finishing the caisson work and the intake bridge erection. All of these works should be completed by the end of June, 1984.

2.3. PIPELINE

2.3.1. Transportation of Pipes

All pipes should be transported from the fabrication workshops to the installation sites by trailer trucks after passing the shop inspection. It will be possible to transport the pipes two times a day in doing some overtime works and transporting two or three pieces of pipes by a trip if suitable cradles to be mounted on the trailer truck. Thereby, three to five units of trailer trucks will be necessary for transportation of the total number of the pipe.

2.3.2. Temporary Works

Following temporary works should be provided prior to the pipe installation works.

a. Preparation of temporary use land
b. Relocation or removal of existing facilities such as houses, cottages, fences, electric poles, pipes etc.
c. By-passes, detours or temporary bridges at road crossing points
d. Jacking facilities at highway crossing points Route-3 and Route-36
e. Coffer dams, river diversions or dewatering facilities at river crossing points
f. Sheet piling or some particular facilities at swampy areas, if necessary

2.3.3. Earth Works

The trench excavation may be done by a hydraulic excavator and a bulldozer, and the excavated material should be deposited beside the said trench for backfilling.

The backfill may be done by a bulldozer and man power. The compaction of filling materials shall be executed by a manually operating rammer or a vibrating compactor for lower portion and a tamping roller or a tire roller for upper portion.

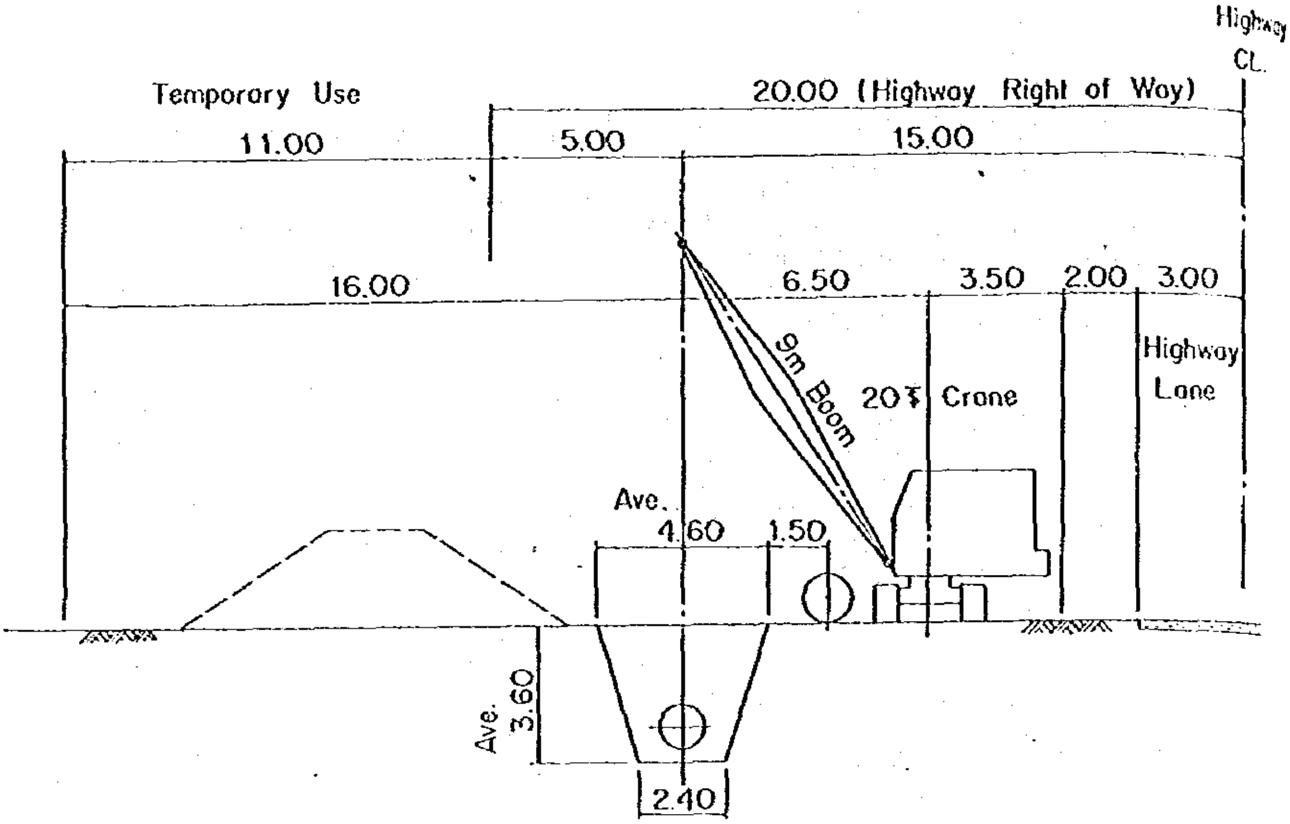
If the excavated materials are treated in accordance with the specification requirements, they can utilize as the sand bed or selected back filling materials.

If the excavated materials are left after used for backfill, they may be spread and arranged within the right of way area.

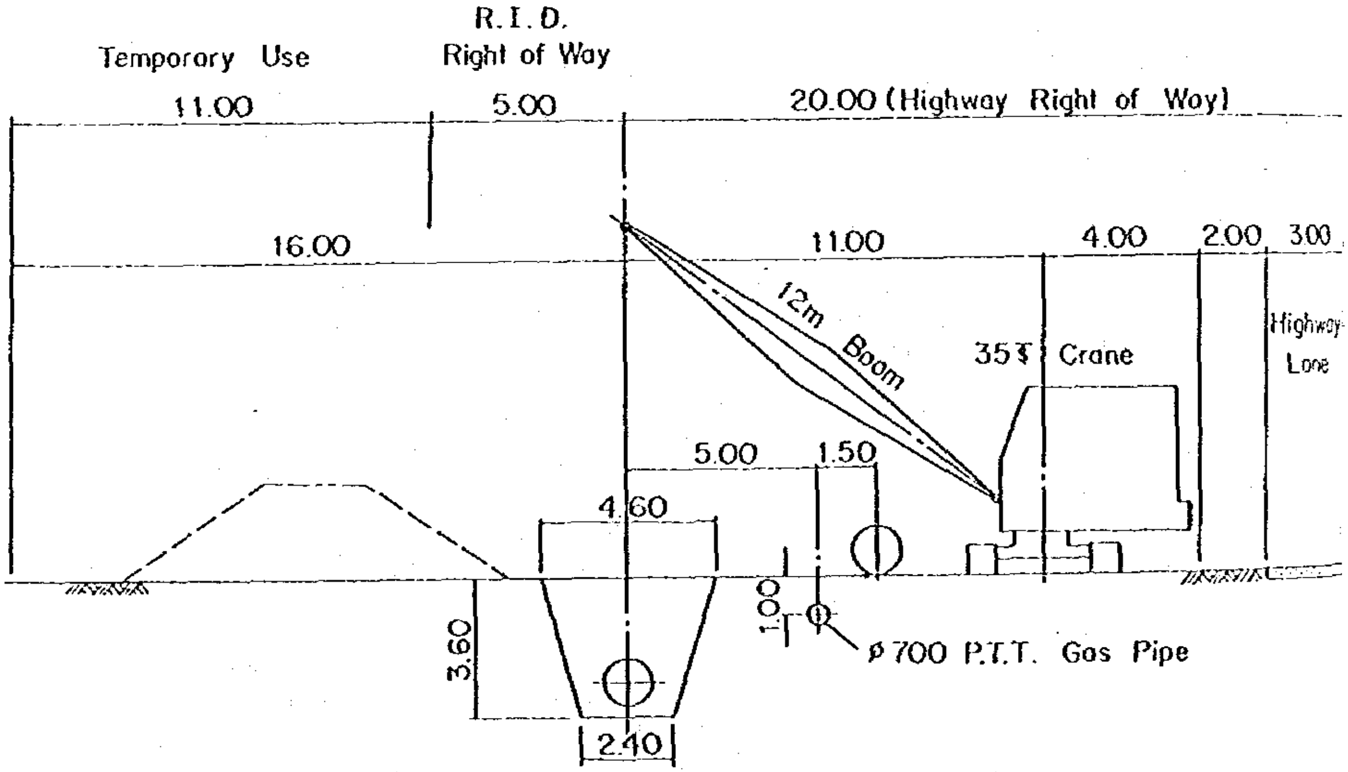
The hydraulic backfill may be applied when the dry-work method is difficult.

2.3.4. Pipe Laying

Handling and laying of pipes may be carried out by a crawler crane or truck crane. For the pipeline from KM 15 $\pm$ to 21 $\pm$, along P.T.T. Gas pipeline a 35-ton class crane should be used for executing the long reach works, but for other places a 20-ton class crane may



CASE - I (Standard Case)



CASE - II (Along Gas Pipeline)

FIG. 2-3-1.

STANDARD CROSS SECTION OF PIPELINE INSTALLATION be used. An attached illustration (Fig. 2-3-1) shows a general idea of pipe laying works.

The welding should be done inside the pipes and a blower should be used for air ventilation of welding and coating of pipeline works. An engine-operated welder or a portable diesel generator may be used instead of ordinary electric power.

The pipe laying will be executed in the wet season at hilly lands, while in the dry season at swampy areas or river crossing places. Then, the working site should be moved on from one place to the other according to the working conditions.

2.3.5. Jacking

At the crossing points with Route-36 and 3, the jacking method should be applied to pipe installation works so as to prevent the highways from being damaged.

Special double tube pipes for jacking and segments for welding joints cover of the pipe should be fabricated at the factory. Furthermore a heading shoe and a pushing ring should be provided to be utilized at both jacking sites repeatedly.

The driving pit should be provided at first. The pit may have surrounding sheet-pile wall and struts, and a back concrete block.

The jacking work will start from the driving pit by using hydraulic jacks. A mobile crane (5 tons) or a gantry crane may be provided for the handling of the pipe and the removal of excavated materials. The first jacking will be carried out at the Route-3 site and then all the equipment and facilities will be transfered to the Route-36.

2.3.6. Field Welding Inspection

The field welding inspection should be made by X-Ray. The instrument of X-Ray should be provided one set for each two pipe installation sites at least, and a X-Ray specialist should control the inspection work. The backfill work can be done after passing the X-Ray inspection.

2.4. HEAD TANK

2.4.1. Excavation

The major excavation works may be done by a bulldozer, and pit and trench excavation by a backhoe shovel. The excavated materials may be spoiled at adjacent area by bulldozer.

2.4.2. Concreting

The concrete should be supplied from the central batching plant by transit mixers, and the concrete should be placed by concrete pump or crane and buckets. The concrete lift also can be used. In any case, supplying and placing concrete should be carried out without interruption because high quality and water proof concrete is required.

The formworks for the head tank may employ ordinary metal or plywood panel forms. Scaffold should be installed inside and outside of the tank wall for concreting, prestressing and water proof coating.

Wall concreting is planned to lifting up by 1.8 meters high each. The numbers of concreting are planned by 18 times as shown in the attached Fig. 2.4-1.

Particular attention should be paid to removing the laitance on the construction joints and pouring the cement mortar just before the successive concreting to make sure of a water-proof joint on the thin concrete wall.

2.4.3. Prestressing

Each vertical prestressing rod should be connected with couplings of the piece rods. Then, vertical prestressing rods should be fabricated by three stages depending on the concreting process. Prestressing of vertical rods should be conducted at the top of the wall after finishing wall concreting.

Tensioning of horizontal prestressing strands should be made from the top of the wall to the bottom each circumference set of strands by each.

2.4.4. Other Works

The water-proof coating on inside surface of wall should be done after finishing of the prestressing. The installation of the ladder and the morning glory pipe will be made finally.

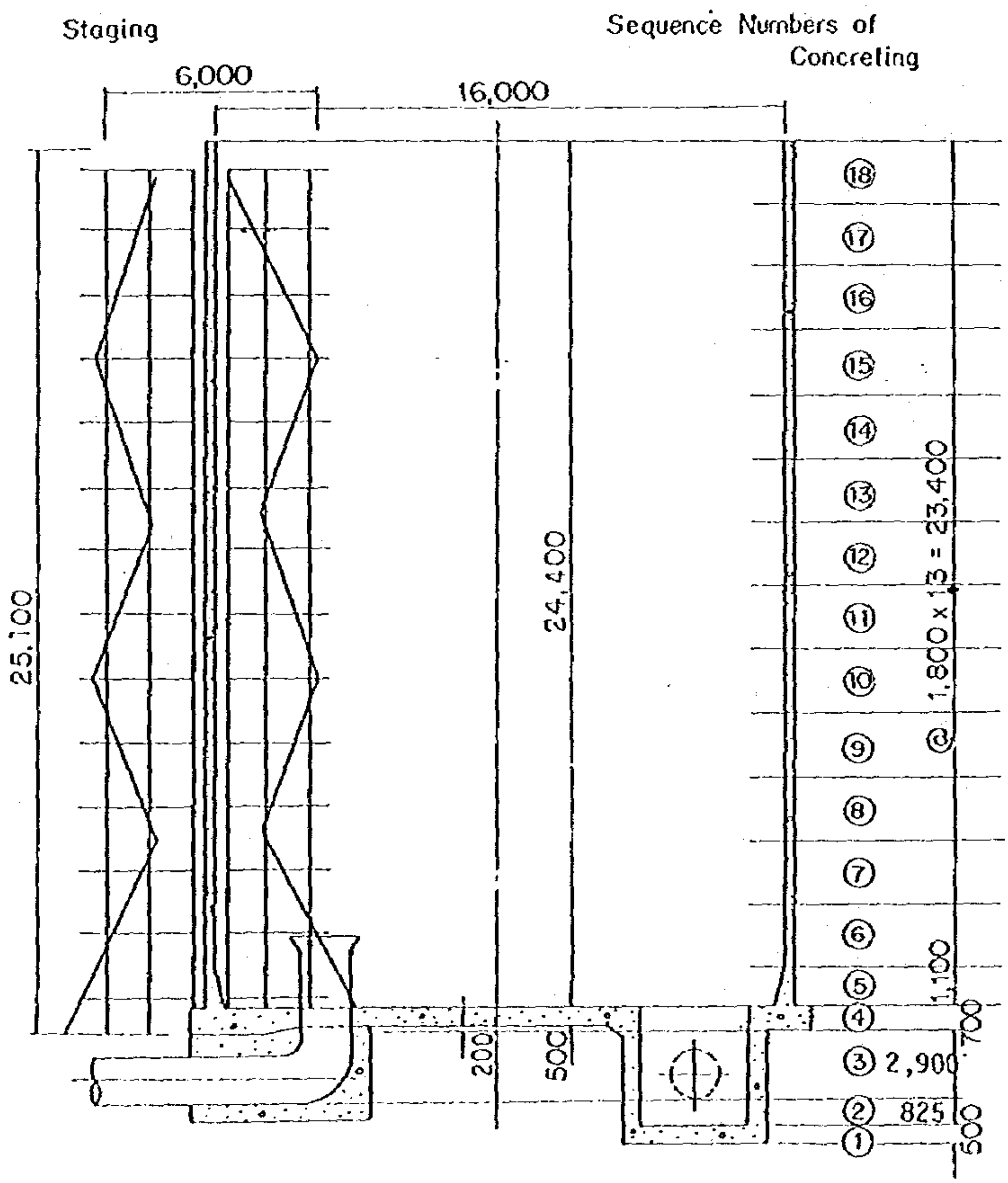
2.4.5. Spillway

The spillway works can be executed in paralleled with the head tank works, but the installation work of concrete pipeline should be carried out after finishing the main pipeline installation.

2.5. RECEIVING FACILITIES

Major works are excavation of 9,000 cu.m, embankment of 24,000 cu.m and membrane sheeting of 12,000 sq.m. The concrete structures are of small size. The total concrete volume was estimated at 1,300 cu.m only.

FIG. 2-4-1. Construction Method of Head Tank



The borrow pit should be provided near the site by the Contractor. The equipment for earth works will be a hydraulic excavator, bulldozers, a shovel loader, dump trucks, a tire roller and vibrating compactors.

The concrete should be transported from the central batching plant by transit mixers.

2.6. CONSTRUCTION EQUIPMENT LIST

Description	Specification	Unit	Remark
Intake Facilities			
Bulldozer	D 6 Class	2	
Hydraulic Excavator	0.6 m^3 Class	1	
Shovel Loader (Wheel)	1.5 m^3 Class	2	
Dump Truck	8 t Class	8	
Tire Roller	8 t Class	1	
Vibrating Compactor	110 kg Class	1	
Clamshell with Special crab	0.6 m^3	1	
Pile Driver	24 m leader	1	
Diesel Hammer	22 Class	1	
Crawler Crane	20 t Class	2	for P.C. beam erection
Pontoon	200 t Class	1	for clamshell mounting
Barge	100 t Class	2	for excavated material
Tugboat	20 t x 150 H.P.	1	for barge
Motor Boat	10 H.P.	1	for supervision
Concrete Pump	20 m^3/hr	1	
Concrete Vibrator	∅27 x 0.65 KW	4	for caisson work
- do -	∅38 x 1 KW	4	for general use
Air Compressor	35 P.S.	1	
Dewatering Pump	∅2" - ∅3"	4	
Pipeline			
Bulldozer	D 5 Class	6	
Hydraulic Excavator	0.6 m^3 Class	6	
Shovel Loader (Wheel)	1.5 m^3 Class	1	
Dump Truck	8 t Class	2	
Tire Roller	8 t Class	6	
Vibrating Compactor	110 kg Class	6	
Concrete Vibrator (Engine)	∅38 x 5 P.S.	12	
Dewatering Pump	∅2" - ∅3"	12	
Crawler Crane	35 t Class	1	
- do -	20 t Class	6	
Truck Crane	20 to Class	1	for miscellaneous works

Diesel Generator	50 KVA	9	
Welder	300 A	9	3 sets extra for
Blower	1.5 KW	9	special bend points
X-Ray Examination Instrument		3	sets
Wheel Crane	5 t	1	for jacking
Hydraulic Jack	100 t x 4	1 set	- do -
Pile Driver	24 m leader	1	- do -
Diesel Hammer	22 Class	1	- do -
Head Tank			
Bulldozer	D 6 Class	1	
Hydraulic Excavator	0.6 m^3 Class	1	
Tire Roller	8 t Class	1	
Vibrating Compactor	110 kg Class	1	
Crawler Crane	20 t Class	1	for R.C. pipe laying
Tower Crane	10 t x 10 KW	1	for head tank
Concrete Pump	20 m^3/hr	1	
Concrete Vibrator	∅38 x 1 KW	5	
Dewatering Pump	∅2	1	
Prestressing Jack and Pump		1 set	
Air Compressor	20 P.S.	1	
Receiving Facilities			
Bulldozer	D 6 Class	2	
Hydraulic Excavator	0.6^3 Class	1	
Shovel Loader (Wheel)	1.5 m^3 Class	2	
Dump Truck	8 t Class	6	
Tire Roller	8 t Class	1	
Vibrating Compactor	110 kg Class	2	
Concrete Vibrator	∅38 x 1 KW	3	
Dewatering Pump	∅2" - ∅3"	3	
Batching Plant			
Shovel Loader (Wheel)	1.5 m^3 Class	1	
Concrete Batcher without mixer	20 m^3/hr	1	
Transit Mixer	5 m^3	3	
- do -	3 m^3	3	
Water Supply Pump	2"	1	