SECTION 5 GROUND CONDITIONS OF EX-MINING LAND

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GROUND CONDITIONS OF EX-MINING LAND

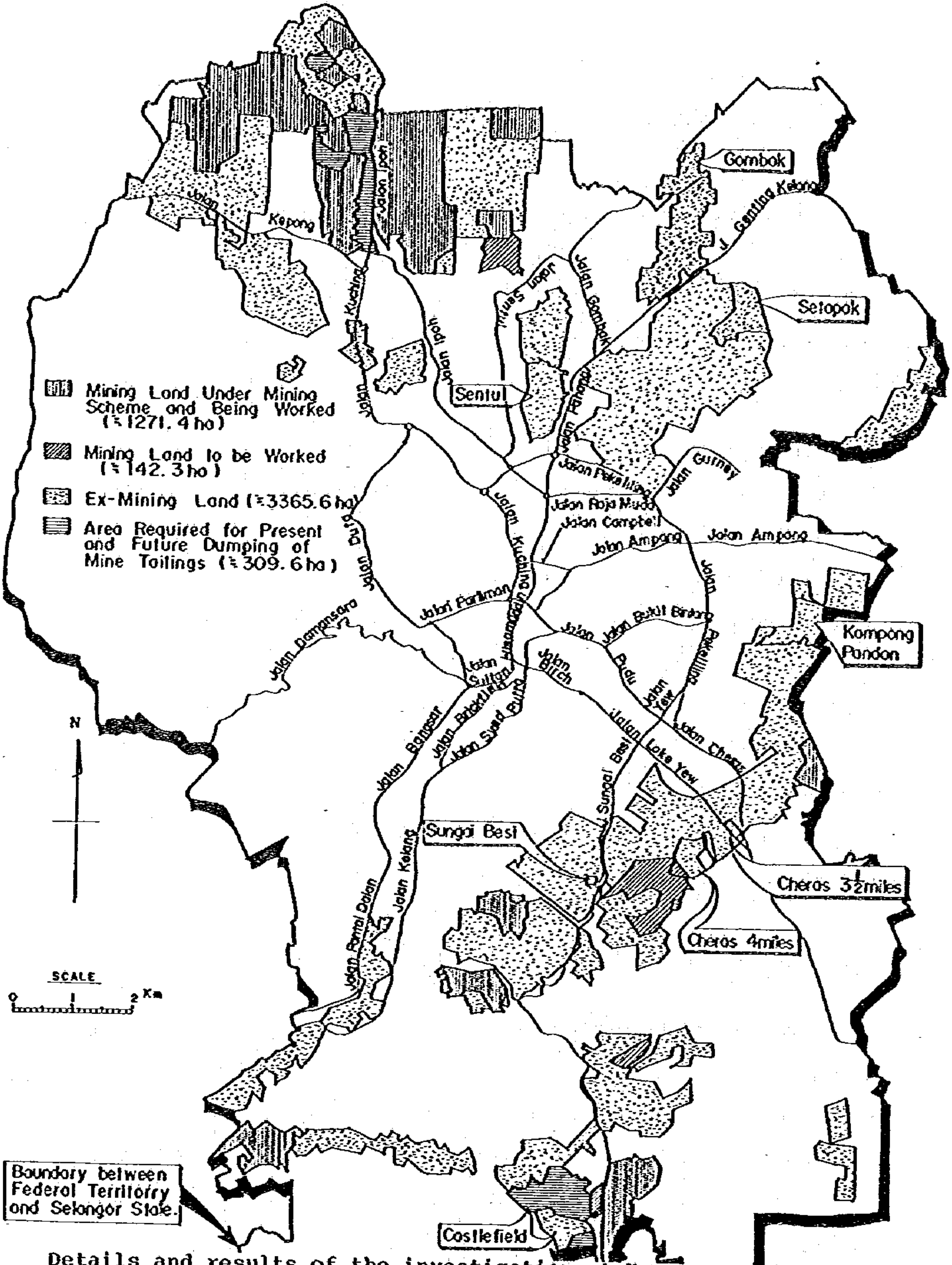
5. GROUND CONDITIONS OF EX-MINING LAND

In order to determine general ground conditions of ex-mining land, in addition to the relatively extensive soil investigations performed at Kampong Pandan and at Sentul, soil investigations were performed during Phase II at several other ex-mining lands.

The first part of this section describes the soil investigations carried out at these additional sites. Next, the ground conditions at each site are examined and described. Based on the results of the investigations, several models of the foundation ground of ex-mining land are postulated for engineering and economic analyses in the sections that follow. In the final part of this section, the investigation methods used to determine the foundation ground of ex-mining land are evaluated and effective investigation methods for the provision of structural design parameters are proposed.

5.1 Details of Ground Investigations

Ground investigations were performed at the eight sites shown in Fig. 5-1. Details of the investigations performed at the Kampong Pandan and Sentul sites are presented in Section 4. Described hereafter are the contents and results of investigations performed at the remaining six ex-mining sites.



Details and results of the investigation at Kampong Pandan and Senful are presented in Section 4, those for the other 6 sites are presented in Section 5.

Fig. 5-1 Location of Sites Investigated

In the course of preliminary investigations, each site was field surveyed and existing information, such as topographic maps, mining records and boring data was collected. Swedish soundings and exploratory borings were performed at three sites and laboratory soil tests were carried out on soil samples retrieved during the borings. Data obtained from the investigation of each site are listed in Table 5-1. The number and depth of Swedish soundings and borings performed at three sites, i.e. Gombak, Setapak, and Castlefield, and the types and number of laboratory soil tests carried out on samples from each site are summarized in Tables 5-2 and 5-3.

5.2 Results of Investigations

The results of the investigation of each site are presented in soil profiles. Detailed data obtained during these investigations such as the results of Swedish soundings, borings, and laboratory soil tests are compiled in Volume 2, Appendices. The ground conditions at each site are briefly described below:-

5.2.1 Gombak

The earthwork at this site, the northernmost one investigated, had already been completed for the construction of five blocks of 5-storey flats for the Phase I construction. Prior to this investigation, some Mackintosh soundings and two borings had been carried out by

Table 5-1 Available Data from Each Site

	Site Plan	Topo-graphic Map	Mining Record	City Hall		Study Team			
				Mackin-tosh Sounding	Bor-ing	Mackin-tosh Sounding	Swedish Sound-ing	Percis-sion Boring	Rotary Boring
1. Gombak	Yes	-	Yes	21	2	-	12	-	-
2. Setapak	Yes	Yes	Yes	-	10	-	15	-	4
3. Sentul *1	Yes	Yes	Yes	105	14	5	25	13	10
4. Kampong Pandan *1	Yes	Yes	Yes	-	-	31	-	5	-
5. Cheras 3-1/2 miles	Yes	-	-	-	25	-	-	-	-
6. Cheras 4 miles	Yes	-	Yes	-	15	-	-	-	-
7. Sungai Besi	Yes	-	Yes	12	6	-	-	-	-
8. Castle-field *2	Yes	Yes	Yes	-	-	-	19	-	3

Remarks: *1 Comprehensive investigations performed. Refer Section 4.

*2 Recent ex-mining site.

Table 5-2a Summary of Swedish Sounding (1)

- Gombak and Setapak -

Site	Sounding No.	Ground Level (RL m)	Sounding Depth (m)	Groundwater Table* (GL ± m)	Remarks
Gombak	GSW-1	54.99	27.00	-2.34 ∿ -2.35	
	2	55.03	9.70	-2.40	
	3	54.87	14.30	-0.95	
	4	54.73	1.90*	-	*Sounding was terminated at this depth due to existence of rocks
	5	54.90	12.40	-0.20 ∿ -0.21	
	6	56.96	15.40	-1.60 ∿ -1.70	
	7	56.70	15.95	-1.40 ∿ -1.60	
	8	56.77	16.55	-1.60 ∿ -1.80	
	9	57.04	16.30	-3.00 ∿ -3.28	
	10	57.03	20.00	-3.00 ∿ -3.40	
	11	59.48	20.00	-1.23 ∿ -1.24	
	12	59.12	20.00	-0.85 ∿ -0.93	
	Sub-Total	12 locations	189.50 m	-	
Setapak	PSW-1	43.54	4.45	+0.07	
	2	45.54	10.75	-0.72 ∿ -0.76	
	3	47.74	12.78	-1.23 ∿ -1.29	
	4	46.28	10.70	-0.02 ∿ -0.04	1m from PBH-3
	5	46.33	11.00	-0.12 ∿ -0.19	
	6	44.01	6.80	-0.04	
	7	47.15	5.15	-0.77 ∿ -0.78	
	8	46.71	14.50	-0.20 ∿ -0.22	
	9	46.81	9.95	-0.30	1m from PBH-2
	10	47.18	13.00	-0.30	
	11	46.54	10.75	-0.20	1m from PBH-1
	12	48.20	10.75	-1.10	
	13	47.41	11.25	-0.72 ∿ -0.73	1m from PBH-4
	14	47.91	11.20	-1.08 ∿ -1.09	
	15	48.17	6.43	-0.79 ∿ -0.91	
	Sub-Total	15 locations	149.46 m	-	
	Grand Total	27 locations	338.96 m	-	

*Ground-water tables were observed on 18th and 19th Jan. 1981

Table 5-2b Summary of Swedish Sounding (2)

- Castlefield -

Site	Sounding No.	Ground *1 Level (TBM ± m)	Sounding Depth (m)	Groundwater *2 Table (BL ± m)	Remarks
North Site	CNSW-1	+ 0.94	7.00	-3.10 ∿ -3.20	
	CNSW-2	+ 0.05	18.15	-1.85 ∿ -1.87	1m from CNBH-1
	CNSW-3	- 0.39	11.29	-1.84 ∿ -1.85	
	CNSW-4	- 0.82	18.07	-1.28 ∿ -1.30	1m from CNBH-2
	CNSW-5	- 1.08	18.03	-0.88 ∿ -1.05	
	CNSW-6	- 2.85	15.55	-0.18 ∿ -0.24	1m from CNBH-3
	CNSW-7	- 3.04	11.27	-0.05 ∿ -0.26	
	CNSW-8	- 3.84	12.10	-0.05 ∿ -0.07	
	CNSW-9	- 3.88	12.15	-0.06 ∿ -0.23	
	CNSW-10	- 3.93	13.40	-0.05 ∿ -0.16	
	CNSW-11	- 3.92	12.65	+ 0.05	
	CNSW-12	- 3.95	13.78	+ 0.05	
	Sub-Total	12 locations	163.44 m	-	
South Site	CSSW-1	- 0.10	8.85	- 0.4	
	CSSW-2	- 0.36	9.50	+0.10 ∿ +0.28	
	CSSW-3	- 0.49	10.60	+0.07 ∿ +0.20	
	CSSW-4	- 0.42	12.15	+0.04 ∿ +0.05	
	CSSW-5	- 0.28	9.34	+0.06 ∿ -0.28	
	CSSW-6	- 0.25	6.85	+0.06 ∿ -0.58	
	CSSW-7	+ 0.51	7.65	-0.14 ∿ -0.70	
	Sub-Total	7 locations	64.94 m	-	
Total		19 locations	228.38 m	-	
Grand Total*3		46 locations	567.34 m	-	

*1 Ground level reference is a temporary bench mark located on the Sungai Besi Bridge of the KL-Seremban Highway near the site.

*2 Groundwater tables were observed on 17th to 19th Jan. 1981.

*3 Total of Tables 5-2a and 5-2b.

Table 5-3 Detail of Exploratory Borings and Laboratory Soil Tests

Site			Setapak					Castlefield North				Grand Total
Boring No.			PBH-1	PBH-2	PBH-3	PBH-4	Total	CNBH-1	CNBH-2	CNBH-3	Total	
Boring Work	Boring Length (m)	Soil	17.41	8.45	11.83	17.00	54.69 m	21.26	12.25	22.35	55.86 m	110.55 m
		Rock	0.50	0.50	1.12	0.50	2.62 m	0.50	1.00	1.00	2.50 m	5.12 m
		Total	17.91	8.95	12.95	17.50	57.31 m	21.76	13.25	23.35	58.36 m	115.67 m
	Standard Penetration Test (Nos)		9	3	9	13	34 Nos.	11	10	19	40 Nos.	74 Nos.
	Undisturbed Soil Sampling		7	3	2	4	16 Nos.	6	2	3	11 Nos.	27 Nos.
Laboratory Soil Test	Physical Property Test (Sets)		10	5	2	5	22 Sets	11	5	5	21 Sets	43 Sets
	Unconfined Compression Test (Nos.)		-	-	-	-	0	3	-	-	3	3 Nos.
	U-U Triaxial Compression Test (Sets)		7	2	2	2	13 Sets	5	3	2	10 Sets	23 Sets
	Consolidation Test (Nos.)		6	2	2	3	13 Nos.	6	2	4	12 Nos.	25 Nos.
	Vane Shear Test (Nos.)		-	-	-	-	0	-	-	1	1 No.	1 No.

the Kuala Lumpur City Hall. Data were collected from this previous investigation and combined with that derived from Swedish soundings conducted by the consultants at twelve other locations. Fig. 5-2 shows the layout of the proposed flats and the locations of the borings and soundings. The results of these soundings and of visual inspection of the site environs were combined with information from mining records and topographic maps to draw the soil profile shown in Fig. 5-3. Most of the foundation ground is loose sand with 2- to 4m- thick lenses of very soft clay sandwiched within. In the vicinity of Swedish sounding GSW-12, a thick layer of very soft clay with a total thickness of 15m was found.

5.2.2 Setapak

Setapak, located in the northeastern sector of the Federal Territory, is the second most northerly site investigated. The earthwork for the construction of ten 5-storey flats which comprise Phase I of the construction scheme had just been completed. Ten borings had previously been performed at the Phase I site by the Kuala Lumpur City Hall. Therefore, the study team investigated the areas of the site where borings had not been performed.

In the investigation, Swedish soundings and borings were carried out at 15 locations and 4 locations respectively. Fig. 5-4 shows the site layout and all the locations of soundings and borings. Figs. 5-5a to 5-5c are

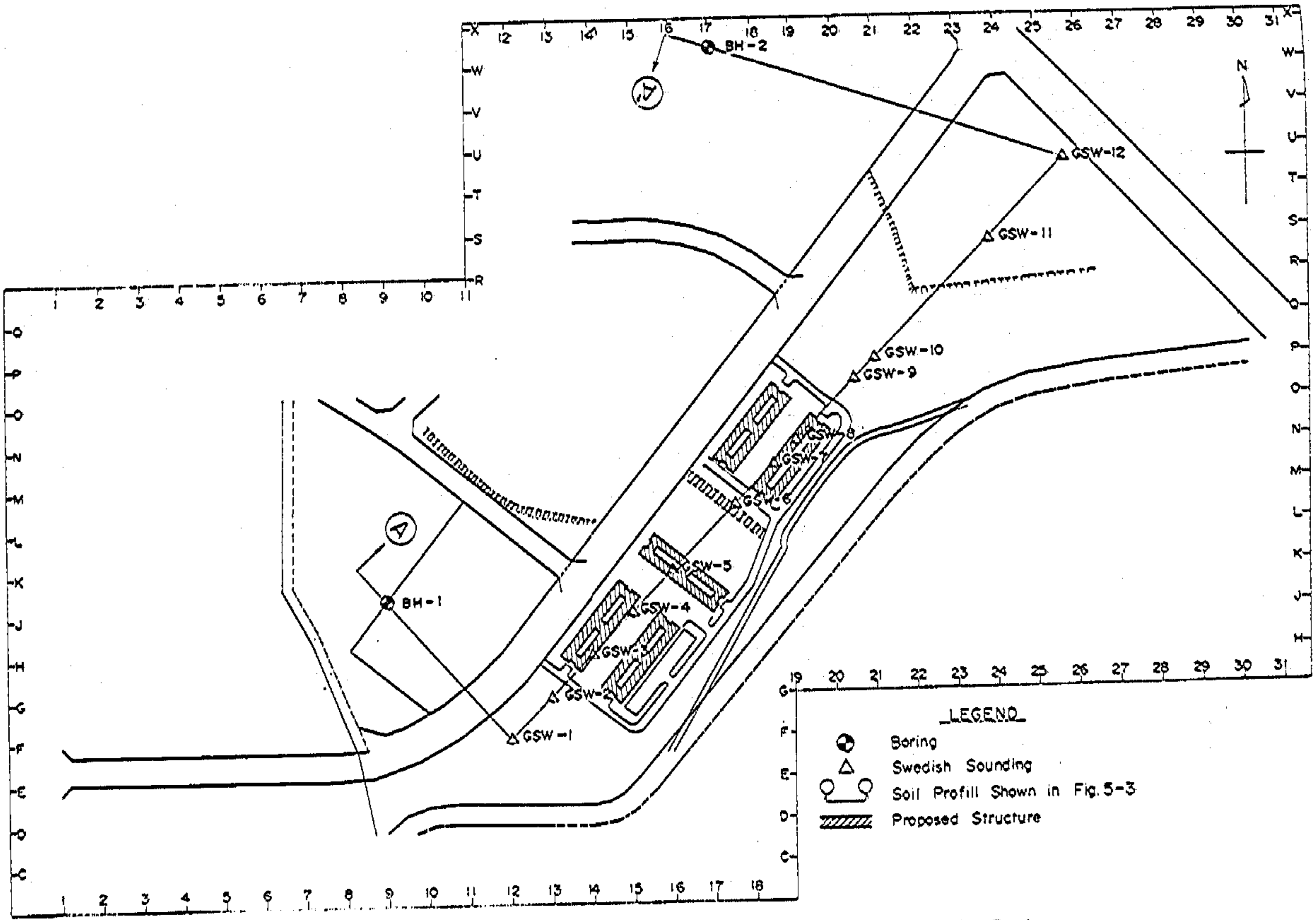


Fig. 5-2 Site Layout and Location of Soundings and Borings
— Gormbak —

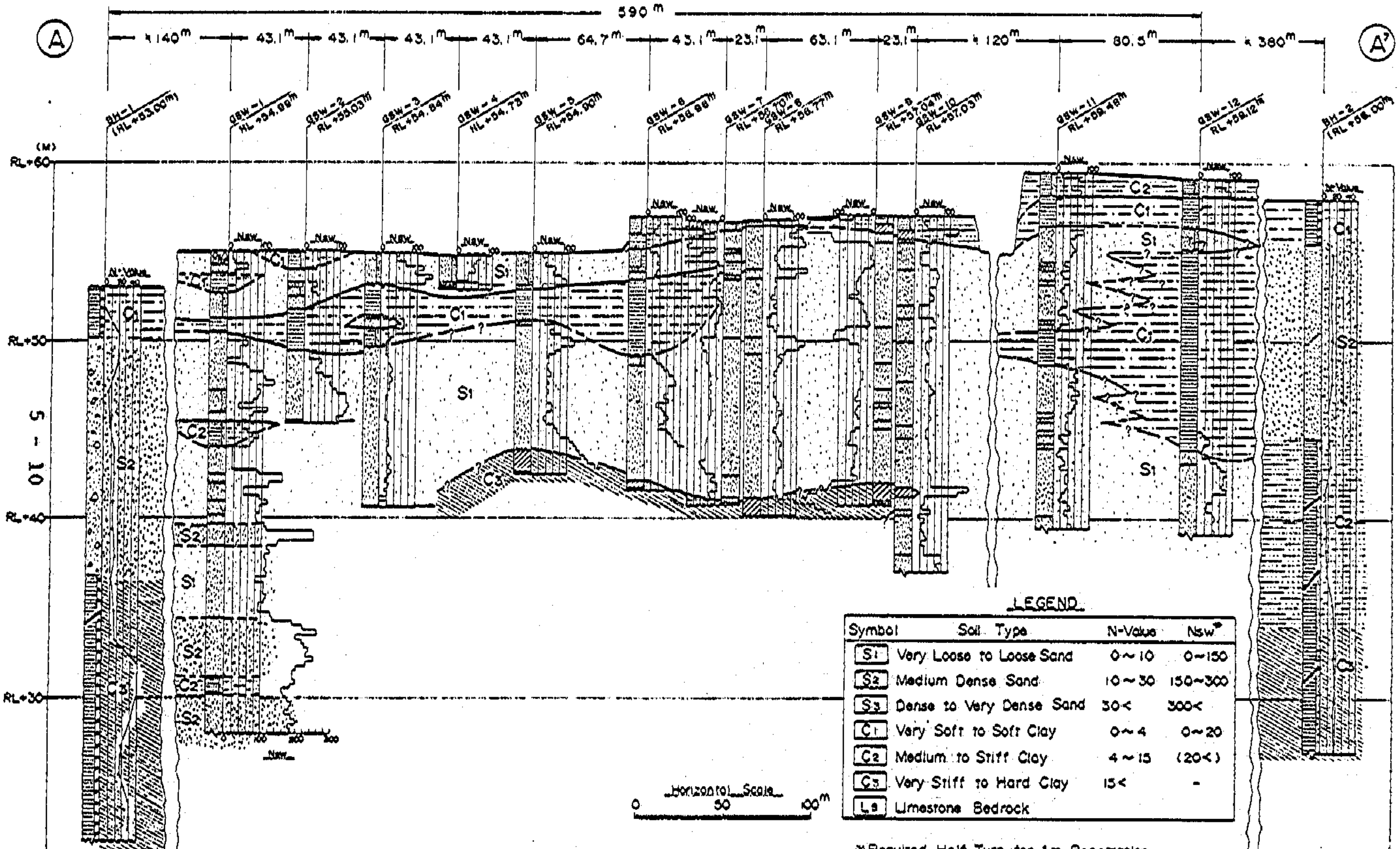
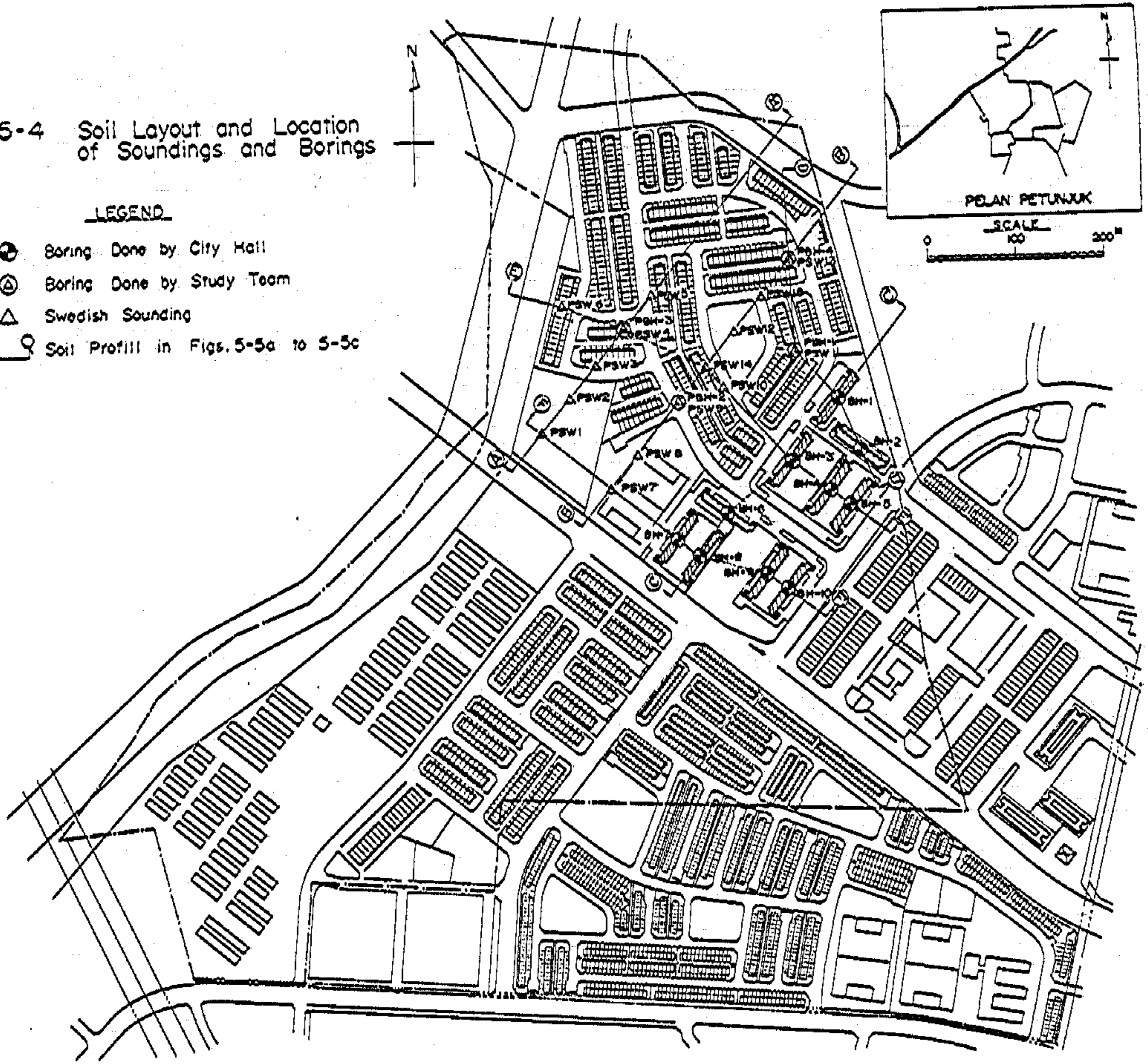


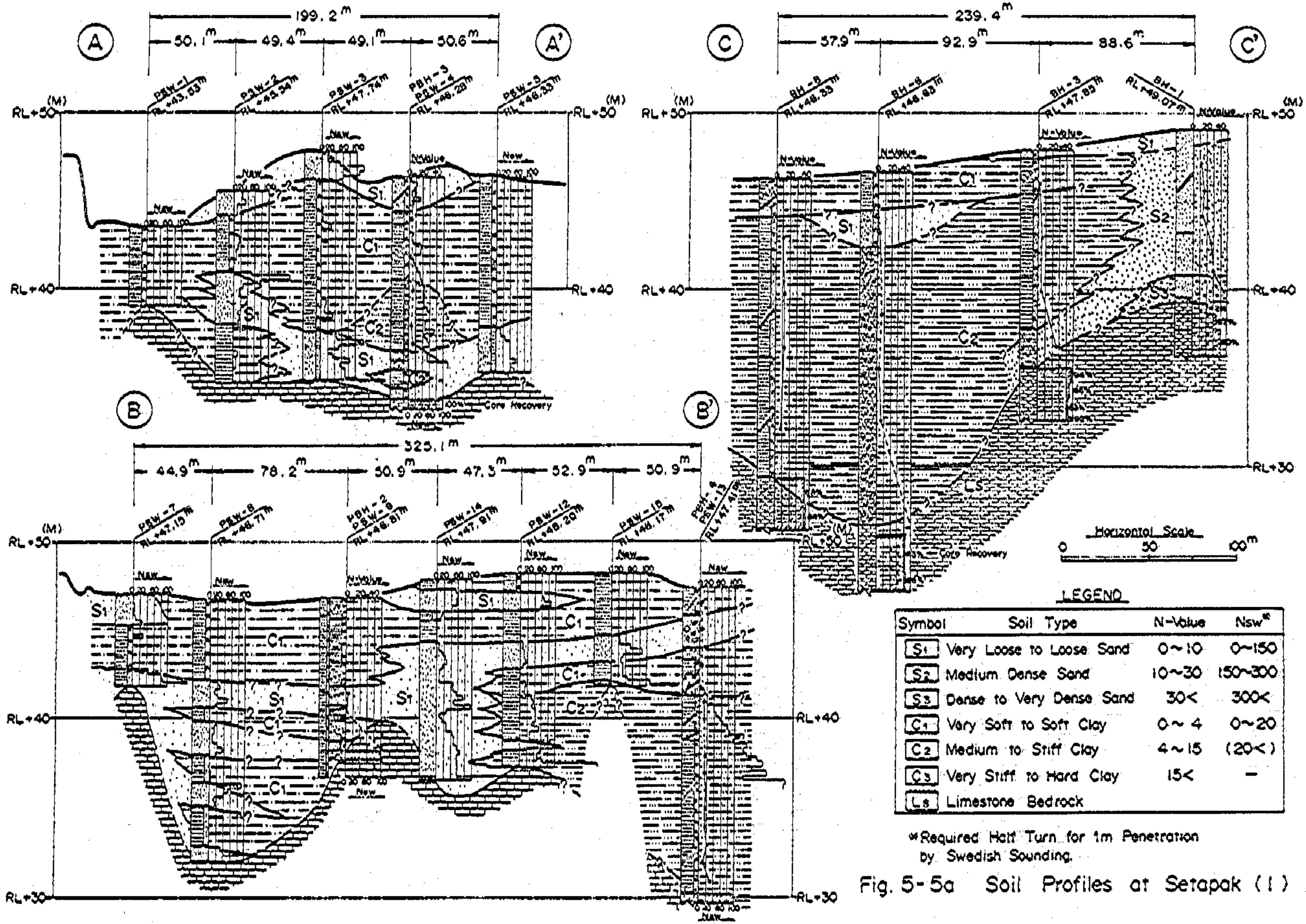
Fig. 5-3 Soil Profile at Gombak

Fig. 5-4 Soil Layout and Location of Soundings and Borings

LEGEND

- Boring Done by City Hall
- Boring Done by Study Team
- Swedish Sounding
- Soil Profill in Figs. 5-5a to 5-5c



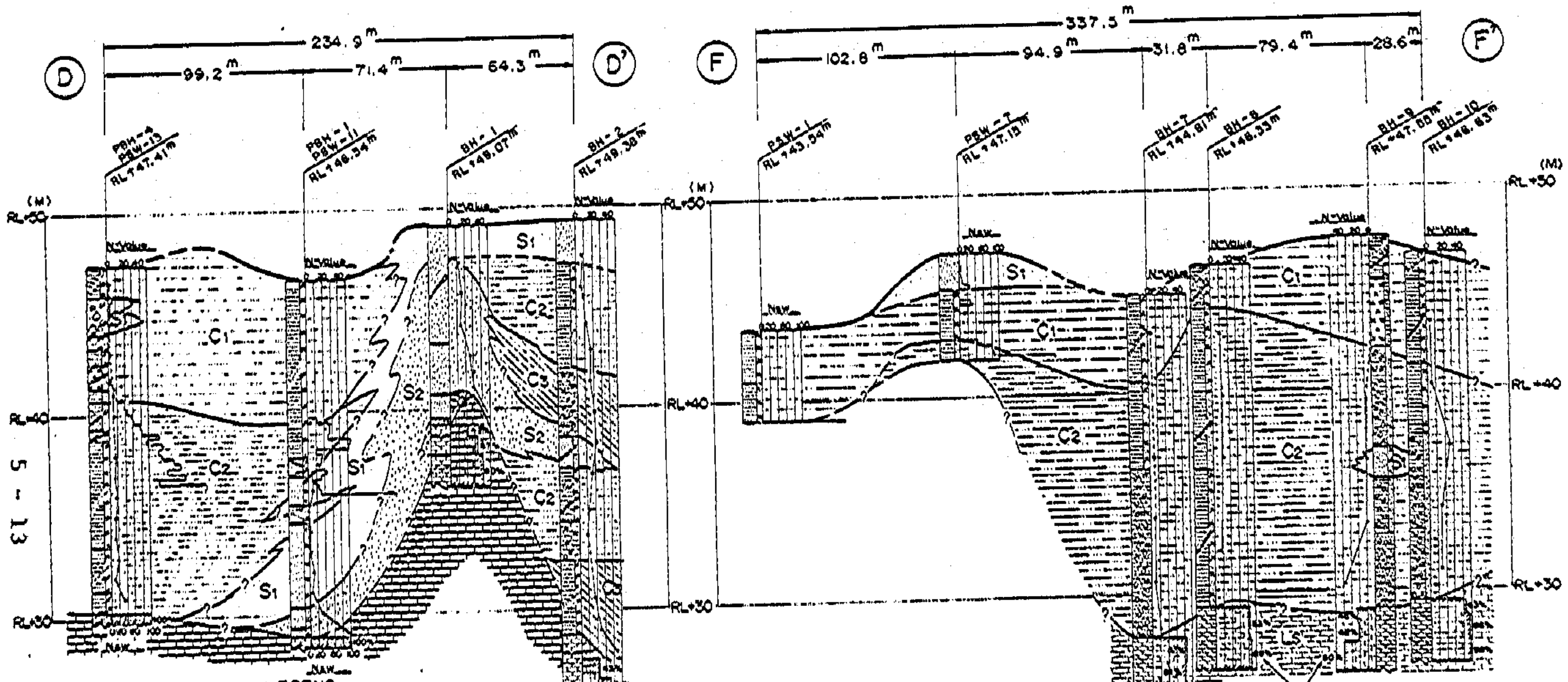


LEGEND

Symbol	Soil Type	N-Value	Nsw*
S1	Very Loose to Loose Sand	0~10	0~150
S2	Medium Dense Sand	10~30	150~300
S3	Dense to Very Dense Sand	30<	300<
C1	Very Soft to Soft Clay	0~4	0~20
C2	Medium to Stiff Clay	4~15	(20<)
C3	Very Stiff to Hard Clay	15<	–
Ls	Limestone Bedrock		

*Required Half Turn for 1m Penetration by Swedish Sounding.

Fig. 5-5a Soil Profiles at Setapak (1)



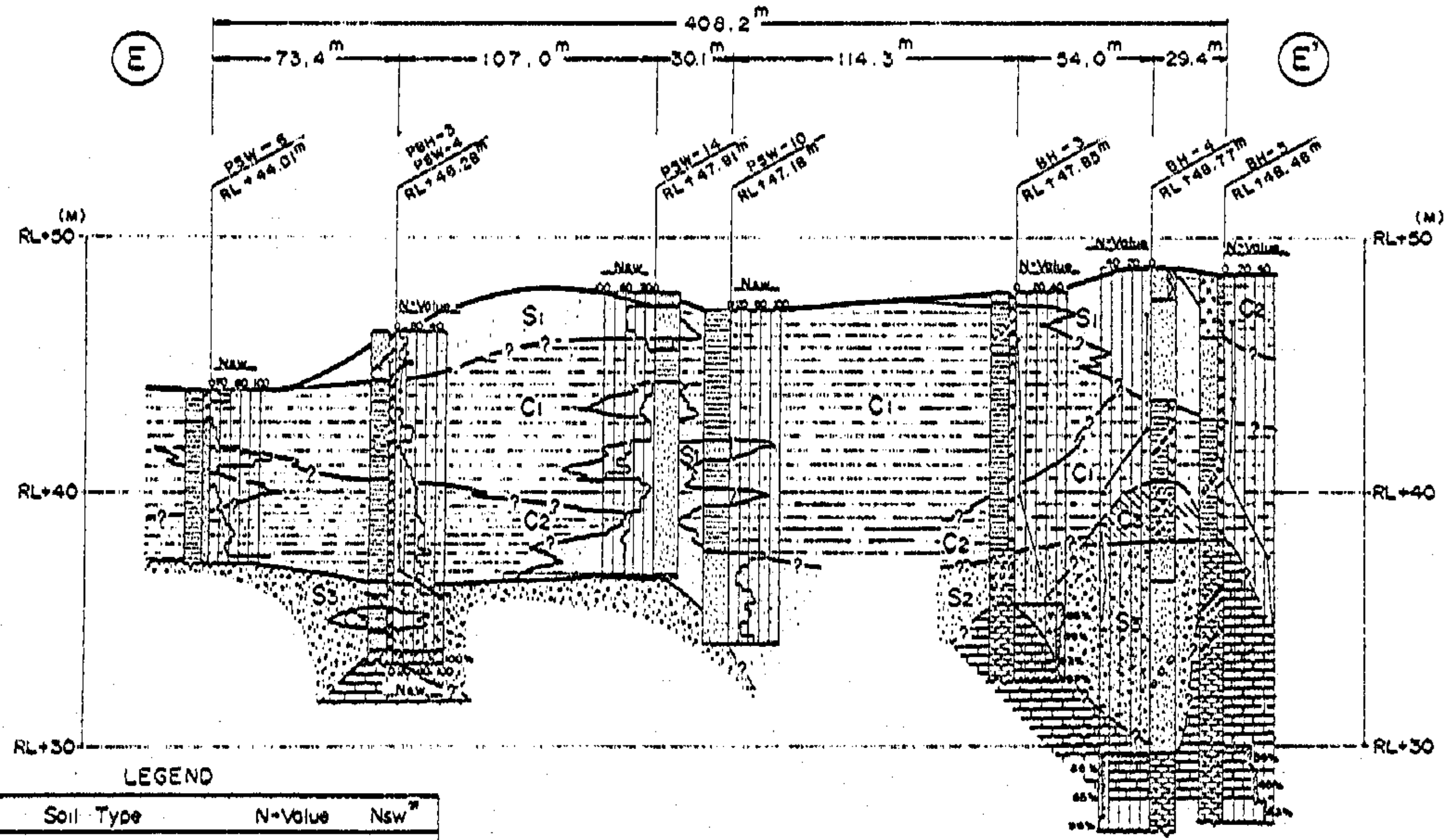
LEGEND

Symbol	Soil Type	N-Value	Nsw*
S1	Very Loose to Loose Sand	0~10	0~150
S2	Medium Dense Sand	10~30	150~300
S3	Dense to Very Dense Sand	30<	300<
C1	Very Soft to Soft Clay	0~4	0~20
C2	Medium to Stiff Clay	4~15	(20<)
C3	Very Stiff to Hard Clay	15<	-
Ls	Limestone Bedrock		

*Required Half Turn for 1m Penetration by Swedish Sounding.

Horizontal Scale
0 50 100m

Fig. 5-5b. Soil Profiles at Setapak (2)



LEGEND

Symbol	Soil Type	N-Value	Nsw*
S1	Very Loose to Loose Sand	0~10	0~150
S2	Medium Dense Sand	10~30	150~300
S3	Dense to Very Dense Sand	30<	300<
C1	Very Soft to Soft Clay	0~4	0~20
C2	Medium to Stiff Clay	4~15	(20<)
C3	Very Stiff to Hard Clay	15<	-
Ls	Limestone Bedrock		

* Required Half Turn for 1m Penetration by Swedish Sounding.

Horizontal Scale
0 50 100m

Fig. 5-5c Soil Profile at Setapak (3)

soil profiles of the site which were compiled after referring to the mining records and old topographic maps, etc.

Soft clayey soils are widely distributed near the ground surface throughout the site, underlying rather thick deposits of medium to stiff sandy or gravelly clay. Some loose sand layers are interbeded in the clayey soil layer, occasionally in the shape of lenses. The top of the limestone bedrock is found to undulate to an extreme degree as is usual in ex-mining land.

In the course of exploratory boring, 16 undisturbed soil samples were taken, mainly from soft clayey soils. Laboratory soil tests were performed on these samples with the results being summarized in Table 5-4.

Figs. 5-6 and 5-7 show the plottings of physical properties of soft clayey soil versus depth. Fig. 5-8 shows a classification of soils on a plasticity chart. Fig. 5-9 shows undrained shear strength, preconsolidation pressure and compression index versus depth. The major ranges of the parameters of soft clayey soil and some notes on them are summarized in Table 5-5. Fig. 5-10 shows e-log p curves of the soft clays and Fig. 5-11 shows coefficient of consolidation versus consolidation pressure curves.

Table 5-4 Summary of Soil Tests on Samples from Setapak

Boring No.	PBH-1										PBH-2					PBH-3		PBH-4				
Sample No.	UD-1	UD-2	UD-3	UD-4	UD-5	UD-6 Top	UD-6 Bottom	UD-7 Top	UD-7 Middle	UD-7 Bottom	UD-1 Top	UD-1 Middle	UD-1 Bottom	UD-2	UD-3	UD-1	UD-2	UD-2 Top	UD-2 Middle	UD-2 Bottom	UD-3	UD-4
Sample depth	0.50m 1.30m	2.00m 2.80m	3.50m 4.30m	5.00m 5.80m	5.80m 6.60m	7.00m 7.35m	7.35m 7.80m	8.50m 8.65m	8.65m 8.80m	8.80m 9.10m	1.00m 1.20m	1.20m 1.45m	1.45m 1.85m	4.00m 4.85m	7.00m 7.85m	3.00m 3.80m	7.00m 7.80m	3.00m 3.15m	3.15m 3.35m	3.35m 3.85m	5.00m 5.85m	8.00m 8.50m
Condition of sample*	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD	UD
Natural water content, %	61.8	76.1	46.0	40.4	44.2	41.9	67.4	63.4	29.0	22.0	50.0	40.3	61.2	44.8	45.4	26.9	33.0	39.1	28.1	36.7	21.0	32.2
Specific gravity	2.593	2.639	2.657	2.640	2.666	2.634	2.651	2.620	2.622	2.626	2.560	2.692	2.647	2.643	2.619	2.628	2.865	2.644	2.673	2.664	2.630	2.655
Wet density, g/cm³	1.62	1.58	1.78	1.78	1.77	1.79	1.58	1.66	1.92	2.02	1.69	1.78	1.60	1.79	1.76	1.95	1.94	1.81	1.90	1.86	2.04	1.86
Dry density, g/cm³	1.00	0.90	1.22	1.27	1.23	1.26	0.94	1.02	1.49	1.66	1.13	1.27	0.99	1.24	1.21	1.54	1.46	1.30	1.48	1.45	1.69	1.41
Natural void ratio	1.59	1.94	1.18	1.08	1.17	1.09	1.81	1.58	0.76	0.59	1.27	1.12	1.67	1.14	1.16	0.71	0.96	1.03	0.80	0.96	0.56	0.89
Degree of saturation, %	100	100	100	99	100	100	99	100	100	99	100	97	97	100	100	100	98	100	93	100	99	97
Atterberg limits: Liquid limit, %	50.5	52.7	61.8	56.5	44.7	45.6	87.6	82.4	41.9	-	58.5	33.3	61.0	43.8	53.8	61.0	66.6*	42.0	-	35.1	41.8	81.0*
Atterberg limits: Plastic limit, %	22.6	21.1	21.9	21.7	17.2	17.8	32.1	30.3	18.3	-	27.4	16.2	23.1	16.7	20.4	22.8	26.9	17.2	-	15.6	14.3	25.8
Atterberg limits: Plasticity index	27.9	31.6	39.9	34.8	27.5	27.8	55.5	52.1	23.6	-	31.1	17.2	37.9	27.1	33.4	38.2	39.7	24.8	-	19.5	27.5	55.2
Grain size analysis: Gravel, %	0	0	0	0	0	0	0	0	0	2	1	4	0	0	0	0	12	0	27	0	0	3
Grain size analysis: Sand, %	2	30	18	25	40	23	6	0	34	63	23	45	4	16	1	7	27	31	46	5	54	27
Grain size analysis: Silt, %	30	28	32	32	22	25	19	17	17	19	25	23	38	40	39	39	15	29	11	60	18	13
Grain size analysis: Clay & colloid, %	68	42	50	44	38	52	75	83	49	16	51	28	58	44	60	54	46	40	16	35	28	57
Grain size analysis: Max. diameter, mm	0.250	0.420	0.420	0.420	0.420	0.420	0.420	0.074	2.00	4.76	4.76	9.52	0.420	0.420	0.105	0.42	9.52	0.42	9.52	0.25	2.00	4.76
Grain size analysis: Diam. at 60%	0.0019	0.052	0.024	0.041	0.072	0.019	-	-	0.046	0.19	0.016	0.10	0.0065	0.022	0.0053	0.0087	0.058	0.053	0.64	0.022	0.12	0.0089
Grain size analysis: Diam. at 10%	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Visual soil description	Silty clay	Silty clay w/sand	Silty clay w/sand	Silty clay w/sand	Silty clay w/sand	Silty clay	Silty clay	Silty clay	Silty clay	Silty sand	Silty clay	Silty clay	Silty clay	Silty clay	Silty clay		Silty clay				Clayey sand	Silty clay
Unified soil classification	CH	CH	CH	CH	CL	CL	CH	CH	CL	SC	CH	SC	CH	CL	CH	CH	CH	CL	(SC)	CL	SC	CH
Triaxial compression test: Angle of internal friction	0°	0°	0°	0°	0°	0°	0°	-	-	-	-	-	-	0°	0°	0°	0°	-	-	-	0°	0°
Triaxial compression test: Cohesion, kg/cm2	0.05	0.04	0.06	0.06	0.08	0.09	0.10	-	-	-	-	-	-	0.06	0.20	0.75	0.37	-	-	-	0.69	0.72
Triaxial compression test: Condition of drainage	u-u	u-u	u-u	u-u	u-u	u-u	u-u	-	-	-	-	-	-	u-u	u-u	u-u	u-u	-	-	-	u-u	u-u
Consolidation test: Preconsolid. Press. kg/cm2	(0.26)	-	(0.30)	(0.48)	-	-	-	-	-	-	-	-	-	-	0.90			-	-			
Consolidation test: Compression index	0.08	-	0.32	0.32	0.30	-	0.26	-	-	0.055	-	-	-	0.27	0.42			-	-			

Remarks : * UD denotes undisturbed samples.

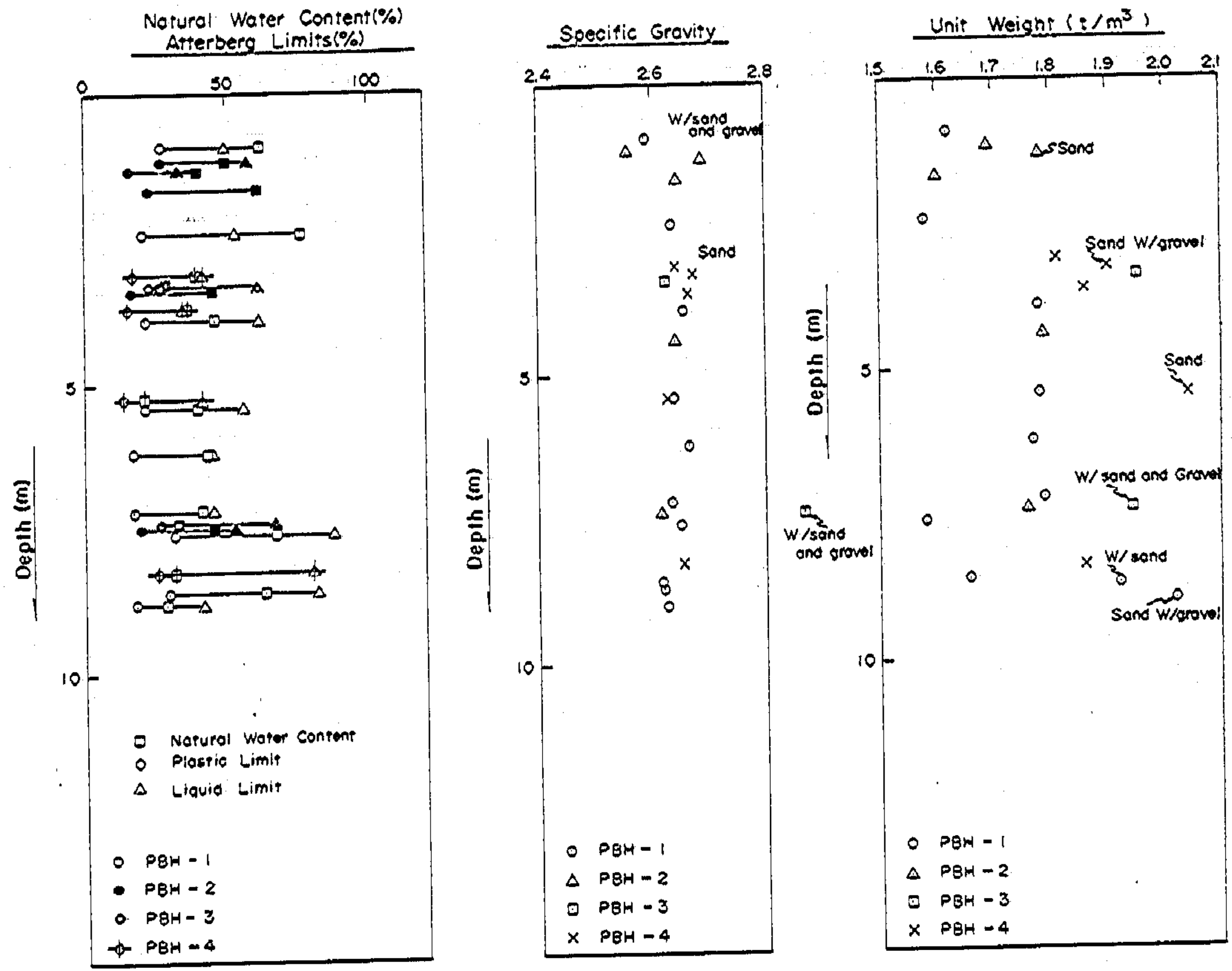


Fig. 5 – 6 Depth v.s. Physical Properies of Sample from Setapak

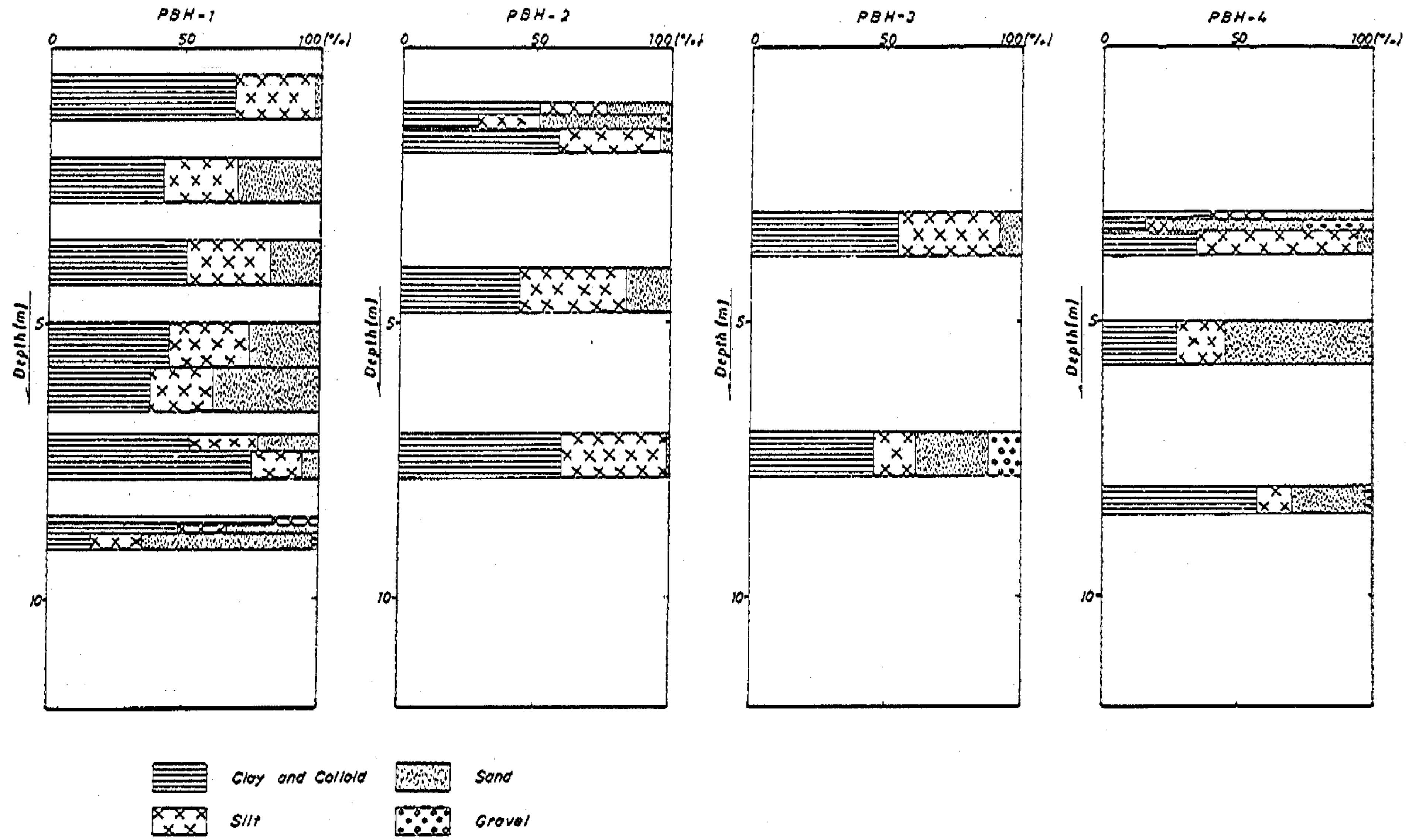
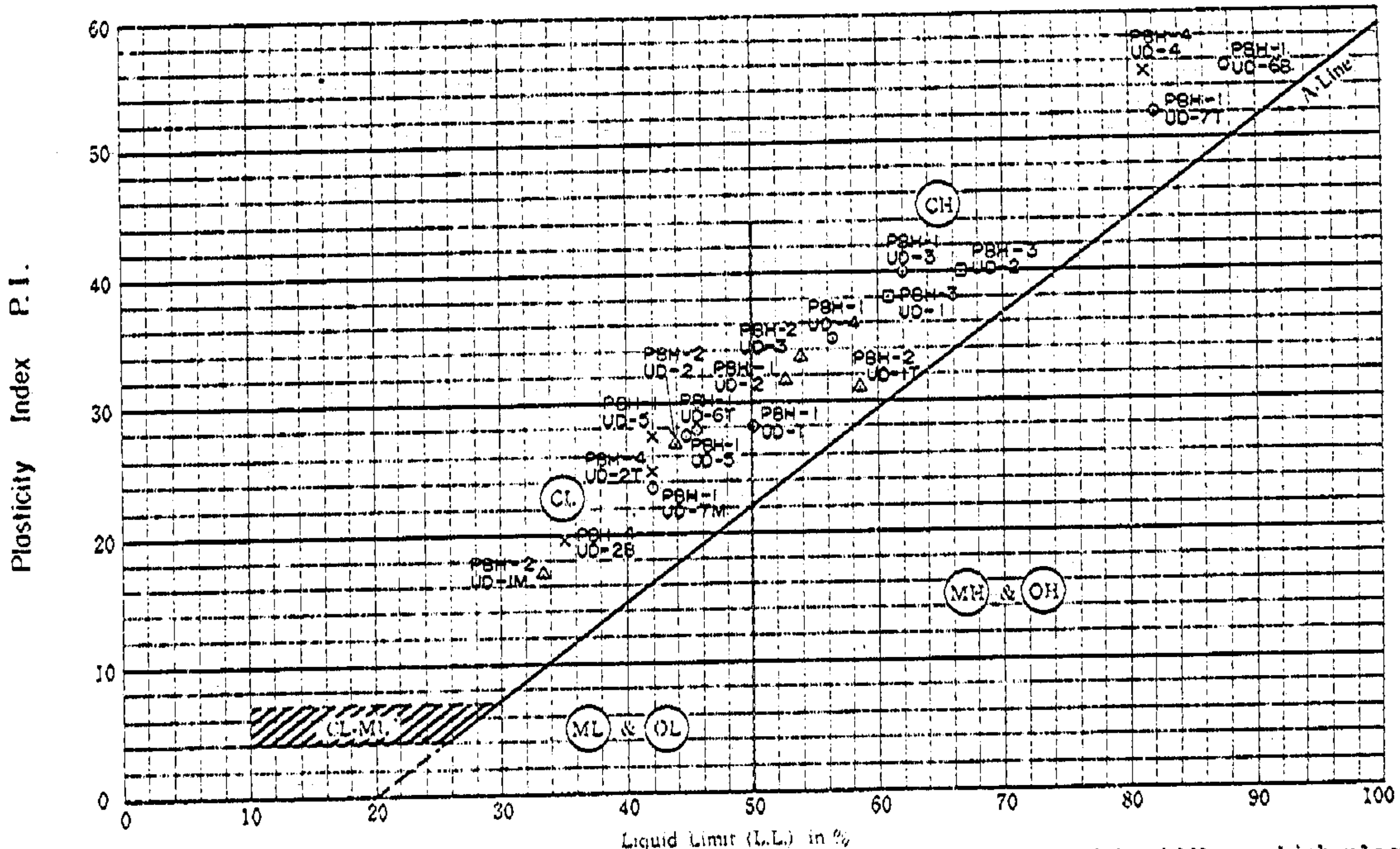


Fig. 5-7 Grading Texture of Samples from Setapak



ML: Inorganic silt with some plasticity, very fine sand, rock flour, silty or clayey fine sand and clayey silt

CL: Clay containing gravels with low or middle plasticity sandly clay and silty clay

OL: Organic silt with low plasticity and organic silty clay

MH: Inorganic silt with middle or high plasticity, micaceous or diatomaceous fine sandy soil and silty soil

CH: Inorganic clay with high plasticity

OH: Organic clay with middle or high plasticity and organic silt

Fig. 5 - 8 Classification of Samples from Setapak by Plasticity Chart

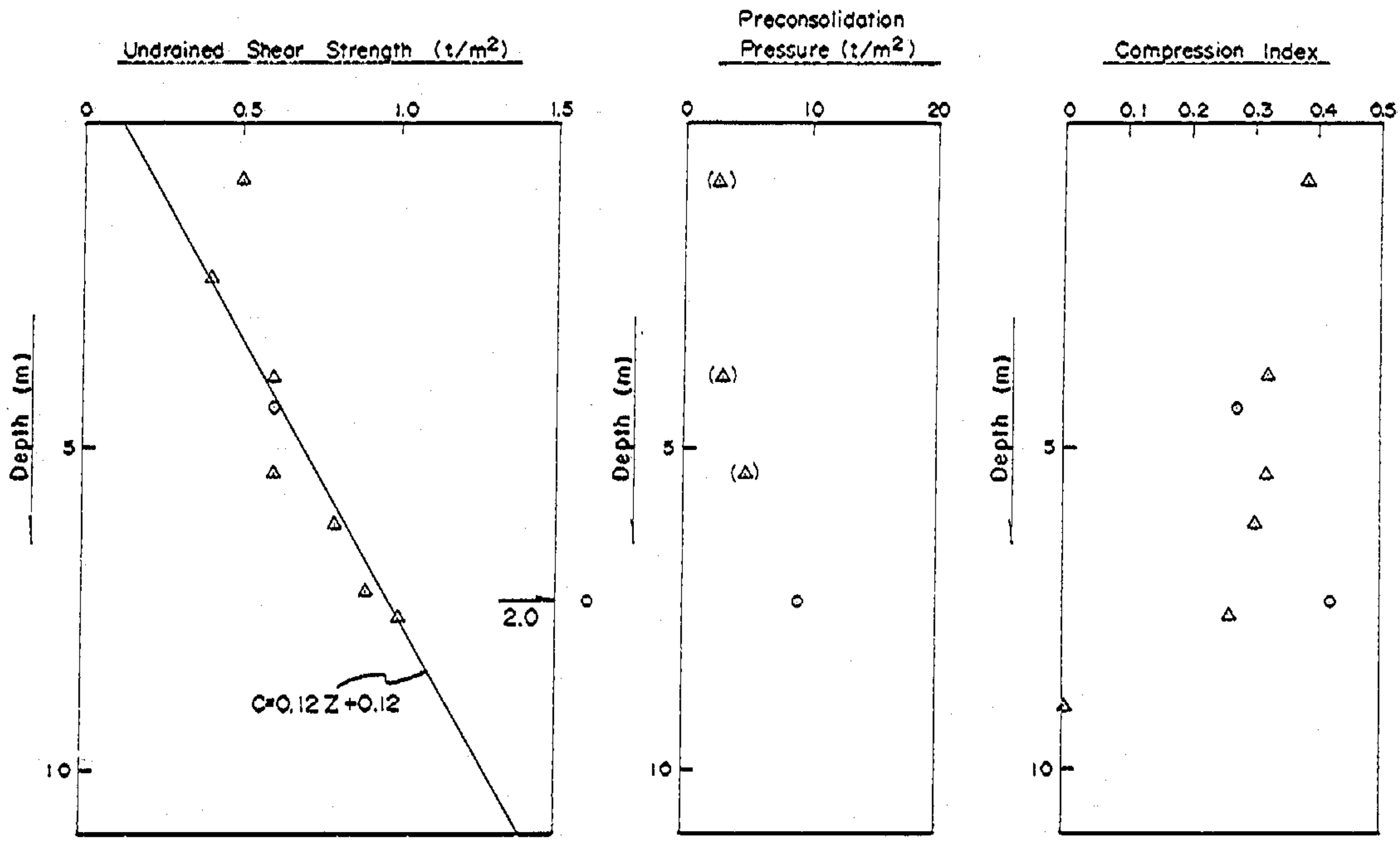


Fig. 5 - 9 Depth v.s. Undrained Shear Strength, Preconsolidation Pressure and Compression Index — Setapak —

Table 5-5 Physical and Mechanical Properties of Soft Clayey Soils of Setapak

Item	Major Range	Tendency
Physical Properties		
Natural Water Content	20 ∿ 80%	Decrease with Depth
Specific Gravity	2.6 ∿ 2.7	Almost Constant
Wet Density	1.6 ∿ 1.8 t/m^3	Scattered in Wide Range
Liquid Limit	30 ∿ 80%	Scattered in Wide Range
Plastic Limit	15 ∿ 30%	Almost Constant
Gravel Content	0 ∿ 10%	
Sand Content	0 ∿ 45%	
Silt Content	15 ∿ 60%	
Clay and Colloid Content	35 ∿ 75%	
Unified Soil Classification	CL or CH	
Mechanical Properties		
Undrained Shear Strength	0.2 ∿ 1.0 t/m^2	Increase with Depth $C \doteqdot 0.12Z + 0.12\ t/m^2$
Preconsolidation Pressure	2 ∿ 10 t/m^2	Increase with Depth
Compression Index	0.25∿0.40	

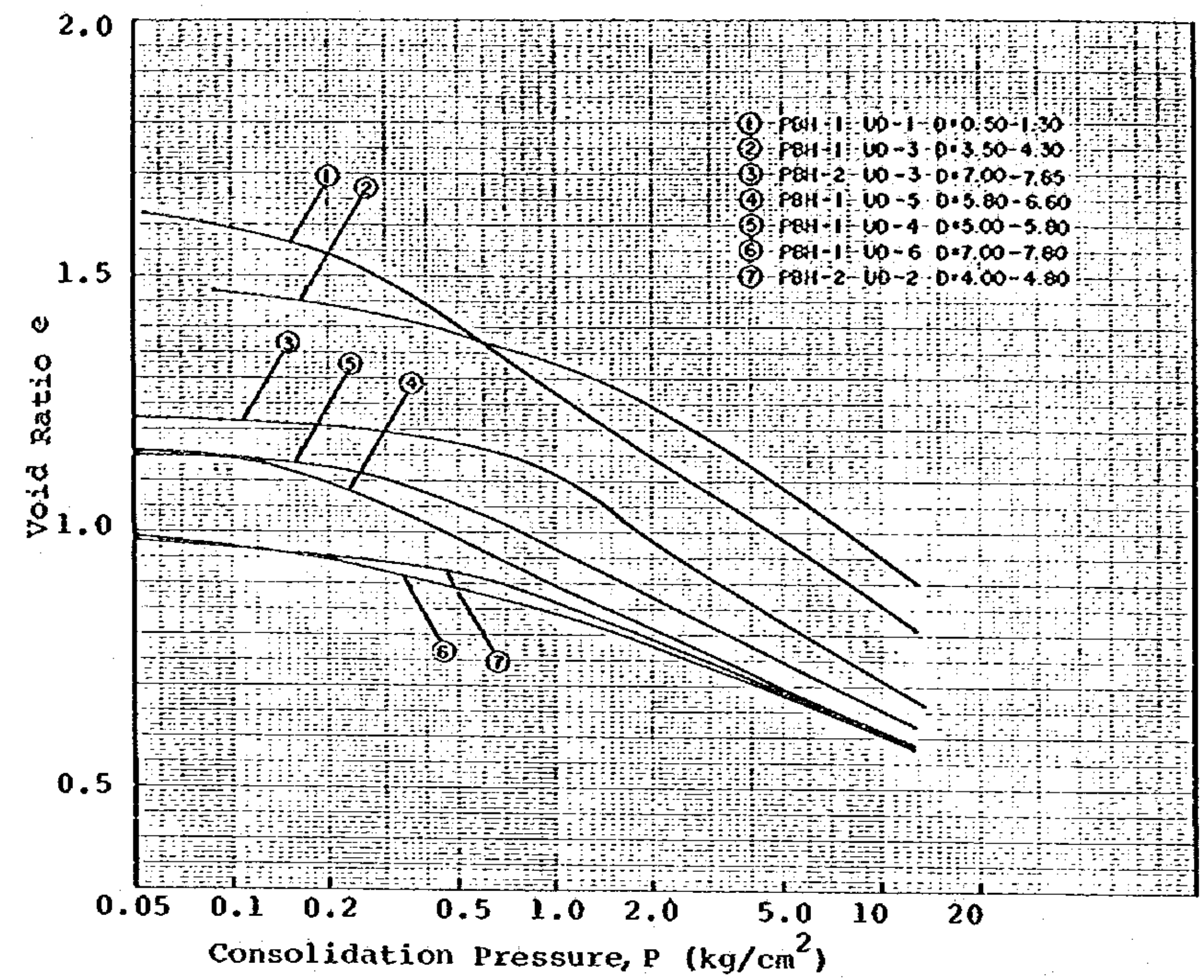


Fig. 5-10 e ~ Log P Curves - Setapak -

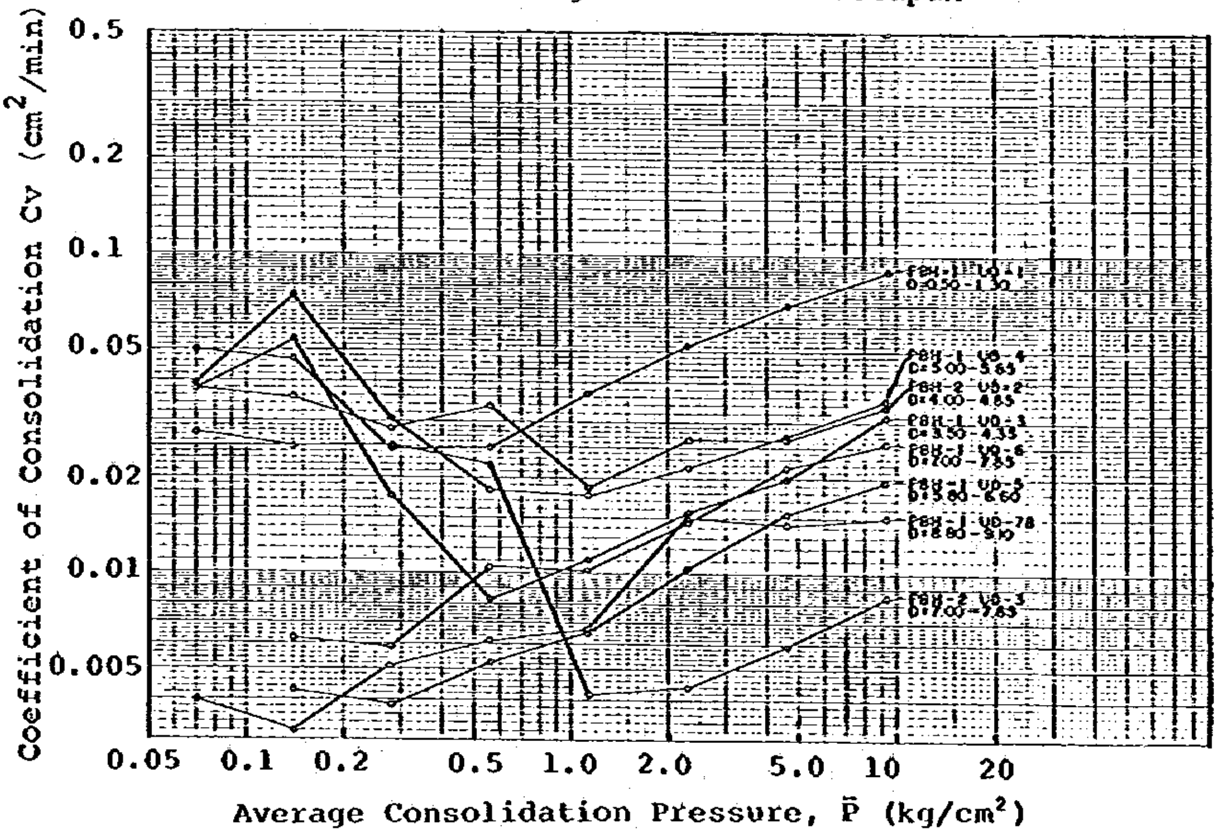


Fig. 5-11 Coefficient of Consolidation v.s. Average Consolidation Pressure - Setapak -

5.2.3 Cheras 3-1/2 Miles

The Cheras 3-1/2 miles site is on the northern suburbs of Chears Township, located in the centre of the Federal Territory. On the northwest side of this site, i.e. Phase 3A, four blocks of 18-storey flats have already been erected and, on the southeast side, i.e. Phase 3B, the pile foundations of five 18-storey flats are being constructed.

From the City Hall of Kuala Lumpur, the records of twenty-five borings previously performed were obtained. Fig. 5-12 shows the locations of the borings together with the layout of the flats. Fig. 5-13a and 5-13b are the soil profiles compiled from these records.

Above the major layers of stiff to hard clayey soils and medium to very dense sandy soils in the areas of Phases 3A and 3B, medium stiff clayey soils and loose sand are deposited. There is a high probability that the major layers of stiff to hard clayey soils and medium to very dense sandy soils are composed of Old Alluvium or weathered bedrock. The clayey soils at Phases 3B and 3C sites are somewhat stiffer than that at the other ex-mining sites, making the ground relatively superior as foundation ground. On the other hand soft clay and loose sand are distributed in the area of the Phase 3A site. The soft clay is particularly thick at the locations of boreholes BH-2, BH-7 and BH-8.

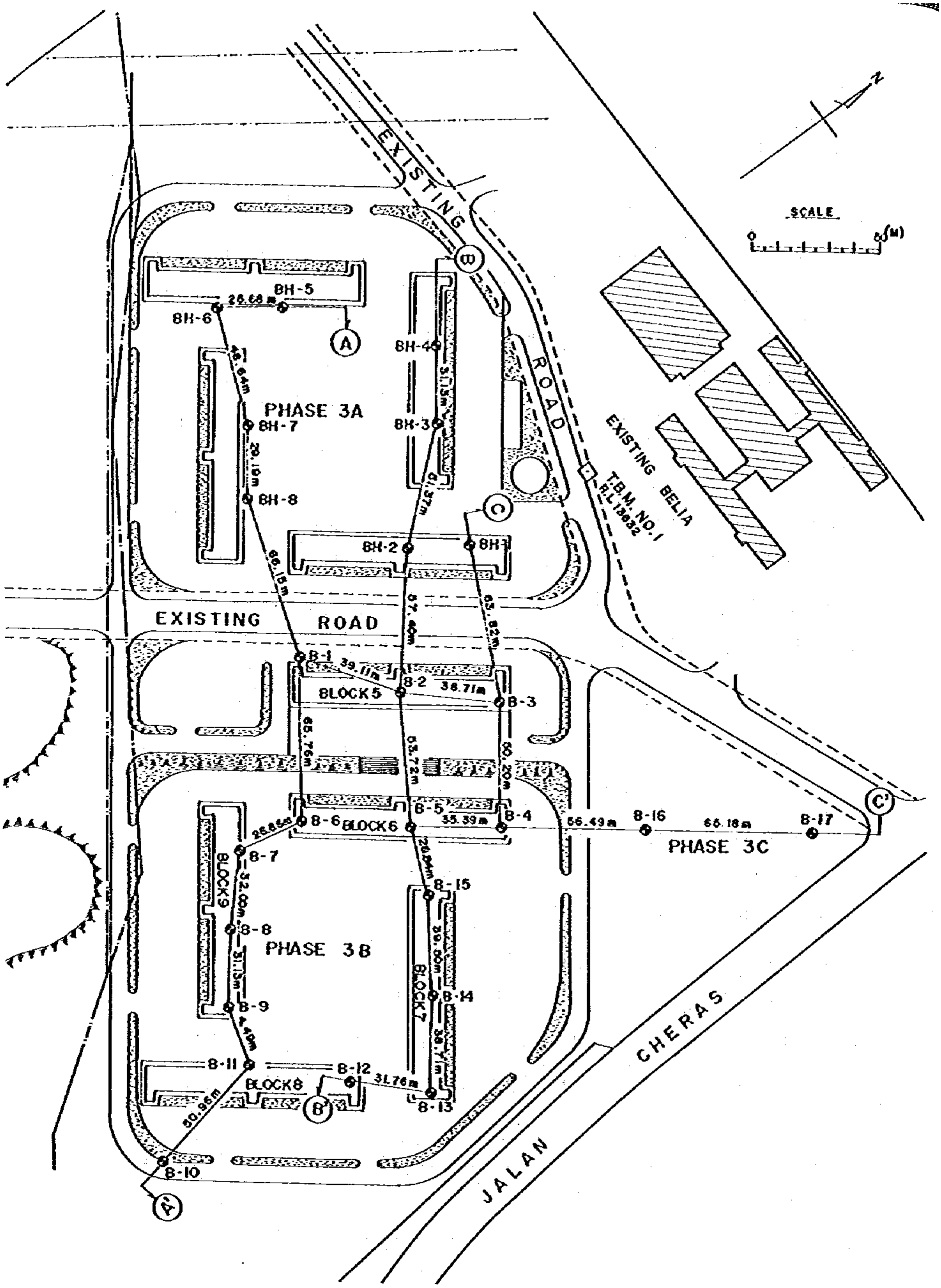


Fig. 5-12 Site Layout and Location of Borings
— Cheras 3½ miles —

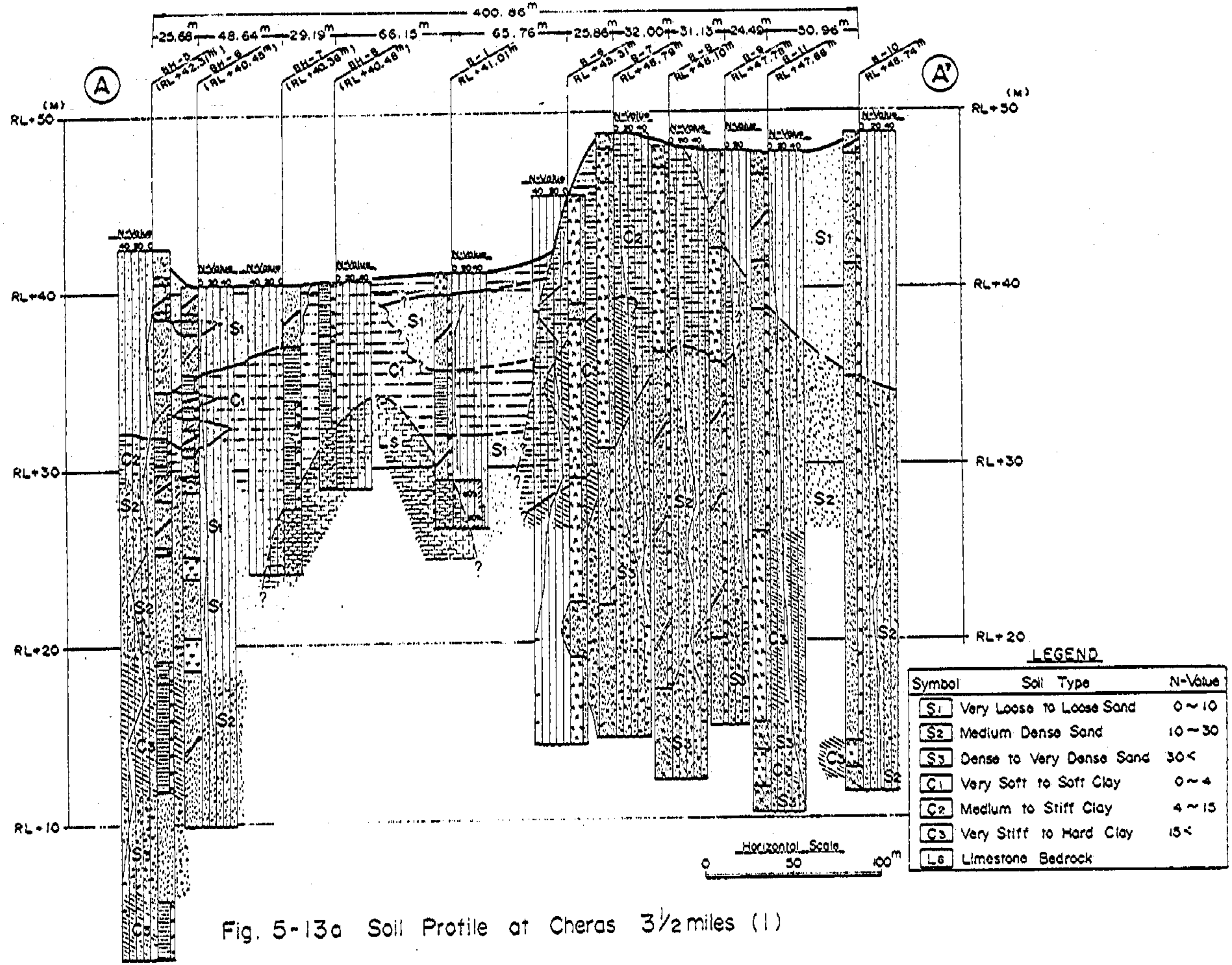


Fig. 5-13a Soil Profile at Cheras 3½ miles (1)

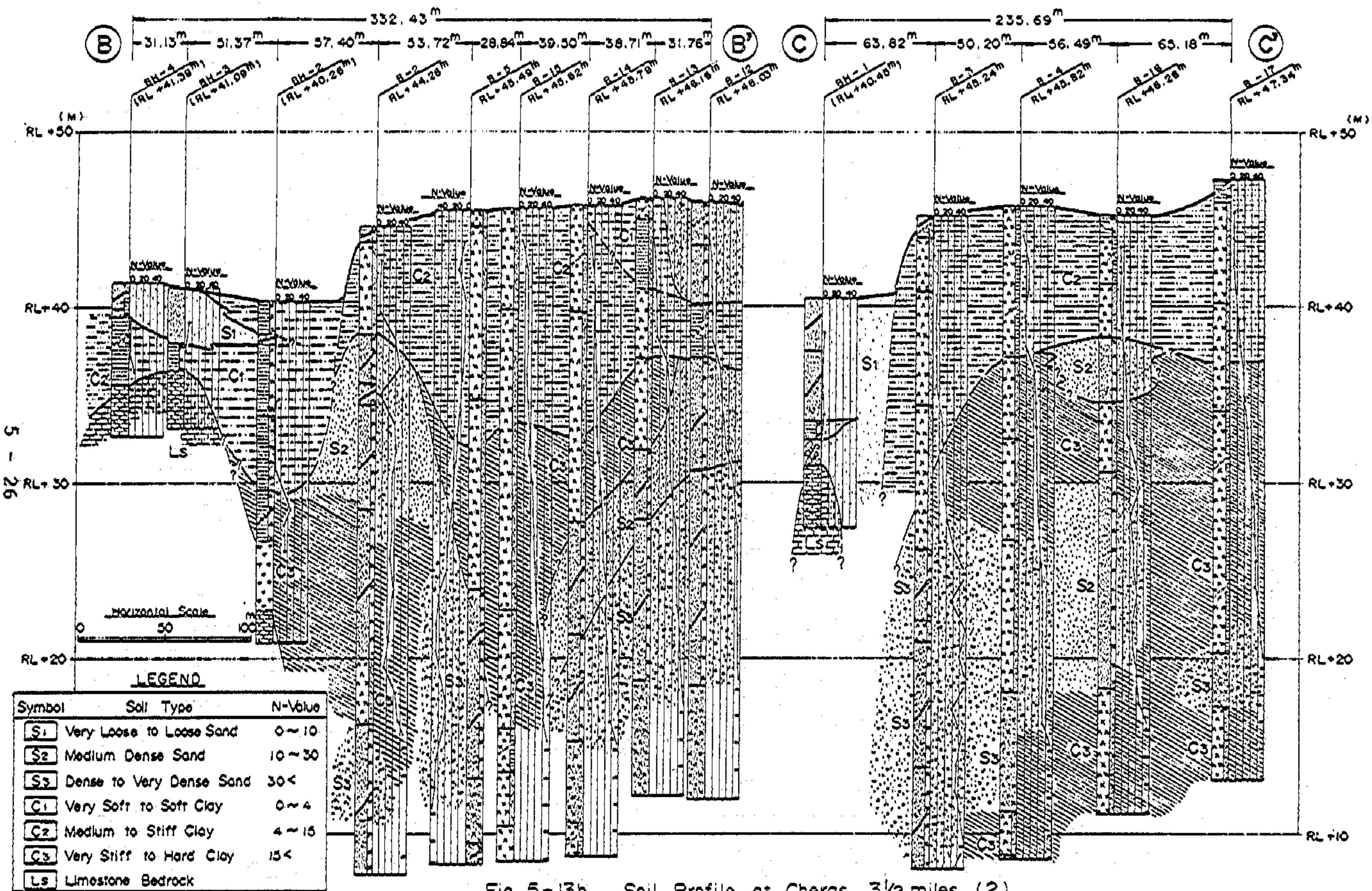


Fig. 5-13b Soil Profile at Cheras 3½ miles (2)

The surface configuration of the limestone bedrock is very undulating. The confirmed difference in the level of the limestone surface is more than 35m.

5.2.4 Cheras 4 Miles

A housing development of mainly 5-storey flats on the Cheras 4-miles site has been planned. The Cheras 4 miles site is located to the north of Cheras Township. Fig 5-14 shows the location of borings done by the Kuala Lumpur City Hall and the site layout.

According to the boring records, the site consists mainly of sandy soils and almost all the area was covered with loose sandy soils (Figs. 5-15a to 5-15c).

Soft clayey soils were found at 2 corners of the site. On the north side of the site, the ground surface consists of a layer of soft clay about 6m thick, whereas on the east side, the same consists of a soft clay layer about 15m in depth and is covered with loose sand.

5.2.5 Sungai Besi

The site of Sungai Besi, to the west of the Cheras Township, abuts the southern border of an old low-cost housing estate. For this site, data from 12 Mackintosh soundings and 6 borings were obtained from the City Hall. The locations are shown in Fig. 5-16. Fig. 5-17 is the soil profile of the site, which was compiled according to the

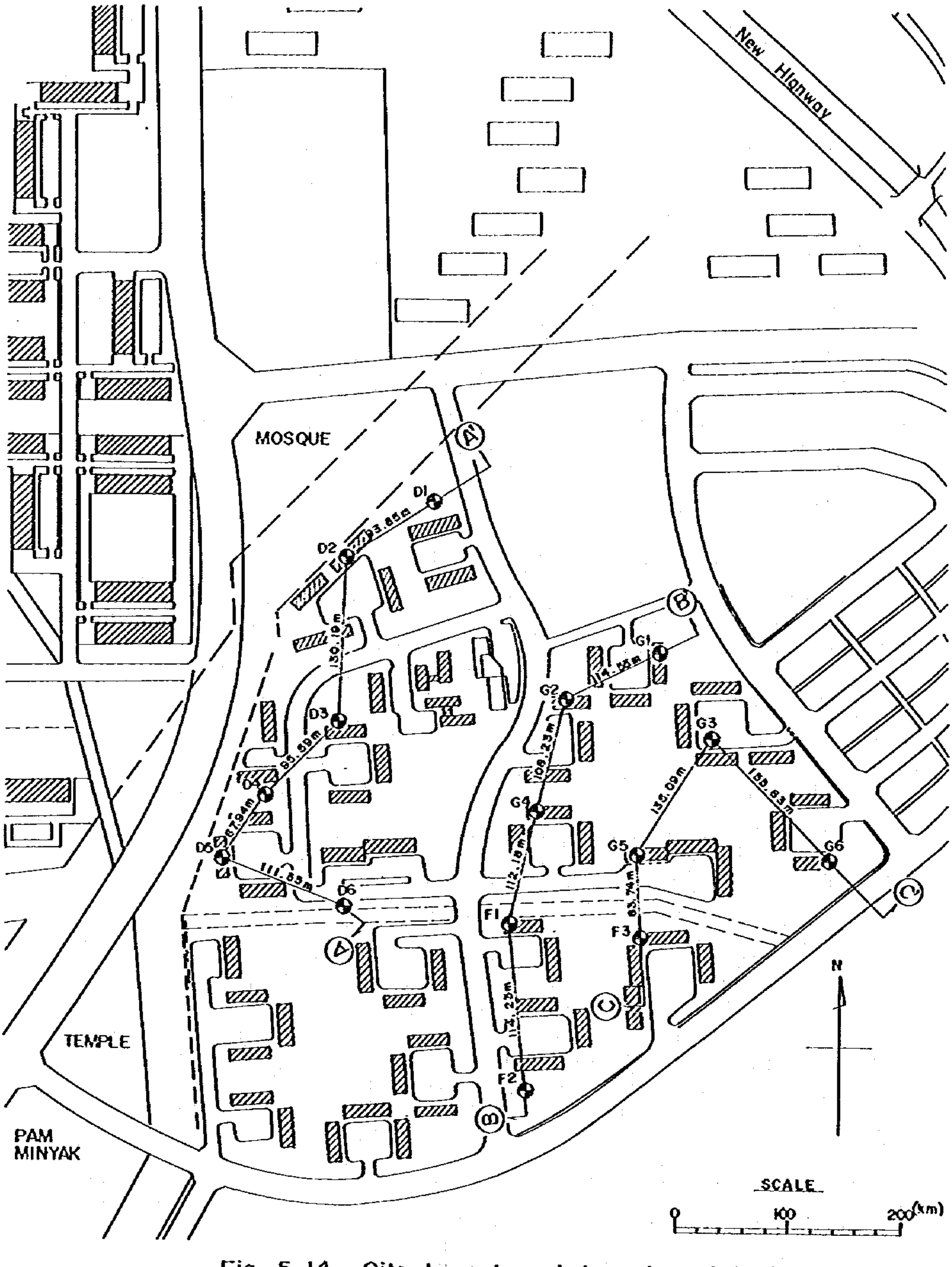


Fig. 5-14 Site Layout and Location of Borings
— Cheras 4mils —

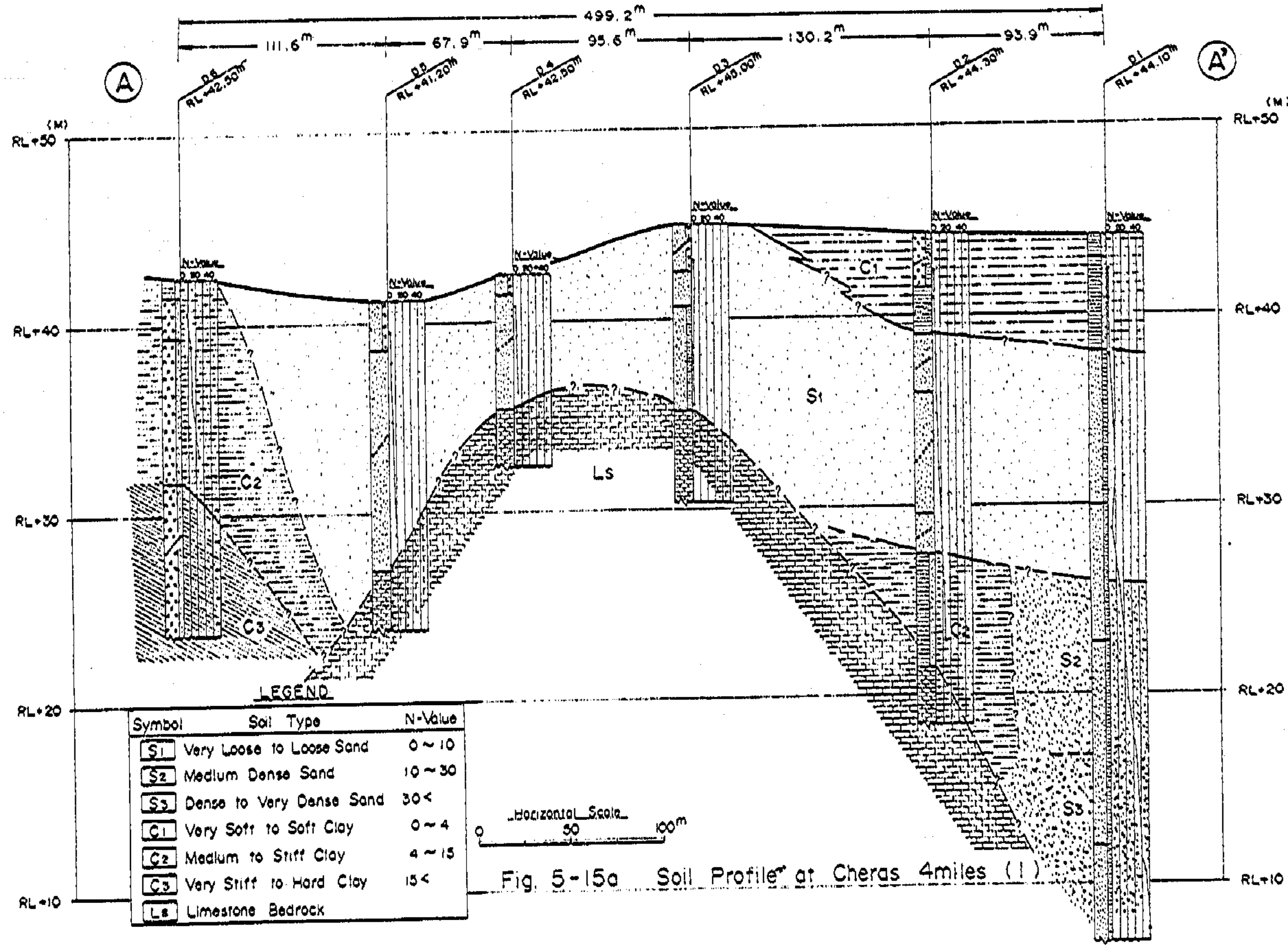
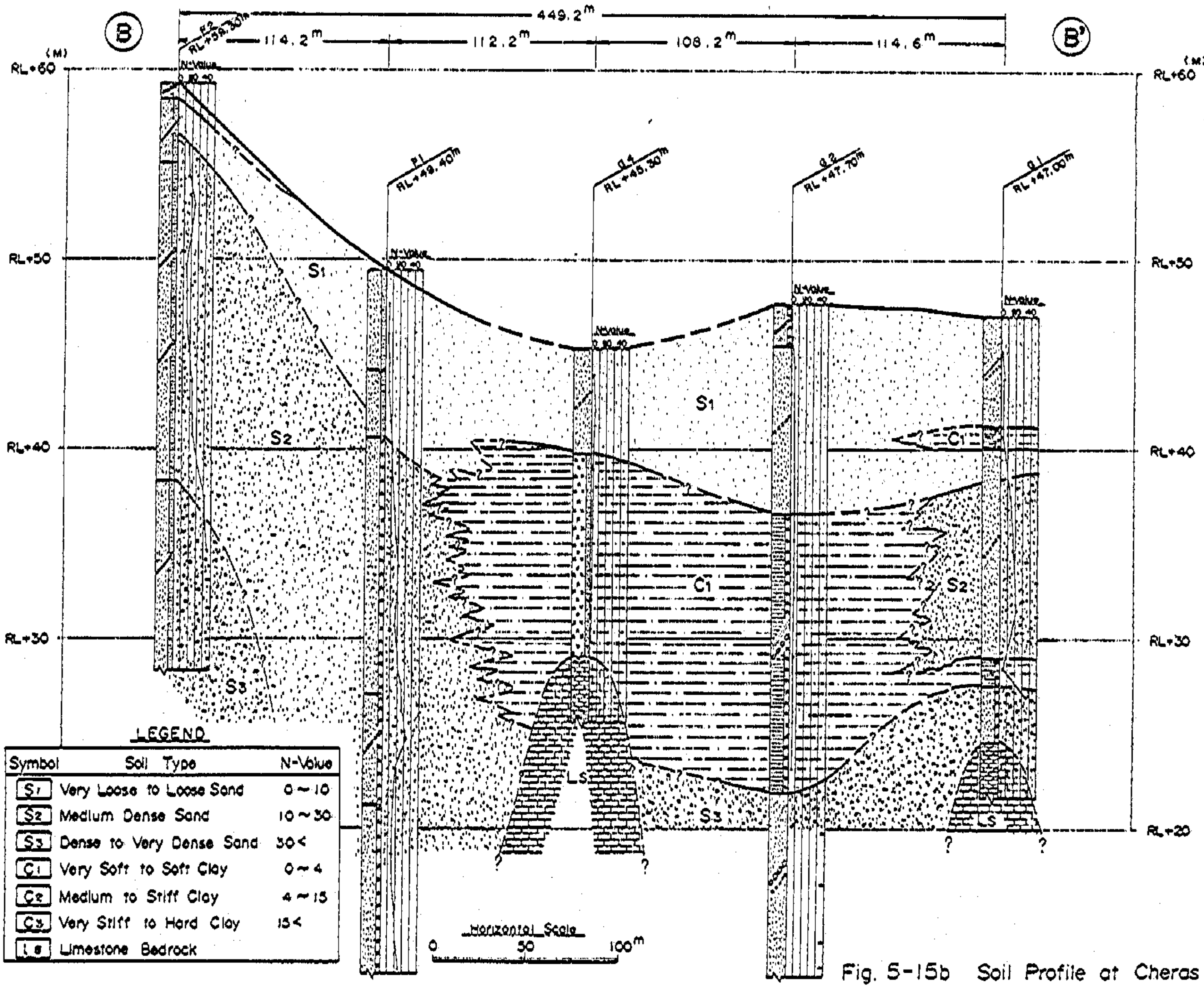


Fig. 5-15a Soil Profile at Cheras 4miles (1)



LEGEND

Symbol	Soil Type	N-Value
S1	Very Loose to Loose Sand	0 ~ 10
S2	Medium Dense Sand	10 ~ 30
S3	Dense to Very Dense Sand	30<
C1	Very Soft to Soft Clay	0 ~ 4
C2	Medium to Stiff Clay	4 ~ 15
C3	Very Stiff to Hard Clay	15<
Ls	Limestone Bedrock	

Fig. 5-15b Soil Profile at Cheras 4miles (2)

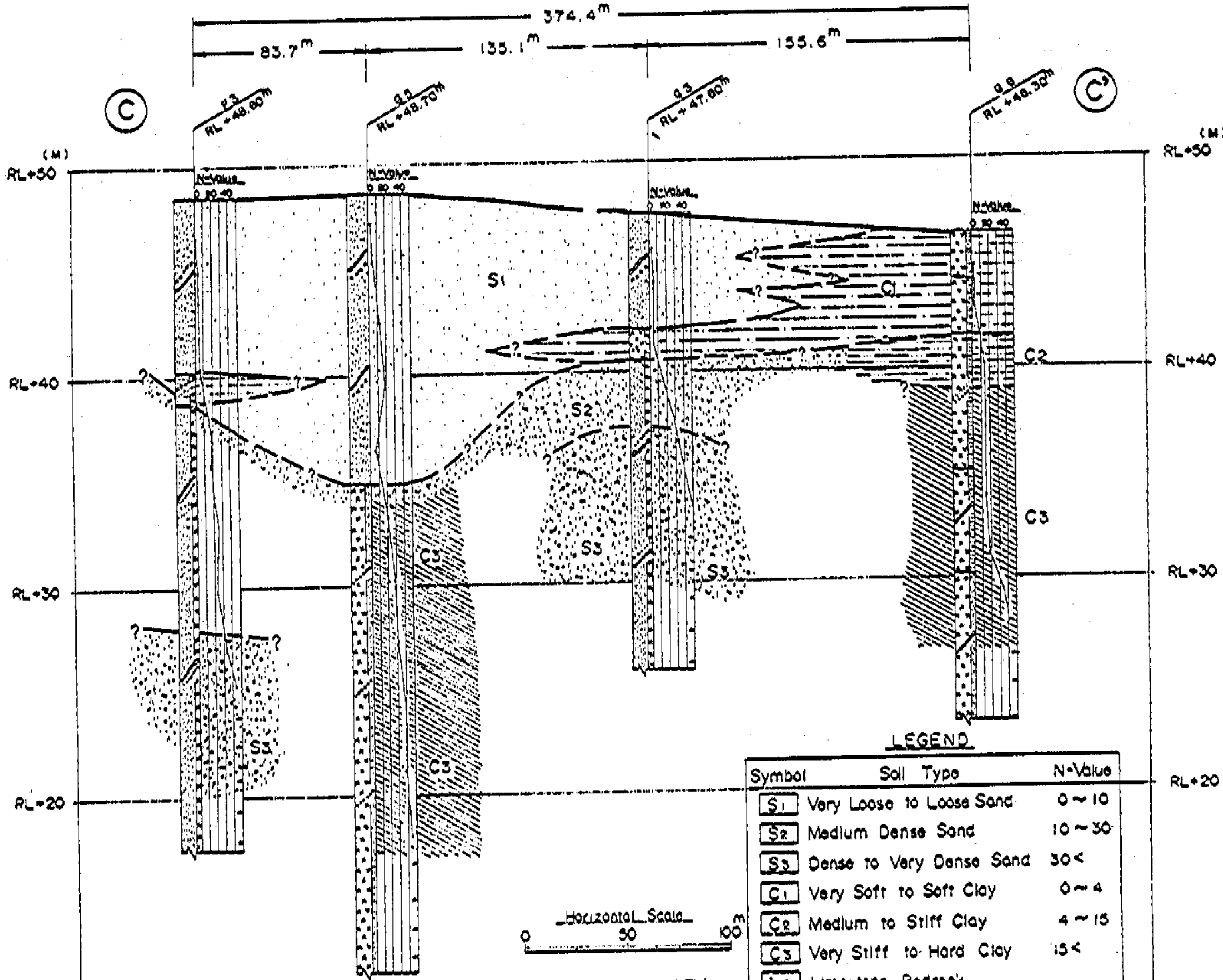


Fig. 5-15c Soil Profile at Cheras 4miles (3)

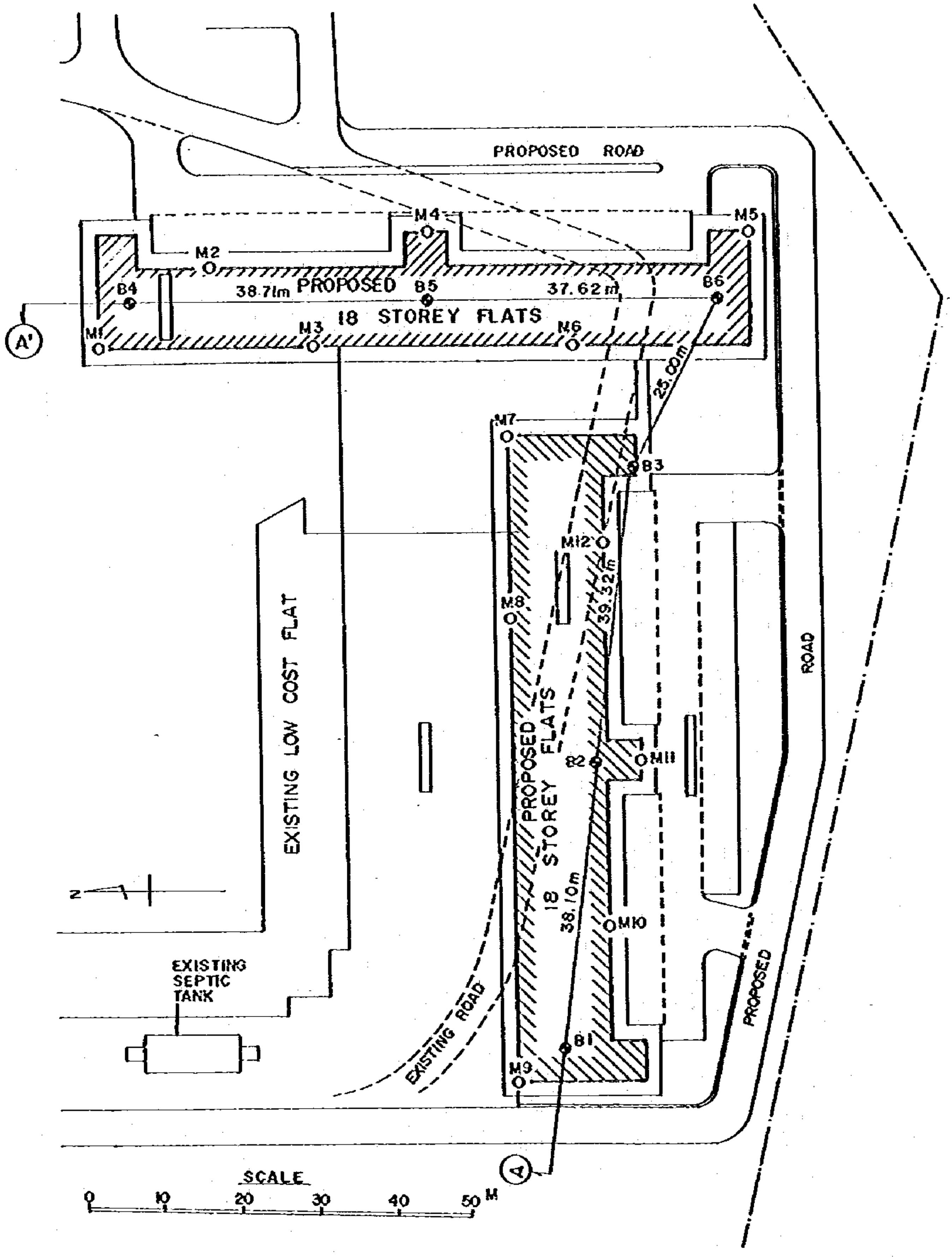


Fig. 5-16 Site Layout and Location of Borings — Sungai Besi —

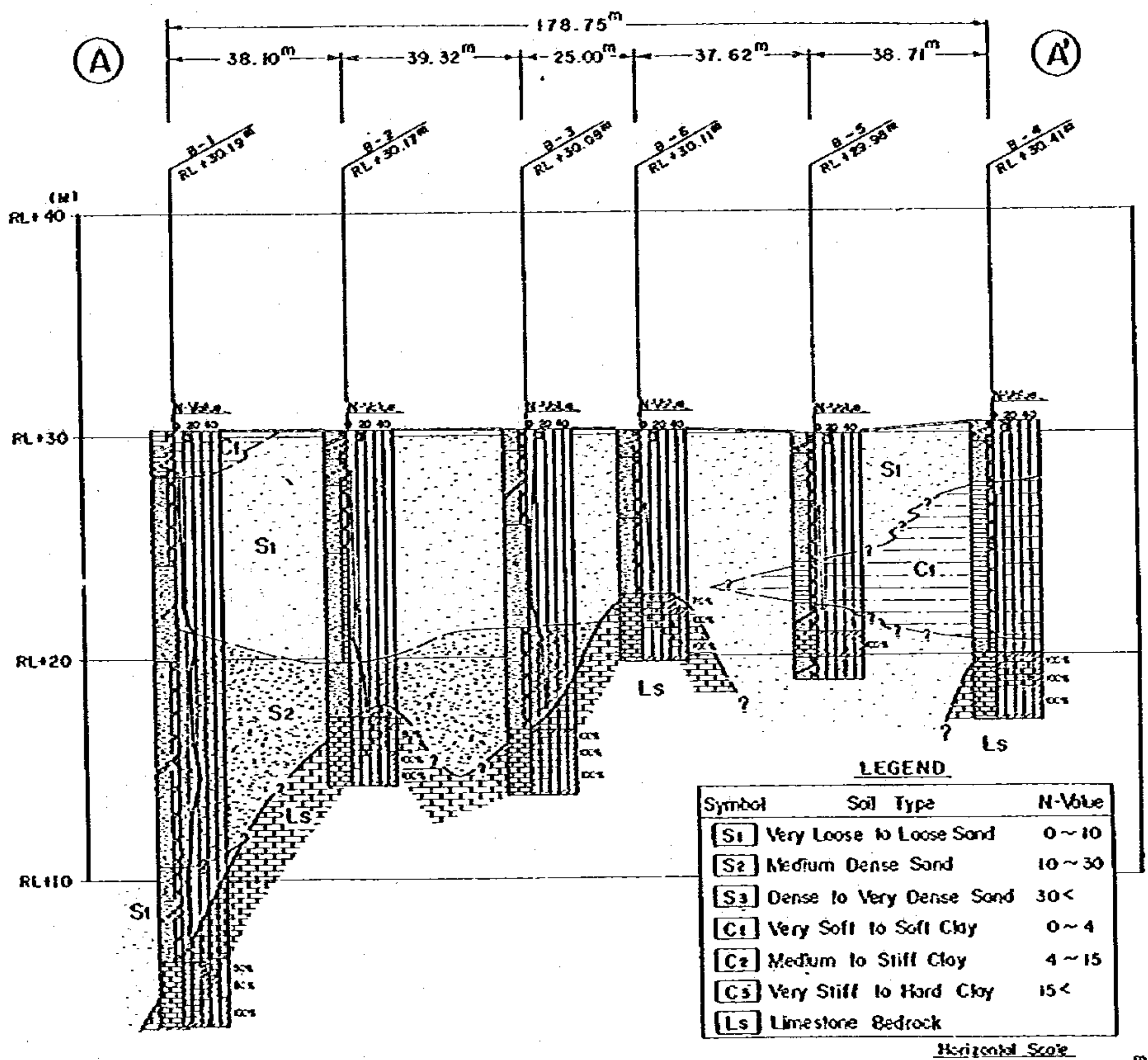


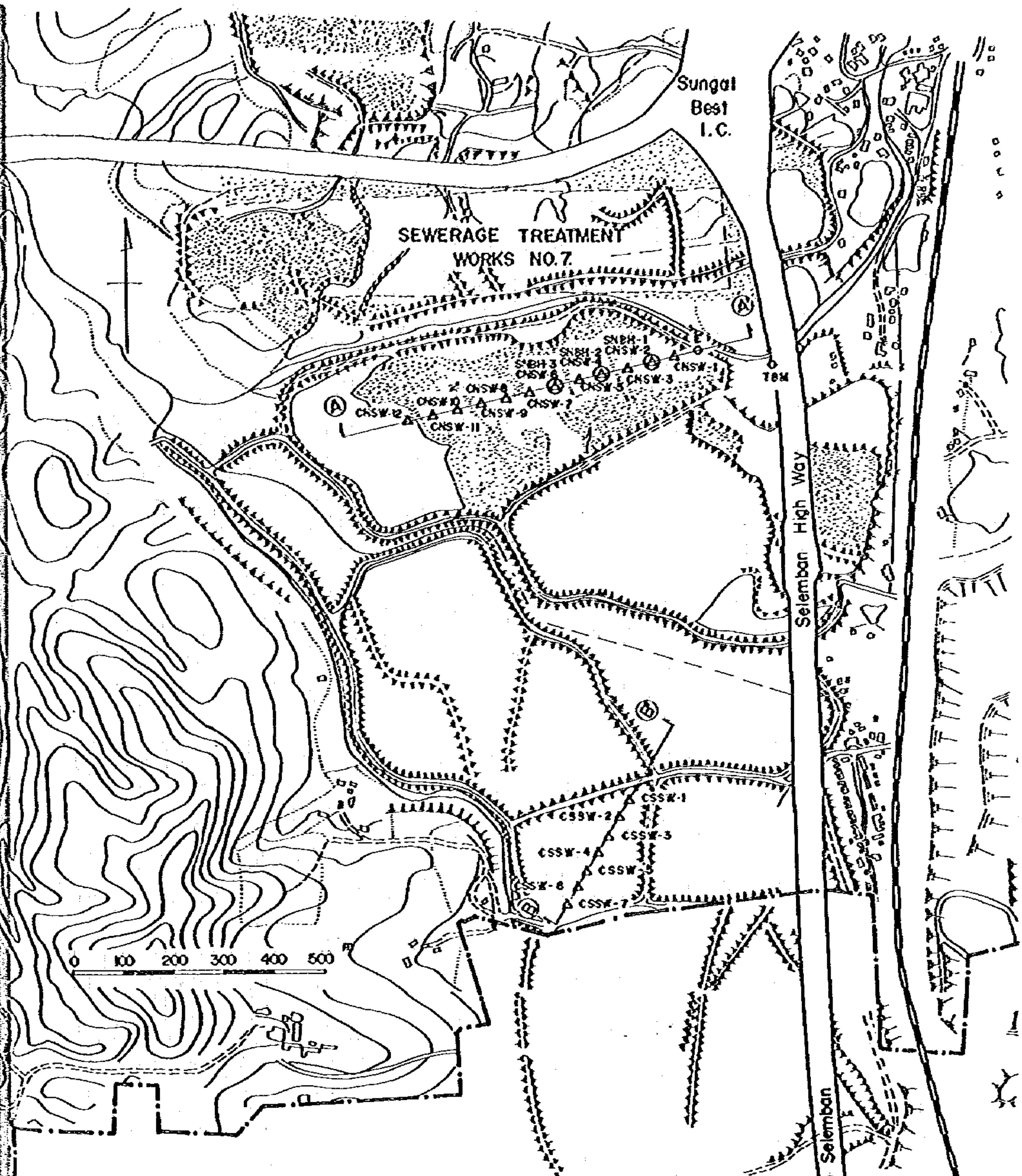
Fig. 5-17 Soil Profile at Sungai Besi

sounding and boring records. The soils at the site are generally sandy, but 2 to 7m of soft clay is sandwiched in the sand between boreholes B-4 and B-5. The maximum difference in the level of the limestone surface is about 16m.

5.2.6 Castlefield

The Castlefield site is located at the southern tip of the Federal Territory on the west side of the Kuala Lumpur-Selemban Highway. For the soil investigation, a measurement line was established at the north site and another was set-up at the south site as shown in Fig. 5-18. At the north site, twelve Swedish soundings and three borings were performed. Laboratory soil tests were carried out on samples obtained by borings. At the south site, only Swedish soundings were carried out. Based on the results of the borings and soundings carried out at both sites, the soil profiles shown in Fig. 5-19 were compiled.

The main subsurface soil at both sites is soft clay, but at the eastern one-third of the northern site, a loose sand layer is deposited near the ground surface. The maximum thickness of the soft clay layer is about 13m at CNSW-12. The strength of the soft clay at the two sites is such that people can walk on it only with difficulty. As shown in Fig. 5-18, much of the area between the north site and the south site (areas surrounded by banks) consist of extremely weak clayey soils, so weak that people can not

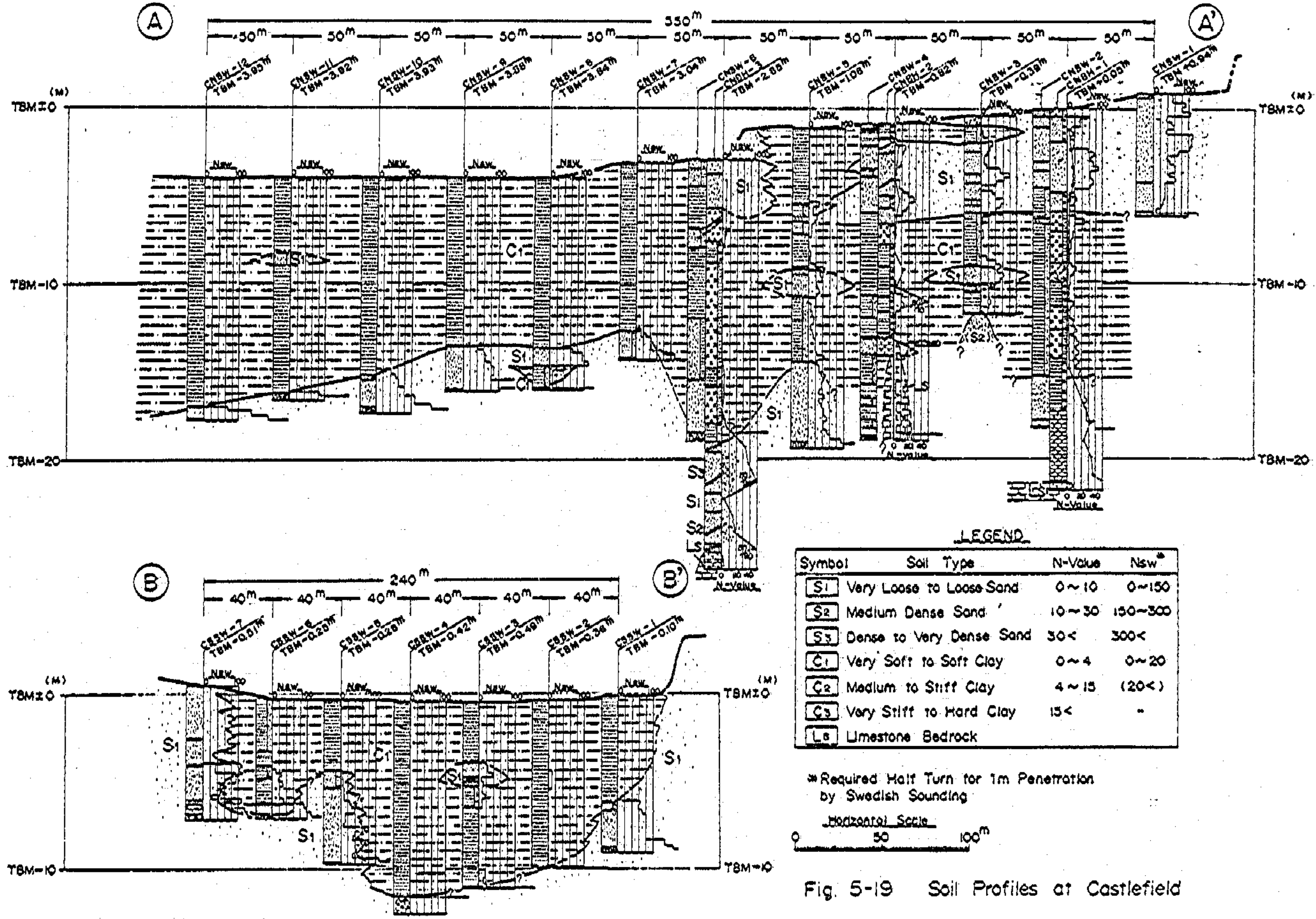


△ Swedish Sounding

○ Boring

○ TBM Temporary Bench Mark

Fig. 5-18 Site Plan and Location of Borings and Swedish Soundings
— Castlefield North and South —



Symbol	Soil Type	N-Value	Nsw*
S1	Very Loose to Loose Sand	0 ~ 10	0 ~ 150
S2	Medium Dense Sand	10 ~ 30	150 ~ 300
S3	Dense to Very Dense Sand	30<	300<
C1	Very Soft to Soft Clay	0 ~ 4	0 ~ 20
C2	Medium to Stiff Clay	4 ~ 15	(20<)
C3	Very Stiff to Hard Clay	15<	"
Ls	Limestone Bedrock		

Fig. 5-19 Soil Profiles at Castlefield

walk on them, and of stagnant muddy water.

Results of laboratory soil tests on 11 undisturbed samples obtained mainly from soft clayey layers by exploratory borings carried out at north site are summarized in Table 5-6. Fig. 5-20 shows the differences in physical properties of the soft clayey soils at the north site versus depth. Fig. 5-21 shows the grading texture of the soft clayey soils. Silt content of the soils is quite high compared with other sites. Classification of the soils on a plasticity chart is shown in Fig. 5-22. Most of the plottings fall in the area of CH, CL and MH. Fig. 5-23 shows undrained shear strength, preconsolidation pressure and compression index versus depth. The physical and mechanical properties of soft clayey soils are summarized in Table 5-7. It should be noted that all the parameters shown are on the rather stiffer side because data on samples from shallow depths are not sufficient. Fig. 5-24 shows e-log p curves of the soft clayey soils and Fig. 5-25 shows coefficients of consolidation for them.

Soil properties of major layers to be encountered in the ex-mining land are summarized in Table 5-8. The values shown in the table are those most representative to the layer and not the possible ranges. For very soft clay, soft clay, very loose sand and loose sand, the parameters indicated are those when the layers appear as the surface layer. The parameters shown in the table in parentheses are estimated by engineering judgement.

Table 5-6 Summary of Soil Tests on Samples from Castlefield North

	Boring No.	CNBH-1											CNBH-2					CNBH-3				
	Sample No.	UD-1 Top	UD-1 Middle	UD-1 Bottom	UD-2 Top	UD-2 Bottom	UD-3 Top	UD-3 Bottom	UD-4	UD-5 Top	UD-5 Bottom	UD-6	UD-1 Top	UD-1 Bottom	P-6	UD-2 Top	UD-2 Bottom	UD-1 Top	UD-1 Bottom	UD-2 Top	UD-2 Bottom	UD-3
	Sample depth	6.00 ~ 6.20	6.20 ~ 6.55	6.55 ~ 6.80	7.50 ~ 7.75	7.75 ~ 8.30	9.00 ~ 9.50	9.50 ~ 9.80	11.50 ~ 12.30	13.00 ~ 13.35	13.35 ~ 13.80	16.00 ~ 16.80	6.00 ~ 6.55	6.55 ~ 6.80	7.15 ~ 7.45	9.00 ~ 9.60	9.60 ~ 9.80	3.00 ~ 3.65	3.65 ~ 3.80	6.00 ~ 6.65	6.65 ~ 6.80	9.00 ~ 9.80
	Condition of sample	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed	Disturbed Undisturbed
	Natural water content, %	62.3	28.6	41.5	57.1	63.3	50.2	42.3	44.3	41.2	41.7	58.6	55.7	62.7	60.5	62.8	62.4	49.9	67.5	68.1	73.3	64.5
	Specific gravity	2.640	2.652	2.689	2.463	2.495	2.631	2.660	2.758	2.785	2.792	2.624	2.624	2.646	2.611	2.629	2.627	2.651	2.561	2.651	2.581	2.681
	Wet density, g/cm³	1.58	1.85	1.73	1.64	1.65	1.69	1.71	1.79	1.83	1.87	1.64	1.67	1.68	-	1.60	1.60	1.75	1.59	1.59	1.53	1.61
	Dry density, g/cm³	0.97	1.44	1.22	1.04	1.01	1.13	1.20	1.24	1.30	1.32	1.03	1.07	1.03	-	0.98	0.99	1.17	0.95	0.95	0.88	0.98
	Natural void ratio	1.71	0.84	1.20	1.36	1.47	1.34	1.21	1.22	1.15	1.12	1.54	1.45	1.56	-	1.68	1.67	1.27	1.70	1.80	1.92	1.74
	Degree of saturation, %	96	90	93	100	100	100	92	100	100	100	100	100	100	-	99	99	100	100	100	98	99
Atterberg limits	Liquid limit, %	69.6	-	42.3	43.2	51.0	52.0	35.9	51.6	48.0	39.6	58.8	57.3	-	-	57.0	56.0	-	54.2	65.0	70.8	54.9
	Plastic limit, %	36.0	-	24.2	28.2	33.1	26.0	22.1	24.3	23.5	24.4	30.1	34.6	-	-	35.3	31.8	-	31.0	34.0	31.4	27.3
	Plasticity index	33.6	-	18.1	15.0	17.9	26.0	13.8	27.3	24.5	15.2	28.7	22.7	-	-	21.7	24.2	-	23.2	31.0	39.4	27.6
Grain size analysis	Gravel, %	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0
	Sand, %	1	16	1	13	6	1	27	0	2	6	0	11	50	2	1	0	29	13	2	0	1
	Silt, %	46	70	60	60	60	50	38	49	58	63	15	48	29	47	50	51	47	57	48	11	43
	Clay & colloid, %	53	14	39	26	34	49	35	51	40	31	85	41	21	51	49	49	24	28	50	89	56
	Max. diameter, mm	0.105	0.84	0.105	4.76	0.84	0.105	0.420	0.074	0.250	0.250	0.028	0.420	2.00	0.420	0.105	0.040	2.000	4.76	0.250	0.011	0.105
	Diam. at 60%	0.0062	0.036	0.012	0.030	0.014	0.0090	0.043	0.0073	0.013	0.017	0.0013	0.012	0.160	0.068	0.0069	0.0069	0.035	0.019	0.0069	-	0.0057
	Diam. at 10%	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	Visual soil description	Clayey Silt	Clayey Silt	Clayey Silt	Clayey Silt	Clayey Silt	Clayey Silt	Sandy Silt	Clayey Silt	Clayey Silt	Clayey Silt	Silty Clay	Sandy Clayey Silt	Silty Sand	Silty Clay	Clayey Silt	Clayey Silt	Clayey Silt with Sand	Clayey Silt	Clayey Silt	Silty Clay	Clayey Silt
	Unified soil classification	MH	-	(ML)	ML	(MH)	(MH)	(CL)	MH	CL	(ML)	CH	MH	(SM)	-	MH	MH	(SM)	MH	MH	CH	MH
Unconfined compression test	Undisturbed sample, kg/cm²	-	-	-	0.46	-	-	-	-	0.53	-	-	-	-	-	-	-	-	-	-	-	-
	Remoulded sample, kg/cm²	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	Sensitivity ratio	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	Strain at failure, %	-	-	-	5.5	-	-	-	-	9.5 ~15.0	-	-	-	-	-	-	-	-	-	-	-	-
Triaxial compression test	Angle of internal friction	-	-	-	-	0°	0°	-	0°	-	0°	0°	0°	-	-	0°	0°	-	-	0°	-	0°
	Cohesion, kg/cm²	-	-	-	-	0.28	0.20	-	0.30	-	0.31	(0.19)	(0.15)	-	-	0.18	0.22	-	-	0.09	-	0.12
	Condition of drainage	-	-	-	-	U-U	U-U	-	U-U	-	U-U	U-U	U-U	-	-	U-U	U-U	-	-	U-U	-	U-U
Consolidation test	Preconsolidation pressure, kg/cm²	-	-	-	-	-	0.86	-	(1.1)	-	-	-	-	-	-	2.4	-	-	-	-	-	-
	Compression index	-	-	0.28	-	0.38	0.58	-	0.48	-	0.44	0.40	-	0.52	-	0.92	-	0.42	0.65	0.62	-	0.56
Lab. Vane Shear	Max. Shear Strength (kg/cm²)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.059	-	-	-	-

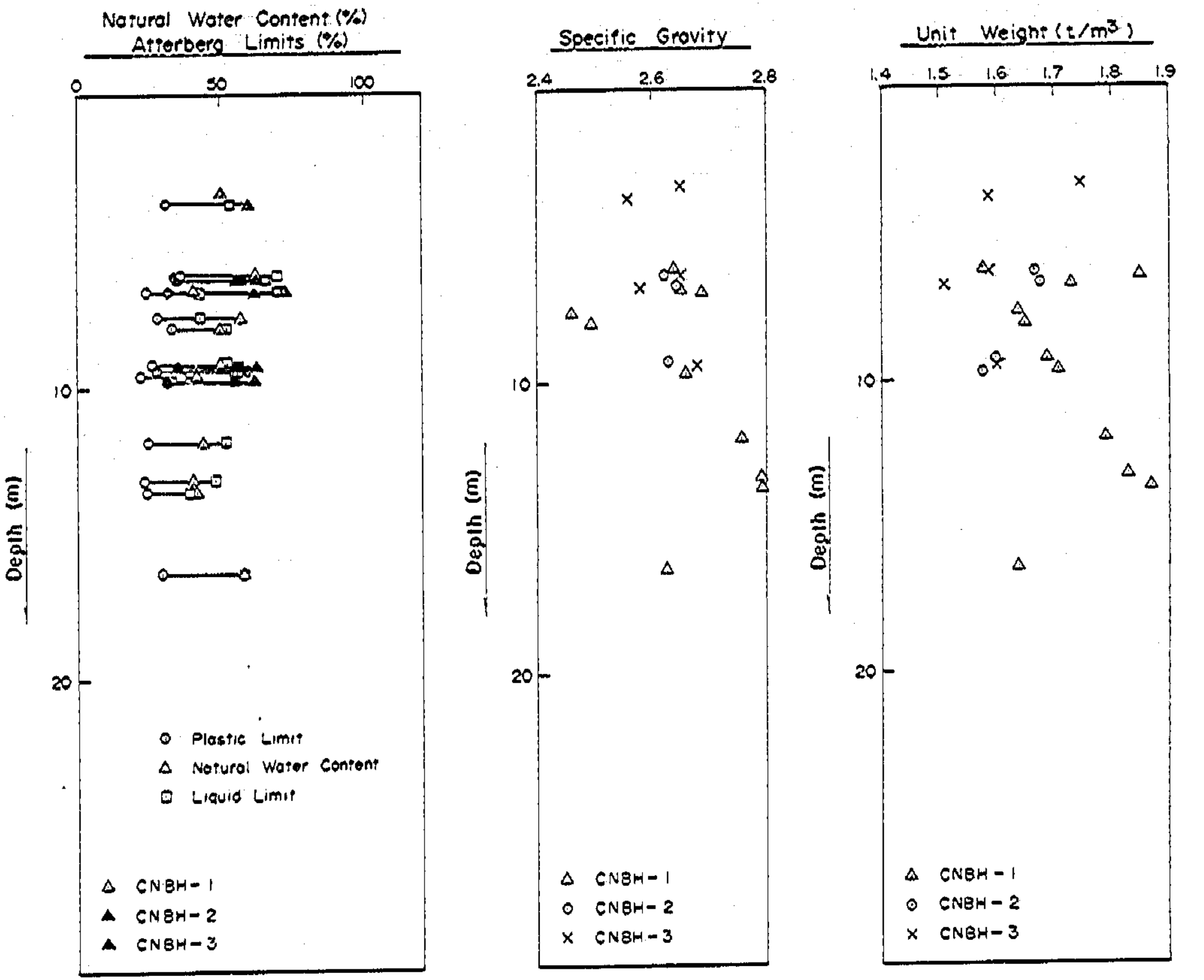


Fig. 5-20 Depth v.s. Physical Properies of Samples from Castlefield North

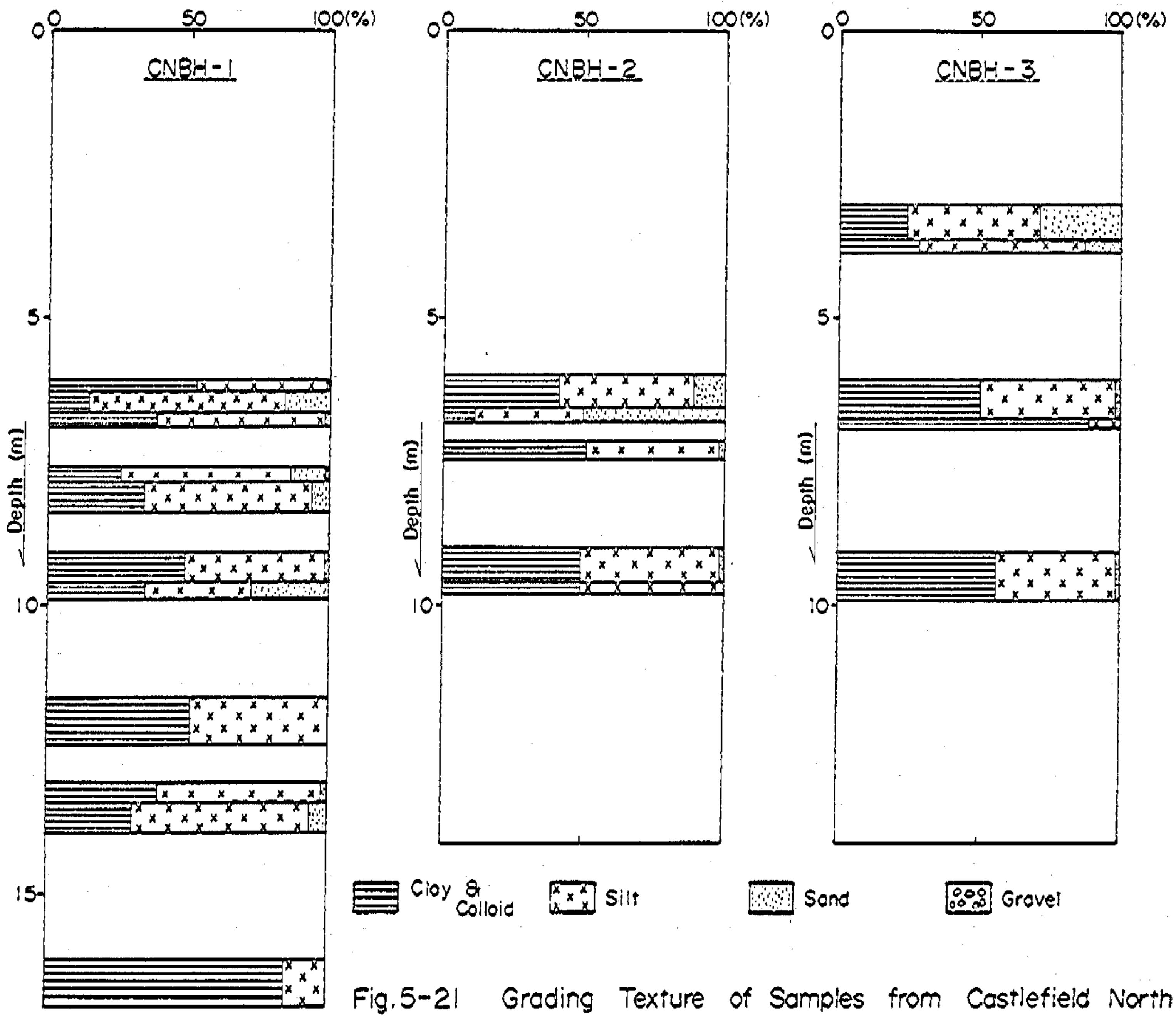
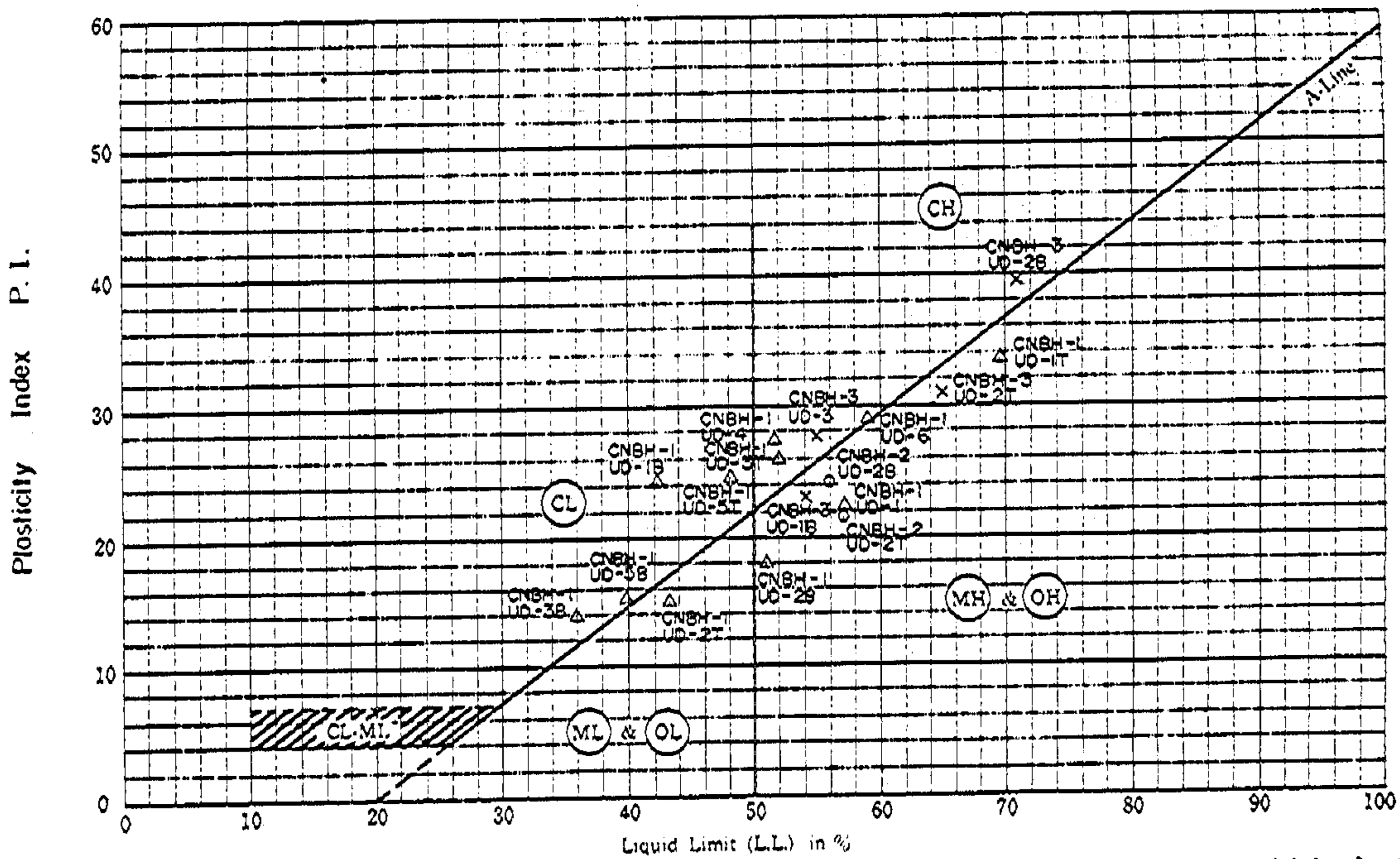


Fig.5-21 Grading Texture of Samples from Castlefield North



ML: Inorganic silt with some plasticity, very fine sand, rock flour, silty or clayey fine sand and clayey silt

CL: Clay containing gravels with low or middle plasticity sandly clay and silty clay

OL: Organic silt with low plasticity and organic silty clay

MH: Inorganic silt with middle or high plasticity, micaceous or diatomaceous fine sandy soil and silty soil

CH: Inorganic clay with high plasticity

OH: Organic clay with middle or high plasticity and organic silt

Fig. 5-22 Classification of Samples from Castlefield North by Plasticity Chart

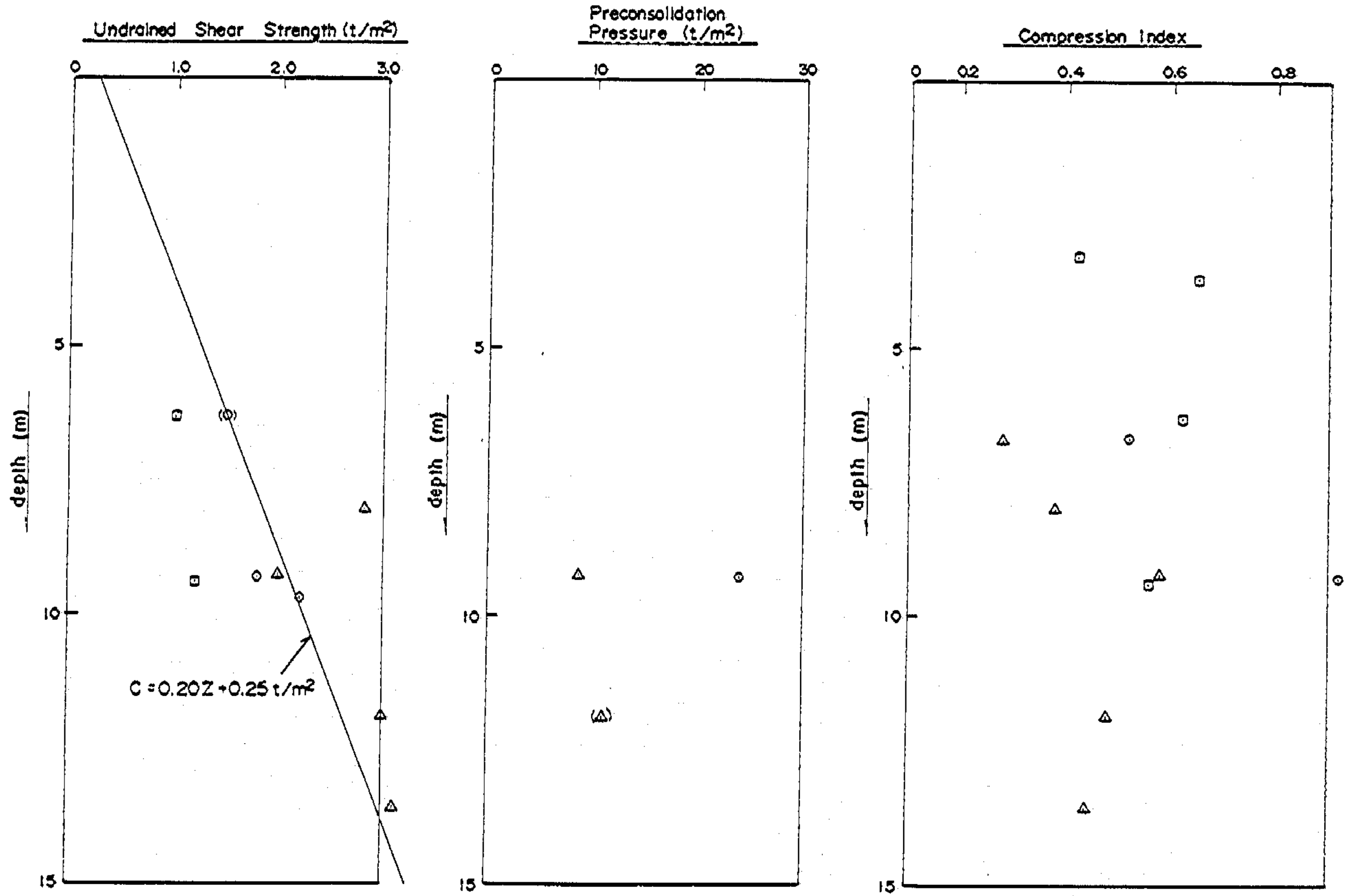


Fig. 5-23 Depth v.s. Undrained Shear Strength Preconsolidation Pressure and Compression Index — Castlefield North —

Table 5-7 Physical and Mechanical Properties of Soft Clayey Soils of Castlefield North

Item	Major Range	Tendency
Physical Properties		
Natural Water Content	30 ∿ 70%	Decrease with Depth
Specific Gravity	(2.45)∿2.6 ∿ 2.8	Scattered in Wide Range
Wet Density	(1.5)∿1.6 ∿ 1.85	Scattered in Wide Range
Liquid Limit	35 ∿ 70%	Scattered in Wide Range
Plastic Limit	25 ∿ 35%	
Gravel Content	0 ∿ 2%	
Sand Content	0 ∿ 30%	Silt Content is Quite high ; Scattered in Wide Range
Silt Content	25 ∿ 70%	
Clay and Colloid Content	10 ∿ 75%	
Unified Soil Classification	CL, CH or MH	
Mechanical Properties		
Undrained Shear Strength	0.5 ∿ 3.0 t/m^2	Increase with Depth $C \doteqdot 0.20Z + 0.25\ t/m^2$
Preconsolidation Pressure	7 ∿ 11 t/m^2	
Compression Index	0.30 ∿ 0.65	

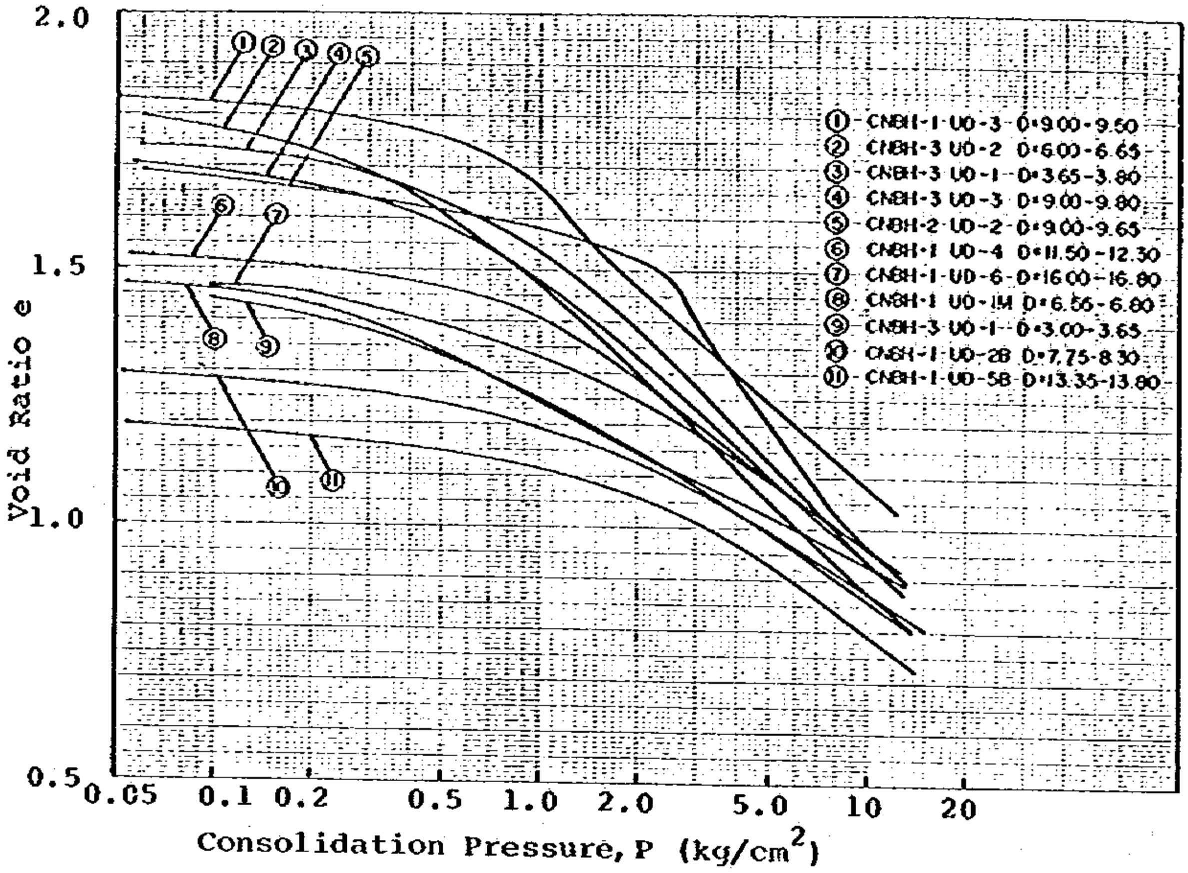


Fig. 5-24 e ~ Log P Curves - Castlefield North -

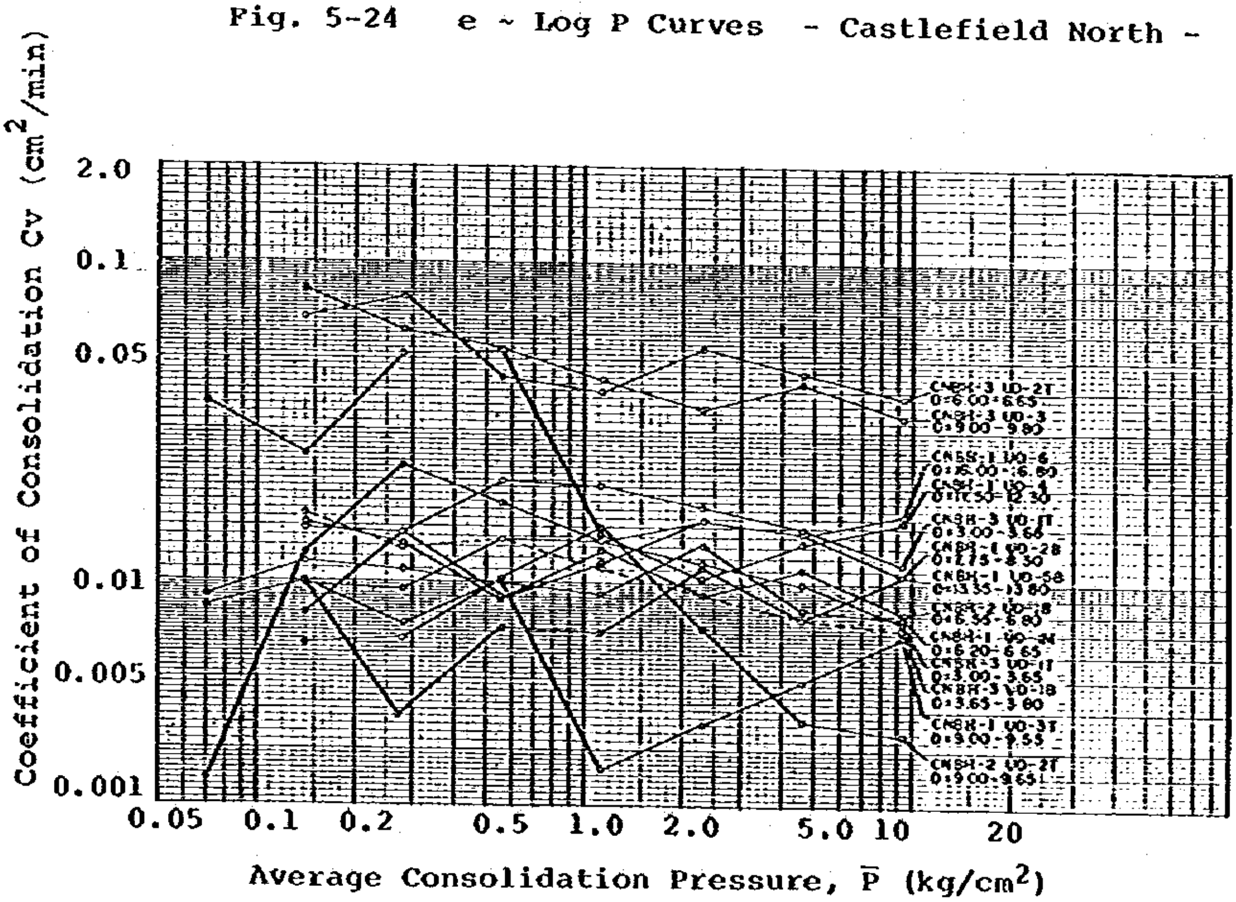


Fig. 5-25 Coefficient of Consolidation v.s. Average Pressure - Castlefield North -

Table 5-8 Summary of Soil Properties of Each Layer

Type of Soil	Colour	N-Value	Nsw *1	Wet Density γ_t (t/m^3)	Undrained Shear Strength C_u (t/m^2)	Angle of Internal Friction ϕ (Degree)	Compression Index C_c	Co-efficient of Consolidation, C_v (cm^2/sec)	Rate of *2 Distribution at Ground Surface (%)	Major Origin and Distribution
Floating Mud	Grey, White, Yellowish Brown, Reddish Brown	0	0	(1.1-1.3)	(0)	-	-	-	0	Young slime in mining pond, e.g. Kampong Pandan, Sentul
Very Soft Clay *3	"	0-1	0	≒1.4 (1.3-1.5)	0.1-0.3	-	0.5-0.8	0.04-0.1	40	Slime deposits, e.g. Sentul Castlefield
Soft Clay *3	"	2-4	0-20	1.5-1.7	1.0-1.5	-	0.5-0.7	0.05-0.2	10	Slime deposits, e.g. Sentapak, Sentul, Castlefield
Medium Stiff Clay	Grey, Reddish Brown, Yellowish brown	4-8 -(15)	20-	(1.7-2.0)	(2.5-5.0)	-	(0.2-0.3)	-	5	Old clayey tailing, unexcavated Old Alluvium, weathered bedrock, e.g. Cheras 3-1/2 miles, Cheras 4 miles
Very Stiff to Hard Clay	Reddish Brown, Yellowish Brown	>15	-	(1.9-2.2)	>10	-	-	-	0	Unexcavated Old Alluvium, weathered bedrock, e.g. Cheras 3-1/2 miles
Very Loose Sand *3	White, Yellowish brown	1-3	0-50	(1.4-1.6)	-	(25-28)	-	-	20	Ex-mining sand, e.g. Kampong Pandan, Cheras 4 miles, Castlefield
Loose Sand *3	"	≒5 (4-10)	50-150	(1.6-1.8)	-	≒30 (28-33)	-	-	20	Ex-mining sand, e.g. Gombak; Kampong Pandan, Sungai Basi, Castlefield
Medium Dense Sand	Grey, Yellowish Brown, Reddish Brown	10-30	150-300	(1.8-2.0)	-	(33-40)	-	-	5	Ex-mining sand, unexcavated Old Alluvium, weathered bedrock, e.g. Cheras 3-1/2 miles
Medium Dense Sand	"	>30	>300	(2.0-)	-	(>40)	-	-	0	Unexcavated Old Alluvium, weathered bedrock, e.g. Cheras 3-1/2 miles
Limestone Bedrock	White, Grey, Yellowish Brown	Refusal	Refusal	2.6-2.8	-	-	-	-	0	-

*1 Required half turn for 1m penetration under 100kg weight by Swedish sounding

*2 Approximate rate of distribution at the ground surface of ex-mining land excluding mining pond

*3 Indicates surface layer

Note: Values in parentheses are estimates based on engineering judgement

The value indicated in the table should be used as a guide for preliminary estimation because the subsurface ground condition of ex-mining land is extremely variable. As regards the foundation study for any specific project, it is necessary to confirm soil properties by individual investigation at the proposed site.

5.3 Ground Model of Ex-Mining Land

The subsurface ground conditions, described above, at the various former tin-mining sites were classified into several categories from an engineering point of view, and a model of each type was made.

Broadly speaking, the distribution of soil at ex-mining land is as follows: Bedrock layers and yet unexcavated soil ranging in type from weathered bedrock to old alluvium, which will be the pile-supporting layers, are overlain by very soft to soft clay and/or very loose to loose sand. Depending on the use of the land during mining operations, sand and/or clay are deposited differently. As a result, ground conditions such as a sand-rich area, a clay-rich area, or an intermediate area are encountered. Foundation engineering approaches to such varied types of ground conditions as outlined above differ fundamentally. These differences are influenced by 2 major factors, ground bearing pressure and consolidation settlement.

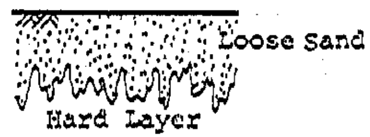
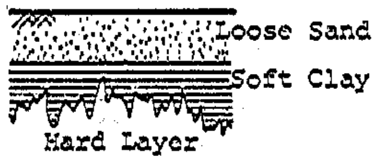
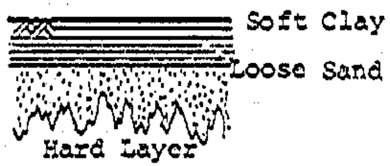
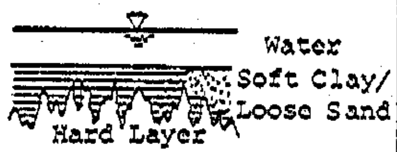
With regard to the first, in cases where the subsurface soil is soft clay, a top soil layer of better earth or the removal of the soft clay is necessary before the commencement of construction. In the event that the surface soil is composed of sand, heavy machinery can be introduced without pre-treatment. Where the area is covered by water, drainage is necessary and land reclamation should be carried out before commencement of construction.

As regards the second factor, the emplacement of additional earth or the construction of structures supported by direct foundations will induce further consolidation settlement even in those cases where consolidation due to overburdening has already been completed. This settlement will continue over the long term depending on the thickness of soft clay layers and their consolidation properties.

In consideration of the above-mentioned factors, subsurface ground conditions of ex-mining land have been classified into 5 types, i.e. Type A through Type E. Table 5-9 summarises the classification of ex-mining land and the relationship between the occurrence of the five types of foundation ground and the major mining operation performed. Of the sites investigated, relatively numerous examples of sites having foundation ground of each type are listed. The following are the features of each ground type.

Type A is a mining hole backfilled with sandy soil only. The ground will support light structures with only surface compaction. No consolidation settlement will take place.

Table 5-9 Types of Ground in Ex-mining Land
Classified from Engineering Viewpoint

Type of Deposit on Bedrock or Other Boaring Layer	Relation to Mining Operation	Examples of Ex-Mining Land
Type A 	Tailing Area near Tailing Point	Kampong Pandan Cheras 3-1/2 miles Cheras 4 miles Sungai Besi
Type B 	Tailing Area and/or Slime Pond Covered Later with Sandy Tailing or Sandy Dumping	Gombak, Kampong Pandan, Cheras 3-1/2 miles, Cheras 4 miles Sungai Besi, Castlefield
Type C 	Tailing Area far from Tailing Point, or Slime Pond	Sentul, Gombak Kampong Pandan, Cheras 3-1/2 miles, Cheras 4 miles
Type D 	Slime Pond, Tailing Area far from Tailing Point	Sentul, Setapak Castlefield
Type E 	Old Mining Hole	Pond of Kampong Pandan Setapak

Type B ground is composed of a soft clayey deposit covered with sand. In the majority of cases, it will support light structures but consolidation settlement will occur.

Type C ground comprises sandy soil deposits covered with soft clay. It requires additional fill or replacement with better soils before commencement of construction. Ground conditions in Type C are superior to those of Type D as the term of consolidation settlement is shorter and the sandy layer will act as a better support for pile foundations.

Type D is a mining hole filled with slime/soft clay only. This type of ground required a substantial amount of fill and given this situation, the term necessary for consolidation settlement will be very long.

Type E ground conditions indicate a mining hole that has not been completely backfilled and in the majority of cases standing water is present. This type requires a large amount of fill before construction. Settlement problems may also be encountered if soft clay deposits exist.

Figs. 5-26a and 5-26b show examples of ground-type classification maps for Kampong Pandan and Sentul. The maps are reliable for only the locations near the boring and sounding positions as those further away have been estimated using mining records and reconnaissance data. Accordingly, the maps show only probable configurations with the result that further detailed investigation is recommended for the design of each individual structure. However, these maps are useful for land-use planning such as layout of housing estates, ect. Very rough estimates of the proportionate distribution of each type of ground at each site including those of Kampong Pandan and Sentul are listed in Table 5-10. It should be kept in mind that these estimates apply only to the areas in which borings and soundings were performed. The average estimates listed in the table are weighted averages based on the investigated area of each site. As seen from the table, the average proportions of the types of foundation ground found at ex-mining land are; about 20% each of Types A and D, about 25% of Types B and C, and about 10% of Type E.

5.4 Reccomeded Method of Field Ground Investigations

In order to perform the field ground investigation economically and efficiently, it is recommended that it be carried out on a step-by-step basis from the preliminary investigation up through the detailed investigation as shown in Fig. 5-27.

5.4.1 Preliminary Ground Investigations

A preliminary investigation should be performed as a first step and should comprise data collection and site inspection by geotechnical engineers. As regards collection of data, it is desirable that the following be collected for

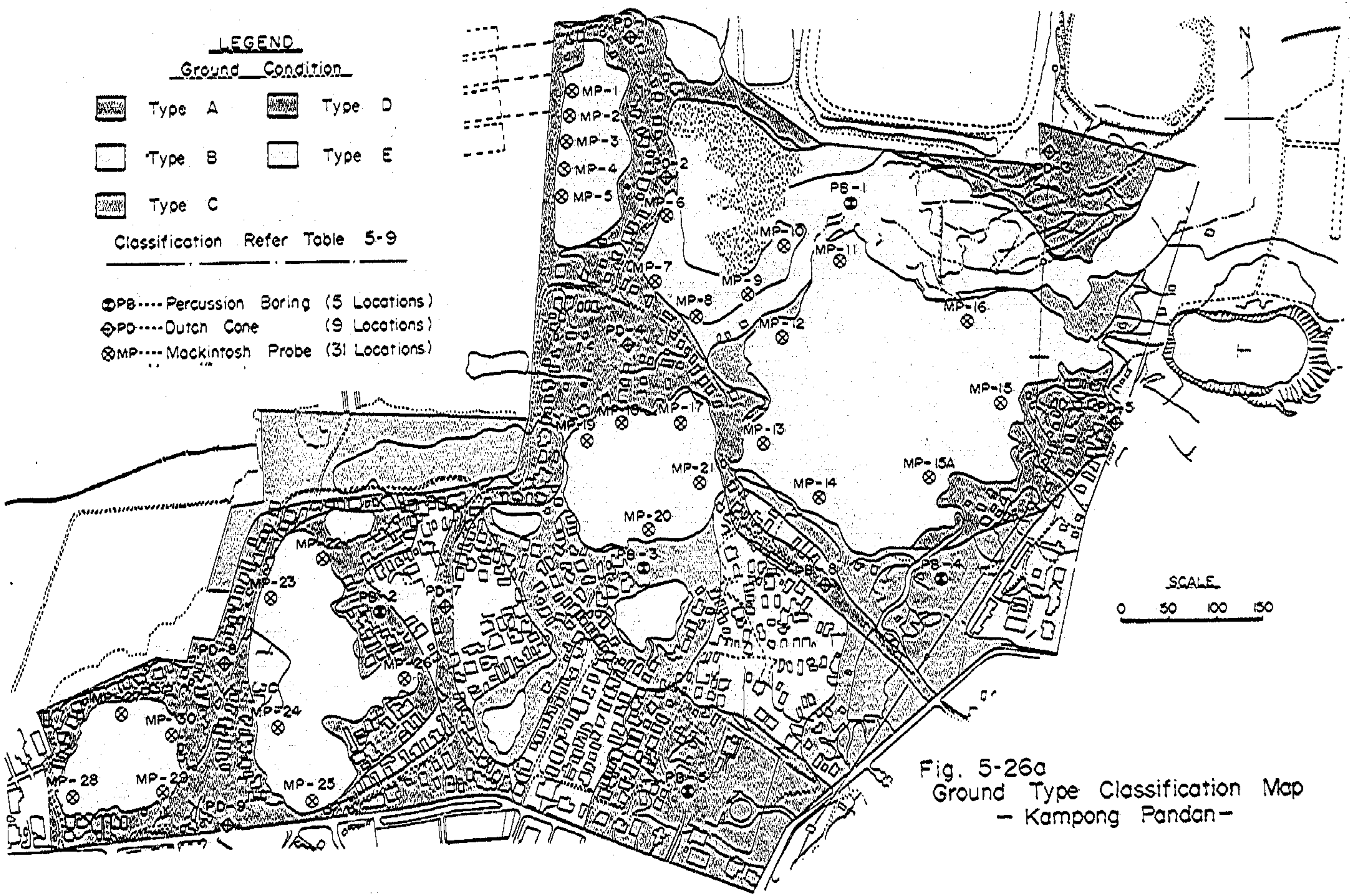


Fig. 5-26a
Ground Type Classification Map
— Kampong Pandan—

Table 5-10 Types of Ground Distribution in Eash Site

(%)

Type of Ground \ Site	Sentul	Kampong Pandan	Gombak	Setapak	Cheras 3-1/2 miles	Cheras 4 miles	Sungai Besi	Castlefield	Average
Type A	10	25	15	5	20	40	70	5	19
Type B	25	15	40	15	20	30	30	25	24
Type C	35	15	35	5	60	30	-	-	23
Type D	20	5	10	60	-	-	-	70	21
Type E	10	40	-	15	-	-	-	-	13
Approximate Area (ha)	46	43	22	18	8.5	17	6	20	Total Area 180.5

Note: The percentages indicated above are rough estimates based on areas where boring or sounding data were available

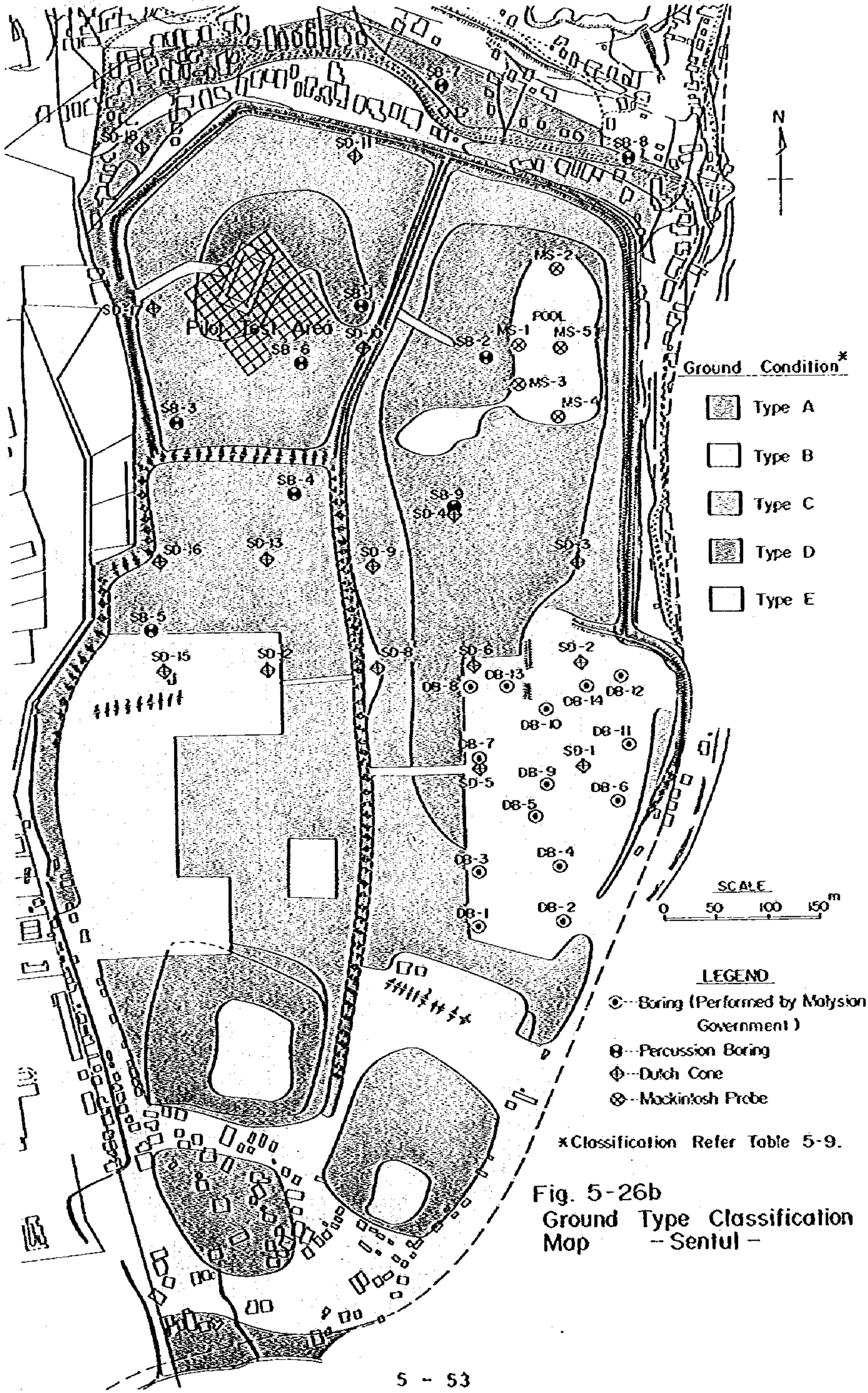


Fig. 5-26b
Ground Type Classification
Map - Sentul -

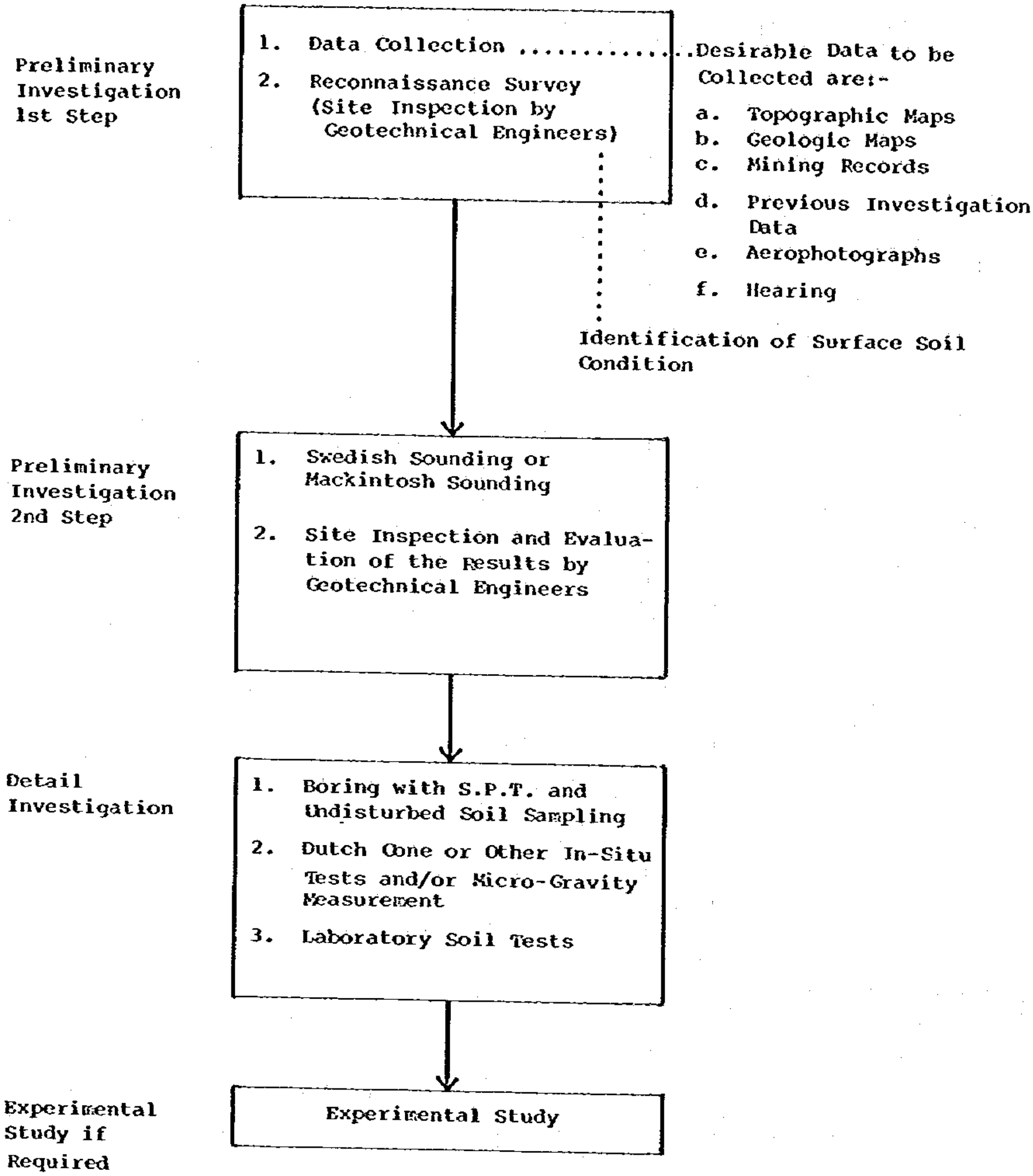


Fig. 5-27 Steps of Ground Investigation

the purpose of compiling a rough soil profile:

1) Topographic maps

2) Geologic maps

3) Mining records

4) Previous investigation data

5) Aerophotographs

6) Information by interviewing

   i) site inspectors of mining departments,

   ii) mining company which was operating at the site in question, and

   iii) dwellers around the site.

In the secound step of the preliminary investigation, a number of soundings should be carried out to obtain soil profiles and penetration resistance of soils. Two basic sounding methods are employed for this purpose, the Mackintosh probe sounding method and the Swedish sounding method. The former is widely used in Malaysia and is a good sounding technique, both in terms of cost and ease of operation. Its only drawbacks are the obvious penetration resistance encountered and the fact that this method does not provide any information on soil type. The operational costs of the Swedish sounding method are slightly higher with the equipment being slightly more expensive than that for the Mackintosh probe tests. However, this method can be used to identify differences in soil type and, in addition, its penetration capacity is greater than that of the Mackintosh type.

Based on this two-step preliminary investigation, the predominant type of material deposited, the areas where clayey or sandy materials are distributed and the average depth to the berdrock will be known. With this information, a preparatory foundation study can be initiated. This study will also assist in indicating necessary information to be obtained during detailed subsurface investigations.

5.4.2 Detailed Ground Investigations

For detailed investigation of ex-mining land, the following types of investigations are generally found to be useful:

1) Exploratory borings together with standard penetration tests and undisturbed samplings
2) Dutch cone or other in-situ tests to obtain in-situ mechanical properties of soils
3) Micro-gravity measurements and rock boring to investigate bedrock profiles
4) Laboratory soil tests to obtain physical and mechanical properties of the ground

Depending on foundation design requirements, appropriate investigation methods will be selected and the results analysed.

5.4.3 Experimental Study

Depending on the ground condition and the size and importance of proposed structures, experimental studies may be carried out in advance of actual construction. Experimental studies may be costly but result in the most accurate prediction of ground behaviour. In this feasibility study, an experimental embankment was constructed at Sentul. The contents and results of the same are presented in Section 6.

SECTION 6 STUDY OF GROUND IMPROVEMENT METHODS FOR SOFT MATERIAL

SECTION 6

STUDY OF GROUND IMPROVEMENT METHODS FOR SOFT MATERIAL

6. STUDY OF GROUND IMPROVEMENT METHODS FOR SOFT MATERIAL

As described before, ex-mining land is mainly covered with soft material. Therefore, it is necessary to improve this soft material before construction of permanent structures. Especially for soft clayey deposits, appropriate countermeasures are required to cope with long term consolidation settlement and ground weakness. Generally, soft clays increase in strength with the progress of consolidation settlement.

There are two major concerns in terms of consolidation settlement, i.e. ① total (final) amount of settlement and ② the time rate (speed) of settlement. These two aspects greatly affect the cost and time required for the improvement of soft ground. Therefore, it is most important to predict the total amount of settlement and the time rate of settlement as accurately as possible.

The amount and time rate of settlement are generally analysed based on the results of laboratory consolidation tests on undisturbed soil samples. However, the predictions based on the results of laboratory tests can often be different from the actual behaviour of the ground at the site. Especially, the prediction of the time rate of consolidation settlement is most difficult. On many occasions, it can be faster in the field than in the prediction. The main reason for this difference is the difficulty in establishing the drainage condition of the field. The actual field condition can be different from the conditions estimated by the test results on small soil specimens.

Hence, experimental studies at the site become very important in predicting the behaviour of the ground and the effectiveness of the soft ground improvement methods.

In this section, the contents of the experimental study carried out at the Sentul site and its results are presented and discussed. The results of laboratory tests carried out to study the effectiveness of chemical stabilization methods on soft clayey soil in ex-mining lands are also described and discussed. Furthermore, various ground improvement methods are studied and discussed in the following part together with their cost aspects.

6.1 Experimental Study on Ground Improvement Methods at Sentul - Preloading and Sand Drains -

An experimental study was carried out at the pilot test area in Sentul. For the study, two ground improvement methods are selected, i.e. preloading with sand drains and preloading without drains, to observe in-situ behaviour of soft clayey materials. Fig. 6-1 shows the location of the test embankment and Fig. 6-2 shows the work progress of the experimental study.

6.1.1 Basic Principle of Preloading

The basic principle of preloading utilizes the consolidation characteristics of cohesive soil. The followings are their typical characteristics.

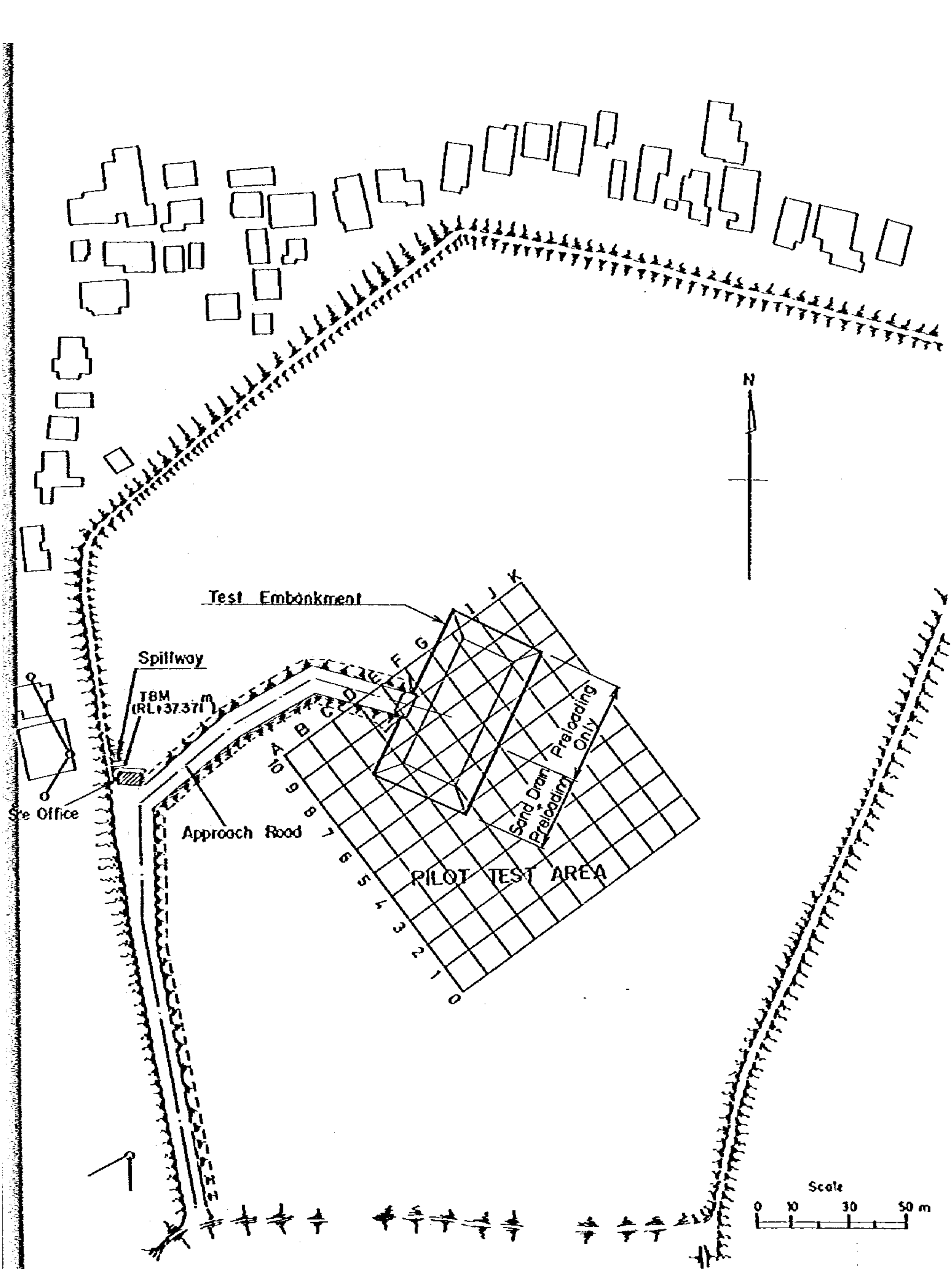


Fig. 6-1 Location of Test Embankment

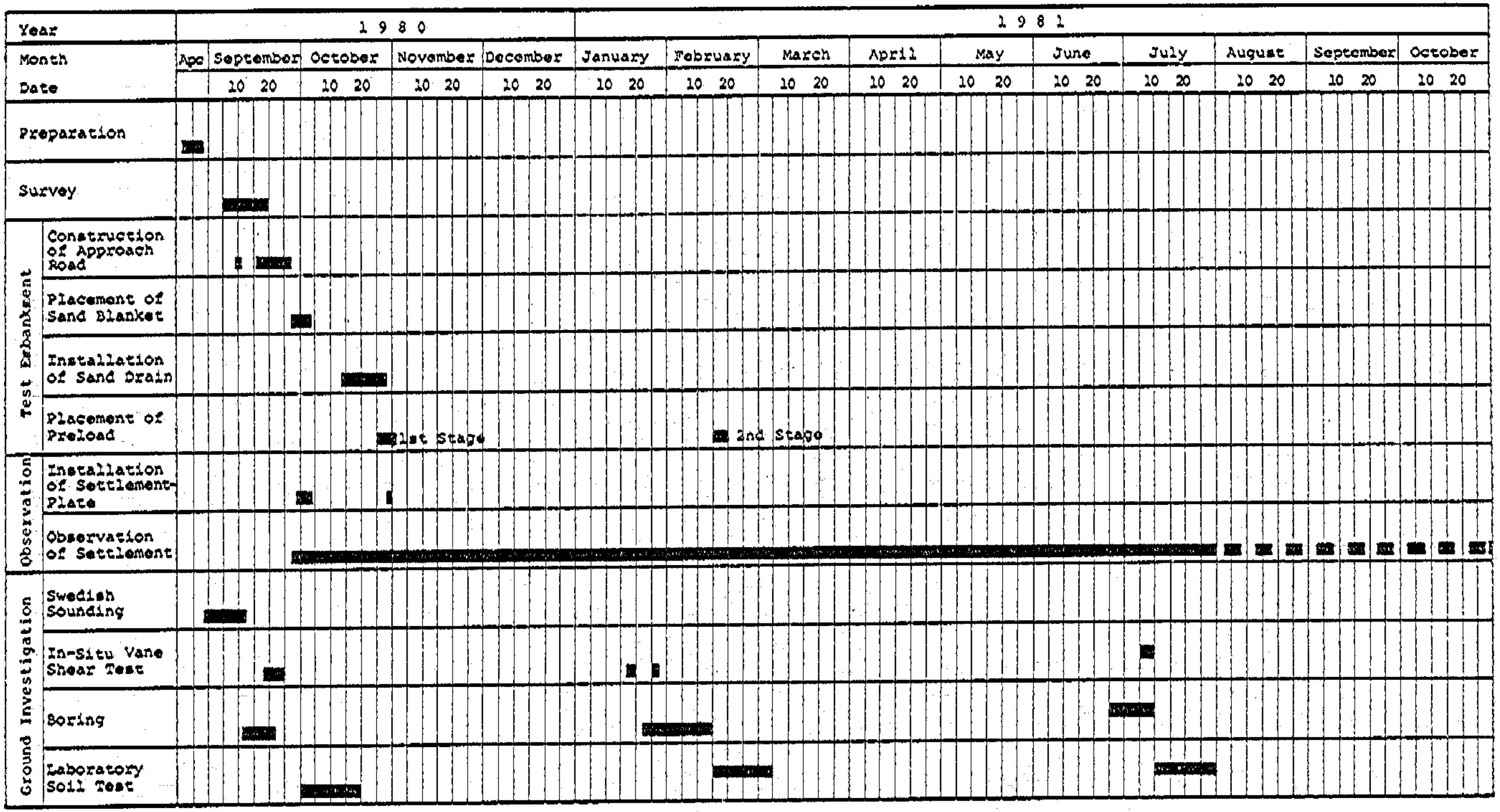


Fig. 6-2 Experimental Ground Improvement Work Progress Chart

1) When a soil element at normally consolidated state is imposed a load, resultant decrement of void ratio is large enough as shown by the points A and B in Fig. 6-3a. During this phase, undrained shear strength is increased as shown in Fig. 6-3b.

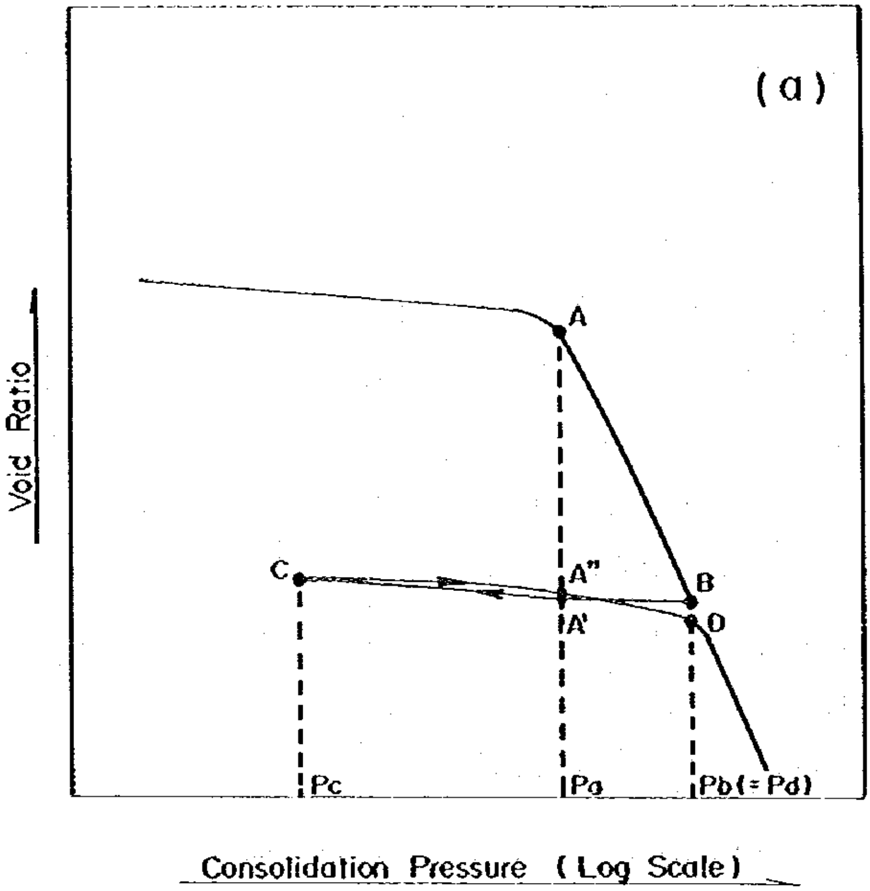
2) Then removing the load, state of the soil turns into overconsolidated state and resultant increment of void ratio and decrement of undrained shear strength is negligibly small.

3) Imposing the same load again, resultant decrement of void ratio is also neglibible as shown by the points C and D in Fig. 6-3a.

Therefore, the method of densification of the foundation soil under a load placed in advance of construction of the permanent structure, usually a temporary load being slightly heavier than that of the permanent structure, is to be a reasonable and useful method as shown in Fig. 6-4.

The main purpose is to eliminate most or all of the post construction primary consolidation, and sometimes a portion of the secondary compression. Preloading is also effective for densifying all loose and soft soils and thus increasing their strength.

For earth structures founded on soft soils, the permanent earth structure as well as the surcharge load, usually in the form of additional fill, are placed simultaneously. In the case of foundations other than earth structures, a preload is



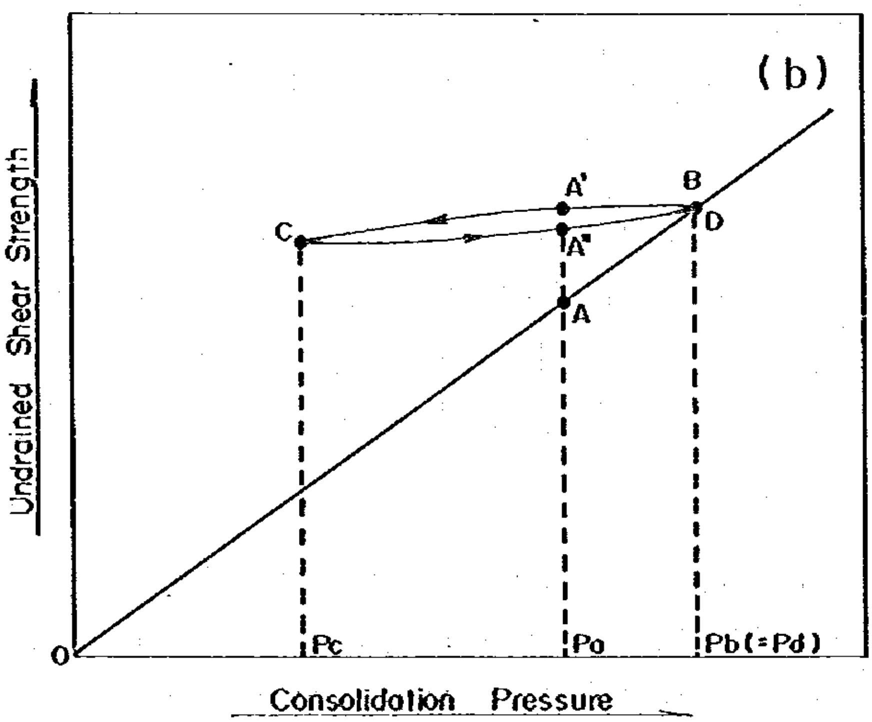


Fig. 6-3 Conceptional Drawing of Consolidation Characteristics of Cohesive Soil

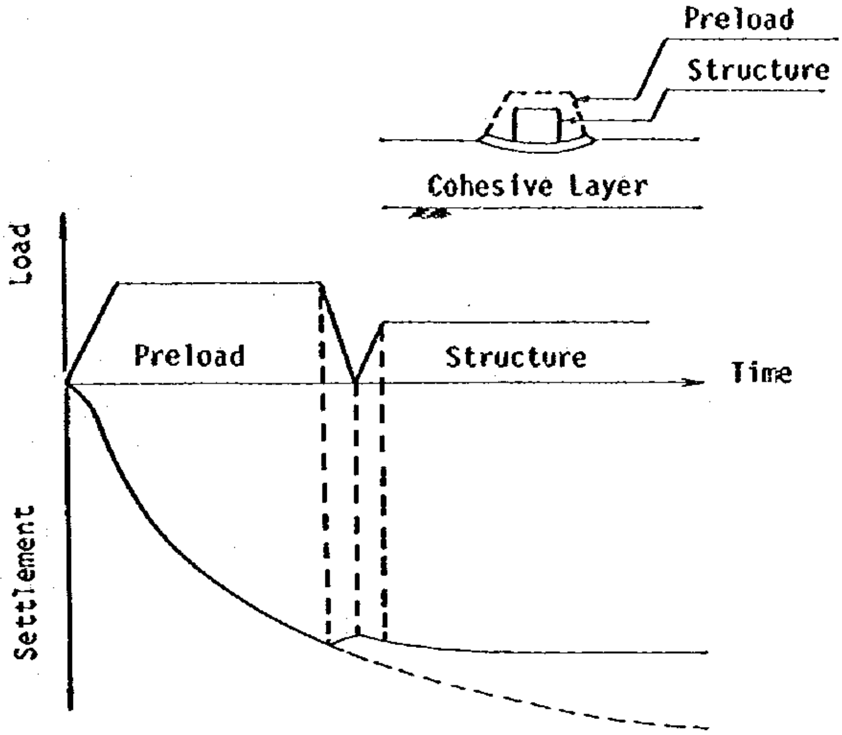


Fig. 6-4 Descriptive Drawing of Preloading Method

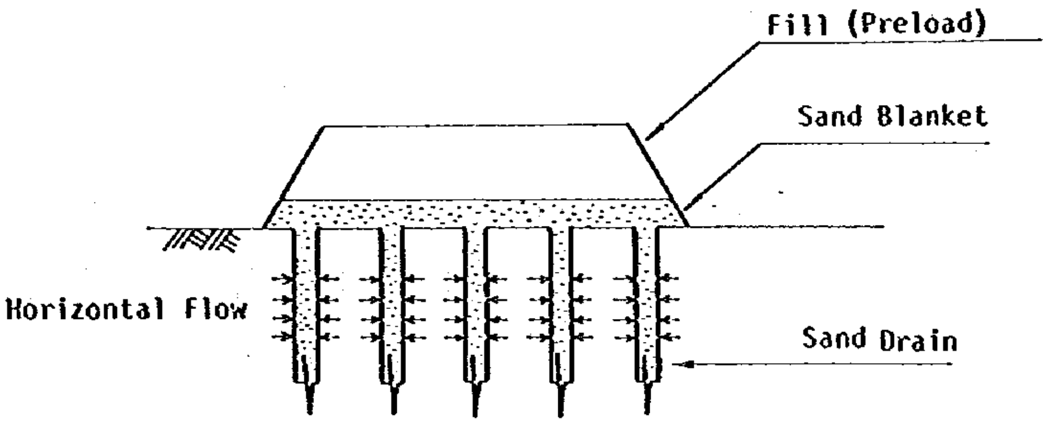


Fig. 6-5 Preloading with Sand Drains

placed on the proposed construction site before the construction. However, construction cannot begin until the preload period is over and the surcharge load removed.

Preloading is a powerful, reasonable and economical method. However, a long period is required for consolidation settlement to be completed. Since this period is proportional to H^2, where H denotes the length of drainage path of the soil, it can be shorter effectively by shortening the length of drainage path H.

Vertical drain is a method which can accelerate the consolidation of thick layers of soft, fine-grained soils with low permeability and inadequate drainage layers, by installing permeable piles into the ground which induce ground water to flow horizontally (Fig. 6-5).

6.1.2 Selection of Test Embankment Location

The location of the test embankment was selected at the pilot test area where comprehensive ground investigations had been performed in the Phase I study. In addition to the investigations in Phase I, Swedish soundings were carried out at 71 locations in the Phase 2 study to sound the thickness of soft clay layers. Based on the sounding results, an isothickness map of the soft clay layers was prepared as shown in Fig. 6-6. The test embankment was located in the portion where the soft clay is relatively thick. The thickness of the soft clay is about 12 to 15 m within the area of the test embankment. Fig. 6-7 shows a soil profile along the longitudinal section of the test embankment.

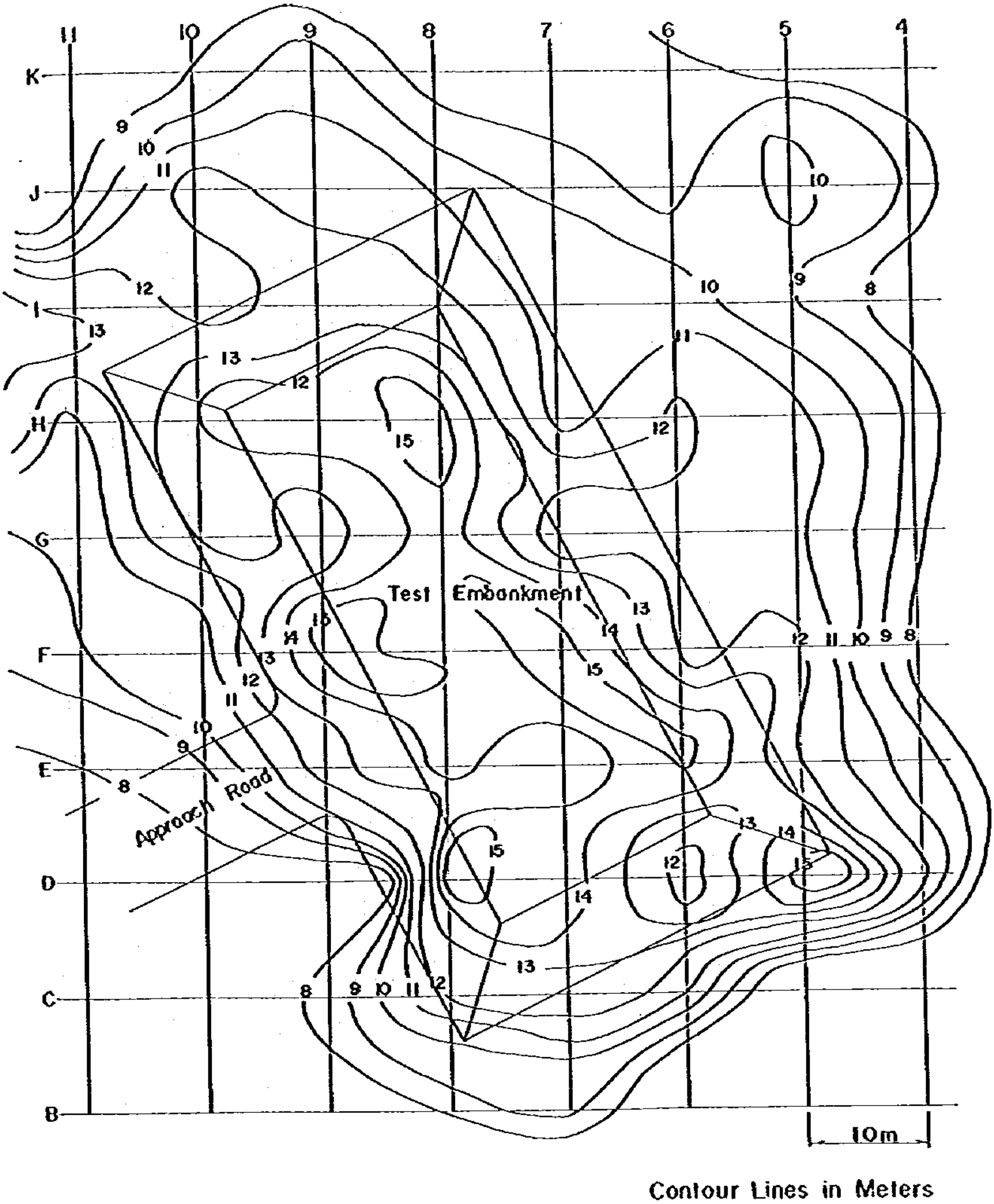


Fig. 6-6 Thickness of Soft Clay Layers

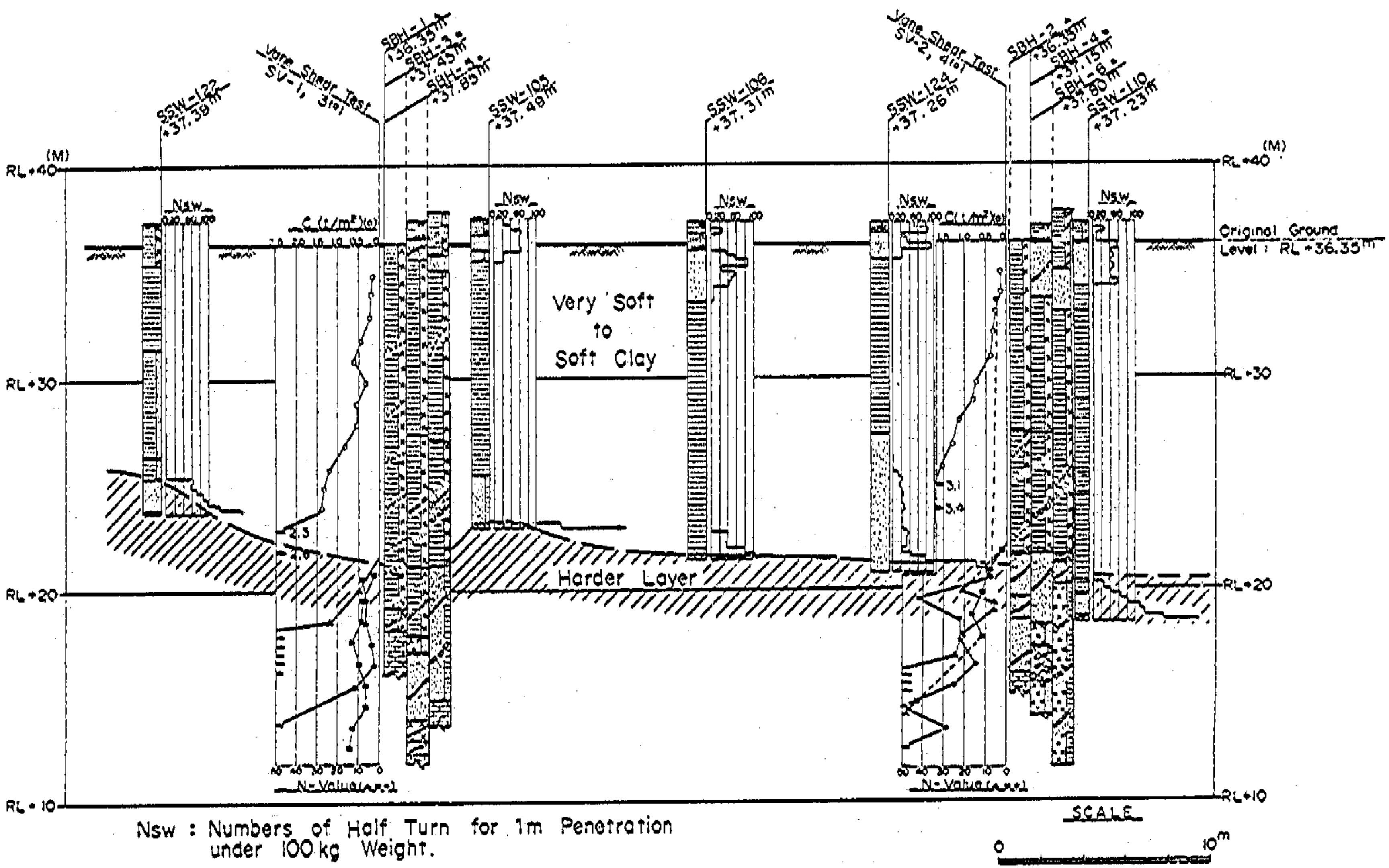


Fig. 6-7 Soil Profile at Test Embankment Area

6.1.3 Contents of Experimental Ground Improvement Work

As shown in Fig. 6-8, the experimental ground improvement work was composed of:-

1) Placement of Sand Blanket
2) Installation of Sand Drains
3) Placement of Preload

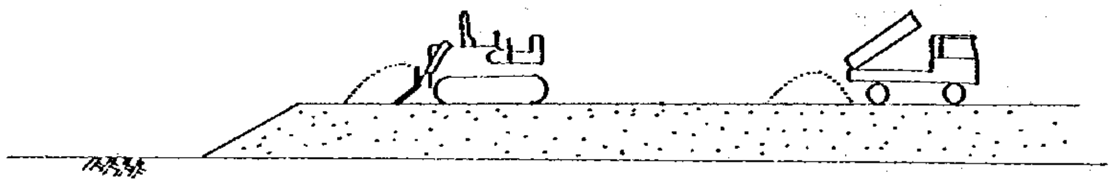
(1) Placement of Sand Blanket

Initially, it was planned that a sand blanket be placed only over the sand drain area. However, to improve the workability of the placement of the preload, the sand blanket was placed over the entire area of the test embankment. It was necessary to place a thicker sand blanket, i.e. about 1.5 to 2.0 m, in the area where sand drains were to be installed as the sand drain installation machine was heavier than the other earthwork machines. On the other hand, the thickness of the sand blanket is about 0.5 m at the area of preload only.

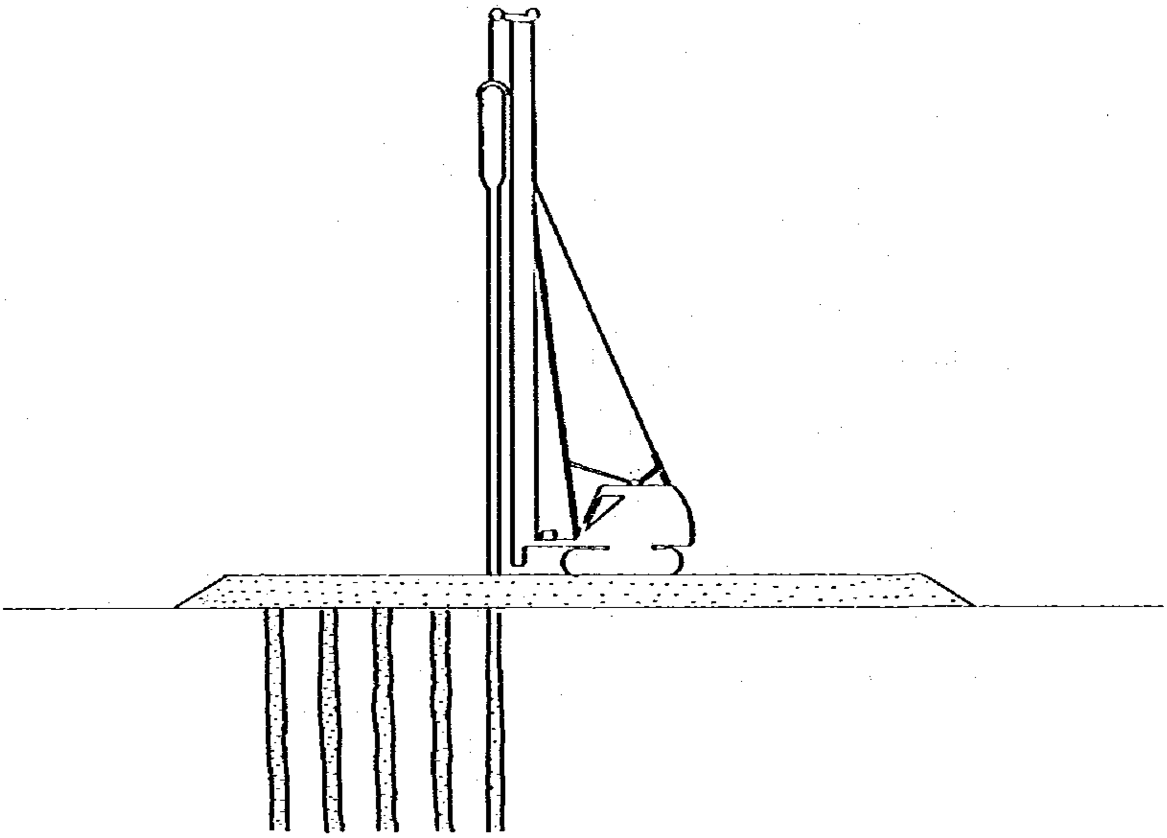
The mateiral used for the sand blanket was ex-mining sand which was washed and separated during tin-ore separation. The grading textures of the sand used for the sand blanket are shown in Fig. 6-9. The sand is clean and its permeability is estimated about 0.01 cm/sec.

(2) Installation of Sand Drains

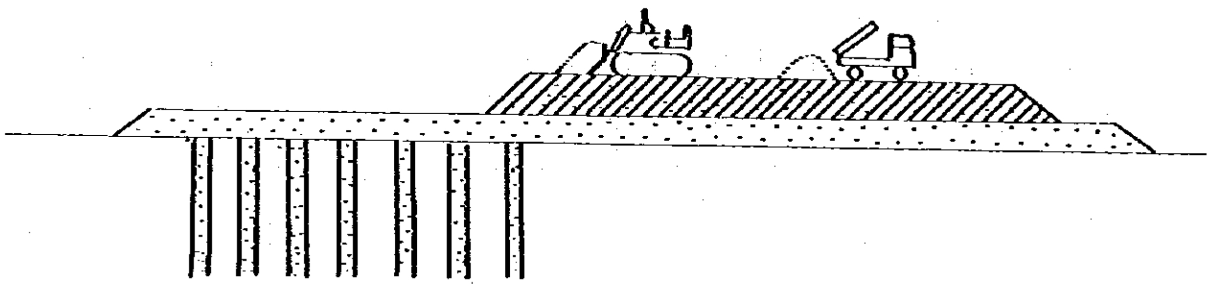
Installation of sand drains was carried out from the 13 to 28 Oct. 1980 by Fudo Construction Co., Ltd. Singapore Branch. As shown in Fig. 6-10, the layout of the



1. Placing Sand Blanket



2. Installation of Sand Drain



3. Placing Preload

Fig.6-8 Sequence of Sand Drain Installation and Preloading

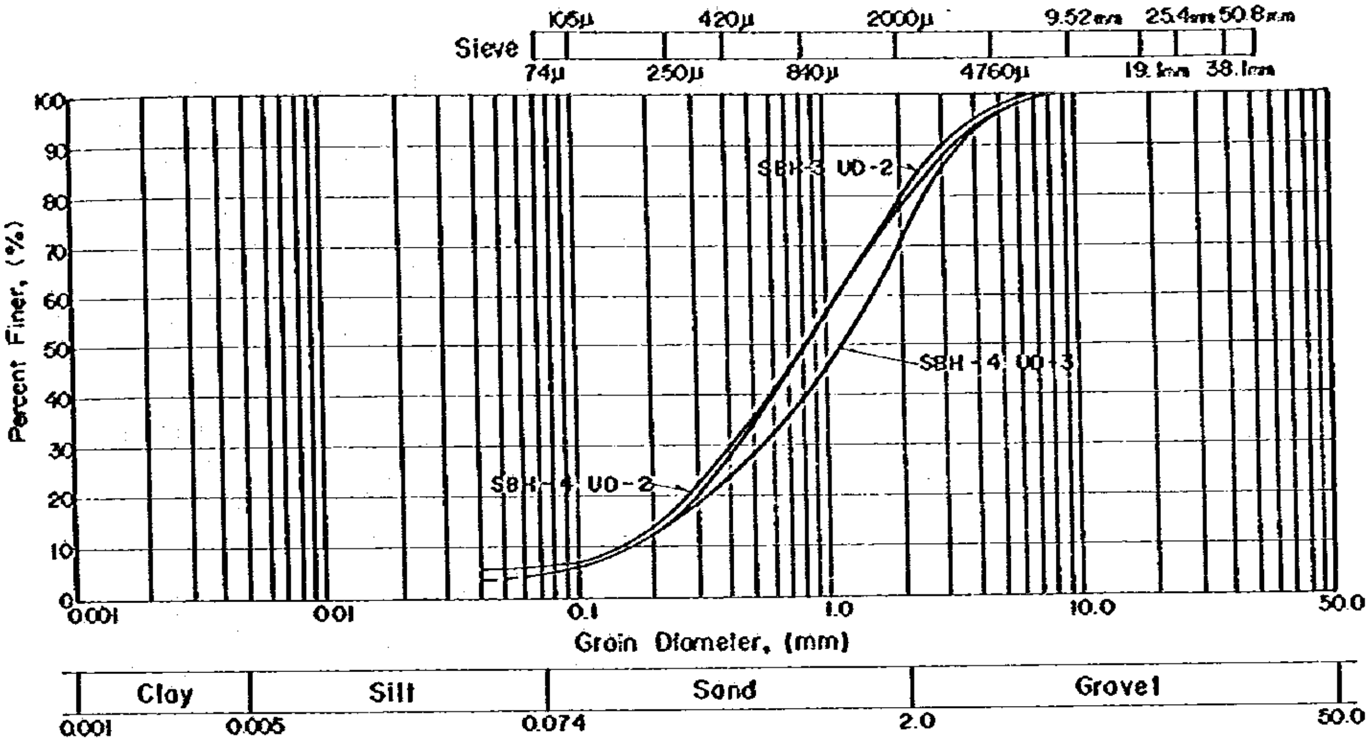


Fig.6-9 Grading Texture of Sand used for Sand Blanket and Sand Drain

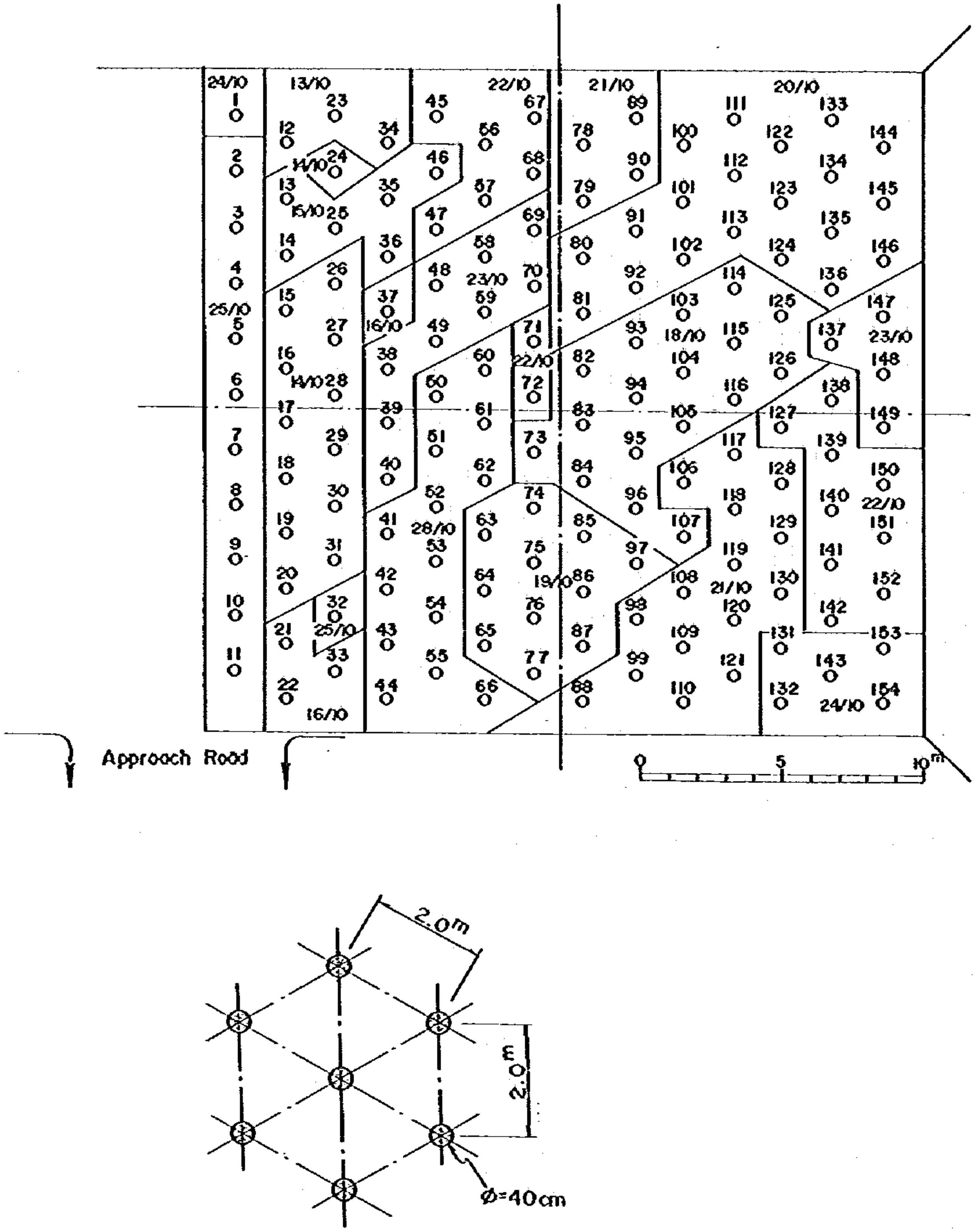


Fig. 6-10 Layout of Sand Drain Installation

sand drains is triangular with 2 m pitches and a diameter of 40 cm. The sand drains installed numbered 154 and were located in an area about 20 m by 20 m. The total length of sand drains installed was 2,260.6 m and the average length of each sand drain was 14.7 m. Fig. 6-11 shows the progress of sand drain installation. Due to the excessive softness of the ground, the progress was very slow when compared with the general rate of work. The material used for sand drains was the ex-mining sand from the same source as that of the sand blanket. The volume of sand used for sand drains is shown in Fig. 6-11.

(3) Placement of Preload

Preloading was executed in two stages as the underlying clay was too soft to place the intended amount of preload at once.

The first preload was placed immediately after completion of the sand drain installation. It was deposited during the period 26 Oct. to 1 Nov. 1980 to a level of about 1.3 m higher than the original ground level. Due to the sinking of the preload into the original soft ground, the average thickness of preloaded earth, including the sand blanket, was 3.5 m in the area where sand drains were installed and 1.6 m in the area with no sand drains. The configuration of the test embankment at the completion of the first stage of preload placement is shown in Figs. 6-12 and 6-13.

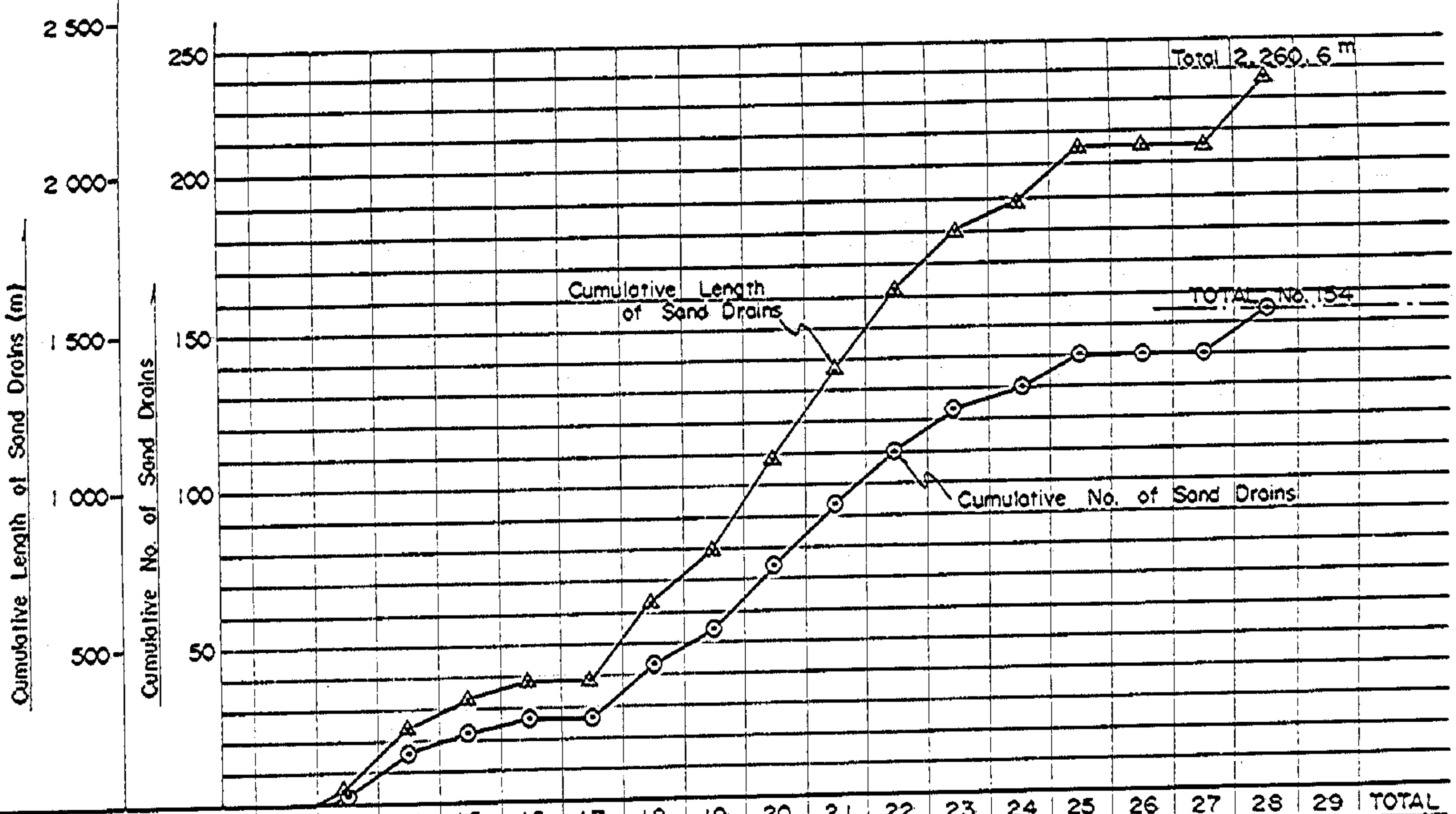
The second preload embankment was made on top of the first after a period of 110 days had elapsed from the time of the first preload placement. The second preload was placed during 16 to 20 Feb. 1981 to a level of about 1.75 m higher than that of the original ground. The average thickness of the second stage of preloaded earth was 1.0 m for the area with sand drains and 0.5 m for that with no sand drains. The configuration of the embankment at the completion of the second stage of preload placement is shown in Figs. 6-14 and 6-15.

The material used for both stages of the preload embankment was lateritic soil derived from a cut site at Jalan Duta. The grading texture of the soil used is shown in Fig. 6-16.

6.1.4 Results of Settlement Observation

Settlement of the ground under the load of the test embankment was measured by settlement plates installed during placement of the embankment. The location of the settlement plates is indicated in Figs. 6-12 to 6-15. A bench mark was fixed on the retaining wall near the spillway as shown in Fig. 6-1 (see Page 6-3) and was surveyed at RL + 37.371 m.

Observation of the settlement was conducted on a continuous basis subsequent to placement of the test embankment. Observation of the settlement commenced on 2nd November 1980, as the placement of the test embankment was completed on that day.



Date	Oct. 1980	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	TOTAL
Work Executed (Daily Basis)	No. of Sand Drains	0	3	13	6	4	0	17	11	20	19	17	13	6	11	0	0	14		154 Nos
	Total Length of Sand Drains	0	44.2	194.5	88.5	55.6	0	248.4	161.4	295.6	276.1	248.8	190.1	85.9	162.2	0	0	209.3		2260.6 m
	Sand Used (m^3)	0	7.15	31.9	13.2	10.45	0	40.7	25.3	46.2	41.8	41.25	33.55	14.3	24.2	0	0	30.8		360.8 m^3
Cumulatives	No. of Sand Drains	0	3	16	22	26	26	43	54	74	93	110	123	129	140	140	140	154		
	Length of Sand Drains	0	44.2	238.7	327.2	382.8	382.8	631.2	792.6	1088.2	1364.3	1613.1	1803.2	1889.1	2051.3	2051.3	2051.3	2260.6		

Fig. 6-11 Progress of Sand Drain Installation

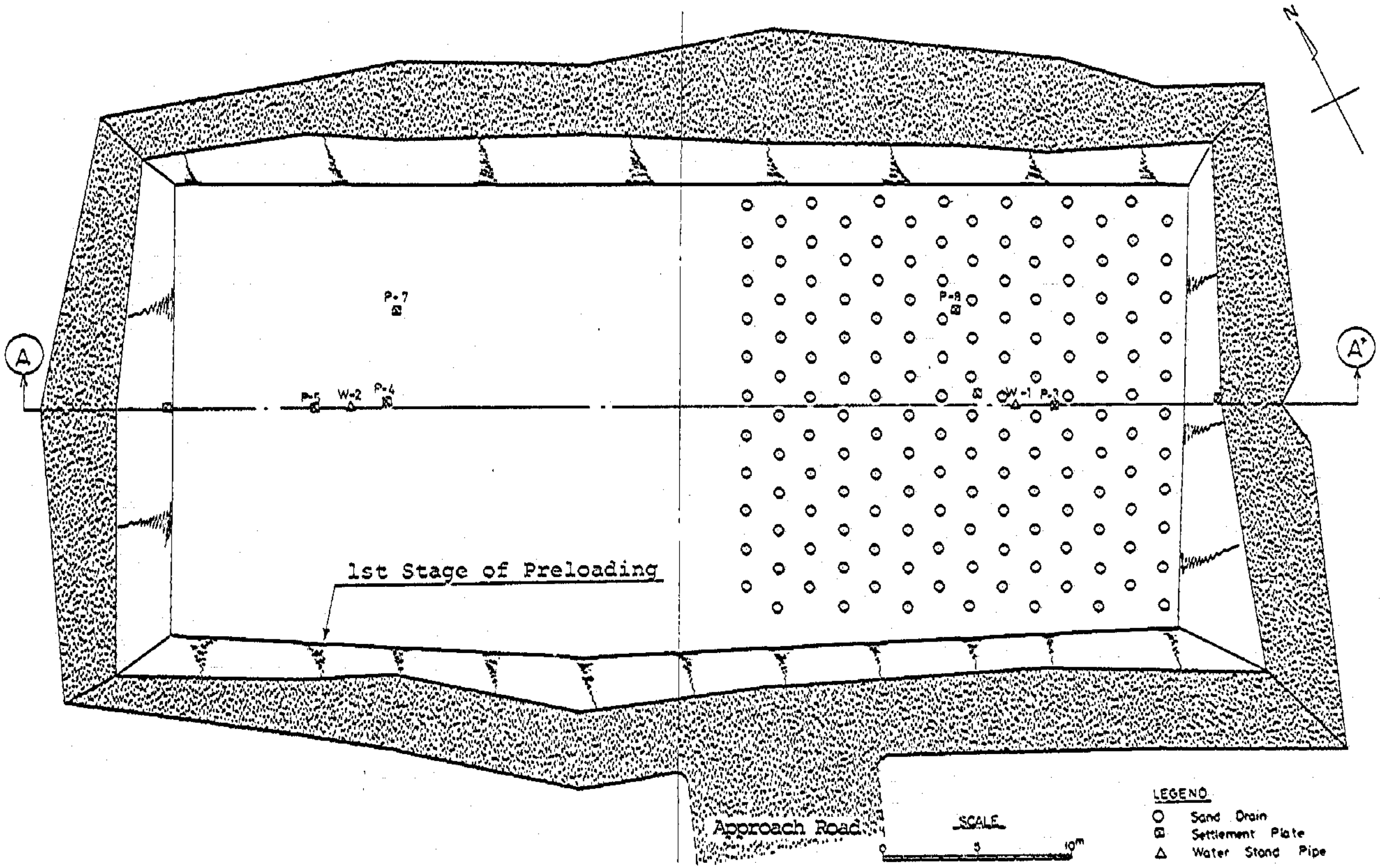


Fig. 6-12 Configuration of Test Embankment at Preload Placement and Locations of Sand Drains, Settlement Plates and Water Stand Pipes

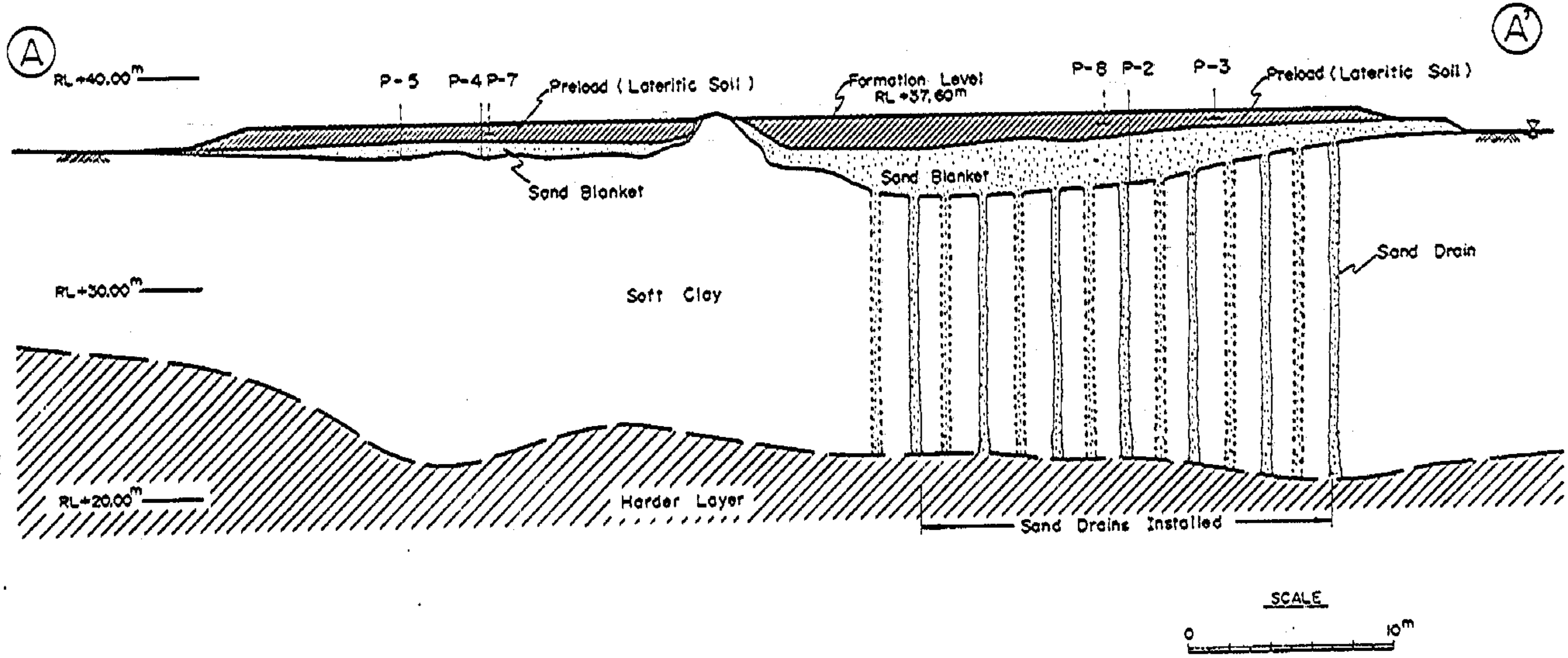


Fig. 6-13 Cross Section along Center Line of Test Embankment
(After 1st Embankment)

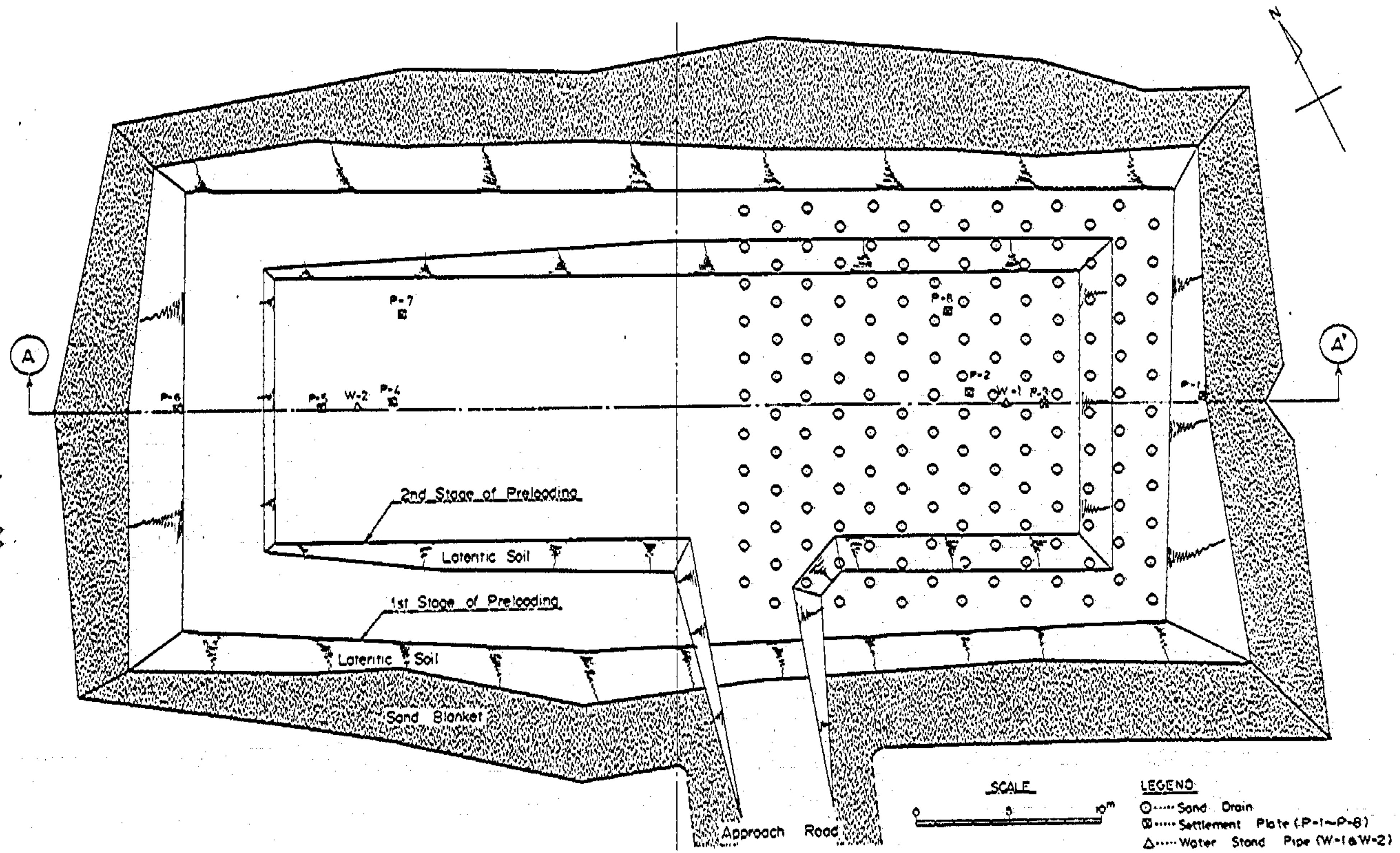


Fig. 6-14 Final Configuration of Test Embankment at Preload Placement and Locations of Sand Drains, Settlement Plates and Water Stand Pipes

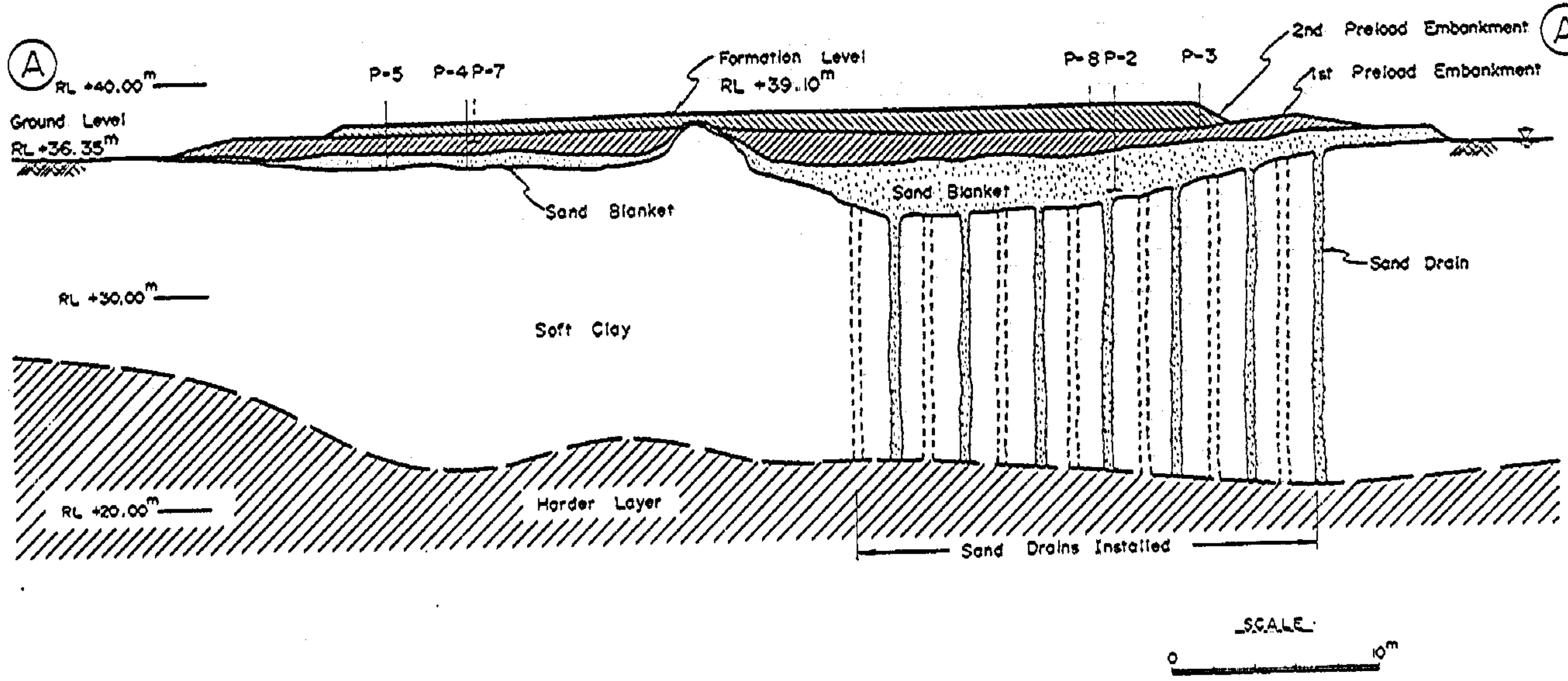


Fig. 6-15 Cross Section along Center Line of Test Embankment
(After 2nd Embankment)

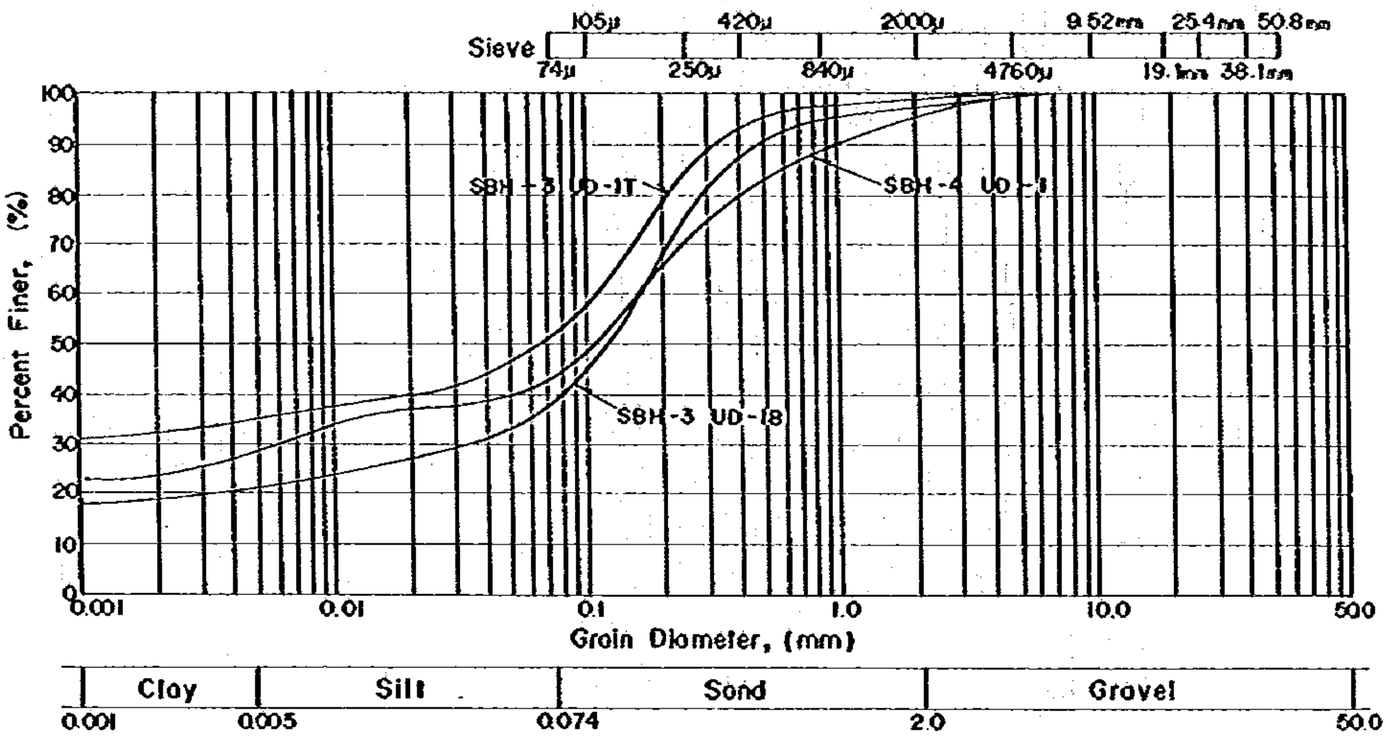


Fig. 6-16 Grading Texture of Lateritic Soils used for Preloading

In Figs. 6-17a to 6-17c, time versus settlement curves are plotted in arithmetric scale. The results are also plotted in root-time versus settlement curves in Figs. 6-18a and 6-18b, and logarithm of time versus settlement curves in Figs. 6-19a to 6-19d. In all of these curves, indications are that settlement due to the consolidation of soft clay is still in progress and that primary consolidation is not completed. It is also known that the rate of settlement in the area with sand drains is much faster than the rate in the area without sand drains. Three settlement plates installed in the area with sand drains (P-2, P-3 and P-8) show similar patterns and magnitudes of settlement, indicating the satisfactory performance and accuracy of the experiment. Similar observations can be made regarding the three settlement plates installed in the zone without sand drains (P-4, P-5 and P-7).

6.1.5 Study on Observation Results

The total amount and time rate of consolidation settlement are analysed by the following two methods: -

1) the results of laboratory consolidation tests performed on undisturbed samples obtained from the site, and
2) the results of settlement observations under the test embankment.

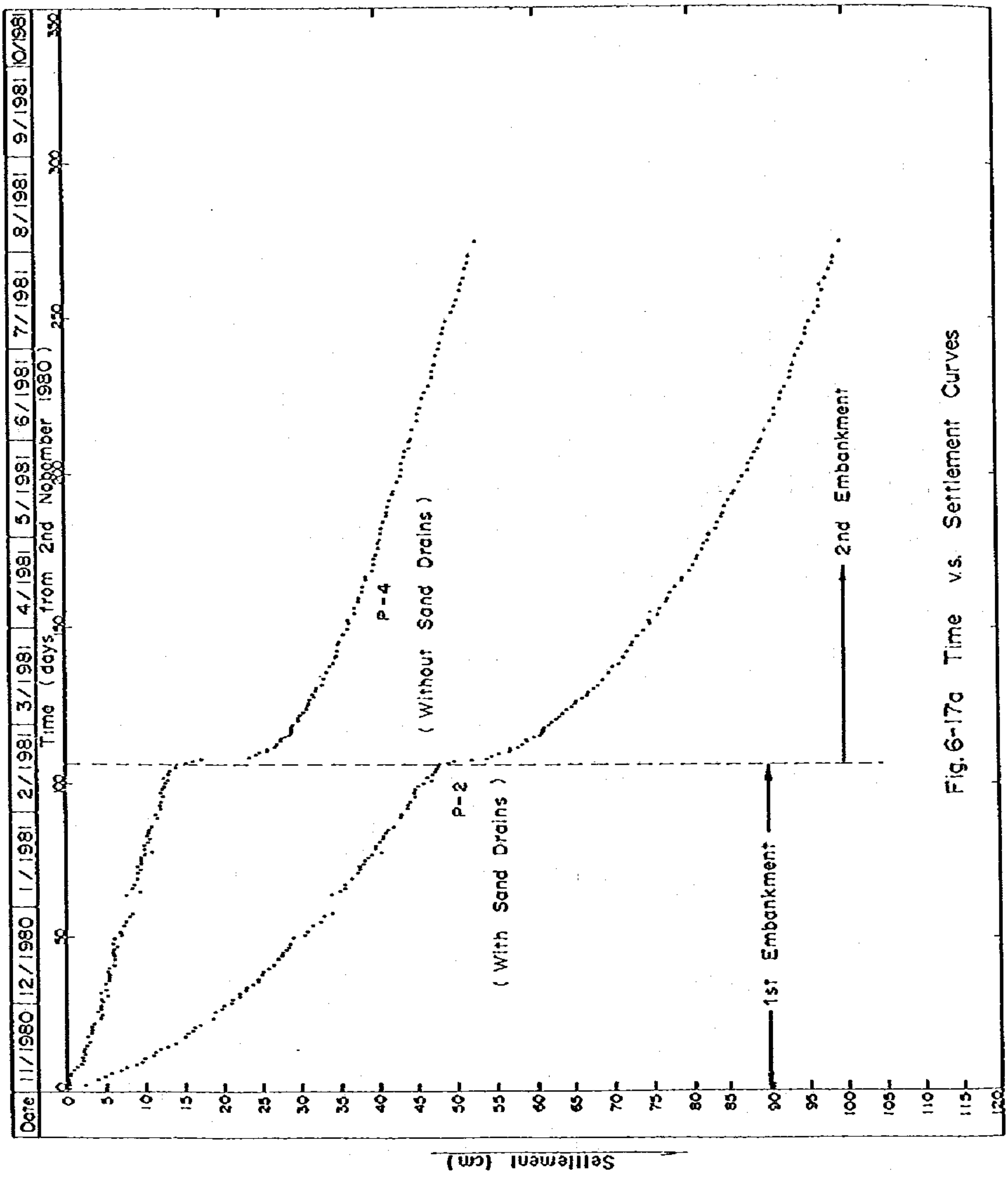


Fig. 6-17a Time v.s. Settlement Curves

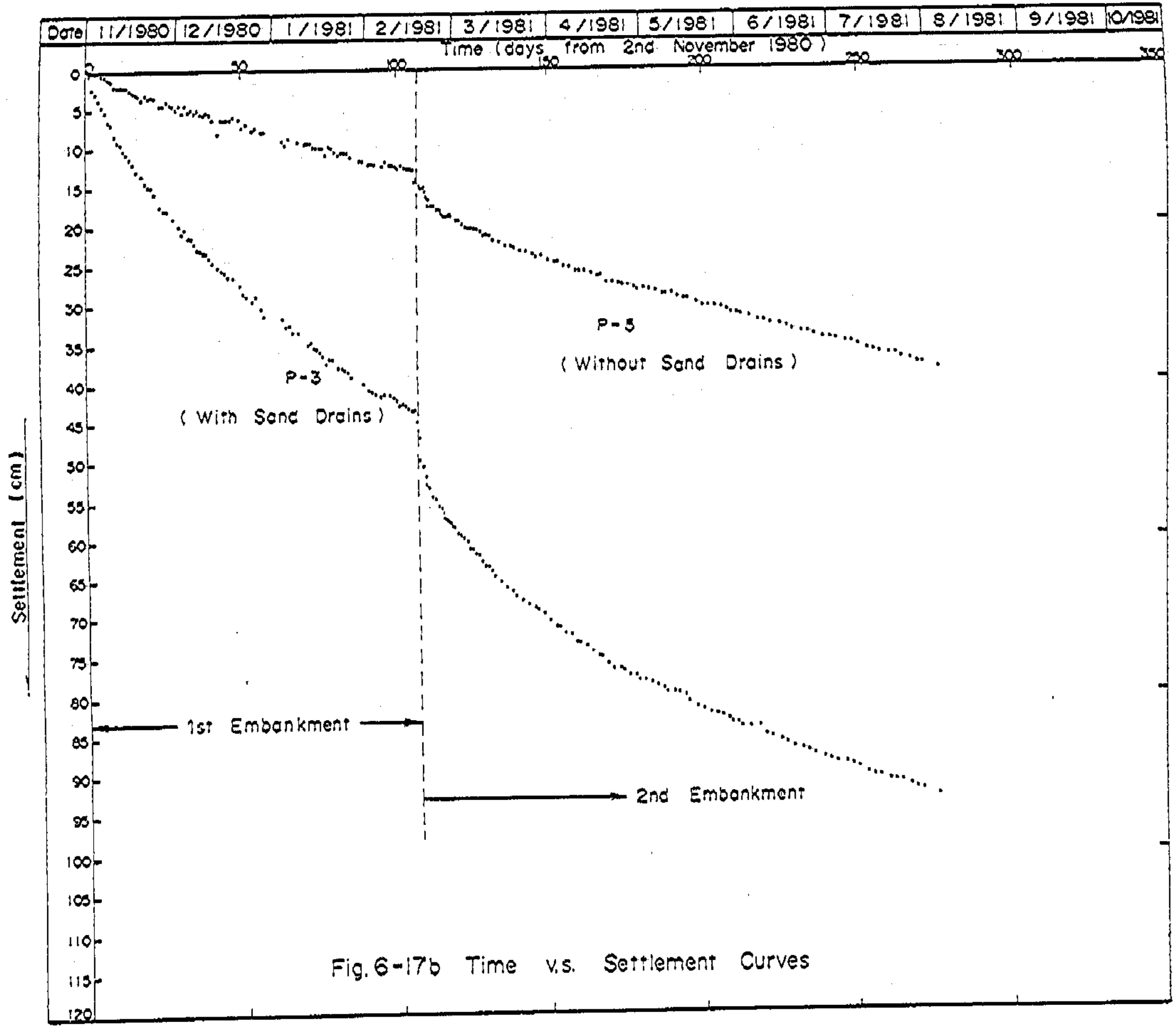


Fig. 6-17b Time v.s. Settlement Curves

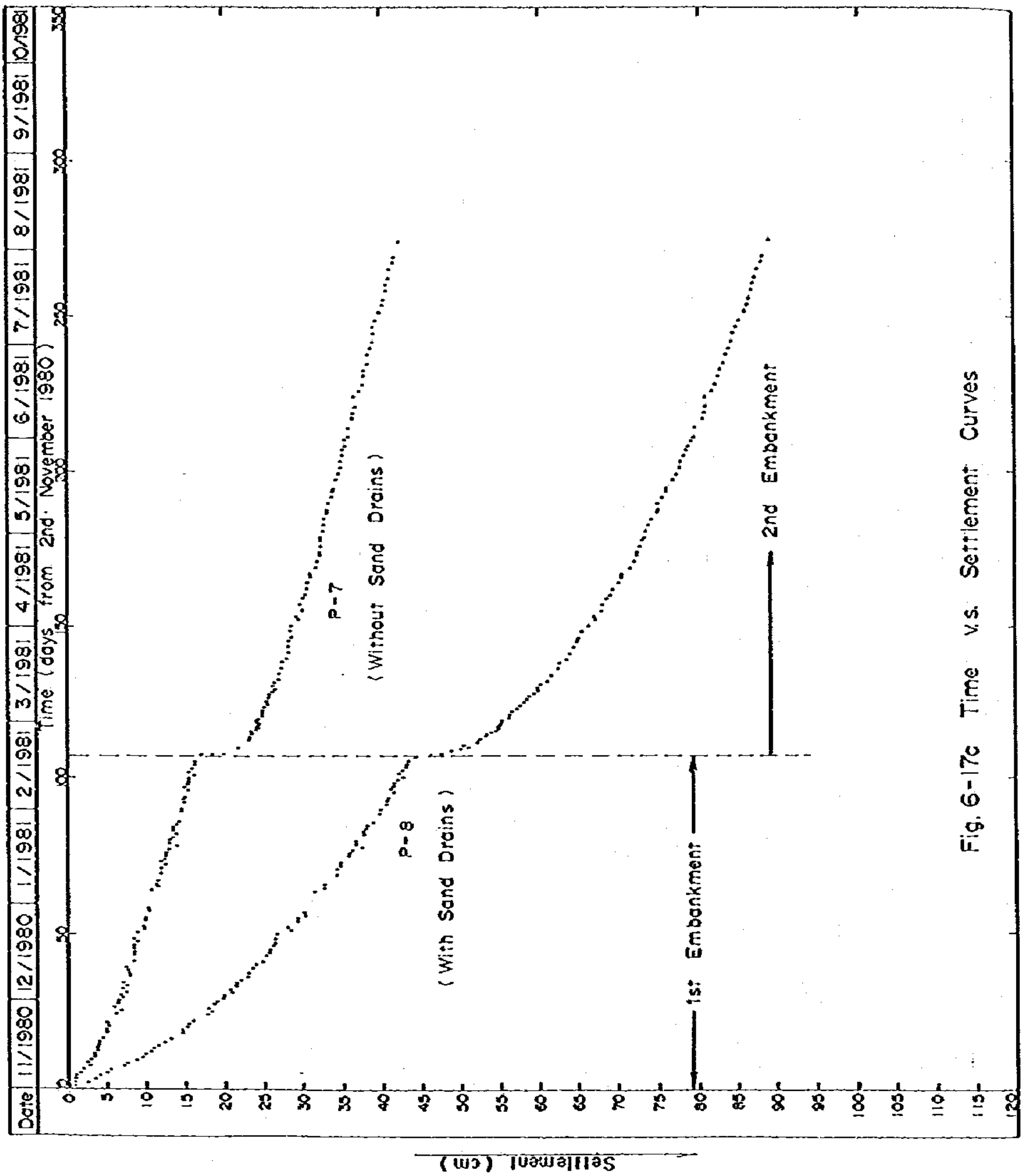


Fig. 6-17c Time v.s. Settlement Curves

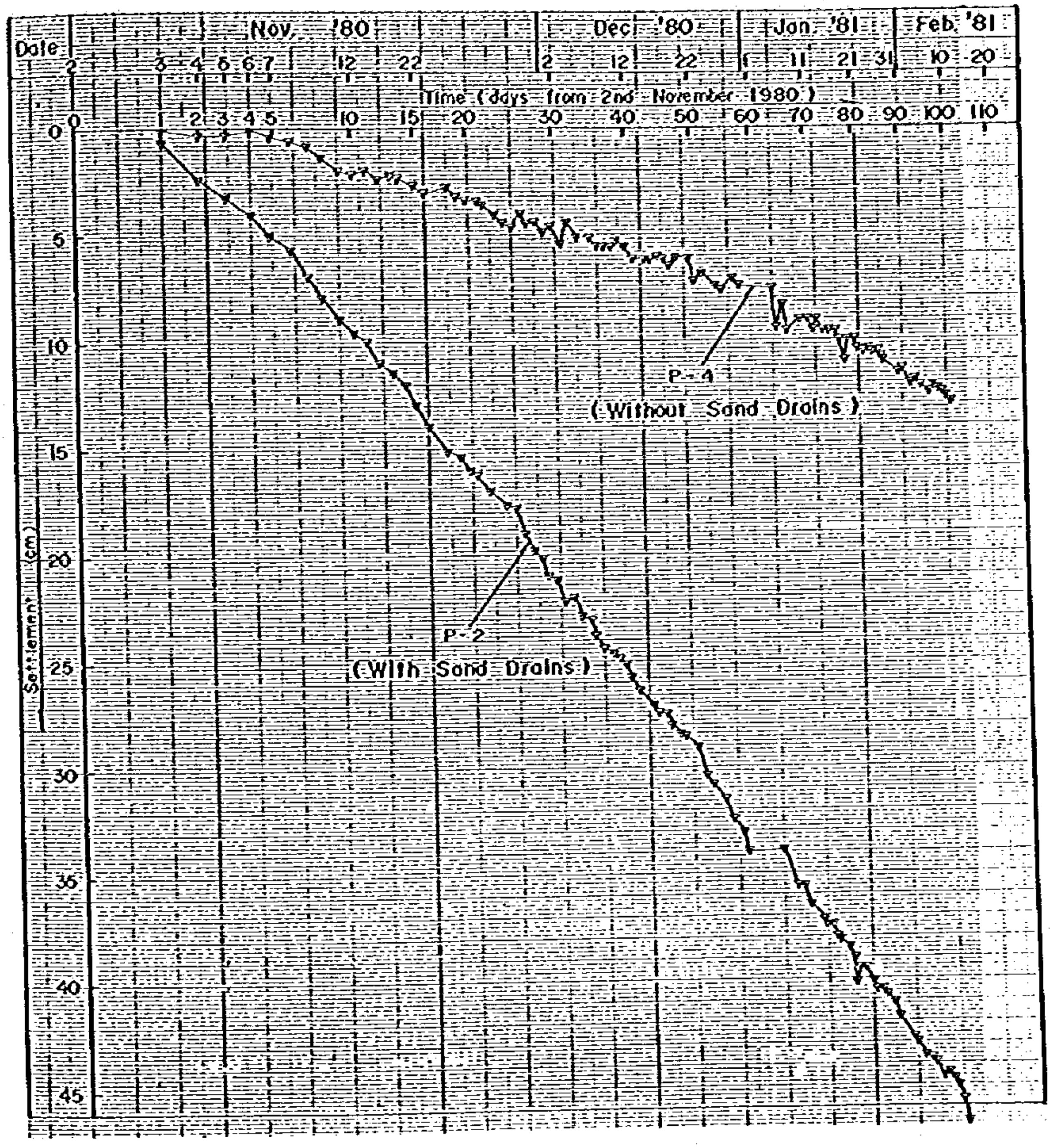


Fig. 6-18a Root-Time vs. Settlement Curves
(For 1st Preloading)

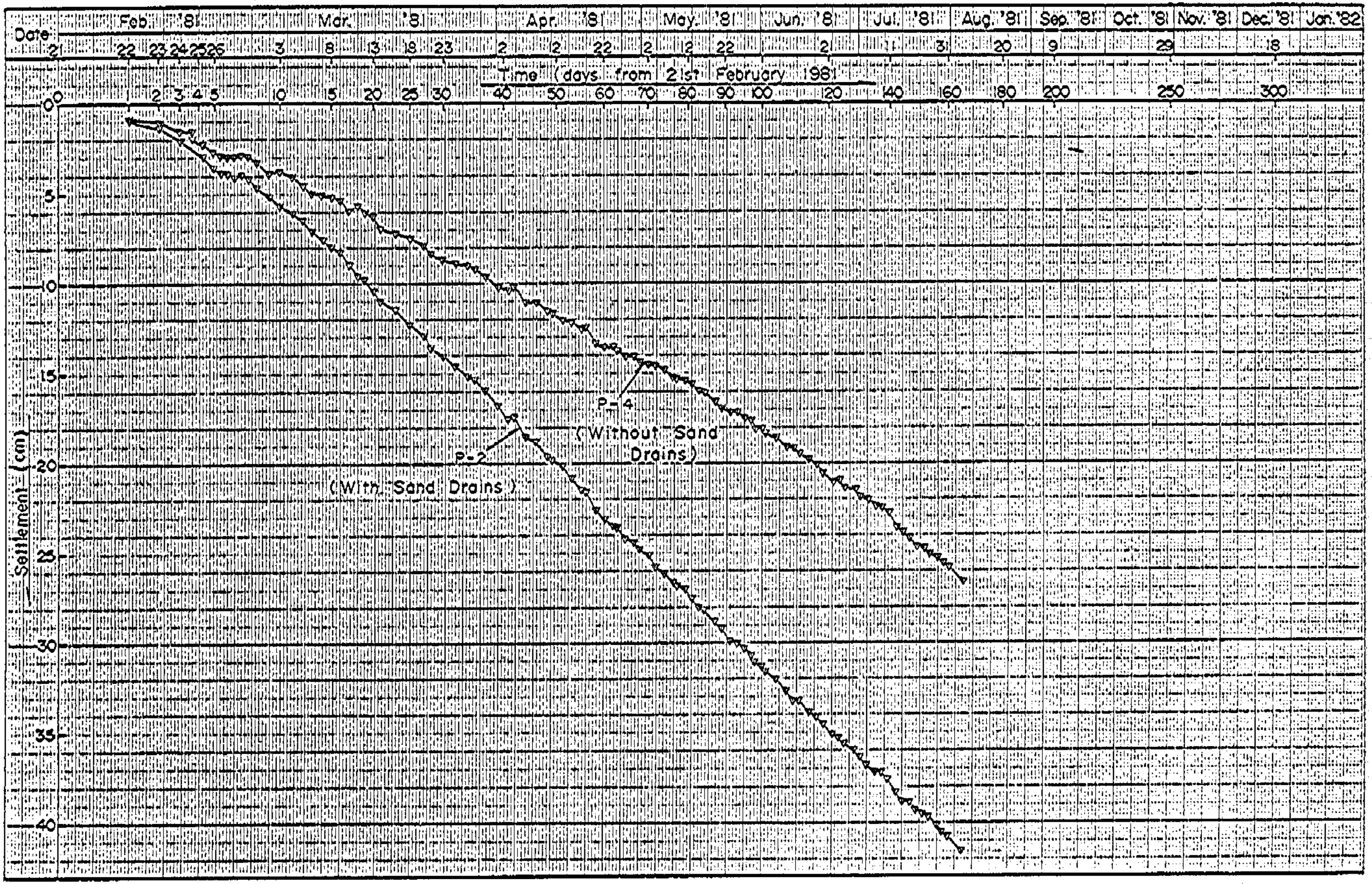


Fig. 6-18b Root-Time v.s. Settelement Curves
(For 2nd Preloading)

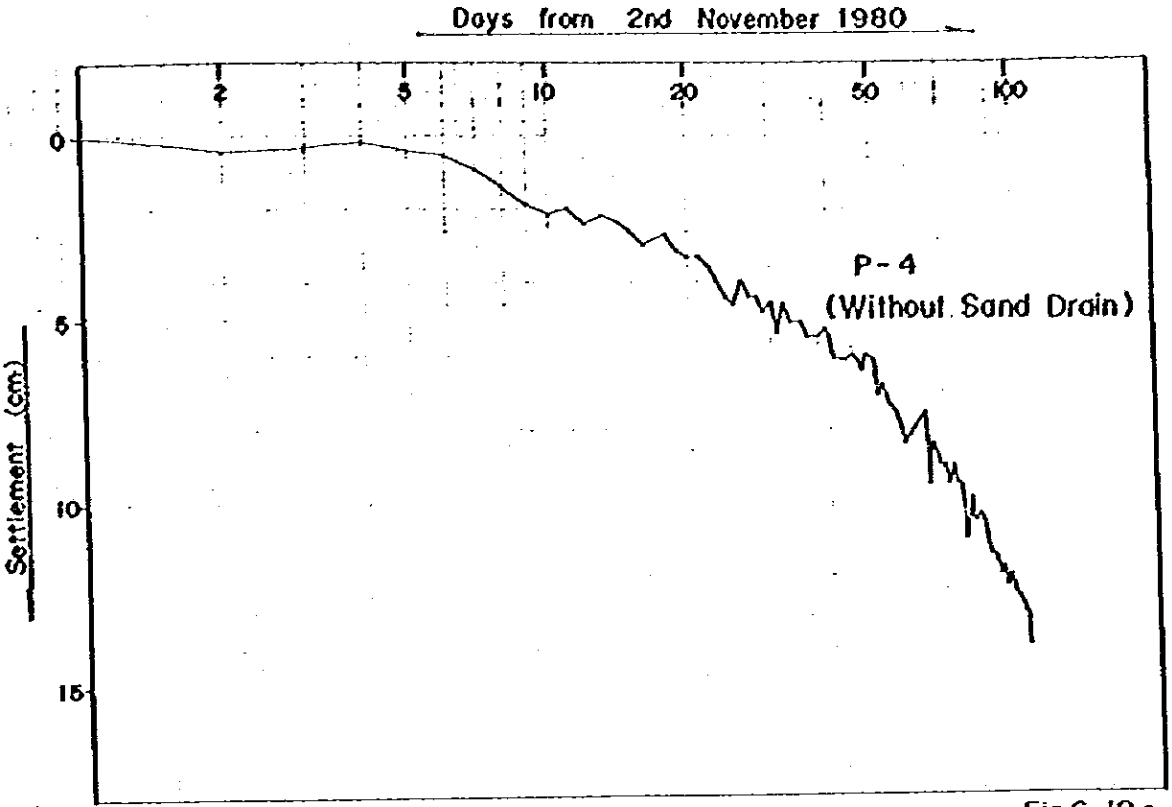


Fig.6-19a

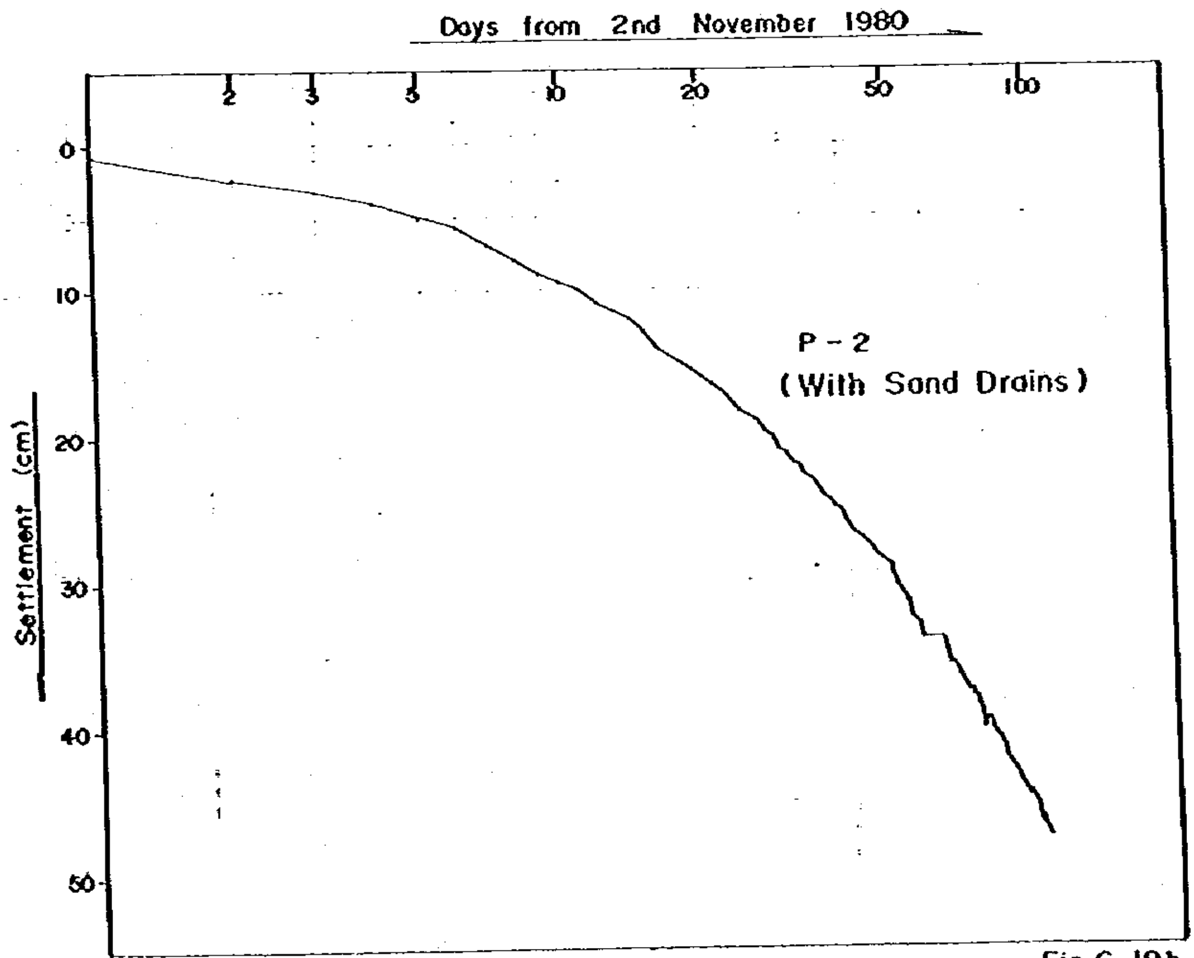


Fig.6-19b

Fig. 6-19a & b Logarithm of Time v.s. Settlement Curves (For 1st Preloading)

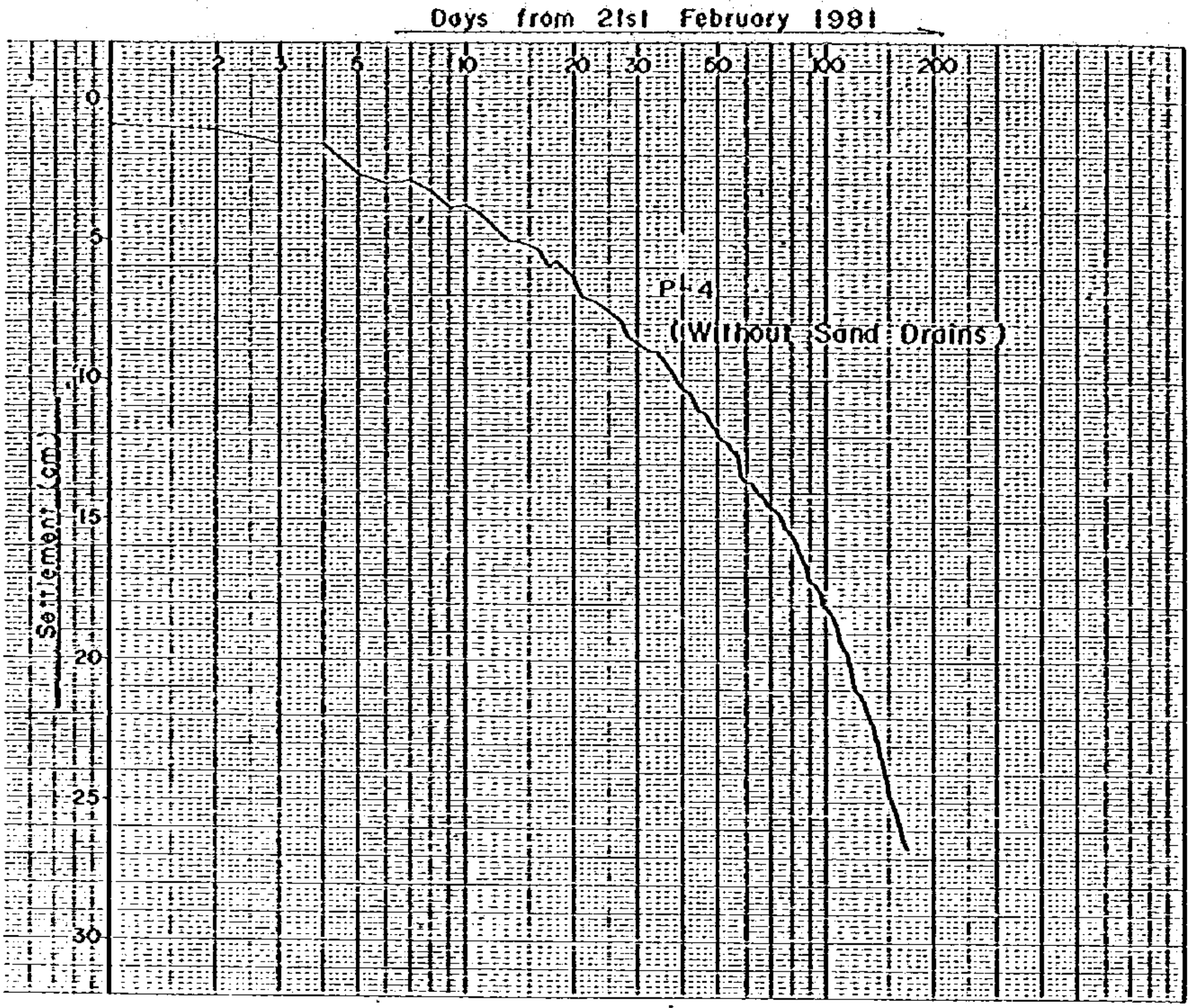


Fig.6-19c.

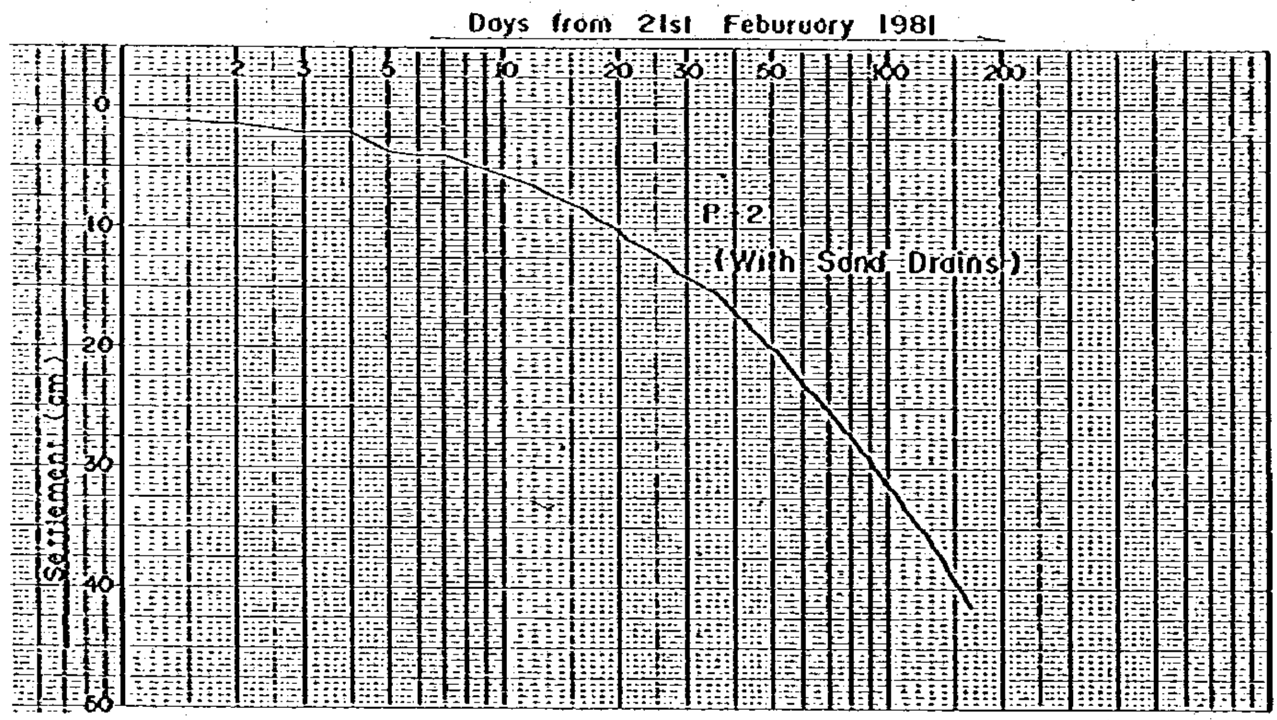


Fig.6-19d

Fig.6-19c & d Logarithm of Time v.s. Settlement Curves
(For 2nd Preloading)

(1) <u>Total Amount of Consolidation Settlement</u>

Results of the estimation of the total (final) amount of consolidation settlement are shown in Table 6-1. For the first stage test embankment of 1.3 m, the total amount of consolidation settlement, estimated as based on the results of laboratory soil tests, is about 80 cm for both areas (with or without sand drains). On the other hand, when analysed by the hyperbolic fitting method, the results of the settlement observations predict total settlement of 30 to 40 cm for the area without sand drains and 75 to 82 cm for the area with sand drains.

Table 6-1 Estimation of Total Settlement

		After Laboratory Soil Tests	After Field Observation (by Hyperbolic Fitting Method)
Under 1st Stage Preload	Without Sand Drains	80 cm	30 ∿ 40 cm
	With Sand Drains	80 cm	75 ∿ 82 cm
Under 2nd Stage Preload	Without Sand Drains	65 cm	61 ∿ 69 cm
	With Sand Drains	65 cm	69 ∿ 77 cm

The second stage of the preload embankment was placed on the first stage embankment to reach a height of 1.75 m from the original ground level. The total amount of consolidation settlement to be effected by the 2nd embankment is about 65 cm for both areas as based on the results of laboratory soil tests. On the other hand, the total amount of consolidation settlement for the 2nd embankment as predicted by the field settlement observation is 61 to 69 cm for the area with no sand drains and 69 to 77 cm for that with sand drains. Both of the total consolidation settlement amounts for the 2nd embankment as predicted by laboratory tests and field observation results show good agreement.

(2) Time Rate

The time required to complete 50% of the consolidation settlement, t_{50}, and that for 90%, t_{90}, are also estimated by two methods, i.e. after laboratory tests and after actual observation results.

(a) After Laboratory Soil Test

The average of the coefficient of consolidation, c_v, obtained from laboratory tests was 0.05 cm^2/min. By applying this value to the field drainage condition, the times required for 50% and 90% consolidation were estimated. Drainage conditions used in the estimation were:-

1) Area without Sand Drains

Drained at upper and lower boundaries, i.e.

$$H = 1/2 \times 15 \text{ m} = 7.5 \text{ m}$$

2) Area with Sand Drains

Diameter of sand drains is 40 cm with layout of the same being triangular on 2 m centres. Thus, effective drainage diameter, de, is 2.10 m. Coefficient of consolidation in horizontal direction, c_h, is assumed to be same as vertical direction.

(b) After Field Observation

From the results of field observations of settlement, and by using the hyperbolic fitting method, t_{50} and t_{90} are obtained.

The times required to complete 50% and 90% of the consolidation settlement are obtained by the two methods and are shown in Tables 6-2a and 6-2b. The following observations can be made on the results shown in these tables.

(a) Effect of Sand Drains

The effects of sand drains are clearly seen through the field experimental study. The rate of settlement in the area with sand drains was much faster than the area without sand drains. Days needed for t_{50} and t_{90} in the sand drain area were about one-third of those in areas without sand drains.

Table 6-2a Estimation of the Time Rate of Settlement (For 1st Stage Preload)

Degree of Consolidation	Sand Drain		After Laboratory Soil Tests	After Field Observation (by Hyperbolic Fitting Method)
50%	Without Sand Drains	t_{50}	1540 days* (4.2 years)	200 ∿ 230 days#
		c_v	$5 \times 10^{-2} cm^2/min$#	$3.3 \sim 3.8 \times 10^{-1} cm^2/min$*
	With Sand Drains	t_{50}	51 days*	80 ∿ 85 days#
		c_h	$5 \times 10^{-2} cm^2/min$#	$3.0 \sim 3.2 \times 10^{-2} cm^2/min$*
90%	Without Sand Drains	t_{90}	6600 days*	2100 ∿ 2300 days#
		c_v	$5 \times 10^{-2} cm^2/min$#	$1.4 \sim 1.5 \times 10^{-1} cm^2/min$*
	With Sand Drains	t_{90}	190 days*	720 ∿ 770 days#
		c_h	$5 \times 10^{-2} cm^2/min$#	$1.2 \sim 1.4 \times 10^{-2} cm^2/min$*

Known from test or experiment

* Estimated from known value

Table 6-2b Estimation of the Time Rate of Settlement
(For 2nd Stage Preload)

Degree of Consolidation	Sand Drain		After Laboratory Soil Tests	After Field Observation (by Hyperbolic Fitting Method)
50%	Without Sand Drains	t_{50}	1300 days*	330 days‡
		c_v	$5 \times 10^{-2}cm^2/min$‡	$2.0 \times 10^{-1}cm^2/min$*
	With Sand Drains	t_{50}	51 days*	140 days‡
		c_h	$5 \times 10^{-2}cm^2/min$‡	$4.6 \times 10^{-2}cm^2/min$*
90%	Without Sand Drains	t_{90}	5800 days*	3000 days‡
		c_v	$5 \times 10^{-2}cm^2/min$‡	$9.6 \times 10^{-2}cm^2/min$*
	With Sand Drains	t_{90}	190 days*	1270 days‡
		c_h	$5 \times 10^{-2}cm^2/min$‡	$0.75 \times 10^{-2}cm^2/min$*

* Known from test or experiment

‡ Estimated from known value