A-10 Trenches

Tab.A-10-1 List of Trenches

A-10 Trenches

Tab.A-10-1 List of Trenches

Table A-10-1 List of Trenches

Location	No. of Trenches	Name of Trenches
Penstock Route "A"	22	T-3, T-4, T-15, T-16, T-17, T-18, T-20, T-21, T-22, T-23, T-25, T-26, T-27, T-28 T-29, T-30, T-31, T-32, T-33, T-34, T-35, T-33B
Penstock Route "B"	25	T-3A, T-4A, T-4B, T-5A, T-6A, T-7A, T-8A, T-9A, T-10A, T-11A, T-11B, T-12A, T-13A T-13B, T-14A, TE-2, TE-3, TE-4, TE-4A, TE-4B, TE-4C, TE-5, TE-6, TE-7, TE-8
P/S Site "A"	28	T-1, T-2, T-4-2, T-5, T-6, T-6-2, T-7, T-8, T-9, T-10, T-11, T-12, T-13, T-1A, T-2A C-1, C-2, C-3, C-4, C-5, C-6, C-7, C-8, C-9, C-10, C-11, C-12, C-13
P/S Site Right Bank	3	T-23, T-E, T-F
Total	78	

Note:Detail data have not been offered yet.

A-10 Trenches

Tab.A-10-1 List of Trenches

A-10 Trenches

Tab.A-10-1 List of Trenches

Table A-10-1 List of Trenches

Location	No. of Trenches	Name of Trenches
Penstock Route "A"	22	T-3, T-4, T-15, T-16, T-17, T-18, T-20, T-21, T-22, T-23, T-25, T-26, T-27, T-28 T-29, T-30, T-31, T-32, T-33, T-34, T-35, T-33B
Penstock Route "B"	25	T-3A, T-4A, T-4B, T-5A, T-6A, T-7A, T-8A, T-9A, T-10A, T-11A, T-11B, T-12A, T-13A T-13B, T-14A, TE-2, TE-3, TE-4, TE-4A, TE-4B, TE-4C, TE-5, TE-6, TE-7, TE-8
P/S Site "A"	28	T-1, T-2, T-4-2, T-5, T-6, T-6-2, T-7, T-8, T-9, T-10, T-11, T-12, T-13, T-1A, T-2A C-1, C-2, C-3, C-4, C-5, C-6, C-7, C-8, C-9, C-10, C-11, C-12, C-13
P/S Site Right Bank	3	T-23, T-E, T-F
Total	78	

Note:Detail data have not been offered yet.

FEASIBILITY DESIGN

（フィージビリティ設計）

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FEASIBILITY DESIGN

（フィージビリティ設計）

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APPENDIX FEASIBILITY DESIGN

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1. Diversion Tunnel

1) Calculation of Diversion Tunnel Capacity

a) Out line of Diversion

Items	Unit	Descrip-tion	Remarks
Design flood	m3/sec	410. 0	10 year return period
Length of tunnel	m	225. 0	Left bank
Cofferdam type	-	Concrete gravity dam	

b) Calculation of Capacity

b)-1 D = 5. 50 m

(1) Shape of Closs Section and Dimensions

Shape of the diversion tunnel is applied Horseshoe type section as shown in attached figure.

r: Radius = D / 2	(m)	2. 750
r1 = r2 = 2r		5. 500
$\theta 1 = \theta 2$	(Degree)	24. 295278
π :Pi		3
H1= r1 * sinθ 1	(m)	2. 2629
H2= r - H1	(m)	0. 4871
B1= r1 * (1-cosθ 1)	(m)	0. 4871
B2= r1 - B1	(m)	5. 0129
Ho= r * cosθ o	(m)	

(2) Calculation of Flowing Area and Wetted Perimeter

(2)-1 Calculation of Flowing Area

$$A = Ao - ((\theta o * \pi * r^2/180) - ho*(r^2-ho^2)^{(1/2)})$$

Where Ao : Inner cross-section area = 3. 317 * r^2 (m2)
ho : Water depth from the tunnel horizontal axis (m)
θ o : Acos(ho / r)

(2)-2 Calculation of Wetted Perimeter

$P = Po - (2*\theta o*\pi*r/180)$

Where   Po : Total perimeter length = 6.543 * r   (m)

(3) Calculation of Capacity (Discharge)

(3)-1 Open Flow

$Q = A * V = A * (1/n) * (I)^{(1/2)} * (R)^{(2/3)}$

Where   n : Coefficient of roughness (Concrete)   0.01400
I : Gradient of tunnel(slope)=1 / s   0.05556
s =   18.00
R : Hydrouric depth = A / P

r =   2.750 m
Ao = 3.317 * r^2 =   25.085 m2
Po = 6.543 * r =   17.993 m

ho (m)	H=r+ho (m)	A (m2)	P (m)	R=A/P (m)	V (m/sec)	Q (m3/sec)
0.20	2.95	14.303	9.753	1.4666	21.732	310.846
0.40	3.15	15.397	10.155	1.5161	22.219	342.096
0.60	3.35	16.478	10.562	1.5601	22.646	373.172
0.80	3.55	17.542	10.976	1.5982	23.014	403.697
1.00	3.75	18.581	11.400	1.6300	23.318	433.264
1.20	3.95	19.589	11.836	1.6550	23.556	461.430
1.40	4.15	20.558	12.291	1.6727	23.723	487.700
1.60	4.35	21.480	12.768	1.6823	23.814	511.514
1.80	4.55	22.344	13.278	1.6828	23.819	532.208
2.00	4.75	23.138	13.832	1.6728	23.725	548.957
2.20	4.95	23.848	14.453	1.6500	23.508	560.628
2.40	5.15	24.450	15.187	1.6099	23.126	565.410
2.50	5.25	24.699	15.629	1.5803	22.842	564.170
2.60	5.35	24.905	16.168	1.5404	22.455	559.240
2.65	5.40	24.986	16.505	1.5139	22.197	554.622
2.70	5.45	25.050	16.942	1.4785	21.850	547.347
2.75	5.50	25.085	17.993	1.3942	21.011	527.056

(3)-2 Pipe Flow

In case of the piping flow for the diversion tunnel, relation between the river water level and the tunnel discharge is estimated with the following formulas:

$$H1 = v^2 * (1 + f) * 1/2g + H2$$

$$Q = \{ A^2 * 2g * (H1 - H2)/(1 + f)\}^{(1/2)}$$

Where
H1 : River water level at the inlet
v : Average velocity in the tunnel = Q / A
Q : Discharge
A : Inner cross-sectional area = 3.317 * (D/2)^2
H2 : River water level at the outlet
= EL 421.00 + D
f : Coefficient of loss

$$f = fe + ft$$

fe : Coeffient of loss at the inlet = 0.200
(with Circular bell mouth)
ft : Tunnel friction loss

$$ft = 2g * n^2 * L / D^{(4/3)}$$

n : Coeffient of roughness = 0.014
L : Length of diverdion = 225.000
D : Tunnel diameter 5.500

H1 (m)	H2=D (m)	D=2r (m)	A (m2)	f	Q (m3/sec)
6.5	6	6	25.085	0.289032	97.815
7.5	6	6	25.085	0.289032	138.332
8.5	6	6	25.085	0.289032	169.421
9.5	6	6	25.085	0.289032	195.631
10.5	6	6	25.085	0.289032	218.722
11.5	6	6	25.085	0.289032	239.598
12.5	6	6	25.085	0.289032	258.795
13.5	6	6	25.085	0.289032	276.664
14.5	6	6	25.085	0.289032	293.446
15.5	6	6	25.085	0.289032	309.319
16.5	6	6	25.085	0.289032	324.417
17.5	6	6	25.085	0.289032	338.842
18.5	6	6	25.085	0.289032	352.678
19.5	6	6	25.085	0.289032	365.992
20.5	6	6	25.085	0.289032	378.837
21.5	6	6	25.085	0.289032	391.261
22.5	6	6	25.085	0.289032	403.303
23.5	6	6	25.085	0.289032	414.995
24.0	6	6	25.085	0.289032	420.720
24.5	6	6	25.085	0.289032	426.367

b)-2 D = 6.00 m

(1) Shape of Closs Section and Dimensions

Shape of the diversion tunnel is applied Horseshoe type section as shown in attached figure.

r: Radius = D / 2	(m)	3.000
r1 = r2 = 2r		6.000
θ1= θ2	(Degree)	24.295278
π :Pi		3
H1= r1 * sinθ1	(m)	2.4686
H2= r - H1	(m)	0.5314
B1= r1 * (1-cosθ1)	(m)	0.5314
B2= r1 - B1	(m)	5.4686
Ho= r * cosθo	(m)	

(2) Calculation of Flowing Area and Wetted Perimeter

(2)-1 Calculation of Flowing Area

A = Ao - {(θo * π * r ^2/180) - ho*(r^2-ho^2) ^(1/2)}

Where Ao : Inner cross-section area = 3.317 * r^2 (m2)
ho : Water depth from the tunnel horizontal axis (m)
θo : Acos(ho / r)

(2)-2 Calculation of Wetted Perimeter

P = Po - (2*θo*π*r/180)

Where Po : Total perimeter length = 6.543 * r (m)

(3) Calculation of Capacity (Discharge)

(3)-1 Open Flow

Q = A * V = A * (1/n) * (I) ^(1/2) * (R) ^(2/3)

Where n : Coefficient of roughness (Concrete) 0.01400
I : Gradient of tunnel(slope)=1 / s 0.05556
s = 18.0
R : Hydrouric depth = A / P

r = 3.000 m
Ao = 3.317 * r^2 = 29.853 m2
Po = 6.543 * r = 19.629 m

ho (m)	H=r+ho (m)	A (m2)	P (m)	R=A/P (m)	V (m/sec)	Q (m3/sec)
0.20	3.20	16.913	10.603	1.5951	22.984	388.731
0.40	3.40	18.107	11.005	1.6453	23.464	424.855
0.60	3.60	19.290	11.411	1.6904	23.891	460.860
0.80	3.80	20.457	11.823	1.7303	24.265	496.385
1.00	4.00	21.601	12.242	1.7645	24.584	531.042
1.20	4.20	22.718	12.672	1.7927	24.845	564.416
1.40	4.40	23.799	13.116	1.8144	25.045	596.052
1.60	4.60	24.838	13.579	1.8292	25.181	625.442
1.80	4.80	25.826	14.065	1.8363	25.246	652.011
2.00	5.00	26.755	14.582	1.8348	25.232	675.079

(3)-2 Pipe Flow

In case of the piping flow for the diversion tunnel, relation between the river water level and the tunnel discharge is estimated with the following formulas:

H1 = v^2 * (1 + f) * 1/2g + H2

Q = [A^2 * 2g * (H1 - H2)/(1 + f)]^(1/2)

Where H1 : River water level at the inlet
v : Average velocity in the tunnel = Q / A
Q : Discharge
A : Inner cross-sectional area = 3.317 * (D/2)^2
H2 : River water level at the outlet
f : Coefficient of loss

f = fe + ft

fe : Coeffient of loss at the inlet = 0.200
(with Circular bell mouth)
ft : Tunnel friction loss

ft = 2g * n^2 * L / D^(4/3)

n : Coeffient of roughness = 0.014
L : Length of diversion = 225.000
D : Tunnel diameter 6.000

H1 (m)	H2=D (m)	D=2r (m)	A (m2)	f	Q (m3/sec)
6.5	6.00	6.00	29.853	0.279279	82.626
7.5	6.00	6.00	29.853	0.279279	143.113
8.5	6.00	6.00	29.853	0.279279	184.758
9.5	6.00	6.00	29.853	0.279279	218.609
10.5	6.00	6.00	29.853	0.279279	247.879
11.5	6.00	6.00	29.853	0.279279	274.040
12.5	6.00	6.00	29.853	0.279279	297.913
13.5	6.00	6.00	29.853	0.279279	320.010
14.5	6.00	6.00	29.853	0.279279	340.677
15.5	6.00	6.00	29.853	0.279279	360.160
16.5	6.00	6.00	29.853	0.279279	378.641
17.5	6.00	6.00	29.853	0.279279	396.262
18.5	6.00	6.00	29.853	0.279279	413.132
19.0	6.00	6.00	29.853	0.279279	421.313
19.5	6.00	6.00	29.853	0.279279	429.339

b)-3 D = 6.50 m

(1) Shape of Closs Section and Dimensions

Shape of the diversion tunnel is applied Horseshoe type section as shown in attached figure.

r : Radius = D / 2 (m)	3.250
r1 = r2 = 2r	6.500
θ1 = θ2 (Degree)	24.295278
π : Pi	3
H1 = r1 * sinθ1 (m)	2.6744
H2 = r - H1 (m)	0.5756
B1 = r1 * (1-cosθ1) (m)	0.5757
B2 = r1 - B1 (m)	5.9243
Ho = r * cosθo (m)	

(2) Calculation of Flowing Area and Wetted Perimeter

(2)-1 Calculation of Flowing Area

A = Ao - {(θo * π * r^2/180) - ho*(r^2-ho^2)^(1/2)}

Where Ao : Inner cross-section area = 3.317 * r^2 (m2)
ho : Water depth from the tunnel horizontal axis (m)
θo : Acos(ho / r)

(2)-2 Calculation of Wetted Perimeter

P = Po - (2*θo*π*r/180)

Where Po : Total perimeter length = 6.543 * r (m)

(3) Calculation of Capacity (Discharge)

(3)-1 Open Flow

Q = A * V = A * (1/n) * (I)^(1/2) * (R)^(2/3)

Where n : Coefficient of roughness (Concrete) 0.01400
I : Gradient of tunnel(slope)=1 / s 0.05556
s = 18.0
R : Hydrouric depth = A / P

r = 3.250 m
Ao = 3.317 * r^2 = 35.036 m2
Po = 6.543 * r = 21.265 m

ho (m)	H=r+ho (m)	A (m2)	P (m)	R=A/P (m)	V (m/sec)	Q (m3/sec)
0.20	3.45	19.742	11.454	1.7236	24.202	477.792
0.40	3.65	21.036	11.856	1.7743	24.675	519.064
0.60	3.85	22.320	12.261	1.8205	25.101	560.267
0.80	4.05	23.590	12.670	1.8618	25.480	601.053
1.00	4.25	24.839	13.087	1.8980	25.809	641.055
1.20	4.45	26.062	13.512	1.9288	26.087	679.876
1.40	4.65	27.253	13.949	1.9538	26.312	717.091
1.60	4.85	28.406	14.400	1.9727	26.481	752.234
1.80	5.05	29.514	14.869	1.9849	26.590	784.787
2.00	5.25	30.568	15.363	1.9898	26.634	814.158

(3)-2 Pipe Flow

In case of the piping flow for the diversion tunnel, relation between the river water level and the tunnel discharge is estimated with the following formulas:

H1 = v^2 * (1 + f) * 1/2g + H2

Q = { A^2 * 2g * (H1 - H2)/(1 + f)}^(1/2)

Where H1 : River water level at the inlet
v : Average velocity in the tunnel = Q / A
Q : Discharge
A : Inner cross-sectional area = 3.317 * (D/2)^2
H2 : River water level at the outlet
f : Coefficient of loss

f = fe + ft

fe : Coeffient of loss at the inlet = 0.200
(with Circular bell mouth)
ft : Tunnel friction loss

ft = 2g * n^2 * L / D^(4/3)

n : Coeffient of roughness = 0.014
L : Length of diversion = 225.000
D : Tunnel diameter 6.500

H1 (m)	H2=D (m)	D=2r (m)	A (m2)	f	Q (m3/sec)
6.5	6.50	6.50	35.036	0.271254	0.000
7.5	6.50	6.50	35.036	0.271254	137.570
8.5	6.50	6.50	35.036	0.271254	194.553
9.5	6.50	6.50	35.036	0.271254	238.278
10.5	6.50	6.50	35.036	0.271254	275.140
11.5	6.50	6.50	35.036	0.271254	307.616
12.5	6.50	6.50	35.036	0.271254	336.977
13.5	6.50	6.50	35.036	0.271254	363.976
14.5	6.50	6.50	35.036	0.271254	389.107
15.5	6.50	6.50	35.036	0.271254	412.710
16.0	6.50	6.50	35.036	0.271254	424.020
17.0	6.50	6.50	35.036	0.271254	445.778
18.0	6.50	6.50	35.036	0.271254	466.523
19.0	6.50	6.50	35.036	0.271254	486.384

b)-4 D = 7.00 m

(1) Shape of Closs Section and Dimensions

Shape of the diversion tunnel is applied Horseshoe type section as shown in attached figure.

r: Radius = D / 2	(m)	3.500
r1 = r2 = 2r		7.000
θ1= θ2	(Degree)	24.295278
π :Pi		3
H1= r1 * sinθ1	(m)	2.8801
H2= r - H1	(m)	0.6199
B1= r1 * (1-cosθ1)	(m)	0.6199
B2= r1 - B1	(m)	6.3801
Ho= r * cosθo	(m)	

(3) Calculation of Flowing Area and Wetted Perimeter

(2) Calculation of Flowing Area

A = Ao - {(θo * π * r^2/180) - ho*(r^2-ho^2)^(1/2)}

Where Ao : Inner cross-section area = 3.317 * r^2 (m2)
ho : Water depth from the tunnel horizontal axis (m)
θo : Acos(ho / r)

(2)-2 Calculation of Wetted Perimeter

P = Po - (2*θo*π*r/180)

Where Po : Total perimeter length = 6.543 * r (m)

(3) Calculation of Capacity (Discharge)

(3)-1 Open Flow

Q = A * V = A * (1/n) * (I)^(1/2) * (R)^(2/3)

Where n : Coefficient of roughness (Concrete) 0.01400
I : Gradient of tunnel(slope)=1 / s 0.05556
s = 18
R : Hydrouric depth = A / P

r = 3.500 m
Ao = 3.317 * r^2 = 40.633 m2
Po = 6.543 * r = 22.901 m

ho (m)	H=r+ho (m)	A (m2)	P (m)	R=A/P (m)	V (m/sec)	Q (m3/sec)
0.20	3.70	22.788	12.304	1.8520	25.390	578.578
0.40	3.90	24.182	12.706	1.9032	25.856	625.264
0.60	4.10	25.568	13.110	1.9502	26.280	671.927
0.80	4.30	26.939	13.518	1.9928	26.661	718.233
1.00	4.50	28.292	13.933	2.0307	26.998	763.828
1.20	4.70	29.621	14.354	2.0636	27.289	808.335
1.40	4.90	30.921	14.785	2.0914	27.533	851.351
1.60	5.10	32.186	15.228	2.1136	27.728	892.442
1.80	5.30	33.409	15.686	2.1299	27.870	931.131
2.00	5.50	34.585	16.162	2.1398	27.957	966.888

(3)-2 Pipe Flow

In case of the piping flow for the diversion tunnel, relation between the river water level and the tunnel discharge is estimated with the following formulas:

$$H1 = v^2 * (1 + f) * 1/2g + H2$$

$$Q = [A^2 * 2g * (H1 - H2)/(1 + f)]^{(1/2)}$$

Where : H1 : River water level at the inlet
v : Average velocity in the tunnel = Q / A
Q : Discharge
A : Inner cross-sectional area = 3.317 * (D/2)^2
H2 : River water level at the outlet
f : Coefficient of loss

$$f = fe + ft$$

fe : Coeffient of loss at the inlet = 0.200
(with Circular bell mouth)
ft : Tunnel friction loss

$$ft = 2g * n^2 * L / D^{(4/3)}$$

n : Coeffient of roughness = 0.014
L : Length of diversion = 225.000
D : Tunnel diameter 7.000

H1 (m)	H2=D (m)	D=2r (m)	A (m2)	f	Q (m3/sec)
7.0	7.00	7.00	40.633	0.264550	0.000
8.0	7.00	7.00	40.633	0.264550	159.971
9.0	7.00	7.00	40.633	0.264550	226.233
10.0	7.00	7.00	40.633	0.264550	277.078
11.0	7.00	7.00	40.633	0.264550	319.942
12.0	7.00	7.00	40.633	0.264550	357.706
13.0	7.00	7.00	40.633	0.264550	391.848
14.0	7.00	7.00	40.633	0.264550	423.244
15.0	7.00	7.00	40.633	0.264550	452.467
16.0	7.00	7.00	40.633	0.264550	479.913

2. Coffer Dam

1) Hight of Coffer Dam

Upsream coffer dam (hu) = (H1-L/S) + Free board + Depth of fou

Downstream coffer dam (hd) = H2 + Free board + Depth of foundati

Coffer Dam	D=5.5 m	D=6.0 m	D=6.5 m	D=7.0 m
H1 (m)	23.5	18.5	15.5	14.0
H2 (m)	5.5	6.0	6.5	7.0
Water L. at Inlet	444.5	439.5	436.5	435.0
Hight of hu	14.5	9.5	6.5	5.0
Hight of hd	8.5	9.0	9.5	10.0

Free board = 1.00 m

Depth of foundation= 2.00 m

L/S = 12.0

2) Quantity of Coffer Dam

Area of coffer dam =(b1+b2)/2*h

b1 = 2.00 m
b2 = b1 + m * h

D	m	b2	h	A
D = 5.5 m				
Upstream c.dam	0.7	12.2	14.5	102.6
Downstream c.dam	1.0	10.5	8.5	53.1
D = 6.0 m				
Upstream c.dam	0.7	8.7	9.5	50.6
Downstream c.dam	1.0	11.0	9.0	58.5
D = 6.5 m				
Upstream c.dam	0.7	6.6	6.5	27.8
Downstream c.dam	1.0	11.5	9.5	64.1
D = 7.0 m				
Upstream c.dam	0.7	5.5	5.0	18.8
Downstream c.dam	1.0	12.0	10.0	70.0

Dam volume = A * L

Coffer dam	L1 (m)	L2 (m)	Lm (m)	V=A*Lm (m3)
D = 5.5 m				
Upstream c.dam				
Crest hight (EL)		445.5		
Length	14.0	45.0	29.5	3,026.3
Downstream c.dam				
Crest hight (EL)		427.5		
Length	15.0	21.0	18.0	956.3
D = 6.0 m				
Upstream c.dam				
Crest hight (EL)		440.5		
Length	14.0	37.0	25.5	1,290.0
Downstream c.dam				
Crest hight (EL)		428.0		
Length	15.0	22.0	18.5	1,082.3
D = 6.5 m				
Upstream c.dam				
Crest hight (EL)		437.5		
Length	14.0	33.0	23.5	653.0
Downstream c.dam				
Crest hight (EL)		428.5		
Length	15.0	22.0	18.5	1,186.3
D = 7.0 m				
Upstream c.dam				
Crest hight (EL)		436.0		
Length	14.0	30.0	22.0	412.5
Downstream c.dam				
Crest hight (EL)		429.0		
Length	15.0	22.0	18.5	1,295.0

L1 : Bottom length
L2 : Crest length

3. Calculation of Cost

Unit : 10^3 Colones

Items	Unit	Price	Quantity	Cost
D = 5.5 m, t=45 cm				
Cofferdam, Concrete	m3	9,500	3,983	37,835
Tunnel excavation	m3	4,700	7,673	36,061
Tunnel lining	m3	12,800	2,029	25,966
Reinforcement	ton	93,600	31	14,979
Others (15 %)	Ls			17,226
Total				132,067
D = 6.0 m, t=50 cm				
Cofferdam, Concrete	m3	9,500	2,372	22,536
Tunnel excavation	m3	4,700	9,178	43,139
Tunnel lining	m3	12,800	2,462	31,507
Reinforcement	ton	93,600	32	14,577
Others (15 %)	Ls			16,764
Total				128,523
D = 6.5 m, t=50 cm				
Cofferdam, Concrete	m3	9,500	1,839	17,474
Tunnel excavation	m3	4,700	10,538	49,528
Tunnel lining	m3	12,800	2,655	33,981
Reinforcement	ton	93,600	34	15,147
Others (15 %)	Ls			17,420
Total				133,550
D = 7.0 m, t=55 cm				
Cofferdam, Concrete	m3	9,500	1,708	16,221
Tunnel excavation	m3	4,700	12,291	57,767
Tunnel lining	m3	12,800	3,148	40,300
Reinforcement	ton	93,600	35	17,143
Others (15 %)	Ls			19,715
Total				151,146

Resalt of cost calculation, the 6.00 m of Diversion tunnel is most economically, therefore 6.00 m of the diversion tunnel will be applied.

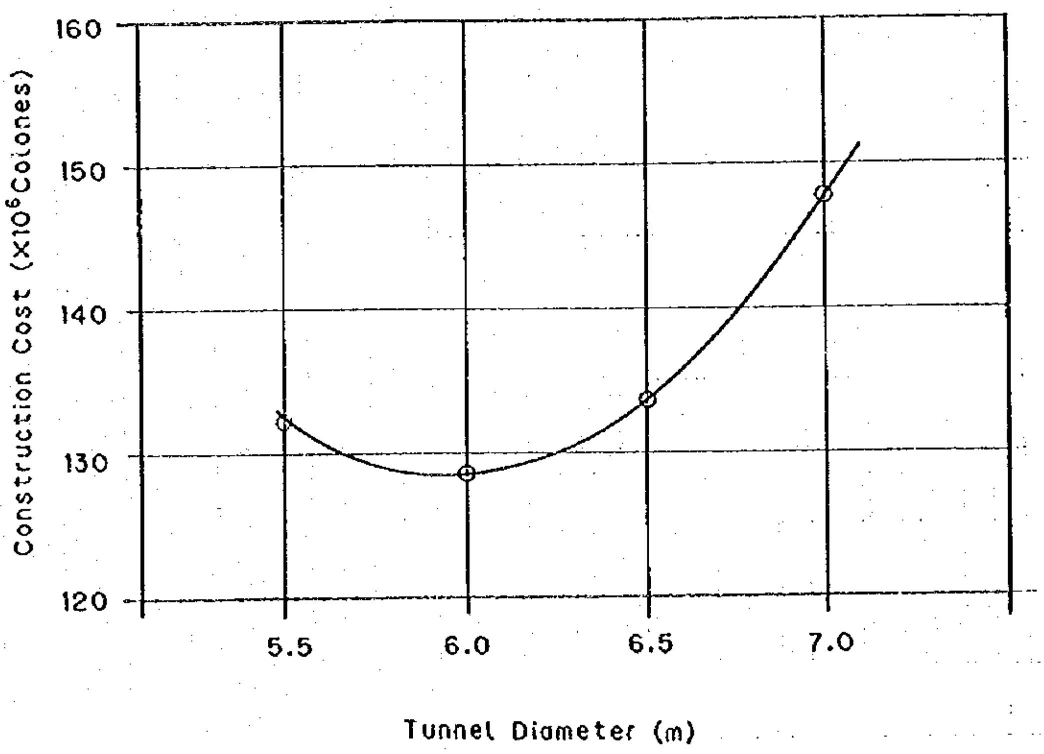


Fig.A–11–1 Relation Curve between
Construction Cost and Tunnel Diameter

A-11-1-2 Calculation of Discharge Capacity of Spillway

1) Calculation of Dischrge Capacity of Spillway

(1) Basic Condition

Design flood (Qf)	(m3/sec)	1,600.00
Dam type	Concrete gravity	
Reservoir water level H.W.L (m) L.W.L (m)		 477.40 470.00
Dam foundation (EL)		417.00

(2) Calculation of Discharge Capacity

$Qf = n * C' * B * H^{1.5}$

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

$Cd = 2.200 - 0.0416 * (Hd / W)^{.990}$

$C = 1.60 * [\{1 + 2 * a * (H/Hd)\} / \{1 + a * (H / Hd)\}]$

In case of $n \gtrsim 2$, $b / s' \gtrsim 0.8$

$Md = 0.0756 * (Hd / B)^{.5}$

In case of $n \gtrsim 2$, $b / s' \lesssim 0.8$

$Md = 0.0756 * (Hd / B)^{.5} * \{ 1 / n + 1.465 * (n-1)/n * (b / S')^{1.7} \}$

Where
- Qf: Discharge (m3/sec)
- n : Number of chute
- B : Width of chute (m)
- H : Water depth from reservoir water level to weir crest (m)
- C' : coefficient of discharge with pier effect considered
- C : Coefficient of discharge without pier effect
- Md: Reduction ratio of discharge coefficient due to piers
- Hd: Water depth from design reservoir water level to weir crest (m)
- W : Height of crest (m)
- Cd: Value of C when H is equal to Hd
- a : Constant
- S' : Distance from crest to pier head (m)
- b : Thickness of pier (m)

Qf	m3/sec	1,600.00
W	m	53.00
B	m	12.50
n	pc	2.00

1st stage   C'-1 =   2.05 (1st approximate value)

Hd = {Qf / (n * C'-1 * B)}^(2/3) =   9.915 m

Cd = 2.200 - 0.0418*(Hd/w)^0.990 =   2.192

Md = 0.0756*(Hd/B)^0.5   =   0.067

a = (Cd - 1.60) / (3.20 - Cd)   =   0.587

C = 1.60 * [{1 + 2 * a * (H/Hd)} / {1 + a *(H /Hd)}]

H = Hd

C = 1.60 * {(1 + 2 * a) / (1 + a)} =   2.192

C' = C* {1 - Md * (H / Hd)^1.5 }   =   2.044

C'-1 > C'

2nd stage   C'-2 =   2.044

Hd = (Qf / n * C'-2 * B)^(2/3) =   9.934 m

Cd = 2.200 - 0.0418*(10.539/50)^0/990 =   2.192

Md =   0.067

a =   0.587

C = 1.60 * {(1 + 2 * a) / (1 + a)} =   2.192

C' = C* {1 - Md * (H / Hd)^1.5 }
=   2.044

C'-2 = C'

∴ H= Hd=   9.934 m , C' =   2.044

Velocity of approach   Wide of waterway=   105.00

Va = Qf / Aapp. =   0.24 m/sec

Velocity of approach head

ha = Va^2 / 2*g =   0.003 m

Discharge at any reservoir water level

$$C = 1.60*\{(1+2*a*(H/Hd))/(1+a*(H/Hd))\}$$
$$C' = C*(1-Md*(H/Hd)^{1.5}$$
$$Q = n * C' * B * H^{1.5}$$

Qf	m3/sec	1,600.00
W	m	53.00
B	m	12.50
n	pc	2.00

a	n	B	Hd
0.587	2.00	12.50	9.93

H (m)	Reservoir W. Level	C	C'	Q (m3/sec)
0.00	467.47			
1.00	468.47	1.689	1.685	42.14
2.00	469.47	1.769	1.758	124.30
3.00	470.47	1.841	1.819	236.34
4.00	471.47	1.906	1.871	374.29
5.00	472.47	1.965	1.915	535.33
6.00	473.47	2.019	1.952	717.09
7.00	474.47	2.068	1.981	917.44
8.00	475.47	2.114	2.005	1,134.38
9.00	476.47	2.156	2.024	1,366.01
10.00	477.47	2.194	2.037	1,610.50
11.00	478.47	2.231	2.046	1,866.09
11.65	479.12	2.253	2.049	2,037.38
12.00	479.47	2.264	2.051	2,131.09

2) Stilling Basin

(1) Ski-jump type (Flip bucket)

L = Hv * {Sin 2θo + (Sin^2 2θo + 4 * W * Cos^2 θo * 1 / Hv)^0.5 }

uo = { 2g * (Hy + W) }^0.5

θ = Cos^-1 Cosθo /(1 + W / Hy)^0.5

Hv = Vo^2 / 2g

Vo = q / h1, or Vo = 0.9 * {2g * (Hd + H)}^0.5

Where

Hv :	Velocity head at end sill = Vo^2 / 2g	(m)	
W :	Height from basin water level to end sill		6.50 m
θo:	Projection angle	(Degree)	
Hd+H:	Dam height+Depth of overflow		62.93 m
q :	Q / B = Q/(n*B+3.00*(n-1)) =		23.93 m3/sec/m
Q :	100 years return period =		670.00 m3/sec

Calculation

θo (°)	Sin 2θo	Cosθo	Vo (m/sec)	Hv (m)	L (m)	UO (m/sec)	θ
10.00	0.34202	0.98481	31.61	50.98	57.30	33.56	
15.00	0.50000	0.96593	31.61	50.98	68.92	33.56	
20.00	0.64279	0.93969	31.61	50.98	80.14	33.56	
25.00	0.76604	0.90631	31.61	50.98	90.17	33.56	
30.00	0.86603	0.86603	31.61	50.98	98.40	33.56	
35.00	0.93969	0.81915	31.61	50.98	104.33	33.56	
40.00	0.98481	0.76604	31.61	50.98	107.63	33.56	

Sin 2θo = 2*Sin θo * Cos θo

Orbit of water vein

y = (1 + tan^2θo) * x^2 / (4 * Hv) − tanθo * x

θo = 30°
tan 30° = 0.57735

x (m)	y (m)
10.00	-5.12
20.00	-8.93
30.00	-11.44
40.00	-12.63
50.00	-12.52
60.00	-11.10
70.00	-8.37
80.00	-4.34
90.00	1.00
100.00	7.65
110.00	15.61
120.00	24.88

Radius of bucket (r)
r = 3hb～8hb

hb = Q / (B * v)

v = C*(2*g*H)^0.5

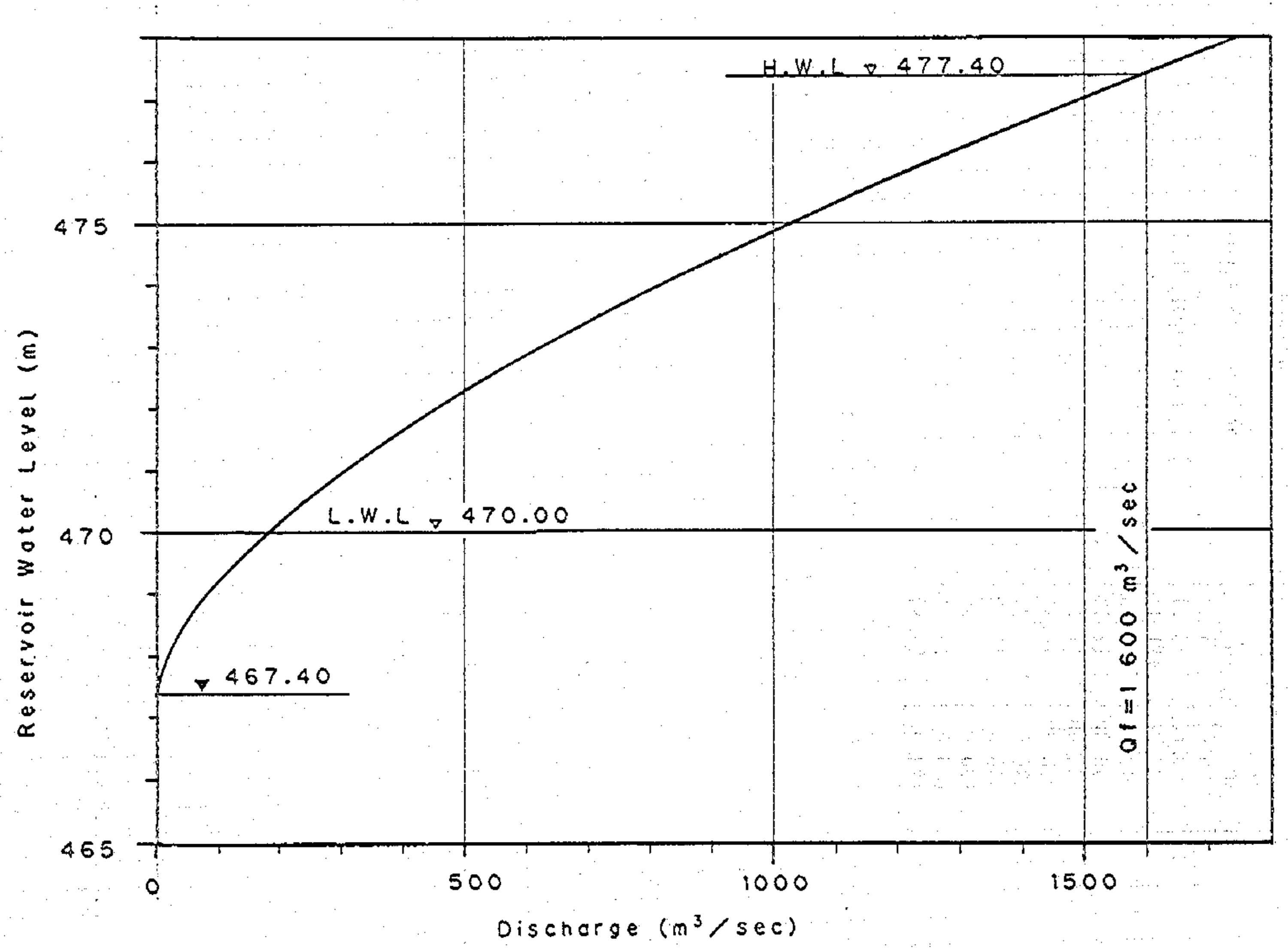


Fig.A-11-2 Discharge Capacity of Spillway

A-11-1-3 Calculation of Outlet Works

1) Reservoir and Intake

High Water Level	(EL)	477.40
Low Water Level (LWL)	(EL)	470.00
Sedimentation Surface	(EL)	460.00
Center of Intake	(EL)	461.00

2) Location of Inlet for Outlet Works

The inlet will be provided at center of the spillway direction below EL 461.00 m as shown sketch.

3) Study of Conduit Pipe Diameter

Outlet should be possible to lerease annual average inflow of 15 m3/sec at low water level of 470.0 m.

the lereasing discharge (Qo) can be calculated by below equation:

$$Qo = Ao * C * \{2g * (H + Zo) / (1 + \Sigma hf)\}^{0.5}$$

Where,

Ao : Cross-sectional area of coduit pipe (m2)

C : Coefficient of discharge for the main valve
Hollow jet valve = 0.80 ~0.83
Cone sleeve valve = 0.85

H : Static head at the inlet (m)

zo : Differential head between Center of inlet and Outlet (m)

Σf : Coefficient of the head loss in the conduit pipe

a) Calculation of Loss Coefficient

(1) Entrance loss coefficient

fe = 0.20 (With rectangular bell mouth)

(2) Friction loss coefficient

f = (2 * g * n^2 / R^(4/3)) * L

Where,

n : Kutter friction coeffient = 0.012

R : Hydrauric radius (m)
= Ao / P (m) = 0.25 * D

Ao : Cross-sectional area (m2)

P : Wetted primeter (m) = $\pi * D$

L = Length of the conduit (m) = 50.00

f = 0.89606 * D^-(4/3)

(3) Bend loss coefficient

fb = fb1 * fb2 = 0.10

r / D = 3.00 → fb1 = 0.10
θ = 90 → fb2 = 1.00

(4) Total

Σf = fe + f + fb = 0.30 + 0.89606/D^-(4/3)

b) Calculation of Discharge

Design conditions

Q	m3/sec	15.00	at EL 470.0 m
HWL	m	477.40	
LWL	m	470.00	
Inlet CL	m	461.00	Conduit cennter
Valve CL	m	437.50	
Length of Conduit	m	50.00	

therefore
H = W.L - Inlet CL =
Z = Inlet CL - Valve CL = 23.50 m
C = (Hollow jet valve) 0.80

Calculation of discharge at any reservoir water level

Q' = (π *D^2/4) *C* [2g * (H + Z) / [1+0.30+0.89606*D^-(4/3)]]^0.5

W.L	H (m)	Z (m)	Q' (m3/sec)		
			1.10	1.20	1.30
477.40	16.40	23.50	14.71	17.88	21.37
477.00	16.00	23.50	14.64	17.79	21.26
475.00	14.00	23.50	14.26	17.34	20.72
473.00	12.00	23.50	13.88	16.87	20.16
471.00	10.00	23.50	13.48	16.38	19.58
470.00	9.00	23.50	13.28	16.14	19.29
469.00	8.00	23.50	13.07	15.89	18.99
467.00	6.00	23.50	12.65	15.38	18.37
465.00	4.00	23.50	12.21	14.85	17.74
463.00	2.00	23.50	11.76	14.30	17.08
461.00	0.00	23.50	11.29	13.72	16.40

Resalt of the caluculation, the diameter of conduit is to be 1.20 m

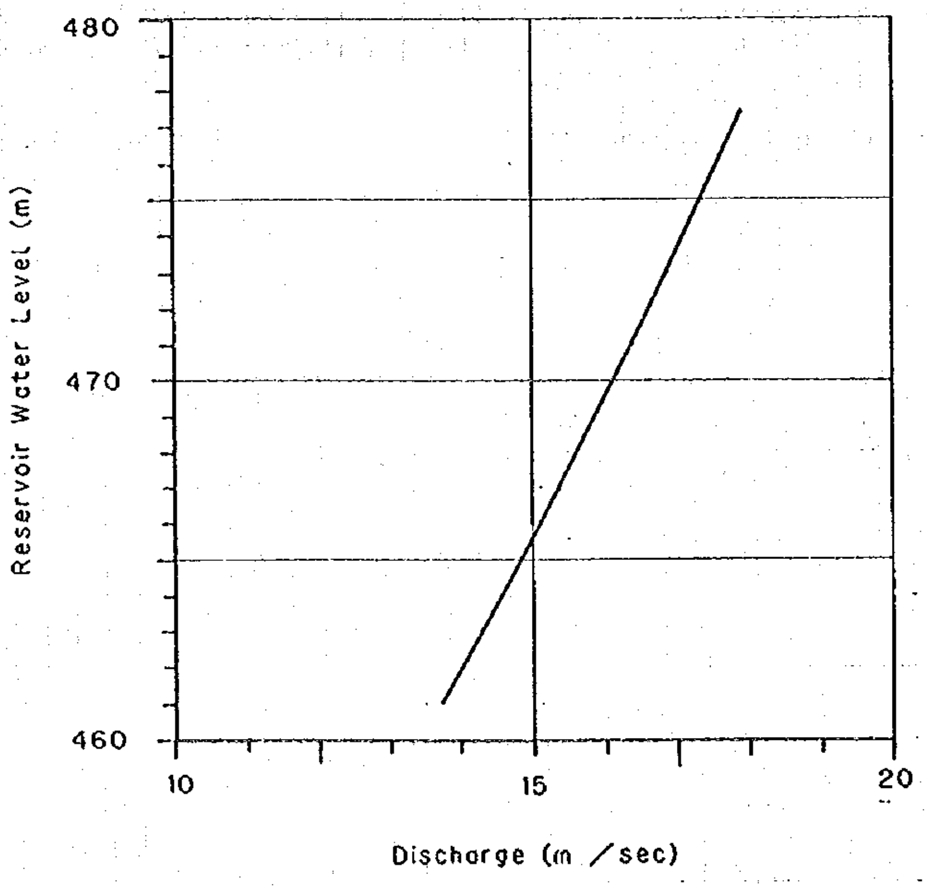


Fig.A-11-3 Discharge Capacity of Dam Outlet

A-11-1-4 Calculation of Freeboard

The elevation of dam crest is determined enough high not to be lower than the reservoir water level adding wave heights and allowances.

1) Basic Formula

$$Ht \geqq \max.(H.W.L + hw + he + ha,\ \text{or}\ \max.W.L + hw)$$

Where

- Ht : Elevation of dam crest
- hw : Height of wave due to wind
- he : Height of wave due to earthquake
- ha : Allowance considering delayed gate operation

· hw = 0.00086 * V^1.1 * F^0.45 (S.M.B method)

V	Design wind velocity(avarage for 10 minutes) (m/sec)	30.00
F	Fetch (m)	1,000.00

he = 1/2 * k * τ * (g * Ho)^0.5 * 1/π

k	Seismic coefficient	0.15
τ	Earthquake period (sec)	1.00
g	Acceleration of gravity (m/sec^2)	9.80
Ho	Height from reservoir level to foundation rock (Ho=H.W.L-EL417.0) (m)	60.40

ha : As shown below table

Concrete dam		0.50
Fill dam		1.00

2) Calculation of Freeboard

In case of concrete dam

H.W.L	Normal high water level	477.40
hw	0.00086 * V^1.1 * F^0.45	0.81
he	1/2 * k * τ * (g * Ho)^0.5 * 1/π	0.58
ha	Concrete dam	0.50
Total(m)		479.29

Therefor, the elevation of dam top is decided EL 479.40 m.

A-11-1-5 Calculation of Surgung

1) Design Conditions

Reservoir			
H.W.L	m	477.400	
L.W.L	m	470.000	
Headrace Tunnel	Circlar type		
Diameter	m	3.100	
Length	m	5,549.8	
Area	m2	7.549	Cross-sectional area
Surge Tank	Restricted orifice type		
Lower Diameter	m	8.000	
Area	m2	50.272	Cross-sectionsl area
Elevation	m	445.000	
Upper Diameter	m	8.000	
Area	m2	50.272	
Elevation	m	500.000	
Orifice Diameter	m	1.500	
Coefficient of loss			
Up surging		0.800	
Down surging		0.700	
Discharge			
Qmax	m3/sec	27.000	
Qmin	m3/sec	26.700	=Qmax * (Hmin/Hmax)^0.5

2) Calculatin Case

Calculation of the surging is performed for two cases :
rapid load rejection and rapid load increase from 1-unit operation to 2-uint operation.

a) Rapid Load Rejection (Closing time t = 0.000 sec)

Tunnel diameter	m	3.100	Q = 27.00 m3/sec
Tunnel length	m	5,549.8	
Loss coefficient		1.533	C=ho/vo^2
Type of surge tank	(Restricted Orifice)	2	制水口は,2をインプット
Number of sections		2	低い方の断面から記入
Elevation Area	m m2	445.000 50.272	Bottom section
Elevation Area	m m2	500.000 50.272	Top section
Orifice diameter	m	1.500	
Coefficient of discharge Inflow Outlet		 0.800 0.700	
Time varietion for Discharge (Load rejection)			Q=Qmax.→Q=0 t= 0 sec
Number of deta		3	
Time Discharge	sec m3/sec	0.000 27.000	Start of closing Discharge at load rejecting
Time Discharge	sec m3/sec	0.000 0.000	Closing time at finish of closing
Time Discharge	sec m3/sec	700.000 0.000	Surging calculation time
Reservoir water level	m	477.400	H.W.L
Integrals Time interval	sec	1.000	
Starting time	sec	0.000	
Finishing time	sec	510.000	Surging calculating time
Output (interval)	sec	30.000	任意時間をインプット

b) Rapid Load Increas (Opening time = 12.0 sec)

Tunnel diameter	m	3.100	Q = 27.000 m3/sec
Tunnel length	m	5,549.8	
Loss coefficient		1.533	C=ho/vo^2
Type of surge tank	Restricted Orifice	2	制水口は，2をインプット
Number of sections		2	低い方の断面から記入
Elevation Area Elevation Area Orifice diameter	m m2 m m2 m	445.000 50.272 500.000 50.272 1.500	Bottom section Top section
Coefficient of discharge Inflow Outlet		0.800 0.700	
Time varietion for Discharge (Load rejection) Number of deta		3	Q=1/2*Qmax. → Q=Qmax. t= 12 sec
Time Discharge	sec m3/sec	0.000 13.500	Start of rapid increase Discharge before increase
Time Discharge	sec m3/sec	12.000 27.000	Opening time Discharge at full opening
Time Discharge	sec m3/sec	700.000 27.000	Surging calculation time
Reservoir water level	m	470.000	L.W.L
Integrals Time interval	sec	1.000	
Starting time	sec	0.000	
Finishing time	sec	510.000	Surging calculation time
Output (interval)	sec	30.000	任意時間をインプット

3) Result of Calculation

a) Rapid Load Rejection

Time (t)	Surging Water Le. (EL)	Surging 上昇高さ (m)	Velocity 流速 (m/sec)	Discharge 流量 (m3/sec)	Remarks
0.000	457.784	19.616	3.577	27.000	
30.000	471.827	5.572	2.693	0.000	
60.000	482.085	-4.685	1.870	0.000	
90.000	488.720	-11.320	1.079	0.000	
120.000	491.820	-14.420	0.299	0.000	
150.000	491.425	-14.025	-0.464	0.000	
180.000	487.970	-10.570	-1.014	0.000	
210.000	482.839	-5.439	-1.206	0.000	
240.000	477.555	-0.155	-1.101	0.000	
270.000	473.188	4.212	-0.816	0.000	
300.000	470.346	7.054	-0.435	0.000	
330.000	469.326	8.074	-0.014	0.000	
360.000	470.200	7.200	0.390	0.000	
390.000	472.642	4.758	0.665	0.000	
420.000	475.900	1.500	0.750	0.000	
450.000	479.131	-1.731	0.660	0.000	
480.000	481.659	-4.259	0.447	0.000	
510.000	483.048	-5.948	0.162	0.000	

Top surging water level　　492.078 m, after 132.0 sec

Bottom surging water level　　457.784 m, after 0.0 sec

b) Rapid Load Increase

Time (t)	Surging Water L. (EL)	Surging 下降高さ (m)	Velocity 流速 (m/sec)	Discharge 流量 (m3/sec)	Remarks
0.000	465.094	4.907	1.789	13.500	
30.000	459.136	10.864	2.088	27.000	
60.000	453.459	16.541	2.547	27.000	
90.000	449.805	20.195	2.974	27.000	
120.000	447.888	22.112	3.312	27.000	
150.000	447.262	22.738	3.547	27.000	
180.000	447.464	22.536	3.682	27.000	
210.000	448.087	21.913	3.738	27.000	
240.000	448.828	21.172	3.739	27.000	
270.000	449.503	20.497	3.712	27.000	
300.000	450.026	19.974	3.674	27.000	
330.000	450.375	19.625	3.636	27.000	
360.000	450.568	19.432	3.605	27.000	
390.000	450.641	19.359	3.583	27.000	
420.000	450.635	19.365	3.570	27.000	
450.000	450.585	19.415	3.564	27.000	
480.000	450.520	19.480	3.563	27.000	
510.000	450.457	19.543	3.565	27.000	

Top surging water level　　465.094 m, after 0.00 sec

Bottom surging water level　　447.250 m, after 155.0 sec

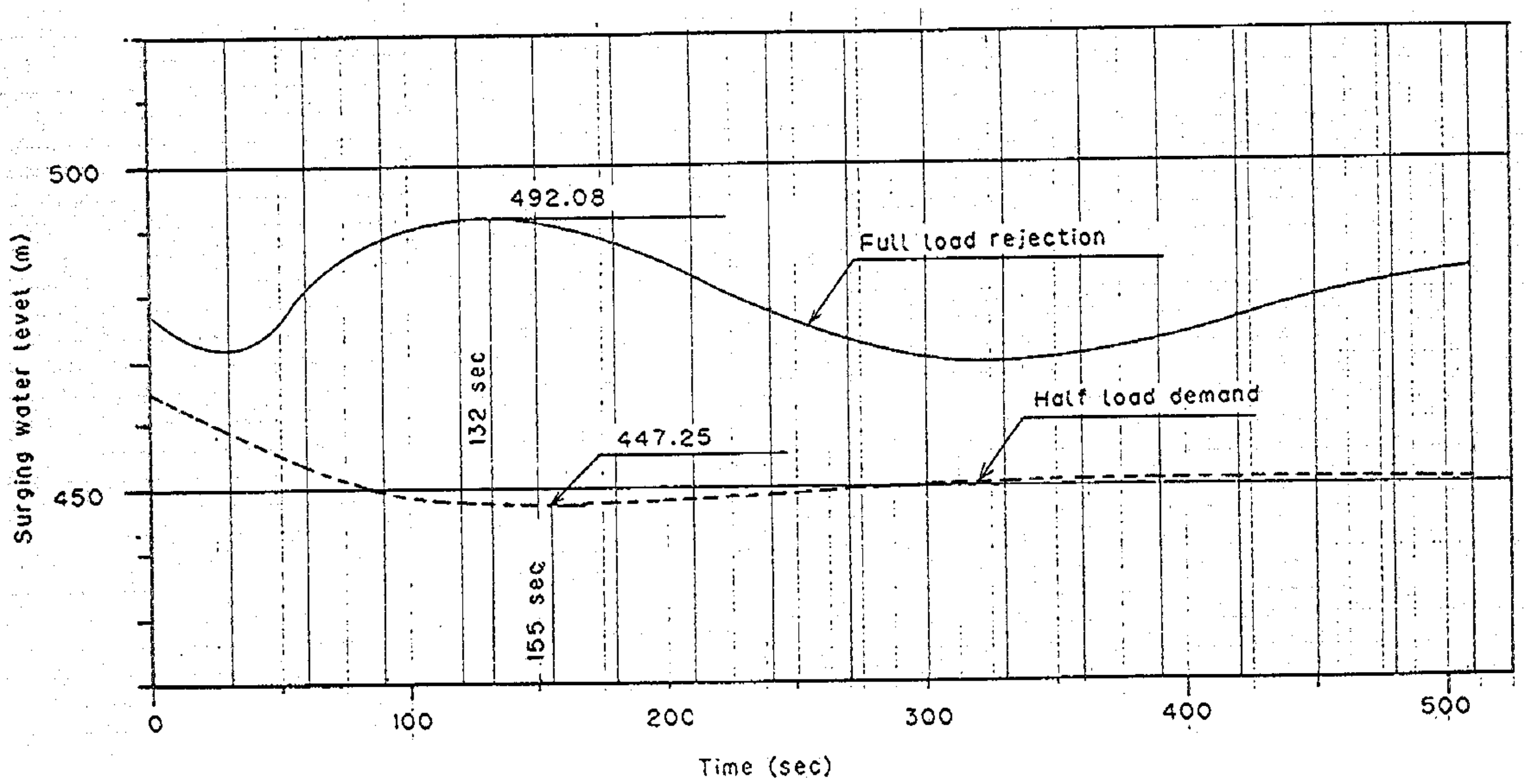
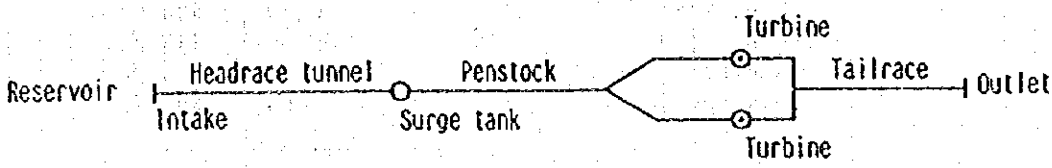


Fig.A-11-4 Surging Curve

A-11-1-6 Calculation of Water Hammer Pressure

Penstock steel liner is installed in the under ground.

One line steel penstock is installed from end of the surge tank to just before of the power station, and after that it bifucates to connect No. 1 and No. 2 turbine as shown below



1) Calculation Conditions

Reservoir H.W.L (EL. m) L.W.L (EL. m)	 477.40 470.00
Headrace tunnel Length (m) Cross-sectional area (m2)	 5,559.81 7.549
Surge tank Top of shaft elvetion (EL, m) Height of basement of shaft (EL, m) Diameter of shaft (m) Cross-sectional area (m2)	 500.00 445.00 8.00 50.272

Penstock	Length (m)	Diameter (m)	Average Diameter (m)	Cross Area (m2)
Center of S.T	0.00	3.10	3.10	7.549
1	10.00	3.10	3.10	7.549
1～2	6.00	3.00	3.05	7.307
2～VIP3 BC	155.27	3.00	3.00	7.070
VIP3 BC～VIP3 EC	12.54	2.80	2.90	6.606
VIP3 EC～VIP4 BC	230.11	2.80	2.80	6.158
VIP4 BC～VIP4 EC	12.55	2.60	2.70	5.726
VIP4 EC～VIP5 BC	426.66	2.60	2.60	5.310
VIP5 BC～VIP5 EC	12.55	2.40	2.50	4.909
VIP5 EC～VIP6 BC	227.66	2.40	2.40	4.524
VIP6 BC～VIP6 BC	12.54	2.20	2.30	4.155
VIP6 EC～Bifucation	450.34	2.20	2.20	3.802
Sub-Total	1,556.22			
Bifucation～End	31.59	1.25	1.25	1.227

Turbine Maximum discharge 13.5m3/sec*2= Number of unit Height of turbine center (EL, m) Closing time (sec)	 27.00 2 79.50 12.00
Outlet Tail water level (EL, m)	 84.00

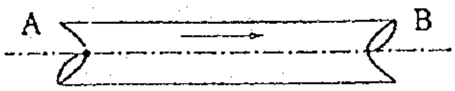
2) Calculation Formula and Boundary Conditions

(a) Calculation Formula

$$HA,(t) \pm S \cdot QA,(t) = HB,(t-\frac{L}{a}) \pm S \cdot QB,(t-\frac{L}{a})$$

where,

$HA,(t)$	:	Pressure of point A at time (t)
$QA,(t)$	:	Discharge of point A at time (t)
$HB,(t-\frac{L}{a})$	:	Pressure of point B at time $(t-\frac{L}{a})$
$QB,(t-\frac{L}{a})$	:	Discharge of point B at time $(t-\frac{L}{a})$
S	:	Constant = $a/g.A$
a	:	Pressure propagation velocity
g	:	Acceleration due to gravity
A	:	Cross-sectional area of penstock
L	:	Length of penstock



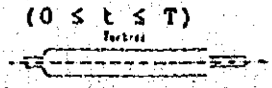
(b) Boundary Conditions

- Boundary condition at closure

In case of linearity closing, the boundary condition is formed as follows:

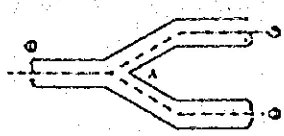
$$QA,(t) = (1 - \frac{t}{T}) \cdot \sqrt{HA,(t) - HB,(t)}$$

T	:	Closing time
t	:	Random time in the closing time $(0 \le t \le T)$



- Boundary condition at branch

$$\{Q\ 1,(t) = Q\ 2,(t) + Q\ (3),(t)\}$$



- Boundary condition at intake (reservoir)

$$HA,(t) = HA,\ 0$$

3) Result of Calculation

Calculation is performed by electric computer per 0.01 second. Resalts is as shown Fig. A-II-I-6, the maximum water hammer is estimated 24.3 % of the stasic water pressure.

$$H_w / H_o = (573.0 - 84.0) / (477.4 - 84.0) = 1.24$$

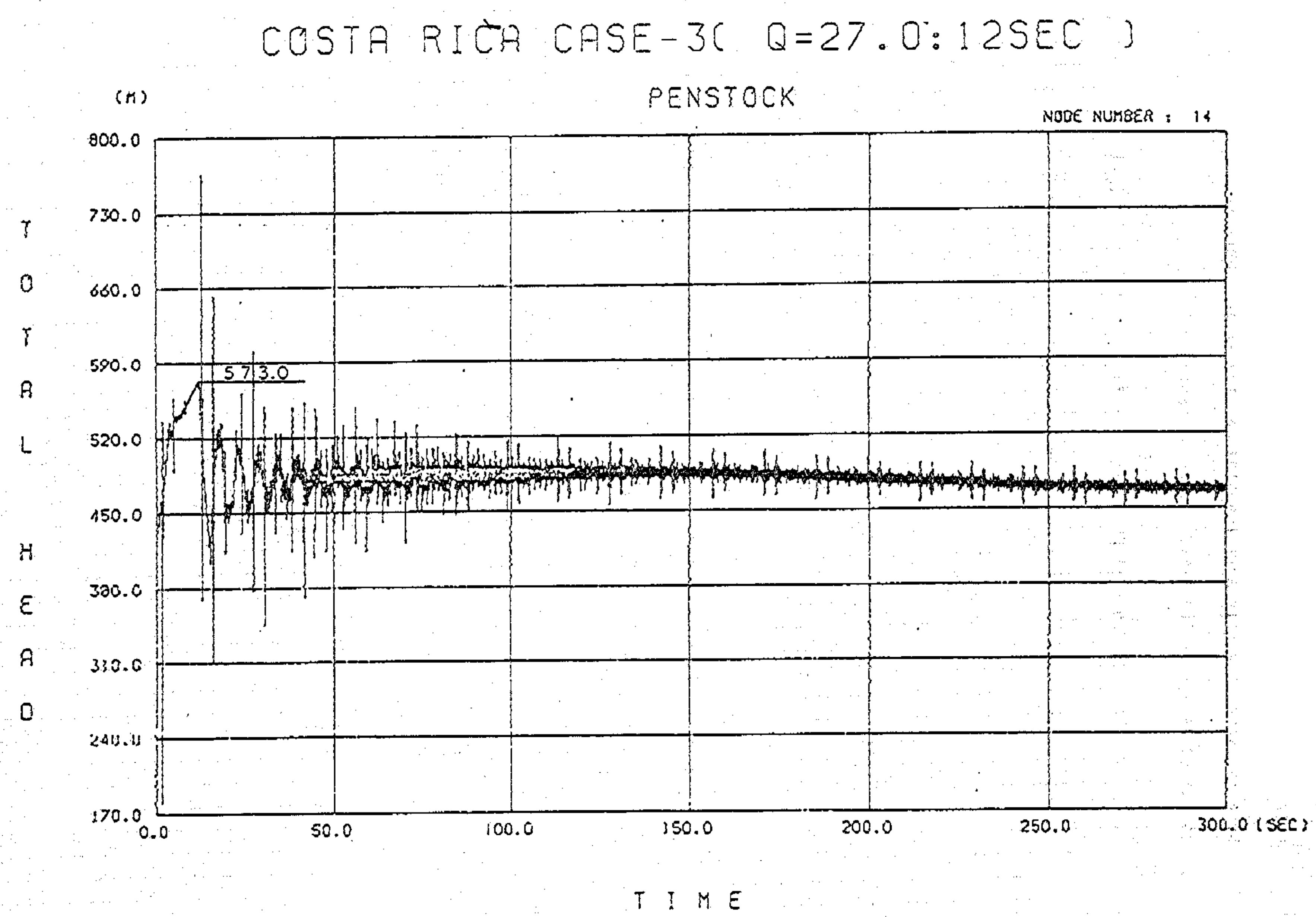


Fig.A—11—5 Water Hammer Pressure Curve

A-11-1-7 Rating Curve at Powerhouse

Rating curve at the Power plant site was calculated by Manning formula because there laked actual observation data.

1) Flow Area and Wetted Perimeter

The river cross section for calculation is located on the juction from Power house (Outlet) and River way arranged as attached figure.

EL	h (m)	Wetted Perimeter (m)	Cross s. Area (m2)	Hydraulic radius (m)	
82.5					
83.0	0.5	41.6	20.3	0.48657	
84.0	1.0	44.9	62.3	1.38783	
85.0	1.0	48.1	106.3	2.20939	
86.0	1.0	51.3	152.3	2.96632	
87.0	1.0				

2) Calculation of Discharge

River flow calculates by Manning formula.

V = I^(1/2) * 1/n * R^(2/3)

Where, I:Gradient of the river = 1/50= 0.02000
n:Kutter friction coefficient = 0.040
R:Hydraulic radius

Q = A * V

EL	V (m/sec)	Q (m3/sec)	Remarks
82.5	0.0	0.0	100 Return period
83.0	2.187	44.3	flood is 100 m3/sec
84.0	4.399	273.8	
85.0	5.997	637.2	
86.0	7.299	1,111.3	

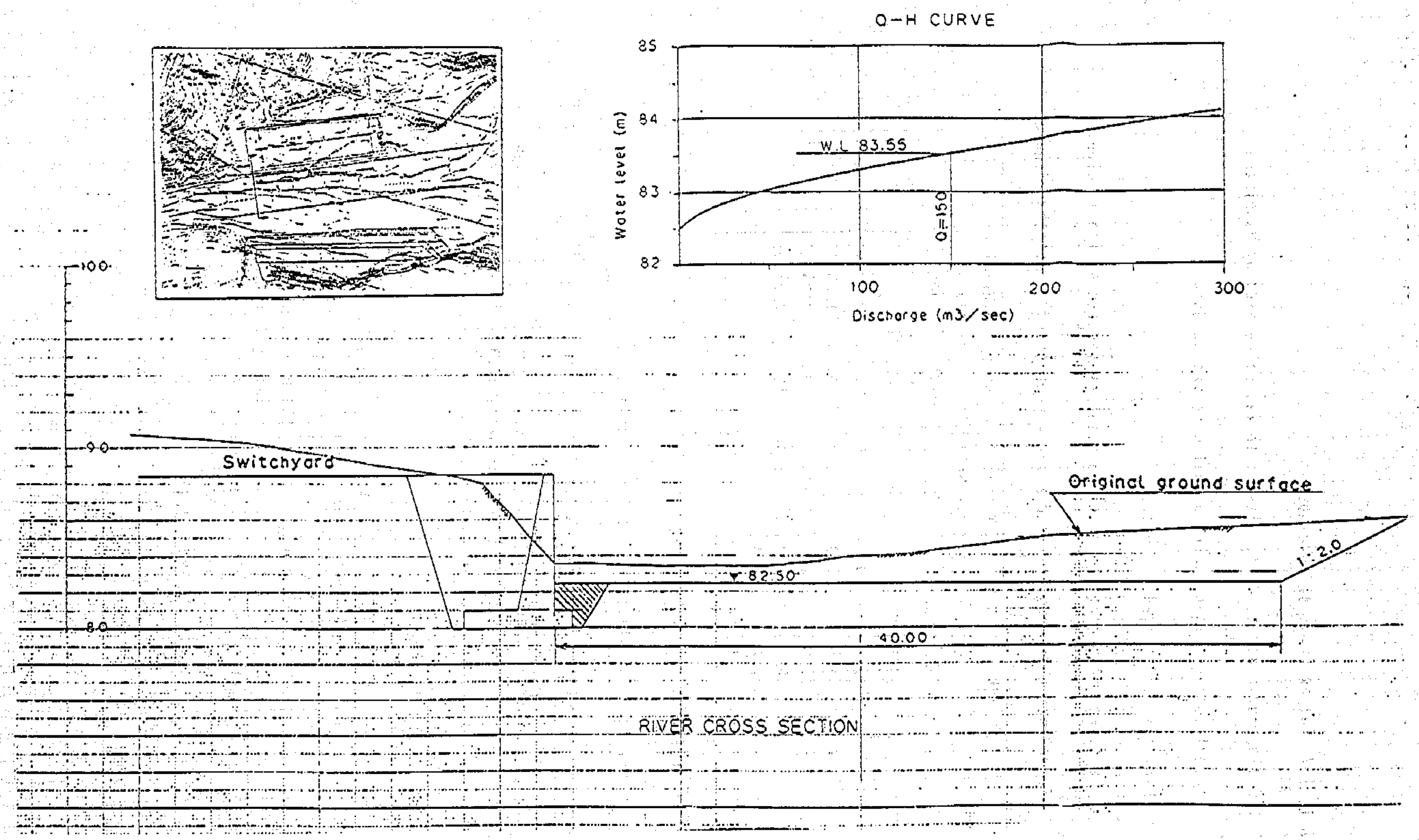


Fig.A—11—6 Rating Curve at Power Plant Site

A-11-1-8 Calculation of Effective Head

1) Calculation of Head Loss

(1) Intake Head Loss

(1)-1 Trashrack Loss (Screen)

$$hl\text{-}1 = \beta * \sin\theta * (t / b)^{(4/3)} * V^2 / 2g$$

Where

hl-1	Trashrack loss (m)	--
β	Coefficient for the sectional shape of screen bars	1.60
θ	Angle of the screen (Dgree)	73.30
t	Thickness of screen bars (mm)	10.00
b	Size of screen mesh (Span) (mm)	100.00
V	Flow velocity at face of screen (m/sec) = Q / A	
A	Flow area at face of screen (m2) (A = H8.50m*B6.50m)	55.25
Q	Discharge (m3/sec) (m3/sec)	--

$\therefore$ hl-1 = 1.07 *10^-6 * Q^2

(1)-2 Entrance Loss

$$hl\text{-}2 = f1 * v^2 / 2g$$

Where

hl-1	Entrance loss (m)	--
f1	Coefficient of entrance loss (with rectangular bell mouth)	0.20
V	Flow velocity after entrance (Q / A)	--
A	Flow area after entrance (m2) (A=π*D^2/4, D= 3.10 m)	7.549

$\therefore$ hl-2 = 179.07 *10^-6 * Q^2

(1)-3 Total Loss at Intake

hl = hl-1 + hl-2= 180.15 *10^-6 * Q^2

(2) Headrace Tunnel Head Loss

(2)-1 Friction Loss

h2-1 = { 124.5 * n^2 / D^(4/3) } * L * V^2 /2g

Where

h2-1	Friction loss (m)	--
n	Kutter's coefficient of roughness	0.013
D	Tunnel inner diameter (m)	3.10
L	Length of headrace tunnel (m)	5,559.81
	(From Intake to End of Surge tank)	
V	Flow velocity (Q / A) (m/sec)	--
A	Cross sectional area (m2)	7.549

∴ h2-1 = 23,172.44 *10^-6 * Q^2

(2)-2 Bending Loss

h2-2 = fb1 * fb2 * v^2 / 2g

Where

h2-2	Bending loss (m)	
fb1	Loss coefficient determined by the ratio of bending radius ρ to the penstock D (ρ/D) (See Fig. (a)) or fb1 = 0.131 + 0.1632*(D/ρ)^(7/2)	
fb2	Ratio of the loss for a center angle θ to the loss for center angle of 90° (See Fig. (b)) or fb2 = (θ° / 90°)^0.5	

Calculation

IP	θ (Degree)	ρ (m)	fb1	fb2	h2-2 (m) *10^-6*Q^2
From Intake to End of Surge tanke					
V-IP. 1	48.00000	15.00	0.1317	0.7303	649.85
V-IP. 2	47.88541	15.00	0.1317	0.7294	649.07
H-IP. 1	41.95091	30.00	0.1311	0.6827	604.77
H-IP. 2	9.62110	60.00	0.1310	0.3270	289.50
Total	(h2-2)				2,193.19

fb1 = 0.131 + 0.1632 *(D/σ)^(7/2)
fb2 = (θ / 90°)^0.5

(2)-3 Total of Headrace Tunnel Head Loss

h2 = h2-1 + h2-2 = 25,365.63 *10^-6 * Q^2

(3) Penstock Head Loss

(3)-1 From Surge tanke end to Penstock's bifucation

(3)-1-1 Friction Loss

h3-1-1 = { 124.5 * n^2 / D^(4/3) } * L * V^2 /2g

Where

h3-1-1	Friction loss (m)	--
n	Kutter's coefficient of roughness	0.0115
D	Penstock inner diameter (m)	--
L	Length of penstock (m)	--
V	Flow velocity (Q / A) (m/sec)	--
A	Cross sectional area	--

Calculation

D (m)	Dm (m)	D^(4/3)	L (m)	A (m2)	h3-1-1 (m) *10^-6 * Q^2
3.10～3.00	3.05	4.423	6.00	7.307	21.34
3.00	3.00	4.327	155.270	7.070	603.19
3.00～2.80	2.90	4.136	12.500	6.606	58.18
2.80	2.80	3.946	229.560	6.158	1,288.46
2.80～2.60	2.70	3.760	12.520	5.726	85.31
2.60	2.60	3.575	426.660	5.310	3,555.55
2.60～2.40	2.50	3.393	12.550	4.909	128.92
2.40	2.40	3.213	227.040	4.524	2,899.52
2.40～2.20	2.30	3.036	12.540	4.155	200.96
2.20	2.20	2.861	450.340	3.802	9,147.51
Total	(h3-1-1)		1,544.98		17,988.94

(3)-1-2 Bending Loss

h3-1-2 = fb1 * fb2 * v^2 / 2g

Where

h3-1-2	Bending loss (m)	
fb1	Loss coefficient determined by the ratio of bending radius ρ to the penstock D (ρ/D) (See Fig. (a)) or fb1 = 0.131 + 0.1632*(D/ρ)^(7/2)	
fb2	Ratio of the loss for a center angle θ to the loss for center angle of 90° (See Fig. (b)) or fb2 = (θ° / 90°)^0.5	

Calculation

IP	θ (Degree	ρ (m)	D (m)	fb1	fb2	h3-2 (m) *10^-6*Q^2
HIP-3	52.32	30.00	3.00	0.1311	0.7624	102.00
VIP-3	47.77	15.00	2.90	0.1315	0.7285	112.02
VIP-4	47.82	15.00	2.70	0.1314	0.7289	149.04
VIP-5	47.94	15.00	2.50	0.1313	0.7299	202.87
VIP-6	47.89	15.00	2.30	0.1312	0.7294	282.85
Total	(h3-1-2)					565.93

$fb1 = 0.131 + 0.1632 * (D/\sigma)^{(7/2)}$
$fb2 = (\theta / 90°)^{0.5}$

(3)-1-3 Branch Loss (Bifucation)

h3-1-3 = fb * v^2 / 2g

Where

h3-1-3	Branch loss (m)	--
fb	Coefficient of branch loss	0.50
V	Velocity of before entrance (Q/A) (m/sec)	
A	Cross sectional area of penstock (m2) D = 2.20 m	3.802

∴ h3-1-3 = 1,764.94 *10^-6 * Q^2

(3)-1-4 Contraction of Loss

h3-1-4 = fgc * V^2 / 2g

Where

h3-1-4	Conraction loss (m)	--
fgc	Coefficient of contraction loss	--
V	Velocity of after contraction (Q/A) (m/sec)	
A1	Cross sectional area before cont. (m2)	
A2	Cross sectional area after cont. (m2)	

Calcculation

Sect.	D1	D2	A1	A2	A2 / A1
B.P	3.10	3.00	7.55	7.07	0.94
VIP-3	3.00	2.80	7.07	6.16	0.87
VIP-4	2.80	2.60	6.16	5.31	0.86
VIP-5	2.60	2.40	5.31	4.52	0.85
VIP-6	2.40	2.20	4.52	3.80	0.84

Sect.	θ	fgc	h3-2-4 (*10^-6*Q^2)
B. P	0.89	0.00	
YIP-3	0.86	0.00	0.00000
YIP-4	0.86	0.00	0.00000
YIP-5	0.86	0.00	0.00000
YIP-6	0.86	0.00	0.00000
Total			0.00000

(3)-1-4 Sub Total (From End of Surge tank to Bifucation)

h3-1 =h3-1-1 + h3-1-2 + h3-1-3 + h3-1-4

= 20,319.82 * 10^-6 * Q^2

(3)-2 From Bifucation to Power Plant

(3)-2-1 Friction Loss

h3-2-1 = [124.5 * n^2 / D^(4/3)] * L * V^2 /2g

Where

h3-2-1	Friction loss (m)	--
n	Kutter's coefficient of roughness	0.0115
D	Penstock inner diameter (m)	1.25
L	Length of penstock (m)	25.79
V	Flow velocity (q / A) (m/sec)	--
A	Cross sectional area	1.227

∴ h3-2-1= 10,681.06 * 10^-6 * q^2

(3)-2-2 Bend Loss

h3-2-2 = fb1 * fb2 * v^2 / 2g

Where

h3-2-2	Bending loss (m)	
fb1	Loss coefficient determined by the ratio of bending radius ρ to the penstock D (ρ/D) (See Fig. (a)) or fb1 = 0.131 + 0.1632*(D/ρ)^(7/2)	
fb2	Ratio of the loss for a center angle θ to the loss for center angle of 90° (See Fig. (b)) or fb2 = (θ° / 90°)^0.5	

D = 1.25 m
ρ = 15.00 m
θ = 35.00 degree

fb1 = 0.131 , fb2 = 0.624

∴ h3-2-2 = 2,767.49 * 10^-6 * q^2

(3)-2-3 Butterfly valve Loss

h3-2-3 = fv * V^2 *1 / 2g

Where

h3-2-3	Butterfly valve loss (m)	--
fv	Coefficient of valve loss (t/D)	0.90
t	Thickness of valve disc (m)	
D	Penstock inner diameter (m)	
V	Velocity of before entrance(q/A) (m/sec)	--
A	Cross sectional area of penstock (m2) D = 1.25 m	1.227

∴ h3-2-3 = 30,482.74 *10^-6 * q^2

(3)-2-4 Sub Total (From Bifucation to Power Plant)

h3-2 = h3-2-1 + h3-2-2 + h3-2-3
= 43,931.29 *10^-6 * q^2

(3) Total of Penstock Loss

h3 : h3-1 = 20,319.82 *10^-6 * Q^2
+ h3-2 = 43,931.29 *10^-6 * q^2

(4) Tailrace (at Outlet)

h4 = fge * V^2 / 2g

Where:

h4	Enlarging head loss	
fge	Coefficient of enlarging loss	1.00
A	Cross sectional area before enlarging (m2) = 2.00 * 4.00 =	8.00
V	Velocity of outlet before enlarging (m/sec) = q / A	

∴ h4 = 797.19388 *10^-6 * q^2