THE GOVERNMENT OF MAURITIUS
MINISTRY OF ENERGY, WATER RESOURCES AND POSTAL SERVICES
CENTRAL WATER AUTHORITY

THE DETAILED DESIGN ON THE PORT LOUIS WATER SUPPLY PROJECT IN MAURITIUS

FINAL REPORT (1)

DATA BOOK FOR LOT- I

MARCH 1991

JAPAN INTERNATIONAL COOPERATION AGENCY

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PART I STRUCTURAL CALCULATION

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1. Structural calculation of River Diversion

1.1 General

The structural analyses are largely divided into three of the tunnel, inlet portal and outlet portal portions.

The whole tunnel will be lined with concrete. Two types of tunnel lining are applied in accordance with geological conditions: that is, Type I (50 cm in lining thickness) will be applied for $D_M \sim C_H$ class of rock. Type II (80 cm in lining thickness) will be applied for $C_L \sim C_M$ class of rock. The diversion tunnel is a permanent structure which is intended to be used for the waterway for water supply, and therefore, all the concrete linings shall be the reinforced concrete structure. Principal features of both tunnel types are given in Table 1.1.1.

Design loads for the analyses are as shown in Table 1.1.2.

The maximum internal and external loads which will determine the design of the structures are the water pressure to act after impounding the reservoir (water level in the dam expansion scheme). As for the tunnel upstream of the plug where the internal and external water pressures are balanced in the normal condition, the following extreme loading conditions have to be taken into consideration: (i) the external water pressure due to the remaining groundwater when the reservoir water level will suddenly drawdown, and (ii) the internal water pressure to act to the lining before the external water pressure will act to the lining. These extreme cases of loadings are rarely caused temporarily, and therefore, an allowable stress increased by 65% is applied in accordance with the standard. With regard to the loadings during the river diversion, particular examinations are considered unnecessary since its loading is very small as compared with the mentioned extreme loading cases even though the increase of the allowable stress is taken into consideration in the extreme cases.

In consideration that the internal and external water pressure will be balanced in the usual condition, application of consolidation grout pressure of 2 kg/cm^2 is considered sufficient. The grout pressure of backfill grout is 2kg/cm^2 at maximum which is same as or less than the consolidation grout pressure. Hence, a particular examination for backfill grout pressure is omitted. In consideration that the grout pressure is also very tentative, the allowable stress is increased to 210 t/m^2 in concrete compressive stress and 18 kg/cm^2 in concrete shearing.

Concrete and steel properties applied are presented in Table 1.1.3.

Rock loads will be supported with the tunnel supports erected during tunnel excavation. Thus, no rock loads are considered to be imposed on the concrete lining. Dead loads are also neglected because it is negligibly small compared with others.

The Otto-Frey-Bear's theory is applied for analyzing the tunnel structure against the internal and external loadings. The cylindrical shall theory is applied for examination against the grout pressure.

As for the inlet portal and outlet portal, the structures are examined by the frame structural analyses.

(2) Analysis for tunnel

Structural calculation is made on the basis of the following methods and assumption.

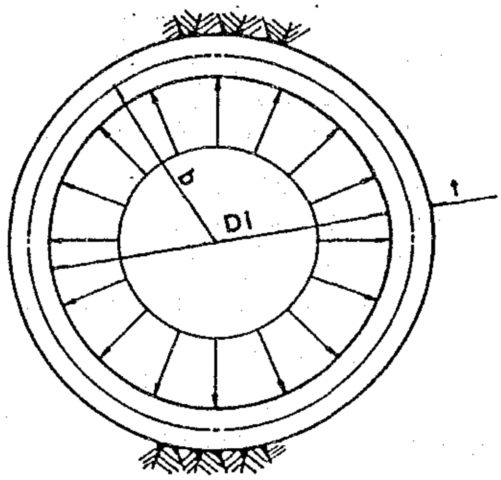
<u>Marks used in the calculation</u>

D_i : internal diameter of tunnel
t : thickness of concrete lining
b : radius of neutral axis of concrete lining (= r + t/z)
c : radius of external axis of concrete lining (= r + t)
P_i : internal water pressure
P_e : external water pressure
P_g : Grouting pressure
E_r : Elastic modulus of rock
E_s : Elastic modulus of reinforcement bar
E_c : Elastic modulus of concrete
n_c : Poisson's ratio of concrete
n_r : Poisson's ratio of rock
mc : Poisson's number of concrete
m_r : Poisson's number of rock
A_s : sectional area of reinforcement bar

a) Stress due to internal water pressure

For the calculation of stress due to internal water pressure, Otto-Frey-Bear's method is applied with the under conditions:

- Rock around tunnel lining exists infinitely,
- Concrete and rock are homogeneous and isotropic, and
- boundary between concrete lining and rock is continuous.



When the lining concrete is not reinforced by steel bar, according to Otto-Frey-Bear's method,

$$\sigma_t{}^c = Pi \times \left\{ \frac{\lambda(\frac{1}{2} + \frac{t}{Di})^2 - \frac{1}{4}}{\frac{t}{Di}(1 + \frac{t}{Di})} + \left(\frac{1}{4(\frac{b}{Di})^2} \times \frac{(\lambda - 1)(\frac{1}{2} + \frac{t}{Di})^2}{\frac{t'}{Di}(1 + \frac{t}{Di})} \right) \right\}$$

where,

$\sigma_t{}^c$ = stress in concrete in tangential direction

$$\lambda = \frac{Pc}{Pi} = \left\{ \frac{1}{2 \times \frac{t}{Di} \times (1 + \frac{t}{Di})} \right\} / \left\{ \frac{m_r' + 1}{m_r'(\frac{E_r'}{E_c'})} + \frac{(m_c' - 1)(\frac{1}{2} + \frac{t}{Di})^2 + \frac{1}{4}(m_c' + 1)}{m_c' \times \frac{t}{Di}(1 + \frac{t}{Di})} \right\}$$

where,

$$m_r' = m_r - 1$$

$$m_c' = m_c - 1$$

$$E_r' = \frac{m_r{}^2}{m_r{}^2 - 1}$$

$$E_c' = \frac{m_c^2}{m_c^2 - 1} E_c$$

$$b = (Di + t)/2$$

When the lining concrete is reinforced by steel bar, stress analysis is conducted by extending the above equations.

$$\sigma_t^s = Pi \frac{1}{2(\frac{As}{Di})} \left[\frac{1}{1 + 4.6E_s(\frac{As}{Di}) \left\{ \frac{\log 2}{E_c'} + \frac{m_r' + 1}{2.3m_r' \times E_r'} + \frac{1}{E_c'} \log(0.5 + \frac{t}{Di}) \right\}^{-1}} \right]$$

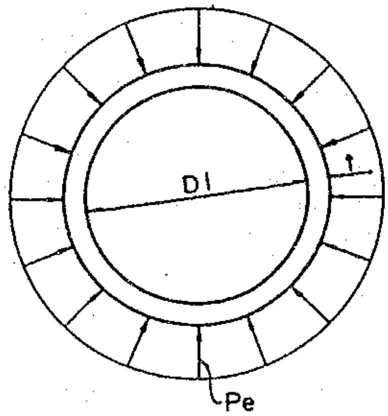
where;

σ_t^S = tensile stress in reinforced bar in tangential direction

b) Stress due to external water pressure

External water pressure is assumed to act as an uniform load along the concrete lining.

The compressive stress which occurs in tunnel lining is obtained from Otto-Frey-Bear's method.



$$\sigma_c = Pe \left[\frac{X}{X - \frac{m_c' - 1}{m_c' + 1} \times \frac{i}{j^2} \times Y} + \frac{Y}{\frac{m_c' + 1}{m_c' - 1} \times X - \frac{i}{j^2} \times Y} \right]$$

where;

$$X = \frac{1}{2Es \times \frac{As}{Di}} \times \frac{m_c' \times E_c'}{m_c' + 1} + 1$$

$$Y = \frac{1}{2Es \times \frac{As}{Di}} \times \frac{m_c' \times E_c'}{m_c' - 1} - 1$$

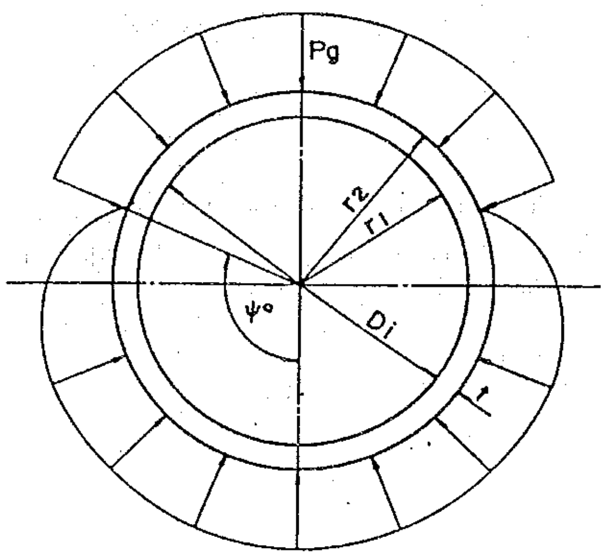
$j = 1 + 2 \times \frac{t}{Di}$

σ_c = compressive stress in concrete

c) Stress due to grouting pressure

The theory of thin cylindrical shell is applied to calculate the stress due to grouting pressure and calculation is executed as follows.

This calculation is obtained from the following equation by trial and error method.



The angle of boundary point is expressed by ψ°

$$\{\frac{3\bar{a}^2}{\eta} - \frac{b}{a}(\eta - 3) + \alpha(\eta - 3)(\psi_o - \pi)\} \sin\psi_o \cos\psi_o +$$

$$2\{\frac{\bar{a}}{\eta}(2 - \eta) - \bar{b}\,\bar{\eta} + (\eta - 1)(\psi_o - \pi)\} \sin^2\psi_o +$$

$$\{\bar{b}\pi\alpha - \frac{\bar{a}^2}{\eta} - 2\eta + \bar{a}\beta(\psi_o - \pi)\}(\psi_o - \pi) = 0$$

where;

$$\bar{a} = \sqrt{\eta^2 - 1}$$

$$\eta^2 = 1 + \frac{3Er}{4Ec} \times \frac{1 + \frac{t}{Di}}{\frac{1}{2} + \frac{t}{Di}} \left\{1 + \frac{1}{\frac{t}{Di}}\right\}^3 \times \frac{1 - \nu c^2}{1 + \nu r}$$

$$\alpha = \sqrt{\frac{\eta - 1}{2}}$$

$$\beta = \sqrt{\frac{\eta + 1}{2}}$$

$$\bar{b} = 1 + \frac{1}{3}\left(\frac{t}{Di} / \left(1 + \frac{t}{Di}\right)\right)^2 \times \eta^2$$

$$R = \{\alpha(3 - \eta) \sin \psi_o \cos \psi_o - 2\{1 + (\eta - 1) \sin^2 \psi_o + \bar{a}\beta (\psi_o - \pi)\}\}\eta$$

$$Q_1 = \alpha(3 + \eta) \sin \psi_o \cos \psi_o + 2\sin^2\psi_o - \bar{a}\beta (\psi_o - \pi)$$

$$Q_2 = \beta(\eta - 3) \sin \psi_o \cos \psi_o + 2\bar{a}\sin^2\psi_o \cos \psi_o - \bar{a}\alpha (\psi_o - \pi)$$

$$\sigma_{1,2} = \frac{N}{t} \pm \frac{6M\psi}{t_2}$$

$$N\psi = \frac{Pg \cdot Di}{2} \left\{1 - \frac{(\beta Q_2{}^2 - \alpha Q1)}{R \sin \psi_o}\right\} \cos \psi$$

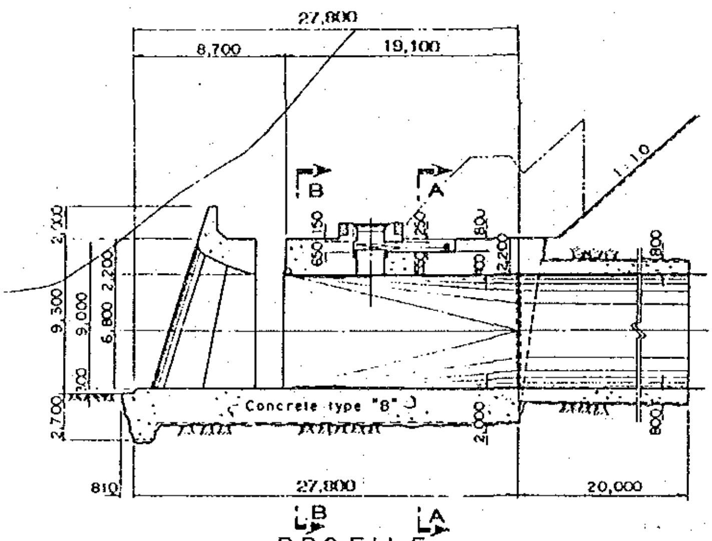
$$M\psi = -\frac{Pg \cdot Di^2}{4} \left\{-\frac{1}{a}\left(\frac{\eta^2}{R} Q_2 + \bar{a}\right) + \frac{\beta Q_2{}^2 - \alpha Q1}{R \sin \psi_o} \times \cos \psi\right\}$$

d) Result

Table 1.1.5 presents results of the tunnel analysis for internal and external pressures by the Otto-Frey-Bear's theory. Table 1.1.6 presents results of tunnel analysis for grout pressure by the theory of thin cylindrical shell.

Reinforcement bar arrangements determined through the above analysis and stress in each case are summarized in Table 1.1.4.

(3) Analysis for inlet portal



PROFILE

Sections to be analyzed are shown above (Section A-A and Section B-B).

(a) Combination of loads

Case-1: during flood

Water pressure, dead load, earth pressure and reaction ($= \frac{V^2}{2g}$)

Case-2: Just before opening of bulkhead gate of river outlet

Water pressure (up to El. 189.0 of spillway crest), dead load, earth pressure and reaction

Case-3: during gate installation (Section B only)

Gate load, dead load, earth pressure and reaction

(b) Load

i) Section A-A: Case 1

Water pressure : Pw= Pex (total static head) - Pin = $\frac{V^2}{2g}$

$$= \frac{(520/44.04)^2}{19.6} = 7.1 \text{ t/m}^2$$

Dead load : Dead loads to be considered are the ones under water.

Upper slab	$2.2 \times 1.4 = 3.1$ t/m^2
Side wall	$2.0 \times 1.4 = 2.8$ t/m^2

Earth pressure :

	Ka γ_s H
Top:	$0.5 \times 1.1 \times 1.1 = 0.6$ t/m^2
Bottom:	$0.5 \times 1.1 \times 9.0 = 5.0$ t/m^2

Reaction : $(11.0 \times 10.8 - 2.0 \times 10.8 - 44.04) \times 1.4/10.8 = 6.9$ t/m^2

ii) Section A-A: Case 2

Water pressure :

Upper slab	$189.0 - 138.0 = 51.0$ t/m^2
Side wall (top)	$189.0 - 136.9 = 52.1$ t/m^2

	Side wall (bottom)	$189.0 - 128.0 = 61.0$ t/m^2	
	Bottom slab	$189.0 - 127.0 = 62.0$ t/m^2	

Dead load	:	Upper slab	$2.2 \times 2.4 = 5.3$ t/m^2
		Side wall	$2.0 \times 2.4 = 4.8$ t/m^2
Earth pressure		:Top:	0.6 t/m^2
		Bottom:	5.0 t/m^2

Reaction : Dead load of upper slab & side wall buoyancy

$$\frac{(11.0 \times 10.8 - 2.0 \times 10.8 - 44.04) \times 2.4 - 11.0 \times 10.8}{10.8}$$

$= 0.8$ t/m^2

iii) Section B-B: Case 1

Water pressure :$Pw = \dfrac{(520/6.8^2)^2}{19.6} = 6.5$ t/m^2

Dead load	:	Upper slab	$2.2 \times 1.4 = 3.1$ t/m^2
		Side wall	$2.0 \times 1.4 = 2.8$ t/m^2
Earth pressure		:Top:	0.6 t/m^2
		Bottom:	5.0 t/m^2

Reaction : $(11.0 \times 10.8 - 2.0 \times 10.8 - 6.8^2) \times 1.4/10.8 = 6.6$ t/m^2

iv) Section B-B: Case 2

Water pressure		:Upper slab	51.0 t/m^2
		Side (top)	52.1 t/m^2
		Side (bottom)	61.0 t/m^2
		Bottom slab	62.0 t/m^2
Dead load	:	Upper slab	5.3 t/m^2
		Side wall	4.8 t/m^2
Earth pressure		:Top:	0.6 t/m^2
		Bottom:	5.0 t/m^2

Reaction : $$\frac{(11.0 \times 10.8 - 2.0 \times 10.8 - 6.8^2) \times 2.4 - 11.0 \times 10.8}{10.8}$$

$= 0.3$ t/m^2

v) Section B-B: Case 3

Gate load : $\frac{\text{Gate weight}}{\text{Bottom area}}$

$= \frac{50}{7.5 \times 1.1} \doteqdot 6.0 \ t/m^2$

Dead load : Upper slab $2.2 \times 2.4 = 5.3 \ t/m^2$

Side wall $2.0 \times 2.4 = 4.8 \ t/m^2$

Earth pressure :Top: $0.5 \times 1.95 \times 1.1 = 1.1 \ t/m^2$

Bottom: $0.5 \times 1.95 \times 9.0 = 8.8 \ t/m^2$

Reaction : $\frac{(10.8 \times 2.2 + 6.8 \times 2.0 \times 2) \times 2.4}{10.8} + 6.0 = 17.3 \ t/m^2$

(c) Allowable stress

Case	Increment Ratio	Concrete		Re-bar
		σ_{ca} (kg/cm^2)	τa (kg/cm^2)	σ_{sa} (kg/cm^2)
1	50%	105	12.8	2,700
2	50%	105	12.8	2,700
3	30%	91	11.1	2,340

(d) Analysis

SECTION A-A:

The section and dimension of Section A-A are shown in Fig. 1.1.2. The loading diagrams are given in Fig. 1.1.3. The structural analyses for Section A-A are made in Table 1.1..7. Its results are all shown in the bending moment, shearing force and axial force diagrams in Fig. 1.1.5 and 1.1.6.

As understood from the analysis results, the minimum reinforcement arrangement with D19 @300 will be sufficient for Section A-A.

An examination on stirrups are made as follows:

$$\tau = \frac{Q}{B \cdot j \cdot d}$$

B : width (cm)

j : 0.875

d : effective height (cm)

Mem	Spot	B (cm)	d (cm)	Q (ton)	τ (kg/cm^2)	τ_a (kg/cm^2)
2	2	100	240	188.6	8.98	12.8
6	6	100	170	164.9	11.09	12.8
13	14	100	240	188.6	8.98	12.8
15	15	100	240	188.7	8.99	12.8
17	1	100	240	188.1	8.96	12.8

Stirrups are not required in calculation, but 2-D16 @300 is applied to the corners of the structure for caution.

SECTION B-B:

The section and dimension of Section B-B are shown in Fig. 1.1.2. The loading diagrams are as seen in Fig. 1.1.4. The structural analyses for Section B-B are made in Table 1.1.8. Its results are presented in the bending moment, shearing force and axial force diagrams in Fig. 1.1.7 to 1.1.9.

Table 1.1..9 presents the calculations of internal stress in the reinforced concrete for the reinforcement arrangement provided based on the structural analyses.

Followings are examinations on the necessary stirrups:

Mem	Spot	B (cm)	S (cm)	τ (kg/cm^2)	A_v (cm)	Application
1 9	2 9	100	30	14.9	9.4	4-D19 = 11.46
10 12	11 12	100	30	14.6	9.1	4-D19

$$A_v = \frac{(\tau - \tau_a/2) \cdot B \cdot S}{\sigma_{sa}}$$

A_v : required area of stirrup (cm^2)
S : pitch of stirrup (cm)

Based on the above, the stirrups of 4-D19 are provided in Section B-B.

(4) Analysis for outlet transition

(a) Combination of load

Case-1: During diversion

Dead load + water pressure (Ground WL. - Tunnel center)

Case-2: Grouting condition

Dead load + Backfill grout pressure

(b) Load

Dead load : $1.0 \times 2.4 = 2.4\ t/m^2$

Water pressure : G.WL - Tunnel Center

$140 - 128.4 = 11.6\ t/m^2$

Grout pressure : $2\ kg/cm^2 = 20\ t/m^2$ (backfill grout)

Reaction :

Case-1 $R = (3.9 \times \pi + 3.4 \times 2) \times 1.0 \times 2.4/8.8$

$\doteqdot 5.2\ t/m$

Case-2 $R = 5.2 +$

$2 \times (2.019 \times 20 \times \cos 15° + 2.019/2 \times 20 \times \cos 45°)/8.8$

$= 17.3\ t/m$

(c) Allowable stress

Case	Increment Ratio	Concrete σ_{ca} (kg/cm^2)	Concrete τa (kg/cm^2)	Re-bar σ_{sa} (kg/cm^2)
1	30%	91	11.1	2,340
2	strength	210	18.0	3,000

(d) Analysis

The section, dimension and loading diagram for the outlet transition portion are shown in Fig. 1.1.10. Structural analysis for the outlet transition portion is given in Table 1.1.10. Fig. 1.1.11 and 1.1.12 present its results by the bending moment, shearing force and axial force diagrams. Table 1.1.11 presents the calculation of internal stress in the reinforced concrete for the reinforcement arrangement provided based on the structural analysis.

(5) Analysis for outlet portal

(a) Combination of load

. Dead load + earth pressure + surcharge load

Dead load	:	$1.0 \times 2.4 = 2.4$ t/m^2
Earth pressure	:	$Ka \cdot \gamma \cdot H$
		$= 0.5 \times 1.95H$
Surcharge load	:	1.0 t/m2
Earth pressure due to surcharge		$0.5 \times 1.0 = 0.5$ t/m^2

Water pressure is neglected because it will be drained through weep holes provided in wing wall.

Above condition is considered to be normal condition. So allowable stresses are:

$$\sigma_{ca} = 70 \text{ kg/cm}^2$$
$$\sigma_{sa} = 1{,}800 \text{ kg/cm}^2$$
$$\tau_a = 8.5 \text{ kg/cm}^2$$

(b) Analysis

The section, dimension and loading diagram for the outlet portal are shown in Fig. 1.1.13. The structural analysis is made in Table 1.1.12. Fig. 1.1.14 shows the bending moment, shearing force and axial force diagrams based on the structural analysis made in Table1.1.12. Table 1.1.13 presents the calculation of internal stress of reinforced concrete for the reinforcement arrangement provided on the basis of the structural analysis.

(6) Analysis for steel support

As mentioned, the load of rock to be loosened by tunnel excavation works is assumed to be supported by the steel support. Hence, the safety of steel support to be installed in examined herein.

The examination is made by checking the stress to be given to the steel support in the following steel support arrangements:

Tunnel Type	Steel Support Dimension (H-shape steel, mm)	Interval of Steel Support (m)
Type I	200 x 200 x 12	1.5
Type II	200 x 200 x 12	1.0

Loading conditions are based on those proposed by Terzaghi: that is, Terzaghi proposes the following rock loads in accordance with geological conditions in the tunnel excavation work.

Geological Condition	Height of rock to act as load (m)
CL ~ CM class	0.25 B ~ 0.35 (B+H)
CM ~ CH class	0 ~ 0.25 B

where, B : Width of tunnel excavation (= 7.8 m)
H : Height of tunnel excavation (= 7.938 m)

Thus, the rock load to act to the steel support is assumed as follows:

Tunnel Type	Steel support interval (m)	Height of rock to act as load (m)	Rock load (ton/each•m)
Type I	1.5	0.25 B = 1.95	7.31
Type II	1.0	0.35 (B+H) = 5.5	13.75

Note: In the above calculation of rock load, the unit weight of rock is assumed to be r=2.5 ton/m^3.

The stress in the steel support is calculated by the following equation:

$$\delta = T/A \pm M/Z$$

where, T: Axial force (ton)
A: Sectional area of H-shape steel (= 71.53 cm^2)
M: Bending moment (t•m)
Z: Section modules of H-shape steel (= 498 cm^2)

The structural model is shown in Fig. 1.1.15. The axial force (Ti) at each point of the steel support and maximum bending moment (Mmax) are calculated by the method of Procter and White in Table 1.1.14 for the tunnel Type-I and Table 1.1.15 for the tunnel Type-II, respectively.

Based on the calculated maximum axial force and bending moment, the maximum stress in the steel support is found as follows:

Tunnel Type	Max. Axial Force (t)	Max. Bending Moment (t•m)	Rock load (ton/each•m)	
			δ max (kg/cm^2)	δ min (kg/cm^2)
Type I	24.12	0.57	452	223
Type-II	50.98	1.21	956	470

As seen above, the stress in the steel support is sufficiently less than the allowable stress of 1,800 kg/cm^2. The sufficient allowance given to the steel support design is considered reasonable in consideration of uncertainties involved in the assumptions of loading.

2 Structure Calculation of Diversion Gate

2.1 General

One complete set of slide type diversion gate with guide frames and hoist tower is provided on the diversion tunnel inlet structure for closing the diversion tunnel.

The gate consists of a gate leaf, guide frames and a steel structure of hoist tower for assembling the gate leaf and for operation of the gate.

The gate is operated by a temporarily installed wire rope winch through rope sheaves provided on the hoist tower.

Slide type gate is selected for the gate by the following reasons:

(1) Less operation force due to low head at operation
(2) Simple structure which makes the fabrication easy and minimize possible troubles
(3) Low construction cost

The diversion gate leaf and guide frames are designed under the condition of reservoir water level of El. 189.0 m which is the spillway overflow crest elevation. The above design condition is based on the consideration that the reservoir water level rise after closing the diversion gate will be very rapid due to a small reservoir storage capacity and that the diversion gate structure should withstand the full reservoir water pressure for the necessary works in the diversion tunnel after the gate closure.

A steel hoist tower is provided on the concrete slab of the diversion tunnel inlet.

The allowable stresses for the diversion gate are applied 1.5 times of the allowable stresses for the permanent hydromechanical structures such as intake gates etc., since the diversion gate is installed as a temporary equipment for construction of this project.

The design data of the diversion gate are summarized below.

Type	:	Steel slide gate
Quantity	:	1 set
Clear Span	:	6.80 m
Clear height	:	6.80 m
Design water level	:	El. 189.00 m

Sill elevation	:	El. 129.00 m
Design head of gate	:	60.00 m
Water seal	:	4 edges rubber seal on downstream face of gate
Type of hoist	:	Steel hoist tower with winch
Operation	:	Temporary winch

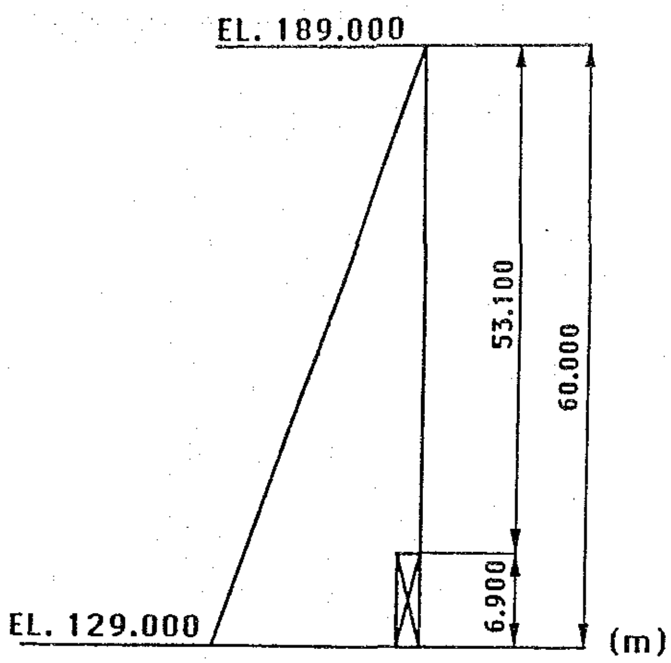
Design of diversion gate is shown in Fig. 1.2.1 to 1.2.2. Structural analyses of the diversion gate structure are made hereunder.

2.2 Structural Analysis of Diversion Gate

(1) Design conditions

Type	:	Sluice gate
Quantity	:	1 set
Clear span	:	6.8 m
Clear height	:	6.8 m
Max. design head	:	60 m (El. 189 - El. 129)
Max. deflection	:	1/800 of supporting span
Sealing method	:	4 edges rubber seal at downstream face of gate
Corrosion allowance	:	Not considered
Closing operation	:	Handled by temporarily installed winch

(2) Total hydraulic load



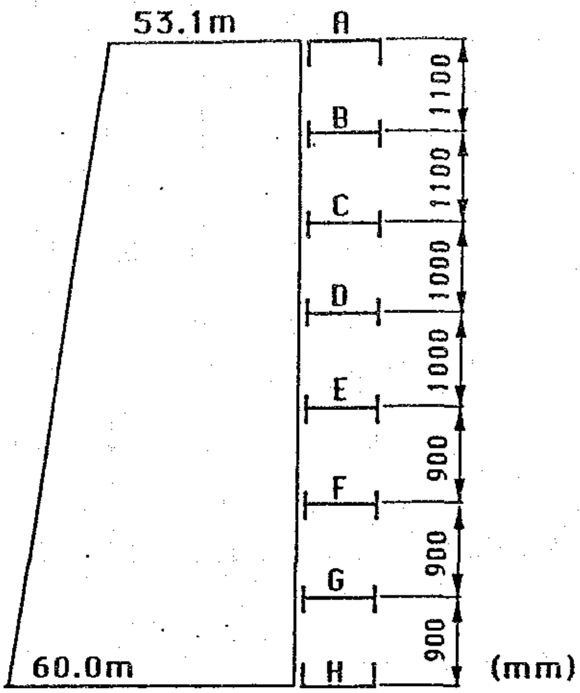
Pt = 0.5 x ($H2^2$ - $H1^2$) x B x Gw = 2732 (ton)

where,

Pt	=	Total hydraulic load	
H1	=	Design head at gate top	53.100 (m)
H2	=	Design head at gate bottom	60.000 (m)
B	=	Sealing span	7.000 (m)
Gw	=	Specific gravity of water	1.000 (ft/m^3)

(3) Main horizontal beams

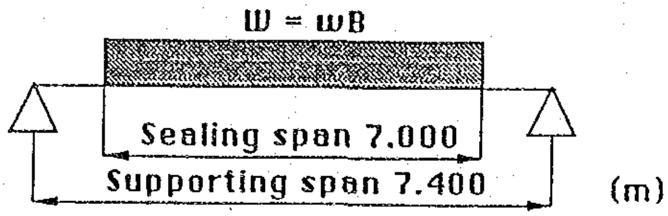
(a) Arrangement and reaction force (w) of main horizontal beams



A-beam	0.5 x (53.100 + 53.650) x 0.550 = 29.356 (t/m)
B-beam	0.5 x (53.650 + 54.750) x 1.100 = 59.620 (t/m)
C-beam	0.5 x (54.750 + 55.800) x 1.050 = 58.039 (t/m)
D-beam	0.5 x (55.800 + 56.800) x 1.000 = 56.300 (t/m)
E-beam	0.5 x (56.800 + 57.750) x 0.950 = 54.411 (t/m)
F-beam	0.5 x (57.750 + 58.650) x 0.900 = 52.380 (t/m)
G-beam	0.5 x (58.650 + 59.550) x 0.900 = 53.190 (t/m)
H-beam	0.5 x (59.550 + 60.000) x 0.450 = 26.899 (t/m)

The strength calculation has to be made for A-beam and B-beam which will be subject to the maximum load in each type of beam.

(b) Bending moment and shearing force



Mm = W x (2 x L - b)/8

Sm = W/2

where,

Mm	=	Max. bending moment	
Sm	=	Max. shearing force	
W	=	Water pressure load on each beam (ton)	
B	=	Sealing span	7.000 (m)
L	=	Supporting span	7.400 (m)

A-beam

W = 29.356 x 7.000 = 205.492 (ton)

Mm = 205.492 x (2 x 7.400 - 7.000)/8 = 200.355 (ton-m)
= 20035500 (kg-cm)

Sm = 205.492/2 = 102.746 (ton)
= 102746 (kg)

B-beam

W = 59.620 x 7.000 = 417.340 (ton)

Mm = 417.340 x (2 x 7.400 - 7.000)/8 = 406.907 (ton-m)
= 40690700 (kg-cm)

Sm = 417.340/2 = 208.670 (ton)
= 208670 (kg)

(c) Bending stress and shearing stress

$\sigma m = Mm/Z$

$\tau m = Sm/Aw$

where,

σm = Max. bending stress

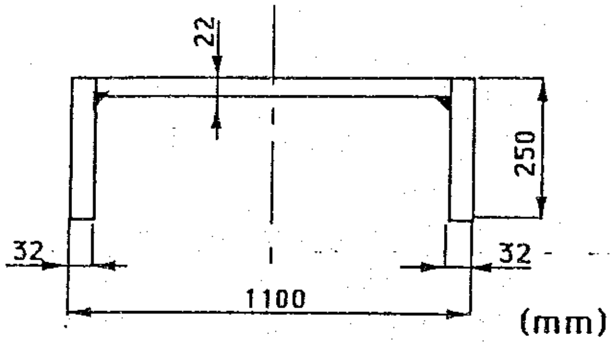
τm = Max. shearing stress

I = Moment of inertia (cm^4)

Z = Modulus of section (cm^3)

Aw = Area of web (cm^2)

A-beam



I = 660241 (cm^4)

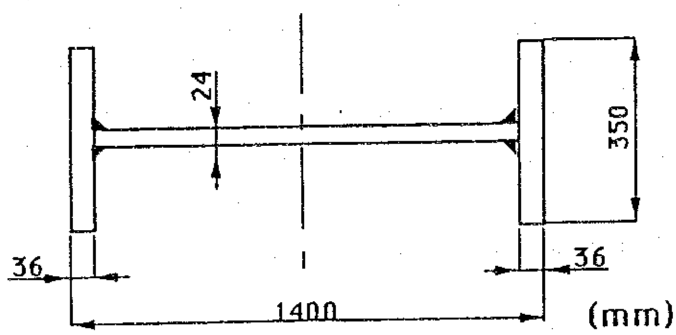
Z = 12004 (cm^3)

Aw = 228 (cm^2)

σm = 20035500/12004 = 1669 (kg/cm^2) < 2700 (kg/cm^2)

τm = 102746/228 = 451 (kg/cm^2) < 1575 (kg/cm^2)

B-beam



I = 1640793 (cm^4)

Z = 23440 (cm^3)

Aw = 264 (cm^2)

$\sigma m = 40690700/23440 = 1736\ (kg/cm^2) < 2700\ (kg/cm^2)$

$\tau m = 208670/264 = 790\ (kg/cm^2) < 1575\ (kg/cm^2)$

(d) Deflection

$\delta m = W \times (L^3 - L \times B^2/2 + B^3/8) / (48 \times E \times 1)$

where,

δm = Max. deflection

W = Water pressure load on each beam (kg)

E = Young's modulus (kg/cm^2)

A-beam

$$\delta m = \frac{205492 \times (740^3 - 740 \times 700^2/2 + 700^3/8)}{(48 \times 2.1 \times 10^6 \times 660241)} = 0.824\ (cm)$$

$\delta m/L = 0.824/740 = 1/898 < 1/800$

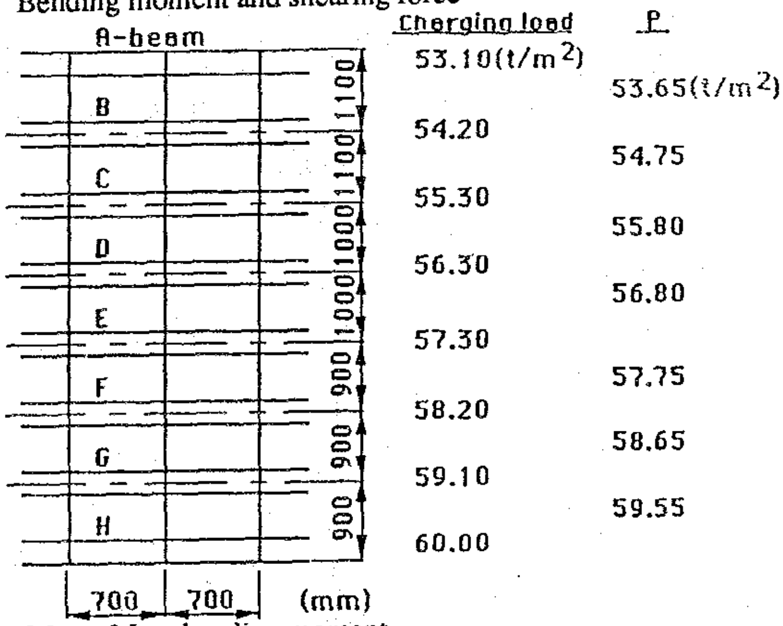
B-beam

$$\delta m = \frac{417340 \times (740^3 - 740 \times 700^2/2 + 700^3/8)}{(48 \times 2.1 \times 10^6 \times 1640793)} = 0.673\ (cm)$$

$\delta m/L = 0.673/740 = 1/1100 < 1/800$

(4) Vertical girders

(a) Bending moment and shearing force



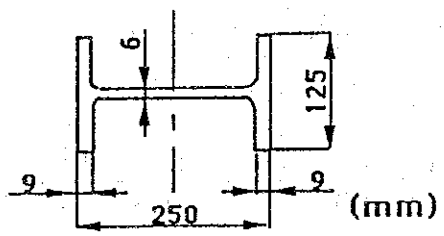
Mm = Max. bending moment

Sm = Max. shearing force

No.	P (kg/cm^2)	a (cm)	b (cm)	Mm (kg-cm)	Sm (kg)
1	5.365	70	110	491345	14083
2	5.475	70	110	501419	14372
3	5.580	70	110	408503	12695
4	5.680	70	110	415823	12922
5	5.775	70	90	326769	11117
6	5.865	70	90	331861	11290
7	5.955	70	90	336954	11463

(b) Bending stress and shearing stress

(b-1)

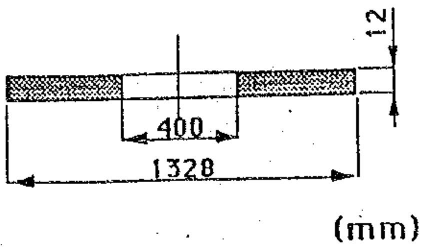


Z = Modulus of section 324.0 (cm^3)
Aw = Area of web 13.9 (cm^2)

No.	Bending stress (kg/cm^2)	Shearing stress (kg/cm^2)
1	1516	1013
2	1548	1034
3	1261	913
4	1283	930
5	1009	800
6	1024	812
7	1040	825

Allowable bending stress: 2700 (kg/cm^2)
Allowable shearing stress: 1575 (kg/cm^2)

(b-2)

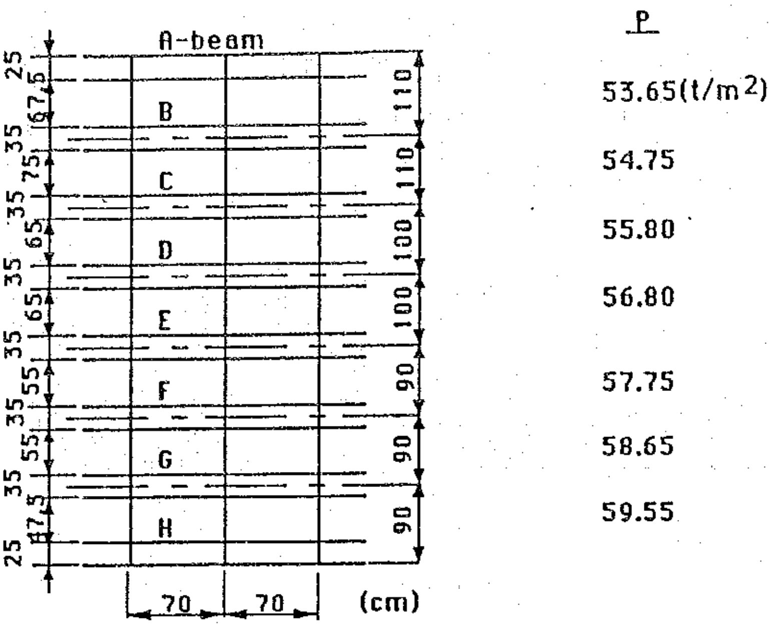


Z	= Modulus of section	3430.8 (cm^3)
Aw	= Area of web	111.4 (cm^2)

No.	Bending stress (kg/cm^2)	Shearing stress (kg/cm^2)
1	143	126
2	146	129
3	119	114
4	121	116
5	95	100
6	97	101
7	98	103

Allowable bending stress: 2700 (kg/cm^2)
Allowable shearing stress: 1575 (kg/cm^2)

(5) Skin plate



$\sigma = (k \times a^2 \times p/t^2)/100$

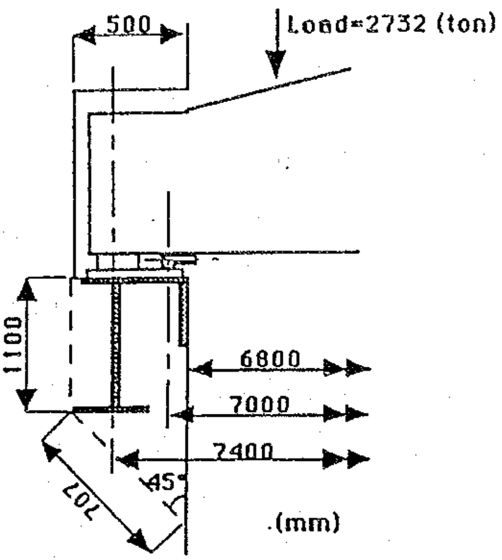
where,

σ = Bending stress (kg/cm^2)
k = Coefficient by "b/a"
a = Short span of plate (cm)
b = Long span of plate (cm)
p = Water pressure (kg/cm^2)
t = Thickness of skin plate (cm)

No.	a	b	b/a	k	p	t	σ
1	67.5	70	1.04	32.6	5.365	2.3	1506
2	70	75	1.07	34.0	5.475	2.3	1724
3	65	70	1.08	34.4	5.580	2.3	1533
4	65	70	1.08	34.4	5.680	2.3	1561
5	55	70	1.27	40.6	5.775	2.3	1341
6	55	70	1.27	40.6	5.865	2.3	1362
7	47.5	70	1.47	45.0	5.955	2.3	1143

Allowable bending stress: 2700 (kg/cm^2)

(6) Guide frame



(a) Shearing area of concrete section

$A1 = (110.0 + 70.7) \times 690.0 = 124683\ (cm^2)$

$A2 = (110.0 \times 50.0 + 0.5 \times 50.0 \times 50.0) \times 2 = 13500\ (cm^2)$

$A = A1 + A2 = 124683 + 13500 = 138183\ (cm^2)$

(b) Shearing stress of concrete

$\tau = load/2A$

$= 2732000/(2 \times 138183) = 9.89\ (kg/cm^2) < 12.75\ (kg/cm^2)$

(7) Weight of the gate leaf

	Weight/unit	Q'ty	Weight
(a) Main horizontal beams			
1100 x 250 x 22 x 32 x 7400	2254 (kg)	2	4508 (kg)
1400 x 350 x 24 x 36 x 7400	3316 (kg)	6	19896 (kg)
(b) Skin plate			
6950 x 7600 x 23	9537 (kg)	1	9537 (kg)

(c)	Vertical beams			
	250 x 125 x 6 x 9 x 6950	206 (kg)	5	1030 (kg)
	1328 x 12 x 6950 - 400 x 400 x 12 x 7	764 (kg)	4	3056 (kg)
(d)	Side beams			
	1100 x 200 x 22 x 32 x 6950	1942 (kg)	2	3884 (kg)
		Total		41911 (kg)
				x 1.2 ≑ 51 (ton)

(8) Strength of the steel supporting structure

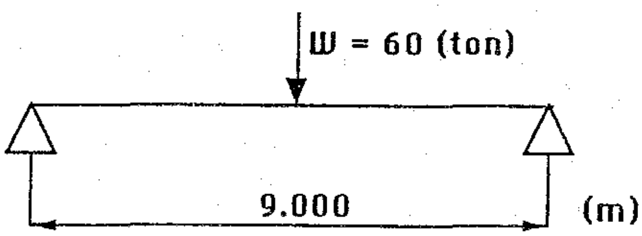
(a) Operation load

W = w + F = 59.4 ≑ 60 (ton)

where,

W = Total operation load (ton)
w = Weight of gate leaf 51 (ton)
F = Friction load due to guide frame (2m head) 8.4 (ton)

(b) Beam

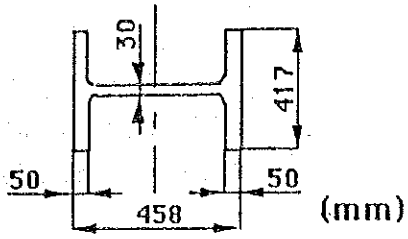


Mm = WL/4 = 13500000 (kg-cm)
Sm = W/2 = 30000 (ton)

where,

W = Total operation load (ton) 60000 (kg)
L = Beam span 900 (cm)

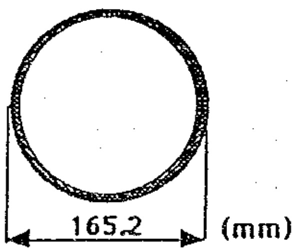
Beam section



Z = 8170 (cm^3)
Aw = 107.4 (cm^2)

σm = 13500000/8170 = 1652 (kg/cm^2) < 2700 (kg/cm^2)

(c) Main posts



Steel pipe: Dia. 165.2 (mm) x Thickness 5 (mm)

$Pk = \pi^2 E1/kL^2$ = 5795 (kg/cm^2)
$Pa = (W + w)/2A$ = 1265 (kg/cm^2)

Pa < Pk

where,

Pk	=	Critical backling pressure (kg/cm2)	
Pa	=	Acting load by gate leaf (kg/cm2)	
E	=	Young's modulus (kg/cm2)	
I	=	Moment of inertia	808 (cm4)
L	=	Post height	850 (cm)
k	=	Safety factor	4
W	=	Total operation load	60000 (kg)
w	=	Beam weight	3735 (kg)
A	=	Sectional area of main post	25.2 (cm^2)