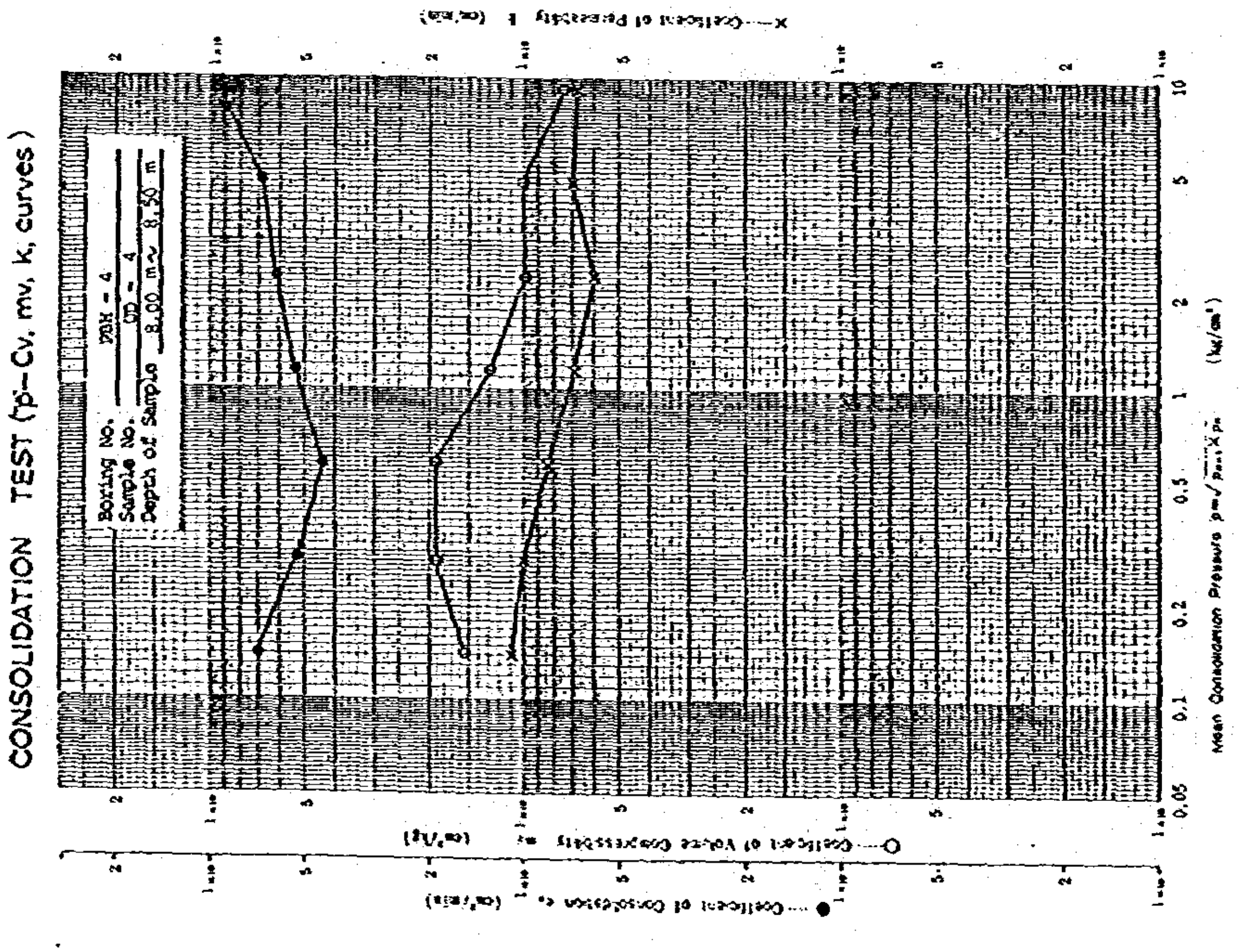

CONSOLIDATION TEST (p̄-Cv, mv, k, curves)
Boring No. PBH - 4
Sample No. UD - 4
Depth of Sample 8.00 m~ 8.50 m
Mean Consolidation Pressure (kg/cm²)
Coefficient of Consolidation cv (cm²/min)
Coefficient of Volume Compressibility mv (cm²/kg)
Coefficient of Permeability k (cm/min)

F.4 Results of Laboratory Soil Tests on Samples from Castlefield North

Summary of Soil Test (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	12.35 - 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	1.67
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	-	-	-	-

TRIAXIAL COMPRESSION TEST (Mohr's circle)

Boring No. CNBH-1 Sample No. UD-2Bottom

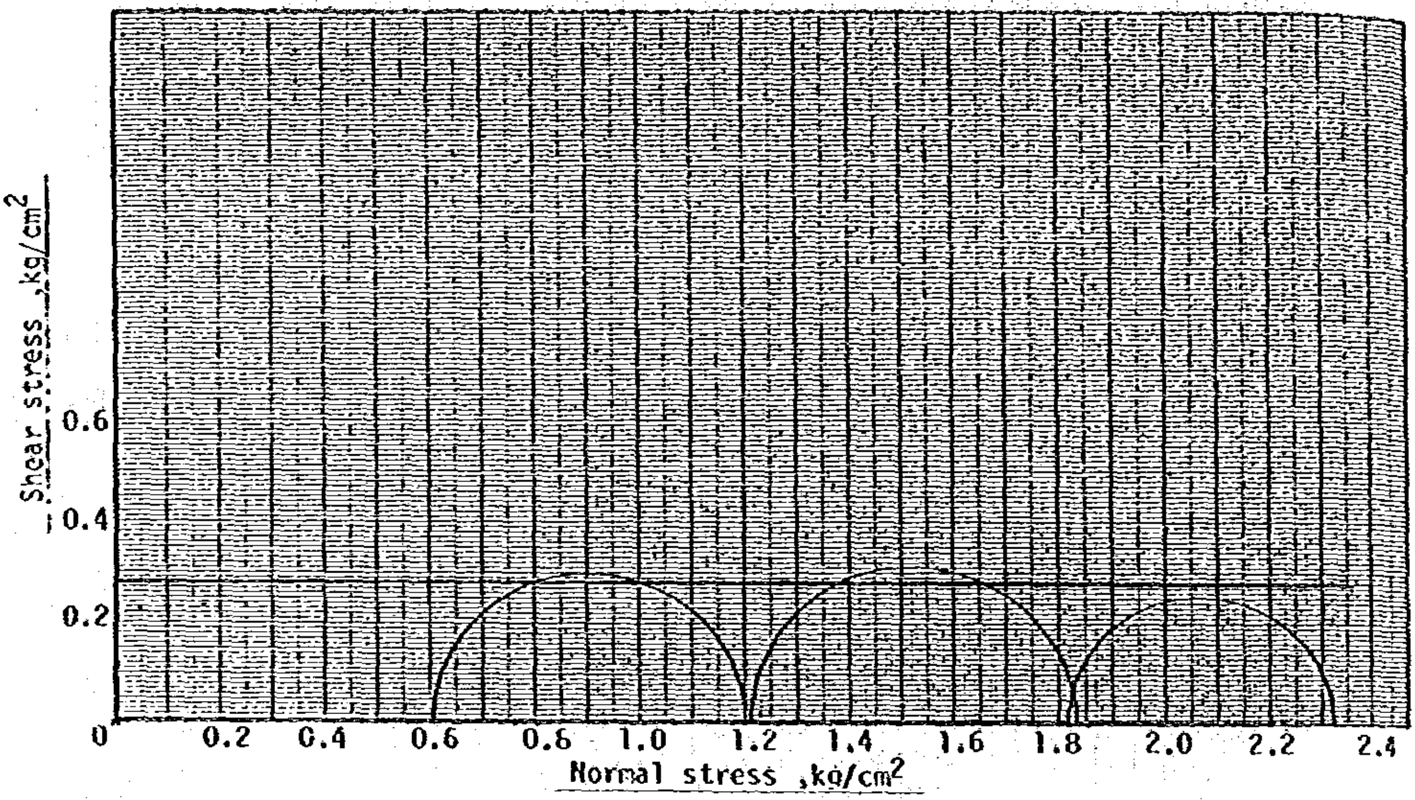
Project 267

Depth of Sample 7.25 m ~ 8.30 m

Angle of internal friction 0°

Condition of drainage U-U

Cohesion 0.23 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Boring No. CNBH-1 Sample No. UD-3Top

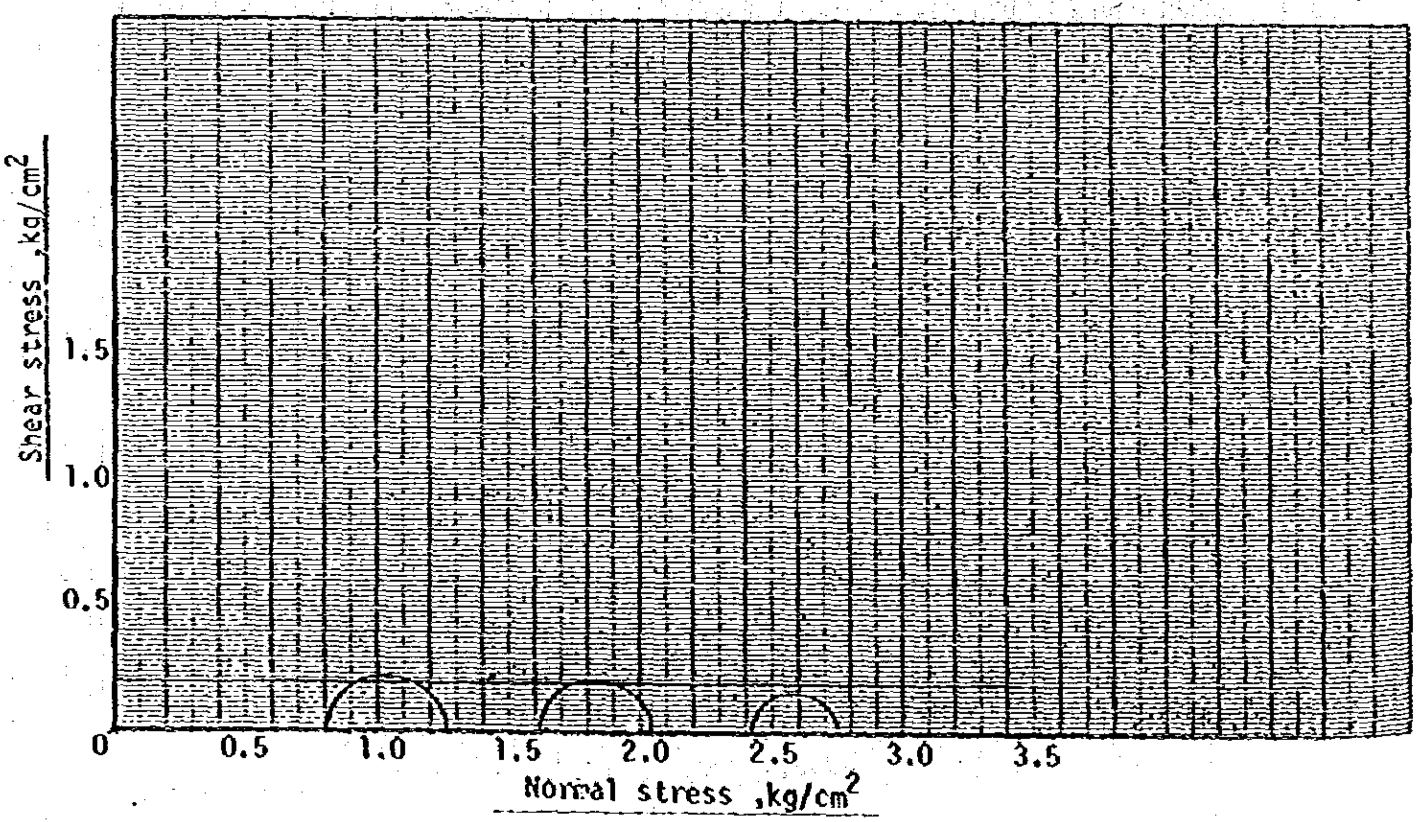
Project 267

Depth of Sample 9.00 m ~ 9.50 m

Angle of internal friction 0°

Condition of drainage U-U

Cohesion 0.20 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Project 267

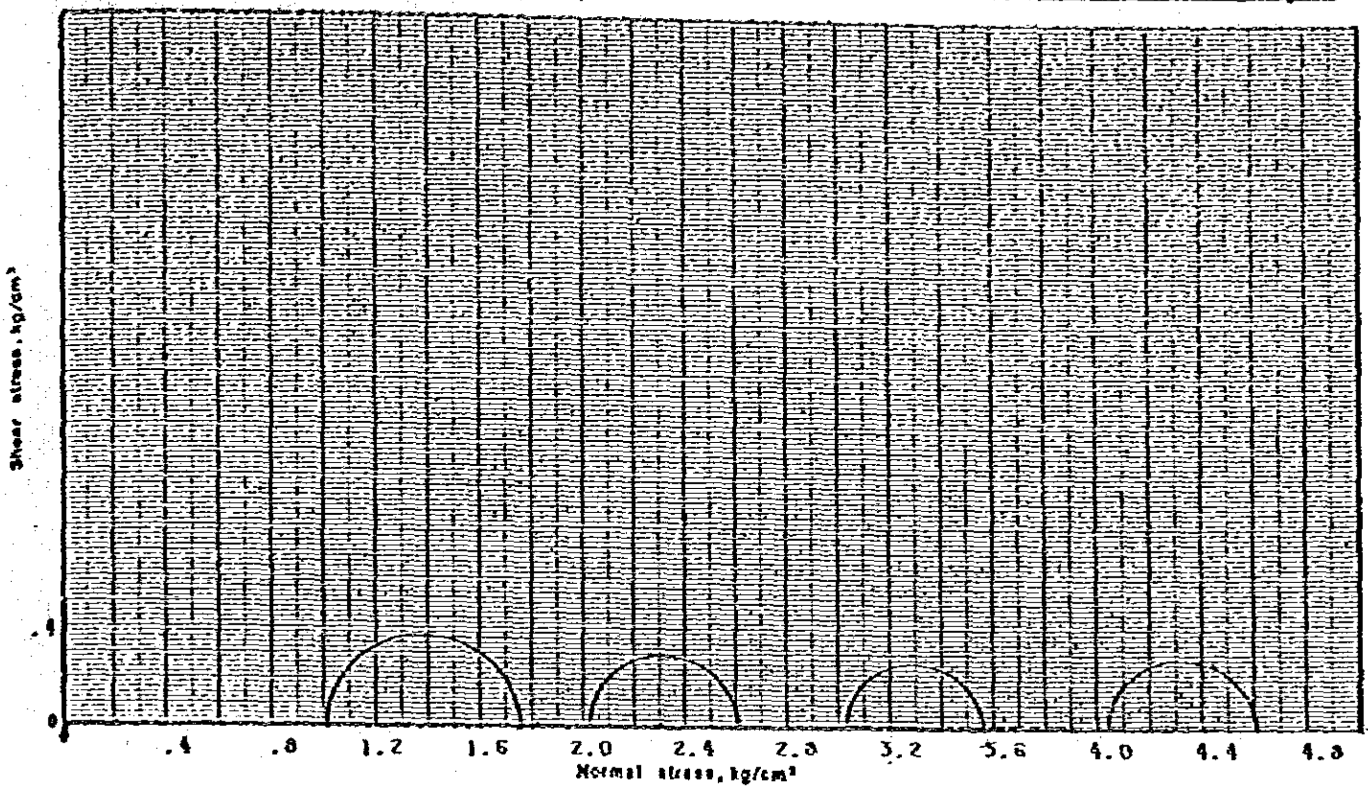
Condition of drainage U-U

Boring No. CNBH-1 Sample No. UD-4

Depth of Sample 11.50 m - 12.30 m

Angle of internal friction 0°

Cohesion 0.30 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Project 267

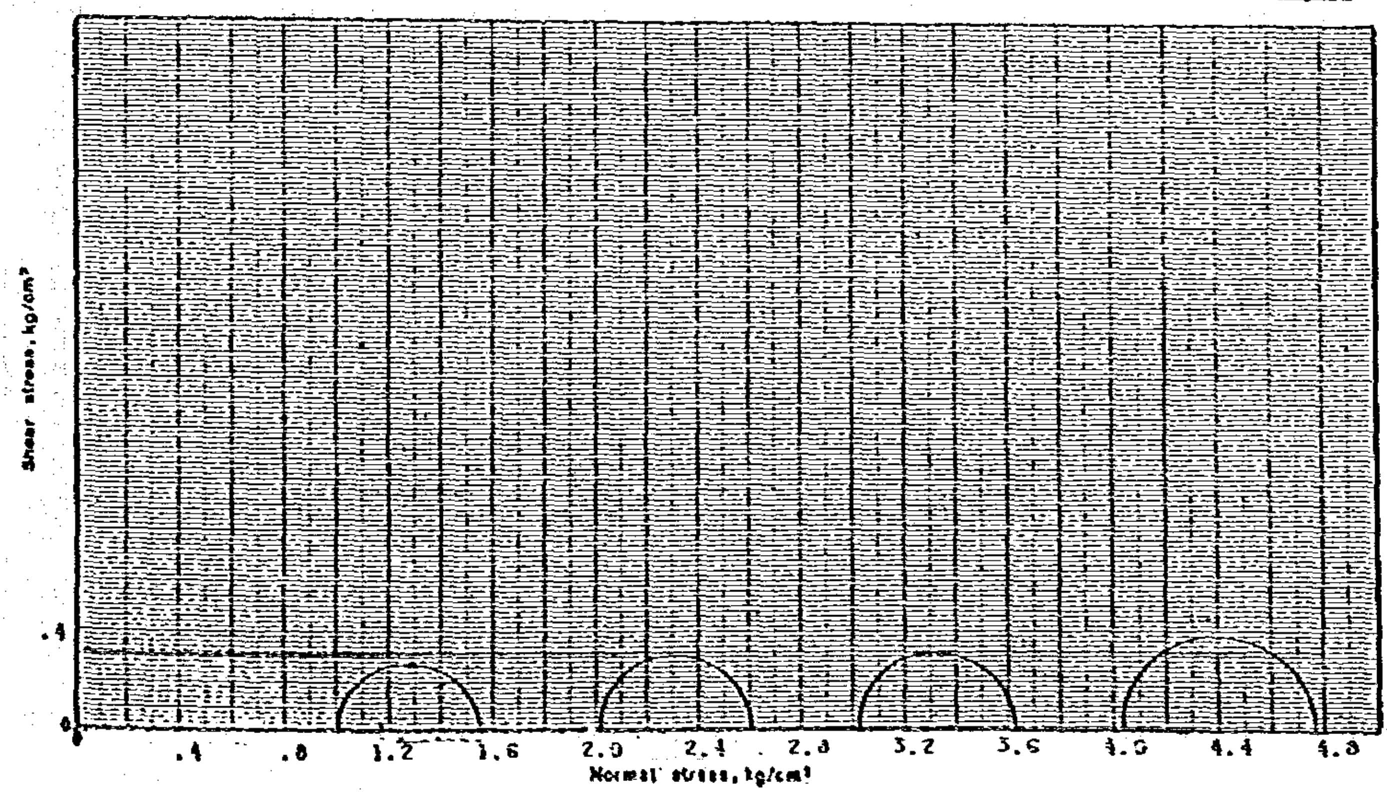
Condition of drainage U-U

Boring No. CNBH-1 Sample No. UD-5Bottom

Depth of Sample 13.35 m - 13.80 m

Angle of internal friction 0°

Cohesion 0.31 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Project: 257

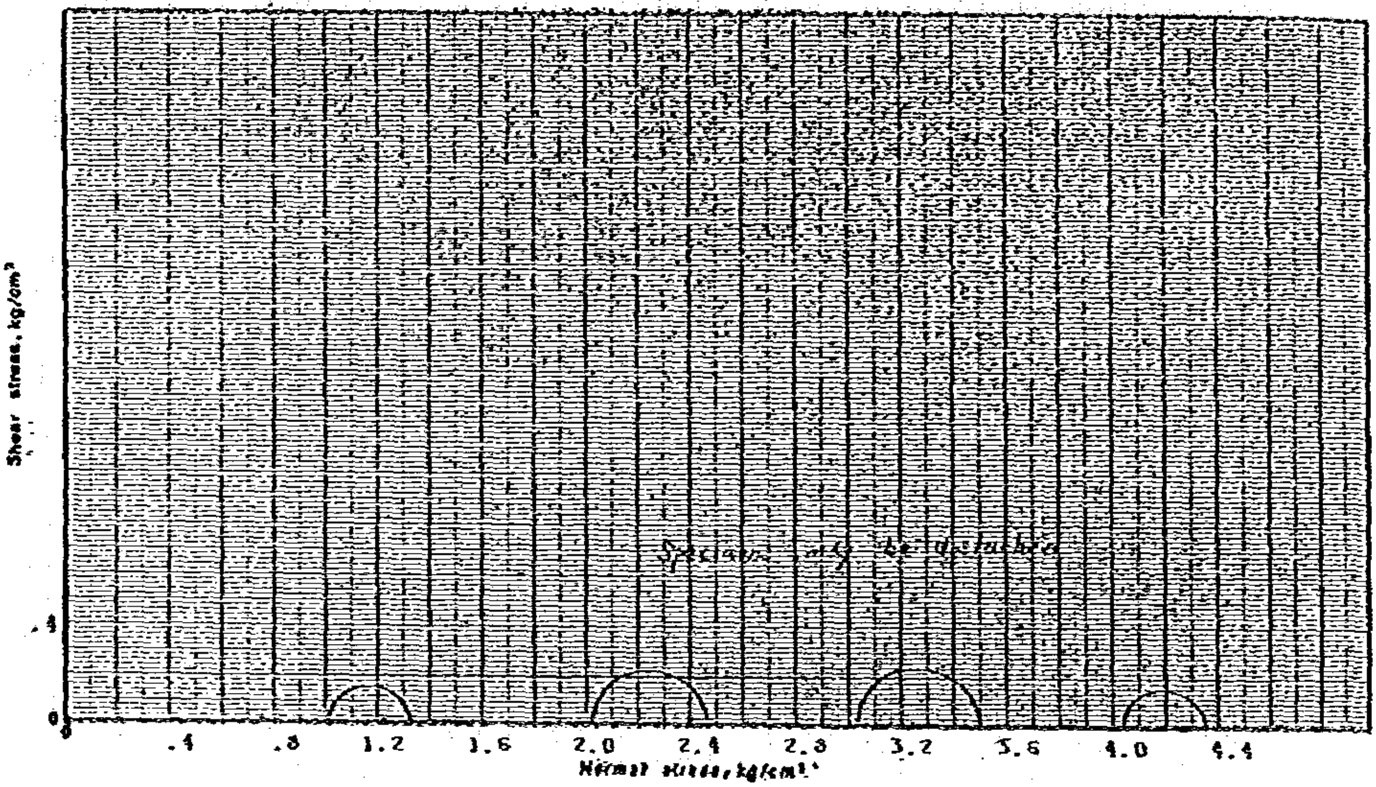
Condition of drainage: U-U

Boring No.: CNBH-1 Sample No.: UD-6

Depth of Sample: 16.00 m - 16.80 m

Angle of internal friction: 0°

Cohesion: (0.19) kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Project: 257

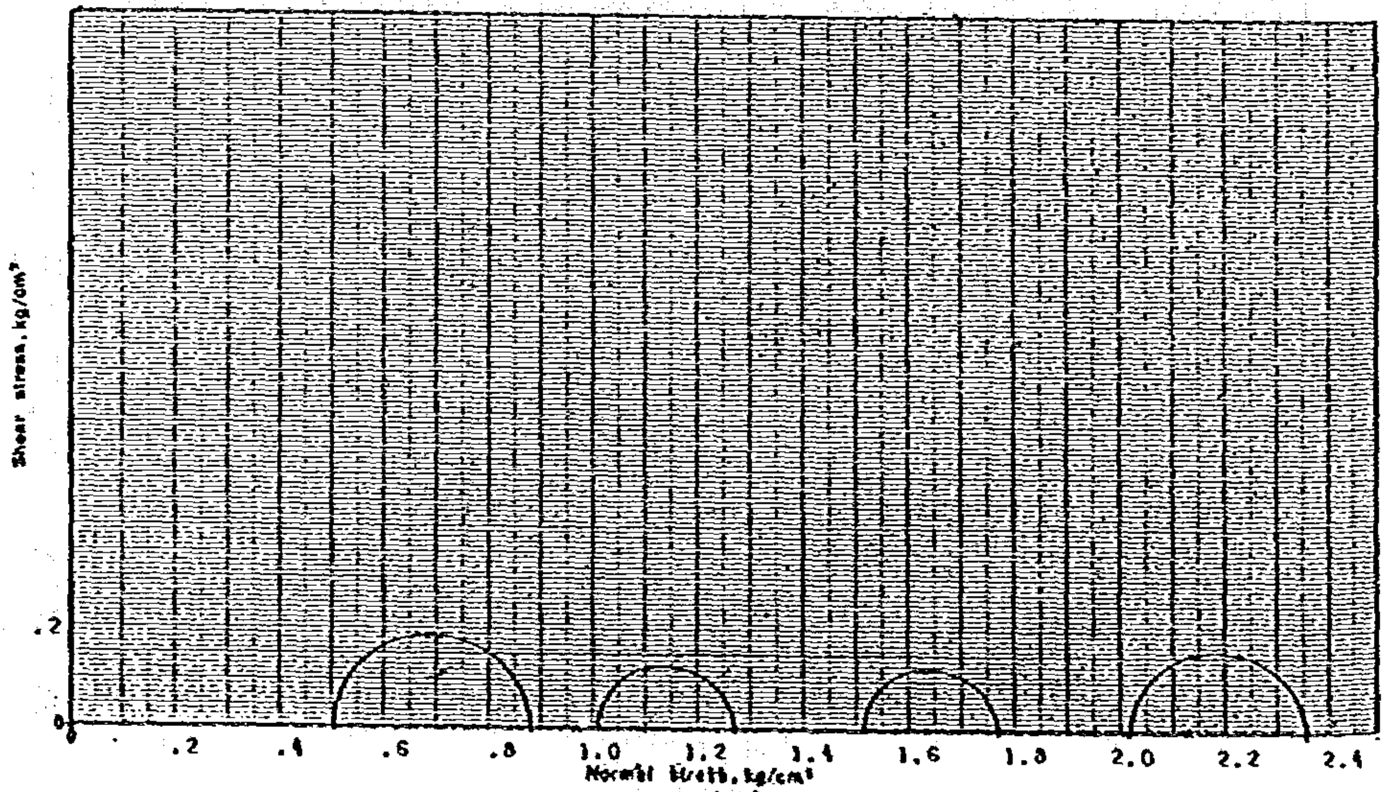
Condition of drainage: U-U

Boring No.: CNBH-2 Sample No.: UD-1 Top

Depth of Sample: 6.00 m - 6.55 m

Angle of internal friction: 0°

Cohesion: 0.15 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Project 267

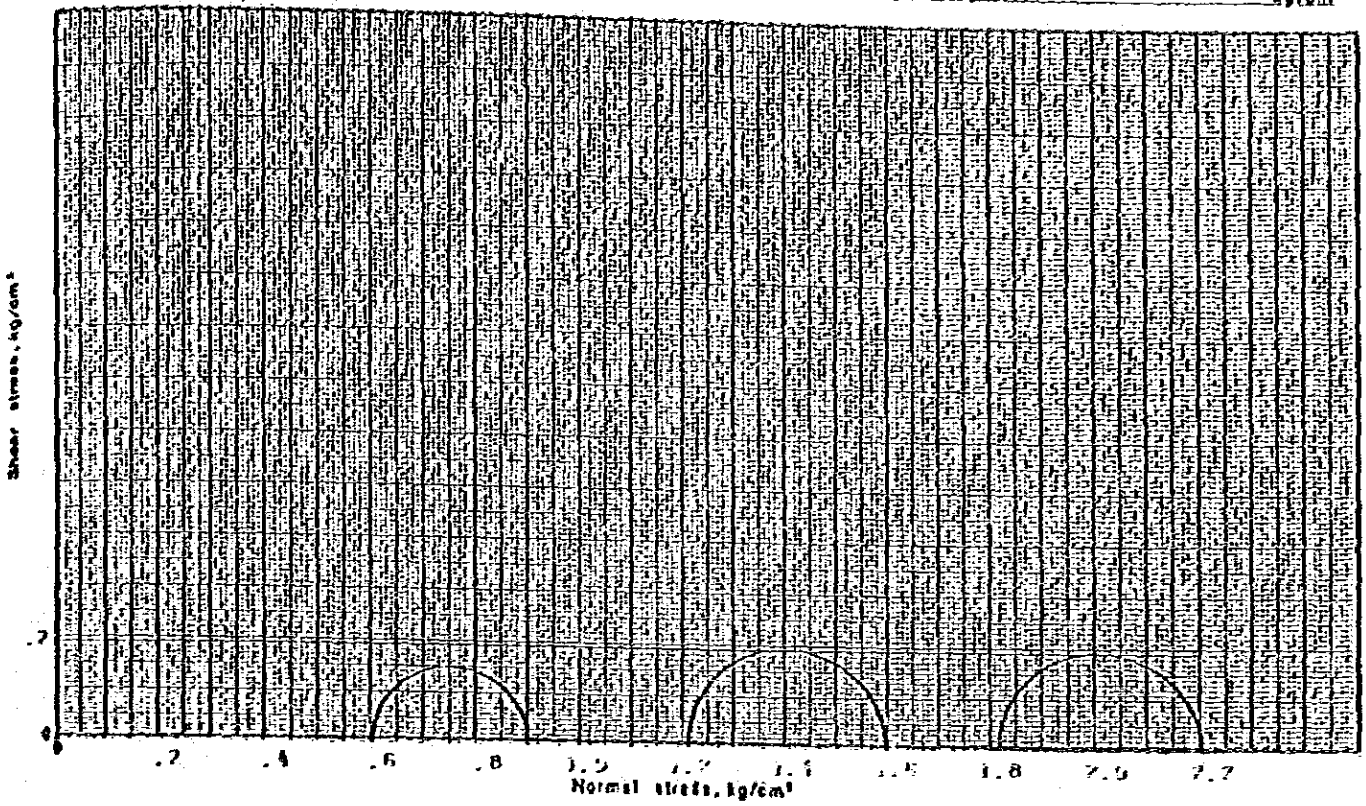
Condition of drainage U-U

Boring No CHBH-2 Sample No UD-2Top

Depth of Sample 9.30 m - 9.60 m

Angle of internal friction 0°

Cohesion 0.18 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Project 267

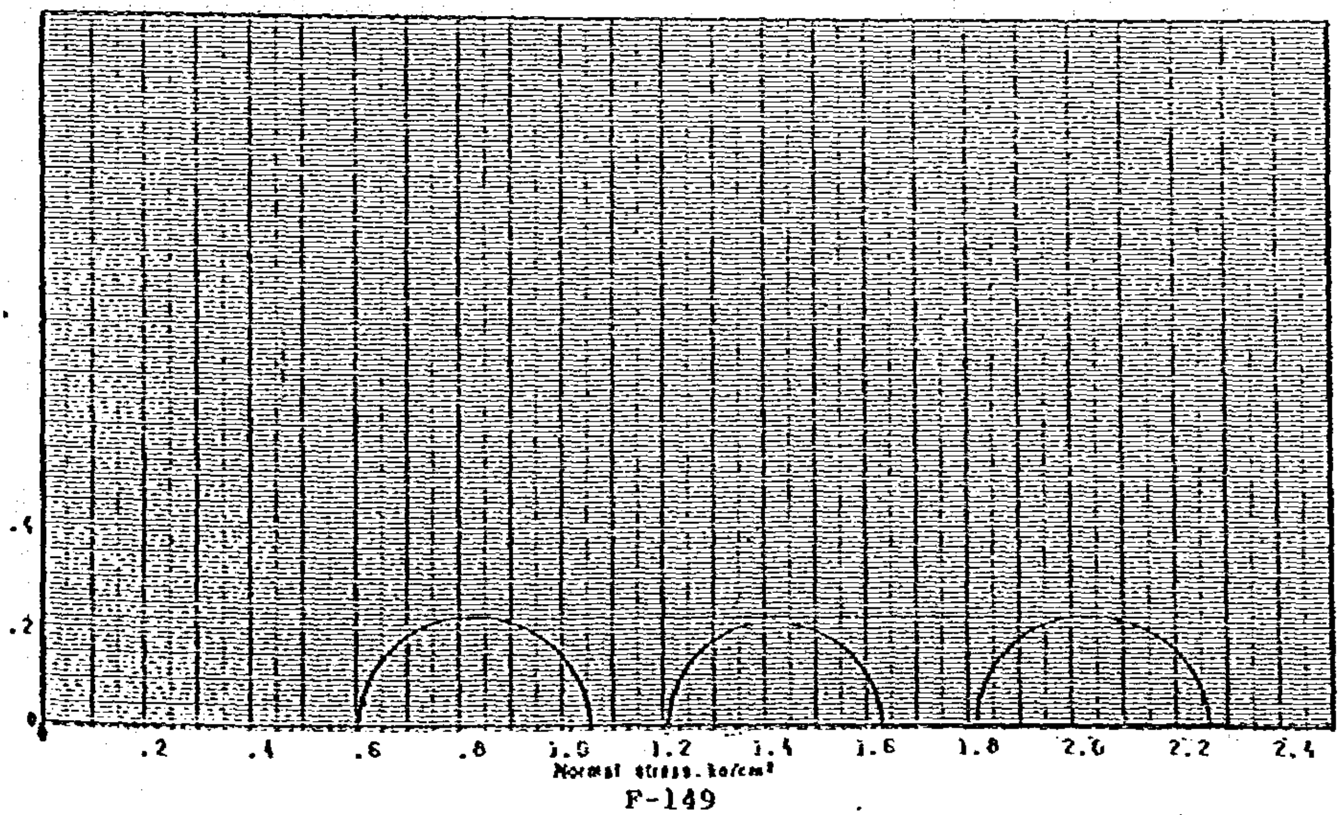
Condition of drainage U-U

Boring No CHBH-2 Sample No UD-2Bottom

Depth of Sample 9.60 m - 9.80 m

Angle of internal friction 0°

Cohesion 0.22 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Boring No. CNBH-3 Sample No. UD-3

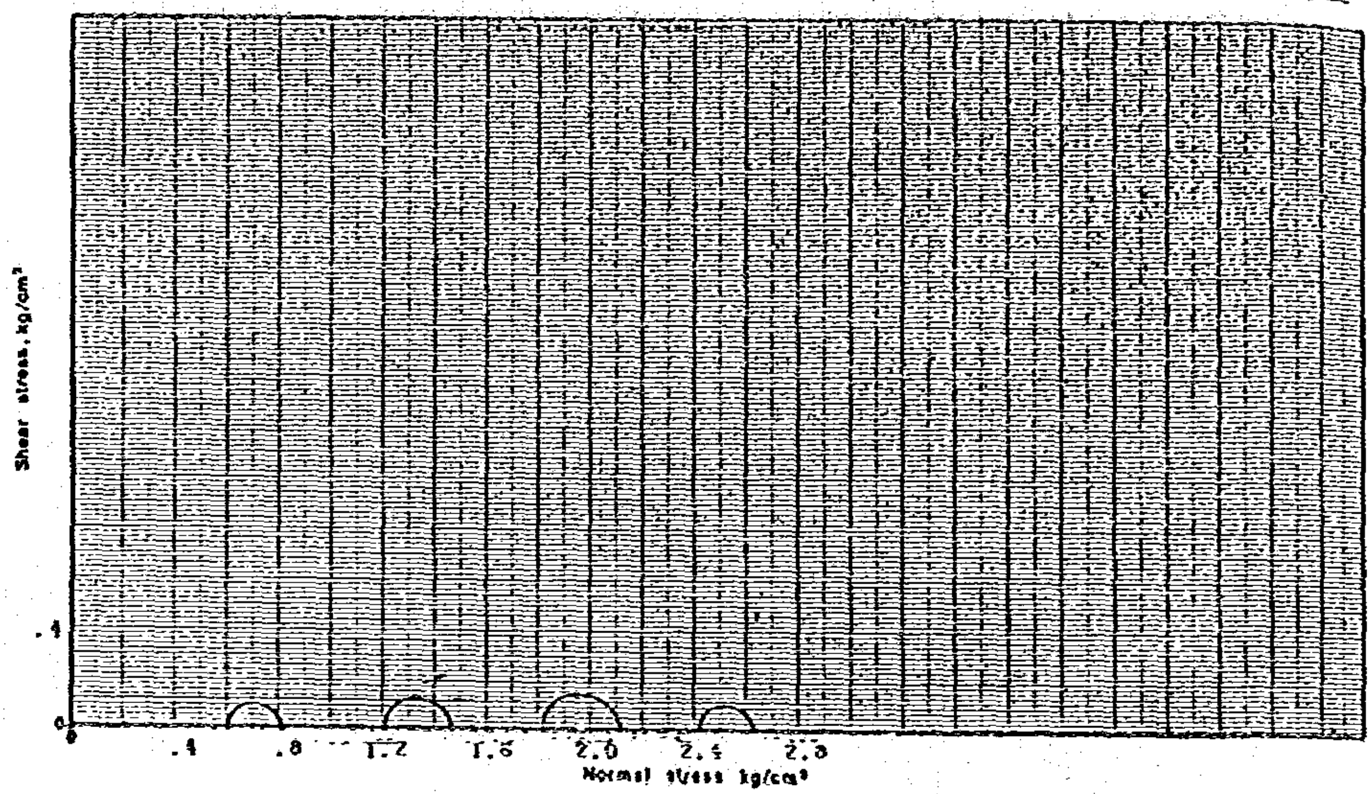
Depth of Sample 9.00 m - 9.50 m

Project 267

Angle of internal friction 0°

Condition of drainage U-U

Cohesion 0.12 kg/cm²



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Boring No. CNBH-3 Sample No. UD-2 Top

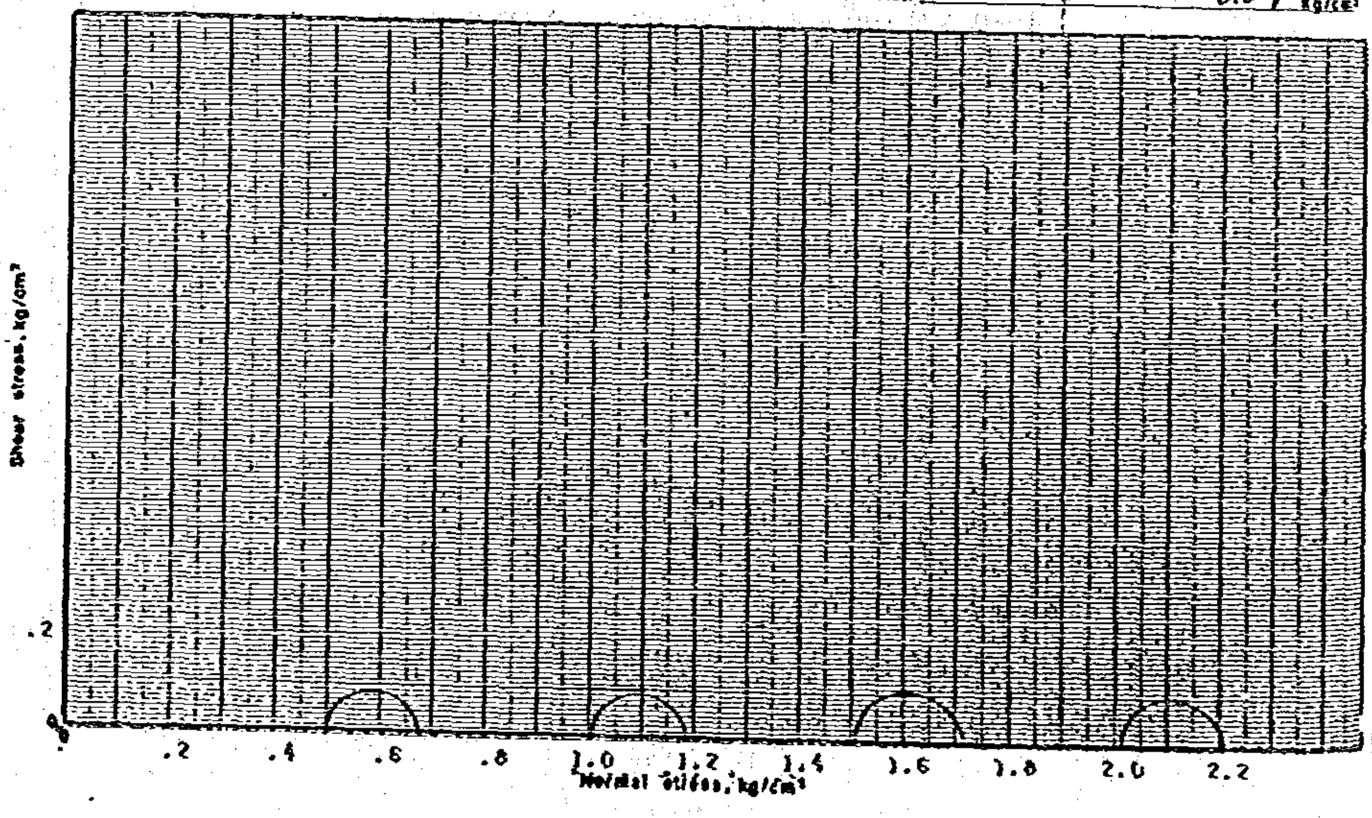
Depth of Sample 6.00 m - 6.65 m

Project 267

Angle of internal friction 0°

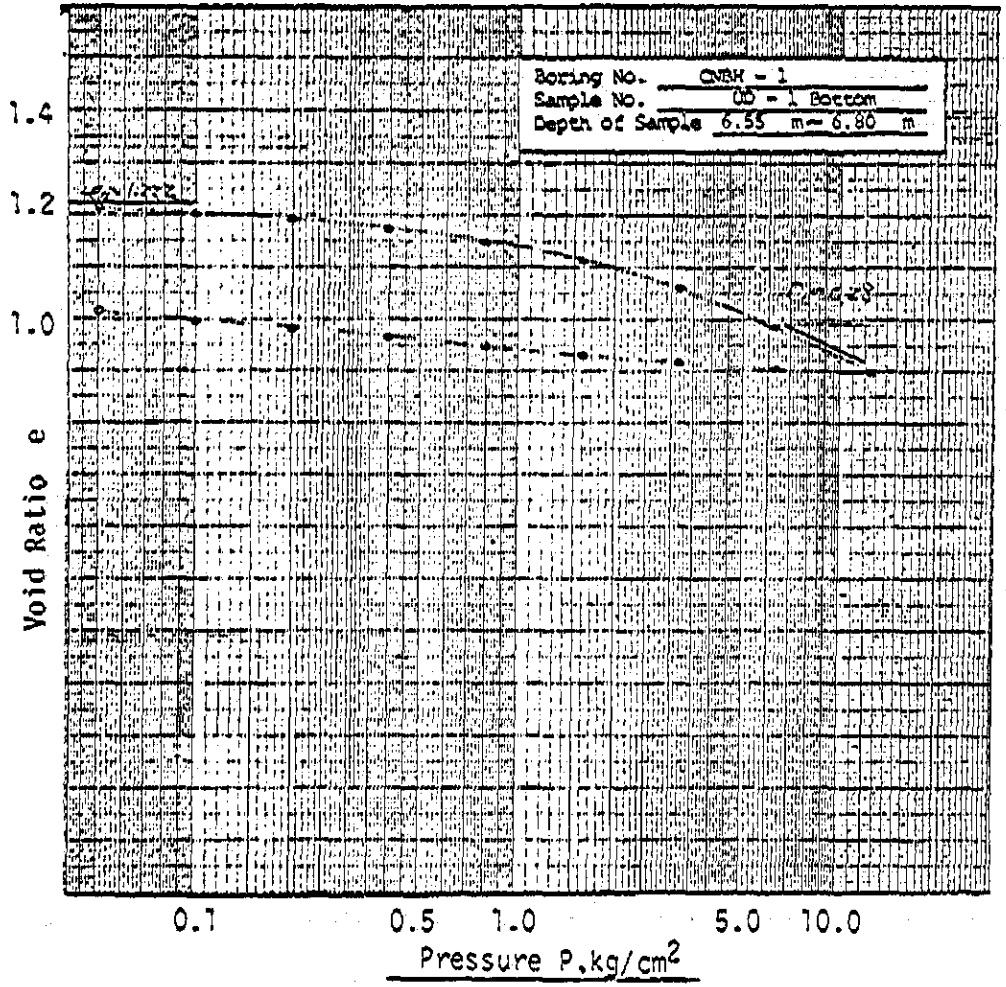
Condition of drainage U-U

Cohesion 0.09 kg/cm²



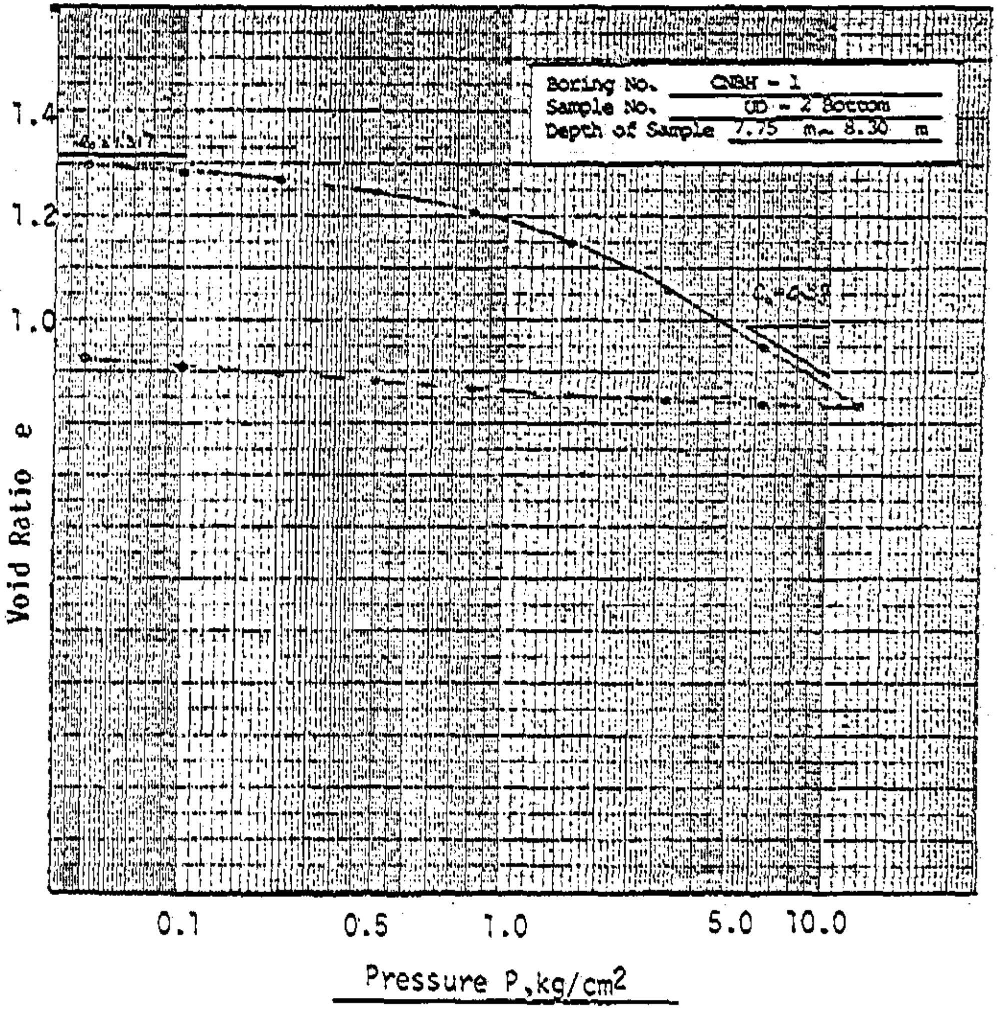
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_0	Preconsolidation Pressure σ'_p (kg/cm²)	Compression Index Cc	Symbol
UD-1	[illegible]	[illegible]	1.222	—	0.28	⊙
[illegible]						△



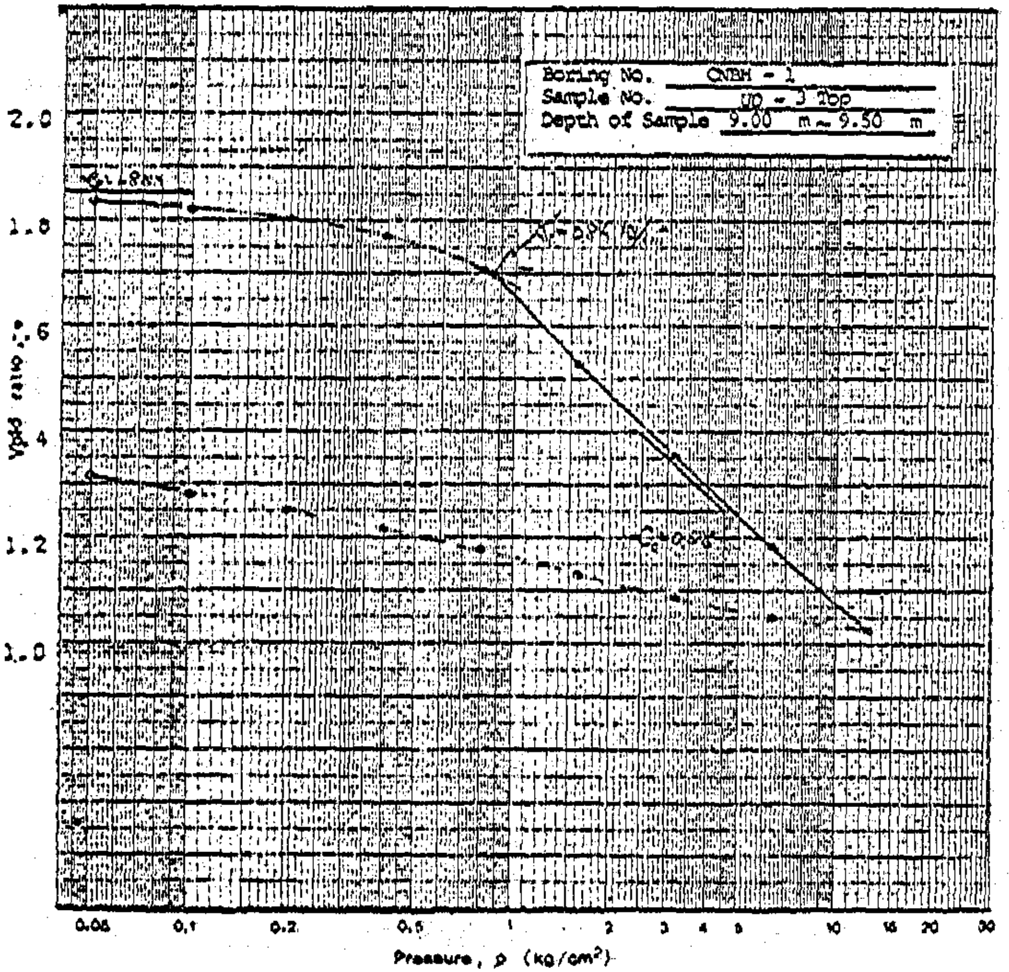
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_0	Preconsolidation Pressure σ'_p (kg/cm²)	Compression Index Cc	Symbol
UD-2B	7.75 ~ 8.30	51.0	1.317	—	0.38	⊙
						△



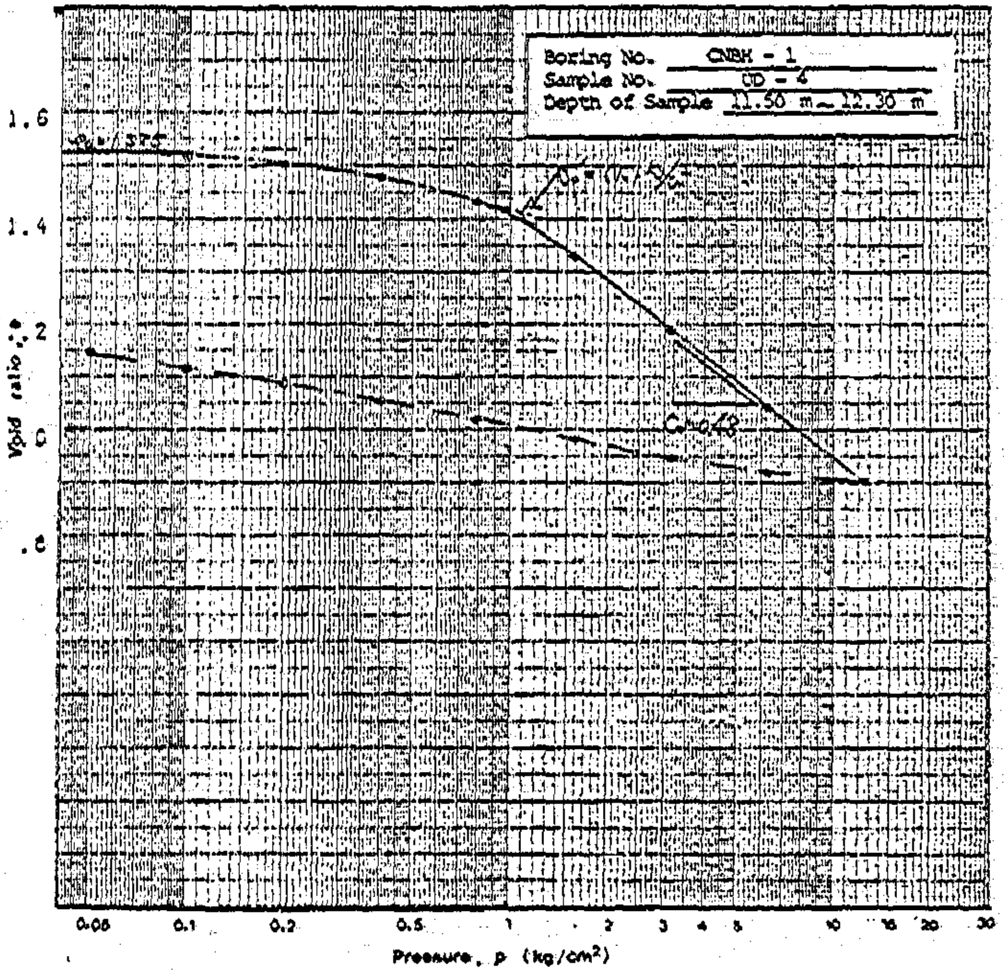
CONSOLIDATION TEST (e-log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ_p (kg/cm²)	Compression Index Cc	Symbol
UD-3	9.00~9.50	55.0	1.864	0.86	0.58	⊙
T-1						△



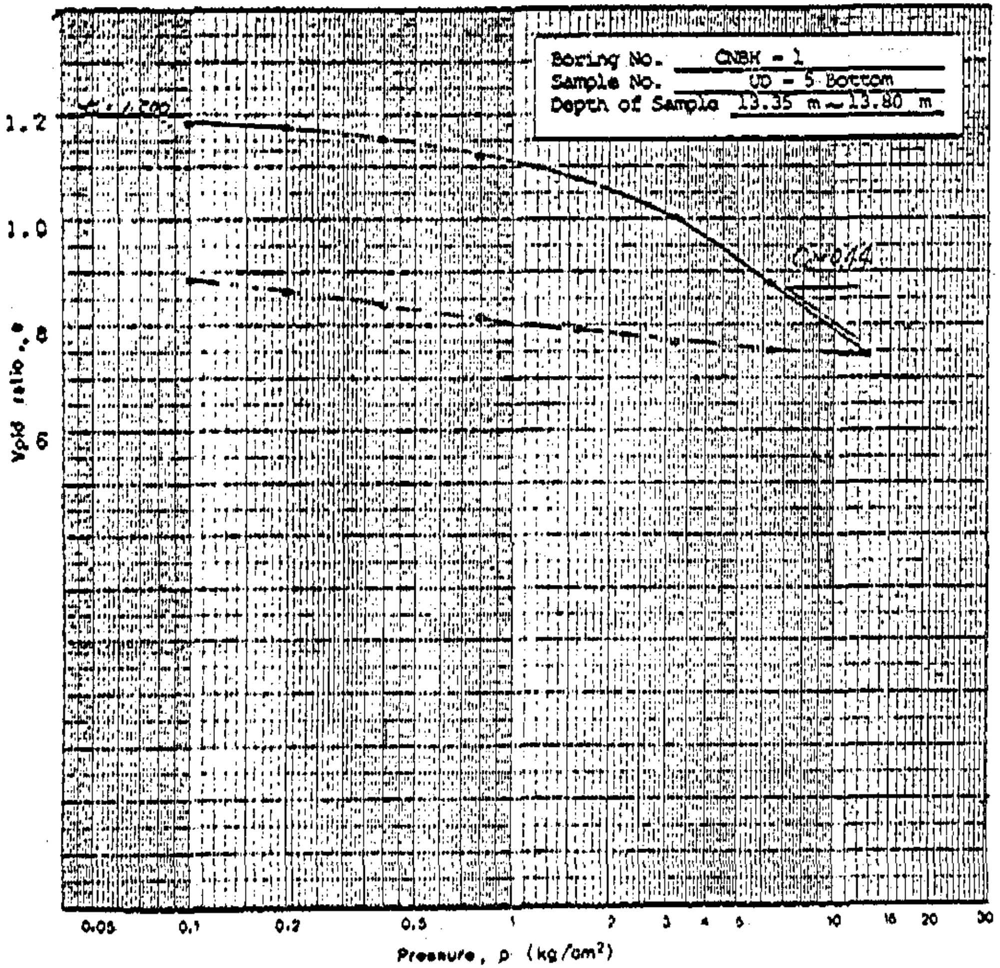
CONSOLIDATION TEST (e-log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ_p (kg/cm²)	Compression Index Cc	Symbol
UD-4	11.50~12.30	51.6	1.525	(1.1)	0.48	⊙
						△



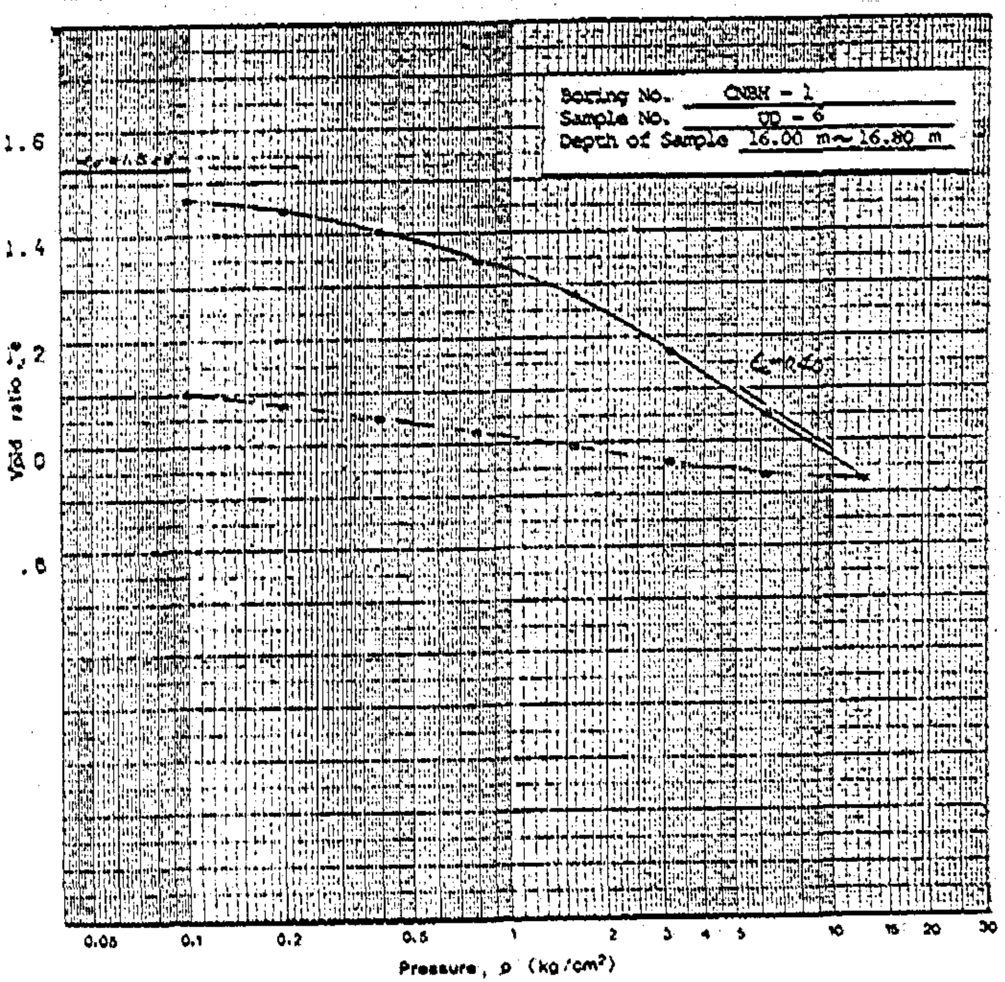
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm^2)	Compression Index Cc	Symbol
UD-5	13.35 ~ 13.80	34.6	1.200	--	0.44	⊙
bottom						△



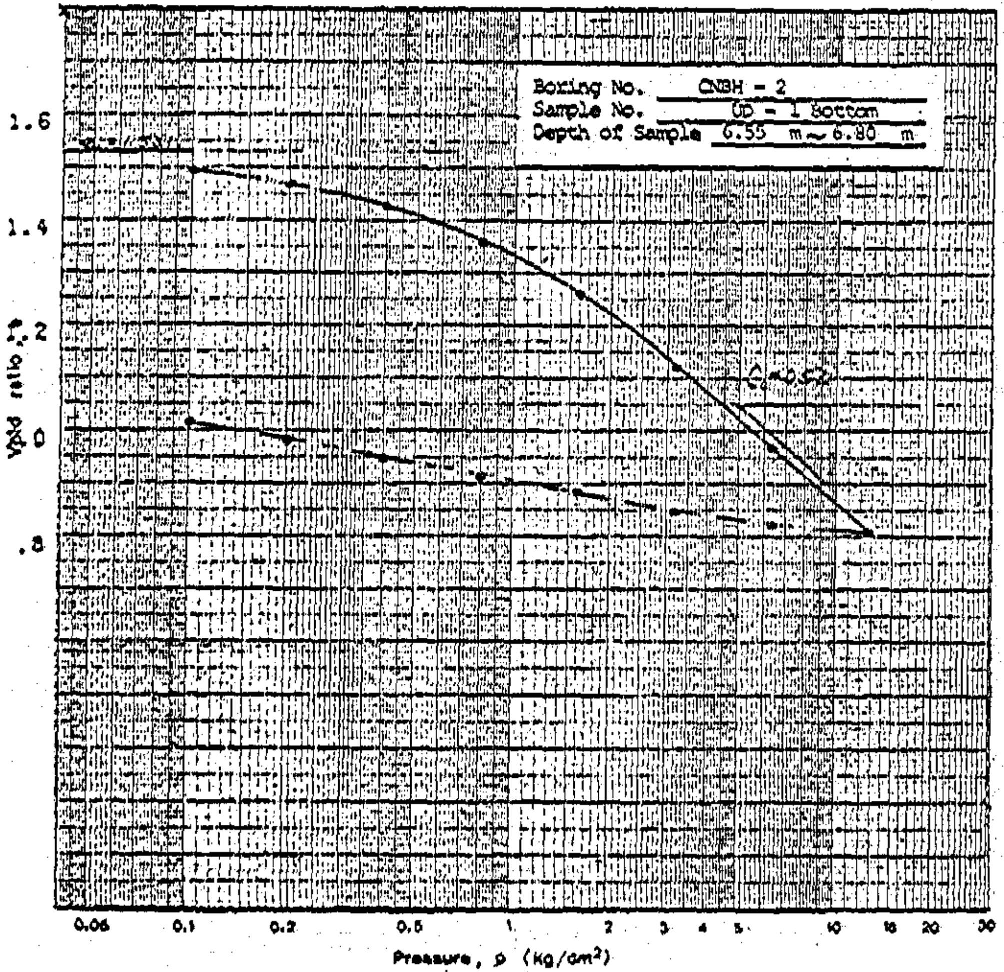
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm^2)	Compression Index Cc	Symbol
UD-6	16.00 ~ 16.80	58.8	1.528	—	0.40	⊙
						△



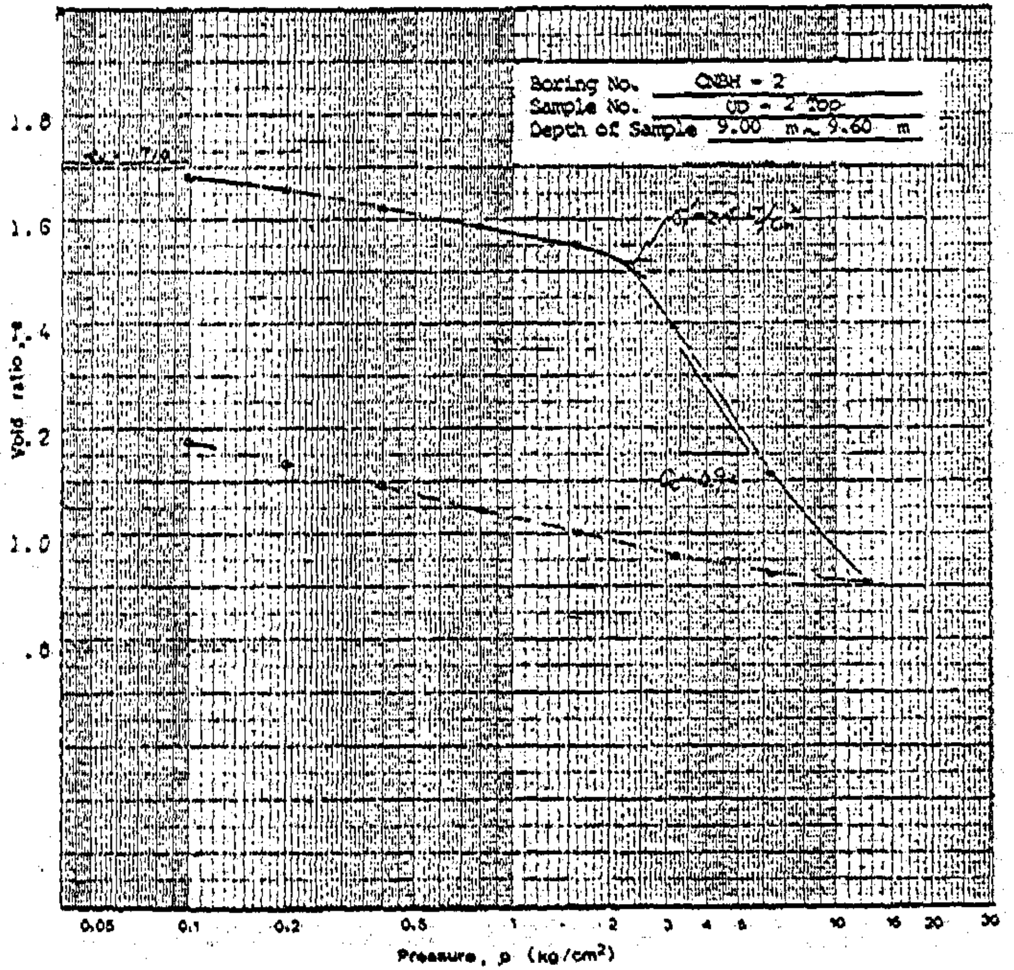
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm²)	Compression Index Cc	Symbol
UD-1	6.55 - 6.80	58.8	1.531	—	0.52	⊙
Bottom						△



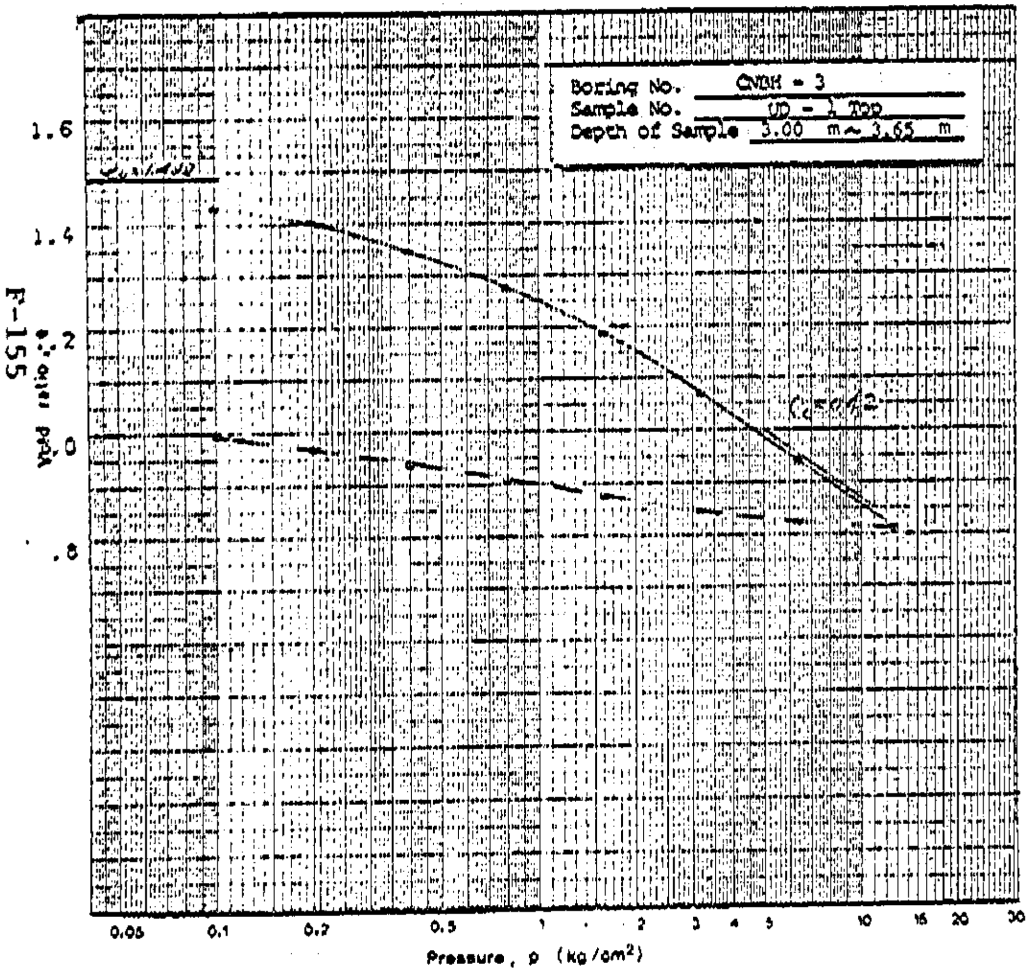
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm²)	Compression Index Cc	Symbol
UD-2	9.00 - 9.60	57.0	1.70	[illegible]	[illegible]	⊙
Top						△



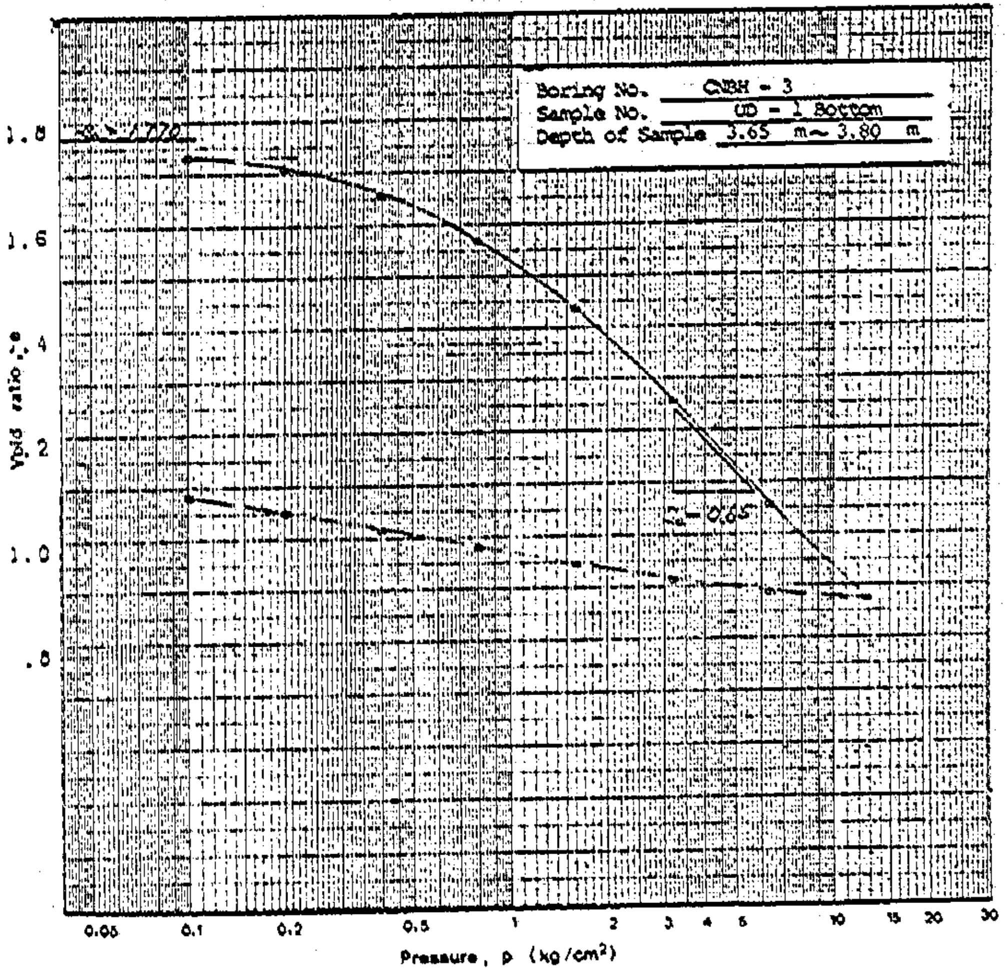
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ_p (kg/cm²)	Compression Index Cc	Symbol
UD-1	3.00 ~ 3.65	—	1.488	—	0.42	⊙
Top						△



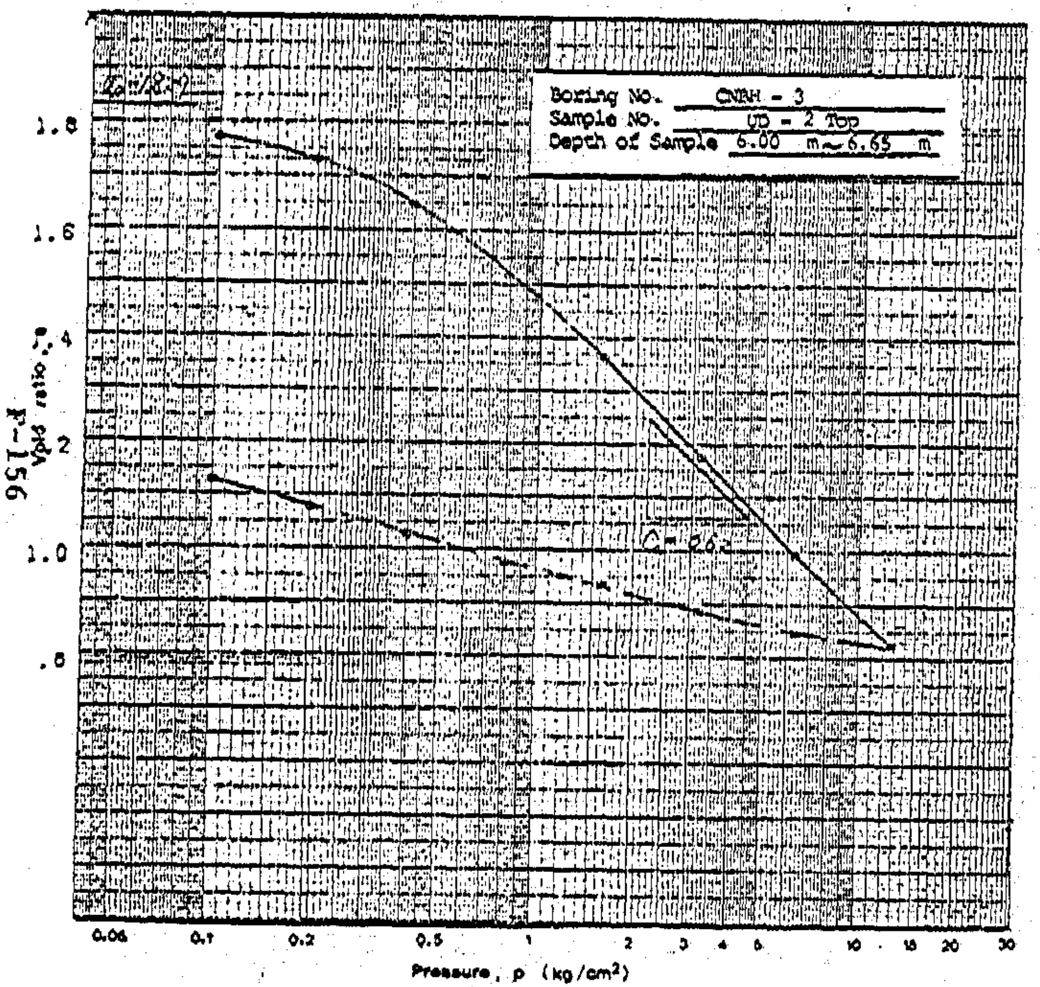
CONSOLIDATION TEST (e - log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ_p (kg/cm²)	Compression Index Cc	Symbol
UD-1	3.65 ~ 3.80	54.2	1.770	—	0.65	⊙
Bottom						△



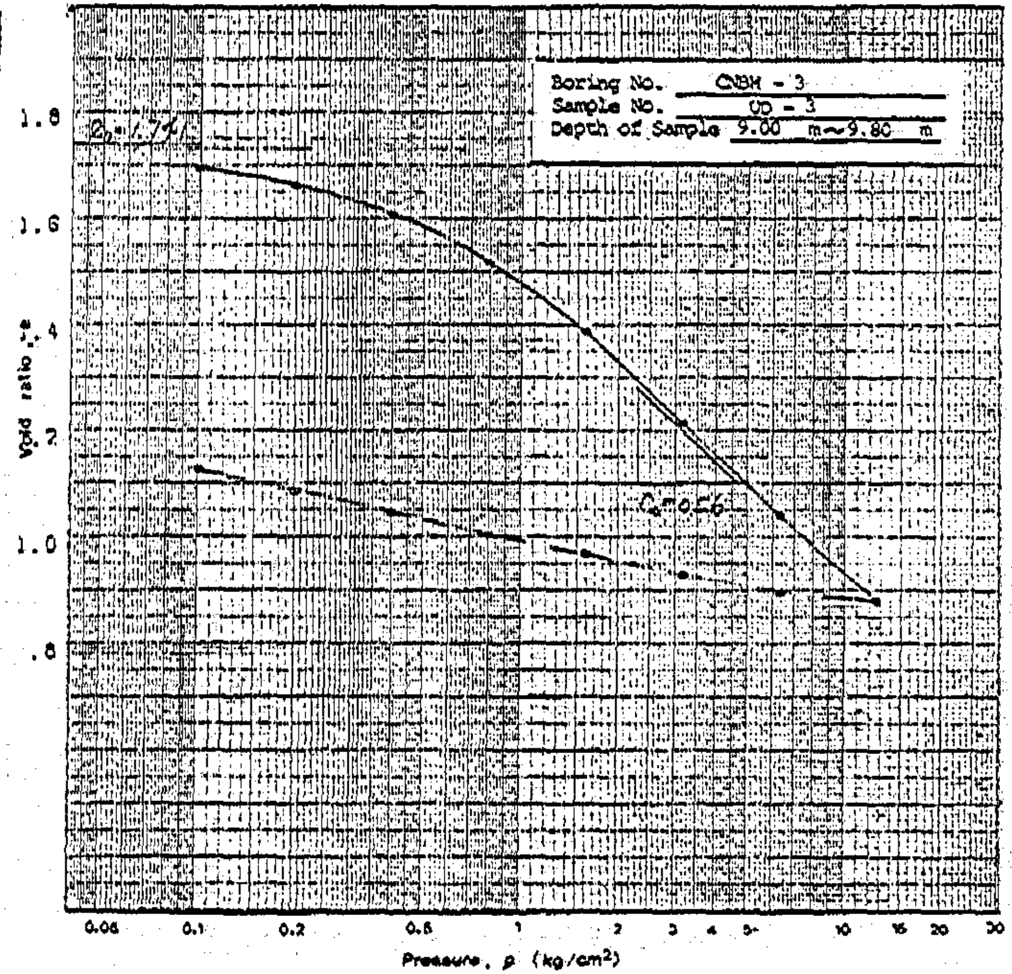
CONSOLIDATION TEST (e-log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm2)	Compression Index Cc	Symbol
UD-2	6.00~6.65	65.0	1.829	—	0.62	⊙
Top						△

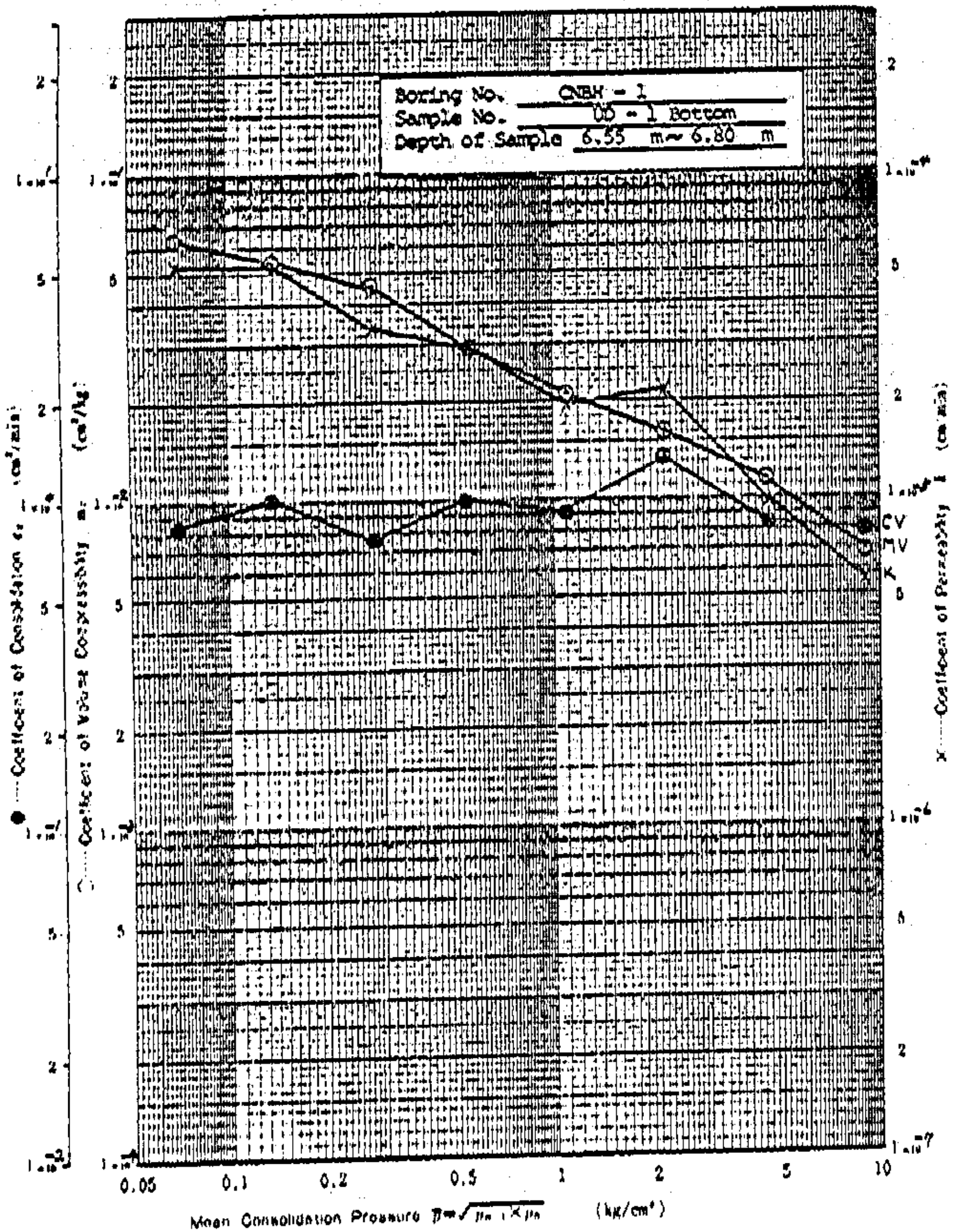


CONSOLIDATION TEST (e-log p curves)

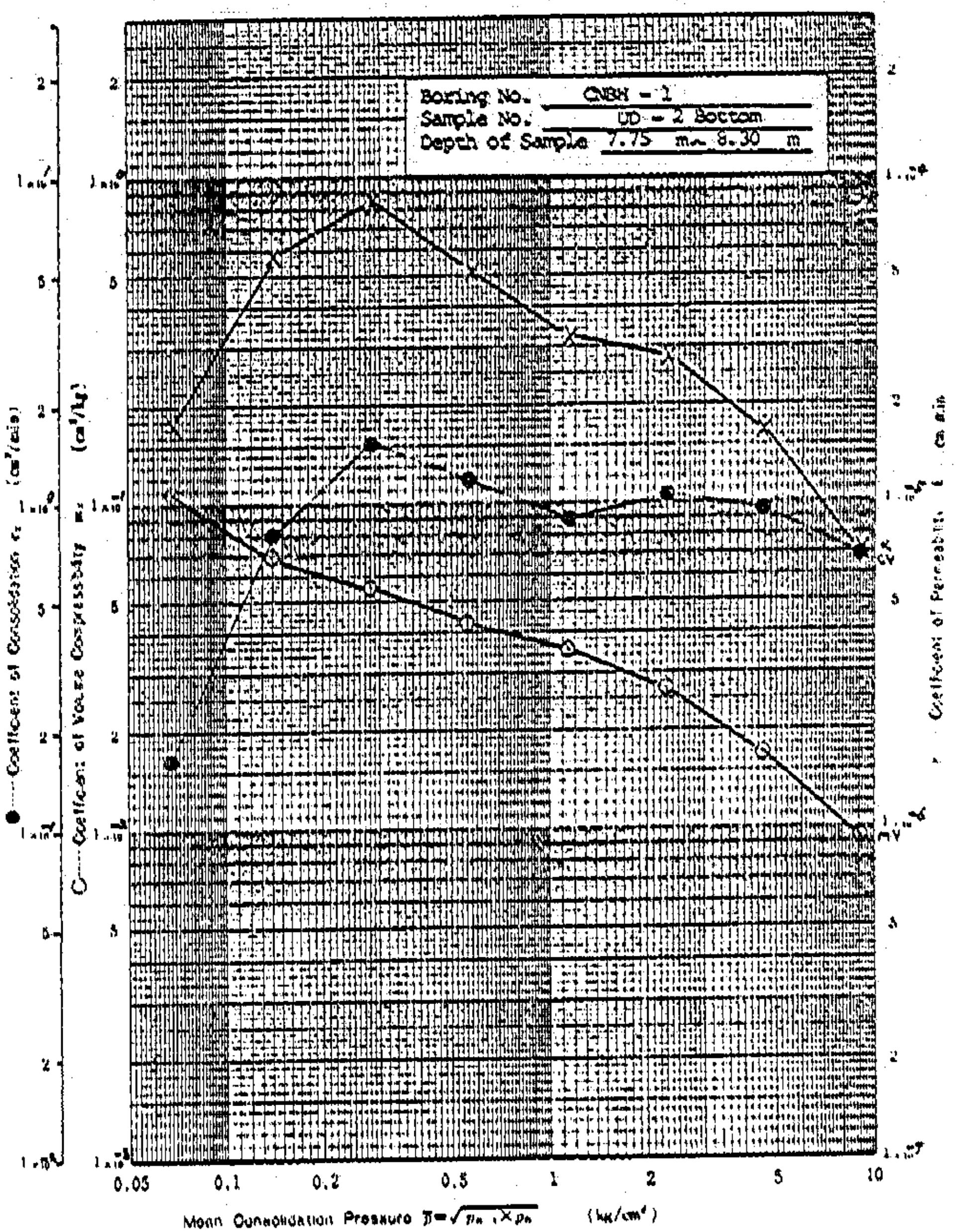
Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm2)	Compression Index Cc	Symbol
UD-3	9.00 ~ 9.80	57.9	1.741	—	0.56	⊙
						△



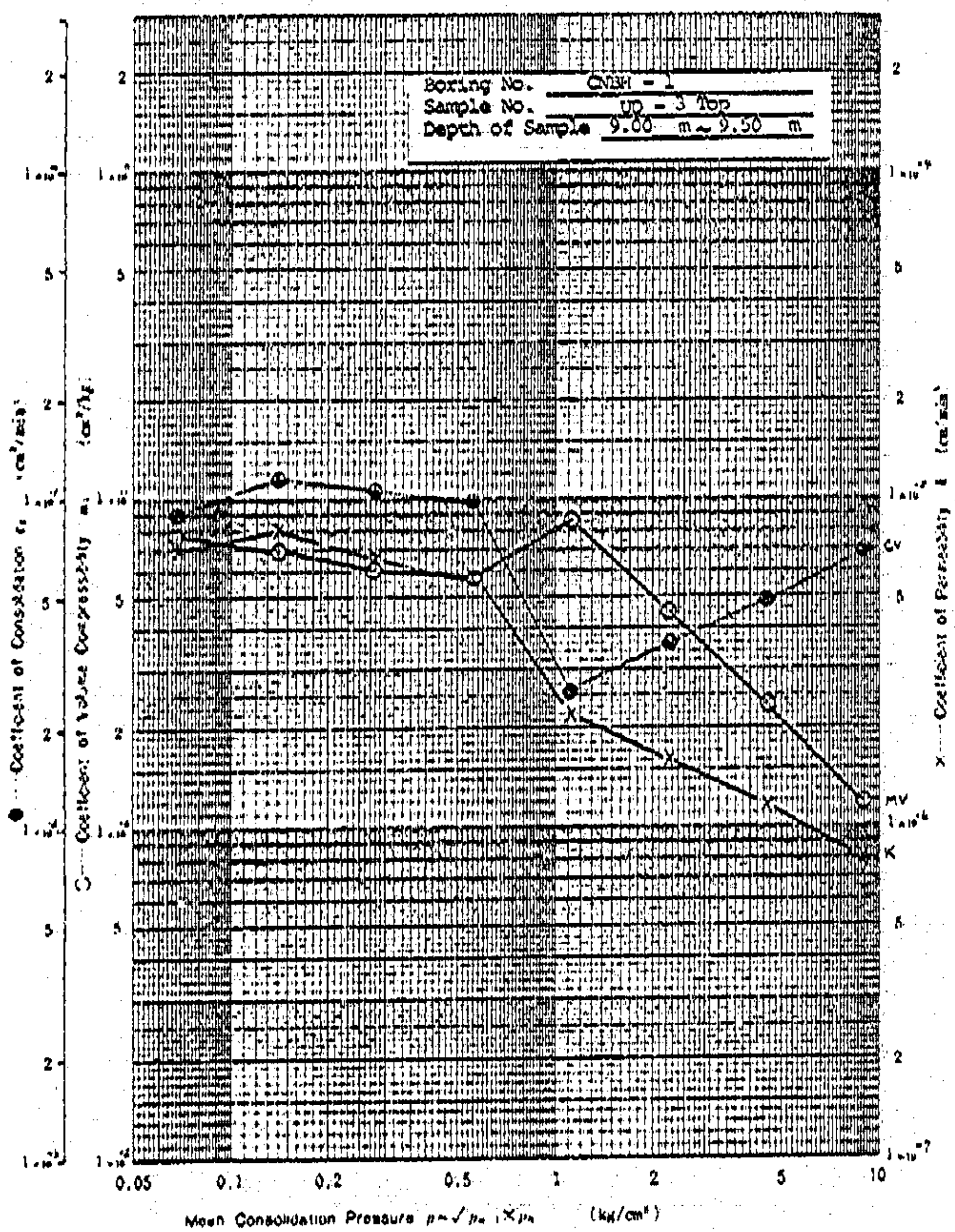
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



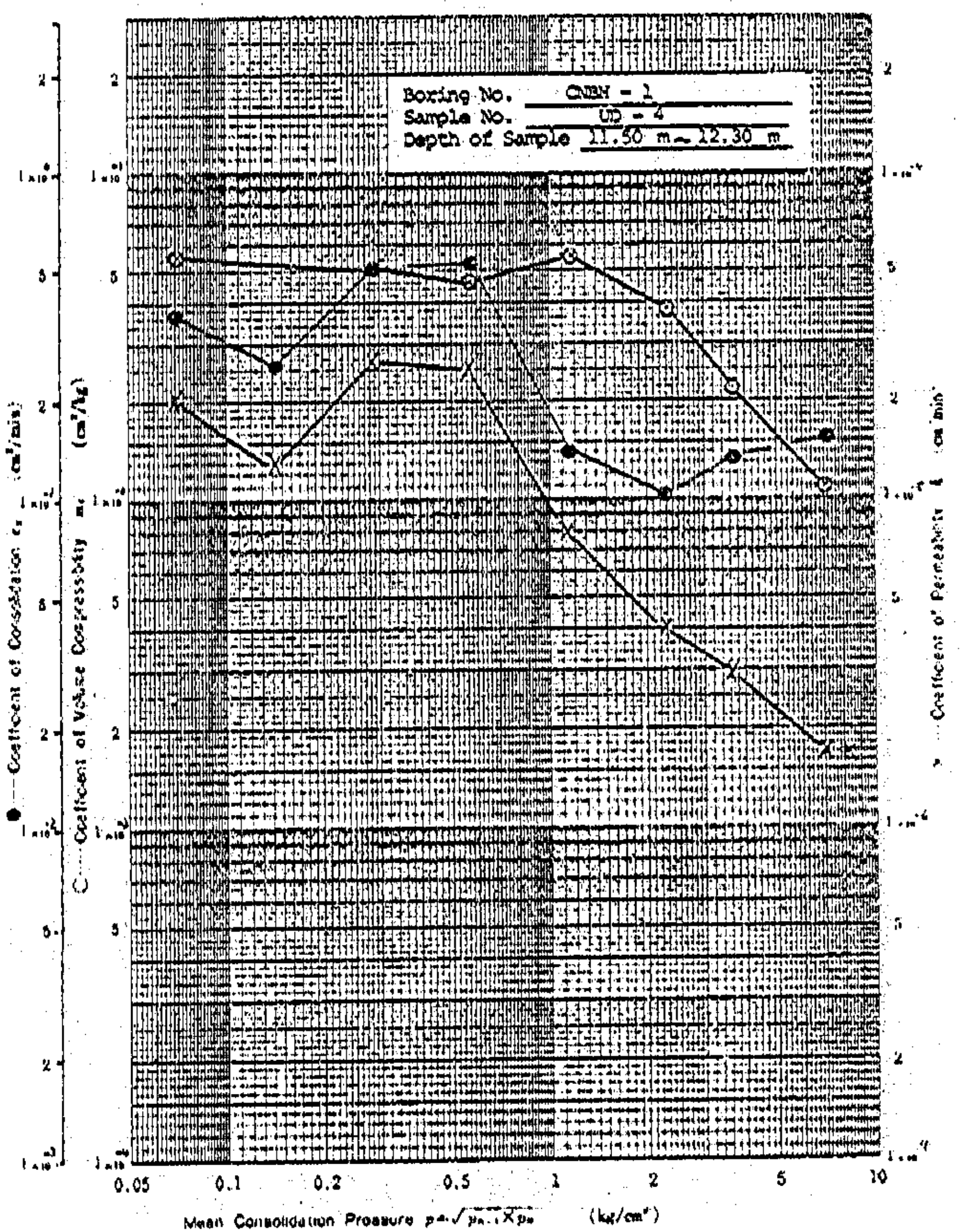
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



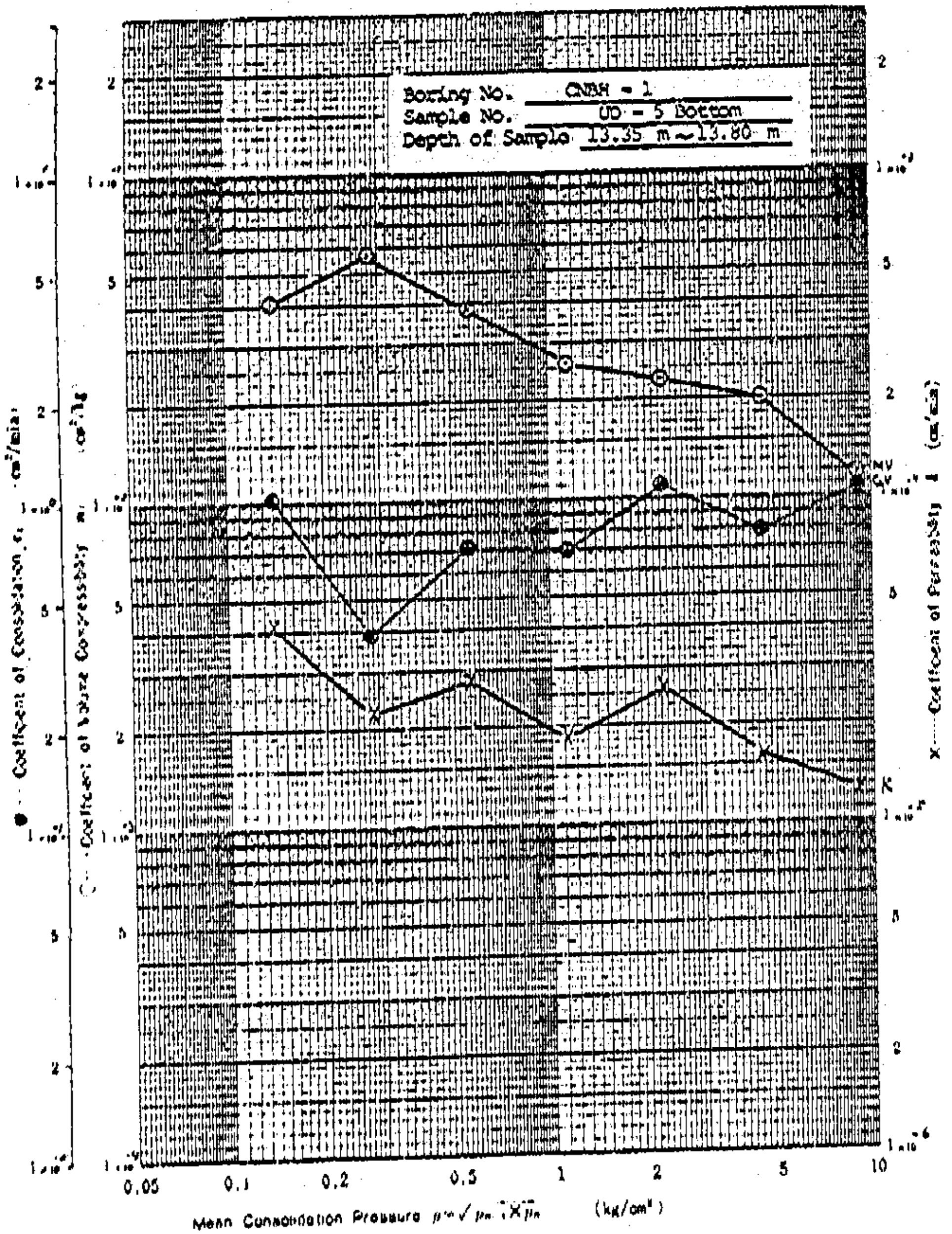
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



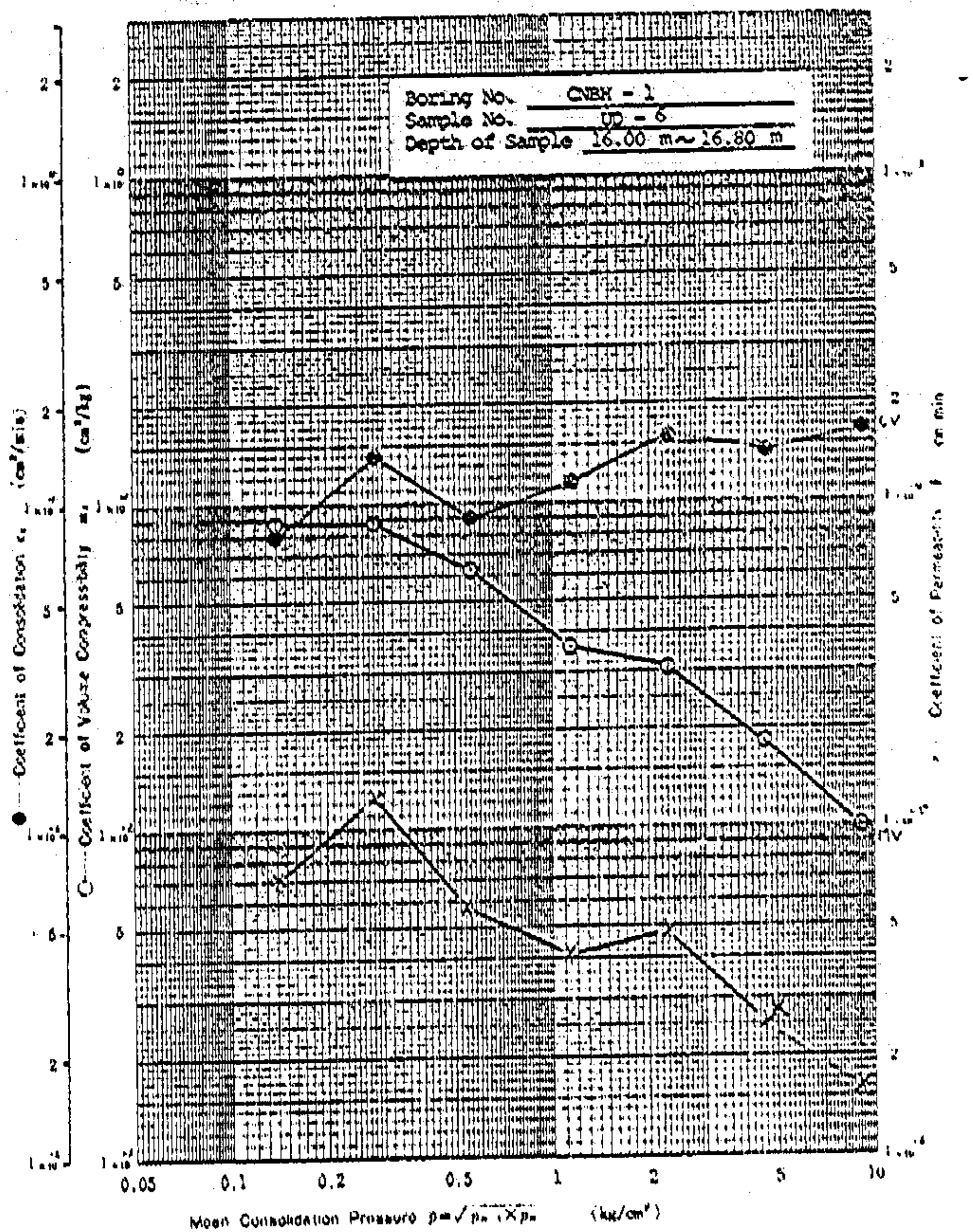
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



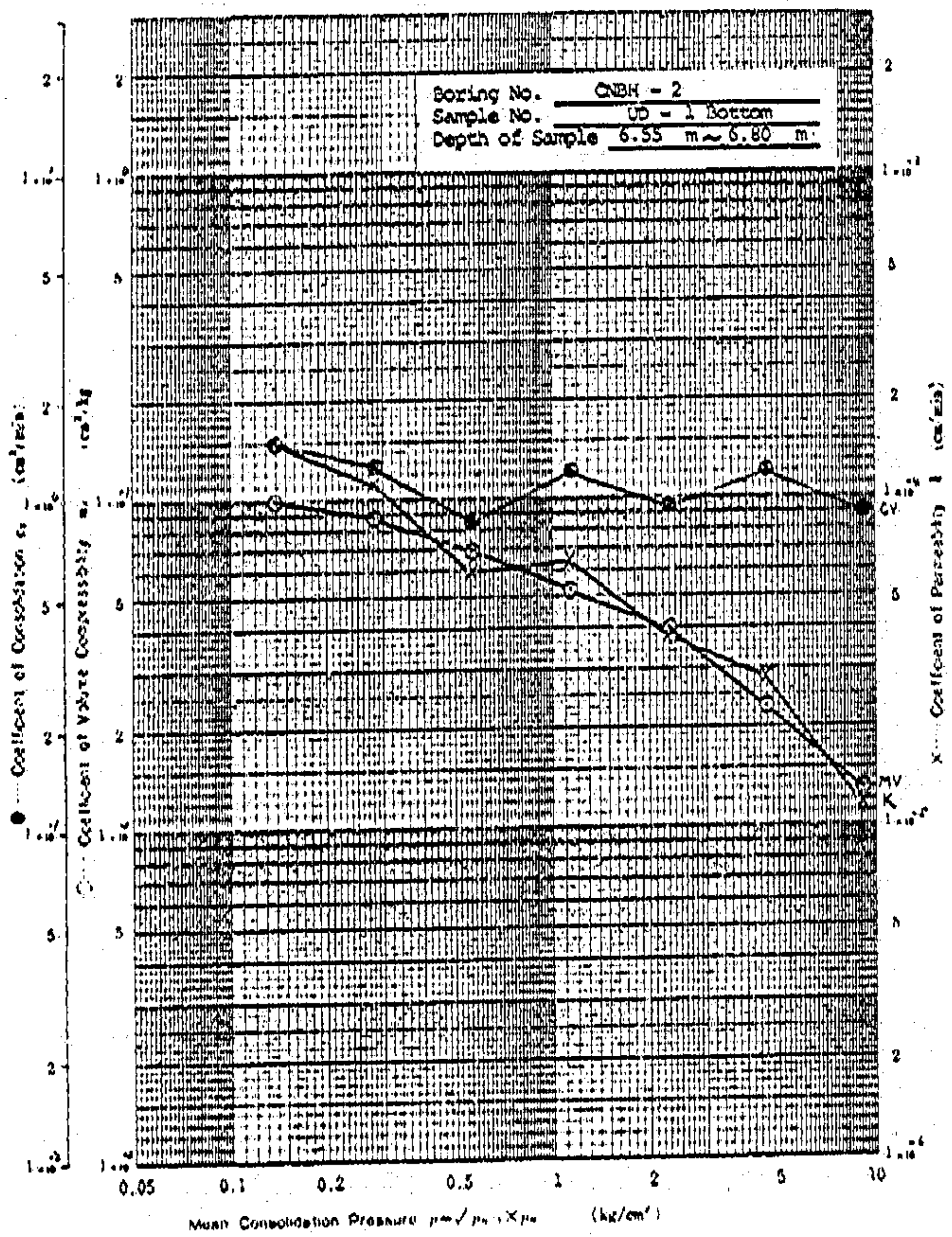
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



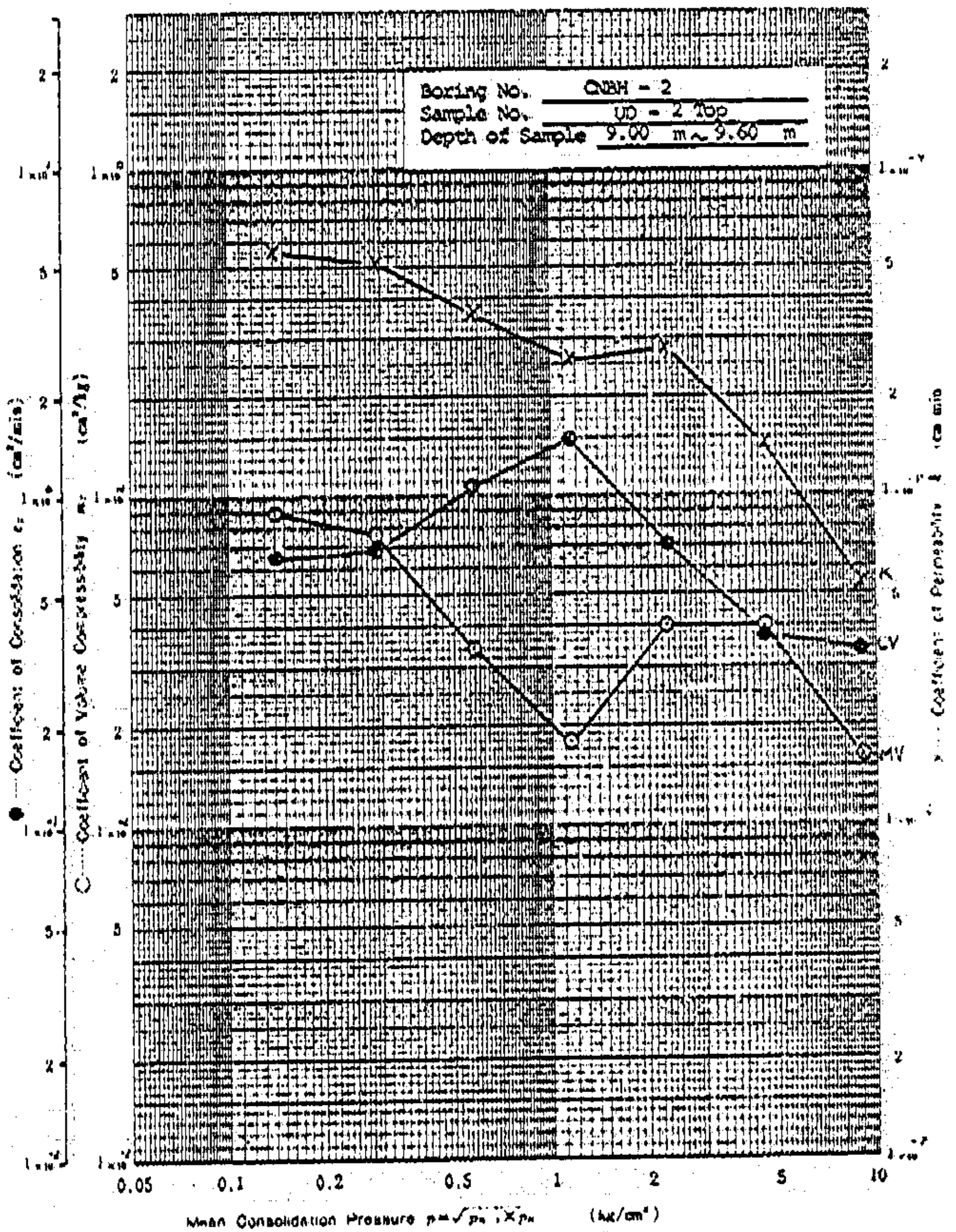
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



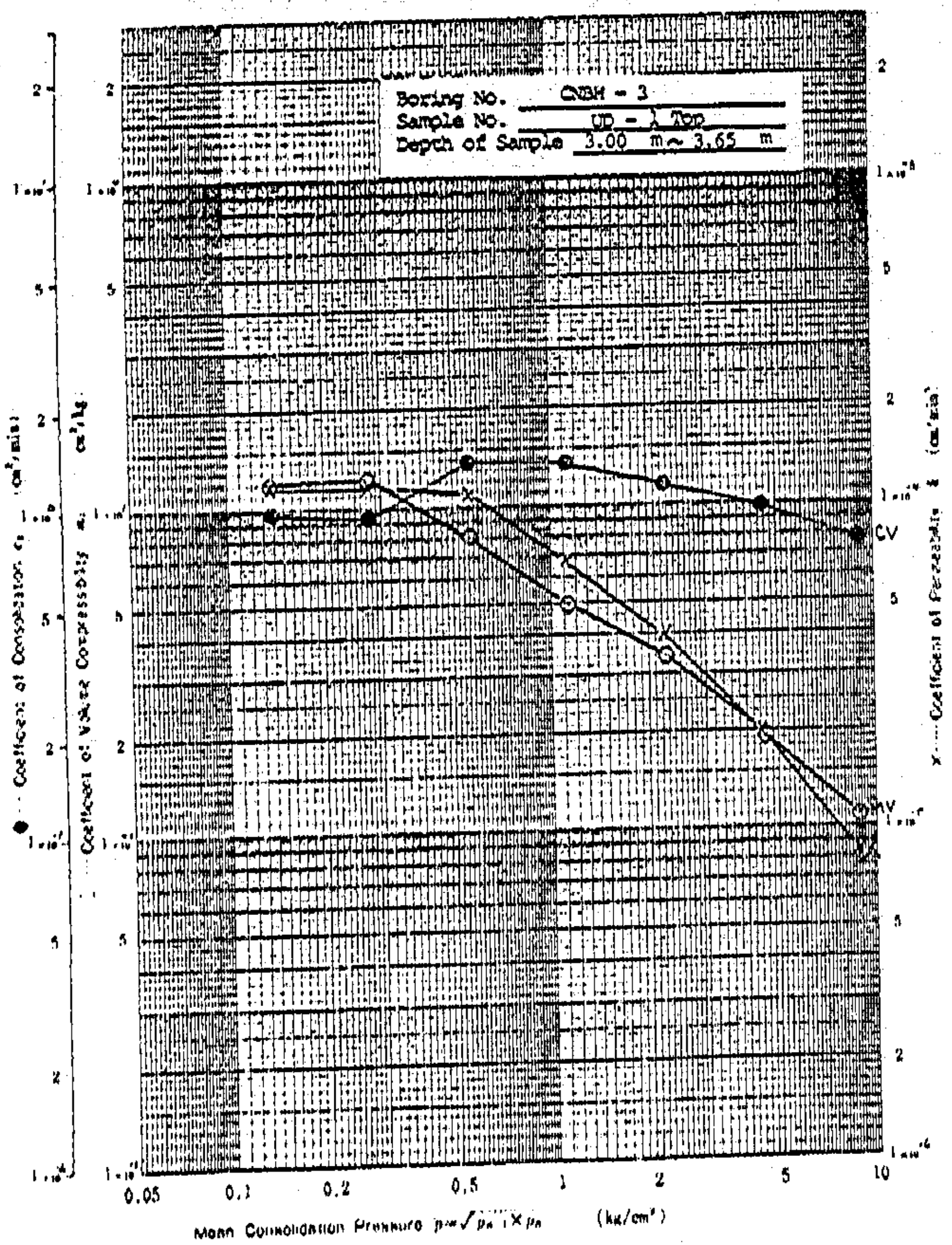
CONSOLIDATION TEST (p̄–Cv, mv, k, curves)



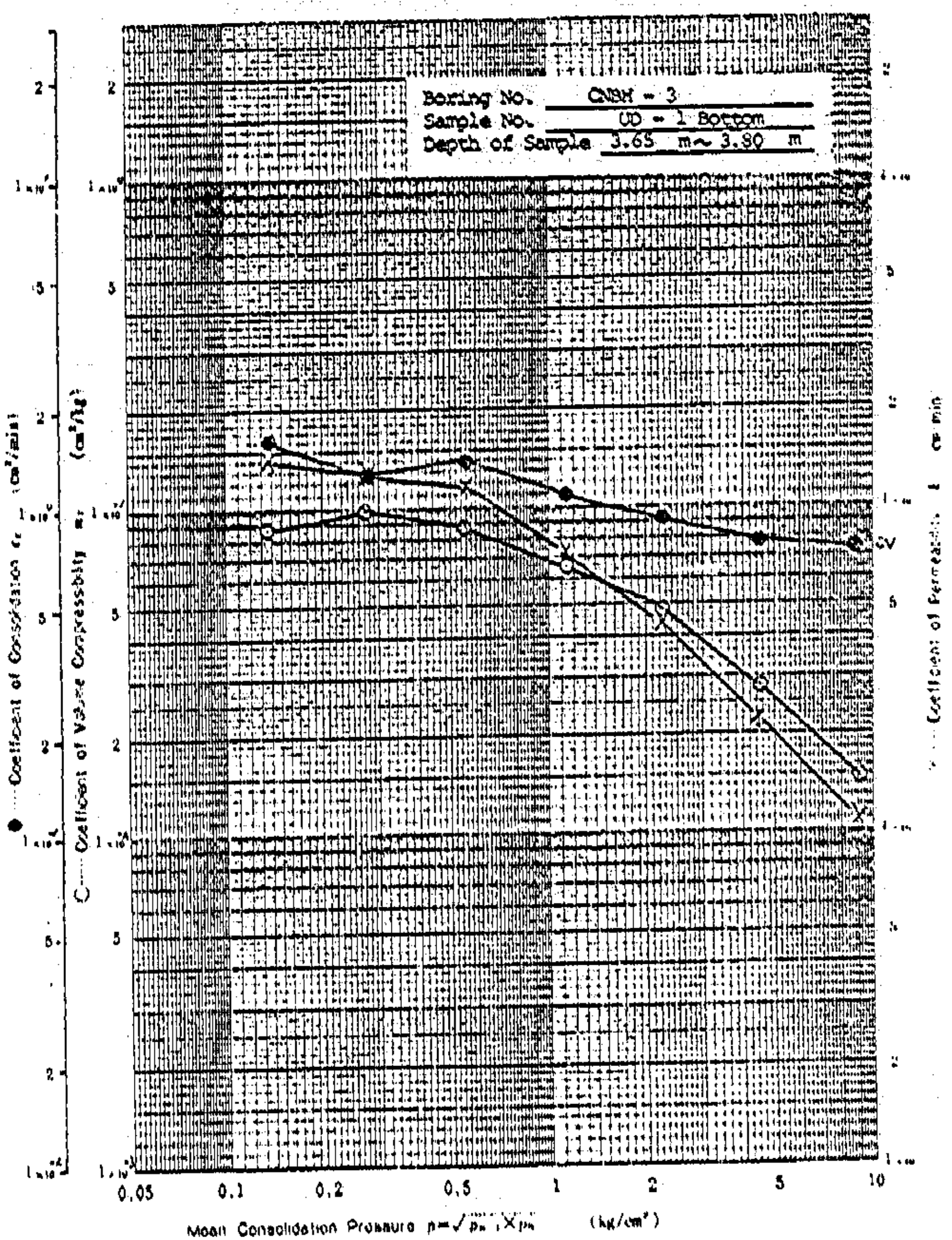
CONSOLIDATION TEST (p̄–Cv, mv, k, curves)



CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



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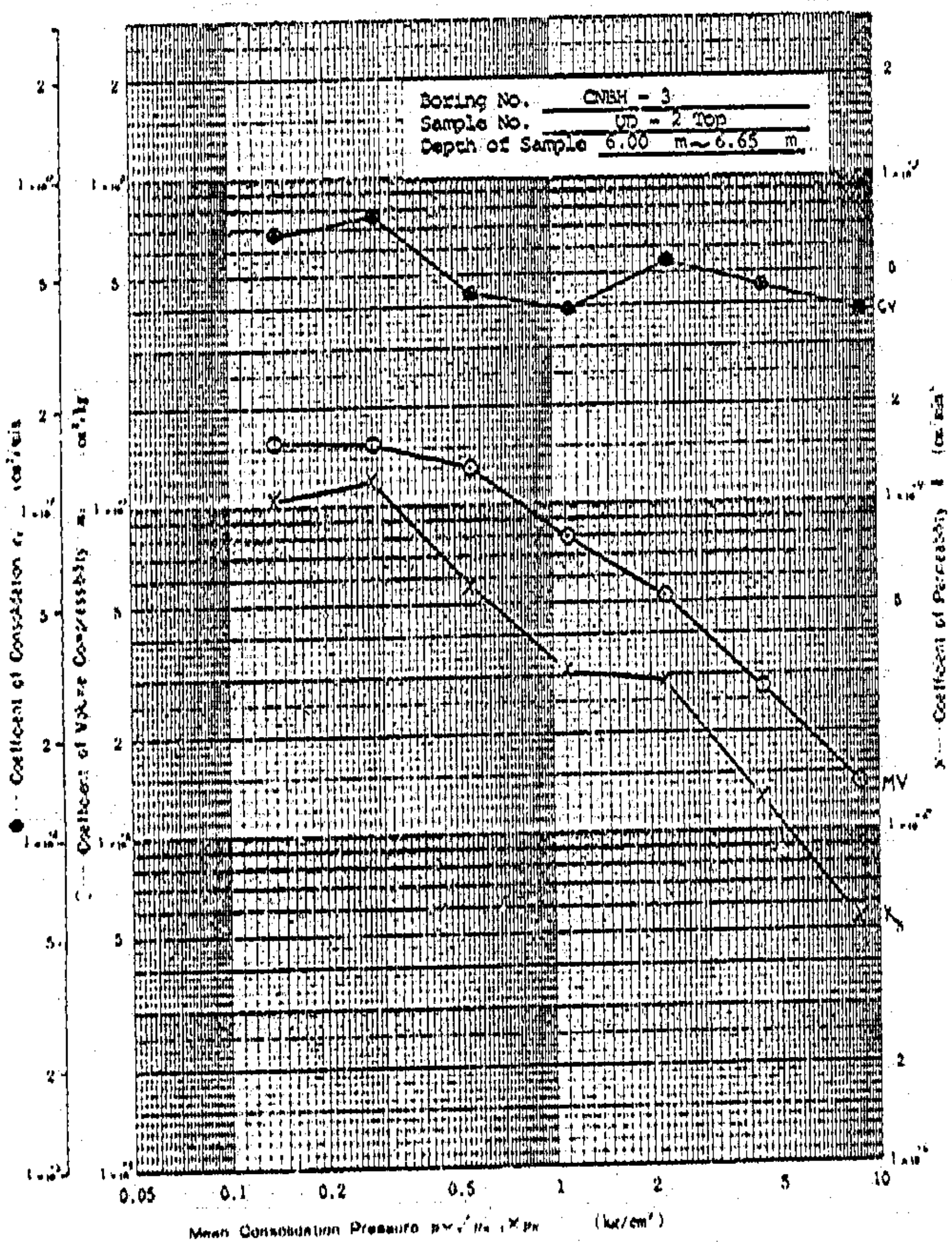

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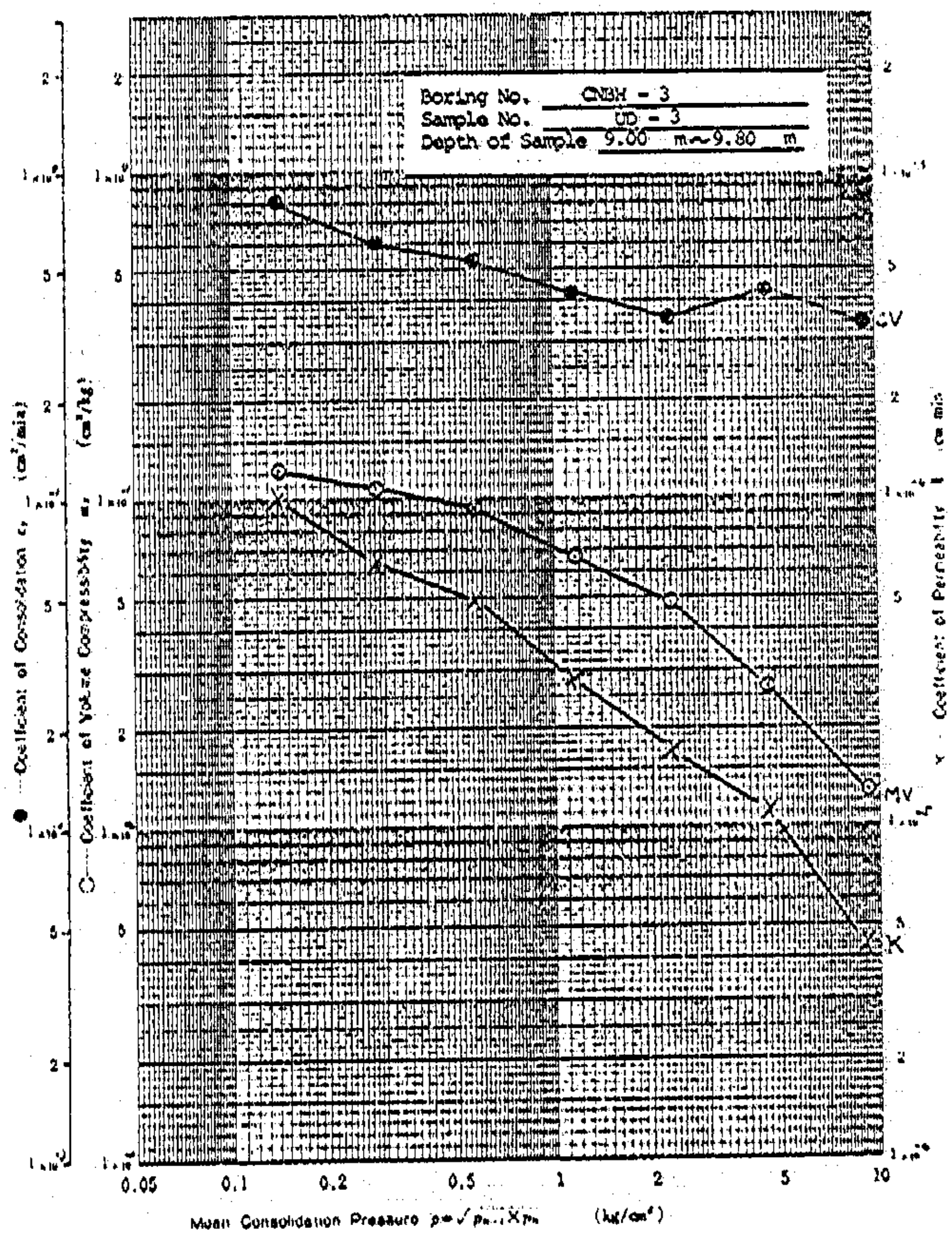


資料G　　地盤改良工法の紹介

資　料　G

地盤改良工法の紹介

G. DETAILED DESCRIPTION OF GROUND IMPROVEMENT METHODS

INTRODUCTION

Volume stability and soil strength, two major concerns of geotechnical engineers, are always present as design and construction constraints. In many areas, soil deposits do not meet either one or both of these requirements. Such soils commonly are classified as "soft," and may be composed of loose sands or silts, soft clays or organic soils, or combinations thereof.

Housing development at ex-mining land involves many constructions of structures that are built on soft, reclaimed land. Hence, the identification of such deposits and the determination of methods to modify the load distributions, to remove and replace the soft soil, or to improve the soft ground in place so that the strength and stability requirements under particular loading conditions are satisfied are major tasks in geotechnical engineering. Several methods have been developed and used as listed in Table G-1. The applicability of each methods depends on the satisfaction of a set of variables peculiar to the method and the site. Descriptions of the various methods and their application are presented in this chapter.

G. 1 Preloading

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

Table G-1 Classifications of Ground Improvement Methods

Principle			Measure	Method		Purpose of Ground Improvement			Suitable Material
						Strength	Compressibility	Dynamic Characteristic	
						Increase in strength, Increase in coefficient of deformation	Improvement of compressibility, Into less-compressible material	Improvement of dynamic characteristics, Prevention of liquefaction	
Soil Improvement	Increasing Soil Density	Dewatering	Consolidation	Pre-Loading		△	○		Clay, Silt
				Pre-loading with Vertical Drains	Sand Drain	△	○		"
					Paper Drain	△	○		"
					Fabric Drain	△	○		"
			Dewatering	Pumping	Wellpoint	△	○～△		Clay, Silt
					Deep Well	△	○～△		"
		Compaction	with Machine	Vibro-Rod		○	○	○	Sand, Sandy Silt
				Vibro-Floatation		○	○	○	"
				Sand Compaction Pile		○	○	○	Sand, Silt, Clay
				Dynamic Consolidation		○	○	○	Sand, Silt
				Roller Compaction		○	○	△	All Material
	Hardening		Chemical	Cement or Lime		○	○		Clay, Silt, (Sand)
				Chemical Composer Pile		○	○	○	"
Replacement			Excavation	Replacement by Excavation	Full Excavation	○	○		All Material
					Partial Excavation	△	△		"
			Compulsory Replacement	Compulsory Replacement	With Weight of Fill	△	△		Clay, (Silt)

○ effective

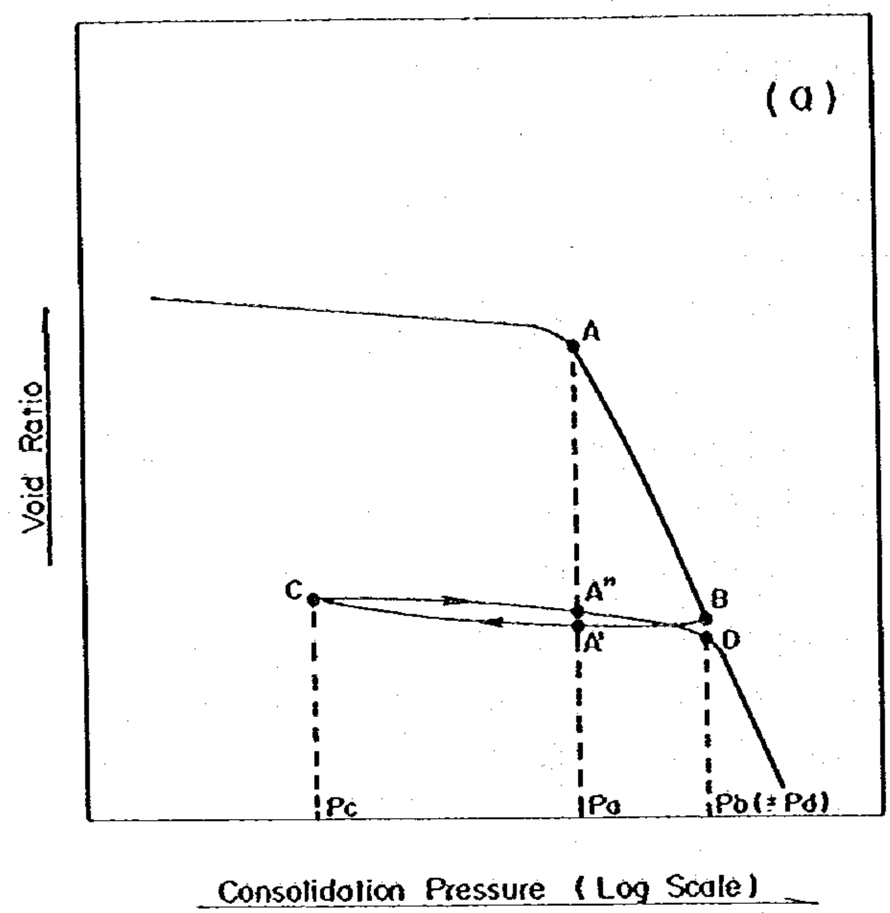
△ subordinate

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. G-1a. During this phase, undrained shear strength is increased as shown in Fig. G-1b.
2) Then removing the load, state of the soil turns into overconsolidated state and resultant increment of voild ratio and decrement of undrained shear strength is negligibly small.
3) Imposing the same load again, resultant decrement of void ratio is also negligible as shown by the points C and D in Fig. G-1a.

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. G-2.

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



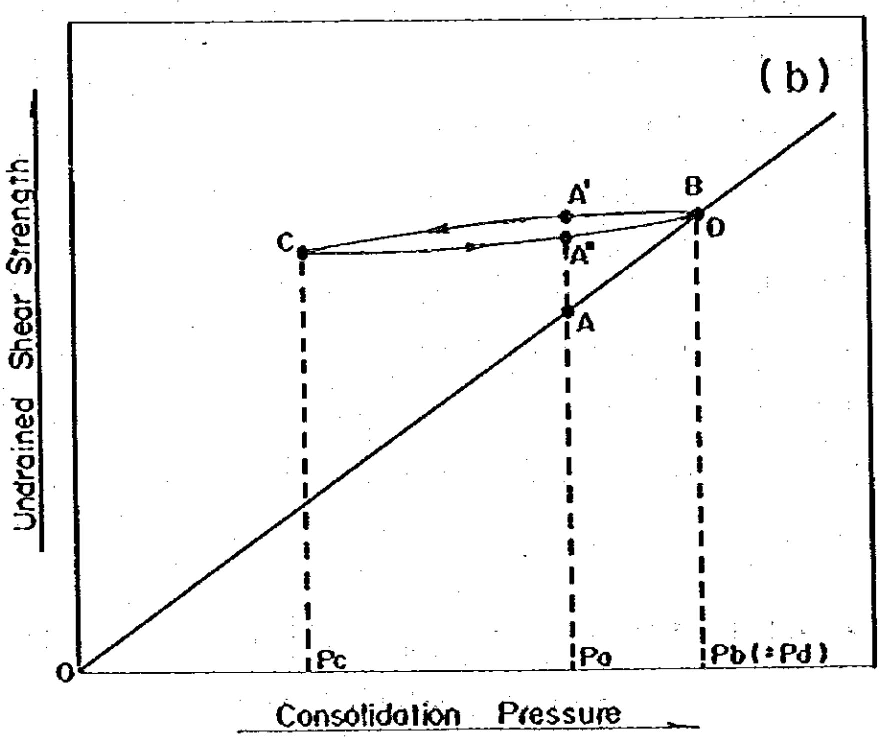


Fig. G-1 Conceptional Drawing of Consolidation Characteristics of Cohesive Soil

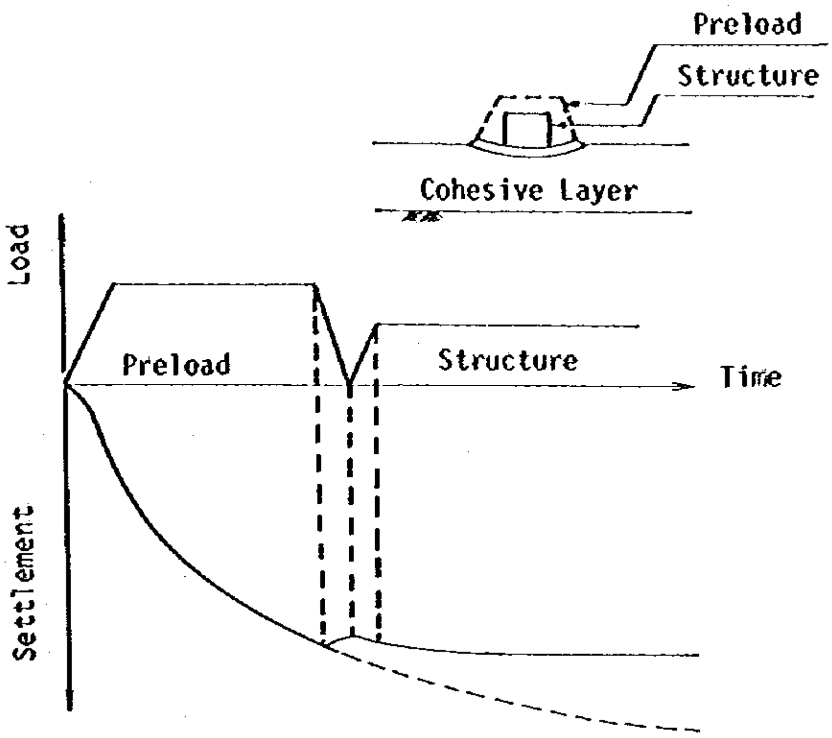


Fig. G-2 Descriptive Drawing of Preloading Method is placed on the proposed construction site before the construction. However, construction cannot begin until the preload period is over and the surcharge load removed.

The surcharging period and load intensity depend on the following:

1) The shear strength of the soil to be loaded---The purpose of surcharging is to compress the soil without causing a failure. Thus the load intensity must be limited so that the resulting induced stresses do not exceed the strength of the underlying soil.

2) The coefficient of consolidation of the foundation soils and the length of drainage paths---These parameters control the rate at which excess pore pressures are dissipated.

3) The amount of preconsolidation desired---Tolerable post-construction settlements for the foundations of buildings, storage tanks, industrial facilities or bridges vary from 0 in(0 cm) to 2 in(5 cm); for earth embankments, industrial work yards, etc. from 10 in(25 cm) to 30 in(76 cm)---provided that the differential settlements are not excessive. Thus, the post-construction settlement tolerances determine the required amount of preconsolidation, which in turn determines the intensity of the surcharge load and the length of the preload period.

The materials used for preloading depend on availability and utility. For embankment construction either a single stage surcharge or a rolling surcharge can be used. A single stage surcharge is normally a layer of soil superimposed on the full length of the embankment. At the end of the preloading period, this extra material is removed and used either for building and dressing the foreslopes of the embankment, for other earthwork in the vicinity, or wasted. A rolling surcharge covers only a portion of an embankment. At the end of the preloading period, it is moved and used as surcharge on an adjacent section. At the conclusion of the project, only a small volume of surcharge material has to be disposed of.

G. 2 Preloading with Vertical Drain

Preloading is a powerful, reasonable and economical method. However, an extremely 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.

G. 2. 1 Sand Drain

Sand drains are circular wells filled with free draining, coarse clean sand (Fig. G-3). The diameters of sand drains range from 14 in (36 cm) to 30 in (76 cm) with spacings ordinarily from 5 ft (1.5 m) to 15 ft (4.6 m), usually in a triangular or square pattern as shown in Fig. G-4.

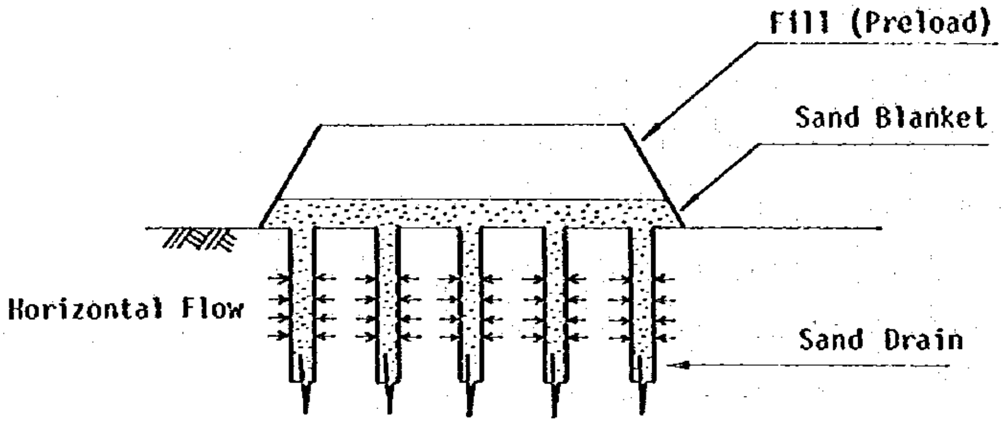


Fig. G-3 Typical Cross Section of Preloading with Sand Drain

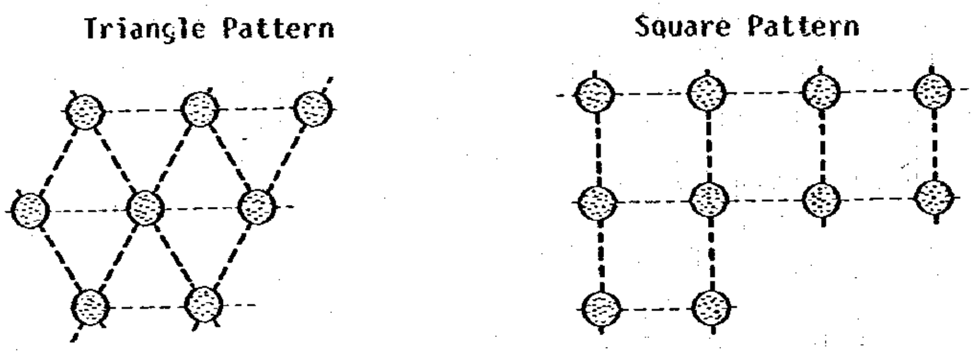


Fig. G-4 Layout of Sand Drain Piles

At the ground surface, the sand drains are connected with a continuous blanket of sand 1 ft. (30 cm) to 3 ft (90 cm) thick that acts as a collector and drainage layer.

The success of sand drains depends not only on the use of rational design methods, but on the techniques used to install the sand drains as well. Sand drains can be installed by the full-displacement method that utilizes a driven closed-end mandrel. Non-displacement installation is achieved by a variety of jetting techniques, or by drilling the wells with an auger. Fig. G-5 illustrates schematically the method of sand drain installation.

Sand drains are unnecessary (and often ineffective) in peaty soils. Such deposits have initially high permeability, so primary consolidation can be achieved quickly. Sand drains have limited effectiveness because secondary compression may represent the major portion of the settlement of these soils.

The surcharging of soft soils can cause lateral movement at the edges. If slip zones across the sand drains, the drains will be broken and rendered totally ineffective. Thus, if lateral flow of soft soils is expected, sand drains should not be installed.

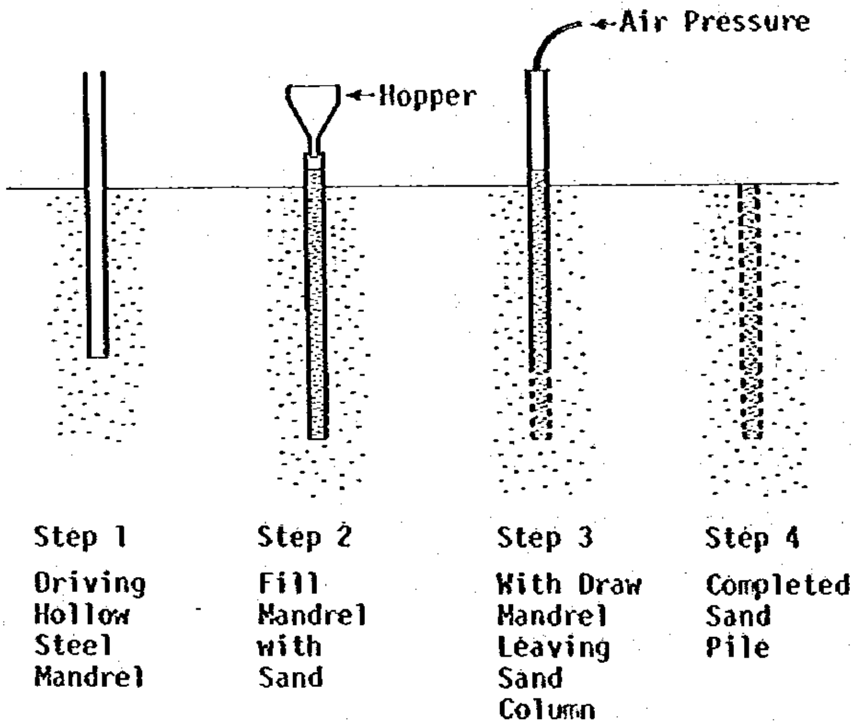


Fig. G-5 Installation of Sand Drain

G. 2. 2 Paper Drain and Fabric Drain

Besides sand drain methods, there are several vertical drain methods. They are paper drain, fabric drain methods and so on, of which names come from what their piles (drain wells) are made of. Though theoretical background of these methods are based on the same principles mentioned above, their engineering properties slightly differ. More advanced properties for paper drain method than sand drain method are as follows.

1) For paper drain method, more uniform and easier execution work can be accomplished.
2) Paper drain method is applicable to so weak ground that sand drain method can not be applicable.

In contrast with these benefits, there are unobjectionable problems for paper drain method. Above all, permeability and shear strength of paper drains are less than those of sand drains.

Hence, paper drain method needs numerous numbers of drain wells to gain high permeability. And increment of shear strength by replacement of the soft ground with drains can not be expected, as is possible for sand drain method.

G. 3 Dewatering by Pumping

In engineering practice difficulties with soils are almost entirely due, not to the soil elements, but to the water contained in the voids. It is generally known fact that water can have the deleterious effects on soil and earth retaining structures.

1) Upward-flowing water can cause a quick condition.
2) An increase in pore water pressure for a given total stress will cause a reduction in the effective stress and thus soil strength.
3) Water can add a very significant lateral thrust to earth retaining structures like retaining walls.

In soil engineering it is frequently highly desirable and sometimes essential to remove pore water from the soil or at least to reduce the pressure of the pore water. Lowering the phreatic surface can cause an increase in the effective stress within the soil and thus compress the soil. Frequently such dewatering is used in conjunction with preloading to improve the soil at a given site.

Dewatering can be a very useful and economical technique for improving soil; however, the soil engineer must examine the situation at hand, giving consideration to such factors as: (a) the probable effectiveness of the dewatering; (b) the amount of water that must be removed; (c) the time required for the dewatering; and (d) possible damage to nearby structures.

It is necessary to calculate, with reasonable accuracy, the total quantity to be pumped to achieve the required degree of lowering, the number of wellpoints or wells that will be required, their depth and yield.

To accomplish these procedures, the following equations are proposed asuming some hypotheses.

For the confined aquifer or artesian condition;

$$Q = \frac{2\pi kb(H-ho)}{\ln (R/ro)}$$

For the water tabel condition;

$$Q = \frac{\pi k \ (H^2 - ho^2)}{\ln (R/ro)}$$

Where, as shown in Fig. G-6

b: thickness of the confined pervious stratum

H: elevation of the original piezometric surface above the impermeable base

ho: elevation of the operating level of the pumping well above the base

R: radius of the Dupuit island

ro: radius of the well

k: coefficient of permeability

Also, it is necessary to assess the cost of the installation of wells and running costs of the pumps as well as their capacity. These calculations are dependent upon the permeability of the ground to be drained.

G. 3. 1 Wellpoint

The wellpoint method is based on a mechanism to put the interior of the ground in a vacuum condition, and it is possible

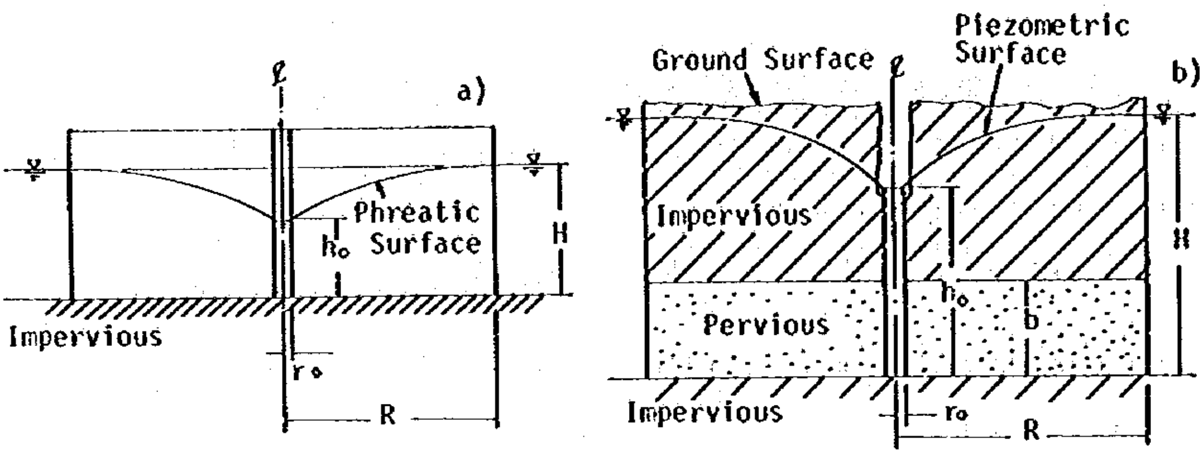


Fig. G-6 Vertical Section a) through Gravity Well, b) through Artesian Well

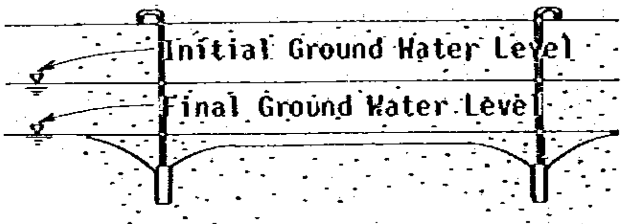


Fig. G-7.a Soil Stabilization

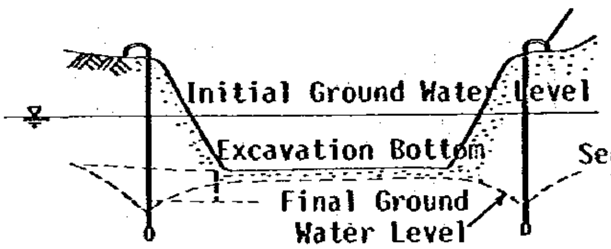


Fig. G-7.b Single-Stage Excavation

Fig. G-7.c Multiple Excavation

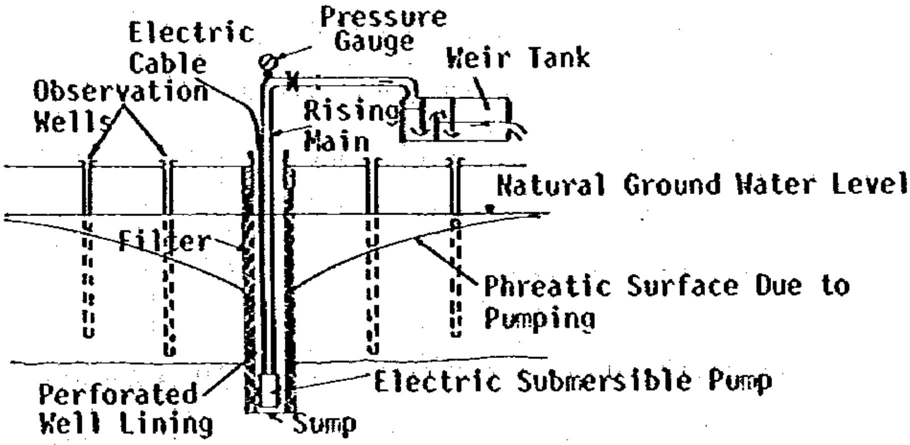


Fig. G-8 Typical Deep Well with Observation Wells for forcible drainage to be achieved not only from sand strata, but also silty strata of coefficient of permeability 10^{-4} to 10^{-5} cm/sec.

The theoretical head is 10 m, but actually, because of various losses, it is not possible to lift water to that height. The causes of the losses are mainly friction between wellpoint screens and surrounding soil, and internal friction as water passes through the wellpoint screen, riser pipe, swing joint and header pipe. Therefore, the actual depth to which water level can be lowered is a maximum of a round H = 5 to 6 m. Figs. G-7a to G-7c are examples of the wellpoint method applied to excavation and soil stabilization.

The amount of lowering by a single stage system is limited by the physical bounds of suction lift. In a clean medium and coarse sand some 5.5 m of lowering may be achieved, but the lowering achievable in a silty fine sand will be about 30% less. For greater depths multi-stage systems are required.

G. 3. 2 Deep Well

For deeper soil stabilization and excavations, it is usually advisable to use deep wells rather than the wellpoint system. Deep wells represent a reliable method of groundwater lowering in all types of permeable soils and, because of the facility they provide for the installation of selected or tailor-made filters, they are particularly appropriate to difficult ground conditions such as variable soils and mult-layer aquifers. The correct selection of filters is vital to the successful operation of a deep well system. A typical deep well and arrangement for a pumping test are shown in Fig. G-8 in page G - 14.

The amount of lowering that can be achieved with a deep well installation depends mainly on the spacing between wells, the amount of penetration of the wells into the aquifer, and the mechanical capacity of the submersible pumps in the wells. The yield of a well in drift deposits is somewhat insensitive to well diameter.

Deep wells are particulary suitable for controlling the groundwater where artesian or subartesian conditions exist at a site.

G. 3. 3 Shallow Filter Well

These are produced by a method which is a synthesis of the deep well and wellpoint methods of water lowering. The installation of the wells is basically identical to that of deep bored wells with the facility to ensure satisfactory filtering, but the individual riser pipes in each well are connected by way of a common suction header main to a well-point pump.

The cost of installation of wells is significant and so the shallow well system is more appropriate to use for a static installation that has to be pumped for more than a few months. On a congested site the use of a shallow well system may be preferred to a wellpoint system because of the smaller number of risers hindering the construction operations.

G. 4 Mechanical Compaction

When the use of deep foundations or any other ground improvement method with controlled fills becomes impractical or costly, artifical compaction to substantial depth may be considered.

The compaction of in-place soil has a significant influences on the engineering behaiviours of the compacted soil, those are increasing the relative density and decreasing the ground settlement. From Figs. G-9 and G-10, it is clear that the bearing capacity, which is a increasing function of N-values, and the stability have a great advantages by densification that is its main effect. The compaction of the ground can be successfully acomplished so long as the nature of the ground is fairly granular and permeable.

Granular soils are often encountered extending to considerable depths at densities too low to support planned engineering works especially if subjected to dynamic loading such as caused by earthquakes. Often it is required to densify deposits to as much as 100 ft (30 m) in depth.

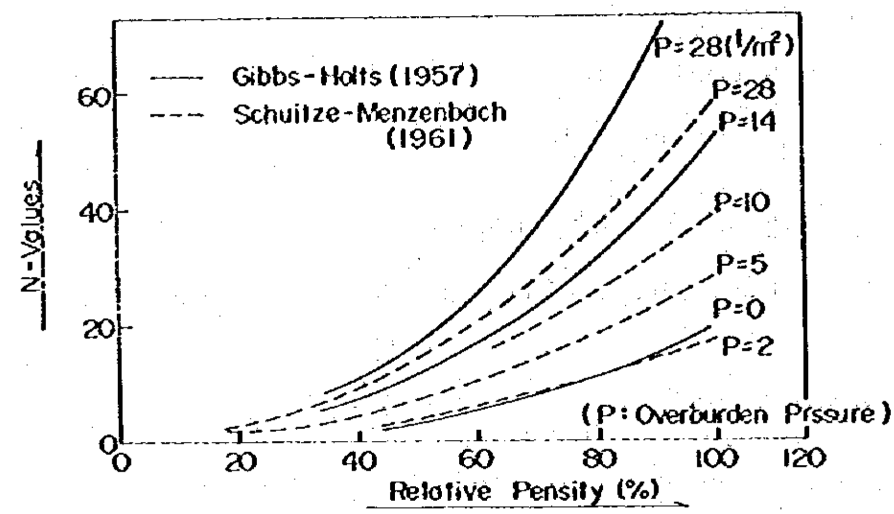


Fig. G-9 Relation between N-Values and Relative Density Dr

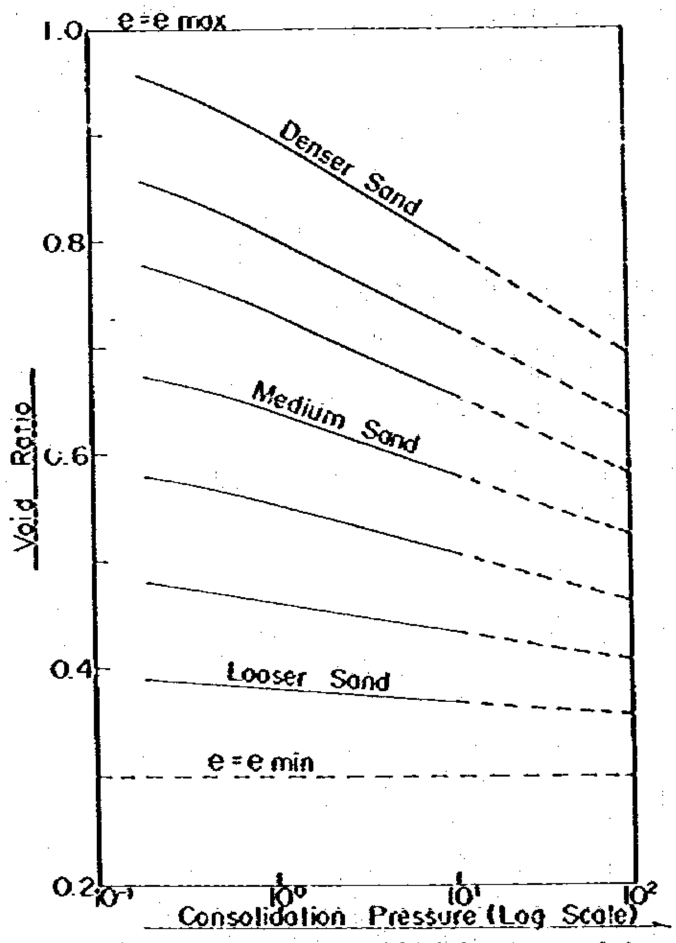


Fig. G-10 Relation between Void Ratio and Log Pressure on Sandy Soils with Various Densities

G. 4. 1 Vibro Rod

Experience with vibratory pile driving equipment led to the observation that granular soil was densified to a high degree in the vicinity of piles driven with this equipment. It was found that granular soil deposits of low density could be compacted effectively by driving and extracting a large open ended pipe, which is a vibro rod, in regulated patterns in the deposit, using the vibratory pile driver.

The vibro rod found to be the most effective was an open end pipe 30inches(76cm) in diameter and a 5/8inches(9.5mm) wall thickness.

Densification of the soil takes place both inside and outside of the pipe. The induced vibration is vertical with an amplitude of 10-25 mm. The frequency of the vibrator can be varied but normally is 15 Hz. No water jets are used in the vibro rod method; hence, its greatest effectiveness is in saturated sands. The process does not require addition of material around the probe as in vibroflotation, but rather, the increase in density of the soil is achieved by settlement of the overlying soil. A surcharge of sand is normally placed atop the deposit, in order to make up for soil used in the densification.

Densification of the deposit, as in vibroflotation, is partially dependent upon the weight of the overlying soil; therefore, it is advantageous to place the required surcharge before the work begins. The upper 3 to 5 ft (0.9 to 1.5 m) usually will require compaction by surface rolling. Although the cost per probe is less by this method than by vibroflotation, smaller spacings are generally required to achieve the same densification.

G. 4. 2 Vibroflotation

The oldest method of densifying granular soils to such depth is the process known as vibroflotation. This process remains one of the most effective methods of compacting deep deposits of granular soils. Soils having a maximum of about 20% fines, of which not more than 3% are active clays, are suitable for compaction by vibroflotation.

The uniqueness of the method lies in the vibroflot, a torpedo-shaped vibration generator specifically designed for this purpose. It is basically a cylindrical shell about 6 ft (2 m) long and 16 in (40 cm) in diameter that weights 3,500 lb. (1,600kg). An eccentric weight inside the cylinder rotates at 1,800rmp to develop large, horizontal forces. Water may be jetted from the bottom and the top of the vibroflot, under a pressure of 60 psi (42.2 ton/m^2)

At the spot to be compacted the vibroflotation is jetted into the soil. The compaction has four basic steps as shown in Fig. G-11.

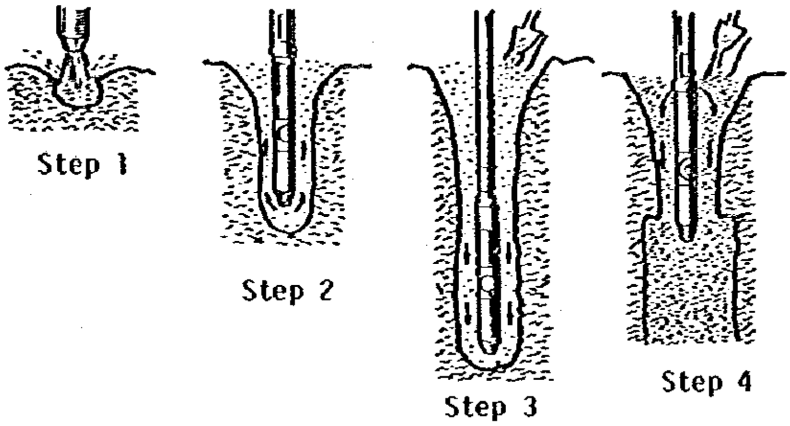


Fig. G-11 Compaction Process by Vibroflotation

Step 1 : At start, lower jet is opened full.

Step 2 : Water is introduced more rapidly than it can drain away. This creates a momentary "quick" condition ahead of the equipment which permits the vibrating machine to settle of its own weight to the desired depth.

Step 3 : The water from the lower jet is transferred to the top jets and the pressure and volume is reduced just enough to carry the sand to the bottom of the hole.

Step 4 : Actual compaction takes place during the intervals between the one-foot lifts which are made in returning the vibroflot to the surface. The vibrator is first allowed to operate at the bottom of the crater until the desired density around the lower part of the machine is attained. By raising the vibrator step by step and simultaneously backfilling, the entire depth of soil is compacted.

The relative density of the material is highest at the periphery of the vibroflot and decreases radially outward. The holes are spaced about 6 ft (2 m) to 12 ft (4 m) apart. Relative densities of 75% or more can generally be achieved. The upper 3 to 5 ft (0.9 to 1.5 m) will usually require densification by surface rolling.

Sand deposits as deep as 100 ft (30 m) have been compacted by this method. In some cases, slightly cohesive stratified soils and granular soils with cohesive lumps have been densified using this method.

G. 4. 3 Sand Compaction Pile

Compaction piles are piles driven solely for the purpose of densifying loose soil. Densification results from two effects: (1) Displacement of material equal to the pile volume; and (2) the effects of vibration during driving.

In the past measures such as the driving of wood piles on close centres in a grid pattern have been used. Compaction piles consisting of sand, gravel, or crushed stone have also been found to be effective. Such piles are constructed in a multi-step process by driving a hollow steel mandrel with a false bottom to the required depth, filling the mandrel and partially withdrawing and re-driving the mandrel in stages to compact the remaining material by vibration and compress it into the surrounding subsoil layer. At the completion of this process, a column of material, i.e. the sand compaction pile, remains in the hole. Fig. G-12 illustrates this procedure.

It can be said that the sand compaction pile method combines the draining effects of sand drains with the effects of vibration compact and is applicable for both sandy and cohesive soils. Principles of the sand compaction pile method are summarized in Fig. G-13.

In the case of sandy soil, the ground is compacted to below the critical void ratio, thus preventing liquefaction of ground during earthquakes. Thereafter, firmly compacted sand piles are utilized to increase the shearing resistance of the ground. The combined results help ensure that compression settlement is practically eliminated.

Order of Execution Steps

1. Install the pipe at the specified site on the ground.
2. Excite the vibrator to penetrate the pipe into the ground. When there is a hard layer which makes it difficult to penetrate the pipe, water jet or air jet is co-used to force penetration.
3. When penetration reaches a certain depth.
4. A certain quantity of sand is thrown into the pipe from the upper hopper and while drawing up the pipe up to the specified level, sand in the pipe is forced out into the bored hole by compressed air.
5. The pipe is redriven into the hole and sand is compacted by vibration. Sand is compressed into the surrounding subsoil layer.
6. Again feed sand into the pipe and draw up the pipe to the specified height.
7. By repeating the above process, complete the compozer pile up to the ground level.

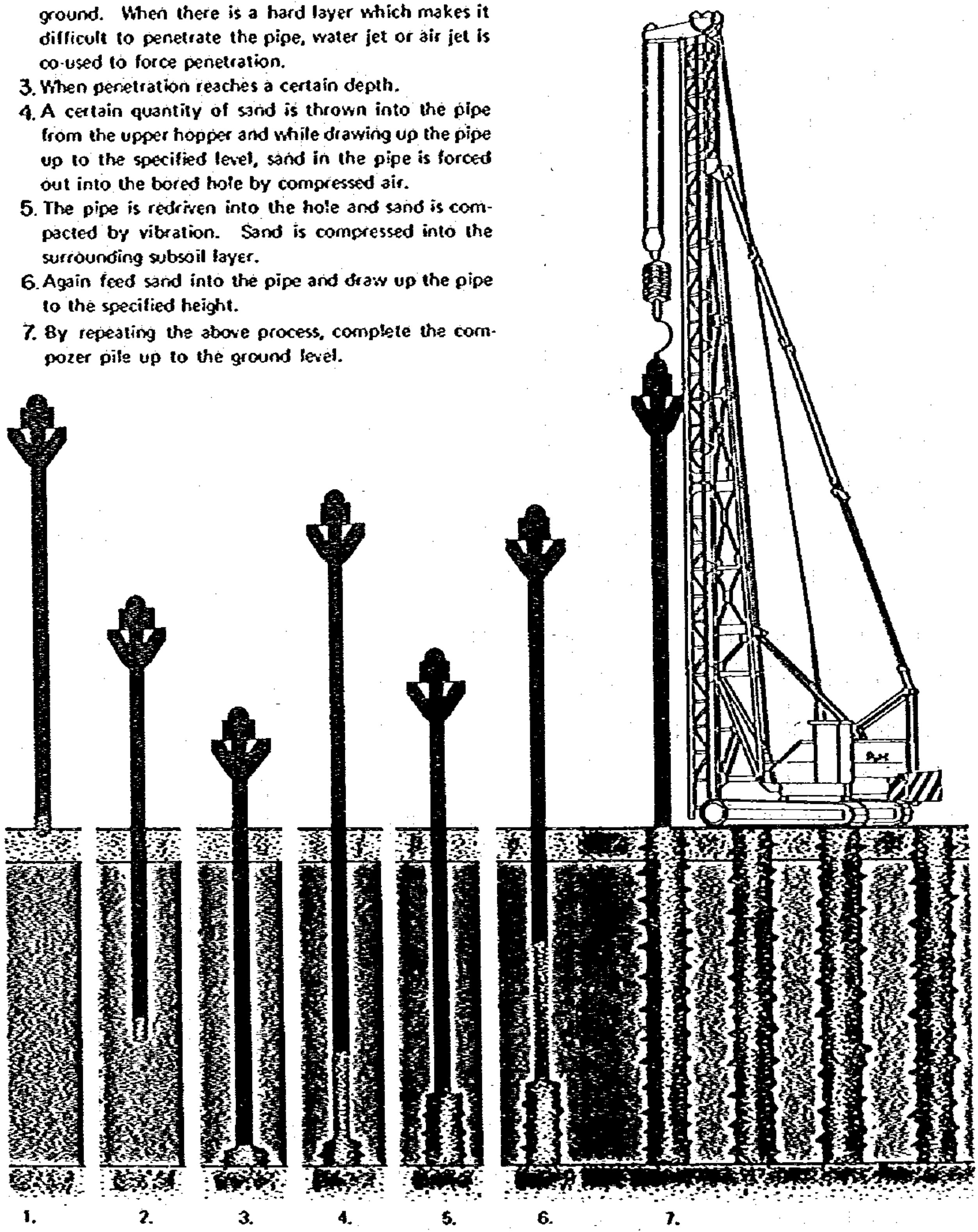
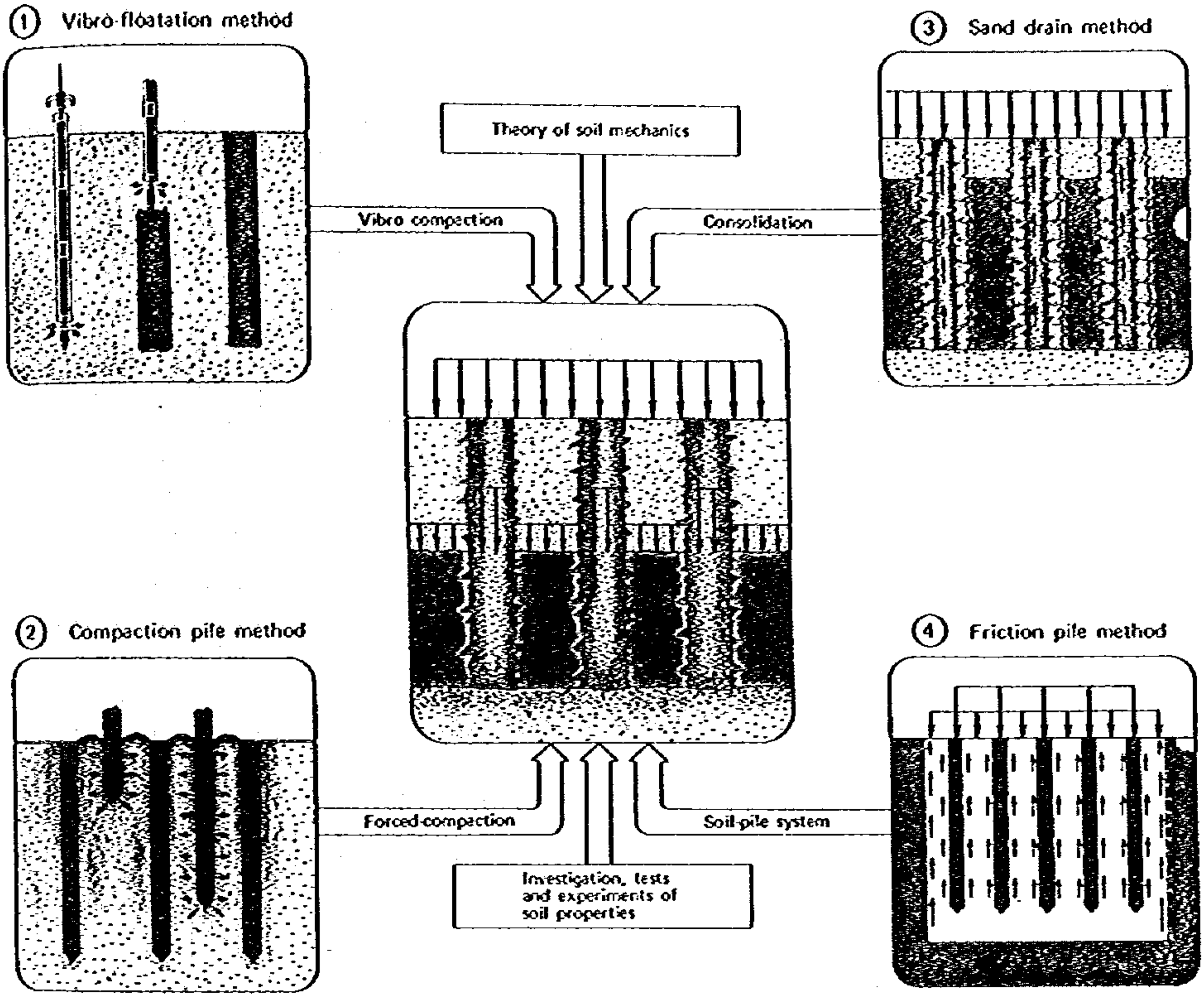


Fig. G-12 Procedure of Sand Compaction Pile Installation



This method consists of driving in well compacted sand piles of large diameter into loose sandy or soft clayey subsoils under vibration loading caused by the Compozer.

For sandy subsoils

1. The sand pile itself is compacted to a remarkable density, and the intermediate subsoil is also compacted to meet the required strength; thus the average subsoil strength is materially increased.
2. Vibratory compaction effect efficiently works on the loose sandy subsoils.
3. Since the subsoil is compacted to the level of critical void ratio, almost no settlement will be caused and the subsoil conditions stable against vibration may be achieved.

For clayey subsoils

1. Bearing capacity as a composite subsoil formed by combined sand piles and clay subsoils is displayed. Under a normal loading speed of superstructure construction, bearing capacity increases without causing any destruction.
2. Overall shear resistance is increased and it in turn helps in preventing subsoil failure.
3. Through replacement effect, stress concentration effect and preconsolidation effect, settlement will be greatly reduced, and besides the term required for stabilization will be extremely shortened.

Fig. G-13 Principles of Sand Compaction Pile Method