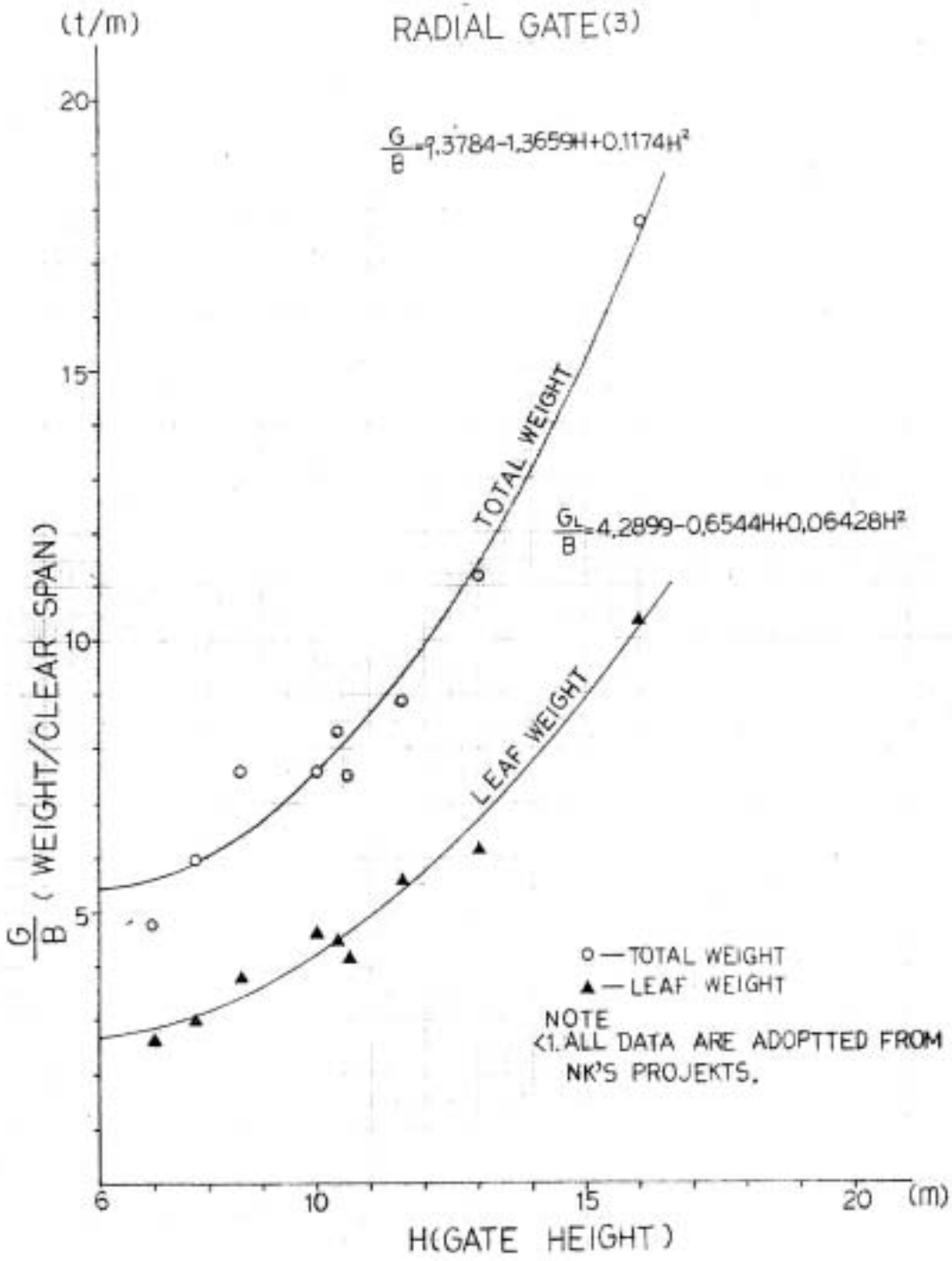
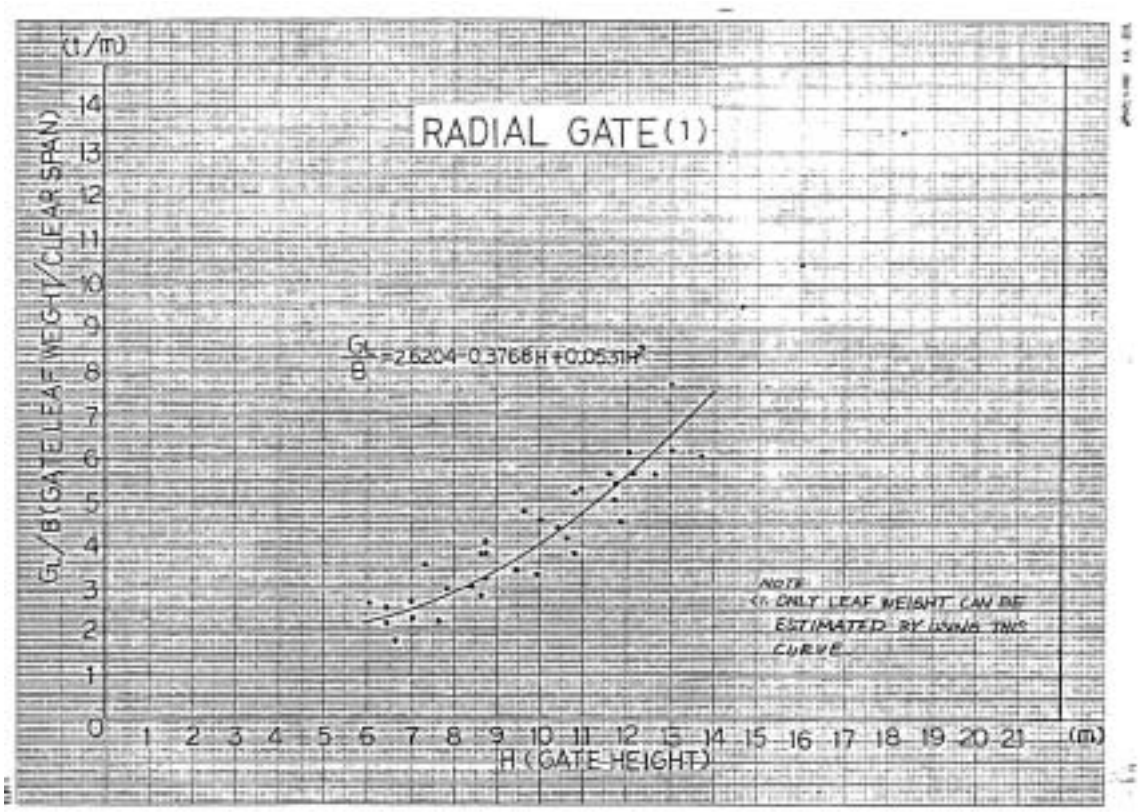
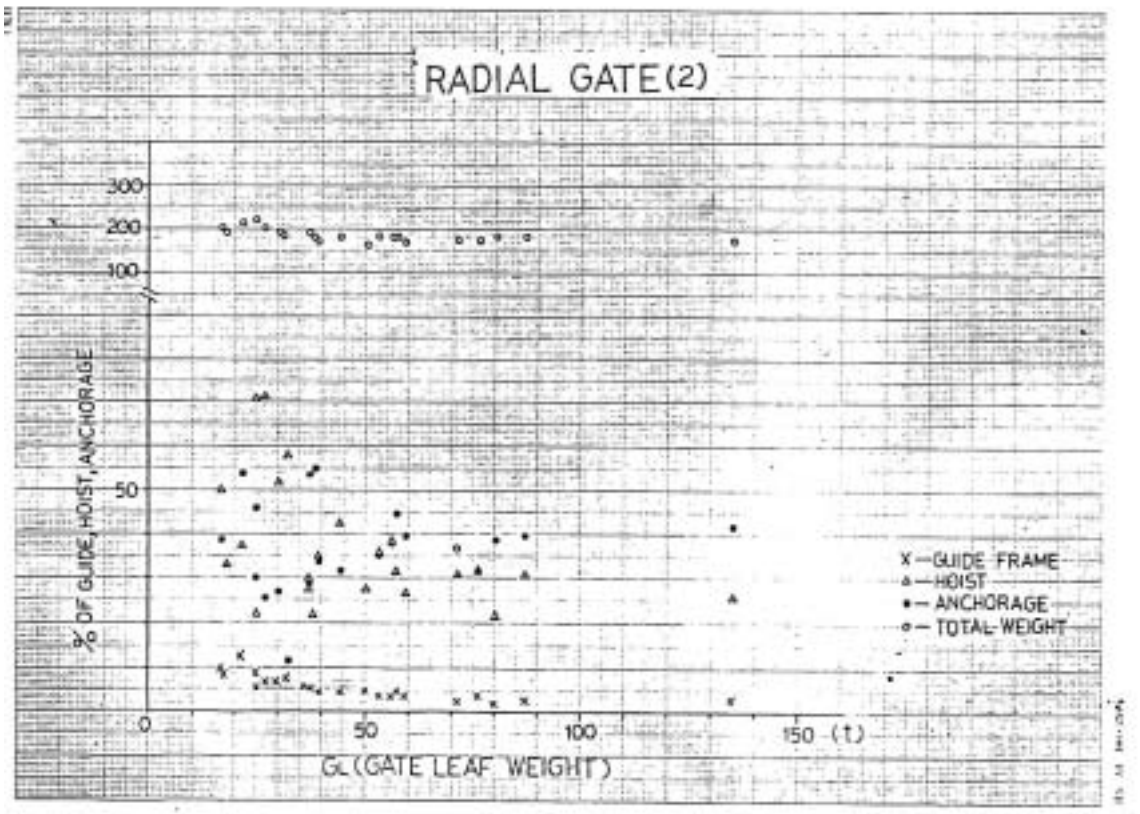
Appendix 1-4 Weight of Gates

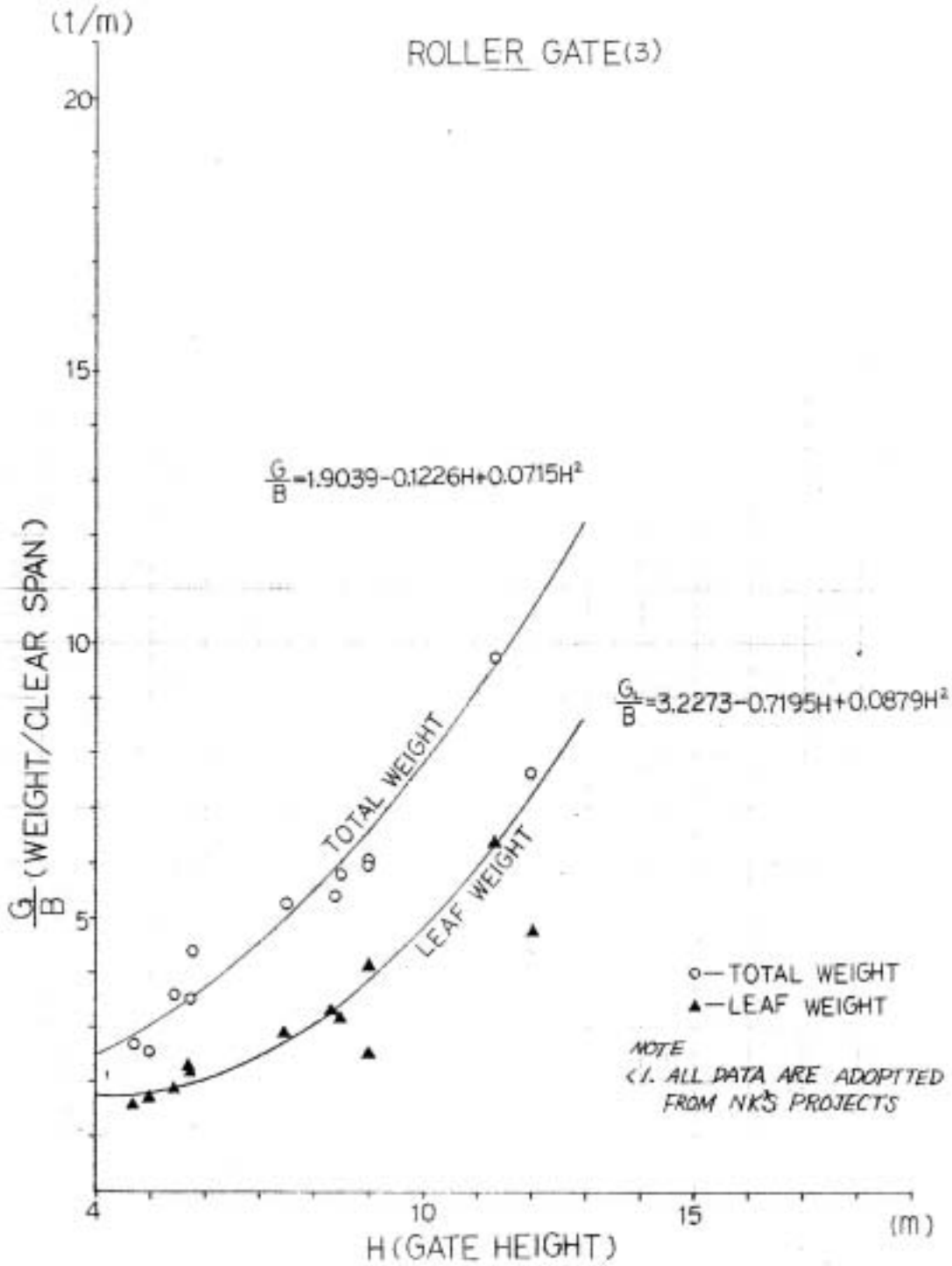


Source: Manual of Gates, Nippon Koei Co., Ltd.

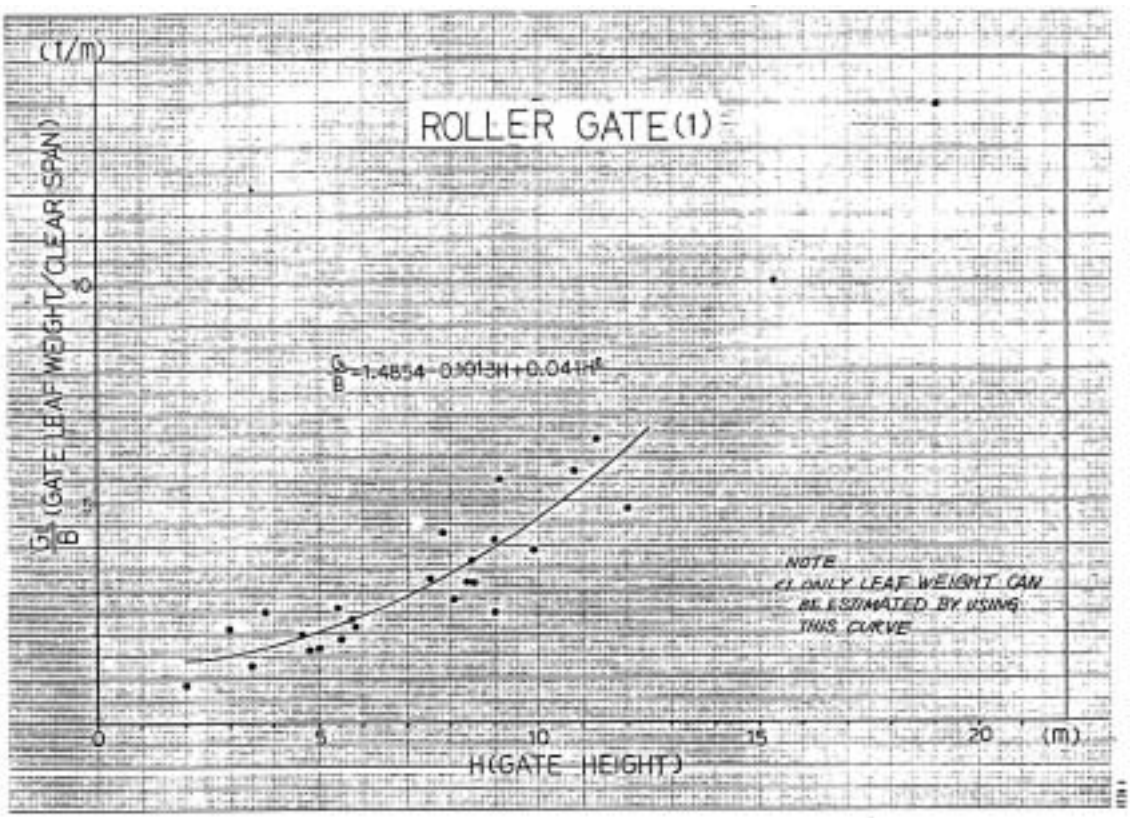


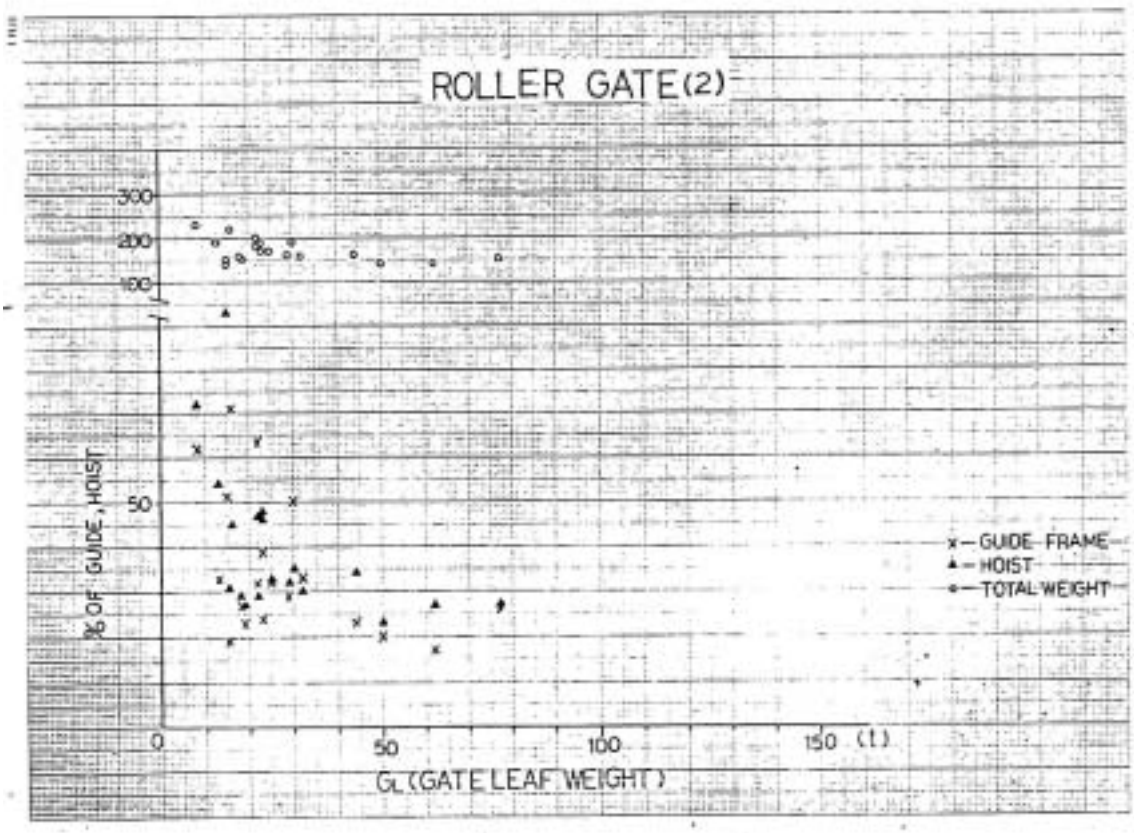


Source: Manual of Gates, Nippon Koei Co., Ltd.

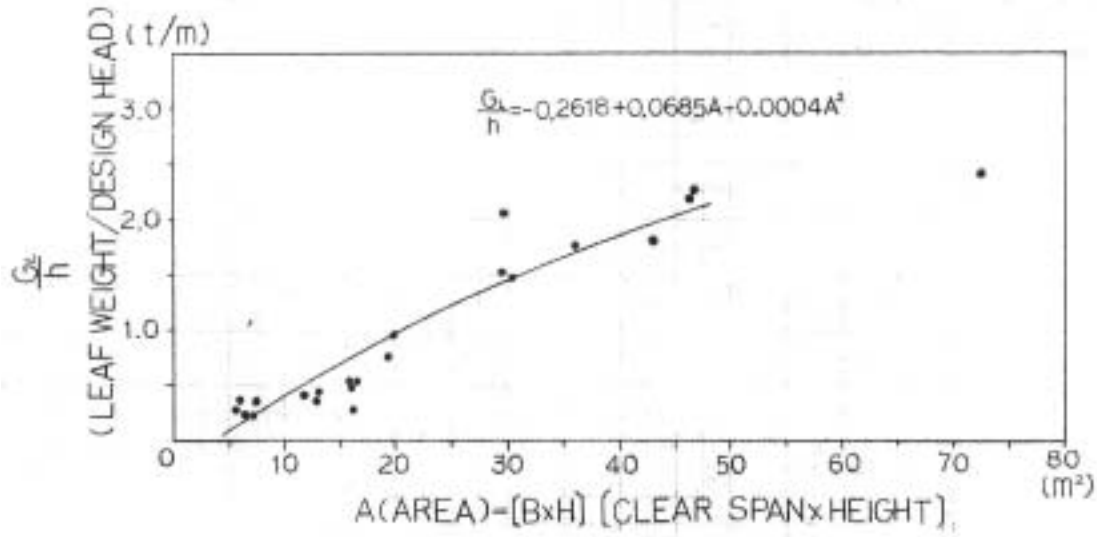


Source: Manual of Gates, Nippon Koei Co., Ltd.





Source: Manual of Gates, Nippon Koei Co., Ltd.

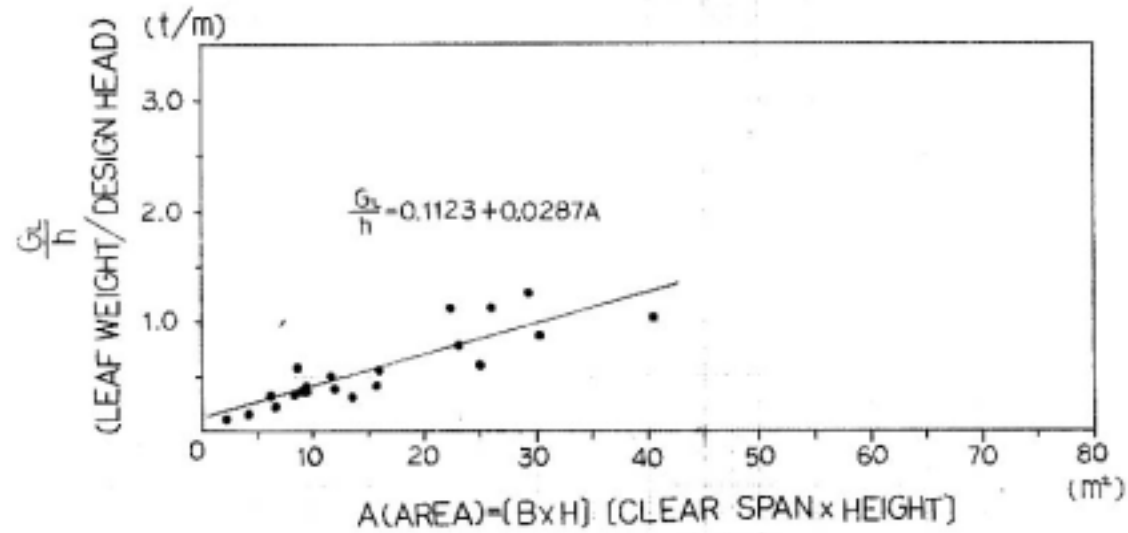


Source: Manual of Gates,

Nippon Koei Co., Ltd.

NOTE
① ONLY LEAF WEIGHT CAN BE ESTIMATED BY USING THIS CURVE.

HIGH HEAD ROLLER GATE

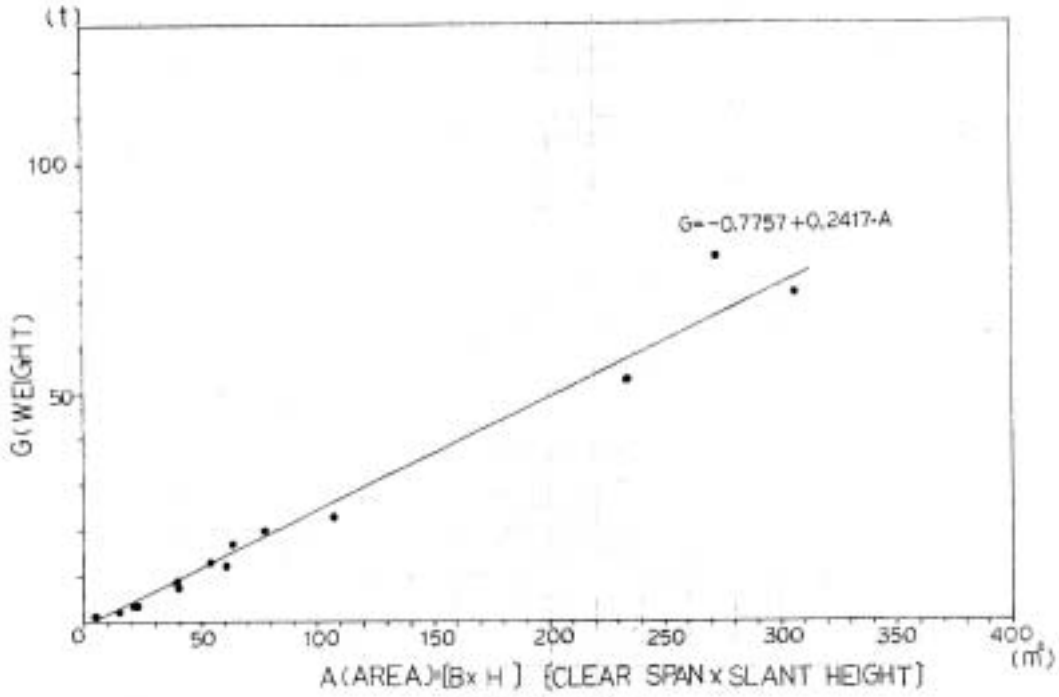


Source: Manual of Gates,

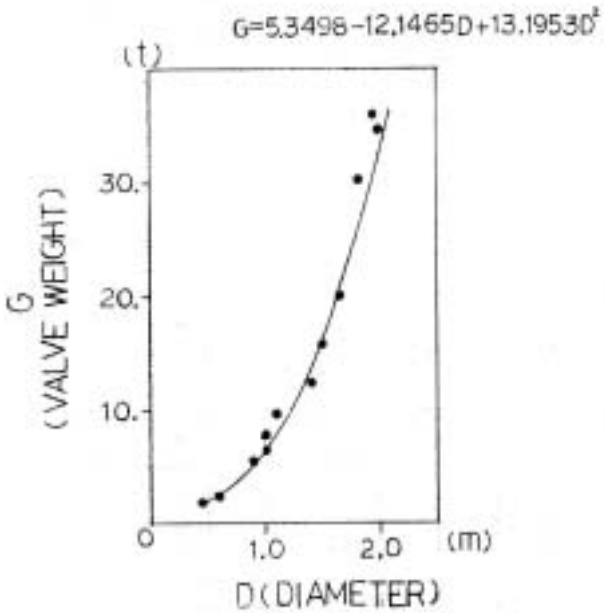
Nippon Koei Co., Ltd.

NOTE
①. ONLY LEAF WEIGHT CAN BE ESTIMATED BY USING THIS CURVE.

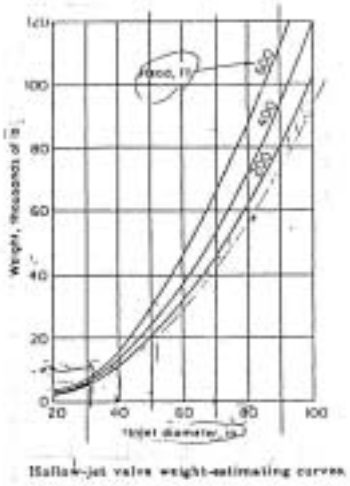
SLIDE GATE



TRASH RACK



NOTE
<1. OPERATED BY HYDRAULIC UNIT.



Hollow-jet valve weight-estimating curves

(Applied Hydraulics)
1 inch = 2.54 cm
1 ft = 0.3048 m
1 lb = 0.45359 kg

HOLLOW JET VALVE

Source: Manual of Gates, Nippon Koei Co., Ltd.

Appendix 2 Computer Programs

Computer Programs for Hydraulic Analyses

Appendix 2-1	Uniform Flow
Appendix 2-2	Non-uniform Flow
Appendix 2-3	Head Loss
Appendix 2-4	Water Flow Profile in Open Chute Spillway
Appendix 2-5	Side Spillway
Appendix 2-6	Surging Analysis
Appendix 2-7	Water Hammer
Appendix 2-8	Flood Routing for Reservoir with Non-gated Spillway
Appendix 2-9	Hydraulic Design of De-silting Basin

Computer Programs for Structural Analyses

Appendix 2-S1	Stability Analysis of Anchor Blocks
Appendix 2-S2	Stress Analysis for Reinforced Concrete

Computer Programs for Miscellaneous Uses

Appendix 2-T1	Coordinates Calculation
Appendix 2-T2	Area Calculation of Polygon

Source: Nippon Koei Co., Ltd

Program (1) Uniform Flow Analysis in Open Channel

Manning's Formula

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

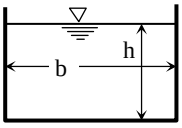
$$Q = V \cdot A$$

Input Data	
Result	
To run	CTRL + f

(1) Rectangular channel

$$A = b \cdot h$$

$$R = \frac{h}{1 + 2h/b}$$

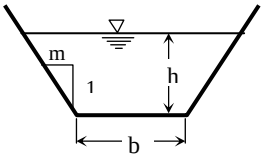


Discharge	Q(m^3/sec)	4.00
Roughness	n	0.0160
Slope	1 / ###	700
Width	b (m)	1.96
Depth	h (m)	1.279
Velocity	v (m/s)	1.595

(2) Trapezoidal channel

$$A = h(b + mh)$$

$$R = \frac{h(b + mh)}{b + 2h\sqrt{1 + m^2}}$$

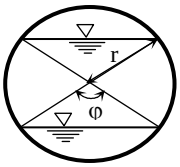


Discharge	Q(m^3/sec)	0.65
Roughness	n	0.02
Slope	1 / ###	27
Width	b (m)	1.5
Wall slope	m	0.5
Depth	h (m)	0.161
Velocity	v (m/s)	2.554

(3) Circular channel

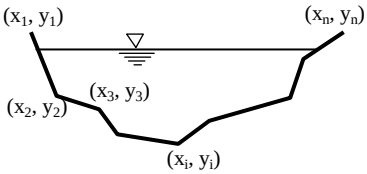
$$A = \frac{1}{2} r^2 (\varphi - \sin\varphi)$$

$$R = r \frac{\varphi - \sin\varphi}{2\varphi}$$



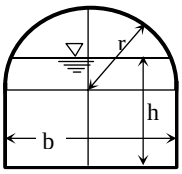
Discharge	Q(m^3/sec)	4.5
Roughness	n	0.014
Slope	1 / ###	1000
Diameter	D (m)	2
Depth	h (m)	1.652
Velocity	v (m/s)	1.621

(4) Coordinates input



Discharge	Q(m^3/sec)	200
Roughness	n	0.025
Slope	1 / ###	500
(x_1, y_1)	0.000	6.500
(x_2, y_2)	2.000	3.500
(x_3, y_3)	5.000	2.000
(x_4, y_4)	7.000	0.000
(x_5, y_5)	11.000	0.000
(x_6, y_6)	13.600	6.500
(x_7, y_7)		
(x_8, y_8)		
(x_9, y_9)		
(x_{10}, y_{10})		
Depth	h (m)	6.257
Velocity	v (m/s)	3.544

(5) D-Shape



2 r = b

Discharge	Q(m^3/sec)	4.50
Roughness	n	0.0140
Slope	1 / ###	1000
Width	b (m)	2.00
Depth	h (m)	1.446
Velocity	v (m/s)	1.573

Program (2) Non-Uniform Flow Analysis (Input)

Title	Main Intake H-Q Curve	
Initial WL.	999.000	Input arbitrary WL.(m), or **888** for critical WL. of rectangular section, or **999** for uniform depth for rectangular/trapezoidal/circular sections
Slope	891.608	Input slope ### (1 / ###) only when B4=999

Sec. No.	Distance (m)	Discharge (m3/s)	Roughness n	Bed EL. (m) No need for Shape-5	Wall crest EL. (m) For freeboard in need be	Shape								
						1 (rectangle)	width (m)							
						2 (trapezoid)	B.width(m)	side slope						
						3 (circle)	dia.(m)							
						4 (D-shape)	width (m)	dia.(m)						
						5 (coordinate)	x1(m)	y1(m)	x2(m)	y2(m)	x3(m)	y3(m)	x4(m)	y4(m)
1	50.000	20.400	0.014	1,377.080	1,380.480	3	3.400							
2	10.000	20.400	0.014	1,377.136	1,380.536	3	3.400							
3	10.000	20.400	0.014	1,377.147	1,380.547	3	3.400							
4	12.768	20.400	0.014	1,377.159	1,380.559	4	3.400	3.400						
5	12.000	20.400	0.014	1,377.173	1,380.573	4	3.400	3.400						
6	12.000	20.400	0.014	1,377.186	1,380.586	4	3.400	3.400						
7	5.000	20.400	0.014	1,377.200	1,380.600	4	3.400	3.400						
8	5.000	20.400	0.014	1,377.200	1,380.600	1	3.400							
9	5.000	20.400	0.014	1,377.200	1,380.600	1	3.400							
10	5.000	20.400	0.014	1,377.200	1,380.600	1	3.400							
11	5.600	20.400	0.014	1,377.200	1,380.600	1	3.400							
12	5.000	20.400	0.014	1,377.200	1,382.200	1	3.400							
13	3.000	20.400	0.014	1,377.200	1,382.200	1	10.000							
14	3.000	20.400	0.014	1,377.200	1,382.200	1	10.000							
15		20.400	0.014	1,377.200	1,382.200	1	10.000							

Program (2) WATER SURFACE PROFILE RESULTS

Title : **Main Intake H-Q Curve**

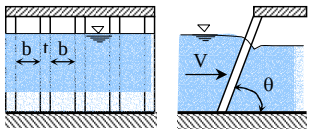
	Shape
Rectangle	1
Trapezoid	2
Circle	3
C & R	4
(x, y)	5

No.	Q	EL.	Distance	n	Width	Dia.	Depth	WL.	F.B.	V	Shape
	(m3/s)	(m)	(m)		(m)	(m)	(m)	(m)	(m)	(m/s)	
1	20.400	1,377.080	0.000	0.014		3.400	2.957	1,380.037	0.443	2.434	3
2	20.400	1,377.136	50.000	0.014		3.400	2.957	1,380.093	0.443	2.434	3
3	20.400	1,377.147	60.000	0.014		3.400	2.957	1,380.104	0.443	2.434	3
4	20.400	1,377.159	70.000	0.014	3.400	3.400	3.046	1,380.205	0.354	2.078	4
5	20.400	1,377.173	82.768	0.014	3.400	3.400	3.042	1,380.215	0.358	2.079	4
6	20.400	1,377.186	94.768	0.014	3.400	3.400	3.039	1,380.225	0.361	2.081	4
7	20.400	1,377.200	106.768	0.014	3.400	3.400	3.035	1,380.235	0.365	2.082	4
8	20.400	1,377.200	111.768	0.014	3.400		3.068	1,380.268	0.332	1.956	1
9	20.400	1,377.200	116.768	0.014	3.400		3.072	1,380.272	0.328	1.953	1
10	20.400	1,377.200	121.768	0.014	3.400		3.075	1,380.275	0.325	1.951	1
11	20.400	1,377.200	126.768	0.014	3.400		3.079	1,380.279	0.321	1.949	1
12	20.400	1,377.200	132.368	0.014	3.400		3.083	1,380.283	1.917	1.946	1
13	20.400	1,377.200	137.368	0.014	10.000		3.276	1,380.476	1.724	0.623	1
14	20.400	1,377.200	140.368	0.014	10.000		3.276	1,380.476	1.724	0.623	1
15	20.400	1,377.200	143.368	0.014	10.000		3.276	1,380.476	1.724	0.623	1

Program(3) Head Loss Calculation for 2 Units Operation in Nam Lan Small Hydro Scheme

:Input :Result

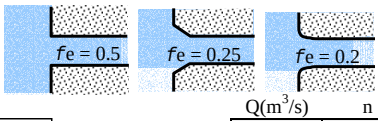
FSWL at Regulating Pond	688.000
TWL at Tailrace	618.500
Discharge (m^3/sec)	0.650
Combined Efficiency of Turbine & Generator	0.733
Units of Turbine & Generator	2



PENSTOCK

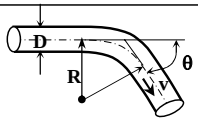
		B(m)	H(m)	f_r	Velocity (m/s)	Head Loss (m)	Coefficient x 10^{-6} Q^2
(1) Screen	$h_{11} = f_r \cdot \frac{v_1^2}{2g}$; $f_r = 2.34(\sin\theta)(\frac{t}{b})^{4/3}$	3.00	0.30	0.97	0.722	0.0258	61,098.51

		D (m)	f_e			
(2) Inlet	$h_{12} = f_e \cdot \frac{v_2^2}{2g}$	0.80	0.20	1.293	0.0171	40,386.31

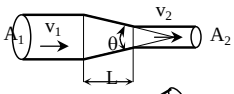


		Q(m^3/s)	n	D (m)	L (m)			
(3) Friction	$h_{13} = \frac{124.5n^2}{D^{4/3}} L \frac{v^2}{2g}$	0.65	0.0120	0.80	178.49	1.293	0.3676	870,063.87
		0.33	0.0120	0.40	3.55	2.586	0.0738	174,573.35

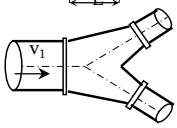
		Q(m^3/s)	R	D(m)	θ (°)			
(4) Bend	$h_{14} = \{0.131 + 0.1632 \cdot (\frac{D}{R})^{3.5}\} \cdot (\frac{\theta}{90})^{0.5} \cdot \frac{v^2}{2g}$	0.65	2.400	0.80	23.77	1.293	0.0059	13,956.83
		0.65	2.400	0.80	2.06	1.293	0.0017	4,108.71
		0.65	2.400	0.80	9.04	1.293	0.0036	8,607.09
		0.65	2.400	0.80	17.24	1.293	0.0050	11,886.14
		0.33	1.200	0.40	30.00	2.586	0.0265	62,718.09



		D_1(m)	D_2(m)	L(m)	$f_{transition}$			
(5) Transition	$h_{15} = f_{gc} \frac{v_2^2}{2g}$							



		D_0(m)	f_b			
(6) Branch	$h_{16} = f_b \cdot \frac{v_1^2}{2g}$	0.80	0.500	1.293	0.0427	100,965.78



		Q(m^3/s)	D(m)	f_{valve}			
(7) Inlet valve	$h_{17} = f_v \cdot \frac{v^2}{2g}$	0.33	0.40	0.250	2.586	0.0853	201,931.57

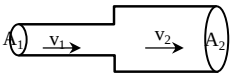
(8) Others	Round-up	0.0050	11,833.92
Sub-total (1) - (8)		0.6600	1,562,130.18

TAILRACE

Discharge Q(m^3/s): 0.33

		n	D_1(m)	D_2(m)	L(m)	Velocity (m/s)	Head Loss (m)	Coefficient x 10^{-6} Q^2
(1) Friction in transition	$h_{21} = \frac{2.37 \cdot n^2 Q^2 L}{D_2 - D_1} \cdot (\frac{1}{D_1^{13/3}} - \frac{1}{D_2^{13/3}})$							

		D_1(m)	D_2(m)	f_{ge}			
(2) Enlargement	$h_{21} = f_{ge}\{1 - (\frac{A_1}{A_2})\}^2 \cdot \frac{v_1^2}{2g}$						



		D(m)	f_{exit}			
(3) Exit	$h_{23} = f_{exit} \cdot \frac{v^2}{2g}$	0.80	1.000	0.647	0.0213	50,482.89

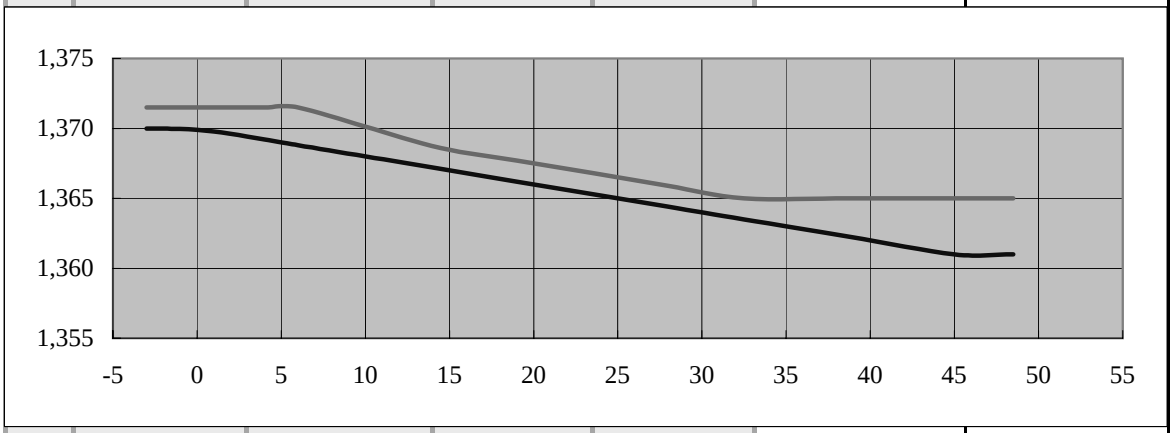
(4) Others	10%	0.0021	5,048.29
Sub-total		0.0235	55,531.18
Total of Head Loss		**0.6835**	**1,617,661.36**

Gross Head (m)	**69.500**
Effective Head (m)	**68.817**
Power Output (kW)	**320**

Program (4) Program Input Form for Spillway Water Surface Profile (Rectangle or Trapezoidal Section)

(no need to input)

No.	Bottom EL. (m)	Horiz. Distance (m)	Width (m)	Radius (m)	Accum. Distance (m)	Wall Crest (m)
1	1,370.000	1.02	40.0	0.0	-3.00	1,371.500
2	1,370.000	1.98	40.0	20.0	-1.98	1,371.500
3	1,369.902	1.94	40.0	0.0	0.00	1,371.500
4	1,369.612	2.06	40.0	0.0	1.94	1,371.500
5	1,369.200	2.00	40.0	0.0	4.00	1,371.500
6	1,368.800	4.00	40.0	0.0	6.00	1,371.500
7	1,368.000	4.50	40.0	0.0	10.00	1,370.135
8	1,367.100	4.50	40.0	0.0	14.50	1,368.600
9	1,366.200	4.50	40.0	0.0	19.00	1,367.700
10	1,365.300	4.50	40.0	0.0	23.50	1,366.800
11	1,364.400	4.50	40.0	0.0	28.00	1,365.900
12	1,363.500	6.25	40.0	0.0	32.50	1,365.000
13	1,362.250	6.25	40.0	0.0	38.75	1,365.000
14	1,361.000	3.50	40.0	0.0	45.00	1,365.000
15	1,361.000	0.00	40.0	0.0	48.50	1,365.000



Note: The solution may not converge at the discharge of 20 m3/s around. At that case, it may be solved by adjusting the initial water depth H1 of Sub Water_Depth() and difference H2 smaller

Program (4) Water Surface Profile in Spillway (Output)

Regulating Pod : Spillway
Discharge(m^3/sec) : 24.000
Roughness coefficient : 0.014

No.	EL. (m)	Width (m)	Radius (m)	H.D. (m)	A.H.D. (m)	C.D. (m)	Depth (m)	WL. (m)	V (m/s)	F.B. (m)	Fr.
1	1,370.000	40.000	0.000		0.000	0.343	0.343	1,370.343	1.748	0.655	1.000
2	1,370.000	40.000	20.000	1.020	1.020	0.342	0.343	1,370.343	1.748	0.655	1.000
3	1,369.902	40.000	0.000	1.980	3.000	0.344	0.236	1,370.139	2.539	0.668	1.754
4	1,369.612	40.000	0.000	1.942	4.942	0.345	0.173	1,369.787	3.477	0.682	2.825
5	1,369.200	40.000	0.000	2.058	7.000	0.346	0.139	1,369.342	4.303	0.693	3.899
6	1,368.800	40.000	0.000	2.000	9.000	0.346	0.124	1,368.926	4.846	0.699	4.660
7	1,368.000	40.000	0.000	4.000	13.000	0.346	0.108	1,368.110	5.546	0.708	5.705
8	1,367.100	40.000	0.000	4.500	17.500	0.346	0.100	1,367.202	5.996	0.713	6.413
9	1,366.200	40.000	0.000	4.500	22.000	0.346	0.096	1,366.298	6.230	0.716	6.792
10	1,365.300	40.000	0.000	4.500	26.500	0.346	0.094	1,365.396	6.353	0.717	6.995
11	1,364.400	40.000	0.000	4.500	31.000	0.346	0.094	1,364.496	6.396	0.718	7.065
12	1,363.500	40.000	0.000	4.500	35.500	0.346	0.093	1,363.595	6.439	0.718	7.136
13	1,362.250	40.000	0.000	6.250	41.750	0.346	0.093	1,362.345	6.439	0.718	7.136
14	1,361.000	40.000	0.000	6.250	48.000	0.344	0.093	1,361.094	6.439	0.718	7.084
15	1,361.000	40.000	0.000	3.500	51.500	0.343	0.108	1,361.108	5.578	0.708	5.698

Program (5) Water Surface Profile in Side Spillway by De-Marchi (Input)

Title		Main Intake Side Spillway
Discharge at D/S Edge	[m3/s]	17.000
WL. at D/S Edge	[m]	1,379.980
EL. of Spillway Crest	[m]	1,379.900
Length of Spillway	[m]	30.000
Width of Channel	[m]	15.000
Floor EL. Of D/S Edge	[m]	1,377.200

: Imput

Program (5) Water Surface Profile in Side Spillway (Output)

Main Intake Side Spillway

Qtotal	18.235	[m3/s]
Qspill	1.235	[m3/s]
Qdown	17.000	[m3/s]

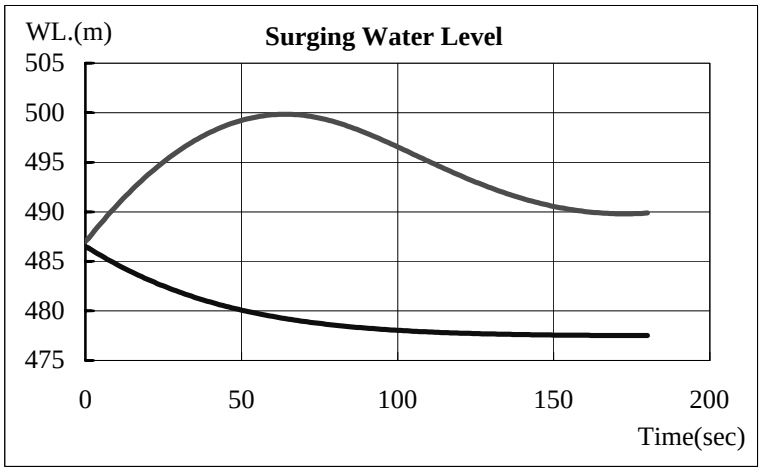
Spillway crest EL.	1,379.900	[m]
Spillway length	30.000	[m]
Channel width	15.000	[m]
Channel bottom EL.	1,377.200	[m]

Distance [m]	Qspill [m3/s/m]	Qtotal [m3/s]	Depth [m]	WL. [m]	Velocity [m/s]	H/E	ϕ(H/E)	Fr.
0.000		17.000	2.780	1,379.980	0.408	0.997	-10.228	0.078
1.000	0.042	17.042	2.780	1,379.980	0.409	0.997	-10.256	0.078
2.000	0.042	17.083	2.780	1,379.980	0.410	0.997	-10.283	0.078
3.000	0.042	17.125	2.780	1,379.980	0.411	0.997	-10.311	0.079
4.000	0.042	17.166	2.780	1,379.980	0.412	0.997	-10.339	0.079
5.000	0.042	17.208	2.780	1,379.980	0.413	0.997	-10.366	0.079
6.000	0.041	17.249	2.780	1,379.980	0.414	0.997	-10.394	0.079
7.000	0.041	17.291	2.780	1,379.980	0.415	0.997	-10.422	0.079
8.000	0.041	17.332	2.780	1,379.980	0.416	0.997	-10.449	0.080
9.000	0.041	17.374	2.780	1,379.980	0.417	0.997	-10.477	0.080
10.000	0.041	17.415	2.780	1,379.980	0.418	0.997	-10.505	0.080
11.000	0.041	17.456	2.780	1,379.980	0.419	0.997	-10.533	0.080
12.000	0.041	17.498	2.780	1,379.980	0.420	0.997	-10.560	0.080
13.000	0.041	17.539	2.780	1,379.980	0.421	0.997	-10.588	0.081
14.000	0.041	17.580	2.779	1,379.979	0.422	0.997	-10.616	0.081
15.000	0.041	17.621	2.779	1,379.979	0.423	0.997	-10.643	0.081
16.000	0.041	17.662	2.779	1,379.979	0.424	0.997	-10.671	0.081
17.000	0.041	17.703	2.779	1,379.979	0.425	0.997	-10.699	0.081
18.000	0.041	17.744	2.779	1,379.979	0.426	0.997	-10.726	0.082
19.000	0.041	17.786	2.779	1,379.979	0.427	0.997	-10.754	0.082
20.000	0.041	17.827	2.779	1,379.979	0.428	0.997	-10.782	0.082
21.000	0.041	17.868	2.779	1,379.979	0.429	0.997	-10.809	0.082
22.000	0.041	17.908	2.779	1,379.979	0.430	0.997	-10.837	0.082
23.000	0.041	17.949	2.779	1,379.979	0.431	0.997	-10.865	0.083
24.000	0.041	17.990	2.779	1,379.979	0.432	0.997	-10.892	0.083
25.000	0.041	18.031	2.779	1,379.979	0.433	0.997	-10.920	0.083
26.000	0.041	18.072	2.779	1,379.979	0.434	0.997	-10.948	0.083
27.000	0.041	18.113	2.779	1,379.979	0.435	0.997	-10.976	0.083
28.000	0.041	18.153	2.779	1,379.979	0.436	0.997	-11.003	0.083
29.000	0.041	18.194	2.779	1,379.979	0.436	0.997	-11.031	0.084
30.000	0.041	18.235	2.779	1,379.979	0.437	0.997	-11.059	0.084

Program (6) SURGING ANALYSIS (Outout)

Discharge(m3/s)	5
HWL. (m)	493
LWL. (m)	489.5
Headrace	
Dia. (m)	2.132
Length(m)	319.1
Surge Tank	
Dia. (m)	4
EL. (m)	0
Port dia. (m)	0.8
Cin	0.9
Cout	0.6
Turbine	
Closing (sec)	0.01
Opening (sec)	0.01
Headloss (m)	
load rejection	6
load demand	12
Load Rejection	100 % to 0 %
Load Demand	50

2	2.343	2.2
900	1878.7	100



Time (sec)	Up Surging under Full Load Rejection			Down Surging under Partial Load Demand		
	WL. (m)	Q(headrace) (m3/s)	Q(turbine) (m3/s)	WL. (m)	Q(headrace) (m3/s)	Q(turbine) (m3/s)
0	487	5	5	486.5	2.5	2.5
1	487.329636	4.939095932	0	486.3353001	2.534110583	5
2	487.7197549	4.865589233	0	486.1406938	2.57486592	5
3	488.1040073	4.791659396	0	485.9493224	2.615409625	5
4	488.4823607	4.717335259	0	485.7611683	2.655720571	5
5	488.8547848	4.6426435	0	485.5762121	2.695779028	5
6	489.2212513	4.567608811	0	485.3944329	2.7355666	5
7	489.5817339	4.49225406	0	485.2158086	2.775066157	5
8	489.9362078	4.416600433	0	485.0403156	2.81426177	5
9	490.2846503	4.340667572	0	484.8679291	2.85313865	5
10	490.6270396	4.264473692	0	484.6986232	2.891683094	5
11	490.9633559	4.188035701	0	484.5323709	2.929882428	5
12	491.2935802	4.111369295	0	484.3691443	2.967724959	5
13	491.6176949	4.034489058	0	484.2089146	3.005199919	5
14	491.9356837	3.957408547	0	484.051652	3.042297426	5
15	492.2475311	3.880140371	0	483.8973263	3.079008435	5
16	492.5532226	3.802696268	0	483.7459062	3.115324697	5
17	492.8527446	3.725087164	0	483.5973602	3.151238719	5
18	493.1460845	3.647323244	0	483.4516559	3.186743725	5
19	493.4332303	3.569414004	0	483.3087605	3.221833621	5
20	493.7141708	3.491368301	0	483.1686408	3.25650296	5
21	493.9888954	3.413194407	0	483.031263	3.290746908	5
22	494.2573944	3.334900047	0	482.8965933	3.324561214	5
23	494.5196583	3.256492444	0	482.7645972	3.357942179	5
24	494.7756784	3.177978354	0	482.6352401	3.39088663	5
25	495.0254466	3.099364104	0	482.5084872	3.42339189	5
26	495.268955	3.020655617	0	482.3843034	3.455455753	5
27	495.5061965	2.941858448	0	482.2626536	3.487076461	5
28	495.7371641	2.862977809	0	482.1435023	3.518252679	5
29	495.9618515	2.784018592	0	482.0268143	3.548983474	5

Program (6) Sursing Analysis Data Input

: input data

Basic Data

Discharge	(m^3/sec)	5.000
H.W.L. (m)	(m)	493.000
L.W.L. (m)	(m)	489.500
Max head loss (Load rejection)	(m)	12.000
Min. head loss (Load increase)	(m)	6.000

Note: velocity head is included

Headrace Data

Nos. of section		4			
Tunnel inner diameter	(m)	2.132	2.000	2.343	2.200
Tunnel section length	(m)	319.100	900.000	1,878.700	100.000

Surse Tank Data

Nos. of section		1			
Tank diameter	(m)	4.000			
Bottom elevation	(m)	0.000			
Diameter of port	(m)	0.800			
Discharge coefficient（inflow）		0.900			
Discharge coefficient（outflow）		0.600			

Turbine Data

Closing time	(sec)	0.010
Opening time	(sec)	0.010

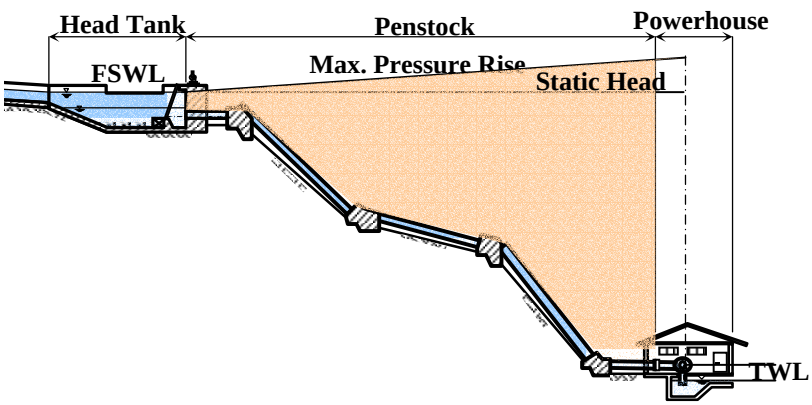
Program (7) Water Hammer Analysis for Penstock without Y-Branch

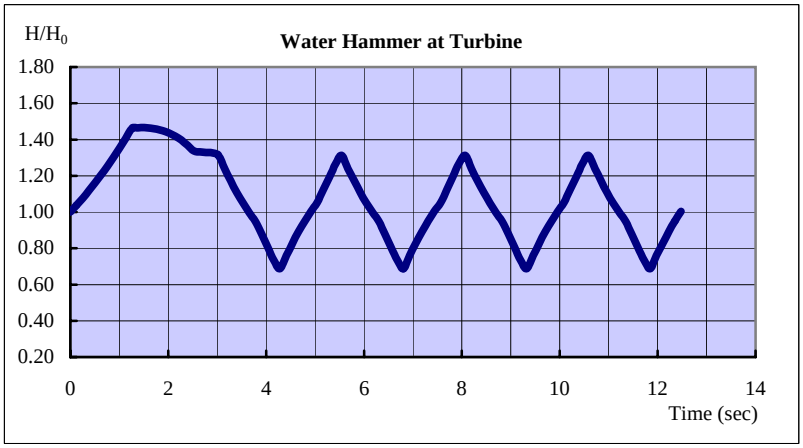
FSWL at Head Tank	(m)	160.0
TWL at Powerhouse	(m)	0.0
Design Discharge	(m^3/s)	8.0
Initial Velocity at Turbine	(m/s)	2.55
Closing Time of Turbine	(sec)	3.0
Calculation Step	(sec)	0.100

Penstock Pipes

	L_i (m)	D_i (m)	t_i (mm)
1	600	2	16.384
2			
3			
4			
5			

CTRL + f for computation





Time (sec)	H/H_0	Head (m)
0.000	1.000	160.00
0.126	1.037	166.00
0.252	1.077	172.27
0.378	1.118	178.84
0.504	1.161	185.70
0.630	1.205	192.88
0.756	1.252	200.38
0.882	1.301	208.23
1.008	1.353	216.45
1.134	1.406	225.03
1.261	1.463	234.01
1.387	1.465	234.45
1.513	1.466	234.49
1.639	1.463	234.08
1.765	1.457	233.18
1.891	1.448	231.73
2.017	1.436	229.68
2.143	1.418	226.95
2.269	1.397	223.48
2.395	1.370	219.18
2.521	1.337	213.96
2.647	1.332	213.18
2.773	1.329	212.60
2.899	1.327	212.29
3.025	1.312	209.96
3.151	1.241	198.49
3.277	1.172	187.59
3.403	1.108	177.34
3.529	1.049	167.87
3.655	0.996	159.29
3.782	0.949	151.78
3.908	0.878	140.46
4.034	0.806	129.00
4.160	0.733	117.30
4.286	0.688	110.04
4.412	0.759	121.51
4.538	0.828	132.41
4.664	0.892	142.66
4.790	0.951	152.13
4.916	1.004	160.71
5.042	1.051	168.22
5.168	1.122	179.54
5.294	1.194	191.00
5.420	1.267	202.70

Program (7) Water Hammer Reference Equations

Wave Velocity of Water Hammer

$$a = \frac{1}{\sqrt{\frac{w_0}{g}[\frac{1}{K} + \frac{1}{E} \cdot \frac{D}{t}]}}$$

Equivalent Sectional Area

$$A_m = \frac{\sum L_i}{\sum (L_i / A_i)}$$

Equivalent Wave Velocity

$$a_m = \frac{\sum L_i}{\sum (L_i / a_i)}$$

$$t_i = t_1 + (i-1) \cdot \mu$$

$$\psi_i = 1 - \frac{i \cdot \mu}{T}$$

Allievi's Equations

(1) 1st phase

$$H_1^{'} - 1 = 2\rho \cdot (1 - \psi_1 \sqrt{H_1^{'}})$$

$$\mu = \frac{2L}{a}$$

$$H_1^{'2} - (2 + 4\rho + 4\rho^2 \psi_1^2) H_1^{'} + (1 + 2\rho)^2 = 0$$

$$\rho = \frac{a \cdot V_0}{2g \cdot H_0}$$

$$\rho = \frac{a \cdot V_0}{2g \cdot H_0}$$

$$H_i^{'} = \frac{h_i + H_0}{H_0}$$

(2) after 2nd phase

$$H_{i-1}^{'} + H_i^{'} - 2 = 2\rho \cdot (\psi_{i-1} \sqrt{H_{i-1}^{'}} - \psi_i \sqrt{H_i^{'}})$$

$$H_i^{'2} - (B - 2A) \cdot H_i^{'} + A^2 = 0$$

$$A = H_{i-1}^{'} - 2 - 2\rho\psi_{i-1} \sqrt{H_{i-1}^{'}}$$

$$B = (2\rho\psi_i)^2$$

Program (8) Flood Routing for Reservoir with Non-gated Spillway (Input)

$$\frac{dV}{dt} = \frac{dV}{dH} \cdot \frac{dH}{dt} = Area(H) \cdot \frac{dH}{dt} = Q_{in} - Q_{out}$$

$$\frac{dH}{dt} = \frac{Q_{in} - Q_{out}}{Area(H)}$$

The above equation can be solved numerically by Runge- Kutta Method.

Initial WL. (m)	Time Step (hour)
150.00	0.50

WL - Area

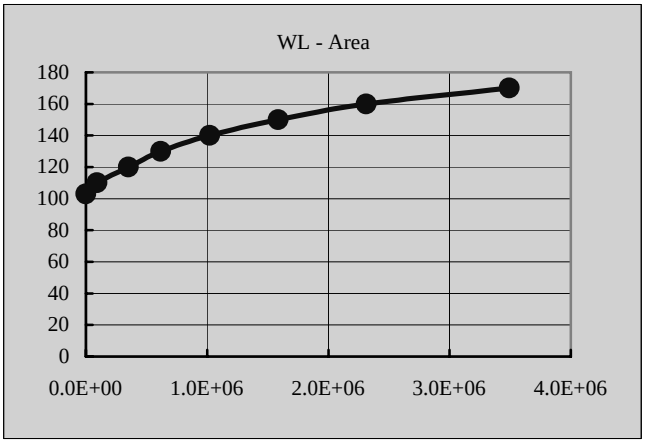
	WL. (m)	Area (m2)
1	103.00	0
2	110.00	93,000
3	120.00	351,000
4	130.00	617,000
5	140.00	1,021,000
6	150.00	1,587,000
7	160.00	2,313,000
8	170.00	3,493,000
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

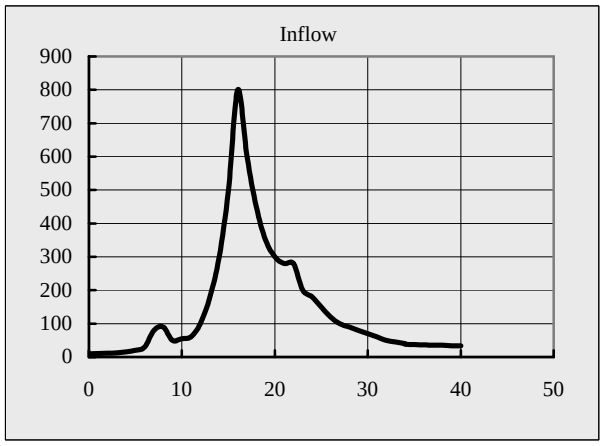
Outflow

	WL. (m)	Outflow (m3/sec)
1	150.00	0.00
2	151.00	72.00
3	152.00	203.65
4	153.00	374.12
5	154.00	576.00
6	155.00	804.98
7	156.00	1,058.18
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Inflow

	Time (hour)	Inflow (m^3/sec)		Time (hour)	Inflow (m^3/sec)
1	0.00	10.00	36	34.00	38.00
2	1.00	11.00	37	36.00	36.00
3	2.00	11.50	38	37.00	35.00
4	3.00	12.00	39	38.00	35.00
5	4.00	15.00	40	39.00	34.00
6	5.00	20.00	41	40.00	34.00
7	6.00	30.00	42		
8	7.00	80.00	43		
9	8.00	90.00	44		
10	9.00	50.00	45		
11	10.00	55.00	46		
12	11.00	60.00	47		
13	12.00	100.00	48		
14	13.00	180.00	49		
15	14.00	300.00	50		
16	15.00	500.00	51		
17	16.00	800.00	52		
18	17.00	600.00	53		
19	18.00	450.00	54		
20	19.00	350.00	55		
21	20.00	300.00	56		
22	21.00	280.00	57		
23	22.00	280.00	58		
24	23.00	200.00	59		
25	24.00	180.00	60		
26	25.00	150.00	61		
27	26.00	120.00	62		
28	27.00	100.00	63		
29	28.00	90.00	64		
30	29.00	80.00	65		
31	30.00	70.00	66		
32	31.00	60.00	67		
33	32.00	50.00	68		
34	33.00	45.00	69		
35	34.00	40.00	70		

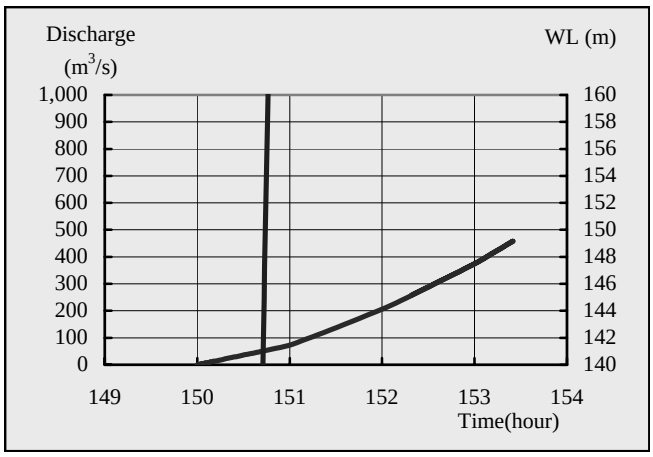


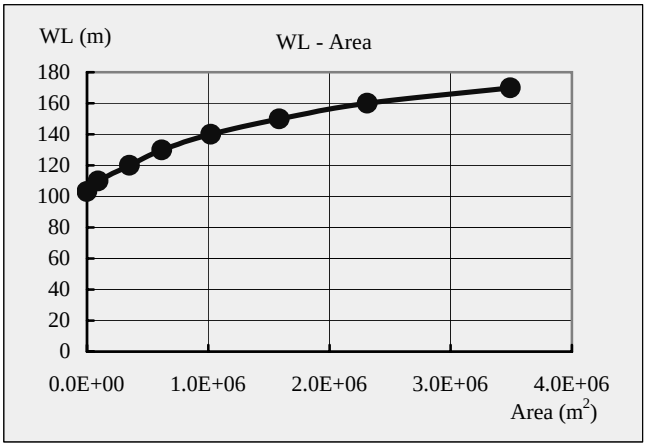


Program (8) Flood Routing for Reservoir with Non-gated Spillway (Input)

Project :
Case :

Time (hour)	WL (m)	Inflow (m)	Outflow (m)
0.00	150.00	10.00	0.00
0.50	150.01	10.50	0.80
1.00	150.02	11.00	1.58
1.50	150.03	11.25	2.33
2.00	150.04	11.50	3.04
2.50	150.05	11.75	3.71
3.00	150.06	12.00	4.35
3.50	150.07	13.50	5.01
4.00	150.08	15.00	5.73
4.50	150.09	17.50	6.55
5.00	150.10	20.00	7.51
5.50	150.12	25.00	8.68
6.00	150.14	30.00	10.15
6.50	150.18	55.00	12.68
7.00	150.24	80.00	16.96
7.50	150.31	85.00	22.04
8.00	150.38	90.00	27.10
8.50	150.43	70.00	31.16
9.00	150.46	50.00	33.37
9.50	150.48	52.50	34.75
10.00	150.50	55.00	36.21
10.50	150.52	57.50	37.74
11.00	150.55	60.00	39.35
11.50	150.58	80.00	41.71
12.00	150.63	100.00	45.41
12.50	150.71	140.00	51.11
13.00	150.82	180.00	59.39
13.50	150.98	240.00	70.78
14.00	151.19	300.00	96.39
14.50	151.44	400.00	129.86
15.00	151.76	500.00	171.59
15.50	152.15	650.00	229.15
16.00	152.62	800.00	308.82
16.50	153.02	700.00	378.71
17.00	153.26	600.00	427.61
17.50	153.38	525.00	451.70
18.00	153.42	450.00	457.91
18.50	153.39	400.00	451.85
19.00	153.32	350.00	437.86
19.50	153.23	325.00	419.66
20.00	153.13	300.00	400.17
20.50	153.03	290.00	381.01
21.00	152.95	280.00	365.04
21.50	152.87	280.00	351.70
22.00	152.80	280.00	340.42
22.50	152.73	240.00	327.65
23.00	152.63	200.00	310.52
23.50	152.52	190.00	292.14
24.00	152.42	180.00	275.03
24.50	152.32	165.00	258.58





Example of Programmed Hydraulic Calculations

1. Installation Width of Bar Screen

The river flow forms a control section on the screen of weir.
For the discharge at the 2nd stage (Q = 0.86m^3/s), accordingly, the critical depth (h_c) of a rectangular section is calculated as follows:

$$hc = \left(\frac{\alpha Q^2}{g B^2} \right)^{\frac{1}{3}}$$

Where,	a:	Adjustment coefficient		=1.1	
	Q:	Discharge	for the 1st stage	=0.43	m^3/s
		Discharge	for the 2nd stage	=0.86	m^3/s
	g:	Gravity acceleration		=9.8	m/s^2
	B:	Width of canal		=12	m

h_c= 0.0832 m

Specific energy at the control section (Eo) is calculated as follows:

Eo=1.5h_c= 0.1248 m

The minimum length of a bar screen (Lo) required for water intake (Q_0) is calculated as follows:

$$Lo = \frac{Qo}{\mu \psi Bo \sqrt{2gEo}}$$

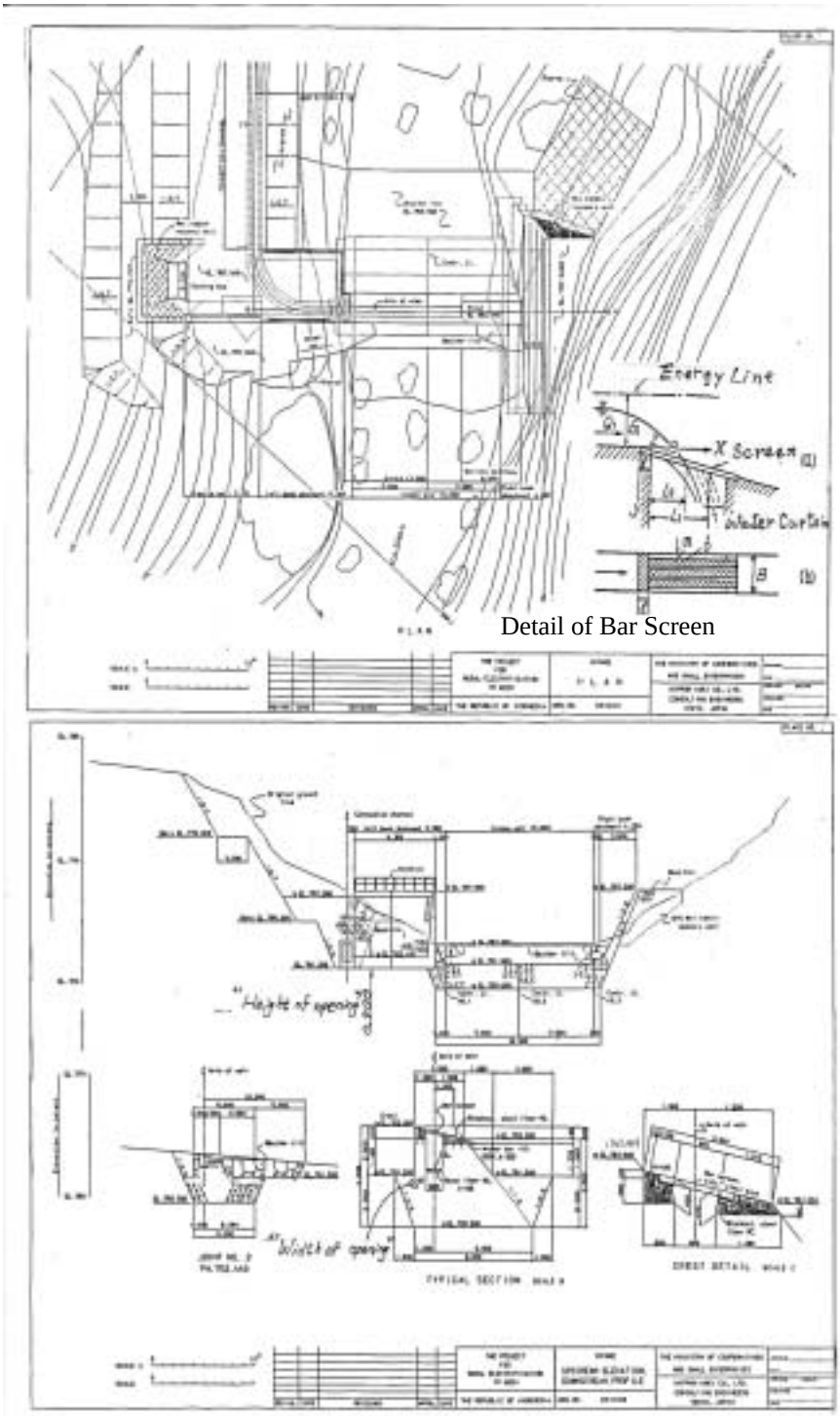
Where,	m:	Inflow coefficient		=0.6	
	y:	Opening degree of bar screen	=Sa/Bo	=0.2308	(=0.03/(0.03+0.1))
	a:	Gap width of bar (refer to the next page)		=0.03	m
	b:	Width of bar (refer to the next page)		=0.1	m
	Q_o:	Discharge		=0.86	m^3/s
	B_o:	Width of intake to install bar screen		=12	m

Lo= 0.3309 m

Taking into account partial blocking and easy maintenance, the installation width of bar screen is calculated as follows:

L= 0.8 m =2.42 Lo (Q 2.1m^3/s =2.42 x 0.86m^3/s)

Therefore, adoption of the installation width of bar scree, i.e. 0.8 m provides a guaranttee of planned water intake (0.86 m^3/s) against partial blocking even at 50 % of blocking.



Reference DWG. of Intake Facilities

2. Maximum Inflow at The Time of Flood

Similar to "1. Installation Width of Bar Screen", the design flood forms a control section on the screen of weir. For the design flood (Q = 360 m^3/s), accordingly, the critical depth (hc) of a rectangular section is calculated as follows:

$$hc = \left(\frac{\alpha Q1^2}{gB^2} \right)^{\frac{1}{3}}$$

Where,	a:	Adjustment coefficient	=1.1
	Q_1:	Design flood	=360 m^3/s
	g:	Gravity acceleration	=9.8 m/s^2
	B:	Width of canal	=18 m

h_c= 3.554 m

Specific energy at the control section (E_1) is calculated as follows:

E_1=1.5h_c= 5.331 m

In addition, the specific energy at the control section (E_1) is developed for the water depth at the most upstream end of bar screen (h_1) as follows:

$$E_1 = h_1 + \frac{V_1^2}{2g} = h_1 + \frac{Q_1^2}{2gh_1^2B^2}$$

Where,	h_1:	Water depth at the most upstream end of bar screen	(m)
	h_2:	Water depth at the most downstream end of bar screen	(m)
	V_1:	Velocity at the most upstream end of bar screen	(m/s)

Accordingly, the water depth at the most upstream end of bar screen (h_1) is calculated as follows:

h_1=	4.1407	-->	4.141 m
E_1=	5.331		

In addition, the length of bar screen (Lo) satisfies the following equation:

$$Lo = \frac{E_1}{\mu\psi}\left(\frac{h}{E_1}\sqrt{1-\frac{h_1}{E_1}} - \frac{h_2}{E_1}\sqrt{1-\frac{h_2}{E_1}} \right)$$

Where,	Lo=	0.8 m
	E_1=	5.331 m
	h_1=	4.141 m
	m=	0.6
	y=	0.230769

Accordingly, the water depth at the most downstream end of bar screen (h_2) is calculated as follows:

h_2= 2.560469 --> 2.560 m
L_o= 0.80076

The discharge at the most downstream end of bar screen, Q_2(m^3/s) is calculated as follows:

$$Q_2 = B_2 h\sqrt{2g(E_1 - h_1}$$

Q_2= 339.6 m^3/s

Based on the assumption that the section of collecting channel provided below bar screen, the discharge possible to flow down from gaps of bar screens is calculated as follows:

$Q=Q_1-Q_2$= 20.4 m^3/s

3. Section of Collecting Channel

By use of Manning's formula, the water depth of uniform flow at the downstream end of collecting channel is calculated as follows:

$$Q = \frac{A}{n} R^{2/3} S^{1/2} = \frac{bho}{n}\left(\frac{bho}{b + 2ho}\right)^{2/3} S^{1/2}$$

$$V = \frac{Q}{A}$$

Where,	Q:	Discharge (m^3/s)		
	A:	Area (m^2)		
	V:	Velocity (m/s)		
	n:	Roughness coefficient		=0.015
	R:	Hydraulic radius (m)		
	S:	Slope of collecting channel	=1/20	=0.05
	b:	Width of collecting channel (m)		=0.8 m

The water depth (ho) is calculated as follows:

For the discharge planned at the 1st stage,
ho= 0.155302 --> 0.155 m
Qo= 0.4300 m^3/s
V= 3.461 m^3/s

For the discharge planned at the 2nd stage,
ho= 0.25083 --> 0.251 m
Qo= 0.8600 m^3/s
V= 4.286 m^3/s

For the design discharge,
ho= 3.37835 --> 3.378 m
Q= 20.3000 m3/s
V= 7.511 m3/s

The depth of collecting channel at the downstream end, H (m) is calculated as follows by use of the following formula:

$$H = ho + K\frac{V^2}{2g}$$

Where, K: 0.6 (For S=1/20-1/30)

For the discharge planned at the 1st stage,
H= 0.522 m

For the discharge planned at the 2nd stage,
H= 0.813 --> 0.900 m

For the design discharge,
H= 5.105 m

Therefore, the height of opening is determined at 0.9m.

Namely, the crest elevation of overflow weir is determined at EL.763.5 and the sill level of collecting channel is determined at EL.761.94.

Due to the height of collecting channel at the downstream end, i.e. 1.56 m (= EL.763.5 - EL.761.94) less than 5.105 m, the flow at the time of flood provides a submerged discharge.

Inflow from the opening, Qm(m^3/s) is calculated as follows:

$$Qm = Ca\sqrt{2gh}$$

Where,	C:	Discharge coefficient		=0.55
	a:	Discharge area	=0.8x0.9	=0.72 m^2
	h:	Water depth	=(763.5+3.55)-761.94	=5.11 m

Qm= 3.963 m^3/s --> 4.0 m3/s

4 Determination of Dimension for De-Silting Basin

Min. size of sand grains	d=	0.3	mm
Critical velocity	u=	0.2	m/s
Discharge to be taken	Qo=	0.86	m^3/s

Assuming width of	B=	2.0	m
Calculatig water depth:	h=Qo/(B ･u)=	2.15	m (= 0.86/2.0/0.2)

Length of de-silting basin (L) is calculated as follows:
L=K.･h･u /Vg

Where,	K:	Safety factor	=1.5
	Vg:	Critical velocity for settling of min. size gra	=0.04 m/s

L= 16.125 --> 17 m (= 0.15x2.15x0.2/0.04)

Adopting the slope of de-siltig basin: S as

S= =1/50 =0.02

Due to the requirement for the length of transition part upstream of the de-silting basin enlarging its width, L_1 is to be longer than the width of de-silting basin: 2.0 m.
Accordingly,

L_1= 2.0 m

Froude numbetr at sand flushing: Fr is calculated as follows by use of Manning's formula

$$Q = \frac{A}{n} R^{2/3} S^{1/2} = \frac{bho}{n}\left(\frac{bho}{b+2ho}\right)^{2/3} S^{1/2}$$

$$V = \frac{Q}{A}, Fr = \frac{V}{\sqrt{gho}}$$

Where,	Q:	Discharge	=0.86 m^3/s
	A:	Area (m^2)	
	V:	Velocity (m/s)	
	n:	Roughness coefficient	=0.015
	R:	Hydraulic (m)	
	S:	Slope on bottom of de-silting basin	=0.02
	b:	Width of de-silting basin	=2 m

F_r is calculated as follows by a trial calculation:

ho=	0.166812	-->	0.167 m
Q=	0.86	m3/s	
V=	2.578	m/s	
F_r=	2.02		

Against this F_r, the required width of sand flushing channel and the required length of transition part to the sand flushing channel are as fokkows:

b'/b=	0.8	b'=	1.6 m	
L2/b=	0.5	L2=	1.0 m	
-->Taking an allowance: 2 times		L2=	2.0 m	(= 1.0 x 2)

Adopting the slope of sand flushing channel: S as

S'= =1/10 =0.1

The critical depth of de-silting channel: h_c is calculated as follows:

$$hc = \left(\frac{\alpha Q^2}{gb'^2}\right)^{\frac{1}{3}}$$

Where, a= =1.1
Q= =0.86 $m^{3/}s$
g= =9.8 m/s^2
b'= =1.6 m

h_c= 0.319 m

Adopting the critical depth with some allowance for the height of de-silting channel: H' (m)
H'= 0.7 m

In the case of the de-silting basin fully filled with water and the gates fully opened, the discharge of sand flushing: Q_m is calculated as follows:

$$Qm = Ca\sqrt{2gh}$$

Where,	C:	Discharge coefficient			=0.55
	a:	Area of flushing		=b'・H'	=1.12 m2
	h:	Hydraulic radius	2.59	+0.25	=2.84 m

Q_m= 4.596 m3/s --> 4.6 m3/s

The depth of uniform flow: ho (m) is calculated as follows by use of Manning's formula

$$Q_0 = \frac{A}{n}R^{2/3}S^{1/2} = \frac{bh_0}{n}\left(\frac{bh_0}{b+2h_0}\right)S^{1/2}$$

Where, Qo= 4.6 m^3/s
n= 0.015
S= 0.100
b= 1.6 m

By a trial calculation, h_0 is calculated as follows:

ho= 0.34983 --> 0.350 m < 0.7 m
Qo= 4.600038 m3/s
V= 8.218 m/s

5 Capacity of Spillway

Maximum inflow	Qm=	4.0 m^3/s
Discharge to be taken	Qo=	0.43 m^3/s
		0.86 m^3/s
Design discharge of spill	Qs=	3.57 m^3/s
		3.14 m^3/s
Width of overflow weir	B=	12.0 m

The overflow discharge of weir is calculated as follows by use of the Ishihara & Ida's formula:

$$Q = CBh^{3/2}$$

$$C = 1.785 + \left(\frac{0.00295}{h} + 0.237\frac{h}{W}\right)(1+\varepsilon)$$

Where,	Q:	Overflow discharge			
	B:	Width of overflow weir	(m)		=12 m
	h:	Depth of overflow weir	(m)		
	W:	Height from canal bottom to weir top	=2.13	(m)	
	e:	0.55(W-1)			

By a trial calculation, h is calculated as follows:

For the discharge planned for the 1st stage:

h=	0.295242	-->	0.295	m
C=	1.854469			
Q=	3.57000	m3/s	3.57	

For the discharge planned for the 2nd stage:

h=	0.271313	-->	0.271	m
C=	1.851581			
Q=	3.14000	m^3/s	3.14	

6 Determination on Section of Waterway

The water depth of uniform flow (h_0) at the beginning point of waterway is calculated as follows by use of Manning's formula:

$$Q_0=\frac{A}{n}R^{2/3}S^{1/2}=\frac{bh_0}{n}\left(\frac{bh_0}{b+2h_0}\right)S^{1/2}$$

Where,	Qo=	0.43	m^3/s
		0.86	m^3/s
	n=	0.015	
	S=	=1/1000	=0.001
	b=	1.0	m

By a trial calculation, ho is calculated as follows:

For the discharge planned for the 1st stage:

ho=	0.51045	-->	0.510	m
Qo=	0.43000	m^3/s		
V=	0.842	m/s		

For the discharge planned for the 2nd stage:

ho=	0.87521	-->	0.875	m
Qo=	0.86000	m^3/s		
V=	0.983	m/s		

The free board: F_b (m) is calculated as follows:

$F_b=0.05ho+V^2/2g+0.15$ =0.24302 --> 0.24 m

The required heght: D (m) is calculated as follows:

$D=ho+F_b$ =0.88 +0.24 =1.12 m

Accordingly, the section of waterway is determined as follows:

Width: 1.0 m Height: 1.2 m

The required height above the overflow weir of side-spillway is calculated as follows

D=ho+h	=0.88	+0.27	=1.15 m	<	1.2 m

7 Transition Channel between Collecting Channel and De-Silting Basin

The water depth of uniform flow: ho (m) in the transition channel is calculated as follows by use of Manning's formula:

$$Q = \frac{A}{n} R^{2/3} S^{1/2} = \frac{bho}{n}\left(\frac{bho}{b+2ho}\right)^{2/3} S^{1/2}$$

$$V = \frac{Q}{A}$$

Where,	Q:	Discharge (m^3/s)	For the 1st stage		=0.43
			For the 2nd stage		=0.86
			For the flood conditic		=4
	A:	Area (m^2)			
	V:	Velocity (m/s)			
	n:	Roughness coefficient			=0.015
	R:	Hydrauli r (m)			
	S:	Slope of Collecting Channel		1/50	=0.02
	b:	Width of Collecting Channel			=0.8 m

By a trial calculation, ho is calculated as follows:

For the discharge planned for the 1st stage:

ho=	0.212632	-->	0.213 m
Qo=	0.4300 m^3/s		
V=	2.528 m^3/s		

For the discharge planned for the 2nd stage:

ho=	0.349331	-->	0.349 m
Qo=	0.8600 m^3/s		
V=	3.077 m^3/s		

For the flood conditions:

ho=	1.18578	-->	1.186 m
Q=	4.0000 m^3/s		
V=	4.217 m^3/s		

In the case when the excess inflow is considered in addition to the discharge planned for the 2nd stage:

ho=	0.69453	-->	0.695 m
Q=	2.1000 m^3/s		
V=	3.780 m^3/s		

The following formula which could be obtained by combination of Iwagaki's formula and Manning's formula is given to calculate the minimum velocity in the collecting channel fo transportation of inflow sediment particles: Vc (m/s).

$$Vc = \frac{0.029}{n} d^{1/2} \left(\frac{bh}{b+2h} \right)^{1/6}$$

Where, d: diameter of grains (cm) = 3

In the case of Q = 0.43m^3/s,

n=	0.015					
d=	=3	cm				
b=	0.8	m				
h=	0.213	m				
Vc=	2.41	m/s	<	V=	2.53	m/s

In the case of Q = 2.1m$^{3/}$s

n=	0.015					
d=	=3	cm				
b=	0.8	m				
h=	0.695	m				
Vc=	2.66	m/s	<	V=	3.78	m/s

Project: **Namlan Small Hydro (Samle)** **Anchor Block No.** **1**

Item		unit	Input
Discharge	Q	m^3/s	0.65
Head	H	m	4.00
Diameter of Pentstock (u/s)	ϕ_1	m	0.80
Diameter of Penstock (d/s)	ϕ_2	m	0.80
Thickness of Penstock (u/s)	t_1	mm	6.00
Thickness of Penstock (d/s)	t_2	mm	6.00
Distance to Expansion Joint (u/s)	L_1	m	15.00
Distance to Expansion Joint (d/s)	L_2	m	0.00
Distance to Saddle (u/s)	l_1	m	6.00
Distance to Saddle (d/s)	l_2	m	6.00
Penstock Slope (u/s)	α_1	°	4.57
Penstock Slope (d/s)	α_2	°	28.34
Center angle of Bend	ψ	°	23.77
Width of Anchor Block	w	m	2.00
Fillet of Anchor Block	b	m	0.30
Seismic Coefficient	k_h	-	0.00
Unit Weight of Concrete	γ_c	t/m^3	2.30
Friction coeff. (saddle)	f	-	0.50
Stability			
Safety Factor for Sliding	F_s	-	0.50
Bearing Stress of Foundation	σ	t/m^2	100.00

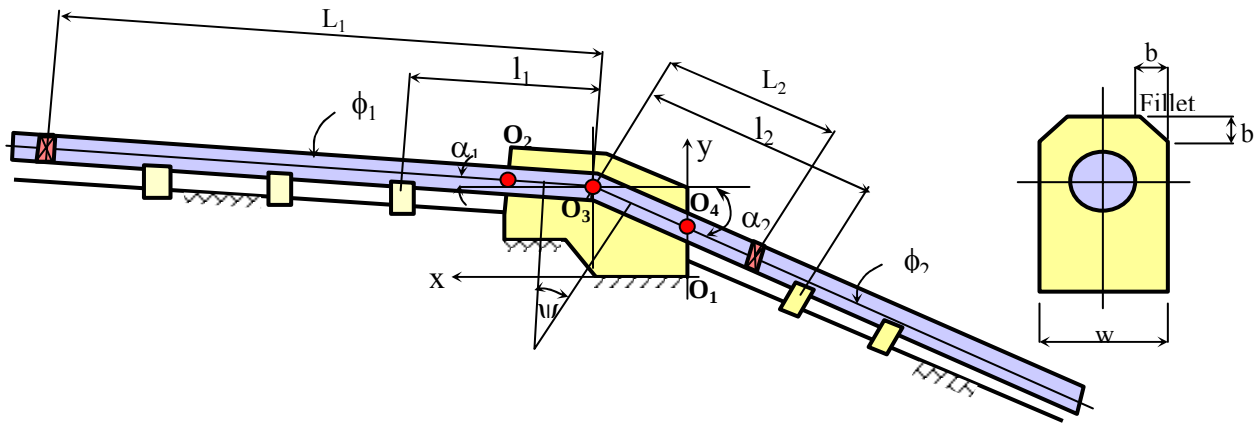
1. Anchor Block		
No.	x(m)	y(m)
1	0.000	0.000
2	2.000	0.000
3	2.500	1.000
4	4.000	1.000
5	4.000	2.157
6	3.848	4.051
7	1.739	3.882
8	0.000	2.994
9	0.000	0.000
10		
11		
12		
13		
14		
15		

Max	4.000
Min	0.000
L	4.000

2. Fillet		
No.	x(m)	y(m)
1	3.848	4.051
2	1.739	3.882
3	0.000	2.994
4	0.000	2.603
5	1.826	3.588
6	3.872	3.752
7	3.848	4.051
8		
9		
10		
11		
12		
13		
14		
15		

Pipe Location

O_1	0.000	0.000
O_2	3.920	3.154
O_3	2.000	3.000
O_4	0.000	1.921



Force	Magnitude (ton)	V (ton)	H (ton)	x (m)	y (m)	Mr (t·m)	Mo (t·m)
(1) Penstock Pipe (u/s)	1.857	1.851	-0.148	2.000	3.000	4.146	
(2) Penstock Pipe (d/s)	1.640	1.443	-0.778	2.000	3.000	5.222	
(3) Axial Force of Penstock Pipe (u/s)	3.856	0.307	3.844	2.000	3.000	0.614	11.531
(4) Axial Force of Penstock Pipe (d/s)	0.000	0.000	0.000	2.000	3.000	0.000	0.000
(5) Friction (u/s)	0.016	0.001	0.016	2.000	3.000	0.003	0.048
(6) Friction (d/s)	0.000	0.000	0.000	2.000	3.000	0.000	0.000
(7) Centrifugal Force	0.035	-0.035	0.007	2.000	3.000		0.091
(8) Water Pressure at Bend	0.828	-0.810	0.171	2.000	3.000		2.132
(9) Reducer Bend	0.000	0.000	0.000	2.000	3.000	0.000	0.000
(10) Anchor Block	54.368	54.368	0.000	1.937	0.000	105.290	
(11) Seismic	0.000	0.000	0.000	0.000	1.959		0.000
Total		57.126	3.111			115.276	13.802
							101.473

Stability

	unit	unit	Result	Allowable Range	Judge
①	Safety factor for over turning (e<L/	m	0.224	< 0.667	OK
②	Safety factor for sliding (|H|/V<Fs)		0.054	< 0.500	OK
③	Bearing stress σ_1=ΣV/(wL)*(1+6e/	t/m2	9.537	< 100.000	OK
④	Bearing stress σ_2=ΣV/(wL)*(1-6e/l	t/m2	4.745	< 100.000	OK

Program (S2) STRESS CALCULATION FOR REINFORCED CONCRETE (Input)

: input

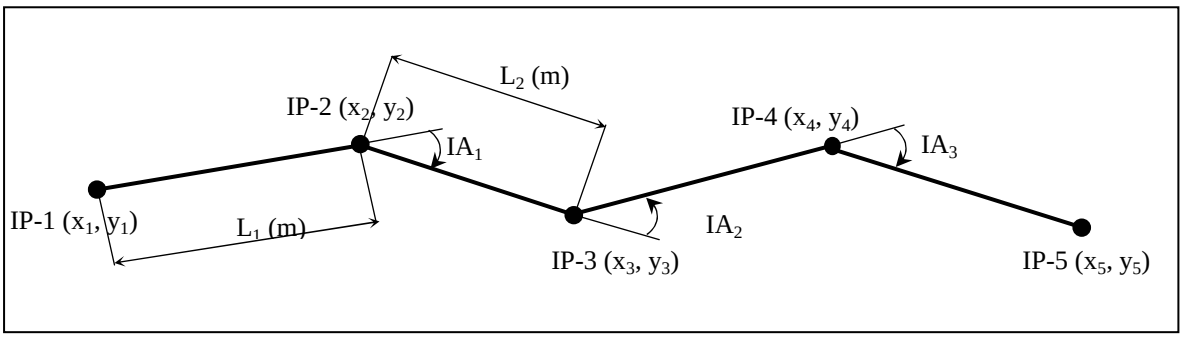
		unit	1	2	3	4	5	6
Moment	M	(t.m)	66.26	111.78	93	106.21	112.24	
Shear	S	(t)	13.51	24.81	27.72	34.2	39.31	
Axial Force	N	(t)	0	0	0	0	0	
Width	B	(cm)	100	100	100	100	100	
Height	H	(cm)	120	120	120	120	120	
Effective Height	d	(cm)	110	110	110	110	110	
Cover for comp.	d'	(cm)			10			
Re-bar(tension)	As	(cm^2)	67.6	67.6	67.6	67.6	67.6	
Re-bar arrange			2-D25@150	2-D25@150	2-D25@150	2-D25@150	2-D25@150	
Re-bar(comp.)	As'	(cm^2)						
Re-bar arrange								
Allowable Stress	sc	(kg/cm^2)	78.0	78.0	78.0	78.0	78.0	
	ss	(kg/cm^2)	2,340.0	2,340.0	2,340.0	2,340.0	2,340.0	
	t	(kg/cm^2)	10.4	10.4	10.4	10.4	10.4	

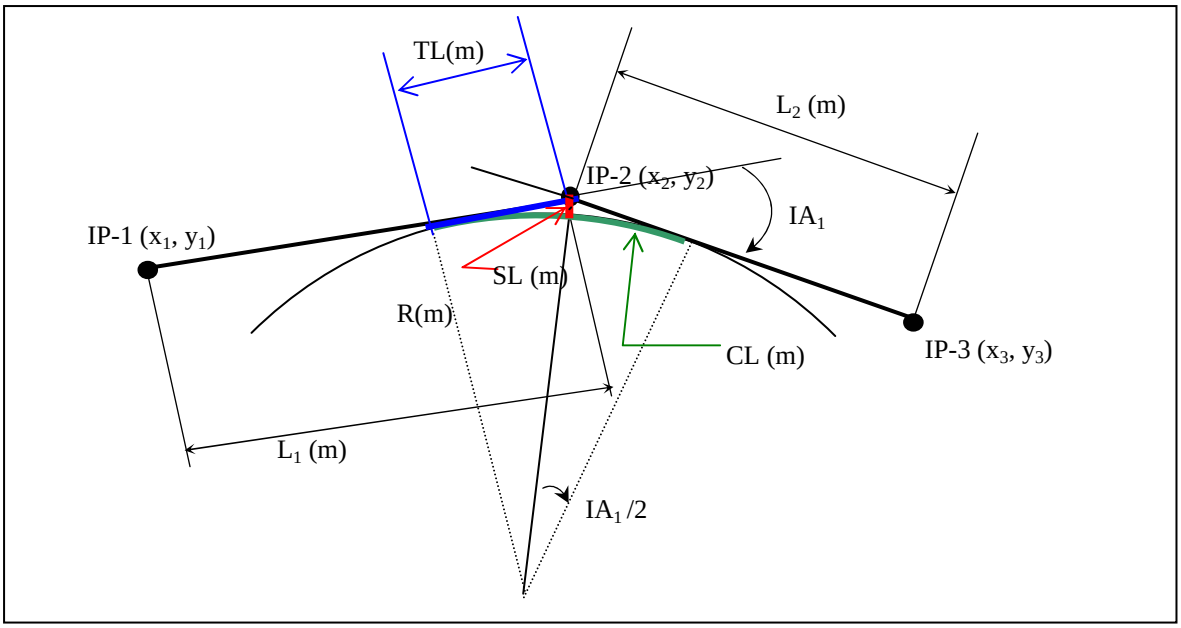
Re-bar	mm	D13	D16	D19	D22	D25	D29	D32
Area	cm^2	1.267	1.986	2.865	3.871	5.067	6.424	7.942
Unit weight	kg/m	0.995	1.56	2.25	3.04	3.98	5.04	6.23
@100	cm2	12.70	19.90	28.70	38.70	50.70	64.20	79.40
@150	cm2	8.45	13.30	19.10	25.80	33.80	42.90	53.00
@200	cm2	6.34	9.93	14.30	19.40	25.30	32.10	39.70
@250	cm2	5.07	7.94	11.50	15.50	20.30	25.70	31.80
@300	cm2	4.22	6.61	9.54	12.90	16.90	21.40	26.50
5 nos.	cm2	6.34	9.93	14.30	19.40	25.30	32.10	39.70
6 nos.	cm2	7.60	11.90	17.20	23.20	30.40	38.50	47.70
7 nos.	cm2	8.87	13.90	20.10	27.10	35.50	45.00	55.60
8 nos.	cm2	10.10	15.90	22.90	31.00	40.50	51.40	63.50
9 nos.	cm2	11.40	17.90	25.80	34.80	45.60	57.80	71.50
10 nos.	cm2	12.70	19.90	28.70	38.70	50.70	64.20	79.40
11 nos.	cm2	13.90	21.90	31.50	42.60	55.70	70.70	87.40
12 nos.	cm2	15.20	23.80	34.40	46.50	60.80	77.10	95.30
13 nos.	cm2	16.50	25.80	37.30	50.30	65.90	83.50	103.00
14 nos.	cm2	17.70	27.80	40.10	54.20	70.90	89.90	111.00

Program (S2) STRESS CALCULATION FOR REINFORCED CONCRETE (Output)

Structure Member			TI#9 Left Abutment Wing Wall (upstream side)					
			Section-I	Section-II	Section-III	Section-IV	Section-V	
Moment	M	(t.m)	66.26	111.78	93.00	106.21	112.24	
Shear	S	(t)	13.51	24.81	27.72	34.20	39.31	
Axial Force	N	(t)	0.00	0.00	0.00	0.00	0.00	
Width	B	(cm)	100.00	100.00	100.00	100.00	100.00	
Height	H	(cm)	120.00	120.00	120.00	120.00	120.00	
Effective Height	d	(cm)	110.00	110.00	110.00	110.00	110.00	
Cover for comp.	d'	(cm)			10.00			
Re-bar (tension)	As	(cm2)	67.60	67.60	67.60	67.60	67.60	
Re-bar arrange			2-D25@150	2-D25@150	2-D25@150	2-D25@150	2-D25@150	
Re-bar (comp.)	As'	(cm2)						
Re-bar arrange								
Compression	σc	(kg/cm2)	35.69	60.21	50.10	57.21	60.46	
		(kg/cm2)						
Tension	σs	(kg/cm2)	1,007.61	1,699.83	1,414.24	1,615.13	1,706.82	
		(kg/cm2)						
Shear	τ	(kg/cm2)	1.39	2.55	2.85	3.52	4.04	
Allowable stress	σca	(kg/cm2)	78.00	78.00	78.00	78.00	78.00	
	σsa	(kg/cm2)	2,340.00	2,340.00	2,340.00	2,340.00	2,340.00	
	τa	(kg/cm2)	10.40	10.40	10.40	10.40	10.40	
Note								

Program (12) COORDINATES CALCULATION (Reference Equations)





$$L_n = \sqrt{(x_{n+1} - x_n)^2 + (y_{n+1} - y_n)^2}$$

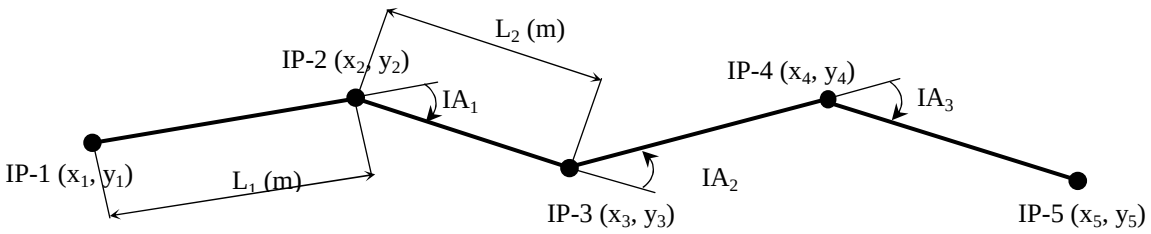
$$IA_n = \frac{\arccos\left((x_n - x_{n-1}) + (x_{n+1} - x_n)\right) + \left((y_n - y_{n-1}) + (y_{n+1} - y_n)\right)}{\sqrt{(x_n - x_{n-1})^2 + (y_n - y_{n-1})^2}\sqrt{(x_{n+1} - x_n)^2 + (y_{n+1} - y_n)^2}}$$

$$TL_n = R \times \tan\frac{IA_n}{2}$$

$$CL_n = R \times 2\pi\frac{IA_n}{360}$$

$$SL_n = \sqrt{TL_n^{\ 2} + R^2} - R$$

Program (12) COORDINATES CALCULATION (Input and Output)



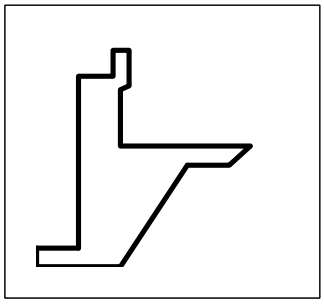
IP	x (m)	y (m)	R (m)	L (m)	IA (°)	CL (m)	TL (m)	SL (m)
1	299,047.500	440,744.500		161.577				
2	299,196.500	440,807.000	120.000	224.371	26.8455063	56.225	28.638	3.370
3	299,420.300	440,791.000	120.000	198.522	7.8885880	16.522	8.274	0.285
4	299,614.500	440,749.800	50.000	50.331	64.8142280	56.561	31.740	9.223
5	299,626.000	440,700.800	50.000	180.136	10.9808815	9.583	4.806	0.230
6	299,633.000	440,520.800	50.000	165.465	76.7278682	66.958	39.578	13.769
7	299,470.600	440,552.500	100.000	109.946	33.7852094	58.966	30.368	4.509
8	299,369.200	440,510.000	100.000	56.304	0.5920846	1.033	0.517	0.001
9	299,317.500	440,487.700	120.000	222.670	34.7026973	72.681	37.494	5.721
10	299,099.200	440,531.600	70.000	95.776	65.8411554	80.440	45.321	13.390
11	299,078.000	440,625.000	100.000	45.689	3.7427245	6.532	3.267	0.053
12	299,065.000	440,668.800	100.000	77.696	3.5143551	6.134	3.068	0.047
13	299,047.500	440,744.500	50.000	161.577	80.2604752	70.040	42.149	15.395
14	299,196.500	440,807.000						
15								
16								
17								
18								
19								
20								

For calculation : CTRL + f

Program (13) Area and Center of Gravity for Polygons

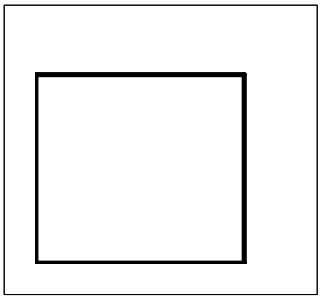
Input Coordinates → Ctrl + a → Output

No.	x (m)	y (m)
1	0.000	0.000
2	2.000	0.000
3	3.590	5.300
4	4.590	5.300
5	5.090	6.300
6	2.000	6.300
7	2.000	9.250
8	2.200	9.450
9	2.200	11.300
10	1.820	11.300
11	1.820	9.950
12	1.000	9.950
13	1.000	0.970
14	0.000	0.970
15	0.000	0.000
16		
17		
18		
19		
20		



Area (m^2)	18.607
x (m)	1.993
y (m)	4.729

No.	x (m)	y (m)
1	0.000	0.000
2	5.000	0.000
3	5.000	5.000
4	0.000	5.000
5	0.000	0.000
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		



Area (m^2)	25.000
x (m)	2.500
y (m)	2.500

No.	x (m)	y (m)
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Area (m^2)	9.090
x (m)	2.159
y (m)	3.071