IRRIGATION ENGINEERING CENTER PROJECT
IN
THE KINGDAM OF THAILAND

DETAIL DESIGN REPORT
ON
THE MODEL INFRASTRUCTURE IMPROVEMENT WORK

MAY 1988

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

The Irrigation Engineering Center Project is a five-year-cooperation project aiming at development and consolidation of appropriate technology to be applied to planning, designs and constructions of irrigation and drainage facilities including transfer of knowledge to encourage backbone engineers in Thailand at the same time. The Project has been started its activities in the preceding since April 1, 1985 in order to contribute to promotion of agricultural infrastrucure improvement projects for stable food production in Thailand.

The team, headed by Mr.Noritaka KAWAGUCHI, Chief of First Laboratory Construction, National Research Institute of Agricultural Engineering, Ministry of Agriculture, Forestry and Fisheries was dispatched to Thailand from February 13, 1988 to March 31, 1988 for the purpose of detail design of the testing canal facility on the soft soil foundation as the model infrastructure improvement project in order to contribute to solve technical problems concerning the establishment of investigation and planning methods and design criteria for canal facilities constructed on the soft soil foundation.

This report represents the results of the field survey and a subsequent study in Japan. We hope that this report will be serve as a guideline for the model infrastructure improvement project being expected in the future.

Lastly, we take this opportunity to express our deep gratitude to all those who were concerned with us for the close cooperation and assistance they extend to the team throughout the survey period.

May 1988

Kazumi MIYAMOTO

Director
Agricultural Development
Cooperation Department,
Japan International
Cooperation Agency, JICA

LOCATION MAP OF THE PROJECT SITE

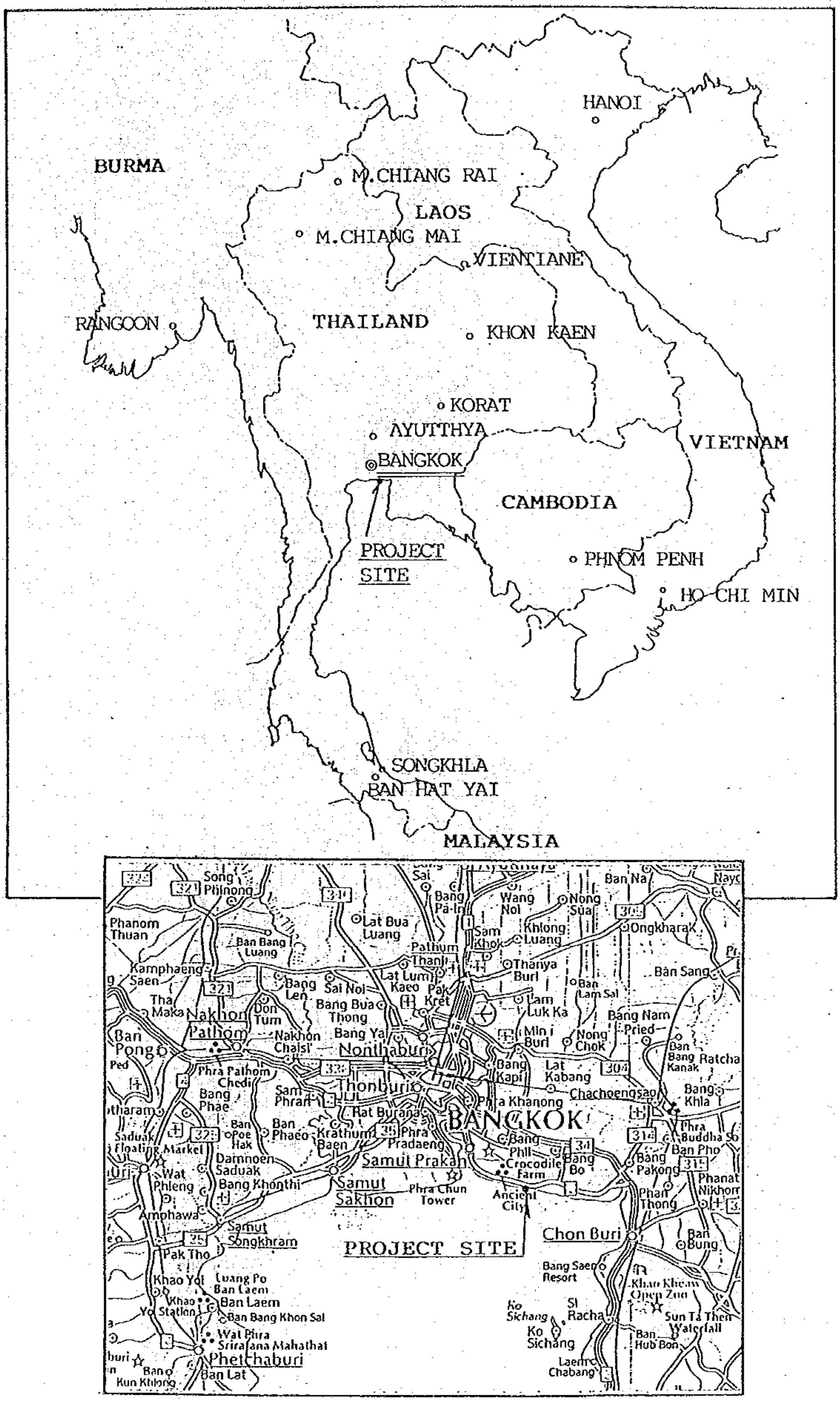
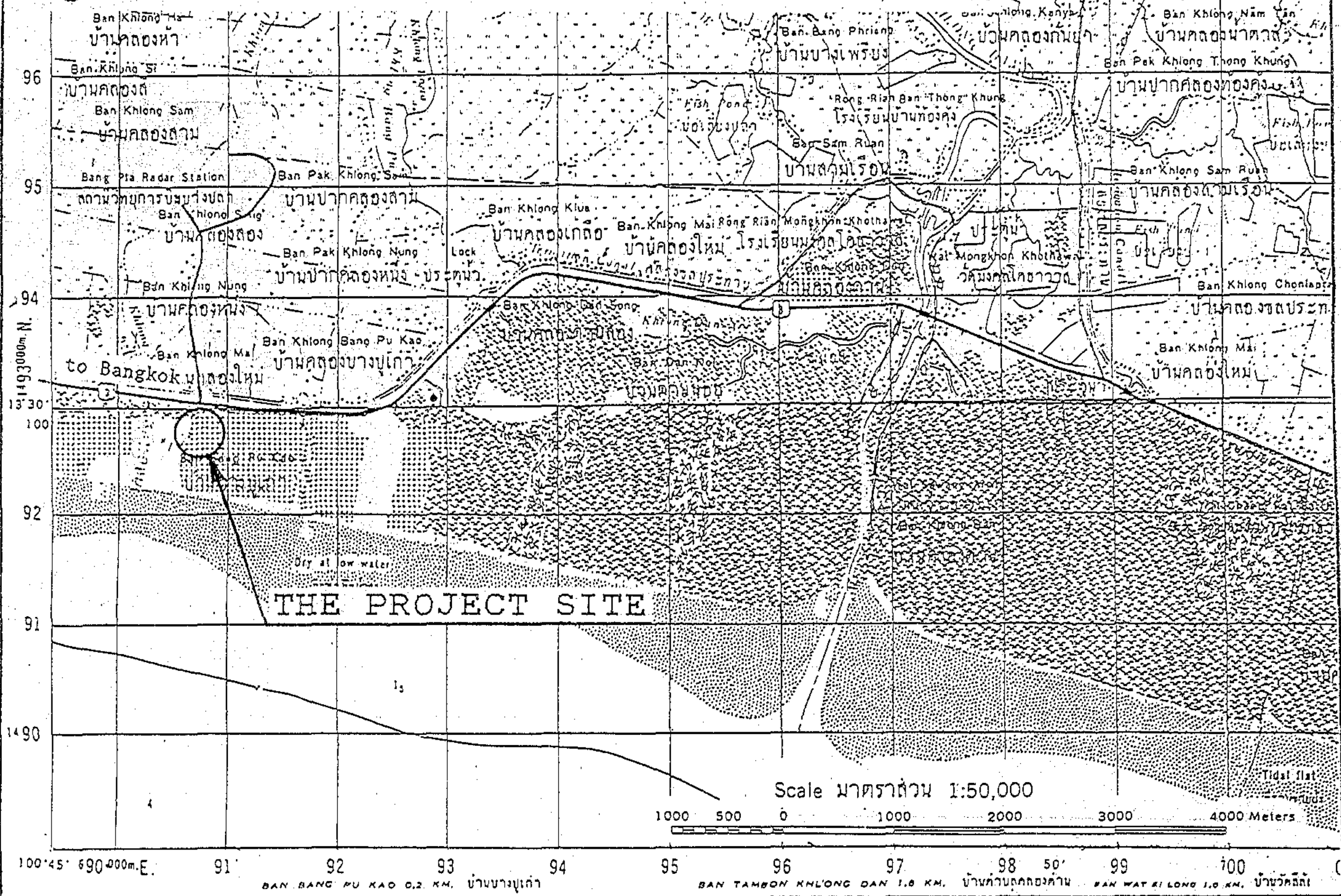
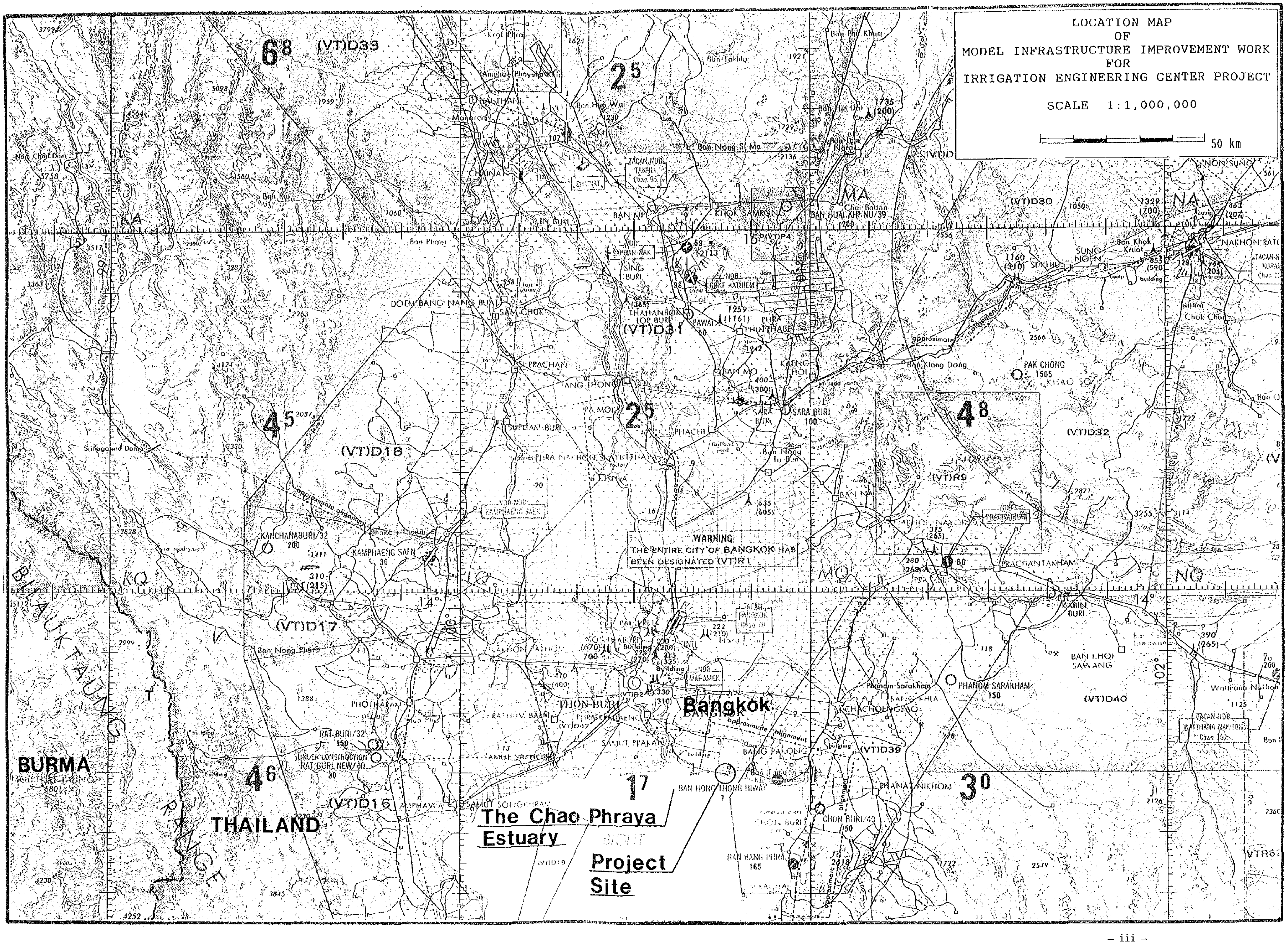


Fig. 2.2 LOCATION MAP OF THE PROJECT SITE

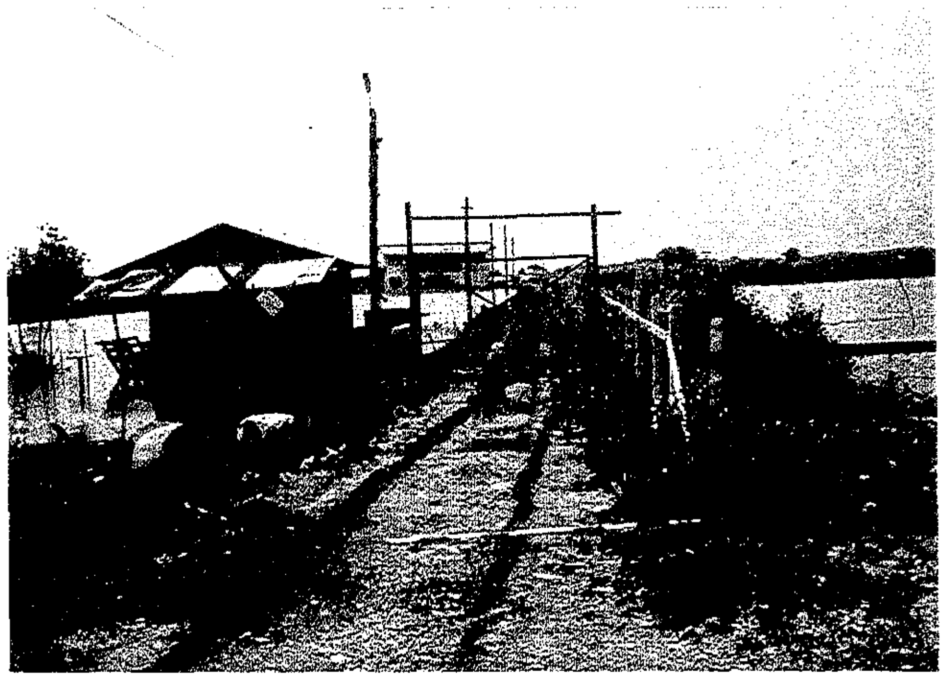



LOCATION MAP
OF
MODEL INFRASTRUCTURE IMPROVEMENT WORK
FOR
IRRIGATION ENGINEERING CENTER PROJECT
SCALE 1:1,000,000
50 km
Bangkok
The Chao Phraya Estuary
Project Site
BURMA
THAILAND
WARNING
THE ENTIRE CITY OF BANGKOK HAS BEEN DESIGNATED (VT)R1
SARA BURI
KANCHANABURI/32
KAMPHAENG SAEN
PHOTHARAM
RAT BURI/32
PHANOM SARAKHAM
CHON BURI/40
PHANAT NIKHOM
BAN BANG PHRA
KABIN BURI
PRACHANTAKHAM
PAK CHONG
NAKHON RATC
SAMUT SONGKHRAM
SAMUT PRAKAN
BANG PAKONG
CHACHOENGSAO
SING BURI
SUPHAN BURI
CHAINAT
KHOK SAMRONG
DOEM BANG NANG BUAT
SAM CHUK
ANG THONG
BAN HONG THONG HIWAY
(VT)D33
(VT)D31
(VT)D32
(VT)D30
(VT)D18
(VT)D17
(VT)D16
(VT)D39
(VT)D40
(VT)R9

No. 1 An Overall View of the Test Site

No. 2 The Nothern End of the Test Site facing along with the State Road

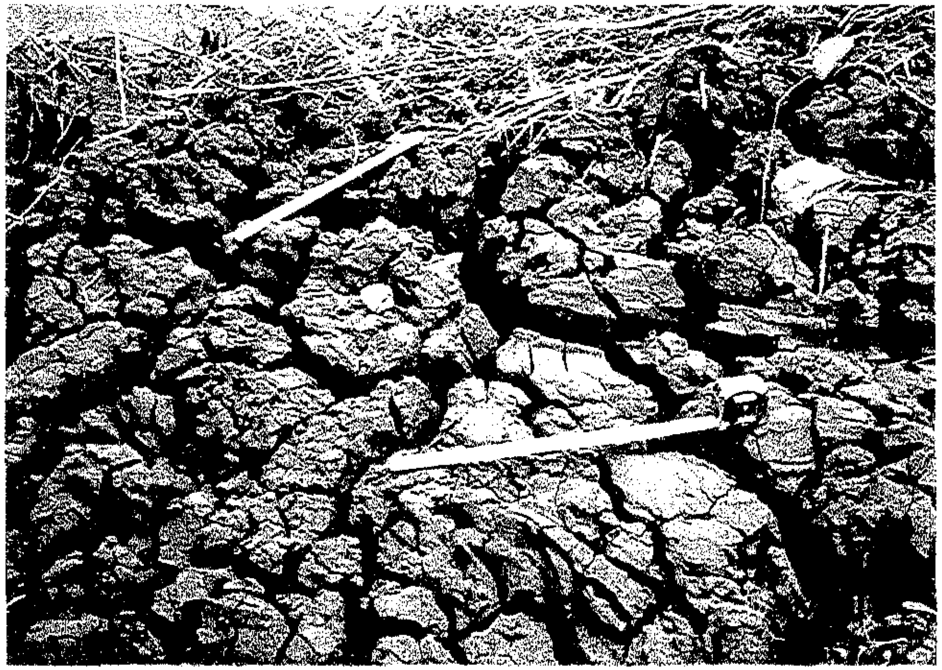
Slope improvement work is to be carried out from the line 25 m apart from the fences by soil cement column method.
(The diameter of the column : 1,000 mm
The length : 6.0~15.0 m)



No. 3 The Eastern End of the Test Site

The line 25 m to the right of the fences becomes the top of slopes.

The elevation of the top of the slopes : 0.50 m
The elevation of the toe of the slopes : -3.50 m
The Inclination of the slopes : 1 : 40
Untreated section



No. 4 A View of Drying and Shrinking of Bangkok Clay

The depth of cracks : about 50 cm
The width of cracks : about 5 cm

No. 5 A View Geotechnical Investigation in the Test Site

OUTLINE AND MAIN ITEMS OF THE PROJECT

I.OUTLINE OF THE PROJECT

1.Testing Canal Facility

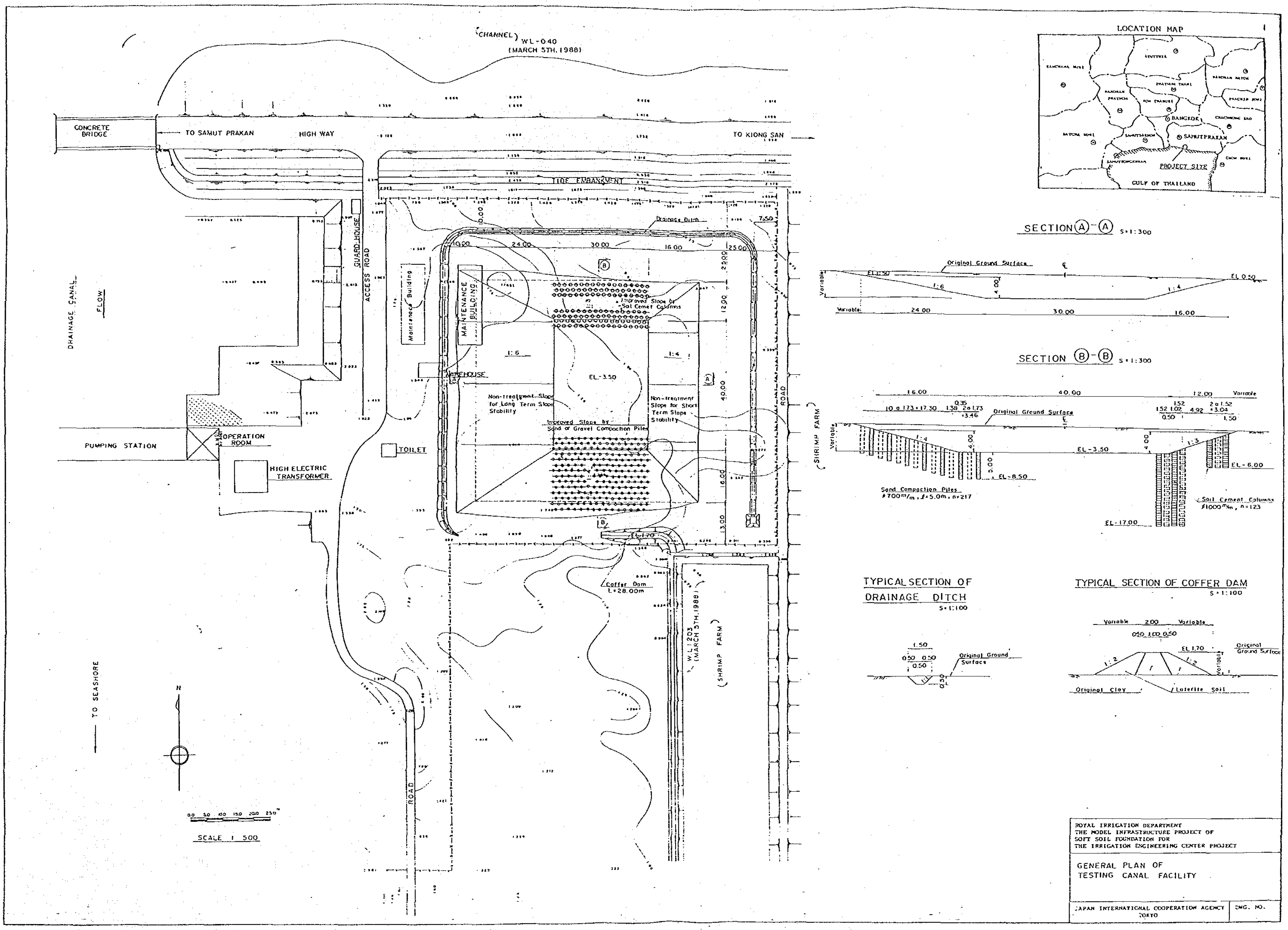
1)Depth of Canal : 4m
2)Size of Bottom of Canal : 40mx30m
3)Slope of Canal :

Type of Slope	Slope Gradient	Location
a.Non-treatment slope for short term slope stability	1:4	West side
b.Non-treatment slope for long term slope stability	1:6	East side
c.Improved slope by sand compaction piles	1:4	South side
d.Improved slope by soil cement columns	1:3	North side

2.Installation of Monitoring System(Procured by JICA)

II.MAIN WORK ITEMS OF THE PROJECT

Work Items	Specification	Quantity
1.Construction of Testing Cnal Facility		
1)Excavation of testing canal facility	Equipment&manpower	15,700 m³
2)Foundation improvement by sand compaction piles	Casing:0.4m Diameter:0.7m	1,085 m
3)Foundation improvement by soil cement columns	Diameter:1.0m	1,257 m
4)Installation of Displacement piles	Wooden piles	104 piles
2.Installation of Monitoring System		
1)Inclinometer(auto-measuring)	Measuring horizontal displacement in ground	1 places
Inclinometer(manual-reading)	-ditto-	5 places
2)Extensometer	Measuring horizontal displacement of ground surface	1 places
3)Differencial settlement gauge	Measuring vertical displacement in ground	7 places
4)Piezometer	Measuring pore water pressure	6 places
5)Water stand pipe	Measuring subterranean water	2 places
6)Measuring unit	Switch box, data logger	1 unit


(CHANNEL) WL-0.40 (MARCH 5TH, 1988)
CONCRETE BRIDGE
TO SAMUT PRAKAN
HIGH WAY
TO KLONG SAN
TIDE EMBANKMENT
LOCATION MAP
BANGKOK
SAMUTPRAKAN
PROJECT SITE
GULF OF THAILAND
SECTION (A)-(A) S=1:300
Original Ground Surface
SECTION (B)-(B) S=1:300
Sand Compaction Piles
Soil Cement Columns
DRAINAGE CANAL
FLOW
GUARD HOUSE
ACCESS ROAD
Maintenance Building
MAINTENANCE BUILDING
PUMPHOUSE
PUMPING STATION
OPERATION ROOM
HIGH ELECTRIC TRANSFORMER
TOILET
Drainage Ditch
Non-treatment Slope for Long Term Stability
Non-treatment Slope for Short Term Slope Stability
Improved Slope by Sand or Gravel Compaction Piles
Improved Slope by Soil Cement Columns
EL.-3.50
Coffer Dam L=28.00m
W.L.1.203 (MARCH 5TH, 1988)
(SHRIMP FARM)
ROAD
TO SEASHORE
SCALE 1:500
TYPICAL SECTION OF DRAINAGE DITCH S=1:100
TYPICAL SECTION OF COFFER DAM S=1:100
Original Ground Surface
Original Clay
Laterite Soil
ROYAL IRRIGATION DEPARTMENT
THE MODEL INFRASTRUCTURE PROJECT OF SOFT SOIL FOUNDATION FOR THE IRRIGATION ENGINEERING CENTER PROJECT
GENERAL PLAN OF TESTING CANAL FACILITY
JAPAN INTERNATIONAL COOPERATION AGENCY TOKYO
DWG. NO.

CONTENTS

CHAPTER 1 BASIC PLAN FOR THE MODEL INFRASTRUCTURE PROJECT

The testing canal facility constructed on the soft soil foundation as a model infrastructure project has a great role for the Irrigation Engineering Center Project (hereinafter referred to as IEC Project). The testing canal facility is constructed in order to carry out the case studies concerning the planning and the design standardization for the facilities to be constructed on the soft soil foundation. Taking the above situation into consideration, the objectives of the model infrastructure project are concluded as mentioned below. The objectives indicate the specific cooperation schemes of IEC Project for the facilities constructed under the model infrastructure project.

1) Objectives of the Model Infrastructure Project

The objectives of the Model Infrastructure Project throughout the execution of the detailed design, construction and monitoring are as follows:

a. Setting up the monitoring system for mechanical behaviour of the excavated soft soil foundation

b. Obtaining the mechanical behaviour of the excavated soft soil foundation

c. Examining the applicability of the method of slope stability analysis using circular slip surface for the excavated soft soil foundation

d. Studying the applicability of prediction for the stress and deformation occuring in the excavated soft soil foundation by the Finite Element Method (F.E.M.) using an Elasto-viscoplastic model

e. Suggestions and recommendations on the design and investigation for the soft soil foundation

2) Location of the Project Site

The model infrastructure Project for soft soil foundation shall be carried out in the area neighboring with the Charoenraj Pumping Station, about 40 km south-east of the center of Metropolitan Bangkok. The site of the Model Infrastructure Project has been provided by the Royal Irringation Department (herein after referred to as R1D).

3) Basic Plan for the Testing Canal Facility

Taking the objectives of the project and the project site conditions into account, the testing canal facility shall have four slopes consisting of the non-treatment slope structures and the improved slope structures as mentioned below:

a. Non-treatment slope structures

Two slope sections are planned to be constructed to leave them in the form of an untreated natural state. One of the non-treatment slope structures is to be applied for the study of long-term stability. The other non-treatment slope structure is to be used for the study of short-term stability.

b. Improved slope structures

The improved slope structures shall be undertaken for the following purposes;

(1). Countermeasures against damage and failure of the existing structures, and preparation work

(2). Study of the effects of improved methods in the improved slope structures

As examples of countermeasures among the various available improvement methods for the soft soil foundation, the final improvement methods shall be selected from three already proposed methods (sand compaction treatment, soil cement colum treatment and gravel compaction pile treatment).

4). Monitoring System

The monitoring system shall be utilized in the following way:

a. Auto-measurement monitoring system

b. Measurement and observation by topo-survey work

The auto-measurement monitoring system shall be installed at the project site. Also, the observation data obtained from the auto-monitoring system shall be recorded at the project site. The data obtained from the monitoring system shall be analized in I.E.C..

The auto-measurement monitoring instruments shall be supplied by means of JICA's procurement method.

CHAPTER 2 OUTLINE AND OBJECTIVES OF THE MODEL INFRASTRUCTURE PROJECT

2-1 Location and Present Conditions of the Project Site

The Project Site for the testing canal facility provided by RID is located about 40 km south-east of the center of Metropolitan Bangkok as the crow flies and about 600 m inland from the coast of the gulf of Thailand. The location of the project site is about latitude 13°30' north and longitude 100°45' east, under the administrative management of Samutphakan Regency in the middle of Thailand.

The project area was obtained for the construction of the Charoenraj Pumping Station, and is now utilized as quarters for the people working at the Pumping Station.

As shown in Fig. 2.1.1, the project site is a square-shaped area of about 1.5 ha and is bordered by National Road Route No.3 to the north by the Charoenraj Pumping Station and its related drainage canal to the west, and by private shrimp ponds to the east. The narrow area (about 50m in width) from the Project Site to the seashore along the drainage canal mentioned above is RID's property. The construction road utilized for the construction of the Pumping Station is being remained as damaged and slided along the drainage canal.

The ground elevation of the project site ranges from EL. 1.75 m at the highest point to EL. 0.13 m at the lowest point and the ground surface within the site of less than about EL. 1.00 m suffers from tidal intrusion and becomes inundated with sea water. The inundated area is estimated to be about 0.90 ha.

The typical profile of geological conditions obtained from the geological investigation performing in the project site are shown in Fig. 2.1.2 and Fig. 2.1.3.

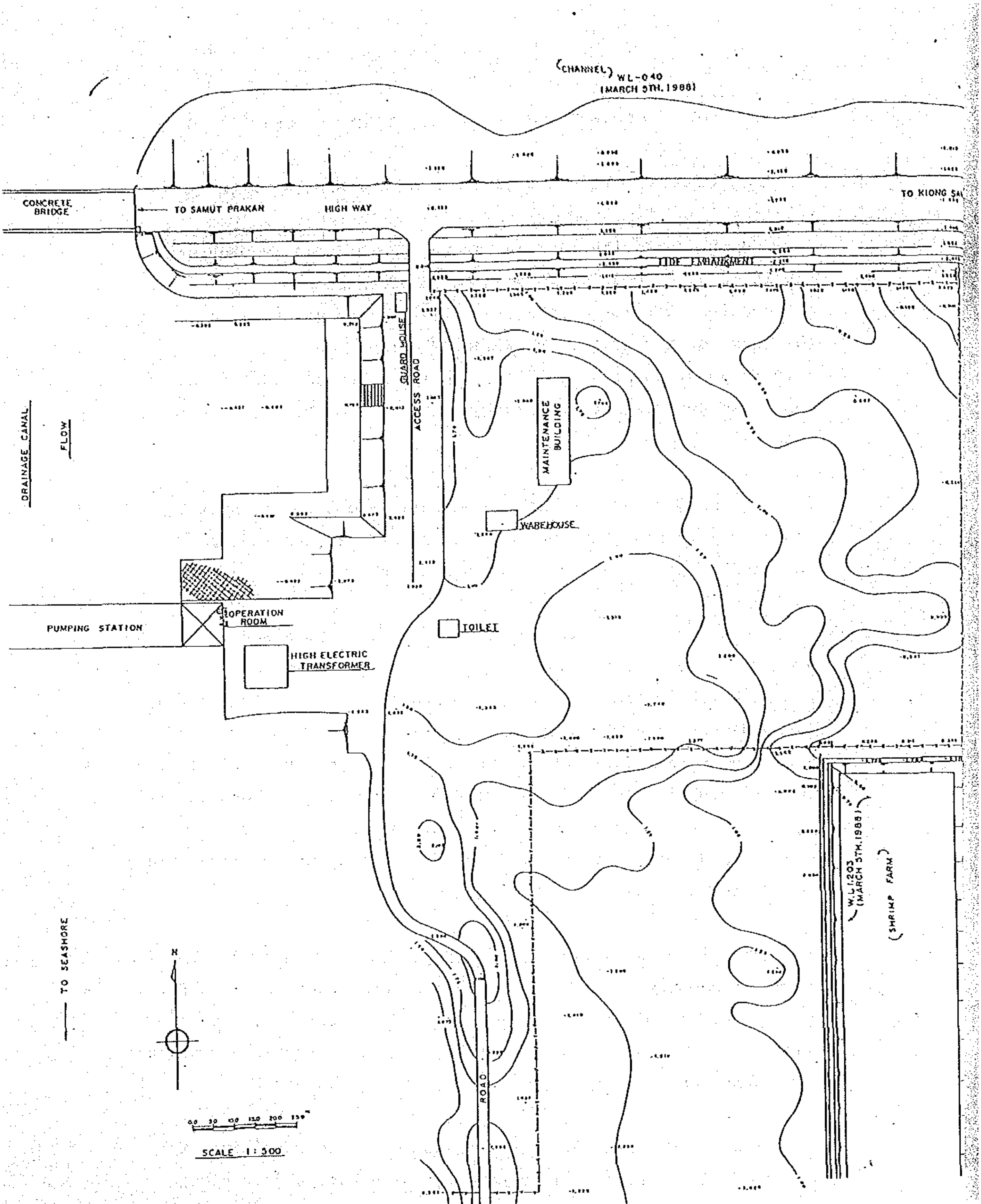


Fig.2.1.1 Topographic Condition of the Project Site

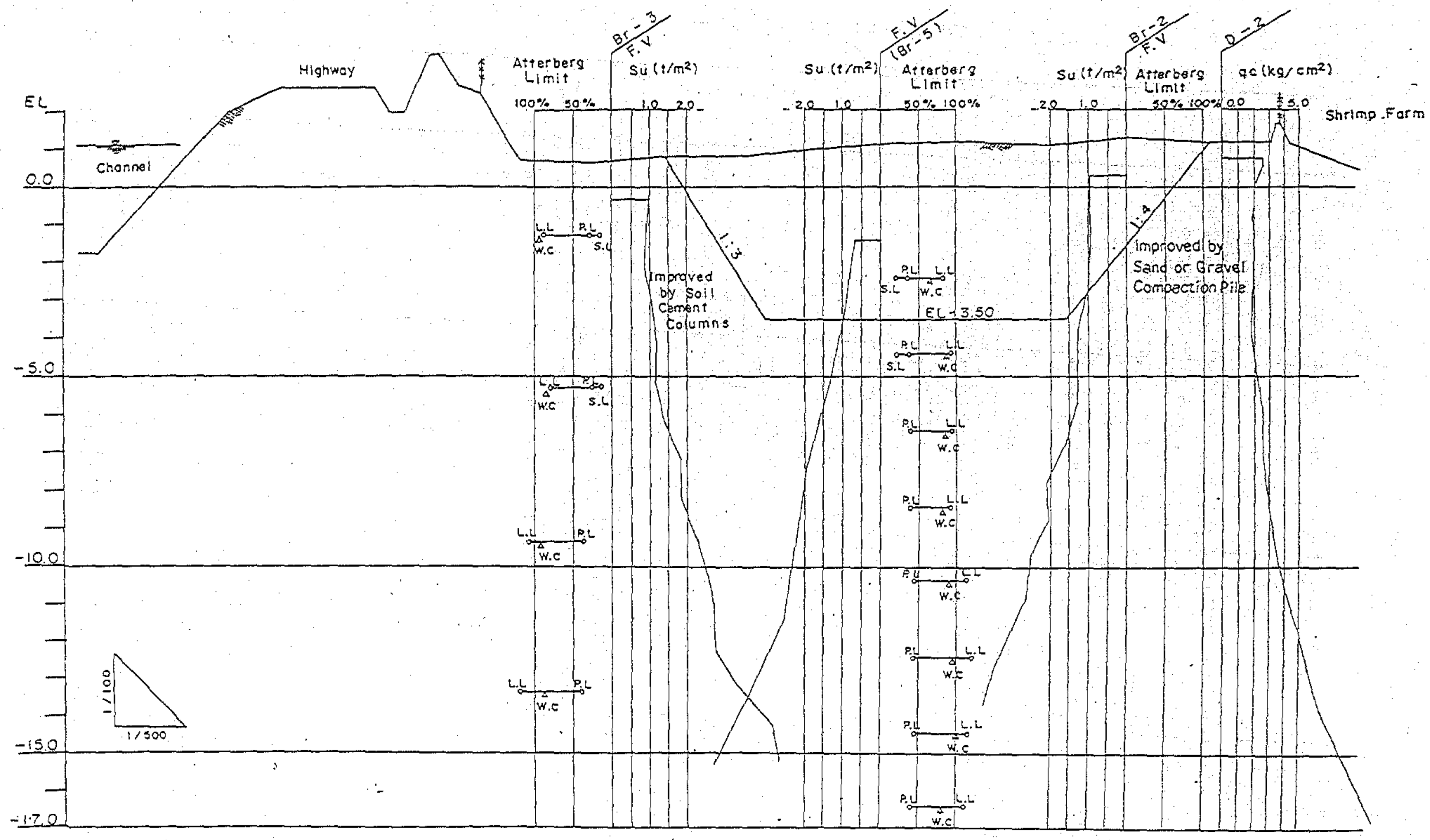


Fig.2.1.2 Geological Profile of the Project Site(1)

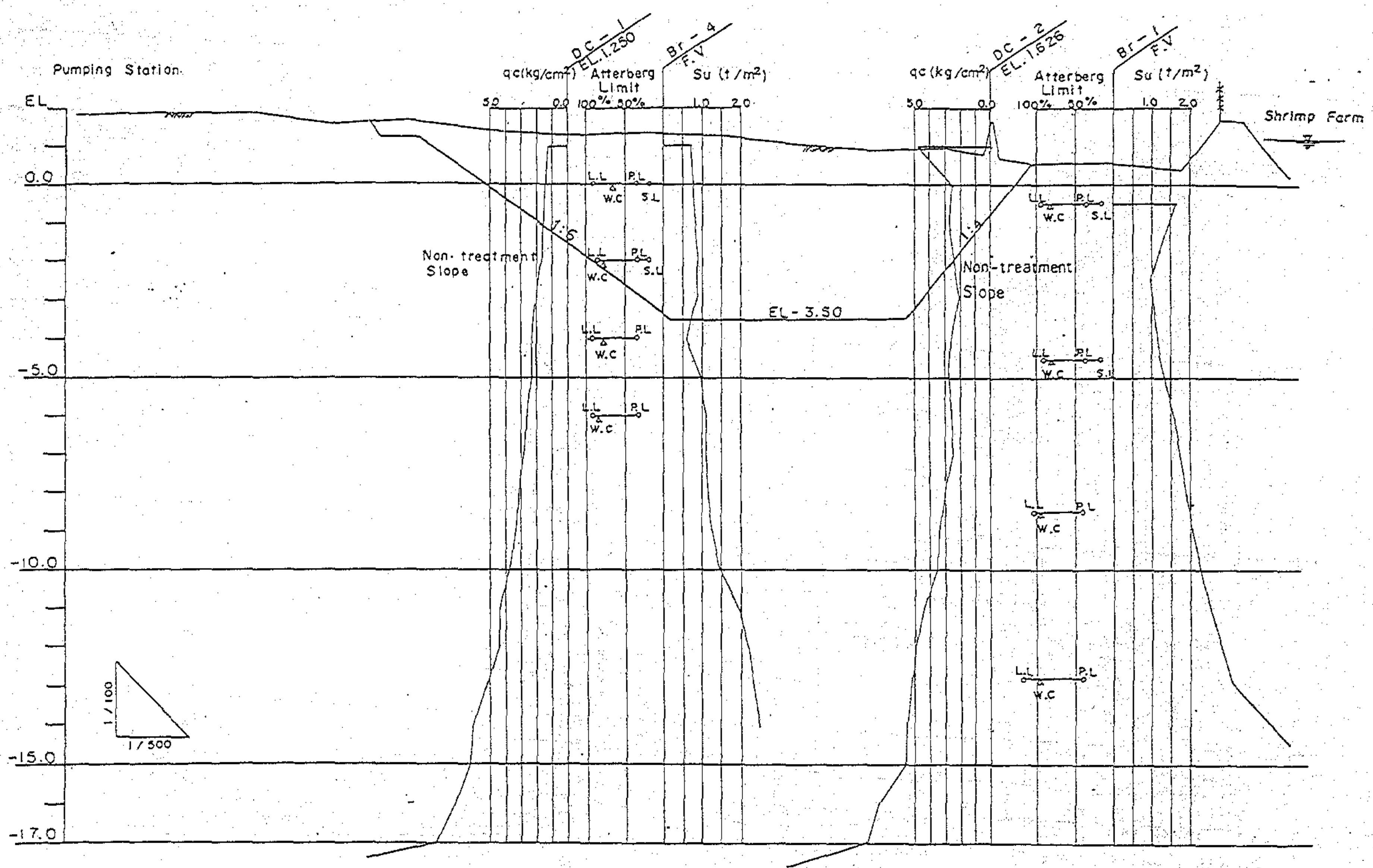


Fig.2.1.3 Geological Profile of the Project Site(2)

2-2 Objectives of the Testing Canal Facility and Types of Structures

According to the Basic Plan described in the CHAPTER 1, the construction of the testing canal facility will be carried out in order to obtain;

1) the mechanical behaviour of the excavated soft soil foundation

2) suggestions and recommendations on the design and investigation for the soft soil foundation.

Taking the area of the project, the limitation of the construction cost and the safety for the existing structures around the project area into considerations, the testing canal facility shall have four slopes as specified below:

- Depth of the testing canal facility : H=4.0m (Canal bed elevation: EL -3.50m)

- Non-treatment structure : Two (2) slopes, one slope for the study of short-term slope stability and another slope for the study of long-term slope stability

- Improved structure : Two (2) slopes, one slope treated by the use of soil cement column as a counter-measure against damage and failure of the National Road Route No.3 and another slope treated by the use of sand compaction piles for the study of effects of improved method on the improved slope

CHAPTER 3 GEOTECHNICAL INVESTIGATION AND SOIL TEST

In order to obtain the geotechnical data required for the detailed design work, the following investigations and tests were performed by RID and Siam Testing Soil Co. (herein after referred to as STS Co.)

3-1 In-Situ Tests

1). Objectives of the Tests

a. Dutch cone tests

The profile of geotechnical information is obtained from the dutch cone tests. The relation between undrained strength cohesion, c, of the foundation and cone bearing capacity is also obtained from the results of these tests.

b. Field vane shear tests

(1) The field vane shear tests are performed by drilling work.
(2) The shear strengths are the directly obtained on the basis of the Ko-condition in the field from the field vane shear tests.
(3) These tests offer the strength anisotropy of the foundation by performing tests in which the vane sizes are changed.
(4) The sensitivity ratio is also obtained form the results of these tests which are performed in the disturbed condition of the foundation.

c. Sampling

Undisturbed samples for laboratory tests are taken by using a thin wall tube sampler in the drilling work (5 holes. Depth of four holes, Br-1, Br-3, Br-4, is 15 metres. Depth of Br-5 is 17 metres).

2). Quantites of In-Situ Tests

Table 3.1.1 shows the quantites of in-situ tests and Fig. 3.1.1 shows the locations where the in-situ tests were performed.

3). Results of In-situ Tests

A summary of the results of In-situ tests are shown in Fig. 3.1.2 to Fig. 3.1.7.

The results of the in-situ tests are attached in the "ATTACHED DATA AVAILABLE" of this report.

3-2 Laboratory Tests

The following laboratory tests required for the detailed design work were performed by RID staff and STS Co.

1) Objectives

a. Physical property tests

(1) The physical characteristics of soft soil are obtained from the physical property tests.

(2) The physical property tests supply the correlation between the physical properties and the parameters using slope stability analysis and the Elasto-Viscosity Finite Element Model.

b. Mechanical property tests

i). Ko-note triaxial compression tests ($\overline{\text{CKoUC}}$)

(1) For the purposes of reproducing the stress condition occurring in the field of the soft soil foundation with consolidated undrained condition, the triaxial compression tests are performed by setting the Ko note stress condition.

(2) The shear strength necessary for the circular slip method are obtained from the results of these tests.

(3) The determination method and procedure of undrained shear strength used for slope stability analysis are referred to Chapter 4. The nine parameters for the Finite Element Analysis using the elasto-viscoplastic model are determined from the results of these tests.

ii). Consolidation tests

(1) The three parameters for the Finite Element Analysis using the elasto-viscoplastic model are determined from the results of these tests.

(2) The shear strengths for circular slip methods are determined by taking into account the characteristics of consolidation and swelling in the existing foundation.

c. Chemical property tests

(1) The chemical characteristics of soft soil are obtained from chemical property tests.

(2) The chemical characteristics (mainly leaching and

chemical swelling) are applied for the index of examining long-term slope stability.

2). Quantities of Laboratory Tests

Table 3.2.1 shows the items and quantities of laboratory tests.

3). Summary of Results of Laboratory Tests

The summary of the results of the Laboratory tests are shown in Fig. 3.1.2 to Fig. 3.1.7.

The results of the laboratory tests are attached in the "ATTACHED DATA AVAILABLE" of this report.

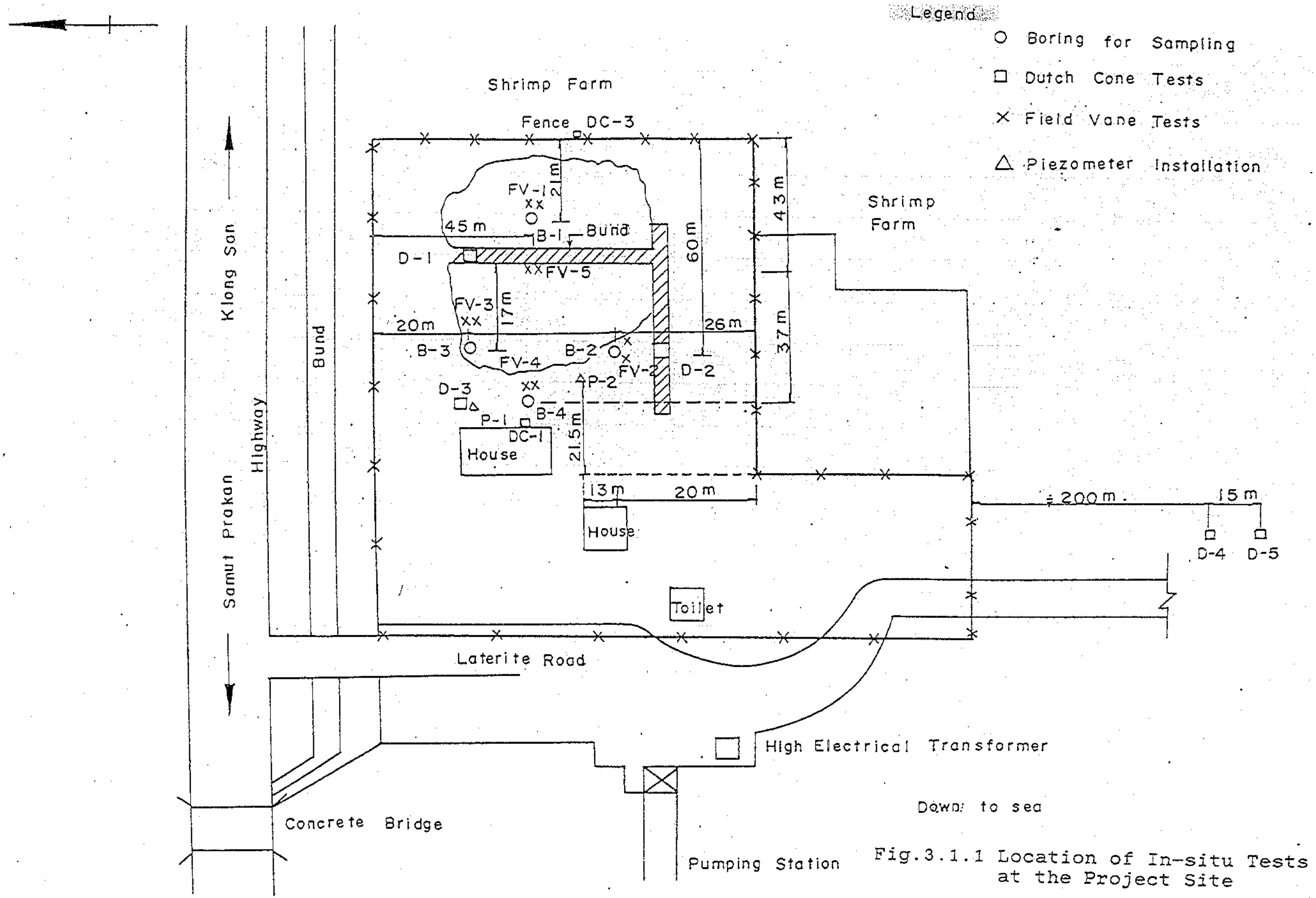


Fig.3.1.1 Location of In-situ Tests at the Project Site

Table 3.1.1 Items and Quantities of In-situ Tests

Items / Depth from GL (m)	Thin Wall Sampling by Boring					Field Vane Tests (2 Holes / Location)						Dutch Cone Test
	Br-1	Br-2	Br-3	Br-4	Br-5	Near Br-1	Near Br-2	Near Br-3	Near Br-4	Near Br-5	FV-5	
-1.0	○	○	○	○	○	○	○	○	○	○	○	DC-1, DC-2, DC-3 } at the Project site Depth : 20m Measuring : at every 1.0 m D-1, D-2, D-3 } at the Project site D-4, D-5 } at the sea side Depth : 20m Measuring: at every 20 cm
-3.0	○	○	○	○	○	○	○	○	○	○	○	
-5.0	○	○	○	○	○	○	○	○	○	○	○	
-7.0	○	○	○	○	○	○	○	○	○	○	○	
-9.0	○	○	○	○	○	○	○	○	○	○	○	
-11.0	○	○	○	○	○	○	○	○	○	○	○	
-13.0	○	○	○	○	○	○	○	○	○	○	○	
-15.0	○	○	○	○	○	○	○	○	○	○	○	
-17.0					○							
-20.0												
Amount	8	8	8	8	9	16	16	16	16	23	16	
Total	42 Samples					103 Times						8 Places

Table 3.2.1 Items and Quantities of Laboratory Tests

Items / Depth from GL (m)	Physical Properties of Soil ① Specific Gravity ② Natural Moisture C. ③ Wet Density ④ Grain Size ⑤ Liquid Limit ⑥ Plastic Limit ⑦ Shrinkage Limit					Mechanical Tests: Triaxial Compression Tests $\overline{\text{CKoUC}}$			Mechanical Tests: Consolidation Tests			Chemical Properties 1 PH 2 Chloride C. 3 Sultate C. 4 Organic M. 5 Salinity 6 Mineral C.		Physical Properties of Sand 1 Specific Gravity 2 Grain Size
	Br-1	Br-2	Br-3	Br-4	Br-5	Br-1	Br-3	Br-5	Br-1	Br-3	Br-5	Br-1	Br-2	
-1.0	①~⑦	①~⑦	①~⑦	①~⑦	①~⑦	○	○	○	○	○	○*	○	○	Sand taken from Mae Klong river and Chao Phya river.
-3.0	①~③			①~⑦	①~⑦	○		○	○		◎			
-5.0	①~⑦	①~⑦	①~⑦	①~⑥	①~⑥	○	○	○	○	○	◎	○	○	
-7.0	①~③	①~③		①~⑥	①~⑥	○		○	○		○*			
-9.0	①~⑥	①~⑥	①~⑥		①~⑥	○	○	○	○	○	○*	○	○	
-11.0	①~③	①~③		②,③	①~⑥	○		○	○		○*			
-13.0	①~⑥	①~⑥	①~⑥		①~⑥	○	○	○	○	○	○*	○	○	
-15.0	①~③	①~③		②,③	①~⑥	○		○	○		○*			
-17.0					①~⑥						○*			
Amount	8	7	4	6	9	8	4	8	8	4	9	4	4	
Total	34 Samples					20 Samples			21 Samples			8 Samples		4 Samples

Remarks ; ○* : One cycle & secondary consolidation
◎ : Two cycle & secondary consolidation

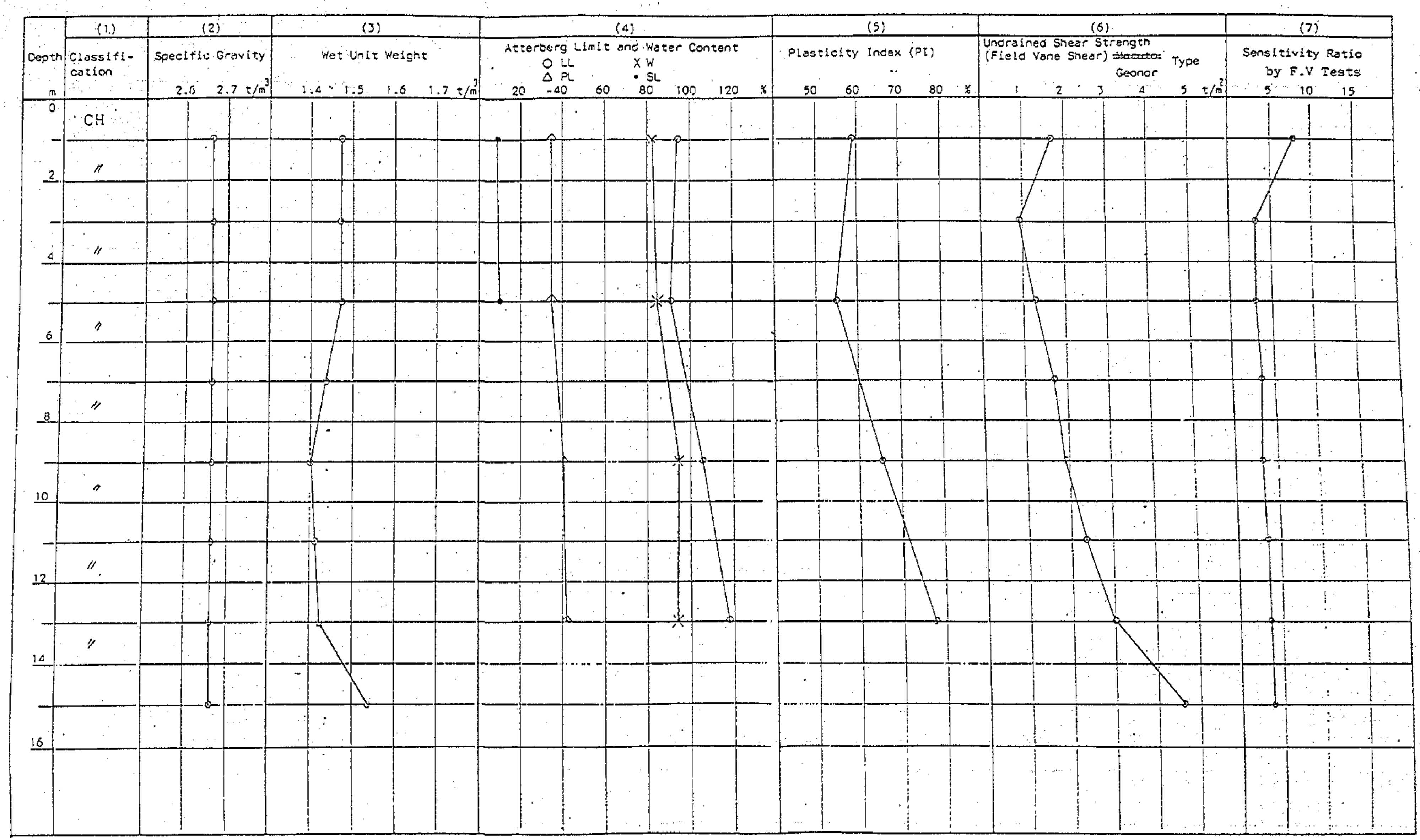


Fig.3.1.2 Summary of Test Results at Boring No.Br-1

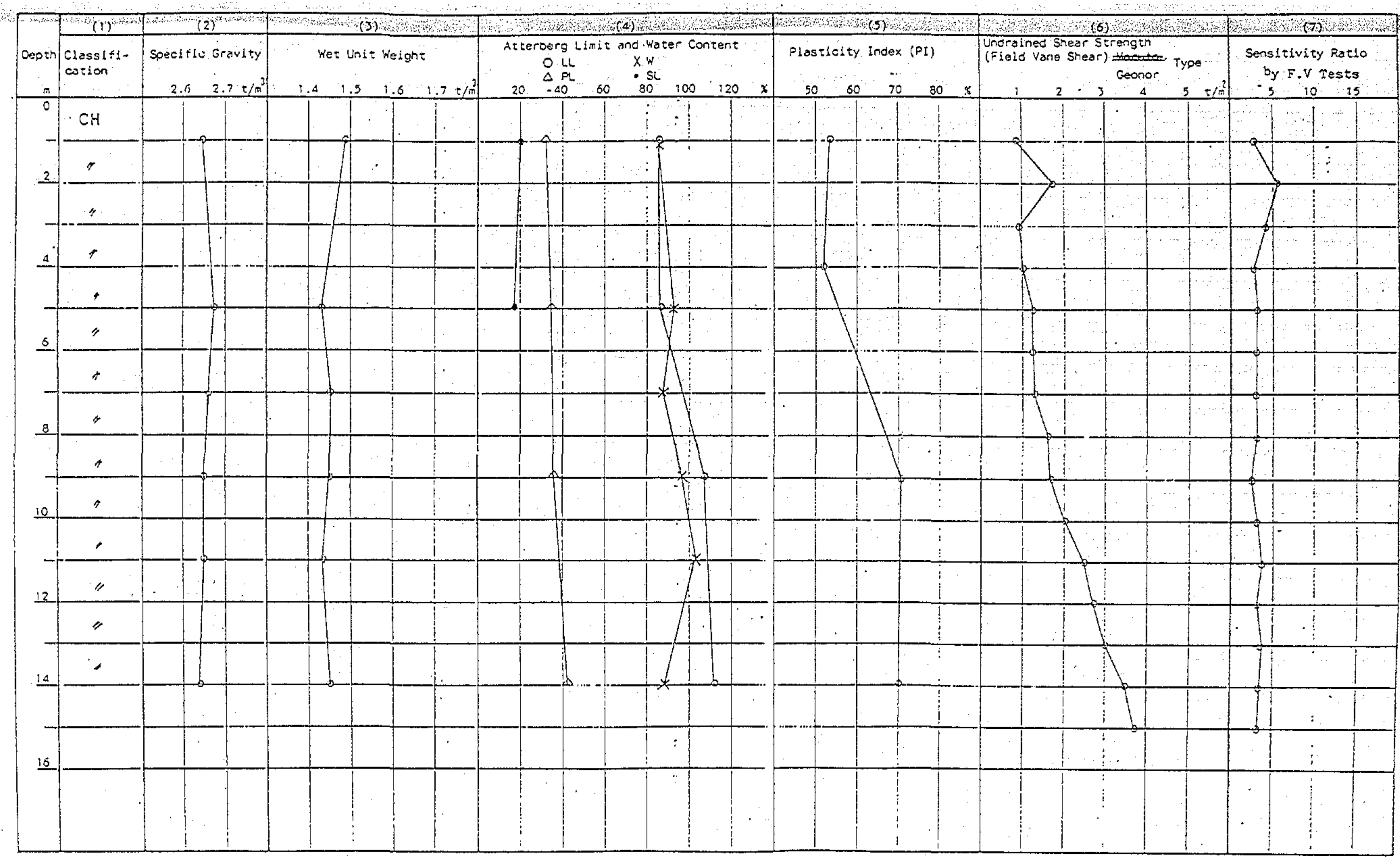


Fig.3.1.3 Summary of Test Results at Boring No.Br-2

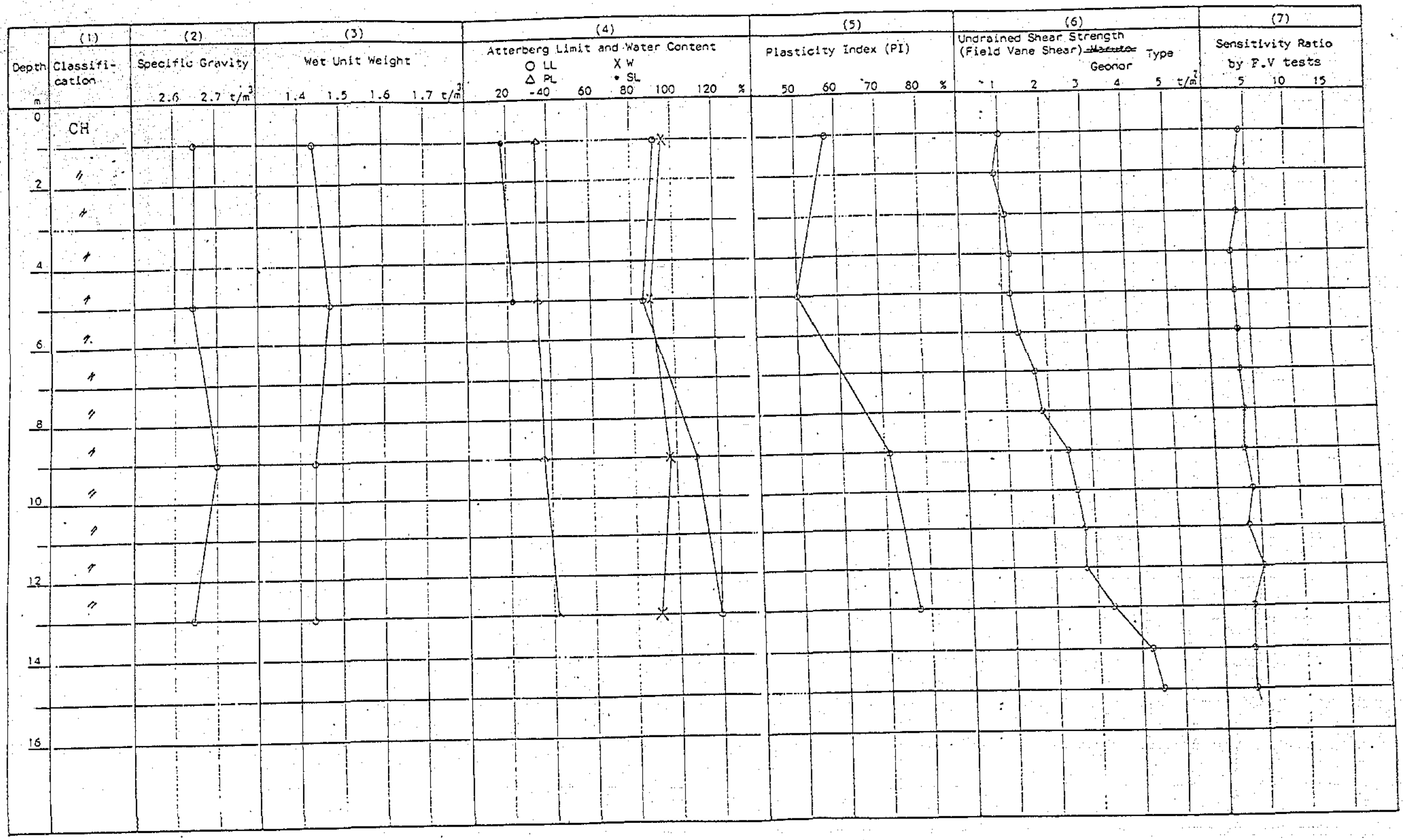


Fig.3.1.4 Summary of Test Results at Boring No.Br-3

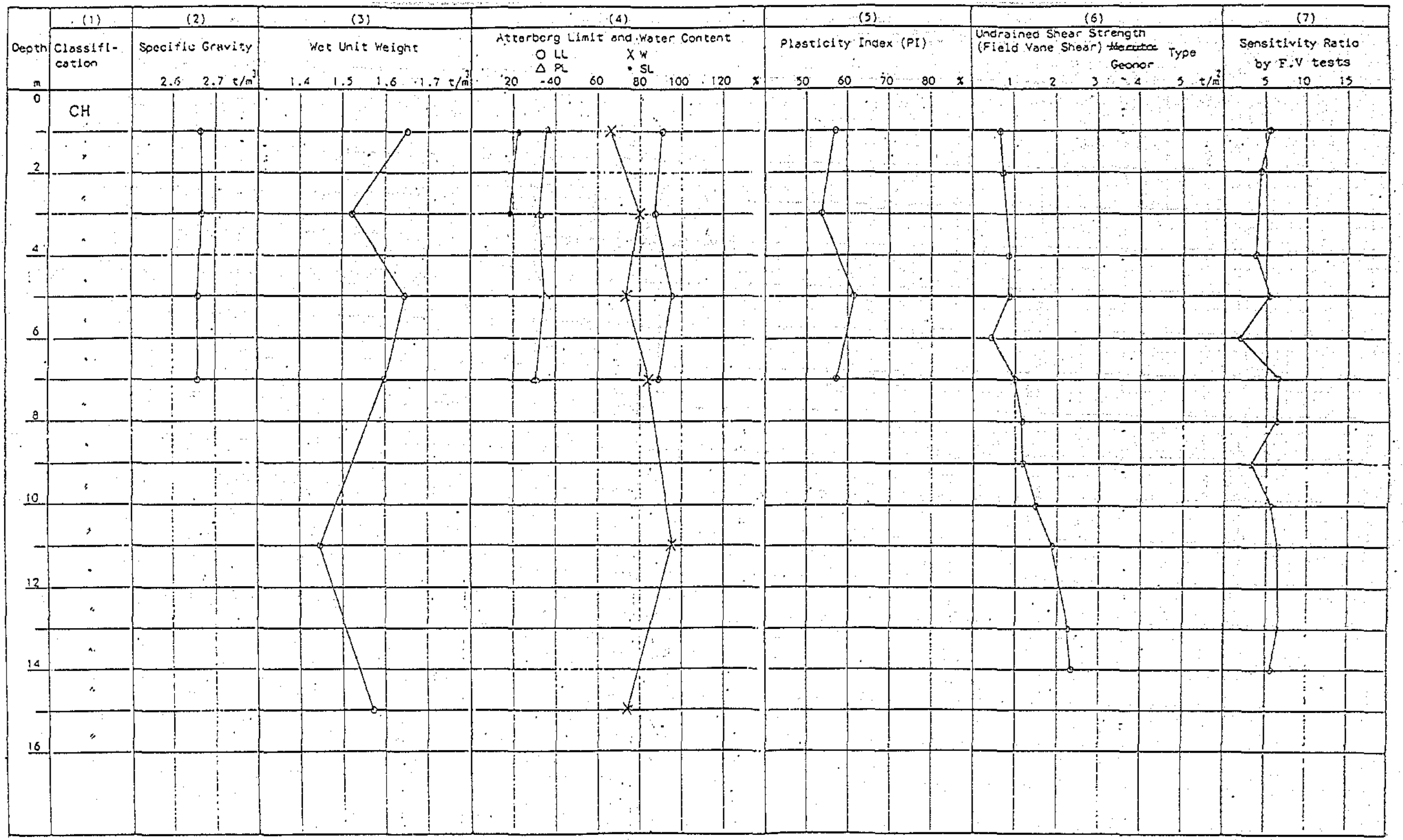


Fig.3.1.5 Summary of Test Results at Boring No.Br-4

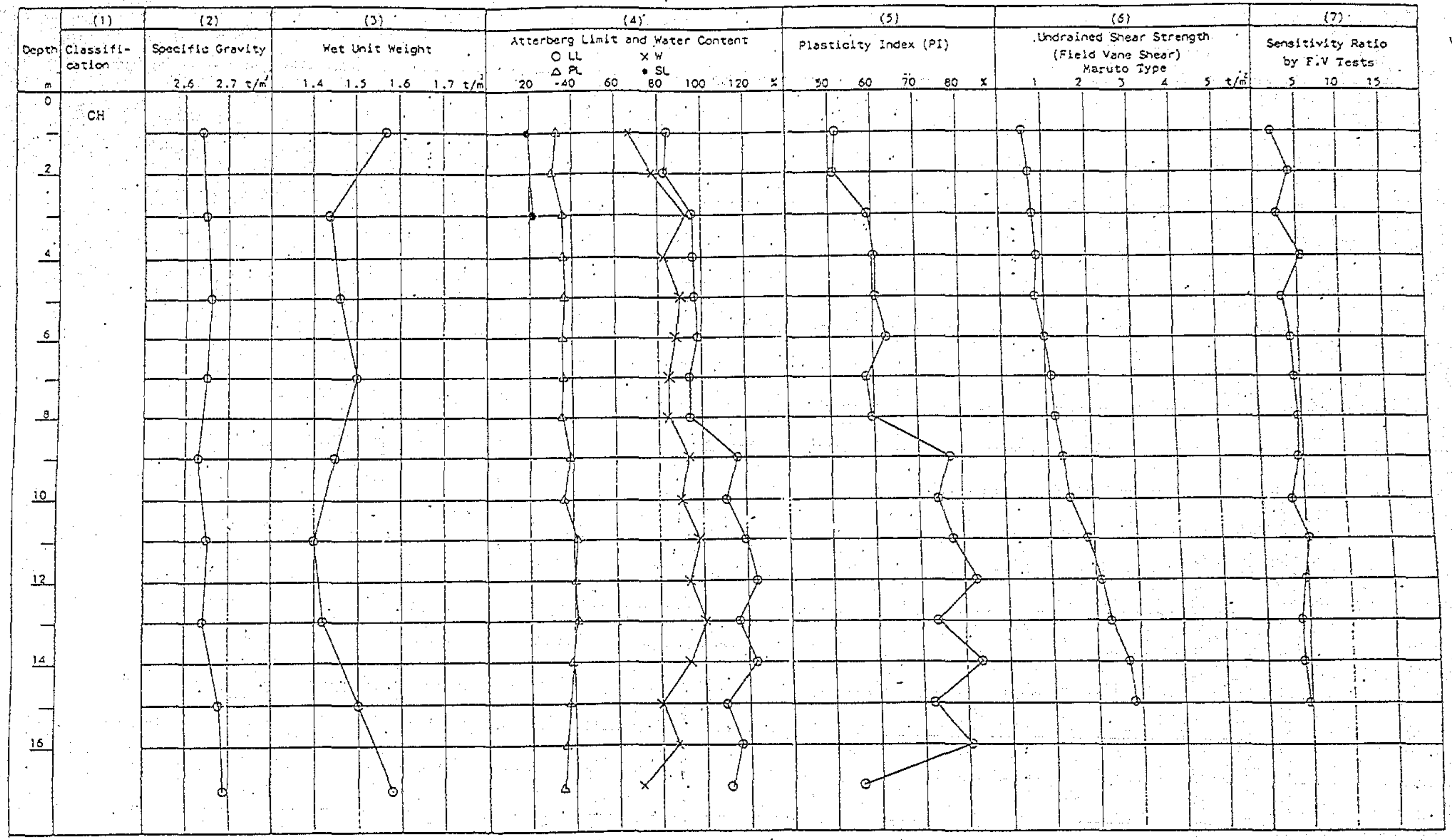


Fig.3.1.6 Summary of Test Results at Boring No.Br-5

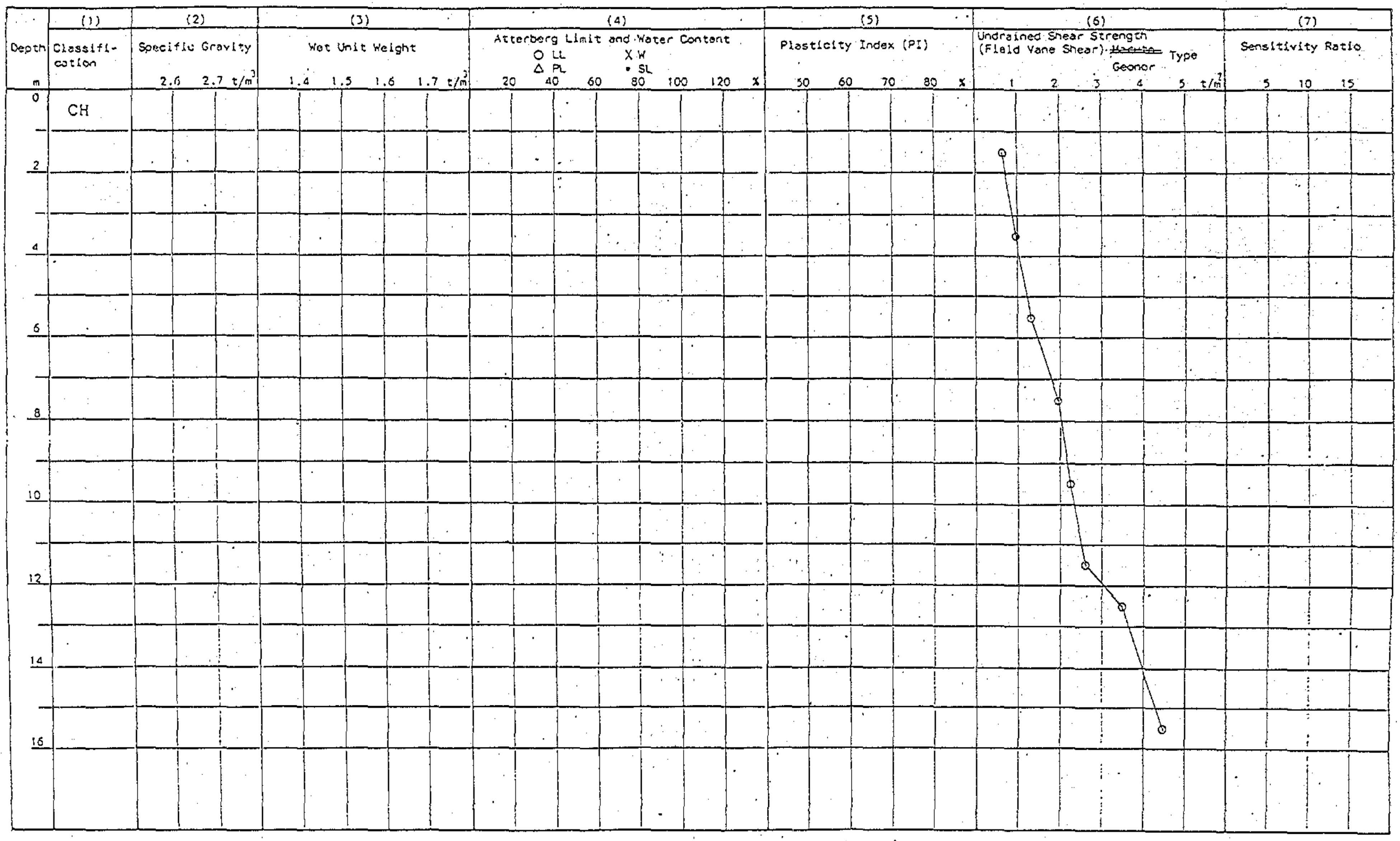


Fig.3.1.7 Summary of Test Results at Field Vane Test FV-5

3-3 Evaluation on the Results of Geotechnical Investigation and Soil Tests

The data arrangement of the results of in-situ tests and their evaluation are carried out in accordance with the flow chart shown in Fig. 3.3.1.

A. Physical properties of clay

- 1 Classification of soil
- 2 Specific gravity (Gs)
- 3 Natural moisture contents (Wn)
- 4 Liquid limit (LL)
- 5 Plasticity limit (PL) Atterberg limit
- 6 Plasticity index (PI)
- 7 Shrinkage limit
- 8 Wet density (t)
- 9 Grain size distribution

The geological structure and mechanical characteristic of clay foundation are grasped from the physical properties.

B. Chemical properties of clay

- 1 PH
- 2 Chloride content
- 3 Sulfate content
- 4 Organic matter content
- 5 Salinity
- 6 Clay mineral

Based on the chemical properties,

1) Design of mix proportion for admixture of soil cement columns.

Fig.3.3.1 Flow Chart of Data Arrangement of In-situ and Laboratory Tests(1)

2) Grasping ingredients of clay mineral and study on leaching and swelling occurring in soft clay so as to refer to analysis for the long term slope stability.

C. Mechanical properties of soil

- 1 Field vane tests →
  - 1) Comparison between data used STS's vane and Maruto's one
  - 2) Study on ZH and ZV (H/D=1:1, 2:1)
  - 3) Correlation between PI and Bjerrum's coefficient
- 2 Dutch cone tests
- 3 Consolidation tests →
  - 1) Study on Cc and Cs
  - 2) Plot mv and Cv
  - 3) Secondary consolidation coefficient α^{e}
  - 4) Velocity of volumetric strain Vo (confirm t_{90})s
  - 5) Plot and check value of pre-consolidation pressure
  - 6) Plot eo of void ratio
  - 7) Plot $e \sim \log 10$ P curves and σ'Vi

    ↓

    Value of σ'v to be studied on swelling (expansion)

    ↓
  - 8) Plot OCR of original ground (before excavation) by depth

i) $\alpha = 0.434 \cdot \frac{-dV}{d(\log t)}$

ii) $\alpha = 0.434 \cdot \frac{-de}{1 + eo}$

$= \frac{0.434}{1 + \varepsilon o} \cdot \frac{-de}{(\log t)}$

Fig.3.3.1 Flow Chart of Data Arrangement of In-situ and Laboratory Tests(2)

9) Calculation of values of Ko and Ki

↓

Comparison with theoretical value (critical state model)

10) Initial value of coefficient of permeability, Ko

4 $\overline{\text{CKo Vc}}$ Tests →

1) Plot C', φ' and Af by depth

2) Stress pass in q'∿p' flame

↓

i) Critical stress ratio, M

($\leftarrow \overline{\text{CM}}$ M=(q/p')crit)

ii) Dilatancy coefficient, D

$$D = \frac{\lambda - K}{(1+eo)\cdot M}$$

(eo: initial void ratio)

↓

Distribution chart of each mechanical parameters by depth

Fig.3.3.1 Flow Chart of Data Arrangement of In-situ and Laboratory Tests(3)

1) The results of tests on physical properties of clay in the test site are shown in Fig. 3.3.2 as follows. Physical properties of Bangkok clay obtained from Proposed New Airport Area are also compared with those results.

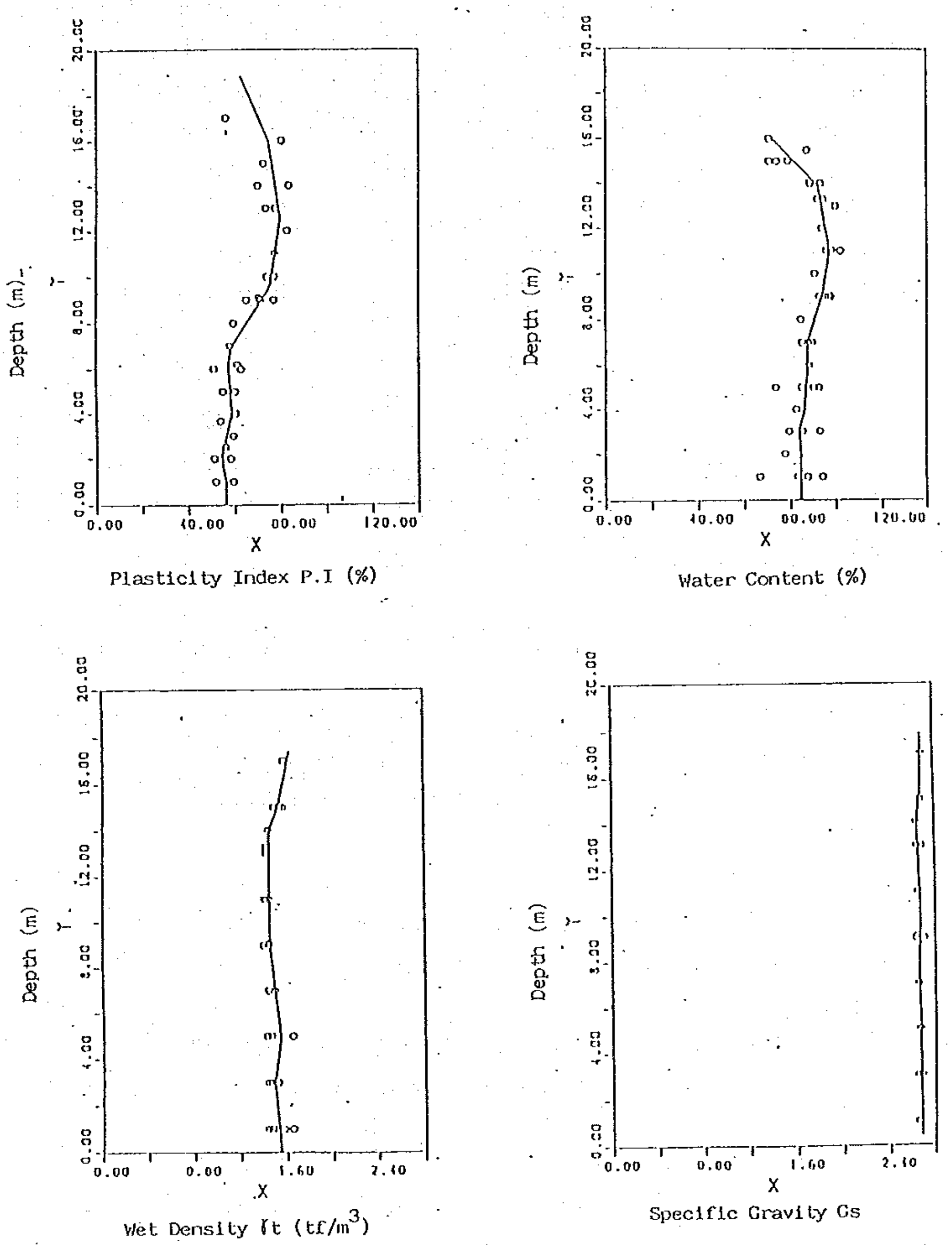


Fig.3.3.2 Physical Properties of Soil at Project Site

Table 3.3.1 Comparison of Physical Properties Clay at Project Site and Other Site

Item	Bangkok Clay in the TestSote	Bangkok Clay in other places *1
Soil Classification	CH	CH
Specific gravity (Gs)	2.62 to 2.70	2.65 to 2.70
Liquid Limit (%)	86.0 to 120.0	70.0 to 130.0
Plastic Limit (%)	32.0 to 42.0	40.0 to 60.0
Plasticity Index	50.0 to 80.0	30.0 to 50.0
Wet Weight	1.40 to 1.65	1.45 to 1.70
Natural Moisture Ratio	65.0 to 100.0	70.0 to 110.0
Void Ratio when pre-consolidated	1.70 to 2.70	2.00 to 3.00

*1 Reported by Dr. Surachat on "Basic properties of Bangkok Clay and Ariake Clay" in the International Exchange Seminar held on July, 1987 in Japan

The test site is located at the coastal area, on the other hand, Proposed New Airport Area is located comparatively in inland area as shown in Fig. 3.3.3, therefore, its geological properties are somewhat different from those of the test site where plasticity index, P.I. (%) in the test site is 20%～30% higher than that in Proposed New Airport Area.

This affects connection of undrained shear strength, Su, by field vane test considerably as mentioned in Chapter 4. As to the other items the test results of two kinds of Bangkok clay show similar tendencies.

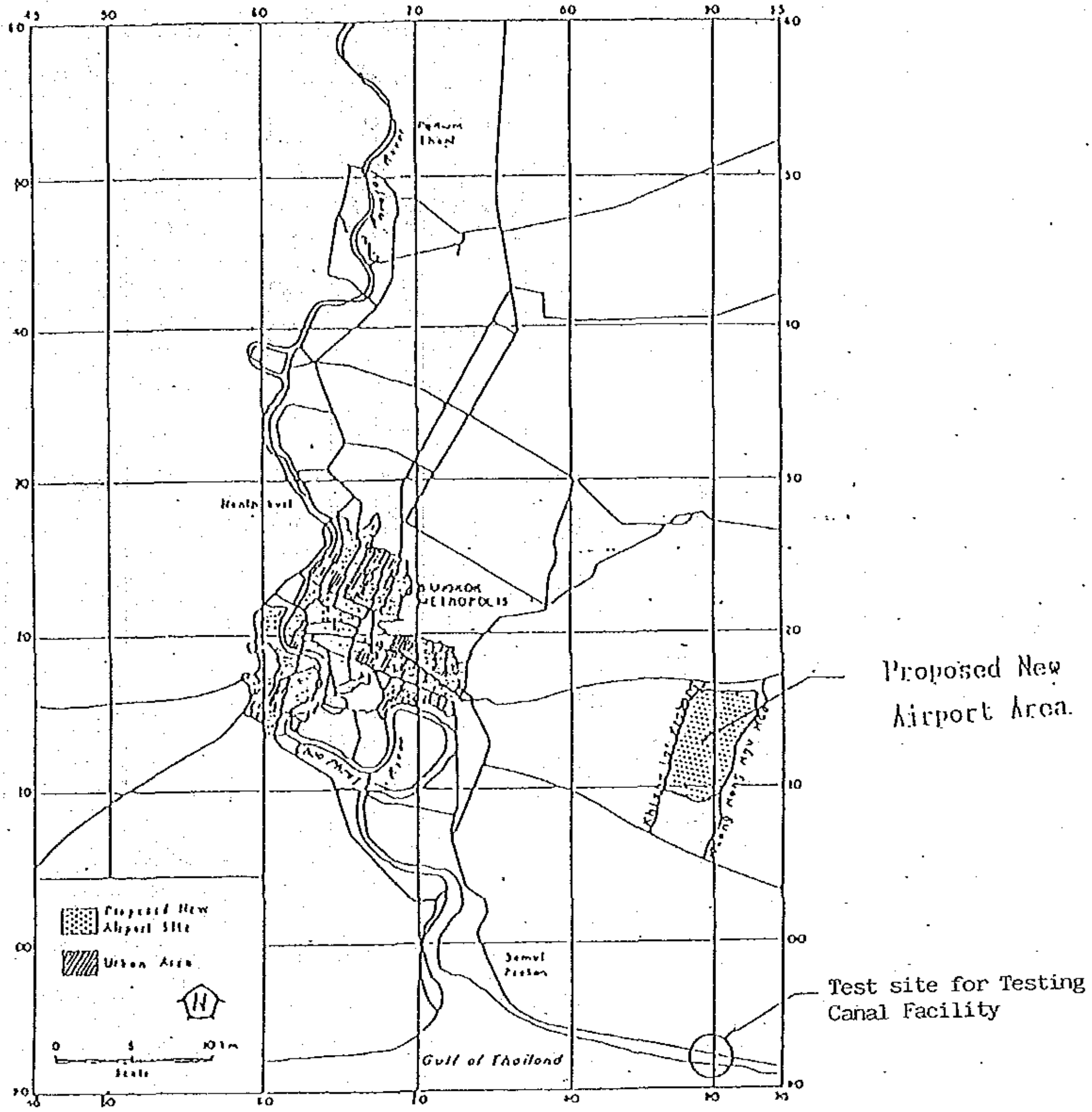


Fig.3.3.3 Locations of Project Site and Proposed New Airport Area

2) Chemical Properties of Clay

In regard of chemical properties of Bangkok Clay in the test site, data obtained from samples of boring No. 1 and No. 2 are as shown in the following Table 3.3.2.

Table 3.3.2 Chemical Properties of Soil at Project Site

Boring No.	Sample No.	Depth, m	pH	Salinity ppt	Cl^- ppm	$SO_4^=$ ppm	O.M. %	Remark
B-1	PST-1	1.00-1.00	6.7	26	14576	400	4.8	In the ponded area
	PST-3	5.00-5.00	7.2	21	11162	260	2.9	
	PST-5	.00-9.00	7.0	16	9291	345	3.5	
	PST-7	13.30-14.10	7.9	14	7616	365	3.6	
B-2	PST-1	1.00-1.00	7.6	23	11000	91	3.8	on land.
	PST-5	5.00-5.00	7.4	22	13101	31	2.9	
	PST-9	.10-9.90	7.1	12	9430	30	3.2	
	PST-14	14.00-14.80	8.3	14	7731	90	3.7	

Values of pH are within the range from 6.7 to 8.3 and the surface portion of clay exhibits slightly acidic properties. It is possible to improve the strength of clay very much by adding burnt lime (calcium oxide) or cement. Salinity decrease from the surface of 26 ppt to the deeper portion of 14 ppt, which suggests that influence of leaching is not so serious. Organic matters (OM%) is from 2.9% to 4.8% and show high value in the surface portion (around the depth from 1 m to 2 m).

This is necessary to be considered for the design of mix proportion of soil cement column.

The result of X-ray analysis on the clay is shown in Fig. 3.3.4. The result is not enough for the determination of a main component composing the clay, however, the main component seems to

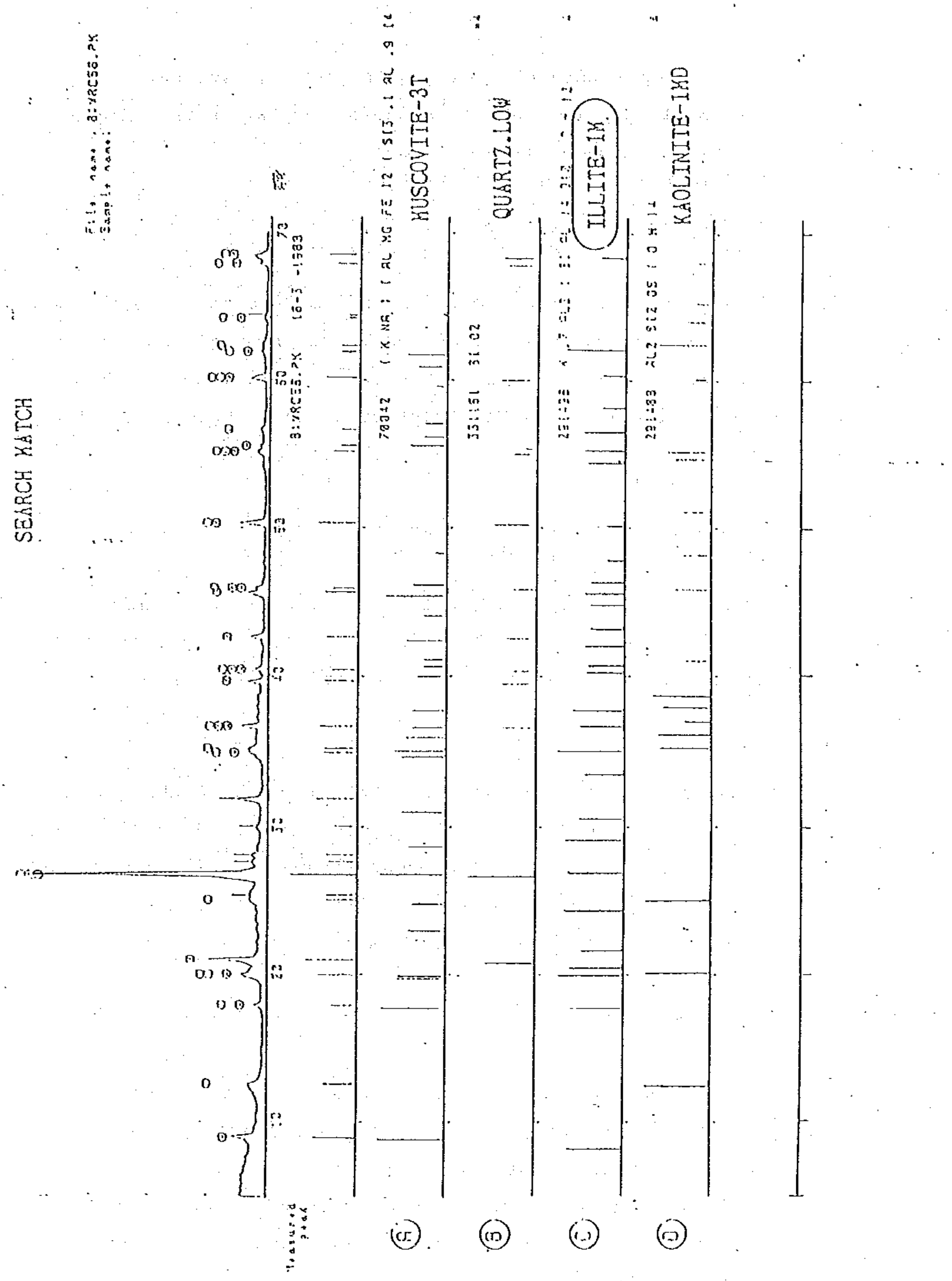


Fig.3.3.4 Results of Spectrum Analysis by X-Ray

be a kind of illite clay considering the form of spectra distribution. If so, its rebound by absorbing water is thought to be not so big as montmorillonite's one.

3) Mechanical Properties of Clay

Mechanical properties are examined by field vane test, dutch cone test, standard consolidation test and by Ko triaxial consolidation test to obtain undrained shear strength, qu value, consolidation properties (swelling index Cs, compression index Cc, initial void ratio eo) etc., which are studied in the following.

(1) Field vane test → undrained shear strength

Distribution by depth and average values of the results of field vane test are shown in Fig. 3.3.5. According to the figure the strength in the surface portion is slightly high because of drying shrinkage and aging.

The strength increases linearly from 2 ∿ 3 m to about 14 m deep, and the soil layer within this range is judged to be a homogeneous clay layer. The shear strength is small compared with the strength of clay in the Proposed Airport Area and seemed to exhibit properties of marine clay in the coast.

(2) Dutch cone test → value of cone penetration resistance

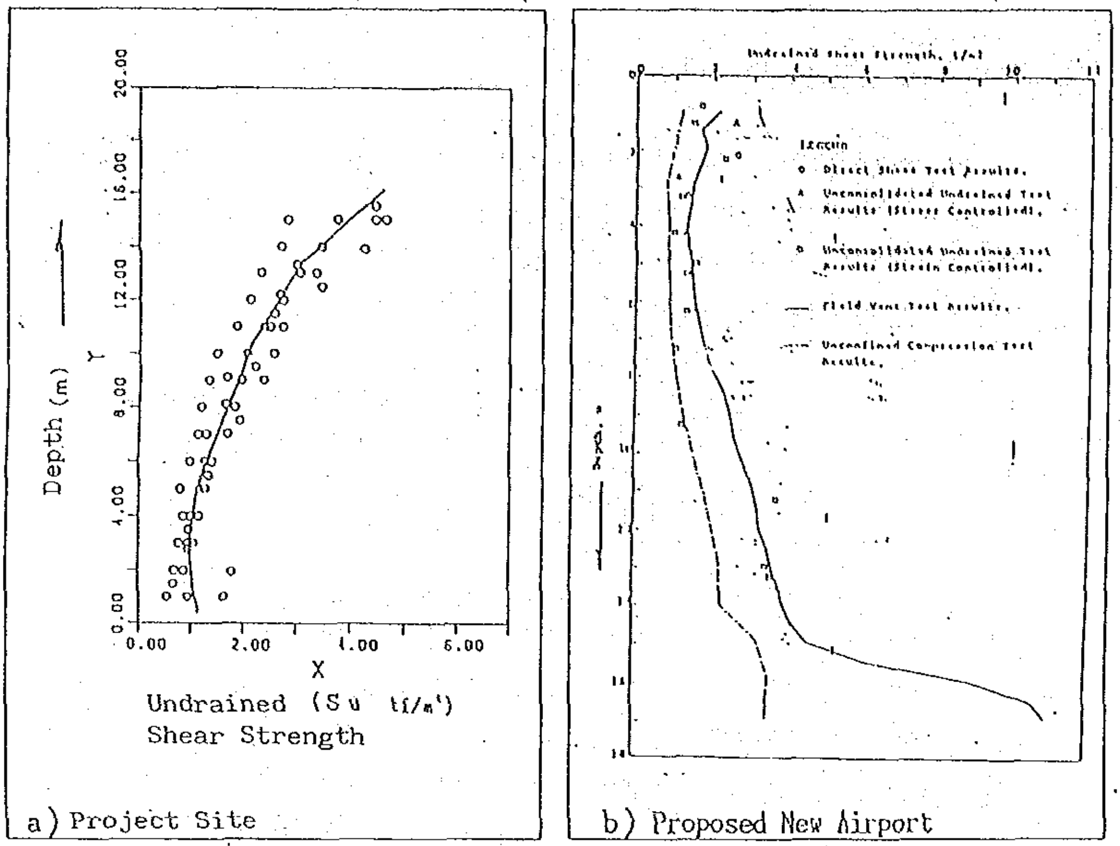
As shown in Fig. 3.3.6, results obtained between the test point D1 and D5 exhibit almost the same tendency, and properties of horizontal strength of the test site seems nearly homogeneous as well as the result of field vane test. The existence of stiff clay is found in the earth about 18 m deep.

(3) Standard consolidation test

Standard consolidation test by depth was carried out using samples obtained from Boring 1, 3 and 5 as mentioned in 3-2. The following factors plotted by depth based on the results of the test are shown in Fig. 3.3.7 ∿ 3.3.9.

1)	Void ratio	eo (pre-consolidation pressure condition)
2)	Pre-consolidation pressure	$\sigma'vo$
3)	Compression index	Cc
4)	Swelling index	Cs
5)	Coefficient of consolidation	Cv (cm^2/s)
6)	Coefficient of volume compressibility	mv (cm^2/s)
7)	Coefficient of permeability	K $(= mv\ Cv\ \gamma\omega \cdot 10^{-3} cm/s)$

Judging from Fig. 3.3.7, the void ratio exceeds 2.5 in the layer from 3.0 m to 8.0 m deep, and this value is somewhat bigger than values of other layers, which suggest that undrained shear strength may be slightly weak compared with other layers' strength. The permeability coefficient generally exhibits small values of 10^{-7} cm/sec level as shown in Fig. 3.3.8. Compression index, swelling index, coefficient of consolidation and coefficient of volume compressibility are shown in Fig. 3.3.9, and these values seem to be in a reasonable level as marine clay.



a) Project Site

b) Proposed New Airport

Fig.3.3.5 Undrained Shear Strength by Field Vane Tests(Su)

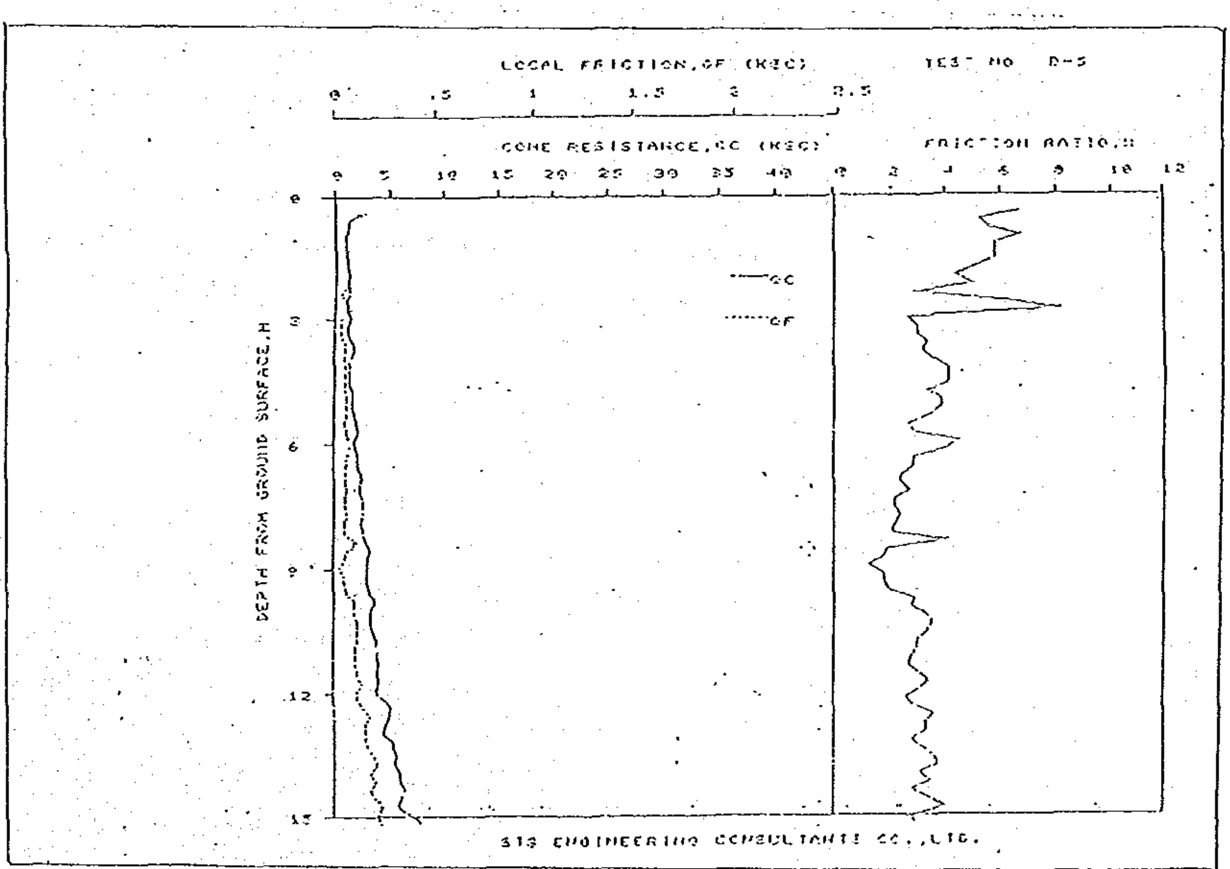


Fig.3.3.6 Cone Resistance by Dutch Cone Tests(qc)

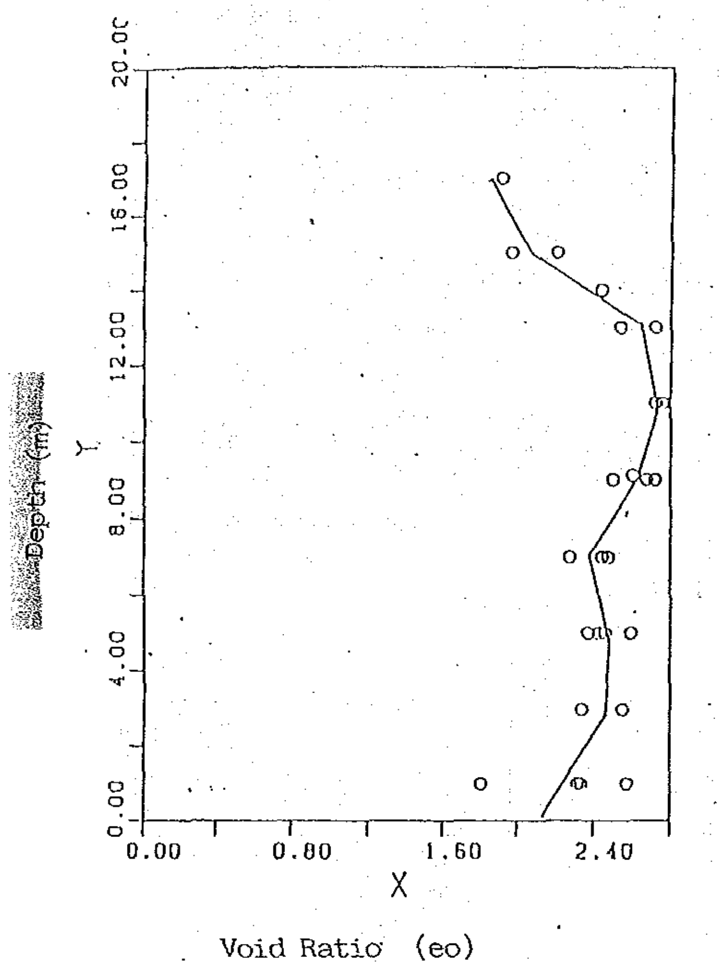


Fig.3.3.7 Void Ratio(eo)

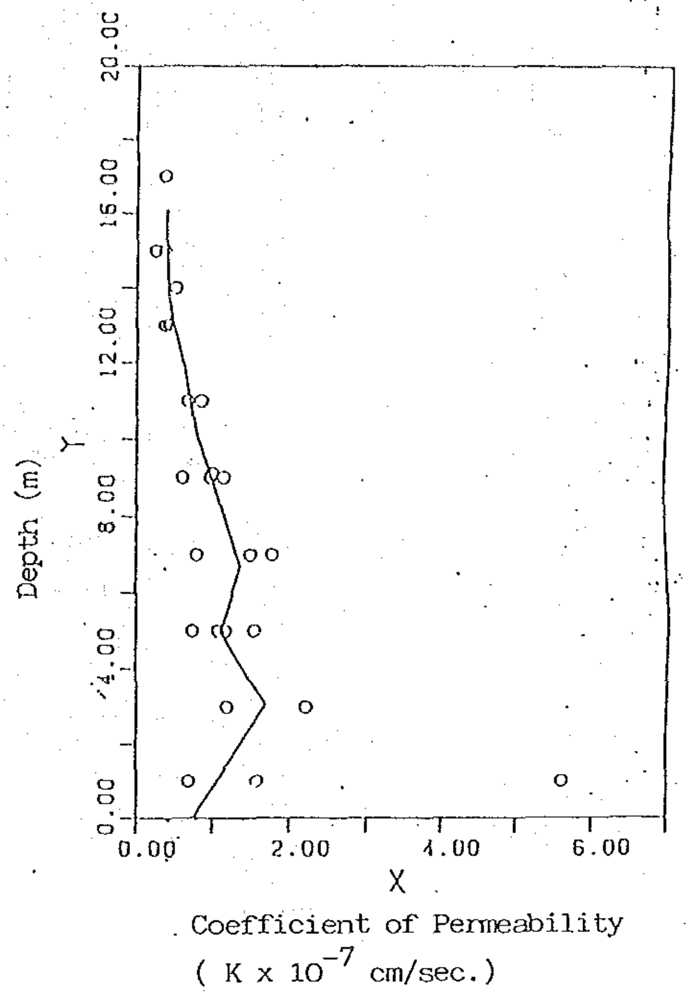


Fig.3.3.8 Coefficient of Permeability(k)

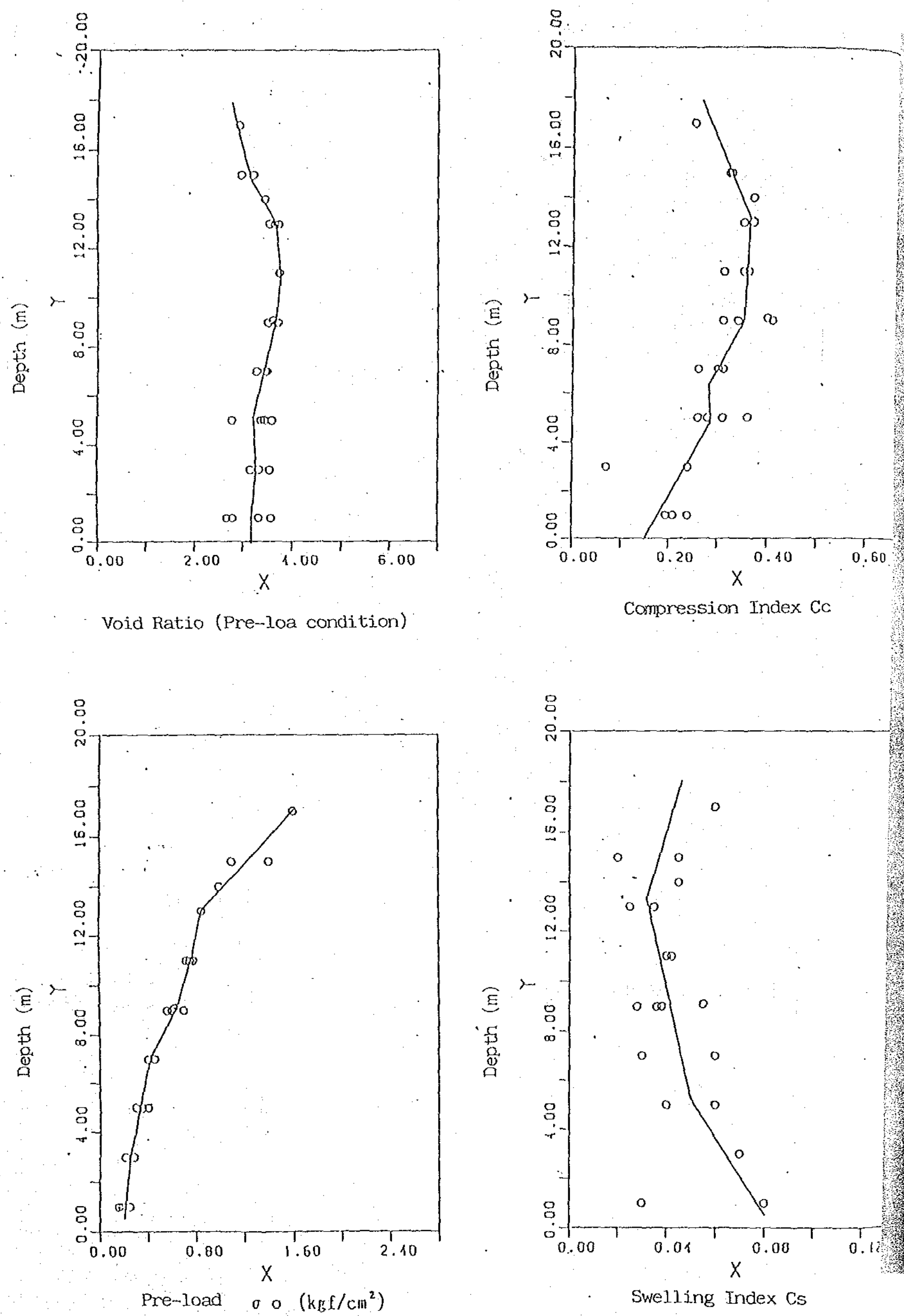


Fig.3.3.9 Distribution of Mechanical Coefficients

(4) Ko triaxial compression test

Internal friction angle in terms of effective stress .is obtained from the results of Ko triaxial compression test where anisotropy is taken into account. Using the value of ϕ, critical stress ratio M which is applied to FEM analysis is calculated by the following equation and plotted in Fig. 3.3.10.

The value of ϕ' in the surface layer is big as well as the reuslt of field vane test as far as the figure concerned, however, the value in the layer deeper than 5 m doesn't increase so much, which suggest that test samples get disturbance.

Critical stress ratio $M = \dfrac{6 \sin \phi'}{3 - \sin \phi'}$

where ϕ': internal friction angle in terms of effective stress, using value of $(\sigma_1' - \sigma_3')f$.

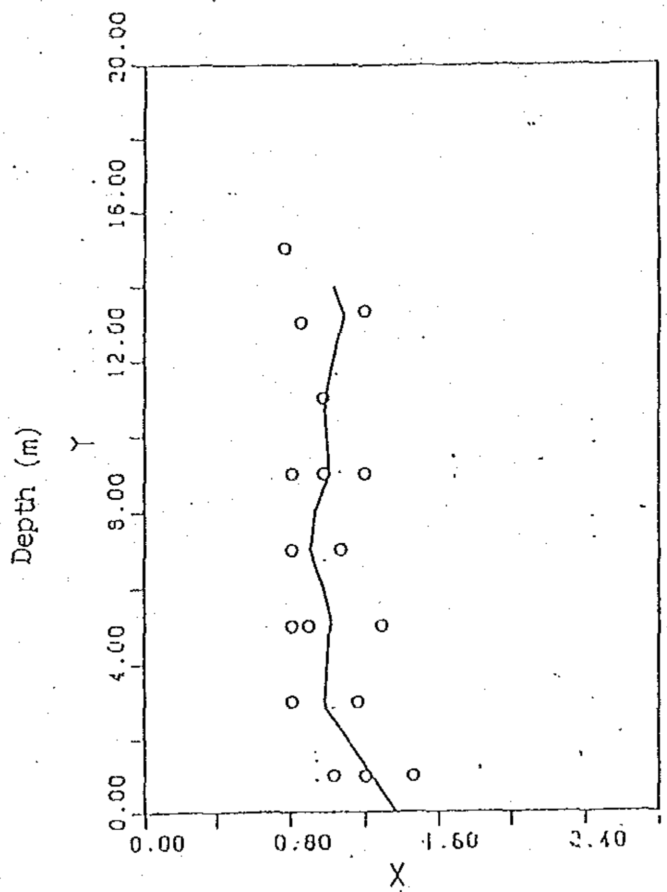


Fig.3.3.10 Critical Stress Ratio(M)

Judging from the obtained results of Bangkok Clay in the test site through various tests over mechanical properties (1) ∿ (4) mentioned above, the properties of the clay is considered to be reasonable in general, therefore, those properties are applied to the determination of parameters for slope stability and elasto-visco-plastic FEM analysis in chapter 4.

Table 3.3.3 Distribution of Plasticity Index(PI %)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	59.3					51.7
1.50						
1.95		53.6				
2.00			58.1			51.1
2.50				55.7		
2.80						
3.00						59.3
3.50						
3.66				53.8		
4.00						60.5
5.00	55.0					60.4
5.50						
6.00		51.7	50.7			63.0
6.20				61.5		
7.00				57.8		58.6
7.50						
8.00						59.8
8.10						
9.00	65.4		71.5			77.3
9.10		70.6				
9.50						
10.00	77.4					74.2
10.10						
11.00						77.7
11.50						
12.00						82.9
12.20						
12.50						
13.00			77.7			73.0
13.30						
13.90						
14.00		70.1				83.5
15.00						72.4
15.50						
16.00						80.3
17.00						56.2

Table 3.3.4 Distribution of Moisture Content(W %)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	83.2	87.5	94.2	66.4	66.8	
1.50						
1.95						
2.00					77.5	
2.50						
2.80						
3.00	85.2			79.4	93.0	
3.50						
3.66						
4.00					82.6	
5.00	84.9	92.6	90.3	73.8	90.0	
5.50						
6.00					88.2	
6.20						
7.00	89.7	87.8		85.3	85.4	
7.50						
8.00					84.9	
8.10						
9.00	95.8		98.4		93.1	
9.10		97.2				
9.50						
10.00					91.0	
10.10						
11.00	98.1	102.5		96.2	98.8	
11.50						
12.00					94.2	
12.20						
12.50						
13.00					100.2	
13.30	94.7		92.3			
13.90						
14.00		88.9			93.5	
15.00	70.9			74.3	79.2	
15.50					87.5	
16.00					70.9	
17.00						

Table 3.3.5 Distribution of Wet Density(γt)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	1.47	1.49	1.44	1.65	1.57	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	1.47			1.52	1.44	
3.50						
3.66						
4.00						
5.00	1.47	1.43	1.47	1.65	1.46	
5.50						
6.00						
6.20						
7.00	1.44	1.45		1.50	1.50	
7.50						
8.00						
8.10						
9.00	1.40		1.44		1.45	
9.10		1.45				
9.50						
10.00						
10.10						
11.00	1.41	1.43		1.45	1.40	
11.50						
12.00						
12.20						
12.50						
13.00			1.44		1.42	
13.30	1.43					
13.90						
14.00		1.45				
15.00	1.54			1.57	1.50	
15.50						
16.00						
17.00					1.58	

Table 3.3.6 Distribution of Specific Gravity(Gs)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5
1.00	2.66	2.65	2.65	2.67		2.64
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	2.65			2.68		2.65
3.50						
3.66						
4.00						
5.00	2.68	2.67	2.65	2.65		2.66
5.50						
6.00						
6.20						
7.00	2.65	2.66		2.65		2.65
7.50						
8.00						
8.10						
9.00	2.63		2.7			2.63
9.10		2.65				
9.50						
10.00						
10.10						
11.00	2.65	2.65	2.65			2.65
11.50						
12.00						
12.20						
12.50						
13.00	2.68					2.64
13.30						
13.90						
14.00		2.64				
15.00	2.67					2.68
15.50						
16.00						
17.00						2.69

Table 3.3.7 Distribution of Void Ratio(eo)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	2.315	2.335	2.574		1.805	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	2.339				2.552	
3.50						
3.66						
4.00						
5.00	2.371	2.596	2.431		2.462	
5.50						
6.00						
6.20						
7.00	2.478	2.445			2.275	
7.50						
8.00						
8.10						
9.00	2.678		2.72		2.502	
9.10		2.604				
9.50						
10.00						
10.10						
11.00	2.723	2.753			2.763	
11.50						
12.00						
12.20						
12.50						
13.00			2.539		2.722	
13.30						
13.90						
14.00		2.439				
15.00	1.963			2.202		
15.50						
16.00						
17.00				1.91		

Table 3.3.8 Distribution of Pre-consolidation Pressure('vm kgf/cm)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	0.25		0.16		0.19	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	0.28				0.21	
3.50						
3.66						
4.00						
5.00	0.4	0.35	0.3		0.35	
5.50						
6.00						
6.20						
7.00	0.45	0.4			0.46	
7.50						
8.00						
8.10						
9.00	0.7		0.6		0.56	
9.10		0.62				
9.50						
10.00						
10.10						
11.00	0.72	0.78			0.75	
11.50						
12.00						
12.20						
12.50						
13.00			0.85		0.85	
13.30						
13.90						
14.00		1.0				
15.00	1.4				1.1	
15.50						
16.00						
17.00					1.6	

Table 3.3.9 Distribution of Compression Index(Cc)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	0.24		0.21		0.105	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	0.24				0.07	
3.50						
3.66						
4.00						
5.00	0.31	0.36	0.28		0.26	
5.50						
6.00						
6.20						
7.00	0.3	0.26			0.31	
7.50						
8.00						
8.10						
9.00	0.41		0.34		0.31	
9.10		0.4				
9.50						
10.00						
10.10						
11.00	0.36	0.35			0.31	
11.50						
12.00						
12.20						
12.50						
13.00			0.35		0.37	
13.30						
13.90						
14.00		0.37				
15.00	0.32				0.325	
15.50						
16.00						
17.00					0.25	

Table 3.3.10 Distribution of Swelling Index(Cs)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	0.08		0.03		0.08	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	0.07				0.07	
3.50						
3.66						
4.00						
5.00	0.06	0.04	0.06		0.04	
5.50						
6.00						
6.20						
7.00	0.06	0.06			0.03	
7.50						
8.00						
8.10						
9.00	0.028		0.036		0.038	
9.10		0.055				
9.50						
10.00						
10.10						
11.00	0.04	0.042			0.04	
11.50						
12.00						
12.20						
12.50						
13.00			0.025		0.035	
13.30						
13.90						
14.00		0.045				
15.00	0.02				0.045	
15.50						
16.00						
17.00					0.06	

Table 3.3.11 Distribution of Compression Index(Cv 10 cm/sec)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	8.3		13.9		2.39	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	11.7				4.19	
3.50						
3.66						
4.00						
5.00	6.9	8.4	8.7		7.5	
5.50						
6.00						
6.20						
7.00	5.6	9.1			19.71	
7.50						
8.00						
8.10						
9.00	5.1		15.00		10.04	
9.10		11.2				
9.50						
10.00						
10.10						
11.00	5.3	7.5			7.34	
11.50						
12.00						
12.20						
12.50						
13.00			5.6		5.36	
13.30						
13.90						
14.00		6.4				
15.00	5.1				6.04	
15.50						
16.00						
17.00					9.26	

Table 3.3.12 Distribution of Coefficient of Volume Compressibility(vm cm/kgf)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	0.226		0.32		0.17	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	0.24				0.3	
3.50						
3.66						
4.00						
5.00	0.16	0.21	0.22		0.205	
5.50						
6.00						
6.20						
7.00	0.15	0.16			0.095	
7.50						
8.00						
8.10						
9.00	0.115		0.139		0.129	
9.10		0.133				
9.50						
10.00						
10.10						
11.00	0.121	0.118			0.095	
11.50						
12.00						
12.20						
12.50						
13.00			0.085		0.075	
13.30						
13.90						
14.00	0.045				0.065	
15.00						
15.50						
16.00						
17.00					0.03	

Table 3.3.13 Distribution of Coefficient of Permeability(k 10cm/sec)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	1.57		5.6		0.69	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	2.21				1.18	
3.50						
3.66						
4.00						
5.00	1.07	0.74	1.54		1.16	
5.50						
6.00						
6.20						
7.00	0.8	1.49			1.77	
7.50						
8.00						
8.10						
9.00	0.61		1.14		0.97	
9.10		0.99				
9.50						
10.00						
10.10						
11.00	0.67	0.85			0.68	
11.50						
12.00						
12.20						
12.50						
13.00			0.43		0.37	
13.30						
13.90						
14.00		0.51				
15.00	0.24				0.36	
15.50						
16.00						
17.00					0.36	

Table 3.3.14 Distribution of Critical Stress Ratio(M)

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	1.03		1.46		1.2	
1.50						
1.95						
2.00						
2.50						
2.80						
3.00	0.81				1.16	
3.50						
3.66						
4.00						
5.00	0.81		1.29		0.9	
5.50						
6.00						
6.20						
7.00	0.81				1.07	
7.50						
8.00						
8.10						
9.00	1.2		0.98		0.81	
9.10						
9.50						
10.00						
10.10						
11.00	0.98					
11.50						
12.00						
12.20						
12.50						
13.00			0.86			
13.30	1.2					
13.90						
14.00						
15.00	0.77					
15.50						
16.00						
17.00						

Table 3.3.15 Distribution of Undrained Shear Strength by Field Vane Tests

深度(m)	NO.1	NO.2	NO.3	NO.4	NO.5	NO.5'
1.00	1.63	0.92	0.96			0.54
1.50					0.68	
1.95		1.79				
2.00			0.88			0.69
2.50						
2.80		0.96				
3.00	0.96		1.06			0.77
3.50					0.96	
3.66						
4.00		1.00	1.16			0.85
5.00	1.24	1.28	1.16			0.79
5.50					1.34	
6.00		1.28	1.41			0.99
6.20						
7.00	1.67	1.30	1.71			1.13
7.50					1.93	
8.00			1.83			1.19
8.10		1.65				
9.00	1.95		2.38			1.33
9.10		1.67				
9.50					2.22	
10.00		2.08	2.58			1.49
10.10						
11.00	2.38	2.5	2.74			1.80
11.50					2.58	
12.00			2.74			2.12
12.20		2.68				
12.50					3.45	
13.00		3.05	3.35			2.32
13.30	2.99					
13.90			4.25			
14.00		3.45				2.71
15.00	4.65	3.75	4.45			2.83
15.50					4.45	
16.00						
17.00						

CHAPTER 4 DESIGN FOR TESTING CANAL FACILITY

4-1 Determination of Slope Gradients of the Testing Canal Facility (Preliminary Analysis)

As described in the CHAPTER 1, BASIC PLAN FOR THE MODEL INFRASTRUCTURE PROJECT, the proposed testing canal facility consists of the following four (4) types of construction structures:

a) Non-treatment slope structure for short-term slope stability study

b) Non-treatment slope structure for long-term slope stability study

c) Two (2) improved slope structures

Two (2) improved slope structures are selected from three already proposed methods (sand compaction pile treatment, soil cement treatment and gravel compaction pile treatment) based on the results of the preliminary design analysis.

Fig. 4.1.1 shows the outline of the testing canal facility.

1) Conditions and Procedure of Preliminary Analysis

The preliminary analysis for the testing canal facility was performed by the circular slip method using total stress analysis in the computer system of IEC.

The preliminary analysis conditions and procedure were as follows:

(1) For the non-treatment slopes, the depths of the testing canal facility shall be 4 m and 3 m. The preliminary analysis shall be performed for three (3) kinds of slope gradients, that is, 1:3, 1:5 and 1:7, on each canal

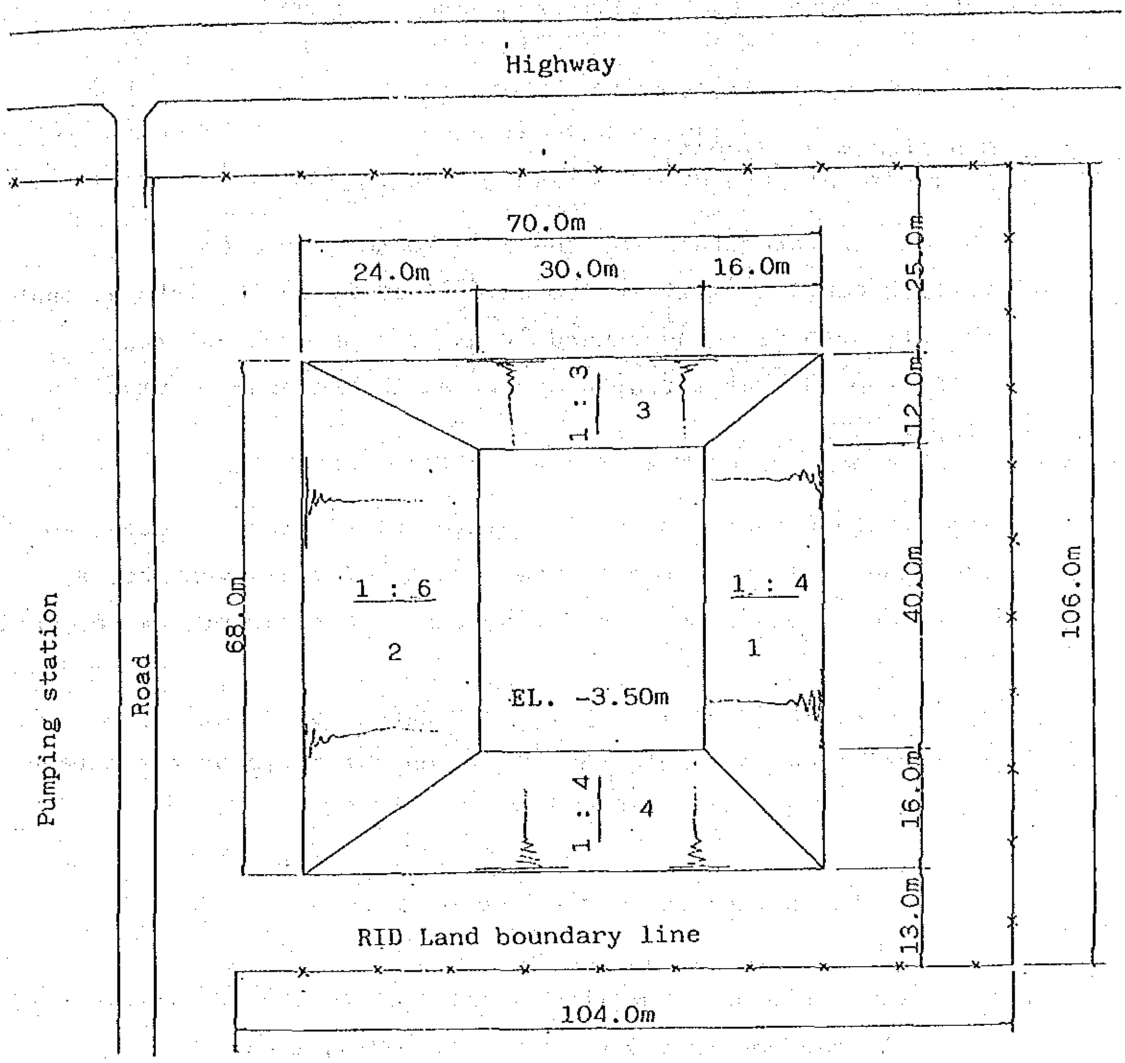
depth considering the strength decrease of the foundation caused by excavation work.

(2) For the non-treatment slopes, the preliminary analysis shall be performed for five (5) kinds of slope gradients, that is, 1:3, 1:4, 1:5, 1:6 and 1:7, on each testing canal depth (4 m and 3 m) considering the strength decrease of foundation caused by excavation work and the anisotropy of the strength occurring in the non-treatment slopes.

(3) Based on the results obtained from the above specified analysis (2), the non-treatment slope gradient applied for the short-term slope stability study shall be selected from the case indicating a safety factor value of a little less than 1.0. Also, the non-treatment slope gradient applied for the long-term slope stability study shall be selected from the case indicating a safety factor value of a little greater than 1.0.

(4) For the improved slope, improvement method and slope gradient of slope for the National Road Route No. 3 shall be decided in order to keep certain slope stability against damage and failure of the road. The testing canal depth shall be the same condition as determined in the above (3).

(5) For the improved slope, improvement method and slope gradient for another improved slope shall be selected in order to study effectiveness of the improvement method.



Section	Treatment	Slope gradient
1	Non-treatment for short term stability	1 : 4
2	Non-treatment for long term stability	1 : 6
3	Treatment by soil cement columns	1 : 3
4	Treatment by sand or gravel compaction piles	1 : 4

Fig.4.1.1 Outline of Testing Canal Facility

2) Determination Method of Design Parameters

i) Non-Treatment Slopes

The design parameters for the non-treatment slopes in the testing canal facilities are determined by taking into account the strength decrease caused by excavation work and the anisotropy of the strength occurring in the non-treatment slopes.

The design parameters are calculated by the results obtained from the field vane tests. The results of field vane tests depend on the strain rate during the vane shear process of soil.

Therefore, it is necessary to correct the shear strength obtained from the field vane tests on the basis of the strain rate during the vane shear process of soil.

a) Strength Decrease due to Excavation Work

The ratio of strength decrease due to the excavation work ,A, can be expressed by the following equation on the basis of the Hvorslev failure criteria.

$$A = \frac{Sun}{Su} = \eta^{-\lambda} \cdot \frac{\kappa + (\sigma fn/Pc)\ \tan\ \phi e}{\kappa + (\sigma fn/Pc)\cdot \tan\ \phi e} \quad \cdots\cdots\cdots\cdots \quad (4.1.1)$$

where,

Sun : Undrained Strength under the condition of over consolidation

Su : Undrained Strength under the condition of normal consolidation

η : Overconsolidation ratio

κ : Coefficient of cohesion

λ : Ratio of compression index under the normal consolidation Process, C_c to expansion index under the rebounding process. C_s (λ = Cs/Cc)

P_c : Maximum pressure for normal consolidation (The pressure P_c is equal to the effective in situ overburden pressure for normally consolidated clays)

σ_{fn} : Normal stress occurring in preconsolidated clay

σ_t : Normal stress occurring in normally consolidated clay.

ϕe : Effective internal friction angle

The consolidation process used in the above equation is presented in Fig. 4.1.2 .

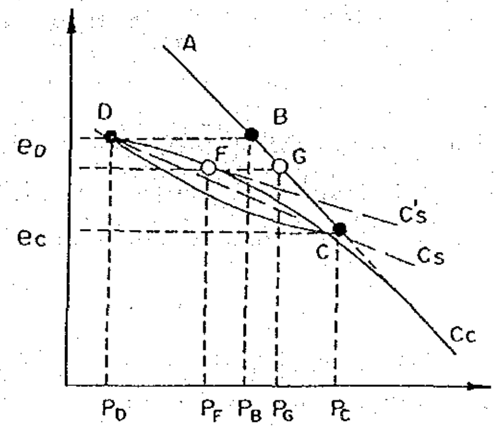


Fig.4.1.2 e-logP Curve

In the case where the value of overconsolidation ratio, n, is less than 10.0, it is said that the relation between overconsolidation ratio and ratio of strength decrease indicates linear under the logarithnic plots shown in Fig. 4.1.3.

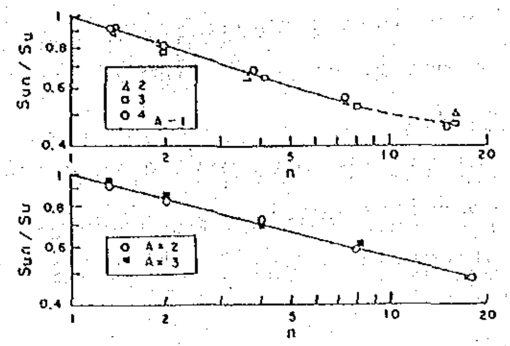


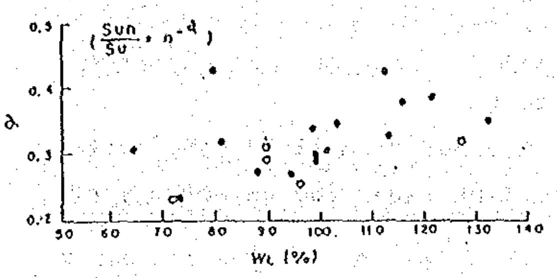
Fig.4.1.3 Relation between Overconsolidation Ratio and Ratio of Strength Decrease

From the above experimental relation, instead of Equation, the following equation can be applied approximately for the determination of design value of the strength decrease due to the excavation work of the testing canal facility.

$$A = \frac{Sun}{Su} = \eta - \lambda$$

where, α : Constant parameter of strength decrease

Fig. 4.1.4a and 4.1.4b present the values of constant parameter of strength decrease obtained from Ariake clay in Japan.



Undisterbed Ariake Clay : ●

Disterbed Ariake CLay : o

Fig.4.1.4a α -W Relation

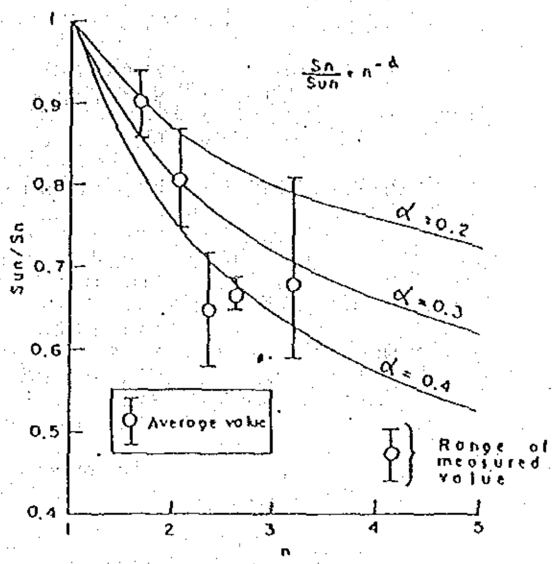


Fig.4.1.4b

Fig.4.1.4 Value of Constant Parameter of Strength Dcrease Obtained from Ariake Clay

It is found that the constant parameter of strength decrease has a range of about 0.2 ∿ 0.4 from Fig. 4.1.4a and 4.1.4b.

In the preliminary design stage, 0.3 (the average value of range of constant parameter obtained from Ariake clay) is adopted as the constant parameter of Strength decrease due to the excavation work of the testing canal facility.

b) Anisotropy of Strength and Corrected Coefficient of Strain Rate

The anisotropy of strength occurring in soil and corrected coefficient of strain rate during vane shear are estimated by Bjerrum's method using the plasticity index obtained from the physical properties tests.

Bjerrum proposed the following equation to obtain the design shear strength of soil from the results of the field vane tests in 1973.

$$
\begin{aligned}
Su^* &= Su \cdot \mu_A \cdot \mu_R \\
&= Su \cdot \mu \quad \text{.............................} \quad (4.1.2)
\end{aligned}
$$

where, Su^* : Design shear strength

Su : Shear strength obtained from field vane tests

μ_A : Corrected coefficient regarding anisotropy of strength

μR : Corrected coefficient regarding Strain rate of shearing process

μ : Bjerrum's coefficient ($\mu = \mu A.\mu R$)

The relationship between the Bjerrum's coefficient, μ, and the plasticity index is presented in Fig. 4.1.5.

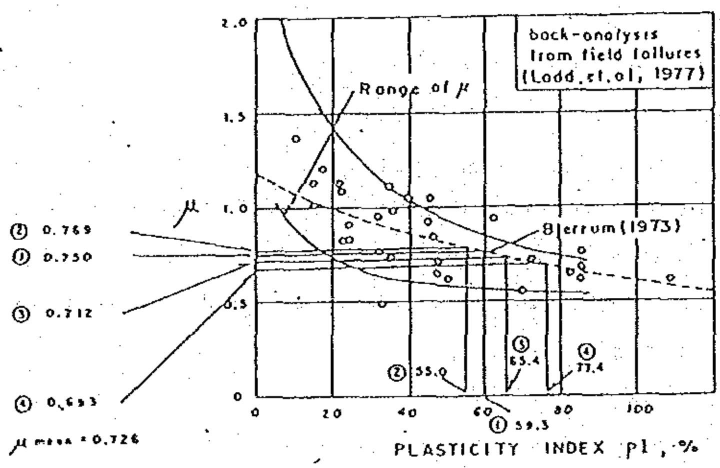


Fig.4.1.5 Relationship between Bjerrum's Coefficient and Plasticity

ii) Improved Slopes

The design parameters applied for the stability analysis of the improved slopes in the testing canal facilities are determined from the assumption that the improved slopes consist of composite ground of the original clay and treatment material.

The shear strength for the original clay including the treatment slopes are also taken into account along with the strength decrease caused by excavation work, the anisotropy of the strength, and the strain rate of the shear process under the field vane tests.

The shear strength for the treatment materials are determined by not taking into account the anisotropy of the strength.

a) Design Unit Weight

The design unit weight of the treatment slopes improved by sand or gravel compaction piles or soil cement columns are determined from the volume ratio of sand (or gravel) compaction piles or soil cement columns to the original clay material calculated by the following equation.

$$\gamma t = \gamma t_1 \, Ass + \gamma t_2 (1 - Ass) \quad \text{..................} \quad (4.1.3)$$

where, γt : Design unit weight of the treatment slopes improved by sand or gravel compaction piles or soil cement columns (composite ground)

γt_1 : Design unit weight of sand or gravel compaction piles or soil cement columns (treatment material)

γt_2 : Design unit weight of original clay material

Ass : Volume ratio of sand (or gravel) compaction piles or soil cement columns (treatment material) to original clay material

b) Design Shear Strength

The design shear strength of the improved slopes (composite ground) are determined from the volume ratio of sand (or gravel) compaction piles or soil cement columns (treatment material) to the original clay material.

① Design Parameters for the Improved Slope Structures by Sand and Gravel Compaction Piles

Since it is considered that the effect of disturbance of original clay foundation caused by the construction of sand or gravel compaction piles can be canceled against the strength increase of the original clay foundation due to the drainage effect by the sand or gravel compaction piles, the strength decrease of the original clay foundation due to the disturbance by the construction of the sand or gravel compaction piles is not taken into account in the design shear strengths of the improved slopes.

The strength increase of the original clay foundation caused by the drainage effect is taken into account in the design shear strengths of the improved slopes.

The design shear strengths of the improved slopes improved by sand or gravel compaction piles, τ is calculated by the following equations.

$$\tau = (1 - Ass)\cdot(Su + \sigma c \Delta c/\Delta p \cdot U) + Ass\ \sigma n' \cdot \tan\phi \quad \ldots. \quad (4.1.4)$$

where, τ : Design shear strength of treatment slopes

improved by sand or gravel compaction piles

Su ; Design shear strength of original clay foundation

ϕ ; Internal friction angle of sand or gravel compaction piles

$\sigma n'$: Effective normal stress caused by overburden load

Λss: Volume ratio of sand (or gravel) compaction piles to original clay material

σc : Confined pressure

$\Delta c/\Delta p$: Ratio of Strength increase

U : Mean degree of consolidation

② Design Parameters for the Improved Slope Structure by Soil Cement Columns

The design shear strength is determined from the volume ratio of the soil cement column to the original clay material.

It is naccessary to consider the safety factor for the effect of the disturbance of the original work of the soil cement columns in the design shear strength of the treatment slope improved by the soil cement column.

The design shear strength, τ is computed by the following equation.

$$\tau = \frac{1}{\eta b} (Cp \cdot \Lambda ss + (1 - \Lambda ss) \cdot Su) \quad \ldots\ldots\ldots\ldots \quad (4.1.5)$$

where,

τ : Design shear strength of treatment slope improved by Soil Cement Column

C_p : Unconfined strength of Soil Cement Column

Su : Design shear strength of original clay foundation

Λss: Volume ratio of Soil Cement Columns to original clay material

ηb : Safety Factor for the Effect of Disturbance of original Clay foundation due to Soil Cement Columns

3). Calculation of Design Parameters

i) Non-Treatment Slopes

a) Design Unit Weight

The design values of the unit weight in the non-treatment slopes are calculated by using the unit weight obtained from the samples taken by boring hole No. 1.

The layer components of deposit model and the design values of the unit weight in the non-treatment slopes are shown in Fig. 4.1.6.

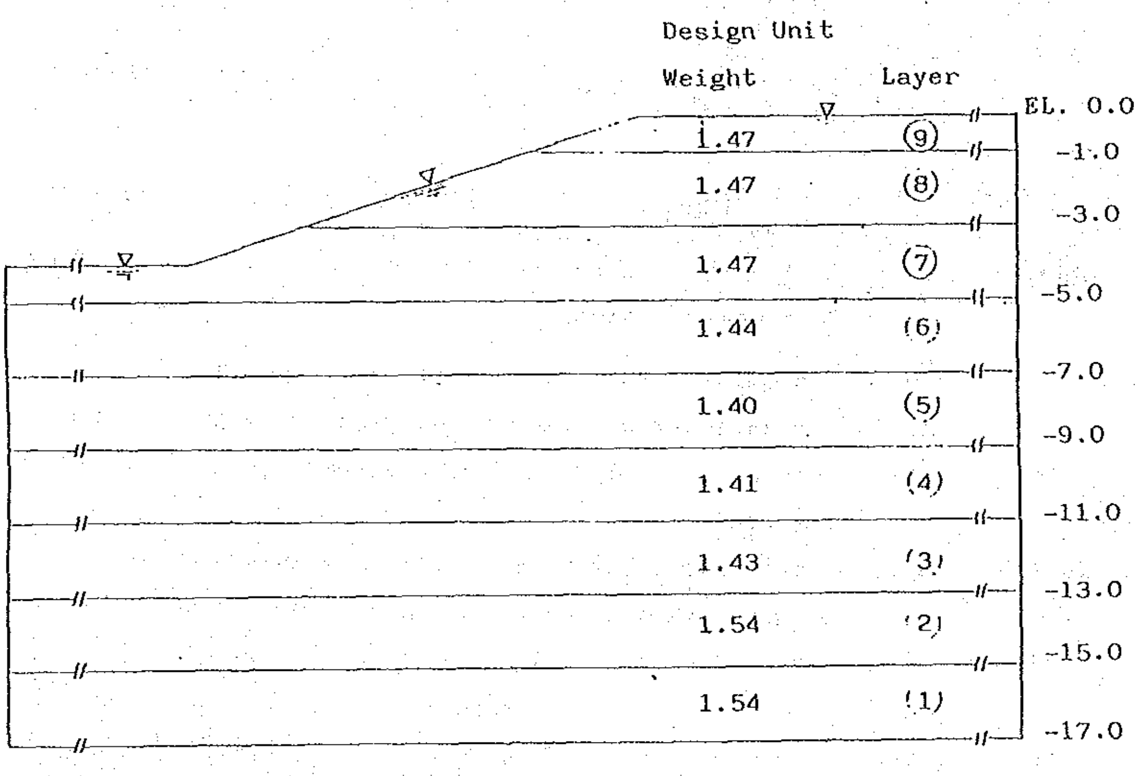


Fig.4.1.6 Deposit Layer Components and Design Unit Weight

b) Design Shear Strength

The design values of the shear strength are estimated based on the undrained shear strength obtained from the field vane tests by taking into account the calculated values of the Bjerrum's coefficient and the strength decrease caused by excavation work.

The design values of shear strength in the non-treatment slopes are caluculated by using the undrained shear strength obtained from the field vane tests performed at the point neighbouring boring hole No. 1.

① Ratio of Strength Decrease caused by Excavation Work

The values of ratio of strength decrease are calculated by using the equation of the basis of the results obtained from the consolidation tests for the samples taken from boring hole No. 1. The strength decrease zone is assumed as shown in Fig. 4.1.7. The values of ratio of strength decrease are presented in Table 4.1.1.

② Bjerrum's Coefficient and Ratio of Strength Decrease

The values of Bjerrum's coefficient are computed by using the plasticity index obtained from the samples taken from boring hole No. 1 on the basis of Bjerrum's curve shown in Fig 4.1.9.

The values of ratio of Strength decrease calculated by using Bjerrum's curve are shown in Table-4.1.2.

Also, Table-4.1.1 shows the results obtained from the field vane tests, the values of ratio of strength decrease and the design values of the shear strength on each deposit layer component model.

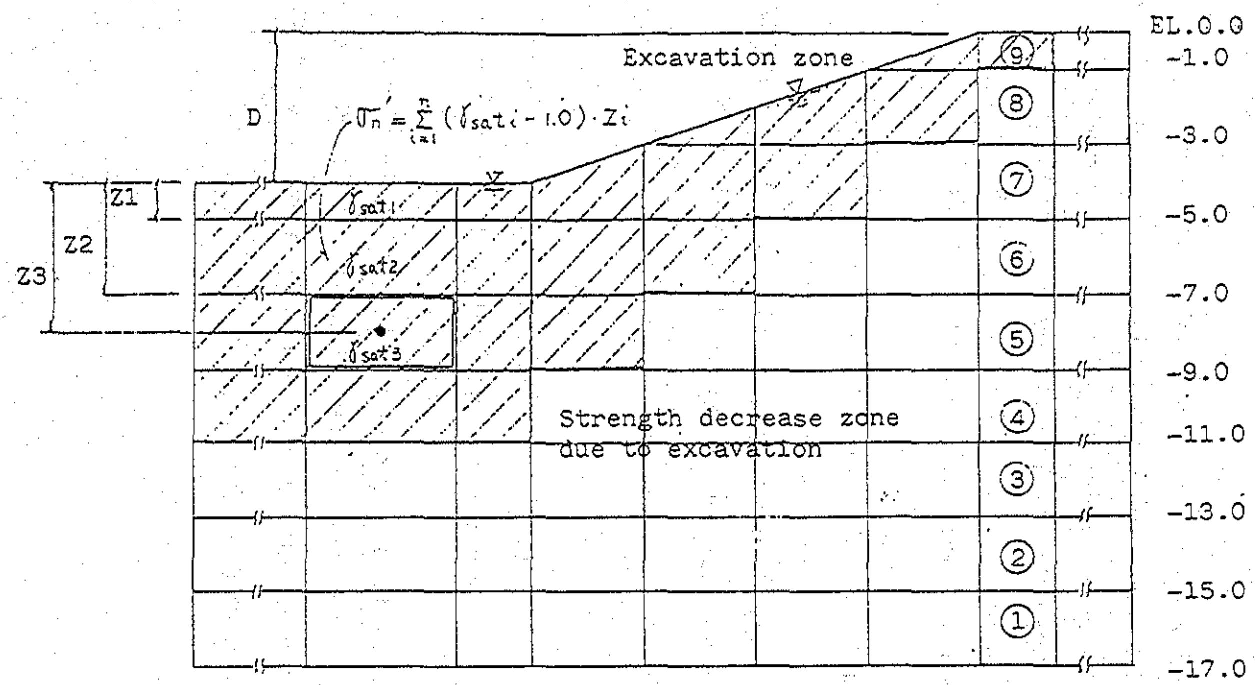


Fig.4.1.7 Strength Decrease Zone for Non-treatment Slopes

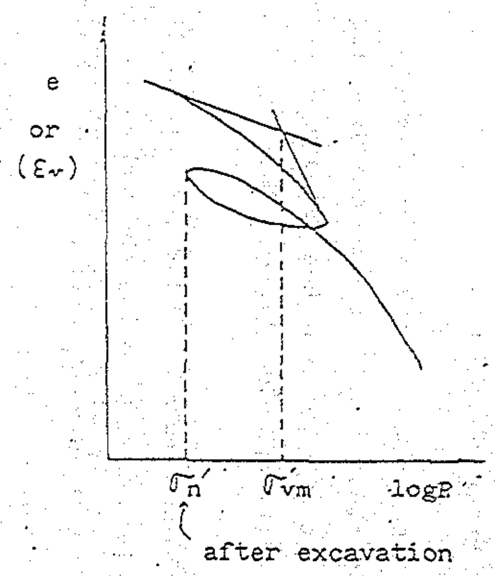


Fig.4.1.8 e-logP Curve

Table 4.1.1 Ratio of Strength Decrease for Non-treatment Slopes

EL	Layer	σ'_{vm}	σ'_n	OCR	γ_t	α	μ_A	μ_B	Su	Su*	Remarks
-1.0	9	2.5	–	–	1.47	–	–	–	–	–	
-3.0	8	2.5	–	–	1.47	–	–	–	–	–	
-5.0	7	2.8	0.235	11.915	1.47	0.3	0.750	0.476	0.960	0.342	
-7.0	6	4.0	0.940	4.255	1.47	0.3	0.769	0.648	1.240	0.618	
-9.0	5	4.5	1.880	2.394	1.44	0.3	0.769	0.769	1.670	0.988	
-11.0	4	7.0	2.820	2.482	1.40	0.3	0.712	0.761	1.950	1.057	
-13.0	3	7.2	–	–	1.41	–	0.712	–	2.380	1.695	
-15.0	2	9.0	–	–	1.43	–	0.673	–	2.990	2.412	
	1	14.0	–	–	1.54	–	0.673	–	4.650	3.130	

Remarks :
- σ'_{vm} : Preconsolidation pressure (tf/m^2)
- σ'_n : Effective normal stress after excavation work (tf/m^2)
- OCR : Overconsolidation ratio after excavation
- α : Constant parameter of strength decrease, $Sun/Sn = OCR^{-\alpha}$
- μ_A : Bjerrum's correction factor
- μ_B : Correction factor ($= OCR^{-\alpha}$)
- Su : Shear strength from F.V. test data (tf/m^2)
- Su* : Shear strength for design (tf/m^2), $Su^* = \mu_A . \mu_B . Su$

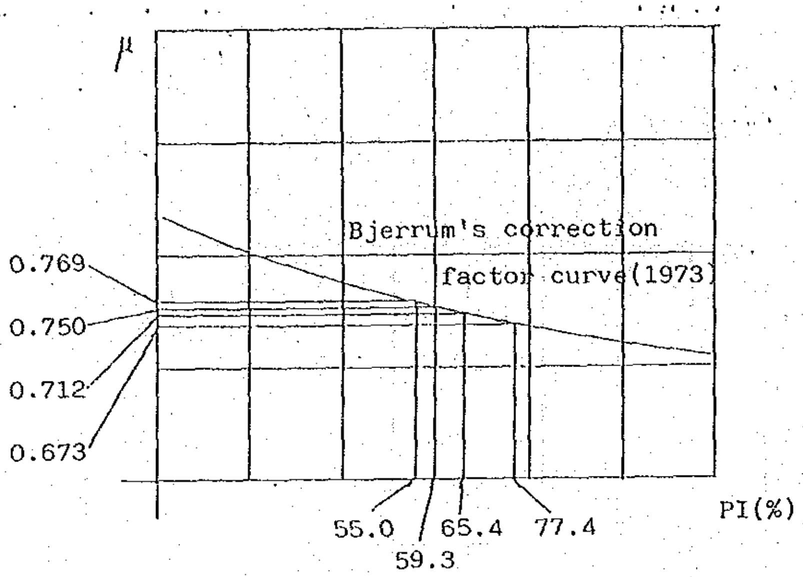


Fig.4.1.9 Bjerrum's Correction Factor Curve

Table 4.1.2 Strength Decrease Ratio Obtained from Bjerrum's Correction Curve

EL	Layer	PI(%)	μ	Remarks
-1.0	9	59.3	0.750	
-3.0	8	59.3	0.750	
-5.0	7	59.3	0.750	
-7.0	6	55.0	0.769	
-9.0	5	55.0	0.769	
-11.0	4	65.4	0.712	
-13.0	3	65.4	0.712	
-15.0	2	77.4	0.750	
17.0	1	77.4	0.750	

Remarks, PI : Plasticity Index (%)

μ : Bjerrum's correction factor

ii) Treatment Slopes

a). Design Unit Weight

① Treatment slope by Sand Compaction Piles

The design unit weight of sand compaction pile is obtained by assuming that the relative density of sand is set at 50% after the compaction.

The relationship between a relative density and a void ratio is presented by the following equation:

$$Dr = \frac{e_{max} - e}{e_{max} - e_{min}} \quad \text{...............} \quad (4.1.6)$$

where,
- Dr : Relative density
- e_{max} : Maximum void ratio
- e_{min} : Minimum void ratio
- e : Void ratio

In case $e_{max} = 1.10$, $e_{min} = 0.35$,

$$e = (1-Dr) \cdot e_{max} + Dr \cdot e_{min}$$
$$= (1-0.50) \times 1.10 + 0.50 \times 0.35$$
$$= 0.725$$

The design unit weight, $\gamma t1$ is computed by the following equation:

$$\gamma t_1 = \frac{Gs \cdot (1 + W/100)}{1 + e} \cdot \gamma w \quad \text{............} \quad (4.1.7)$$

where,
- γt_1 : Design unit weight of sand compaction pile
- Gs : Specific gravity
- W : Moisture content
- γ_w : Unit weight of water

In case $e=0.725$ under the assumptions, $Gs=2.58$ and $W=7\%$, the following value for the design unit weight is obtained:

$$\gamma t_1 = 1.60\ t/m^3$$

The design unit weight of the improved zone (composite ground) in the treatment slope structure by the sand compaction pile method is calculated by the use of the equation (4.1.3).

(2) Treatment Slope by Gravel Compaction Piles

The design unit weight of gravel compaction pile is calculated by the use of the equations (4.1.6) and (4.1.7) as the same manner with the sand compaction pile method.

The assumpted values for each parameter can be presented as follows:

$$Dr = 0.50$$
$$e_{max} = 1.20$$
$$e_{min} = 0.35$$
$$Gs = 2.70$$
$$W = 5.0\%$$

From the above assumptions and the equations (4.1.6) and (4.1.7),

$$e = 0.775$$
$$qt_1 = 1.597\ t/m^3$$

The design unit weight of the improved zone (composite ground) in the treatment slope structure by the gravel compaction pile method is calculated by the use of the equation (4.1.3) as well.

③ Treatment Slope by Soil Cement Column

The design unit weight of soil cement column is determined by the volume ratio of soil cement column to original clay material. As mentioned below, the cement volume which is required to conform to the specified design strength was presented as 150 kg/m^3. Therefore, the design unit weight of soil cement column is obtained from the following equation taking not into account the water weight of cement slurry:

$$
\begin{aligned}
\gamma t_1 &= \text{amount of Mixed Cement} \\
&\quad + \text{Unit Weight of Clay} \\
&= 0.150 + 1.450 \\
&= 1.600 \text{ t/m}^3
\end{aligned}
$$

The design unit weight of the improved zone (composite ground) in the treatment slope structure by the soil cement column method is calculated by the use of the equation (4.1.3) as well.

b) Design Shear Strength

① Treatment Slope by Sand Compaction Pile

From the experience of sand compaction piles constructed in Japan, the blow counts of standard penetration test, N, obtained from the sand compaction pile is assumed to be 4 blow to the ground surface.

On the other hand, Gibbs and Holtz proposed the relationship between relative density of sand (Dr) and blow count, N, in 1957 a presented in Fig 4.1.10.

The internal friction angle of the sand compaction pile is determined from assumption where the relative density of sand after compaction can reach about 50% by using Gibbs and Holtz relationship between relative density Dr and blow count N.

Under the assumptions mentioned above, therefore, a value of 30 degrees can be determined as the internal friction angle of a sand compaction pile by applying the relationship between the relative density and internal friction angle of sand shown in Table 4.1.3 which was proposed by Meyerhof (1956).

The value of the confined factor* to original clay material by sand compaction pile is assumed to be a 20 percent increase in original strength of the clay material on the basis of past experience.

The design values of shear strength in the original clay material existing in the treatment slope are calculated by using the undrained shear strength obtained from the field vane tests performed at the point neighbouring with boring hole No. 1 taking into account the calculated values of the Bjerrum's coefficient (Corrected coefficient regarding Anisotropy of strength and Strain rate of shearing process) and the strength decrease caused by excavation work.

The volume ratio of sand material to the original clay material in the improved zone is set at 10 percent from the results of the sand compaction piles constructued in Japan and the economic viewpoint of the construction cost for the treatment slope structures by sand compaction piles..

---

*Remark:

Confined factor;

$$\tau = (1 - Ass) \cdot (Su^* + \overbrace{\sigma_c \cdot \frac{\Delta C}{\Delta P} U}^{\text{Confined factor}}) + Ass \cdot \sigma n' \cdot \tan \phi$$

Ass : Improved volume ratio

τ : Shear Strength

σ_n' : Effective normal stress

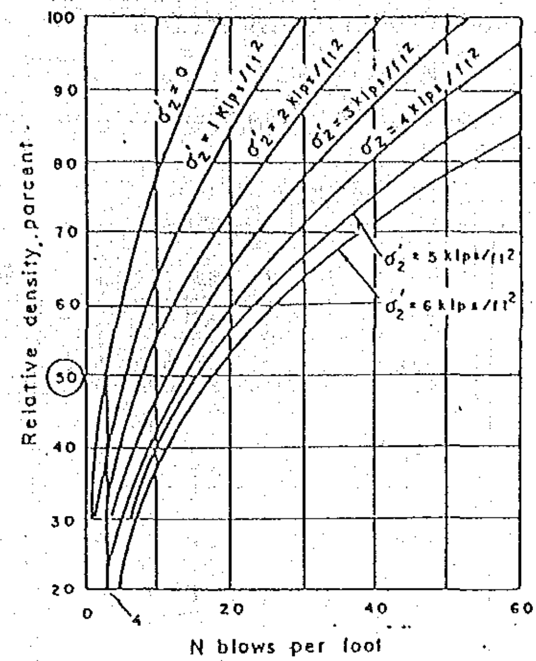


Fig.4.1.10 Relation between Relative density(Dr) and Blow Count(N) Proposed by Gibbs and Holtz (1953)

Table 4.1.3 N-Dr-σ Relationship Proposed by Meyerhof (1956)

Sand Condition	Relative Density ·Dr	N-Value	Internal friction angle (ø')
very loose	< 0.2	< 4	< 30
loose	0.2~0.4	4 ~ 10	30 ~ 35
compact	0.4~0.6	10 ~ 30	35 ~ 40
dense	0.6~0.8	30 ~ 50	40 ~ 45
very dense	>0.8	> 50	>45

The design values of the shear strength in the improved zone (composite ground) by sand compaction piles are shown in Table 4.1.4 & Fig 4.1.11.

Table 4.1.4 Design Parameters of Improved Zone by Sand Compaction Piles

The mobilized shear strength in the improved zone are calculated by the following equation ;

$$\tau = (1-Ass).(Su^{*}+\sigma_c.\frac{\Delta C}{\Delta P}.U)+Ass.\sigma_n'.\tan\phi$$

EL	Layer	μA	μB	Su	Su*	Ass	ϕ	$\sigma_c \cdot \frac{\Delta C}{\Delta P} \cdot U$
-1.0	–	–	–	–	–	–	–	–
-3.0	–	–	–	–	–	–	–	–
-5.0	7	0.750	0.476	0.960	0.342	0.1	30	0.2
-7.0	6	0.769	0.648	1.240	0.618	0.1	30	0.2
-9.0	5	0.769	0.769	1.620	0.988	0.1	30	0.2
-11.0	4	0.712	0.761	1.950	1.057	0.1	30	0.2
-13.0	3	0.712	1.000	2.380	1.695	0.1	30	0.2
-15.0	2	0.673	1.000	2.990	2.012	0.1	30	0.2
	1	0.673	1.000	4.650	3.130	0.1	30	0.2

Original clay (μA – Su*)   Sand compaction piles (ϕ)

Improved zone (EL -3.0 to -9.0)

Strength decrease zone (EL -3.0 to -11.0)

Remarks,

μA : Bjerrum's correction factor

μB : ($=OCR^{-\alpha}$, $\alpha=0.3$) Coefficient of strength decrease

Su : Shear strength from F.V. test data

Su* : ($=\mu A \cdot \mu B \cdot Su$) Shear strength for design

Ass : Improved volume ratio

ϕ : Internal friction angle of sand.

$\sigma_c \cdot \frac{\Delta C}{\Delta P} \cdot U$: Coefficient of strength decrease in clay by drainage

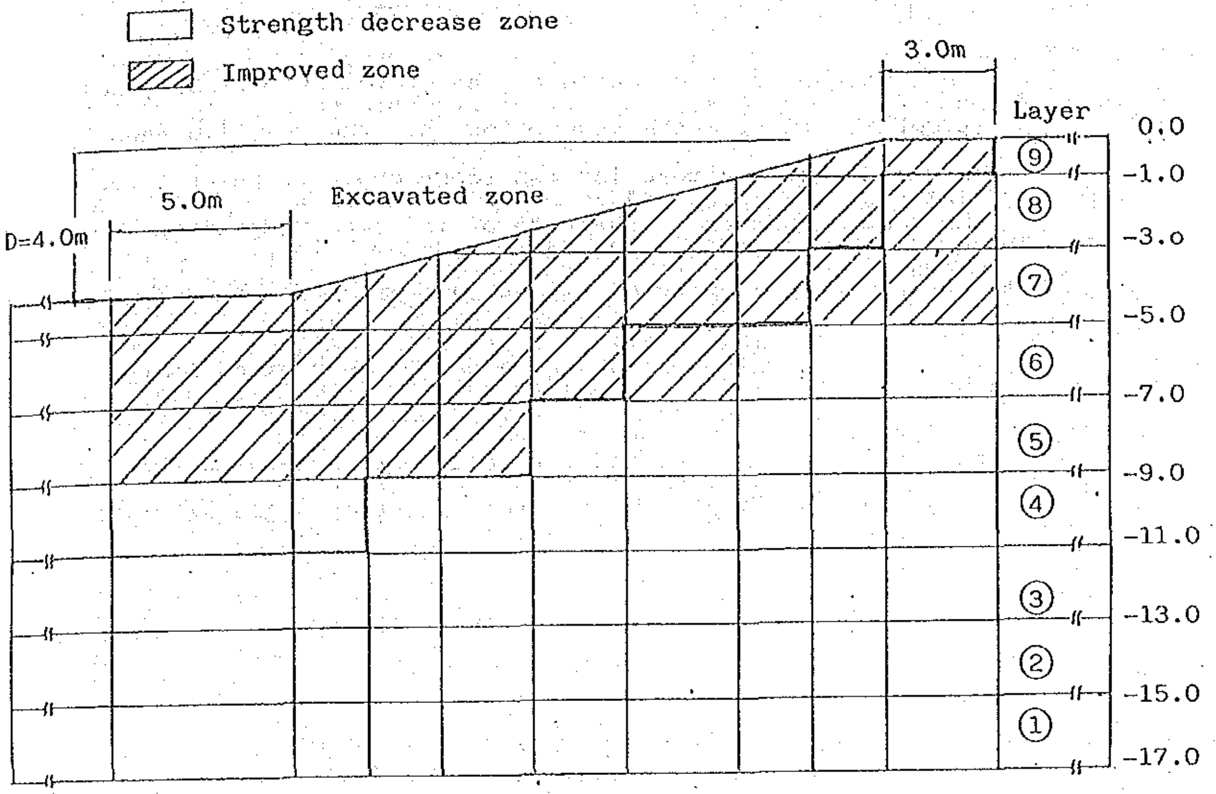


Fig.4.1.11 Strength Decrease Zone and Improved Zone by Sand Compaction Piles

② Treatment Slope by Gravel Compaction Piles

The internal friction angle of the gravel compaction piles is determined from the assumption that the relative density of gravel after compaction can reach about 50 percent for the same reason as in the case of the sand compaction piles.

Therefore, a value of 33 degree can be determined as the internal friction angle of gravel compaction piles by using the Table 4.1.5 presented the relationship between the relative density and the internal friction angle of gravel material.

Table 4.1.5 Relative Density and Internal Friction Angle of Gravel Material

Grain Size and Grain Size Distribution	Relative Density (Dr)		
	> [illegible]0 %	70 ~ 50 %	< 50 %
	Dense	Medium	Loose
Coarse Sand and Fine Sand are Distributed Uniformly	34 ~ 38°	32 ~ 34°	28 ~ 30°
Well Distributed Coarse Sand and Poor Distributed Sand and Gravel are Mixed	37 ~ 45°	33 ~ 36°	30 ~ 33°
Well Distributed Sand and Well Distributed Gravel are Mixed	40 ~ 45°	36 ~ 41°	33 ~ 36°

Source : ' NEW FILLDAM ENGINEERING '

Edited by The Electric Power Civil Engineering Association in 1981

The value of the confined factor to original clay material by gravel compaction piles is assumed to be 20 percent increase in the original strength of the clay material for the same reason as the case of the sand compaction piles.

The design values of the shear strength in the original clay material existing in the improved slope are calculated by using the undrained shear strength obtained from the field vane tests by taking into account the calculated values of the Bjerrum's coefficient and the strength decrease by the same determination method of non-treatment slopes. The volume ratio of gravel material to the original clay material in the improved zone is set at 10 percent from the same reason as the case of the sand compaction piles.

The design values of the shear strength in the improved zone (composite ground) by gravel compaction piles are shown in Table 4.1.6 and Fig. 4.1.12.

Table 4.1.6 Design Parameters and Mobilized Shear Strength in the Improved Zone by Gravel Compaction Piles

ɔmpaction Piles

The mobilized shear strength in the improved zone are calculated by the following equation ;

$$\tau = (1-Ass)\cdot(Su^{*}+\sigma_c\cdot\frac{\Delta C}{\Delta P}\cdot U)+Ass\cdot\sigma_n'\cdot\tan\phi$$

EL	Layer	μA	μB	Su	Su*	Ass	ϕ	$\sigma_c\cdot\frac{\Delta C}{\Delta P}\cdot U$
	–	–	–	–	–	–	–	–
-1.0	–	–	–	–	–	–	–	–
-3.0 -5.0	7	0.750	0.476	0.960	0.342	0.1	33	0.2
-7.0	6	0.769	0.648	1.240	0.618	0.1	33	0.2
	5	0.769	0.769	1.620	0.988	0.1	33	0.2
-9.0	4	0.712	0.761	1.950	1.057	0.1	33	0.2
-11.0 -13.0	3	0.712	1.000	2.380	1.695	0.1	33	0.2
	2	0.673	1.000	2.990	2.012	0.1	33	0.2
-15.0	1	0.673	1.000	4.650	3.130	0.1	33	0.2

Original clay (μA – Su*) Sand compaction piles (ϕ)

Improved zone (layers 5–7); Strength decrease zone (layers 4–7)

Remarks,

- μA : Bjerrum's correction factor
- μB : ($=OCR^{-\alpha}$, $\alpha=0.3$) Coefficient of strength decrease
- Su : Shear strength from F.V. test data
- Su* : ($=\mu A\cdot\mu B\cdot Su$) Shear strength for design
- Ass : Improved volume ratio
- ϕ : Internal friction angle of sand
- $\sigma_c\cdot\frac{\Delta C}{\Delta P}\,U$: Coefficient of strength decrease in clay by drainage

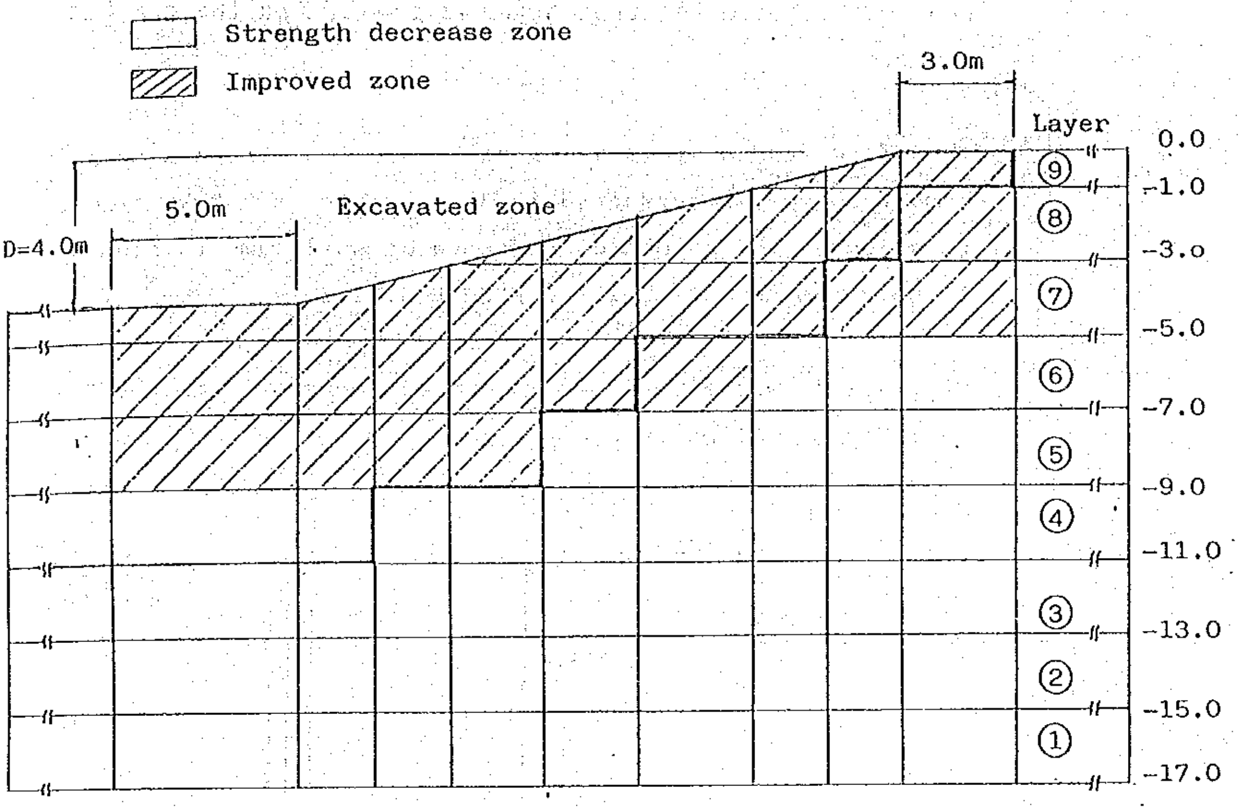


Fig.4.1.12 Strength Decrease Zone and Improved Zone by Gravel Compaction Piles

(3) Treatment Slope by Soil Cement Columns

From the experience of the soil cement columns constructed in Japan, the values of design strength of the soil cement column range from 1.0 to 4.0 kg f/cm^2 as shown in FIGURE-4.1.13. The volume ratio of the soil cement column to original clay material can be decreased by taking the higher design strength value into consideration. However, it is not desirable to take the design strength of the soil cement column as too high a value because the original clay