8.2 Design of Proposed Pump

1) Basic Design Conditions

a) Pump Capacity and Operation Hours

The capacity of the pumps for irrigation water intake can be computed by two methods ; one is by seasonal maximum water requirements excluding effective rainfall and the other is by curves developed from maximum gross water requirements for 10 (ten) to 20 (twenty) years including effective rainfall.

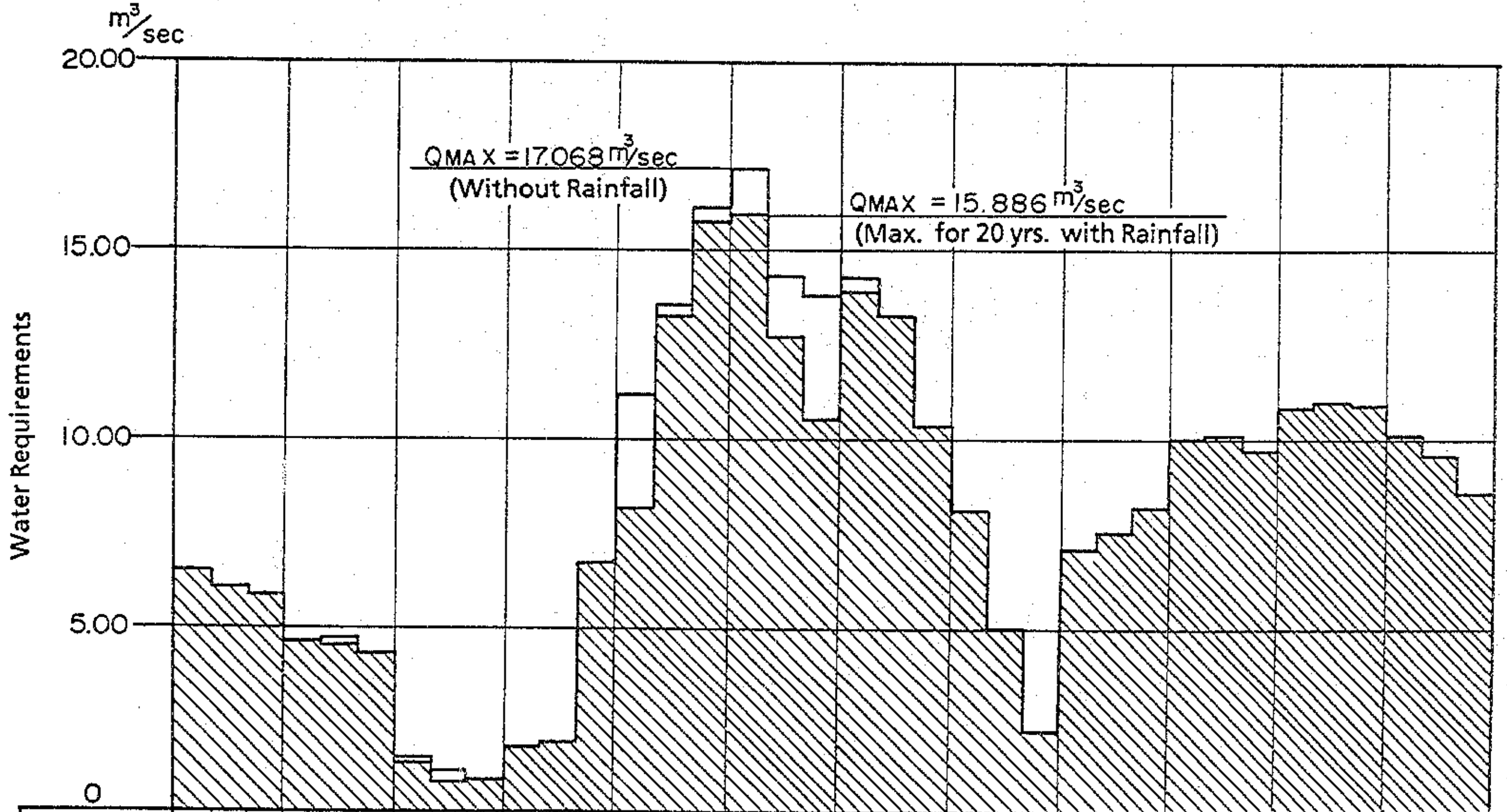
The proposed pump capacities can be estimated by the aforesaid methods as shown below:

Method		(Unit : m^3/s)
i)	Max. water req. excluding rainfall	17.068
ii)	Curve of max. gross water req. for 20 years	15.886
iii)	Curve of max. gross water req. for 10 years	15.630

In the Project Area, the maximum water requirements without effective rainfall appears at the first 10 days of September in the wet season. The pump capacity, therefore, should be reasonably determined based on the gross water requirements with effective rainfall. Consequently, the maximum gross water requirements with effective rainfall included for 10 - 20 years are in the range of 15.630 to 15.886 m^3/s. Considering small difference between the values of 10 years and 20 years, the design pump capacity is estimated by $Q = 16.00\ m^3/s$ taking the curve for the 20-year value with some allowance.

The operation hours of the proposed pumps are planned for 24 hrs/day considering the fact that the facilities are to be provided as an irrigation water resource.

FIGURE 8 - 2 SEASONAL WATER REQUIREMENTS



Q (m^3/sec) (Without Rainfall)	6.550	6.029	5.838	4.595	4.419	4.229	1.387	0.782	0.818	1.712	1.806	6.663
Q (m^3/sec) (With Rainfall)	6.550	6.029	5.838	4.595	4.640	4.229	1.407	1.007	0.827	1.747	1.806	6.663
Month	1	2	3	1	2	3	1	2	3	1	2	3
	4			5			6			7		

Q (m^3/sec) (Without Rainfall)	8.053	13.205	15.630	15.886	12.760	10.438	13.871	13.243	10.366	8.091	4.963	2.104
Q (m^3/sec) (With Rainfall)	11.171	13.584	16.067	17.068	14.312	13.797	14.231	13.243	10.366	8.091	4.963	2.104
Month	1	2	3	1	2	3	1	2	3	1	2	3
	8			9			10			11		

Q (m^3/sec) (Without Rainfall)	7.062	7.473	8.052	9.987	10.080	9.676	10.797	10.937	10.865	10.082	9.558	8.566
Q (m^3/sec) (With Rainfall)	7.062	7.473	8.052	9.987	10.080	9.676	10.797	10.937	10.865	10.082	9.558	8.566
Month	1	2	3	1	2	3	1	2	3	1	2	3
	12			1			2			3		

Hatching area shows max. net water requirements with rainfall for 20 years.

TABLE 8-1 WATER REQUIREMENTS FOR WET SEASON PADDY CROPPING

Month		Seasonal water req'ment without rainfall (m^3/s)	Seasonal max. gross water req'ment with rainfall (m^3/s)	
			For 20 yrs.	For 10 yrs.
7	1	1.747	1.712	1.712
	2	1.806	1.806	1.806
	3	6.663	6.663	2.828
8	1	11.171	8.053	7.825
	2	13.584	13.205	13.205
	3	16.067	15.630	15.630•
9	1	17.068•	15.886•	13.628
	2	14.312	12.760	12.760
	3	13.797	10.438	9.492
10	1	14.231	13.871	13.871
	2	13.243	13.243	13.243
	3	10.366	10.366	10.366
11	1	8.091	8.091	8.091
	2	4.963	4.963	4.716
	3	2.104	2.104	2.104

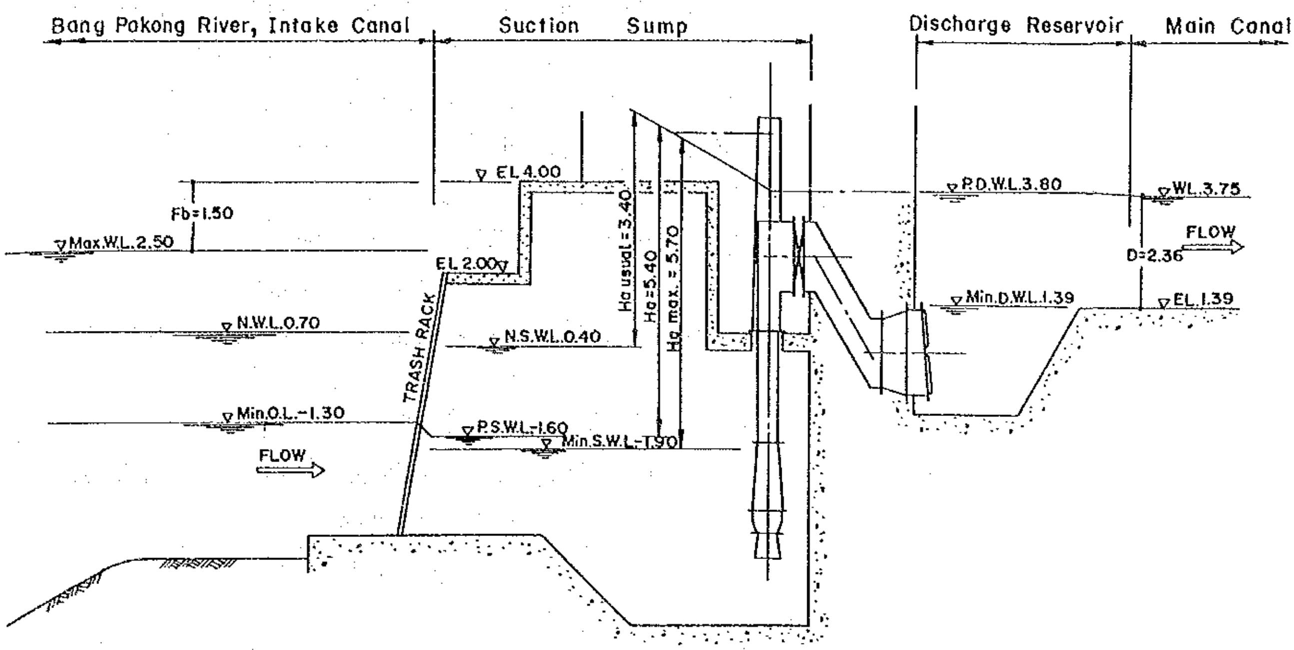
Note : The records for rainfall will cover the latest 10 to 20 years depending upon the rainfall records available for 1968 - 1987.

b) Water Level

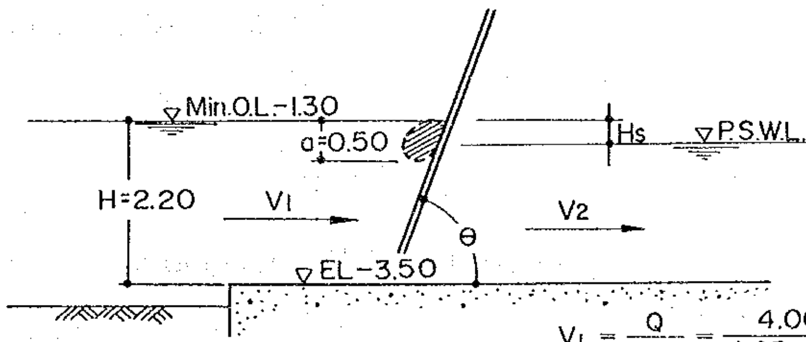
The design suction water level of the proposed pumps is taken at P.S.W.L. (-)1.60 m, taking into account head losses due to conveyance and trashrack on the basis of the minimum operating water level Min. O.L. (-)1.30 m of Bang Pakong Reservoir. The lowest suction water level is designed at Min. S.W.L. (-) 1.90 m including a 0.30 m allowance for unforeseeable suction water level lowering because of trouble cause by a great deal of trapped trash.

The design pump discharge water level is taken at P.D.W.L. 3.80 m based on the design data of the main irrigation canal.

FIGURE 8-3 FIGURE ON RELATED WATER LEVEL



Head Losses Due to Trashrack



$$V_1 = \frac{Q}{B \cdot H} = \frac{4.00}{4.05 \times 2.20} = 0.45 \, m/s$$

The trashrack head losses can be estimated by the following equation:

$$Hs = 6.69 \times \sin\theta \cdot (t/b)^{4/3} \cdot \exp(0.074 \times \gamma a \times a/H) \cdot V^2/2g$$

Where, H_s	:	Head losses due to trashrack (m)
a	:	Height of trashes adsorbed (= 0.50 m)
γa	:	Trash weight by unit volume (= 200 kg/m^3)
θ	:	Sloping angle of trashrack (= 70°)
H	:	Water depth at the upstream of the trashrack (= 2.20 m)
t	:	Thickness of trashrack bars (= 0.009 m)
b	:	Clear spacing of trashrack bars (= 0.050 m)
g	:	Gravity added (= 9.8 m/s^2)

$$H_s = 6.69 \times \sin 70° \times (0.009/0.050)^{4/3} \cdot \exp(0.074 \times 200 \times 0.50/2.20) \times 0.45^2/19.6 = 0.191 \text{ m}$$

The intake head losses and some allowance shall be added to the above result to get the trashrack head losses by Hs = 0.30 m as design value.

2) Number of Pump Units Required and Bore

The number of pump units required and the bore will be determined according to the following conditions:

i) To meet the varied water requirement

It is desirable to meet the seasonal variation of the irrigation water requirements under the control by the number of pump units to be provided. Application of increase/decrease in the number of units and a combination of size of the units would be most appropriate.

ii) To lighten damage due to machine trouble

Installation of the plural number of the pump units is desirable as a back up in case of machine troubles.

iii) To have the parts of the equipment interchangeable

Installation of units with the same capacity is desirable so that essential parts of the equipment and devices will be interchangeable.

iv) To be more economical in initial and operation costs

The initial cost will be more expensive in case of the large number of pumps to be installed, but the adaptability of the facilities to fluctuations of the water demand will be reduce in the operation costs.

Under the above-mentioned conditions the proposed number of pump units will be determined so as to minimize both the initial and operation costs.

a) Proposed Number of Pump Units

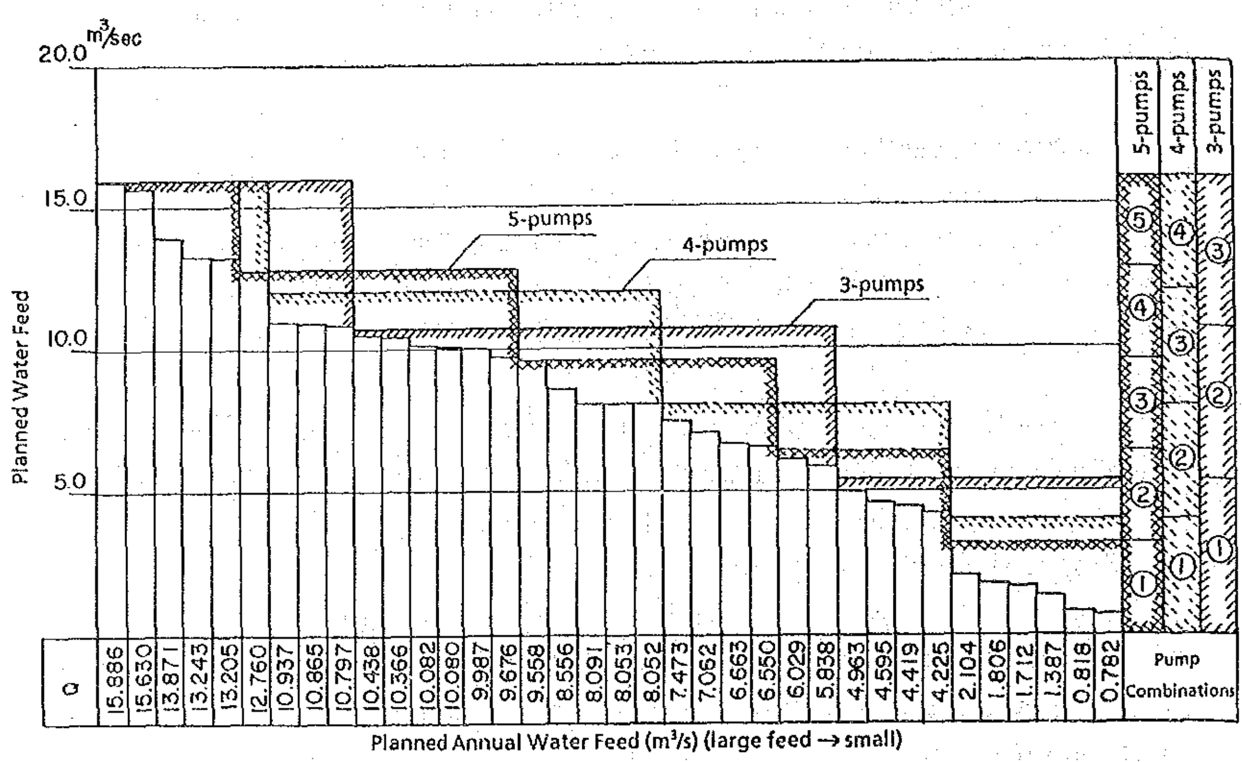
All the pump units to be adopted will have the same capacity in terms of interchangeability of the parts, operability, and easy operation and maintenance works. The following alternative study on three to five units will be made to consider operational flexibility in appropriately meeting the variated water requirements and operational safeguards against potential problems. For reference, in this plan, no stand-by pump units will be provided due to the fact that the proposed pumps are used for irrigation water intake.

The standard bore of the pumps are shown as follows by number of units:

	No. of Pump (units)	Discharge/unit (m^3/min)	Bore (mm)
Case - 1	3	320	ø1,500
Case - 2	4	240	ø1,350
Case - 3	5	192	ø1,200
(Case - 4	6	160	ø1,200)

Adaptability between the proposed number of pump units and planned annual water feed is shown in Figure 8-4.

FIGURE 8-4 PROPOSED NO. OF PUMPS & PLANNED ANNUAL WATER FEED



Adaptability ratio (α) for each case is as follows so that the increasing number of units brings favorable results:

3 - pumps $\alpha = 75.9\%$

4 - pumps $\alpha = 77.8\%$

5 - pumps $\alpha = 81.4\%$

b) Decision on Number of Proposed Pump Units

The following comparison table explains the relationship between the adaptability of pump numbers to annual water feed and economy including the initial costs and running costs for each case in the alternative study.

TABLE 8-2 COMPARISON OF PROPOSED NUMBERS OF PUMP UNITS

Item \ Case	Case - 1	Case - 2	Case - 3
Bore × Units	ø1,500 mm × 3	ø1,350 mm × 4	ø1,200 mm × 5
Adaptability to Water Feed Planned Annual W. Feed Total W. Feed for 20 yrs.	 a = 75.9% a = 61.3%	 a = 77.8% a = 67.1%	 a = 81.4% a = 75.7%
Economic Evaluation Construction Cost Pump Installation Cost Running Cost 1* Total (Ratio)	'000฿ 31,341 209,440 163,963 404,744 (100%)	'000฿ 34,254 220,080 154,316 408,650 (101.0%)	'000฿ 35,891 233,040 151,474 420,405 (103.9%)

1* The running costs quoted cover the total running costs for twenty (20) years within the life of the pump facilities.

As clarified by the above table, better adaptability to annual water feed can be secured by large number of the pump units provided while, contrarily, more disecomony including running costs would result form the large number.

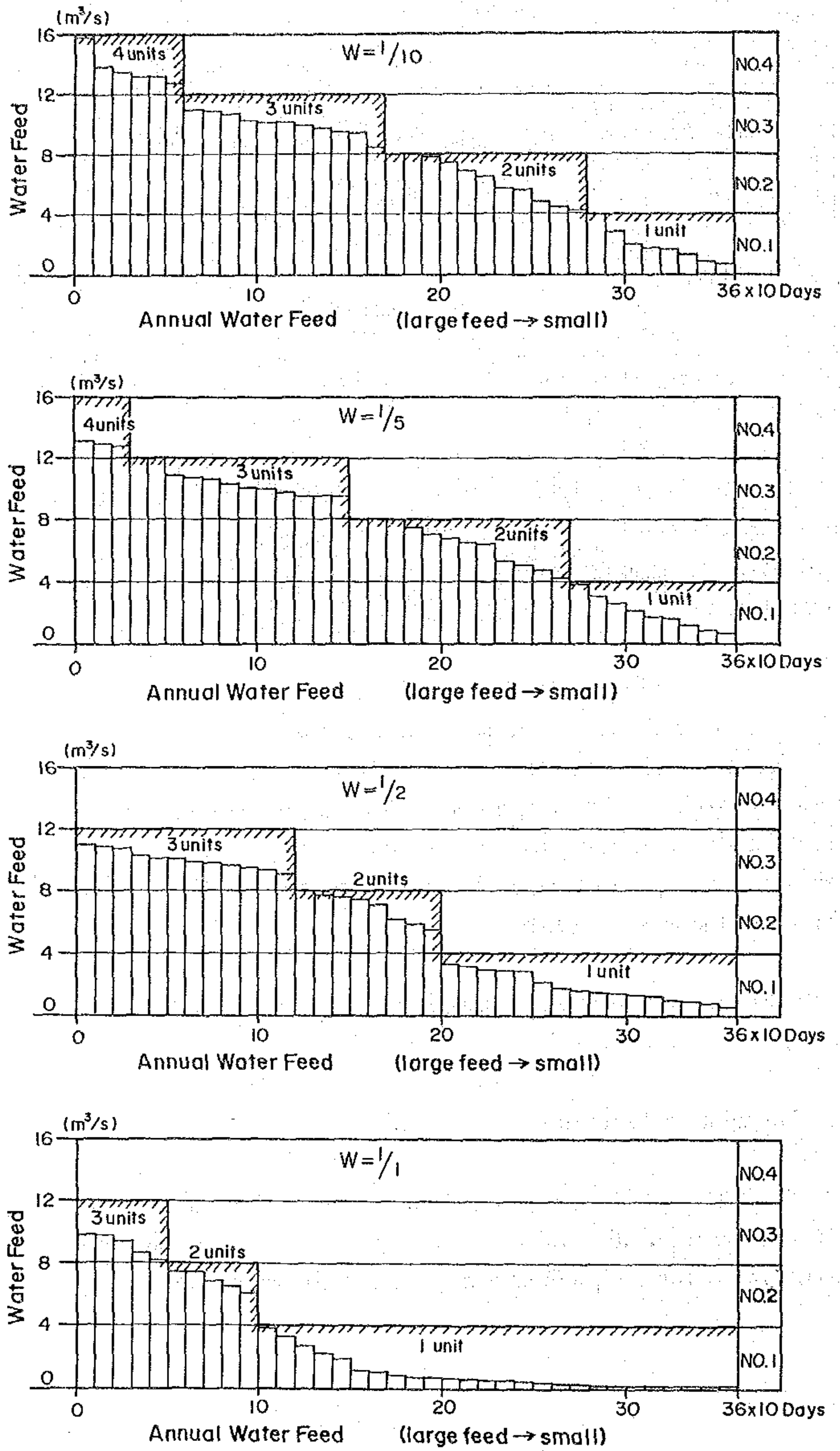
In principle, in the Project, the water feed shall be controlled by means of the number of pump operation units, and the better control can be obtained by high adaptability to the water feed. It is, therefore, desirable to provide possibly large number of the pump units. In the respects of the economy, the cases for three (3) and four (4) pump units would bring about almost the same results. The running costs are more economical in the case for four (4) unit than that for three (3) units.

As a result of the above comprehensive study, the Case-2 for four (4) pumps is recommended for the Project.

c) Decision of Pump Bore

The bore of the proposed pumps should be determined through the relation between discharge per one unit of the pump and the standard bore according to the design discharge per one unit of the pump.

FIGURE 8-5 PUMP OPERATION UNITS FOR ANNUAL WATER FEED (In each return period)



The design discharge of the pumps in the Project is 16.00 m^3/s for four (4) units, which come to 4.00 m^3/s per unit (240.00 m^3/min). Under such conditions, the pump bore will be determined at 1,350 mm.

3) Determination of Pump Type

The types of proposed pump and shaft can be roughly determined according to the given total head. An applicable range of the types of the pump and shaft for the given total head is as follows:

TABLE 8-3 PUMP TYPES AND TOTAL HEAD

Model \ Shaft Type	Horizontal	Vertical
Axial Flow	3 m or less	Under 5 m
Mixed Flow	3 to 7 m	Over 4 m

The total head given for the proposed pump facilities is about 6.0 m, and the pump types to be recommended to the Project are:

i) Horizontal shaft-type mixed flow pump
ii) Vertical shaft-type mixed flow pump

The vertical shaft-type mixed flow pump would be appropriate since the proposed pump facilities must not cause any harmful cavitation in the operation range.

4) Pump Head

a) Actual Static Head (Ha)

The actual static head at the design point of the pumps is as follows:

Actual Head (Ha) = Design Discharge Water Level
- Design Suction Water Level
= P.D.W.L. 3.80 - P.S.W.L. (-)1.60 = 5.40 m

b) Head Loss ($h\ell$)

i) Designed conditions

① Designed discharge capacity

$Q = 240\ m^3/min = 4.00\ m^3/sec$ per pump

② Designed discharge pipe bore

Pump discharge pipe bore $D_1 = 1{,}350$ mm
Tapered discharge pipe bore $D_2 = 1{,}650$ mm

③ Pipe cross-sectional areas

$A_1 = \pi \cdot D_1^2/4 = 1.431\ m^2$ for D_1
$A_2 = \pi \cdot D_2^2/4 = 2.138\ m^2$ for D_2

④ Flow velocity in the pipe

$V_1 = Q/A_1 = 2.795$ m/sec for D_1
$V_2 = Q/A_2 = 1.871$ m/sec for D_2

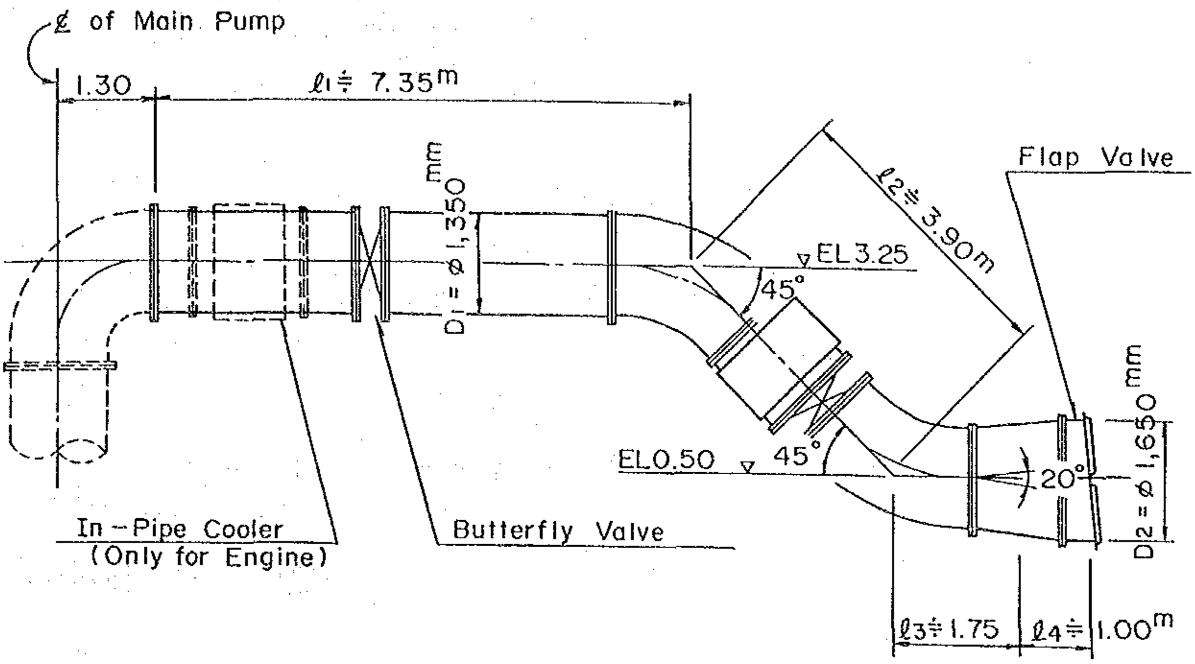
⑤ Velocity head

$V_1^2/2g = 0.399$ m for D_1
$V_2^2/2g = 0.179$ m for D_2

⑥ Pipe length

For, $L_1 = \ell_1 + \ell_2 + \ell_3 = 12.95$ m for D_1
For, $L_2 = \ell_4 = 1.00$ m for D_2

FIGURE 8-6 PIPING DIMENSIONS



ii) Head losses for each element

① Friction losses for D_1 pipes

By applying Darsy Weisback formula

$$hf_1 = \lambda_1 \cdot \frac{L_1}{D_1} \cdot \frac{V_1^2}{2g}$$

where $\lambda_1 = (0.02 + 1/2000\,D_1) \times 1.5 = 0.0306$

$hf_1 = 0.0306 \times \frac{12.95}{1.35} \times 0.399 = 0.117$ .... (1)

② Friction losses for D_2 pipes

$\lambda_2 = (0.02 + 1/2000\,D_2) \times 1.5 = 0.0305$

$hf_2 = 0.0305 \times \frac{1.00}{1.65} \times 0.179 = 0.003\,\mathrm{m}$ .... (2)

④ Head losses due to butterfly valve

$$h_{bv} = f_{bv} \cdot \frac{V_1}{2g} \times 0.30 \times 0.399 = 0.120 \text{ m} \quad \cdots\cdots\cdots \quad (4)$$

where $f_{bv} = 0.30$

⑤ Head losses due to 45° bend

$$h_b = f_b \cdot \frac{V_1^2}{2g} \times 0.12 \times 0.399 \times 2 = 0.096 \text{ m} \quad \cdots\cdots \quad (5)$$

where $f_b = 0.12$

⑥ Head losses due to gradual expansion of pipe

$$h_{ge} = f_{ge} \cdot (V_1\text{-}V_2)^2/2g = 0.43 \times (2.795\text{-}1.871)^2/19.6 = 0.019 \text{ m} \quad (6)$$

where $f_{ge} = 0.43$

⑦ Head losses due to flap valve

$$h_v = f_v \cdot (V_2^2/2g) = 0.62 \times 0.179 = 0.111 \text{ m} \quad \cdots\cdots \quad (7)$$

where $f_v = 0.62$

⑧ Head losses due to water release

$$h_0 = f_0 \cdot \frac{V_2^2}{2g} = 1.0 \times 0.179 = 0.179 \text{ m} \quad \cdots\cdots\cdots\cdots \quad (8)$$

where $f_0 = 1.0$

iii) The sum of head losses

For pump driven by motor
$H\ell_1 = (1) + (2) + (4) + (5) + (6) + (7) + (8) = 0.645 \text{ m} \doteqdot 0.70 \text{ m}$

For pump driven by engine
$H\ell_2 = (1) + (2) + (3) + (4) + (5) + (6) + (7) + (8) = 0.785 \text{ m} \doteqdot 0.80 \text{ m}$

c) Total Head (H)

Total Head (H) = Ha + Hℓ
= 5.40 + 0.70 = 6.10 m (For driven by motor pump)
= 5.40 + 0.80 = 6.20 m (For driven by engine pump)

5) Study on Cavitation

a) Study Conditions

Design discharge/unit	Q = 240 m^3/min
Design actual pump head	ha = 5.40 m
Lowest discharges water level	Min. D.W.L. 1.39 m
Operatable minimum suction water level	Min. S.W.L. (-)1.90 m
Suction water level at minimum actual head	S.W.L. 1.39 m
Finished floor elevation of pump room	EL. 4.30 m
Total pump head	

Horizontal shaft type ht = ha + hl = 5.40 + 0.80 = 6.20 m
Vertical shaft type ht = ha + hl = 5.40 + 0.70 = 6.10 m

b) Conditions for Planning

	Axial Flow Pump	Mixed Flow Pump
Specific Speed (Ns)	1,500~1,600	900~1,000
Revolution Speed (N)	400 rpm	250 rpm
Suction Specific Speed (S)	1,200	1,300
Pipe Losses Head		
Horizontal shaft type (hls)	0.30 m	0.30 m
Vertical shaft type (hls)	0 m	0 m
Atomspheric pressure (Pa)	10.33 m	10.33 m
Saturated vapor pressure (Pv)	0.33 m	0.33 m
Allowable suction head (β)	0.50 m	0.50 m

c) Study Result

The vertical shaft type mixed flow pumps will not cause cavitation in every range of operation, but other types of pumps will be.

TABLE 8-4 STUDY ON CAVITATION BY HORIZONTAL SHAFT PUMP

Pump Type		Axial Flow Pump		Mixed Flow Pump		Remarks
Bore		φ 1,350 mm		φ 1,350 mm		
Study Conditions		Lowest S.W.L	Min. ha	Lowest S.W.L	Min. ha	
Discharge Water Level (D.W.L)	m	1.39	1.39	1.39	1.39	
Suction Water Level (S.W.L)	m	(-) 1.90	1.39	(-) 1.90	1.39	
Total Head (ht)	m	6.20	6.20	6.20	6.20	
Minimum Actual Head (hamin)	m	3.29	0	3.29	0	
Actual Head Ratio (hamin/ht)	–	0.53	0	0.53	0	
Loss Head of Pipe (h_ℓ)	m	0.80	0.80	0.80	0.80	
Loss Head Ratio (h_ℓ/ht)	–	0.13	0.13	0.13	0.13	
Capacity Ratio (q)	–	1.15	1.28	1.19	1.38	
Coefficient (α)	–	1.50	2.55	1.40	2.70	
Re. NPSH at the Design Point (Hsvo)	m	8.93	8.93	4.29	4.29	
Re. NPSH at the Max. Capacity (Hsv)	m	13.40	22.77	6.01	11.58	
Allowable Actual Suction Head (Hs_2)	m	(-) 4.20	(-) 13.57	3.19	(-) 2.38	9.20-Hsv
Basic Elevation (EL)	m	6.13	6.13	6.13	6.13	
Actual Suction Head ($H's_2$)	m	8.03	4.74	8.03	4.74	
Judgement $Hs_2 \geqq 0$ $Hs_2 \geqq H's_2$		NO	NO	NO	NO	
		×		×		

TABLE 8-5 STUDY ON CAVITATION BY VERTICAL SHAFT PUMP

Pump Type		Axial Flow Pump		Mixed Flow Pump		Remarks
Bore		φ 1,350 mm		φ 1,350 mm		
Study Conditions		Lowest S.W.L	Min. ha	Lowest S.W.L	Min. ha	
Discharge Water Level (D.W.L)	m	1.39	1.39	1.39	1.39	
Suction Water Level (S.W.L)	m	(-) 1.90	1.39	(-) 1.90	1.39	
Total Head (ht)	m	6.10	6.10	6.10	6.10	
Minimum Actual Head (hamin)	m	3.29	0	3.29	0	
Actual Head Ratio (hamin/ht)	–	0.54	0	0.54	0	
Loss Head of Pipe (h_2)	m	0.70	0.70	0.70	0.70	
Loss Head Ratio (h_2/nt)	–	0.11	0.11	0.11	0.11	
Capacity Ratio (q)	–	1.16	1.29	1.20	1.39	
Coefficient (α)	–	1.50	2.65	1.40	2.80	
Re. NPSH at the Design Point (Hsvo)	m	8.93	8.93	4.29	4.29	
Re. NPSH at the Max. Capacity (Hsv)	m	13.40	23.66	6.01	12.01	
Allowable Actual Suction Head (Hs_2)	m	3.90	14.16	(-) 3.49	2.51	–(9.50-Hsv)
Basic Elevation (EL)	m	(-) 3.53	(-) 3.53	(-) 3.53	(-) 3.53	
Actual Suction Head ($H's_2$)	m	1.63	4.92	1.63	4.92	
Judgement $H's_2 \geqq Hs_2$		NO	NO	OK	OK	
		×		○		

8.3 Design of Prime Mover

1) Determination of Prime Mover Type

The type of prime mover will be determined taking into account the local conditions for the relevant energy sources, running of pumps, O/M, environment, etc. together with its economy.

In general, there are two types of prime mover for the pumps; motors and diesel engines. The more suitable type will be selected for the Project.

a) Combination of Prime Movers

A combination of prime movers for the pumps will be determined so that at least one unit of the main pumps can be driven in a power suspension, considering the local power supply conditions. Under such conditions, a emergency generator should be installed for operation of at least one unit of the main pumps together with the appurtenant equipment.

The combination of prime movers will be studied comparatively with the following four cases:

Case-1: Motors for all pumps
Case-2: Three motors and one diesel in co-use with motor
Case-3: Three motors and one diesel only
Case-4: Two motors and two diesels only

In Case-1, the emergency generator's capacity should be enough to operate one unit of the main pumps with the appurtment equipment.

In Case-2 to Case-4, the emergency generator's capacity should be enough to operate the appurtenant equipment only.

TABLE 8-6 COMBINATION PATTERNS OF PRIME MOVERS

CASE / ITEM	1	2	3	4
Type & Number of Pumps	Vertical Shaft Type Mixed Flow Pump ø 1,350 mm x 4 units			
Output of Prime Movers	350 Kw x 4 Sets	350 Kw x 3 Sets 350 Kw x 1 (500 Ps)	350 Kw x 3 Sets 500 Ps x 1	350 Kw x 2 Sets 500 Ps x 2
Combination of Prime Movers	P – M 350 Kw P – M 350 Kw P – M 350 Kw P – M 350 Kw GENERATOR (3300V) 1250 KVA	P – M 350 Kw P – M 350 Kw P – M 350 Kw M 350 Kw P – D.E. 500 Ps GENERATOR (380V) 60 KVA	P – M 350 Kw P – M 350 Kw P – M 350 Kw P – D.E. 500 Ps GENERATOR (380V) 60 KVA	P – M 350 Kw P – M 350 Kw P – D.E. 500 Ps P – D.E. 500 Ps GENERATOR (380V) 60 KVA

b) Determination of Prime Mover Type

For successful selection of the suitable prime movers of the proposed pumps, the following table is prepared to show the operation conditions of the proposed pump units for the 20 year period from 1968 to 1987.

TABLE 8-7 ANNUAL OPERATION HOURS OF EACH PUMP

Pump	Max.	Min.	Mean
	(hr)	(hr)	(hr)
No. 1 Pump	8,784	8,760	8,766
No. 2 Pump	4,284	3,081	3,661
No. 3 Pump	1,645	460	852
No. 4 Pump	146	0	24

The economic comparison of the prime movers for installation costs and running costs is as follows:

TABLE 8-8 ECONOMIC COMPARISON OF PRIME MOVERS IN COMBINATION

(Unit ; '000฿)

Case	Case-1	Case-2	Case-3	Case-4
Construction Cost	33,189	34,254	34,254	34,254
Pump Installation Cost	233,216	229,440	220,080	234,080
Running Cost *1	168,131	168,131	154,316	143,853
Total	434,536	431,825	408,650	412,187
(Ratio)	(106.3%)	(105.7%)	(100%)	(100.9%)

*1 The running costs quoted cover the total running cost for twenty (20) years within the life of the pump facilities.

Electric motors are positively recommended as the pump prime movers for the Project in view of the operation conditions, but it is considered reasonable to adopt one diesel engine, as insurance against serious problems. Case-3 with one diesel engine provided is found most economical and advantageous. Case-3 therefore, should be adopted as the combination of pump prime movers for Project. It will include three (3) electric motors and one (1) diesel engine.

2) Determination of Prime Mover Output

The output of the proposed pump prime movers can be computed by the following equation:

$$P = \frac{K \cdot \gamma \cdot Q \cdot H}{\eta_P \cdot \eta_G} \cdot (1 + R)$$

Where, P : Output of prime movers (kw or ps)
K : Conversion factors (0.163 in kw, 0.222 in ps)
γ : Specific gravity of water (1.0)
Q : Pump discharge (m^3/min)
H : Total pump head (m)
η_P : Pump efficiency (%) × 1/100
η_G : Transmission efficiency (%) × 1/100
R : Extra-load factor to prime Mover
(0.15 for motors and 0.20 for diesel engines)

i) Output of Motors

$$P_M = \frac{0.163 \times 1.0 \times 240 \times 6.1}{0.835 \times 0.96} \times (1 + 0.15) = 342.5 \div 350 \text{ kw}$$

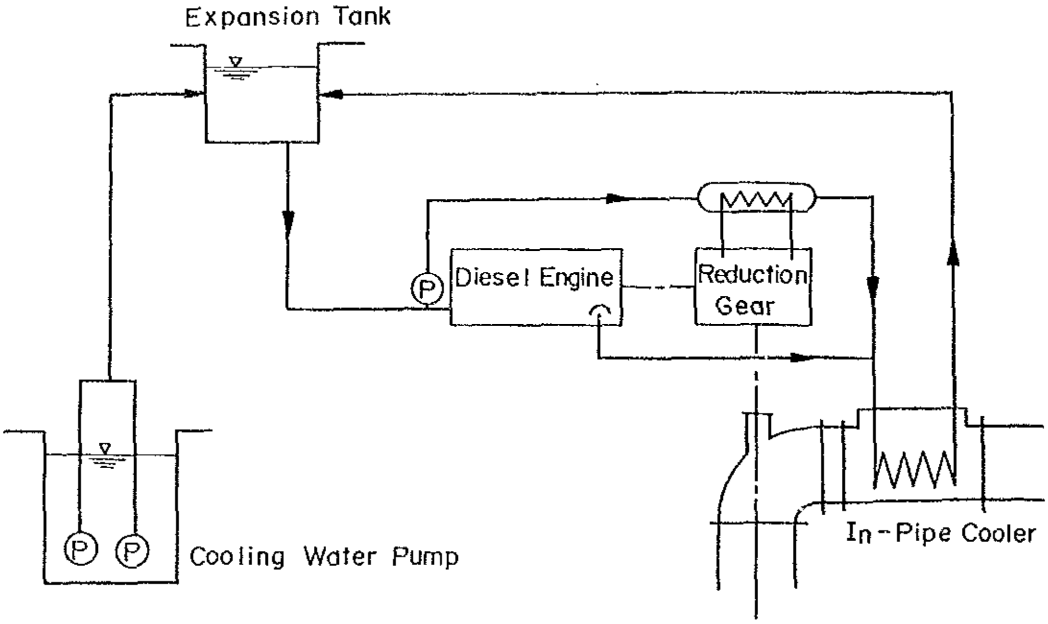
ii) Output of Diesel Engine

$$P_E = \frac{0.222 \times 1.0 \times 240 \times 6.2}{0.835 \times 0.96} \times (1 + 0.20) = 494.5 \div 500 \text{ ps}$$

8.4 Design of Auxiliary and Ancillary Equipment

1) Auxiliary Equipment for Cooling Water System

a) Flow of Cooling Water System



For the purpose of simplifying and ensuring the auxiliary water supply system, the ceramic bearing shall be applied to the main pump, which does not required a lubricating water. Thereof, the diesel engine and reduction gear shall be made cool through in-pipe cooler system, where, the diesel engine for power generator will equip with radiator.

b) Quantity of Cooling Water Supply

i) Quantity of cooling water for main diesel engine

$$Q_{WE} = \frac{P_E \cdot B_E \cdot H_u \cdot \tau}{\Delta T \cdot 60 \cdot C \cdot P} \times n$$

Where, Q_{WE} : Required water quantity
P_E : Output of diesel engine
B_E : Specific fuel consumption
H_u : Calorific values of fuel (Light oil 10,300 kcal/kgf)
τ : Cooling water thermal radiation ratio : 0.3

ΔT : Temperature difference of cooling water at between inlet and outlet (°C)
C : Specific heat of cooling water (1 kcal/kgf · °C)
P : Specific gravity of cooling water (1 kgf/ℓ)
n : Number of unit

$$\text{Let, } Q_{WE} = \frac{500 \times 0.2 \times 10{,}300 \times 0.3}{15 \times 60 \times 1 \times 1} \times 1$$
$$= 343.3;\ \text{round to } 350\ \ell/\text{min}$$

The supply water is estimated to be 1% of cooling water for main diesel engine in quantity, taking into account cooling water losses due to evaporations leakage from gland portions etc, as follows:

$$\text{Supply water quantity (q)} = 350 \times 0.01$$
$$= 3.5;\ \text{round to } 5\ \ell/\text{min}$$

ii) Expansion tank capacity

The capacity of expansion tank is corresponding quantity of water supply required for one hour and given below:

$$V = 5 \times 60$$
$$= 300;\ \text{use } 500\ \ell$$

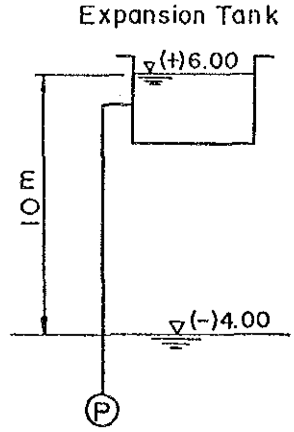
iii) Water Feeding Pump

This pump, feeding the supply water to the expansion tank, has a capacity to fill up the tank by about 5 minutes operations and is provided with two (2) units, one for normal operations and other for standby.

Let, Water discharge (q) = 500ℓ/5 min = 100 ℓ/min

Total pump head (Ht);

Actual pump head (Ha)	= 10 m
Head losses due to pumping, etc. (Hf) about =	5 m
Total	15 m

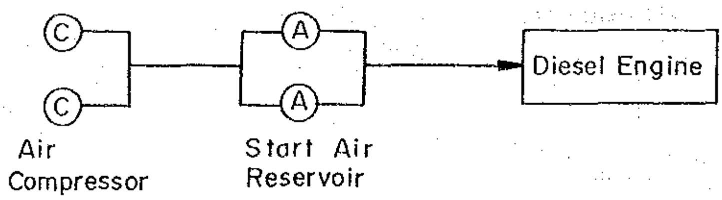


Specifications of Pump

Type	:	Submersible Motor Pump
Bore	:	40 mm
Discharge	:	0.1 m^3/min
Total pump head	:	15 m
Motor	:	0.75 kw
No. of Unit	:	2 units (incl. one for standby)

2) Auxiliary Equipment for Air Start System

a) Flow of Air Starting System



b) Air Start Reservoir

The air start reservoir has a capacity enough to perform starting of more than 3 times by automatic operations and/or more than 5 times by mannual operations, as given below:

$$V = \frac{\pi}{4} \times D^2 \times S \times \frac{Z}{2} \times \frac{P_0}{P_2 - P_1} \times N \times R$$

Where,
- V : Air start reservoir capacity (m^3)
- D : Diameter of cylinder (m)
- S : Stroke of piston (m)
- Z : No. of cylinder entered starting air
- P_0 : Atmospheric pressure; 1 kgf/cm^2
- P_1 : Minimum operating pressure; 10 kgf/cm^2
- P_2 : Initial pressure in the air reservoir; 30 kgf/cm^2
- N : No. of starting times by manual; 6
- R : Cranking revoluting speed necessary for starting

For the capacity of air start reservoir for 500 ps diesel engine, let:

$$V = \frac{\pi}{4} \times 0.165^2 \times 0.21 \times \frac{6}{2} \times \frac{1}{30 - 10} \times 6 \times 13$$
$$= 0.053 \text{ m}^3$$

Then, expecting the starting of three (3) times by automatic operation, the double capacity obtained in the above equations as done by manual operation is applied; that is,

$$V = 0.053 \times 2$$
$$= 0.106 \text{ m}^3\text{; use } 150\ell$$

c) Air Compressor

The air compressor generally has the capacity which is able to fill one air reservoir with the air applying from the atomosphic pressure to the specified pressure within 30 to 60 minutes operations. Nevertheless, the following two (2) cases are examined to determine the specifications of the subject compressor.

i) Filling one air reservoir (150ℓ) with air applying pressure from 0 to 30 kgf/cm^2

$$A = \frac{V \times (P_2 - P_1) \times 60}{T \times \eta c \times P_0}$$

Where,
- A : Capacity necessary for air compressor
- V : Air reservoir capacity
- P_2 : Pressure in the air reservoir after air filling; 30 kgf/cm^2
- P_1 : Pressure in the air reservoir before air filling; 0 kgf/cm^2
- T : Time required for air filling (min)
- ηc : Average filling up efficiency of compressor; 0.55
- P_0 : Atomospheric pressure; 1 kgf/cm^2

$$A = \frac{0.15 \times (30 - 0) \times 60}{30 \times 0.55 \times 1.0}$$
$$= 15.8\ m^3/hr$$

ii) Filling two (2) air reservoirs (300 ℓ) with air applying pressure from 22 to 30 kgf/cm^2

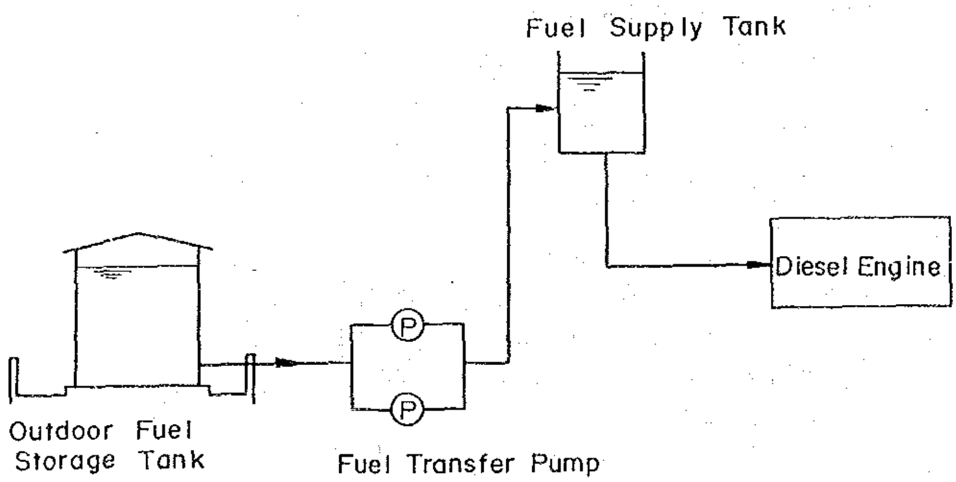
$$A = \frac{0.3 \times (30 - 22) \times 60}{30 \times 0.55 \times 1.0}$$
$$= 8.45\ m^3/hr$$

Then, the air compressor is specified as follows:

Type	:	Vertical type two stage air cooled compressor
Capacity	:	19.3 m^3/hr
Delivery Pressure	:	30 kgf/cm^2
Output of Motor	:	3.7 kw
No. of Unit	:	2 units (incl. one for standby)

3) Auxiliary Equipment for Fuel System

a) Flow of Fuel System



b) Fuel Consumptions

$$q = \frac{B_E \cdot P_E}{W_f}$$

Where, q : Fuel consumptions
B_E : Specific fuel consumptions
P_E : Output of diesel engine
W_f : Specific gravity of fuel (Light oil; 0.83 kgf/ℓ)

For 500 ps diesel Engine:

$$q_1 = \frac{0.2 \times 500 \text{ ps}}{0.83}$$
$$= 120.5\text{; round to } 121\ \ell/\text{hr}$$

For diesel engine for 60 kVA electric power generator:

$$q_2 = \frac{0.22 \times 80 \text{ ps}}{0.83}$$
$$= 21.2\text{; round to } 22\ \ell/\text{hr}$$

c) Fuel Storage Tank

The capacity of fuel storage tank is determined to correspond to the fuel consumption required for the continuous operation of main pump for 100 hours and power generator for 5 hours more and given as follows:

$$Q = (q_1 \times 100 + q_2 \times 5) \times \alpha$$

Where, Q : Fuel supply tank capacity (ℓ)
α : Allowance factor 1.2

Let, $Q = (121 \times 100 + 22 \times 5) \times 1.2$
$= 14{,}652\ \ell$; use 15 kℓ

Then, the specification is below:

Type	:	Steel plate-made, on-ground, upright type tank
Capacity	:	15 kℓ
No. of Unit	:	1 unit

d) Fuel Supply Tank

The capacity is equivalent to the fuel consumption necessary to operate all diesel engines for 2 to 3 hours and given as follows:

$V = \Sigma q \cdot t$
Where, V : Fuel supply tank capacity
$V = (121 + 22) \times (2\sim3)$
$= 286\sim429\ \ell$; use $390\ \ell$

Specification show below:

Type	:	Steel plate-made, cube type
Capacity	:	390ℓ
No. of Unit	:	1 unit

e) Fuel Transfer Pump

This pump has a capacity to feed fuel to the fuel supply tank for about 30 minutes.

The discharge capacity (Q) is given as follows:

$Q = V/t$
$= 390/30$
$= 13\ \ell/\text{min}$; use $35\ \ell/\text{min}$

Specification shows below:

Type	:	Gear pump
Bore	:	20 mm
Discharge Capacity	:	35 ℓ/min
Output of Motors	:	0.4 kw
No. of Unit	:	2 units (incl. one for standby)

4) In-plant Drainage Pump

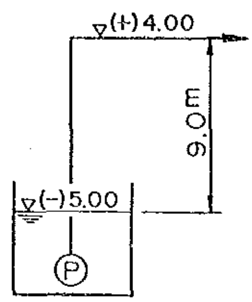
The proposed pump installation floor is semi-two floor type. The floor level for pump installation (EL. 1.70m) is below the ground level (EL. 4.00 m). Thereby, the bilge water come out on the pump installation floor is required to be wasted by the in-plant drainage pump installed in the drainage tank.

Discharge Capacity;

The capacity is designed to drain the water in the drainage tank (3.80 m × 4.05 m × 6.25 m) for 10 hours.

Then,

$$q = (3.80 \times 4.05 \times 6.25) \div (10 \times 60)$$
$$= 0.15\ m^3/min$$



Total Pump Head (Ht)

Actual pump Head (Ha)	=	9.0 m
Various head lesses due to pipes, etc. (Hf)	=	3 m
Total		12 m

Specification of Pump:

Type	:	Submersible motor pump
Bore	:	50 mm
Discharge Capacity	:	$0.15\ m^3/min$
Total Pump Head	:	12 m
Output of Motors	:	0.75 kw
No. of Unit	:	2 units (incl. one for standby)

5) Air Quantity for Ventilation of Pump Room

The ventilation air quantity is determined by calculating the air quantity required for dissipating the heat generated by the equipment and burning diesel engine fuel.

a) Quantity of heat generated by electric motor

$$Q_M = \frac{P_M \cdot (1 - \eta_M)}{\eta_M} \times 860$$
$$= \frac{350 \times (1 - 0.93)}{0.93} \times 860 \times 3 = 67{,}968\ \text{kcal/h}$$

Where, Q_M : Heat quantity generated by electric motor (kcal/h)

P_M : Electric motor output power (kw)

η_M : Electric motor efficiency

b) Quantity of Heat Generated by Diesel Engine (water-cooled)

$$Q_E = P_E \cdot B_E \cdot H_u \cdot f$$
$$= 500 \times 0.20 \times 10{,}300 \times 0.025 = 25{,}750 \text{ kcal/h}$$

Where, Q_E : Heat quantity generated by diesel engine (kcal/h)
P_E : Diesel engine output power (ps)
B_E : Specific fuel consumption (kg/ps · h)
H_u : Heat quantity generated by fuel burning (10,300 kcal/kg for Light oil)
f : Heat ratio dissipated from the surface of a diesel engine; 0.04 for high-speed diesel engines (1200 rpm or higher); 0.02 - 0.025 for low-speed diesel engines (600 - 1200 rpm or lower)

c) Quantity of Heat Generated by Gear Reducer

$$Q_B = K_B \cdot P_S \cdot (1 - \eta_B)$$
$$= 95 \times (500 \times 1 - \frac{350\text{kw}}{0.75} \times 3) \text{ ps} \times (1 - 0.96) = 7{,}220 \text{ kcal/h}$$

Where, Q_B : Quanity of heat generated by gear reducer (kcal/h)
P_S : Transmitted power
η_B : Transmision efficiency;
0.96 for right angle or parallel axis gear (1 stage)
0.94 for right angle or parallel axis gear (2 stages)
0.97 for planetary gear reducer
K_B : Coefficient; 95 ($\frac{\text{kcal/h}}{\text{ps}}$ · dissipating specific heat)

d) Radiant Heat of Building

$Q_c \doteqdot 36{,}000$ kcal/h

e) Ventilating Fan Air Quantity

The ventilation capacity required for preventing the room from rising temperature is given by the following equation:

$$V_F = \frac{Q_M + Q_E + Q_B + Q_C}{60 \cdot \gamma \cdot Cp \cdot \Delta T}$$

$$= \frac{67{,}968 + 25{,}750 + 7{,}220 + 36{,}000}{60 \times 1.2 \times 0.24 \times 5} = 1{,}585\ m^3/min$$

Where, V_F: Capacity of ventilating equipment (m^3/min)

γ : Weight per unit volume of air; 1.2 (at 30°C) (kg/m3)

C_P: Specific heat at constant pressure of air; 0.24 (kcal/kg · °C)

ΔT: Temperature rise; 5-10°C (difference from ambient temperature)

f) Specification of Ventilator

Type	:	Roof type ventialter
Diameter	:	ø 900 mm
Capacity	:	350 m^3/min
Static pressure	:	5 mm Aq.
Output	:	2.2 kw
Number of Unit	:	5 units

8.5 Design of Intake Canal and Intake

1) Intake Canal

The intake canal will be a facility to convey the water from the Bang Pakong River to the intake of the pumping station.

a) Study on Sedimentation Basin

The minimum size of sand particles to be settled in sedimentation basins is generally 0.3 mm in diameter, depending on those conditions of gradation curve of sediments inflow, amount of sediments in canals, etc.

The particle size of the sediment in the Bang Pakong River will include less than 5 percent by over 0.3 mm diameter and be found most by silt and clay with a diameter of 0.074 mm, judging from the gradation tests of the sand taken along the surface of the riverside. In consequence, the suspended particles included the suspended sediment are deemed most as silty and clayey materials, and there will be little adverse effects expected to pump impeller as well as to sedimentation on the canal bed.

In other respect, since the whisky factory near the Tha Lat River has been taking the river water by about 240 thousand cubic meters to use for the factory after storing in the reservoir where very little sediments can be found, it is considered unnecessary to provide a sedimentation basin for the pumping station. It is , therefore, the proposed pumping station shall not provide sedimentation basin, but a intake canal with the dimensions, have-in-after to as discussed, at the upstream of the pumping site so as to prevent inflow of sediment materials from flowing into the pumps.

Subject particles's diameter : $d = 0.3$ mm

Length of sand settling channel : $L = K \cdot \frac{h}{Vg} \cdot U$

Where,

L : Length sand settling channel (m)

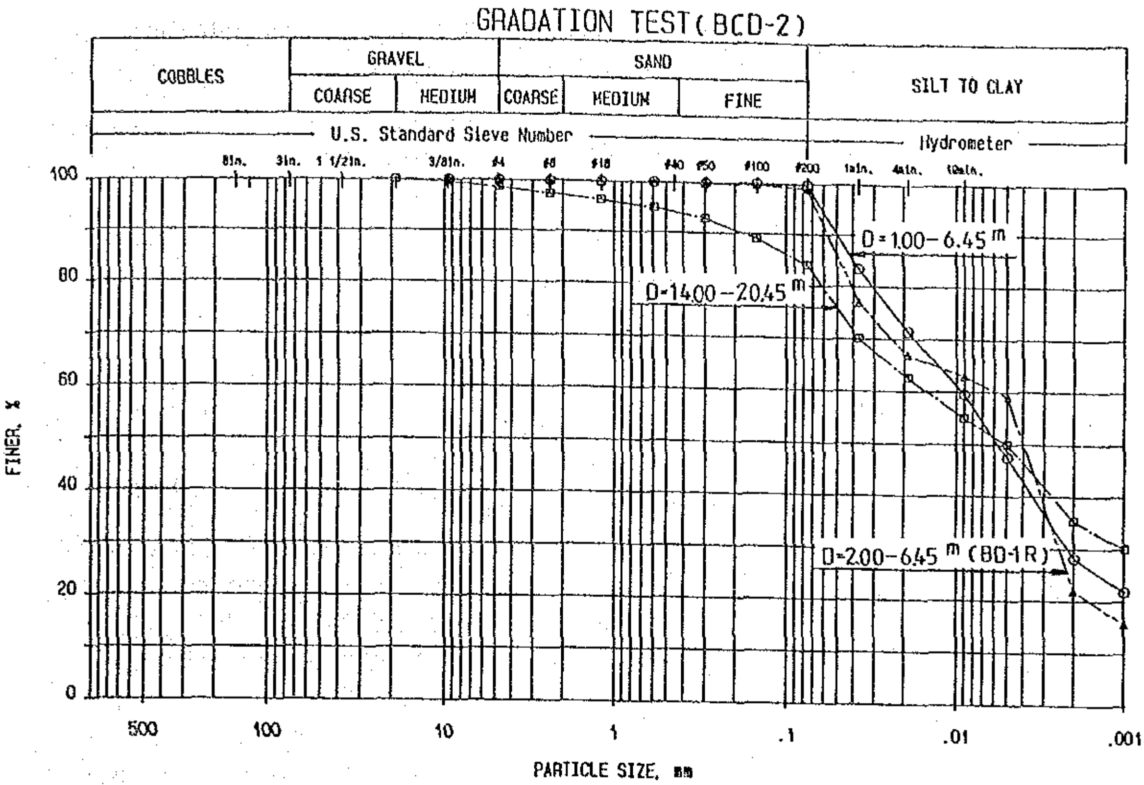
K : Safety factor (1.5)

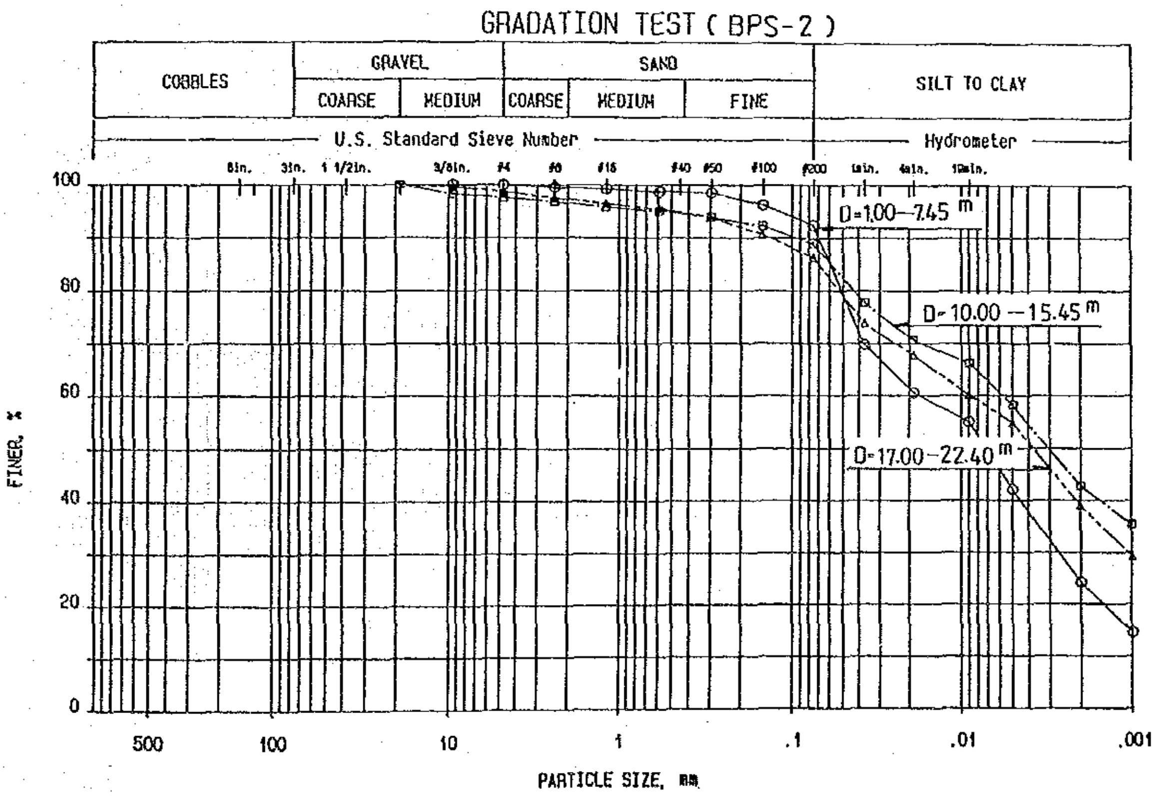
h : Water depth above the settlement (m)

U : Average velocity in the sand settling channel (0.15 ~ 0.30 m/sec)

Vg : Critical falling velocity of the smallest particle to be settled (m/sec)
$Vg = 0.025$ (m/sec) for particle with 0.3 mm diameter

FIGURE 8-7 GRADATION TEST FOR SOILS NEARBY THE RIVER

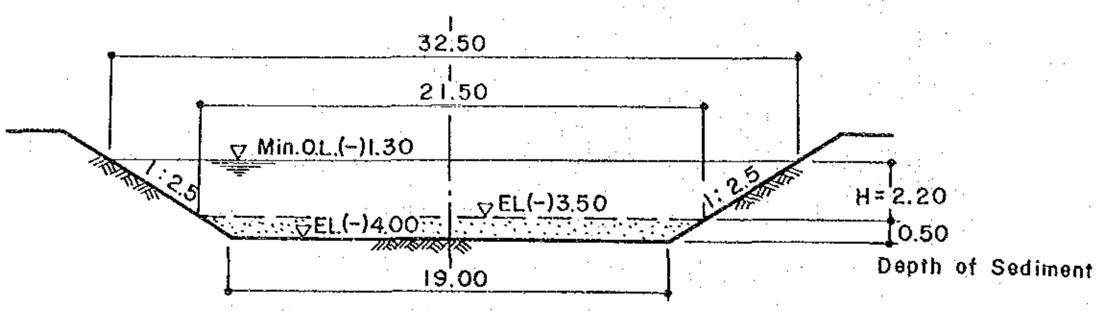




b) Dimemsion

The section of intake canal is determined to have a mean velocity of less than 0.30 m/s in the sand settling channel, as shown below:

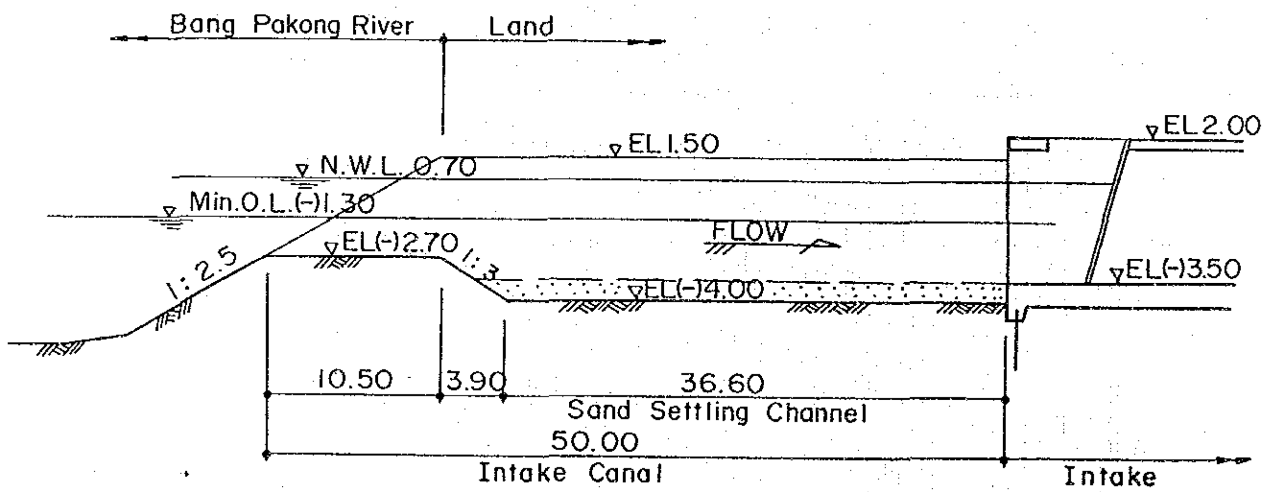
FIGURE 8-8 CROSS SECTION OF INTAKE CANAL



Flow area $A = 1/2 \times (21.50 + 32.50) \times 2.20 = 59.40\ m^2$
Mean Velocity $V = 16.00\ m^3/s / 59.40\ m^2 = 0.269\ m/s \leqq 0.30\ m/s$

The length of intake canal including the sand settling channel with a length of 36.60 m is determined to be 50.00 m, as shown below:

FIGURE 8-9 PROFILE OF INTAKE CANAL (UNIT : M)



c) Others

The intake canal will have stone pitching to prevent the canal bed and side slope from scouring by river flow. There will also be a floating net system in the upstream section of the canal to protect intake structures from floating trashes.

2) Intake

The intake will provide trashracks for protection of the pumps and stop-log grooves for maintenance and repair works.

a) Sill Elevation

The sill elevation of the intake will be determined so as to be an velocity of less than 0.50 m/s in the upstream of the trashrack. Taking the water depth at the upstream of trashrack to be 2.20 m, the inflow velocity is expected to be less than 0.50 m/s ($4.00/4.05 \times 2.20 = 0.45$ m/s). The sill elevation is EL. (-)3.50 m, which is 2.20 m below the minimum operating level of (-)1.30 m.

b) Width of Concrete Plank

The intake structure downstream form the trashrack will be covered with concrete plank. The width of concrete plank will be 4.0 m for trashes removal works and 5.0 m for the traffic path.

c) Elevation of Concrete Plank

The elevations of concrete plank for trashes removal works had better lower as much as possible for convience of doing the works manually. From this standpoints, the floor level of plank will be EL. 2.00 m in the elevations, which is a little higher than ordinary high water level (w = 1/10 yrs.).

The floor level of concrete plank for traffic will be EL. 4.00 m in the elevation, which is the same with the ground level for the pumping stations.

d) Effective Bar Spacing of Trashrack

The trashes on the racks will be taken away manually. The effective clearance between the bars will be 50 mm taking into consideration the pump bore 1,350 mm.

8.6 Design of Suction Sump

1) Suction Water level

The suction water level is determined according to the basic design flow conditions.

Design suction water level	:	P. S. W. L. (-) 1.60 m
Lowest suction water level	:	Min. S. W. L. (-) 1.90 m
Highest suction water level	:	Max. W. L. (+) 2.50 m (Maximum water level at the site of the pumping station)

2) Dimensions of Suction Sump

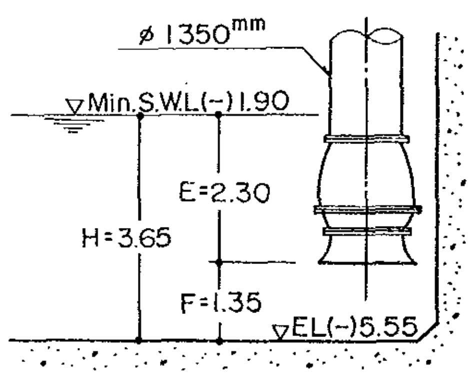
The shape and size of the suction sump will be determined so as to maintain water level and flow stably without entering air into the suction pipes and causing vertex motion in the sump.

a) Water Depth in Suction Sump

Water depth in the sump is determined by the E and F dimensions as shown in Figure 8-10 which are the depths required to submerge the suction pipes. For the pump with a 1,350 mm dia., the necessary water depth (H) is given, below:

$$H = E + F = 2.30 + 1.35 = 3.65\,m$$

FIGURE 8-10 WATER DEPTH IN SUCTION SUMP

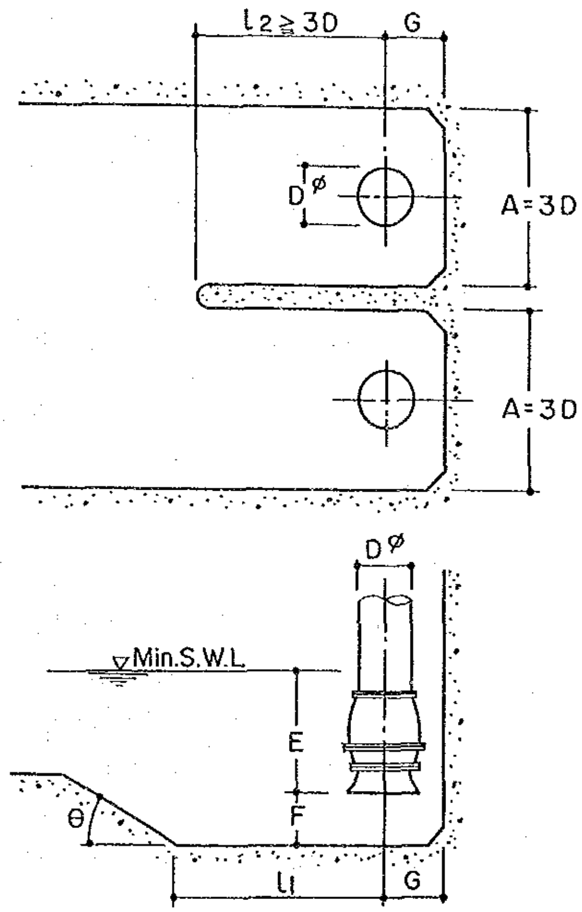


The sill elevation of the suction sump, therefore, is decided at EL. (-) 5.55 m.

b) Size of Suction Sump

The size of suction sump will be determined by the shape shown in Figure 8-11 in according to the number of sumps to be provided.

FIGURE 8-11 SHAPE OF SUCTION SUMP



In case $\theta = 30°; \ell_1 \geqq 3D$
$\theta = 45°; \ell_1 \geqq 4.5D$

$A = 3 \cdot D = 3 \times 1.35 = 4.05$
$\ell_1 = 3 \cdot D$ or more $= 3 \times 1.35 = 4.05$ m or more $(\theta \leqq 30°)$

The length of the suction sump will be 16.00 m in total taking into consideration the aforesaid measurements and the scale of the proposed pump facilities. The backside space of the sump will be used for the storage of cooling water.

c) Floor Elevation of Suction Sump

There will be foundations to place the pumping equipment and prime movers on the floors of the pumping station. The elevation of these floors will be determined as follows:

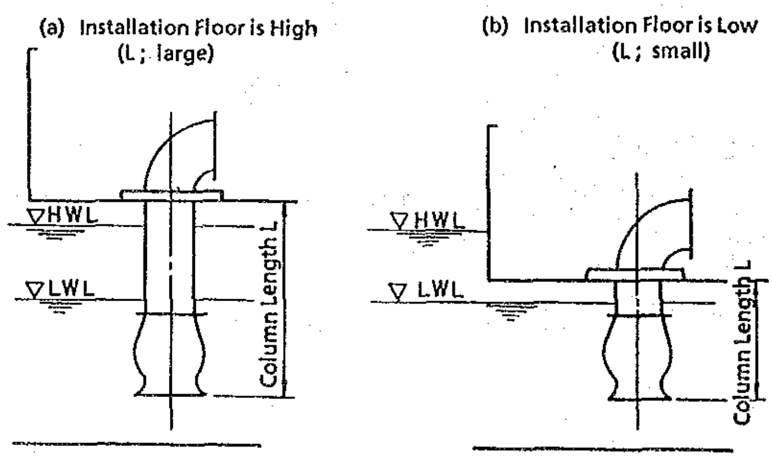
i) Elevation of prime mover floor

The elevation of the prime mover floor must be sufficient to ensure that it is not submerged by any high-level flood water. Under such consideration, the said elevation will be more than EL. 4.00 m taking into account Max. W. L. 2.50 m of design flood water level at the site of the pumping station and freeboard of 1.50 m.

ii) Elevation of pump floor

For vertical shaft-type pumps, if the installation floor is high, the initial cast is high because of the need for a long column of pump. On the other hand, in the case of lower pump installation floor, the pump column length become shorter and consequently the construction costs is smaller but it is necessary to protect the pump room from entering the flood water.

FIGURE 8-12 PUMP COLUMN LENGTH



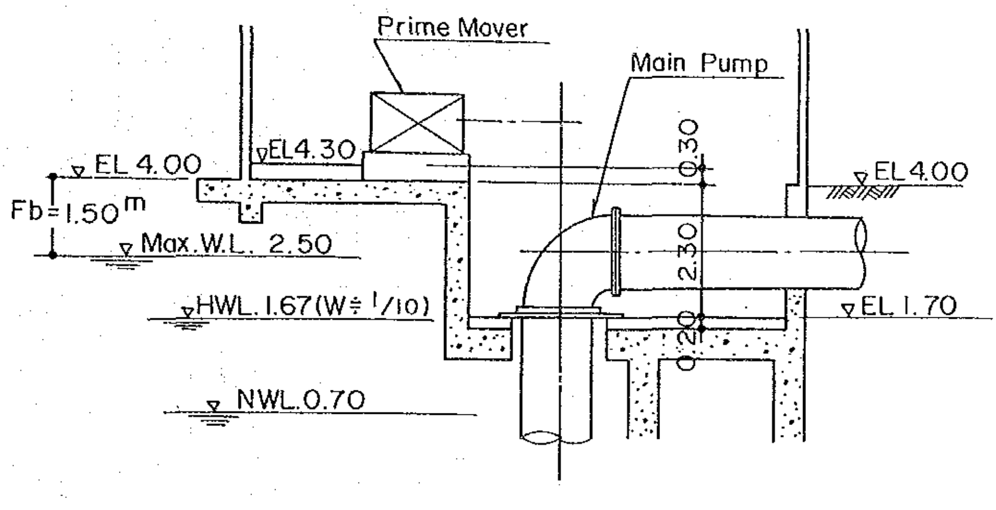
In the design of this sump, the pump installations floor elevation will be EL. 1.70 m, which is slightly higher than the maximum flood water level at Tha Khai for the lastest 8 years, H. W. L. 1.67 m (w ≑ 1/10 return period), for the following reasons:

① As compared to the case that the pumps and prime mover are installed on the same floor level against the design flood water level (Max. W. L. 2.50 m), the pump column length in the proposed floor elevation is shortened by 2.60 m and both costs for pumping facilities and pump house become economical.

② Although there might be some fear of flood water intrusion in the pump room, a water tight pump base and in-plant drainage pump are required to prevent from intruding flood water.

③ By taking EL. 1.70 m in the pump room floor elevation, the frequency of suffering from the flood water intrusion become lesser.

FIGURE 8-13 FLOOR ELEVATION OF SUCTION SUMP



8.7 Design of Pump House

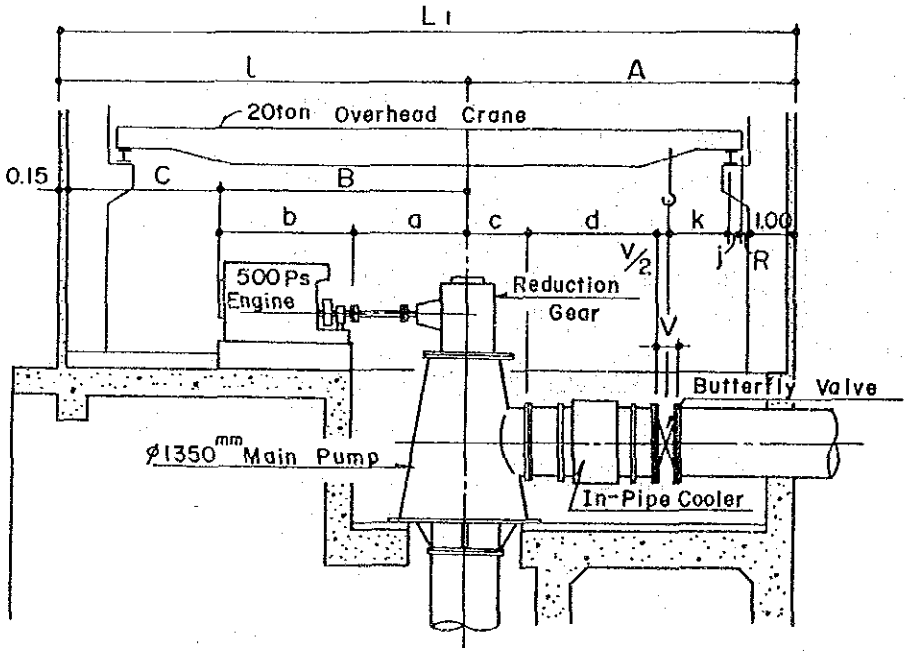
The scale of the pump house will be determined taking into account both the space required for installation of the necessary pump equipment and the space required to carry out successful operation and maintenance. The following areas will be included :

Pump room	; Main Pump (ø 1,350 mm × 4 units with vertical shaft-type mixed flow pump), Prime movers, Auxiliary equipment and devices
Electricity-room	; Electric panels
O/M room	; Office, Lavatory
Others	; Space for material entrance, including disassembly and reassembly of equipment

1) Length of Pump Room

The length of the pump room will be decided according to the length of the diesel-driven pump unit.

FIGURE 8.14 LENGTH OF PUMP ROOM



A $= c + d + v/2 + kjR + 1.00$ (Column width)
$= 1.30 + 2.70 + 0.40/2 + 1.20 + 0.32 + 1.00 = 6.72 ÷ 6.80$ m
B $= a + b = 2.00 + 3.00 = 5.00$
C $= 3.00$ m
ℓ $= B + C + 0.15 = 0.15 ÷ 8.20$ m

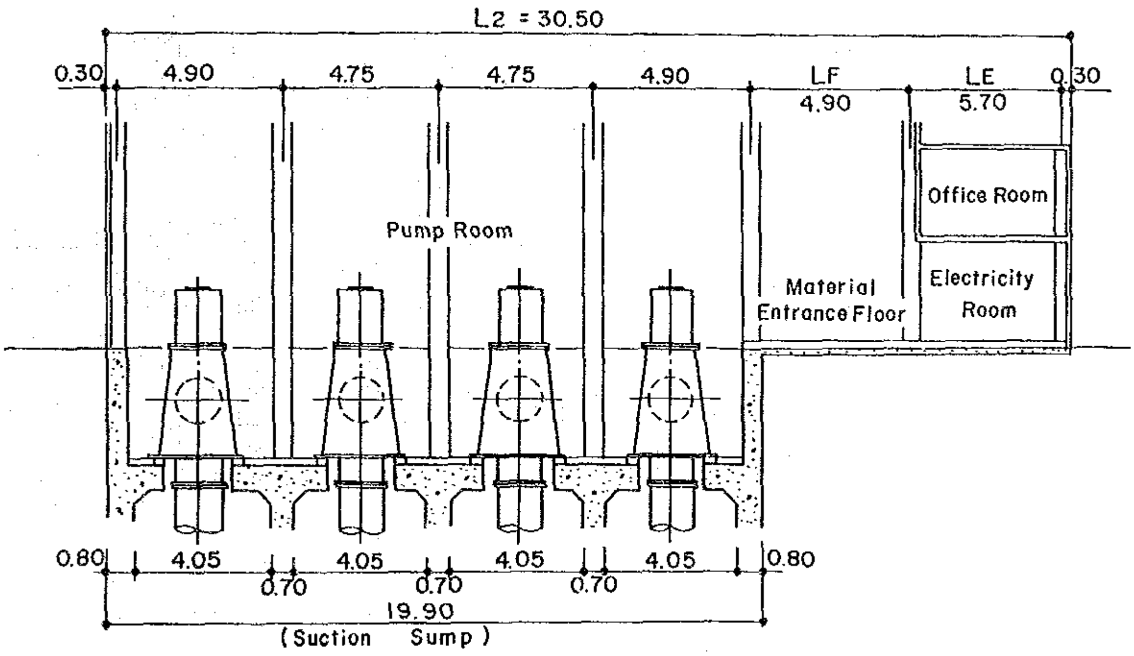
As a result of the above, the total length of the pump room (L_1) can be obtained as follows :

$$L_1 = A + \ell = 6.80 + 8.20 = 15.00\,\text{m}$$

2) Width of Pump House

The total width of the pump house will consist of the floor width of the pump installations, the material entrance, and the electricity room.

FIGURE 8-15 TOTAL WIDTH OF PUMP HOUSE



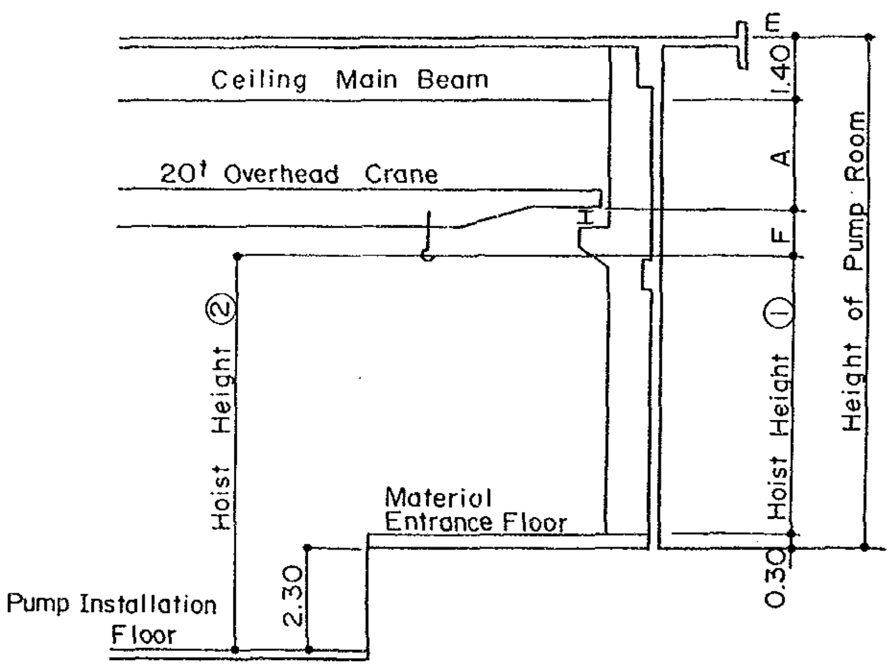
As shown in Figure 8-15, the total width of the pump house (L_2) is decided at 30.50 m including 4.90 m for the material entrance floor (L_F) and 5.70 m for the electricity room(L_E).

3) Height of Pump House

a) Pump Room

The height of the pump room will be determined according to the hoist height of the overhead crane and the crane measurements.

FIGURE 8-16 HEIGHT OF PUMP ROOM



The hoist height of the vertical shaft-type mixed flow pump with 1,350 mm dia. and the measurements of the 20 ton overhead crane are as follows:

Hoist height ① = 5.70 m
Hoist height ② = 8.50 m
F = 0.80 m
A = 2.50 m

As a result, the height of the pump room can be obtained as 11.00 m by the following calculation.

$$H_1 = 0.30 + 5.70 + 0.80 + 2.50 + 1.40 = 10.70 \text{ m} \leqq 11.00 \text{ m}$$
$$H_2 = 8.50 + 0.80 + 2.50 + 1.40 - 2.30 = 10.90 \text{ m} \leqq 11.00 \text{ m}$$

b) Electricity Room

The building for the electricity room will have two stories. The first floor (ground floor) is to be used as the electricity room, while the second floor is to be used for the office spaces.

The height of first floor room is 4.00 m, as considered the height of electric power receiving and distribution panels, etc.

8. 8 Design of Discharge Reservoir

The proposed discharge reservoir will be so designed as to allow the water released through the pipes to be promptly dissipated in the energy ensuring smooth flow into the following canal.

1) Discharge Water Level

The discharge water level under the canal design conditions is as follows:

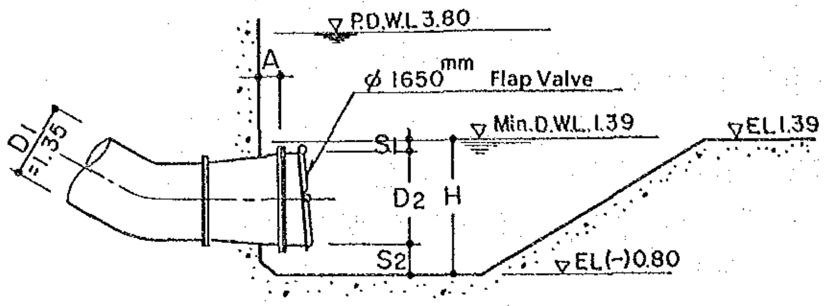
Design discharge water level = P.D.W.L. 3.80 m
Lowest discharge water level = Min. D.W.L. 1.39 m
(=Sill elevation of the main canal)

2) Dimensions of Discharge Reservoir

a) Water Depth in Discharge Reservoir

Since the vertical shaft-type pump is adopted for this pumping station, the end of the discharge pipe will be fixed by the lowest discharge water level to ensure effective dissipation of hydraulic energy, although it is not necessary for it to be fully submerged.

FIGURE 8 - 17 WATER DEPTH IN DISCHARGE RESERVOIR



$S_1 \geqq 0$ m, $D_2 = 1.65$, $S_2 \geqq 0.40$ m $\quad\quad \therefore H \geqq 2.05$ m

Then, the sill elevation of reservoir will be EL. (-)0.80 m.

b) Plane Scale of Discharge Reservoir

Determination of the discharge reservoir width depends on the installation intervals of the main pump units. The total net reservoir width will be 18.30 m. The reservoir length will be 43.50 m including open transition for smooth discharge the water into the main canal.

8. 9 Design of Waste Way

In the course of this detailed design, it was learned that the constructions of main irrigation canal to be connected with the pumping station may not be completed in time for the completion of this pumping station, the waste way and water measurement facilities, thereby, will be provided as a tentative facility for the test operations of pumps after its installation.

1) Design Water Discharge

The design water discharge for waste way is applied to be about 130% of design pump discharge a unit, as learned of the pump performance curve, planning the test operation of pump to be conducted one by one, and then given as follows:

$Q = Qp \times 1.30 = 4.00 \times 1.30 = 5.20$; use 5.5 m^3/s

where, Q : Design water discharge of waste way (m^3/s)

Qp : Design pump discharge a unit (4.00 m^3/s)

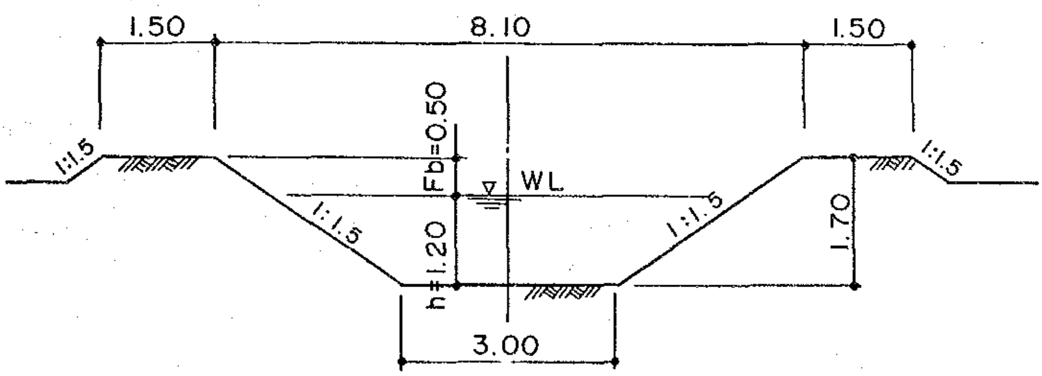
2) Water Measurement Facilities

The contracted rectangular weir will be provided as a water measurement instrument, which is comparatively simple in the structure and highly accurate in the measurement.

3) Channel Section of Waste Way

The waste way traverse the exiting pond in its middle section. The channel sections will be applied by Type-A and Type-B in the sections downstream and upstream from the said pond, respectively. The channel bottom elevations and slope of the channels will be determined by giving the water level not to exceed EL. 1.30 m in the existing pond. The channel sections are shown as follows:

FIGURE 8-18 TYPCIAL SECTION OF TYPE-A



Where, I = 1/550

A = 5.76 m^2

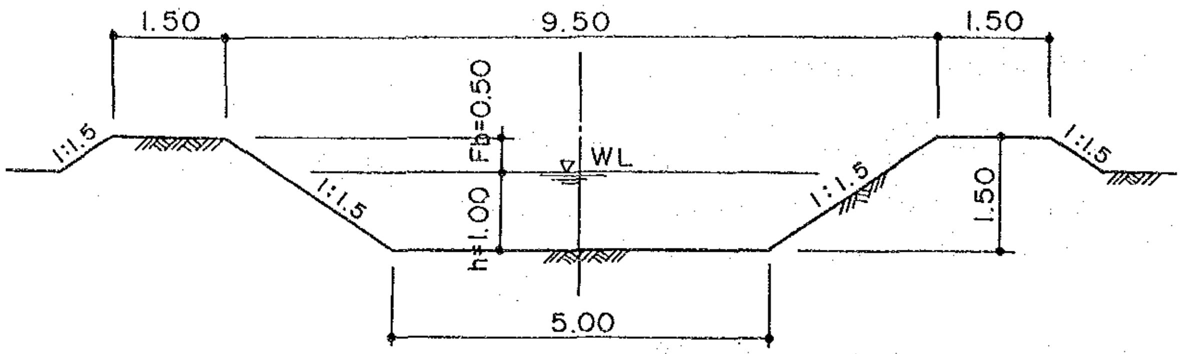
P = 7.38 m

$R^{2/3} = (5.76/7.33)^{2/3} = 0.852$

$V = \frac{0.852}{0.035} \times (\frac{1}{550})^{1/2} = 1.04$ m/sec

$Q = 5.76 \times 1.04 = 5.99 > 5.5$ m^3/sec

FIGURE 8-19 TYPICAL SECTION OF TYPE-B



Where,
$$I = 1/750$$
$$A = 6.50\ m^2$$
$$P = 8.61\ m$$
$$R^{2/3} = (6.50/8.61)^{2/3} = 0.829$$
$$V = \frac{0.829}{0.035} \times (\frac{1}{750})^{1/2} = 0.865\ m/sec$$
$$Q = 6.50 \times 0.865 = 5.62 > 5.5\ m^3/sec$$

8.10 Design Criteria of Pump House

1) Regulation, Specification and Standard

a) General

Additional Regulation No. 6 for Control of the Construction by Ministry of Interior, Thailand - 1985.

b) Reinforced Concrete

Building Code Requirements for Reinforced Building - ASD of the American Institute of Steel Construction (AISC).

c) Loads

"Minimum Design Loads for Building and Other Structures" ANSI A58.1 - 1982, American National Standard (ANSI)

2) Dead Loads

		W_{DL} (kg/m^2)
a)	Roof	520
b)	Concrete Block Wall	120
c)	Office Room	460
d)	Roof for 2nd Story	640
e)	Electricity Room	1,130
f)	Material Entrance Floor	1,130

3) Live Loads

		W_{LL} (kg/m^2)
a)	Roof	100
b)	Office Room	250
c)	Storage	600
d)	Corridor	250
e)	Electricity Room	500
f)	Material Entrance Floor	500

4) Wind Loads

Minimum wind pressure : q
Wind speed V = 30 m/sec
$\therefore$ q $= 0.0625\,(30)^2 = 56.25$ kg/m^2 say 56.3 kg/m^2

H	q (kg/m^2)
$H \leqq 10$ m	$50 < 56.3 \therefore 56.3$
$< H \leqq 20$ m	80

H : Height of structure

Wind coefficient is in accordance with ANSI A58.1 6. Wind Loads.

5) Reinforced concrete

In accordance with ACI 381M - 89, Ultimate Strength Method, except the factor loads and combination loads are as follows:

$$U = 1.7\,D + 2L$$
$$U = 0.75\,(1.7\,D + 2L + 2W)$$
$$U = 0.9\,D + 1.3\,W$$

Compressive Strength of Concrete;
$fc = 210\ kg/cm^2$ (20.6 M Pa)

Yield Strength of Reinforcement Steel Bar;
$fy = 3{,}000\ kg/cm^3$ (294 MPa)

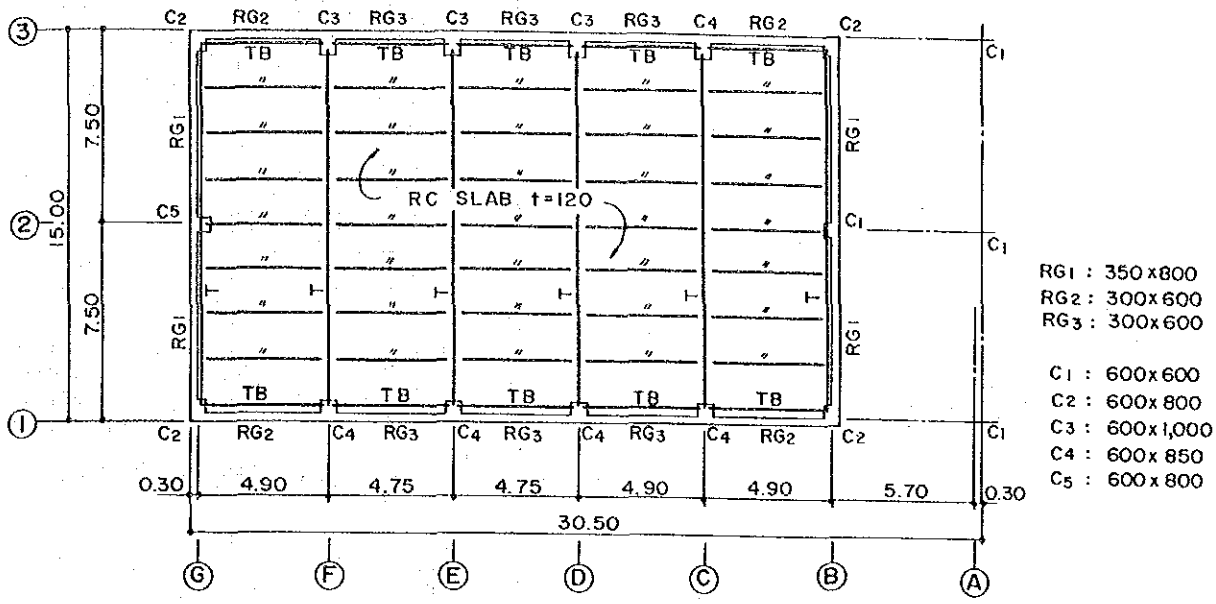
6) Steel Structure

In accordance with "Specification for Structural Steel Building - ASD" of the American Institute of Steel Construction (AISC), 1989.

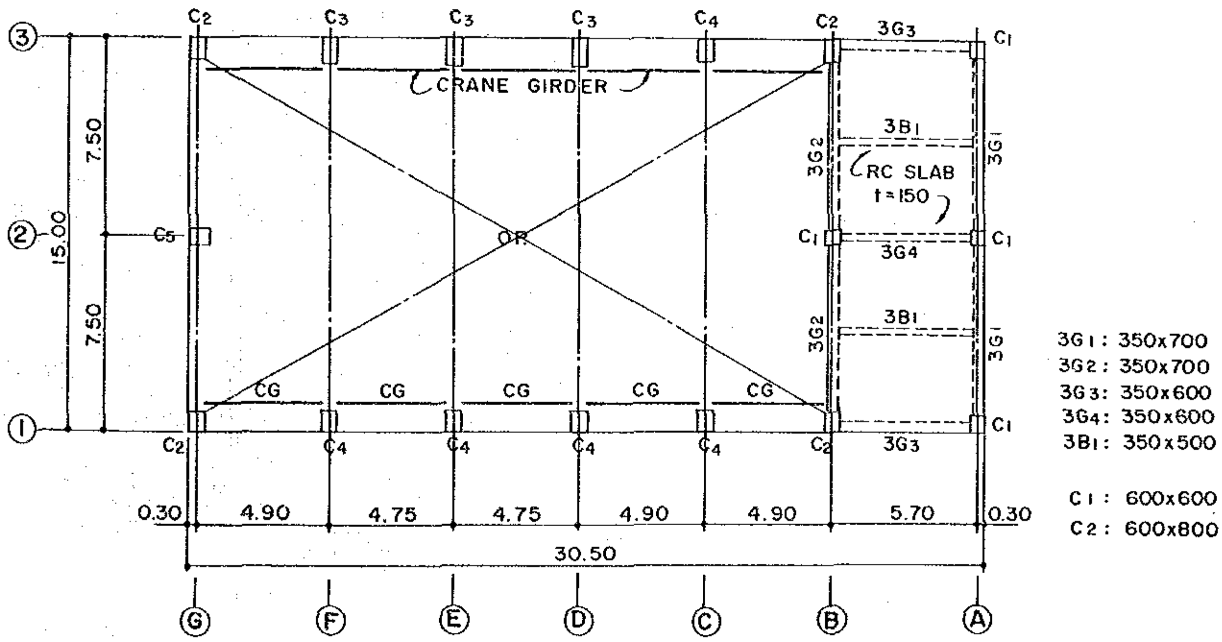
Minimum Yeild Stress;
$Fy = 2{,}400\ kg/cm^2$ (235 MPa)

Maximum Allowable Tension and Flexure Stress;
$Ft = Fb = 1{,}200\ kg/cm^2$

FIGURE 8 - 20 ROOF FRAMING PLAN

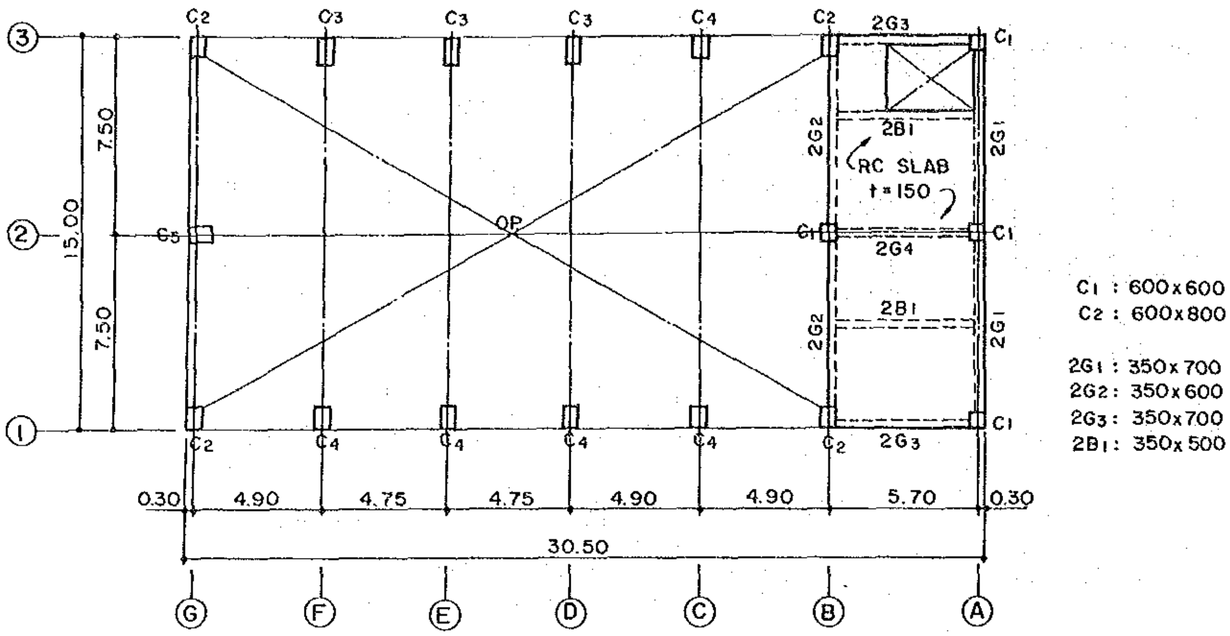


ROOF FRAMING PLAN
(GL+11.00m)

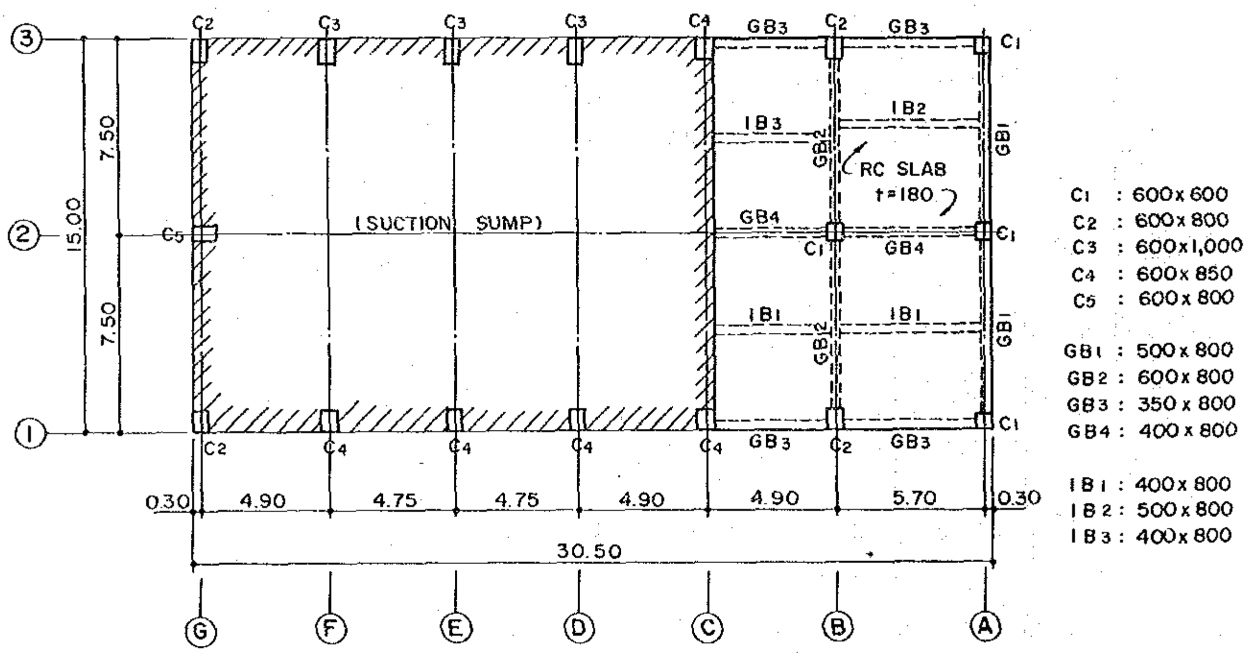


ROOF FRAMING PLAN
(GL+7.50m)

FIGURE 8 - 21 FLOOR FRAMING PLAN

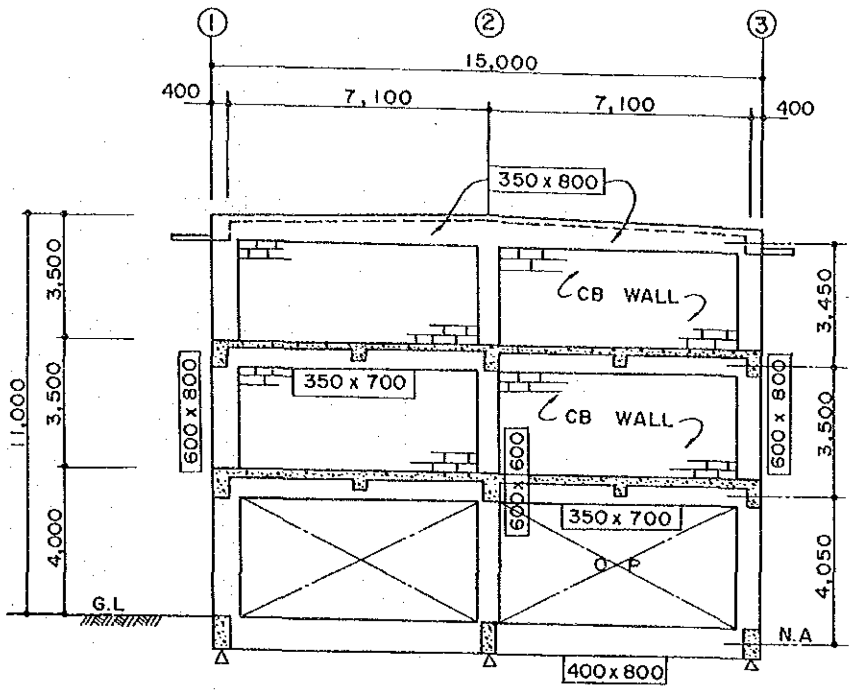


2nd FLOOR FRAMING PLAN
(GL+4.00m)

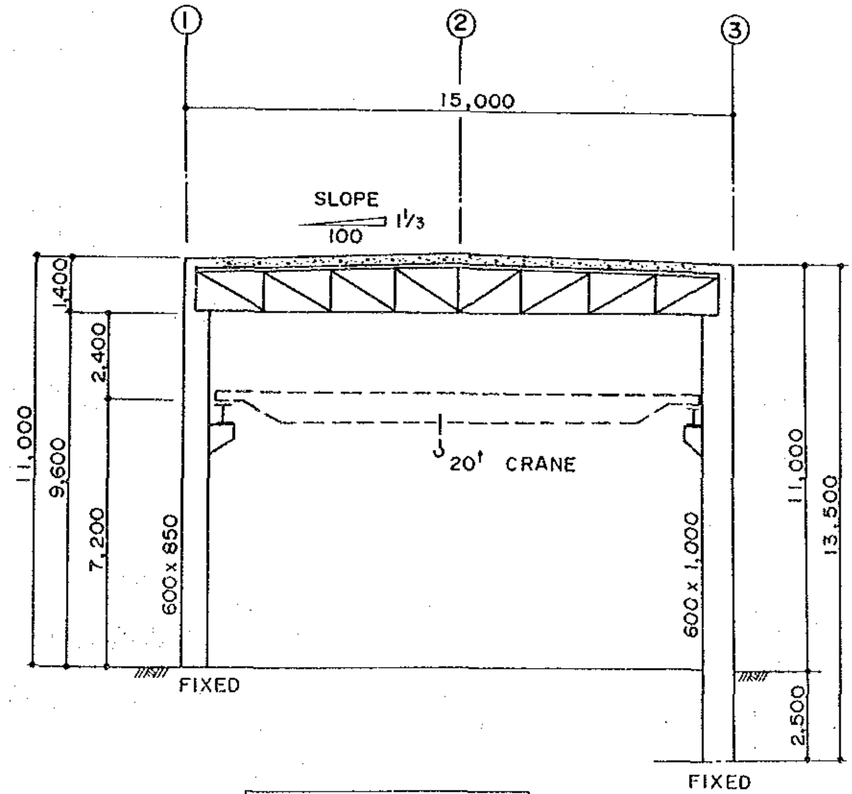


1st FLOOR FRAMING PLAN
(GL±0)

FIGURE 8-22 TYPICAL TRANSVERSE FRAME

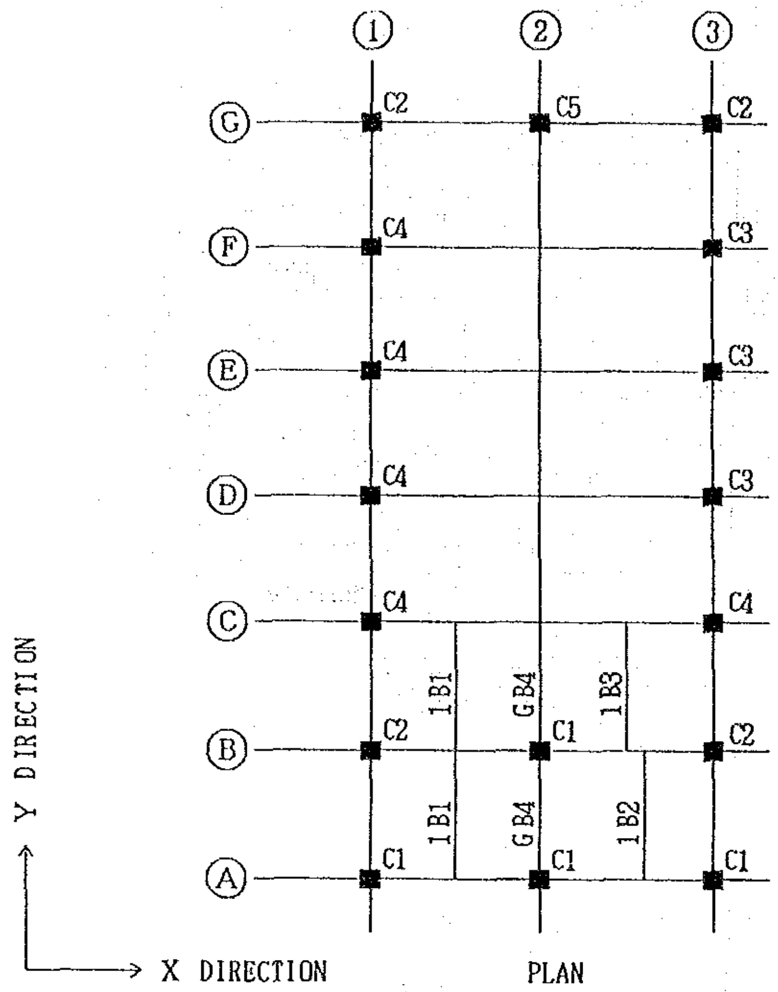


Ⓑ FRAME



Ⓓ Ⓔ Ⓕ FRAME

FIGURE 8-23 COLUMN LOADING DIAGRAM



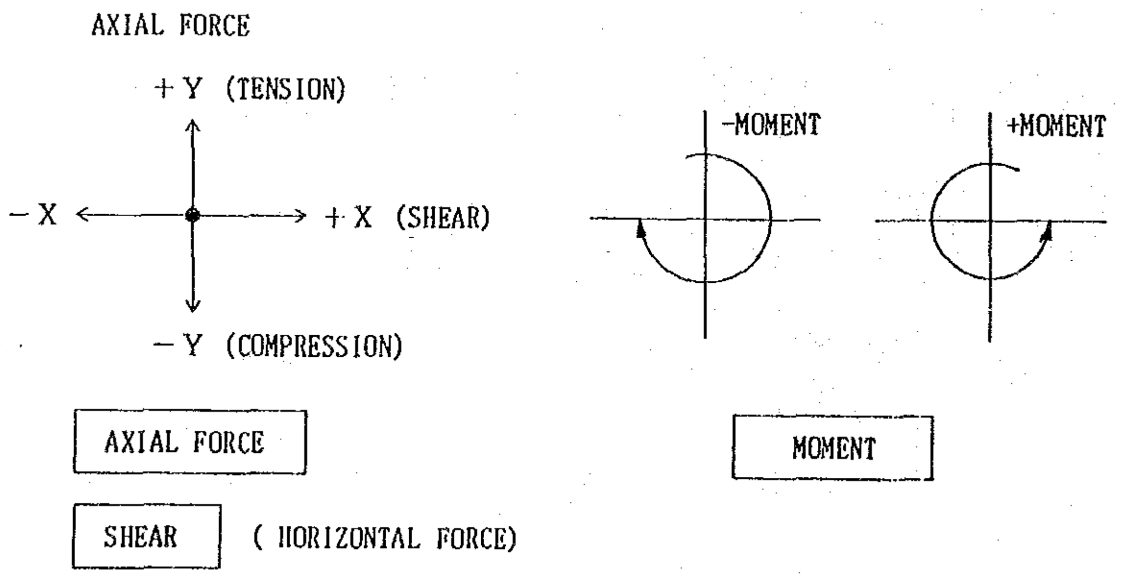


TABLE 8-9 LOADING DATA OF COLUMNS

NON-FACTORED LOADING DATA

UNIT : TON AND TON-METER

	COL.MARK	DEAD LOAD					LIVE LOAD					WIND LOAD					
		AXIAL	X-DIRECTON		Y-DIRECTON		AXIAL	X-DIRECTON		Y-DIRECTON		X-DIRECTON			Y-DIRECTON		
		FORCE P	SHEAR V	MOMENT M	SHEAR V	MOMENT M	FORCE P	SHEAR V	MOMENT M	SHEAR V	MOMENT M	AXIAL P	SHEAR V	MOMENT M	AXIAL P	SHEAR V	MOMENT M
C 1	A-1 A-3	-75	-1.7	0	-1.2	0	-10	-0.3	0	-0.13	0	+0.6 -0.6	+0.44 -0.44	0	-5 +5	-2 +2	0
C 1	A-2	-110	0	0	-0.9	0	-20	0	0	-0.2	0	0	+0.65 -0.65	0	0	+2.6 -2.6	0
C 2	B-1 B-3	-125	-2.0	0	-1.2	0	-42	-0.3	0	-0.13	0	+1.7 -1.7	+1.1 -1.1	0	+4 -4	-2.2 +2.2	0
C 2	B-2	-150	0	0	-0.9	0	-30	0	0	+0.2	0	+1.4 -1.4	0	0	0 0	+2.7 -2.7	0
C 4	D-1 to F-1	-60	0	-7.4	0	0	-30	-2.2 +2.2	-5.5 -37	0	0	0	+4.0 -4.0	+27 -27	0 0	-0.6 +0.6	+3.6 -3.6
C 2	G-1	-45	-1.2	+4	0	0	-26	+1.1	-6.0	0	0	+0.3 -0.3	-0.6 +1.3	+3.5 -5	+0.5 -0.5	-0.5 +0.5	+3.0 -3.0
C 3	D-3 to F-3	-70	0	+8	0	0	-30	+2.2 -2.2	0 +43	0	0	0	+5.0 -5.0	+39 -39	0 0	-0.6 +0.6	+3.6 -3.6
C 2	G-3	-50	+1.1	-5.1	0	0	-26	+3.0	-13.0	0	0	-0.3 +0.3	-1.3 +0.3	+5.4 -2.0	+0.5 -0.5	-0.5 +0.5	+3.0 -3.0
C 5	G-2	-50	+0.1	-0.6	0	+12.1	-2	+0.4	-2.2	0	+1.0	+0.1 -0.1	+0.3 -0.3	-1.6 +1.6	0 0	-6.8 +6.8	+38 -38
C 4	C-1	-70	0	+7.4	0	0	-35	-2.2 +2.2	-5.5 -37	0	0	0 0	+4.0 -4.0	+27 -27	0 0	-0.6 +0.6	+3.6 -3.6
C 4	C-3	-80	0	+8	0	0	-35	+2.2 -2.2	0 +43	0	0	0 0	+5.0 -5.0	+39 -39	0 0	-0.6 +0.6	+3.6 -3.6
	FLOOR BM(1B1) C END	-15	0	0	0	0	-5	0	0	0	0	0	0	0	0	0	0
	GRADE BM(GB4) C END	-25	0	0	0	0	-10	0	0	0	0	0	0	0	0	0	0

− : CONPRESSON
+ : TENSION

8-11 Table and Figure

TABLE 8-11-1 TEN-DAY WATER REQUIREMENTS (WITHOUT RAINFALL)

Season	Mon		W.Paddy (Trans.) Req.	W.Paddy (Trans.) A=8910ha	W.Paddy (Broad.) Req.	W.Paddy (Broad.) A= 990ha	W.Paddy (Broad.) Req.	W.Paddy (Broad.) A=1980ha	Soybean Req.	Soybean A= 280ha	Groundnuts Req.	Groundnuts A= 920ha	Mungbean Req.	Mungbean A=1780ha	Orchard Req.	Orchard A=4160ha	Vegetable Req.	Vegetable A=(1420)ha	F.Pond Req.	F.Pond A= 400ha	S.Pond q	S.Pond A= 981ha	W.Supply V=2.2 MCM/Year	TOTAL
Dry	4	1	—	m^3/s	—	m^3/s	4.35	m^3/s 1.675	0.63	m^3/s 0.042	3.08	m^3/s 0.669	0.44	m^3/s 0.185	2.69	m^3/s 2.643	3.37	720 m^3/s 0.573	—	m^3/s	0.6319	m^3/s 0.689	m^3/s 0.073	m^3/s 6.550
		2	—		—		2.48	0.955	—		1.77	0.385	1.69	0.711	2.69	2.643	3.37	0.573	—		0.6319	0.689	0.073	6.029
		3	—		—		0.79	0.304	—		0.56	0.122	3.41	1.434	2.69	2.643	3.37	0.573	—		0.6319	0.689	0.073	5.838
Wet	5	1	—		—		—		—		—		4.93	2.073	1.99	1.955	2.90	0.493	—		—		0.073	4.595
		2	—		—		—		—		—		5.04	2.119	1.99	1.955	2.90	0.493	—		—		0.073	4.640
		3	—		—		—		—		—		4.06	1.707	1.99	1.955	2.90	0.493	—		—		0.073	4.229
	6	1	—		—		—		—		—		1.73	0.727	—		1.14	240 0.065	16.30	0.545	—		0.070	1.407
		2	—		—		—		—		—		0.66	0.277	—		1.14	0.065	17.80	0.595	—		0.070	1.007
		3	—		—		—		—		—		0.11	0.046	—		1.14	0.065	19.31	0.646	—		0.070	0.827
	7	1	—		5.11	0.984	—		—		—		—		—		—		20.72	0.693	—		0.070	1.747
		2	—		6.54	1.259	—		—		—		—		—		—		14.25	0.476	—		0.070	1.806
		3	2.78	4.318	8.06	1.552	—		—		—		—		—		—		6.65	0.222	—		0.070	6.663
	8	1	5.16	8.943	9.46	1.822	—		—		—		—		—		2.10	0.119	6.39	0.214	—		0.073	11.171
		2	6.64	11.508	8.67	1.670	—		—		—		—		—		2.10	0.119	6.39	0.214	—		0.073	13.584
		3	8.27	14.334	6.89	1.327	—		—		—		—		—		2.10	0.119	6.39	0.214	—		0.073	16.067
	9	1	8.84	15.321	6.89	1.327	—		—		—		—		—		2.70	0.153	6.00	0.201	—		0.066	17.058
		2	7.24	12.548	7.01	1.350	—		—		—		—		—		2.70	0.153	5.83	0.195	—		0.066	14.312
		3	6.97	12.080	6.97	1.342	—		—		—		—		—		2.70	0.153	4.67	0.156	—		0.066	13.797
	10	1	7.21	12.496	7.21	1.388	—		—		—		—		—		2.94	0.167	3.42	0.114	—		0.066	14.231
		2	6.72	11.647	6.72	1.294	—		—		—		—		—		2.94	0.167	2.05	0.069	—		0.066	13.243
		3	5.25	9.099	5.25	1.011	—		—		—		—		—		2.94	0.167	0.68	0.023	—		0.066	10.356
Dry	11	1	3.86	6.690	3.86	0.743	—		—		—		—		—		0.82	0.046	16.29	0.545	—		0.066	8.091
		2	2.21	3.830	2.21	0.426	—		—		—		—		—		0.82	0.046	17.76	0.594	—		0.066	4.963
		3	0.70	1.213	0.70	0.135	—		—		—		—		—		0.82	0.046	19.24	0.643	—		0.066	2.104
	12	1	—		—		5.18	1.995	—		—		—		3.80	3.734	1.69	1,420 0.567	20.92	0.699	—		0.066	7.062
		2	—		—		6.75	2.600	0.07	0.005	0.08	0.017	—		3.80	3.734	1.69	0.567	14.49	0.484	—		0.066	7.473
		3	—		—		8.42	3.243	0.70	0.046	0.76	0.165	—		3.80	3.734	1.69	0.567	6.90	0.231	—		0.066	8.052
	1	1	—		—		10.14	3.905	1.63	0.108	1.72	0.374	—		3.65	3.587	3.36	1.127	6.86	0.229	0.5417	0.590	0.066	9.987
		2	—		—		9.55	3.678	2.73	0.181	2.86	0.622	—		3.65	3.587	3.36	1.127	6.86	0.229	0.5417	0.590	0.066	10.080
		3	—		—		7.86	3.027	3.74	0.247	3.69	0.802	—		3.65	3.587	3.36	1.127	6.86	0.229	0.5417	0.590	0.066	9.676
	2	1	—		—		8.52	3.282	4.70	0.311	4.51	0.980	—		3.59	3.528	3.59	1.204	7.25	0.242	1.0833	1.181	0.070	10.797
		2	—		—		8.67	3.339	5.12	0.339	4.79	1.041	—		3.59	3.528	3.59	1.204	7.05	0.236	1.0833	1.181	0.070	10.937
		3	—		—		8.62	3.320	5.17	0.342	4.75	1.032	—		3.59	3.528	3.59	1.204	5.64	0.189	1.0833	1.181	0.070	10.865
	3	1	—		—		8.87	3.416	5.56	0.368	5.17	1.123	—		3.32	3.262	2.42	720 0.412	4.33	0.145	1.1736	1.279	0.077	10.082
		2	—		—		8.24	3.174	3.99	0.264	4.62	1.004	—		3.32	3.262	2.42	0.412	2.60	0.087	1.1736	1.279	0.077	9.558
		3	—		—		6.42	2.473	2.07	0.137	4.13	0.897	—		3.32	3.262	2.42	0.412	0.37	0.029	1.1736	1.279	0.077	8.565
TOTAL			(E=0.595) 110.03 MCM		(E=0.595) 15.57 MCM		(E=0.595) 35.08 MCM		(E=0.490) 2.04 MCM		(E=0.490) 7.96 MCM		(E=0.490) 8.16 MCM		(E=0.490) 48.97 MCM		(E=0.490) 12.81 MCM		(E=0.900, A=0.650) 8.16 MCM		(E=0.900) 9.65 MCM		2.20 MCM	260.63 MCM

FIGURE 8 - 11 - 1 TEN-DAY WATER REQUIREMENTS FOR 20-YEAR (WITH RAINFALL)

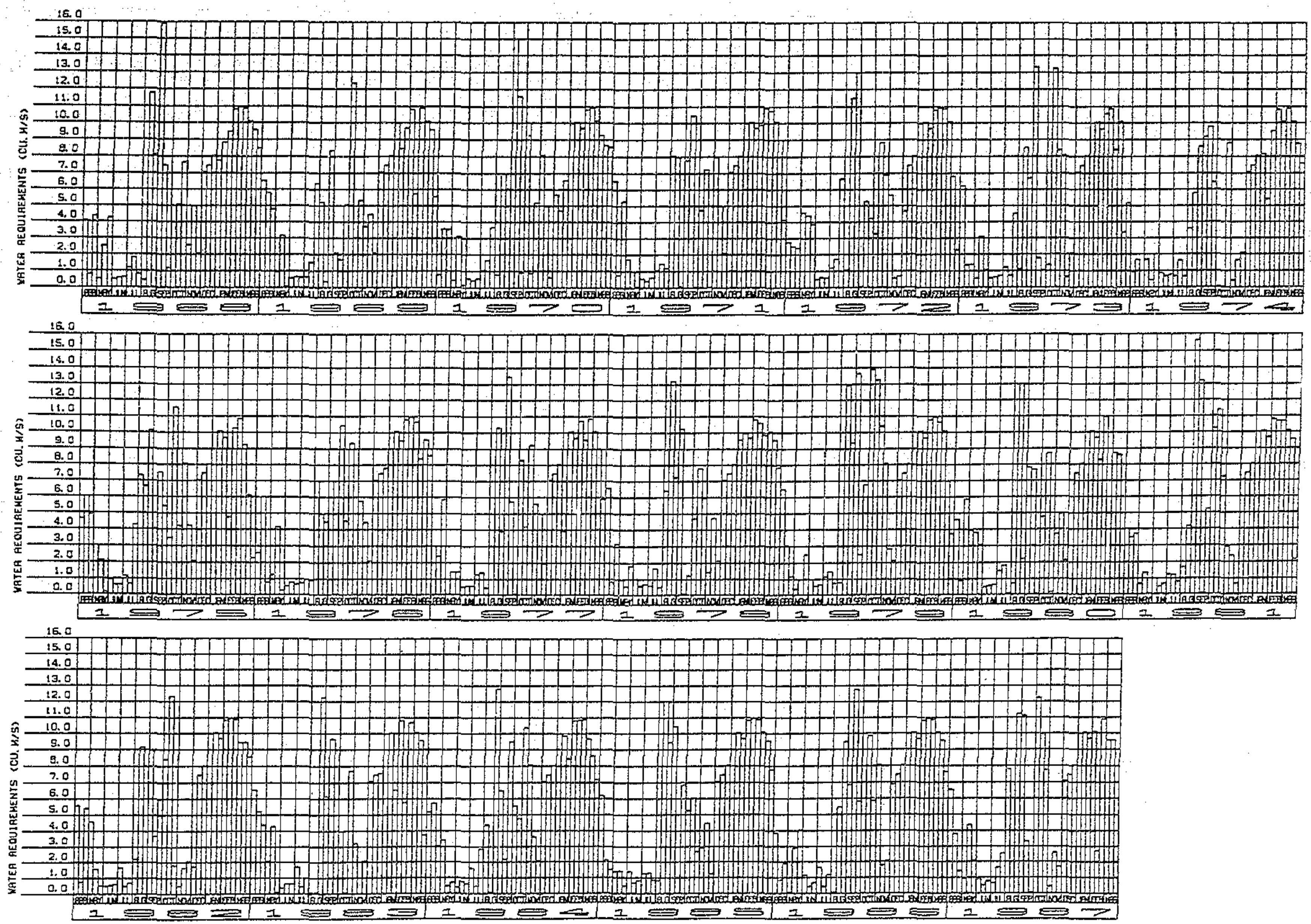


TABLE 8-11-2 PUMP OPERATION COST FOR EACH CASE

Item \ No. of Pumps	3 − Pumps		4 − Pumps (1)	4 − Pumps (2)		4 − Pumps (3)		5 − Pumps	
Type of Prime Mover	Motor	Engine	Engine	Motor	Engine	Motor	Engine	Motor	Engine
Prime Mover output	470kw	680PS	350kw	350kw	500ps	350kw	500ps	280kw	410ps
Required Input	461.2kw		342.3kw	342.3kw		342.3kw		275.6kw	
No. of Prime Mover	2	1	4	3	1	2	2	4	1
Total Annual Operation Hours	10,988hr	69hr	13,303hr	13,279hr	24hr	12,427kw	876hr	15,686hr	15hr
1) Electric Charge	B		B	B		B		B	
a) Demand Charge	(470X2X167X12)		(350X4X167X12)	(350X3X167X12)		(350X2X167X12)		(280X4X167X12)	
(1.67 B/kw·Month)	1,883,760		2,805,600	2,104,200		1,402,800		2,244,480	
b) Energy Charge	(461.2X10,988X1.23)		(342.3X13,303X1.23)	(342.3X13,279X1.23)		(342.3X12,427X1.23)		(275.6X15,686X1.23)	
(1.23 B/kw·hr)	6,233,228		5,600,948	5,590,844		5,232,127		5,317,365	
Sub-Total	8,116,988		8,406,548	7,695,044		6,634,927		7,561,845	
2) Fuel Cost <*>									
300∼ 500PS···1.41 B/Ps·hr	(680X69X1.73)			(500X24X1.73)		(500X876X1.73)		(410X15X1.93)	
500∼1,000PS···1.27 B/Ps·hr	81,171		−	20,760		757,740		11,869	
3) Annual Operation Cost (Total)	8,198,159		8,406,548	7,715,804		7,392,667		7,573,714	
	'000 B		'000 B	'000 B		'000 B		'000 B	
Twenty Years Operation Cost	163,963		168,131	154,316		147,853		151,474	

<*> ; 300∼ 500 PS ····· 0.20 kg/Ps·hr × 1/0.83 kg/ℓ × (8B/ℓ) = 1.93 B/Ps·hr

500∼1,000 PS ····· 0.18 kg/Ps·hr × 1/0.83 kg/ℓ × (8B/ℓ) = 1.73 B/Ps·hr

TABLE 8-11-3 PRIME MOVER OUTPUT FOR EACH CASE

Item \ No. of Pumps		3 – Pumps		4 – Pumps		5 – Pumps	
Pump Bore	(mm)	φ 1,500		φ 1,350		φ 1,200	
Discharge/Unit (Q)	(m^3/min)	320		240		192	
Actual Head (ha)	(m)	5.40		5.40		5.40	
Type of Prime Mover		Motor	Engine	Motor	Engine	Motor	Engine
Loss Head (h_a)	(m)	0.80	1.00	0.70	0.80	0.70	0.90
Total Head (H)	(m)	6.20	6.40	6.10	6.20	6.10	6.30
Pump Efficiency (η_p)		0.84		0.835		0.83	
Transmission Efficiency (η_G)		0.96		0.96		0.96	
Extra Coefficient (R)		0.15	0.20	0.15	0.20	0.15	0.20
Prime Mover Output (P)		(461.2) 470KW	(676.6) 680PS	(342.3) 350KW	(494.5) 500PS	(275.6) 280KW	(404.4) 410PS

$$P = K \cdot \frac{r \cdot Q \cdot H}{\eta_p \cdot \eta_G} \cdot (1 + R)$$

Where K ; coefficient (0.163 in KW, 0.222 in Ps)

r ; specific gravity of water (1.0)

TABLE 8-11-4 HEAD LOSSES OF DISCHARGE PIPE FOR EACH CASE

Item \ No. of Pumps			3 − Pumps		4 − Pumps		5 − Pumps	
Pump Bore		(mm)	φ 1,500		φ 1,350		φ 1,200	
Discharge/Unit		(m^3/s)	5.333		4.000		3.200	
Discharge Pipe Bore		(m)	1.50	1.80	1.35	1.65	1.20	1.50
Area		(m^2)	1.767	2.545	1.431	2.138	1.131	1.767
Velocity		(m/s)	3.018	2.095	2.795	1.871	2.829	1.811
Velocity Head		(m)	0.465	0.224	0.399	0.179	0.408	0.167
Friction Loss Coefficient		(λ)	0.0305	0.0304	0.0306	0.0305	0.0306	0.0305
Water Conveyrance Slope $I=\lambda\cdot\frac{1}{D}\cdot\frac{V^2}{2g}$		($\times 10^{-3}$)	9.455	3.783	9.044	3.309	10.404	3.396
Loss Heads (m)	Butterfly Valve	$f_v=0.30$		0.140		0.120		0.122
	Bend (45° × 2)	$f_B=0.12$		0.112		0.096		0.098
	Enlarge of Area	$f_E=0.46$		0.020		0.020		0.024
	Flap Valve		($f_v=0.57$)	0.128	($f_v=0.62$)	0.111	($f_v=0.68$)	0.114
	Water Release	$f_o=1.0$		0.224		0.179		0.167
	Friction ($\ell \fallingdotseq 13.5$ m)			0.127		0.121		0.139
	In-Pipe Cooler (Only for Engine)	$f_c=0.35$		0.163		0.140		0.143
Total Loss Head	For Motor	(m)	0.751 ≒ 0.80		0.647 ≒ 0.70		0.664 ≒ 0.70	
	For Engine	(m)	0.914 ≒ 1.00		0.787 ≒ 0.80		0.807 ≒ 0.90	

TABLE 8-11-5 PUMP OPERATION HOURS FOR EACH CASE (1968-1987)

(Hour/Year)

No. of Pumps / Year	3-Pumps			4-Pumps				5-Pumps				
	No. 1	No. 2	No. 3	No. 1	No. 2	No. 3	No. 4	No. 1	No. 2	No. 3	No. 4	No. 5
1968	8,784.0	2,316.7	171.5	8,784.0	3,750.5	871.4	146.5	8,784.0	4,532.4	2,257.7	380.9	123.1
1969	8,760.0	1,751.3	25.2	8,760.0	3,263.2	621.0	0	8,760.0	4,303.8	1,752.5	154.8	0
1970	〃	2,258.0	134.9	〃	3,594.2	954.1	99.7	〃	4,239.0	2,387.4	351.1	64.6
1971	〃	2,186.1	0	〃	3,833.5	626.4	0	〃	4,659.8	2,194.1	33.6	0
1972	8,784.0	2,156.3	56.1	8,784.0	3,342.3	972.4	0	8,784.0	4,161.5	2,217.1	303.8	0
1973	8,760.0	2,219.9	127.3	8,760.0	3,607.2	943.0	9.6	8,760.0	4,245.7	2,224.2	393.9	0
1974	〃	1,953.4	0	〃	3,576.9	460.1	0	〃	4,157.5	1,956.9	32.8	0
1975	〃	1,916.3	0	〃	3,422.5	619.7	0	〃	4,537.3	1,893.3	85.7	0
1976	8,784.0	2,076.9	0	8,784.0	3,612.5	642.8	0	8,784.0	4,526.8	2,120.5	46.2	0
1977	8,760.0	2,130.9	71.1	8,760.0	3,676.9	784.6	14.8	8,760.0	4,446.2	2,235.6	224.8	0
1978	〃	2,252.1	60.3	〃	3,820.0	839.0	0.3	〃	4,633.4	2,269.6	199.0	0
1979	〃	3,001.2	275.0	〃	4,284.1	1,644.6	62.3	〃	4,826.8	3,071.0	825.3	0
1980	8,784.0	2,002.4	52.9	8,784.0	3,581.7	585.6	0	8,784.0	4,338.1	1,942.6	193.8	0
1981	8,760.0	2,574.7	235.5	8,760.0	3,625.6	1,314.2	145.8	8,760.0	4,096.4	2,738.9	531.2	116.2
1982	〃	2,271.4	24.5	〃	3,688.2	790.2	0	〃	4,176.2	2,430.2	179.2	0
1983	〃	1,806.5	23.9	〃	3,081.1	700.9	0	〃	3,963.6	1,805.2	151.1	0
1984	8,784.0	2,288.0	47.9	8,784.0	3,843.6	851.8	0	8,784.0	4,666.9	2,325.0	218.8	0
1985	8,760.0	2,357.4	12.7	8,760.0	3,474.0	919.9	0	8,760.0	4,414.0	2,471.9	159.6	0
1986	〃	2,390.0	42.1	〃	3,912.7	928.7	0	〃	4,628.6	2,494.6	208.5	0
1987	〃	2,535.2	20.0	〃	3,946.6	962.7	0	〃	4,566.7	2,598.5	220.2	0
Max.	8,784.0	3,001.2	275.0	8,784.0	4,284.1	1,644.6	146.5	8,784.0	4,826.8	3,071.0	825.3	123.1
Min.	8,760.0	1,751.3	0	8,760.0	3,081.8	460.1	0	8,760.0	3,963.6	1,752.5	32.8	0
Mean	8,766.0	2,222.2	69.0	8,766.0	3,660.5	851.6	23.9	8,766.0	4,405.5	2,269.3	244.8	15.2

Note ; 1) No.1 pump operation hour is calculated in case of value control.
2) Operation hour except No.1 pump is calculated in case of on-off control.

FIGURE 8 - 11 - 2 SCHEMATIC DIAGRAM FOR IRRIGATION NETWORK IN BANG PAKONG LEFT BANK AREA

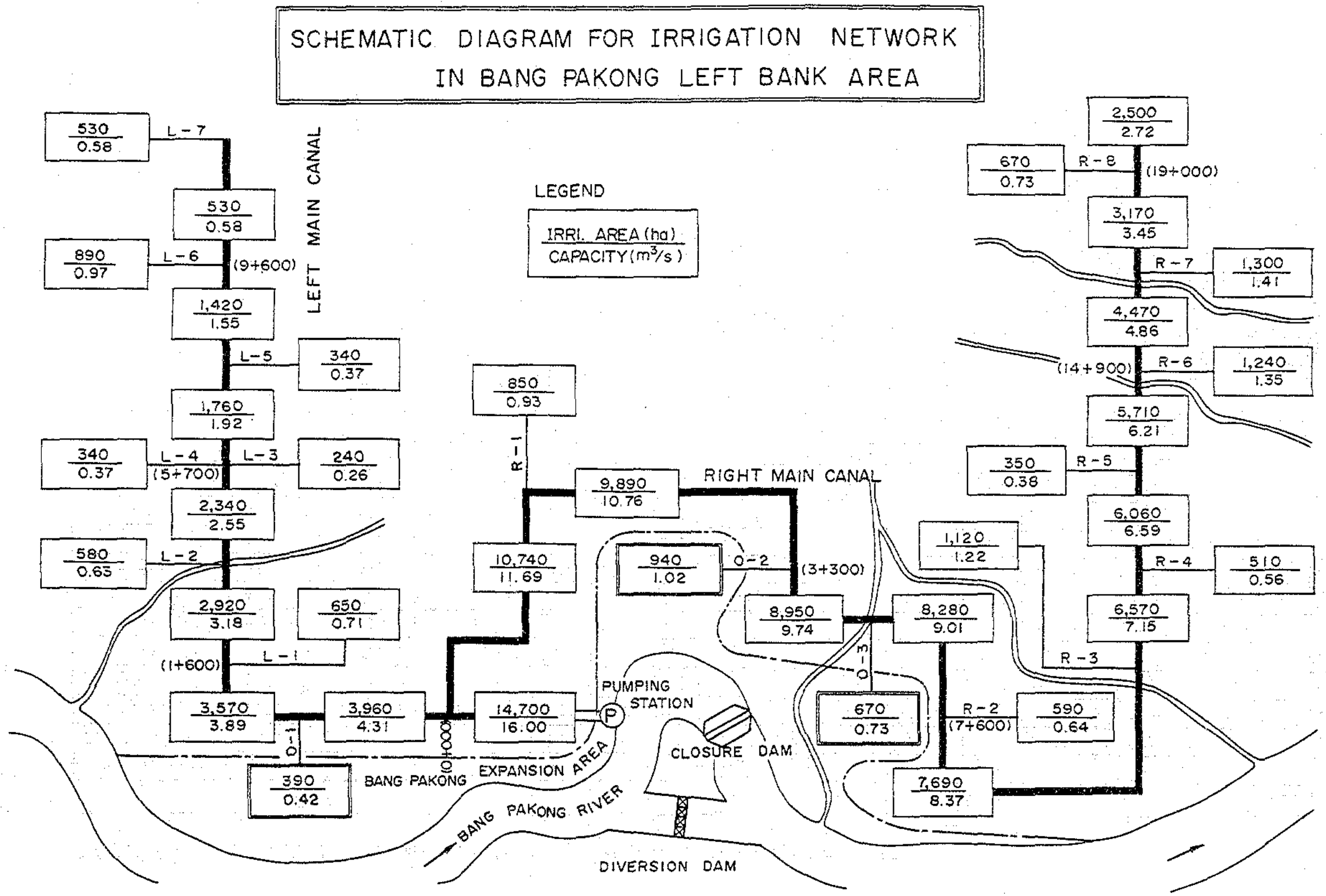
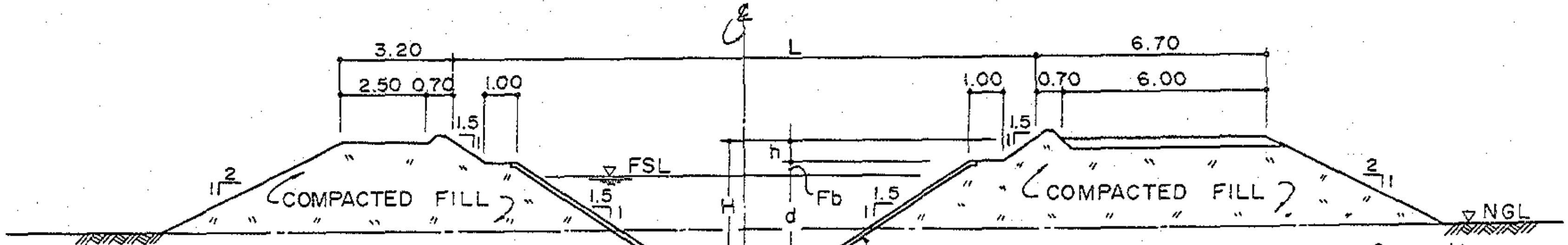


TABLE 8-11-6 MAIN CANAL DIMENSIONS

TYPICAL SECTION OF MAIN CANAL



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

$$n = 0.015$$

MAIN CANAL DIMENSIONS

STATION		Q (m³/s)	A (m²)	V (m/s)	R (m)	I	B (m)	d (m)	Fb (m)	h (m)	H (m)	L (m)	REMARKS
MAIN CANAL													
0+131	1+330	16.00	20.154	0.795	1.492	1/12000	5.00	2.36	0.34	0.60	3.30	16.90	FSL 3.75
LEFT MAIN CANAL													
0+000	1+600	4.31	6.440	0.677	0.865	1/8000	2.00	1.51	0.29	0.40	2.20	10.60	FSL 3.50
1+600	5+700	3.18	5.105	0.626	0.770	1/8000	1.80	1.34	0.26	0.40	2.00	9.80	
5+700	9+600	1.92	3.497	0.553	0.639	1/8000	1.40	1.13	0.27	0.30	1.70	8.50	
9+600	12+000	0.58	1.435	0.409	0.407	1/8000	1.00	0.70	0.20	0.30	1.20	6.60	
RIGHT MAIN CANAL													
0+000	3+300	11.69	17.717	0.660	1.402	1/16000	4.60	2.23	0.37	0.60	3.20	16.20	FSL 3.50
3+300	7+600	9.74	15.435	0.631	1.311	1/16000	4.20	2.10	0.30	0.50	2.90	14.90	
7+600	14+900	8.37	13.797	0.610	1.245	1/16000	3.80	2.02	0.28	0.50	2.80	14.20	
14+900	19+000	4.86	9.174	0.535	1.023	1/16000	2.80	1.71	0.29	0.40	2.40	12.00	
19+000	24+000	2.72	5.927	0.465	0.828	1/16000	2.00	1.43	0.27	0.40	2.10	10.30	

APPENDIX - 9 : DESIGN OF CONTROL SYSTEM

APPENDIX - 9. DESIGN OF CONTROL SYSTEM

LIST OF CONTENTS

9.1 Location of Gauging Station

9.1.1 Figure

FIGURE 9-1-1 LOCATION OF BAN SANG STATION (PRACHIN RIV.)

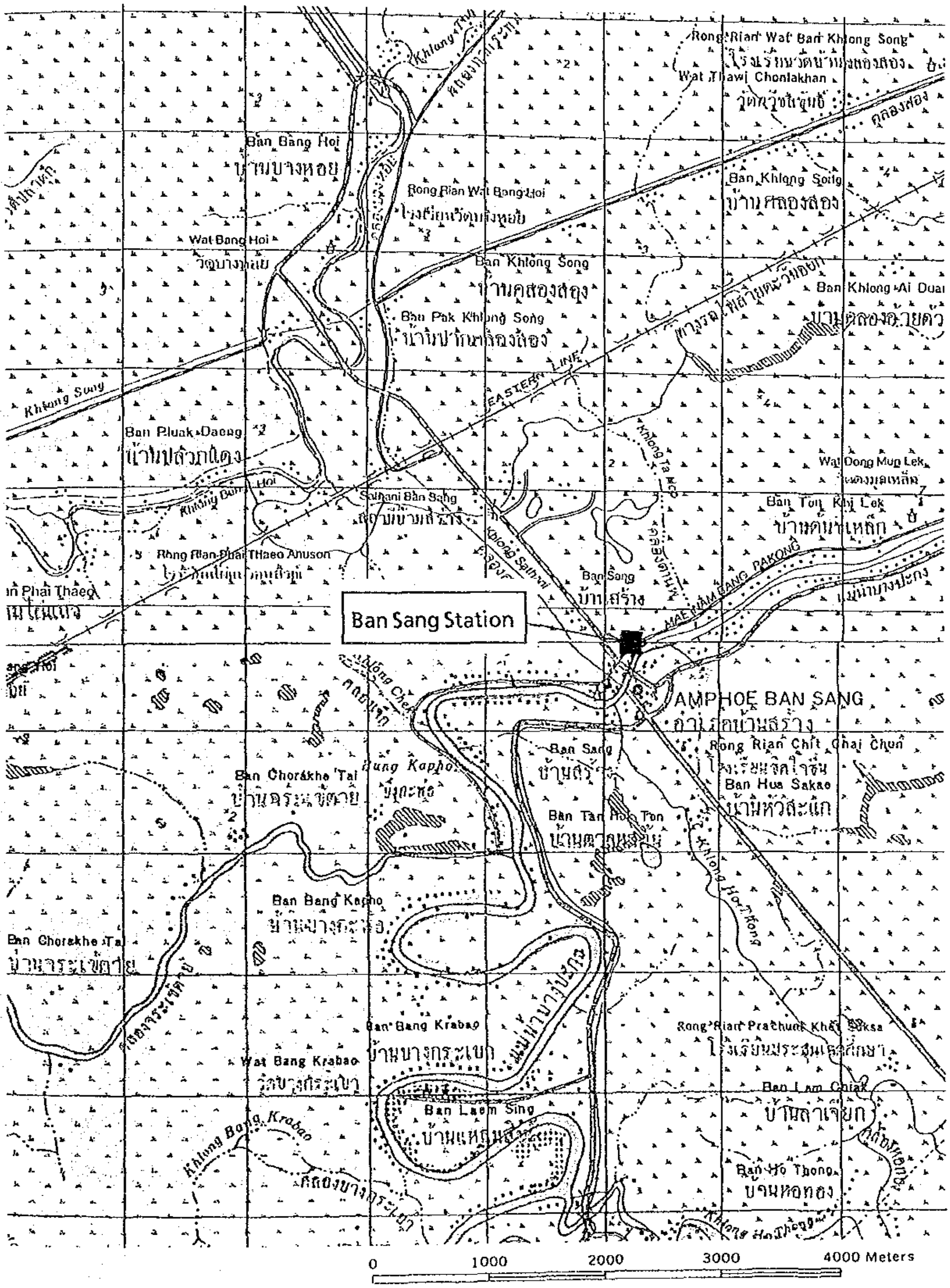
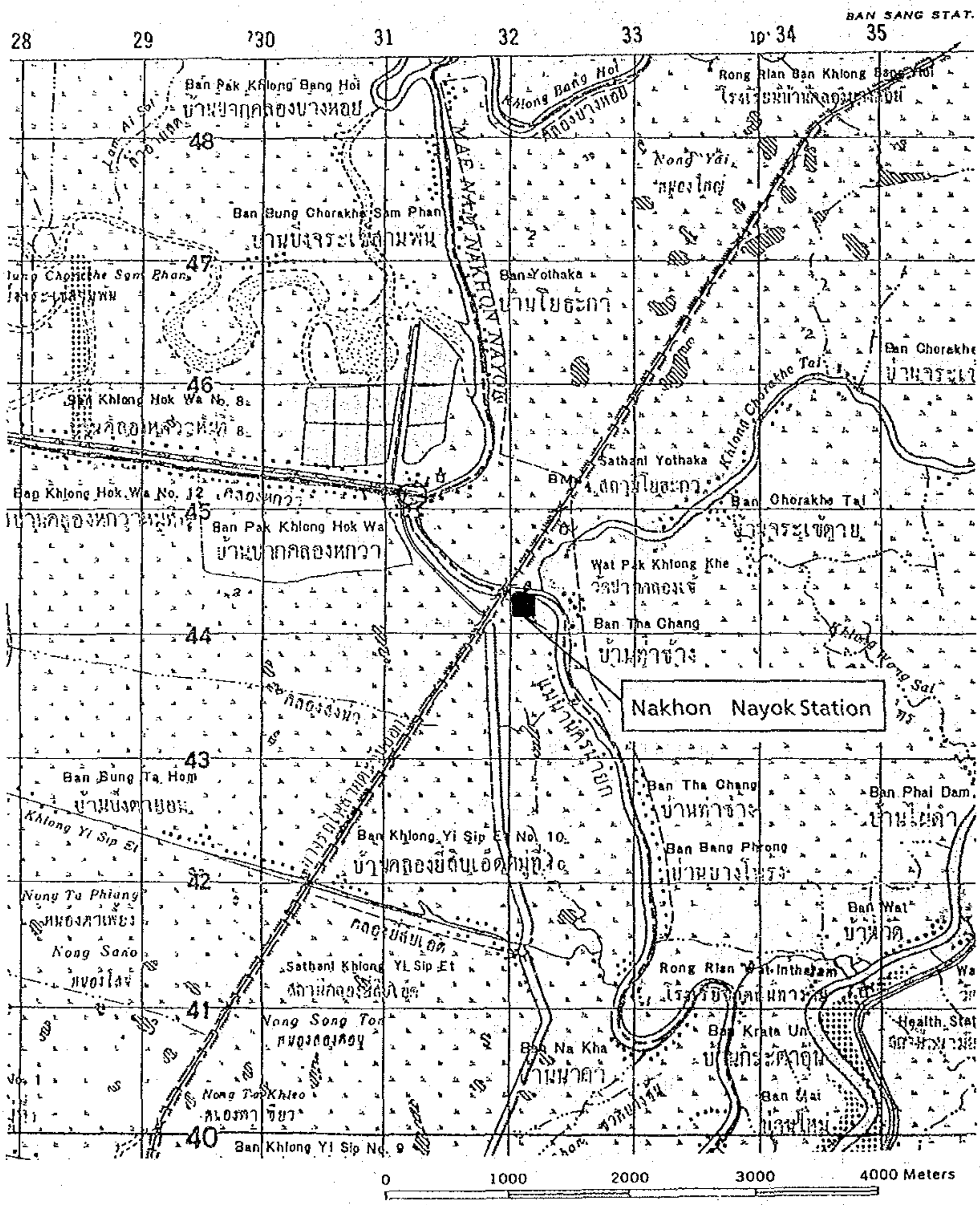


FIGURE 9-1-2 LOCATION OF NAKHON NAYOK STATION (NAKHON NAYOK RIV.)



9.2 Gate Opening Depth Instrument

The gate opening depth instrument proposed for the project facilities will be singled out form the following three (3) type instruments available in the market.

(1) Lifting Device Rotational Value Detection Method

The gate opening depth is detected by measuring the rotational value of lifting drum and converted its value into the electric signal.

(2) Gate Leaf Movement Value (Messenger Wire) Method

The gate opening depth is presented by measuring the moved value of messenger wire furnished to the gate leaf and indicated by converting its value into the electric signal.

(3) Read Switch Dection Method

The method is to indicate the position of gate leaf by detecting the operations (ON - OFF) of read switches installed on the gate side guide.

As presetned in the following comparision table, the moved value of gate leaf (messenger wire) and read switch detections method are introduced for the appropriate instrument in terms of a high accuracy.

Item	Gate Leaf Movement Value Method (Messenger Wire Method)	Read Switch Method
Accuracy	± 2 cm in extent for 0 - 10 m ± 10 cm in extent for 0 - 50 m	± 2 cm in extent
Merit	· Comparatively simple mechnism with a high accuracy in detection · Fasile combination with various transmitters (Signal detections system)	· A high accuracy becuase gate leaf motions is directly detected · An accuracy unchanged in joining. several measuring logs together.

Item	Gate Leaf Movement Value Method (Messenger Wire Method)	Read Switch Method
Others		· Taking case to install the measuring log involved read switch in a vertical positions in relations to magnet, a distance between two installation lines must be within ± 5 mm for the entire length (lifting height) · To prevent a foreign substance from entering in between the measuring log and magnet. · The gate and side guide are complicated in the installtion works and hard in the inspections and maintenance.

Arranaging the these matters mentioned above, the gate leaf movement value (Messenger wire) method is proposed, as shwon in the following table.

Item	Lifting Devices Rotational Value Detection Method	Gate Leaf Movement Value (Messenger wire) Method	Read Switch Detections Method
Measuring Range	○	○	△
Composit Accurancy	△	○	◎
Structure/Installations	○	○	△
Inspections/Maintenance	○	○	△
Comprehensive Conclusion	△	◎	○

Note : ◎ :Excellent, ○ : Better, △ :Good

9.3 Location of ITV Camera

As for location to install ITV camera, the following cases for each upstream and downstream side were taken up for the examinations. The locations was adopted to the case 3 in the upstream and case 3 in the downstream taking into account dead angle and risk of observation.

① Angle looked at the camera location to the dam axis (°)
② A width of dam out of visual field observed by two cameras (m)
③ A width of dam out of visual field in case of camera trouble in the other river side (m)
④ Focal distance necessary to zoom up a gate width of 17 m within a display of monitor (mm)

For the upstream cameras

Case 1 : Located at about 50 m upstream from the dam axis in the left and right side major bed of the river, respectively - 2 cameras installations

Case 2 : Located at about 100 m upstream from the dam axis in the left and right side major bed of the river, respectivly - 2 cameras installation

Case 3 : Located at about 200 m upstream from the dam axis in the left and right side major bed of the river, respectively - 2 cameras installations

Cases	① (°)	② (m)	③ (m)	④ (mm)	Remarks
Case 1 (50 m × 2 sets)	60 ~ 23	10	69	22	
Case 2 (100 m × 2 sets)	72 ~ 47	0	28	44	
Case 3 (200 m × 2 sets)	70 ~ 55	0	17	80	adopted

For the downstream cameras

Case 1 : Located at about 50 m downstream from the dam axis in the left and right side major bed of the river, respectively - 2 cameras installations

Case 2 : Located at about 100 m downstream from the dam axis in the left and right side major bed of the river, respectivly - 2 cameras installation

Case 3 : Located at the pier road bridge about 200 m downstream from the dam axis - 2 cameras installations

Case 4 : Locationed at the same bridge pier - 1 camera installation

Cases	① (°)	② (m)	③ (m)	④ (mm)	Remarks
Case 1 (50 m × 2 sets)	59 ~ 20	68	110	22	
Case 2 (100 m × 2 sets)	73 ~ 48	0	64	44	
Case 3 (200 m × 2 sets)	90 ~ 71	0	20	80	adopted
Case 4 (200 m × 1 sets)	90 ~ 66	11	150	80	

9. 4 ITV Camera and Lens

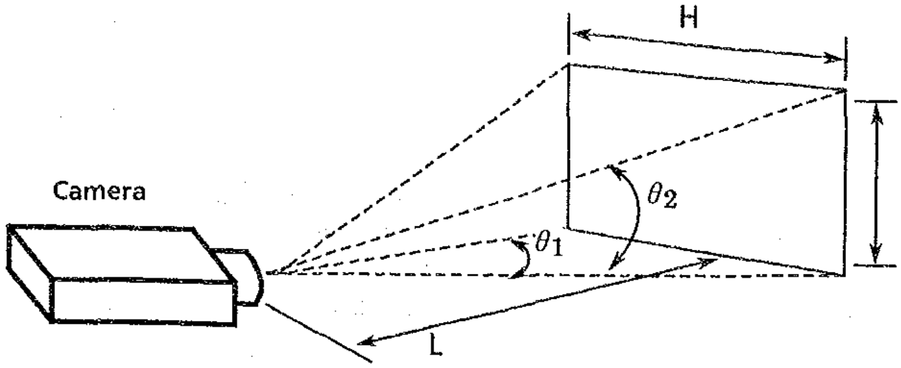
9. 4. 1 Combination of Camera and Lens

Typical Zooming Lens (Conformity Device ; 1/2 inch)

Focal Length	Picture Angle	
	Case of Wide	Case of Telephoto
	θ_1 θ_2	θ_1 θ_2
6 times type 8 ~ 48 mm	43.6° × 33.4°	7.7° × 5.7°
10 times type 8 ~ 80 mm	43.6° × 33.4°	4.6° × 3.4°
14 times type 10 ~ 140 mm	35° × 27°	2.5° × 2.0°

Notes, θ_1 ; Horizontal Picture Angle (degree)
θ_2 ; Vertical Picture Angle (degree)

9.4.2 Lens Selection



The relation of f, L, H, V is as follows.

$$H = \frac{6.5 \times L}{f}$$

$$V = \frac{4.8 \times L}{f}$$

f ; Focal Length (mm)
L ; Object Distance (m)
H ; Horizontal Field of View (m)
V ; Vertical Field of View (m)

In this project, lens will be proposed as follows.

L = 200 m
H = 17 m (= 34 m/2 ※)

$$\therefore f = \frac{6.5 \times L}{H}$$

$$= \frac{6.5 \times 200}{17}$$

= 76.5
= 8 ~ 80 mm (10 times type)

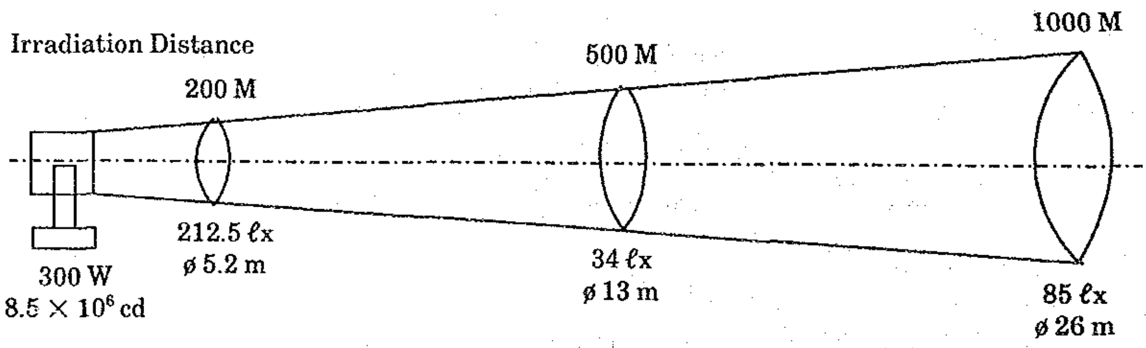
※ 1 span of each gate is 34 m

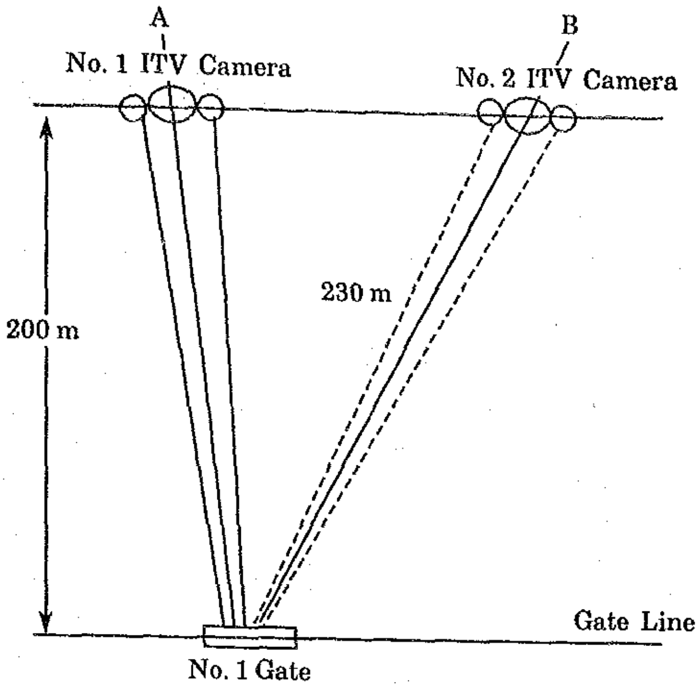
9. 5 Lighting for Gates

9. 5. 1 Illumination of Xenon-Serach Light for Gates

Xenon-Search Light's Substantial Data

Class of Protection	Light Body IP 45
For Lamp	H - 3 (300 WATT)
Front Glass	232 mmø 5mm^t
Reflector Rotation	200 mmø
Elevation Angle	UP 30 deg, DOWN 30 deg.
Azimuth Angle	360 deg.
Max. Luminous Intensity of Beam (cd)	300 W 8.5×10^6 cd at 2°
Appament Power	300 W (700 VA) [1ø 200 V 3.2 A]
Body Material	STAINLESS STEEL & MILD STEEL





A. No. 1 ITV Camera

1. Case by 1 Search-light

500 m, 34 Lx, DIA 13 mø

$$34\,\mathrm{Lx} \times \left(\frac{500}{200}\right)^2 = 212.5\,\mathrm{Lx} \doteqdot 210\,\mathrm{Lx}$$

$$13\,\mathrm{m}\phi \times \left(\frac{200}{500}\right) = 5.2\,\mathrm{m}\phi \doteqdot 5\,\mathrm{m}\phi$$

2. Case by 2 Serach-Lights

210 Lx × 2 = 420 Lx

B. No. 2 ITV Camera to use in Combination

1. Case by 1 Search-light

Distance : 230 m

$$34\,\mathrm{Lx} \times \left(\frac{500}{230}\right)^2 = 160\,\mathrm{Lx}$$

$$13\,\mathrm{m}\phi \times \left(\frac{230}{500}\right) = 6\,\mathrm{m}\phi$$

2. Case by 2 Search-Lights

160 Lx × 2 = 320 Lx

9. 5. 2 Gate Lighting

Illumination calculater of gate lighting by flux method. (out door projecter)

$$E = \frac{F \cdot U \cdot M}{N \cdot A \cdot D}$$

E	:	Average Lux
F	:	Beam Flux = 23,000 ℓm
U	:	Beam Utilize Factor = 0.68
M	:	Compensation Factor (Sea or River) = 0.55
N	:	Nos. of Light
A	:	Area 30 m × 12 m = 360 m^2
D	:	Depreciation Factor = 1.5

$$E = \frac{23{,}000 \times 0.68 \times 0.55}{360 \times 1.5} \doteqdot 16 \text{ Lx}$$

$$16 \times 4 = 64 \text{ Lx}$$

9. 6 Data Transmision System (Bang Pakong to Bangkok)

9. 6. 1 Instruments (Bang Pakong to Bangkok for Future Extension)

1) Control House

Item	Description	Q'ty
1. 1	Modification for DPE (Data Processing Equipment)	1 L.S
	1. Data Output for DSE (Data Sending Equipment)	1
1.2	DSE (Data Sending Equipment)	1 L.S
	1. Processing Unit	1
	2. Communication Panel	1
	3. Cabinet	1
	4. Power Supply Unit	1
	5. Transformer	1
1. 3	CPE (Cable Protection Equipment)	1 L.S

2) Bangkok IEC Center

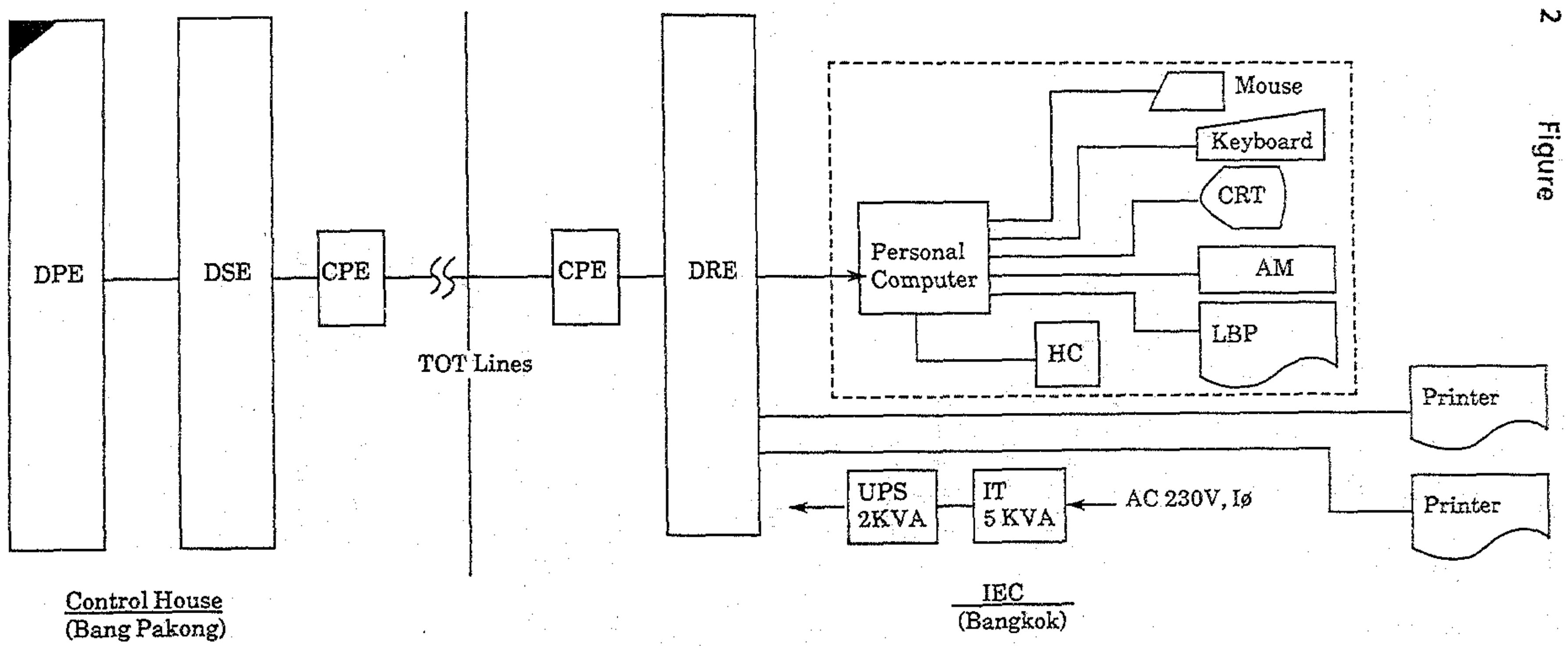
Item	Description	Q'ty
2. 1	DRE	1 L.S
	1. Processing Unit	1
	2. Printing Processing Unit	1
	3. Output I/O Unit	1
	4. Transformer	1
	5. Communication Panel	1
	6. Cabinet	1
	7. Power Supply Unit	1
2. 2	Personal Computer	1 L.S
	1. Hard Ware	1
	2. Soft Ware	1
2. 3	Printer	2 L.S
2. 4	Isolation Transformer, 5 KVA	1 L.S
2. 5	UPS, 2 KVA	1 L.S
2. 6	Spare PCB	1 L.S
2. 7	Spare Power Supply	1 L.S
2. 8	Spare nad Accessories	1 L.S

3) Test Tools

Item	Description	Q'ty
3. 1	Digitial Synchroscope	1 L.S
3. 2	Data Communication Analyzer	1 L.S
3. 3	Level Meter	1 L.S
3. 4	Digital Mulitmeter	1 L.S
3. 5	Oscilloscopc (Portabel type)	1 L.S
3. 6	Tool	1 L.S

9.6.2 Figure

FIGURE 9.6.1 FROM BANG PAKONG TO BANGKOK DATA TRANSMISSION SYSTEM



DSE : Data Sending Equipment
DRE : Data Receiving Equipment
CPE : Cable Protection Equipment
□ : New
◩ : Reconstruct

APPENDIX - 10 DESIGN OF ELECTRICAL FACILITIES

APPENDIX - 10. DESIGN OF ELECTRICAL FACILITIES

LIST OF CONTENTS

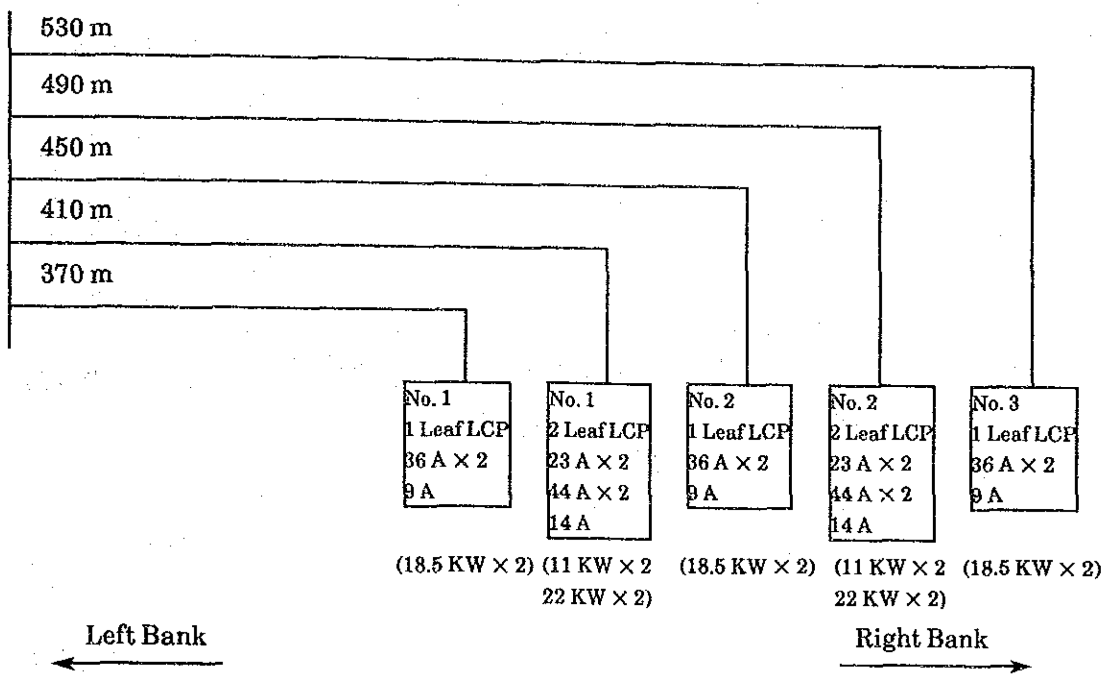
10.1 No. 1 Substation (At the Diversion Dam Area)

1) Gate Motor

Regulating Gate	:	380 V 28.9 KVA × 2, 15.1 KVA × 2
Flood Gate	:	380 V 23.6 KVA × 2

$$23.6\,\text{KVA} \times 2 + 15.1\,\text{KVA} \times 2 + 28.9\,\text{KVA} \times 5.2 \times 2 + 12\,\text{KVA} = 390\,\text{KVA}$$

2) Voltage Drop & Cable Size for Local Control Panel



No. 1 1 Leaf LCP

$$\left.\begin{aligned} E_{R1} &= \frac{30.8 \times 370\,(36 \times 2 + 9)}{1000 \times 100 \times 380} = 0.024 \\ E_{S1} &= \frac{30.8 \times 370\,(180 \times 2 + 9)}{1000 \times 100 \times 380} = 0.11 \end{aligned}\right\} \text{CV } 100^{\square} \times 3 + \text{CV } 60^{\square} \times 1$$

No. 1 2 Leaf LCP

$$\left.\begin{aligned} E_{R2} &= \frac{30.8\times410\times(23\times2+44\times2+14)}{1000\times150\times380} = 0.033 \\ E_{S2} &= \frac{30.8\times410\times(23\times2+227\times2+14)}{1000\times150\times380} = 0.114 \end{aligned}\right\} \text{CV } 150^{\square}\times3+\text{CV } 100^{\square}\times1$$

No. 2 1 Leaf LCP

$$\left.\begin{aligned} E_{R3} &= \frac{30.8\times450\times(36\times2+9)}{1000\times150\times380} = 0.020 \\ E_{S3} &= \frac{30.8\times450\times(180\times2+9)}{1000\times150\times380} = 0.090 \end{aligned}\right\} \text{CV } 150^{\square}\times3+\text{CV } 100^{\square}\times1$$

No. 2 2 Leaf LCP

$$\left.\begin{aligned} E_{R4} &= \frac{30.8\times490\times(23\times2+44\times2+14)}{1000\times200\times380} = 0.030 \\ E_{S4} &= \frac{30.8\times490\times(23\times2+227\times2+14)}{1000\times200\times380} = 0.102 \end{aligned}\right\} \text{CV } 200^{\square}\times3+\text{CV } 100^{\square}\times1$$

No. 3 1 Leaf LCP

$$\left.\begin{aligned} E_{R5} &= \frac{30.8\times530\times(36\times2+9)}{1000\times150\times380} = 0.023 \\ E_{S5} &= \frac{30.8\times530\times(180\times2+9)}{1000\times150\times380} = 0.120 \end{aligned}\right\} \text{CV } 150^{\square}\times3+\text{CV } 100^{\square}\times1$$

3) Control House & Substation

Lighting & Cooler 50 KW
Isolation Trasformer 20 KW

$$\frac{50+20}{0.85} = 83 \text{ KVA}$$

4) Control House & Training Center

Control House	284 KW
Training Center	280 KW
Dormitory	94 KW

$$\frac{284 + 280 + 94}{0.85} \times 0.7 = 542 \text{ KVA}$$

5) ITV & Observation House

ITV Moving Device	500 W/0.85	= 588 VA
Search Light	700 VA × 2	= 1400 VA

$$\left(\frac{588 + 1400}{1000}\right) \times 4 \text{ sets} = 8 \text{ KVA}$$

6) Gate Lighting & Hoist House

Gate Light : 380 V 400 W × 40 sets

Ditto Ballaster Starting Current: 1.7 A × 1.2 = 2.04 A

$$\frac{2.04 \text{ A} \times 40 \times 380 \text{ V}}{1000} \doteqdot 31 \text{ KVA}$$

7) Transformer Capacity

$$\text{Tr KVA} = \Sigma \text{KVA} \times 1.2 = (390+83+542+8+31)\times 1.2 = 1{,}054 \times 1.2$$
$$= 1{,}265 \text{ KVA}$$

Transformer Capacity 1,500 KVA

8) Condenser for Improvement of Power Factor

$$\text{Q KVA} = 1500 \text{ KVA} \times 0.7 (\sqrt{1/(0.85^2) - 1} - \sqrt{1/0.95 - 1})$$
$$= 1050 \text{ KVA} \times (\sqrt{0.38} - \sqrt{0.108})$$
$$\doteqdot 300 \text{ KVA}$$

Core Loss : 25 KVA 300 + 25 = 325 KVA

Condenser Capacity : 360 KVA

9) Emergency Generator for Diversion Dam

Calculation Basis

Motor Load : 3ø, 50 HZ, 380 V, 22 KW sets operation at one time

Motor Capcity : $28.9 \times 2 = 57.8$ KVA

IT, Lighting & Air Condtioner 40 KVA

Generator Capacity = $(40+57.8\times5.2)\times0.7 = 238$ KVA → 270 KVA

10. 2 No. 2 Substation (At the Pumping Station Area)

1) Main Motor : Cage Rotor with Reactor Type Starter

350 KW $\times 3 = 1{,}050$ KW

Operation Rule : 350 KW $\times 2 + 350$ KW $\times 3.9$

$$\frac{(350 \times 2) + (350 \times 3.9)}{0.9 \times 0.9} = 2{,}550 \text{ KVA}$$

2) Auxiliary Equipment, Lighting and Others

100 KVA

3) Transformer Capacity KVA

Tr KVA = $(2{,}550 + 100) \times 1.2 = 3{,}180$ KVA For Starting

Transformer Capacity = 3,000 KVA

4) Condenser for Improvement of Power Factor

$$Q\,\text{KVA} = \frac{350\,\text{KW}}{0.93} \times \left(\sqrt{1/(0.85^2)-1} - \sqrt{1/0.95-1}\right)$$

$$\doteqdot 109\ \text{KVA}$$

Core Loss 50 KVA

$109 \times 3 + 50 \doteqdot 377$ KVA

Condenser Capacity = 400 KVA

5) Emergency Generator for Pumping Station

Calcuation Basis

Discharge valve	3.7 KW	→	4.6 KVA
Priming pump for gear	0.4 KW	→	0.5 KVA
Priming pump for engine	0.4 KW	→	0.5 KVA
Cooling water pump	0.75 KW	→	0.9 KVA
Compressor	3.7 KW	→	4.6 KVA
Fuel pump	0.4 KW	→	0.5 KVA
Drainage pump	0.75 KW	→	0.9 KVA
Others			17.0 KVA
	Total		29.5 KVA

$$Q\,KVA = \frac{(29.5 - 4.6 \times 2) + 4.6 \times 2 \times 6}{1.3}$$

$$= \frac{20.3 + 55.2}{1.3}$$

$$= 58.1\,KVA \rightarrow 60\,KVA$$

10. 3 Residential Area (RID Plan For Future at the Diversion Dam Area)

1) Nos. of Families

High Class	17	Families
Middle Class	48	〃
Low Class	396	〃
Total	461	Families

2) Application

High Class	$17 \times 20\,A \times 220\,V \times 1/0.8$	= 94 KVA
Middle Class	$48 \times 10\,A \times 220\,V \times 1/0.8$	= 132 KVA
Low Class	$396 \times 5\,A \times 220\,V \times 1/0.8$	= 545 KVA
	Total	771 KVA

(3) Transformer Capacity

Demand Factor 60%

$$Tr\ KVA = 771 \times 0.6 \times 1.2 = 463 \times 1.2 = 556\ KVA \doteqdot 600\ KVA$$

Transformer Capacity 300 KVA × 2 sets

APPENDIX - 11 : CONSTRUCTION PLAN

APPENDIX - 11. CONSTRUCTION PLAN

LIST OF CONTENTS

11.1 Plan for Use of High Water Content Excavated Soil in the O/M Building Area

The O/M building will be constructed in the area with approximately 550,000 m^2 and in the height 1.0 ~ 1.2 m. Excavated soft soil with a volume of 750,000 m^2 from diversion dam shall be used for embankment material.

The production volume per day is

$$750{,}000\ m^3 \div (11M \times 24D) = 2{,}840\ m3/day$$

The soil will be diffused with approx. 20 cm in thickness, hence the required area will be 2,840 $m^3 \div 0.2\ m = 14{,}200\ m^2$

Out of 550,000 m^2, 400,000 m^2 area will be used for its operation in about 1 month's period (400,000 $m^2 \div$ 14,200 m^2/day = 28 days or 1 month).

This means that one cycle of one layer will take about one month, which period is sufficient to dry up the soil. This area is only intend for the building and therefore high compaction is not necessary.

The number of the equipment that will be required for the following works are as follows:

Spreading 21 ton bulldozer : 2 units

$$2{,}840\ m^3 \div (150\ m^3/hr \times 10\ hours) = 2$$

Compaction 15 ton Swamp bulldozer : 3 units

This bulldozer will be prepared for scarifying and compaction purpose.

$$4{,}000\ m^2/hr \times 10\ hours \times 0.7 \div 14{,}200\ m^2 = 2\ \text{times pass per day.}$$

By assuming 6 times pass in drying up the soil, then 3 sets swamp bulldozer shall be prepared.

For the finishing work, 2 units of grader and 2 units of tire roller will be necessary.

In conclusion, the required number of units of the following equipment will be as follows;

Bulldozer (21 ton)	:	2 units
Swampdozer (15 ton)	:	3 units
Grader (12 ft)	:	2 units
Tire roller (15 ton)	:	2 units
Total		7 units

11. 2 Study for Heavy Equipment Trafficability

If back-hoe type shovel is used at around EL. (-)9.0 m in diversion dam foundation area, a bulldozer may be required for assistance. Generally, the following Cone-Indexes of each equipment are to be applied to determine the trafficability.

Kind of Equipment	Cone-Indix qc (kg/cm^2)
Supper Swampdozer	$2 \leqq$
Swampdozer	$3 \leqq$
Middle Class Bulldozer	$5 \leqq$

The equation applied between qc and compressive strength qu as above stated are as follows:

$$qc = 5\, qu$$

Average qu at Bang Pakong Diversion Damsite soil layer EL. (-)1.0 ~ EL. (-)7.0 m is qu = 0.4 (kg/m^2)

$$\therefore\; qc = 5 \times 0.4 = 2.0\ (kg/m^2)$$

This means that the soil condition has lower limit of supper swampdozer's trafficability.

Generally we recommend
$qc \geqq 3 \sim 5\ kg/m^2$

The soil tests show a very high Liquidity-Index (3.0 ~ 3.5), or high Sensitivity Ratio soil. In view of this any bulldozer can not be utilized on this very soft soil, and can only assist the backhoe's operation on the EL. (-)9.0 m ground for material gathering purposes.

11.3 Production Volume by Construction Equipment

Production volume by construction equipment was estimated by mean's of empirical equations.

The formulae applied and the production volume of each equipment are shown in the following pages.

a) Dragline 2 m^3

$$Q = \frac{3{,}600 \times q \times f \times E}{Cm}$$

Where,
- Q ; Production volume per hour (m^3/hr)
- q; One cycle performance volume (m^3), $q = q_o \times 0.9$
- f; Swell factor q_o; nominal bucket capacity
- E; Coefficient of work
- Cm; One cycle time (sec.)

$$\begin{aligned} q &= 2\,m^3 \times 0.9 = 1.8\,m^3 \\ f &= 0.85 \\ E &= 0.6 \\ Cm &= 40\ sec \end{aligned}$$

$$Q = \frac{3{,}600 \times 1.8 \times 0.85 \times 0.5}{40} = 80\ m^3/Hr$$

b) Backhoe Shovel 1 m^3

$$Q = \frac{3{,}600 \times q \times f \times E}{Cm}$$

Where,
- Q ; Production volume per hour (m^3/hr)
- q; One cycle performance volume (m^3), $q = q_o \times 0.9$

f; Swell factor q_o; nominal bucket capacity

E; Coefficient of work

Cm; One cycle time (sec.)

$$q = 1\,m^3 \times 0.9 = 0.9\,m^3$$
$$f = 0.85$$
$$E = 0.55$$
$$Cm = 30\,sec$$

$$Q = \frac{3{,}600 \times 0.9 \times 0.85 \times 0.5}{30} = 50\,m^3/Hr$$

c) Dump Truck 11t

$$Q = \frac{3{,}600 \times q \times f \times E}{Cm}$$

Where, Q ; Production volume per hour (m^3/hr)

q; One cycle performance volume (m^3)

f; Swell factor

E; Coefficient of work

Cm; One cycle time (min.)

$q =$ T/W (m^3)

T; Allowable loading volume of dump truck, (t)

W; Unit weight, (t/m^3)

Cm = 0.005L + 10.5 (min)

L; Distance, (m)

T= 11 t, W = 1.4 t/m^3 $\therefore$ q = 11/1.4 = 7.85 m^3

Case-1 L = 1 km Cm = 0.005L + 10.5 = 15.5 min

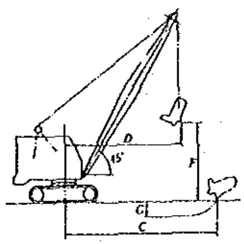
$$Q = \frac{60 \times 7.85 \times 0.85 \times 0.7}{15.5} = \; = 18m^3/Hr$$

Case-2 L = 5 km Cm = 0.005L + 10.5 = 35.5 min

$$Q = \frac{60 \times 7.85 \times 0.85 \times 0.7}{35.5} = \; = 8m^3/Hr$$

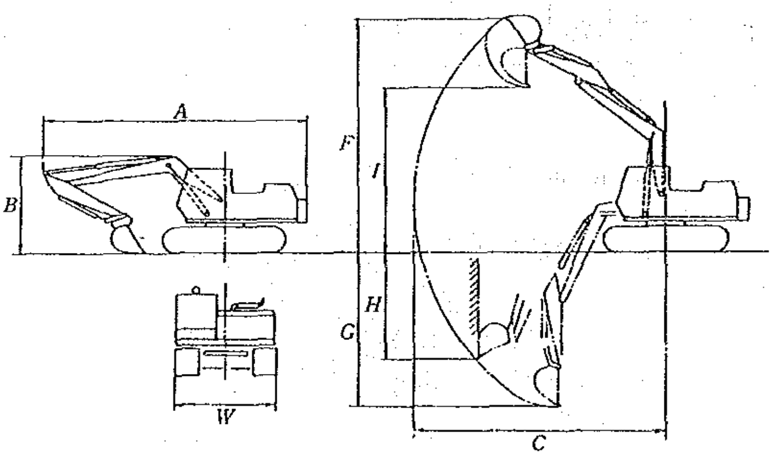
d) Dragline Specifications

Make	Model	Bucket Capacity Struck (m³)	Operating Weight (t)	Ground Pressure (kgf/cm²)	(A) Boom Length Standard (m)	(A) Boom Length Max. (m)	Hoist Rope Speed (m/min)	Drag Rope Speed (m/min)	Working Range (at 45 Deg. of Boom Angle) (C) Max. Cutting Radius (m)	(G) Max. Digging Depth (m)	(F) Max. Dumping Height (m)	(D) Radius at Max. Dumping Height (m)	Primary Resolved Weight (t)	Remark
Ishikawajima Construction Machinery	CCH 250	0.8	28.3	0.61	13.0	16.0	80/40	80/40	13.9	8.0	6.7	10.5	25.8	
	CCH 300	0.8	26.9	0.55	13.0	16.0	80/40	80/40	13.9	8.0	6.7	10.5	24.1	
	CCH 350	0.8	31.4	0.46	13.0	16.0	80/40	80/40	13.9	8.0	6.7	10.5	21.7	
	CCH 400	0.8	33.8	0.47	13.0	16.0	80/40	80/40	13.9	8.0	6.7	10.5	23.5	
	DCH 6020	2.0	73.6	0.85	21.34	21.34	70/35	70/35	22.5	8.7	11.9	17.6	(27.2)	
KOBE STEEL	7035	1.0	34.4	0.49	9.14	18.29	70.0	70.0	16.5	3.8	7.5	12.9	25.5	
	7045	1.0	41.4	0.56	9.14	18.29	70.0	70.0	21.3	5.5	10.9	15.2	28.0	
	7055	2.0	44.2	0.57	12.19	18.29	60.0	60.0	21.3	8.5	10.8	15.4	20.0	
	7065	2.0	49.0	0.58	12.19	18.29	60.0	60.0	21.3	8.5	10.8	15.5	24.0	
SUMITOMO (S.H.I) CONSTRUCTION MACHINERY	LS-78 RM	0.8	32.1	0.57	9.50	18.5	60	2.1	17.67	9.97	9.41	14.97	25.0	
	LS-118 RM	1.0	39.7	0.52	12.20	18.3	50	50	16.79	9.83	9.25	14.89	30.1	
	LS-78 RHD-5	0.8	42.6	0.63	10.0	19.0	80	80	21.3	13.2	10.8	15.3	27.5	
	LS-108 RGD-5	1.0	43.1	0.60	10.0	19.0	80	80	21.4	13.2	10.0	15.4	28.3	
	LS-118 RHD-5	1.0	46.3	0.59	9.30	18.45	80	80	21.0	12.5	9.6	15.3	28.3	
	LS-120 RHD-5	1.9	65.0	0.79	12.20	24.40	60	60	27.0	12.2	12.6	19.2	33	
	LS-458 HD	3.6	83.1	0.90	12.20	30.5	70	70	31.2	11.7	13.2	24.5	(29.5)	
	LS-468 HD	4.8	102.9	0.76	18.30	36.6	70	70	37.0	13.9	17.5	28.8	(31.9)	
	184 M	6.9	372	1.3	42.7	45.7	97.5	55.5	36.6	27.4	20.7	36.6	–	
	195 M	10.7	603	1.7	51.8	–	124.9	99.7	46.1	30.8	24.9	46.1	–	
NIPPON SHARYO	DH 300 III	0.8	33.5	0.56	10	16	65/32.5	65/32.5	17.0	10.0	10.0	12.6	23.3	
	DH 350 III	0.8	39.9	0.58	10	16	65/32.5	65/32.5	17.0	10.0	10.0	12.6	26.1	
	DH 400 III	0.8	42.2	0.58	10	16	65/32.5	65/32.5	17.0	10.0	10.0	12.6	27.3	
	DH 500 III	0.8	48.9	0.65	13	16	65/32.5	65/32.5	17.0	10.0	10.0	12.6	29.7 (17.8)	



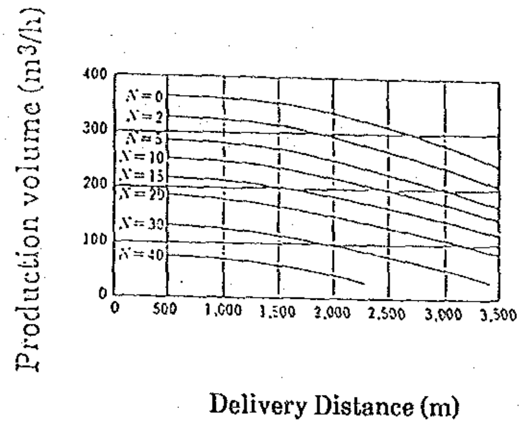
e) Backhoe Specifications

Make	Model	Backet Capacity		Opera-ting Weight	Ground Pressure	Working Range						Dimensions at Transport			Primary Resolved Weight	Remark
		Heaped	Struck			(C) Max. Cutting Radius	(G) Max. Digging Depth	(H) Depth of Vertically Digging	(F) Max. Cutting Height	(I) Max. Dumping Height	Max. Digging Force	(A) Overall Length	(W) Overall Width	(B) Overall Height		
		m³	m	t	kgf/cm²	m	m	m	m	m	tf	m	m	m	t	
KOUBE STEEL	SK 60	0.25	0.22	6.4	0 .34	6.42	4.20	3.47	7.36	5.29	4.8	6.02	2.20	2.61	—	
	SK 100	0.4	0.35	10.5	0 .36	7.70	5.08	4.41	7.66	5.49	7.5	7.24	2.49	2.73	—	
	SK 120	0.45	0.38	11.5	0 .38	8.25	5.57	4.87	8.50	6.06	7.7	7.60	2.49	2.73	—	
	SK 120 LC	0.45	0.38	11.7	0 .36	8.25	5.57	4.87	8.50	6.06	7.7	7.60	2.49	2.73	—	
	SK 200	0.7	0.59	18.7	0 .44	9.85	6.67	6.00	9.59	6.76	11.4	9.32	2.80	2.91	—	
	SK 200 LC	0.7	0.59	19.5	0 .41	9.85	6.67	6.00	9.59	6.76	11.4	9.32	2.80	2.91	—	
	SK 220	0.9	0.76	22.9	0 .51	10.31	7.00	6.04	9.72	6.83	13.8	9.98	2.99	3.07	—	
	SK 220 LC	0.9	0.76	23.5	0 .48	10.31	7.00	6.04	9.72	6.83	13.8	9.98	3.19	3.07	—	
	SK 300-2	1.2	1.0	29.5	0 .61	11.13	7.38	6.19	10.35	7.15	18.0	10.85	3.20	3.22	—	
	SK 300 LC-2	1.2	1.0	30.5	0 .59	11.13	7.38	6.19	10.35	7.15	18.0	10.85	3.20	3.22	—	
	SK 300 HD	1.2	1.0	30.5	0 .63	11.13	7.38	6.19	10.35	7.15	18.0	10.85	3.20	3.22	—	
	SK 400-2	1.6	1.4	41.5	0 .78	12.03	7.80	6.94	7.80	7.58	22.5	11.67	3.35	3.45	—	
	SK 400 LC-2	1.6	1.4	42.4	0 .75	12.03	7.80	6.94	7.80	7.58	22.5	11.67	3.35	3.45	—	
	SK 400 HD	1.6	1.4	43.2	0 .82	12.03	7.80	6.94	7.80	7.58	22.5	11.67	3.35	3.45	—	
	SK 600	2.0	1.8	58.8	1 .10	13.50	8.50	6.89	11.50	7.60	23.6	13.41	3.80/ 3.93	4.75	(19)	
	SK 1350	6.4	5.6	130.0	1 .52	14.60	8.40	6.44	12.73	8.12	50.6	15.33	5.49	5.94	28	
	SK 100 W	0.4	0.35	11.2	—	7.32	4.26	3.51	8.79	6.37	7.3	6.84	2.49	3.59	—	



f) Pump Dredger

Production Curve for 1,000 PS Pump Dredger by Soil Conditions (clay or silty clay).



11. 4 Cost Comparison by Excavation Methods for Diversion Canal

1) Case-1 Pump Dredging

Stripping (include top layer soil)	400,000 m^3 × 70B/m^3 =	28,000,000B
Dredging	2,000,000 m^3 × 60B/m^3 =	120,000,000B
Dumped rock on the riverbed	10,000 m^3 × 500B/m^3 =	5,000,000B
Temporary access road	L.S.	7,000,000B
Total		160,000,000B

2) Case-2 Open-Cut Excavation

a) Excavation and Dumping

Cost:	Dragline	2 m^3	4 unit	68 unit-month × 360,000 B/M =	24,480,000B
	Backhoe	1 m^3	4 unit	67 unit-month × 156,000 B/M =	10,452,000B
	Dump Truck	11 t	40 unit	1,094 unit-month × 72,000 B/M =	78,768,000B
	Bulldozer	21 t	2 unit	58 unit-month × 216,000 B/M =	12,528,000B
	Swampdozer	15 t	5 unit	145 unit-month × 168,000 B/M =	24,360,000B
	Foreman "A"	2×29M		58 unit-month × 12,000 B/M =	696,000B
	Foreman "B"	4×29M		116 unit-month × 8,400 B/M =	974,000B
	Labour	20×29M		580 unit-month × 4,800 B/M =	2,784,000
	Misc.			LS =	2,000,000B
	Total				157,042,000B

b) Dewatering

Sets up of cut-off wall by 12 m long light weight steel sheet pile, perimeter of cut-off wall are computed and shown in Fig. 11-2'.

$$720\,m + 420 + 900 + 360 + 420 + 180 = 3{,}000\,m$$

The width of sheet pile is 50 cm, which the required number of sheet pile is 6,000 pcs.

Tonnage of pile: $33.6\,kg/m \times 6{,}000\,pcs \times 12\,m = 2{,}420\,t$

Cost:	Material price (Buy-Buck)	10,000 B/t	
	Driving & extracting cost	2,500 B/t	
	Sub-total	12,500 B/t	
	∴ 12,500 × 2,420 =	30,250,000①	
	Dewatering by pumps		
	Submersible pump	ø150m/m 70,000 B/pc × 10 pcs =	700,000
	Operating cost	6,000 B/M × 10 pcs × 25 M =	1,500,000
	Misc. cost		300,000
	Sub-total		2,500,000②
	Total ① + ②		32,750,000B

c) Temporary Road

i) Surface ground

(800 m + 300 m) × 8 m	6 NOS	52,800 m^2
400 m × 8 m	3 NOS	9,600 m^2

ii) EL. (-)9.0 m Ground

(800 m + 300 m) × 8 m	3 NOS	26,400 m^2
100 m × 8 m	3 NOS	2,400 m^2
Total road area		91,200 m^2

Used crusher run and laterite mix for base course material is 30 cm in thickness.

Cost:	Base Course: 91,200 $m^2 \times 0.3 \times 300$ B/m^3 =	8,208,000
	Misc. Cost	292,000
	Sub-Total	8,500,000 ①

iii) Access road from surface to EL. (-)9.0 m ground

3,000,000 × 2 NOS =	6,000,000 ②
Total ① + ②	14,500,000 B

d) Spoil Bank

Total excavation volume	2,400,000 m^3
Leaved bank at upper & downstream	300,000 m^3
(Dredge by pump in final stage)	
Balance	2,100,000 m^3

Among this volume topsoil (200,000 m^3), good soil at lower layer (100,000 m^3) can be used for embankment materials, and 1,800,000 m^3 soil must be disposed, the bank height is assumed at 2.5 m, required spoil bank as follows;

1,800,000 $m^3 \div 2.5$ m $\times 1.2 = 86$ Ha (540 Rai)

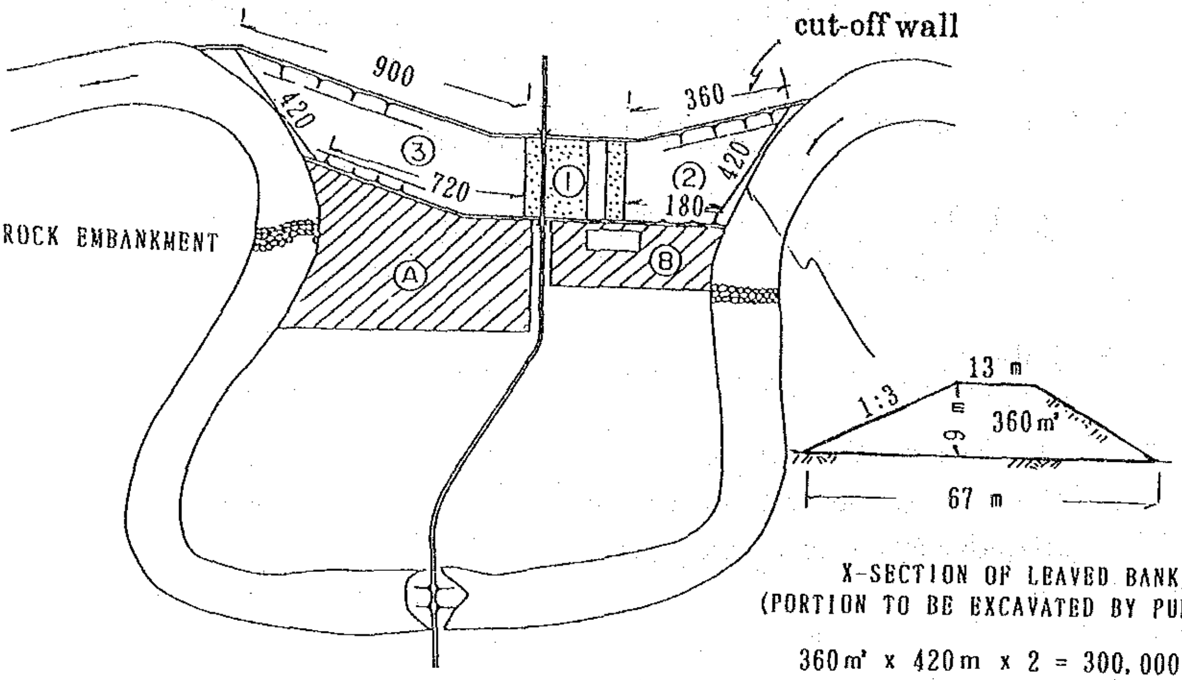
Cost Land rental

10,000B/Rai/year × 5 year × 540 Rai =	27,000,000
Road Maintenance Expense	3,000,000
Total	30,000,000 B

e) Removed Both Side Banks by Pump Dredger

Cost	300,000 $m^3 \times 60$B/m^3 =	18,000,000 B
	Grand Total a) ~ e)	252,292,000 B

FIGURE 11 - 2' EXPLANATION MAP OF EXCAVATION PLAN



X-SECTION OF LEAVED BANK
(PORTION TO BE EXCAVATED BY PUMP DREDGER)

$360\,m^2 \times 420\,m \times 2 = 300,000\,m^3$

11.5 Consideration of Spoil Flowing Down and Precipitation in Case of Discharging Excavated Materials into Old River Course by Pump Dredgers

11.5.1 Rough Estimate of Mean River Velocity

A flood analysis of Bang Pakong river was made by unsteady flow analysis method by the Study Team.

As a result, the followings were found out;

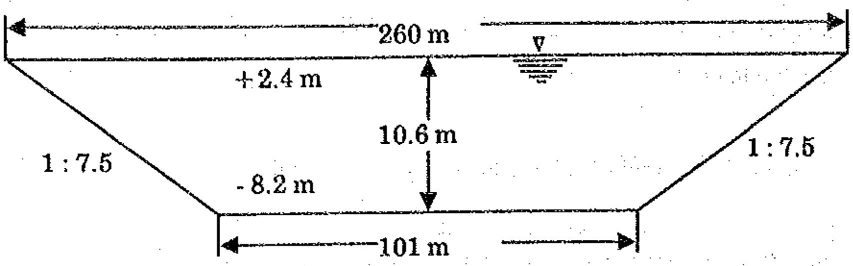
Design flood discharge	:	Q = 1,600 m^3/sec
Design flood level	:	Max. WL = 2.4 m
Design flood depth	:	H = 10.6 m
Mean river-bed slope	:	Io = 1/4,000
Flood surface slope at design flood level	:	I = 2.40/76,500 = 1/32,000

In the river cross section at the design flood level;

Water surface width	:	190 ~ 260 m
Cross sectional area	:	1,300 ~ 1,900 m^2 (1,750 m^2 on an average)

The river flow being unsteady flow, the extent and direction of river velocity are momentarily changed. However, since it takes some time to determine the range of the river velocity by using unsteady flow analysis method, it will be calculated for the present by uniform flow analysis at the site 76.5 km upstream from the estuary where the water level is assumed to be ± 0 m MSL. After the range of velocity is obtained by the unsteady flow analysis, the result will be re-examined.

1) Rough estimate of river cross sectional area in which the design flood discharge is able to flow down.



$Q = 1{,}600\ m^3/sec$

$A = 1{,}913\ m^2$

$V = Q/A = 0.836\ m/sec$

$P = 261.4\ m$

$R = A/P = 7.318\ m$

$R^{2/3} = 3.778$

$I = 1/32{,}000$

$I^{1/2} = 5.586 \times 10^{-3}$

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

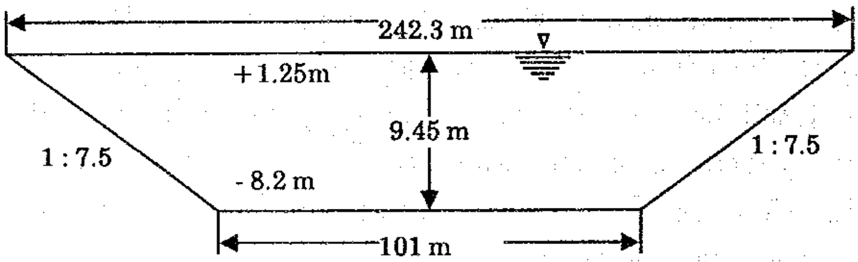
$\quad = 0.0252 \doteqdot 0.025$

2) Estimate of Mean River Velocities in Cases of Three Stages of River Discharges

a) In case of $Q = 500\ m^3/sec$

Assuming that a river water level is + 1.25 m,

$$H = 9.45\ m \quad I = 1.25/75{,}600 = 1/60{,}500$$



$P = 244.0\ m$

$A = 1{,}147.0\ m^2$

$R = 4.701\ m$

$R^{2/3} = 2.806$

$Q = 500\ m^3/sec$

$V = 0.436\ m/sec$

$I^{1/2} = nV/R^{2/3}$

$\quad = 0.025 \times 0.436/2.806$

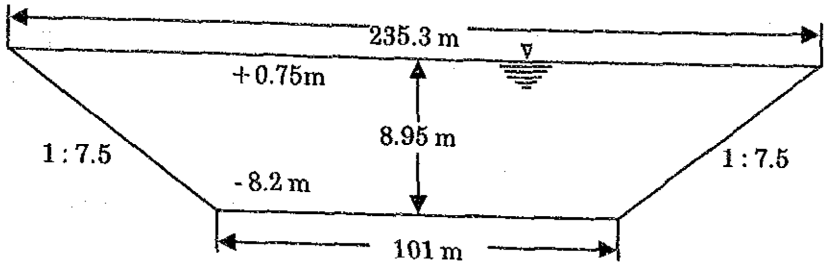
$\quad = 3.885 \times 10^{-3}$

$$\text{Calculated } Ic = 1/62{,}900 \doteqdot 1/60{,}500 = I$$

b) In case of $Q = 300\ m^3/sec$

Assuming that a river water level is + 0.75 m,

$$H = 8.95\ m \qquad I = 1.25/76{,}500 = 1/100{,}800$$



$P = 236.4\ m^3/sec$

$A = 1{,}052.7\ m^2$

$R = 4.453\ m$

$R^{2/3} = 2.708$

$Q = 300\ m^3/sec$

$V = 0.285\ m/sec$

$I^{1/2} = 0.030 \times 0.285/2.708$

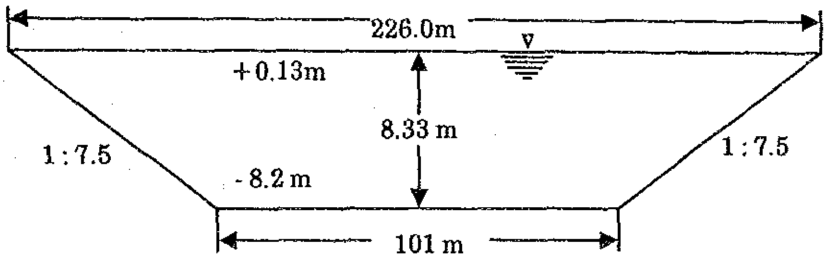
$= 3.157 \times 10^{-3}$

$$Ic = 1/100{,}300 \doteqdot 1/100{,}800 = I$$

c) In case of $Q = 100\ m^3/sec$

Assuming that a river water level is + 0.13 m,

$$H = 8.33\ m \qquad I = 0.13/76{,}500 = 1/581{,}500$$



$P = 227.0\ m$

$A = 941.1\ m^2$

$R = 4.147\ m$

$R^{2/3} = 2.581$

$Q = 100\ m^3/sec$

$V = 0.1062\ m/sec$

$I^{1/2} = 0.030 \times 0.1062/2.581$

$= 1.234 \times 10^{-3}$

$$Ic = 1/65{,}600 \doteqdot 1/581{,}500 = I$$

d) Summary of calculated mean river velocities

Q	V
1,600 m^3/sec	0.836 m/sec
500 m^3/sec	0.436 m/sec
300 m^3/sec	0.286 m/sec
100 m^3/sec	0.106 m/sec

11. 5. 2 Estimation of River Velocities 2m above Riverbed in Cases of Three Stages of River Discharges

It is premised that a pipe delivery discharging spoil into the old river course by a pump dredger is always set up 2 m above the riverbed.

(by Bazin formula) $u = V + \{8 - 24\,(y/H)^2\}\,(HI)^{1/2}$

u : River velocity 2m above riverbed. That is, H-y = 2m (m/sec)
v : Mean river velocity in the concerned river cross section (m/sec)
y : Water depth from river surface to pipe delivery (m)
H : Water depth from river surface to riverbed (m)
I : Mean river surface slope

1) In Case of $Q = 500\ m^3/sec$

$$
\begin{aligned}
u &= 0.436 + \{8 - 24\,(7.45 / 9.45)^2\}\,(9.45 / 60{,}500)^{1/2} \\
&= 0.436 - 6.918 \times 1.249 \times 10^{-2} \\
&= 0.436 - 0.086 = 0.350\ \text{m/sec}
\end{aligned}
$$

2) In Case of $Q = 300\ m^3/sec$

$$
\begin{aligned}
u &= 0.285 + \{8 - 24\,(6.95/8.95)^2\}\,(8.95 / 100{,}800)^{1/2} \\
&= 0.285 - 6.472 \times 0.941 \times 10^{-2} \\
&= 0.285 - 0.061 = 0.224\ \text{m/sec}
\end{aligned}
$$

3) In Case of $Q = 100\ m^3/sec$

$$
\begin{aligned}
u &= 0.106 + \{8 - 24\,(6.33/8.33)^2\}\,(8.33 / 581{,}500)^{1/2} \\
&= 0.106 - 5.858 \times 0.378 \times 10^{-2} \\
&= 0.106 - 0.022 = 0.084\ \text{m/sec}
\end{aligned}
$$

4) Summary of Calculated River Velocities 2m above Riverbed

Q	U
500 m^3/sec	0.350 m/sec
300 m^3/sec	0.224 m/sec
100 m^3/sec	0.084 m/sec

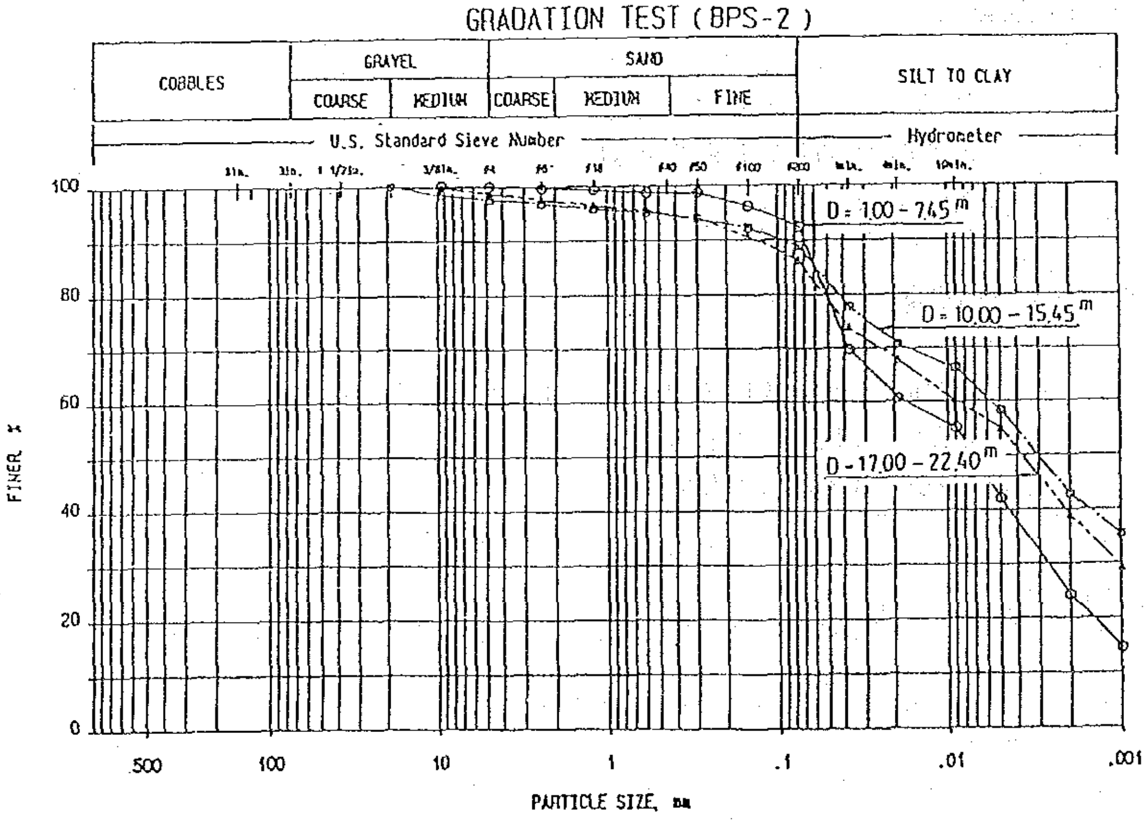
11. 5. 3 Result of Gradation Analysis of Soil to be Excavated

Considering the soil at BPS-2 shown on the below figure, the gradation is composed of sand of 7.6%, silt of 50.4% and clay of 42.0%.

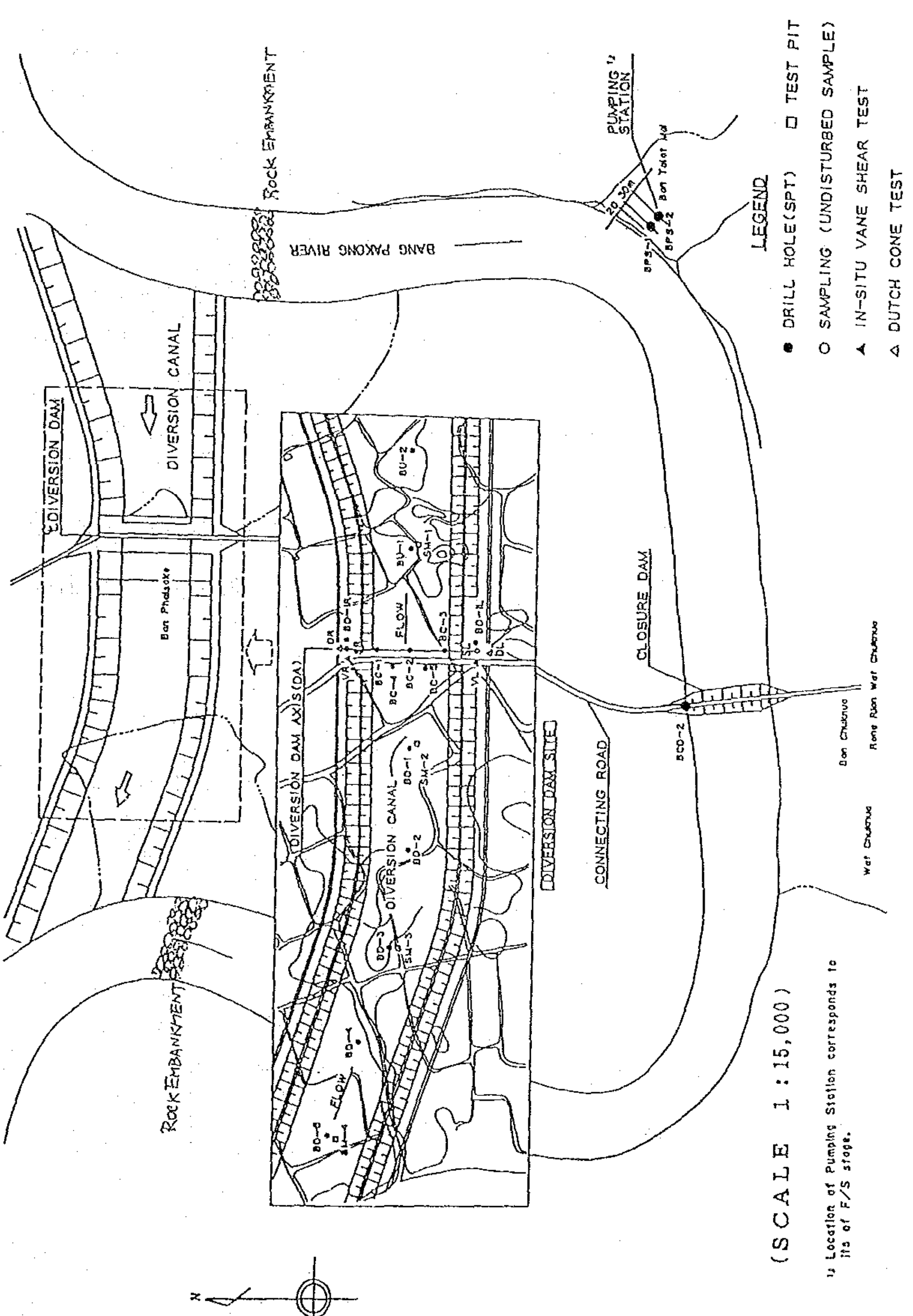
Judging from the grain size accumulation curve, the grain sizes over

d = 0.001mm occupies 85%,
d = 0.01 mm occupies 54%, and
d = 0.1 mm occupies 5%,

GRADATION TEST (BPS-2)



Location Map of Investigations and Tests



11. 5. 4 Spoil Flowing Down and Precipitation

1) Precipitation Velocity and Time Required

Assuming that the water specific gravity of the river is 1.064, the figure "Relation between Grain Size and Critical Precipitation Velocity" indicates that;

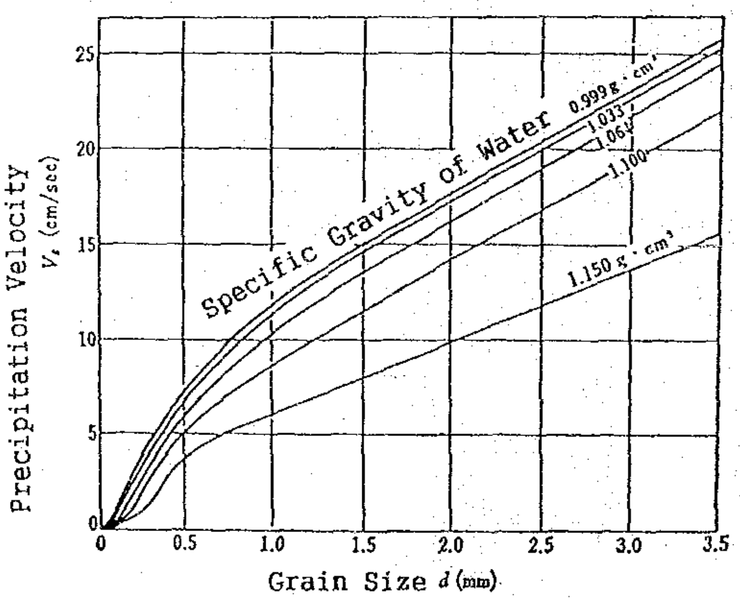
in case of d = 0.001 mm, the critical precipitation velocity will be,
$Vg_{0.001} = 0.013$ cm/sec $= 1.3 \times 10^{-4}$ m/sec

in case of d = 0.01 mm, the critical precipitation velocity will be,
$Vg_{0.01} = 0.11$ cm/sec $= 1.1 \times 10^{-3}$ m/sec

Therefore, the precipitation time required will be,

$T_{0.001} = 2.0/1.3 \times 10^{-4} = 15{,}358$ sec $= 4.27$ hr
$T_{0.01} = 2.0/1.1 \times 10^{-3} = 1{,}818$ sec $= 0.51$ hr

FIGURE RELATION BETWEEN GRAIN SIZE AND CRITICAL PRECIPITATION VELOCITY



2) Distance Required for Soil Precipitation in Designed Minimum Grain Size

$L = (H - y)u/Vg$

L : Distance required for soil precipitation in design minimum grain size (m)
H : Water depth (m)
y : Depth from water surface to pipe delivery (m)
It is assumed, $H - y = 2m$.
u : River velocity at y point 2m above riverbed (m/sec)
Vg: Critical precipitation velocity of design minimum grain size to be precipitated (m/sec)

a) In Case of $Q = 500\ m^3/sec$

$u = 0.350$ m/sec
$\therefore L_{0.001} = 2.0 \times 0.350/1.3 \times 10^{-4} = 0.538 \times 10^4 = 5{,}380\ m \doteqdot 5.4\ km$
$L_{0.01} = 2.0 \times 0.350/1.1 \times 10^{-3} = 636\ m$

b) In Case of $Q = 300 m^3/sec$

$u = 0.224$ m/sec
$\therefore L_{0.001} = 2.0 \times 0.224/1.3 \times 10^{-4} = 0.345 \times 10^4 = 3{,}450\ m \doteqdot 3.5\ km$
$L_{0.01} = 2.0 \times 0.224/1.1 \times 10^{-3} = 407\ m$

c) In Case of $Q = 100 m^3/sec$

$u = 0.084$ m/sec
$\therefore L_{0.001} = 2.0 \times 0.084/1.3 \times 10^{-4} = 0.129 \times 10^4 = 1{,}290\ m \doteqdot 1.3\ km$
$L_{0.01} = 2.0 \times 0.084/1.1 \times 10^{-3} = 153\ m$

d) Distance Required for Soil Precipitation in Designed Minimum Grain Size

For precipitating grain size over d = 0.001 mm;
in case, $Q = 500\ m^3/sec$, a distance of about 5.4 km will be required,
in case, $Q = 300\ m^3/sec$, a distance of about 3.5 km will be required and
in case, $Q = 100\ m^3/sec$, a distance of about 1.3 km will be required,

For precipitating grain size over d = 0.01 mm;

in case, Q = 500 m^3/sec, a distance of about 538 m will be required,

in case, Q = 300 m^3/sec, a distance of about 345 m will be required and

in case, Q = 100 m^3/sec, a distance of about 129 m will be required,

11.5.5 Conclusion

It is presumed that the section of the old river course usable for spoil discharging is about 5.0 km of 5.77 km in total.

A closure dam will be constructed about in the middle of the old river course in the very near future.

It will be unavoidable that a part of spoil is temporarily precipitated on the closure damsite, because spoil discharging works precede the closure dam construction.

In case, the river discharge Q = 500 m^3/sec, if dredged spoil is discharged at the upmost part of the old river course, almost all amount of grain size over d = 0.001 mm corresponding to 85% of the discharged spoil amount will be able to be precipitated by the lowest part of the old river course.

In case, the river discharge Q = 100 m^3/sec, if dredged spoil is discharged only at the upper reaches 1.3 km form the lowest part of the old river course, all amount of grain size over d = 0.001 mm corresponding to 85% of the discharged spoil amount will be able to be precipitated.

The spoil in grain size over d = 0.01 mm corresponding to 54% of the discharged spoil amount will be precipitated within 500 m downstream from the delivery of pipe.

Two rock embankments of which crest levels are 3 m under the river water surface will be built for keeping spoil from flowing down at the upmost and lowest sites of the old river course. (cf. Figure)

According as a reduction of the old river cross sectional area by spoil precipitation, the mean river velocity will become large. However, since a diversion canal to cut the river course short is passed through soon, the river discharge through the old river course will be divided into two, and be diminished.

Finally in case, the closure dam is completed, it will become nil.

11.5.6 Result of Unsteady Flow Analysis

The result of unsteady flow analysis is shown as follows;

Q (m^3/sec)	V (m/sec)
1,600	1.01
500	0.33 ± 0.02
300	0.20 ± 0.02
200	0.14 ± 0.01
100	0.07 ± 0.01

Assuming the velocities estimated by unsteady flow analysis at the site concerned, as V', and comparing the velocities in uniform flow calculation V with the above V' max, the result will be as follows;

Q (m^3/sec)	V (m/sec)	V' max (m/sec)	V/V' max
1,600	0.836	1.01	-
500	0.436	0.35	1.25
300	0.286	0.22	1.30
100	0.106	0.08	1.33

As a result, V/V' max were ranged between 1.25 and 1.33.

In the sedimentation calculation, the distance to be required for precipitation of min. grain size of particles, C = 1.5 is usually taken as the safety coefficient. Similarly this time, regarding V/V' max as the safety coefficient, the above-stated conclusion drawn from the velocities in uniform flow, V, will be able to be used as it is, further with safety coefficient.

Therefore, the conclusion will not be revised.

11.6 Calculation of River Water Turbidity at Lowest Point of Old River Section

The dimensions of pump dredgers for diversion canal excavation are as follows:

18" × 2 sets .... 1200 p.s. each

Excavating capacity (Qe) will be,

$Qe = 2 \text{ sets} \times 250 \text{ m}^3/\text{hr} = 500 \text{ m}^3/\text{hr} = 0.139 \text{ m}^3/\text{sec}$

(Discharging capacity (Qd) will be

$Qd = 500 \text{ m}^3/\text{hr} \times 10 = 5{,}000 \text{ m}^3/\text{hr}$ mixed with soil and water)

1) In Case of $Q = 500 \text{ m}^3/\text{sec}$

The critical precipitation velocity of soil grains at the lowest point of the old river section (Vg) will be,

$Vg = (H\text{-}y)\, u/L = 2.0 \times 0.350 \text{ m/sec}/5{,}000 \text{ m}$

$= 0.00014 \text{ m/sec} = 0.014 \text{ cm/sec}$

(Time required for precipitation (Ts) will be,

$Ts = 2.00 \text{ m}/0.00014 \text{ m/sec} = 14{,}286 \text{ sec} = 3.97 \text{ hr}$)

It was found out from the figure "Soil Grain size and Critical Precipitation Velocity" that the excavated soil grain sizes smaller than 0.0011 mm flowed down.

Judging from the Soil Grain Accumulation Curve, the total soil amount flowing down is 15.5% of all.

Therefore, total soil amount flowing down (Qe') will be,

$Qe' = Qe \times 0.155 = 0.139 \text{ m}^3/\text{sec} \times 0.155 = 0.0215 \text{ m}^3/\text{sec}$

Assuming that the total soil amount flowing down is conveyed by river discharge $Q = 500 \text{ m}^3/\text{sec}$, and soil unit weight is 1.60 g/cm^3, mean turbidity (Tmean) will be, Tmean $= 0.0215\ m^3/sec/500 = 0.0000430\ m^3/1m^3$

(soil amount) (river water amount)

$= 0.0430$ lit./m3 $= 0.0430 \times 1.6$ kg/lit.

$= 0.0688\ kg./m^3 = 68.8\ g/m^3 = 68.8$ p.p.m

2) In Case of $Q = 300\ m^3/sec$

The critical precipitation velocity of soil grains at the lowest point of the old river section (Vg) will be,

Lowest point of the old river section (Vg) will be,

$Vg = (H - y)\ u/L = 2.0\ m \times 0.224\ m/sec/5{,}000\ m$

$= 0.0000896\ m/sec = 0.00896\ cm/sec$

(Time required for precipitation (Ts) will be,

$Ts = 2.00\ m/0.0000896\ m/sec = 22{,}321\ sec = 6.20\ hr$)

It was found out from the figure "Soil Grain Size and Critical Precipitation Velocity" that the excavated soil grain sizes smaller than 0.00036 mm flowed down.

Judging from the Soil Grain Accumulation Curve, the total soil amount flowing down is about 8% of all.

Therefore, total soil amount flowing down (Qe') will be,

$Qe' = Qe \times 0.08 = 0.139\ m^3/sec \times 0.08 = 0.01112\ m^3/sec$

Assuming that the total soil amount flowing down is conveyed by river discharge Q = 300 m/sec, and soil unit weight is $1.60\ g/cm^3$, mean turbidity (Tmean) will be,

Tmean $= 0.01112\ m^3/sec/\ 300\ m^3/sec = 0.0371\ lit/m^3$

$= 0.0371\ lit./m^3 \times 1.6\ kg/lit. = 0.0594\ kg/m^3$

$= 59.4\ g/m^3 = 59.4$ p.p.m

3) In case of $Q = 100\ m^3/sec$

The critical precipitation velocity of soil grains at the lowest point of the old river section (Vg) will be,

$Vg = (H - y)\ u/L = 2.0\ m \times 0.084\ m/sec/5{,}000\ m$
$= 0.0000336\ m/sec = 0.0036\ cm/sec$
(Time required for precipitation (Ts) will be,
$Ts = 2.00\ m/0.0000336\ m/sec = 59{,}524\ sec = 16.53\ hr$)

It was found out from the figure "Soil Grain Size and Critical Precipitation Velocity" that the excavated soil grain sizes smaller than 0.000134 mm flowed down.

Judging from the Soil Grain Size Accumulation Curve, the total soil amount flowing down is about 2% of all.

Therefore, total soil amount flowing down (Qe') will be,
$Qe' = Qe \times 0.02 = 0.139\ m^3/sec \times 0.02 = 0.00278\ m^3/sec$
Assuming that the total soil amount flowing down is conveyed by river discharge $Q = 100\ m^3/sec$, and soil unit weigh is $1.60\ g/cm^3$, mean turbidity (Tmean) will be,

$$\begin{aligned} Tmean &= 0.00278\ m^3/sec/100\ m^3/sec = 0.0278\ lit./m^3 \\ &= 0.0278\ lit./m^3 \times 1.6\ kg/lit. = 0.0445\ kg/m^3 \\ &= 44.5\ g/m^3 = 44.5\ p.p.m \end{aligned}$$

4) In Case of $Q = 12m^3/sec$ (in the most droughty month)

Since the river velocity becomes nearly zero, all the excavated soil amount will be able to be settled within 5 km.

The mean turbidity at the lowest point of the old river section will be very low, then not change from the mean turbidity at the entrance of the old river section.

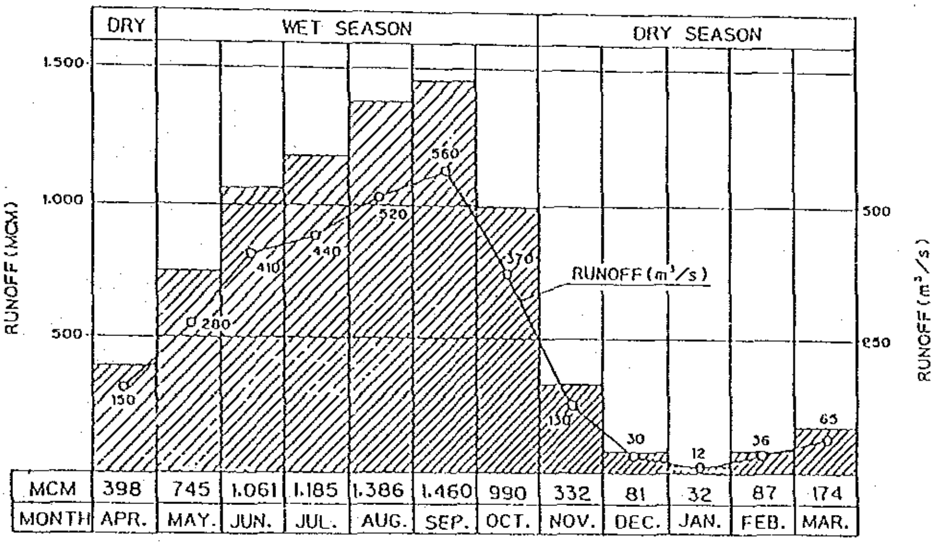
5) Summary of Calculated Mean Turbidity by River Discharge (Tmean)

Q	Tmean
500 m^3/sec	68.8 p.p.m
300	59.4
100	44.5
12	20-0

6) Conclusion

If the dredged materials dumped into the old river course should unexpectedly have a bad effect upon the river water quality, the method for depositing the dredged materials would be reexamined by RID.

AVERAGE MONTHLY RUNOFF OF BANG PAKONG RIVER



If the dredged materials dumped into the old river course has a bad effect upon the river water quality, the method for depositing the dredged materials should be reexamined.

11.7 Ground Water Control for Excavation Work

1) Necessity of Avoid Seepage Water

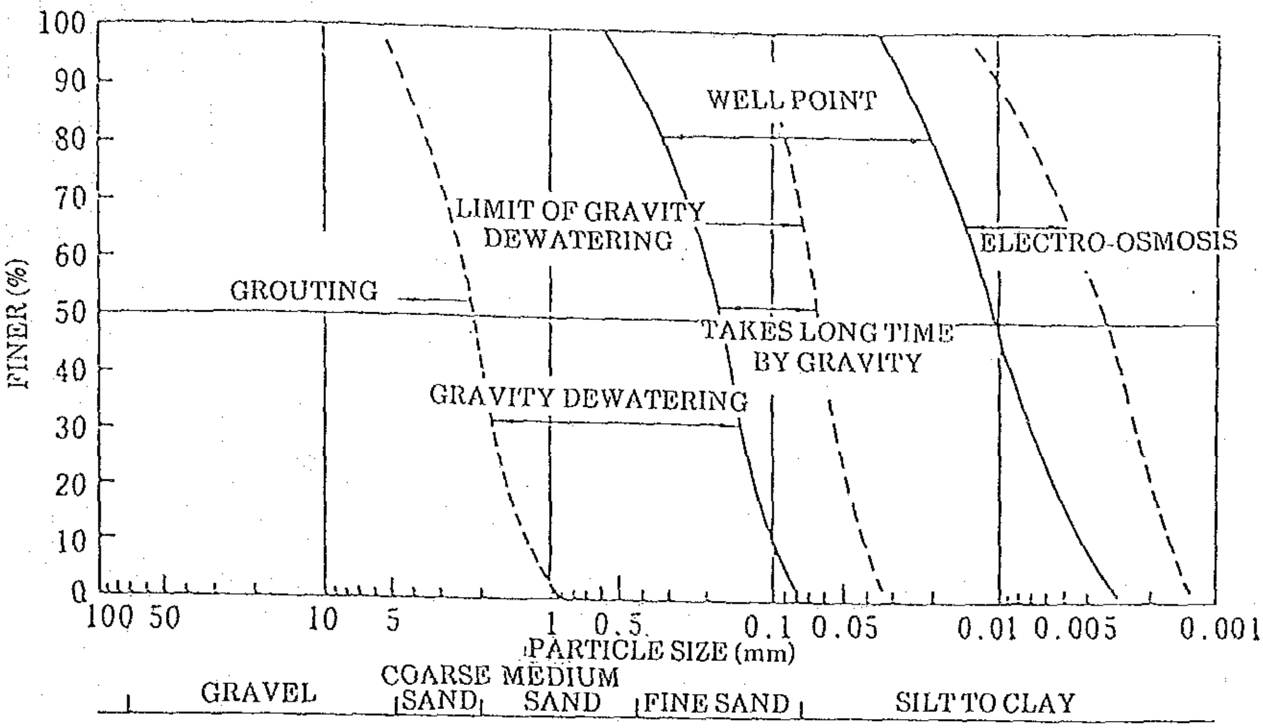
Excavation for the diversion dam will be made on the layer found at EL (-) 9.0 meters to EL 1.5 meters which is geologically specified as a soft clayey layer with N-value of 1 to 2, cohesion of 0.1 to 0.5 kg/cm^2. Generally the extreme moisture content clayey soil becomes high liquidity, and its very difficult for Control, and when there are under ground water stream or well water, its seepage force activated to stream directions, then excavated slope becomes to unstable. Therefore its necessary applied some countermeasure for slope protections.

2) Method of Control for Underground Water

The method of Control for underground water are classified as following table.

- Reduce ground water level
  - Gravity dewatering
    - Pit
    - Deep well
    - Underdrain, Open channel
  - Forced dewatering
    - Well point
    - Electro-Osmasis
    - Vacuum dewatering
- Cut off ground water
  - Ground solidifying
    - Grouting
    - Freezing
  - Continuous wall
    - Slurry wall
    - Sheet wall
    - Steel sheet pile wall
    - Continuous concrete wall
  - Compressed air

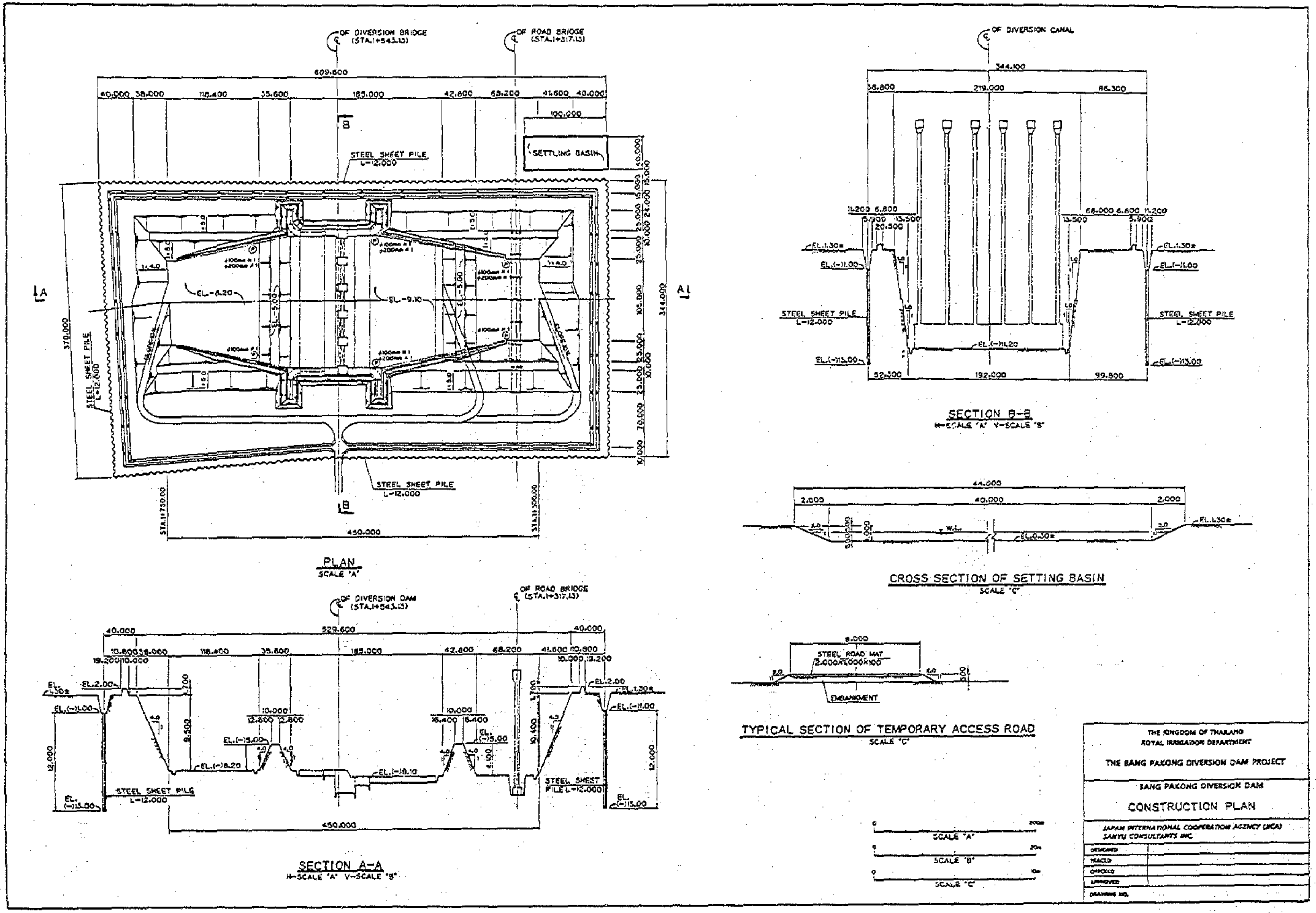
The relation of soil grading and suitable ground water control methods as shown in following Figure;



3) Selection of Suitable Method

The results of the soil tests around the dam site is shown that, gradation is smaller than The Well Point limits, and fitted The Electro-Osmosis limits. But there was only few experiences for the Electro-Osmosis method, so that when Well Point method could not be applied for dewatering, should be chosen another methods.

Then for this case we recommended The Steel Sheet Pile cut off wall method. This method is most suitable, and effectual.


PLAN
SCALE 'A'
SECTION A–A
H–SCALE 'A' V–SCALE 'B'
SECTION B–B
H–SCALE 'A' V–SCALE 'B'
CROSS SECTION OF SETTING BASIN
SCALE 'C'
TYPICAL SECTION OF TEMPORARY ACCESS ROAD
SCALE 'C'
SETTLING BASIN
STEEL SHEET PILE L=12.000
STEEL ROAD MAT 2.000×1.000×100
EMBANKMENT
℄ OF DIVERSION BRIDGE (STA.1+543.13)
℄ OF ROAD BRIDGE (STA.1+317.13)
℄ OF DIVERSION CANAL
℄ OF DIVERSION DAM (STA.1+543.13)
SCALE 'A'
SCALE 'B'
SCALE 'C'
THE KINGDOM OF THAILAND
ROYAL IRRIGATION DEPARTMENT
THE BANG PAKONG DIVERSION DAM PROJECT
BANG PAKONG DIVERSION DAM
CONSTRUCTION PLAN
JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)
SANYU CONSULTANTS INC.
DESIGNED
TRACED
CHECKED
APPROVED
DRAWING NO.

APPENDIX - 12 : CONSTRUCTION COST ESTIMATION

APPENDIX - 12. CONSTRUCTION COST ESTIMATION

LIST OF CONTENTS