3.5 Power and Power Generation

In the current Interim Report (3) of the Detailed Design, the type and height of Jatibarang Dam was again reconsidered. A rockfill type dam of 77.0 m high was employed instead of a concrete gravity type dam of 77.5 m high. In connection with other modifications of dam design, total head for hydropower generation changed from 62.7 m to 65.99m and as the result, installed capacity of the hydropower station was revised to 1,500kW, as shown below.

$$
\begin{aligned}
P &= 9.8 \times E_T \times E_G \times He \times Q \\
&= 9.8 \times 0.858 \times 0.951 \times 64.3 \times 3.0 \\
&= 1{,}542 \text{ kW} \\
&\fallingdotseq 1{,}500 \text{ kW}
\end{aligned}
$$

where, $E_T = 0.858$: efficiency of turbine

$E_G = 0.951$: efficiency of generator

$He = NWL148.9m - TWL82.91m - h_{loss}$: Net head

$= 64.3m$

$h_{loss} = 1.690m$: Head loss

$Q = 3.0m^3/s$: Discharge

The annual average generations of present and future stages are 6,020 MWh and 8,640 MWh, respectively.

6,020 MWh = 5,790 MWh × 1,500/1,440
8,640 MWh = 8,307 MWh × 1,500/1,440

Item	Optimum Scale Study	Definitive Plan
Reservoir NWL (EL m)	148.6	148.9
Tailrace WL (EL m)	85.9	82.91
Gross Head (m)	62.7	65.99
Power Output (kW)	1,440	1,500
Annual Energy (MWh)		
present stage	5,790	6,020
future stage	8,307	8,640

3.6. Design of Penstock

3.6.1 Design Head

The design head of the outlet works is generally determined on the basis of the maximum water pressure head in the conduit system. The maximum water pressure head is computed considering the effects of the static head and the water hammer pressure head in the pipe system. The static head can be obtained by subtracting the elevation at the calculated point from the reservoir water level. The water hammer pressure is generally computed by solving the equation of continuity and equation of motion of fluid in consideration of the characteristics of the pipe system.

(A) Methodology

The water hammer is a pressure transient in a pipe system due to a rapid reduction of flow velocity caused by an adjustment of the setting of a control gate or a change in the operation of a turbine. There are a lot of the methods used for the water hammer analysis. In this study, a FORTRAN computer program made by unsteady flow equations was employed for computing the water hammer in the conduit system of the Jatibarang Power Station.

(a) Basic Equations

The equation of motion and the continuity equation used for the water hammer analysis are

Equation of Motion

$$\frac{\partial h}{\partial x} = -\frac{1}{g}\frac{\partial v}{\partial t} - \alpha v^2 \qquad (1)$$

Equation of Continuity

$$\frac{\partial h}{\partial t} = -\frac{c^2}{g}\frac{\partial v}{\partial x} \qquad (2)$$

Where

v : Velocity in the pipeline

h : Water head

α : Hydraulic loss coefficient in the pipeline

c : Velocity of pressure wave

g : Acceleration of gravity

t : Time

If the head loss is neglected, then, the above equations become as follows

$$h = h_0 + F(t + \frac{x}{c}) + f(t - \frac{x}{c})$$

$$v = v_0 + \frac{g}{c}[F(t + \frac{x}{c}) + f(t - \frac{x}{c})$$

By further transformation, the following expressions can be obtained:

$$h - h_0 + \frac{c}{g}(v - v_0) = 2f(t - \frac{x}{c}) \quad (3)$$

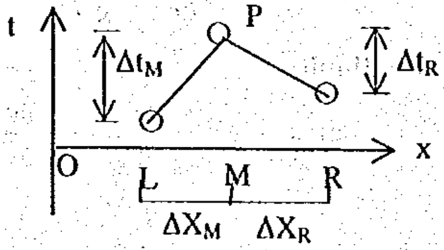
$$h - h_0 - \frac{c}{g}(v - v_0) = 2F(t + \frac{x}{c}) \quad (4)$$

Where F and f are the heads of backward and forward waves at the moment of $t-x/c$ and $t+x/c$; h_0 is the initial pressure head; v_0 is initial velocity; and x represents the distance between the calculated point and the reference point.

The water hammer at any point can be computed by solving Eq.(3) and Eq.(4). For example, the pressure head at point M depicted in Sketch "A" at the moment of t, h_p, can be determined in the following manner, using known pressures at points L and R.

The water hammer travel time from point L to point M, Δt_M, is expressed as follows:

$$\Delta t_M = \Delta x_M / c_M \quad (5)$$



Sketch "A"

The travel time from R to M is

$$\Delta t_R = \Delta x_R / c_R \tag{6}$$

From the correlation of the location of the three points, L, M and R, the following formulae can be obtained

$$x_M = x_L + \Delta x_M \tag{7}$$

$$x_M = x_R - \Delta x_R \tag{8}$$

By combining Eqs. (3), (5) and (7), the following is obtained:

$$h_P + \frac{c_M}{g} v_P = h_{L,t-\Delta t_M} + \frac{c_M}{g} v_{L,t-\Delta t_M} \tag{9}$$

Also, Eqs. (4), (6) and (8) yield the following:

$$h_P - \frac{c_R}{g} v_P = h_{R,t-\Delta t_R} + \frac{c_R}{g} v_{R,t-\Delta t_R} \tag{10}$$

Where the right-handed terms in Eq.(9) and Eq.(10) represent the values at the moment of t-Δt, and the left-handed terms are the ones at the moment of t.

By adopting the terms $h' = h_p/h_o$, $q' = A_M v_p/Q_o$, $\rho_M = c_M Q_o/(2gA_M h_o)$, and $\rho_R = c_R Q_o/(2gA_R h_o)$, Eq.(9) and Eq.(10) can be transformed into the following dimensionless forms:

$$h' + 2\rho_M q' = h'_L + 2\rho_L q'_L \tag{11}$$

$$h' - 2\rho_R q' = h'_R - 2\rho_R q'_R \tag{12}$$

In which, A_M and A_R are the sectional areas of the pipeline at points M and R, and h_o and Q_o are the static head and the discharge under steady state condition, respectively.

If the head loss of the waterway is taken into account, the Eqs. (11) and (12) become as follows:

$$h' + 2\rho_M q' = K_1 \tag{13}$$

$$h' - 2\rho_R q' = K_2 \tag{14}$$

Where

$K_1 \quad = \quad h'_L + 2\rho_L q'_L - \alpha_M |q'_L| q'_L$

$K_2 \quad = \quad h'_R - 2\rho_R q'_R + \alpha_R |q'_R| q'_R$

$\alpha_M \quad = \quad \Delta h_{L0}/h_0$

$\alpha_R \quad = \quad \Delta h_{R0}/h_0$

Δh_{L0} = Head loss between points L and M under the condition of Q_o

Δh_{R0} = Head loss between points M and R under the condition of Q_o

If the right-handed terms in above equations are known, the h' and q' at point M located between L and R, can be easily obtained by a step-by-step method, starting with a steady state condition.

(b) Specific Formulae at Different Kinds of Calculated Points

The h' and q' at a specific point obtained by substituting the specific parameters for Eq.(13) and Eq.(14) are expressed as follows:

i) <u>at middle point</u>

Eliminating h' from Eq.(13) and Eq.(14), then

$$q' = \frac{K_1 - K_2}{2(\rho_m + \rho_R)} \qquad (15)$$

Substituting Eq.(15) for Eq.(14), the following can be obtained

$$h' = K_1 - 2\rho_M q' \qquad (16)$$

Sketch "B"

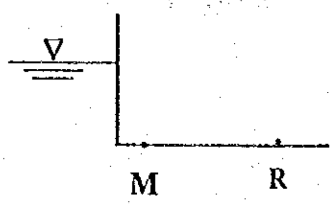
ii) <u>at upstream end with a free surface</u>

Considering that the head at point M is always being kept as the static head, h_o, then

$$h' = h/h_o = h_o/h_o = 1 \qquad (17)$$

and from Eq.(14), then,

$$q' = \frac{h' - K_2}{2\rho_R} \tag{18}$$



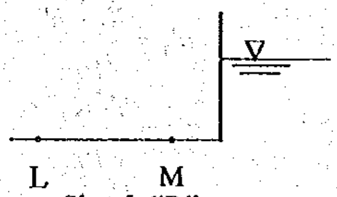
Sketch "C"

iii) at downstream end with a free surface

Similar to the above, h' and q' can be obtained

$$h' = h/h_o = h_o/h_o = 1 \tag{19}$$

$$q' = \frac{K_1 - h'}{2\rho_M} \tag{20}$$

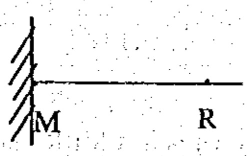


Sketch "D"

iv) at upstream closing end

Since q' is 0.0 at the upstream closing end, then

$$h' = K_2 \tag{21}$$

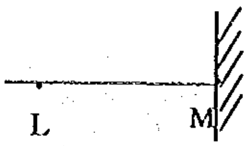


Sketch "E"

v) at downstream closing end

Since q' is 0.0 at the downstream closing end, then

$$h' = K_1 \tag{22}$$



Sketch "F"

vi) **at branching point**

The pressure head, h', and the discharges, q'_1, q'_2 and q'_3 can be obtained by

$$h' + 2\rho_{ML} q'_1 = K_1 \quad (23)$$

$$h' - 2\rho_R q'_2 = K_2 \quad (24)$$

$$h' - 2\rho_{RR} q'_3 = K_3 \quad (25)$$

$$q'_1 = q'_2 + q'_3 \quad (26)$$

Rearranging the Eqs.(23)~(25), then

$$q'_1 = \frac{K_1 - h'}{2\rho_{ML}} \quad (27)$$

$$q'_2 = \frac{h' - K_2}{2\rho_R} \quad (28)$$

$$q'_3 = \frac{h' - K_3}{2\rho_{RR}} \quad (29)$$

Substituting Eqs.(27)~(29) into Eq.(26) and rearranging, then

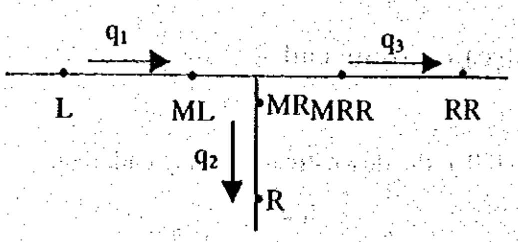
$$h' = \left(\frac{K_1}{\rho_{ML}} + \frac{K_2}{\rho_R} + \frac{K_3}{\rho_{RR}}\right) \Bigg/ \left(\frac{1}{\rho_{ML}} + \frac{1}{\rho_R} + \frac{1}{\rho_{RR}}\right) \quad (30)$$

Where

K_3 = $h'_{RR} - 2\rho_{RR} q'_{RR} + \alpha_3 |q'_{RR}| q'_{RR}$

α_3 = $\Delta h_3 / h_0$

Δh_3 = Head loss between points MRR and RR under the condition of Q_o



Sketch "G"

vii) <u>at closing end of turbine</u>

The discharge through the turbine under the normal condition, Q_o, can be obtained by

$$Q_o = A_o (2gy_0)^{1/2} \tag{31}$$

Where

- A_0 : Sectional area of the pipe
- y_0 : Effective head under the normal condition

The discharge through the turbine during closing period, Q, can be expressed as

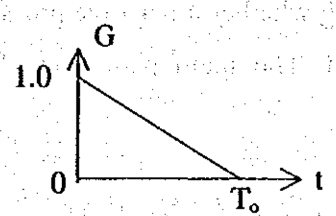
$$Q = GA_0 (2gy)^{1/2} \tag{32}$$

Where

- y : Effective head during closing period
- G : Relative opening of the guide vane, $G = 1 - t/T_0$
- T_0 : Closing time of the guide vane

From Eq.(31) and Eq.(32), the ratio of Q/Q_0, q', can be calculated as follows

$$q' = Q/Q_0 = G(y/y_0)^{1/2} \tag{33}$$



Sketch "H"

Rearranging the above equation, the following dimensionless equation can be obtained

$$q'^2 = G^2 \cdot (h'_{ML} - h'_{MR})\,\beta \tag{34}$$

Where

- $\beta = h_o/y_o$
- $h'_{ML} = h_{ML}/h_o$
- $h'_{MR} = h_{MR}/h_o$

Solving Eq.(13), Eq.(14) and Eq.(34), then

$$q' = -K_3 + (K_3^2 - K_4)^{1/2} \quad (35)$$

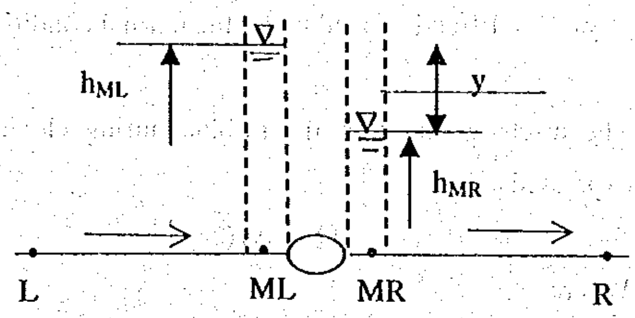
$$h'_{ML} = K_1 - 2\rho_{ML} q' \quad (36)$$

$$h'_{MR} = K_2 + 2\rho_R q' \quad (37)$$

Where

$K_3 = G^2\beta(\rho_{ML}+\rho_R)$

$K_4 = G^2\beta\,(K_2 - K_1)$



Sketch "I"

(B) Calculated Cases and Parameters

The calculations were carried out for six cases shown in Table 3.6.1. The Cases 1 thru 4 were adopted for determining the design head of the pipe system, and Cases 5 and 6 are for checking whether a negative pressure occurs in the tailrace when the turbine was closed. The main parameters for calculations were as follows.

Calculated Parameters

Maximum Design Discharge		6.0	m^3/s
Maximum Power Discharge		3.0	m^3/s
Roughness Coefficient	Concrete	$n_c = 0.0125$	
	Steel	$n_s = 0.0115$	
Centerline of Turbines (℄ Distributors)	EL	84.70	m
Unit Weight of Water		$W_0 = 1{,}000$	kgf/m^3
Acceleration of Gravity		$G = 9.8$	m/s^2
Thickness of Pipe		$t = 9.0$	mm

Elastic Modulus of Steel	$E = 2.1\times10^{10}$	kgf/m^2
Elastic Modulus of Concrete	$E_C = 2.1\times10^{9}$	kgf/m^2
Elastic Modulus of Rock	$E_R = 1.2\times10^{8}$	kgf/m^2
Elastic Modulus of Water	$K = 2.0\times10^{8}$	kgf/m^2
Poisson's Ratio of Rock	$\nu = 0.3$	

Closing time: The closing time of the guide vanes is one of the most important design considerations. The timing of closure of the guide vanes will affect the water hammer pressure.

However, the closing time of the guide vanes is generally determined by the characteristics of the generator and the turbine system. The manufacturers usually guarantee the functioning of the equipment on the basis of the design pressure and the ratio of the momentary speed variation, both of which are specified in the contract.

This closing time of 3.5 seconds was adopted for the water hammer analysis in this study.

Wave velocity: The wave velocities along different portions were calculated using the following expressions:

(i) Along Intake and Tailrace

$$c = 1 \Big/ \sqrt{\frac{w_0}{g}\left\{\frac{1}{K}+\frac{2}{E_r}\right\}} \qquad (38)$$

(ii) Along the Steel Pipe embedded into Tunnel

$$c = 1 \Big/ \sqrt{\frac{w_0}{g}\left\{\frac{1}{K}+\frac{2r}{Et}(1-\lambda)\right\}} \qquad (39)$$

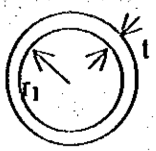
$$\lambda = \frac{r^2}{Et} \Big/ \left\{\frac{r^2}{E\cdot t}+\frac{(r_S{}^2-r^2}{2r_s E_c}+\frac{(m_r+1)r}{m_r E_r}\right\}$$

(iii) Along the Portions embedded into Soil within the Powerhouse

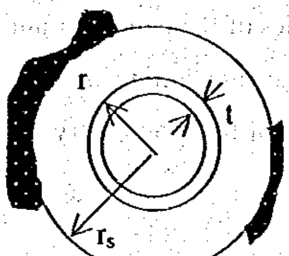
$$c = 1 \Big/ \sqrt{\frac{w_0}{g}\left\{\frac{1}{K}+\frac{2r_1}{Et}\right\}} \qquad (40)$$

Where,

K	:	Modulus of compressibility of water
E_c	:	Modulus of elasticity of concrete
E	:	Modulus of elasticity of steel
E_r	:	Modulus of elasticity of the surrounding rock
m_r	:	Poisson's ratio for the surrounding rock
w_0	:	Unit weight of water
r	:	External radius of the pipe
r_1	:	Internal radius of the pipe
r_s	:	Excavated radius of the tunnel
g	:	Acceleration of gravity



Sketch "J"



Sketch "K"

If the diameter of the steel pipe and medium around the steel pipe are different along the reach between two calculated points, an equivalent wave velocity and equivalent area will be adopted in the portion. The equivalent wave velocity, c_m, and the equivalent sectional area, A_m, are expressed in the following manner:

$$c_m = \frac{\sum L_i}{\sum L_i / a_i} \tag{41}$$

$$A_m = \frac{\sum L_i}{\sum L_i / A_i} \tag{42}$$

In which, L_i, A_i and c_i are the length, sectional area, and wave velocity of the calculated reach, respectively.

The calculated points are depicted in Fig.3.6.1 and the locations of the calculated points are described in Table 3.6.2.

The distances from the entrance of the intake, sectional areas and wave velocities are summarised in Table 3.6.3. The head losses for different cases are listed in Table 3.6.4.

(C) Calculated Results

The maximum and minimum water pressures at each point for the six cases are summarised in Tables 3.6.5 and 3.6.6 and shown in Figs. 3.6.2 and 3. It can be found that the maximum water pressures at each point for Case 1 are the largest among the six cases, and the minimum water pressures at each point for Case 6 are the smallest among the six cases. The water pressures for all six cases are larger than 0.0, that means no negative pressure occurs in the whole pipe system.

The variances of the water pressures at the upstream and downstream ends of the turbine for the six cases are depicted in Fig. 3.6.4. The water pressures at the upstream end of the turbine for the six cases become the maximum before the complete closing. The increase of water pressures due to water hammer for the cases with branching tend to reach the static head rapidly, compared to the non-branching cases.

The discharge fluctuations at the control gate of the main water supply pipe (No.13) for Cases 3 and 5 are shown in Fig. 3.6.5. The releasing discharges vary suddenly during the process of closing of the generation system.

(D) Determination of Design Head

The maximum water pressures and pressures which increase due to water hammer at the upstream end of the turbine for the six cases are listed in Tables 3.6.7. Although the pressure increase due to water hammer for Case 2 is slightly larger than the one for Case 1, the maximum water pressure for Case 1 is larger than that for Case 2. In addition, from the above-mentioned results, the maximum water pressures at each point for Case 1 are all larger than those for Cases 2 thru 6. Therefore, the maximum water pressures for Case 1 are adopted as the design heads in the whole pipe system.

Finally, the design head at the upstream end of the turbine is taken as 101.0 m in consideration of some allowance. The design head distribution from the entrance of the steel pipe(No.3) to the upstream end of the turbine (No.17) is shown below.

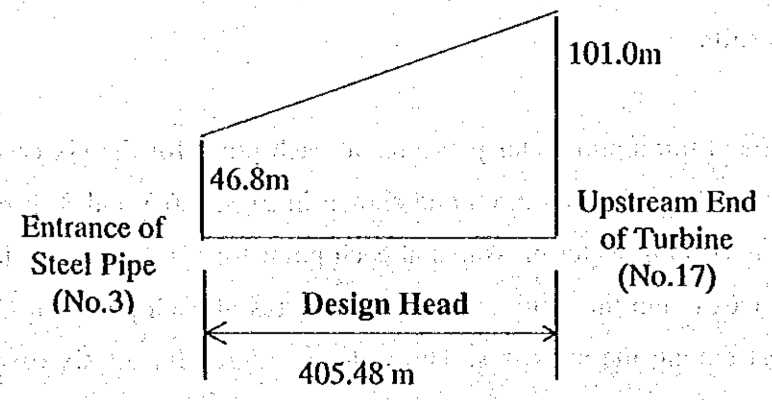


Table 3.6.1 Calculated Cases and Purposes

Case	Calculated Parameters						Purpose	Remarks
	Reservoir Water Level (EL.m)	Tail Water Level (TWL) (EL.m)	Downstream Water Level for Calculation (DWL) (EL.m)	Discharge (m3/s)				
				Total	Generation	Water Supply		
1	155.30	87.79	87.79	3.0	3.0	0.0	to compute design head	MWL
2	151.80	84.80	84.80	3.0	3.0	0.0		SWL
3	148.90	82.70	83.268	6.0	3.0	3.0		NWL
4	148.90	82.63	83.268	3.0	3.0	0.0		NWL
5	136.00	82.70	83.268	6.0	3.0	3.0	to check negative pressure	LWL
6	136.00	82.63	83.268	3.0	3.0	0.0		LWL

Notes: NWL=normal water level; LWL=lower water level;
SWL=surcharge water level (during 100 year return period flood);
MWL=maximum water level (during PMF);
TWL = the water level of the downstream river channel at the outlet section of tairace;
DWL is defined as the downstream water level for water hammer analysis, DWL is the same as TWL for Cases 1 and 2, DWLs for Cases 3 thru 6 are set as the water levels at the section upstream of air shaft, which are obtained by a non-uniform calculation.

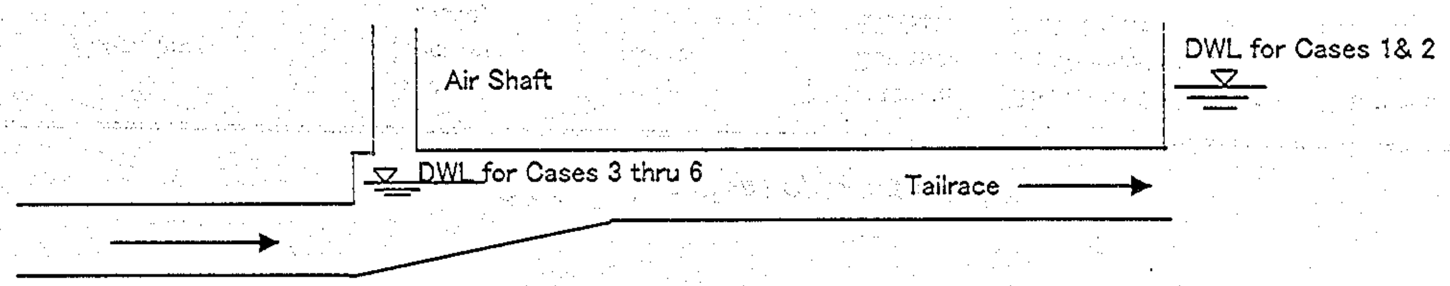


Table 3.6.2 Locations of Calculated Points

No.	Location	Elevation of Section (EL.m)	Reach	Interval Distance ΔL (m)	Distance of Waterway for Generation from No.1, Lg (m)	Distance of Waterway for Main Water Supply from No.1, Ls(m)
1	Entrance of lower part of inclined intake	130.000		0.00	0.00	0.00
2	Downstream face of emergence gate	115.505	No.1 - No.2	24.94	24.94	24.94
3	Entrance of steel pipe	110.600	No.2 - No.3	4.07	29.01	29.01
4	5m downstream of HIP1	105.378	No.3 - No.4	88.57	117.58	117.58
5	Dam axis	97.869	No.4 - No.5	110.26	227.84	227.84
6	Start of HIP2	93.050	No.5 - No.6	70.77	298.61	298.61
7	End of HIP2	90.638	No.6 - No.7	35.43	334.03	334.03
8	End of VIP2	84.700	No.7 - No.8	87.33	421.36	421.36
9	Branching point	84.700	No.8 - No.9	6.75	428.11	428.11
10	Start of main water supply pipe	84.700		0.00		428.11
11	Branching point of water supply pipe	85.900	No.10 - No.11	9.71		437.82
12	Just downstream of branching	85.900		0.00		437.82
13	Control gate of main water supply pipe	85.900	No.12 - No.13	6.50		444.32
14	Start of branch water supply pipe	85.900				
15	Control gate of branch water supply pipe	85.450	No.14 - No.15	9.10		
16	Start of generation supply pipe	84.700			428.11	
17	Just upstream of turbine	84.700	No.16 - No.17	6.36	434.47	
18	Draft-tube exit	81.250			437.92	
19	Air shaft for cases of NWL & LWL	81.800	No.18 - No.19	12.21	450.13	
	Tailrace exit for cases of MWL & SWL	83.300		50.64	488.56	

Notes: NWL=normal water level; LWL=lower water level;
SWL=surcharge water level (during 100 year return period flood); MWL=maximum water level (during PMF).

Table 3.6.3 Equivalent Sectional Area and Equivalent Wave Velocity

Section	Length Li(m)	Diameter D(m)	Area A(m²)	Interval Distance L(m)	Equivalent Area A(m²)	Pipe Thickness (mm)	Wave Velocity a(m/s)	Equivalent Wave Velocity a_m(m/s)	Remarks
No.1 - No.2 (entrance of lower part of inclined intake - entrance of pipe)	24.94	2.02	3.20	24.94	3.20	9.0	672.54	672.54	Intake
No.2 - No.3 (emergence gate)	1.60	1.58	1.96			9.0	672.54		Intake
	2.47	1.40	1.54	4.07	1.68	9.0	911.28	799.70	Penstock
No.3 - No.4 (5m after HIP1)	88.57	1.40	1.54	88.57	1.54	9.0	911.28	911.28	
No.4 - No.5 (dam axis)	110.26	1.40	1.54	110.26	1.54	9.0	911.28	911.28	
No.5 - No.6 (start of HIP2)	70.77	1.40	1.54	70.77	1.54	9.0	911.28	911.28	
No.6 - No.7 (end of HIP2)	35.43	1.40	1.54	35.43	1.54	9.0	911.28	911.28	
No.7 - No.8 (end of VIP2)	87.33	1.40	1.54	87.33	1.54	9.0	911.28	911.28	
No.8 - No.9 (branching point)	6.75	1.40	1.54	6.75	1.54	9.0	888.74	888.74	
No.10 - No.11 (just branching - branching point of water supply	9.71	0.65	0.33	9.71	0.33	9.0	1077.61	1077.61	Main water supply pipe
No.12 - No.13 (just after branching - control gate)	6.50	0.65	0.33	6.50	0.33	9.0	1077.61	1077.61	Main water supply pipe
No.14 - No.15 (branch water supply pipe: just after branching - control	9.10	0.25	0.05	9.10	0.05	9.0	1244.97	1244.97	Branch water supply pipe
No.16 - No.17 (penstock: just after branching - before turbine)	2.00	1.40	1.54			9.0	888.74		Penstock
	2.86	1.11	0.97			9.0	948.56		
	1.50	0.80	0.50	6.36	0.88	9.0	1030.26	946.23	
No.18 - No.19 (draft-tube exit section -air shaft)	2.08	4.49	15.81			9.0	672.54		For NWL & LWL
	5.00	3.65	10.48			9.0	672.54		
	5.13	2.26	4.00	12.21	6.45	9.0	672.54	672.54	
(air shaft - tailrace exit)	5.40	2.71	5.78			9.0	672.54		For SWL & MWL
	33.04	2.26	4.00	50.64	4.57	9.0	672.54	672.54	

Table 3.6.4(1/6) Head Loss along Each Reach (Case 1)

($Q=3\ m^3/s$ and MWL=EL.155.3m)

Reach	Length (m)	Discharge Q(m³/s)	Type of Loss	Formula	Head Loss (m)	Sum of Loss (m)
No.1 - No.2	24.939	3.0	Trash racks	$0.000013Q^2$	0.000	
			Friction	$0.001119Q^2$	0.010	
			Entrance	$0.000664Q^2$	0.006	0.016
No.2 - No.3	4.071	3.0	Friction	$0.000823Q^2$	0.007	
			Bend (VIP1)	$0.002347Q^2$	0.021	0.029
No.3 - No.4	88.573	3.0	Friction	$0.020058Q^2$	0.181	
			Bend (HIP1)	$0.002806Q^2$	0.025	0.206
No.4 - No.5	110.256	3.0	Friction	$0.024969Q^2$	0.225	0.225
No.5 - No.6	70.769	3.0	Friction	$0.016026Q^2$	0.144	0.144
No.6 - No.7	35.425	3.0	Friction	$0.008022Q^2$	0.072	
			Bend (HIP2)	$0.001990Q^2$	0.018	0.090
No.7 - No.8	87.331	3.0	Friction	$0.019777Q^2$	0.178	
			Bend (VIP2)	$0.000612Q^2$	0.006	0.184
No.8 - No.9	6.748	3.0	Friction	$0.001528Q^2$	0.014	
			Bend (HIP3)	$0.001327Q^2$	0.012	0.026
No.10 - No.11 (water supply pipe)	9.710	0.0	Branching	$0.000215Q^2$	0.000	
		0.0	Friction	$0.131576Q^2$	0.000	
			Bend (VIP3 & HIP4)	$0.097806Q^2$	0.000	
			Gradual contraction	$0.003240Q^2$	0.000	0.000
No.12 - No.13	6.500	0.0	Branching	$0.013886Q^2$	0.000	
			Friction	$0.088079Q^2$	0.000	0.000
No.14 - No.15	9.095	0.0	Branching	$0.004628Q^2$	0.000	
			Friction	20.139796Q	0.000	0.000
No.16 - No.17	6.360	3.0	Branching	$0.000646Q^2$	0.006	
		3.0	Friction	$0.010740Q^2$	0.097	
			Gradual contraction	$0.001210Q^2$	0.011	0.113
No.18 - No.19 (before air shaft)	12.206	3.0	Draft-tube	$0.020766Q^2$	0.187	
			Friction	$0.000137Q^2$	0.001	
			Miter bend	$0.000233Q^2$	0.002	
			Gradual contraction	$0.000143Q^2$	0.001	
			Abrupt enlargement	$0.000587Q^2$	0.005	
(after air shaft)	38.436	3.0	Friction	$0.000730Q^2$	0.007	
			Entrance due to air shaft	$0.001765Q^2$	0.016	
			Gradual contraction	$0.000029Q^2$	0.000	
			Exit	$0.003189Q^2$	0.029	0.248
Σ					1.280	

Table 3.6.4(2/6) Head Loss along Each Reach (Case 2)

(Q=3 m^3/s and SWL=EL.151.8m)

Reach	Length (m)	Discharge Q(m^3/s)	Type of Loss	Formula	Head Loss (m)	Sum of Loss (m)
No.1 - No.2	24.939	3.0	Trash racks	$0.000017Q^2$	0.000	
			Friction	$0.001119Q^2$	0.010	
			Entrance	$0.000664Q^2$	0.006	0.016
No.2 - No.3	4.071	3.0	Friction	$0.000823Q^2$	0.007	
			Bend (VIP1)	$0.002347Q^2$	0.021	0.029
No.3 - No.4	88.573	3.0	Friction	$0.020058Q^2$	0.181	
			Bend (HIP1)	$0.002806Q^2$	0.025	0.206
No.4 - No.5	110.256	3.0	Friction	$0.024969Q^2$	0.225	0.225
No.5 - No.6	70.769	3.0	Friction	$0.016026Q^2$	0.144	0.144
No.6 - No.7	35.425	3.0	Friction	$0.008022Q^2$	0.072	
			Bend (HIP2)	$0.001990Q^2$	0.018	0.090
No.7 - No.8	87.331	3.0	Friction	$0.019777Q^2$	0.178	
			Bend (VIP2)	$0.000612Q^2$	0.006	0.184
No.8 - No.9	6.748	3.0	Friction	$0.001528Q^2$	0.014	
			Bend (HIP3)	$0.001327Q^2$	0.012	0.026
No.10 - No.11 (water supply pipe)	9.710	0.0	Branching	$0.000215Q^2$	0.000	
		0.0	Friction	$0.131576Q^2$	0.000	
			Bend (VIP3 & HIP4)	$0.097806Q^2$	0.000	
			Gradual contraction	$0.003240Q^2$	0.000	0.000
No.12 - No.13	6.500	0.0	Branching	$0.013886Q^2$	0.000	
			Friction	$0.088079Q^2$	0.000	0.000
No.14 - No.15	9.095	0.0	Branching	$0.004628Q^2$	0.000	
			Friction	20.139796Q	0.000	0.000
No.16 - No.17	6.360	3.0	Branching	$0.000646Q^2$	0.006	
		3.0	Friction	$0.010740Q^2$	0.097	
			Gradual contraction	$0.001210Q^2$	0.011	0.113
No.18 - No.19 (before air shaft)	12.206	3.0	Draft-tube	$0.020766Q^2$	0.187	
			Friction	$0.000137Q^2$	0.001	
			Miter bend	$0.000233Q^2$	0.002	
			Gradual contraction	$0.000143Q^2$	0.001	
			Abrupt enlargement	$0.000587Q^2$	0.005	
(after air shaft)	38.436	3.0	Friction	$0.000730Q^2$	0.007	
			Entrance due to air shaft	$0.001765Q^2$	0.016	
			Gradual contraction	$0.000029Q^2$	0.000	
			Exit	$0.003189Q^2$	0.029	0.248

Σ 1.280

Table 3.6.4(3/6) Head Loss along Each Reach (Case 3)

($Q=6\ m^3/s$ and NWL=EL.148.9m)

Reach	Length (m)	Discharge $Q(m^3/s)$	Type of Loss	Formula	Head Loss (m)	Sum of Loss (m)
No.1 - No.2	24.939	6.0	Trash racks	$0.000023Q^2$	0.001	
			Friction	$0.001119Q^2$	0.040	
			Entrance	$0.000664Q^2$	0.024	0.065
No.2 - No.3	4.071	6.0	Friction	$0.000823Q^2$	0.030	
			Bend (VIP1)	$0.002347Q^2$	0.084	0.114
No.3 - No.4	88.573	6.0	Friction	$0.020058Q^2$	0.722	
			Bend (HIP1)	$0.002806Q^2$	0.101	0.823
No.4 - No.5	110.256	6.0	Friction	$0.024969Q^2$	0.899	0.899
No.5 - No.6	70.769	6.0	Friction	$0.016026Q^2$	0.577	0.577
No.6 - No.7	35.425	6.0	Friction	$0.008022Q^2$	0.289	
			Bend (HIP2)	$0.001990Q^2$	0.072	0.360
No.7 - No.8	87.331	6.0	Friction	$0.019777Q^2$	0.712	
			Bend (VIP2)	$0.000612Q^2$	0.022	0.734
No.8 - No.9	6.748	6.0	Friction	$0.001528Q^2$	0.055	
			Bend (HIP3)	$0.001327Q^2$	0.048	0.103
No.10 - No.11 (water supply pipe)	9.710	6.0	Branching	$0.000754Q^2$	0.027	
		3.0	Friction	$0.131576Q^2$	1.184	
			Bend (VIP3 & HIP4)	$0.097806Q^2$	0.880	
			Gradual contraction	$0.003240Q^2$	0.029	2.121
No.12 - No.13	6.500	3.0	Branching	$0.013886Q^2$	0.125	
			Friction	$0.088079Q^2$	0.793	0.918
No.14 - No.15	9.095	0.0	Branching	$0.004628Q^2$	0.000	
			Friction	20.139796Q	0.000	0.000
No.16 - No.17	6.360	6.0	Branching	$0.000969Q^2$	0.035	
		3.0	Friction	$0.010740Q^2$	0.097	
			Gradual contraction	$0.001210Q^2$	0.011	0.142
No.18 - No.19	12.206	3.0	Draft-tube	$0.020766Q^2$	0.187	
			Friction	$0.000137Q^2$	0.001	
			Miter bend	$0.000233Q^2$	0.002	
			Gradual contraction	$0.000143Q^2$	0.001	
			Abrupt enlargement	$0.000111Q^2$	0.001	0.193
Σ					7.049	

Table 3.6.4(4/6) Head Loss along Each Reach (Case 4)

(Q=3 m^3/s and NWL=EL.148.9m)

Reach	Length (m)	Discharge $Q(m^3/s)$	Type of Loss	Formula	Head Loss (m)	Sum of Loss (m)
No.1 - No.2	24.939	3.0	Trash racks	$0.000023Q^2$	0.000	
			Friction	$0.001119Q^2$	0.010	
			Entrance	$0.000664Q^2$	0.006	0.016
No.2 - No.3	4.071	3.0	Friction	$0.000823Q^2$	0.007	
			Bend (VIP1)	$0.002347Q^2$	0.021	0.029
No.3 - No.4	88.573	3.0	Friction	$0.020058Q^2$	0.181	
			Bend (HIP1)	$0.002806Q^2$	0.025	0.206
No.4 - No.5	110.256	3.0	Friction	$0.024969Q^2$	0.225	0.225
No.5 - No.6	70.769	3.0	Friction	$0.016026Q^2$	0.144	0.144
No.6 - No.7	35.425	3.0	Friction	$0.008022Q^2$	0.072	
			Bend (HIP2)	$0.001990Q^2$	0.018	0.090
No.7 - No.8	87.331	3.0	Friction	$0.019777Q^2$	0.178	
			Bend (VIP2)	$0.000612Q^2$	0.006	0.184
No.8 - No.9	6.748	3.0	Friction	$0.001528Q^2$	0.014	
			Bend (HIP3)	$0.001327Q^2$	0.012	0.026
No.10 - No.11 (water supply pipe)	9.710	0.0	Branching	$0.000215Q^2$	0.000	
		0.0	Friction	$0.131576Q^2$	0.000	
			Bend (VIP3 & HIP4)	$0.097806Q^2$	0.000	
			Gradual contraction	$0.003240Q^2$	0.000	0.000
No.12 - No.13	6.500	0.0	Branching	$0.013886Q^2$	0.000	
			Friction	$0.088079Q^2$	0.000	0.000
No.14 - No.15	9.095	0.0	Branching	$0.004628Q^2$	0.000	
			Friction	20.139796Q	0.000	0.000
No.16 - No.17	6.360	3.0	Branching	$0.000646Q^2$	0.006	
		3.0	Friction	$0.010740Q^2$	0.097	
			Gradual contraction	$0.001210Q^2$	0.011	0.113
No.18 - No.19	12.206	3.0	Draft-tube	$0.020766Q^2$	0.187	
			Friction	$0.000137Q^2$	0.001	
			Miter bend	$0.000233Q^2$	0.002	
			Gradual contraction	$0.000143Q^2$	0.001	
			Abrupt enlargement	$0.000111Q^2$	0.001	0.193

Σ 1.225

Table 3.6.4(5/6) Head Loss along Each Reach (Case 5)

(Q=6 m^3/s and LWL=EL.136.0m)

Reach	Length (m)	Discharge Q(m^3/s)	Type of Loss	Formula	Head Loss (m)	Sum of Loss (m)
No.1 - No.2	24.939	6.0	Trash racks	$0.000227Q^2$	0.008	
			Friction	$0.001119Q^2$	0.040	
			Entrance	$0.000664Q^2$	0.024	0.072
No.2 - No.3	4.071	6.0	Friction	$0.000823Q^2$	0.030	
			Bend (VIP1)	$0.002347Q^2$	0.084	0.114
No.3 - No.4	88.573	6.0	Friction	$0.020058Q^2$	0.722	
			Bend (HIP1)	$0.002806Q^2$	0.101	0.823
No.4 - No.5	110.256	6.0	Friction	$0.024969Q^2$	0.899	0.899
No.5 - No.6	70.769	6.0	Friction	$0.016026Q^2$	0.577	0.577
No.6 - No.7	35.425	6.0	Friction	$0.008022Q^2$	0.289	
			Bend (HIP2)	$0.001990Q^2$	0.072	0.360
No.7 - No.8	87.331	6.0	Friction	$0.019777Q^2$	0.712	
			Bend (VIP2)	$0.000612Q^2$	0.022	0.734
No.8 - No.9	6.748	6.0	Friction	$0.001528Q^2$	0.055	
			Bend (HIP3)	$0.001327Q^2$	0.048	0.103
No.10 - No.11 (water supply pipe)	9.710	6.0	Branching	$0.000754Q^2$	0.027	
		3.0	Friction	$0.131576Q^2$	1.184	
			Bend (VIP3 & HIP4)	$0.097806Q^2$	0.880	
			Gradual contraction	$0.003240Q^2$	0.029	2.121
No.12 - No.13	6.500	3.0	Branching	$0.013886Q^2$	0.125	
			Friction	$0.088079Q^2$	0.793	0.918
No.14 - No.15	9.095	0.0	Branching	$0.004628Q^2$	0.000	
			Friction	20.139796Q	0.000	0.000
No.16 - No.17	6.360	6.0	Branching	$0.000969Q^2$	0.035	
		3.0	Friction	$0.010740Q^2$	0.097	
			Gradual contraction	$0.001210Q^2$	0.011	0.142
No.18 - No.19	12.206	3.0	Draft-tube	$0.020766Q^2$	0.187	
			Friction	$0.000137Q^2$	0.001	
			Miter bend	$0.000233Q^2$	0.002	
			Gradual contraction	$0.000143Q^2$	0.001	
			Abrupt enlargement	$0.000111Q^2$	0.001	0.193
Σ					7.056	

Table 3.6.4(6/6) Head Loss along Each Reach (Case 6)

(Q=3 m^3/s and LWL=EL.136.0m)

Reach	Length (m)	Discharge Q(m^3/s)	Type of Loss	Formula	Head Loss (m)	Sum of Loss (m)
No.1 - No.2	24.939	3.0	Trash racks	$0.000227Q^2$	0.002	
			Friction	$0.001119Q^2$	0.010	
			Entrance	$0.000664Q^2$	0.006	0.018
No.2 - No.3	4.071	3.0	Friction	$0.000823Q^2$	0.007	
			Bend (VIP1)	$0.002347Q^2$	0.021	0.029
No.3 - No.4	88.573	3.0	Friction	$0.020058Q^2$	0.181	
			Bend (HIP1)	$0.002806Q^2$	0.025	0.206
No.4 - No.5	110.256	3.0	Friction	$0.024969Q^2$	0.225	0.225
No.5 - No.6	70.769	3.0	Friction	$0.016026Q^2$	0.144	0.144
No.6 - No.7	35.425	3.0	Friction	$0.008022Q^2$	0.072	
			Bend (HIP2)	$0.001990Q^2$	0.018	0.090
No.7 - No.8	87.331	3.0	Friction	$0.019777Q^2$	0.178	
			Bend (VIP2)	$0.000612Q^2$	0.006	0.184
No.8 - No.9	6.748	3.0	Friction	$0.001528Q^2$	0.014	
			Bend (HIP3)	$0.001327Q^2$	0.012	0.026
No.10 - No.11 (water supply pipe)	9.710	0.0	Branching	$0.000215Q^2$	0.000	
		0.0	Friction	$0.131576Q^2$	0.000	
			Bend (VIP3 & HIP4)	$0.097806Q^2$	0.000	
			Gradual contraction	$0.003240Q^2$	0.000	0.000
No.12 - No.13	6.500	0.0	Branching	$0.013886Q^2$	0.000	
			Friction	$0.088079Q^2$	0.000	0.000
No.14 - No.15	9.095	0.0	Branching	$0.004628Q^2$	0.000	
			Friction	20.139796Q	0.000	0.000
No.16 - No.17	6.360	3.0	Branching	$0.000646Q^2$	0.006	
		3.0	Friction	$0.010740Q^2$	0.097	
			Gradual contraction	$0.001210Q^2$	0.011	0.113
No.18 - No.19	12.206	3.0	Draft-tube	$0.020766Q^2$	0.187	
			Friction	$0.000137Q^2$	0.001	
			Miter bend	$0.000233Q^2$	0.002	
			Gradual contraction	$0.000143Q^2$	0.001	
			Abrupt enlargement	$0.000111Q^2$	0.001	0.193
Σ					1.227	

Table 3.6.5 Maximum Water Pressures for Different Cases

No.	Lg (m)	Ls(m)	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Maximum	
			H0 (m)	Hmax (m)	H0 (m)	Hmax (m)	H0 (m)	Hmax (m)	H0 (m)	Hmax (m)	H0 (m)	Hmax (m)	H0 (m)	Hmax (m)	H0 (m)	Hmax (m)
1	0.00	0.00	25.30	25.30	21.80	21.80	18.90	18.90	18.90	18.90	6.00	6.00	6.00	6.00	25.30	25.30
2	24.94	24.94	39.80	41.49	36.30	37.99	33.40	34.20	33.40	35.09	20.50	21.30	20.50	22.26	41.49	41.49
3	29.01	29.01	44.70	46.76	41.20	43.26	38.30	39.32	38.30	40.37	25.40	26.43	25.40	27.56	46.76	46.76
4	117.58	117.58	49.92	59.12	46.42	55.63	43.52	50.27	43.52	52.71	30.62	37.44	30.62	40.19	59.12	59.12
5	227.84	227.84	57.43	73.52	53.93	70.04	51.03	64.82	51.03	67.11	38.13	52.03	38.13	54.90	73.52	73.52
6	298.61	298.61	62.25	83.44	58.75	79.97	55.85	74.04	55.85	77.09	42.95	61.21	42.95	65.10	83.44	83.44
7	334.03	334.03	64.66	88.20	61.16	84.73	58.26	78.61	58.26	81.84	45.36	65.74	45.36	69.95	88.20	88.20
8	421.36	421.36	70.60	98.87	67.10	95.41	64.20	89.85	64.20	92.48	51.30	76.85	51.30	80.88	98.87	98.87
9	428.11	428.11	70.60	99.34	67.10	95.88	64.20	90.24	64.20	92.95	51.30	77.23	51.30	81.36	99.34	99.34
10		428.11	70.60	99.34	67.10	95.88	64.20	90.24	64.20	92.95	51.30	77.23	51.30	81.36	99.34	99.34
11		437.82	69.40	98.16	65.90	94.69	63.00	85.71	63.00	91.77	50.10	72.30	50.10	80.20	98.16	98.16
12		437.82	69.40	98.16	65.90	94.69	63.00	85.71	63.00	91.77	50.10	72.30	50.10	80.20	98.16	98.16
13		444.32	69.40	98.17	65.90	94.70	63.00	84.24	63.00	91.77	50.10	70.62	50.10	80.21	98.17	98.17
14			69.40	98.16	65.90	94.69	63.00	85.71	63.00	91.77	50.10	72.30	50.10	80.20	98.16	98.16
15			69.85	98.62	66.35	95.15	63.45	86.18	63.45	92.22	50.55	72.77	50.55	80.66	98.62	98.62
16	428.11		70.60	99.34	67.10	95.88	64.20	90.24	64.20	92.95	51.30	77.23	51.30	81.36	99.34	99.34
17	434.47		70.60	100.11	67.10	96.65	64.20	90.99	64.20	93.72	51.30	78.01	51.30	82.15	100.11	100.11
18	437.92		6.54	7.66	3.55	4.67	2.01	2.20	2.01	2.21	2.01	2.21	2.01	2.21	7.66	7.66
19	450.13		4.92	5.16	1.93	2.17	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	5.16	5.16
	486.66		4.49	4.49	1.50	1.50									4.49	4.49

Notes: NWL=normal water level; LWL=lower water level;
SWL=surcharge water level (during 100 year return period flood); MWL=maximum water level (during PMF);
Lg = distance of waterway for generation from No.1; Ls =distance of waterway for main water supply from No.1;
H0 =static head; Hmax= maximum water pressure including water hammer pressure.

Table 3.6-6 Minimum Water Pressures for Different Cases

No.	Lg (m)	Ls(m)	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Minimum	
			H0 (m)	Hmin (m)	H0 (m)	Hmin (m)	H0 (m)	Hmin (m)	H0 (m)	Hmin (m)	H0 (m)	Hmin (m)	H0 (m)	Hmin (m)	H0 (m)	Hmin (m)
1	0.00	0.00	25.30	25.30	21.80	21.80	18.90	18.9	18.90	18.9	6.00	6	6.00	6	6.00	6.00
2	24.94	24.94	39.80	38.02	36.30	34.52	33.40	32.347	33.40	31.609	20.50	19.4	20.50	18.628	19.40	18.63
3	29.01	29.01	44.70	42.52	41.20	39.01	38.30	37.19	38.30	36.103	25.40	24.249	25.40	23.104	24.25	23.10
4	117.58	117.58	49.92	40.65	46.42	37.14	43.52	42.52	43.52	34.258	30.62	29.567	30.62	20.978	29.57	20.98
5	227.84	227.84	57.43	41.21	53.93	37.69	51.03	49.13	51.03	34.807	38.13	36.222	38.13	21.201	34.81	21.20
6	298.61	298.61	62.25	41.08	58.75	37.55	55.85	53.372	55.85	34.647	42.95	40.464	42.95	20.834	34.65	20.83
7	334.03	334.03	64.66	41.13	61.16	37.60	58.26	55.424	58.26	34.704	45.36	42.516	45.36	20.795	34.70	20.80
8	421.36	421.36	70.60	43.01	67.10	39.47	64.20	60.628	64.20	36.591	51.30	47.72	51.30	22.502	36.59	22.50
9	428.11	428.11	70.60	42.95	67.10	39.41	64.20	60.525	64.20	36.539	51.30	47.617	51.30	22.442	36.54	22.44
10		428.11	70.60	42.95	67.10	39.41	64.20	60.525	64.20	36.539	51.30	47.617	51.30	22.442	36.54	22.44
11		437.82	69.40	41.56	65.90	38.01	63.00	57.204	63.00	35.129	50.10	44.296	50.10	21.031	35.13	21.03
12		437.82	69.40	41.56	65.90	38.01	63.00	57.204	63.00	35.129	50.10	44.296	50.10	21.031	35.13	21.03
13		444.32	69.40	41.45	65.90	37.91	63.00	56.286	63.00	35.021	50.10	43.379	50.10	20.913	35.02	20.91
14			69.40	41.56	65.90	38.01	63.00	57.204	63.00	35.129	50.10	44.296	50.10	21.031	35.13	21.03
15			69.85	41.90	66.35	38.36	63.45	57.653	63.45	35.47	50.55	44.746	50.55	21.363	35.47	21.36
16	428.11		70.60	42.95	67.10	39.41	64.20	60.525	64.20	36.539	51.30	47.617	51.30	22.442	36.54	22.44
17	434.47		70.60	42.87	67.10	39.33	64.20	60.382	64.20	36.456	51.30	47.475	51.30	22.352	36.46	22.35
18	437.92		6.54	5.39	3.55	2.39	2.01	1.805	2.01	1.807	2.01	1.796	2.01	1.797	1.80	1.80
19	450.13		4.92	4.68	1.93	1.69	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
	486.66		4.49	4.49	1.50	1.50									1.50	1.50

Notes: NWL=normal water level; LWL=lower water level;
SWL=surcharge water level (during 100 year return period flood); MWL=maximum water level (during PMF);
Lg = distance of waterway for generation from No.1; Ls =distance of waterway for main water supply from No.1;
H0 =static head; Hmin= minimum water pressure including water hammer pressure.

Table 3.6.7 Water Pressures at just Upstream End of Turbine

Case	Static Head H0 (m)	Maximum Water Pressure Hmax (m)	Maximum Water Hammer Pressure Hwh (m)	Ratio of Hwh/H0 (%)	Design Head (m)
1	70.60	100.11	29.51	42	101.0
2	67.10	96.65	29.55	44	
3	64.20	90.99	26.79	42	
4	64.20	93.72	29.52	46	
5	51.30	78.01	26.71	52	
6	51.30	82.15	30.85	60	

Notes: Ratio = ratio of water hammer pressure to static head;
Static head = reservoir water level minus the elevation of turbine;
Design head = Hmax + allowance

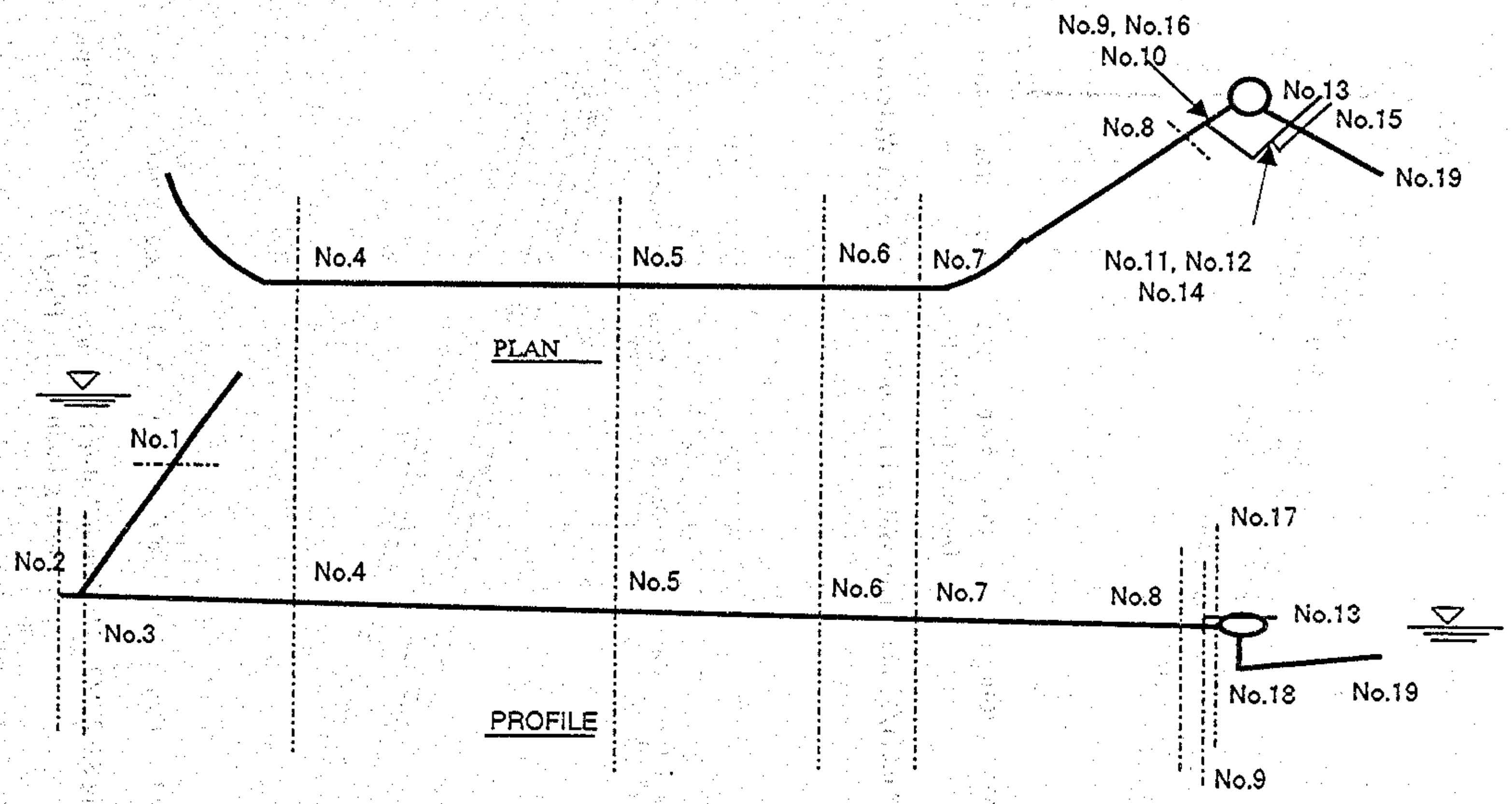


Fig.3.6.1 Schematic Profile of Outlet Works

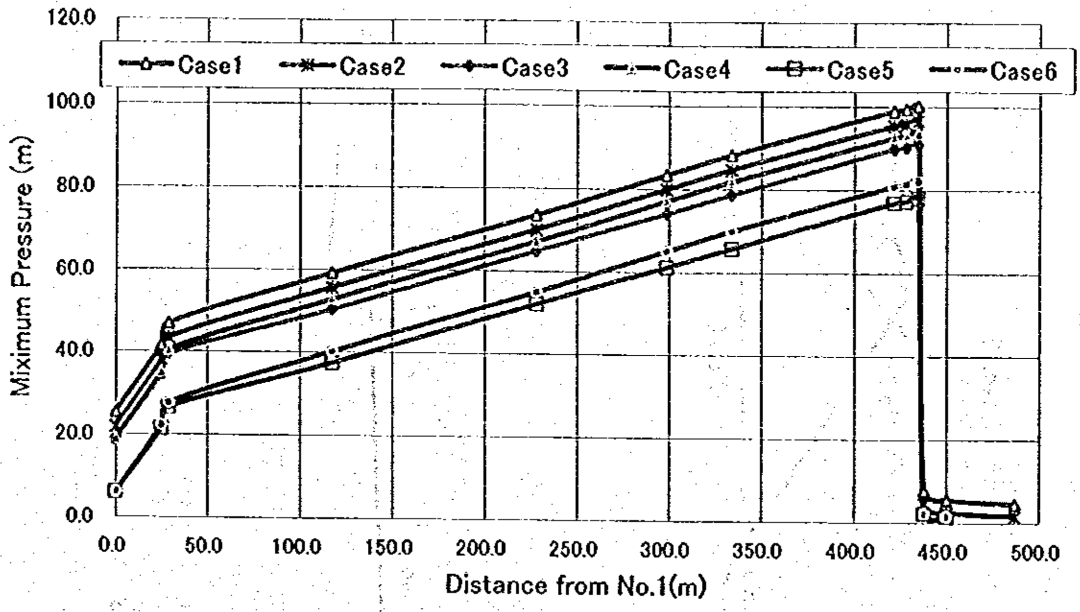


Fig.3.6.2(1/2) Maximum Water Pressures in Generation Waterway

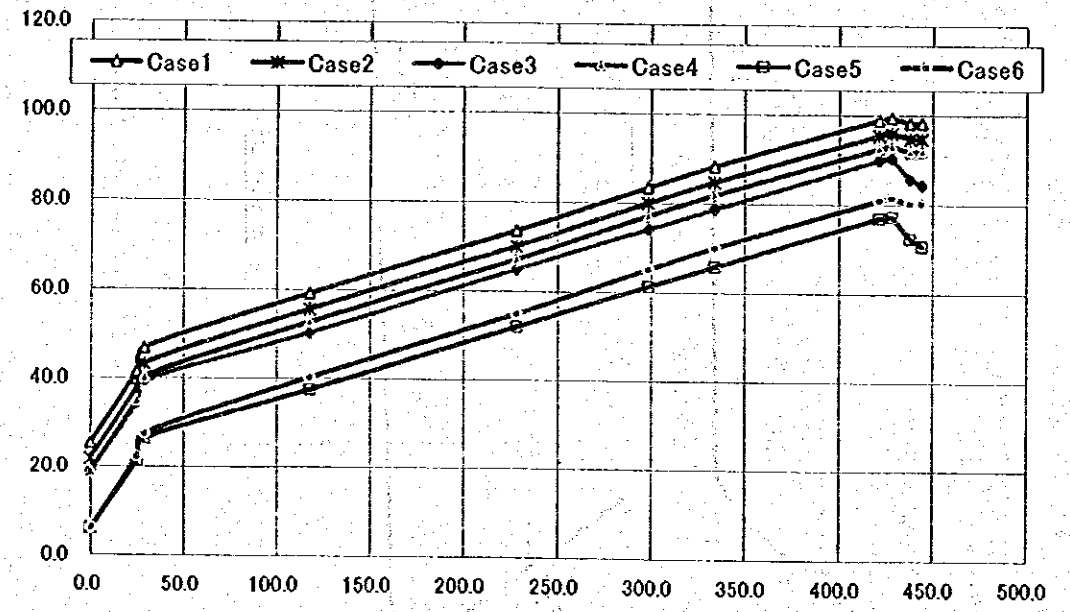


Fig.3.6.2(2/2) Maximum Water Pressures in Water Supply Pipe (including Main Outlet Pipe)

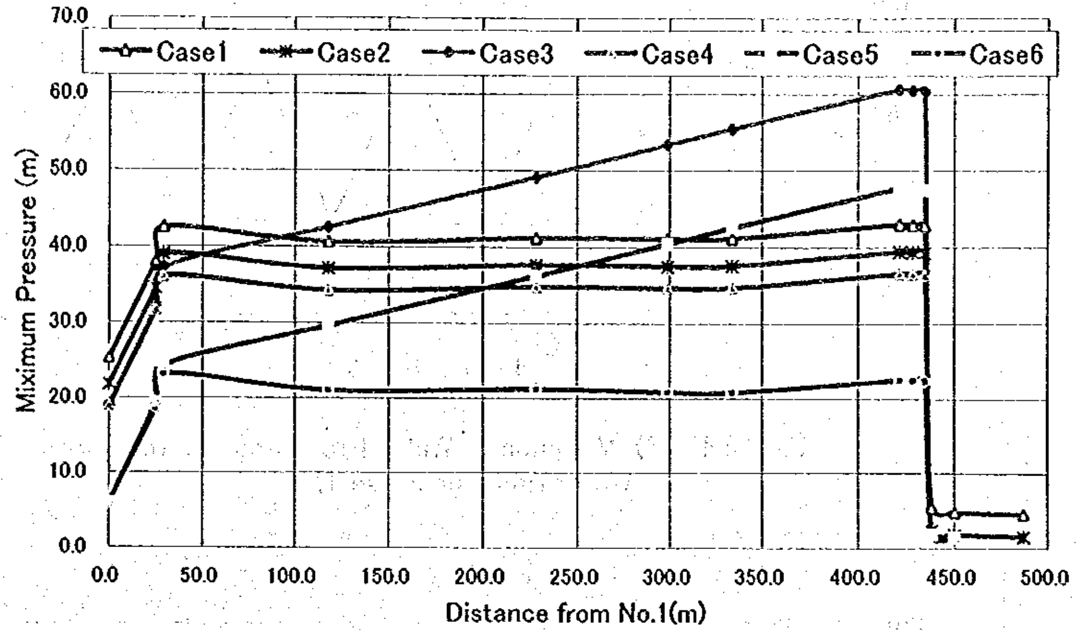


Fig.3.6.3(1/2) Minimum Water Pressures in Generation Waterway

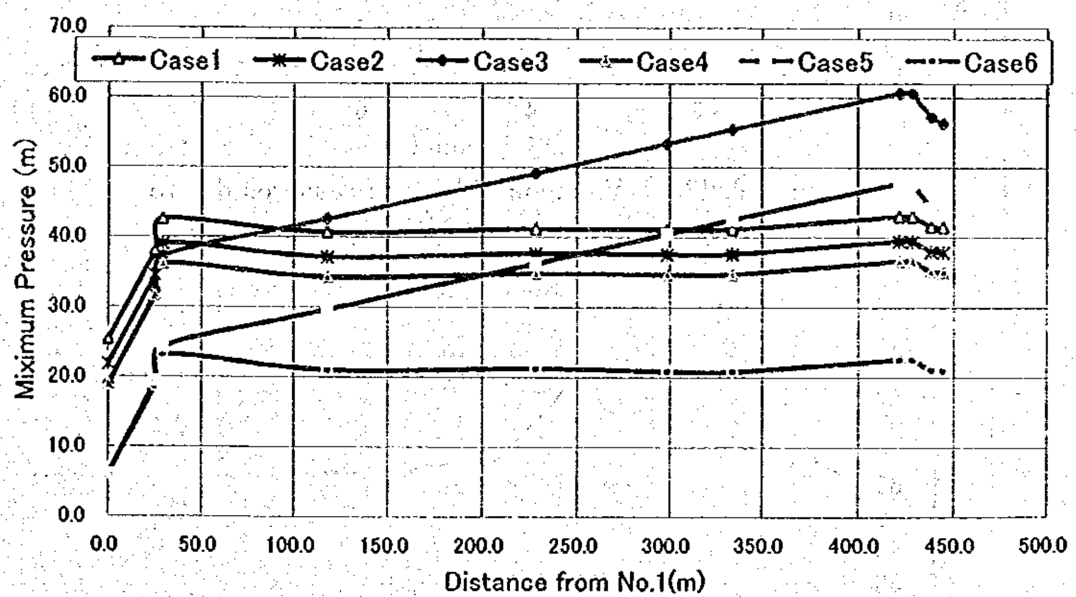


Fig.3.6.3(2/2) Minimum Water Pressures in Water Supply Pipe (including Main Outlet Pipe)

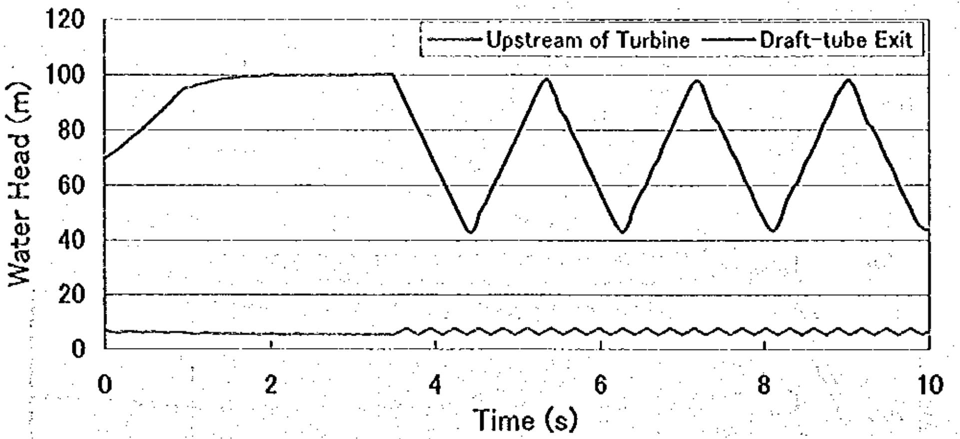


Fig.3.6.4(1/6) Variance of Water Pressures due to Water Hammer (Case 1)

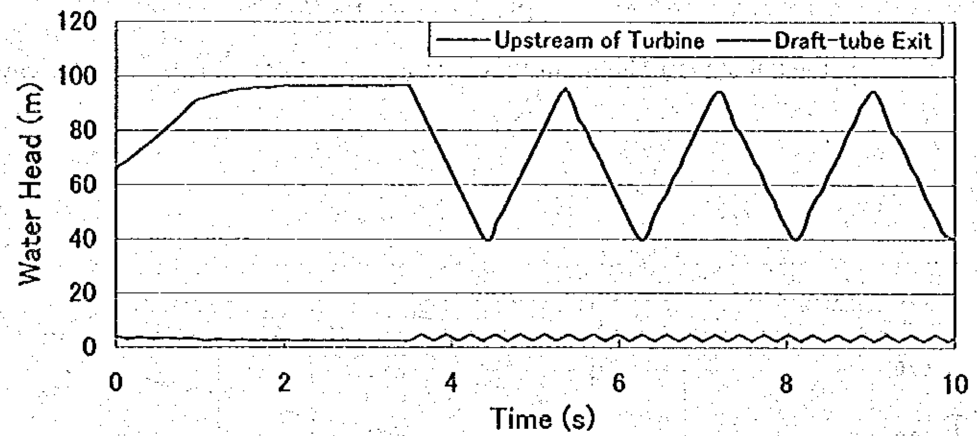


Fig.3.6.4(2/6) Variance of Water Pressures due to Water Hammer (Case 2)

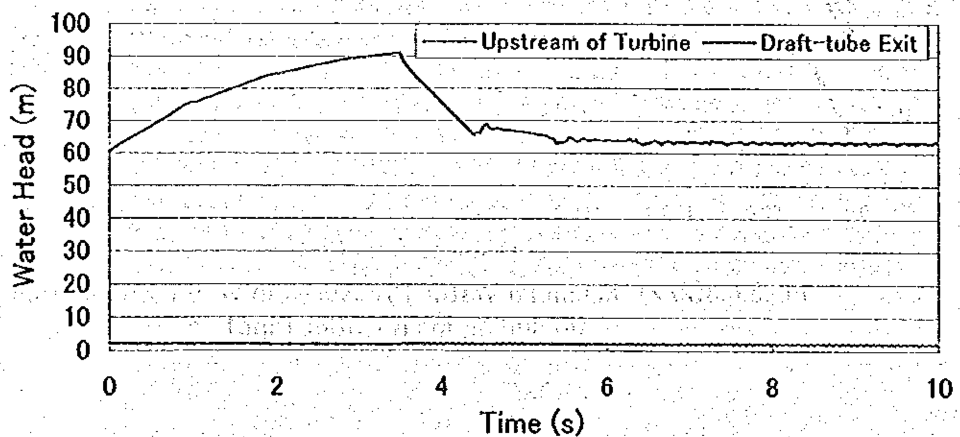


Fig.3.6.4(3/6) Variance of Water Pressures due to Water Hammer (Case 3)

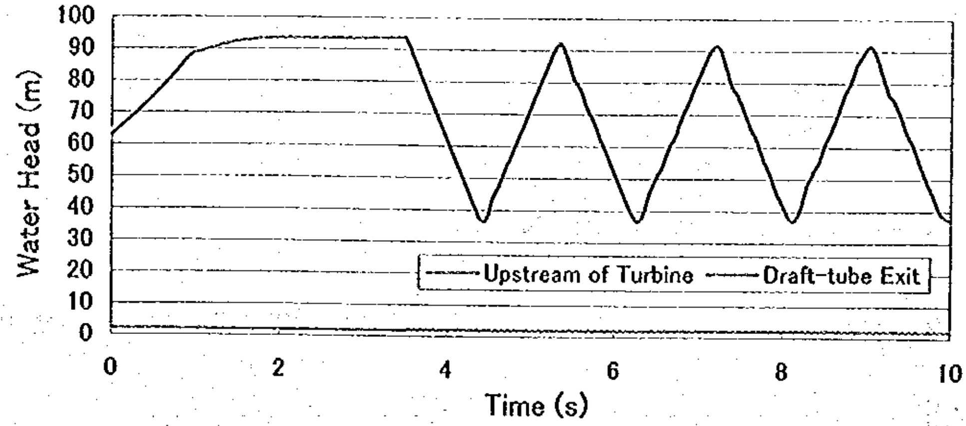


Fig.3.6.4(4/6) Variance of Water Pressures due to Water Hammer (Case 4)

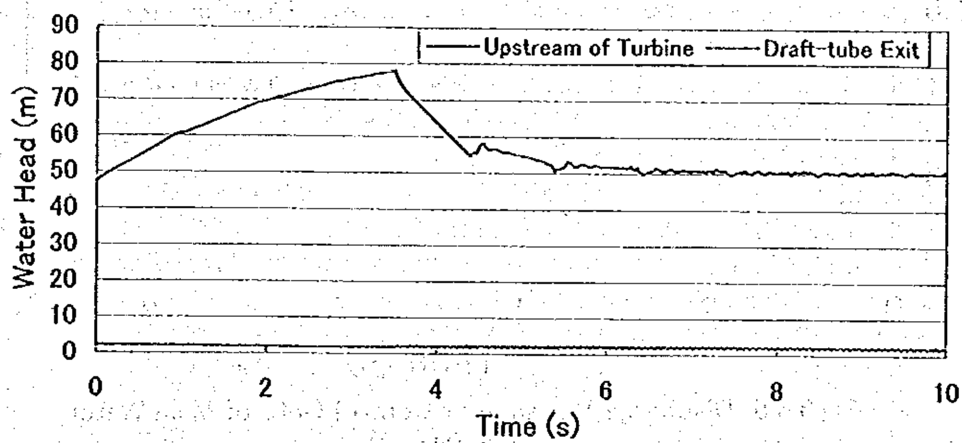


Fig.3.6.4(5/6) Variance of Water Pressures due to Water Hammer (Case 5)

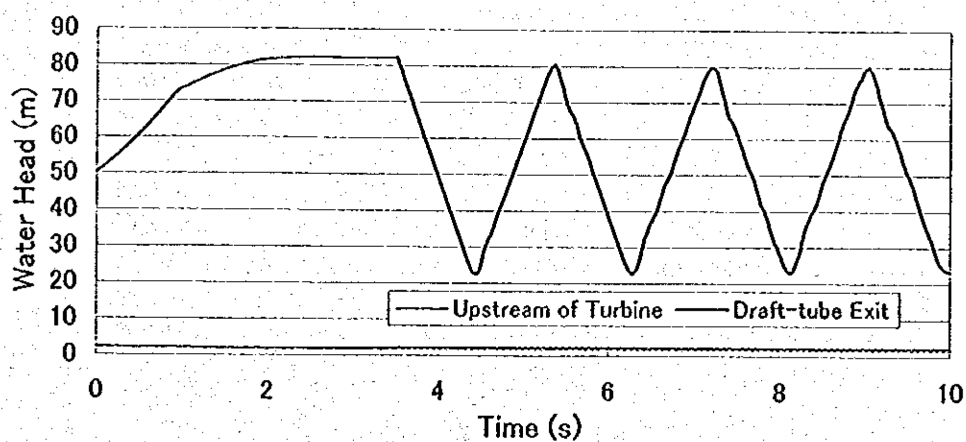


Fig.3.6.4(6/6) Variance of Water Pressures due to Water Hammer (Case 6)

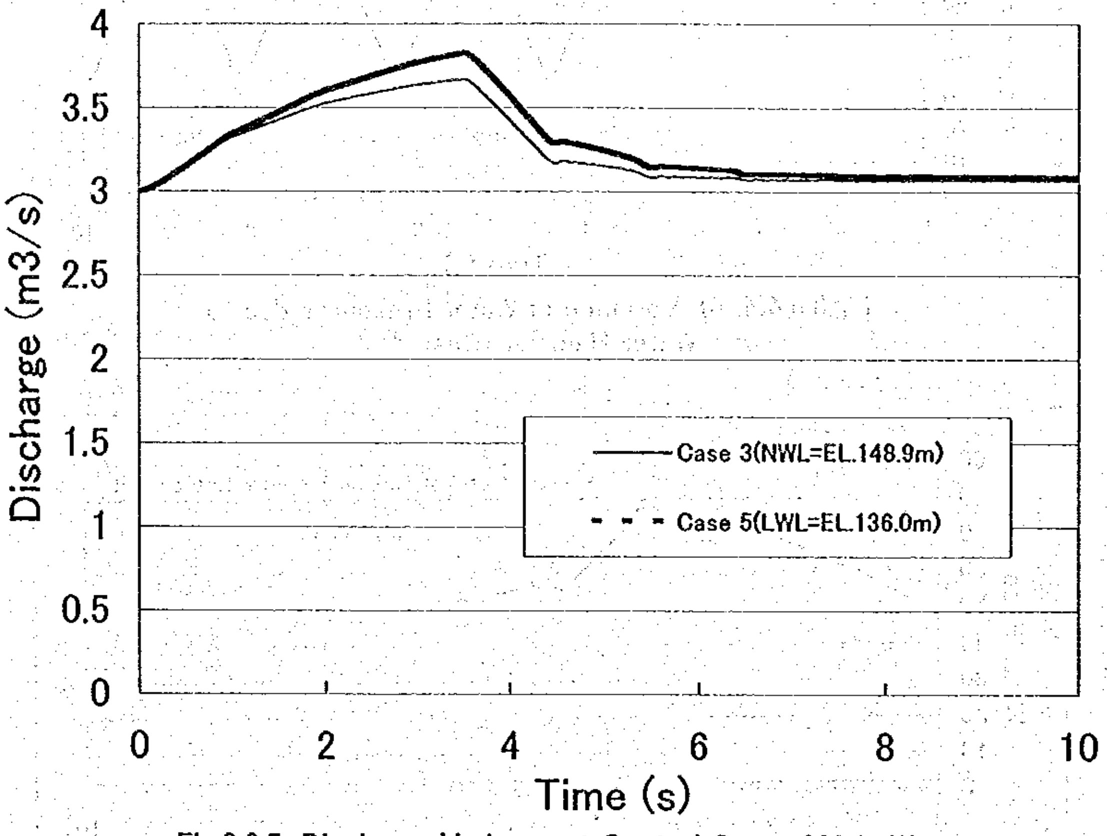


Fig.3.6.5 Discharge Variance at Control Gate of Main Water Supply (No.13)

3.6.2 Design of Penstock

(1) General

Longitudinal profile and section of penstock are shown in Fig.3.6.6.

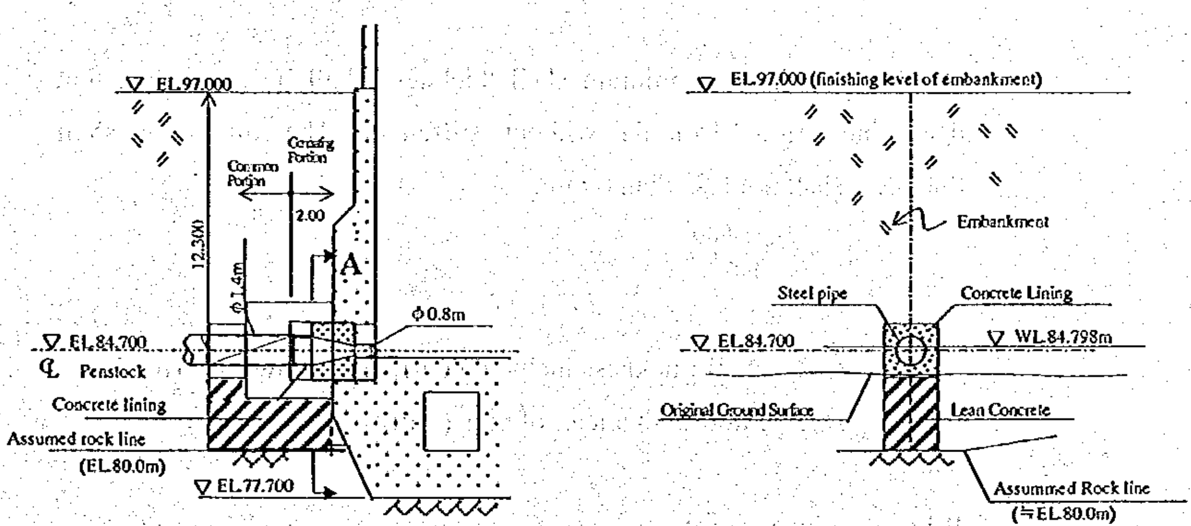
Lean concrete shall be placed for the foundation to prevent settlement of penstock. Embankment shall be conducted up to EL 97m.

(2) Design head

The maximum internal pressure acting on the steel penstock is sum of hydrostatic pressure and pressure increase by water hammering as shown below.

Hydrostatic pressure	70.60m
Pressure increase by water hammering	29.41m
Maximum internal pressure	100.10m

Notes: Case-1 Reservoir water level: 155.30m (during PMF)
Tailrace water level: 87.79m (during PMF), Q=3.0m^3/sec
EL of turbine center, EL of penstock center: 84.70m



A-A Section

Fig. 3.6.6 Profile and Section of Penstock (Generating Portion)

(3) Load conditions

The penstock steel liners shall be designed for the following conditions.

1) When water is fully filled in the pipe
2) When the pipe is empty

1) When water is fully filled in the pipe.

In case that when water is fully filled in the pipe, internal pressure is most influential load.

Tensile stress due to internal pressure is calculated as follows;

$$\sigma = \frac{PD}{2t} \text{ or } \frac{PD}{2(t_0 - \varepsilon)}$$

where, σ: stress (kgf/cm^2)

P: maximum hydraulic pressure at a place to determine stress (kgf/cm^2)

D: internal diameter subtracting the 1/2 of corrosion allowance from the internal surface of the pipe (cm)

t: thickness excluding corrosion allowance = t_0-ε (cm)

ε: allowance thickness for corrosion and wear (=0.15cm)

On the other hand, minimum shell thickness shall be more than that determined from formula without stiffeners. The minimum shell thickness shall not less than 6 mm.

$$t_{min} = \frac{D_0 + 800}{400}$$

where, t_{min}: shell thickness including corrosion allowance. (mm)

D_0: internal diameter of pipe (mm)

2) When the pipe is empty

Since lining concrete will be placed around steel pipe, external pressure acts on concrete lining, so no stress in the pipe will generate due to the external pressure.

(4) Calculation results

Allowable strength of penstock steel (SM400)

Item	Thickness < 16mm	16mm <thickness < 40mm	thickness > 40mm
Allowable tensile stress	1,350kgf/cm²	1,300kgf/cm²	1,150kgf/cm²

Calculation results are given in the table below.

description		Unit	Section-A	Section-B
σ_{as}	allowable tensile stress	kgf/cm²	1,350	1,350
η	joint efficiency		0.90	0.90
σ_{a0}	design allowable stress	kgf/cm²	1,215	1,215
P	internal maximum pressure	kgf/cm²	10.01	10.01
D_0	inside diameter	Cm	140	80
t_0	thickness (including ε)	Mm	8.0	8.0
ε	corrosion allowance	Mm	1.5	1.5
σ_s	calculated stress	kgf/cm²	1,078	616

3.6.3 Design of Concrete Lining for Steel Pipe

(1) Design parameter and structural model

Unit weight

Unit weight of reinforced concrete	2.5 tf/m³
Unit weight of embankment (saturated)	1.9 tf/m³
Unit weight of embankment (in water)	0.9tf/m³
Water	1.0 tf/m³

Earth pressures

coefficient of earth pressure 0.5

Embankment and ground water levels

Embankment surface level: EL. 97.0 m.

Ground water level is assumed to be at EL 84.798 m (corresponding to the 100 year return period flood)

Ground water level is assumed to be at EL 84.798 m
(corresponding to the 100 year return period flood)

Allowable strength of concrete and reinforcement:

	Item	Normal (kgf/cm²)	Earthquake (kgf/cm²)	Remarks
Concrete (K-225 class)	Compressive strength, σ_{ck} (28th day)	225		
	Allowable bending compressive stress	75 *1	112 *2	*1 $\sigma ca = \sigma_{ck}/3$ *2 $\sigma cy = 1.5\ \sigma ca$
	Allowable shearing stress	8	12*3	*3 $\tau\ cy = 1.5\ \tau\ ca$
Reinforcement	Allowable tensile stress	1,800	2,700 *4	*4 $\sigma sy = 1.5\ \sigma sa$

The structural model for analysis is shown in Fig. 3.6.7.

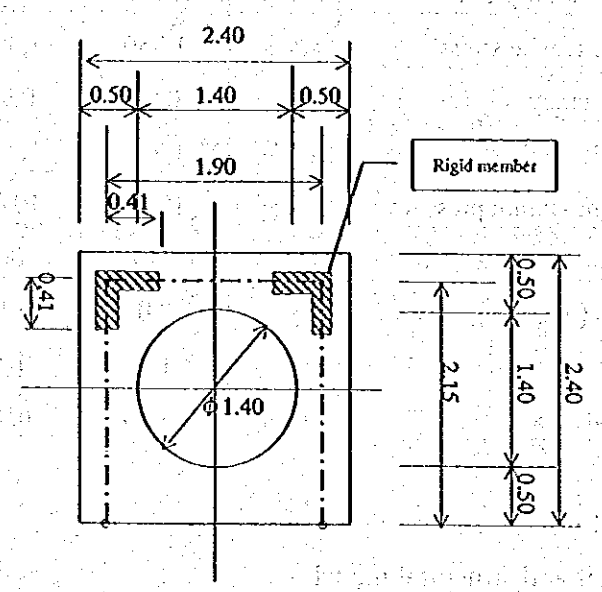


Fig 3.6.7 Structural Model

(2) Load conditions

Following loads are considered as shown in Fig. 3.6.8.

(i) Dead Load

(ii) Surface Live Load

(iii)Earth and Water Pressure

1) Earth pressure

Earth pressure acting on buried concrete lining is considered as follows:

1) Earth pressure

Earth pressure acting on buried concrete lining is considered as follows:

Case-1 : For long-term conditions

$$P_V = \gamma \cdot h$$

The vertical (Pv) and horizontal (Ph) earth pressures are given below ;

$$P_H = K \cdot \gamma \cdot h$$

where, K : coefficient of steady earth pressure (=0.5)

γ : unit weight of soil (1.9tf/m^3)

h : height of soil fill

Case-2 : When embankment is completed

In case of the rigid foundation, vertical earth pressure is bigger than weight of covering soil due to existence of downward friction force in the column of covering soil above the structure. With time passes, earth pressure will be stabilized in the above condition.

AASHO gives the following formulae:

h>1.7 B

$$P_V = \gamma(1.9h - 0.97B)$$

h<1.7B

$$P_V = 2.59B \cdot \gamma(e^k - 1)$$

$$K = 0.395h / B$$

where, B : width of the structure

γ : unit weight of soil

h : height of soil fill

Surface live load

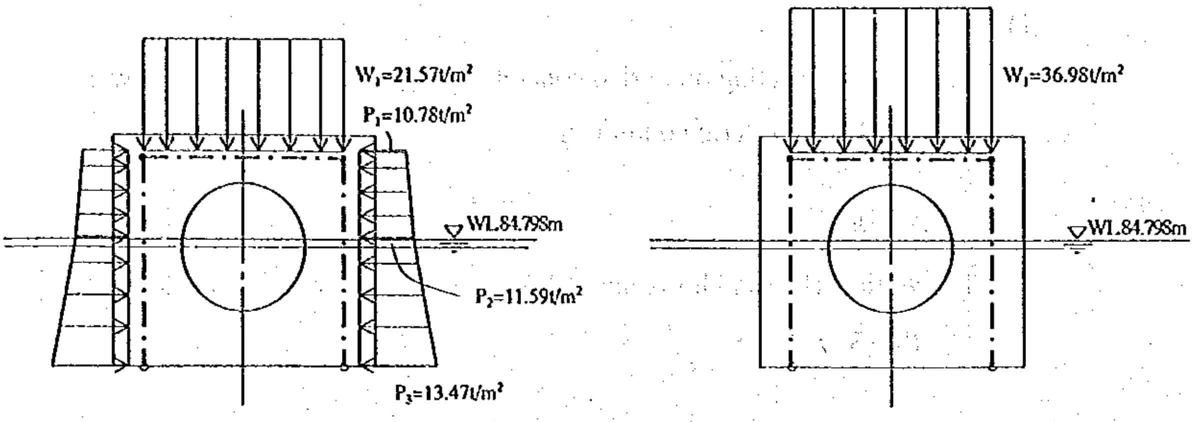
The vertical (Pv) and horizontal (Ph) load by surface live load are given as follows;

$$P_V = q$$

$$P_H = K \cdot q$$

where, K : coefficient of steady earth pressure (=0.5)

q : surface live load (1tf/m^2)



1) For long-term conditions 2) When embankment is completed

Fig. 3.6.8 Earth and Water Pressures

(3) Calculation results

The results of the structural analysis for each case are shown in Fig.3.6.9.
The reinforcement arrangements are given in the following tables and Fig.3.6.10.

Mem.	Sec.	Load case *1	Moment *2 (tf-m)	Normal Force*3 (tf)	Shear Force (tf)	Amount of Reinforcement		Stress of Steel and Concrete (kgf/cm²)		
						Outer side	Inner side	Concrete	Steel	Shear
Upper	Side	1	-2.84.	17.65	12.86	D16@0.3 m	D22@0.3 m	12	60	2.0
		2	2.57	4.92	21.19	(6.619cm²/m))	(12.903cm²/m)	13	350	3.2
	Center	1	0.63	17.65	0	D16@0.3 m	D22@0.3 m	5	80	0.0
		2	8.29	4.92	0	(6.619cm²/m))	(12.903cm²/m)	42	1560	0.0
Side	Top	1	-3.84	23.14	12.94	D16@0.3 m	-	17	110	2.0
		2	-7.44	37.78	4.71	(6.619cm²/m))	-	35	400	0.7
	Center	1	3.11	24.50	0.46	D16@0.3 m	-	13	10	0.1
		2	-5.41	38.34	4.49	(6.619cm²/m))	-	23	70	1.0

Notes: *1 : Case-1 : For long-term conditions Case-2 : When embankment is completed
*2 : + tension in inside - : tension in outside
*3 : + tension - : compression

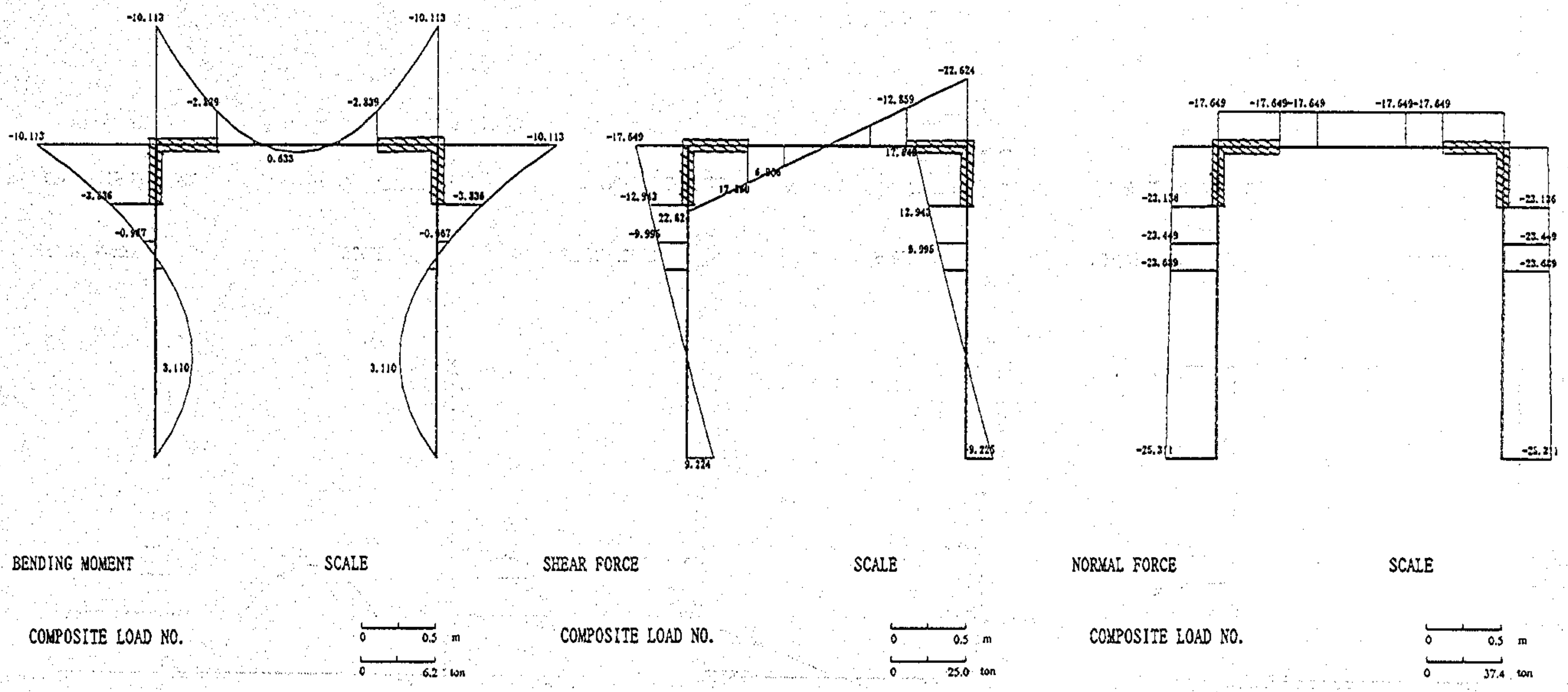


Fig.3.6.9(1/2) Sectional Force (Case-A : for long-term conditions)

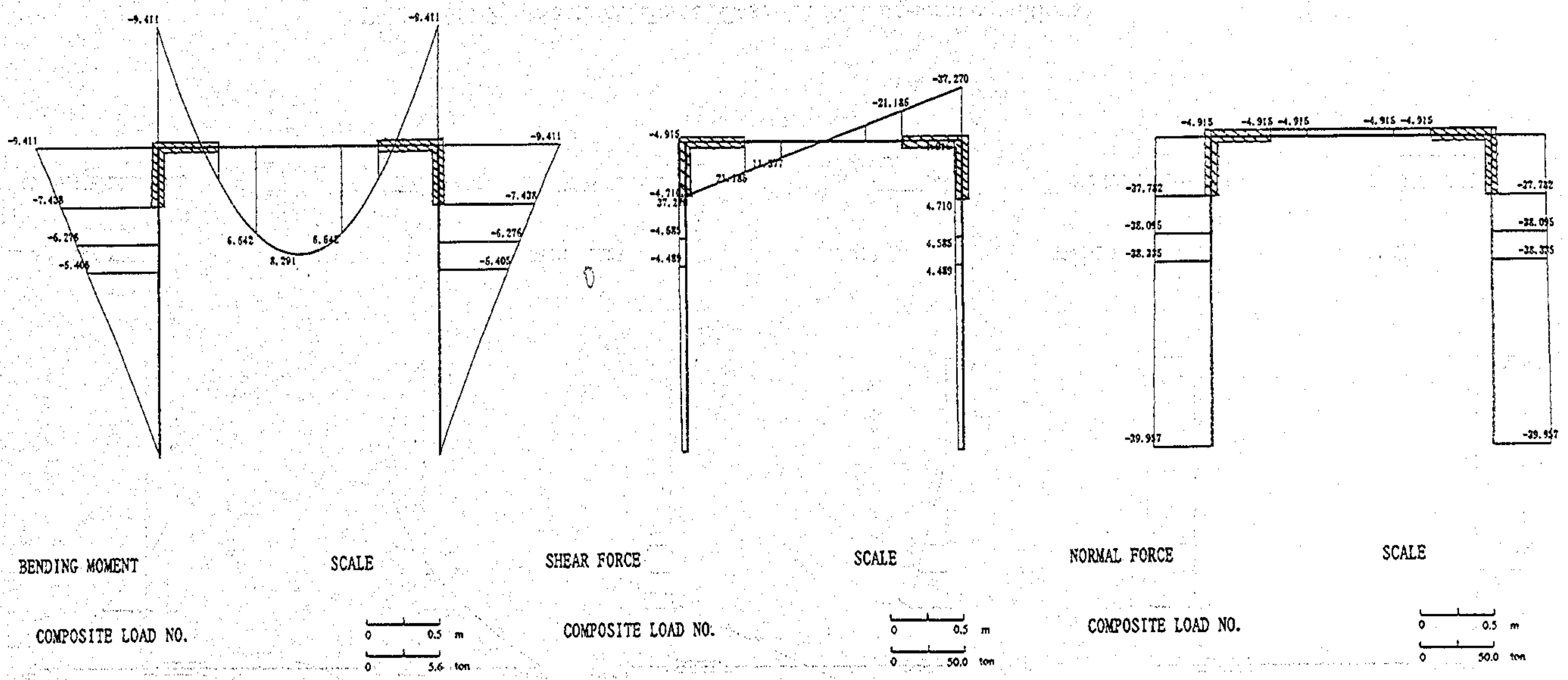


Fig.3.6.9(2/2) Sectional Force (Case-B : when embankment is completed)

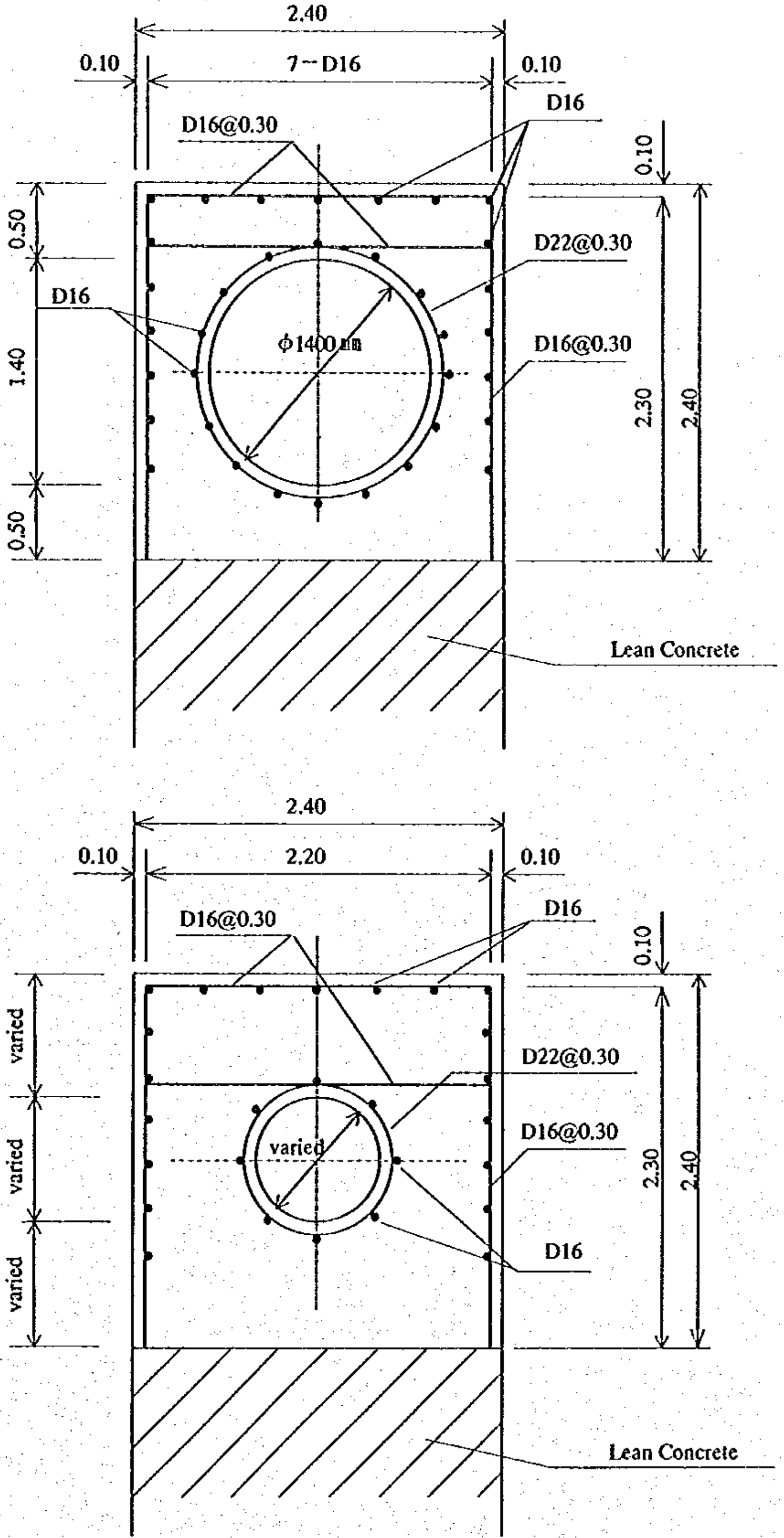


Fig3.6.10 Reinforcement Arrangement of Lining Concrete of Penstock
(Generating Portion)