SOIL IMPROVEMENT BY USING VERTICAL DRAINS AND PRELOADING

BY

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A Project Report

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ABSTRACT

The present study is concerned with the performance of vertical drains and preloading installed in soft clay soil at a selected location of Bangladesh. The performance study was limited to the improvement of shear strength and settlement behaviour of the soft soil. Sand drains of 200mm diameter were installed in square patterns at a spacing of 1.77 m to a depth of 7.62 m . The project was undertaken by the Public Works Department of Bangladesh in Sunamgonj district with an aim to improve the existing soil conditions at the site for the proposed District Judge Court and District Collectorate Building Construction Projects.

The top soil at the site, up to a depth of approximately 6 m , is composed of very soft to soft clay . The soil was found to be unsuitable for the construction of shallow foundations and accordingly the soil improvement scheme was undertaken . The method of soil improvement consisted of removal of top 2 m of soft soil , installation of vertical drains, laying of 0.75m of sand cushion. and the application of 2.75m of soil preloading. Standard penetration and undrained shear strength tests were carried out for the soil at different depths prior to and after two months of application of preload. The ultimate bearing capacity as calculated from the results of standard penertation test and using Terzaghi's (1948) formula showed

an increase in the range of 0 to 135 percent. The increase in strength is found to decrease with depth. It was also observed that at a distance of approximately 0.75 times the spacing of the sand drains , the increase in shear strength was negligible. The calculated settlement at 50% consolidation based on the consolidation test results and Terzaghi's (1943) method showed an overestimation in comparison to actual results.

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LIST OF ABBREVIATIONS AND NOTATIONS

AIT	Asian Institute of Technology
BH	Bore hole
BUET	Bangladesh University of Engineering and Technology
E.G.L.	Existing Ground Level
F.G.L.	Finish Ground Level
F.M	Fineness Modulus
F.S	Factor of Safety
D	Disturbed Sample
D.C.	District Collectorate
D.J.	District Judges
HBRI	Housing and Building Research Institute
kN/m^2	Kilo-newton per square metre
kN/m^3	Kilo-newton per cubic metre
LL	Liquid Limit
N	Number of blows per feet of penetration
PI	Plasticity Index
PL	Plastic Limit
P.W.D.	Public Works Department
q_u	Unconfined Compression Strength
SPT	Standard Penetration Test
U	Undisturbed Sample
UNDP	United Nations Development Programme
WL	Water Content

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1 INTRODUCTION

1.1 General

In the last few decades there has been a growing demand for construction on sites underlain by thick strata of soft soils. In such circumstances, soil improvement is required to prevent bearing capacity failures and/or to avoid excessive total and differential settlements. One of the many methods to achieve these aims is to preload the site in combination with the installation of vertical drains in which the soil undergoes the process of consolidation.

During one dimensional consolidation, the pore water is squeezed out mainly in the direction of applied load. When vertical drains are installed, pore water also escapes in the lateral direction towards the drains and flows freely along the drains vertically to a drainage blanket placed on the soil surface or to other highly permeable layers deeper down the soil. Thus , the installation of vertical drains in a homogeneous, clay layer will reduce the length of flow paths, thereby reducing the time of consolidation.

There are mathematical formulations to evaluate the performance of a soil layer particularly in relation to shear strength and settlement, when subjected to the process of consolidation. Due to the over simplified assumptions about the characteristics used in the derivation of a theory, the predicted and observed values may sometimes be differ.

Hence it is, always, worthwhile to study the performance of a project in order to examine the relation between predicted and observed parameters.

The Public Works Department of Bangladesh has undertaken a project to improve the subsoil conditions at the sites of District Judge Court and District collectorate Court buildings in Sunamgonj District. The top soil at the site was soft clayey soil and found to be unsuitable for construction of any shallow foundations . As such a soil improvement project using vertical drains and preloading was undertaken. It expected that the analysis of the soil data of the site will yield certain valuable information regarding the performance of soil improvement scheme. And hence it decided to carry a systematic study.

Subsoil investigation data of the two sites were collected. The top soil upto a depth of 6 m were found to comprise of very soft to soft clayey soil and organic materials. The soil improvement project was designed using 200mm diameter sand drains up to a depth of 7.62 m. The data were collected for settlement and strength of the soil layer under consideration (at 50 % degree of consolidation) in order to meet the outlined subsequently.

1.2 Objective of Study

The main objectives of the present study are :-

(I) To analyse the data in order to compare the actual results of settlements with the predicted results .

(II) To observe the performance in relation to shear strength of soil.

1.3 Methodology of Study

The methodology of study consists of :-

(I) The collection of SPT, shear strength, consolidation, load test and other related soil data prior to and after the installation of vertical drains.

(II) The review of existing literature on vertical drains and preloading .

(III) The analysis of data for the comparison of predicted and field results of settlements.

(IV) The analysis of data for the shear strength of soil prior to and after the improvement.

1.4 Organization of Study

The study is presented in five chapters, the first of which is an introduction. Chapter-II presents review of relevant literature. Chapter-III describes the statement of the problem. Chapter-IV presents the analysis of results and discussions . Finally , Chapter-V discusses the conclusion of the present work and recommendation for future work.

2 REVIEW OF LITERATURE

2.1 General

There are several methods currently being used , all over the world for the improvement of soft soils. Of them, frequently used methods are: (i)Vertical drains with preloading (ii) Stone columns (iii) Dynamic compaction (iv) Thermal theatment (v) Electro-Osmosis (vi) Injection and grouting and (vii) Admixture stabilization. For the detailed description of the methods reference can be made of Mitchell and Katti (1981) and Lee, White and Ingles (1983). In Bangladesh the most common method being vertical drains with preloading. The present study is mainly concerned with the review of existing literature on the theory and practice of vertical drains.

2.2 Vertical Drains

Vertical drains have been employed for almost half a century to promote rapid consolidation of relatively thick deposits of soft fine-grained soils . The first installation appears to have been in California, U.S.A. in 1934 (PROTER, 1936)* using 500 mm and larger diameter sand drains. Initially these drains were installed mainly by means of closed-ended mandrels which caused a cosiderable thickness of smear around the drains. The smear problem was overcome in the Netherlands during 1950s with the development of jetted sand drains . This method had its own problems, notably the additional cost of large jetting pumps and

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* Cited by Singh(1986)

the difficulties of disposing of large quantities of water The principal alternative to large diameter sand drains were the much smaller band-shaped cardboard wicks first employed by KJELLMAN (1948) in Sweden. These early band drains proved to be susceptible to rotting, particularly in acidic groundwaters and the use of these drains was not common until the last decade. In the 1970s, the rising costs of providing large quantities of suitable sand for sand drains and the great technical advances in the manufacture of man made fabrics led to the development and the use of increasing numbers of band drains produced from polyethylenes, PVC, polypropylenes, etc. In the following sections the various available vertical drains are briefly described.

2.3 Types of Vertical Drains

The most widely used vertical drains are the traditional sand drain, the prefabricated sand drain or sandwick and band drains.

2.3.1 Sand Drains

These are formed by infilling sand into a hole in the ground. The holes may be formed in a number of ways. Large diameter sand drains may also tend to act as strengthening columns in soft soils and modify the selttlement behaviour of the structure. Because of a number of problems, these types of vertical drains are rarely used now a days.

2.3.2 Sandwicks

These drains are prepared in a filter stocking and placed in a pre-drilled hole or into a mandrel (HUGHES and CHALMERS, 1972)*. Polpropylene woven and melt-bonded fabrics are now used as materials for filter stocking. The filter stockings allow the sandwicks to be extremely flexible and tenacious which makes these particularly attractive in a number of situations.

2.3.3 Plastic Band Drains

A prefabricated flat band drain consists of a central core, whose function is to act as a free draining channel, surrounded by a thin filter sleeve, which prevents the soil surrounding the drain from entering the central core but allows free entry to the core of excess pore pressure, as well as providing channels for the discharge of water. The core also provides strength during installation and resistance to crusing from the soil pressure. Band drains are generally placed by displacement methods.

Types of plastic band drains- Currently there are a large number of band drains available and a cursory examination suggests that there is much similarity between the majority of them. A list of various types of prefabricated drains is presented in Table 2.1.

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*Cited by Singh(1986)

Table 2.1 Prefabricated Vertical Drains

Type	Remarks
Cardboard drain	Full displacement of small volume
Fabric covered plastic drain	Full displacement of small volume
Plastic drain without jacket	Full displacement of small volume

Chracteristics of prefabricated drains - Probably the most relevent features for the design and performance of prefabricated band shaped drains are their hydraulic properties, e.g. the discharge capacity of the drain cross-section and the filter permeability. The filter permeability should not be less than the surrounding soil mass in which these drains are placed. The mechanical chracteristics of the band shaped drains, like the tensile strength of the core and filter, and the buckling strength of the core are also important in view of the stresses to which drains are subjected during installation, due to large displacements and because of large vertical settlements.

Potential advantages of prefabricated drains - In both uniform and non uniform soils, the use of prefabricated drains has many advantages, such as, (a) the increased rate of gain in shear strength of the clay enables the load to be applied more rapidly than would otherwise have been possible, often allowing a better utilization of construction plant. Furthermore , in case of embankments, steeper side slopes and the avoidance of the use of berms may be possible when such vertical drains are employed. Thus, the total volume of earth fill required may be reduced and the rate of construction increased, The consequent savings in cost may be appreciably more than the outlay on the drain installation. (b) the increased rate of consolidation of clay results in a reduction of the time required for primary settlement to take place . Consequently, structures can be built or embankments can be put into commission far earlier than would otherwise have been possible, or the subsequent maintenance costs can be greatly reduced, (c) Many soft clay strata contain thin bands, or partings, of silt or sand. Instability of embankments or tanks built on such strata is sometimes due to the horizontal spread of excess pore pressure along these bands or partings (TERZAGHI,1943).Prefabricated vertical drains can relieve these excess pore pressures, and thus, avoid the occurrence of instability.

Advantage over sand drains - Because of smaller volume of the installed drains compared to the volume of treated soil, the degree of disturbance associated with the installation of prefabricated drains are appreciably lower than that occurs in case of conventional displacement sand drains. Due to this facts, the pre-fabricated drains are appropriate for applications in sensative clays and soils having high anisotropy of the permeability coefficient Another important advantage over sand drains is the simplicity of installation procedure, low cost, and high installation speed.

2.4 Consolidation by Vertical Drains

The main aim of consolidation by vertical drains is to reduce the consolidation time by shortening the drainage path. Many theories have been developed and a lot of development has been achieved in this area.

2.4.1 Development of Vertical Drain Theories - The development of vertical drain theories by various authors and solutions are summarized herein . RENDULIC (1935)* developed radial flow consolidation theory as an extention of TERZAGHI's one dimensional vertical flow consolidation theory. KJELLMAN (1937) developed the solution of radial flow consolidation based on equal strain theory. He also introduced nominal diameter of band-shaped drains. BARRON (1948) presented a complete solution of sand drain theories, taking into

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* Cited by Singh(1986)

consideration the effects of smear and drain well resistance. He introduced two types of solutions, the "Equal strain" and "Free strain". SCHIFFMAN (1958)* introduced the effect of linear construction loading and varying permeability of the soil during consolidation. RICHART (1959) developed BARRON's theoritical solutions into practical design chart. HANSBO (1961) developed a solution for radial flow consolidation based on his theory on the exssumption that at small hydraulic gradient of flow, Darcy's law is not valid. YOSHIKUNI and NAKANODO (1974) presented a rigorous solution of consolidation process including the effect of drain resistance. HANSBO (1979,1981) presented simple formulae based on equal strain theory, taking into consideration the effect of well- resistance and smear.

2.4.2 Barron's Theory - BARRON(1948) presented the first exhaustive treatment of the problem of the consolidation of a soil cylinder containing a central drain. His theory was based on all of the simplifying assumptions of Terzaghi's one dimensional consolidation theory. Besides these, he made the following additional assumptions:

(a) All vertical loads are initially carried by excess pore pressure u .

(b) All compressive strains within the soil mass occurs in vertical directions.

(c) The zone of influence of each well is a circle.

(d) Load distribution is uniform over the area.

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* Cited by Singh(1986)

(e) Shear strains developed in the foundation by differential settlements will have no effect on the consolidation process.

(f) Initial excess pore pressure, Uo is uniform throughout the soil mass at t = 0.

(g) The external radius, r_e is impervious or because of symmetry no flow occurs across this boundary, i.e. du/dr=0 when $r = r_e$.

(i) Excess pore pressure at upper boundary of the soil mass (z=0) is zero when t>0 in case of drainage at top.

(j) The lower horizontal boundary of soil mass (Z=H) is impervious or because of symmetry no flow occurs across this boundary, i.e. du/dr=0 at Z=H.

The basic differential equation in polar coordinates which applies to vertical flow problem is expressed as:

$$C_h \left(\frac{1}{r}\frac{du}{dr} + \frac{du^2}{dt^2} \right) = \frac{du}{dt} \qquad (2.1)$$

in which $C_h = \dfrac{k_h m_v}{\gamma w}$

Where: m_v = coefficient of volume change .

k_h = coefficient of horizontal permeability .

γ_w = unit weight of water.

r = radius, a coordinant in the cylindrical system.

u = excess pore water pressure.

t = time of consolidation.

BARRON's theory solves the problem of consolidation around vertical drains under the following boundary conditions which represent two boundaries of real field situations.

(a) Free vertical strain - The assumption is that the vertical stress on a surface remains constant during the consolidation process and the resulting surface displacements are, thus, non uniform. The solution of Eq. 2.1 for the given case of free strain, in terms of excess pore pressure at any point due to combined radial and vertical consolidation at a given time t in the contained soil cylinder having ideal drain well is given by BARRON (1948) as:

$$U_{r,z} = \frac{U_r . U_z}{U_o} \qquad (2.2)$$

Where,
- U_r = excess pore water pressure due to radial flow.
- U_z = excess pore water pressure due to vertical flow.
- $U_{r,z}$ = excess pore water pressure due to combined radial and vertical flow.
- U_o = initial uniform excess pore pressure.

(b) Equal vertical strain - The assumption is that the vertical surface displacement is constant throughout the drained area and the resulting vertical stress at the surface is thus non uniform. In case of equal strain, the solution for Eq 2.1 is given by BARRON(1948) as:

$$U_r = \frac{U_o}{r_e^2 F(n)} \left[r_e^2 \ln \frac{r_e}{\gamma_w} - \frac{r_e^2 - \gamma_w^2}{2} \right] \exp \frac{-8T_h}{F(n)} \qquad (2.3)$$

$$F(n) = \frac{n^2}{n^2-1} \ln(n) - \frac{3n^2-1}{4n^2} \qquad (2.4)$$

Where : r_e = radius of equivalent soil cylinder.

r_w = radius of drain.

T_h = dimensionless time factor for horizontal flow

$$= \frac{C_h t}{de^2}$$

de = diameter of equivalent soil cylinder.

n = ratio of diameter of equivalent soil cylinder to diameter of well.

F(n) = drain spacing factor.

In this case , the average degree of consolidation with respect to radial flow becomes:

$$U_h = 1 - \exp(-8T_h)/F(n) \qquad (2.5)$$

in which U_h is the average degree of consolidation with respect to radial flow. Thus, the time of consolidation is estimated from:

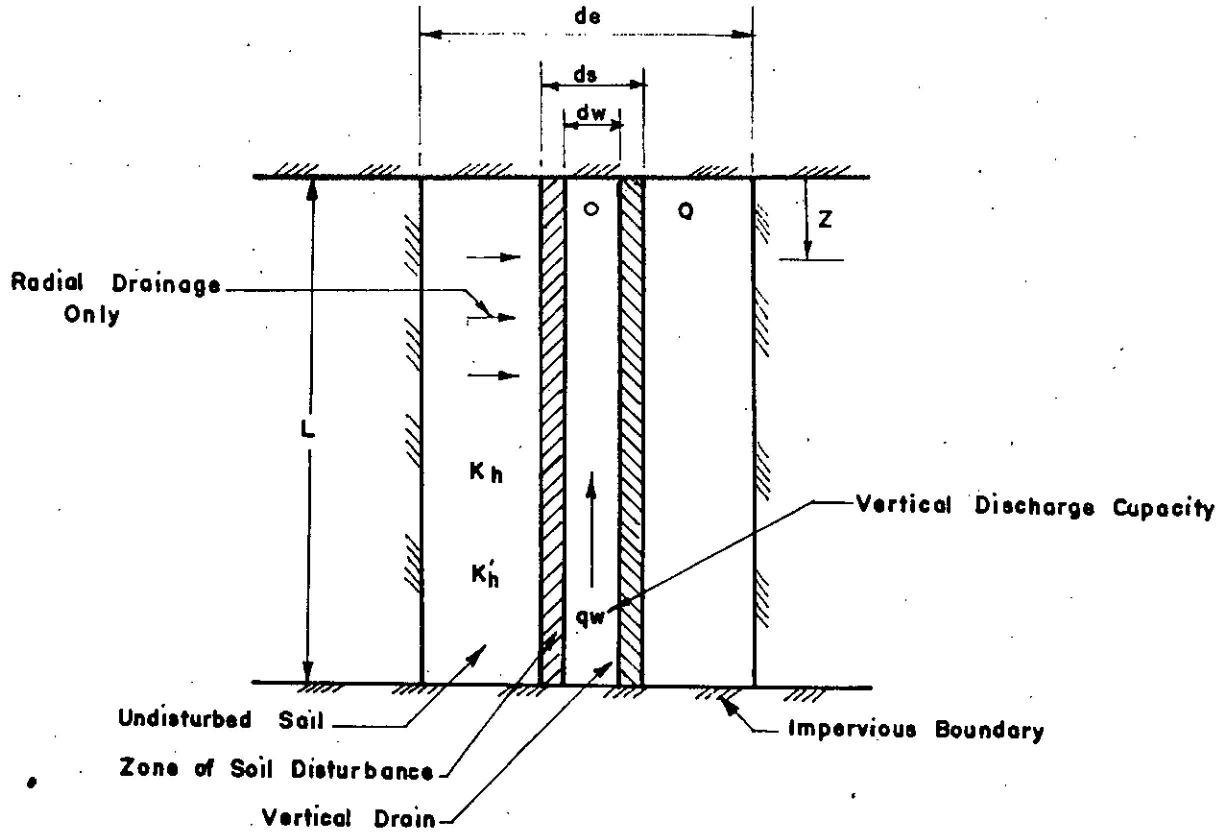


Fig. 2.1 Schematic Diagram of Vertical Drain

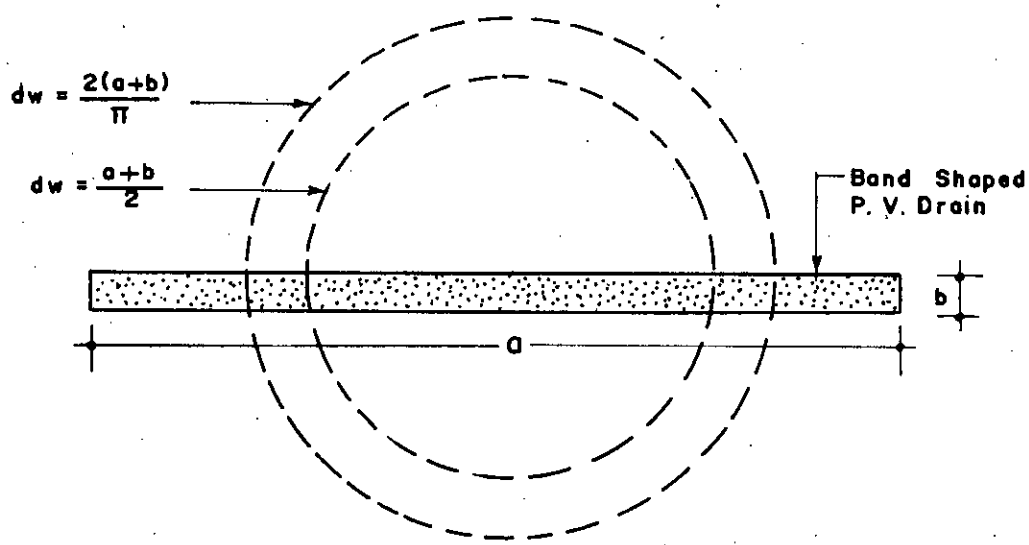


Fig. 2.2 Equivalent Diameter of a Vertical Drain.

$$t = \frac{d_e^2}{8C_h} \ln \frac{1}{1-U_h} F(n) \qquad (2.6)$$

F(n) is given by Eq. 2.4 and d_e is the diameter of equivalent soil cylinder (Fig. 2.1)

Effect of smear and well resistance - The above given formulas are valid under the ideal conditions i.e. the drain has infinite permeability and that its installation does not change the characteristics of the surrounding clay which govern the consolidation process. In reality , the installation of the drain causes a disturbance of the surrounding clay mass which leads, depending on sensitivity and macrofabric, to more or less pronounced changes in its flow and stiffness characteristics. Also, the commonly adopted hypothesis of an infinite permeability of the drain well becomes unacceptable, because in reality head loss will occur due to the resistance of well to flow. This effect becomes more pronounced in case of prefabricated band drains which usally have a relatively small cross-section.

2.4.3 Hansbo's Theory - HANSBO (1979, 1981) presented for equal strain conditions, a closed form solution which allows for a ready compution of the effects of well resistance and smear on drain performances. He made the same assumptions as made earlier by BARRON (1948) which was used in deriving his solutions for drains wells. The solution for average degree of radial consolidation is given by HANSBO (1979) as:

$$U_h = 1 - \exp \frac{-8T_h}{Fc(n)} \qquad (2.7)$$

which is similar to earlier solution given by Eq. 2.5 , the factor F(n) being Fc(n). The expression for F(n) is given by Eq. 2.4 . The time for radial consolidation is given as :

$$t = \frac{d_e^2 Fc(n)}{8C_h} \ln \frac{1}{1-U_h} \qquad (2.8)$$

(1) Effect of well resistance - Because of smaller diameter, in case band drains, the effect of well resistance is more significant. The effect of well resistance is considered by imposing the condition $dQ_2 = dQ_1$ on the continuity of flow through the drain. The factor F(n) now takes the form $F_w(n)$ and may be expressed as

$$F_w(n) = \ln(n) - 0.75 + z(\ 2l - z\ ) \frac{k_h}{q_w} \qquad (2.9)$$

Where: $F_w(n)$= factor for the effect of well resistance.

q_w = discharge capacity of the drain .

dQ_1 = quantity of water entering the drain well.

dQ_2 = quantity of water leaving the drain well.

k_h = coefficient of horizontal permeability.

(2) Effect of smear - The effect of soil remoulding caused by drain may be included in the analysis of radial consolidation, assuming that an annulus of smeared clay with an internal diameter ds exists around the drain. Within this annulus the soil is strongly remoulded and its coefficient of permeability k_h is lower than k_h' which characterises the

undisturbed clay. This circumstance imposes an additional continuity condition of the flow at the boundary between the smeared and undisturbed zone, which causes a further modification of the factor F(n) and takes the form as (HANSBO, 1979):

$$F_s(n) = \frac{k_h}{k_h'} \ln(s) - \ln(s) \qquad (2.10)$$

Where: k_h' = coefficient of horizontal permeability in smeared zone.

s = smear ratio.

(3) Combined effect of smear and well resistance - Considering both the effects of well resistance and smear, F(n) finaly takes the form (HANSBO ,1979) as:

$$Fc(n) = \ln \frac{n}{s} \frac{k_h}{k_h'} \ln(s) - 3/4 + Z(2l - Z) \frac{k_h}{q_w} \qquad (2.11)$$

Where: Z = considered depth of soil.

l = length of the drain.

2.5 Theory of Consolidation

2.5.1 One Dimensional Consolidation

TERZAGHI (1943) established the well-known theory of one dimensional consolidation. This theory takes into account only primary consolidation due to vertical drainage. The basic differential equation for one dimensional consolidation is given by :

$$du/dt = Cv\ du^2/dz^2 \qquad (2.12)$$

in which: $Cv = k_h m_v / \gamma w$

where: Cv = coefficient of consolidation in vertical direction.

k_v = coefficient of vertical permeability.

The approximate solution for degree of consolidation, Uv, in case of constant initial excess pore pressure Uo is given by:

$$Uv = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M} \exp(-MTv) \qquad (2.13)$$

Uv = average degree of consolidation due to vertical flow.

M = (2m+1)/2

m = an integer.

Tv = dimensionless time factor for vertical flow
= Cvt^2/H^2

H = depth of clay layer.

It has been found that Eq. 2.13 may be represented with high precision by the following empirical expressions (TAYLOR, 1948) as:

When Uv < 60 % , $Uv = 2(Tv/\pi)^{0.5}$ (2.14a)

When Uv > 60 % , $Uv = 1 - (8/\pi^2) \exp(-\pi^2/4Tv)$ (2.14b)

2.5.2 Two and Three dimensional Consolidation - In three dimensional consolidation the drainage occurs in radial planes as well as in the conventional vertical direction. The differential equation of consolidation can be expressed as:

$$du/dt = Cv(d^2u/dx^2 + d^2u/dy^2 + d^2u/dz^2) \qquad (2.15)$$

For two dimensional consolidation, the differential equation can be written as:

$$du/dt = Cv(du^2/dx^2 + d^2u/dz^2) \qquad (2.16)$$

In polar coordinates the above equation will take a form

$$du/dt = C_{vh}(d^2u/dr^2 + du/rdr) \quad (2.17)$$

Where, C_{vh} equals the coefficient of consolidation due to combined radial and vertical flow.

CARRILO (1942) resolved Eq. 2.17 into two parts, namely radial and vertical drainage. The average degree of consolidation due to the combined drainage is then expressed as :

$$U = 1 - (1 - U_h)(1 - U_v) \quad (2.18)$$

in which U is the average degree of consolidation due to radial and vertical flow. For further details of the topic reference can be made to RIXNER et al (1986).

2.6 Stress distribution in a Soil Mass

Considerations of deformations of underlying compressible materials will require determination of the distribution of stresses within and under the load of the structure. Numerous researchers have proposed approximate methods of calculation of stress distribution, but due to complex nature of soil and boundary conditions none of them represent the actual conditions encountered in the field.

2.7 Pore Pressure Prediction

Two stages are involved in predicting excess pore pressures generated by an undrained loading on a saturated clay. First, the relationship between the change in total stress and excess pore pressure must be established. Second, the change in total stress induced by the applied load must be determined. Some of the methods for computing excess pore pressure from total stress changes are briefly discussed the following sections.

In one dimensional case, the pore pressure (u) is equal to the change in vertical total stress,

$$u = p_z \tag{2.19}$$

in which Pz is the stress increment in vertical direction, z .

For a soil considered to be an elastic material subjected to a three dimensional change in stress, u is equal to the change in octahedral normal stress and can be express as :

$$u = 1/3\ (\ p_x + p_y + p_z\ ) \tag{2.20}$$

in which Px, Py are the stress increments in x and y directions

SKEMPTON and HENKEL suggested a semi-empirical method to estimate pore water pressure as :

$$U = B\ Px\ A\ (\ Pz - Px\ ) \tag{2.21}$$

$$U = V_{oct} + a\tau_{oct} \tag{2.22}$$

in which,

A and B = SKEMPTON's pore pressure coefficients.

V_{oct} = Change in octahedral stress.

τ_{oct} = Change in octahedral shear srtess.

a = HENKEL's parameter.

In both SKEMPTON and HENKEL's methods, the soil is considered to be inelastic and tends to experience volume changes when pure shear is applied. SKEMPTON's method is strictly valid only for triaxial stresses. HENKEL'S method is more general. It considers that the excess pore pressure has two components, one due to change in volumetric stress and the other due to change in deviatoric stress. The deviatoric component is related to the change in octahedral shear stress. When SKEMPTON's parameter A is determined in a triaxial test, the relationship between A and HENKEL's parameter a (BISHOP and HENKEL, 1962) is :

$$a = (3A - 1)/(2)^{0.5} \tag{2.22}$$

Thus, the prediction of excess pore pressure is influenced to an important degree by the method used to calculate total stress change i.e. stress distribution.

2.8 Settlement Analysis

Because of the complex nature of soil and the influence of stratification, none of the methods developed so far can accurately predict settlements. A theoretical analysis of the settlement process is indispensable because the results permit the engineer at least to recognize the magnitude and the distribution of settlement under the structure. The component of settlement consists of immediate , primary consolidation, and secondary settlements.

Method of Settlement Analysis

In the past , numerous methods have been used to calculate the immediate and final settlements . Some of these are described subsequently in brief.

One dimensional method - This method was developed by TERZAGHI (1943) and involves the numerical summation of different vertical strains beneath the foundation assuming only one dimensional vertical deformation. The soil compressibility properties can be determined from oedometer tests. For normally consolidated clay, the total settlement, S_T can be expressed as :

$$S_T = \sum_{0}^{H} \left[\frac{Cc}{1+e_o} \log \left(\frac{Po + \Delta p}{Po} \right) dH \right] \qquad (2.23)$$

For overconsolidated clay, the settlement can be calculated considering two distinct cases as follows.

(a) Case 1 : When (Po + Δp) < Pc

$$S_T = \sum_{0}^{H} \left[\frac{Cc}{1+e_o} \log \left(\frac{Po + \Delta p}{Pc} \right) dH \right] \qquad (2.24)$$

(b) Case 2 : When (po + Δp) > Pc

$$S_T = \sum_{0}^{H} \left[\frac{Cr}{1+e_o} \log \left(\frac{Pc}{Po} \right) + \frac{Cc}{1+e_o} \log \left(\frac{Po + \Delta p}{Pc} \right) \right] dH \qquad (2.25)$$

Where : Cc = compression index

Cr = recompression index

e_o = initial void ratio

Pc = maximum past pressure

Δp = increase in stress in vertical direction.

dH = thickness of soil layer.

Elastic method - THe elastic method is based on the approach described by DAVIS and POULOS (1968). This method involves the use of elastic theory to calculate the immediate and final settlements. For homogeneous soil, the immediate settlement, Si is :

$$Si = \frac{q\ B\ I}{Eu} \tag{2.26}$$

in which: q = intensity of surcharge loading.

I = influence value of loading for immediate settlement.

B = width of foundation.

Eu = undrained elastic modulus of soil.

For non homogeneous stratum, Si is given as :

$$Si = \frac{1}{Eu} [\ Pz - \nu\ (\ Px + Py\ )]\ dH \tag{2.27}$$

Where:

Px, Py, Pz = stress increments in x,y,z direction respectively.

ν = undrained poisson's ratio (0.5)

It is common practice to determine Eu over a range of deviator stress equal to half the ultimate stress ($Eu = E_{50}$), but it is probably a worthwile refinement to determine Eu over the range of stress relevent to the particular problem, since the experimental stress-strain relationship may not be linear even for stresses less than 50% of the ultimate stress . Also , the value of Eu will decrease significantly even with the slightest disturbance in the case of normally consolidated clays (SKEMPTON et al, 1955). The value of Eu used in Eq. 2.25 also be suitably chosen as the average value with respect to variations with depth.

In three dimensional approach involving several strata of different types, the total final settlement may be obtained by summation of strains in the individual strata as :

$$S_T = 1/E' \; (P_z - \nu' P_x - \nu' P_y) \; dH \qquad (2.28)$$

Where E', ν' are the elastic modulus and Poisson's ratio, respectively, for drained condition.

If, however, the clay stratum is reasonably homogeneous, and appropriate average values of E' and ν' can be assigned to the clay for the whole depth, S_T may be calculated from elastic displacement theory as :

$$S_T = q \; B \; I/E' \qquad (2.29)$$

Where I is the influence factor given by elastic displacement theory.

Skempton-Bjerrum method - In an attempt to take more satisfactory account of the three dimensional nature of settlement, SKEMPTON and BJERRUM (1955) developed a method in which the immediate settlement is taken into account. The final settlement is calculated based on the theory of elasticity as :

$$S_T = S_i + S_c \qquad (2.30)$$

in which

$$S_i = \frac{q\ B\ (1-v^2)\ I}{Eu} \quad (2.31)$$

$$Sc = \mu \int m_v\ dH \quad (2.32)$$

$$\mu = A + e(1-A) \quad (2.33)$$

$$e = \frac{\int Px\ dH}{\int Pz\ dH} \quad \text{for } (\ 0 < e > 1\ ) \quad (2.34)$$

In the above expression, μ is a correction factor and A is the SKEMPTON's pore pressure parameter. BJERRUM (1972) has proposed an expression, as given in Eq. 2.35 to calculate primary settlement, Sc, by considering the initial state of overconsolidation.

$$Sc = \mu_{oc} \sum_{o}^{H} m_v (Pc - Po) dH + \mu_{cp} \sum_{o}^{H} \frac{Cc}{1+e_o} \log \left(\frac{Po+\Delta p}{Pc}\right) \quad (2.35)$$

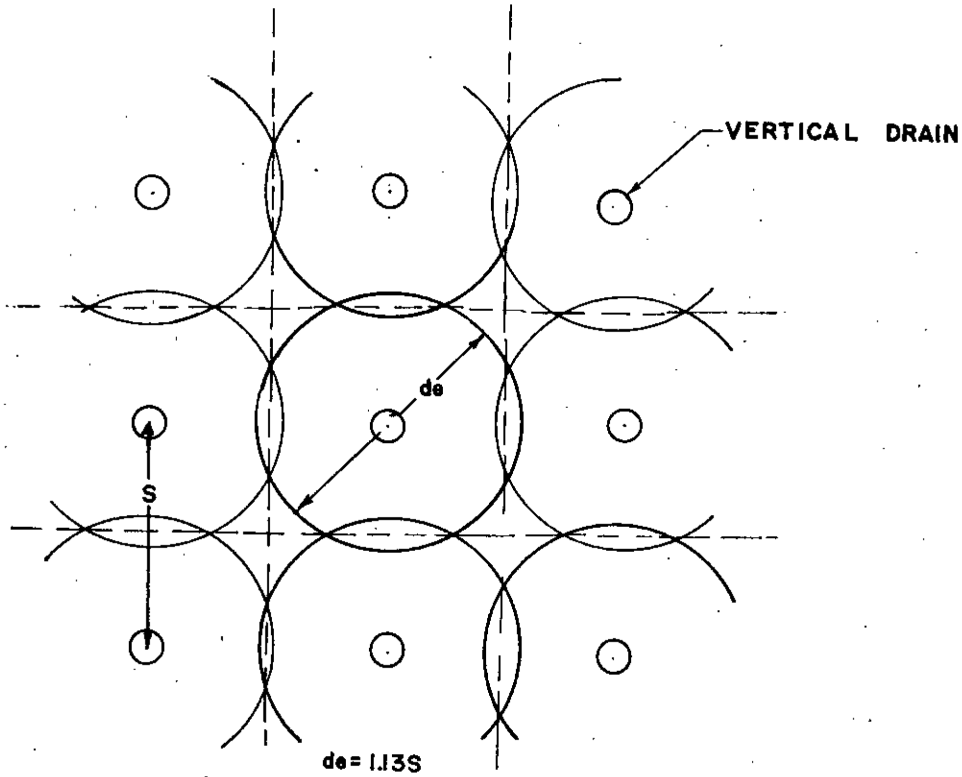
Where: μ_{oc}= the pore pressure correction factor for overconsoliated state for A_f = 0.3 to 0.5

μ_{cp}= the pore pressure correction factor for nomally consolidated state for A_f = 0.5 to 1.0

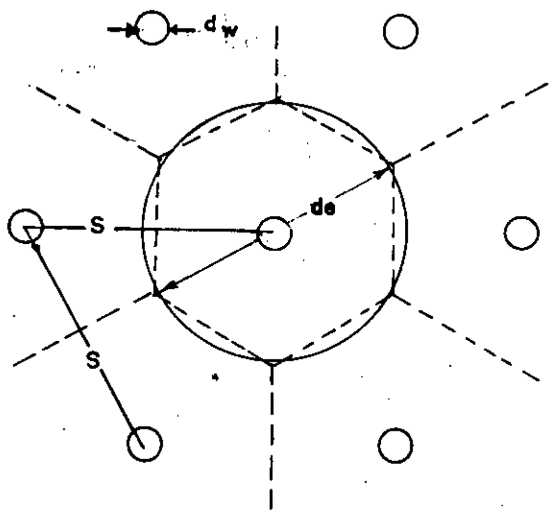
2.9 Theoretical Considerations For Design of Drains

In the design of drains , the following factors are generally taken into consideration.

Equivalent Diameter of a Band-shaped drain -- The vertical drains were assumed to have circular cross-section. A band shaped drain, therefore, has to be converted into a circular cylindrical drain producing the same effect on the consolidation process as the band shaped drain. According



(a) SQUARE PATTERN



(b) TRIANGULAR PATTERN

Fig. 2.3 Relationship of Drain Spacing to Drain Influence Zone (a) Square Pattern (b) Triangular Pattern.

to original idea in this matter by KJELLMAN (1948) and verification by finite element analysis (HANSBO, 1979), it was found that the band shaped and circular drains will lead to parctically the same consolidation process provided their circumferences are same. Accordingly, the equivalent diameter dw of a band shaped drain with width "a" and thickness "b" is considered, Fig.2.2 ,and can be express by :

$$dw = 2(a+b)/\pi \qquad (2.36)$$

Drain pattern -- Vertical drains are installed either in square or triangular pattern. In principle the triangular drain spacing is the most economical (BARRON, 1948; KJELLMAN, 1948) and the degree of deviation from truely square or triangular patterns will have a much more appreciable effect on the results. For instance, calcuting the consolidation time for a square pattern, as opposed to a triangular, results in an increase of consolidation time by about 20 % (FELLENIUS, 1981)* . On the other hand, for equal consolidation times, the calculated number of drains per unit area is the same in both patterns. The square and triangular patterns are shown in Fig. 2.3 .

Dischage Capacity of Different Drains -- In case of prefabricated drains the discharge capacity will be a function of lateral earth pressure against the drain sleeve. Thus. the filter will be partly squeezed into the channel system of the core by the pressure of the surrounding

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* Cited by Singh(1986)

soil and this will doubtlessly reduced the channel volume and consequently the discharge capacity. Also the fines on the pore water may not be retained by filter and may therefore cause a gradual decrease of discharge capacity or even clogging. This problem of reduced discharge capacity is known as well resistance.

Drainage Blanket -- The primary function of providing drainage blanket over the area where the drains are to be installed is to provide a free draining outlet for the water discharged from the drains as shown in Fig. 3.7. This also can provide a suitable working platform for heavy equipments which may otherwise be unable to gain access to the area.

Soil Drainage Prameters -- The accuracy of any design method is obviously limited by the accuracy with which the soil drainage parameters can be assessed. It is evident that the correct methods of site investigations carried out under strict supervision will enable a more appropriate design to be undertaken . The important soil drainage parameters are C_v , C_h , K_v . Both laboratory and insitu tests should be carried out to obtain relevant soil parameters.

2.10 Use of Vertical Drains in Bangladesh

Although the application of soil improvement technique in Bangladesh with vertical drains and preloading, started only a few years ago, the recent development of vertical drains has increased its use significantly.

In Bangladesh, vertical drains were successfully installed to some projects to treat soft clayeyey soil to a depth down to several metres by the Public Works Department. application of soil improvement by using vertical drains and preloading were successfully completed by the Foundation Consultants Ltd. (1986) to several projects in Bangladesh.

3 STATEMENT OF THE PROBLEM

3.1 Introduction

The Public Works Department (P.W.D.) of Bangladesh undertaken the construction project of District Headquarter complex buildings of Sunamgonj. in 1986 . Subsurface investigation was carried out at the project site. The investigation reveals that the subsurface soils are too weak to support a economic spread foundation at shallow depth. And accordingly, a programme was undertaken to improve the soil conditions by using the method vertical of drains and preloading.

In the following sections the project site, the sub surface investigation programm including the related loboratory soil tests, and the method used for the soil improvement purpose are briefly outlined.

3.2 Project Site

Sunamgonj is located about 250 kms north east of Dhaka city. A plan of the construction site is shown in Fig. 3.1 .

3.3 Soil profile

Foundation Consultants (1987) reported that geologically the project site lies on a syncline of folded hills of Sylhet district. The surface rock of the locality is composed of reddish to brownish gray silt and clay with sand, which is older than the adjacent alluviums. Probably the surface rock is of Dupi Tila formation . Altitude of the project area rises to 15m to 18m.

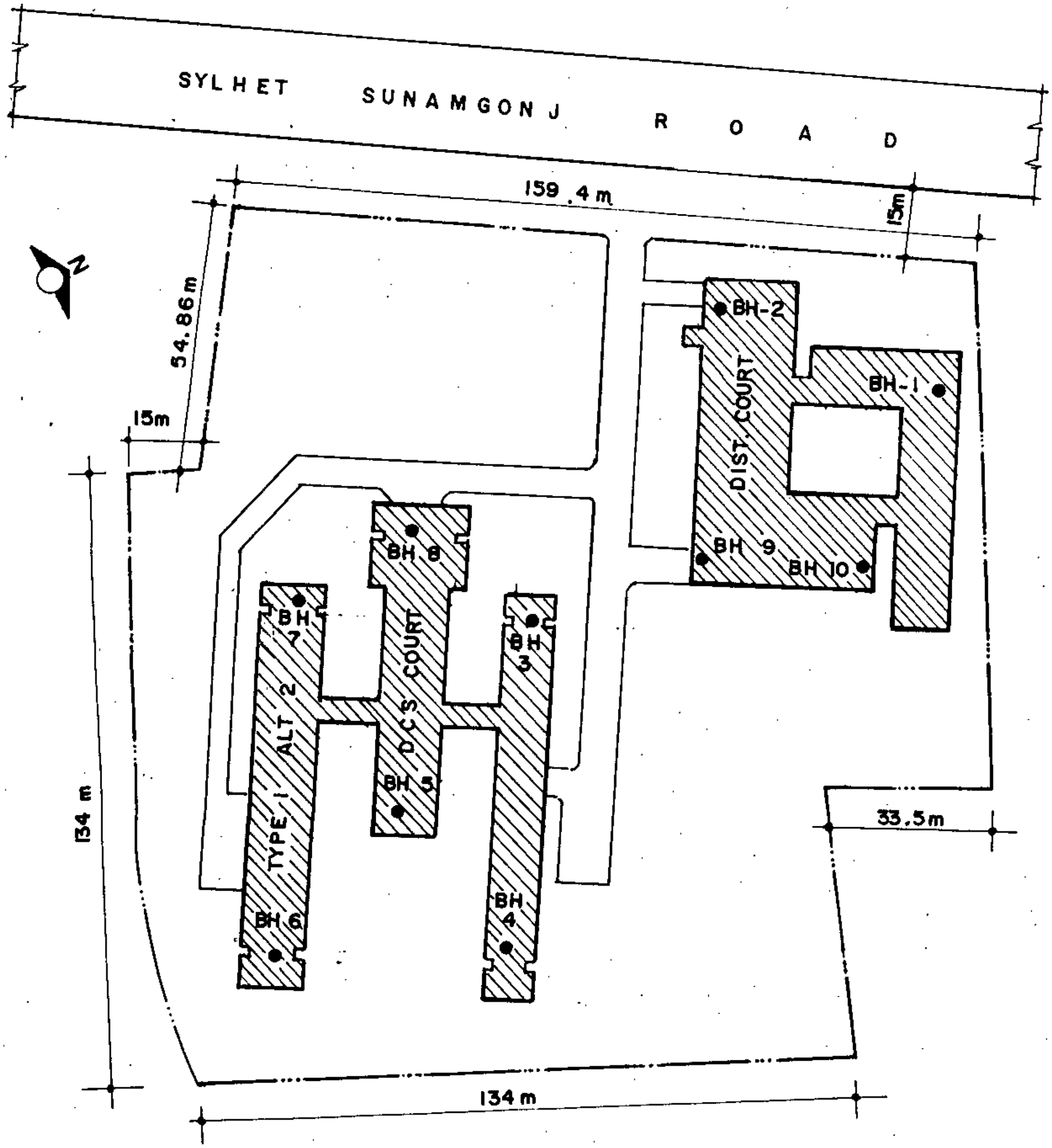


Fig. 3.1 Site Plan of Project Area with Bore Hole Locations.

3.4 Ground water table

The ground water table varies with the season, during the rainy season it lies about 250mm below the ground surface, while during dry season it goes down to 2.0m below the ground surface . An average value of 1.5 m below the ground surface may be considered for the groundwater table.

3.5 Sub-soil Condition

Foundation Consultants (1987) and Geotechnical Engineers and Consultants (1986) performed , in favour of P.W.D. the soil investigations both in the field and in the laboratory, to evalutate the mechanical , physical and engineering properties of the sub soil with a view to recommend the safe and economical foundation design for the proposed structure.

(a) Boring -- To collect necessary information 10 numbers of exploratory borings were done at the site, as shown on the location plan, Fig. 3.1 . The bore holes were drilled down to a depth of 15m from ground level using 100mm diameter casing.

(b) Standard Penetration Test -- The standard penetration tests were performed at every 1.5 m interval in all the bore holes. The tests were executed by using a split spoon samplers of 35 mm internal and 50 mm outer diameter, and a 63.5 kg hammer falling freely from a constant height of 750 mm on

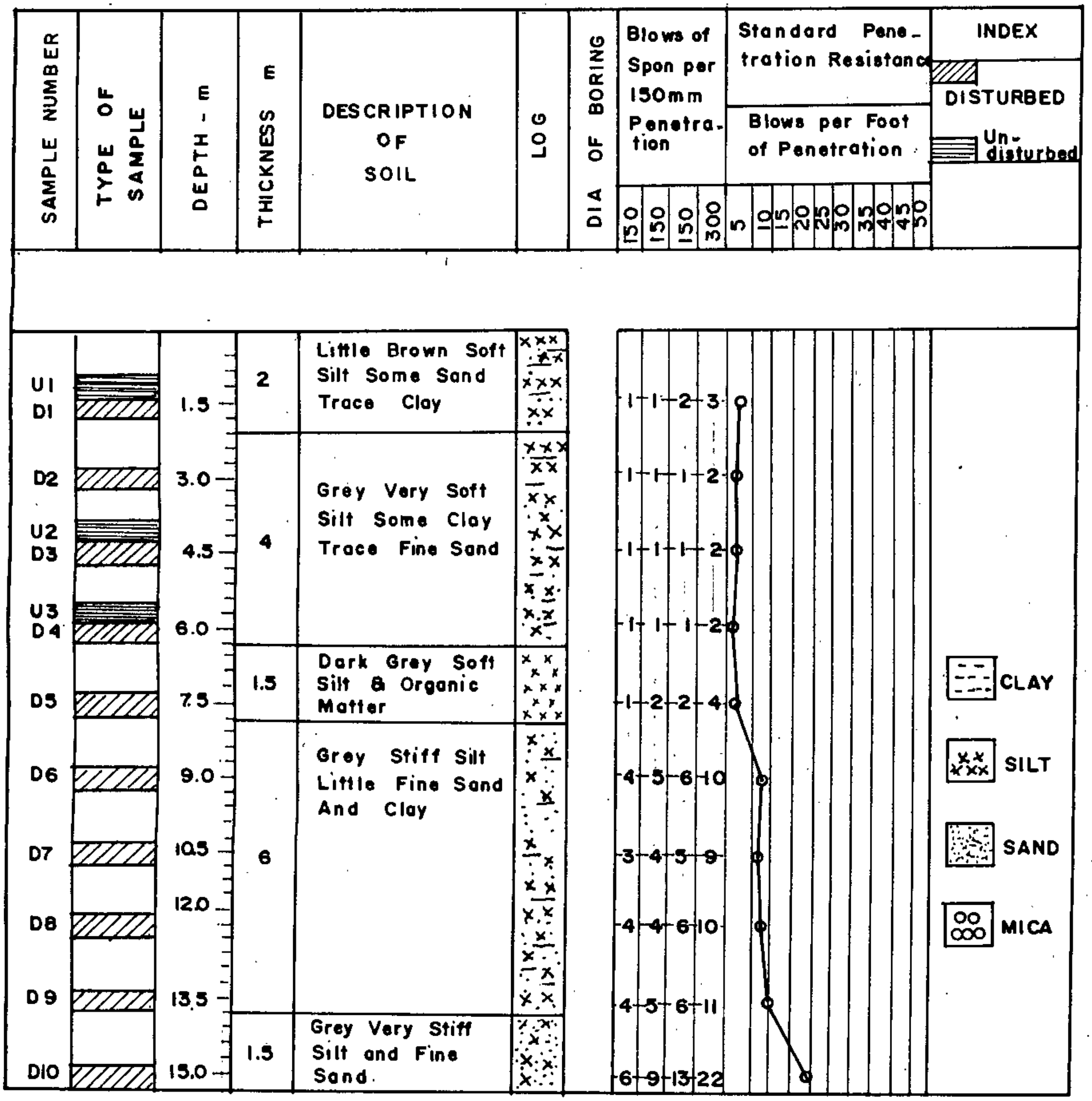


Fig 3.2a Typical Bore Hole Log (D.C'S Court, BH - 1)

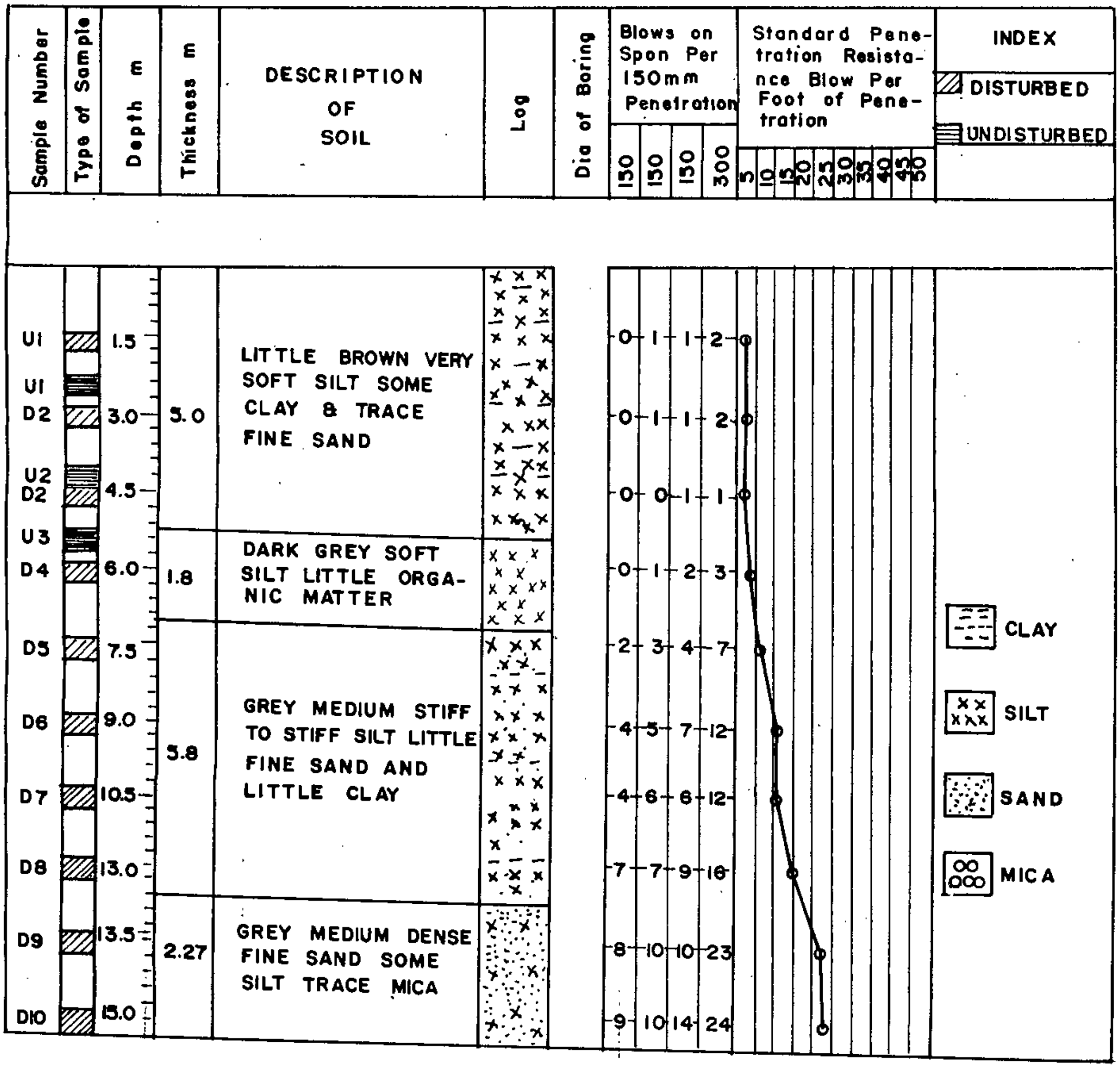


Fig. 3.2b Typical Bore Hole Log (D. J.'S Court, BH – 3)

to the drill rod. The N-values are shown on the bore charts. Fig.3.2 shows typical bore hole log with N-values.

(c) Collection of Disturbed Soil Samples -- Disturbed soil samples were collected at different depths. These soil samples were duly classified in order to reconstruct a depth wise stratification chart of bore-holes and to evalute the over all subsoil condition of the investigated site. The disturbed soil samples were preserved in tight polythene bags and were taken to the laboratory for detailed analysis.

(d) Collection of Undisturbed Soil Samples -- Disturbance to the soil alter its physical and mechanical properties and hence it is necessary to collect undisturbed soil samples to perform laboratory tests in order to obtain soil parameters , which eventually lead to the evaluation of bearing capacity and settlement characteristics of the subsoil layers. 48 numbers of undisturbed soil samples were collected from the bore holes with the help of 75 mm diameter Shelby tubes. The depth of collection of samples have been shown on the bore charts in Figs. 3.2 . Immediately after the extraction of the undisturbed soil samples, the two ends of the Shelby tubes were sealed with wax and were taken to the laboratory.

Laboratory Tests -- Different laboratory tests were performed on both disturbed and undisturbed soil samples collected from the site to evaluate the properties for foundation design.

The following tests were performed:

1. Grain size analysis
2. Specific gravity tests
3. Atterberg limit tests
4. Moisture content
5. Unconfined compression tests
6. Consolidation tests
7. Density tests
8. Direct shear tests

Grain size analysis , atterberg limit and specific gravity tests were carried out to ascertain the detailed composition of the soil. These tests also help in classifying the soil properly for geotechnical interpretation. Details of properties of soils are summerized in Tables 3.1 and 3.2 . Typical grain size distribution curves are shown in Fig.3.3.

Unconfined compression, density and direct shear tests provide data on the shear strength characteristics of the sub-soil, from which bearing capacity of the underlying soil can be estimated . Details of the engineering properties of soils are summerized in Table 3.3 . The typical unconfined compression test, consolidation test and direct shear test results are presented in Figs. 3.4, 3.5 and 3.6 .

Table 3.1 : Properties of Soil (Before improvement)

Bore No.	Sample No.	Depth (m)	Grain Size Analysis			Atterberg Limits			Specific Gravity
			Sand %	Silt %	Clay %	L.L.	P.L.	P.I.	
1	D1	1.4-1.8	36	56	8	24	17	7	2.63
	U2	4.0-4.4	9	67	24	38	24	14	2.66
	D6	9.0-9.5	19	64	17	32	21	11	2.65
	D10	12.3-12.5	52	44	4	-	-	-	-
2	U1	2.4-2.9	11	74	15	30	20	10	2.64
	D4	5.9-6.4	7	80	13	31	22	9	2.62
	D7	10.5-11.0	29	62	9	27	19	8	2.63
	D9	13.6-14.0	34	60	6	-	-	-	-
3	U1	2.1-2.6	10	64	26	40	25	15	2.67
	D3	4.4-4.9	7	71	22	37	25	12	2.66
	D8	12.0-12.5	19	65	16	35	24	11	2.64
	D10	15.1-15.5	62	38	0	-	-	-	-
4	D1	1.4-1.8	6	67	27	39	23	16	2.65
	U1	3.7-4.1	8	68	24	42	28	14	2.68
	U3	9.8-10.2	9	70	21	36	25	11	2.63
	D9	13.6-14.0	16	65	19	-	-	-	-
5	D1	1.4-1.8	5	70	25	44	28	16	2.68
	U1	2.4-2.9	7	67	26	47	32	15	2.67
	D5	7.5-7.9	12	66	22	40	27	13	2.65
	D8	12.5-12,5	15	67	18	-	-	-	-

Table 3.1 : Properties of soil (Before improvement)

Bore No.	Sample No.	Depth (m)	Grain Size Analysis			Atterberg Limits			Specific Gravity
			Sand %	Silt %	Clay %	L.L.	P.L.	P.I.	
6	D1	1.4-1.8	11	65	24	34	22	12	2.63
	U1	4.0-4.4	6	67	27	48	31	17	2.69
	U3	8.5-9.0	13	70	17	31	21	10	2.64
	D10	15.0-15.5	19	69	12	-	-	-	-
7	D2	2.9-3.4	6	69	25	42	25	17	2.64
	U2	5.5-5.9	8	66	26	47	29	18	2.68
	D7	10.5-11.0	11	70	19	36	23	13	2.65
	D9	13.6-14.0	34	60	6	-	-	-	-
8	U1	0.9-1.4	9	68	23	39	27	12	2.66
	D6	9.0-9.5	16	64	20	33	23	10	2.65
	D8	12.0-12.5	24	58	18	30	21	9	2.62
	D10	15.0-15.5	38	62	0	-	-	-	-
9	U1	2.4-2.9	9	65	26	41	27	14	2.66
	U3	5.2-5.6	6	70	24	38	26	12	2.64
	D6	9.0-9.5	8	73	19	30	21	9	2.62
	D10	15.0-15.5	60	38	2	-	-	-	-
10	D1	1.4-1.8	4	74	22	37	24	13	2.65
	U2	3.7-4.1	7	73	20	33	23	10	2.63
	D5	7.5-7.9	14	68	18	28	20	8	2.62
	D8	12.0-12.5	56	40	4	-	-	-	-

Table 3.2 : Unit Weight and Water Content of Soil (Before improvement)

Bore Hole	Sample No.	Depth (m)	Water Content %	Bulk Unit Weight kN/m³	Dry Unit Weight kN/m³	Saturated Unit Weight kN/m³
1	U1	1.0-1.5	19-20	17	15	
	U2	4.0-4.4	24-25	18	14	18
	U3	4.5-5.0	37-38	17	12	
2	U1	2.0-2.5	20-22	17	14	
	U2	4.0-4.5	26-27	18	14	18
	U3	7.0-7.5	53-54	14	9	
3	U1	2.0-2.5	24-25	17	14	
	U2	4.0-4.5	32-34	16	12	18
	U3	5.0-5.5	48-50	15	10	
4	U1	3.7-4.1	22-24	18	14	
	U2	5.2-5.6	44-47	16	11	18
	U3	9.8-10.2	20-23	19	15	
5	U1	2.1-2.6	28-30	16	13	
	U2	4.0-4.5	58-60	15	9	17
	U3	5.2-5.6	53-55	16	10	

Table 3.2 : Unit Weight and Water Content of soil (Before improvement)

Bore Hole	Sample No.	Depth (m)	Water Content %	Bulk Unit Weight kN/m^3	Dry Unit Weight kN/m^3	Saturated Unit Weight kN/m^3
6	U1	4.0-4.5	24- 25	17	14	18
	U2	7.0-7.5	55-58	15	10	
	U3	8.5-9.0	23-26	18	14	
7	U1	2.1-2.6	27-29	17	13	18
	U2	5.5-6.0	24-27	18	14	
	U3	8.5-9.0	40-46	17	12	
8	U1	1.2-1.5	29-31	16	12	18
	U2	4.0-4.5	54-56	15	10	
	U3	10.1-10.5	21-23	19	15	
9	U1	2.1-2.5	28-29	16	13	18
	U2	4.0-4.4	23-25	17	14	
	U3	5.5-6.0	59-62	14	9	
10	U1	2.1-2,5	25-28	17	13	18
	U2	4.0-4.5	28	17	14	
	U3	5.5-6.0	45-56	15	11	

Table : 3.3 : Engineering Properties of Soil (Before improvement)

Bore No.	Sample No.	Depth (m)	Consolidation: Void Ratio e_o	Consolidation: Comp. Index C_c	Uncon. Comp. Test: Uncon. Comp. Strength kN/m²	Uncon. Comp. Test: Strain at Failure %	Direct Shear Test: Cohesion KN/m²
1	U1	1.0-1.5	0.78	0.18	29	7	14
	U2	4.0-4.5	0.84	0.20	24	9	12
	U3	7.0-7.5	0.98	0.26	26	13	10
2	U1	2.0-2.5	0.80	0.19	21	12	13
	U2	4.0-4.5	0.88	0.21	24	15	17
	U3	7.0-7.5	1.30	0.35	10	7	7
3	U1	2.0-2.5	0.77	0.17	22	14	10
	U2	4.0-4.5	0.89	0.20	16	10	7
	U3	5.0-5.5	1.15	0.30	14	8	5
4	U1	3.7-4.1	0.80	0.20	33	12	14
	U2	5.2-5.6	1.40	0.40	22	8	9
	U3	9.8-10.2	0.69	0.14	83	7	36
5	U1	2.1-2.6	0.90	0.22	14	16	9
	U2	4.0-4.4	1.48	0.45	12	7	6
	U3	5.2-5.6	1.36	0.36	16	7	7

C

Table : 3.3 : Engineering Properties of Soil (Before improvement)

Bore No.	Sample No.	Depth (m)	Consolidation: Void Ratio e_o	Consolidation: Comp. Index C_c	Uncon. Comp. Test: Uncom. Comp. Strength kN/m²	Uncon. Comp. Test: Strain at Failure %	Direct Shear Test: Cohesion kN/m²
6	U1	4.0-4.5	0.76	0.17	19	14	12
	U2	7.0-7.5	1.39	0.34	29	7	11
	U3	8.5-9.0	0.68	0.13	74	10	41
7	U1	2.1-2.6	0.76	0.18	21	17	7
	U2	5.5-6.0	0.82	0.19	36	12	21
	U3	8.5-9.0	1.25	0.32	39	10	13
8	U1	1.2-1.5	0.90	0.22	12	15	7
	U2	4.0-4.5	1.44	0.38	14	8	5
	U3	10.1-10.5	0.70	0.15	97	9	55
9	U1	2.1-2.6	0.92	0.22	21	18	14
	U2	4.0-4.4	0.86	0.20	29	15	19
	U3	5.5-6.0	1.48	0.46	13	6	7
10	U1	2.1-2.6	0.81	0.19	15	16	11
	U2	4.0-4.5	0.92	0.23	37	14	16
	U3	5.5-6.0	1.42	0.42	22	8	9

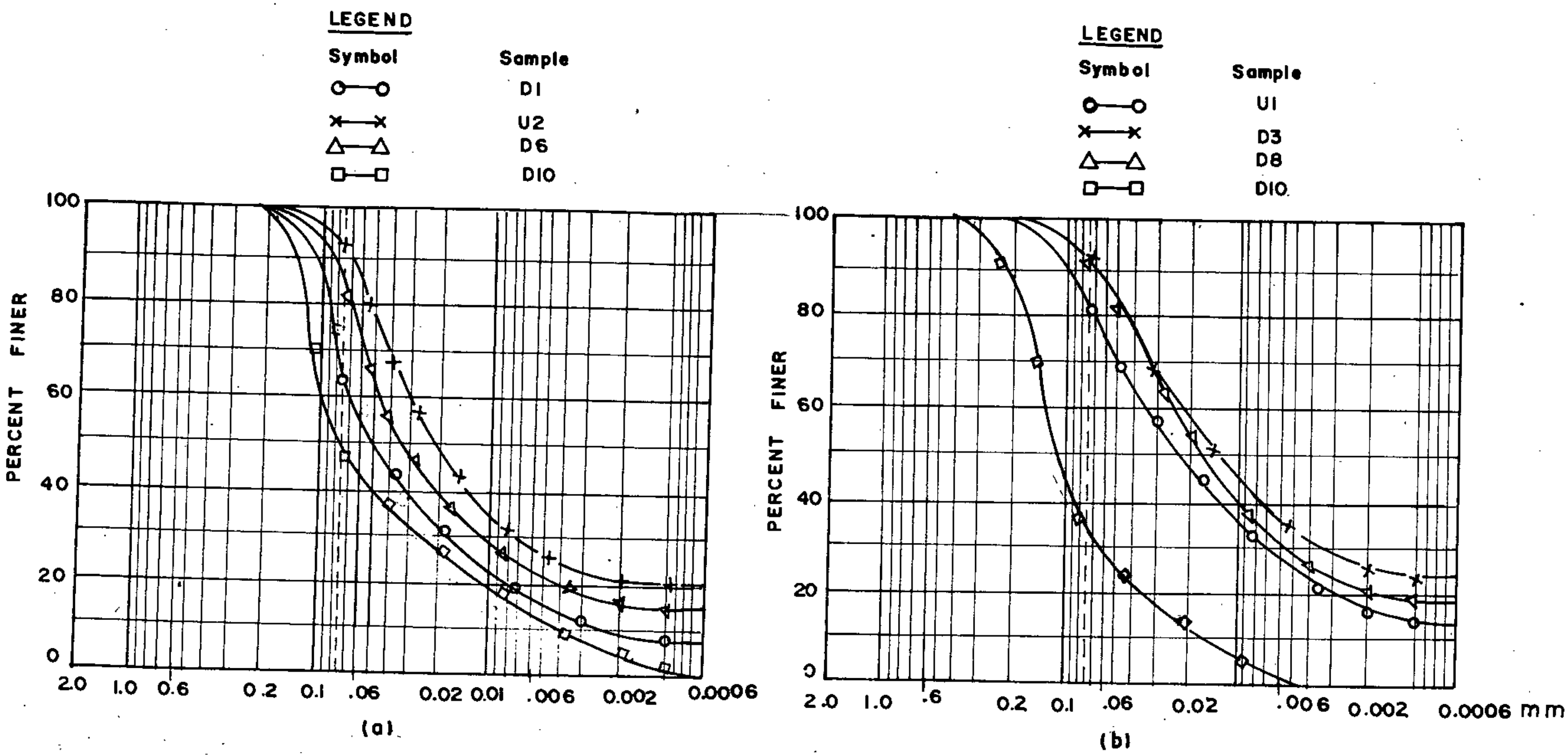


Fig. 3.3 Typical Grain Size Distribution Diagram (a) Bore Hole No. 1 (D.C.'S Court) (b) Bore Hole No. 3 (D. J.'S Court)

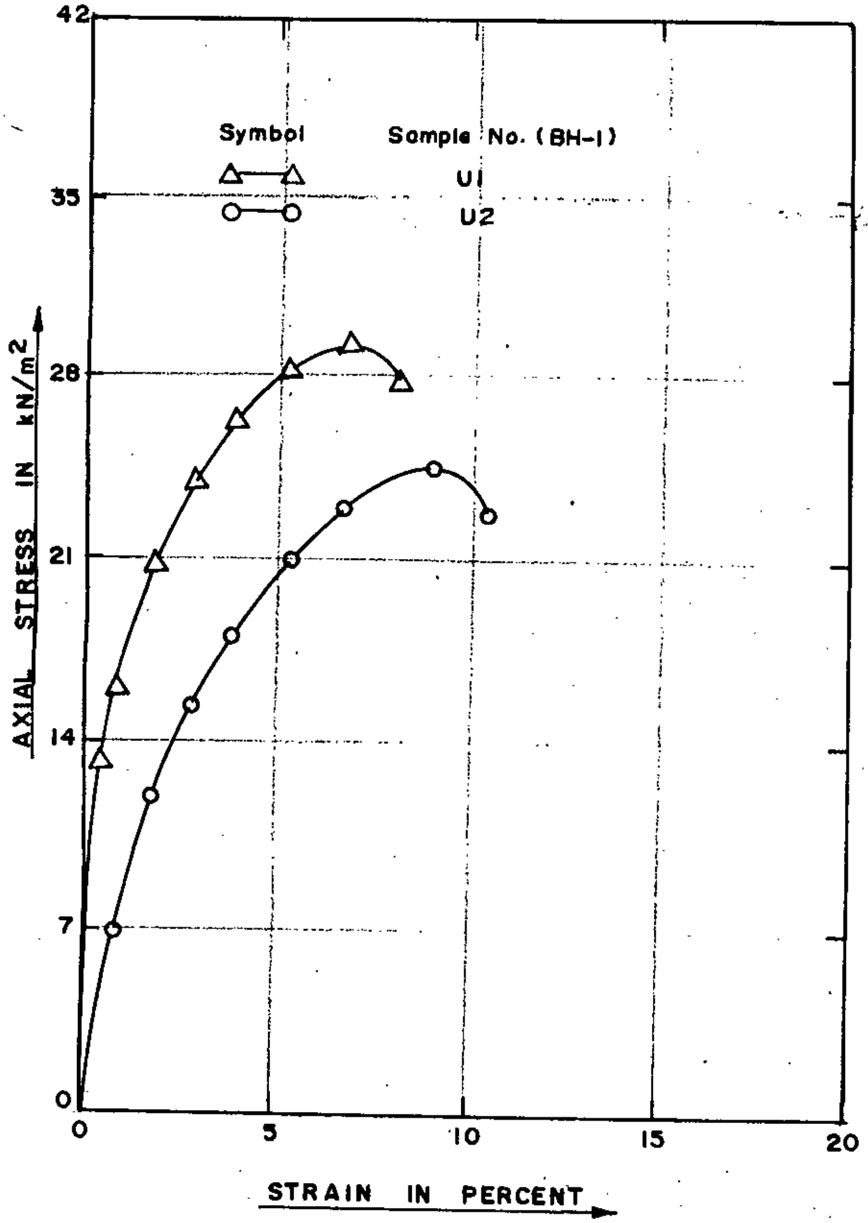


Fig. 3.4a Stress - Strain Diagram
(Unconfined Compression Test)

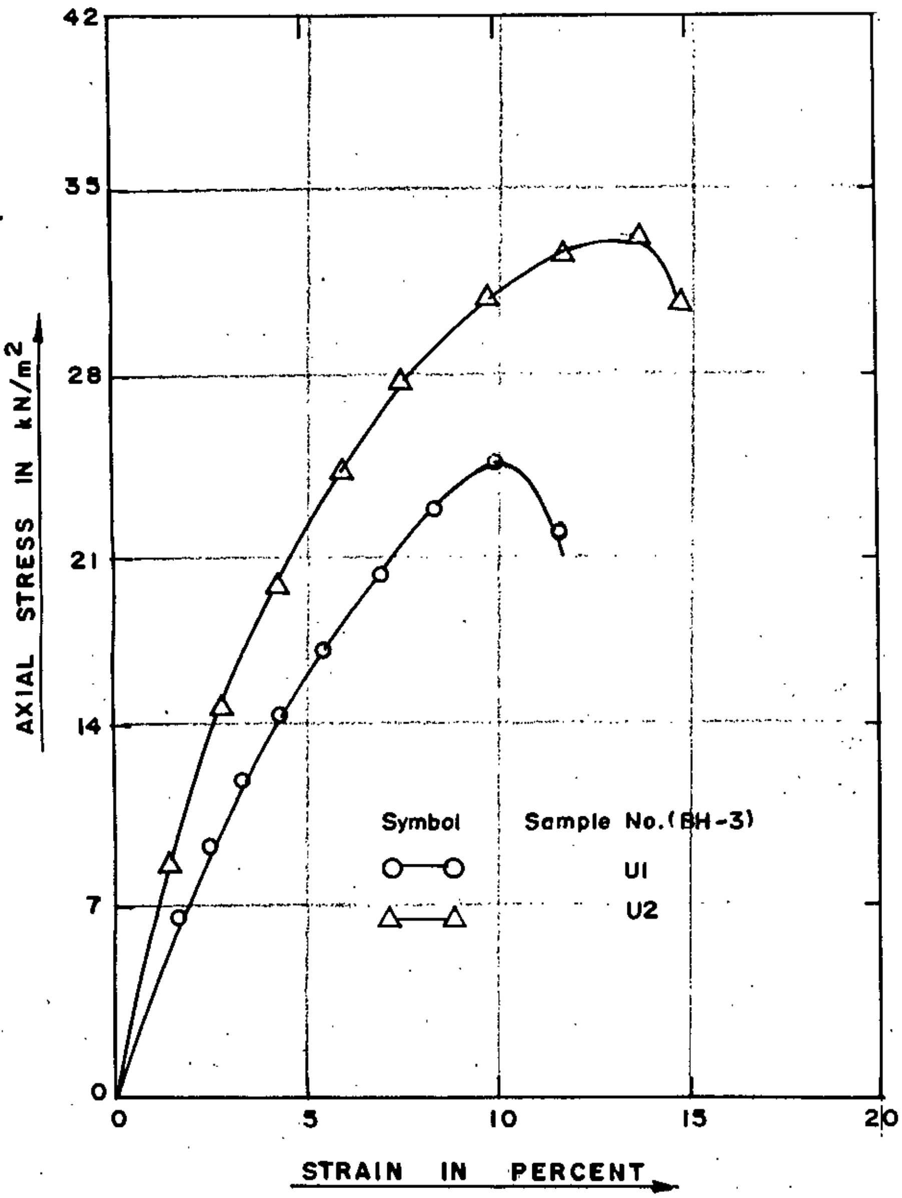
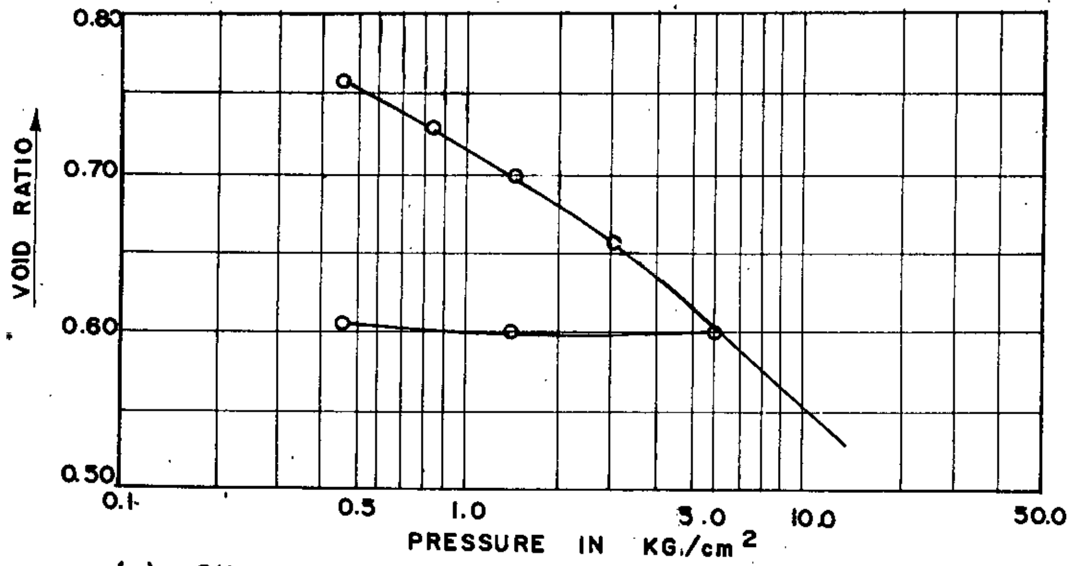
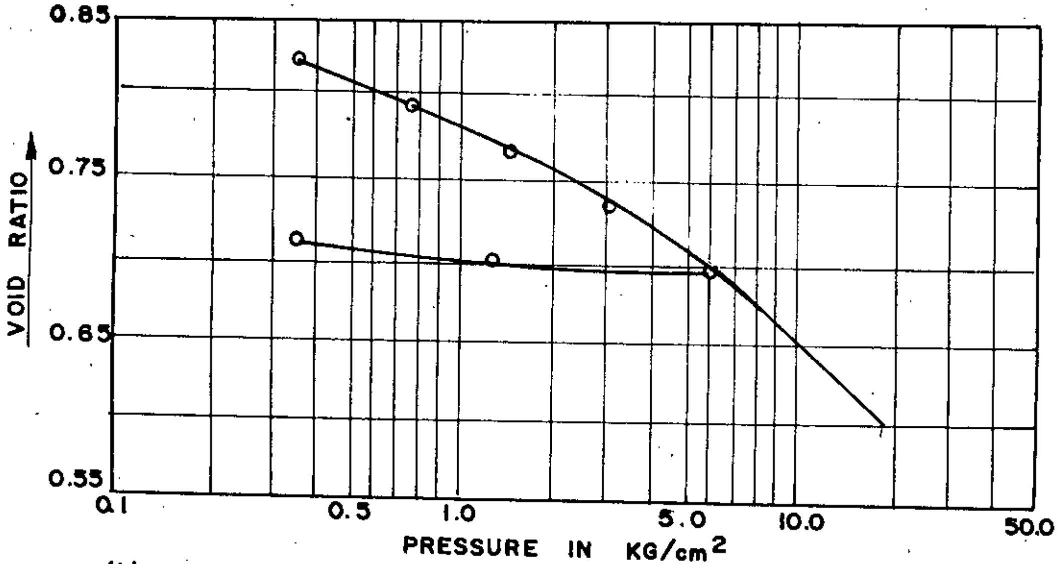


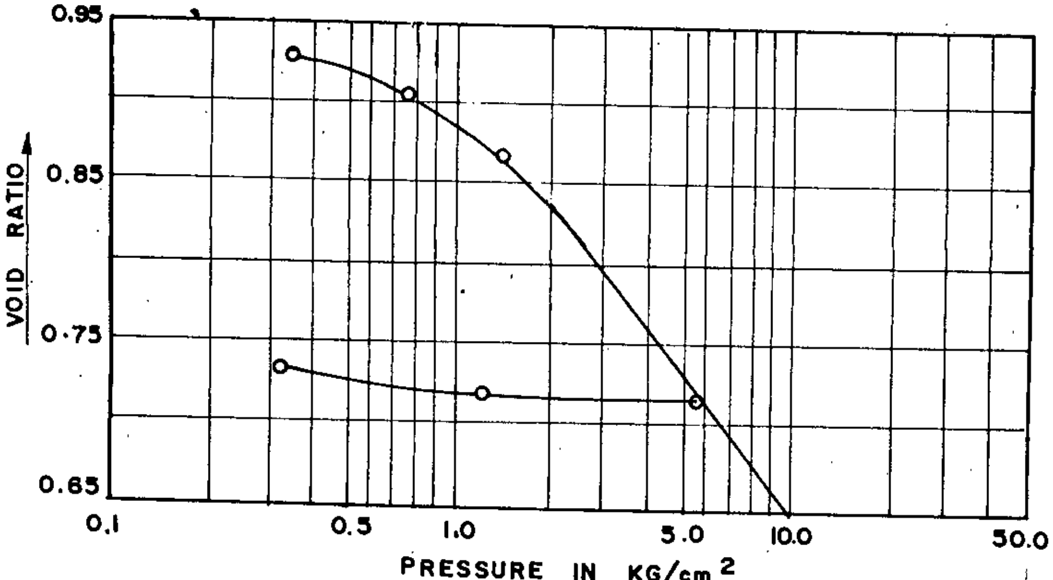
Fig. 3.4b Typical Stress – Strain Diagram
(Unconfined Compression Test)



(a) BH - I, SAMPLE - UI



(b) BH - I, SAMPLE - U2



(c) BH - I, SAMPLE - U3

Fig. 3.5a Typical e - logp Curve (Consolidation Test)

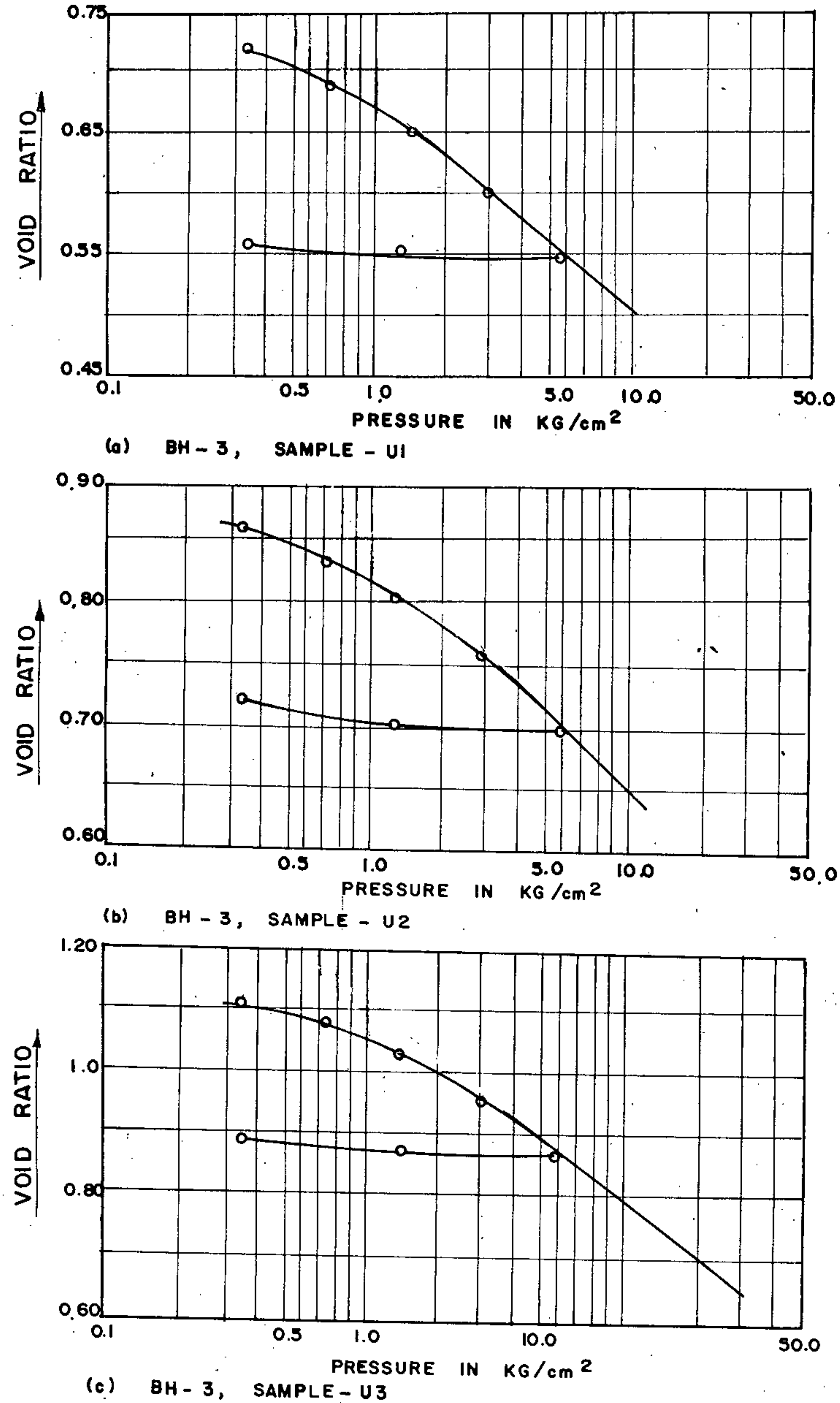
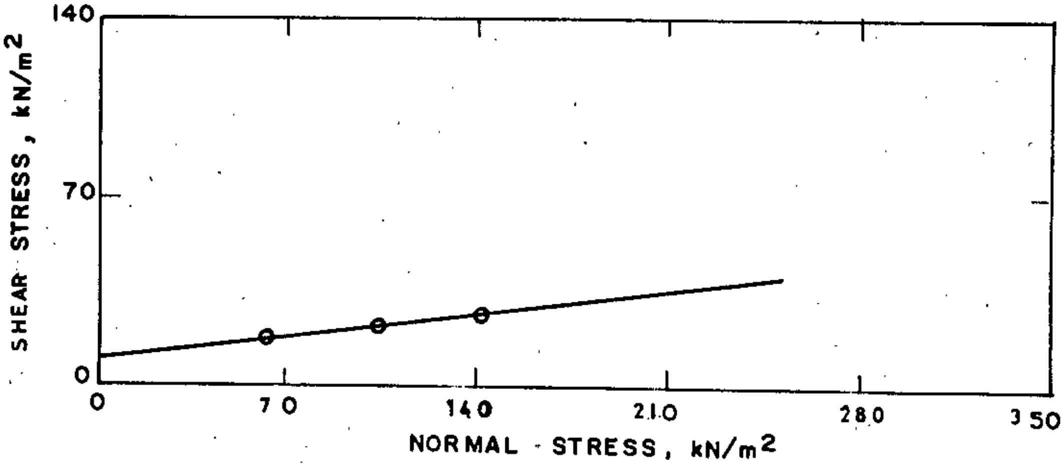
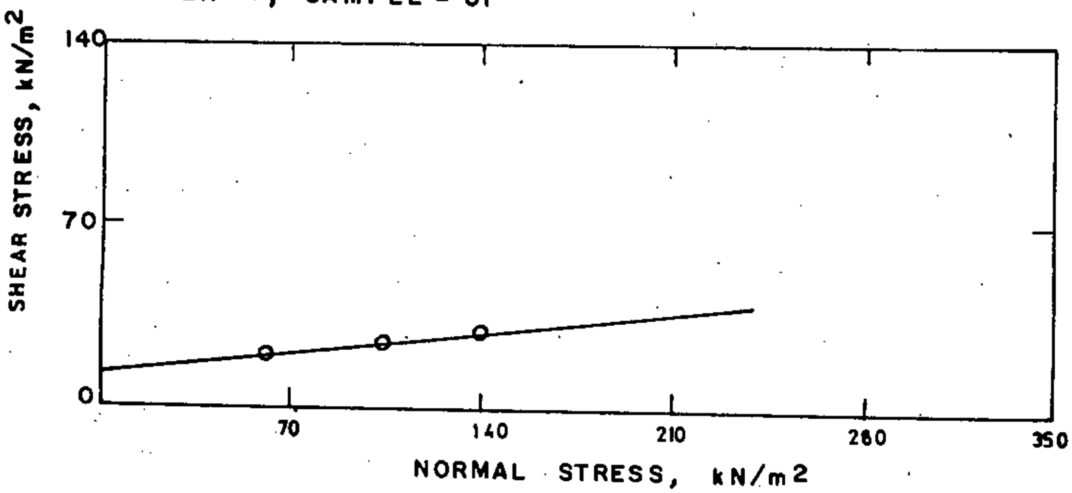


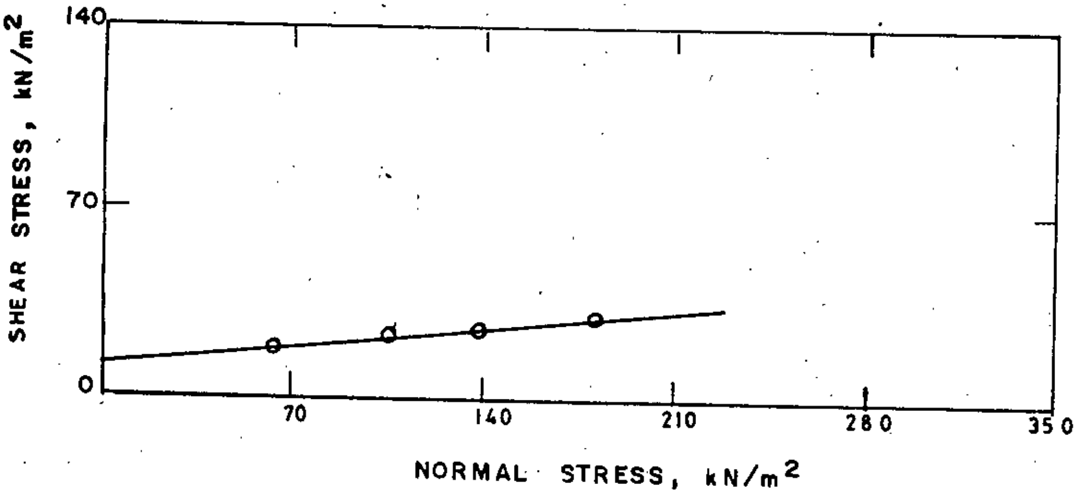
Fig. 3.5b Typical e - logP - Curve (Consolidation Curve)



(a) BH - I, SAMPLE - UI

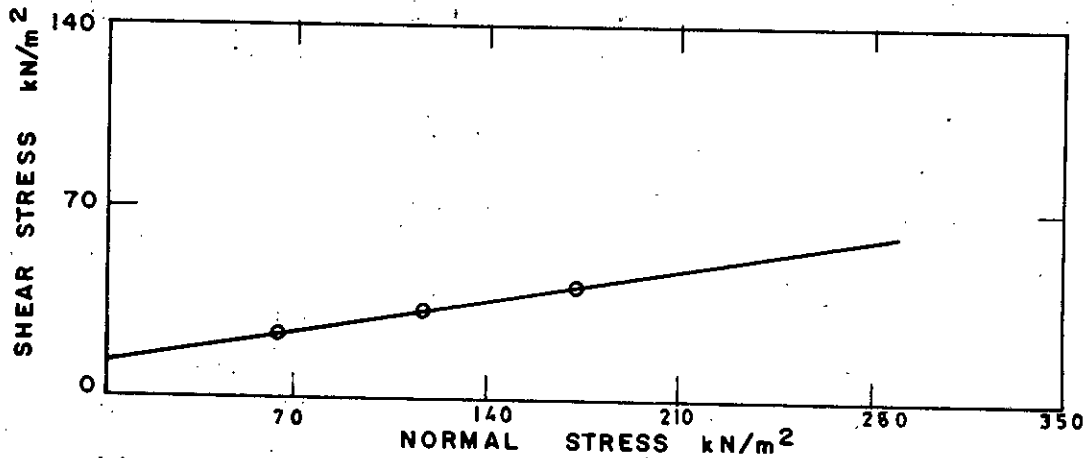


(b) BH - I, SAMPLE - U2

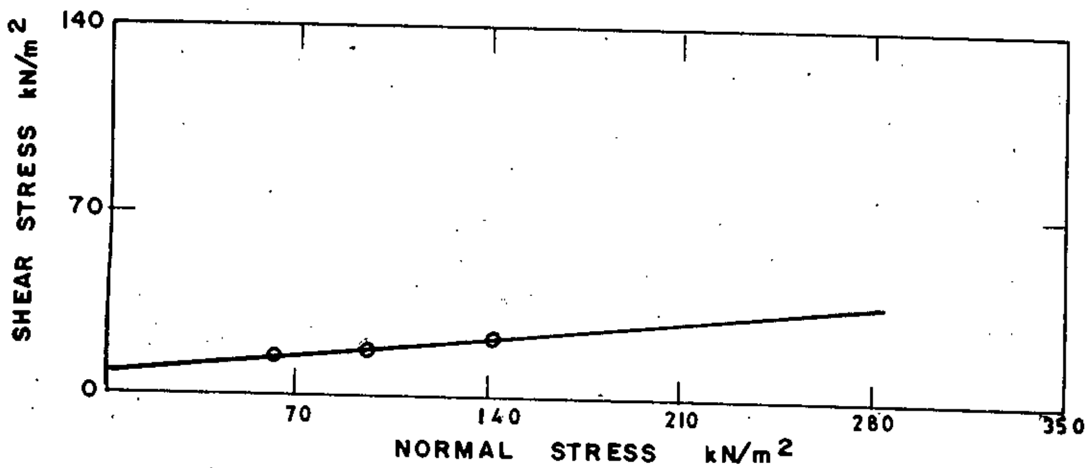


(c) BH - I, SAMPLE - U3

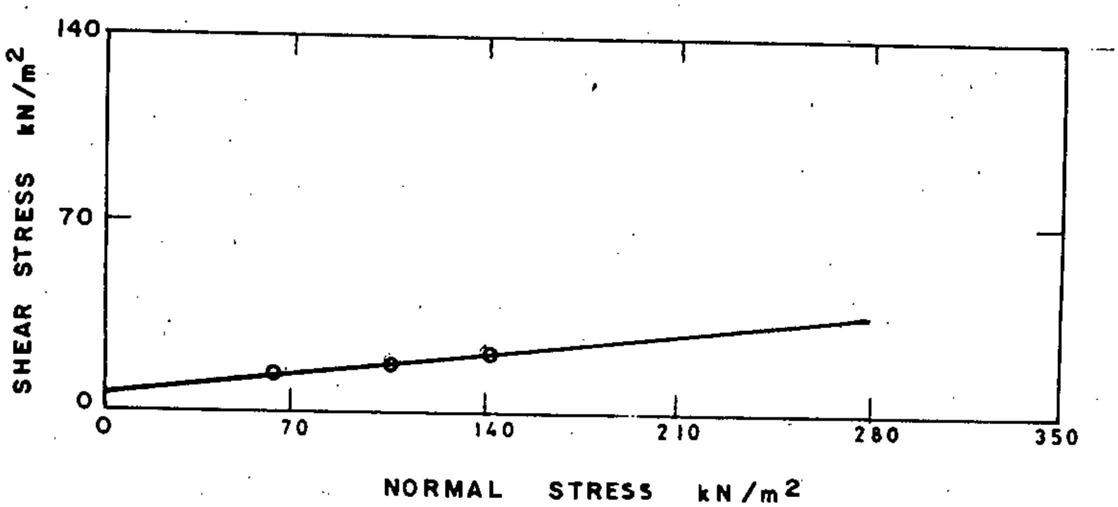
Fig. 3.6a Typical Normol Stress, Sheor-Stress Relation (Direct Sheor Stress)



(a) BH - 3, SAMPLE - U1



(b) BH - 3, SAMPLE - U2



(c) BH—3, SAMPLE - U3

Fig. 3.6b Typical Normal Stress, Shear Stress Relation.
(Direct Shear Stress)

3.6 Bearing Capacity of Soil

The top soil up to 7.5m depth comprises of very soft to soft clay silt and soft layer of organic materials. The average bearing capacity at the site was found in the range of 16 - 156 kN/m^2 (F.S = 3) upto a depth of 4.5 m. The settlement analysis shows a vertical settlement in the order of 320mm. The site seemed to be unsuitable for a spread foundation at shallow depth unless any proper treatment of the soil at the site is done. And as such, the vertical drains and preloading method being common in Bangladesh was decided for the purpose.

3.7 Installation of Vertical drains

3.7.1 Prepration of Ground for Drain Installation -- The preparation was carried out in two stages, (1) removal of top soil and (2) construction of drainage blanket.

(1) Removal of top soil -- In order to break the arching effect of the top soil, earth upto a depth of about 2m of the entire area of the proposed building was scooped out.

(2) Construction of drainage blanket -- About o.75m thick granular (F.M = 2.5) drainage blanket was constructed over the excavated area where the drains were to be installed . The drainage blanket served two purposes - to provide a suitable platform for the equipment used for the drain installation and to serve as a free draining outlet for the water to be discharged from the drains.

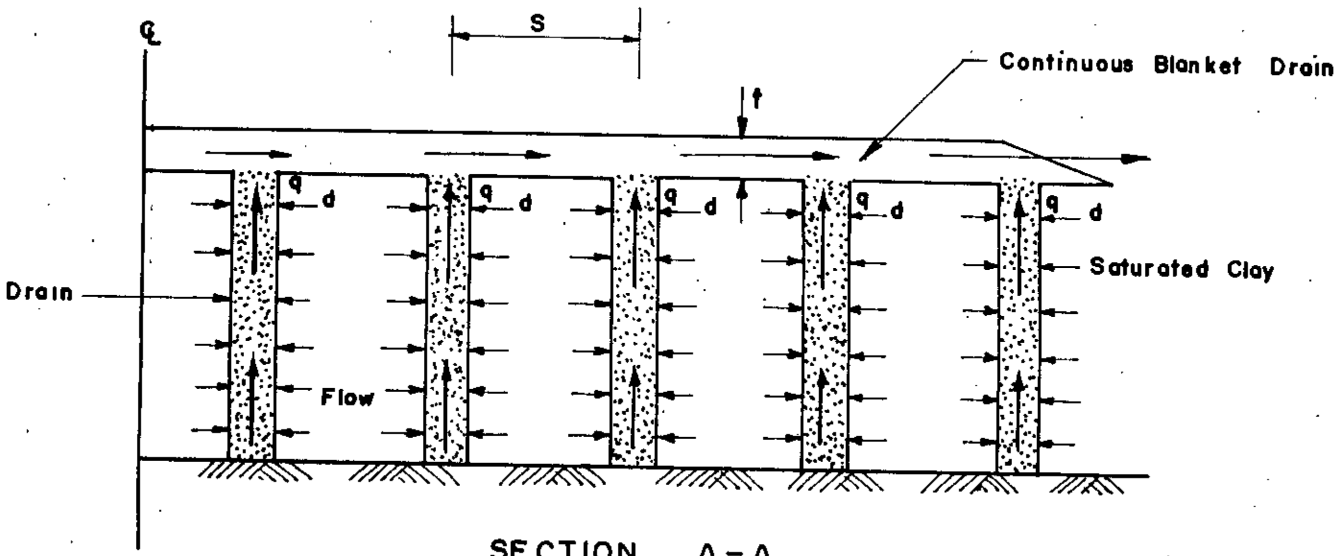
3.7.2 Spacing, Pattern, Diameter and Depth of Drains - According to the design as presented in Appendix A, sand drains were installed in square pattern at 1.77 m centres. The layout of the vertical drains are shown in Fig. 3.7. The 200 mm diameter sand drains were installed to a depth of 7.62 m which was the full depth of soft clay at the site.

3.7.3 Installation of Drains -- The vertical drains were installed with the help of a machine , making 200mm diameter bore-holes upto 7.62 m depth , driving casing pipe, back filling the bore-holes with coarse sand (F.M=2.5) and compacting sand by monkey.

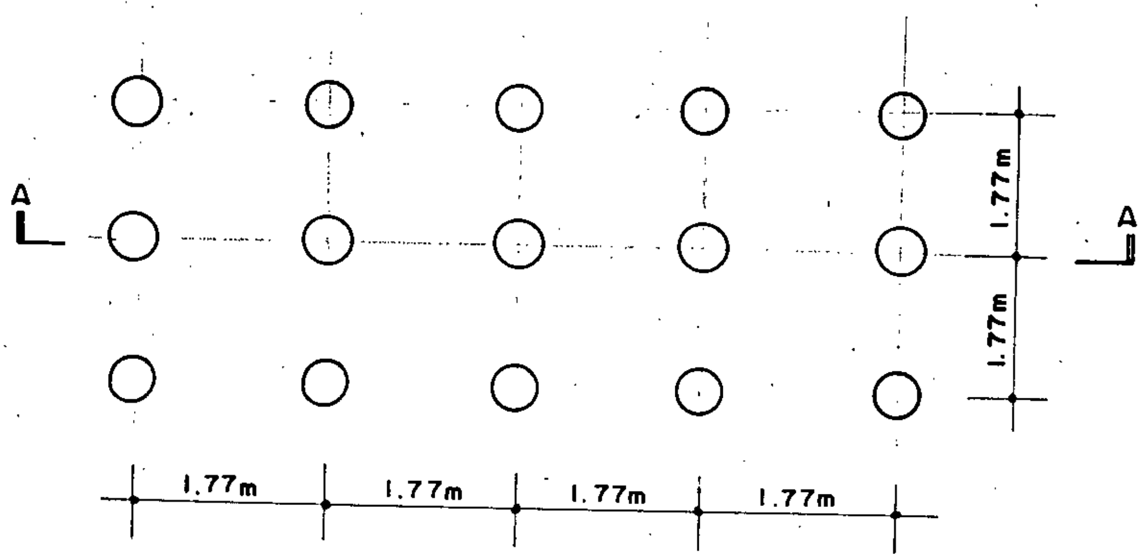
3.7.4 Rate and Time of Placement of Drains -- The rate of drain installation was 15 drains in average per day. It took more than 6 months to finish the installation work.

3.8 Preloading

The essential idea behind the concept of precompression of soft cohesive soil consists of subjecting the foundation to a load, the intensity and geometry of which is suitable to anticipate partially or entirely the settlements of the foundation .The project site was a low land area. As mentioned earlier, earth upto a depth of 2 m was scooped out from the existing ground level at the site of the proposed building. Fig. 3.8 . The vertical drains were then installed. The scooped area was then refilled with sand (F.M.=2.5) to a thickness of 0.75m. The site was then filled with local sand to a height of 2.75m.



SECTION A-A



LAYOUT OF VERTICAL DRAIN

Fig. 3.7 Plan and Section of Vertical Drain Arrangement.

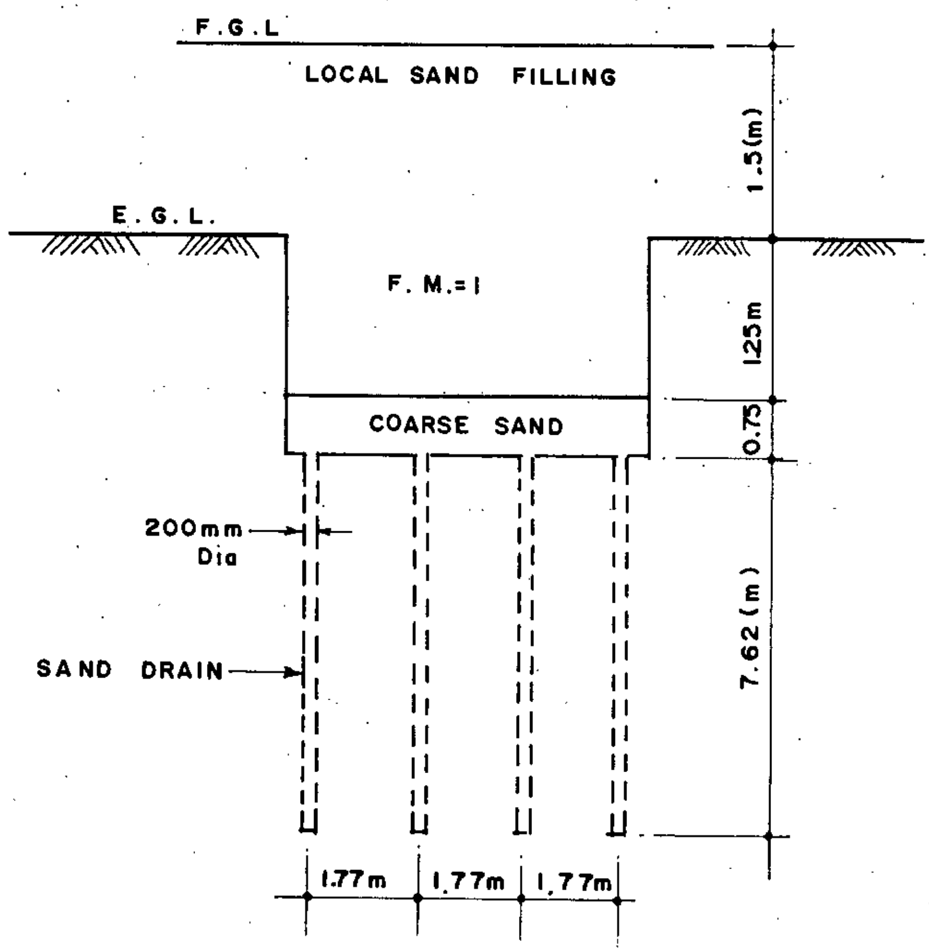


Fig. 3.8 Schematic Diagram of Vertical Drains with Blanket and Preload.

RESULTS AND DISCUSSIONS

4.1 Introduction

Subsurface soil explorations were carried out in two phases to collect the pertinent soil engineering data of the site. The first phase of the investigation was done at the preliminary stage of the project in order to obtain the information about existing soil condition. This includes ten number of boreholes at the two sites at different locations as shown previously in Figs. 3.1 and 3.2 . The The second phase of the investigation was done making thirteen boreholes at the two sites two months after the improvement treatment was initiated.

The soil results prior to treatment, as shown in Figs. 4.1 and 4.2 , show that the top layers at both the sites are composed of very soft to soft clayey silt which are normally unsuitable without any treatment, for a shallow foundation . Accordingly, the soil improvement scheme was undertaken by using sand drains of 7.62 m depth and 0.2m diameter at a spacing of 1.77 m. The arrangements of sand drains are shown in Fig. 3.7 . In the following sections the soil conditions , prior to and after improvement, are described and the results obtained , due to the treatment are discussed.

4.2 Standard Penetration Resistance

The Standard Penetration Resistance Tests were performed at every 1.5 m interval in all the bore holes. The SPT values varies from 1 to 16 upto a depth of 9 m before the improvement of soil. After improvement by vertical drains and preloading

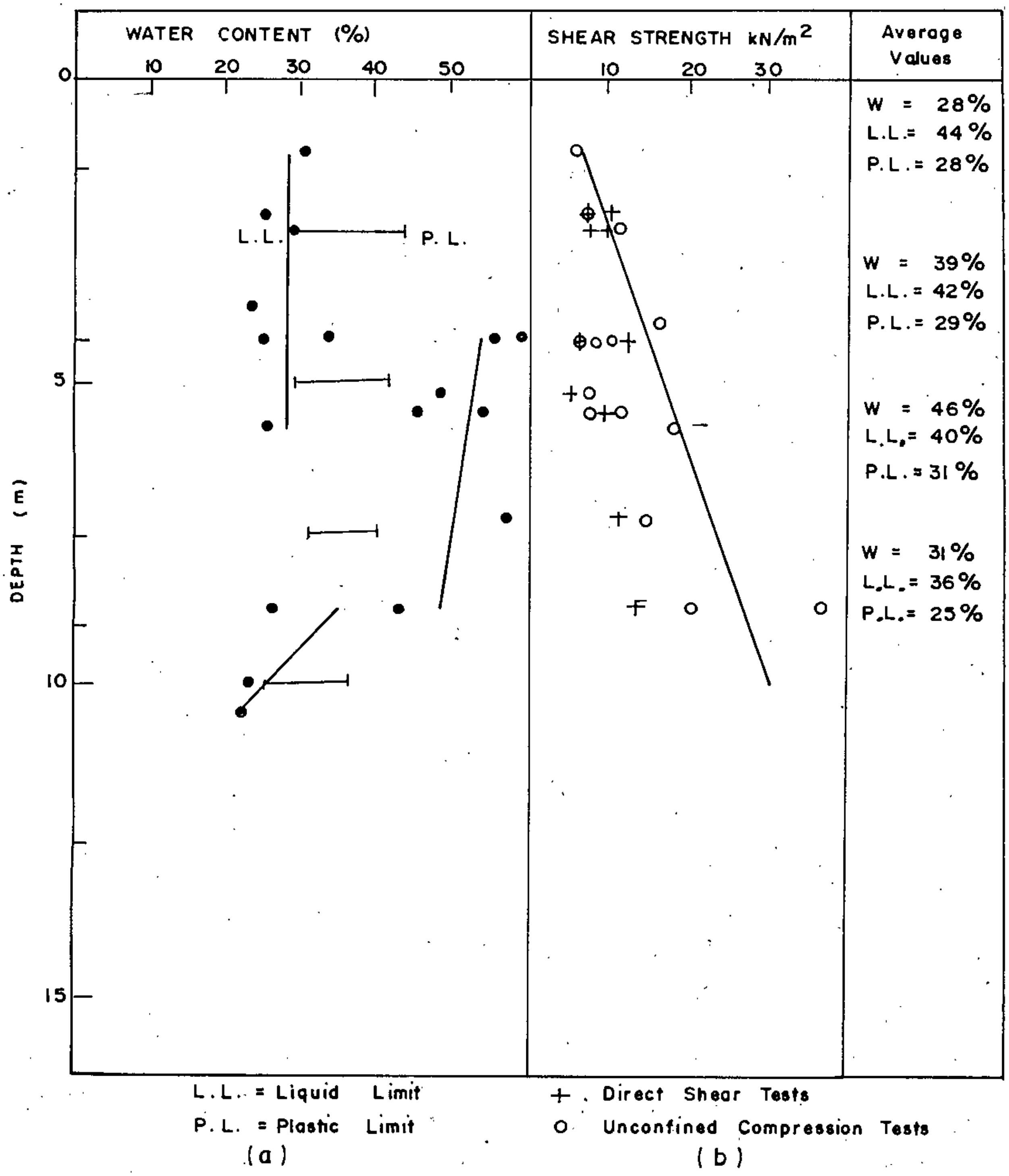


Fig. 4.1a Soil Properties of D. C's Court (a) Water Content.
(b) Shear Strenth.

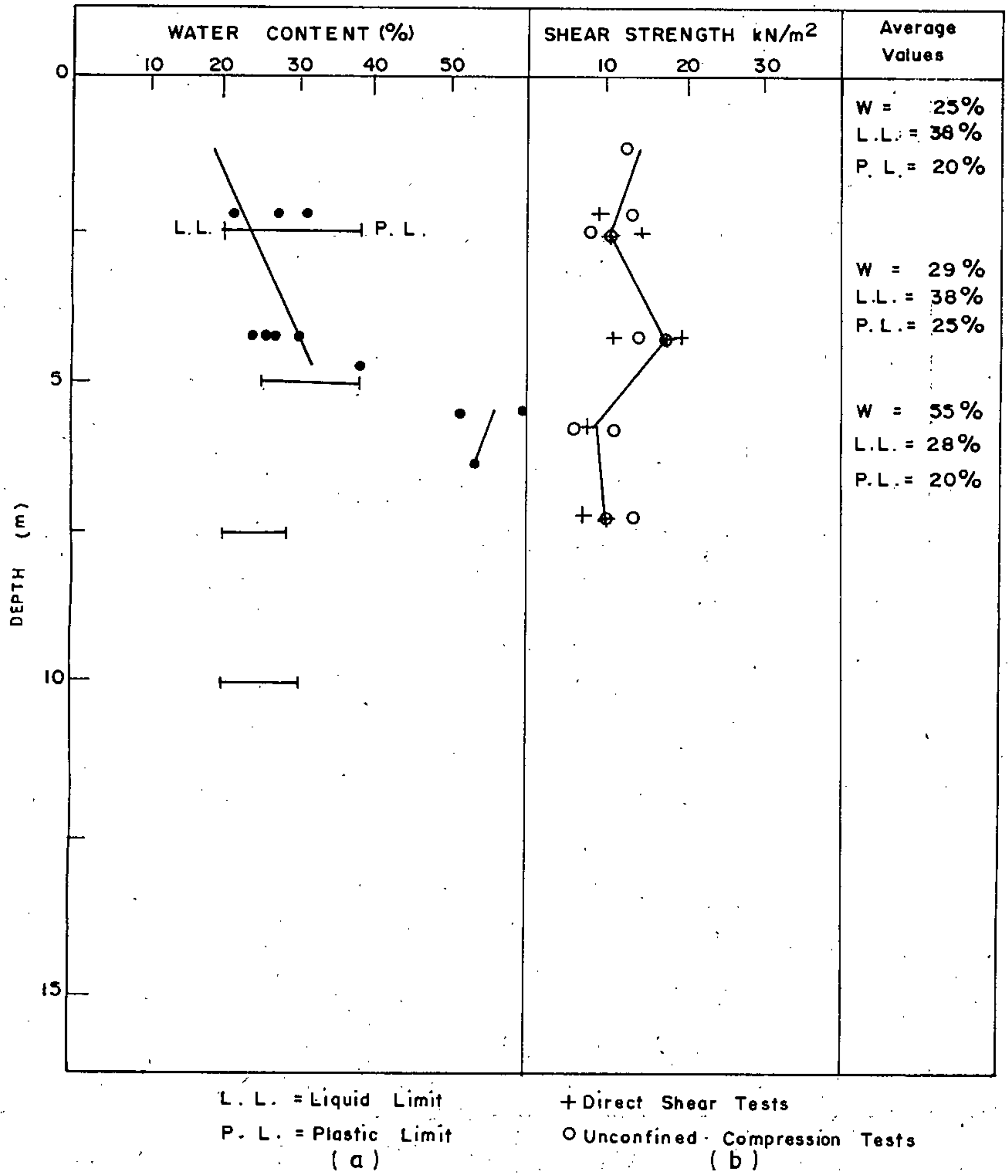


Fig. 4.1b Soil Properties of D.J's Court (a) Water Content. (b) Shear strenth.

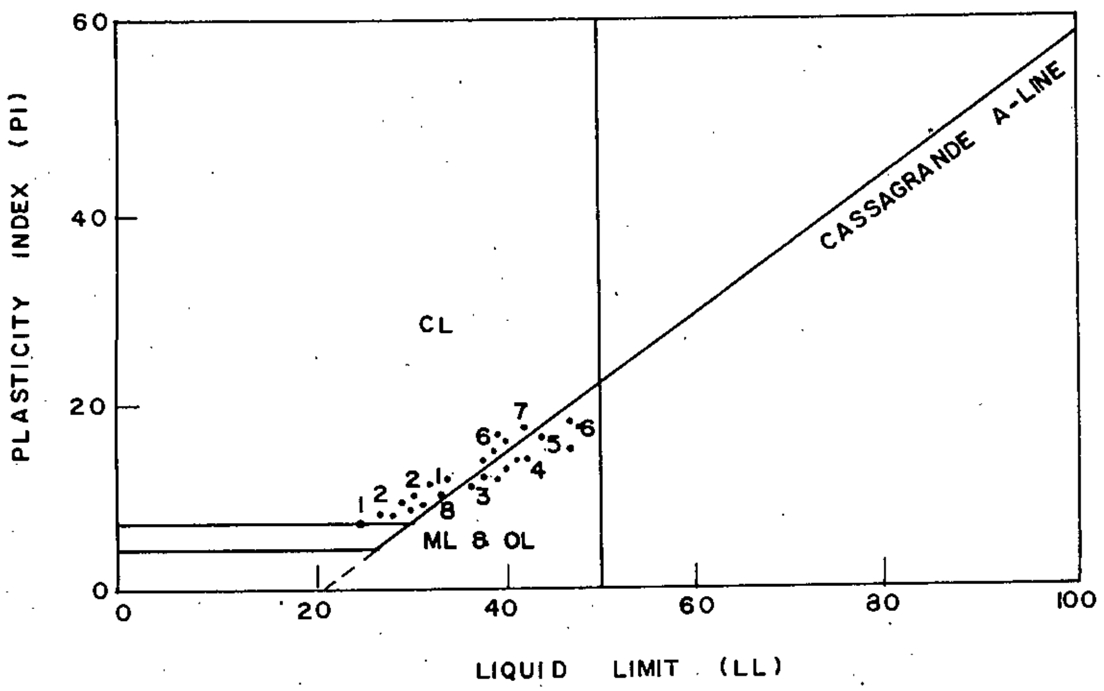


Fig. 4.2 Plasticity Chart for Laboratory Classification of Soil.
(D. C.'s Court and D. J.'s Court)

of 60 kN/m^2 , the SPT values were found to vary in between 3 and 18 . The results of the Standard Penetration Resistance tests are summerized in Tables 4.1 and 4.2 and also presented in Fig. 4.3 . The rate of increase of SPT values varies from 150% to a negligible . The comparison of SPT values prior to and after improvement is shown in Fig. 4.4 . The rate is higher at the shallower depths and for softer consistency of soil. The reason may be that at shallower depth the intensity of imposed load is higher. Similarly, for soils having softer consistency the stiffness is lower and resulting in a large deformation hence higher stiffness and SPT values.

4.3 Bearing Capacity of Soil

Before the installation of the vertical drains , the ultimate bearing capacity as calculated using the SPT values and Terzaghi's (1948) formula was found in the range of 48 kN/m^2 to 864 kN/m^2 upto a depth of 9.0 m . After the installation of vertical drains and preloading (60 kN/m^2), the ultimate bearing capacity at the site was found in the range of 216 kN/m^2 to 600 kN/m^2 up to depth of 9.0 m at 50% consolidation. Between 3.0 m to 6.0m the bearing capacity increases rapidly because of presence thick clayey silt layer. The results of the bearing capacities before and after the installation of vertical drains are shown in Tables. 4.3 and 4.4. A comparison of bearing capacities are presented in Table 4.5 and shown Fig 4.5.Similar reasoning may be made for the increase of SPT in Art. 4.2 .

DEPTH(m)	ELEVATION	STANDARD PENETRATION VALUES				DEPTH
		6	6	6	Total 12	
		2	3	3	6	
1.5		2	3	4	7	
3.0		3	3	6	9	
4.5		3	4	5	9	
6.0		4	6	6	12	
7.5		3	6	8	14	
9.0						

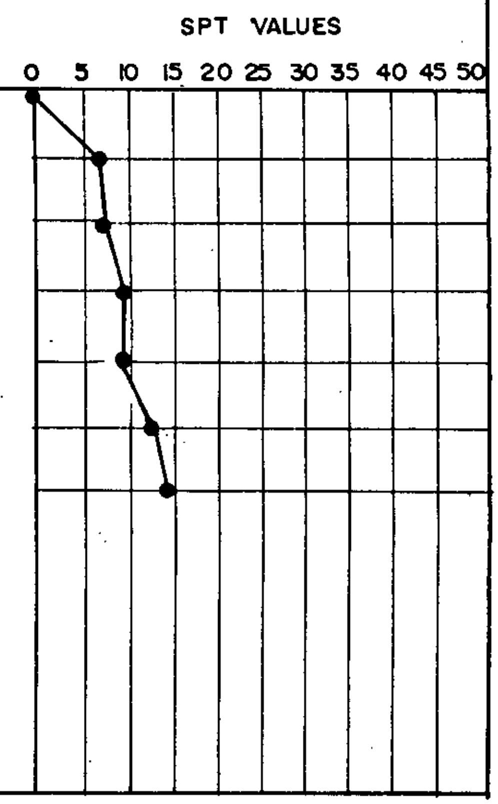


Fig. 4.2a Typical SPT Value with Depth (After Improvement)
(D.C.S Court, BH-6, Group-2)

DEPTH (m)	ELEVATION	STANDARD PENETRATION VALUE				DEPTH
		6	6	6	Total 12	
1.5		2	3	3	6	
3.0		1	2	2	4	
4.5		1	2	3	5	
6.0		2	3	5	8	
7.5		3	3	6	9	
9.0		4	5	7	12	

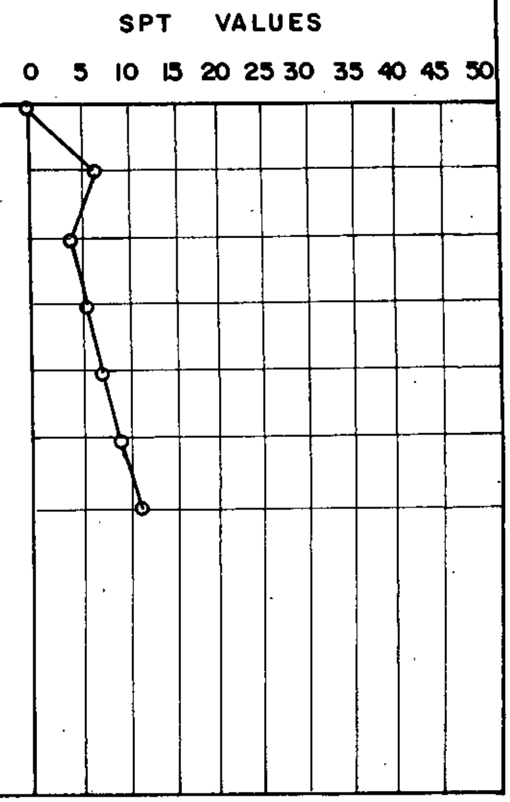


Fig. 4.2b – Typical SPT Value with Depth (After Improvement)
(D.J.'S Court; BH - I, Group-8)

Table 4.1 : Field SPT Values (Before improvement)

Depth (m)	1.5	3.0	4.5	6.0	7.5	9.0	10.5	12.0	13.5	15.0
BH No. 1	3	2	2	2	4	10	9	10	11	22
BH No. 2	2	2	2	3	3	12	13	13	15	20
BH No. 3	2	2	1	3	7	12	12	16	23	24
BH No. 4	2	2	3	10	12	16	16	16	14	17
BH No. 5	1	1	3	3	5	12	14	10	13	14
BH No. 6	1	2	2	6	8	15	15	16	17	17
BH No. 7	1	2	3	4	9	11	14	15	15	18
BH No. 8	1	1	4	5	6	8	10	15	14	22
BH No. 9	1	2	3	3	7	9	10	15	17	23
BH No. 10	1	3	3	7	13	14	22	-	-	-

N.B.
D.C.'s Court = BH Nos. 3, 4, 5, 6, 7 & 8
D.J.'s Court = BH Nos. 1, 2, 9 & 10

Table - 4.2a : N-Values (After improvement)

Depth (m)		3.0	4.5	6.0	7.5	9.0	10.5
Group 2	BH 6	6	7	9	9	12	14
	BH 7	4	6	8	9	11	13
	BH 8	3	4	4	6	7	7
Group 4	BH 4	5	6	7	10	11	13
	BH 5	5	6	10	11	12	15
Group 6	BH 2	5	3	11	17	18	18
	BH 3	7	9	10	7	8	11
Group 8	BH 1	6	4	5	8	9	12
	BH 2	7	5	5	9	10	14
	BH 3	5	5	6	8	10	11
Group 9	BH 5	5	5	6	8	9	11
	BH 6	3	5	5	7	8	12
Group 10	BH 4	6	5	7	8	10	13

Table 4.2b : N Values (After improvement)

	Field Values					
Depth (m)	3.0	4.5	6.0	7.5	9.0	10.5
Group 2	4	6	7	8	10	11
Group 4	5	6	8	10	11	15
Group 6	5	6	10	12	13	14
Group 8	6	5	5	8	10	12
Group 9	4	5	5	7	8	11
Group 10	6	5	7	8	10	13

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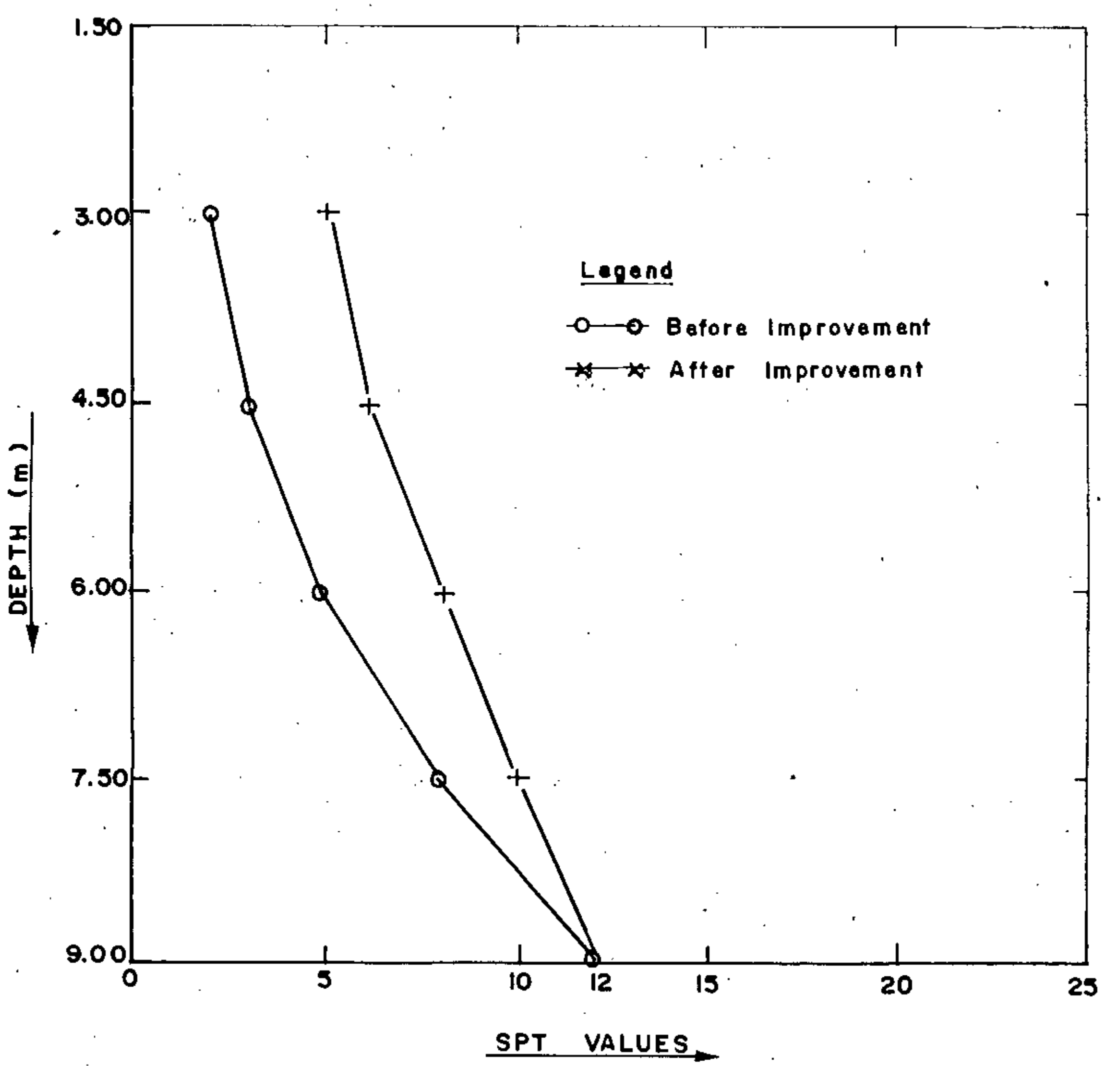

Fig 4.3a SPT Values Prior to and After Improvement.
(D.C'S Court)

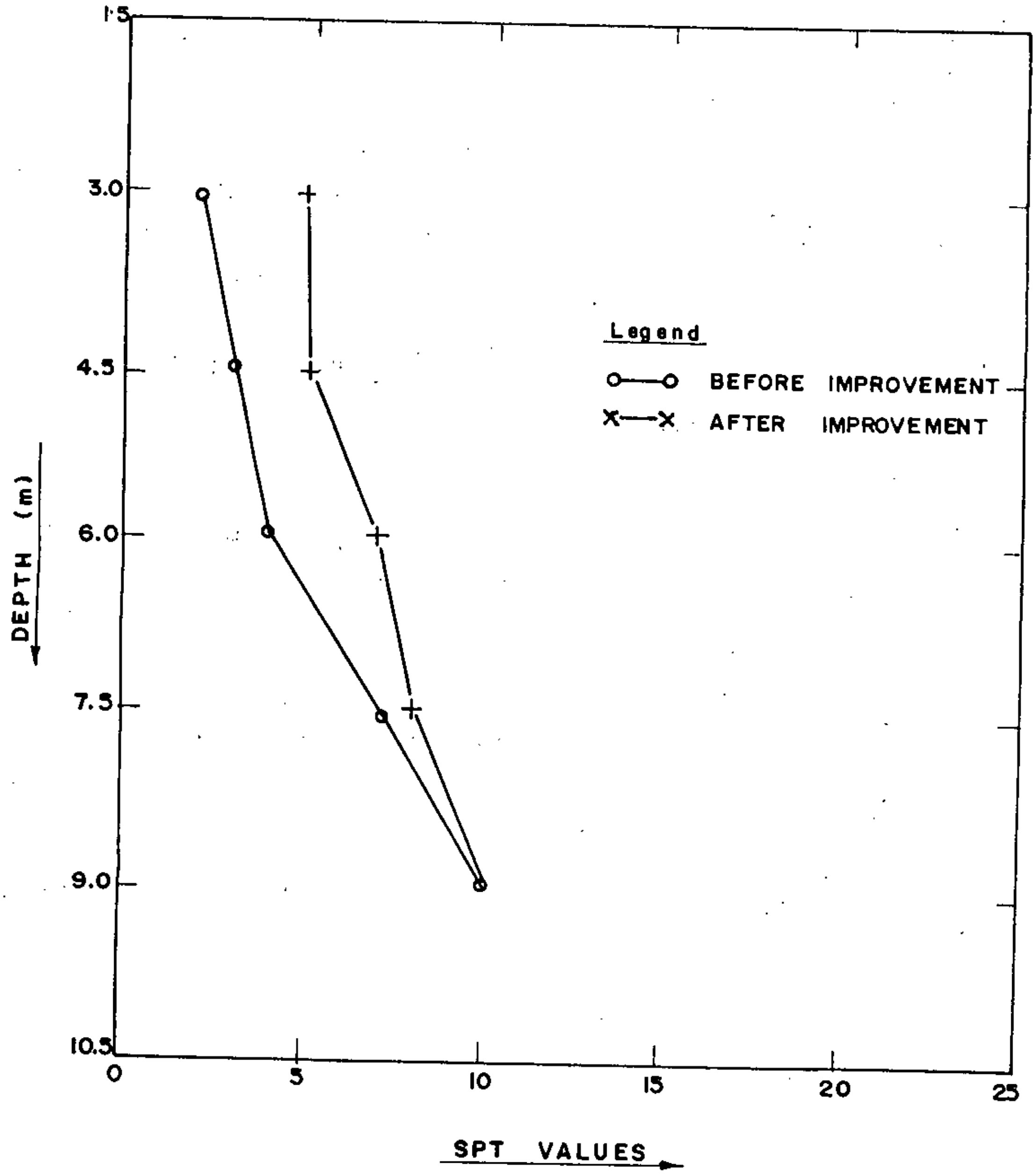

Fig. 4.3b SPT Values Prior to ond After Improvement. (D. J.'S Court)

Table : 4.3 : Ultimate Bearing Capacity in kN/m^2 : (Before improvement)

Depth (m)	1.5			3.0			4.5		
From Diff. Methods	N Value	q_u	Direct Shear Test	N Value	q_u	Direct Shear Test	N Value	q_u	Direct Shear Test
BH 1	120	108	63	96	96	69	96	108	60
BH 2	96	78	63	96	84	78	120	102	102
BH 3	96	84	51	72	90	39	96	66	33
BH 4	96	-	-	120	-	-	312	138	84
BH 5	48	-	-	96	57	48	144	51	36
BH 6	72	-	-	96	-	-	192	81	75
Bh 7	72	-	-	120	84	51	168	-	-
BH 8	48	45	33	120	-	-	216	60	36
BH 9	72	-	-	120	84	78	144	123	87
BH 10	96	-	-	144	171	60	240	156	90

Depth (m)	6.0			7.5			9.0		
From Diff. Methods	N Value	q_u	Direct Shear Test	N Value	q_u	Direct Shear Test	N Value	q_u	Direct Shear Test
BH 1	144	-	-	336	-	-	456	-	-
BH 2	144	-	-	360	45	63	600	-	-
BH 3	240	63	-	456	-	-	576	-	-
BH 4	528	96	78	672	-	-	768	-	-
BH 5	192	69	48	408	-	-	624	-	-
BH 6	336	-	-	552	129	81	720	333	255
BH 7	312	159	138	480	-	-	600	177	90
BH 8	264	-	-	336	-	-	432	-	-
BH 9	240	57	78	384	-	-	456	-	-
BH 10	480	96	60	648	-	-	864	-	-

Table- 4.4 : Ultimate Bearing Capacity in kN/m^2 (After installation of vertical drains)

Depth (m)	3.0		4.5		6.0	
From Diff. Methods	N Values	N and q_u Values	N Values	N and q_u Values	N Values	N and q_u Values
Group 2	240	179	312	268	360	313
Group 4	264	224	360	268	480	403
Group 6	288	268	408	268	552	492
Group 8	264	268	240	224	312	224
Group 9	216	179	264	224	336	268
Group 10	264	268	288	224	360	313

Depth (m)	7.5		9.0		10.5	
Group 2	432	357	504	447	528	492
Group 4	552	492	624	537	672	626
Group 6	600	537	672	582	720	671
Group 8	432	358	528	447	576	537
Group 9	408	357	504	403	576	537
Group 10	432	357	552	447	624	582

Table- 4.5 : Comparison of Ultimate Bearing Capacity from SPT Values in kN/m^2

	Before installation of drains						After installation of drains					
Depth(m)	1.5	3.0	4.5	6.0	7.5	9.0	3.0	4.5	6.0	7.5	9.0	10.5
D.C.'s Court *	72	104	188	312	484	620	264	360	464	528	600	640
D.J.'s Court **	96	114	150	252	432	594	248	264	336	424	528	592

* D.C.'s Court = BH No. 3, 4, 5, 6, 7, 8 and Group No. 2, 4, 6
** D.J.'s Court = BH No. 1, 2, 9, 10 and Group No. 8, 9, 10

Table- 4.6 : Results of Plate Load Test Data (After installation of drains)

Group	Time (hr)	load on Plate kN/m^2	Settlement at Design Load * (mm)	Settlement at twice the Design Load (mm)	Safe Bearing Pressure in kN/m^2
Group 2	6	107	13.85	24.09	55
Group 4	4	107	10.0	18.11	64
Group 6	5	107	4.71	12.93	81
Group 8	20	107	3.18	12.50	70
Group 9	16	107	7.02	22.33	60
Group 10	5	107	6.02	20.43	60

* Design load = 53.5 KN/m^2

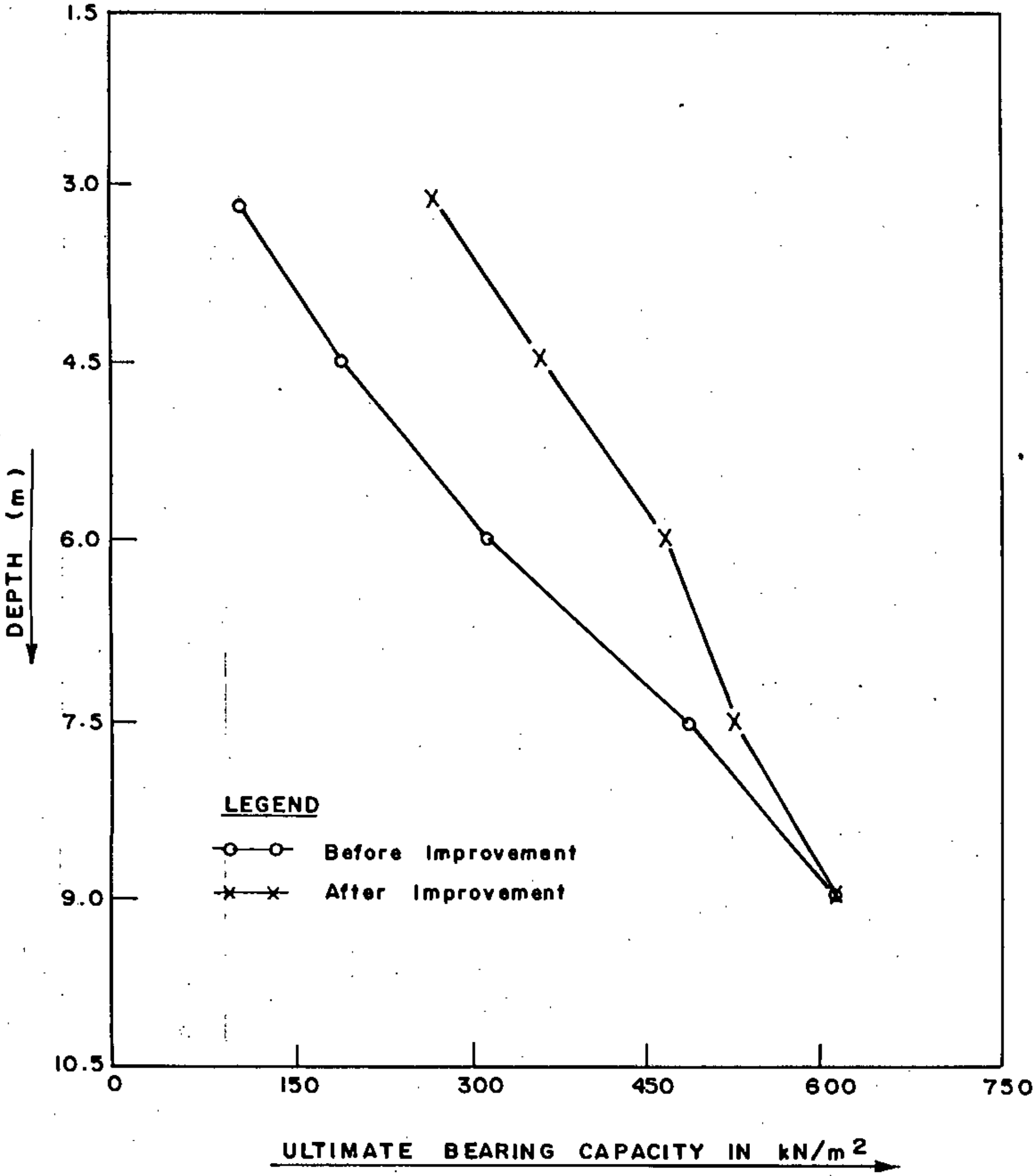

Fig. 4.5a Ultimate Bearing Capacity Prior to and After Improvement (D. C.'s Court)

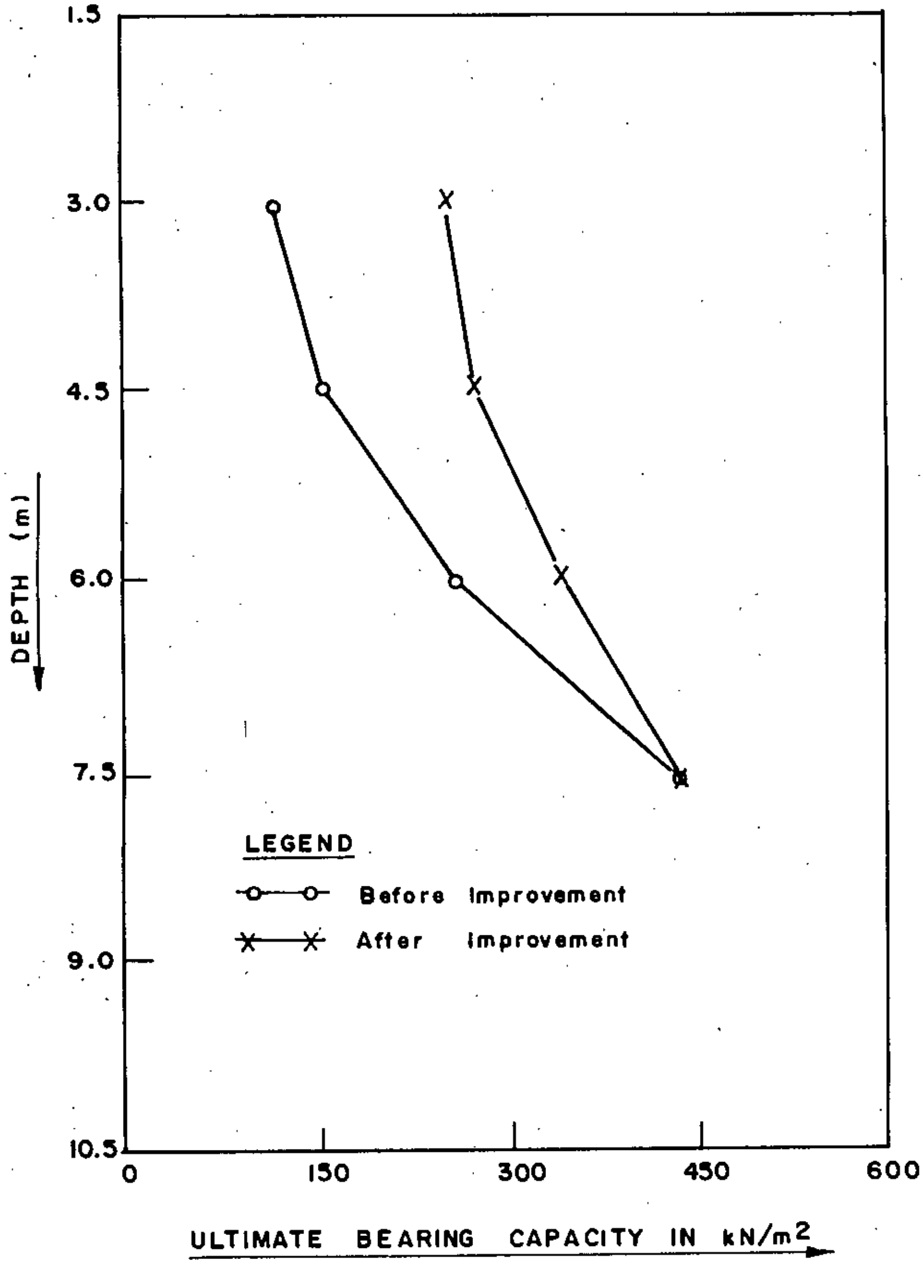

Fig. 4.5b Ultimate Bearing Capocity Prior to and After Improvement (D. J.'S Court)

4.4 Bearing Capacity from Load Tests

Six numbers of plate load tests were done at the site two months after installation of vertical drains and preloading. The results of the load tests are summerized in Table 4.6 . Typical load settlement curve of a test is shown in Fig. 4.7 and its data are presented in Table 4.7 . It is found from the Table 4.6, the settlement is less than 14mm at design load (53.5 kN/m^2) and is less than 25mm at twice the design load.

From load tests data, the allowable bearing capacities were found to be 67 kN/m^2 at D.C.'s court buildind site and 63 kN/m^2 at D.J.'s court building site. It appears that the bearing capacity as calculated by using Terzaghi's (1948) formula and SPT value gives an overestimated value.

4.5 Settlement of Soil

To calculate the settlements, the whole depth of the soft soil was divided into several layers as suggested by SIMONS and MENZIES (1975). The soil parameters corresponding to each layers were obtained from Table 3.3. The predicted settlements by using Terzaghi's (1943) method are given in Table 4.8 . The amount of predicted settlement ranges from 0.11 m to 0.23 m at 50% consolidation. The average total settlement was estimated as 0.17m. After two months of the installation of vertical drains, only 50% consolidation was completed as shown in Fig. 4.6 and the amount of settlement of the soil layer was observed to be 0.125 m . This shows an overestimation of settlement while using Terzaghi's(1943) formula.

Table : 4.7 Typical Plate Load Test Data (After improvement)

Design Load = 0.5 TON/SFT ; Ultimate Load = 1.0 TON/SFT
Area of Plate = 1 ft². Location : D.C.'s Court ; Group = II

Date	Time	Dial Gaze Reading kg/cm²	Transferred Load on Plate Tsf	Settlement Gaze Reading			Average Settlement mm
				M_1	M_2	M_3	
20.4.89	16-22	5	0.25	640	690	680	6.70
	16-28	5	0.25	820	800	750	-
	16-45	5	0.25	886	850	810	-
	17-00	5	0.25	1000	1000	1000	-
	17-30	5	0.25	1128	1200	1120	-
	18-00	5	0.25	1142	1225	1130	11.65
	18-02	10	0.50	1270	1370	1360	13.33
	18-05	10	0.50	1300	1380	1378	-
	18-15	10	0.50	1318	1390	1390	-
	18-30	10	0.50	1328	1400	1402	-
	18-45	10	0.50	1335	1410	1410	13.85
	18-47	15	0.75	1610	1700	1685	16.65
	19-00	15	0.75	1730	1820	1770	-
	19-15	15	0.75	1810	1880	1830	18.80
	19-30	20	1.00	1990	1996	1950	19.78
	20-00	20	1.00	2210	2241	2226	-
	20-05	20	1.00	2300	2305	2330	-
	20-15	20	1.00	2315	2320	2340	-
	20-45	20	1.00	2348	2392	2380	-
	22-00	20	1.00	2404	2415	2410	24.09

Table 4.8 : Predicted Settlement of Soil

$$S_T = \frac{c_c}{1+e_o} H \log_{10} \frac{Po+\Delta p}{Po}$$

Bore Hole	Layer (m)	P_o kN/m²	Δp kN/m²	Void ratio e_o	Comp. Index c_c	$\frac{c_c}{1+e_o}$	$\frac{Po+\Delta p}{Po}$	Thickness (H)	S_T (m)
1	0 -1.5	13	60	0.78	0.18	0.10	5.62	1.5	0.11
	1.5-6.0	44	60	0.84	0.20	0.11	2.36	4.5	0.18
2	0 -6.5	40	60	0.84	0.20	0.11	2.50	6.5	0.28
3	0 -5.3	29	60	0.83	0.19	0.10	3.07	5.3	0.26
	5.3-7.0	48	60	1.15	0.30	0.14	2.25	1.7	0.08
4	0 -5.2	36	60	0.80	0.20	0.11	2.67	5.2	0.24
	5.2-7.0	65	60	1.40	0.40	0.17	1.92	1.8	0.09
5	0 -4.0	28	60	0.90	0.22	0.12	3.14	4.0	0.24
	4.0-7.0	53	60	1.42	0.40	0.17	2.13	3.0	0.17
6	0 -7.0	42	60	0.76	0.17	0.10	2.43	7.0	0.27
7	0 -6.7	41	60	0.79	0.19	0.11	2.46	6.7	0.29
8	0 -3.6	26	60	0.90	0.22	0.12	3.31	3.6	0.22
9	0 -5.2	25	60	0.89	0.33	0.17	3.40	5.2	0.47
10	0 -5.2	26	60	0.87	0.21	0.11	3.31	5.2	0.30
	5.2-6.0	59	60	1.42	0.42	0.17	2.02	0.8	0.04

The average total settlement = 0.32 m

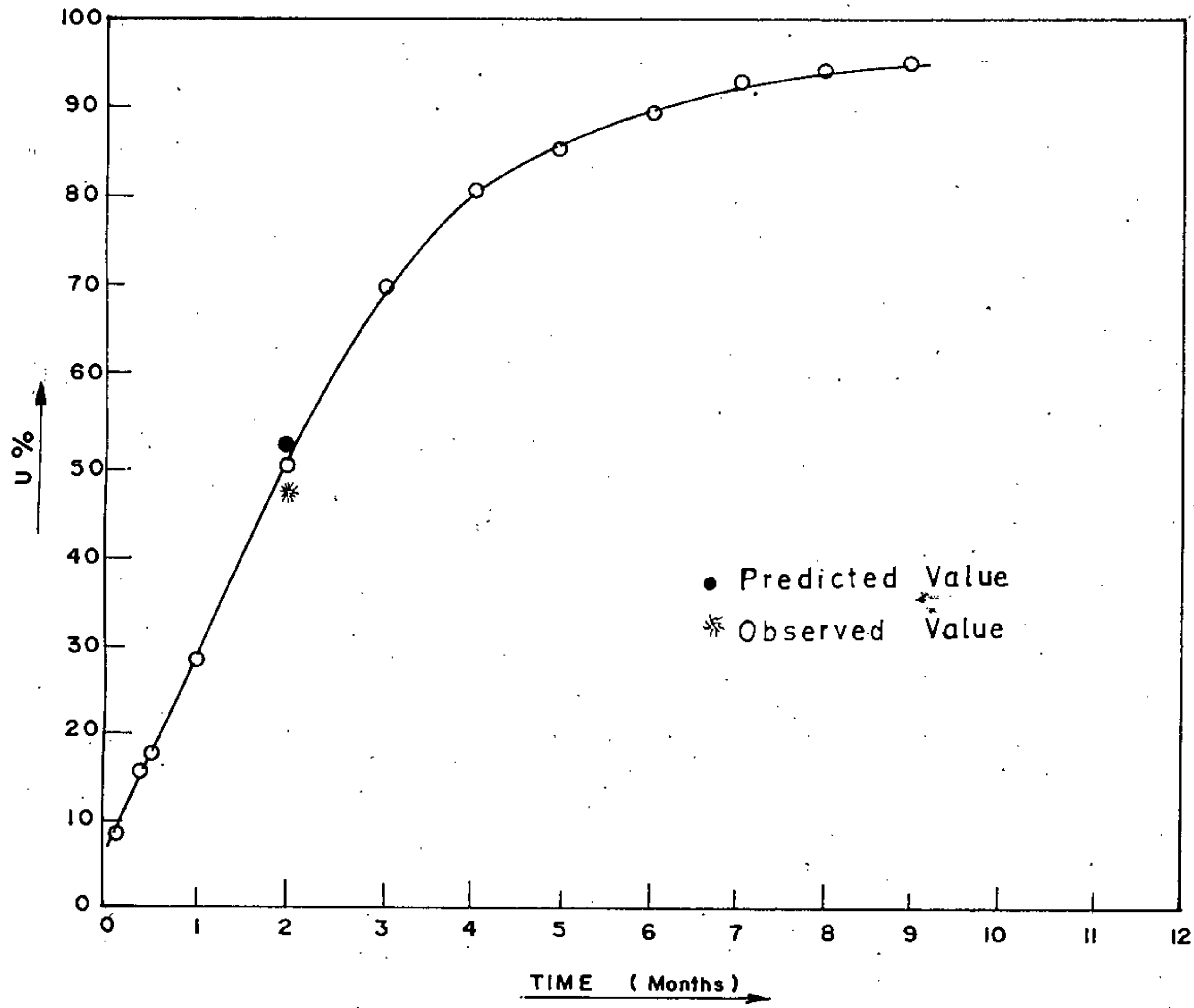

Fig 4.6 Average Time Consolidation Rates.

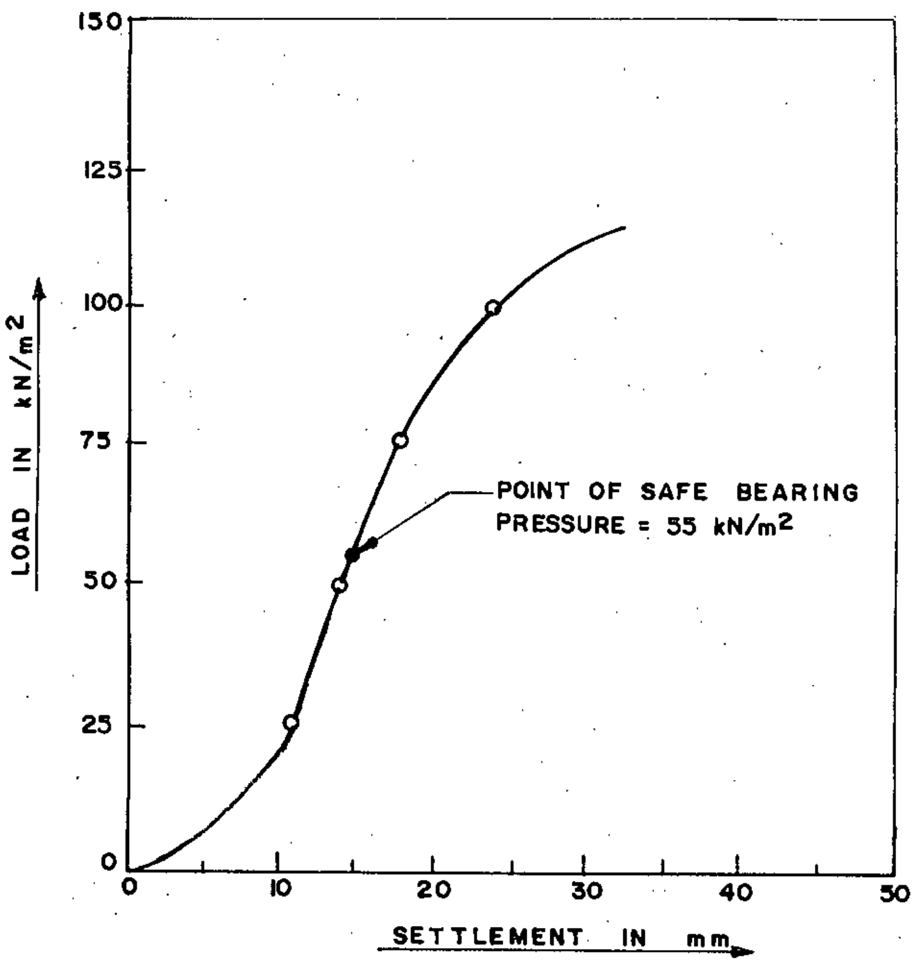

Fig. 4.7a Typical Load Settlement Curve
(D.C.S Court; Group II)

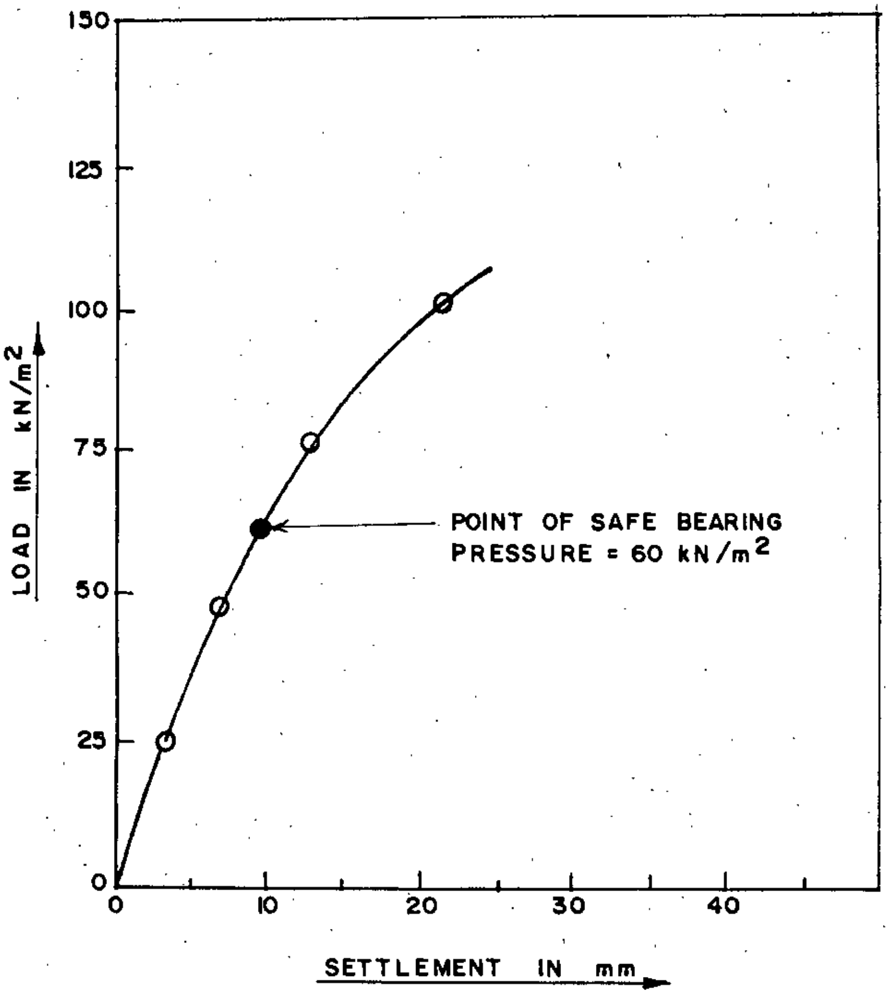

Fig. 4.7 b Typical Load Settlement Curve.
(D. J.'S Court Group IX)

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

5.1 Conclusions

The objective of the present study was to investigate the shear strength and hence the bearing capacity, and settlement characteristics of a soft soil layer (WL = 19-59, LL = 24-48, PI = 7-17), due to installation of vertical sand drains (with n = 10) and preloading . The following conclusions can be drawn from the study.

(I) The increase in shear strength of soil (at 50 % consolidation of soil) decrease with depth. The rate of increase varies from 135 % at surface to negligible at the bottom end level of sand drains. The increase of shear strength almost ceases at a depth of 0.75 times the spacing of sand drains from the bottom of sand drains.

(II) The settlement of a soil layer as calculated using Terzaghi's (1943) formula and consolidation test data is conservative.

5.2 Recommendations for Future Study

The study is based on the data of an ongoing project of soil improvement using vertical drains and preloading. It is worthwhile to carryout further study on the following aspects:

(I) The behaviour of soft soil layer in relation of shear strength and bearing capacity at different degree of consolidation and preloading.

(II) The deformation behaviour of soft soil layer due to preload of different intensity.

(III) The variations of strength and deformation with depth at different degree of consolidation.

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Appendix : A

Design For Vertical Drains

For squre pattern, $d_e = 1.13\ S$

Where: d_e = diameter of the cylinder of influence of drain.
d_w = diameter of sand drains = 0.2 m
S = Spacing of vertical drains

We have , $T_v = Cvt/H^2$

Whre, $C_v = 8.43 \times 10^{-8}\ m^2/sec = 7.28 \times 10^{-3}\ m^2/day$
(From HBRI soil report)
t = 180 days, H = 7.62 m (Assume),

$$\text{or, } T_v = 7.28 \times 10^{-3} \times 180/7.62^2 = 0.0226$$

From graph of consolidation due to vetical drain, U_v = 17 %
Assume, U = 85 %

$$\text{or, } U = 1 - (1 - Uv)(1 - U_h)$$
$$0.85 = 1 - (1 - 0.17)(1 - U_h)$$
$$U_h = 0.82 = 82\ \%$$

For ideal case, drain spacing factor,

$$F(n) = Ln\ (d_e/d_w) - 3/4$$

Time required to achieve U_h,

$$t = (d_e^2\ /8C_h\ )\ F(n)\ Ln\ [1/(1-U_h)]$$

Assume, $C_h/Cv = 1$

By trial, spacing of vertical drains can be found:

Assume d_e m	F(n)	t(months)	$n = d_e/d_w$
1	0.86	0.83	5
1.5	1.26	2.77	7.5
2.0	1.55	6.00	10

Therefore, d_e = 2.0 m , or, $S = 2.0/1.13 = 1.77$ c/c

Appendix : B

Calculation of Bearing Capacity

The relationship between the standard penetration resistance, the consistency of soil, and the allowable bearing capacity as indicated in the accompanying table (Terzaghi and Peck, 1948) is very approximate.

Consistency of soil	N (standard penetration resistance)	Allowable bearing pressure* (Square footing) in Tsf
Very soft	0 - 2	0.00 - 0.30
Soft	2 - 4	0.30 - 0.60
Medium	4 - 8	0.60 - 1.20
Stiff	8 - 15	1.20 - 2.40
Very stiff	15 - 30	2.40 - 4.80
Hard	30 +	4.80 +

*Ultimate bearing capacity is equal to three times the allowable.

For calculation of bearing capacity, the average corrected N- Values are taken upto a depth of 2 m (width of foundation)