4. GEOLOGICAL SURVEY AT PERJAYA HEADWORKS SITE

4.1 General Feature

Another alternative site of the headworks is found at Perjaya village; 3 km downstream of the Martapura highway bridge. The river bed is covered with a large quantity of quartz sand and pumice which have been derived from severe erosion of the rhyolite tuff and volcanic ashes. The left bank of the river is formed with flat flood plain covered with alluvial deposits. The right bank of the rivers is formed with hilly land, of which height is about 86 m in altitude. The slope of the right hill is steep.

4.2 Drilling Investigation

Geological investigation by test drilling was carried out at four boreholes selected along the proposed weir axis; two holes on the left bank, one hole on the right bank and one in the river bed, of which the former three have been completed so far. Total proposed drilling depth would be 100 m (See Fig. III-7).

In addition, the standard penetration test and field permeability test were carried out at each bore hole (See Fig. III-8).

4.3 Geological Conditions of Headworks Site

According to the geological survey by means of test drilling at the Perjaya site, the foundation of the Perjaya headworks site consists of Pliocene-Pleiostocene formations, an alternating layers of sandstone and claystone (See Fig. III-9).

Cl layer

This layer is composed of sandy clay exposed on the top of the both banks. It is slightly weathered and rather loose in consolidation, having standard penetration values of 7 - 11. It is rather permeable, having rather high value; $K = 5 \times 10^{-3} - 3 \times 10^{-3}$ cm/sec, in the constant head tests.

S1 layer

This layer is composed of quartz sand and gravel, loosely with some silty clay. S1 layer can be observed on the core of the drilling hole BH 2-1 and BH 2-2. The boring results indicate that the permeability coefficient (K) of this layer is about 2 x 10^{-3}- 1.5 x 10^{-4} cm/sec, and standard penetration test value shows 20 - 30 at the BH 2-1 and 50 at the BH 2-2, representing weak or medium consolidation.

S1' layer

This layer compared to S1 layer. It has gray color, very dense and fine to medium sand with clay. Standard penetration tests show N-value of 50.

C2 layer

This layer consists of clay to sandy clay. C2 layer is absent at BH 2-2. It is bluish gray colored in fresh conditions and medium consolidated. The permeability coefficients obtained by constant head methods are very small; about 7.6 x 10^{-6} at the BH 2-4 and 2 x 10^{-3} - 2.7 x 10^{-4} at BH 2-1.

S2 layer

This layer is composed of alternating layers of sandstone and claystone. Its fresh faces are very dense compact having standard penetration value of over 50. This layer is made up largely of sandstone and contains tuff breccia, claystone and some carbonized organic matters.

Permeability coefficients obtained by water pressure method and constant method are in between 4.29 x 10^{-4}cm/sec - 2 x 10^{-5}cm/sec.

4.4 Foundation of Weir

As seen in Fig. III-9, alternating layers of sandstone and claystone (S2 layer) are found at the depth of 11 - 5 meters from ground surface. These alternating layers indicate over 50 in standard penetration value. Field permeability tests were carried out at each borehole along the proposed weir line, and the results are summarized in Table III-2-1 and III-2-2. General permeability coefficients in the layer of S1 are rather high; between 2 x 10^{-3} and 2 x 10^{-4} cm/sec, and a cut-off wall will be required to minimize water leakage through this layer. General permeability coefficients in the layer of S2 (the alternating layers of sandstone and claystone) are between 4 x 10^{-4} and 2 x 10^{-5} cm/sec and no treatment is required.

Judging from the above results, the alternating layers of sandstone and claystone would be compact and massive enough for the foundation of the proposed headworks.

5. GEOLOGICAL SURVEY AT RANAU REGULATING DAM SITE

5.1 General Feature

Lake Ranau is situated at about 100-km south of Martapura. The water level of the lake lies at about 540 m above mean sea level.

Structural feature of Lake Ranau is rectangular volcano-tectonic basin which was formed by a caldera shaped subsidence. This volcano-tectonic depression is characterized by the presence of a huge amount of acid pumice tuffs (Ignimbrite). These tuffs are rhyolitic and partly welded, which flows widely overlie along the Semango Valley on the top of the Barisan Mountains in Pleistocene time.

The only one outlet of the lake water is situated at the north-western corner of Lake Ranau. The Selabung river flows from the outlet to north-west direction following the line of the Semango Valley. The regulating dam site is selected at about 2-km downstream of the outlet. The width of the river is about 50 m and its mean water depth is 0.5 m. Hard rhyolitic tuff bed forms the river bed and numbers of tuff boulders are scattered over the whole width of the river bed with a few deposits of sands and gravels.

5.2 Drilling Investigation

The purpose of the geological investigation on Ranau regulating dam site is to study the details of engineering geology and stratigraphy. For this purpose, test drilling was carried out at five bore holes selected along the weir axis; one hole on the left bank, one hole on the left bank and three in the river bed (See Fig. III-10). Total drilling depth would amount to 55 m. In addition, field permeability tests were carried out at each bore hole (See Fig. III-11).

5.3 Geological Conditions of Regulating Dam Site

According to the drilling results foundation of the Ranau Regulating Dam consists of Ryolitic and welded tuff (See Fig. III-12).

1st layer

The 1st layer extends over the top of both banks, being composed of silty clay and sand, and this layer is 1.6 - 4-meter thick. This layer is generally brown or blackish brown, and consists of loose medium to coarse grain sand and silty clay. The results of field permeability test show the coefficient of around 2.0×10^{-4} cm/sec in the constant head tests.

2nd layer

Below the first layer lies gray colored fresh and hard ryolitic and welded tuff (Ignimbrite), which is approximately 10 meters thick. The results of field permeability test show very small (impermeable) coefficient; around 7.70×10^{-7} cm/sec in the water pressure test.

3rd layer

The 3rd layer consists of weakly welded sandy tuff. Core recovery of the layer is nearly nil, though the foregoing sample of this layer showed drilling cuttings. It is gray color and medium to coarse grained. The fragment consists of quartz, feldspar, igneous rock and mica. The layer is situated at 10 - 12-meter depth. The results of field permeability test show small coefficient of around $5 \times 10^{-5} - 9 \times 10^{-7}$ cm/sec in the water pressure test. The layer is partially rather permeable in the constant head test.

5.4 Foundation of Weir

It is known from the result of the drilling investigation that the base rock of the proposed dam site is composed of the ryolitic and welded tuff (Ignimbrite), having a thickness of over 20 meters (See Fig. III-11).

The base rock is lithologically divided into two types of layers, namely, 2nd layer and 3rd layer. The 2nd layer is composed of fresh massive hard ryolitic and welded tuff, whereas the 3rd layer is formed weakly with welded sandy tuff.

The results of field permeability test carried out at each borehole along the proposed dam site line are summarised in Table III-3-1 and III-3-2. General permeability coefficient in the base rock is between 4×10^{-4} and 3×10^{-6} cm/sec.

Judging from the above results, this site would have favourable geological condition as the foundation of the proposed dam.

Nevertheless, there is problem to be solved through further detailed investigation. More test boreholes should be drilled around the proposed dam axis to investigate further detailed lithofacies, bearing strength and permeability of the 3rd layer. These are very important for the design of foundation treatment.

6. CONSTRUCTION MATERIALS

6.1 Rock Materials

There is a quarry site located at Bukit Mapas, about 15-km south from Martapura, which is being used for producting materials for concrete aggregate and road metal (See Fig. III-1).

The rocks available at this quarry site are blocks of andesite, which are exposed at the surface. The size of present quarry is 30 meters in width and one kilometer in length. Providing the depth of quarry to be 10 - 20 meters, the quantity of rock materials available from this quarry would amount to 300,000 m^3 in gross. In order to have a more accurate idea about the available quantity of materials, however, the detailed investigation by means of test boring is needed.

Rock samples were collected from surface exposures and tested for their compression strength at the soil and road laboratory in Palembang. The test results are shown in Table III-5. According to the results of the laboratory rock mechanical test, the compression strength of the moistened andesite is 462 kg/cm^2. Judging from this value, this rock can be used as concrete aggregate materials and for other construction purposes such as road metalling, masonry work, etc.

Table III-1-1 RESULTS OF FIELD PERMEABILITY TEST AT PRACAK

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BH 1-1	3-4	Constant	7.05×10^{-5}	-	-
	4-6	"	7.40×10^{-5}	-	-
	6-7	"	1.11×10^{-4}	-	-
	7-10	"	2.84×10^{-4}	-	4.78
	10-12.4	"	8.64×10^{-5}	-	5.55
	12.4-15	"	4.62×10^{-6}	-	"
	15-18	"	3.45×10^{-5}	-	3.66
	18-21	Pressure	2.33×10^{-5}	1	5.65
		"	3.25×10^{-5}	3	"
		"	3.66×10^{-5}	5	"
		"	3.45×10^{-5}	3	"
		"	2.88×10^{-5}	1	"
BH 1-2	0-2	Constant	2.07×10^{-3}	-	1.80
	2-4	"	2.37×10^{-3}	-	1.80
	4-6	"	7.87×10^{-6}	-	1.85
	6-8	"	2.20×10^{-5}	-	1.98
	8-10	"	1.46×10^{-5}	-	2.00
	10-12	"	7.09×10^{-6}	-	3.00
	12-14	"	7.25×10^{-6}	-	"
	14-16	"	6.62×10^{-6}	-	"
	16-18	"	9.03×10^{-6}	-	"
	18-20	"	1.91×10^{-5}	-	"
	20-22	"	2.09×10^{-5}	-	"
	22-25	"	2.71×10^{-5}	-	"
BH 1-4	3-6	Constant	6.30×10^{-5}	-	0.15
	6-9	"	5.55×10^{-4}	-	0.15
	18-21	"	6.57×10^{-6}	-	10.93
	21-25	"	2.39×10^{-6}	-	13.50

Table III-1-2 RESULTS OF FIELD PERMEABILITY TEST AT PRACAK

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BI pr.2	0-3	Constant	1.29×10^{-3}	-	-
	3-6	"	1.96×10^{-3}	-	3.5
	6-9	"	9.18×10^{-5}	-	4.5
	9-12	"	8.44×10^{-5}	-	3.2
	12-15	"	3.7×10^{-6}	-	4.2
	15-18	"	4.3×10^{-6}	-	12.2
	18-20.5	"	2.32×10^{-4}	-	3.7
	20.5-23	"	6.29×10^{-5}	-	3.2
	23-25	"	7.41×10^{-5}	-	3.2
BI pr.2	0-3	Constant	8.77×10^{-3}	-	-
	3-4.5	"	1.38×10^{-5}	-	1.20
	4.5-6	"	1.3×10^{-5}	-	1.35
	6-9	"	2.21×10^{-4}	-	1.45
	12-15	Pressure	2.28×10^{-6}	-	1.70
	15-16.5	Constant	5.61×10^{-3}	-	1.40
	16.5-18	"	2.67×10^{-2}	-	1.25
	18-19.5	"	8.28×10^{-3}	-	1.30
	21-22	"	3.6×10^{-3}	-	1.30
	22-25	"	4.3×10^{-6}	-	1.30

Table III-1-3 RESULTS OF FIELD PERMEABILITY TEST AT PRACAK

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BI pr.3	0-3	Constant	6.35×10^{-5}	-	2.0
	3-6	"	4.72×10^{-5}	-	2.0
	6-9	"	1.53×10^{-5}	-	4.0
	9-12	"	1.06×10^{-5}	-	5.4
	12-15	"	2.44×10^{-5}	-	9.35
	15-18	Pressure	3.33×10^{-4}	-	-
	19-21	"	1.39×10^{-4}	-	14.5
	21-24	"	2.13×10^{-4}	-	14.5
BI pr.4	0-3	Constant	3.88×10^{-5}	-	-
	3-6	"	8.53×10^{-5}	-	0.4
	6-9	"	4.01×10^{-5}	-	1.76
	9-12	"	2.18×10^{-5}	-	1.76
	12-15	"	3.9×10^{-5}	-	4.5
	15-18	Pressure	1.3×10^{-6}	-	5.46
	18-21	"	2.16×10^{-5}	-	5.70
	21-24	"	1.7×10^{-5}	-	5.70

Table III-2-1 RESULTS OF FIELD PERMEABILITY TEST AT PERJAYA

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BH 2-1	0-3	Constant	4.98×10^{-3}	-	0.56
	3-6	"	2.92×10^{-3}	-	0.60
	6-9	"	5.03×10^{-4}	-	0.59
	9-12	"	1.98×10^{-3}	-	0.60
	12-15	"	2.71×10^{-4}	-	2.01
	15-18	"	2.36×10^{-4}	-	2.01
	18-21	"	2.90×10^{-5}	-	2.01
	21-25	Pressure	2.19×10^{-5}	1	2.01
		"	2.49×10^{-5}	3	2.01
		"	6.62×10^{-5}	5	2.01
		"	3.26×10^{-5}	3	2.01
		"	3.08×10^{-7}	1	2.01
BH 2-2	6-9	Constant	8.31×10^{-4}	-	2.50
	9-12	"	1.49×10^{-4}	-	2.00
	12-15	"	1.30×10^{-4}	-	"
	15-18	Pressure	8.1×10^{-6}	1	2.90
		"	4.28×10^{-5}	3	"
		"	2.87×10^{-4}	5	"
		"	1.25×10^{-4}	3	"
		"	4.09×10^{-5}	1	"
	21.2-25	"	6.55×10^{-5}	1	2.50
		"	3.27×10^{-4}	3	"
		"	4.92×10^{-4}	5	"
		"	5.02×10^{-4}	3	"
		"	2.27×10^{-4}	1	"

Table III-2-2 <u>RESULTS OF FIELD PERMEABILITY TEST AT PERJAYA</u>

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BH 2-4	9-12	Constant	7.60×10^{-6}	-	1.83
	12-15		4.32×10^{-5}	-	"
	15-18		2.86×10^{-4}	-	2.11
	18-21		2.07×10^{-4}	-	"
	21-25		2.38×10^{-4}	-	2.18

Table III-3-1 <u>RESULTS OF FIELD PERMEABILITY TEST AT RANAU</u>

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BH 4-1	0-3	Constant	1.57×10^{-4}	-	-
	3-7.5	Pressure	1.49×10^{-6}	1	1.01
		"	8.71×10^{-7}	3	"
		"	7.87×10^{-7}	5	"
		"	1.04×10^{-6}	3	"
		"	1.77×10^{-6}	1	"
	7.5-12	"	2.84×10^{-6}	1	"
		"	1.69×10^{-6}	3	"
		"	2.87×10^{-6}	5	"
		"	2.78×10^{-6}	3	"
		"	4.30×10^{-6}	1	"
	12-16	"	1.02×10^{-6}	1	0.22
		"	9.86×10^{-7}	3	"
		"	8.93×10^{-7}	5	"
		"	1.08×10^{-6}	3	"
		"	1.79×10^{-6}	1	"
	16-20	Constant	3.12×10^{-4}	-	0.38

Table III-3-2 RESULTS OF FIELD PERMEABILITY TEST AT RANAU

No.	Depth	Method	Permeability coefficient	Pressure	Water table
	m-m		K = cm/sec	kg/cm^2	m
BH 4-2	0-3	Constant	2.16×10^{-4}	-	-
	3-7	Pressure	1.63×10^{-6}	1	2.40
		"	8.62×10^{-7}	3	"
		"	7.70×10^{-7}	5	"
		"	9.55×10^{-7}	3	"
		"	1.42×10^{-7}	1	"
	7-12	"	1.30×10^{-6}	1	"
		"	7.51×10^{-7}	3	"
		"	7.25×10^{-7}	5	"
		"	9.07×10^{-7}	3	"
		"	1.48×10^{-6}	1	"
	12-16	"	2.71×10^{-6}	1	2.20
		"	2.37×10^{-6}	3	"
		"	3.02×10^{-6}	5	"
		"	2.83×10^{-6}	3	"
		"	3.94×10^{-6}	1	"
	16-20	Constant	4.63×10^{-5}	-	"
	20-25	"	4.29×10^{-5}	-	2.18

Table III-4 RESULTS OF SOIL MECHANICS TESTS

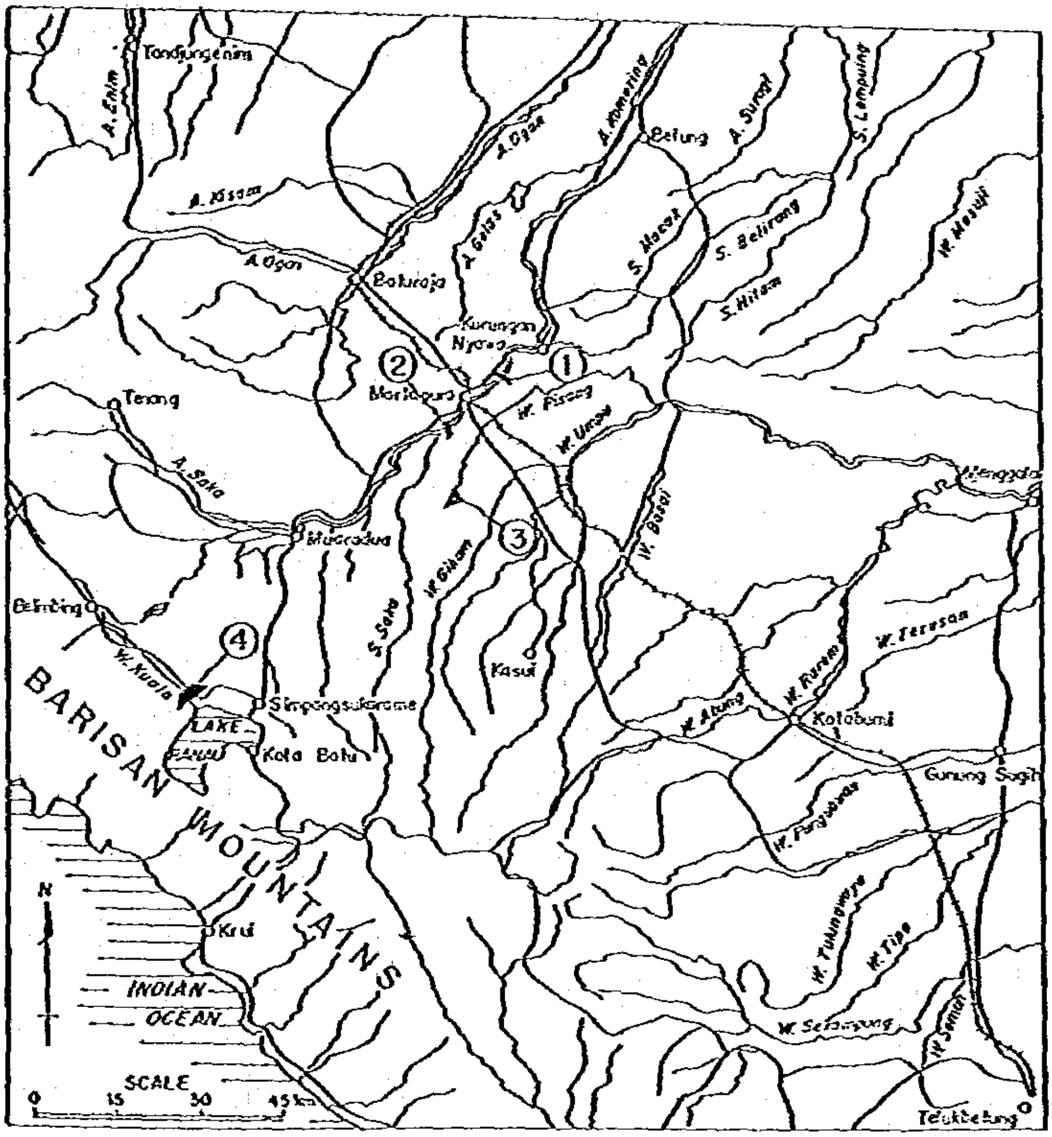
Boring No.	Depth	Moisture Content	Volume Weight	Specific Gravity	Direct Shear				Triaxial				Unconfined Compression		Consolidation			Atterberg			Grain Size Analyses			
		Wn	γ_n	γ_s	ϕ_m	cm	ϕ_u	Cu	ϕ	C	ϕ'	C'			Cc	Cv	k	Wl	Wp	Ip	Gravel	Sand	Silt	Clay
	m	%	t/m^3	t/m^3	degree	kg/cm^2	degree	kg/cm^2	degree	kg/cm^2	degree	kg/cm^2	u	Cu		cm^2/sec	cm^2/sec	%	%	%	%	%	%	%
BIPr.1	1.50- 1.90	36.710	1.910	2.620	-	-	-	-	-	-	-	-	2.85	4.43	0.18	$1.36x10^{-3}$	$4.33x10^{-7}$	89.60	30.33	59.27	-	0.50	65.50	34.0
BIPr.1	14.60-15.00	34.570	1.810	2.590	-	-	-	-	4.00	0.10	9.00	0.05	-	-	0.21	$9.48x10^{-4}$	$2.35x10^{-7}$	54.00	27.50	26.50	-	25.00	60.00	15.00
BIPr.1	17.60-18.00	48.335	1.750	2.780	26	0.09	16	0.13	-	-	-	-	-	-	0.33	$7.05x10^{-4}$	$3.31x10^{-7}$	97.40	41.00	56.40	-	8.00	80.00	12.00
BIPr.1	19.40-19.80	26.505	1.760	2.695	37	0.00	36	0.02	-	-	-	-	-	-	0.14	$1.13x10^{-2}$	$1.56x10^{-7}$	-	-	-	-	65.00	32.00	3.00
BIPr.2	1.60- 2.00	42.240	1.720	2.595	-	-	-	-	23.50	0.400	34.00	0.300	-	-	0.12	$8.26x10^{-4}$	$4.71x10^{-7}$	-	-	-	-	15.00	75.00	10.00
BIPr.2	10.50-10.90	23.110	1.820	2.675	-	-	-	-	-	-	-	-	1.45	0.73	0.17	$3.89x10^{-3}$	$6.90x10^{-7}$	67.60	39.17	28.43	-	5.00	78.00	17.00
BIPr.3	1.00- 1.40	40.890	1.860	2.720	44.50	0.195	39	0.195	-	-	-	-	-	-	1.66	$5.68x10^{-3}$	$4.89x10^{-7}$	111.40	32.795	78.605	6.00	27.00	32.00	35.00
PIPr.3	5.10- 5.50	54.910	1.860	2.735	34	0.145	34	0.00	-	-	-	-	-	-	0.42	$3.83x10^{-4}$	$6.86x10^{-7}$	100.60	45.86	54.74	0.50	0.00	74.50	25.00
BIPr.4	1.40- 1.80	32.545	1.840	2.740	30	0.42	34.25	0.165	-	-	-	-	-	-	0.23	$2.13x10^{-3}$	$1.59x10^{-7}$	91.20	31.65	59.55	3.00	24.00	45.00	28.00
BIPr.4	5.15- 5.55	30.815	2.040	2.715	36.50	0.16	32.50	0.10	-	-	-	-	-	-	0.14	$3.74x10^{-2}$	$5.30x10^{-7}$	34.40	13.66	20.74	-	51.00	33.00	16.00

Table III-5 RESULTS OF ROCK TESTS

Sample	Specific gravity	Strength of Compression (kg/cm^2)	Remarks
No.1	2.702	462.44	48 hours Moistened
No.2	2.864	3,699.50	24 hours dried

Test Date: 17 November 1980

Fig. II - 1 LOCATION MAP



LEGEND

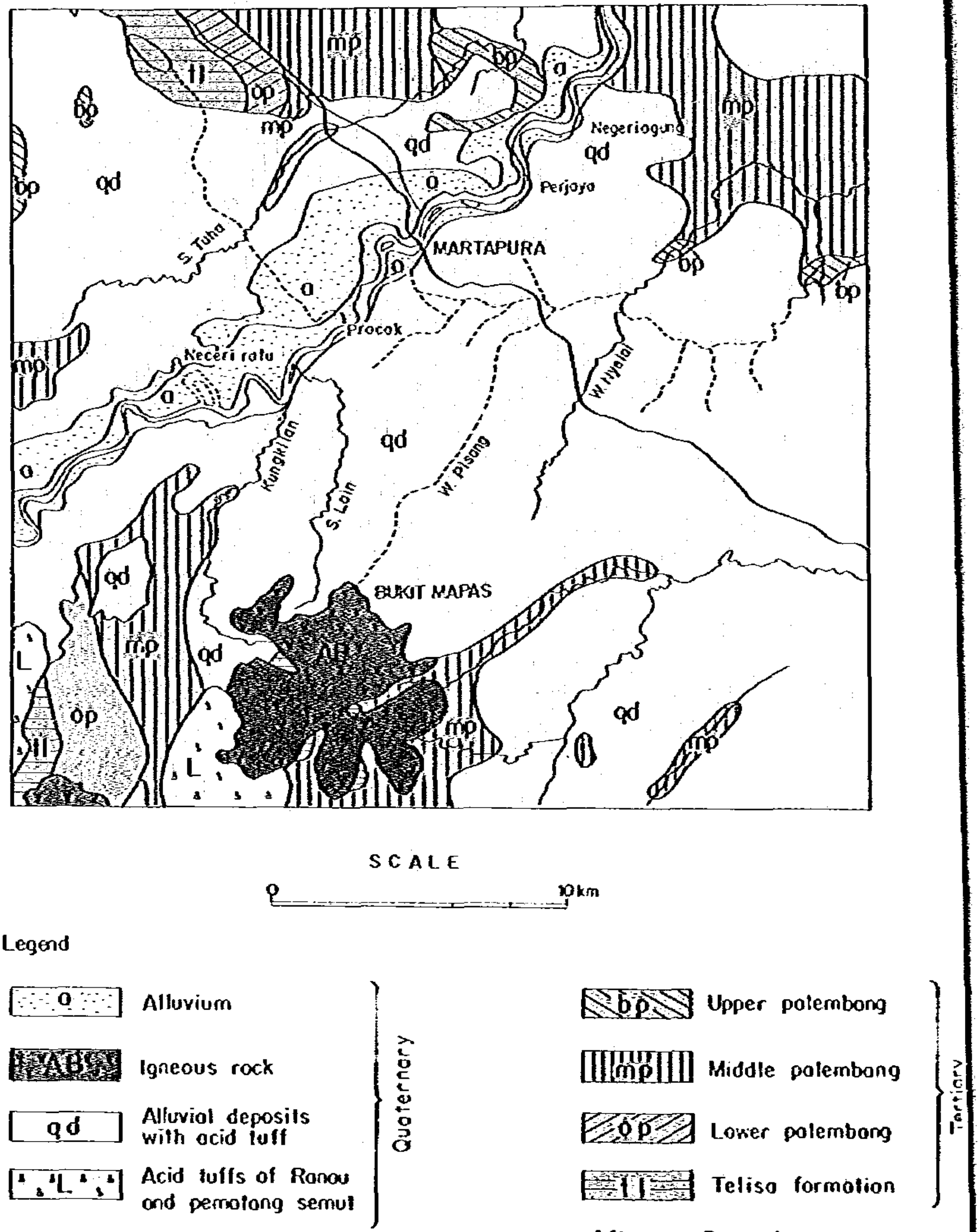
① Perjaya Weir Site

② Pracok Weir Site

③ Bukit Mapas quarry Site

④ Ranau Dam Site

Fig. Ⅲ-2 GENERAL GEOLOGICAL MAP



After van Bemmelen

Fig. III-3 LOCATION MAP OF TEST BORINGS AT PRACAK SITE

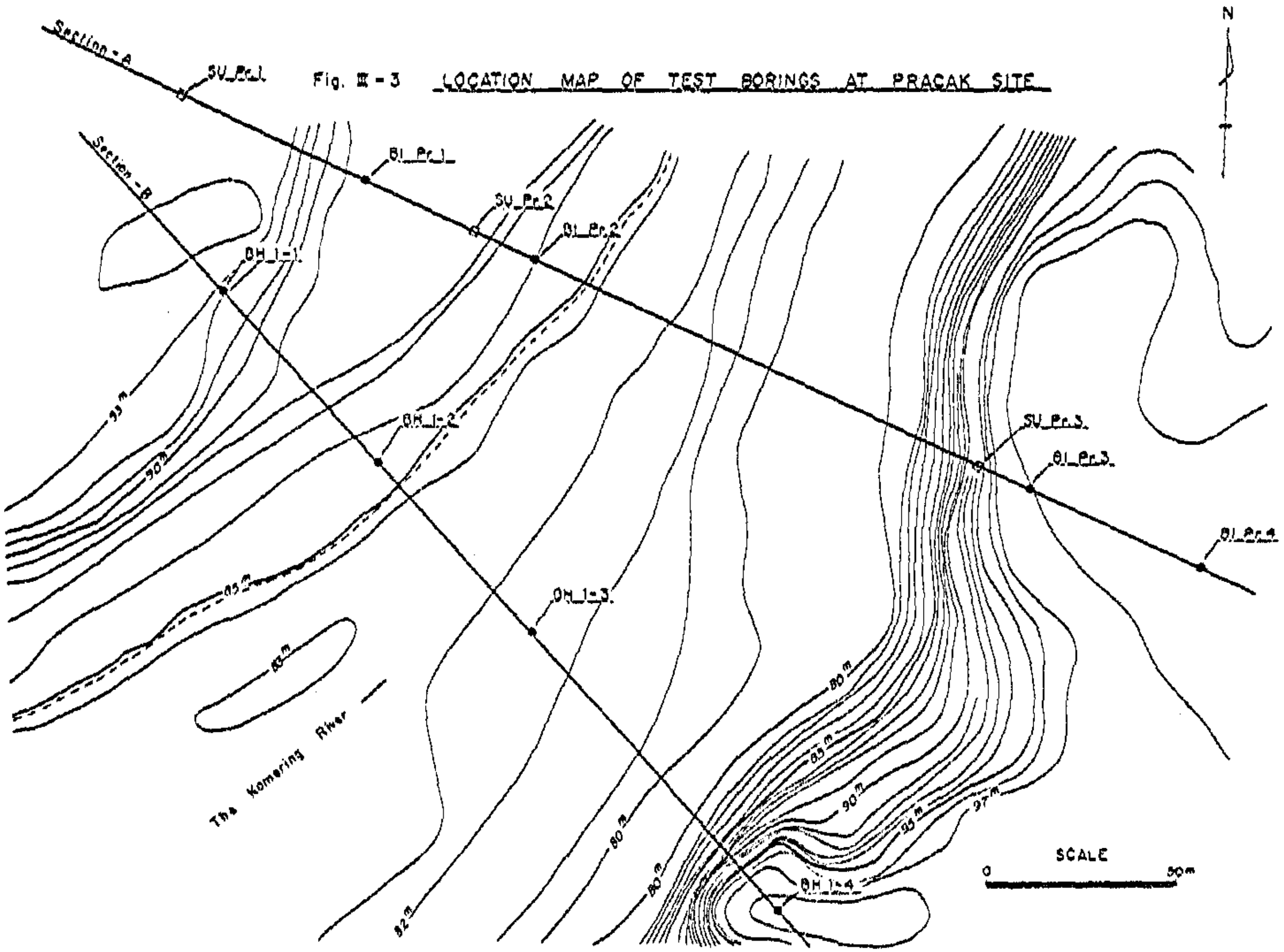


Fig. Ⅲ-4-1

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. 81. Pra

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PRACAK		
ELEVATION OF GROUND SURFACE	88.46 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	2 APR - 12 APR 1980
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
2 APR		87.96	Soil	dark brown, with organic matters
	1			
	2		Clay	brown color
	3	84.96		
	4		Sand and gravel	grayish brown poorly sorted, slightly compact
	5			
3 APR	6	82.45		
		92.26	Cobble	grayish brown w/clay
	7	81.66	Gravel	grayish brown w/f.sand
		81.36	Gravel&Cobb	grayish brown w/clay
4 APR	8		Sandy and gravel	grayish brown, slightly compact medium to fine grained subrounded
	9			
5 APR	10	77.96		
	11	77.26	Gravel	grayish brown, w/sand
6 APR	12	75.86	Sand and gravel	grayish brown, slightly compact, mixed with rounded gravel of 2-4cm
8 APR	13		Sandy clay	gray, stiff, plastic
	14			
	15	73.45		
9 APR	16		Clay stone	gray, hard
	17			
	18	70.45		
10 APR	19		Sand	gray, fine to medium grained well sorted subangular, well rounded well compact
	20	68.45		
11 APR	21		Sand and gravel	gray, verydense compact, the forming fragment consists of quartz, sediment rocks subangular, well rounded poorly sorted.
	22			
12 APR	23			
	24			
	25	63.45		

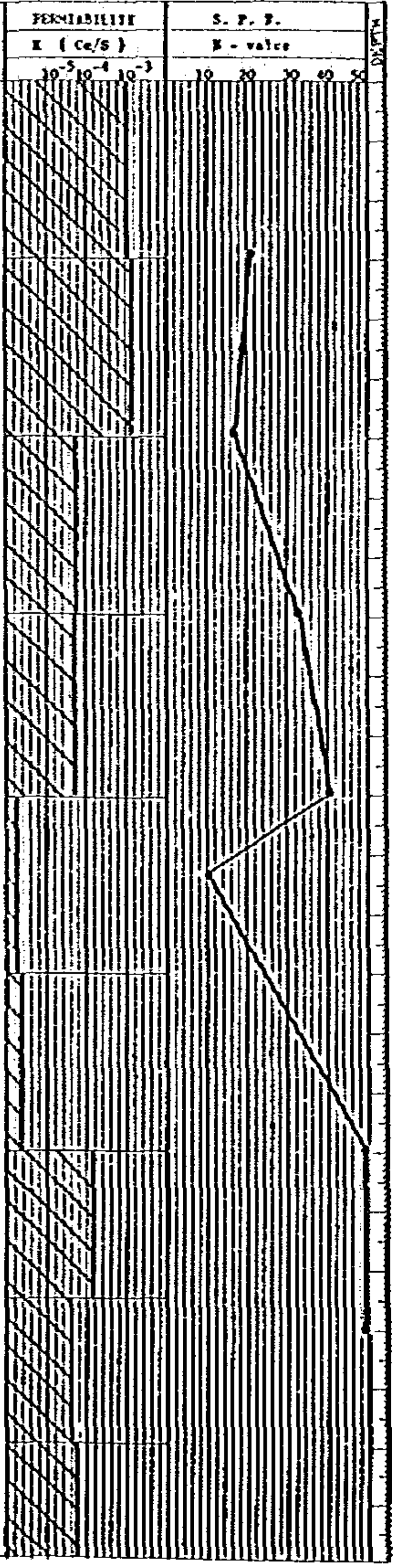
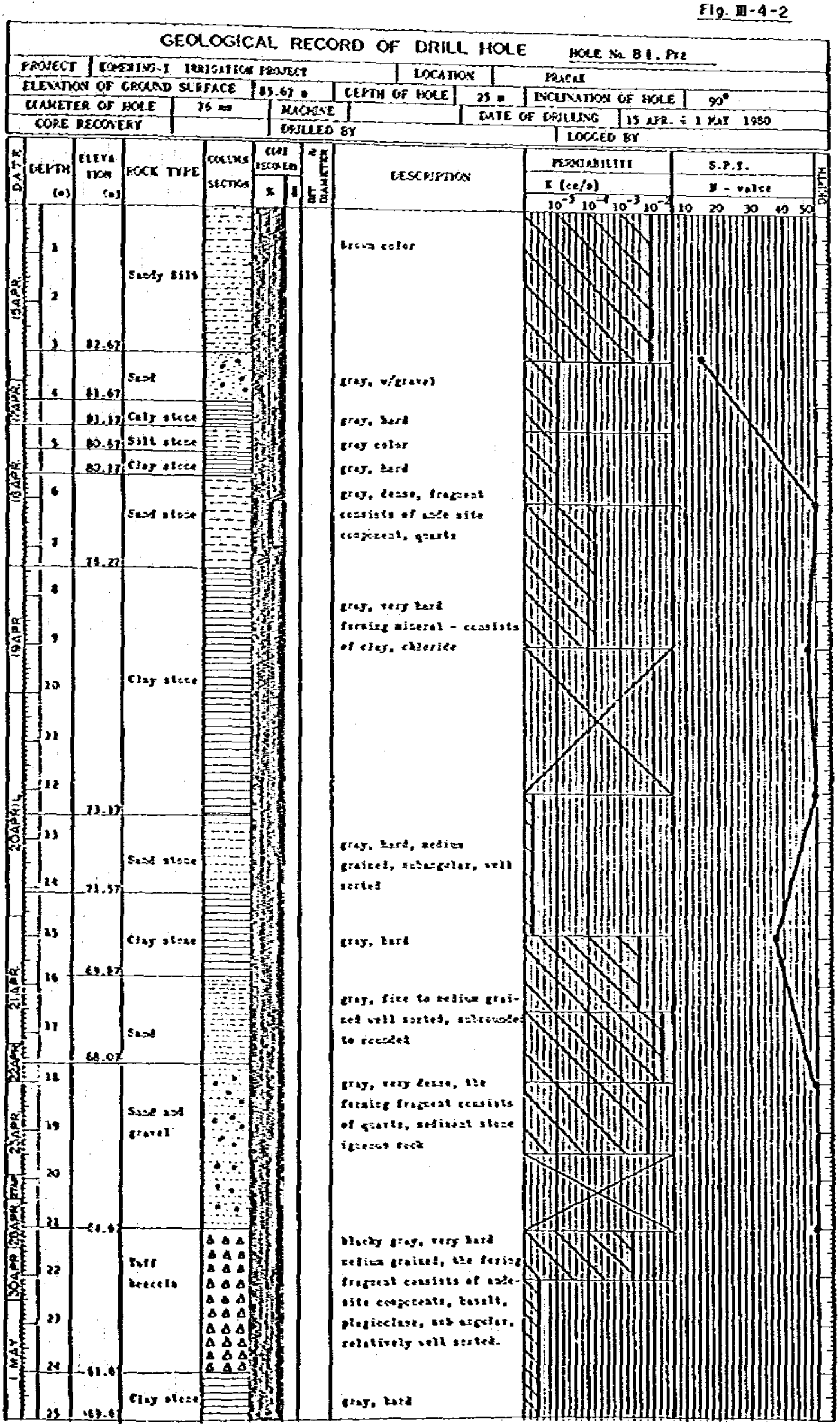


Fig. III-4-2



GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. B I. Pre

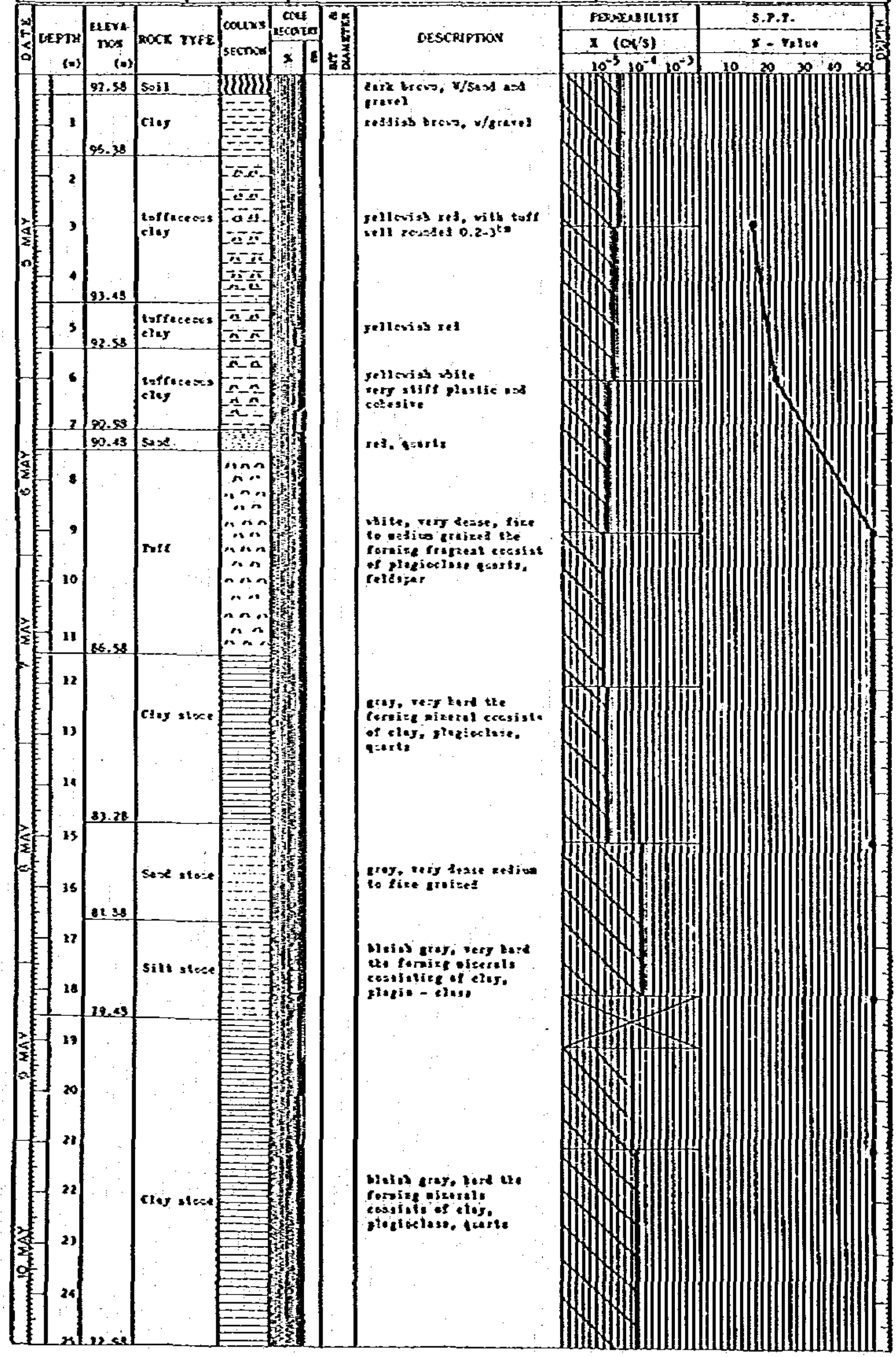
PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PRACAK
ELEVATION OF GROUND SURFACE	85.67 m	DEPTH OF HOLE	25 m
INCLINATION OF HOLE	90°	DIAMETER OF HOLE	76 mm
MACHINE		DATE OF DRILLING	15 APR. ÷ 1 MAY 1980
CORE RECOVERY		DRILLED BY	
LOGGED BY			

DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
3	82.67	Sandy Silt	brown color
4	81.67	Sand	gray, w/gravel
	81.17	Caly stone	gray, hard
5	80.67	Silt stone	grey color
	80.17	Clay stone	gray, hard
	78.27	Sand stone	gray, dense, fragment consists of ande site component, quartz
	73.17	Clay stone	gray, very hard forming mineral - consists of clay, chloride
14	71.57	Sand stone	gray, hard, medium grained, subangular, well sorted
16	69.87	Clay stone	gray, hard
	68.07	Sand	gray, fine to medium grained well sorted, subrounded to rounded
21	64.6	Sand and gravel	gray, very dense, the forming fragment consists of quartz, sediment stone igneous rock
24	61.6	Tuff breccia	blacky gray, very hard medium grained, the foring fragment consists of andesite compcreate, basalt, plagioclase, sub angular, relatively well sorted.
25	69.6	Clay stone	gray, hard

GEOLOGICAL RECORD OF BORING

HOLE No. 81.Pi3

PROJECT	KOMERING-I IRRIGATION PROJECT		LOCATION	P R A C A K	
ELEVATION OF GROUND SURFACE	79.98 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	3 MAY - 10 MAY 1950
CORE RECOVERY		DRILLED BY		LOGGED BY	



DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
		97.58	Soil	dark brown, W/Sand and gravel
	1	96.38	Clay	reddish brown, w/gravel
5 MAY	2 3 4	93.48	tuffaceous clay	yellowish red, with tuff well rounded 0.2-3^{cm}
	5	92.58	tuffaceous clay	yellowish red
	6 7	90.58	tuffaceous clay	yellowish white very stiff plastic and cohesive
		90.48	Sand	red, quartz
6 MAY	8 9 10			
7 MAY	11	86.58	Tuff	white, very dense, fine to medium grained the forming fragment consist of plagioclase quartz, feldspar
	12 13 14	83.28	Clay stone	gray, very hard the forming mineral consists of clay, plagioclase, quartz
8 MAY	15 16	81.38	Sand stone	grey, very dense medium to fine grained
	17 18	79.45	Silt stone	bluish gray, very hard the forming minerals consisting of clay, plagio - class
9 MAY	19 20 21 22			
10 MAY	23 24 25	72.58	Clay stone	bluish gray, hard the forming minerals consists of clay, plagioclase, quartz

Fig III-4-4

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. B1.Pr4

PROJECT	KOMERING - I IRRIGATION PROJECT	LOCATION	P R A C A I		
ELEVATION OF GROUND SURFACE	98.19 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	14 MAY - 21 MAY 1990
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION	DEPTH
14 MAY		97.89	Soil	Dark brown, contain organic	
	1	97.39	Clay	Dark brown, W/sand	
	2 3 4	93.99	Clay	Brownish red, hard, with gravels fragment consist of quartz and sediment stone.	
	5 6 7 8	90.19	Clay	Yellowish red, very stiff contained to gravel the fragment consist of quartz and tuff (pumice).	
15 MAY		89.59	Clay	Yellowish white stiff w/sand.	
	9 10 11		Tuff	White, very dense, fine to medium grained the forming fragment consists of plagioclase quartz, feldspar.	
	12	85.93	Carbonized organic	Black color.	
16 MAY	13 14	83.99	Clay Stone	Gray, hard.	
		83.69	Silt Stone	Gray, hard.	
	15	82.83	Clay Stone	Gray, hard.	
	16 17 18	79.49	Sand Stone	Gray, very dense fine to medium grained, the forming fragment consists of andesite, quartz plagioclase.	
17 MAY 21 MAY	19 20 21 22 23 24	73.99	Clay Stone	Bluish gray, very hard.	
	25	73.19	Sand Stone	Gray, the forming fragment consists of quartz, feldspa	

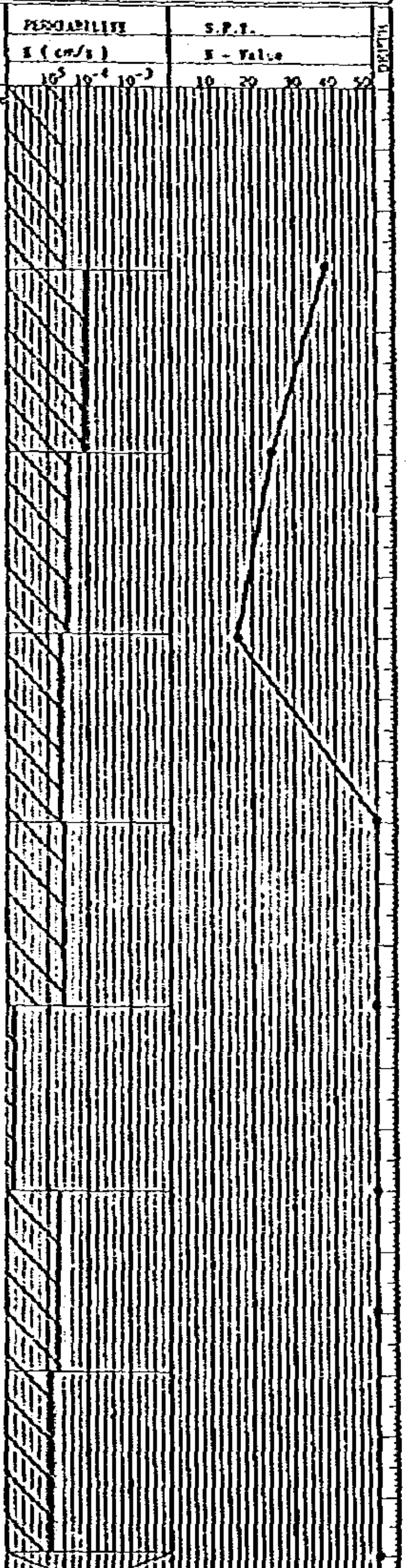
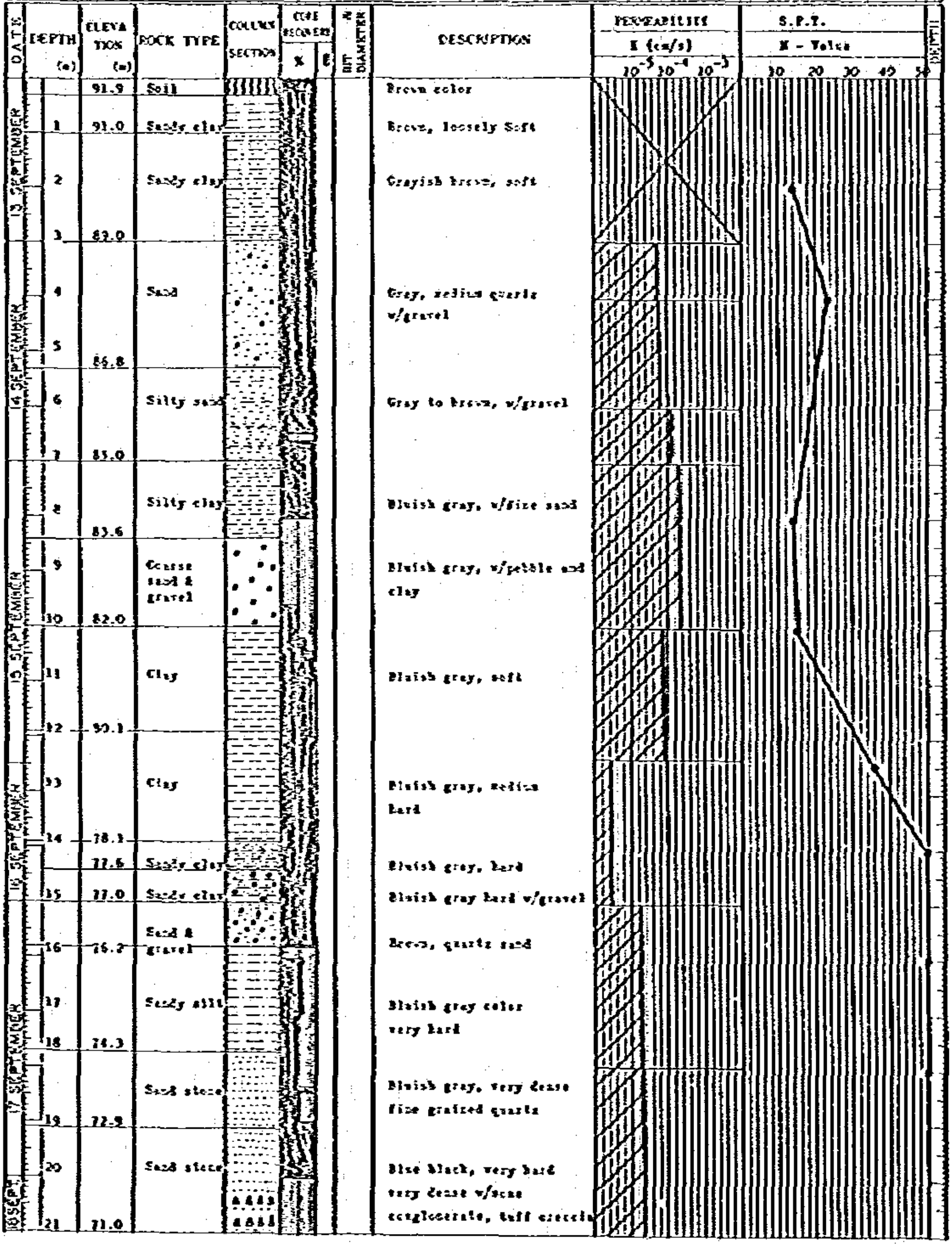


Fig. III-4-5

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH 1-1

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PRACAK
ELEVATION OF GROUND SURFACE	92.0 m	DEPTH OF HOLE	21 m
INCLINATION OF HOLE	90°	DIAMETER OF HOLE	76 mm
MACHINE		DATE OF DRILLING	13 Sept - 18 Sept 1980
CORE RECOVERY		DRILLED BY	
LOGGED BY			



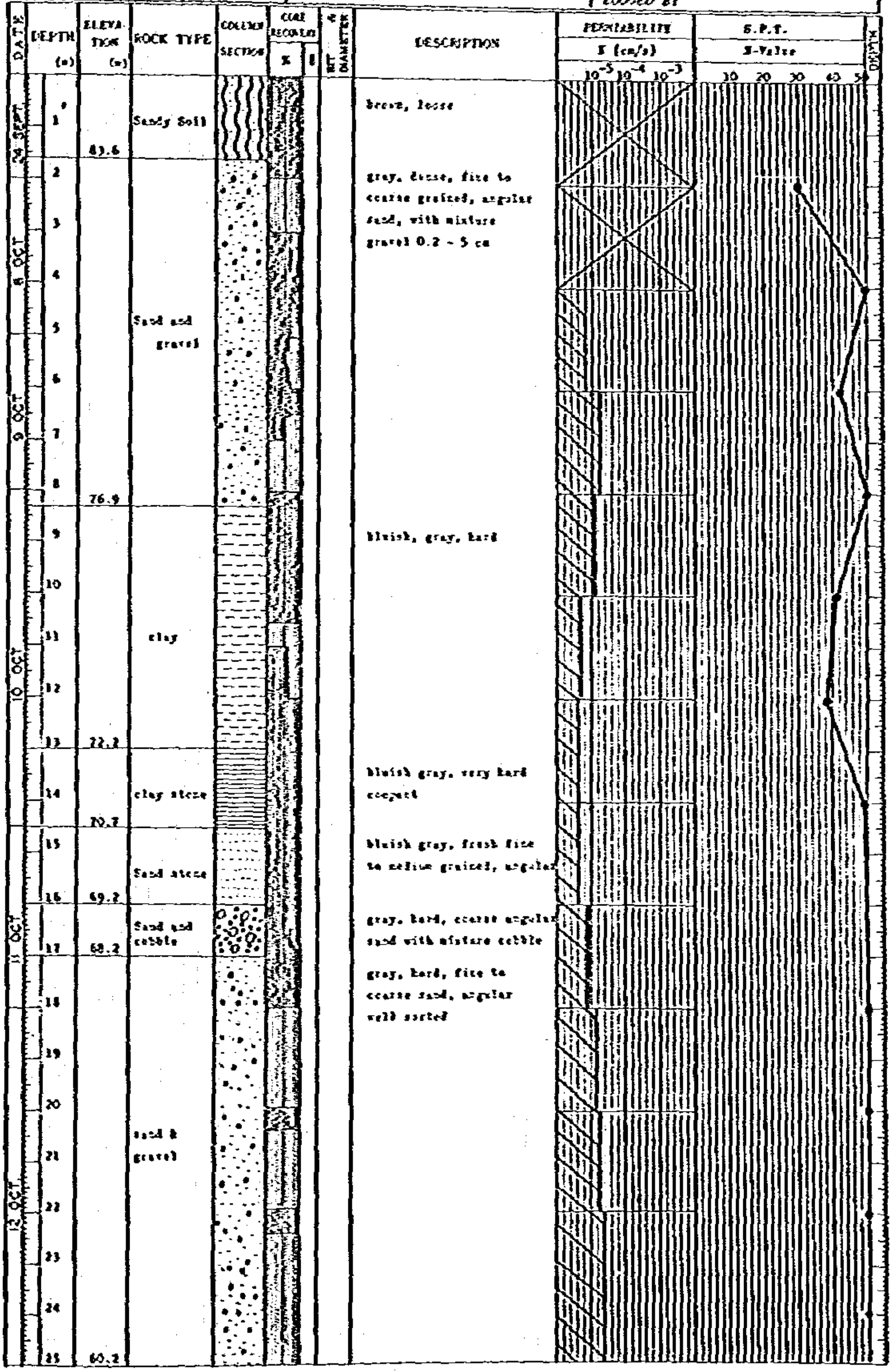
DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
13 SEPTEMBER		91.9	Soil	Brown color
	1	91.0	Sandy clay	Brown, loosely Soft
	2		Sandy clay	Grayish brown, soft
	3	89.0		
14 SEPTEMBER	4		Sand	Gray, medium quartz w/gravel
	5	86.8		
	6		Silty sand	Gray to brown, w/gravel
	7	85.0		
15 SEPTEMBER	8		Silty clay	Bluish gray, w/fine sand
		83.6		
	9		Coarse sand & gravel	Bluish gray, w/pebble and clay
	10	82.0		
	11		Clay	Bluish gray, soft
	12	80.1		
16 SEPTEMBER	13		Clay	Bluish gray, medium hard
	14	78.1		
		77.6	Sandy clay	Bluish grey, hard
	15	77.0	Sandy clay	Bluish gray hard w/gravel
			Sand & gravel	Brown, quartz sand
	16	76.2		
17 SEPTEMBER	17		Sandy silt	Bluish grey color very hard
	18	74.3		
			Sand stone	Bluish gray, very dense fine grained quartz
	19	72.9		
18 SEPT.	20		Sand stone	Blue black, very hard very dense w/some conglomerate, tuff breccia
	21	71.0		

Fig Ⅲ-4-6

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH 1-2

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PRACAK		
ELEVATION OF GROUND SURFACE	85.2 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	24 Sept - 12 Oct. 1980
CORE RECOVERY		DRILLED BY		LOGGED BY	



DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
1	83.6	Sandy Soil	brown, loose
2–8	76.9	Sand and gravel	gray, dense, fine to coarse grained, angular sand, with mixture gravel 0.2 - 5 cm
9–13	72.2	clay	bluish, gray, hard
14	70.7	clay stone	bluish gray, very hard compact
15–16	69.2	Sand stone	bluish gray, fresh fine to medium grained, angular
17	68.2	Sand and cobble	gray, hard, coarse angular sand with mixture cobble
18–25	60.2	sand & gravel	gray, hard, fine to coarse sand, angular well sorted

Fig III-4-7

GEOLOGICAL RECORD OF BORING

HOLE No. BH I-4

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PRACAK		
ELEVATION OF GROUND SURFACE	98 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	21 OCT - 25 OCT
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	COLUMN SECTION	CORE RECOVERY %	BIT DIAMETER	DESCRIPTION
	1	96.5	Clay with gravel				brownish-red, stiff
	2						
21 OCT	3		Tuffaceous-Clay				brownish-red, stiff
	4	93.5					
	5						
	6		Tuffaceous-Clay				brownish-gray very-stiff
	7	90.6					
22 OCT	8		Tuffaceous-Sand stone				yellow-brown, dense, fine to coarse grain sand angular Poorly sorted, the fragment Consisting of pumice tuff, quartz, sediment stone
	9	88.2					
	10		Tuff				yellow-white, hard, fine to medium size, consist of tuff fragments pumice angular well sorted partly brownish-red color
	11						
	12	86					
	13		Clay				bluish-gray, very stiff hard
23 OCT	14						
	15						
	16	82					
	17		Sand stone				bluish-gray, fresh, fine to medium grain sand, angular, well sorted the fragment consist of pumice quartz sedimentstone rather well cemented by clay
	18						
24 OCT	19	79					
	20		Gravely Sand				bluish-gray, very dense the sand fragment consist of quartz, igneous rock, sediment stone, angular poorly sorted.
	21						
	22	76					
25 OCT	23		Clay stone				bluish-gray, fresh
	24	73.3					
	25	73	Sand stone				bluish-gray, fresh, fine to medium grain, angular well sorted, the fragment consist of pumice, Sediment stone

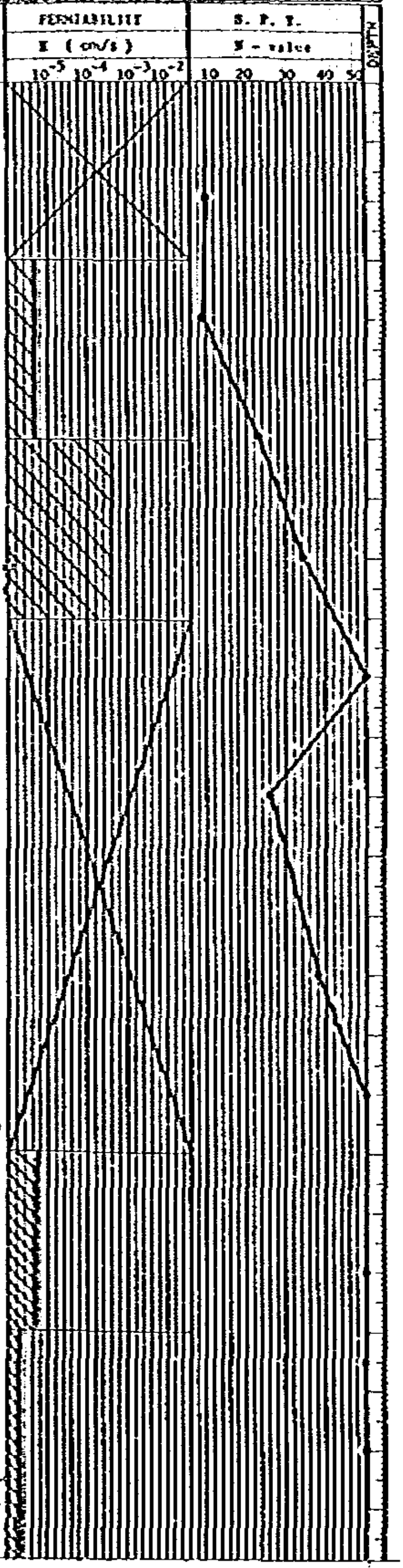


Fig. III-5 GEOLOGICAL CROSS SECTION OF PRACAK SITE (Section-A)

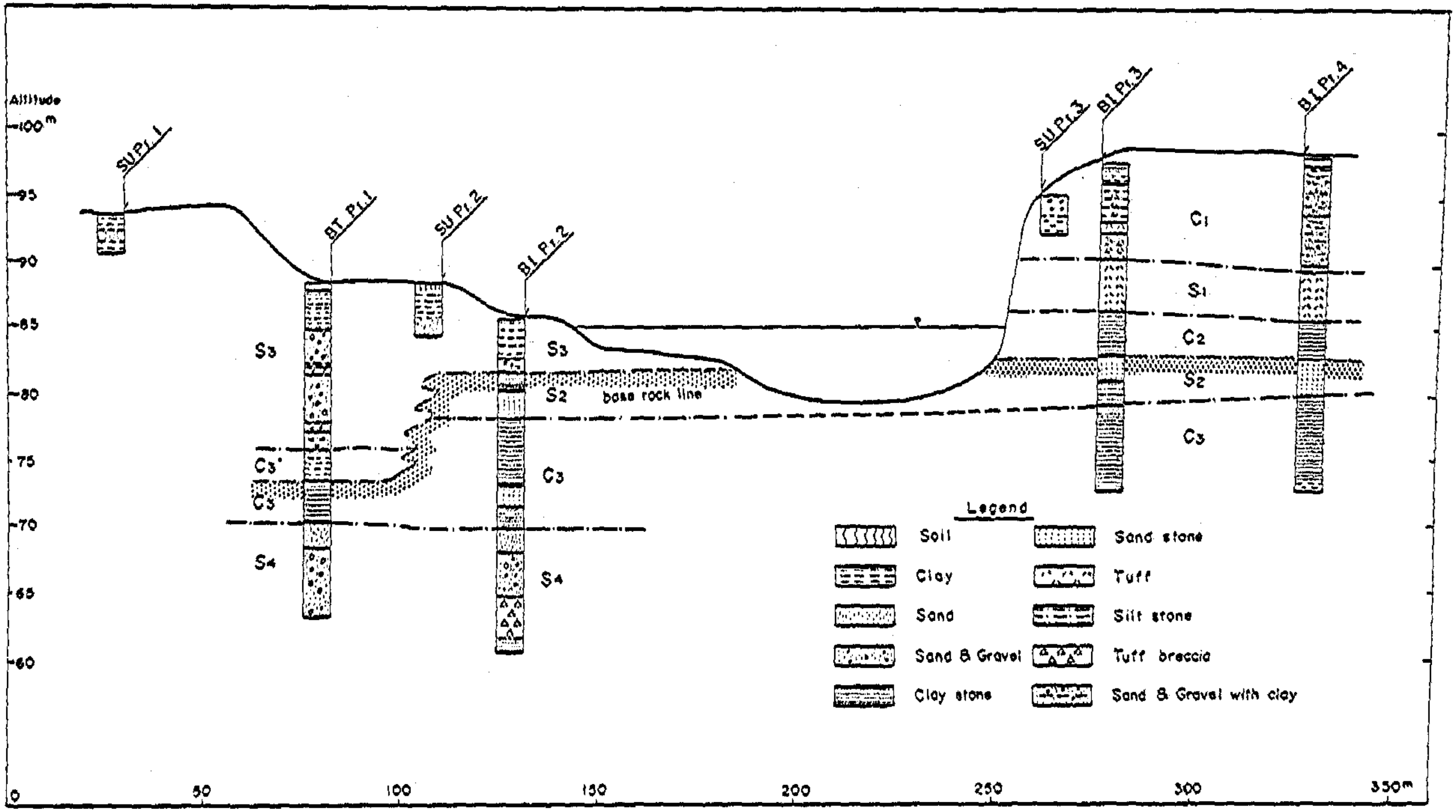


Fig. III-6 GEOLOGICAL CROSS SECTION OF PRACAK SITE (Section-B)

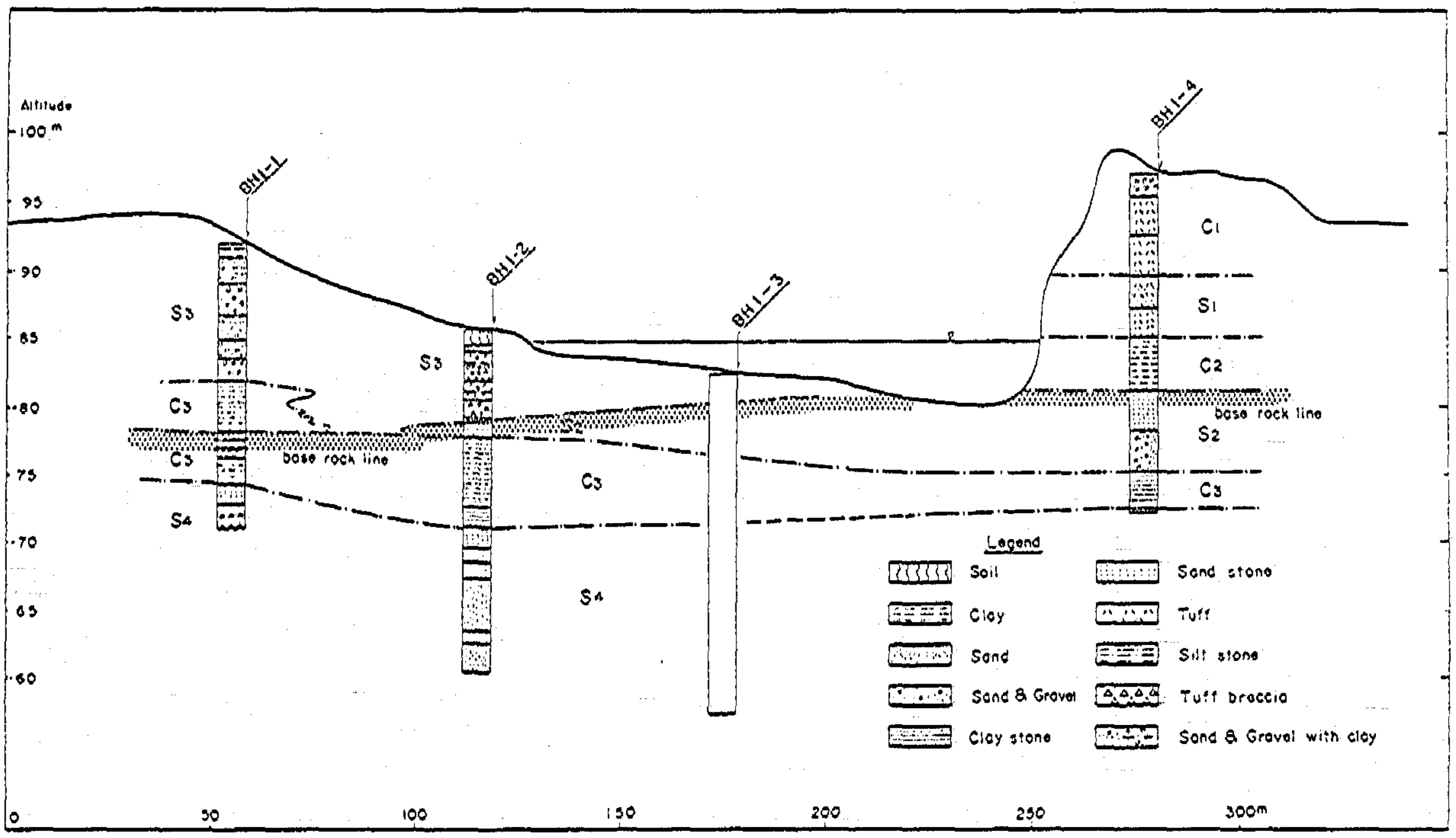


Fig. III-7 LOCATION MAP OF TEST BORINGS AT PERJAYA SITE

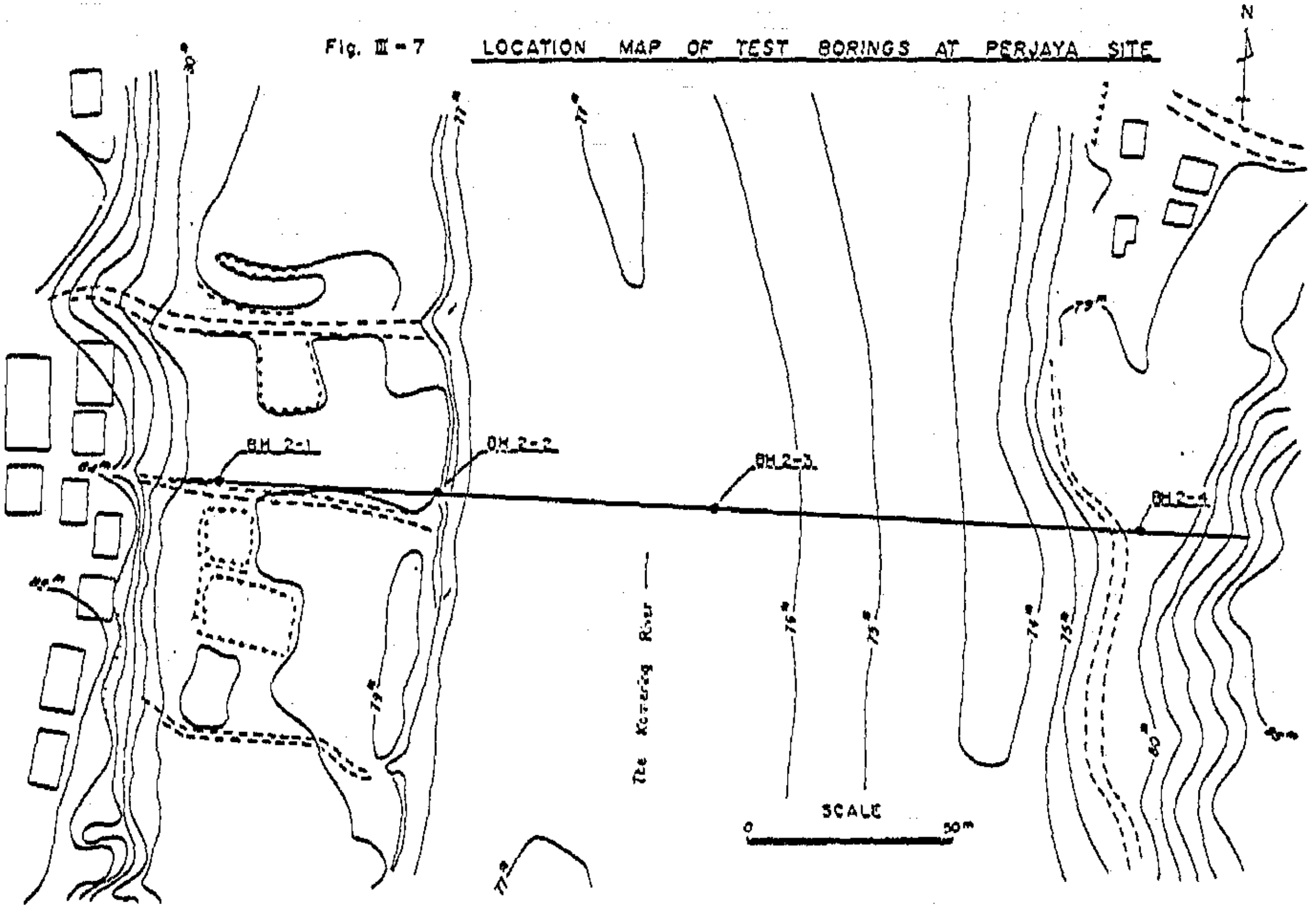
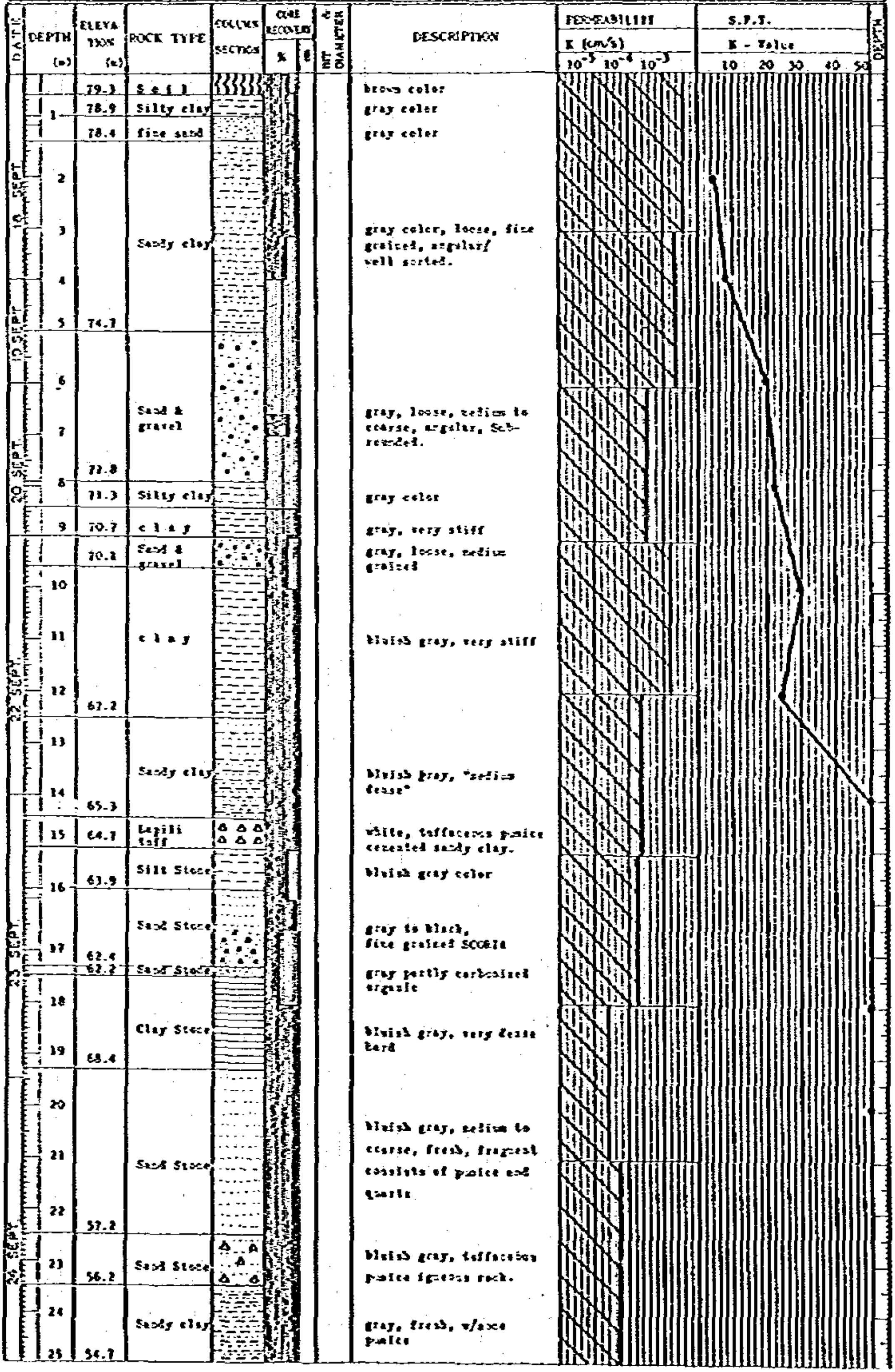


Fig. III-8-1

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH 2-1

PROJECT	KOMERING I IRRIGATION PROJECT	LOCATION	PERJAYA		
ELEVATION OF GROUND SURFACE	79.7 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	18 SEPT - 24 SEPT 1980
CORE RECOVERY		DRILLED BY		LOGGED BY	



DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
		79.3	Soil	brown color
	1	78.9	Silty clay	gray color
		78.4	fine sand	gray color
18 SEPT	2 3 4 5	74.7	Sandy clay	gray color, loose, fine grained, angular/ well sorted.
19 SEPT	6 7	72.8	Sand & gravel	gray, loose, medium to coarse, angular, Sub-rounded.
20 SEPT	8	71.3	Silty clay	gray color
	9	70.7	clay	gray, very stiff
		70.2	Sand & gravel	gray, loose, medium grained
22 SEPT	10 11 12	67.2	clay	bluish gray, very stiff
	13 14	65.3	Sandy clay	bluish gray, "medium dense"
	15	64.7	Lapilli tuff	white, tuffaceous pumice cemented sandy clay.
	16	63.9	Silt Stone	bluish gray color
23 SEPT	17	62.4	Sand Stone	gray to black, fine grained SCORIA
		62.2	Sand Stone	gray partly carbonized organic
	18 19	60.4	Clay Stone	bluish gray, very dense hard
	20 21 22	57.2	Sand Stone	bluish gray, medium to coarse, fresh, fragment consists of pumice and quartz
24 SEPT	23	56.2	Sand Stone	bluish gray, tuffaceous pumice igneous rock.
	24 25	54.7	Sandy clay	gray, fresh, w/some pumice

Fig. III-8-2

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH 2-2

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PERJAYA		
ELEVATION OF GROUND SURFACE	79.0 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	27 SEPT - 16 OCT 1980
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	DESCRIPTION
27 SEPTEMBER	1	77.5	Sandy Soil	brown, loose, w/some roots of plant
	2	76.4	Sandy Clay	brownish gray stiff
	3	75.2	Clayey fine sand	brownish gray, very fine grained, Sub angular
	4 5 6	72.8	Sand & gravel	brown, very dense fine to medium grained angular quartz
28 SEPT.	7	72.0	Sand	gray, very hard fine to medium
5 OCT.	8	70.6	Sand	gray, very dense fine to medium
6 OCT.	10 11	68.0	Sand	grayish white, very dense fine to medium angular well Sorted
15 OCT.	12	66.4	Clay stone	bluish gray, fresh, compact
14 OCTOBER	13 14	65.2	Tuff breccia	gray, component consist of pumice, angular, matrix consist of sand.
	15	63.9	Sand stone	gray, fresh, fine to medium, angular well sorted.
	16	62.5	Tuff breccia	gray, component consist of pumice, angular, matrix consist of sand
	17	61.8	Sand stone	bluish gray, fine to medium
15 OCT.	18 19 20	59.0	Silty stone	gray, fresh
16 OCTOBER	21 22	56.4	Sand stone	bluish gray, fresh fine to medium grained well sorted
	23 24	54.7	Sand stone	bluish gray, fresh fine to medium grained well sorted
	25	54.0	clay stone	bluish gray, fresh

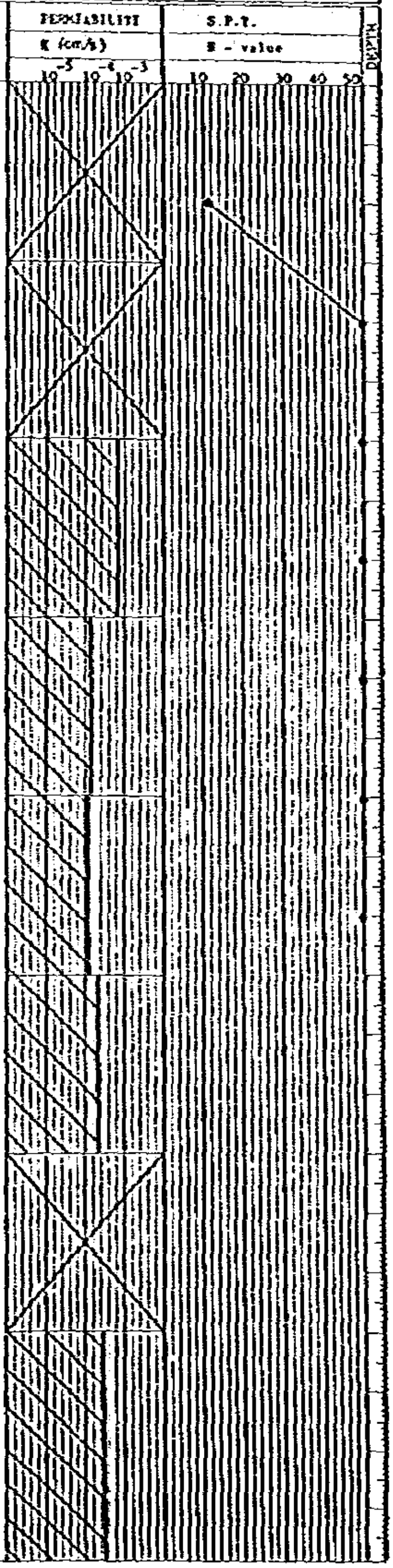


Fig III-8-3

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No BH 2-4

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	PERJAYA		
ELEVATION OF GROUND SURFACE	79.8 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	21 Oct. 24 Oct.
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	COLUMNAR SECTION	CORE RECOVERY %	CORE RECOVERY ∅	BIT DIAMETER	DESCRIPTION
21 OCT.		79.3	Soil					Silty clay, blakish-brown w/organic matters
	1	78.3	Silty Sand					brown, loose, fine grain anguler, well sorted
	2 3	76.2	Clay					gray color, stiff
	4 5 6	73	Sand					gray, very dense, fine to medium grain, angular, well sorted, the fragment consists of pumice, quartz w/clay
	7 8 9 10 11	68.2	Caly					gray color, hard
22 OCT.	12 13	66.8	Gravelly sand					gray, fine to medium, poorly cemented, gravel ∅ 0.2 – 4.5 cm angular
	14 15 16 17 18	61.3	Sand					gray, fine to coarse angular, well sorted, consist of quartz fragment, igneous and sediment stone
23 OCT.	19 20	59.5	Sand					black, fine to medium grain angular well sorted, w/organic matter
	21	58.3	Sand					gray, fine to medium grain angular well sorted
24 OCT.	22 23	56.5	Silty sand					gray, fine to medium grain angular, well sorted, ∅ 0.2 – 4.5 cm
	24 25	54.8	Silty clay					blaish gray, fresh with organic matter, black color

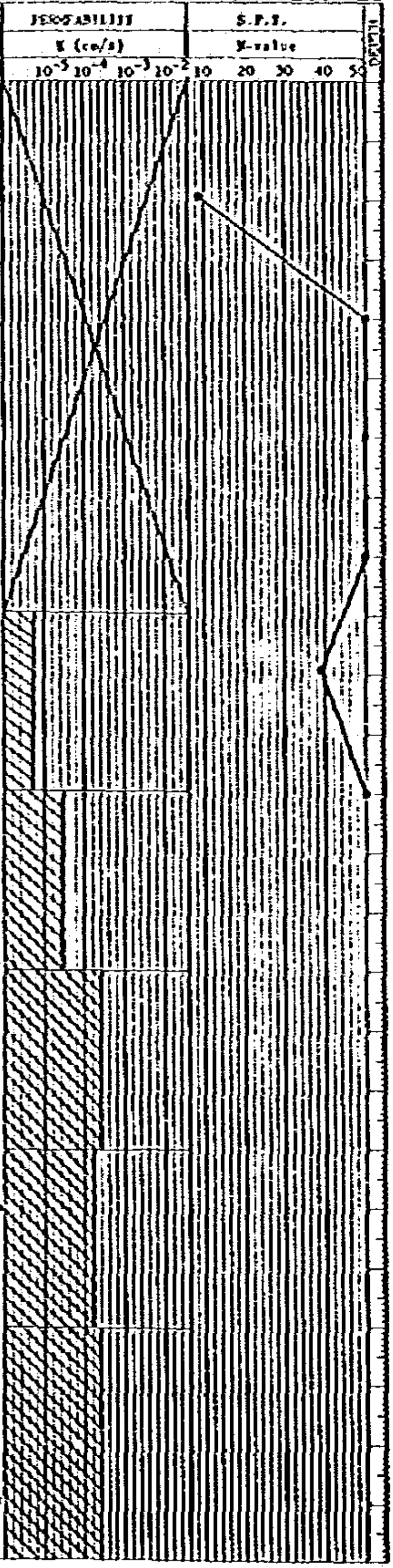


Fig. III - 9 GEOLOGICAL CROSS SECTION OF PERJAYA SITE

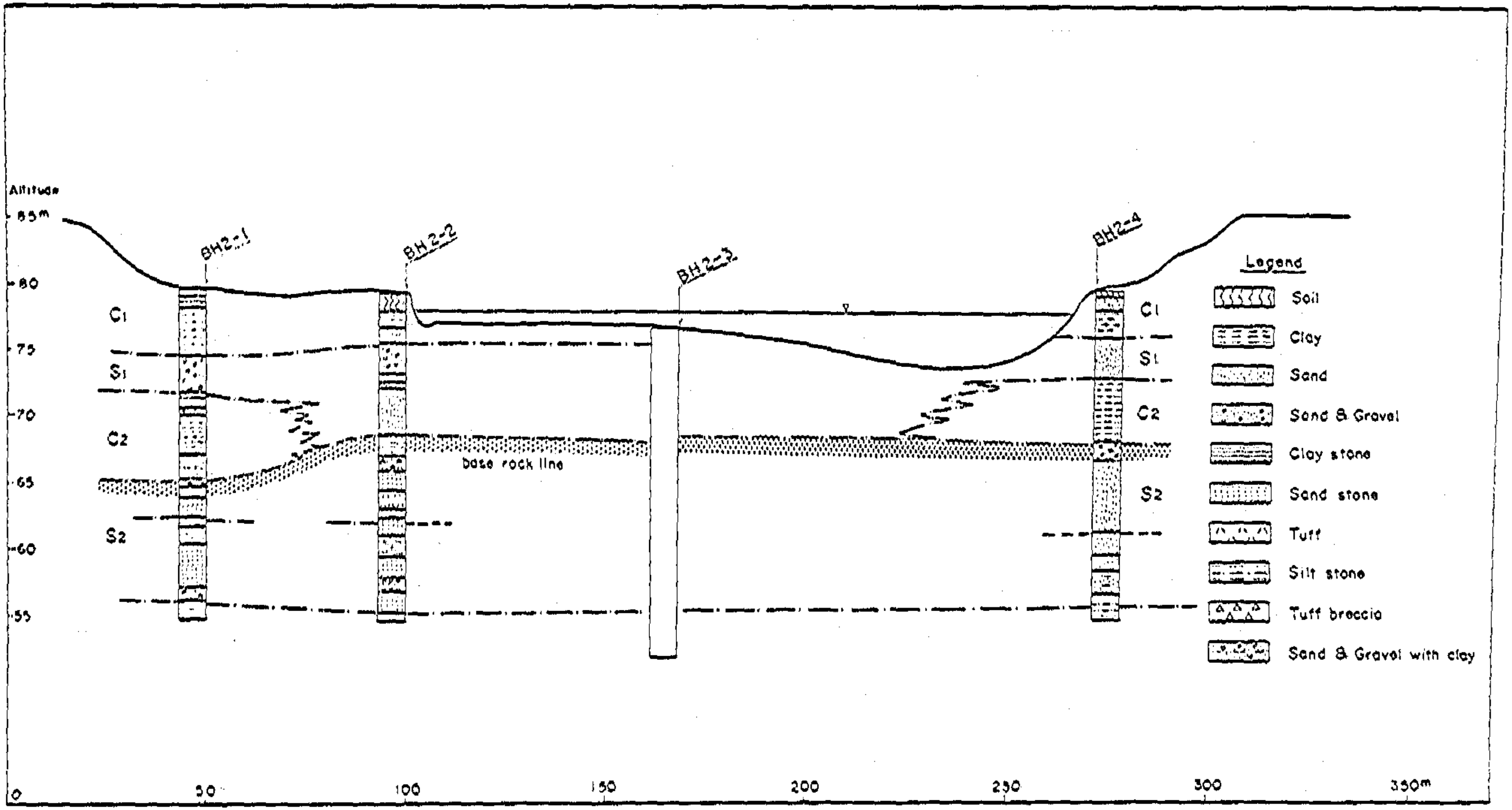


Fig. III-10 LOCATION MAP OF TEST BORINGS AT RANAU REGULATING DAM SITE

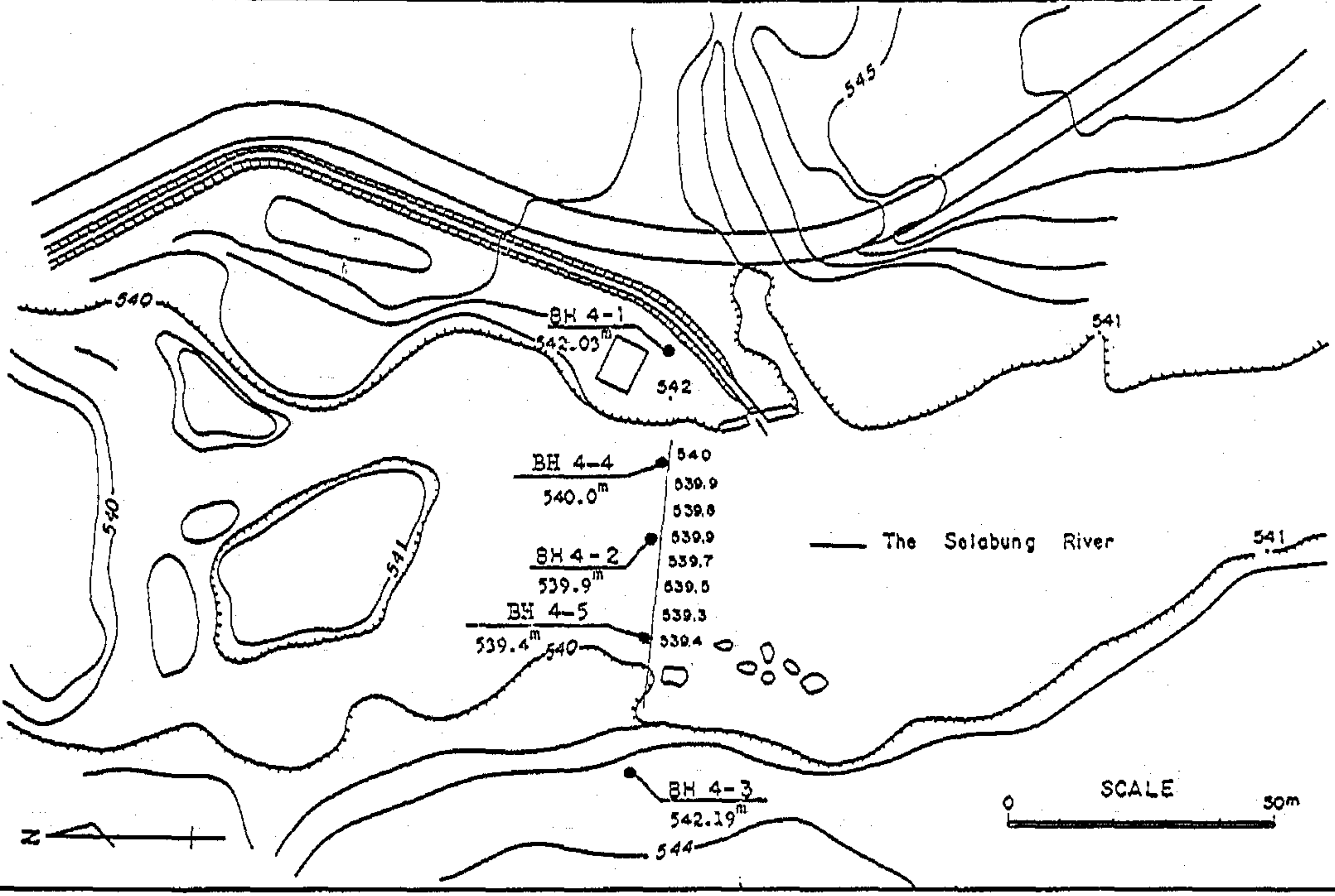


Fig. Ⅲ-11-1

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH4-1

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	RANAU		
ELEVATION OF GROUND SURFACE	542.03 m	DEPTH OF HOLE	20 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	15 NOV - 17 NOV
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	COLUMN SECTION	CORE RECOVERY %	RQD	BIT DIAMETER	DESCRIPTION
	1	540.83	Silty Sand					brown, loose, fine to medium grained, the fragment consist of quartz feldspar, igneous rock, angular, well sorted
		540.43	Sand					
15 NOV	2 – 10	531.83	Ryolitic & welded tuff (Ignimbrite)					redish brown, loose medium to coarse grained, consist of fragment quartz, mica, igneous rock, angular, well sorted gray, fresh, consist of fragment quartz, igneous rock, pumice, mica, medium to coarse grained, angular well sorted, cemented by silicate with igneous rock component ∅0.2 - 5cm angular
16 NOV								
17 NOV	11 – 20	522.03	Weakly welded Sandy tuff (Cuttings)					gray, consist of sand, medium to coarse grained angular, quartz, feldspar, pumice igneous rock mica

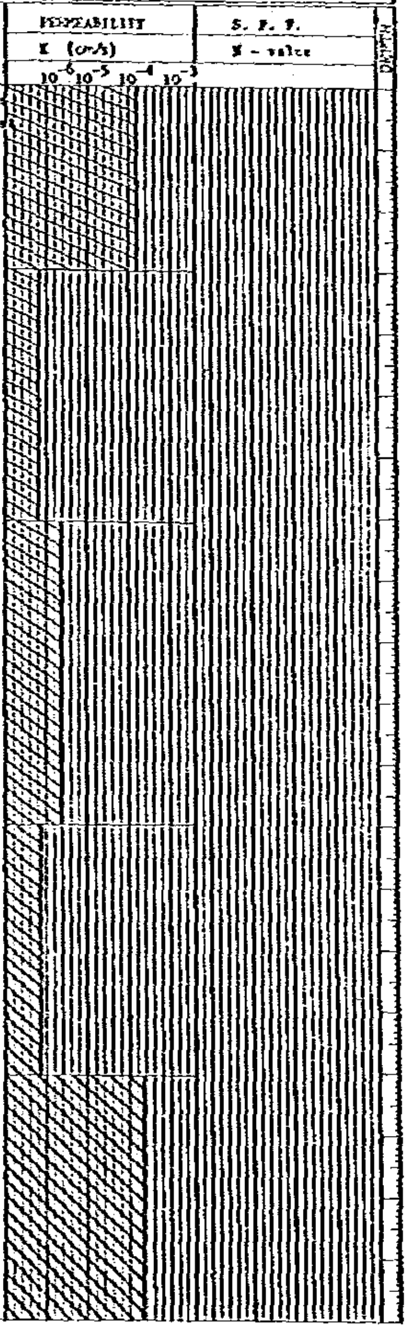
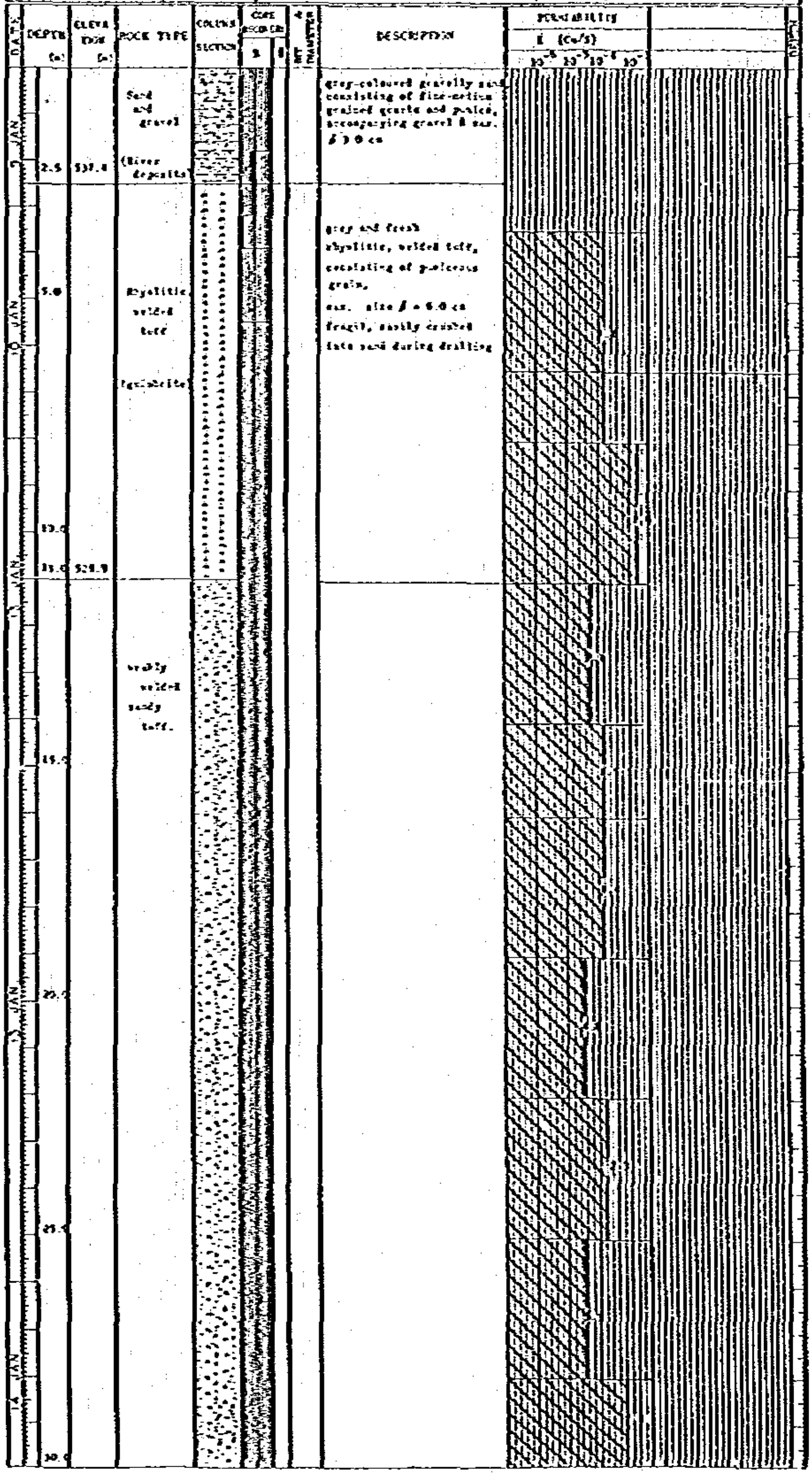


Fig. Ⅲ-11-2

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH 4 - 2

PROJECT	KEMERING - I IRRIGATION PROJECT	LOCATION	RANAU
ELEVATION OF GROUND SURFACE	539.9 m	DEPTH OF HOLE	30 m
INCLINATION OF HOLE	90°	DIAMETER OF HOLE	76 mm
MACHINE		DATE OF DRILLING	9 - 14 JAN. 1981
CORE RECOVERY		DRILLED BY	
LOGGED BY			



grey-coloured gravelly sand consisting of fine-medium grained quartz and pumice, accompanying gravel ϕ max. ϕ 3.0 cm

grey and fresh rhyolitic, welded tuff, consisting of pumiceous grain, max. size ϕ = 6.0 cm fragil, easily crushed into sand during drilling

Fig. III-11-3

GEOLOGICAL RECORD OF BORING

HOLE No. BH4-3

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	RANAU		
ELEVATION OF GROUND SURFACE	542.19 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	21 NOV.- 26 NOV.
CORE RECOVERY		DRILLED BY		LOGGED BY	

DATE	DEPTH (m)	ELEVATION (m)	ROCK TYPE	COLUMN SECTION	CORE RECOVERY %	BIT DIAMETER	DESCRIPTION
21 NOV	1		Silty clay				blackish brown, soft
		540.55					
	2		Clayly Sa				blackish brown, very stiff, medium to coarse grained fragment consist of quartz feldspar, mica and igneous rock angular well sorted
	3						
	4	538.15					
	5		Ryolitic and welded tuff (Ignimbrite)				gray, fresh, medium to coarse grained, the fragment consist of quartz feldspar, igneous rock, pumice, mica with component of igneous rock, $\phi 0.2 - 5^{cm}$ angular
	6						
	7						
25 NOV	8						
	9						
	10						
25 NOV	11						
	12	530.15	Weakly welded Sandy tuff (Cuttings)				gray, medium to coarse grained the fragment consist of quartz, feldspar igneous rock and mica
25NOV	13	529.15	Ryolitic and welded tuff				
		528.75					
	14						gray, fresh
	15						
	16						
	17		Weakly welded Sandy tuff (Cuttings)				
	18						gray color
	19						
26 NOV	20						
	21						
	22						
	23						
	24						
	25	517.15					

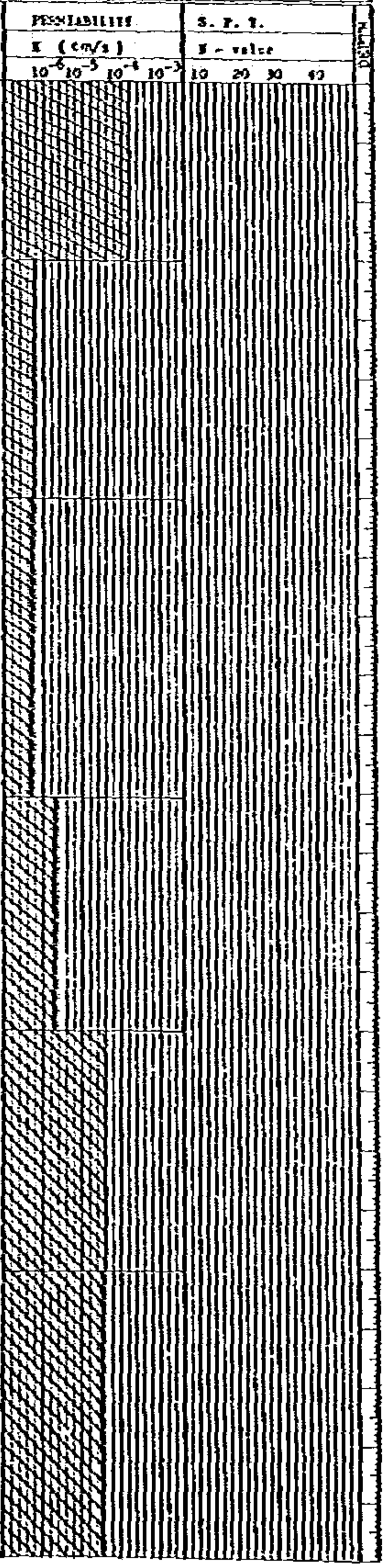
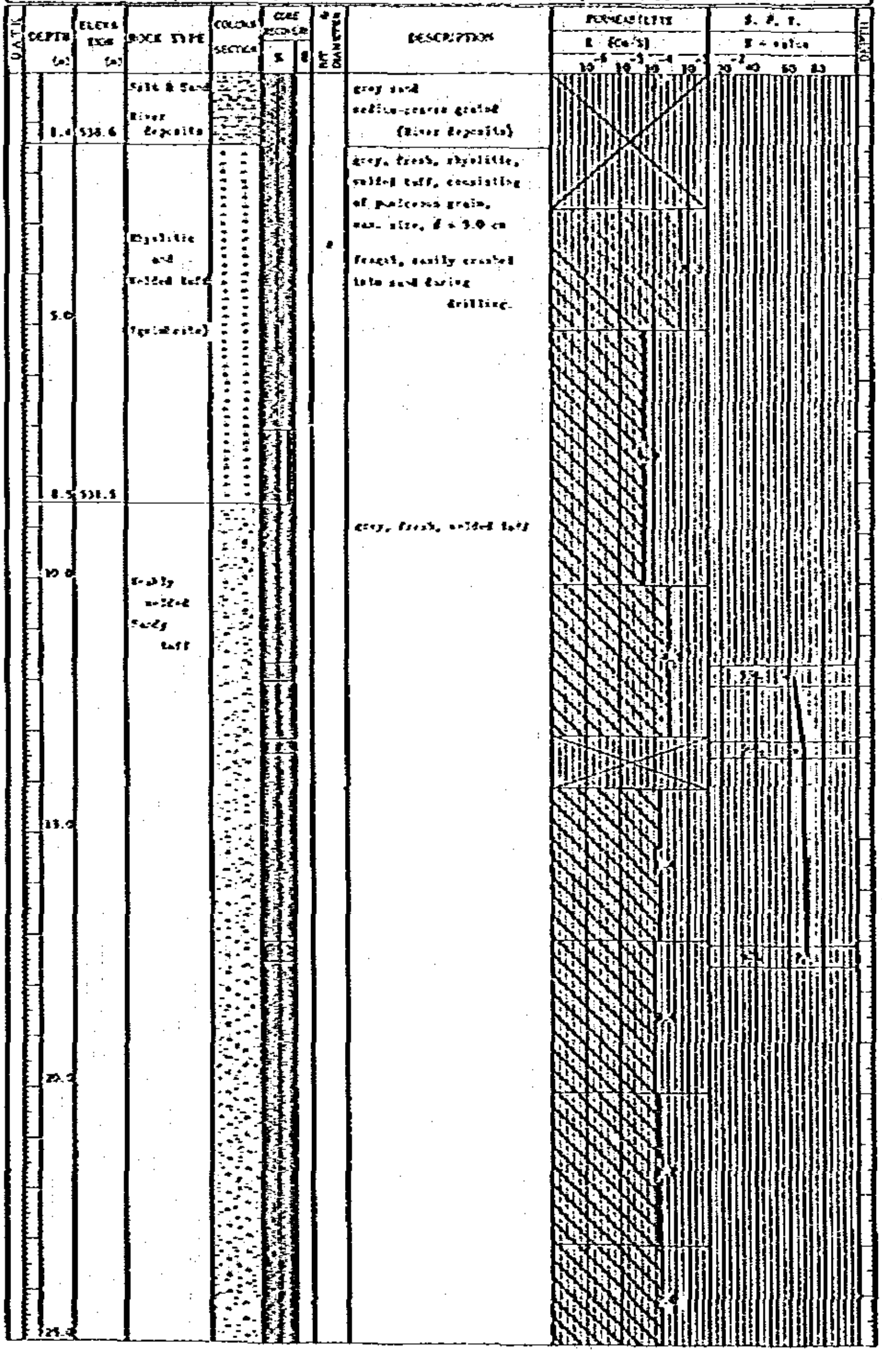


Fig. III-(1)-4

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH-4-4

PROJECT	KOMERING-I IRRIGATION PROJECT	LOCATION	RANAU		
ELEVATION OF GROUND SURFACE	540.0 m	DEPTH OF HOLE	25 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	27, DEC. '83 - 1, JAN. '84
CORE RECOVERY		DRILLED BY		LOGGED BY	



grey sand
medium-coarse grained
(River Deposits)

grey, fresh, rhyolitic, welded tuff, consisting of pumiceous grain, max. size, ø = 3.0 cm

fragil, easily crushed into sand during drilling.

grey, fresh, welded tuff

Fig. III-11-5

GEOLOGICAL RECORD OF DRILL HOLE

HOLE No. BH 4-5

PROJECT	KOMERING-II IRRIGATION PROJECT	LOCATION	RANAU		
ELEVATION OF GROUND SURFACE	539.4 m	DEPTH OF HOLE	30 m	INCLINATION OF HOLE	90°
DIAMETER OF HOLE	76 mm	MACHINE		DATE OF DRILLING	17 - 19 JAN. 1981
CORE RECOVERY		DRILLED BY		LOGGED BY	

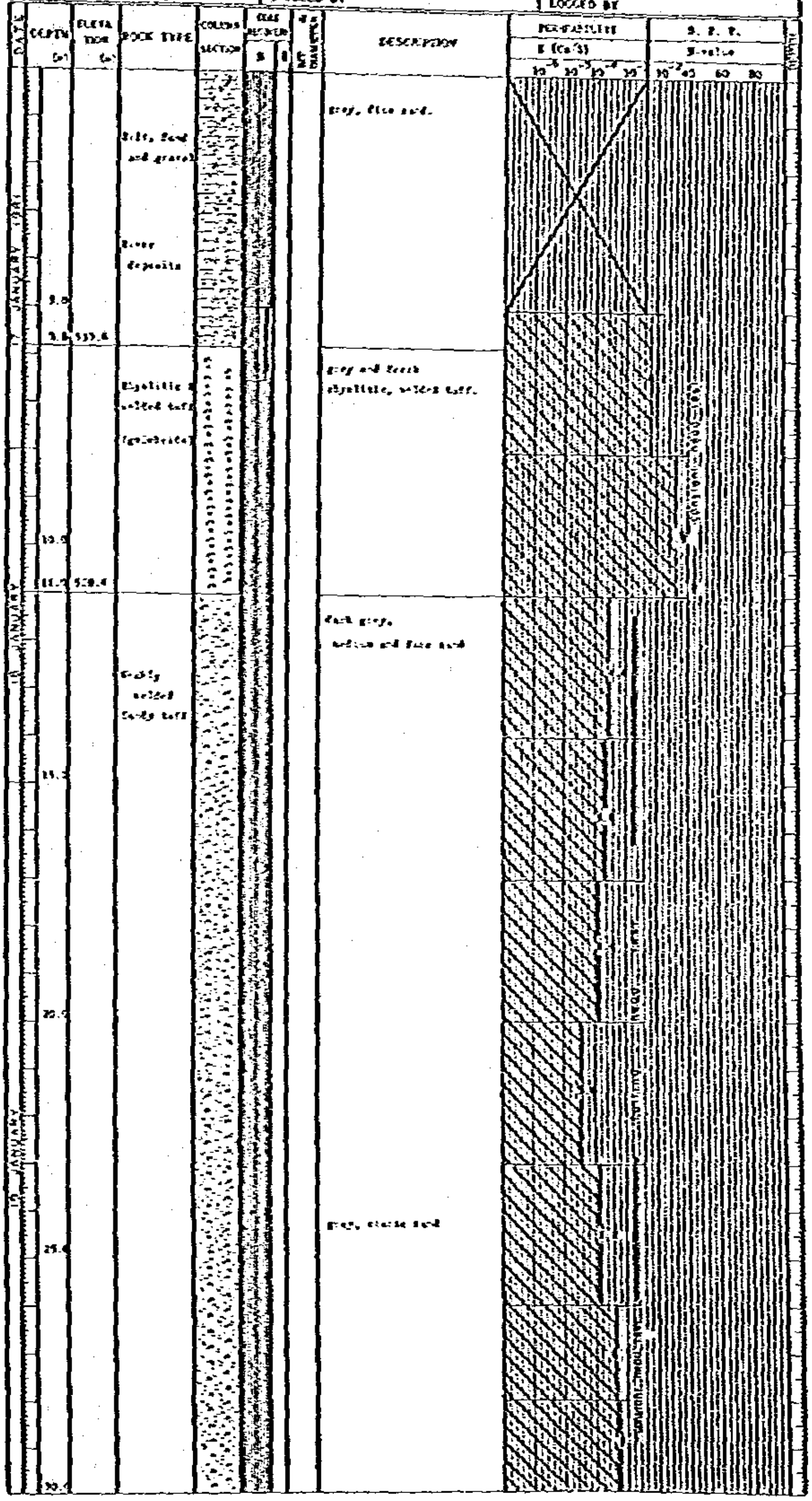
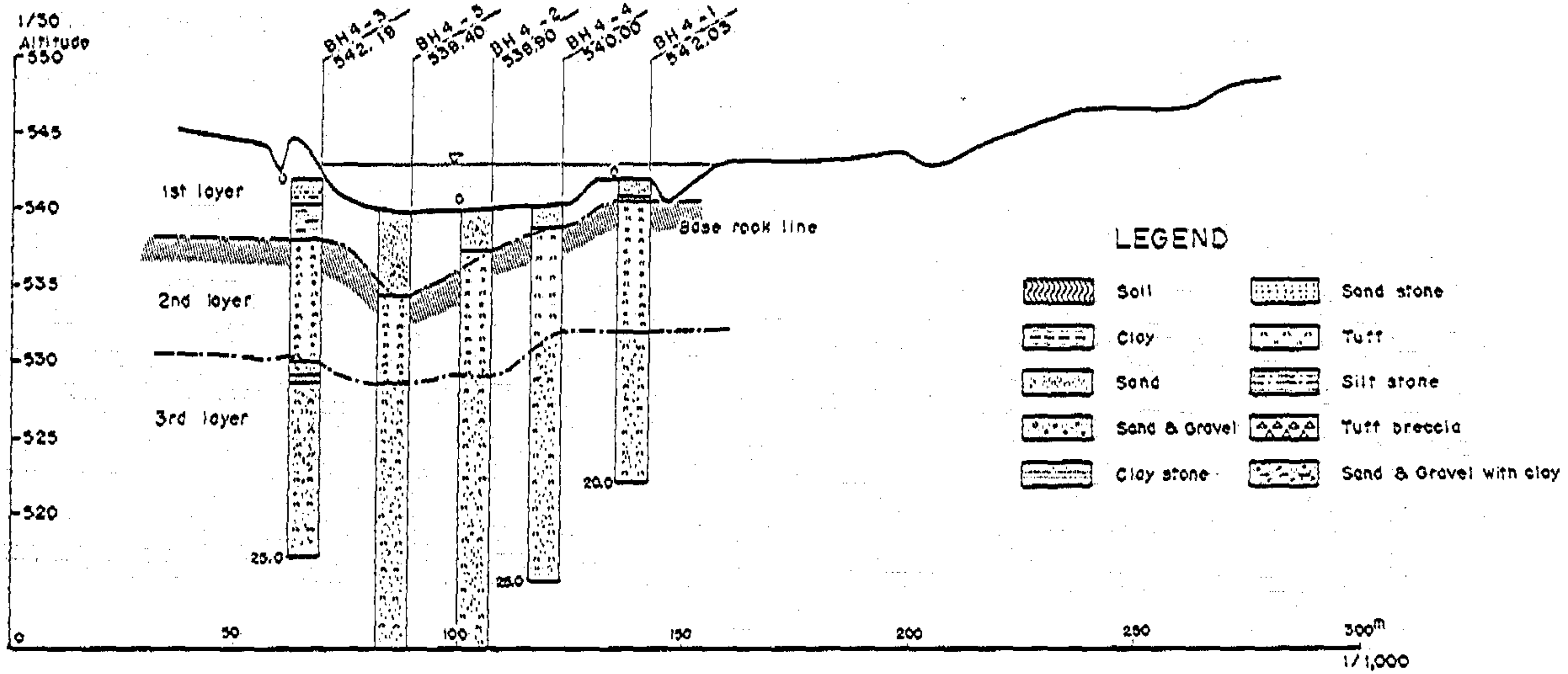


Fig. III - 12. GEOROGICAL CROSS SECTION OF RANAU REGULATING DAM



ANNEX IV

SOIL MECHANICS

ANNEX - IV

SOIL MECHANICS

1. INTRODUCTION

1.1 Purpose

The present soil survey was carried out for the following four purposes:

(1) to know the physical and mechanical properties of soil along the proposed canal alignments, particularly for the headreach and main canals,

(2) to know the foundation conditions at the proposed major structure sites such as settling basin, bifurcation structure, aqueducts, check gates and culverts, etc.,

(3) to know the depth of groundwater table which should be considered in preparing the construction plan and in estimating the construction costs, and

(4) to know the availability and suitability of construction materials such as sand, gravel and embankment materials.

1.2 Method of Investigation

The field survey and test on the soil mechanics and concrete aggregate were carried out during the period from August 22 to October 11, 1980. The item and quantity of field survey are shown in the following table.

Item	Quantity
1. Soil mechanics	
Test pit	14 sites
Auger boring	19 sites
Cone-penetration test	8 sites
Mechanical boring	7 sites
Standard penetration test	7 boreholes
Soil sampling	132 samples

to be continued

2. Concrete aggregate

Sampling

- Gravel	9 sites
- Sand	12 sites

The hole number and depth are shown in Table IV-1.

The soil samples collected from test pits and auger boreholes were sent to University of Sriwijaya for the physical and mechanical tests. These tests were carried out following the ASTM Standard. The measurement of ground water table was also carried out in each test hole.

In addition, the reconnaissance for concrete aggregate was carried out along the Komering river for the extent from the Perjaya headworks site to 35 km upstream of the headworks site and on the Ogan river around Baturaja. The number of samples collected are 6 sand-gravel mixed samples and 3 sand samples on the Komering river and 3 sand-gravel mixed samples on the Ogan river.

Table IV-2 shows the items and quantities for soil physical and mechanical tests and aggregate tests.

2. SOIL MECHANICS

2.1 General Soil Conditions along the Canal Routes

The area along the canal routes is roughly classified into two main parts: one is gentle hilly area 20 to 30 meters higher than the paddy field and the other is paddy field area.

The geology of the hilly area is of fluvial cohesive soil of the diluvial, which does not include gravel in most part. Main strata of the hilly area along the canal routes are as follows:

H-1: This stratum consists of reddish brown and cohesive soil lying in the upper part of the hilly area and its thickness is less than 3 meters. This stratum sometimes includes a small amount of gravel with thin layer.

H-2: This stratum consists of light gray soil spotted with red soil. The light gray soil is highly plastic and cohesive, and the red soil is composed of extremely weathered gravel being rich in ferric component partially reserving old structure. This stratum lies at medium depth in this hilly area and its thickness is large.

H-3: This stratum consists of fluvial volcanic ashes of pale orange color, lying mainly under the stratum H-2. The plasticity and cohesion are lower than those of the stratum H-2.

H-4: This stratum consists of light gray colored soil containing a small amount of algae, and are well consolidated. This stratum lies mainly under the stratum H-3. The plasticity and cohesion are the highest of all the strata in the hilly area.

H-5: This stratum is composed of soft and light gray colored claystone. This stratum is partially distributed along the upper reach of the Pracak headreach.

H-6: This stratum consists of sandy soil lying under the soils mentioned above.

On the other hand, the geology of the paddy field area is of stratified alluvium, which is composed of cohesive soils and sands of various textures. The horizontal distribution of clay and sand layers in this area should further be clarified through detailed survey. The strata in the area are largely categorized into two, i.e. sand strata and clay strata.

The sand strata in the paddy field area are further divided into following three classes according to grain size distribution.

P-1: This stratum is composed of poorly graded coarse sand, and its color is gray or olive brown. This stratum is distributed in the limited portion of the project area.

P-2: This stratum consists of well graded sandy soil with small quantity of silt, including humus to some extent. Its color is similar to that of the stratum P-1. This stratum is widely distributed over the paddy field area.

P-3: This stratum consists of clayey and silty sand. The stratum has various colors such as light gray, yellowish gray, brownish gray and reddish brown. This stratum is also widely distributed. The above sands contain particles of quartz, feldspar, mica, etc.

The clay strata are classified into following two classes,

P-4: This stratum comprizes inorganic clay and silt. It is of various colors, in which light gray color is predominant. This soil is solidified but degree of solidification is small and after disturbed, it becomes sticky. The plasticity is medium to high. The distribution of this stratum is dominant to the other strata in the paddy field area.

P-5: This stratum is composed of organic clay. The thickness of this stratum is mostly less than one meter. It is of the olive black color and poorly distributed.

Columnar sections at the test pits and the boreholes are shown in Fig. IV-1-(a) through Fig. IV-1-(e).

2.2 Test Results and Comments

2.2.1 Physical Properties of Soils

(1) Specific Gravity (Gs)

Mean values of specific gravity of the soils in the project area vary from 2.57 to 2.63 for cohesive soil in hilly area, 2.59 to 2.63 for sandy soil in paddy field, 2.54 for cohesive soil in paddy field and 2.48 for organic soil. Histograms of measured values are shown in Fig. IV-2.

The physical tests on inorganic soils in this area show somewhat low value of specific gravity as compared with usually observed values of 2.6 - 2.8. This reveals the fact that the compaction curve is very close to the zero-air-void curve drawn for the values of Gs mentioned above. It is, therefore, considered reasonable that actual value of Gs of cohesive soil ranges at least over 2.6.

These low values of Gs observed in the test are considered to be affected by insufficient time of boiling for driving out the entrapped air in the soil completely. Long time of boiling is especially important in the case where the soil sample is prepared in absolutely dry condition. This fact is readily recognized in the test results where the value of Gs for sand is reasonably high as compared with that for cohesive soil.

(2) Gradation

Most of the soil samples were tested for their natural gradation. For a few samples, however, gravels over 9.52 mm were removed before tests.

Mean values of percent finer than No.200 sieve (Rp) are 62 - 82% for cohesive soils in hilly area and paddy field, and Rp of the sandy soils are 10% for the strata P-1 and P-2 in paddy field and 24% for the stratum P-3. Histograms of Rp are shown in Fig. IV-3. In cohesive soils, the content of clay (under 5 μ) is higher than that of silt (5 - 74 μ).

The value of Rp could be one of the indications in judging appropriateness of soil materials for the impervious zone of embankment. In general, materials with Rp of over 20% are judged to be appropriate. On the other side, however, cracking is anticipated for materials which have extremely high value of Rp. According to USBR standards, it is noted that the upper boundary of Rp should be 50 - 80%.

Taking the above-mentioned conditions into account, soils in the stratum P-3 are considered good as an impervious material. Moreover, judging from gradation curves, soils in the strata P-1 and P-2 are good as a material of compacted soil-cement if it is planned to use these materials in the lining.

(3) Consistency

The results of Atterberg Limit tests are summarized in a plasticity chart in Fig. IV-4. Mean values of plastic index (Ip) of soils in all strata are arranged in the following order: P-3 → P-4 → (H-1, H-2, H-3) → P-5 → H-4.

Summary of Consistency

Stratum	Liquid Limit (%)			Plasticity Index		
	lowest	highest	mean	lowest	highest	mean
H-1	60	83	73	31	52	43
H-2	51	115	81	26	71	47
H-3	51	110	82	26	66	45
H-4	54	130	89	26	87	57
P-3	45	55	49	22	28	24
P-4	34	88	56	13	42	28
P-5	71	108	94	36	56	48

It is generally known that plastic soils with Ip of over 15% have high resistance against piping. This condition is sufficiently satisfied for all soils in the project area.

(4) Water Content

The test results on natural water contents of soils are summarized in the following table. Soils under the groundwater table are excluded from the tests.

Summary of Water Content (%)

Stratum	lowest	highest	mean
H-1	21.6	36.5	27.3
H-2	25.1	57.3	44.2
H-3	20.7	47.2	37.7
H-4	31.0	37.9	33.7
P-2	15.9	34.2	23.1
P-3	10.9	35.3	19.6
P-4	17.6	46.1	35.7

Soils in the stratum H-2 have somewhat high water content among other soils. Because of the fact that the natural water content of soil generally increases in proportion to the content of silt and clay, the tests on the soils in the strata P-2 and P-3 showed rather low value of water content.

The relationship between the natural water content and the optimum water content is commented later.

(5) Unit Weight

The results of the unit weight test are summarized in the following table. Tests were carried out by using the slide caliper method.

Summary of Unit Weight

Stratum	wet density γ_t (g/cm^3)			water content w (%)			dry density γ_d (g/cm^3)		
	lowest	highest	mean	lowest	highest	mean	lowest	highest	mean
H-2	1.46	1.72	1.61	51.5	58.4	55.0	0.95	1.12	1.04
H-3	1.44	1.59	1.52	39.8	42.7	41.3	1.01	1.14	1.08
H-4	1.56	1.74	1.66	32.3	43.0	37.6	1.15	1.32	1.21
H-5	1.76	2.00	1.87	35.5	50.0	42.1	1.22	1.38	1.32
P-2	1.67	1.69	1.68	19.7	21.4	20.5	1.38	1.41	1.39
P-3	1.34	1.70	1.58	26.0	38.5	35.3	1.06	1.23	1.17

Soft claystone (H-5) is sufficiently consolidated and has reasonably large value of density. The same thing is noted for solidified silty sand in the stratum P-2. The tests on the other clayey soils showed dry densities of 1.04 - 1.21 g/cm^3, which are considered reasonable.

Although soils in the strata H-2, H-3 and H-4 have similar gradation, the tests showed somewhat different value of γ_d. This reason is not clear because of lack of available data on the relationship between overburden pressure and γ_d. It is therefore recommended that the design value of γ_t be taken on the safe side in the ranges listed above. For instance, it is considered in safe side to take the stratum value of γ_t in the examination of cut slope stability in the natural ground. This is due to that the driving forces increase remarkably due to increase of γ_t, while large increase of shearing resistance is not expected for cohesive soils having low angle of internal friction.

2.2.2 Mechanical Properties of Soils

(1) Compaction Characteristics

The results of standard proctor compaction test are summarized in the following table.

Summary of Compaction Test

Stratum	/1 γdmax (g/cm^3)	/2 Wopt (%)	Wf/3 minus Wopt (%)	Wf minus W95/4 (%)	Wf minus W90/5 (%)	γdf/6 (g/cm^3)	Df/7 (%)
H-1	1.34	31.8	4.7	-1.9	-6.7	1.30	96.9
H-2	1.31	34.2	15.1	9.8	5.3	1.11	84.9
H-3	1.48	25.6	10.4	5.2	1.1	1.31	88.6
H-4	1.40	29.0	13.6	8.0	3.6	1.20	85.7

Remarks, /1; maximum dry density

/2; optimum moisture

/3; field moisture

/4; highest water content to obtain 95 percents of γd max

/5; highest water content to obtain 90 percents of γd max

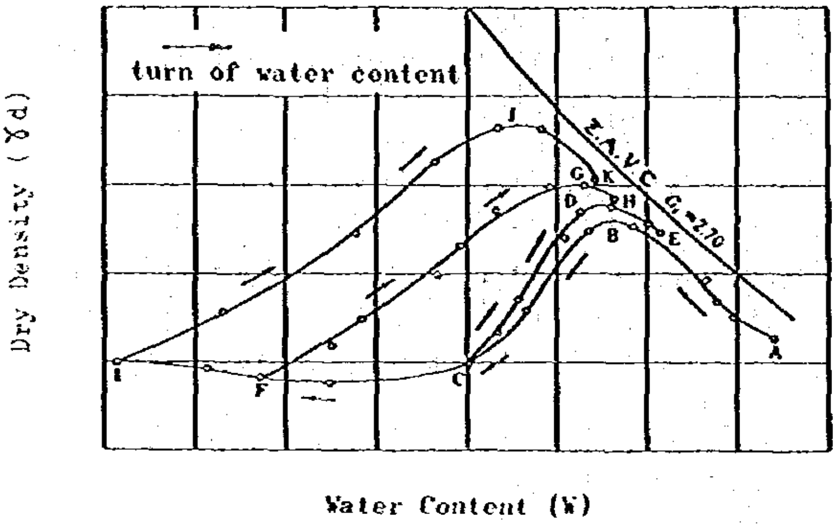
/6; dry density at field moisture on compaction curve

/7; D-value for γdf
i.e., (γdf/ γd max) x 100%

Considerably large difference is observed between Wf and the optimum moisture Wopt for all soils except the stratum H-1. The same results are observed in the most parts due to the fact that compaction tests are carried out following the dry-method/1 in Indonesia.

It is generally known for fine soils that compaction curve obtained by means of the dry method changes due to the change of the initial water content of soils in the stage of sample preparation as shown in the following figure. If the samples are prepared in more dry condition, the maximum dry density becomes larger and Wopt becomes smaller. This tendency is remarkably seen in volcanic ash.

/1; Dry-method
In the dry method, soil samples are initially prepared in air-dried condition. Water is then added in steps to make samples having different water content.



The abovementioned tendency can be explained by the facts that compaction energy can be transmitted more easily to the soil in the mold whose particles are congealed in larger size such as gravel, and that absorbed water around soil particles is removed in drying stage, but can not be recovered even by adding water in compaction tests.

In the present compaction tests, soil samples were dried in the preparation stage to have water contents about 23% lower than Wf for soils in the stratum H-1 and about 30% lower than Wf for other soils. Since such a large extent of drying cannot be considered in actual construction, γd max and Wopt should be determined by means of the non-dry method, in which the samples are dried gradually in steps from Wf. The values of Wopt to be obtained by the non-dry method can be estimated for soils in the strate H-2, H-3 and H-4 to be about 5% lower than those listed above, and for the soils in the stratum H-1 to be similar value to the present results.

From the viewpoint of compaction control, high standard value of compacted dry density is desirable, and it is also desirable for soils in the project area to decrease water content in advance up to the value near Wopt to seek high resistance against sliding, settlement and cracking for impervious embankment.

On the other hand, Wf of all the soils in hilly area are higher than Wopt. Wf of the soil in the stratum H-1 is nearly W97[1], and Wf of the soil in the stratum H-2 is slightly higher than W90. Those of H-3 and H-4 are between W90 and W95. However, the differences between Wf and W95 are so small as less than 5 percent. This indicates that Wf is easily reduced to W95 in the field work by air drying.

Based on the test results and its consideration, and by taking construction conditions into account, it is recommended that standard D-value[2] of field compaction should be about 95% for these materials.

From the abovementioned compaction tests, it may be concluded that the soils in the hilly area can be compacted well in the order of H-1, H-4, H-3 and H-2.

Compaction tests were not carried out on the samples from paddy field. Judging from physical properties of soils, however, it is concluded that the material in the stratum P-4 has similar compaction characteristics to the material in the stratum H-4, because they showed similar values in water content, specific gravity, consistency and content of fine soil.

(2) Unconfined Compression

Unconfined compression tests were performed on both undisturbed and remolded samples.

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[1] higher side of water content in the compaction curve to obtain the dry density of 97% of γd max.

[2] D - value = $\gamma d / \gamma d$ max x 100 (%)
For example, D95 means D - value = 95%

(a) Undisturbed samples

Test results are summarized in the following table.

Summary of Unconfined Compression Test with Undisturbed Specimens

Stratum	Sample Name	Condition of Specimens γd (g/cm^3)	 Wf (%)	Unconfined Compression Strength qu (kg/cm^2)	Strain at failure $\varepsilon 50$ (%)	Deformation Coefficient E50(kg/cm^2)
H-2	80TP-3 Z=5.0m	1.03	52.6	0.88	3.3	57.8
	80TP-9 Z=4.5m	1.05	57.3	1.48	5.1	62.8
H-3	80TP-4 Z=5.0m	1.08	41.3	0.62	2.9	34.4
H-4	80TP-10 Z=2.0m	1.21	37.6	0.62	2.0	42.3
H-5	80TP-1 Z=2.0m	1.32	42.1	4.13	2.3	410.0
P-2	80AB16(b) Z=1.0m	1.39	20.5	0.84	2.7	58.3
P-3	80TP-2 Z=2.0m	1.19	38.4	0.39	4.7	11.4

Soils in the strata P-2 and P-3 are classified in sand (SW - SC). Although value of unconfined compression strength (qu) is not so high, sufficient shearing resistance as a base foundation is expected for these soils due to the effect of angle of internal friction.

High value of qu of materials in the stratum H-5 reveals the fact that this stratum consists of solidified silt or claystone consolidated under considerably high overburden pressure (Po)/1. This

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/1; In Skempton's equation:
Cu/Po = 0.11 + 0.0037 · Ip, Cu = qu/2 = 2.07, and Ip = 28 for this soil. This yields Po ≑ 10 kg/cm^2, being equivalent to about 50 meters of overburden.

stratum, however, is not so hard and can easily be excavated by use of small backhoe shovel.

The values of qu for the materials in the strata H-2, H-3 and H-4 vary in the range of 0.62 - 1.48 kg/cm^2. These are considered reasonable comparing with the results of triaxial compression tests on similar samples.

For saturated clayey soils, strength parameters are determined from the unconfined compression tests as the internal friction angle (ϕu) = 0 and the cohesion (Cu) = qu/2. For the materials tested, however, $\phi u \neq 0$ and $Cu < qu/2$, because they are unsaturated soils. It is therefore recommended that the design parameters for undisturbed condition be determined on the basis of all data including results of triaxial compression tests.

(b) Remolded Samples

The results of unconfined compression tests on the samples compacted at D95 and D90 are summarized in Table IV-3. There is a little difference in qu-value due to the difference in soil types from the strata H-1 and H-4.

All samples were compacted at field moisture of Wf. The relation between qu and water content is plotted in Fig. IV-5.

Although triaxial compression tests are needed to be conducted to determine the design strength of fill materials, some comments can be drawn from the shear strength of materials tested by taking its qu-value into account: i.e. Cu can be estimated from qu. assuming Cu = qu/2.

The values of Cu measured for the samples compacted at D-95 range from 0.22 to 1.44 kg/cm^2, and those for the samples compacted at D-90 are in the range from 0.21 to 1.14 kg/cm^2. These values of Cu at D-95 are not so small for cohesive soils, and the materials tested are therefore considered appropriate for the use as fill materials with respect to shearing resistance.

(3) Triaxial Compression

Triaxial tests under unconsolidated-undrained condition (U-U test) were performed on undisturbed samples which have similar natural conditions to those used in unconfined compression tests. Mohr's stress circles at failure and strength envelopes are shown in Fig. IV-6. The strength parameters (Cu, ǿu) thus obtained are:

for the stratum H-2; Cu = 0.25 kg/cm^2, ǿu = 26^{o}-30'
for the stratum H-4; Cu = 0.35 kg/cm^2, ǿu = 24^{o}-00'

It is pointed out that the measured values of Cu are somewhat small and those of ǿu are relatively large as compared with the parameter values obtained for usual cohesive soils.

There is a little difference in shear strength depending on the soil types. As mentioned before, the mean value of qu for the materials in the strata H-2, H-3, H-4 and H-5 is about 0.9 kg/cm^2. The stress circles at failure obtained in the unconfined compression tests are compared with the strength envelopes ditermined from the triaxial compression tests as shown in Fig. IV-7. The tests show a good correspondence in low stress range of the normal stress (σ). The mean value of triaxial test results; Cu = 0.3 kg/cm^2, is therefore proposed as the design value of cohesion. Since there is no reliable data on the measured values of ǿu, it is desirable to use the design value of ǿu on the safe side. Considering various situations, the design value of ǿu is taken as ǿu = 12^{o} which is 50 percent of the mean value of triaxial test results.

2.3 Analysis

2.3.1 Classification

Based upon the results of the grain size analysis and of the consistency test, the soils distributed in the project area can be classified as follows according to the Unified Classification Method. The method is shown in Table IV-4.

(1) Clay and Silt in hilly area and Paddy field

Stratum H-1;	Major parts of this stratum are classified into "CH" soil and only a small parts are classified into "GC".
Stratum H-2;	Two thirds of this stratum are classified into "CH" soil and the rests are "MH".
Stratum H-3;	Soil in this stratum is classified as "MH" according to the ASTM Designation but additional classification will be required for this kind of volcanic soil and it is additionally classified as "VH".
Stratum H-4;	Major parts of this stratum are composed of "CH" soil and partly "MH".
Stratum H-5;	Soil of this stratum is classified into "MH".
Stratum P-4;	Around 70% of this stratum is classified into "CH" to "MH" soil and the rest is "CL" to "ML" soil.
Stratum P-5;	This stratum is classified into "OH" soil.

(2) Sand in Paddy Field

Since it is difficult to classify the sand in paddy field according to its distributed depth, location and its color, unified classification by gradation is adopted to distinguish the stratum as follows.

By Gradation	Name of Stratum
SP	P - 1
SW	P - 2
SM or SC	P - 3

This classification of soils is related to the relative desirability for the canal embankment material following the standard of USBR as shown in Table IV-5. From this table, it is judged that most of the soils distributed in the project area are usable as the embankment materials for the canal construction.

2.3.2 Foundation along the Canal Routes

Major analyses on the foundation along the canal routes are as follows:

i) Allowable bearing capacity for the related structures.

ii) Allowable bearing capacity for the canal embankment.

iii) Permeability of foundation.

Explanation on second and third items are given later in this report.

Allowable bearing capacity of the foundation is defined as the capability of the foundation in terms of strength and deformation, and it is expressed by the ultimate bearing capacity divided by safety factor.

The ultimate bearing capacity for the structure is related to the foundation shape of structure and to the soil shear strength expressed by angle of internal friction (ϕu) and cohesion (Cu). The shear strength parameters ϕu and Cu will be defined by the soil mechanical test and will be estimated from the soil sounding also.

(1) Soil Sounding

Soil sounding investigation was conducted using standard penetrometer and double-tube type cone-penetrometer. The results are summarized in Fig. IV-8 - Fig. IV-9. It is shown in these figures that the cone-index (qc) and N-values of the soils in the project area do not increase according to the soil depth.

The test results are also summarized in the following table according to the said soil classification.

Stratum	qc (kg/cm^2)	N-value
H-1, (clay)	12 - 14	15 - 20
H-2, (")	12 - 15 over	5 - 31
H-3, (volcanic ash)	10 - 15 over	14 - 19
H-4, (clay)	13 - 15 over	nil
P-1, P-2 and P-3 (sand)	15 over	13 - 50
P-4, (clay)	9 - 15 over	4 - 31
P-5, (organic clay)	nil	4 - 12

The value of qc are 9 - 15 over for clay and 15 over for sand but N-values do not show the clear difference between clay and sand.

Following relations are generally known between ϕu, Cu and qc, N-value.

- For clayey soil ($\phi u = 0$)

$$Cu \fallingdotseq qc/20 \quad (\because qu \fallingdotseq qc/10, \quad qu = 2 \cdot Cu)$$

$$Cu \fallingdotseq N/16 \quad (\because qu \fallingdotseq N/8, \quad qu = 2 \cdot Cu)$$

(unit of qu, qc, Cu; kg/cm^2)

- For sandy soil

$$\phi u = \sqrt{12 \cdot N} + 20$$

(unit of ϕu; degree)

Using these relations, the figures in the above table will be converted as follows:

	by qc	by N-value	
Stratum	Cu (kg/cm^2)	Cu (kg/cm^2)	ϕu (degree)
H-1	0.60 - 0.70	0.94 - 1.25	-
H-2	0.60 - 0.75 over	0.31 - 1.94	-
H-3	0.50 - 0.75 over	0.88 - 1.19	-
H-4	0.65 - 0.75 over	nil	-
P-1,P-2,P-3	-	-	32 - 44
P-4	0.45 - 0.75 over	0.25 - 1.94	-
P-5	nil	0.25 - 0.75	-

From this Table, Cu value will be in the range between 0.25 - 1.94 kg/cm^2 for the strata H-1, H-2, H-3, H-4 and P-4 and ϕu value for sand will be 32 - 44 degree.

(2) Shear strength

Based on the above analyses and the results of unconfined compression tests and triaxial tests, the values of Cu and ϕu for canal foundation design are estimated as follows:

Clay Cu = 0.3 kg/cm^2, ϕu = 12°-00'
Sand Cu = 0 , ϕu = 30°-00'

These values only give average characteristics of the soils in the project area. Therefore, it should be necessary to conduct further soil mechanic tests for the design of each major canal structures.

(3) Ultimate Bearing Capacity

Shear failure of the foundation is classified into two types: one is general shear failure and the other is local shear failure. The local shear failure is such failure that the failure point on the load increment is not clearly defined and it occurs often in clayey soil. The clayey soils are distributed in the major parts of the project area. The Terzaghi formula is adopted to obtain the ultimate bearing capacity of clayey soil.

$$qf = \frac{2}{3} \alpha \cdot Cu \cdot Nc + \gamma_1 \cdot Df \cdot Nq + \beta \cdot \gamma_2 \cdot Nr$$

where,

qf: Ultimate bearing capacity.

Cu: Average cohesion of the soil in the foundation layer.

γ_1: Unit weight above the bottom level of foundation structure.

γ_2: Unit weight under the bottom level of foundation structure.

Df: Depth of overburden (from ground surface to the bottom level of foundation structure).

Nc,Nq,$N\gamma$: Bearing capacity factor (refer to Table. IV-6)

α,β : Shape factor (refer to Table IV-7)

For the unit weight of layer, "γsub" below groundwater surface or seepage surface and "γt" above them should be utilized.

Foundation depth from the ground surface should be more than 2 times of the width of structure foundation in adopting the above Terzaghi formula.

The calculation of ultimate bearing capacity against local failure about clayey soil foundation is made for the condition that the basement of structure is of square shape. Result of the calculation is shown in the following table.

Ultimate Bearing Capacity (t/m^2)

	Df (m)		
B (m)	1	2	3
4	27.5	31.3	35.0
8	28.8	32.6	36.3
12	30.2	33.8	37.6

$Nc = 8.68$, $Nq = 2.24$, $Nr = 0.48$

$\gamma_1 = \gamma_2 = \gamma_t = 1.67$, $Cu = 3.0$ t/m^2, $\phi u = 12^\circ$

2.3.3 Embankment Material for Canal

(1) Excavated Material

Excavated soils in canal construction should fully be utilized as the embankment material, if the soil is suitable for embankment. Since it is considered that the soils produced from the canal excavation will be of all types of the soils distributed in the project area, the study on the characteristics and adoptability of the soils and the estimate of the available volume are made for all types of the soils as follows:

(a) Clay and Sand in Paddy Field

These are alluvial deposits in the paddy field area and distributed on irregular alternate layer. These soils are classified as the strata of P-1, P-2, P-3, P-4 and P-5. The thickness of the layer varies from 0.2 to 5.0 meters but mostly from 0.5 to 1.5 meters.

Color of clay layer is light gray, gray, brown, dark olive and olive black but majority of clay has light gray to gray color. Among them, olive black to black colored clay contains humus and is unsuitable as embankment material. Other colored clays have similar characteristics to the abovementioned clay, but can be utilized as embankment material. These clays are of fine graded texture of silt and clay, having high plasticity, large dry strength, and large dry shrinkage.

The strength of compacted clay would be about $Cu = 4\ t/m^2$ according to the unconfined compression test. Permeability will be sufficiently low without treatment. As the clay layer below groundwater level has already been consolidated, the water content will not be much different between the clay layers above and below groundwater level. For the excavation work, heavy machine will be workable under the dry work condition. This can also be instified by the cone-index value (qc) of more than 9.

The soils in the sand strata of P-1, P-2 and P-3 are categorized in "SC", "SM", "SW" and "SP" according to their gradations. The soils categorized in "SC" can be utilized for the canal embankment, but the soils categorized in "SP" are not suitable for the embankment due to the fearfulness of piping action in the embankment. On the other hand, the soils categorized in the "SM" and "SW" can not be utilized for the canal embankment without protection of the embankment surface against the canal erosion and seepage. For the use of this material to the canal embankment, it is necessary to line the wetted perimeter of canal with clayey materials or other impervious materials.

(b) Clay in Hilly Area

Clay in the hilly area is grouped in four strata; H-1, H-2, H-3 and H-4. The thickness of these strata varies from 0.5 to 7 meters and mostly from 2 to 3 meters. The mechanical characteristics of these clays except H-3 are similar to that of the clay in the stratum P-4. But clay in the stratum H-3 is porous volcanic soil of semi-impervious and comparatively compressive material. With the available data, it can be concluded that the soils in the strata H-1, H-2 and H-4 are suitable: the soil in H-1 is the most for the embankment material, but the soil in H-3 is unsuitable because of large settlement.

(c) Ratio of Excavated Soil Usable to the Canal Embankment

Canal routes are divided into 8 reaches, namely from Reach-A to Reach-H, in order to estimate the ratio of excavated soil usable to the canal embankment.

Reach-A;	From the Pracak intake to IP.8 of the Pracak headreach. For most part of this reach, canal has cut section except some flat portion. Borrow pit can be found in hilly area along the headreach. The flat land area from the intake down to 4 km is covered with river bed material and excavated material is mostly sand.
Reach-B;	The headreach portion from IP.8 to the bifurcation structure. Canal section will have cut & bank portions. Excavated volume may be short for embankment, but borrow pit can be found along the headreach.

Reach-C; North Main Canal from BP. to IP.4.
The canal route passes through flat paddy field and embankment volume will exceed the excavated volume. The borrow pit for this reach can be found along the canal but drainage problem will be encountered in case the embankment materials are taken from the flat paddy field adjacent to the canal. Therefore, careful study is required for this matter.

Reach-D; North Main Canal from IP.4 to EP.
The canal route passes through the foot of hilly area. The excavated materials will be enough for the canal embankment.

Reach-E; South Main Canal from BP. to IP.2.
Similar to Reach-C, it is difficult to obtain sufficient volume for embankment material along the canal route because of drainage problems in the paddy field.

Reach-F; South Main Canal from IP.2 to IP.5.
Borrow pits can be found at the left side of the Pisang river and the embankment material can be supplied easily.

Reach-G; South Main Canal from IP.5 to EP.
Similar to Reach-D, the embankment materials can easily be obtained without much difficulty.

Reach-H; Pisang Main Canal.
Embankment material will be available along the canal.

The estimated result for the available quantity of the excavated soil for the use of embankment is summarized in the following table.

Ratio of Available Excavated Soil for Impervious Embankment Material

Division	Reach	Excavated Depth from Ground Surface		
		3 meters	5 meters	10 meters
Headreach	A	70	55	45
	B	75	55	-
North Main Canal	C	50	45	45
	D	65	60	65
South Main Canal	E	65	55	-
	F	70	60	-
	G	70	70	75
Pisang Main Canal	H	90	90	-

(Unit: Percentage)

Remarks; " - " indicates that the available data are not sufficient to analize the ratio.

2.3.4 Stability of Canal Side Slope

The stability analysis for canal side slope is made using the Taylar's Method for which the slope stability chart is shown in Fig. IV-10. This analysis is made for both the embankment and cut slopes.

(1) Stability of Embankment Slope

Soil mechanical coefficients of embankment materials in the project area are estimated as follows:

Unit weight of soil; Based on the results of compaction tests, the dry density of 95% of γd_{max} is taken as a standard.

$\gamma d = 1.10\ t/m^3$

$w = 45\ \%$

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

Shear strength parameters;

Since triaxial compression test on disturbed sample is not conducted, the results of unconfined compression test are used for analysis of shear strength. Taking angle of internal friction (ϕu) to be zero, cohesion (Cu) of $3\ t/m^2$ is obtained

The following table shows the result of stability analysis of embankment immediately after the construction. Safety factor is taken to be 1.5 in the analysis.

Required Angle of Embankment Slope

height of embankment (m)		2	4	6	8
angle of embankment slope	β (degree)	90	90	66	41
	m (1/tan β)	vertical		0.44	1.15

(2) Stability of Cut Slope

It is better to analize separately the stability of cut slope of the canals to be constructed in the hilly area and the paddy field area, because the unsolidified sand layer with high ground water level is expected in the paddy field but mostly clayey soil in hilly area.

The cut slope of the canal to be constructed in the hilly area is analized also using the Taylor's method. The soil mechanical coefficients used in the analysis are as follows:

Unit weight of soil: $\gamma t = 1.67\ t/m^3$

(Based on the unit weight test) $w = 45\ \%$

$\gamma d = 1.15\ t/m^3$

Shear strength parameters: (based on the analysis in the subsection 2-3-2)

$Cu = 3\ t/m^2$

$\phi u = 12^{o}\text{-}00'$

The results of analysis are summarized in the following table.

Required Angle of Cut Slope
for Safety Factor of 1.5

Cutting height (m)		2	4	6	8	10
Angle of cut slope	β (degree)	90	90	77	54	38
	m (1/tanβ)	vertical		0.23	0.73	1.28

2.3.5 Seepage

(1) Characteristics of Seepage

Seepage will occur through the wetted perimeter of canal. The magnitude of seepage loss will depend on the depth of canal water and groundwater table.

Fig. IV-11 shows the typical patterns of canal seepage. The case-(a) will occur under the condition that the permeability coefficient of embankment (kb) is less than that of canal bottom (kf). The case-(b) shows the excavated canal on two soil layers of which the upper layer has smaller permeability than that of lower layer. The case-(c) shows the excavated canal on two soil layers of which the upper layer has larger permeability than that of lower layer.

If the constant water level is maintained in the canal, the seepage rate will not be affected by the fluctuation of groundwater level under the condition of $hw < Hb$ (case-(a)) and the condition of $hw > Hb$(case-(b)). Other than these, the seepage rate will be changed according to the fluctuation of groundwater table. The lower groundwater table will cause more canal seepage.

(2) Coefficient of Permeability

The coefficient of permeability of embankment depends on void ratio (density) and saturation ratio of soil after the compaction. The difference of coefficient between the maximum and minimum is sometimes 100 times even for the same embankment material.

The minimum coefficient of permeability can be obtained at wetter condition than the optimum moisture content under the same compaction energy. In case of clayey soils, they are damp and have a high plasticity, and it will be relatively easy to attain the coefficient of less than $kv = 5 \times 10^{-6}$ cm/sec.

The coefficient of permeability will be estimated as shown in the following table.

Soil Material	Coefficient of Permeability (Kf)	Data Used for the Estimation of Coefficient
Clay in hilly area	1×10^{-5} cm/sec.	groundwater in test pit, water content and density of soil
Clay in paddy field	1×10^{-6} cm/sec.	grain size of soil, consolidation level
Sand in SW strata	5×10^{-3} cm/sec.	grain size analysis shown in Table IV-8 and IV-9

(3) Recommended Embankment Section

Based on the abovementioned study, the recommendations on the design of embankment are made as follows:

In case the canal bottom is permeable and seepage loss is estimated to be more than allowable amount, earth lining or cut-off trench should be provided as shown Fig. IV-12-(a) and (b).

In case the amount of seepage through canal bottom is allowable or the canal bottom is improved against leakage, it is recommended to design the canal bank as shown in Fig. IV-12-(c) and (d). The outer surface of embankment should be protected by sod facing against the slope failure as shown in Fig. IV-12-(c) and (d).

2.3.6 Groundwater Surface

The depth of groundwater table in each test hole in the dry season is shown in Table IV-10. Using these data, the iso-depth contour map is made along the canal routes as shown in PLATE No.4. During the course of survey, confined groundwater was found at only one site in the upper stream area of South Main Canal. This matter shows that permeability of clay in paddy field area is very low.

3. CONCRETE AGGREGATE

3.1 General

The survey on concrete aggregates was made along the rivers in and around the project area. Relatively large rivers such as the Pisang and Belitang and several creeks run through the project area. However, these river beds are mostly covered with clay, silt and fine sand, and it will be difficult to obtain the aggregate materials from these rivers. On the other hand, the survey revealed that aggregates could be obtained from the Komering and Ogan rivers at the places as shown in PLATE No.5. This plate also shows the sampling sites of materials.

3.2 *Results of Aggregate Test*

Fig. IV-13 shows the results of sieve tests for 9 gravel samples. These results were used to decide the ratio of available gravel material for concrete coarse aggregates (10 mm up).

Table IV-11 shows the results of specific gravity test for gravel and sand, and of absorption test for gravel. These results are compared with the standard values of ASTM, i.e. absorption value of less than 3% and specific gravity of 2.5. As the results, it may be concluded that the gravel collected along the upstream of the Komering from Martapura is suitable as the concrete aggregates. On the other hand, the gravel collected in the downstream of Martapura does not fully satisfy the standard. Before getting the final conclusion, however, the adoptability of gravel as the concrete aggregate should be checked by compression test.

The results of sieve test on the sand collected both in the Komering and the Ogan rivers as well as the standard range for fine aggregate are shown in Fig. IV-14. This Fig. shows that the specific gravity of all the sand tested is reasonably high ranging from 2.64 to 2.79. As for the distribution of grain size of the sand, all the gradation curves are within the standard range up to 80% and the curves above 80% are closed to the boundary. Accordingly, it is concluded that most sand materials can be used. As the fine aggregate after washing out silt contained in sands.

3.3 Analysis

3.3.1 Availability of Concrete Aggregate

Fine aggregate will be available along every reach of the Komering river and its volume will be enough for the construction use.

Gravels are available at the curves and shoals of the Komering river and particularly, the high gravel contents are found at the upstream of the Komering river as shown in Fig. IV-13.

The maximum gravel diameter is about 60 mm in the upstream of the Komering river and 100 mm in the Ogan river. Since the shape of gravel is round for the Komering river and sub-angular for the Ogan river, the workability of concrete will be better for the gravel from the Komering river.

The available amount of gravel material is roughly estimated based on the reconnaissance. The estimated amount is as shown in Table IV-12

3.3.2 Recommended Quarry Sites of Gravel

The proposed quarry sites are broadly grouped into three. The first group including Block-A through E in PLATE. No.5 has sufficient amount of gravel and the shortest transportation distance to the project site; around 10 km. The second group is located in the downstream of the Komering river from Martapura, i.e. Block-F. This block will not cover the required quantity of the material and the quality of gravel is inferior to that in the first group. The third one, Block-G and H, is located near Baturaja on the Ogan river. This group has no problem on the quantity and quality but the transportation distance to the project site is very long; more than 50 km. Considering the above, the first group, i.e. the upstream of the Komering river from Martapura is recommended as the quarry site of the gravel to be used for concrete work.

Table IV-1 SURVEY HOLES AND EACH DEPTH

	Hole NO.	Depth of holes(m)	Ground Level (EL.m)	Location
Headreach	BH 3-1	25.0	90	IP.1-220^{m}
	80TP-1	2.5	89.5	NO.11-10^{m}
	BH 3-2	25.0	102.5	IP.2+30^{m}
	BH 3-3	20.0	85.1	NO.23
	80TP-2	5.0	86.5	NO.30-20^{m}
	BH 3-4	20.0	95.0	NO.43
	BH 3-5	25.0	88.1	NO.51
	80TP-3	10.0	98	IP.5-20^{m}
	80TP-4	10.0	93	NO.76
	BH 3-9	15.0	84.0	IP.7
	80AB-12	3.3	83.6	NO.111+70^{m}
	80AB-13	2.4	82.0	NO.123
	80AB-11	5.0	78.9	IP.8
	80AB-10	4.3	78.3	IP.9
	80AB-9	5.0	73.7	EP. (Bifuracation)
North Main Canal	80TP-14	10.0	74	NO.53+50^{m}
	80AB-5	3.2	73	IP.1
	BH 3-10	25.0	72.2	IP.2-460^{m}
	80TP-5	2.0	88.5	IP.4-850^{m}
	80TP-12	4.0	70	IP.6+700^{m}
North Main Canal	80TP-13	4.5	70	IP.9-500^{m}
	80TP-6	5.0	65	IP.12+1500^{m}
	80AB-19	3.5	70	IP.26+400^{m}
	80TP-7	8.0	60	IP.28+500^{m}
South Main Canal	80AB-16	3.8	66.5	IP.1+500^{m}
	80AB-15	5.0	66.0	IP.1+1680^{m}
	80AB-14	5.0	63.5	NO.125
	80AB-1	5.0	63.0	NO.158(Turnout to Pisang Area)
	80AB-2	3.3	60	IP.5
	80TP-9	10.0	56	IP.10+500^{m}
	80TP-11	7.0	60	IP.15+500^{m}
	80TP-8	2.0	55	IP.20
	80AB-4	5.0	55	IP.20+30^{m}
	80AB-18	5.0	55	IP.25+500^{m}
	80AB-17	5.0	48	IP.35+2000^{m}
Pisang Main Canal	80TP-10	3.0	68	IP.3
South Main Canal (Secondary plan)	80AB-6	5.0	74	IP.1'-720^{m}
	80AB-7	1.5	71.8	IP.2'
	80AB-8	5.0	69.4	IP.3'
	80AB-3	5.0	66	IP.6'-50^{m}

Remarks

(1) '80TP','80AB'and'BH'indicate test pit, auger borehole and mecanical borehole respectively.

(2) Total number in this table increases from upstream to downstream each canal.

(3) Auger boring was used to excavate the lower section deeper than 5 meters in test pit.

Table IV-2 <u>LABORATORY TEST ITEM AND ITS QUANTITY</u>

		Test Item	Quantity
SOIL TEST	Physical Test	Water content	107 nos
		Specific gravity	132 nos
		Gradation analysis (including sedimentation analysis for 50 samples)	132 nos
		Liquid limit	94 nos
		Plastic limit	94 nos
		Shrinkage limit	20 nos
		Unit weight	7 samples
	Mechanical Test	Compaction test	8 samples
		Unconfined compression test	
		- Undisturbed sample	26 nos
		- Remolded sample	12 nos
		Triaxial compression test under U-U condition (with undisturbed sample)	2 sets
		Permeability test (with remolded sample)	2 nos
AGGREGATE TEST		Specific gravity	
		- Gravel	9 nos
		- Sand	12 nos
		Sieve analysis	
		- Gravel	9 nos
		- Sand	12 nos
		Absorption	
		- Gravel	9 nos

Table IV-3 RESULTS OF UNCONFINED COMPACTION TEST WITH REMOLDED SPECIMENS

Sample Name	Condition of Specimen			U.C. Strength	Strain at failure	Diformation Coefficient	Remarks
	D-value (%) /1	W_f /2 (%)	γd (g/cm^3)	qu(kg/cm^2)	εf (%)	E_{50}(kg/cm^2)	
80TP-3	95	47.2	1.23	2.88	5.0	144	CH, (H-2)
Z=4.5 m	90	"	1.16	2.28	3.0	163	
80TP-4	95	35.4	1.47	1.02	9.0	21.3	CH, (H-2)
Z=3.0 m	90	"	1.39	0.93	8.0	23.3	
80TP-9	95	51.3	1.27	0.43	8.0	6.7	CH, (H-2)
Z=5.0 m	90	"	1.20	0.42	11.0	5.3	
80TP-11	95	36.5	1.33	1.23	4.0	39.7	VH, (H-3)
Z=3.0 m	90	"	1.26	0.90	3.0	40.9	
80TP-13	95	31.8	1.33	1.27	4.0	39.7	MH-CH,(H-4)
Z=3.0 m	90	"	1.26	0.97	3.0	38.8	
80TP-14	95	36.5	1.27	1.10	3.0	91.7	CH, (H-1)
Z=2.0 m	90	"	1.21	0.84	2.0	84.0	

/1, D-value = ($\gamma d / \gamma d_{max}$)· x 100%

/2, W_f = field moisture

Condition of Specimen is explained in the following figure.

Compaction Curve

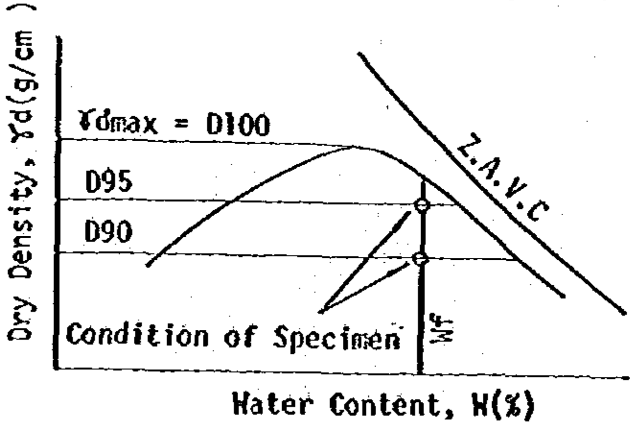
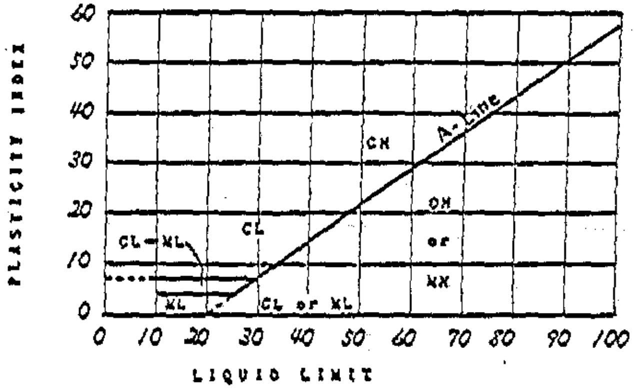


Table IV-4 UNFINED SOIL CLASSIFICATION

PRINCIPAL DIVISION			GROUP SYMBOLS	TYPICAL NAMES	CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS More than half of material is larger than No200 sieve size. ※1	GRAVELS More than half of coarse fraction is larger than No4 sieve size	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels,gravel—sand mixture,little or no fines.	Determine percentage of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No200 sieve size) coarse grained soils are classified as follows. Less than 5%—GW,GP,SW,SP More than12%—GM,GC,SM,SC 5%to 12% —Borderline cases requiring use of dual symbols.	Cu greater than 4 ※3 Cc between one and 3 ※4	
			GP	Poorly graded gravels,gravel—sand mixture,little or no fines.		Not meeting all gradation requirements for GW	
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels,poorly graded gravel—sand—silt mixtures.		Atterberg limits below 'A' line,or PI less than4	Above 'A' line with PI between4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels,poorly graded gravel—sand—silt mixtures.		Atterberg limits above A line with PI greater than7	
	SANDS More than half of coarse fraction is smaller than No4 sieve size	CLEAN SANDS (Little or no fines)	SW	Well graded sands,gravelly sands, little or no fines.		Cu greater than6 ※3 Cc between one and 3 ※4	
			SP	Poorly graded sands,gravelly sands, little or no fines.		Not meeting all gradation requirement for SW	
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, poorly graded sand—silt mixtures.		Atterberg limits below 'A' line or PI less than4	Above 'A' line with PI between4 and 7 are borderline cases requiring use of dualsymbols.
			SC	Clayey sands, poorlly graded sand—clay mixtures.		Atterberg limits above 'A' line with PI greater than7	
FINE GRAINED SOILS More than half of material is smaller than No200 sieve size.	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands rock flour,silty or clayey fine sands with slight plasticity.			
			CL	Inorganic clays of low to medium plasticity,gravelly clays,sandy clays,silty clays,lean clays.			
			OL	Organic silts and organic silt—clays of low plasticity.			
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts,micaceous or diatomaceous fine sandy or silty soils,elastic silts			
			CH	Inorganic clays of high plasticity, fat clays.			
			OH	Organic clays of medium to high plasticity.			
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.			
VOLCANIC COHESIVE SOILS ※2			VH	Volcanic ashes.			



PLASTICITY CHART

by ASTM Designation : D2487—69

※1 All sieve sizes on this chart are U.S. standard.

※2 Addition division of Japanese soil classification system.

※3 $Cu=D_{60}/D_{10}$ coefficient of uniformity.

※4 $Cc=(D_{30})^2/(D_{60} \times D_{10})$: coefficient of curvature.

Table IV-5 SOIL TYPE - MECHANICAL PROPERTIES (1)

(a) Important Mechanical Properties

Group Symbols	Permeability when compacted	Shear strength when compacted and saturated	Compressibility when compacted and saturated	Workability as a construction material
GW	Pervious	Excellent	Negligible	Excellent
GP	Very pervious	Good	Negligible	Good
GM	Semi-pervious to impervious	Good	Negligible	Good
GC	Impervious	Good to fair	Very low	Good
SW	Pervious	Excellent	Negligible	Excellent
SP	Pervious	Good	Very low	Fair
SM	Semi-pervious to impervious	Good	Low	Fair

(to be continued) .....

Table IV-7 SOIL TYPE - MECHANICAL PROPERTIES (2)

Group Symbols	Permeability when compacted	Shear strength when compacted and saturated	Compressibility when compacted and saturated	Workability as a construction material
SC	Impervious	Good to fair	Low	Good
ML	Semi-pervious to impervious	Fair	Medium	Fair
CL	Impervious	Fair	Medium	Good to fair
OL	Semi-pervious to impervious	Poor	Medium	Fair
MH	Semi-pervious to impervious	Fair to poor	High	Poor
CH	Impervious	Poor	High	Poor
OH	Impervious	Poor	High	Poor
VH	Semi-impervious	Poor	High	Poor

Table IV-5 SOIL TYPE-MECHANICAL PROPERTIES (3)

(b) Relative Desirability for Various Uses (No.1 is considered the best)

Group Symbols	Rolled Earthfill Dams			Canal Sections		Foundations		Roadways	
	Homogeneous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Seepage Important	Seepage Not Important	Fills	Surfacing
GW	-	-	1	1	-	-	1	1	3
GP	-	-	2	2	-	-	3	3	-
GM	2	4	-	4	8	1	4	9	5
GC	1	1	-	3	1	2	6	5	1
SW	-	-	3 /a	6	-	-	2	2	4
SP	-	-	4 /a	7 /a	-	-	5	4	-
SM	4	5	-	8 /a	9 /b	3	7	10	6
SC	3	2	-	5	2	4	8	6	2
ML	6	6	-	11	4 /b	6	9	11	-
CL	5	3	-	9	3	5	10	7	7
MH	8	8	-	12	6	8	12	13	-
CH	7	7	-	10	5 /c	9	13	8	-
VH	9	9	-	13	7 /b	10	14	12	-
OL		Unsuitable				7	15	-	-
OH		"				11	11	-	-
PT		"				-	-	-	-

/a = if gravelly
/b = erosion critical
/c = volume change critical

Table IV-6 BEARING CAPACITY FACTOR

ϕ_u	Nc	Nq	Nr
0	5.71	1.00	0
5	6.72	1.39	0
10	8.01	1.94	0
15	9.69	2.73	1.2
20	11.9	3.88	2.0
25	14.8	5.60	3.3
30	19.1	8.32	5.4
35	25.2	12.8	9.6
40	34.8	20.5	19.1

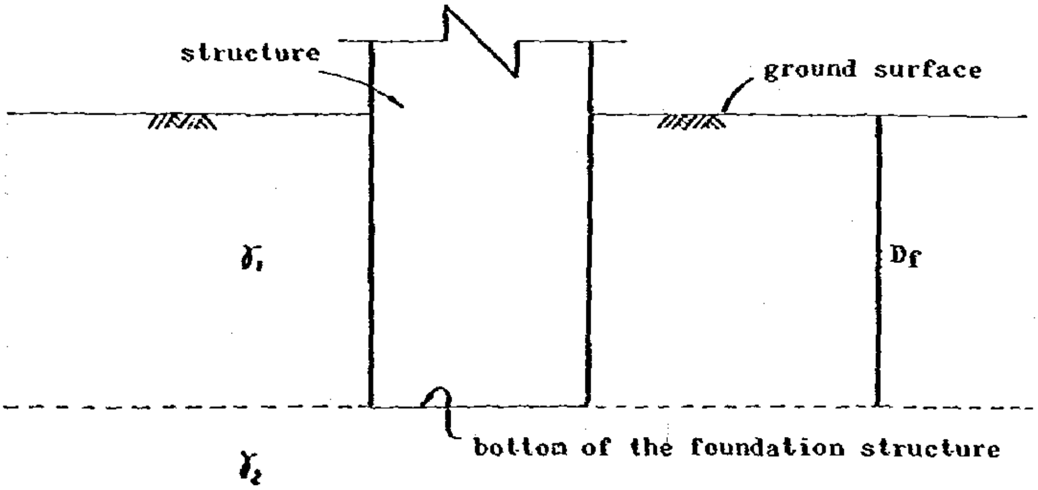


Table IV-7 SHAPE FACTOR OF BEARING CAPACITY

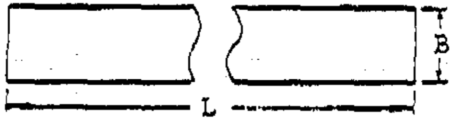
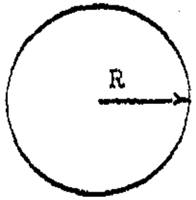
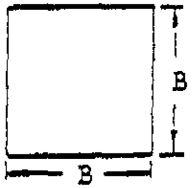
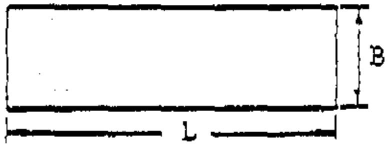
Shape of Foundation Load		Shape Factor	
		α	β
Continuous L B		1.0	0.5·B
Circular		1.3	0.6·R
Square		1.3	0.4·B
Rectangular		1 + B/3L	0.5·B(1–B/3L)

Table IV-8 EFFECTIVE GRAIN SIZE OF SAND

D_{10} (mm)

Classification	SC - SM	SW	SP
1	0.0030	0.075	0.08
2	0.0030	0.110	0.16
3	0.0075	0.085	0.06
4	0.0330	0.095	0.28
5	0.0300	0.060	0.20
6	0.0022	0.090	0.24
7	0.0010	0.060	0.38
8	0.0044	0.100	
9	0.0060	0.065	
10		0.095	
11		0.050	
12		0.055	
13		0.032	
Average	0.0100	0.075	0.20

Table IV-9 COEFFICIENT OF PERMEABILITY FOR SAND

	SC - SM	SW	SP
$\overline{D_{10}}$ (cm)	1.0×10^{-3}	7.5×10^{-3}	2.0×10^{-2}
$(D_{10})^2$	1.0×10^{-6}	5.6×10^{-6}	2.0×10^{-4}
k (cm/sec)	1.0×10^{-4}	5.6×10^{-3}	4.0×10^{-2}

Utilized Hazen's Formula : $k = C \cdot D_{10}$

C; constant
(adopt 100)

Table IV-10 DEPTH OF GROUNDWATER TABLE

	Hole No.	GROUND WATER TABLE			Hole No.	GROUND WATER TABLE	
		Depth below GL (m)	Elevation WL (m)			Depth below GL (m)	Elevation WL (m)
Headreach	BH 3-1	4.3	85.7	South Main Canal	80 AB-16	3.3	63.0
	80 TP-1	1.2	88.3		" 15	3.3	62.7
	BH 3-2	7.2	95.3		" 14	1.1	65.4
	BH 3-3	2.1	83.0			(-3.3)/1	(63.0)
	80 TP-2	3.6	82.9		" 1	0.2	62.8
	BH 3-4	4.0	91.0		" 2	2.0	58.0
	BH 3-5	4.3	83.8		80 TP-9	deeper than	
	80 TP-3	6.5	91.5			10.0	46.0
	" 4	8.5	84.5		" 11	4.3	55.7
	BH 3-9	3.4	80.6		" 8	1.5	53.5
	80 AB-12	0.9	82.7		80 AB-4	1.5	53.5
	13	2.0	80.0		" 18	4.5	50.5
	11	1.0	77.9		" 17	2.0	46.0
	10	1.3	77.0				
	9	3.8	69.9	Pisang Main Canal	80 TP-10	2.5	65.5
North Main Canal	80 TP-14	2.0	72.0	South Main Canal (Secondary Plan)	80 AB-6	1.1	72.9
	80 AB-5	2.5	70.5		" 7	3.0	68.8
	BH 3-10	8.6	63.6		" 8	5.0	64.4
	80 TP-5	1.1	87.4		" 3	4.3	61.7
	" 12	1.8	68.2				
	" 13	2.9	67.1				
	" 6	1.3	63.7				
	80 AB-19	2.0	68.0				
	80 TP-7	2.4	57.6				

/1: Artesian head = 3.5 - 1.1 = 2.4 m

Table IV-11 RESULTS OF SPECIFIC GRAVITY TEST AND ABSORPTION TEST

Division	Sample No.	Gravel		Sand
		Specific Gravity (S.S.D)	Water Absorption (%)	Specific Gravity
Komering river upstream of Martapura	No.1	–	–	2.64
	No.2	–	–	2.70
	No.3	–	–	2.66
	No.4	2.58	1.62	2.69
	No.5	2.54	1.95	2.79
	No.6	2.53	2.53	2.70
Komering river downstream of Martapura	No.7	2.48	2.46	2.66
	No.8	2.46	3.52	2.76
	No.9	2.47	2.90	2.73
Ogan river	No.10	2.62	1.08	2.71
	No.11	2.59	1.17	2.75
	No.12	2.53	2.09	2.79

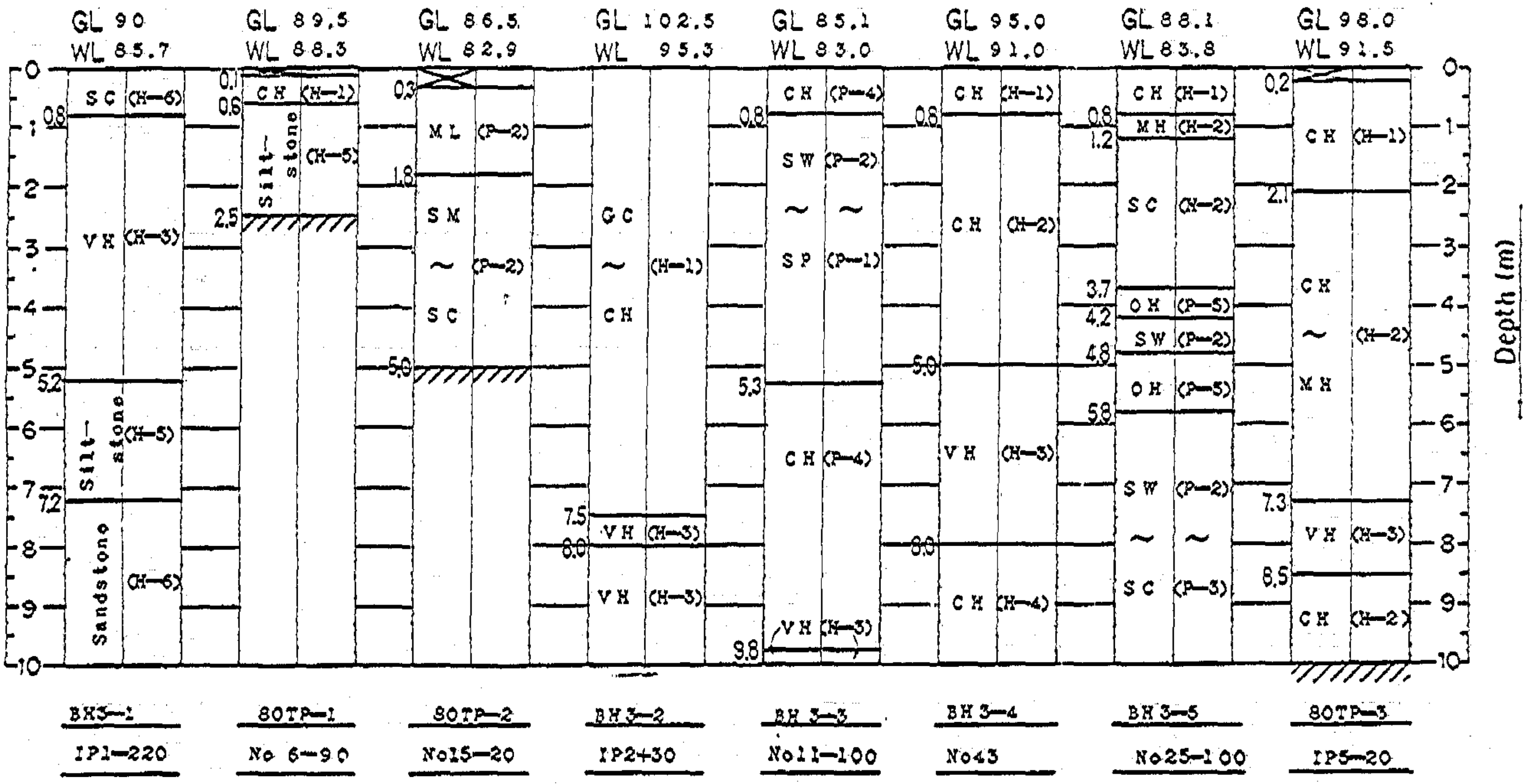
Remark: 'S.S.D' indicates condition on Saturated Surface-Dry.

Table IV-12 AVAILABLE AMOUNT OF GRAVEL

Proposed Site		Mean of /2 Carrying Distance (km)	Representative Sample	Total Earth Volume = Area x Thickness (m^2) x (m)	Utility Earth Volume (x 40%, m^3)	Content of Gravel (%)	Volume of Gravel (m^3)	Recovery (x 90%) (m^3)
River	Block /1							
Komering (Upper than Martapura)	A	21	No.6	721,000 x 2 = 1,442,000	576,000	50%	288,000	259,000
Upper Stream	B	16	No.5	1,031,000 x 2 = 2,062,000	824,000	40%	329,000	296,000
	C	11	-	842,000 x 2 = 1,684,000	673,000	30%	201,000	180,000
	D	7	No.4	858,000 x 2 = 1,716,000	686,000	30%	205,000	184,000
Lower Stream	E	3	-	1,724,000 x 2 = 3,448,000	1,379,000	30%	413,000	371,000
Komering (Lower than Martapura)	F	15	No.7, 8, 9	2,230,000 x 2 = 4,460,000	1,784,000	30%	535,000	481,000
	G	55	No.10	788,000 x 2 = 1,576,000	630,000	70%	441,000	396,000
Ogan	H	50	No.11	512,000 x 2 = 1,024,000	409,000	50%	204,000	183,000
	I	55	No.12	960,000 x 2 = 1,920,000	768,000	50%	384,000	345,000

Remarks, /1: Refer to PLATE NO. 8 /2: Distance between the Pracak Headworks and each Block.

Fig.IV-1-(a) COLUMNAR SECTION BY SOIL CLASSIFICATION



Headreach (In case of Pracak Headworks)

Fig.IV-1-(b) COLUMNAR SECTION BY SOIL CLASSIFICATION

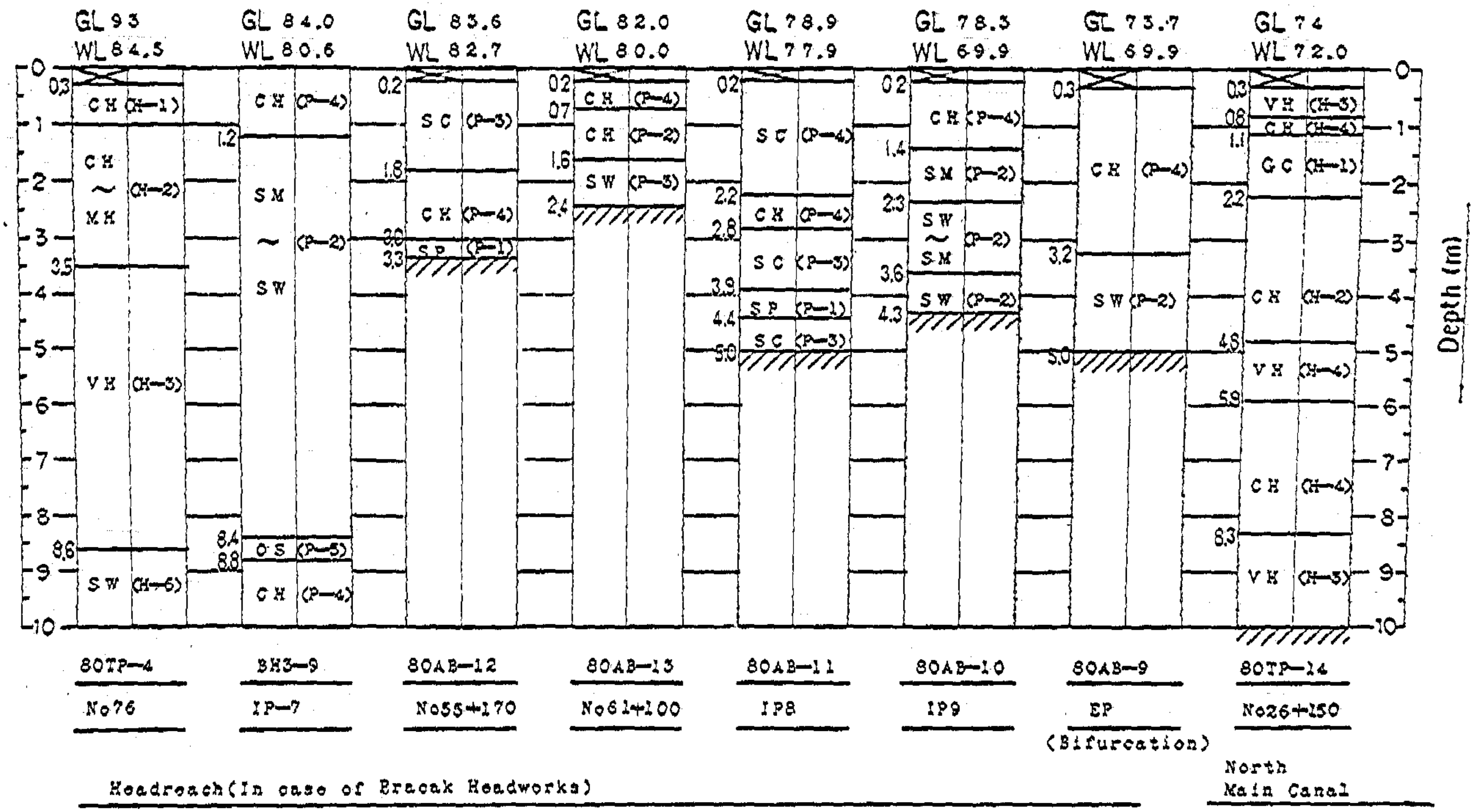


Fig.IV-1-(c) COLUMNAR SECTION BY SOIL CLASSIFICATION

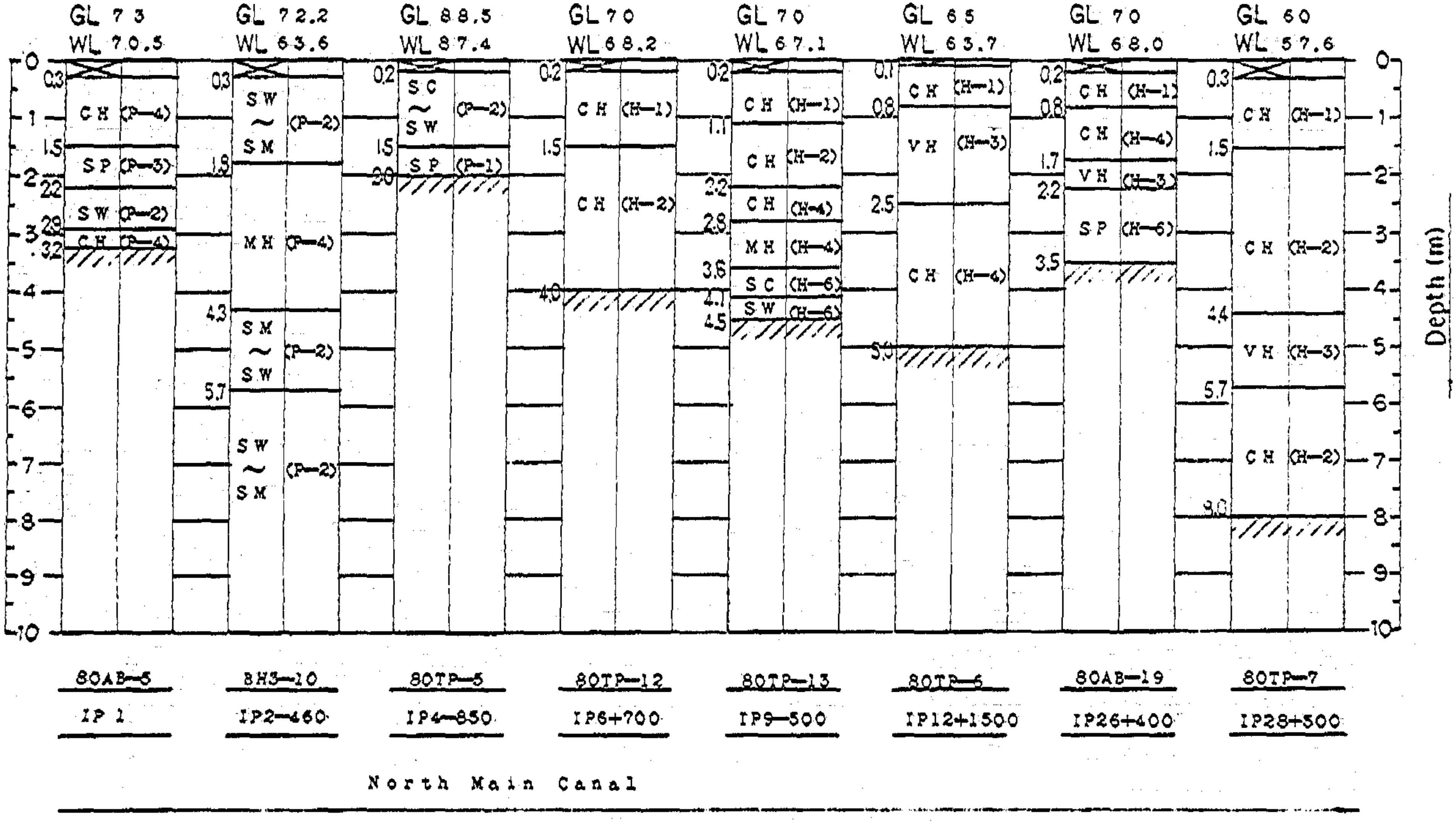


Fig.IV-1-(d) COLUMNAR SECTION BY SOIL CLASSIFICATION

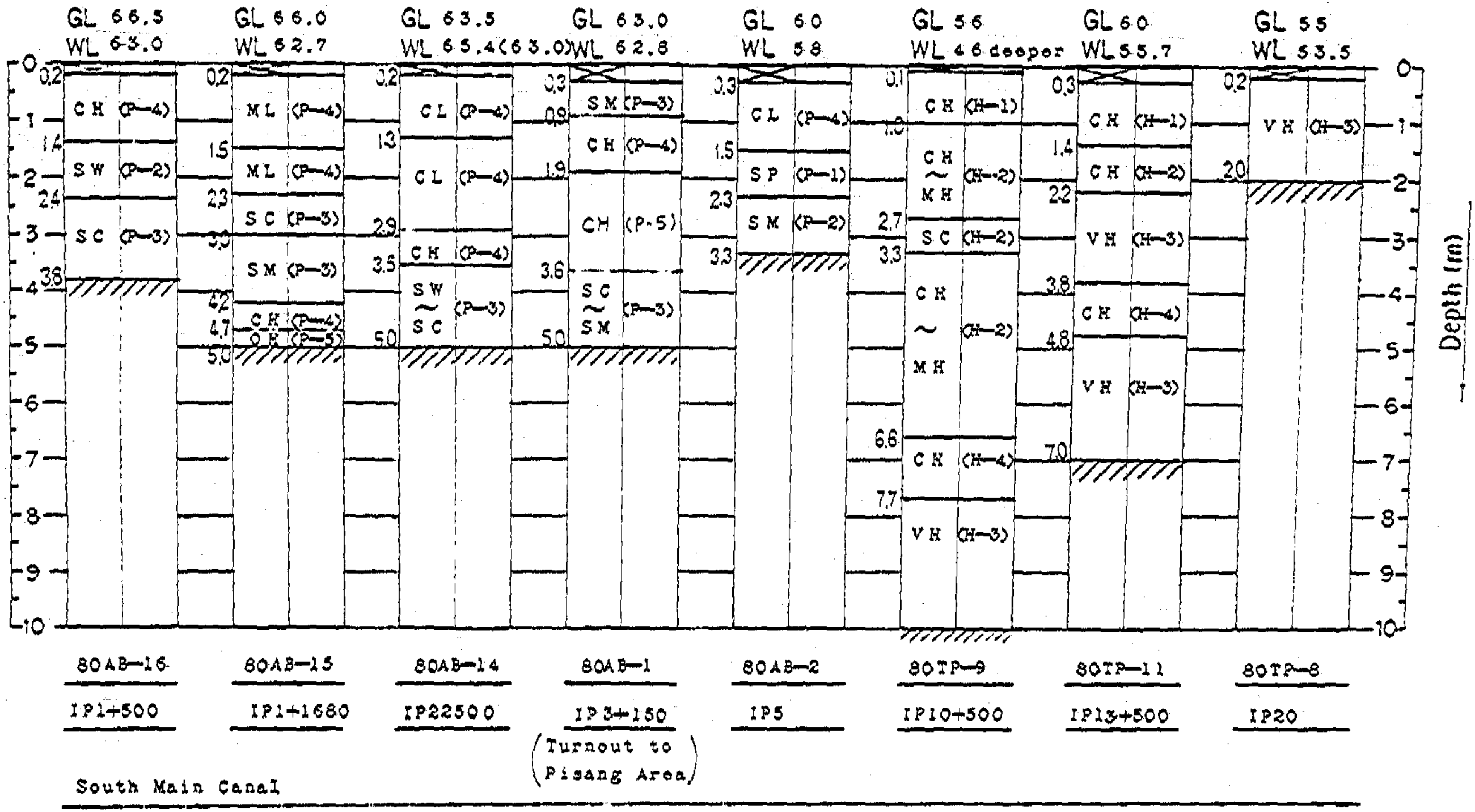


Fig.IV-1-(c) COLUMNAR SECTION BY SOIL CLASSIFICATION

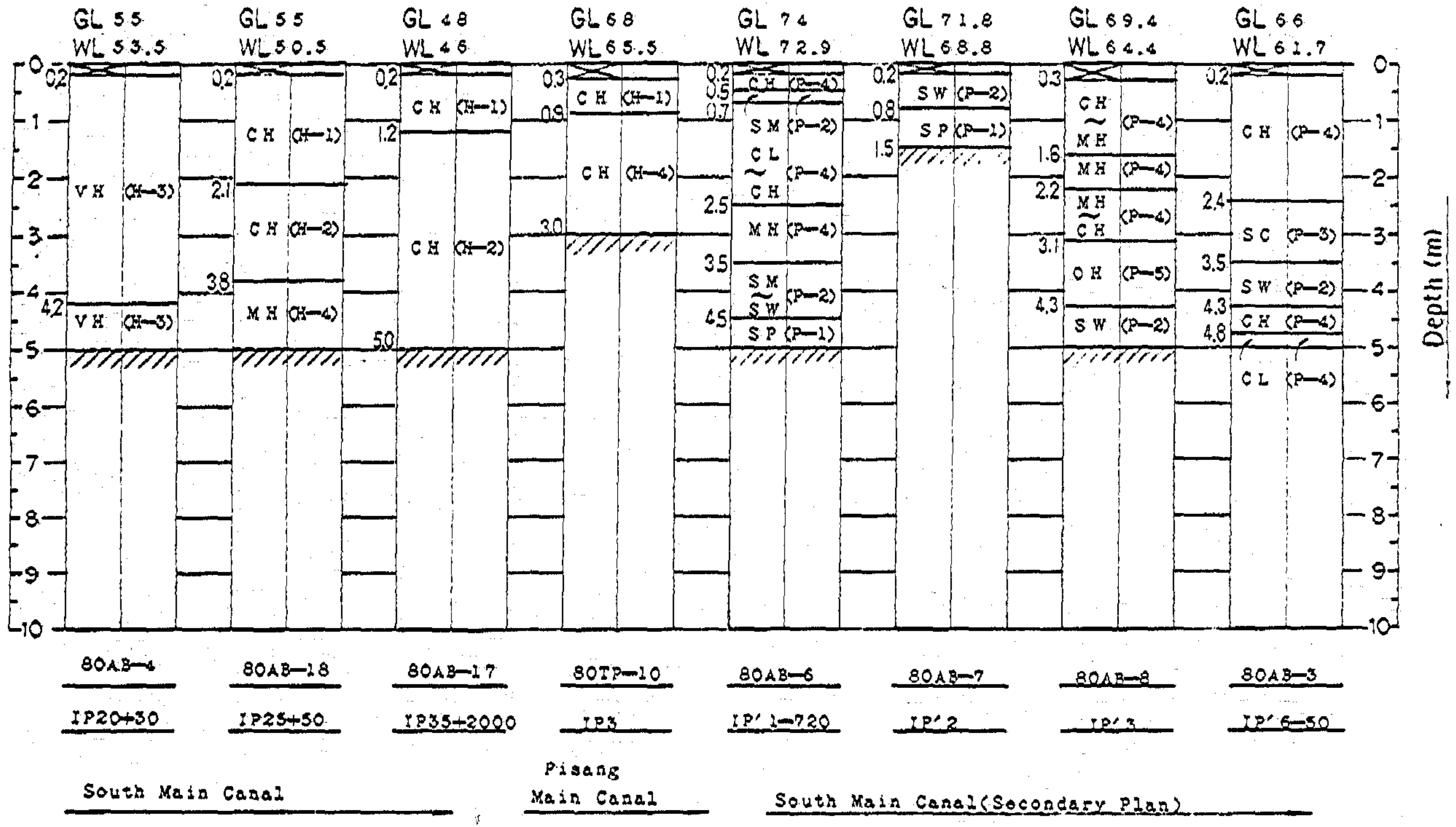
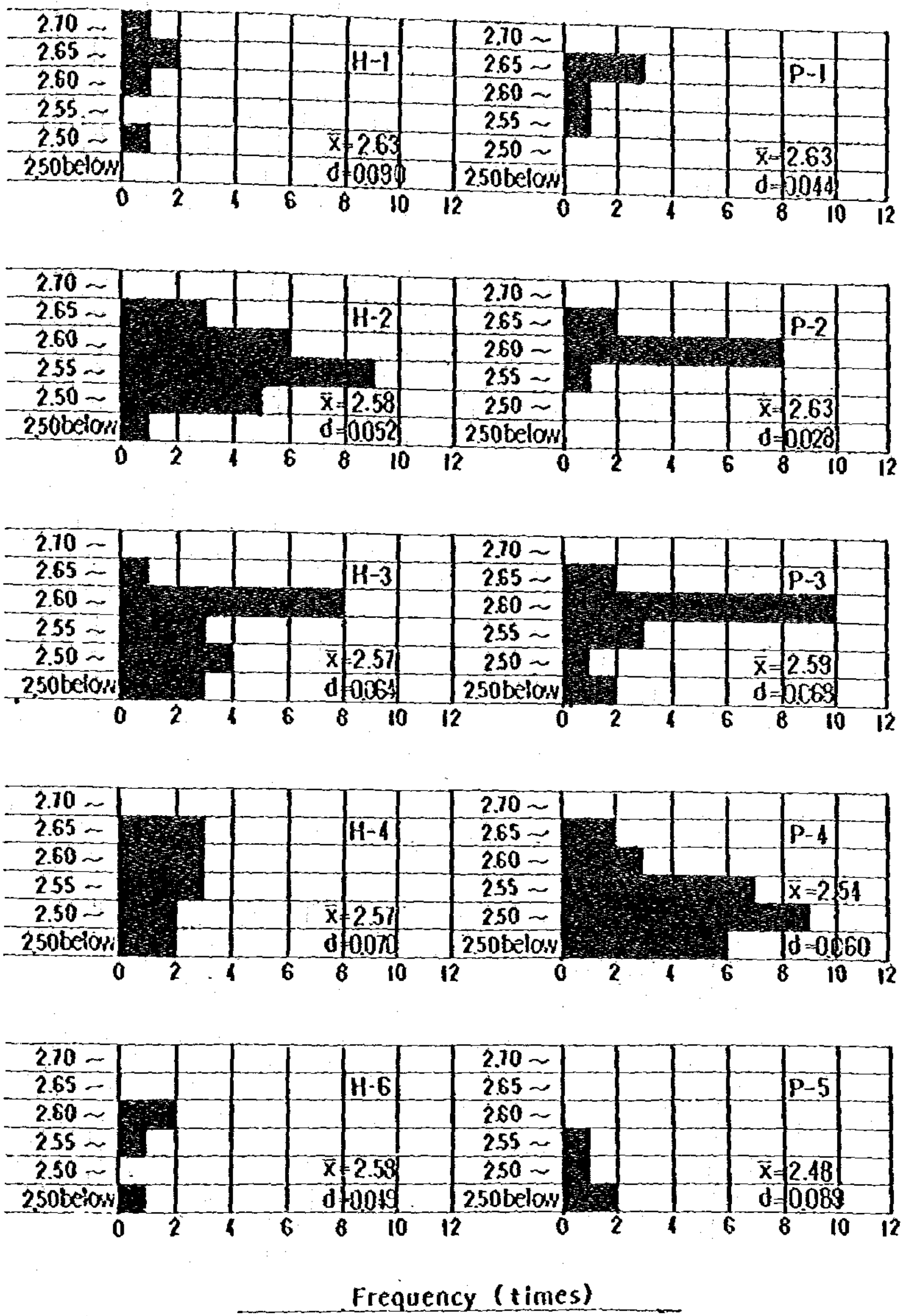


Fig.IV-2 HISTOGRAM OF MEASURED SPECIFIC GRAVITY

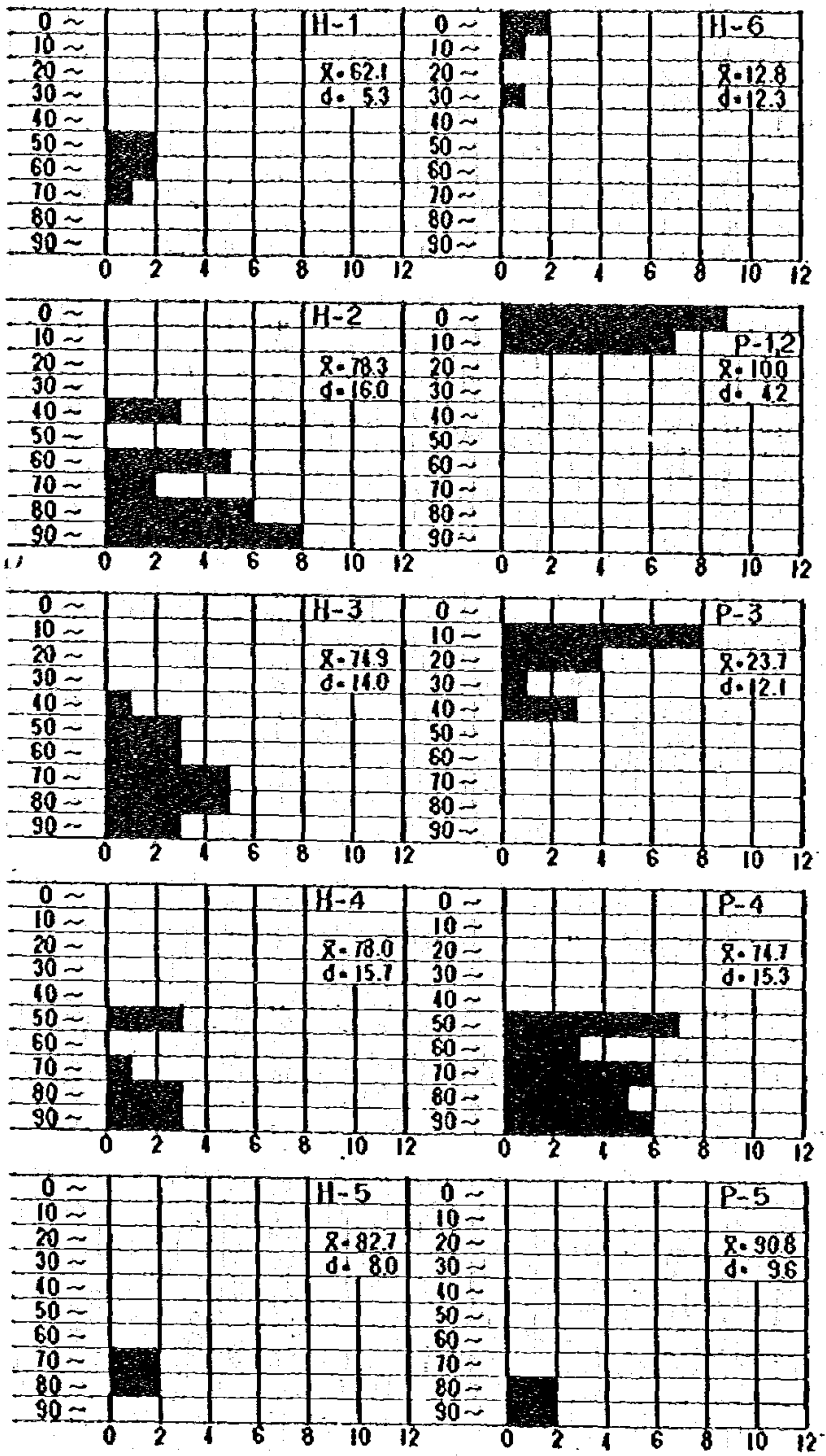


Note:

x̄ : average

d : standard deviation

Fig.IV-3 HISTOGRAM OF PERCENT FINER THAN NO.200 SIEVE



Note: Frequency (times)

X̄ : average

d : standard deviation

Fig. IV-4 PLASTICITY CHART

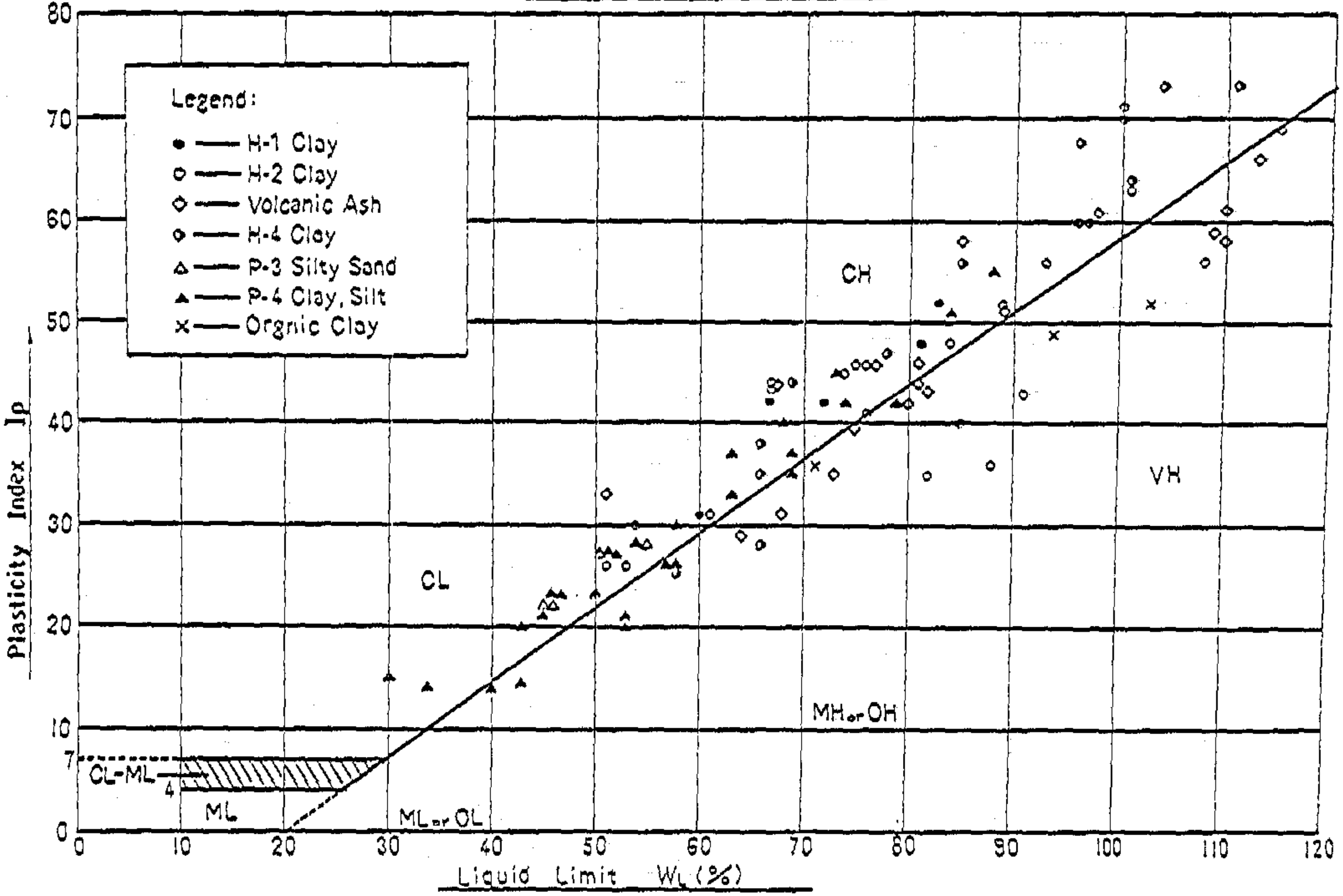


Fig.IV-5 RELATION BETWEEN q_u AND W

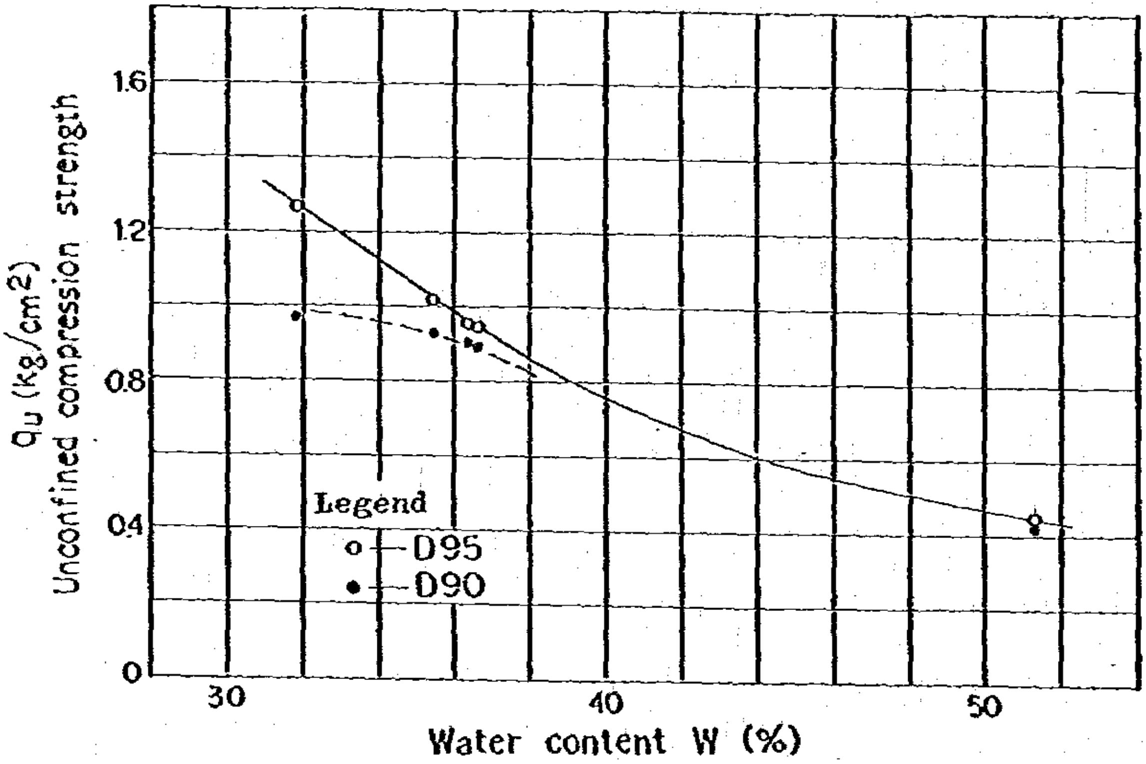


Fig. IV-6 MOHR'S CIRCLE OF TRIAXIAL COMPRESSION TEST ON FOUNDATION SOIL UNDER U-U CONDITION

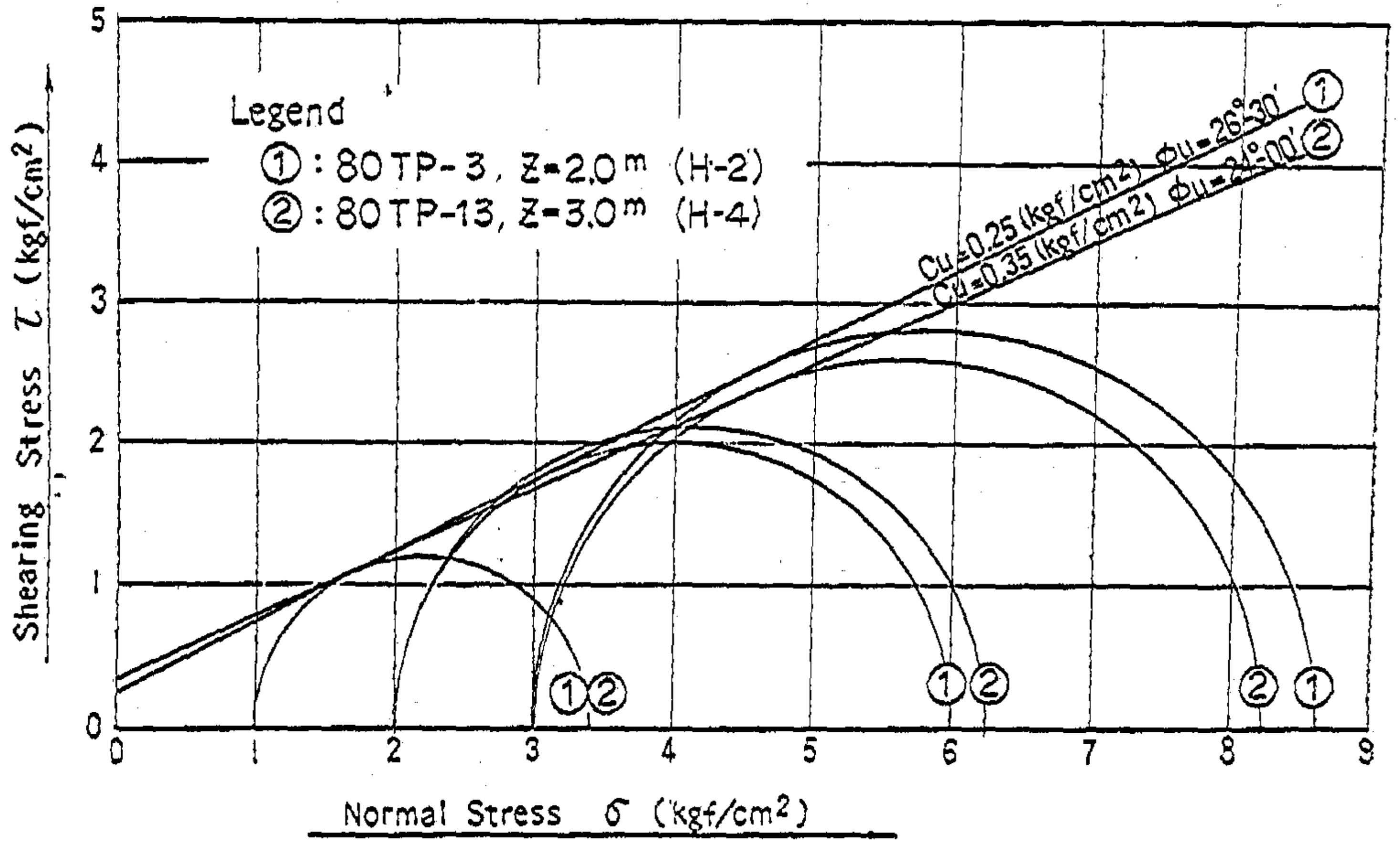


Fig. IV-7 COMPARISON BETWEEN q_u AND MOHR-COULOMBS STRENGTH ENVELOPES

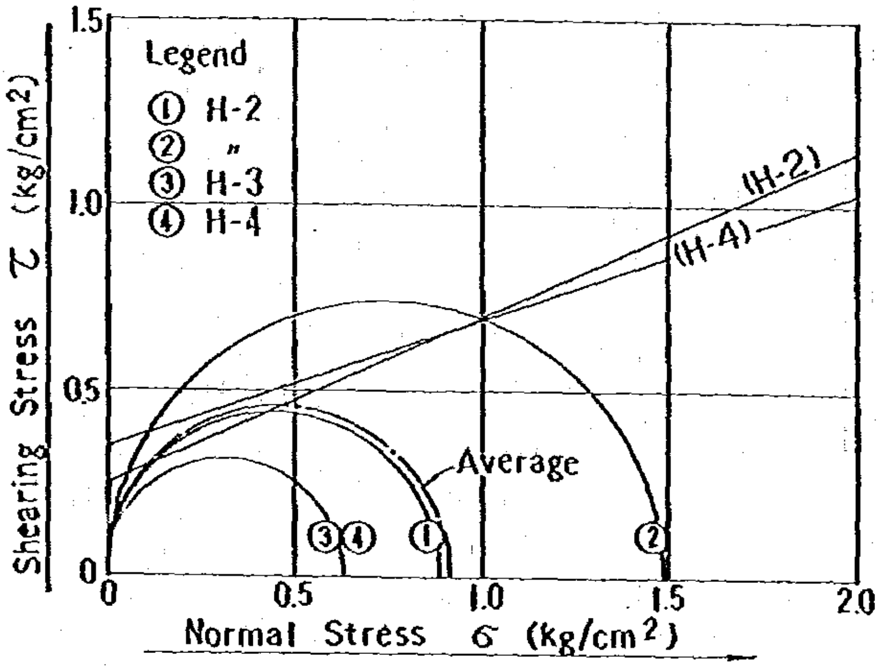


Fig.IV-8 RECORD OF STANDARD PENETRATION TEST

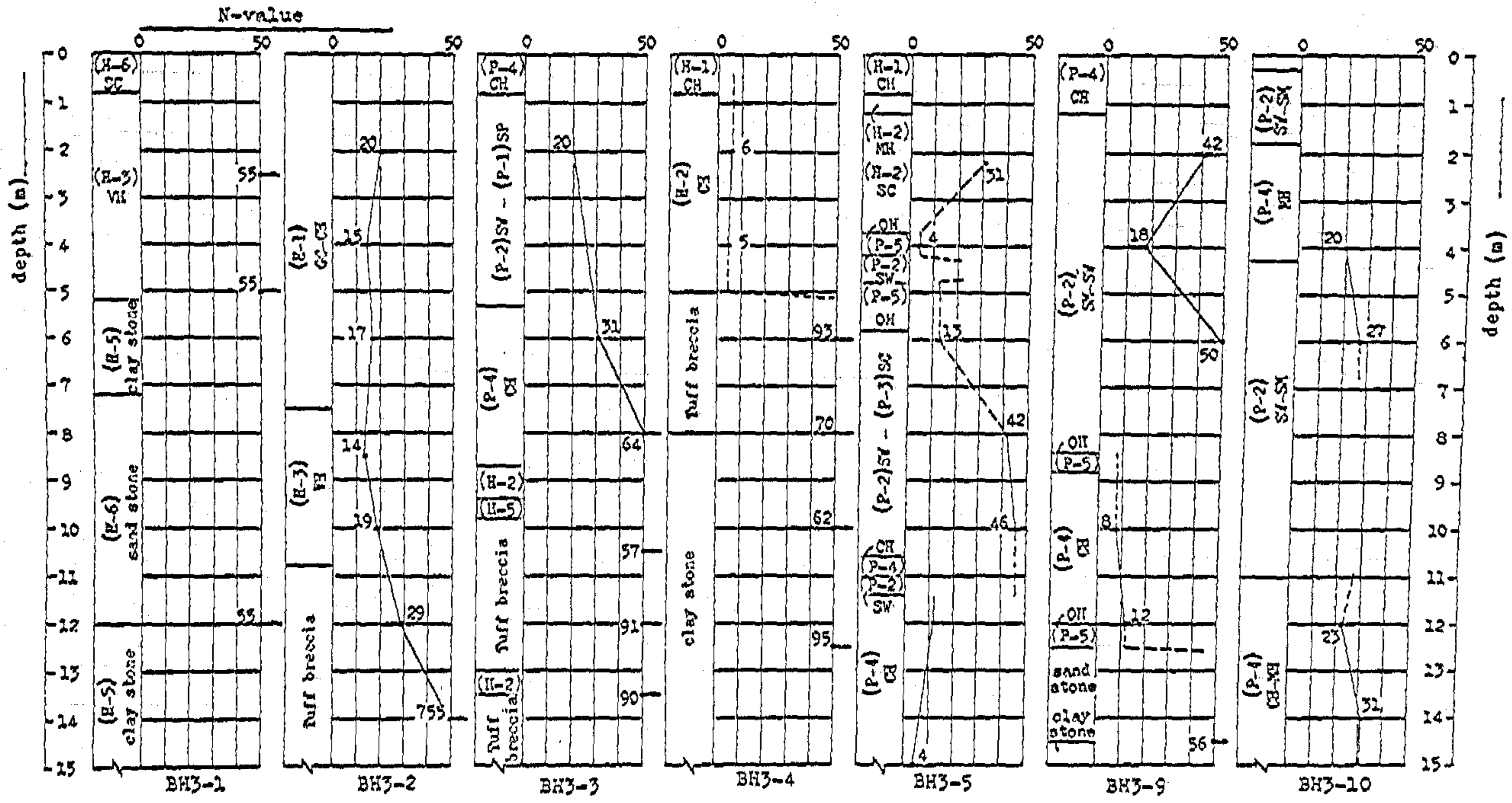
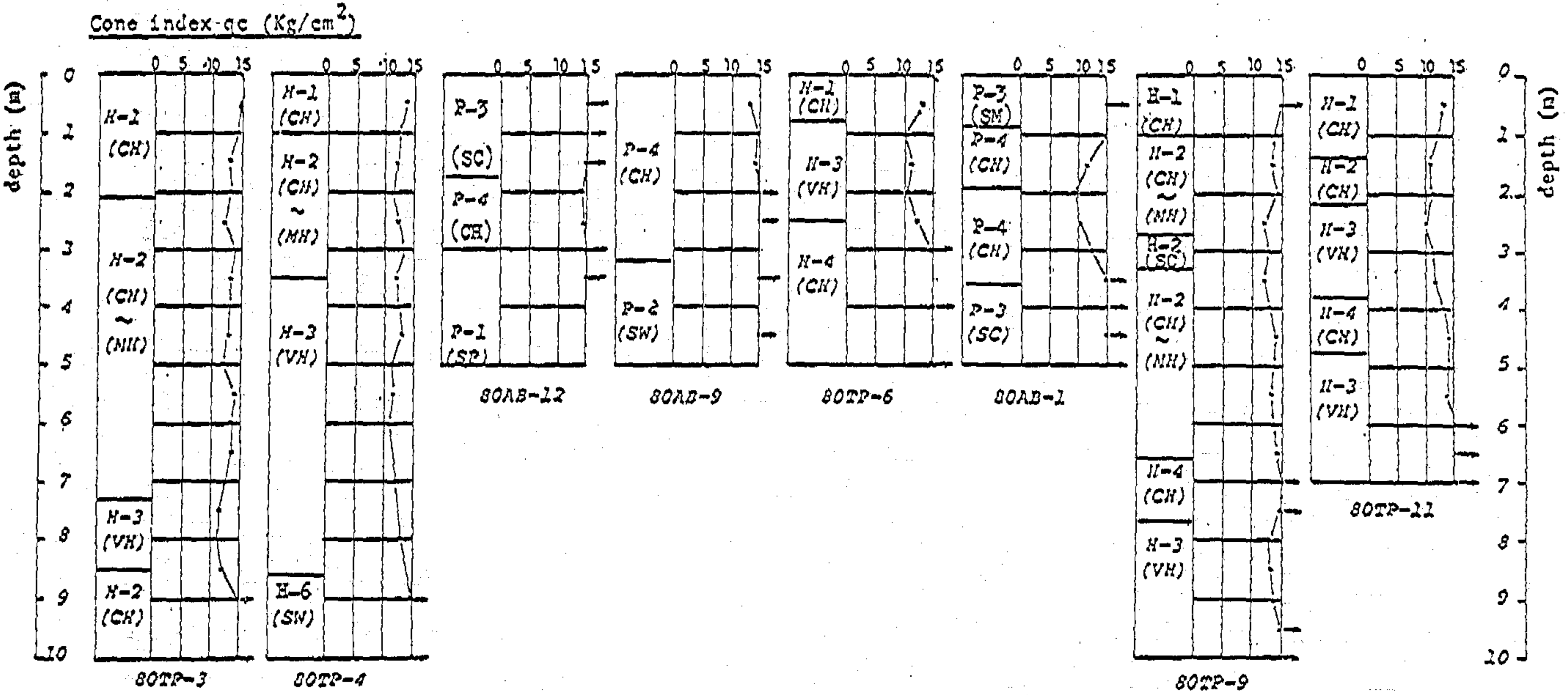
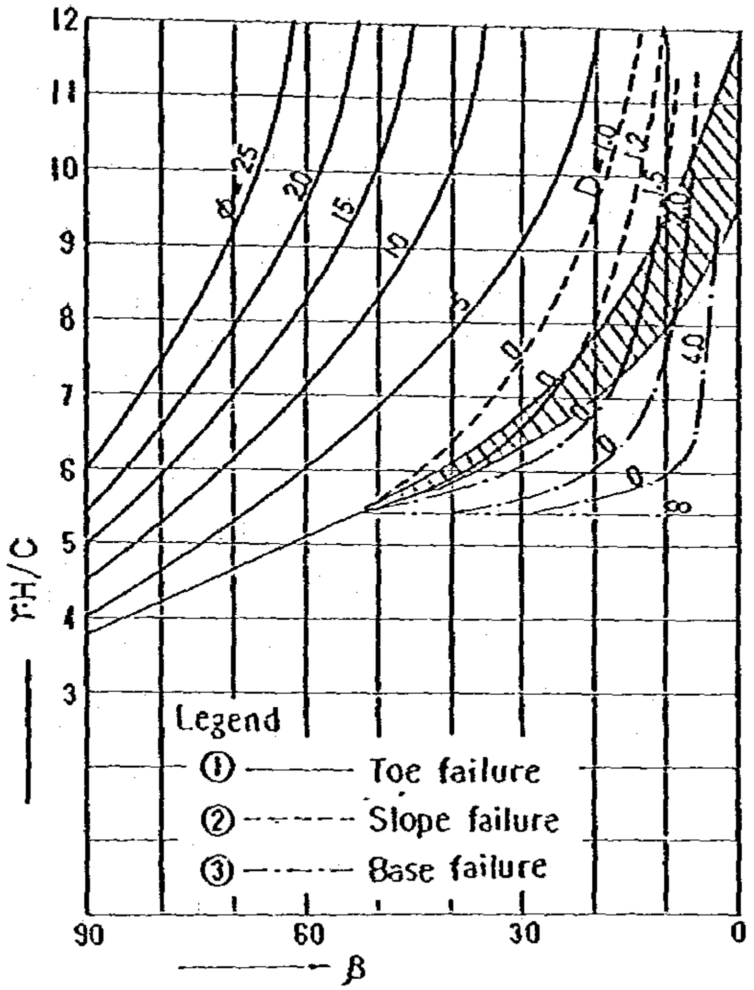


Fig.IV-9 RECORD OF CONEPENETRATION TEST



→ : This symbol means that the cone index is more than 15 kg/cm^2.

Fig.IV-10 TAYLOR'S SLOPE STABILITY CHART



Example

Condition, $\gamma = \gamma t = 1.60$ t/m^3

C = Cu/Fs = 3.0/1.5 = 2.0 t/m^2

$\phi = \phi_u/Fs = 0/1.5 = 0$ degree

(Fs: safety factor)

H = 6 m

Result, $\gamma H/C = 1.60 \times 6/2.0 = 4.8$

from above chart

β = 66 degree

Fig.IV-11 FORM OF SEEPAGE FROM CANAL

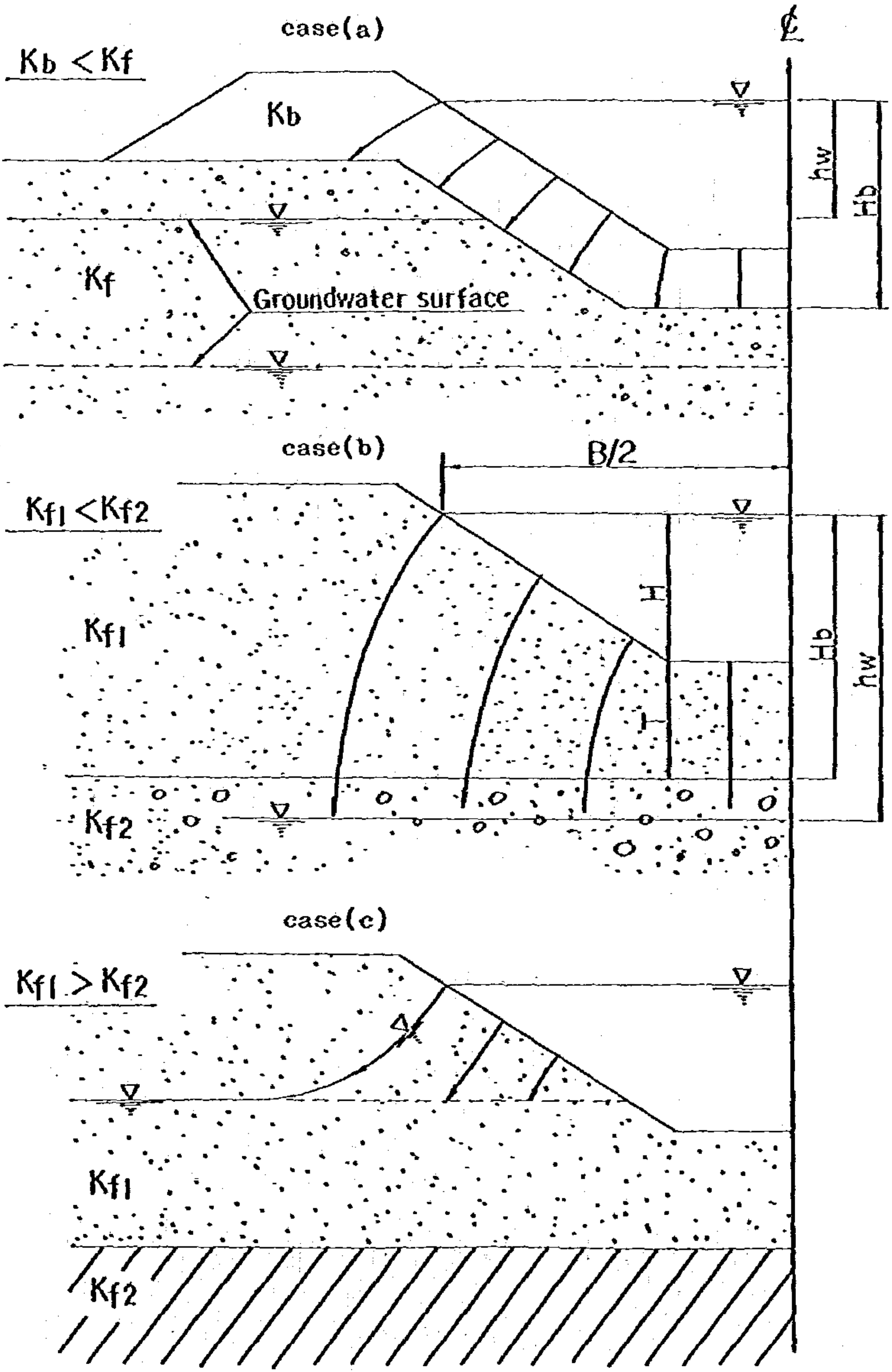


Fig.IV-12 PROTECTION MEASURE FOR SEEPAGE

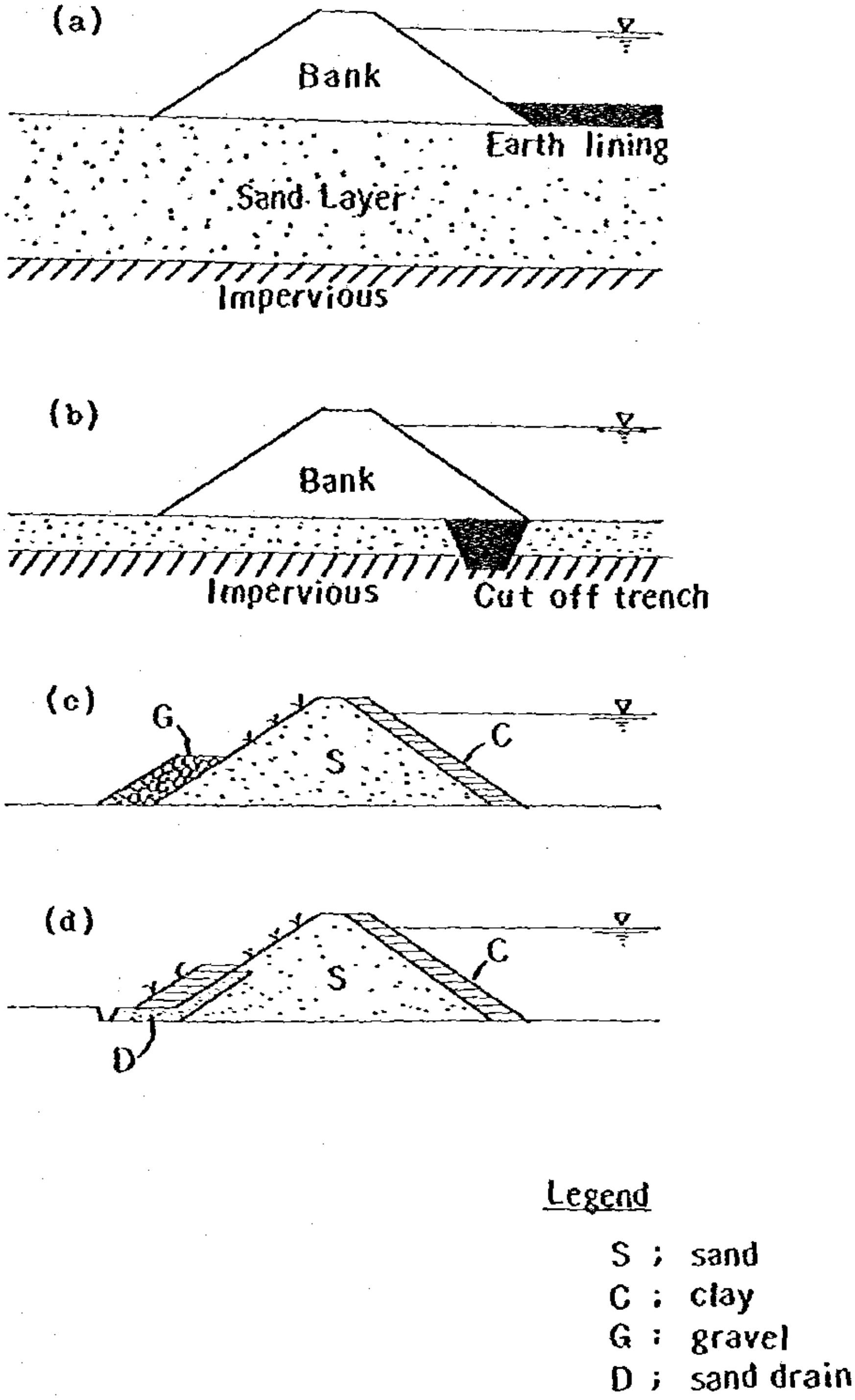
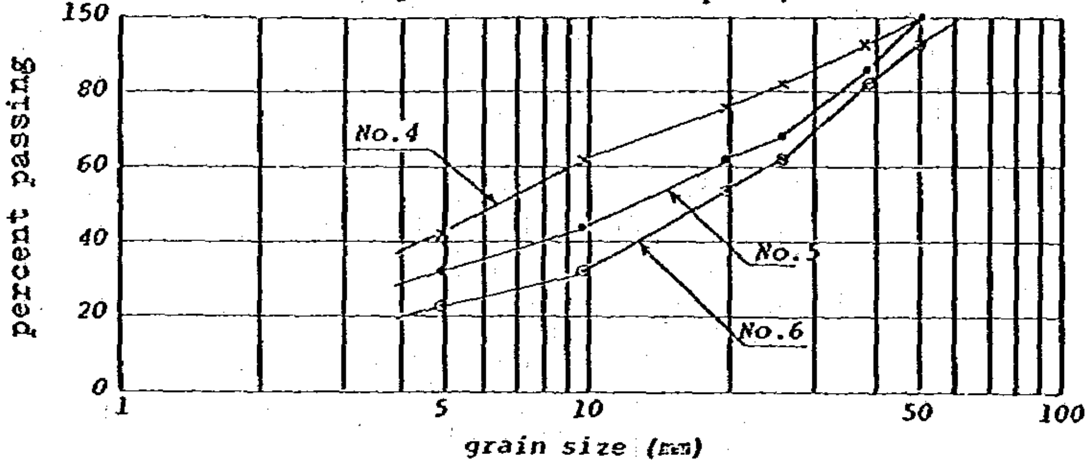
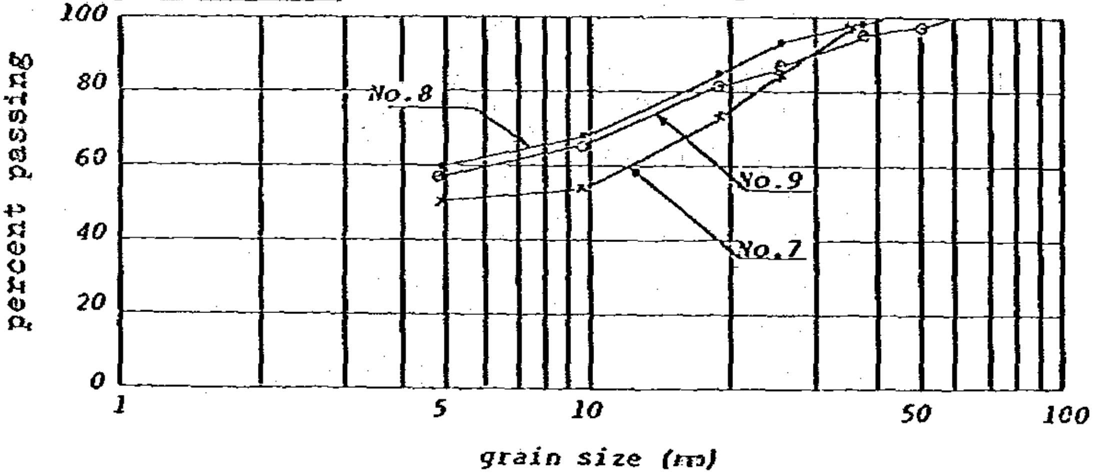


Fig.IV-13 GRAIN SIZE ACCUMULATION CURVE (GRAVEL)

KOMERING RIVER (Upstream of Martapura)



KOMERING RIVER (Downstream of Martapura)



OGAN RIVER

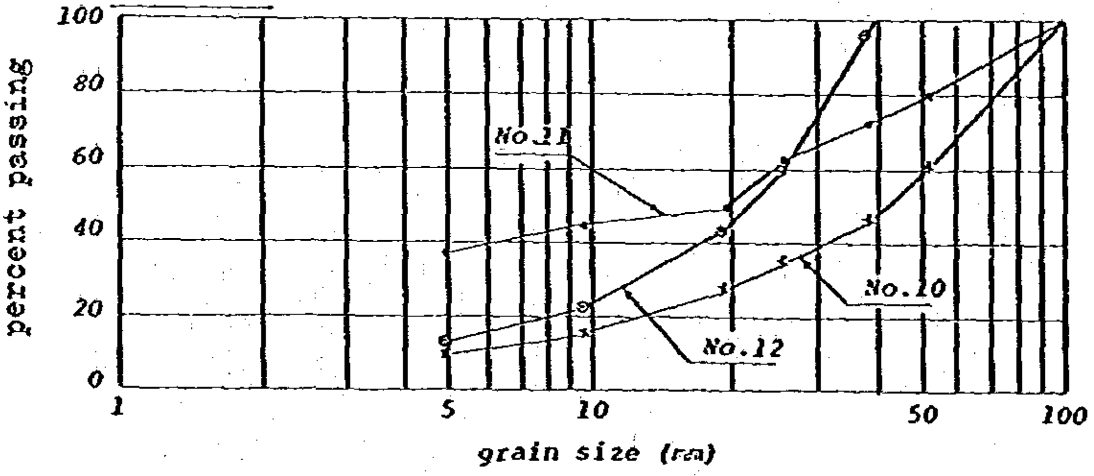
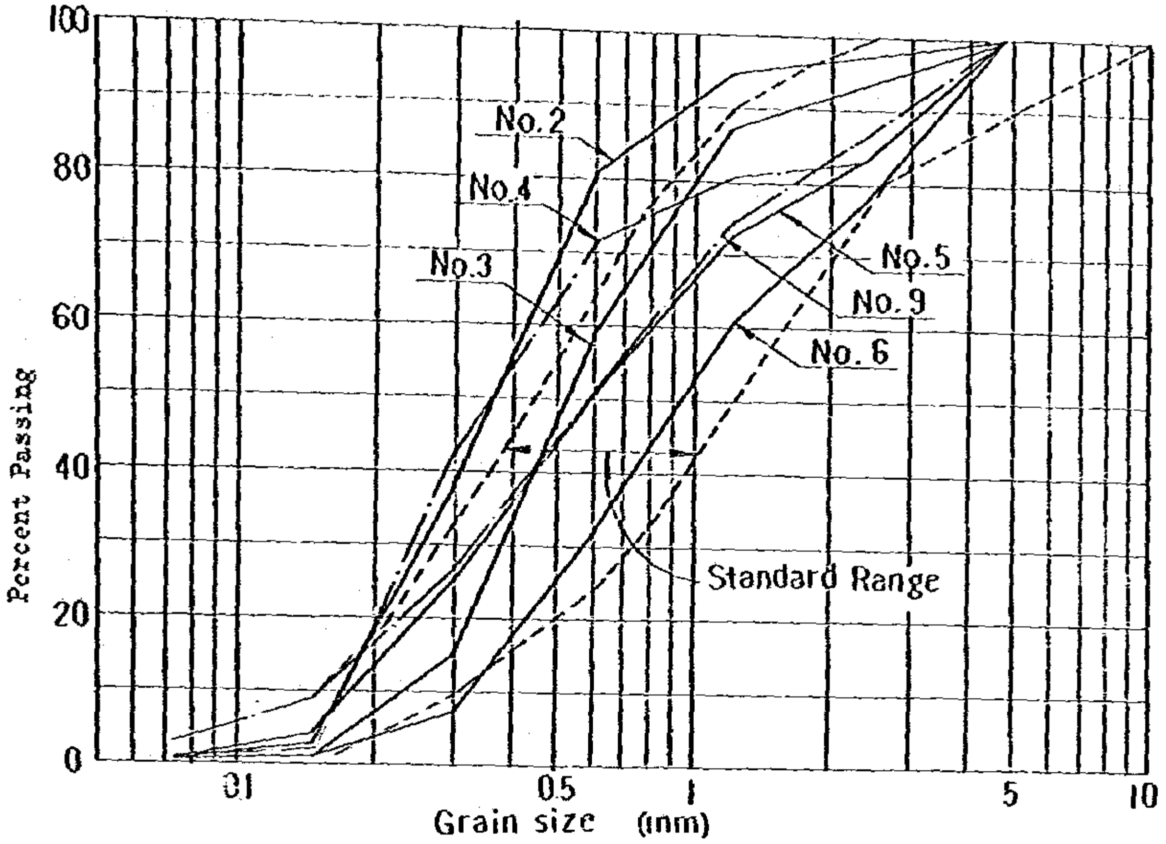
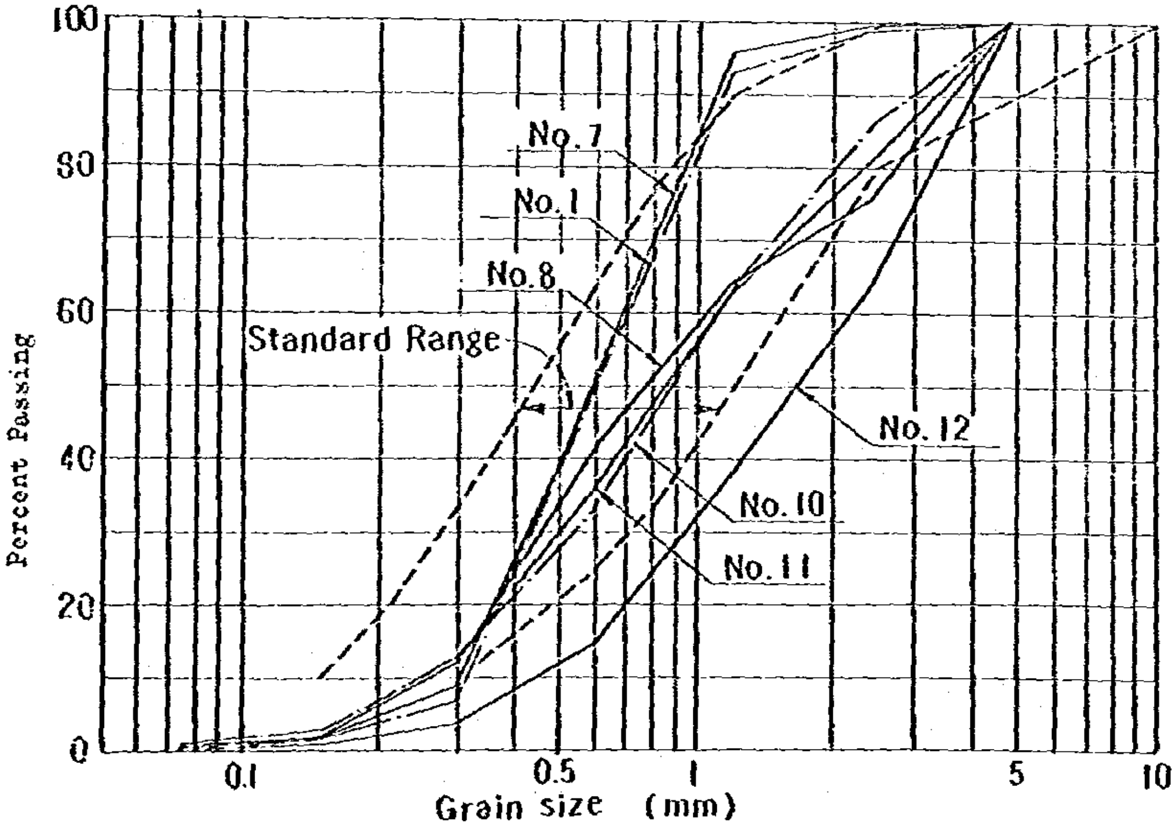


Fig.IV-14 GRADATION OF FINE AGGREGATE





JICA