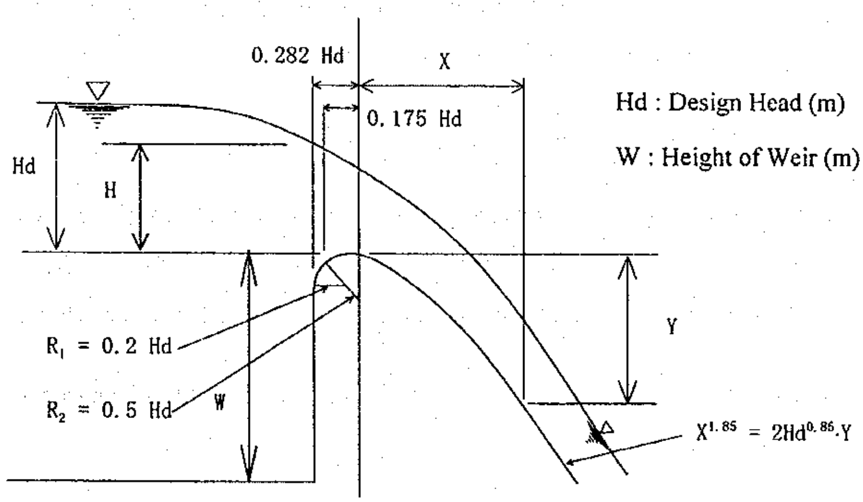
- By applying an ungated type, easier and more economical maintenance work can be expected.

(2) Crest Shape and Overflow Discharge

Crest shapes of ungated overflow weir which approximate the profile of the under nappe of a jet flowing over a sharp-crested weir provide the ideal form for obtaining optimum discharges without creating negative pressure at the overflow crest. The shape of a profile depends on the design head, the inclination of the upstream face and the height of the weir.

The profile shape for a crest with a vertical upstream face is illustrated below. This Harrold's standard shape of the overflow weir is adopted.



Harrold's Standard Shape of Overflow Crest

The discharge over a crest with Harrold's standard shape is given by the following formulas:

$$Q = C \cdot B \cdot H^{3/2}$$

$$C = C' \cdot \{1 - Md \cdot (H/Hd)^{1.5}\}$$

$$Md = 0.0756 \cdot (Hd/B)^{0.5}$$

$$C' = 1.60 \frac{1 + 2a(H/Hd)}{1 + a(H/Hd)}$$

$$a = \frac{Cd - 1.6}{3.2 - Cd}$$

$$Cd = 2.20 + 0.416\ (Hd/W)^{0.99}$$

Where,

Q : overflow discharge (m^3/sec)

C : variable coefficient of discharge considering pier contraction

B : crest length (m)

H : overflow head at crest (m)

C' : variable coefficient of discharge

Md : reduction factor of pier contraction

a : constant value estimated from Cd

Hd : design head (m)

Cd : coefficient of overflow discharge when H and Hd are equal

W : height of overflow weir (m)

(3) Overflow Weir of Service Spillway

Shown in Fig. 7.4.4 is the typical cross section with Harrold's standard shape of the service spillway assuming that the design head (Hd) is 2.9 m. The top of overflow crest is set at EL. 148.9 m corresponding to the Normal Water Surface.

When the crest length of the weir is 15 m, the overflow discharge is calculated as shown in Fig. 7.4.5.

According to the flood control plan, when a 100-year probable flood with peak inflow discharge of 290 m^3/s flows into the dam reservoir, the outflow is regulated and the maximum outflow becomes 120 m^3/s at the net water surface of EL. 151.32 m (refer to Fig. 7.4.6). The Surcharge Water Surface is set at EL. 151.8 m, which corresponds to the flood control capacity 3,100,000 m^3 including 20 % allowance. The outflow discharge at the Surcharge Water Surface is estimated at 155 m^3/s.

(4) Overflow Weir of Emergency Spillway

Shown in Fig. 7.4.4 is the typical cross section with Harrold's standard shape of the emergency spillway assuming that the design head (Hd) is 3.5 m. The emergency spillway is located at the both sides of the service spillway. The top of overflow crest is set at EL. 151.8 m corresponding to the Surcharge Water Surface.

When the crest length of the emergency spillway is 60 m in total, the overflow discharge is calculated as shown in Fig. 7.4.7. The emergency spillway will pass the probable maximum flood (PMF) in combined operation with the service spillway. The total discharge of them is given in Fig. 7.4.8.

The Maximum Water Surface, which corresponds to the design inflow discharge of 1,600 m^3/s (PMF), is set at EL. 155.3 m. After regulating the inflow through the reservoir, the outflow becomes 1,310 m^3/s (refer to Fig. 7.4.9).

Set up on Side Channel Section

Flow over the crest falls into a side channel opposite the overflow weir, turns an approximate right angle, and continues into a control portion. The theory of flow in a side channel is based on the law of conservation of linear momentum, assuming that the only forces producing motion in the channel result from the fall in the water surface in the direction of the channel axis.

The design formula and concept for side channel section is given as follows:

① Slope of overflow weir, m, is 1 : 0.7, and channel wall of the opposite bank is vertical wall.

② Bottom slope of side channel, i_1, is designed to be not steeper than 1/13.

③ Ratio of bottom width, B, at the downstream end of the side channel, to the water depth, *d*, (d/B) is designed to be about 0.5.

④ Froude number, Fr, at the end of side channel is assumed to be Fr < 0.5, generally Fr = 0.44.

$$Fr = \frac{V}{\sqrt{gD}}$$

where,

V	:	flow velocity (m/s)
g	:	gravity acceleration (9.8 m/s^2)
$D = \left(\frac{A}{T}\right)$	:	hydraulic depth (m)
A	:	cross sectional area (m^2)
T	:	width of water surface (m)

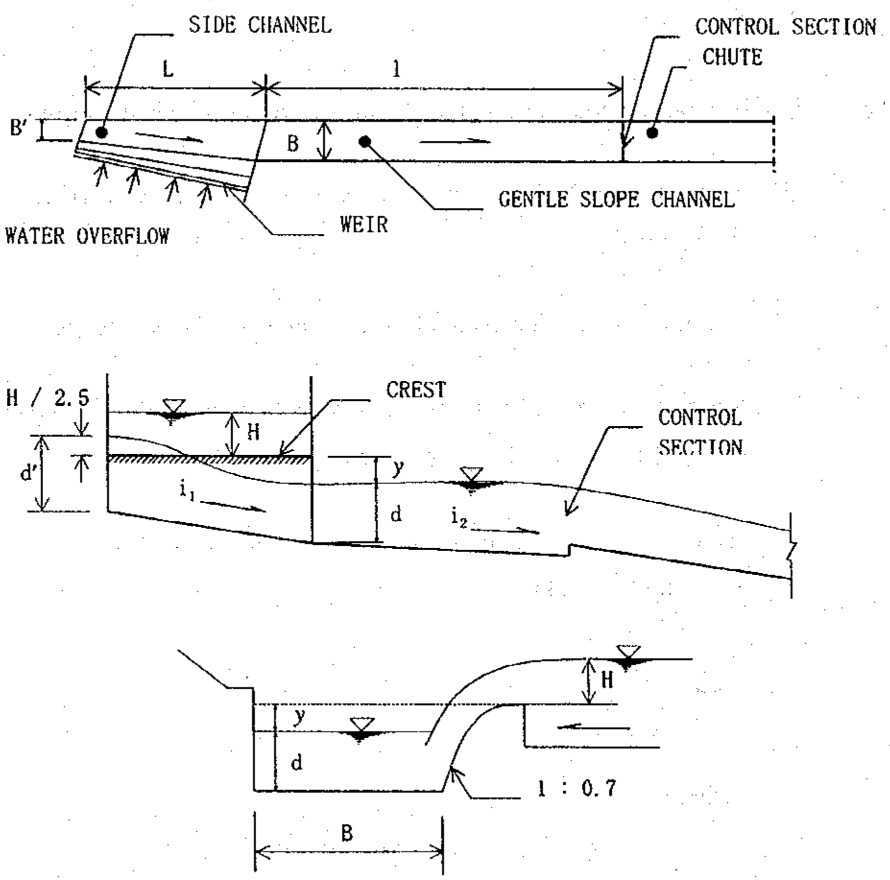
⑤ Water depth (above crest level) at the upper end of the side channel shall be lower than 1/2.5 of overflow depth.

⑥ Gentle slope channel is designed to meet the formula mentioned in ④ and must be gentle enough.

⑦ Overflow weir is arranged at the downstream end of the gentle slope channel so that discharge flows into the chute smoothly.

⑧ Each value shall meet the following formula:

$$\frac{d^3 B^2 \left(1+\frac{m}{2}\frac{d}{B}\right)^3}{\left(1+m\frac{d}{B}\right)} = \frac{Q^2}{gFr^2}$$



Side Channel

In case of bottom width, Bx, at distance, x, from the downstream end of the side channel is obtained by formula

$$Bx = B\left\{1-(1-\alpha)\frac{x}{L}\right\}$$

Where,

B : bottom width at the downstream end of the side channel (m)

B' : bottom width at the upstream end of the side channel (m)

α : B' / B

x : distance from the downstream end of the side channel (m)

L : length of the side channel (m)

Length of gentle slope channel, l, shall be designed to meet the following formula.

$$l \geq 4d$$

Using these formulas, dimension of the side channel is decided as follows:

Q (m^3/s)	m	Fr	B (m)	B' (m)	d (m)	i_1	i_2	L (m)	l (m)
1,310	1:0.7	0.44	24	18	10.991	0	0	14.373	44.600

For any short reach of the side channel, the momentum at the beginning of the reach plus any increase in momentum due to the external forces must equal the momentum at the end of the reach. Flow profile in the side channel is calculated based on momentum equation as follows:

$$\Delta h = \frac{Q_1(v_1+v_2)}{g(Q_1+Q_2)}\left(\Delta v + \frac{q v_2 \Delta x}{Q_1}\right)$$

Where,

Δh : a rise in water level in the Δx section (m)

Q_1 : discharge of downstream section (m^3/s)

Q_2 : discharge of upstream section (m^3/s)

v_1 : average velocity at the downstream section (m/s)

v_2 : average velocity at the upstream section (m/s)

q : inflow per unit width (overflow discharge) ($m^3/s/m$)

Δv : $v_1 - v_2$

g : gravity acceleration (9.8 m/s^2)

Δx : distance between sections (m)

Calculation on flow profile is carried out by the trial method from downstream to upstream section. By use of the above equation, the water surface profile can be determined for any particular side channel by assuming successive short reaches of channel once a starting point is cleared.

The trial-and-error computation results of water surface profile are shown in Table 7.4.1. The resulting water surface profile at PMF in the side channel is shown in Fig. 7.4.10.

Chute Structure

Discharge generally passes through the critical stage in the spillway control portion and enters the chute as shooting flow. To avoid a hydraulic jump in the chute, the flow must remain at the supercritical stage throughout the length of the chute.

The flow velocity will be determined by Manning formula:

$$v = \frac{1}{n} R^{2/3} S^{1/2}$$

Where,

v	:	flow velocity (m/s)
R	:	hydraulic radius = A/P (m)
A	:	flow section area (m^2)
P	:	wetted perimeter (m)
n	:	Manning's roughness coefficient
S	:	energy gradient

The velocity and depth of free surface flow conform to the principle of the conservation of energy as expressed by the Bernoulli's theorem. It states "The absolute energy of flow at any cross section is equal to the absolute energy at a downstream section plus intervening loses of energy". As applied to the figure as shown below, this relationship can be expressed as follows:

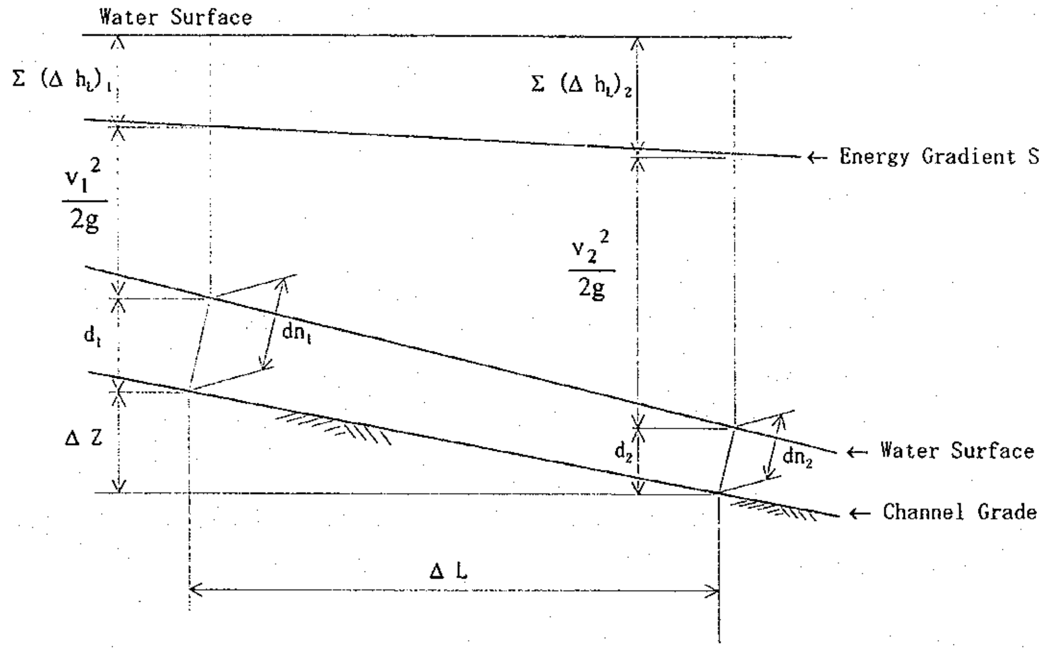
$$\Delta Z + d_1 + \frac{v_1^2}{2g} = d_2 + \frac{v_2^2}{2g} + \Delta h_L$$

$$\Delta h_L = S \cdot \Delta L = \frac{S_1 + S_2}{2} \cdot \Delta L$$

Where,

ΔZ	:	difference of elevation (m)
d_1, d_2	:	vertical depth of flow (m)

v_1, v_2	:	velocity (m/s)
g	:	gravity acceleration (9.8 m/s^2)
Δh_L	:	energy losses (m)
S	:	average energy gradient in the section
S_1, S_2	:	energy gradient at the section
ΔL	:	distance between sections (m)



<u>Flow Depth in Chute</u>

The coefficient of roughness n, will depend on the nature of the channel surface. For conservative design, the frictional loss should be maximized when evaluating the depth of flow. For determining the depth of flow in a concrete line channel, a value of n about 0.018 should be assumed, in order to account for air swell, wave action, etc.

About the freeboard of the chute wall, the surface roughness, wave action and air bulking shall be considered. They are related to the velocity and energy content of the flow and the energy per unit width can express in terms of velocity and depth of flow. An empirical expression based on this relationship which gives a reasonable indication of desirable freeboard values is as follows:

$$F = 0.61 + 0.037 \cdot v \cdot d^{1/3}$$

Where,

F : freeboard (m)

v : velocity (m/sec)

d : depth of flow (m)

From these explanations, the water surface profile in the chute and wall height considering freeboard are calculated as shown in Table 7.4.2.

To avoid the water to spring away from the floor and reduce the surface contact pressure, the floor shape for convex curvature should be made slightly flatter than the trajectory of a free-discharging jet. It is issuing under a head equal to the specific energy of flow as it enters the curve. The curvature should approximate a shape defined by the following equation:

$$y = x \cdot \tan\theta + \frac{x^2}{K[4*(d+hv)\cos^2\theta]}$$

Where,

θ : slope angle of floor upstream from the curve (degree)

d : depth of flow (m)

hv : energy of velocity (m/sec)

To assure positive pressure along the entire contact surface of the curve K should be equal to or greater than 1.5.

The calculated results are shown in Fig. 7.4.11.

Stilling Basin

The design discharge of energy dissipater should be the largest figures of the following discharges. In this case, the design discharge of stilling basin becomes 340 m^3/s.

- Outflow capacity at the Surcharge Water Surface (155 m^3/s)
- 100-year probable flood discharge (340 m^3/s), which is estimated by the hydrological model of the damsite itself having the catchment area of 53 km^2.

Regarding the energy dissipater type, a hydraulic jump basin with endsill is employed.

The height of sidewall shall be designed so as not to overflow it when the maximum outflow discharge (Q_d = 1,310 m^3/s) after regulating the design discharge for emergency spillway (PMF) flowed into the stilling basin.

The design formula for stilling basin is given as follows:

$$v_1 = 0.95\ \sqrt{2g(H_s - H_b)}$$

$$h_1 = \frac{Q}{A} = \frac{Q}{B \cdot v_1}$$

$$F_1 = \frac{v_1}{\sqrt{gh_1}}$$

$$h_2 = \frac{h_1}{2}\left(\sqrt{1+8F_1^{\ 2}} - 1\right)$$

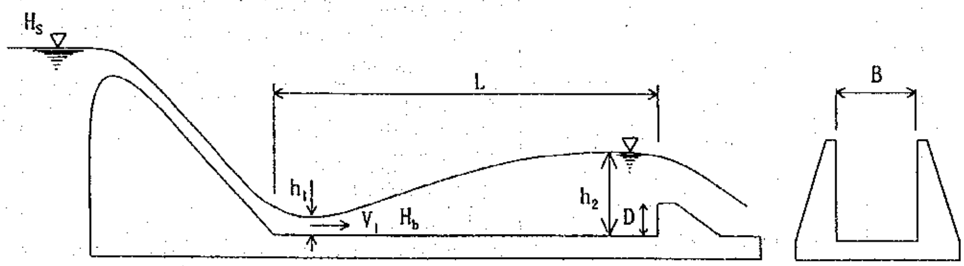
$$L \geq 4.5\ h_2$$

$$\frac{D}{h_1} = \frac{\left(1+2F_1^{\ 2}\right)\sqrt{1+8F_1^{\ 2}} - 1 - 5F_1^{\ 2}}{1+4F_1^{\ 2} - \sqrt{1+8F_1^{\ 2}}} - \left(\frac{\sqrt{g}}{2}F_1\right)^{\frac{2}{3}}$$

$$H = \left(\frac{Q_d}{2B}\right)^{\frac{2}{3}}$$

Where,

- v_1 : velocity at entrance (m/s)
- g : gravity acceleration (9.8 m/s^2)
- H_s : water surface in side channel (EL. m)
- H_b : apron elevation (EL. m)
- h_1 : water depth at entrance (m)
- Q : design discharge (m^3/s)
- B : width of apron (m)
- F_1 : Froude number
- h_2 : conjugate depth (m)
- L : length of apron (m)
- D : height of endsill (m)
- H : overflow depth (m)
- Q_d : design discharge for sidewall height (m^3/s)



The width and elevation of apron are determined at 24 m and EL. 82.5 m respectively considering the topographical condition at the downstream section.

Using the above formulas, dimension of the stilling basin was decided as follows.

Item	Symbol	Unit	Values
Water Surface in Side Channel	H_s	EL. m	144.7
Apron Elevation	H_b	EL. m	82.5
Velocity at Entrance	v_1	m/s	33.2
Water Depth at Entrance	h_1	m	0.427
Design Discharge	Q	m^3/s	340.0
Width of Apron	B	m	24.0
Froude Number	F_1	-	16.2
Conjugate Depth	h_2	m	9.58
Length of Apron	L	m	43.12 (44.0 m)
Height of Endsill	D	m	6.01 (6.5 m)

The flow depth over the endsill is determined by the maximum outflow discharge (Q_d = 1,310 m^3/s). The depth is estimated at 9.0 m. Therefore, the height of the sidewall becomes 15.5 m (= 9.0 m + 6.5 m). The calculated results are shown in Fig. 7.4.12.

7.4.3 Structural Design

Loading Condition to be Considered

The safety of the spillway structure should be verified through detailed structural calculations. The combination of loads needed for the structural calculation is given hereunder.

for Structures facing Reservoir Water

Case	Condition of Reservoir	Combination of Loads
1	Normal Water Surface	Self weight Earth pressure with earthquake (100 %) Hydrostatic pressure Hydrodynamic pressure (100 %) Inertial force during seismic motion (100 %) Uplift pressure
2	Surcharge Water Surface	Self weight Earth pressure with earthquake (50 %) Hydrostatic pressure Hydrodynamic pressure (50 %) Inertial force during seismic motion (50 %) Uplift pressure
3	Maximum Water Surface	Self weight Earth pressure Hydrostatic pressure Uplift pressure
4	Empty Reservoir	Self weight Inertial force during seismic motion (50 %)

for Structures not facing Reservoir Water

Case	Condition	Combination of Loads
1	Normal Condition	Self weight Earth pressure Hydrostatic pressure Uplift pressure
2	Earthquake Condition	Self weight Earth pressure with earthquake (100 %) Hydrostatic pressure Hydrodynamic pressure (100 %) Inertial force during seismic motion (100 %) Uplift pressure

Structural Stability

The spillway structures should be founded on rock and should be safe against shear and overturning. It is an usual practice to determine the cross sectional area for a two-dimensional design process based on the assumption that the structure consists of a number of cantilevered beams which are independent of each other. The required conditions of the structural safety are described hereinafter.

(1) Conditions for Safety against Shear

Regarding shear safety, an evaluation should be made using Henny's formula for the contact plane of the rock foundation.

$$SF = \frac{\tau_0 \cdot l + f \cdot V}{H}$$

Where,

SF	:	safety factor
H	:	total shearing force acting in the shear plane per unit width (tf/m)
V	:	total normal force acting on the shear plane per unit width (tf/m)
τ_0	:	shear strength of rock foundation (tf/m^2)
l	:	length of shear plane (m)
f	:	coefficient of internal friction of rock foundation

(2) Overturning

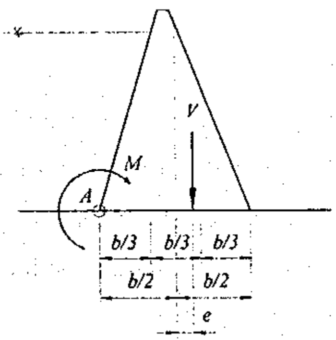
When the location of the resulting force of a load is within the central one-third point, tensile stress in the vertical direction is not produced at the upstream face of the structure.

The following formula is used for the stability evaluation.

$$e = \frac{b}{2} - \frac{M}{V}$$

Where,

- b : width of base (m)
- M : total moment at point A per unit width (tf-m/m)
- V : total normal force acting on the shear plane per unit width (tf/m)
- e : eccentricity (m)



(3) Bearing Capacity of Foundation

The maximum principal stress in the foundation must be kept within allowable rock bearing capacity, which derived from the following:

$$q_1 = \frac{V}{b} \cdot \left(1 + \frac{6e}{b}\right)$$

$$q_2 = \frac{V}{b} \cdot \left(1 - \frac{6e}{b}\right)$$

Where,

- q_1 : maximum principal stress (tf/m^2/m)
- q_2 : minimum principal stress (tf/m^2/m)
- V : total normal force acting on the shear plane per unit width (tf/m)
- b : width of base (m)
- e : eccentricity (m)

Required Conditions

The following conditions shall be satisfied in stability calculations:

for Structures facing Reservoir Water

Case	Condition of Reservoir	Earthquake	Structural Stability	
			Against Shear	Against Overturning
1	Normal Water Surface	100 %	SF ≥ 4	$e = \frac{b}{2} - \frac{M}{V} < \frac{b}{6}$
2	Surcharge Water Surface	50 %	SF ≥ 4	$e = \frac{b}{2} - \frac{M}{V} < \frac{b}{6}$
3	Maximum Water Surface	0 %	SF ≥ 4	$e = \frac{b}{2} - \frac{M}{V} < \frac{b}{6}$
4	Empty Reservoir	50 %	SF ≥ 4	$e = \frac{b}{2} - \frac{M}{V} < \frac{b}{6}$

for Structures not facing Reservoir Water

Case	Condition	Earthquake	Structural Stability	
			Against Shear	Against Overturning
1	Normal Condition	0 %	SF ≥ 4	$e = \frac{b}{2} - \frac{M}{V} < \frac{b}{6}$
2	Earthquake Condition	100 %	SF ≥ 4	$e = \frac{b}{2} - \frac{M}{V} < \frac{b}{3}$

Notes SF : safety factor
b : width of base (m)
M : total moment at point A per unit width (tf-m/m)
V : total normal force acting on the shear plane per unit width (tf/m)
e : eccentricity (m)

Material Properties

Material properties to be used for structural calculation are summarized below:

(1) Shear Strength of Foundation Rock

Rock Class	Estimated Shear Strength	Coefficient of Internal Friction
CL class	τ_o=30 tf/m^2	0.7
CM-L class	τ_o=45 tf/m^2	0.8
CM-H class	τ_o=50 tf/m^2	0.8

(2) Properties of Construction Material

Material	Unit Weight (tf/m^3)
Reinforced Concrete (Thick Structure)	2.35
Reinforced Concrete (Thin Structure)	2.50

Material	Wet Density (tf/m³)	Submerged Density (tf/m³)	Internal Friction Angle (degree)
Impervious Material (Dam)	2.11	1.19	30.0
Semi-pervious Material (Dam)	2.11	1.27	35.0
Pervious Material (Dam)	1.94	1.16	45.0
Sandy Soil (Backfill)	1.90	0.90	35.0

Results of Structural Design

Sections analyzed are shown in Fig. 7.4.13. The results of stability analysis are summarized as follows:

Parts			Loading Condition	Structural Stability		
				Against Shear	Against Overturning (m)	
				SF ≥ 4	e	<B/6 or <B/3
Overflow Weir	Service Spillway		Normal Water Surface	6.626	1.078	B/6 = 2.552
			Surcharge Water Surface	6.165	0.191	
			Maximum Water Surface	5.774	1.490	
			Empty Reservoir	30.199	0.574	
	Emergency Spillway		Normal Water Surface	5.795	0.441	B/6 = 2.385
			Surcharge Water Surface	5.450	0.889	
			Maximum Water Surface	4.967	1.589	
			Empty Reservoir	20.259	1.405	
Control Portion	Left Side	Upper	Normal Condition	6.887	1.296	B/6 = 2.197
			Earthquake Condition	4.081	2.614	B/3 = 4.393
		Lower	Normal Condition	7.445	0.785	B/6 = 0.878
			Earthquake Condition	4.373	1.603	B/3 = 1.757
	Right Side	Upper	Normal Water Surface	4.297	1.879	B/6 = 2.537
			Surcharge Water Surface	5.081	1.964	
			Maximum Water Surface	4.895	2.175	
			Empty Reservoir	10.143	0.771	
		Impervious Zone	Normal Water Surface	4.051	2.093	B/6 = 3.217
			Surcharge Water Surface	4.433	2.018	
			Maximum Water Surface	4.834	1.859	
			Empty Reservoir	5.612	1.409	
		Lower	Normal Condition	7.010	1.055	B/6 = 2.027
			Earthquake Condition	4.133	2.355	B/3 = 4.053
Chute	i=1:4		Normal Condition	24.133	0.060	B/6 = 0.483
			Earthquake Condition	12.863	0.262	B/3 = 0.967
	i=1:2		Normal Condition	35.965	0.085	B/6 = 0.450
			Earthquake Condition	18.371	0.153	B/3 = 0.900
Stilling Basin			Normal Condition	6.709	1.108	B/6 = 1.567
			Earthquake Condition	4.053	2.530	B/3 = 3.133

7.4.4 Consolidation Grouting

Consolidation grouting is carried out (1) to strengthen the spillway foundation, (2) to reduce settlements in the foundation, (3) to fill the void with cement slurry between the foundation and the concrete placed and (4) to improve permeability of the foundation. The consolidation grouting area is limited to the upstream side of the spillway foundation from the dam axis.

(1) General

(a) Target Lugeon Value

Since consolidation grouting aims to strengthen the foundation and fill gaps or cracks between concrete and the foundation, target lugeon value is not set.

(b) Area and Depth of Consolidation Grouting

The consolidation grouting is limited to be executed in the foundation of the spillway upstream of the dam axis. The depth of the grouting is set at 5.0 m.

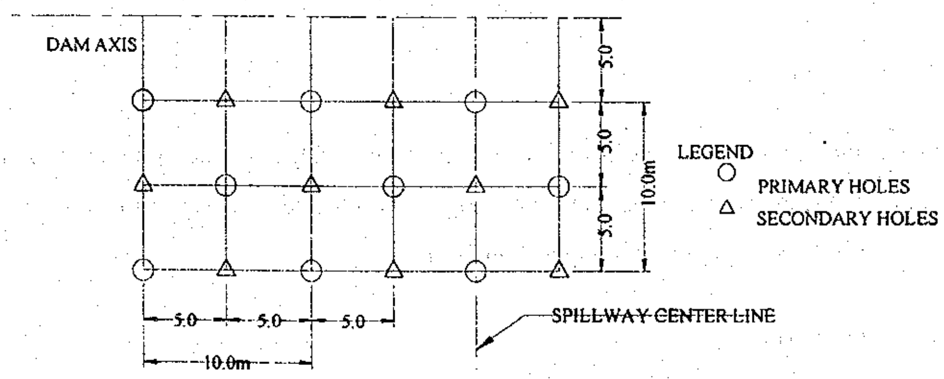
(c) Time of Consolidation Grouting Works

Consolidation grouting is carried out three (3) weeks after placing concrete for the base slab of the spillway.

(2) Grouting Works

(a) Arrangement of Grout Holes and Work Procedure

The designed holes are distributed with 5.0 m x 5.0 m mesh and the primary holes are carried out followed by the secondary holes. The distribution of primary and secondary holes is illustrated below.



Arrangement of Consolidation Grouting Holes

(b) Length of Stage

The length of each stage is set at 5.0 m

(c) Additional Holes and Completion of Consolidation Grouting

Additional holes are not carried out. When the designed holes are completed, consolidation grouting works is completed.

(d) Displacement of Ground Surface

Ground displacement meter is installed to manage displacement of the ground surface by consolidation grouting. The ground displacement meter shall have accuracy of 0.01 mm and measurement range of 20 mm with automatic recording function.

(3) Water Pressure Test

Water pressure test is carried out at all designed holes to get basic data for the decision of grouting pressure and initial mix proportion of grout milk. The standard length of stage is 5.0 m and packer is placed at the slab concrete where is the immediate upside of the test stage. P - Q curves (Pressure - Quantity Curve) shall be prepared based on the test data to calculate lugeon values and to grasp the existence of critical pressure.

7.4.5 Spillway Bridge

(1) Function of the Bridge

The Spillway Bridge will function as an access road to the dam management complex. It will not be used for public transportation.

(2) Type of Bridge

From the maintenance point of view, concrete bridge is recommended. Among concrete bridge types, PC girder type bridge was selected because of low construction cost with the span length of longer than 20 m.

(3) Design Criteria

The Spillway Bridge is designed based on the following design criteria.

- Span length : 23.34 m
- Width : 5.0 m
- Clearance above the water surface at the bridge when PMF is discharged through the spillway (EL. 152.083 m) : larger than 1.0 m
- Live load and other common design criteria : described in Design Criteria Repot (volume 1), July 1998

(4) Design Result

The design results of the bridge are shown in Fig. 7.4.14.

7.5 Diversion Facilities

7.5.1 Layout of Structures

Required Function

The objective of the diversion tunnel is to divert the streamflow around or through the damsite during the construction period. It can minimize serious potential flood damage to the work in progress.

Diversion structures consist of main cofferdam, inlet and outlet portals, and diversion tunnel. The tunnel is designed to be capable of managing a 25-year probable flood that has been worked out as 280 m^3/s. The crest elevation of main cofferdam is determined by taking freeboard of 0.5 m against 25-year probable flood with the peak discharge of 280 m^3/sec.

Layout of Diversion Structures

Considering meandering of the river course at the upstream of the damsite and the location of the other structures, it is advantageous to choose the left abutment for the diversion tunnel.

Location of the inlet portal is determined at the concave side on the left bank of the river so that the tunnel length will result shortest. This location is approximately 600 m upstream from the dam axis. The outlet portal is planed to easily make the centerline of discharge into the original center of the river flow. The outlet portal is designed to be projected to Kreo River to protect the outlet from clogging by fallen masses of rocks.

The longitudinal gradient of diversion tunnel is determined straight by the elevation of inlet and outlet portals together with consideration of other conditions such as the height of access road, temporary facilities and so on.

The elevations of the inlet and outlet beds are set at EL. 98.500 m and EL 83.800 m respectively considering the riverbed elevation. The longitudinal gradient is set at 1/30 considering flow velocity and the elevation of the inlet bed.

The main cofferdam is located upstream of the inlet portal to stop the streamflow through the damsite.

The layout plan and profile are shown in Figs. 7.5.1 and 7.5.2.

Geological Features along Diversion Tunnel

Based on the results of the borehole drilling and adits excavation around tunnel alignment, the geological features are clarified as shown in Fig. 7.5.3 and general features are described hereunder.

(1) The geology along the tunnel consists of pyroclastic rocks and sedimentary rocks of Neogene to Quaternary. The uniaxial compressive test results for this rocks show qu = 30 ~ 50 kgf/cm².

The elastic wave exploration was conducted at the dam axis whose geological features are considered almost same as the one at the tunnel alignment. The elastic wave velocity at the dam axis was detected at v = 2,500 ~ 3,000 m/s.

Judging from this test result, the geology along the tunnel alignment is considered to be soft rock. It is classified as DI d2 or DII a ~ DII d2 in the rock classification criteria issued by Japan Society of Civil Engineering (JSCE) (refer the Table 7.5.1). This rock classification together with the competence factor mentioned in Item (4) is used to determine the dimensions of support or thickness of tunnel lining.

(2) Through visual observation of the face and the sidewall of the adits excavated in the geological survey, the rocks fairly stands itself without support and there is no sign of expansion nor squeezing ground water inflow.

(3) Though the rocks have been fairly consolidated, it is easily broken by hammer, marked by fingernail and also disintegrated by fingertips.

(4) Competence factor (Gn) calculated by the following equation shows Gn = 2.5 ~ 4.

$$Gn = \frac{q_u}{\partial H}$$ where : q_u = uniaxial compression strength (tf/m^2)

γ = unit weight of rock mass (tf/m^3)

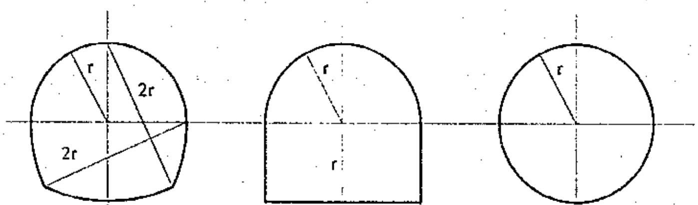
H = covering depth (m)

7.5.2 Hydraulic Design

Internal Cross Section of Diversion Tunnel

(1) Type of Cross Section

In general, following three types of tunnel section are employed for soft rock mass, (1) Standard horseshoe type, (2) Top half round with bottom half square type and (3) Round type.



Type (1) Type (2) Type (3)

The advantage and disadvantage of these three types are compared in the table below.

Evaluation	Type (1)	Type (2)	Type (3)
Advantage	There is only a small concentration of stress at the corner of wall and invert. Construction period is shorter than type (3).	Construction period is shorter than type (3).	There is no concentration of stress at concrete lining.
Disadvantage	No disadvantage	Stress may be concentrated at the corner of wall and invert, which may cause clacks or collapse of lining.	Since the work of invert is complicated, construction period is longer than other two types.
Evaluation	Applicable	Not Applicable	Not Applicable

From the result of comparison in the above table, type (1) Standard Horseshoe type is selected.

(2) Dimensions of Internal Cross Section

In the case of open channel flow in the standard horseshoe shaped tunnel, 80 % of the height of tunnel will be used as actual flow section for the design discharge to avoid the creation of self-priming phenomenon as common practice.

Dimensions of internal cross section are designed based on the conditions as shown below:

- Shape of cross section : standard horseshoe with 2r
- Design discharge : Q = 280 m^3/s (25-year provable flood)
- Type of flow : open channel (80 % of tunnel height)
- Longitudinal gradient : $i = 1/30$
- Roughness coefficient : $n = 0.015$ (concrete lining)

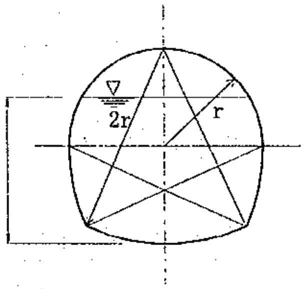
Internal cross section is decided as follows:

- Height and width : 5.6 m (r = 2.8 m)

Discharge amount was calculated in accordance with the hydraulic formula as shown below:

$$Q = \frac{1}{n} A \cdot R^{2/3} \cdot i^{1/2}$$

Where,



R : hydraulic radius (m) = A/P

A : flow sectional area (m^2)

P : wetted perimeter (m)

n : Manning's roughness coefficient = 0.015

i : longitudinal gradient = 1/30

Inlet Portal

The layout of the inlet structure is shown in Fig. 7.5.4. The water will be dammed up by 1.5 m at the entrance of the tunnel so that design discharge of 280 m^3/s can be introduced into the tunnel smoothly by super critical flow. The overflow crest is set at EL. 103.0 m. This wall may act as barrier and prevent the tunnel from flowing sand and cobbles. Dimensions of the inlet structure were designed in accordance with the following formula.

$$H = (1 + fe)\frac{V_2^{\,2}}{2g} + h_2$$

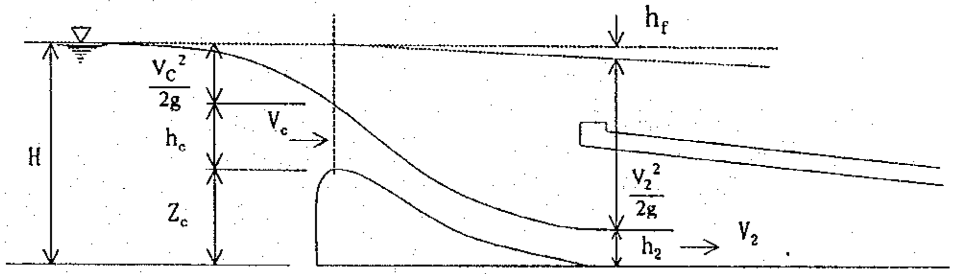
$$Zc = (1 + fe)\frac{V_2^{\,2}}{2g} + h_2 - \frac{V_c^{\,2}}{2g} - hc$$

$$V_c = \frac{Q_c}{Bh_c}$$

$$h_C = \sqrt[3]{\frac{Q_c^{\,2}}{gB^2}}$$

Where,

H	: total water depth (m)
fe	: entrance energy loss coefficient (0.2)
V_2	: flow velocity at entrance of tunnel (m/s)
h_2	: flow depth at entrance of tunnel (m)
g	: gravity acceleration (9.8 m/s^2)
V_c	: critical flow velocity at top of overflow crest (m/s)
h_c	: critical water depth at overflow crest (m)
Q_c	: design discharge (m^3/s)
B	: bottom width of overflow crest (m)
Zc	: height of overflow weir (m)



Q_c (m^3/s)	B (m)	h_c (m)	V_c (m/s)	V_2 (m/s)	h_2 (m)	fe	H (m)	Zc (m)
280	5.6	6.342	7.884	12.444	4.480	0.2	13.961	4.448

7.5.3 Structural Design

Typical Cross Section of Diversion Tunnel

(1) Tunnel Excavation Method

In deciding the tunnel excavation method, following two (2) points shall be taken into consideration together with the geological conditions mentioned in the succeeding section.

- To adopt a method not to loosen the rock mass as much as possible.
- Occurrence of local concentration of stress in the rock mass shall be avoided.

Considering the above points, it is quite necessary to finish the excavated surface smooth and to make exposure time of the rock mass as short as possible. To realize these, the application of tunneling machine for soft rock and New Austrian Tunneling Method (NATM) are adopted for the excavation of the diversion tunnel and supporting system. The internal section of the tunnel has an area of about 40 m^2. Then "Upper half advancing excavation method" is applied which is commonly applied for tunnels with the same areas.

(2) Supporting System for Tunnel Excavation

NATM provides rock bolts and shotcrete to the excavated face immediately after the completion of one cycle of excavation by tunneling machine to protect the rock mass from fall rocks and to strengthen week layer and furthermore expecting formation of arch action in the rock mass.

Working efficiency of rock bolt and shotcrete is summarized in the following tables.

Working Efficiency of Rock bolt

No.	Working Efficiency	Description
1	Fastening effect (Suspending effect)	Rocks loosened by excavation are fastened to sound rock mass so that they would not fall. This is the principal effect.
2	Building-up beam effect	In case of rock mass made of bedding strata, rock bolts can fastened them and they will serve as a composite beam.
3	Tensioning effect	Tensioned rock bolts will give the compressed strength to the rock mass.
4	Building-up arch effect	Rock mass fastened by rock bolts around the tunnel builds up arch effect.

Working Efficiency of Shotcrete

No.	Working Efficiency	Description
1	Bonding effect	Bonding effect between shotcrete and the excavated rock surface will assist arch effect of the rock mass.
2	Effect of solid member	Rock mass covered with relatively thick shotcrete will act as one solid member. This is very effective at soft rock and soil ground.
3	Distribution effect of external force	Earth pressure can be distributed to steel support or rock bolts.
4	Reinforcing effect of weak zone	Shotcrete filled in defects will avoid concentration of earth pressure and reinforce weak zone.
5	Covering effect	Shotcrete will prevent the excavated surface from weathering, control groundwater inflow and prevent wash out of fine particles from rock mass.

When the support design is carried out, the geological conditions and method of construction, etc. shall be considered. Construction materials are also selected for determining the support pattern.

Standardized selection method of tunnel support categories is expressed in the Table 7.5.2. For the diversion tunnel, competence factor is relatively small because of the soft rock, The following works are planed as the supporting system for the diversion tunnel.

- Primary Shotcrete	:	Immediately after the completion of the one circle of excavation, the primary shotcrete with the thickness of 50 mm is sprayed to protect excavated internal face temporary from collapse or relaxation.
- Wire mesh	:	Steel mesh (5 mm in diameter, 150 mm x 150 mm) is installed on the primary shotcrete to reduce spring back of the secondary shotcrete and to reinforce the shotcrete.
- Steel support	:	H steel with the size of 125 mm x 125 mm x 6.5 mm x 9 mm is applied for the steel support with the interval of 1.0 m taking the rock mass conditions into account.
- Secondary shotcrete	:	Secondary shotcrete with the thickness of 100 mm is sprayed after the installation of the steel support to protect the surface of the rock mass from collapse or relaxation.
- Rock bolts	:	Rock bolts with 25 mm in diameter are driven into the rock mass to prevent the rock mass from its deep collapse or relaxation. The length of the rock bolt is 3.0 m at the standard section and 4.0 m at the inlet and outlet sections, while the interval of the rock bolt is 1.2 m in circumference and 1.0 m in longitudinal direction.

(3) Thickness of Concrete Lining

The design thickness of the lining is determined mainly by the internal tunnel width together with geological strength of rock mass, ground water pressure and so on.

The following thickness is generally applied for the design thickness of the lining with steel support.

Internal Tunnel Width (m)	Design Thickness of Concrete Lining (cm)
3	20 ~ 40
5	30 ~ 50
10	40 ~ 70

Considering the geology of soft rock with the uniaxial compression strength of 30 ~ 50 kgf/cm^2, which was classified as DI d2 or DII a ~ DII d2, 50 cm of the design thickness is applied for the concrete lining with steel support.

(4) Typical Cross Section of Diversion Tunnel

The typical cross section of the diversion tunnel is shown in Fig. 7.5.5.

Structural Analysis of Tunnel before Concrete Lining

Structural analysis of the tunnel is conducted to confirm the design mentioned above by finite element method (FEM).

(1) Section for Analysis

The analysis is conducted for the plug section where the area of the excavation section is larger than the standard section.

(2) Conditions for Analysis

(a) Analytical Domain

The analytical domain is limited to 300 m wide and 170 m high as shown in Fig. 7.5.6 together with mesh. The boundary conditions of the domain are horizontally fixed, vertically movable for side boundary, and both horizontally and vertically fixed for bottom boundary.

(b) Properties of Material and Rock Mass

The properties of the supporting material are shown in the table below.

Structure	Modulus Elasticity (kgf/cm^2)	Sectional Area (cm^2)	Moment Inertia (cm^4)	Section Modulus (cm^3)	Remarks
Shotcrete (15 cm)	34,000	1,500	28,125	3,750	1.0 m wide
Steel Support (H 125 mm)	2,100,000	30	839	134	for 1 piece

The properties of the rock mass are shown in Fig. 7.5.7.

(c) Analytical Steps

In accordance with the construction procedure, the analysis is conducted with five (5) steps as shown in Fig. 7.5.8.

(3) Results of Analysis

The deformation map is shown in Fig. 7.5.9 as the results of the analysis. The settlement of the land surface above the tunnel is calculated at only 0.09 mm which is judged to be safe enough.

The deformation of the lining concrete is calculated as shown in the table below.

Location	Crown	Spring (Vertical)	Spring (Horizontal)	Invert
Deformation (cm)	0.8	0.1	0.4	1.0

The above figures of deformation of the supporting system are considered not to be harmful at any portions.

The active stresses to the shotcrete and steel support are calculated as shown in the table below.

Item	Unit	Crown	Spring	Invert
Max. Axial Force for Shotcrete	(tf)	92.8	15.0	60.0
Max. Bending Moment for Steel Support (H 125 mm)	(tf·m)	0.2	1.9	0.2

While the allowable compressive strength of the shotcrete and the allowable bending moment of the steel support are calculated as below.

(a) Allowable Compressive Strength of Shotcrete

Allowable compressive stress (f_{ck}) of the shotcrete is defined as f_{ck}= 60 kgf/cm^2.

Increment of allowable compressive stress for temporary works is 1.5.

Sectional area of the shotcrete (A) is 1,500 cm^2.

Allowable compressive strength of the shotcrete (N_r)

$N_r = f_{ck} \times A = 60 \times 1{,}500 \times 1.5 = 135{,}000$ kgf = 135 tf > 92.8 tf (crown)

(b) Allowable Bending Moment of Steel Support

Allowable compressive stress due to bending of steel support (H 125), (σ a)

σ a = 1,400 kgf/cm^2

Increment of allowable compressive stress for temporary works is 1.5.

Section modulus (Z) = 134 cm^3

Allowable bending moment of the steel support (H 125), (Mr)

Mr = σ_a x Z x 1.5 =1,400 x 134 x 1.5 = 281,400 kgf·cm = 2.8 tf·m > 1.9 tf·m (Spring)

Therefore, the active stresses for both of the shotcrete and the steel support are smaller than the allowable stresses.

(c) Fracture Safety Factor (Point Safety Factor) and Maximum Shear Strain

Fracture safety factor (point safety factor) and maximum shear strain are calculated for the evaluation of safety of the rock mass due to the excavation of the tunnel.

As a result of the calculation, the contour line maps of fracture safety factor and maximum shear strain are shown in Figs. 7.5.10 and 7.5.11, respectively.

As for the fracture safety factor, the safety factors exceed 1.0 at any portions. The critical shear strain of the rock mass, which has 50 kg/cm^2 of unconfined compressive strength, is considered about 1.0 %. Since the maximum calculated shear strain of the rock mass is 0.5 %, the shear strain does not exceed the critical value at any portions.

Structural Analysis of Tunnel after Concrete Lining

(1) Method of Analysis

The concrete lining is presumed as a rigid frame structure connected with short strait members on elastic supports. However, the connecting points of the side wall and the invert are considered to be pin connections. Fig. 7.5.12 shows the model of analysis.

(2) Conditions for Analysis

(a) Design Conditions

The design conditions for the concrete lining are shown in the table below.

Item	Unit	Design Value	Remarks
Allowable Compressive Stress	kgf/cm^2	70	
Allowable Shear Stress	kgf/cm^2	3.6	
Thickness of Concrete Lining	cm	50	Standard Section
Thickness of Concrete Lining	cm	100	Plug Section
Covering of Steel Bar	cm	10	
Coefficient of Sub-grade Reaction	tf/m^2	100,000	E=10,000 kgf/cm^2
Ground Water Level	m	29 to 58	
Grout Pressure	kgf/cm^2	6.0	

(b) Analysis Cases

The analysis conducted for the following five (5) cases.

Case	Status	Lining Weight	Grouting	Ground Water Level	Increment of Allowable Stress
1	Normal	applied	None	H = 29 m	1.0
2	Reservoir Filling	applied	None	H = 58 m	1.0
3	Grouting (1)	applied	Grout at Crown	H = 29 m	1.5
4	Grouting (2)	applied	Grout at Side Wall	H = 29 m	1.5
5	Grouting (3)	applied	Grout at Invert	H = 29 m	1.5

(3) Results of Analysis

The structural analysis for the concrete lining is conducted using the maximum stress resultants of the members for the above each case. The results of the analysis are summarized in the table below.

Result of Structural Analysis for Case 1

Item	Unit	Crown	Side Wall	Invert
Max. Bending Moment	tf·m	0.5	3.2	
Max. Axial Force	tf	27.0	29.8	
Max. Shear Force	tf	1.8	6.8	
Compressive Stress by Bending Moment (Con.)	kgf/cm^2	6.6	13.6	
Tensile Stress by Bending Moment (Con.)	kgf/cm^2	-	1.7	
Shear Stress (Con.)	kgf/cm^2	0.4	1.4	
Tensile Stress by Bending Moment (Bar)	kgf/cm^2	-	-	
Allowable Compressive Stress (Con.)	kgf/cm^2	70.0	70.0	70.0
Allowable Tensile Stress (Con.)	kgf/cm^2	2.6	2.6	2.6
Allowable Shear Stress (Con.)	kgf/cm^2	3.6	3.6	3.6
Allowable Tensile Stress (Bar)	kgf/cm^2	1,600.0	1,600.0	1,600.0

Steel Bar Arrangement for Crown, Side Wall, Invert : None

However, steel bars are provided for the Side Wall and Invert as precaution reinforcement.

Result of Structural Analysis for Case 2

Item	Unit	Crown	Side Wall	Invert
Max. Bending Moment	tf·m	1.2	6.5	6.8
Max. Axial Force	tf	61.0	61.3	61.8
Max. Shear Force	tf	4.0	13.7	14.7
Compressive Stress by Bending Moment (Con.)	kgf/cm^2	15.1	28.5	29.5
Tensile Stress by Bending Moment (Con.)	kgf/cm^2	-	-	-
Shear Stress (Con.)	kgf/cm^2	1.0	3.4	3.6
Tensile Stress by Bending Moment (Bar)	kgf/cm^2	-	29.4	19.4
Allowable Compressive Stress (Con.)	kgf/cm^2	70.0	70.0	70.0
Allowable Tensile Stress (Con.)	kgf/cm^2	2.6	2.6	2.6
Allowable Shear Stress (Con.)	kgf/cm^2	3.6	3.6	3.6
Allowable Tensile Stress (Bar)	kgf/cm^2	1,600.0	1,600.0	1,600.0

Steel Bar Arrangement for Crown : None
Steel Bar Arrangement for Side Wall : D 13 -- @ 200 mm
Steel Bar Arrangement for Invert : D 13 -- @ 200 mm

Result of Structural Analysis for Case 3,4 and 5

Item	Unit	Crown	Side Wall	Invert
Max. Bending Moment	tf·m	18.2	25.2	29.4
Max. Axial Force	tf	30.6	50.7	46.0
Max. Shear Force	tf	19.9	24.7	16.6
Compressive Stress by Bending Moment (Con.)	kgf/cm^2	28.9	25.2	48.1
Tensile Stress by Bending Moment (Con.)	kgf/cm^2	-	-	-
Shear Stress (Con.)	kgf/cm^2	2.2	2.7	1.8
Tensile Stress by Bending Moment (Bar)	kgf/cm^2	1,075	1,022	1,951
Allowable Compressive Stress (Con.)	kgf/cm^2	105.0	105.0	105.0
Allowable Tensile Stress (Con.)	kgf/cm^2	3.9	3.9	3.9
Allowable Shear Stress (Con.)	kgf/cm^2	5.4	5.4	5.4
Allowable Tensile Stress (Bar)	kgf/cm^2	2,100.0	2,100.0	2,100.0
Case		Case 3	Case 4	Case 5

Steel Bar Arrangement for Crown : D 13 -- @ 200 mm
Steel Bar Arrangement for Side Wall : D 13 -- @ 200 mm
Steel Bar Arrangement for Invert : D 13 -- @ 200 mm

Steel bar arrangement for both standard and plug sections are illustrated in Fig. 7.5.13.

7.5.4 Plug Works

Layout of plug works is shown in Fig. 7.5.14.

(1) Plug Method

In general, there are two (2) plug method for a diversion tunnel. The one is to demolish the lining concrete at the plug section and expose rock mass. Then plug concrete is placed at the plug section.

The other method is to place plug concrete at the plug section after chipping the surface of the lining concrete which is 50 cm thicker than the standard section.

The second method is applied for the diversion tunnel of the project from the reasons mentioned below.

- Since the rock mass is soft rock, the demolition of the lining concrete may damage the rock mass.
- Time for the demolition of the lining concrete can be saved.

(2) Location and Necessary Length of Plug Section

The location of the center line of the plug section is same as the dam axis to attain the continuous seepage protection effect by curtain grout.

As for the length of the plug section is decided in general about 0.5 to 0.6 times of the design head of 60 m. Then the plug length is decided at 35 m.

(3) Cooling Method of Plug Concrete

Since the plug concrete is mass concrete, cooling of the plug concrete is necessary. The cooling of the plug concrete is made by means of iced water which flow in cooling pipes provided in the plug concrete. The construction period of the diversion tunnel is limited to one dry season, therefore, the cooling pipes are designed to be densely provided to make the cooling period as short as possible.

The one lift of plug concrete is placed in 2.0 m and the cooling pipes are provided at each lift of concrete. The interval of the cooling pipes is decided at 1.5 m to attain the final temperature of 28 °C, which is equivalent to the one of the rock mass.

(4) Grouting around Plug Works

(a) Consolidation Grouting

The consolidation grout is provided to clog cracks or voids to lower permeability in the rock mass at the plug section. The dimensions of the consolidation grout are as follows:

- Length of Grout : L = 5.0 m
- Longitudinal Interval : L = 3.0 m
- Length of Grouting Section : L = 35.0 m (same as the plug section)
- Numbers at a section : n = 8 (45° each)

(b) Curtain Grouting

The curtain grout is provided to protect seepage along the diversion tunnel. The location of the curtain grout is the center of the plug section, which is same as the dam axis. The dimensions of the curtain grout are as follows.

- Length of Grout : L = 10.0 m
- Numbers of Grout : n = 18 (20° each)

7.5.5 Closure Gate

For closing the diversion tunnel and damming up the reservoir, steel made closure gate is provided as follows:

Type of Gate	:	Steel Made Closure Gate		
Number of Gate	:	1 set		
Design Water Level	:	EL. 120.000 m		
Sill Elevation	:	EL 98.500 m		
Lintel Seal Elevation	:	EL 104.150 m		
Clear Span	:	5.600 m		
Type of Hoist	:	Lowering by Truck Crane		
Corrosion Allowance	:	0.0 mm		
Allowable Stress				
For Steel	:	Tension , Compression 0.75 σ y as a basic shearing 0.75 σ y/$\sqrt{3}$		
For Concrete	:	Bearing	:	1.5×60 kgf/cm^2
	:	Shearing	:	1.5×4.0 kgf/cm^2
	:	Bond	:	1.5×7.0 kgf/cm^2

7.6 Outlet Facilities

7.6.1 Layout of Structures

Required Function

The outlet facilities are to assure the reservoir yield, which is required for municipal and industrial water supply to Semarang City and river maintenance flow to the area downstream of the dam. The maximum out flow discharge is 2.69 m^3/s, which corresponds to the required flow at the Simongan weir site in the present stage.

In the future, the deficit in the water supply from Jatibarang Reservoir would be supplemented by the proposed Mundingan Dam and Inter-basin water transfer project, which were proposed in the Master Plan. After the completion of the above two (2) facilities, it is expected that the maximum outflow will increase to 6.0 m^3/s. The outlet facilities are designed to accommodate the planed maximum out flow discharge of 6.0 m^3/s in the future stage.

The minimum outflow from the intake facilities is 0.26 m^3/s which corresponds to the maintenance flow between the damsite and the confluence with Garang River.

The hydropower is generated subordinately using the released water necessary for downstream water use.

Layout of Structures

The outlet facilities are located in the right bank of Jatibarang Multipurpose Dam The Plan and profile of the outlet facilities are shown in Figs. 7.6.1 and 7.6.2.

The intake structure is located at the right abutment upstream of the toe of the dam body. It is designed as the inclined type with 1.0 vertical to 1.4 horizontal with U-shaped and box-shaped cross sections as shown in Figs. 7.6.3 and 7.6.4. The bulkhead gate is operated when the outlet pipe is necessary to be drained for inspection, maintenance and repair without lowering the reservoir water surface. Th floor elevation of a bulkhead gate operation is set at EL. 157.000 m, which is the same as the dam crest elevation.

All excavated slopes around the inclined intake structure are protected with shotcrete. The slope has a 1.5 m wide berm at every 7.5 m high interval, and cut slope is 1.0 vertical to 0.8 horizontal. The thickness of shotcrete is 10 cm and a drain pipe of 5 cm in diameter is provided at every 2 m^2 area of the slope surface.

The outlet pipe is installed in the outlet tunnel located in the right abutment. A branched pipe is connected with the outlet pipe just before the hydropower station to release the water directly, without passing through the hydropower station.

A control gate installed at the downstream end of the outlet pipe regulates the flows. The minimum and maximum design discharges of 0.26 m^3/s and 6.0 m^3/s with the Low Water Surface (EL. 136.0 m) are considered. Since the discharge control of the gate at openings of less than 10 % is inferior in accuracy, the control gates of ϕ 0.65 m and ϕ 0.25 m in diameter with jet flow gate type are required. The layout of the control gate is shown in Fig. 7.6.5.

The elevations of the inlet and outlet are set at EL. 111.5 m and EL 84.7 m respectively considering the topographic condition. The longitudinal gradient becomes 1/14.65.

The geology of the layer in which the outlet tunnel passes is confirmed to be almost same as the one of the diversion tunnel with pyroclastic rock and sedimentary rock. The property of the geology is also same as the one of the diversion tunnel with unconfined compressive strength (qu) of 30 to 50 kgf/cm^2, which is classified as soft rock. (refer to Fig. 7.6.6)

7.6.2 Hydraulic Design

Diameter of Outlet Pipe

The outlet pipe is installed in the outlet tunnel located in the right abutment. A branched pipe is connected with the outlet pipe just before the hydropower station to release the water directly, without passing through the hydropower station.

The diameter of the outlet pipe is determined at $D = (4*Q/3.14/V)^{0.5} = 1.4$ m, using the condition of the maximum discharge $Q = 6.0\ m^3/s$ and the velocity $V = 4.0$ m/s.

Discharge through Control Gate

The discharge through the control gates of ϕ 0.65 m and ϕ 0.25 m in diameter are calculated using the following basic equation:

$$Q = C \cdot A \cdot \sqrt{2 \cdot g \cdot (H - \Sigma h_i)}$$

$$\Sigma h_i = \frac{Q^2}{2g} \cdot \Sigma \frac{f_i}{A_i^2}$$

Accordingly,

$$Q = \sqrt{\frac{2g \cdot C^2 \cdot A^2 \cdot H^2}{1 + C^2 \cdot A^2 \cdot \Sigma \frac{f_i}{A_i^2}}}$$

Where,

Q : discharge (m^3/sec)

A : area of clear opening of control gate (m^2)

C : discharge coefficient (refer to the table below)

Gate open percent (%)	Discharge Coefficient C
5	0.02
10	0.04
20	0.10
30	0.17
40	0.26
50	0.35
60	0.47
70	0.57
80	0.67
90	0.75
100	0.81

H : depth from reservoir water surface to control gate centerline (m)

Σh_i : total losses of head (m)

g : gravity acceleration (9.8 m/sec2)

f_i/A_i^2 : head loss coefficient divided by area related to loss (refer to Table 7.6.1 and Fig. 7.6 7)

When the reservoir is at the Low water Surface EL. 136.0 m, the discharge through each control gate is calculated as follows:

Jet Flow Gate	Gate Open	Item	Unit	Calculated Value
ϕ 650 mm	100 %	C	-	0.81
		A	m^2	0.332
		f_i/A_l	$1/m^4$	11.802
		H	m	50.1 (EL. 136.0 – EL. 85.9)
		Q	m^3/s	6.19
	10 %	C	-	0.04
		A	m^2	0.332
		f_i/A_l	$1/m^4$	11.802
		H	m	50.1 (EL. 136.0 – EL. 85.9)
		Q	m^3/s	0.42

Jet Flow Gate	Gate Open	Item	Unit	Calculated Value
ϕ 250 mm	100 %	C	-	0.81
		A	m^2	0.049
		f_i/A_i	$1/m^4$	660.795
		H	m	50.55 (EL. 136.0 – EL. 85.45)
		Q	m^3/s	0.87
	10 %	C	-	0.04
		A	m^2	0.049
		f_i/A_i	$1/m^4$	660.795
		H	m	50.55 (EL. 136.0 – EL. 85.45)
		Q	m^3/s	0.06

Relationship between discharge and gate opening height is calculated in Table 7.6.2 and discharge-rating curve is shown in Fig. 7.6.8.

7.6.3 Structural Design

Standard Section of Outlet Tunnel

(1) Tunnel Excavation Method

Since the condition and property of the rock mass of the outlet tunnel is almost same as the one of the diversion tunnel, the same excavation method as the diversion tunnel is adopted. Namely, New Austrian Tunneling Method (NATM) with smaller size tunneling machine is employed. Since the cross sectional area of the outlet tunnel is much smaller than that of the diversion tunnel, excavation is conducted in the full-face excavation method.

(2) Cross Section of Outlet Tunnel

(a) Type of Cross Section

From the same reason in the selection of the cross section type of the diversion tunnel, the horse shoe type is selected for the outlet tunnel.

(2) Dimensions of Cross Section

The dimensions of the cross section of the outlet tunnel are decided from the size of excavation equipment and the size of an outlet pipe with the diameter 1.4 m and its installation works.

The excavation of the tunnel will be executed by tunneling equipment. Considering the size of the tunneling equipment, the internal width of 2.2 m is necessary at the minimum.

On the other hand, the outlet tunnel contains a steel pipe line with a diameter of 1.40 m to introduce reservoir water to the hydropower station. The internal section of the outlet tunnel is decided to have enough working space for transporting, installation and welding the steel pipes. For working space, 0.5 m on both sides of the steel pipeline is taken, then the internal diameter of the tunnel becomes 2.4 m.

(3) Supporting System for Tunnel Excavation

NATM provides and rock bolts and shotcrete to the excavated face immediately after the completion of one cycle of excavation by tunneling machine to protect the rock mass from fall rocks and to strengthen week layer and furthermore expecting formation of arch action in the rock mass.

Standardized selection method for tunnel support categories is expressed in the Table 7.5.2. For the outlet tunnel, competence factor is relatively small because of the soft rock, The following works are planed as the supporting system for the outlet tunnel.

- Wire mesh	:	Steel mesh (5 mm in diameter, 150 mm x 150 mm) is installed on the excavated surface to reduce spring back of the shotcrete and to reinforce the shotcrete.
- Steel support	:	H steel with the size of 100 mm x 100 mm x 6.0 mm x 8 mm is applied for the steel support with the interval of 1.5 m taking the rock mass conditions into account.
- Shotcrete	:	Shotcrete with the thickness of 100 mm is sprayed after the installation of the steel support to protect the surface of the rock mass from collapse or relaxation.
- Rock bolts	:	Rock bolts with 22 mm in diameter are provided at the arch section with angle of 57 degrees. The length of the rock bolt is 1.5 m at the standard section, while the interval of the rock bolt is 1.5 m in longitudinal direction.

The area between the steel pipeline and the supporting system of shotcrete and steel support is filled with concrete after the completion of the installation of the pipeline.

Therefore, no concrete lining is provided.

(4) Typical Cross Section of Outlet Tunnel

The typical cross section of the outlet tunnel is shown in Fig. 7.6.9.

Structural Analysis of Tunnel before Concrete Filling

Structural analysis of the tunnel is conducted to confirm the design mentioned above by finite element method (FEM).

(1) Section for Analysis

The analysis is conducted for the plug section where the area of the excavation section is larger than the standard section.

(2) Conditions for Analysis

(a) Analytical Domain

The analytical domain is limited to 300 m wide and 170 m high as shown in Fig. 7.6.10 together with mesh. The boundary conditions of the domain are horizontally fixed, vertically movable for side boundary, and both horizontally and vertically fixed for bottom boundary.

(b) Properties of Material and Rock Mass

The properties of the supporting material are shown in the table below.

Structure	Modulus Elasticity (kgf/cm^2)	Sectional Area (cm^2)	Moment Inertia (cm^4)	Section Modulus (cm^3)	Remarks
Shotcrete (10 cm)	34,000	1,000	12,500	2,500	1.5 m wide
Steel Support (H 100 mm)	2,100,000	21.59	378	75.6	for 1 piece

The properties of the rock mass are shown in Fig. 7.6.11.

(c) Analytical Steps

In accordance with the construction procedure, the analysis is conducted with five (5) steps as shown in Fig. 7.6.12.

(3) Results of Analysis

The deformation map is shown in Fig. 7.6.13 as the result of the analysis. The settlement of the land surface above the tunnel is calculated at only 0.01 mm which is judged to be safe enough.

The deformation of the lining concrete is calculated as shown in the table below.

Location	Crown	Spring (Vertical)	Spring (Horizontal)	Invert
Deformation (cm)	0.2	0.1	0.1	0.3

The above figures of deformation of the supporting system are considered not to be harmful at any portions.

The active stresses to the shotcrete and steel support are calculated as shown in the table below.

Item	Unit	Crown	Spring	Invert
Max. Axial Force for Shotcrete	(tf)	38.5	19.0	19.3
Max. Bending Moment for Steel Support (H 100 mm)	(tf·m)	0.1	0.4	0.1

While the allowable compressive strength of the shotcrete and the allowable bending moment of the steel support are calculated as below.

(a) Allowable Compressive Strength of Shotcrete

Allowable compressive stress (f_{ck}) of the shotcrete is defined as f_{ck}= 60 kgf/cm^2.

Increment of allowable compressive stress for temporary works is 1.5.

Sectional area of the shotcrete (A) is 1,000 cm^2.

Allowable compressive strength of the shotcrete (N_r)

$N_r = f_{ck} \times A = 60 \times 1{,}000 \times 1.5 = 90{,}000$ kgf = 90 tf > 38.5 tf (crown)

(b) Allowable Bending Moment of Steel Support

Allowable compressive stress due to bending of steel support (H 100), (σ a)

σ a = 1,400 kgf/cm2

Increment of allowable compressive stress for temporary works is 1.5.

Section modulus (Z) = 75.6 cm^3

Allowable bending moment of the steel support (H 125), (Mr)

Mr = σ_a x Z x 1.5 =1,400 x 75.6 x 1.5 = 158,760 kgf·cm = 1.5 tf·m > 0.4 tf·m (Spring)

Therefore, the active stresses for both of the shotcrete and the steel support are smaller than the allowable stresses.

(c) Fracture Safety Factor (Point Safety Factor) and Maximum Shear Strain

Fracture safety factor (point safety factor) and maximum shear strain are calculated for the evaluation of safety of the rock mass due to the excavation of the tunnel.

As a result of the calculation, the contour line maps of fracture safety factor and maximum shear strain are shown in Figs. 7.6.14 and 7.6.15, respectively.

As for the fracture safety factor, the safety factors exceed 1.0 at any portions. The critical shear strain of the rock mass, which has 50 kg/cm^2 of unconfined compressive strength, is considered about 1.0 %. Since the maximum calculated shear strain of the rock mass is 0.5 %, the shear strain does not exceed the critical value at any portions.

7.6.4 Plug Works

Layout of plug works is shown in Fig. 7.6.16.

(1) Plug Method

As the area between the pipeline and the supporting system is filled with concrete for the whole length of the outlet tunnel, no independent plug concrete is provided. However, the thickness of the filling concrete is increased by 30 cm in radius of the tunnel at the plug section.

(2) Location and Necessary Length of Plug Section

The location of the center line of the plug section is same as the dam axis to attain the continuous seepage protection effect by curtain grout. The length of the plug section is 20 m.

(3) Grouting

(a) Consolidation Grouting

The consolidation grout is provided to clog cracks or voids in the rock mass at the plug section. The dimensions of the consolidation grout are as follows:

- Length of Grout : L = 3.0 m
- Longitudinal Interval : L = 3.0 m
- Length of Grouting Section : L = 20.0 m (same as the plug section)
- Numbers at a section : n = 8 (45° each)

(b) Curtain Grouting

The curtain grout is provided to protect seepage along the outlet tunnel. The location of the curtain grout is the center of the plug section, which is same as the dam axis. The dimensions of the curtain grout are as follows:

- Length of Grout : L = 10.0 m
- Numbers of Grout : n = 18 (20° each)

7.6.5 Mechanical Structures for Outlet Facilities

Mechanical structures for outlet facilities such as bulkhead gate, emergency gate, trash rack, steel conduit, guard gates and control gates are designed. The features clarified by the detailed design are summarized in the followings.

Bulkhead Gate

The bulkhead gate equipped at the inclined intake structure is operated when the steel conduit is necessary to be drained for inspection, maintenance and repair without lowering the reservoir water level.

Since the bulkhead gate is always operated under the balanced pressure condition. It can not be operated in any emergency case, which needs to close the steel conduit under moving water condition. However, special consideration should be taken to ensure the condition of pressure balance. For this purpose, the bulkhead gate leaf is provided with two (2) water filling valves of 15 cm in diameter.

Layout of the bulkhead gate is shown in Fig. 7.6.17 and the main features are described as follows:

Type of Gate	:	Steel Made Fixed Wheel Gate (with 2 Water Filling Valves)
Number of Gate	:	1 set
Design Water Level	:	NWL. 148.900 m + Wave Height 0.8 m
Sill Level	:	EL. 113.471 m
Lintel Seal Elevation	:	EL. 114.488 m
Clear Span	:	2.0 m
Clear Height	:	1.4 m
Gate Height	:	2.0 m
Type of Hoist	:	Electrically Driven 1-Motor, 1-Drum Wire Rope Hoist
Operating Speed	:	0.1 m/min ± 10 %
Raising Operation	:	Water Pressure Balance
Operation	:	Local Control
Power Source	:	380 V, 50 Hz, 3 ϕ, 4 W
Raising Height	:	68.0 m more
Seismic Intensity	:	k = 0.16

Emergency Gate

The emergency gate equipped at EL. 115.0 m of the inclined intake structure can be operated if the reservoir water level has to be drawn down to the lower elevation than the Low Water Surface EL. 136.000 m.

Since the emergency gate is always operated under the balanced pressure condition. It can not be operated in any emergency case, which needs to close the steel conduit under moving water condition.

Layout of the emergency gate is shown in Fig. 7.6.18 and the main features are described as follows:

Type of Gate	:	Steel Made Slide Gate
Number of Gate	:	1 set
Design Water Level	:	WL. 130.000 m + Wave Height 0.8 m
Sill Level	:	EL. 115.000 m
Lintel Seal Elevation	:	EL. 115.900 m
Clear Span	:	2.0 m
Clear Height	:	1.4 m

Gate Height	:	1.65 m
Type of Hoist	:	Electrically Driven 1-Motor, 2-Drum Wire Pope Hoist and Lifting Beam
Operating Speed	:	0.1 m/min ± 10%
Raising Operation	:	Water Pressure Balance (WL. 136.0 m - WL. 131.0 m)
Operation	:	Local Control
Power Source	:	380 V, 50 Hz, 3 ϕ , 4 W
Raising Height	:	2.0 m (Normal), 67.0 m more (Maintenance)
Seismic Intensity	:	k = 0.16

Trash Rack

The trash rack consisting of steel bar elements and supports is provided at the upstream face of the inclined intake structure. It can catch debris and rubbish so as to prevent them from flowing into the steel conduit and to damage the hydropower equipment and control gates.

The main features are described as follows:

(1) For Bulkhead Gate

Type	:	Steel Made Fixed Trash Rack
Quantity	:	1 set
Top of Trash Rack	:	EL. 156.400 m
Bottom of Trash Rack	:	EL. 130.000 m
Breadth	:	2.0 m
Inclination Angle	:	1 : 1.4
Bar Thickness	:	12.0 mm (include corrosion allowance)
Bar Breadth	:	65.0 mm
Design Pressure	:	0.2 kgf/cm^2

(2) For Emergency Gate

Type	:	Steel Made Fixed Trash Rack
Quantity	:	1 set
Top of Trash Rack	:	EL. 116.476 m
Bottom of Trash Rack	:	EL. 115.000 m
Breadth	:	3.0 m
Depth	:	0.6 m
Height	:	2.54 m
Inclination Angle	:	1 : 1.4

Bar Thickness	:	9.0 mm (include corrosion allowance)
Bar Breadth	:	90.0 mm
Design Pressure	:	0.2 kgf/cm^2

Steel Conduit

The steel conduit for outlet pipe is installed inside the tunnel located in the right abutment. After installation of the steel conduit, the space between excavated rock surface and steel conduit is filled with concrete. A branched pipe is connected with the steel conduit just before the hydropower station to release the water directly without passing through the hydropower station.

The main features are described as follows:

Number of Lanes	:	1 set
Transition Pipe	:	□ 1.4 m-○ 1.4 m
Outlet Pipe	:	ϕ 1.4 m, ϕ 0.65 m, ϕ 0.25 m
Length of Transition Pipe	:	1.15 m
Length of Outlet Pipe	:	ϕ 1.4 m - 399.511 m (Transition Pipe-Branch)
		ϕ 0.65 m - 16.210 m (Branch-Guard Gate)
		ϕ 0.25 m - 9.056 m (Branch-Guard Gate)
Design Internal Pressure	:	MWL. 155.500 m + Wave Height 0.8 m
		SWL. 151.800 m + Water Hummer
Design External Pressure	:	NWL. 148.9 m + Wave Height 0.8 m (Intake-Dam Axis)
		Grouting Pressure 2.0 kgf/cm2
Safety Factor for Buckling	:	1.5

Control Gate

The control gate installed at the downstream end of the branched outlet pipe regulates the discharge required, when hydropower generation is not in operation. The required discharge varies from the minimum amount of 0.26 m^3/s up to the maximum amount of 6.0 m^3/s with the Low Water Surface (EL. 136.0 m). The control gate should be satisfactorily operated at all openings without vibration or serious cavitation damage.

The main features are described as follows:

(1) ϕ 0.65 m Control Gate

Type of Gate	:	Jet Flow Gate (ϕ 0.65 m)

Number of Gate	:	1 set
Design Water Level	:	SWL. 151.800 m + Water hummer
Operating Water Level	:	MWL. 155.300 m + Wave Height 0.8 m
Elevation of Orifice Center	:	EL. 85.900 m
Type of Hoist	:	Electrically Driven Screw Spindle Type
Operating Speed	:	0.1 m/min ± 10 %
Operation	:	Local Control
Power Source	:	380 V, 50 Hz, 3 ϕ, 4 W

(2) ϕ 0.25 m Control Gate

Type of Gate	:	Jet Flow Gate (ϕ 0.25 m)
Number of Gate	:	1 set
Design Water Level	:	SWL. 151.800 m + Water Hummer
Operating Water Level	:	MWL. 155.300 m + Wave Height 0.8 m
Elevation of Orifice Center	:	EL. 85.450 m
Type of Hoist	:	Electrically Driven Screw Spindle Type
Operating Speed	:	0.1 m/min ± 10 %
Operation	:	Local Control
Power Source	:	380 V, 50 Hz, 3 ϕ, 4 W

Guard Gate

The guard gate is installed just upstream of the control gate for the purpose of emergency use. The gate is always kept at its fully opened position and operated to shut off the flow of water in case of repair of the control gate. This gate is usually operated under balanced pressure/no-flow condition, except for closure in emergencies.

The main features are described as follows:

(1) ϕ 0.65 m Guard Gate

Type of Gate	:	High Pressure Slide Gate (ϕ 0.65 m)
Number of Gate	:	1 set
Design Water Level	:	SWL. 151.800 m + Water Hummer
Operating Water Level		Raising : Water Pressure Balance
		Lowering : MWL. 155.300 m + Wave Height 0.8 m
Elevation of Center	:	EL. 85.900 m
Type of Hoist	:	Electrically Driven Screw Spindle Type

Operating Seed	:	0.1 m/min ± 10 %
Operation	:	Local Control
Power Source	:	380 V, 50 Hz, 3 ϕ , 4 W

(2) ϕ 0.25 m Guard Gate

Type of Gate	:	High Pressure Slide Gate (ϕ 0.25 m)
Number of Gate	:	1 set
Design Water Level	:	SWL. 151.800 m + Water Hummer
Operating Water Level		Raising : Water Pressure Balance
		Lowering : MWL. 155.300 m + Wave Height 0.8 m
Elevation of Center	:	EL. 85.450 m
Type of Hoist	:	Electrically Driven Screw Spindle Type
Operating Seed	:	0.1 m/min ± 10 %
Operation	:	Local Control
Power Source	:	380 V, 50 Hz, 3 ϕ , 4 W

7.7 Hydropower Generation

7.7.1 Detailed Design of Main Facilities

Layout of plant and structure of the powerhouse are shown in Figs. 7.7.1 to 7.7.11.

Features of Structures

(1) Intake : Intake is used in common both for power generation purpose and water utilization purpose

(2) Penstock

- Type	:	Embedded steel penstock
- Quantity	:	1 Lane
- Total length	:	4.900 m (CL)
- Inside diameter of pipe	:	ϕ 1,400 ~ ϕ 800 mm
- Design internal pressure		
• Maximum static head	:	71 m
• Maximum water hammer	:	30 m
• Maximum design pressure	:	101 m
• Maximum discharge	:	3.0 m^3/sec

• Minimum shell thickness	:	9 mm
• Material	:	Rolled steel for welded structure
• Corrosion allowance	:	1.5 mm

(3) Powerhouse

- Location	:	Direct downstream of dam toe
- Type	:	Semi-underground type, Reinforced concrete structure
- Dimensions of generating room		
• Width × Length	:	9.0 m × 15.0 m
• Floor elevation	:	EL. 84.9 m
• Height	:	EL. 97.5 - 84.9 = 12.6 m

(4) Afterbay

- Width × Height	:	5.1 m × 3.1 m
- Total length	:	4.25 m
- Sill elevation	:	EL. 79.7 m

(5) Tailrace Channel

- Width × Height	:	2.0 m × 2.0 m
- Total length	:	50.96 m
- Sill elevation	:	EL. 82.3 m

(6) Gate

- Type	:	Steel Slide Gate
- Quantity	:	1 set
- Clear span	:	2.0 m
- Clear height	:	2.0 m
- Design water level	:	EL. 87.742 m
- Sill elevation	:	EL. 82.500 m
- Design head	:	5.242 m
- Sealing method	:	4 edges rubber seal at the upstream face of the gate (turbine side)
- Corrosion allowance	:	0 mm
- Deflection main horizontal beam	:	Less than 1/600 of supporting span
- Hoisting method	:	Manual operation type hoisting system

Generating Equipment

(1) Connection Diagram

The single line diagram of the Jatibarang hydropower station is shown in Fig. 7.7.12.

(2) Layout of Powerhouse

General plan of machine room is shown in Fig. 7.7.13. Detail of layout shall be modified according to the dimension of actual equipment. Longitudinal section and Latitudinal section are shown in Fig. 7.7.14.

(3) Hydraulic Turbine

(a) Waterway System Design Data

Basic data used for design of waterway system of the Jatibarang Hydropower Station are presented below, as determined in Clause 6.3.3.

(i) Reservoir Water Level

High water level	EL. 151.600 m
Design water level	EL. 148.900 m
Low water level	EL. 138.000 m

(ii) Tailrace Water Level

Normal water level (at 3.0 m3/s) EL.82.910 m

(iii) Static Head

Maximum gross head	68.69 m (= 151.600 - 82.910)
Design gross head	65.990 m (= 148.900 - 82.910)
Minimum gross head	55.090 m (= 138.000 - 82.910)

(iv) Waterway Head Loss 1.690 m

(v) Net Head (Effective Head)

Maximum net head	67.000 m (= 68.690 - 1.690)
Design net head	64.300 m (= 65.990 - 1.690)
Minimum net head	53.570 m (= 55.090 - 1.520)

(b) Type of Hydroturbine

The horizontal shaft type Francis turbine is applied. (refer to 6.6.3 (1)(a))

(c) Turbine Output

The number of unit is selected to be one (1) unit.

The turbine output (Pt) is calculated by the following formula:

$$\text{Turbineoutput}\,\text{Pt}\,(\text{kW}) = g \times Q \times H_e \times \eta_t = 9.8 \times 3.0 \times 64.3 \times 0.658$$
$$\approx 1{,}630\,(\text{kW})$$

Where,

g	:	gravitational acceleration constant = 9.8 (m /s²)
Q	:	rated turbine discharge (m³/s) = 3.0 (m³/s)
He	:	rated head (m) = 64.3 (m)
ηt	:	turbine efficiency assumed as 85.8 %

(d) Rated Speed and Specific Speed

(i) Relations between Rotational Speed (n) and Specific Speed (ns)

$$n = \frac{n_s \times H_d^{\frac{5}{4}}}{P_t^{\frac{1}{2}}}$$

Where,

n	:	rotational speed (rpm)
ns	:	specific speed (m-kW)
Hd	:	design head (m)
Pt	:	turbine output (kW)

(ii) Limit specific speed (nslim) and Trial rotational speed (n')

As for the limit specific speed (nslim) of Francis turbine, the following formula is recommended by The Japanese Electrotechnical Committee (JEC) and its rough estimation is given and shown in Fig. 7.7.15.

nslim = {21,000/(Hd+25)} + 35 = 269.1 (m-kW)

From the above limit specific speed (nslim), the limit of trial rotational speed (n') is calculated by the following formula:

$$n' = \frac{n_{s\lim} \times H_d^{\frac{5}{4}}}{P_t^{\frac{1}{2}}} = 1{,}214(rpm)$$

Where,

n'	:	trial rotational speed (rpm)
nslim	:	limit specific speed = 269.1 (m-kW)
Hd	:	design head = 64.3 (m)
Pt	:	turbine output = 1,630 (kW)

(iii) Applicable rotational speed

The relationship among the number of poles of the generator, frequency of the power system and applicable rotational speed (na) of the machine is shown below.

$$n_a = \frac{120 \times f}{P} \leq n' = 1{,}214 \text{ (rpm)}$$

Where,

na	:	applicable rotational speed can be selected from three types = 1000 : 750 : 600 (rpm)
f	:	frequency = 50 (Hz)
p	:	number of poles = 6 : 8 : 10

(iv) Rated rotational speed

Generally, the turbine becomes compact as the above applicable rotational speed that is derived from limit specific speed (nslim) is higher. However, consideration of cost performance such as turbine efficiency, flywheel-effect, turbine setting level and maintainability for machines, the rated rotational speed (n) of the hydroturbine for Jatibarang Hydropower Station is selected at 750 rpm.

(v) Design specific speed

Consequently, the design specific speed (ns) is calculated as shown below:

$$n_s = \frac{n \times H_d^{\frac{5}{4}}}{P_t^{\frac{1}{2}}} = 165.9$$

Where,

ns	:	design specific speed (m-kW)
n	:	rated rotational speed = 750 (rpm)
Hd	:	design head = 64.3 (m)
Pt	:	rated turbine output = 1,630 (kW)

(e) Stability Condition of Hydroturbine

Condition: TM > 2TW, Result : 4.26 > 3.50

Where,

TM	:	time of accelerated rotor speed = (π n0 / 60) 2 (GD2 / P0) = 4.26 (sec)
TW	:	time of accelerated water column velocity = L Q0 / g f H0 = 1.32 (sec)
n0	:	rated rotation of hydroturbine = 750 (rpm)
GD2	:	fly-wheel effect = 4.5 (ton-m^2)
P0	:	power Output = 1,630 (kW)
L	:	penstock length = 428.1 (m)
Q0	:	discharge = 3.0 (m^3/s)
G	:	gravity acceleration = 9.8
F	:	area of penstock section = 1.54 (m^2)
H	:	design head = 64.3 (m)

(f) Turbine Setting Level

(i) General Description

The center line level of the shaft of the horizontal turbine is to be decided properly on the basis of various conditions such as the mechanical characteristics of hydraulic runner, water level at the tailrace (TWL), the type of rock foundation at the site and the building layout.

Another matter to be taken into account is cavitation that occurs mainly on the rear surface of the runner blades. It is also outbroken on various parts of the runner such as the band, crown, discharge ring and others besides the draft tube inlet.

The countermeasures to prevent cavitation are as follows:

- Properly designed runner contour, and
- Runner setting at a level beneath the water discharge level with adequate submergence (H_S). The relations between turbine setting level and static draft head is illustrated in Fig. 7.7.16 (1/2)

(ii) Cavitation Coefficient

The index expressing the degree of cavitation induced is given by Prof. Thoma as the cavitation coefficient (σp) which is expressed and calculated as follows:

$$\sigma p = \frac{H_a - H_v - H_s}{H_e}$$

$$= 0.109$$

Where,

σp	:	plant cavitation coefficient
H_e	:	effective head = 64.3 (m)
H_a	:	atmospheric pressure = 10.248 (m Aq.)
H_v	:	vapor pressure at the water temperature = 0.316 (m Aq.)
H_s	:	static draft head (where the most probable location of the lowest pressure on the runner located) subtracted by the elevation of the discharge outlet water depth = 2.900 (m)

(iii) Recommended cavitation coefficient (σr)

The recommended cavitation coefficient (σr) is calculated by the formula of the Kansai Electric Power Company Inc.(KEPCO) as shown below (refer to Fig. 7.7.16 (2/2)).

σr = ans2 - bns + c = 0.109

Where,

a = 0.000004, b = 0.0003, c = 0.0488

Since the calculated plant cavitation coefficient (σp) of Jatibarang Hydropower Station is the same value as one of the recommended cavitation coefficients (σr), it means that the designed turbine setting level is suitable.

(g) Runner

The runner is advisable to be made of stainless steel contained with 13% chromium and more than 4% nickel and cast in one piece to prevent runner from cavitation damages. The characteristics of turbine efficiency are recommended that the efficiency under partial load is high and the lower limit of operational discharge is small.

(h) Inlet Valve

In general inlet valves, such as the butterfly valve, biplane valve, sluice valve, needle valve or spherical valve are used for the purpose of hydraulic turbine gate. Among these, the butterfly valve and biplane valve are used for a small size power station with a low head and small discharge of water flow. For the Jatibarang Hydropower Station these are suitable valves.

(4) Generator

(a) Type and Rated Capacity

One (1) set of generator will be installed in the powerhouse and coupled directly to the hydraulic turbine. The type of generator is horizontal shaft, 3-phase alternating current synchronous generator with a salient pole revolving field.

The parameters of generator are as follows;

(i) Power Factor of Generator

The rated power factor of the generator shall be decided considering the requirements of the power system in which a generator is required to be operated. The rated power factor of a generator that is connected to a transmission line of lower voltage, is usually 80% to 90% lagging. The rated power factor of the generator of Jatibarang Hydropower Station shall be 80% lagging.

(ii) Rated Capacity of Generator

Rated capacity of generator is obtained as shown below

$$
\begin{aligned}
Pg &= Pt \times \eta g / Pf = 1{,}630 \times 0.951 / 0.8 \\
&= 1{,}938 \text{ (kVA)} \\
&\fallingdotseq 2{,}000 \text{ (kVA)}
\end{aligned}
$$

Where,

Pg : rated generator output (kVA)

Pt : rated turbine output = 1,630 (kW)

ηg : generator efficiency assumed as 95.1 %

Pf : rated power factor taken as 80 % lagging.

(b) Type of Generator

Horizontal shaft type generator is selected to be coupled with hydroturbine directly.

(c) Excitation System

The brushless exciting system is adopted to the Jatibarang Hydropower Station. The system has excellent maintainability.

(d) Main Circuit and Station Service Circuit

(i) The generator is connected to the main transformer by the main generator circuit via a generator circuit breaker and current transformers. The neutral point of the generator winding will be grounded through current transformers and a grounding resistor. Station service supply circuits will be branched off from the main generator circuit. Single line connection diagram of powerhouse for main circuits and station services are shown in Fig. 7.7.12.

(ii) The generator voltage (6.6 kV) will be stepped up to the transmission voltage (20 kV) through the main transformer.

(iii) High voltage terminals of the main transformer will be connected to the outdoor 20 kV switchgear via 20 kV power cables.

(5) Control System of Hydropower Station (including outdoor substation)

(a) Operation and Control at Powerhouse

The following remote control is available by means of a control panel (cubicle or desk) installed in the control room of the powerhouse.

(i) The generator unit operation and control is carried out by one operator, so-called "One man control". Unit start/stop, excitation, parallel in and load control are conducted by operating a main control switch (No.1).

(ii) The turbine power control is possible by operating a control switch (No.7-65), but the automatic turbine power control in proportion to a dam water level is not available.

(iii) The unit voltage control is performed by operating a control switch (No.7-90).

(iv) If faults occur in a running generator unit, the unit is brought to either "emergency stop (No.86-1)", "quick stop (No.86-2)"or "normal fault stop (No.86-5)" automatically.

(v) The unit operating condition and a kind of faults are indicated immediately by announciators on the control panel.

(vi) The substation equipment control, protection and supervision is performed both in manually and automatically.

(vii) Discharge from the turbine is calculated as a parameter of the output of the generator unit and the dam water level that is communicated from the dam control system.

(6) Substation

(a) Configuration of Substation

(i) The substation is constructed outdoor nearby the powerhouse. Regarding the single line connection diagram of substation is referred to Fig. 7.7.12.

(ii) 20 kV switchgears and outdoor equipment shall be of the standard manufacture widely applied in the PLN existing substations. The layout of the substation is referred to Fig. 7.7.17.

(iii) The 6.6/20 kV main transformer, whose capacity is 2,000 kVA, is installed at the substation and the transformer is connected to the 20 kV transmission line through the 20 kV switchgear.

(vi) The primary circuit of a transformer is connected to the generator through the 6.6 kV power cable from the powerhouse.

(v) The secondary circuit of a transformer is connected to the overhead transmission line through the 20 kV switchgear and power cable.

(vi) The control, protection and supervision for substation equipment are carried out by a control panel installed at the control room of the powerhouse and each metalclad swichgears.

(vii) The amount of generated electricity and consumed electricity of the Hydropower Station is measured by means of Metering Out Fit (MOF) installed in the substation.

(b) Main Transformer

Type	:	three phase, oil immersed (ONAN), outdoor
Frequency	:	50 Hz
Capacity (Continues rating)	:	2,000 kVA (refer to generator capacity for estimation)
Rated voltage		
- Primary	:	6.6 kV
- Secondary	:	20 kV
Tap changer	:	Off load - tap changer
- Volt range	:	22 - 21 - 20 (R) - 19 - 18 (kV)
- Tap position	:	5
Winding connection		
- Primary	:	Delta
- Secondary	:	Star
- Vector group	:	Ynd5

7.7.2 Preliminary Design of Transmission Line

Plan of Transmission Line Connection

In principle, the Jatibarang Hydropower Station will be connected directly to the Krapyak Substation by a new 20 kV transmission line. The connection diagram of transmission line is referred to Fig. 7.7.18.

The distribution line is to supply electricity for general users at stable voltage and frequency. Generally, however, it may have more frequent shutdown of electricity supply due to its complicated network as compared to the transmission line.

The transmission lines for power stations should deliver the generated power to the transmission grid constantly. On the other hand, any transmission line fault, if happened, would cause remarkable rise in the voltage and frequency. To keep a distribution line with stable voltage and frequency, a transmission line should not be connected directly to a distribution line.

Therefore, the transmission line from Jatibarang Hydropower Station should be connected to Krapyak substation directly.

Transmission Facilities

(1) Conductor

All Aluminum Alloy Cord (AAAC) 120 mm^2 (standard conductor of PLN) is used for the 20 kV transmission line. The limit current of the conductor is about 300 A. The current of the transmission line is calculated as follows:

$$I = \frac{P/\sqrt{3}}{E} = \frac{2{,}000/\sqrt{3}}{20} = 57.7(A)$$

Where,

I	:	transmission current	= 57.7 (A)
P	:	transmission power	= 2,000 (kVA)
E	:	transmission voltage	= 20 (kV)

(2) Supporting Structures

(a) 20 kV Transmission lines shall be of the standard manufacture widely applied to the existing PLN distribution lines. Standard assembly of supporting structures for transmission lines is referred to Fig. 7.7.19.

(b) The great part of sections of new transmission line will be constructed by single post of concrete pole parallel to the existing distribution line.

(c) Fore the heavy duty part such as the crossing of a river or deep valley, steel towers or H type posts are used. The assembly diagrams of these structures are shown in Figs. 7.7.20 and 7.7.21.

(d) The line from the Jatibarang Hydropower Station (No. 1) to the Natural Park Gate to Goa Kreo (No. 37) will be placed along the new road to be constructed for this project. The entire route of this transmission line is shown in Fig. 7.7.22.

7.8 Access Road

7.8.1 General

The purposes of the access roads are to provide the better function to the damsite than the existing narrow roads and to manage the new facilities developed by the Jatibarang Multipurpose Dam Construction Project.

The following access roads are designed:

Access Road	Start Point	End Point	Accumulative Distance
Left Bank Access Road	Dam Left Abutment EL. 157.000 m	Left Side Public Road EL. 184.349 m	858.0 m
Right Bank Access Road	Dam Right Abutment EL. 157.000 m	Right Side Public Road EL. 158.823 m	1,688.1 m
Access Road to Hydropower Station	Dam Right Abutment EL. 157.000 m	Hydropower Station EL. 97.000 m	656.5 m
Access Road to Intake Structures	Dam Right Abutment EL. 157.000 m	Inclined Intake Structure EL. 157.000 m	207.1 m

The general plan of the access roads is shown in Fig. 7.8.1.

7.8.2 Geometric Design

The design of the access road is made according to the standard specification for geometric design of rural highway No. 13/1970, Directorate Generals of Bina Marga.

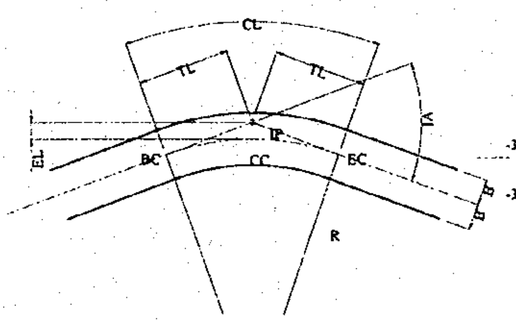
(1) Basic Design Criteria

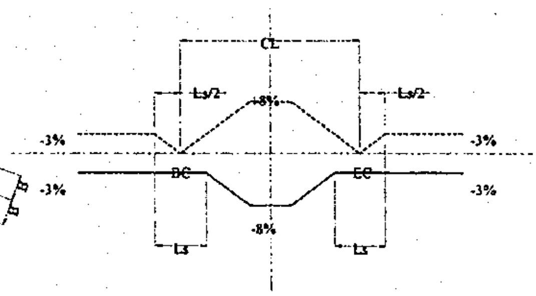
Description	Characteristic of Access Road	
Average traffic intensity per day (LHR)	500<LHR<1,500	< 500
Design speed	30 km/h	20 km/h
Road width - Pavement - Shoulder	 4.0 m 0.5 m	 4.0 m 0.5 m
Maximum longitudinal gradient	11.0 %	12.0 %
Sight distance	30.0 m	20.0 m
Desirable radius of curvature	65.0 m	30.0 m
Minimum radius of curvature	30.0 m	15.0 m
Minimum length of curvature	350/(inter angle) or 40.0m	280/(inter angle) or 40.0m
Superelevation - Pavement · Normal · Maximum - Shoulder - Run-off ratio	 3.0 % 8.0 % 6.0 % Less than 1/75	 3.0 % 8.0 % 6.0 % Less than 1/50
Type of loading should be in accordance with Guide Line for Highway and Bridge Design Loads, SNI-1725-1989		

The following design speeds are applied:

Access Road	Design Speed (km/h)
Left Bank Access Road	30.0
Right Bank Access Road	30.0
Access Road to Hydropower Station	20.0
Access Road to Intake Structure	20.0

(2) Horizontal Alignment





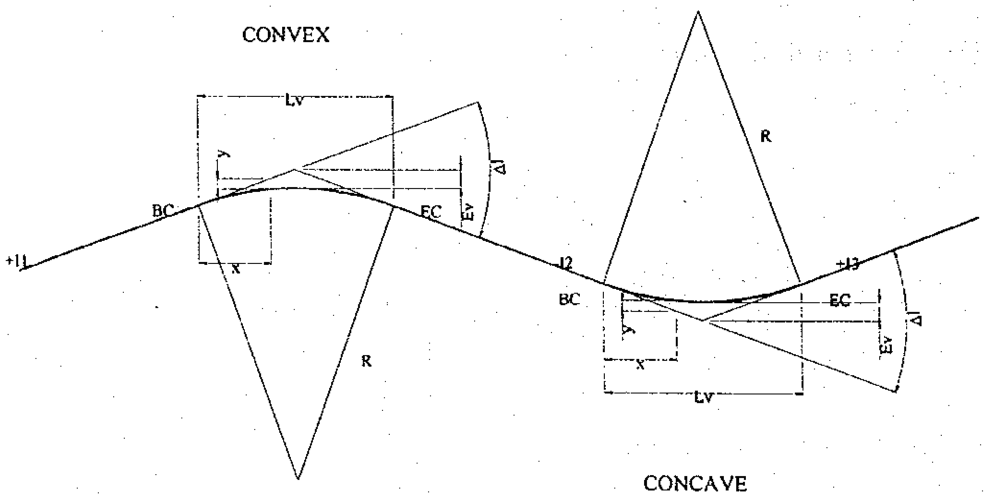
Where,

IP	:	intersection point
IA	:	intersection angle (degree)
R	:	radius of curvature (m)
CL	:	length of curve (m)
BC	:	beginning of curve
CC	:	center of curve
EC	:	end of curve
TL	:	distance between IP and BC, and IP and EC (m)
EL	:	distance between IP and curve (m)
Ls	:	superelevation run-off length = $(B \cdot \Delta I)/q$
B	:	half of road width (m)
ΔI	:	difference between superelevation (%)
q	:	superelevation run-off ratio (%)

Superelevation for each Radius of Curvature

Design Speed 30 km/h		Design Speed 20 km/h	
Radius of Curvature (m)	Maximum Superelevation (%)	Radius of Curvature (m)	Maximum Superelevation (%)
220 – 500	3.0	100 – 200	3.0
150 – 220	3.0	70 – 100	3.0
110 – 150	4.0	50 – 70	4.0
80 – 110	5.0	40 – 50	5.0
60 – 80	6.0	30 – 40	6.0
40 – 60	7.0	20 – 30	7.0
30 – 40	8.0	15 – 20	8.0

(3) Vertical alignment



Where,

BC : beginning of curve

EC : end of curve

I1, I2, I3 : longitudinal gradient (%)

R : radius of curvature (m)

ΔI : difference between longitudinal gradients (%)

Lv : length of vertical convex or concave (m)

Formula for vertical shifting is given as follows:

$$E_V = \frac{\Delta i \cdot L_V}{800}$$

$$y = \left(\frac{x}{\frac{1}{2}L_V}\right)^2 \cdot E_V = \frac{\Delta I}{200 \cdot L_V} \cdot x^2$$

Where,

E_V = vertical shifting (m)

x = distance (m)

y = vertical sifting (m)

The length of vertical Convex and Concave is as follows:

Difference between Gradients (ΔI)	Design Speed 30 km/h		Design Speed 20 km/h	
	Lv (m)	Vertical Curve Length R (m)	Lv (m)	Vertical Curve Length R (m)
1	25	2,500	20	2,000
2	25	1,250	20	1,000
3	25	830	20	670
4	25	630	20	500
5	25	500	20	400
6	25	420	20	330
7	25	360	20	290
8	25	310	20	250
9	25	280	20	220
10	25	250	20	200
11	28	250	20	180
12	30	250	20	170
13	33	250	20	150
14	35	250	20	140

7.8.3 Cross Section

(1) Typical Cross Section

The typical cross sections for cut and embankment type are shown in Fig. 7.8.2.

Design standard of embankment layer, drainage and slope protection is explained hereinafter.

(2) Design of Layer

(a) Sub-grade

The road will be made by spreading out on the filling of rock (or earth) or the cutting of rock (or earth). The filling as sub-grade should be compacted until 98% of the maximum dry compaction according to the AASHO Design Standard. The minimum thickness is 1.0 meter.

(b) Sub-base Course

The gradation of the sub-base course is to be decided in accordance with the AASHO Design Standard T.96, where a part of material passing through No. 200 sieve should be less than 2/3 of material passing through No. 40 sieve. The maximum percentage of material retained on No. 8 sieve is 50 %. Thickness of Sub-Base Course are as follows:

In Case of Cut Sub-Grade		In Case of Fill Sub-Grade	
Thickness	Sub-base Material	Thickness	Sub-base Material
0.3 m	Crushed Stone	0.4 m	Crushed Stone

Note : Fill sub-grade is constructed using selected common and unsound rock materials extracted from the required excavation nearby the road construction areas.

(c) Base Course

The construction can be made of dry bound macadam or water bound macadam, which is consisting of gravel or crushed stone bounded with fine aggregate as one or more layers on the sub-grade or sub-base course. The gradation of these layers is decided in accordance with the AASHO Design Standard T.96.

(d) Surface Course (pavement)

The following pavement design standard is applied; "Guide for Flexible Pavement Design (Petunjuk Perencanaan Tebel Perkerasan lentur Jalan Raya dengan metode analisa komponen : SKBI-2.3.26.1987, UDC : 625.73(02), Bina Marga).

(3) Drainage Design Standard

Drainage facilities design is based on rainfall intensity in a year return period as stipulated in Bina Marga Standard (No. 008/T/BNKT/1190, Petunjuk Desain Drainase Permukaan Jalan)

To determine the flow in the road drainage facilities the modified rational formula is used and the dimensions of the road drainage facilities are determined using Manning's Formula.

(4) Slope Protection

Slope protection work is planned to protect slope surface from erosion and weathering and to stabilize it by covering it with vegetation or structure. An optimum method should be selected in consideration of topographical, geological and meteorological conditions.

Slope protection work using vegetation is a sod facing method, while the slope protection works by structure are stone masonry, block masonry work, concrete leaning wall works and shotcrete.

7.9 Building Works

7.9.1 General

For the operation, maintenance and monitoring of Jatibarang Multipurpose Dam, operation/management office is planned at the damsite.

This section covers design of proposed buildings and houses involved in the project, such as dam administration building, staff houses, mushola for the personals in charge of dam operation and maintenance, hydropower station building, garage and guard house.

These buildings and houses classified into two (2) groups and their component are shown bellow:

Dam Management Complex

Components of buildings/houses	Type/Size	Unit
- Dam Administration Building	3 story; total area ± 594.010 m^2 RC frame, brick wall and tile roof	1
- Staff House 1 (Guest House)	1 story; total area ± 74.416 m^2 Brick wall and tile roof	1
- Staff House 2	1 story; total area ± 49.110 m^2 Brick wall and tile roof	4
- Mushola	1 story; total area ± 72.300 m^2 Brick wall and tile roof	1

Hydropower Station Complex

Components of buildings/house	Type/Size	Unit
- Hydropower Station Building	2 story; total area ± 389.640 m^2 RC frame, brick wall and steel roof	1
- Garage	1 story; total area ± 183.600 m^2 RC Frame, brick wall and steel roof	1
- Guard House	1 story; total area ± 14.275 m^2 Brick wall and slab roof	1

7.9.2 Dam Management Complex

Dam Management complex will be constructed on the excavated right spillway ridge area as a main facilities for the personals in charge of operation and maintenance of Jatibarang Multipurpose Dam including hydropower generation. The task and activities to be undertaken in these buildings are presumed as follows:

- Operation and maintenance of dam and its appurtenant structures/facilities;
- Maintenance of dam reservoir;
- Flood observation and warning for dam operation; and
- Observation and monitoring on seepage, deformation, etc., for a safety of dam after the construction.

Site plan and floor area table for the dam management complex are shown in Figs. 7.9.1 and 7.9.2.

Room Layout

(1) Dam Administration Building

Dam administration building is single building with total covered area amount 594.010 m^2 and its layout is made based on the manning arrangement and the space requirement of dam control equipment for execution of the above-mentioned task and activities.

Organizational structure for this building composed of for (4) sections; namely manager office, operation section, maintenance section and administration section.

Based on this organizational structure, the office spaces are so arranged to realize efficiency and smooth flow of the office activities and to meet with the security requirements for the building facilities and working personal. The elevation and floor plan are shown in Figs. 7.9.3 and 7.9.4.

(2) Staff House 1 (Guest House)

This facility is constructed to accommodate as a temporary staying space, for administration office worker or guest. This building can be use for 2 - 4 peoples by using two rooms that provided. Total covered area is amount 74.416 m^2, which have 6 rooms arrangement shown as follow:

- Two (2) Bed Room
- One (1) Living Room
- One (1) Dinning Room
- One (1) Pantry
- Two (2) Toilet

(3) Staff House 2

Four (4) permanent living quarters are proposed on the excavated hill top at right wing of dam area, due to accommodate the personals of the dam administration building with their living spaces. Those buildings completed with all convenience facilities for living those personals and his family, such as toilet, bath, kitchen etc. Each of those buildings is covered amount 49.11 m^2, which arranged face to face along the asphalt pavement.

- One (1) Living Room

- One (1) Dinning Room
- Two (2) Bed Room
- One (1) Kitchen
- One (1) Toilet

(4) Mushola

Mushola is using standard layout that usually applied to ordinary mushola/mosque, structures as follow: one big room as a praying room arranged between two washing rooms and toilet, support by entrance terrace and one small storage at behind.

Structural Design

(1) Administration Building

Dam administration building as the main building in the complex is single mass with tree (3) story. Ground floor dimension is 14.0 x 20.0 m plus entrance canopy 7.0 x 4.0 m, second floor dimension 14.0 x 20.0 m and third floor as a Watching room 13.0 x 4.0 m.

The building is made of reinforcement concrete framing and floor slabs. Standard span is 6.0 m long, and secondary beam is providing where necessary. For floor 120-mm, slab according to the floor load. All exterior and interior wall is used brick wall, with little variation of terracotta and wood materials.

Roof of the building is used tile roof with steel truss support. These steel frames are using angular bars and their connections are using nuts-bolts and by welding system. The tile is using Ceramic roof tile that specified as referenced.

The first floor level is determine at level of EL. 160.60 m, which is 600-mm higher than the asphalt layer of surrounding road, and 3.60 m higher than the parking area in front. Foundation are placed on the level of EL. 162.00 m, square-shape spread type foundation are used.

(2) Staff House 1 (Guest House)

This building is a one (1) story building with 'Y' shape. The structure made by reinforcement concrete column and beam, tile roof with steel truss frame, while the partition are used 150-mm brick wall. The floor level is determining on EL. 165.40 m which 400-mm higher than parking and surrounding area level to prevent entering of rainwater.

(3) Staff House 2

These four buildings are arranged along the surrounding road to the guesthouse at highest level. The first rows are constructed at level EL. 162.0 m, and the second are at level EL. 163.5m. The floor levels are 750 mm higher than the ground level. The structures are used reinforcement concrete for column and beam, brick wall used for partition and steel truss frame for the roof.

(4) Mushola

The one story building is covered amount 9.0 x 9.0 m area and constructed by using reinforcement concrete of column and beam. The partitions are using brick wall 150-mm, while the roof structure is using steel truss frame below the ceramic roof tile finish. The floor level is 400 mm higher than ground level at EL. 164.00 m.

Architectural Design

(1) Finishes

The architectural design finishes are designed to be simple, economical and easily available in the local market. The following finishes are adopted for the proposed structures:

(a) Floor finishes

- Ceramic tile : all room use ceramic tile

(b) Wall finishes

- Paint on cement plaster : all R.C. and brick wall excepted interior surface at lavatory, toilet and pantry.
- Wood skirting 900-mm height : Wall along the corridor at all level story, chief room, meeting room, hall and lobby at administration building.
- Ceramic tile : Lavatory, toilet and pantry

(c) Ceiling finishes

- Gypsum board : All room at the administration building

		except corridor, lobby, lavatory and toilet. Dinning room, bedroom and pantry at guest house.
- Waterproof gypsum board	:	Toilet, lavatory
- Wooden lumbersering	:	Corridor and lobby at administration building. Terrace living room at guesthouse.
- Fiber cement board	:	All room at staff house, mushola and terrace at administration building.

(d) Roof finishes

- Ceramic roof tile	:	All building and houses

(2) Landscaping

Several ancillary structures and developments are considered to further enhance the architectural features and function of the area around this dam management complex.

(a) Gravel and Soil Embankment

Sub-grade condition around the site is made of hard rock will be excavated until level as designed for each building. The area around the building will be provided with 10-cm thick gravel layer and 30 cm thick topsoil layer. The gravel will mainly function for storm water drainage and the topsoil layer is utilized for planting and sodding.

(b) Asphalt Pavement for driveway and parking

As access to the dam management complex from the dam crest road and out side area, asphalt paved is used. The parking area in front of dam administration office will be used as main parking area for dam, so the capacity is big enough. However, for special proposes parking area are available, such as service parking for dam administration building and guesthouse parking.

(c) One block pavement for pedestrian way

For the pedestrian access to each building, cone block pavement is designed. The level determined 100 mm higher than asphalt layer.

(d) Planting

Plantations are used for as shown propose:

- Sun shine protection : Special kind of trees are used for this propose in several area, such as parking area and along the pedestrian way.

- Ground cover : To cover the excavated area selected grass will be use, especially adopted from the local area kind.

- Bordering : This function can accommodate by using shrubs which selected as designed.

- Direction element : Arrangement of trees along the access road will be show the direction, especially columnar trees.

- Decoration element

(3) Water Drainage

Storm water drainage system around this complex area consist of two components:

- Storm water drainage on asphalt layer such as access road and parking area and around buildings is drainage off through U-type drainage ditch.
- Storm water drainage for green area is carried through the gravel layer of embankment.

7.9.3 Hydropower Station Complex

Three (3) buildings included in this complex, namely hydropower station building, garage and guard house. This complex will be embanked at the right side area of the spillway stilling basin on the level EL. 97.00 m. The hydropower station building is the main facilities of the power generation. Another two building are just to support the main building.

Site plan and floor area table for the hydropower station complex are shown in Figs. 7.9.5 and 7.9.6.

(1) Hydropower Station Building

Dam hydropower station building is a single building with two stories above the ground and three stories under the ground. Total floor area of those two stories above is amount 389.64 m^2.

The layout is made based on the power generator layout, which installed at the lowest story of the basement. Therefore, the design of the room layout designed for the activities Three activities are accommodating in these hydropower station building, mentioned (1) Maintenance activities; (2) operational control activities; and (3) Office activities.

The elevation and floor plan are shown in Figs. 7.9.7 and 7.9.8.

(2) Garage

This garage building provided to accommodate for service car parking and maintenance utility storage, so the layout of this building is simple. One big open room is in left position with dimension 15.0 m x 9.0 m, and another one storage 5.0 m x 6.0 m at right position. The level determines at 150 mm higher from the ground level at EL. 97.00 m, it maenad for convenience for car access.

(3) Guard House

At the entrance gate will be provide one guardhouse for the security of those facilities above. This single building divided to be 2 rooms, one room for guard space the other provided for sanitary facilities such as toilet and bath convenience of personal who in charge.

Structure Design

(1) Hydropower Station Building

Main building is single unit 30.00 m x 11.00 m with canopy at entrance area. The building is made of reinforcement concrete framing and floor slab at apart of it. Span of main frame is 11.00 m long for the room over the generator, which also used for the base of the crane runaway. The secondary beams are used at office room slab. The slab floor thickness is 150 mm.

The partitions of the exterior wall are using brick wall with 150 mm thick, while the interior wall not all use brick wall but also gypsum board and glass.

Roof of this building is made of steel frame structures using angular bars and their connection are mostly using nuts-bolts and welding connection. On top of these frames, steel roof will be provided. As a roof of apart of this building, polycarbonate roof sheet and concrete roof slab with asphalt water proofing membrane are employed.

(2) Garage

The garage building is single simple building which using reinforcement concrete framing with just small part using brick wall for the partition. Span of the framing is 6.00 m long, and distance between them is 5.00 m. Steel roof with singe sloop (1-% slope) will be used over these framing.

Square-shape spread foundations are placed at 1.00-m bellow the ground floor at EL. 97.00 m.

(3) Guard House

The guard house building is single simple building which using reinforcement concrete framing with brick wall for the partition. Roof is using reinforce concrete slab with asphalt water proofing membrane under layer. For foundation just using stone masonry foundation.

Architectural Design

(1) Finishes

Same with the Dam Management complex, the architectural finishes are designed to be simple, economical and available at local market except for the special room. These special materials are installed for specials propose according to the activities that accommodate by those rooms.

These following finishes materials are adopted for proposed buildings:

(a) Floor finishes

- Ceramic tile : Office room, lobby, dinning room, pantry, locker, lobby and toilet at Hydropower station building. Guard's room, toilet and terrace at guard house.

- Rubber flooring : Assembling room and catwalk at

Hydropower station building.

- Access floor	: Control room at Hydropower station building.
- Mortar trowel	: All room at garage.

(b) Wall finishes

- Brick wall 150 mm	: All buildings are using brick wall material, but part sections of the exterior wall are covered by terracotta tile.
- Ceramic tile	: toilet locker and pantry.

(c) Ceiling finishes

Gypsum board	: Office room, lobby and control room at Hydropower station.
Waterproof gypsum board	: Toilet and locker at Hydropower station building.
Fiber cement board	: All room at guard house.

(d) Roof finishes

- Steel roof	: Use as a main roof at Hydropower station building and garage.
- Polycarbonate roof sheet	: Use at entrance canopy as a decoration element.
- Concrete slab	: With waterproof asphalt layer. Guard house and overhang roof at hydropower station building and garage.

(2) Landscaping

(a) Gravel and Soil Embankment

The site where this complex will be construct is a filled area, which filled until level EL. 97.00 m. Over this layer will be covered by 100 mm gravel and 300 mm thick topsoil layer. The gravel will mainly function as water drainage and the topsoil layer will use for planting and sodding.

(b) Asphalt Pavement for driveway and parking

The pavement will use asphalt layer for driveway, access road and parking space.

(c) Planting

Selected trees will use for sunshine protection at several places; another will cover by grass.

(3) Water Drainage

Storm water drainage system around this complex area consist of two components:

- Storm water drainage on asphalt layer such as access road and parking area and around buildings is drainage off through U-type drainage ditch.
- Storm water drainage for green area is carried through the gravel layer of embankment.

7.10 Approach Bridge to Goa Kreo Cave

Since the Normal Water Surface Elevation of the Jatibarang Dam Reservoir is higher than the existing approach road to Goa Kreo, a new bridge is designed for pedestrian only.

The location of the approach bridge to Goa Kreo Cave is shown in Figs. 7.10.1.

(1) Type of Bridge

As mentioned in CHAPTER 6, after comparison of two types, namely RC girder bridge and suspension bridge, RC girder type was selected from economical reason.

(2) Geological Condition

The geological structure of the bridge site is composed of Damar Formation of the latter period of Tertiary to Quaternary. Damar Formation is an alternation composed of tuffaceous sandstone, conglomerate, and tuff. It shows almost horizontal geological structure. These sedimentary rocks are without apparent cracks but weathered and softened to a deep portion of the ground. Although, no boring investigation has been done at the bridge site, according to the field reconnaissance, it is estimated that the foundation rock with N value over 30 is located 2 to 3 meters below the ground surface.

(3) Design Criteria

(a) Span length : 17.0 m × 4 spans

(b) Width : 2.0 m

(c) Clearance above the Surcharge Water Surface (EL. 151.8 m) : 0.6 m

(d) Live load; 500 kg/m^2 (BINAMARGA standard for a pedestrian bridge)

(e) Seismic load; based on the Design Criteria Report (VOLUME III)

For seismic load, following coefficient was adopted.

Teq = Kh · I · Wr

Where,

Teq : total base shear force in the direction beign considered (kN)

Kh : coefficient of horizontal seismic loading

Kh= C · S

Where,

C : base shear coefficient for the appropriate zone, period and site condition (=0.15 for zone 4)

S : structure type factor (=1.0 for RC type)

I : safety factor of Importance of structure (= 1.0 for pedestrian bridge)

Wr : total nominal weight of structure object to seismic acceleration (kN)

(4) Foundation Structure

As the foundation rock is shallow as 2 to 3 meters from the ground surface, spread foundation was adopted

(5) Design Result

Design result is shown in Fig. 7.10.2.