3) Verification of Runoff Models

A large number of flow gauging stations are in operation in the whole watershed (C.A. = 17,660 sq.km). The station kgt. 3 (C.A. = 7,520 sq.km), which occupies 45% of the whole watershed, has been selected for the model verification.

MAJOR FLOODS OBSERVED AT THE STATION KGT. 3 ARE AS IN THE FOLLOWING TABLE

Kgt. 3 Major Floods Observed

No. 1	2,220.0 cu.m/s	6, Oct. 1990
2	1,111.0	23, Sep. 1969
3	1,064.0	20, Oct. 1983
4	1,056.5	3, Oct. 1978
5	1,042.0	31, Aug. 1942

(Records : 1941-1990)

As shown in the above, the recorded maximum is 2,220 cu.m/s in 1990 and the exceeding probability is equivalent to 1/26,248; i.e. the magnitude was extreme.

Verification of the runoff model has thus been made according to the 1983 flood due not only to its proximity to the present implying less deviation in the pattern of land use but also to the availability of the observed sea level records during the time.

4.2.3 Hydraulic Analysis of Flood

1) Method of Hydraulic Analysis

The watershed is, as aforementioned, composed of relatively low-elevated rolling hills and of flat and low land along the River, and in the latter so-called ponding irrigation is practiced by storing flooded water in the fields.

The land use being as it is, a continuous reservoir model composed of river channel functions and dead storage functions, like those of major beds and ponding-irrigated fields, has been employed to determine key parameters for structural design such as gate crest elevation and to estimate the state of flooding and inundation.

In the hydraulic model, the runoff discharges obtained by dividing the whole watershed into 54 blocks, and by the hourly sea levels at the estuary of the Bang Pakong River, are the upstream and downstream boundary conditions respectively. And for the river channel functions, an unsteady-flow approximate model has been employed because of its compatibility with the flat and low land areas. The storage functions of major beds and others have also been taken into the model.

2) Conditions for Analysis

i) Inflow

Inflow to the diversion dam is calculated by the flood runoff model.

ii) Sea Level

Sea levels have been observed and recorded hourly at the estuary of the Bang Pakong River since Sep. 1981. The records from Sep. 1983, shown in the following table, are applied in the hydraulic analysis.

Highest high-water level	(+) 2.02 m(MSL)
High water level	(+) 1.27 〃
Lower high-water level	(+) 1.00 〃
Mean sea level	(+) 0.16 〃
Higher low-water level	(-) 0.82 〃
Low water level	(-) 0.95 〃
Lowest low-water level	(-) 1.67 〃

(Records : 1982-1991)

iii) Storage Volume in Flooded Block

Storage volumes in major bed sections at present have been calculated from cross-sections of the river course, while those taken after flood dike construction are obtained by the use of 1 : 10,000 topo-maps.

iv) River Channel

The hydraulic parameters such as flow area and hydraulic radius have been calculated from cross-sections of the river taken along 200 km at 2.0 km intervals by the Survey Division, RID. The whole river channel to the confluence with the Nakon Nayok River has been divided into 30 short channel sections as shown in Fig. 4-1. Referring to the study in the field and on the Chao Phraya River[1/], the values of the coefficient of roughness have been decided as 0.031 in the downstream and 0.023 in the upstream reaches respectively. The unit time interval of calculation Δt is 30 seconds.

3) Verification of Hydraulic Model

The subject flood for verification is the same flood in Sep. 1983 as for the runoff model. The model has been verified with the observed flood water levels along the river.

Figure 4-2 shows the result of the verification, wherein the observed figures are water levels at major sluices[2/] along the river. It indicates that the simulated water levels are fairly close to the observed ones.

Figure 4-3 shows the temporal behavior of the water level along the 30 channel sections from the estuary to the confluence with the Nakon Nayok River.

4) Design Flood and Maximum Water Level

By applying the design rainfall (W=1/50) to the verified hydraulic model, the design flood discharge and maximum water level can be determined.

Regarding the construction of flood protection dikes in the river system, no master plan for construction has been established yet. Two alternative cases, with and without the dike along the left bank upstream and

** 1/ "Flow Analysis on Tidal Domain in Chao Phraya Rier in Thailand" in "Water and Soils, 1988". n=0.025 - 0.035 and n=0.022 respectively up and downstream from the RID Head Office location.

* 2/ Pages 3-90, "Environmental Assessment Report" by Kaseset Univ.

FIGURE 4-1 BLOCKING DIAGRAM OF BANG PAKONG RIVER

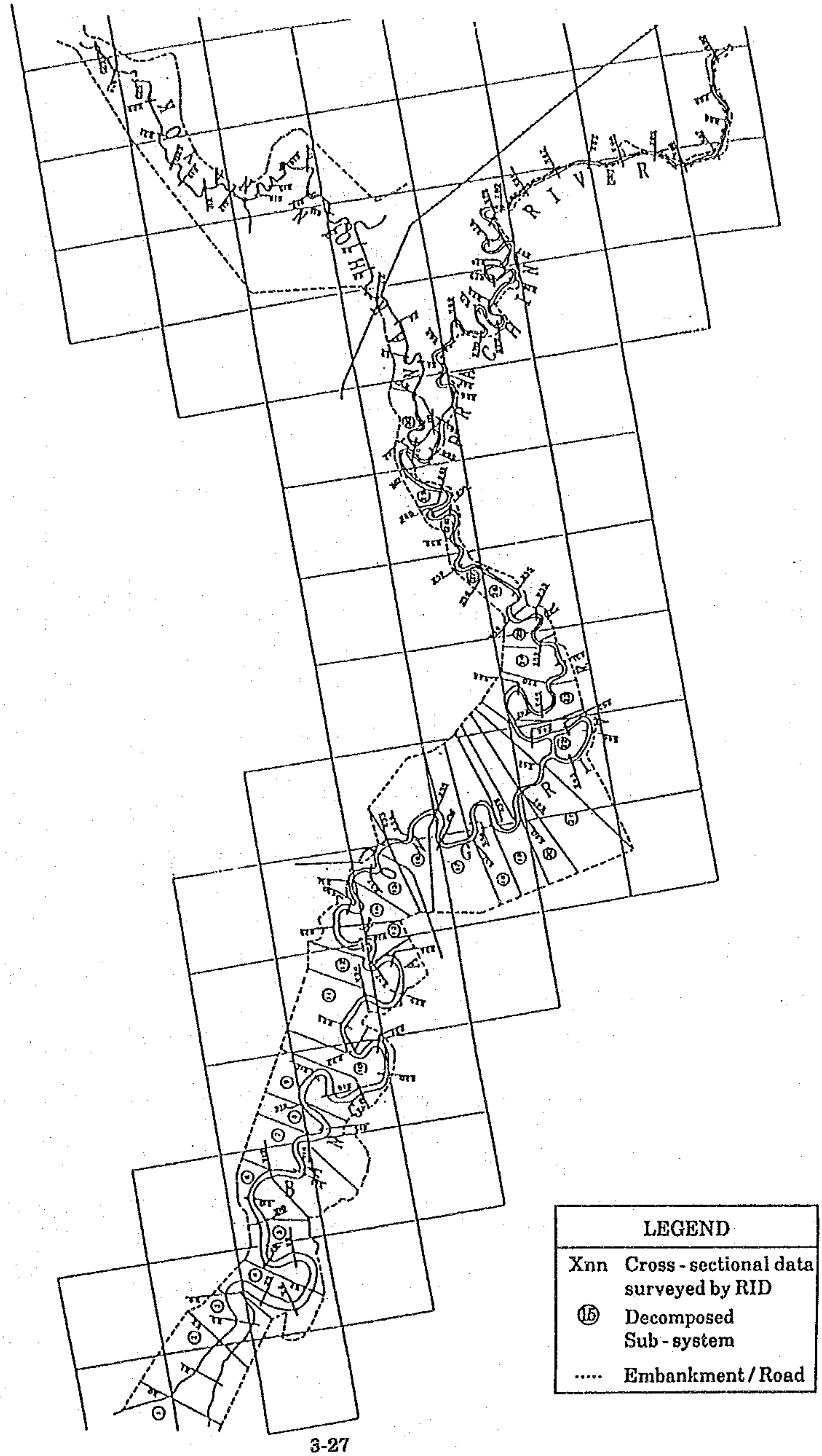


FIGURE 4-2 RESULT OF HYDRAULIC ANALYSIS IN OCTOBER 1983 (HYDRAULIC PROFILE)

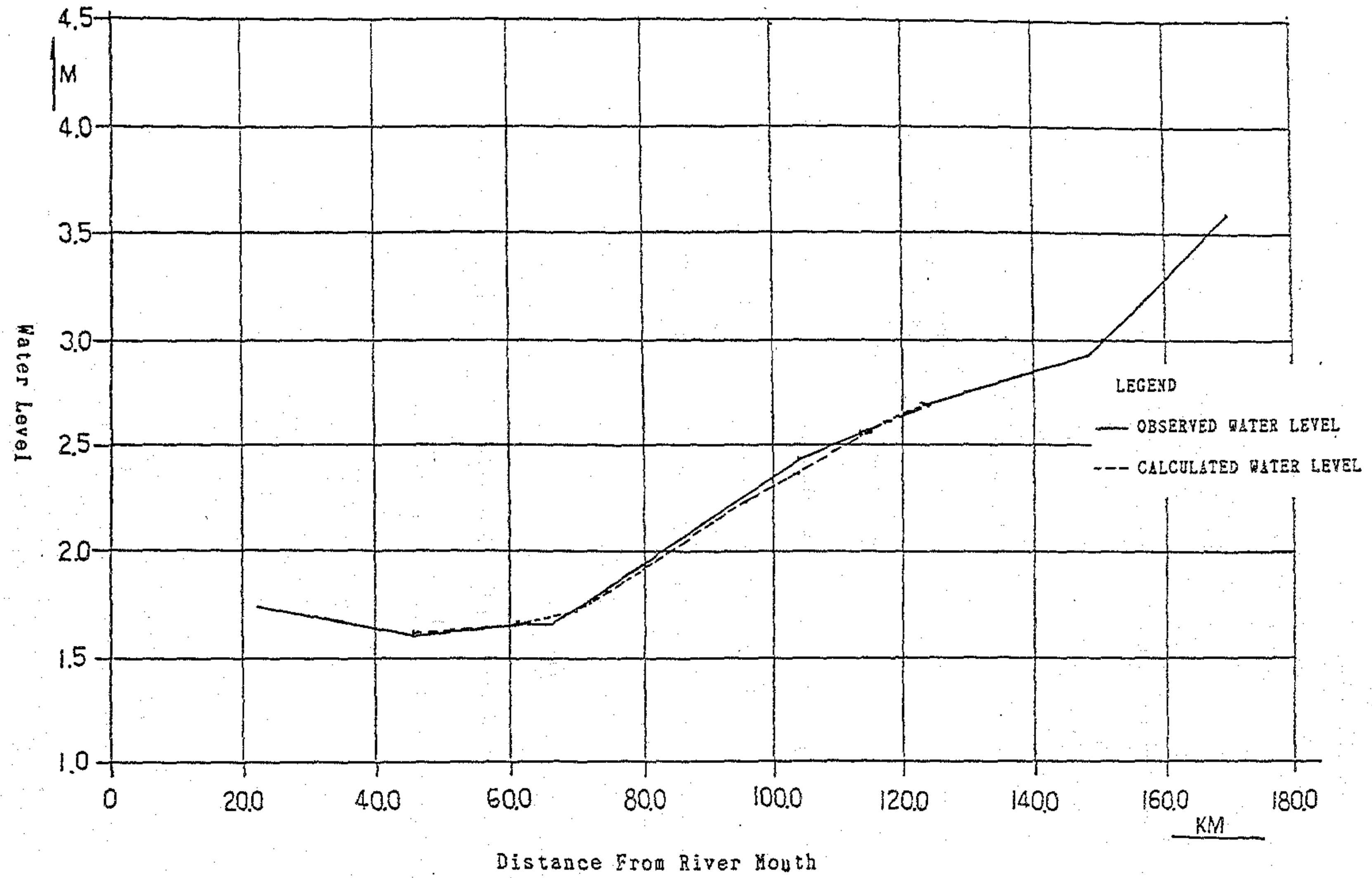
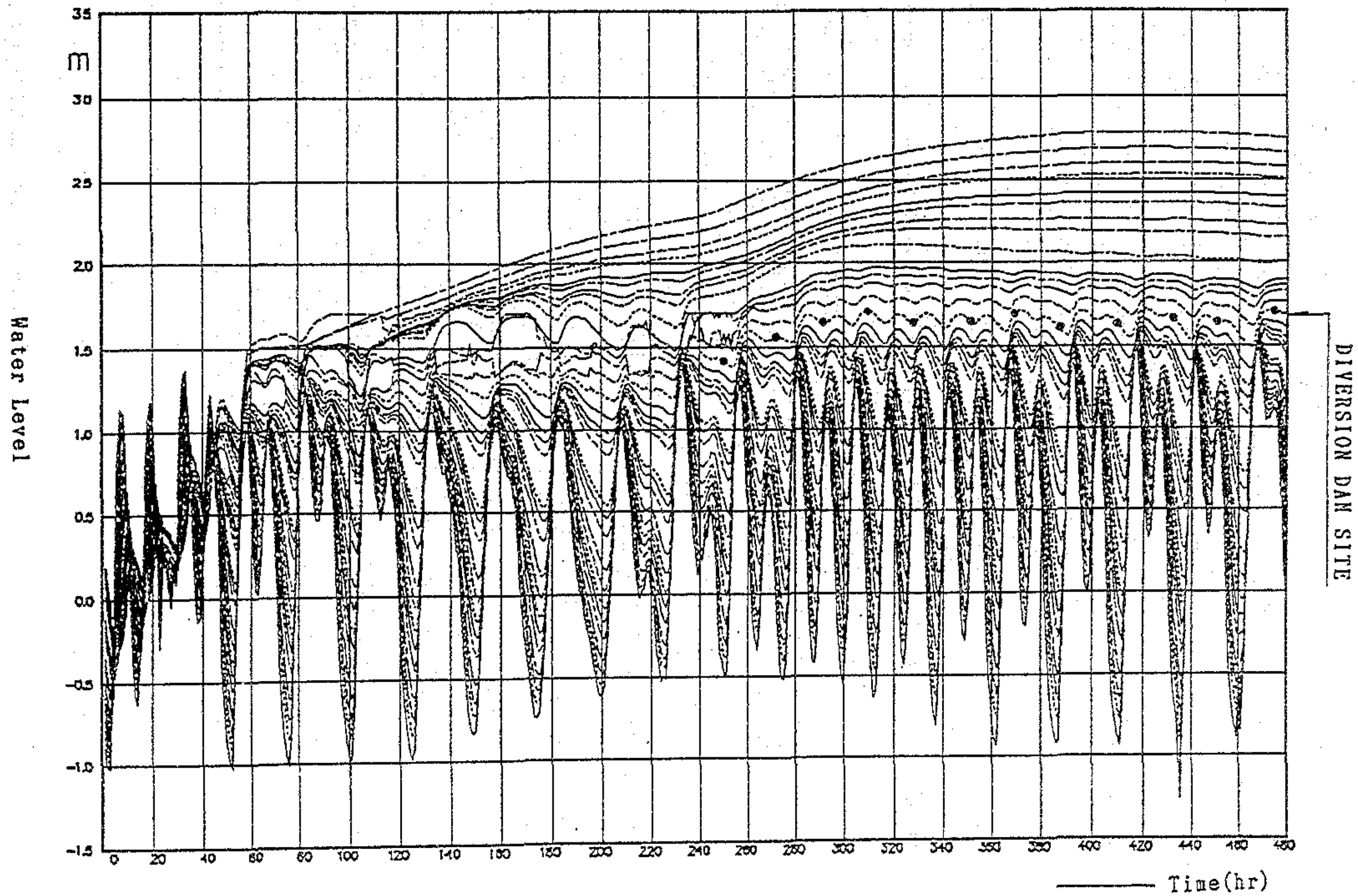


FIGURE 4-3 RESULT OF HYDRAULIC ANALYSIS IN OCTOBER 1 983 (TEMPORAL BEHAVIOR OF WATER LEVEL)



downstream from the diversion dam are studied. Alignment of the dike is assumed by taking account of the boundary of Chachoengsao Irrigation Project areas and existing roads.

Figures 4-4 and 4-5 indicate the results of analyses in the 'without' and 'with' flood dike cases respectively and, as in Figures 4-3, show water levels divided into 30 channel sections. Flood discharge and maximum water levels at the dam site are given as 1,590.6 cu.m/s and (+) 2.36 m before dike construction and 1593.4 cu.m/s and (+) 2.30 m after dike construction respectively.

Consequently, the maximum water level and flood discharge are herein decided as (+) 2.40 m and 1,600 cu.m/s by rounding up from (+) 2.36 m and 1593.4 cu.m/s respectively. The field standing water level along Khlong Tha Thong Long is lowered by 0.35 m from (+) 2.60 m to (+) 2.25 m by flood dike construction, however the duration of water logging in the fields is not substantially reduced due to the same difficulty in gravity drainage, governed by the river water level.

4. 3 Water Level in Dry Season

1) Analysis Model and Analysis Conditions

All gates of the diversion dam are kept closed during the dry season, and functions of the river channel change substantially.

The river channel model applied for the flood analysis of flow behavior before and after the dam construction and to examine the magnitude of its effects. The analysis conditions require that the gates be closed and that there be no flow in the channel including side inflows. Meanwhile the sea levels applied in the downstream area are the same as those for 1983.

2) Results of Analysis

Results of the analysis are as shown in Figure 4-6 and indicate that the upstream water level at the dam becomes elevated by about 0.30 m after gate

FIGURE 4-4 FLOOD WATER LEVEL OF 50 YEARS PROBABILITY (BEFORE CONSTRUCTION OF DIVERSION DAM)

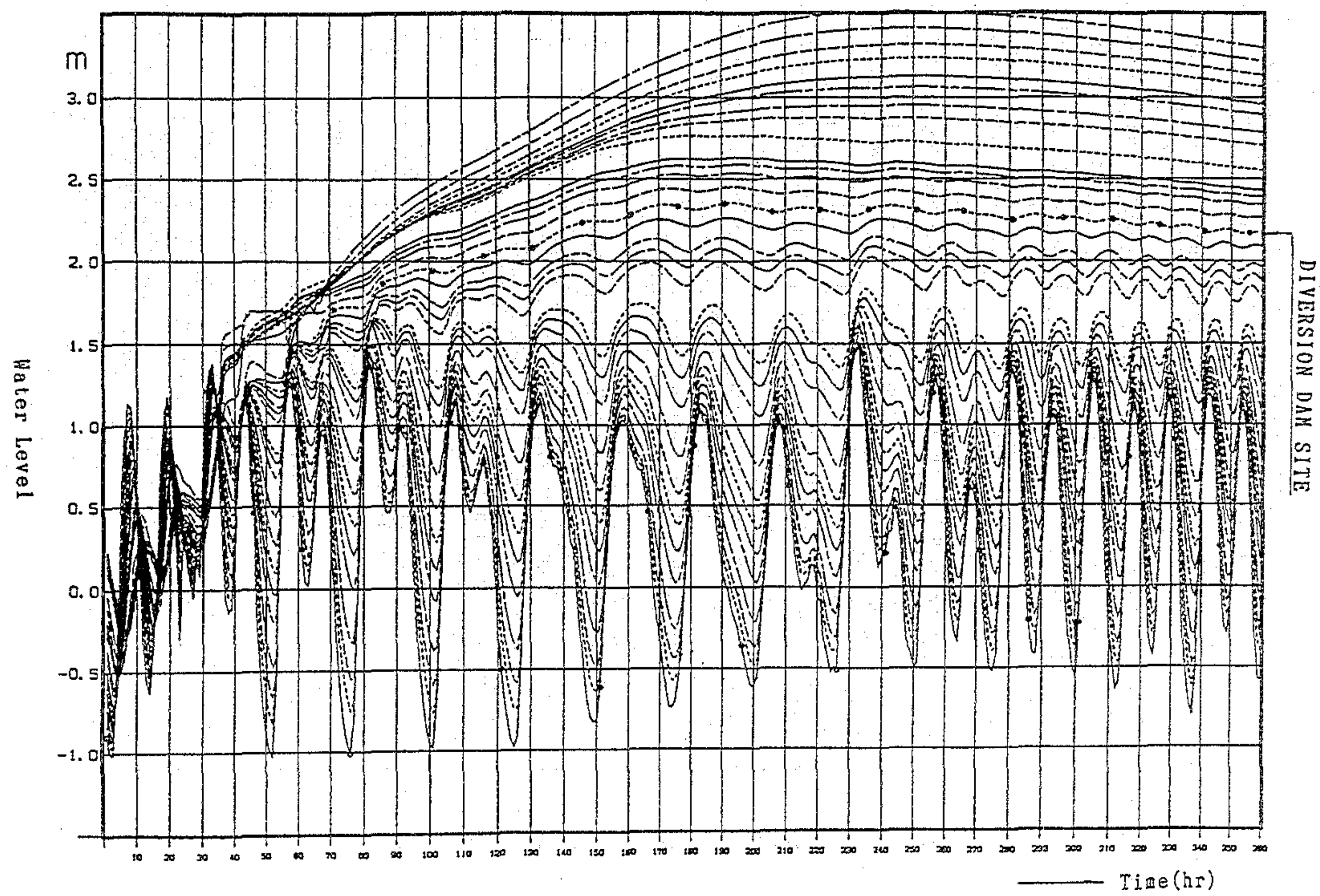


FIGURE 4-5 FLOOD WATER LEVEL OF 50 YEARS PROBABILITY (AFTER CONSTRUCTION OF DIVERSION DAM)

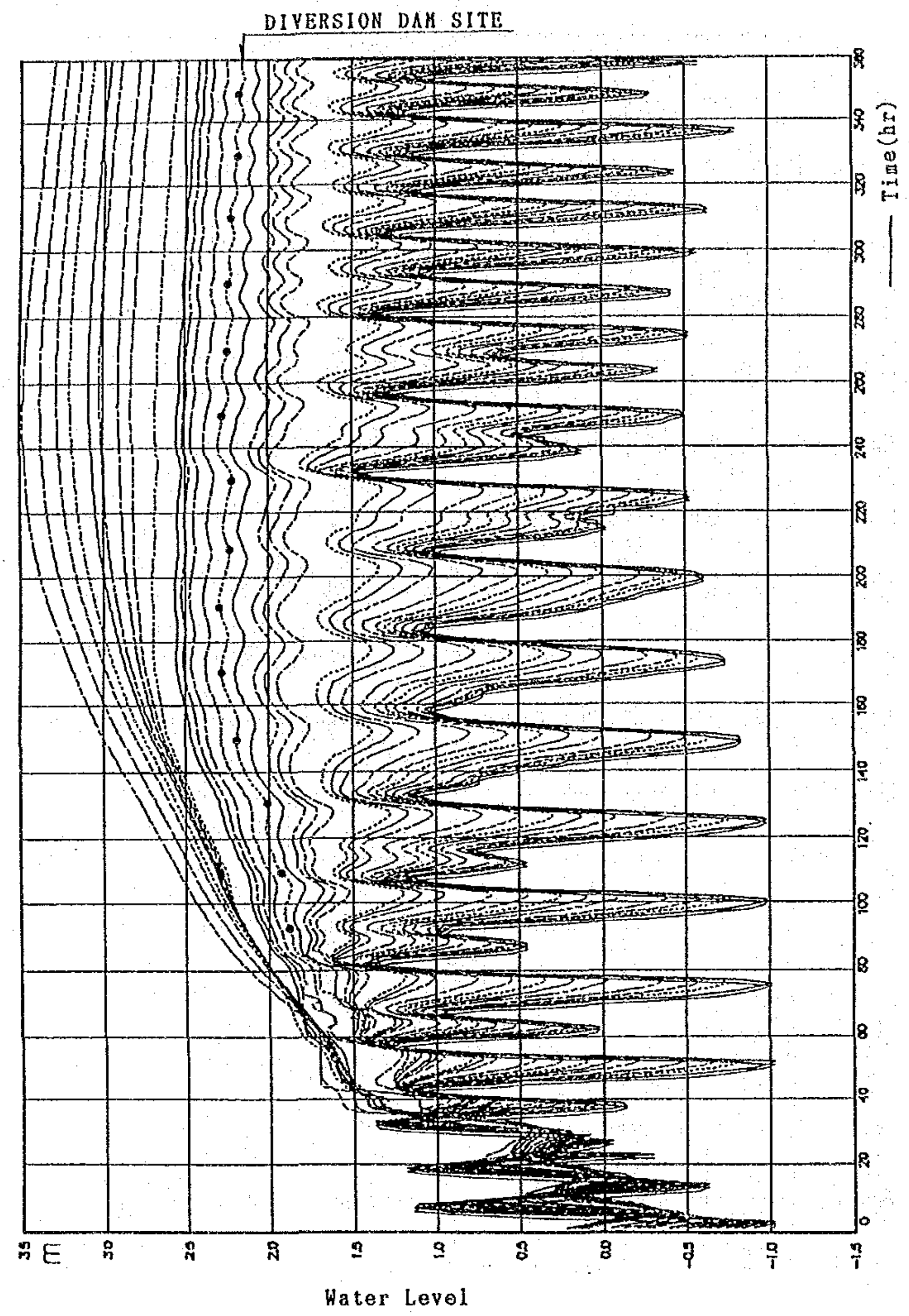
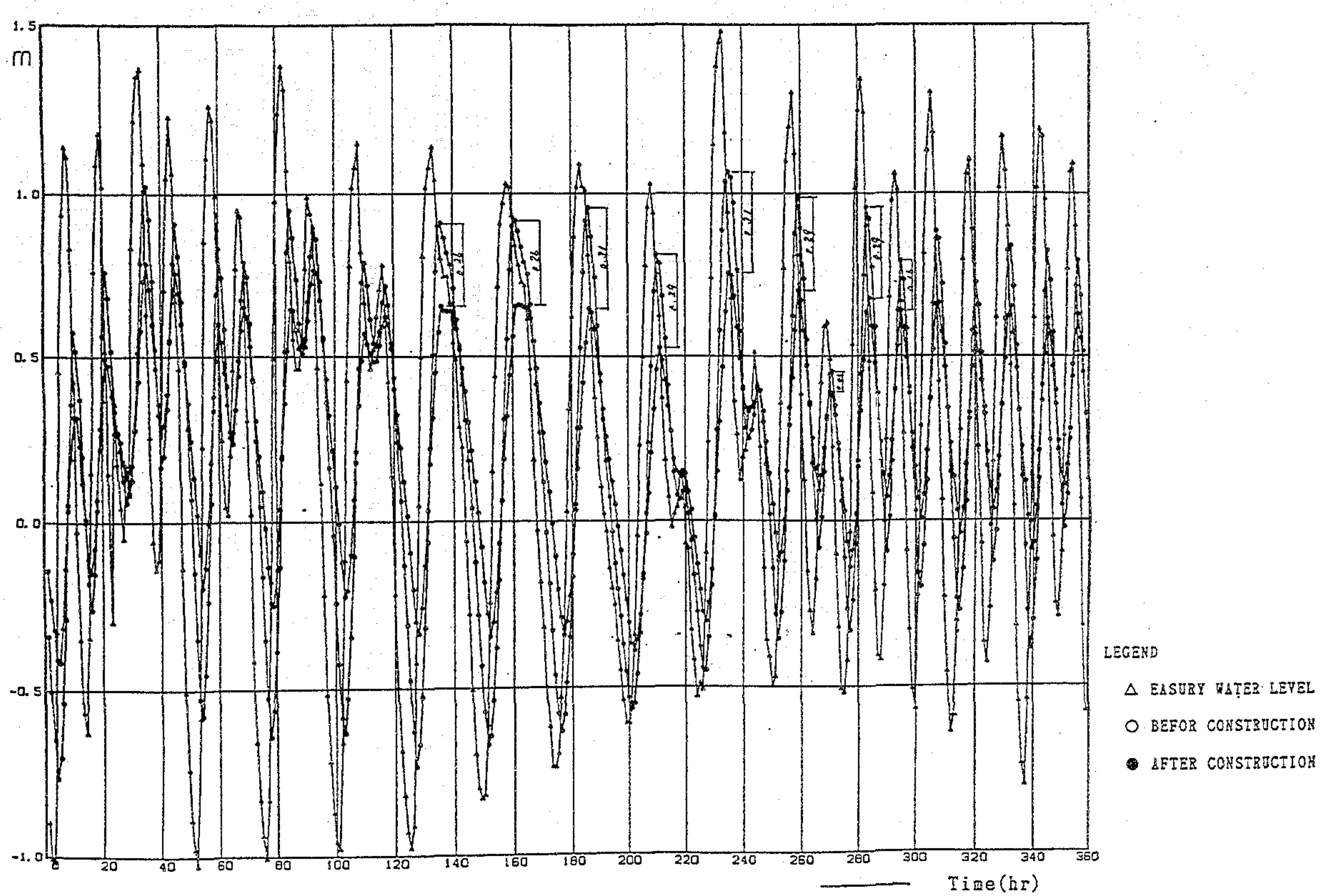


FIGURE 4-6 FLUCTUATION OF WATER LEVEL WITH AND WITHOUT DIVERSION DAM IN DRY SEASON



closure. However, the height is still lower than the sea level at the estuary and does not exceed the crest of the minor bed revetments.

On the other hand downstream from the dam the water level partly exceeds the revetments crest. Some countermeasures may be required such as alteration of land use on the major bed or elevation of the revetments crest.

4.4 Operation Rule of Tide Protection Gate

Tide protection gates have to satisfy the functions of protecting from salt water intrusion, storing fresh water in the channel, assuring pumping operation and securing safe release of river flow without damage during floods. The gates, therefore must be operated properly.

In order to assume the required operation methodology, by using of the aforesaid hydraulic model and 3 consecutive days of rainfall with 1/2 probability, the hydraulic behavior upon and during gate closure was examined.

1) Timing of Closure

Timing of the gate closure has to be decided so as to satisfy protection from tide intrusion and conservation for the required storage water at the same time. 50 cu./s of channel flow can work for the tide protection, while the water balance study indicates that about 30 cu.m/s of channel flow does not cause a water shortage.

Accordingly, the timing of the gate opening could be June when the flow exceeds 80 cu.m/s. As for the closure, the discharge through the opened gates at the time cannot be quantified so that, replacing the gauging site, it may have to be in November when the flow is reduced to 30 cu.m/s at kgt. 22 and the salt concentration at the tidal protection gates is below the permissible level.

2) Control Water Level and Storage Volume

Considering the ground elevation (+)1.20 m upstream of the dam, the normal water level needs to be about (+)0.70 m at the highest, so as to leave an allowance of about 0.5 m. With the design storage of 30 MCM unchanged the minimum operating level then becomes (-)1.30 m.

3) River Maintenance Flow and Design Water Intake

In coordination with RID, it has been concluded that the river maintenance flow would be 0.1 cu.m/s/100 sq.km, the same as the F/S and the appropriate design water intake by pumps would be 16 cu.m/s.

4) Operation Method of Gates

The gates are 30 m-wide and 5 spans, and among them 2 gates are designed as double leaf gates with upper and lower leaves so that both partial release of water by overtopping of the lowered upper gates and total release by full opening can be managed.

Operation of the tide protection gates must be governed by the following fundamental operations.

a) Total Closing Operation

When the water level in the Bang Pakong Reservoir is lower than the sea level, the whole gates are closed.

b) Total Opening Operation

In cases when release of a large quantity of water is required such as during floods, the gate are fully opened for the release. Even during the transitional period from wet season to dry season, release of floods is promptly managed by monitoring the water level in the upstream with telemeters.

c) Overtopping Release Operation

For control of the responsible release or release of minor floods to maintain the normal water level, the overtopping release operation is made by lowering the two regulating gates. At the time of operation lowering of the upper leaves is adjusted from time to time.

d) Submerged Release Operation

When the water level in the Bang Pakong Reservoir is considerably higher than the sea level, submerged release operation by hoisting the flood gates is made in order to dispose of the sea water that remains in the reservoir.

RULES FOR OPERATION OF TIDE PROTECTION GATES

1) Tide Protection Gate

The dimensions and the key design figures for the operation of the tide protection gates are as follows.

a) Bang Pakong Diversion Dam

Gate crest elevation		EL (+)1.80 m(MSL)
Gate sill elevation		(-)8.20
Flood gate		$^{H}10.00$ m × $^{W}30.00$ m × 3
Regulating gate:	upper leaf	$^{H}3.10$ m × $^{W}30.00$ m × 2
	lower leaf	$^{H}6.90$ m × $^{W}30.00$ m × 2
Hoisting speed		0.30 m/min

b) Bang Pakong Reservoir

Active storage	30 MCM
Normal water level	(+)0.70 m(MSL)
Minimum operation level	(-)1.30 m(MSL)
River maintenance flow	2.49 cu.m/s

2) Method of Reservoir Water Level Calibration

Water level in the reservoir is calculated from the gauge reading at the upstream side of the Bang Pakong Diversion Dam

3) Method of Inflow Calibration

Inflows to the reservoir with total gate closure of the flood gates are calibrated from the water level difference on both sides of the gates and the state of the upper regulating gate openings.

Inflows with the gates fully open or fully closed during high tide, are calculated based on those at kgt. 22.

4) Method of Gate Operation

a) Gate Names

The gates in the set of 5 tide protection gates are named in order from the right bank to the left bank, "No.1 Flood Gate", "No.2 Flood Gate", "No.3 Flood Gate", "No.1 Regulating Gate" and "No.2 Regulating Gate".

b) Whole Opening Operation

In the complete opening of the gates in the transitional period between the dry season and the wet season, the flood gates are opened 0.30 m at a time in the order of No.2 Gate, No.1 Gate and then No.3 Gate. After full opening of the 3 flood gates, both upper leaves of the regulating gates are hoisted till the crest elevation reaches (+)1.80 m. Then the lower leaves of No.1 and No.2 Regulating Gates are alternately hoisted 0.30 m at a time. After that, both upper leaves are fully hoisted.

c) Whole Closure Operation

In the complete closing of the gates in the transitional period between the wet season and the dry season, the lower leaves of No.1 and No.2 Regulating Gates are alternately closed 0.30 m at a time. The flood gates are them closed 0.30 m at a time in the order of No.2 Gate, No.1 Gate and then No.3 Gate. The upper leaves of No.1 and No.2 Regulating Gates are finally closed alternately 0.30 m at a time.

d) Operation of Regulating Gates

For release of river maintenance flow while the gates are fully closed or in order to maintenance the normal water level of (+)0.70 m, the upper leaves of the regulating gates are promptly operated to cause overtopping.

e) Operation for Desalinization

When removal of saline remaining in the reservoir becomes necessary, No.2 Flood Gate is opened at a time when there is a large difference in water level between each side of the gate.

f) Release of Minor Floods

For release of minor floods while the gates are fully closed, the upper leaves of both regulating gates are lowered until the crest elevation of the lower leaves, if necessary.

When the upper leaves are fully lowered down and the monitored water levels in the upstream reaches suggest that the water level in the reservoir exceeds (+)1.30 m, the flood gates are opened 0.30 m at a time in the order of No.2 Gate, No.1 Gate and No.3 Gate so as to maintain a water level below the upper-most water level of (÷)1.30 m. During the above operation, the water levels at the diversion dam and the upstream telemeter sites have to be carefully monitored so as to prevent the reservoir water level from falling below the normal water level of (+)0.70 m through excessive release.

5) Use of Water in the Reservoir

Intakes of irrigation water from the reservoir are made at water levels between (+)0.70 m and (-)1.30 m through the pumping station upstream of the diversion dam. The maximum total intake is 16.0 cu.m/s.

6) Operational Record

Whenever the regulating gates or flood gates are operated, the following are recorded.

a) Reasons for operation
b) Names of gates operated, time of start and end of an operation, and height of the gate opening
c) Reservoir water levels, sea levels, inflow, discharge and pumping intake at the start and end of an operation.

CHAPTER 5. BASIC DESIGN OF DIVERSION CANAL

5. 1 Location and Plane Figure of the Diversion Canal

The diversion canal will be constructed to make a short-cut for the meandering section. It will be located at the site and have the plane figure as shown in Figure 5-1 for the following reasons.

1) The diversion canal will have the plane figure on the upstream side that the left bank side of the diversion dam is in a straight line. The angle of the right bank opening will be 20 degrees with a radius of 1000 meters, four times the canal width, so as to ensure smooth inflow of the river water. Downstream from the diversion dam, both banks of the canal will contact the river at an angle of 20° and with a curve of 1000 m radius.

2) The straight portion of the diversion canal will be extended as far as possible so that both the diversion dam and the road bridge can be constructed over the canal.

5. 2 Hydraulic Design

5. 2. 1 Design Conditions

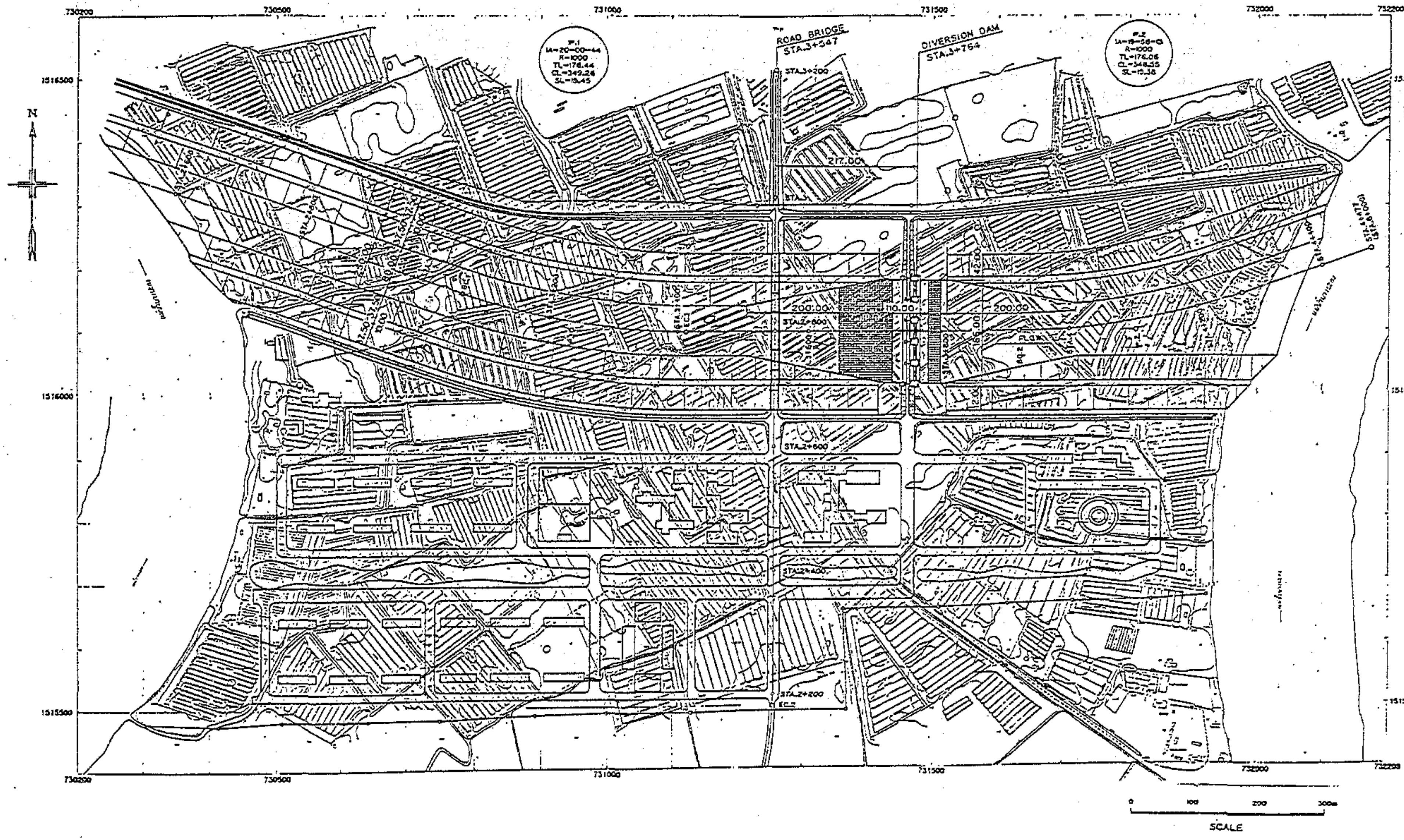
1) Design Flood Discharge

The flood analysis has revealed that the design flood discharge by 50-year probability is $Q = 1600 m^3/s$.

2) Design Flood Depth

The flood analysis has found that the maximum water level at the diversion dam site is Max. WL 2.40 meters, while the design river bed elevation is EL (-) 8.20 meters. Consequently, the design flood depth is H = 10.60 meters.

FIGURE 5-1 PLAN OF DIVERSION CANAL



3) Profile Slope

a) River bed slope

The average slope of the river bed for a distance of about 10 kilometers around the diversion dam is $I_0 = 1/4000$. The portion ($L = 5.77$ km) between Sta. 2 + 230 and Sta. 8 + 000 will be short-cut for construction of the diversion dam, and the river alignment will be shortened by 3.52 kilometers for constructing the diversion canal ($L_2 = 2.25$ km). The river slope of $I_0 = 1/4000$ cannot be changed because of the stability of the river bed. Under these conditions, the difference in river bed elevation, $\triangle H = 3520 \text{ m} \times 1/4000 =$ approx. 0.9 meters, which will be caused by the construction of the diversion dam, should be treated with a drop structure to be provided at the diversion dam site.

b) Flood surface slope

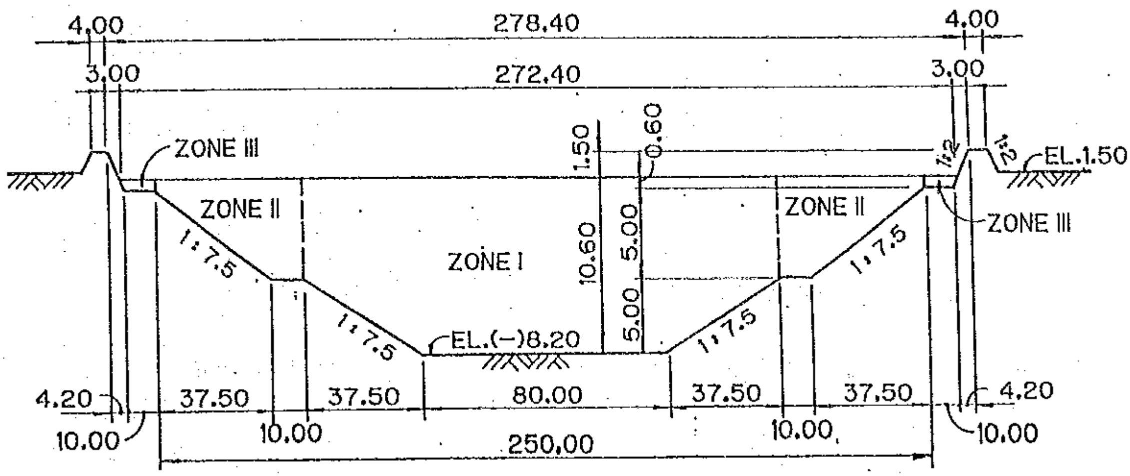
The profile of the Bang Pakong River indicates that the diversion dam site is located at a point 76.5 kilometers upstream from the estuary, having a Max. WL of 2.40 meters. The flood surface slope, therefore, is obtained as $I = 2.40/76500 =$ approx. 1/32000.

4) Section of the River

Currently, the river surface width at the diversion dam extends from 190 to 260 meters and the cross-sectional area of the river is from 1300 to 1900 m^2, 1750 m^2 on average. Consequently, 250 meters will be taken for the width, while 1750 m^2 for the cross-sectional area of the diversion canal.

5.2.2 Hydraulic Calculation

FIGURE 5-2 TYPICAL SECTION OF DIVERSION CANAL
(Sta. 3 + 746)



1) Zone I:

$A_1 = \frac{1}{2}(80.0 + 155.0) \times 5.0 + 155.0 \times 5.60 = 1{,}455.50\ m^2$
$P_1 = 80.0 + 37.83 \times 2 = 155.66\ m$
$R_1 = 1455.50/155.66 = 9.351\ m$
$N_1 = 0.025$
$I_1 = 1/32{,}000$
$V_1 = 1/0.025 \times 9.351^{2/3} \times (1/32{,}000)^{1/2} = 0.99\ m/s$
$Q_1 = 1.455.50 \times 0.99 = 1.441\ m^3/s$

2) Zone II:

$A_2 = \frac{1}{2}(10.0 + 47.5) \times 5.0 \times 2 + 47.5 \times 0.60 \times 2 = 344.50\ m^2$
$P_2 = (10.0 + 37.83) \times 2 = 95.66\ m$
$R_2 = 344.50/95.66 = 3.601\ m$
$N_2 = 0.025$
$I_2 = 1/32{,}000$
$V_2 = 1/0.025 \times 3.601^{2/3} \times (1/32{,}000)^{1/2} = 0.53\ m/s$
$Q_2 = 344.50 \times 0.53 = 183\ m^3/s$

3) Zone III:

$A_3 = \frac{1}{2}(10.0 + 11.2) \times 0.60 \times 2 = 12.72\ m^2$
$P_3 = (10.0 + 1.34) \times 2 = 22.68\ m$
$R_3 = 12.72/22.68 = 0.561\ m$
$N_3 = 0.030$

$I_3 = 1/32{,}000$
$V_3 = 1/0.030 \times 0.561^{2/3} \times (1/32{,}000)^{1/2} = 0.13$ m/s
$Q_3 = 12.72 \times 0.13 = 2\ m^3/s$

Therefore, the total section area and the total discharge are as follows;

Total Section Area; $A = 1455.5 + 344.5 + 12.7 = 1{,}813\ m^2 > 1{,}750\ m^2$
Total Discharge; $Q = 1{,}441 + 183 + 2 = 1{,}676\ m^3/s > 1{,}600\ m^3/s$

5.3 Slope Protection Works

5.3.1 Slope by Excavation

Excavation for the diversion canal 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, cohesiveness of 0.1 to 0.5 kgf/cm^2.

As a result of a stability analysis of the excavated slope, the slope will be 1 : 7.5 and 10 m-wide berms shall be provided at every 5 meter height of the excavation works so that the stability required can be ensured.

For reference, the existing river bank has a slope of 1 : 3.0 to 8.0 but, since the flood flow velocity is as small as 0.5 m/s, there has been no bank scouring caused from river flow. Under such circumstances, there has been no slope protection work planned for the excavated slope of the diversion canal.

5.3.2 Embankment Slope

The embankment height of dike to be constructed along the diversion canal is as low as about 2.0 meters, and the embankment is affected by river flow only in flooding. The embankment slope will be 1 : 2.0 with vegetation cover as slope protection considering the flow velocity is as low as 0.5 m/s or below.

CHAPTER 6. BASIC DESIGN OF DIVERSION DAM

6.1 Location of the Diversion Dam

The location of the proposed diversion dam was selected, in the Feasibility Study, at a site about 76 kilometers upstream from the Bang Pakong River estuary. The location of the diversion dam is proposed at the point of Sta. 3 + 764.45 of the diversion canal for the following reasons.

1) The diversion dam will probably be located upstream of the diversion canal so as to lessen the effect of the overflow at the gates to the lower part of the Bang Pakong River.

2) About 150 meters in the length of the diversion canal upstream from the diversion dam will be constructed in a straight line to smooth the plane figure of the transition portion of the canal.

6.2 Elevation of Gate Sill and Other Major Parts of the Structures

6.2.1 Gate Sill Elevation

The gate sill elevation is designed to ensure the smooth rundown of flood discharge. The gate sill elevation will be EL (-) 8.20 meters considering that the elevation of the diversion canal is EL (-) 8.18 meters at the diversion dam site.

6.2.2 Crest Elevation of the Gate

1) Crest Elevation of the Gate

The crest elevation of the gate is designed according to the high water level (H.W.L.) in the sea and wave height.

High Water Level : H.W.L 1.30 m
Wave Height : By S.M.B. method
(Sverdrup - Munk - Bretschneidr)

$$Hw = 0.00086 \cdot V^{1.1} \cdot F^{0.95}$$

where, Hw : Wave height (m)
V : Wind velocity 30m/s (Max. 55 knots)
F : Fetch 340 m (double the length of the diversion dam)

$$Hw = 0.00086 \times 30^{1.1} \times 340^{0.45} = 0.50 \text{ m}$$

The crest elevation of the gate can be obtained as follow:

H.W.L 1.30 m + 0.50 m = EL 1.80 m

2) Crest Elevation of the Lower Leaf Gate

Since the water level of the reservoir can be controlled by only the upper leaf gate, the crest elevation of the lower leaf gate will be decided at the same elevation as the minimum operating level of the reservoir by Min. O.L (-) 1.30 meters.

6. 2. 3 Pier Height

The crest elevation of the piers will be determined according to the following equation.

Crest Elevation = Maximum water level + Freeboard ① + Gate height + Freeboard ② + Crest thickness

where, Maximum water level : Max. W.L. 2.40 m

Freeboard ① : Distance between maximum water level and bottom of gate sill hoisted up (1.50 m)

Gate Height : EL 1.80 - EL (-) 8.20 = 10.00m

Freeboard ② : Distance between gate crest and bottom of crest plate, when the gate includes such structures, as spoiler, sieve, stopping hook, etc. and freeboard of hoist is 2.00 m.

Crest thickness : 1.50 m

Pier crest elevation $=$ Max. W.L. 2.40 + 1.50 + 10.00 + 2.00 + 1.50
$=$ EL 17.40 m

6. 2. 4 Retaining Wall Crest Elevation

The crest elevation of the retaining wall can be calculated as follows.

Crest Elevation $=$ High Water Level (H.W.L 1.30 m)
+ Wave Height (0.50 m) = EL 1.80 m

6. 3 Determination of the Gate Span

6. 3. 1 Cross Section for Flood Discharge

The mean cross-sectional area of the Bang Pakong river is A. = 1750 m^2, and the total length of the diversion dam is required to be more than 165 meters so that the cross-sectional area of the river can be secured at more than 1750 m^2 at the point of the diversion dam site.

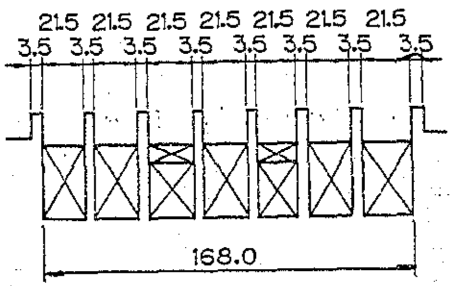
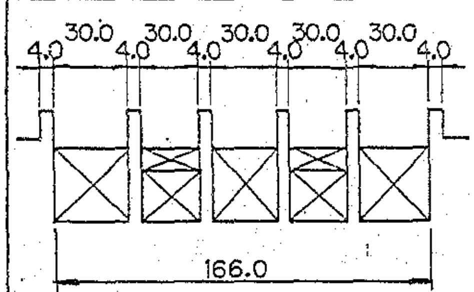
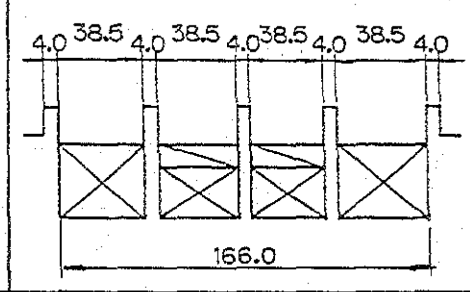
Total Dam Length $=$ Cross-Sectional Area/Water Depth
$=$ 1750/10.60 = 165 m

6. 3. 2 Determination of Gate Span

The gate span length will be determined taking into consideration the magnitude of the design flood discharge, the gate manufacturing technique, and economic factors.

In general, the span length of a diversion dam for a river with a design flood discharge of 1,600 m^3/s is more than 20 meters. As a result of an alternative study on three different span lengths of 20, 30, and 40 meters respectively, the span of 30 meters with 5 gates is recommended in view of the manufacturing technique and economic factors as shown in Table 6-1 on Comparison of Gate Span.

Table 6-1 Comparison of Gate Span

Item	Case-A : 20 m scheme	Case-B : 30 m scheme	Case-C : 40 m scheme
Rough Sketch			
Gate Dimensions	Flood Gate : 21.5 m × 10 m 5 sets Regulating G. : 21.5 m × 10 m × 2 sets	F. G. ; 30.0 m × 10 m × 3 sets R. G. ; 30.0 m × 10 m × 2 sets	F. G. ; 38.5 m × 10 m × 2 sets R. G. ; 38.5 m × 10 m × 2 sets
Stability	Span ratio to leaf height : 1/2.2 > 1/15, stable in structure.	Span ratio to leaf height: 1/3 > 1/15, stable in structure	Span ratio to leaf height: 1/3.9 > 1/15, stable in structure
Operation and Maintenance	Easier in O and M works with hoisting motor capacity of 30 kw per one gate	Normal O and M works with hoisting motor capacity of 44 kw per gate	More disadvantageous in O and M works with hoisting motor capacity of 60 kw per one gate.
Economically	Flood Gate : 464.0 M.฿ Regulating G. : 220.0 M.฿ Pier : 57.3 M.฿ Total : 741.5 M.฿	Flood Gate : 365.4 M.฿ Regulating G. : 312.0 M.฿ Pier : 46.1 M.฿ Total : 723.5 M.฿	Flood Gate : 390.0 M.฿ Regulating G. : 488.6 M.฿ Pier : 38.4 M.฿ Total : 917.0 M.฿
Overall Appraisal	○	◎	△

6.4 Piers

The structure of the piers should not in any way obstruct gate operation. They should be stable, and if possible reduce problems with the flow of flood discharge.

6.4.1 Pier Length

The pier length at the upper part of the diversion dam is designed to be 9.0 meters in the direction of the thalweg allowing for space for gate construction works and the stability of the piers. The lower part of the piers will have a round shape at both ends of the upper and downstream sides. The total length will be 19.0 meters including the upper length and the length reserved for the construction of the O/M bridge.

6.4.2 Pier Thickness

The pier thickness will be calculated by the following empirical equation with the pier height and gate span length as parameters, which was derived from the data of piers constructed in Japan.

$$t_p = 0.12\,(D_p + 0.2\,B_i) \pm 0.25$$

where, t_p : Pier thickness (m)

D_p : Pier height (EL 16.90 - EL (-) 9.10 = 26.00 m)

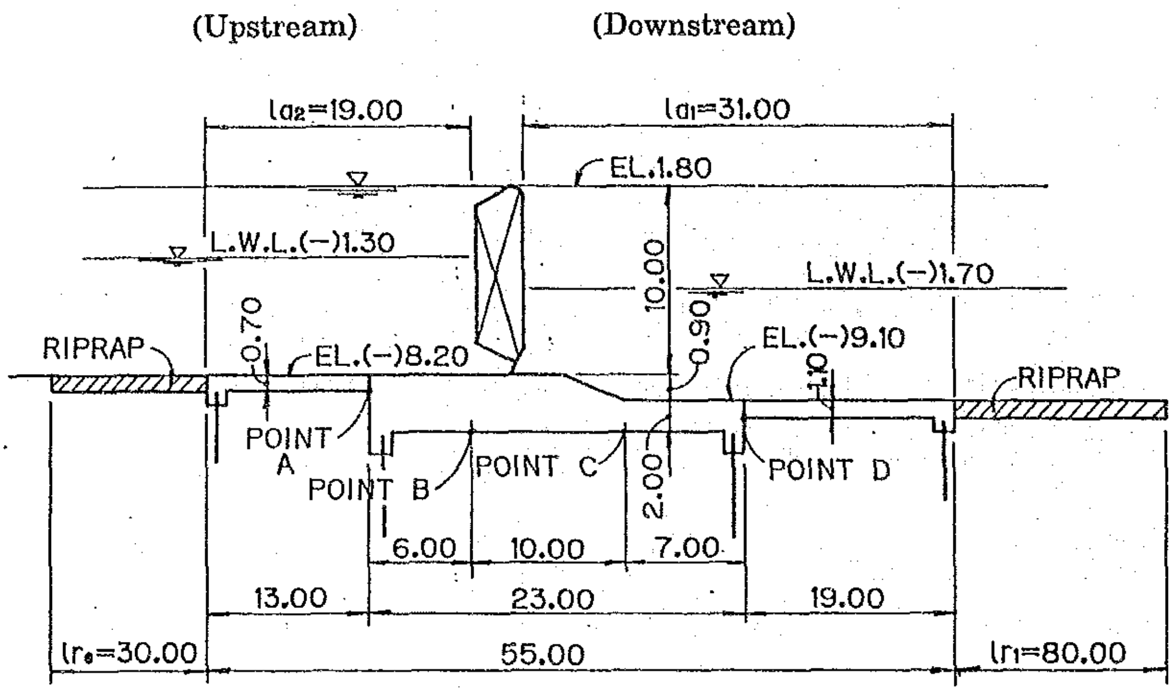
B_i : Span length of gate (30.0 m)

$$t_p = 0.12\,(26.50 + 0.2 \times 30.0) \pm 0.25$$
$$= 3.65 \text{ to } 4.10 \text{ m}$$

The pier thickness is determined to be 4.0 meters taking into account the guide rail depth of the gate leaf and pier stability.

6.5 Apron and Riprap

The apron and riprap is structured to protect the river bed both upstream and downstream of the gates from scouring.



6.5.1 Downstream

1) Apron Length

The apron length can be obtained by the following equation.

$$l_{a1} = 0.9\,C\,\sqrt{D_1}$$

where, l_{a1} : Length of downstream apron (m)

C : Bligh's coefficient 18 (fine particle sand or sedimentary sand)

D_1 : Height from min. tide level to gate crest
(EL. 1.80 - Min. W.L. (-) 1.70 = 3.50 m)

$$l_{a1} = 0.9 \times 18 \times \sqrt{3.50} = 30.3\ \text{m} < 31.0\ \text{m}$$

2) Creep Length

Sufficient length of creep must be secured in the foundation and back surface of the retaining wall so that the occurrence of piping caused by the creeping that results from the water level differences upstream of the gate and downstream of the gate can be prevented. The minimum creep length to be ensured should be taken as the larger value of the two methods which can be obtained by the following equations respectively.

a) Bligh's Method :

$L_{B1} \geqq C \cdot \Delta H_1$

where, L_{B1} : Creep length measured along the foundation bottom (m)

C : Bligh's coefficient (18)

ΔH_1 : Water level differences upstream of the gate and downstream of the gate (H.W.L. 1.80 - L.W.L (-) 1.70 = 3.50 m)

Therefore; $L_{B1} \geqq 18 \times 3.50 = 63.0$ m

At the end of the apron, 3.0 meter-long steel pile shall be driven for prevention of scouring and expansion of the foundation. And the design creep length is:

$$\text{Design creep length } L'_{B1} = 55.0 + 3.0 \times 2 \times 2 + 0.7 + 2.2 + 0.9 + 1.1$$
$$= 71.9 \text{ m} > L_{B1} = 63.0 \text{ m}$$

b) Lane's Method

$L_{L1} \geqq C' \Delta H_1$

where, L_{L1} : Weighted creeping length (m)

C' : Lane's coefficient 8.5 (fine particle sand or sedimentary sand)

ΔH_1 : Water level differences upstream and downstream (3.50 m)

Therefore : $L_{L1} \geqq 8.5 \times 3.50 = 29.8$ m

Design creep length $L'_{L1} = 1/3 \times 55.0 + 3.0 \times 2 \times 2 + 0.7 + 2.2 + 0.9 + 1.1$

$= 35.2\ \text{m} > L_{L1} = 29.8\ \text{m}$

3) Apron Thickness

The apron thickness can be obtained from the following equation with a minimum thickness of 0.50 m.

$$t_1 \geqq \frac{4}{3} \times \frac{\Delta H_1 - H_{f1}}{\gamma - 1}$$

where,

t_1	:	Apron thickness (m)
ΔH_1	:	Water level difference between up and downstream (3.50 m)
H_f	:	Head loss of water percolation up to the check point (m)
γ	:	Specific weight of apron (2.2)

a) Point C

Creep length $l_c = 29.0 + 3.0 \times 2 + 0.7 + 2.2 = 37.9\ \text{m}$

$$H_{fc} = \frac{lc}{L_1} \times \Delta H_1 = \frac{37.9}{71.9} \times 3.50 = 1.84\ \text{m}$$

$$t_c = \frac{4}{3} \times \frac{3.50 - 1.84}{2.2 - 1} = 1.84 \fallingdotseq 2.0\ \text{m}$$

b) Point D

Creep length $l_d = 36.0 + 3.0 \times 2 \times 2 + 0.7 + 2.2 + 0.9 = 51.8\ \text{m}$

$$H_{fd} = \frac{51.8}{71.9} \times 3.50 = 2.52\ \text{m}$$

$$t_d = \frac{4}{3} \times \frac{3.50 - 2.52}{2.2 - 1} = 1.09\ \text{m} \fallingdotseq 1.1\ \text{m}$$

4) Riprap Length

The riprap will be provided at the places where there may be some fear of local river bed scouring to be caused by diversion dam construction, taking into account the conditions of the river bed and discharges found around the gate. And the riprap will be structured to withstand the river flow and to dissipate any discharge.

· **Bligh's Method;**

$l_{r1} = L_{R1} - l_{a1}$

$L_{R1} = 0.67\,C\,\sqrt{\Delta H_1 \cdot q_1} \cdot f_1$

where,

l_{r1}	:	Riprap length (m)
L_{R1}	:	Total length of protection structures including apron length (l_{a1}) and riprap length (l_{r1}). (m)
C	:	Bligh's coefficient (18)
ΔH_1	:	Height from the water level downstream to the gate crest. (EL 1.80 - EL (-) 1.70 = 3.50 m)
q_1	:	Design flood discharge per unit width 1600/30 × 5 = 10.67 $m^3/s/m$
f_1	:	Safety factor (1.5 for movable weird)

There fore : $L_{R1} = 0.67 \times 18 \times \sqrt{3.50 \times 10.67} \times 1.5 = 110.5$ m

$l_{r1} = 110.5 - 31.0 = 79.5\text{ m} \doteqdot 80\text{ m}$

6. 5. 2 Upstream

1) Apron Length

The apron length can be obtained by the following equation.

$l_{a2} = 0.6\,C\,\sqrt{D_2}$

where, D_2 ; Height from Min. operation level at upstream to gate crest.
EL 1.80 - Min. O.L (-) 1.30 = 3.10 m

$l_{a2} = 0.6 \times 18 \times \sqrt{3.10} = 19.0$ m

2) Creep Length

a) Bligh's Method

$L_{B2} \geqq C \cdot \triangle H_2 = 18 \times 3.10 \quad = 55.8\,m$

Design creep length $L'_{B2} \quad = 55.0 + 3.0 \times 2 \times 2 + 1.1 + 0.9 + 2.2 + 0.7$

$= 71.9\,m > L_{B2} = 55.8\,m$

b) Lane's Method

$L_{L2} \geqq C' \cdot \triangle H_2 = 8.5 \times 3.10 \quad = 26.4\,m$

Design creep length $L'_{L2} \quad = 1/3 \times 55.0 + 3.0 \times 2 \times 2 + 1.1 + 0.9 + 2.2 + 0.7$

$= 35.2\,m > L_{L2} = 26.4\,m$

3) Apron Thickness

$$t_2 \geqq \frac{4}{3} \times \frac{\triangle H_2 - H_{f2}}{\gamma - 1}$$

a) Point A

Creep length $l_a = 42.0 + 3.0 \times 2 \times 2 + 1.1 + 0.9 + 2.2 = 58.2\,m$

$$H_{fa} = \frac{58.2}{71.9} \times 3.10 = 2.51\,m$$

$$t_a = \frac{4}{3} \times \frac{3.10 - 2.51}{2.2 - 1} = 0.66\,m \doteqdot 0.70\,m$$

b) Point B

Creep length $l_b = 36.0 + 3.0 \times 2 + 1.1 + 0.9 = 44.0\,m$

$$H_{fb} = \frac{44.0}{71.9} \times 3.10 = 1.90\,m$$

$$t_b = \frac{4}{3} \times \frac{3.10 - 1.90}{2.2 - 1} = 1.33\,m < 2.9\,m$$

4) Riprap Length

• Bligh's method :

$l_{r2} = L_{R2} - l_{a2}$

$L_{R2} = 0.67 \cdot C \sqrt{\Delta H_2 \cdot q_2} \cdot f_2$

where, C : Bligh's coefficient 18

ΔH_2 : EL 1.80 - Min O.L. (-) 1.30 = 3.10 m

q_2 : $CB\Delta H_2^{3/2} / L = 2.0 \times 30.0 \times 3.10^{3/2} / 166.0 = 1.97\ m^3/s/m$

f_2 : Safety factor, 1.5 in case of gate.

$L_{R2} = 0.67 \times 18 \times \sqrt{3.10 \times 1.97} \times 1.5 = 44.7\ m$

$l_{r2} = 44.7 - 19.0 = 25.7\ m \doteqdot 30.0\ m$

6. 6 Retaining Wall

6. 6. 1 Height of Retaining Wall

The foundation of the retaining wall will be on an elevation 2.0 meters below the apron surface, and an elevation of EL (-) 10.2 m upstream and EL (-) 11.1 downstream.

Since the crest elevation of the wall is EL 1.8 meters, the wall height is 12.0 meters on the upstream side while 12.9 meters on the downstream side, respectively.

6. 6. 2 Wall Type

There are four types of walls: i) Gravity type, ii) Inverted T-shape type, iii) L-shape type, and iv) Counterfort wall type. And the counterfort wall type is adopted for the Project for the following reasons.

1) This type requires a lower volume of concrete than the other types.
2) An economical height for walls of this type ranges from 7.0 meters to 13 meters.

6.7 Gate

6.7.1 Gate Type

1) Flood Gate

There are three types of gates, the Girder type, Trass type and Shell type. The Shell type is adopted for the following reasons.

a) The Girder type is successfully applicable only when the span is less than 15 meters.

b) The Shell type and Trass type can be applied to those gates whose span ranges from 15 to 80 meters.

c) Due to the fact that flood gates receive hydraulic pressure from both sides, up and downstream, the Girder and Trass types cannot be adopted in this Project.

2) Regulating Gate

Double leaf gates and Flap gates can be adopted as regulating gates to serve for water intake for irrigation, domestic/industrial water supply, and to prevent sea water intrusion. The following table shows the comparison of the above two types of gates with their merits and demerits.

Item	Double Leaf Gate		Shell Type with Flap	
	Normal	Reverse	Normal	Reverse
- Height of Gate	Hc > 1 / 12 L		Hc > 1 / 3 H or 3.0 m	
- Down pull of Lower Leaf	Small	Large	Large	Small
- Effect of Tidal Wave	Small	Medium	Large	Small
- Adherence of Sea Animals	Inside of Upper Leaf	Inside of Lower Leaf	Flap and Lower Leaf	Flap and Lower Leaf
- Stability of Water Flow	Smooth	Smooth	Unsteady	Unsteady

Double leaf gates are high in stability as well as reliability in operation, although inferior to flap gates in economy. Accordingly, double leaf gates will be employed in the Project; especially, the double leaf wheel type gates with hydraulic advantages. For reference, two sets of flood gates will be provided to secure stable operation.

6.7.2 Specifications for Gate Design

1) Design Conditions of the Gate

a) Design Water Level

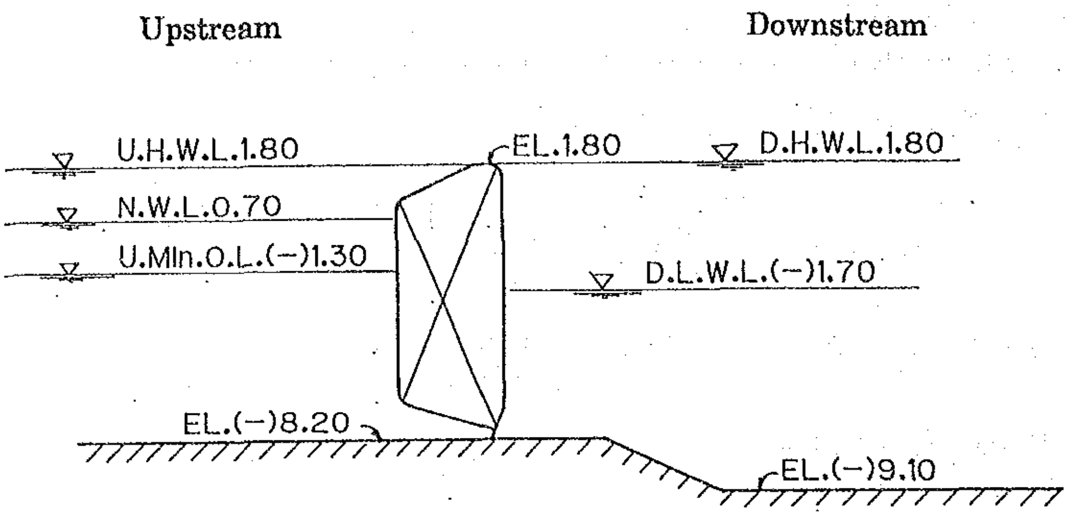
The design water level for the gates is as follows:

- The upstream high water level (U.H.W.L) is designed at EL. 1.80 meters of the crest elevation of the gate.
- The upstream low water level (U.L.W.L) is designed at EL (-) 1.30 meters of the minimum operation level of the reservoir.
- The downstream high water level (D.H.W.L) is EL 1.80 meters of the crest elevation of the gate.
- The downstream low water level (D.L.W.L) is EL (-) 1.70 meters of the lowest low water level in the sea.

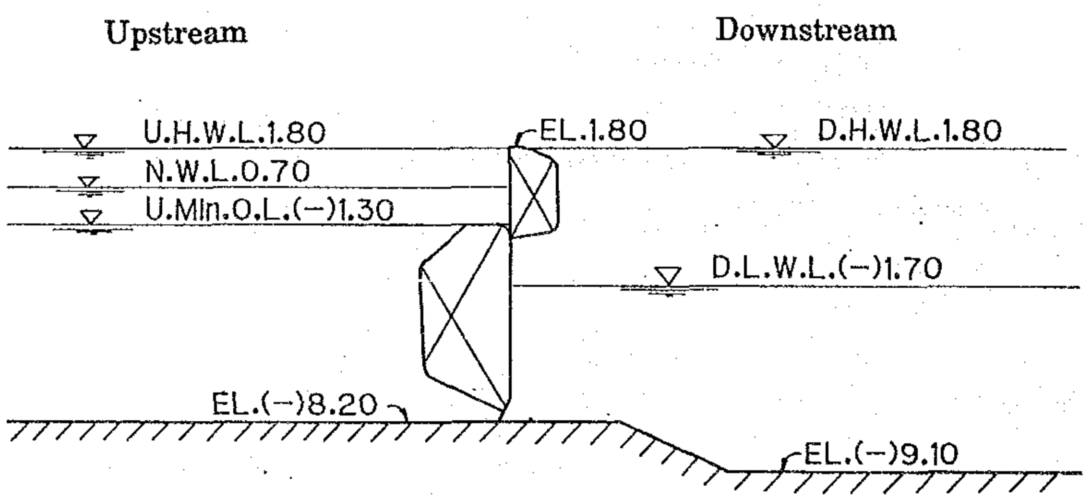
b) Sediment Depth

The sediment at upstream of the gate is not acute because of the large reservoir.

c) Flood Gate



d) Regulating Gate



e) Water Level Conditions for Designing

	Upstream	Downstream	Water level difference
Case I :	H.W.L 1.80 m	L.W.L (-) 1.70 m	3.50 m
Case II :	L.W.L. (-) 1.30 m	H.W.L 1.80 m	3.10 m

2) Major Items of Gate Specifications

Item	Flood Gate	Regulating Gate
Gate type	Single Shell Roller Gate	Double Shell Roller Gate
Clear span	30.0 m	30.0 m
Upper leaf height	-	3.1 m
Lower leaf height	10.0 m	6.9 m
Quantity	3 sets	2 sets
Gate sill elevation	EL. (-) 8.20 m	EL (-) 9.10 m
Gate crest elev. of upper	-	EL 1.80 m
Gate crest elev. of lower	EL. 1.80 m	EL (-) 1.30 m
Sealing Type	3 - sides rubber seal	3 - sides rubber seal
Hoist	2 - motors, 2 - drams wire rope winch	2 - motors, 2 - drams wire rope winch
Operating speed	0.3 m/min	0.3 m/min
Lifting height	12.1 m (EL. 3.90 m)	12.1 m (EL. 3.90 m)
Operation method	Local and remote	Local and remote
Design water level difference	Upstream : 3.50 m Downstream : 3.10 m	Upstream : 3.50 m Downstream : 3.10 m
Operating water level	H.W.L. 1.80 m	H.W.L 1.80 m

6.8 Hoist House

6.8.1 Size of Hoist House

The gate hoist house is designed to protect the hoisting and electric equipment and facilities from exposure to sun shine and rain water which inhibit smooth operation of the gates. The size of the hoist house is decided according to the types of hoisting equipment on the piers and operation methods.

The hoist house for the proposed gates will be 12.5 meters in thalweg direction, 10.0 meters in diversion dam center direction, and 4.5 meters in height.

6. 8. 2 Structure of Hoist House

The proposed hoist house is designed to be a reinforced concrete building. It will be necessary to provide a chain block for inspection and repairs on the ceiling which will be made of reinforced concrete. The walls will be reinforced concrete block.

Since the hoist house building will have the appearance of a colourless rectangular box, it is deemed better that the total building is designed to meet the architectural design standards of Thailand.

6. 9 Operation and Maintenance (O and M) Bridge

The O and M bridge connected with the hoist house is a bridge to be provided for rendering O and M services for the facilities.

6. 9. 1 Bridge Width

The O and M bridge will have sufficient width to carry out successful O and M works as well as emergency operation of the gates.

The proposed bridge will have a width of 5.0 meters so that a 10-ton truck crane can pass through for installation of stop-log.

6. 9. 2 Span

The O and M bridge will have a clear span length of 30.0 meters and a subbeam span of 33.15 m $\times$ 5 spans taking into account the gate span length.

6. 9. 3 Bridge Type

As O and M bridges, in general, steel bridges, I-beam bridges with prestressed concrete and hollow box type bridges with prestressed concrete can be used. For the proposed O and M bridge, the hollow box type bridge with prestressed concrete is recommended for the following reasons and in reference to Table 6-2 Comparison of O & M Bridge Types.

1) The hollow box type bridges are more economical in many respects than the other types of bridges, although the weight is heavier.

2) O and M works such as painting, etc. can be rendered more easily than with other types.

3) Since the beam height of this type is lower than that of the others, it is easier to provide embankments, access roads and so forth.

6. 9. 4 Beam Seat Elevation

The beam seat elevation is determined as follows with consideration for gate operation safe clearance, wave height, freeboard.

Beam seat elevation = Maximum water level + Freeboard
= Max. W.L. 2.40 m + 1.50 m
= EL 3.90 m

Table 6-2 Comparison of O & M Bridge Types

Scheme	Type of Superstructure	Dimensions of Super.	Economy	Specific Features	Overall Appraisal
A	Prestressed Concrete Bridge Girder I - Section	- Span length : 33.15 m - Beam length : 33.95 m - Span : 5	143%	- Dead load reaction is largest - O and M works are not required - Economically, initial cost is largest - No need for bent method and easy in construction.	△
B	Prestressed Concrete Hollow Box Girder	- Span length : 33.15 m - Beam length : 33.95 m - Span : 5	100%	- O and M works are not required - Most economical - No need for bent method and easy in construction	◎
C	Simple Composite Steel Girder	- Span length : 33.30 m - Beam length : 33.90 m - Span : 5	103%	- Dead load reaction is least - Bent method is required and high technique is necessary in construction	○

CHAPTER 7. BASIC DESIGN OF CLOSURE DAM

7.1 General

The closure dam is planned for construction across the Bang Pakong river at about 300 m in the east from Ban Chuknua situated on the left bank of the same river route.

Topographically, around the dam site the river course is about 250 m wide, the lowest elevation in the river bed is 7.1 m below mean sea level and there is a very gentle slope at both banks with a slope angle of about 15° on the left bank and about 10° on the right bank respectively.

The result of bore-holed drilling at BCD - 2 which was sunk to 20.3 m in depth on the right bank of the closure dam indicates that the dam foundation is a typical weak ground composed of alluvial deposits with fine particles of silt and clay.

The bearing capacity of the foundation can be classified into 3 layers from the N-value as described below:

- The upper layer consists of very weak alluvial clay and silt with about 1 in N-value, and distributes until reaching the elevation around -12.0 m.
- The intermediate layer is located approximately between -12.0 m to -16.0 m in elevation with about 15 in N-value.
- The base layer distributes below about -16.0 m in elevation with more than 40 in N-value and can be regarded mostly as a firm foundation.

The typical section of the closure dam is decided taking into account the topographical and geological conditions mentioned above as well as the following items concerning the basic dimensions, zoning, embankment materials and foundation treatment.

7. 2 Basic Dimensions and Zoning of Closure Dam

The closure dam has a 3.9 m crest elevation without taking into account the camber of 0.3 m high and the 12.0 m crest width. These values are determined in consideration of the water level in the closing river and utilization of the crest as a road after completion of the dam.

For the deepest part of the dam foundation, the excavation should be done to a depth of -12.0 m with removal of the upper weak layer, setting the dam height at 15.9 m.

The closure dam which is symmetrical with regard to the axis of the dam, mostly consists of an earth fill zone with a rock zone at both upstream and downstream toes of the dam in order to prevent lateral moving-out of the embanked materials from the earth fill zone. The riprap zone is planned at both upstream and downstream slopes of the earth fill zone in order to prevent the materials from moving and being washed away by the wave action.

The slant in both upstream and downstream slopes is determined by the stability analysis of the preliminary study with 1 vertical to 5.0 horizontal for the earth fill slope and 1 on 2.0 for the rock fill slope respectively.

7. 3 Embankment Materials & Stability Analysis

The embankment materials for the earth fill zone will be hauled from the borrow area which can be expected to be located about 20 km to the east of the dam site. These materials are expected to have a value of about 25° in the angle of internal friction.

However, the exact location of the borrow area and the properties of the materials for the earth fill zone should be examined in detail through field investigations and soil tests in the laboratory during the Phase II Study.

It is planned to purchase excavated sandstone for rock zone materials from a quarry site near Chonburi City which is located about 60 km from the dam site. This sandstone is hard and sound and resistant to weathering.

The excavated materials from the diversion channel, which consist of fine particles of silt and clay are not suitable for the earth fill zone considering the underwater embankment, consolidation settlement and stability of the closure dam.

In the case that those excavated materials were utilized as the embankment materials for the earth fill zone with the removal of the upper layer in the foundation, an estimate of the settlement and the safety factor is obtained through preliminary consolidation and stability analyses, the results of which are as follows;

1) Result of Consolidation Analysis

Total settlement of dam body is estimated to be about 160 cm during and after construction of the closure dam. The obtained consolidation time corresponding to the degree of consolidation for the dam is shown in the following table under 2 drainage conditions.

	Degree of consolidation	
	50%	80%
Single drainage condition	about 23 years	about 68 years
Double drainage condition	about 4 years	about 12 years

2) Result of Stability Analysis

The stability analysis on the closure dam with slopes 1 vertical to 5.0 horizontal for the earth fill zone and 1 to 2.0 for the rock zone is carried out by the slip circle method under the following conditions and the results are indicated in the table below.

Case	Condition of analysis	Factor of Safety	Remarks
Case 1	Constant water level H.H.W.L of EL. 2.10 m	1.253 > 1.20	H.H.W.L; height high water level,
Case 2	Constant water level L.L.W.L of EL -1.70 m	1.132 > 1.20	L.L.W.L; lowest
Case 3	Drawdown of water level H.H.W.L. to L.L.W.L.	0.926 < 1.10	low water level

Judging from the results of the consolidation and stability analyses on the closure dam, it seems that excavated materials even if obtained about 8 m below the ground surface at the diversion canal are unsuitable for the earth fill embankment due to the extreme amount of consolidation settlement and consolidation time as well as the stability of the dam body.

7.4 Foundation Treatment

In case the the dam foundation is located at the upper layer which consists of very weak alluvial deposits having about 1 in N-value, the analysis for stability of the dam is executed by the slip circle method against sliding failure taking the following conditions into account and the results are as below;

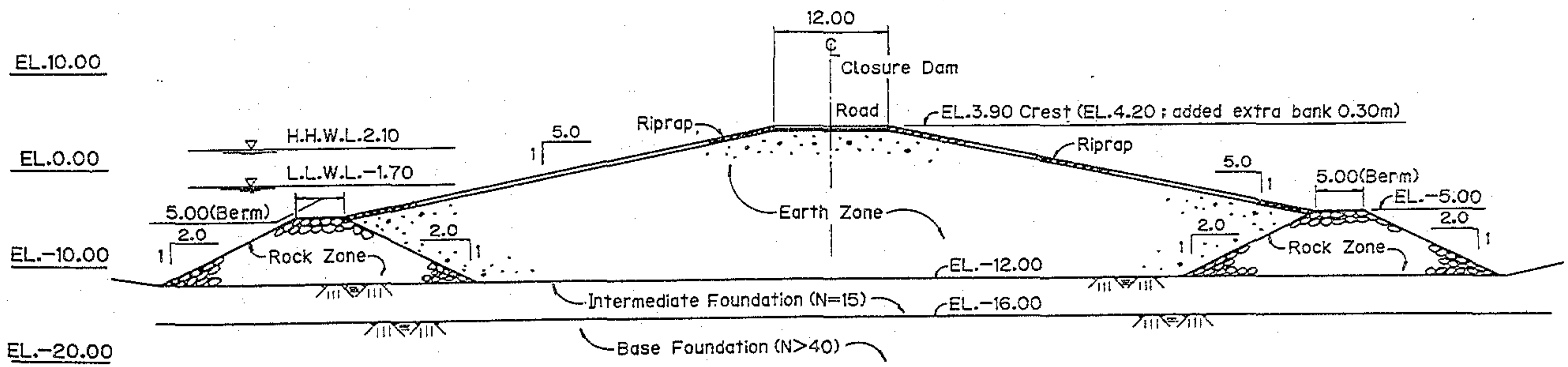
Case	Condition of analysis	Factor of Safety	Remarks
Case 1	Constant water level H.H.W.L of EL. 2.10 m	1.249 > 1.20	H.H.W.L; height high water level, L.L.W.L; lowest low water level
Case 2	Constant water level L.L.W.L of EL -1.70 m	1.098 > 1.20	
Case 3	Drawdown of water level H.H.W.L. to L.L.W.L.	1.034 < 1.10	

In the above table the safety factor suggests there would be a possible to probable occurrence of the sliding failure through the deep slip circle. Therefore, it is proposed that the foundation of the closure dam should be located at the intermediate layer which can be expected to be about 15 in N-value with removal of the upper layer.

If the soil improvement method was employed to improve the properties of the materials for the upper layer of the whole foundation of the closure dam instead of the removal method mentioned above, there would inevitably be a considerable increase in construction time and cost. The soil improvement method is therefore, rejected as an unfavorable method for the whole foundation of the closure dam. However, considering the distribution of weak layers in both banks of the river, it would be impossible to remove those layers from the foundation of the dam at both abutments, therefore, one of the following methods should be partially employed: the sand compaction pile method or an other appropriate method that will accelerate the consolidation and increase the strength constants, avoiding consolidation settlement and stability problems in the foundation.

Based on the above considerations, a typical section to be adopted for the closure dam is shown in Figure 7-1.

FIGURE 7-1 TYPICAL SECTION OF CLOSURE DAM



CHAPTER 8. BASIC DESIGN OF ROAD AND ROAD BRIDGE

8.1 Road

8.1.1 Route Alignment

RID plans to build a new road beginning from an existing road which branches from the trunk road 304 on the left bank of the Bang Pakong River and reaches to Chuknua village on the river bank. The new road will link up with an existing road on the right bank, after crossing the river and the diversion canal. The existing branch road on the left bank has been paved with laterite materials with a total width of 9.0 meters and a favourable plane alignment.

If the proposed branch road was to be constructed as an extension of the existing branch road on the left bank, Chuknua village houses would have to be removed as obstacles of access road construction. Furthermore, the plane alignment as a whole would not be favourable due to the fact that the road would have to be a considerable distance to the west of the road bridge.

Therefore, the proposed road will be constructed about 300 meters east of the end of the existing road on the left bank avoiding the residential area of Chuknua village.

The crossing point of the road over the diversion canal is decided 200 meters downstream from the diversion dam so as to lessen the effect of the river discharge through the diversion dam.

1) Radius of Curve

The following table shows the relationship between the design speed (V), and the curve radius (R), length (L), width (ΔW), and the superelevation of curve (i).

TABLE 8-1 DIMENSIONS OF ROAD CURVE

Design Speed (km/hr)	Radius of Curve		Length of Curve (m)	width of Curve (m)	Superelevation of Curve (%)
	Min (m)	Stand. (m)			
60	150	200	100	0.25 (0)	9 (8)
80	280	400	140	0	9 (7)

Note : The figures in ( ) are for the standard radius.

As learned from the above table, the curve radius for a road with a design speed of 60 km/hr is decided at more than 200 meters and that of 80 km/hr at more than 400 meters.

Since the distance from the starting point to the point IP_1 is as short as 152.6 meters with the intersecting angle of 65°55' 43", the necessary curve radius is 200 meters. Consequently, the design speed for the portion of 300 meters from Sta. 0 + 000 to Sta. 0 + 300 will be 60 km/hr and the others 80 km/hr. The curve radius for the IP_2 and IP_3 will be designed at 500 meters.

2) Transition Portion

Curve widening in the proposed road is not required because the related curve radius is more than 200 meters, and there is no need to provide transition portions for the proposed road.

8. 1. 2 Longitudinal Section

1) Longitudinal Slope

The road design criteria set by the Highway Department of Thailand indicate that the longitudinal slope should be four (4) percent maximum for roads that are classified Class 4 with a design speed in the range of 60 to 80 km/hr and with flat gentle topography. The proposed road, providing no drains, should be flat with minimum longitudinal slope.

2) Design Crest Elevation of the Road

The design crest elevation of the road at each point should be as follows.

a) The design crest elevation of the starting point (Sta. 0) will be taken at 2.20 meters, the same elevation as that of the existing road crest.

b) The design crest elevation from the point at the river closure (Sta. 0 + 970) to the point (Sta. 1 + 220) will be EL 4.20 meters to meet the closure dam crest elevation including the amount of camber.

c) The design road crest elevation for the portion from the point of road bridge (Sta. 2 + 640) to the point (Sta. 2 + 960) will be EL 5.20 meters to meet the road bridge elevation.

d) Since the proposed road must be constructed to cross the Bang Pakong river via the high water channel, the road crest elevation will possibly be lowered to ensure the smooth discharge of flood water. And the design road elevation will be EL 1.80 meters taking into consideration the average elevation of EL 1.50 meters, stripping thickness of 0.3 meters, pavement thickness of 0.6 meters, and the highest water level recorded in the past 10 years of 1.86 meters.

8.1.3 Road Cross Section

1) Road Width

The vehicle road shall have two lanes with width of 6.0 meters (one lane 3.0 meters × 2), and the shoulder width is designed by 1.5 meters for one side to make 3.0 meters with both sides. Consequently, the total road width is 9.0 meters.

2) Cross Slope

According to the design criteria of the Highway Department of Thailand, the cross slope of the proposed road should be 3.5 percent.

3) Embankment Slope

The slope of the road embankment will be 1 to 2.0 in accordance with the design criteria of the Highway Department.

8. 1. 4 Pavement Works

1) Design Conditions

- Daily traffic capacity (large size vehicles)
  : A - traffic (100 - 250/day)
- Design CBR : 3%
- Minimum thickness of each paved layer

Surface course	:	5.0 cm
Base course	:	10.0 cm
Sub-base course	:	10.0 cm

2) Pavement thickness required (T_A)

$$T_A = \frac{3.84 \cdot N^{0.16}}{C.B.R.^{0.3}}$$

where, T_A : Thickness required (cm)

N : Number of wheels of vehicles passing for 10 years. (150,000 wheels/dir.)

C.B.R. : C.B.R. for road grade (3%)

T_A is obtained as about 19 cm.

3) Design Pavement Thickness

$$T'_A = a_1 \cdot T_1 + a_2 . T_2 + a_3 . T_3$$

where, T'_A : Design pavement thickness (cm)

T_1, T_2 & T_3 : Thickness of surface course, base course and sub-base course, respectively (cm)

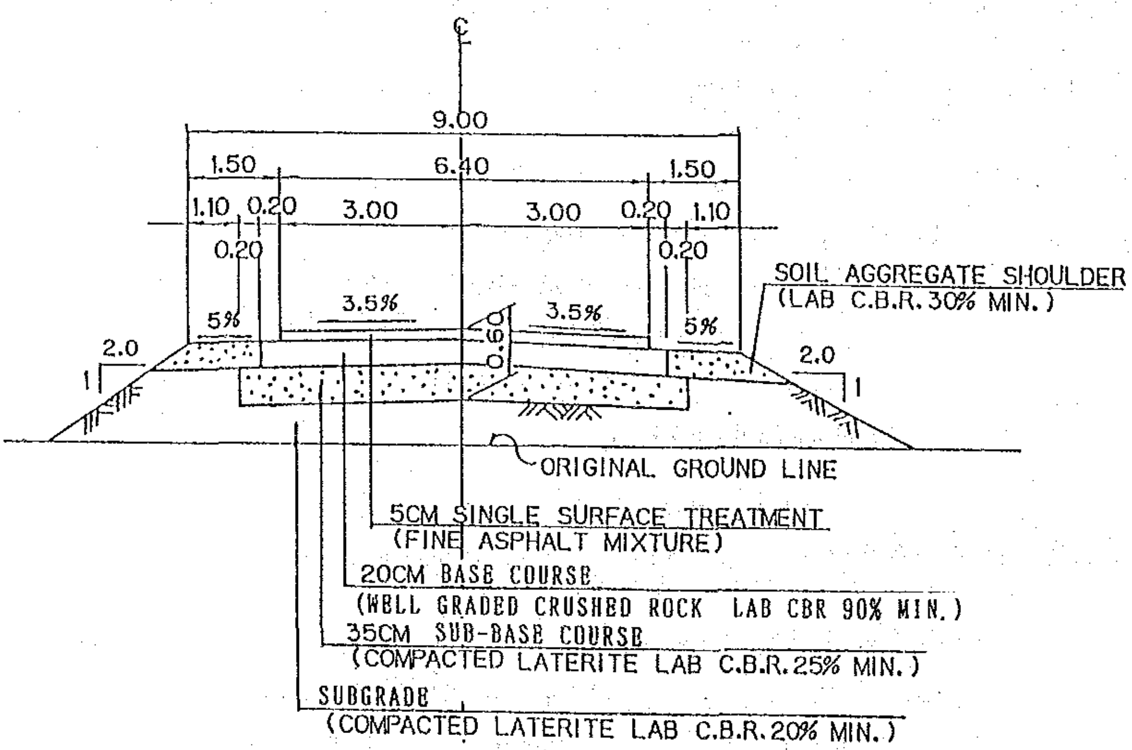
a_1, a_2 & a_3 : Equivalent conversion coefficient of each course by 1.00, 0.35 and 0.20, respectively.

TABLE 8-2 COMPARISON OF DESIGN PAVEMENT THICKNESS OF COURSES

Scheme	Surface C. 2000฿/m³	Base Course 450 ฿/m³	Sub-base C. 265฿/m³	Con. Thick. T'A (cm)	Total Thick. H (cm)	Paving Cost (฿/m²)
Case - A	5 cm	10 cm	55	19.5	60	
	100 ฿/m²	45 ฿/m²	146			291
Case - B	5	15	45	19.3	65	
	100	68	119			287
Case - C	5	20	35	19.0	60	
	100	90	93			283
Case - D	5	25	30	19.8	60	
	100	113	80			293

From the above table, the pavement construction of the proposed road will have a thickness of 5.0 cm for the surface course, 20 cm for the base course, 35 cm for the sub-base course, and a total thickness of 60 cm.

FIGURE 8-1 TYPICAL CROSS SECTION OF ROAD

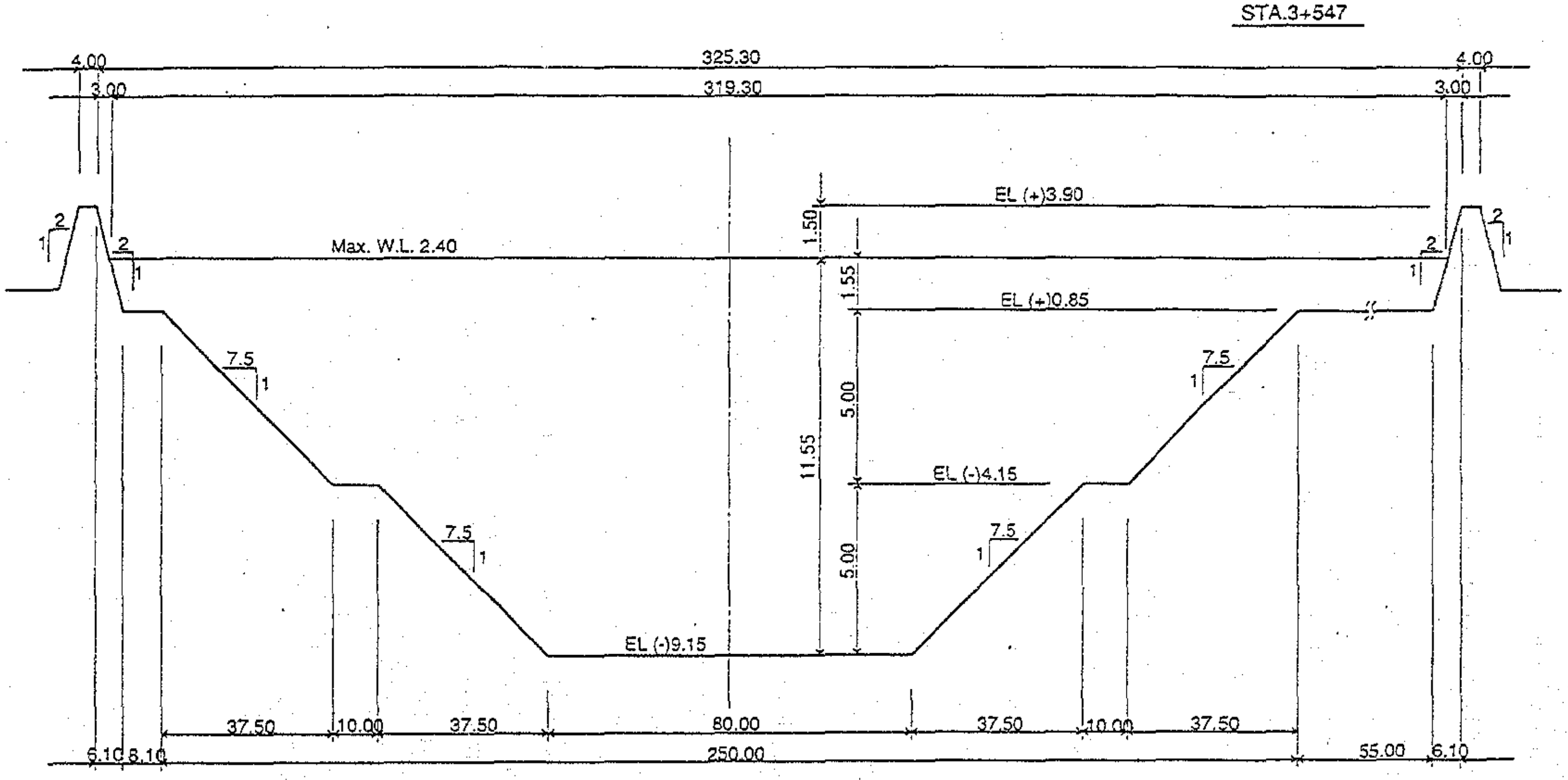


8. 2 Road Bridge

8. 2. 1 Basic Design Conditions

1)	Road class	: Class 4 by design criteria of the Highway Department in Thailand
2)	Design speed	: 80 km/hr
3)	Design traffic	: 300 to 1000 vehicles/day
4)	Bridge class	: Class 1 (TL - 20)
5)	Bridge length	: 251.65 meters
6)	Bridge construction	: For vehicles 2 lanes × 3.0 m = 6.0 m For shoulder 2 × 1.50 m = 3.0 m Total width 9.0 m
7)	Route alignment	: straight
8)	Inclined angle	: 90 degrees
9)	Pavement	: Asphalt pavement with 6.0 cm thickness for vehicles lanes.
10)	Cross slope	: 3.5 % for vehicles lane
11)	Longitudinal slope	: Level
12)	Special load	: Lighting facilities
13)	River planning	
	River name	: Bang Pakong river
	Location of bridge	: Sta. 3 + 547
	Design flood discharge	: Q = 1600 m^3/s
	Maximum water level	Max. W. L. 2.40 m
	Design crest dike elevation	: EL. 3.90 m
	Design river bed elevation	: EL. (-) 9.15 m
	Design river bed slope	: I = 1/4,000
	Design cross section	: as Figure 8-2

FIGURE 8-2 DESIGN CROSS SECTION OF DIVERSION CANAL

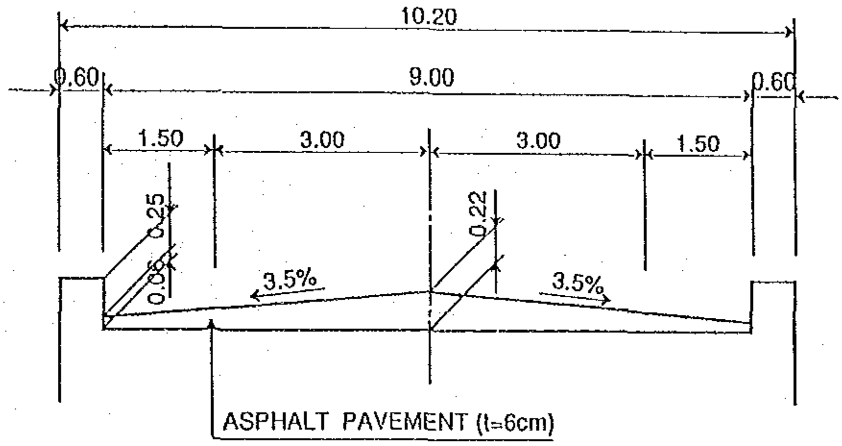


8.2.2 Alignment Plan

1) Cross Alignment

The cross alignment of the road bridge will be the same as that of the road as shown in Figure 8-3.

FIGURE 8-3 TYPICAL CROSS SECTION OF ROAD BRIDGE



2) Longitudinal Alignment

The longitudinal alignment of the proposed road bridge will be level. The road bridge surface elevation will be EL. 5.20 meters allowing for the fact that the beam seat elevation should be kept at more than EL 3.90 meters, which is the dike elevation.

Bridge surface elevation = Design dike elevation + Beam height + Pavement thickness
= EL. 3.90 m + 1.00 m + 0.22 m
= EL. 5.12 m ≒ EL. 5.20 m

8. 2. 3 Bridge Length

The bridge length will be determined taking into account the fact that the river cross section must have the ability to cope with the flow of design flood discharge of $Q = 1{,}600 m^3/s$ at the maximum water level (Max. W. L. 2.40 m) in making the abutments front surface (EL. 0.85 m) contact the berm shoulder. The following equation should be applied to obtain the bridge length.

$$
\begin{aligned}
\text{Bridge length} &= \text{River width} + 2 \times \text{Beam seat width} \\
&= 250\,\text{m} + 2 \times 0.80\,\text{m} \\
&= 251.60\,\text{m} \doteq 251.65\,\text{m}
\end{aligned}
$$

8. 2. 4 Type of Superstructure and Span

The type of superstructure and span length should be determined taking into consideration economy, ease of construction and of O and M works, along with a comprehensive and comparative study of the following seven (7) types and their respective span lengths.

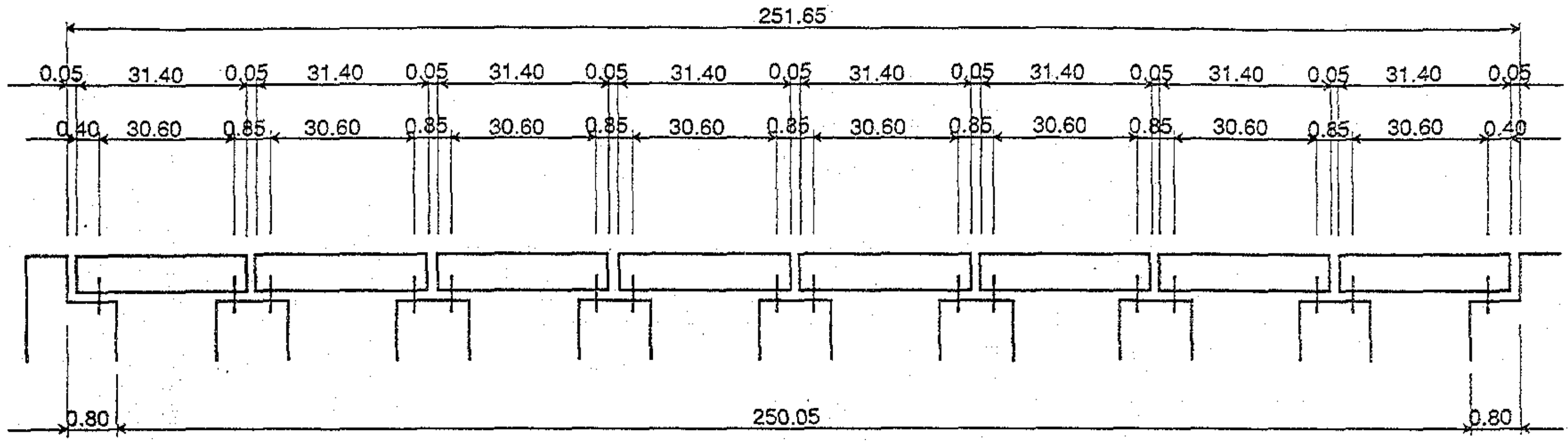
Superstructure	Span	Name of Case
P.C. I - Section Girder	8 spans	A - 1
	9 spans	A - 2
P.C. Hollow Box Girder	7 spans	B - 1
	8 spans	B - 2
	9 spans	B - 3
Steel Simple Composite Girder	5 spans	C - 1
	6 spans	C - 2

The most suitable type of superstructure and the related span length is determined to be Case B-2 (Hollow Box P.C. bridge : 30.6 m × 8 span) in view of the results of a comparative study on economy, merits and demerits of construction and O and M works as shown in Table 8-3.

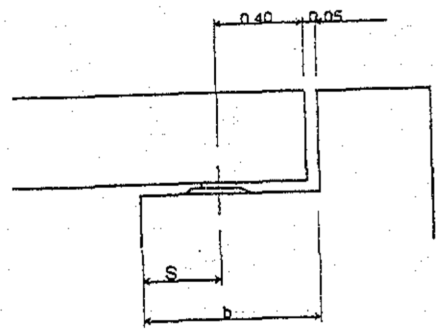
TABLE 8-3 COMPARISON OF SUPERSTRUCTURE TYPE AND SPAN

Scheme	Type of Superstructure	Span	Economy	Specific Features	Overall Appraisal
A - 1	Prestressed Concrete Bridge Girder I - Section	30.6m × 8	130%	- Dead load reaction is largest - O and M works are not required - Economically, initial cost is largest - No need for bent method and easy in construction	△
A - 2		27.2m × 9	133%		△
B - 1	Prestressed Concrete Hollow Box Girder	35.1m × 7	104%	- O and M works are not required - Most economical - No need for bent method and easy in construction	△
B - 2		30.6m × 8	100%		◎
B - 3		27.2m × 9	103%		○
C - 1	Simple Composite Steel Girder	49.7m × 5	110%	- Dead load reaction is least - Bent method is required and high technique is necessary in construction	△
C - 2		41.3m × 6	105%		△

FIGURE 8-4 ELEVATION OF ROAD BRIDGE



Elevation of Road Bridge



S = 20 + 0.5×ℓ
= 20 + 0.5×30.60
= 35.3 cm

b = 0.353 + 0.40 + 0.05 = 0.80 m

Detail of Beam Seat

8. 2. 5 Infrastructure

1) Abutment

The abutment type will be a common inverted T-shape type because of having a height of 6.00 meters with pile foundation.

2) Pier

The piers will be an ellipse balcony type made of reinforced concrete, taking into consideration the fact that they will be constructed beneath the river discharge.

8. 2. 6 Foundation Works

The foundation works will be a type of PC concrete pile foundation with a 600 mm dia. (9 - 21m) taking into account the fact that the bearing layer is a silty clay layer (N>40) at EL (-) 19.00m and the groundwater table is high because the bearing layer is based 10 meter lower than the design river bed.

CHAPTER 9. BASIC DESIGN OF PUMPING STATION

9.1 Site Selection of Pumping Station

Site selection of the proposed pumping station will be made so as to find the most advantageous site through comprehensive study of the following conditions.

i) Topographic Conditions

- The main irrigation canal between the pumping site and the area it serves should be as short as possible.

- The site should have limited inflow of sediment and drifting materials upstream from the diversion dam and closure dam, and continuously stable water intake by pumping should be ensured.

ii) Geological Conditions

- It must be possible to carry out foundation works safely and economically.

iii) Environmental Conditions

- Construction of the proposed pumping station should not interfere with its neighboring facilities, buildings, etc.

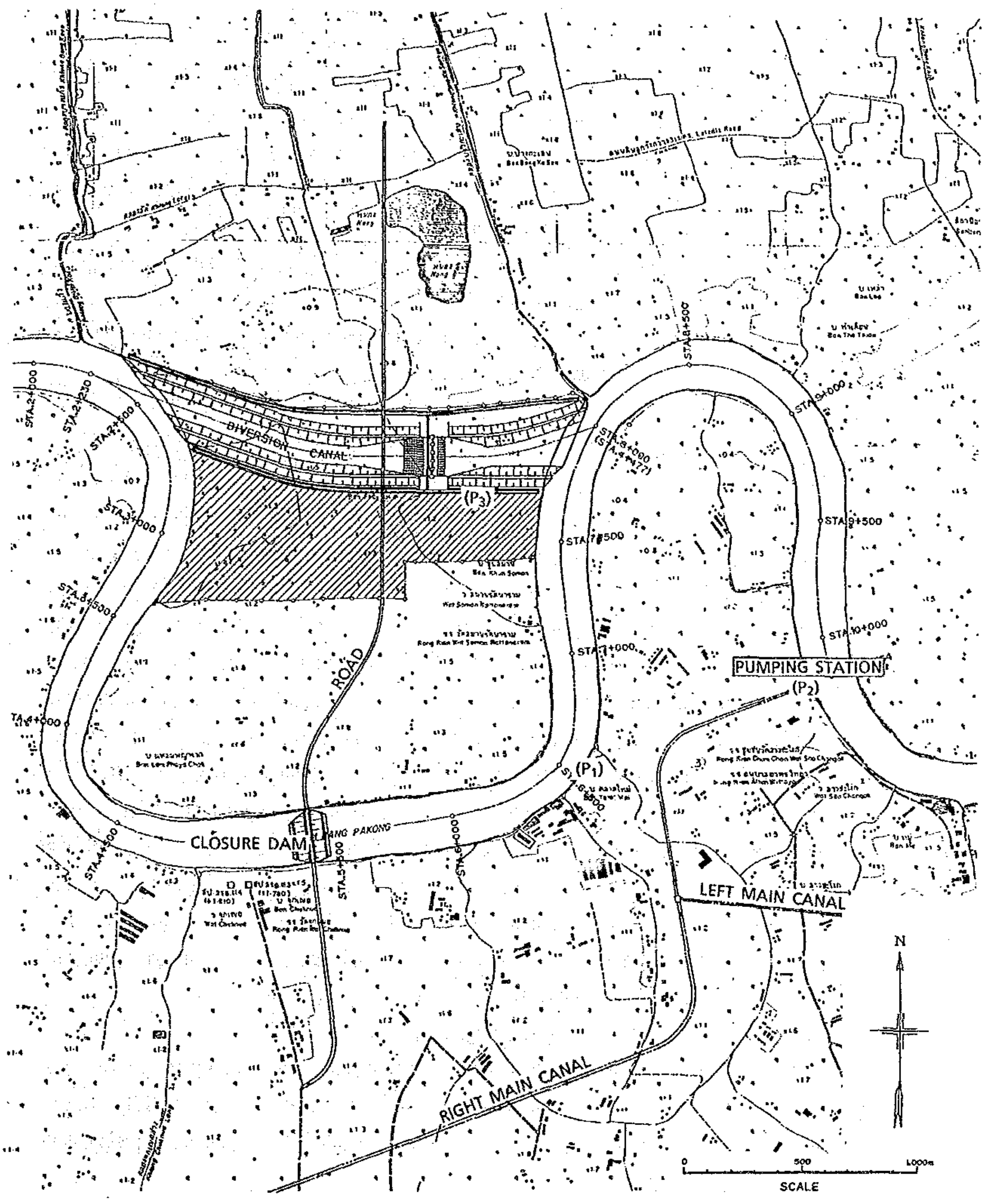
- Noise and vibration from the pumping operation must not become a public nuisance to the neighboring area.

iv) Others

- Land acquisition should be easy.
- Power supply for pump driver must be secured easily.
- Easy O/M works must be ensured.

As a result of the comparative study for the pumping site at the three locations (P1), (P2) and (P3) illustrated in Figure 9-1 considering the

FIGURE 9-1 LOCATION OF PUMPING STATION



above-mentioned conditions, site (P2) has been selected for the following reasons:

i) Site (P1) is located nearly in the middle of the left bank of the Bang Pakong River between the entrance of the diversion canal and the closure dam. After completion of the Project, the flow velocity of the river for this portion would be below 0.01m/s even with full pumping operation. Accordingly, the river water would remain almost motionless. Flushed sand and other organic materials would sediment in the upstream portion of the river together with suspended materials in the river to result in a decrease in the cross section of the river. And also, waste water from animal husbandry and kitchen services might pollute the river water resulting in poor water quality and thick growth of water weed. Smooth flushing in the wet season would be impeded resulting in great O/M costs.

   Site (P1), therefore, is considered unsuitable for the proposed pumping site.

ii) Site (P3) is located along the diversion canal upstream of the diversion dam. Disadvantages of this site were found although it is advantageous in the operation and maintenance works of the pumping station. The main irrigation canal of (P3) would have to be laid at a right angle to the Bang Pakong river flow. It is, therefore, considered unfavorable from the river control viewpoint because the canal embankment with a crest level of EL. 5.00 m, would become an obstacle for smooth discharge of flood water overflowing the river course. Additionally, the main canal would cross over the existing river course parallel with the closure dam which would increase the volume of construction and the total main canal length would be 1.0 km or more longer than that in the case of (P2). (P3), therefore, would be considerably less economical than (P2).

iii) Site (P2) is located about 2.3 km upstream from the entrance of the diversion canal on the left bank of the Bang Pakong River. The site is at a point where little sand sediment will occur upstream of the diversion canal entrance in the Bang Pakong River, and the water course runs near the left bank. Further more, rather easy linking of the main canals can be secured at this point. Although there is an elementary school and a junior high school about 300 m upstream from the (P2) site, little fear exists for noise pollution from the various works and operations. And selection of (P2) will not bring any trouble with river control works and will be more economical than (P3).

9. 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 excluding rainfall appears in the first decade during September in the wet season. The said requirements should be reasonably taken as the gross water requirements including 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 $Q=16.00m^3/s$ 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.

TABLE 9-1 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 For 20 yrs.	Seasonal max. gross water req'ment with rainfall 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

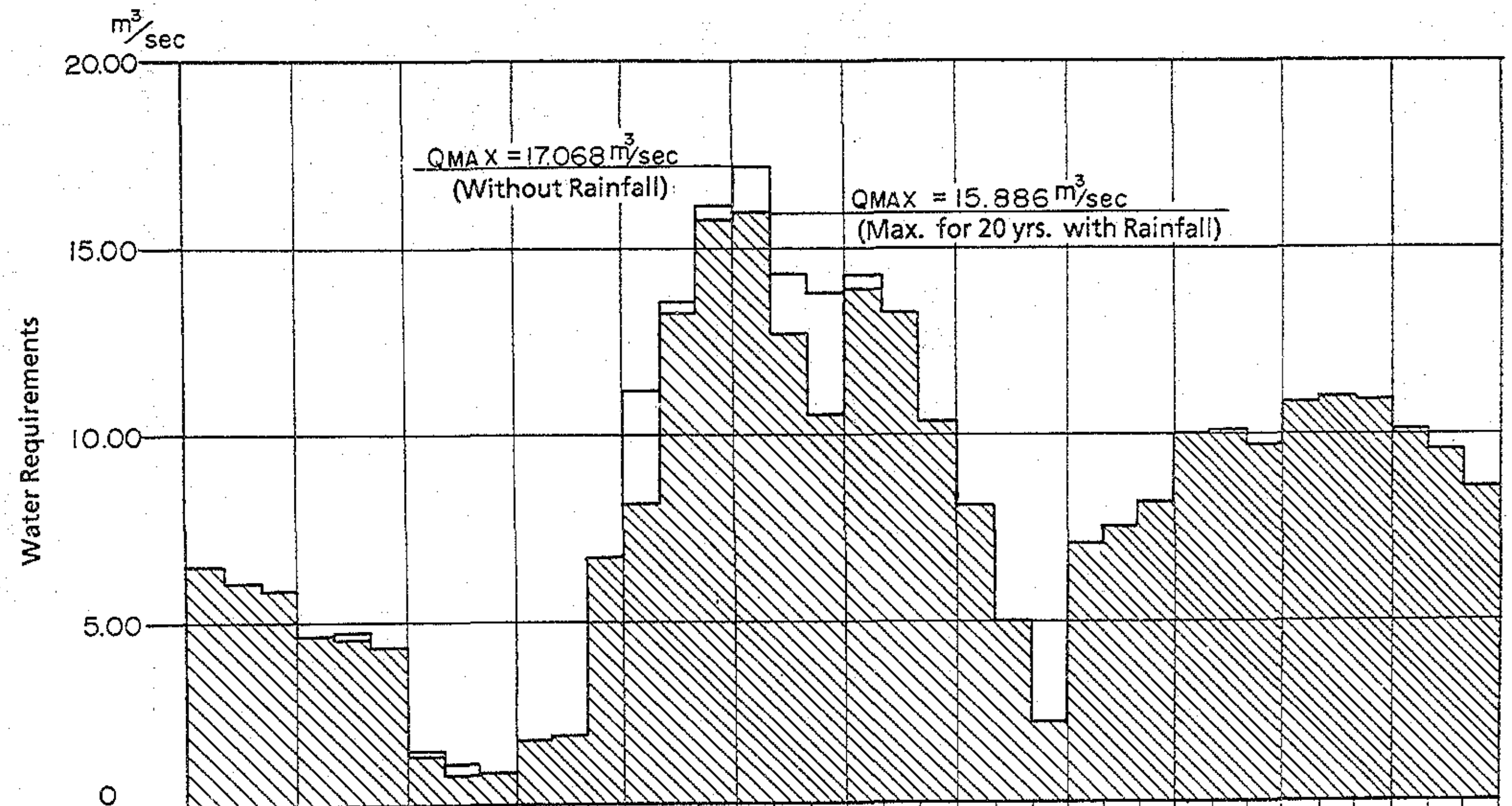
Note : The records for rainfall will cover the latest 10 to 20 years depending upon the rainfall records available for 1968 - 1987.

b) Water Level

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

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

FIGURE 9-2 SEASONAL WATER REQUIREMENTS

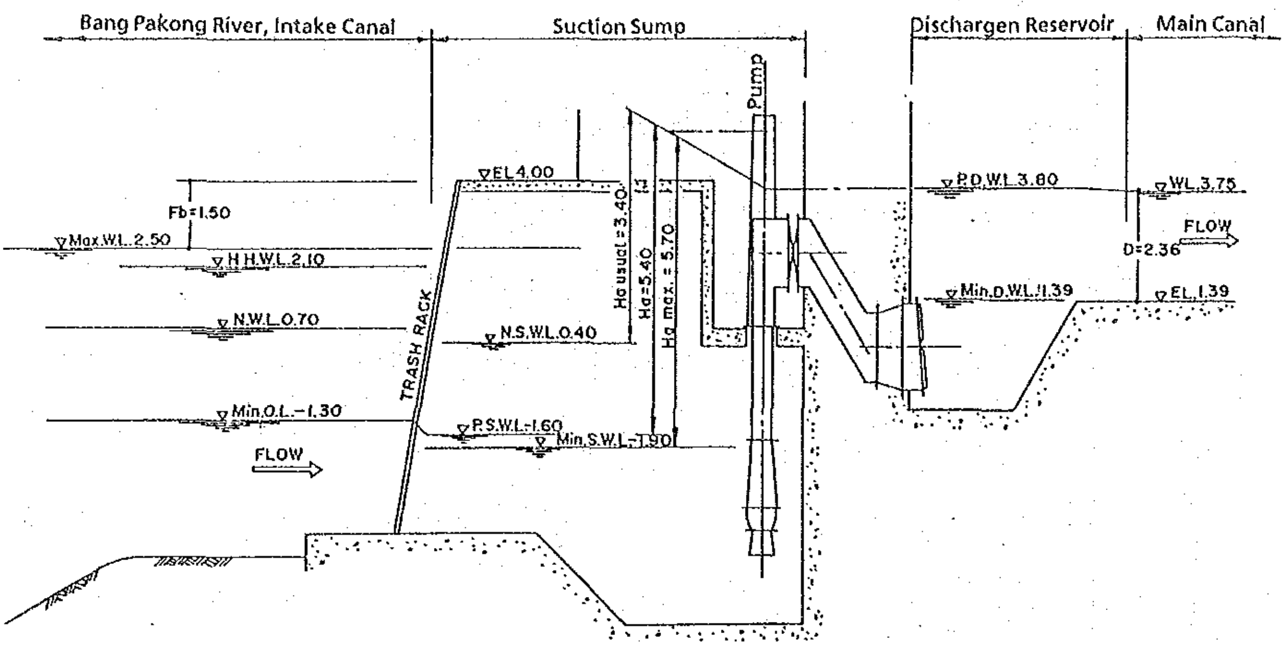


Month	4			5			6			7			8			9		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Q (m^3/sec) (Without Rainfall)	6.550	6.029	5.838	4.595	4.419	4.229	1.387	0.782	0.818	1.712	1.806	6.663	8.053	13.205	15.630	15.886	12.760	10.438
Q (m^3/sec) (With Rainfall)	6.550	6.029	5.838	4.595	4.640	4.229	1.407	1.007	0.827	1.747	1.806	6.663	11.171	13.584	16.067	17.068	14.312	13.797

Month	10			11			12			1			2			3		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Q (m^3/sec) (Without Rainfall)	13.871	13.243	10.366	8.091	4.963	2.104	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)	14.231	13.243	10.366	8.091	4.963	2.104	7.062	7.473	8.052	9.987	10.080	9.676	10.797	10.937	10.865	10.082	9.558	8.566

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

FIGURE 9-3 FIGURE ON RELATED WATER LEVEL



2) Number of Pump Units Required and Bore Diameter

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

i) To Meet the Fluctuating Water Requirements

It is desirable to meet the seasonal fluctuation of the irrigation water requirements with a variation in 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 Limit Problems

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

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 because of the large number of pumps to be installed, but the adaptability of the facilities to fluctuations of the water demand will be reduce 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

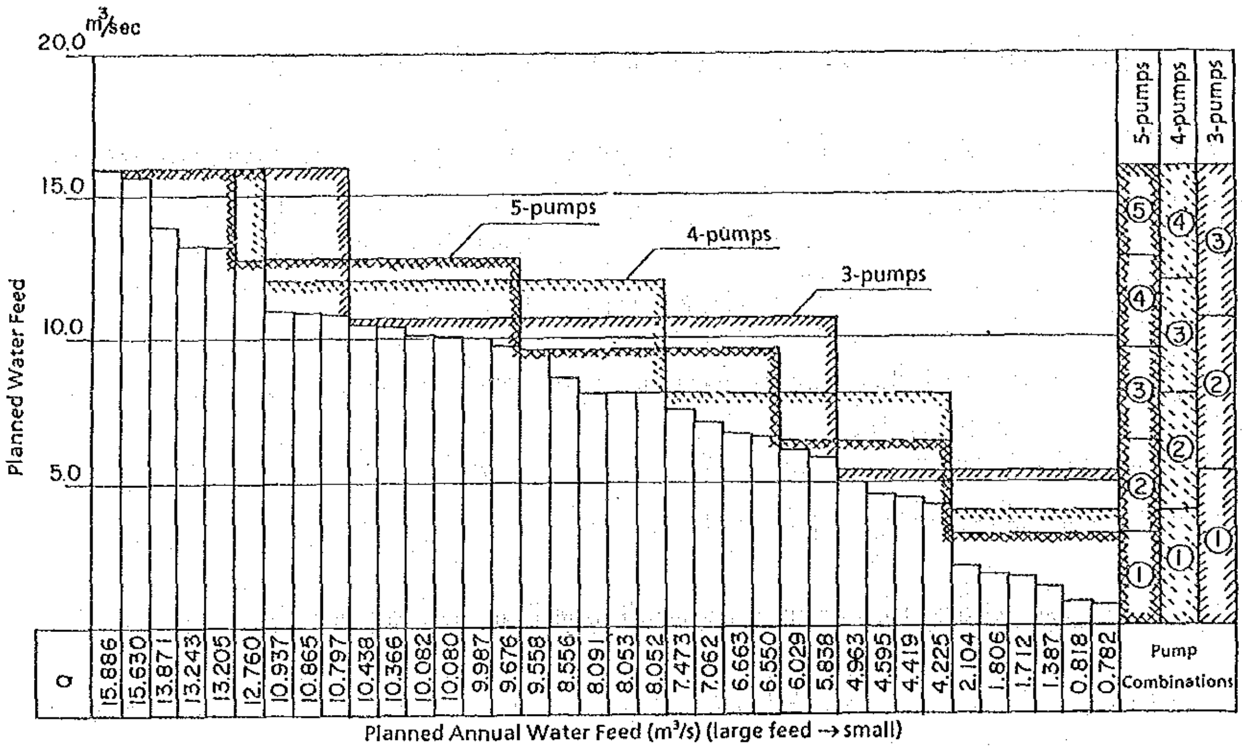
All the pump units to be adopted will have the same capacity in terms of interchangeability of the parts, operability, and easy operation and maintenance works. The following alternative study on three to five units will be made to consider operational flexibility in appropriately meeting fluctuating 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 diameters of the pumps are show as follows by number of units:

	No. of Pumps (units)	Discharge/unit (m^3/min)	Bore Dia, (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 9-4.

FIGURE 9-4 PROPOSED NO. OF PUMPS & PLANNED ANNUAL WATER FEED



Adaptability ratio (α) in each case is as follows so that the increasing number of units brings favourable 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 feed and economy including the initial costs and running costs for each case in the alternative study.

TABLE 9-2 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	a=75.9%	a=77.8%	a=81.4%
Total W. Feed for 20 yrs.	a=61.3%	a=67.1%	a=75.7%
Economic Evaluation	'000 ฿	'000 ฿	'000 ฿
Construction Cost	31,341	34,254	35,891
Pump Installation Cost	209,440	220,080	233,040
Running Cost 1/	163,514	154,206	151,406
Total	404,295	408,540	420,337
(Ratio)	(100%)	(101.0%)	(104.0%)

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 larger number of the pump units provided while, contrarily, more diseconomy including running costs would result from the larger 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) units 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 Bore Diameter

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

The design discharge of the pumps in the Project is 16.00 m^3/s by four (4) units, which come to 4.00 m^3/s per unit (240.00 m^3/min). Under such conditions, the pump bore diameter 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 9-3 PUMP TYPES AND TOTAL HEAD

Shaft Type / Model	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 rage.

4) Pump Head

The head at the design point for the pumps is as follows

Actual Head (Ha)	=	Planned Discharge Water Level
	-	Planned Suction Water Level
	=	P.D.W.L 3.80 - P.S.W.L (-)1.60 = 5.40 m
Total Head (H)	=	Actual Head (Ha)
	+	Loss Head of Discharge Pipe (hl)
	=	5.40 + 0.70 = 6.10 m (By Motor)
	=	5.40 + 0.80 = 6.20 m (By Engine)

9. 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 cut, 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 and/or the peripheral devices.

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 pump and the peripheral devices.

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

TABLE 9-4 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 9-5 ANNUAL OPERATION HOURS OF EACH PUMP

Pump	Max.	Min	Mean
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 9-6 ECONOMIC COMPARISON OF PRIME MOVERS IN COMBINATION

(Unit ; '000 B)

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	154,206	143,824
Total	434,536	431,825	408,540	412,158
(Ratio)	(106.4%)	(105.7%)	(100%)	(100.9%)

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

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) 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 ; Coefficient (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 ; Prime mover allowance coefficient (0.15 for motors and 0.20 for diesel engines)

i) Output of Motors

$$P_M = \frac{0.163 \times 1.0 \times 240 \times 6.1}{0.835 \times 0.96} \times (1 + 0.15) = 342.3 \doteqdot 350 \text{ KW}$$

ii) Output of Diesel Engine

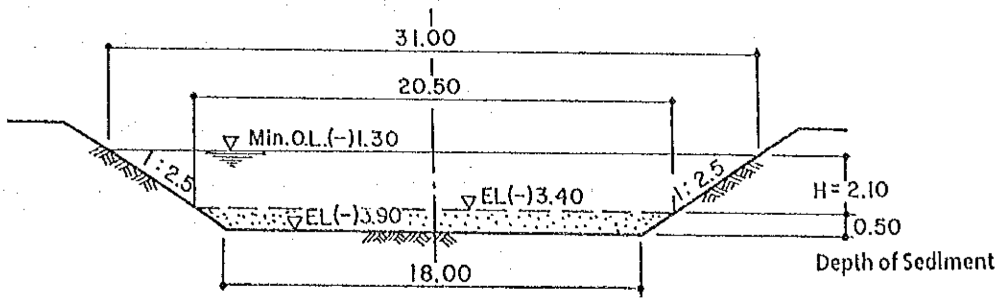
$$P_E = \frac{0.222 \times 1.0 \times 240 \times 6.2}{0.835 \times 0.96} \times (1 + 0.20) = 494.5 \doteqdot 500 \text{ Ps}$$

9.4 Design of Intake Canal and Intake

1) Intake Canal

The intake canal will be a facility to convey water taken from the Bang Pakong River to the intake of the pumping station and it will function as a sedimentation basin to protect the pumps. (Mean velocity ; 0.15 ~ 0.30 m/s, Flowing duration ; 30 ~ 60 sec)

FIGURE 9-5 CROSS SECTION OF INTAKE CANAL

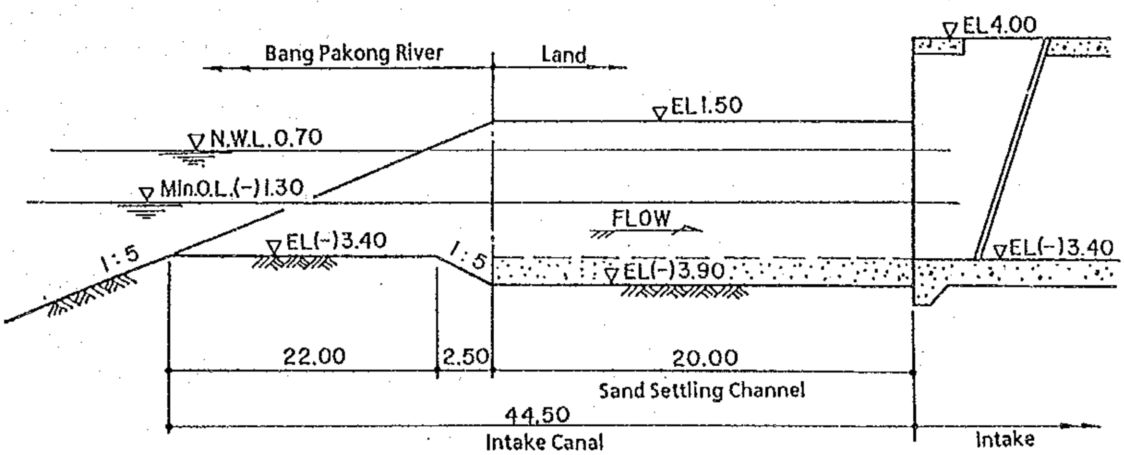


Flow area $A = 1/2 \times (20.50 + 31.00) \times 2.10 = 54.08 \text{ m}^2$

Mean velocity $V = 16.00 \text{ m}^3/\text{s} / 54.08 \text{ m}^2 = 0.296 \text{ m/s} \leqq 0.30 \text{ m/s}$

The length of the proposed sand settling channel will be 20.0 m so as to allow a flowing duration of about 60 seconds, and the intake canal will have the length shown below.

FIGURE 9-6 PROFILE OF INTAKE CANAL



The intake canal will have stone pitching to prevent the canal bed and banks from scouring by river discharge. There will also be a floating net system in the upstream section of the canal to prevent floating trashes into the intake structure.

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 that the inflow velocity can be less than 0.50 m/s upstream of the trashrack. Taking the water depth upstream of the trashrack to be 2.10 m, the inflow velocity is expected to be less than 0.50 m/s ($4.00/4.05 \times 2.10$ = 0.47 m/s). The sill elevation is taken at EL. (-)3.40 m, which is 2.10 m lower than the minimum operating level of (-)1.30 m.

b) Width of Top Slab

The top of the intake downstream from the trashrack will be covered with a slab of total width 6.0 m, including 2.0 m for trash cleaning works and 4.0 m for vehicles.

c) Effective Mesh Size of Trashrack

The trapped trash will be cleaned manually. The effective mesh size will be 50 mm taking into consideration the pump bore diameter 1,350 mm.

9.5 Design of Suction Sump

1) Suction Water level

The suction water level is determined as flows according to the basic design 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) Size of Suction Sump

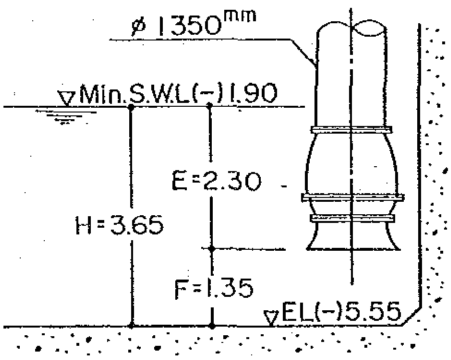
The shape and size of the suction sump will be determined so that the water level and flow can be stably maintained without sucking air into the suction pipes and causing whirling flow in the sump.

a) Water Depth in Suction Sump

Water depth in the sump is determined by the E, F dimensions and the depth required to submerge the suction pipes. For the pumps with a 1,350 mm dia., the necessary water depth (H) can be shown in the equation.

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

FIGURE 9-7 WATER DEPTH IN SUCTION SUMP

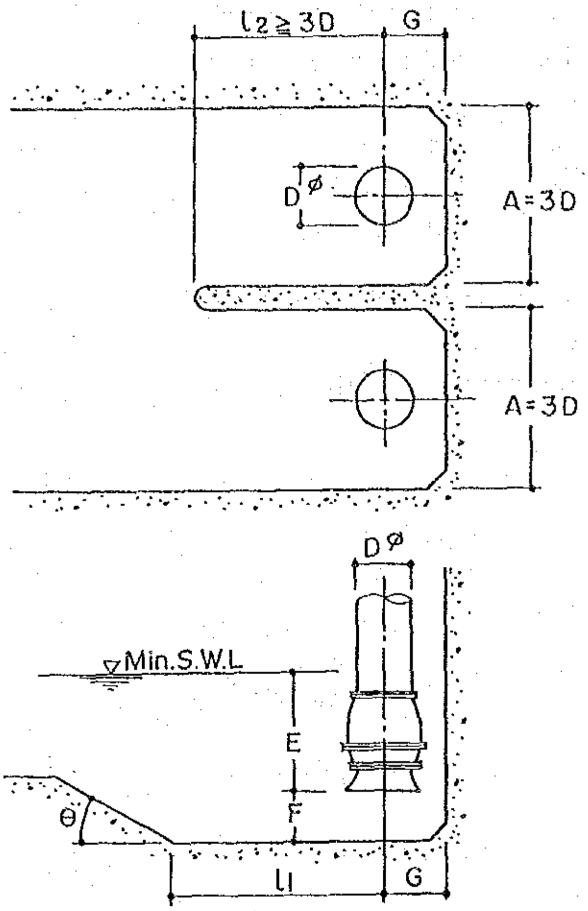


The sill elevation of the suction sump, therefore, will be decided at EL. (-)5.55 m.

b) Size of Suction Sump

The size of suction sump will be determined by the shape shown in Figure 9-8 and the number of sumps to be provided.

FIGURE 9-8 SHAPE OF SUCTION SUMP



In case θ = 30°, $\ell 1 \geqq 3D$
θ = 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 total in taking into consideration the aforesaid measurements and the scale of the proposed pump facilities. The space at the back of the sump will be used for the storage of cooling water.

c) Floor Elevation of Suction Sump

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

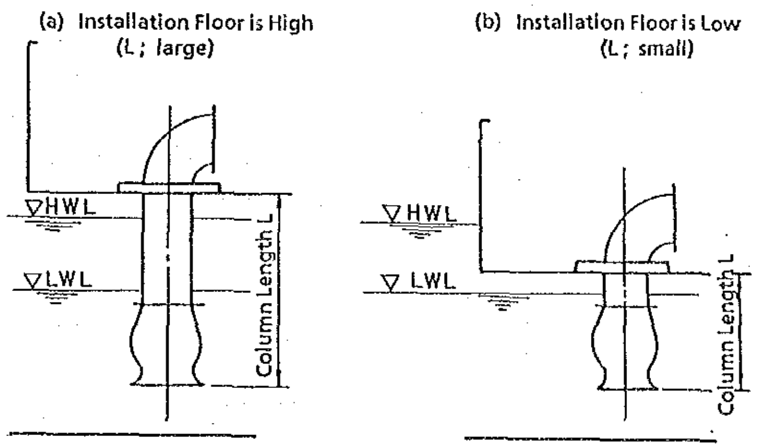
i) Elevation of Prime Mover Floor

The elevation of the prime mover floor will be sufficient to ensure that it is not submerged by any high-level flood water. With such a consideration, the said elevation will be more than EL. 4.00 m including 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 cost is high because of the need for a long column. However, it is low,though less costly, flood water protection must be carried out for the pump room.

FIGURE 9-9 PUMP COLUMN LENGTH

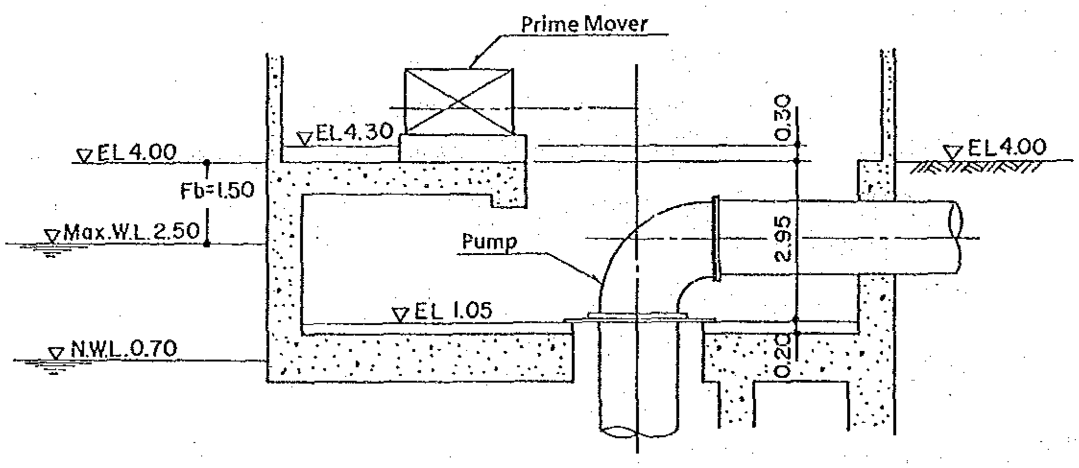


In this Project, the floor elevation will be EL. 1.05 m, slightly higher than the normal water level of the Bang Pakong Reservoir (N.W.L. 0.70 m), for the following reasons.

① The pump column can be shortened by 1.80 m, reducing the costs of both pump installation and construction of the pumping station.

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 can prevent flood water from intruding.

FIGURE 9-10 FLOOR ELEVATION OF SUCTION SUMP



9. 6 Design of Pump House

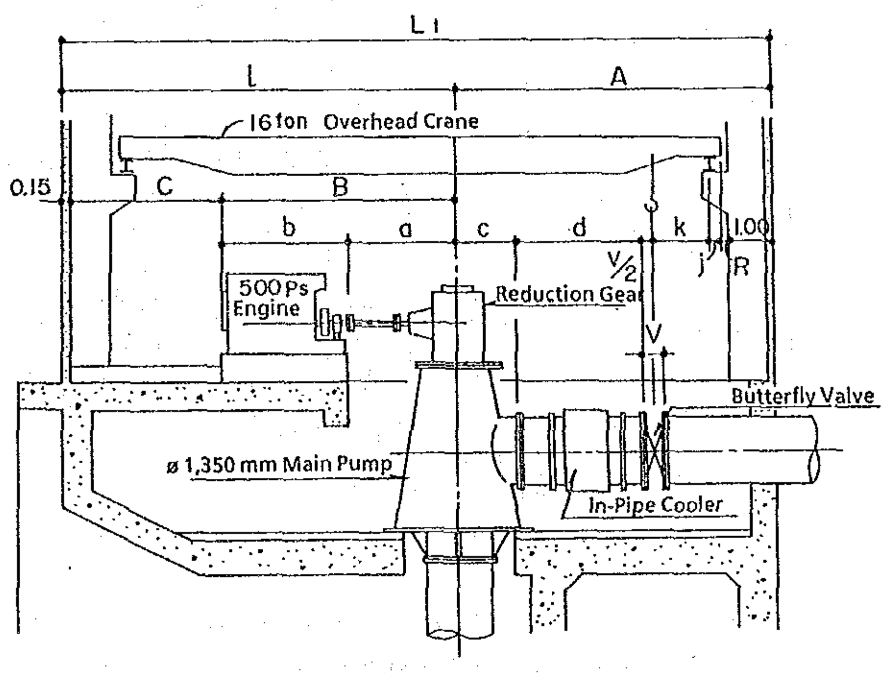
The scale of the pump house will be determined taking into account both the space required for installation of the necessary pump equipment and the space required to carry out successful control and maintenance. The following areas will be included:

Pump room	;	Main Pump (ø1,350 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 9-11 LENGTH OF PUMP ROOM



A = c + d + v/2 + k + j + R + 1.00 (Column thickness)

= 1.30 + 2.70 + 0.40/2 + 1.10 + 0.37 + 1.00 = 6.67 ≐ 6.70 m

$B = a + b = 1.95 + 2.95 = 4.90\ \text{m}$

$C = 3.00\ \text{m}$

$\ell = B + C + 0.15$ (Wall thickness)

$= 4.90 + 3.00 + 0.15 = 8.05 \doteqdot 8.30\ \text{m}$

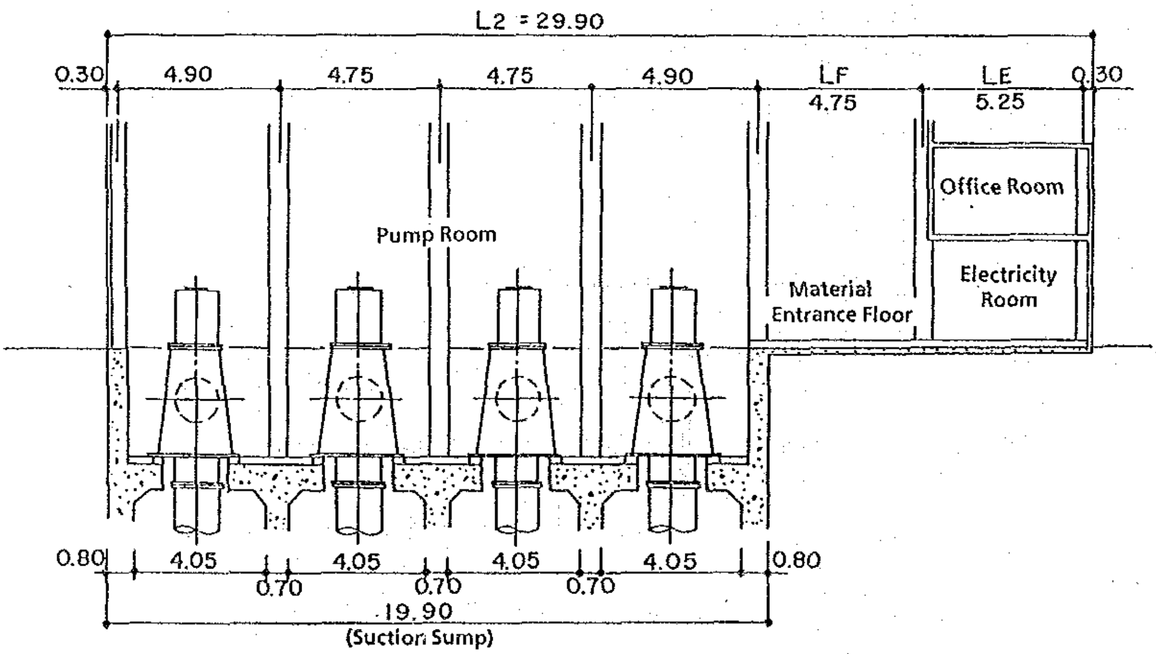
As a result of the above, the total length of the pump room (L_1) can be obtained as follows

$$L_1 = A + \ell = 6.70 + 8.30 = 15.00\ \text{m}$$

2) Width of Pump House

The total width of the pump house will be determined depending on the floor width of the pump installations, the material entrance, and the electricity room.

FIGURE 9-12 TOTAL WIDTH OF PUMP HOUSE



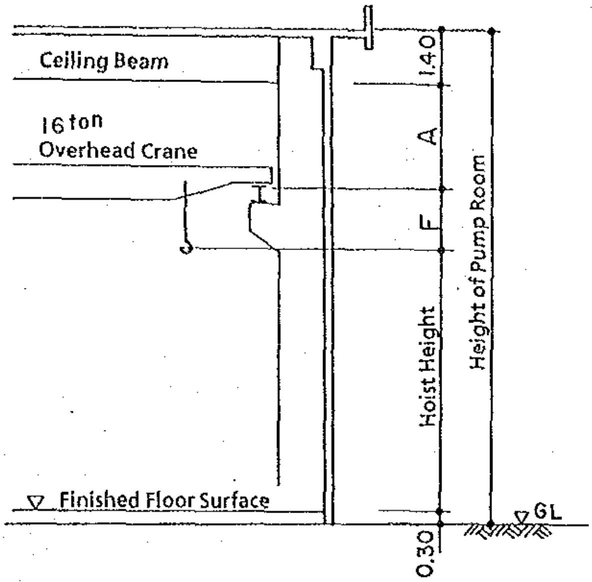
As shown in Figure 9-12, the total width of the pump house (L_2) is decided at 29.90 m including 4.75 m for the material entrance floor (L_F) and 5.25 m for the electricity room (L_E).

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 9-13 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 16 ton overhead crane are as follows.

Hoist height = 6.20 m
F = 0.70 m
A = 2.40 m

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

$$H = 0.30 + 6.20 + 0.70 + 2.40 + 1.40 = 11.00\,\mathrm{m}$$

b) Electricity Room

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

9.7 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 ensuring smooth flow into the following canal.

1) Discharge Water Level

The discharge water level under the basic conditions is as follows.

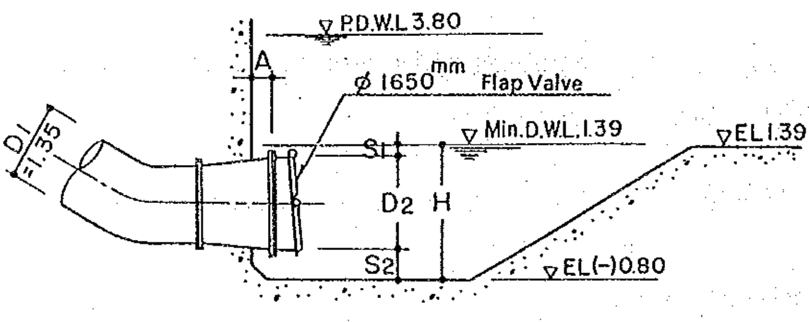
Design discharge water level = P.D.W.L. 3.80 m
Lowest discharge water level = Min. D.W.L. 1.39 m
(=Sill elevation of the main canal)

2) Scale of Discharge Reservoir

a) Water Depth in Discharge Reservoir

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

FIGURE 9-14 WATER DEPTH IN DISCHARGE RESERVOIR



$S_1 \geqq 0$ m, $D_2 = 1.65$, $S2 \geqq 0.40$ m $\therefore H \geqq 2.05$ 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 between the main pump units. The total net reservoir width will be 18.30 m. The reservoir length will be 42.50 m including open transition for smooth discharge into the main canal.

9. 8 Foundation of Pumping Station

Although borehole drilling has not been carried out yet in the investigation for the pumping station site, it may be presumed from similar investigation results on the site of the diversion dam that a sufficiently bearable foundation would be found at a depth greater than EL (-)18.00 (-)20.00 m, pile foundations will be applied to the pumping station site.

CHAPTER 10. BASIC DESIGN OF CONTROL SYSTEM

10.1 Objectives of Control System

A control system for the Bang Pakong diversion dam project will be introduced for the proper operation of the diversion dam and pumping station. The use of this centralized control system, is expected to save control time and ensure the safety of the facilities and surroundings of the diversion dam. Another objective is the effective utilization of water resources and fair distribution of the water.

The general plan of the control system is shown in Figure 10-1.

The Bang Pakong Diversion Dam Project includes the diversion dam, the pumping station, 8 water level gauges and 2 salinity instruments. The monitoring of Tha Lat diversion dam (existing), Rabom dam (existing), Khlong Si Yat dam and sea level gauge at the estuary will be added to the system according to the full plan of the Tha Lat River Basin Development Project.

10.2 Scope of Control System and Control Level

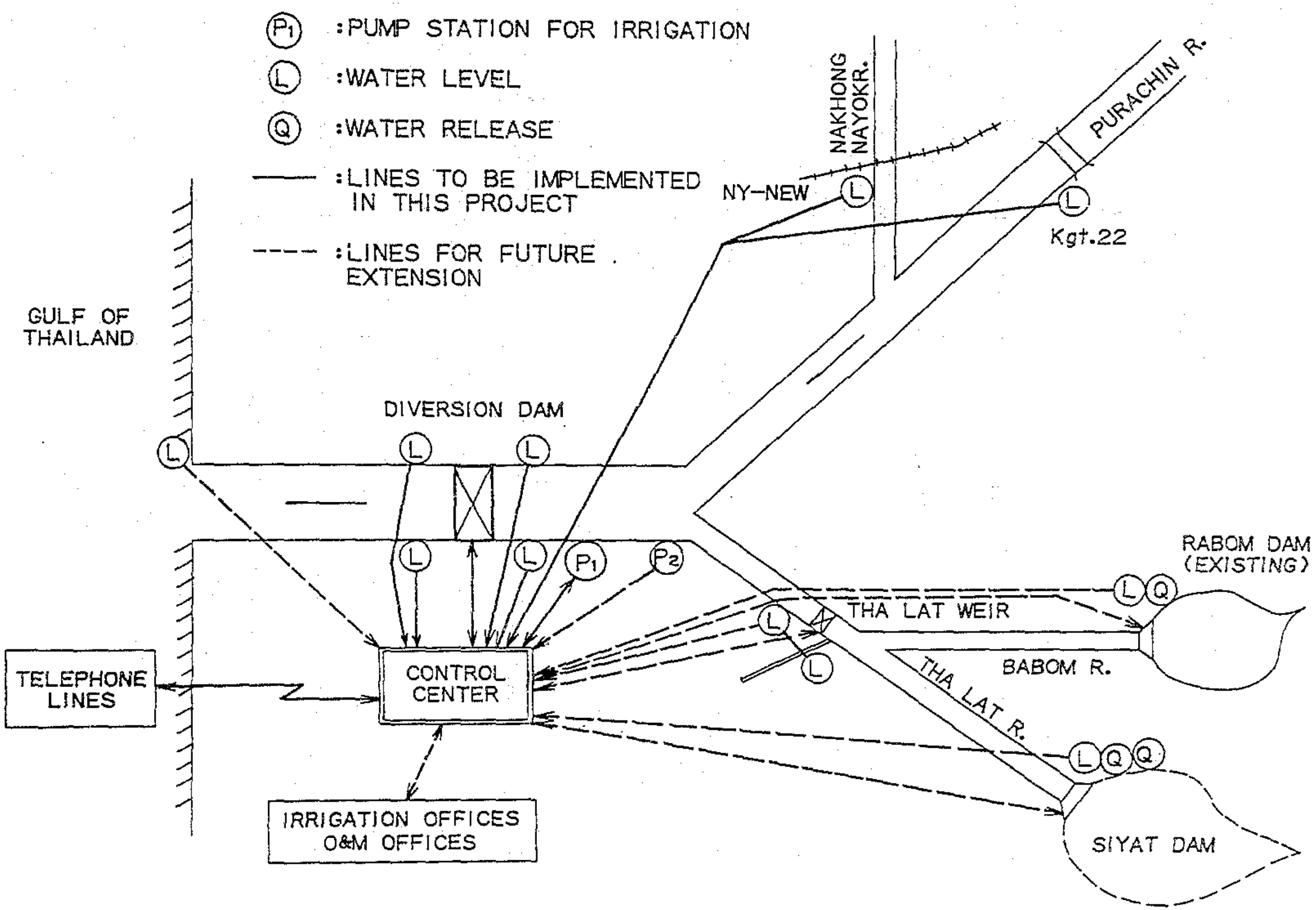
The control system should be simple to operate and maintain. Therefore, a monitoring system should be adopted for the main system.

The control and monitoring system for electric facilities is convenient for the operation and maintenance of the dispersed electric facilities, such as the hoist house, substation of the diversion dam, pumping station and residential area on the O/M building site.

10.2.1 Scope of Control System

The facilities for the centralized control and monitoring system are shown in Table 10-1. The reasons for this selection of facilities are shown below.

FIGURE 10-1 GENERAL PLAN OF CONTROL SYSTEM



1) Control of the diversion dam is possible by monitoring water levels downstream and upstream of the diversion dam and in the Prachin and the Nakhong Nayok rivers.

2) Both remote control and monitoring of the main facilities, that is the diversion dam and the pumping station, as possible.

TABLE 10 - 1 FACILITIES FOR CONTROL AND MONITORING

Name of Facilities	Monitoring	Control	Remarks
1. Diversion Dam	○	○	
2. Pumping Station ①	○	○	Irri. W. Supply, Fish, Others
3. Pumping Station ②	(○)		Industry
4. Upstream Water Level at Diversion Dam	○		2 Spots
5. Downstream Water Level at Diversion Dam	○		2 Spots
6. Prachin River	○		1 Spot
7. Nakhong Nayok River	○		1 Spot
8. Tide at Estuary	(○)		1 Spot
9. Salinity at Diversion Dam	○		2 Spots
10. Khlong Si Yat Dam	(○)		
11. Rabom Dam	(○)		
12. Tha Lat Diversion Dam	(○)		

Note: Facilities with ( ) are not included in Bang Pakong Diversion Dam Project

The following facilities are not included in this Project because a total system including these facilities is not immediately necessary.

a) Tha Lat diversion dam(existing)
b) Rabom dam (existing)
c) Khlong Si Yat dam
d) Pumping station ②, Tide at Estuary

10. 2. 2 Control Level

The control level of the control center and other facilities is shown below.

1) Control Center

a) Telemetry System

The following items will be operated by a telemetry system. But, online control using collected information is not included in this system.

Indication of collected information
Accumulation of -do-
Recording of -do-
Processing of -do-

b) Control and Monitoring of Electric Facilities

The following items will operated by this system.

On-off tele-control of electric facilities
Indication and monitoring of gauges

2) Prachin and Nakhong Nayok river

Water levels will be transmitted to the control center by radio (VHF).

3) Diversion Dam and Pumping Station

a) Telemetry System

Water levels will be transmitted to the control center by cable or communication circuit.

b) Control and Monitoring of Facilities

The following items will be operated by this system.

- Manual on-off control of facilities
- Indication and monitoring of gauges

10.3 Outline of Control System

The outline of the control system is shown in Figure 10-2.

10.3.1 Location and Function of Central Control Room

1) Location

The central control room is located in one room of the control house, which is located on the left bank at the site of the diversion dam.

2) Function

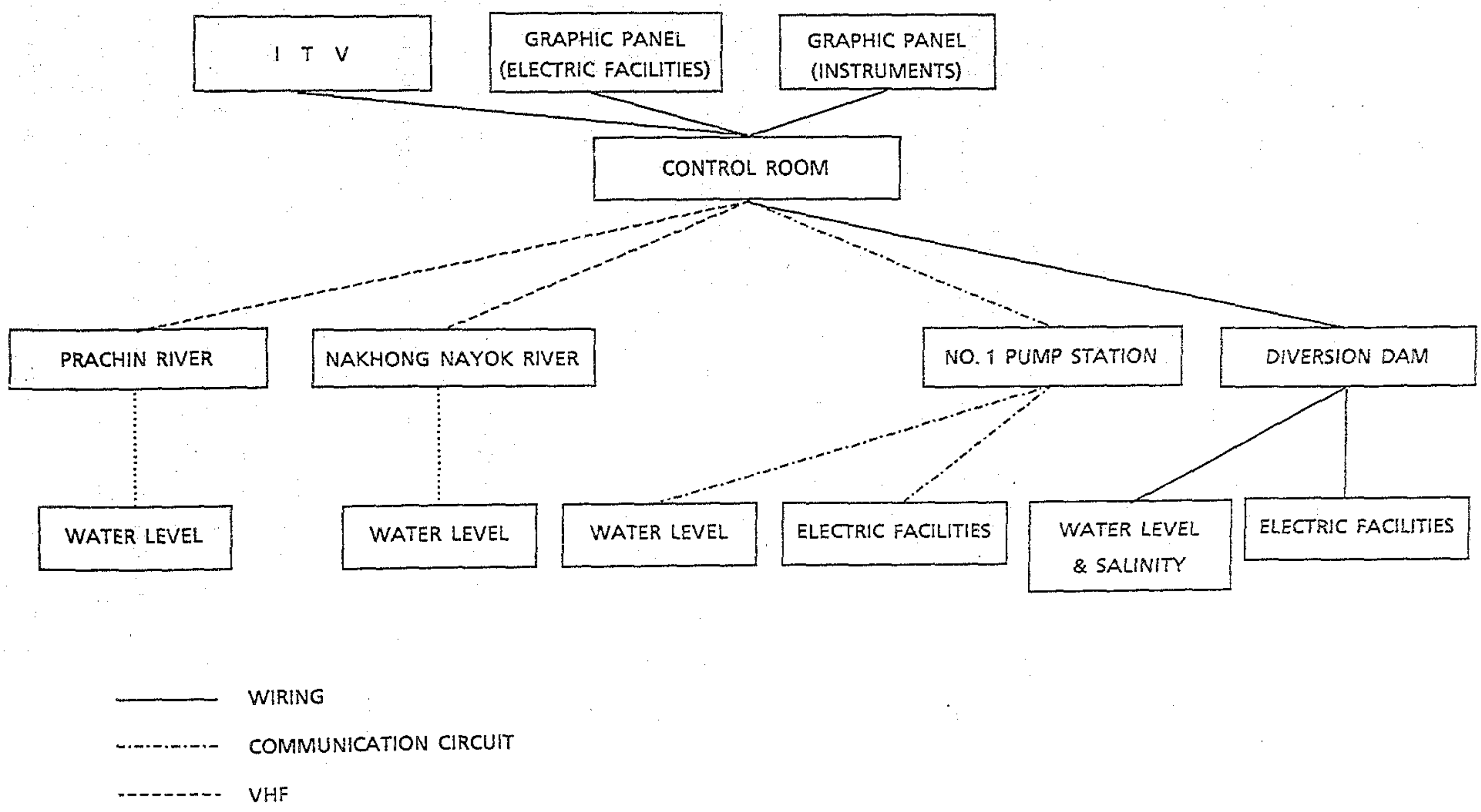
The central control room is the supervisory room for the diversion dam, pumping station and all facilities of this Project. The main functions of this system are as follows.

- Monitoring of water levels and discharges
- Remote control of diversion dam, pumping station and other electric facilities
- Information processing
- Monitoring of diversion dam by ITV (Industrial T.V)
- Information by paging

Main instruments for this control room are as follows.

- Central operation console
- Graphic panel
- Measuring and monitoring device
- Control and monitoring device
- Information processing device
- Alarming device
- ITV device
- Paging device

FIGURE 10-2 OUTLINE OF SUPERVISORY SYSTEM



10. 3. 2 Composition

1) Graphic Panel for Measuring and Monitoring

· The graphic panel should be designed to include the full plan of the Tha Lat River Basin Development Project. For future alteration, the materials of the graphic panel should be mosaic type.

· Indication items on the graphic panel are as follows.

- · Indicators
- · Water levels
- · Discharges form diversion dam and pumping station
- · Condition of regulating and flood gates
- · Condition of pumps

2) Graphic Panel for Electric Facilities

· The condition of substations and electric facilities at the diversion dam, pumping station and residential area will be indicated on the graphic panel.

· The kind of indication will be the on-off condition of each loads system.

10. 3. 3 Transmission Method and Circuit of Information

The transmission method depends on the distance from the control center.

1) Distance Between Control Center and On-Site

- · Prachin river .... 35 km
- · Nakhong Nayok river .... 30 km
- · Pumping station .... 2 km or 7 km*
- · Diversion dam .... 1 km

* distance between control center and pumping station depends on route.

2) Transmission by Radio (VHF)

The next stations are far from the control center, therefore, radio circuit (VHF 139 MHZ) will be adopted in this case.

- Prachin river
- Nakhong Nayok river

3) Transmission by Communication Circuit

The pumping station is 2 km (direct) or 7 km by (another route) from the control center. It is therefore better to use a communication circuit between the control center and the pumping station for fear that wrong values and operation errors may occur because of transmission loss if normal control cables are used.

4) Transmission by Control Cable

Water level gauges and electric facilities at the diversion dam site are less than 1 km away from the control center. Therefore, control cables should be used for the route between the control center and these facilities.

5) Telephone Circuit

A public telephone circuit will be installed in the control room. It will be used for communication with RID head office etc.

6) Monitoring and Paging

Two monitoring cameras will be installed on the upstream and downstream sides of the diversion dam. Television pictures will be transmitted to the control room by an ITV device. The angle of the monitoring cameras should be variable and a zooming device should be added because of the 170 m width of the diversion dam.

A paging device will be installed at each of the following places for announcements that are transmitted from the control room.

- Upstream and downstream of the diversion dam

- Pumping station
- Control room
- Control house
- Training center
- Apartment

10. 3. 4 Outline of Control System

The outline of the control system is shown in Figure 10-3.

1) Scope of Centralized Monitoring System

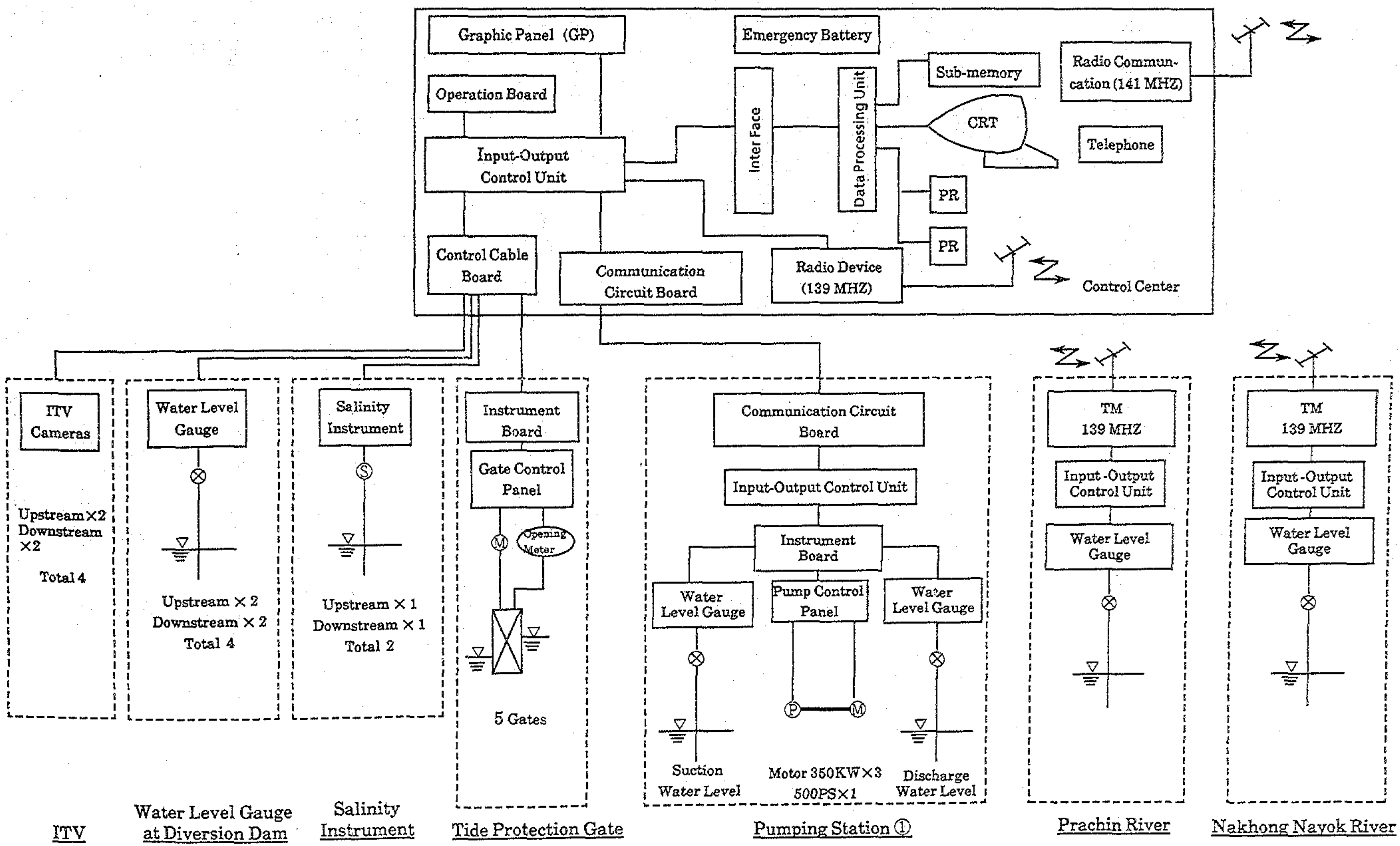
a) Scope of Monitoring and Measuring System

Location	Measuring Items	Transmission Method
Prachin river	Water level ×1	VHF
Nakhong Nayok river	Water level ×1	VHF
Pumping station	Water level ×2	Communication circuit
Diversion dam	Water level ×4	Control cables
-do-	Salinity instrument ×2	-do-
-do-	ITV camera ×4	Coaxial or beam cable
-do-	Condition of gates ×5	Control cable

b) Scope of Control and Monitoring System of Electric Facilities

Location	Control & Monitoring items	Transmission method
Diversion	Power unit	Control cable
-do-	Sub-station	-do-
-do-	Gate motor	-do-
-do-	Emergency power unit	-do-
Pumping station	Power unit	Communication circuit
-do-	Sub-station	-do-
-do-	Pump motor	-do-
-do-	Emergency power unit	-do-

FIGURE 10 - 3 COMPOSITION OF CONTROL SYSTEM



2) Function of Control and Monitoring System

a) Monitoring and Measuring

The purpose of monitoring and measuring is to collect, record and accumulate data. Discharge from the diversion dam and pumping station should be computed from the differences between water levels.

b) Control and Monitoring of Electric Facilities

The on-off controls of all devices and switches will be operated by this system. Also, voltage, electric current, power rate and power factor will be indicated, recorded and accumulated.

c) Alarming System

Warnings of an emergency including information on temperature and function of the control system, will be conveyed by indicators and buzzers.

3) Layout of Control Room

The layout of the control room is shown in Figure 10-4.

The control room should be constructed with a free-type floor to enable wiring and future extension.

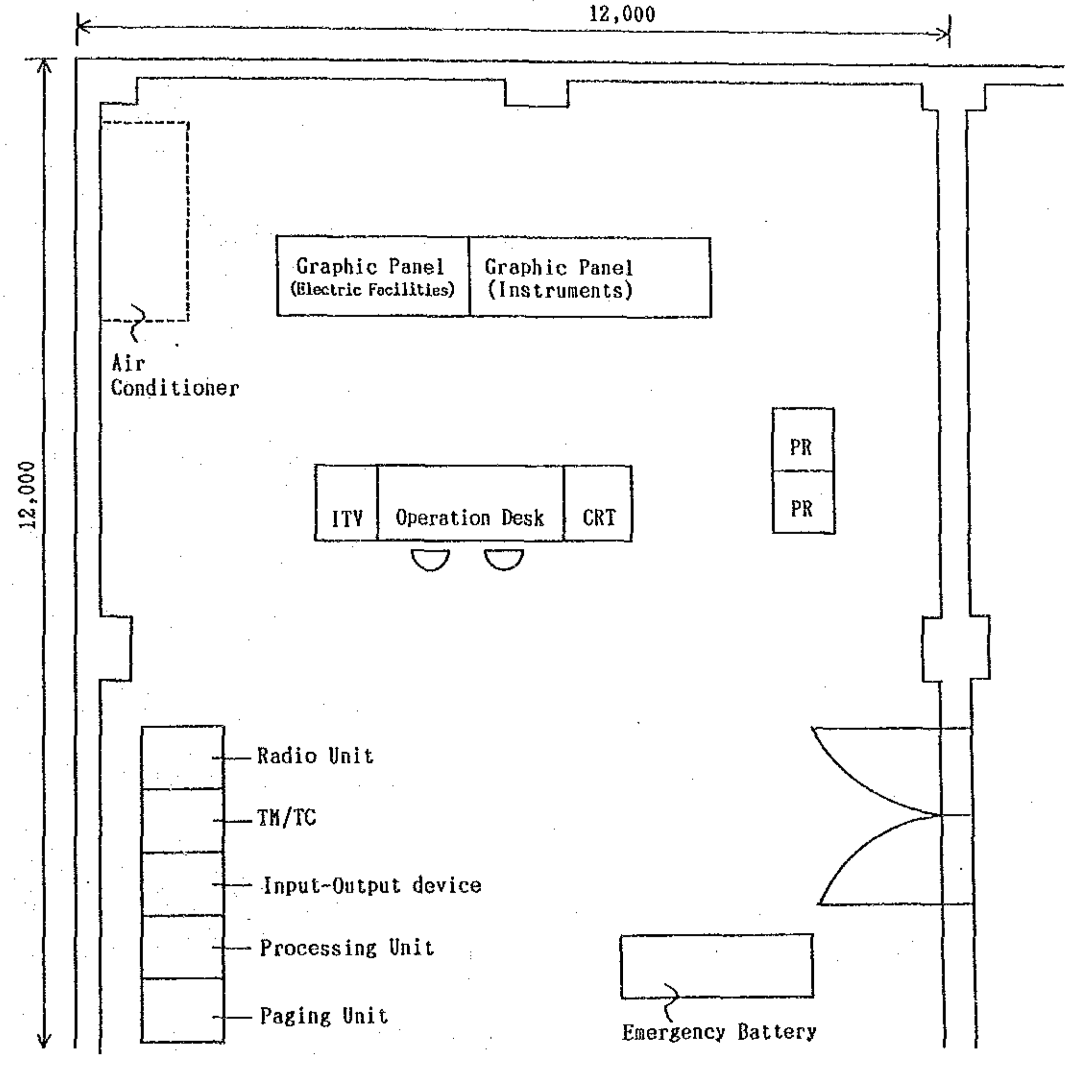
10. 3. 5 Gate Control

It may be possible to operate the tide protection gates by remote control depending on the results of the estimate of the degree of gate opening which in turn depends on the water level data.

There are three phases in the above-mentioned system. And these phases are shown below.

1) Measuring data and data-transmission
2) Estimate of the gate opening and closing
3) Operation of the gates

FIGURE 10-4 LAYOUT OF CONTROL ROOM



Therefore, one manual input is provided in the system as shown below.

Measurement and data-transmission of water level and gate opening/closing positions

↓

Estimate of the gate control by using the water level and gate positioning data, for the next step by personal computer

↓

(Manual input of the data to the remote control panel)

↓

Operation of the gates by remote control

10. 4 Composition of Instruments

10. 4. 1 Water level Gauge

1) Location

a) Gauging Station for River

Two water level gauging stations have been chosen after a site survey of the rivers. These stations are as follows.

Prachin river (KGT.22)	; catchment area	9,260 km2	(63%)
Nakhong Nayok river	; -do-	1,910 km2	(13%)

Notes ; the value of ( ) means the area's percentage of the total catchment area of the diversion dam of 14,730 km2

The existing water level gauging station KGT. 22 (Prachin river) has been observed by RID using a staff gauge. This station is 35 km from the diversion dam.

There is no existing gauging station around the confluence with Prachin river and the Nakhong Nayok river. Therefore, the new gauging station should be constructed at the crossing point between railway and the Nakhong Nayok river. This new gauging station will be 30 km from the diversion dam.

The catchment area of these two stations occupies 76% of the total catchment area of the diversion dam.

b) Gauging Station for Diversion Dam and Pumping Station

Two water level gauging stations should be constructed both upstream and downstream of the diversion dam, for the operation of the tide protection gates. In all, there need to be four gauging stations at the diversion dam site.

Also, one gauging station should be constructed at both the suction sump and the discharge reservoir of the pumping station, for monitoring the pumping discharge.

2) Type of Water Level Gauge

Floating type water level gauges will be adopted for these stations because of their frequent use in the river. This type is available for long term observation and maintenance.

10. 4. 2 Salinity Instrument

1) *Location*

Two salinity instruments will be installed both upstream and downstream of the diversion dam. The upstream instrument will be used for monitoring water quality, and the downstream one will be used for operation of the tide protection gates.

Salinity at three depths should be monitored for each station.

2) Type of Salinity Instrument

The salinity instrument consists of a sensor for the temperature of water and specific conductance.

Salinity is estimated from this specific conductance.

10. 4. 3 ITV Monitoring Device

The ITV monitoring device consists of a monitoring camera and a television set. Two monitoring cameras each will be installed both upstream and downstream of the diversion dam. Altogether, four cameras are needed for this system. Television sets will be installed in the control room for monitoring. And, flood lighting devices are needed for night.

1) Transmission Distance and Movement Method

Transmission distance .... 1 km
Movement method .... real time monitoring system

2) Monitoring Camera

Lightness of subject .... 10 - 10,000 LX
Lens of camera .... powered zooming
Efficiency of turning .... over 300° (horizontal)
over 60° (vertical)

3) Television Set and Operation Desk

Television set .... 14 inches, color
Operation desk .... camera operation desk

10. 4. 4 Telemetry System and Graphic Panel

The telemetry system consists of the following three phases and it is shown in Figure 10-2.

Prachin and Nakhong Nayok river ..	radio wave (VHF)
Pumping station	TM/TC
Diversion dam area	control cables

1) TM/TC

Transmission way	communication circuit
Transmission velocity	200 bit/sec
Transmission method	circulating method
Linkage method	1 : 1

2) Graphic Panel

The graphic panel will feature digital displays and lamps for water levels, discharge, gate opening/closing and the condition of pumps.

10. 4. 5 RADIO DEVICE AND CONTROL DEVICE

1) Radio Device (VHF)

A radio wave device will be adopted for the transmission of water levels of the Prachin and Nakhong Nayok rivers.

Transmission way	139 MHZ
Transmission velocity	200 bit/sec
Transmission method	polling method
Linkage method	1 : 2, available to 1 : 7

According to the Tha Lat River Basin Development Project, monitoring stations will be increased in future. Therefore, it is economical to use the 1 : N linkage method.

2) Control Device

A control device will be used for the control and monitoring of electric facilities. Graphic panel will be installed for the electric facilities.

a) Control Board

The control board will be a standing desk type, and the electric facilities for the diversion dam, pumping station, residential area and each sub-station will be operated from this control board. The on-off control of the gate motors and pump motors will also be operated from this point. The condition of these electric facilities will be indicated on the graphic panel.

b) Graphic Panel for Electric Facilities

The following items will be indicated on the graphic panel for electric facilities.

- Electric circuit of each substations and main loads
- On-off condition of main electric circuit
- Emergency circuit and relay
- Voltage, electric current, and power factor of each sub-station