PERMEABILITY AND CONSOLIDATION CHARACTERISTICS

OF

NORMALLY CONSOLIDATED CLAYS

A Thesis

by

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ABSTRACT

Permeability and consolidation characteristics of three saturated normally consolidated silty clays of medium plasticity and one slightly plastic silt have been studied. Normally consolidated samples of each soil were prepared by applying a vertical pressure of 1/4 tsf to a slurry having a water content of twice the liquid limit of the soil. Constant head permeability and one dimensional consolidation tests were performed on specimen of each soil. Permeability tests were run by applying back pressure under hydraulic gradients varying from 293 to 857. Consolidation tests were performed with a stress increment ratio of unity and duration of each loading was approximately 24 hours.

Permeability parameters (n and C) and coefficient of permeability (k) were computed from both permeability and consolidation test results while compressibility parameters (C_c and e_1), coefficient of consolidation (C_v) and constrained modulus (D) were determined from consolidation test results.

It has been found that value of C increases with decreasing liquid limit (LL) and plasticity index (PI) while the values of C_c and e_1 generally increase with the increase in LL and PI. The magnitude of n of the soils studied did not vary significantly. Coefficient of consolidation increases somewhat linearly with logarithm of effective vertical pressure ($\bar{\sigma}$). However, C_v decreases at higher values of $\bar{\sigma}$. For a

particular value of $\bar{\sigma}$, C_v decreases with increase in LL and PI. The analysis of test results reveals that both k and C_v can be determined with reasonable accuracy from consolidation test using square root of time fitted curves. On the basis of test results two theoretical models and one empirical equation have been proposed. One model relates k with void ratio e while the other relates D with $\bar{\sigma}$. The empirical equation evaluates the value of C_c as a function of $\bar{\sigma}$ and e. These proposed models and empirical equation are expected to be useful in estimating coefficient of permeability and settlement of structures.

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NOTATIONS

ASTM	- American Society for Testing of Materials
a	- cross-sectional area of capillary tube
a_v	- coefficient of compressibility
C	- Permeability parameter which is the antilogarithm of the vertical intercept of log $[k(1+e)]$ versus loge plot
C_c	- compressibility parameter called compression index
C_k	- an index which is the slope of logk versus log $\bar{\sigma}$ plot
C_s	- pore shape factor
C_v	- coefficient of consolidation
D	- value of minimum D of D versus $\bar{\sigma}$ plot
D_{10}	- effective grain size
D_s	- A hyphothetical diameter of spherical grains
e	- void ratio
e_o	- initial void ratio
e_1	- compressibility parameter which is the equilibrium void ratio under 1 tsf effective vertical pressure i.e. $\bar{\sigma}_1$
$e_{\frac{1}{4}}$	- equilibrium void ratio under $\frac{1}{4}$ tsf effective vertical pressure
$e_{\frac{1}{2}}$	- equilibrium void ratio under $\frac{1}{2}$ tsf effective vertical pressure
e_2	- equilibrium void ratio under 2 tsf effective vertical pressure
g	- acceleration due to gravity
H	- height of sample
H_t	- thickness of layer undergoing settlement

H_{dr} - longest drainage path

i - hydraulic gradient

k - coefficient of permeability

K - specific or absolute coefficient of permeability

LL - liquid limit

log - base 10 logarithm

n - permeability parameter which is the slope of log $[k(1+e)]$ versus loge plot

n' - porosity

PI - plasticity index

Q - flow

Q_{α} - flow rate at steady state of flow

q - flow quantity in unit time

R - radius of capillary tube

R_H - hydraulic radius of capillary

S - specific surface area per unit volume of total particles

S_v - surface area per unit of total volume

S_t - settlement

S - deformation (% of height)

T - tortuosity

t - time

T_v - time factor

t_{50} - time required for 50% consolidation

t_{90} - time required for 90% consolidation

u - excess pore water pressure

U_z	- percent consolidation
v	- flow through unit cross-sectional area
w	- water content
z	- coordinate of flow direction
γ_w	- unit weight of water
γ_{sat}	- saturated unit weight of soil
μ	- absolute coefficient of viscosity of permeating fluid
$\bar{\sigma}$	- effective vertical pressure
$\Delta\sigma$	- pressure increment
$\bar{\sigma}_o$	- minimum value of $\bar{\sigma}$ of D versus $\bar{\sigma}$ plot
$\bar{\sigma}_1$	- unit effective vertical pressure = 1 tsf (1 ton = 2000 lb).

CHAPTER 1

INTRODUCTION

1.1 General

Two important problems foundation engineers are often required to deal with are the accurate prediction of rate of flow of water through soil and its deformation behavior. For example, the seepage through the core of an earth dam and underseepage through the insitu soils supporting the dam are of primary importance to the designer in his evaluation of water loss and dam stability. Similarly, the designer of a coefferdam needs to calculate the rate of flow into the excavation in order to select the appropriate pumping system. The computation of the rate of settlement of structures on clays, solution of drainage problems for highways and airports, checking stability of slopes for embankments and cuts, all these require a knowledge of rate of flow of water through the soil. A foundation engineer is also concerned with the deformation characteristics of soil because he has to know by how much a soil will be compressed under external load and at what rate the compression will take place. These information are important in order to make reliable estimates of the amount and time of settlement of foundations and possible volume change of an earthwork.

From historic past numerous cases of settlement and slope failures have been reported from different parts of the world. The most classic example of a settlement problem is the case of the "Leaning Tower of Pisa" in Italy which suffered a total settlement of almost ten feet and a differential settlement of about six feet in a time span of about eight hundred years (Jumikis, 1962). In the modern world, Mexico city has become a museum of settlement failures. The slope failure of railway embankment at Vita Sikudden in Sweden in 1918 was due to an exceptional water filtration in the embankment fill during unusually heavy rains before the slide took place (Jumikis, 1962). Extensive ruptures of slope in the Panama Canal cuts are cited as examples of slope failures. The failure of Malpaset Dam in France in 1961 was caused by excessive seepage pressures (Cedergren, 1967).

Slope and settlement failures are not unusual in Bangladesh. The River Transport Terminal Building of Inland Transport Authority at Narayanganj has suffered considerable damage due to differential settlement of the foundation beyond tolerable limits (Kabir, 1978). Safiullah (1977) investigated the causes of failure of slopes of the Dhaka-Narayanganj Demra embankments during 1974 flood. The failure of this embankment was due to hydraulic conditions which created instability. In this particular embankment neither toe filter nor central impermeable core was provided. As a result, progressive erosion from land side toe of the embankment occurred due to high seepage pressure at the exit point and absence of filter at such locations.

At present Bangladesh is moving forward with large development projects including construction of high rise buildings, oil storage tanks, long span bridges, harbour and port structures, flood protection embankments and barrages. So, during this stage of infrastructure development in Bangladesh, a detailed knowledge and sound understanding of flow rate of water through soil and deformation behaviour of soil are of utmost importance to the civil engineers in this country.

1.2 Permeability and Consolidation

Any given mass of soil consists of solid particles of various sizes with continuous interconnected void spaces. The continuous void spaces in a soil permit water to flow from a point of high energy to a point of low energy. Permeability is defined as the property of a soil which permits the flow or seepage of fluids (liquids) through its interconnected void spaces and it is analogous to electrical conductivity. Problems involving stability, settlement and seepage are all dependent on the permeability characteristics of soils.

When a compressive load is applied to a saturated fine grained soil mass, at the instant of application of load almost all the applied pressure is transferred to the pore water, because water is virtually incompressible in comparison with the compressibility of the soil structure. The excess hydrostatic pressure initiates flow of water to drain out of the voids and the soil mass begins to decrease in volume. The decrease in volume of a soil mass under stress is called compression. A portion of the applied stress is transferred to the mineral skeleton, which in turn causes a reduction in the excess pore pressure. This process involving a gradual compression occuring simultaneously with a flow of water out of the soil mass and with a gradual transfer of the applied pressure from the pore water to the mineral skeleton is termed as consolidation. Conventionally, the process of consolidation is divided into primary consolidation and secondary consolidation. The reduction in volume which is solely due to the flow of water from the voids under excess hydrostatic pressure is called primary consolidation or primary time effect and is compatible with the Terzaghi (1943) consolidation theory. Even after the reduction of all excess hydrostatic pressure to zero, the soil mass continues to decrease in volume with time and is unaccounted by the Terzaghi theory. This process of reduction in volume is referred to as secondary consolidation or secular time effect.

Permeability and consolidation studies on sand are usually given less importance than on clay because sandy soils are highly permeable and consequently compression takes place instantaneously after the application of external load. However, permeability and consolidation studies on clays subjected to long term loading are very important because in such a case compression occurs at a slow rate due to very low permeability of clays. To study the permeability and consolidation characteristics of clay, it has been divided into two groups such as normally consolidated and overconsolidated, because permeability and consolidation properties of these two types of clay are not similar. A soil is said to be normally consolidated if the present effective overburden pressure is the maximum to which the soil has ever been subjected. On the other hand a soil is called overconsolidated if the present effective overburden pressure is less than the maximum to which the soil was ever subjected in the past.

Permeability of clays can be determined in both laboratory and field. Field determination of permeability is often complicated, time consuming and expensive. Therefore, permeability of clays is usually evaluated in the laboratory with the help of properly designed equipments. This research is principally devoted to the study of permeability and consolidation characteristics of normally consolidated silty clay and silts. Permeability and consolidation properties have been evaluated in the laboratory from constant head permeability test by applying back pressure and from one dimensional consolidation test.

1.3 Area of Research

Samarasinghe et al (1982) suggested a theoretical model to formulate a unique relationship between permeability and void ratio of saturated normally loaded clay. This relationship was used for deriving the governing differential equation of consolidation for normally consolidated clay. The derivation shows a theoretical relationship between coefficient of consolidation and vertical effective consolidation pressure. Stamatopoulos and Cotzias (1978) studied compressibility of clays in terms of constrained modulus which is the change of effective vertical consolidation pressure divided by the resulting change in vertical unit deformation or strain. They constituted a model that relates constrained modulus and consolidation pressure. The model is frequently used in Europe and some parts of North America to estimate settlement on the basis of constrained modulus. Estimation of settlement on the basis of constrained modulus is not used in Bangladesh.

The present research is undertaken with a view to verify and establish the findings as reported by Samarasinghe et al (1982) for different types of normally consolidated clays, particularly for silty clays and slightly plastic silts available in Bangladesh. Besides, attempts were made to propose a simple model relating constrained modulus and effective vertical consolidation pressure for normally consolidated silty clay and silts of Bangladesh. This model can be used to estimate settlement of foundations on normally consolidated silty clays and silts.

This research will be effective in establishing permeability-void ratio, coefficient of consolidation-effective consolidation pressure and void ratio-effective consolidation pressure relations for normally consolidated silty clays and silts. The above mentioned relations will provide the magnitudes of permeability and compressibility parameters of the soils. Correlation of these parameters with other index properties will also be available. All these findings are expected to establish a better understanding of the fundamentals of permeability and consolidation behavior of normally loaded local clays. The model relating constrained modulus and effective consolidation pressure can be successfully utilized to predict the settlement of structures on normally consolidated silty clays and silts.

CHAPTER 2

LITERATURE REVIEW

2.1 Coefficient of Permeability

From the results of permeability tests on saturated sand Darcy (1856) established that

$$v = ki \qquad 2.1$$

where, v = flow rate through unit cross-sectional area.

k = Darcy's constant called the coefficient of permeability.

i = hydraulic gradient.

A number of investigators subsequent to the classic experiments of Darcy have given results which are in agreement with Eq. 2.1 and thus this equation has become known as Darcy's law.

Darcy's law asserts that the rate of flow through a porous medium is directly proportional to the applied hydraulic gradient. That is, flow rate versus hydraulic gradient relationship is linear and any graph plotted to relate this will pass through the origin. The mathematical simplicity of Darcy's law makes it very useful in connection with analytical treatment of steady and transient flow problem in soils.

2.2 Validity of Darcy's Law and Non-Darcy Behavior

Darcy's law given by Eq. 2.1 is valid for laminar flow of water through saturated soils only. Several studies have been made to investigate the range over which Darcy's law is valid, and an excellent summary of these works is reported by Muskat (1937). For laminar flow conditions in soils, it can be conservatively assumed that Reynolds number R_e has an upper limit of unity (Lee et al, 1968) Thus,

$$R_e = \frac{\gamma_w D_{10} v}{\mu g} \leq 1 \qquad 2.2$$

here, v = flow through unit cross-sectional area or flow velocity

D_{10} = effective diameter of soil particle

γ_w = unit weight of water

g = acceleration due to gravity

μ = coefficient of absolute viscosity.

The linear relationship of flow rate with hydraulic gradient has been reported by numerous investigators. For example, Terzaghi (1925a) and Macey (1942) obtained a linear relation for the clays used in their classic permeability studies. Michaels and Lin (1954a) report studies of Muskat (1937) showing similar relationship for kaolinite. Also, Low (1960) presented data showing linearity for Na-montmorillonite.

Non-Darcy behavior has been reported by numerous investigators. As early as in 1898, King (1898) used various porous media including sand, sandstone and glass capillaries to perform permeability experiments. In all the cases, he observed that the flow velocity increase was more than proportional to the hydraulic gradient. Von Engelhardt and Tunn (1955) observed that water flow in sand stones containing less than 5 percent clay did not obey Darcy's law. The relationship between flow rate and gradient took a curvilinear shape. From the experimental results of permeability tests on unconfined and remoulded clays Lutz and Kemper (1959) observed that increasing the gradient increased the flow rate totally out of accord with Darcy's law. This was true for Na-clays. They explained that this resulted from a breakdown of water structure, in which the diffuse layer of cations and the water structure extended to distances greater than the radius of pores between particles.

Hansbo (1960) reported tests on four natural undisturbed clay samples in an extremely sensitive flow rate measuring system under equal and constant temperature conditions. His results are illustrated in Fig. 2.1. For hydraulic gradients greater than 10, the results show linearity of flow rates and gradients in all the samples. For gradients less than 10, the flow rate versus gradient relationships deviated from linearity in three of the four samples.

During the test he also observed that even at zero hydraulic gradient, the flow of water in clays was never found to be at completed stand-still, but sometimes a gradual increase in flow did occur. Thus he concluded that Darcy's law is not always valid for small hydraulic gradients and mentioned that this deviation from Darcy's law ought to depend on the interaction between skeleton grains and pore water. Based on experimental results, the relationship of flow rate and hydraulic gradient for normally consolidated clays shown by Hansbo is reproduced in Fig. 2.2. Kemper (1960) also observed deviations from Darcy's law and mentioned that this was due to the electroviscous effect. The electroviscous effect is explained as the production of an electrical potential that retards ionic movement, and through electroviscous drag, the movement of water is affected. Martin (1962) mentioned that curvilinear velocity-gradient relationship is due to plugging and unplugging of flow channels in the skeleton pore caused by seepage forces. Gupta and Swartzendruber (1960) found that the hydraulic conductivity of quartz sand for boiled deionized water decreased markedly during prolonged flow. The cause of the reduction was explained to be related to the activity and growth of bacteria.

Olsen (1962) from his experiments observed discrepancies between measured flow rates in liquid saturated clays and those predicted from Darcy's law. This he attributed to unequal pore sizes in the clays. Olsen (1963) further reported that

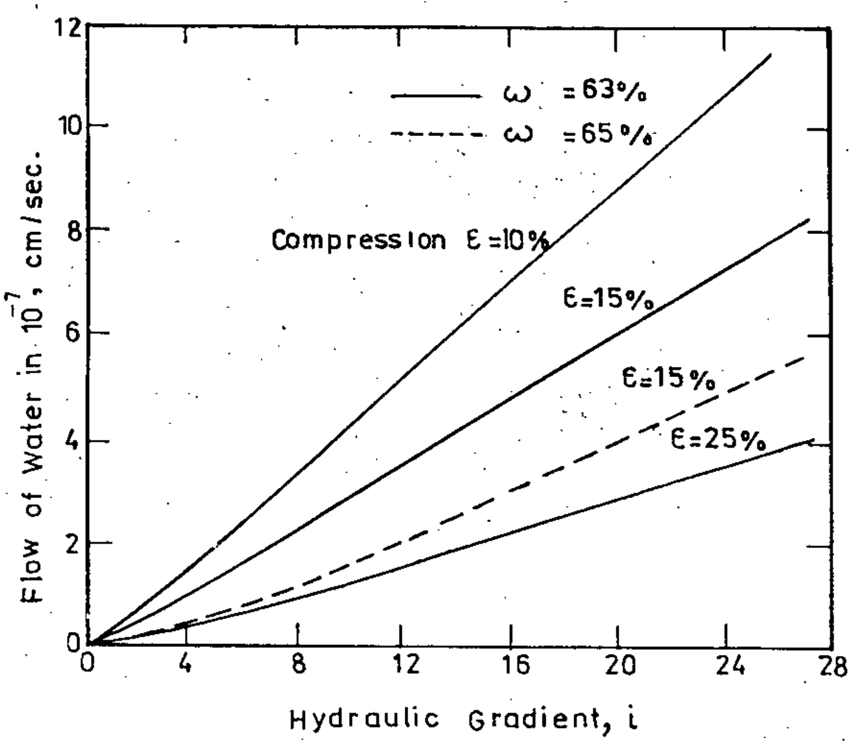


Fig. 2.1 Non-Darcy behavior (after Hansbo, 1960).

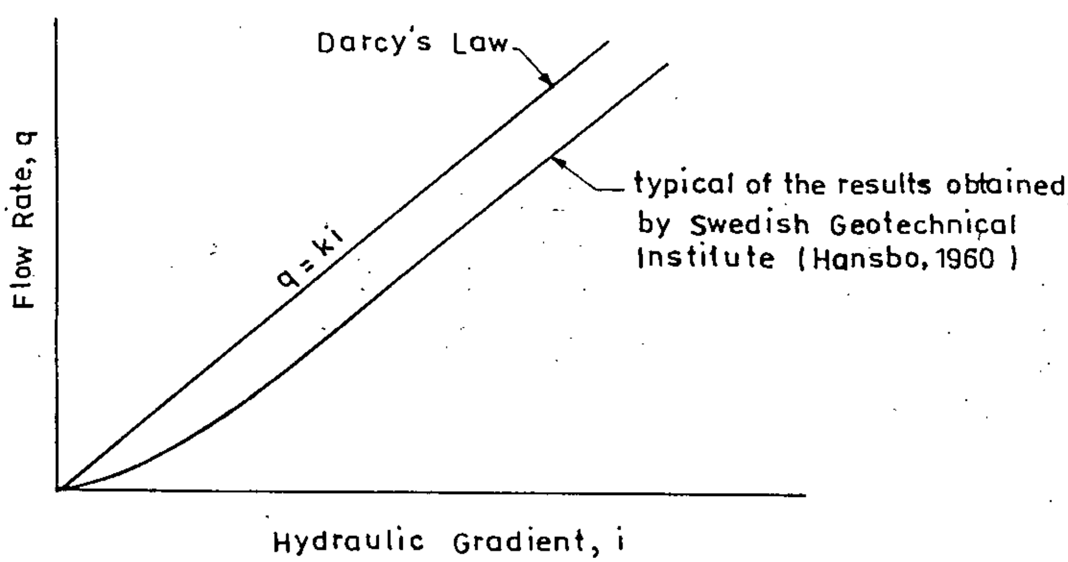


Fig. 2.2 Hydraulic flow rates versus hydraulic gradient for a normally consolidated clay (after Hansbo, 1960).

the flow behavior not following Darcy's law is due to change in fabric of the soil caused by migration of fine particles during the flow of the fluid through the soil and according to his observations the deviation from Darcy's law increases with increasing polarity and decreasing clay content. Mitchell and Younger (1966) observed the non-Darcy behavior of compacted silty clay samples and suggested that particle migration is more likely the cause in this case than the abnormal water properties.

2.3 Prediction of Permeability

Darcy's law was the result of simple experimental observations. Many attempts have been made to achieve a theoretical relation between permeability and size of soil grains, and size and shape of pore spaces through which flow occurs. Many of these relations have been derived from the Hagen-Poiseuille's equation for viscous flow through a small capillary tube of radius R given by the following expression:

$$q = \frac{\gamma_w \, i}{8\mu} R^2 a \qquad 2.3$$

where, q = flow quantity in unit time

γ_w = unit weight of permeating fluid (water)

μ = absolute coefficient of viscosity of permeating fluid

a = cross-sectional area of tube

i = hydraulic gradient

A model may then be considered to consist of a bundle of capillary tubes arranged parallel to each other with a ratio of pore-cross-section to total area equal to porosity n' of the soil. On the basis of this parallel-tube capillary model permeability may be expressed as

$$k = C_s \, R_H^2 \, \frac{\gamma_w}{\mu} \, n' \qquad 2.4$$

Here, k = coefficient of permeability

C_s = pore shape factor which takes into account the variation in shape of the capillary

R_H = Hydraulic radius of the capillary

Although the pore size may be related to the dimension of the grains composing the soil and that a collection of grains can be arranged in various assemblage containing different pore sizes and there can be no unique relationship between grain size and pore size in a soil mass. Therefore, in order to correlate between model and actual soil condition an average pore diameter should be used. This average diameter concept is used by Kozeny (1927) as a function of hydraulic radius, that is the ratio between flow area in a capillary and its wetted perimeter. When mathematically represented the permeability equation on the basis of above concept becomes

$$k = C_s \frac{\gamma_w}{\mu} \frac{n'^3}{S_v^2}$$

In equation 2.5 S_v is the surface area per unit of total volume.

The classic Kozeny-Carman equation, originally proposed by Kozeny (1927) and improved by Carman (1956) is given by the following equation

$$k = \frac{1}{C_s S^2 T^2} \frac{\gamma_w}{\mu} \frac{e^3}{1+e} \qquad 2.6$$

Here, S = specific surface area per unit volume of total particles

T = tortuosity which takes into account of the winding path actually followed by permeating fluid

e = void ratio of soil.

The Kozeny-Carman equation works well for describing coarse grained soils such as sand and some silts. For these case, the coefficient of permeability bears a linear relation to $e^3/(1+e)$. However, serious discrepancies are observed when Kozeny-Carman equation is applied to clayey soils (Olsen 1961, 1962).

Taylor (1948) showed that the permeability k of sand can be expressed as

$$k = C_s D_s^2 \frac{\gamma_w}{\mu} \frac{e^3}{1+e}$$

$$= C \frac{e^3}{1+e} \qquad 2.7$$

Here, $C = C_s D_s^2 \frac{\gamma_w}{\mu}$

D_s = a hyphothetical diameter of spherical grains.

Eq. 2.7 is based on considering flow through a porous media similar to flow through a bundle of capillary tubes.

The theoretical models developed by Kozeny, Kozeny-Carman and Taylor take into account of many factors e.g. pore shape factor, tortuosity, grain size, specific surface, viscosity and unit weight of pore fluid. In the laboratory, it is difficult to measure and represent all these parameters for a particular type of soil. Substituting all these parameters into a single coefficient, Samarasinghe et al (1982) suggested a theoretical model to predict the permeability of normally consolidated clay from laboratory test. The model suggested by him is as follows

$$k = C \frac{e^n}{1+e} \qquad 2.8$$

here, C = a constant in the same unit as k

n = a constant depending on type of soil.

Both C and n are positive Eq. 2.8 shows that for a particular value of n a plot of k versus $e^n/(1+e)$ results in a straight line passing through the origin as shown in Fig. 2.3. Eq. 2.8 can be written as follows

$$\log [k(1+e)] = n \log e + \log C \qquad 2.9$$

Eq. 2.9 shows that a plot of log $[k(1+e)]$ versus loge results in a straight line. Consequently, n is the slope of the straight line and log C is the vertical intercept as shown in Fig. 2.4. n and C are referred to as permeability parameters. Fig. 2.5 gives examples of k versus $e^n/(1+e)$ relationship for various porous materials including soil. The values of n reported in this figure are obtained from experimental data plotted in a manner shown in Fig. 2.4.

The theoretical model (Eq. 2.8) suggested by Samarasinghe et al has been verified experimentally for a sandy clay soil. In order to establish this model further experimental works should be carried out for different types of clay, especially for silty clays and silts available in different regions of Bangladesh. Moreover, a review of the published work for soils of Bangladesh reveals that no information is yet available to relate coefficient of permeability with void ratio for saturated normally consolidated clays and silts.

2.4 Factors Affecting Coefficient of Permeability

The coefficient of permeability of a soil is influenced by both permeant and soil characteristics. Eqs.2.6 and 2.7 provide an idea of the factors that affect the coefficient of permeability of soil.

2.4.1 Effect of Permeant Characteristics

Equations 2.6 and 2.7 show that both viscosity and unit weight of the pore fluid affect the value of the coefficient

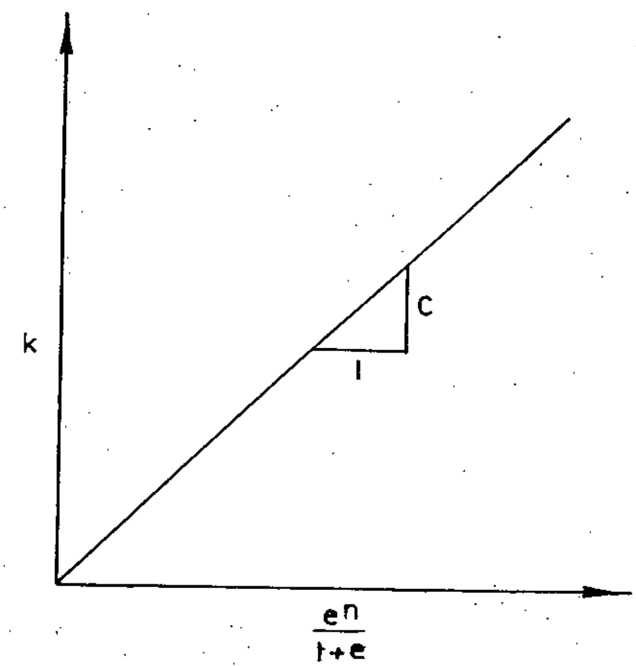


Fig. 2.3 k versus $e^n/(1+e)$.

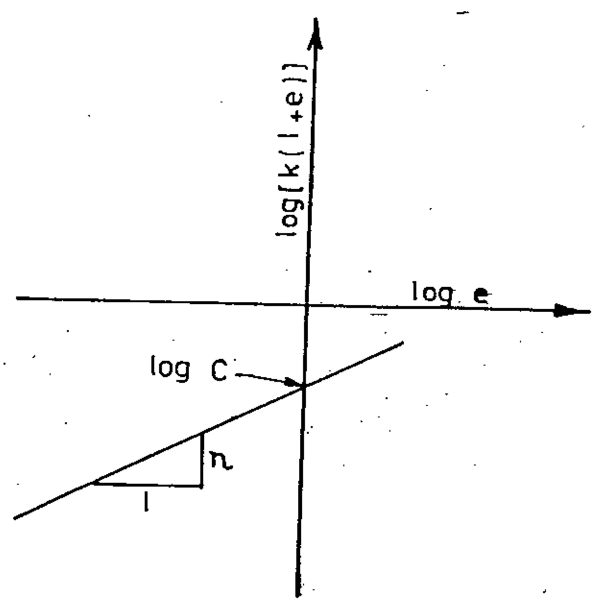


Fig. 2.4 log $[k(1+e)]$ versus loge.

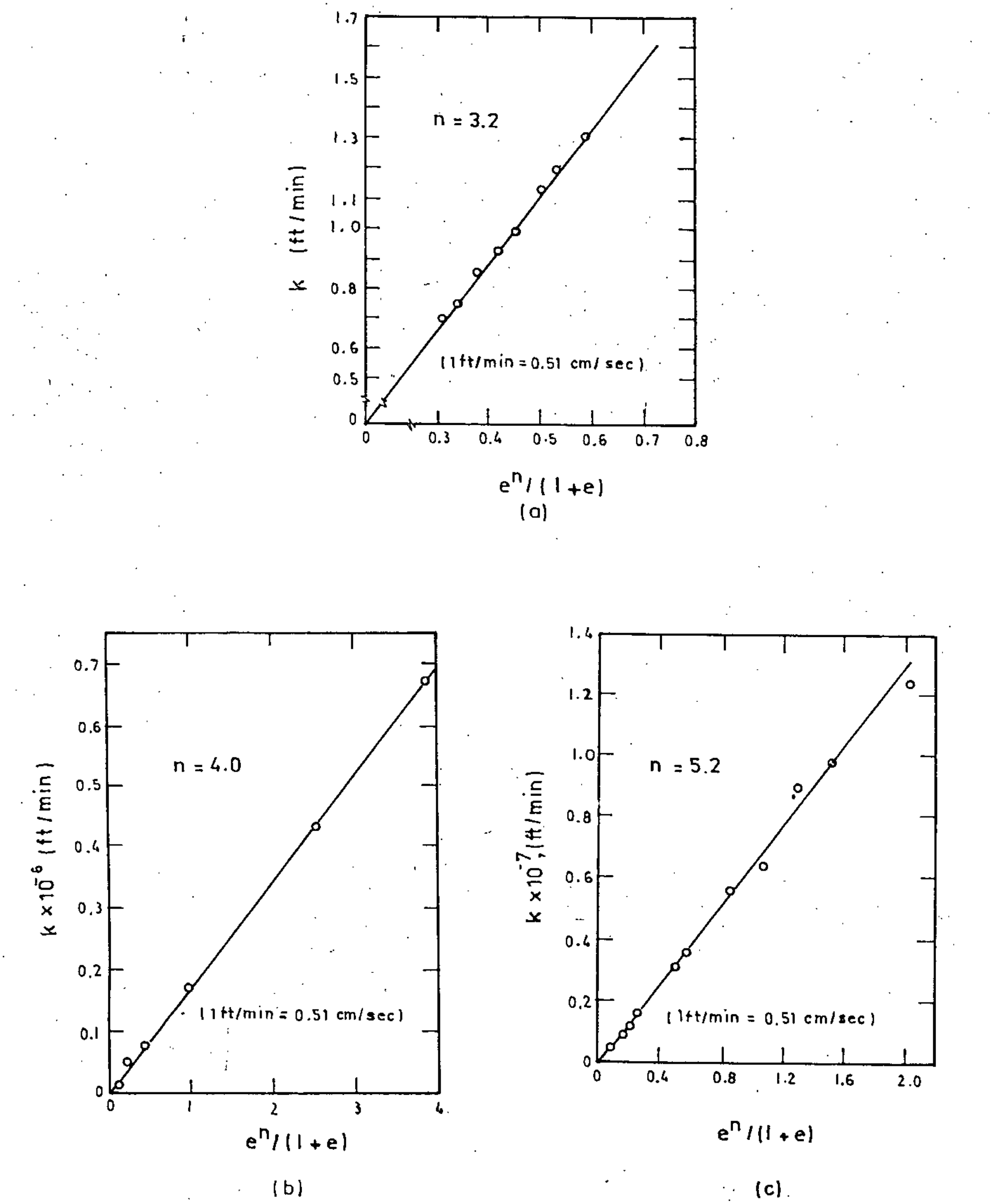


Fig. 2.5 k versus $e^n/(1+e)$ relationship for various porous materials (a) for crushed glass (after Loudon, 1952) (b) for normally consolidated kaolinite (after Michaels and Lin, 1954) (c) for New Liskeard clay (after Raymond, 1966).

of permeability. Permeability is directly proportional to the unit weight and inversely proportional to the viscosity of the permeant. These two permeant characteristics can be eliminated as variables by defining another permeability called the specific or absolute permeability as

$$K = \frac{k\mu}{\gamma_w} \qquad 2.10$$

In any soil the coefficient K has the same value for all permeants and all temperatures as long as the void ratio and fabric of the soil skeleton are not changed.

Michaels and Lin (1954a) conducted tests on kaolinite to examine whether fluid properties other than viscosity and unit weight did affect permeability in clay systems. Kaolinite, initially packed with water was replaced carefully by displacing the water with dioxane, acetone and drynitrogen in sequence. When the kaolinite was initially packed in an organic liquid and was subsequently replaced with dry nitrogen, its permeability in the original organic liquid and in the gas was the same. When the permeability of the clay in nitrogen was measured and compared with its original permeability of water, the permeability in nitrogen always exceeded the water permeability by 40 to 60 percent. The results thus showed that the permeability decreased markedly with increasing polarity of the permeant. They explained that kaolinite was less permeable to organic liquids, water and aqueous solutions than to gases as due to more complete dispersion of the solid in presence of more highly polar permeant fluids. They also observed that in gases and organic liquids, the phenomena of colloidal effects, such as absorbed liquid surface films, and electro-osmotic counterflow did not affect permeability. However, these effects did occur in water or aqueous solutions with small influence on the permeability.

Later Michaels and Lin (1954b) carried out further permeability test on kaolinite and found that the permeability reduction due to electroviscous effect was only about 5 percent. This was found by successively increasing the salt concentration of the permeant water to decrease the electrical double layers of the particles, and thereby the electroviscous effects. The maximum increase in permeability was only about 10 percent.

2.4.2 Effect of Soil Characteristics

The following five characteristics of soil influence the value of coefficient of permeability (Lambe and Whitman, 1969).

1. Particle size
2. Void ratio
3. Composition
4. Fabric
5. Degree of saturation

Eq. 2.7 suggests that coefficient of permeability varies with the square of some particle diameter. A relationship between permeability and particle size is much more reasonable in silts and sands than in clays, since in silts and sands the particles are more nearly equidimensional. Hazen's (1892) equation for coefficient of permeability gives values based on effective grain size D_{10} only. The observations of Hazen were limited to filter sands of grain size between 0.1 and 3.0 mm of fairly uniform grain size. Hazen's equation, therefore, is only applicable for uniform fine sand. It should not be used for graded sand or clay. Slichter (1899) devised an equation which is also applicable for uniform sand. Terzaghi (1925b) used an empirical relation that was extended to cover sand of non-uniform grain size and variable grain shape.

It is known that the coefficient of permeability of a given soil is some function of the void ratio if the pore fluid remains the same. Many authors have suggested various void ratio functions for permeability (Franzini, 1951; Lambe and Whitman, 1969; Taylor 1948; Samarasinghe et al, 1982).

For any clay Taylor (1948) found that a plot of void ratio to natural scale against the coefficient of permeability to logarithmic scale can be approximated by a straight line. However, the experimental results do not always support such a linear relationship (Michaels and Lin, 1954a; Raymond, 1966) as shown in Fig. 2.6.

Lambe (1954) showed that for fine grained soils k versus $e^3/(1+e)$ is not a straight line. However, Lambe's experimental results showed that generally, the plot of void ratio versus log-permeability is approximately a straight line.

The Kozeny-Carman equation (Eq. 2.6) does not successfully explain the variation of the coefficient of permeability with void ratio for clayey soils (Olsen 1961, 1962). The marked degree of variation between the theoretical and experimental values arise from several factors, including deviations from Darcy's law, high viscosity of the pore water (Terzaghi 1925a; Macey 1942), tortous flow paths (Lambe, 1958), Electrolytic coupling (Elton, 1948) and unequal pore sizes (Micheals and Lin, 1954a).

The influence of soil composition on permeability is of major importance with clays (Cornell, 1951). The very large influence composition can have on clay permeability is shown in Fig. 2.7. The magnitude of permeability variation with soil composition ranges widely. Fig. 2.7 shows that the ratio of permeability of calcium montmorillonite to that of potassium montmorillonite at a void ratio of 7 is approximately 300. It further shows that the permeability of kaolinite is a hundred times that of montmorillonite.

The fabric component of structure is one of the most important characteristics influencing permeability (Mitchell et al, 1965). At the same void ratio, fine grained soils with

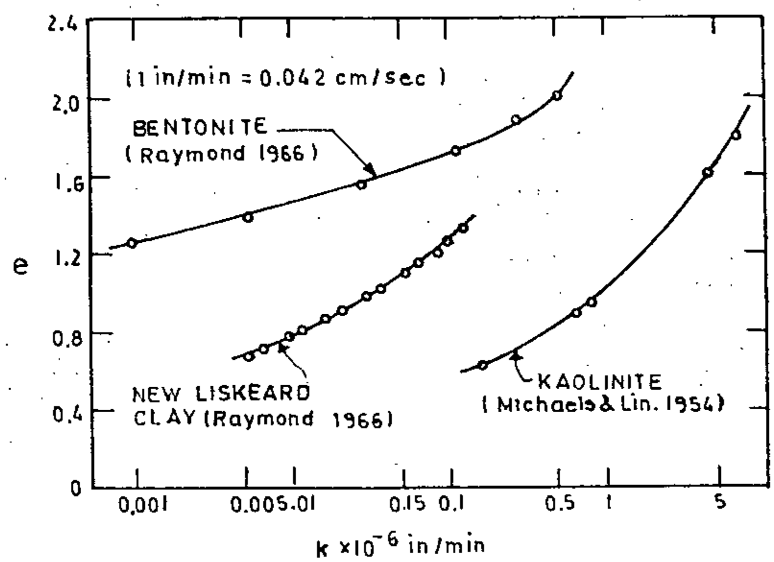


Fig. 2.6 Nonlinear relationship between e and log k. (after Samarasinghe et al, 1982).

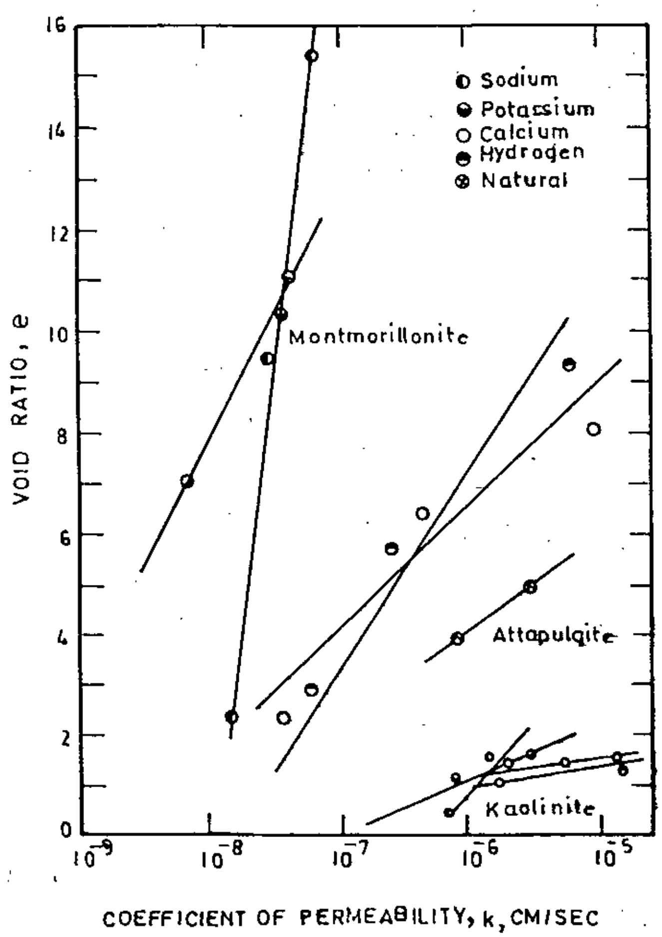


Fig. 2.7 Influence of soil composition on permeability (after Cornell, 1951).

a flocculated structure have higher coefficient of permeability than those with dispersed structure. This fact is demonstrated in Fig. 2.8 for the case of a silty clay. Macrostructure is also of considerable importance. A stratified soil has much higher permeability for flow parallel to stratifications than it does for flow perpendicular to stratification.

The degree of saturation has an important influence on permeability of soil (Mitchell et al, 1965). An increase in saturation leads to an increase in the coefficient of permeability. Fig. 2.9 shows the effect of degree of saturation on coefficient of permeability for a silty clay.

2.5 Methods of Determination of Coefficient of Permeability in Laboratory

The three most common laboratory methods for determining the coefficient of permeability are the following:

1. Constant head test
2. Falling head test
3. Indirect determination from consolidation test

The constant head test is performed by measuring the quantity of flow in a specified period of time through soil specimen of known cross-sectional area and length under a constant hydraulic gradient. In the constant head permeability test, coefficient of permeability can be determined by application of back pressure in soil specimen placed in specially designed permeameter.

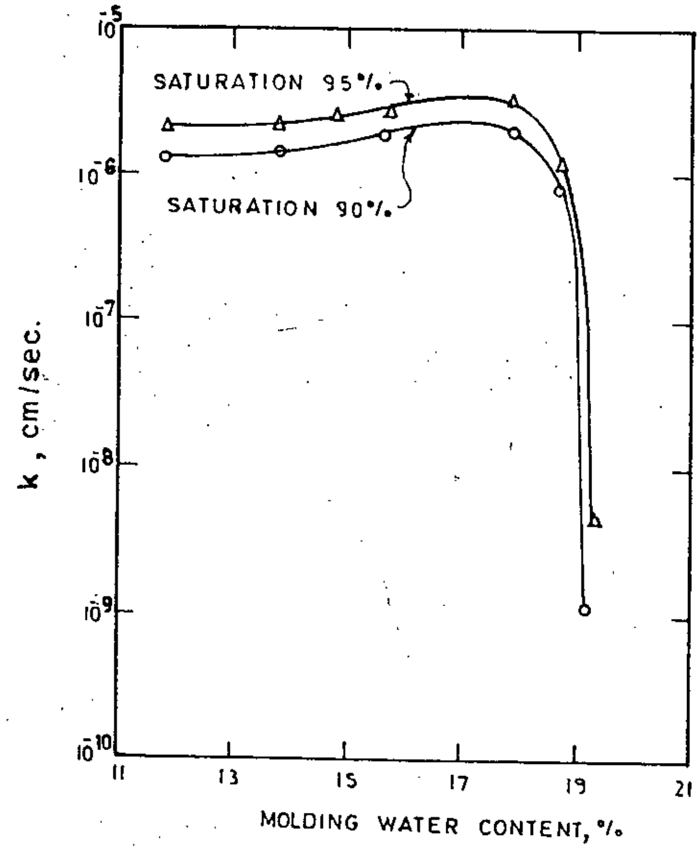


Fig. 2.8 Dependence of permeability on the structure of a silty clay (after Mitchell et al, 1965).

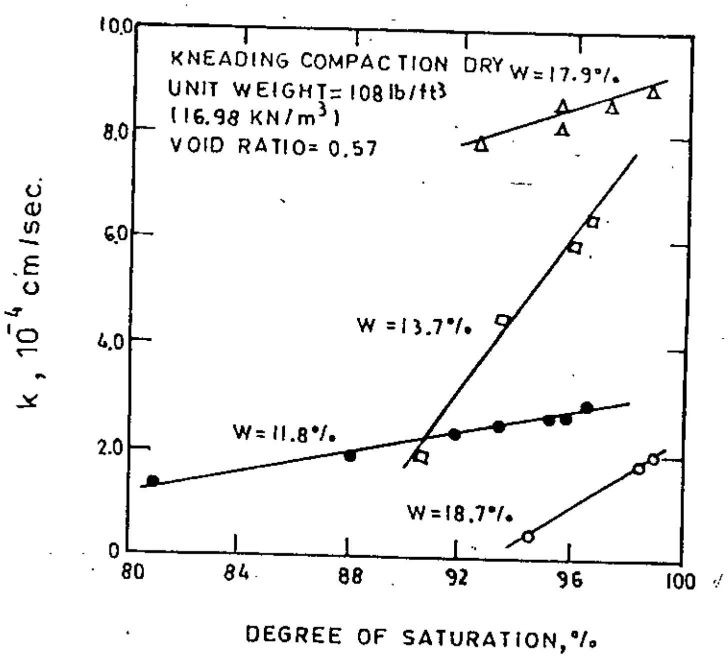


Fig. 2.9 Influence of degree of saturation on permeability of compacted silty clay (after Mitchell et al, 1965).

In falling head permeability test, coefficient of permeability is determined by measuring fall of head in a stand pipe of known cross-sectional area due to flow of water in a specific time period through the soil specimen of known cross-sectional area and length. Hvorslev (1951) discussed the limitations of falling head permeability test and he concluded that falling head permeability tests on fine grained soils lead to unreliable results because of swelling and reconsolidation of the samples due to variation of pore water pressure during the test. Standard test procedure for determining coefficient of permeability in laboratory is described by Lambe (1951).

The coefficient of permeability of a soil can be computed from its rate of consolidation under a given stress increment because the rate of consolidation of a soil depends on its coefficient of permeability. The coefficient of permeability of a soil can be determined from the following equation

$$k = \frac{\gamma_w \, C_v \, (\frac{de}{d\bar{\sigma}})}{1+e} \qquad 2.11$$

when e-log $\bar{\sigma}$ plot is linear, as in case of normally consolidated soil,

$$k = \frac{\gamma_w \, C_v \, C_c}{2.303(1+e)\,\bar{\sigma}}$$

where $\bar{\sigma}$ = average value of effective consolidation pressure

C_v = coefficient of consolidation

C_c = compression index

e = void ratio of soil

γ_w = unit weight of water

2.6 Terzaghi's One-Dimensional Consolidation Theory

When consolidation takes place due to expulsion of pore water vertically from a laterally confined mass, the consolidation is considered as one dimensional. Terzaghi (1943) developed a mathematical statement for one dimensional consolidation process on the basis of some assumptions and the partial differential equation developed for saturated soils is given as:

$$\frac{\partial u}{\partial t} = \frac{k}{\gamma_w} \cdot \frac{1+e}{a_v} \cdot \frac{\partial^2 u}{\partial z^2} = C_v \frac{\partial^2 u}{\partial z^2} \qquad 2.13$$

where, k = coefficient of permeability

e = void ratio

a_v = coefficient of compressibility

u = excess pore water pressure

z = co-ordinate of flow direction

C_v = coefficient of consolidation

Eq. 2.13 can be solved with proper boundary conditions assuming u to be the product of two functions i.e the product of a function of z and a function of t. The solution of Eq. 2.13 can be expressed by the following relationships

$$U_z(\%) = f(T_v, z/H_{dr}) \qquad 2.14$$

here, U_z = percent consolidation

T_v = time factor

H_{dr} = longest drainage path

The details of Terzaghi's one dimensional consolidation theory and its mathematical solution are available in "Theoretical Soil Mechanics" by Terzaghi (1943).

2.7 Deviations From Terzaghi's Theory

Expression for the rate of consolidation was first developed by Terzaghi (1943) for the special case of one dimensional flow from laterally confined soil. However important abberations are observed between predictions made by the Terzaghi theory and observations of the time rate of consolidation, both in the field and during laboratory tests. The most notable difference in the time-deformation curves occur when the theoretical primary curve approaches its 100 percent consolidation. It is found that the experimental time consolidation curve is in agreement with Terzaghi's theory of consolidation only upto about 60 percent consolidation. This difference between observed consolidation is due to the secondary time effects and defined as secondary compression.

2.8 Laboratory Evaluation of Consolidation Characteristics

Compressibility of a fine grained soil is usually determined directly by performing a laboratory compression test, called one dimensional consolidation test. In this test, undisturbed or remoulded sample is fit into a ring or cylinder so that the soil sample is confined against lateral displacement and compressive loading is imposed on the soil. For known magnitudes of load, the amount of compression and the time required for compression to occur are recorded. The test is usually performed by imposing a series of increasing compressive loadings and determining time-rate-of-compression data for each increment of loading. Detail procedure for one dimensional consolidation test is available in Annual Book of ASTM standards, 1979, Part 19.

Recently, two other one-dimensional consolidation test procedures have been developed which are much faster but yet give reasonably good results. The methods are (1) constant rate-of-strain-consolidation (CRSC) test and (2) the controlled gradient consolidation (CGC) test. CRSC test was developed by Smith and Wahls (1969) and CGC test was developed by Lowe et al (1969).

2.9 Methods of Determination of Coefficient of Consolidation from Laboratory Tests

For a given stress increment, the coefficient of consolidation C_v can be determined from laboratory observations of time versus dial reading. Two graphical procedures are commonly used for this are the logarithm of time method proposed by Casagrande and Fadum (1940) and the square root of time method proposed by Taylor (1942). There are also two other useful methods, which are proposed by Su (1958) and Sivaram and Swamee (1977).

2.10 Compression Index-Initial Void Ratio Relationship

Compression index C_c determined from laboratory consolidation tests on clays have been plotted against initial void ratio e_o by several authors. The empirical equations derived by them are as follows:

$$C_c = 0.30\ (e_o - 0.27) \tag{2.15}$$

$$C_c = 0.54\ (e_o - 0.35) \tag{2.16}$$

$$C_c = 0.75\ (e_o - 0.60) \tag{2.17}$$

$$C_c = 0.44\ (e_o - 0.30) \tag{2.18}$$

$$C_c = 0.44\ (e_o - 0.36) \tag{2.19}$$

Eq. 2.15 was derived by Hough (1969) for remoulded clays. Eqs. 2.16 and 2.17 were developed by Nishida (1956) and Sowers (1962) respectively for undisturbed clays. Eqs. 2.18 and 2.19 have been derived by Serajuddin and Ahmed (1967) and Kabir (1978) for undisturbed fine grained soils of Bangladesh. All the above equations are approximate and hence should be used for initial computation only. Relationship between compression index and void ratio for saturated normally consolidated clays and silts of Bangladesh is not a available in published research work and therefore attempts should be made to relate compression index and void ratio for saturated normally consolidated clays and silts available in Bangladesh.

2.11 Relationship between Coefficient of Consolidation and Effective Vertical Pressure

Samarasinghe et al (1982) derived a theortical relationship between coefficient of consolidation C_v and effective vertical pressure $\bar{\sigma}$ for normally consolidated clay. The relationship is as follows:

$$C_v = \frac{2.303C}{\gamma_w C_c} \left(-C_c \log \frac{\bar{\sigma}}{\bar{\sigma}_1} + e_1\right)^n \bar{\sigma} \qquad 2.20$$

Here, $\bar{\sigma}_1$ = unit effective vertical pressure

= 1 tsf

C_c = compression index i.e slope of e-log $\bar{\sigma}$ curve

e_1 = value of void ratio when effective vertical pressure is $\bar{\sigma}_1$

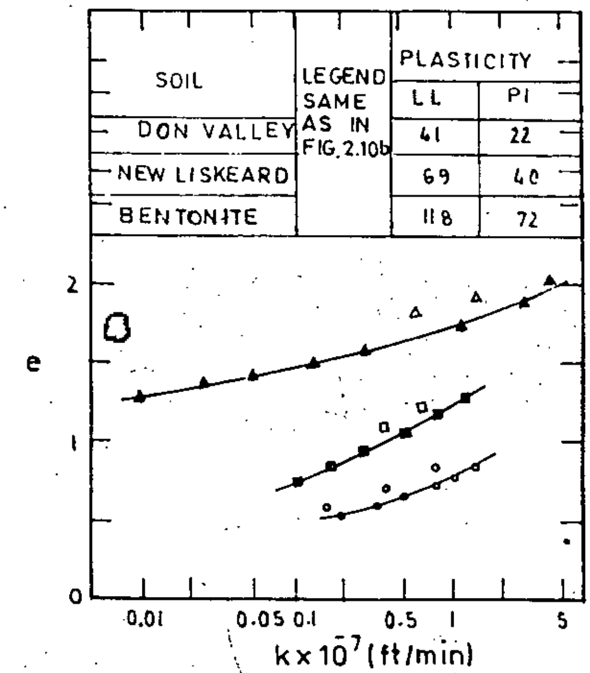
C_c and e_1 and called compressibility parameters. Eq. 2.20 is derived without assuming that k is constant and that a linear relationship exists between e and $\bar{\sigma}$, as assumed in the Terzaghi theory. The equation derived assumes that e-log $\bar{\sigma}$ plot can be approximated by a straight line, which is applicable to normally consolidated soils. The details of derivation of Eq. 2.20 is given in Appendix-C.

2.12 Permeability and Consolidation Characteristics of Normally Consolidated Clays

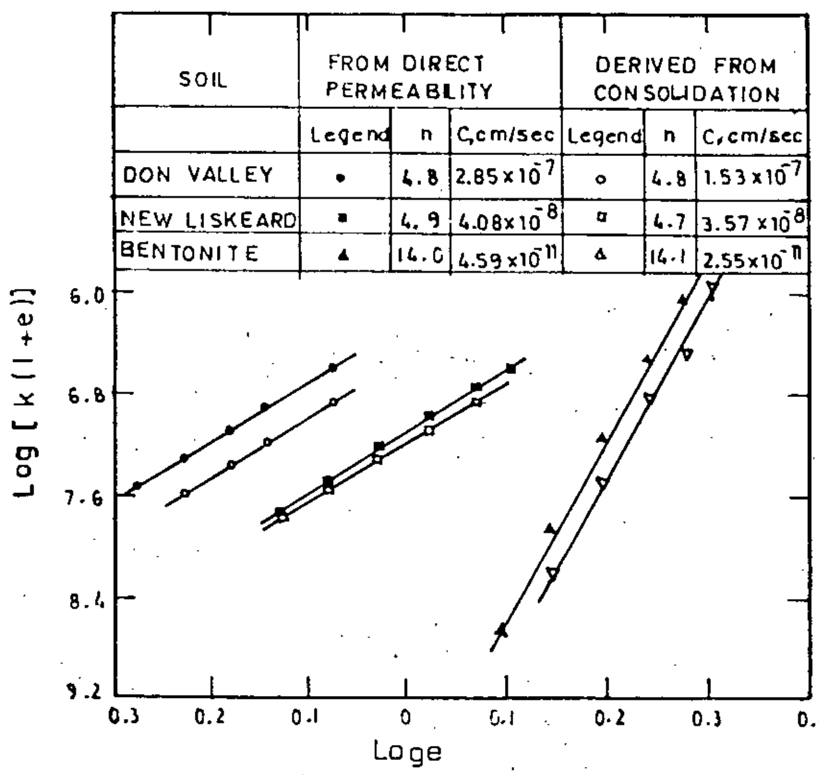
Raymond (1966) conducted hydrostatic consolidation test and permeability tests on the same specimen. He presented results for three different clays ranging from medium to high plasticity. All specimens were artificially sedimented. Consolidation tests were performed with a stress increment ratio of 1 (i.e a load ratio of 2), and direct permeability tests were run on the same specimen at the end of each stress increment. The directly measured permeability was compared with the permeability derived indirectly from the consolidation test results.

Void ratio versus permeability plots from both the direct and indirect tests on the same specimen of each of the three soils are shown in Fig. 2.10(a). Corresponding $\log[k(1+e)]$ versus log e plots are shown in Fig. 2.10(b). From Fig. 2.10(a) it appears that permeability of each soil did not vary significantly depending on the method of test. From Fig. 2.10(b) it is observed that the permeability parameter n is about the same for each soil. However permeability parameter C varies with the method of test. Typical linear e-log $\bar{\sigma}$ plots for the three soils are shown in Fig. 2.11.

Samarasinghe et al (1982) proposed a unique relationship (Eq.2.8) between permeability and void ratio. This relationship is then used for deriving the governing differential equation of consolidation for normally consolidated soils without assuming



(a)



(b)

Fig. 2.10 Permeability plots from two different methods on same specimen (a) e versus log k plots (b) log [k(1+e)] versus loge plots (after Raymond, 1966).

that k is constant and that a linear relationship exists between e and $\bar{\sigma}$, as assumed in the Terzaghi theory. The equation derived assumed that e-log $\bar{\sigma}$ plot can be approximated by a straight line, which is applicable to normally consolidated soils. It is shown that C_v has a direct relationship to $\bar{\sigma}$. The theoretical relationship between C_v and $\bar{\sigma}$ is compared with experimental results which utilize the Terzaghi theory. To demonstrate the relationships between k and e and between C_v and $\bar{\sigma}$, Samarasinghe et al (1982) conducted laboratory tests on a greyish sandy clay with liquid limit 27 and plastic limit 14. A direct permeability test as well as standard incremental loading consolidation test were run for the same specimen. The specimen was artificially sedimented to get a normally consolidated sample. A series of consolidation tests were run increasing the load at regular time intervals. The permeability tests were run at the end of each load increment when the primary consolidation was complete. Figs. 2.12 and 2.13 show, respectively, the compressibility and permeability plots. From these two linear plots compressibility and permeability parameters were determined. The C_v - log $\bar{\sigma}$ curve obtained by using Eq. 2.20 is shown Fig. 2.14 together with the experimental points obtained from log time and square root of time fitting methods for each pressure increment. Fig. 2.14 shows that both the theoretical and experimental curves have the same shape.

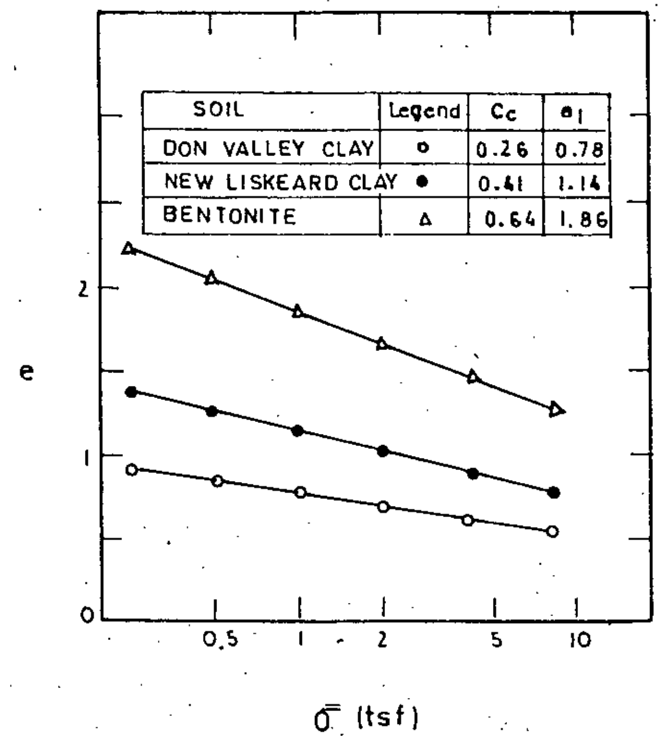


Fig. 2.11 Typical compressibility characteristics (after Raymond, 1966).

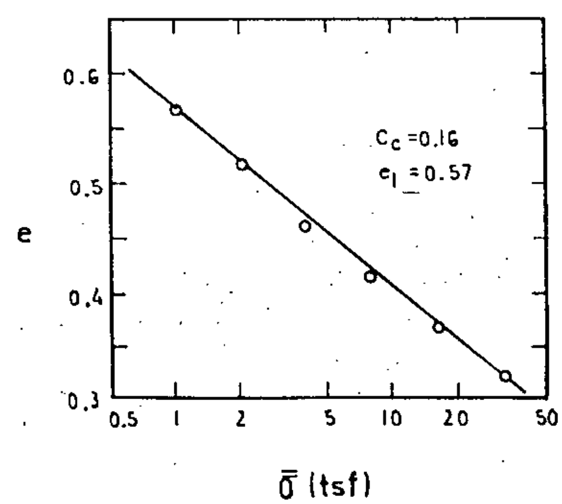


Fig. 2.12 Compressibility plot for greyish sandy clay (after Samarasinghe et al, 1982).

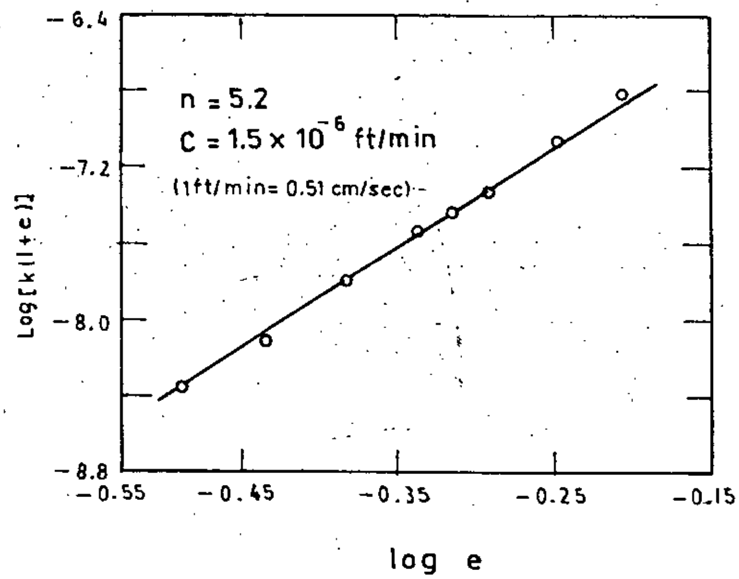


Fig. 2.13 Permeability plot for greyish sandy clay (after Samarasinghe et al, 1982).

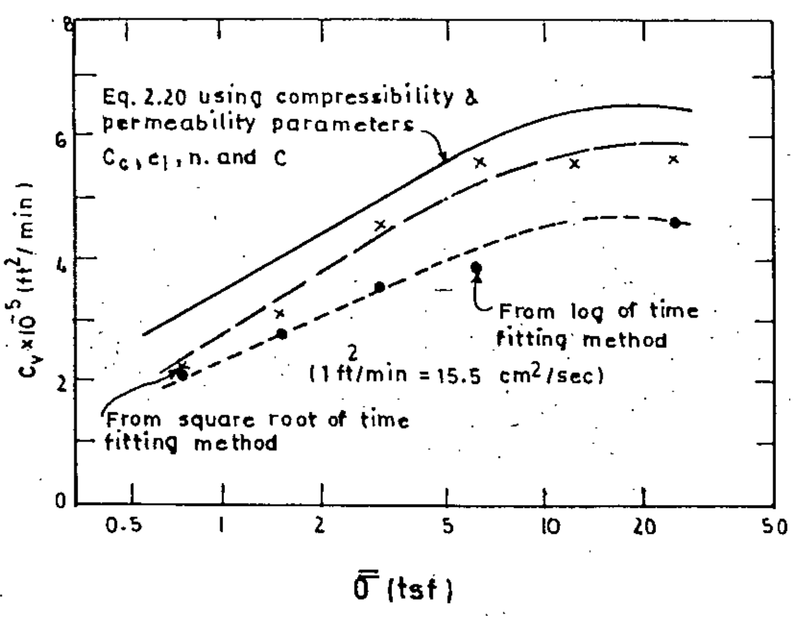


Fig. 2.14 Relationship between C_v and log $\bar{\sigma}$ for greyish sandy clay (after Samarasinghe et al, 1982).

Samarasinghe et al (1982) observed the shape of theoretical and experimental C_v - log $\bar{\sigma}$ curves for a sandy clay soil only. In order to confirm their findings regarding the shape of theoretical and experimental C_v - log $\bar{\sigma}$ curves more research should be done with different types of soils e.g. silty clay, silts etc.

2.13 Relation between Constrained Modulus and Effective Vertical Consolidation Pressure

Stamatopoulos and Kotzias (1978) plotted constrained modulus D against effective vertical consolidation pressure $\bar{\sigma}$ (linear scale). Such a plot has been found to have the characteristic U shape as shown in Fig. 2.15. This shape has been reported explicitly by Stamatopoulos and Kotzias (1973) and also encountered incidentally by Janbu (1969) and by Wissa et al (1971) while investigating special applications of the oedometer test. This shape holds for soils of variable type and origin e.g. clay, silt, sand, organic, sedimentary, residual and so on.

The D against $\bar{\sigma}$ curve is approximated by

$$D = a/\bar{\sigma} + b\bar{\sigma} + c \qquad 2.21$$

where a, b and c are arithmetic coefficients and are given by the following expressions:

$$b = \frac{\bar{\sigma}(D - D_o)}{(\bar{\sigma} - \bar{\sigma}_o)^2} \qquad 2.22$$

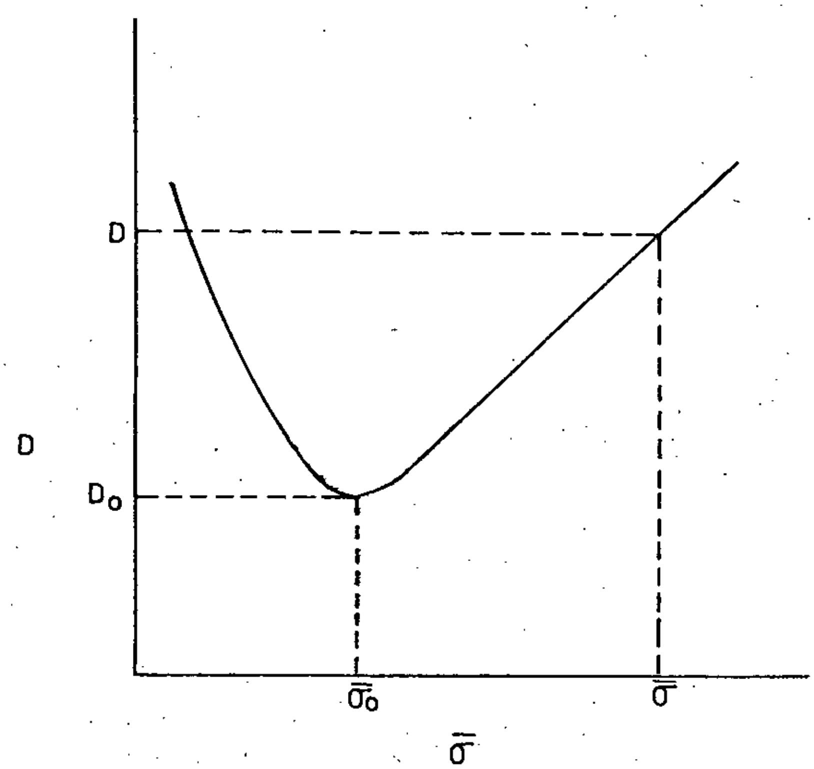


Fig. 2.15 Characteristic shape of D against $\bar{\sigma}$ curve (after Stamatopoulos and Kotzias, 1973).

$$a = \bar{\sigma}_o^2 b \qquad 2.23$$

$$c = D_o - 2b\,\bar{\sigma}_o \qquad 2.24$$

where $\bar{\sigma}_o$, D_o correspond to the point of minimum D and $\bar{\sigma}$, $\bar{\sigma}$ to some point near the end of curve ($\bar{\sigma} > \bar{\sigma}_o$). Coefficients a and b are always positive whereas c can be either positive or negative.

Soils of high compressibility have b values of the order of 10 and soils of low compressibility have b values of the order of 50 (dimensionless). Soils of high compressibility have a values of the order of 5 kg^2/cm^4 and soils of low compressibility have a values of the order of 500 kg^2/cm^4. Soils of high compressibility have c values between 5 and 15 kg/cm^2. On the other hand, the c values of relatively incompressible soils can vary between wide limits of -100 kg/cm^2 to + 100 kg/cm^2.

2.14 Procedure of Settlement Calculation

Stamatopoulos and Kotzias (1978) proposed a procedure for settlement calculation which is briefly discussed below:

The change in vertical unit deformation or strain ΔS is tabulated with the corresponding consolidation pressure σ, and for each increment of pressure $\Delta\sigma$ the constrained modulus $D = \Delta\sigma/\Delta S$ is calculated. Each value of D is taken to correspond to the midpoint of the pressure increment. The value of D is then plotted against $\bar{\sigma}$.

In order to estimate the settlement of layer of thickness H subjected to a change of pressure $\Delta\sigma$ the value of D corresponding to the applied pressure is chosen from D against $\bar{\sigma}$ plot and this is used in the simple equation

$$\text{Settlement } S_t = \frac{\Delta\sigma}{D} H_t \qquad 2.25$$

It is noted that it is not required to calculate void ratios or C_c as an intermediate step in the calculation of settlement.

Studies on the relationship between constrained modulus and effective vertical pressure are important because such relationship leads to an approximate but convenient method of estimating settlements through constrained modulus. However, estimation of settlement on the basis of constrained modulus has not yet been used in Bangladesh. Research should be carried out to establish relation between constrained modulus and effective vertical pressure for saturated normally consolidated silty clays and silts available in Bangladesh.

2.15 Summary of the Literature Review

The literature review presented in this chapter may be summarized as follows:

i) In sand Darcy's law is generally found to be valied. In clay, however, the law was found, in general, to be valid, except under some special situations.

ii) Permeability of soil depends on the characteristics of both the permeant and the soil. The permeant characteristics include viscosity, unit weight and nature of the permeant while the soil characteristics include particle size, void ratio, composition, fabric and degree of saturation.

iii) Constant head permeability test by application of back pressure is suitable and reliable to determine permeability of fine grained soils. Falling head test on fine grained soils leads to unreliable results because of swelling and reconsolidation of the samples due to variation of pore water pressure during the test. Permeability of clays can be determined indirectly from consolidation test.

iv) For a particular type of soil, a unique relationship exists between the coefficient of permeability and void ratio for normally consolidated clays.

v) Permeability parameters n and C can be obtained from a linear plot of log $[k(1+e)]$ versus loge. Available test results have indicated that n increases with the increase in liquid limit and plasticity index.whereas C decreases with increasing liquid limit and plasticity index.

vi) Compressibility parameters C_c and e_1 for normally consolidated clays can be obtained from a linear plot of e versus log $\bar{\sigma}$.

vii) For normally consolidated clay C_v - $\bar{\sigma}$ relation can be determined directly from Eq. 2.20 by finding n, C, C_c and e_1. This eliminates the use of time fitting methods for every stress increment. Eq. 2.20 is strictly valid only for the range of $\bar{\sigma}$ in which e-log $\bar{\sigma}$ plot is linear. The theoretical C_v -log $\bar{\sigma}$ curves drawn by utilizing Eq. 2.20 and the experimental C_v -log $\bar{\sigma}$ curves drawn by utilizing Terzaghi's theory have the same shape.

viii) Constrained modulus of clay is related to effective vertical pressure by more or less simple relation. This relation can be used to estimate settlement on the basis of constrained modulus.

CHAPTER 3

THE RESEARCH SCHEME

3.1 Introduction

Permeability and consolidation of saturated normally consolidated clays are two important properties that must be investigated properly. Some of the theoretical developments that have taken place in this field have been reviewed in the previous chapter. Samarasinghe et al (1982) has proposed a model to predict permeability of normally consolidated clays. This model is based on earlier relations provided by Kozeny and Carman (Kozeny, 1927 ; Carman, 1956) and Taylor (1948). Also using this model, Samarasinghe et al proposed another model for consolidation characteristics of normally consolidated clays. These have been discussed in the previous chapter. The models are yet to be verified for different types of clay, particularly for silty clays and silts available in Bangladesh. It has also been shown in the literature review that simple relation exists between constrained modulus and effective vertical pressure. Such relation is likely to provide simple method of estimating settlement in compressible soils. This approach has not been investigated for normally consolidated silty clays and silts and is therefore included in this research.

3.2 Objective of the Research

The broad aim of this research was to verify the existing models and to develop an appropriate model for permeability and compressibility behavior of saturated normally consolidated silty clays and silts. To achieve this the following evaluations were considered necessary for this research.

i) Determination of permeability parameters n and C from both permeability and consolidation tests.

ii) Determination of compressibility parameters C_c and e_1 from consolidation test.

iii) Correlation of the permeability and compressibility parameters with index properties of soils.

iv) Establishment of the nature of relation between coefficient of permeability and void ratio of normally consolidated silty clays and silts.

v) Establishment of the nature of relation between coefficient of consolidation C_v and effective vertical pressure $\bar{\sigma}$ for normally consolidated silty clays and silts.

This research also aimed at developing a simple model to relate constrained modulus D and effective vertical pressure for normally consolidated silty clays and silts. Such a model can be used as a simple method of settlement estimation by using Eq. 2.25.

3.3 Research Scheme

The whole research was divided into the following phases:

i) In the first phase, index properties of the soils were determined in order to classify them. Organic matter of the soils were also determined.

ii) In the second phase, some equipments and accessories were designed and fabricated locally for conducting one dimensional consolidation test and constant head permeability test. The locally made components included a consolidation permeability test unit for testing soils in saturated condition, a flow volume measuring unit and a mould for making porous stone.

iii) In the third phase, in order to check the effectiveness of the complete experimental set-up a constant head permeability test as well as a standard incremental loading consolidation test were run for a clayey soil available in the laboratory and thus successful installation of the complete experimental set-up was ensured.

iv) In the final phase, one dimensional consolidation and constant head permeability tests were performed for the selected soils.

The experimental programme that followed is depicted by flow chart is shown in Fig. 3.1.

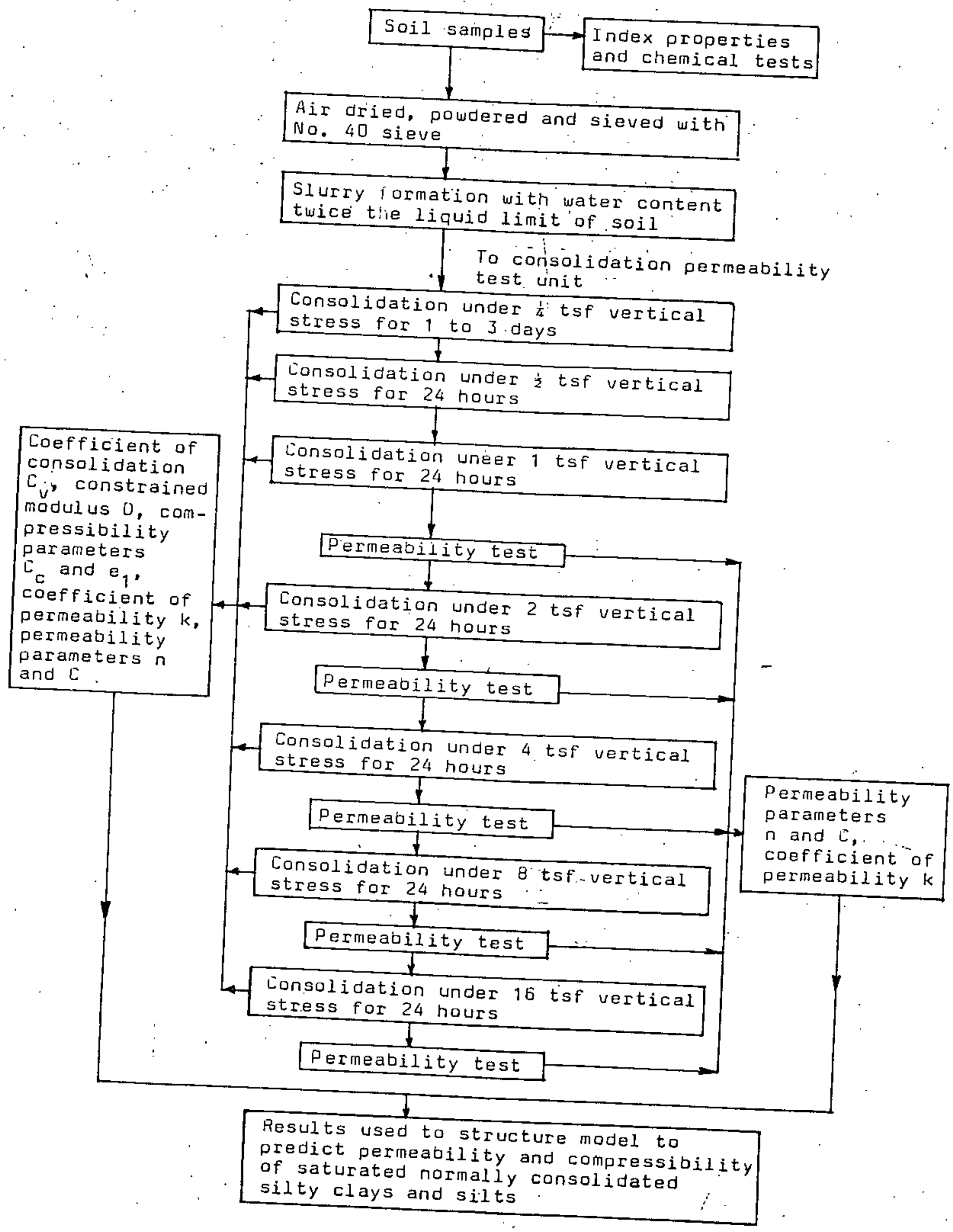


Fig. 3.1 Flow chart for experimental programme.

Four soils with varying index properties were used for this research. Consolidation and permeability tests were repeated for the first two soils and consistent results were obtained for each of these soils. Therefore, consolidation and permeability tests were not repeated for the remaining two soils. So, a total number of six cylindrical samples were tested. Each of the sample was artificially sedimented by applying a vertical pressure of 1/4 tsf to obtain saturated normally consolidated sample. Diameter of each sample prepared was 2.50 inch (63.5 mm) and the height varied between 3.66 cm and 4.35 cm. For each sample consolidation test was run with a stress increment ratio of unity i.e a load ratio of two. The vertical consolidation pressures applied were 1/2 tsf, 1 tsf, 2 tsf, 4 tsf, 8 tsf and 16 tsf. Duration of each loading step was approximately 24 hours. Constant head permeability test was run at the end of stress increment when primary consolidation appeared to be complete. For each sample five sets of permeability test were run at the end of 1 tsf, 2 tsf, 4 tsf, 8 tsf and 16 tsf vertical consolidation pressure. So, a total number of 30 permeability tests were performed. During permeability tests, the applied hydraulic gradients varied from 293 to 857 and void ratio ranged between 0.479 and 0.926.

3.4 Soils Used

Four soils were selected for this research. Of them two were natural soil and the other two were reconstituted at laboratory. Of the natural soil, one was collected from the campus of Bangladesh University of Engineering and Technology, Dhaka from a depth of eight to ten feet, the other was collected from Kaliakoir in Gazipur district from a depth of six to eight feet. The soils were designated as follows:

Soil-I	Collected from BUET campus, Dhaka
Soil-II	Collected from Kaliakoir, Gazipur
Soil-III	Reconstituted
Soil-IV	Reconstituted

The physical and chemical properties of the soils used are listed in Table 3.1. The grain size distribution curves of the soils are shown in Fig. 3.2.

Table 3.1 Results of Physical and Chemical Properties of Soils

	Soil-I	Soil-II	Soil-III	Soil-IV
A. Physical Properties				
i) Grain size distribution				
Sand in percent	13	11	12	13
Silt in percent	65	89	75	79
Clay in percent	22	0	13	8
Percent passing No.200 sieve	89.5	92	91	91.5
ii) Specific gravity	2.68	2.72	2.69	2.70
iii) Atterberg Indices:				
Liquid limit	40	33	36	35
Plastic limit	20	27	22	24
Plasticity index	20	6	14	11
iv) Activity number	0.909	-	1.076	1.375
v) Group classification	CL	ML	CL	CL
B. Chemical properties				
Organic matter (%)	0.34	0.62	0.43	0.49

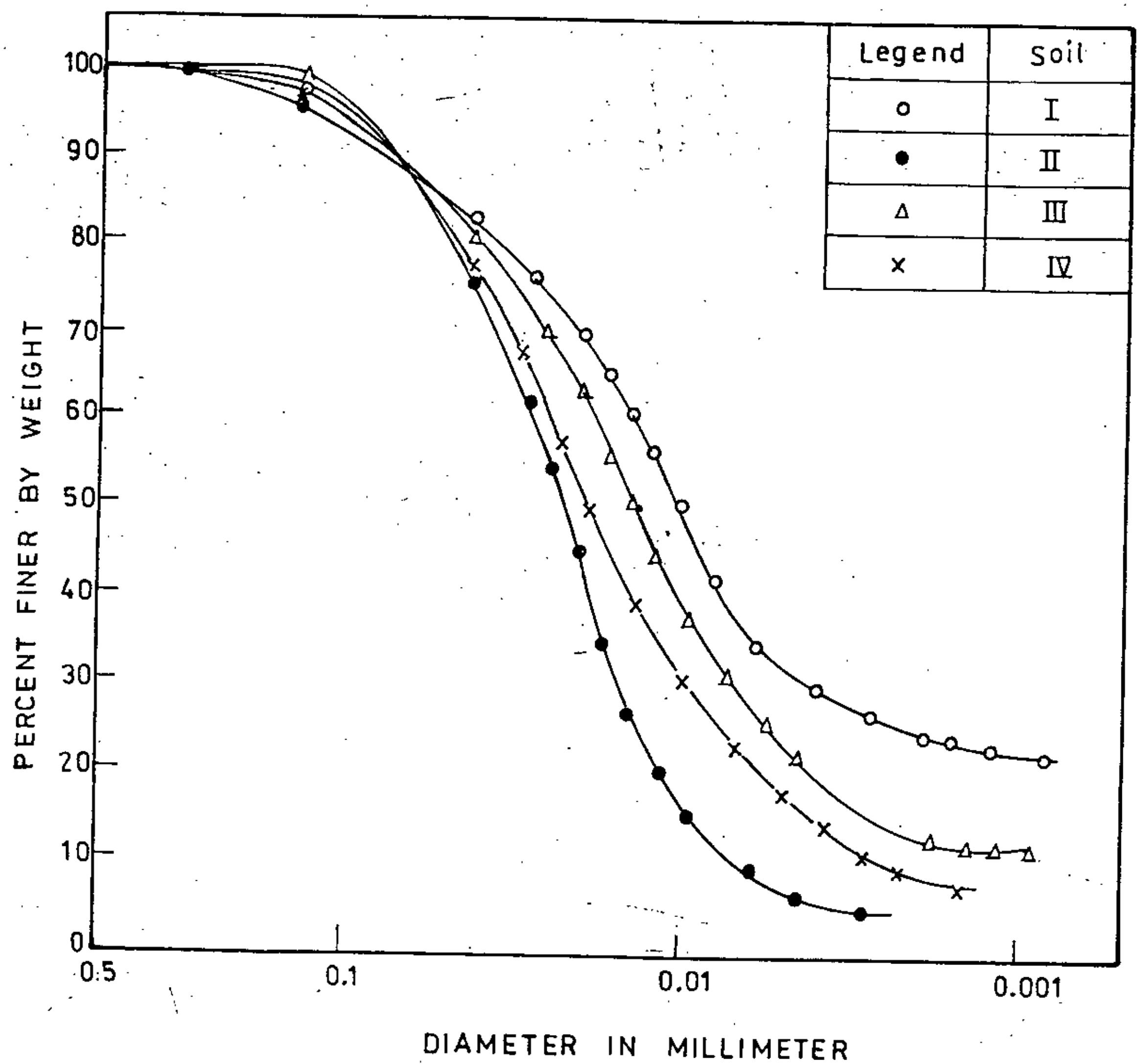


Fig. 3.2 Grain size distribution curves of the soils tested.

CHAPTER 4

LABORATORY INVESTIGATIONS

4.1 Introduction

The investigations in the laboratory were conducted in accordance with the programme outlined in Art. 3.3. The details of experimental set-up and experimental procedure are discussed in this chapter.

4.2 Test Procedures for Classifying Soils

The index properties of the soils used were determined in accordance with the procedures specified by American Society for Testing and Materials (ASTM, 1979). ASTM Standard D423-66(1972) was followed for liquid limit LL, D424-59(1971) followed for plastic limit PI, D854-58(1972) for specific gravity and D422-63(1972) for grain size distribution. The soils were then classified according to unified soil classification system based on ASTM Standard D2487-69(1975). The test results along with their classification and grain distribution are presented in Table 3.1. The grain size distribution curves have been shown in Fig. 3.2.

4.3 Test Procedure for the Determination of Organic Matter

Two gms of oven dry sample after passing through 2 mm sieve was taken into a beaker of 150 ml size. Distilled water was added to the sample to give a 1:2 soil-to-water-ratio and covered the beaker with a ribbed watch-glass. Initially 30% H_2O_2 was added in increments of 5 to 10 ml in order to oxidize the organic matter. For complete subside of frothing, constant stirring was done with a low heat (65^{o}-$70^{o}C$) for 10 to 20 minutes using a water bath. When the sample had lost its dark colour, it was transferred into a centriguge tube for washings of the solution. After several washings, the sample was placed in an oven at a temperature of $105^{o}C$-$110^{o}C$ and weighed to the nearest 0.001 gm after 24 hours. The percentage loss of the sample was calculated as organic content of the sample. The organic matter of each soil is less than 1 percent.

4.4 Equipments Developed for Consolidation and Permeability Test

For permeability and consolidation test the following main equipments were used:

i) A Consolidation permeability test unit

ii) A Flow volume Measuring unit

iii) A Constant Pressure Apparatus

Consolidation and permeability test and flow volume measuring units were designed and fabricated locally the details of which are provided in Arts. 4.5 and 4.6. As porous stones of 2 inch (50.8 mm) diameter were not available in the laboratory, a technique was also developed to prepare the same. This is detailed in Appendix-A.

4.5 Consolidation Permeability Test Unit

It was observed that the apparatus used by Olsen (1962) was relatively simple and could be easily duplicated here. So, Olsen's apparatus with some modifications a consolidation. Permeability test unit made of brass was designed and fabricated. A schematic diagram of the consolidation permeability test unit is shown in Fig. 4.1.

The test unit consists of two major parts, a lower bottom part for sample placement and an upper loading piston. The cylindrical bottom part is $1\frac{1}{8}$" (28.57 mm) thick, $5\frac{1}{2}$" high and has a base diameter of $4\frac{3}{4}$" (120.65 mm).

The principal features of the bottom part are the followings:

i) A central hole of $3\frac{1}{2}$" (88.9 mm) height and $2\frac{1}{2}$" (63.5 mm) diameter for enclosing the sample.

ii) Under the central hole there is a groove of $\frac{1}{2}$" height and 2" diameter for placing porous stone to secure bottom drainage.

iii) Two drilled holes, each 3/16" diameter, at the bottom of the groove extended upto the side walls of the test unit at a height of $\frac{1}{2}$" from the base. Two valves, a and b, are fitted with these holes. These holes were provided for bottom drainage and upward permeation through the sample.

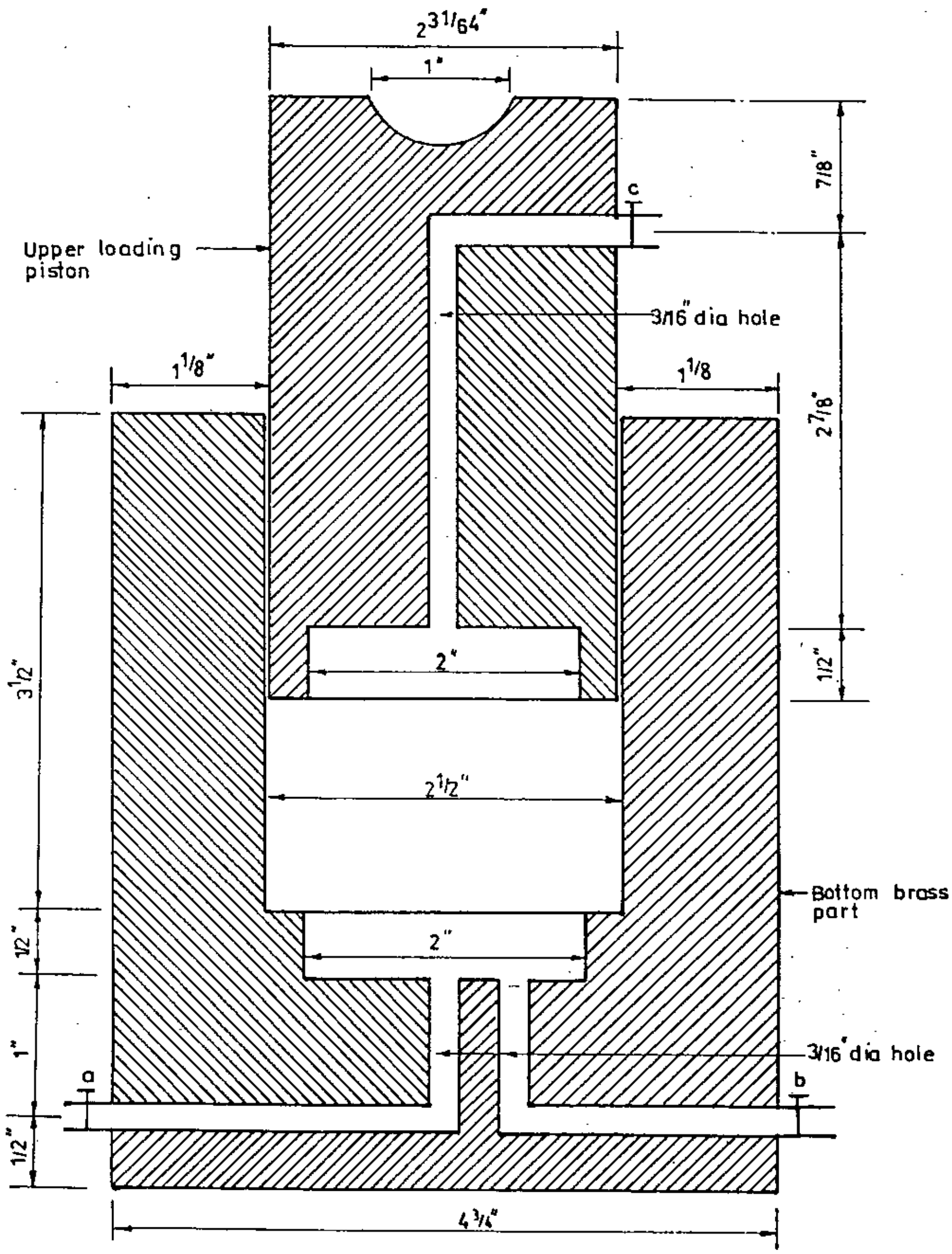


Fig. 4.1 Consolidation permeability test unit.

The loading piston is of $4\frac{1}{4}$" (107.95 mm) height and $2\frac{31}{64}$" (63.1 mm) diameter. The diameter of the loading piston was made 1/64" less than that of central hole for sample placement to eliminate side friction between the loading piston and the side walls of the central hole of bottom part.

The loading piston consists of the following features:

i) A groove of $\frac{1}{2}$" height and 2" diameter for setting the upper porous stone for top drainage.

ii) A hole of 3/16" diameter drilled centrally from the base and extended upto the side wall of the piston at a height of 7/8" down the top of the piston. This hole was provided for top drainage and to collect permeant discharge. This hole has a valve attachment c at the side wall end.

iii) A semi-elliptical groove of 3/8" diameter at center was made centrally top of the piston for setting a steel ball to transfer load from the loading plate to the loading piston.

4.6 Flow volume Measuring Unit

Bishop and Henkel (1964) developed an apparatus to measure flow volume and to apply back pressure in triaxial tests. An apparatus was designed and fabricated to measure the flow rate during constant head permeability test. This apparatus is similar to that developed by Bishop and Henkel with some modifications to accommodate the present test conditions. The whole unit rests on a rigid base plate with three permanent legs. A schematic diagram of this apparatus is shown in Fig. 4.2.

The salient features of the unit are as follows:

i) An inner 100 ml x 0.2 ml precision bore glass burette B and an outer clear acrylic tube T.

ii) A lower circular body made of brass. This body, which is $1\frac{3}{8}$" (34.92 mm) high and $3\frac{1}{2}$" (88.9 mm) diameter, has layers of grooves of different diameters for mounting inner burette and outer acrylic tube and for seating O-ring. Two holes of 5/32" diameter, one at the base of inner burette and the other at the base of outer acrylic tube were drilled and extended upto the side walls. Two valves, d and e were fitted with these holes.

iii) An upper circular body also made of brass of 1" height and $3\frac{1}{2}$" diameter. This body has a groove of $1\frac{25}{64}$" diameter for holding the acrylic tube. A hole of 7/16" diameter was drilled and extented upto the top of the upper body. At this end a bleed valve f was fitted. Arrangement was also made to keep the inner burette in upright position.

iv) Three brass rods of 5/16" diameter were fixed on the periphery of the lower body and extended vertically upto

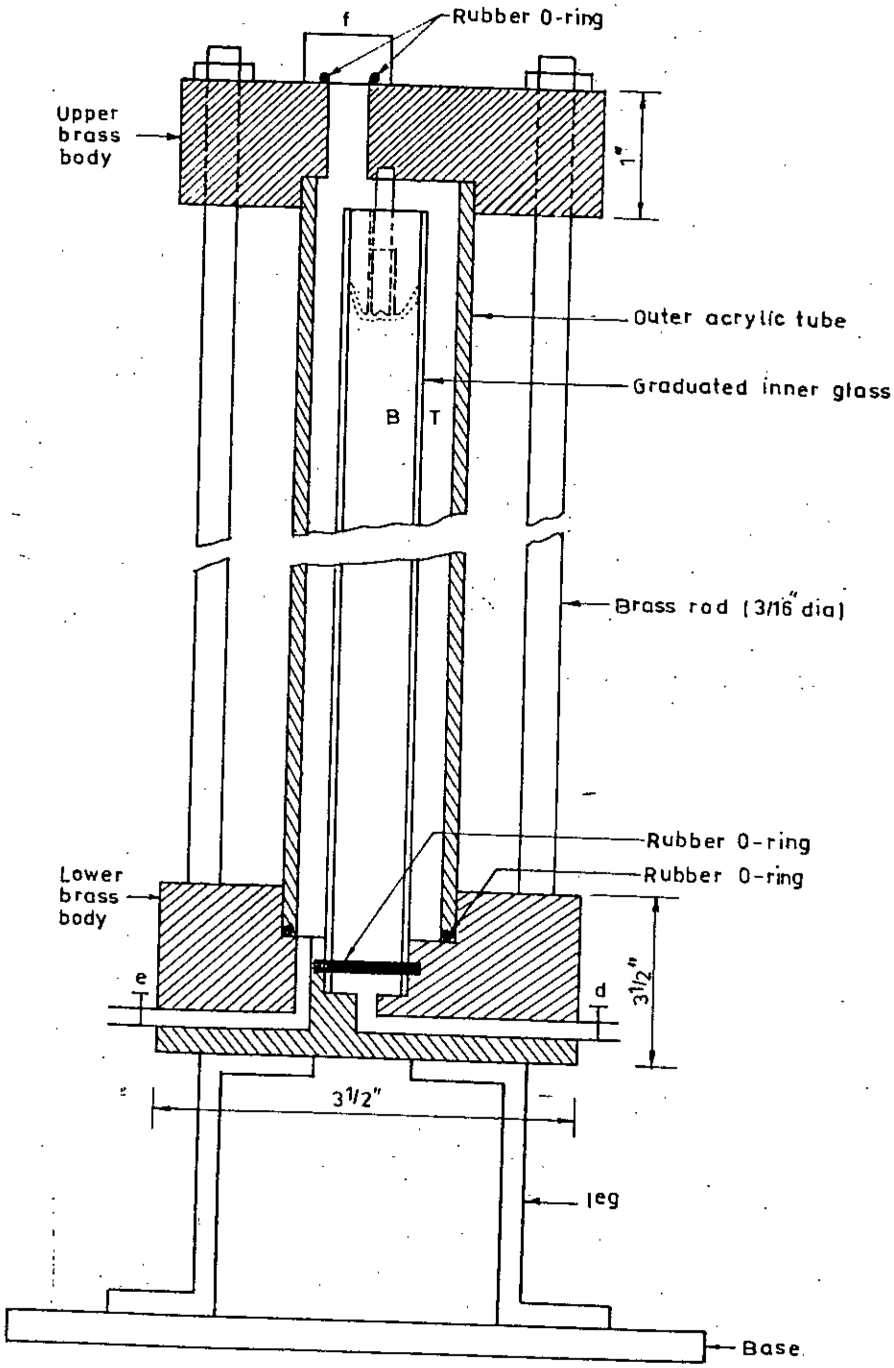


Fig. 4.2 Details of Flow volume measuring unit.

the top of upper body and bolted there to keep the whole unit firmly in vertical position.

4.7 Preparation of Reconstituted Soil

The reconstituted soils were prepared by mixing two different types of soils. One of the soils had about 25 percent clay while the other had about 85 percent silt. Before mixing of the soils, the coarse particles were removed by sieving with No. 30 sieve and the two soils were mixed in dry powdered form at different proportions. Two silty clay soils of medium plasticity resulted in the above process of preparation.

4.8 Preparation of Normally Consolidated Sample

Arrangements for the preparation of normally consolidated sample is shown in Fig. 4.3. About 175 gms of air dried sample was taken. This sample was thoroughly mixed with a water content equal to twice the liquid limit of the soil to form a slurry to ensure full saturation. This slurry was stored in a dessicator for 24 hours to prevent any moisture loss.

The consolidation permeability test unit was placed on a lever-arm consolidometer. The inner wall of the consolidation permeability test unit was lubricated to prevent side friction between the sample and side walls. A porous stone with two filter papers on it was placed at the bottom of the central hole of the test unit. Another porous stone was placed at the bottom. The side wall of the loading piston was lubricated to prevent friction between the side wall of the piston and side wall of the central hole of consolidation permeability test unit.

The slurry was poured directly into the test unit. Care was taken to avoid segregation of particles. The loading piston was gently placed on the slurry and kept for about 30-60 minutes (depending on the type of soil). The valves a, b and c were kept opened during this stage to allow drainage through top and bottom of the slurry. A total vertical consolidation pressure of $\frac{1}{4}$ tsf was applied on the slurry in two to four increments (depending on the type of soil). The pressure was exerted by the load on the lever arm which is transmitted to the loading piston through a loading plate and a steel ball. The loading was continued for 1 to 3 days (depending on the type of soil) till the completion of consolidation which was indicated by constant reading of strain gauge G. Normally consolidated sample was thus prepared.

4.9 One Dimensional Consolidation Test

Fig. 4.3 shows the consolidation testing arrangements schematically. Series of consolidation tests were run on normally consolidated sample with a stress increment ratio of 1 (i.e a load ratio of 2). The vertical stresses applied

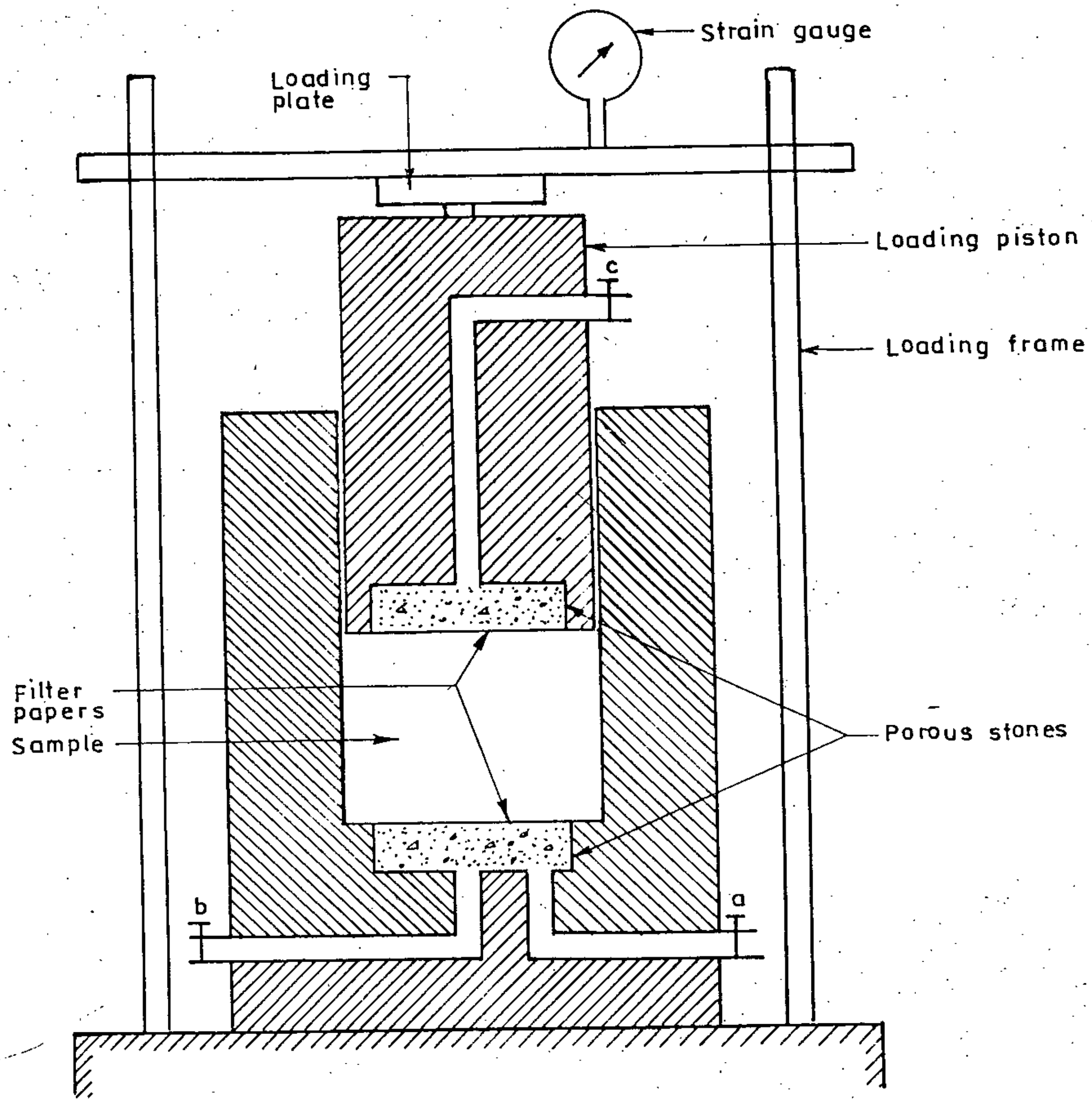


Fig. 4.3 Arrangements for the preparation of normally consolidated sample and for conducting one dimensional consolidation test.

were $\frac{1}{2}$ tsf, 1 tsf, 2 tsf, 4 tsf, 8 tsf, and 16 tsf. Duration of each load step was approximately 24 hours. For each load increment settlement was recorded by strain gauge, G with elapsed time at increasing intervals of $\frac{1}{4}$, $\frac{1}{2}$, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes. During all these tests valves a, b and c were kept opened to allow drainage through top and bottom of the sample. This test was performed in accordance with procedure described by ASTM standard D 2435-70.

4.10 Constant Head Permeability Test

Consolidation permeability test unit, with constant pressure apparatus and the flow volume measuring unit, was used as permeameter for constant head permeability test. Fig. 4.4 shows the arrangements schematically to perform this test.

Permeability tests were run at the end of stress increment when the primary consolidation was complete. For each sample five sets of test were run at the ends of 1, 2, 4, 8 and 16 tsf vertical effective consolidation pressure. This test was done according to the following steps:

1) The inner burette B and the outer acrylic tube T were completely filled and overflowed through bleed valve f with cooled boiled water from the reservoir by opening valves g and e. Valve d was kept closed during this operation After filling with water, valves g and e were closed.

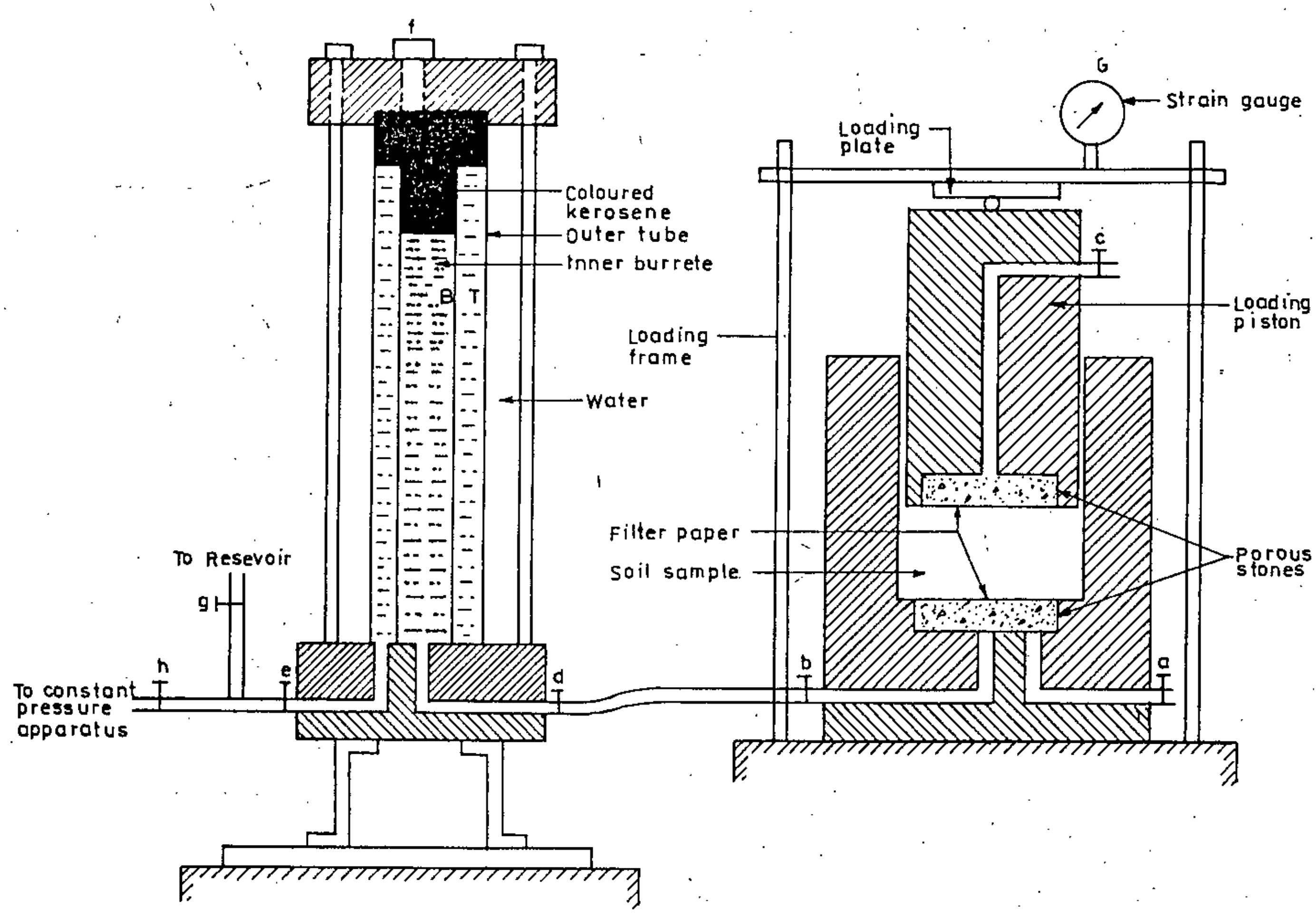


Fig. 4.4 Arrangements for conducting constant head permeability test.

2) The line between d and b was connected by flexible tube and was flushed by opening d and b. Valve a was kept closed. During this operation water level inside the burette B was lowered to some fixed mark. Valves d and b were then closed.

3) The water level inside the outer acrylic tube T was lowered to some extent by opening valve e. Valve e was then closed.

4) Coloured kerosene was added through the bleed valve f, when the vacant space within the burette B and acrylic tube T was completely filled through bleed valve f. In this way kerosene-water interface was made. Valve f was then closed.

5) The line between h and e was flushed by opening valve h and operating the constant pressure apparatus. Valve e was kept closed. Constant pressure was maintained for at least half an hour. This was indicated by the pressure gauge of constant pressure apparatus.

6) Valves e and d were opened slowly. No fluctuation of pressure indicated by the pressure gauge was ensured.

7) Position of kerosene-water interface within the glass burette B was noted. Valve b was opened slowly. Under constant hydrostatic pressure water began to flow through the soil from bottom to top. The flow through the soil was recorded with elapsed time at increasing intervals such as, 1,2,4,9, 16,25,.... 121, 144, 169 minutes and so on until the steady state of flow was reached.

The same procedure as mentioned above was followed for each stress increment.

The steady rate of flow (Q_α) was determined from the slope of the straight line portion of flow discharge (Q) versus time (t) plots. These plots are shown in Figs. 5.18 to 5.21. Using Darcy's law, the coefficient of permeability was then determined corresponding to the steady rate of flow.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 Introduction

In this chapter one dimensional consolidation and constant head permeability test results are analysed to establish a relation between coefficient of permeability and void ratio. Determination of permeability and compressibility parameters and correlation of these parameters with index properties of soil are also attempted. A relationship between constrained modulus (D) and effective vertical pressure for normally consolidated silty clays and silts is reported. On the basis of available results, some equations for evaluation of parameters for settlement estimation have been proposed.

5.2 One Dimensional Consolidation Test Results

In consolidation test normally consolidated soil samples were prepared and tested under stress increments of 1/2, 1, 2, 4, 8 and 16 tsf. The details of soil sample preparation and testing procedure are described in Arts. 4.8 and 4.9. Typical time deformation curves for soil-I under 1 tsf consolidation pressure are shown in Figs. 5.1 and 5.2. Fig. 5.1 shows dial reading versus logarithm of time plot while Fig. 5.2 shows the same plot in dial reading versus square root of time. Table B-1 to B-4 of Appendix-B furnish

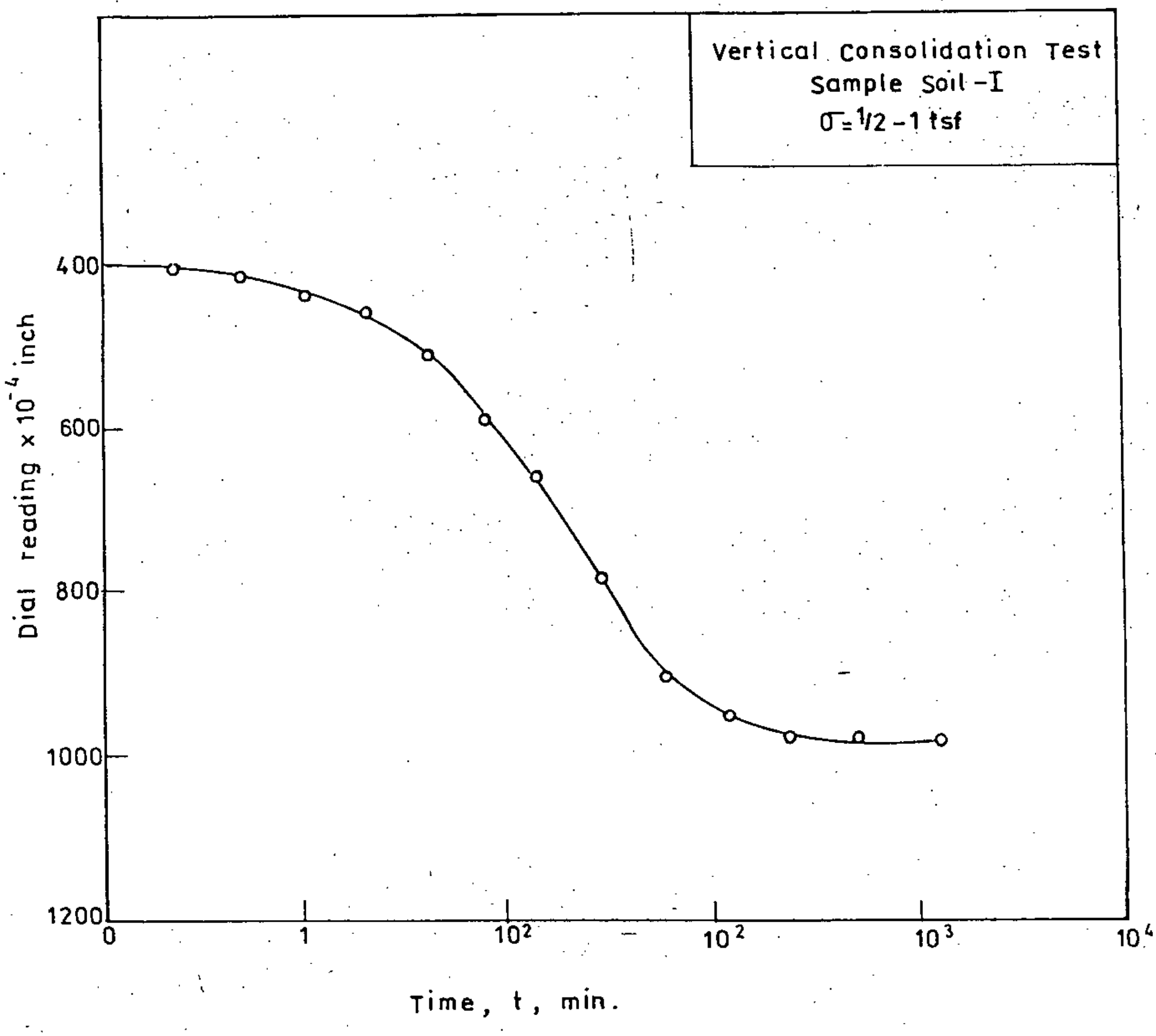


Fig. 5.1 Typical dial reading versus log time plot.

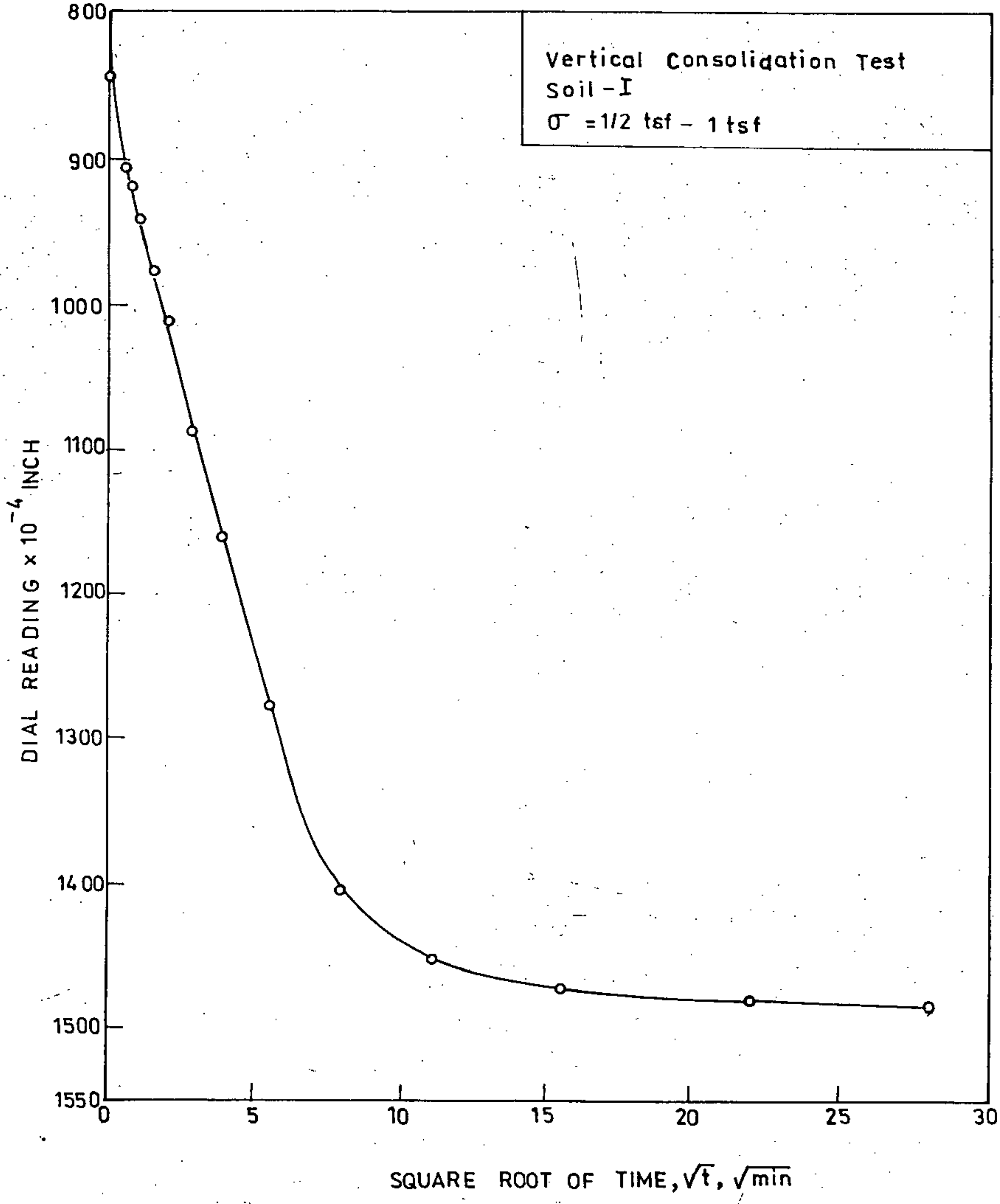


Fig. 5.2 Typical dial reading versus square root of time plot.

time deformation readings for the four soils used in this research for all the stress increments. From the two time deformation curves that is log fitting and square root fitting curves, times corresponding to 50% consolidation (t_{50}) and 90% consolidation (t_{90}) respectively were calculated for each soil. t_{50} was determined by method proposed by Casangradde and Fadum (1940) while t_{90} was determined by method proposed by (Taylor (1942). Coefficient of consolidation C_v and coefficient of permeability k were calculated for each stress increment using both t_{50} and t_{90}. The C_v and k values were determined by using the following equations from Terzaghi's one dimensional consolidation theory (Terzaghi,1943).

$$C_v = \frac{0.197\ H^2}{t_{50}} \tag{5.1}$$

$$C_v = \frac{0.848\ H^2}{t_{90}} \tag{5.2}$$

$$k = \frac{0.197\ H^2\ \gamma_w\ C_c}{2.303(1+e)\bar{\sigma}\ t_{50}} \tag{5.3}$$

$$k = \frac{0.848\ H^2\ \gamma_w\ C_c}{2.303(1+e)\bar{\sigma}\ t_{90}} \tag{5.4}$$

Here, H = average length of drainage path, that is half the height of sample at the end of 100% consolidation for a given stress increment.

C_c = Compression index

$\bar{\sigma}$ = aveage effective vertical pressure for a given stress increment

e = void ratio.

Summary of the consolidation test results for the soils are presented in Table 5.1. It shows that C_v values for normally consolidated silty clays used in this study i.e soils I, III and IV ranged between 5.04×10^{-4} and 97.27×10^{-4} cm^2/sec whilst that for normally consolidated silt i.e. Soil-II varies from 73.66×10^{-4} to 249.6×10^{-4} cm^2/sec. Serajuddin (1967) performed consolidation tests on a large number of undisturbed clay and silt samples collected from different locations in Bangladesh. He reports that the C_v-values for undisturbed clay and silt samples collected from North East Zone (Dhaka, Mymensingh and Sylhet districts) varied between 0.92×10^{-4} to 900×10^{-4} cm^2/sec. It is likely that most of the samples tested by him were normally loaded with some samples slightly preconsolidated and also that the samples were natural specimens.

5.3 Permeability Test Results

During permeability test, samples were tested at the range of hydraulic gradients from 293 to 857 which on the basis of Eq. 2.2 showed that values of Reynolds number were all less than unity. Hence the samples were tested under laminar flow condition and consequently Darcy's equation was used to compute coefficient of permeability for the soils.

Table 5.1 Summary of Consolidation Test Results

Soil	$\bar{\sigma}$ (tsf)	Sample ht. at the end of 100% consolidation 2H, cm	Void ratio	Avg. $\bar{\sigma}$ (tsf)	Avg. void ratio	Fitting time (sec)		C_v ($10^{-4}cm^2/sec$)		k ($10^{-8}cm/sec$)	
						t_{50}	t_{90}	$\frac{.197H^2}{t_{50}}$	$\frac{.848H^2}{t_{90}}$	$\frac{.197H^2\gamma_w C_c}{2.303(1+e)\sigma t_{50}}$	$\frac{.848H^2\gamma_w C_c}{2.303(1+e)\sigma t_{90}}$
1	2	3	4	5	6	7	8	9	10	11	12
	0.25	3.805	1.01	-	-	-	-	-	-	-	-
	0.5	3.646	.926	.375	.968	1356	4256	5.04	6.93	8.289	11.39
	1.0	3.483	.84	.75	.883	951	2820	6.583	9.55	5.649	8.196
I	2.0	3.32	.754	1.5	.797	702	2107	8.126	11.64	3.645	5.228
	4.0	3.16	.67	3.0	.712	530	1664	9.76	13.37	2.30	3.15
	8.0	2.998	.584	6.0	.627	438	1405	10.669	14.306	1.32	1.77
	16.0	2.858	.509	12.0	.546	406	1288	10.4	14.106	.678	0.92
	0.25	4.353	1.134	-	-	-	-	-	-	-	-
	0.5	4.12	1.021	.375	1.077	120	366	73.66	104.0	118.18	166.9
	1.0	3.937	.93	.75	.9755	77	241	103.0	142.7	85.5	120.9
II	2.0	3.775	.841	1.5	.8855	58	168	125.6	186.6	55	82.78
	4.0	3.576	.753	3.0	1.217	45	127	148.0	224.0	34.4	52.09
	8.0	3.396	.665	6.0	.709	36	106	172.0	243.0	20.99	29.67
	16.0	3.223	.58	12.0	.6225	30	93	179.8	249.6	11.5	16.045

Table 5.1 Contd...

1	2	3	4	5	6	7	8	9	10	11	12
	0.25	3.8	.953	-	-	-	-	-	-	-	-
	0.5	3.647	.875	.375	.914	645	1993	10.58	14.75	16.0	22.3
	1.0	3.497	.798	.75	.836	446	1410	14.1	19.184	11.09	15.1
III	2.0	3.351	.723	1.5	.76	329	1033	17.57	24.05	7.20	9.87
	4.0	3.203	.647	3.0	.685	252	826	21.0	27.56	4.497	5.9
	8.0	3.053	.57	6.0	.608	225	730	21.42	28.4	3.18	3.18
	16.0	2.906	.494	12.0	.532	228	781	19.2	24.1	1.128	1.416
	0.25	3.66	.89	-	-	-	-	-	-	-	-
	0.5	3.523	.818	.375	.854	237	582	26.74	46.95	37.27	65.42
	1.0	3.395	.752	.75	.785	162	421	36.4	60.22	26.366	43.62
IV	2.0	3.25	.677	1.5	.714	117	320	46.43	73.07	17.45	27.46
	4.0	3.13	.615	3.0	.646	88	244	56.81	88.37	11.15	17.328
	8.0	2.996	.546	6.0	.58	68	204	68.08	97.27	6.939	9.914
	16.0	2.87	.481	12.0	.513	66	192	64.2	94.98	3.417	5.056

Permeability tests were performed for each of the four soils at the end of primary consolidation at stress increments of 1,2,4,8 and 16 tsf. The primary consolidation was assumed to be complete at the end of 24 hours of loading when increase in soil deformation was nearly zero. Details of test procedure for permeability test is outlined in Art. 4.10. From the analysis of permeability test results coefficient of permeability was determined from the steady state flow rate at different applied hydraulic gradients. Summary of the permeability test results for all the soils is shown in Table 5.2. This table shows that, for normally leaded silty clay, coefficients of permeability determined from constant head test are in the range of 4.01×10^{-7} to 7.36×10^{-9} cm/sec with dry density and void ratio of the samples in the range of 90.68 to 113.458 pcf and 0.479 to 0.837 respectively. Table 5.2 also shows that, coefficients of permeability for silty samples varies from 1.23×10^{-6} to 1.35×10^{-7} cm/sec. Dry density and void ratio of the silty samples are in the range of 87.77 to 1,07.085 pcf and 0.579 to 0.926 respectively. Serajuddin and Ahmed (1967) performed laboratory permeability test by falling head method on six undisturbed clay samples of medium plasticity. The samples were collected from an area of Dhaka by excavation at shallow depth. Coefficient of permeability of the samples were in the range of 4.0×10^{-6} to 3.6×10^{-8} cm/sec with natural dry density of the samples in the range of about 77 to 100 pcf.

Table 5.2 Summary of Permeability Test Results

Soil	Hydraulic gradient, i	Void ratio	Water content (%)	Wet density (pcf)	Steady state flow rate (cm^3/min)	k (10^{-8} cm/sec)
I	293.05	.837	31.23	119.0	.0413	7.42
	368.73	.752	28.06	121.76	.033	4.7
	452.42	.666	24.85	124.84	.0243	2.83
	621.77	.582	21.71	128.21	.0179	1.54
	857.1	.507	18.92	131.41	.012	0.736
II	259.348	.926	34.04	117.64	.607	123.2
	271.58	.839	30.87	120.29	.4026	78.03
	342.218	.752	27.68	123.23	.337	51.8
	366.43	.663	24.64	126.69	.179	26.1
	442.95	.597	21.28	129.87	.114	13.54
III	291.71	.796	29.59	120.63	.081	14.1
	265.428	.72	26.79	123.2	.0636	9.17
	446.05	.645	23.97	126.04	.0451	5.32
	601.518	.568	21.11	126.19	.0337	2.95
	842.75	.492	18.3	132.56	.0233	1.46
IV	300.412	.75	27.8	122.51	.2289	40.1
	313.95	.675	25.0	125.23	.149	24.98
	391.137	.613	22.72	127.64	.120	16.13
	476.757	.544	20.19	130.56	.0832	9.189
	639.98	.479	17.74	133.58	.0615	5.061

In this research, permeability test was performed on saturated normally consolidated samples by the application of back pressure. Undisturbed samples tested by Serajuddin and Ahmed were taken from shallow depth. So the samples were likely to be preconsolidated. Since, they performed permeability test by falling head method the samples were likely to swell during test resulting in higher permeability. Also the samples tested were natural clay and therefore likely to have different stress history and fabric. A comparison of the results of the present research with that of Serajuddin and Ahmed reveals that the permeability values obtained by the latter to be higher for nearly similar type of soil. This deviation in results may be due to differences in test method, stress history and fabric of the soil.

5.4 Void Ratio-Effective Vertical Pressure Relationship

Void ratio e versus logarithm of effective vertical pressure (at the end of primary consolidation) plots for the four soils studied are shown in Fig. 5.3. From the figure it can be observed that for normally consolidated silty clays and silts e-log $\bar{\sigma}$ plots are straight lines. Similar observation was made by Raymond (1966) for three normally consolidated clays of medium to high plasticity and by Samarasinghe et al (1982) for a normally consolidated sandy clay of medium plasticity. Compressibility parameters, i.e. compression index

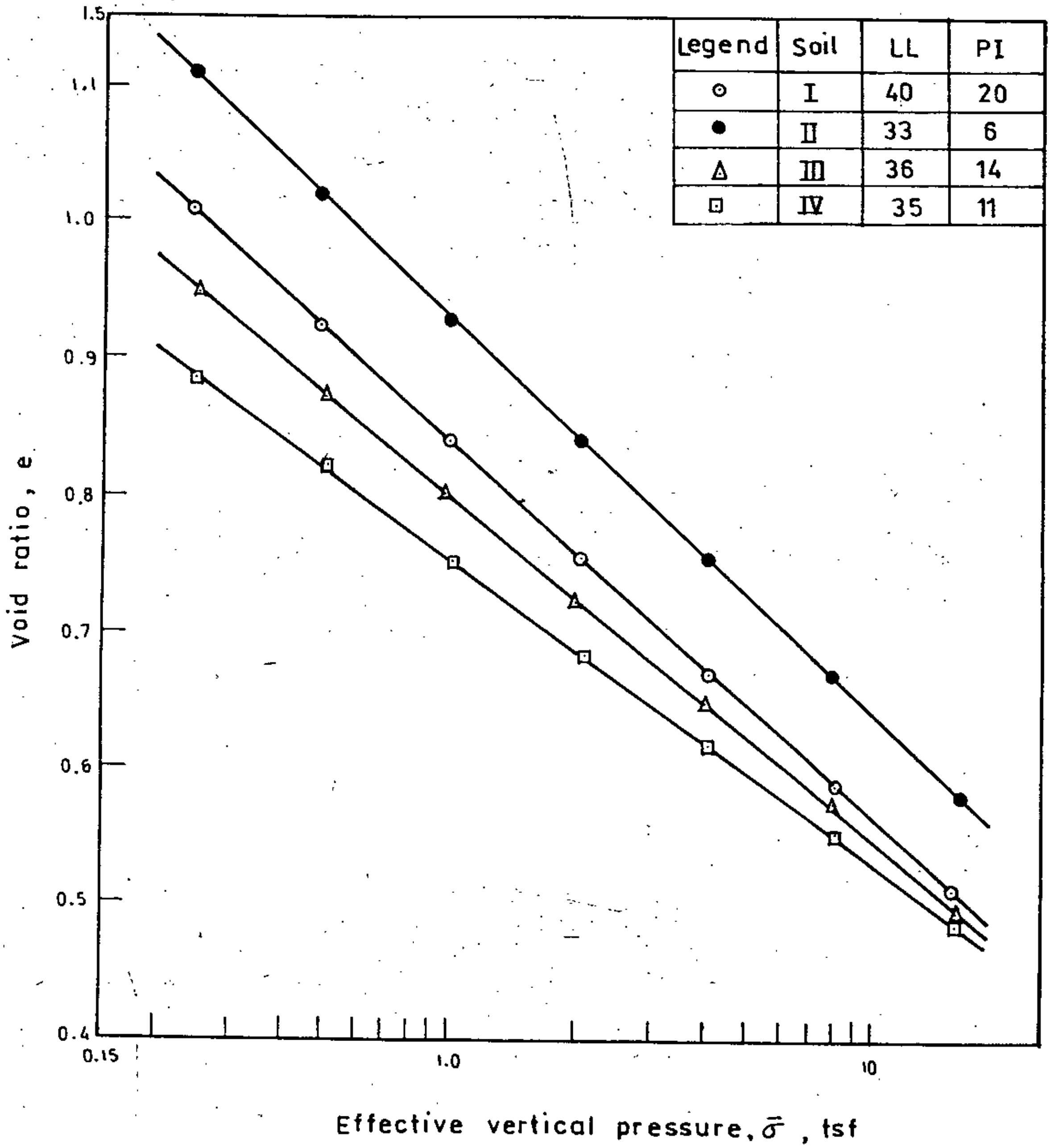


Fig. 5.3 Void ratio versus log effective vertical pressure curves.

C_c and e_1 were determined from e-log $\bar{\sigma}$ plots. C_c is the slope of e-log $\bar{\sigma}$ plot and e_1 is the void ratio corresponding to the effective vertical pressure of 1 tsf. The values of compressibility parameters of the soils studied are presented in Table 5.3 It shows that C_c for the soils range between 0.223 and 0.28 while e_1 varies from 0.752 to 0.93 for the four different soils.

Except for the Soil-II, which is a slightly plastic silt, compressibility parameter C_c i.e compression index, increases with increasing LL and PI. Skempton (1953), Raymond (1966) and Serajuddin and Ahmed (1967) all showed that for plastic clays C_c increases with increasing LL and PI. Similarly, for all the soil except Soil-II, the compressibility parameter e_1 increases with the increase in LL and PI. Raymond (1966) demonstrated that value of e_1 increases with increasing LL and PI for plastic clays. However, for Soil-II, both the values of C_c and e_1 are higher than the corresponding values for the other soils even though both LL and PI for Soil-II is less. It, therefore, appears that for non-plastic or slightly plastic soils different relationship between C_c and e_1 with LL and PI exists than for soils with medium to high plasticity. However, this aspect requires further investigation.

5.5 Void ratio-Permeability Relationship

For each soil, coefficient of permeability k was determined from constant head permeability test using Darcy's equation and also from consolidation test data by using Eq. 5.3 based on t_{50} and Eq. 5.4 using t_{90} value. The e-log k plots for the soils are shown in Figs. 5.4 and 5.5. In Fig. 5.4, void ratio has been plotted against coefficient of permeability computed from constant head permeability test while Fig. 5.5 shows plotting of e versus k determined from consolidation tests. It is observed from Figs. 5.4 and 5.5 that coefficient of permeability of each soil decreases with decreases with decreasing void ratio and also that e-log k relationship is non-linear. Similar relationship had previously been reported by Raymond (1966) for normally consolidated clays and by Ahmed (1977) for undisturbed soft Bangkok clay.

Variation of coefficient of permeability k (in log scale) with plasticity index PI is shown in Fig. 5.6. In this k corresponds to a particular value of void ratio marked within the figure. The value of k used in Fig. 5.6 has been determined from consolidation test results using equation 5.3. Since k-values obtained by other methods, such as constant head permeability test or consolidation test using eqn. 5.4 were not significantly different, shows similar plot and therefore is not presented. From Fig. 5.6 it is evident that log k-PI plots are approximately S-shaped curves. The plottings also show that for any particular void ratio, coefficient of permeability for normally consolidated silty clays and silts decreases with the increase in plasticity index of the soils. The log [k(1+e)] versus loge plots of the soils investigated

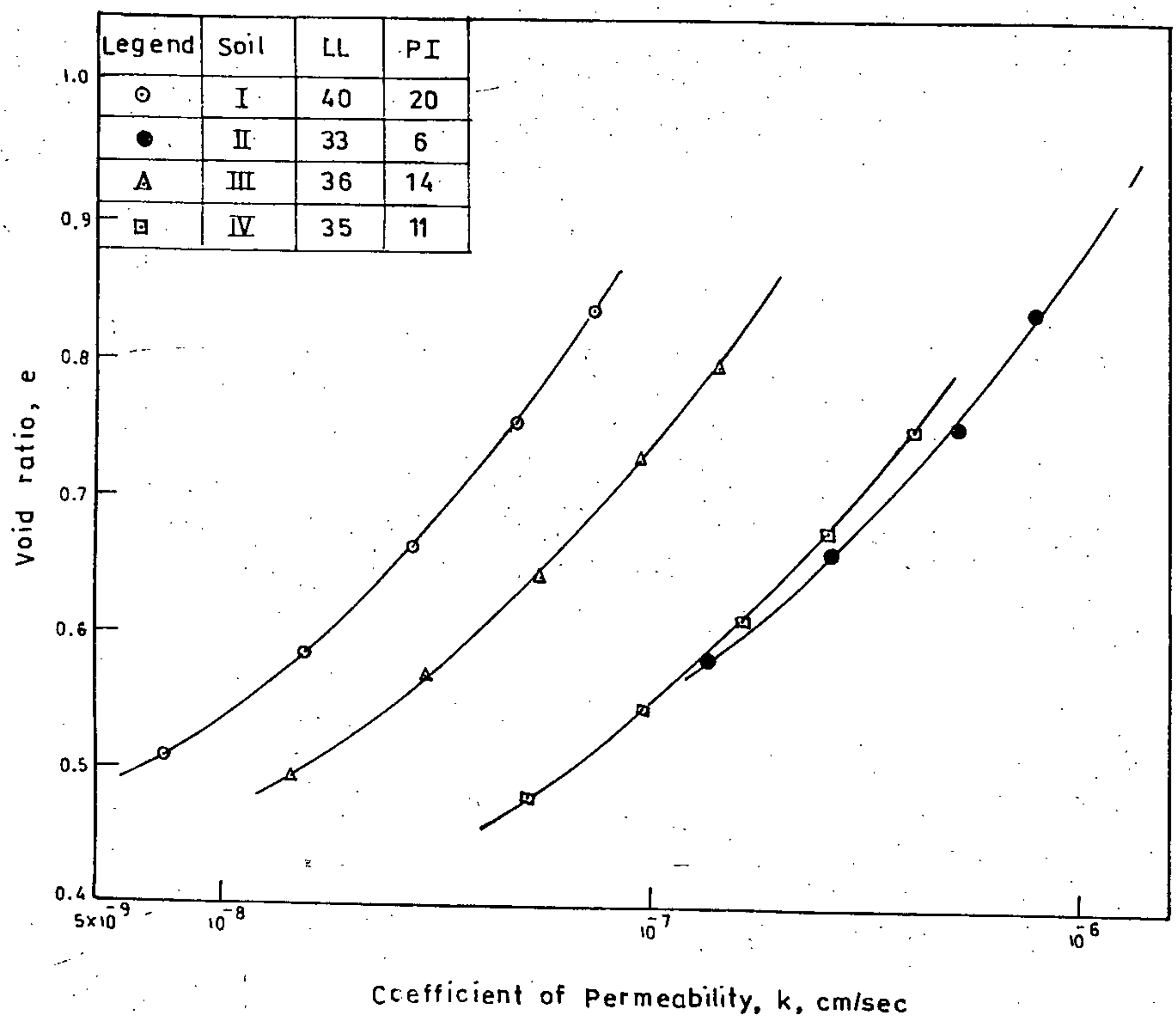


Fig. 5.4 Void ratio versus log coefficient of permeability (determined from constant head permeability test) plots.

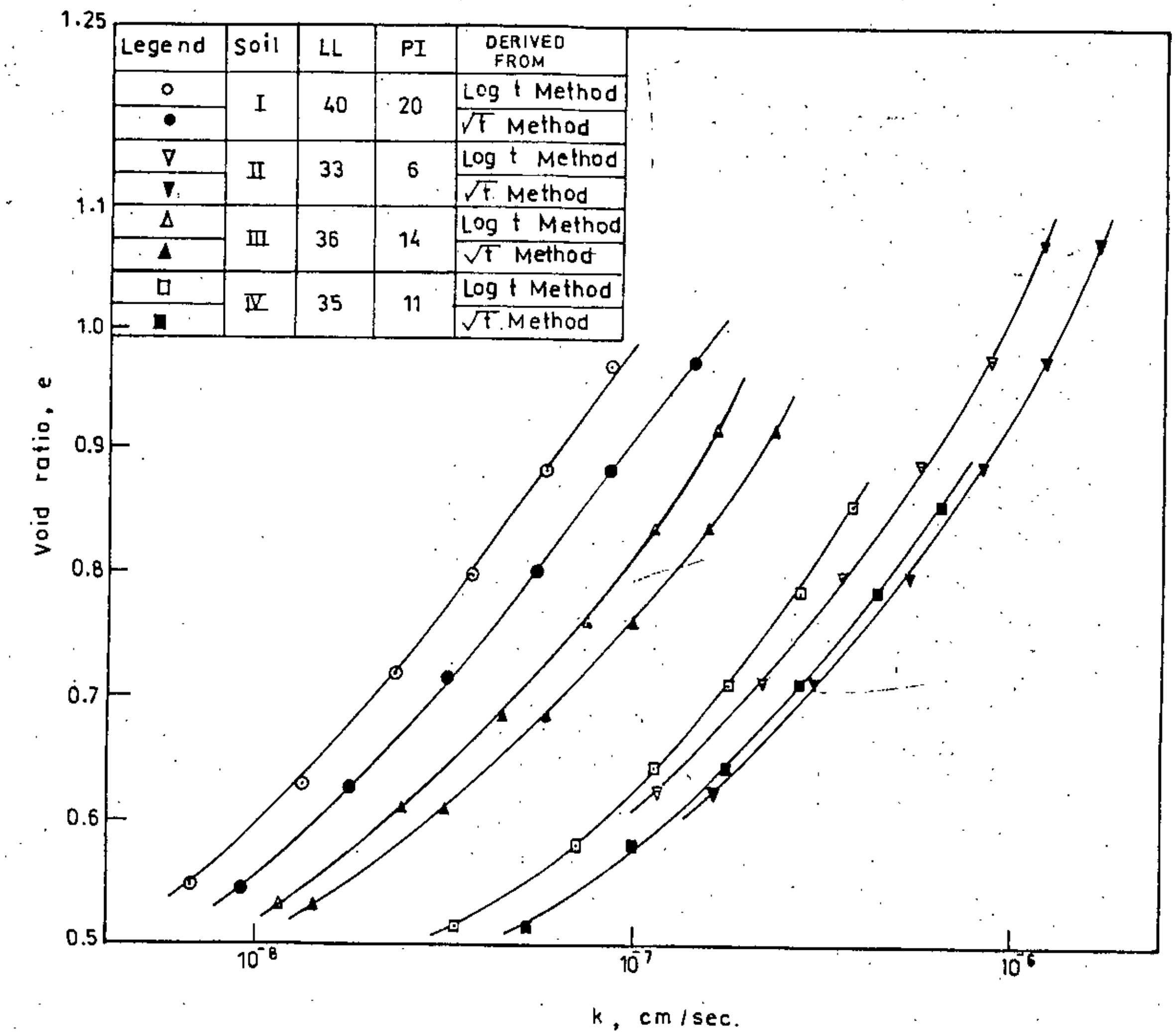


Fig. 5.5 Void ratio versus log coefficient of permeability (computed from consolidation test) plots.

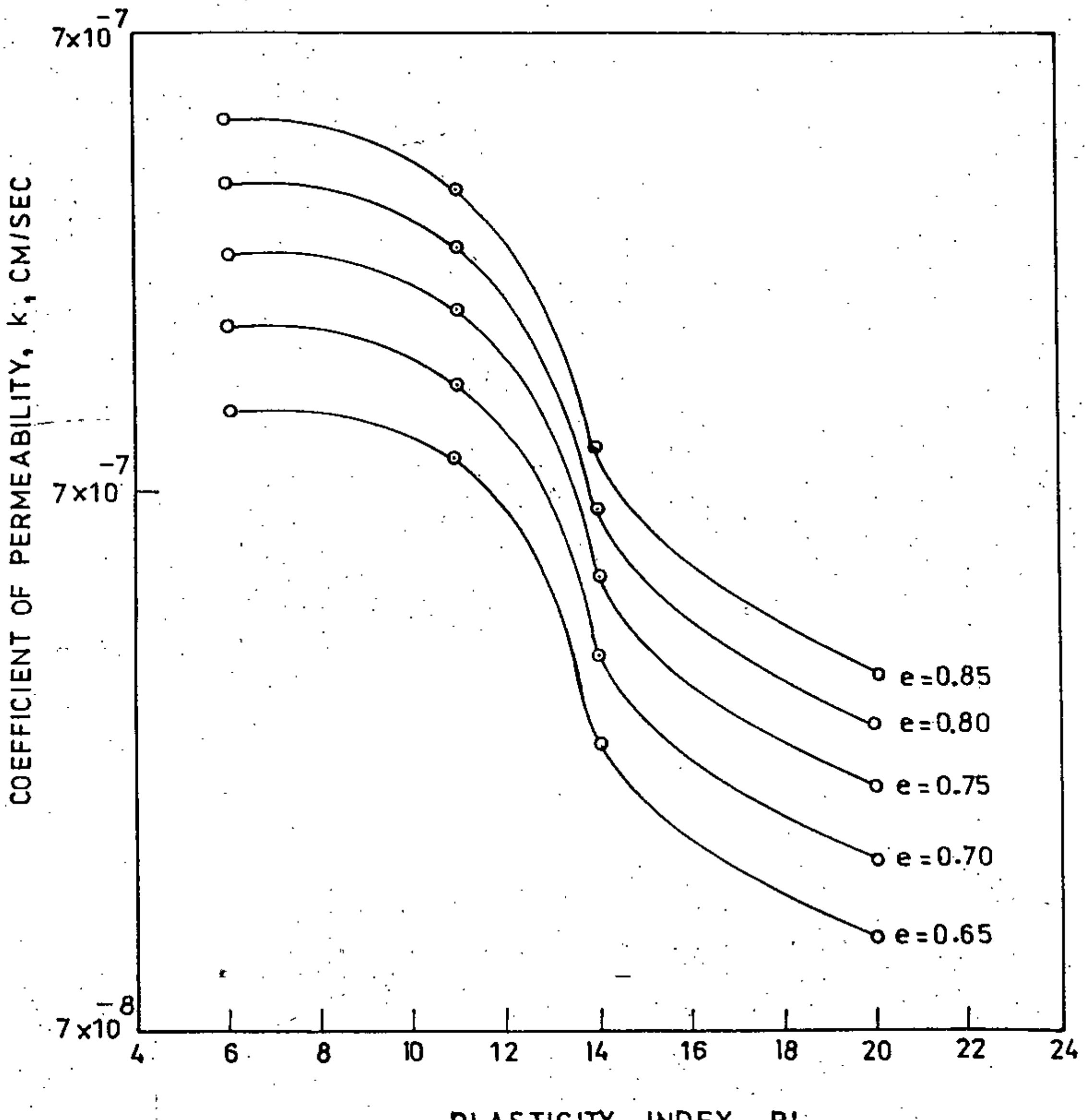


Fig. 5.6 Variation of coefficient of permeability (computed from consolidation test) with plasticity index.

are shown in Figs. 5.7 and 5.8. In Fig. 5.7 k values are obtained from permeability tests while the k-values in Fig.5.8 are obtained from consolidation test. From the plots of Figs.5.7 and 5.8 it is observed that log [k(1+e)] versus loge plots for normally consolidated clays and silts are straight lines. Identical relations were obtained by Raymond (1966) and Samaraasinghe et al (1982) for normally loaded clays.

From the plots of Figs. 5.7 and 5.8 permeability parameters n and C were computed.

Parameter n is the slope of log [k(1+e)] versus loge plot whereas logarithm of parameter C is the vertical intercept of the same plot. The magnitudes of n and C are tabulated in Table 5.4. This table shows that n and C obtained from permeability test ranged from 4.8 to 5.0 and 3.2×10^{-7} to 35.48×10^{-7} cm/sec. respectively. Values of n and C computed from consolidation test (log t method) varied from 4.72 to 5.18 and 1.94×10^{-7} to 18.55×10^{-7} cm/sec respectively. Values of n and C obtained from consolidation test ($\sqrt{t}$ method) varied from 5.0 to 5.3 and 2.818×10^{-7} to 28.2×10^{-7} cm/sec. Thus permeability parameter C computed from permeability test is higher than that determined from consolidation test. However, permeability parameter n did not vary significantly with type of test.

From Table 5.4 it can be inferred that C increases with decreasing values of LL and PI. Similar behavion was observed by Raymond (1966) for three normally consolidated clays.

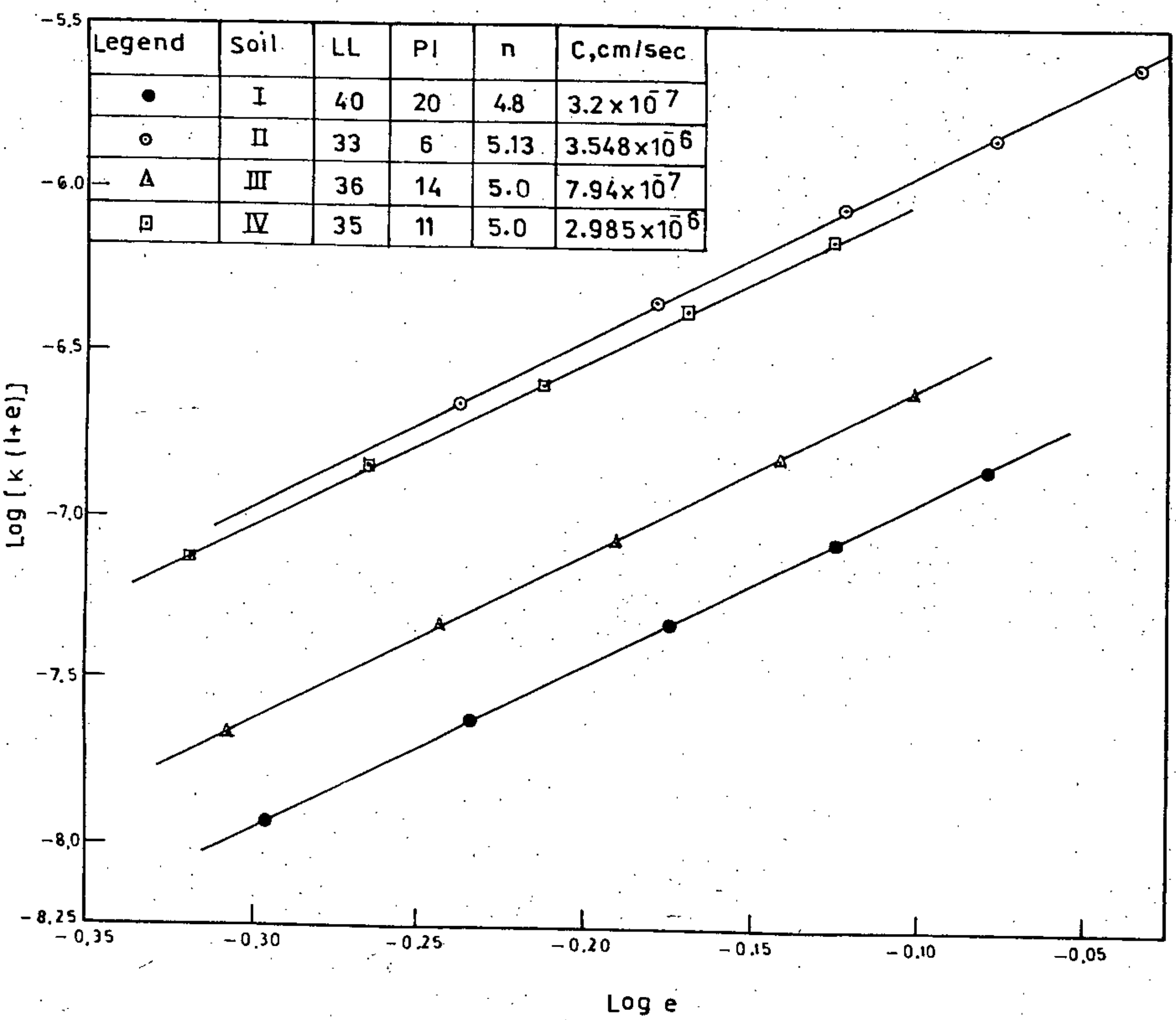


Fig. 5.7 Relationship-between log [k(1+e)] and loge
(k determined from constant head permeability test).

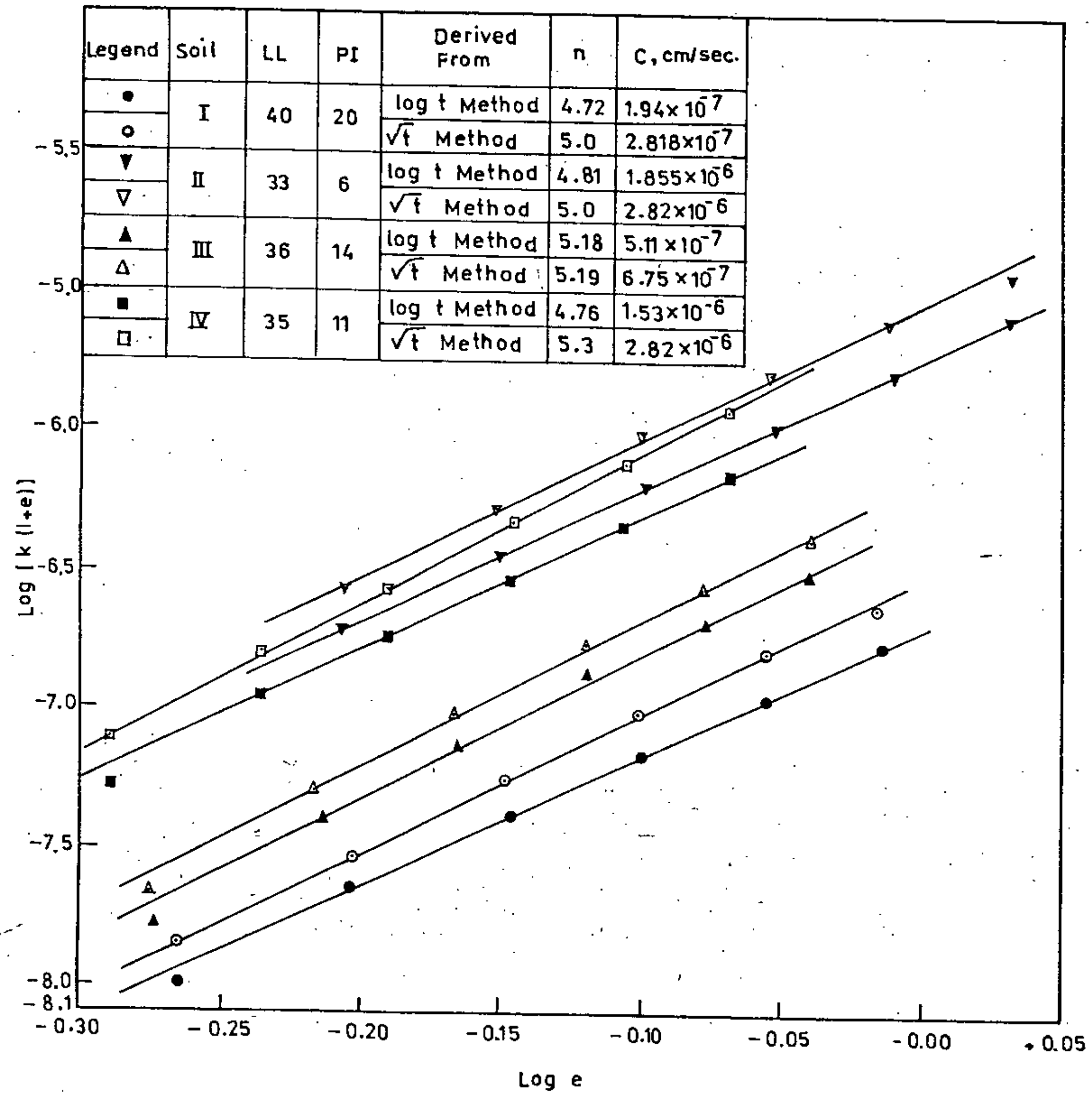


Fig. 5.8 Relationship between log [k(1+e)] and loge (k computed from consolidation test).

Table 5.3 Values of Compressibility Parameters of the Soils Investigated

Soil	LL	PI	Saturated water content under ‾ = 1 tsf (%)	Compressibility Parameters	
				C_c	e_1
I	40	20	31.23	0.28	0.84
II	33	6	34.94	0.29	0.93
III	36	14	29.59	0.25	0.798
IV	35	11	27.8	0.225	0.752

Table 5.4 Values of Permeability Parameters of the Soils Investigated

Soil	LL	PI	Permeability parameters					
			From constant head permeability test		From one dimensional consolidation test			
					log time method		Square root of time method	
			n	C (10^{-7} cm/sec)	n	C (10^{-7} cm/sec)	n	C (10^{-7} cm/sec)
I	40	20	4.8	3.2	4.72	1.94	5.0	2.818
II	33	6	5.13	35.48	4.815	18.55	5.0	28.2
III	36	14	5.0	7.94	5.18	5.11	5.19	6.75
IV	35	11	5.0	29.85	4.76	15.3	5.3	28.2

The nature of variation of C with plasticity index can be observed from Fig. 5.9 which shows the plottings of log C versus PI. Raymond reports that for a particular type of clay n is essentially same irrespective of method of determination i.e test type but there is a tendency of increase in n value for soils with increasing PI. Raymond tested three soils e.g. Don valley clay, New Liskeard clay and Bentonite having PI equal to 22, 40 and 72 respectively. All the specimens were normally consolidated. Values of n determined from permeability test for Don valley clay, New Liskeard clay and Bentonite are 4.5, 5.2 and 13.9 respectively. In this research, three normally consolidated clays with PI equal to 20, 14 and 11 and one slightly plastic silt with PI equal to 6 have been studied. Although no definite relation between n and PI could be observed, there is a trend of slight increase in n with decrease in PI when n values are computed from permeability test. This finding, therefore, somewhat deviates from the relation observed by Raymond. This aspect, therefore, requires further investigation.

The theoretical model (Samarasinghe et al, 1982) for the prediction of permeability of normally consolidated clays suggests that for a particular value of n, a plot of k versus void ratio function $e^n/(1+e)$ would result in a straight line passing through the origin. In order to observe this relation coefficient of permeability computed from constant head permeability test has been plotted against void ratio function

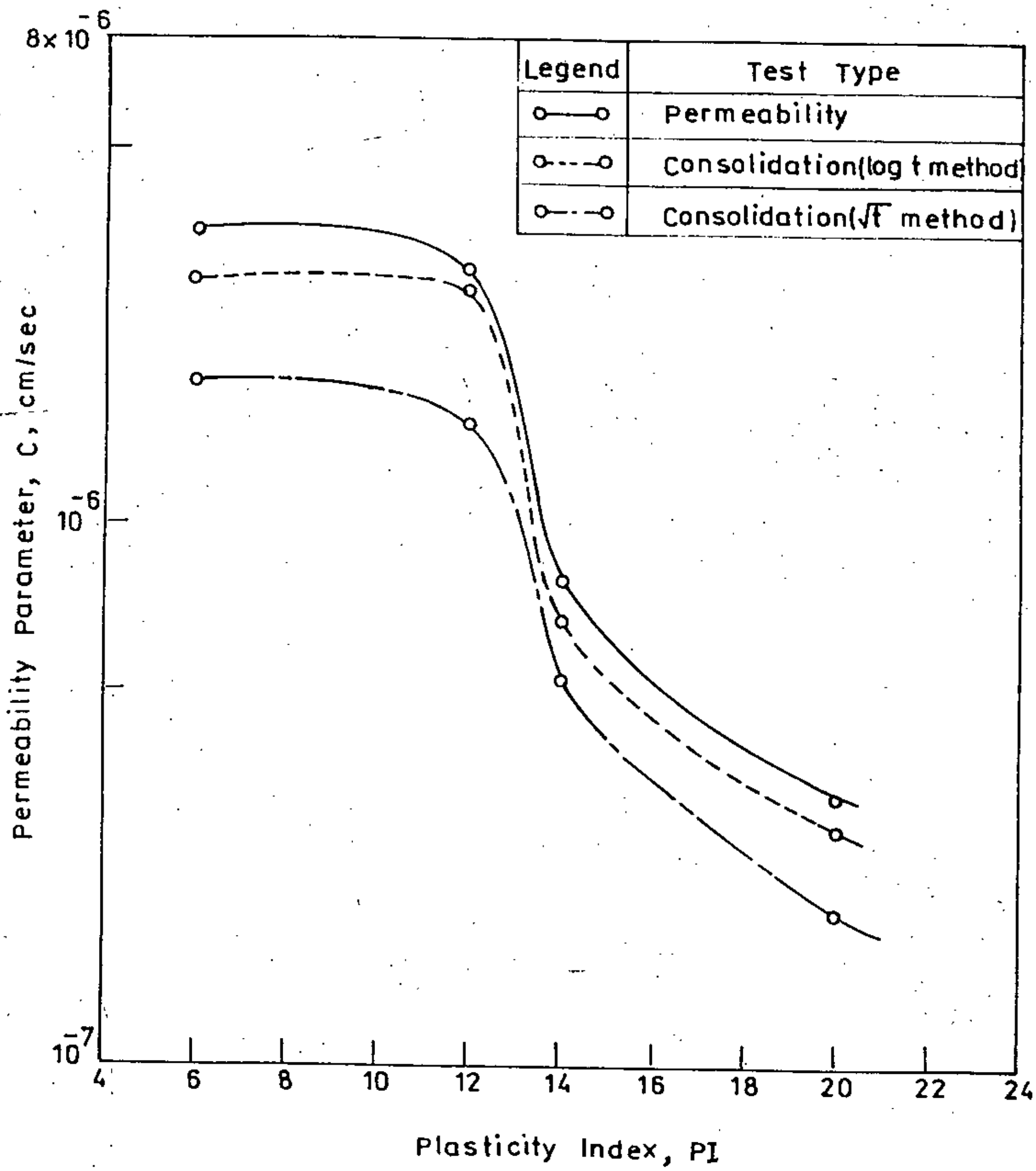


Fig. 5.9 Variation of permeability parameter C with plasticity index for various methods of testing.

$e^n/(1+e)$. Fig. 5.10 shows plottings of k versus $e^n/(1+e)$ for the soils studied. The values of n reported in Fig. 5.10 have been obtained from log $[k(1+e)]$ versus loge plots shown in Fig. 5.6 plotted from permeability test data. Fig. 5.8 and 5.9 show that for normally consolidated silty clays and silts as well, k versus $e^n/(1+e)$ plot is a straight line passing through the origin. Hence, for normally consolidated silty clays and silts coefficient of permeability can be directly related to void ratio function $e^n/(1+e)$ by Eq. 2.8 proposed by Samarasinghe et al (1982) which has been experimentally verified.

5.6 Coefficient of Consolidation - Effective Vertical Pressure Relationship

For each soil, coefficient of consolidation, C_v has been determined from consolidation test using Eq. 5.1 based on t_{50} and Eq. 5.2 based on t_{90}. C_v has also been calculated from Eq. 2.20. In order to calculate C_v from Eq. 2.20, n, C, C_c and e_1 were required and the values of n and C were obtained from log $[k(1+e)]$ versus loge plot, k being determined from permeability test using Darcy's equation. The values of C_c and e_1 shown in Table 5.3 were computed from e-log $\bar{\sigma}$ plot for each soil. The values of C_v corresponding to different effective vertical pressure $\bar{\sigma}$ calculated by using Eq. 2.20 are presented in Table 5.5. Fig. 5.11 shows the plotting of

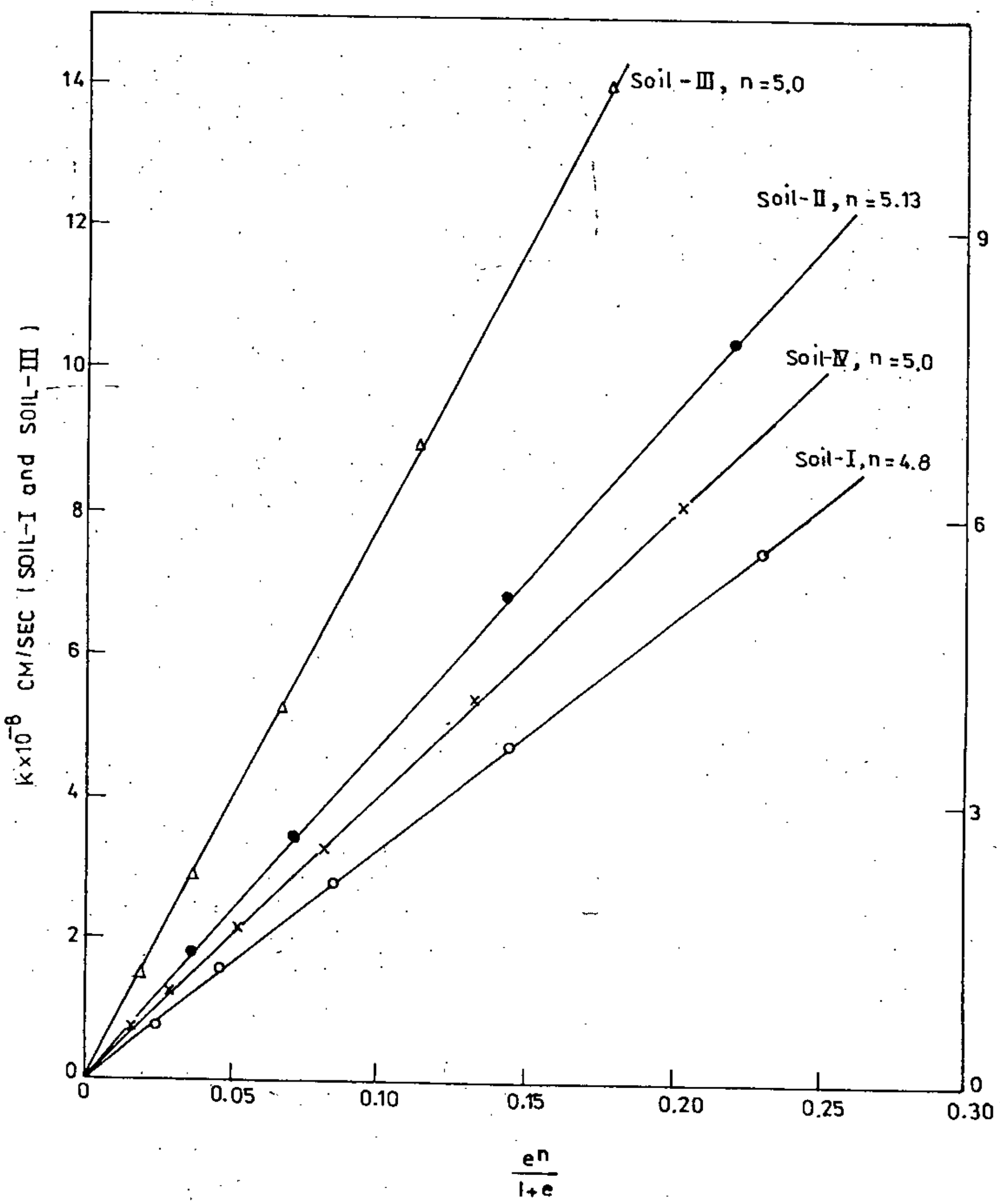


Fig. 5.10 k versus $e^n/(1+e)$ plots of the soils investigated (k determined from constant head permeability test).

Table 5.5 Values of Coefficient of Consolidation (Calculated from Eq. 2.20)

$\bar{\sigma}$ (tsf)	Coefficient of consolidation, $C_v \times 10^{-4}$ cm^2/sec			
	Soil-I	Soil-II	Soil-III	Soil-IV
0.25	6.723	115	13.77	41.3
0.5	8.844	150.8	18.2	55.7
1.0	11.178	190	23.2	72.68
2.0	13.457	229.6	28.29	91.08
4.0	15.25	262	32.65	108.75
8.0	16.02	279	35.2	122.36
16.0	15.23	272	34.79	127.7

C_v versus log $\bar{\sigma}$ for Soil-I. Similar plottings for Soil-II, Soil-III and Soil-IV are shown in Figs. 5.12, 5.13, and 5.14 respectively. It is evident from the above mentioned figures that C_v obtained from consolidation tests increases somewhat linearly with log $\bar{\sigma}$ upto a value of $\bar{\sigma}$ of 3 tsf. At higher $\bar{\sigma}$ -values the curves become non-linear. At $\bar{\sigma}$ greater than about 6 to 8 tsf there is a tendency of decrease in C_v value. Similar nature of C_v - log $\bar{\sigma}$ relationship was reported by Gorman et at (1978) and Samarasinghe et al (1982) for normally consolidated clays. The decrease in C_v at higher effective stresses is due to decrease of ratio H^2/t_{50} or H^2/t_{90} at higher effective stresses. From Figs. 5.11 to 5.14 it is also evident that the theoretical curves drawn by utilizing Eq. 2.20 and the experimental curves drawn by utilizing Tergahi's theory i.e. Eqs. 5.1 and 5.2 have the same shape. This confirms the findings previously reported by Samarasinghe et al (1982) for a normally consolidated sandy clay. It is also inferred that Eq. 2.20 can be used to predict the nature of C_v - log $\bar{\sigma}$ relationship of normally consolidated silty clays and silts available in Bangladesh.

Variation of C_v with index property can be observed from the plottings of Fig. 5.15. In Fig. 5.15 log C_v corresponding to different effective vertical stresses has been plotted against the plasticity index PI of the soils studied. The values of C_v have been calculated by using Eq. 2.20. Fig. 5.15 shows that in case of normally loaded silty clays

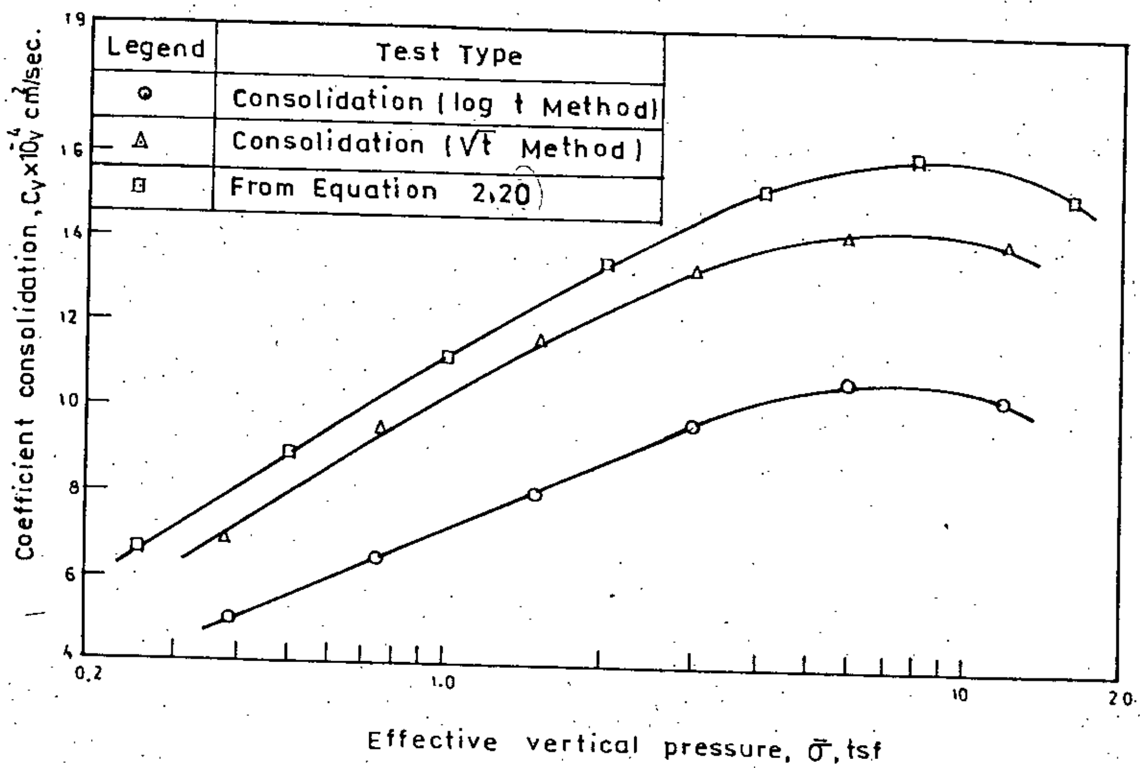


Fig. 5.11 Relationship between C_v and log $\bar{\sigma}$ for Soil-I.

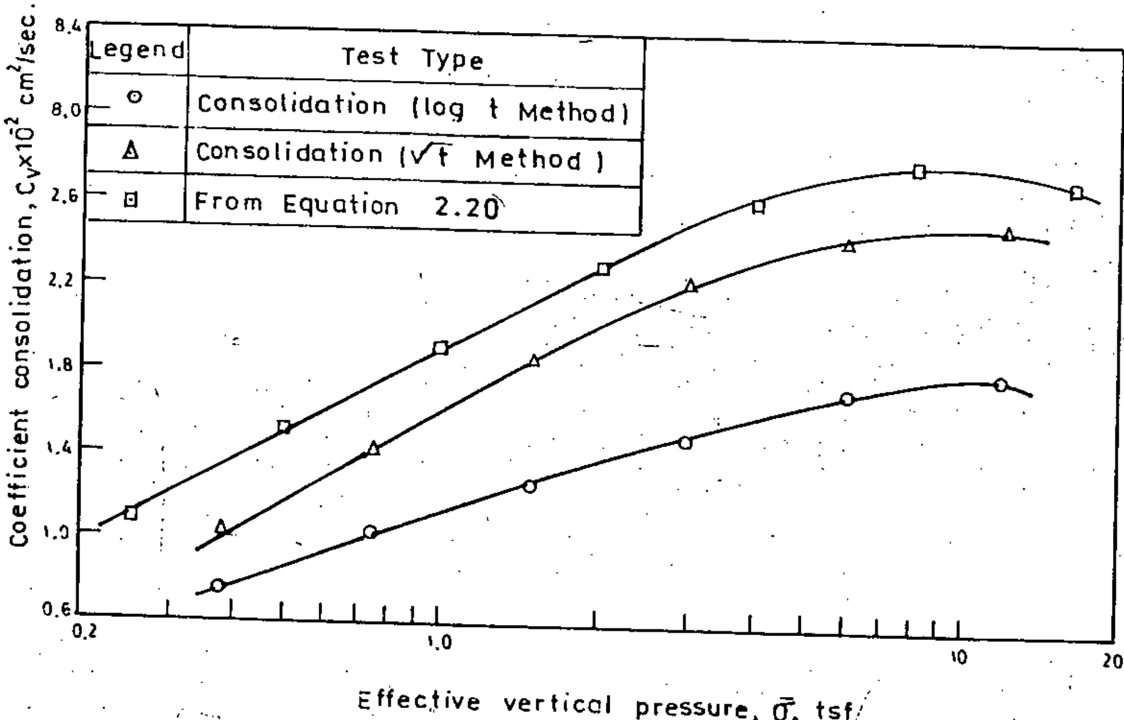


Fig. 5.12 Relationship between C_v and log $\bar{\sigma}$ for Soil-II.

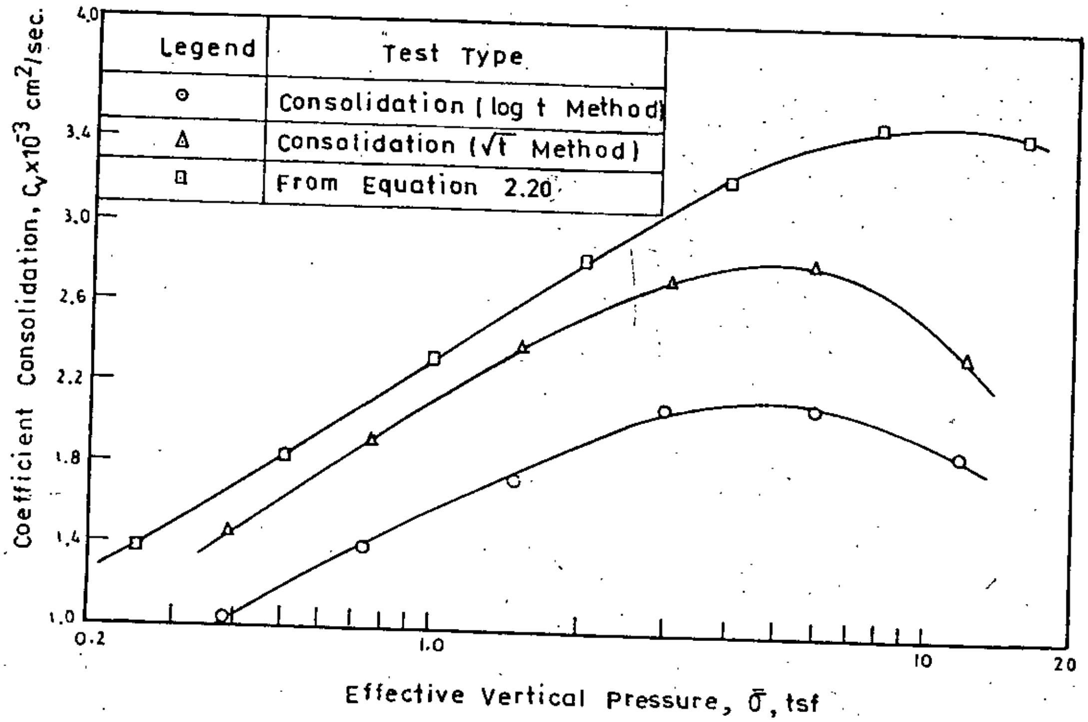


Fig. 5.13 Relationship between C_v and log $\bar{\sigma}$ for Soil-III.

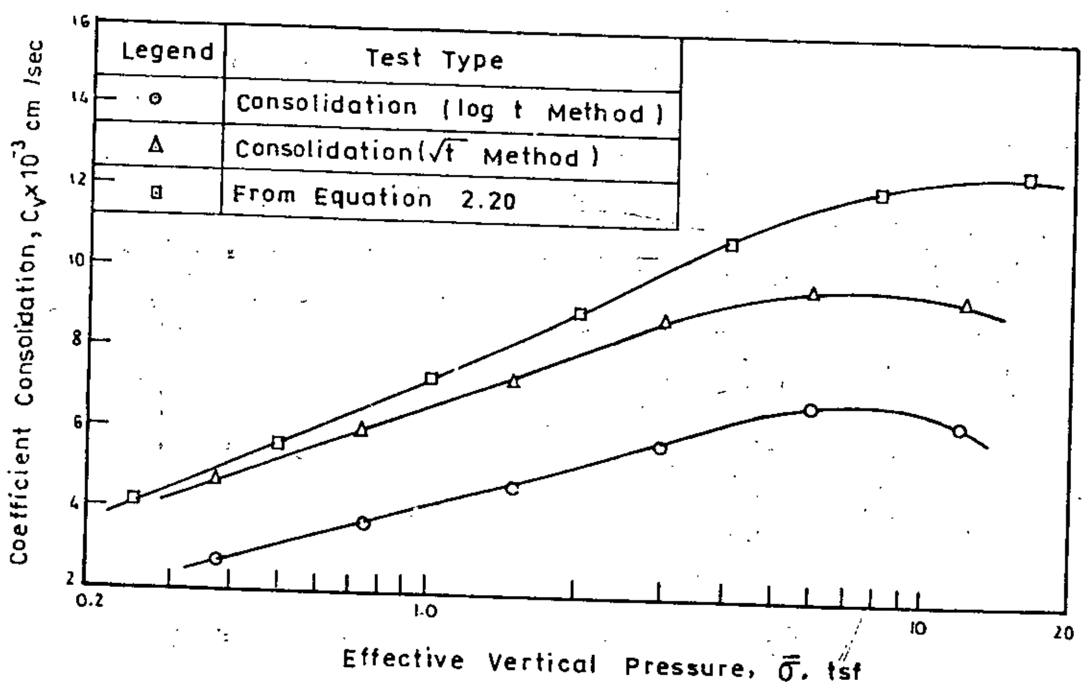


Fig. 5.14 Relationship between C_v and log $\bar{\sigma}$ for Soil-IV.

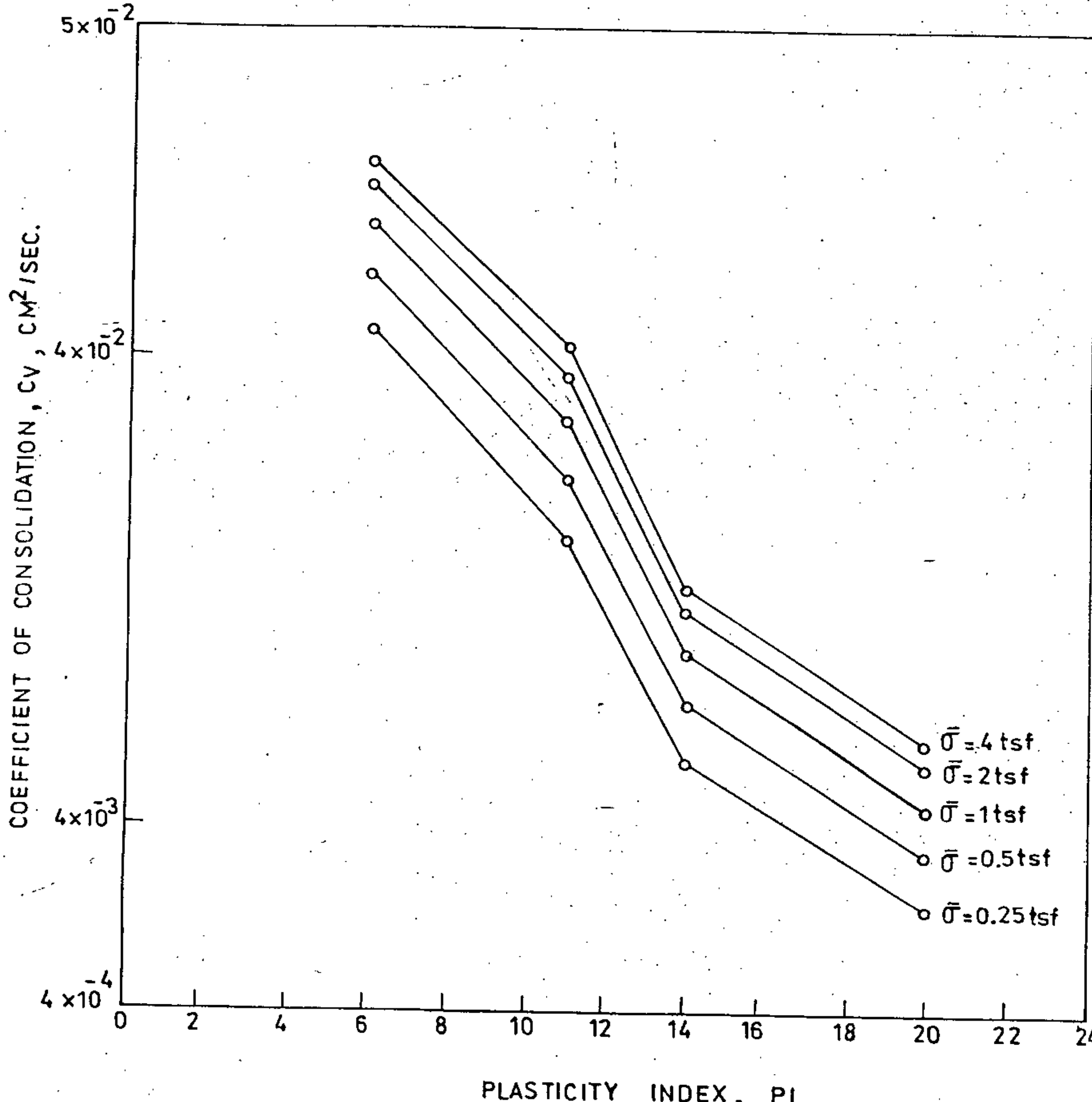
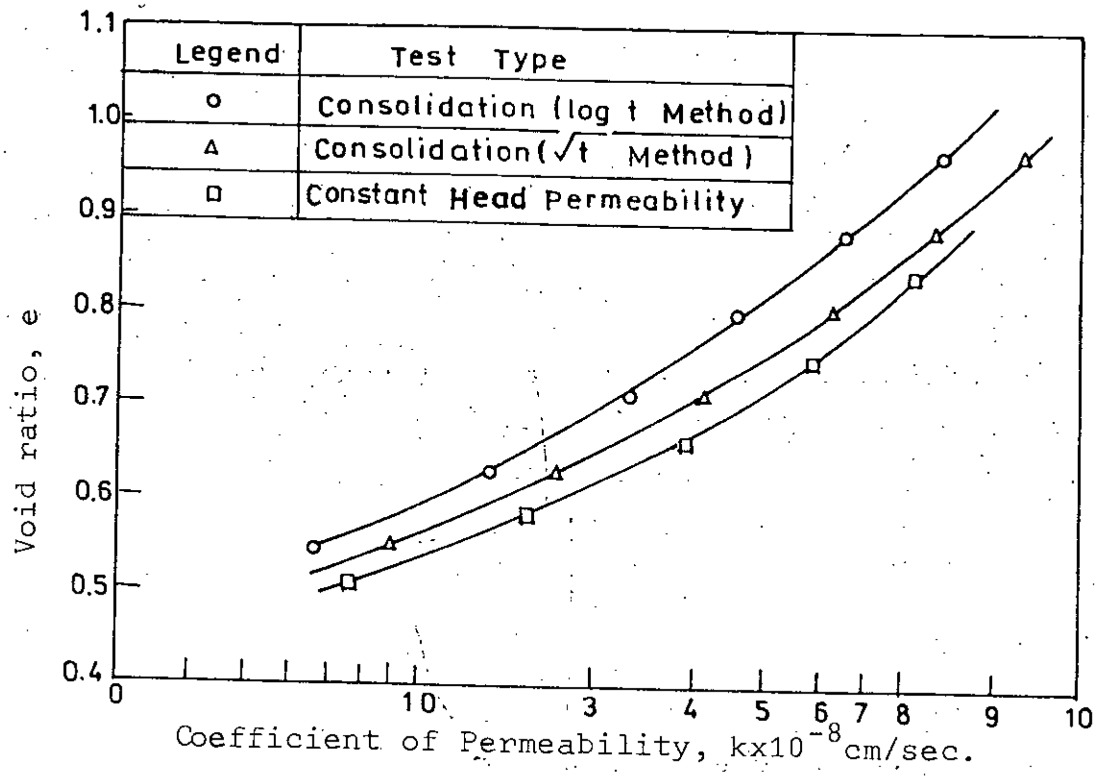


Fig. 5.15 Variation of coefficient of consolidation with plasticity index.

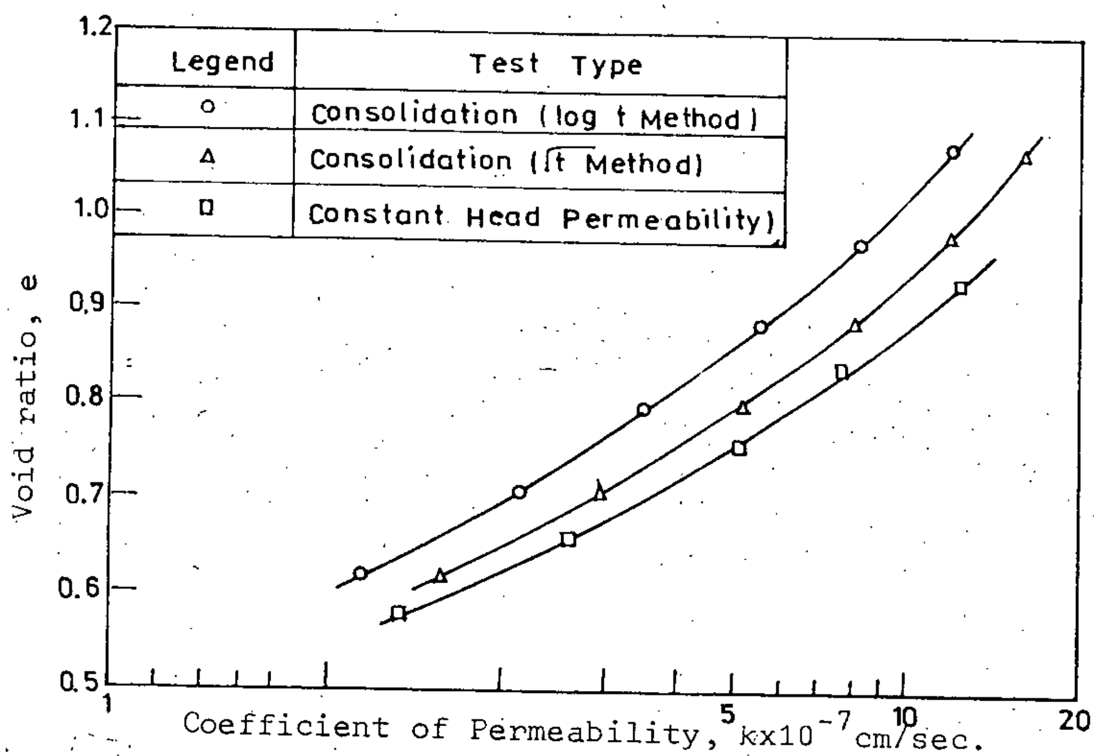
and silts C_v decreases with the increase in plasticity index. C_v values computed by other methods such as consolidation test using Eqs. 5.1 and 5.2 show similar relationship with PI and, therefore, is not presented here.

5.7 Comparison of Coefficient of Permeability and Coefficient of Consolidation Determined by Different Methods

Coefficient of permeability was computed from constant head permeability test and also from consolidation test using logt and $\sqrt{t}$ plots. Figs. 5.16(a) and 5.16(b) show the plottings of e versus logk for Soil-I and Soil-II respectively. From these figures it is evident that at any void ratio k determined from constant head permeability test is higher than that obtained from consolidation test estimated by using either logt or $\sqrt{t}$ plot. It is also evident from these figures that k computed from consolidation test using $\sqrt{t}$ plots is higher than that computed from the same test but using logt plots. Higher permeability values when determined from consolidation test using $\sqrt{t}$ plots is due to that at any stress increment the ratio T_v/t_{90} is higher than the ratio T_v/t_{50}. Similar results as stated above were obtained in case of Soil-III and Soil-IV. A comparison of k-values determined by different-methods for the soils studied is shown in Table 5.6. Permeability determination from constant head permeability test by applying back pressure involves more controlled operations than that from consolidation test utilizing Terzaghi's theory.



(a)



(b)

Fig. 5.16 Relationship between e and log k (a) for Soil-I (b) for Soil-II.

Table 5.6 Comparison of Coefficient of Permeability and Coefficient of Consolidation Determined by Different Methods

Soil	Coefficient of Permeability			Coefficient of consolidation		
	$^{*}k_{\sqrt{t}}/k_{logt}$	$^{**}k_p/k_{logt}$	$k_p/k_{\sqrt{t}}$	$^{+}C_{v_{\sqrt{t}}}/C_{v_{logt}}$	$^{++}C_{v_{Eq}}/C_{v_{logt}}$	$C_{v_{Eq}}/C_{v_{\sqrt{t}}}$
I	1.34-1.45	1.59-1.67	1.10-1.21	1.34-1.45	1.43-1.58	1.05-1.12
II	1.39-1.54	1.73-1.83	1.22-1.29	1.38-1.51	1.54-1.71	1.10-1.26
III	1.25-1.39	1.54-1.82	1.14-1.40	1.25-1.39	1.47-1.83	1.10-1.46
IV	1.42-1.75	1.75-2.20	1.16-1.49	1.43-1.75	1.75-1.96	1.09-1.32

* $k_{\sqrt{t}}$ = coefficient of permeability determined from consolidation test using $\sqrt{t}$ method i.e. t_{90} value

k_{logt} = coefficient of permeability determined from consolidation test using logt method i.e. t_{50} value

** k_p = coefficient of permeability determined from constant head permeability test

\+ $C_{v_{\sqrt{t}}}$ = coefficient of consolidation determined from consolidation test using $\sqrt{t}$ method i.e. t_{90} value

$C_{v_{logt}}$ = coefficient of consolidation determined from consolidation test using logt method i.e. t_{50} value

++ $C_{v_{Eq}}$ = coefficient of consolidation determined from Equation 2.20

Moreover Terzaghi's consolidation theory is based on several assumptions. Therefore, determination of permeability from constant head test is more reliable than consolidation test. However, it has been found that coefficient of permeability determined from constant head permeability test compares more closely with that calculated from consolidation test results using t_{90} value than that obtained from t_{50} value. From Table 5.6 it can be observed that k-value obtained from permeability test is 10 to 49% higher than that obtained from consolidation test using t_{90} value while it is 54 to 120% higher in case of using t_{50} value.

It, therefore, seems reasonable for most practical purposes at least, to determine coefficient of permeability from consolidation test ($\sqrt{t}$ method) which involves relatively simple operations as compared with constant head permeability test by application of back pressure.

From Figs. 5.11 to 5.14 it is evident that, for each soil at any effective vertical stress, C_v calculated from Eq. 2.20 is higher than those computed from consolidation test involving t_{50} and t_{90}. It is also evident that C_v determined from consolidation test involving t_{90} is higher than that determined from same test but involving t_{50}. In Eq. 2.20, C_v has been calculated by using compressibility parameters C_c and e_1 and permeability parameters n and C obtained from constant head permeability test. A comparison of C_v - values determined by different methods is shown in Table 5.6. C_v - values calculated from Eq. 2.20 is more reliable than those computed from consolidation test involving t_{50} and t_{90} because Eq. 2.20 is derived without assuming that k is constant and that a linear relationship exists between e and $\bar{\sigma}$, as assumed in the Terzaghi theory. However, Table 5.6 shows that values of C_v determined from consolidation test involving t_{90} do not vary significantly from that determined from Eq. 2.20. C_v calculated from Eq. 2.20 is 5 to 46% higher than that computed from consolidation test using t_{90} value while it is 43 to 96% higher when computed from consolidation test using t_{50} value. So for use in any practical purpose, coefficient of consolidation can be determined from consolidation test using Eq. 5.2 involving t_{90} rather than from using Eq. 2.20 which involves so many parameters.

5.8 Effect of Vertical Effective Pressure on Coefficient of Permeability

The log $\bar{e}$ versus log $\bar{\sigma}$ relations for the four soils studied have been plotted in Fig. 5.17. Permeability values obtained from constant head permeability test have been used to plot the curves in Fig. 5.17. From this figure it can be observed that, coefficient of permeability decreases with the increase in effective vertical pressure . During the process of consolidation pore volume decreases and the soil particles are likely to become more oriented in the direction perpendicular to the direction of load, so permeability decreases with

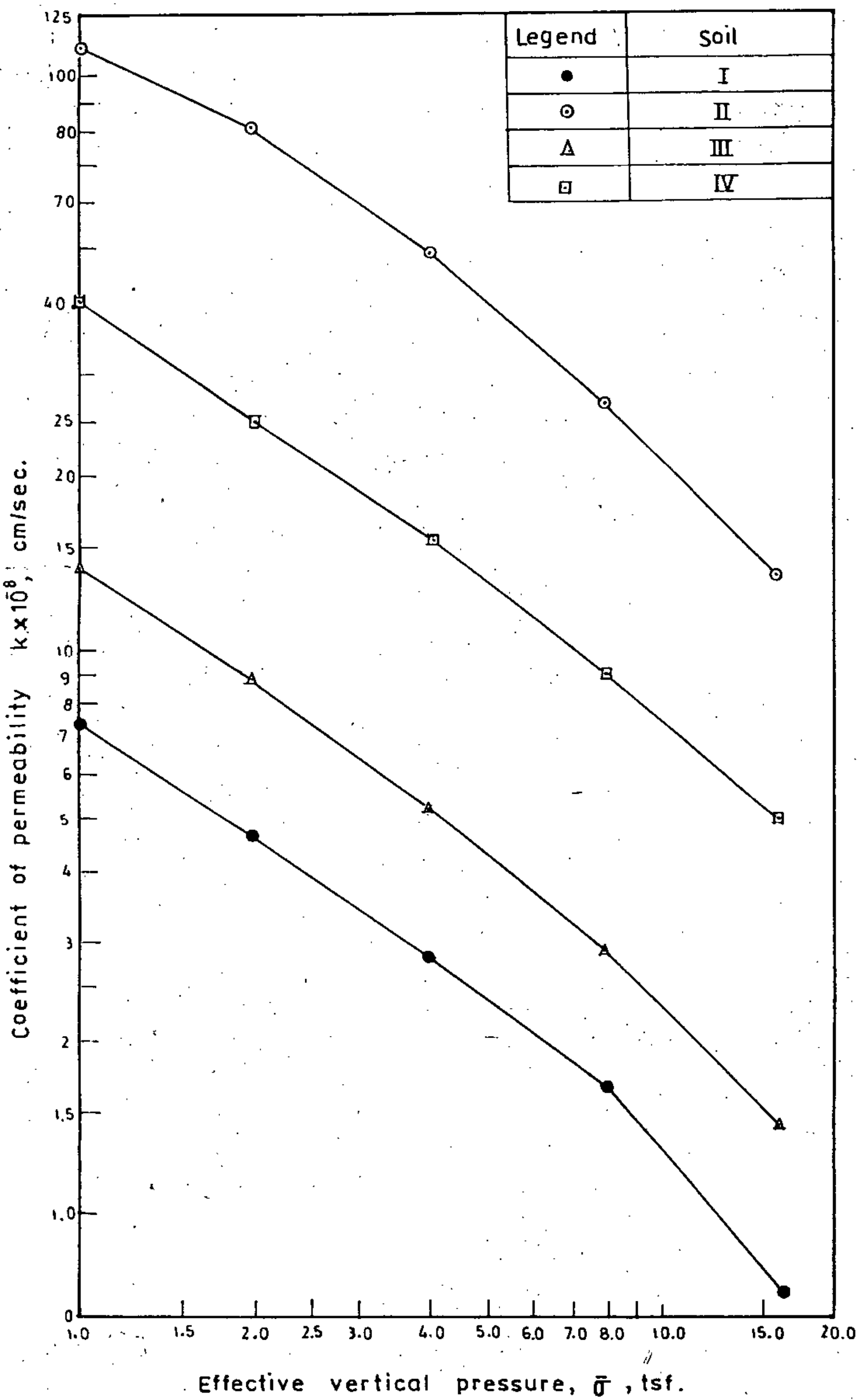


Fig. 5.17 Variation of coefficient of permeability with effective vertical pressure.

consolidation under increased consolidation pressure. The rate of decrease of permeability with the increase in consolidation pressure has also been studied in terms of an index C_k which is the slope of log k versus log $\bar{\sigma}$ plot. The values of C_k for Soil-I, Soil-II, Soil-III and Soil-IV at higher consolidation pressure (8 to 16 tsf) are approximately 1.2, 1.10, 1.11 and 0.88 respectively. However, at lower consolidation pressure (1 to 4 tsf), the respective values of C_k for Soil-I, Soil-II, Soil-III and Soil-IV are approximately 0.697, 0.62, 0.644 and 0.69. Therefore, normally consolidated silty clays and silts are more susceptible to decrease in permeability at higher stress levels than at lower stress levels. Moreover, at higher stress levels, the values of C_k of the soils studied did not differ significantly but at lower stress levels the variation is more or less prominent. However, no definite trend in the variation of C_k value depending on the type of soil has been noticed.

5.9 Variation of Rate of Flow with Time

During constant head permeability test the flow Q through the sample for different time intervals were recorded. Fig. 5.18 shows Q versus t plots for Soil-I where t is the cumulative time against which total flow is plotted. Similar plottings for Soil-II, Soil-III and Soil-IV are shown in Figs. 5.19, 5.20 and 5.21 respectively. From the steep slope of the initial nonlinear portion of these curves it is evident that just

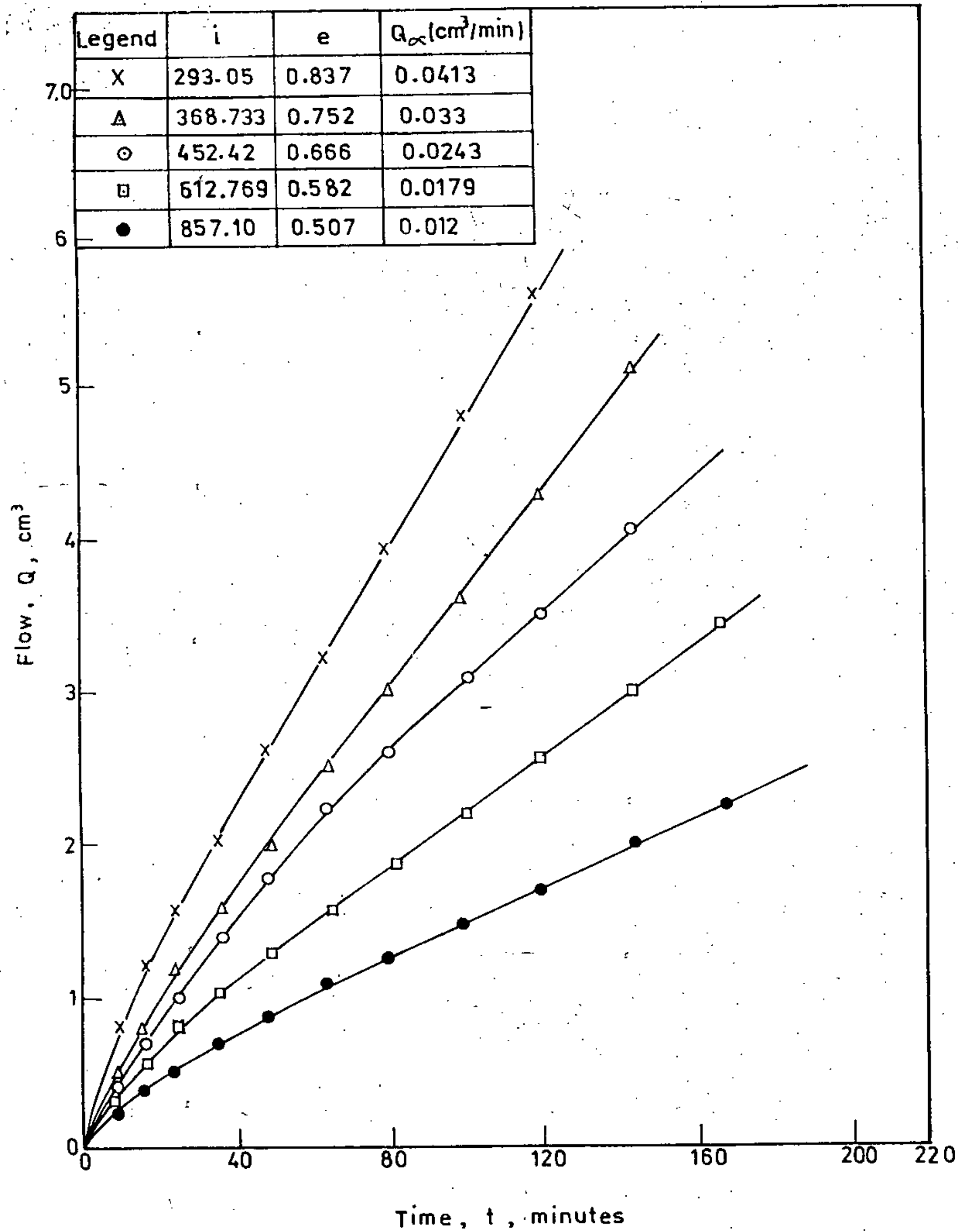


Fig. 5.18 Flow versus time plot for Soil-I.

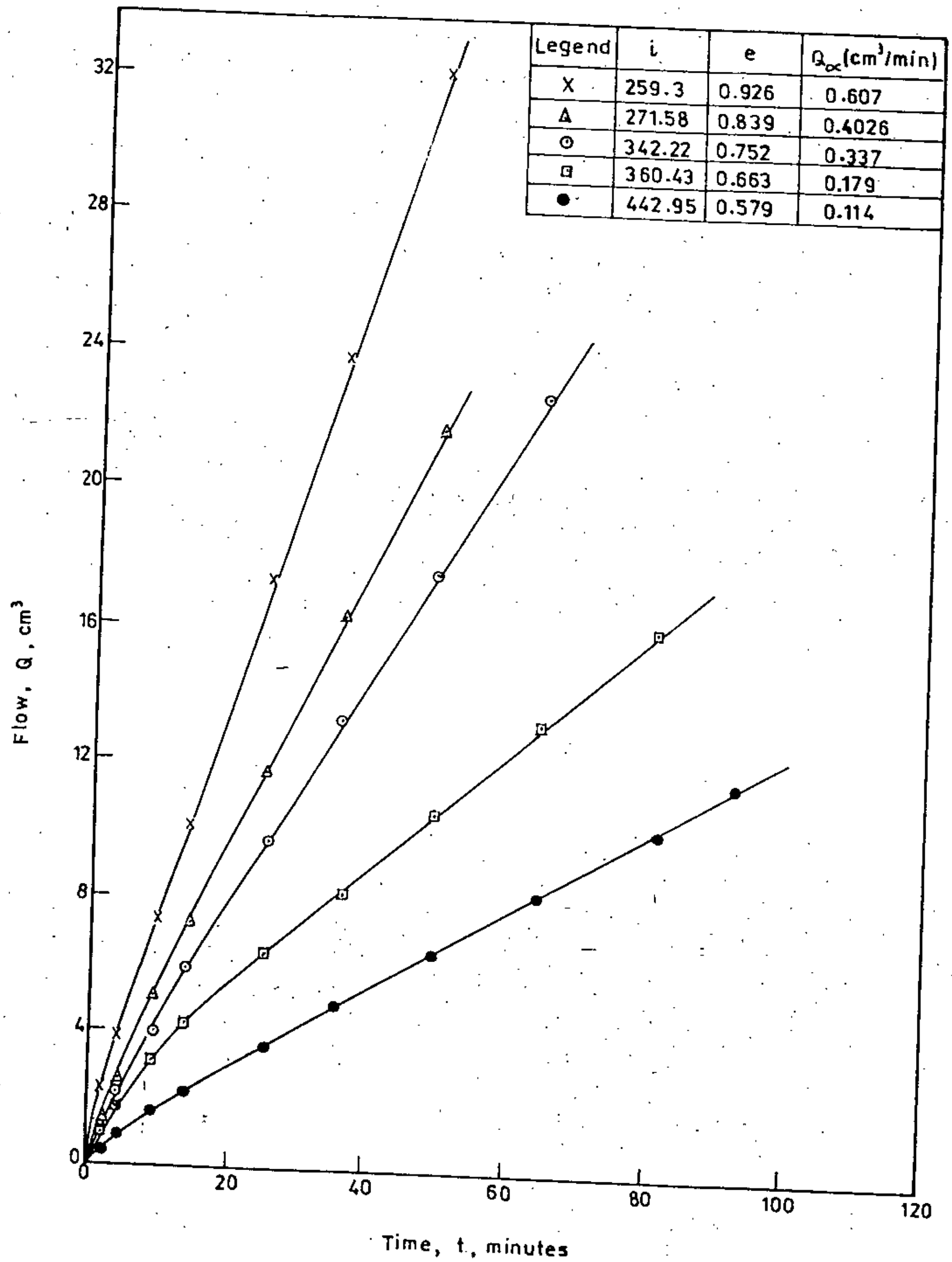


Fig. 5.19 Flow versus time plot for Soil-II.

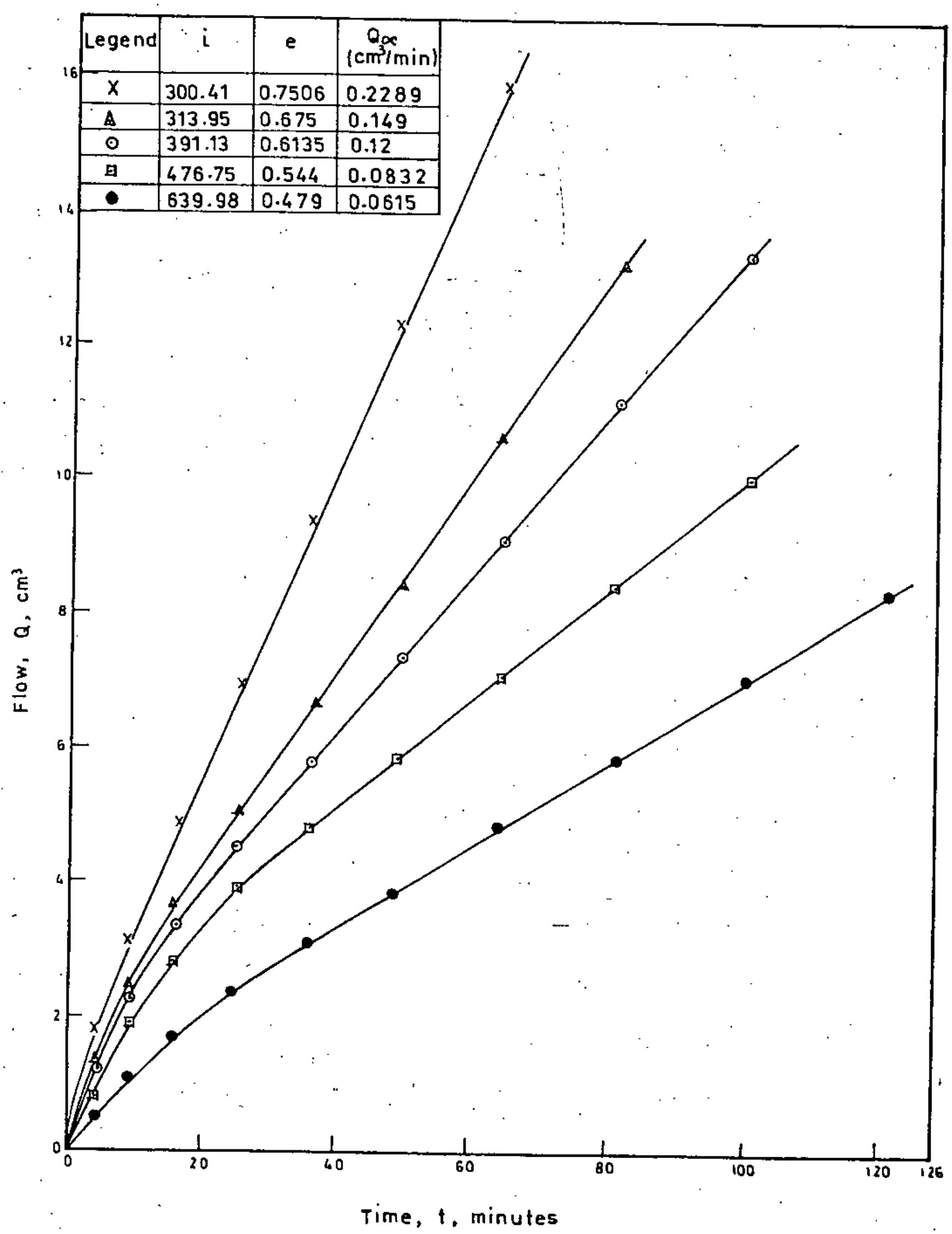


Fig. 5.20 Flow versus time plot for Soil-III.

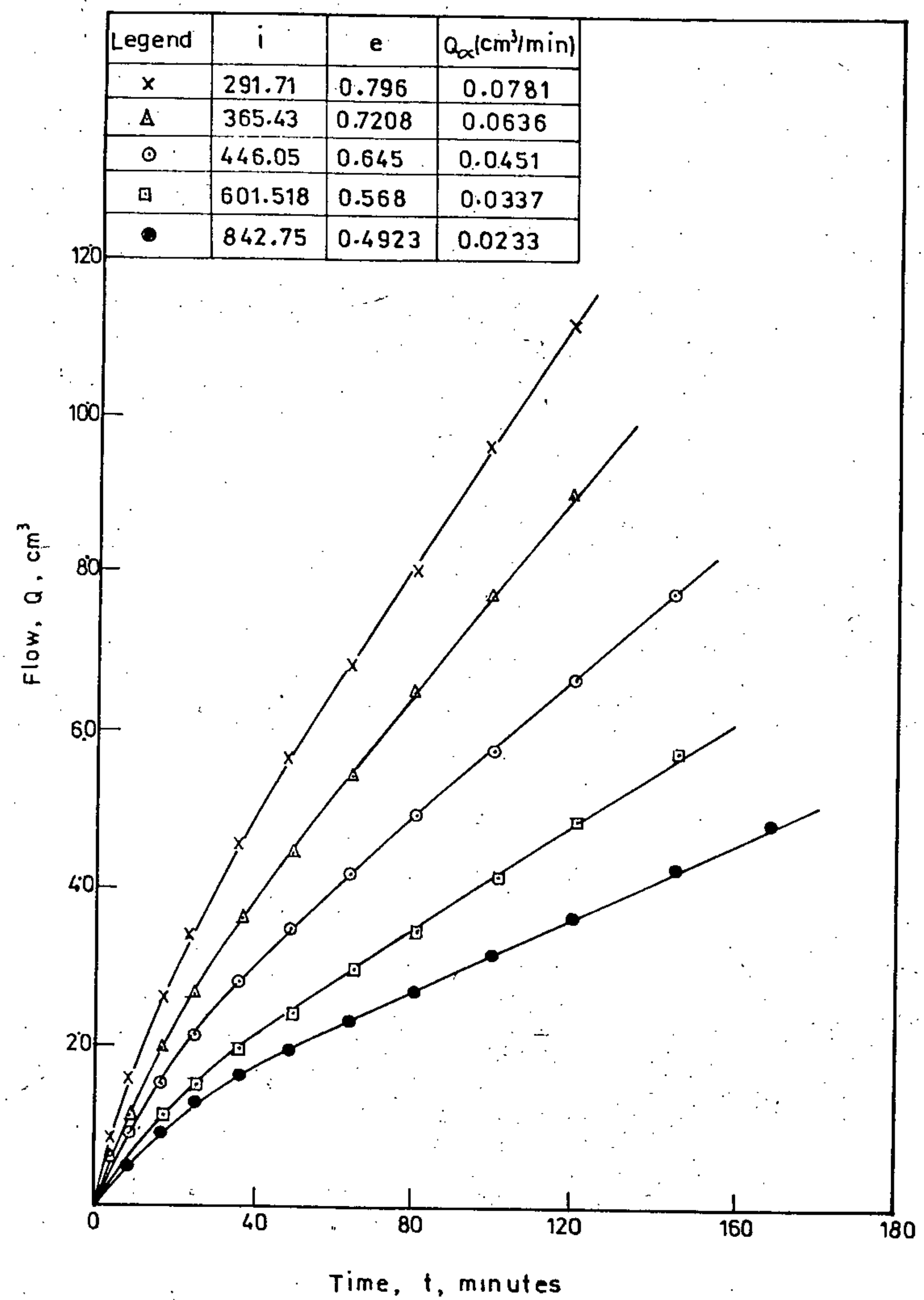


Fig. 5.21 Flow versus time plot for Soil-IV.

after the application of hydraulic pressure the rate of flow is high. The rate of flow subsequently decreases with time and finally attains a steady value after a time depending upon the type of soil and to lesser extent on applied hydraulic pressure. For Soil-I and Soil-II which contain 22 percent and 13 percent clay respectively, time required to attain steady flow is about 50 to 80 minutes. For Soil-II containing no clay this time required is less and is about 15 to 25 minutes. While for Soil-IV containing 8% clay it is 30 to 45 minutes. The initial high rate of flow and later attaining a steady value is probably due to the combined effect of decrease of hydraulic gradient with time, change of fabric of the soil and migration of soil particles during the flow. From the foregoing description, it, therefore, seems essential that in the measurement of coefficient of permeability, the rate of flow corresponding to steady state should be considered.

5.10 Compression Index-Void Ratio Relationship

Attempts have been made to relate compression index C_c with equilibrium void ratio under different effective vertical stresses. The values of C_c have been obtained from consolidation test on soil samples for Soil-I, II, III and IV. In Fig. 5.22 these C_c values have been plotted as open circles against e_1, in which e_1 is the equilibrium void ratio corresponding to 1 tsf effective vertical pressure. On the basis of regression analysis of these values a correlation line shown as a firm line in Fig. 5.22 is drawn. Samarasinghe et al (1982) and Raymond (1966) also published data on C_c and e_1 values for four normally consolidated clays of medium to high plasticity (LL = 27 to 118 and PI = 14 to 72). The data from Raymond are plotted as open triangles and those from Samarasinghe as open square in Fig. 5.22. These plottings lie close to the line drawn on the basis of present test results. All these data including those obtained for Soil-I to IV are reanalysed and a correlation line shown in dotted in Fig. 2.22 is drawn. The equation of firm line (for Soils I, II, III and IV) is as follows:

$$C_c = 0.36\,(e_1 - 0.11) \qquad 5.5$$

The equation of the dotted line for all the soils is as follows:

$$C_c = 0.39\,(e_1 - 0.14) \qquad 5.6$$

The correlation coefficient for the linear plots expressed by Eqs. 5.5 and 5.6 are 0.928 and 0.994 respectively.

In Fig. 5.23 C_c has been plotted against equilibrium void ratio under 1/4 tsf ($e_{\frac{1}{4}}$), $\frac{1}{2}$ tsf ($e_{\frac{1}{2}}$) and 2 tsf (e_2) effective vertical pressure respectively. The relations may be expressed by the following empirical expressions:

$$C_c = 0.323\,(e_{\frac{1}{4}} - 0.186) \qquad 5.7$$

$$C_c = 0.33\,(e_{\frac{1}{2}} - 0.11) \qquad 5.8$$

$$C_c = 0.40\,(e_2 - 0.10) \qquad 5.9$$

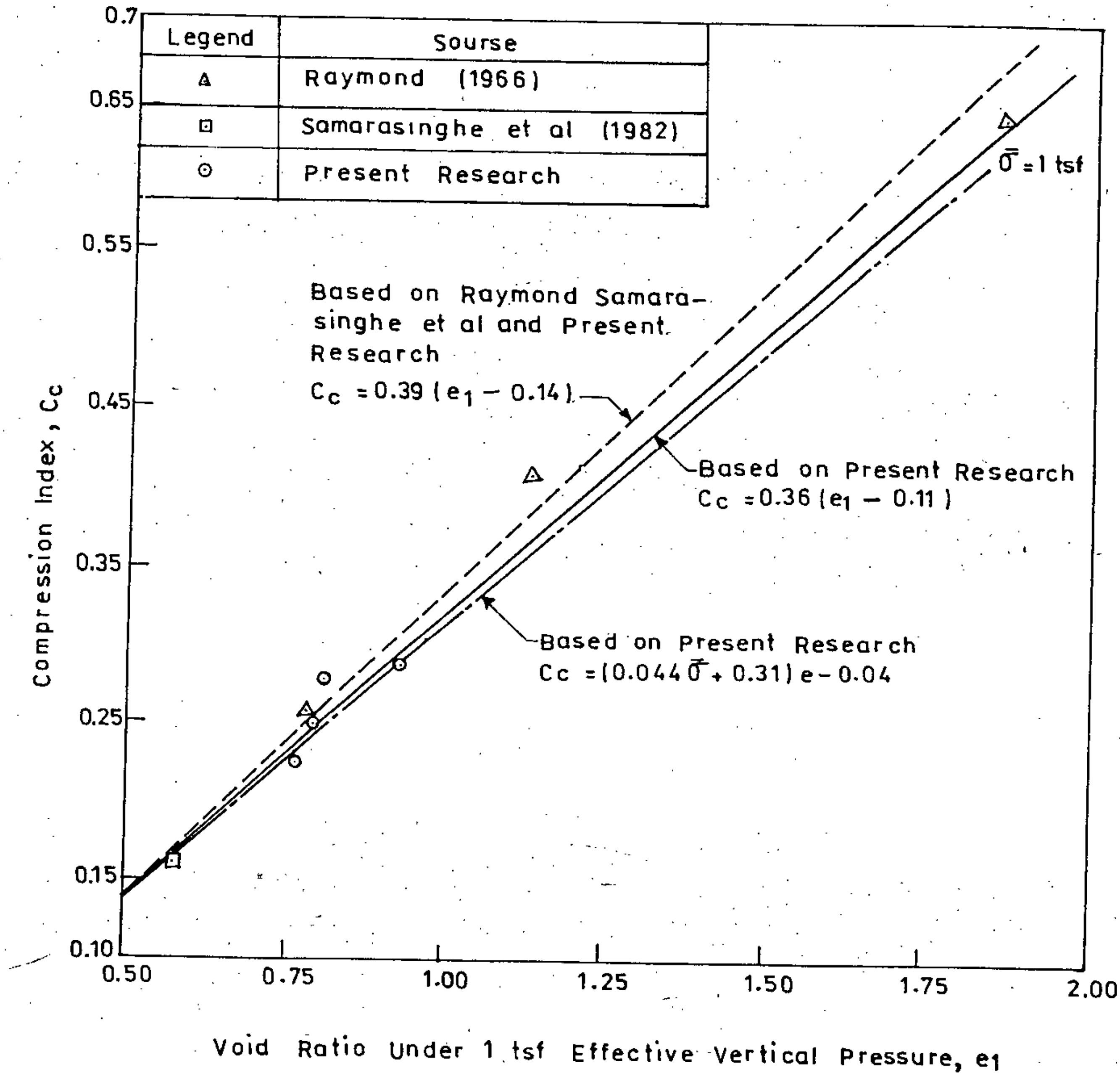


Fig. 5.22 C_c versus e_1 plots.

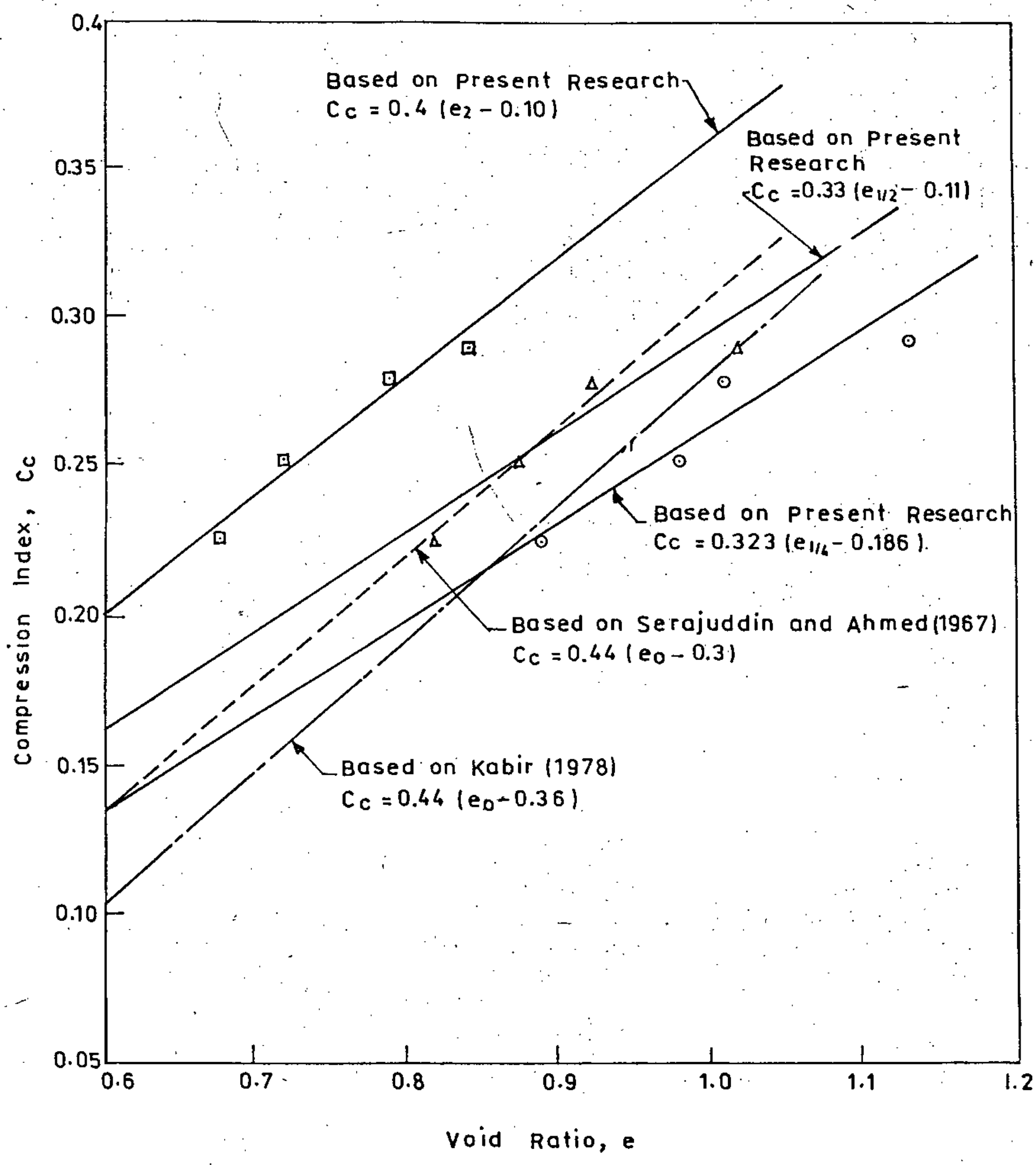


Fig. 5.23 Compression Index - Void ratio relations.

The correlation coefficients of Equations 5.7, 5.8 and 5.9 are respectively 0.927, 0.944 and 0.918.

Serajuddin and Ahmed (1967) and Kabir (1978) have shown relations between C_c and initial void ratio e_o for a number of undisturbed plastic clays and silts of Bangladesh and suggested empirical relations. In Fig. 5.23 empirical relations suggested by them have been plotted. From the curves of Figs. 5.22 and 5.23 it is evident that at a particular effective vertical pressure, C_c increases with the increase in equilibrium void ratio and also that for a given equilibrium void ratio C_c increases with increasing effective overburden pressure. Therefore, C_c is a function of both void ratio and effective overburden pressure. From Fig. 5.23 it is also apparent that the effective overburden pressure of the soil samples tested by Serajuddin and Ahmed was in the range of $\frac{1}{4}$ to $\frac{1}{2}$ tsf and of those tested by Kabir was about less than $\frac{1}{4}$ to $\frac{1}{4}$ tsf.

On the basis of Eqs. 5.5, 5.7, 5.8 and 5.9, the following equation expressing C_c as a function of void ratio and effective overburden pressure has been derived.

$$C_c = (0.044\,\bar{\sigma} + 0.31)\,e - 0.04 \qquad 5.10$$

Eq. 5.10 has been derived from the results of normally consolidated silty clays and silts with liquid limit ranging from 33 to 40 and plasticity index varying between 6 and 20. A plot of Eq. 5.10 for $\bar{\sigma}$ = 1 tsf has been shown by the chain line in Fig. 5.22. This equation can be used to estimate compression index C_c of normally consolidated silty clays and silts provided the effective overburden pressure $\bar{\sigma}$ and void ratio corresponding to that pressure are known.

5.11 Constrained Modulus - Effective Vertical Pressure Relationship

During consolidation test the soil sample was laterally confined. Attempt has been made to find the relation between constrained modulus D and effective vertical pressure $\bar{\sigma}$. Constrained modulus has been obtained by dividing the change in pressure (pressure increment by the resulting vertical strain i.e. deformation per unit height of sample which developes at the end of primary consolidation. The values of constrained modulus for the four soils used in this research are presented in Table 5.7.

A plot of D versus $\bar{\sigma}$ for each soil is shown in Fig. 5.24. From this figure it is clear that for each soil D versus $\bar{\sigma}$ plot can be represented by a straight line passing through the origin. The equations of such lines with their respective correlation coefficients are presented in Table 5.8. Since, the slopes of these lines do not vary significantly, a dotted line such as shown in Fig. 5.24 may be considered presenting the average of all these plots determined by regression analysis. The equation of this line is

$$D = 14.47\ \bar{\sigma} \qquad 5.11$$

Table 5.7 Magnitudes of Constrained Modulus of the Soils Investigated

Applied pressure (tsf)	Pressure increment (tsf)	Deformation, ΔS (% of height)				Average pressure (tsf)	Constrained modulus, D (tsf)			
		Soil					Soil			
		I	II	III	IV		I	II	III	IV
0.25	0.25	-	-	-	-	-	-	-	-	-
	0.25	4.178	5.35	4.024	3.74	0.375	5.98	4.67	6.21	6.68
0.5										
	0.5	4.472	4.438	4.116	3.633	0.75	11.8	11.26	12.147	13.76
1.0										
	1.0	4.667	4.625	4.176	4.272	1.50	21.42	21.61	23.94	23.40
2.0										
	2.0	4.82	4.768	4.419	3.688	3.0	41.5	41.93	45.25	54.21
4.0										
	4.0	5.104	5.036	4.663	4.284	6.0	78.36	79.43	85.78	93.35
8.0										
	8.0	4.668	5.09	4.817	4.205	12.0	171.3	157.0	166.0	190.2
16.0										

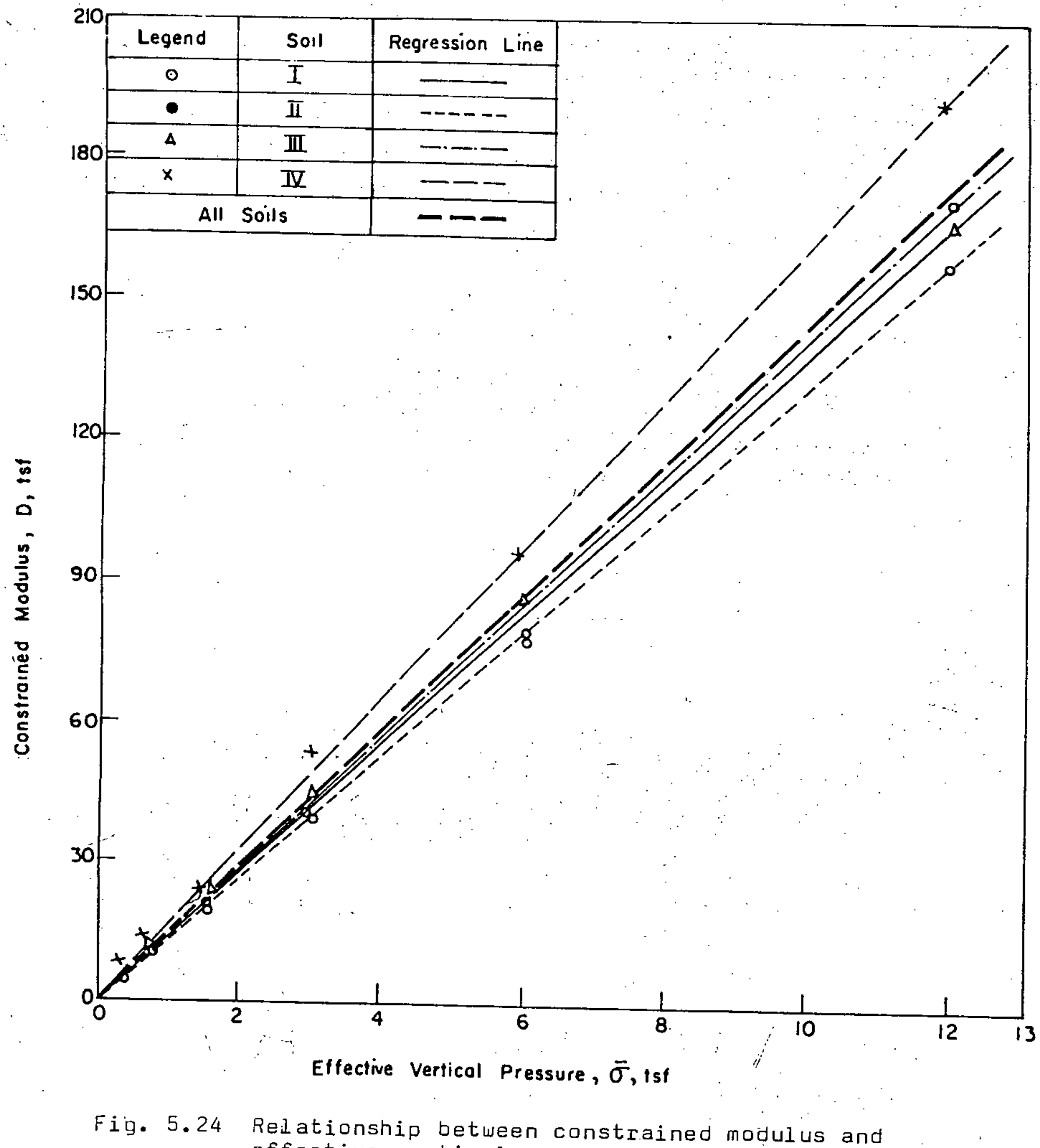


Fig. 5.24 Relationship between constrained modulus and effective vertical pressure.

Eq. 5.11 has been derived from the results of normally consolidated silty clays and silts with liquid limit varying from 33 to 40 and plasticity index varying between 6 and 20. Correlation coefficient of Eq. 5.11 is 0.9992. Eq. 5.11 may be used as a simple method for estimation of settlements by using the expression:

$$S_t = \frac{\Delta\sigma}{D} H_t \qquad 5.12$$

here $\Delta\sigma$ = change in effective vertical pressure

H_t = thickness of layer undergoing settlement

$D = 14.47\ \bar{\sigma}$.

Table 5.8 Equations and correlation coefficients of D vs. $\bar{\sigma}$ plots

Soil	Equation	Correlation coefficient
I	$D = 13.98\ \bar{\sigma}$	0.9988
II	$D = 13.37\ \bar{\sigma}$	0.999
III	$D = 14.36\ \bar{\sigma}$	0.998
IV	$D = 16.15\ \bar{\sigma}$	0.9991

Details of estimation of settlement in the center of a raft foundation by using Eq. 5.12 and also by using the conventional method i.e. $S = \frac{H_t\ C_c}{1+e_o} \log \frac{\bar{\sigma} + \Delta\sigma}{\bar{\sigma}}$ are presented in Appendix-D. For this purpose a raft 40 ft wide and 60 ft long supported on Soil-I has been taken. The Soil-I is assumed to be extended upto infinite depth. Settlement has been estimated for a depth of 90' below the base of foundation. 90' depth has been divided into 9 layers with layer thickness of 10 ft. It has been found that for layers in which value of $\frac{\Delta\sigma}{\bar{\sigma}}$ is higher (0.5 to 1.0) settlement estimation by Eq.5.12 is approximately 25% to 50% higher than that estimated by using conventional method. However, for layers in which value of $\frac{\Delta\sigma}{\bar{\sigma}}$ is relatively low (0.1 to 0.33), settlement estimation from Eq. 5.12 is approximately 3% to 17% higher than that estimated in conventional method. The total settlement for 90' depth calculated from Eq. 5.12 is 1.28 times greater than that estimated in conventional method. Similar observations were found for other soils as well. It is therefore inferred that for higher values of $\frac{\Delta\sigma}{\bar{\sigma}}$ i.e at shallow depth Eq. 5.12 overestimates settlement of foundation while for lower values of $\frac{\Delta\sigma}{\bar{\sigma}}$ i.e at higher depths settlement estimated on the basis of Eq. 5.12 agrees much closely with that estimated in conventional method.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The important findings and conclusions drawn on various aspects of this research may be summarized as follows:

1. Based on permeability test results of three saturated normally consolidated silty clays and one slightly plastic silt, liquid limit and plastic limit of which varied from 30 to 40 and 6 to 20 respectively, it can be concluded that the coefficient of permeability (k) of saturated and normally consolidated silty clays and silts can be related to its void ratio by the equation $k = C\frac{e^{5.0}}{1+e}$. However, the value of C is not constant and ranged between 2×10^{-7} and 30×10^{-7} cm/sec. for the soils tested.

2. The magnitude of the parameter C generally increases with the decrease in liquid limit and plasticity index, magnitude of C also depends upon method of determination i.e. type of test. For the soils studied in this research, value of C computed from permeability test is higher than that computed from consolidation test.

3. The relationship between void ratio and logarithm of coefficient of permeability for saturated and normally consolidated silty clays and silts is nonlinear and for each soil coefficient of permeability decreased with the decrease

in void ratio. It has also been observed that coefficient of permeability decreases with increasing plasticity index and liquid limit. These findings are in agreement with those reported by previous investigators for normally consolidated clays.

4. For saturated normally consolidated silty clays and silts coefficient of permeability determined from constant head permeability test by applying back pressure is higher than that obtained from consolidation test. It has been found from this study that coefficient of permeability computed from consolidation test using square root of time method is closer to that obtained from permeability test and, therefore, can be used for practical purposes.

5. Compressibility parameters C_c and e_1 of normally consolidated clays generally increase with the increase in liquid limit and plasticity index of soil.

6. For saturated normally consolidated silty clays and silts coefficient of consolidation C_v increases almost linearly with logarithm of effective vertical pressure $\bar{\sigma}$. However, value of C_v decreases at higher values of effective vertical pressure. At any particular value of $\bar{\sigma}$, C_v decreases with increasing liquid limit and plasticity index.

7. The magnitude of C_v calculated from Eq. 2.20 i.e $C_v = \frac{2.303}{\gamma_w C_c} (-C_c \log \frac{\bar{\sigma}}{\bar{\sigma}_1} + e_1)^n \bar{\sigma}$ is usually higher than that

determined from consolidation test by use of time-fitting techniques. Magnitude of C_v calculated from Eq. 2.20 agrees more closely with that determined from consolidation test using t_{90} than that using t_{50}. This suggests that it is more convenient to determine the value of C_v from consolidation test using t_{90} than using Eq. 2.20 which involves many parameters.

8. Compression index C_c is not a function of void ratio only as shown in the empirical relations proposed by the previous investigators (Nishida, 1956; Sowers, 1962; Serajuddin and Ahmed. 1967; Hough, 1969; Kabir, 1978). It has been found that, C_c is a function of both void ratio and effective overburden pressure. On the basis of present study, the following empirical relation has been derived.

$$C_c = (0.044\,\bar{\sigma} + 0.31)e - 0.04$$

In the above relation $\bar{\sigma}$ is the effective overburden pressure in tsf.

9. Based on consolidation test results of the soils investigated, it can be concluded that the constrained modulus D of saturated normally consolidated silty clays and silts can be related to effective vertical pressure $\bar{\sigma}$ by the equation $D = 14.47\,\bar{\sigma}$. This equation should be used as a simple method of estimating settlements by utilizing Eq. 5.12 i.e $S_t = \frac{\Delta\sigma}{D} H_t$ where $\Delta\sigma$ is the change in effective vertical pressure, $D = 14.47\,\bar{\sigma}$ and H_t is the thickness of layer undergoing settlement. It has been observed that for higher values

of $\Delta\sigma/\bar{\sigma}$ i.e at shallow depths Eq. 5.12 overestimates settlement. However, for lower values of $\Delta\sigma/\bar{\sigma}$ i.e at higher depths settlement estimated on the basis of Eq. 5.12 agrees much closely with that estimated by using conventional method.

6.2 Recommendations for Further Study

Several aspects of the work presented in this thesis require further study. Some of the important areas of further research may be listed as follows:

1. In the present research, permeability and consolidation behavior of three silty clays and one slightly plastic silt have been investigated. All the soils were saturated and normally consolidated. It would be interesting to know the changes that take place in structure and fabric of the soil during loading and permeation. This would clarify the physical concepts regarding flow and consolidation mechanism.

2. To observe the effect of stress history on permeability and consolidation characteristics, the same investigation may be carried out with samples of overconsolidated soils having different overconsolidation ratio.

3. In the present research no definite correlation between index properties of the soils and the parameters n and e_1 have been found. Further research, therefore, should be carried out on these aspects with soils having widely varying values of liquid limit and plasticity index.

4. In this investigation, soil samples were laterally confined. Consolidation test was performed with a stress increment ratio of unity and duration of each loading step was approximately 24 hours. Permeability tests were done under high hydraulic gradients. In order to observe the effect of testing conditions this investigation may be carried out with different stress increment ratios and duration of loading. Attempts should be made to perform permeability test under low hydraulic gradients and to perform consolidation test on isotropically consolidated soil samples because flow through a soil actually takes place under low hydraulic gradients and soil can deform both laterally and vertically.

5. Further research may be carried out with undisturbed soil samples collected from different regions of Bangladesh to check whether the proposed models and empirical relation can be used for natural soils as well.

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APPENDIX-A

PROCEDURE FOR PREPARING POROUS STONE

Because of non-availability of porous stone of 2" diameter in the laboratory, attempt was made to prepare it. At first a mould made of brass with three detachable parts, a top, bottom and middle, was designed. The three parts of the mould were designed in such a way that the exact diameter and height of the porous stone would be $1\frac{15}{16}$" and 1/2" respectively. A schematic diagram of the mould assembly is shown in Fig. A.1.

The following steps were followed in making porous stone:

1) About 2000 gms of Sylhet sand was cleaned by wash-sieving. The washed sample was ovendried for 24 hours at 105-110^{o}C and then colled at room temperature.

2) The sample was then sieved mechanically and only the portion passing sieve no. 20 and retained on sieve no. 50 was sampled for the purpose.

3) For making a single stone about 55 gms of sand was taken and thoroughly mixed with Araldite HV 100 (Hardner) and Araldite AV 100 (Adhesive). The total volume of araldite, hardner plus adhesive, was approximately 10% by volume of sand used.

4). The inner wall of the middle part of the mould and the contact surfaces of the detachable portions were lubricated properly to prevent sticking of sand.

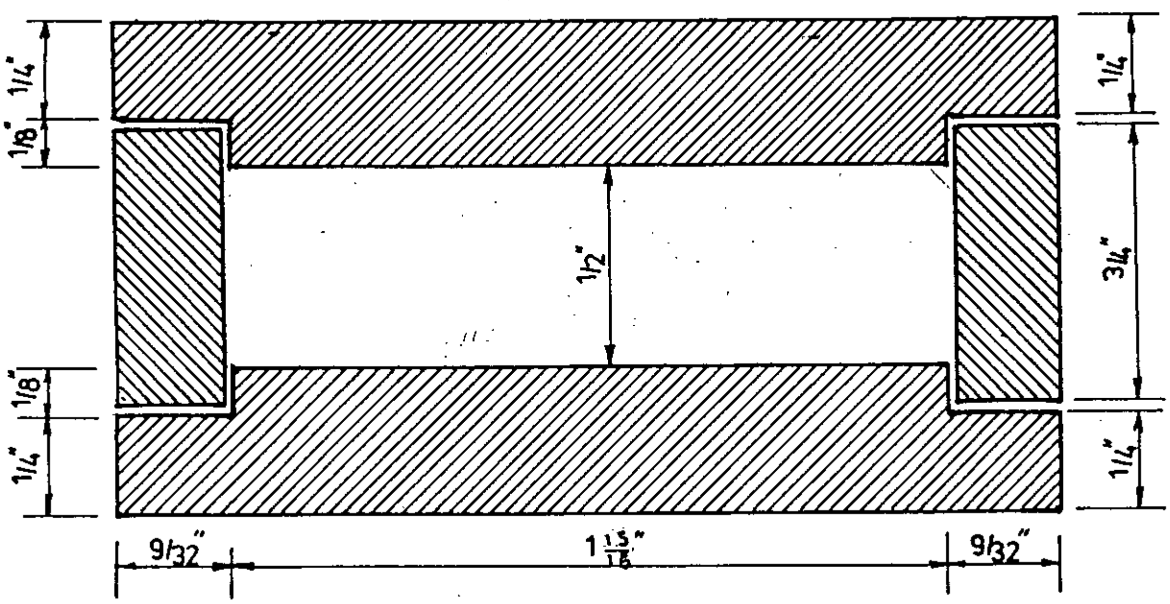


Fig. A.1 Details of mould for making porous stone.

5) The middle and bottom parts of the mould were assembled and sand mixed with araldite was placed in three layers, each layer tamped uniformly all over the surface by a 3/8" dia brass rod.

6) The top part of the mould was attached, pressed firmly and later released.

7) The whole assembly was kept in a jack for 4-6 hours. After that period, the top and bottom parts of the mould were detached.

8) The stone just formed from sand araldite mix was removed from the middle part of the mould. The porous stone thus prepared was dried at room temperature for another 24 hours to impart hardness.

The porous stones were boiled with water for complete saturation and then cooled. Falling head permeability tests were performed for each porous stone. Coefficient of permeability of the porous stones varied between 1.7×10^{-2} cm/sec to 1.96×10^{-2} cm/sec.

APPENDIX-B

CONSOLIDATION READINGS

Table B.1 Consolidation Readings for Soil-I

Time, t (minute)	Dial Reading (10^{-4} inch)					
	Applied pressure (tsf)					
	1/2	1	2	4	8	16
1/4	216	904	1551	2183	2840	3464
1/2	225	914	1561	2192	2851	3480
1	240	940	1585	2220	2875	3504
2	272	978	1633	2273	2930	3548
4	320	1008	1696	2343	3007	3609
8	379	1089	1762	2435	3117	3690
15	442	1161	1842	2516	3179	3763
30	574	1288	1960	2640	3303	3863
60	744	1407	2076	2714	3360	3908
120	803	1452	2109	2740	3377	3928
240	832	1472	2116	2748	3381	3935
480	838	1478	2121	2755	3385	3940
1440	844	1485	2130	2761	3390	3944

Table B.2 Consolidation Readings for Soil-II

Time, t (minute)	Dial Reading (10^{-4} inch)					
	Applied pressure (tsf)					
	1/2	1	2	4	8	16
1/4	391	1310	1991	2688	3417	4208
1/2	451	1360	2061	2786	3535	4320
1	540	1440	2186	2915	3662	4445
2	678	1580	2308	3041	3800	4568
4	811	1709	2413	3145	3887	4618
8	928	1796	2498	3204	3935	4636
15	1019	1829	2530	3238	3962	4645
30	1082	1852	2549	3256	3970	4650
60	1115	1858	2553	3260	3972	4651
120	1117	1860	2556	3262	3975	4652
240	1120	1862	2559	3264	3977	4654
480	1121	1863	2560	3265	3977	4655
1440	1122	1865	2561	3267	3978	4655

Table B.3 Consolidation Readings for Soil-III

Time, t (minute)	Dial Reading (10^{-4} inch)					
	Applied pressure (tsf)					
	1/2	1	2	4	8	16
1/4	213	860	1450	2038	2618	3212
1/2	222	872	1470	2042	2624	3219
1	244	890	1486	2072	2659	3249
2	301	366	1551	2158	2748	3340
4	366	1044	1639	2260	2863	3448
8	440	1142	1721	2381	2996	3578
15	542	1227	1857	2478	3071	3665
30	698	1334	1929	2540	3127	3713
60	767	1375	1958	2554	3140	3720
120	790	1391	1970	2558	3143	3724
240	798	1395	1975	2560	3145	3726
480	803	1399	1978	2562	3148	3728
1440	811	1405	1983	2564	3151	3732

Table B.4 Consolidation Readings for Soil-IV

Time, t (minute)	Dial Reading (10^{-4} inch)					
	Applied pressure (tsf)					
	1/2	1	2	4	8	16
1/4	210	799	1320	1888	2397	2900
1/2	215	806	1331	1009	2437	2959
1	242	829	1390	1970	2540	3028
2	341	951	1528	2081	2626	3136
4	458	1049	1660	2183	2728	3234
8	560	1160	1758	2246	2780	3282
15	642	1206	1798	2277	2802	3307
30	708	1240	1815	2288	2817	3313
60	736	1244	1820	2291	2820	3315
120	740	1246	1823	2293	2822	3318
240	742	1248	1824	2295	2823	3320
480	743	1250	1825	2296	2824	3321
1440	746	1252	1828	2298	2826	3323

APPENDIX-C

DERIVATION OF THE RELATIONSHIP BETWEEN COEFFICIENT OF CONSOLIDATION AND EFFECTIVE VERTICAL PRESSURE

For an infinitesimal element of volume dxdydz, the time-rate-of-change of volume for one-dimensional flow is as given by $k \dfrac{\partial^2 h}{\partial z^2}$ dxdydz, where h is the total head at any point in the element dxdydz, z is the coordinate of flow direction and k is coefficient of permeability of the element dxdydz. The pore volume of the element dxdydz is dxdydz $\dfrac{e}{1+e}$, where e is the void ratio of the element. Since, all changes in volume must be changes in pore volume, a second expression for the time rate of change volume is $\dfrac{\partial}{\partial t}$ (dxdydz $\dfrac{e}{1+e}$). Equating this expression to the first one gives,

$$k \frac{\partial^2 h}{\partial z^2} dxdydz = \frac{\partial}{\partial t} \left(dxdydz \frac{e}{1+e}\right)$$

or $$k \frac{\partial^2 h}{\partial z^2} dxdydz = \frac{dxdydz}{1+e} \frac{\partial e}{\partial t}$$

or $$k \frac{\partial^2 h}{\partial z^2} = \frac{1}{1+e} \frac{\partial e}{\partial t} \qquad (C.1)$$

Only heads due to hydrostatic excess pressure will tend to cause flow in the case under consideration. Thus h in Eq. C.1 may be replaced by $\dfrac{u}{\gamma_w}$ giving

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e} \frac{\partial e}{\partial t}$$

or $$\frac{\partial e}{\partial t} = \frac{k(1+e)}{\gamma_w} \frac{\partial^2 u}{\partial z^2} \qquad (C.2)$$

in which, t = time

γ_w = unit weight of water

u = excess pore pressure

z = coordinate of flow direction

For normally consolidated soils whose e-log $\bar{\sigma}$ plot can approximated by a straight line, the void ratio can be expressed as

$$e = - C_c \log \frac{\bar{\sigma}}{\bar{\sigma}_1} + e_1 \qquad (C.3)$$

in which C_c = compression index

e_1 = the value of void ratio when the effective stress is $\bar{\sigma}_1$

It is convenient to choose $\bar{\sigma}_1$ = unity, even if the straight line has to be projected backward to reach this stress. Substituting $\bar{\sigma} = \sigma - u$ into Eq. C.3 in which σ = total applied stress to the soil and u = pore water pressure, and differentiating with respect to u,

$$\frac{\partial e}{\partial t} = \frac{C_c}{2.303(\sigma - u)} \qquad (C.4)$$

Since the total stress, $\Delta\sigma$, is constant

$$\frac{\partial e}{\partial t} = \frac{\partial e}{\partial u} \frac{\partial u}{\partial t} \tag{C.5}$$

Substituting $\frac{\partial e}{\partial t}$ from Eq. C.5 and then $\frac{\partial e}{\partial u}$ from Eq.C.4

$$\frac{\partial u}{\partial t} = \frac{2.303\ k(1+e)}{\gamma_w\ C_c} (\sigma - u) \frac{\partial^2 u}{\partial z^2} \tag{C.6}$$

As consolidation progresses k and e also vary. Therefore substituting $k = C\frac{e^n}{1+e}$ and e from Eq. C.3 into Eq. C.6 can relate these to effective stress. Upon simplification

$$\frac{\partial u}{\partial t} = \frac{2.303\,C}{\gamma_w C_c} \left[- C_c \log \frac{(\sigma - u)}{\sigma_1} + e_1\right]^n (\sigma - u) \frac{\partial^2 u}{\partial z^2} \tag{C.7}$$

Eq. C.7 is the governing differential equation of consolidation valid for the range of $\bar{\sigma}$ in which the e-log $\bar{\sigma}$ plot approximates a straight line. Terzaghi assumed the multipher of $\frac{\partial^2 u}{\partial z^2}$ on the right hand side of Eq.C.7 to remain constant for a single stress increment and termed it the coefficient of consolidation, C_v. It is a measure of the rate of settlement of a soil layer. Thus

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \tag{C.8}$$

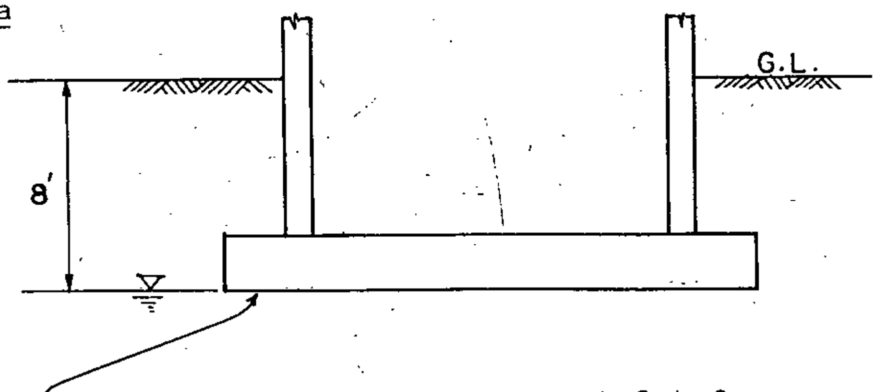
Comparison of Eqs. C.7 and C.8 shows that in effect

$$C_v = \frac{2.303C}{\gamma_w\ C_c} (-C_c \log \frac{\bar{\sigma}}{\bar{\sigma}_1} + e_1)^n \bar{\sigma} \tag{C.9}$$

APPENDIX-D

COMPUTATION OF SETTLEMENT

General Data



D.L + permanent LL at base = 1.2 tsf

Soil-I extended upto infinite depth

LL = 40, PI = 20

γ_{sat} = 125 psf

C_c = 0.28

Computations for values of $\bar{\sigma}$:

$\frac{8\times125+5(125-62.5)}{2000}$ = 0.656 tsf at 5 ft below the base of raft

$\frac{8\times125+15(125-62.5)}{2000}$ = 0.9687 tsf at 15 ft below the base of raft

$\frac{8\times125+25(125-62.5)}{2000}$ = 1.28 tsf at 25 ft below the base of raft

$\frac{8\times125+35(125-62.5)}{2000}$ = 1.59 tsf at 35 ft below the base of raft

$\frac{8\times125+45(125-62.5)}{2000}$ = 1.9 tsf at 45 ft below the base of raft

$\frac{8\times125+55(125-62.5)}{2000}$ = 2.218 tsf at 55 ft below the base of raft

$$\frac{8x125+65(125-62.5)}{2000} = 2.53 \text{ tsf at 65 ft below the base of raft}$$

$$\frac{8x125+75(125-62.5)}{2000} = 2.84 \text{ tsf at 75 ft below the base of raft}$$

$$\frac{8x125+85(125-62.5)}{2000} = 3.156 \text{ tsf at 85 ft below the base of raft}$$

Computations of $\Delta\sigma$ and e_o

Values of $\Delta\sigma$ at mid-depth of each layer have been calculated by multiplying the net pressure at the base of raft by the corresponding influence coefficient found from Fadum (1948) chart.

Values e_o at mid-depth of each layer have been found from e-log $\bar{\sigma}$ plot of Soil-I shown in Fig. 5.3. Values of $\Delta\sigma$ and e_o at mid-depth of different layers are tabulated below:

Layer	Depth below the base of raft in ft	$\Delta\sigma$ in tsf	e_o
1	5	0.70	0.87
2	15	0.683	0.835
3	25	0.644	0.80
4	35	0.537	0.78
5	45	0.518	0.76
6	55	0.453	0.745
7	65	0.378	0.73
8	75	0.336	0.715
9	85	0.28	0.7

Settlement Computation

Settlements at the mid-depth of each layer (10' thick) are shown below in a tabular form:

Layer	$\frac{\Delta\sigma}{\bar{\sigma}}$	$S_t = \frac{H_t C_c}{1+e_o}(1+\frac{\Delta\sigma}{\bar{\sigma}})$	$S_t = \frac{\Delta\sigma}{14.47\bar{\sigma}} H_t$
1	1.0.66	5.66"	8.45"
2	0.704	4.238"	5.848"
3	0.503	3.304"	4.172"
4	0.337	2.38"	2.79"
5	0.27	1.99"	2.25"
6	0.204	1.55"	1.69"
7	0.149	1.17"	1.238"
8	0.118	0.95"	0.98"
9	0.088	0.729"	0.735"
		Total=21.97"	Total=28.153"