APPENDIX VIII

MAIN FACILITIES PLANNING

APPENDIX VIII

MAIN FACILITIES PLANNING

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1. Pateswari Intake Work

1-1. Intake Water Design Level and River Discharge

The relationship between annual minimum water levels and annual maximum and minimum discharges were examined based on past available recoreds (Tables Ⅷ-1-1 and Ⅷ-1-2, Figures Ⅷ-1-1 and Ⅷ-1-2).
As a result, it is recognized that the fluctuations of the river bed elevation was caused by the following:

a) the bed is raised by Large flooding,
b) the bed is lowered by a serios of medium scale floods,
c) and the bed is raised by a serios of small floods.

It is also recognized that the bed is at a raised level at present.

Considering the above three points, the water design level was determined. The poissible minimum water level in the durable period of the structure is to be 25.5m which approximately corresponds to the lowest recorded water level (see Fig. Ⅷ-1-1).

As for available minimum river discharge, it is proposed to adopt 73m³/s, which is the probable minimum annual 355th river discharge for a 5 year recurrence. The water design diversion requirement of 42.78m³/s was obtained from the requirements of the third 10 day-period in April of 1979, when the minimum river discharge was observed as 80.1 m³/s on March 31. The second maximum water diversion requirement from 1979 to 1988 was given as 40.9m³/s in the second 10day-period in May of 1987, when the minimum river discharge was observed as 65.4m³/s on May 31. Accordingly, the ratios of water intake to river discharge in the above cases are given as follows:

Table Ⅷ-1-1 Pateswari Minimum Water Levels and Minimum Discharges

Year	Minimum Water levels	Minimum Discharges	Occurrence Date Day, Month
1964-65	25.92 m	N A m3/s	25.3
1965-66	25.59	N A	20.3
1966-67	25.47	N A	11.3
1967-68	25.50	N A	15.4
1968-69	25.71	88.0	6.4
1969-70	25.85	72.2	25.3
1970-71	25.83	78.7	13.4
1971-72	N A	N A	—
1972-73	25.88	62.3	25.3
1973-74	25.94	79.0	1.4
1974-75	26.38	84.9	15.4
1975-76	26.40	71.3	29.3
1976-77	25.88	62.5	30.3
1977-78	26.05	90.6	1.4
1978-79	26.17	80.1	31.3
1979-80	26.08	76.9	18.3
1980-81	26.12	56.0	5.4
1981-82	26.17	60.2	23.4
1982-83	26.00	90.6	12.2
1983-84	26.03	86.3	23.4
1984-85	26.08	N A	10.4
1985-86	26.43	N A	30.3
1986-87	26.40	69.9	9.4
1987-88	26.24	65.4	31.3
1988-89	26.22	59.7	6.4

Source: BWDB

Table Ⅷ-1-2 Pateswari Maximum Water Levels and Maximum Discharges

Year	Maximum Water levels	Maximum Discharges	Occurrence Date Day, Month
1964-65	29.93 m	N A m3/s	——
1965-66	30.26	N A	——
1966-67	29.42	N A	——
1967-68	29.64	N A	——
1968-69	30.08	N A	——
1969-70	29.17	2,360	18.7
1970-71	29.48	4,670	28.9
1971-72	N A	N A	——
1972-73	25.10	4,240	1.8
1973-74	29.09	993	9.8
1974-75	30.24	1,640	5.8
1975-76	29.52	1,160	26.7
1976-77	29.38	7,190	3.7
1977-78	30.15	809	18.8
1978-79	29.69	1,020	18.7
1979-80	29.95	1,120	9.9
1980-81	29.93	659	17.8
1981-82	29.64	283	3.6
1982-83	30.29	979	11.7
1983-84	30.55	867	14.9
1984-85	30.86	N A	18.9
1985-86	29.93	N A	14.7
1986-87	29.72	892	3.8
1987-88	30.31	5,670	12.8
1988-89	30.68	4,200	29.8

Source: BWDB

Fig. Ⅷ-1-1 Fluctuation of Past Annual Minimum Water Levels and Discharges

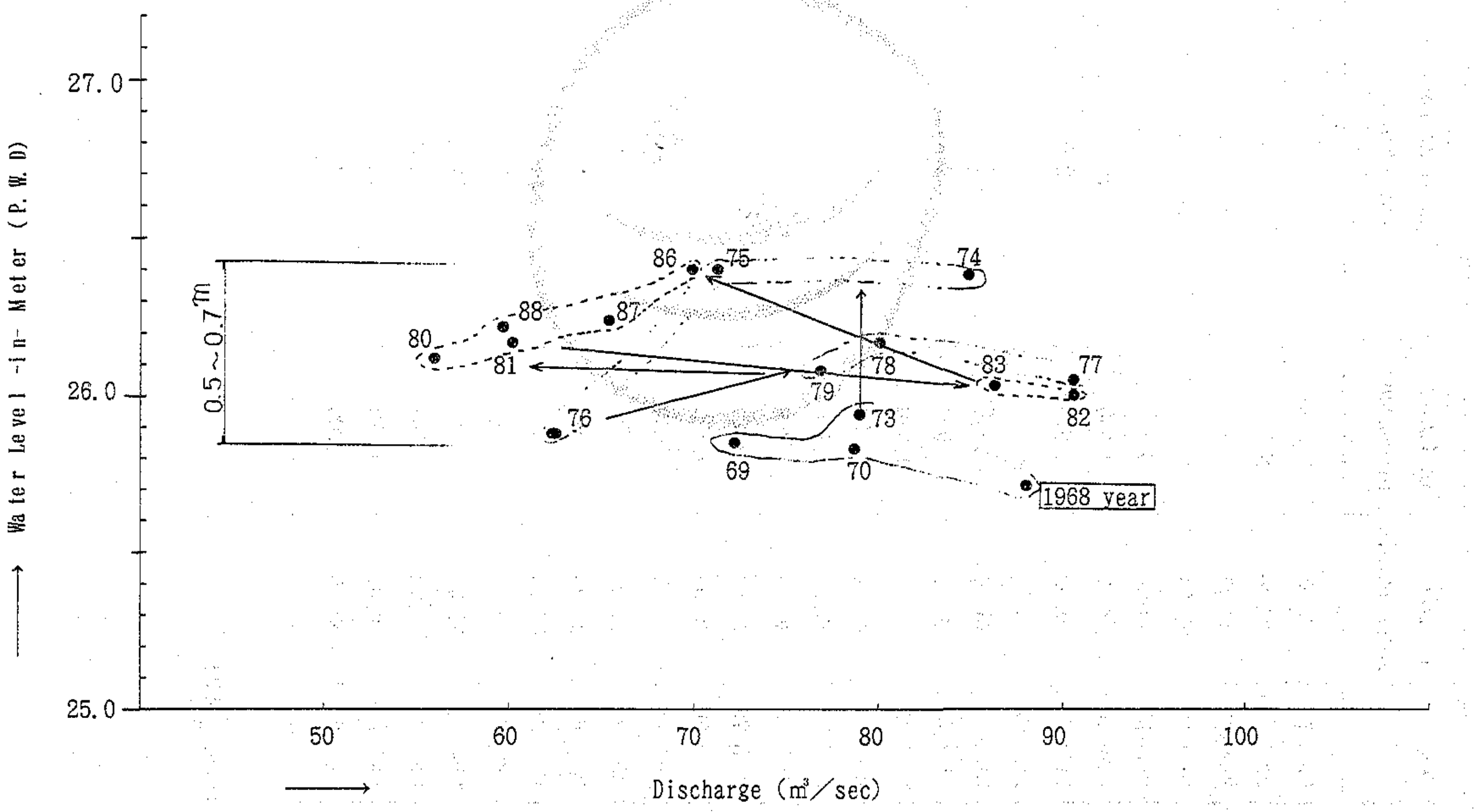


Fig. Ⅷ-1-2 Fluctuation of Past Annual Maximum Water Levels and Discharges

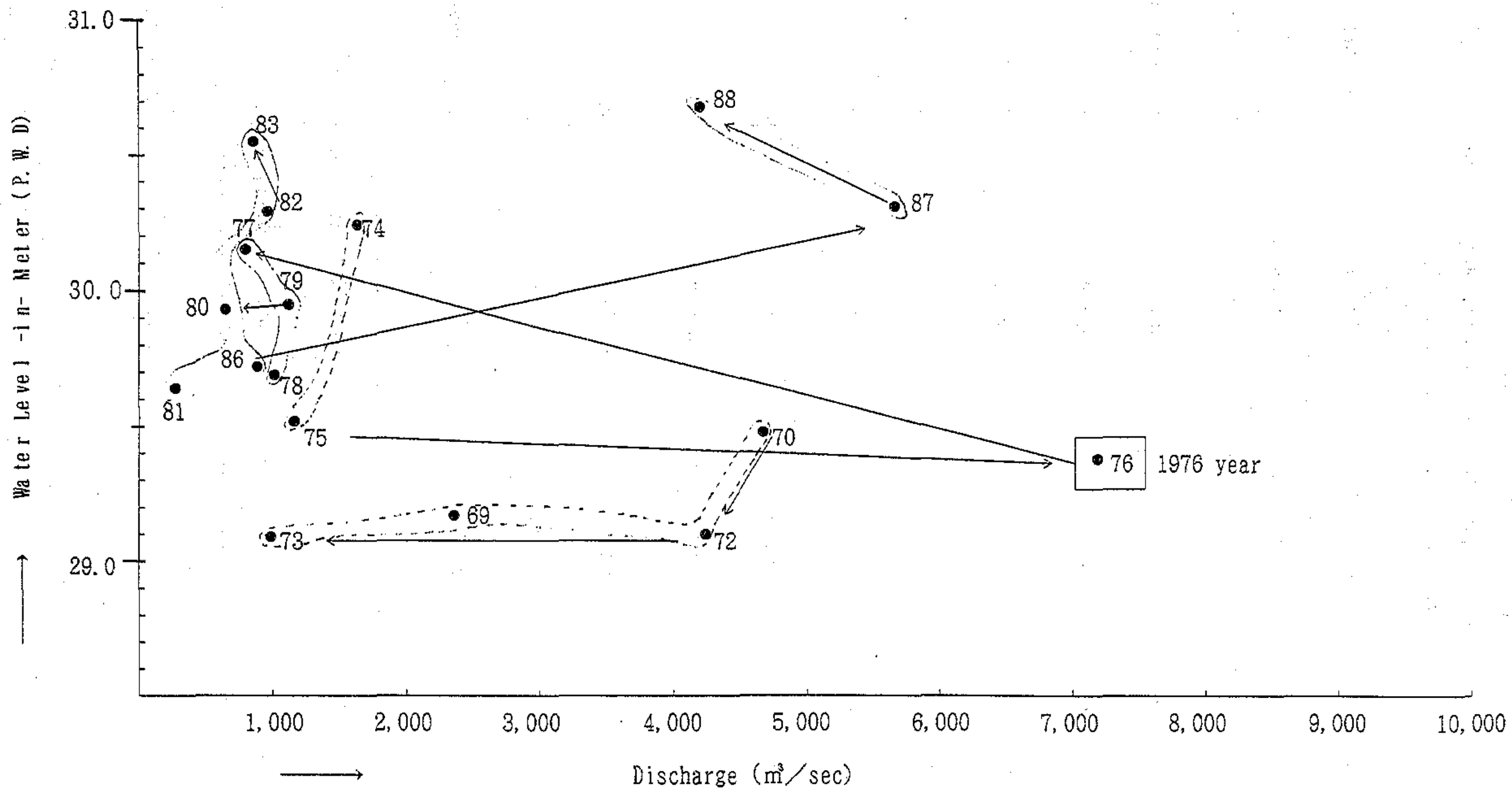


Table Ⅷ-1-3 Pateswari Pump Station
Gross Annual Irrigation Water Requirement
(Total Irrigable Area; A=29, 450 ha)

Year		Gross Irri. Req. (peak) (m^3/s)	Gross Irri. Req. (Million m^3)	Remarks	
1979	Apr-3	42.78	587.0	1	Plan year
1980	Apr-3	38.72	527.0	3	
1981	Jun-1	33.84	549.3	2	
1982	May-1	34.64	519.9	4	
1983	May-1	33.44	480.7	8	
1984	Feb-1	30.09	449.1	10	
1985	May-1	36.28	483.6	7	
1986	May-3	34.83	496.3	6	
1987	May-2	40.90	464.1	9	
1988	Apr-3	31.69	503.0	5	Average year
Total			5,060.0	Million m^3	
Average			506.0	Million m^3	

Table Ⅷ-1-4 (a) Pateswari Water Intake Ratio, Dudhkumar River

Plan Year; 1979

MARCH	1	Design Discharge (m³/s) A	25.81				
	〈1 ~ 10〉	Probable River Discharge (m³/s) B	(1/2) 90.9	(1/5) 75.4	(1/10) 68.2	(1/20) 62.7	(1/30) 59.9
		Water Intake Ratio A/B	0.28	★ 0.34	0.38	0.41	0.43
	2	Design Discharge (m³/s) A	25.93				
	〈11 ~ 20〉	Probable River Discharge (m³/s) B	(1/2) 88.3	(1/5) 75.0	(1/10) 68.4	(1/20) 63.1	(1/30) 60.3
		Water Intake Ratio A/B	0.29	★ 0.35	0.38	0.41	0.43
	3	Design Discharge (m³/s) A	25.37				
	〈21 ~ 31〉	Probable River Discharge (m³/s) B	(1/2) 86.2	(1/5) 72.8	(1/10) 67.4	(1/20) 63.6	(1/30) 61.9
		Water Intake Ratio A/B	0.29	★ 0.35	0.38	0.40	0.41

Table Ⅷ-1-4 (b) Pateswari Water Intake Ratio, Dudhkumar River

APRIL	1 〈1 ∫ 10〉	Design Discharge (m³/s) A	27.40				
		Probable River Discharge (m³/s) B	(1/2) 92.3	(1/5) 76.5	(1/10) 68.8	(1/20) 62.6	(1/30) 59.5
		Water Intake Ratio A/B	0.30	★ 0.36	0.40	0.44	0.46
	2 〈11 ∫ 20〉	Design Discharge (m³/s) A	37.29				
		Probable River Discharge (m³/s) B	(1/2) 101.6	(1/5) 82.6	(1/10) 75.4	(1/20) 70.4	(1/30) 68.2
		Water Intake Ratio A/B	0.37	★ 0.45	0.49	0.53	0.55
	3 〈21 ∫ 31〉	Design Discharge (m³/s) A	42.78				
		Probable River Discharge (m³/s) B	(1/2) 128.0	(1/5) 93.1	(1/10) 80.5	(1/20) 72.3	(1/30) 68.6
		Water Intake Ratio A/B	0.33	★ 0.46	0.53	0.59	0.62

TableⅧ-1-5 Pateswari Past Flow-Duration Table, Dudhkumar River

(Discharge: Ten Daily Averages)

Unit ; m³/sec

Year \ Month	JAN			FEB			MAR			APR			MAY			JUN		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
1969	170	160	144	130	115	103	96	96	127	109	135	149	126	232	449	298	599	781
1970	154	141	139	119	100	110	100	94	83	89	109	• 289	• 421	352	264	• 1330	873	824
1971	191	175	166	• 155	• 150	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
1972	NA	NA	NA	NA	NA	NA	NA	NA	NA	119	165	184	249	• 687	• 851	1290	1340	• 2040
1973	147	132	119	109	91	81	87	73	68	86	87	94	102	155	135	198	567	755
1974	102	100	115	130	123	117	114	108	110	119	141	235	392	478	325	509	830	800
1975	161	152	147	152	143	• 136	• 135	• 128	• 131	102	116	137	157	194	217	303	446	739
1976	108	101	94	97	96	85	83	80	73	• 66	• 70	121	239	218	211	650	• 2300	1820
1977	132	130	121	112	106	98	93	90	89	• 133	• 178	219	322	264	322	467	478	426
1978	156	137	131	125	124	119	106	96	92	84	99	146	167	194	324	235	403	891
1979	• 183	• 176	144	111	95	93	93	90	85	88	90	93	129	187	166	132	198	195
1980	135	123	112	102	94	91	93	82	89	93	112	147	165	190	217	249	453	476
1981	134	129	109	89	77	• 67	• 59	72	• 63	81	84	• 65	• 69	• 79	• 89	• 120	• 126	• 119
1982	• 75	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	184	144	150	210	215	NA
1983	141	144	129	119	115	113	107	95	108	NA	100	112	269	351	338	348	NA	NA
1984	168	165	• 182	• 155	135	108	107	103	93	NA	NA	NA	NA	NA	NA	NA	NA	NA
1985	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
1986	NA	NA	NA	NA	NA	NA	NA	NA	NA	75	83	113	177	166	149	188	272	NA
1987	111	98	89	82	78	75	74	85	85	92	86	92	131	160	165	190	222	402
1988	99	94	87	81	• 75	77	70	68	70	• 63	75	89	80	177	219	200	413	440
1989	77	• 71	• 66	• 70	77	76	64	• 62	70	—	—	—	—	—	—	—	—	—
Max Q	183	176	182	155	150	136	135	128	131	133	178	289	421	687	851	1330	2300	2040
Mean Q	136	131	123	114	106	97	93	89	90	93	108	143	199	249	270	407	608	769
Min Q	75	71	66	70	75	67	59	62	63	63	70	65	69	79	89	120	126	119
Probable Q (1/5)	—			—			75.4	75.0	72.8	76.5	82.6	93.1	—			—		

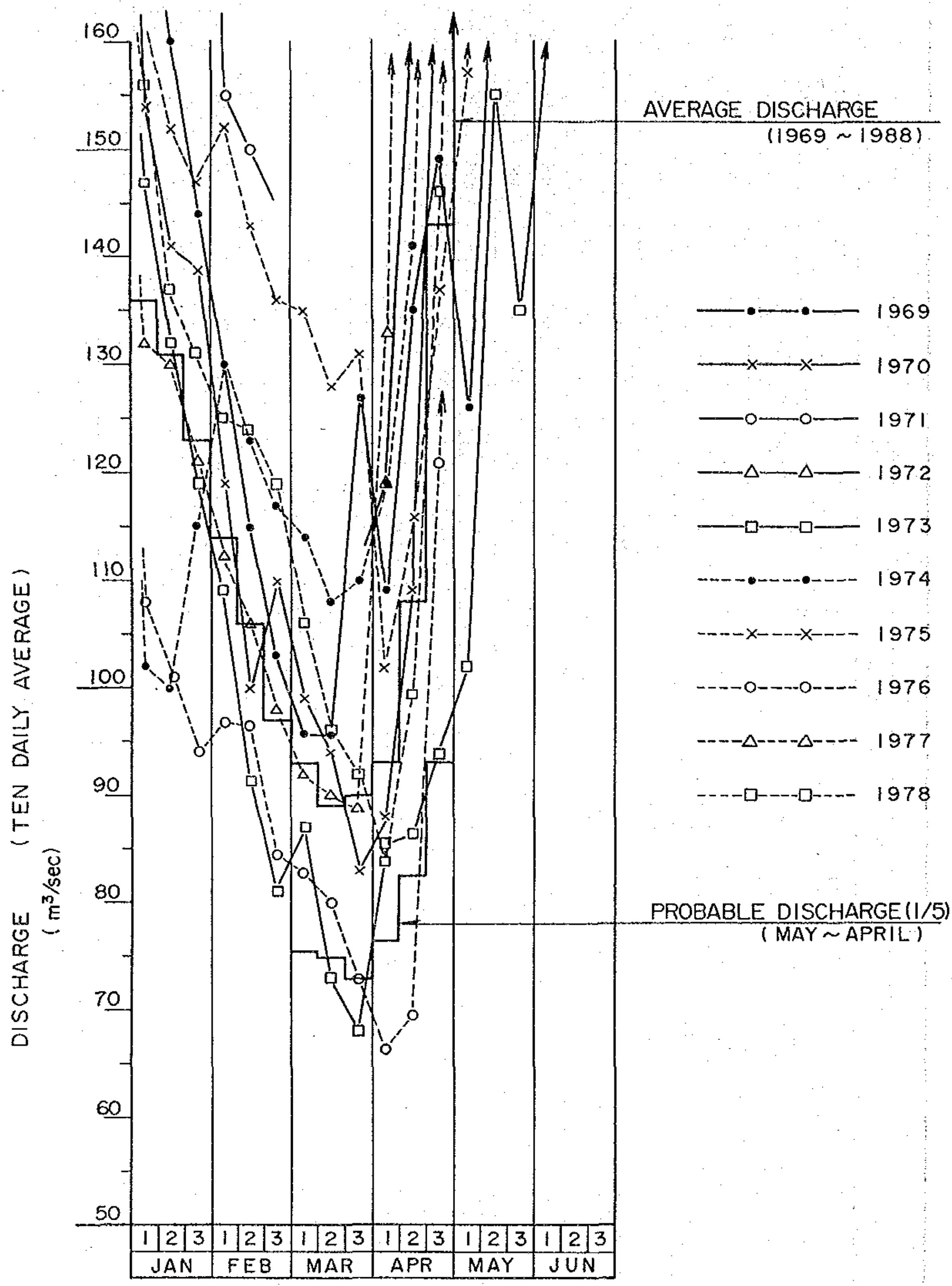


Fig. Ⅷ-1-3(a) Pateswari Past Flow-duration Curve, Dudhkumar River

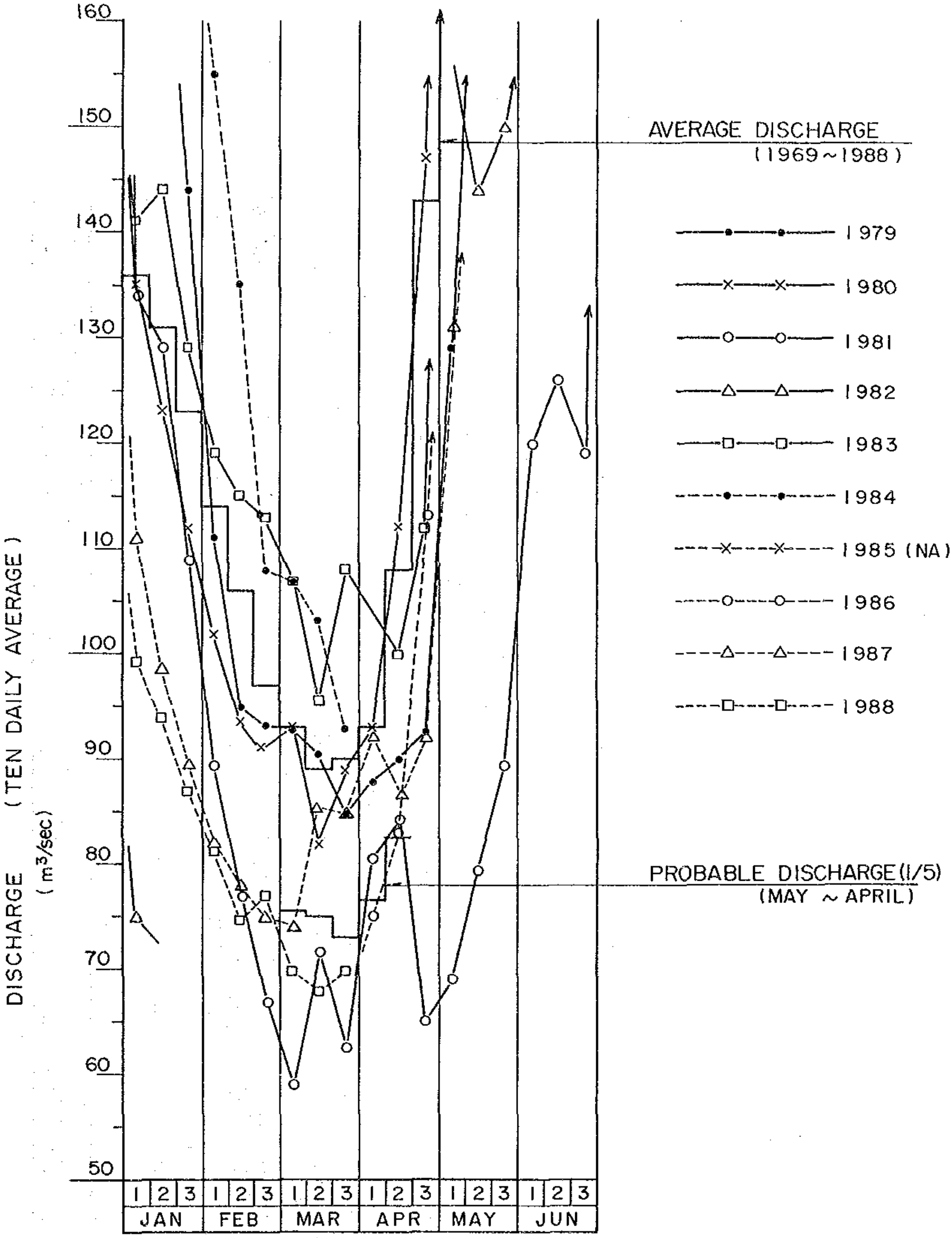
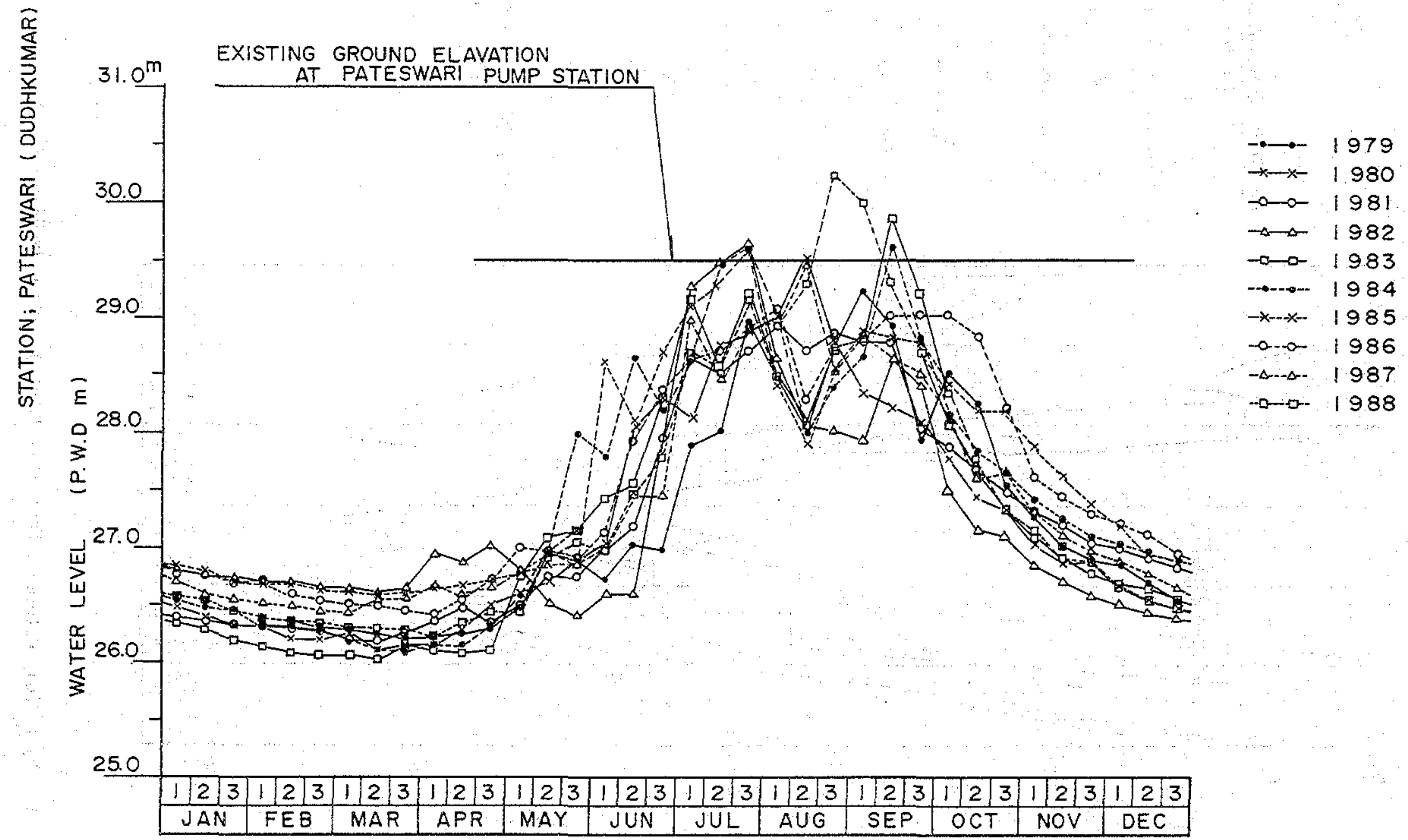


Fig. Ⅷ-1-3(b) Pateswari Past Flow-duration Curve, Dudhkumar River

Fig. Ⅷ-1-4 Pateswari Past Annual Water Levels of Ten Daily Average



Cases	Water intake	River discharge	Water intake ratio
Design values	42.78 m^3/s	93.1 m^3/s (Probable 1/5)	0.46 (third 10 day period in April)
Year:1979 (Plan year)	42.78	93.0	0.46 - do -
Year:1988 (Average year)	31.69	89.0	0.36 - do -

From these water intake ratios, it is judged that the water diversion requirements can be taken from the river without any particular difficulties. (refer to Table Ⅷ-1-3~5 and Fig. Ⅷ-1-3~4)

1-2. Groin Work

Groin work is planned for the purpose of maintaining the channel and the water depth near the right bank by the directional turning of the stream's flow towards the opposite (right) bank. The proposed groin work is outlined as follows.

(1) Direction

It is at a right angle against the river's flow.

(2) Number

In general, installing only one strong groin increases water flow disturbance and causes strong scouring near the groin. Therefore, it is proposed to set up a number of groins which work with even resisting powers.

The groins are to be progressively set in length the shorter starting downstream and the longest being upstream as to reduce the scouring action by lowering the groin's resistance upstream.

(3) Length

As to not decrease the cross-sectional area of the river as much as possible, the length of the groins are planned at less than 10% of the river's width. The crest of the groin work is put on a down grade of 1/10 ~1/100 towards the river's center.

(4) Height

The height of the groin's crest is set at 0.5 ~1.0 meters over the mean water level, taking flooding into consideration.

(5) Interval

Groin intervals are set as about 1.5~2.0 times the groin's length.

(6) Foundation

To cope with the scouring action, especially at the tip of the groin, protection work is to be done at the foot.

Groin work plan diagrams are shown in Fig. Ⅷ-1-5, and Fig. Ⅷ-1-6.

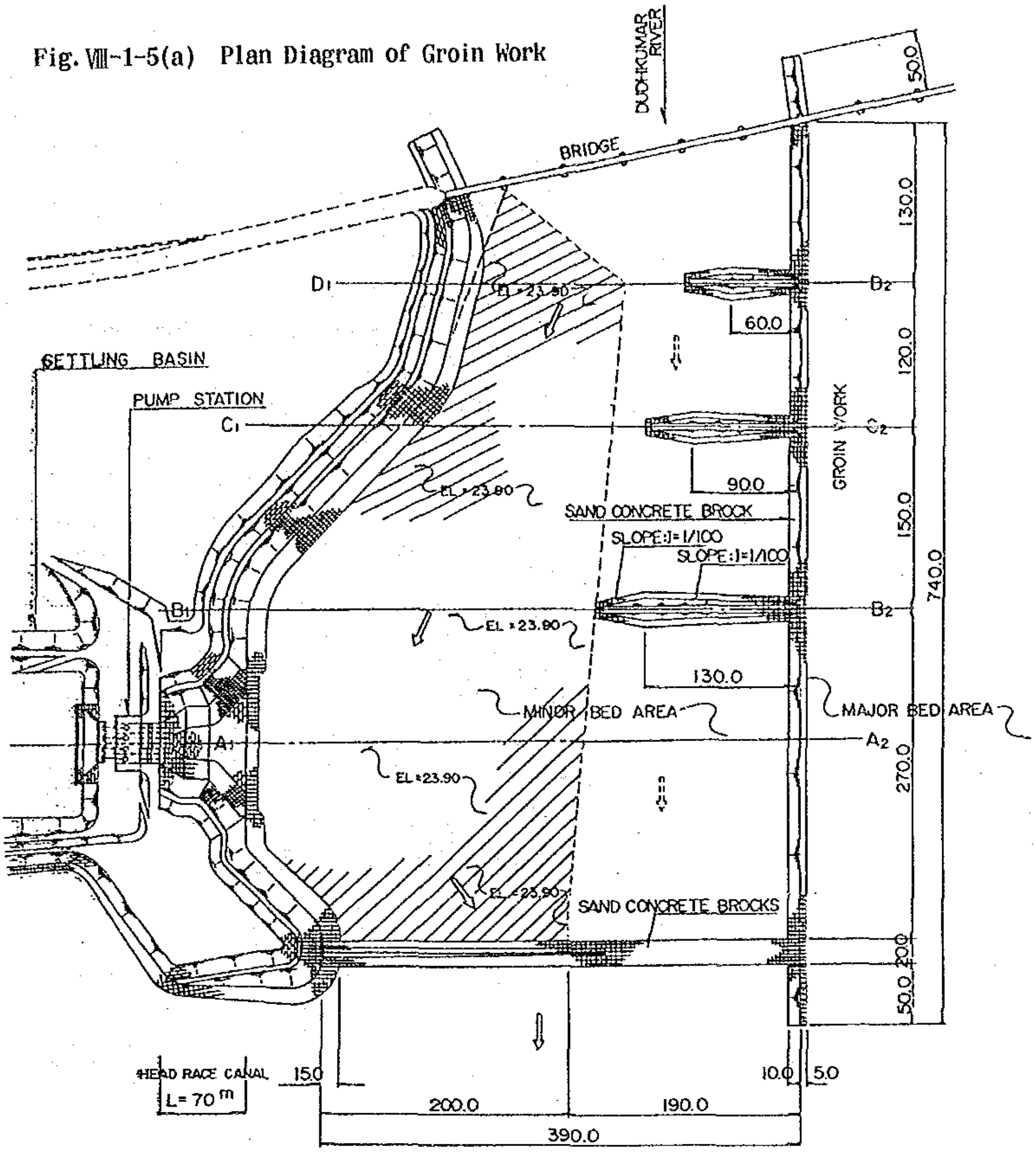
1-3. Inlet Basin

An inlet basin is planned to secure a smooth water intake and for settling sand. The scale for the settling basin is determined with the following formula.

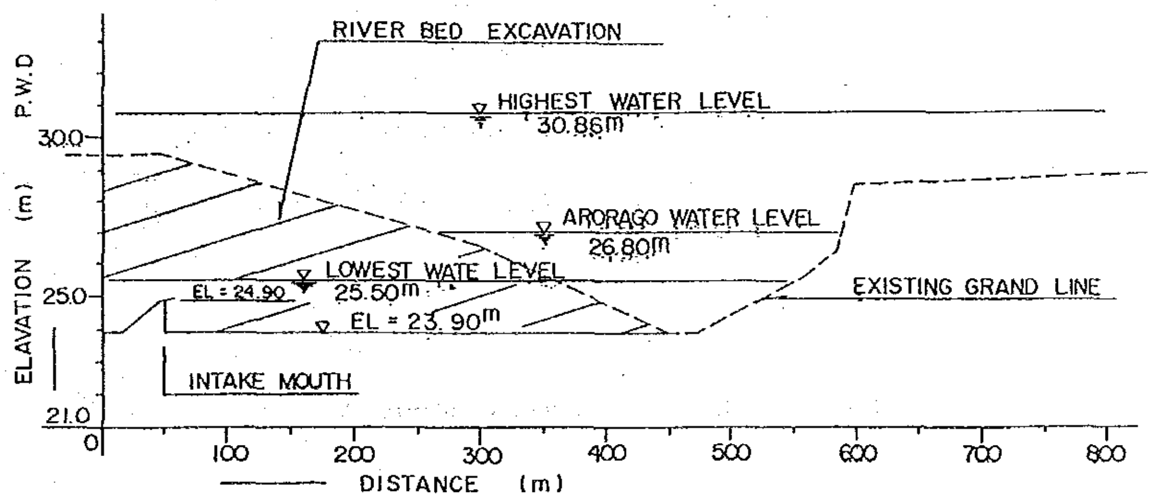
$$L = K \frac{Q}{B \cdot Vg} \rightarrow A = B\,L = K \frac{Q}{Vg} \; (m^2)$$

where, L ; Length of the settling basin (m)
B ; Width of - do -
A ; Area of - do -
K ; Safety factor 2.0
Q ; Discharge 42.78 m^3/sec
Vg ; Settling velocity of sediment minimum size (mm)
Vg = 0.005 m/sec (Grain size : 0.1mm)

Fig. Ⅷ-1-5(a) Plan Diagram of Groin Work



A1 — A2



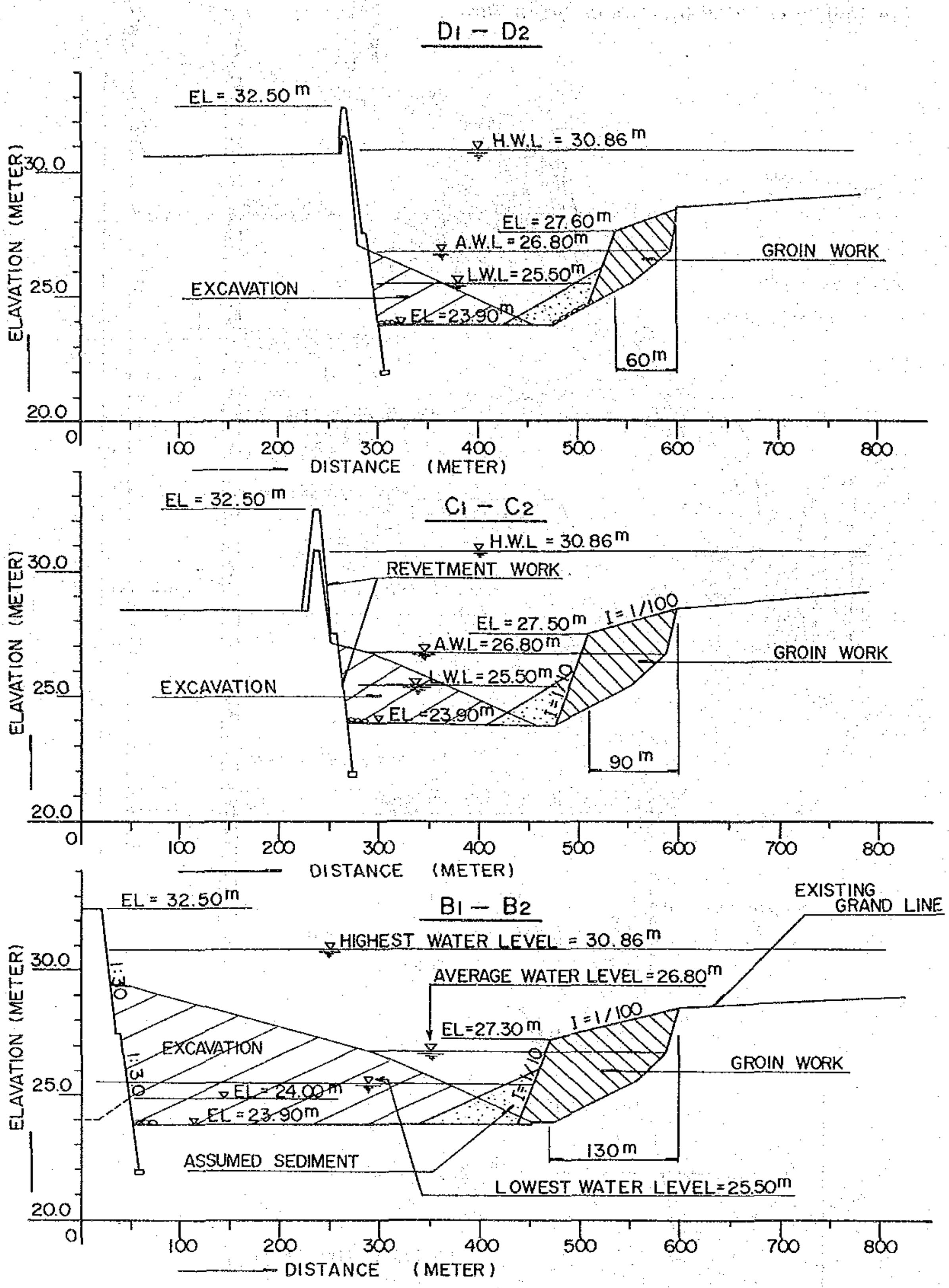
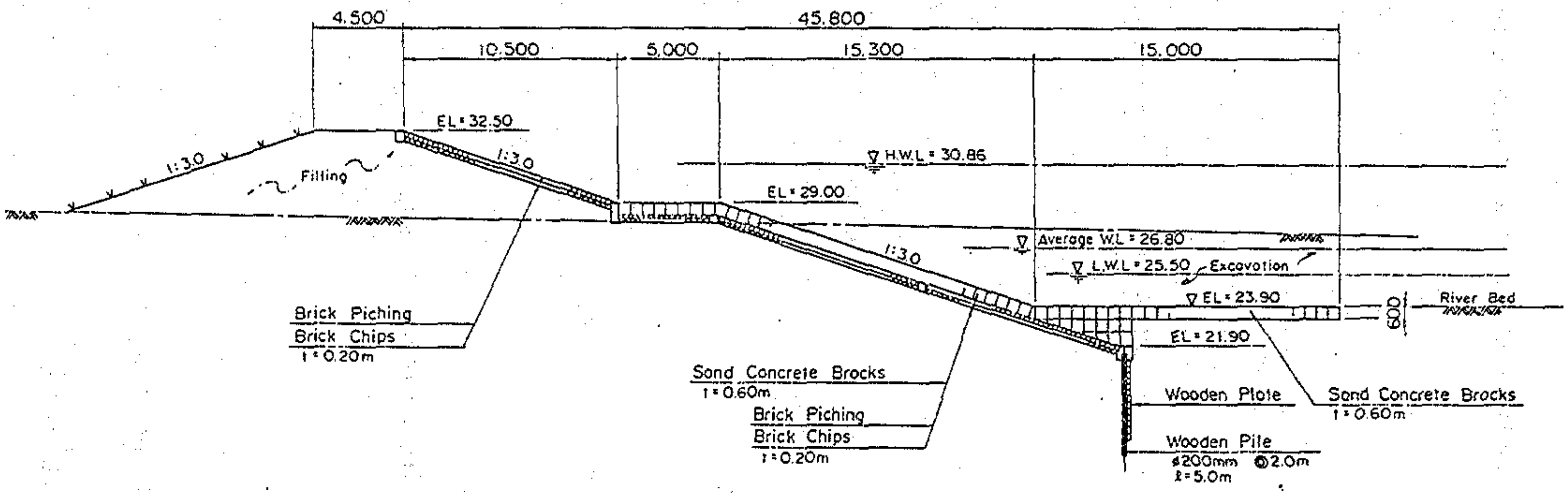


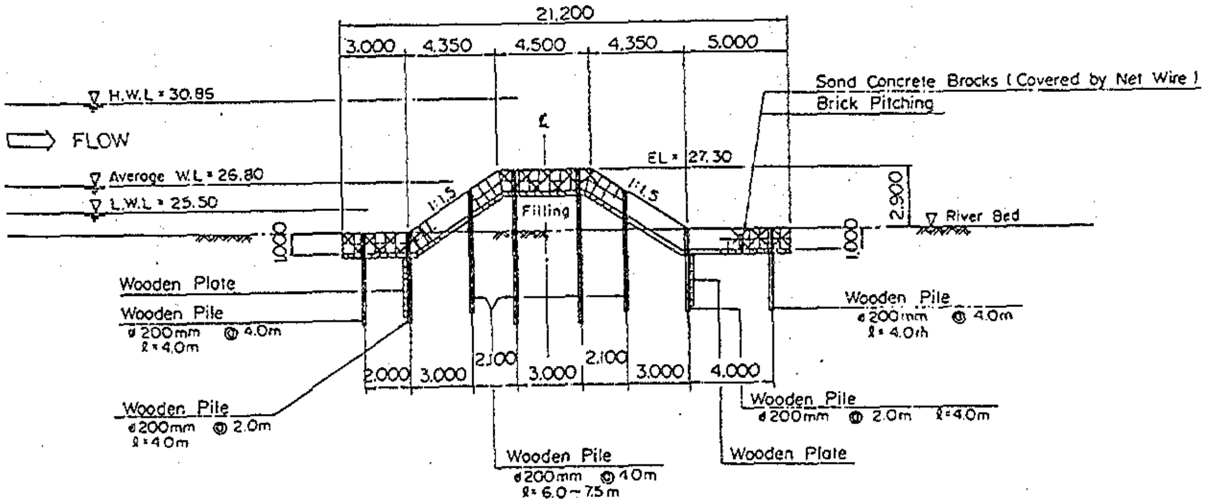
Fig. Ⅷ-1-5(b) Plan Diagram of Groin Work

Fig. Ⅷ-1-6 Standard Cross Section of River Revetment Work and Groin Work

River Revetment Work



Groin Work



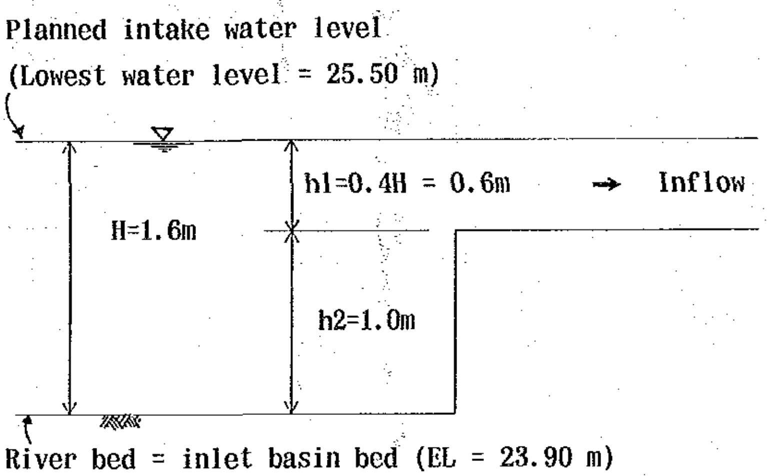
$$A = K \frac{Q}{B \cdot Vg} = 2.0 \frac{42.78}{0.005} = 17,112 \rightarrow 18,000 \ m^2$$

The results from the grain size analysis at the pump station site are shown in Fig. Ⅷ-1-7.

1-4. Intake Mouth

(1) Intake Sill

An intake sill is planned to prevent sediment inflow at the lowest water level.



(2) Intake Width

The intake width is decided with the following formula :

$$B = \frac{Q}{h1 \cdot V} \ (m)$$

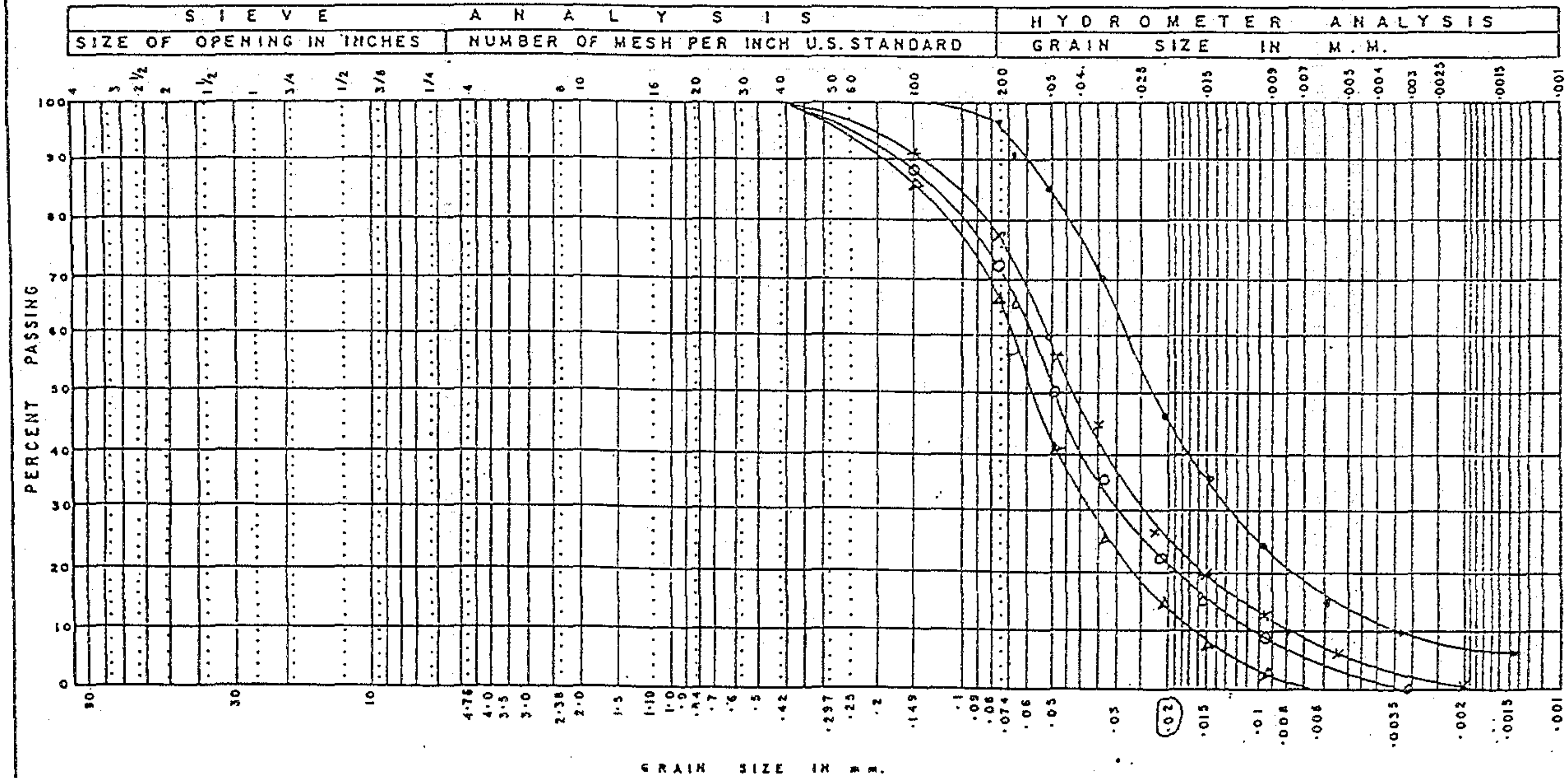
where, B ; Intake width (m)

Q ; Intake discharge = 42.78 m^3/sec

h1 ; Inflow depth (m)

V ; Intake velocity = 0.6 m/sec

Fig. Ⅷ-1-7 Soil Particle Size Distribution



COBBLES	COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	FINES (SILT OR CLAY)

LOCATION	BORING NO	SAMPLE NO	Depth - m From	To	SOIL CLASSIFICATION (UNIFIED SYSTEM)	
Pateswari Pump Station	1	D-1	0.50	0.96	Medium compressible SILT, trace fine sand	-●-●-●-
		D-2	1.50	1.96	Non-plastic SILT, some fine sand	-×-×-×-
		D-3	2.50	2.96	Non-plastic SILT, some fine sand	-○-○-○-
		D-4	4.00	4.46	Non-plastic SILT, some fine sand	-△-△-△-

$$B = \frac{42.78}{0.6 \times 0.6} = 119 \rightarrow 120 \text{ m}$$

1-5. Head Race Cannal and Pump Suction Sump

The width of the head race canal is to be designed as to introduce water smoothly from the intake mouth to the pump suction sump. This plan is shown in Fig. Ⅷ-1-8.

The velocity of each section is shown below.

$$V = \frac{Q}{B \cdot H} \text{ (m/sec)}$$

section	Discharge	Width	Depth	Velocity
	(Q) m³/s	B (m)	H (m)	V (m/s)
A	42.78	120.0	0.6	0.60
B, C	- do -	60.0	1.6	0.46
D	- do -	3.0^m x 8 = 24^m	3.4	0.52
E	- do -	7.0^m x 4 = 28^m	5.8	0.26

1-6. Intake Point Non-Uniform Flow Calculation

Non-uniform flow calculations were carried out to reconfirm water intake.

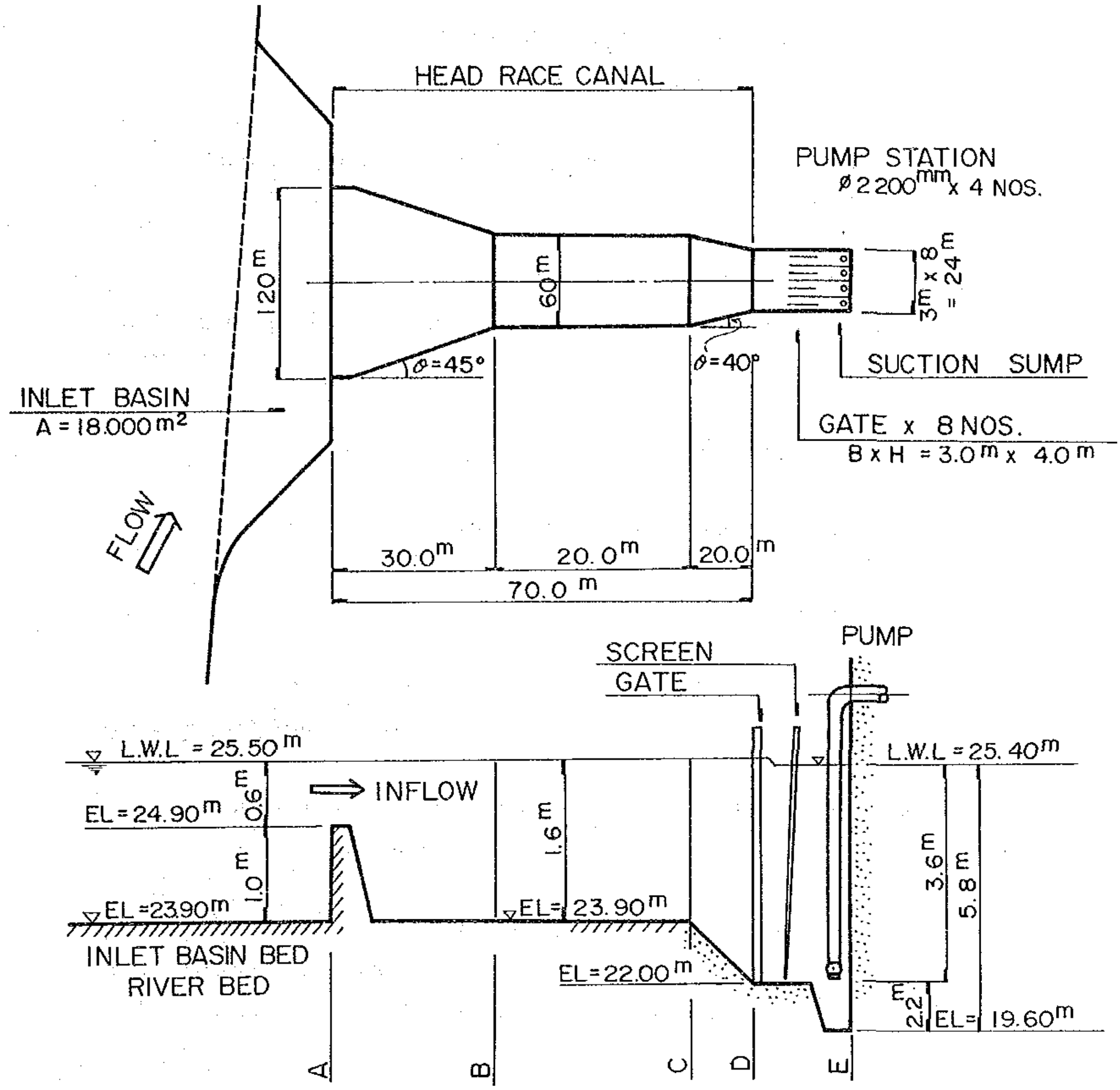


Fig. Ⅷ-1-8 Head Race Canal and Pump Suction Sump

(1) Basic Calculation Conditions

a) To be calculated from the lower reach side.
Starting water level H=1.6 m, L,W,L=25.50 m

b) The calculated river cross section numbers were 16 sections.
(refer to Fig. Ⅷ-1-9)

c) River discharge : Q = 82.6 m³/sec
This discharge is probable Q (1/5) in the second 10 day period of April.

d) Intake discharge : Qp = 42.78 m³/sec

e) River roughness coefficient n = 0.025

(2) Analysis results

- Water depth of the upper reach side (16 section number) :
H = 1.605m ⇒ +0.005m

- Starting water depth of the lower reach side (1 section number) :
H = 1.600m

In other words, the water depth of the lower reach side after water intake is 1.595m therefore intake work is not a problem. Namely, the velocities of these sections are about 0.10~0.30 m/sec. (refer to Table Ⅷ-1-6)

But the range in change of the river's minimum water level is about 0.5~0.7m , in order for this to be changed the river bed and a variation in the width of the water's surface need to be changed. (refer to Fig. Ⅷ-1-1)

Considering this point the proposed backwater ground sill work is to be set and is expected to secure intake work indirectly near the lower reach side of the intake point.

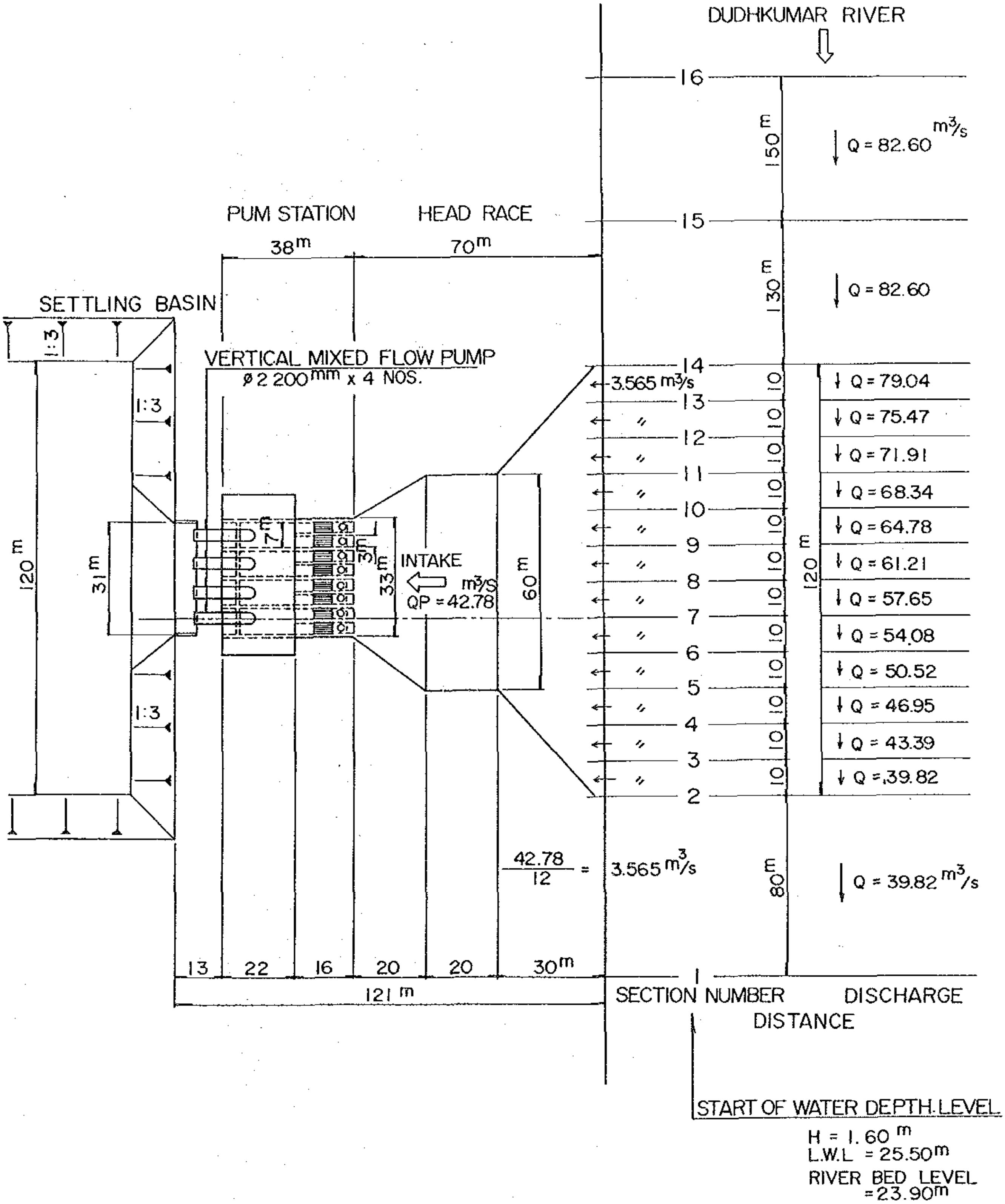


Fig. Ⅷ-1-9 Diagram of Non-uniform Flow Calculation

TableⅧ-1-6 Non-Uniform Flow Caluculation Results

Station	Section	Distance (m)	Total Distance (M)	Water Depth (m)	River Bed Level (m)	Water Surface Level (m)	Velocity (m/s)	Discharge (m3/s)	Flow Area (m2)	Roughness Coefficient
0	①	0.000	0.000	1.600	23.900	25.500	0.095	39.82	420.03	0.025
1	2	80.000	80.000	1.600	23.900	25.500	0.082	39.82	488.31	0.025
2	3	10.000	90.000	1.600	23.900	25.500	0.082	39.82	488.31	0.025
3	4	10.000	100.000	1.600	23.900	25.500	0.089	43.39	488.29	0.025
4	5	10.000	110.000	1.600	23.900	25.500	0.093	46.95	504.31	0.025
5	6	10.000	120.000	1.600	23.900	25.500	0.100	50.52	504.31	0.025
6	7	10.000	130.000	1.600	23.900	25.500	0.107	54.08	504.32	0.025
7	8	10.000	140.000	1.600	23.900	25.500	0.114	57.65	504.33	0.025
8	9	10.000	150.000	1.600	23.900	25.501	0.118	61.21	520.34	0.025
9	10	10.000	160.000	1.600	23.900	25.500	0.124	64.78	520.33	0.025
10	11	10.000	170.000	1.600	23.900	25.500	0.131	68.34	520.30	0.025
11	12	10.000	180.000	1.600	23.900	25.500	0.138	71.91	520.27	0.025
12	13	10.000	190.000	1.600	23.900	25.500	0.145	75.47	520.25	0.025
13	14	10.000	200.000	1.600	23.900	25.500	0.152	79.04	520.23	0.025
14	15	130.000	330.000	1.600	23.900	25.500	0.283	82.60	292.06	0.025
15	⑯	150.000	480.000	1.605	23.900	25.505	0.307	82.60	269.12	0.025

2. Type of Pumping Station at Pateswari

2-1. Type of Pumping station

The irrigation water required for the Project will be diverted directly from the Dudhkumar river. In the case, where the irrigation water is lifted from the river by pumps, the following three types of structures are generally compared:

(1) Screw Pump Type Structure

(2) Inclined Pump Type Structure

These pump facilities are installed alone on the side slope of the river embankment. These types are applicable when the river's course is stable.

(3) Vertical Mixed Flow Pump Type Structure

This pump's facilities are installed inside the existing river bank with connection canals (head race) constructed to conduct water.

The above three pump types were carefully compared in due consideration with their adaptability to the sites, construction costs and their O & M cost. The following definitions were employed in the study:

- Design Discharge Qp = 42.78 m^3/sec
- Water Design Level
  - Lowest Water Level (L.W.L) = 25.50^m
  - Highest Water level (H.W.L) = 30.86^m
  - Outlet Pond Water Level
    - Water Delivery Level = 33.52^m
    - Lowest Water Level for Delivery = 30.52^m

Based upon the previous definitions, the main facilities for the pump systems are to be designed as per the following Table Ⅷ-2-1:

For this connection, the following pump types were excluded from the comparative study for the following reasons:

a) Horizontal Mixed Flow Pump

- Driving operations are difficult when the pump starts being that it requires to be active with the filling of pump water. The Pump's suction performance is a problem when dealing with a 4^{m} variation range and particularlywhen lifting low water levels. In any case the suction head variation is;

$NPSH_{av} \geqq NPSH_{re}$ (for Cavitation)

NPSH ; Net Positive Suction Head

av ; Available

re ; Required

b) Submergible Moter Pump

- There are no large boring facilities.
- The motor cost is extremely high.
- The supply of spare parts is very difficult.
- O & M is very difficult and expensive.

c) Floating Type Pump

- A barge for large bore floating type pumps can not be installed being that it requires a $1.5^{m} \sim 2.0^{m}$ water depth in the dry seasons.

2-2. Comparative Study for Pumping Station

Three types of pumping stations were comparatively studied and summarized in Table Ⅷ-2-2～4.

Regarding the general lay-out, see drawings in Fig. Ⅷ-2-1～3.

The results of this comparative study, show that the vertical mixed flow type pump station is recommended for the project due to the following reasons.

- The construction cost is cheaper than the other two types.
- O & M costs are lower than the other two types.

Table Ⅷ-2-1 Main Dimension of Each Pump Type

Items \ Pump Type	Screw Pump	Inclined Mixed Flow Pump	Vertical Mixed Flow Pump
Design Discharge (m^3/sec)	42.78	42.78	42.78
Number of Pumps	12 NOS	22 NOS	4 NOS
Bore (mm)	φ3,400	φ1,000	φ2,200
Capacity	3.57 m^3/sec = 214 m^3/min	1.95 m^3/sec = 117 m^3/min	10.70 m^3/sec = 642 m^3/min
Total Head (m)	9.2	9.0	8.6
Pump Speed (r.p.m)	26	493	225
Moter Poer (Kw/Unit)*1	500	250	1,220
Total Moter Power (Kw)	6,000	5,500	4,880
Pump Efficiency (%)	75	82	86

*1

$$P = \frac{K \cdot r \cdot Q \cdot H}{np \cdot ng} (1+R) \text{ KW}$$

where : P ; Pump Power (KW)
K ; 0.163
r ; Specific Gravity of Water 1.00
Q ; Pump Discharge (m^3/min)
H ; Total Pump Head (m)
np ; Pump Efficiency (0.75, 0.82, 0.86)
ng ; Moter Transmission Efficiency 0.95
R ; Pump Surplus Factor 0.10

Table Ⅷ-2-2 Special Characteristics of Each Type

Items \ Pump Type	Screw Pump	Inclined Mixed Flow Pump	Vertical Mixed Flow Pump
1. Driving & Operation	- ON-OFF drive at water suction level. - Available drive below L.W.L and drive is easy.	- ON-OFF drive at water suction level. - Not available drive below L.W.L	- ON-OFF drive at water suction level. - Not available drive below L.W.L
2. Maintenance	- Painting of impellor is required one time in per 1 to 2 year.	- Pump disassembly and inspection are done by Crane. - For large bores, reassembly is required with highly skilled personel.	—— do. left ——
3. Passing of Dust	- Possible to lift water with considerable amounts of dust.	- Possible to pass dust through screen.	- —— do. left ——
4. Sedimentation	- Dredging is needed at the intake mouth in the beginning of the dry season every year as to avoid a certain degree of sedimentation.	- —— do. left ——	- —— do. left ——
5. Pump Efficiency	- Low	- Medium	- High
6. Building and Civil Work			
· Space	- Large with many pumps	- Large with many pumps	- Small
· Structure	- Simple	- Simple	- Simple
· Unit floor Load	- Medium	- Medium	- Large
7. Initial Cost			
· Pump facilities	- Expensive	- Expensive	- Cheap
· Civil works	- — do —	- Cheap	- Cheap
8. Lining Cost	- Expensive	- Expensive	- Cheap
9. General Evaluation Ranking	- 3	- 2	- ◎1

Table Ⅷ-2-3 Cost Comparision for Each Type

$\times 10^3$ TK

Item \ Type	Screw Pump Type	Inclined Mixed Flow Pump Type	Vertical Mixed Flow Pump Type
1. Initial Cost			
(1) Pump Facilities Cost	898,000	901,000	656,000
(2) Pump House	26,250	35,060	27,000
(3) Civil Work	90,720	20,160	83,580
Total	1,014,970	956,220	766,580
(Initial Cost)	× 0.1588 *2	× 0.1588	× 0.1588
2. Annual Cost	161,177	151,847	121,732
3. Annual O & M Cost *1	61,840	57,677	49,744
Total Annual Cost (2 + 3)	223,017 (130%)	209,524 (122%)	◎ 171,476 (100%)

*1 ; Refer to TableⅧ-2-4

*2 ; Refer to (0.1588)

The estimation formula for the capital recovery ratio is shown as follows:

$$P = \frac{i(1+i)^n}{(1+i)^n - 1} \times (1 + \text{Interest Ratio for Construction Periods})$$

Interest Ratio = $\alpha \times f' \times t$

where : i ; Interest 12%

α ; Former's share 40%

f' ; Interest of Farmer's share 14%

t ; 5 years construction period

n ; Durable Life = 30 year

$$P = \frac{0.12 \times (1+0.12)^{30}}{(1+0.12)^{30} - 1} \times (1 + 0.4 \times 0.14 \times 5) = 0.1588$$

Table VIII-2-4 Annual O & M Cost for Each Type

$\times 10^3$ TK

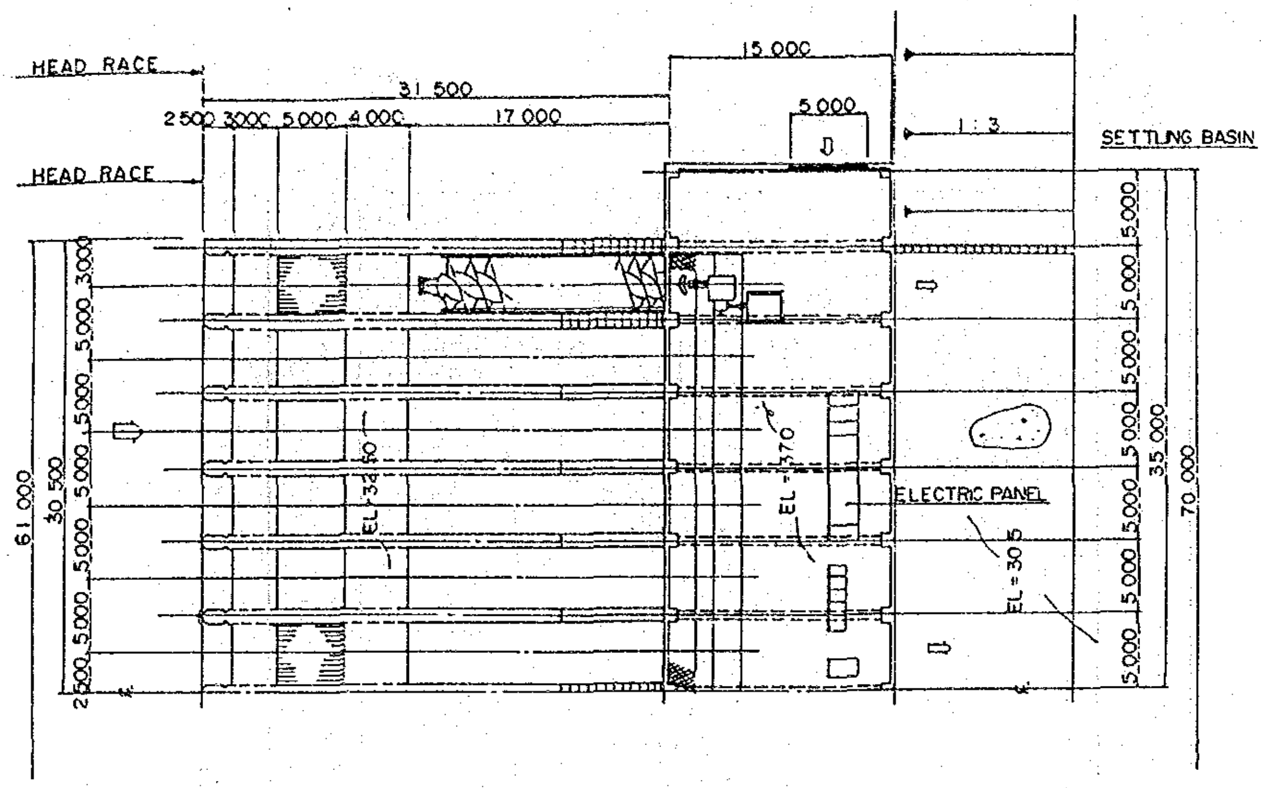
Item \ Type	Screw Pump Type	Inclined Mixed Flow Pump Type	Vertical Mixed Flow Pump Type
1. Civil Work	$10^{Persons} \times 20^{days} \times 12^{M} \times 50^{TK}$ = 120	— do — = 120	— do — = 120
2. Maintenance Work for Pump Facilities (2% of Pump Facilities Cost. Excluding Tax)	$551,000 \times 10^{3TK} \times 0.02$ = 11,020	$553,000 \times 10^{3TK} \times 0.02$ = 11,060	$417,000 \times 10^{3TK} \times 0.02$ = 8,340
3. Electricity - Service Charge	$350^{TK/M} \times 12^{M}$ = 4	— do — = 4	— do — = 4
- Basic Charge	$40^{TK/KW/M} \times 12^{M} \times 6,000^{KW}$ *1 = 2,880	$40^{TK/KW/M} \times 12^{M} \times 5,500^{KW}$ *1 = 2,640	$40^{TK/KW/M} \times 12^{M} \times 4,880^{KW}$ *1 = 2,342
- Utility Charge	$4.25^{TK/KWH} \times 0.25 \times 3.286^{hr/r} \times 6,000^{KW} + 1.8^{TK/KWH} \times 0.75 \times 3,286^{hr} \times 6,000^{KW}$ = 47,564	$4.25^{TK/KWH} \times 0.25 \times 3.286^{hr/r} \times 5,500^{KW} + 1.8^{TK/KWH} \times 0.75 \times 3,286^{hr} \times 5,500^{KW}$ = 43,601	$4.25^{TK/KWH} \times 0.25 \times 3.286^{hr/r} \times 4,880^{KW} + 1.8^{TK/KWH} \times 0.75 \times 3,286^{hr} \times 4,880^{KW}$ = 38,686
4. Dredging (Man Power and Sub-Sand Pump $\phi 150^{mm} \times 11^{kw} \times 2^{NOS}$)	$8,000 m^3/year \times 29^{TK/m^3}$ = 232 $15^{d} \times 24^{hr/d} \times 11^{KW} \times 2 \times 2.5^{TK/KW}$ = 20 = 252	— do — = 232 — do — = 20 = 252	— do — = 232 — do — = 20 = 252
Total Annual O & M Cost	61,840 (124%)	57,677 (116%)	49,744 (100%)

*1 : Total Pump Power (KW)
*2 : Annual Operation Hours (Average Year) T(hour) → Average Year = 1988

$T = V/Qp \times 3,600^{sec} = 506,000^{*3} \times 10^{3} m^3/42.78 m^3/s \times 3,600^{sec} = 3,286^{hr}$

where, V ; Annual Total Water Intake Volume = 506 Million m^3/year
Qp ; Total Pump Capacity $42.78 m^3$/sec

*3 : Refer to Table VIII.1-3.



MAIN DIMENSIONS OF PUMP	
TYPE	SCREW PUMP
BORE	ø 3 400
CAPACITY	3.57 m^3/sec = 214 m^3/min
TOTAL PUMP HEAD	9.2 m
PUMP SPEED	26 rpm
MOTOR POWER	500 KW
UNITS	12 NOS
TOTAL MOTOR POWER	6 000 KW

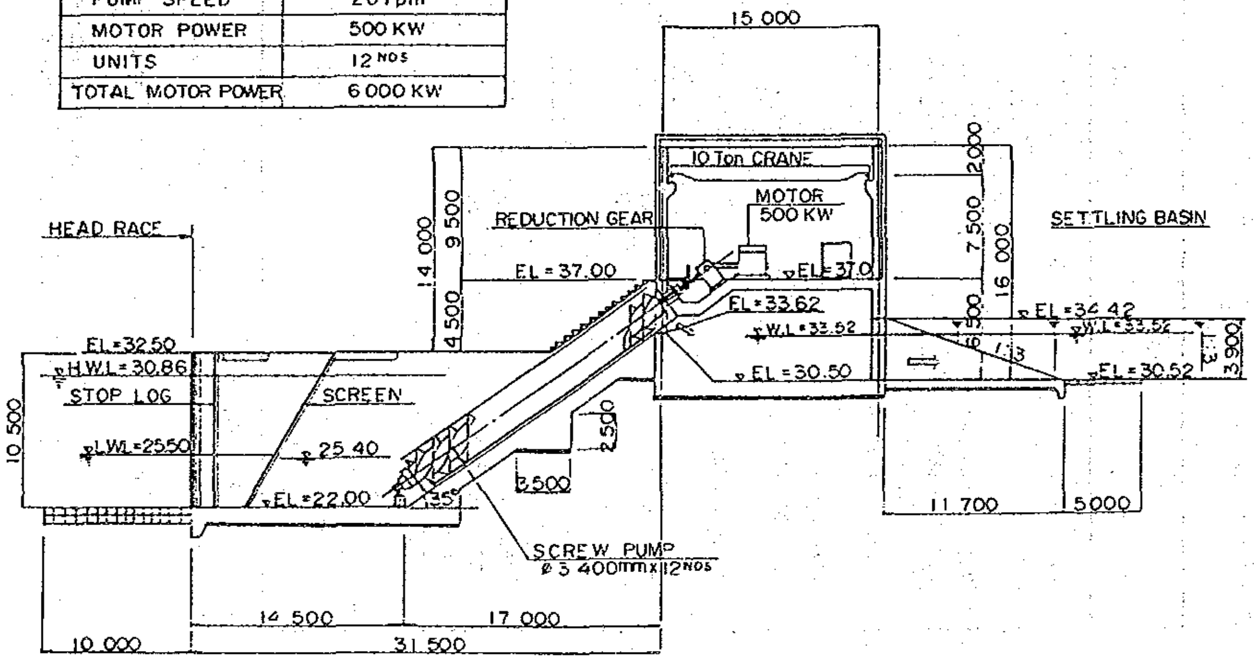
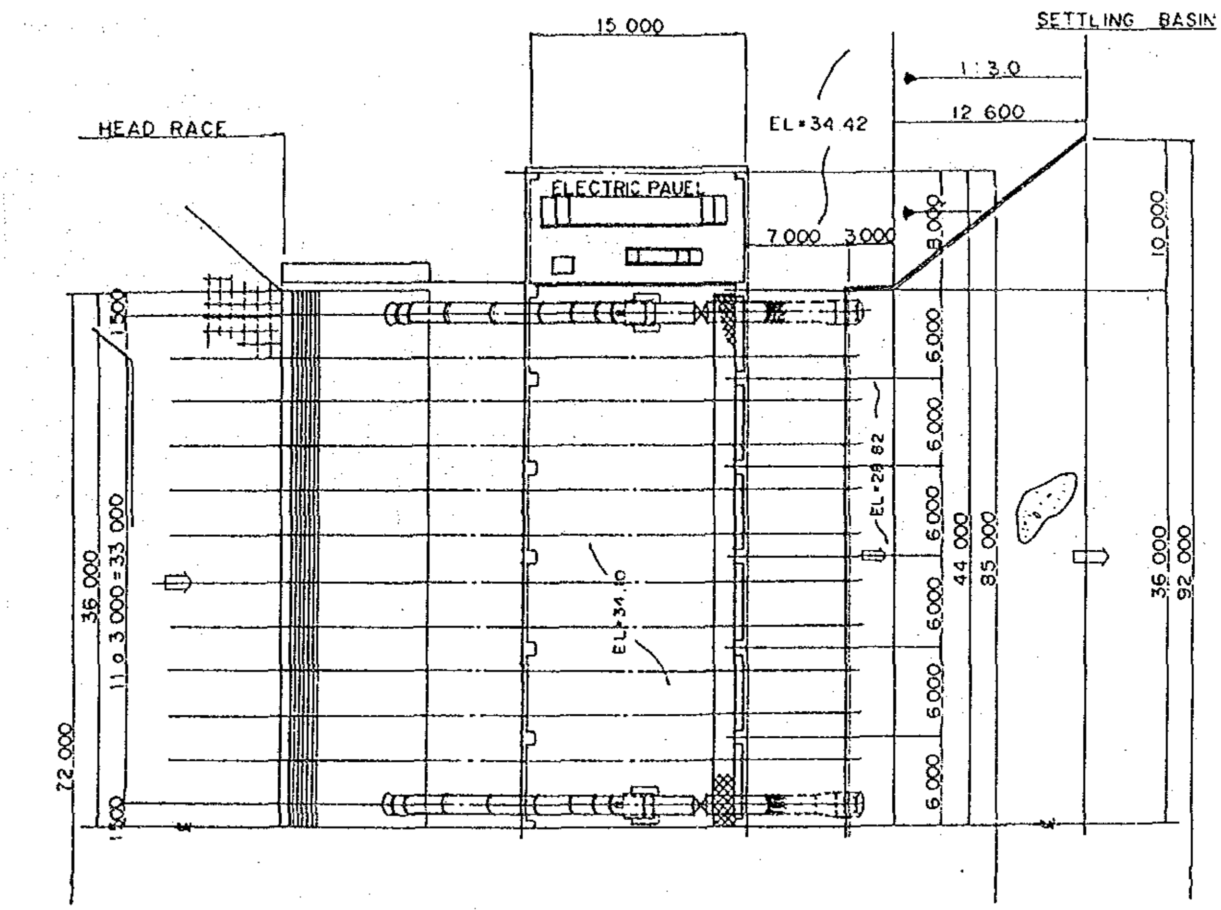


Fig. Ⅷ-2-1 Screw Pump Dimensions



MAIN DIMENSIONS OF PUMP	
TYPE	INCLINED MIXED FLOW PUMP
BORE	ø 1000mm
CAPACITY	1.95m³/sec=117 m³/min
TOTAL PUMP HEAD	9m
PUMP SPEED	493rpm
MOTER POWER	250KW
UNITS	22 NOS
TOTAL MOTOR POWER	5 500 KW

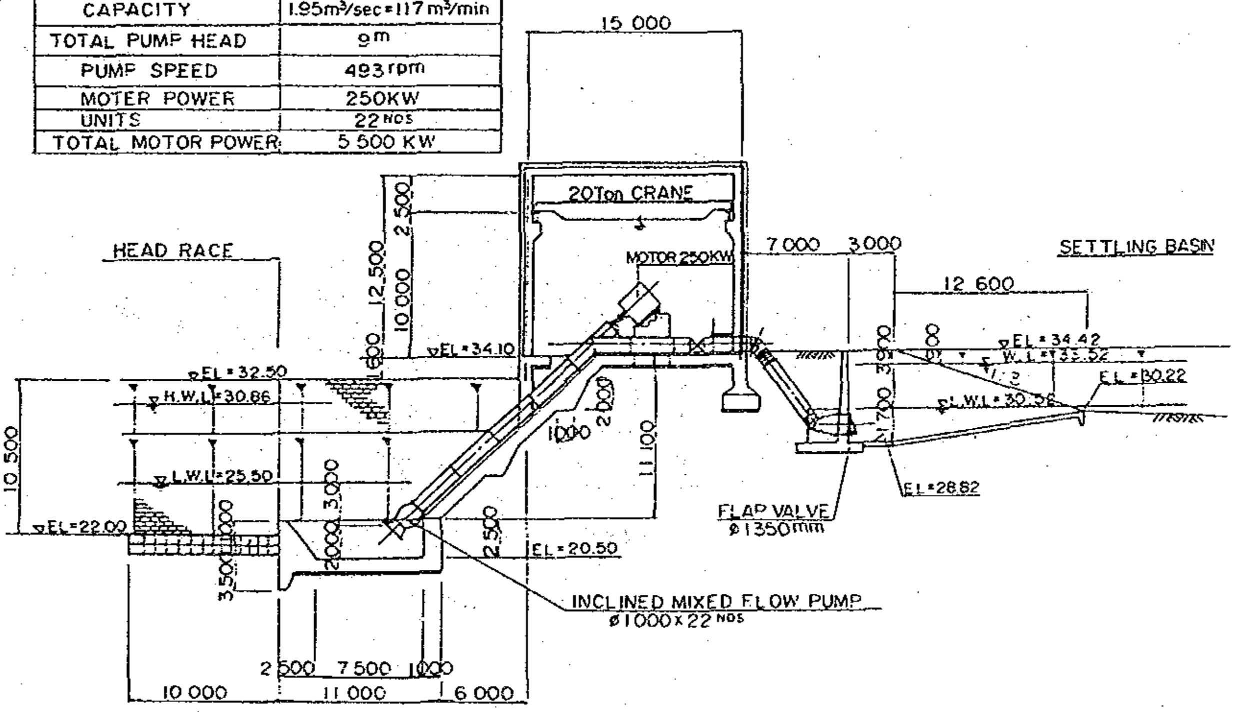
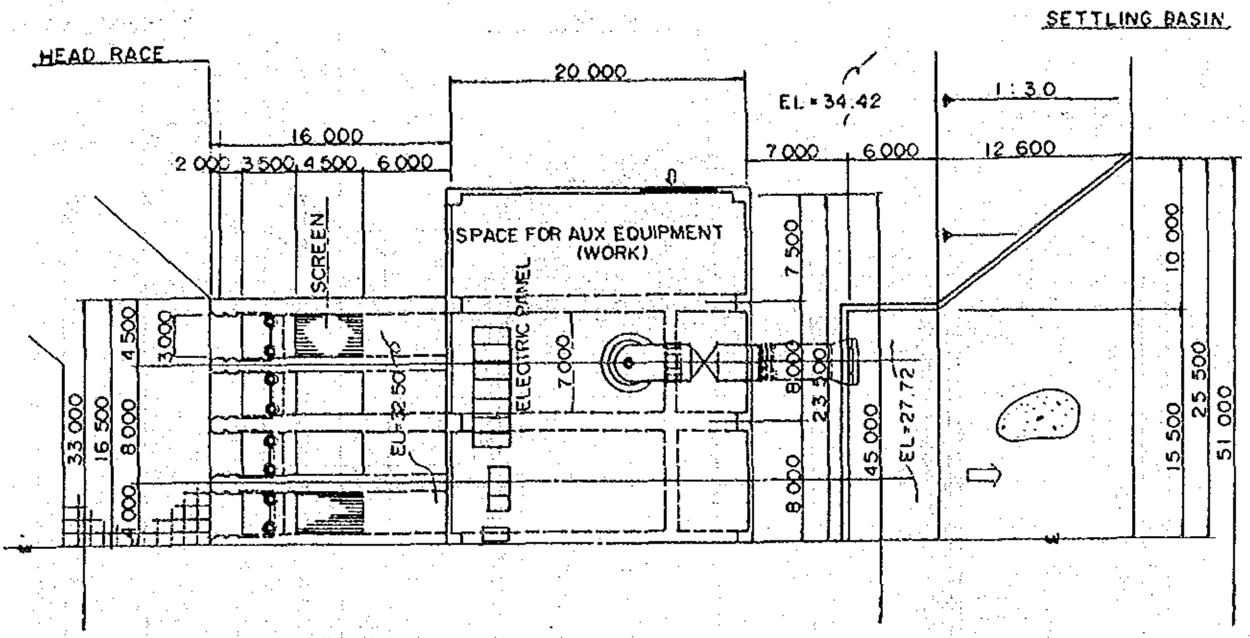


Fig. VIII-2-2 Inclined Mixed Flow Pump Dimensions



MAIN DIMENSIONS OF PUMP	
TYPE	VERTICAL MIXED FLOW PUMP
BORE	ø 2 200 mm
CAPACITY	10.70 m³/sec = 642 m³/min
TOTAL PUMP HEAD	8.6 m
PUMP SPEED	225 rpm
MOTOR POWER	1 220 KW
UNITS	4 NOS
TOTAL MOTOR POWER	4 880 KW

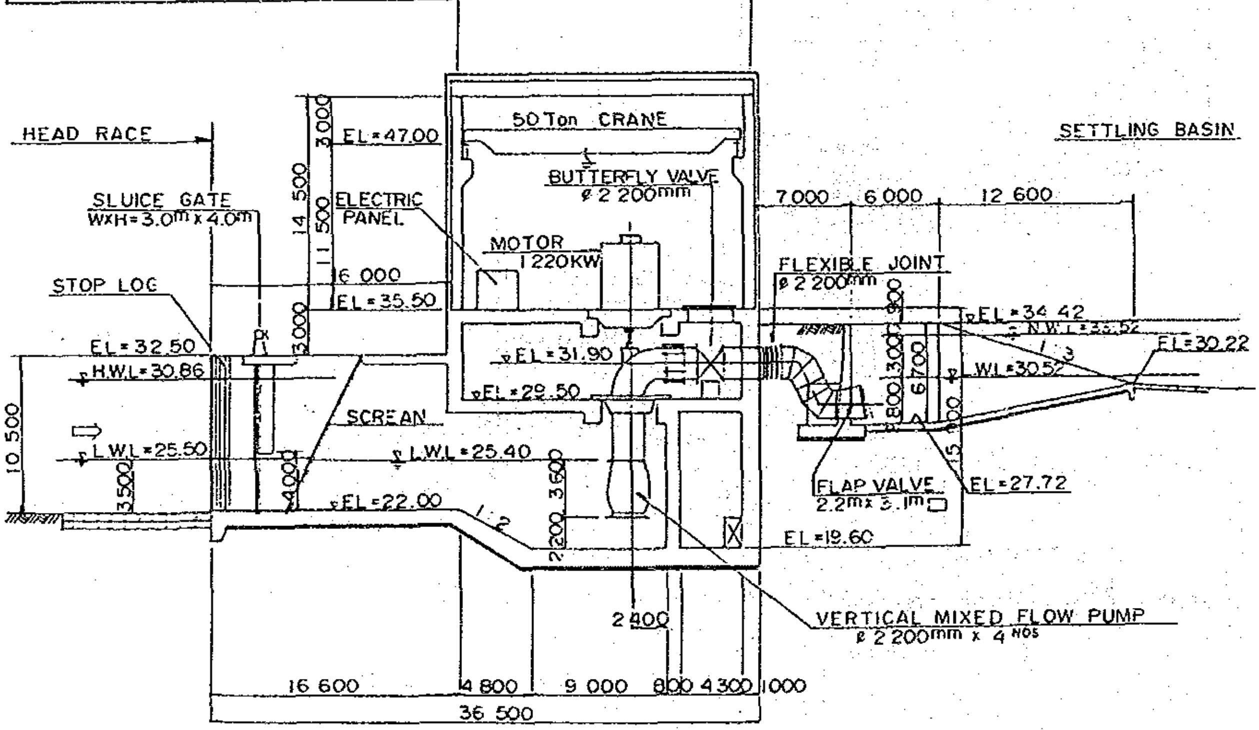


Fig. VIII-2-3 Vertical Mixed Flow Pump Dimensions

3. Pateswari Pumping Station

3-1. Number of Pumps

The number of vertical mixed flow type pumps is to be planned considering comprehensively the following points:

(1) To respond to the fluctuations of the required intake water during a year.

(2) To keep initial costs and running costs at a reasonable level,

(3) Risk diversification,

(4) and machinery transportation limitations.

Note* a), the gross water requirements for 10 days intervals in 1979, when the maximum annual water consumption is required, and in 1988, when the average is required, were analysed (Table Ⅷ-1-3, Fig. Ⅷ-3-1～2).

3-2. Comparative Study for the Number of Pumps

It seems that the larger the pump's bore is the higher the efficiency becomes (Table Ⅷ-3-1), and the better the efficiency the lower the running cost.

Cosidering the initial cost and the running cost, the following cases were compararED.

(1) Case - 1 3 Pumps

(2) Case - 2 4 Pumps

(3) Case - 3 5 Pumps

The main pump facility systems are designed as in Table Ⅷ-3-2～3.

Table Ⅷ-3-1 Pump Efficiency

Bore	Low Head Pump Type			
	Horizontal Type		Vertical Type	
	Mixed Flow	Axial Flow	Mixed Flow	Axial Flow
mm	%	%	%	%
600	79	77	78	76
700	80	78	79	77
800	81	79	80	78
900	82	80	81	79
1000	83	81	82	80
1200	84	82	83	81
1350	84.5	82.5	83.5	81.5
1500	85	83	84	82
1650	85.5	83.5	84.5	82.5
1800	86	84	85	83
2000	86	84	85	83
2200			86	84
2400			86	84
2600			87	85
2800			87	85

Fig. Ⅷ-3-1 Gross Irrigation Water Requirements and Pump Numbers

— Pateswari Pump Station, Plan Year: 1979 —

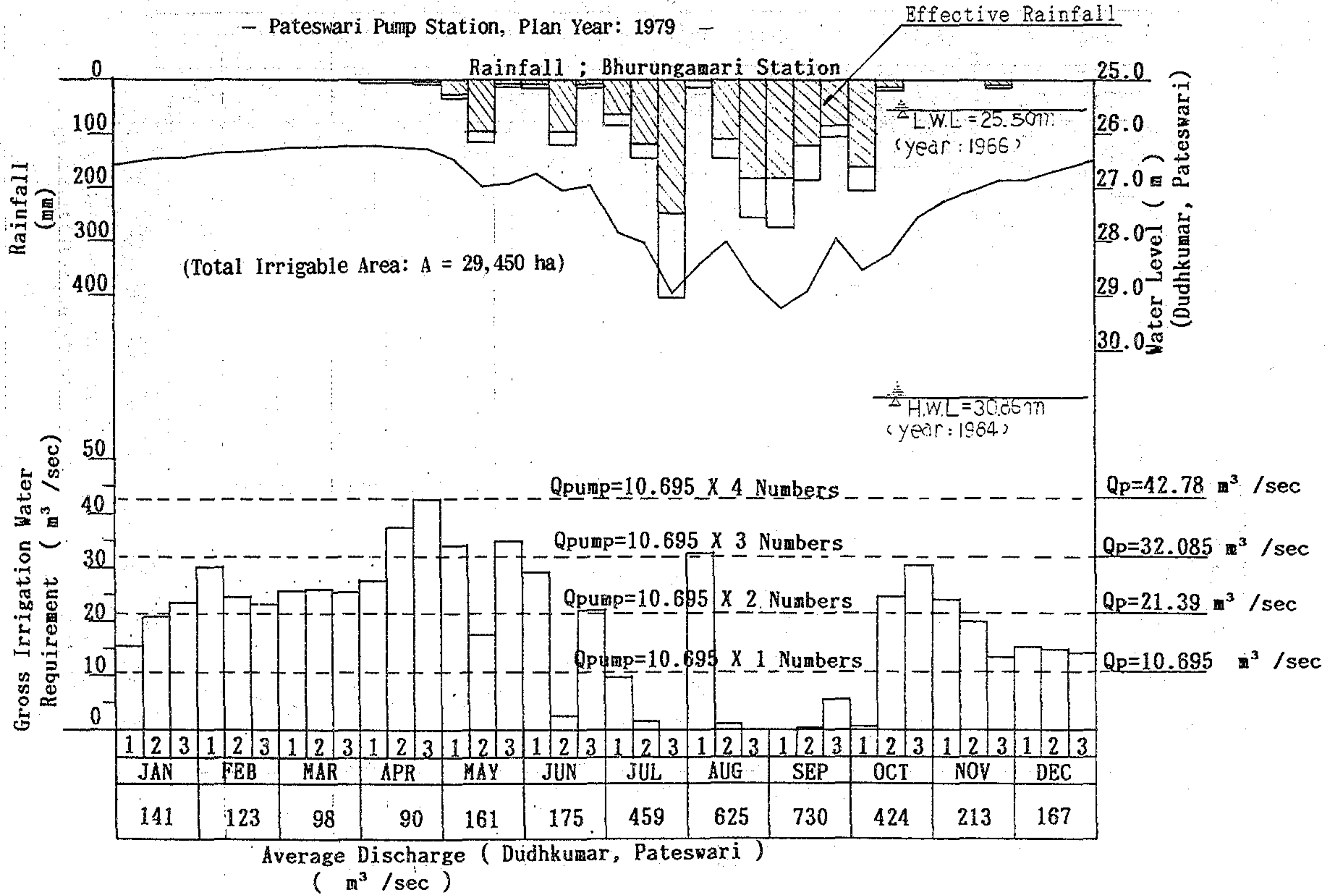


Fig. VIII-3-2 Gross Irrigation Water Requirements

— Pateswari Pump Station, Average Year: 1988 —

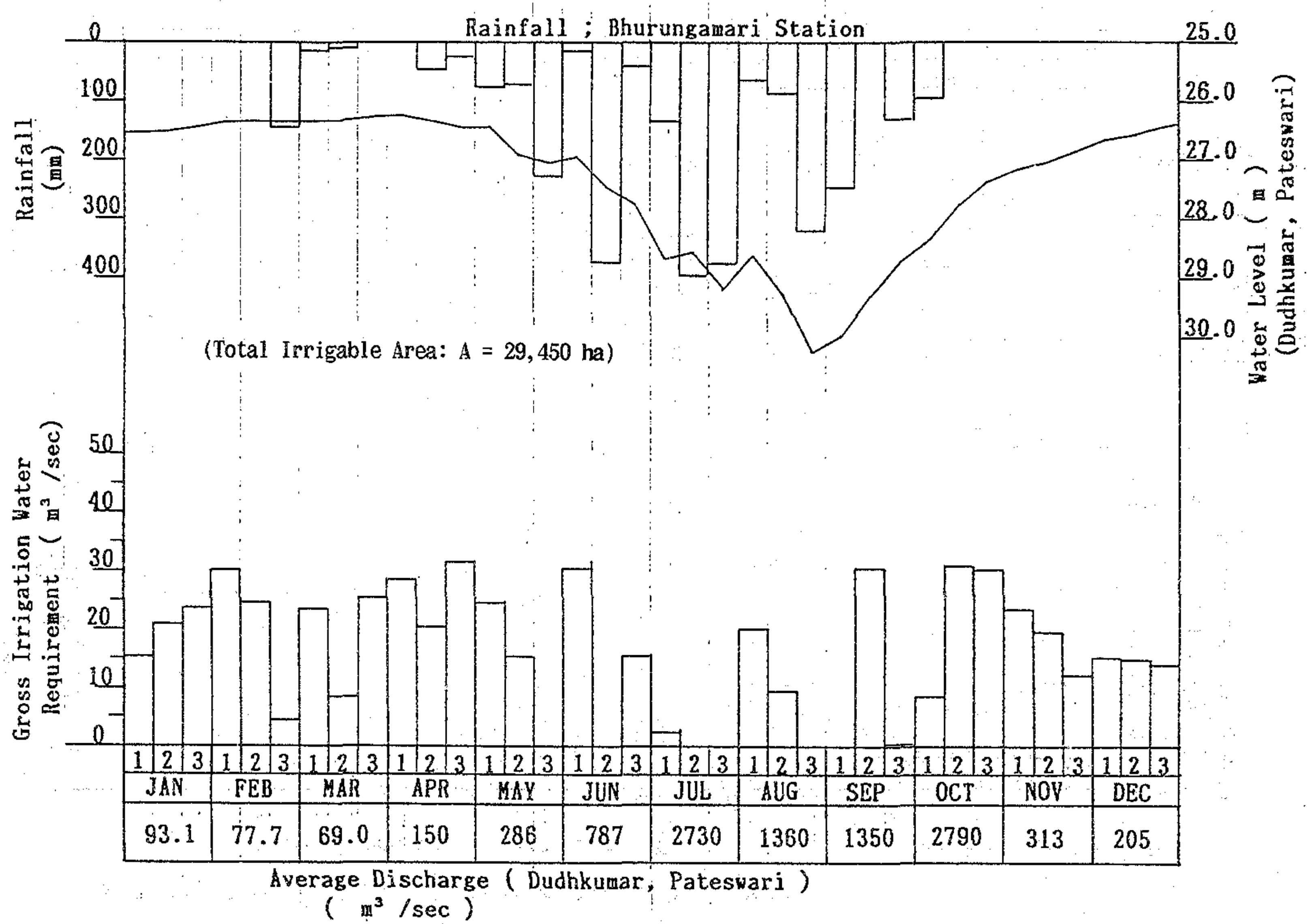


Table Ⅷ-3-2 Main Dimension of Each Pump Type
(Vertical Mixed Flow Pump)

Items \ Type	3 Pumps	4 Pumps	5 Pumps
1. Pump Sepcifications Design Discharge (m^3/sec)	42.78	42.78	42.78
· Bore (mm)	ϕ 2,600	ϕ 2,200	ϕ 2,000
· Capacity (m^3/sec)	14.26=856 m^3/min	10.70=642 m^3/min	8.56= 514 m^3/min
Total Head (m)	8.6	8.6	8.8
Pump Speed (rpm)	184	225	245
Motor Power (kw/Unit) ★-1	1,600	1,220	1,010
Total Motor Power(kw)	4,800	4,880	5,050
Pump Efficiency (%)	87	86	85
2. Initial Cost			
Pump Facilities Cost	High	Low	Medium
Building and Civil Work	Low	Medium	High
3. Running Cost	Medium	Low	High
4. Working Ratio When One Pump Is In Trouble (risk diversification)	Low (67%)	Medium (75%)	High (80%)
5. Building Space (m)	22 × 40	20 × 45	19 × 46
6. Required Irrigation Water Match Operation	Low	Medium	Good
7. General Evaluation Rank	3	◎ 1	2

★-1

$$P = \frac{K \cdot r \cdot Q \cdot H}{np \cdot ng} (1 + R) \text{ kw}$$

where ;
P ; Pump Power (kw)
K ; 0.163
r ; Specific Water Gravity 1.00
Q ; Pump Discharge (m^3/min)
H ; Total Pump Head (m)
np ; Pump Efficiency
ng ; Motor Transmission Efficiency 0.95
R ; Pump Surplus Factor 0.10

Table Ⅷ-3-3 Pump Discharge and Bore

Pump Bore	Pump Discharge	Remarks
φ 1500	255 ~ 325 m^3/min	Pump type; vertical mixed flow pump
φ 1650	325 ~ 400	
φ 1800	400 ~ 480	
φ 2000	480 ~ 600	
φ 2200	600 ~ 740 ★	
φ 2400	740 ~ 850	
φ 2600	850 ~ 1,000	
φ 2800	1,000 ~ 1,150	

Source; Design Criteria for Pump Station, Ministry of Agriculture, Forestry and Fishery, Japan

Three pumping station types were comparatively studied and summarized in Table- Ⅷ-3-4～5.

Regarding the general lay-out, see drawings in Fig- Ⅷ-3.3～4.

The results show that the installation of four pumps is considered to be the most suitable. The characteristic curve of φ 2,200mm vertical mixed flow pump is shown in Fig.-Ⅷ-3-5.

Table Ⅷ-3-4 Cost Comparision For Each Vertical Mixed Flow Type

(Unit : $\times 10^3$ TK)

Type / Item	V. Mixed Flow P ϕ 2,600mm × 3NOS	V. Mixed Flow P ϕ 2,200mm × 4NOS	V. Mixed Flow P ϕ 2,000mm × 5NOS
1. *Initial Cost*			
(1) Pump Facilities Cost	708,000	656,000	673,000
(2) Pump House	26,400	27,000	26,220
(3) Civil Work	81,060	83,580	85,050
Total Cost (1)+(2)+(3)	815,460 × 0.1588	766,580 ×0.1588	784,270 × 0.1588
2. Aunual Cost	=129,495	=121,732	=124,542
3. Annual O & M Cost ★1	50,090	49,744	51,253
Total Annual Cost (2+3)	179,585× 10^3TK (105%)	171,476× 10^3TK (100%)	175,795× 10^3TK (103%)

★1 Refer to Table Ⅷ-3-5.

Table Ⅷ-3-5 Annual O & M Cost for Each Vertical Mixed Flow Type Pump

(Unit; $\times 10^3$TK)

Item \ Type	V. Mixed Flow Pump ϕ2,600mm × 3 NOS	V. Mixed Flow Pump ϕ2,200mm × 4 NOS	V. Mixed Flow Pump ϕ2,000mm × 5 NOS
1. Civil Work	10persons ×20days×12M ×50TK = 120	10persons ×20days×12M ×50TK = 120	10persons ×20days×12M ×50TK = 120
2. Maintenance Work for Pump Facilities (2% of Pump Facilities Cost and excluding Tax)	468,000 $\times 10^3$ ×0.02 = 9,360	417,000 $\times 10^3$ ×0.02 = 8,340	421,000 $\times 10^3$ ×0.02 = 8,420
3. Electricity			
- Service Charge	350TK/M ×12M = 4	350TK/M ×12M = 4	350TK/M ×12M = 4
- Basic Charge	40TK/kw/M ×12M ×4,800kw★1 = 2,304	40TK/kw/M ×12M ×4,880kw★1 = 2,342	40TK/kw/M ×12M ×5,050kw★1 = 2,424
- Utility Charge	4.25TK/KWH×0.25×3.286hr★2 ×4,800kw +1.8TK/KWH ×0.75×3.286hr ×4,800kw =38,050	4.25TK/KWH×0.25×3.286hr★2 ×4,880kw +1.8TK/KWH ×0.75×3.286hr ×4,880kw =38,686	4.25TK/KWH×0.25×3.286hr★2 ×5,050kw +1.8TK/KWH ×0.75×3.286hr ×5,050kw =40,033
4. Dredging (Man Power and Sub-Sand Pump ϕ150mm × 11kw ×2NOS)	8,000m^3/year×29TK/m^3 = 232 15d ×24hr×11kw×2 ×2.5TK/KWH =20 = 252	8,000m^3/year×29TK/m^3 = 232 15d ×24hr×11kw×2 ×2.5TK/KWH =20 = 252	8,000m^3/year×29TK/m^3 = 232 15d ×24hr×11kw×2 ×2.5TK/KWH =20 = 252
Total Annual O & M Cost	50,090$\times 10^3$TK (101%)	49,744$\times 10^3$TK (100%)	51,253$\times 10^3$TK (103%)

★1: Total Pump Power (kw)

★2: Annual Operation Hours (Average Year) T(hr) ⟶ Average Year = 1988

T =V/Qp★3×3,600sec=506,000 $\times 10^3$ m^3/42.78m^3/sec ×3,600sec=3,286 hr/year

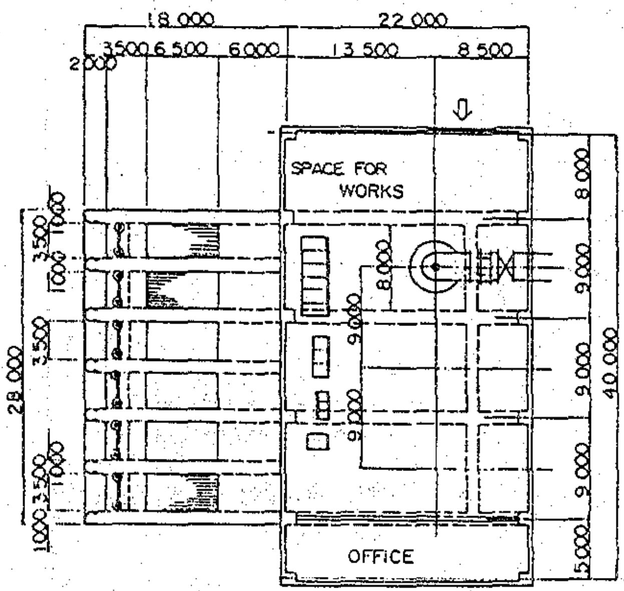
★3: Refer to Table- Ⅷ-1-3

V ; Annual Total Water Intake Volume=506 Million m^3/year

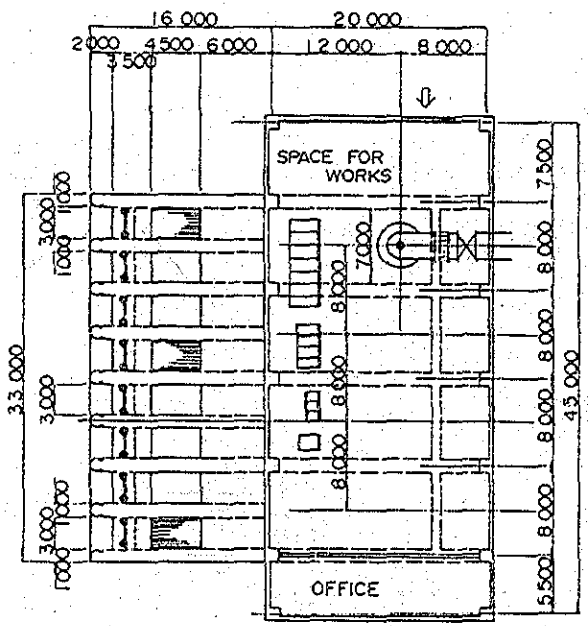
Qp ; Total Pump Capacity =42.78 m^3/sec

Fig. Ⅷ-3-3 Plan of Each Vertical Flow Pump Type

Plan with 3 Pumps
(Case-1)



Plan with 4 Pumps
(Case-2)



Plan with 5 Pumps
(Case-3)

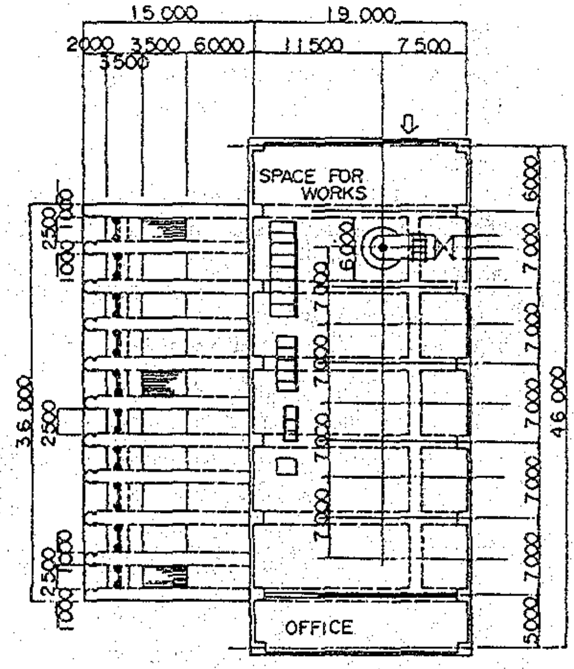
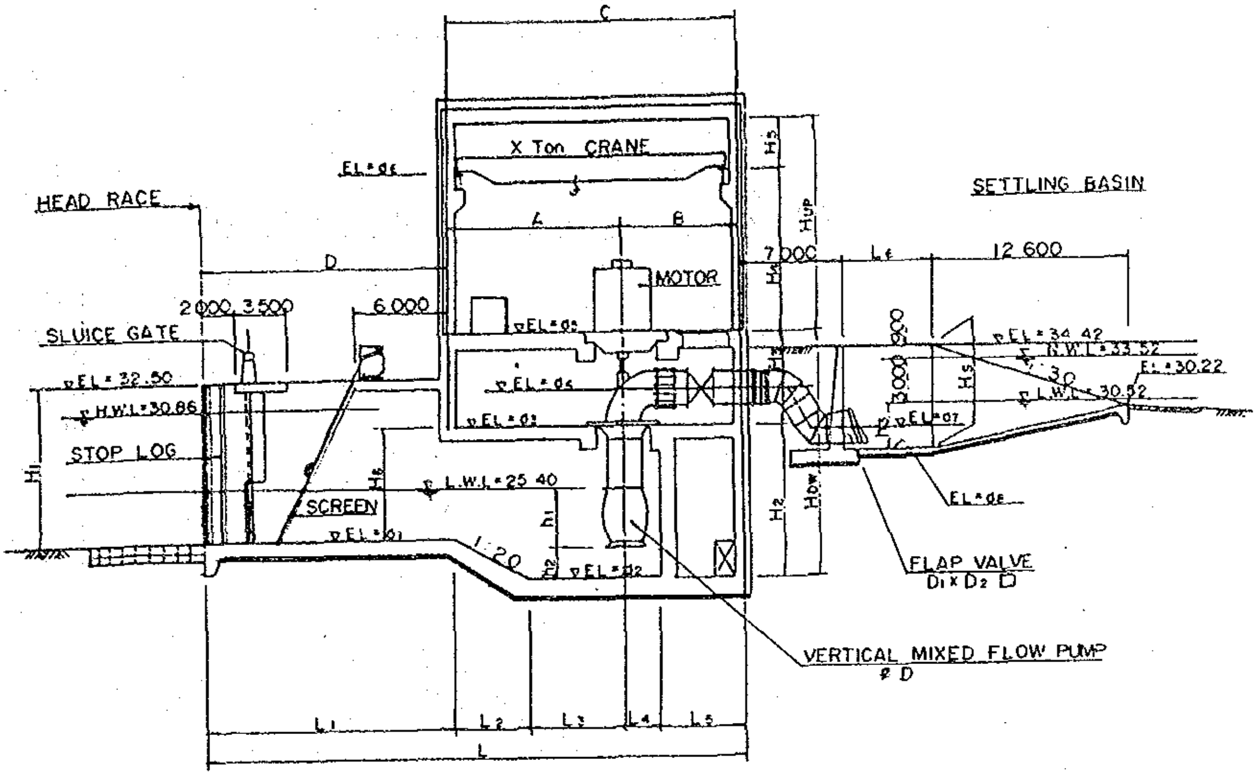


Fig. Ⅷ-3-4 Profile of Each Vertical Flow Pump Station Type



DIMENSION TABLE OF PUMP STATION

(Unit; Meter)

		A	B	C	D	Elevation (P.W.D m)								Water Depth (m)			Hs
						d_1	d_2	d_3	d_4	d_5	d_6	d_7	d_8	h_1	h_2	h_3	
Case-1	φ2,600mm × 3 NOS	13.5	8.5	22.0	18.0	21.50	18.60	28.90	31.60	35.90	48.40	28.92	27.32	4.2	2.6	3.2	7.1
Case-2	φ2,200mm × 4 NOS	12.0	8.0	20.0	16.0	22.00	19.60	29.50	31.90	35.50	47.00	29.12	27.72	3.6	2.2	2.8	6.7
Case-2	φ2,000mm × 5 NOS	11.5	7.5	19.0	15.0	22.40	20.10	29.80	32.00	35.10	46.60	29.22	27.92	3.1	2.0	2.6	6.5

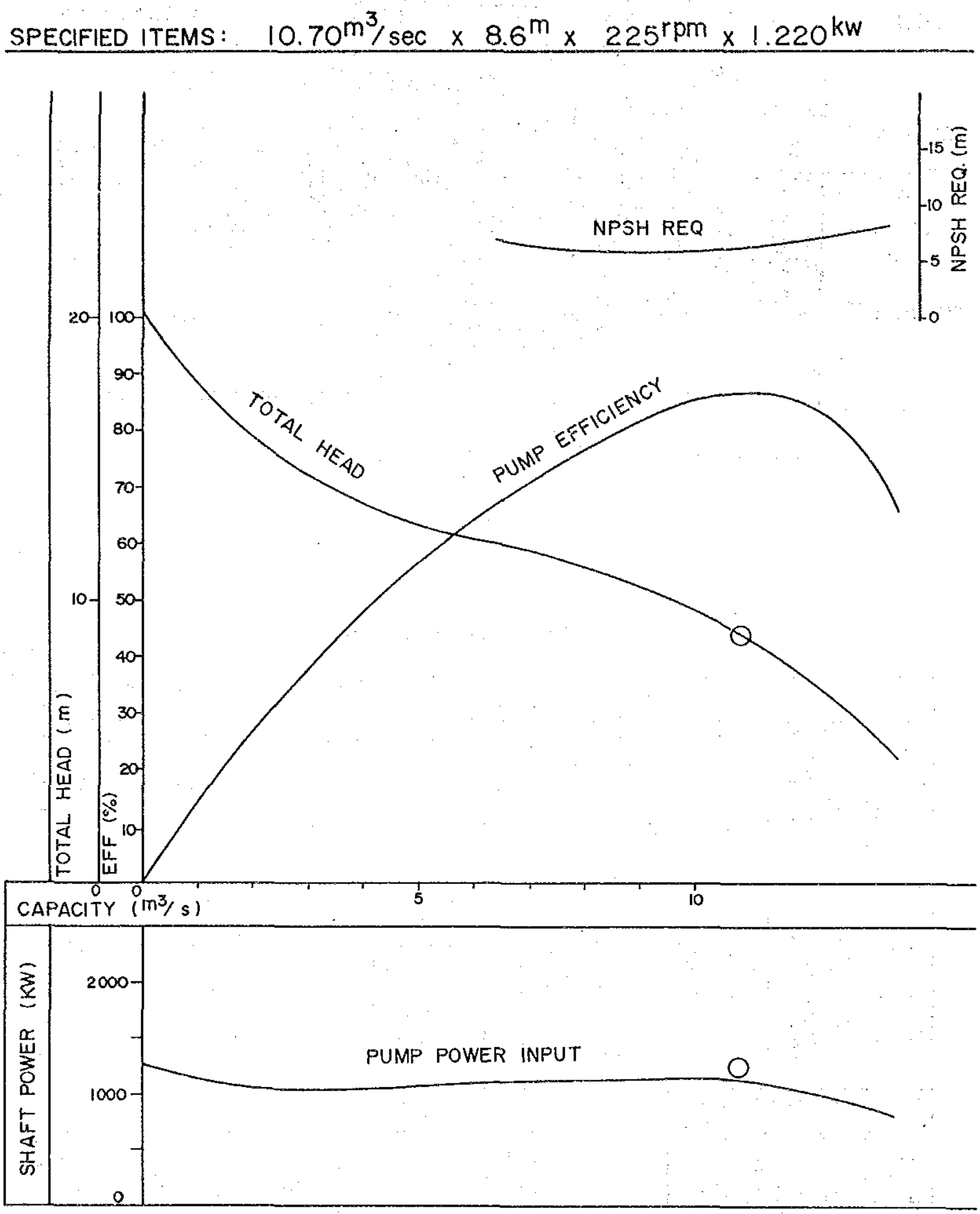
		L_1	L_2	L_3	L_4	L_5	L	L_6	H_1	H_2	H_3	H_4	H_5	H_6	How	Hup	FLAP VALVE
Case-1	φ2,600mm × 3 NOS	17.4	5.3	7.8	2.8	5.7	39.5	7.0	11.0	10.3	7.0	12.5	4.0	7.4	17.3	16.5	2.6 ×3.7
Case-2	φ2,200mm × 4 NOS	16.1	4.8	6.6	2.4	5.6	35.5	6.0	10.5	9.9	6.0	11.5	3.0	7.5	15.9	14.5	2.2 ×3.1
Case-2	φ2,000mm × 5 NOS	15.4	4.6	6.0	2.2	5.3	33.5	5.0	10.1	9.7	5.3	11.5	3.0	7.4	15.0	14.5	2.0 ×2.8

	Crane ×Ton	Sluice Gate W ×H ×0 NOS
Case-1	80	3.5m×5.0m× 6 NOS
Case-2	50	3.0m×4.0m× 8 NOS
Case-3	50	2.5m×4.0m× 10 NOS

Item \ Case	1	2	3
Type	Vertical Mixed Flow Pump	Vertical Mixed Flow Pump	Vertical Mixed Flow Pump
Bore	φ2,600mm	φ2,200mm	φ2,000mm
Capacity	14.26m³/sec =856m³/min	10.70m³/sec =642m³/min	8.56m³/sec =514m³/min
Total Pump Head	8.6m	8.6m	8.8m
Pump Speed	184rpm	225rpm	245rpm
Motor Power	1,600kw	1,220kw	1,010kw
Units	3 NOS	4 NOS	5 NOS
Total Motor Power	4,800kw	4,880kw	5,050kw
Pump Efficiency	87%	86%	85%

Fig. Ⅷ-3-5 Characteristic Curve of Vertical Mixed Flow Pump

(ϕ2,200mm)



3-3. Pump Station Layout

Based on the "Design Criteria for Pump Stations, Ministry of Agriculture, Forestry and Fishery, Japan" the dimensions of the suction sump and the space for the pump setting were obtained and thus the layout for the pump station is shown in Fig.Ⅷ-3-6.

3-4. Outlet Pond and Settling Basin

The outlet pond is to be smoothly connected with the settling basin directly, so that the velocity in the outlet pond is kept within 0.5 m/sec and the average velocity in the settling basin falls between 0.10 and 0.20 m/sec to assure a good settling effect.

(1) Settling Basin Scale

The scale of the settling basin is calculated with the following formula.

$$L = K \frac{Q}{B \cdot Kg}$$

Where : L ; length of the settling basin (m)

B ; width of - do -

K ; safety factor 2.0

Q ; discharge 42.78 ㎥/sec

Vg ; settling velocity with minimum size sediment (mm)

Vg = 0.002 m/sec (Grain size d = 0.02 mm)

$$L = 2.0 \frac{42.78}{120 \times 0.002} \fallingdotseq 360 \text{ m}$$

The area for the settling basin (A) is obtained below:

$$A = B \times L = 120 \text{ m} \times 360 \text{ m} = 43,200 \text{ m}^2$$

(2) The Velocity for Each Section of the Outlet Pond and the Settling Basin

The velocity for each section of the outlet pond and settling basin is given as follows:

$$V = \frac{Q}{B \cdot H} \quad (m/sec)$$

Section	Discharge (Q) m³/s	Width (B) m	Water depth (H) m	Velocity (V) m/s
Outlet Pond	42.78	31.0	5.80	0.24
Settling Basin	- do -	51.0	3.30	0.25
Settling Basin	- do -	120.0	4.02	0.09

Fig. VIII-3-6(a) Profile of Pateswari Pumping Station

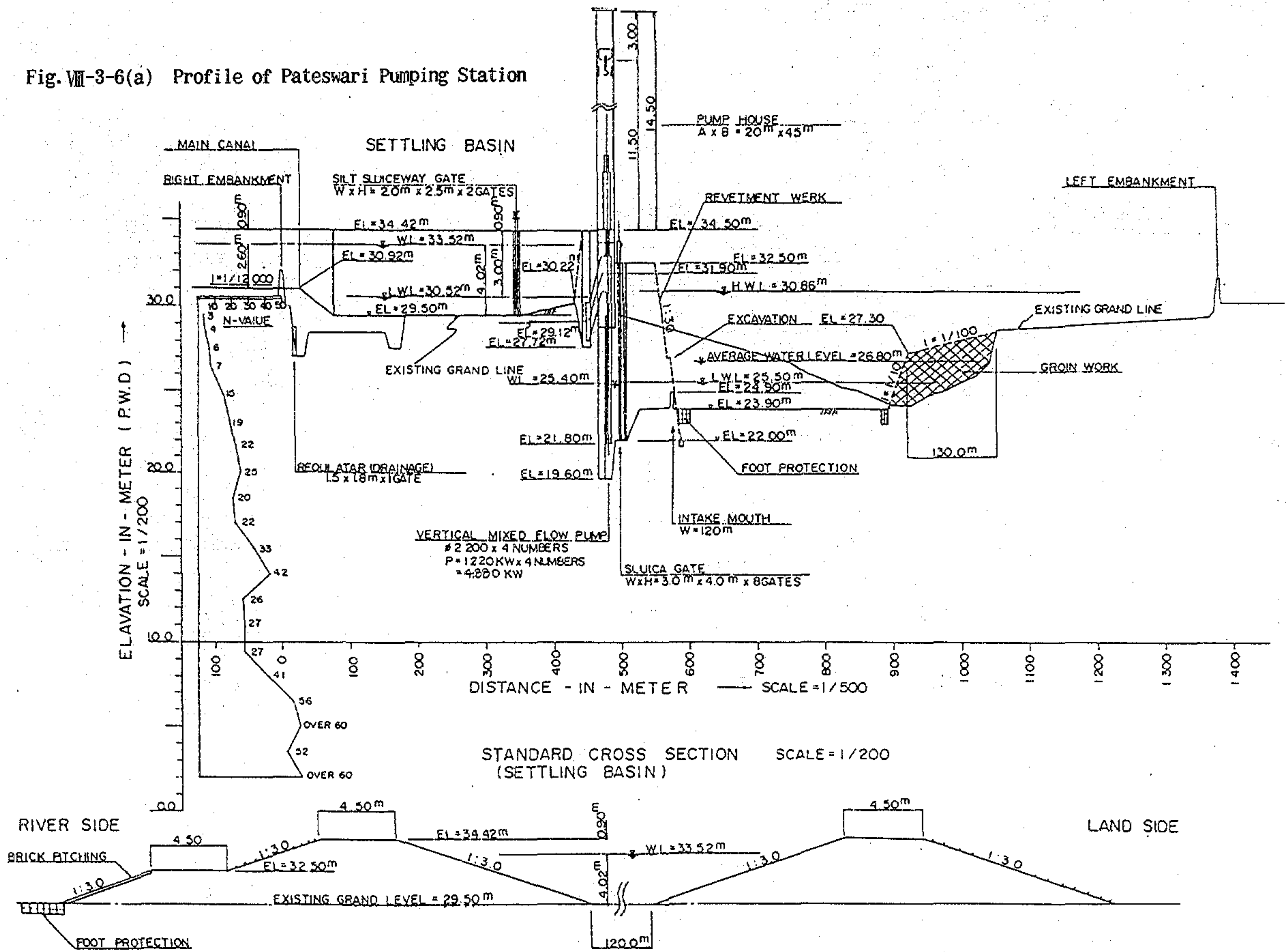
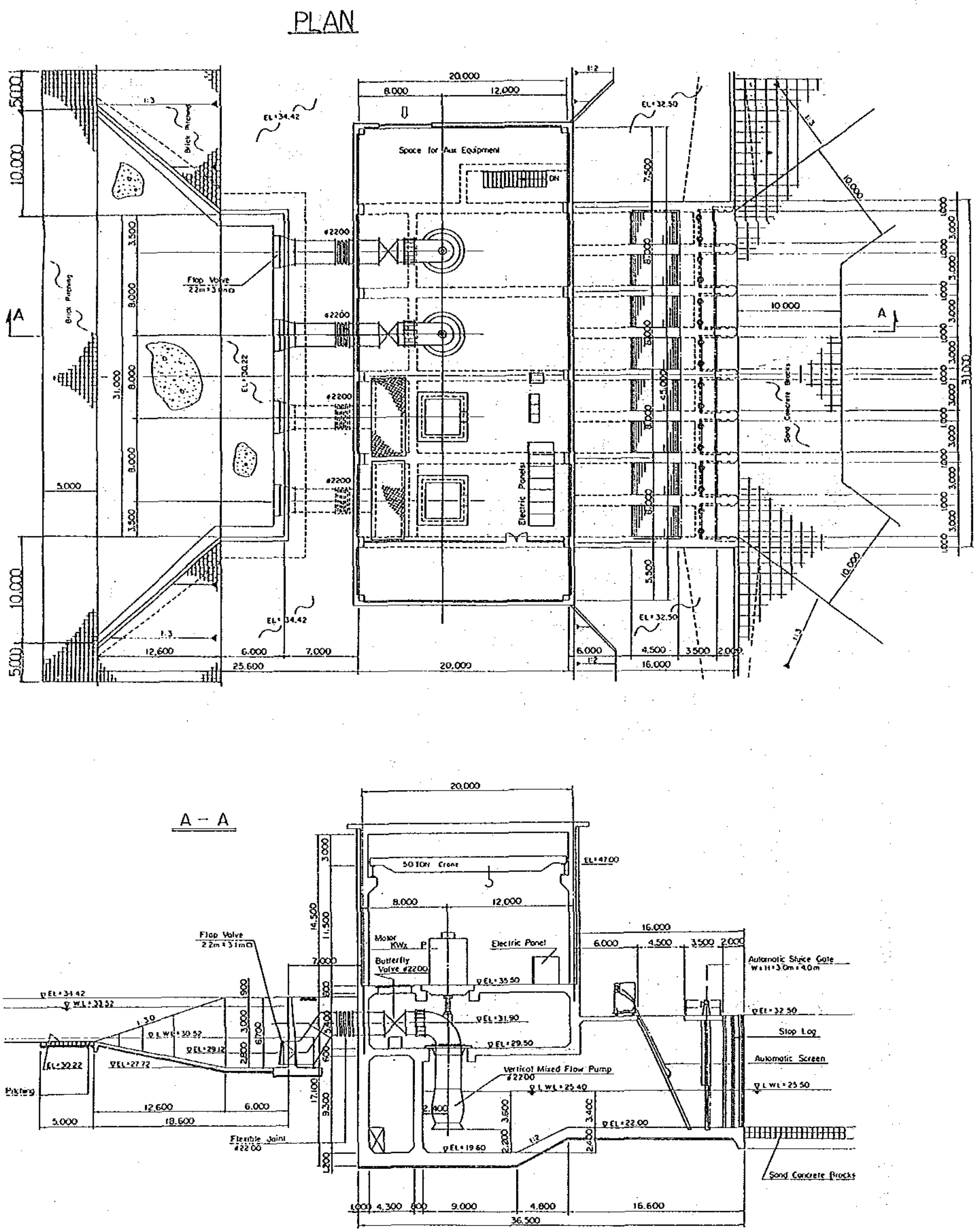
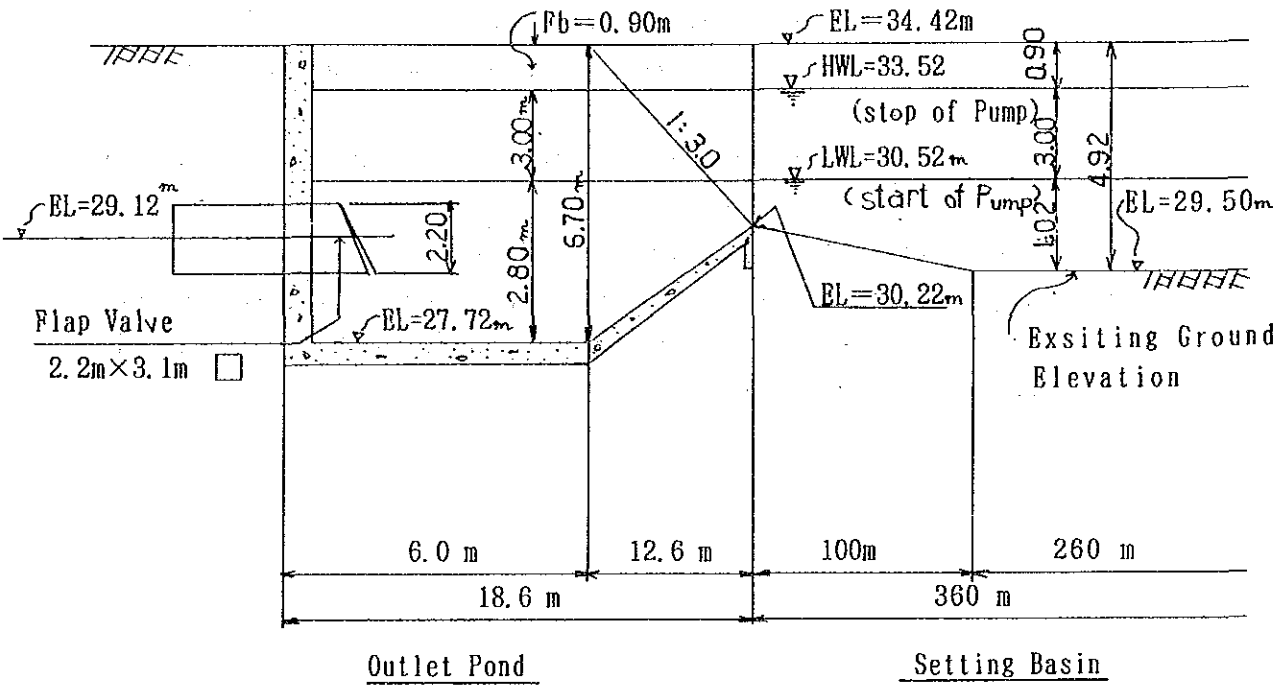


Fig. VIII-3-6(b) Layout of Pateswari Pumping Station



(3) Pumping Working Hours Ability of the Settling Basin's Volume

Considering the ON-OFF hours of the pump, the Water depth of the settling basin is planned as follows.



Settling Basin Volume $V = A \times H = 43{,}200\ m^2 \times 3.0\ m$

$= 129{,}600\ m^3$

Pump Working Hours Ability (T)

$$T = \frac{V}{Qp} = \frac{129{,}600}{42.78\ m^3/sec \times 60\ sec} = 50\ min$$

↓

Water Depth H = 3.0 m

3-5. Power Supply

The existing transmission lines to be used are 11 KV standard. In order to supply power to the proposed pump station 11 KV transmission lines for the new construction site are required and should be connected with the most adjacent point of the existing line. The length required for this new construction transmission line is about 5.5 km.

The electrical facilities required for the connections are as follows.

Incoming Board——Connection Board——Transformer Board ——Transformer

- Standard Main Motor : 3.3KV
- Transformer Capacities : 7,000~8,000KVA

A worry is the possibility of a voltage drop on the existing transmission line when the pump is on. Considering this worry, the required electric power must certainly be secured.

4. Reversible Pumping Station at the Existing Tangoumari Regulator Site.

4-1. River Bank Shifting Conditions

River Bank shifting conditions at the proposed reversible pump station site were analysed based on ① a Topographical Map (S=1/15,800, surveyed in 1965), ② Aerial Photographs (S=1/50,000, surveyed in 1975 and 1983) and ③ a River Cross Section Survey in 1990. The River bank's shifting conditions and the annual minimum water level's movement conditions are shown in Fig- Ⅷ-4-1~4.

The results are shown below :

(1) According to past recorded annual minimum water levels, the river bed's shifting conditions at each point are :

- Dudhkumar River at Pateswari ;
  Tends to gradually rise. (recently the rising value is about 0.3m)
- Brahmaputra River at Noonkawa ;
  Tends to go down on a large scale. (Dropping value is about 2.0m)
- Brahmaputra River at Bahadurabad ;
  Tends to rise gradually. (rising value is about 0.6m)

Accordingly, the river bed's changing conditions have been repetitive thus fluctuating up/down based on many years of observed data.

(2) River Bank Shifting Conditions at the Reversible Pump Station.

The River bank's shifting conditions from the Dudhkumar River confluence point to the proposed reversible pump station site was recorded as being in the moving of 200m ~1,300m length with the cardinal point of the existing embankment.

The River bank's shifting conditions directly near the Dudhkumar River confluence point has been repeatedly causing scouring conditions/accumulated sediment conditions, but the proposed reversible pump station site has presently been forcibly scouring.

The existing embankment is situated at 250 m from the present river bank.

Fig. Ⅷ-4-1 River Bank Shifting Record Near Tangonmari Regulator Site

(Proposed Reversible Pump Station Site)

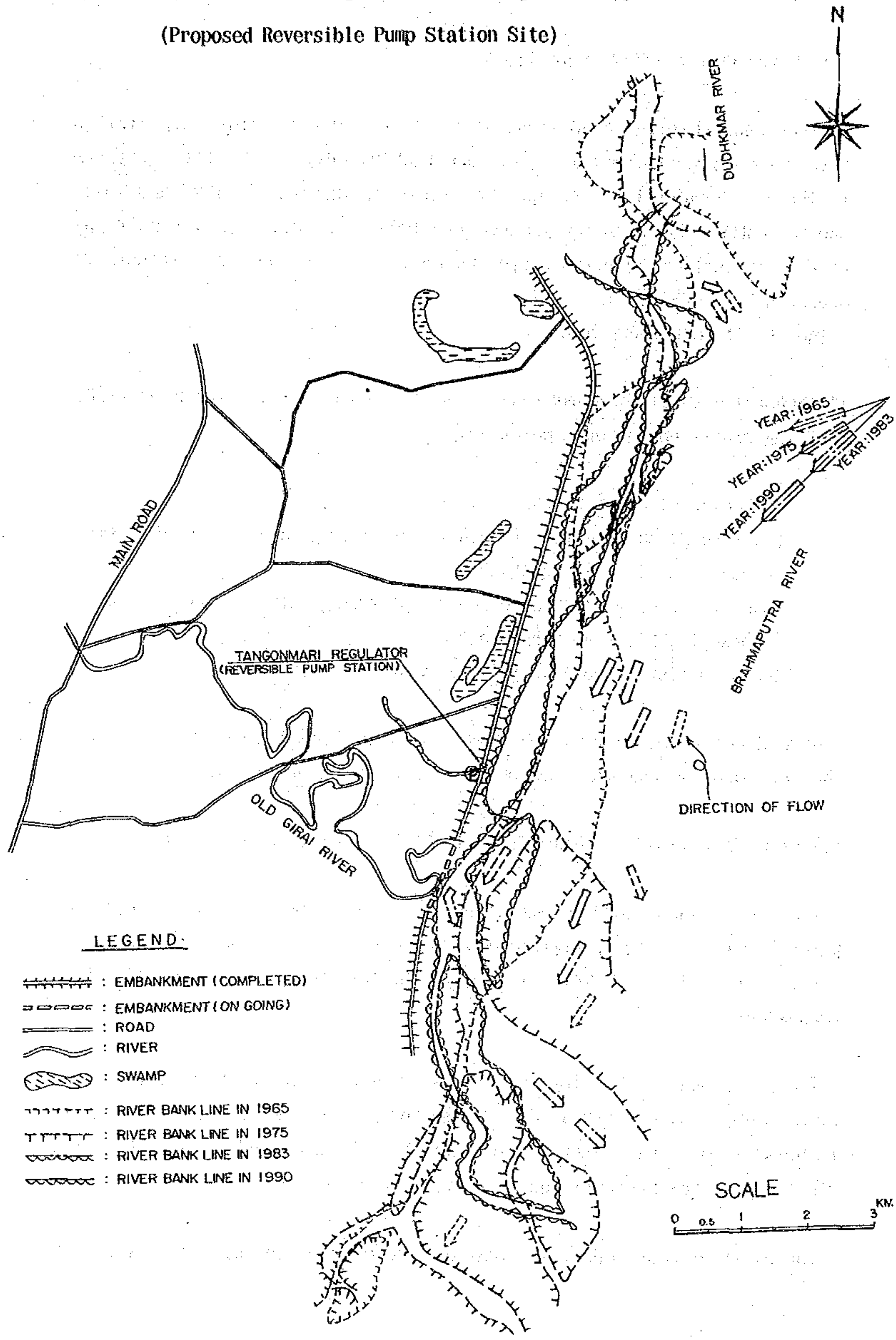


Fig. VIII-4-2 Moving Conditions of Annual Minimum Water Level

(Dudhkumar River at Pateswari and Brahmaputra River at Noonkawa)

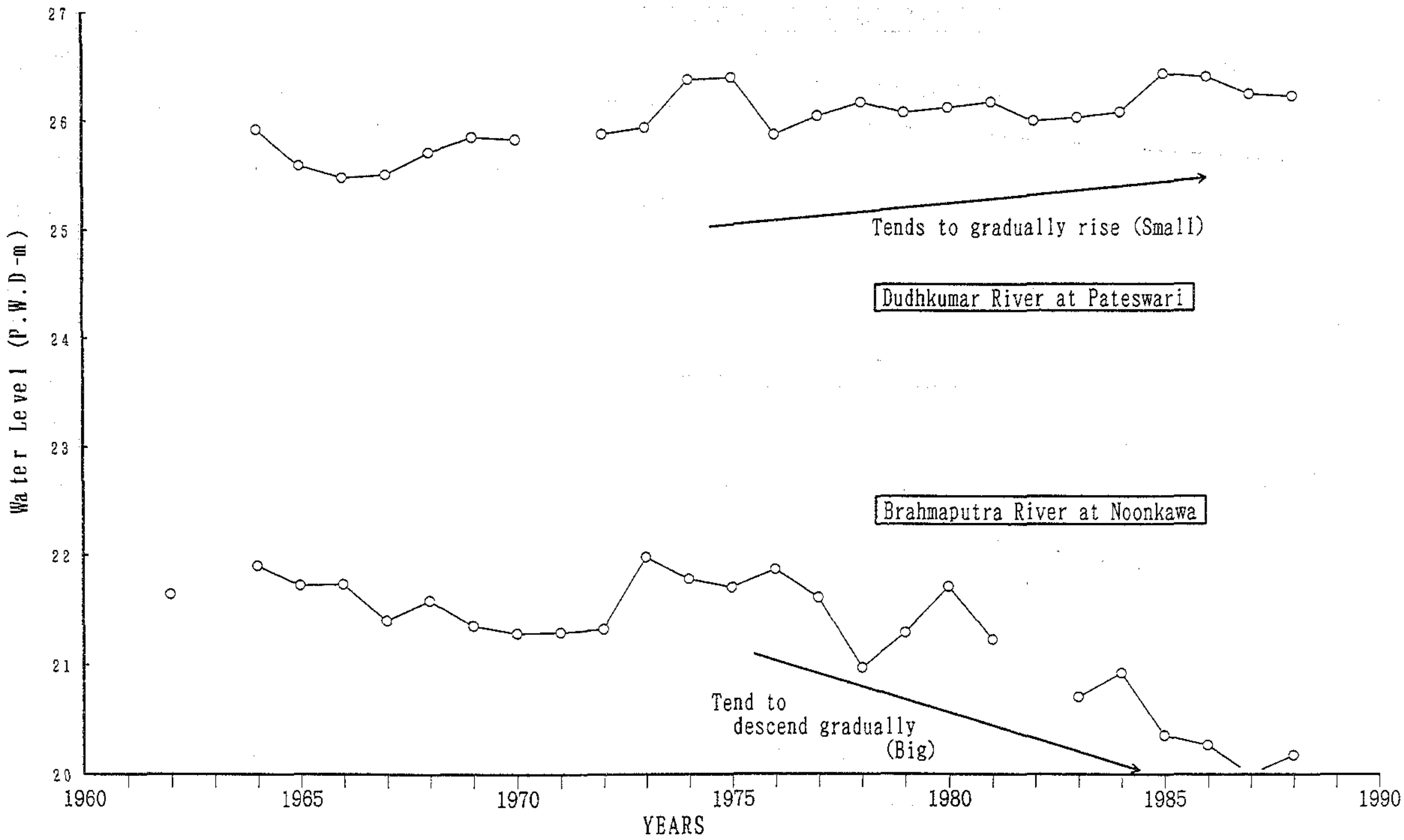


Fig. Ⅷ-4-3 Moving Conditions of Annual Minimum Water Level

(Brahmaputra River at Chilimari and Bahadurabad)

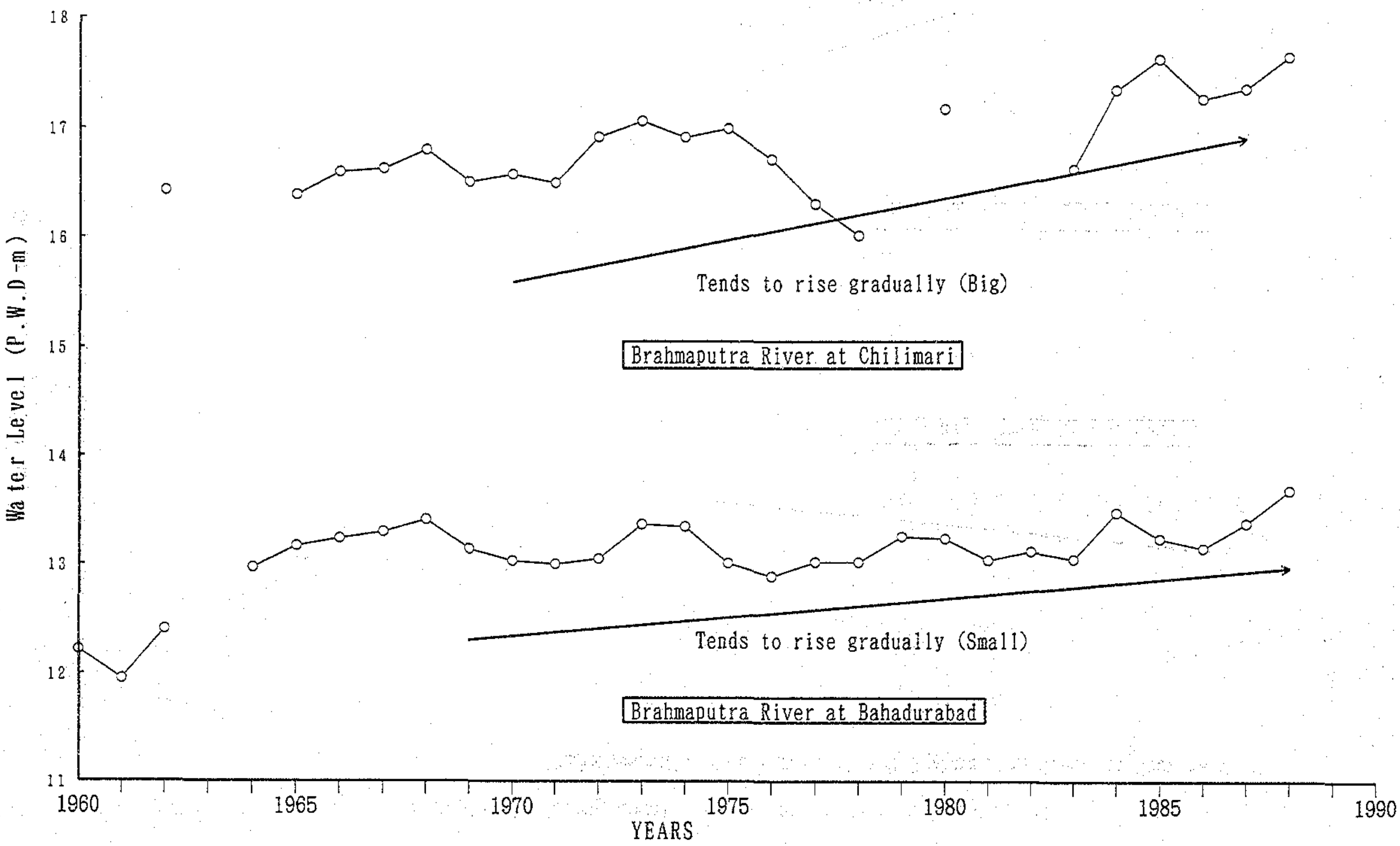
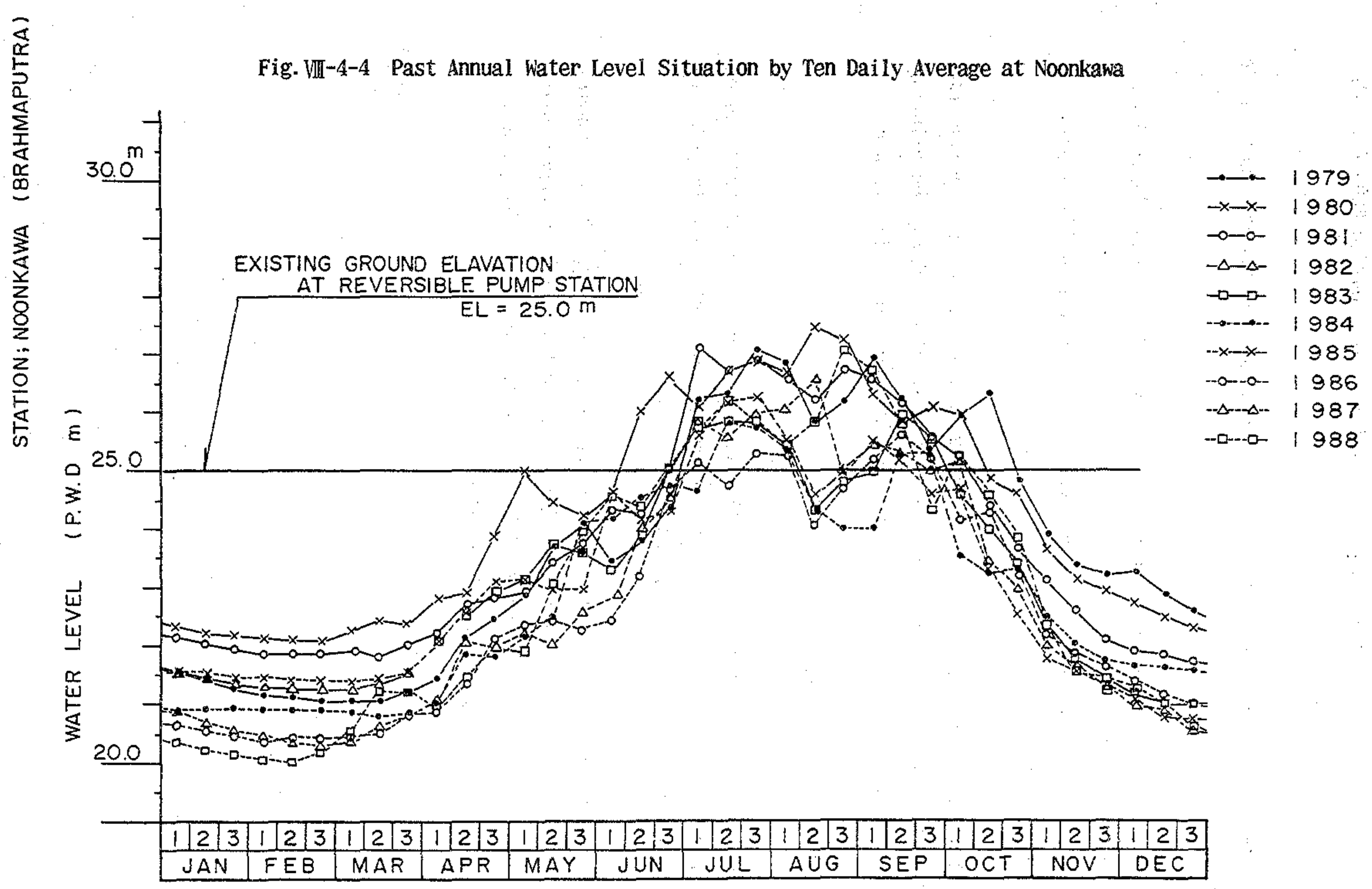


Fig. Ⅷ-4-4 Past Annual Water Level Situation by Ten Daily Average at Noonkawa



Accordingly, for the future, the river's bank movement range at the proposed pump station site is presupposed at being 100m to 600m from the existing embankment.

(3) The water level changes of the past 10 year at Noonkawa are shown in Fig. Ⅷ-4-4.

- Variation range of the water level in the rainy season (Jun ~Oct)
  W.L = 24~27 m (P.W.D)
- Variation range of the water level in the dry season (Nov ~Apr)
  W.L = 20~23 m (P.W.D)

In order to maintain a 25m elevation (P.W.D) at the proposed reversible pump station site, the river's water level is almost above the existing ground level at the planned pumping site throughout the rainy seasons.

(4) Intake Water Problems

Treatment for accumulated sedimentation in the head race needs to be carried out at the beginning of the dry season every year in order to secure the designed discharge. (Q=4.87 m^3/sec)

The proposed reversible pump station determined for the Tangonmari regulator site is considered suitable according to it's topographic conditions and is to be designed with a smaller intake discharge, and is prone to the same problems as the above.

4-2. River Bed shifting conditions

The river bed's shifting conditions at the proposed reversible pump station site were analysed based on past recorded water level and discharge data during a 25 year period.

Annual minimum & maximum water level conditions at each observed station and the annual minimum & maximum discharge at Bahadurabad are shown in Table-Ⅷ-4-1~2, Fig- Ⅷ-4-5~8.

(1) Annual Minimum Water Level

a) Variations of the Annual Minimum Water Level.

- Noonkawa Station

Year ; 1962~1980 →L.W.L ; ranges of 21 ~22m (P.W.D)
Year ; 1981~1988 →L.W.L ; ranges of 20 ~21m (P.W.D)

At the Noonkawa station point large degradations in the river bed have been occuring by about 2.0 m during the last 10 years.

- Brahadurabed station

Year ; 1962~1980 →L.W.L ; similar to the Noonkawa station concerning movement within a 5 year cycle.

On the other hand, the Chilimari station point has been raising with large gradations in the river bed of approximatly 1.6 m in the last 10 years as the L.W.L range is 16.0~17.6m (P.W.D).

b) Annual Minimum Water Surface Gradient Movements.

(Section distance is about 35 KM from Noonkawa to Chirimari)

- Year ; 1962~1980 →Minimum Water Surface Gradient :
I = 1/6,600~1/7,800 (Steep)
- Year ; 1981~1988 →Minimum Water Surface Gradient :
I = 1/8,600~1/13,900 (Gentle)

As previously mentioned the minimum water surface gradients vary a great deal during the last ten years.

Accordingly, the results support that the upper reach sides are to be scoured and that the lower reach sides are to be sedimented when the river bed has a gentler gradient.

In other words, considering an equilibrium gradient, it is supposed in one process for it to be returned from a gentle gradient condition to a steep gradient conditions.

(2) Annual Maximum Water Level

a) Annual Maximum Water Level Movement.

- Nookawa station

Year ; 1962~1980 → H.W.L : ranges of 27.3 ~28.1m (P.W.D)
Year ; 1981~1987 → L.W.L : ranges of 26.2 ~26.7m (P.W.D)
} different value : 1.9m

The Annual maximum water levels went down about 1.0 m in the last 10 years, the water Level surpassed the maximum water level during the big flood in 1988.

The Brahadurabad station point level has been fluctuating up/down for the past 10 years cycle and that the variation ranges are smallar than at the Noonkawa point keeping within 1.5 m.

b) Annual Maximum Water Surface Gradient Movements.

(Section Distance is about 35 km from Noonkawa to Chirimari)

Year ; 1962~1980 →Maximum Water Surface Gradient
I = 1/8,100~1/11,700 (Slightly steep)

Year ; 1981~1988 →Maximum Water Surface Gradient
I = 1/11,500 ~1/16,400 (Gentle)

The results show that the river bed has been fluctuating up/down (Steep Gradient→ Gentle Gradient→Steep Gradient).

Table Ⅷ-4-1 Minimum and Maximum Water Levels at Noonkawa and Chilimari

Unit : m

Year	Noonkawa				Chilimari			
	Rank	MIN WL	Rank	MAX WL	Rank	MIN WL	Rank	MAX WL
1962 - 63	17	21.64	2	☆28.07	4	16.43	1	★25.07
1963 - 64	N A		N A		N A		N A	
1964 - 65	24	21.90	9	27.71	N A		N A	
1965 - 66	20	21.72	11	27.59	3	16.39	22	23.26
1966 - 67	21	21.73	15	27.45	8	16.60	17	23.81
1967 - 68	14	21.40	17	27.39	10	16.63	13	23.97
1968 - 69	15	21.58	14	27.53	12	16.80	12	24.04
1969 - 70	13	21.35	12	27.58	6	16.51	18	23.79
1970 - 71	9	21.28	6	27.88	7	16.58	7	24.19
1971 - 72	11	21.29	20	27.11	5	16.49	11	24.08
1972 - 73	17	21.32	4	27.94	13	16.92	10	24.09
1973 - 74	25	21.98	7	27.79	16	17.07	16	23.88
1974 - 75	22	21.78	3	27.99	14	16.92	4	24.46
1975 - 76	18	21.70	13	27.55	15	17.00	19	23.78
1976 - 77	23	21.87	10	27.63	11	16.72	15	23.90
1977 - 78	16	21.61	8	27.77	2	☆16.31	9	24.11
1978 - 79	7	20.97	16	27.41	1	★16.02	20	23.68
1979 - 80	10	21.29	18	27.32	N A		N A	
1980 - 81	19	21.71	5	27.94	17	17.18	6	24.25
1981 - 82	8	21.23	19	27.30	N A		N A	
1982 - 83	N A		N A		N A		N A	
1983 - 84	5	20.70	21	26.71	9	16.63	8	24.12
1984 - 85	6	20.92	24	26.45	19	17.35	5	24.26
1985 - 86	4	20.35	23	26.51	22	17.63	14	23.93
1986 - 87	3	20.26	25	26.20	18	17.27	21	23.44
1987 - 88	1	★19.99	22	26.70	20	17.36	3	24.56
1988 - 89	2	☆20.17	1	★28.10	21	17.65	2	☆25.06

Table Ⅷ-4-2 Maximum Water Level, Maximum Discharge, Minimum Water Level and Minimum Discharge at Bahadurabad.

Year	Rank	MIN W.L (m)	Rank	MAX Q (m^3/sec)	Rank	MIN W.L (m)	Rank	MIX Q (m^3/sec)
1960 - 61	23	19.48	15	64,800	2 ☆	12.22	14	3,790
1961 - 62	28	18.91	26	53,800	1 ★	11.95	21	3,990
1962 - 63	7	19.99	7	69,400	3	12.41	10	3,540
1963 - 64		N A	23	56,400		N A	13	3,770
1964 - 65	11	19.84	18	63,100	5	12.97	3	3,170
1965 - 66	16	19.69	16	64,200	17	13.17	7	3,500
1966 - 67	19	19.62	8	68,900	20	13.24	17	3,950
1967 - 68	21	19.50	6	69,600	22	13.30	11	3,620
1968 - 69	13	19.80	19	62,300	26	13.41	16	3,880
1969 - 70	12	19.84	24	56,000	15	13.14	23	4,020
1970 - 71	25	19.20	4	75,000	10	13.03	1 ★	2,860
1971 - 72	1 ★	20.75		N A	6	13.00		N A
1972 - 73	27	18.98	10	66,600	12	13.05	2 ☆	3,140
1973 - 74	9	19.88	9	67,300	24	13.37	5	3,280
1974 - 75	3	20.26	2 ☆	91,100	23	13.35	4	3,200
1975 - 76	20	19.60	27	52,200	7	13.01	25	4,050
1976 - 77	10	19.87	14	65,600	4	12.89	20	3,990
1977 - 78	6	19.99	11	66,600	8	13.02	27	4,530
1978 - 79	17	19.63	21	56,600	9	13.02	24	4,050
1979 - 80	14	19.78	13	66,100	21	13.26	26	4,330
1980 - 81	5	20.10	20	61,200	19	13.24	22	4,020
1981 - 82	22	19.48	12	66,500	11	13.04	8	3,500
1982 - 83	24	19.42	25	55,900	14	13.12	18	3,980
1983 - 84	8	19.93	22	56,500	13	13.05	9	3,530
1984 - 85	4	20.11	3	77,000	27	13.48	15	3,840
1985 - 86	18	19.62	17	63,800	18	13.24	12	3,660
1986 - 87	26	19.15	28	43,100	16	13.15	6	3,360
1987 - 88	15	19.71	5	74,000	25	13.38	19	3,990
1988 - 89	2 ☆	20.62	1 ★	98,600	28	13.68	28	5,240

Fig. Ⅷ-4-5 Annual Minimum Surface Water Level and Discharge Fluctuation at Noonkawa and Bahadurabad

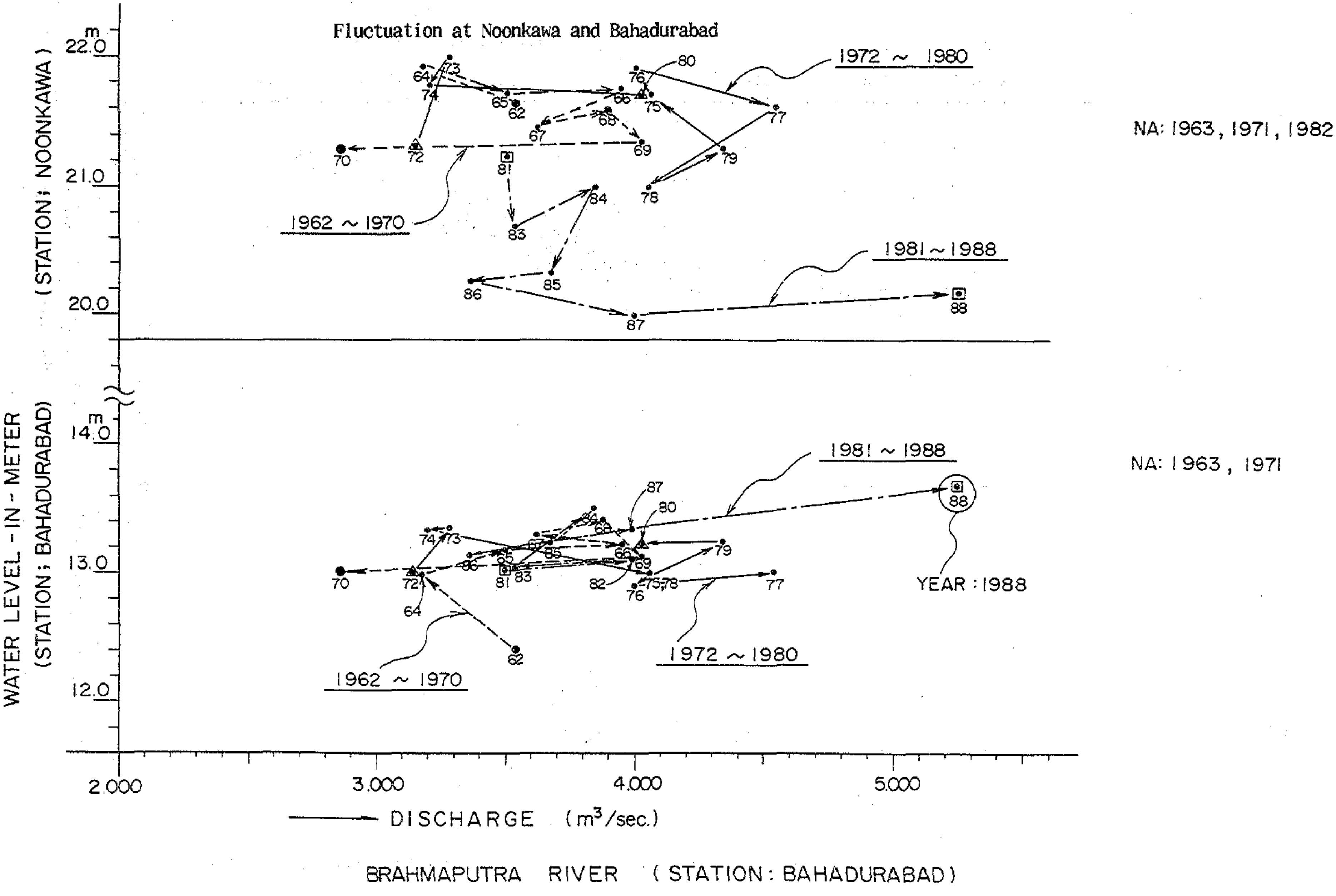
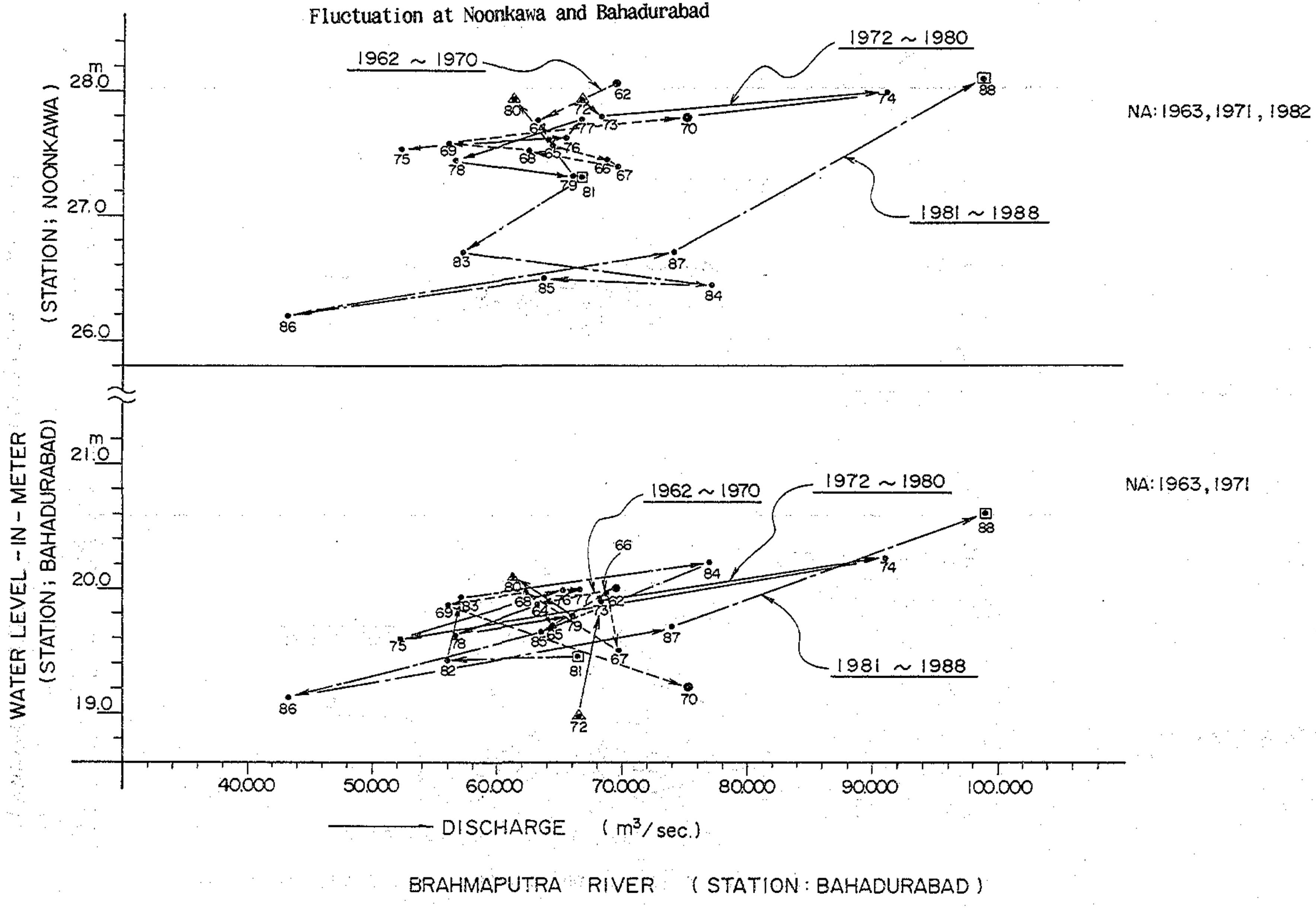


Fig. Ⅷ-4-6 Annual Maximum Surface Water Level and Discharge Fluctuation at Noonkawa and Bahadurabad



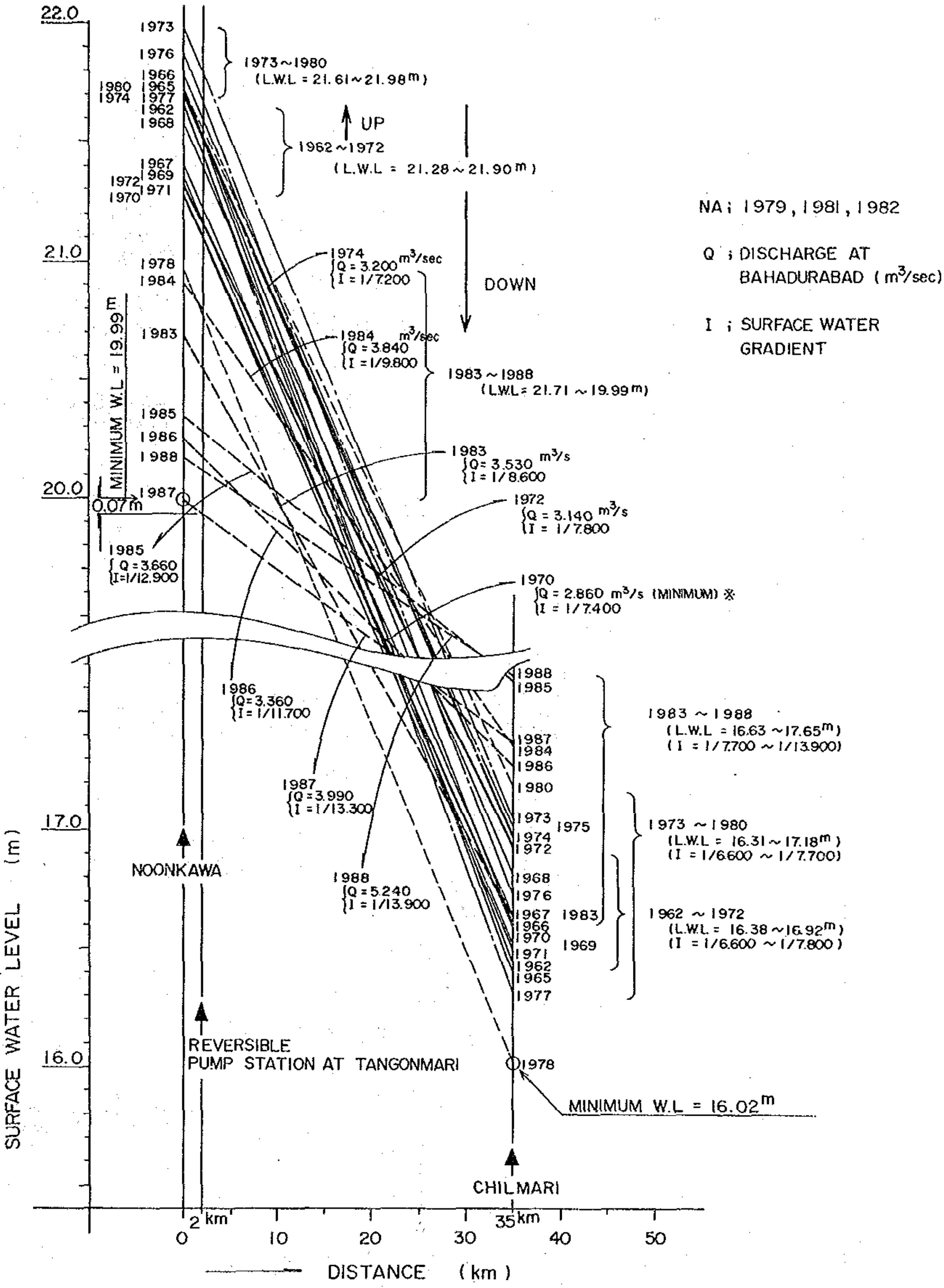


Fig. VIII-4-7 Annual Minimum Water Surface Gradient Movements

(Section: Noonkawa ~ Chilimari)

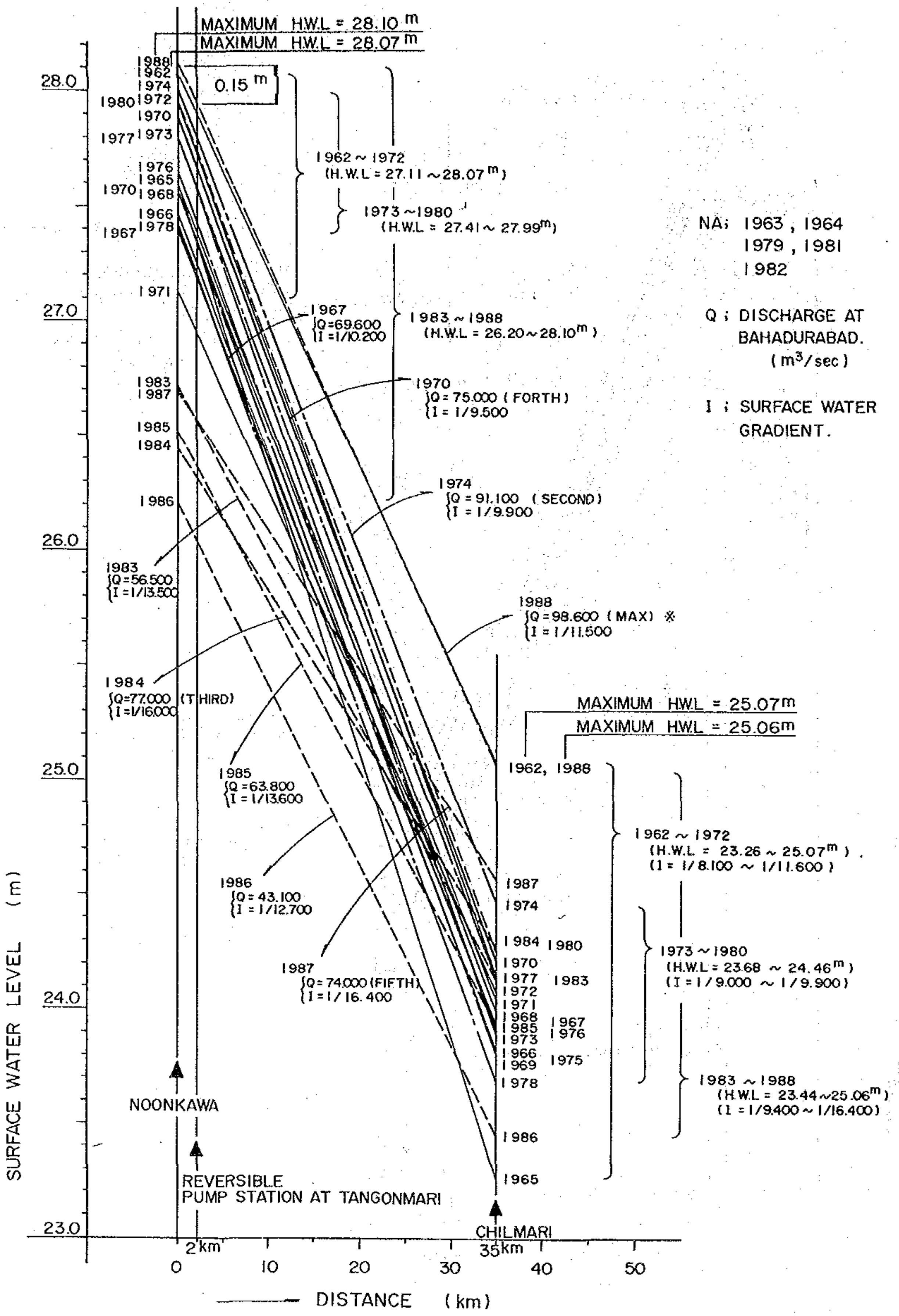


Fig. Ⅷ-4-8 Annual Maximum Water Surface Gradient Movements

(Section: Noonkawa ~ Chilimari)

4-3. Design Low Water Level for Water Intake

The lowest water level during the past 10 years for the intake site is obtained as follows:

- L.W.L. = 19.99*1 - 0.07*2 = 19.92 ⇨ 19.90
  - *1 ; The lowest water level at the Noonkawa station which was recorded in 1987.
  - *2 ; The water surface drop obtained from the river gradient and the 2 km of distance between the Noonkawa station and the pump station site.

Considering the previously mentioned large range of the past low water level fluctuation and the supposed discharge reduction caused by artificial activities in the future, the design low water levels for the proposed pump station are planned as follows:

- For the intake regulator, conduit and the pump suction sump,
  L.W.L = 19.9 - 1.0 *3 = 18.9 m
  - *3 ; A planned drop of the low water level to secure the intake during the durable period of the pump station.
- For head race canal, for which the canal bed can easily be cut down responding to a possible future drop of the water level,
  L.W.L = 19.9 m

4-4. Outline of the Reversible Pump Station

Irrigation water requirements in 1979 (the maximum annual water consumption) and 1988 (the average annual water consumption) are shown in Table-Ⅷ-4-3 and Fig- Ⅷ-4-9~10.

- Pump total discharge :
  - Design discharge for water intake : QI = 4.87 m^3/sec
  - Design discharge for pumping drainage : QD = 5.00 m^3/sec
- Pump number : 3
- Pump type : Vertical mixed flow pump
- Pump bore : ϕ900 mm
- Total pump head : 8 m

- Pump power : 200KW/unit
- Total pump power : 200×3 = 600KW
- Pump efficiency : 81 %

(1) Basic Water Level Condition

Use \ Water Level	Suction Water Level	Outlet Water Level	Actual Head
Irrigation	L.W.L = 19.8 m	W.L = 27.3 m	7.5 m
Drainage	L.W.L = 24.5 m	H.W.L = 28.1 m	3.6 m

(2) Operation Specifications for Irrigation and Drainage

- Irrigation Lifting

  Q H

  98 m^3/min × 8 m × 490 rpm × 200 KW

  (1.63 m^3/sec)

- Drainage Lifting

  Q H

  146 m^3/min × 4.2 m × 140 KW

  (2.43 m^3/sec)

A characteristic curve of a vertical mixed flow pump is shown in Fig- Ⅷ-4-11.

Table Ⅷ-4-3 Annual Gross Irrigation Water Requirements for Tangonmari Pump Station
(Total Irrigable Area ; A = 3,350 ha)

Year		Gross Irri. Req. (peak) (m^3/s)	Gross Irri. Req. (Million m^3)		Remarks
1979	Apr-3	4.87	66.6	1	Plan year
1980	Apr-3	4.40	59.8	3	
1981	Jun-1	3.85	62.5	2	
1982	May-1	3.94	58.9	4	
1983	May-1	3.80	54.6	8	
1984	Feb-1	3.42	51.0	10	
1985	May-1	4.13	54.8	7	
1986	May-3	3.96	56.4	6	
1987	May-2	4.65	52.7	9	
1988	Apr-3	3.60	57.0	5	Average year
average			57.4	Million m^3	

4-5. Reversible Pump Station Operation Systems

The operation systems were designed as shown in Fig.Ⅷ-4-12 to facilitate and make safe the operation and maintenance of the irrigation and drainage system.

Fig. VIII-4-9 Gross Irrigation Water Requirements and Pump Numbers

– Tangonmari Pump Station, Plan Year: 1979 –

(Total Irrigable Area: A = 3,350 ha)

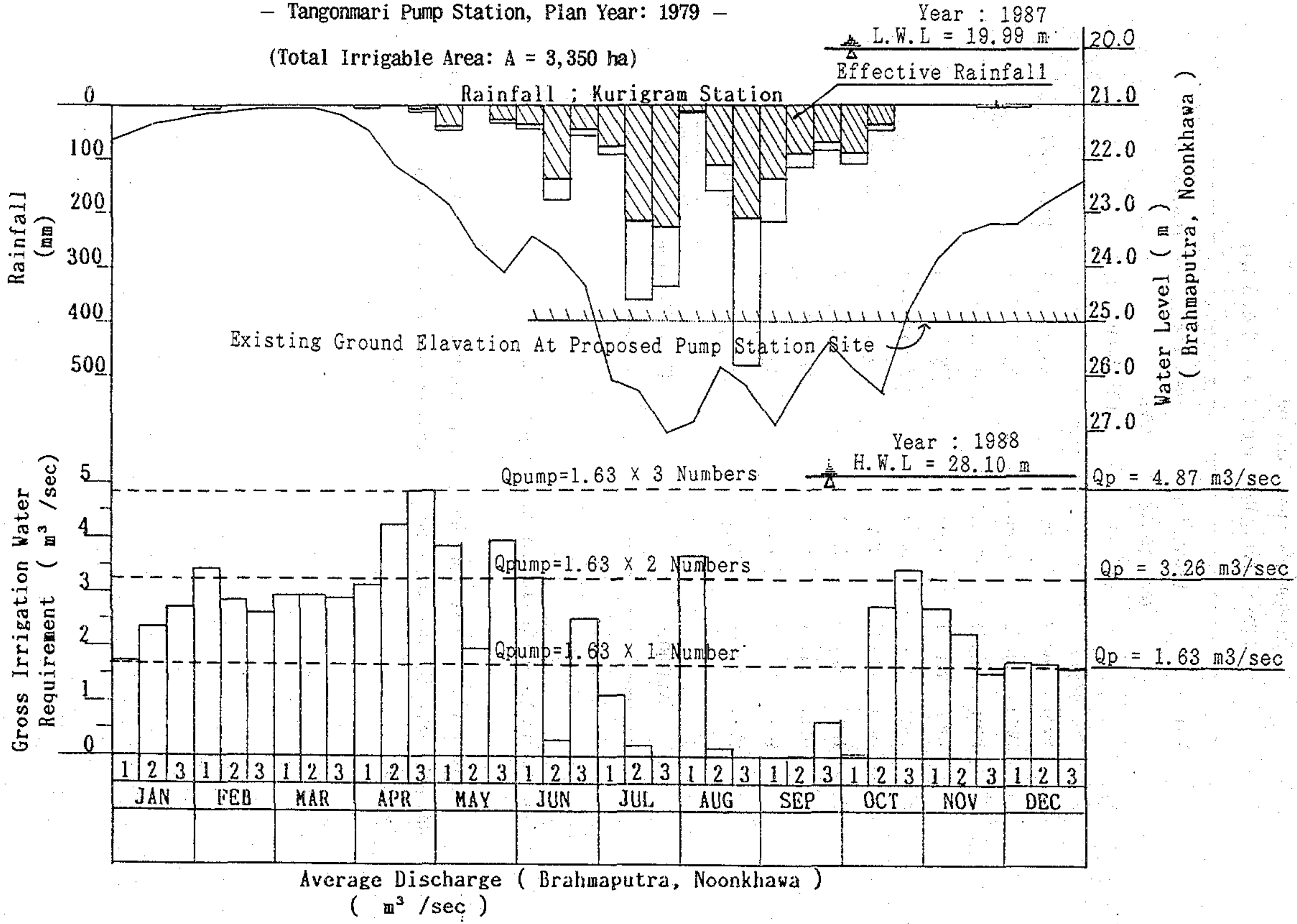


Fig. VIII-4-10 Gross Irrigation Water Requirements

– Tangonmari Pump Station, Average Year: 1988 –

(Total Irrigable Area: A = 3,350 ha)

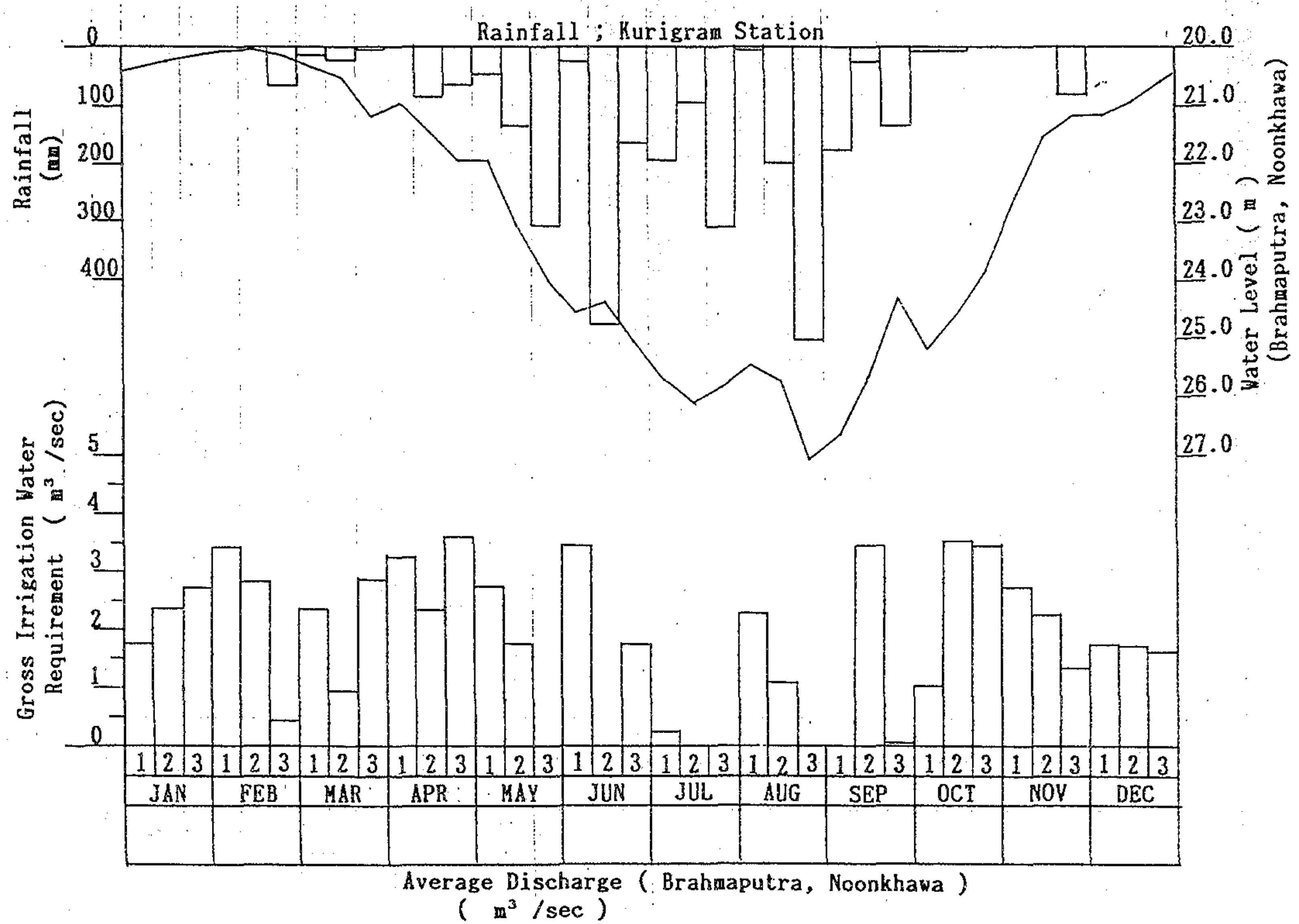


Fig. Ⅷ-4-11 Characteristic Curve of Vertical Mixed Flow Pump

(Reversible Pump Station : ϕ900 mm)

SPECIFIED ITEMS: $1.63^{m^3}/sec = 98^{m^3}/min \times 8.0^{m} \times 490^{rpm} \times 200^{kw}$

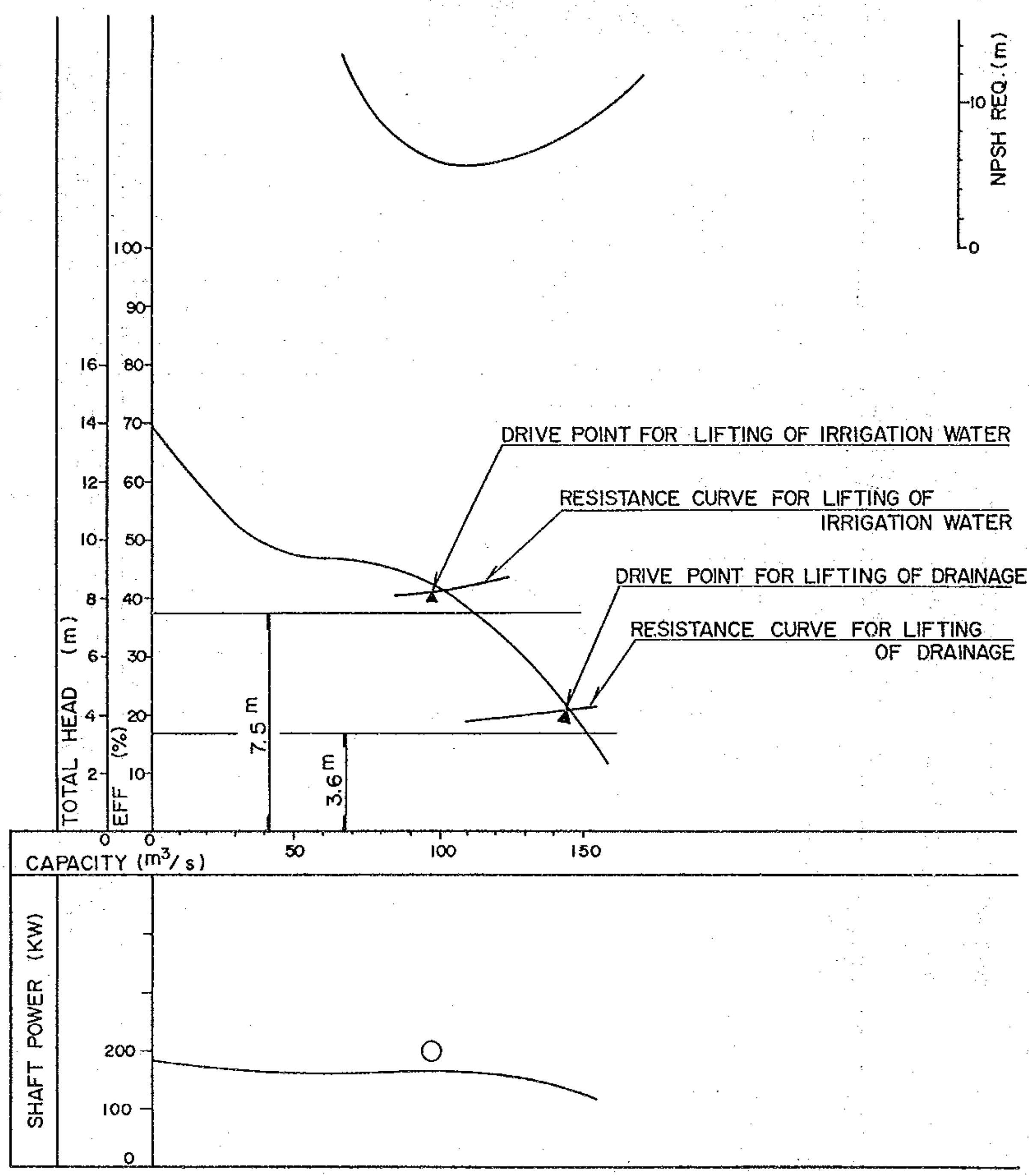
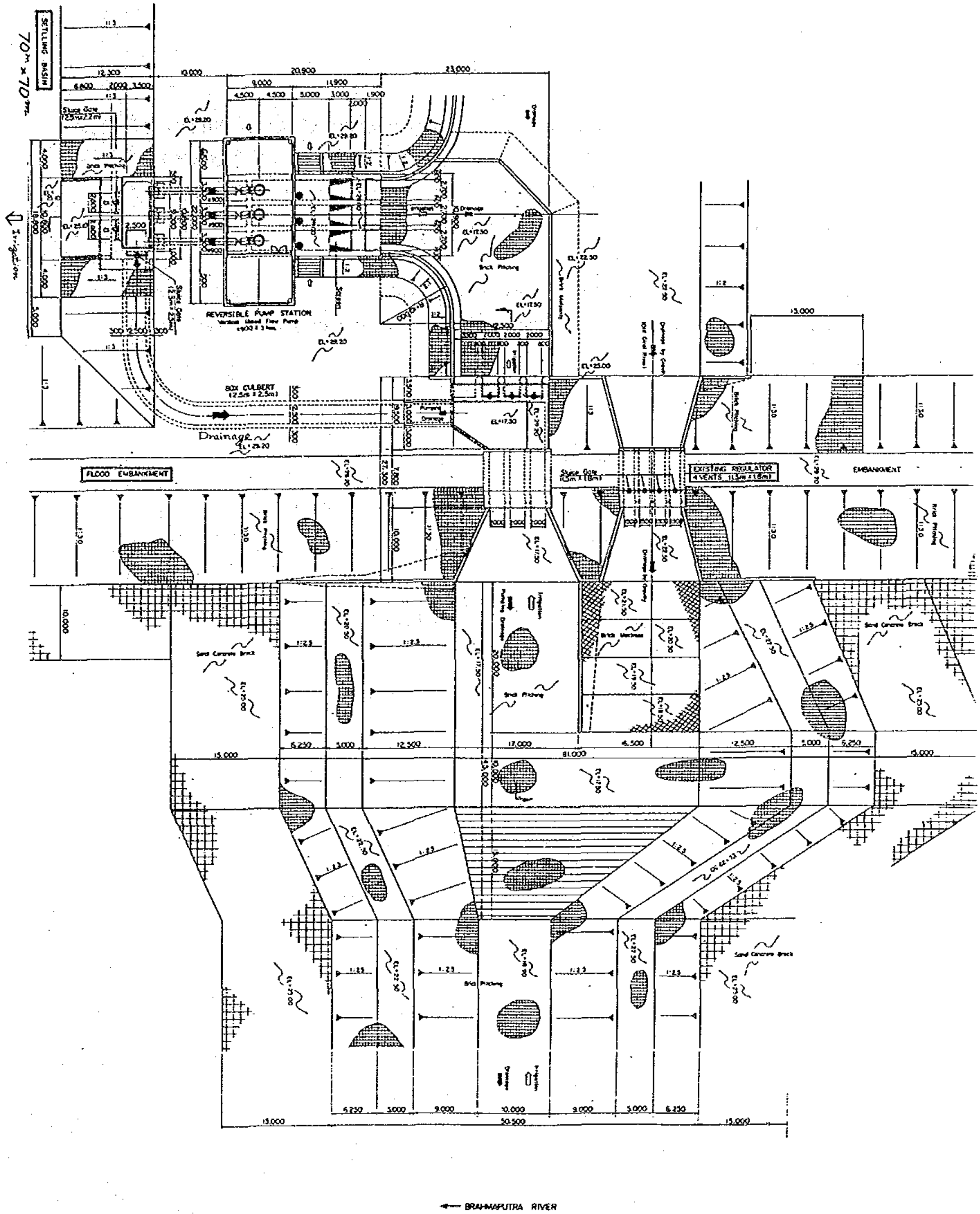


Fig. Ⅷ-4-12 Reversible Pump Station Operation Systems



← BRAHMAPUTRA RIVER

4-6. Design of Reversible Pumping Station

(1) Basic Conditions

Design Discharge :

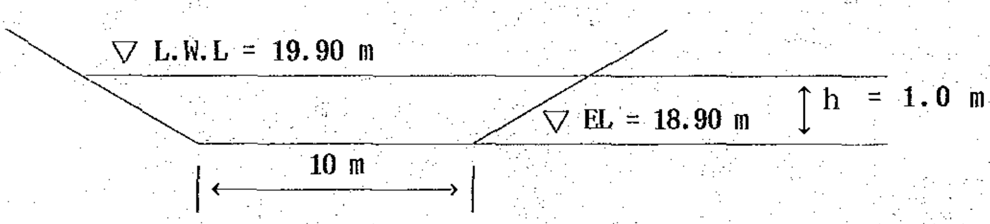
Q1 = 4.87 m^3/sec (for Irrigation)

☆1 QD = 7.30 m^3/sec (for Drainage)

☆1 : To be planned by the pump's characteristic curve, if available. (QD = 14.6m^3/min×3 NOS = 4.38m^3/min = 7.3m^3/sec)

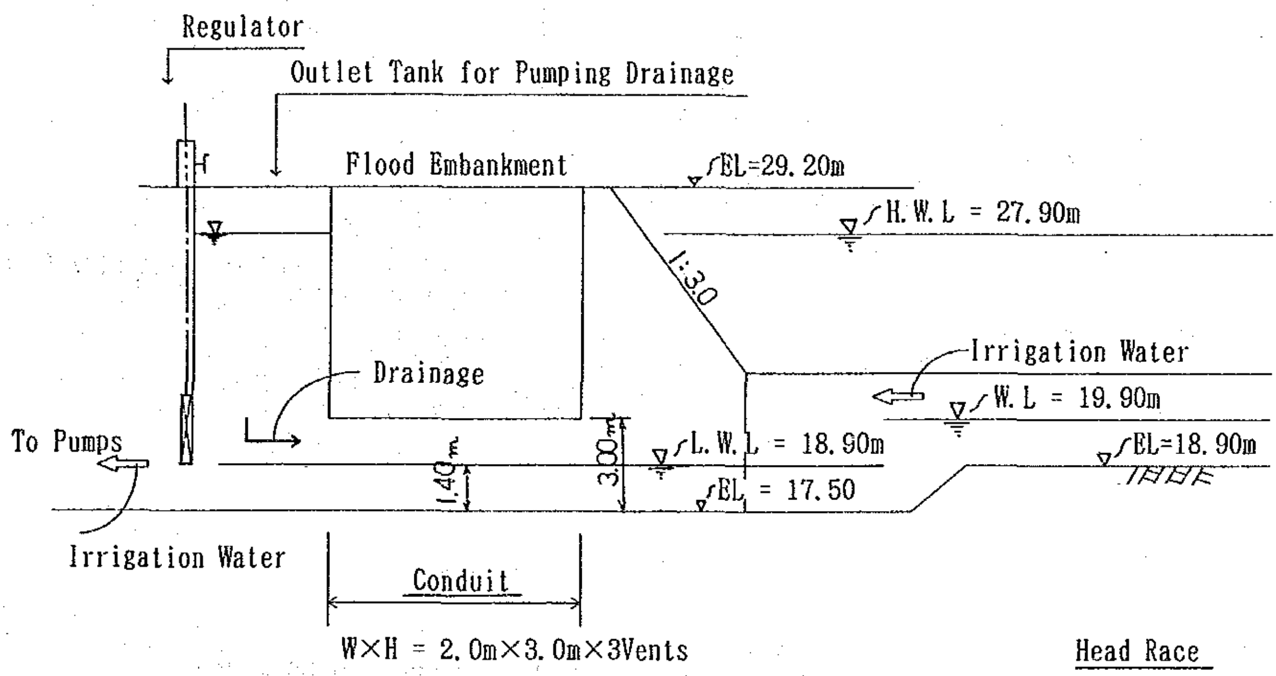
(2) Bed width and Sill Height of the Head Race.

Considering the existing canal conditions, the bed's width for the head race was determined and what is require is to keep the existing canal width for gravity drainage at least.



$$V = Q/A = \frac{4.87\ m^3/sec}{10 \times 1.0} = 0.48\ m/sec$$

(3) Scale and Sill Height of the Conduit and the Regulator (Gate)



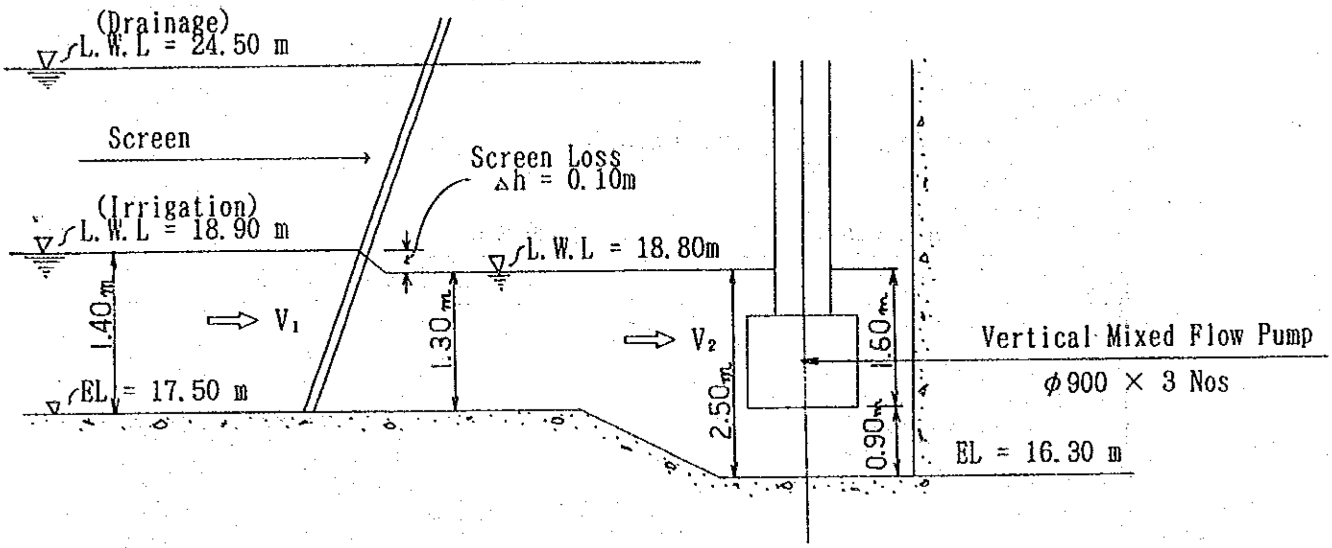
- Intake for Irrigation Water

$$V = \frac{Q}{W \times H} = \frac{4.78\ m^3/sec}{2.0\ m \times 1.4\ m \times 3\ Vents} = 0.57 \fallingdotseq 0.60\ m/sec$$

- Outlet for the pumping Drainage Water.

$$V = \frac{Q}{W \times H} = \frac{7.30\ m^3/sec}{2.0\ m \times 3.0\ m \times 3\ Vents} = 0.41\ m/sec$$

(4) Pump Suction Sump



$$V_1 = \frac{Q}{W \times H} = \frac{4.78 \text{ m}^3}{2.7 \text{ m} \times 1.40 \text{ m} \times 3 \text{ Nos}} = 0.42 \text{ m/sec} \geqq 0.50 \text{ m/sec}$$

$$V_2 = \frac{Q}{W \times H} = \frac{4.78}{2.7 \text{ m} \times 2.5 \text{ m} \times 3 \text{ Nos}} = 0.24 \text{ m/sec}$$

(5) Scale of the Outlet Pond and the Settling Basin

a) Settling Basin

The scale of the settling basin is determined through the following formula.

$$L = K \cdot \frac{Q}{B \cdot Vg}$$

where :

L : length of settling basin (m)

B : Width of - do - 70 m

K : Safety factor 2.0

Q : Design discharge 4.78 m^3/sec

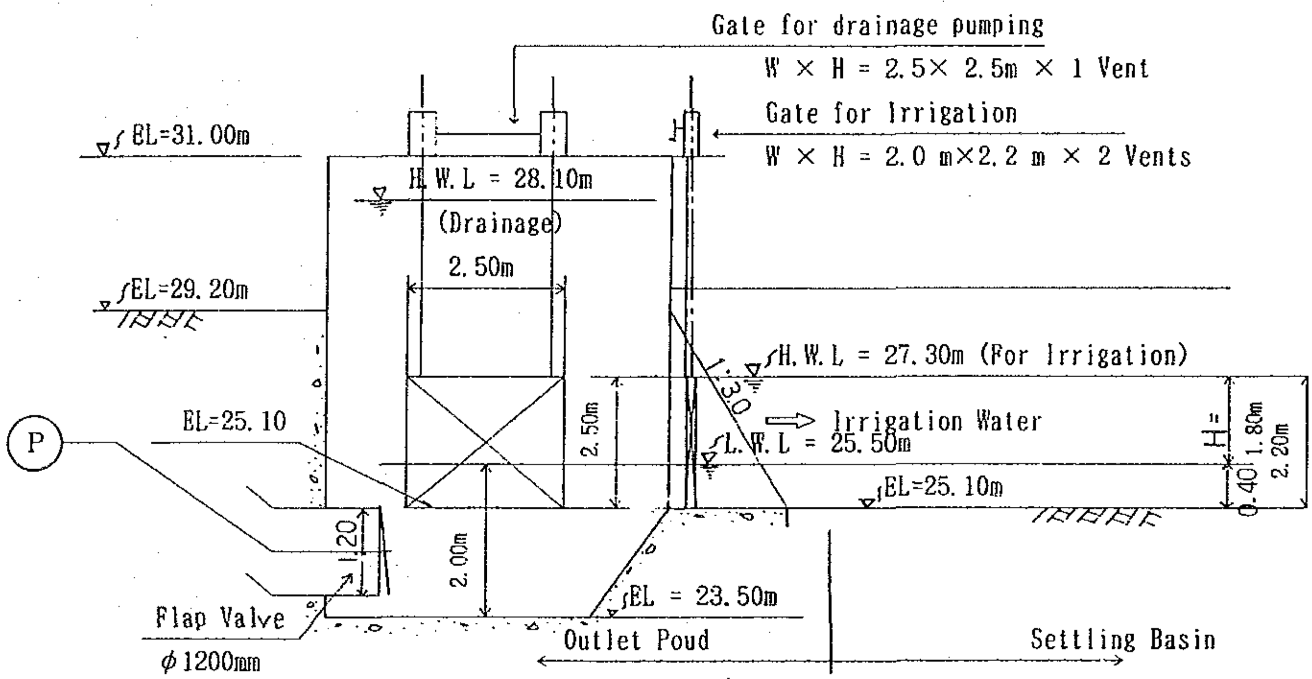
Vg : Settling velocity with minimum sediment size (mm)

Vg = 0.002 m/sec (Grain size d = 0.02 mm)

$$L = 2.0\ \frac{4.78}{70 \times 0.002} = 68 \rightarrow 70\ m$$

$$\therefore A = B \cdot L = 70\ m \times 70\ m = 4,900\ m^2$$

b) Outlet Pond



- Working hours for the setting basin pumping :

Settling Basin Volume (V)

$$V = W \times L \times H = 70\ m \times 70\ m \times 1.8m = 8.820\ m^3$$

Pump Working Hours (T)

$$T = \frac{V}{Qp} = \frac{8,820\ m^3}{4.78\ m^3/s \times 60\ sec} = 31\ min$$

⇩

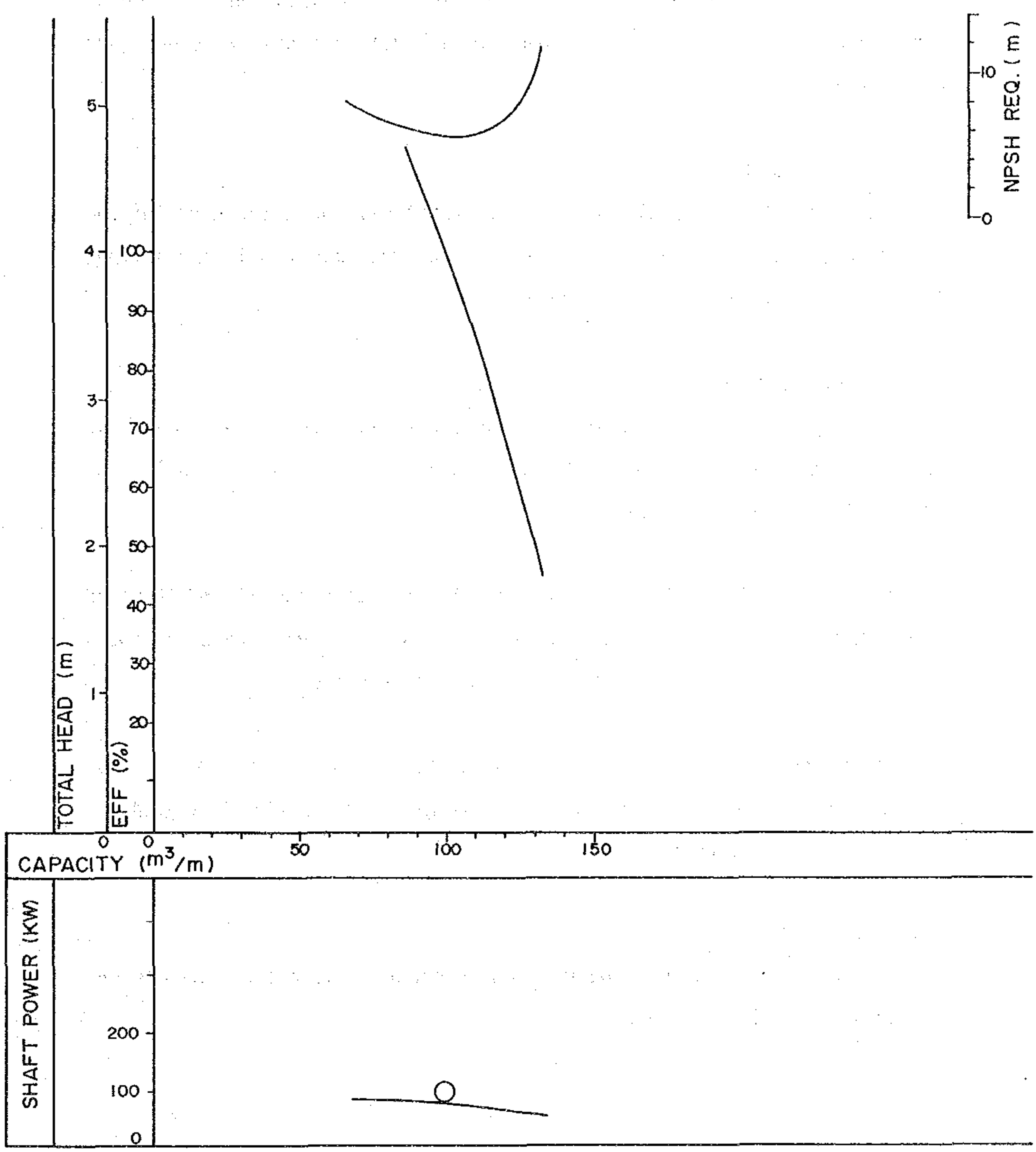
$$H = 1.8\ m$$

Considering the pumps on-off hours the pumps break hours are planned for more than 30 minutes to protect the pumps when they are used during the period of maximum irrigation requirements.

Fig. Ⅷ-4-13 Characteristic Curve of Vertical Axial Flow Pump

(Begomganj Drainage Pump Station : φ900 mm)

SPECIFIED ITEMS : 100^{m^3}/min x 4^m x 423 rpm x 100 kw



5. Irrigation Canals

5-1. Standard Irrigation Canals

In accordance with the results of the comparative study on canal cross senctions mentioned in 5-3., the standard irrigation canals were designed accordingly.

(1) Standard canal types

As for the standard canal cross sections, 10 types were proposed for the main canals, 15 types for the secondary / sub-secondary canals and 3 types for the tertiary canals, respectively.

(2) Hydraulic analysis and canal dimensions

Water-depths and velocities of canals were calculated by using the Manning's formula, and heights of canal embankments were given considering these water depths and freeboards.

The results of hydraulic analysis as well as dimensions of each type of main canals, secondary / sub-secondary canals and tertiary canals are shown inTable Ⅷ-5-1, Table Ⅷ-5-2 and TableⅧ-5-3, respectively.

Besides, the typical cross sections of main canals, secondary / sub-secondary canals and the tertiary canals are given in Fig.Ⅷ-5-1, Fig.Ⅷ-5-2 and Fig. Ⅷ-5-3, respectively.

(3) Canal lengths

The lengths of main, secondary / sub-secondary and tertiary canals are listed up in Table Ⅷ-5-4.

Table Ⅷ-5-1 Hydraulic and Structural Dimensions for Main Canals

Canal Type	Q (m^3/s)	m_1	b (m)	d (m)	V (m/s)	fb (m)	H (m)	B (m)
M-Ⅰ	42.78	2.5	22.0	2.56	0.59	0.89	3.45	39.25
	40.08	2.5	22.0	2.52	0.59	0.88	3.40	39.00
M-Ⅱ	23.04	2.0	15.0	2.06	0.58	0.84	2.90	26.60
	20.66	2.0	15.0	1.94	0.56	0.81	2.75	26.00
	18.82	2.0	15.0	1.84	0.55	0.76	2.60	25.40
M-Ⅲ	17.04	2.0	14.0	1.95	0.49	0.80	2.75	25.00
	16.37	2.0	14.0	1.91	0.48	0.79	2.70	24.80
	15.64	2.0	14.0	1.86	0.47	0.79	2.65	24.60
	15.07	2.0	14.0	1.83	0.47	0.77	2.60	24.40
	14.05	2.0	14.0	1.75	0.46	0.75	2.50	24.00
	13.67	2.0	14.0	1.73	0.45	0.77	2.50	24.00
M-Ⅳ	16.27	2.0	14.0	1.75	0.53	0.75	2.50	24.00
M-Ⅴ	14.31	1.5	13.0	1.61	0.58	0.74	2.35	20.05
M-Ⅵ	11.35	1.5	12.0	1.72	0.45	0.73	2.45	19.35
	11.21	1.5	12.0	1.71	0.45	0.74	2.45	19.35
M-Ⅶ	9.12	1.5	9.0	1.51	0.53	0.69	2.20	15.60
M-Ⅷ	6.79	1.5	7.0	1.46	0.51	0.69	2.15	13.45
M-Ⅸ	5.30	1.5	5.5	1.43	0.49	0.67	2.10	11.80
M-Ⅹ	4.50	1.5	4.5	1.31	0.53	0.64	1.95	10.35

Note: 1) Roughness coefficient (n) = 0.025

2) Canal slope (I); Type M-Ⅰ, Ⅲ, Ⅵ ………………… 1/12,000

Type M-Ⅱ, Ⅳ ………………………… 1/ 9,000

Type M-Ⅴ, Ⅶ, Ⅷ, Ⅸ ………… 1/ 7,000

Type M-Ⅹ ………………………………… 1/ 5,000

Table Ⅷ-5-2 Hydraulic and Structural Dimensions for Secondary and Subsecondary Canals

Canal Type	Q (m^3/s)	m_1	b (m)	d (m)	V (m/s)	fb (m)	H (m)	B (m)
S- 1	4.50	1.5	4.50	1.43	0.47	0.67	2.10	10.80
S- 2	4.00	1.5	4.00	1.29	0.52	0.61	1.90	9.70
S- 3	3.50	1.5	4.00	1.20	0.50	0.60	1.80	9.40
S- 4	3.00	1.5	3.50	1.17	0.49	0.58	1.75	8.75
S- 5	2.50	1.5	3.00	1.14	0.47	0.56	1.70	8.10
S- 6	2.00	1.5	3.00	1.01	0.44	0.54	1.55	7.65
S- 7	1.50	1.5	2.50	0.94	0.41	0.51	1.45	6.85
S- 8	1.00	1.5	2.50	0.75	0.37	0.45	1.20	6.10
S- 9	3.00	1.5	3.00	1.09	0.59	0.56	1.65	7.95
S-10	2.50	1.5	2.50	1.07	0.57	0.53	1.60	7.30
S-11	2.00	1.5	2.50	0.95	0.54	0.55	1.50	7.00
S-12	1.50	1.5	2.00	0.89	0.50	0.51	1.40	6.20
S-13	1.00	1.5	2.00	0.72	0.45	0.48	1.20	5.60
S-14	0.75	1.5	1.50	0.70	0.42	0.45	1.15	4.95
S-15	0.50	1.5	1.00	0.65	0.39	0.45	1.10	4.30

Note: Canal slope(I); Type S-1 .... 1/7,000
Type S-2 ~S-8 .... 1/5,000
Type S-9 ~S-15 .... 1/3,000

Table Ⅷ-5-3 Hydraulic and Structural Dimensions for Tertiary Canals

Canal Type	Q (m^3/s)	m_1	b (m)	d (m)	V (m/s)	fb (m)	H (m)	B (m)
T- 1	0.50	1.5	1.00	0.50	0.57	0.40	0.90	3.70
T- 2	0.30	1.5	0.50	0.48	0.51	0.42	0.90	3.20
T- 3	0.10	1.5	0.30	0.32	0.40	0.38	0.70	2.40

Note: Canal slope(I) = 1/1,000

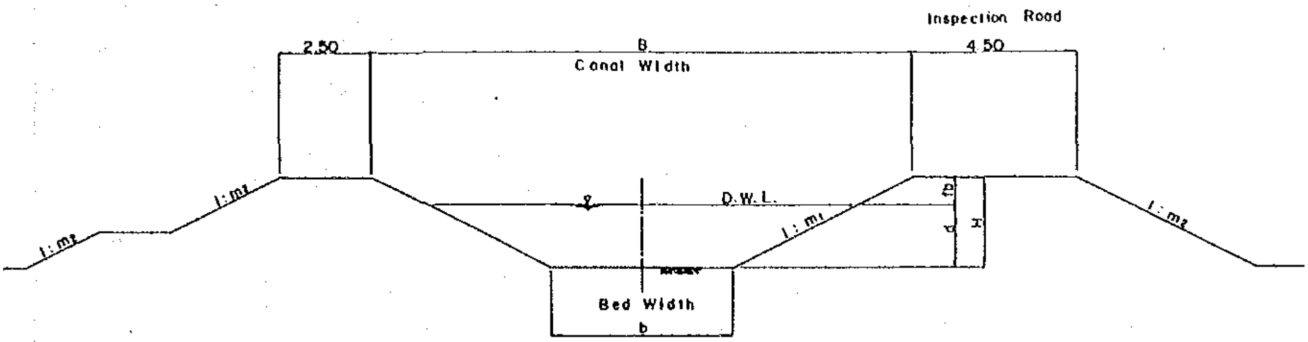


Fig. Ⅷ-5-1 Typical Cross Section of Proposed Main Canal

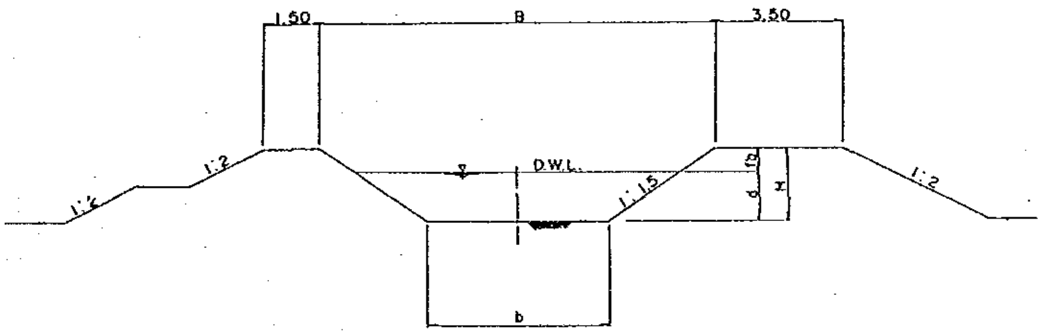


Fig. Ⅷ-5-2 Typical Cross Section of Proposed Secondary and Sub-secondary Canals

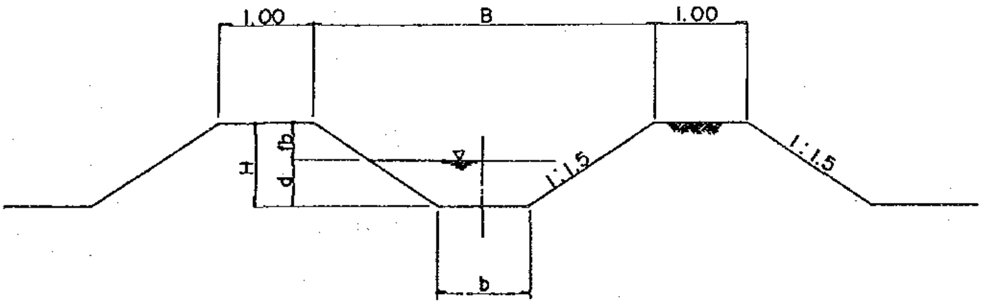


Fig. Ⅷ-5-3 Typical Cross Section of Proposed Tertiary Canal

TableⅧ-5-4 List of Canal Length

CANAL NAME	CANAL TYPE	CANAL ELNGTH (m)	CANAL NAME	CANAL TYPE	CANAL ELNGTH (m)
1. Main canal			M_1S_9	S-7	3,000
M	M-Ⅰ	4,400	M_1S_{10}	S-4	3,600
M_1	M-Ⅱ	5,200	$M_1S_{10}S_1$	S-7	5,000
〃	M-Ⅳ	1,400	$M_1S_{10}S_2$	S-8	5,500
〃	M-Ⅴ	4,200			
〃	M-Ⅶ	4,800	M_2S_1	S-14	3,200
〃	M-Ⅹ	4,000	M_2S_2	〃	1,900
M_2	M-Ⅲ	12,200	M_2S_3	S-15	3,000
〃	M-Ⅵ	6,200	M_2S_4	S-8	3,600
〃	M-Ⅷ	500	M_2S_5	S-5	1,900
〃	M-Ⅸ	4,500	$M_2S_5S_1$	S-8	2,700
Total		47,400	$M_2S_5S_2$	S-14	2,500
			M_2S_6	S-1	4,600
2. Sec/sub-Sec. Canal			$M_2S_6S_1$	S-7	3,800
MS_1	S-7	4,000	$M_2S_6S_2$	S-6	5,000
MS_2	〃	6,000	M_2S_7	S-7	2,900
M_1S_1	S-5	6,400	M_2S_{8-1}	S-9	1,300
M_1S_2	S-6	4,200	M_2S_{8-2}	S-11	3,000
M_1S_3	S-5	500	$M_2S_8S_1$	S-14	2,100
$M_1S_3S_1$	S-6	5,900	$M_2S_8S_2$	〃	2,000
$M_1S_3S_2$	S-14	3,600	$M_2S_8S_3$	S-15	2,700
M_1S_4	S-6	4,600	M_2S_9	S-10	10,500
M_1S_5	S-5	2,000	RS_1	S-6	4,500
$M_1S_5S_1$	S-8	4,500	RS_2	S-3	3,000
$M_1S_5S_2$	S-14	1,600	RS_2S_1	S-5	6,400
M_1S_{6-1}	S-4	600			
M_1S_{6-2}	S-5	3,100	Total		156,300
$M_1S_6S_1$	S-14	2,300			
$M_1S_6S_2$	〃	1,600	3. Tertiary Canal	T-1	211,000
$M_1S_6S_3$	〃	1,500		T-2	143,000
M_1S_7	S-4	1,100			
$M_1S_7S_1$	S-8	3,700	Total		354,000
$M_1S_7S_2$	S-7	4,100			
M_1S_8	S-6	2,200			
$M_1S_8S_1$	S-8	3,500			
$M_1S_8S_2$	S-15	1,600			

5-2. Appurtenant Structures

As for the major appurtenant structures, the following structures are planned on the main canals, secondary sub-secondary canals and tertiary canals:

- Bifurcations,
- Check structures,
- Escapes,
- River crossing structures, (Syphon type, Aqueduct type, Culvert type) and
- Road crossing structures, (Bridge type, Culvert type)

These structures are listed in Table Ⅷ-5-5, and the standard figures areshown in " VOLUME 3 DRAWINGS ".

Table Ⅷ-5-5 List of Appurtenant Structures

(1/3)

Canal Name	CheсK	Bifurcation			Syphon	Aqueduct	Road Crossing		Drainage Culvert		Escape		Remarks
		A	B	C			Bridge	Culvert	Box Type	Pipe Type	A Type	B Type	
M			2				5		1				
M_1	3	9	2		1		21		3		2		
M_2	2	4	7		1		24		4		2		
MS_1				5		1		14		9		1	
MS_2				5				13	2	9		1	
M_1S_1				8			6	18	1	10		1	
M_1S_2				6			5	8		11		1	
M_1S_3			2	1			1	4		8		1	
$M_1S_3S_1$				5			6	16		3		1	
$M_1S_3S_2$				2				10		1		1	
M_1S_4				6		1	4	12	3	1		1	
M_1S_5			2	2		4		4		7		1	
$M_1S_5S_1$				4		1		19		2		1	
$M_1S_5S_2$				2				2		1		1	
M_1S_6			3	4		1	5	7	3	9		1	
$M_1S_6S_1$				2				11		2		1	
$M_1S_6S_2$				2				9		1		1	
$M_1S_6S_3$				2				6		1		1	
M_1S_7			2	2		2	1	8		9		1	
$M_1S_7S_1$				3				19		4		1	
$M_1S_7S_2$				5				23		4		1	

Canal Name	Check	Bifurcation			Syphon	Aqueduct	Road Crossing		Drainage Culvert		Escape		Remarks
		A	B	C			Bridge	Culvert	Box Type	Pipe Type	A Type	B Type	
M_1S_8			2	3		1	3	6	1	4			
$M_1S_8S_1$				3				12		2		1	
$M_1S_8S_2$				2				10	1	1		1	
M_1S_9				6				24		7		1	
M_1S_{10}				3			5	7	1	12			
$M_1S_{10}S_1$				4				16		5		1	
$M_1S_{10}S_2$				5				13		4		1	
M_2S_1				3		4		10	1	4		1	
M_2S_2				3				6				1	
M_2S_3				2				15		3		1	
M_2S_4				3		1		13	2	3		1	
M_2T_1				2				4				1	
M_2S_5			2	2		3	1	11	1	3			
$M_2S_5S_1$				3				15		1		1	
$M_2S_5S_2$				3				11	1	1		1	
M_2T_2				1				2				1	
M_2S_6			3	6	1	2	4	11	1	13			
$M_2S_6S_1$				4				27	1	3		1	
$M_2S_6S_2$				2			8	21		3		1	
M_2S_7				4		1		26		6		1	
M_2S_8			3	4		3	8	15		8			
$M_2S_8S_1$				2				11	2	1		1	

Canal Name	Check	Bifurcation			Syphon	Aqueduct	Road Crossing		Drainage Culvert		Escape		Remarks
		A	B	C			Bridge	Culvert	Box Type	Pipe Type	A Type	B Type	
$M_2S_8S_2$				2				16		1		1	
$M_2S_8S_3$				2				9	1			1	
M_2S_9				7			13	17		14		1	
RS_1			1	7		1	5	14	1	5		1	
RS_2		2		3			1	4	1	1			
RS_2S_1				7			15	31	1	14		1	
Total	5	15	31	164	3	26	141	580	33	211	4	35	

5-3. Comparative Study on Canal Cross Sections

In order to find the optimal cross sections for irrigation canals, a comparative study has been done from structual stability and economic view points.

(a) Comparative cases

The following cases for the design water depths and for the canal discharges were put in the comparative study:

Discharge (m^3/s)	Water depths (m)
40.0	3.0, 2.5 & 2.0
20.0	2.5, 2.0 & 1.5
15.0	2.0, 1.8 & 1.5
10.0	1.8, 1.5 & 1.2

(b) Basic design conditions

Basic canal design conditions for the comparative study are as follows:

1) Hydraulic shape

A type of irrigation canal is planned as an unlined trapezoid type from aneconomical viewpoint.

2) Canal bed gradient

The longitudinal bed slope of a canal is selected considering the maximum and minimum allowable velocities in the canal and the present topographic conditions.

3) permissible velocity

The maximum allowable velocity is to be 0.6 m/sec considering the soil characteristics of this area, and the minimum, 0.3 m/sec, as a non-silting and a non-weed velocity. The Manning's formula is used as a mean velocity formula to plan canals, as presented below:

$V = 1/n \cdot R^{2/3} \cdot I^{1/2}$

$Q = A \cdot V$

Where, V; Mean velocity (m/sec)

n; Coefficient of roughness = 0.025

R; Hydraulic mean radius (m)

I; Canal slope

A; Cross sectional area (m^2)

Q; Design canal discharge (m^3/sec)

4) Freeboard

The formura for freeboard is given bellow with a maximum limit of 0.90 m:

$Fb = 0.25\ d\ \ + 0.30$

Where, Fb; Freeboard (m)

d; Water depth for design discharge (m)

5) Side slope

Side slopes of the filling embankments of canals are determined througha slope stability analysis, and considered the hydraulic grade line through theembankment in case of outside slopes.

As a result, the side slopes of the canals are planned as shown in the following table:

Embankment height (m)	Side slope inside	Side slope outside
less than 2.5	1.0: 1.5	1.0: 2.0
2.5 ～ 3.0	1.0: 2.0	1.0: 2.0
3.0 ～ 3.5	1.0: 2.5	1.0: 2.5
3.5 ～ 4.0	1.0: 3.0	1.0: 3.0
above 4.0	above 3.5	above 3.5

The side slopes of the cutting embankments of the canals are planned as 1.0:1.5

6) Bank top width

The bank top width for one side of the main canals is to be 4.5 m as an inspection road, and that for the other side, 2.5 m.

7) Borrow pits

Borrow pits are required for obtaining embankment meterial. They are to belaid at both the outsides of a canal.
The maximum depth of the borrow pits is to be 2.0 m with a cutting slope of 1:1.

(c) Results of the comparative study

Based on the hydlaulic design the dimensions, the land widths for construction, the earth volumes for cutting and filling and the construction costs of canals in comparative cases were calculated as shown in Table Ⅷ-5-6.

As a result, the following standard water depths were selected for each case of canal discharges, which give the optimal standard cross sections:

Canal discharge (m^3/sec)		Standard water depth (m)
Q = 40	....	d = 2.5
Q = 20	....	d = 2.0
Q = 15	....	d = 1.8
Q = 10	....	d = 1.5
Q = 5	....	d = 1.4

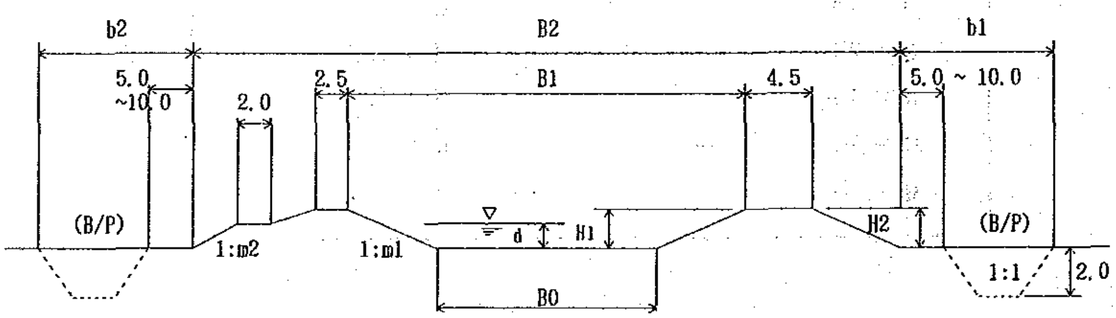
In case a canal discharge is small, the following enpirical formula is adoptedto determine a canal bed width and a water depth.

$d = 0.5 \cdot b^{1/2}$ where, d; Water depth. (m)
b; Bed width (m)

TableⅧ-5-6 Comparison of Canal Cross Sections

Discharge (m3/s) Bed slope	Water depth d (m)	Embankment hight (m)		Side slope		Canal width (m)			Borrow pit width (m)		Earth volume (m3)	Canal land width (m)	Canal cost (TK/m)
		H1	H2	m1	m2	B0	B1	B2	b1	b2			
Q=40 i=1/12,000	3.0	4.0	5.1	3.0	3.5	15	39	84	65	61	220	210	5,871
	2.5	3.4	5.0	2.5	3.5	22	39	83	64	61	217	208	5,798
	2.0	2.8	4.9	2.0	3.5	33	44	87	67	64	230	218	6,123
Q=20 i=1/9,000	2.5	3.4	2.9	2.5	2.5	9	26	50	19	16	67	85	1,979
	2.0	2.8	2.8	2.0	2.0	14	25	45	17	15	51	77	1,618
	1.5	2.2	2.7	1.5	2.0	25	32	52	19	17	58	88	1,844
Q=15 i=1/12,000	2.1	2.9	2.6	2.0	2.0	12	24	43	15	14	49	72	1,537
	1.8	2.6	2.6	2.0	2.0	14	24	43	16	13	45	72	1,465
	1.5	2.2	2.5	1.5	2.0	21	28	47	16	14	46	77	1,528
Q=10 i=1/7,000	1.8	2.6	2.3	2.0	2.0	7	17	35	14	11	39	60	1,248
	1.5	2.2	2.2	1.5	2.0	10	17	35	12	10	32	57	1,094
	1.2	1.8	2.1	1.5	2.0	15	20	37	12	10	29	59	1,058

Note: 1) Unite price Earth work: 18TK/m^3
Land cost : 9.1TK/m^2



5-4. Slope Stability Analysis

Side slopes of the filling embankments of canals were determined through aslope stability analysis. In case of outside slopes, the hydlaulic grade lines through the embankments were also considered.

(1) Slope stability computation

1) Soil data

The following investigated soil deta at the proposed Pateswari pumping station site were adopted:

- Dry density .... 1.4 t/m³
- Wet density .... 1.8 t/m³
- Submerged density .... 2.0 t/m³
- Angle of internal friction ..... 16°
- Cohesion .... 0.7 t/m²

2) Safety factor

A safety factor in the slip circle method is as follow:

$$Fs = Rf/Sf \geq 1.5$$

Where, Fs; Safety factor

Rf; Resisting moment (t・m)

Sf; Sliding moment (t・m)

3) Computational cases

The following 6 cases were computed:

	Embankment height (m) (1) outside	(2) Inside
Case 1	5.0	3.5
Case 2	3.0	3.0
Case 3	3.0	2.5

4) Results of slope stability analysis

The analysis results of side slopes and the minimum safety factors for each case are shown as follows:

	Embankment height (m)	Side slope	Minimumt safety factor
Case 1-(1)	5.0	1: 3.5(outside)	1.5
Case 1-(2)	3.5	1: 2.5(inside)	1.9
Case 2-(1)	3.0	1: 2.0(outside)	1.5
Case 2-(2)	3.0	1: 2.0(inside)	2.1
Case 3-(1)	3.0	1: 2.0(outside)	1.6
Case 3-(2)	2.5	1: 1.5(inside)	2.0

Details of the analysis results are presented in Fig. Ⅷ-5-4 (1)~Ⅷ-5-4 (6).

(2) Provision of outside berms for canal embankments

1) Hydraulic grade line

The gradient of saturation line in the embankment depends mainly on the characteristics and relative placement of the different types of embankment materials.

As for hydraulic gradients, the value of 1:5 (vertical to horizotal) was applied empirically by considering the soil material as silty sand in this area.

2) Outside berms

In order to prevent the emergence of the hydraulic grade line above the outer toe of the embankment, outside berms were recommended with a 2.0 m widthon one side of the canal embankments. This embankment has a bank top width smaller than an inspection road side, for the main, secondary and sub-secondary canals. It is more economical to put the outside berm than to flatten thebank slope wholly.

Fig. VIII-5-4(1) Slope Stability Analysis Results for Case 1-(1)

Y\X	24.00	25.00	26.00	27.00	28.00	29.00	30.00	31.00
26.00	1.553	1.483	1.448	1.444	1.485	1.545	1.633	1.744 (Safety factor)
	(19.0)	(19.0)	(19.0)	(19.0)	(19.0)	(19.0)	(19.0)	(19.0) (Radius)
25.00	1.564	1.486	1.446	1.443	1.470	1.526	1.608	1.725
	(18.0)	(18.0)	(18.0)	(18.0)	(18.0)	(18.0)	(18.0)	(18.0)
24.00	1.583	1.492	1.446	1.442	1.456	1.507	1.585	1.692
	(17.0)	(17.0)	(17.0)	(17.0)	(17.0)	(17.0)	(17.0)	(17.0)
23.00	1.610	1.506	1.454	1.443	1.454	1.490	1.562	1.662
	(16.0)	(16.0)	(16.0)	(16.0)	(16.0)	(16.0)	(16.0)	(16.0)
22.00	1.627	1.527	1.466	1.450	1.454	1.477	1.541	1.636
	(15.0)	(15.0)	(15.0)	(15.0)	(15.0)	(15.0)	(15.0)	(15.0)
21.00	1.640	1.559	1.487	1.462	1.457	1.478	1.521	1.613
	(14.0)	(14.0)	(14.0)	(14.0)	(14.0)	(14.0)	(14.0)	(14.0)
20.00	1.654	1.579	1.520	1.479	1.471	1.480	1.510	1.589
	(13.0)	(13.0)	(13.0)	(13.0)	(13.0)	(13.0)	(13.0)	(13.0)
19.00	1.670	1.602	1.565	1.508	1.486	1.493	1.518	1.574
	(12.0)	(12.0)	(12.0)	(12.0)	(12.0)	(12.0)	(12.0)	(12.0)

Minimum safety factor : Fs = 1.442
Critical circle : X = 27.00 m
Y = 24.00 m
R = 17.00 m
Resisting moment : Rf = 579.0 tm
Sliding moment : Sf = 401.5 tm

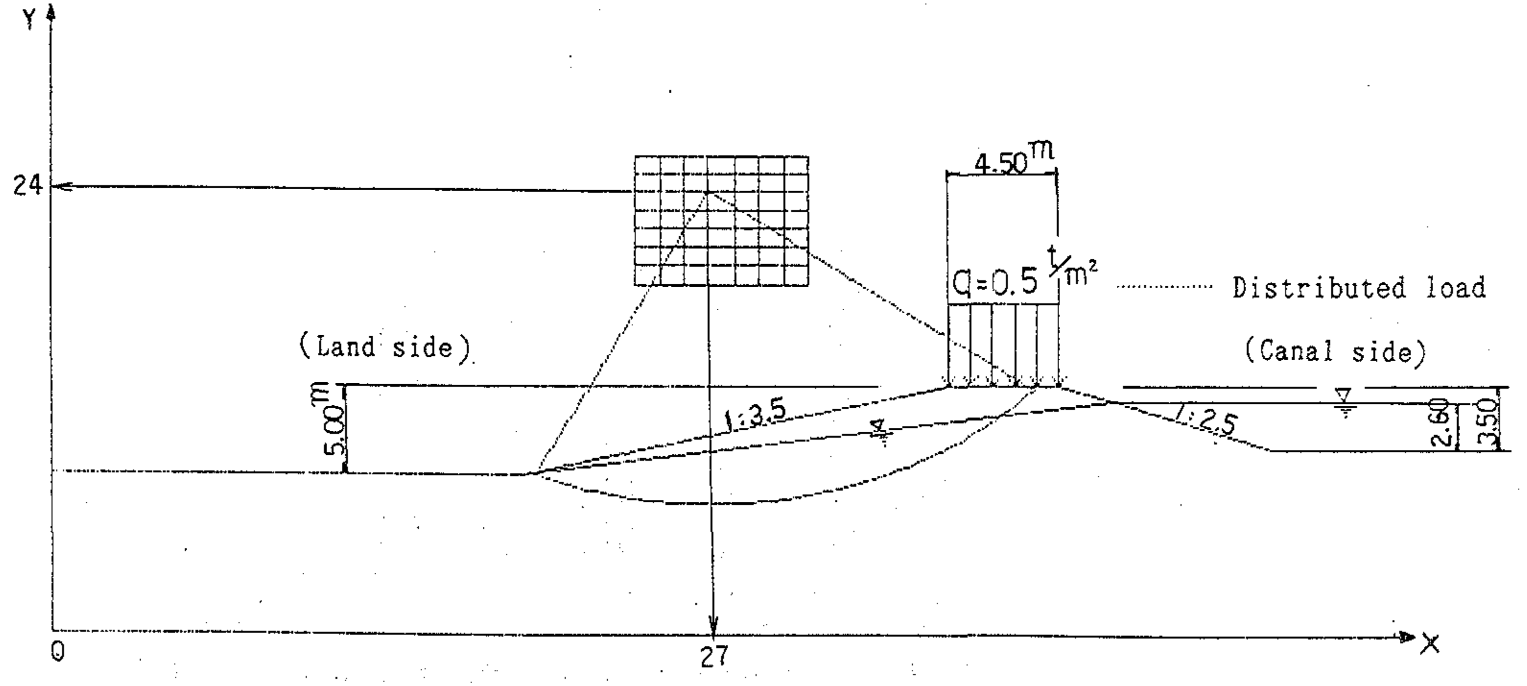


Fig. Ⅷ-5-4(2) Slope Stability Analysis Results for Case 1-(2)

Y\X	20.00	21.00	22.00	23.00	24.00	25.00
20.00	2.501 (12.0)	2.237 (12.0)	2.121 (12.0)	2.070 (11.0)	2.111 (11.0)	2.310 (11.0)
19.00	2.584 (11.0)	2.237 (11.0)	2.083 (11.0)	2.029 (10.0)	2.053 (10.0)	2.196 (10.0)
18.00	2.758 (10.0)	2.268 (10.0)	2.044 (10.0)	1.991 (10.0)	2.005 (9.0)	2.154 (9.0)
17.00	2.936 (9.0)	2.360 (9.0)	2.047 (9.0)	1.955 (9.0)	1.963 (9.0)	2.082 (9.0)
16.00	3.067 (8.0)	2.505 (8.0)	2.105 (8.0)	1.940 (8.0)	1.930 (8.0)	2.039 (8.0)
15.00	3.214 (7.0)	2.606 (7.0)	2.239 (7.0)	1.965 (7.0)	1.916 (7.0)	2.016 (7.0)

Fs = 1.916
X = 24.00 m
Y = 15.00 m
R = 7.00 m
Rf = 118.7 tm
Sf = 62.0 tm

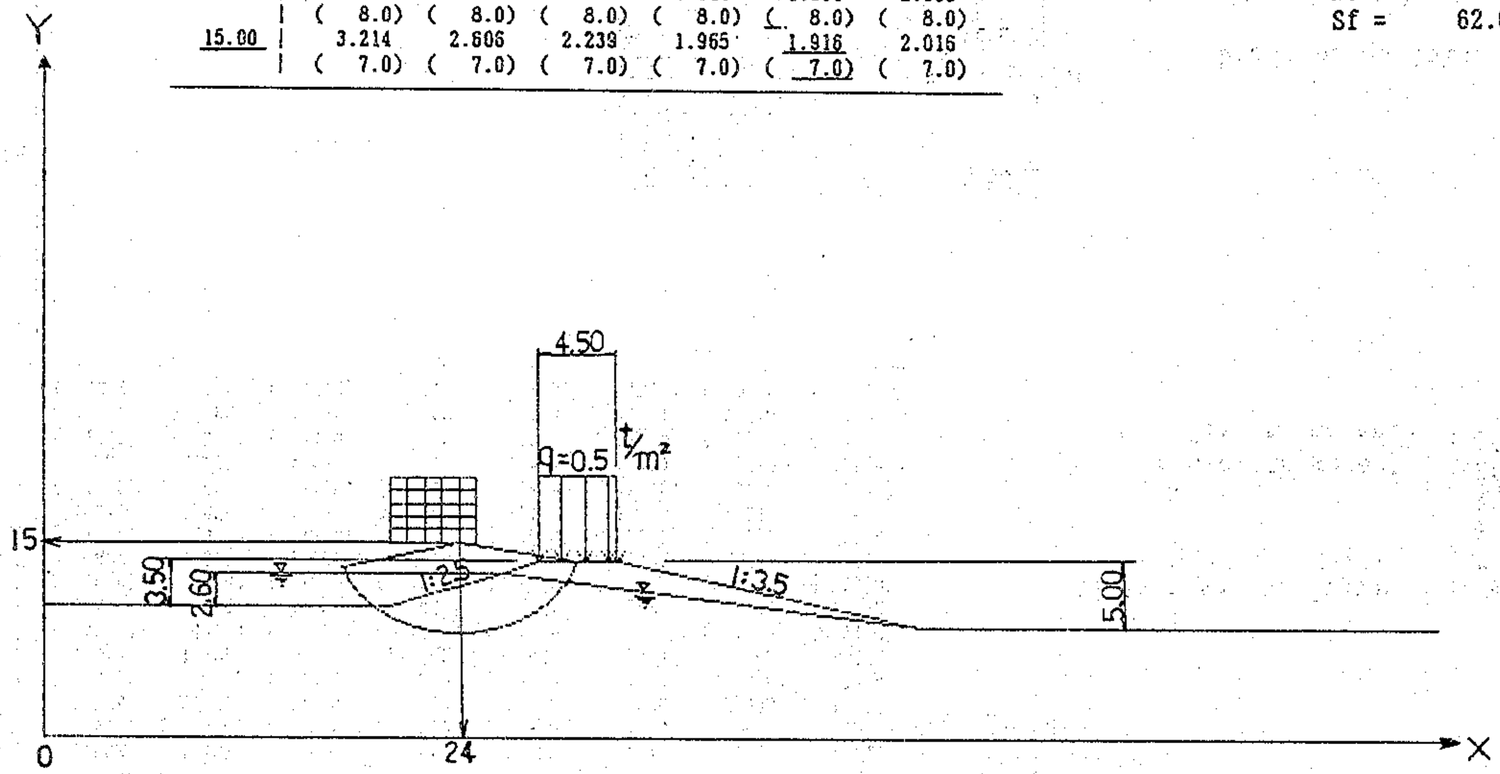


Fig. Ⅷ-5-4(3) Slope Stability Analysis Results for Case 2-(1)

Y\X	33.00	34.00	35.00	36.00	37.00	38.00
18.50	2.112	1.763	1.618	1.580	1.653	1.884
	(9.5)	(9.5)	(9.5)	(9.5)	(9.5)	(9.5)
17.50	2.238	1.773	1.593	1.545	1.592	1.819
	(8.5)	(8.5)	(8.5)	(8.5)	(8.5)	(8.5)
16.50	2.494	1.816	1.582	1.516	1.558	1.764
	(7.5)	(7.5)	(7.5)	(7.5)	(7.5)	(7.5)
15.50	2.782	1.926	1.588	1.498	1.546	1.725
	(6.5)	(6.5)	(6.5)	(6.5)	(6.5)	(6.5)
14.50	3.122	2.152	1.649	1.507	1.561	1.756
	(5.5)	(5.5)	(5.5)	(5.5)	(5.5)	(5.5)
13.50	3.583	2.336	1.867	1.581	1.628	1.830
	(4.5)	(4.5)	(4.5)	(4.5)	(4.5)	(4.5)

Fs = 1.498
X = 36.00 m
Y = 15.50 m
R = 6.50 m
Rf = 86.0 tm
Sf = 57.4 tm

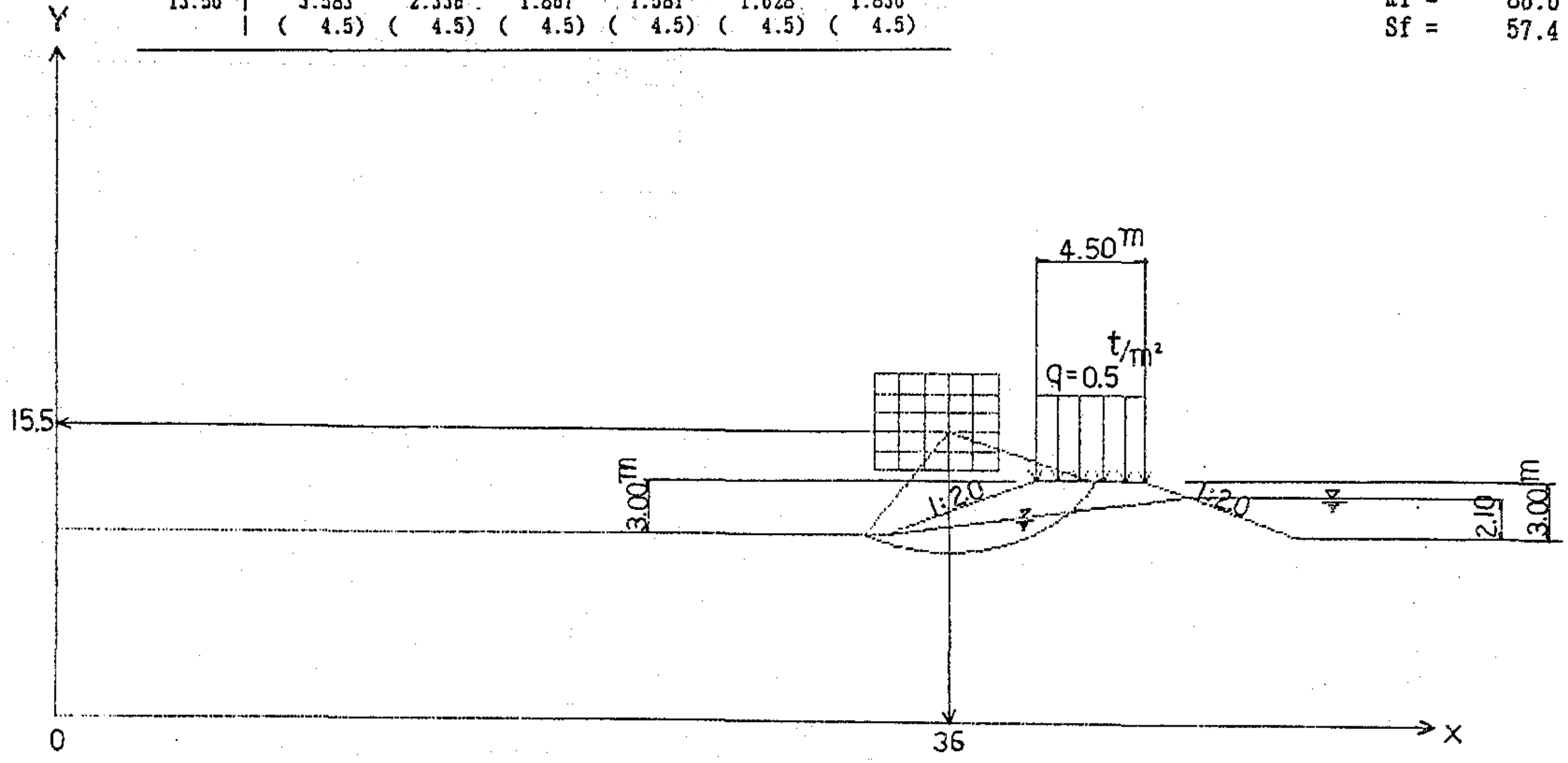


Fig. Ⅷ-5-4(4) Slope Stability Analysis Results for Case 2-(2)

Y\X	21.00	22.00	23.00	24.00	25.00
23.00	2.246	2.358	2.621	3.250	4.377
	(13.0)	(13.0)	(13.0)	(13.0)	(13.0)
22.00	2.209	2.303	2.492	3.074	4.107
	(12.0)	(12.0)	(12.0)	(12.0)	(12.0)
21.00	2.187	2.254	2.381	2.810	3.811
	(11.0)	(11.0)	(11.0)	(11.0)	(11.0)
20.00	2.172	2.199	2.315	2.670	3.459
	(10.0)	(10.0)	(10.0)	(10.0)	(10.0)
19.00	2.152	2.119	2.242	2.512	3.234
	(10.0)	(10.0)	(10.0)	(9.0)	(9.0)

Fs = 2.119
X = 22.00 m
Y = 19.00 m
R = 10.00 m
Rf = 154.7 tm
Sf = 73.0 tm

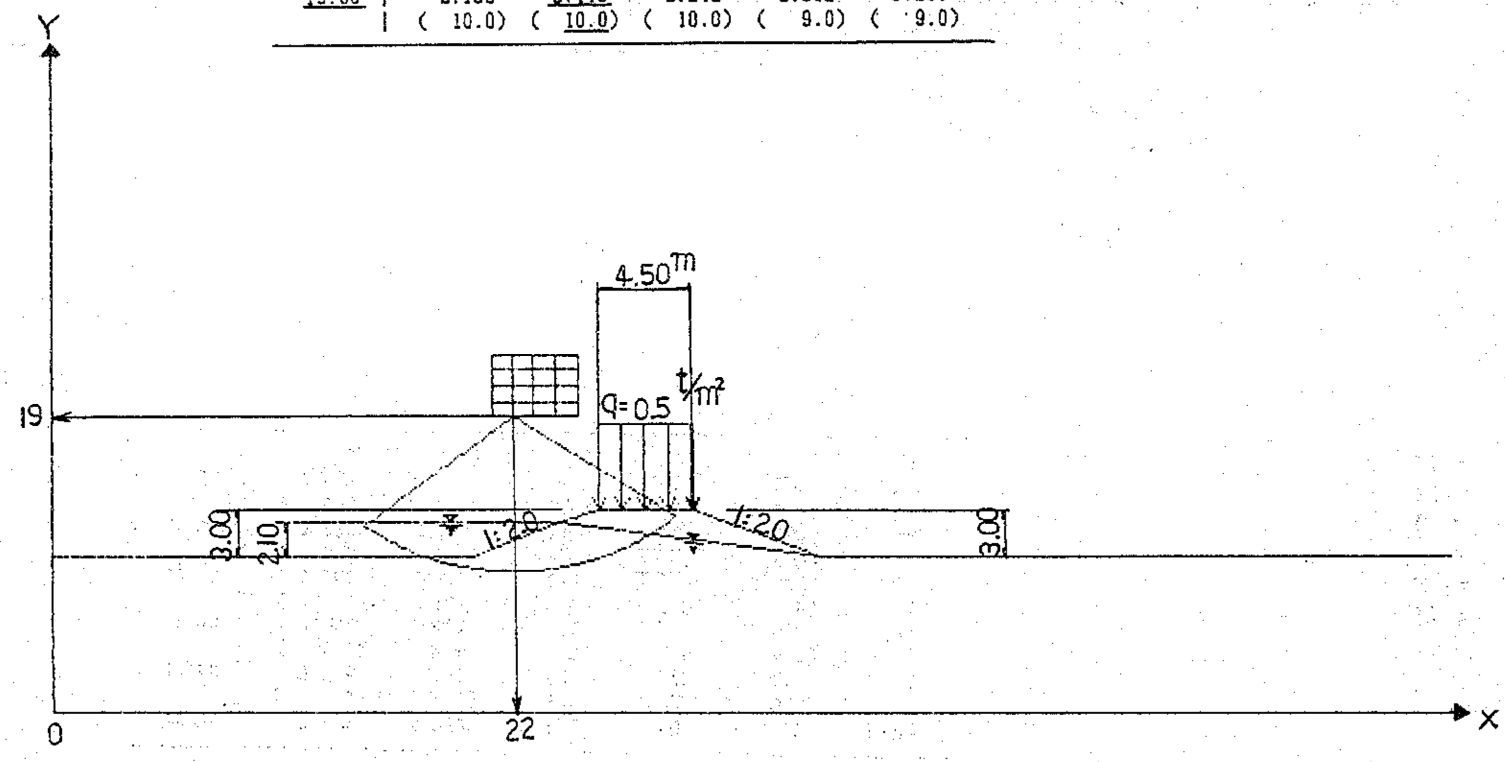


Fig. Ⅷ-5-4(5) Slope Stability Analysis Results for Case 3-(1)

Y\X	33.00	34.00	35.00	36.00	37.00	38.00	39.00
19.00	15.617	4.636	2.534	1.939	1.714	1.643	1.690
	(10.0)	(10.0)	(10.0)	(10.0)	(10.0)	(10.0)	(10.0)
18.00	29.819	5.565	2.805	1.975	1.695	1.609	1.646
	(9.0)	(9.0)	(9.0)	(9.0)	(9.0)	(9.0)	(9.0)
17.00	999.000	6.976	3.267	2.059	1.690	1.580	1.624
	(999.0)	(8.0)	(8.0)	(8.0)	(8.0)	(8.0)	(8.0)
16.00	999.000	9.423	3.697	2.243	1.706	1.558	1.606
	(999.0)	(7.0)	(7.0)	(7.0)	(7.0)	(7.0)	(7.0)
15.00	999.000	14.417	4.287	2.523	1.777	1.559	1.603
	(999.0)	(6.0)	(6.0)	(6.0)	(6.0)	(6.0)	(6.0)
14.00	999.000	999.000	5.155	2.762	1.989	1.613	1.621
	(999.0)	(999.0)	(5.0)	(5.0)	(5.0)	(5.0)	(5.0)
13.00	999.000	999.000	6.591	3.060	2.150	1.813	1.716
	(999.0)	(999.0)	(4.0)	(4.0)	(4.0)	(4.0)	(4.0)

Fs = 1.558
X = 38.00 m
Y = 16.00 m
R = 7.00 m
Rf = 78.9 tm
Sf = 50.6 tm

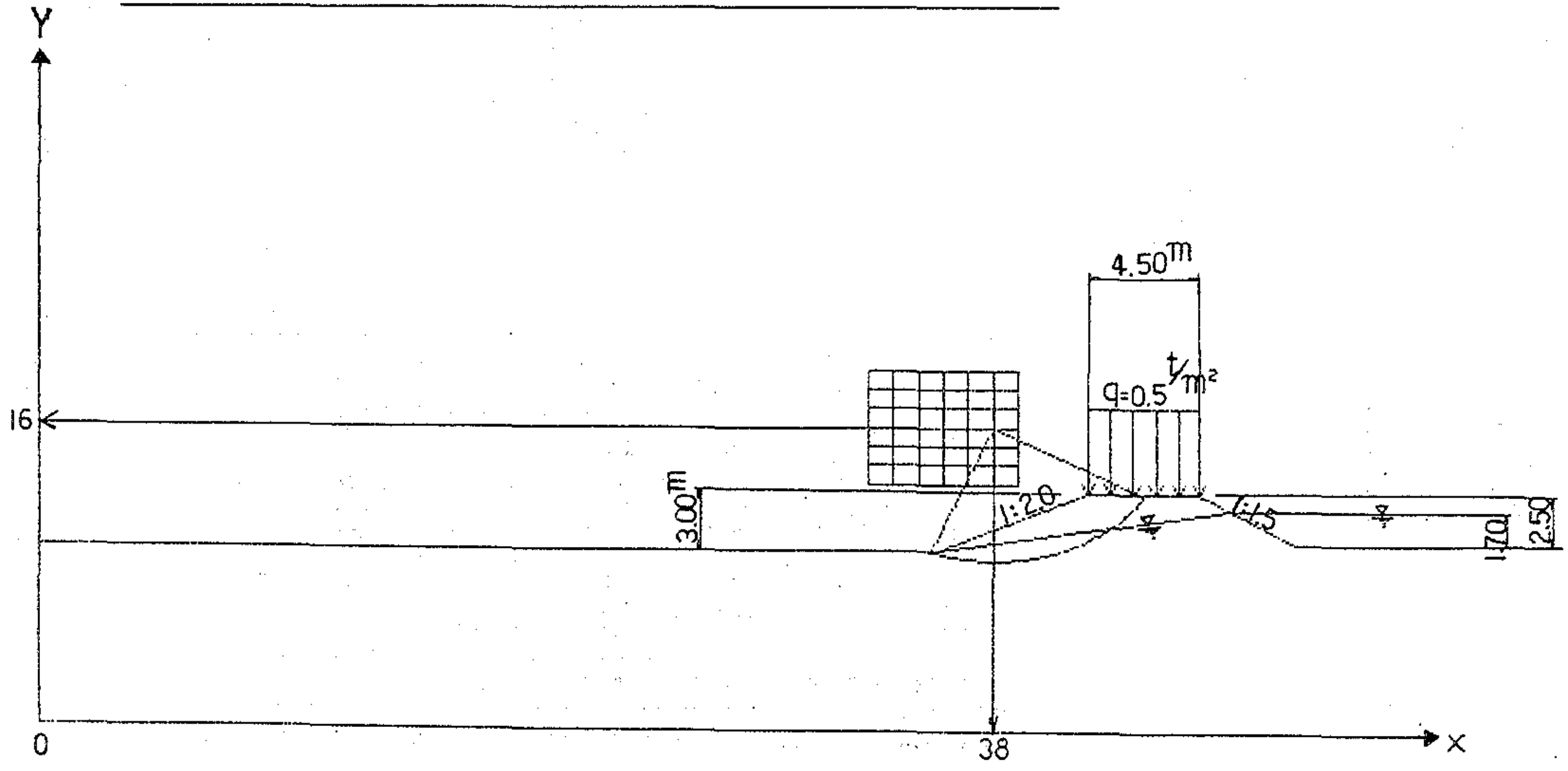
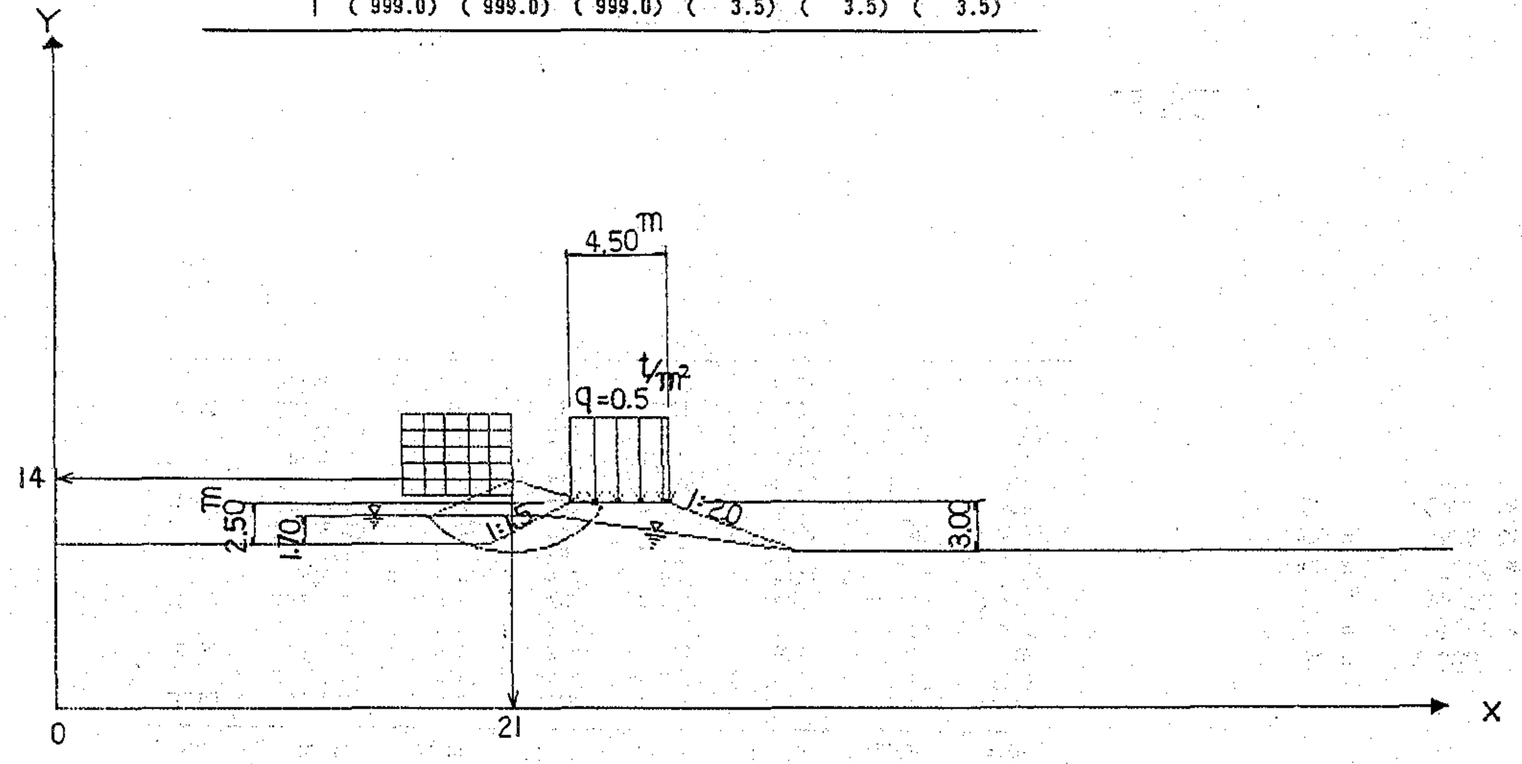


Fig. Ⅷ-5-4(6) Slope Stability Analysis Results for Case 3-(2)

Y＼X	16.00	17.00	18.00	19.00	20.00	21.00
18.00	999.000	29.357	4.365	2.723	2.313	2.239
	(999.0)	(8.5)	(8.5)	(8.5)	(8.5)	(8.5)
17.00	999.000	999.000	5.871	2.883	2.257	2.137
	(999.0)	(999.0)	(7.5)	(7.5)	(7.5)	(7.5)
16.00	999.000	999.000	10.219	3.196	2.231	2.046
	(999.0)	(999.0)	(6.5)	(6.5)	(6.5)	(6.5)
15.00	999.000	999.000	20.529	4.193	2.278	1.972
	(999.0)	(999.0)	(5.5)	(5.5)	(5.5)	(5.5)
14.00	999.000	999.000	999.000	6.036	2.537	1.951
	(999.0)	(999.0)	(999.0)	(4.5)	(4.5)	(4.5)
13.00	999.000	999.000	999.000	9.780	3.424	2.065
	(999.0)	(999.0)	(999.0)	(3.5)	(3.5)	(3.5)

Fs = 1.951
X = 21.00 m
Y = 14.00 m
R = 4.50 m
Rf = 32.1 tm
Sf = 16.4 tm



APPENDIX Ⅸ

COST ESTIMATE

APPENDIX IX

COST ESTIMATE

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1. UNIT PRICE

(1) Unit Price of Labours
(2) Unit Price of Materials
(3) Unit Price of Civil Works
(4) Unit Price of Inland Transportation
(5) Tax Rate of Imported Items

Table IX-1-1 Unit Price of Labours

Description	Unit Price (Tk/day) (8 hours per day)
1. Head Labour	75
2. Unskilled Labour	30
3. Semi Skilled Labour	35
4. Skilled Labour	50
5. Head Mason	75
6. Mason	60
7. Head Mistry	75
8. Rod Mistry	60
9. Head Carpenter	75
10. Carpenter	60
11. Plumber	60
12. Glaziar	60
13. Electrician	60
14. Painter	60
15. Blacksmith	75
16. Majhi with Dingi Boat (Boat man)	75

Source: Schedule of Rates for Project-IV, BWDB, Rangpur (Oct. 1989)

Table IX-1-2 Unit price of Materials

Materials	Component (%) Local/Foreign	Unit	Unit Price (Tk) L.C	F.C	Total
1. Brick					
- First class	100/0	100 nos.	180	0	180
- Second class	100/0	100 nos.	150	0	150
- Picked jhama	100/0	100 nos.	180	0	180
- Brick bats	100/0	m3	542	0	542
2. Stone/Boulders					
- Stone shingles	100/0	m3	710	0	710
- Boulders (75-100mm)	100/0	m3	1,050	0	1,050
- Boulders (above 300mm)	100/0	m3	1,700	0	1,700
- Boulders (above 400mm)	100/0	m3	1,800	0	1,800
3. Cement	25/75	50kg	50	150	200
4. Sand					
- F.M. 0.5 (min.)	100/0	m3	50	0	50
- F.M. 1.0 (min.)	100/0	m3	100	0	100
- F.M. 1.5-1.8	100/0	m3	210	0	210
- F.M. above 1.8-2.0	100/0	m3	280	0	280
- F.M above 2.0-3.0	100/0	m3	350	0	350
5. M.S. Rod	40/60	ton	8,400	12,600	21,000
6. M.S. Angle/Plate	40/60	ton	10,000	15,000	25,000
7. Wood					
- Jack	100/0	m3	14,000	0	14,000
- Jarul/Garjan	100/0	m3	12,350	0	12,350
- Jam/Sheel Korai	100/0	m3	10,500	0	10,500
8. 12 S.W.G.Hexagonal wire net	40/60	m2	26	39	65
9. 12 S.W.G.Plain wire	40/60	kg	16	24	40
10. Steel sheet pile	0/100	ton	0	30,000	30,000

Source: Schedule of Rates for Project-IV, BWDB, Rangpur (Oct. 1989)

Table IX-1-3 Unit price of Civil Works

Description	Unit	Unit Price(Tk)
1. Earth Work		
1) Stripping	100 m2	36
2) Construction of irrigation canal dike/embankment/road by mechanical compaction	100 m3	1,767
3) ——— do ———by manual labour	100 m3	1,281
4) Excavation of intake channel /pump station	100 m3	2,880
5) Excavation of foundation trenches (except slushy soil)	100 m3	1,499
6) —— do ——in slushy soil	100 m3	1,637
7) Earth filling in foundation, back filling	100 m3	1,136
8) Local sand(F.M.0.5 min.) filling in foundation with ramming	100 m3	5,439
9) Earth work in constructing cross dam in flowing channel	100 m3	1,281
10) Earth carring by truck(initial lead 1 km)	m3	56
11) Turfing	100 m2	220
2. Concrete Work		
A) Foundation/Plinth/Floor		
1) Aggregate: Picked jhama bricks		
- C.C. (1:3:6)	m3	1,829
- R.C.C. (1:2:4)	m3	2,287
- R.C.C. (1:1.5:3)	m3	2,632
2) Aggregate: Stone shingles		
- C.C. (1:3:6)	m3	1,974
- R.C.C. (1:2:4)	m3	2,416
- R.C.C. (1:1.5:3)	m3	2,729

(continued)

3) Aggregate: Crushed stones

- R.C.C. (1:2:4)	m3	2,446
- R.C.C. (1:1.5:3)	m3	3,044

B) Superstructure

1) Aggregate: Picked jhama bricks

- C.C. (1:3:6)	m3	1,843
- R.C.C. (1:2:4)	m3	2,300
- R.C.C. (1:1.5:3)	m3	2,645

2) Aggregate: Stone shingles

- C.C. (1:3:6)	m3	1,987
- R.C.C. (1:2:4)	m3	2,430
- R.C.C. (1:1.5:3)	m3	2,743

3) Aggregate: Crushed stones

- R.C.C. (1:2:4)	m3	2,897
- R.C.C. (1:1.5:3)	m3	3,057

3. Concrete Block

1) Aggregate: Picked jhama bricks

- C.C. (1:5:10)	m3	1,592
- C.C. (1:4:8)	m3	1,724
- C.C. (1:3:6)	m3	1,913

2) Aggregate: Stone shingles

- C.C. (1:5:10)	m3	1,799
- C.C. (1:4:8)	m3	1,900
- C.C. (1:3:6)	m3	2,057
- C.C. (1:2:4)	m3	2,502

4. Sand Cement Block

- in prop. (1:8)	m3	1,412
- in prop. (1:6)	m3	1,606

(continued)

5. Brick Work

		(1st cl.)	(2nd cl.)
A) Brick masonry			
- mortar (1:2)	m3	1,644	1,510
- (1:3)	m3	1,475	1,341
- (1:4)	m3	1,380	1,246
- (1:5)	m3	1,306	1,172
- (1:6)	m3	1,262	1,128
B) Brick pitching			
1) t= 12.7 cm			
- mortar (1:3)	m2	188	171
- (1:4)	m2	176	159
- (1:5)	m2	167	150
2) t= 7.62 cm			
- mortar (1:3)	m2	116	106
- (1:4)	m2	109	99
- (1:5)	m2	102	92

6. Plastering

1) t= 1.27 cm			
- mortar (1:2)	m2		54
- (1:3)	m2		46
- (1:4)	m2		41
- (1:6)	m2		34

(continued)

2) t= 1.90 cm

- mortar (1:2)	m2	77
- (1:3)	m2	61
- (1:4)	m2	54
- (1:6)	m2	43

7. Form Work

1) Wooden form

- t= 2.54cm	m2	127
- t= 2.54cm (for column/stair case/arch,etc.)	m2	140
- t= 3.18cm (for roof slab)	m2	159
- t= 3.18cm (for deck slab/ girder/beem,etc.)	m2	205

2) Metal form

- for foundation,etc.	m2	145
- for wall,etc.	m2	167
- for soffit	m2	323

8. Reinforcement Work

- by wire binding	ton	24,610
- by welding	ton	25,200

9. Road Works

1) Brick flat soling

- 1st class	m2	67
- 2nd class	m2	57

(continued)

2) Brick-on-edge soling		
(herring bone-bond)		
- 1st class	m2	110
- 2nd class	m2	93
3) Brick-on-edge over one brick flat soling		
- 1st class	m2	177
- 2nd class	m2	149
4) Bitumenous carpetting	m2	151
(t= 5.08 cm)		
10. R.C.C. Pipe		
1) φ 300 (t= 3.8 cm, l= 1.83m)	m	344
2) φ 600 (t= 6.35cm, l= 1.22m)	m	760
3) φ 800 (t= 10.0cm, l= 0.91m)	m	1,349
4) φ1000 (t= 13.0cm, l= 0.91m)	m	2,077
11. R.C.C. Pile (1:2:4)		
1) 30cm x 30cm	m	860
2) 35cm x 35cm	m	1,015
3) 40cm x 40cm	m	1,280
12. Steel Sheet Pile (U-II)	m2	3,977
13. Brick mattress	m2	294
(t= 15cm, with 12x10cm mesh hexagonal wire)		

(continued)

14. Timber pile		
1) φ 150	m	187
2) φ 200	m	211
15. Dewatering (21 l/sec)	hr	38
16. Bailing water out (2 cusec)	hr	86
17. Wood Work		
1) Jam/Sheel Korai	m3	14,067
2) Jarul/Garjan	m3	16,260
3) Jack	m3	18,210

Table IX-1-4 Unit Price of Inland Transportation

1. By Truck *)

1) Khulna to Kurigram .... 893 Tk/ton
2) Monjla to Kurigram .... 944 Tk/ton
3) Chittagong to Kurigram .... 1,357 Tk/ton

2. By Railway **)

1) Chittagong to Dhaka .... 840 Tk/ton
2) Dhaka to Kurigram .... 920 Tk/ton

1,760 Tk/ton

Source: *) Schedule of Rates for Transportation of Food Grain, Cements, M.S.Rod, Empty Seeds, Relief Materials by Road, 1989- 90, Ministry of Food

**) Percel Office, Dhaka Railway Station

Table Ⅸ-1-5 Tax Rate of Imported Items

Item	Tax Rate(%) (added to CIF)	Chapter
- Cement	20	25.23
- M.S.rod (steel bar)	50	73.10
- Steel sheet pile	5	73.11
- Pipes and valves	50	73.20
- Pumps	50	84.10
- Motors	20	84.08
- Crane	50	84.22
- Agricultural machinery	100	84.24
- Harvesting and threshing machinery	50	84.25
- Weighing machinery	50	84.20
- Heat-treating machinery	20	84.17
- Sprayer	20	84.21
- Earthworking machinery	50	84.23
- Printing machinery	25	84.34
- Typewriters	50	84.51
- Machine tools	20	84.45
- Hand tools	50	84.49
- Welding tools & Appliance	50	84.50
- Calculators	20	84.52
- Duplicating machines	50	84.54
- Other office machines	20	84.54
- Transformers	50	85.01
- Generators	25	85.08
- Switch gear	100	85.19
- Insulated electric wire	100	85.23
- Pontoons	10	89.01
- Jeeps and Pickups	20	87.02
- Surveying & hydrological instruments	20	90.14
- Instruments for physical or chemical analysis	20	90.25
- Fertilizer	0	31.00
- Furniture	20	94.00

Source: Customs and Sales Tax 1989, Internal Resources Division, Ministry of Finance

2. SUMMARY OF COSTS

(1) Summary of Project Cost
(2) Summary of Pump Station Cost
(3) Summary of Irrigation Canal Cost
(4) Summary of Irrigation Facilities Cost

PROJECT COST COMPONENT

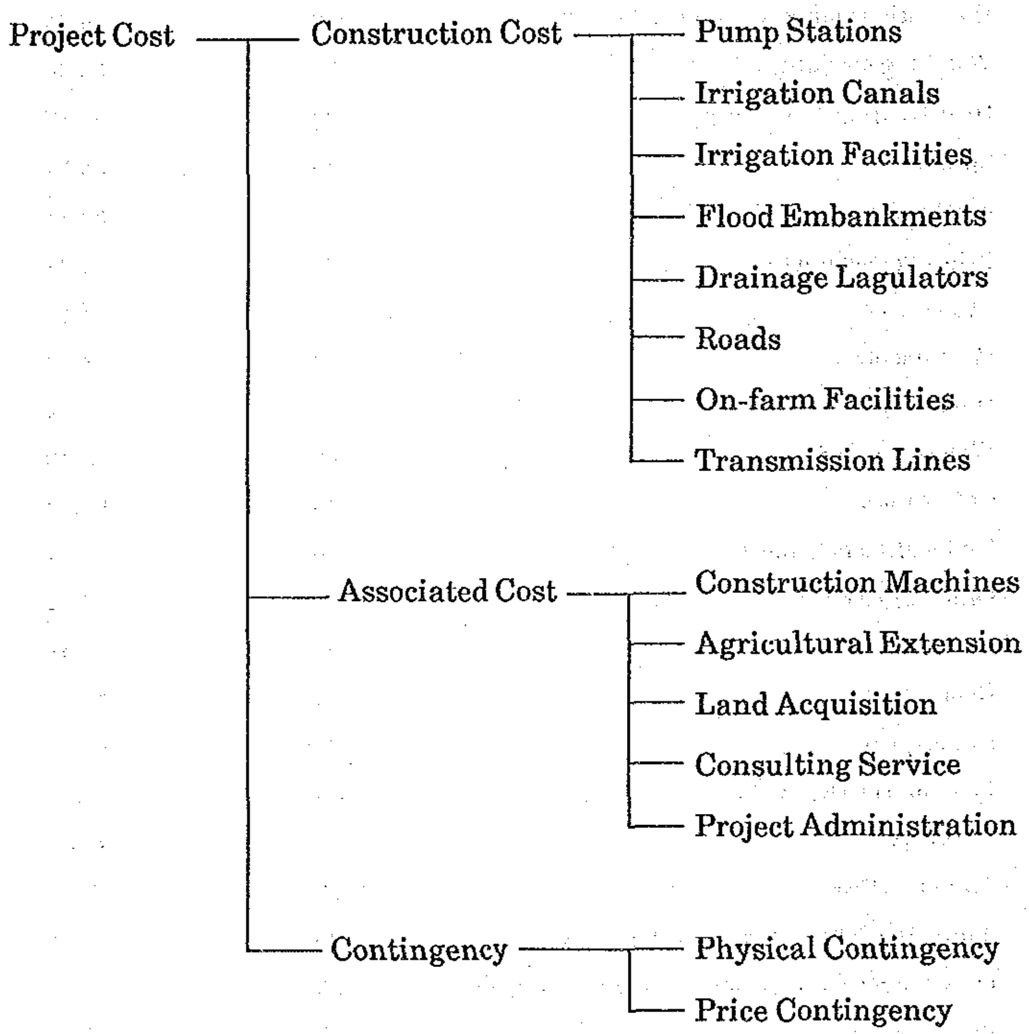

Table IX-2-1 Summary of Project Cost

Description	Cost (× 1,000 Tk)			
	L.C.	F.C.	Tax	Total
I Construction Cost				
1. Pump Stations				
- Main pump staton	156,866	477,035	181,370	815,271
- Reversible pump station	56,435	116,663	41,410	214,508
- Drainage pump station	27,875	91,328	38,050	157,253
Sub-total	241,176	685,026	260,830	1,187,032
2. Irrigation Canals				
- Main pump area	139,274	-	-	139,274
- Reversible pump area	7,285	-	-	7,285
Sub-total	146,559	-	-	146,559
3. Irrigation Facilities				
- Main pump area	176,203	70,018	-	246,221
- Reversible pump area	14,687	4,655	-	19,342
Sub-total	190,890	74,673	-	265,563
4. Flood Embankments				
- Main pump area	35,121	-	-	35,121
- Reversible pump area	15,979	-	-	15,979
Sub-total	51,100	-	-	51,100
5. Drainage Regulators				
- Main pump area	31,949	20,741	-	52,690
- Reversible pump area	5,801	3,708	-	9,509
Sub-total	37,750	24,449	-	62,199
6. Roads				
- Main pump area	2,510	-	-	2,510
- Reversible pump area	5,568	-	-	5,568
Sub-total	8,078	-	-	8,078

Continued

Description	Cost (× 1,000 Tk)			
	L.C.	F.C.	Tax	Total
7. On-Farm Facilities				
- Main pump area	94,534	12,722	-	107,256
- Reversible pump station	10,753	1,447	-	12,200
Sub-total	105,287	14,169	-	119,456
8. Transmission Lines				
- Main pump area	2,801	10,144	4,592	17,537
- Reversible pump area and Drainage Pump	2,004	7,386	3,405	12,795
Sub-total	4,805	17,530	7,997	30,332
Total I.	785,645	815,847	268,827	1,870,319
II. Associated Cost				
1. Construction Machines	3,201	57,258	27,519	87,978
2. Agricultural Extension	12,765	6,605	1,865	21,235
3. Land Acquisition				
- Main pump area	134,698	-	-	134,698
- Reversible pump area	11,976	-	-	11,976
Sub-total	146,674	-	-	146,674
4. Consulting Service	36,608	199,207	-	235,815
5. Administration				
- Administration	38,742	11,552	1,630	51,924
- Overhead (5%)	49,244	-	-	49,244
Sub-total	87,986	11,552	1,630	101,168
Total II.	287,234	274,622	31,014	592,870
III. Physical Contingency	78,884	87,311	29,635	195,830
IV. Price Escalation	354,844	194,703	52,651	602,198
V. Grand Total	1,506,607	1,372,483	382,127	3,261,217

Table Ⅸ-2-2 Summary of Pump Station Cost

Description	Cost (× 1,000 Tk)			
	L.C.	F.C.	Tax	Total
		(C.I.F.)		
(1) Main Pump station				
1. Pump Facilities				
- Pump φ2,200 × 4	-	202,500	101,250	303,750
- Motor 1,250 kw × 4	-	74,100	14,820	88,920
- Valves	-	22,500	11,250	33,750
- Pipes	-	18,000	9,000	27,000
- Crane	-	9,200	4,600	13,800
- Screen & trash machine	-	28,100	14,050	42,150
- Gate	-	15,000	7,500	22,500
- Electric boad and others	-	37,800	18,900	56,700
(Sub-total)	-	407,200	181,370	588,570
2. Inland Freight	2,700	-	-	2,700
3. Installation	31,900	11,600	-	43,500
4. Civil Works				
- Grom work	14,644	6,881	-	21,525
- Riverrevetment and race	36,893	15,732	-	52,625
- Pump station	42,858	25,274	-	68,132
- Settling basin	8,971	2,248	-	11,219
(Sub-total)	103,366	50,135	-	153,501
5. Building Work	18,900	8,100	-	27,000
Total	156,866	477,035	181,370	815,271

(Continued)

Description	L.C.	F.C.	Tax	Total
	Cost (× 1,000 Tk)			
		(C.I.F.)		
(2) Reversible Pump Station				
1. Pump Facilities				
- Pump φ900 × 3	-	32,700	16,350	49,050
- Motor 200 kw × 3	-	7,800	1,560	9,360
- Valves	-	3,300	1,650	4,950
- Pipes	-	3,900	1,950	5,850
- Crane	-	4,700	2,350	7,050
- Screen & trash machine	-	10,900	5,450	16,350
- Electric boad and others	-	24,200	12,100	36,300
(Sub-total)	-	87,500	41,410	128,910
2. Inland Freight	680	-	-	680
3. Installation	7,900	4,900	-	12,800
4. Civil Works				
- Riverrevetment and head race	15,880	8,884	-	24,764
- Pump station	17,693	7,652	-	25,345
- Settling basin	3,775	2,231	-	6,006
- Regulator	6,307	3,696	-	10,003
(Sub-total)	43,655	22,463	-	66,118
5. Building work	4,200	1,800	-	6,000
Total	56,435	116,663	41,410	214,508

(Continued)

Description	Cost (× 1,000 Tk)			
	L.C.	F.C.	Tax	Total
		(C. I. F.)		
(3) Drainage Pump Station				
1. Pump Facilities				
- Pump ϕ900 × 3	-	29,900	14,950	44,850
- Motor 100 kw × 3	-	4,000	800	4,800
- Valves	-	3,300	1,650	4,950
- Pipes	-	3,900	1,950	5,850
- Crane	-	4,700	2,350	7,050
- Screen & trash machine	-	10,900	5,450	16,350
- Electric boad and others	-	21,800	10,900	32,700
(Sub-total)	-	78,500	38,050	116,550
2. Inland Freight	610	-	-	610
3. Installation	6,500	4,600	-	11,100
4. Civil Works	16,565	6,428	-	22,993
5. Building work	4,200	1,800	-	6,000
Total	27,875	91,328	38,050	157,253

Table IX-2-3 Summary of Irrigation Canal Cost

Description	Cost (× 1,000 Tk)		
	L.C.	F.C.	Total
(1) Main Pump Area			
1. Main canal	74,650	-	74,650
2. Secondary/sub-secondary canal	38,534	-	38,534
3. Tertiary canal	26,090		26,090
Total	139,274	-	139,274
(2) Reversible Pump Area			
1. Secondary/sub-secondary canal	4,701	-	4,701
2. Tertiary canal	2,584	-	2,584
Total	7,285	-	7,285

Table IX-2-4 Summary of Irrigation Facilities

Description	Cost (× 1,000 Tk)		
	L.C.	F.C.	Total
(1) Main Pump Area			
1. Bifurcation	22,121	11,026	33,147
2. Check	18,064	9,840	27,904
3. Escape	10,846	6,672	17,518
4. Syphon	10,290	5,414	15,704
5. Aqueduct	9,341	3,771	13,112
6. Road crossing	79,972	23,461	103,433
7. Drainage crossing	25,569	9,834	35,403
Total	176,203	70,018	246,221
(2) Reversible Pump Area			
1. Bifurcation	2,276	1,130	3,406
2. Escape	136	32	168
3. Aqueduct	823	373	1,196
4. Road crossing	8,809	2,124	10,933
5. Drainage crossing	2,643	996	3,639
Total	14,687	4,655	19,342

3. BREAKDOWN OF COSTS

(1) Cost Breakdown of Civil Works for Pump Station
(2) Cost Breakdown of Irrigation Canal
(3) Cost Breakdown of Irrigation Facilities
(4) Cost Breakdown of Flood Embankment
(5) Cost Breakdown of Drainage Ragulator
(6) Cost Breakdown of Road
(7) Cost Breakdown of On-farm Facilities
(8) Cost Breakdown of Transmission Line
(9) Cost Breakdown of Construction Machines
(10) Cost Breakdown of Agricultural Extension Facilities
(11) Cost Breakdown of Land Acquisition
(12) Cost Breakdown of Consulting Service
(13) Cost Breakdown of Project Administration

Table IX-3-1 Cost Breakdown of Civil Works for Pump Stations

Work Item	Unit	Q'ty	Unit Price (TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Station						
1. Groin work						
- Earth work	m3	4,200	29	121	-	121
- Brick pitching	m2	8,720	150	1,308		1,308
- S.C. block	m3	8,720	1,606	7,548	6,456	14,004
- Wire net	m2	10,900	80	447	425	872
- Timber pile (φ200x8.0m)	nos	480	1,688	810	-	810
- Timber pile (φ200x5.0m)	nos	790	1,055	833	-	833
-Wooden board	m3	220	16,260	3,577	-	3,577
(Sub-total)				14,644	6,881	21,525
2.River revetment & head race						
- Excavation	m3	246,900	29	7,160	-	7,160
- Earth filling	m3	17,300	18	311	-	311
- Turfing	m2	4,180	2	8	-	8
- Brick chips	m3	6,300	542	3,414	-	3,414
- Brick pitching	m2	31,500	150	4,725	-	4,725
- S.C. block	m3	21,060	1,606	18,230	15,592	33,822
- C.C.	m3	220	1,829	262	140	402
- Form	m2	900	127	114	-	114
- Timber pile (φ200x5.0m)	nos	450	1,055	474	-	474
-Wooden board	m3	135	16,260	2,195	-	2,195
(Sub-total)				36,893	15,732	52,625

(continued)

Work Item	Unit	Q'ty	Unit Price (Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
3. Pump station						
- Earth filling	m3	62,470	18	1,124	-	1,124
- Turfing	m2	1,510	2	3	-	3
- R.C.C.	m3	6,970	2,446	10,416	6,632	17,048
- C.C.	m3	254	1,829	303	162	465
- Form	m2	15,430	167	2,576	-	2,576
- Reinf. bar	ton	627	24,610	7,530	7,900	15,430
- R.C.C. pile (400x400x16m)	nos	304	20,480	4,396	1,830	6,226
- R.C.C. pile (300x300x10m)	nos	72	8,600	439	180	619
- Sheet pile	m2	1,465	3,977	553	5,273	5,826
- Dewatering	hr	216,000	38	8,208	-	8,208
- Bailing water out	hr	20,000	86	1,720	-	1,720
- Miscel. work (15%)				5,590	3,297	8,887
(Sub-total)				42,858	25,274	68,132
4. Settling basin						
- Earth filling	m3	158,900	18	2,860	-	2,860
- Turfing	m2	8,800	2	18	-	18
- R.C.C.	m3	640	2,416	937	609	1,546
- C.C.	m3	32.0	1,829	38	21	59
- Form	m2	1,650	167	275	-	275
- Reinf. bar	ton	57.6	24,610	692	725	1,417
- Brick pitching	m2	5,800	342	1,983	-	1,983
- Sheet pile	m2	100	3,977	38	360	398
- Gate(2.0x2.5m)	nos	2	600,000	960	240	1,200
- Miscel. work (15%)				1,170	293	1,463
(Sub-total)				8,971	2,248	11,219
Total				103,366	50,135	153,501

(continued)

Work Item	Unit	Q'ty	Unit Price (Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
(2) Reversible Pump Station						
1. River revetment & head race						
- Excavation	m3	75,600	29	2,192	-	2,192
- Brick Pitching	m2	22,000	150	3,300	-	3,300
- S.C. block	m3	12,000	1,606	10,388	8,884	19,272
(Sub-total)				15,880	8,884	24,764
2. Pump station						
- Excavation	m3	9,380	29	272	-	272
- Earth filling	m3	27,400	18	493	-	493
- R.C.C.	m3	2,370	2,446	3,542	2,255	5,797
- C.C.	m3	69.0	1,829	82	44	126
- Form	m2	5,570	167	930	-	930
- Reinf. bar	ton	213	24,610	2,558	2,683	5,241
- R.C.C. pile (400x400x10m)	nos	76	12,800	687	286	973
- Sheet pile	m2	385	3,977	145	1,386	1,531
- Dewatering	hr	144,000	38	5,472	-	5,472
- Bailing water out	hr	14,000	86	1,204	-	1,204
- Miscel. work (15%)				2,308	998	3,306
(Sub-total)				17,693	7,652	25,345
3. Settling basin						
- Earth filling	m3	30,670	18	552	-	552
- Turfing	m2	3,300	2	7	-	7
- R.C.C.	m3	130	2,416	190	124	314
- C.C.	m3	15.0	1,829	18	9	27
- Form	m2	460	167	76	-	76
- Reinf. bar	ton	11.7	24,610	140	147	287
- Brick pitching	m2	1000	171	171	-	171

(continued)

Work Item	Unit	Q'ty	Unit Price (Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
- R.C.C. pile (300x300x20m)	nos	48	17,200	586	240	826
- Sheet pile	m2	295	3,977	111	1,062	1,173
- Gate(2.0x2.2m)	nos	2	520	832	208	1,040
- Gate(2.5x2.5m)	nos	1	750	600	150	750
- Miscel. work (15%)				492	291	783
(Sub-total)				3,775	2,231	6,006
4. Regulator						
- Excavation	m3	5,022	29	146	-	146
- Earth filling	m2	3,800	18	68	-	68
- R.C.C.	m3	460	2,416	673	438	1,111
- C.C.	m3	33.0	1,829	39	21	60
- Form	m2	1,410	167	235	-	235
- Reinf. bar	ton	41.4	24,610	497	521	1,018
- Brick pitching	m2	1,900	171	324	-	324
- R.C.C. pile (400x400x10m)	nos	58	12,800	524	218	742
- Sheet pile	m2	440	3,977	166	1,584	1,750
- Dewatering	hr	24,000	38	912	-	912
- Bailing water out	hr	2,000	86	172	-	172
- Gate(2.0x3.0m)	nos	720,000	3	1,728	432	2,160
- Miscel. work (15%)				823	482	1,305
(Sub-total)				6,307	3,696	10,003
Total				43,655	22,463	66,118

(continued)

Work Item	Unit	Q'ty	Unit Price (Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
(3) Drainage Pump Station						
- Excavation	m3	11,500	29	333	-	333
- Earth filling	m2	25,000	18	450	-	450
- R.C.C.	m3	2,270	2,416	3,324	2,160	5,484
- C.C.	m3	81.0	1,829	97	51	148
- Form	m2	5,600	167	935	-	935
- Reinf. bar	ton	203	24,610	2,438	2,557	4,995
- Brick pitching	m2	1,500	171	256	-	256
- R.C.C. pile (400x400x10m)	nos	106	12,800	958	398	1,356
- Sheet pile	m2	100	3,977	38	359	397
- Dewatering	hr	115,000	38	4,370	-	4,370
- Bailing water out	hr	11,000	86	946	-	946
- Gate(1.5x1.8m)	nos	108,000	3	259	65	324
- Miscel. work (15%)				2,161	838	2,999
Total				16,565	6,428	22,993

Table IX-3-2 Cost Breakdown of Irrigation Canal

Work Item	Unit	Q'ty	Unit Price (TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Area						
1. Main canal						
- Type M-1	m	4,400	4,299	18,915	-	18,915
- Type M-2	m	5,200	1,506	7,831	-	7,831
- Type M-3	m	12,200	1,424	17,372		17,372
- Type M-4	m	1,400	1,300	1,820	-	1,820
- Type M-5	m	4,200	1,176	4,939	-	4,939
- Type M-6	m	6,200	1,219	7,557	-	7,557
- Type M-7	m	4,800	1,408	6,758	-	6,758
- Type M-8	m	500	1,096	548	-	548
- Type M-9	m	4,500	1,077	4,846	-	4,846
- Type M-10	m	4,000	1,016	4,064	-	4,064
(Sub-total)				74,650	-	74,650
2. Secondary/sub- secondary canal						
- Type S-1	m	4,600	481	2,212	-	2,212
- Type S-4	m	5,300	362	1,918	-	1,918
- Type S-5	m	13,900	346	4,809	-	4,809
- Type S-6	m	21,900	300	6,570	-	6,570
- Type S-7	m	28,800	274	7,891	-	7,891
- Type S-8	m	23,500	207	4,864	-	4,864
- Type S-9	m	1,300	333	432	-	432
- Type S-10	m	10,500	317	3,328	-	3,328
- Type S-11	m	3,000	288	864	-	864
- Type S-14	m	22,300	193	4,303	-	4,303
- Type S-15	m	7,300	184	1,343	-	1,343
(Sub-total)				38,534	-	38,534

(continued)

Work Item	Unit	Q'ty	Unit Price (TK)	Cost (× 1,000 Tk) L.C	F.C	Total
3. Tertiary canal						
- Type T-1	m	187,300	81	15,171	-	15,171
- Type T-2	m	134,800	81	10,919	-	10,919
(Sub-total)				26,090	-	26,090
(Total)				139,274	-	139,274
(2) Reversible Pump Area						
1. Secondary/sub-secondary canal						
- Type S-3	m	3,000	379	1,137	-	1,137
- Type S-5	m	6,400	346	2,214	-	2,214
- Type S-6	m	4,500	300	1,350	-	1,350
(Sub-total)				4,701	-	4,701
2. Tertiary canal						
- Type T-1	m	23,700	81	1,920	-	1,920
- Type T-2	m	8,200	81	664	-	664
(Sub-total)				2,584	-	2,584
(Total)				7,285	-	7,285

Table IX 3-3 Cost Breakdown of Irrigation Facilities

Work Item	Unit	Q'ty	Unit Cost	Cost (× 1,000 Tk)		
			(× 1,000 Tk)	L.C.	F.C.	Total
(1) Main Pump Area						
1. Bifurcation						
- Type A	nos	13	1,044	8,450	5,122	13,572
- Type B	nos	30	383	7,350	4,140	11,490
- Type C	nos	147	55	6,321	1,764	8,085
Sub-total				22,121	11,026	33,147
2. Check						
- Type A-1	nos	1	9,918	6,634	3,284	9,918
- Type A-2	nos	2	6,685	8,612	4,758	13,370
- Type B	nos	2	2,308	2,818	1,798	4,616
Sub-total				18,064	9,840	27,904
3. Escape						
- Type A-1	nos	2	4,816	5,810	3,822	9,632
- Type A-2	nos	2	2,557	2,792	2,322	5,114
- Type B	nos	33	84	2,244	528	2,772
Sub-total				10,846	6,672	17,518
3. Syphon						
- Type A-1	nos	1	7,540	4,884	2,656	7,540
- Type A-2	nos	1	6,511	4,274	2,237	7,511
- Type B	nos	1	1,653	1,132	521	1,653
Sub-total				10,290	5,414	15,704

(Continued)

Work Item	Unit	Q'ty	Unit Cost (× 1,000 Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
5. Aqueduct						
- Type A-1	nos	1	1,835	1,243	592	1,835
- Type B-1	nos	1	1,106	754	352	1,106
- Type B-2	nos	1	1,810	1,216	594	1,810
- Type B-3	nos	1	1,061	724	337	1,061
- Type B-4	nos	1	584	415	169	584
- Type B-5	nos	1	1,196	823	373	1,196
- Type B-6	nos	5	376	1,380	500	1,880
- Type B-7	nos	14	260	2,786	854	3,640
Sub-total				9,341	3,771	13,112
6. Road crossing						
1) Bridge type						
- Type A-1	nos	2	1,979	2,824	1,134	3,958
- Type A-2	nos	2	1,512	2,162	862	3,024
- Type A-3	nos	2	884	1,302	466	1,768
- Type B-1	nos	1	1,712	1,280	432	1,712
- Type B-2	nos	10	1,169	8,920	2,770	11,690
- Type B-3	nos	4	929	2,836	880	3,716
- Type B-4	nos	2	547	880	214	1,094
- Type C-1	nos	4	1,320	4,016	1,264	5,280
- Type C-2	nos	6	880	4,098	1,182	5,280
- Type C-3	nos	6	727	3,378	984	4,362
- Type C-4	nos	6	500	2,430	570	3,000
- Type C-5	nos	7	440	2,499	581	3,080
- Type D-1	nos	4	824	2,376	920	3,296
- Type E-1	nos	2	869	1,294	444	1,738
- Type E-3	nos	26	479	9,932	2,522	12,454
- Type F-1	nos	3	656	1,503	465	1,968
- Type F-2	nos	4	440	1,404	356	1,760
- Type F-3	nos	29	376	8,932	1,972	10,904
(Sub-total)				62,066	18,018	80,084

(Continued)

Work Item	Unit	Q'ty	Unit Cost (× 1,000 Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
2) Box culvert type						
- Type A-1	nos	4	167	428	240	668
- Type A-2	nos	24	126	1,968	1,056	3,024
- Type A-3	nos	56	97	3,640	1,792	5,432
- Type C-2	nos	9	77	468	225	693
- Type C-3	nos	13	61	533	260	793
(Sub-total)				7,037	3,573	10,610
3) Pipe culvert type						
- Type A-2	nos	3	46	108	30	138
- Type A-3	nos	9	38	279	63	342
- Type B-1	nos	8	39	256	56	312
- Type B-2	nos	101	31	2,626	505	3,131
- Type B-3	nos	304	29	7,600	1,216	8,816
(Sub-total)				10,869	1,870	12,739
Sub-total				79,972	23,461	103,433
7. Drainage crossing						
1) Box culvert						
type	nos	1	1,370	870	500	1,370
- Type A-1	nos	1	853	547	306	853
- Type A-2	nos	2	720	930	510	1,440
- Type A-3	nos	2	586	762	410	1,172
- Type A-4	nos	3	514	1,008	534	1,542
- Type A-5	nos	8	440	2,320	1,200	3,520
- Type A-6	nos	9	351	2,115	1,044	3,159
- Type A-7	nos	1	1,363	855	508	1,363
- Type B-1	nos	1	1,064	671	393	1,064
- Type B-2	nos	1	716	456	260	716
- Type B-3	nos	1	615	394	221	615
- Type B-4				10,928	5,886	16,814
(Sub-total)						

(Continued)

Work Item	Unit	Q'ty	Unit Cost	Cost (× 1,000 Tk)		
			(× 1,000 Tk)	L.C.	F.C.	Total
2) Pipe culvert type						
- Type A-1	nos	6	124	570	174	744
- Type A-2	nos	18	115	1,602	468	2,070
- Type A-3	nos	37	108	3,108	888	3,996
- Type A-4	nos	8	96	608	160	768
- Type A-5	nos	103	79	6,592	1,545	8,137
- Type B-1	nos	4	189	556	200	756
- Type B-2	nos	6	173	768	270	1,038
- Type B-3	nos	9	120	837	243	1,080
(Sub-total)				14,641	3,948	18,589
Sub-total				25,569	9,834	35,403
(2) Reversible Pump area						
1. Bifurcation						
- Type A	nos	2	1,044	1,300	788	2,088
- Type B	nos	1	383	245	138	383
- Type C	nos	17	55	731	204	935
Sub-total				2,276	1,130	3,406
2. Escape						
- Type B	nos	2	84	136	32	168
Sub-total				136	32	168
3. Aqueduct						
- Type B-5	nos	1	1,196	823	373	1,196
Sub-total				823	373	1,196

(Continued)

Work Item	Unit	Q'ty	Unit Cost	Cost (× 1,000 Tk)		
			(× 1,000 Tk)	L.C.	F.C.	Total
4. Road crossing						
1) Bridge type						
- Type E-2	nos	3	566	1,329	369	1,698
- Type E-3	nos	2	479	764	194	958
- Type F-2	nos	13	440	4,563	1,157	5,720
- Type F-3	nos	3	376	924	204	1,128
(Sub-total)				7,580	1,924	9,504
2) Pipe culvert type						
- Type B-2	nos	4	31	104	20	124
- Type B-3	nos	45	29	1,125	180	1,305
(Sub-Total)				1,229	200	1,429
Sub-total				8,809	2,124	10,933
5. Drainage crossing						
1) Box culvert type						
- Type A-5	nos	3	514	1,008	534	1,542
(Sub-total)				1,008	534	1,542
2) Pipe culvert type						
- Type A-1	nos	5	124	475	145	620
- Type A-2	nos	2	115	178	52	230
- Type A-5	nos	11	79	704	165	869
- Type B-1	nos	2	189	278	100	378
(Sub-total)				1,635	462	2,097
Sub-total				2,643	996	3,639

Table Ⅸ-3-4 Cost Breakdown of Flood Embankment

Work Item	Unit	Q'ty	Unit Cost (TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Area						
1. Construction of Embankment						
- Type A-1	m	6,200	1,378	8,543	-	8,543
- Type B-1	m	14,510	902	13,088	-	13,088
(Sub-total)				21,631		21,631
2. Repairs/improvement of Embankment						
- Major repair	m	8,200	825	6,765	-	6.765
- Minor repair	m	26,900	250	6,725	-	6,725
(Sub-total)				13,490		13,490
Total				35,121		35,121
(2) Reversible Pump Area						
1. Construction of Embankment						
- Type A-2	m	400	2,499	999	-	999
- Type B-2	m	970	1,867	1,810	-	1,810
(Sub-total)				2,809		2,809
2. Repairs/improvement of Embankment						
- Major repair	m	4,300	1,400	6,020	-	6,020
- Minor repair	m	14,300	500	7,150	-	7,150
(Sub-total)				13,170		13,170
Total				15,979		15,979

Table Ⅸ-3-5 Cost Breakdown of Drainage Regulator

Work Item	Unit	Q'ty	Unit Cost (×1,000TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Area						
1. Type A	Nos	2	3,286	3,932	2,640	6,572
2. Type B	Nos	1	4,082	2,488	1,594	4,082
3. Type D	Nos	1	8,572	5,225	3,347	8,572
4. Type E	Nos	1	9,797	5,971	3,826	9,797
5. Type F	Nos	1	23,667	14,333	9,334	23,667
Total				31,949	20,741	52,690
(2) Reversible Pump Area						
1. Type B	Nos	1	4,082	2,488	1,594	4,082
2. Type C	Nos	1	5,427	3,313	2,114	5,427
Total				5,801	3,708	9,509

Table Ⅸ-3-6 Cost Breakdown of Road

Work Item	Unit	Q'ty	Unit Cost (TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Area						
1. Type-A	m	2,200	992	2,182	-	2,182
2. Type-B	m	500	655	328	-	328
Total				2,510	-	2,510
(2) Reversible Pump Area						
1. Type-B	m	8,500	655	5,568	-	5,568
Total				5,568	-	5,568

Table IX-3-7 Cost Breakdown of On-farm Facilities

Work Item	Unit	Q'ty	Unit Cost (TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Area						
- On-farm Facilities	ha	29,450	3,642	94,534	12,722	107,256
(2) Reversible Pump Area						
- On-farm Facilities	ha	3,350	3,642	10,753	1,447	12,200

Table IX-3-8 Cost Breakdown of Transmission & Telephone Line

Work Item	Unit	Q'ty	Unit Price (×1,000TK)	Cost (× 1,000 Tk) L.C	F.C	Total
(1) Main Pump Area						
1. Pump station (11.0 kv)	km	5.5	510	605	2,200	2,805
2. Check gates	km	12.0	745	1,956	6,984	8,940
3. Tax (50%)				-	4,592	4,592
4. Tel. line	lps			240	960	1,200
Total				2,801	14,736	17,537
(2) Reversible Pump Area	and Drainage Pump station					
1. Pump station (11.0 kv)	km	17.0	510	1,860	6,810	8,670
2. Tax (50%)				-	3,405	3,405
3. Tel. line	lps			144	1,576	720
Total				2,004	10,791	12,795

Table IX-3-9 Cost Breakdown of Land Construction Machines

	Description	No.	Unit Price (× 1,000 Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Tax
1.	Dragline (0.8m3)	2	7,370	-	14,740	7,370
2.	Crawler crane (25 ton)	2	4,920	-	9,840	4,920
	Crawler crane (50 ton)	1	7,250	-	7,250	3,625
3.	Tower crane	1	6,580	-	6,580	3,290
4.	Vibro hammer (40 kw)	2	1,420	-	2,840	1,420
5.	Generator (270 KVA)	2	1,850	-	3,700	740
6.	Vibratory roller (10 ton)	1	2,640	-	2,640	1,320
	Vibratory roller (1.6 ton)	4	550	-	2,200	1,100
7.	Spare parts (15%)			-	7,468	3,734
8.	Inland freight (250 ton × 1,357 Tk/ton)			339	-	-
9.	Handling charge (5%)			2,862	-	-
	Total			3,201	52,258	27,519

Table Ⅸ-3-10 Cost Breakdown of Agricultural Extension Facilities

Work Item	Unit	Q'ty	Unit Price (×1,000 Tk)	Cost (× 1,000 Tk) L.C.	F.C.	Total
1. Pilot farm	ha	6.0	300	1,800	-	1,800
2. Land acquisition	ha	7.0	125	875	-	875
3. Building						
- Office and accommodation	m2	500	5	2,500	-	2,500
- Store house	m2	300	4.5	1,350	-	1,350
- Miscel. work (20%)				770	-	770
Sub-total				4,620	-	4,620
4. Equipment						
- Office equip.	set	1		500	1,000	1,500
- Farm machinery	set	1		1,000	2,000	3,000
- Furnitures	set	1		400	800	1,200
- communication	set	1		250	500	750
Sub-total				2,150	4,300 (Tax: 1,300)	6,450
5. Vehicle						
- Jeep	nos	3	650	-	1,950	1,950
- Pickup	nos	1	550	-	550	550
- Motorcycle	nos	20	45	-	900	900
Sub-total				-	3,400 (Tax: 565)	3,400
6. Personnel cost (Laboures)	20persons×1,800Tk/month ×12months× 5years			2,160	-	2,160
7. Miscellaneous(10%)				1,160	770	1,930
8. Total				12,765	8,470 (Tax: 1,865)	21,235

Table IX-3-11 Cost Breakdown of Land Acquisition

Work Item	Area	Unit Price	Cost (× 1,000 Tk)		
	(ha)	(× 1,000 Tk)	L.C.	F.C.	Total
(1) Main Pump Area					
1. Pump station	14.4	91	1,310	-	1,310
2. Irrigation canals					
- Main	397	91	36,127	-	36,127
- Sec./sub-sec.	483	91	43,953	-	43,953
- Tertiary	521	91	47,411	-	47,411
3. Flood embankment	64.8	91	5,897	-	5,897
Total			134,698		134,698
(2) Reversible Pump Area					
1. Pump station	5.6	91	509	-	509
2. Irrigation canals					
- Sec./sub-sec.	54.5	91	4,960	-	4,960
- Tertiary	51.9	91	4,723	-	4,723
3. Flood embankment	19.6	91	1,784	-	1,784
Total			11,976	-	11,976

Table Ⅸ 3-12 Cost Breakdown of Consulting Service

Description	Unit	Q'ty	Unit Cost (Tk)	Cost (× 1,000 Tk) L.C.	F.C.
A. Detailed Design Stage					
1. Foreign currency					
1) Remuneration	month	83	550,000	-	45,650
2) Direct cost					
- International travel expense	trip	20	95,000	-	1,900
- International communication cost	month	12	22,000	-	264
- Printing and duplicating cost	l.s			-	800
- Equipment cost	l.s			-	2,000
3) Contingency (10%)				-	5,061
Sub-total				-	55,675
2. Local currency					
1) Remuneration	month	114	45,000	5,130	-
2) Direct cost					
- Per diem allowance					
· Foreign	day	2,490	2,000	4,980	-
· Local	day	3,420	1,000	3,420	-
- Local communication and transportation	l.s.			150	-
- Boring and soil investigation	l.s.			1,600	-
- Office rental fee	month	12	25,000	300	-
3) Contingency (10%)				1,558	-
Sub-total				17,138	-

(Continued)

Description	Unit	Q'ty	Unit Cost (Tk)	Cost (× 1,000 Tk) L.C.	F.C.
B. Construction supervision stage					
1. Foreign currency					
1) Remuneration	month	233	550,000	-	128,150
2) Direct cost					
- International travel expense	trip	16	95,000	-	1,520
- International communication cost	month	64	11,000	-	704
- Printing cost	l.s.			-	110
3) Contingency (10%)				-	13,048
Sub-total				-	143,532
2. Local currency					
1) Remuneration	month	4	45,000	180	-
2) Direct cost					
- Per diem allowance					
· Foreign	day	7,350	2,000	14,700	-
· Local	day	120	1,000	120	-
- Local communication and transportation	l.s.			900	-
- Office rental fee	month	72	25,000	1,800	
3) Contingency (10%)				1,770	-
Sub-total				19,470	-
Total				36,608	199,207

Fig. IX-3-1 Proposed Schedule for Consulting Services

Description	Man-Month		1st. Year			2nd. Year			3rd. Year			4th. Year			5th. Year			6th. Year			7th. Year		
	F	L	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
I. Detailed Design																							
1. Team Leader	11	11																					
2. Hydrologist	2	12																					
3. Irrigation and Drainage Engineer	5	5																					
4. Soil Scientist	3	3																					
5. Geologist	5	5																					
6. Surveyor	5	12																					
7. Design Engineer (Pump)	8	8																					
8. -do- (Pump)	8	8																					
9. -do- (Canal)	8	8																					
10. -do- (Canal)	8	8																					
11. -do-(Architecture)	3	3																					
12. -do- (Mechanical)	4	4																					
13. Construction Planning	3	3																					
14. Cost Estimate	3	3																					
15. Specialist for Tender Document	3	3																					
16. Agronomist	2	10																					
17. Economist	2	8																					
Sub-total	83	114																					
II. Construction Supervision																							
II-1. Tendering																							
1. Team Leader	2	2																					
2. Mechanical Eng.	1	1																					
3. Civil Engineer	1	1																					
4. Cost Estimate	1																						
Sub-total	5	4																					
II-2. Construction																							
1. Team Leader	63																						
2. Pump Engineer	39																						
3. Canal Engineer	44																						
4. Mechanical Eng.	24																						
5. Geologist	16																						
6. Economist	2																						
7. Surveyor	40																						
Sub-total	228	0																					

Table IX-3-13 Cost Breakdown of Project Administration

1. Personnel cost

1) Detailed design stage (×1,000 Tk)
Project office staff
5,000 Tk/month × 7 persons × 12 months = 420

2) Construction stage

a. Superintending engineer
7,000 tk/month × 12 months × 6 years × 1 person = 504

b. Excutive engineer
5,700 Tk/month × 12 months × 6 yeras × 3 persons = 1,231

c. Sub-divisional engineer
4,300 Tk/month × 12 months × 6 years × 8 persons = 2,477

d. Assistant engineer
2,700 Tk/month × 12 months × 6 years × 12 persons = 2,333

e. Sub-divisional office staff

- Sub-assistant	3,100 Tk/mounth	×3 persons	=	9,300Tk
- Accountant	2,700 Tk/mounth	×3 persons	=	2,700 Tk
- Clerk	2,300 Tk/month	× 1 person	=	2,300 Tk
- Typist	2,100 Tk/month	× 1 person	=	2,100 Tk
- Surveyer	2,000 Tk/month	× 2 persons	=	4,000 Tk
- Stone keeper	1,800 Tk/month	× 1 person	=	1,800 Tk
- Driver	2,500 Tk/month	× 1 person	=	2,500 Tk
- Tracer	1,800 Tk/month	× 2 persons	=	3,600 Tk
- Chaukider	2,100 Tk/month	× 1 person	=	2,000 Tk
- Night guard	2,000 Tk/month	× 3 persons	=	6,000 Tk
- Sweeper	1,500 Tk/month	× 1 person	=	1,500 Tk
- Orderly peon	1,800 Tk/month	× 1 person	=	1,800 Tk
	Sub-total			39,600 Tk

39,600 Tk/mounth × 12 months × 6 years ×8 nos. = 22,810

f. Superintending engineer's office staff
39,600 Tk/month × 12 months × 6 years = 2,851

3) Personel cost total 32,626

2. Equipment cost for construction supervision (× 1,000 Tk)

- Jeep (4 × 4)	650,000 Tk × 6 nos. =	3,900
- Pickup	550,000 Tk × 3 nos . =	1,600
- Motorcycle (70cc)	45,000 Tk × 12 nos. =	540
- Theodlite	130,000 Tk × 2 nos. =	260
- Leveling instrument	50,000 Tk × 6 nos. =	300
- Compass	25,000 Tk × 6 nos =	150
- Current meter	75,000 Tk × 2 nos. =	150
- Transceiver	300,000 Tk × 1 no. =	300
- Walkie-talkie	20,000 Tk × 10 nos. =	200
- Personal computer	350,000 Tk × 1 no. =	350
- Duplicating machine	150,000 Tk × 1 no. =	150
- Spare parts (10%)		720
	Total	8,670
		(Tax; 1,630)

3. Repair and maintenance cost

- Vehicle repair 5,550,000 Tk × 10% × 6 years =	3,330
- Vehicle fuel 7.2 Tk/1 × 151/day × 300 days × 9 nos. × 6 years =	1,750
- Office supply (10%)	508
Total	5,588

4. Training cost
- 4 persons/year × 6 years = 24 persons
- 20 days/1 time

1) Foreign currency portion		
- International travel expenses	80,000 Tk × 24 =	1,920
- Accommodation charge	2,200 Tk × 20 days × 24 =	1,056
- Allowance	1,000 Tk × 20 days × 24 =	480
- Domestic transportation charge	22,000 Tk × 2 × 24 =	1,056
2) Local currency portion		
- Domestic transportation charge	2,000 Tk × 24 =	48
- Allowance	1,000 Tk × 20 days × 24 =	480
3) Training cost total		5,040

5. Total cost for project administration 51,924