2.5 Hydraulic Design of Sand Settling Basin

2.5.1 Necessity and Location of Sand Settling Basin

The sand blown into the conveyance canal will flow to downstream, if the transport capacity of the conveyance canal is larger than the critical friction force as discussed previous section of 2.2. However, the sand will settle where the transport capacity is less than the critical friction force and the sedimentation will occur and blockade the flow area of the conveyance canal. A sand settling basin shall be provided at the upstream of such a section with potential sedimentation.

On the other hand, operation of pumps for sand mixed water will cause the abrasion of pump units which will induce the decrease of efficiency and the trouble. Due to sand will blown into the conveyance canal of open canal section where the drifting sand dune is not prevailing, the sand settling basin shall be provided at immediate upstream of suction sump of No.7 Pumping station.

(a) Hydraulic Dimension of Open Canal and Box Culvert
The hydraulic dimensions of open canal and box culvert are shown in the following tables. The maximum discharge and the minimum discharge correspond to the discharge in July and November.

Table 2.5-1 Hydraulic Dimensions of Open Canal and Box Culvert

	Symbol	unit	Open Canal		Box Culvert	
			Min. Discharge	Max. Discharge	Min. Discharge	Max. Discharge
Design Discharge	Q	m^3/s	8.09	32.48	8.09	32.48
Water Depth	H	m	1.17	2.55	0.84	2.29
Flow Area	A	m^2	16.78	43.61	12.43	33.89
Wetted Perimeter	P	m	17.23	23.40	21.52	33.12
Hydraulic Radius	R	m	0.974	1.864	0.578	1.023
Roughness Coefficient	1/n		55	55	67	67
Hydraulic Gradient	I		1 / 12,500	1 / 12,500	1 / 5,000	1 / 5,000
Velocity	V	m/s	0.484	0.745	0.658	0.962
Friction Velocity	U_*	cm/s	2.76	3.82	3.37	4.48
Critical Tractive Particle Size	d_c	mm	1.0	1.8	1.4	2.5
Max. Particle Size	d_{max}	mm	0.8	0.8	0.8	0.8
Required Friction Velocity	U_{*r}	cm/s	2.10	2.10	2.10	2.10
Safety Factor of Friction	F_f		1.31	1.82	1.60	2.13

(b) Necessity and Location of Sand Settling Basin

The critical friction velocity of the sand particle with maximum diameter 0.8 mm is 2.10 cm/sec, whereas the minimum transport capacity of the open canal or box culvert is 2.76 cm/sec. The sand blown into the open canal will flow to the No.7 Pump station through the open canal and the culvert. In order to avoid the pumping of the sand mixed water, the sand settling basin shall provided at the immediate upstream of the No.7 Pumping station.

2.5.2 Design Conditions

(1) Design Discharge

The design discharges of the sand settling basin shall be determined by the pump operation. The minimum and maximum design discharges are 1 unit and 3 units of the pump capacity, respectively. The design discharges are as following table.

Table 2.5-2 Design Discharges of Sand Settling Basin

Discharge	Pump Operation (unit)	Discharge (m^3/s)
Max. Discharge	3	32.48
Min. Discharge	1	10.83

(2) Hydraulic Design Conditions at BP. of Sand Settling Basin

(a) Designed Elevation of Canal Bottom

Designed elevation of the canal bottom at beginning point of the sand setting basin: KM 108.46618 of fore sight and KM 108.480 of back sight.

Elevation of the Canal Bottom: EL. 7.42 m

(b) Designed Water Level of Canal

Design water levels of the at beginning point of the sand settling basin are shown as following table. The detailed hydraulic analyses are shown in Appendix A.2.5-1.

Table 2.5-3 Designed Water Level of Canal

	Min. Discharge	Max. Discharge
Discharge (m^3/s)	10.83	32.48
Elevation of Bottom	EL. 7.42 m	EL. 7.42 m
Water Depth	1.39 m	2.55 m
Water Level	LWL. 8.81 m	HWL. 9.97 m

(3) Grain Size of Settling Sediment in diameter:

Grain size of sand dune blown into conveyance canal has been investigated with ranged 0.2 to 0.8 mm.

Maximum Size: $d_{max.}$ = 0.8 mm

Minimum Size: $d_{min.}$ = 0.2 mm

2.5.3 Design of Sand Settling Basin

(1) Bottom Elevation of Sand Settling Basin

The bottom elevation of the sand settling basin at the end sill is decided at EL.4.40 m as the same elevation of the suction sump of the pump station.

(2) Width of Sand Settling Basin

In order to completely settle the sand sediment with diameter of 0.3 mm-0.8 mm, the design velocity of the sand settling basin is decided around 0.15 m/sec.

Table 2.5-4 Velocity in the Sand Settling Basin

Descriptions	Min. Discharge	Max. Discharge
Design Discharge (m^3/s)	10.83	32.48
Width of Settling Basin (m)	36.00	36.00
Effective Water Depth (m)	4.40 (8.80 - 4.40)	5.56 (9.96 – 4.40)
Effective Flow Area (m^2)	158.40	200.16
Velocity of Flow (m/s)	0.068	0.163
Optimum Velocity (m/s)	Approx. 0.15	Approx. 0.15

In consideration of the width of the suction sump of the pumping station, the width of the sand settling basin is decided at 36.0 m of Stage I.
In order to secure the flow even during the extrusion of sand from the sand settling basin, the basin shall be divided into two parts.
The method of division is in proportion to the number of pumps; 4 units which is to be erected in Stage I. The width of the settling sub-basins is decided each 18 m.

(3) Length of Sand Settling Basin

It is planned that the sands with grain size 0.3 mm - 1.0 mm settle completely within the sand settling basin. The necessary length of the sand settling basin is given in the following equation;

$$L = F \cdot H \cdot V / V_g$$

Where, L : Required length of sand settling basin (m)

H : Effective water depth (m)

V : Velocity in sand settling basin (m/s)

V_g: Final speed of settling (m/s)

F : Safety factor of sand settling

Table 2.5-5 Required Length of the Sand Settling Basin

Descriptions	Min. Discharge	Max. Discharge
Design Discharge (m^3/s)	10.83	32.48
Water Depth : H (m)	4.40	5.56
Velocity of Flow : V(m/s)	0.068	0.163
Final Speed : V_g (m/s)	0.022	0.022
Safety factor : F	2.0	2.0
Required length : L (m)	27.20	82.39

Therefor, the length of the sand settling basin is decided at 85.0 m.

(4) Sediment Volume of Sand Settling Basin

The sediment volume of the sand settling basin shall be decided based on the volume of drift sand, the grain size distribution, the volume of sediment in the conveyance canal, the frequency of extrusion of sediment sand, etc.

(a) Volume of Drift Sand in Open Canal

Expected volume of sand would be estimated based on following observation, canal length and some assumptions.

- Deposited sand in the open canal is frequently observed on the one side of inclined wall and on the half-length of bed. (H_c = 4.50m, SS = 1:2.0, L_b = 12.0/2 = 6.0 m)
- The proportion of deposited canal length is supposed 10 % in totally.
- Mean deposited thickness is measured 0.35m (0.2 to 0.5 m).
- Proposed open canal length is 14.6 km

Sediment volume in the canal

$$V_o = (5.0^{0.5} \times 4.50 + 12.0/2) \times 0.35 \times 14{,}600 \times 10\ \% = 8{,}200\ m^3./year$$

(b) Sediment Volume in Settling Basin

Depend on grain size distribution test, mean percentage passing of d=0.3 mm is reported as 46 %.

	Percentage passing of d = 0.3 mm
No.1 Sample (KM 72 from Suez Siphon)	56 %
No.2 Sample (KM 25 from Suez Siphon)	36 %
No.3 Sample (KM 15 from Suez Siphon)	47 %
Average :	46 %

Sediment volume in the sand settling basin

$$V_1 = V_o \times (1-0.46) \times Q_1/Q_t = 8{,}200 \times 0.54 \times 32.48/52.66 \fallingdotseq 2{,}700\ m^3./year$$

(c) Design sediment depth of settling basin

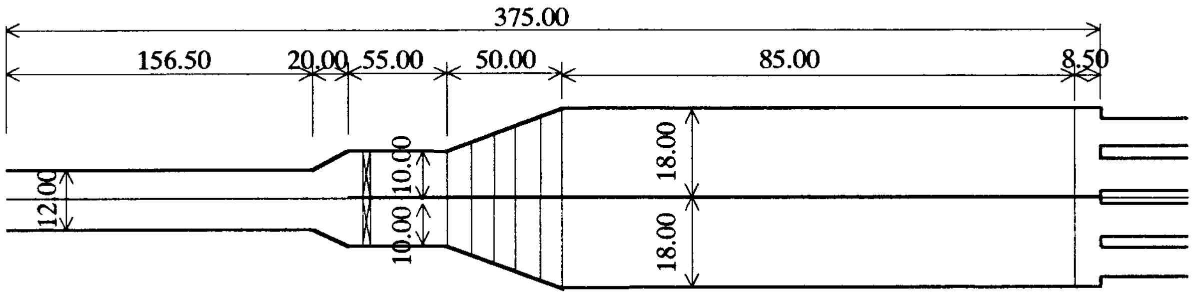
- Removal of deposited sand — One time per year
- Effective length of basin — Le = 85.0 m
- Safety factor — F=1.5

Design sediment depth: $D = V_1 / (W \cdot L) = 2{,}700 \times 1.5 / (36.0 \times 85.0) = 1.32$ m $\Rightarrow$ 1.50m

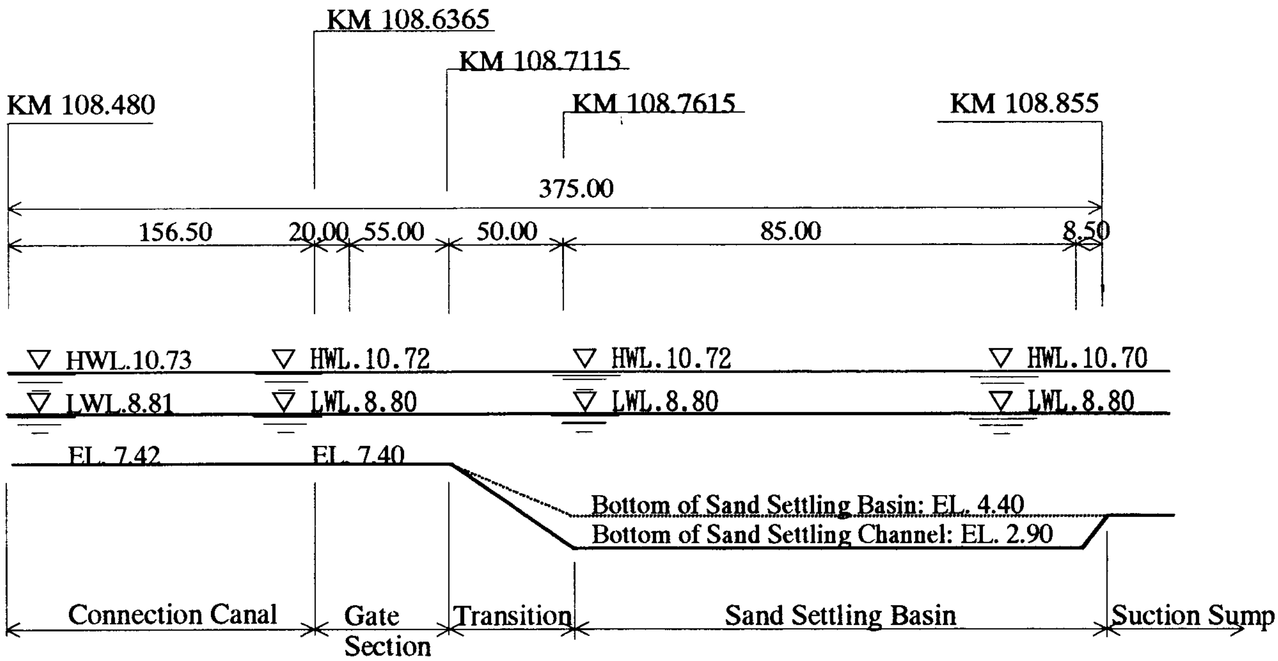
Design bottom elevation of settling basin:
Bottom elevation = EL.4.40 m - 1.50 m = EL.2.90 m

(5) Hydraulic Analysis of Sand Settling Basin

(a) Plan and Profile of Sand Settling Basin



Plan



Profile

(b) Designed Water Level at Each Station

The loss heads of the each section are as follows, and the detailed hydraulic analyses are shown in Appendix A.2.5-1.

- Loss head between KM 108.480 and KM 108.6365

  Min. & Max. Discharge: H_1 = 156.50 x 1/12,500 = 0.01 m
- Loss head between KM 108.6365 and KM 108.7115

  Min. Discharge: H_2 = 75.00 x 1/2 x (1/12,500 + 1/22,768) = 0.00 m

  Max. Discharge: H_2'= 75.00 x 1/2 x (1/12,500 + 1/31,771) = 0.00 m
- Loss head between KM 108.7115 and KM 108.7615

  Min. Discharge: H_3 = 50.00 x 1/2 x (1/22,768 + 1/2,500,000) = 0.00 m

  Max. Discharge: H_3'= 50.00 x 1/2 x (1/31,771 + 1/833,333) = 0.00 m
- Loss head between KM 108.7615 and KM 108.855

  Min. Discharge: H_4 = 93.50 x 1/2 x (0.0000004 + 0.0000004) = 0.00 m

  Max. Discharge: H_4'= 93.50 x 1/2 x (0.0000012 + 0.0000012) = 0.00 m

The hydraulic dimensions at each station are shown as following table.

Table 2.5-6 Designed Hydraulic Dimensions at Each Station

Station	Min. Discharge (Qmin. = 10.83 m^3/s)		Max. Discharge (Qmax. = 32.48 m^3/s)	
	Bottom Elevation	Water Level	Bottom Elevation	Water Level
KM 108.48000	EL.7.42 m	LWL.8.81 m	EL.7.42 m	HWL.10.73 m
KM 108.63650	EL.7.41 m	LWL.8.80 m	EL.7.41 m	HWL.10.72 m
KM 108.71150	EL.7.40 m	LWL.8.80 m	EL.7.40 m	HWL.10.72 m
KM 108.71650	EL.4.40 m	LWL.8.80 m	EL.4.40 m	HWL.10.72 m
KM 108.85500	EL.4.40 m	LWL.8.80 m	EL.4.40 m	HWL.10.72 m

According to the above analysis, the maximum and minimum designed water level in the sand settling basin are HWL. 10.70 m and LWL. 8.80 m, respectively.

(6) Removal Method of Deposited Sand in Settling Basin

The dimensions and conditions of the settling basin are summarized as follows.

- Topographical condition

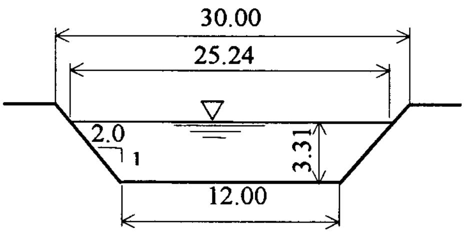
  Settling basin and pumping station are surrounded by hills or sand dune with elevation of EL.30 m approximately and designed ground level of this area is around EL13.00 m.
- Dimensions of settling basin mentioned above are as follows.

Stage	Design Volume	Width	Length	Sediment Depth
Stage I	2,700 cu.m	18.0x2=36.0 m	85.0 m	1.3 m

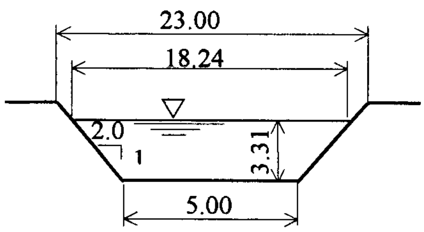
- Elevation of bed level is EL.2.90 m.
- Wheel shovel loader and dump truck can be operated in the basin after dewatering by submerged sand pump.

(7)Cross Section of Connection Canal for Stage II

The cross section of connection canal for stage I will be designed as same as the No.2 open canal with the bottom width of 12.0 m. The cross section of connection canal for stage II will be designed as same the velocity of the stage I. The cross section of connection canal of stage I and II are as follows;



Cross Section of Stage I



Cross Section of Stage II

Table 2.5-7 Hydraulic Dimensions of Connection Canal

	Symbol	Unit	Stage I		Stage II	
			Min. Discharge	Max. Discharge	Min. Discharge	Max. Discharge
Design Discharge	Q	m^3/s	10.83	32.48	10.83	20.18
Bottom Width	B	m	12.00	12.00	5.00	5.00
Water Depth	H	m	1.17	3.31	1.17	3.31
Flow Area	A	m^2	16.78	61.63	8.59	38.46
Wetted Perimeter	P	m	17.24	26.80	10.24	19.80
Hydraulic Radius	R	m	0.973	2.300	0.839	1.942
Roughness Coefficient	1/n		55	55	55	55
Velocity	V	m/s	0.645	**0.527**	1.261	**0.525**
Hydraulic Gradient	I		1/ 7,011	1/ 33,068	1/ 1,505	1/ 26,591

2.6 Hydraulic Design of No.7 Pumping Station

2.6.1 Hydraulic Conditions of No.7 Pumping Station

(1) Design Condition of Pumping Station

Hydraulic design conditions of No.7 Pumping Station can be summarized the following Table 2.6-1 based on the conclusion of basic design of the pumping station and detailed hydraulic analysis of delivery pressured pipeline.

Table 2.6-1 Design Water Level and Discharge of N0.7 Pumping Station

Description	Unit	Hydraulic Condition	Remarks
No. of pump unit -No. of operation unit -No. of standby unit -Total pump unit	Unit	 3 1 4	
Design discharge (Q) -Average discharge per one unit -Total discharge	cu.m/sec	 10.827 32.481	
Suction water level (Hs) -High suction water level -Design suction water level -Low suction water level	m	 9.90 (10.70) 9.90 8.80	The water level in parenthesis indicates after completion of Stage II scheme.
Discharge water level (Hd) -Design discharge water level -Low discharge water level	m	 92.90 92.50	
Design static lifting head (Ha)	m	83.00	Hd – Hs = 92.90-9.90
Hydraulic losses at design point -Pumping Station (Hl) -Pipeline -Total	m	 1.78 14.74 16.52 (=16.60)	 Refer to Table 2.6-3 Refer to Appendix A.2.3-3.
Design total lifting head (Ht)	m	99.60	Ha + Hl = 83.00+16.60

(2) Hydraulic Calculation of Pumping Station

Hydraulic calculation of head losses of the pumping station was carried out by the following conditions;

(a) Discharge for one unit of main pump

Expected actual discharges by combined unit operation of the pump will be estimated by expected pump performance curve which is made standard pump performance curve.(Refer to Figure 2.6-3)

Table 2.6-2 Pump Discharge

Pump operation unit	Total discharge (m^3/s)	Discharge for one unit (m^3/s)	Remarks
3 pumps	32.481	10.827	Design discharge
2 pumps	23.930	11.965	
1 pump	12.900	12.900	

(b) Summary of the hydraulic calculation

The summary of hydraulic calculation is shown in the following Table 2.6-3, and the detailed are tabulated in Appendix A2.6-1.

Screen losses will be considered constant head loss which is calculated at low suction water level (LWL 8.80) and at maximum discharge for one pump unit (12.900m^3/s).

Table 2.6-3 Head Losses of Pumping Station

Description	Unit	Pump operation unit			Remarks
		3 Pumps	2 Pumps	1 Pump	
1.Discharge					
Total discharge	m^3/s	32.481	23.930	12.900	
One unit discharge	m^3/s	10.827	11.965	12.900	
2.Head losses of P.S.					
Screen	m	0.200	0.200	0.200	
Suction pipe	m	0.219	0.268	0.311	
Delivery pipe	m	0.793	0.965	1.120	
Header Pipe	m	0.344	0.356	0.375	
Pipeline(H.P～KM108.985)	m	0.216	0.120	0.036	
Total head losses of P.S.	m	**1.772**	**1.909**	**2.042**	

2.6.2 Summary of the Selection of Pump Type

(1) General Description

Based on the design discharge and given conditions of conveyance canal stated in previous section, the pump type can be decided. Figure 2.6-1 indicates a general guide for selecting the large scale pump best suited to meet various head and capacity requirements. However, in selecting the type of best pump suited to a particular situation, economics of plant construction, efficiency of the units, and operation and maintenance costs should be considered.

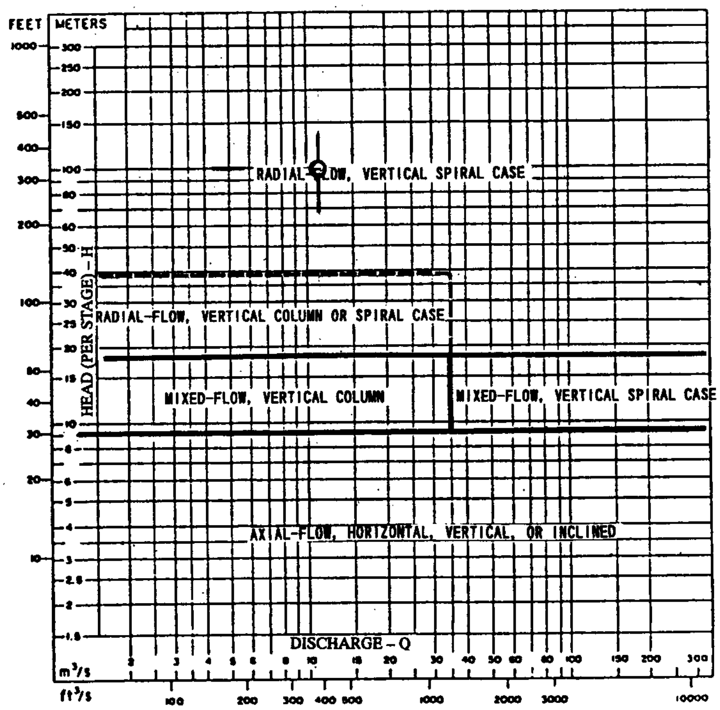


Figure 2.6-1 Large Pump Type Selection Guide
(Bibliography : The USBR Engineering Monograph No. 40)

Following items, in general, shall be studied for design of suitable pumps type based on given conditions.

- Selection of pump types : Centrifugal (radial flow) pump by Figure 2.6-1.
- Determination of pump shaft types : Vertical type by Figure 2.6-1, and shaft arrangement.
- Determination of pump floor types: Concrete embedded casing, suction elbow type because of large capacity pump.
- Determination of No. of pumps : 4 units in Stage I and 3 units in Stage II based on capacity for water demand(one stand-by unit is including in each stage).
- Selection of prime mover and power transmissions : Direct-connected electrical

motor in economical power cost.

- Selection of operation control methods : Manual operation from electric room or machine side, and one man control / on-off control (preset value control) or automatic control (feedback control, preset value control) from field control room.
- Basic layout of pumping facilities : Arrangement of main and auxiliary equipment is shown in later section.

(2) Shaft Arrangement

Horizontal centrifugal pumps are set on a pump floor above maximum water surface elevation to prevent possible flooding of the motors. Each pump is equipped with a flared suction tube extending down into the suction sump. Vertical centrifugal pumps usually have intake tubes and elbows leading from the plant suction sump to the pump. The pump casing is set below normal water surface and may be set on a pump floor or, in the case of large units, embedded in concrete.

Comparison of horizontal and vertical centrifugal pump is shown in Table 2.6-4. MED practices of electric motor installation are to be above maximum water level on the pump house. Therefore, shaft arrangement of the project shall be vertical type.

Table 2.6-4 Comparison Table for Pump Shaft Arrangement

Shaft arrangement / Item	Vertical shaft	Horizontal shaft
Area to be used for pump unit	Narrow space is required than horizontal shaft type	Wide space is required than vertical shaft type
Filling water of Pump	Filling water equipment (Vacuum pump etc.) is not necessary	Filling water equipment (Vacuum pump etc.) is necessary
Suction performance of pump (NPSH)	Good: High speed pump can be adopted due to positive NPSH ava.	No Good: Low speed pump than vertical shaft type will be adopted due to negative NPSH ava.
Readiness for Starting of pump unit	Easier than horizontal shaft type	Complicate than vertical shaft type
Readiness for operation / Maintenance	Pump internal checking is complicate. However, reliability is very high due to less number of auxiliary equipment	Pump internal checking is easy. However, reliability is less than vertical shaft type due to many number of auxiliary equipment.
Manufacturing	In case of large vertical pump the casing will be manufactured by steel plate and embedded in concrete.	So large horizontal pump is not common in the world due to the difficulty of casting of pump casing.
Vibration	Since the pump rotating parts can be suspended vertically, no fatigue strength analyses is required.	In case of so large horizontal pump, the shaft deflection due to own rotating parts weight will cause the fatigue limit somehow.
Pump efficiency	Approx. 91 percent	Approx. 1 percent less than vertical shaft type due to low specific speed and scale effect (smaller than vertical one on impeller size)
Decision	Applied Applicable	Not Applicable

In comparison of vertical and horizontal pump, following items will be effect in view stand of the economical judgement.

1. Pump size : There is restriction for adoption of the large horizontal pump in manufacturing and vibration program so that number of the horizontal pump will increase than one of the vertical pump.
2. Manufacturing : There is limitations for casting of the large horizontal pump casing due to the foundry capacity so that the number of the pump shall be increased and reflects to increase the pump cost. On the other hand, the vertical pump casing can be manufactured by the steel plate, and be no limitation in size on manufacturing.
3. Pump cost : The vertical pump casing is embedded in the concrete so that it weight will be smaller than one of the horizontal pump to be placed on the floor, and be low cost..
4. Construction cost of pump station: The horizontal pump requires the large space than the vertical pump.
5. Motor setting position : If the motor is kept the above suction sump maximum water level, cavitation will be caused. And if the motor is set below the water level, there is a risk that the motor will be immerse in the water.

From the above mentioned reason, **the vertical shaft type pump** was finally selected for the Project.

2.6.3 Study on Cavitation Phenomenon

(1) General

Since pumps are often operated at a point other than the design point because of the difference in water level between the inside and outside or depending on the number of pumps operated, suction specific speed should be found from the maximum capacity (%) in the pump operation range in Figure 2.6-2. If ratio of maximum capacity and planned one is large, pump design point will be shifted to large capacity side within a range which pump efficiency is not affected so that civil work cost for construction of pumping station can be saved.

(2)Determination of Pump submergence

To determine pump installation level and speed, the influence of cavitation should be checked.
Cavitation terms conditions within the pump where, owing to a local pressure drop, cavities filled with water vapor are formed; these cavities collapse as soon as vapor bubbles reach region of higher pressure on their way through the pump. As a result of the study and accumulated experience, pumps now operate at higher speeds and are safer against

cavitation damage such as vibration, noise, drop in efficiency, impeller vane pitting and corrosion fatigue of metals.
The experimental relationship between the impeller eye velocity at cut-off capacity and the suction pressure gives a satisfactory means for predicting cavitation for low specific speed pumps. Thoma's cavitation constant σ be defined as;

$$\sigma = H_{svo} / H = \text{constant}$$

Where,

H_{svo}: Required NPSH (m)

H_{sv} : Available NPSH ($H_s + P_a - P_v - h_l - 0.5 \geqq H_{svo}$) (m)

P_a : Atmospheric pressure (m)

H_s : Suction static head (m)

P_v : Saturated vapor pressure (m)

h_l : Head loss in suction pipe (m)

H : Total head (m)

0.5: Allowance (m)

Suction condition of pumps in respect to cavitation can be expressed by a criterion known as suction specific speed and defined as;

$$S = N\, Q^{1/2} / H_{svo}^{\ 3/4}$$

Where,

S: Suction specific speed (1,400 m-m^3/min. constant for volute pumps)

Q: Capacity at design point (m^3/min)

N: Pump rotational speed (r/min)

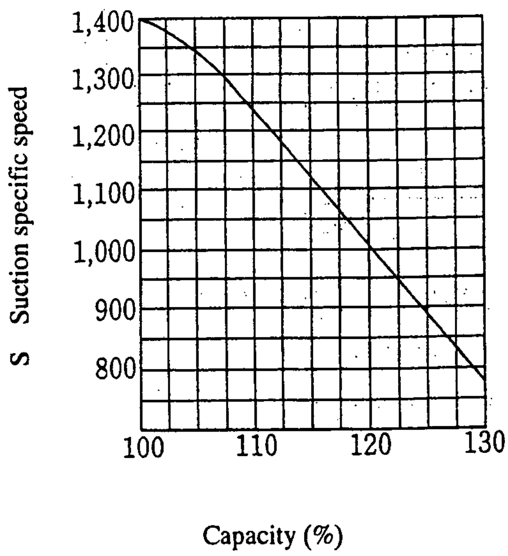


Figure 2.6-2 Suction Specific Speed of a Volute Pump

The submergence of the unit can be found using the pump speed as follows ;

$$Hs \geqq H_{svo} - Pa + Pv + h_l + 0.5 = (N\ Q_d^{1/2}/S)^{4/3} - 10.33 + 0.33 + 0.32 + 0.5$$

$$= (375 \times 714^{0..5} / 1{,}250)^{1.333} - 9.18 = 6.85 \fallingdotseq 7.0\ m$$

Where

Hs : Suction head (m)

Q_d : Capacity at design point (11.90 m^3/sec = 714 m^3/min)

S : Suction specific speed at maximum discharge (12.90/11.90=108.4% design point capacity. S=1,250 m-m^3 / min-r/min)

h_l : Head loss in suction pipe (0.32 m)

Therefore, the pump impeller were set at elevation 1.6 m which deducted suction head (Hs) 7.0 m from lowest suction water level (LSWL 8.60m) at the mouth of suction pipe.
As for determination of capacity at design point , it were decided by try and error method considering pump operation range within. And, since capacity Qd of 11.90 m^3/sec were decided as best efficiency point of the pump, design head can be calculated using laws of pump scaling as follows;

$$(Hd / Hn)^{1/2} = (Qd / Qn)^{1/3}$$

$$Hd = Hn\ (Qd / Qn)^{2/3} = 99.6 \times (10.827 / 11.9)^{0.666} = 93.5\ m$$

Where,

Qn : Rated capacity (10.827 m^3/sec)

Hn : Rated head (99.6 m)

(3) Determination of pump specific speed

Pump specific speed Ns for best efficiency condition under pump speed of 375 r/min is calculated as follows ;

$$Ns = N\ Q_d^{1/2} / H_d^{3/4} = 375 \times 11.9^{0.5} / 93.5^{0.75} = 43.0\ m\text{-}m^3/s\text{-}r/min\ (333\ m\text{-}m^3/min\text{-}r/min)$$

2.6.4 Specification of No.7 Pumping Station

From the above mentioned discussion, specification of main pump and motor of No.7 PS can be summarized in the Table 2.6-5 and the detailed shall be referred Interim Report (2).

Table 2.6-5 Specification of Main Pumps

Description	Unit	Specification	Remarks
1) Pump type			
-Type		Vertical shaft Single suction Centrifugal	
-Number of pump unit (stand-by)	unit	3 + (1)	
-Capacity per unit	m^3/s	10.827	
-Total lifting head	m	99.6	
-Revolution	r/min	375	
2) Main Motor			
-Type		Vertical shaft Synchronous Totally enclosed with CACW cooler	
-Out put	KW	13,000	
-Voltage x Frequency	KVxHz	11x50	
-No. of pole (Revolution)	(r/min)	16(375)	
-Insulation class		F	
-Starting method		Auto-Transformer	

Main pump' expected performance curve which is made by using standard pump performance curve (Ns=330 r/min-m-m^3/s), is shown Figure 2.6-3

2.6.5 Function of Bi-plane Valve and Surge Tank

(1) Bi-plane Valve

The type of valve selected for any given installation will depend primarily on service conditions to be encountered. And it also depends on the initial cost, and cost of maintenance.

In this project, bi-plane type butterfly valve is selected for purpose stopping water, reverse flow prevention and water hammer prevention but not intend to control flow rate because pumps are operating by unit number control methods. Figure 2.6-4 shows cylinder/counter weight driven bi-plane valve to be used as discharge valve and check valve.

The valve is normally open by hydraulic cylinder and closed by counter weight in time of aprox. 120 second. When the pump is accidentally stopped such as power failure and back flow starts, the valve is first closed to prevent most water from backward then slowly closed by the hydraulic cylinder (acting as dash pot) while releasing some of the water. The closing time of the valve is controlled by the hydraulic cylinder (acting as dash pot) after the optimum time has been determined by the analyzing by the transients of the water hammer according to pipeline conditions.

Aim of selection is on the initial cost, and cost of maintenance that bi-plane valve have a cost than the spherical valve.

The bi-plane valve can be used in maximum static head of 300 meter.

(2) Surge Tank

Function of surge tank hydraulics are discussed in the forwarding section of 2.7.4 and 2.7.6.

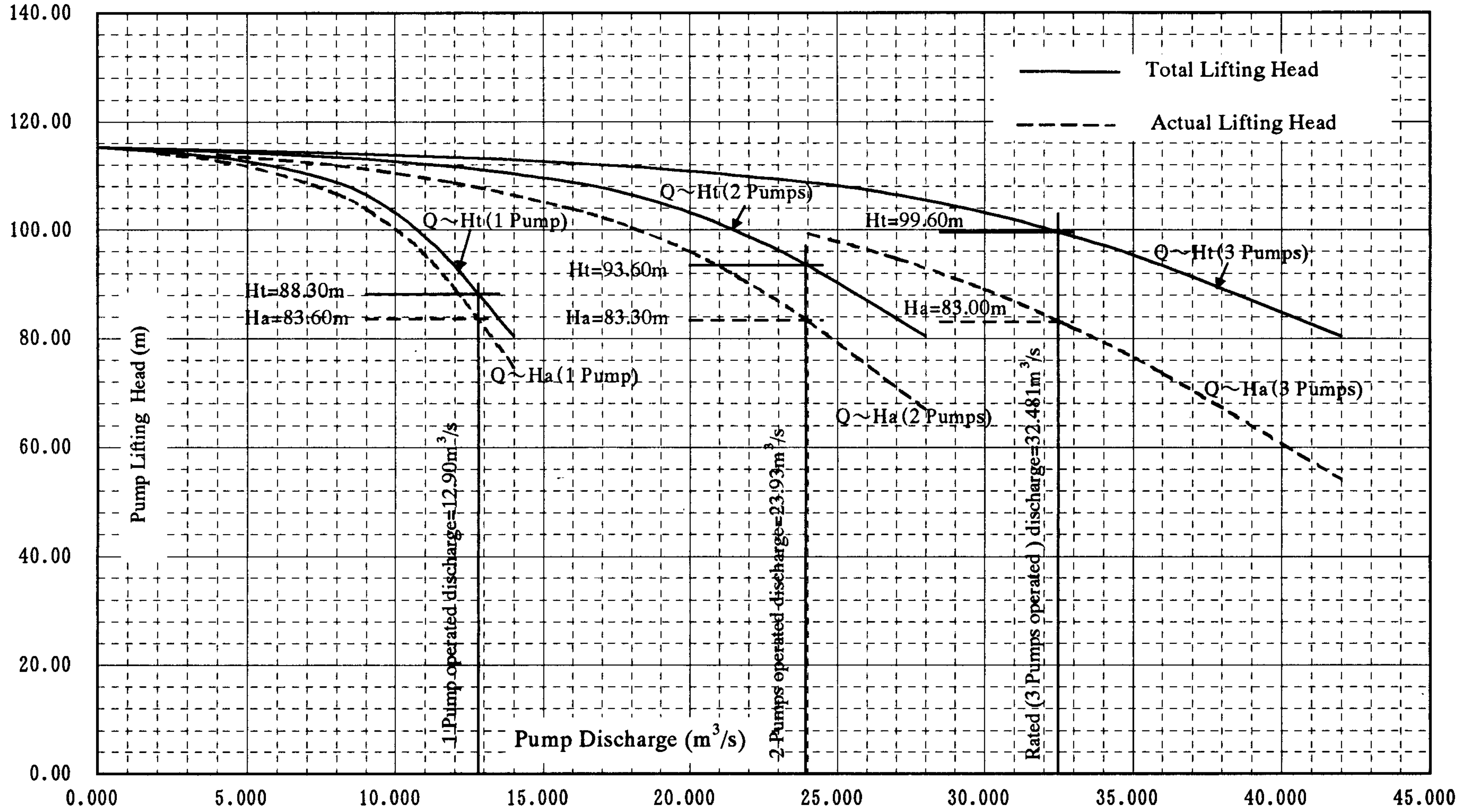


Figure 2.6-3 Expected Pump Performance Curve

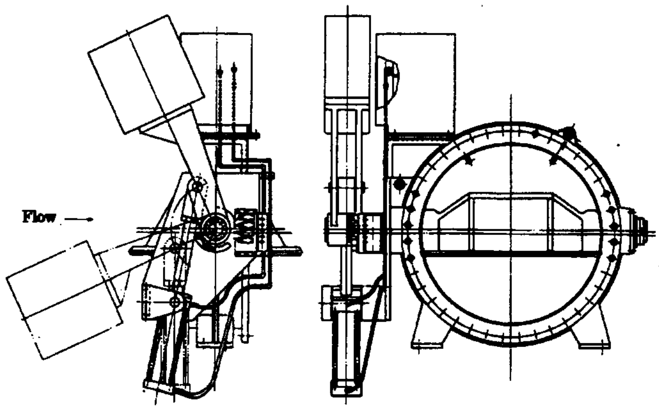


Figure 2.6-4 Bi-plane Valve cylinder/counter operating type

2.6.6 Pump Operation Plan

(1) Water Requirement and Water Distribution Plan

Water users of the project are agriculture (irrigation) and industrial sector. NSDO determined that irrigation water projection shall be followed the proposed land use plan for respective development stages but industrial water is divided into around 50% for each stages. Monthly base water demand projection, therefore, can be tabulated in the Table 2.6-6.

Table 2.6-6 Water Demand Projection

Month	Stage I Irrigation	Stage I Industry	Stage I Total	Stage II Irrigation	Stage II Industry	Stage II Total	Grand Total
Jan.	10.74	2.96	13.70	6.32	2.82	9.14	22.84
Feb.	12.69	2.96	15.65	7.46	2.82	10.28	25.93
Mar.	14.40	2.96	17.36	8.47	2.82	11.29	28.65
Apr.	12.42	2.96	15.38	7.30	2.82	10.12	25.50
May	11.95	2.96	14.91	7.03	2.82	9.85	24.76
Jun.	20.00	2.96	22.96	11.76	2.82	14.58	37.54
Jul.	**29.52**	**2.96**	**32.48**	**17.36**	**2.82**	**20.18**	**52.66**
Aug.	24.63	2.96	27.59	14.49	2.82	17.31	44.90
Sep.	12.42	2.96	15.38	7.30	2.82	10.12	25.50
Oct.	5.61	2.96	8.57	3.30	2.82	6.12	14.69
Nov.	5.13	2.96	8.09	3.02	2.82	5.84	13.93
Dec.	8.79	2.96	11.75	5.17	2.82	7.99	19.74
Average	**14.02**	**2.96**	**16.98**	**8.25**	**2.82**	**11.07**	**28.05**

Source: JICA F/S report 1997.

(2) Simulated Discharge of the Proposed Main Pump

Construction of the No.7 Pumping Station (PS) will be carried out dividing into two stages, for 4 units of Stage I and 3 units of Stage II development including each one unit of standby for respective stage.

Actual operation of No.7 PS in Stage I is proposed simplifying method such as numbers of pump unit and hourly control within the month to meet fluctuated water demand of beneficiaries. As tabulated in the Table 2.6-6, monthly base water demand will fluctuate from 100 % in July to 25 % in November for Stage I and from 100% in July to 26% in November after completion of Stage II development. Due to the high lifting pump with long distance delivery pressured pipelines, discharge capacity of the pumps based on numbers of pump operation buries (increase) about 20 % of the full operation of three units in Stage I for example. This range shall be decided within allowable limit of pump efficiencies. Prospected discharge for each pumps can be tabulated in the Table 2.6-7.

Table 2.6-7 Simulated Discharge of Pump

Description	Unit	Stage I			Stage II	
		1 Pump	2 Pump	3 Pump	1 Pump	2 Pump
1. Total discharge	m^3/s	**12.900**	**23.930**	**32.481**	**12.570**	**21.654**
2. Head losses of Pumping Station						
Screen	m	0.20	0.20	0.20	0.20	0.20
Suction pipe	m	0.31	0.27	0.22	0.30	0.22
Delivery pipe	m	1.12	0.96	0.79	1.06	0.79
Header pipe	m	0.38	0.36	0.35	0.37	0.35
Pipeline (H.P. ~ KM 108.985)	m	0.04	0.12	0.22	0.06	0.22
① **Sub total**	m	**2.05**	**1.91**	**1.78**	**1.99**	**1.78**
3. Head losses of Pipeline (KM 108.985 ~ KM 118.360)						
Diameter x row		ϕ2,400mm x 3 row			ϕ2,400mmx2row	
Discharge of 1 row	m^3/s	4.300	7.977	10.827	6.285	10.827
② **Head losses of Pipeline**	m	**2.65**	**8.36**	**14.74**	**5.40**	**14.78**
4. Total head losses (① + ②)		4.70	10.27	16.52	7.39	16.52
③ **(Round up)**	m	**4.70**	**10.30**	**16.60**	**7.40**	**16.60**
5. Static head						
Discharge water level	m	92.50	92.80	92.90	92.70	92.90
Suction water level	m	8.90	9.50	9.90	9.90	9.90
④ **Static head**	m	**83.60**	**83.30**	**83.00**	**82.80**	**83.00**
6. Total head (③ + ④)						
⑤ **Total head**	m	**88.30**	**93.60**	**99.60**	**90.20**	**99.60**

(3) Pump Operation Manners for the Monthly Water Demand

The required monthly pump operation hours based on the simulated discharge of the main pumps can be tabulated in the Table 2.6-8. Discharge volume of each pump unit are estimated from the Figure 2.6-4.

Table 2.6-8 Monthly Pump Operating Hours

Month	Water Requirement (1000m³)	No.1 Pump		No.2 Pump		No.3 Pump		Total Operating Hours (hr/day)
		Hours (hr/d.)	Discharge Volume (1000m³)	Hours (hr/d.)	Discharge Volume (1000m³)	Hours (hr/d.)	Discharge Volume (1000m³)	
Jan.	1,183.68	24.00	1,108.72	1.74	74.96	0	0.00	25.74
Feb.	1,352.16	24.00	1,094.55	5.98	257.61	0	0.00	29.98
Mar.	1,499.90	24.00	1,082.04	9.70	417.86	0	0.00	33.70
Apr.	1,328.83	24.00	1,096.28	5.40	232.55	0	0.00	29.40
May	1,288.22	24.00	1,099.95	4.37	188.27	0	0.00	28.37
Jun.	1,983.74	24.00	1,040.86	21.89	942.88	0	0.00	45.89
Jul.	2,806.29	24.00	935.43	24.00	935.43	24.00	935.43	72.00
Aug.	2,383.78	24.00	991.73	24.00	991.73	10.27	400.32	58.27
Sep.	1,328.83	24.00	1,096.28	5.40	232.55	0	0.00	29.40
Oct.	740.45	15.94	740.45	0	0.00	0	0.00	15.94
Nov.	698.98	15.05	698.98	0	0.00	0	0.00	15.05
Dec.	1,015.20	21.86	1,015.20	0	0.00	0	0.00	21.86

(a) Effective Volume of Canal as Regulating Function

In order to clarify discrepancy inflow from the upper reach of conveyance canal and simulated discharge of No.7 PS, and allowable regulating water volume in the conveyance canal, the following study was carried out.

In the study, water level dropping limit from the proposed water supply level in each number of pump operation assumed 1 m taking into account safety of side slop of the open canal section. The effective volume as regulating function of the canal for on-off operation of the pumps is tabulated in the Table 2.6-9.

Table 2.6-9 Effective Volume of Regulating Function

Case	Discharge (m^3/s)	Fluctuation of water level (m)	Effective volume of regulating function (m^3)
1 unit operating	12.900	1.00	102,700
2 units operating	23.930	1.00	120,200
3 units operating	32.481	0.97	114,000

(b) Maximum Continuously Operating Hours of Pumps

Intermediate operation of No.7 SP shall be applied in order to adjust discrepancy between water requirement of each month and actual pump lifting capacity. The maximum continuously operating hours of pumps are shown in the Table 2.6-10.

Table 2.6-10 Maximum Continuously Operating Hours of Pumps

Month	Water Requirement (m^3/s)	Pump (unit)	Actual Discharge (m^3/s)	Differential Discharge (m^3/s)	Effective Volume (m^3)	Operating Hours (hr)
Jan.	13.70	2	23.930	10.23	120,200	3.26
Feb.	15.65	2	23.930	8.28	120,200	4.03
Mar.	17.36	2	23.930	6.57	120,200	5.08
Apr.	15.38	2	23.930	8.55	120,200	3.91
May	14.91	2	23.930	9.02	120,200	3.70
Jun.	22.96	2	23.930	0.97	120,200	34.42
Jul.	32.48	3	32.481	0.00	114,000	∞
Aug.	27.59	3	32.481	4.89	114,000	6.48
Sep.	15.38	2	23.930	8.55	120,200	3.91
Oct.	8.57	1	12.900	4.33	102,700	6.59
Nov.	8.09	1	12.900	4.81	102,700	5.93
Dec.	11.75	1	12.900	1.15	102,700	24.81

(c) Maximum Continuously Stopping Hours of Pumps

The maximum continuously stopping hours of pumps are shown in the Table 2.6-11.

Table 2.6-11 Maximum Continuously Stopping Hours of Pumps

Month	Water Requirement (m^3/s)	Pump (unit)	Actual Discharge (m^3/s)	Differential Discharge (m^3/s)	Effective Volume (m^3)	Stopping Hours (hr)
Jan.	13.70	1	12.900	0.80	120,200	41.74
Feb.	15.65	1	12.900	2.75	120,200	12.14
Mar.	17.36	1	12.900	4.46	120,200	7.49
Apr.	15.38	1	12.900	2.48	120,200	13.46
May	14.91	1	12.900	2.01	120,200	16.61
Jun.	22.96	1	12.900	10.06	120,200	3.32
Jul.	32.48	2	23.930	8.55	114,000	3.70
Aug.	27.59	2	23.930	3.66	114,000	8.65
Sep.	15.38	1	12.900	2.43	120,200	13.46
Oct.	8.57	0	0	8.57	102,700	3.33
Nov.	8.09	0	0	8.09	102,700	3.53
Dec.	11.75	0	0	11.75	102,700	2.43

(d) Operation Pattern of No.7 Pumping Plants

The monthly base operation patterns of the No.7 Pumping Station for Stage I, therefore, can be developed from the Table 2.6-8 as shown in the Table 2.6-12. Many operation pattern can be considered from the Table 2.6-8 and Table 2.6-11. Therefore, NSDO shall consider most appropriate operation plan as well as mobilization plan of staff based on the progress and requirement of the Project.

2.6.7 Study on Beginning and End of Pump Operation

(1) Beginning of Pump Operation

Prior to main pump operation at beginning of pump operation, pipeline is requested to fill entirely by water with small rate of discharge. The discharge to fill water is to be less than 10% of design discharge of main pump. The pump system of 2 sets of $9m^3/min.(\phi 200mm$ and pump shaft power of 240kw) for this purpose is studied at Chapter No.7 Pumping Station, and this will be operated to pour individually water into the each pipeline. The required time to pour the water entirely one row of pipeline is estimated approximately 1.7 days / 2 sets of pump. For this purpose, the butterfly valve shall be needed at immediate downstream of header pipe.

(2) End of Pump Operation or at Maintenance of Pipeline

The butterfly valves shall be completely closed at the end of pump operation or for maintenance work of one of the pipeline that may be necessary. In general when irrigation will be stopped and it is needed to keep full by water in the pipeline until beginning of irrigation. On the other hand it is needed to drain water to the settling basin by the blow off pipeline for the maintenance work of each pipeline independently.

Table 2.6-12 Monthly Operating Pattern of No.7 Pumping Station (1/3)

Month	Water Req. (m^3/s)	Required Hours (hr/day)	No. of Pump (unit)	1st day				2nd day				3rd day				4th day			
				6	12	18	24	6	12	18	24	6	12	18	24	6	12	18	24
Jan.	13.70	25.74	No.1																
			No.2																
			No.3																
			No.4																
Feb.	15.65	29.98	No.1																
			No.2																
			No.3																
			No.4																
Mar.	17.36	33.70	No.1																
			No.2																
			No.3																
			No.4																
Apr.	15.38	29.40	No.1																
			No.2																
			No.3																
			No.4																

Table 2.6-12 Monthly Operating Pattern of No.7 Pumping Station (2/3)

Month	Water Req. (m^3/s)	Required Hours (hr/day)	No. of Pump (unit)
May	14.91	28.37	No.1
			No.2
			No.3
			No.4
Jun.	22.96	45.89	No.1
			No.2
			No.3
			No.4
Jul.	32.48	72.00	No.1
			No.2
			No.3
			No.4
Aug.	27.59	58.27	No.1
			No.2
			No.3
			No.4

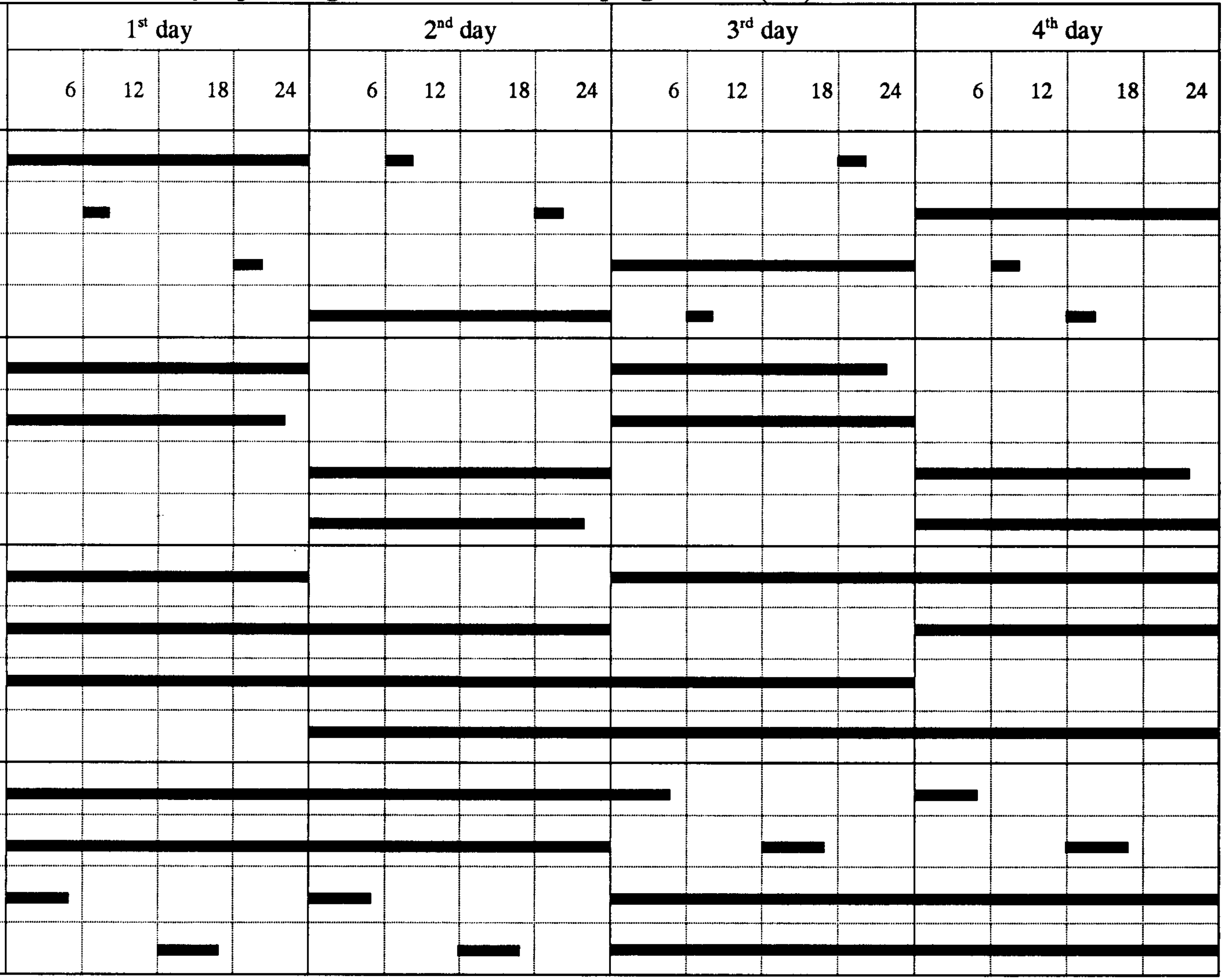


Table 2.6-12 Monthly Operating Pattern of No.7 Pumping Station (3/3)

Month	Water Req. (m^3/s)	Required Hours (hr/day)	No. of Pump (unit)	1st day				2nd day				3rd day				4th day			
				6	12	18	24	6	12	18	24	6	12	18	24	6	12	18	24
Sep.	15.38	29.40	No.1																
			No.2																
			No.3																
			No.4																
Oct.	8.57	15.94	No.1																
			No.2																
			No.3																
			No.4																
Nov.	8.09	15.05	No.1																
			No.2																
			No.3																
			No.4																
Dec.	11.75	21.86	No.1																
			No.2																
			No.3																
			No.4																

2.7 Water Hammer Analysis of Delivery Pressured Pipeline

2.7.1 Water Hammer Analysis and Simulation Program

(1) Transient Phenomena and its Theoretical Analysis at Pump Sudden Stop upon Pump Delivery Pressured Pipeline.

In occasion of pump sudden stop by electrical failure, it has well known that pressure variation due to water hammer in the pipeline and speed variation of the turbine rotor occur, and experiences have established practical method of these analysis.

Water-column separation will be occurred due to the force of inertia of water in the pipeline, it will cause excessive pressure drop and sudden evaporation by the vacuum condition in the pipeline and it results unusual high pressure due to rejoinder of water-column subsequent to water separation.

It is requested to analyze such transient phenomena and to secure the pipeline, pump equipment and other auxiliary machines against such excessive drop and high pressures

(2) Theoretical Analysis

So far the estimation of up and down surge pressure of the water hammer had been done using the simplified calculation chart or such as some formulas given an outline of pressure or volume for surge tanks. Although these are only for the rough estimation and being luck of reliability.

The theoretical analysis of transient phenomena has been established by the following method with computer estimation.

Programming of the theoretical analysis has been composed with the method of characteristics on time series analysis, based on the elasticity mass of pipe and compression of water by the basic equations of motion and continuity.

The program is composed of full consideration of characteristics of facilities relations between functions and its variations for example, such as relations between discharge, pump torque and lifting head in case of pumps, relations between opening, loss head and its variable time in case of valves, and relations between water levels or pressures and variable time in case of each major points of facilities.

(3) Computation Facilities and Functions of Program

Scope of applied facilities and functions of the program are composed the following items;

Several places of suction and discharge work, pumps and its type, pipelines including lateral pipelines, divergences and confluence of pipeline, valves and its type, check valves, air chambers, surge tanks, vacuum breakers and condenser.

2.7.2 Composition of the Input Data

Input dada of the program are as follows:

a. No. of pipe rows; One of 3 rows

b. Time interval; by a second

c. Velocity of sound and pressure wave; by the adequate formulas

d. Atmospheric steam pressure; 10.33m

e. Fluid density; $1.00 g/cm^3$

f. Data of pipeline

No.of division of pipeline; up to 40

Diameter and thickness of division pipe; 2.40m t=22mm

Length of division; 200m to 400m

Head loss of pipeline; approx.15.0m for peak discharge

Kinds of division facilities; inflow, outflow (other valves, divergences)

g. Characteristics of pumps; characteristics of normal and reverse flows of turbine with energy dissipation

h. Characteristics of valves

Coefficient of head loss of valves (relation with valve opening and coefficient)

Characteristics of valve operation (relation with time and valve opening)

Characteristics of check valves (relation with valve opening and coefficient)

Initial pressure, valve torque, weight, radius of leaf, moment of inertia and others

i. Initial water level of suction point; 9.90m at peak discharge

j. Initial water level of discharge point; 92.90m at peak discharge

k. Surge tank data

Section area; assumed by case study

Entrance head loss of surge tanks; assumed by case study

Initial water level of surge tanks; assumed by case study

l. Specific condition

Specific pump head; 100m at peak discharge

Specific discharge; 10.83 cum/s

Specific speed of rotation; 375 rpm

Specific torque; 974 pump axial forth/specific discharge

m. Time for computation; until 100s

n. Initial data for pump; conditions of pump operation

o. Initial discharge; 10.83 cum/s

p. Initial rpm; 375 rpm

q. Pump & motor GD^2 $113 t\text{-}m^2$

r. Time of pump trip; as assumed

2.7.3 Composition of the Output Data

As the result of calculation, the followings may be output shown by the appropriate form of the sheets.

a. Discharge valve data

Time;	
Valve opening;	as assumed by case study
Head loss by valve;	0.8 of velocity head
Discharge	10.83cum/s to minus by analysis

b. Check valve data

Time;	
Valve opening;	as estimation by analysis
Head loss by check valve;	0.8 of velocity head

c. Surge tanks data;

Time;	
Water level of surge tanks;	as estimation by analysis
Entrance head loss of surge tanks;	as estimation by analysis
Water pressure at branch of pipe;	as estimation by analysis

d. Data at specific points of pipeline

Maximum & Minimum pressures and time; as estimation by analysis

e. Pump & pipe data

Time;	
Pump head;	as estimation by analysis
Discharge;	as estimation by analysis
Rotation speed of pump;	as estimation by analysis

Water pressure of specific points of pipeline; as estimation by analysis

2.7.4 Summarized Case Study on Water Hammer Analysis

As stated in the Interim Report (2), two sets of one way type surge tanks are effectively proposed to the pipeline to prevent from abnormally up and down surge pressure by water hammer.

Many case studies have been carried out for determination of surge tank system to cope with the measures of water hammer pressures.

The studies have been carried out by the following steps; (a) valve control condition at immediate down stream of pump, (b) type of outflow condition of discharge tank and proposed number of surge tank, (c) initial water level and section area of surge tank and (d) dimensions of connection pipe of surge tank. The judgements of each analysis are summarized as follows;

And results of the detail studies are shown in Appendix A.2.7-1.

(1) Valve conditions of immediate downstream of pump

A	Check valve controlled closer time of 40 seconds	(○)
B	Check valve proportional closer time of 40 seconds	(△)
C	Check valve non controlled (quick response 12 seconds)	(×)

As shown Appendix A.2.7-1 (1), the case A of controlled closer time (opening 100% to 10% by 10 seconds and opening 10% to 0 % by 30 seconds) is the most adequate because of no vacuum pressure and minimum required capacity of surge tank. And the case B is secondary adequate because of no vacuum pressure but maximum required capacity of surge tank and the case C is rejected because of vacuum pressure of 9 m.
Check valve of the case A is in generally used Bi-plane valve operated by cylinder/counter weight, and the optimum closer time shall be determined by the transient condition of the pipeline as indicated above.

(2) Type of outflow of discharge tank and proposed number of surge tank

A→D One way surge tank 2- proposed and direct flow to discharge tank
with 30cu.m of backward flow (○)
(Ave. $1.0m^3/s \times 30sec = 30m^3$ of reverse flow)
A→E One way surge tank 2- proposed and discharge by flap valve (×)
A→F One way surge tank 2- proposed, discharge by flap valve
and conventional surge tank (△)

As shown Appendix A.2.7-1 (2), the case A→D of two proposed surge tanks and 30 cu.m of required capacity of discharge tank is the most adequate and the case A→F of two proposed one way surge tank, discharge by flap valve and one conventional surge tank is the secondary adequate. Although the case A→F shall not be accepted to the project because of economical view point.

(3) Initial water level and section area for surge tank

A→D→G	ST-1; Initial water level 54m and section area 60sq.m.	(×)
A→D→H	ST-1; Initial water level 56m and section area 60sq.m.	(○)
A→D→I	ST-1; Initial water level 56m and section area 40sq.m.	(×)
A→D→J	ST-2; Initial water level 66m and section area 15sq.m.	(×)
A→D→K	ST-2; Initial water level 68m and section area 15sq.m.	(△)
A→D→L	ST-2; Initial water level 68m and section area 5sq.m.	(○)

As shown Appendix A.2.7-1 (3), the case A→D→H of initial water level 56m with section area 60 sq.m of ST-1 (surge tank No,1) and the case A→D→L of initial water level 68m with section area 5 sq.m of ST-2 (surge tank No,2) is the most adequate because of no vacuum pressure, non of suction air to the main pipe and economical reason. The case A→D →K is the secondary adequate but uneconomical compared to A→D→L.

Judgement
○ ; Adequate
△ ; Inadequate by the reason of uneconomical
× ; Rejected

(4) Size and number of connection pipe of surge tank

Proposed connection pipe
ST-1; Satisfactory ϕ 1500mm×2 units and ϕ 1500mm×1 unit
Unsatisfactory (ϕ 1000～ϕ 1200mm×1unit) supposed to suction of air in the main pipeline
ST-2; Satisfactory ϕ 700mm×2 units and ϕ 700mm×1 unit
Unsatisfactory (ϕ 500mm×1 unit) supposed to suction of air in the main pipeline

2.7.5 Comparison of Theoretical Analysis and Experimental Measures of the Field.

As stated above, the program has been composed of full considerations of characteristics of each facility. The comparison between theoretical analysis and experimental measures of the field will not be done but according to the past many cases of the estimation by this program , the accuracy of the analysis may be satisfactory with certain reliability.
We may submit for instance the comparison results between the theoretical analysis by computer and the measured transient phenomena on three cases in circulating water system for thermal and nuclear power plants which is shown in Appendix A.2.7-2 Theoretical Water Hammer Calculation Comparison with Experimental Result.

2.7.6 System Reliability of Water Hammer Protection

The system of the water hammer protection is consist of bi-plane valve at immediate downstream of main pump and two places of one way surge tank. Reliability of each facility are discussed as follow;

(1) Reliability of the Bi-plane Valve

The function of the bi-plane valve is to be certainly shut the reverse water flow with a rule of closer time whenever water hammer will be occurred. Another function of the valve is to protect pump from reverse water flow. The valve mechanism is rather simple and the valve will actuate by counter weight as hydraulically release oil pressure without any electricity. It is important to maintain valve and valve control panel periodically to actuate in a rule of closer time by the mechanical engineer. Isolating valve mounted at immediate downstream of the bi-plane valve will be used to check the valve leaf it's self and sealing rubber from inside of pipe.

As discussed above, reliability of the bi-plane valve is to keep appropriate maintenance works by the mechanical engineer according to the maker's specification or manuals.

(2) Reliability of the One Way Surge Tank

There are two sets of mechanical component with swing type check valve and ball type float valve at a one way surge tank for one row of pipeline, and these are mechanically very simple and operated by hydraulic force. If one set of equipment does not functioned due to unknown factors or luck of maintenance works, remainder of one set of equipment may be functioned. Then the system will be functioned sufficiently as long as adequate maintenance works. Furthermore, there are some considerations to be safety, that are strainer to clean water to prevent being stuffed float valve by the debris and by-pass pipeline to fill water without float valve if damaged as shown in the drawing DPP-231 and DPP-237.

2.8 Hydraulic Design of Discharge Tank

2.8.1 Hydraulic Conditions of Discharge Tank

(1) Hydraulic Conditions of Discharge Tank

- Design discharge: (Minimum and maximum design discharge are the pump capacity of 1 and 3or 5 units, respectively.)

	Stage I	Stage II (overall)
Max. Discharge	32.48 m^3/sec.	54.14 m^3/sec.
Min. Discharge	10.83 m^3/sec	10.83 m^3/sec.

- Designed elevation of the canal bottom at the end of the Discharge Tank : KM 118.560

Elevation of the canal bottom: EL. 88.57 m

- Design water level at the end of the Discharge Tank : KM 118.560

	Stage I	Stage II
Max. Discharge	HWL. 91.12 m	HWL. 91.94 m
Min. Discharge	LWL. 89.96 m	LWL. 89.96 m

- Required volume of Discharge Tank for countermeasure of water hammer as discussed previous section of 2.7.

$V_t = 30\ m^3 \times 1.2 = 36\ m^3$

(2) Selection of the Location

The location of the discharge tank was decided preliminary in consideration with topography on the conveyance canal, designed water surface, economical feasibility and environmental impacts. The main considerations for selection of the discharge tank are shown below:

- To minimize the length of pipeline for reducing the project cost, which requires a high construction cost.
- To select the place with a comparatively large acreage as the site for the discharge tank.
- To select the place with flat topography, less affection of drifting sand dune, natural vegetation which may indicate stable environmental condition.
- To select the place with suitable ground surface elevation for smooth connection with No.3 open canal.

Based on the considerations mentioned above, a result of field survey/topographic survey, the site of the discharge tank is finally decided as indicated below.

- Station: KM 118.360 (BP + 31.860 km)
- Ground elevation: approximately EL.91 m

(3) Selection of Discharge Tank Type

(a) General

Several types of facilities can be considered to prevent reverse flow from the discharge tank i.e. i) rectangular weir, ii) round weir, iii) siphon, iv) non-return valve. The type of the discharge tank shall be decided in consideration of flow rate of discharge, relation of discharge water level and downstream water level, space of land required, elevation of original ground surface, etc.

(b) Alternatives of Discharge Tank Type

The characteristics of the alternatives of discharge tank are shown in the following table.

Table 2.8-1 Alternatives of Discharge Tank Type

Type	Rectangular Weir type	Round weir type	Siphon type	Non-return valve type
Description	• Straight weir	• Round weir	• Siphon at the down-stream end of the discharge pipe	• Non-return valve at the downstream end of the discharge pipe
Hydraulic character	• Head loss equal to the overflow depth occurs. (Head loss:1.0m) • Deviated flow tends to occur.	• Head loss equal to the overflow depth occurs. (Head loss:1.0m) • Deviated flow hardly occurs.	• Head loss is less than weir type. (Head loss:0.5m) • Deviated flow tends to occur.	• Head loss is small. (Head loss:0.4m) • Deviated flow tends to occur. • Discharge tank shall be requested countermeasure for water hammer, when the pumping system has High pumping head and long length pipeline. • Except two one way surge tanks, additional conven-tional surge tank may be necessary because nega-tive pressure shall be occurred at location behind the flap valve when it closed.
Structural character	• Straight structure ensures easy construction.	• Curved structure result in hard construction	• Curved structure on curved discharge pipe result in hard construction.	• Installation of non-return Valve only makes construction easy.
Past example	• Head loss can be Decreased by Lengthening the weir .Many examples for large discharge and less limita-tion of land	• Largest head loss. As discharged vertically upward, many examples for medium to small discharge.	• Least head loss. Many examples for large discharge with medium to high pumping head in case initial siphon flow are formed by themselves. • No.6 Pumping station of this project	• Little head loss. Many examples for wide range of discharge with Low pumping head. • No.5 Pumping Station of this Project.
Economic	(1,000 L.E.)			
1.Initial Cost	1,109	1,331	1,370	14,377
2.Annual Cost	70	84	87	226
3.Energy Cost	55,820	55,820	55,540	55,483
4.Overall ann. Cost (2+3)	55,890 (1.005)	55,904 (1.005)	55,627 (1.000)	55,709 (1.001)

From the above comparative table, "Rectangular weir type" of discharge tank was adopted because, there are no technical hazard than other alternatives and no different initial cost and energy costs.

2.8.2 Basic Design of Discharge Tank

(1) Basic Concepts of the Discharge Tank

The considerations for discharge tank design are as follows.

(a) Shape and Size of discharge tank

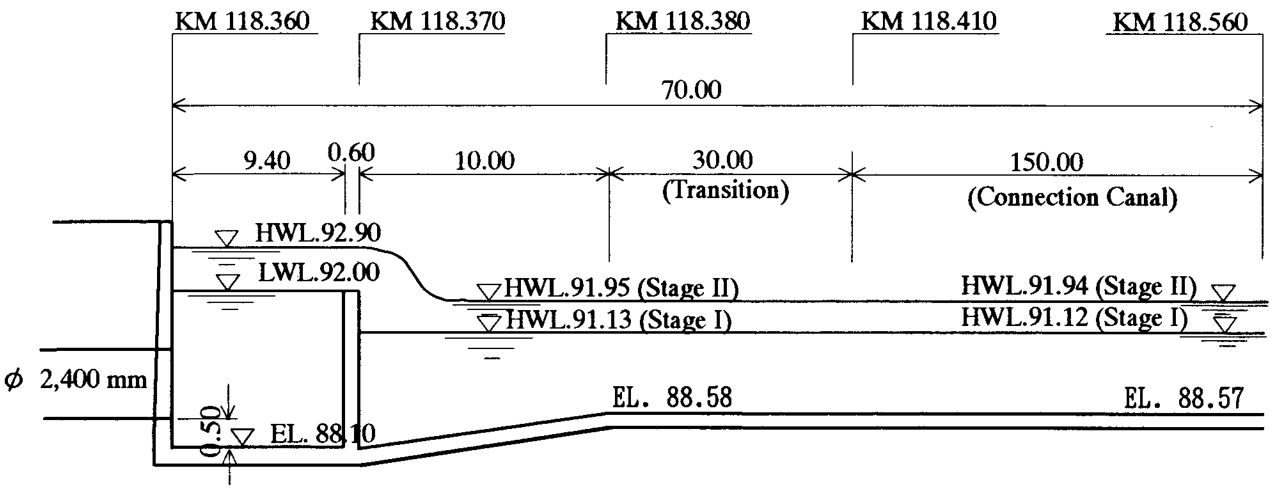
In order to reduce high flow velocity in the discharge tank produced a partial rise in water level and much shape head loss in the tank, such velocity should be reduced in the discharge tank, and then the water flow direction changed so that water is directed to the open canal.

The flow velocity therefor should be below 0.30 to 0.50m/sec in the discharge tank. Insufficient size or improper shape of the discharge tank will produce reflected waves: or step waves on the side wall opposite the discharge pipe. To prevent such waves, a proper distance of the side wall opposite the discharge pipe end should be changed by inclining the wall or by sloping the bottom of the tank. Also, reflected waves produced on the opposite side wall should be prevented from travelling directly to the next pump discharge pipe outlet.

(b) Discharge Tank Sill

The upper end of the discharge pipe should be located at a level 30cm or more below the lowest discharge water level. The discharge tank sill should be about 40 to 60cm below the lower end of the discharge pipe. The width of the discharge tank is often determined on the basis of the relationship between the discharge pipe arrangement and the width of discharge weir.

(2) Basic Design of Discharge Tank



Profile

Dimensions and elevations of discharge tank are as follows.

(a) Width of Weir

In order to reduce high flow velocity in the discharge tank produced a partial rise in water level and much shape head loss in the tank, such velocity should be reduced in the discharge tank, and then the water flow over the weir so that water is directed to the open canal.

In order to keep the velocity in the discharge tank less than 0.3 m/s ~ 0.5 m/s, the crest length of the weir shall be about 3 times of the diameter of the delivery pipe (3 x 2.40 m$\fallingdotseq$7.50 m). 7.5 m width of weir per one unit gives the velocity of discharge tank as follows;

The flow velocity in the discharge tank should be given by following formula.

$V = Q / (W \cdot H)$

where, V : Flow velocity in the discharge tank (m/s)
Q : Discharge of one row pipeline (m^3/s)
W : Width of upstream discharge tank (m)
H : Water depth in the discharge tank (m)
H = HWL. 92.90 m – EL.88.10 m = 4.80 m

$V = 10.827 / (7.50 \times 4.80) = 0.30$ m/s

(b) Length of Discharge Tank

In order to protect reflected waves or step waves on the side-wall opposite the discharge pipe, the length of the discharge tank shall be about 4 times of the diameter of the delivery pipe. Therefor, the length of the discharge tank shall be provided 9.40 m ($\fallingdotseq$ 4 x 2.40 m).

(c) Crest Elevation of Weir

The crest elevation of the weir shall be so planned as not to allow the back-flow from the downstream of the conveyance canal in case of sudden stop of the pump operation. The full water supply level at the immediate downstream of the conveyance canal is EL.91.95 m at the stage II completion. Therefor, the crest elevation of the weir is decided at EL.92.00 m, a little higher than the HWL. 91.95 m.

- Crest Elevation of Weir : EL. 92.00 m ($\fallingdotseq$ HWL.91.95 m)

(d) Upper End Elevation of Delivery Pipe

In order to keep the required volume of discharge tank for countermeasure of water hammer, the upper end elevation of the delivery pipe shall be designed as follows;

$H = Vr / (B \times L) + Hc$

Where, H : Water depth from top of delivery pipe (m)
Vr : Required volume of discharge tank for countermeasure of water hammer. (36.00 m^3)
B : Width of weir (7.50 m)
L : Length of discharge tank (9.40 m)
Hc: Allowance (0.50 m)

$H = 36.00 / (7.50 \times 9.40) + 0.50 = 1.01$ m$\fallingdotseq$ 1.00 m

- Upper End of Discharge Pipe : EL. 91.00 m (= LWL. 92.00 m – 1.00 m)

(e) Discharge Tank Sill

- Discharge Tank Sill : EL.88.10 m (= EL. 91.00 m - 2.40 m - 0.50 m)

(f) Depth of Over Flow at the Weir and Water Surface Level in the Discharge Tank

$D = 1.5\,(q^2/g)^{1/3}$

Where, D : Depth of over flow at the weir (m)
q : Discharge per meter (m^3/s)
$q = 10.827 / 7.50 = 1.444\ m^3/s$
g : Gravitational acceleration (m/s^2)
$g = 9.8\ m/s^2$

$D = 1.5\,(1.444^2/9.8)^{1/3} = 0.90\ m$

Therefor, the high and low water surface level should be 92.90 m (= EL.92.00m + 0.90m) and 92.00m, respectively.

(3) Hydraulic Analysis of Discharge Tank

The loss heads of the each section are as follows, and the detailed hydraulic analyses are shown in Appendix A.2.8-1.

- Loss head between KM 118.560 and KM 118.410
  Stage I & II: H_1 = 150.00 x 1/12,500 = 0.01 m
- Loss head between KM 118.410 and KM 118.380
  Stage I : H_2 = 30.00 x 1/2 x (1/12,500 + 1/41,428) = 0.00 m
  Stage II : H_2'= 30.00 x 1/2 x (1/12,500 + 1/39,691) = 0.00 m
- Loss head between KM 118.380 and KM 118.370
  Stage I : H_3 = 10.00 x 1/2 x (1/41,428 + 1/26,579) = 0.00 m
  Stage II : H_3'= 10.00 x 1/2 x (1/39,691 + 1/18,224) = 0.00 m

The hydraulic dimensions at each station are shown as following table.

Table 2.8-2 Designed Hydraulic Dimensions at Each Station

Station	Stage I: Max. Discharge ($Q_1 = 32.48\ m^3/s$)		Stage II: Max. Discharge ($Q_2 = 54.14\ m^3/s$)	
	Bottom Elevation	Water Level	Bottom Elevation	Water Level
KM 118.560	EL. 88.57 m	HWL. 91.12 m	EL. 88.57 m	HWL. 91.94 m
KM 118.410	EL. 88.58 m	HWL. 91.13 m	EL. 88.58 m	HWL. 91.95 m
KM 118.380	EL. 88.58 m	HWL. 91.13 m	EL. 88.58 m	HWL. 91.95 m
KM 118.370	EL. 88.10 m	HWL. 91.13 m	EL. 88.10 m	HWL. 91.95 m
KM 118.360	EL. 88.10 m	HWL. 92.90 m	EL. 88.10 m	HWL. 92.90 m