4th layer : The 4th layer consists of silty clay and fine sand and shows more than 40 in the N-value. This fine sand layer is intercalated between clay layers with a lenticular shape and its continuation is not clear. The silty clay layer is well compacted and consolidated as a whole and distributed evenly. As a result, it can be judged to have sufficient bearing capacity.

2) Site of Closure Dam

On both banks of the closure dam, there is no 3rd layer (fine-coarse sand layer as mentioned above) and the silty clay is continuously spread reaching about 30 m in depth. The thickness of an extremely soft layer in this site is almost the same at the sites of the diversion dam and the diversion canal, reaching 9 m on the left bank and 12.5 m on the right bank (3 to 5 m at the riverbed) respectively.

3) Site of Pumping Station

At the site of the pumping station, it is remarkable that the thickness of the 2nd layer (clay and silt layer with intercalation of silty sand layers) and 3rd layer (fine-coarse sand layer) is from 6 to 7 m in average. However, the typical stratigraphy in this site seems to correspond, as a whole, to the sites of the diversion dam and diversion canal, and the thickness of an extremely weak layer is comparatively well correlated to the sites mentioned above, reaching about 10 m.

2. 3 Soil Mechanics Property of Alluvial Deposit

A series of soil tests on samples of materials taken from bore-holes and test pits was executed in order to clarify the physical and mechanical properties of the alluvial deposits extending around the diversion damsite.

The results of the soil tests refer to Appendix 2. 7 and the brief descriptions for the physical and mechanical properties of the alluvial deposit are as follows;

1) Physical Properties

a) Specific Gravity

The distribution of specific gravity in depth according to the distributed samples from the bore-holes is shown in Figure 2 - 2.

The specific gravity is erratically distributed over a range of 2.59 to 2.84 with considerable irregularity in depth, however, this is not inconsistent with the data obtained in previous similar tests.

b) Field Moisture Content

The distribution of field moisture content in depth is shown in Figure 2 - 3, in which the widely distributed values range between 20 and 100%, however, it can be divided into 2 layers at a depth of about 8 m.

The field moisture content in the upper layer decreases significantly with an increase of depth, and finally may reach about 50% at a depth of 8 m. On the other hand, the lower layer decreases gradually in depth and approaches a value of about 20% around a depth of 22 m.

c) Grading Distribution

Grading distribution in depth is shown in Figure 2 - 4(a) to 2 - 4(c). As are seen in most specimens, the extremely soft layer reaching less than 8 m in depth is composed of typical fine particles such as clay and silt materials (less than 0.075 mm in diameter) with more than 95% in weight. On the other hand, the sand materials are found prominently in some specimens at more than 8 m in depth, however, most of the specimens contain many fine particles such as clay and silt materials and show the same tendency in the grading distribution in the upper layer.

d) Liquid and Plastic Limits

According to liquid and plastic limits in depth as shown in Figure 2 - 3, the value approximately corresponds to a range of 35 to 55% in the liquid limit and of 20 to 40% in the plastic limit, having the same tendency to decrease gradually in accordance with the distance below the ground surface.

e) Plasticity Chart

The relationship between the plasticity index and liquid limit is shown in the plasticity chart of Figure 2 - 5.

The greatest value is distributed in the upper part along the A line which indicates that the deposits consist mainly of inorganic clay belonging to CL (ML, OL, MH and OH partially distributed) under the Unified Soil Classification System with low to medium plasticity and less cohesion.

f) Plastic Index

The distribution of plastic index (PI = LL - PL) in depth is indicated in Figure 2 - 6, in which there is no extreme variation in depth and all data are within the range of 10 to 20%.

g) Consistency Index

Regarding the distribution of consistency index (IC = (LL-Wf) / PI) in depth, it can be classified into 2 layers at a depth of about 8 m as shown in Figure 2 - 6. The upper layer corresponds to the values below zero in the consistency index, in which it is generally known that the soil state is induced to the liquid state by remolding and disturbance because the field moisture content exceeds the liquid limit, resulting in an unstable condition. On the contrary, in the lower layer, the values of consistency index remains in the range of 0.5 to 1.5 in a stable condition.

h) Liquidity Index

As for the liquidity index (IL = (Wf-PL) / PI), the distribution in depth is mainly divided into 2 layers with the same tendency for the consistency index as shown in Figure 2 - 6. The upper layer of about 8 m in thickness is shown to be generally over 3.0 in the liquidity index indicating very soft ground with extreme sensitivity.

On the other hand, the lower layer corresponds to a value of less than 1.0 in the liquidity index indicating a stable condition.

2) Mechanical Properties

a) Shear Strength

The distribution of shear strength (shear strength means cohesion) in depth by the in-situ vane test, triaxial compression test and unconfined compression test, is shown in Figure 2 - 7, in which the values are widely distributed ranging between 0.1 and 2.5 kgf/cm^2. However, it seems to be classified into 2 layers at a depth of about 8 m below the ground surface. The upper layer corresponds to the value of 0.1 to 0.5 kgf/cm^2, in which there is no definite tendency in depth and mostly distributes in a range of 0.15 to 0.4 kgf/cm^2. On the contrary, the lower layer, shows a range of 0.5 ~2.5 kgf/cm^2 and the value in creases gradually in accordance with an increase in depth.

b) Pre-consolidation Stress

As for the pre-consolidation stress of the alluvial deposits, Figure 2 - 8 shows a relationship between the consolidation yield stress and overburden load in depth.

In the above figure, the relationship in depth is comparatively well correlated, therefore, the alluvial deposits can be recognized as a normal consolidated clay layer.

c) Deformation Modulus

Deformation modulus of the alluvial layer is estimated from the results of lateral loading tests in bore-holes executed at the sites of the road bridge, closure dam and pumping station. The relationship between N-value and deformation modulus is shown in Figure 2 - 9. The relationship mentioned above is well correlated except the extremely soft layer indicating less than 4 in the N-value, and the deformation modulus shows 60 to 70 kgf/cm^2 in the case of the N-value of around 10, 100 to 150 kgf/cm^2 for around 20 and about 200 kgf/cm^2 for around 30.

2.4 Embankment Materials

A reconnaissance for embankment materials of the closure dam was made around the project area, and a suitable borrow area was found near the Ban Lum Maha Chai Village which is located about 35 km east-north-east of the closure dam site.

In the borrow area, it can be expected to obtain a sufficient quantity for the construction of the closure dam, and the mechanical property of borrow materials is summarized in Table 2 - 1 and Figure 2 - 10.

The borrow materials shows 2.62 to 2.70 in specific gravity and contains 58 to 90% sand, 6 to 33% silt, 6 to 11% clay in grading distribution and is classified into SM under the Unified Soil Classification System as silty sand. Judging from the quantity and quality of the materials available in the borrow area, these materials can be sufficiently utilized as embankment materials for the closure dam.

TABLE 2-1 RESULTS OF SOIL TEST (DISTURBED SAMPLES AT BORROW AREA TEST-PITS)

1. Physical Tests

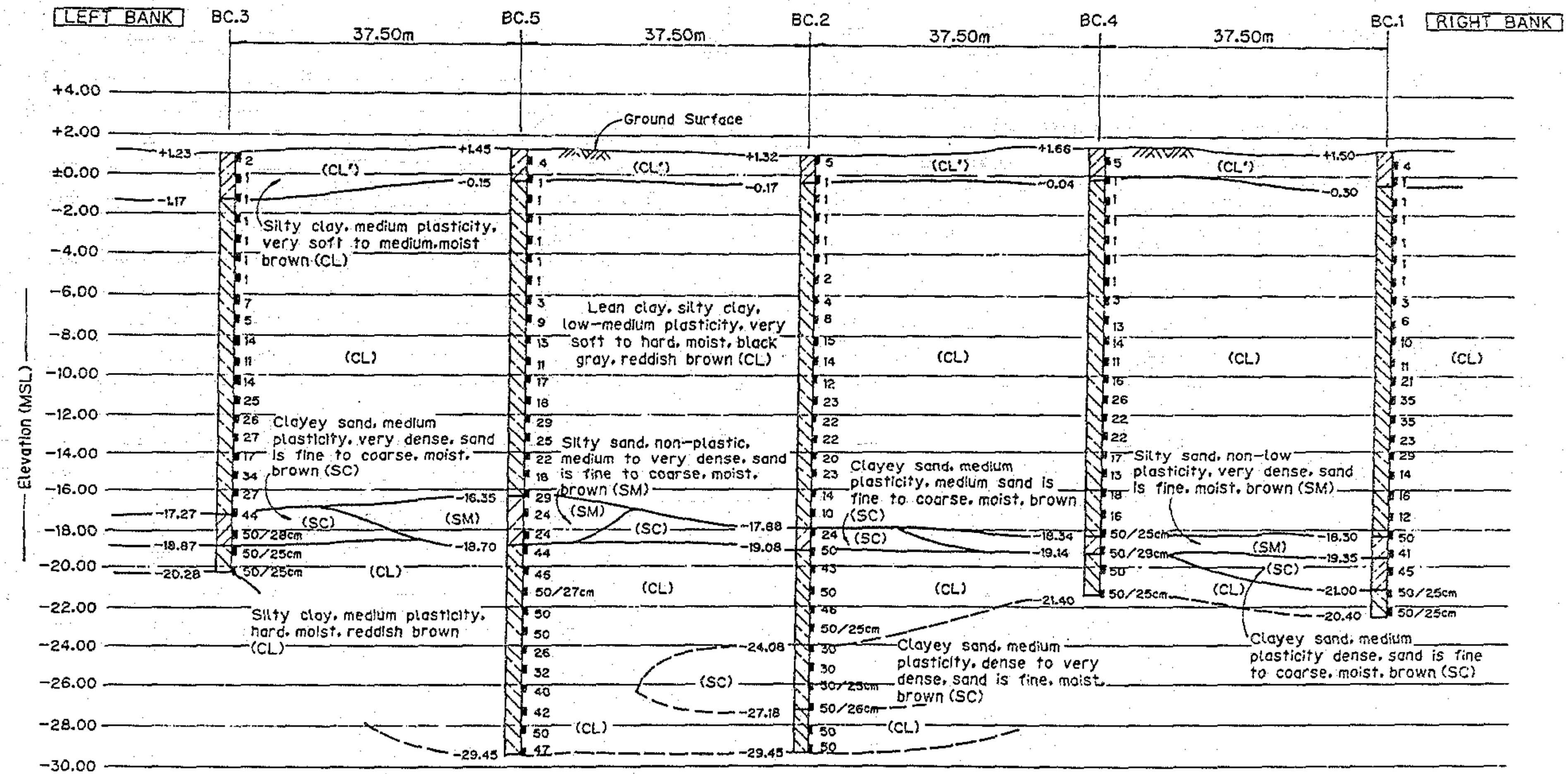
No. of Hole	Depth (m)	Unified, Soil Classification System	Field Moisture Contents, Wf (%)	Specific Gravity, Gs	Gradation Distribution (%) Sand	Silt	Clay	Liquid Limit LL (%)	Plastic Limit PL (%)	Plastic Index, PI
No. 1	-	SM	4.7	2.66	82.6	6.2	11.2	-	-	-
No. 2	-	SM	2.0	2.65	79.4	14.8	5.8	-	-	-
TP-1	0.00~1.00	SM	6.6	2.70	71.5	28.5		-	-	-
	1.00~2.00	SC	9.0	2.60	58.0	32.0	10.0	19.8	11.8	8.0
	2.00~3.00	SM-SC	1.9	2.60	58.0	33.0	9.0	18.4	12.7	5.7
TP-2	0.00~1.00	SM	1.6	2.62	82.0	18.0		-	-	-
	1.00~2.00	SM	1.0	2.62	79.0	21.0		-	-	-
	2.00~3.00	SM	4.2	2.64	72.0	28.0		-	-	-
TP-3	0.00~1.00	SM	0.9	2.65	80.0	20.0		-	-	-
	1.00~2.00	SM	1.0	2.65	77.5	22.5		-	-	-
	2.00~3.00	SP-SM	0.9	2.65	90.0	10.0		-	-	-

2. Mechanical Tests

No. of Hole	Depth (m)	Compaction Test Max-γd *1 (gr/cm³)	Wopt *2 (%)	Direct Shear Test C-U Condition C (kgf/cm²) *3	ø(°) *4	Coefficient of Permeability K, (cm/sec)
TP-1	0.00~1.00	2.021	7.6	0.12	34.4	6.25×10^{-7}
	1.00~3.00	2.004	9.7	0.27	28.1	8.70×10^{-6}
TP-2	0.00~2.00	1.851	6.0	0.01	31.1	1.18×10^{-5}
	2.00~3.00	2.112	6.4	0.05	34.1	1.46×10^{-6}
TP-3	0.00~2.80	1.887	4.9	0.00	33.0	3.11×10^{-5}

Note ; The direct shear test and permeability test have been carried out at 95% of the maximum dry density condition with 2% wet side of the optimum moisture content.
*1 maximum dry density, *2 optimum moisture content,
*3 cohesion, *4 internal friction angle,

FIGURE 2 - 1(a) GEOLOGICAL PROFILE ON DIVERSION DAM AXIS



SYMBOLS

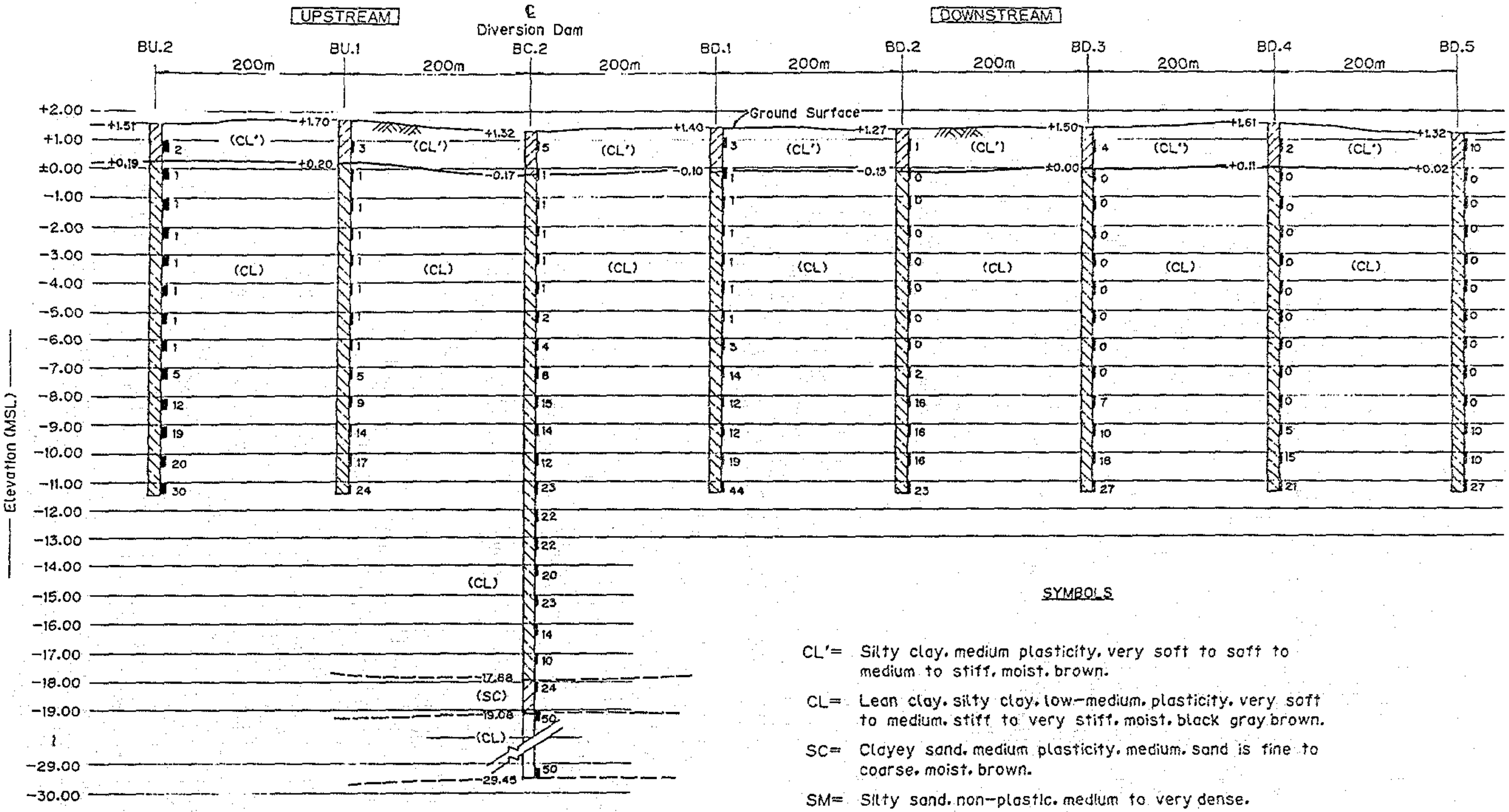
CL'= Silty clay, medium plasticity, very soft to soft to medium to stiff, moist, brown.

CL= Lean clay, silty clay, low-medium, plasticity, very soft to medium, stiff to very stiff, moist, black gray brown.

SC= Clayey sand, medium plasticity, medium, sand is fine to coarse, moist, brown.

SM= Silty sand, non-plastic, medium to very dense, sand is fine to coase, moist, brown.

FIGURE 2 - 1(b) GEOLOGICAL PROFILE OF DIVERSION CANAL



SYMBOLS

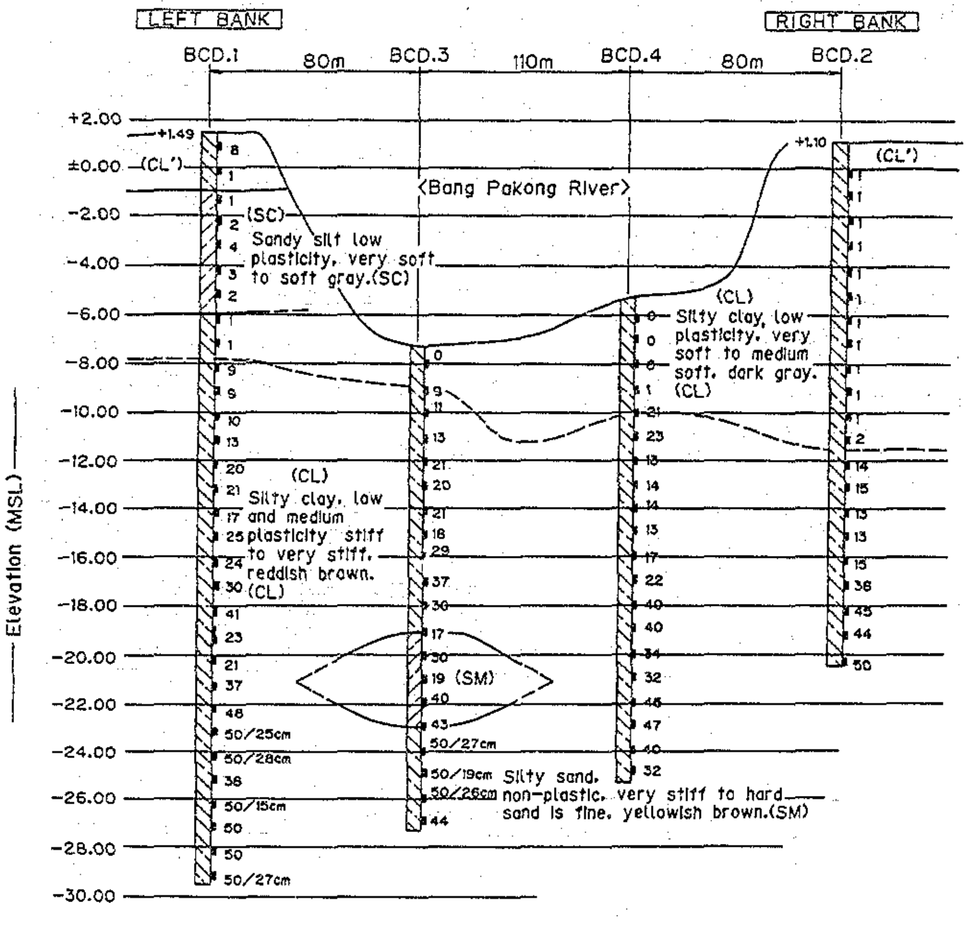
CL'= Silty clay, medium plasticity, very soft to soft to medium to stiff, moist, brown.

CL= Lean clay, silty clay, low-medium, plasticity, very soft to medium, stiff to very stiff, moist, black gray brown.

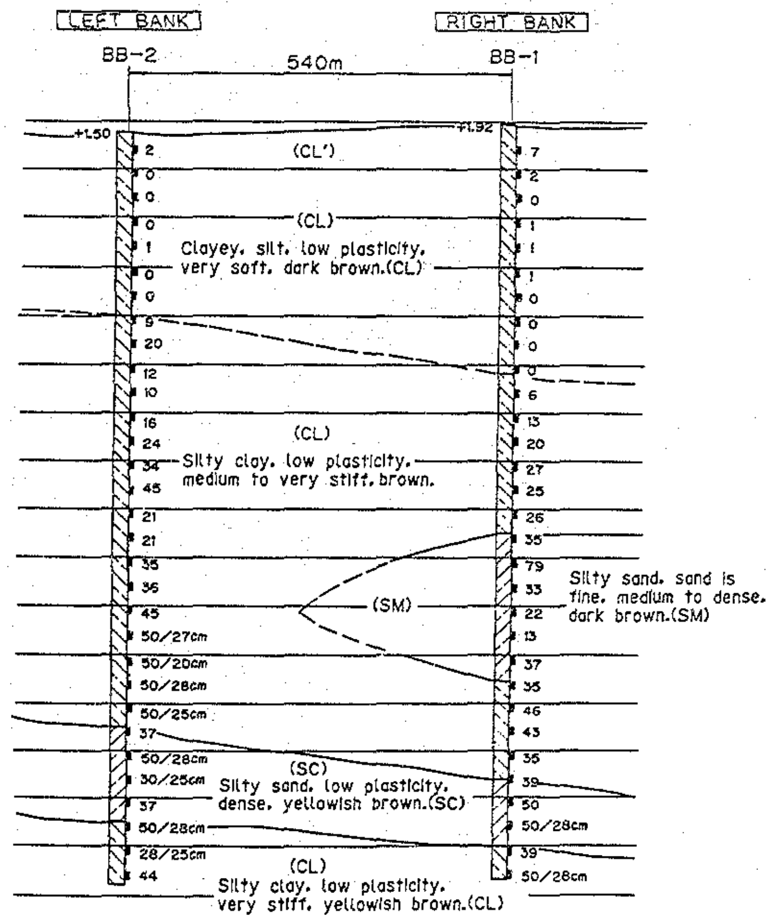
SC= Clayey sand, medium plasticity, medium, sand is fine to coarse, moist, brown.

SM= Silty sand, non-plastic, medium to very dense, sand is fine to coase, moist, brown.

FIGURE 2 - 1(c) GEOLOGICAL PROFILE AT CLOSURE DAM AND ROAD BRIDGE



CLOSURE DAM



ROAD BRIDGE

SYMBOLS

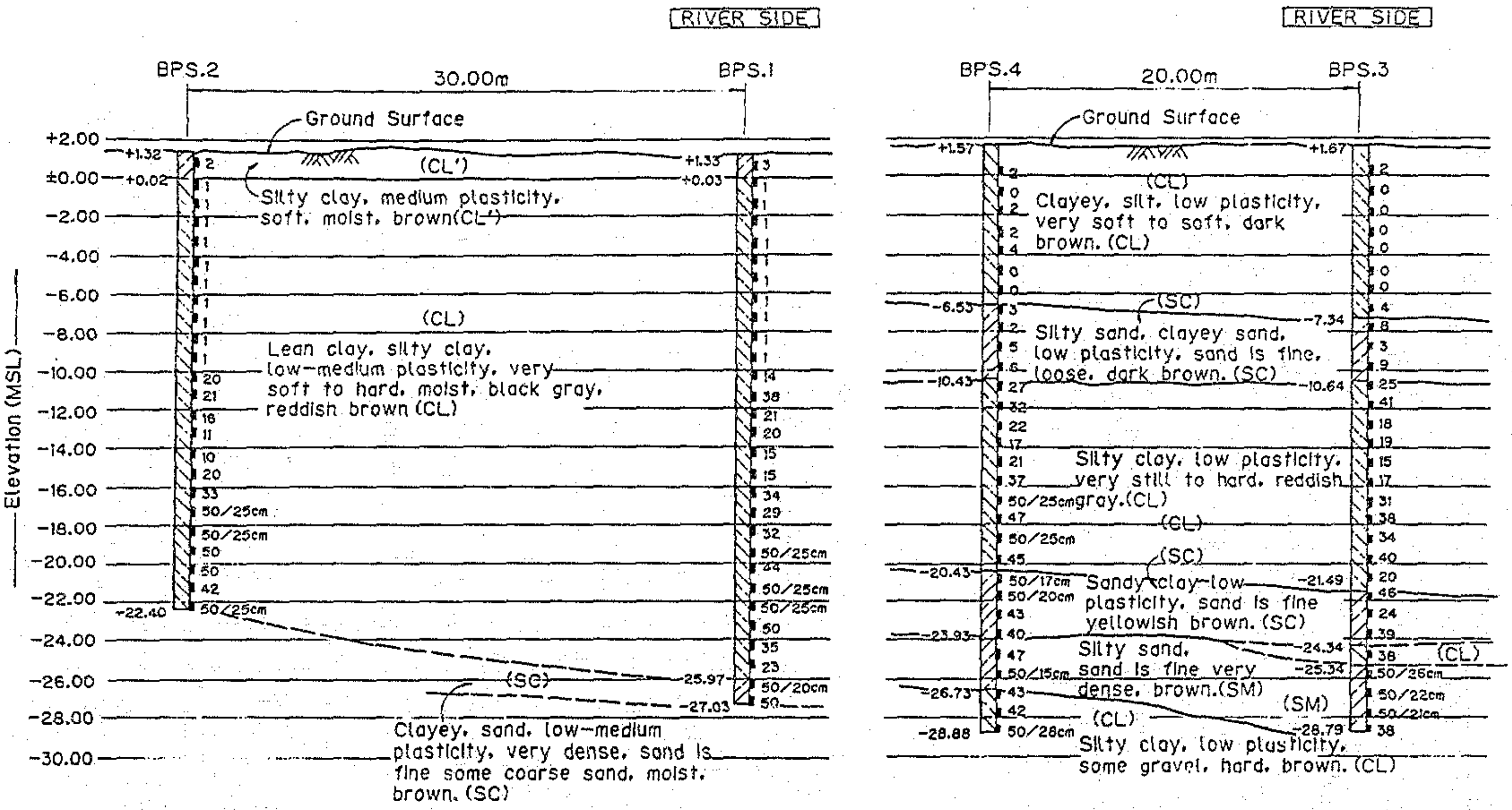
CL'= Silty clay, medium plasticity, very soft to soft to medium to stiff, moist, brown.

CL= Lean clay, silty clay, low-medium, plasticity, very soft to medium, stiff to very stiff, moist, black gray brown.

SC= Clayey, sand, medium, plasticity, medium sand is fine to coarse, moist, brown.

SM= Silty sand, non-plastic, medium to very dense, sand is fine to coase, moist, brown.

FIGURE 2 - 1(d) GEOLOGICAL PROFILE AT PUMPING STATION



SYMBOLS

CL'= Silty clay, medium plasticity, very soft to soft to medium to stiff, moist, brown.

CL= Lean clay, silty clay, low-medium, plasticity, very soft to medium, stiff to very stiff, moist, black gray brown.

SC= Clayey, sand, medium plasticity, medium, sand is fine to coarse, moist, brown.

SM= Silty sand, non-plastic, medium to very dense, sand is fine to coase, moist, brown.

FIGURE 2-2 RELATIONSHIP BETWEEN SPECIFIC GRAVITY AND DEPTH (DISTURBED SAMPLES BY BORING CORES)

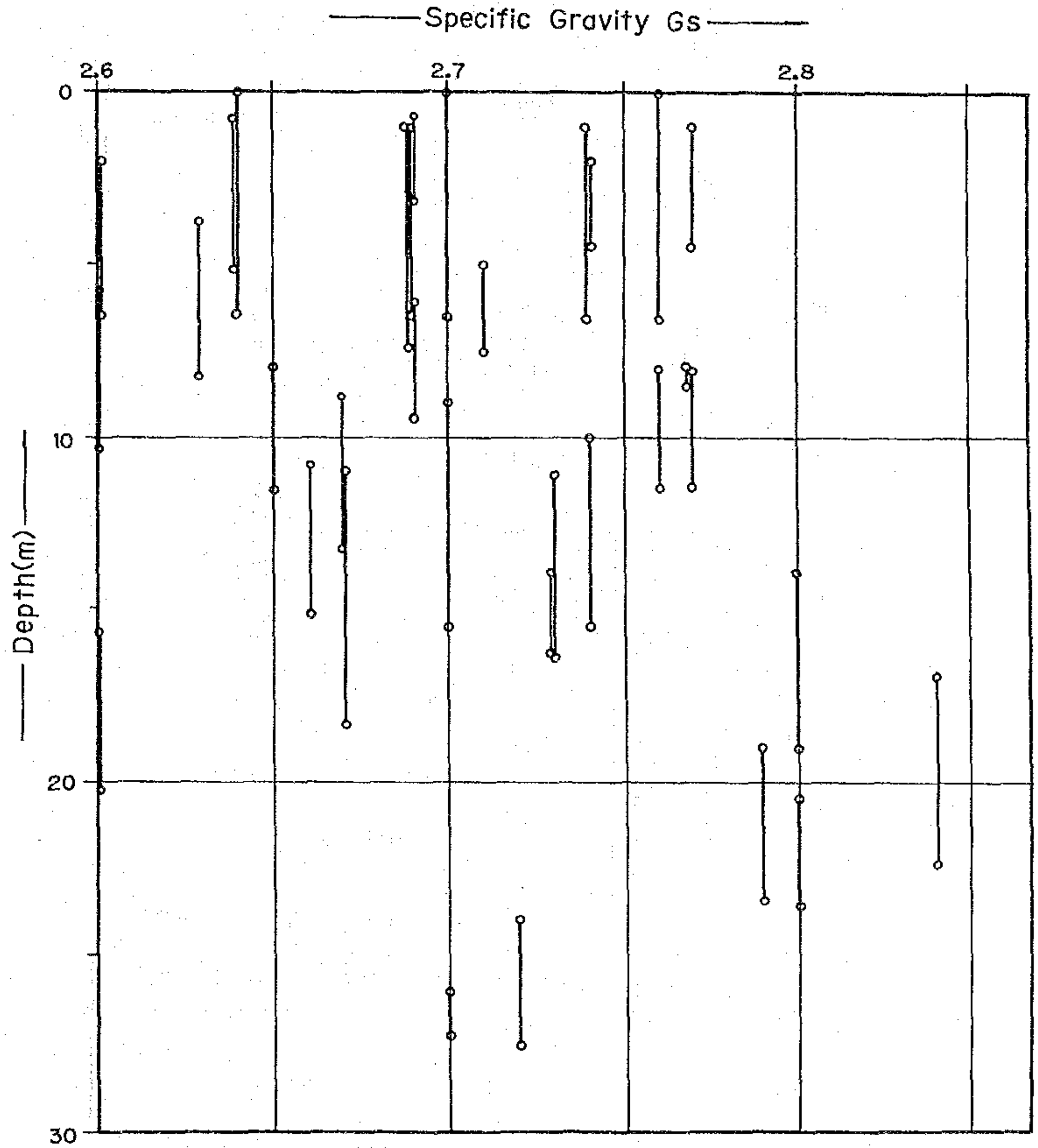


FIGURE 2-3 RELATIONSHIP BETWEEN PHYSICAL PROPERTIES AND DEPTH (DISTURBED SAMPLES BY BORING CORES)

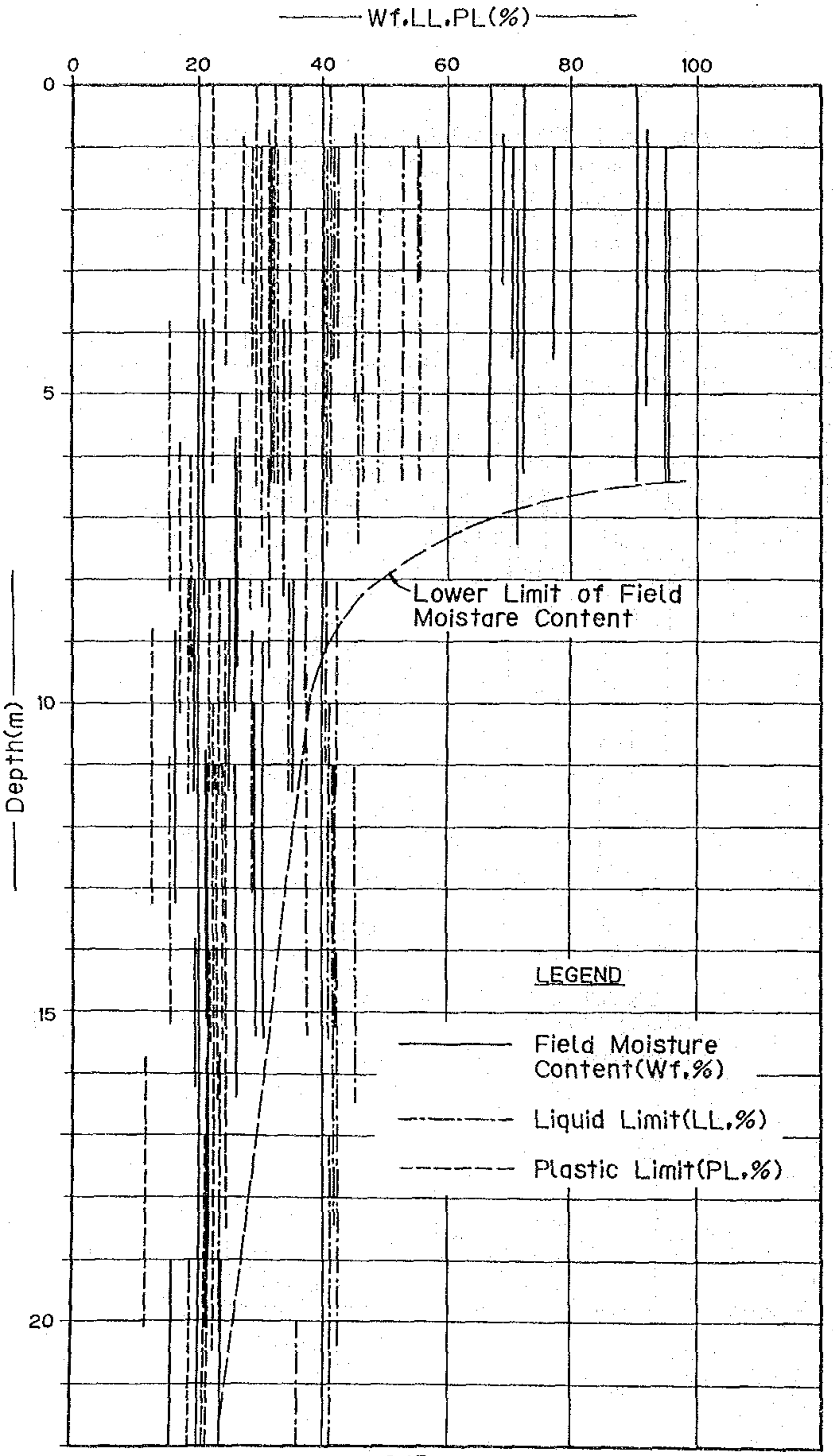


FIGURE 2 - 4(a) GRAIN SIZE DISTRIBUTION CURVE OF SOIL (1)

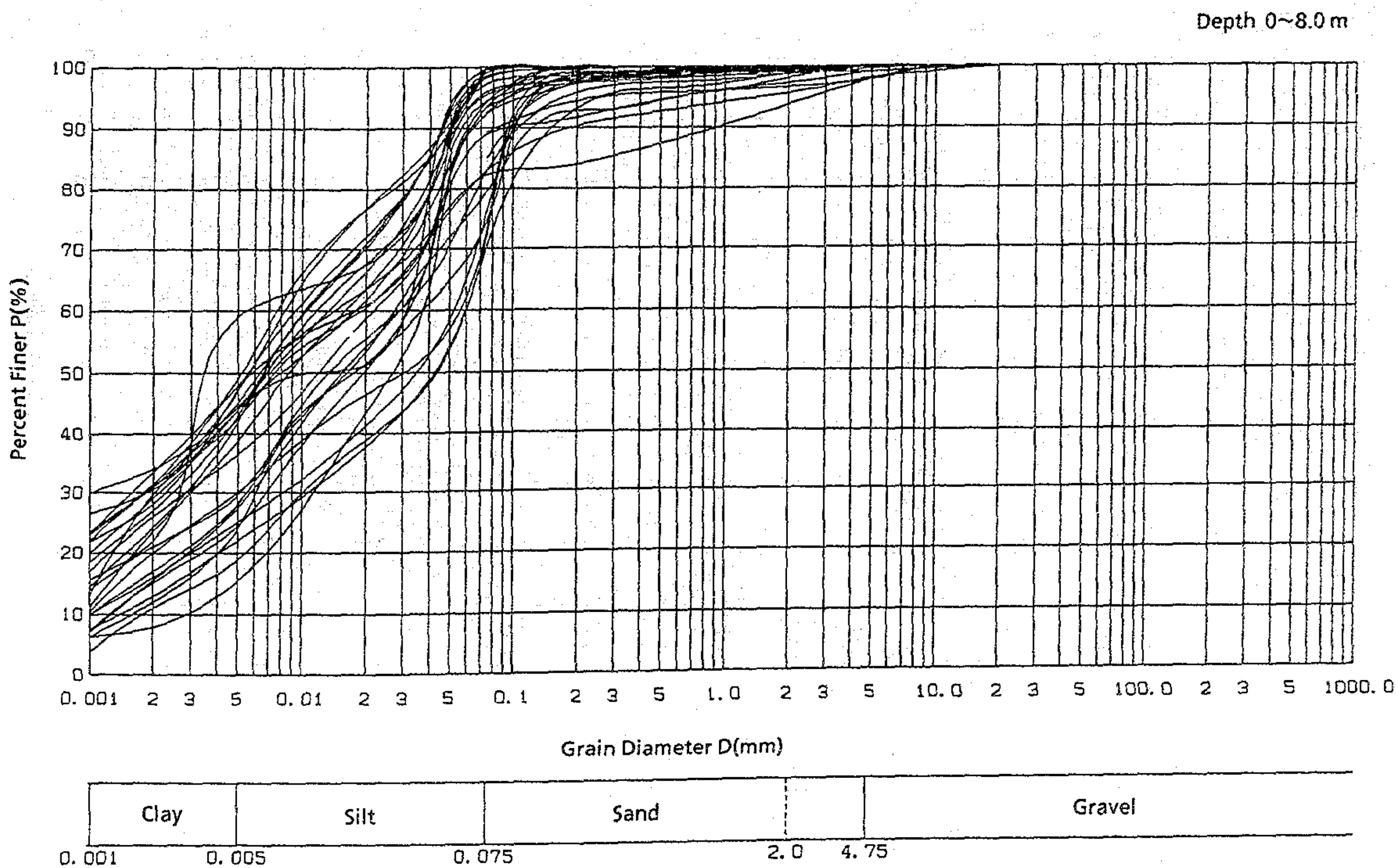


FIGURE 2 - 4(b) GRAIN SIZE DISTRIBUTION CURVE OF SOIL (2)

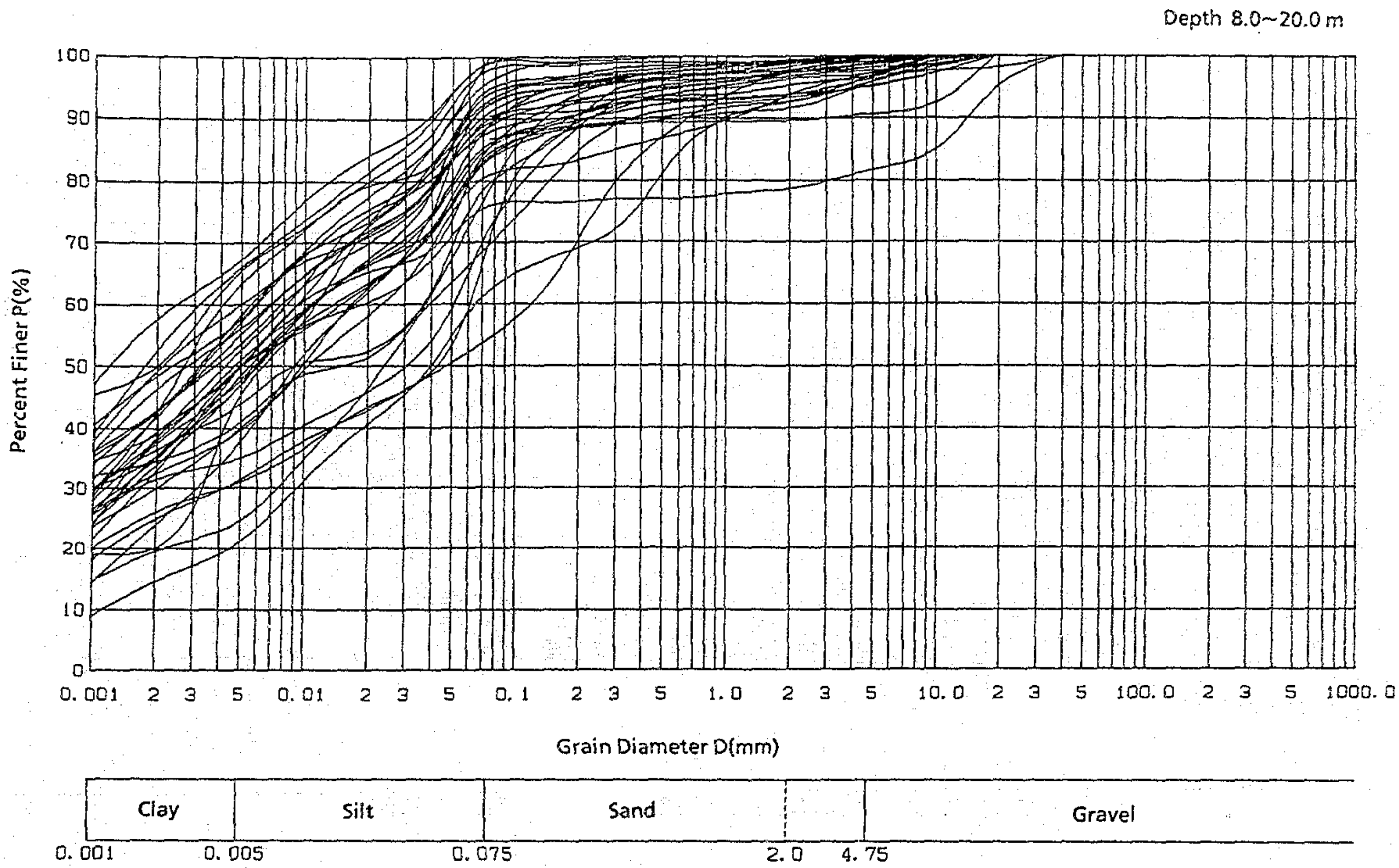


FIGURE 2 - 4(c) GRAIN SIZE DISTRIBUTION CURVE OF SOIL (3)

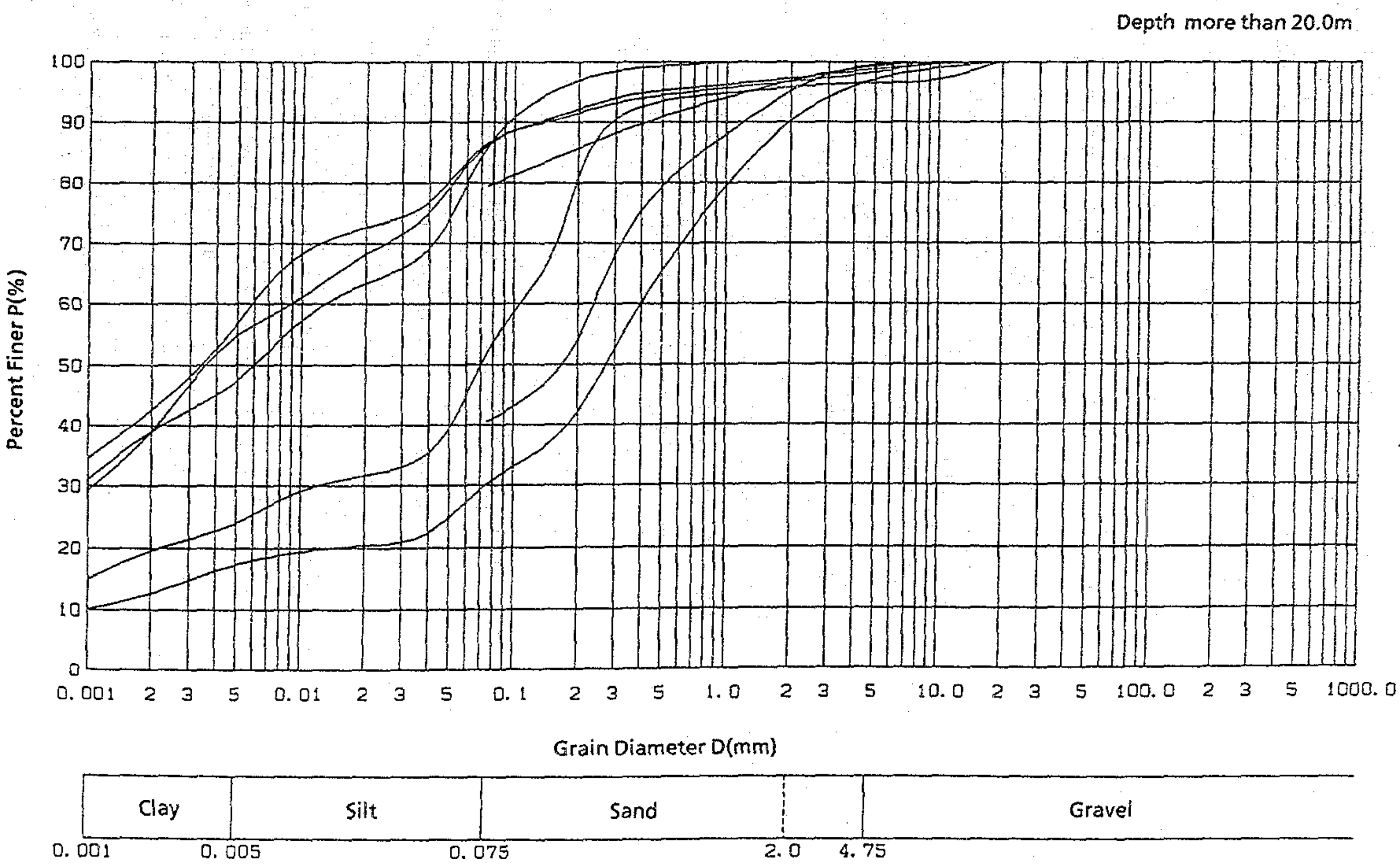


FIGURE 2-5 PLASTICITY CHART OF SOIL

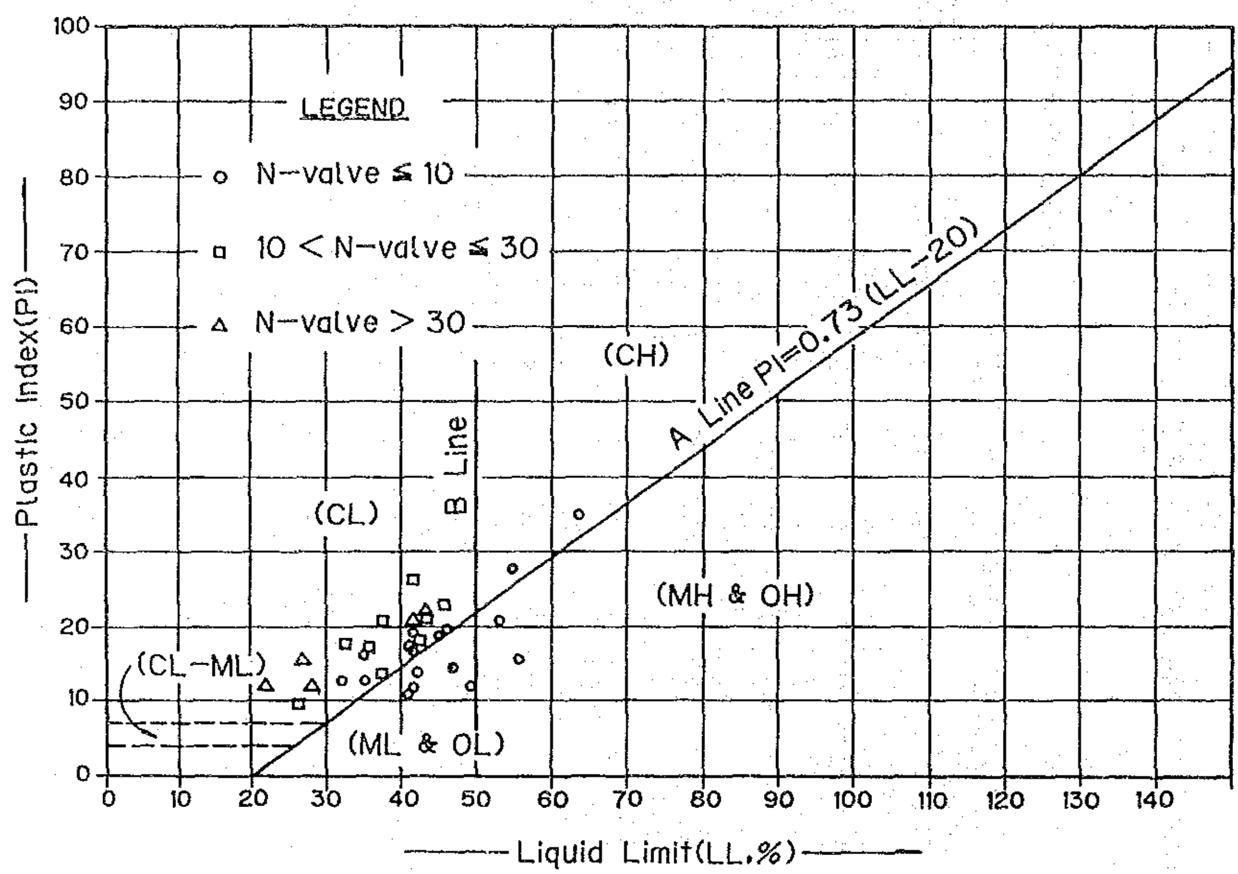


FIGURE 2-8 RELATIONSHIP BETWEEN PRE-CONSOLIDATION STRESS AND DEPTH

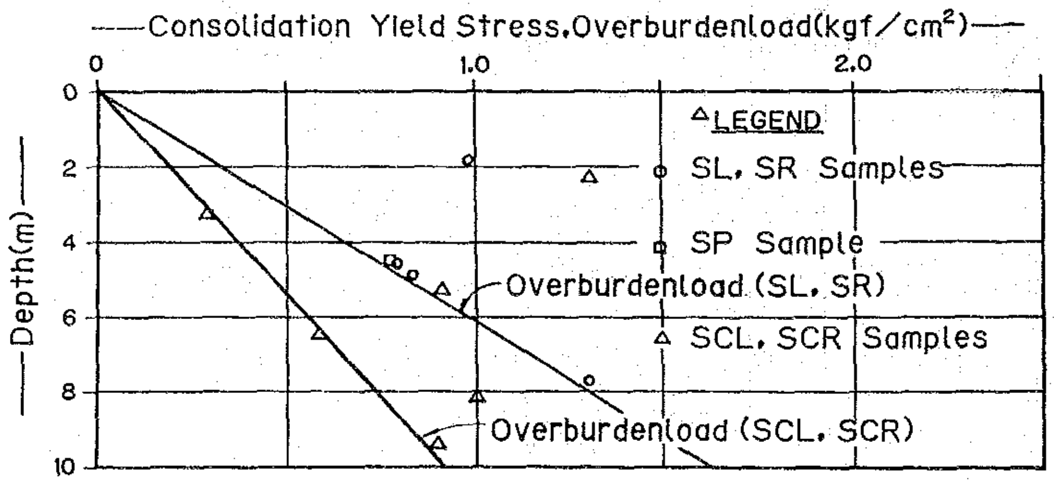


FIGURE 2-6 RELATIONSHIP BETWEEN PLASTIC INDEX, CONSISTENCY INDEX, LIQUIDITY INDEX AND DEPTH (SAMPLES BY BORE-HOLES)

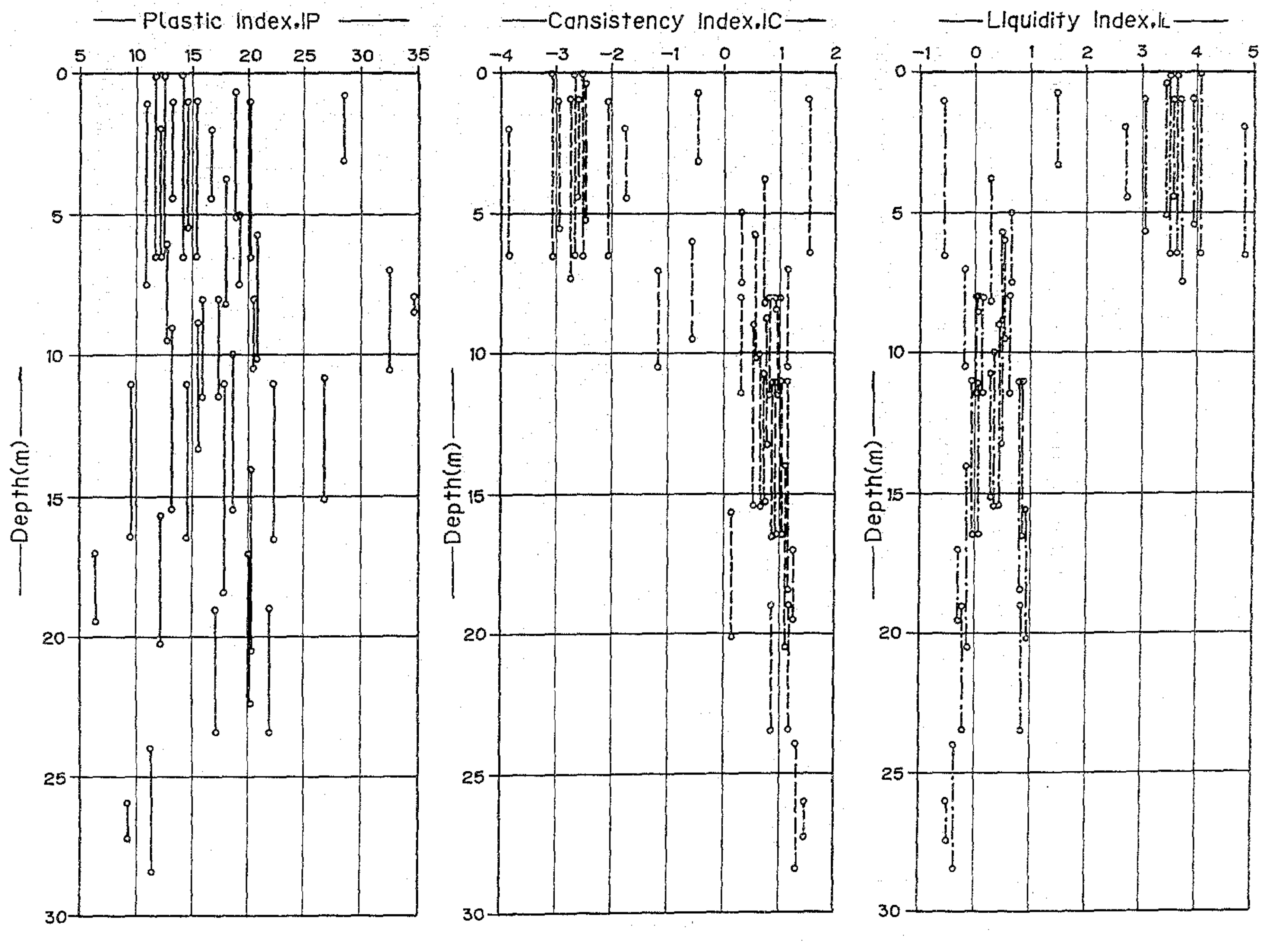


FIGURE 2-7 RELATIONSHIP BETWEEN SHEAR STRENGTH (COHESION) AND DEPTH

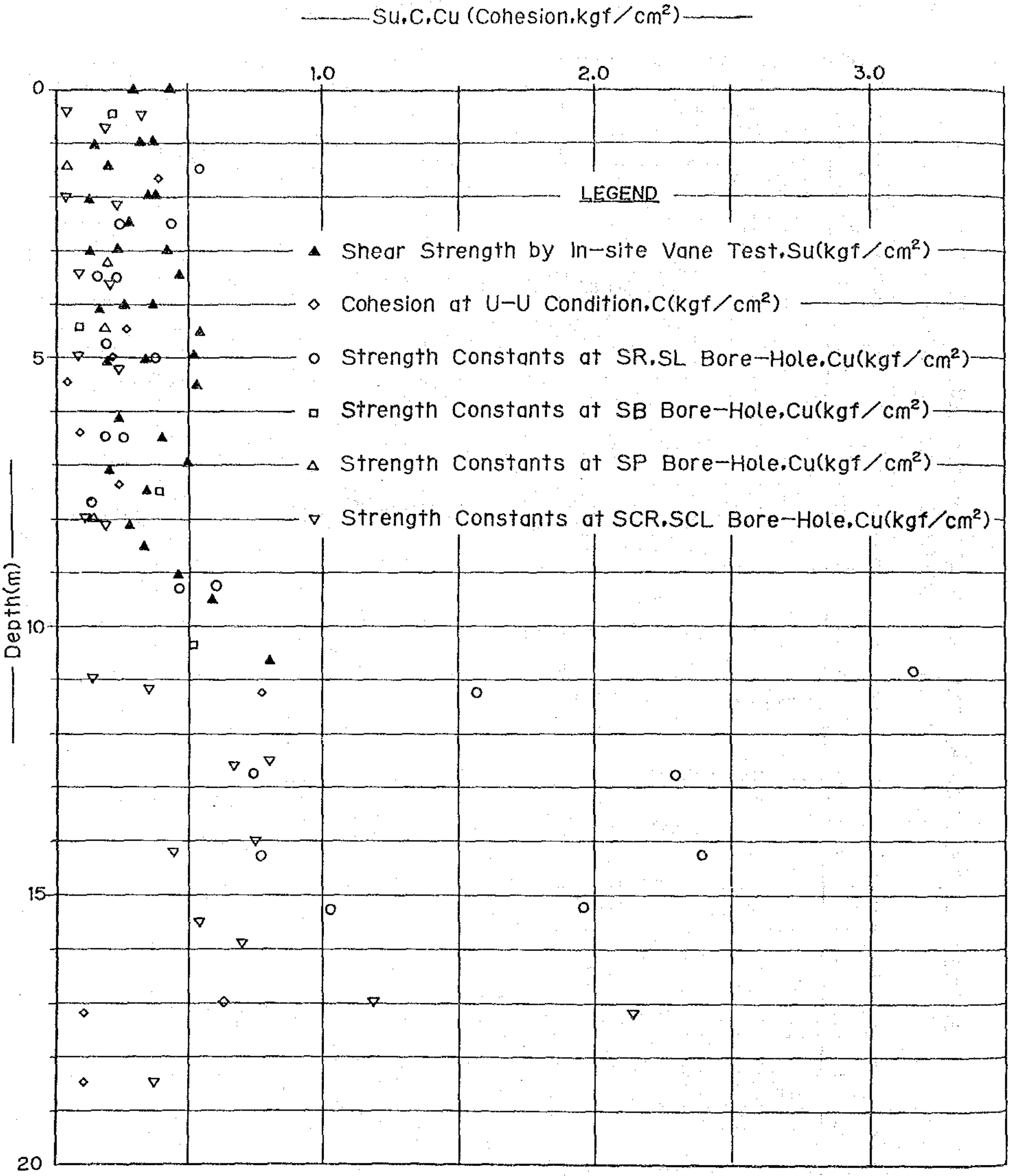


FIGURE 2 - 9 RELATIONSHIP BETWEEN N-VALUE AND COEFFICIENT OF DEFORMATION

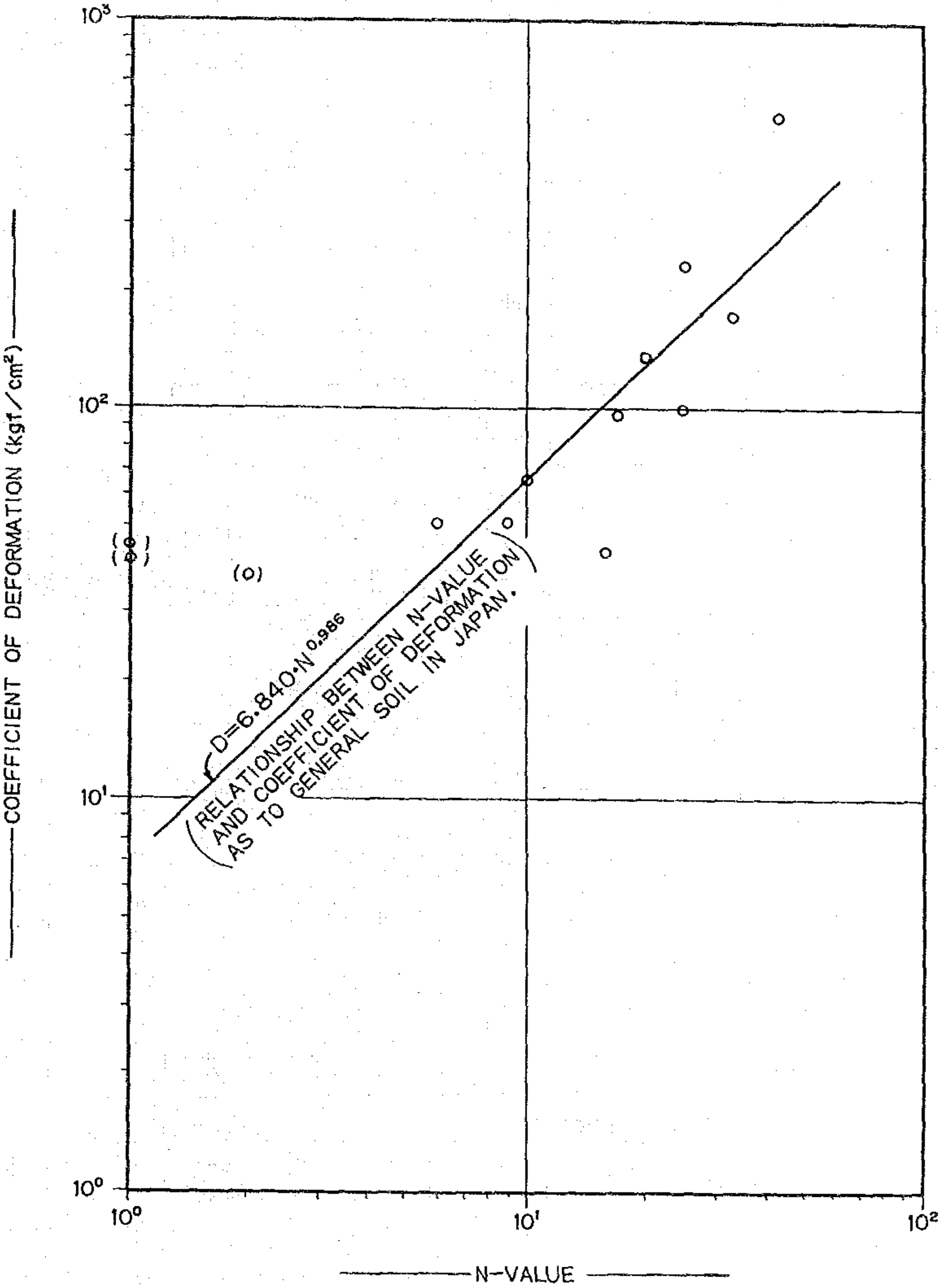
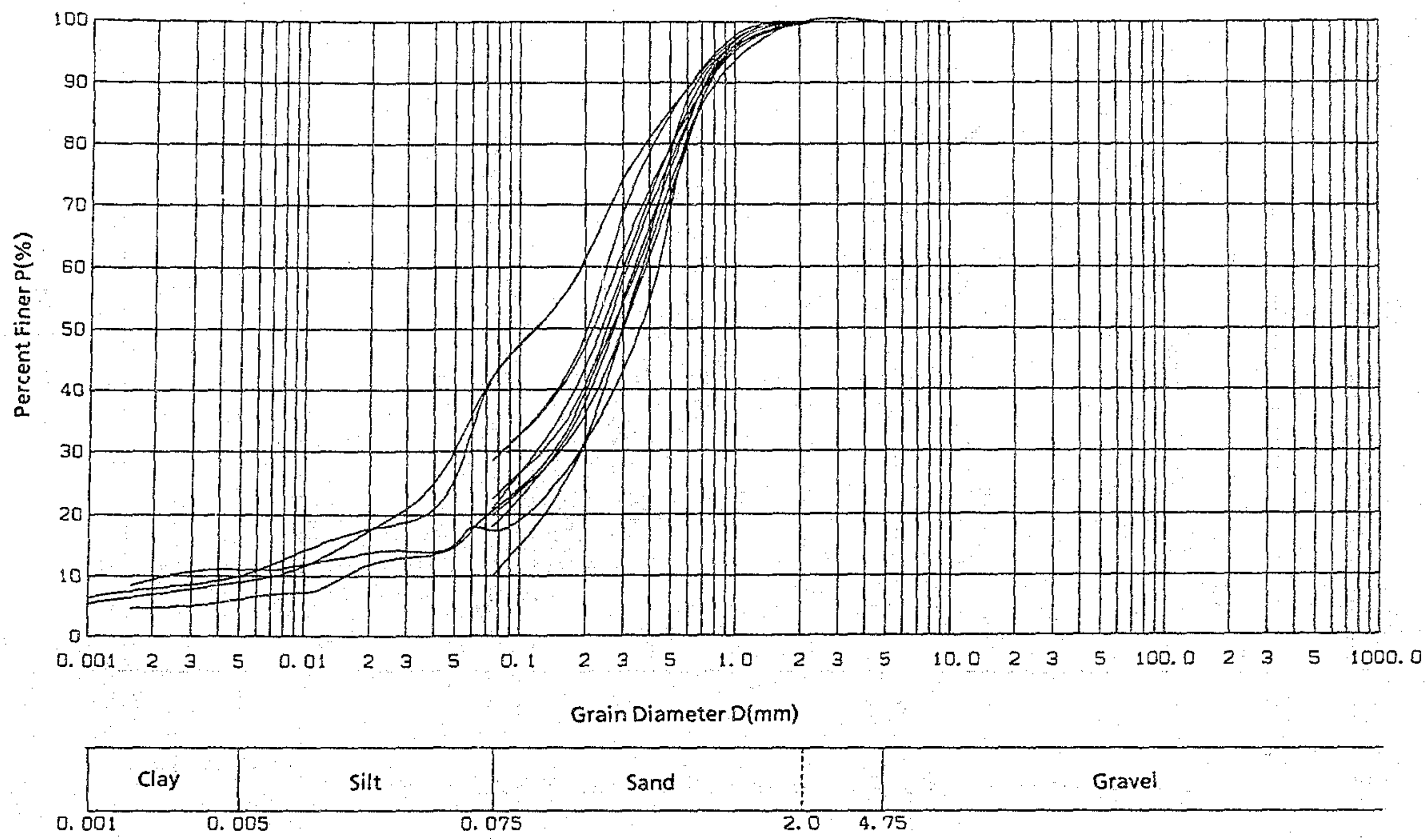


FIGURE 2 - 10 GRAIN SIZE DISTRIBUTION CURVE BY BORROW AREA SAMPLES



CHAPTER 3. DESIGN CRITERIA

3.1 General

The project facilities are designed according to the design information given in this chapter. The other design information necessary for the complete design of the project facilities can be found in the following design standards.

- Design standards for headworks, filldam, pump facilities and canal works in land improvement projects, established by Agricultural Structure Improvement Bureau, Ministry of Agriculture, Forestry and Fisheries of Japan.
- Japanese government ordinance for road structures and Japanese design standard for road bridges.
- Design standard of roads and standard drawings of road structures, established by the Highway Department of Thailand.

In designing the project facilities based on these design standards, the design standards of Thailand should be applied preferentially over the Japanese design standards.

3.2 General Design Information for Structure

1) Allowable Stresses of Construction Materials

a) Allowable Stress of Reinforced Concrete

Allowable Stress (kg/cm^2)		28 day Concrete Strength (kg/cm^2)		
		180	210	240
Bending Compressive Stress		81	94.5	108
Shear Stress	Beams	4	4.2	4.5
	Slabs	8	8.5	9
Bond Stress	Round Bar	7	7.5	8
	Deformed Bar	14	15	16
Bearing Stress		54	63	72
Structures to be applied		Others	Slabs, walls and piers of main structures	Slab of bridge

The modular ratio (modulus of elasticity of steel/modulus of elasticity of concrete) of 10 will be used for the design of the project facilities.

b) Allowable Stress of Plain Concrete

Allowable Stress (kg/cm^2)	28 Day Concrete Strength (kg/cm^2)	
	180	140
Bending Compressive Stress	45	35
Bending Tensile stress	2.5	2
Bearing Stress	54	42

c) Allowable Tensile Stress of Steel

- Deformed bar (SD30) $\sigma sa = 1{,}400\ kg/cm^2$
- Round bar (SR24) $\sigma sa = 1{,}200\ kg/cm^2$
- Structural steel (SS41) $\sigma sa = 1{,}200\ kg/cm^2$
- Steel sheet pile (SY30) $\sigma sa = 1{,}400\ kg/cm^2$

2) Loadings

a) Dead Loads

The dead-load weights are as follows;

- Reinforced concrete $\gamma c = 2.4\ t/m^3$
- Plain concrete $\gamma c = 2.2\ t/m^3$
- Water $\gamma w = 1.0\ t/m^3$
- Sea water $\gamma e = 1.03\ t/m^3$
- Dry earth $\gamma e = 1.6\ t/m^3$
- Wet earth $\gamma e = 1.8\ t/m^3$
- Saturated earth $\gamma e = 2.0\ t/m^3$
- Steel $\gamma s = 7.85\ t/m^3$

b) Live Loads

Structures on which heavy wheels pass over the side of the structure should be designed for the wheel loads, and where heavy wheels do not pass through the side of the structure, a live load of 300 kg/m^2.

c) Seismic Loads

The seismic loads are not considered for the design of the project facilities.

3) Bang Pakong Reservoir Plan

- Design flood	:	1,600 m^3/s
- Maximum Water Level (Max. W.L.)		
at the site of Diversion Dam	:	EL.2.40 m
at the site of Pumping Station	:	EL.2.50 m
- Normal Water Level (N.W.L.)	:	EL. 0.70 m
- Minimum Operating Level (Min. O.L.)	:	EL. - 1.30 m
- Active Storage	:	30 MCM

4) Sea Level

- High Water Level (H.W.L.)	:	EL. 1.30 m
- Low Water Level (L.W.L.)	:	EL.-1.00 m
- Highest High Water Level (H.H.W.L.)	:	EL.2.10 m
- Lowest Low Water Level (L.L.W.L.)	:	EL.-1.70 m

3.3 Diversion Dam

The general design considerations for the diversion dam are as follows;

a) No navigation lock is provided. Boats and ships are prohibited from passing through the diversion dam even when the gates open. The two jetties are planned on the left bank of Bang Pakong river. One is constructed upstream of the closure dam and the another is downstream.

b) No fish ladder is constructed in the Bang Pakong Diversion Dam Project. However, the approximate location of a fish ladder will be selected and preliminary design drawings will be provided for future implementation.

c) A floating net system is provided upstream from the diversion dam.

d) The O/M bridge will be designed for a live load of TL-20t and its width will be 5 meters or more for the passage of a truck crane for maintenance and repair work on the tide protection gates.

e) A stop log necessary for maintenance and repair work on the tide protection gates will be provided.

f) An emergency generator is provided for the operation of the tide protection gates even at the time of power failure.

3.4 Pumping Station

The general design considerations for the pumping station are as follows;

a) The discharge water level is determined as EL. 3.80 m by adding a head loss due to the increased length of the canal and a difference in water level in the transitional section, to the designated water level of EL. 3.70 m at the beginning of the main canal shown in the F/S report.

- Head loss due to an increased length of canal:

  = (increased length of canal) × (gradient of canal invert)

  = 600 m × (1 /12,000) = 0.05 m

- Difference in water level in transitional section:

  = (1 + fgc) × (velocity head)

  = $1.3 \times (0.815^2/(2 \times 9.8)) = 0.05$ m

  (fgc: coefficient of head loss due to change of canal section)

b) The floating net system will be provided in front of the entrance of the intake canal, and also trash racks will be provided in the intake canal.

c) The pump house will be designed to have enough space for disassembling and repairing the pump facilities in addition to the space for the main pumps, incoming/distribution facilities, office, etc. The area of the office should be enough for a staff of more than three. A bedroom is not required.

d) At least one of the main pumps should be designed to be operated even in a power failure.

e) The site of the pumping station includes the sites of the intake canal, pump house and discharge reservoir. The layout plan should take into account the space required for the above mentioned structures and for maintenance.

3.5 Road and Road Bridge

The general design considerations for the road and road bridge are as follows;

a) The width of the road will be 9 meters and road bridge will be 13 meters. The road section is composed of a 6 meter wide roadway and two shoulders of 1.5 meters wide each.

b) The road and road bridge should be designed for a design speed of 60 ~ 80 km/hr.

c) The road is planned to be paved with asphalt and the cross grade will be 3.5%.

d) Drainage ditches along the road are not required.

e) The road bridge is designed for a live load of TL-20t.

f) Because the passage of boats and ships under the road bridge is prohibited, it is not necessary to determine a clearance level.

g) Lighting facilities will be installed along the road and road bridge.

3.6 Buildings

The general design considerations for buildings are as follows;

a) The allowable stresses of the construction materials in building works are as shown in chapter 3.2 "General Design Information for Structure".

b) The wind load factors for various height zones are as follows;

less than 10 m high	50 kg/m^2
10 to 20 m high	80 kg/m^2
20 to 40 m high	120 kg/m^2

c) The design loads on the floor for various types of building are as follows;

garages, machine room	500 kg/m^2
office	300 kg/m^2
residence	150/m^2

d) Fire extinguishers will be provided in the buildings. For every 100 m^2 of floor area there will be one fire extinguisher. A floor area less than 100 m^2 will have at least one fire extinguisher.

CHAPTER 4. DESIGN OF DIVERSION CANAL

4.1 Location and Plane Figure of the Diversion Canal

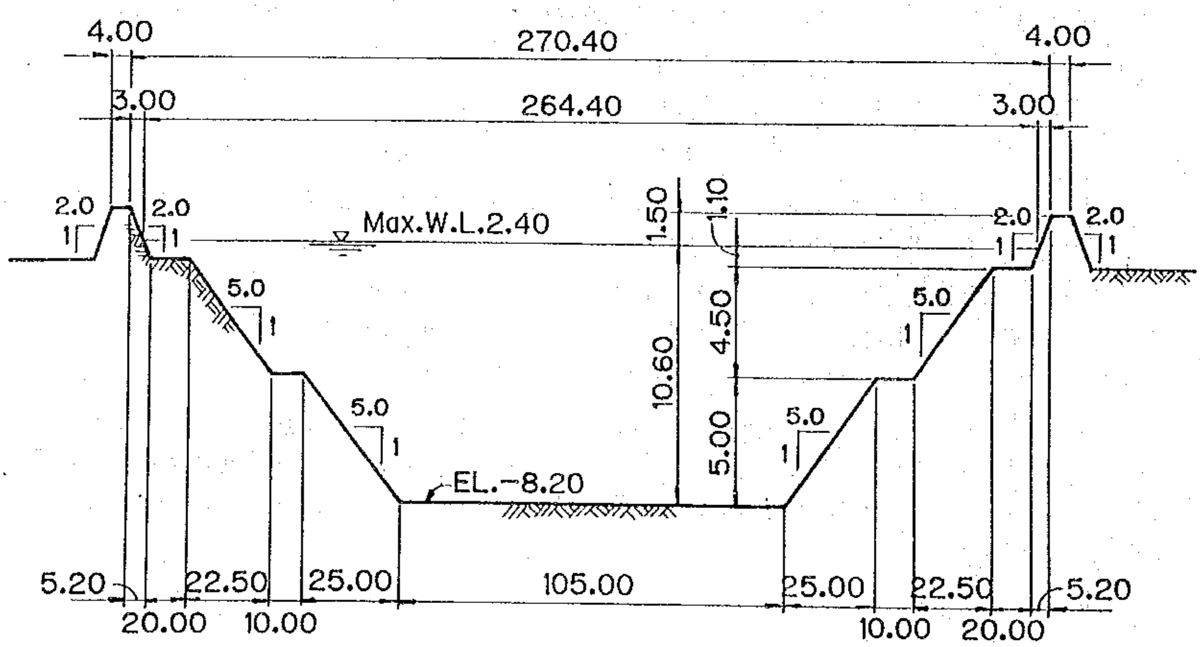
The location of the diversion canal was selected at a site about 71.0 km upstream from the estuary of the Bang Pakong river, so that the proposed canal to be constructed for making a short-cut of the meandering section of the river.

The straight portion of the diversion canal should be as long as possible so that both the diversion dam and a road bridge can be provided over the canal. The diversion canal shall be jointed with the Bank Pakong river at a 20 degree contact angle together with curve of 1,000 m radius which is about 4 times the canal width at both the up-and-downstream points, so that possible smooth in-and-outflow of the river discharge can be ensured.

4. 2 Typical Section of Diversion Canal

The design section of the canal shall be as the illustration in Figure 4-1, in consideration of the existing dimensions of the river section by 230 m water surface width, 11 m water depth, sectional area, and the result of the stability analysis for the excavated slope.

FIGURE 4 - 1 TYPICAL SECTION OF DIVERSION CANAL
(STA. 1 + 534.13)



Flow area	:	A	=	1/2(105.0 + 155.0)×5.0 + 1/2(175.0 + 220.0) × 4.5
				+ 1/2(260.0 + 264.4) × 1.1 = 1,827.17 m^2 > 1,750 m^2
Wetted perimeter	:	p	=	105.0 + (25.5 + 10.0 + 229.5 + 20.0 + 2.46) × 2
			=	266.82 m
Hydraulic radius	:	R	=	1827.17/266.82 = 6.848 m
Coeff. of roughness	:	n	=	0.023
Hydraulic grade	:	I	=	1/32,000
Velocity of flow	:	V	=	$1/0.023 \times 6.848^{2/3} \times (1/32{,}000)^{1/2} = 0.88$ m/s
Discharge	:	Q	=	1,827.17 × 0.88 = 1,608 m^3/s > 1,600m^3/s

4.3 Stability Analysis for Slope of Diversion Canal

The diversion canal will be constructed by excavating weak alluvial ground consisting of fine silt and clay materials. It is generally well-known that initial cohesion in such ground is decreased considerably due to looseness caused by unloading of the upper layers. Consequently, a successful stability analysis for slope excavation is required in consideration of the aforesaid initial decrease in cohesion of the materials.

1) Conditions of Analysis

The slip circle method shall be applied to the stability analysis for diversion canal slope in taking the representative section in the following conditions.

- Table 4-1 shows the constants of density, shearing strength, initial cohesion, internal friction angle for embankment materials for the diversion canal slope.

TABLE 4-1 INPUT CONSTANTS ON STABILITY ANALYSIS AT DIVERSION CANAL

Zoning		Depth (m)	Thickness (m)	Density (tf/m^3)			Shear Strength	
				γt *1	γsat *2	γ *3	C (t/m^3) *4	ϕ (°) *5
Existing Ground Materials	①	0 ~ 2.0	2.0	1.74	1.98	0.98	2.0	0.0
	③	2.0 ~ 4.0	2.0	1.47	1.48	0.48	2.0	0.0
	③	4.0 ~ 6.0	2.0	1.47	1.49	0.49	2.0	0.0
	④	6.0 ~ 8.0	2.0	1.47	1.53	0.53	2.0	0.0
	⑤	8.0 ~ 10.0	2.0	1.75	1.82	0.82	6.5	0.0
	⑥	10.0 ~ 12.0	2.0	1.75	1.88	0.88	6.5	0.0
	⑦	12.0 ~ 20.0	8.0	2.05	2.05	1.05	21.7	0.0
	⑧	20 m	-	2.05	2.05	1.05	-	-
Embankment Materials		-	-	1.77	1.83	0.83	0.8	20.0

Notes: *1 Wet density *2 Saturated density *3 Submerged density
*4 Initial cohesion *5 Internal friction angle

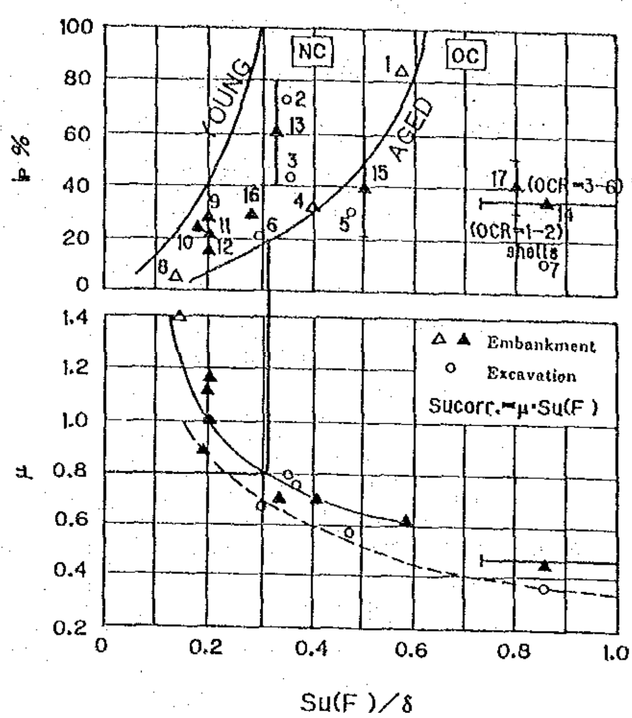
- The decreasing ratio of initial cohesion in the excavation works shall be obtained by the following equations, which are found in the Detailed Design Report by Irrigation Engineering Center of RID for the Model Infrastructure Improvement Works (1988).

$$Su = Su' \cdot \mu_A \cdot \mu_B$$
$$M_B = OCR^{-\alpha} = (\sigma_{vm}/\sigma_n)^{-\alpha}$$

where,
- Su : Decreased cohesion
- Su' : Initial cohesion
- μ_A : Coefficient A
- μ_B : Coefficient B
- OCR : Over consolidation ratio
- σ_{vm} : Pre-consolidation pressure
- σ_n : Effective normal stress after excavation

For reference, the value of μ_A is a corrected value of the shearing strength obtained by field vane test. The value of μ_A shall be 0.8 by the plasticity index (Ip) of the existing ground from Figure 4-2. For depths of more than 12 m, the vane shearing tests have not been carried out yet and shearing strength has been estimated μ_A at 1 from the results of soil mechanical test.

FIGURE 4-2 MODIFIED COEFFICIENT FOR SHEAR STRENGTH BY FIELD VANE TEST



Note

Nc ; Normally consolidated clay

OC ; Over consolidated clay

OCR ; Over consolidation ratio

YOUNG ; Young normally consolidated clay

AGED ; Aged normally consolidated clay

LEGEND

1 Bangkok (Eide & Holmberg, 1972)

2 Fiumicino (Calabresi & Burghignoli, 1977)

3 San Francisco Bay (Duncan and Buchignani, 1973)

4 Onsoy (Berre, 1973)

5 Kimola (Kankare, 1969)

6 Postgiro (Aas, 1979)

7 Malmo (Pusch, 1968)

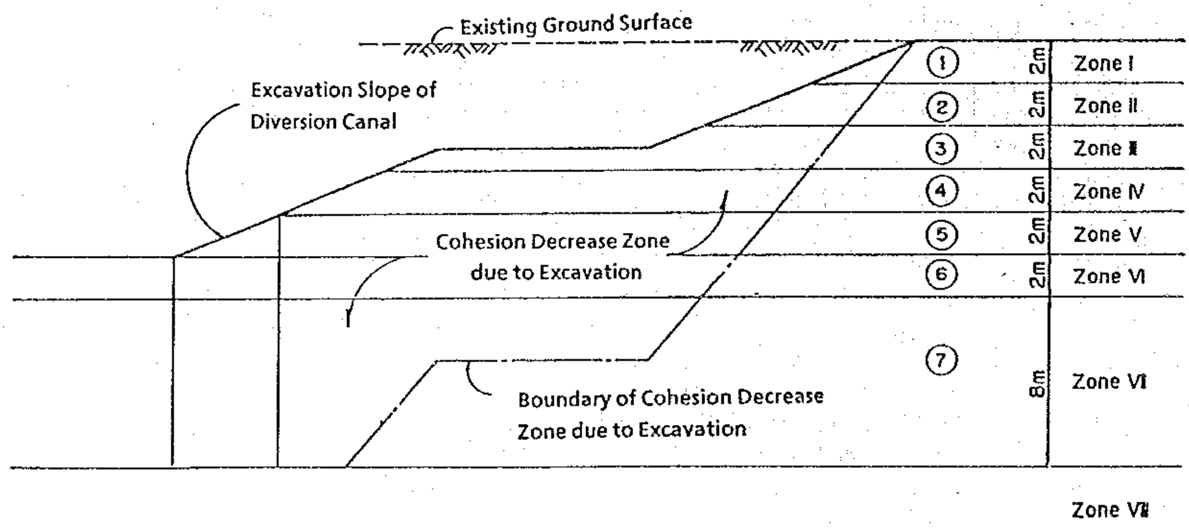
8 Ellingsrud (Aas, 1979)

9~17 MIT cases (Lascasse *et al*, 1978)
(15, 16, 17 no failure)

- The loosening range caused by excavation shall remain at twice as much as excavation depth, however, for the layer deeper than 20 m from the ground surface, influence of looseness shall not be taken into consideration, but firmness with N-value more than 40.

- Estimation of the cohesion decrease from excavation is made at each two-meter depth interval in considering variation of stress before and after excavation and the results are shown in Appendix 4. 3. And the cross-section model used for stability analysis is illustrated in Figure 4-3.

FIGURE 4-3 MODEL OF SLOPE AT DIVERSION CANAL



- The stability analysis is made of the excavation slope by the slip circle method. The following equation is used for the necessary calculation.

$$F.S = \Sigma [(N - U) \cdot \tan \phi + C \cdot l] / \Sigma T$$

where, F.S ; Factor of safety
N ; Normal force acting on slip circle of each slice
U ; Residual pore pressure acting on slip circle of each slice

ϕ	;	Internal friction angle of materials on slip circle of each slice
C	;	Cohesion of materials on slip circle of each slice
l	;	Arc length of slip circle of each slice
T	;	Tangential force acting on slip circle of each slice

- The following three water level conditions are taken for the stability analysis.

TABLE 4-2 CASES FOR THE STABILITY ANALYSIS

Case	Condition	Water Level
Case - 1	Constant	Highest high water level *1
Case - 2	Constant	Lowest low water level *2
Case - 3	Sudden drop	Drop from highest high water level to lowest low water level

Note: i) *1 : EL. 2.40, *2 : EL (-) 1.70
ii) Occurrence of residual pore pressure is taken into account for the Case 3.

2) Result of Stability Analysis

The calculations of stability analysis were carried out repeatedly with different slip circles to obtain the smallest value of the safety factor. The results of Analysis for the cases with slopes 1 : 4.0 and 1 : 5.0 are shown in Table 4-3.

The allowable safety factor taken in the stability analysis should be comprehensively determined taking into consideration the specific features of materials, input constants of materials, occurrence frequency of drawdown, important of facilities to be proposed, analysis method, etc.

In this study, the values of the safety factors for the respective cases are taken with an allowance of 0.2 for Case 1 and 2, and 0.1 for Case 3, while considering differences in drawdown frequency in all the cases. As for the results, the allowable safety factors are obtained as less than 1.20 for Case 1 and 2, while 1.10 for Case 3 respectively.

TABLE 4-3 RESULTS OF SLOPE STABILITY ANALYSIS OF DIVERSION CANAL

Case	Slope of Excavation						Water Level
	1 : 4.0			1 : 5.0			
	F.S.		F.Sa	F.S.		F.Sa	
Case - 1	1.738	>	1.20	1.880	>	1.20	H.H.W.L. 2.10
Case - 2	1.174	<	1.20	1.278	>	1.20	L.L.W.L. (-) 1.70
Case - 3	1.080	<	1.10	1.175	>	1.10	H.H.W.L. 2.10 → L.L.W.L. (-) 1.70

The safety factors resulting from this analysis are less than the allowable values for the case of excavation slope with 1 : 4.0 for Case 2 and Case 3, while more than the allowable factors with 1 : 5.0 for every case. Under these conditions, the analysis results show that the stable excavation slope of the proposed diversion canal should be taken as 1 : 5.0

4.4 Slope Protection Works

The slope protection works shall be designed in consideration of the erosion of dikes and the diversion canal by flood flows and waves. (Refer to Appendix 4.3.3)

4.4.1 Allowable Velocity

Allowable velocity was studied for such a variety of protection works as earth lining, sodding, and riprap works, and the results of E.W. Lane's experiment and field investigation were applied to the earth-lining canal, actual results for sodding, and those by Isbsh's formula for riprap works.

As a result, the allowable flow velocity for the earth lining works is in the range from 0.67 to 1.02 m/s, for sodding works in a range from 1.0 to 2.5 m/s, and for riprap works (30 cm dia.) by 2.68 m/s, respectively.

4. 4. 2 Design of Slope Protection Works

The flow velocity will be 0.88 m/s, when the design flood flows down. Amplitude of the waves caused by large-size boat with about 30 passengers will be in the range of 30 to 40 cm, while the one from wind will be 15 to 20 cm.

Velocity resulting from 40cm amplitude waves will be about 2.2 m/s. The allowable velocity of the proposed diversion canal by the respective construction method and the flow velocity in the diversion canal are shown in Table 4-4.

TABLE 4 - 4 ALLOWABLE VELOCITY

Podision		Allowable Velocity (m/s)			Velocity (m/s)	
		Earth Lin.	sodding	Riprap	Flow	Wave
Up and Down	Dike	0.80	1.0 ~ 2.5	2.68	0.88	-
	Upper	0.67	〃	〃	〃	2.2
	Lower	0.97	〃	〃	〃	-
Middle	Dike	0.84	〃	〃	〃	-
	Upper	0.67	〃	〃	〃	2.2
	Lower	1.07	〃	〃	〃	-

It can be learned from the above table that the dike protection shall be made with sodding works against flood flow and the upper part of the excavation is planned to provide riprap works against flood flow and waves, but the lower part of the excavated slope shall be earth-lined in considering safety against the flood flow and the least influence by waves.

CHAPTER 5. DESIGN OF DIVERSION DAM

5. 1 Location of Diversion Dam

In the Feasibility Study Report and the Basic Design Report, the proposed diversion dam site was selected at the straight section of the diversion canal, about 71 km upstream from the Bang Pakong River estuary.

5. 2 Elevation of Gate Sill and Other Major Parts of the Structures

5. 2. 1 Gate Sill Elevation

The gate sill elevation shall be designed so as to ensure a smooth discharge of flood water. The gate sill elevation is determined as EL (-)8.20 m, since the diversion canal bed elevation at the diversion dam site is EL. (-)8.18 m.

5. 2. 2 Gate Crest Elevation

The gate crest Elevation is designed at EL. 1.80 m taking into consideration H.W. L. 1.30 m together with a wave height of 0.5 m.

5. 2. 3 Pier Height

The pier crest elevation is designed at EL. 17.40 m based on a design flood discharge by Max. W.L. 2. 40 m together with freeboard ① of 1.50 m, gate height of 10.00 m, freeboard ② of 2.00 m, and crest slab thickness of 1.50 m.

5. 2. 4 Retaining Wall Crest Elevation

The crest elevation of the retaining wall is designed at EL.1.80 m in taking the high water level (sea level) by W.L. 1.30 m and freeboard by 0.50 m into consideration.

5. 3 Determination of Gate Span

In the Basic Design Report, the gate span was determined at span length 30 m with 5 gates, taking into account the magnitude of the design flood discharge, the gate manufacturing technique, as well as economic factors.

5. 4 Piers

The gate piers shall be the type of structure to ensure smooth gate operation as well as to be dynamically stable in permitting smooth flow of flood discharge.

In the Basic Design Report, the piers were designed to be 26.5 m in height, 19 m length, and 4.0 m in thickness, respectively.

5. 5 Apron and Riprap

Both the apron and the riprap shall be provided so that the river beds both up-and downstream of the gates can be protected from scouring by river discharge. The Basic Design Report determines that the upstream apron has a length of 13.0 m and thickness of 0.7 m, midstream apron 24.0 and 2.0 to 2.9m, and downstream apron 18.0 m and 1.1 m, respectively.

The riprap shall be have a length of 30 m for the upstream, while 80 m for the downstream, respectively.

5. 6 Retaining Wall

According to the Basic Design Report, the retaining wall should be 12.0 to 12.9 m in height. The retaining wall type shall be the counterfort wall type for the Project in view of its economy.

5.7 Gates

5.7.1 Flood Gates

The flood gates are such a large type that have 30 m in span length and 10 m in gate leaf height for three units. And the gates shall be of shell roller type with two units of motors and also drams, respectively.

5.7.2 Regulating Gates

The upper gates leaves of the regulating gates shall have span lengths of 30 m and leaf heights of 3.1 m for two units so that the gates can control the water level of the reservoir in ordinary conditions of the water level and drought as well. The double leaf gates are inferior in economy to the flap gates, but highly reliable in operability and have stable structure with high rigidity. In such view, the double leaf gates are adopted by the Project especially, the double leaf normal type gates which have hydraulic advantages.

5.7.3 Hoist House

The hoist house is designed so that the hoisting and electric equipment and facilities can be protected from exposure to the sunshine, rain water and wind that are obstructions to effective operation of the gates. The hoist house shall have the scale where the length in the thalweg direction is 12.5 m, in dam axis direction 10.0 m, and in height 4.5 m. The hoist house shall be constructed of reinforced concrete, and concrete blocks will be used for walls but not columns.

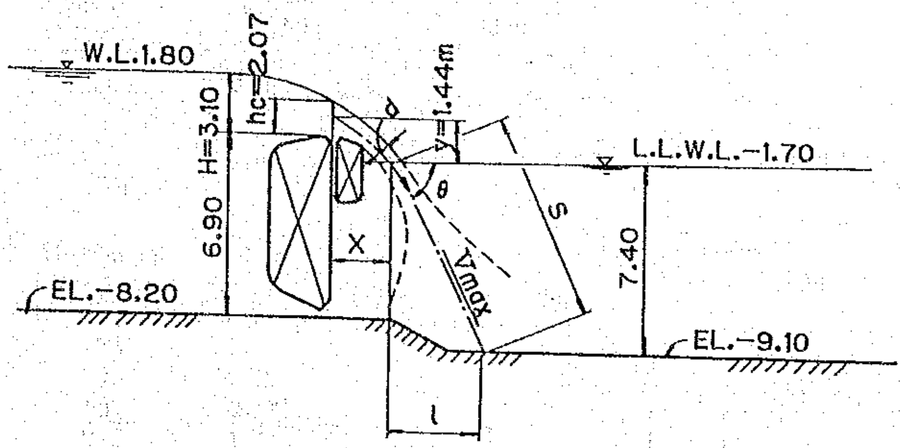
5.7.4 Operation and Maintenance (O & M) Bridge

The O & M bridge shall be 5.0 m in width allowing 10-ton truck cranes to pass across to install stop logs. The bridge shall have a clear span of 30.0 m and span length of 32.60 m by 5 spans in taking into account the gate span length. The bridge shall be constructed with prestressed concrete of hollow box type from the viewpoint of economy, ease of O & M work such as painting, etc. and comparatively low beam height.

5.8 Hydraulic Analysis for Riprap

5.8.1 Discharge through Gates

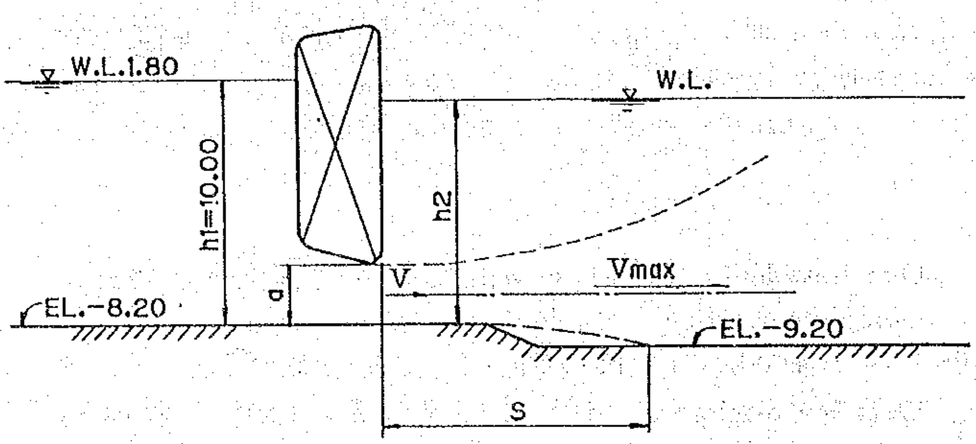
1) Upper Gate Leaf of Regulating Gate



- Overflow depth ; H = W.L. 1.80 - EL (-)1.30 = 3.10 m
- Complete overflow coefficient;
  $C = 1.838\,(1 + 0.0012/3.10)\,(1 - \sqrt{3.10/28.30}\,/10.0) = 1.783$
- Overflow discharge; $Q = 1.783 \times 28.3 \times 3.10^{3/2} = 275\ \mathrm{m^3/s}$
- Critical depth; $hc = 2/3H = 2/3 \times 3.10 = 2.07$ m
- Vertical distance of dropping water vein;
  $y = 1/2 \times 2.07 + 0.40 = 1.44$ m
- Horizontal distance of dropping water vein;
  $x/H = 1.155\,\{(y/H) + 0.333\}^{0.500}$
  $x = 1.155 \times 3.10\,\{(1.44/3.10) + 0.333\}^{0.500} = 3.19$ m
- Velocity of center of water vein;
  $V = \sqrt{2 \times 9.8 \times 3.50} = 8.28$ m/s
- Thickness of water vein; $d = 275/28.3 \times 8.28 = 1.17$ m
- Inclination angle of water vein;
  $\theta = \tan^{-1}(1.500 \times 3.19/3.10) = 0°\,59'$
- Velocity of water vein;
  $s \leqq 5.82\,d$: $V\max = V$
  $s > 5.82\,d$: $V\max = 2.41V\sqrt{d/s}$

Penetrating Dis. s (m)	Horiz. Dis. ℓ (m)	Water Depth h (m)	Velocity V max (m/s)
10.0	10.0	0.2	6.38
30.0	30.0	0.5	3.94
50.0	50.0	0.9	3.05
100.0	100.0	1.7	2.16
200.0	200.0	3.4	1.53
300.0	300.0	5.1	1.25
430.0	429.9	7.4	1.04

2) Flood Gate



- Water depth, upstream; h_1 = WL.1.80 - EL(-) 8.20 = 10.00 m
- Opening distance of gate; a (m)
- Discharge; $Q = C_1 \cdot a \cdot B \cdot \sqrt{2gh_1}$ (m/s)
- Width of discharge; $B = 30.0 \times 3 = 90.0$ m
- Water depth, downstream; h_2 (m)

Opening a (m)	Depth of D. h_2 (m)	h_1/a	h_2/a	Coeff. of D. C_1	Discharge Q (m^3/s)	Velocity V (m/s)
0.30	9.15	33.33	30.50	0.20	76	2.81
0.60	9.20	16.67	15.38	0.18	136	2.52
0.90	9.25	11.11	10.28	0.16	181	2.23
1.50	9.30	6.67	6.20	0.14	265	1.96
2.10	9.35	4.76	4.45	0.13	344	1.82
3.00	9.40	3.33	3.13	0.11	416	1.54
3.90	9.45	2.56	2.42	0.09	442	1.26
4.80	9.45	2.08	1.97	0.075	454	1.05

Opening a (m)	Discharge Q (m^3/s)	Velocity V (m/s)	Vel. of Apron Vmax (m/s)	Vel. of Riprap V'max (m/s)	Remarks
0.30	76	2.81	0.67	0.37	· S is 31 m at Apron
0.60	136	2.52	0.84	0.47	· S is 100 m at Riprap
0.90	181	2.23	0.92	0.51	
1.50	265	1.96	1.04	0.58	
2.10	344	1.82	1.14	0.64	
3.00	416	1.54	1.15	0.64	
3.90	422	1.26	1.08	0.60	
4.80	454	1.05	1.00	0.55	

5. 8. 2 Design of Riprap Works

The velocity of discharge to be released through the gates will reach the maximum when the river discharge is 275 m^3/s under control of water level in the reservoir by operating the upper gate leaves. In cases, where the maximum velocity will be 6.83 m/s at the immediate downstream of the gates, 3.94 m/s at the downstream end of the apron, and 2.16 m/s at the downstream end of the riprap works, respectively. The water vein in these cases, however, will not reach the river bed at a depth of 1.7 m. It is about 430 m downstream from the gates that the water vein can reach the river bed, where the flow velocity will be 1.04 m/s in the river bed. Consequently, the downstream riprap works will not be required for water release through the upper gates of the regulating gates.

In other respects, the flood gate operation for minor floods or flash floods will result in the maximum velocity of 2.81 m/s in the immediate downstream area from the gates, 1.15 m/s at the downstream end of the apron and 0.64 m/s at the downstream end of the riprap, respectively. This indicates that there will be no fear of scouring, downstream of the riprap works in considering the velocity below the allowable velocity of 1.0 m/s for the river bed.

5.8.3 Design of Revetment Works

The extent of revetment works is studied to protect the river bed from scouring due to the velocity of water released through gate operation.

As a result of the analysis on the regulating gate operation, it is found that the velocity caused from water release from the upper gate has no effect causing scouring. The maximum velocity resulting from the flood gate operation is estimated at 1.15 m/s at the downstream end of the apron, and 0.95 m/s at a point 46 m downstream from the gates. In view of the fact that the allowable velocity of the river protection is 1.0 m/s, the revetment works shall cover an extent up to 10 m downstream of the road bridge, considering the velocity below the allowable velocity of 1.0 m/s for the river bed.

5.9 Pier Stability Analysis

5.9.1 Design Conditions

1) Water level

a) Design max. water level : Max. W.L. 2.40 m

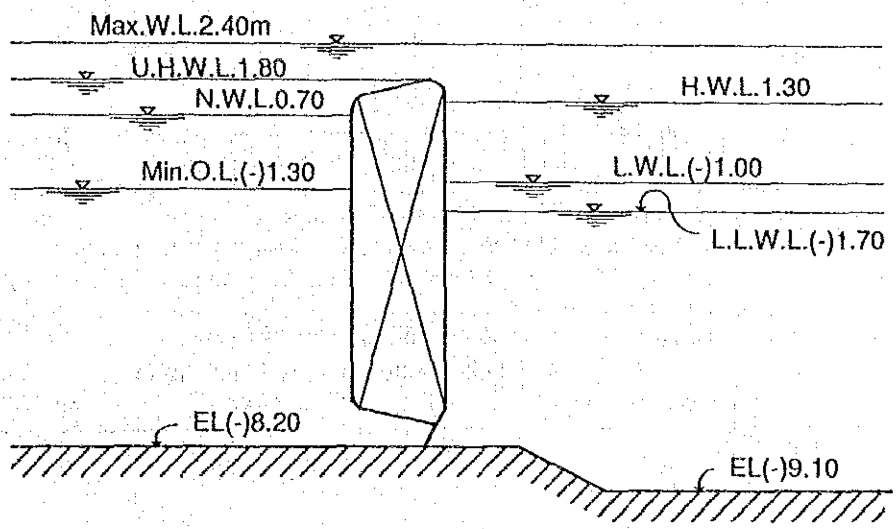
b) Upstream water level

- High water level : U.H.W.L 1.80 m
- Normal water level : N.W.L. 0.70 m
- Minimum water level : Min. W.L. (-)1.30 m

c) Downstream water level

- High water level : H.W.L 1.30 m (Wave height 0.50 m)
- Low water level : L.W.L. (-) 1.00 m
- Lowest low water level : L.L.W.L. (-)1.70 m

Upstream Downstream



2) Load Conditions

The gate weight and reaction of the O & M bridge, which act upon the pier can be shown as follows.

Gate Weight

Load	Unit	Flood Gate	Reguta. Gate	Remarks
Gate Weight	t/set	300	350	
Hoist Weight	〃	50	100	
Operation Load	〃	700	800	Increase gate weight

Reaction O & M Bridge

Reaction	Unit	Center-Pier	Abut-Pier	Remarks
Reaction to Dead Load	t/set	320	160	
Reaction to Live Load	〃	100	50	
Reaction tp Wind Load	〃	20	10	

3) Cases of Stability Analysis

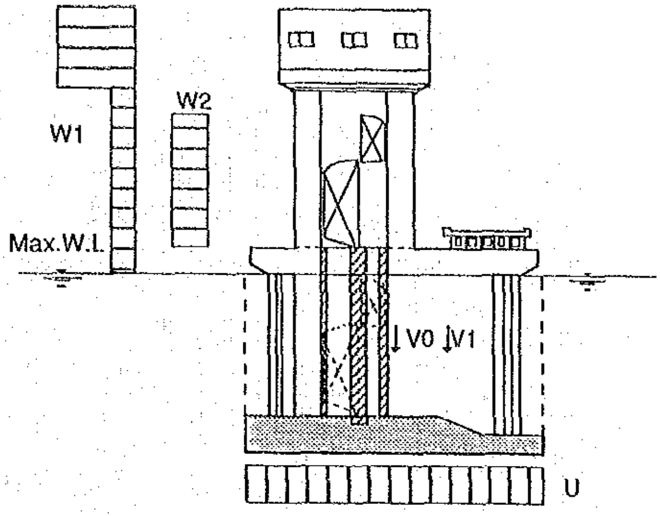
Case	Direction	Gate	Condition	Water Level		Remarks
				Upstream	Downstream	
1	Thalweg	Open	Max. flood	Max. W.L. 2.40 m		
2	〃	Close	Minor flood	U.H.W.L. 1.80 m	L.L.W.L. (-) 1.70m	
3	〃	〃	Spring tide	Min O.L (-)1.30 m	H.W.L. 1.30 m	Wave height 0.5 m
4	〃	Open	Construction	-	-	
5	Dam center	Close	Normal	U.H.W.L. 1.80 m	H.W.L. 1.80 m	
6	〃	〃	〃	Min O.L.(-)1.30m	L.L.W.L. (-)1.70 m	
7	〃	Open	Construction	-	-	

5.9.2 Piers of Analysis

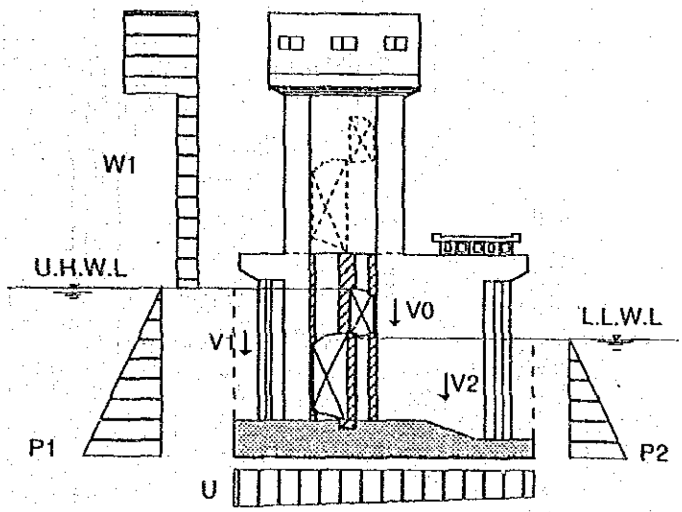
The stability analysis shall be made for the following piers.

1) Center pier to support the flood gate and the regulating gate.
2) Abut pier to support the flood gate on one side only.

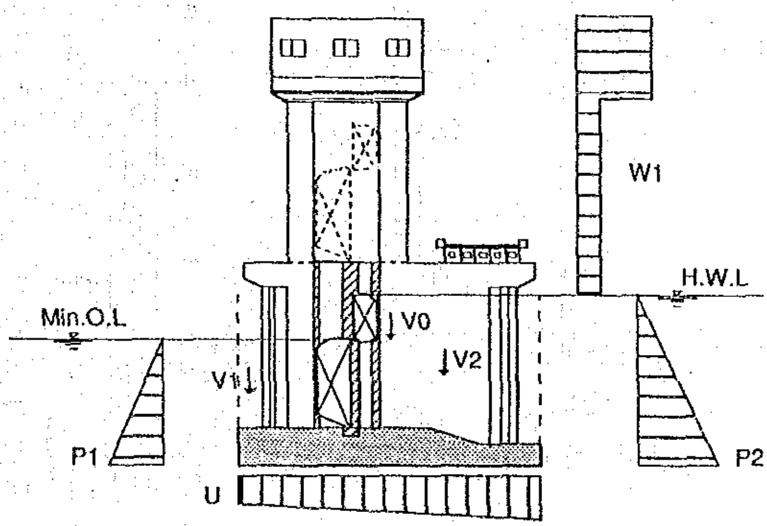
Case 1

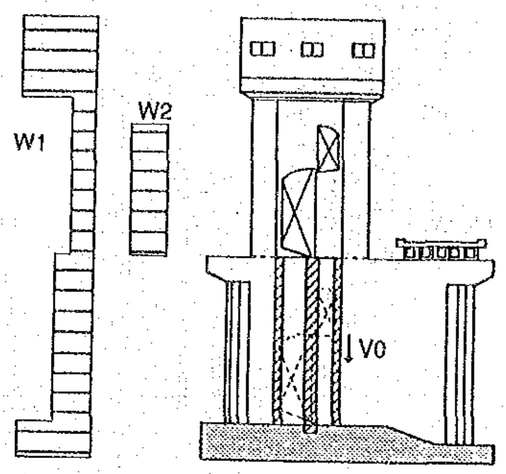
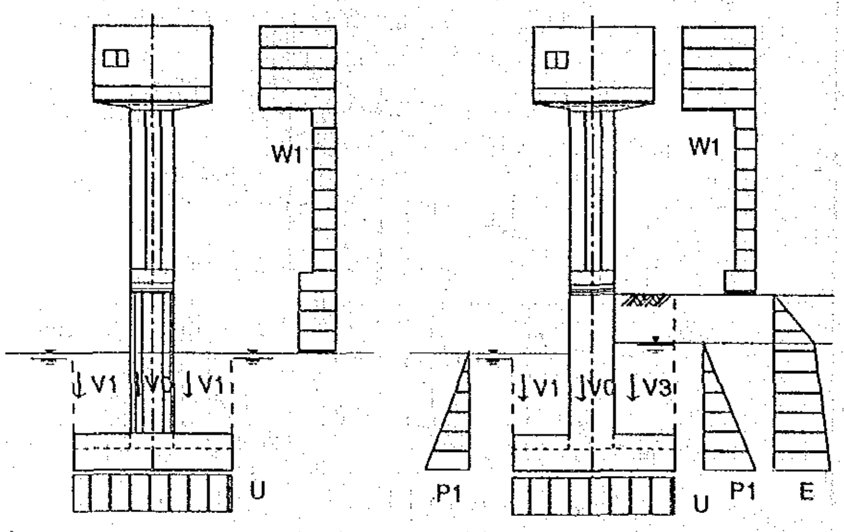
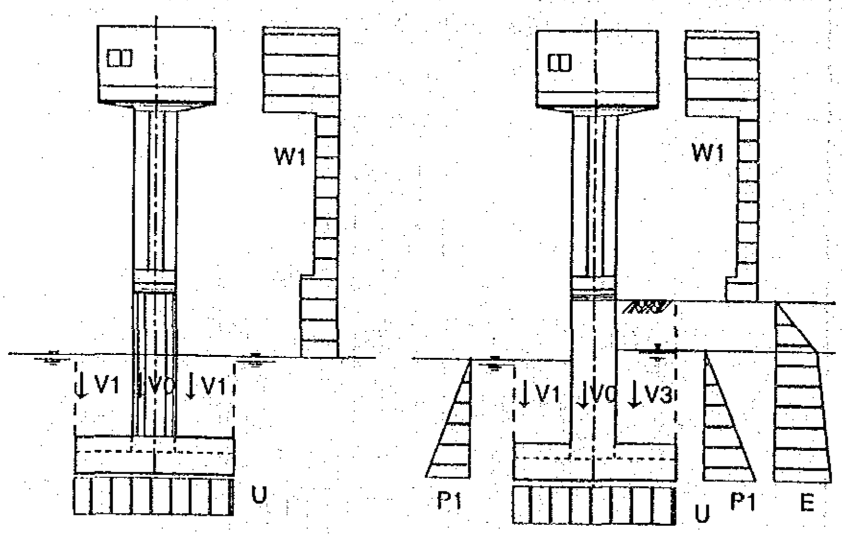


Case 2



Case 3



Case 4	
Case 5	
Case 6	

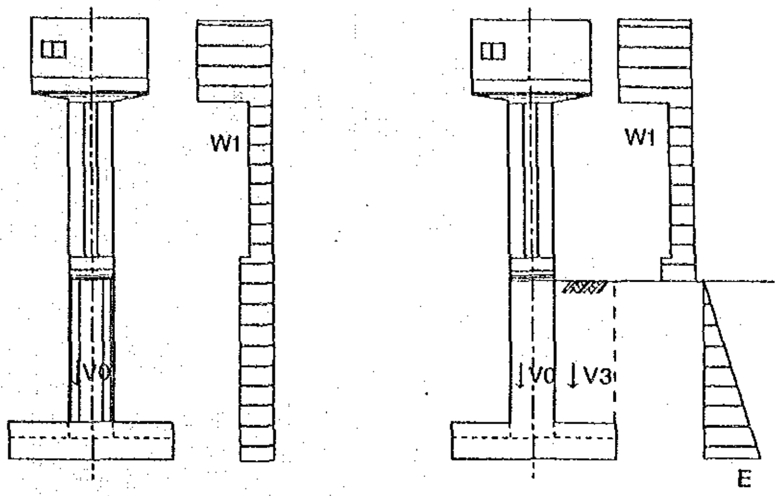
Case 7	
Note	V_0 ; Dead load pier, gate and bridge $V_{1,2}$; Dead load water V_3 ; Dead load water and earth $P_{1,2}$; Water pressure P_3 ; Activing earth pressure W_1 ; Wind pressure pier, hoist house, and Bridge W_2 ; Wind pressure gate U ; Buoyancy or uplift

FIGURE 5 - 1 ELEVATION OF PIER

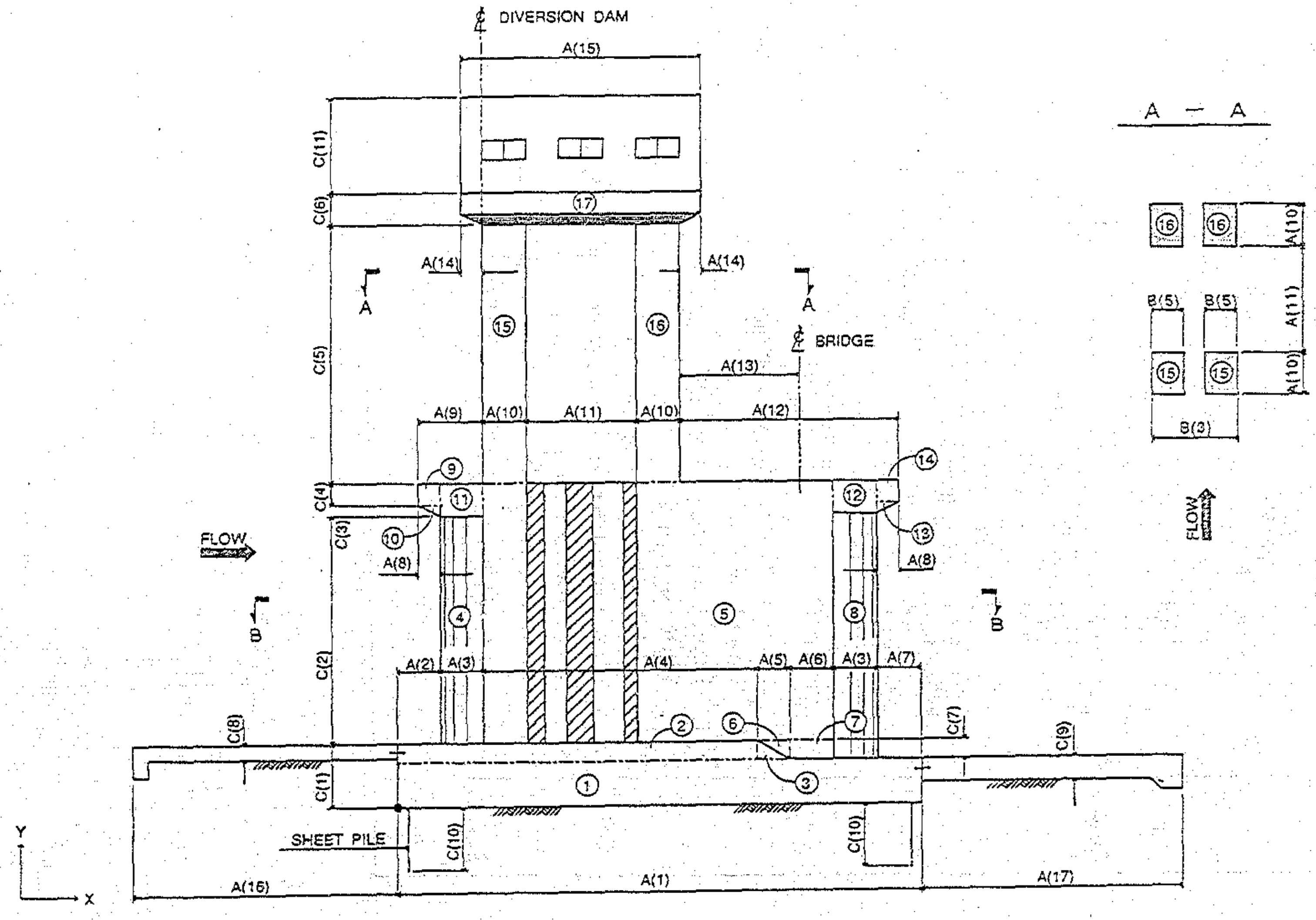
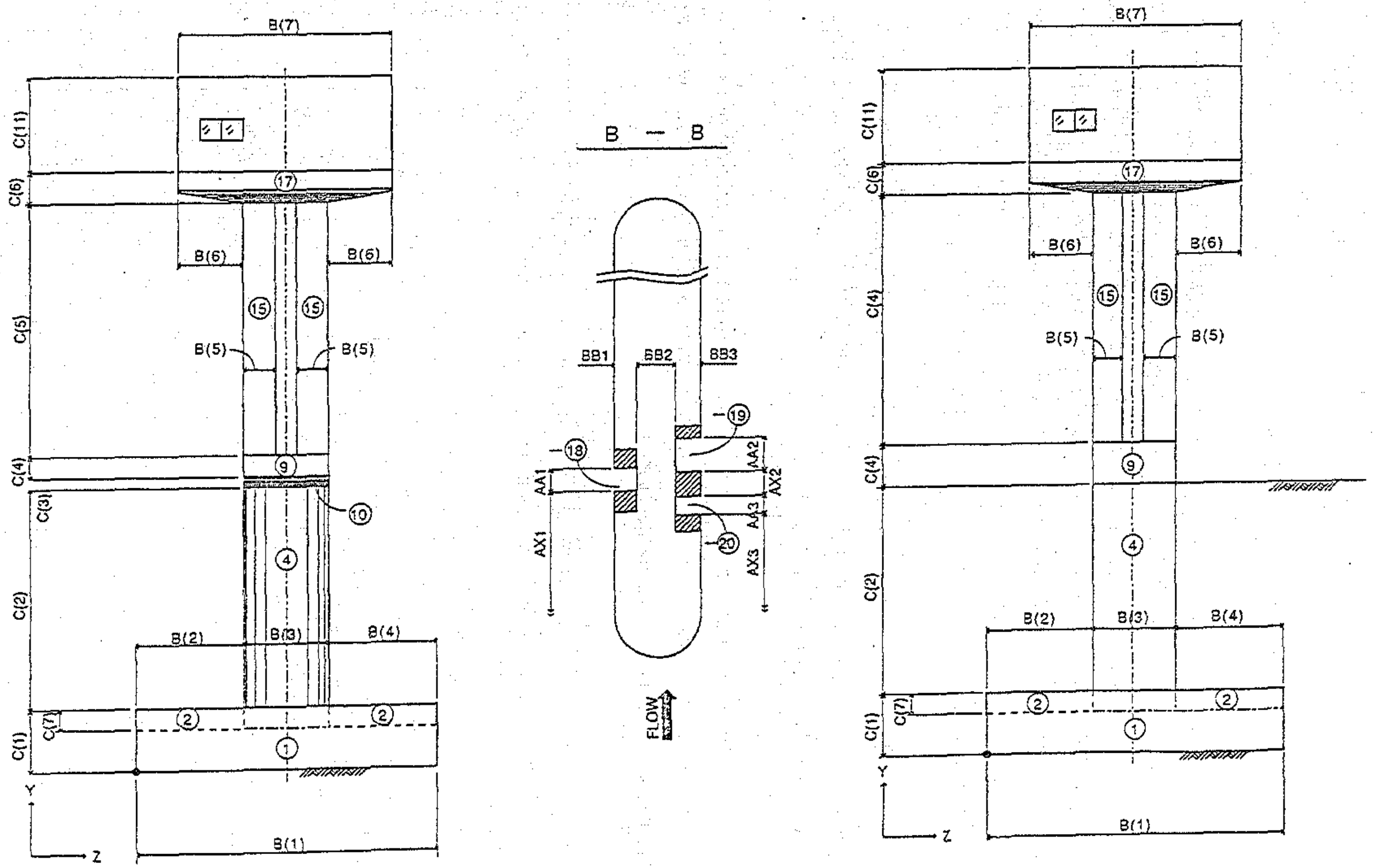


FIGURE 5 - 2 SECTION OF PIER



5. 9. 3 Results of Stability Analysis

The results of the pier stability analysis are as shown in Table 5-1. The safety factors of the center pier as regards sliding will become 3.75 at the maximum in the thalweg direction with gates closed, while 1.5 at the minimum with gates opened in the spring tide, respectively. The eccentric distance along the dam center direction against an overturn of the dam will be 1.52 m $<$ 2.00 m when the gates are closed due to the river water level reaching 1.80 m. The maximum soil reaction will be 37.82 tons/m^2.

The stability of the abut piers against sliding will be 3.26 $>$ 1.5, and the eccentric distance of safety will become 1.38 m $<$ 2.00 m and the maximum soil reaction will be 43.73 t/m^2.

Both the center and abut piers require the provision of pile foundations because the soil reaction ranges widely from 37.2 to 43.73 t/m^2 with a large N-value of 10 to 15 in the clay silt layers.

The pier bottom slab shall be 24 m in the thalweg direction and 12 m in dam center direction in considering the pile layout and upper figure of the piers.

TABLE 5-1 CALCULATION RESULTS OF PIER STABILITY

Pier	Case	Stabilizing moment		Overturning moment		Against sliding		Against overturning		Soil reaction	
		V (t)	V · x (t · m)	H (t)	H · y (t · m)	Factor of safety Fs	Fsa	e (m)	B/6 (m)	Q1 (t/m^2)	Q2 (t/m^2)
Center-pier	1	6894.91	87405.31	118.30	2427.00	34.97	> 1.5	0.32	< 4.00	25.88	22.00
	2	6830.02	85434.44	1120.51	8021.92	3.66	> 1.5	(-) 0.67	< 4.00	19.77	27.66
	3	6919.96	82524.56	1108.27	7887.77	3.75	> 1.5	1.21	< 4.00	31.32	16.73
	4	6168.05	79778.81	162.94	3253.70	22.71	> 1.5	0.41	< 4.00	23.59	19.24
	5	6194.35	28802.20	43.25	1029.51	85.94	> 1.5	1.52	< 2.00	37.82	5.20
	6	6920.72	41673.36	59.03	1201.66	70.35	> 1.5	0.15	< 2.00	25.86	22.20
	7	6168.05	37230.63	90.86	1406.34	40.73	> 1.5	0.19	< 2.00	23.47	19.36
Abut-pier	1	6435.89	81125.00	72.30	1551.74	53.41	> 1.5	0.36	< 4.00	24.38	20.31
	2	8097.98	100331.75	686.96	4852.20	7.07	> 1.5	(-) 0.21	< 4.00	26.65	29.59
	3	8068.75	94161.37	682.92	4780.27	7.09	> 1.5	0.92	< 4.00	34.48	21.55
	4	8030.28	100890.37	93.84	2056.49	51.34	> 1.5	0.31	< 4.00	30.03	25.74
	5	7458.09	41277.91	1372.96	6801.48	3.26	> 1.5	1.38	< 2.00	43.73	8.06
	6	8830.37	61149.91	1532.03	6278.74	3.46	> 1.5	(-) 0.21	< 2.00	27.38	33.94
	7	8565.81	62137.97	1259.75	6767.30	4.08	> 1.5	(-) 0.46	< 2.00	22.84	36.64

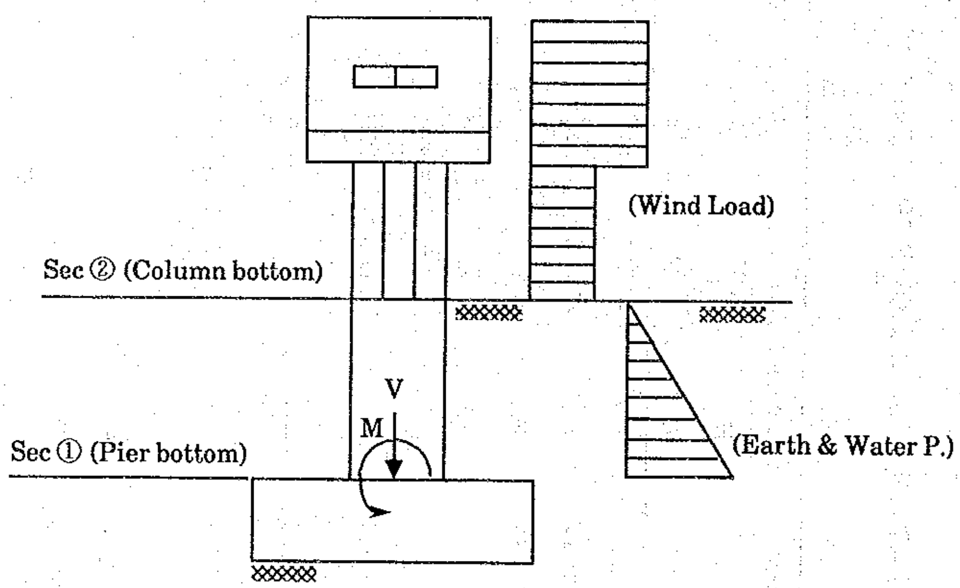
5. 10 Structural Calculation of Piers

5. 10. 1 Piers

1) Analysis Sections

The center and abut piers shall be designed with a cantilever receiving axial force and bending moment with the following two sections.

For further references, the calculation for the steel reinforcing bars required and the analysis on stress shall be made on the basis of the unit width by 1.0 m, and the load per unit based on the effective length with blocks out.



2) Water Level

The water levels for making structural calculations shall be taken as the combinations giving the most adverse influence on both piers.

- Upstream · · · · Low water level: Min. O.L. (-) 1.30
  Remaining water level: W.L. 0.80 m

- Downstream · · · Lowest low water level: L.L.W.L. (-) 1.70 m
  Remaining water level: W.L.0.30 m

3) Results of Structural Calculation

The results of the structural calculation are as shown in Table 5-2.

For both piers, since a large axial force works on their lower parts while bending moment works only a little, no tensile strength will occur. And the necessary quantity of the reinforcements can be estimated to a minimum in quantity as D25 @250 As = 19.64 cm^2/m.

Contrarily, there will occur tension stress on the upper part of the piers due to having a column structure with four posts. The quantity of reinforcements, consequently, will be able to be designed with D28 @125 (double) As = 98.56 m^2/m.

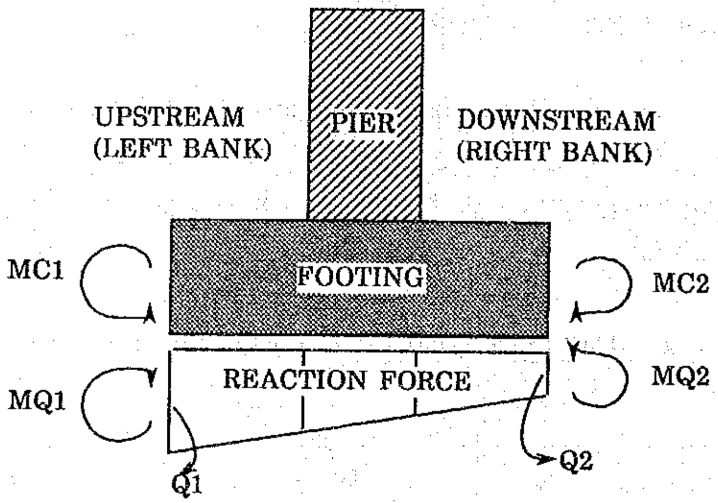
TABLE 5-2 RESULTS OF STRUCTURAL CALCULATION FOR PIERS

Item			Center Pier		Abut Pier	
			sec ①	sec ②	sec ①	sec ②
Dimensions	b	(cm)	100.00	100.00	100.00	100.00
	h	(cm)	400.00	140.00	400.00	140.00
	d	(cm)	390.00	125.00	390.00	125.00
Moment	M	(t·m)	174.78	128.06	254.41	154.31
Vertical Force	V	(t)	446.40	54.27	248.80	22.67
Horizontal Force	H	(t)	6.21	4.07	43.24	4.07
Req. Reinf.	As	(cm^2)	0.00	62.61	0.00	91.19
Reinf.	Dia.	(mm)	D25	D28	D25	D28
	Pitch	(mm)	@250	@125 (×2)	@250	@125 (×2)
	As	(cm^2)	19.64	98.56	19.64	98.56
Stress	σc	(kg/cm^2)	—	50	—	58
	σs	(kg/cm^2)	—	904	—	1287
	τ	(kg/cm^2)	—	0.3	—	0.3
Allowable Stress	σca	(kg/cm^2)	94.5	94.5	94.5	94.5
	σsa	(kg/cm^2)	1400.0	1400.0	1400.0	1400.0
	τa	(kg/cm^2)	4.2	4.2	4.2	4.2

5. 10. 2 Pier Bottom Slab

1) Calculation Method

The structural calculation of the pier bottom slab can be made on the cantilever as shown in the following illustration and based on the results of the soil reaction obtained from stability analysis of the piers.



2) Results of Structural Calculation

The results of the structural calculation are shown in Table 5 - 3 and - 4. The center pier shall be designed with 1/4 of the reinforcement in quantity required for the main bars as distributing bars, since the bending moment and shear strength are small in the thalweg direction.

The required quantity of the reinforcement shall be designed by D25 @125 (double) As = 78.56 cm^2/m in the dam center direction, while in the downstream D25 @125 (three fold) As = 117.84 cm^2/m, since the bottom slab thickness is 200 cm, and a stirrup by bars shall be designed in the thalweg direction with shear stores by τ = 6.2 kg/cm^2 > 5.4 kg/cm^2.

For the abut piers, the required quantity of the reinforcement shall be designed with 1/3 of the main bars as distribution bars in the thalweg direction because of small bending moment and sear strength.

For the upstream section, the reinforcement shall be designed as D28 @125 (double) As = 98.56 cm^2/m in the dam center direction, while for the downstream section, designed as D28 @125 (threefold) As = 147.84 cm^2. And the stirrup bar shall be designed at the downstream bottom slab in the dam center direction, so that the shear stress will become τ = 7.3 $kg/cm^2 > 5.4$ kg/cm^2.

5. 10. 3 Pier Upper Slab

The structure calculation is made for the center pier which is deemed most critical in view of its load. The load on the slab is considered to distribute as shown in the following illustration, and slab A and B are trapezoidal cantilever supported by columns, and the slab C is a hexagonal beam fixed at both sides.

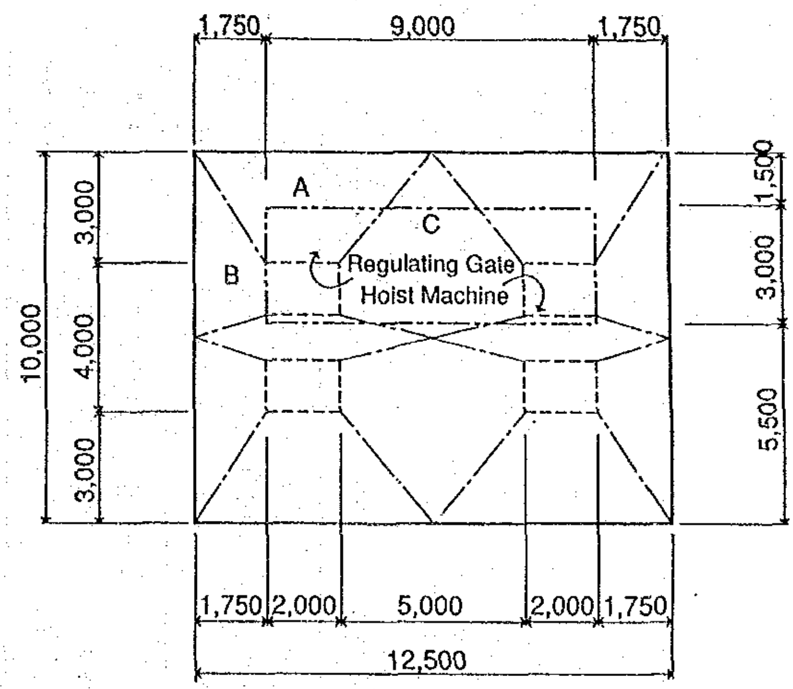


TABLE 5-3 RESULT OF STRUCTURAL CALCULATION FOR BOTTOM SLAB OF CENTER PIER

Iitem			Thalweg Direction		Dam Center Direction	
			Upstream	Downstream	Upstream	Downstream
Dimensions	b	(cm)	100.00	100.00	100.00	100.00
	h	(〃)	290.00	290.00	290.00	200.00
	d	(〃)	270.00	270.00	270.00	175.00
Moment	M	(t・m)	0.00	0.00	290.29	298.93
Shearing Force	S	(t)	0.00	0.00	107.34	110.61
Requir. Reinf.	As	(cm^2)	0.00	0.00	82.84	136.83
Reinf.	Dia.	(mm)	D20	D20	D28	D28
	Pitch	(〃)	@125 (×2)	@125 (×2)	@125 (×2)	@125 (×3)
	As	(cm^2)	50.24	50.24	98.56	147.84
Stress	σc	(kg/cm^2)	0.00	0.00	37	66
	σs	(〃)	0.00	0.00	1184	1301
	τ	(〃)	0.00	0.00	4.6	7.3
Allowable Stress	σca	(kg/cm^2)	94.5	94.5	94.5	94.5
	σsa	(〃)	1400	1400	1400	1400
	τa	(〃)	8.4	8.4	8.4	5.4

TABLE 5-4 RESULT OF STRUCTURAL CALCULATION FOR BOTTOM SLAB OF ABUT PIER

Iitem			Thalweg Direction		Dam Center Direction	
			Upstream	Downstream	Upstream	Downstream
Dimensions	b	(cm)	100.00	100.00	100.00	100.00
	h	(〃)	290.00	200.00	290.00	200.00
	d	(〃)	265.00	180.00	270.00	175.00
Moment	M	(t · m)	54.87	50.08	245.72	254.36
Shearing Force	S	(t)	27.54	24.93	90.80	94.00
Requir. Reinf.	As	(cm^2)	15.31	20.85	69.72	115.50
Reinf.	Dia.	(mm)	D16	D16	D25	D25
	Pitch	(〃)	@125 (×2)	@125 (×3)	@125 (×2)	@125 (×3)
	As	(cm^2)	32.16	48.24	78.56	117.84
Stress	σc	(kg/cm^2)	11	16	34	60
	σs	(〃)	676	619	1247	1373
	τ	(〃)	1.2	1.6	3.9	6.2
Allowable Stress	σca	(kg/cm^2)	94.5	94.5	94.5	94.5
	σsa	(〃)	1400	1400	1400	1400
	τa	(〃)	8.4	8.4	8.4	5.4

1) Load Conditions

The loads working on the upper slab are shown as follows.

a) Dead Load of Upper Slab: 2.4 t/m^3

b) Loading (uniform load)

- Cinder concrete : $2.2\ t/m^3 \times 0.20\ m = 0.44\ t/m^2$
- Regulating gate (operating) : 400 t
- Hoist machine weight : 100 t
- Uniform load : $(400+100)/(9.00\times3.00)+0.44=18.96\ t/m^2$

c) Hoist House Load (at slab edge)

$W = 1.50\ t/m^2 \times 10.00 \times 12.50 = 187.50\ t$

Per one meter of the wall : $187.50/(10.00 + 12.50) = 4.17\ t/m$

2) Results of Structural Calculation

The results of the structural calculation are shown in Table 5-5. For the upper slab, the quantity of reinforcement required will be given by D25 @125 (double) As = 78.56 cm^2/m at the middle part of C at both ends in the thalweg direction, and in the dam center direction, the required quantity for the A, overhang, can be obtained as D25 @125 (double) As = 78.56 cm^2/m.

For the bottom slab, since the maximum quantity required at the middle part, C (Center) will be As = 2.58 cm^2/m, the required quantity shall be determined to be more than 1/2 of that of the tension side bars by D25 @125 As = 39.28 cm^2/m.

5.11 Design of Pier Foundation Works

5.11.1 Load Conditions

The design of the pier foundation works shall be made along with both the thalweg and dam center directions. According to the stability analysis

TABLE 5-5 RESULT OF STRUCTURAL CALCULATION FOR UPPER SLAB

Item			A	B
Dimensions	b	(cm)	100.00	100.00
	h	(〃)	150.00	150.00
	d	(〃)	135.00	135.00
Moment	M	(t · m)	108.94	38.60
Shearing Force	S	(t)	35.56	9.35
Req. Reinf.	As	(cm^2)	63.17	21.61
Reinf.	Dia.	(mm)	D25	D25
	Pitch	(〃)	@125 (×2)	@125 (×2)
	As	(cm^2)	78.56	78.56
Stress	σc	(kg/cm^2)	46	16
	σs	(〃)	1136	403
	τm	(〃)	3.0	0.8
Allowable Stress	σca	(kg/cm^2)	95	95
	σsa	(〃)	1400	1400
	τa	(〃)	4.2	4.2

Item			C (End)	C (Center)
Dimensions	b	(cm)	100.00	100.00
	h	(〃)	150.00	150.00
	d	(〃)	135.00	140.00
Moment	M	(t · m)	55.31	27.66
Shearing Force	S	(t)	46.46	0.00
Req. Reinf.	As	(cm^2)	71.42	2.58
Reinf.	Dia.	(mm)	D25	D25
	Pitch	(〃)	@125 (×2)	@125
	As	(cm^2)	78.56	39.28
Stress	σc	(kg/cm^2)	23	15
	σs	(〃)	577	562
	τm	(〃)	3.4	0.0
Allowable Stress	σca	(kg/cm^2)	95	95
	σsa	(〃)	1400	1400
	τa	(〃)	4.2	4.2

results obtained previously, the load conditions shall be adopted on the horizontal force and eccentric distance with large values as follows.

	Case	Direct.	V. Force V (t)	H. Force H (t)	Ecc. Dis. e (m)	Moment M (t - m)
Center Pier	1	Thalweg	6,920	1,110	1.21	8,380
	2	Dam Center	6,200	50	1.52	9,430
Abut Pier	1	Thalweg	8,070	690	0.92	7,430
	2	Dam Center	7,460	1,380	1.38	10,300

5.11.2 Study on Construction Method for Pier Foundation

The allowable bearing capacity of the ground for the respective piers can be estimated based on the aforesaid load conditions and shown as follows. As load works more than allowable bearability in every case, the direct foundation method has been found to be not adoptable.

	Direction	Ultimate bear. Capacity Vu (t)	Safety Factor Fs	Allowable bear. Capacity Va (t)	F. Force V (t)
Center Pier	Thalweg	7,407	3	2,469	< 6,920
	Dam Center	6,576	3	2,192	< 6,200
Abut Pier	Thalweg	9,507	3	3,162	< 8,070
	Dam Center	5,442	3	1,814	< 7,460

Under the conditions, the foundation, being important in relating to piers, gates and the O/M road bridge, shall be constructed by pile foundation method with high reliability and easy confirmation on bearability.

5.11.3 Determination of Pile Length

The propose foundation piles length shall be determined by L = 10.0 m as learned from the relevant geological boring logs so that the piles can be rooted by length more than diameter of the piles into favourable layers with N-value of more than 30.

5. 11. 4 Study on the Type of Piles

As mentioned already, since Class-' A' PC piles have some problems in bearing capacity and their maximum diameter available by 600 mm in TIS cannot keep the piles' intervals by 2.5 m at the minimum, the PC piles cannot be used for the foundation works to require more close intervals than the above.

Consequently, the pier foundation works shall be made with PHC piles or steel piles with large and reliable strengths.

And the comparative study has been made between PHC and steel piles in various aspects. As a result, the proposed pile foundation shall be constructed with steel piles of SKK 490 with 800 mm in diameter and 9.0 mm in thickness for reasons of economy.

5. 11. 5 Study on Pile Arrangement

As the study results on load conditions and economic comparison, the reliable strength and displacement of the pile head are estimated for each pier. As the results of the aforesaid study, the compressive stress intensity for the piles at the center pier is σc = 1,721 kg/cm^2 < 1,900 kg/cm^2 in the thalweg direction, and the steel piles of SKK 490 with diameter of 800 mm and length of 10.0 m shall be adopted in 50 positions: 10 pieces × 5 rows will be prepared for the center piers.

On the other hand, the compressive stress intensity for the abut piers will be σc =1,853 kg/cm^2 < 1,900 kg/cm^2 in the dam center direction, and the steel piles of SKK 490 with diameter of 800 mm and the length of 10 m will be adopted in 55 pieces: 11 pieces × 5 rows will be prepared for the abut piers.

5. 11. 6 Design of Pile Foundation for Apron

1) Load Conditions

The load conditions working upon each apron per block can be shown as follows.

	Size (m)			V. Force	Remarks		
	Width	Length	Thickness	V (t/Block)	Gate	Up. W.L.	Down.W.L.
Up.Apron	12.00	13.00	0.70	1,800	-	Max. 2.40 m	Max. 2.40 m
Mid.Apron	8.00	24.00	2.90 to 2.00	2,710	Open	〃	〃
Dow.Apron	9.00	18.00	1.10	2,090	-	〃	〃

2) Study on the Construction Method for Apron Foundation

Allowable bearing capacity of the ground can be estimated with load conditions applied to each apron and the results are shown in the following table. The direct foundation method cannot be applied to this work because the load working upon it exceeds the allowable bearing capacity.

	Ultimate Bear. Capacity Vu (t)	Safety Factor Fs	Allowable Bear. Capacity Va (t)	Vertical Force V (t)
Up. Apron	5,070	3	1,690	< 1,800
Mid. Apron	6,864	3	2,228	< 2,710
Dow. Apron	5,265	3	1,755	< 2,090

The replacement method for foundations will not be applied to these foundation works because sufficiently bearable layers cannot be found at the shallow part of the ground. The pile foundation method, as a consequence, shall be adopted to the works.

3) Determination of Pile Length

The pile length for the aprons can be determined as follows so the pile tips can penetrate into the effectively arable layers with N-values more than 30. And in that case, the pile penetration depth shall be over the diameter of the these piles.

	Pile Length (m)	Remarks
Up. Apron	12.00	
Mid. Apron	10.00	
Dow. Apron	10.00	

4) Determination for Type of Pile

The type of pile foundation to be proposed for the aprons shall be constructed with Class- 'A' PC piles in considering the fact that the aprons will not be subject to large horizontal forces. The diameter of the pile is determined through economic comparison of the pile foundation at the middle apron with the largest load on the proposed aprons. And economically, the piles in the TIS standard with diameter of 300 mm and length of 10 to 12 m shall be selected.

5) Number of Piles to be Used

According to the designed pile arrangement, the number of piles to be used can be determined as follows in considering the vertical force on each apron and allowable bearable capacity.

	V. Force V (t)	Allowable Bear. Capacity Va (t/piece)	Required Number (Piece)	Arranged Pile Number (Price)
Up. Apron	1,800	88	20.5	$6 \times 4 = 24$
Mid. Apron	2,710	79	34.3	$9 \times 4 = 36$
Dow. Apron	2,090	71	29.4	$8 \times 4 = 32$

5. 12 Stability and Structural Calculation for Retaining Walls

The retaining walls proposed for the diversion dam have the following two types, and the stability and structural calculations have been made for these two types.

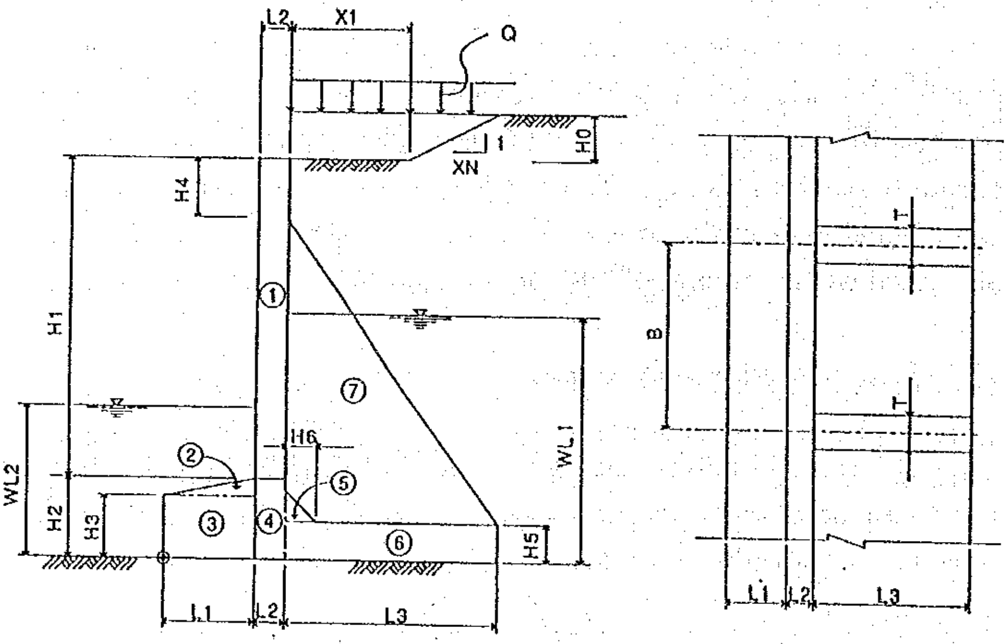
- Counterfort wall type .... Type W1, Type W2
- Inverted T-shape type .... Type W3

5. 12. 1 Stability Calculation

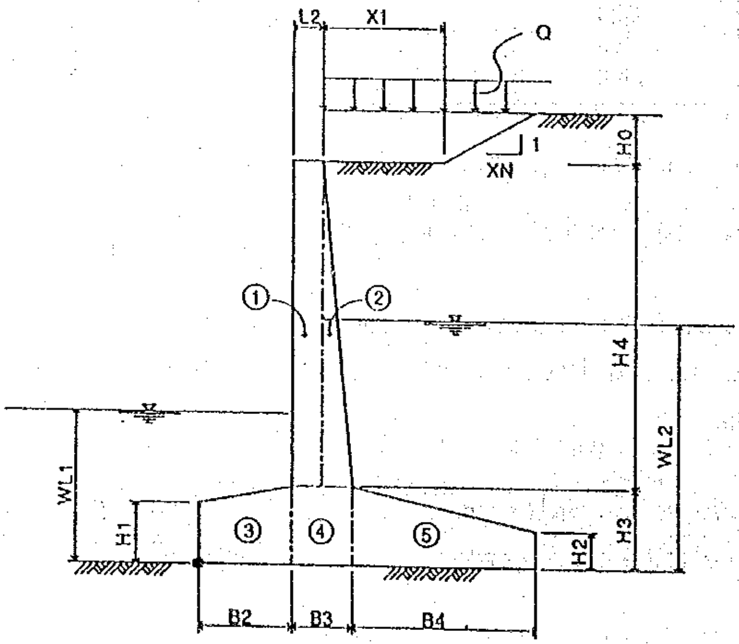
1) General Figures

The following figures show the Counterfort type wall and the Inverted T-shape type wall.

General Figure of Counterfort Wall Type



General Fique of Inverted T-Shape Type



2) Water Level

The stability analysis of the wall is made based on the water level conditions at the lowest low water level at the front of each wall and residual water level of the upstream at the gate crest elevation of EL. 1.80 m and of the downstream at the high water level (sea level) of H.W.L. 1.30 m at the back of the wall.

	Front of Wall	Back of Wall	Remarks
Upstream	Min. O.L.(-) 1.30 m	W.L. 0.80 m	W1-1, W1-2,W2-1, W3
Downstream	L.L.W.L.(-) 1.70 m	W.L. 0.30 m	W1-3, W1-4, W2-2

3) Case Studies on Stability Analysis

The two case studies as follows have been carried out on the necessary stability analysis.

Case 1: After construction
Case 2: Under construction

4) Results of Stability Analysis

The wall stability analysis has been made to show the results in Table 5-6.

The Counterfort type walls of W1-1 and W1-4 with front footing slabs are designed to ensure the safety factors against sliding.

The Counterfort type walls of W1-2, W1-3, W2-1 and W2-2 without front footing slabs are designed to ensure safety against overturning of the structures under construction.

The Inverted T-shape type walls are deemed safe against both sliding and overturning, but the design of footing slab shall be made in considering the arrangement of the foundation piles.

5. 12. 2 Structural Calculation

1) Calculation Method

The walls and back footing slabs will be supported by the counterfort wall in a design of continuous beams. The front footing slabs are designed with cantilevers against soil reactions.

The walls and footing slabs of the Inverted T-shape type walls are supported by cantilevers to secure stability against earth pressure and soil reaction.

2) Results of Structural Calculation

The counterfort walls are 1.0 m at a minimum for securing the height of walls in a range from 12.0 m to 12.9 m. And the minimum thickness of the vertical walls is 0.5 m. All thicknesses of walls are minimum. The analyses of the designed reinforcement and stress are shown Appendix 5. 12. 2.

TABLE 5-6 RESULT OF STABILITY ANALYSIS

Type		Vertical Force ΣV (t)	Stabilizing Moment $\Sigma V \cdot x$ (t·m)	Horizontal Force ΣH (t)	Overturning Moment $\Sigma H \cdot y$ (t·m)	Against Sliding Factor of safety Fs	Fsa	Against Overturning E (m)	B/6 (m)	Soil Reaction Q1 (t/m²)	Q2 (t/m²)
W 1 − 1	Case 1	534.04	2297.09	211.06	925.81	1.52	> 1.5	0.68	< 1.08	44.63	10.14
	Case 2	142.20	434.00	0	0	−	> 1.2	0.20	< 1.08	27.58	18.98
W 1 − 2	Case 1	808.14	3651.29	211.06	925.81	2.30	> 1.5	0.63	< 1.33	49.52	17.83
	Case 2	187.80	510.72	0	0	−	> 1.2	1.28	< 1.33	15.34	0.31
W 1 − 3	Case 1	861.45	3912.64	233.85	1084.95	2.21	> 1.5	0.72	< 1.33	55.21	16.58
	Case 2	199.14	535.83	−	−	−	> 1.2	1.31	< 1.33	16.45	0.15
W 1 − 4	Case 1	620.84	2853.19	233.85	1084.95	1.59	> 1.5	0.65	< 1.17	46.08	13.05
	Case 2	159.90	517.03	−	−	−	> 1.2	0.23	< 1.17	27.35	18.34
W 2 − 1	Case 1	950.80	3781.61	276.49	1212.80	2.06	> 1.5	0.80	< 1.17	58.21	10.91
	Case 2	196.78	452.66	−	−	−	> 1.2	1.20	> 1.17	57.03	− 0.80
W 2 − 2	Case 1	1148.66	5186.30	306.35	1421.29	2.25	> 1.5	0.72	< 1.33	56.33	16.74
	Case 2	230.56	610.76	−	−	−	> 1.2	1.35	> 1.33	19.33	− 0.12
W 3	Case 1	26.09	66.03	8.56	20.77	1.85	> 1.5	0.01	< 0.58	7.64	7.26
	Case 2	11.95	15.96	−	−	−	> 1.2	0.41	< 0.58	5.81	1.01

5.13 Design of Retaining Wall Foundation Works

5.13.1 Load Conditions

The retaining wall foundation works are designed under the following load conditions according to the results of the stability analysis conducted as above. The load conditions as below shall be applied for each block between expansion or contraction joints from the results of the analysis.

Type	V. Force V (t)	H. Force H (t)	Ecc. Dis. e (m)	Moment M (t-m)
W1-1	1,780	705	0.68	1,210
W1-2	1,750	460	0.63	1,110
W1-3	2,585	705	0.72	1,860
W1-4	2,070	780	0.65	1,350
W2-1	4,370	1,270	0.30	1,300
W2-2	5,050	1,350	0.72	3,640
W3	265	90	0.01	5

5.13.2 Study on Construction Methods for Foundation Works

The allowable bearing capacity of the proposed ground can be estimated based on the load conditions of each wall to be shown in the following table. In this case, the direct foundation construction method cannot be adopted due to the fact that the load working on the ground exceeds the allowable bearing capacity.

The pile foundation method, therefore, shall be adopted in the same manner as the case for the pier foundation works.

Type	Ultimate Bear. Capacity Vu (t)	Safety Factor Fs	Allowable bear. Capacity Va (t)	V. Force V (t)
W1-1	741	3	247	< 1,780
W1-2	657	3	219	< 1,759
W1-3	954	3	318	< 2,585
W1-4	834	3	278	< 2,070
W2-1	990	3	330	< 4,370
W2-2	1,092	3	364	< 5,050
W3	576	3	192	< 265

5. 13. 3 Determination of Pile Length

The pile length to be used for the foundation works of the walls is determined at 10 m, the same length as those for the pier foundation, and the pile tip shall penetrate sufficiently into favourably bearable layers with N-values more than 30, and the penetration length of the piles into such supportable layers shall be more than the diameter of the said piles.

5. 13. 4 Study on Type of Proposed Pile

Since the proposed foundation ground has a larger horizontal force than vertical force, neither PC pile or PHC pile shall be employed. And steel piles with high reliability in strength shall be used for the wall foundation piles.

As a result of study, those piles specified to Standard SKK 400 or 490 will be selected and the diameter well-suited to the works will be determined as follows:

The economic study can find that the W1-1 will has a large ratio of horizontal force to vertical force, having many superiority in its quality, and the proposed diameters of the piles to be used shall be 600 mm to 900 mm.

The general comparative study on the piles has resulted in the design whereby the steel piles to be adopted shall be of SKK 490 type with diameter of 800 mm with thickness of 12 mm.

The pile foundation for W3 type walls shall adopt PC piles of TIS Standard with diameter of 600 mm because the load working on the ground will be considerably smaller than that in other cases. (Refer to Appendix-5. 13)

5. 13. 5 Study on Pile Arrangement

The reliable stress of the piles and displacement of pile heads can be estimated for each wall. The relevant study can clarify the quantity of piles required for the works, as shown in the following table.

Type	Quantity Required (Piece)	Remarks	
W1-1	5 + 4 + 5 = 11	σc = 1,775 kg/cm^2	< 1,900 kg/cm^2
W1-2	4 × 3 = 12	σc = 1,617 〃	<1,900 〃
W1-3	4 × 4 = 16	σc = 1,877 〃	<1,900 〃
W1-4	3 × 5 = 15	σc = 1,863 〃	<1,900 〃
W2-1	11 + 9 + 7 = 27	σc = 1,790 〃	<1,900 〃
W2-2	10 + 8 + 7 + 6 = 31	σc = 1,776 〃	<1,900 〃
W3	2 × 4 = 8	σc = 120 〃	< 170 〃

5. 14 Design of Gates

5. 14. 1 Design of Flood Gates

1) Design Conditions

°	Gate Type	Single Steel Shell Roller Gate
°	Gate Span	30.0 m
°	Leaf Height	10.0 m
°	Gate Quantity	3 sets
°	Design Water Level	

	Case 1	Case 2
Upstream W.L.	EL. 1.80 m	EL.(-) 1.30 m
Downstream W.L.	EL.(-)1.70 m	EL. 1.80 m

°	Operating Water Level	Upstream EL. 1.80 m, Downstream EL.(-) 1.70 m
°	Gate Sill Elevation	EL.(-) 8.20 m
°	Sealing Type	3 side rubber seal at the downstream
°	Hoist Type	2-motors, 2-drams, wire rope winch
°	Operating Speed	0.3 m/min
°	Operation Method	Local and remote
°	Lifting Height	12.1 m
°	Wind Load	0.3 t/m^2
°	Allowable Thickness	Upstream: 1.0 mm Downstream 1.5 mm
°	Allowable Deflection	below 1/800

2) Design of Gate Leaves

The stress and deflection of the gate leaf are shown in Table 5-7.

TABLE 5-7 STRESS AND DEFLECTION

Case	Water Pressure (t)	Hor. Bending Moment (t-m)	Max. Stress (kg/cm^2)	Allowable Stress (kg/cm^2)	Deflection		
					Horizontal	Vertical	Allowable
Case 1	855.70	3,369.33	1,086	1,200	1/1,377	1/25,623	1/800
Case 2	839.16	3,304.21	1,065	1,200	1/1,407	1/25,625	1/800

3) Design of Main Roller

The contact stress of the main roller is shown in Table 5-8.

TABLE 5-8 CONTACT STRESS OF MAIN ROLLER

Case	Water Pressure (t)	Roller Load (t)	Roller Radius (mm)	Contact Stress (kg/cm^2)	Allowable Stress (kg/cm^2)
Case 1	855.70 t	115.0	70.0	8,031	9,000
Case 2	839.16 t	110.0	70.0	-	-

4) Hoisting Equipment

a) Hoisting Load

The hoisting load with gate leaves can be specified as amounting to 300 tons, 28.1 tons of roller resistance, 3.9 tons of friction of sealing rubbers, and 47.3 tons of downpull, totaling about 400 tons.

b) Design of Motors

Two units of electric motor of 16.05 kw power are to be installed, taking into consideration 400 tons of operation load, 0.3 m per min. for operation speed, 0.62 of mechanical efficiency, etc. And finally, the electric motors shall be provided by two units with 18.50 kw × 2.

5. 14. 2 Design of Regulating Gates

1) Design Conditions

° Gate Type	Double Steel Shell Roller Gate	
° Gate Span	30.0 m	
° Leaf Height	10.0 m (Upper Leaf: 3.1 m, Lower Leaf: 6.9m)	
° Gate Quantity	2 sets	
° Design Water Level	Case 1	Case 2
	Upstream W.L. EL. 1.80 m	EL.(-) 1.30 m
	Downstream W.L. EL.(-)1.70 m	EL. 1.80 m
° Operating Water Level	Upstream EL. 1.80 m, Downstream EL.(-) 1.70 m	
° Gate Sill Elevation	EL.(-) 8.20 m	
° Sealing Type	3 side rubber seal at the downstream	
° Hoist Type	2-motors, 2-drams, wire rope winch	
° Operating Speed	0.3 m/min	
° Operation Method	Local and remote	
° Lifting Height	12.1 m	
° Wind Load	0.3 t/m^2	
° Allowable Thickness	Upstream: 1.0 mm Downstream 1.5 mm	
° Allowable Deflection	below 1/800	

2) Design of Gate Leaf

The stress and deflection of the gate leaf are shown in Table 5-9.

TABLE 5-9 STRESS AND DEFLECTION

Position	Case	Water Pressure (t)	Hor. Bending Moment (t-m)	Max. Stress (kg/cm^2)	Allowable Stress (kg/cm^2)	Deflection Horizontal	Deflection Vertical	Deflection Allowable
Upper Leaf	Case 1	156.85	629.37	971	1,200	1/1,103	1/4,263	1/800
	Case 2	161.21	646.86	998	1,200	1/1,073	1/4,264	1/800
Lower Leaf	Case 1	659.69	2,739.27	1,186	1,200	1/1196	1/7,834	1/800
	Case 2	674.66	2,656.46	1,150	1,200	1/1,235	1/7,834	1/800

3) Design of Main Roller

The specific information on the contact stress of the proposed main roller is shown in Table 5-10.

TABLE 5-10 CONTACT STRESS OF MAIN ROLLER

Position	Case	Water Pressure (t)	Roller Load (t)	Roller Radius (mm)	Contract Stress (kg/cm^2)	Allowable Stress (kg/cm^2)
Upper Leaf	Case 1	156.85	45.0	50.0	8,031	9,000
	Case 2	161.21	45.0	50.0	8,031	9,000
Lower Leaf	Case 1	659.69	100.0	62.5	8,957	9,000
	Case 2	674.66	100.0	62.5	8,957	9,000

4) Hoisting Equipment

a) Hoisting Load

The hoisting load for the upper gate leaf consists of 90 tons of leaf weight, 130 tons of overflow water weight, 7.4 tons of roller resistance, and 6.7 tons of friction of sealing rubbers, totaling to about 235 tons, while that for the lower leaf consists of 20 tons of leaf weight, 126 tons of overflow water weight, 29.2 tons of roller resistance, and 3.9 tons of friction of sealing rubbers to total about 450 tons.

b) Design of Electric Motors

The proposed electric motors for gate operation shall be designed with the following details: Operation load: 235 tons, operation speed: 0.3 m/min., mechanical efficiency: 0.64, and motors: 2 units by power source of 9.14 kw, while, for the lower gate motors: operation load: 450 tons, operation speed: 0.3 m/min., mechanical efficiency: 0.61, and motors: 2 units by power source of 18.92 kw.

Therefore, the proposed electric motors for the gates shall be designed with 11 kw capacity for the upper gates and 22 kw for the lower gates, respectively.

5. 14. 3 Stop Logs

1) Design Conditions

- ° Gate Type · Steel Stop Logs
- ° Gate Quantity Gate Leaf 1 unit
  Guide Fame 5 units
- ° Gate Span 30.0 m
- ° Leaf Height 10.0 m
- ° Gate Sill Elevation EL.(-) 8.20 m
- ° Design Water Depth Upperstream 8.9 m
  Downstream 7.2 m
- ° Sealing Type 3 side rubber seal at the downstream
- ° Allowable Deflection below 1/600

2) Design of Gate Leaf

The specific information on stress and deflection is shown in Table 5-11.

TABLE 5 - 11 STRESS AND DEFLECTION

Member	Load	Bending Moment	Stress		Deflection	
			Max.	Allowable	Max.	Allowable
Lower Portion	0.706 t/m	2.87 t-m	629 kg/cm^2	1,082 kg/cm^2	1/1,096	1/600
Middle Portion	1,080 t/m	4.39	566	1,259	1/982	1/600
Skin Plate	1,700 t/m^2	-	217	1,200	-	-

3) Design of Supporting Posts

The stress and deflection working on the posts are as shown in Table 5-12.

TABLE 5 - 12 STRESS AND DEFLECTION

Member	Load	Bending Moment	Stress		Deflection	
			Max.	Allowable	Max.	Allowable
Vertical Member	1,700 t/m^2	59.99 t-m	947 kg/cm^2	1,038 kg/cm^2	1/700	1/600
Inclined Member	68.02 t	-	688	943	-	-

4) Installing Method

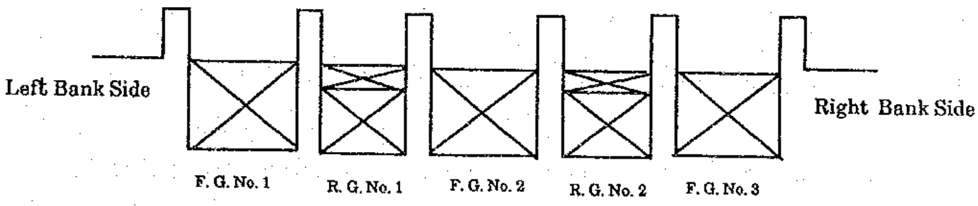
The maximum weight of the proposed steel stop logs is about 3.76 tons per piece in the 11.0 m length of the vertical member. And installation shall be executed by 10-ton truck crane at a point within an area with working radius of about 7.5 m. (Refer to Appendix 5.9.1).

5. 15 Gate Operation Rules

5. 15. 1 Designation of Gates, etc.

1) Gates

The Flood and Regulating Gates are designated from the left bank side to the right bank side, in order, respectively, such as Flood Gate No. 1 to Flood Gate No. 3, Regulating Gate No. 1 and Regulating Gate No. 2, as shown below.



Note: F. G. : Flood Gate
R. G. : Regulating Gate

2) Water Gauge

The water gauges are named as follows:

Water Gauge No. 1 (on the left bank, upstream from the dam)
Water Gauge No. 2 (on the left bank, downstream from the dam)
Water Gauge No. 3 (on the right bank, upstream from the dam)
Water Gauge No. 4 (onthe right bank, downstream from the dam)

The water level indicated by Water Gauge Nos. 1 and 3 must be applied for the gate operation. In the case of an emergency such as mechanical trouble in the said gauges, etc., the water level of Water Gauge No. 2 and No. 4 will be applied.

3) Salt Concentration Meter

The salt concentration meters for measuring a salinity at three (3) points in the water depth are designated as follows:

Salinity Meter No. 1 (on the left bank, upstream from the dam)

Indicator	No. 1-1	for the water depth at EL (+) 0.0 m
Indicator	No. 1-2	for the water depth at EL (-) 1.3 m
Indicator	No. 1-3	for the water depth at EL (-) 5.0 m

Salinity Meter No. 2 (on the left bank, downstream from the dam)

Indicator	No. 2-1	for the water depth at EL (+) 1.3 m
Indicator	No. 2-2	for the water depth at EL (-) 4.0 m
Indicator	No. 2-3	for the water depth at EL (-) 7.0 m

5.15.2 Method of Gate Operation

This rule has tentatively been prepared regarding the basic terms of gauge operation based on the result of analysis on the hydrology and conditions of the river basin to date. Therefore, while applying, this rule shall be improved to make up a reasonable gate operation method by continuing collection and arrangement of fundamental data and taking the comprehensive management of the Bank Pakong river basin.

1) Operation Mode

The operation mode, which categorizes the operation method of tide protection gates to comply with the control requirements, consists of the following four (4) modes:

Control Mode	Ruling Values	Gate Operation	Priority
Small Water Discharge	$Qi < 30\ m^3/s$	Overflow × 1 set	3
Normal Operation	① $30 < Qi < 80\ m^3/s$ ② $80 < Qi < 300\ m^3/s$	Overflow × 1 set Overflow × 2 sets	3
Inverse Flow Prevention	$H1 < H2 + 0.10\ m$	All Gates Closed	2
Flooding	$Q1 > 300\ m^3/s$	All Gates Opened	1

Note: Qi = Inflow to the reservoir, H1 = Water levels of the reservoir, and H2 = Water level downstream from the dam
The inverse flow prevention mode takes priority over the low water discharge and normal operation modes

2) Control System

For the Flood and Regulating Gates, the operations for each control mode are ruled as follows:

Control Mode	Control System	Note
Small Water Discharge	Const. W. L. Control	To control the water level in the reservoir constantly, subject to the discharge of river maintenance flow.
Normal Operation	Const. W. L. Control	To control the water level in the reservoir constantly, lowering the water level in the reservoir for the purpose of discharging the small amount of flood water and restoring the water quantity in the reservoir required for the Project after small amount of water discharge stage.
Inverse Flow Prevention	All Gates Closed	To prevent salt water intrusion into the reservoir.
Flooding	All Gates Opened	To open all gates, in case flood discharge exceeds 300 m^3/s, to keep the adequate passage for flood water.

3) Time for Closing of All Gates

In the transitional period (around November) from the rainy season to dry season, it is required to close all gates in order to prevent salt water from intruding into the reservoir. This period may begin around November 15, according to the river water discharge data for the last eight (8) years.

Nevertheless, estimating the time of salt water intrusion based on the salinity contents for the water sampled once a week at the Chachoengsao bridge, all gates must be opened in time for exceeding a salinity content of 100 mg/ℓ, as specified by the Water Quality Guideline for Plantation of the RID, downstream from the dam, even before November 15. The water sampling at the said bridge continues up to the beginning of water storage, starting in the second week of October.

The water way must be closed by the Regulating Gate No. 1, Regulating Gate No. 2, Flood Gate No. 2, Flood Gate No. 1 and Flood Gate No. 3 in sequence.

4) Time for Opening of All Gates

It has been verified that the salt water intrusion is blocked by small-scale floods in the transitional period (May to July) from the dry to rainy seasons, all gates are opened in this transitional period, where the inflow water discharge exceeds 300 m^3/s.

The gates are opened one by one, following the Flood Gate No. 2, Flood Gate No. 2, Flood Gate No. 3, Regulating Gate No. 1 and Regulating Gate No. 2 in sequence.

After opening all the gates, monitoring of salinity downstream from the dam must be conducted by the Salinity Meter No. 2. The gates must be closed if there is danger of intrusion from salt water.

5) Gate Operation for Desalinization

The gate shall operate to release the reservoir water for desalinization, where the salinity indicated by the Salinity Indicator No. 1-1 exceeds a salinity level of 250 mg/ℓ as specified in the Water Quality Standard for Water Supply of the MOI. When the Salinity Indicator No. 1-1 is not applicable due to lower water level, the salinity shall be applied by the value of Salinity Indicator No. 1-2 and/or No. 1-3. The water is released by the operation of a Flood Gate, when there is a great difference between the water levels in the reservoir and downstream from the dam. According to materials from Kasetsart University, the salt water intrusion is unexpected, where a water flow discharge is over 50 m^3/s at the dam site. Estimating a release water discharge of 50 m^3/s, the height of gate opening shall be determined according to the following table.

Height of Flood Gate Opening for Water Released for Desalinization

Water Level in Reservoir (m, MSL)	Water Level Downstream from the Dam (m, MSL)										
	1.0	0.8	0.6	0.4	0.2	0.0	-0.2	-0.4	-0.6	-0.8	-1.0
1.4	0.9	0.7	0.7	0.6	0.5	0.5	0.4	0.4	0.4	0.4	0.4
1.2	1.2	0.9	0.7	0.7	0.6	0.5	0.5	0.4	0.4	0.4	0.4
1.0	-	1.2	0.9	0.7	0.7	0.6	0.5	0.5	0.4	0.4	0.4
0.8	-	-	1.2	0.9	0.7	0.7	0.6	0.5	0.5	0.4	0.4
0.6	-	-	-	1.2	0.9	0.7	0.7	0.6	0.5	0.4	0.4
0.4	-	-	-	-	1.2	0.9	0.7	0.7	0.6	0.5	0.4
0.2	-	-	-	-	-	1.2	0.9	0.7	0.7	0.6	0.5
0.0	-	-	-	-	-	-	1.2	0.9	0.7	0.7	0.6

Note : This table shows the height of gate opening for the outflow discharge of 50 m^3/s under the gate of one (1) set.

5. 15. 3 Record of Gate Operation

The gate operation records, daily and monthly reports for operation and maintenance shall be prepared in accordance to the report forms.

CHAPTER 6. DESIGN OF CLOSURE DAM

6.1 Topography and Geology

The proposed closure dam shall be constructed about 300 m east of Ban Chuknua, which is located on the left bank of the Bang Pakong River, in crossing the river.

The cross section shape of the river's local topography around the closure dam is that the river bed width is about 230 m, minimum river bed elevation is EL (-)6.8 m, and slope of the both bank is as gentle as about 10-15 degrees.

Judging from the drilling survey for the both abutments and the river bed, the foundation of closure dam are composed of alluvial deposits which can be classified into the following three layes by different N-values.

- The surface layer consists of soft silt and clay with N-value in a range form 1 to 2, which distribute from right abutment to river bed at EL (-)12.0 m and from left abutment to river bed at EL (-)8.0 to (-) 10.0 m. This kind of soft ground can be judged to commonly and horizontally extend in the Project Area in the view that they can be found in the site of diversion dam and diversion canal with similar thickness.

- At the bottom of the surface layer, the intermediate layer with N-values from 10 to 15 distributes in a thickness of between 4.0 and 5.0 m.

- The base layer is found deeper than the intermediate layer extending in EL (-)12.0 to EL (-)16.0m, having N-value from 25 to 40 and over. These data suggest that the said base layer is on a firm foundation.

6.2 Outline of Design

The closure dam shall have a crest elevation of 3.9 m and a crest width of 12 m, taking into consideration the fact of the Bang Pakong River design flood water level of EL 2.40 m and road use of the dam crest after completion but excepting extra-banking of 0.3 m.

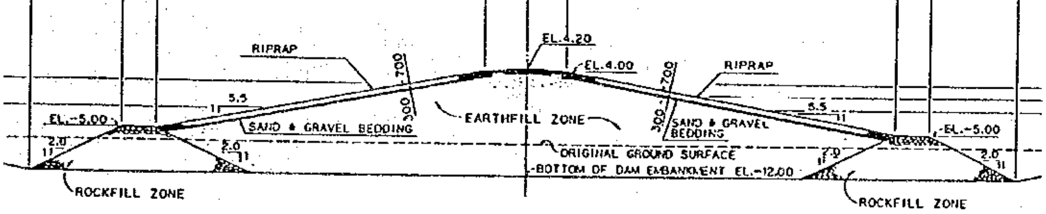
The closure dam shall be designed in symmetry with the dam axis, consisting mostly of an earthfill zone and partly of rockfill zone at the feet of the up-and-downstream slope for preventing the earth embankment materials from washout. Riprap works shall be provided on the earthfill zones for both up-and-downstream slopes for protecting embankment materials from erosion by waves and rainwater.

Both embankment slopes of the closure dam have been determined by stability analysis to result in 1 : 5.5 for the earthfill zone slope and 1 : 2.0 for the rockfill zone slope, respectively.

The main points of the closure dam design are summarized as follows.

1) Study on how to treat the soft layers distribution over the surface part of the dam foundation
2) Design of the sand compaction piles for soil improvement for both abutments
3) Selection of the embankment materials: comparative study of materials available at borrow area with those available from canal excavation

FIGURE 6-1 TYPICAL SECTION OF CLOSURE DAM

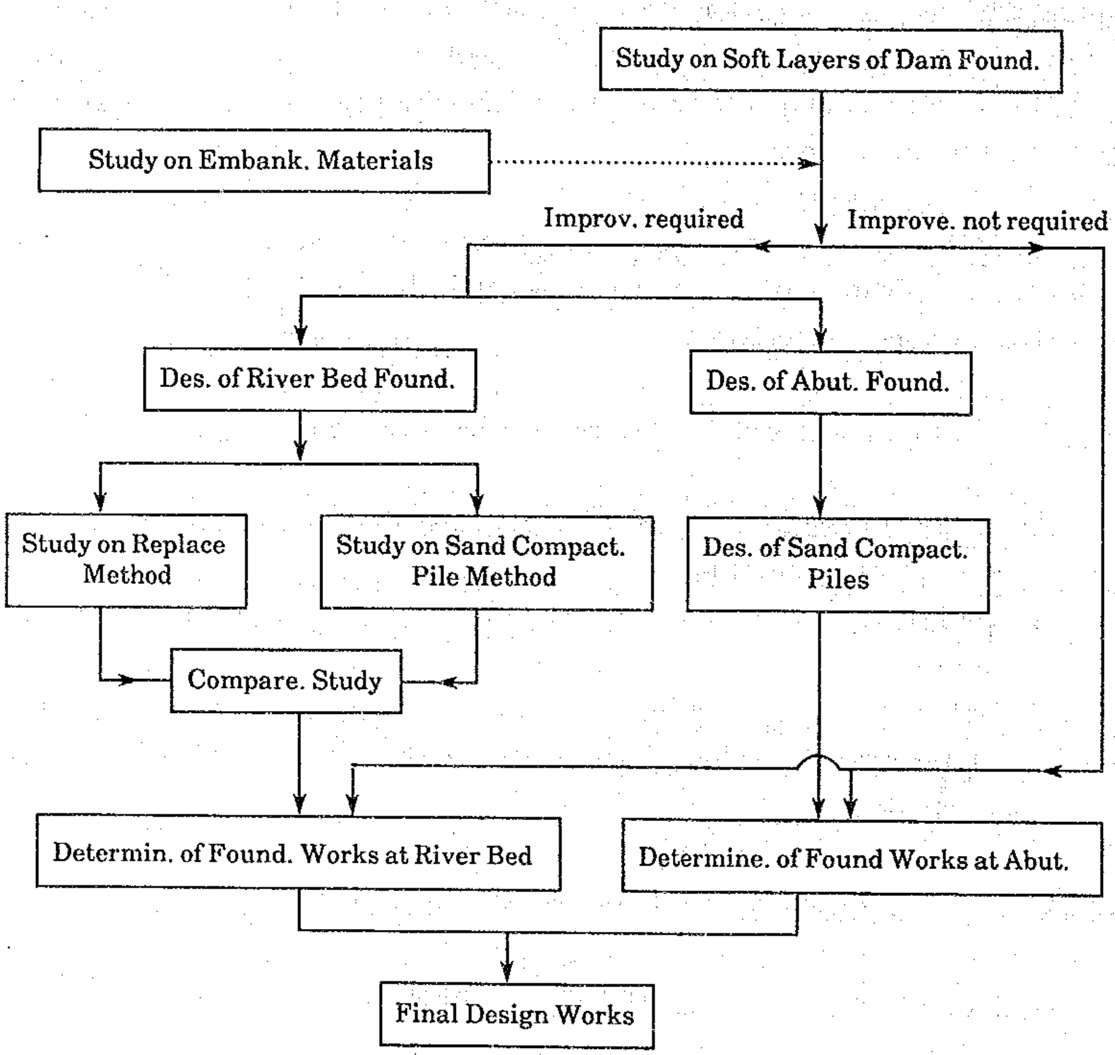


6.3 Design of Dam Foundation

6.3.1 Design Procedures

The following flow chart illustrates the design procedures of the dam foundation.

TABLE 6-1 FLOW CHART OF DESIGN OF DAM FOUNDATION



6.3.2 Study on Soft Layer of Dam Foundation

The soft layers with N-value from 1 to 2 are distributed up to EL (-) 12 m at the dam foundation. The following shows the results of the study on consolidation settlement, and stability analysis by slip circle slice method conducted for the case where a closure dam would be constructed at the site where the aforesaid soft layer foundation would remain unimproved. (Refer to Appendix 6.1.1)

- Consolidation Settlement Calculation

$$S = \frac{e_0 - e_1}{1 + e_0} \cdot H$$

$$= \frac{2.31 - 1.81}{1 + 2.31} \times 400$$

$$= 60.4 \text{ cm}$$

Where:
- S: Settlement (cm)
- e_0: Initial void ratio before loading 2.31
- e_1: Void ratio after loading 1.81
- H: Thickness of soft layers (cm). In this site, the surface soft layer by about 1.0 m thickness consists of soft mud layer, which shall be excavated, and the layer thickness after excavation shall be 400 cm.

- Estimation of Consolidation Time

$$t = \frac{Tu \cdot H^2}{Cv}$$

$$= \frac{0.567 \times 400^2}{86.4}$$

$$= 10{,}500 \text{ days}$$

$$\fallingdotseq 28.8 \text{ year}$$

Where:
- t: Time required for consolidation
- Tu: Time factor (0.567 by 80% for consolidation degree)
- H: Thickness of soft layers (400 cm)
- Cv: Consolidation coefficient 6.0×10^{-3} cm²/min $= 8.64$ cm²/day

- Result of Stability Analysis

The safety factor in rapid drawdown in water level can be expressed as $Fs = 0.896 \leqq 1.10$ and there will be fear for deep slip circle through soft layers as shown in Figure 6-2.