TRIAXIAL COMPRESSION TEST (Mohr's circle)

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

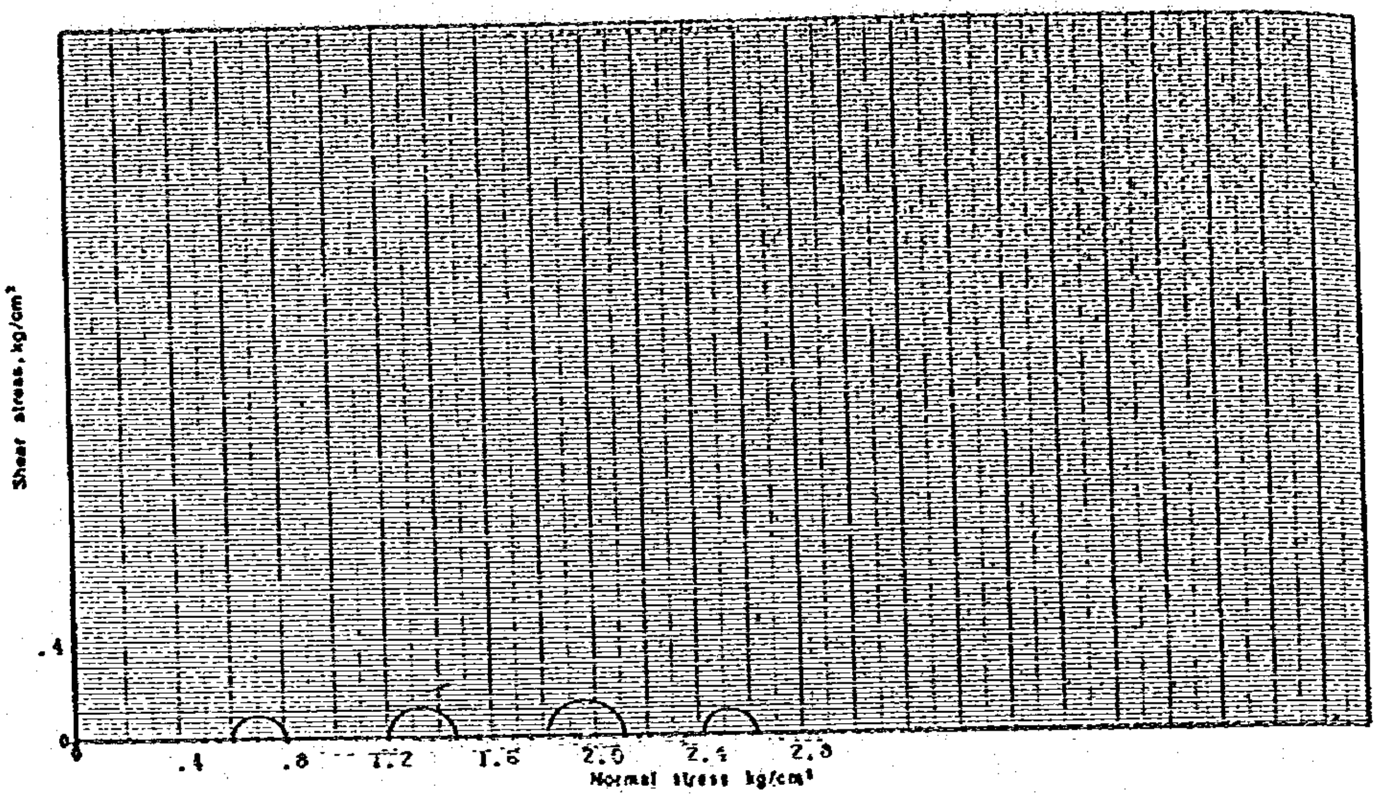
Depth of Sample 9.00 m - 9.80 m

Angle of internal friction 0°

Cohesion 0.12 kg/cm²

Project 267

Condition of drainage U-U



TRIAXIAL COMPRESSION TEST (Mohr's circle)

Boring No. CNBH-3 Sample No. UD-21cp

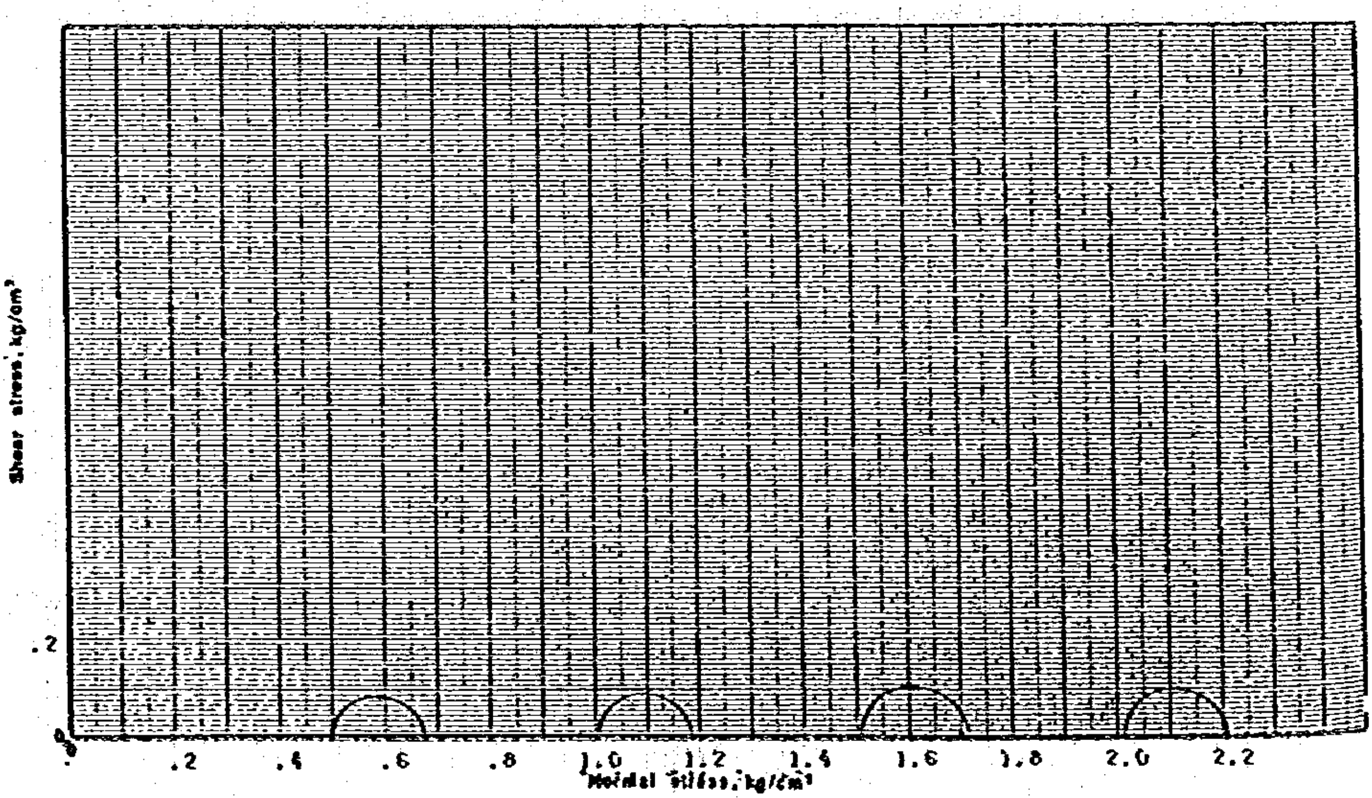
Depth of Sample 6.00 m - 6.65 m

Angle of internal friction 0°

Cohesion 0.09 kg/cm²

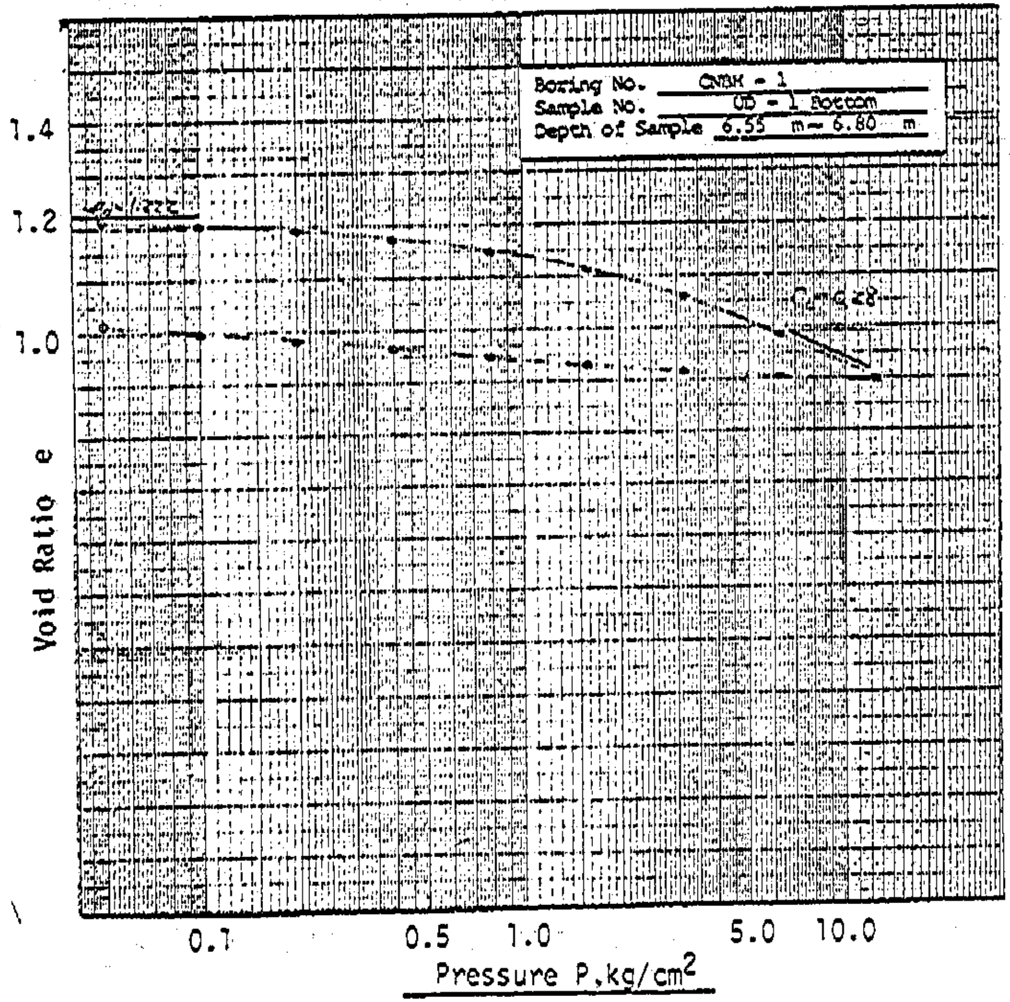
Project 267

Condition of drainage U-U



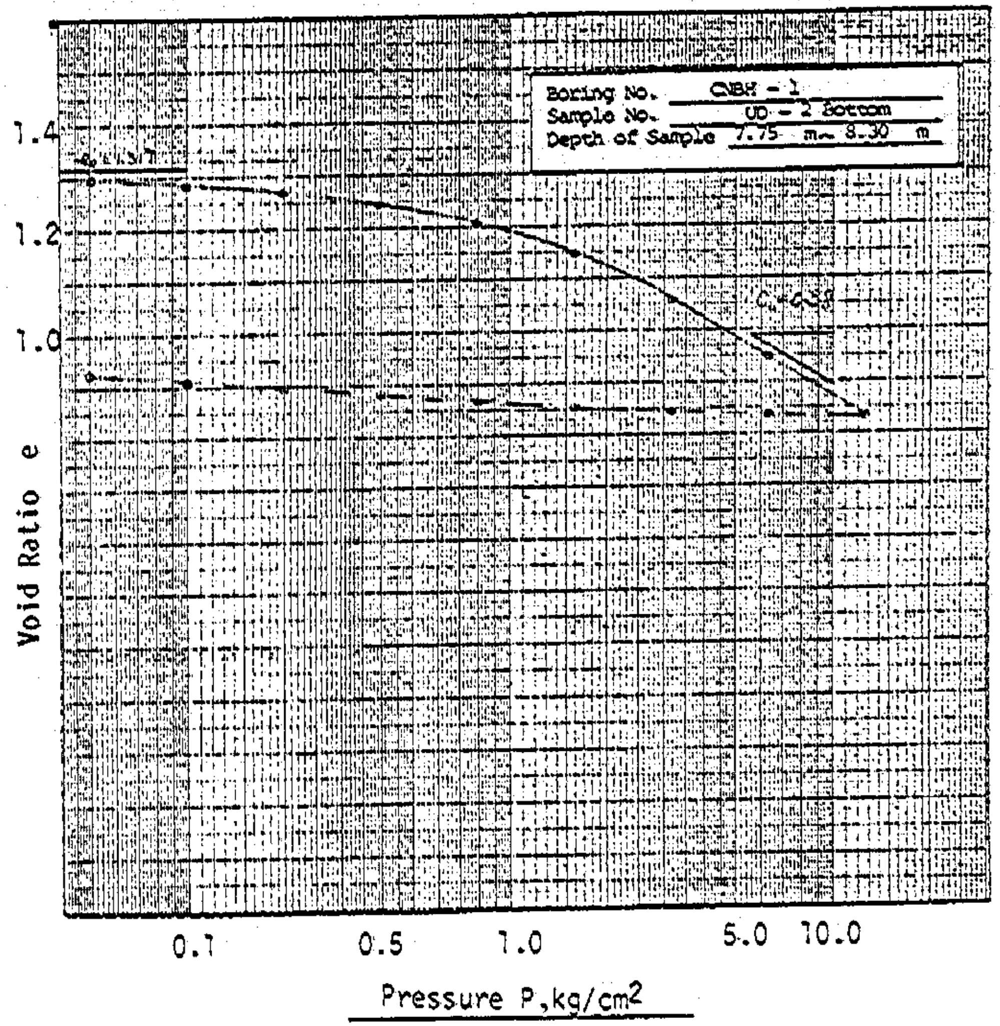
CONSOLIDATION TEST (e-log p curves)

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	[illegible]	[illegible]	1.222	—	0.28	⊙
[illegible]						△



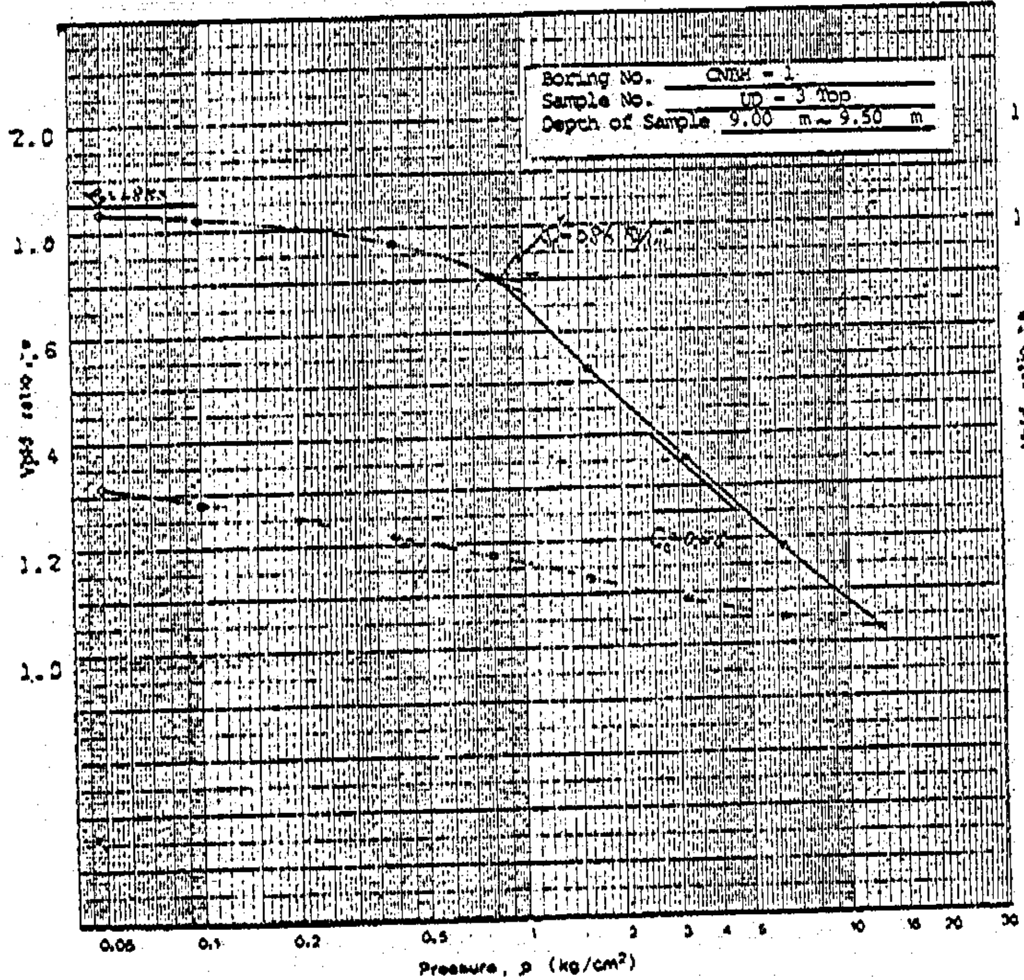
CONSOLIDATION TEST (e-log p curves)

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



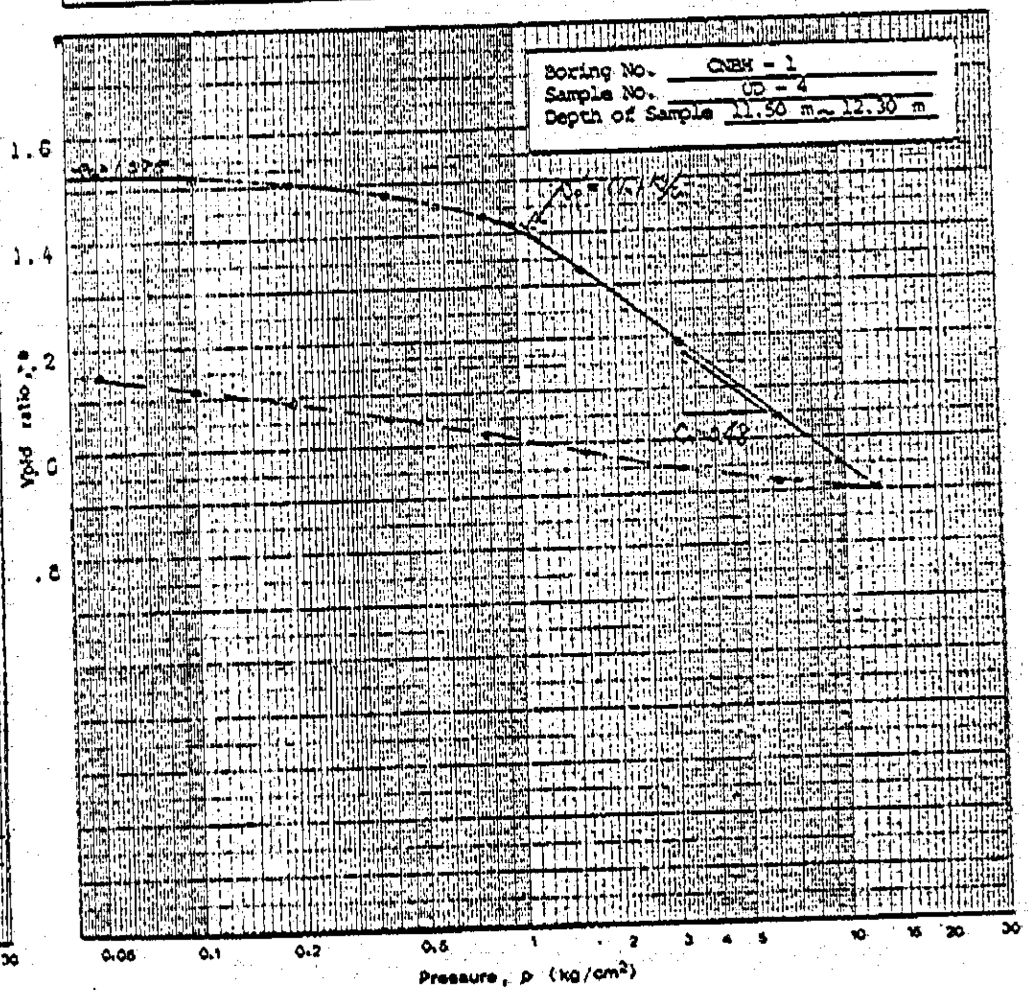
CONSOLIDATION TEST (e-log p curves)

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-3	9.00-9.50	52.0	1.863	0.86	0.58	⊙
Top						△



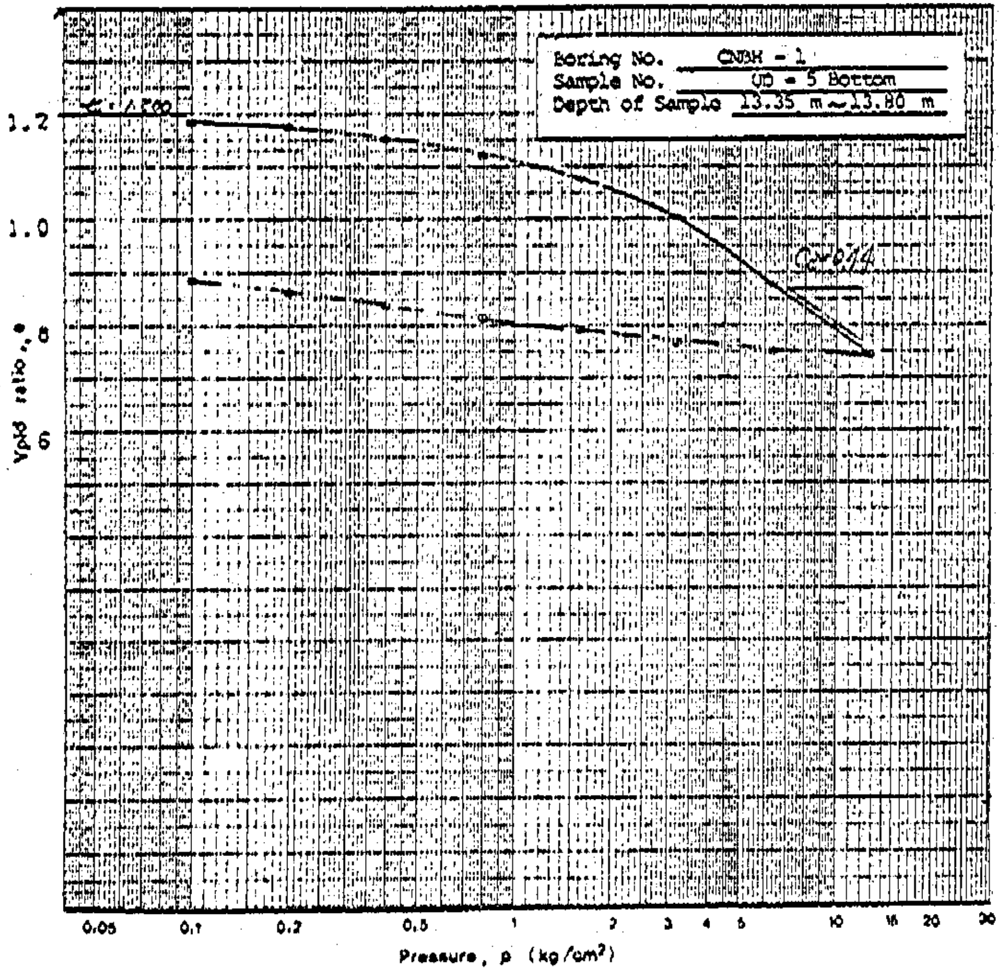
CONSOLIDATION TEST (e-log p curves)

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-4	11.50~12.30	51.6	1.525	(1.1)	0.48	⊙
						△



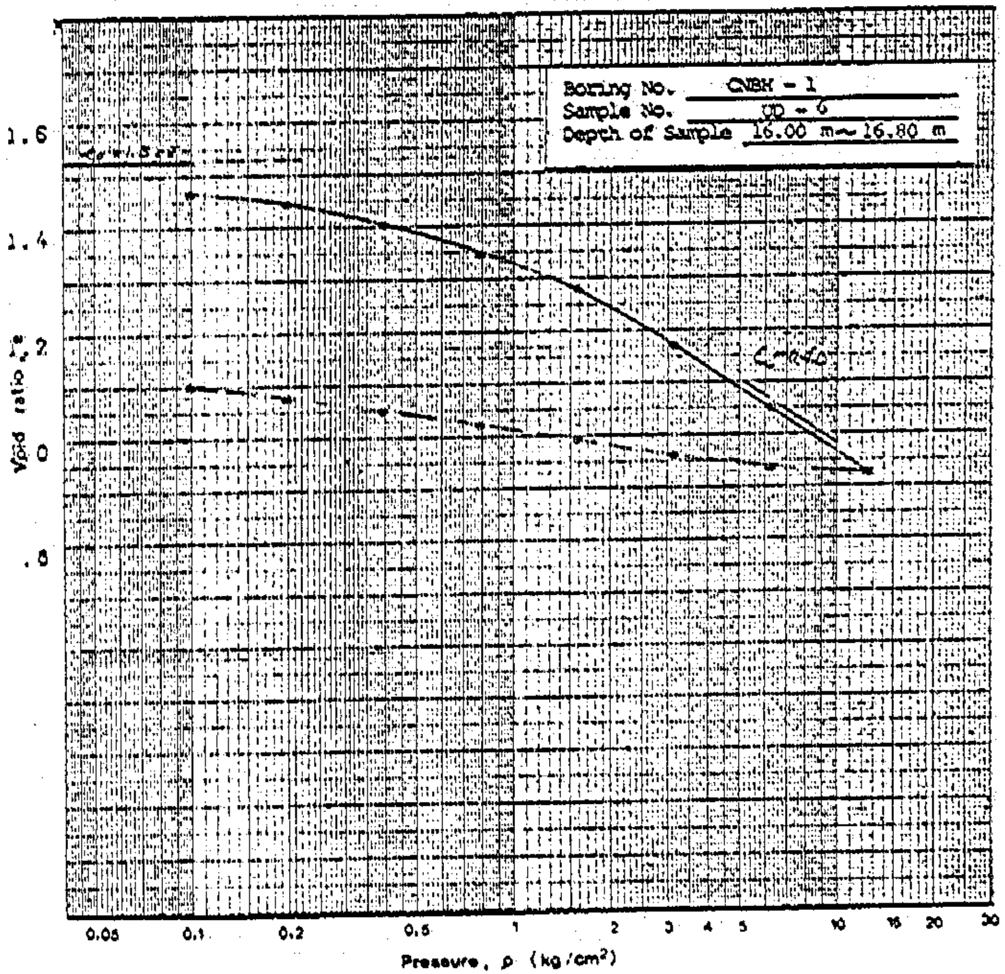
CONSOLIDATION TEST (e-log p curves)

Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_0	Preconsolidation Pressure σ'_p (kg/cm²)	Compression Index Cc	Symbol
UD-5	13.35~13.80	34.6	1.200	--	0.14	⊙
[illegible]						△



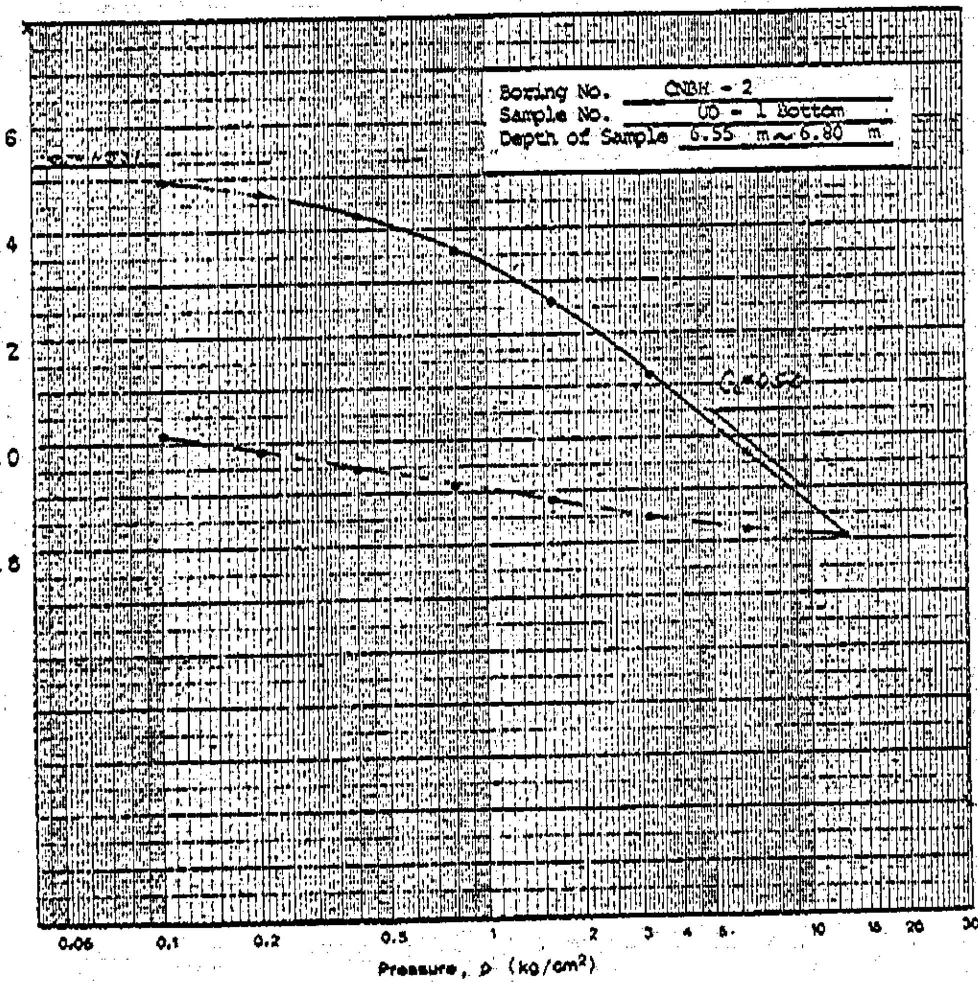
CONSOLIDATION TEST (e-log p curves)

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-6	16.00~16.80	58.2	1.528	—	0.40	⊙
						△



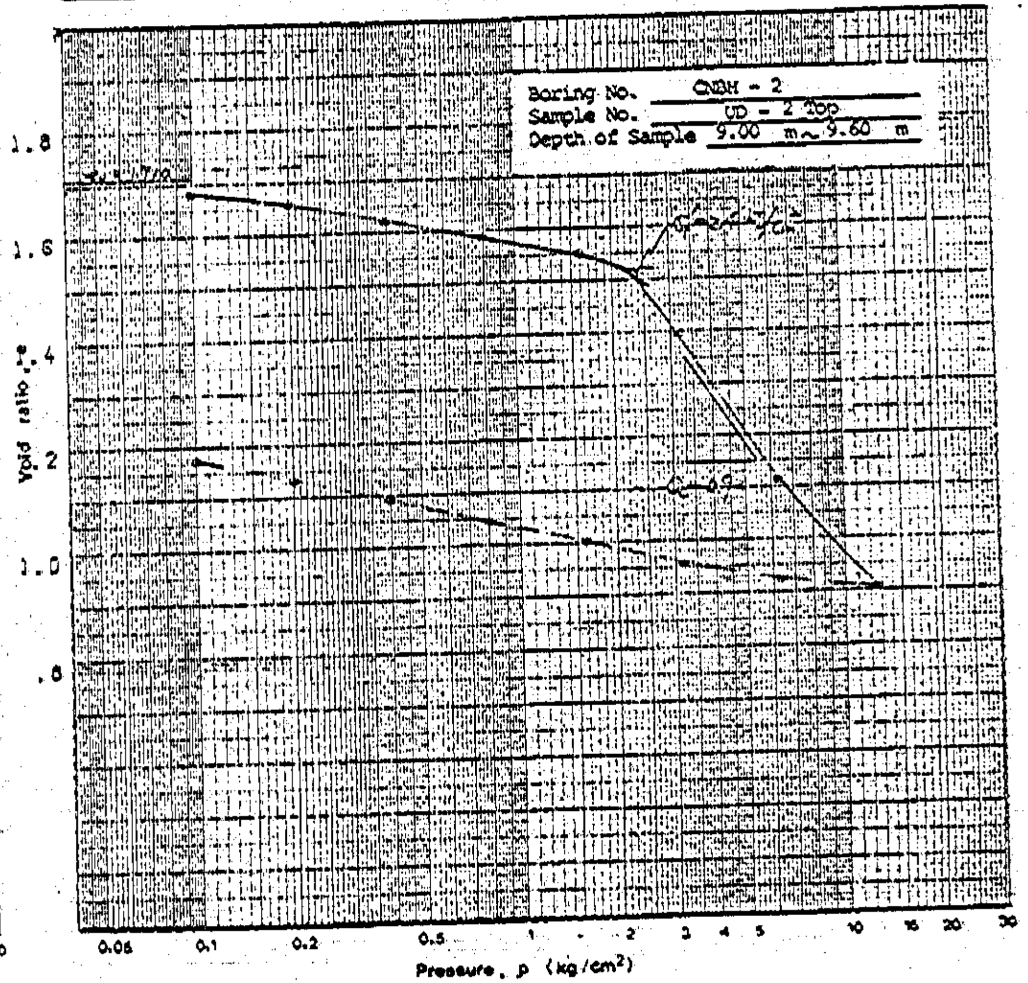
CONSOLIDATION TEST (e - log p curves)

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						△



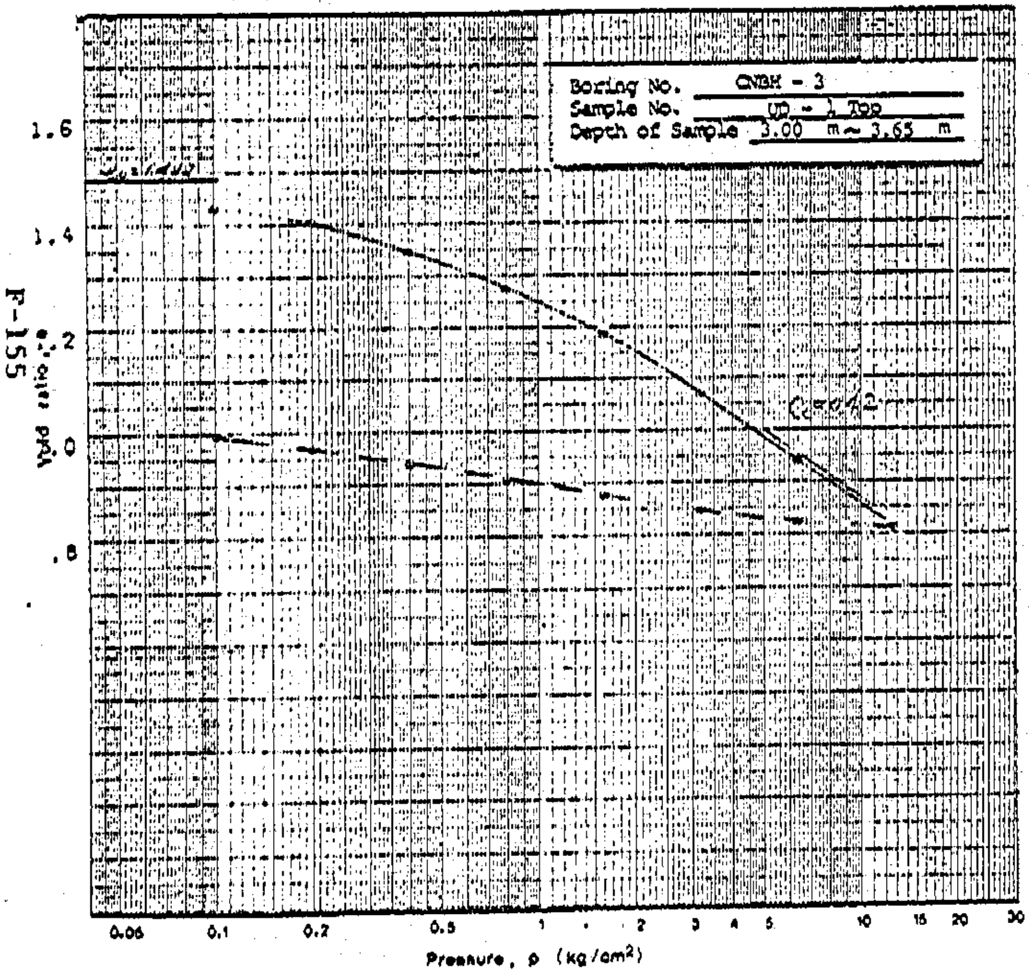
CONSOLIDATION TEST (e - log p curves)

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	1.710	2.4	0.92	⊙
Top						△



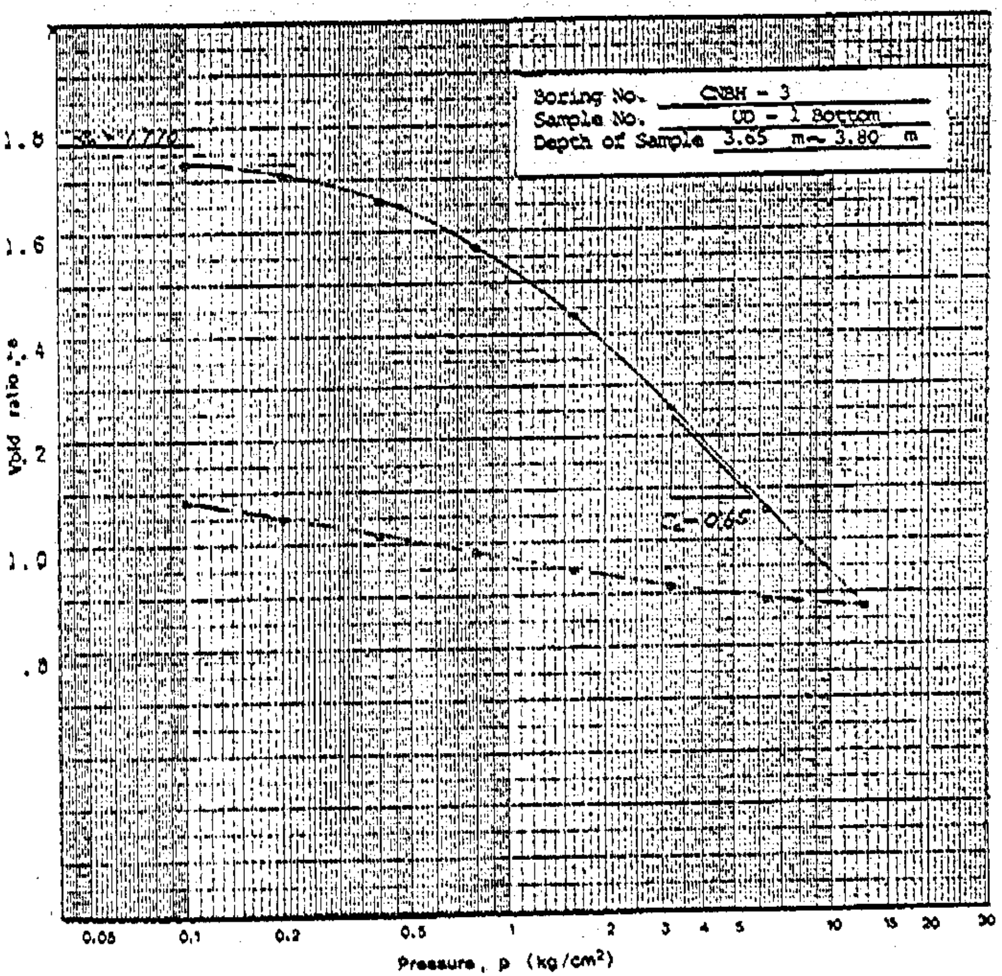
CONSOLIDATION TEST (e - log p curves)

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	—	1.988	----	0.42	⊙
Top						△



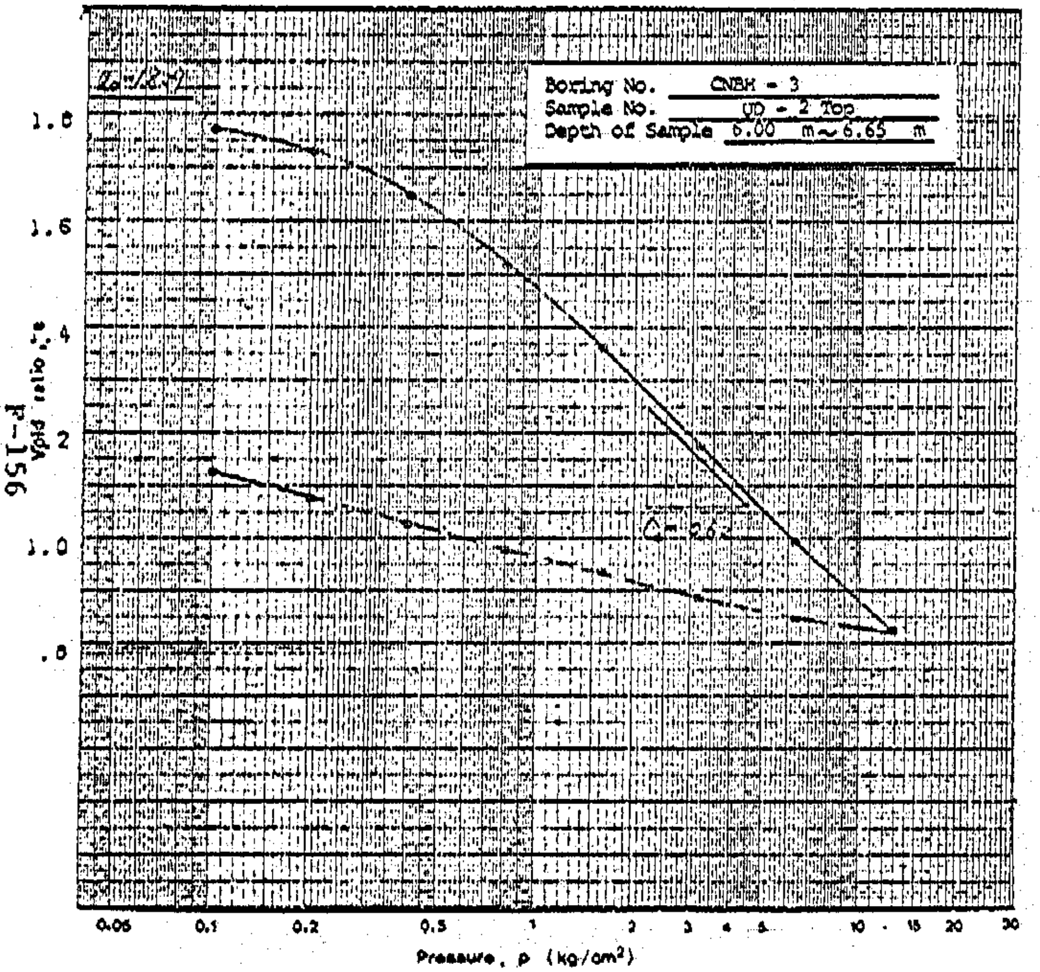
CONSOLIDATION TEST (e - log p curves)

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						△



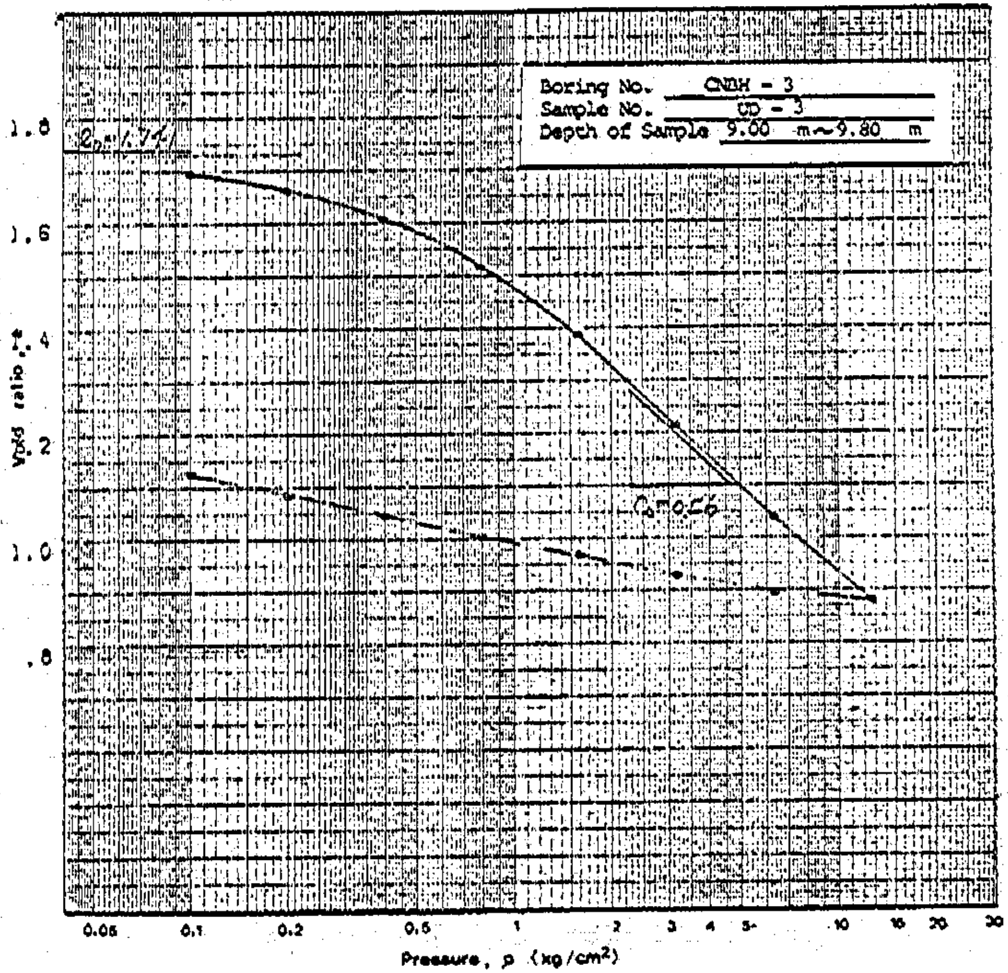
CONSOLIDATION TEST (e - log p curves)

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						△

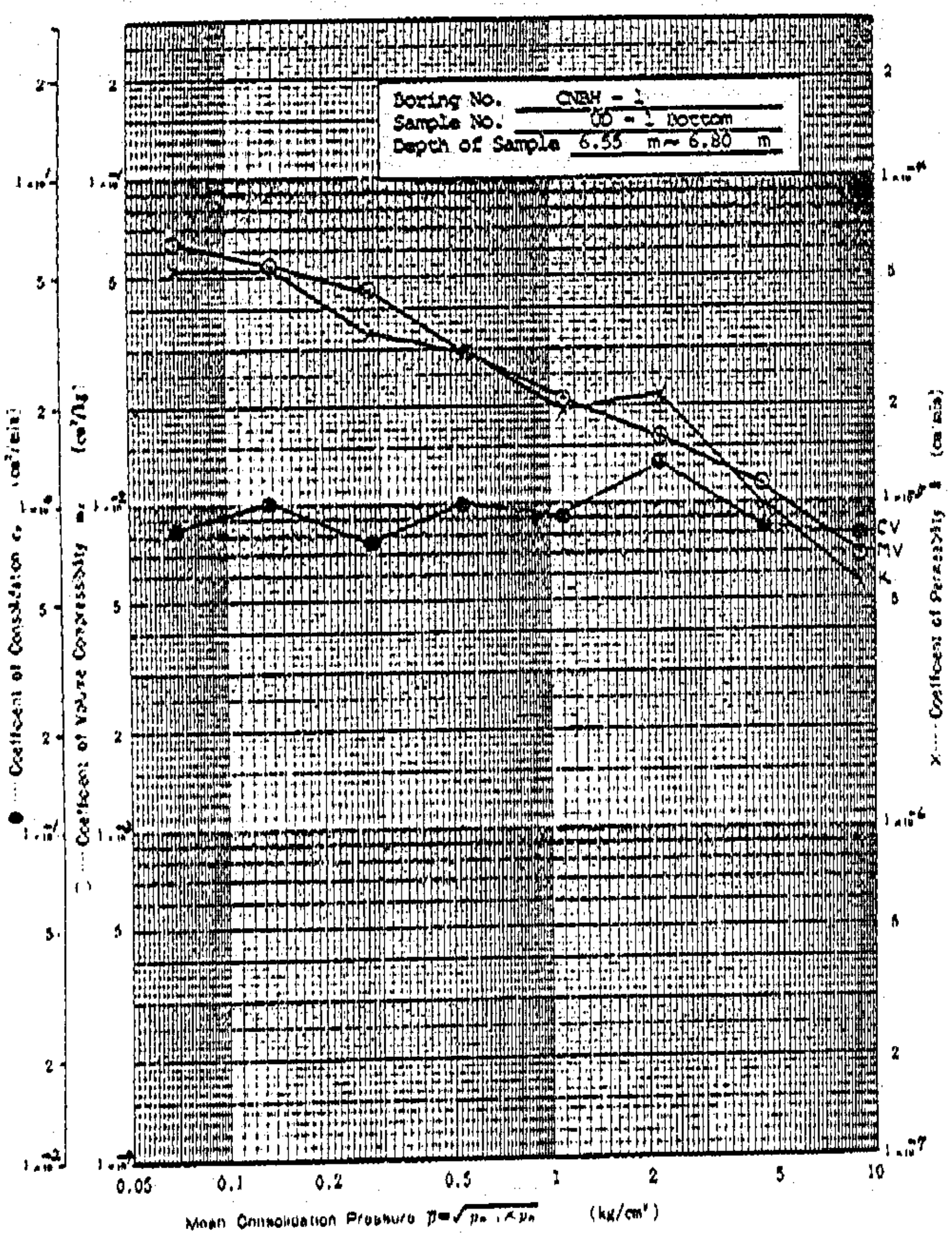


CONSOLIDATION TEST (e - log p curves)

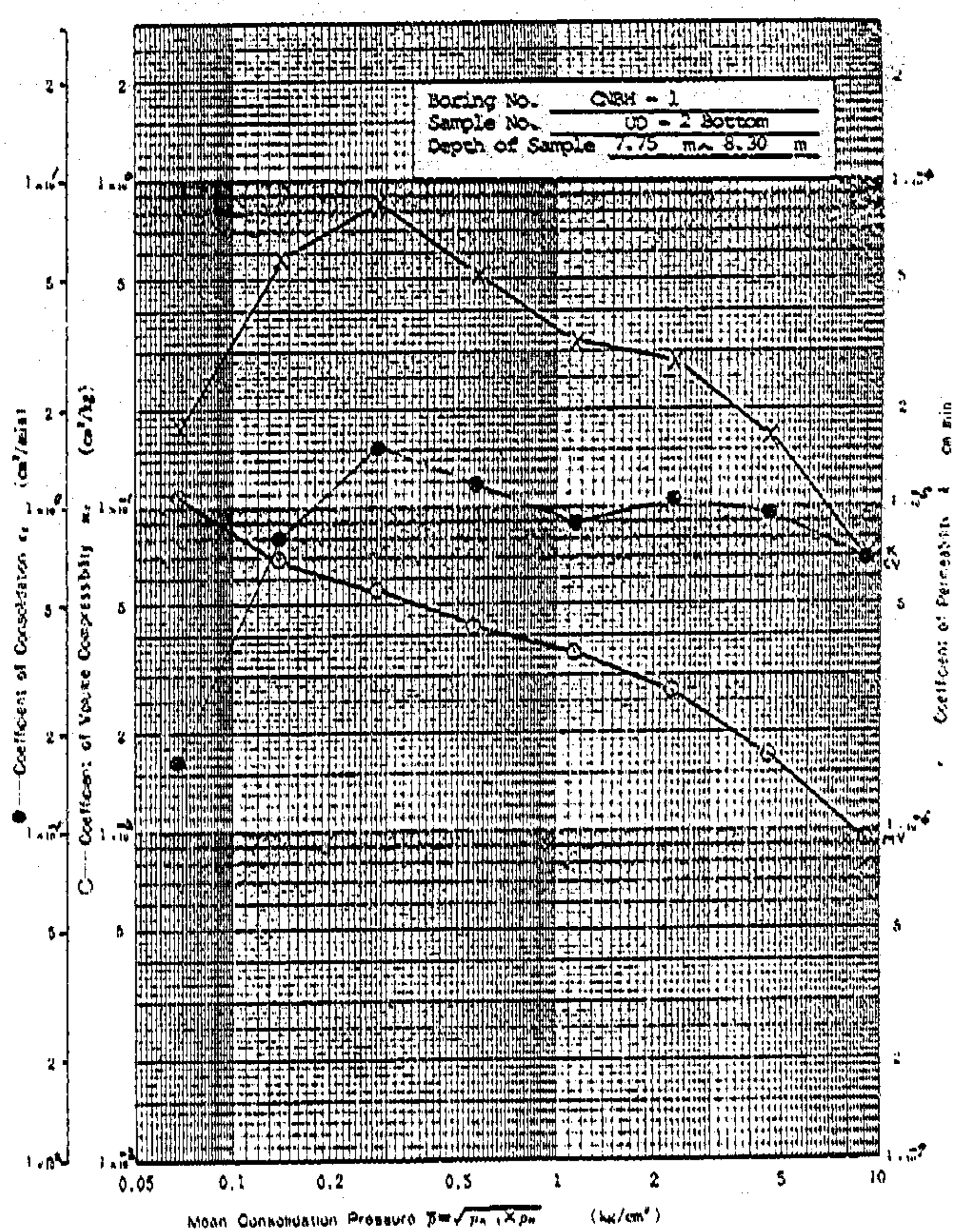
Sample No.	Depth of Sample (m)	Liquid Limit LL (%)	Initial Void Ratio e_o	Preconsolidation Pressure σ'_p (kg/cm2)	Compression Index Cc	Symbol
UD-3	9.00 ~ 9.80	59.9	1.741	—	0.56	⊙
						△



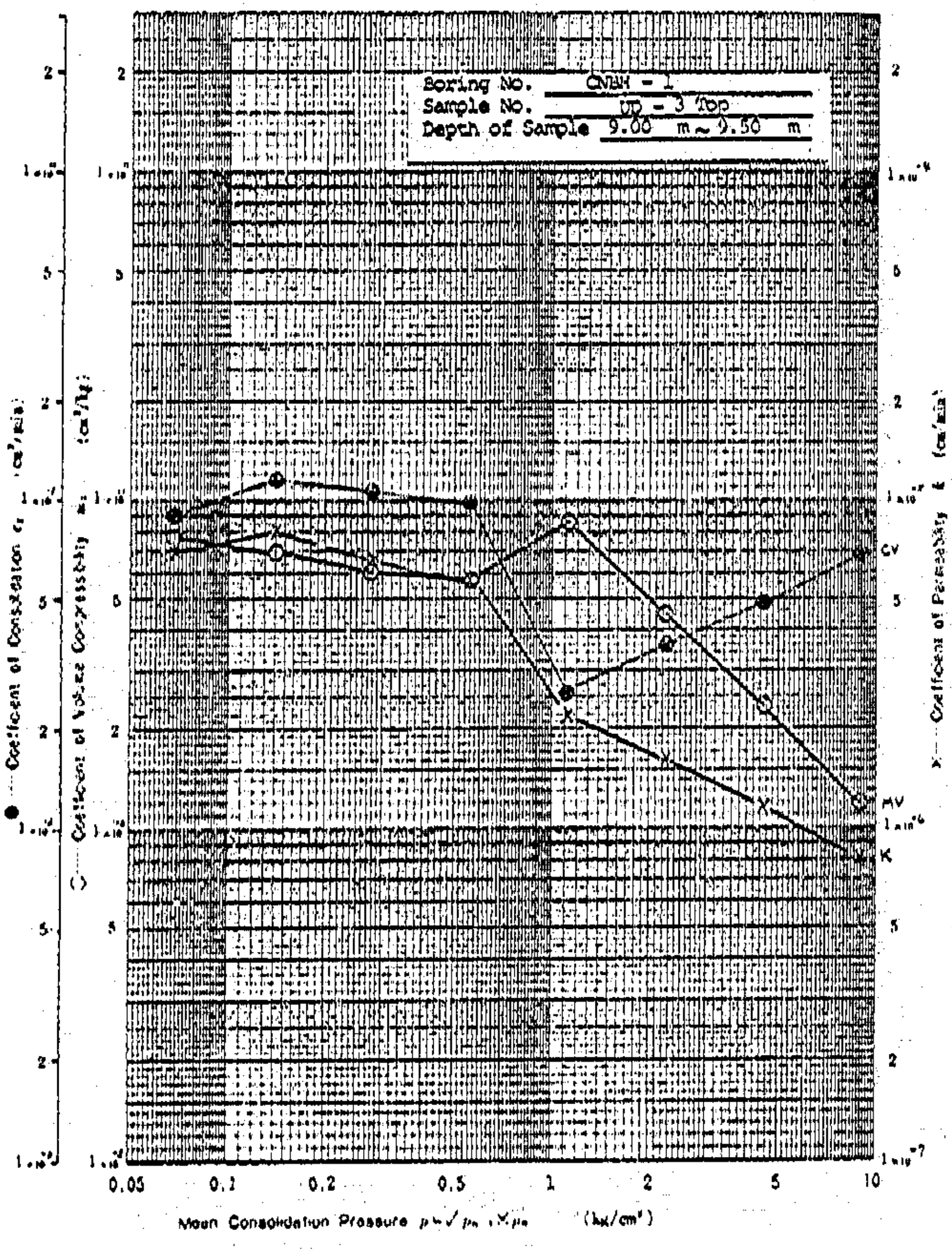
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



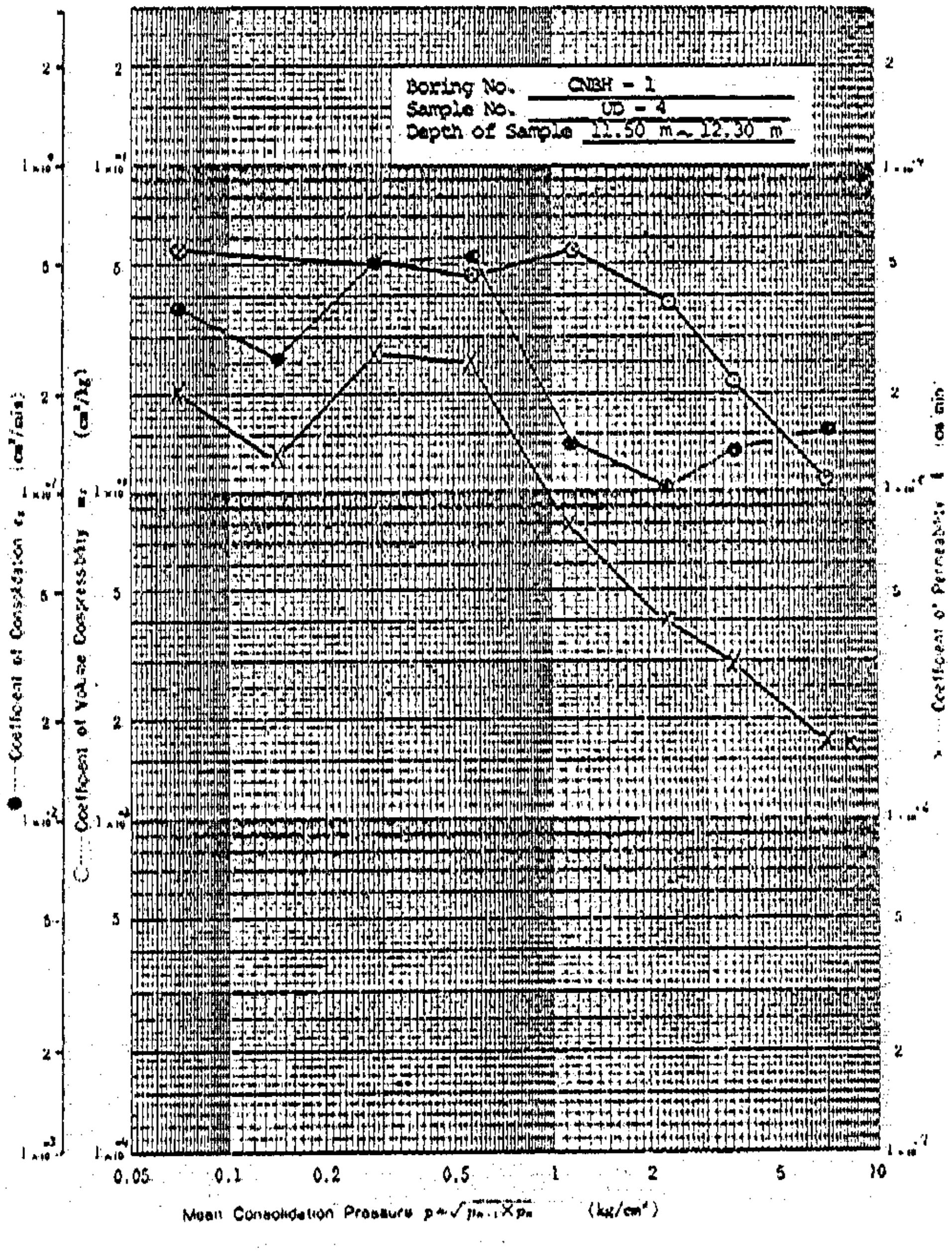
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



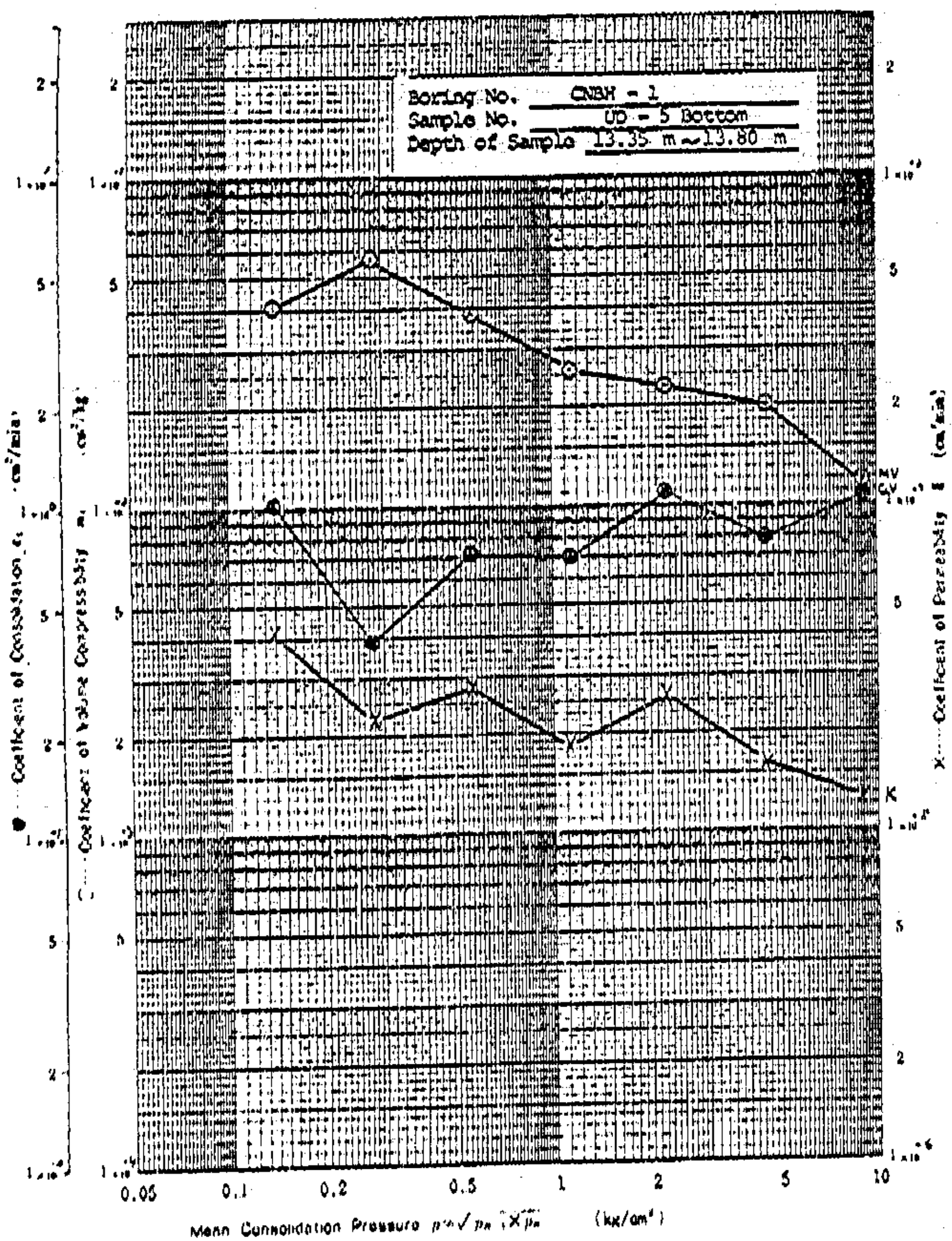
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



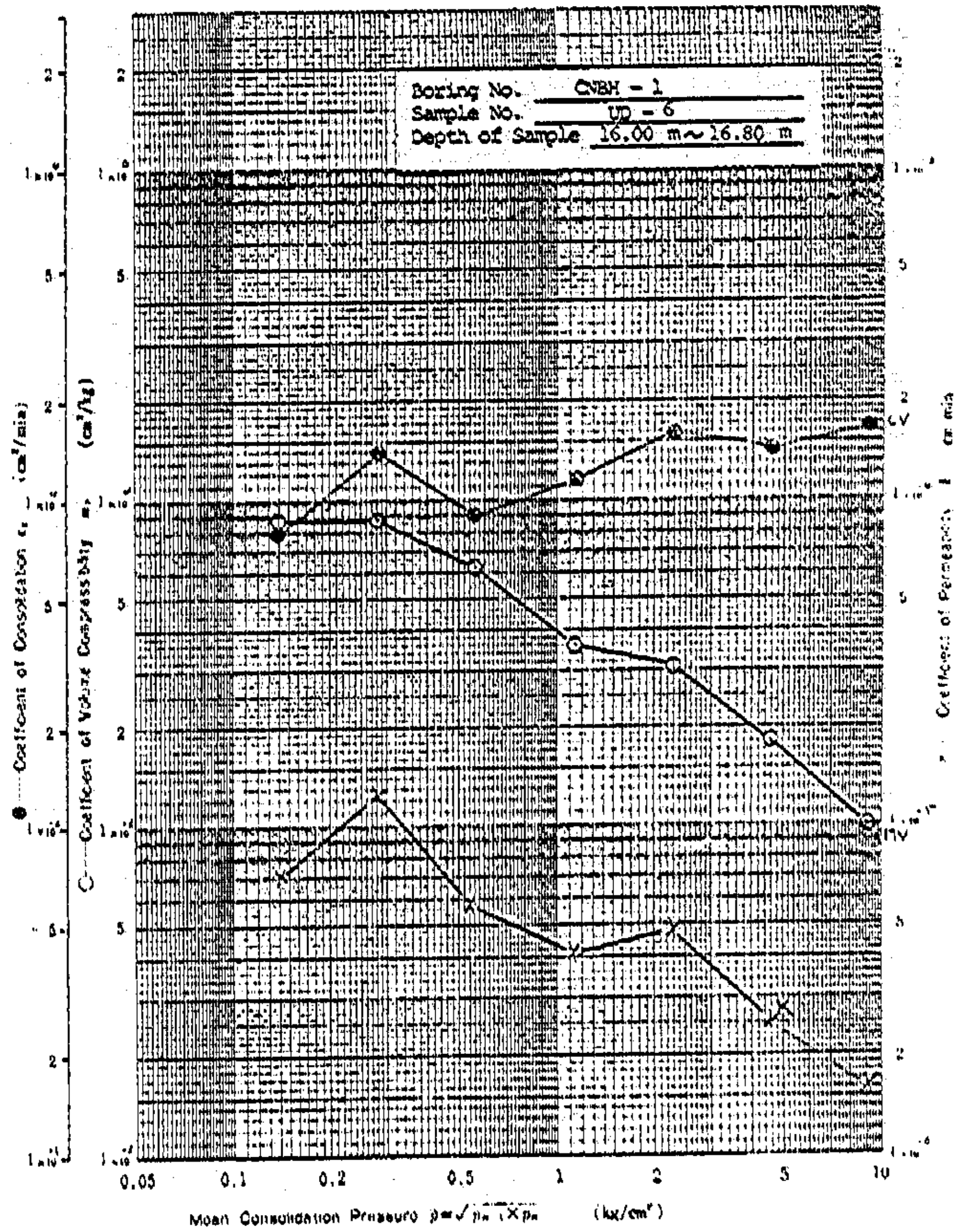
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



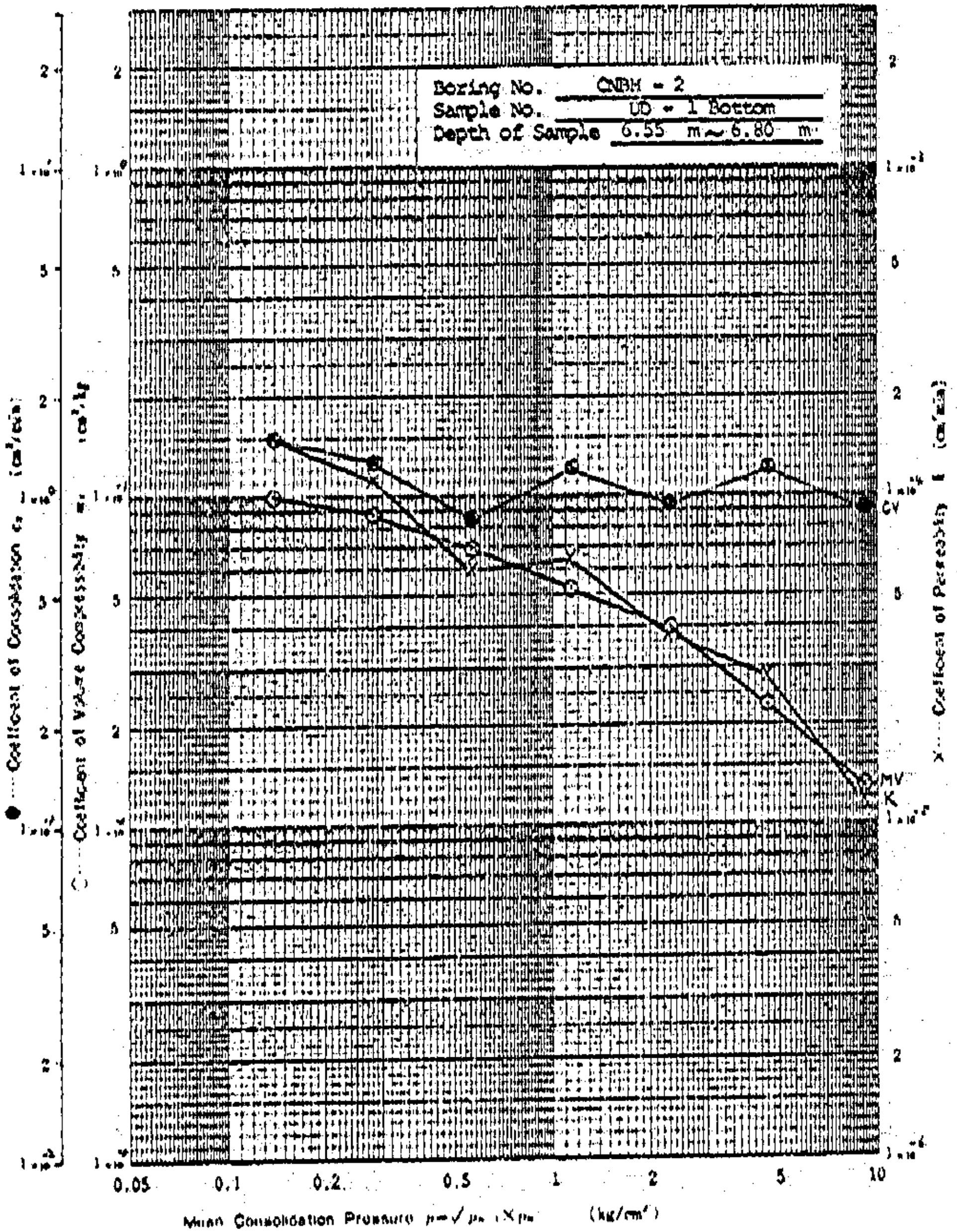
CONSOLIDATION TEST (p̄–Cv, mv, k, curves)



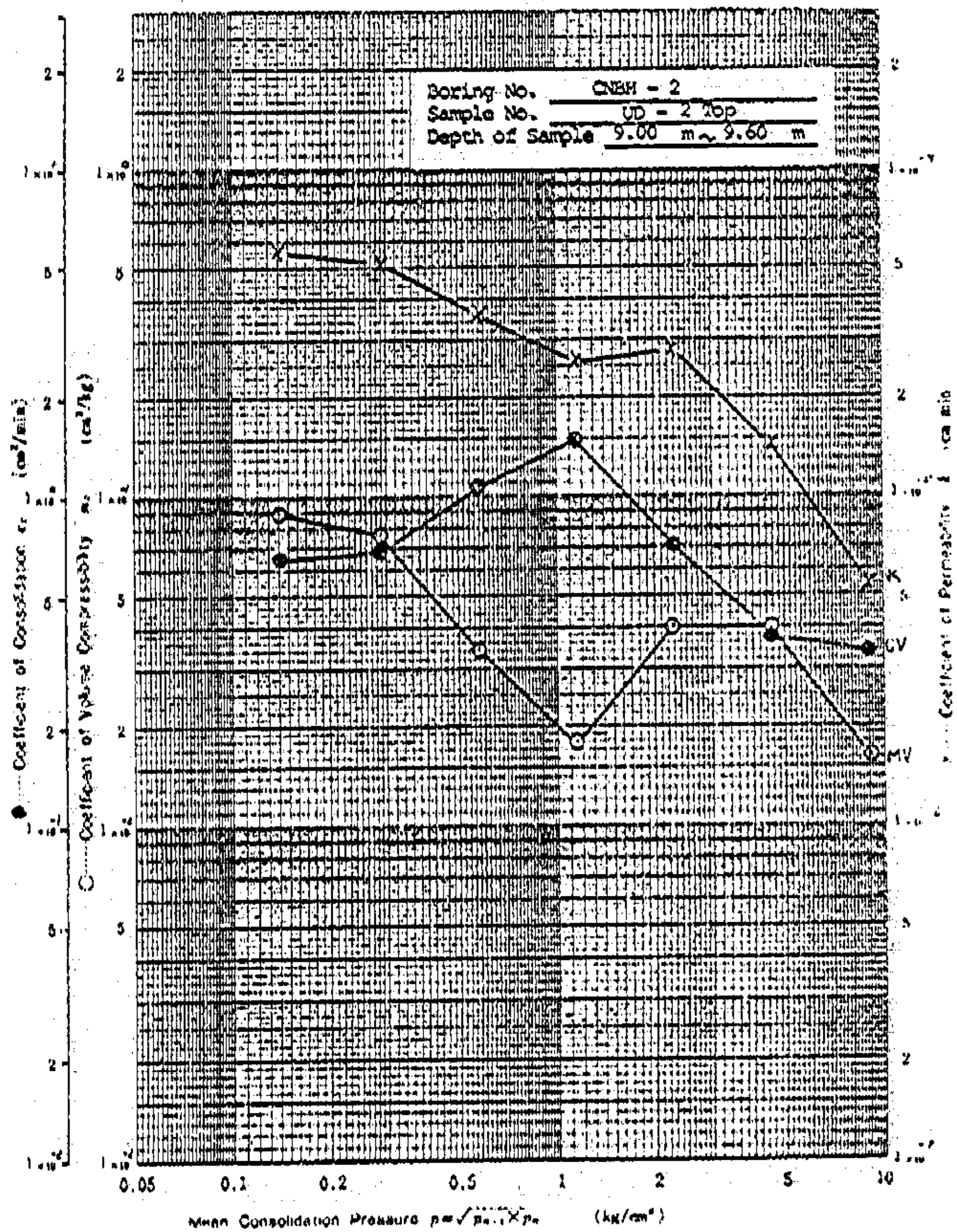
CONSOLIDATION TEST (p̄–Cv, mv, k, curves)



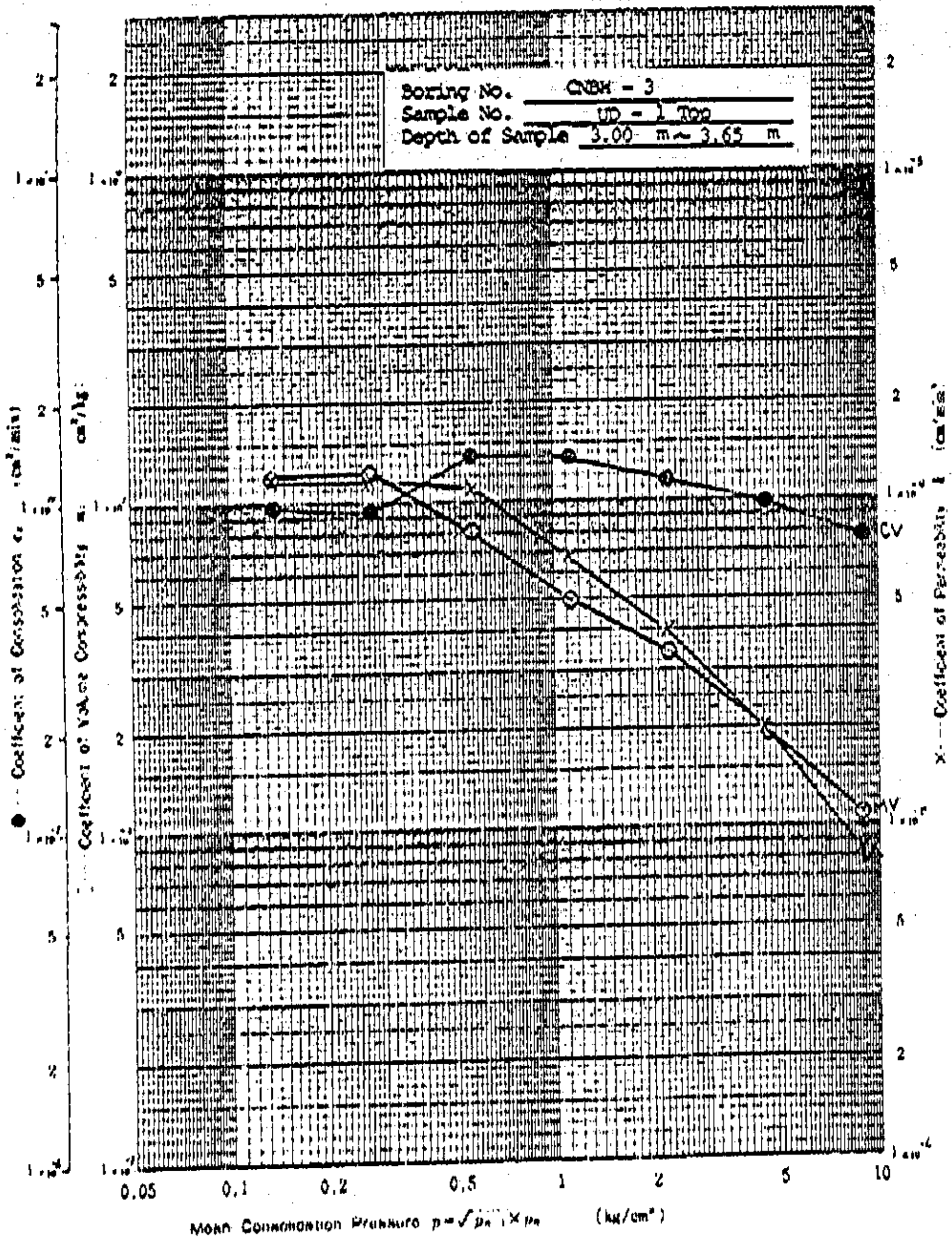
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



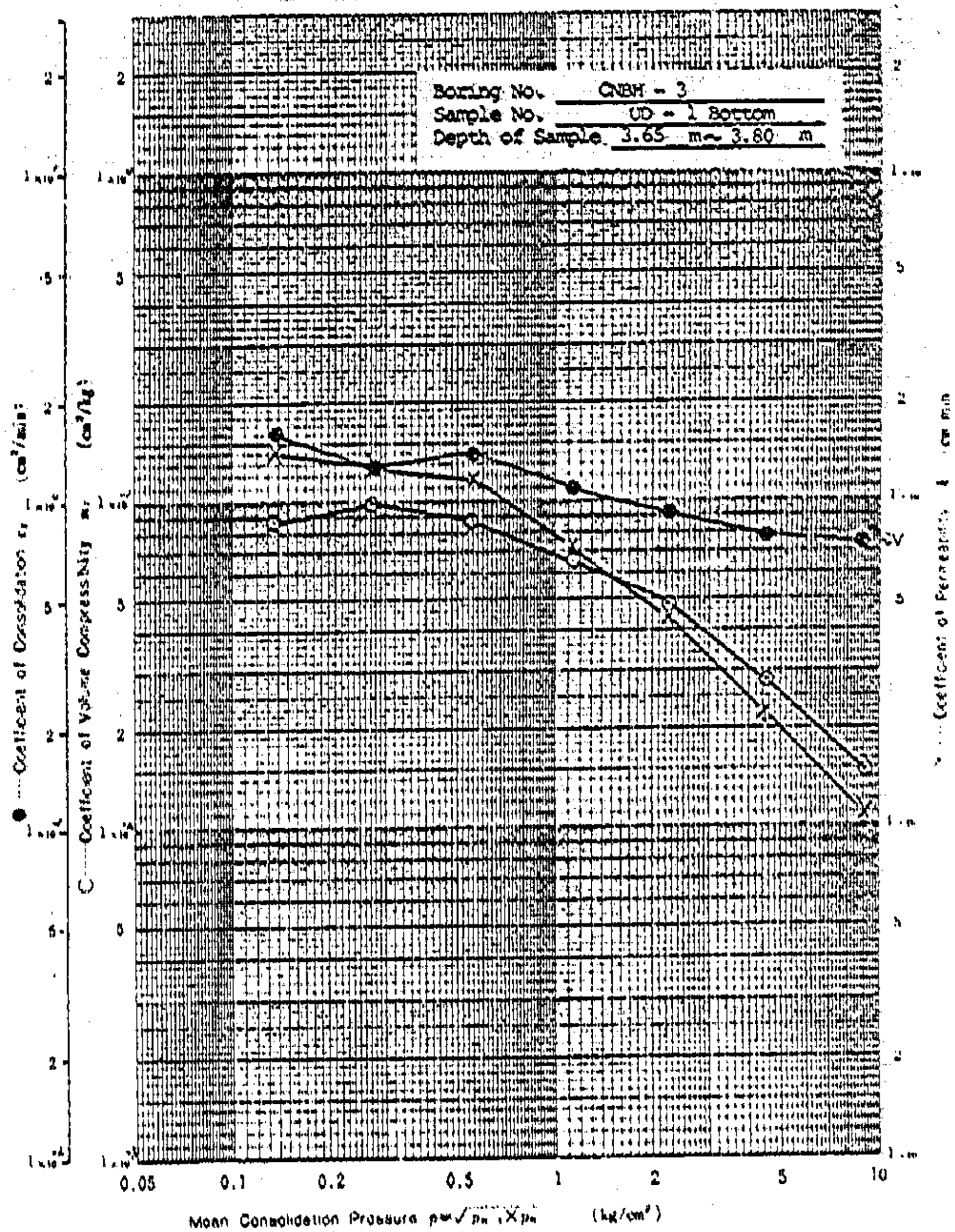
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



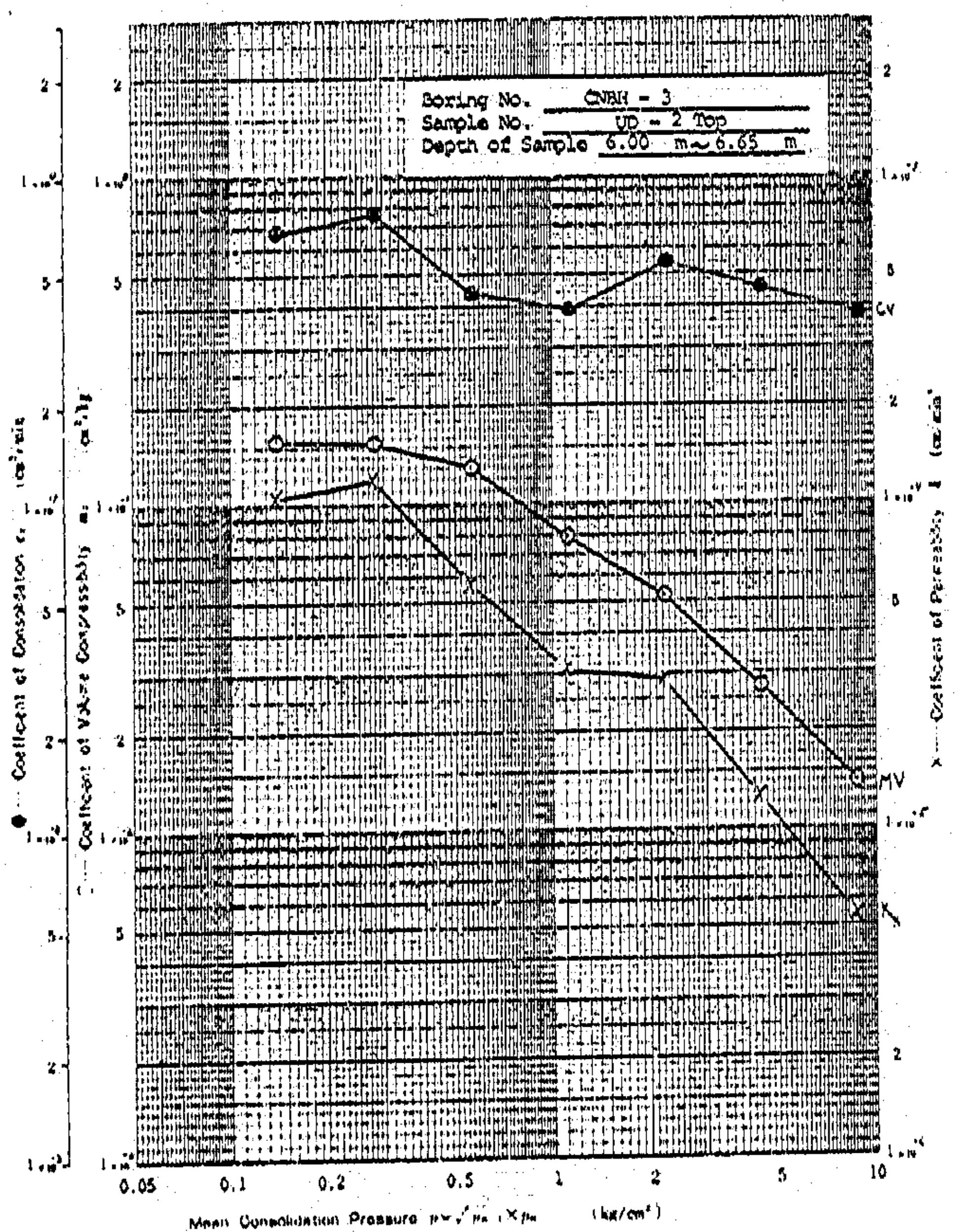
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



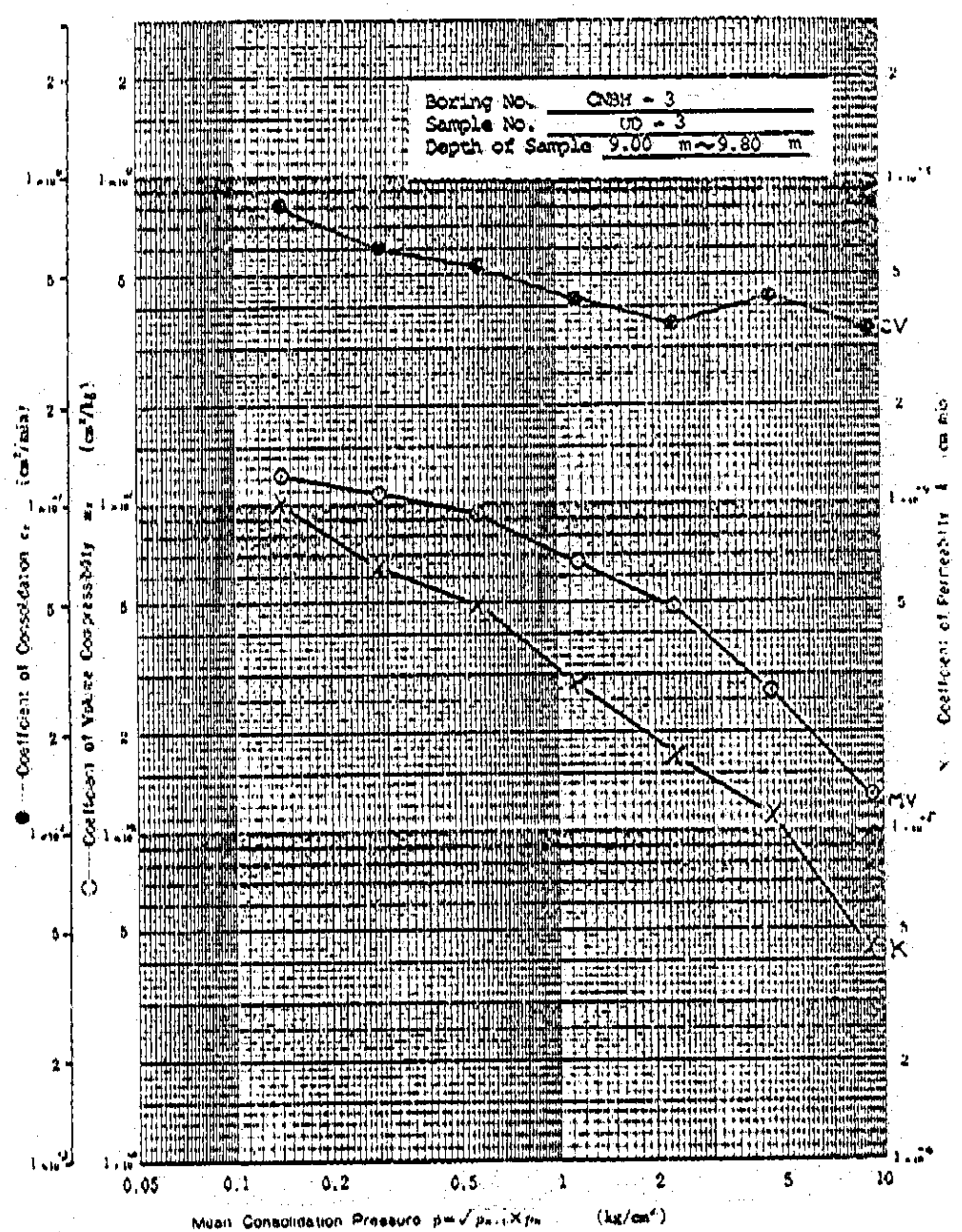
CONSOLIDATION TEST ($\bar{p}$–Cv, mv, k, curves)



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



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



APPENDIX G DETAILED DESCRIPTION OF GROUND IMPROVEMENT METHODS

APPENDIX G

DETAILED DESCRIPTION OF GROUND IMPROVEMENT METHODS

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: Strength — Increase in strength, Increase in coefficient of deformation	Purpose of Ground Improvement: Compressibility — Improvement of compressibility, Into less-compressible material	Purpose of Ground Improvement: Dynamic Characteristic — Improvement of dynamic characteristics, Prevention of liquefaction	Suitable Material
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

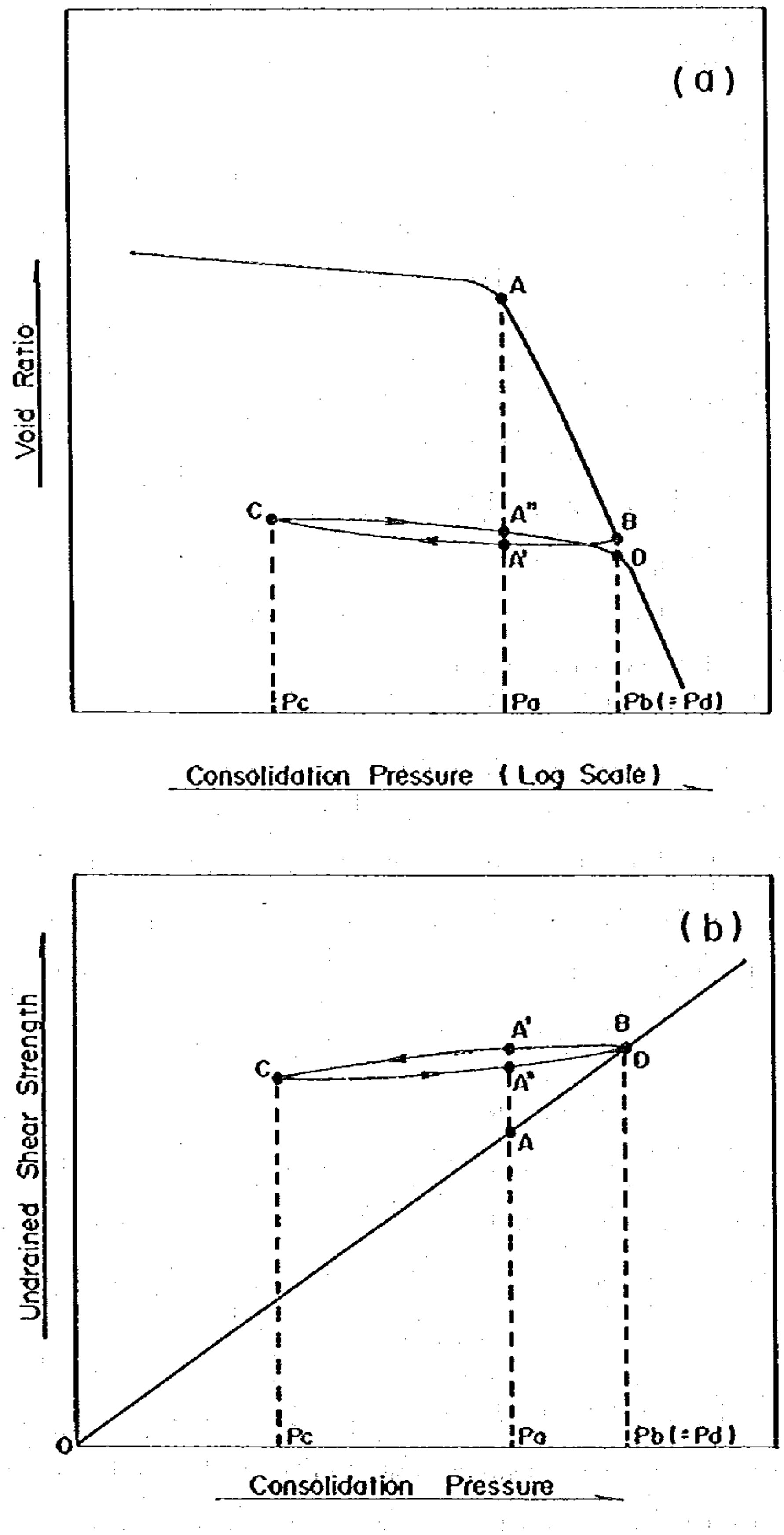


Fig. G-1 Conceptional Drawing of Consolidation Characteristics of Cohesive Soil

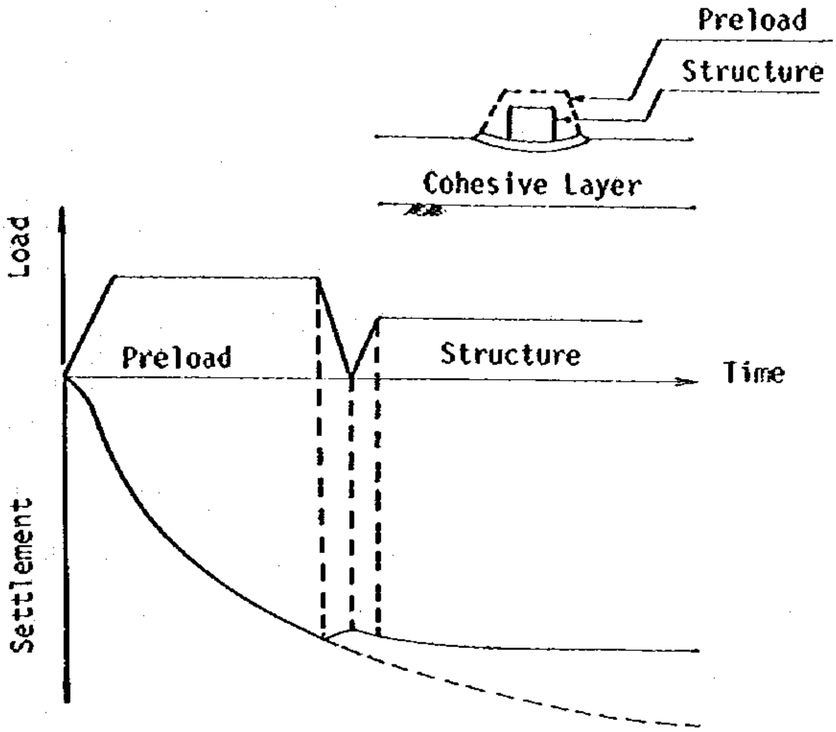


Fig. G-2 Descriptive Drawing of Preloading Method is placed on the proposed construction site before the construction. However, construction cannot begin until the preload period is over and the surcharge load removed.

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 ordinarly 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.

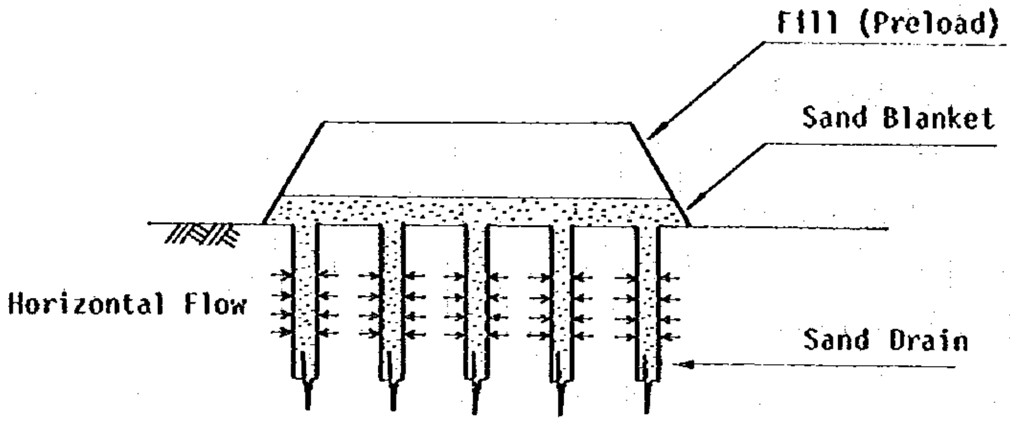


Fig. G-3 Typical Cross Section of Preloading with Sand Drain

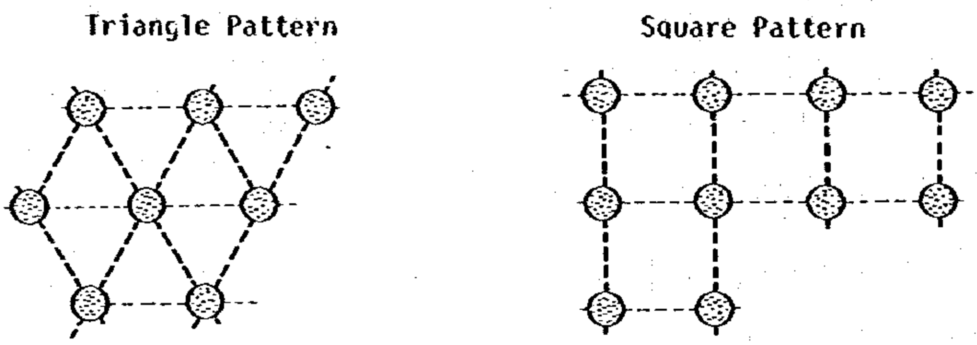


Fig. G-4 Layout of Sand Drain Piles

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.

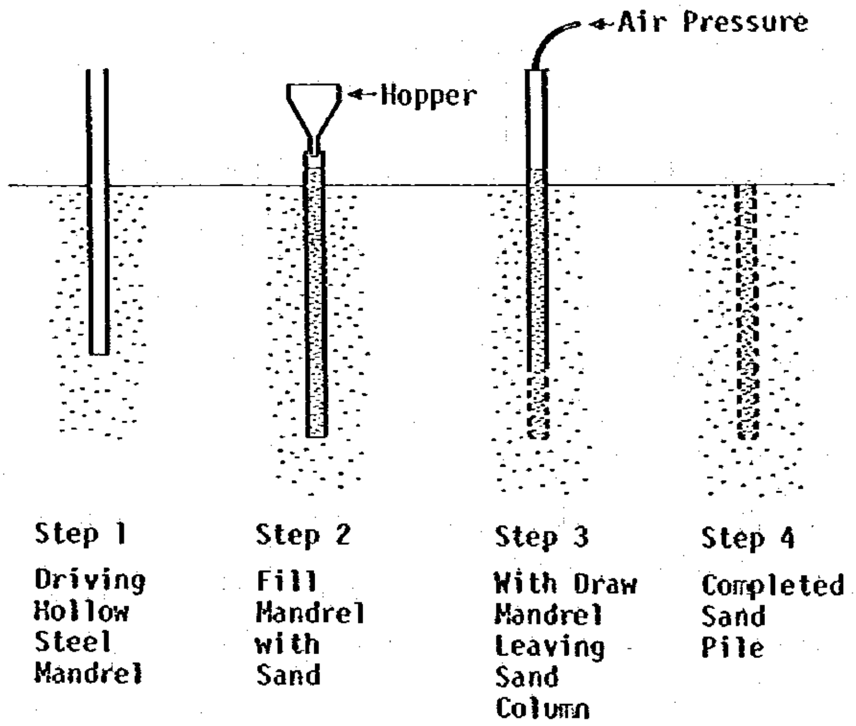


Fig. G-5 Installation of Sand Drain

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

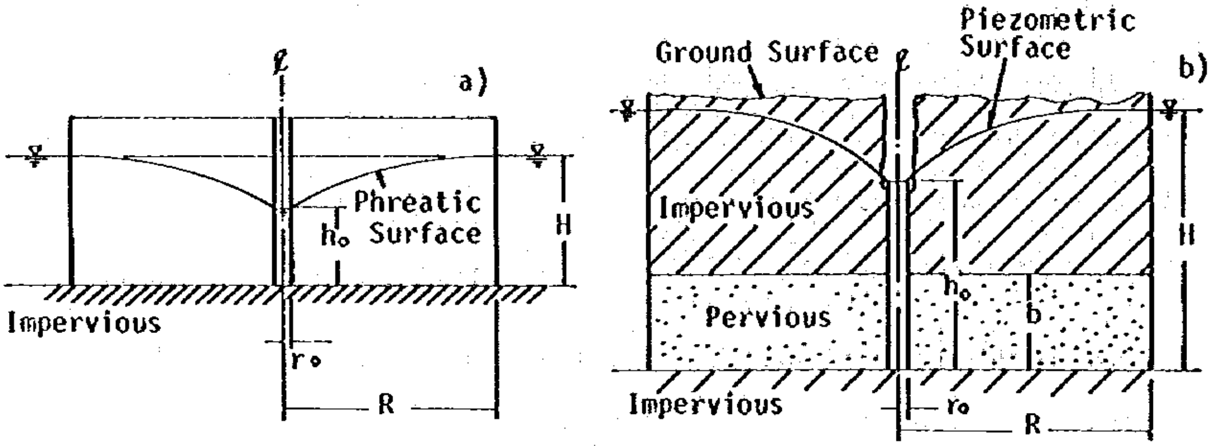


Fig. G-6 Vertical Section a) through Gravity Well,
b) through Artesian Well

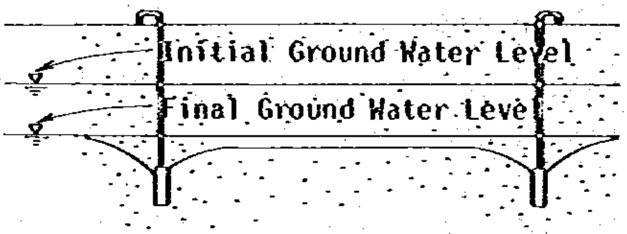


Fig. G-7.a Soil Stabilization

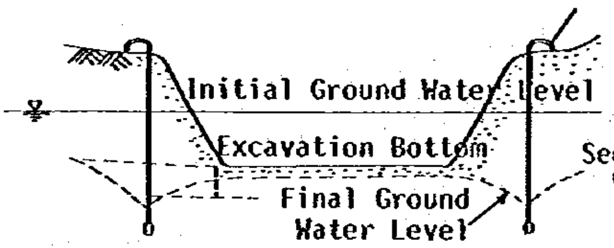


Fig. G-7.b Single-Stage Excavation

Fig. G-7.c Multiple Excavation

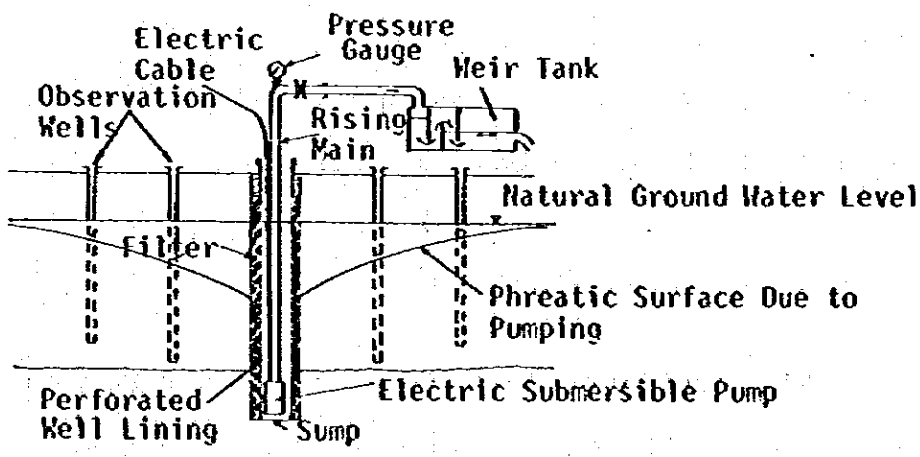


Fig. G-8 Typical Deep Well with Observation Wells for forcible drainage to be achieved not only from sand strata, but also silty strata of coefficient of permeability 10^{-4} to 10^{-5} cm/sec.

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.

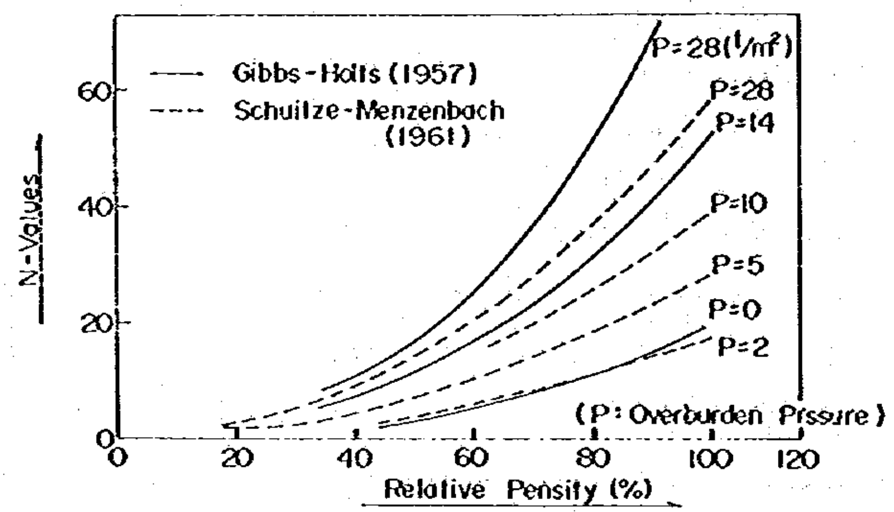


Fig. G-9 Relation between N-Values and Relative Density Dr

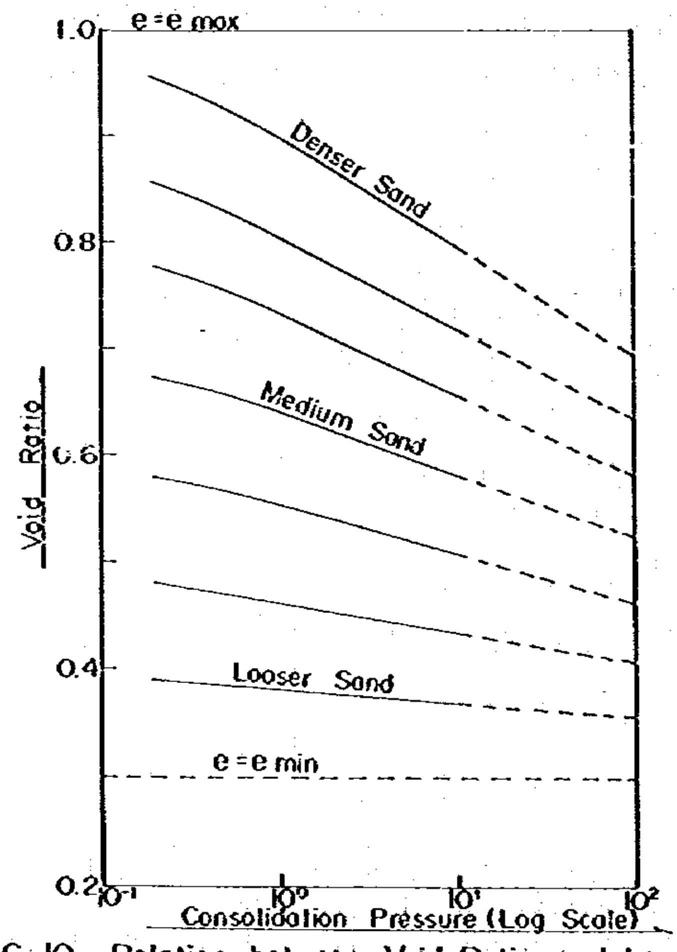


Fig. G-10 Relation between Void Ratio and Log Pressure on Sandy Soils with Various Densities

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.

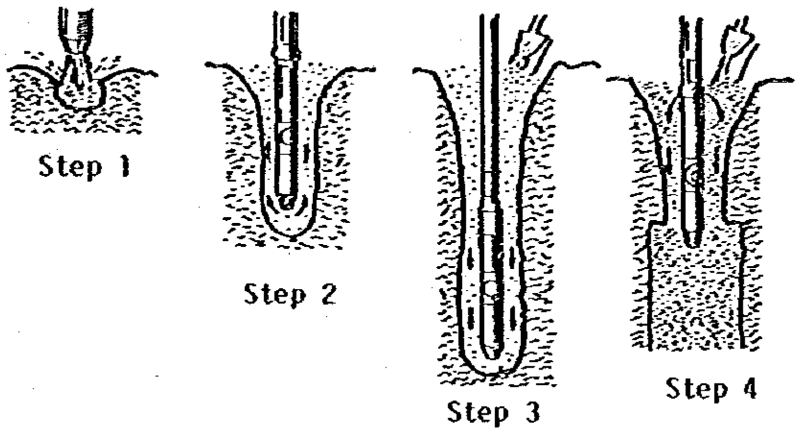


Fig. G-11 Compaction Process by Vibroflotation

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.

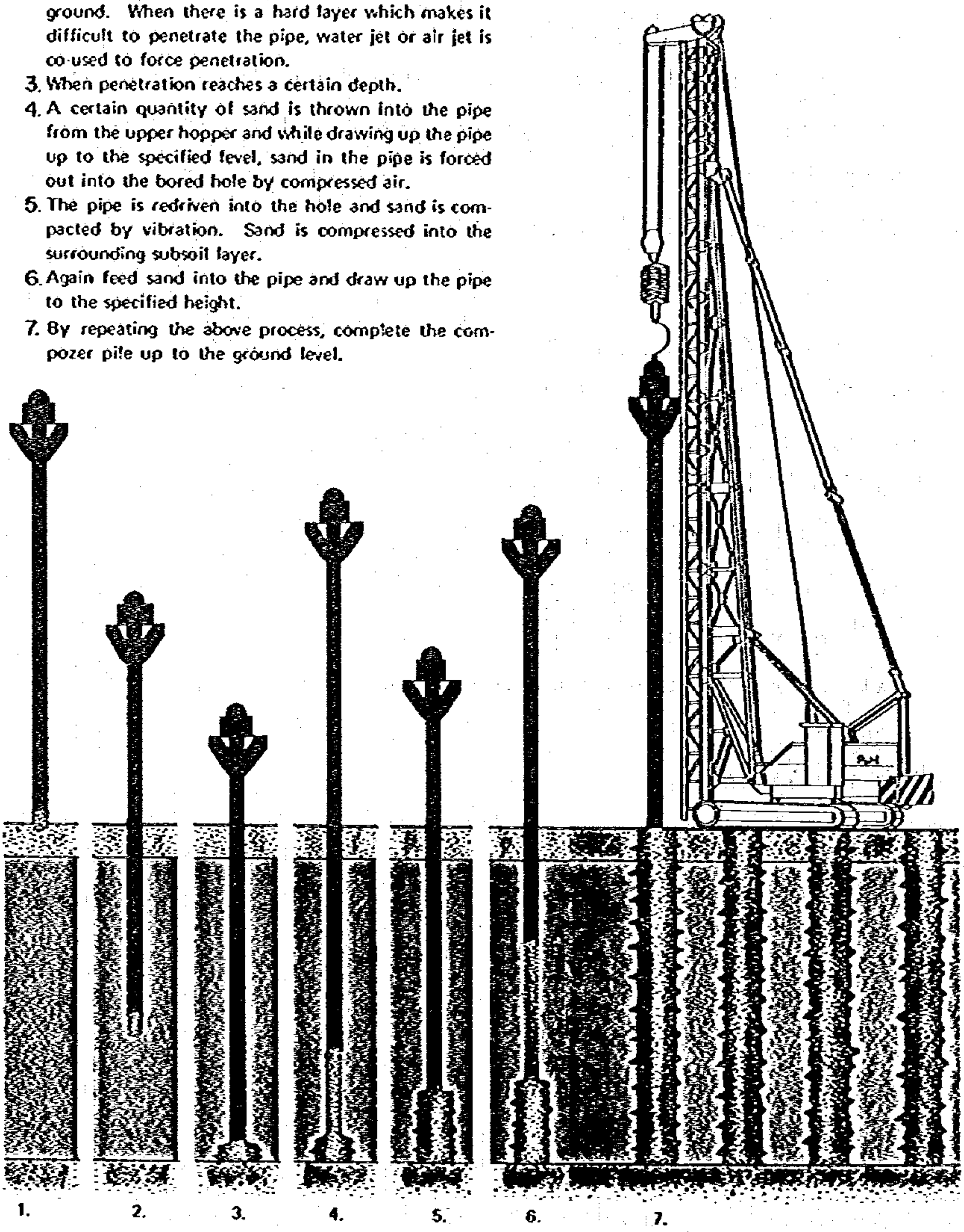
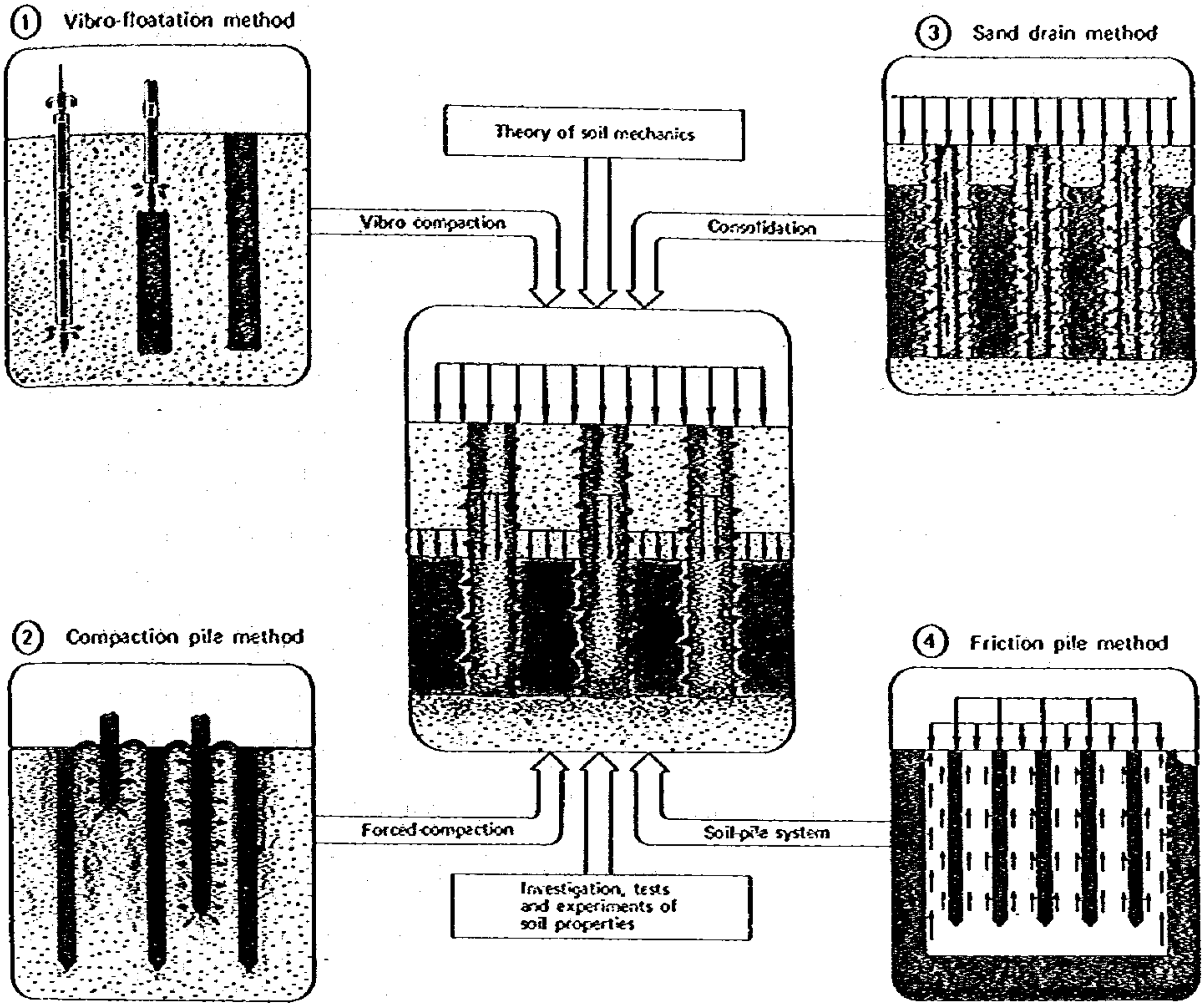


Fig. G-12 Procedure of Sand Compaction Pile Installation



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

The effects of sand compaction piles on soft cohesive soil are that the shearing resistance of the ground is increased by forming a composite ground of cohesive soil and sand piles. This results in a reduction in the amount of consolidation settlement and also in the time necessary for settlement as well.

G.4.4 Dynamic Consolidation

This system employs heavy tamping and was originally used on ballast fills or natural gravelly soils. Its range of applicability has now been extended to alluvial soils and clays. This method is now called Dynamic Consolidation. It involves impacting 5 to 4 ton weights, called pounders, from heights of 20 to 100 ft. (6 to 30 m), according to a predetermined pattern specially evaluated for the particular site. Weights as large as 200 tons are now contemplated for use. Fig. G-14 illustrates execution of Dynamic Consolidation Method. Fig. G-15 shows a typical soil behaviour during dynamic consolidation.

The theory of Dynamic Consolidation followed its application. In granular soils, the high energy impact is believed to cause partial liquefaction of the deposit, thereby allowing the mass to settle into a denser state. However, Dynamic Consolidation of fine-grained soils is not so well understood. Menard and Broise (1975) hypothesized that in clays, the shock waves and high stresses compress the air microbubbles in nearly saturated soils. After repeated impacts, gradual liquefaction occurs accompanied by changes in soil mass permeability due to fissures around the impact points. Finally, the deposit undergoes thixotropic strength recovery, presumably at a higher density.

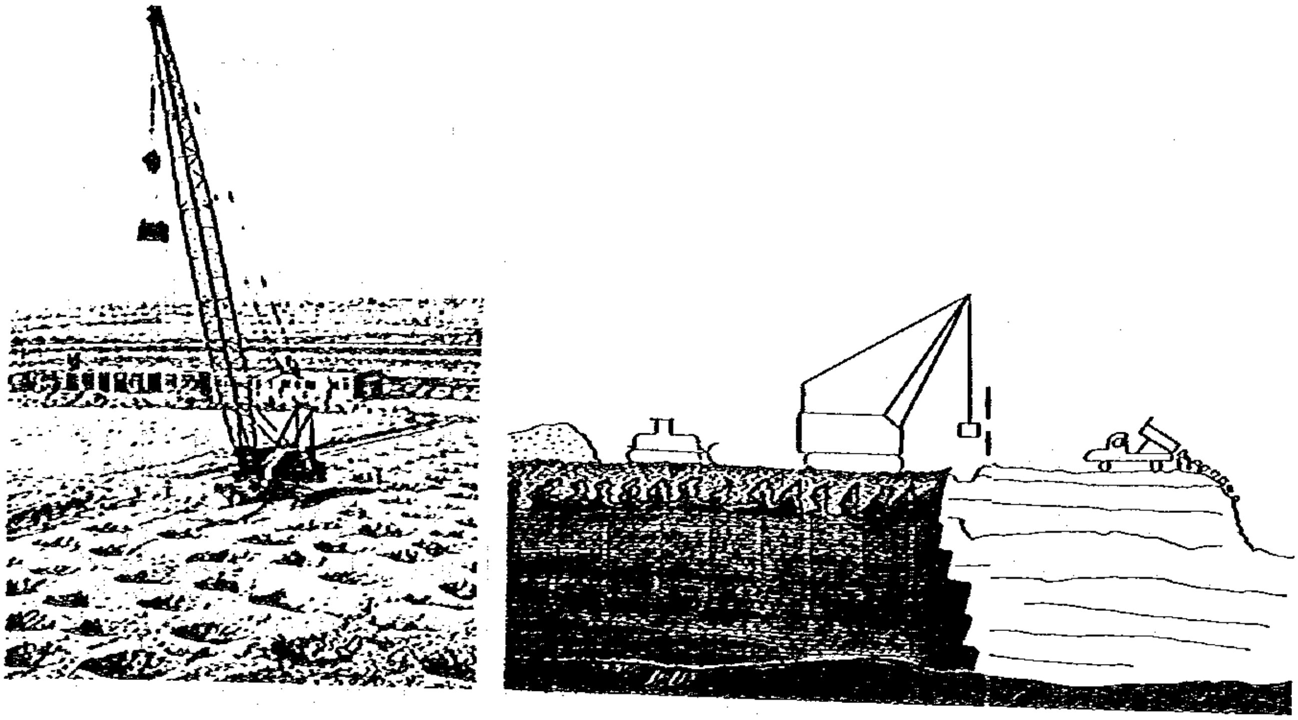
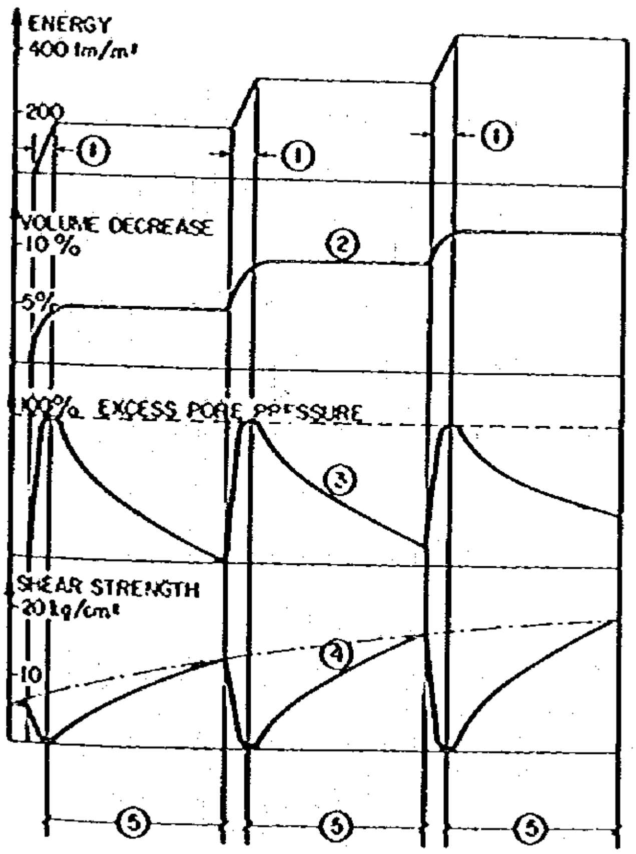


Fig. G-14 Illustration of Dynamic Consolidation Excecution



1) High energy applied at ground surface by impact of falling weight. Energy is applied in three phases in this example.

2) Induced settlement of ground surface typically corresponds to a volume reduction of 5 to 15% in the weak soil being treated.

3) Excess pore pressures increase to the overburden stress during energy application, then dissipate rapidly.

4) Strength of soil first decreases then increases to a higher value.

5) Waiting period of several days to several weeks between phases.

Fig. 4-15 Typical Soil Behaviour During Dynamic Consolidation

To determine the energy requirements for liquefaction, a patented dynamic oedometer has been developed. Dynamic loads are applied to an 11.8inches(30 cm) diameter specimen while allowing static consolidation to occur. The device simultaneously measures pore pressures and horizontal pressures. Although this test does not entirely duplicate field conditions, it has been used successfully to estimate the effect of dynamic consolidation, as well as to optimize the field compaction operation. Other soil investigation methods should also be employed to determine operation methods and parameters, number of phases, and instantaneous settlements. Menard and Broise (1975) recommend boreholes for stratigraphy and in-situ testing with pressurementer, vane, and penetrometer for soil properties. This would be coupled with laboratory testing for moisture contents, grain-size distributions, etc.

The lifting and release of these huge pounders may be from a standard crane of 50 to 120 ton capacity, a tall tri-pod mast or larger specially built cranes. The pounders are dropped numerous times at one location before moving to the next impact position. After the initial pass over the entire site, subsequent passes are accomplished at time intervals based on dissipation rates of the induced pore water pressures. These intervals may be up to three weeks depending upon the particular soil at the site.

Dynamic consolidation is most suitable for projects in rural areas or at construction sites surrounded by a large open area. The very high impact produces vibrations with frequencies on the order of 2 to 12 Hz. Field measurements have shown that at a distance of 98 ft (30 m) away from the point of impact, the wave velocities remain below 5 cm/sec. which can be tolerated by most structures. Surface settlement of 5% to 15% of the total deposit thickness can be achieved.

Environmentally, this method poses very few problems. However, the noise may be objectionable in urban areas. Run-off from induced drainage may carry some fines into surrounding areas. However, this can easily be remedied by proper precautions.

G. 4. 5 Roller Compaction

If the soft layer to be improved is at relatively shallow depth, such as less than 8 ft (2.4 m), roller compaction method is useful and economical.

Compaction of ordinary controlled fills constructed using light surface rollers extends about a foot below the roller. When low density granular soils extend to depths of five to eight feet, heavy rollers are available which can successfully achieve compaction from the surface.

Pneumatic Tire Rollers; Much research and development since the 1950's has resulted in greatly improved rollers today. Development of this equipment was greatly accelerated by the U.S. Army Corps of Engineers because of the rapid escalation of aircraft wheel loads and tire pressures. Fifty and 100 ton pneumatic tire rollers have been developed and used extensively on air-fields, dams and highways.

Vibratory Rollers; Research and experimentation with vibratory compactors, initiated in the late fifties led to the development of effective steel wheel vibratory rollers and various types of vibratory sleds in the sixties. Vibratory frequency was found to be critical. The most effective compaction may be obtained using the least energy if resonance can be achieved.

For a short time steel wheel vibratory rollers as large as about 7,000 pounds (3 tons) were considered as "heavies". They proved so effective in granular soils and reached depths previously believed unreachable, that the size and weight of these rollers accelerated rapidly soon reaching weights as high as 15 tons and delivering a dynamic force of 30 tons or more. Effective compaction to depths as deep as 8 ft (2.4 m) have been reported. However, with the greatly increased vibrational forces the upper one to two feet (0.5 m) of soil became loosened rather than densified.

G. 5 Chemical Stabilization

Because of the high cost and the absence of precise methods to predict and evaluate the results, the use of a chemical stabilization method is limited to corrective measures for post-construction problems and small, isolated soft or loose deposits. This method is performed by injecting fine soils and/or chemical grouting materials, such as cement, lime, and other chemicals or mixtures of these into the problem deposit.

Mixtures of fine soil, portland cement, and water; lime and water; sodium silicate; calcium chloride; polymers, and resins are the most universally used grouting materials. Various surfactants and catalysts, such as calcium lignosulfonate, hydroxylated carboxilic acid, and carbon dioxide gas, are also used to facilitate the dispersion and solidification of grouts in soils and to promote in-situ reaction.

Compaction grouting and hydraulic fracturing (Ruppel, 1970) can be used to compensate for differential settlement of structures or stabilize soft foundations. A commonly-used mixture for compaction grouting is portland cement and fine sand or silt. Mixtures of well-graded sand and silt are also used. Twelve to fifteen percent by weight of portland cement is mixed with the soil and water into a highly viscous grout that is pumped into bored holes at pressures up to 500 psi (352 ton/m^2).

Penetration grouting (penetration of the grout into soil pores) cannot be used to stabilize soils finer than sands with permeabilities of 5 x 10^{-6} m/sec or higher (Mitchell, 1970).

A groutability ratio of 25 or higher is necessary for consistent penetration. The groutability ratio "R" is defined as the ratio of the 15% size of the formation to be grouted to the 85% size of the grout, or D_{15} formation/D_{85} grout.

Soil-cement grouts can be used for the strengthening of loose, coarse sands or the prevention of their liquification under cyclic loading. Fine sand containing 15 to 35 percent by weight of portland cement, mixed with a minimum amount of water to allow the mixture to be pumped, has been used for this purpose (Peignaud, 1971).

Soil tests should be performed to determine the presence of any substances that will counteract the hardening of the grout. This problem may occur particularly near the banks of polluted streams and at industrial plant sites with polluted ground water.

The strength developed by the cement-grouted foundation varies with the grouting techniques as well as with variations in the water-cement ratios. Strengths as high as 70 tons/ft^2 (683 ton/cm^2) can be obtained. Bentonite used with portland cement (Caron, 1972) has produced strengths up to 75 tons/ft^2 (734 ton/cm^2) in sands.

Lime-water slurry has been used to stabilize soft clays as well as stratified alluvial deposits (Karol, 1960). The lime may react with clays, in-situ, after injection. Lime and a wetting agent are mixed with water and injected under high pressure (about 100 psi, or 70 t/m^2). This method has been used for slope stabilization and the treatment of expansive soft soils. The strength developed by the in-situ reaction varies widely. Fig. G-16 illustrates an advanced technique of chemical stabilization of soft clay which is called "CMC Method". Examples of application of chemical stabilization are shown in Fig. G-17.

A large variety of chemical grouts, many of which are proprietary, are used to stabilize fine granular soils. These grouts are expensive; however, they are easier to inject because they do not contain particulate matter. Thus they can penetrate smaller voids than the cement or soil grouts.

Upon hardening, some chemical grouts produce high strengths. Most of them are either monomers that are polymerized in place or phenoplasts. Polymerized chemicals form a gel with a strong-bonded, cross-linked molecular structure. The strength and gel time can be controlled by the amount of catalysts and the temperature. Phenoplasts, upon geling, form polycondensed non-linear structures of high strength.

The permanence of chemical grouts, as well as their extent of penetration, remains somewhat indeterminate. Even though some formulae have been developed to estimate the radial penetration distance, final design decisions are often based on empirical data and field pumping tests.

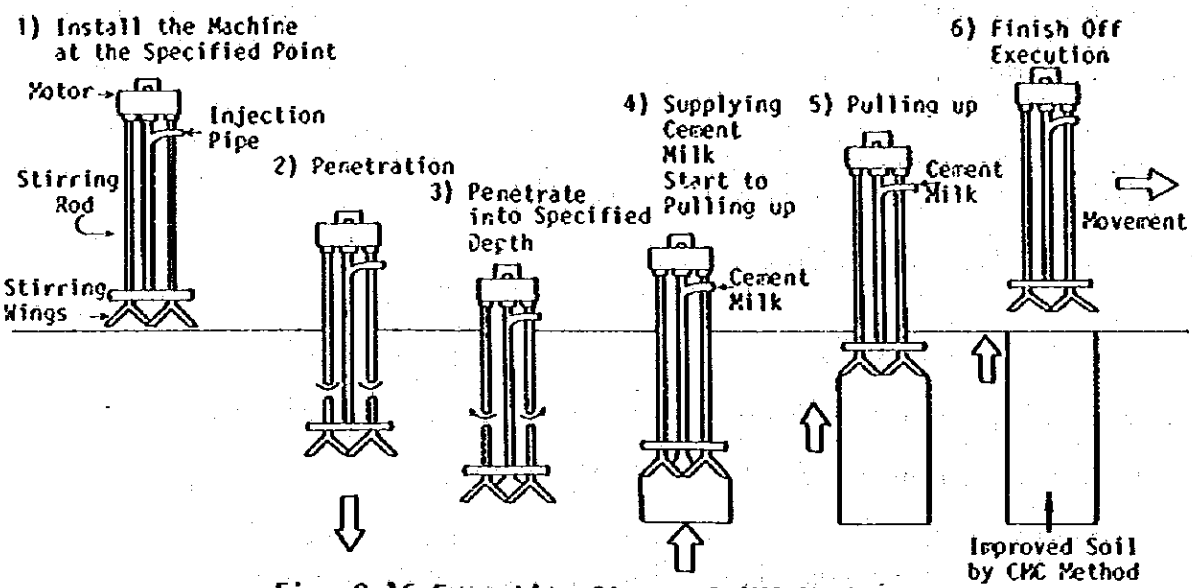
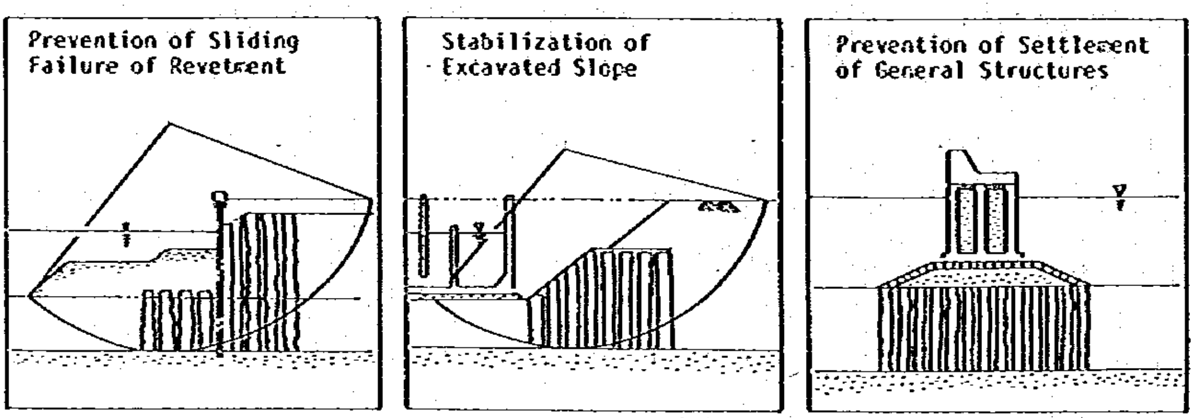


Fig. G-16 Execution Steps of CMC Method



(1) Improvement of soft soils at depth
* Prevention of sliding failure of Revetment
* Improvement of Foundation Soils for Embankment
(2) Prevention of Settlement of General Structures
(3) Stabilization for Excavation
(4) Improvement of Soft Ground at the Foot of Bridges
(5) Intensifying Lateral Resistance of Piles

Fig. G-17 Application of Chemical Stabilization Methods

G. 6 Excavation and Replacement

Where materials such as soft clays are underlain by soil deposits of suitable stability at shallow depths (less than 20 ft or 6m), it is often economical to remove the unsuitable material by excavation and replace it either with special borrow material or, after drying or treatment, replace the original material. The type of borrow material used and the methods of its placement depend largely on the position of the water table.

If the water table is below the excavation line, either fine or coarse grained soils can be used as replacement material, with densification by mechanical means and admixture stabilization with lime, portland cement or other chemical agents, if necessary. The mixing of stabilizing agents may be accomplished either on or off the site. If the excavation site is confined, the mixing of the soil and stabilizer can be done either at a central mixing plant or at a location adjacent to the excavation. The mixture is then placed into the excavation and compacted in layers.

If the water table is above the bottom of the excavation line, and is not lowered by a dewatering system, the excavated material can only be replaced with granular material such as sand, slag, gravel, etc. Additional mechanical compaction or stabilization can be accomplished with vibratory or dynamic methods (Vibroflotation, Vibro-Rod, Dynamic Consolidation, etc.).

Many soft soil deposits are located under water and excavation under such conditions present serious problems to both the designer and the builder. Both the excavation and the placement of the fill material require special consideration of the material properties and careful inspection during excavation operations (Johnson et al., 1971).

Underwater excavation is performed with draglines and dredges. The foreslopes of the trench should be flat designed to be stable to prevent refilling of the excavation trench by slope failures occurring after the excavation of a section. If such failures occur and remain undetected, soft materials may be trapped under the select fill.

In some cases the water level in the excavated trench should be maintained at the phreatic surface, because lowering of the water table would otherwise cause subsidence of the surrounding area (Transportation Research Board, 1975).

Environmental aspects are of utmost importance in the planning and execution of underwater excavation and placement and should be considered in the early planning stages. Disposal of the excavated material without affecting the sheet water flow becomes particularly important in marshlands and flat lowlands subject to tidal inundation. The fate of pollutants dredged from river and harbor bottoms must be considered. In some cases treating the dredged spoil with quick or hydrated lime prior to wasting is an effective means for preventing contamination of adjacent waters with silt and clay fines. Use of the excavated material to create water basins and wildlife refuge islands may, in many cases, be an acceptable alternative.

Granular materials suitable for placement and densification under water are pumped sand, clam shell, gravel, or slag. For large fill projects, sand has been pumped as far as fifteen miles under 120 psi (84.4 ton/m^2) pressure (Starring, 1971).

When the removal of soft cohesive soils by excavation is not feasible or economical, the displacement method can be used to remove the unsuitable material.

Removal by displacement is accomplished by placing sufficient embankment material to cause failures in the underlying soft layers in the direction of least resistance, which normally is ahead of the advancing embankment. These failures create a mudwave that can then be excavated. The rate of any required excavation of the mudwave ahead of the embankment should be about the same as the rate of placement of the embankment material. If this rule is not followed, there is the danger of trapping pockets of soft soils under the embankment.

Lateral movement of the soft soil beyond the limits of construction may sometimes occur resulting in the formation of lateral mudwaves with consequential damage to adjacent structures. However, if the lateral mudwaves present no problems, they act as berms and greatly increase the stability of the embankment. Displacement methods have been successfully used for soft soil deposits up to 65 feet (20m) thick (Weber, 1962).

Soft clays and highly organic soils respond well to displacement. However, if the organic material is composed of large fibers, the soft material tends to become reinforced and will resist the required forward movement. This, in turn, results in the encapsulation of soft materials under the fill, which may cause differential settlements.

APPENDIX H DETAILED DATA FOR COST SUTDY

APPENDIX H

DETAILED DATA FOR COST STUDY

Page

H. DETAILED DATA FOR COST STUDY

Details of the costs for the following items are presented in this section:

1) Preloading and Surface Compaction

2) Ground Improvement other than Preloading

3) Pile Foundation

Subsection H.1 gives details for Items 1) and 2), and Subsection H.2 provides details of 1), 2) and 3).

Summary of Item 1) is shown on Table H-1 (page H-3) and that of Items 2) and 3) is shown on Table H-7 (page H-31). The contents of Tables H-1 and H-7 are used to prepare Tables 6-3 and 8-3 in Main Text.

H.1 Unit Cost of Ground Improvement for Soft Material

(Reference to Table 6-3 in Main Text)

Unit: M$

H.1.1 Basis for Cost Estimation

1) Machine and equipment are assumed to be available in Kuala Lumpur.

2) Construction Area : 40,000 m^2

3) Thickness of Improvement: 5 m (for type B, C ground)

   10 m (for type D ground)

4) Total Installation : 100,000 m

5) Each method will be used for the suitable ground condition and purpose.

H.1.2 Cost Estimation

(1) Preloading

a. (H = 3 m, for 3 x A (bldg))

Type B ground: M$27.6/$m^2$

Type C ground: M$43.5/$m^2$

Type D ground: M$46.5/$m^2$

b. (H = 1.5 m, for 3 x A (bldg))

Type B ground: M$17.4/$m^2$

Type C ground: M$31.8/$m^2$

Type D ground: M$33.3/$m^2$

Details of the cost of preloading are summarized in Table H-1 (Detail explanation are given in Sub-section H.2 below).

Table H-1 Unit Cost of Preloading and Surface Compaction

Ground Condition	Structure	① Thickness of Sand Blanket (m)	② Thickness of Preload/Fill (m)	③ Amount of Settlement (m)	④ Grubbing, Clearing, Spreading & Surface Compaction (M$/m²)	Cost for Loss of Material (M$/m²) ⑤ Sand Blanket ① x M$9.0 (M$/m²)	Preload Earth/Fill ⑥ Due to Settlement ③ x M$6.0 (M$/m²)	Other Loss ⑦ Thickness (m)	Other Loss ⑧-⑦ x M$6.0 (M$/m²)	Cost for Transferring Preload Earth to Next Site ⑨ Thickness (m)	⑩ = ⑨ x M$2.0 (M$/m²)	⑪ Total (M$/m²)	⑫ Cost for 3 x A(bldg) ⑪ x 3
Type A	Low-, Medium- & High-Rise	0	0	0	1.0	-	-	-	-	-	-	1.0	3.0
Type B	Low-Rise	0	1.5	0.15	1.0	0	0.9	0.2	1.2	1.35	2.7	5.8	17.4
	Medium- & High-Rise	0	3.0	0.25	1.0	0	1.5	0.2	1.2	2.75	5.5	9.2	27.6
Type C	Low-Rise	0.5	1.5	0.70	1.0	4.5	1.2	0.2	1.2	1.35	2.7	10.6	31.8
	Medium- & High-Rise	0.5	3.0	0.95	1.0	4.5	2.7	0.2	1.2	2.55	5.1	14.5	43.5
Type D	Low-Rise	0.5	1.5	0.85	1.0	4.5	2.1	0.2	1.2	1.15	2.3	11.1	33.3
	Medium- & High-Rise	0.5	3.0	1.20	1.0	4.5	4.2	0.2	1.2	2.3	4.6	15.5	46.5
Type E	Low-Rise	1.0	5.5	0.80	1.0	9.0	4.8	0.2 + 4.0	25.2	0.5	1.0	41.0	123.0
	Medium- & High-Rise	1.0	6.0	0.85	1.0	9.0	5.1	0.2 + 4.0	25.2	0.95	1.9	42.2	126.6

(2) Wellpoint and Deep Well

a. Wellpoint

- ° Interval of header pipe = 30 m
- ° Interval of wellpoint = 2 m

Therefore, total length of header pipe is 7 x 200 m

= 1,400 m (700 points)

M$388.98/m·month x 1,400 m ÷ 40,000m^2 = M$13.61/m^2/month

Period of operation

U%	50	80
T	0.196	0.567
t	170 days 5.7 months	492 days 16 months

for t_{50}

13.61 x 5.7 = 77.58 ≒ M$ 78/m^2

for t_{80}

13.61 x 16 = 217.76 ≒ M$ 218/m^2

b. Deep Well

i) Assumed ground condition

1) Clayey layer (5 m) is sandwiched by 5 m thick sand layers.

2) Coefficient of permeability of sand layer

$k = 2 \times 10^{-2}$ cm/sec.

3) Ground water will be lowered at least 10 m at any location.

4) Installed at grid points of 30 m x 30 m

(36 wells)/40,000 m^2

5) Diameter of well: 1 m = 2r

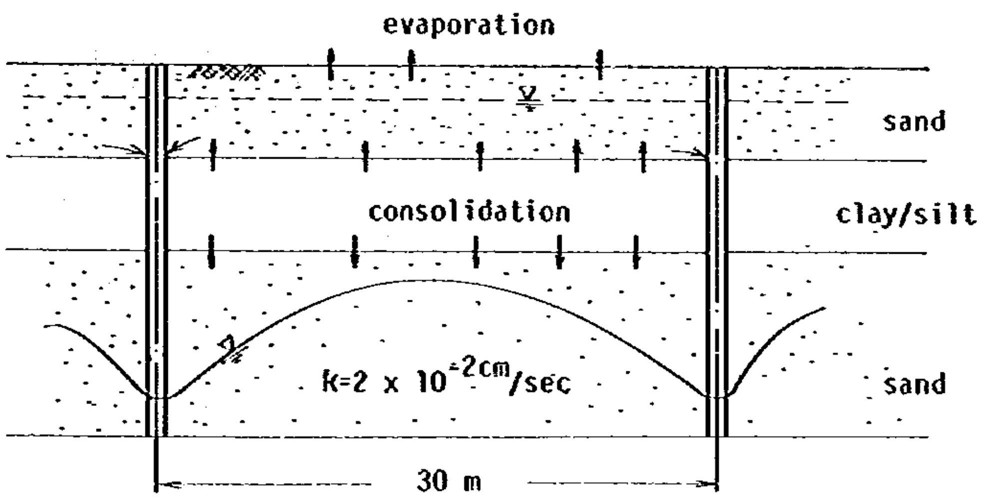
6) Depth of well : 20 m

7) Affective diameter of area: 500 m = R

8) Remaining height of water in wells: 2 m = h

9) Period of operation: t_{50} = 5.7 months

t_{80} = 16 months



ii) Quantity of pumping (Q)

$$k(H^2 - hj^2) = \sum_{i=1}^{n} \frac{Q}{\pi} \ln \frac{R}{r_i} - \sum_{i=1}^{n} \frac{Qi}{\pi} \ln \frac{r_{ij}}{r_i}$$

$Q = 0.3\ m^3/min$

where; $r_1 = r_2 = \cdots = r_n,\ h_1 = h_2 = \cdots = h_n,$

$Q_1 = Q_2 = \cdots = Q_n$, and

$$r_{ij} = \frac{\sqrt{2} + 2}{2} \times 30 = 51.21\ m$$

iii) Capacity of Pump and Diesel Generator

$$P_o = \frac{Q \times (2.0 \times 1.3)}{4.5 \times (1.5 \sim 2)} = 3.57\ HP\ \ (max.)$$

* Submerged pump TS-6C 65 mm 3.7 kW

0.36∿0.225 m^3/min. 49.5 m ∿ 28 m

* Diesel Generator

2.7 x 36 = 133.2 kW

SDG150S 125/150 kW 200/400 V

361/180 A

iv) Timing of Operation

The operation is assumed to be continuous.

v) In Case of 50% Consolidation of Clay Layer

1) Excavation and Installation

M$1,220/m x 720 m = 878,400

2) Pump

M$663/pump x 36 x 5.7 months = M$136,048

3) Generator

M$6,000/month x 1 x 5.7 = 34,200

4) Miscellaneous expenses M$6,000/month; Operation M$2,000/month; Fuel, oil, etc. M$3,000/month

$$\left.\begin{array}{ll}\text{Miscellaneous expenses} & \text{M\$6,000/month}\\ \text{Operation} & \text{M\$2,000/month}\\ \text{Fuel, oil, etc.} & \text{M\$3,000/month}\end{array}\right\}\ \text{M\$11,000}$$

11,000 x 5.7 = 62,700

For 40,000 m^2 ... Total M$1,111,348

M$28/$m^2$

vi) In Case of 80% Consolidation of Clay Layer

1) Same to t_{50} = M$878,400

2) 663 x 36 x 16 = M$381,888

3) 6,000 x 1 x 16 = M$96,000

4) 11,000 x 16 = M$176,000

Total M$1,532,288 for 40,000 m^2

M$39/$m^2$

Table H-2 Cost of Ground Improvement of Soft Material (Unit Cost per 1 m)

(Refer Table 6-3)

Description / Method	Lease of Machine	Operation	Transportation	Installation	Miscellaneous Expenses	Material	Total	Notes
Preloading	-	-	-	-	-	-	-	
Sand Drain (M$/m)	6.58	3.57	0.86	4.25	2.00	1.35	M$18.71/m	
Paper Drain (M$/m)	1.28	0.85	0.80	2.50	0.77	0.50	M$ 6.70/m	
Fabric Drain (M$/m)	1.28	0.85	0.80	2.50	0.77	1.00	M$ 7.70/m	
Wellpoint (M$/m/month)	228.97	28.70	86.56		44.75	-	M$388.98/m /month	Per unit length of header pipe, well point: 10 m deep
Deep Well (M$/m)	252.00	15.28	(76.25)		16.67	-	(-)	
Vibro-Rod (M$/m)	10.49	5.85	1.17	4.50	2.86	1.80	M$26.67/m	
Vibro-floatation (M$/m)	10.28	8.03	1.08	4.50	3.11	1.80	M$28.80/m	
Sand Compaction Pile (M$/m)	16.45	9.18	1.73	4.72	4.00	4.50	M$40.58/m	
Dynamic Consolidation (M$/m2)	30.85	10.84	3.75	9.17	8.34	3.75	-	
Roller Compaction (M$/m2)	—	—	—	—	—	—	M$ 1.0/m2	
Chemical CMC (Cement) (M$/m)	65.80	27.54	4.15	14.16	14.51	38.50	M$164.66/m	
Chemical DLM (Lime)	65.80	27.54	4.15	14.16	14.51	23.10	M$149.26/m	
Full Excavation	-	-	-	-	-	—	M$ 9/m3	

Table H-3 Cost of Ground Improvement for Soft Material (Type B ground, Medium and High Rise)

(Refer Table 6-3)

Method \ Description	Cost per 1m (MS/m)	Cost per*1 5m (MS)	Unit Cost *2 per $1m^2$ (MS/ m^2)	① 1.5 x A(bldg) (MS/m^2)	② Pre-loading 3xA (bldg) (MS/m^2)	① + ② (MS/m^2)	Notes
Preloading	-	-	-	-	27.6	28	3m preloading
Sand Drain	18.71	93.55	23.39	35.09	27.6	63	Space of Drain 2m x 2m Square
Paper Drain	6.70	33.50	21.44	32.16	27.6	60	Space of Drain 1.25m x 1.25m Square
Fabric Drain	7.70	38.50	19.64	29.46	27.6	57	Space of Drain 1.4m x 1.4m Square
Wellpoint	-	-	-	-	-	78 (218)	( ): t_{80} = 16 months
Deep Well	-	-	-	-	-	28 (39)	
Vibro-Rod	26.67	133.35	33.34	50.01	-	50	Space of Pile 2m x 2m Square
Vibrofloatation	28.80	144.00	36.00	54.00	-	54	- ditto -
Sand Compaction Pile	40.58	202.90	50.73	76.10	-	76	- ditto -
Dynamic Consolidation	-	-	33.35	50.03	-	50	-
Roller Compaction	-	-	-	-	-	1	Shallow Depth only
Chemical CMC (Cement)	164.66	823.30	205.83	308.75	-	309	Space of Installation 2m x 2m Square
Chemical DLM (Lime)	149.26	746.30	186.58	279.87	-	280	
Full Excavation	-	-	-	-	-	-	

*1 Thickness of Improvement = 5 m
*2 See conditions in Notes.

Table H-4 Cost of Ground Improvement for Soft Material (Type C ground, Medium and High Rise)

(Refer Table 6-3)

Description / Method	Cost per 1m (M$/m)	Cost per 5m [*1] (M$)	Unit Cost per 1m² [*2] (M$/m²)	① 1.5 x A(bldg) (M$/m²)	② Pre-loading 3 x A (bldg) (M$/m²)	① + ② (M$/m²)	Notes
Preloading	–	–	–	–	43.5	44	3m preloading
Sand Drain	18.71	93.55	23.39	35.09	43.5	79	Sapce of Drain 2m x 2m Square
Paper Drain	6.70	33.50	21.44	32.16	43.5	76	Space of Drain 1.25m x 1.25m Square
Fabric Drain	7.70	38.50	19.64	29.46	43.5	73	Space of Drain 1.4m x 1.4m Square
Wellpoint	-	-	-	-	-	-	-
Deep Well	-	-	-	-	-	-	-
Vibro-Rod	-	-	-	-	-	-	Space of Pile 2m x 2m Square
Vibrofloatation	-	-	-	-	-	-	- ditto -
Sand Compaction Pile	40.58	202.90	50.73	76.10	-	76	- ditto -
Dynamic Consolidation	-	-	-	-	-	-	-
Roller Compaction	-	-	-	-	-	-	Shallow Depth only
Chemical CMC (Cement)	164.66	823.30	205.83	308.75	-	309	Space of Installation 2m x 2m Square
Chemical DLM (Lime)	149.26	746.30	186.58	279.87	-	380	
Full Excavation	-	-	45	-	-	90	Area of Replacement = 2 x A (bldg)

*1 Thickness of Improvement = 5m

*2 See conditions in Notes.

Table H-5 Cost of Ground Improvement for Soft Material (Type D ground, Medium and High Rise)

(Refer Table 6-3)

Description / Method	Cost per 1m (M$/m)	Cost *1 per 10m (M$)	Unit Cost *2 per 1m2 (M$/m2)	① 1.5 x A (bldg) (M$/m2)	② Pre-loading 3xA (bldg) (M$/m2)	① + ② (M$/m2)	Notes
Preloading	-	-	-	-	46.5	47	3m preloading
Sand Drain	18.71	187.10	46.78	70.17	46.5	117	Sapce of Drain 2m x 2m Square
Paper Drain	6.70	67.00	42.88	64.32	46.5	111	Space of Drain 1.25m x 1.25m Square
Fabric Drain	7.70	77.00	39.29	58.94	46.5	105	Space of Drain 1.4m x 1.4m Square
Wellpoint	-	-	-	-	-	-	-
Deep Well	-	-	-	-	-	-	-
Vibro-Rod	-	-	-	-	-	-	-
Vibrofloatation	-	-	-	-	-	-	-
Sand Compaction Pile	40.58	405.80	101.45	152.18	-	152	-
Dynamic Consolidation	-	-	-	-	-	-	-
Roller Compaction	-	-	-	-	-	-	Shallow Depth only
Chemical CMC (Cement)	164.66	1646.60	411.65	617.48	-	617	Space of Installation 2m x 2m Square
Chemical DLM (Lime)	149.26	1492.60	373.15	559.73	-	560	- ditto -
Full Excavation	-	-	-	-	-	-	-

*1 Depth of Improvement = 10m.

*2 See conditions in Notes.

H.2 Unit Cost of Foundations + Soft Ground Improvement

(Reference to Table 8-3 in Main Text)

Unit: M$

H.2.1 Ground Condition A

(1) Unit cost of Surface Compaction by Mechanical Compactor for Low-Rise Housing

a. Conditions

1) Mechanical Compactor: 10 ton-class

2) Rental fee with a driver and fuel : M$16,000/month (one month = 26 days)

3) One day is required to compact an area of 2,500 m^2 (50 m x 50 m) with 10 passes.

4) Surface cleaning, grubbing, etc. will also be performed.

b. Cost per Unit Area

1) Unit rate for surface compaction with mechanical compactor: (M$L6,000/26 days) x 1 day/ 2,500 m^2 = M$0.25/$m^2$

2) Cost for surface cleaning, grubbing, etc. = M$0.75/$m^2$

3) Thus, cost for area preparation is:-

M$0.25 + M$0.75 = M$1.0/$m^2$

(2) Unit Cost of Direct Foundation + Compaction of Sand Layer for Medium-Rise Housing

Unit cost of direct foundation + compaction of sand layer is summarized in Table H-6.

Table H-6 Unit Cost of Direct Foundation + Compaction of Sand Layer for Medium-Rise Housing

Soil Improvement Method *1	Unit Costs of Lease of Machines, Operation, Transport, Installation and Miscellaneous Expenses	Material Unit Cost	Total Unit Cost	Required Unit Cost *2
1.3.1 Vibro Rod	M$24.9/m	M$1.8/m	M$26.7/m	{(M$26.7 x 5m) ÷ $4m^2$} x 1.5 ≑ M$50/$m^2$
1.3.2 Dynamic Consolidation	M$30.3/$m^2$	M$3.0/$m^2$	M$33.3/$m^2$	M$33.3 x 1.5 ≑ M$50/m^2
1.3.3 Vibro Floatation	M$27.0/m	M$1.8/m	M$28.8/m	{(M$28.8 x 5m) ÷ $4m^2$} x 1.5 ≑ M$54/$m^2$
1.3.4 Composer Pile	M$36.1/m	M$4.5/m	M$40.6/m	{(M$40.6 x 5m) ÷ $4m^2$} x 1.5 ≑ M$76/$m^2$

(Notes) *1 Basis for Cost Estimation
- Machine and equipment are assumed to be available in Kuala Lumpur.
- Construction Area: 40,000 m^2, Depth of Improvement: 5 m
- Installation Pitch of 1.3.1, 1.3.3 and 1.3.4 is 2 m x 2 m square.

*2 For above methods, it is necessary to improve the ground area equal to 1.5 times the floor area of building.

(3) Unit Cost of Pile + Surface Compaction for Medium-Rise Housing

a. Treated Timber Pile (6" sq.) + Surface Compaction

i) Condition and Assumption

1) Size of Treated Timber Pile

L = 10.5 m, 6" square

2) Unit Cost per Design Load and Length:

M$0.8/ton·m

3) Structural Design Load: $3.9\ t/m^2$

4) Unit cost of pile is calculated assuming 30% of additional piles are required that the number of piles calculated from the structural design load.

ii) Calculation of Unit Cost

1) Piling

*Unit Cost of Pile

$(3.9\ t/m^2 \times 1.3) \times 10.5\ m \times M\$0.8/ton\cdot m$
$= 5.07 \times 8.4 = M\$42.6/m^2$

*Unit Cost for Joint of Pile (one joint per pile)

$(5.07\ tons \div 20\ tons/pile) \times M\$15/joint$
$= M\$3.8/m^2$

2) Unit Cost of Surface Compaction: $M\$1.0/m^2$

3) Total: $M\$42.6/m^2 + M\$3.8/m^2 + 1.0/m^2 = \underline{M\$47.4/m^2}$

b. <u>Steel Pile (30m Long: See Table 10-3b) + Surface Compaction</u>

i) <u>Condition and Assumption</u>

1) Length of Steel Pile: L = 30 m

2) Unit Cost per Design Load and Length : M$1.0/ton·m

3) Structural Design Load: 3.9 t/m^2

4) Unit cost of pile is calculated assuming 30% of additional piles are required than the number of piles calculated from the structural design load.

ii) <u>Calculation of Unit Cost</u>

1) Unit Cost of Steel Pile

$(3.9\ t/m^2 \times 1.3) \times 30\ m \times$ M$1.0/ton·m

$= 5.07 \times 30 =$ M$152.1/$m^2$

2) Unit Cost of Surface Compaction: M$1.0/$m^2$

3) Total: M$152.1/$m^2$ + 1.0/m^2 = <u>M$153.1/$m^2$</u>

(4) <u>Unit Cost of Pile + Surface Compaction for High-Rise Housing</u>

a. <u>RC Pile + Surface Compaction</u>

i) <u>Condition and Assumption</u>

1) Size of RC Pile: 15" sq., L = 11.5m

2) Structural Design Load: 16.75 tons/m^2

3) Unit Cost per Design Load and Length:

M$0.7/ton·m

4) Unit cost of pile is calculated assuming 30% of additional piles are required than the number of piles calculated from the structural design load.

ii) Calculation of Unit Cost

1) Unit Cost of RC (15" sq.) Pile

$(16.75\ t/m^2 \times 1.3) \times 11.5\ m \times M\$0.7/ton \cdot m$

$= M\$175.3/m^2$

2) Unit Cost of Surface Compaction: $M\$1.0/m^2$

3) Total: $M\$175.3 + M\$1.0 = \underline{M\$176.3/m^2}$

b. Steel Pile (30 m Long: see Table 10-3b) + Surface Compaction

i) Condition and Assumption

1) Length of Steel Pile: L = 30 m

2) Structural Design Load: 16.75 tons/m^2

3) Unit Cost per Design Load and Length : M$1.0/ton·m

4) Unit cost of pile is calculated assuming 30% of additional piles are required than the number of piles calculated from the structural design load.

ii) <u>Calculation of Unit Cost</u>

1) Unit Cost of Steel Pile

$(16.75\ t/m^2 \times 1.3) \times 30\ m \times M\$1.0/ton{\cdot}m$

$= M\$653.3/m^2$

2) Unit Cost of Surface Compaction: $M\$1.0/m^2$

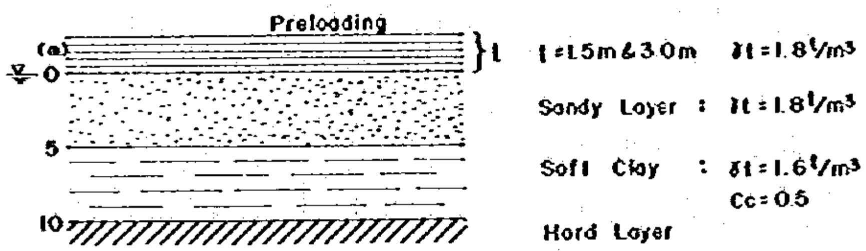
3) Total: $653.3 + 1.0 = \underline{M\$653.3/m^2}$

H.2.2 Ground Condition B

(1) Unit Cost of Preloading + Surface Compaction

a. Conditions

1) Soil Conditions



e ∿ log p curve of SBH-2, UD-4 is used for the calculation of consolidation settlement.

2) Calculation of Settlement

In case of t = 1.5 m

$$P_o = (1.8 - 1.0) \times 5 + (1.6 - 1.0) \times 2.5$$
$$= 5.5\ t/m^2 \rightarrow e_o = 1.40$$

$$P_1 = 5.5 + (1.8 \times 1.5) = 8.2\ t/m^2 \rightarrow e_1 = 1.34$$

$$\therefore S = \frac{1.40 - 1.34}{1 + 1.40} \times 500 = 12.5\ cm \rightarrow 15\ cm$$

In case of t = 3.0 m

$$P_o = 5.5\ t/m^2 \rightarrow e_o = 1.40$$

$$P_1 = 5.5 + (1.8 \times 3.0) = 10.9\ t/m^2 \rightarrow e_1 = 1.30$$

$$\therefore S = \frac{1.40 - 1.30}{1 + 1.40} \times 500 = 21\ cm \rightarrow 25\ cm$$

b. <u>Unit Cost of Preloading (t = 1.5 m) + Surface Compaction for Low-Rise Housing</u>

1) Grubbing, Clearing, Spreading Earth and Surface Compaction: M$1.0/m^2

2) Loss of Material (Normal earth) due to Settlement:
0.15 m x M$6/m^3 = M$0.9/m^2

3) Other Loss of Material: 0.20 m x M$6/m^3 = M$1.2/m^2

4) Transportation for Transferring Preloading Material to Next Site: 1.35 m^3/m^2 x M$2/m^3 = M$2.7/m^2

Total : 1) + ... + 4) = M$5.8/m^2

5) Unit Cost for 3 times the Floor Area of Low Rise [1F] House: M$5.8/m^2 x 3 = <u>M$17.4/m^2</u>

c. <u>Unit Cost of Treated Timber Pile (6" Sq.) + Preloading (t=3.0m) + Surface Compaction for Medium Rise Housing</u>

1) Unit Cost of Treated Timber Pile (6" Sq.)
Same to the unit cost of treated timber pile (6" Sq.) calculated at ii) in a. on page H-14:
M$42.6/m^2 + M$3.8/m^2 = M$46.4m^2

2) Earth Work

i) For Grubbing, Clearing, Spreading Earth and Surface Compaction:

$M\$1.0/m^2$

ii) Loss of Material (Normal earth) due to Settlement:

$0.25\ m \times M\$6/m^3 = M\$1.5/m^2$

iii) Other Loss of Material:

$0.20\ m \times M\$6/m^3 = M\$1.2/m^2$

iv) Transportation for Transferring Preloading Material to Next Site:

$2.75\ m^3/m^2 \times M\$2/m^2 = M\$5.5/m^2$

Total $= M\$9.2/m^2$

v) Unit Cost for 3 times of the Floor Area of Medium Size (5F) Building:

$M\$9.2 \times 3 = M\$27.6/m^2$

3) Total: 1) + 2) $= M\$46.4/m^2 + M\$27.6/m^2 = \underline{M\$74.0/m^2}$

d. <u>Unit Cost of RC Pile (15" Sq.) + Preloading (t=3.0m) + Surface Compaction</u>

1) Unit Cost of RC Pile (15" Sq.)

Same to the unit cost of RC (15" Sq.) Pile calculated at ii) on page H-16: $M\$175.3/m^2$

2) Unit Cost of Preloading + Surface Compaction

Same to the unit cost of earth work calculated at 2) in c. on this page : $M\$27.6/m^2$

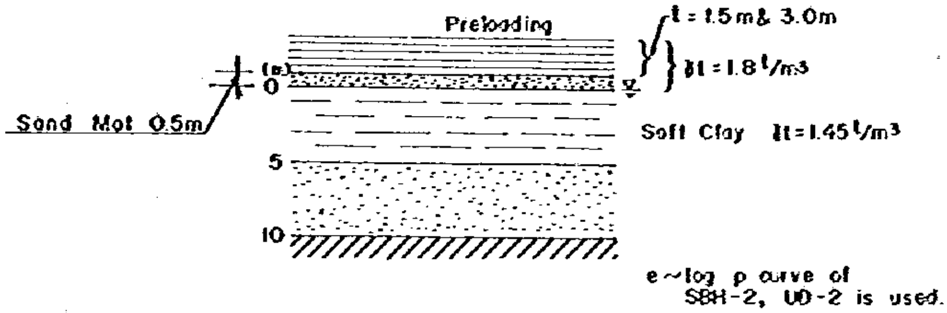
3) Total: 1) + 2) $= M\$175.3/m^2 + M\$27.6/m^2 \doteqdot \underline{M\$202.9/m^2}$

H.2.3 Ground Condition C

(1) Unit Cost of Sand Mat (t = 0.5 m) + Surface Soil (with Surface Compaction) + Preloading

a. Conditions

1) Soil Conditions



2) Calculation of Settlement

In case of t = 0.5 m + 1.5 m = 2 m

P_o = (1.45 - 1.0) x 2.5 = 1.125 t/m² → e_o = 2.23

P_1 = 1.125 + 1.8 x 2.0 = 4.73 t/m² → e_1 = 1.79

$$\therefore S = \frac{2.23 - 1.79}{1 + 2.23} \times 500 \fallingdotseq 70 \text{ cm}$$

In case of t = 0.5 + 3.0 m = 3.5 m

P_o = 1.125 t/m² → e_o = 2.23

P_1 = 1.125 + 1.8 x 3.5 = 7.43 t/m² → e_1 = 1.64

$$\therefore S = \frac{2.23 - 1.64}{1 + 2.23} \times 500 = 91 \text{ cm} \rightarrow 95 \text{ cm}$$

b. Unit Cost for Low-Rise Housing

1) Grubbing, Clearing, Spreading Earth and Surface Compaction: M$1.0/m^2$

2) Loss of Materials due to Settlement:
{ 0.5m (Sand) x $9/m^3$ } + { 0.2m (Normal earth) x M$6/m^2$ }
= M$5.7/m^2$

3) Other Loss of Material: 0.2 m x M$6/m^3$ =M$1.2/m^2$

4) Transportation for Transferring Preloading Material to Next Site: 1.35 m^3/m^2 x M$2/m^3$ = M$2.7m^2$

Total of 1) + ... + 4) = M$10.6/m^2$

5) Unit Cost for 3 times the Floor Area of Low Rise [1F] House: M$10.6/m^2$ x 3 = M$31.8/m^2$

c. Unit Cost of Sand Mat + Surface Soil (with Surface Compaction) + Preloading for Medium Rise Housing

1) Unit Cost of Timber Pile [6" Sq.]

Same to the unit cost of timber pile [6" Sq.] shown at ii) in a. on page H-14: 42.6 + 3.8 = M$46.4/m^2$

2) Earth Work

i) Grubbing, Clearing, Spreading Earth and Surface Compaction:

$M\$1.0/m^2$

ii) Loss of Materials due to Settlement:

{0.5 m (Sand) x $M\$9/m^3$}
+ {0.45 m (Normal earth) x $M\$6/m^3$} = $M\$7.2/m^2$

iii) Other Loss of Material: 0.2 m x $M\$6/m^3$ = $M\$1.2/m^2$

iv) Transportation for Transferring Preloading Material to Next Site:

$2.55\ m^3/m^2$ x $M\$2/m^3$ = $M\$5.1/m^2$

Total $M\$14.5/m^2$

v) Unit Cost for 3 times the Floor Area of Medium Rise Housing

$M\$14.5/m^2$ x 3 = $M\$43.5/m^2$

3) Total: 1) + 2)

$M\$46.4/m^2$ + $M\$43.5/m^2$ = $\underline{M\$89.9/m^2}$

d. Unit Cost of Replacement for Medium Rise Housing

1) Excavation of Soft Clay:

$M\$0.8/m^3$ x $5\ m^3/m^2$ = $M\$4/m^2$

2) Transportation for Removal of Soft Clay:

$M\$2.0/m^3$ x $5\ m^3/m^2$ = $M\$10/m^2$

3) Filling Material: $M\$6/m^3$ x $5\ m^3/m^2$ = $M\$30/m^2$

4) Compation: $M\$1/m^2$

Total of 1) + ... + 4) $M\$45/m^2$