EXPERIMENTAL INVESTIGATION OF PREFABRICATED VERTICAL JUTE DRAIN IN SOFT SOIL

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(Md. Wahidul Islam)

DEDICATION

Dedicated to my parents & wife

CONTENTS

NOTATIONS

AOS = Apparent opening size (mm or micron)

b = Width of the band drain or jute PVD (mm), radius of the circular loaded area (m)

C_c = Compression index

c_u = Consolidated undrained shear strength (kPa)

C_v = Co-efficient of consolidation for one-way drainage (mm^2/min)

C_h/C_{vr} = Coefficient of consolidation for radial drainage (mm^2/min)

cc = Cubic centimeter

D = Influence diameter of sand drain, (m)

d_e or d = Equivalent diameter of the PVD, ($d = 2(b-t)/\pi$), (mm)

Δh = Hydraulic head difference (mm)

Δp = Applied surcharge/stress increase (kPa)

E = Young's modulus of soil (ton/ft^2)

e = Void ratio

e_0 = Initial void ratio

G_s = Specific gravity of soil

H_c = Thickness of consolidating clay layer

i = Hydraulic gradient

Ip = Plasticity index

k = Coefficient of permeability

k_h = Coefficient of horizontal permeability

km = Kilometer

kPa = Kilo-Pascal

LL = Liquid limit

ml = Milliliter

mm = Millimeter

m = Metre

n_s = Porosity of sand drain

O_{95} = 95% Finer than specific sieve/geotextile opening size (mm)

p = Vertical stress in consolidation test (kPa)

PI = Plasticity index

PL = Plastic limit

q = Flow rate (m^3/s), load per unit area (kPa)

q_u = Unconfined compressive strength (kPa)

r_e = Equivalent radius of vertical drains (mm)

S = Spacing of vertical drain

S_c = Consolidation settlement (mm)

S_e = Immediate or elastic settlement (mm)

S_s = Secondary consolidation settlement (mm)

S_u = Undrained shear strength (kPa)

t = Time (min, sec)

t = Thickness of geotextile/natural textile (mm)

tsf = Ton per square foot

T_v = Time factor in one-way drainage (Vertical direction)

T_r = Time factor for radial drainage

U = Degree of consolidation for both vertical and radial drainage

U_v = Degree of consolidation for vertical drainage

U_r = Degree of consolidation for radial drainage

v = Velocity of liquid flow (mm/sec)

w_n = Natural water content

γ_w = Unit weight of water (9.81 kN/m^3)

γ_s = Unit weight of soil (kN/m^3)

ψ = Permittivity (Cross-plane permeability) (s^{-1})

θ = Transmissivity (In-plane permeability) (m^3/s-m)

ν = Poisson's ratio

ε = Strain in the unconfined compression testing

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ABSTRACT

In the present study, efforts were taken to investigate the feasibility of using prefabricated vertical drain (PVD) made of natural fibre like jute in accelerating the consolidation process of soft soil under surcharge. To do it, soft soil parameters consisting of index properties, shear strength properties and consolidation parameters of Mirpur Clay have been investigated following ASTM standards. Sample of jute PVD has been collected from Bangladesh Jute Mills Corporation (BJMC). Besides, DW Twill, a type of jute product that is used mainly for making sack which is also used as filter fabric of jute PVD has also been collected. Mechanical properties of filter fabric have been adopted from the work of Mohy (2005); whereas determinations of hydraulic conductivity of filter fabric as well as of the PVD system as a whole have been undertaken in this research.

Recently, synthetic PVD such as 'Flexi Drain' has been used in the Kaliakoir Bypass Road Project in Bangladesh. The Roads and Highways Department (RHD) of Bangladesh being the sponsor department, has set the specifications for the PVD. It is seen from the test results that jute PVD clearly fulfill most of the specifications set for synthetic PVD. However, transmissivity (in-plane permeability) of the jute PVD sample lacks marginally to achieve required discharge rate at hydraulic gradient of 1.0.

In this study, a cylindrical type steel tank of 930 mm height and 502 mm diameter has been fabricated to investigate the in-soil performance of synthetic PVD and jute PVD. In the first phase, settlement under fixed surcharge has been observed for 4 weeks and time-settlement profile has been compared with that of theoretically (Terzaghi one dimensional) achievable with oedometer test parameters. In the second phase, 'Flexi Drain' was used to run the same set-up of test. This synthetic PVD fulfils all the specifications of Roads and Highways Department. Settlement that took place in this phase was significantly more than that occurred in the first phase.

In the third phase, jute PVD has been used in-soil in the steel tank with same characteristic of soil slurry and observed for 4 weeks. Laboratory test result showed that the performance of jute PVD was similar to synthetic PVD. Cost feasibility analysis shows that synthetic PVD is two times costlier than jute PVD. In addition to its environmental protection qualities, significant amount of cost can be reduced in any project that encounters the challenge of modifying the soft soils by using jute PVD in place of synthetic PVD.

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Soft clays present a number of special problems during and post construction period of any civil engineering work. Such soils have very low shear strength and poses high compressibility, as it mainly comprises of under-consolidated to normally consolidated clays. Due to high compressibility, the foundation on such soils suffers very large amount of settlement. Again, low shear strength of soft soils results in low bearing capacity of foundation. However, excessive settlement of structures that continues for years together is the most pronounced out of all problems. Though cases of soil settlement problem are very common in the world, some of the most dramatic examples of soil settlement are found in Mexico City. Parts of the city are underlain by one of the most troublesome soils in any urban area of the world. Part of Shrine of Lady Guadalupe in Mexico City was built in about 1709 and rest part was constructed in 1622. There existed a ridge of rock beneath the junction of two parts that allowed little settlement, whereas adjacent portions located above deposits of highly compressible clay as much as 60 feet thick, have settled more than 7 feet. The Tower of Pisa in Italy is another example of excessive settlement. In this case, one side has settled more than the other, a behaviour which is called 'differential settlement', which also gave the tower its famous tilt, Coduto (2007). In those days, people did actually have little idea regarding consolidation of clay and used to consider this type of post construction settlement to be of mysterious origin, Peck et al (1973). Settlement problems are not limited to buildings only. This phenomenon equally takes place in bridges, highways, embankments supporting road structure (sub-base, base, pavements) etc if those are underlain by soft clay deposits.

1.2 IDENTIFICATION OF SOFT SOILS

Generally, the term 'soft soil' includes soft clay soils, soils with large fractions of fine particles such as silts, clay soils which contain high moisture content, peat foundations and loose sand deposits near or under the water table, Bergado et al (1996). According to ASTM soil classification system, soil particles smaller than #200 sieve (0.075 mm) constitute fine grained soils. Fine grained soils are again composed of silts (0.075 mm to 0.002 mm) and clays (<0.002 mm). Clays are the smallest particles and are predominantly an aggregate of microscopic and submicroscopic flake-shaped crystalline minerals. The consistency of fine grained soils is expressed qualitatively by the terms such as soft, medium, stiff and hard. This differentiation is not very clear and absolute. Quantitatively soft soil can be identified by its undrained shear strength (S_u), or by its unconfined compression strength (q_u). Soft clays are characterized by high plasticity, high liquid limit, high natural water content and low shear strength. Soils with shear strength less than 25 kPa refer to 'soft soil' whereas less than 12 kPa refers to 'very soft soil', GoI (2005). On the other hand, the SPT N- values are used to ascertain the consistency of the ground and its relative density. According to this identification system, SPT N value less than 2 indicates a 'very soft soil, whereas, the values of 2 to 4 indicate a 'soft consistency' of clay, Bergado et al (1996).

1.3 CHRONOLOGY OF SOFT SOIL IMPROVEMENT TECHNIQUES

Soft soils and problems associated with these soils had been and are still important topics of research to many engineers. Problems related to settlement mainly take place due to reduction of void spaces. In the saturated clay soil, this void is nothing but the pore water. In 1809. the British engineer Thomas Telfrod placed a 17 m deep surcharge fill over a soft clay layer for the purpose of squeezing out the water and consolidating the mud with a view to recognizing qualitative flow of pore water. American engineer William SooySmith in 1892 also recognized that "slow progressive settlements result from the squeezing out of the water from the earth". The first laboratory test to understand the consolidation process was undertaken by J. Fronthard in France in 1914. Although these tests provided some insight. the underlying processes were not yet understood. About the same time. one German engineer Forchheimer developed a crude mathematical model of consolidation. but it was not accurate and did not recognize important aspects of the problem. It was Karl Terzaghi. also a student of Forchheimer. made the major breakthrough in 1919. While he was a teacher of thermodynamics in Istanbul. he conducted his famous consolidation works between 1919 and 1923. which led the way to understanding the consolidation process. Terzaghi first put forward his studies on a scientific basis and his theory of consolidation is recognized as one of the major milestones of geotechnical engineering, Coduto (2007). Terzaghi developed his theory for the time rate of one-dimensional consolidation basing on number of assumptions. One important assumption was 'deformation of soil occurs only in the direction of the load application'. It actually means that consolidation takes place only in the vertical direction because loads are usually applied vertically. So, one method of soft soil improvement technique was developed in the form of preloading or application of surcharge. Since coefficient of permeability in the vertical direction is few times smaller than that in the horizontal direction, pore pressure dissipation becomes slower and continues for longer period. If the thickness of clay layer is more than the width of the clay formation then pore water needs to travel a longer path. This preloading technique, though worked, researches were still going on how to leverage the better coefficient of horizontal permeability. In 1934, first vertical drainage path in the form of sand drain was installed in California, USA. Barron in 1948 put forward the scientific analysis of vertical drains that are artificially created drainage paths which could be installed by one of several methods and which can have a variety of physical characteristics. Installation of vertical drains in the clay reduces the length of the drainage paths and, therefore reduces the time to complete the consolidation process. Sand drains have some inherent drawbacks like smear effect, well resistance and to some extent piling effect that prohibits settlement in that particular drain area. Applications of sand drains for the improvement of soft soil started in East Asia and Southeast Asia by 1950s. Use of sand drains have been reported in Nagasaki, Japan in 1951 by Aboshi (1993), by Choa et al. (1979) in Changi Airport. Singapore. by Chou et al. (1980) in Taiwan, by Agaki(1981) and by Balasubramaniam et al. (1980) in Bangkok, Bergado et al (1996). Few researches had also been conducted in Bangladesh about vertical drains especially sand drains during 1980s and 1990s. Safiullah et al. (1993). Ansari et al. (1993). Kabir et al. (1994) and some other researchers have carried out investigations on vertical drains in Bangladesh. Various Government organizations especially Roads & Highways Department have included the soft soil improvement technique in their specifications through the use of vertical drains. In the Kaliakoir Bypass road project. prefabricated vertical drains have been used successfully.

In the initial years of vertical drains applications. it only used to mean sand drains. Gradually. smaller diameter band-shaped cardboard wickdrains were used by Kjellman

(1948). Revolution took place in this field when PVDs made of geosynthetics were introduced. Synthetic materials dominated this field because of its special characteristics like high strength, low specific gravity, good resilience, chemical inertness and resistance to moth and bacterial growth, Mohy (2005). It reduced the size, shape, and cost and made the installation procedure easier. In Bangladesh these products are imported from foreign countries. Again from engineering point of view, vertical drain needs to remain useful as a drain over the required period and in most cases a few months and rarely over a year for consolidation process as opposed to permanent drains like synthetic PVDs, Abdullah (2008). To avoid environmental degradation with synthetic PVDs and take the potential of jute to serve the same purpose of synthetic materials, some researchers of Southeast Asia started to investigate the engineering feasibility of jute during last three decades. Generally, their findings are in favour of using jute based products in some of the civil engineering projects. In some cases, jute is comparable or to some extent superior to synthetic materials in physical, chemical and engineering qualities.

1.4 BACKGROUND OF THE STUDY

Roads in Bangladesh are generally constructed over the embankments raised above the highest recorded flood level in order to keep the roads operational even during the worst flooding condition. While constructing embankments over soft soils, problems related to bearing capacity failure, embankment instability, excessive settlement, and long period of time required for consolidation become major issues of consideration. In order to increase the consolidation rate and improve the shear strength and compressibility properties of the subsoils, preloading without vertical drains, preloading with sand drains and preloading with synthetic prefabricated vertical drains are commonly employed, GoB (2000 & 2006). Surcharge for preloading is applied with the embankment core materials. A synthetic geotextile separator is laid at the base, i.e. at the interface between the soft subsoil and the embankment materials (surcharge) in order to avoid mudding effect. The filling of the embankment material progresses in stages as the shear strength and the compressibility properties of the subsoil improve due to expulsion of water through sand drain or synthetic PVD. Due to inherent drawbacks of sand drain, its applications have reduced whereas synthetic PVDs have got wider acceptance to many organizations. For high embankments (>4.0m), a synthetic geogrid reinforcement is provided at 3.0 m height from the base in order to avoid local instability of next stages of embankment fill during construction, GoB (2000 and 2006), GoB (2000a and 2006a). Once, the full height of the embankment is attained, the compacted embankment core material is used as the subgrade for construction of flexible road pavement structure.

Generally, road construction projects in Bangladesh require 2 to 3 years to complete. In these projects, synthetic PVDs used for ground improvement needs to function for 6 months to maximum 2 years, after which it becomes redundant. Vertical drain made of natural fibre capable of serving the same purpose may be a better alternative in this regard. Jute, a natural fibre is produced in abundance in Bangladesh. Natural fibres get degraded when they are exposed to the humid weather. In search of improved serviceability, the U.S. Army has investigated the various preservative treatment of jute since 1950. They also considered making fortifications permanent by chemical treatment of jute/sandbags. Adding up to 10% cement to the sand fill or providing a concrete cap were two options for treatment. Besides, old hessian sandbags were known to last longer if they were dipped in cement-wash before they were filled or if they were painted with a cement-wash after they were installed. Koerner (1997). Jute fabric soaked in bitumen was used in military road construction by the Allied Forces in the Burma Front of Southeast Asia during the Second World War. Kabir et al (1994).

Due to its hydrophilic characteristics, untreated jute products are likely to biodegrade in saturated environment by 2 months to 4 months. To investigate the durability of untreated jute products. Rao G. et al (1994) investigated four types of woven jute fabrics embedded in-soil for periods up to 2.5 months and submerged in normal water up to 4 months and tested for changes in stress-strain behaviour. The durability of jute yarns was also assessed by conducting tests with solutions at values of p^H ranging from 4.5 to 9.0. Their studies revealed that the tensile strength of different fabrics falls to zero with 2.5 months in the biotic environment. However, when jute was fully submerged in water for 4 months, the loss was only 35%. Prodhan (2008) found that untreated jute is swelled and degraded within six months in water but some chemical treatments can increase the life span up to 5-20 years. Life span of jute based products can be extended up to 20 years through different treatments and blending, Abdullah (2008). They investigated the durability of jute based products with various types of chemical treatment and showed that the life of these products could be increased basing on design requirement. Prodhan (1994) called this enhancement of design life of jute products as 'Design Biodegradability'. Following their findings, Bangladesh Jute Research Institute (BJRI) has developed 'designed biodegradable' jute products such as jute geotextiles and jute PVD. In these products developed by BJRI, the inherent drawback of biodegradable characteristics of natural fibre has been tried to reduce to a considerable degree.

So far, few investigations have been conducted to find out the performance of jute PVD. Lee et al. (1989) developed jute PVD with coconut coir enveloped by two layers of jute burlap filter for the countries in the South East Asia and studied the variation of discharge capacity with hydraulic gradient. Kabir et al. (1994) carried out performance tests (ex-soil) on jute wickdrain similar to Lee's Test and found out its discharge capacity. As of now, research findings on in-soil performance of jute PVD are not available. Before using jute PVD in any project, its in-soil behaviour needs to be determined.

1.5 OBJECTIVES OF THE STUDY

To address the issues identified above, this research has been undertaken for the following objectives:

(1) To assess the performance of jute PVD and synthetic PVD in soft soil.

(2) To analyze technical and economic feasibility of jute PVD in comparison to synthetic PVD.

1.6 REAEARCH METHODOLOGY

In order to achieve the objectives as mentioned, the following methodologies will be adopted:

(1) Clay soil from a portion of Zia Colony to Mirpur Cantonment Road Project will be collected. All the index properties of the samples will be determined following standard test methods.

(2) Synthetic PVD with known properties will be collected from a project of Bangladesh where it has been used. Jute PVDs made of coconut coir enveloped by jute burlap will be collected from BJMC. The physical and mechanical properties of these samples will be determined employing ASTM tests standards in the BUET geotechnical laboratory.

(3) A cylindrical steel tank of diameter 502 mm and height 930 mm will be fabricated.

(4) Soil slurry will be made of known moisture content. This slurry will be placed in the tank without using jute PVD and settlement with time will be recorded for a given surcharge load. Small portion of the same slurry will be tested in the oedometer for counter checking the settlement result in the tank.

(5) In the same tank, jute PVD and synthetic PVD will be used as vertical drain and time settlement pattern will be assessed to evaluate the performance of both PVDs.

(6) An analysis will be made to determine the economic feasibility of jute PVD in comparison to synthetic PVD.

1.7 THESIS LAYOUT

The complete research work for achieving the stated objectives is divided in number of chapters so that it becomes easier to understand the chronological development of the work. Briefly the contents of each chapter are presented here:

A review on soft soil, its index properties, various correlations for soft soils, soil consistency and consolidation parameters have been discussed in Chapter Two. This chapter also includes the literatures regarding various types of vertical drains. Thereafter, it contains detailed theoretical considerations of sand drains, band/strip drain and PVDs including the effect of spacing of drains, smear and well resistance. Finally, Chapter Two ends with the review of jute/jute products including the research work on durability and engineering properties.

Chapter Three deals with the tests methodologies. Index properties and consolidation parameters of clay soil those were found out following ASTM standards are mentioned in this chapter. This chapter includes the test programme on hydraulic conductivity of jute PVD (Filter and core). The important part of this chapter includes the fabrication of cylindrical steel tank for testing in-soil behaviour of synthetic PVD and jute PVD. In addition to assessing the time- settlement profile of soil slurry under surcharge, vane shear test and unconfined compression tests were also conducted to compare the change in shear strength.

The physical and engineering characteristics of soft soil and their correlation with various parameters have been discussed in Chapter Four. Soil settlement under surcharge without drains, with synthetic PVD and with jute PVD along with change in water content and shear strength has been analyzed with graphs and charts. These settlements also have been compared with theoretically calculated settlement with consolidation parameters (e_0, C_c and C_v) obtained from oedometer tests. Economic analysis in terms of cost comparison with synthetic PVD is included in this chapter.

Chapter 5 presents the major findings and conclusions of the present investigation. Recommendations basing on the present research and also recommendations for further research in this field are also presented in this chapter.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Suitable land for construction of embankments and other infrastructures is becoming scarce day by day. Sometimes construction projects such as embankments, bridges, highways etc are undertaken at sites which are underlain by very soft compressible deposits. Dissipation of excess pore water pressure from these soils under surcharge takes place gradually at a rate controlled by the consolidation parameters of soils and magnitude of surcharge. Since in the traditional preloading method, dissipation takes place only in the vertical direction, in most of the cases, time required to achieve a desired amount of settlement (say 80% to 90%) for any civil engineering work becomes excessive. There are number of techniques that are used to accelerate the consolidation rate in soft soils. Using one of the important techniques i.e. vertical drains. pore water dissipation can be expedited which in turn reduces the compressibility and increases the strength and stiffness properties of soft soils. This improvement is due to reduction in the void spaces. These vertical drains are already available in many forms such as sand drains, wickdrains, prefabricated vertical drains (PVD) etc. PVDs available nowadays are made of synthetic fibres and are imported in Bangladesh from foreign countries. Jute, a natural fibre which is abundant in Bangladesh has the potential to be used in the products that can be employed in place of synthetic prefabricated vertical drains. This review looks into the fundamental concepts of soft soil consolidation characteristics and theories, enhancement of pore water dissipation though simultaneous use of vertical and radial drainage along with mathematical solutions, discussions on requisite specifications of PVD and analysis of potential properties of jute prefabricated vertical drains (Jute PVD).

2.2 CHARACTERIZATION OF SOFT SOIL

Soft soils are generally characterized by low shear strength, high compressibility and large settlement under surcharge or structural load. The consistency of natural cohesive soil deposits is expressed qualitatively by terms such as soft, medium, stiff and hard. However, the meaning of these terms varies widely in different parts of the world depending on whether the local soils are generally hard or generally soft. Soft clays are highly plastic fine-grained soils with moderate to high clay fraction. Soft clays have following typical characteristics, GoI (2005):

a) Predominantly fine-grained i.e. more than 50% of soil passes through #200 sieve.
b) High liquid limit(LL) and plastic limit(PL)
c) High natural water content (w_n, normally more than 34%) and even higher than the LL.
d) Low material permeability.
e) Undrained shear strength usually less than 25 kPa.
f) High compressibility.

2.2.1 Occurrence of Soft Soils

Soft soils are generally recent sediments laid down by rivers, tributaries, lakes and other sediment-carrying water flow in the flood basin. These deposits are characterized by bedding and laminations interrelated with sand or silt seams and are usually subject to repeated desiccation and wetting near the surface. Generally, soft soils exist in the following environments, GoI (2005):

a) In low land areas near coasts where marine sediments are often found.
b) In the vicinity of rivers, especially those which have been subjected to meandering
c) In local depression where the runoff is restricted and the soil contains appreciable amount of organic matter.

In the Indian Sub-Continent, major proportions of soft clays are marine and river delta deposits and are of Pleistocene to recent origin. The colour of these deposits is somewhat blue black, blackish or blue grey. These soft clays are generally lightly over-consolidated with natural water content in the majority of cases greater than 34%. GoI (2005).

Occurrence of soft soils at various depths and at a varying layer thickness is very common in most parts of Bangladesh. These soft clay deposits are mainly available in the alluvial flood plain deposits, depression deposits and estuarine and tidal plain deposits in the country, Taiyab (2005). Most of her area is extremely flat. Bangladesh is situated in the Bengal Deltaic Basin floored primarily with quaternary sediments deposited by the Ganges-Padma, the Brahmaputra-Jamuna and the Meghna river systems and their tributaries and distributaries. Hunt (1976) has grouped soils of Bangladesh into six groups namely, (1) Hill soils, (2) Raised alluvial terrace deposits, (3) The Himalayan piedmont deposits, (4) Alluvial flood plain deposits, (5) Depression deposits and (6) Estuarine and tidal flood plain deposits, Ansari et al (1993). In a study carried out by Serajuddin et al (1993) of the upper strata of about 2m to 3m in 12 greater districts of Bangladesh showed that about 91 percent of tested soil samples are silty and clayey soils and falls under A-4, A-6 and A-7 soil sub-groups according to AASHTO soil classification system. Majority of the soil samples have low to medium plasticity with soil particles 51 to 100 percent finer than # 200 sieve (0.075 mm). These vast alluvial silty and clayey deposits are highly compressible and are more pervious in the direction of bedding plane than perpendicular to it, Safiullah et al (1993).

2.2.2 Consistency of Soft Soil

One of the most significant index properties of fine-grained soils in the natural state is the consistency. In 1911, Atterberg, a Swedish agriculture engineer mentioned that a fine- grained soil can exist in four states namely, liquid, plastic, semi-solid and solid state. The water contents at which the soil changes from one state to the other are known as consistency limits or Atterberg limits. Quantitatively the consistency of an undisturbed cohesive soil may be expressed by the undrained shear strength, S_u or unconfined compressive strength, q_u. On the other hand, in absence of unconfined compressive strength (q_u), a rough estimate can be based on the SPT 'N' value for finding out consistency of soil. It is usually seen that unconfined compressive strength and SPT 'N' value follows a correlation such as $q_u = \frac{N}{8}$, where q_u is in ton per square foot (tsf). Table 2.1 shows qualitative and quantitative expressions for consistency of clays.

Table 2.1: Qualitative and Quantitative Expressions for Consistency of Clays

Serial No	Consistency	Field Identification	SPT 'N' Values	Unconfined Compressive Strength, q_u (tons/ft^2)
1.	Very soft	Easily penetrated several inches by fist	Less than 2	Less than 0.25
2.	Soft	Easily penetrated several inches by thumb.	2 to 4	0.25-0.50
3.	Medium	Can be penetrated several inches by thumb with moderate effort	4 to 8	0.50-1.0
4.	Stiff	Readily indented by thumb but penetrated only with great effort	8 – 16	1.0-2.0
5.	Very stiff	Readily indented by thumbnail	16 – 32	2.0-4.0
6.	Hard	Indented with difficult by thumbnail	> 32	Over 4.0

After Peck et al, (1973) and Bergado et al. (1996)

2.3 PROBLEMS ASSOCIATED WITH SOFT SOILS

The settlement of structures above beds of soft clay, sometimes buried deeply beneath stronger and less compressible materials, may take place slowly and reach large magnitudes. Because of the time lag between the end of construction and the appearance of cracking, such settlements were once considered to be miraculous. The first successful efforts to explain the phenomenon on a scientific basis were made by Terzaghi in 1919. Peck et al. (1973). The low shear strength of soil results in low bearing capacity of foundation. and due to high compressibility. the foundation on such soils suffers very large amount of settlement. Clay with low shear strength is not strong enough to support the most common structures with conventional shallow foundation systems. and therefore. pose a severe problem for economic design of foundation on the soft deposits. Another problem associated with the structures constructed over this soft soil is, severe serviceability problem due to large settlement related to the compressible nature of the soft subsoil. Usually, embankments are constructed to provide a raised and stable subgrade for road structures. Often the alignment of this embankment goes through cross-country directions where it encounters marshy and low lying areas. In Bangladesh, soft clay layers exist in these alignments at different depths. The embankments constructed traditionally over these alignment fail due to large settlements that take place over a long period of time. In Bangladesh, extreme settlements have been noticed in many roads constructed over this type of embankments.

2.4 SETTLEMENT IN SOFT SOILS

Clays are nearly impervious because of attraction of dipolar water molecule to the negatively charged faces of clay minerals. So, water actually remains trapped in the pores. When an increment of load is applied, this pore water can not escape. Since the clay particles tend to squeeze together, pressure develops in the pore water. This pressure tends to make the water flow out. The flow is rapid at first, but as it continues, pressures drops and the rate of flow decreases. As the water is forced out of a clay sample, the particles can move close together. So, the surface of the clay specimens settles. Analysis of settlement in soft soils demands basically two important parameters to be known. These are (1) amount of settlement that will occur in a particular clay layer and (2) total time required to occur the most amount of settlement to occur. When a foundation or embankment is constructed over soft soil, the following three components of settlement take place:

$$S_t = S_e + S_c + S_s \qquad (2.1)$$

Where, S_t = Total settlement

S_e = Immediate settlement or elastic settlement

S_c = Primary settlement or consolidation settlement

S_s = Secondary settlement

2.4.1 Immediate Settlement

This is also commonly known as elastic settlement or initial undrained settlement. This takes place on account of shear strains which occur instantaneously following the application of load. If the clay is saturated, settlements take place at constant volume caused by shear strains beneath the loaded area. Immediate settlement though, is predominant in granular soils, in saturated clays it is of little significance. In this research, immediate settlement will not be considered. However, elastic settlement due to a uniformly loaded area at a depth of z can be given by:

$$S_e = q\frac{1+\nu}{E}b\left[\frac{z}{b}I_1 + (1-\nu)I_2\right] \tag{2.2}$$

Where, q = Load per unit area (t/m^2)

b = Radius of the circular loaded area (m)

v = Poisson's ratio

E = Modulus of elasticity of the soil solid (t/m^2)

I_1= Non dimensional function of z/b

I_2 = Influence number and a function of s/b

s = Distance of a point from the centre of the load (m)

2.4.2 Primary Settlement

This is also known as consolidation settlement. This is a time dependent process and continues for years after years. When load is applied on clay layer, hydraulic gradient is set up and gradually pore water pressure is transferred to the soil skeleton which undergoes volume change. Primary settlement in one dimension can be determined using the famous one dimensional consolidation theory of Terzaghi. To calculate total primary settlement and time-settlement profile, consolidation parameters such as coefficient of consolidation (C_v), compression index (C_c) and initial void ratio (e_o) etc need to be determined from oedometer test. For the calculation of settlement in this research, primary consolidation settlement will be considered. The mathematical computation procedures for primary consolidation settlement will be described in detail in the following sections.

2.4.3 Secondary Settlement

Clays continue to settle under sustained loading at the end of primary consolidation. This happens due to the continued readjustment of clay particles. This is also commonly referred to as 'drained creep' or 'time-dependent creep of the soil skeleton' and main part takes place essentially after dissipation of excess pore water pressure. There is at present no general agreement on how to separate consolidation into its primary and secondary components. In this research, secondary consolidation settlement will not be taken into consideration. However, to calculate secondary settlement we need to define coefficient of secondary consolidation as, Das (1985):

$$C_{\alpha} = \frac{\Delta H_t}{H_t} * \frac{1}{\Delta \log t} \tag{2.3}$$

Where, C_{α} = Coefficient of secondary consolidation

H_t = Thickness of the clay layer

t = time

It has been reasonably established that C_{α} decreases with time in a logarithmic manner and is directly proportional to the total thickness of the clay layer at the beginning of secondary consolidation. Thus, secondary consolidation settlement can be given by:

$$S_s = C_{\alpha} H_{ts} \log \frac{t}{t_p} \tag{2.4}$$

Where, H_{ts} = Thickness of clay layer at the beginning of secondary consolidation

$= H_t - S_c$

t = Time at which secondary compression is required

t_p = Time at the end of primary consolidation

2.4.4 Fundamentals of Consolidation

When a soil with low coefficient of permeability is subjected to rapid loading, there will be insufficient time for drainage of pore water. This will lead to an increase in excess hydrostatic pressure. On the other hand, if a load is applied slowly on a soil such that sufficient time is allowed for pore water to drain out, there will be practically no increase of pore water pressure. When a saturated soil element is subjected to an instant isotropic stress increase of magnitude $\Delta\sigma$, this total increase in stress will be taken by pore water i.e. pore pressure will also increase by Δu. In that case $\Delta\sigma$ will be equal to Δu. After application of surcharge (i.e. at time t>0), the water in the void spaces of the clay layer will be squeezed out and will flow toward permeable layers and pore water pressure will be reduced. This will in turn increase the effective stress. According to Terzaghi (1943), "a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation". Theoretically, at time $t = \infty$, the excess pore pressure will be dissipated by gradual drainage and effective stress will be increased by the same amount of surcharge. So, the "gradual process of increase of effective stress in the clay layer due to the surcharge will result in a settlement which is time-dependent and is referred to as the process of consolidation".

2.4.4.1 Theory of One Dimensional Consolidation

Dissipation of excess pore water pressure takes place toward the permeable layers. Geologically, clay layers are often bounded by permeable sand layer(s) at top and/or bottom. In the radial direction, if clay layer is considered to be infinitely long, flow of pore water is not significant. So, drainage is allowed in the top or bottom that is only in the vertical direction. The theory for the time rate of one-dimensional consolidation was first proposed by Terzaghi (1925). There were some underlying assumptions in the derivation of mathematical equations by Terzaghi which are as follows:

a) The clay layer is homogenous.
b) The clay layer is saturated.
c) The compression of the soil layer is due to the change in volume only, which, in turn, is due to the squeezing out of water from the void spaces.
d) Darcy's Law is valid ($v = ki$, v = discharge velocity, i = hydraulic gradient and k = coefficient of permeability).
e) Deformation of soil occurs only in the direction of the load application.
f) The coefficient of consolidation C_v is constant during the consolidation.

Usually, these assumptions are generally valid for clay formations which have widths greater than the thickness. Chin et al (2000). If the clay layer has the thickness more than the width, horizontal (radial) flow of pore water may be more significant and one dimensional theory tends to underestimate the rate of consolidation.

Terzaghi formulated an equation for one dimensional consolidation and also solved that equation. The basic differential equation of Terzaghi's one dimensional consolidation is,

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \qquad (2.5)$$

Where, u = Pore water pressure,

z = Clay layer thickness in the direction to which drainage takes place.

The solution of this equation is

$$U_{av} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \exp(-M^2 T_v) \tag{2.6}$$

Where, U_{av}= Average degree of consolidation

$$M = (2m + 1)\frac{\pi}{2}$$

m is an integer.

T_v is known as non-dimensional time factor which may be found out from oedometer test in the laboratory.

$$T_V = \frac{C_V t}{H^2} \tag{2.7}$$

Where, C_v = coefficient of consolidation (mm^2/min or m^2/year). C_v is not a soil constant but decreases with increasing vertical stress. Particularly around the preconsolidation pressure, C_v could drop sharply, say by a factor of one-half, although no general rule can be given. Soil disturbance or remolding (more relevant if vertical drains are installed) can similarly lower the C_v value, Hausmann (1990).

H = length of the longest drainage path. If the clay layer is bounded by permeable layers at both top and bottom, then it will be considered to have two-way drainage and H will be half of the total thickness of the clay layer. If clay layer is overlain or underlain by permeable layer in one side only, then H is taken as full thickness of the clay layer. Terzaghi suggested the following equations for U_{av} to approximate the values obtained from Equation (2.6)

For U_{av}= 0 to 53%: $$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \tag{2.8}$$

For U_{av}= 53% to 100% $$T_v = 1.781 - 0.933[\log(100 - U\%)] \tag{2.9}$$

From Equation (2.7), it is clear that if we know the duration of load (t), coefficient of consolidation (C_v) and thickness of clay layer (H), then time factor (T_v) can be calculated. This T_v can be used in Equations (2.8) and (2.9) to have a variation of average degree of consolidation with time. The variation of T_v with U_{av} [Eq. (2.6), (2.8) and (2.9)] is shown in Table 2.2 and Figure 2.1.

Table 2.2: Variation of T_v with U_{av} [Eq. (2.6), (2.8) and (2.9)]

U_{av}%	T_v	U_{av}%	T_v
0	0	60	0.287
10	0.008	65	0.342
20	0.031	70	0.403
30	0.071	75	0.478
35	0.096	80	0.567
40	0.126	85	0.684
45	0.159	90	0.848
50	0.197	95	1.127
55	0.238	100	---

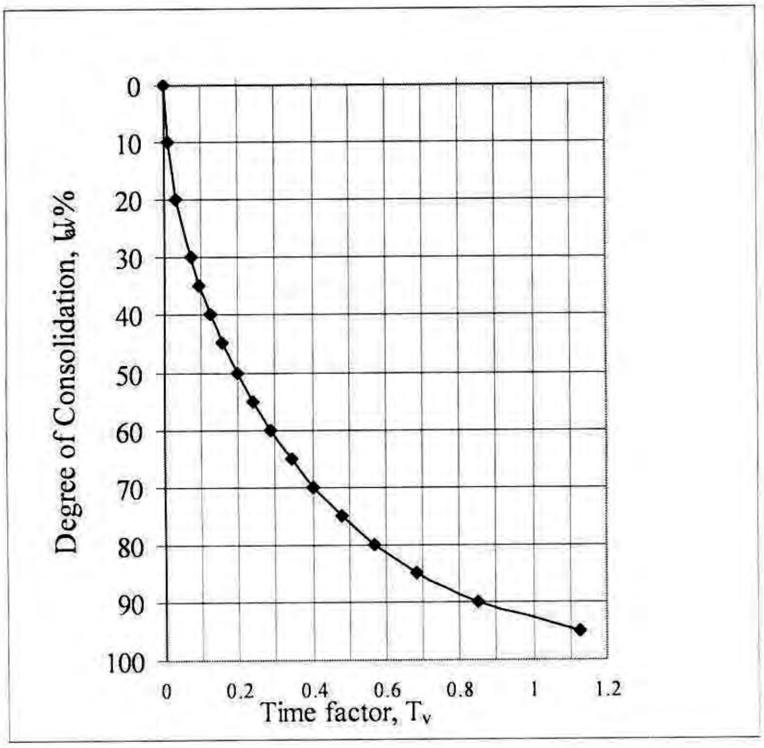

Figure 2.1: Variation of Average Degree of Consolidation with Time Factor

2.4.4.2 Settlement Computation from One Dimensional Consolidation

Settlement of the subsoil supporting the embankment will take place during and after the surcharge filling. It is necessary to evaluate both the magnitude and rate of settlement of the subsoil when designing the embankment so that the settlement in the long term does not influence the serviceability of the embankment. If a clay layer of total thickness H is subjected to an increase of average effective overburden pressure from p_o' to $(p_o' + \Delta p)$, it will undergo a consolidation settlement of ΔH. Hence the strain can be given by:

$$\varepsilon = \frac{\Delta \mathrm{H}}{\mathrm{H}} \tag{2.10}$$

Where, ε is the unit strain. Again, let an element of an undisturbed specimen be assumed to consist of soil solid with a height (H_s) equal to unity and a void space with an additional height equivalent to e_o. The total height of the element will therefore be $(1+ e_o)$. If that specimen is subjected to the same effective stress increase, the void ratio will decrease by Δe. Thus, the strain is equal to

$$\varepsilon = \frac{\Delta \mathrm{e}}{1+\mathrm{e}_0} \tag{2.11}$$

Where, e_o is the void ratio at an effective stress of p_o' or in other word e_o is called initial void ratio.

So, from Eqs. (2.10) and (2.11)

$$\Delta H = \frac{\Delta \mathrm{e}}{1+\mathrm{e}_0} H \tag{2.12}$$

For a normally consolidated clay

$$\Delta \mathrm{e} = \mathrm{C_c} \log_{10}\left(\frac{p_0' - \Delta p}{p_0'}\right) \tag{2.13}$$

C_c is the compression index and it can be found from oedometer test in the laboratory. It actually denotes the compressibility of soil. p_0' is the effective overburden pressure at mid- depth of the clay layer.

$$p_0' = (\gamma - \gamma_w)\frac{H}{2} \tag{2.14}$$

Where, γ = unit weight of moist soil and

γ_w = unit weight of water (9.81 KN/m^3)

But from Eqs. (2.12) and (2.13) consolidation settlement ΔH or in other notation S_c can be given by:

$$S_c = \frac{C_c H}{1+e_0} \log_{10}\left(\frac{p'_c + \Delta p}{p'_c}\right) \tag{2.15}$$

2.5 OPTIONS FOR ADDRESSING THE PROBLEMS OF SOFT SOILS

Generally, alignment of an embankment project passes through well established habitation area to indomitable low lying and difficult terrain. Situations like constructing structures such as industrial parks, multi-storied buildings, bridges and other civil engineering works at locations of soft ground are also common. Often engineers need to estimate economic and structural feasibility before finalizing the project. When engineers are faced with difficult soft soil before the construction of structures, following options are open to them:

a) Avoid the particular site: Alignment is adjusted or relocated so that areas containing difficult and problematic soils can be avoided. This option entails extra cost to the project.
b) Remove and replace unsuitable soils: Removal and replacement is one of the oldest and simplest methods of soil improvement. Where soft and compressible soils exist. soil layer up to certain height is removed and replaced by good quality borrowed material. This is a usual practice for constructing highway embankment in Bangladesh where soft and organic top soils are replaced by imported sand or good quality dredged materials.
c) Design the planned structure accordingly: Structures are designed in a way in which extra height is considered beforehand to accommodate subsequent settlement. Construction is also done in stages to address the time dependent settlement. Construction period stretches even more than 2 to 3 years. This method is usually adopted in Bangladesh for the construction of embankment.
d) Attempt to modify the existing ground: This option calls for modifying in-situ soils by various ground improvement techniques. Though mechanical modifications like static or vibratory rollers. impact roller or plate vibrators

and deep compaction by heavy tamping are used for cohesionless soils, fine grained soils are modified by chemical treatment, preloading, vertical drains etc. Admixtures are used to chemically modify the existing soils. Moreover, soil can be modified by reinforcement such as strips, bars, fibres, meshes etc.

Similar options must be considered in the case where there is a lack of good-quality granular materials needed for the construction of dams, embankments, roads, or foundations. As more and more land becomes subject to urban and industrial development, good construction sites and borrow-areas are difficult to find and the alternative for improving the in-situ soil becomes the best option.

2.6 MODIFICATION TECHNIQUES FOR SOFT SOILS

The ground modification techniques can be grouped under main four headings, Hausmann (1990) and Ansari (1993):

a) Mechanical modification technique
   i. Blasting
   ii. Vibro-compaction
   iii. Vibro-replacement
   iv. Composer system
   v. Other casing drivers.
b) Hydraulic modification technique
   i. Preloading
   ii. Preloading with vertical drains
   iii. Vacuum consolidation
   iv. Electro-osmosis
   v. Lowering the ground water table through pumping from boreholes or trenches
c) Physical and chemical modification technique
   i. Modification by admixtures
   ii. Grouting
   iii. Thermal stabilization
d) Modification by inclusions and confinement
   i. Soil nailing

ii. Ground anchors

iii. Strip, bar, mesh and grid-reinforcement

iv. Flexible geosynthetic sheet reinforcement

This classification of ground improvement techniques is to some extent arbitrary, particularly where one or more of the possible physical, chemical, hydraulic, or mechanical processes are combined. All four main modification techniques and their sub-techniques are employed for improving various types of soils. However, in the following sub-sections, techniques and sub-techniques that are particularly meant for modifying soft cohesive soils are described in brief. Preloading with vertical drains under hydraulic modification is described in detail.

2.6.1 Mechanical Modification Technique

This is an in-situ soil improvement technique. In this technique, soil density is increased by the applications of short-term external mechanical forces, including compaction of surface layers by static, vibratory, or impact rollers and plate vibrators and deep compaction by heavy tamping at the surface or vibration at depth. Many of these methods are proved to be cost effective. Out of all the mechanical modification techniques, only vibro-replacement can be employed for modifying soft soils.

2.6.1.1 Vibro-Replacement Technique

This is applicable only when soil is relatively impervious and cohesive having consolidated undrained shear strength (c_u) in the range of 15 to 50 kPa, Ansari (1993). These soils are readily penetrated with low pressure large volume bottom jets and the displaced materials get transported with the water flow to the surface. On reaching the desired depth, a gravel backfill is tipped around the machine to fall down the annulus against continuing up-flow from the bottom jet. As gravel accumulates at the base of the column, this motion together with vibration tends to ram it into the sides of the bore. Resistance encountered as the machine sinks at each level indicates completion of the column to a diameter depending on the soil resistance, shearing and flushing action. The process is self-compensating in that: diameters are wider in softer strata. The columns are normally about 0.8 m to 1.0 m in diameter. So, in soft soils, the replacement process does not involve gross disturbance between columns. Some variation in the execution method is adopted particularly in case of stable insensitive soils (c_u = 30 to 60 kPa). Ansari (1993). The machine penetrates both by vibratory impact and by its weight. There is no removal of soil which is displaced laterally involving local shearing like driven piling.

2.6.2 Hydraulic Modification Technique

Free pore water is forced out of the soil via drains or wells. This technique is applicable for both coarse-grained and fine-grained soils. For fine-grained soils expulsion of water is expedited by the long term application of external loads (preload), by introduction of additional drainage path or by means of electrical consolidation process.

2.6.2.1 Preloading Technique

The preloading technique is one of the most economical and effective means for improving soft compressible soils. It is a method of preempting potentially damaging settlements on soft soils. The objective of the preloading technique is to cause all or a portion of the settlement (primary or secondary) to occur before the final construction takes place. The soil is precompressed using a permanent or temporary load. When the required degree of consolidation has taken place, only then the final load is placed onto the soil. In the case of a building, the surcharge would normally be equivalent or higher than the expected bearing pressure. Preloading technique may be employed by placing direct load such as earthfill or the same may be achieved by increasing effective stress on the soil by indirect means such as ground-water lowering and vacuum method. The surcharge generally consists of earthfill from a nearby borrow pit. In Bangladesh, dredged material mostly of sand is used for preloading and as embankment core materials. Sometimes in very soft soils, placing of earthfill to design height may induce soil shear failure. In such cases, loads are placed in stages which are commonly known as 'staged construction method'. Figure 2.2 illustrates the soft ground improvement by preloading technique. In the vacuum method, loads are placed on an air tight membrane over the ground. The membrane is sealed in slurry trenches or in impermeable soil around the edges. Suction tubes are put through the membranes, sealed and then connected to vacuum pumps. The negative pressure created by the pumps, causes water in the pores of soil to move towards the surface.

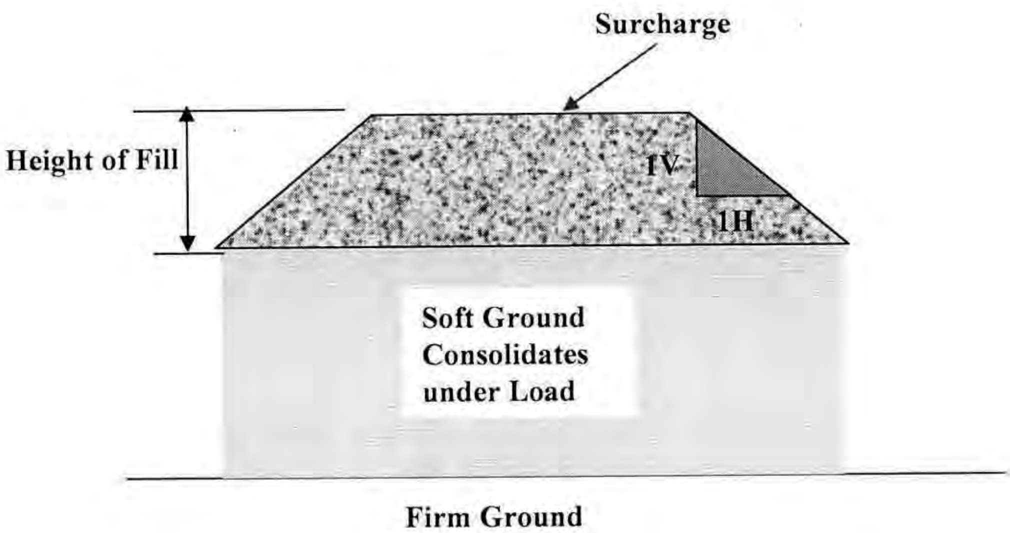

Figure 2.2: Preloading with Earthfill (without Vertical Drains)

2.6.2.2 Preloading with Vertical Drains

Vertical drains alone can not expedite the consolidation process. It can only accelerate preloading techniques such as earthfill method or vacuum method. Vertical drains are only applicable for soft cohesive soils. Without installing vertical drains, settlement of clay soils may extend over many years. Nowadays, highly efficient drain installation methods have been developed, so, preloading combined with vertical drains has become an economic alternative to the deep foundations or other methods of ground improvement. Vertical drains are also used to advantage in the construction of permanent fills, such as highway embankment on soft ground. Vertical drains aid primary consolidation only, because significant water movement is associated with it. Secondary consolidation causes only very small amounts of water to drain from soil; therefore, is not speeded up by vertical drains. Vertical drains may be in the form of sand drains, wickdrains or band-shaped synthetic or natural fibre prefabricated vertical drains. Elaborate concept of vertical drains again will be presented in this chapter. Figure 2.3 illustrates the soft ground improvement by preloading with vertical drains.

2.6.3 Physical and Chemical Modification Technique

This technique deals with modifying soils by the mechanical addition of granular materials into soft ground or mixing chemical compounds such as cement, lime, bitumen and calcium chloride. This technique also comprises of grouting which is executed by injecting fluidized materials into voids of the ground or spaces between the ground and adjacent structures, generally through boreholes and under pressure

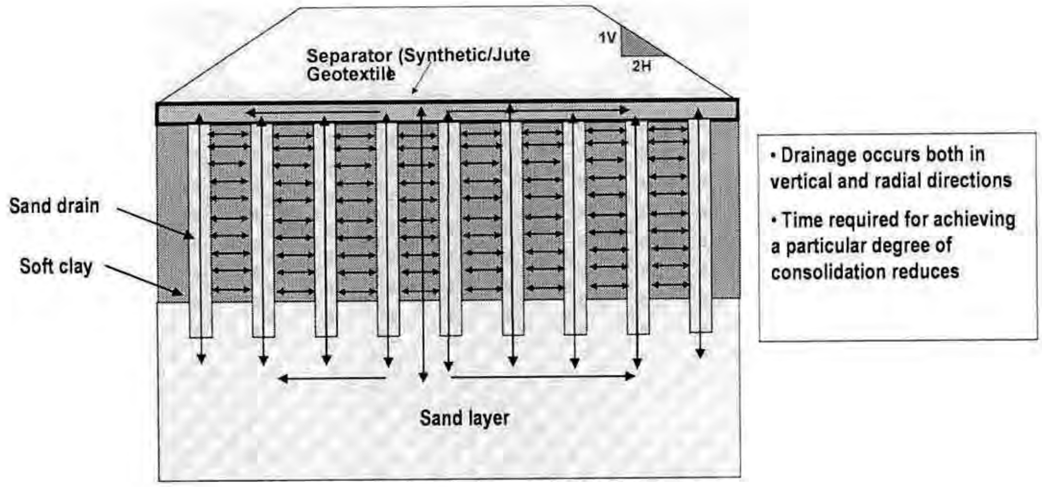

Figure 2.3: Preloading with Vertical Drain

2.6.3.1 Modification by Admixtures

Stabilization by physically mixing additives with surface layers or columns of soil at depth falls under this category. Additives include natural soils, industrial by-products or waste materials, and cementitious and other chemicals which react with each other and/or the ground. Improving the engineering properties of a soil by admixtures is often referred to as 'soil stabilization', particularly in roadworks. Traditional surface stabilization begins with excavating and breaking up of the soil. Then the stabilizer is added if necessary. Soil and additives are mixed thoroughly, compacted and allowed to cure. Most commonly, mix-in-place equipment is used. The term 'deep mixing' is applied to techniques where piles, walls, or foundation blocks are formed by introducing lime, cement, slag and other additives into the soil below the ground surface. The most common artificial additives are:

a) Portland cement (and cement fly-ash)
b) Lime (and lime-fly-ash)
c) Bitumen and tar.

Portland cement is the most common admixture for stabilization technique. When mixed with the soil, it forms a material called soil-cement. The reason for the popularity of additives is that they are applicable to a considerable range of soil types, they are widely available, their costs are relatively low, and they are environmentally acceptable. Figure 2.4 and 2.5 show the techniques of ground modification by lime.

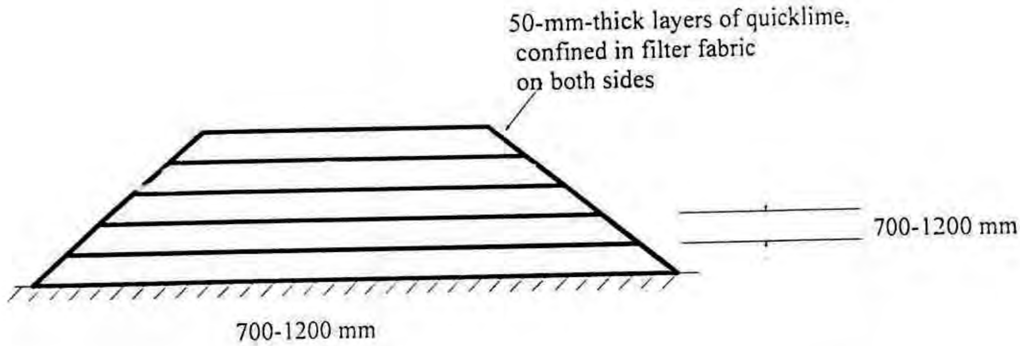

Figure 2.4: Embankment Construction using Quicklime Sandwich.

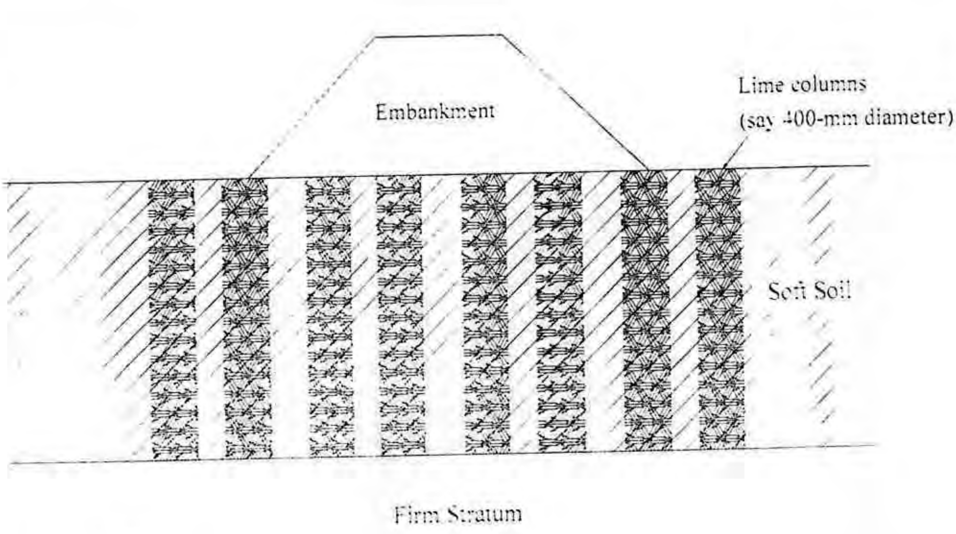

Figure 2.5: Lime Columns below Embankment

2.6.3.2 Modification by Grouting

When additives are injected under pressure into the voids within the ground or between it and a structure, the process is called grouting. It has been extensively used for the past several decades, and is a well-established method of soil improvement. There are two kinds of grouts namely, cementitious grout and chemical grout. Grouting methods are also of four principal types. 'Intrusion grouting' or 'penetration grouting' or 'slurry grouting' consists of filling joints or fractures in rock or soil by injecting grout by pipes. 'Permeation grouting' is the thin grouts into the soil such that they permeate into the voids. Once the grout cures, the porous soil is transformed into near solid mass. Most permeation grouting is performed using chemical grouts. That is why it is often called 'chemical grouting'. 'Compaction grouting' or often called 'displacement grouting' uses a stiff grout (about 25 mm slump) that is injected into the ground under high pressure through a pipe to form a series of inclusions. Compaction grouting is often used to repair structures that have experienced excessive settlement, since it improves the underlying soils and raises the structure back into the position. 'Jet grouting' is the newest method. This method uses a special pipe equipped with horizontal jets that inject grout into the soil at high pressure. The pipes are first inserted to the desired depth, and then they are raised and rotated while injection is in progress, thus forming a column of treated soil. This technique was developed in Japan during the 1960s and 1970s. Based on solubility state, grouts are of three compositions. 'Suspension state' means small particles of solids are distributed in a liquid dispersion medium such as cement and clay in water. 'Emulsion state' describes a two-phase system containing minute (colloidal) droplets of liquid in a disperse phase such as bitumen and water. 'Solution state' means liquid homogenous molecular mixtures of two or more substances. Examples are sodium silicate. organic resins and wide variety of chemical grouts.

2.6.4 Modification by Inclusions and Confinement

Soft soil modification by inclusion and confinement is the latest and updated technology among all ground improvement techniques. It is well known to civil engineers that concrete and soil are similar in regards to their weak tensile strengths. Like concrete. reinforcing materials can be used in the soils to overcome the weakness of soils. Modifications by inclusions and confinement may be achieved by ground anchors, soil nailing and soil reinforcement. The primary purpose of reinforcing a soil mass is to improve its stability, increase its bearing capacity, and reduce settlements and lateral deformation. Reinforcing materials may include natural fibres, strips, special plastic grids, geotextiles, bars, meshes etc.

2.6.4.1 Modification by Ground Anchors

Ground anchors are structural units which transmit forces into stable rock or soils by means of tendons; they replace a support which would otherwise have to be provided by gravity blocks and steel, concrete or timber elements. The simplest form of soil anchors consists of single or multiple plates attached to a rod or cable, either buried, pushed or drilled into the ground, taking on loads up to 150 to 300 KN, Ansari et al (1993). There are many civil engineering situations where lateral, uplift or pullout forces have to be resisted or where confining pressures may have to be generated. Ground anchors may provide a solution to these problems of 'foundations in tension'. Their purposes may be to

- -Tie back sheet piles, slurry walls, and similar temporary excavation support systems.
- -Rehabilitate existing retaining walls.
- -Resist uplift in hydraulic structures, such as dams, weirs and spillways.
- -Stabilize existing and potential landslides.
- -Prevent rock falls in road cuts, tunnels and underground mining.
- -Tie down a pipeline and its foundations.
- -Prevent heaving due to a swelling soil.

In-situ reinforcement is achieved by nails and anchors. The concepts of reinforcement and confinement are very closely associated. Confinement may be produced by internal inclusions or external formwork, supports or abutments.

2.6.4.2 Modification by Soil Nailing

Soil nails are more or less rigid bars driven into soil or pushed into boreholes which are subsequently filled with grout together with in-situ soil. They are intended to form a coherent structural entity supporting an excavation or arresting the movement of an unstable slope. Soil nailing construction is flexible and allows adjusting the direction of the nails to maximize the reinforcing action and construction efficiency. The size of nails varies from thin steel bars to light concrete piles such as root piles and micro-piles. Most of the nail-supported structures are classed as temporary. This is partly due to concerns about corrosion. Uncertainties with regard to design assumptions also suggest caution in the application of the nailing technique.

2.6.4.3 Soil Reinforcement

The term 'reinforced soil' refers to a soil which is strengthened by a material able to resist tensile stresses and which interacts with the soil through friction and/or adhesion. The broader definition of soil reinforcement also includes methods of erosion control and stress transfer via anchors and piles. Soil reinforcement or reinforced earth is constructed of composite material consisting of alternate layers of compacted backfill and tensile reinforcing materials. Inclusions of tensile reinforcements of soils may be of two types: namely; ideally inextensible materials such as metal strips and bars, and ideally extensible materials such as fibres, roots, and geotextiles.

In reinforced earth, soil is made stable by the interaction between soil. which is weak in tension and reinforcements. which is strong in tension. Reinforced earth structures pose flexibility and can tolerate some deformations without distress. As a result they can be used in situations where some ground movements are anticipated. and in combination with other forms of ground improvement. Soil reinforcement is usually done over soft soil but in order to ensure adequate development of friction, normal practice for the preliminary design of reinforced earth structures requires that the percentage of fines (<0.075 mm) in the backfill be less than 15% and the backfill be placed and compacted at a moisture content equal to or less than optimum. Hausmann (1990). This technique allows slope (1V:1H) of the embankment to be steeper than traditional (1V:2H) and walls to be made even vertical. Thus this technique saves extra land acquisition and also reduces the amount of backfill materials. Figure 2.6 illustrates a reinforced earth wall.

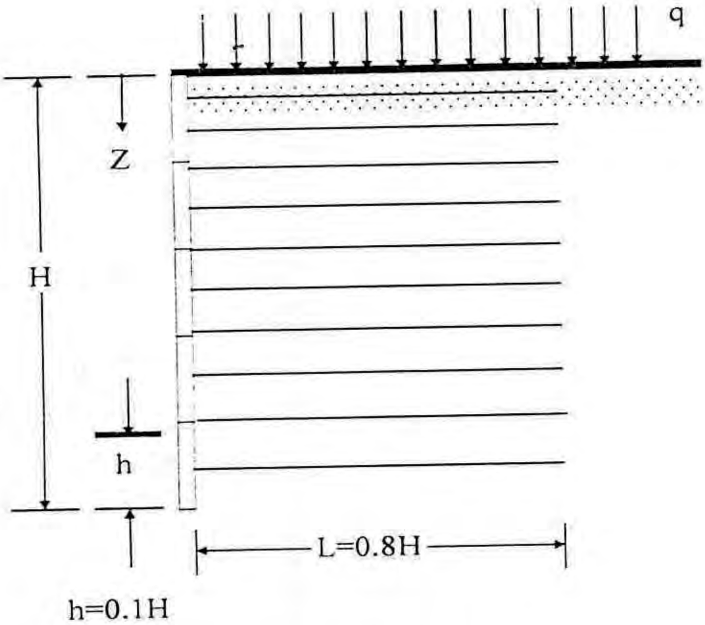

Figure 2.6: Reinforced Earth Wall (with Concrete Face Panel)

2.7 VERTICAL DRAINS

Dissipation of excess pore water pressure from fine grained soils can be accelerated with the use of vertical drains. Vertical drains are artificially–created drainage paths which can be installed by one of several methods and which can have a variety of physical characteristics. The installation of the vertical drains in the clay reduces the length of the drainage path and thereby reduces the time to complete the consolidation process. These are used when preloading alone is not effective to bring expected amount of consolidation within the stipulated time. Vertical drainage can be achieved by the use of sand drains. sand wicks. band drains, prefabricated vertical drains etc. Vertical drains. their types. theoretical analyses. limitations. advantages etc will be described in the following sections.

2.7.1 Background of Development of Vertical Drains

Vertical drains have been employed for more than half a century to promote more rapid consolidation of relatively thick deposits of soft fine grained soils. The first installation of vertical drains dates back to 1934 in California. USA using 20 inch diameter sand drains at 10 feet centres. Atkinson et al (1982). After Second World War. sand drains were used in the reconstruction works of Nagasaki Fishing Port and also the replacement work of a main highway at Kanaura Bay, Okayama Prefecture, bypassing the town of the bay, Aboshi (1993). The basic theory of sand drains was presented by Rendulic (1935) and Barron (1948) and later summarized by Richart (1959), Das (1985). Until 1970, the majority of the vertical drains were used to be large diameter sand drains. In the USA sand drains were constructed by driving down casings or closed-ended mandrels into the soft soils which caused a considerable thickness of smear around the drain. The smear problem was overcome in the Netherlands during the 1950s with the development of jetted sand drains. This method had its own problems, such as the additional cost of large jetting pumps and the difficulties of disposing of large quantities of water. The principal alternative to large diameter sand drains was the much smaller band-shaped cardboard wicks first employed by Kjellman in 1948. These wickdrains became susceptible to biodegradation particularly in acidic groundwaters. In the 1970s, various band drains made of mainly synthetic materials were introduced. Due to the rising cost of providing large quantities of suitable sand for sand drains and the great technical advances led to development and use of band drains made of synthetic fabrics such as polyethylenes, PVC, polypropylenes and polyesters etc. Nowadays, there are a large number of these manufactured drains available in the free-markets. Generally, they consists of a central core, whose function is to act as a free-draining water channel, surrounded by a thin filter jacket, which prevents the surrounding soil entering the central core but allows free entry of excess pore water.

2.7.2 Fundamentals of Vertical Drains

In case of installing vertical drains, two fundamental cases arise:

(1) Free strain case: When the surcharge applied at the ground surface is of flexible nature, there will be equal distribution of surface load. This will result in an uneven settlement at the surface. Preloading with imported sand over soft soil could be an example of such a case.

(2) Equal strain case: When the surcharge applied at the ground surface is rigid, the surface settlement will be same all over. However, this will result in an

unequal distribution of stress. One good example of this category could be mat foundation.

2.7.3 Type of Vertical Drains

Though there were number of shapes for vertical drains, the main two types are either round or flat band-shaped, Mcgown et al (1982). These two types again include few sub groups:

(1) Round Shaped Vertical Drains:

- (a) Sand drains
- (b) Wickdrains
- (c) Wrapped flexible pipes

(2) Band (Strip) Shaped Vertical Drains:

- (a) Synthetic PVD
- (b) Natural Fibre PVD

2.7.3.1 Sand Drains

These are constructed by filling sand into a hole made by casing or mandrel. These holes can be made by number of ways such as displacement method, drilling method and wash boring method. Displacement method creates a smear zone near the drain well circumference thereby reducing the horizontal permeability. On the other hand washing method results in irregularly shaped drains as a result they can often have an effective radius in excess of that estimated. Few commonly adopted methods for installing vertical drains are given in Appendix A. Large diameter sand drains may also act as column or pile effect thereby reducing the actual settlement. Sometimes cost of imported sand and its quantity may preclude the use of sand drains. Besides, problem like bulking of sand during its placement in the drain can be a problem leading to the formation of cavities and collapse on flooding.

2.7.3.2 Wickdrains

These are readymade small diameter sand drains pre-packed in filter stocking. In early days, these used to be covered with woven jute burlap, but polypropylene woven and melt bonded fabrics are now used. Wickdrains provide economy in the sand quantity, offer large variety of drilling methods and least amount of disturbance in the adjacent clay layer. The fabric stocking allow the sandwick to be extremely flexible and tenacious which makes them particularly attractive in number of situations, Mcgown et al (1982).

2.7.3.3 Wrapped Flexible Pipes

These drains consist of flexible, corrugated, plastic pipes surrounded by either a natural fibre or an engineering filter fabric. They can be inserted into the ground using all driving systems but normally they are placed by means of a mandrel which is then removed. A non-recoverable conical sealing and driving tip is used to anchor the pipe on withdrawal of mandrel. They are easy to install and cause little soil disturbance. One advantage of this system is that checks can be made down the pipe whether any clogging has occurred.

2.7.4 Analysis of Sand (Radial) Drainage System

The purpose of a deep vertical drain is to relieve excess pore water pressure by reducing the length of the drainage path through the soil. To estimate the performance of the drains, a number of analytical approaches have been developed, all basically derived from Terzaghi's theory of one dimensional consolidation. This was later extended to take account of radial flow by Rendulic (1935), Carillo (1942) and Barron (1948) and a very informative review of these theories is given by Richart (1957), Mcgown et al (1982).

2.7.4.1 Conventional Sand Drains and Smear Zone

Figure 2.7 shows the basic layout of a traditional sand drain. A clay layer is underlain by rock strata and overlain by permeable sand layer at top. This top sand layer either natural or laid later for filtration purpose is called 'sand blanket'. Ground water level normally is above the clay layer. The theoretical considerations applied for all derivations assume the clay layer to be saturated. In the figure it is seen that pore water travels only a small distance to get into the sand drains and thereafter it moves towards the upper direction through the sand drains. So, initial radial drainage is converted to later vertical drainage. During installation of sand drains, clay particles adjacent to sand drains are disturbed and thereby both vertical and horizontal permeability appreciably reduces. Thus 'Smear Zone' in sand drains is created by the remolding of clay during the drilling operations for building it.

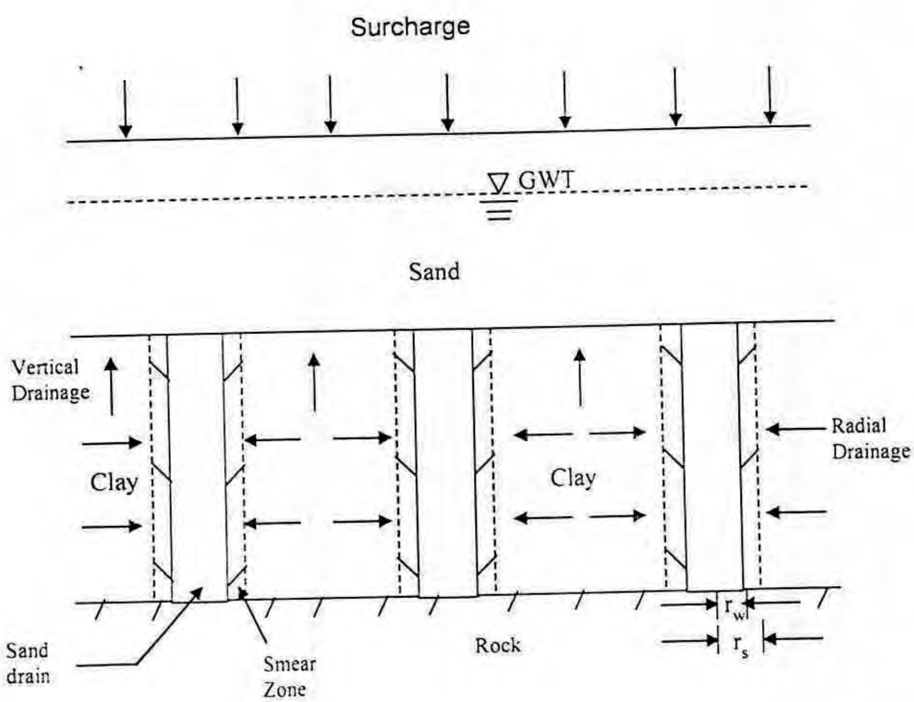

Figure 2.7: Sand Drains with Smear Effect.

2.7.4.2 Mathematical Assessment of Sand Drains

The assessment of the average degree of consolidation due to radial drainage is more difficult than evaluation of vertical consolidation. It is usual to approximate the problem to that of a cylinder of consolidating soil. Figure 2.8 shows the general pattern of the layout of sand drains. In the figure, notations used are:

(1) r_e = Radius of the equivalent influence circle.

(2) r_w = Radius of sand drain well.

(3) r_s = Radial distance from the centre line of the drain well to the farthest point of the smear zone. In case of no smear case, $r_w = r_s$.

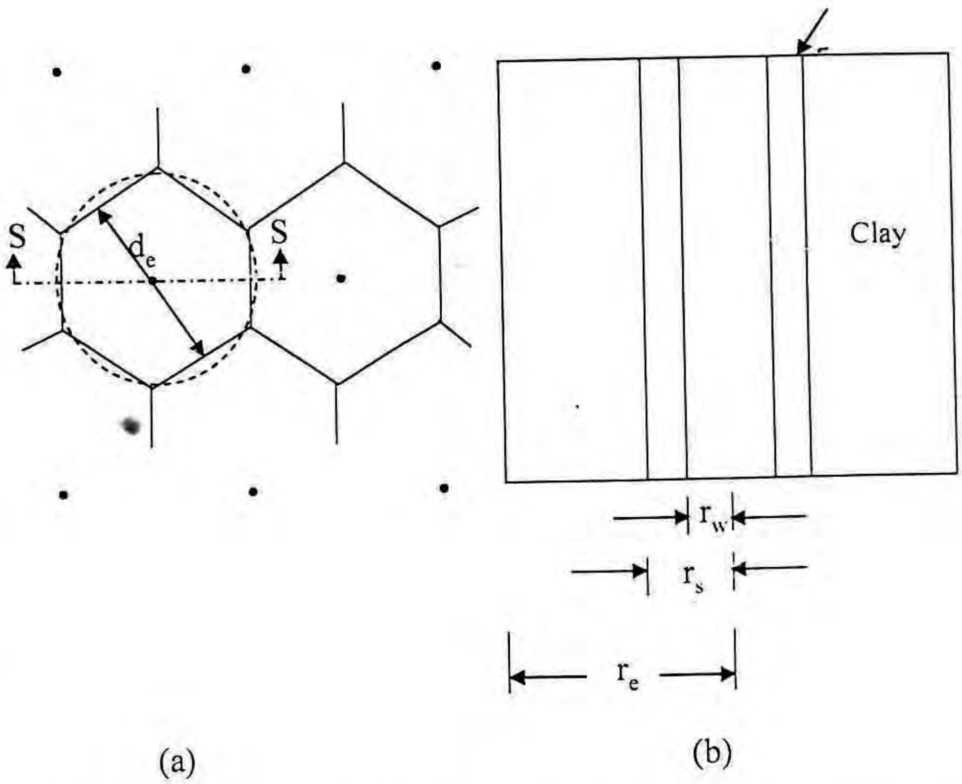

Figure 2.8: General layout of Sand Drains. (a) Plan. (b) Cross section at SS

For radial drainage. Terzaghi's one dimensional consolidation equation can be written in partial differential form as:

$$\frac{\partial u}{\partial t} = C_{vr}\left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}\right) \tag{2.16}$$

Where, u = excess pore water pressure.

r = radial distance measured from centre of drain well.

C_{vr} = coefficient of consolidation in radial direction.

The problem of equal strain case with no smear ($r_w = r_s$) was solved by Barron (1948). Thus the excess pore water pressure at any given time t and radial distance r is given by:

$$u = \frac{4u_{av}}{d_e^2 F(n)}\left[r_e^2 \ln\left(\frac{r}{r_w}\right) - \frac{r^2 - r_w^2}{2}\right] \tag{2.17}$$

Where. $F(n) = \frac{n^2}{n^2-1}\ln(n) - \frac{3n^2-1}{4n^2}$ (2.18)

u_{av} = Average value of pore water pressure through the clay layer

$$= ue^{\lambda} \tag{2.19}$$

$$\lambda = \frac{-8T_r}{F(n)} \tag{2.20}$$

Time factor for radial flow:

$$T_r = \frac{C_{vr}t}{d_e^2} \tag{2.21}$$

The average degree of consolidation due to radial drainage is

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right] \tag{2.22}$$

One important point here is that for $\frac{r_e}{r_w} \phi\, 5$, the free strain and equal strain solutions give approximately the same results for the average degree of consolidation, Das (1985). Figure 2.9 gives the values of time factor T_r for various values of U_r.

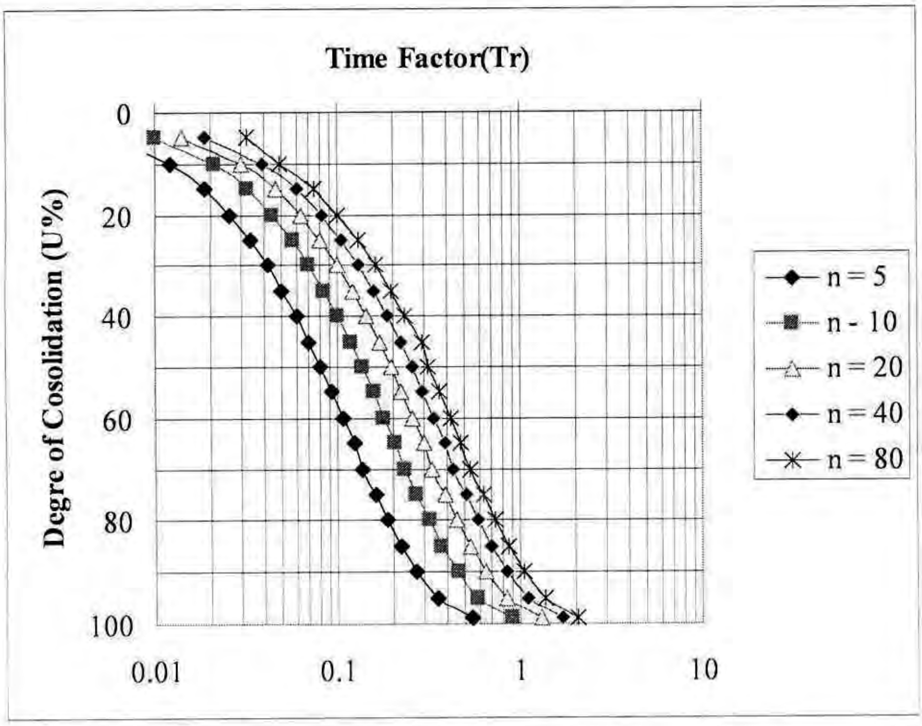

Figure 2.9: Variation of Degree of Consolidation, U_r with Time Factor, T_r (After Das, 1985)

2.7.4.3 Degree of Consolidation for Combined Vertical and Radial Drainage

In the derivation of Terzaghi's one dimensional consolidation, radial drainage was not taken into account. At the same time, when average degree of consolidation for radial drainage was found out, one dimensional consolidation in the vertical direction was disregarded. But actually the dissipation takes place in both directions simultaneously. The combination of two solutions as already stated in Equations (2.6) and (2.22) give the total average degree of consolidation that was first presented by Carillo (1942):

$$U = 1 - (1 - U_v)(1 - U_r) \tag{2.23}$$

U = Average degree of consolidation for simultaneous vertical and radial drainage

U_v = Average degree of consolidation calculated on the assumption that only vertical drainage exists.

U_r = Average degree of consolidation calculated on the assumption that only radial drainage exists.

2.7.5 Synthetic Prefabricated Vertical (Band Shaped) Drains

The first band (strip) shaped vertical drain was developed by the Swedish Geotechnical Institute. It was made of cardboard with internal ducts developed by Kjellman (1948). This type was later superseded by thin fluted PVC drains. Currently, there are more than 50 different types of drains in the market. These are mostly of composite make up; a corrugated or studded inner core wrapped in synthetic filter fabric. Figure 2.10 shows some typical core shapes of band drains. These are very light and easier to handle at site. The installations of band drains cause little disturbance to the adjacent soil. The band drains made of synthetic material are usually driven by displacement method. Auger and washing methods are not usually used. The mandrels used for synthetic drains are hollow and rectangular or trapezoidal in cross section. Similar to Kjellman wickdrain. band drains are generally about 100 mm wide and 2 to 6 mm thick. Hausmann (1990). Details of some synthetic band (strip) shaped drains are given in Appendix A.

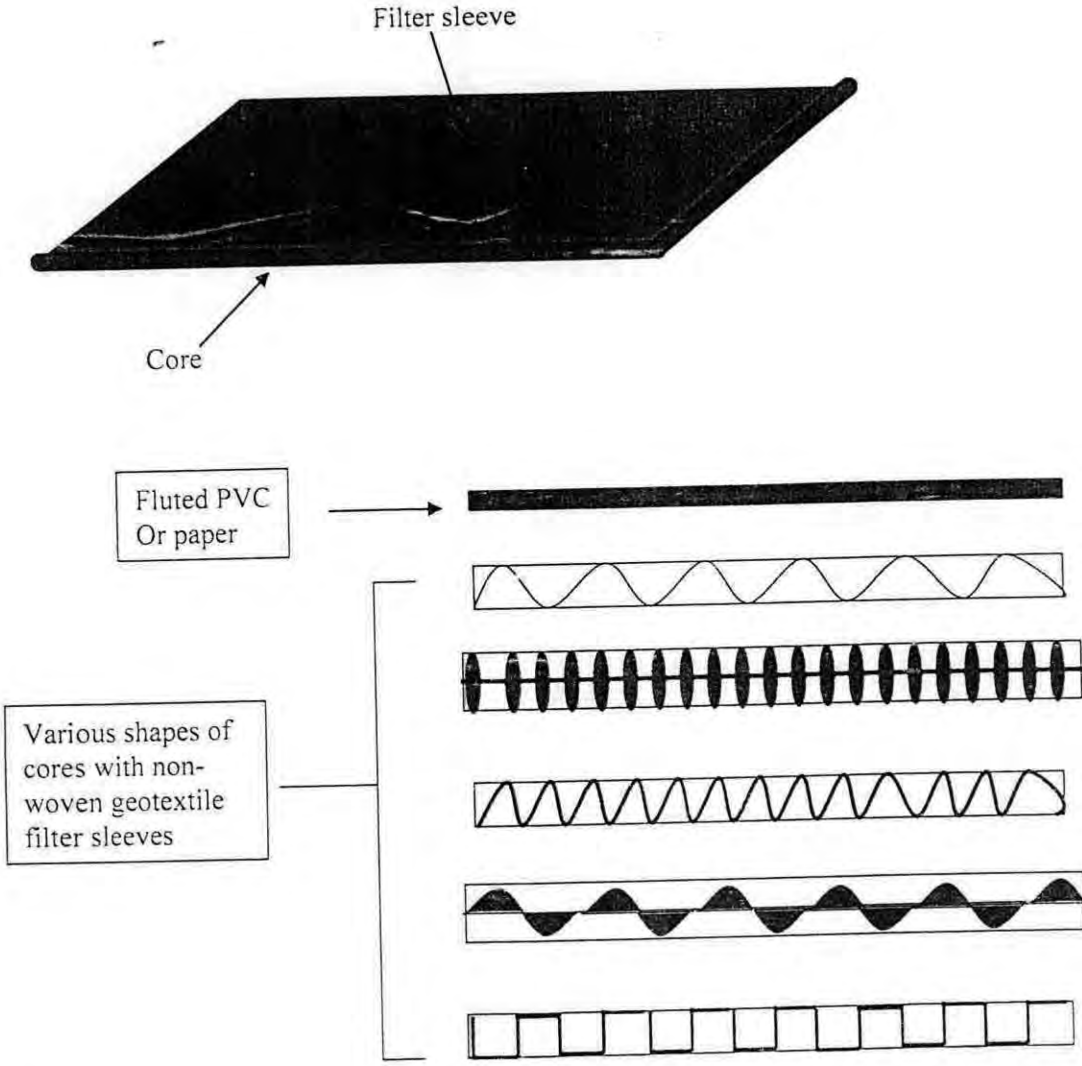

Figure 2.10: Typical Core Shapes of Synthetic (Band Shaped) Drain.

2.7.5.1 Designing with Synthetic PVDs

Design with PVDs here means finding out the spacing of drains once drain sizes (width and thickness) are known. The main focal point is on the time for required consolidation of the subsoil to occur. Generally, the time for 90% consolidation is desired, but other values might also be of interest. This design is possible provided, other data related to drain size, soil permeability, consolidation parameters etc are known. Two approaches to such a design are possible:

The first is equivalent sand drain approach that uses the PVD to estimate an equivalent sand drain diameter and then proceed with the design in a manner of sand drains. This is done by using actual cross-sectional area of the candidate PVD and making it into an open void circle. This open void circle is then increased using the estimated porosity of sand to obtain the equivalent sand drain diameter.

Equal void area method, Koerner (1986):

$$\text{Equivalent diameter, } d_e = \frac{\sqrt{4Btn_d / \pi}}{n_s} \qquad (2.24)$$

Where, B = width of synthetic strip/band drain

t = thickness of strip

$$n_d = \frac{\text{Void area}}{\text{Total cross sectional area of the strip}}$$

n_s = porosity of sand drain

Equal circumference method:

The equivalent diameter of band drain is

$$d_e = \frac{2(b+t)}{\pi} \qquad (2.25)$$

Where, b = width of band drain

t = thickness of band drain

Equation (2.25) is preferred by Hansbo (1979) and is usually more conservative than Equation (2.24), which means it results in a smaller equivalent diameter.

Another approach to PVD design is straightforward than the preceding approach and is the preferable one. Hansbo (1979) developed this equation modifying original Barron's (1948) formula in which time (t) of consolidation is given by the following:

$$t = \frac{D^2}{8c_h}\left[\frac{\ln\frac{D}{d}}{1-\left(\frac{d}{D}\right)^2} - \frac{3}{4} + \frac{1}{4}\left(\frac{d}{D}\right)^2\right]\ln\left(\frac{1}{1-U}\right) \qquad (2.26)$$

Where, t = Time for consolidation

c_h = Radial (C_{vr})/horizontal coefficient of consolidation. (m^2/year or mm^2/min)

D = Influence diameter of drain.

For square grid, D = 1.13*Spacing (m)

For triangular grid, D = 1.05*Spacing (m)

d = Equivalent diameter of the PVD (d = 2(b+t)/π), (m)

U = Average degree of consolidation.

Since $\frac{d}{D}$ is small for synthetic drain, the equation can be reduced to

$$t=\frac{D^2}{8c_h}\left[\ln\frac{D}{d}-\frac{3}{4}\right]\ln\frac{1}{1-U} \quad (2.27)$$

Since the discharge capacity of PVD is not infinite as compared to sand drains, the following revised formula is used which is being also followed by 'Flexi Drain'

$$t=\frac{D^2}{8C_h}\left[\ln\left(\frac{D}{d}\right)-\frac{3}{4}+0.64\pi l^2\left(\frac{k_c}{q_w}\right)\right]\ln\left(\frac{1}{1-U}\right) \quad (2.28)$$

Where, l = Drain length at unilateral flow (m)

k_c = Soil permeability (m/s)

q_w = Discharge capacity of drain (m^3/s).

2.7.5.2 Advantages of Synthetic PVDs over Sand Drains

Synthetic prefabricated vertical drains (synthetic PVD) have number of advantages over traditional sand or wickdrains. Synthetic PVDs can be easily and rapidly installed than other types of vertical drains. In a study, it was observed that 120 Alidrain, a type of synthetic PVD were installed per day. On the other hand, only 7 to 8 sand drains of similar depths were installed per day with same resources, Davies et al (1982). Synthetic PVDs are made of uniform and non degradable materials which can be easily stored and transported. Sand drains require large and heavy rigs whereas synthetic PVDs require lighter equipment. Synthetic PVDs have a tensile strength which helps to preserve continuity. Sand drains are costly whereas synthetic band drains are low cost. Treatment of soft soil site may be possible for only one-fourth the costs of traditional sand drains. Hausmann (1990).

2.7.6 Jute PVD

Jute is a biodegradable natural fibre which is produced in abundance in Southeast Asia particularly in Bangladesh. Though Bangladesh and India are the leading producers of jute, it is also grown in Nepal, China, Thailand, Indonesia, Burma, Brazil, Vietnam, Taiwan, Cambodia and in some other countries of Africa. Before the production and use of geosynthetics, jute used to be applied for versatile uses. Because of its topmost role in earning foreign currency, jute is called "Golden Fibre" of Bangladesh. Nowadays, this fibre has lost its position to many synthetic materials. Researchers are still looking for the potential uses of jute especially as civil engineering material. Multifarious uses of jute may include soil reinforcement, soil separator, filtration, erosion control and soil drainage etc. To achieve rapid consolidation, sand drains and synthetic PVDs are currently being used in many countries. Jute based vertical drains may be used in place of sand drains or synthetic PVDs.

2.7.6.1 Jute Fibre

Jute is one of the world's most important long vegetable fibres, being exceeded in production quantity only by cotton. It is collected mainly from two commercially important species, namely, White Jute (Corchorus capsularis) and Tossa Jute (Corchorus olitorius). It was first used as an industrial raw material for making packaging materials as a replacement for European-grown flax and hemp. A temperate, wet and humid climate with alluvial soil is conducive for the growth of jute. It is a photo-reactive plant and for that long hour of day, light is necessary for its rapid growth, Mohy (2005).

2.7.6.2 Physical Properties of Jute

Jute consists of fibre bundles arranged in several layers between the central hollow woody core and the outer skin. The individual fibres are held together by non-cellulosic materials such as lignin, hemi-cellulose, pectin etc to form fibre strand. Jute's quality is normally characterized by colour, luster, strength, cleanliness, flexibility, length, proportion of roots and moisture content. Naturally, good quality jute should possess good colour. This should be lustrous as more lustrous jute is found to be stronger. An Indian variety of jute known as "Shamla Daisee", although darkish blue in colour, is well known for its high strength, luster and fineness. The fibre should be fine, long and strong and the body should be clean. The percentage of roots should be low. Faults like knots, sticks, specks etc should be absent. The water content must not exceed the standard limit. Important physical properties of jute fibre are listed in Table 2.3. A jute fibre is considered to be of good quality when yarn spun having:

Table 2.3: Important Physical Properties of Jute Fibre

Ser No.	Parameters	Values
1.	Jute cell/ultimate width	15-20 microns
2.	Jute cell/ultimate length	1-6 mm
3.	Jute fibre/ultimate width (Average)	184 m
4.	Jute fibre/ultimate width (Average)	2.5 m
5.	Tenacity	2.7-5.3 g/tex
6.	Specific gravity	1.48
7.	Moisture regain at 65RH/22°C	13.8%
8.	Fineness(gm/100m)	1.4-1.65 tex
9.	Breaking elongation	0.8-1.8%
10.	Refractive index(Parallel)	1.577
11.	Refractive index(Perpendicular)	1.536
12.	Fluorescence with corning filter	Bluish white
13.	Phosphorescence	Yellow
14.	Phosphorescence(time)	15 sec
15.	Swelling in water(Diameter)	20-21%
16.	Swelling in water(Area)	40%
17.	Stiffness(Average)	185
18.	Specific heat	0.324 cal/g/°C
19.	Water retention	70%
20.	Young's Modulus	
	a) White	0.86-1.74 dynes/cm^2x1000
	b) Tossa	0.96-1.94 dynes/cm^2x1000
21.	Modulus of Rigidity	
	a) White	dynes/cm^2x109
	b) Tossa	4.42 dynes/cm^2x109
22.	Linear density	0.94-2.94 tex
23.	Density	1.52-1.59

After Abdullah. (2008)

a) Quality ratio (Q.R.) above 90

b) A low coefficient of variance (C.V.)

c) Fibre possesses high elasticity and low frictional properties.

Q.R. indicates the strength of the yarns, being expressed as the ratio of breaking strength in pound (lb) to the strength of yarn in lb/spindle of 14400 yards and multiplied by 100. C.V. indicates the irregularity in the weight per unit in a short length of the yarn. Q.R. above 90 and C.V. below 23 indicates fibre of good quality. Q.R. above 80 and C.V. below 25 indicates fibre of medium quality whereas Q.R. below 80 and C.V. above 25 means fibre of poor quality, Mohy (2005).

2.7.6.3 Chemical Composition of Jute Fibre

Jute is lignocellulosic fibre composed of cellulose (58%-63%), hemicellulose (21%-22%), lignin (12%-14%), protein (0.8%-1.5%), pectin (0.2%-0.5%), fat and wax (0.4%-0.8%), mineral matter (0.6%-1.2%) and traces of tannin and colouring material. Except for cellulose. all other components are functioning more or less as cementing materials. Stiffness and lower wet strength of jute fibre are closely related to the reinforced resinous structure. It has been observed that the molecular weight, fibre length and degree of polymerization of jute cellulose are comparatively smaller than those of cotton. Jute fibre consists of crystalline, para-crystalline and non-crystalline, amorphous resin. Cellulose is the major portion within the crystalline part of jute fibre. The high moisture regain property of jute fibre compared to other fibres is attributable to these special characteristic of jute fibre. The hemicellulose units of jute are fairly simple and of low molecular weight and susceptible to action of alkali. Lignin is the single-most important component in jute which in fact distinguishes it from other textile fibre. Some of the peculiar characteristics of the fibre such as yellowing or photo degradation are attributed to its presence.

Jute is acidic in nature and its acidity is due to the presence of phenolic hydroxyl group present in lignin. The contribution of cementing materials (lignin. hemicellulose) on the tensile properties of jute fibre both in dry and wet conditions is enormous. It has been observed that when jute is treated with chemical reagents employed in textile pretreatments and bleaching processes. lignin, hemicellulose and other encrusting substances are attacked and to some extent removed. Lignin also protects jute from ultra-violet and visible radiations emitted by sun. Jute is degraded by heat, mildew acids and alkali. Jute burns like other cellulosic fibres.

2.7.6.4 Treatment of Jute Fibre

Jute is a biodegradable natural fibre. This fibre is also hygroscopic. Bangladesh is situated in a tropical, humid and flood affected country. Jute used for geotechnical engineering purposes has to sustain an unfriendly environment where it loses it's all the properties very rapidly. One of the single most problems of using jute in the soil water interaction phase is the biodegradation. To increase the design life of jute, limited works have been carried out by some researchers like Prodhan (1994), and also by few organizations and institutes. Recently a wide range of geo-jute has been developed at BJRI by blending jute with hydrophobic fibre like coir or by modification with bitumen, latex and wax resinous materials. These treatment techniques may enhance the life of jute geotextiles up to or even more than 20 years, Khan (2008). A summary of blending jute with different materials is shown in Table 2.4. Following are three different types of blending:

a) Jute with natural fibre i.e. jute with cotton, coir, flax etc.
b) Jute with synthetic fibre like jute blended with polythene, polypropylene, nylon. polyester, poly-acrylic etc.
c) Modification/Treatment of jute with bitumen, latex, wax, resinous material.

Blending jute with natural fibre like cotton for making finished and apparel products is a common technology where jute is modified with chemical and softening agents. Again union fabrics of jute mixture are made by inserting warp and weft thread with different fibres. For making geotextile materials. low quality cotton is used in blending and union fabrics.

Some researchers have tried to blend jute with synthetic materials like acryline. polyethylene. polypropylene. polyester rayon etc. In this process compatibility of blending items plays the significant role. Modification of mixing fibres is necessary for increasing their compatibility through chemical. bio-chemical or mechanical treatment. An investigation was carried out by Rao. P. et al (1994) at Central Road Research Institute. New Delhi. India on six types of blended jute products. These were non-woven fabrics consisting of varying proportions of Jute (J) and Polypropylene (PP). It is seen from the results that mechanical properties (grab tensile strength, interface friction angle etc) and survivability properties (puncture resistance, index elongation etc) are enhanced when synthetic materials are added with jute. On the other hand, hydraulic property (permittivity) is reduced with the addition of synthetic material.

Table 2.4: Summary of Blending Jute with Different Materials

Type	Composition	Possible Durability	Biodegradability	Moisture content	Mass/unit Area	Tensile Strength (lb)
Woven jute in different structure	Jute	2-6 months	Quick	12-14%	220-280	120-140
Woven jute in different structure	Jute & Coir	5-12 months	Slow	7-10%	220-280	240-660
Woven jute but treated composite	Jute, Bitumen & Carbon	6-48 months	Long run	3-8%	Variable weight	140-700
Non-woven	Jute & blanket	6-18 months	Slow	8-12%	800	300-800
Woven with different construction	Jute & latex	5-20 years	Long run	5-7%	≥800	300-800
Non-woven jute	Jute, blanket & latex	5-20 years	Long run	5-7%	≥800	≥800

After Khan (2008)

2.7.6.5 Use of Jute in Vertical Drains

Inherent drawback of using jute in humid condition is its gradual biodegradation. Very few people could think and still can think of using this material in any permanent or longer duration works. But according to studies so far carried out, it can be deduced that there may be two approaches to use jute in civil engineering purposes. Firstly, use of jute based products may include as transitional layers in the staged construction method, in civil engineering works that have short design life, as a construction aid and repair of structures during emergencies. Secondly, jute can be used in any projects spanning any duration up to 20 years by treating them according to deigned biodegradability. Use of jute in vertical drains falls under first category though some amount of treatment might be necessary. Vertical drains are employed to expedite the consolidation process. As a result construction period is also reduced. With the use of vertical drains, consolidation time may be reduced to 6 months to 2 years which would have otherwise taken many times more duration than this time. After achieving the desired consolidation, vertical drains become useless and if it is synthetic material, it will continue to degrade the environment. Few endeavours have been made to study jute based PVD as an alternative. A flexible prefabricated jute drain made out of jute fabric and coir ropes suitable for consolidating soft compressible soils, has been developed and field tested at Changi Airport by Lee et al. (1989), Rao, G. et al. (1994). They also developed and tested fibre drain (FD) at the National University of Singapore, Kabir et al (1994). These drains were installed up to 22 m depth. For the observed period of over two years, back analysis using observed settlement under reclamation fills showed that these drains have adequate discharge capacity and functions very well, Rao, G. et al. (1994).This FD has been used in several countries in South East Asia. FD is a band shaped products consisting of four strands coconut coir enveloped by two layers of jute burlap filter. The size of FD is 90-100 mm wide and 7-9 mm in thickness. Park et al (www.geosynthetics.net/tech_docs/GeoAsia04Park) carried out tests to evaluate discharge capacity of both plastic board drains (PBD) and fibre drains (FD) using large scale test apparatuses.

Kabir et al. (1994) prepared two types of FDs similar to that developed by Lee et al (1989). Both the types had a couple of layers of twill fabric filter jackets. In one type, twisted coconut coir rope was used. The other type had twisted jute fibre ropes. The drains were 100 mm wide and 10 mm thick, each having four rope channels for hydraulic conveyance. In that test, they found that there is in general, reduction in discharge rate due to increase in confining pressure between 2 kPa to 90 kPa. They also found that discharge ratio for FD with jute rope core at all pressures was lower than that with coconut coir. Abdullah et al. (1994) also carried out research on engineering properties of jute and jute based products at BJRI. They also developed vertical drains with jute and called them 'Banana Drains' (BD).

2.7.6.6 Designing with Jute PVD

Banana Drain (BD) is a special type of fibre drain where jute with higher content of lignin is used as lignin is more resistant to biodegradation. Abdullah et al (1994) developed this Banana Drain with two to three types of woven, nonwoven and netting materials, where the innermost part is enveloped by nonwoven jute materials. The woven outer part is made with specially blended cloth with jute, coir and jute cuttings. This drain works like wickdrain and for the mathematical analysis of BD, an equation developed by Prodhan (1984) has been used for finding out the spacing of BD and time of consolidation. This equation closely resembles the equation developed by Hansbo for synthetic PVD. Prodhan's equation for solving jute PVD is explained below:

$$t = \lambda z \frac{D^2}{8C_r}\left[\ln\left(\frac{D}{d_d}\right) - 0.75\right]\left[\ln\frac{1}{(1-U_r)}\right] \tag{2.29}$$

Where, $\lambda z = \frac{t_{B.D}}{t_{W.D}} \pi 1.0$

$t_{B.D}$ = Time for Banana Drain

$t_{W.D}$ = Time for Band-shaped wickdrain

D = Influence diameter of vertical drains

d_d = Equivalent diameter of drain.

U_r = Average degree of consolidation due to radial drainage.

C_r = coefficient of radial consolidation

$d_d = \frac{2(a+b)}{\pi}$, a = width of drain. b = thickness of the drain.

Here. λz shows the drainage characteristic of BD which is more effective than any type of band shaped drain. However. in this equation, λz needs to be found out. So. with this equation. if drain spacing. drain pattern. drain size and C_r are known. then spacing or time of particular degree of consolidation can be found out. Equation (2.29) is exactly similar to the equation developed by Hansbo (Eq. 2.27) except the factor λz incorporated by Prodhan in his Equation (Eq. 2.29).

2.7.7 Some Controlling Factors of Radial Drainage

Radial drainage through the vertical drains is immensely affected by the smear effect of soil that occurs mainly during drain installation. It is clear from many experiences that soil particles near the drains get severely disturbed thereby reduces the permeability. Performance of drains also depends on the drain spacing, shape and size of the drain well. Drain internal resistance is another factor that slows down the drainage through the channel or core of the drain.

2.7.7.1 Grid Spacing, Shape and Size of Vertical Drains

Usually drains are installed in some triangular or rectangular grid pattern and therefore the problem is not axisymmetric. No analytical solutions exist for these real situations. For triangular spacing of sand drains, the zone of influence of each drain is hexagonal in plan. Figure 2.11 shows both square and triangular grid pattern of vertical drain layout. The diameter of equivalent cylinder of soil surrounding each drain d_e (equivalent diameter is also expressed by 'D') is calculated on the basis of equivalent cross-sectional areas, i.e. for drains on a square grid pattern with a drain spacing of S:

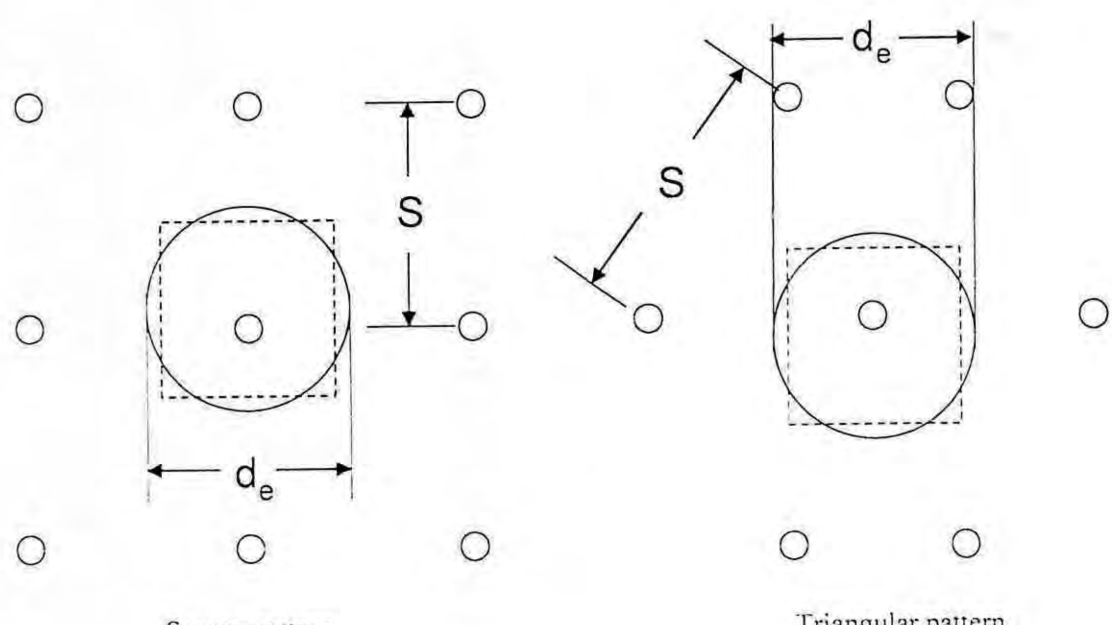

Figure 2.11: Vertical Drain Patterns

$$\pi \frac{d_e^2}{4} = S^2$$

$$d_e \approx 1.13S \tag{2.30}$$

for a triangular grid this becomes:

$$d_e^2 = \frac{2\sqrt{3}}{\pi} S^2$$

$$d_e \approx 1.05S \tag{2.31}$$

Where sand drains, or their modern derivatives like wickdrains or plastic tube drains are used, the cross-sectional area of the drain is the same as that used in the derivation of radial consolidation (Eq. 2.22) and there should not be any shape error. This is not the case with band or strip drains where the flow pattern around the drain is considerably different from the cylindrical case. Thus a strip shaped drain has to be converted into a circular cylindrical drain producing the same consolidation effect as the strip shaped drain. Two different approaches are possible. The first is an equivalent sand drain approach that uses the band drain to estimate an equivalent sand drain diameter and then proceed with design in the manner of sand drains. This is done by taking the actual cross sectional area of the candidate band drain and making it into an open void circle, (Eq. 2.24). This open void circle is then increased using the estimated porosity of sand to obtain the equivalent sand drain diameter, Koerner (1997). Another method follows equal circumference method. (Eq. 2.25).

2.7.7.2 Smear Effect on Sand Drains and PVDs

The effect of smear zones on the consolidation capacity of sand drains was also first analyzed by Barron (1948). The analysis is based on the assumption that the clay in the smear zone will have one boundary with zero excess pore water pressure and the other boundary with an excess pore water pressure which will be time dependent. Das (1985). Based on this assumption:

$$u = \frac{1}{m} u_{av} \left[\ln\left(\frac{r}{r_e}\right) - \frac{r^2 - r_s^2}{2r_e^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right) \ln S \right] \tag{2.32}$$

Where, k_s = coefficient of permeability of smeared zone.

$$S = \frac{r_s}{r_w}$$

$$m = \frac{n^2}{n^2 - S^2}\ln\left(\frac{n}{S}\right) - \frac{3}{4} + \frac{S^2}{4n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right)\ln S \qquad (2.33)$$

It should be pointed out that for the case where no smear is assumed, i.e. S=1, the expression for m reduces to F (n) of Equation (2.18). Barron assumed for the analysis that $\frac{k_h}{k_s}$ was 10 and $\frac{r_s}{r_w}$ was 1/6, as such time required to achieve a particular degree of consolidation would be increased by about 20%. If the thickness of the smeared zone was increased to twice the drain radius, then the effect would be approximately to double the consolidation time. In the case of an 18 inch (457 mm) diameter casing, a typical size for many sand drain installations, the thickness of the remolded zone would be about 90 mm. The effect of such a thick smeared zone of greatly reduced permeability would be to negate any potential beneficial effects of such a drain. Cassagrande and Poulos (1969) therefore concluded that the drains installed by displacement methods were generally uneconomic and cited a number of installations where such drains had not only failed to produce any beneficial effects. but probably caused additional problems due to the disturbance during installations, Atkinson et al (1982).

Currently available PVDs are 100 mm in width, about 3 to 4 mm thick and are installed using a lance about 140 mm wide and 30-40 mm thick. Using the same approach as Cassagrande and Poulos, a smeared zone about 10 mm thick would be expected along the wall of the hole made by the lance. After a period of time when the hole had closed, the smear zone would lie against the filter layer of the drain. Another research showed that a fabric filter initially allows the finer soil particles pass through the filter. i.e. piping occurs. As these smaller particles pass through the fabric, a bridging network of the larger soil particles builds up adjacent to the drain, thus forming a natural graded filter within the soil. the thickness of which was found to be several millimetres. The effect of piping is to remove the clay particles from the smeared zone immediately adjacent to the drain. In the case of the PVDs installed by the typical size of lance used at present. the thickness of the smear zone is similar to the thickness of the natural soil filter created by piping. Consequently. it appears probable that the majority of the smear caused by installation process is removed by formation of the naturally formed graded filter, Atkinson et al (1982).

2.7.7.3 Effect of Drain Resistance on Sand Drains and PVDs

The design of vertical drain system is generally based on the classical theoretical solution developed by Barron(1948) in which the drains are assumed to be functioning as ideal wells, i.e. their permeability is considered infinitely high as compared to that of the soil in which the drains are placed. This assumption is justified when the drain sand fulfils the requirements of an ideal filter material. However, in practice it is doubtful whether such an ideal condition can be achieved. If the permeability of the sand is in the order of 500 1000 m/year, the effect of well resistance cannot be ignored, Hansbo (1982). In general, consideration for drain resistance has been given to the case where drain spacing is comparable to half-depth of the soil layer. For this case, the effect of the well resistance is not excessive, increasing the time required to obtain a particular degree of consolidation by about 25% by comparison with the ideal case of an infinitely permeable drain. Hansbo (1979) modified the equations developed by Barron (1948) for PVD applications. The modified general expression for average degree of radial consolidation, i.e. Eq. (2.22) becomes:

$$U_r = 1 - \exp\left[\frac{-8T_r}{F}\right] \tag{2.34}$$

and , $F = F(n) + F_s + F_r$ (2.35)

Where, F(n) = Effect due to spacing of drains

F_s= Effect due to smear

F_r= Effect due to drain resistance

Since PVDs do not have unlimited discharge capacities as assumed by Barron (1948) in case of sand drains. Hansbo (1979) developed well resistance factor as follows:

$$F_r = \pi z(L - z)\frac{k_h}{q_w} \tag{2.36}$$

Where z is the depth of the drain under consideration. L is the length of the drain having one-way drainage or half this value for two-way drainage, k_h is the coefficient of permeability in the horizontal direction in the undisturbed soil and q_w is the discharge capacity of the drain's longitudinal direction (in-plane), Park et al (www.geosynthetics.net/tech_docs/GeoAsia04Park).

However, in majority of the cases examined by Cassagrande and Poulos (1969), the half depth of the drained stratum was considerably in excess of the drain spacing and the internal resistance of the drains may well have made a significant contribution to the lack of acceleration of the consolidation process. The recent trend towards using band drains, of considerably smaller cross section than sand drains, and with drain lengths of up to 50 m in some cases, makes it imperative to consider the effect of the drain internal resistance. Atkinson et al (1982). Clogging should be an important consideration when the potential for internal resistance is analyzed. If the drain functions correctly in the very early stages of the consolidation, the excess pore water will contain a small proportion of soil particles which may collect within the drain and clog. The discharge capacity of the PVD will be a function of the effective lateral earth pressure against the drain sleeve. In the majority of the cases the filter will be partly squeezed into the channel system of the core by the pressure of the surrounding soil and this will reduce the channel volume and consequently the discharge capacity. The filter permeability should not be higher than required with respect to the discharge capacity. The filter permeability of the existing PVDs (which according to the results of laboratory tests have short-term discharge capacities of maximum 10-25 m^3/year) need not be higher than 0.01-0.05 m/year, Hansbo et al (1982).

2.7.7.4 Effect of Drainage Blanket on Sand Drains and PVDs

Drainage blanket is a layer of sand placed over the existing soft clay formation where sand drains or PVDs are to be installed. This blanket provides a working platform for vertical drains installing equipment. Its primary function is to provide a free draining outlet for the water discharged from the drains. In certain cases, where a large volume of soil is being drained, considerable quantities of water can be discharged into the drainage blanket, particularly in the early stages of consolidation.

2.7.8 Advantages of Vertical Drains

Vertical drains reduce the drainage path. So, it increases the rate of consolidation of the clay under load. Figures 2.12 and 2.13 demonstrate the difference in excess pore water

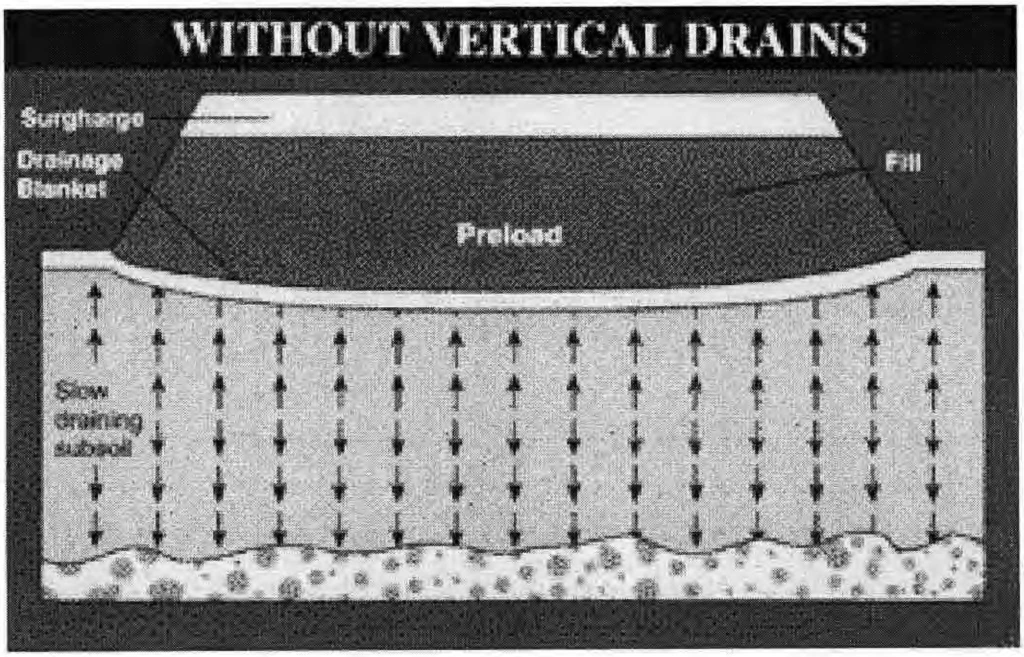

Figure 2.12: Slow Dissipation of Excess Pore Pressure only in the Vertical Direction

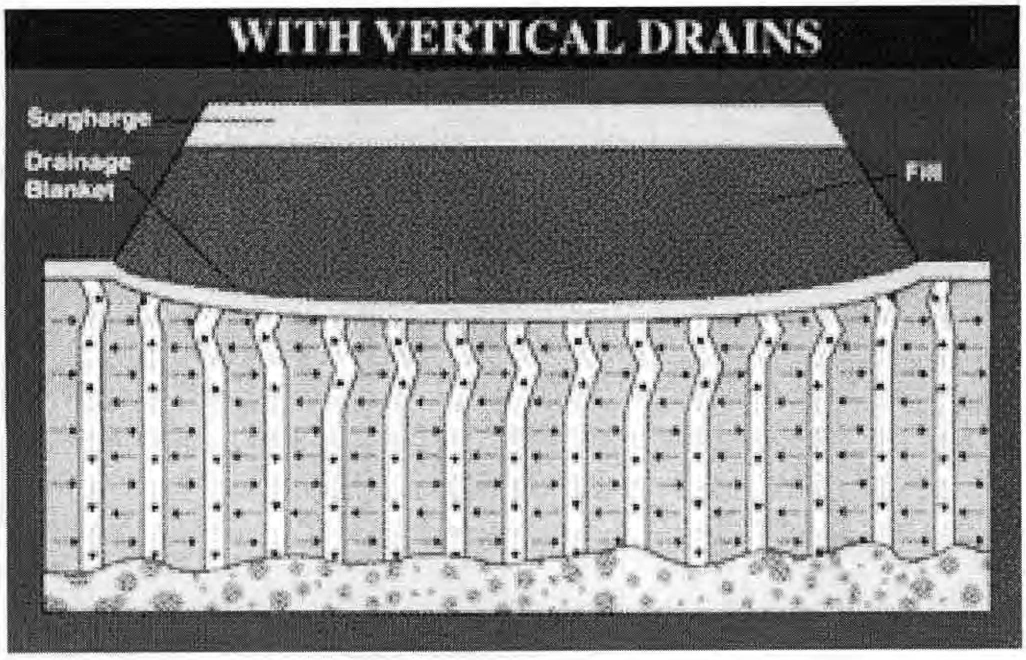

Figure 2.13: Quick Dissipation of Excess Pore Pressure Through Radial and Vertical Direction

dissipation in cases of without and with vertical drains. Besides, in non-homogenous soil, the horizontal (radial) permeability may be considerably greater than the vertical permeability. This anisotropy confers an additional advantage on the use of drains, Atkinson et al (1982). Following are the main advantages:

(1) Time required for primary consolidation can be appreciably reduced by the installation of vertical drains. As a result, structures like embankments can be constructed and can be commissioned much before the time that would have been required without the use of vertical drains. Hence, subsequent maintenance and overhead cost can be considerably reduced.

(2) With the accelerated rate of dissipation of excess pore water through vertical drains, shear strength also increases. This will enable application of subsequent stages of loading more rapidly than would not otherwise have been possible without the use of vertical drains. This increased rate of gain in shear strength will allow the use of heavy construction equipment. As a result rate of construction will be increased.

(3) Many soft clay strata contain thin bands, or partings, of silt or sand. Instability of embankments or tankage built on such strata is sometimes due primarily to the horizontal spread of excess pore pressure along these bands or partings, Atkinson et al (1982). Vertical drains relieve these excess pore pressures and thus avoid the occurrence of instability.

(4) When vertical drains are not used and embankments/structures are constructed without allowing sufficient time for considerable amount (say 90%) of primary consolidation, the serviceability of the embankments will be jeopardized. This will incur extra cost to the project later for reconstruction, renovation, repair, maintenance etc.

2.7.9 Limitations of Vertical Drains

Vertical drains accelerate primary consolidation only, because significant water movement is associated with it. Secondary consolidation causes only very small amounts of water to drain from the soil; secondary settlement therefore is not speeded up by vertical drains. Since vertical drains have virtually no effect on secondary settlement, that is why soils subjected to extreme magnitudes of secondary settlement, such as peat and organic soils, may not benefit considerably from vertical drains. Vertical drains will be only effective where soils are relatively impermeable. According to Rowe (1968) vertical drains may potentially be beneficial in soils with vertical coefficient of consolidation (C_v) of less than 3×10^{-7} m^2/sec (18 mm^2/min). Soils which are more permeable will usually consolidate under a surcharge at an acceptable rate on their own. Vertical drains are particularly effective where a clay deposit contains number of thin horizontal sand or silt lenses (so-called micro-layers). However, if these layers are continuous in a horizontal direction, only little may be gained from vertical drains under a surcharge of limited extent, since rapid drainage of the foundation material then occurs, whether the drains are placed or not, Hausmann (1990)

2.7.10 Property Requirements of PVD

PVDs consist of mainly a filter jacket and a water draining core. Synthetic PVDs are produced by number of manufacturers where filter jackets are synthetic geotextile conforming to number of mechanical and hydraulic properties of drains. Core is made of synthetic channel. But in case of jute PVD product varieties are yet to be developed. Few specimens have been developed that are made of jute and coconut coir. One type was made of two layers of DW twill filter jacket with twisted coconut coir rope in between. The other type was almost same with jute fibre rope in the middle, Kabir et al (1994). As synthetic PVDs are already in use even in Bangladesh, their full specifications namely, physical, mechanical, hydraulic, and endurance properties are already included in the tender and design requirements. However, specifications for jute PVDs are yet to be formulated. For the purpose of research works, specifications for PVDs have been taken as base values. One such important project where soft soil was encountered is Kaliakoir Bypass under Roads & Highways Department. Figures 2.14 to Figure 2.16 show the synthetic PVD and its installation methods. There were number of methods adopted to deal with the improvement of ground. Measures included the use of geosynthetic reinforcement, geotextile separator, PVDs, dewatering etc. The requirements for selecting PVDs were based on three common criteria those are as follows:

(1) Possibility to provide a safe installation of the drain, which is also dependent on the installation procedures.

(2) Long term performance of the drain, during preloading and later with respect to the following properties:

i. The drain should be sufficiently flexible to cope with the anticipated settlement of soil while maintaining continuity and without offering any significant support to the structure. The drain material should be inert and the drain should maintain its properties through the required period of consolidation.

ii. The drain should offer the minimum resistance to the passage of water from the surrounding soil without loss of fines from the soil, and without clogging the filter.

iii. The drain should be capable of transmitting water along its length without significant resistance to flow and should retain its required discharge capacity at the maximum specified working depth.

The PVD materials should conform to the specifications that are listed in the Table 2.5. The requirements for PVD listed in the Table 2.5 refer mainly to the properties of filter jacket and in few cases PVD system as a whole. Kabir et al (1994) have conducted some investigations on hydraulic properties of jute PVD. But a more number of tests have been conducted by number of researchers on physical, mechanical, and hydraulic properties of DW Twill, Canvass, Hessian and other jute products. Rao et al (1994), Mohy (2005). Prodhan (2008) and Abdullah (2008) have found out almost all the properties mentioned in the Table 2.5 and also many other physical, mechanical and engineering properties of jute and jute geotextiles. Since performance of PVD demands a good drainage capacity, in this research only the hydraulic conductivity that includes apparent opening size (AOS), permittivity (cross-plane permeability) and transmissivity (in-plane permeability) will be investigated. Tests results for the rest of the requirements will be used from Mohy (2005) to evaluate the compatibility of jute PVD with the specifications set out by RHD. Test methodology of hydraulic conductivity and their results including the tests results obtained from Mohy (2005) will be analyzed in Chapter 3 and 4.

2.7.10.1 Properties of PVDs Related to Hydraulic Conductivity

The functions of hydraulic conductivity demand the movement of pore water through the geotextile filter whereas, at the same time, it should serve the purpose of retaining

Table 2.5: Technical Specifications Set By R&H for Synthetic PVD (Kaliakoir Bypass)

Ser No	Properties of PVD Jacket & Core	Properties of	Test Designation	Requirements
1.	Apparent Opening Size, μm	Filter	ASTM 4751-87	< 90
2.	Grab Tensile Strength, KN	Filter	ASTM 4632-91	> 0.35
3.	Trapezoidal Tear Strength, KN	Filter	ASTM 4533-91	> 0.10
4.	Puncture Resistance, KN	Filter	ASTM 4833-88	> 0.10
5.	Burst Strength, KN	Filter	ASTM 3786-80 A	> 900
6.	Discharge Capacity at 7 days, 200 kPa at hydraulic Gradient of 1, m^3/year	Composite Drain	ASTM 4716-87	> 500
7.	Equivalent Diameter, mm	Composite Drain		> 50

GoB (1999)

the soil on its upstream side. Both adequate permeability retaining an open structure, and soil retention requiring a tight fabric structure are required simultaneously. A third factor is also involved, that being a long term soil-to-geotextile filter flow compatibility that will not clog excessively during lifetime of the system. Another important property of PVD as a system is the in- plane permeability (transmissivity) that is the way liquid is conveyed within the plane of their structure.

2.7.10.2 Filtration Properties of PVDs

Filtration is defined as the "equilibrium soil-geotextile system that allows for adequate liquid flow with limited soil loss across the plane of the geotextile over a service lifetime compatible with the application under consideration". Koerner (1997). Two of the important contradicting requirements in filtration are permeability and soil retention. These two parameters need further discussion:

Permittivity (Cross-Plane Permeability)

Any given soil mass consists of solid particles of various sizes with 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 may be defined as the property of soil which allows the seepage of fluids through its interconnected void spaces. A simple relation between the discharge velocity and the hydraulic gradient has been given by Darcy:

$$v = ki \tag{2.37}$$

Where, v = Discharge velocity of pore water

i = Hydraulic gradient

k = Coefficient of permeability

From this equation, another relation can be obtained for discharge rate (q):

$$q = kiA \tag{2.38}$$

Where, A = Cross-sectional area of the geotextile filter.

This type of permeability refers to cross-plane permeability when water flow is perpendicular to the plane of fabric. Some of the geotextiles used for this purpose are relatively thick and compressible. For this reason, the thickness is included in the permeability coefficient and is used as a permittivity which is given as:

$$\psi = \frac{k_n}{t} \tag{2.39}$$

Where, ψ = Permittivity,

k_n = Cross-plane permeability coefficient

t = Thickness at a specified normal stress

Soil Retention

When pore water flows through jute PVD filter openings. smaller soil particles are also carried along with the flow. At one time. this leads to an unacceptable situation called *soil piping*. in which the larger soil void spaces are left behind. The velocity of the flow then increases. accelerating the process. until the soil structure begins to collapse. The process is prevented by making the geotextile voids small enough to retain the soil on the upstream side of the fabric. It is the coarser soil fraction that must be initially retained and that is the targeted soil size in the design process. These coarse-sized particles eventually block the finer-sized particles and build up a stable upstream soil structure. There are many formulae available for soil-retention design. most of which use the soil particle size characteristics and compare them to the 95% opening size of the PVD filter, defined as O_{95} of the PVD filter. The ASTM describes this value as apparent opening size (AOS) and it is a dry-sieving method. The simplest of the design methods examines the percentage of soil passing a No. 200 sieve. According to the Task Force # 25, the following is recommended, Koerner (1997):

For soil ≤ 50% passing the No. 200 sieve:

$O_{95} < 0.60$ mm, that is, AOS of the fabric ≥ No. 30 sieve

For soil > 50% passing the No. 200 sieve:

$O_{95} < 0.30$ mm, that is, AOS of the fabric ≥ No. 50 sieve.

Beginning in 1972, a series of direct comparisons of geotextile-opening size (O_{95}, O_{50}, or O_{15}) was made in ratio form to some soil particle size to be retained (d_{90}, d_{50}, or d_{15}).The numeric value of the ratio depends on the geotextile type, the soil type, the flow regime and so on. For example Carroll (1983) recommends the following:

$O_{95} < (2 \text{ or } 3)\ d_{85}$

Where, d_{85} is the soil particle size in mm. for which 85% of the total soil is finer.

Long Term Flow Compatibility

One of the important issues in the PVD functions is the clogging of the channel or core of the PVD. The finer soil particles embed themselves in the core and a measurable reduction in permeability or permittivity occurs. There are guidelines available for non-critical. non-severe cases. but the question can be answered directly by taking a soil sample and the candidate PVD and testing them in the laboratory. Either gradient ratio (GR) test to see that the GR ≤ 3.0. long term flow (LTF) tests to see that the terminal slope of the flow rate versus time curve is adequate for site specific conditions. or a hydraulic conductivity ratio (HCR) with resulting HCR values between 0.7 and 0.3 should be performed. Koerner (1997). A different approach to the clogging issue is simply to avoid situations that have been known to lead to excessive clogging problems. The soil-geotextile compatibility assumes the establishment of a set of mechanisms that are in equilibrium with the flow regime being imposed on the system. There exist a number of possibilities, including upstream soil-filter formation, blocking, arching, partial clogging, and depth filtration.

2.7.10.3 Drainage Properties of PVDs

Pore water is collected from radial directions and PVDs convey this water through their core or channels. This type of flow is known as in-plane drainage function. Drainage is thus defined as the equilibrium soil-to-PVD system that allows for adequate water flow with limited soil loss within the plane of the PVD over a service lifetime with the application under consideration. Almost all geotextiles both synthetic and natural fibre provide some in-plane drainage function, but to widely varying degrees. For example, DW Twill, a filter material of jute PVD in isolation having, by virtue of their fibres crossing over and under one another, can transmit water within the spaces created at these crossover points but to an extremely low degree. When the DW Twill is used to make composite drain system, it's in-plane permeability as a whole increases many times.

Transmissivity (In-Plane Permeability)

In case of flow in the longitudinal direction of PVD. cross sectional area of the drain is a function of width and thickness of the PVD. With increasing normal stress on the PVD, it's thickness also decreases. For this reason, a new term for in-plane permeability is introduced called *transmissivity* which is as follows:

$$\theta = k_p t \tag{2.40}$$

Where. θ = transmissivity

k_p = in-plane permeability coefficient

t = thickness at a specified normal stress.

2.8 DURABILITY OF JUTE BASED ENGINEERING MATERIALS

Jute is a biodegradable natural fibre. The degradation takes place within ecological cycle, climatic conditions and soil properties. Though number of studies has been carried out on the physical, mechanical, hydraulic and other properties of jute, the aspect of durability has not been taken care of by many researchers. Some professional engineers expressed no hope for use of jute in civil engineering applications because of quick degradation of jute in soil-water interaction conditions. Some people comment that jute should have a minimum life of one season when it is addressed for a soil erosion problem whereas, the same for the purpose of drainage should not be less than the time required for the consolidation of the highly compressible soil. Rao, G. et al (1994) have conducted tests on durability of 4 types of jute geotextiles in both soil water environment, submergence in water and submergence in solutions of different p^H. In the first test they buried the samples in a 0.3 m deep pit in Indian Institute of Technology campus. The specimens were exhumed from time to time up to 2.5 months and tested for narrow strip tensile strength. Results showed that strength of the jute fabric as well as initial tangent modulus reduced with time. It was found that jute geotextiles when exhumed from the pit 2.5 months after the burial had strengths so small that they could not even withstand the handling. A number of black spots were observed and fungus growth visible.

In the second case, jute geotextile samples were kept submerged for four months in water and after drying they were again tested for narrow strip tensile strength. Their tests results are presented in Table 2.6. They saw that the reduction of strength was only 35%. The slower rate of reduction in strength in water than in soil indicates that the growth of particular micro-organisms that acted on jute in water are either different or less intense than those that acted on jute when it was in soil. This finding of Rao G. et al (1994) closely matched with the study of Ghose and Basu (1962) who observed reduced biological activity on jute under full submergence. So. jute geotextiles can be used to advantage for surface protection works in canals and river banks where jute comes in contact with a 100% saturated environment. In the third case. jute samples were submerged in aqueous solution of p^H ranging from 4.5 to 9.0. These samples were periodically tested for 14 days. They saw that maximum strength reductions for all types was at $p^H = 5.2$. This result also conformed to the findings of Ghose and Basu (1962) that fungus attack on jute is most severe when p^H is less than 5.8. Thus the lower p^H ranges of acidic environment are most detrimental to the jute fabric

Table 2.6: Reduction in Strength of Jute Geotextiles after 4 Months Submergence in Water

Type	Direction of Yarn	Reduction in Strength, %
A	MD	35
A	XMD	25
B	MD	22
B	XMD	43
C	MD	36
C	XMD	40
D	MD	45
D	XMD	38

Type A = B-Twill, Type B = A-Twill, Type C = Hycee cement, Type D = DW Plain, MD = Machine Direction and XMD = Cross-Machine Direction, After Rao G. et al (1994).

Prodhan (1994) conducted study on the treatment of jute products for increasing their life in the biotic environment. Abdullah (1999) carried out study on various types of jute product with a view to reducing the biodegradability. He prepared few types of jute geotextiles treated with various chemical composition designated by Treatment I, Treatment II. Treatment III and Treatment IV. All these samples were tested for biodegradability, durability, moisture holding capacity in a standard laboratory environment. The results are shown in Table 2.7. It is seen that increasing the durability can be achieved by adopting suitable treatment. In fact. life of jute based product can be extended by controlling the treatment methodology and this can be increased up to or more than even 20 years. According to these findings it is concluded that jute can be treated to convert to "Design Biodegradable" material.

Table 2.7: Biodegradability, Durability, Moisture Holding Capacity of Treated and Untreated Jute

Type of Product	Biodegradability		Durability, Time in Years	Moisture Holding Capacity, %
	Time in Year	Weight Loss, %		
Light Weight Hessian	0.25	30	0.25-0.80	-
Treatment I	1	15	0.50-1.25	9-10
Treatment II	1	10	2.0-5.0	6-8
Treatment III	1	5	>10	5-6
Treatment IV	1	1-3	>10	3-4

After Abdullah (1999)

Figure 2.14: Synthetic PVD Used in the Kaliakoir Bypass, Bangladesh

Figure 2.15: Flexi Drain Installation Method in Kaliakoir Bypass

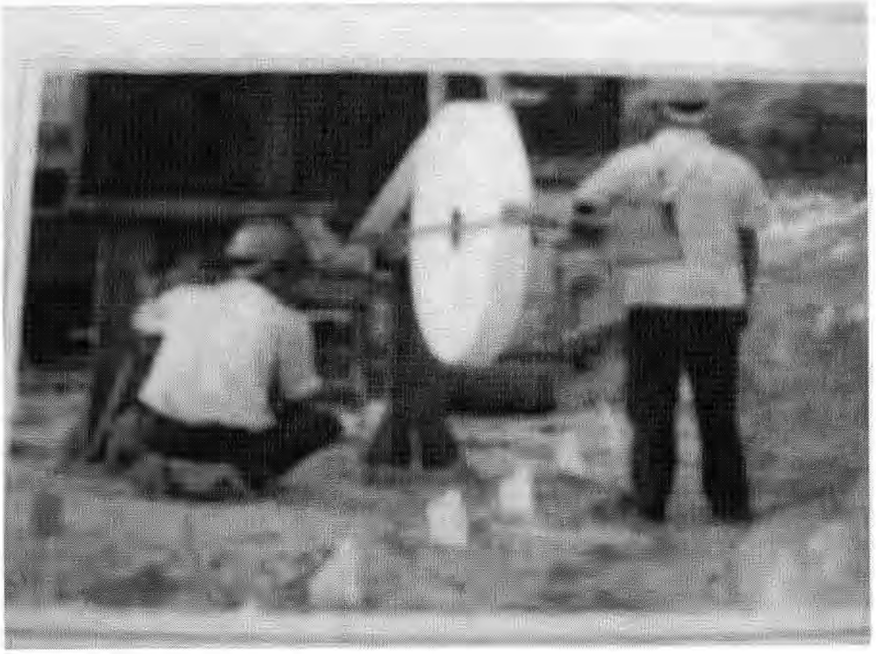

Figure 2.16: Flexi Drain Installation Process in Kaliakoir Bypass

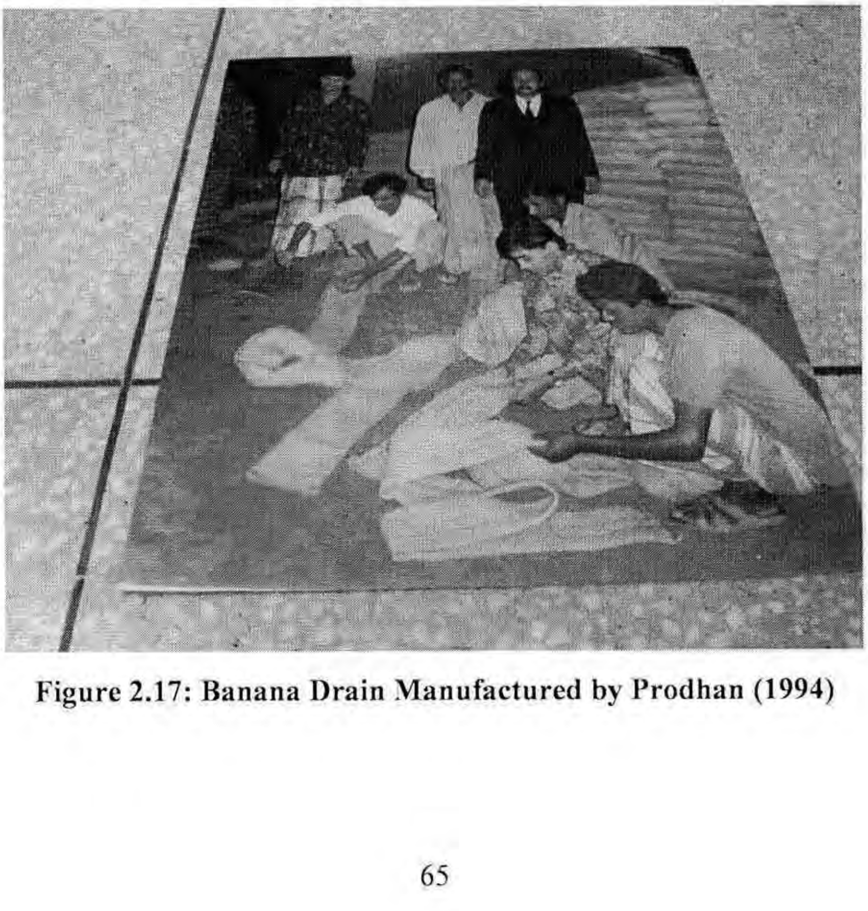

Figure 2.17: Banana Drain Manufactured by Prodhan (1994)

CHAPTER 3

EXPERIMENTAL INVESTIGATIONS

3.1 GENERAL

The investigations in the laboratory were conducted on soft clay collected from Mirpur Cantonment side of Zia Colony to Mirpur Cantonment link road project, Dhaka in order to find out various consolidation characteristics and to analyze time-settlement behaviour. The fabrication of testing equipment, sample collection, preparation and test methodologies are discussed in this chapter. Settlement was observed in the steel tank without vertical drains. Then in-soil performance of synthetic PVD and jute PVD was evaluated in terms of settlements that occurred, and compared with that of first phase. Hereinafter, in this paper, this 'in-soil performance test' of synthetic PVD and jute PVD will be referred to as 'performance test'. The total work carried out in this research is presented in the form of flow chart in Figure 3.1.

3.2 COLLECTION OF SOIL SAMPLES

Construction of 6.3 km long link road project between Zia Colony of Dhaka Cantonment (Opposite to Hotel Radisson Water Garden) and Mirpur Cantonment is being executed by 16 Engineer Construction Battalion (16 ECB) of Bangladesh Army. The project has been sponsored by Dhaka City Corporation (DCC) under Ministry of Local Government Rural Development (LGRD). About 5.0 km length of this 6.3 km road project passes through low land and marshy areas. water bodies. fish ponds etc. where. soft clay soil exists at different layers. DCC (2007). For laboratory testing. soil sample was collected as per ASTM D 420-87 at a chainage of 05-600 km which remains submerged during monsoon. For collection of soil sample. approximately 2 m by 2 m area was excavated to a depth of 2 m to 2.5 m using hand shovels. Proper care was taken to remove any loose material. debris. coarse aggregates and vegetation from the top of the excavated pit. Around 400 kg of samples was collected from the bottom of the excavated pit. Samples were also collected to assess the field unit weight and

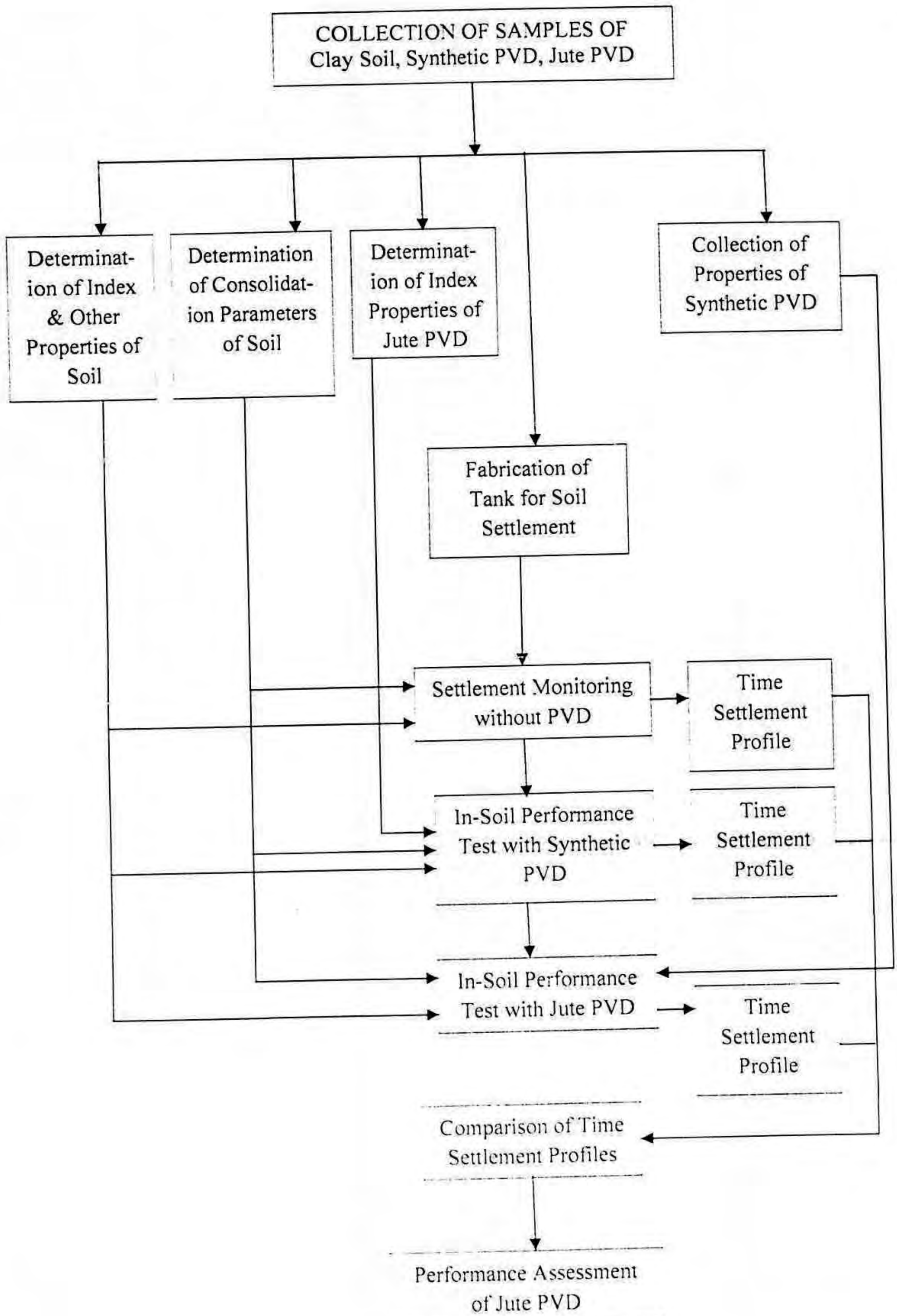

Figure 3.1: Flow Chart of Test Programme

natural water content. All samples were packed in used cement bags and were finally transported to the BUET Geotechnical Laboratory.

3.3 COLLECTION OF SYNTHETIC PVD

For accelerating the settlement, synthetic prefabricated vertical drains (synthetic PVD) are already being used in some of the projects in Bangladesh. Kaliakoir Bypass is one the projects where 'Flexi Drain', a type of synthetic PVD has been used by Abdul Monem Ltd, a contractor of this project. For the laboratory investigation of this product, 2 m of synthetic PVD has been collected from Abdul Monem Ltd. It has got a drain body, a central continuous plastic core made of high molecular polypropylene specially designed to provide high discharge capacity. The core configuration conforms to a corrugated extruded type. The core functions as a free-draining water channel. It also has filter jacket which surrounds the drain body, is a strong and durable non-woven spun-bonded polypropylene filter with high permeability and effective filtering properties allowing free access of pore water into the core while eliminating the movement of soil particles and preventing piping. It is of 100 mm wide and 3.2 ± 0.03 mm thick (Design guide of Flexi-Drain). Its colour is white. Flexi-Drain is available in different core configurations and filter fabrics to suit various soil conditions and engineering practices. Figure 3.2 and 3.3 show the photographs of Flexi-Drain roll, core configuration, filter sleeve etc.

3.4 COLLECTION OF JUTE PVD

In Bangladesh, for the production and development of Jute products, Bangladesh Jute Research Institute (BJRI) and Bangladesh Jute Mills Corporation (BJMC) are the overall responsible organizations. Because of pioneering role in the jute sector by Bangladesh, International Jute Study Group (IJSG) and Jute Diversification Promotion Centre (JDPC) are also located in Bangladesh. Considering the climatic condition, BJRI has developed as many as 50 types of jute products by blending with hydrophobic fibre like coir or by modification with bitumen, latex, wax or resinous materials. Mohy (2005). Jute prefabricated vertical drain (jute PVD) is not produced and marketed commercially neither in Bangladesh nor in any other countries of the world. For the purpose of civil engineering research, sample was developed by BJRI and was prepared in the then Adamjee Jute Mills. After the layoff of this Mill, the machines have been transferred to Latif Bawany Jute Mills located at Demra, Dhaka. Recently in the month of July 2008, BJMC was contacted to produce samples of jute PVD. BJMC had a piece of jute PVD prepared by Latif Bawany Jute Mills and cost estimation has also been done. However, in the laboratory investigation, jute PVD that was made previously in the Adamjee Jute Mills has been used. It is of 90 mm wide and 7 mm thick. This jute PVD consists of four strands of coconut coir (each of 3 mm diameter) covered with jute burlap. The jute burlap used is known as DW (Double Works) Twill according to BJMC specification. In normal uses, this DW Twill is extensively used for making sack. Figure 3.4 and Figure 3.5 show specimens of jute PVD used in the test.

3.5 LABORATORY TESTING PROGRAMME

In order to examine the engineering characteristics of soil samples, a comprehensive laboratory investigation programme was undertaken. Details of laboratory tests carried out, standards followed; type of samples tested and number of tests performed is presented in Table 3.1.

3.6 DETERMINATION OF INDEX AND OTHER PROPERTIES OF SOIL

3.6.1 Soil Sample Preparation

Soil sample collected from field in the month of December contained high water content. Because water table was just 5 to 7 feet below the ground level in that season, the collected soil was nearly saturated. This soil was air-dried in the BUET Geotechnical Laboratory for four weeks until the water content reached to 4 to 5 percent from its natural water content of 38.3%. Soil lumps were broken manually with wooden hammer. Before carrying out routine tests like index properties, soil was further oven dried. Other laboratory investigations were carried out on remolded soils. For conducting actual performance test in the steel tank, soil slurry was made by mechanical mixer.

Table 3.1: Details of Test Programme

Tests Type	Type of Samples	Test Standard	No of Samples Tested
Field unit weight	Field sample	-	01
Natural moisture content	Field soil sample	ASTM D 2216	03
Liquid Limit	Dry soil	ASTM D 4318	02
Plastic Limit	Dry Soil	ASTM D 4318	02
Grain size/Hydrometer Analysis	Oven dried soil	ASTM D 421-2	01
Oedometer consolidation	Remolded sample	ASTM D 4186	03
Vane Shear test	Remolded sample	BS 1377	06
Water content determination	Remolded sample	ASTM D 2216	06
Unconfined compression Test	Consolidated remolded	ASTM D 2166	06
Physical and engineering properties of Jute PVD (thickness, AOS permittivity, transmissivity)	Samples collected from BJMC	ASTM D 5199 ASTM D 4751 ASTM D 4491 ASTM D 4716	22
Consolidation Settlement without vertical drains	Soil slurry in settlement tank	-	01
In-soil Performance of Synthetic PVD	Soil slurry in settlement tank	-	01
In-soil Performance of Jute PVD	Soil slurry in settlement tank	-	01

3.6.2 Determination of Field Unit Weight of the Soil Sample

A metal cylinder having a height of 304.8 mm (12 inch) and an internal diameter of 185 mm was used for this test. Both the ends were open. After clearing the debris and removing the top soil. the metal cylinder was vertically pushed downwards. The pushing was continued up to a soil filling height of 240 mm. Immediately after extracting the cylinder filled with soil from the site; its both ends were waxed to prevent moisture loss. Figure 3.6 shows a metal cylinder used for unit weight measurement. The sample was taken to the BUET Laboratory and weighed. Then the unit weight was found out from the simple calculation of weight-volume relationship.

3.6.3 Natural Water Content of the Soil Sample

After carrying out the field unit weight test, wax was removed from the metal cylinder. Immediately three specimens of small quantity (approx 20 gm each) were weighed and placed in the oven for measuring the water content. Water content was Determined according to ASTM D-2216.

3.6.4 Index Properties of Soil

The disturbed soil samples collected were dried in air for about four weeks and soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. About 500 gms of soil sample was oven dried. The required quantities of soil were then sieved through sieve No. 40 (0.425 mm). The tests carried out to determine the physical and index properties of the disturbed soil included specific gravity, grain size analysis, Atterberg limits tests. The different fraction of sand, silt and clay of sample was found following the MIT Textural Classification System (1931). The soils were classified according to Unified Soil Classification System (ASTM D 2487) and AASHTO classification system.

3.6.4.1 Atterberg Limits Tests of Mirpur Soils

Tests for finding out liquid limit, plastic limit and plasticity index of the samples were carried out on air-dried pulverized samples. The Cassagrande liquid limit device was used for the liquid limit test. Pulverized soils passing through sieve No. 40 (0.425 mm) were used for this test. Care was taken to prepare uniform soil-water paste. By plotting the moisture content versus number of blows in the semi log graph paper, a best fit line was obtained. Water content corresponding to 25 blows determined the liquid limit of the soil. Liquid Limit test was conducted according to ASTM D-4318. For finding out the plastic limit, approximately 50 gms of representative air- dry soil passing through No 40 Sieve was taken. Three specimens were tested for plastic limit following ASTM D-4318.

3.6.4.2 Specific Gravity Test

Specific gravity of the soil sample was determined following ASTM standards D-854. A volumetric flask of 500 ml capacity filled with water was weighed. 50 gms of air-dried powdered soil was put inside the flask and water was added. This soil water mixture was then placed in a boiling container and agitated for 10 to 15 minutes. Vacuum was applied by a pump for removing the entrapped air. After the temperature of the solution came down to room temperature, water was added to bring the level up to 500 ml mark and weight was again recorded. Soil and water was poured into an evaporating dish of known weight and kept for one day for sedimentation. Next day clean water from the upper portion was removed by plastic squeeze bottle and then the evaporating dish with soil water mixture was placed in the oven for 24 hours. Then the mass of the dry soil was determined and specific gravity found out.

3.6.4.3 Sieve and Hydrometer Analysis

In order to analyze the grain size and classify the soil. first of all, sieve analysis following ASTM D-421 was carried out. Approximately 100 gms of oven dried soil was broken into individual particles using mortar and wooden pestle. A stack of sieves was prepared with US sieve #200 at the bottom. The pulverized soil was weighed and then poured on the top of the stack. After shaking the stack of sieves for 10 to 15 minutes, sieves were separated. Soil retained on each sieve and bottom pan was measured. It was found that around half of the quantity of soil passed #200 sieve and rest was retained on the #200 sieve. This retained soil was wash-sieved and only 5% was still retained on the sieve. To carry out hydrometer analysis, a sample of 50 gm oven-dry. well pulverized soil was dispersed in 1 liter of water in a graduated cylinder. After few drop of deflocculating agent was added. the suspension was thoroughly shaken and cylinder was kept upright on plane horizontal table. A hydrometer was inserted in the cylinder and readings were recorded at an interval of 2, 4, 8, 15, 30 minutes and so on. Each time the hydrometer was taken out from the suspension and kept inside another cylinder filled with normal water. The readings were plotted on a semi-log paper and computations were made. Percent finer than 0.002 mm size gave the clay-size fractions. Soil diameters between 0.06 mm and 0.002 mm were silts. This test was conducted following ASTM D-422.

3.7 DETERMINATION OF CONSOLIDATION PARAMETERS

The oedometer consolidation test is usually performed on saturated clayey soils to study the load deformation characteristics of soil mass as a function of time and excess pore pressure. This type of consolidation is one-dimensional; all water-flow and soil movement are in the vertical direction. The two most important soil properties furnished by a oedometer test are (1) the compression index, C_c which indicates the compressibility of the specimen, and (2) the coefficient of consolidation, C_v, which indicates the rate of compression under a load increment. An oedometer consolidation testing programme was chosen for this study in order to investigate the consolidation characteristics of soft Mirpur clay, to use the data to compare the actual settlement in the steel tank. The main focus was to find out the e-logp relationship, volumetric strain, compression index, and coefficient of consolidation. Oedometer consolidation tests were accomplished according to the procedures outlined in ASTM D-2435.

3.7.1 Oedometer Test

Oedometer test and soil settlement in the steel tank were conducted simultaneously. One inch height (25.4 mm) and 2.5 inches (63.5 mm) diameter cylindrical sample was taken from the soil slurry actually prepared for settlement test. Water content determinations were made from the soil slurry since same water content was maintained for both oedometer and settlement tests. The density of soil was also maintained same in both the cases which were 16.51 KN/m^3. The bottom porous stone was placed on the oedometer base and the sample was positioned on the porous stone very carefully. Soaked filter paper and saturated porous stones were used at both ends of the sample. In this test fixed ring container method was used i.e. under applied pressure soil specimen movement was downwards relative to the container. The carefully placed samples were then assembled in the oedometer. Water was added to the consolidation unit to submerge the soil and to keep it saturated. A small seating load of 5 kPa was used to place the equipment in compression before the displacement-recording device was set. Displacements were recorded by deformation dial gauge (readability 0.0001 inch). Since the actual settlement in the steel tank was studied in a low stress condition (approximately less than 30 kpa), oedometer consolidation test was also conducted with low to medium range of stresses. In this test, stresses were applied in the sequence of 10, 20, 40, 100, 200, 100, 50 and 5 kPa. Unload-reload cycle was not studied. The loads were kept unchanged for 24 hours. At the end of testing programme the soil sample was taken out from the oedometer, weighed, oven dried and weighed. After plotting the reading, useful parameters like initial void ratio, compression index, coefficient of consolidation, t_{50}, t_{90}, e-logp relations and c_v-logp relations were determined. Both Taylor and Cassagrande methods were used to calculate t_{90} and t_{50} respectively in order to find out coefficient of consolidation (c_v).

3.8 PROPERTIES OF SYNTHETIC PVD

'Flexi Drain', a type of synthetic PVD has been used for performance test in the steel tank. This is collected from Abdul Monem Ltd, a contractor in the Kaliakoir Bypass Road Project. No tests for engineering properties of synthetic PVD have been conducted in this research. Test properties of this drain have been provided by the manufacturer to the user organization. A list of properties is tabulated in the Table 3.2.

Table 3.2: Properties of Flexi Drain (Synthetic PVD)

Properties of PVD Jacket & Core	Properties of	Test Designation	Synthetic PVD Flexi Drain
Apparent Opening Size, μm	Filter	ASTM 4751-87	75
Grab Tensile Strength, KN	Filter	ASTM 4632-91	0.50
Trapezoidal Tear Strength, KN	Filter	ASTM 4533-91	Not mentioned
Puncture Resistance, KN	Filter	ASTM 4833-88	0.144
Burst Strength, KN	Filter	ASTM 3786-80 A	950 (kPa)
Discharge Capacity at 7 days, 200 kPa at Hydraulic Gradient of 1.0, m^3/year	Composite Drain	ASTM 4716-87	>2500
Equivalent Diameter, mm	Composite Drain	$d_e=2(b+t)/\pi$	65.70

3.9 DETERMINATION OF PROPERTIES OF JUTE PVD

PVD needs to conform to a set of requirements before it is accepted for use by the sponsor organizations. One such specification by Roads and Highways Department of Bangladesh for Kaliakoir Bypass Road Project has already been cited in Chapter 2 of this paper. Those requirements include properties of both filter fabric and drain system as a whole. Mohy (2005) has carried out laboratory tests on jute products which satisfy all those requirements of filter fabric. Besides, he has found out number of other physical, mechanical and endurance properties. This study will cover tests methodologies related to hydraulic conductivity such as thickness, apparent opening size (AOS), permittivity (cross-plane-permeability) of filter fabric and transmissivity (in-plane permeability) of filter fabric & also jute PVD system.

3.9.1 Thickness of Filter Fabric of Jute PVD

Jute PVD consists of filter fabric made of DW Twill and core material made of coconut coir or jute rope. To find out the thickness of PVD as a whole, first of all, method of finding out thickness of DW Twill is discussed. Thickness of geotextile or jute based products is a physical property. This property is often mentioned in the specification for geotextiles. In certain industrial applications, the thickness may be rigidly controlled within specified limits. Bulk and warmth properties of jute based products are often estimated based on their thickness measured before and after abrasion or shrinkage. Mohy (2005). Though, thickness is not indicative of field performance and therefore is not included in the specifications (Kaliakoir Bypass), this value is required in the calculation of some geotextile and geomembrane parameters such as permeability coefficients, permittivity, transmissivity, tensile strength (Index) etc.

Thickness is measured as the distance between the upper and lower surfaces of the fabric, measured at a specified pressure. Koerner (1997). The thickness of geotextiles and geomembranes may vary considerably depending on the magnitudes and the duration of pressure applied. When pressure is increased, thickness decreases. ASTM D 5199-98 specifies that specific sample size and applied pressure should be indicated to ensure all results are comparable. According to this standard, thickness of geotextile should be measured to an accuracy of at least 0.02 mm under pressure of 2.0 ± 0.02 kPa. The thickness testing instrument should have the thickness gage having a base (or anvil) and a free moving presser foot plate whose planar faces are parallel to each other. A gage with a 56.4 mm diameter presser foot (the base should extend at least 10 mm in all directions further than the edge of the 2500 mm^2 circular presser foot) should be used for measurements of geotextiles. This instrument is capable of measuring a maximum thickness of at least 10 mm to an accuracy of at least $\pm$ 0.02 mm.

Test specimens are removed from DW Twill collected from BJMC in a randomly distributed pattern across the width with no specimen taken nearer than 100 mm from the roll edge. From each unit of DW Twill, tests specimens are cut such that the edge of the specimens extends beyond the edge of the presser foot by 10 mm in all directions. Total 10 specimens are taken for the thickness measurement of DW Twill.

An important consideration is to check whether the strain gauges have truly come into equilibrium under placement. The introduction of a weight would induce compression of the fabrics. Thus it is important to read the gauges as quickly as possible once the presser plate is deemed to have come to equilibrium at the standardized duration. According to ASTM D 5199-98, suggested time interval for recording the dial reading is 5 second only whereas the thickness test carried out by Muhammad (1993) shows that jute fabrics continues to compress and it takes about 2 minutes to come to equilibrium, Mohy (2005). Thus the average thickness, T_{avg} is calculated from the summation of thicknesses of all specimens (total 10) divided by the number of specimens. Muhammad (1993) has also tested thickness by taking number of layers of DW Twill together under a wide range of stresses. The average thickness, T_{avg} was calculated from total thickness, T_{total}, when number of layers tested, N, by

$$T_{avg} = T_{total} / N \tag{3.1}$$

Where, T_{avg} = Average thickness of all specimens

T_{total} = Total thickness of number of individual specimens

N = Total number of specimens.

Figure 3.7 shows the thickness measurement instrument and thickness measuring procedure.

3.9.2 Apparent Opening Size (AOS) of Filter Fabric of Jute PVD

The procedure for measuring the apparent opening size was developed by the Corps of Engineers of USA Army to evaluate woven geotextiles. The test has been extended to cover all geotextiles including the non-woven types. AOS is a property of geotextile that determines the soil retention capability and at the same time allowing water to flow. The ideal retention criteria for fabrics should specify an appropriate fabric pore structure in order to provide adequate seepage and to prevent piping in the soil and clogging in the fabric. The AOS is defined as the U.S. standard sieve number that has openings closest in size to the openings in the geotextiles. The equivalent ASTM test is designated as D 4751. The test uses known diameter glass beads and determines the O_{95} size by standard dry-sieving. Sieving is done using beads of successively larger diameters until the weight of beads passing through the test specimen is 5%. This defines O_{95} size of geotextile's opening in millimeters. It may be noticed here that O_{95} value only defines the one particular void size of the geotextiles and not the total pore size distribution.

Calhoun (1972) developed a test for equivalent opening size (EOS) to determine the soil particle retention ability of various fabrics. The test involves in the determination of the size of the rounded sand particles which when sieved through the fabric will pass only 5% or less by weight. The EOS is defined as the "retention on" size of that fraction expressed as U.S. standard sieve numbers. The EOS test only provides a method for determining the relative size of the largest straight through openings in a fabric. Two fabrics may have similar values but dramatically different pore sizes and porosities, for example, those found in woven and non-woven fabrics. The AOS, EOS and O_{95} all refer to the same specific pore size, the difference being that AOS and EOS are sieve numbers, while O_{95} is the corresponding sieve-opening size in millimeters.

As per the ASTM D 4751, a geotextile specimen as shown in Figure 3.8 is placed in a mechanical sieve frame, and then standard glass beads are placed on the geotextile surface. A mechanical sieve shaker shakes the geotextile and the frame laterally. It imparts lateral and vertical motion to sieve, causing the particles thereon to bounce and turn so as to present different orientations to the sieving surface. The procedure is then repeated on new specimens of the same type of geotextile with various sizes of glass beads until its AOS is determined. AOS is that bead size for which 5% or less of the beads pass through the fabric.

In this research, from a roll of DW Twill, initial 1m is discarded. Then a full width swatch 1m long from that roll is taken for preparing the test specimen. Total 4 specimens of 280 mm X 280 mm size are cut to fit the appropriate sieve pan. In this test, sand of different sizes is used instead of glass beads. Sand sizes include (1) passing sieve # 16 but retained on sieve # 30, (2) passing sieve # 30 but retained on sieve # 50, (3) passing sieve # 50 but retained on sieve # 100, (4) passing sieve # 100 but retained on sieve # 200. 50 gms of sand is used for every size. The passed fraction of sand is captured in the pan which is then weighed and fraction passing determined.

Many geotextiles do not have surface films and in general natural geotextiles may not build up much static current during shaking. According to ASTM for this test, the geotextile has to be changed after using a particular uniform size of glass beads or sand to maintain the jute fabric opening at each time of testing. But in this test all four sizes of sand are used on one specimen to evaluate the AOS value. In this way all four specimens are tested. As alternative to dry-sieving method, there are a number of wet-sieving methods. The ISO/DIS 12956 test is a wet-sieving test and will be seeing greater use than dry-sieving in the future. In general, the wet-sieving tests avoid many of the problems of dry-sieving and are more representative of site conditions, Koerner (1997).

3.9.3 Permittivity (Cross-Plane Permeability) of Filter Fabric of Jute PVD

Permeability in the cross-plane direction of jute PVD filter is an important property. This cross-plane permeability through the DW Twill filter fabric is quantified and named as "Permittivity" and designated as Ψ. Permittivity is an indicator of the quantity of water that can pass through a geotextile (or jute based filter fabric) in an isolated condition. As per ASTM D 4439-98, permittivity (Ψ) of geotextile is defined as the volumetric flow rate of water per unit cross sectional area per unit head under laminar flow condition in normal direction through that geotextile. As it is already discussed in determination of thickness that geotextile/jute geotextile is considerably compressed under load, thickness is included in its determination. Thus Eq. (2.39) is rewritten here:

$$\Psi = \frac{k_n}{t} \tag{2.39}$$

Using this equation in the Darcy's formula as follows:

$$q = k_n iA$$

$$= k_n \frac{\Delta h}{t} A$$

$$\frac{k_n}{t} = \Psi = \frac{q}{(\Delta h)(A)} \tag{3.2}$$

Where,

q = flow rate, m^3/s

i = hydraulic gradient (unit less)

Δh = total head loss (m)

A = total area of DW Twill test specimen (m^2)

This test can be conducted in two ways, namely; constant head and falling head method. The ASTM D 4431-98 specifies the standard for constant head of 50 mm. The important test consideration for this test are preconditioning of the fabric, temperature and the use of de-aired water. ASTM D 4491 requires a dissolved oxygen content of less than 6.0 mg/l. Tap water is allowed unless dispute arises, in which case de-ionized water should be used. The constant head test is used when the flow rate of water through the geotextile is so large that it is difficult to obtain readings of head change versus time. In the falling head test, a column of water is allowed to flow through the filter fabric and the readings of head changes versus time are noted. In this case, the flow rate of water through the filter fabric must be slow enough to obtain accurate readings.

As per the ASTM standard the minimum diameter of specimen should be 25 mm. To make the test more representatives of field conditions, numerous attempts to construct a permittivity-under-load device have been made. Generally, a number of layers of geotextile are placed upon one another with an open mesh stainless steel grid on top and bottom. This assembly is placed inside a permeameter and loaded normally via the ceramic balls of approximately 12 mm diameter. Though the normal stress is imposed on the geotextile, flow is nominally restricted. Loading by soil itself (which would definitely affect flow) is completely avoided. The test is standardized by ASTM D 5493.

For the preparation of specimens, 1 m^2 of representative DW Twill is taken and specimens having diameter of 104 mm are cut with a cutting machine of BUET Geotechnical Laboratory so that these fit in the testing apparatus. Total 20 pieces are cut which are then conditioned by soaking in closed container of de-aired water, at a room conditions, for a period of 24 hours. Total height of 20 pieces of DW Twill is measured with a scale. After setting a head difference, flow is allowed to occur in the permeameter. Time duration is taken for collecting 1000 ml of water. With the same head difference, the time durations for collecting 1000 ml of water are recorded for another three times. From the average of time durations and discharge (1000 ml), flow rate q (m^3/s) is found out. The same procedures are repeated for other three head differences. The test methods and conduct of test are shown in the Figures 3.9.

3.9.4 Transmissivity (In-Plane Permeability) of Filter and Jute PVD System

In-plane permeability is an important property of any PVD. Water is collected radially through cross-plane filtration but it is the in-plane drainage property of PVD that takes the water out of the soil. For the flow of water within the plane of the geotextile (e.g. in the utilization of the drainage function), the variation in geotextile thickness (its compressibility under load) is a major issue. Eq. (2.40) is repeated here to explain the in-plane permeability of PVD.

$$\theta = k_p t \tag{2.40}$$

But according to Darcy's Law,

$$q = k_p i A$$

$$= k_p i (W x t)$$

$$k_p t = \theta = q/iW \tag{3.3}$$

Where,

θ = transmissivity of geotextile (m^3/s-m)

k_p = permeability (hydraulic conductivity) in the plane of the geotextile (m/s)

t = thickness of the geotextile

q = flow rate (m^3/s)

W = width of the geotextile (m)

i = hydraulic gradient (dimensionless) = $\Delta h/L$.

Δh = total head loss

L = length of geotextile (m).

As per ASTM D 4716-95, hydraulic transmissivity for a geosynthetic is the volumetric flow rate per unit width of specimen per unit gradient in a direction parallel to the plane of the specimen. A number of test devices are configured to model the Eq. (2.40), where liquid flows in the plane of the geotextile in a parallel flow trajectory: ASTM 4716 and ISO/DIS 12958 use such a device.

This test method is intended either as an index test or as a performance test and used to determine and compare the flow rate per unit width under specific conditions. This test method may be used as an index test for acceptance testing of commercial shipments of geosynthetics but it is advised that information regarding multi-laboratory precision of this test is incomplete. The hydraulic gradient (s) and specimen contact surfaces are selected by the user either as an index test or as a performance test to model a given set of field parameters as closely as possible. Measurements may be repeated under increasing normal stresses selected by the users. Hydraulic transmissivity should be determined only for tests or for specific regions of tests that exhibit a linear flow rate per unit width versus gradient relationship, which is laminar flow.

The test set-up consists of a base and a reservoir. The sturdy metal base has smooth flat bottom and sides are capable of holding a test specimen of sufficient area and thickness. The reservoir is a clear plastic or glass extending the full width of the base. The height of the reservoir is at least equal to the total length of the specimen. A catch trough extending the entire width of the base is provided for collection and measurement of the outflow from the specimen, Mohy (2005). The system is connected with the water flow and facilities are available for varying the hydraulic head. In the reservoir there is also an extra outlet at the top to allow removing of water if anytime head is increased. Normal stresses are applied and varied by using weight disc of 10 kg, 5 kg, and 2 kg along with loads of other accessories. The schematic drawing of test assembly and the sample preparation are shown in Figure 3.10 to Figure 3.12.

This test is carried out in two phases; firstly with only DW Twill and secondly with the composite drain system as a whole. Specimens from DW Twill are cut such that the longer dimension is parallel to the Twill's directions to be tested. According to ASTM, the width and length should be kept as 300 mm and 300 mm respectively, or the length to allow the specimen to extend into the reservoir and weir a distance of 25 mm, whichever is greater. However, in the BUET Geotechnical Laboratory the instrument fabrication is such that it allows a size of the specimen to be 165 mm X 100 mm. So, specimens of the latter size are cut to fit in the test set-up. The thickness of the DW Twill has already been found out in the test that has been conducted (2.38 mm). This piece of specimen is then covered with yellow colour rubber membrane with super glue so that water can only flow through the specimen, not in between the specimen and the membrane. The first test is carried out at varying hydraulic gradient but with same normal stress of 20 kPa. Time is recorded for collecting 100 ml of water. The same procedure is repeated for four times with each hydraulic gradient. In the second phase, jute PVD system as a whole is tested for transmissivity. The width of the specimen closely resembles the instrument set-up i.e. 90 mm. The length is cut as earlier one i.e. 165 mm. The thickness of the jute PVD as a whole is 7 mm. This specimen is again fixed with rubber membrane to make it water tight so that system allows only in-plane flow. As per the ASTM method, this test is performed using a minimum of three applied normal stresses selected from the values of 10, 25, 50, 100, 250 and 500 kPa. Normally three gradients are selected from 0.05, 0.10, 0.25 and 1.0. In the Kaliakoir Bypass Road project, Roads and Highways Department of Bangladesh has specified this test to be conducted with hydraulic gradient of 1.0 which is tried to maintain in this test. Normal stress in this test is varied between 2 kPa to 27 kPa. Discharge capacity at hydraulic gradient of 1.0 and normal stress of 20 kPa is obtained from the graphical plot. Figure 3.13 to Figure 3.14 show the test method of single layer of DW Twill and jute PVD system respectively.

3.10 PERFORMANCE TEST OF PVDs

Soil settlement behaviour was studied in three separate phases. In the first phase, soil slurry was placed in the steel tank and put under surcharge load without using any vertical drains. The total settlement achieved within a time period was recorded and a settlement profile was obtained. In the second phase, in-soil performance of synthetic PVD was investigated and time settlement profile was evaluated in the same way. In the third phase, jute PVD was used in place of synthetic PVD and performance was observed for the same duration. The detail procedures are explained in the following sections.

3.10.1 Description of the Steel Tank and Accessories

A steel tank was fabricated with 6 mm thick steel plate by rolling into a hollow cylinder whose both ends were open. Its height was 0.93 m (36.61 inch). inner diameter was 0.502 m (19.75 inch) and outer diameter was 0.514 m. Bottom end was closed with 6 mm thick round plate and tightened with 16 numbers of 10 mm screws. A 2 mm thick rubber gasket was used between the cylinder and bottom cover plate to make it watertight. It created the condition of one-way drainage. The whole tank was painted black to avoid rust. The inside of the tank was rubbed with sand paper to make it smooth and greased sufficiently to reduce frictional resistance between soil slurry and the tank. Two measuring tapes were fixed inside the tank to note the reading of soil settlement in addition to the use of two dial gauges. A round steel plate of 3 mm thick and 49.5 mm diameter was prepared to place it on top of the soil slurry. It was perforated to allow vertical drainage. This steel plate provided equal strain condition for radial drainage, equal distribution of surcharge on the soil slurry and helped in placing the load easily. A wooden beam of 1050 mm X 150 mm X 100 mm was made that had one perforation each at the ends for inserting the rod and hanging the load. The surcharge loads were applied with the available circular load disc of 10 kg. Since total surcharge could not be accommodated on the load hanging beam, discs were also used by placing an extra cover plate on top of the beam and then additional 10 kg disc place over it. The steel tank with the described attachments is shown in the Figure 3.15 to Figure 3.19.

3.10.2 Preparation of Soil Slurry

HOBART Mechanical mixing machine available at BUET Geotechnical Laboratory was used to prepare soil slurry. Water content of dry soil was first determined. To attain the required percentage of water content (Approximately 43%, which was 5% less than liquid limit) in the slurry, quantity of additional water for per kg of soil was calculated. In each mixing effort, 10 to 15 kg of soil and known quantity of water was poured into the mixing bowl. The mechanical mixer was operated for 10-12 minutes until uniform slurry was made. If there was a lump in the soil, it was broken manually. The prepared slurry was then immediately transferred to the steel tank. While pouring the slurry in the tank, efforts were taken to avoid any air void in the slurry so that consistent and uniform density could be achieved. Figure 3.20 shows the mechanical mixing methods.

3.10.3 Water Content and Density Determination

Though soil slurry was prepared with the addition of calculated quantity of water, water content determination was again carried out. Before the application of surcharge, three samples were taken from various depths and tested for water content according to ASTM D 2216. Density of slurry was found out from total weight of soil-water mixture filled up to known height (825 mm) of the steel tank. The same procedures were repeated after the removal of surcharge at the end of consolidation settlement. In each phase of the test, water content and density determination were done in the same way.

3.10.4 Conduct of Vane Shear Test

Shear strength of soil slurry in the steel tank was found out using vane shear apparatus. Since soil slurry was very soft to be put under unconfined compression test, only the direct reading hand vane tester was used. The instrument used was 'The Pilcon Hand Vane Tester' that has been adapted from B.S. 1377. This is a portable instrument that can be used to determine the in-situ shear strength in the field or on the remolded clays in the laboratory. The instrument comprises a torque head with a direct reading scale which is turned by hand. A non-return pointer indicates the reading. Two types of vanes can be used. 19 mm vane is used for 0-120 kPa of shear strength whereas 33 mm vane is used for measuring 0-28 kPa of shear strength. Since soil slurry was very soft, 33 mm vane was used in the laboratory test. Before conducting the test, the vane was screwed in the vane spindle. The vane was pushed inside the slurry with as little sideways movement as possible; to a depth of about 70-80 mm. Holding the instrument in one hand, the head was rotated clockwise at a speed equivalent to a complete revolution in a minute. After the slurry was sheared, the pointer remained set and a reading was taken. The manufacturer of Pilcon Hand Vane Tester had calibrated the instrument and a constant of 1.145 must be multiplied to convert each reading to shear strength of clay. Figures 3.21 to Figure 3.23 show the vane shear apparatus and method of taking readings.

3.10.5 Performance (Settlement) Test without Vertical Drain

The height of soil slurry was 825 mm and a density of 16.51 KN/m^3 was achieved. After the conduct of vane shear test and water content determination, a sand blanket of 25 mm to 30 mm was placed above the soil slurry for easy drainage. Then a geotextile layer was laid on top of sand blanket. Finally perforated steel plate was placed on the geotextile layer. Circular load disc (25 mm thick and 250 mm diameter) weighing 10 kg each was applied centrally on the perforated steel plate and after a certain height was achieved, wooden beam was fixed on its top. Maximum loads were hung on both sides of the beam. A circular steel plate having diameter of 610 mm was placed on the beam and few pieces of 10 kg load disc were also used on the centre of the beam. It made the system more stable. Total surcharge applied amounted to 27.406 kPa. Two dial gauges were set to take settlement readings. The first dial gauge had the scale of 0.01 mm for each small graduation and maximum measuring capacity at one setting was 10 mm. The second one had the scale of 0.001 inch for each small graduation and maximum measuring capacity was 1 inch at one setting. After the loads were applied, readings were taken in a sequence of 0, 5, 10, 20, 40, 60, 120, 240, 480, 1440 minute and so on. Readings were also noted from measuring tapes. Test was continued until a reasonable settlement with respect to applied surcharge took place. From the parameters of oedometer test, it was found that for occurring 55-60% consolidation would take about 30 days. So, the settlement test was run for 28 days. Then the test set up was dismantled. Reduction of slurry height was also noted from measuring tapes. Vane shear apparatus was used to find out shear strength. Samples were tested for water content. Soil slurry was taken out and prepared for next setting. Figure 3.24 and 3.25 show the test methods.

3.10.6 In-Soil Performance Test with Synthetic PVD

Before soil slurry was put in the steel tank, a piece of 'Flexi Drain' of 900 mm long was hung from a horizontal steel rod placed over the top of the tank. A flat plate of 6"x1.5"x2/8" size that was hung from the synthetic PVD did not touch the bottom of the tank. A gap of 40 mm between flat plate and bottom of the tank was intentionally maintained so that when settlement took place, synthetic PVD remained still upright. It also helped in keeping the synthetic PVD kinks-free and twist-free. While pouring the slurry into the tank, efforts were taken to keep the synthetic PVD always at the centre. The height of soil slurry was 825 mm and a density of 16.51 KN/m^3 was obtained. Vane shear test and water content determination were done as usual and a sand blanket of same height (25 mm to 30 mm) used in the previous test was placed above the soil slurry for easy drainage. A geotextile layer was also laid on top of sand blanket. Finally perforated steel plate was placed on the geotextile layer. Same amount of surcharge (27.406 kpa) was applied with the use of circular disc load, wooden beam, circular steel plate and other accessories. Two dial gauges were set to take settlement readings. After the loads were applied, readings were taken in a sequence of 0, 5, 10, 20, 40, 60, 120, 240, 480, 1440 minute and so on. Readings were also noted from measuring tapes. Test was continued for 28 days and after which the test set up was again dismantled. In this case the average degree of consolidation was also more than that of the test without vertical drains. Vane shear test and water content determination were done. To check the shear strength determined by vane shear test, unconfined compression test was also conducted. Figures 3.26 to 3.31 show the tests with Synthetic PVD.

3.10.7 In-Soil Performance Test with Jute PVD

Before soil slurry was put in the steel tank, a piece of jute PVD of 900 mm long was hung from a horizontal steel rod placed over the top of the tank. It was kept straight and upright in the same way as of test with synthetic PVD. At the bottom, a flat plate of 6"x1.5"x2/8" was attached. This flat plate did not touch the bottom of the tank. A gap of 40 mm between flat plate and bottom of the tank was intentionally maintained so that when settlement took place, jute PVD remained still upright. It also helped in keeping the jute PVD kinks-free and twist-free. While pouring the slurry into the tank, efforts were taken to keep the jute PVD always at the centre. The height of soil slurry was 825 mm and a density of 16.505 KN/m^3 was achieved. After the conduct of vane shear test and water content determination a sand blanket of 25 to 30 mm height used in the previous test was again placed above the soil slurry for easy drainage. Then a geotextile layer was laid on top of sand blanket. Finally perforated steel plate was placed on the geotextile layer.

Same amount of surcharge (27.406 kpa) was applied by the use of circular disc load, wooden beam, circular steel plate and other accessories. Two dial gauges were set to take settlement readings. After the loads were applied, readings were taken in a sequence of 0, 5, 10, 20, 40, 60, 120, 240, 480, 1440 minute and so on. Readings were also noted from measuring tapes. Test was continued for 28 days and after which the test set up was again dismantled. In this case, the average degree of consolidation took place was more than that of test without vertical drains. Vane shear test and water content determinations were done. To check the shear strength determined by vane shear test, this time unconfined compression test was conducted. Figures 3.32 to 3.36 show the tests with jute PVD

3.10.8 Unconfined Compressive Test

After the surcharge load and steel plate was removed, and sand blanket was scrapped of, soil specimens were collected with plastic cylinders. These specimens were extracted from the cylinder and prepared by trimming machine. These were then placed inside a split cylinder of 3 inch height and 1.5 inch diameter and both ends were trimmed. After opening from the split barrel, these specimens were weighed. The specimen was then placed between two loading plates of the unconfined compression testing machine. A dial gauge was attached to record the vertical upward movement. The machine was turned on and corresponding specimen deformations were recorded. Reading was continued until a load reached the peak and then decreased or load reached a maximum value and remained approximately constant or deformations of the specimen were past 20% strain before reaching the peak. Then the specimen was unloaded by lowering the bottom plate. The specimen was then taken out from the two loading plates. ASTM D 2166 was followed in the conduct of unconfined compression test.

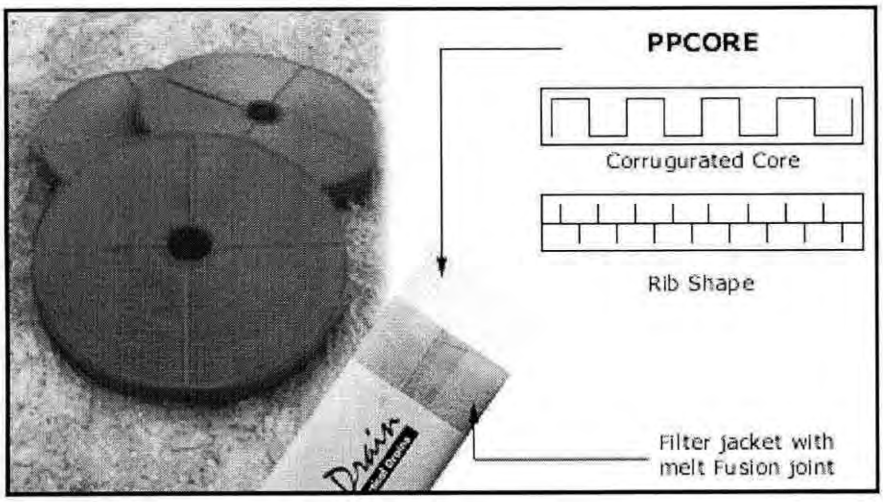

Figure 3.2: Photograph Showing Roll Size, Core Configuration and Jacket of 'Flexi-Drain'

Figure 3.3: Photograph Showing a Piece of 'Flexi-Drain'

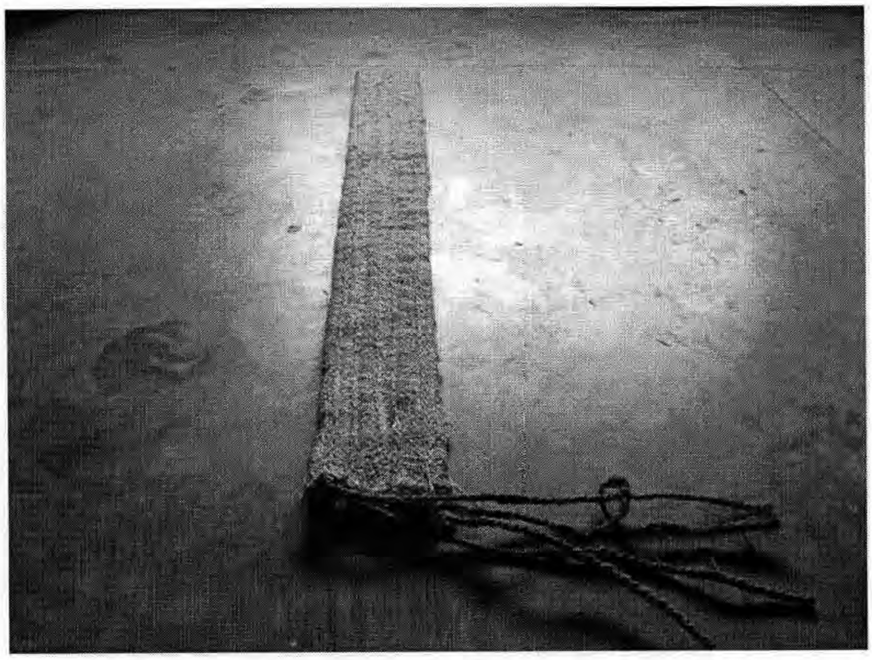

Figure 3.4: Photograph Showing Jute PVD with Coconut Coir Core

Figure 3.5: Photograph Showing Jute PVD with Jute Rope Core

Figure 3.6: Cylinder for Measuring Field Unit Weight.

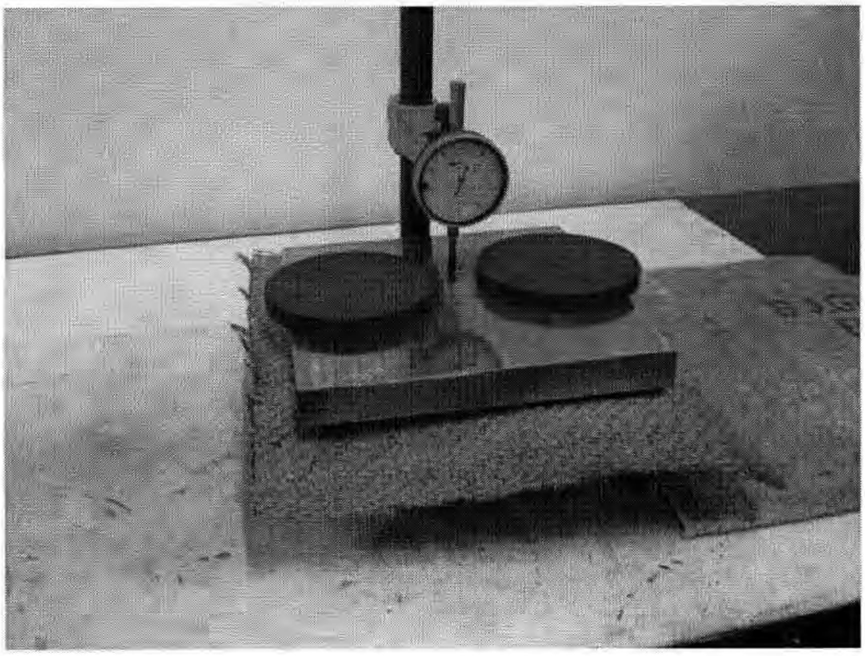

Fig 3.7: Thickness Measurement of DW Twill

Figure 3.8: Photograph Showing DW Twill Cut for AOS Test

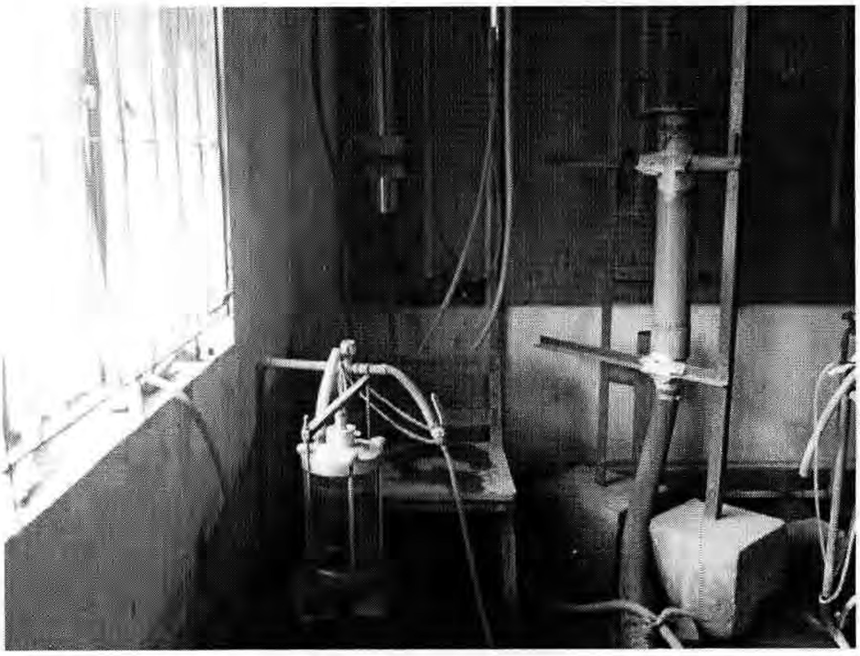

Figure 3.9: Permittivity Test Set-Up Arrangements

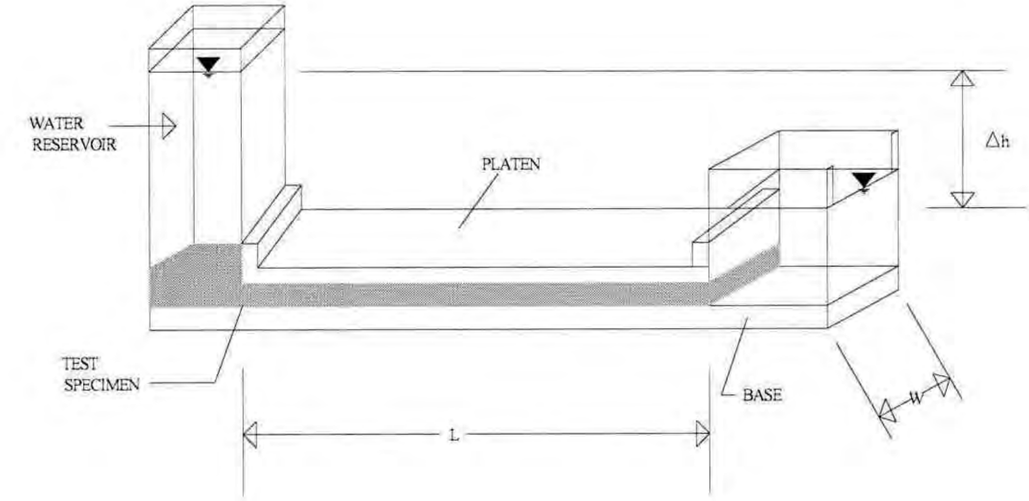

Figure 3.10: Schematic Drawing of Transmissivity Test Assembly (after ASTM Book of Standard, 2004)

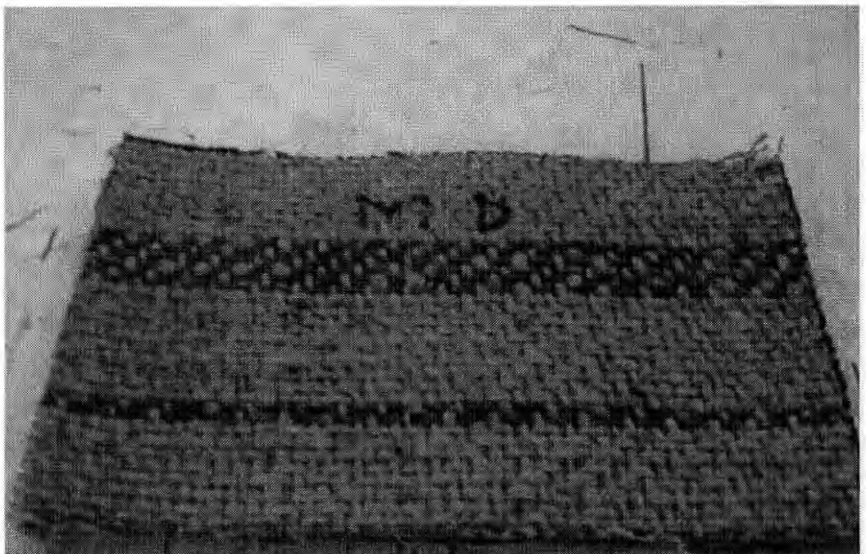

Figure 3.11: DW Twill for Transmissivity (In-Plane) Test

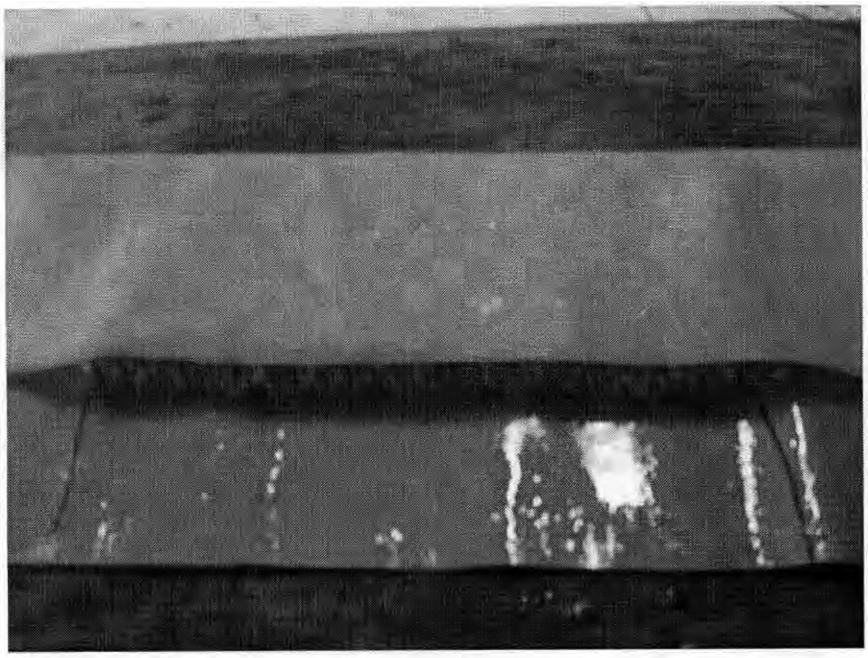

Figure 3.12: Sample of Jute PVD Being Prepared for Transmissivity Test

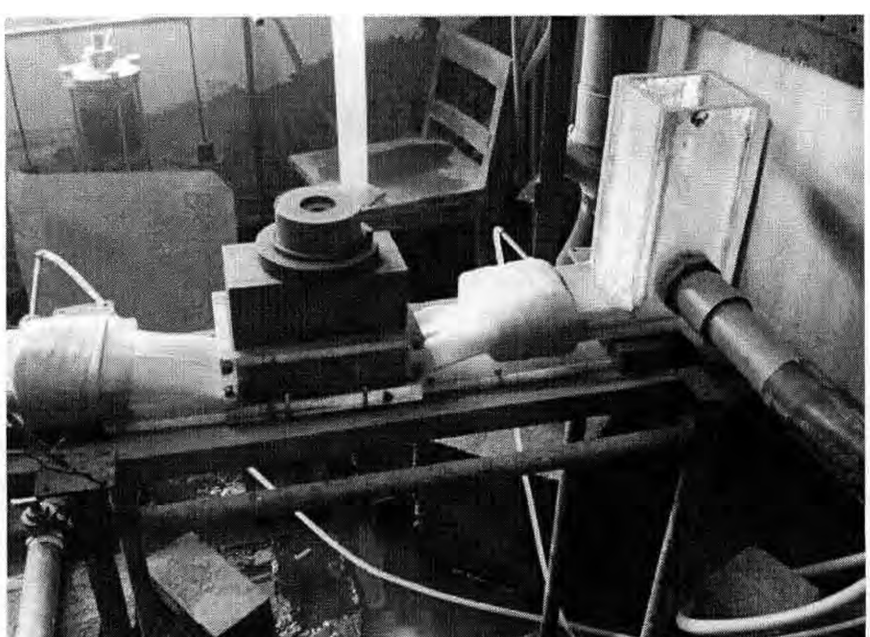

Figure 3.13: Transmissivity Test of Filter Fabric of Jute PVD

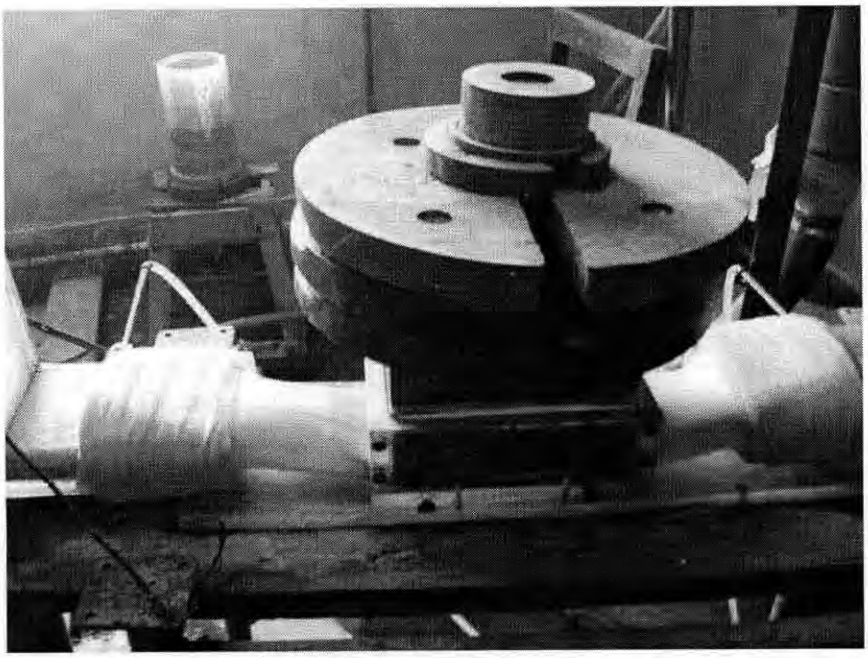

Figure 3.14: Transmissivity Test of Jute PVD as a System

Figure 3.15: Photograph Showing a Steel Tank for Soil Settlement

Figure 3.16: Loading Beam for Suspending Surcharge

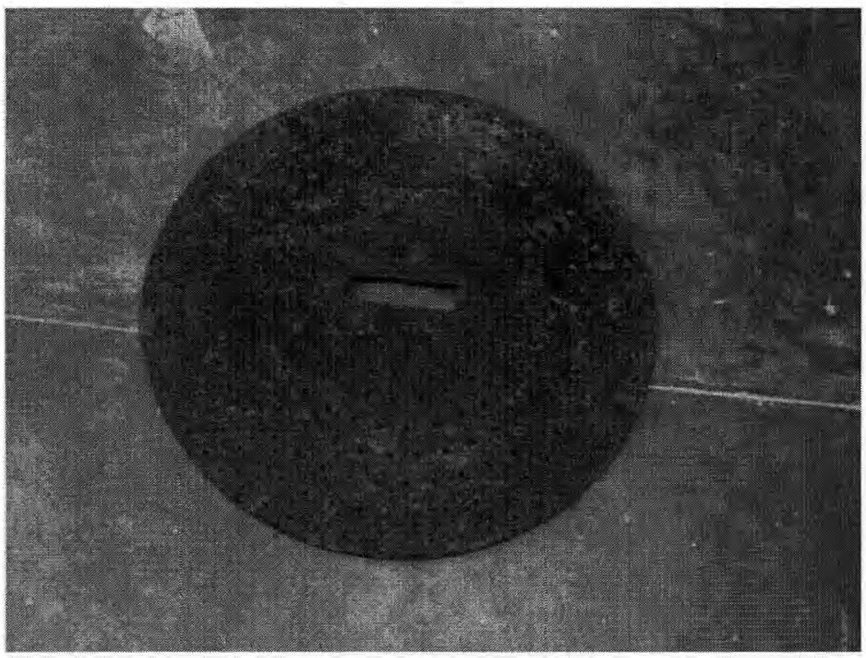

Figure 3.17: Perforated Steel Plate for Equal Strain Case

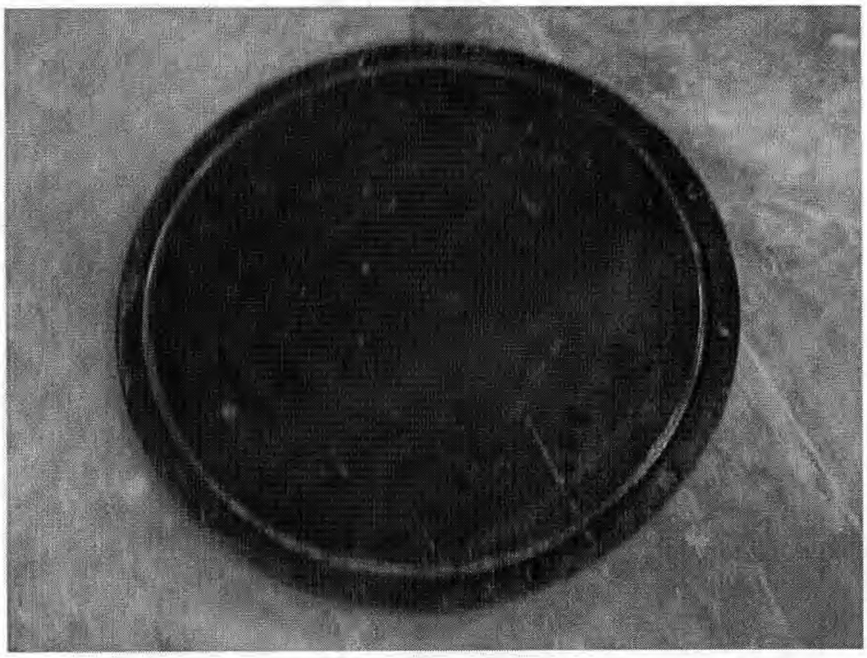

Figure 3.18: Steel Cover Plate Used at the Bottom of Tank

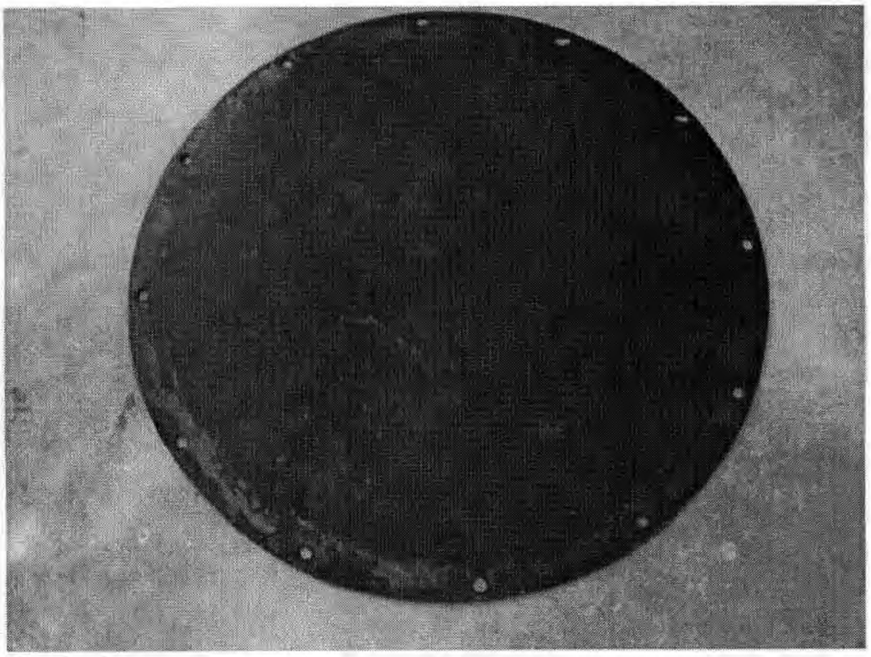

Figure 3.19: 2 mm Thick Rubber Gasket for Making the Tank Watertight

Figure 3.20: HOBART Mechanical Soil Mixture in Operation.

Figure 3.21: Pilcon Hand Vane Tester for Shear Strength Measurement

Figure 3.22: Dial Gauge of Pilcon Hand Vane Tester

Figure 3.23: Shear Strength Being Measured by Pilcon Hand Vane Tester.

Figure 3.24: Soil Being Settled without Vertical Drain.

Figure 3.25: Surcharge Removed after 28 Days (Test without Vertical Drains)

Figure 3.26: Synthetic PVD Hung before Pouring the Slurry (Test with Synthetic PVD).

Figure 3.27: Steel Tank Filled with Slurry (Test with Synthetic PVD).

Figure 3.28: Moist Sand Blanket on Top of Slurry (Test with Synthetic PVD).

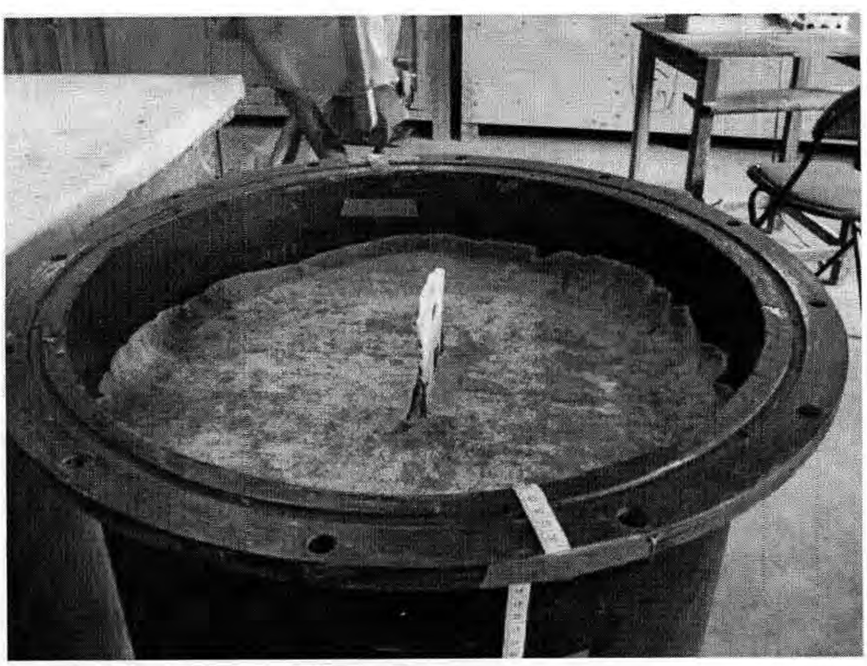

Figure 3.29: Geotextile Filter on the Top of Sand Blanket (Test with Synthetic PVD)

Figure 3.30: Perforated Steel Plate on Top of Geotextile Layer (Test with Synthetic PVD)

Figure 3.31: Surcharge Load Applied for Soil Settlement (Test with Synthetic PVD).

Figure 3.32: Steel Tank Filled with Soil Slurry (Test with Jute PVD).

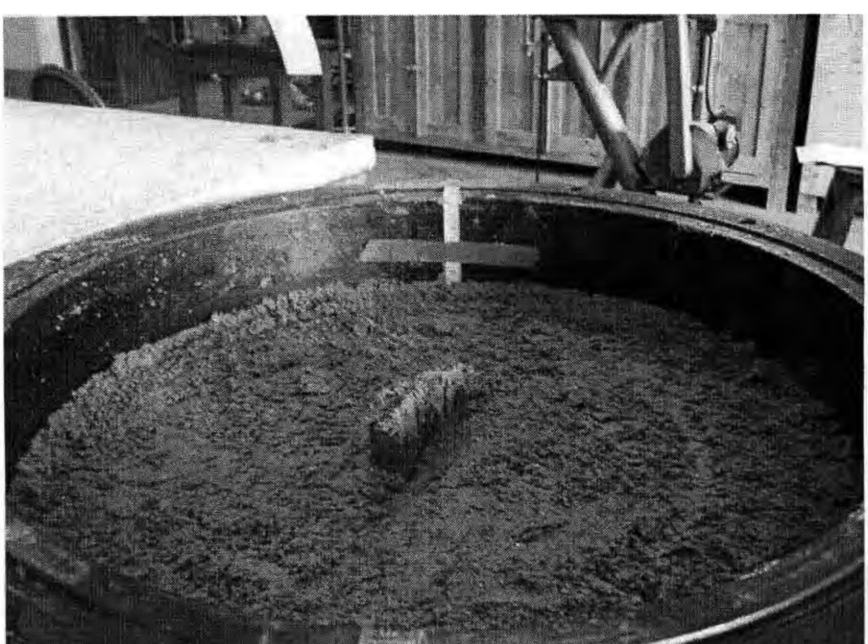

Figure 3.33: Sand Blanket on Top of Slurry (Test with Jute PVD).

Figure 3.34: Geotextile Filter on Top of Sand Blanket (Test with Jute PVD).

Figure 3.35: Perforated Steel Plate on Top of Geotextile Layer (Test with Jute PVD).

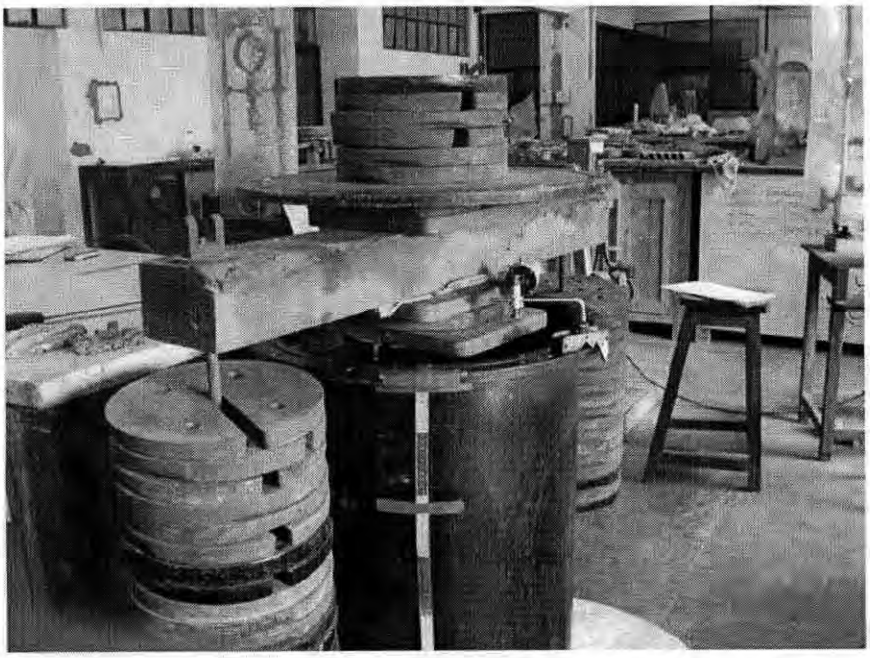

Figure 3.36: Surcharge Load Applied for Settlement (Test with Jute PVD).

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 GENERAL

The results of the laboratory investigations of soils collected from Mirpur Cantonment side of Zia Colony to Mirpur link road, Dhaka are discussed in detail in the following sections of this chapter. Test results of jute PVD are also discussed here. Time-settlement profiles of remolded clay under surcharge load have been investigated in three separate phases, namely, without vertical drain, with synthetic PVD and with jute PVD. These settlements have been compared with the consolidation parameters found from the oedometer tests. Jute PVD has also been compared with synthetic PVD in regards to specifications set for Kaliakoir Bypass by the Roads and Highways Department, Bangladesh.

4. 2 INDEX AND OTHER PROPERTIES OF MIRPUR CLAY

The results of laboratory investigations regarding soil properties are shown in Table 4.1. From investigation, the shear strength of remolded soil is found to be very low (S_u = 6 kPa). The physical and engineering properties comprising specific gravity, index properties, shear strength, and unconfined compressive strength of test soil samples are discussed in the following sections.

4.2.1 Soil Classification

The in-situ soil looked blackish with natural water content but when dried, it turned out to be grey. Once dried it became very stiff. From the laboratory analyses, the particular-size distribution has been obtained from sieve analysis and hydrometer analysis. According to MIT (1931) classification system, the test soil is primarily identified as "5 percent sand, 42 percent silt and 53 percent clay".

Table 4.1 Basic Properties of Test Soil.

Parameters	Symbol	Value
Specific Gravity	G_s	2.70
Liquid Limit	LL	48.4%
Plastic Limit	PL	20.8%
Plasticity Index	PI	27.6%
Natural Water Content	w_n	38.3%
Natural Unit Weight	γ_n	16.9 kN/m^3
% Sand (2 mm to 0.075 mm)	-	5%
% Silt (0.075 mm to 0.002 mm)		42%
% Clay (<0.002 mm)		53%
% of Material Finer than No. 200 Sieve		95%
Unconfined Compressive Strength of remolded and consolidated sample	q_u	12 kPa
Compression Index of remolded sample	C_c	0.12
Coefficient of consolidation	C_v	4.43 mm^2/min
Unified Soil Classification System	CL	Inorganic clay, Low to Medium Plasticity
AASHTO Soil Classification	A-7-6(27)	Clayey soil

In the textural method of classification used by soil scientists of the U.S. Department of Agriculture. only three ranges of particles are specified and materials coarser than 2.0 mm are excluded. So. the percentages of sand, silt and clay-size particles are plotted in a triangular chart used by the U.S. Department of Agriculture and intersection of three sizes gives the classification as silty clay. Soil with particles finer than 0.074 mm would be called silt if its LL is 28% or less and its PI is 6% or less and soil with LL over 28 and PI over 6 is classified as clay. Lambe (1993). According to it. the test soil with LL of 48.4% and PI of 27.6% is classified as clay. According to unified soil classification system (USCS). developed by Arthur Cassagrande for the Corps of Engineers. U.S. Army. soil is classified as CL. This classification means inorganic clays having LL of 50 or less. that is. low to medium compressibility. American Society for Testing and Materials (ASTM) has adopted USCS and incorporated in the ASTM D-2487 and also introduced a plasticity chart for classification of fine-grained soil. The test soil falls above the 'A' line in the plasticity chart in the CL region. As per Association of American State Highway and Transportation Officials (AASHTO), soil containing fine-grained materials is identified by its Group Index in addition to its Group Number. From the particle size distribution plot, the soil falls in group number A-7-6. The group index is calculated ($GI = (F - 35)[0.2 + 0.005(w_L - 40)] + 0.01(F - 15)(I_P - 10)$) to be 27 and the final classification according to AASHTO classification system is A-7-6 (27).

4.2.2 Consistency of Soil

In the field, soil was very soft. From various laboratory investigations, the very soft consistency of soil was also confirmed. Few methods which were used to verify the soft consistency of soils are described in the following sub-sub sections.

4.2.2.1 Natural State of Consistency

Natural state of consistency is one of the most significant index properties of fine-grained soils. No tests for shear strength of in-situ soils were done. A rough estimate of unconfined compressive strength can be based on the simple field tests. For field identification, fist was penetrated several inches easily as an attempt was made, which confirmed the deposit to be very soft (Equivalent unconfined compressive strength, q_u<0.25 tons/sq ft or <23.94 kPa). Peck et al (1973).

4.2.2.2. Correlation with Liquid Limit

The liquid limit is a measure of the shear strength of a soil at some water content. The liquid limit is analogous to shear test, and Cassagrande (1932) has found that each blow to close the standard groove of liquid limit device corresponds to about 1 g/cm^2 (0.0981 kPa) of shear strength. Others have obtained similar results so that one might say that the liquid limit (25 blows) represents for all soils a constant shear strength of between 20 g/cm^2 (1.962 kPa) and 25 g/cm^2 (2.45 kPa). Bowles (1978). The test soil has a liquid limit of 48.4 at which shear strength should be around 2.0 kPa. Though field shear strength was not found out, vane shear test has been conducted on clay slurry at a water content of 43.38%. At this water content, shear strength has been obtained as nearly 6.0 kPa. Shear strength, LL and water content together verify the correlations with each other along with the soft consistency of test soil.

4.2.2.3. Natural Water Content

In natural soft clay soils, water contents in saturated condition vary from 30% to 50%, Das (1997). Soil samples were collected from Mirpur Cantonment area where in the month of December, water level was just few feet below the ground level. So, clay layer at 2m depth was nearly saturated. Natural water content was determined 38.3% in the month of December. According to the range of natural water content mentioned above, the soil was soft clay.

4.2.2.4 Shear Strength from Vane Shear Test

In the laboratory, shear strength was found out by vane shear apparatus. The results of this test in various phases are shown in Appendix B. These results show that the shear strengths obtained at a moisture content 5% less than liquid limit (Before the application of surcharge load for consolidation settlement) are nearly 6 kPa. Vane shear tests after the consolidation settlement confirmed that shear strength improved by 45% to 60% i.e. in the range of 9 to 10 kPa. Shear strength less than 25 kPa specifies soft consistency and shear strength less than 12 kPa indicates a very soft consistency of clay material, GoI (2005). In all cases shear strength remained below 10 kPa which confirmed the very soft consistency of soil.

4.2.2.5 Unconfined Compression Test

Unconfined compression test was conducted on soil slurry only when consolidation settlement was stopped and surcharge was removed. Though it was difficult to prepare the sample for the test due to soft consistency, cautious efforts were taken to get three specimens tested each time. Water content of these samples was near about natural water content (38% to 41%).The results of these tests are plotted and shown in the Appendix B. Unconfined compressive strength was obtained to be more or less 12 kPa. For any soil having unconfined compression strengths below 500 lb/ft^2 (47.88 kPa) indicate a very soft consistency of fined-grained soil, Das (1997). Accordingly this test also confirmed the very soft consistency of test soil.

4.2.2.6 Consistency Correlation from Compression Index and Initial Void Ratio

Compression index and initial void ratio were found out from oedometer tests. There exists a correlation between these parameters and compressibility of clay. Coduto (2007) stated that the range of values of the ratio $c_c/ (1-e_0)$ specifies various consistency of soil. The values greater than 0.35 indicate a very compressible clay. From the compression index and initial void ratio the value came to be 0.052 which indicated that the soil was slightly compressible. This result did not correspond with the specifications mentioned above.

4.2.3 Specific Gravity

There are general ranges of specific gravity for various soils. These ranges vary from 2.63 to 2.9. For clay and silty clay the range is 2.67 to 2.90. The specific gravity test conducted in the geotechnical laboratory of BUET gave a G_s of 2.7. The G_s of 2.7 indicates that the sample was clay.

4.3 TEST RESULTS OF CONSOLIDATION PARAMETERS

Due to low shear strength of soil slurry (6 to 10 kPa), performance tests in the steel tank were carried out under low surcharge load (27.406 kPa). As a result, it was intended that consolidation parameters should also be obtained from oedometer with low to medium stress. In the oedometer tests, loading stress was applied from 10 kPa to 200 kPa whereas unloading was done from 200 to 5 kPa. The soil slurry used for oedometer test was taken from the same slurry that was placed for performance test in the steel tank. The water content and initial density of soil slurry in the oedometer ring were 43.5 % and 16.51 KN m^3 respectively whereas in the settlement tank water content was 43.38% with initial density being same. After the oedometer test, water content reduced to 37.5% and final density increased to 17.3 KN/m^3. In the steel tank, final water content was 40.58%. The values of different consolidation parameters obtained from oedometer consolidation test on the remolded samples are discussed in the following sub-sections.

4.3.1 Void Ratio-Pressure Relationship

The relationships between void ratio (e) and consolidation pressure (p) of oedometer consolidation tests are shown in Figure 4.1 to 4.2. For calculating time factor (Tv), both *square root of time fitting* method (Taylor Method, 1942) and *logarithm of time* method (Cassagrande and Fadum Method, 1940) are used. In the Taylor Method, $\sqrt{t_{90}}$ is obtained whereas in Cassagrande Method, t_{50} is obtained.

4.3.2 Coefficient of Consolidation

The relationships between coefficients of consolidation (c_v) and consolidation pressure (p) for both Taylor Method and Cassagrande Method are shown in Fig. 4.3 to 4.4. Since the performance test was conducted at a stress of (Δp) 27.406 kPa and effective overburden pressure (p_0') at the mid height of soil slurry was 2.76 kPa, so coefficient of consolidation (c_v) corresponding to 30.17 kPa was found out. Figure 4.3 and Figure 4.4 show the methods for finding out coefficient of consolidation. Figure 4.5 to Figure 4.14 show the determination of t_{50} and $\sqrt{t_{90}}$. The c_v values from Taylor Method and Cassagrande Method were 2.45 m^2/year (4.66 mm^2/min) and 4.20 mm^2/min respectively. In this investigation both the values have been averaged which gave c_v value of 4.43 mm^2/min. But for undisturbed sample, usual range of c_v for inorganic clay with plasticity index greater than 25 is 0.19 mm^2/min (0.1 m^2/year) to 19 mm^2/min (10 m^2/year). For remolded clay, the values reduce to 25% to 50% i.e. lowest 0.048 mm^2/min to highest 9.5 mm^2/min, Head et al (1992). The results obtained from laboratory test remained well within the range.

4.3.3 Compression Index

The results of compression index (c_c) at different consolidation pressures (p) of the oedometer consolidation tests on the soil sample have already been shown in Fig. 4.1 to 4.2. In both Taylor and Cassagrande Method, compression index obtained was same i.e. 0.12. This compression index was also taken corresponding to the consolidation pressure level of 30.17 kPa. The magnitude of the compression index varies from soil to soil. Rendon-Herrero (1980) has given some correlations between c_c and type of soils. Terzaghi and Peck gave a correlation for remolded clay as C_c= 0.007(LL-10), Das (1985). The LL of test soil being 48.4, C_c should be 0.268. This value is more than the value obtained in the laboratory investigation. Again, it is found by Isalm (1999) that the c_c value of Dhaka natural clay varies from 0.13 to 0.20 and that of reconstituted Dhaka clay varies from 0.25 to 0.30 all of which are also higher than the value obtained in this investigation. In fact in the laboratory, oedometer test has been conducted with a stress range of 10 kPa to 200 kPa instead of 25 kPa to 800 kPa. So, the compression index has been found at a working stress range. That is the reason of c_c being very low.

4.4 PHYSICAL AND ENGINEERING PROPERTIES OF SYNTHETIC PVD

No laboratory investigations on synthetic PVD were carried out to find out physical, mechanical and hydraulic conductivity characteristics. But in-soil performance test on synthetic PVD was conducted. For compliance with the requirements set by the user organizations, PVD needs to be tested. However, for analyzing its in-soil performance and comparing with jute PVD, test properties have been collected from the manufacturer of synthetic PVD. All these properties have been listed in Chapter 3 in Table 3.2.

4.5 PHYSICAL AND ENGINEERING PROPERTIES OF JUTE PVD

The specific requirements of PVD as set out by the Roads & Highways Department of Bangladesh for Kaliakoir Bypass have already been listed in the Chapter Two of this paper. There are a number of physical and engineering properties that need to be fulfilled for a PVD to be used for that project. 'Flexi Drain', a synthetic PVD has been used finally in that project. Regarding filter jacket of jute PVD, almost all of those properties have been evaluated by Mohy (2005). These results are discussed here to assess if those properties comply with the specifications of Roads & Highways Department of Bangladesh. Besides, hydraulic conductivity being the most important property of any PVD, tests on hydraulic conductivity has been conducted again in the BUET Geotechnical Laboratory. First, tests results related to hydraulic conductivity are discussed and then rest of the results will be discussed from Mohy (2005).

4.5.1 Thickness of Filter Fabric and of Jute PVD System

When water flows through the filter in cross-plane and in-plane direction, the thickness characteristics is incorporated to define that flow with the terminologies called 'Permittivity' (cross-plane permeability) and 'Transmissivity' (in-plane permeability). However, the sponsor organization (Roads and Highways) of PVD has not incorporated thickness in their specifications but the supplier of synthetic PVD (Flexi Drain) has cited thickness in their design manual. The thickness of DW Twill is likely to vary depending on the magnitude and the duration of pressure. As per ASTM D 5199, 10 specimens were cut for testing. The time after which readings were taken was 5 sec after the application of presser plate as suggested by ASTM. Initial readings were taken by the strain gauge before placing the DW Twill. Again after the placement of DW Twill, readings were taken carefully. The average of all 10 readings is found out to be 2.38 mm. It should be mentioned here that tests have been conducted on untreated DW Twill.

Kabir et al. (1994) have also conducted laboratory tests on physical and mechanical properties of jute products. The thicknesses were found 2.82 mm and 1.82 mm by Mohy (2005) and Kabir et al (1994) respectively. The thickness found by Kabir et al (1994) was less because they conducted test on normal Twill whereas test by Mohy (2005) was carried out on DW (Double Works) Twill. The thickness found in this test is smaller than Mohy (2005). This difference might be attributable to the product difference of various jute mills. It is to be mentioned here that the filter layer thickness of synthetic PVD such as 'Flexi Drain' is 0.33 mm only which is of non-woven spun-bonded continuous filament. The thickness of composite jute PVD has not been tested but physically has been measured by scale. This thickness is variable across the surface. Thickness at the surface where core is fixed with stitching is measured to be 8.5 mm to 9.0 mm whereas in between the 2 rows of stitches, this thickness is found to be only 5.5 mm. Thus the average nominal thickness has been found to be 7 mm. On the other hand thickness of composite synthetic PVD (Flexi Drain) has been mentioned to be 3.2 mm.

4.5.2 Apparent Opening Size (AOS) of Jute PVD

The test involves in the determination of the size of the sand particles which when sieved through the fabric will pass only 5% or less by weight. The AOS (or EOS) is defined as the "retention on" size of that fraction expressed as U.S. standard sieve numbers. The AOS, EOS and O_{95} all refer to the same specific pore size, the difference being that AOS and EOS are sieve numbers. while O_{95} is the corresponding sieve-opening size in millimeters. Total four specimens of 280 mm X 280 mm were cut from a full width swatch of DW Twill after discarding 1 m from the very outside of the roll. The size of the specimens was selected so that those fit the sieving pan.

According to ASTM D 4751-87, the specimen was placed in a sieve frame. The sample was secured in such a way that it was taught, without wrinkles or bulges. Care was taken so that the sample was not stretched or deformed such that no changes or distortion in fabric openings occurred. 50 gms of sand fractions were placed on the centre of the fabric started with the smallest size of the sand. Total four sizes of sand were used starting from passing # 100 sieve and retained #200 sieve. The largest size used was the sand passing # 16 sieve and retained on # 30 sieve. A mechanical sieve shaker was used to shake the frame and specimen for 5 minutes. The sand fractions that passed through and that remained on the specimen were taken in pan and separately weighed. The process was repeated by using next larger size sand fractions until the weight of sand passing through the filter fabric were 5% or less. The procedure was repeated on a new specimen of the same type of DW Twill with 50 gms of sand until its apparent opening size was determined.

To find out the AOS of all four specimens, a graphical procedure was adopted. The graph was plotted with the values of percent passing as Ordinate and size of the sand size (mm) as Abscissa on a semi-log paper. A straight line was drawn connecting the two data points representing the sand sizes, which were immediately on either side of the 5% passing ordinate. The particle size in mm (abscissa) at the intersection of the straight line plotted and the 5% passing ordinate was the AOS of the specimen in mm, i.e. the theoretical sand size that would result in exactly 5% passing the specimen.

The result of AOS is given in the Appendix B and the graphical presentation of AOS is shown in the Figure 4.15. In this test. AOS of DW Twill is found out to be 0.85 mm. The required specification of AOS in the Kaliakoir bypass of Roads & Highways Department is < 0.90 mm (90 microns). Mohy (2005) in his study also got AOS as 0.8 mm. (Figure 4.16). Kabir et al (1994) found AOS of Twill to be 0.6 mm. The synthetic PVD (Flexi Drain) has AOS (or pore size. O_{95}) of 0.75 mm (75 microns). From all these tests results and specification requirements, it is seen that the AOS (O_{95}) conforms well into the requirement for jute PVD.

4.5.3 Permittivity (Cross-Plane Permeability)

Constant head method was followed according to ASTM D 4431-98 for the laboratory determination of permittivity of DW Twill. Due attention was given in the important test considerations for this test i.e. preconditioning of fabric (soaking for 24 hours), temperature and use of de-aired water. The specimens were cut by a circular cutting machine with a diameter of 104 mm. Total 20 pieces were used in cylindrical apparatus that made the total height of 69 mm. After setting a constant head, water flow was allowed. Caution was taken to remove any entrapped air from the system. Time was recorded for the flow of 1000 ml of water and the procedure was repeated for another three times. The average time and quantity of flow (1000 ml) gave the flow rate (q, m^3/s).

Though it is a constant head test, flow rate was measured at one constant value of Δh and then the test was again repeated at three different values of Δh. These different values of Δh produced correspondingly different values of q, Koerner (1997). These test results are shown in the Appendix B. To find out the permittivity of specimens, a graphical procedure was adopted. The graph was plotted with the values of flow rate, q as Ordinate and head difference*cross sectional area (Δh*A), as Abscissa on a plain paper. A straight line was derived from the best fit method. The slope of the resulting best fit gave the permittivity (s^{-1}). The graphical presentation is shown in Figure 4.17. The permittivity of DW Twill obtained from this test is 0.12/s. The result obtained by Mohy (2005) was 0.25/s. A minor difference occurred in the procedure adopted in this test and that of Mohy (2005) is that Mohy (2005) collected water for 15 minutes whereas in this test variable time was recorded for collecting 1000 ml water. One important point is that in the PVD specification of Kaliakoir Bypass, permittivity is not included as a requirement to comply.

4.5.4 Transmissivity (In-Plane Permeability)

Transmissivity (m^3/s-m) i.e. the in-plane permeability of geotextile made of natural fibre can be determined by number of test methods. Specifications for PVD in the Kaliakoir Bypass included the test method to be followed as ASTM D 4716. The flow rate per unit width was determined by measuring the quantity of water passing through the test specimen in 15 minutes time. The flow rate was varied by varying normal stress. Specimens were cut from a roll of DW Twill such that the longer dimension was parallel to the Twill's direction to be tested. Only one specimen was tested in the laboratory. The size of the specimen was 165 mm X 100 mm that was glued with rubber membrane on top and bottom to make it water-tight. Hydraulic gradient ($\Delta h/L$) was varied but kept near about at 1.0. In the Kaliakoir Bypass specification for PVD, hydraulic gradient for the test was required to be 1.0. The normal stress was kept at 20.0 kPa. The flow rate ($\frac{Total\ Flow}{Time\ of\ Collection}$) versus hydraulic gradient (Abscissa) is plotted in the Figure 4.18 and flow rate versus hydraulic gradient*width of DW Twill (i*W) is plotted in Figure 4.19. From the best fit curves, discharge rate and transmissivity of single layer of DW Twill are found out. The test yielded the in-plane flow rate at hydraulic gradient of 1.0 at 20 kPa normal stress to be $1.7X10^{-7}$ m^3/s and transmissivity of $1.7X10^{-6}$ m^3/s-m.

Mohy (2005) conducted the same test with varying the normal stress from 10 kPa to 30 kPa. He conducted a range of tests with treated and untreated jute, canvas, DW Twill and Hessian. Rao et al. (1994) also conducted various tests related to engineering properties of 6 types of jute and polypropylene fibres. The results of both Mohy (2005) and Rao et al. (1994) show the similar trend, that the curve shows transmissivity decreasing exponentially. Mohy (2005) got flow rate per unit width at hydraulic gradient of 0.97 to be $1.28X10^{-6}$ m^3/s-m and transmissivity of $1.32X10^{-6}$ m^3/s-m at a normal stress of 20 kPa.

The specifications set by the Roads & Highways Department for the PVD, show that discharge rate (q, m^3/s) should be greater than 500 m^3/year at hydraulic gradient of 1.0 at a normal stress of 200 kPa. But one important point to note here, that this discharge rate is specified for drain system as a whole, not for the filter fabric in isolation. In fact, discharge rate of PVD system as a whole is many times (50 to 200) more than that of filter fabric in isolation. In this research, jute PVD has also been tested as a system for transmissivity, which was not conducted by other researchers. In this test, hydraulic gradient was kept nearly at 1.0; normal stresses were varied between 2.27 kPa and 26.05 kPa. The flow rate versus normal stress is plotted in the Figure 4.20 and hydraulic transmissivity ($\frac{Total\ Flow}{Time * i * W}$) versus normal stress is plotted in Figure 4.21. Since hydraulic gradient is nearly 1.0, both curves as obtained, are nearly of the same pattern. It is seen from the figures that flow rate and hydraulic transmissivity decreases exponentially with the increase of normal stress. Generally, this is true up to a normal stress of 85 kPa above which, increased normal stresses do not decrease the in-plane flow through the fabric, Koerner (1997). Because beyond such stresses, yarn structure is sufficiently tight and dense to hold the load but still convey liquid to convey to some extent. It is seen from the figures that transmissivity at 2.27 kPa was maximum but reduces at a considerable rate up to 17 kPa and then it reduces at a flatter rate.

The flow rate at 20.0 kPa that was obtained in this test is $1.476X10^{-5}$ m^3/s and the transmissivity obtained is $1.45X10^{-4}$ $m^3/s\text{-}m$. With this flow rate, total discharge in one year would be 466.74 m^3 which is close to the specification (>500 m^3/year). However, this discharge rate can be increased in number of ways. A simple method may be to increase the number of inner core by one or two or increasing the diameter of the core.

4.5.5 Grab Tensile Strength

The grab tensile strength is used by almost all manufacturers of geotextiles and this property is invariably incorporated in the specifications of user organizations. The grab tensile test provides an index of the ultimate strength of the specimen at failure. The test results are expressed in units of load (such as pounds) rather than in terms of load per unit width. The standard procedure for conducting this test is ASTM D4632 which is easy to perform, inexpensive, quick and taking only minutes to complete. Specimens are tested for both machine direction (MD) and cross machine direction (XMD). Test specimens are prepared by cutting rectangular section of number of sizes such as 150 mm X 100 mm or 200 mm X 100 mm etc.

Mohy (2005) has conducted grab tensile test on various jute products including DW Twill. Average grab breaking strength of untreated and treated jute products are shown in Figure 4.22 and 4.23. DW Twill provided the best strength out of all types of products both in MD and XMD. He obtained 0.929 KN and 0.75 KN of grab tensile strength for untreated DW Twill in MD and XMD respectively. For the treated DW Twill, grab tensile strengths were 0.995 KN and 0.984 KN for MD and XMD respectively.

According to the specification of Kaliakoir Bypass, and the properties of 'Flexi Drain' supplied by the contractor, it is seen that grab tensile strength is a property of filter jacket and not the property of composite drain. Specifications require a grab tensile strength of greater than 0.35 KN for filter fabric tested as per ASTM D 4632. The results obtained by Mohy (2005) clearly surpass the requirements. Grab tensile strengths of untreated DW Twill are 0.929 KN and 0.75 KN in MD and XMD respectively both of which are greater than 0.35 KN. The grab tensile strength of 'Flexi Drain' is mentioned in the supplier's manual is 0.5 KN. In this case jute PVD clearly fulfills the requirements of PVD.

4.5.6 Trapezoidal Tear Strength

Trapezoidal tearing strength is the force required to break the individual yarns in a fabric. ASTM D 4533-91 codifies this test. As per the standard, an outline of an isosceles trapezoid is marked on a rectangular specimen cut for the determination of tearing strength. The non-parallel sides of the trapezoid marked on the specimen are clamped in parallel jaws of a tensile testing machine. The trapezoid tearing strength method is useful for estimating the relative tear resistance of different directions in the same fabric. In this test, the rectangular specimen is cut as 76.2 mm by 201.6 mm. For the measurement of the tearing strength in the machine (or warp) direction and cross machine (filling yarn), the specimens are cut so that the longer dimension remains parallel to the MD and XMD respectively.

Mohy (2005) has conducted trapezoidal tear test on various jute products such as jute, canvas, hessian and DW Twill both treated and untreated. Average trapezoidal tear strength of untreated and treated jute products are shown in Figure 4.24 and 4.25. DW Twill provided the best strength out of all types of products both in MD and XMD. He obtained 0.464 KN and 0.153 KN of trapezoid tear strength for untreated DW Twill in MD and XMD respectively. For the treated DW Twill trapezoid tear strengths were 0.400 KN and 0.118 KN for MD and XMD respectively. The results show a decrease in the trapezoid tear strength from untreated to treated sample. However, the reasons for such decrease in the strength are not explained.

According to the specification of Kaliakoir Bypass, and the properties of Flexi Drain supplied by the manufacturer, it is seen that trapezoidal tear strength is a property of filter jacket and not the property of composite drain system. Specifications require trapezoidal tear strength of greater than 0.10 KN for filter fabric tested according to ASTM D4533-91. The results obtained by Mohy (2005) clearly fulfill the requirements. Trapezoidal tear strengths of untreated DW Twill are 0.464 KN and 0.153 KN in MD and XMD, both of which are greater than 0.10 KN. The trapezoidal tear strength of

Flexi Drain has not been mentioned in the supplier's manual. However, jute PVD clearly fulfills the requirements of PVD.

4.5.7 Puncture Resistance

Puncture resistance is an important property of any geotextile but also its importance to PVD need not be overemphasized. Puncture resistance tests assess the resistance of geotextile to objects such as stones and stumps under quasi-static conditions. During installation and also during in-soil existence, PVD encounters this sort of objects frequently. PVD filter fabric needs to possess the resistance to objects that might penetrate the filter under load. Such test has been standardized under ASTM D 4833. According to this method, a penetrating steel rod of 8.0 mm diameter is used. The geotextile/jute geotextile specimen is firmly clamped in an empty cylinder with 45 mm inside diameter and the rod is pushed through it via a compression testing machine. Resistance to puncture is measured in force units. The use of this test method is to establish an index value by providing standard criteria and a basis for uniform reporting.

This test is a popular one due to its simplicity and its ability to be automated. It is reported by all manufacturers and listed in most specifications. In this test method, it is important to note the exact shape of the end of the metal rod. Three types are in current use: flat, hemispherical and beveled flat. The interrelationships and difference between these types have not been identified. The ASTM D 4833 specifies the beveled flat type with a 0.8 mm 45° bevel around its circumference. Due to the small size of the device, there is likelihood that there will be difference of the test results even in a single specimen. That is why a larger sized puncture test has been formalized as ISO/DIS 12236 and as DIN 54307. It uses the conventional CBR soil-testing plunger and mold. However, in this research, test results of puncture resistance test conducted under ASTM D 4833 is used which is also called index puncture resistance test.

Mohy (2005) conducted puncture resistance test on various jute products such as jute, canvas, hessian and DW Twill both treated and untreated. Average puncture resistance strength of untreated and treated jute products are shown in Figure 4.26. DW Twill provided the best strength out of all types of untreated samples. He obtained 0.840 KN for DW Twill being the maximum and 0.405 KN for jute samples being the minimum.

In case of treated samples, maximum puncture resistance of 0.4 KN was achieved for canvas but for treated DW Twill, this resistance was 0.305 KN.

According to the specification of Kaliakoir Bypass, and the properties of 'Flexi Drain' supplied by the contractor, it is seen that puncture resistance is a property of filter jacket and not the property of composite drain as a whole. Specifications require puncture resistance of greater than 0.10 KN for filter fabric tested as per ASTM D 4833-88. The results obtained by Mohy (2005) clearly fulfill the requirements. Puncture resistance of untreated DW Twill is 0.840 KN and treated DW Twill is 0.305 KN, both of which are greater than 0.10 KN. The puncture resistance of 'Flexi Drain' is 0.144 KN as mentioned in the supplier's manual. In this case, jute PVD clearly fulfills the requirements of PVD.

4.5.8 Burst Strength

Burst strength is measured by stressing geotextile/jute geotextile such as DW Twill out of the plane, thereby mobilizing tension until failure occurs. Burst strength can be measured in two ways. The first method is covered by ASTM D 3786 which defines burst strength as the force or pressure required in rupturing a textile fabric by distending it with a force applied at right angles to the plane of the fabric, and under specified conditions. In this test an inflatable rubber membrane is used to distort the fabric into the shape of a hemisphere of 30 mm diameter. The diaphragm is expanded by fluid pressure to the point of specimen rupture. Bursting of the fabric occurs when no further deformation is possible. The test is widely used for quality control. Another method uses a large rectangular test specimen and deforms it by an underlying rubber membrane. In this test the fabric remains very close to plane strain conditions. As such the pressure versus strain response yields a very accurate modulus. But it is a difficult test to set up and perform. In this test method, strength may be referred both in force unit (KN) or pressure unit (kPa).

Mohy (2005) conducted burst strength test on various jute products such as jute, canvas, hessian and DW Twill both treated and untreated. Average burst strengths of untreated and treated jute products are shown in Figure 4.27. DW Twill provided the maximum being 2373 kPa and jute provided the minimum being 1245 kPa out of all types of untreated samples. In case of treated samples, maximum puncture resistance of 2530 kPa was achieved for DW Till and 1560 kPa for jute.

According to the specification of Kaliakoir Bypass, and the properties of Flexi Drain supplied by the contractor, it is seen that puncture resistance is a property of filter jacket and not the property of composite drain system. Specifications require puncture resistance of greater than 900 KN for filter fabric tested as per ASTM D 3786-80A. The results obtained by Mohy (2005) clearly fulfill the requirements. Puncture resistance of untreated DW Twill is 2373 kPa and treated DW Twill is 2530 kPa, both of which are greater than specification. The puncture resistance of Flexi Drain is 950 kPa as mentioned in the supplier's manual. In this case jute PVD clearly fulfills the requirements of PVD filter.

4.5.9 Analysis of DW Twill's Test Results

In Chapter Two, required properties of PVD have been listed from Roads & Highways Department for Kaliakoir Bypass Road Project. Most of those properties actually meant for filter jacket. The hydraulic conductivity properties were meant for PVD system as a whole. In the present research, only the properties related to filtration and hydraulic conductivity have been investigated. Properties of other requirements are analyzed from Mohy (2005). The complete range of properties are compiled and compared with the specification as mentioned. In addition, set of properties of synthetic PVD i.e. Flexi Drain is also collected and put side by side for comparison. The total comparison is presented in the Table 4.2. It is seen that except in-plane discharge rate (transmissivity), all the properties fulfill the requirements and even with positive margin. It is very amazing also to note that the test values often exceed that of the synthetic PVD like Flexi Drain. Specification for AOS has been set to be less than 90 micron (0.9 mm). Synthetic PVD has 0.7 mm whereas jute PVD has got the AOS of 0.85 mm (0.8 mm by Mohy). Since the AOS is near the higher limit, after a passage of time at use, soil piping may occur and the possibility of clogging of jute PVD may arise.

4.6 ANALYSIS OF TIME-SETTLEMENT PROFILES

Settlement profiles of soil slurry made of Mirpur clay with known water content was investigated in the steel cylindrical tank in three separate phases. In the first attempt, time-settlement profile was obtained without any use of vertical drains. In the second phase, combined vertical and radial drainage was obtained with the placement of synthetic PVD in the centre of the tank. In the last phase, combined vertical and radial drainage was achieved with the use of jute PVD. Time-settlement results are discussed in detail in the following sections.

Table 4.2: Comparison of Properties of Synthetic and Jute PVDs with RHD Specifications

Properties of PVD Jacket & Core	Propert-ies of	Test Designation	R&H Specificat-ions	Synthetic PVD Flexi Drain	Jute PVD
Apparent Opening Size, μm	Filter	ASTM 4751-87	< 90	75	85
Grab Tensile Strength, KN	Filter	ASTM 4632-91	> 0.35	0.50	0.929 (MD) 0.75 (XMD)
Trapezoidal Tear Strength, KN	Filter	ASTM 4533-91	> 0.10	Not mentioned	0.464 (MD) 0.153 (XMD)
Puncture Resistance, KN	Filter	ASTM 4833-88	> 0.10	0.144	0.840
Burst Strength, KN	Filter	ASTM 3786-80 A	> 900	950 (kPa)	2373 (kPa)
Discharge Capacity at 7 days, 200 kPa at hydraulic Gradient of 1.0, m^3/year	Composite Drain	ASTM 4716-87	> 500	>2500	466.74
Equivalent Diameter, mm	Composite Drain	d_e=2(b+t)/π	> 50	65.70	61.75

4.6.1 Time-Settlement Profile without Vertical Drains

Soil slurry was made in the mechanical mixing machine and subsequently poured in the steel tank. Water content was kept 5% below the liquid limit i.e. 43.38%. Soil slurry was poured in such a way so that no air or vacuum remained in the slurry. Height of soil slurry was kept to 825 mm. The density achieved after filling the steel tank was 16.51 KN/m^3. Since the bottom of the tank was closed, system allowed one-way drainage only. After shear strength was measured by vane shear apparatus, surcharge load of 27.406 kPa was applied.

Settlement readings were noted by two dial gauges and also by two measuring tapes fixed previously in the tank. Settlement was observed for 28 days. To compare these observed results of settlement in the steel tank with the settlement that should have occurred theoretically, first a time-settlement profile has been developed with the consolidation parameters (C_c, C_v, e_0, clay layer thickness) which is shown in the Table 4.3 and Fig. 4.28. Then a time-settlement profile has been drawn with the observed settlement readings. Finally the theoretical settlement (Terzaghi) and observed settlement in the laboratory are compared by plotting them in the same graph which is shown in the Figure 4.29.

The mathematical sample calculations for theoretical settlement are shown in the Appendix C. It is seen that the total settlement (U = 100%) of clay layer thickness of 825 mm at a surcharge of 27.406 kPa would be 44.68 mm. Also from the Figure 4.28, theoretical settlement for 28 days is calculated to be 25.73 mm which is actually 57.6% of total settlement. On the other hand, from the time-settlement observations in the laboratory, it is seen that the total settlement occurred in 28 days is 25.82 mm. So, the actual average degree of consolidation is 57.80%. From the time settlement profile it is seen that initially the actual settlement was more than the theoretical settlement but later on the actual settlement curve without vertical drains became flatter than the theoretical curve.

Vane shear apparatus was used to measure shear strength before and after the surcharge load was applied. The results of vane shear tests have already been shown in the Appendix B. It is seen that shear strength was 6.01 kpa before consolidation settlement and 8.95 kPa after the removal of surcharge. The gain in shear strength was 47%. In this test, quantity of expulsed water was not measured but water content was found out of the soil slurry before and after the application of surcharge. The test was started with 43.38% of water content and after the test, the water content reduced to 40.58%. The results of water content determination is shown in Appendix B and the relation between reduction of water content and reduction in the soil slurry height is shown in the Appendix C.

4.6.2 Time-Settlement Profile with Synthetic PVD

After dismantling the settlement test program of the first phase, steel tank was cleaned and greased. Measuring tapes were again fixed. The same soil with known water content was used to prepare soil slurry for the second phase of the test. In this test, synthetic PVD was placed vertically at the centre of the tank. Efforts were taken to maintain same initial water content of the soil slurry as like first phase of the test. Pouring of slurry was done in the same way as previous phase of the test to achieve same density in the tank. The tank was filled up to the height of 825 mm. Initial water content and shear strength were found out as usual. Same surcharge of 27.406 kPa was applied and settlement readings were recorded for 28 days. After that the loads were removed and again water content and shear strength were determined. Actual settlement of 29.28 mm took place in the steel tank so that the actual average degree of consolidation became 65.53% in 28 days.

4.6.2.1 Theoretical Settlement Profile with Synthetic PVD

When PVD is used, there are two components of consolidation, namely; one-way (vertical) drainage and radial drainage. The contribution of one-way drainage (U_v) in this case is same as that of previous phase (57.6%). For calculating radial drainage, equal strain case has been considered because surcharge applied at the top was distributed equally by placing a perforated rigid steel plate and consequently settlement was same all over. Again, since synthetic PVD was hung before pouring the slurry in the steel tank, there was no smear at the boundary of the drain well. It is also assumed that coefficient of consolidation in the radial direction (C_{vr}) is same as coefficient of consolidation in the vertical direction (C_v).

Since only one synthetic PVD has been used in the centre of the tank of diameter 502 mm, diameter of influence circle (d_e or D) is also taken as 502 mm. Using Eq. (2.21), time factor in the radial direction (T_r) for 28 days becomes 0.7088. Taking into consideration the width and thickness of the synthetic PVD, the degree of consolidation due to radial drainage (U_r) becomes 98.61%. Finally the simultaneous theoretical average degree of consolidation due to vertical and radial drainage is 99.41%, whereas 65.53% settlement actually took place.

In the first phase, without any use of vertical drain, total settlement occurred in 28 days was 25.82 mm. In the second phase, with synthetic PVD used in-soil; 25.82 mm settlement took place in 16.8 days. Now settlement for 16.8 days is analyzed. In 16.8 days, theoretical vertical average degree of consolidation (U_v) is 45.54% and radial average degree of consolidation is 92.33%. So, the combined theoretical average degree of consolidation comes out to be 95.82%, whereas in 16.8 days actual average degree of consolidation occurred 57.8%. The comparison among the observed settlements with synthetic PVD and theoretical settlement with synthetic PVD is presented in the Figure 4.30. Therefore, test results of actual settlement in the steel tank differ with the theoretical settlement as calculated with the parameters found from oedometer test and physical properties of steel tank and synthetic PVD.

4.6.2.2 Gain in Shear Strength after Consolidation with Synthetic PVD

The results of vane shear tests have already been shown in the Appendix B. It is seen that shear strength was 5.86 kPa before consolidation settlement and 9.11 kPa after the removal of surcharge. The gain in shear strength was 55.46%. An attempt was made to conduct unconfined compression test on consolidated soil slurry after the removal of surcharge. Due to soft consistency of slurry, preparation of specimens was difficult. From three specimens, the average unconfined compressive strength (q_u) obtained was 12.3 kPa. As a result, shear strength was 6.15 kPa ($q_u/2$). This shear strength was lower than that obtained by vane shear test. This difference in the shear strength value might have happened due to the very soft consistency of clay which was actually not suitable for unconfined compression test.

In this test, quantity of expulsed water was not measured but water content was found out of the soil slurry before and after the application of surcharge. The test was started with 43.32% of water content and after the test (28 days) the water content reduced to 39.97%. The results of water content determination is shown in Appendix B and the relation between reduction of water content and reduction in the soil slurry height are shown in the Appendix C.

4.6.3 Time-Settlement Profile with Jute PVD

After dismantling the performance test of second phase, steel tank was cleaned and greased. Measuring tapes were re-fixed. The same soil with known water content was used to prepare soil slurry for the third phase of the test. In this test, jute PVD was placed vertically at the centre of the tank. Efforts were taken to maintain same initial water content of the soil slurry. Pouring of slurry was done in the same way as first and second phases of the test to achieve same density in the tank. The tank was filled up to the height of 825 mm. Initial water content and shear strength were found out as usual. Same surcharge of 27.406 kPa was applied and settlement readings were recorded for 28 days. After that the loads were removed and again water content and shear strength were determined. Actual settlement of 30.62 mm took place in the steel tank so that the actual average degree of consolidation became 68.53% in 28 days.

4.6.3.1 Theoretical Settlement Profile with Jute PVD

When PVD is used, there are two components of consolidation, namely; one-way (vertical) drainage and radial drainage. The contribution of one-way drainage in this settlement is 57.6% which is same that occurred in the first phase of the test without vertical drains. For calculating radial drainage, equal strain case has again been considered because surcharge applied at the top was distributed equally by placing a perforated rigid steel plate and consequently settlement was same all over. Again, since drain was hung before pouring the slurry in the steel tank, there was no smear at the boundary of the drain well. It is also assumed that coefficient of consolidation in the radial direction (C_{vr}) is same as coefficient of consolidation in the vertical direction (C_v). Since only one PVD has been used in centre of the tank of diameter 502 mm, diameter of influence circle (d_e or D) is also taken as 502 mm. Using Eq. (2.21) time factor in the radial direction (T_r) was same (0.7088) as test with synthetic PVD. Taking into consideration the width and thickness of the jute PVD, the degree of consolidation due to radial drainage (U_r) becomes 98.33%. Finally the simultaneous theoretical average degree of consolidation due to vertical and radial drainage is 99.29%, whereas the actual settlement took place is 68.53%.

In the first phase, without any use of vertical drain, total settlement occurred in 28 days was 25.82 mm. In the third phase with jute PVD used in-soil: 25.82 mm settlement took place in 14.75 days. After 14.75 days, theoretical vertical average degree of consolidation (U_v) is 42.36% and radial degree of consolidation is 88.45%. So, the combined theoretical average degree of consolidation comes out to be 93.34%, whereas actual average degree of consolidation occurred 57.8%. The comparison among the observed settlement with jute PVD and theoretical settlement with jute PVD is presented in the Figure 4.31. It is seen that, test results of actual settlement in the steel tank differ with the theoretical settlement as calculated with the parameters found from oedometer test and physical properties of steel tank and jute PVD.

4.6.3.2 Gain in Shear Strength after Consolidation with Jute PVD

The results of vane shear tests have already been shown in the Appendix B. It is seen that shear strength was 5.82 kPa before consolidation settlement and 9.24 kPa after the removal of surcharge. The gain in shear strength was 58.76%. Unconfined compression test on consolidated soil slurry after the removal of surcharge was conducted. From three specimens, the average unconfined compressive strength (q_u) obtained was 13 kPa. As a result, shear strength was 6.5 kPa ($q_u/2$). This shear strength was lower than that obtained through vane shear test. This difference in the shear strength value might have happened due to the very soft consistency of clay which was actually not suitable for unconfined compression test. In this test, quantity of expulsed water was not measured but water content was found out of the soil slurry before and after the application of surcharge. The test was started with 43.29% of water content and after the test (28 days) the water content reduced to 39.82%. The results of water content determination is shown in Appendix B and the relation between reduction of water content and reduction in the soil slurry height are shown in the Appendix C.

4.7 DISCUSSIONS ON TESTS RESULTS

4.7.1 Consolidation Parameters

Three important parameters obtained from oedometer test are initial void ratio (e_o), compression index (C_c) and coefficient of consolidation (C_v). The initial void ratio found out from this test is 1.30 which is a reasonable value for clay soil. It has already been shown in the previous section that c_v value of 4.43 mm^2/min (2.33 m^2/year) is well within the range for remolded clay. But the c_c value of 0.12 does not fully correspond to some correlations based on liquid limit such as given by Terzaghi & Peck and others. Also c_c value of clay soil of Dhaka is usually 0.17 and above. This value is less than those values. Soil sample for oedometer test was taken from soil slurry made for settlement test in the steel tank. The degree of saturation (wG_s/e_0) of that sample was 90%. Though saturation was again applied in the oedometer for 24 hours, it was not enough to get 100% saturation. One of the main assumptions of Terzaghi's one dimensional consolidation test is that soil should be 100% saturated. So, the c_c value obtained from test was not a fully accurate parameter. Usually, oedometer test in the laboratory is carried out with a loading range of 25 kPa to 800 kPa. But in this test loading range was 10 kPa to 200 kPa which yielded a c_c value at working stress range.

4.7.2 Properties of Jute PVD Versus Required Specifications

The complete comparison has already been presented in the Table 4.2. Almost all the requirements such as physical properties, mechanical strength, and hydraulic conductivity capabilities as found out in this research and those obtained by Mohy (2005), clearly conform to the requirements. In some of the properties, jute PVD provides even better performance than synthetic PVD such as Flexi Drain. In the jute PVD, it is seen that AOS (0.85 mm) is near to the maximum limit (<0.90 mm). Flexi Drain has got AOS of 0.7 mm. The implication is that. larger pore spaces would allow larger particles to intrude along with the flow of pore water. It will eventually cause soil piping in the drain-well and also tend to clog during the process of drainage. Again in-plane discharge capacity of PVD system as a whole at hydraulic gradient of 1.0 in one year time need to be more than 500 m^3/year. In this case, Flexi Drain supersedes the requirement by 5 times (>2500 m^3/year). whereas, jute PVD falls short of specification which is 466.74 m^3/year. To overcome this situation, the design change in the jute PVD manufacturing is needed. The test sample of jute PVD includes 4 coconut coir cores. In the in-plane permeability, core actually conveys the fluids. So, to conform to the specification, jute PVD may have 5 or 6 cores or larger diameter cores which will increase the hydraulic conductivity. In this test, DW Twill i.e. filter fabric of jute PVD was untreated. Treatment with bitumen or chemicals reduces the pore spaces or pore sizes. For the AOS specification, any amount of treatment will reduce the pore sizes.

4.7.3 Theoretical Settlement (One-Dimensional) Calculation

C_c value and initial void ratio obtained from oedometer test were used to calculate the total settlement under surcharge load in the steel tank. On the other hand, time-settlement profile was obtained from c_v value obtained from the same test. The accuracy of settlement calculation is directly related to these values along with effective overburden pressure. In the calculation of effective overburden pressure, unit weight of water (γ_w, 9.81 KN/m^3) was deducted from unit weight of soil slurry. It would be accurate if soil would be 100% saturated. It has already been mentioned that degree of saturation of soil slurry placed under settlement test was 90%. So the total settlement calculated with these values did not give accurate result.

4.7.4 Settlement without Vertical Drains

In the first phase, time-settlement profile was obtained under 27.406 kPa surcharge and there was no vertical drains used. It is seen from Figure 4.28 and Figure 4.29 that at 28 days, actual settlement (57.8%) is same as theoretical settlement (57.6%) and after that actual settlement curve became flatter than theoretical one. From beginning of the test up to 28 days, actual settlement was more than the theoretical settlement. To explain this issue, the case of degree of saturation and effective overburden pressure again can be considered. Besides, issue of soil slurry density can be used in explaining the pattern of the actual curve. In all phases, efforts were taken to achieve similar density in the settlement tank which was 16.51 KN/m^3. But during pouring the slurry in the tank, it used to be observed that often air vacuum used to remain in the slurry. Under surcharge load this air voids tends to dissipate faster than pore water. So, initially actual settlement was quicker than theoretical settlement. At the top of the soil slurry, sand blanket of 25 to 30 mm, a geotextile layer for filtration and a perforated steel plate for distribution of load were used. This filter and sand blanket also provided some faster settlement that was not considered in theoretical settlement calculation.

4.7.5 Settlement with Synthetic PVD

Actual settlement without vertical drains and actual settlement with synthetic PVD after 28 days were 57.8% and 65.53% respectively. Within the same period, theoretical settlement with synthetic PVD should have been 99.41%. The equation used to calculate the radial drainage was actually developed for sand drains and adapted by Hansbo (1979) for PVD where he included number of factors such as drain spacing, smear and drain-well resistance. It was assumed that sand's permeability was unlimited. But here in this test, synthetic PVD did not allow infinite permeability. Though, smear was avoided in the test, drain internal resistance and clogging could not be controlled. Another reason might be the under-utilization of drainage capacity (In-plane permeability) because of limited filtration capacity (cross plane permeability).

4.7.6 Settlement with Jute PVD

Settlement with the use of Jute PVD was less than that was supposed to occur theoretically (combined vertical and radial) but was more than the settlement that occurred without any vertical drains. Settlement with jute PVD was 68.53%. After 28 days, without vertical drains, settlement was 57.8%. But if we consider the radial drainage, settlement should have been 99.29%. The equation used to calculate the radial drainage was actually developed for sand drains and adapted by Hansbo (1979) for PVD where he included number of factors such as drain spacing, smear and drain-well resistance. It was assumed that sand's permeability was unlimited. The settlements that occurred in 28 days for both jute PVD and synthetic PVD were very close. It is seen that as far as drainage capability is concerned, jute PVD rather provides a better performance. Time-settlement profiles of without vertical drains, with synthetic PVD and with jute PVD are presented in the Figure 4.32. It is seen that jute PVD accelerated the settlement more than the synthetic PVD though in-plane discharge capacity of synthetic PVD is five times more than jute PVD. This issue may be explained as synthetic PVD having under-utilized in-plane discharge capacity. Filtration criterion (O_{95}) of synthetic PVD was conservative than jute PVD. So pore water flow through jute PVD was more than synthetic PVD. However, longevity and durability of jute PVD in saturated condition is not tested. Since the use of vertical drains may be required for six months to 2 years, jute PVD with required treatment will function as demanded.

4.8 ECONOMIC FEASIBILTY OF USING JUTE PVD

Properties of jute PVD jacket, core and PVD as a single system have been investigated. Physical. mechanical and hydraulic properties as obtained from the laboratory investigations are satisfactory to meet the requirements outlined in the specifications of Government organizations like Roads and Highways Department of Bangladesh. Shortfall in the technical requirements can easily be met up through redesign and reinvestigation. Now, economic feasibility needs to be studied for its acceptance by the users.

4.8.1 Case Analysis

In Bangladesh, usually intercity highways are 17 m (60 feet) wide at top surface. These highways are constructed over raised embankment which is actually underlain by soft soil in most parts of its alignment. Again the height of such embankment varies from 1 m to as high as 10 m in some locations. In this analysis. depth of compressible layer is considered to be 10 m. Usually. in an unreinforced embankment. side slope of 1V:2H is used. If embankment is reinforced with synthetic or jute geotextiles. then slope of 1V:1H can safely be employed. In this case study. both types of side slopes will be considered and height of embankment will be taken as 4m. 6m and 8m. Synthetic PVD such as 'Flexi Drain' has been used in the Kaliakoir Bypass Road. The present cost of 'Flexi Drain per metre is little more than Tk. 100.

Jute PVD which has been used in the laboratory investigation was prepared in the then Adamjee jute Mills in 1998. Required expertise and machineries were available in that time and cost per metre was approximately Tk. 10. However, this cost estimate is not documented anywhere, but verbally learned from the individuals engaged in the preparation of jute PVD in that time. With the inflation in almost all the sectors, this price of jute products would also be much more than that. For estimating the latest cost, sample of PVD has been prepared in Latif Bawany Jute Mills in Demra, Dhaka in July 2008 under the direction of BJMC. Due to non-availability of stitching machine and expertise, the job could not be undertaken very easily. However; one piece of sample has been prepared by that mills and estimated cost for 1 m was Tk. 43.79. BJMC high officials were contacted about the cost of newly made jute PVD. It was clarified from them that this product was not commercially produced. The cost which has been estimated by Latif Bawany Jute Mills includes not only the raw materials and labour cost, but also the power, establishment, and other overhead cost. So, when commercial production of this item will be taken, its cost will no way cross Tk. 35 per metre. However, conservatively this cost per metre can be considered as Tk. 45.

In an embankment of 8 m height with side slope of 1V:2H having roadway of 17 m will require 48 numbers of PVD at the bottom. Spacing of PVD has been set to be 1m centre to centre. If side slope is made steeper i.e. 1V:1H with soil reinforcement, then 31 numbers will be required. For 1 km length of embankment, total 48,000 and 31,000 numbers of PVD of 10 m long (depth). will be required in case of 1V:2H and 1V:1H side slope respectively. Cost per number of Flexi Drain will be Tk.1000. So, total cost of synthetic PVD for 1 km embankment of 8m high (1V:2H) becomes Tk. 4,80,00,000/-. For the same case, if jute PVD is used, total cost will be Tk. 2,16,00,000/-. This actually offers a cost saving of 55%. Figure 4.33 and Figure 4.34 show the estimate of PVD quantity and cost comparison of synthetic PVD with jute PVD per km of embankment.

4.8.2 Other Benefits of Jute PVD

Economic benefit of using PVD, be it synthetic or natural fibre, is difficult to estimate. Use of PVD accelerates consolidation, thereby reduces project duration. It saves huge amount of money by saving time. It tries to eliminate post construction settlements thereby also eliminate future serviceability problems of structures. The cost of PVD certainly is less than that would be incurred in future due to non-use of PVD during initial construction. Since synthetic PVD has been used in many countries of the world including Bangladesh, its economic feasibility is already established. As far as cost effectiveness is concerned, jute PVD is preferred to synthetic PVD as is already seen in the calculation above. Jute is a biodegradable natural fibre. Still it can be treated as per required designed life. Considering the worldwide concern on environment, jute PVD can be a better option in meeting the environment preservation requirements. Finally, Bangladesh needs to spend her limited foreign exchange to procure synthetic PVD from abroad. Jute PVD in this case becomes a viable option to reduce the import of synthetic materials

Table 4.3: Theoretical Time Settlement (Terzaghi's One-Dimensional Theory)

Clay Layer Thickness, mm	Cv mm^2/min	Total Settle-ment, mm	Time, min	Time Factor, $T_v = C_v*t/H^2$	Deg of Consolid-ation U(%)	Settlement, mm
825.00	4.43	44.68	0	0	0	0
825.00	4.43	44.68	5	3.251E-05	0.6	0.29
825.00	4.43	44.68	10	6.501E-05	0.9	0.41
825.00	4.43	44.68	20	0.00013	1.3	0.57
825.00	4.43	44.68	40	0.0002601	1.8	0.81
825.00	4.43	44.68	60	0.0003901	2.2	1.00
825.00	4.43	44.68	120	0.0007802	3.2	1.41
825.00	4.43	44.68	240	0.0015603	4.5	1.99
825.00	4.43	44.68	480	0.0031207	6.3	2.82
825.00	4.43	44.68	1440	0.009362	10.9	4.88
825.00	4.43	44.68	2880	0.018724	15.4	6.90
825.00	4.43	44.68	4320	0.028086	18.9	8.45
825.00	4.43	44.68	7200	0.0468099	24.4	10.91
825.00	4.43	44.68	8640	0.0561719	26.7	11.95
825.00	4.43	44.68	10080	0.0655339	28.9	12.91
825.00	4.43	44.68	11520	0.0748959	30.9	13.80
825.00	4.43	44.68	12960	0.0842579	32.8	14.63
825.00	4.43	44.68	14400	0.0936198	34.5	15.43
825.00	4.43	44.68	17280	0.1123438	37.8	16.90
825.00	4.43	44.68	20160	0.1310678	40.9	18.25
825.00	4.43	44.68	23040	0.1497917	43.7	19.51
825.00	4.43	44.68	27360	0.1778777	47.6	21.26
825.00	4.43	44.68	30240	0.1966017	50.0	22.35
825.00	4.43	44.68	33120	0.2153256	52.4	23.39
825.00	4.43	44.68	34560	0.2246876	53.4	23.87
825.00	4.43	44.68	37440	0.2434116	55.5	24.81
825.00	4.43	44.68	40320	0.2621355	57.5	25.71

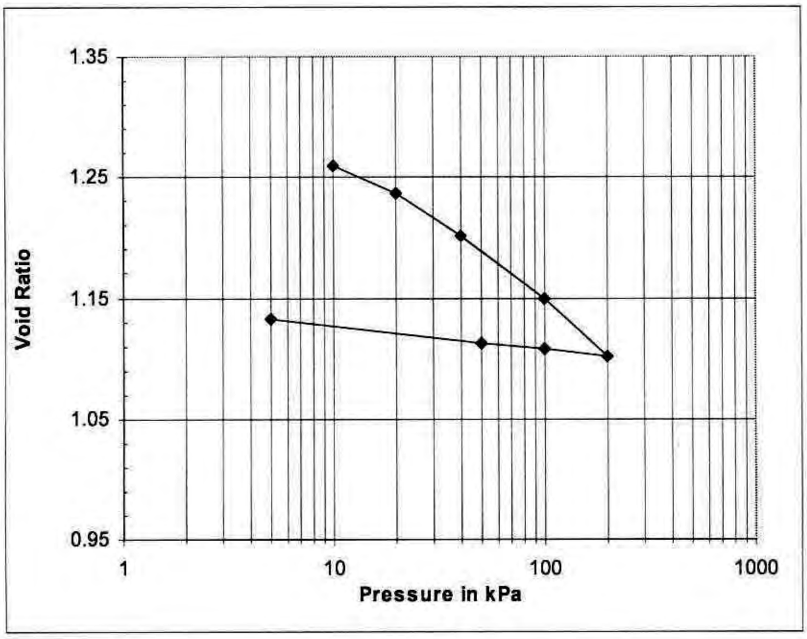

Figure 4.1: Void Ratio Vs Log p Curve ($\sqrt{t_{90}}$ Method)

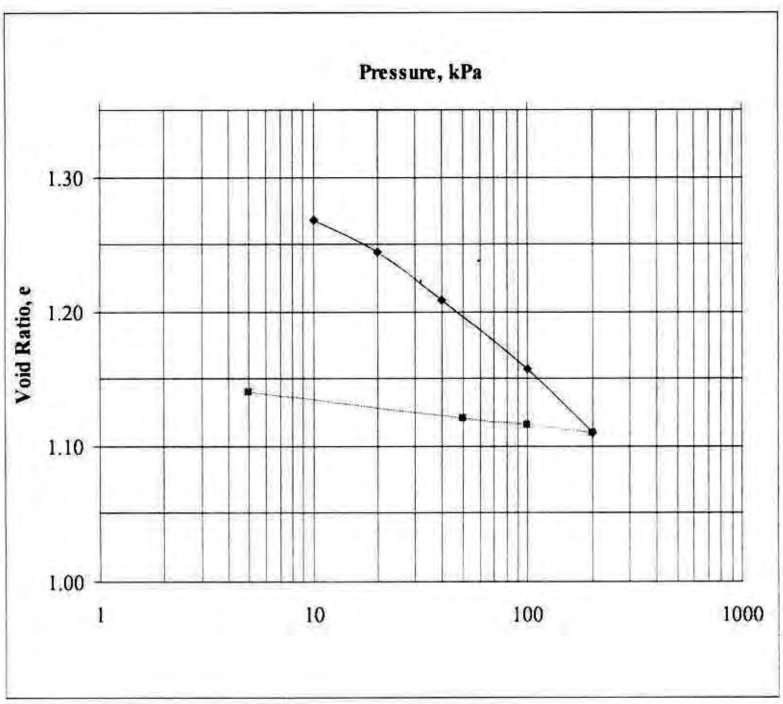

Figure 4.2: Void Ratio Vs Log p Curve (T_{50} Method)

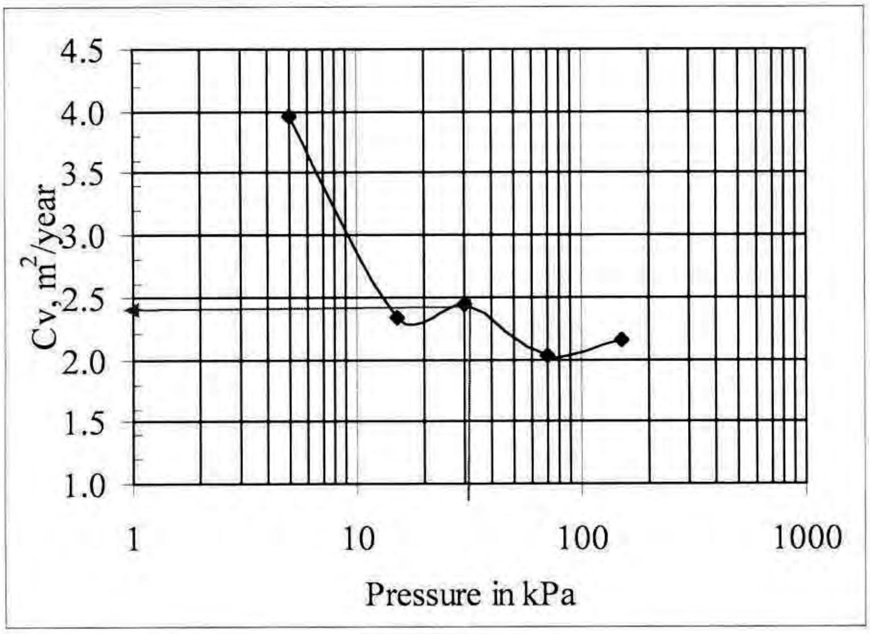

Figure 4.3: Determination of C_v Value ($\sqrt{t_{90}}$ Method)

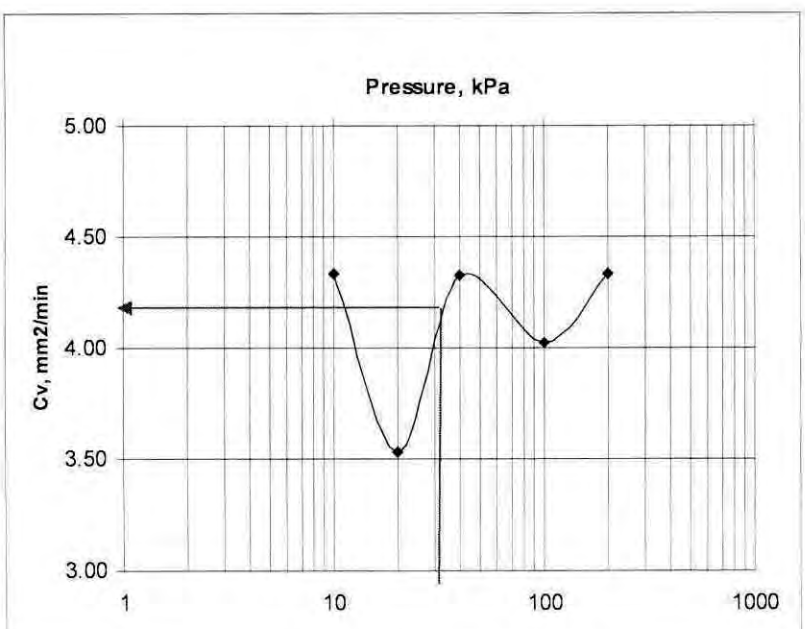

Figure 4.4: Determination of C_v Value (T_{50} Method)

Settlement	% Consolidation
0.01	0%
0.32	100%
0.1650	50%
T_{50}	7.240
t	d
4	0.13208
8	0.17272

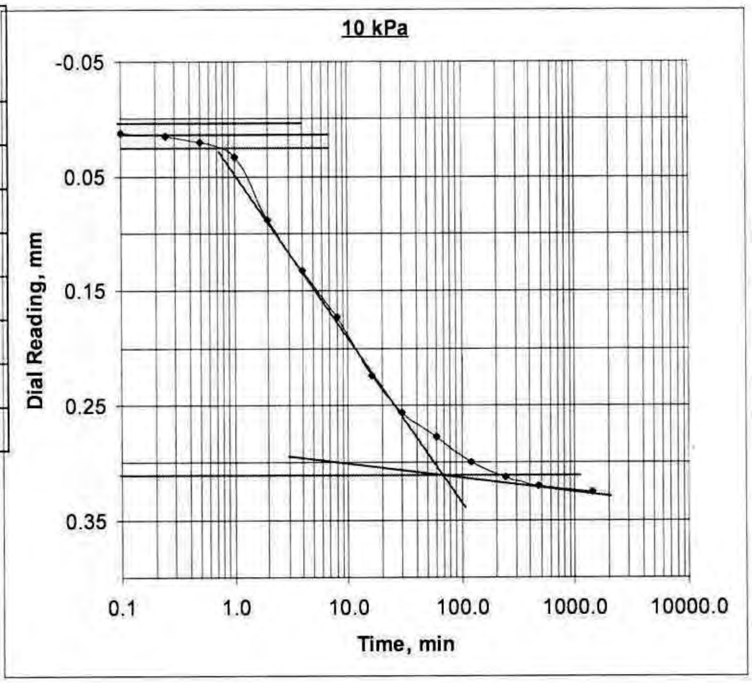

Figure 4.5: Determination of t_{50} by Cassagrande Method

Settlement	% Consolidation
0.34	0%
0.575	100%
0.4575	50%
T_{50}	8.688
t	d
8	0.45466
16	0.48768

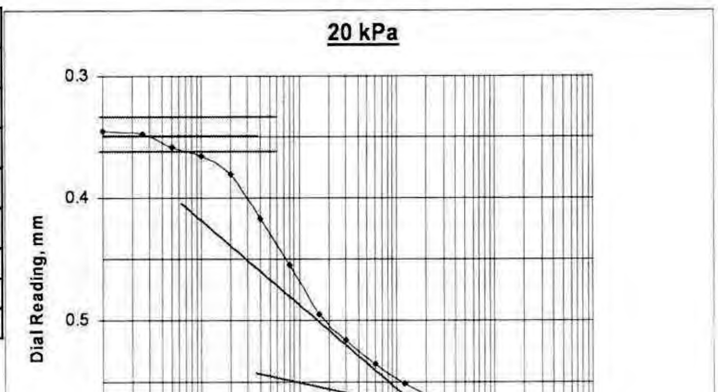

Figure 4.6: Determination of t_{50} by Cassagrande Method.

Settlement	% Consolidation
0.6	0%
0.92	100%
0.7600	50%
T_{50}	6.898
t	d
4	0.72136
8	0.7747

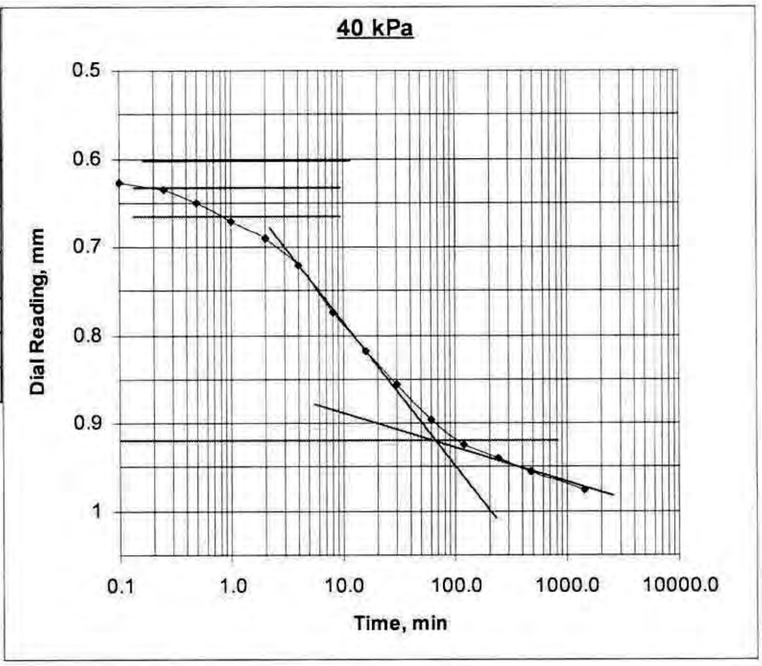

Figure 4.7: Determination of t_{50} by Cassagrande Method.

Settlement	% Consolidation
1.06	0%
1.465	100%
1.2625	50%
T_{50}	7.132
t	d
8	1.26746
16	1.31318

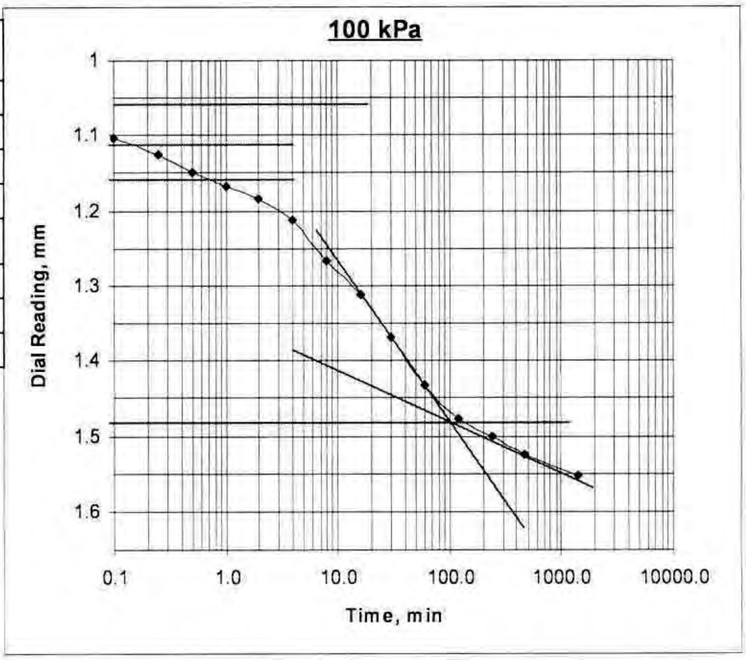

Figure 4.8: Determination of t_{50} by Cassagrande Method.

Settlement	% Consolidation
1.67	0%
2.03	100%
1.8500	50%
T_{50}	6.321
t	d
4	1.8161
8	1.87452

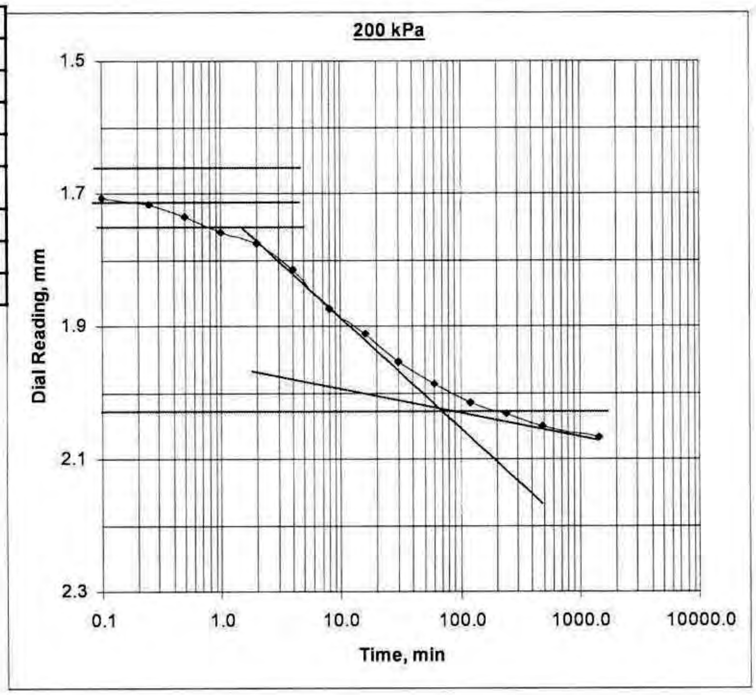

Figure 4.9: Determination of t_{50} by Cassagrande Method.

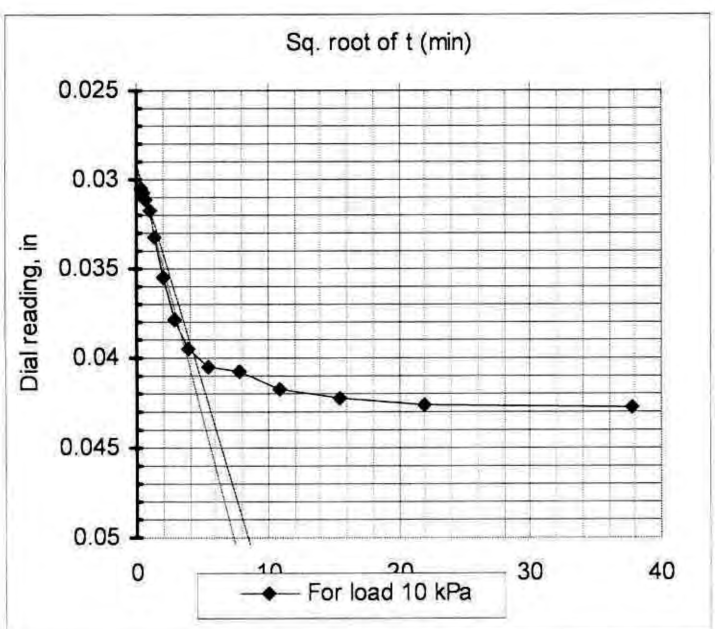

Figure 4.10: Determination of t_{90} by Taylor Method.

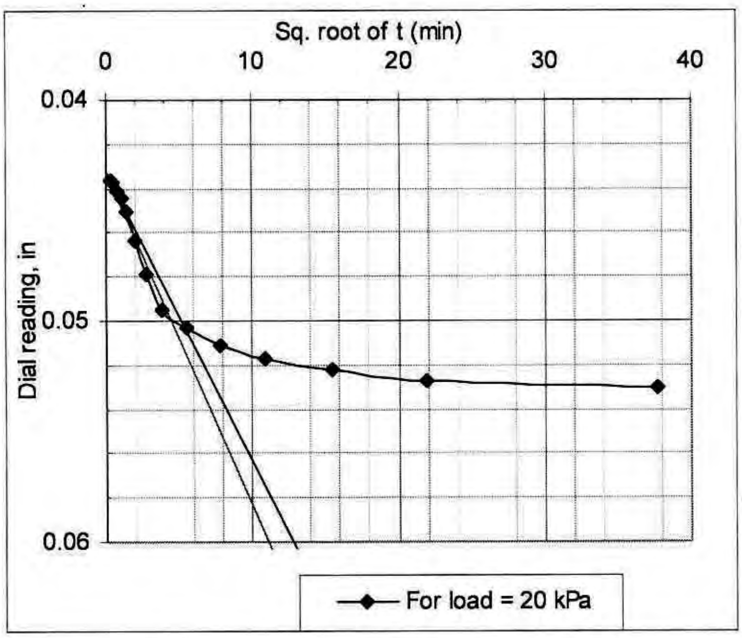

Figure 4.11: Determination of t_{90} by Taylor Method.

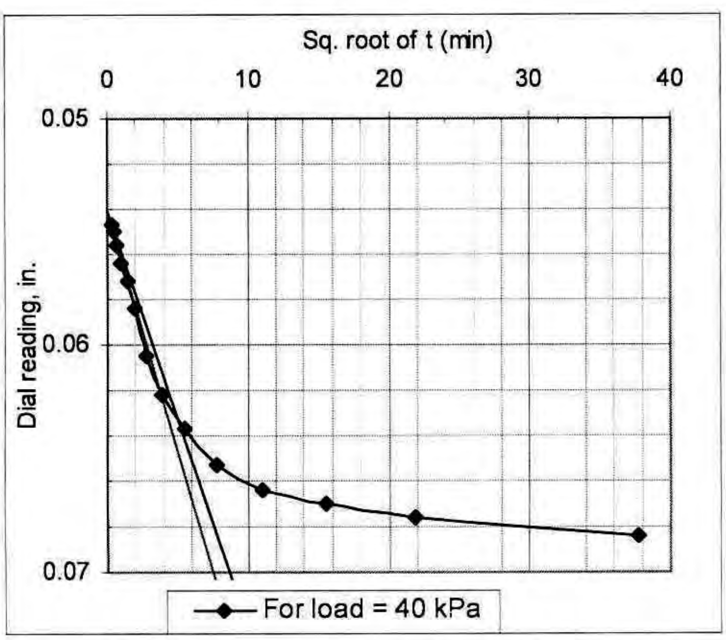

Figure 4.12: Determination of t_{90} by Taylor Method.

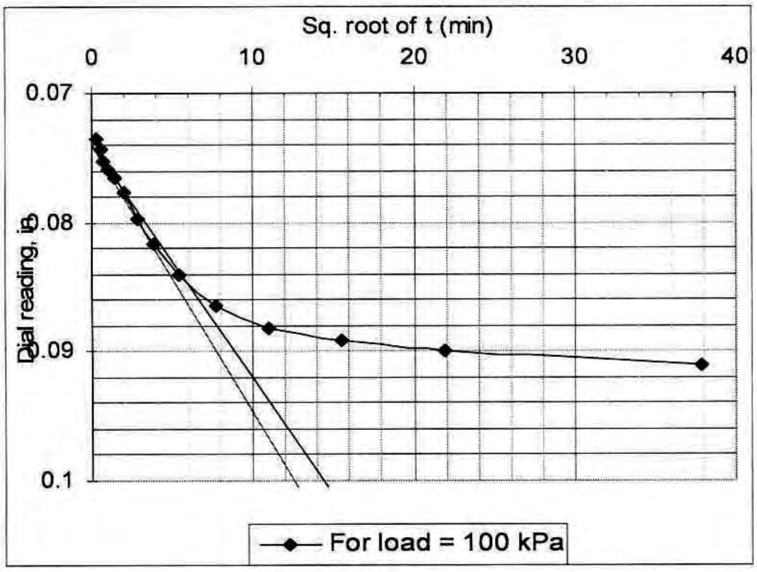

Figure 4.13: Determination of t_{90} by Taylor Method.

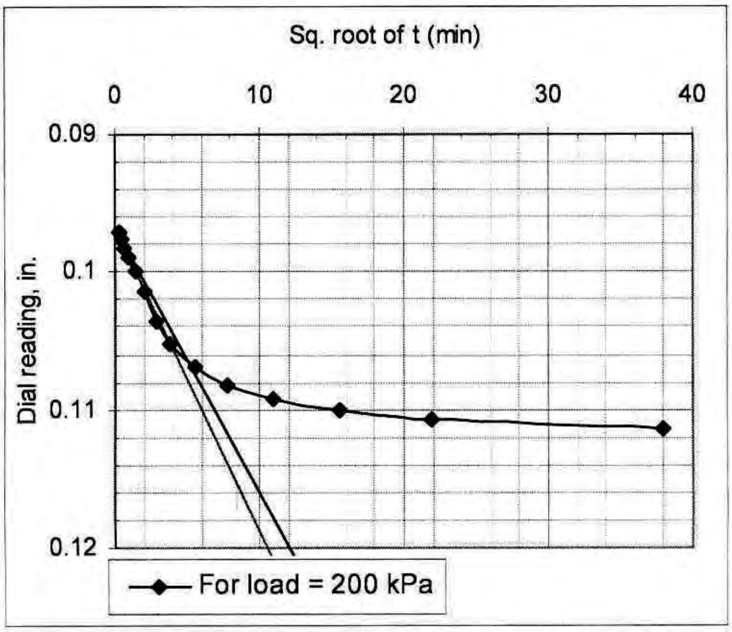

Figure 4.14: Determination of t_{90} by Taylor Method.

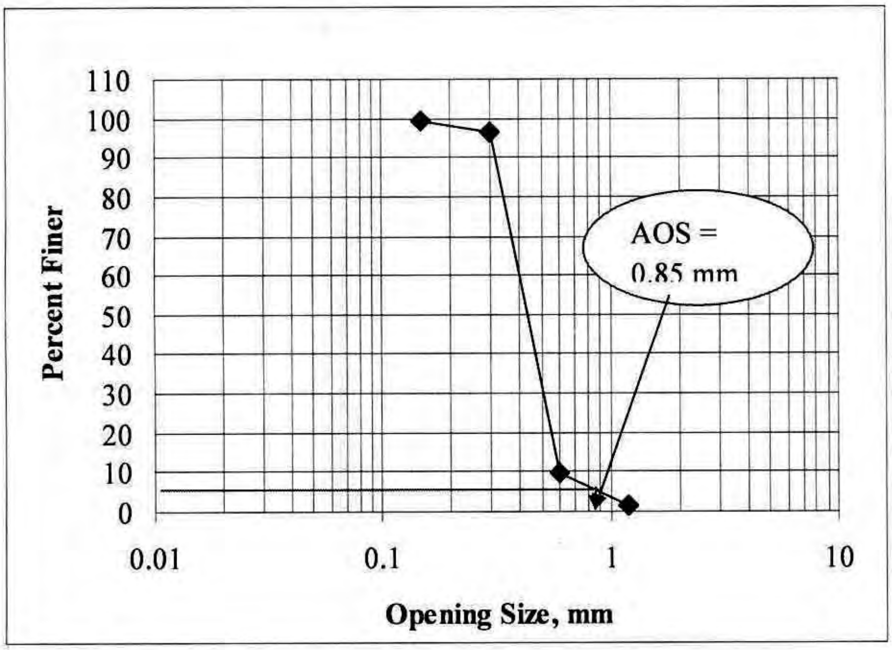

Figure 4.15: Apparent Opening Size of DW Twill

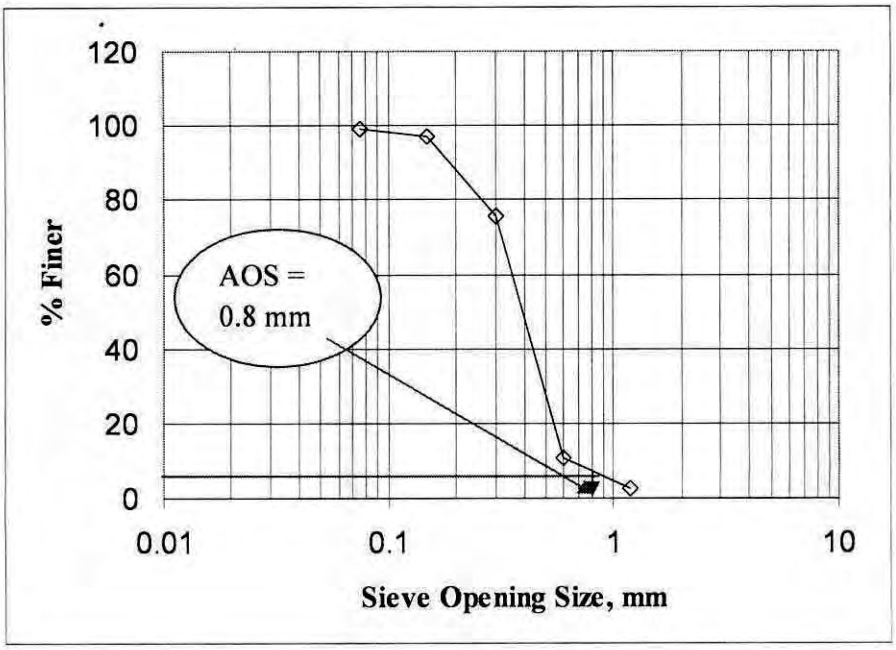

Figure 4.16: Apparent Opening Size of DW Twill, After Mohy (2005)

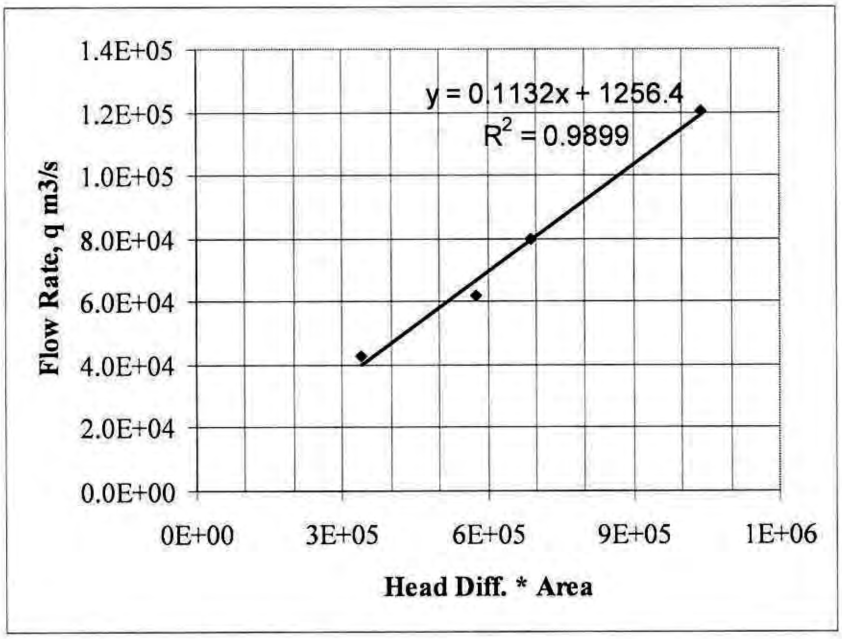

Figure 4.17: Permittivity (Cross-Plane Flow) of DW Twill

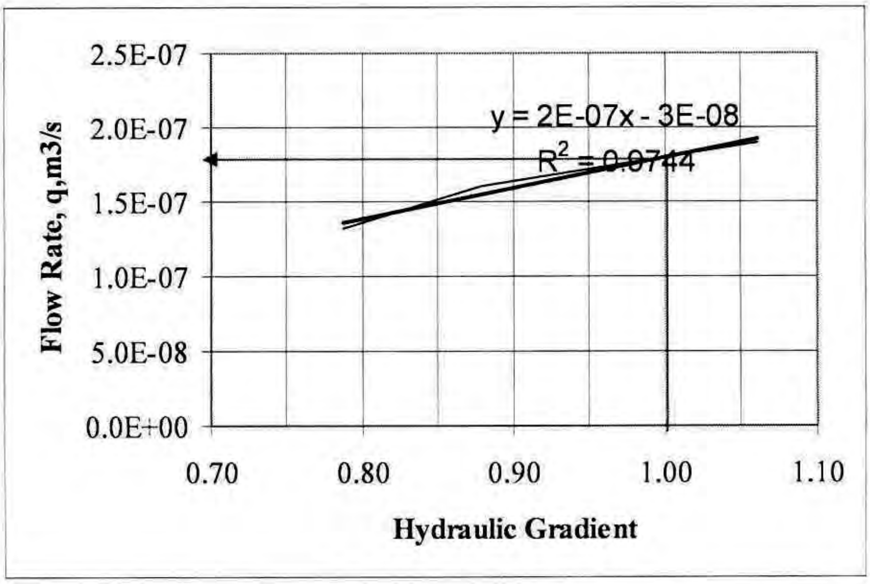

Figure 4.18: Transmissivity (In-Plane Flow) of Single Layer DW Twill

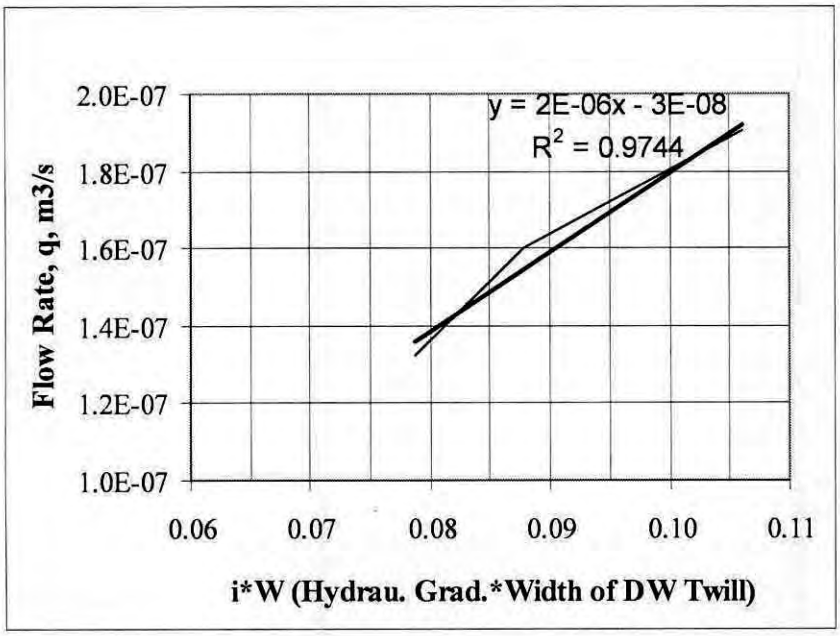

Figure 4.19: Transmissivity of Single Layer DW Twill at 20 kPa

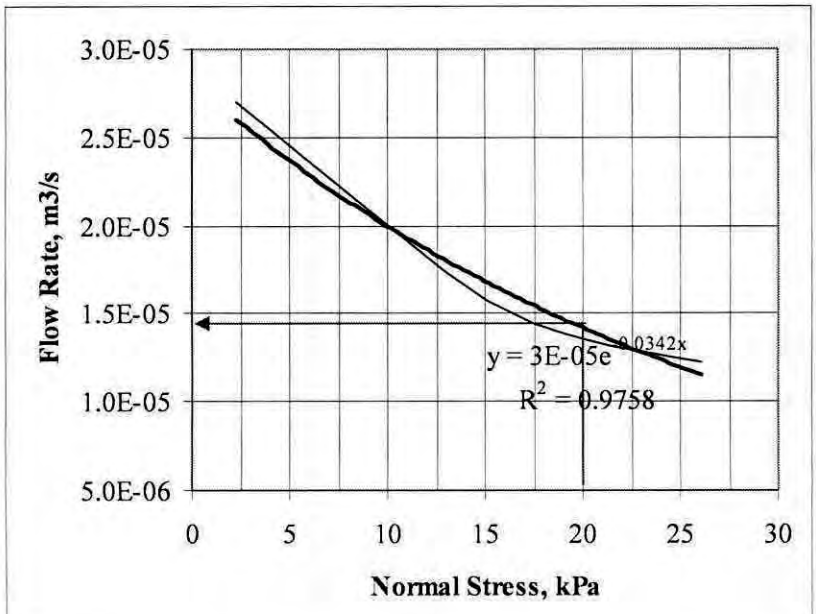

Figure 4.20: Variation of Flow Rate with Normal Stress (Jute PVD)

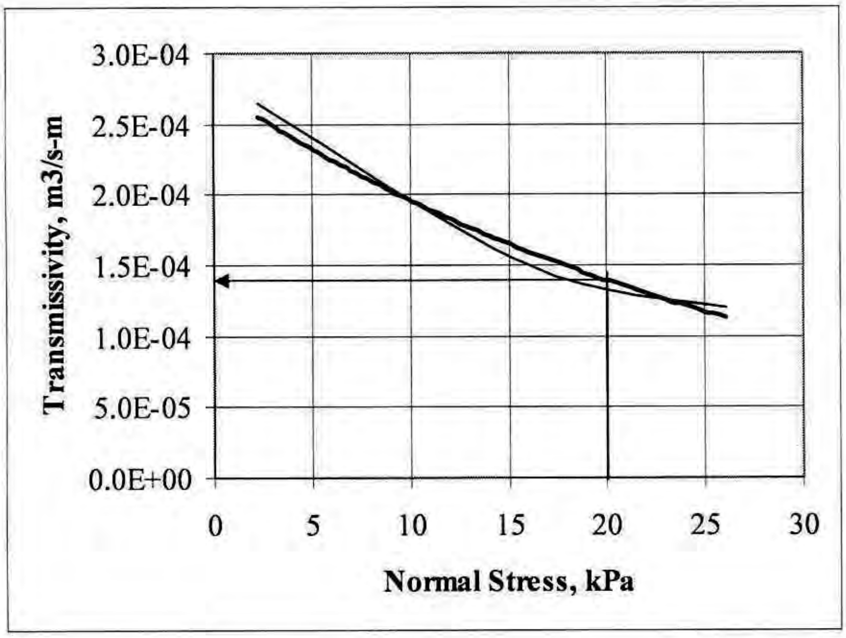

Figure 4.21: Transmissivity of Jute PVD as a Single System

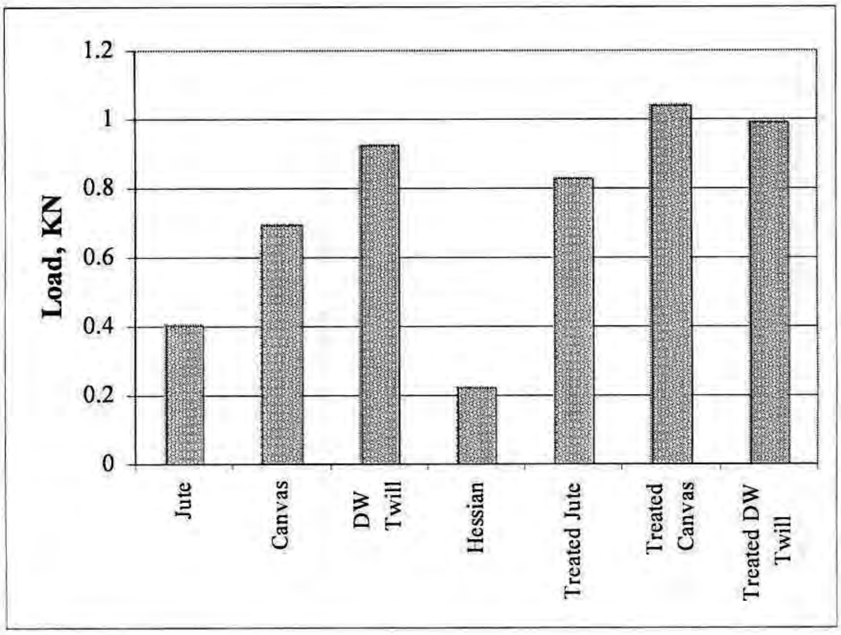

Figure 4.22: Grab Tensile Strength, (MD), After Mohy (2005)

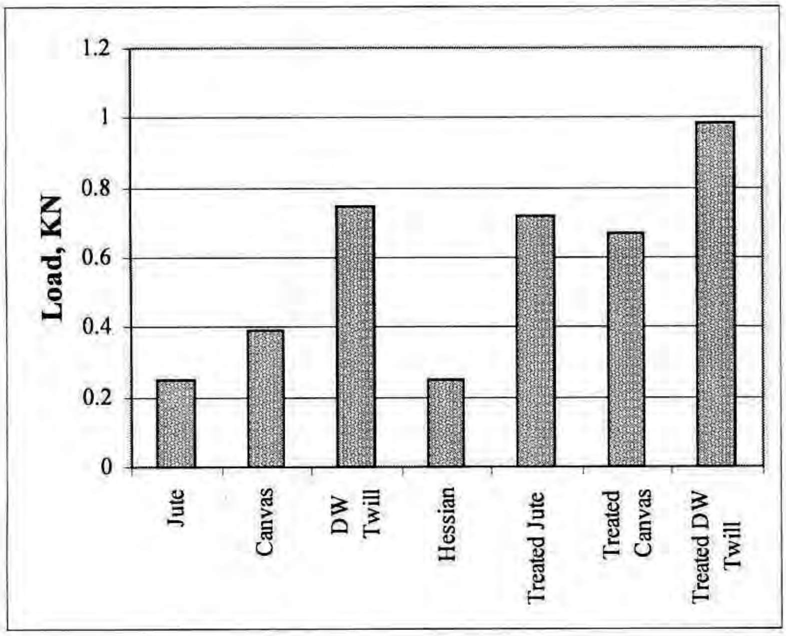

Figure 4.23: Grab Tensile Strength, (XMD), After Mohy (2005)

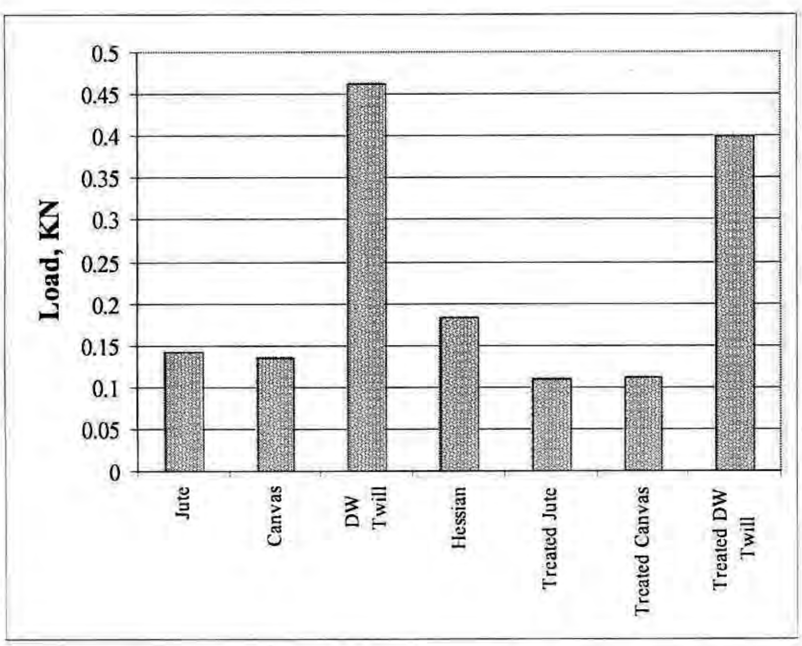

Figure 4.24: Trapezoidal Tear Strength, (MD), After Mohy (2005)

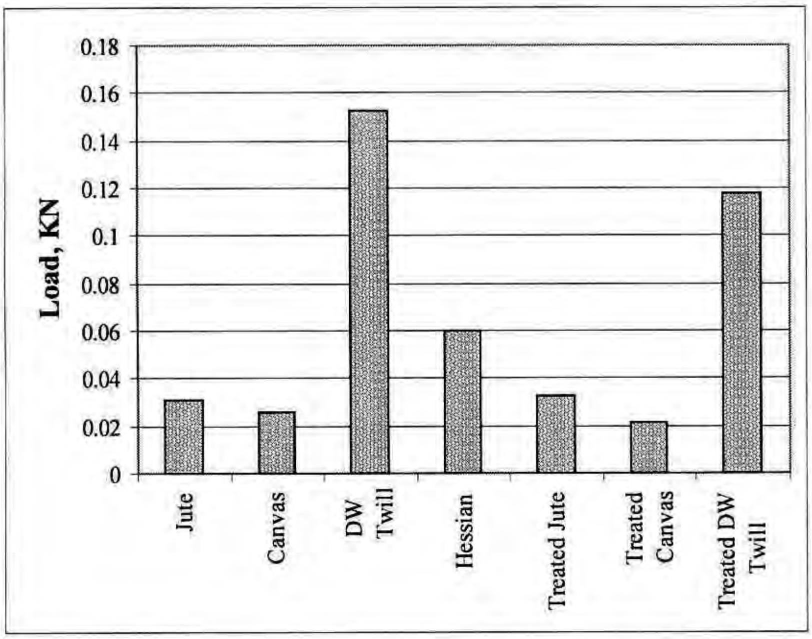

Figure 4.25: Trapezoidal Tear Strength, (XMD), After Mohy (2005)

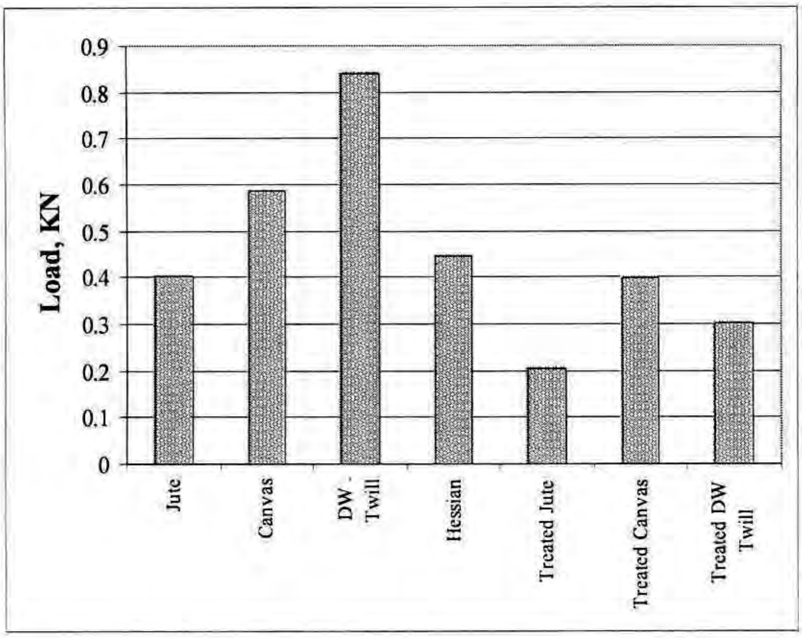

Figure 4.26: Puncture Resistance, After Mohy (2005)

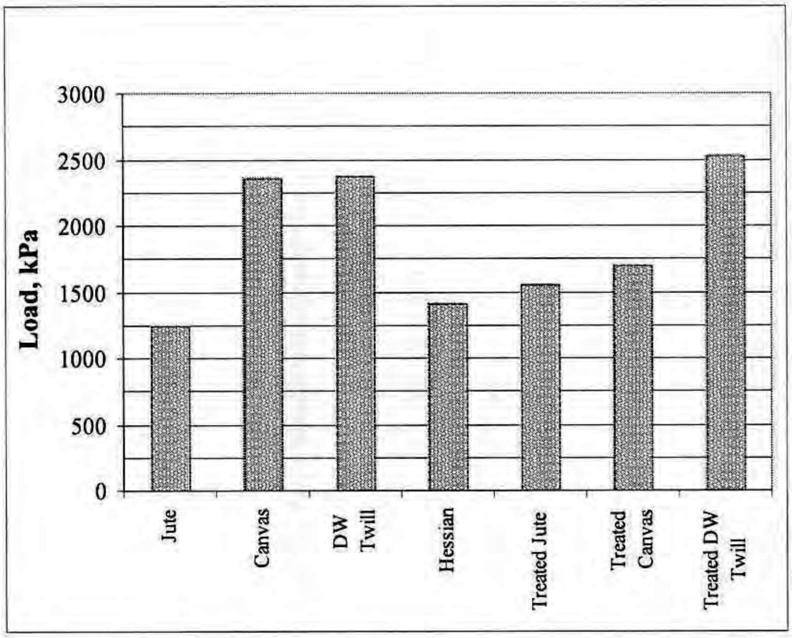

Figure 4.27: Burst Strength, After Mohy (2005)

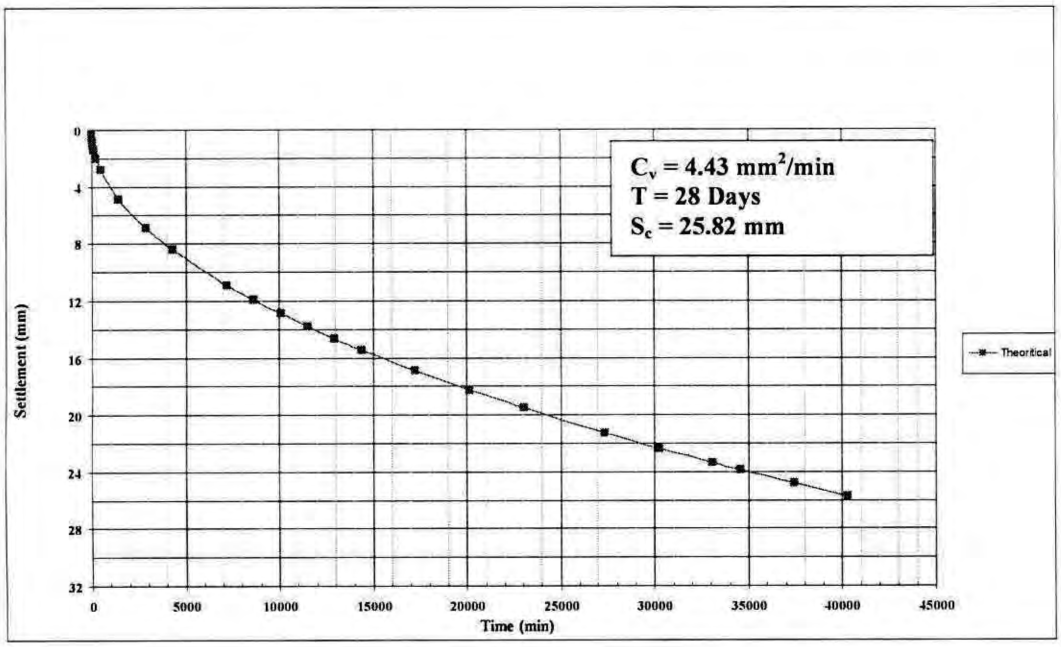

Figure 4.28: Theoretical (Terzaghi) Time-Settlement Curve

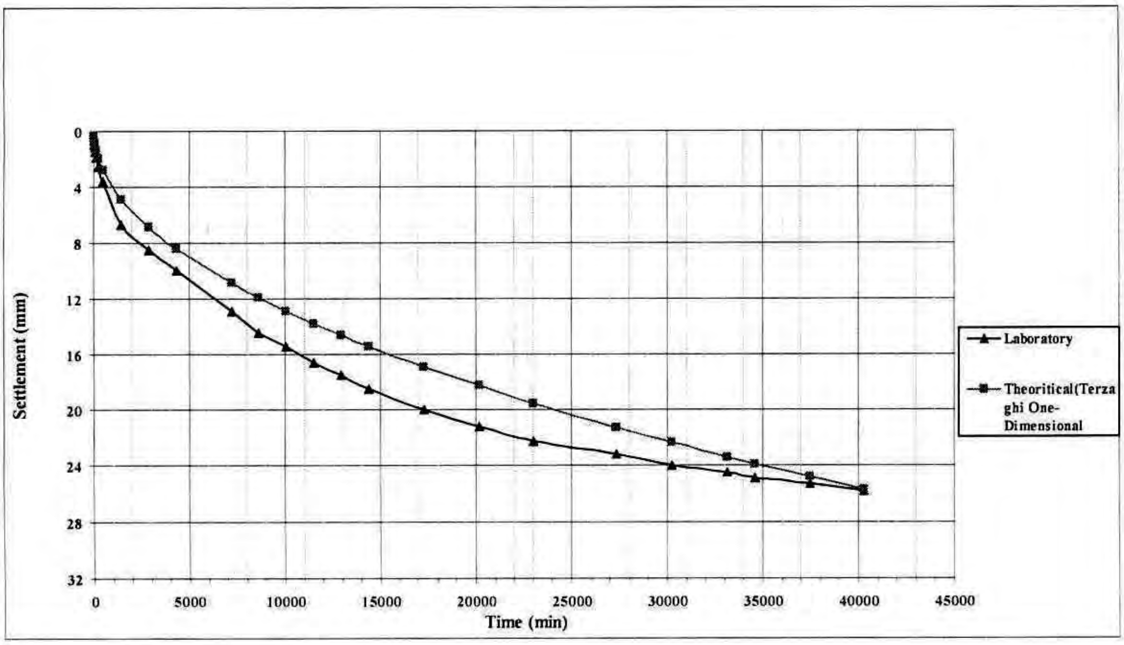

Figure 4.29: Comparison between Theoretical (Terzaghi) and Observed Settlement in the Laboratory (without Vertical Drain)

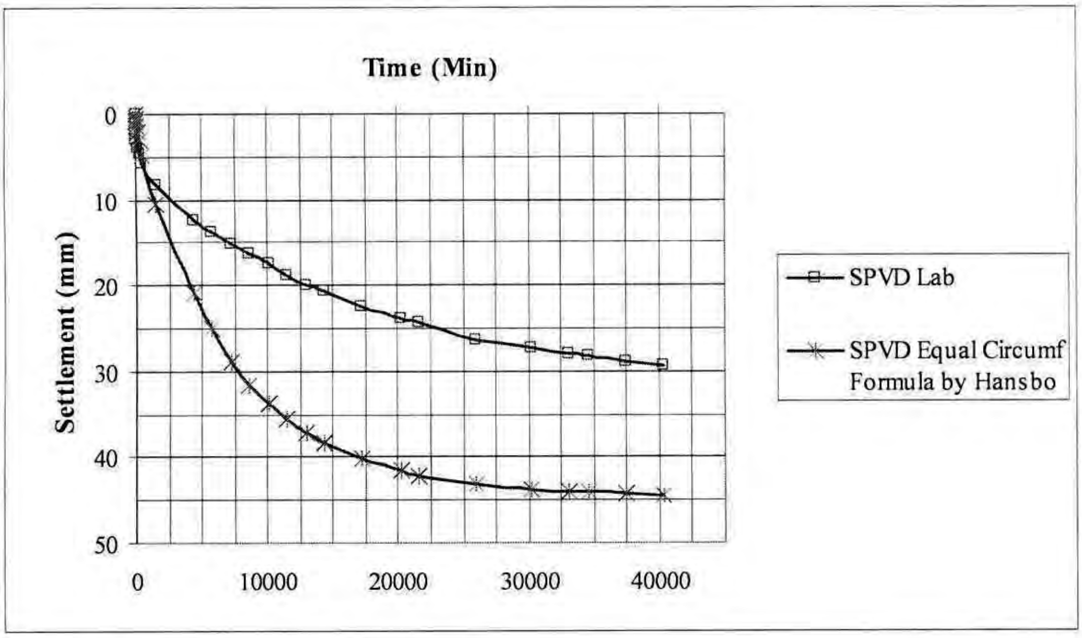

Figure 4.30: Comparison between Theoretical Radial Settlement and Observed Settlement with Synthetic PVD in the Laboratory

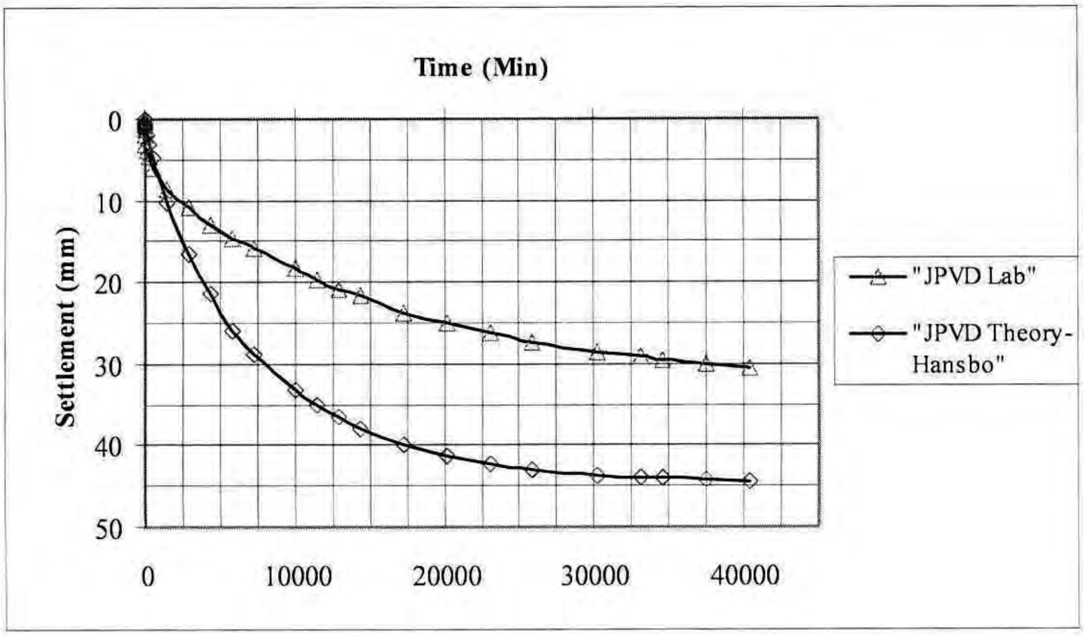

Figure 4.31: Comparison between Theoretical Radial Settlement and Observed Settlement with Jute PVD in the Laboratory

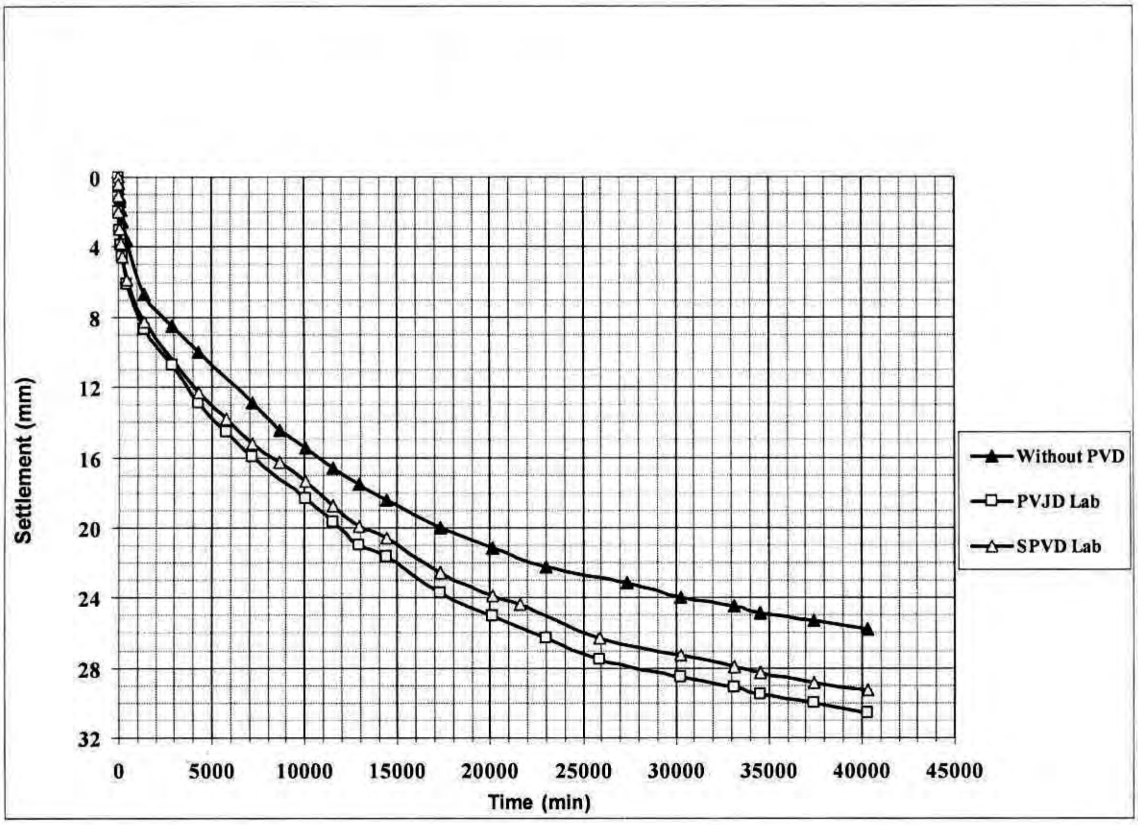

Figure 4.32: Settlement Profiles without PVD, with Synthetic PVD and with Jute PVD

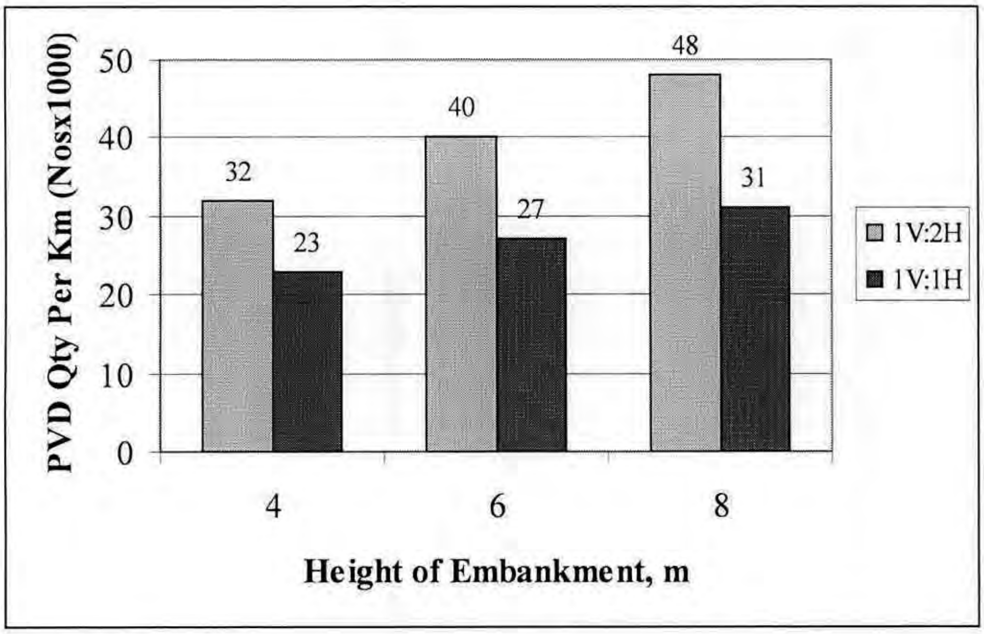

Figure 4.33: Comparison of PVD Quantity for Embankment (1 km) of Different Slopes

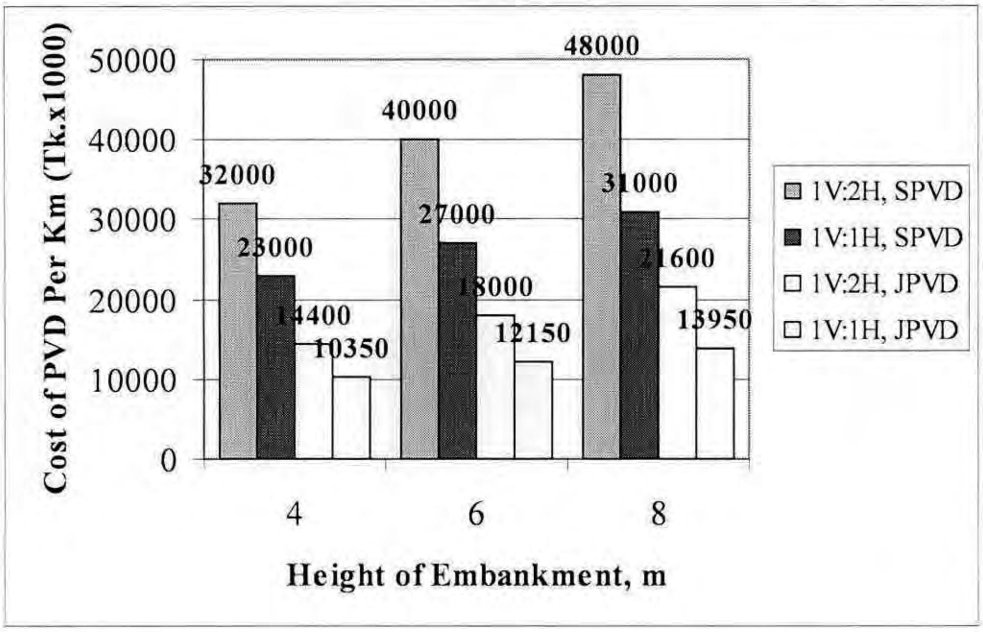

Figure 4.34: Cost Comparison of Synthetic PVD and Jute PVD for Embankment (1 km) of Different Slopes

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The main objective of the study was to investigate the feasibility of developing and using jute based PVD in accelerating the consolidation rate of soft soil under preloading technique. In doing so, properties of clay samples collected from a location of a link road project connecting Zia Colony (opposite to Hotel Radisson Water Garden) and Mirpur Cantonment, Dhaka, have been investigated in the BUET Geotechnical Laboratory. Jute PVD has been developed by BJRI and prepared with the help of BJMC that incorporate four coconut coir cores enveloped with two layers of DW Twill. a type of woven jute product. Settlement under preloading has been observed under three phases such as without vertical drains, with synthetic PVD and with jute PVD. Observed data has been analyzed to evaluate the feasibility of jute PVD.

5.2 CONCLUDING REMARKS

5.2.1 Index Properties of Mirpur Clay

The present study from the beginning, attempted to investigate the basic properties of soft Mirpur Clay. Index properties like Atterberg limits, specific gravity, water content, particle-size distribution etc were found out following standard test procedures. Grain size distribution showed that 53% were clay. 42% were silt and 5% were sand in the soil sample. Soil is classified as silty clay according to the U.S. Department of Agriculture. Atterberg limits included LL of 48.4%. PL of 20.8 and PI of 27.6%. According to unified soil classification system (USCS). soil is classified as CL which means inorganic clays with low to medium compressibility. As per AASHTO soil classification system. soil falls under group of A-7-6 (27). The test programme also included determination of natural water content (w_n) and field unit weight (γ_n) and the results obtained were 38.3% and 16.9 kN/m^3 respectively. Specific gravity of the soil sample was obtained to be 2.7. All of these results confirmed the soft consistency of collected sample.

5.2.2 Consolidation Parameters of Mirpur Clay

Test programme included investigation of consolidation parameters of Mirpur Clay. Oedometer test was performed with a low to medium stress levels, i.e. 10, 20. 40, 100 and 200 kPa as the preloading stress in the settlement tank was low i.e. 27.406 kPa. Compression index (C_c) was obtained to be 0.12 whereas; coefficient of consolidation (C_v) was 4.43 mm^2/min. The same test provided initial void ratio (e_0) of 1.3. With various empirical correlations of C_c, C_v and Atterberg Limits developed by earlier researchers, the soil consistency was verified to be soft.

5.2.3 Undrained Shear Strengths of Clay Sample

Shear strength was not determined on undisturbed sample. It was determined with vane shear apparatus on remolded soil slurry which was also used for performance test. Unconfined compression strength was found out for remolded soil slurry after the unloading of surcharge. Shear strength and unconfined compression strength of remolded soil slurry were 5.5-9.5 kPa and 12-13 kPa (only after removal of surcharge) respectively. These test results confirmed that the consistency of soil was very soft.

5.2.4 Properties of Synthetic PVD

No laboratory investigations on synthetic PVD were carried out to find out physical. mechanical and hydraulic conductivity characteristics. But in-soil performance test on synthetic PVD was conducted. For compliance with the requirements set by the user organizations. PVD needs to be tested. However. for analyzing its in-soil performance and comparing with jute PVD. test properties have been collected from the manufacturer of synthetic PVD. 'Flexi Drain'. a type of synthetic PVD has been used in the test which has fulfilled all the criteria set in the specification of Roads and Highways Department of Bangladesh.

5.2.5 Properties of Jute PVD

Jute PVD has not been manufactured for commercial marketing. PVD samples were prepared in Bangladesh for research purposes by Kabir et al. (1994), Abdullah et al. (1994) and Prodhan (1994) etc. Lee et al (1989) also developed the same samples in Singapore. For the present study jute PVD was prepared by BJMC which was not put under test. Samples prepared previously at Adamjee Jute Mills were collected and tested.

Tests on jute PVD covered a range of physical, mechanical and hydraulic conductivity characteristics. These tests are intended to investigate properties of both filter jacket made of DW Twill and in-built jute PVD system. Only the hydraulic conductivity tests were intended for investigating PVD system performance whereas physical and mechanical tests were needed for filter jacket made of DW Twill. In the present study, tests to find out thickness, apparent opening (AOS), permittivity (cross-plane permeability) and transmissivity (in-plane permeability) were conducted. Test results for complete range of specification are available in the work of Mohy (2005) and some of the test results have been used directly from that research work. Thickness of single DW Twill was found out to be 2.38 mm which was well within the range. AOS was found in this study to be 0.85 mm and that of Mohy was 0.8 mm. But the actual requirement was that the AOS should be less than 0.9 mm.

Transmissivity test has been conducted on filter fabric as single DW Twill and also on PVD system as a whole. In the first case, discharge rate through single DW Twill at 20.0 kPa normal stress was very low (flow rate $1.7X10^{-7}$ m^3/s and transmissivity $1.7X10^{-6}$ m^2/s-m). In-plane discharge rate of PVD system as a whole at a hydraulic gradient of 1.0 was found out to be 466.74 m^3/year but the specification required a value of more than 500 m^3/year. Grab tensile strength, trapezoidal tear strength, puncture resistance and burst strength were also included in the specification. The test results of these properties are taken from Mohy (2005) and compared. It is seen that the test results on DW Twill regarding these properties fulfill the specifications and also, to most extent, match in parallel with that of 'Flexi Drain'. Synthetic PVD provides excellent hydraulic conductivity in the in-plane flow which is even five times more than the requirement. Jute PVD falls short in this regard.

5.2.6 Soil Settlement without Vertical Drains

The main part of the study encompasses the in-soil performance test of jute PVD and compare with that of synthetic PVD. To prepare a base line of time-settlement (theoretical) profile, consolidation parameters (C_c, C_v, e_0, H_c) and time duration of 28 days have been considered. In the development of this profile, only the one-way drainage has been considered according to Terzaghi's theory of one dimensional consolidation. To conduct the performance test in the laboratory, a cylindrical steel tank of 930 mm height and 502 mm diameter was fabricated along with other accessories. In the first phase of the test, soil slurry of the test clay was prepared with mechanical mixer with controlled water content. After pouring the slurry in the tank, density and height of clay layer thickness (825 mm) was measured and recorded. Surcharge of 27.406 kPa was applied and settlement reading taken for 28 days after that the system was dismantled. Settlement reading with time duration was plotted and the resulting time-settlement profile was compared with that of theoretical (Terzaghi Formula) one. It is seen that the observed settlement of 25.82 mm in the steel tank and that of theoretical one were almost same, i.e. 57.8% and 57.5% respectively. But observed settlement profile was more prominent in the initial stages which showed a flatter trend later on.

5.2.7 Performance Test with Synthetic PVD

In the second phase of the test. soil slurry with same water content was prepared and filled in the settlement tank. The density and height of the slurry was maintained as of previous test. Before pouring the slurry in the tank, synthetic PVD was hung vertically and kept straight. Smear effect was avoided. A rigid steel plate was placed at the top to have equal strain case of radial drainage. Test progressed in the same manner as of first phase with applying surcharge of equal amount and settlement observed for 28 days. . The result showed that the total settlement of 29.28 mm took place which was 65.53% of the total settlement. Theoretically. settlement due to both vertical and radial drainage should have been 99.41%. From the profile it is also seen that it took only 16.8 days to achieve the same amount of settlement of the first phase (57.8%). i.e. without the vertical drains.

5.2.8 Performance Test with Jute PVD

In the third phase of the test, same set up was followed except that only jute PVD was used for radial drainage. Water content, density, soil slurry height and all other parameters were identical to the previous phases. Same amount of surcharge was placed and settlement readings were recorded for again 28 days. The time-settlement profile showed that total 30.62 mm settlement took place which was 68.53%. It is also seen that it took only 14.75 days to achieve the settlement (57.8%) that took place in the first phase. But according to the mathematical computation of simultaneous vertical and radial drainage condition, the settlement should have been 99.29%. This incompatibility of test results owe to some factors such as degree of saturation being 90% instead of 100%, occurrence of soil piping and clogging the in-plane channel of the drain etc.

5.3 RECOMMENDATIONS

Based on the findings of the present research, following propositions are recommended:

(1) Vertical drains accelerate the consolidation process thereby reduce the project duration. Due to inherent drawbacks of sand drains, PVD provides a better alternative which is efficient, faster in installation process and cost effective. Nowadays, synthetic PVD is being used in soft soil improvement in Bangladesh where jute PVD should be used as a suitable alternative.

(2) Use of PVD for improving soft soil is meant for a limited time which may be 6 months to maximum 2 years. After achieving the purpose of vertical drainage, a PVD becomes redundant. Synthetic PVD which is already being used in Bangladesh is costly and is a potential source of environmental degradation. Based on the present findings, jute PVD is recommended to be included in the specifications and schedule of rates of all concerned government organizations.

(3) Before the full scale use of jute PVD in the soft soil improvement, a trial embankment should be taken for a short stretch of 100 to 200 m where its performance may be tested on ground. Organizations like Roads and Highways

Department, Local Government Engineering Department, City Corporations etc may take up this sort of initiative and test the feasibility of jute PVD. This initiative, if found positive, is going to save hard-earned foreign currency and also rejuvenate the once-famous jute industry in the domestic market.

5.4 RECOMMENDATIONS FOR FUTURE STUDY

An important aspect of this research was to study the in-soil performance evaluation of jute PVD. Performance evaluation has been conducted in the laboratory where field conditions could not be depicted properly. Refinements and improvements in the design and evaluation are possible if further studies are carried out in this regard. Following are some of the scopes that may be accepted for further study:

(1) It is already seen in the index tests of jute PVD that it falls short in fulfilling the specification of PVD in terms of in-plane discharge rate (466.74 m^3/year as against minimum 500 m^3/year). Since in-plane discharge rate depends on the hydraulic conductivity of core/channel, increasing the number of core rope by one or two will certainly enhance the discharge rate. Another approach may be to increase the diameter of the coconut coir core. Sample of jute PVD should be designed and tested with increased number of core rope/ropes or with larger diameter of core.

(2) Only one type of jute PVD has been tested for in-soil performance evaluation. Since the AOS was also very marginal (0.85 mm as against <0.90 mm), there is likelihood of infiltrating larger soil particles in the core and clog the PVD before its functional life. So, jute PVD should be made with variation in number of core and that of filter fabric having different AOS.

(3) Jute PVD made of untreated jute product is unlikely to sustain a soil-water environment for more than 2.5 to 3 months whereas a PVD should remain functional minimum for six months. In this case, treatment of jute to some extent is indispensable. Treatment might reduce the filtration capability of filter fabric made of DW Twill. The present research has been conducted with

untreated product. So, there is of course a need for conducting the same study with treated jute materials.

(4) Since theoretical formulations regarding vertical drains are basically related to sand drains, performance test with sand drain may be conducted to be more confident in comparing the performance test results of synthetic and jute PVD

(5) Under preloading technique, removal of surcharge and commencement of actual construction is delayed until 80% to 90% settlement is achieved. In the present study, only 69% settlement was achieved in 4 weeks and after that the set-up was dismantled. Future study in the laboratory should try to conduct settlement test and continue until 90% settlement is achieved which would give a better performance evaluation of the test sample.

(6) Due to inherent limitations (clay remolded, smear effect avoided and equal strain case) of laboratory tests, field performance of jute PVD could not yet be determined. A full-scale field test with jute PVD alongside a synthetic PVD for a short stretch of embankment should be undertaken in future.

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Appendix A

Typical Values and Correlations

A.1 Typical Values of C_v

Ser	Soil	Coefficient of Consolidation (C_v)	
		m^2/day	ft^2/day
1.	Boston blue clay(CL) (Ladd & Luscher, 1956)	0.033 ±0.016	0.33 ± 0.16
2.	Organic silt(OH), (Lowe, Zaccheo and Feldman, 1964)	0.00016 – 0.00082	0.0016 – 0.0082
3.	Glacial lake clays(CL), (Wallace and Otto,1964)	0.00055 – 0.00074	0.0055 – 0.0074
4.	Chicago silty clay (CL), (Terzaghi and Peck, 1967)	0.00074	0.0074
5.	Swedish medium sensitive clays (CL-CH), (Holtz and Broms,1972	0.0003 – 0.0006 (Lab) 0.0006 – 0.0033(Field)	0.003 – 0.006(Lab) 0.006 – 0.033(Field)
6.	San Francisco bay mud(CL)	0.0016 – 0.0033	0.016 – 0.033
7.	Mexico City clay (MH), (Leonards and Girault, 1961)	0.0008 – 0.0014	0.008 – 0.014

After Coduto (2007)

A.2 Classification of Soil Compressibility

Ser No	$\frac{C_c}{1+e_0}$ *or* $\frac{C_r}{1+e_0}$	Classification
1.	0-0.05	Very slightly compressible
2.	0.05-0.10	Slightly compressible
3.	0.10-0.20	Moderately compressible
4.	0.20-0.35	Highly compressible
5.	>0.35	Very highly compressible

For soils that are normally consolidated, base the classification on $\frac{C_c}{1+e_0}$. For soils that are overconsolidated, base it on $\frac{C_r}{1+e_0}$, Coduto (2007).

A.3 Correlation of C_c with LL and Water Content

- C_c = 0.009(LL-10), for normally consolidated clays, Terzaghi and Peck (1967).
- C_c = 0.007(LL-10), for remolded clays, Terzaghi and Peck (1967).
- C_c = 0.01w_n, (w_n is the natural water content), for Chicago clays, Azzouz et al. (1976).
- C_c = 0.046(LL-9), for Brazilian clays, Azzouz et al. (1976).
- C_c = 1.21−1.055(e_0-1.87), for Motley clays from Sao Paulo City), Azzouz et al. (1976).
- C_c = 0.208e_0 + 0.083, for Chicago clays, Azzouz et al. (1976).
- C_c = 0.0115w_n, for organic soils, peats, etc. Azzouz et al. (1976).
- C_c = 0.02 + 0.014PI, for natural deep ocean soils, Nacci et al. (1975).

A.4 Typical Values of Specific Gravity (G_s)

Ser No	Soil Type	Specific Gravity(G_s)
1.	Gravel	2.65 – 2.68
2.	Sand	2.65 – 2.68
3.	Silt	2.66 – 2.7
4.	Clay	2.68 – 2.8
5.	Soils with Micas or Iron	2.73 – 3.00
6.	Organic Soils	Variable but may be under 2.00

After Bowles (1978) & Das, (1985)

A.5 Characteristics of Soils with Different Plasticity Indices

Plasticity Index, I_p	Classifications	Dry Strength	Visual-Manual Identifications of Dry Sample
0 – 3	Nonplastic	Very low	Falls apart easily
3 - 5	Slightly plastic	Slight	Easily crushed with fingers
15 – 30	Medium plastic	Medium	Difficult to crush with fingers
>30	Highly plastic	High	Impossible to crush with fingers

A.6: Typical Values of Natural Water Content in a Saturated State

Serial No	Soil Type	Natural Water Content (%)
1.	Loose uniform sand	25-30
2.	Dense uniform sand	12-16
3.	Loose angular-grained silty sand	25
4.	Dense angular-grained silty sand	15
5.	Stiff clay	20
6.	**Soft clay**	**30-50**
7.	Soft organic clay	80-130
8.	Glacial till	10

After Das, (1997)

A.7: Commonly Adopted Methods for Installing Vertical Drains

Serial No	Group Description	Particular Methods	Remarks
1.	Displacement methods	Driving Vibration Pull down(static force) Washing Combinations of above	A mandrel with or without a disposable shoe is used in each case
2.	Drilling Methods	Rotary drill with or without a casing Rotary auger. including continuous standard and hollow flight augers Percussive(shell and auger) methods. with or without casing Hand auger.	
3.	Washing methods	Rotary wash jet Washed open ended casing Weighted wash jet head on flexible hose	Methods in which sand is washed via the jet pipe are not suitable for PVDs.

After Megown et al (1982)

A.8: Details of Some Band Drains

Serial No	Type	Core material	Filter Material	Dimension, mm
1.	Kjellman	Paper	Paper	100x3
2.	PVC	PVC	None	100x2
3.	Geodrain	Polyethylene	Cellulose	95x4
4.	Mebradrain	Polypropylene	Polypropylene or Polyester	95x3
5.	Alidrain	Polyethylene	Polyester	100x6
6.	Colbond	Polyester	Polyester	100x6
7.	Hitek	Polyethylene	Polypropylene	100x6

After Hausmann (1990)

Appendix B

Laboratory Test Results

Hydrometer Analysis

Soil Sample: Grey — Test No: 1

Location: Mirpur cantt — Type of Hydrometer: 152H

Sample No: 1 — Hydrometer No: 2

Date of Test: 26/02/2008 — Wt. of sample: 50 gm

Cr 4.5

Sp. Gravity, G_s: 2.7 — Meniscus Correction, C_m: 0.5 cm

Multiplier, a 1.65*G_s/2.65(G_s-1) 0.988

Dispersing Reagent: 4% Solution of Na HexametaPhosphate

Date	Time	Elapsed time t (min)	Room Temp °C	Hydro Reading Ra	Reading After Meniscus Correction R=Ra-Cm	Effective Depth L cm	Corrected Reading Rc=R-Cr±Ct	Value of K	Particle Size(mm) D=k√(L/t)	Percent Finer= Rc*a*100/Ws
19/02/08	9:11	0.25	22	52.5	53	7.7	48.5	0.013	0.07270	95.9233
19/02/08	9:11	0.5	22	52	52.5	7.75	48	0.013	0.05157	94.9344
19/02/08	9:12	1	22	51.5	52	7.8	47.5	0.013	0.03659	93.9455
19/02/08	9:13	2	22	50.5	51	7.9	46.5	0.013	0.02604	91.9677
19/02/08	9:15	4	22	49	49.5	8.2	45	0.013	0.01876	89.001
19/02/08	9:19	8	22	47.5	48	8.4	43.5	0.013	0.01342	86.0343
19/02/08	9:26	15	22	45.5	46	8.8	41.5	0.013	0.01003	82.0787
19/02/08	9:41	30	22	42.5	43	9.2	38.5	0.013	0.00725	76.1453
19/02/08	10:11	60	22	40	40.5	9.65	36	0.013	0.00525	71.2008
19/02/08	11:11	120	22	37.5	38	10.1	33.5	0.013	0.00380	66.2563
19/02/08	13:11	240	22	34.5	35	10.6	30.5	0.013	0.00275	60.3229
19/02/08	17:11	480	22	31	31.5	11.15	27	0.013	0.00200	53.4006
20/02/08	9:11	1440	22	23	23.5	12.45	19	0.013	0.00122	37.5782
21/02/08	9:36	2910	21.5	19.5	20	13	15.5	0.013	0.00088	30.6559
22/02/08	9:10	4319	21	18	18.5	13.25	14	0.013	0.00074	27.6892
24/02/08	15:32	7581	22	16	16.5	13.6	12	0.013	0.00055	23.7336

Atterberg Limit Test

Soil Sample:	Mirpur Clay, Black at Natural Moisture Content	Test no:	1
		Date:	17/02/08
Depth:	2 m	Tested by:	Md Wahidul Islam

Liquid Limit

No of Blows	Container No	Wt. Container gm	Wt. Cont+ Wet Soil	Wt. Cont– Dry Soil	Wt. Water Ww gm	Wt. Dry Soil, Ws gm	Water Content,%
16	820	7.49	16.09	13.1	2.99	5.61	53.30
21	306	7.3	15.17	12.55	2.62	5.25	49.90
24	608	6.5	12.5	10.55	1.95	4.05	48.15
29	881	7	16.33	13.35	2.98	6.35	46.93
40	740	7.33	14.98	12.65	2.33	5.32	43.80

Plastic Limit

Container No	Wt. Container gm	Wt. Cont+ Wet Soil	Wt. Cont– Dry Soil	Wt. Water Ww gm	Wt. Dry Soil, Ws gm	Water Content,%
206	7.13	11.31	10.61	0.7	3.48	20.11
9	7.68	13.71	12.66	1.05	4.98	21.08
138	7.29	15.41	13.96	1.45	6.67	21.74

Result Summary

Plastic Limit	Natural Water Content.%	Liquid Limit LL. %	Plasticity Index. PI
20.98	38.4	48.4	27.42

Natural Water Content

Soil Sample:	Mirpur Clay, Black at Natural Moisture Content	Test no:	1
		Date:	29/01/08
Depth:	2 m	Tested by:	Md Wahidul Islam

Container No	Wt. Cont. gm	Wt. Cont+ Wet Soil	Wt. Cont+ Dry Soil	Wt. Water Ww gm	Wt. Dry Soil, Ws gm	Water Content,%
39	6.97	22.1	17.87	4.23	10.9	38.81
716	7.1	19.35	15.97	3.38	8.87	38.11
155	7.05	25.61	20.5	5.11	13.45	37.99

Vane Shear Test of Soil Slurry without Vertical Drain (Before Application of Surcharge)

Reading No	Dial Reading	Calibration Factor	Shear Strength, kpa	Average Shear Strength, kPa
1	4.9	1.145	5.61	
	5.2	1.145	5.95	
3	4.8	1.145	5.5	6.01
4	5.3	1.145	6.07	
5	5.6	1.145	6.41	
6	5.7	1.145	6.53	

Vane Shear Test of Soil Slurry without Vertical Drain (After the Removal of Surcharge)

Reading No	Dial Reading	Calibration Factor	Shear Strength kpa	Average Shear Strength kPa
1	7.4	1.145	8.47	
2	7.8	1.145	8.93	
3	8.20	1.145	9.39	
4	7.9	1.145	9.05	8.95
5	7.7	1.145	8.82	
6	7.9	1.145	9.05	

Vane Shear Test When Synthetic PVD Used (Before the Application of Surcharge)

Reading No	Dial Reading	Calibration Factor	Shear Strength kpa	Average Shear Strength kPa
1	5.0	1.145	5.73	
2	5.20	1.145	5.95	
3	4.90	1.145	5.61	
4	5.30	1.145	6.07	**5.86**
5	5.20	1.145	5.95	
6	5.10	1.145	5.84	

Vane Shear Test When Synthetic PVD Used (After the Removal of Surcharge)

Reading No	Dial Reading	Calibration Factor	Shear Strength kpa	Average Shear Strength, kPa
1	8.0	1.145	9.16	
2	7.50	1.145	8.59	
3	7.75	1.145	8.87	**9.11**
4	8.00	1.145	9.16	
5	8.40	1.145	9.62	
6	8.10	1.145	9.27	

Vane Shear Test When Jute PVD Used (Before the Application of Surcharge)

Reading No	Dial Reading	Calibration Factor	Shear Strength kpa	Average Shear Strength kPa
1	4.90	1.145	5.61	
2	5.20	1.145	5.95	
3	4.80	1.145	5.5	
4	5.30	1.145	6.07	**5.82**
5	5.20	1.145	5.95	
6	5.10	1.145	5.84	

Vane Shear Test When Jute PVD Used (After the Removal of Surcharge)

Reading No	Dial Reading	Calibration Factor	Shear Strength, kpa	Average Shear Strength, kPa
1	8.10	1.145	9.27	
2	8.0	1.145	9.16	
3	7.90	1.145	9.05	
4	8.20	1.145	9.39	**9.24**
5	8.30	1.145	9.50	
6	7.90	1.145	9.05	

Specific Gravity Determination

Conducted on(Date)	19/02/2008	26/02/08
Bottle No	A	B
Wt of Bottle+ Water+ Soil(W_1) gm	372.88	372.19
Temperature T in °C	21^0	29^0
Wt of Bottle + Water (W_2) gm	341.85	340.8
Evaporating Dish No	3	9
Wt. of Dish, gm	161.05	151.6
Wt. of Dish + Dry Soil, gm	210.88	201.3
Wt. of Bottle gm	92.77	92.8
Wt. of Dry Soil W_s gm	49.83	49.7
Sp. Gravity of water G_T at T^0 C	0.9998	0.9977
Sp. Gravity of Soil $G_s = G_T * W_s/(W_s - W_1 + W_2)$	2.69	2.71

Unconfined Compression Test-1 (Test with Synthetic PVD)

Proj Name:	M.Sc. Thesis	Job No:	1
Location :	Mirpur Cantt	Date:	06/07/08
Sample No:	1	Wt of wet sample:	157.1 gm
Depth:	2 m	Wt of Dry Sample:	113.20 gm
Initial Diameter:	1.5 inch		
Initial Area Ao=	1.76714 sq in		
Initial Height:	3 inch		
Initial Volume Vo=	5.3014 cu inch		

Load	Axial	Displacem.	Tota	L	Unit	Ao	Corr. Area	Stress
Dial	Load	Dial	Displ.	(in)	Strain	sq. in	Ac=	P/Ac
(.0001 in)	P (lb)	(.001 in)	ΔL (in)		ε=ΔL/L		Ao/(1-ε)	(kPa)
0	0.00	0	0	3	0	1.76714	1.767	0.000
0	0.00	3	0.003	3	0.001	1.76714	1.769	0.000
0	0.00	6	0.006	3	0.002	1.76714	1.771	0.000
0	0.00	9	0.009	3	0.003	1.76714	1.772	0.000
0	0.00	12	0.012	3	0.004	1.76714	1.774	0.000
0	1.19	15	0.015	3	0.005	1.76714	1.776	4.625
0.2	1.25	30	0.03	3	0.01	1.76714	1.785	4.842
0.4	1.31	45	0.045	3	0.015	1.76714	1.794	5.056
0.75	1.42	60	0.06	3	0.02	1.76714	1.803	5.447
1.25	1.58	75	0.075	3	0.025	1.76714	1.812	6.010
2.1	1.84	90	0.09	3	0.03	1.76714	1.822	6.979
3	2.12	120	0.12	3	0.04	1.76714	1.841	7.955
3.75	2.36	150	0.15	3	0.05	1.76714	1.860	8.736
4.8	2.68	180	0.18	3	0.06	1.76714	1.880	9.841
5.5	2.90	210	0.21	3	0.07	1.76714	1.900	10.525
6	3.05	240	0.24	3	0.08	1.76714	1.921	10.970
6.75	3.29	270	0.27	3	0.09	1.76714	1.942	11.678
7.2	3.43	300	0.3	3	0.1	1.76714	1.963	12.041
8	3.67	330	0.33	3	0.11	1.76714	1.986	12.771
8	3.67	360	0.36	3	0.12	1.76714	2.008	12.627
8	3.67	390	0.39	3	0.13	1.76714	2.031	12.484
8	3.67	420	0.42	3	0.14	1.76714	2.055	12.340
8	3.67	450	0.45	3	0.15	1.76714	2.079	12.197

Unconfined Compression Test-2(Test with Synthetic PVD)

Proj Name:	M.Sc. Thesis	Job No:	2
Location :	Mirpur Cantt	Date:	06/07/08
Sample No:	2	Wt of wet sample:	157.8 gm
Depth:	2 m	Wt of Dry Sample:	113.5.0 gm

Initial Diameter:	1.5 inch
Initial Area Ao=	1.76714 sq in
Initial Height:	3 inch
Initial Volume Vo=	5.3014 cu inch

Load	Axial	Displacem.	Tota	L	Unit	Ao	Corr. Area	Stress
Dial	Load	Dial	Displ.	(in)	Strain	sq. in	Ac=	P/Ac
(.0001 in)	P (lb)	(.001 in)	ΔL (in)		ε=ΔL/L		Ao/(1-ε)	(kPa)
0	0.00	0	0	3	0	1.76714	1.767	0.000
0	0.00	3	0.003	3	0.001	1.76714	1.769	0.000
0	0.00	6	0.006	3	0.002	1.76714	1.771	0.000
0	0.00	9	0.009	3	0.003	1.76714	1.772	0.000
0	0.00	12	0.012	3	0.004	1.76714	1.774	0.000
0	1.19	15	0.015	3	0.005	1.76714	1.776	4.625
0	1.19	30	0.03	3	0.01	1.76714	1.785	4.602
0.3	1.28	45	0.045	3	0.015	1.76714	1.794	4.937
0.7	1.41	60	0.06	3	0.02	1.76714	1.803	5.387
1.2	1.56	75	0.075	3	0.025	1.76714	1.812	5.951
2	1.81	90	0.09	3	0.03	1.76714	1.822	6.861
2.7	2.03	120	0.12	3	0.04	1.76714	1.841	7.606
3.5	2.28	150	0.15	3	0.05	1.76714	1.860	8.448
4.25	2.51	180	0.18	3	0.06	1.76714	1.880	9.214
5.2	2.81	210	0.21	3	0.07	1.76714	1.900	10.187
6	3.05	240	0.24	3	0.08	1.76714	1.921	10.970
6.5	3.21	270	0.27	3	0.09	1.76714	1.942	11.403
6.8	3.30	300	0.3	3	0.1	1.76714	1.963	11.605
7.4	3.49	330	0.33	3	0.11	1.76714	1.986	12.123
7.4	3.49	360	0.36	3	0.12	1.76714	2.008	11.987
7.4	3.49	390	0.39	3	0.13	1.76714	2.031	11.851
7.4	3.49	420	0.42	3	0.14	1.76714	2.055	11.715
7.4	3.49	450	0.45	3	0.15	1.76714	2.079	11.578

Unconfined Compression Test-3(Test with Synthetic PVD)

Proj Name:	M.Sc. Thesis	Job No:	3
Location:	Mirpur Cantt	Date:	06/07/08
Sample No:	3	Wt of wet sample:	158.0 gm
Depth:	2 m	Wt of Dry Sampie:	113.6.0 gm
Initial Diameter:	1.5 inch		
Initial Area Ao=	1.76714 sq in		
Initial Height:	3 inch		
Initial Volume Vo=	5.3014 cu inch		

Load	Axial	Displace m.	Tota	L	Unit	Ao	Corr. Area	Stress
Dial	Load	Dial	Displ.	(in)	Strain	sq. in	Ac=	P/Ac
(.0001 in)	P (lb)	(.001 in)	ΔL (in)		ε=ΔL/L		Ao/(1-ε)	(kPa)
0	0.00	0	0	3	0	1.76714	1.767	0.000
0	0.00	3	0.003	3	0.001	1.76714	1.769	0.000
0	0.00	6	0.006	3	0.002	1.76714	1.771	0.000
0	0.00	9	0.009	3	0.003	1.76714	1.772	0.000
0	0.00	12	0.012	3	0.004	1.76714	1.774	0.000
0.2	1.25	15	0.015	3	0.005	1.76714	1.776	4.866
0.4	1.31	30	0.03	3	0.01	1.76714	1.785	5.082
0.75	1.42	45	0.045	3	0.015	1.76714	1.794	5.474
1	1.50	60	0.06	3	0.02	1.76714	1.803	5.744
1.25	1.58	75	0.075	3	0.025	1.76714	1.812	6.010
2	1.81	90	0.09	3	0.03	1.76714	1.822	6.861
2.6	2.00	120	0.12	3	0.04	1.76714	1.841	7.489
3.1	2.15	150	0.15	3	0.05	1.76714	1.860	7.987
4.2	2.49	180	0.18	3	0.06	1.76714	1.880	9.157
5	2.74	210	0.21	3	0.07	1.76714	1.900	9.962
5.6	2.93	240	0.24	3	0.08	1.76714	1.921	10.524
6.2	3.12	270	0.27	3	0.09	1.76714	1.942	11.072
6.8	3.30	300	0.3	3	0.1	1.76714	1.963	11.605
7.1	3.46	330	0.33	3	0.11	1.76714	1.986	12.015
7.1	3.46	360	0.36	3	0.12	1.76714	2.008	11.880
7.1	3.46	390	0.39	3	0.13	1.76714	2.031	11.745
7.1	3.46	420	0.42	3	0.14	1.76714	2.055	11.610
7.1	3.46	450	0.45	3	0.15	1.76714	2.079	11.475

Unconfined Compression Test-4 (Test with Jute PVD)

Proj Name:	M.Sc. Thesis		Job No:	4
Location :	Mirpur Cantt		Date:	06/10/08
Sample No:	4		Wt of wet sample:	156.1 gm
Depth:	2 m		Wt of Dry Sample:	112.50 gm
Initial Diameter:	1.5 inch			
Initial Area Ao=	1.76714 sq in			
Initial Height:	3 inch			
Initial Volume Vo=	5.3014 cu inch			

Load Dial (.0001 in)	Axial Load P (lb)	Displacement Dial (.001 in)	Tota Displ. ΔL (in)	L (in)	Unit Strain ε=ΔL/L	Ao sq. in.	Corr. Area Ac=Ao/(1-ε)	Stress P/Ac (kPa)
0	0.00	0	0	3	0	1.7671	1.767	0.000
0	0.00	3	0.003	3	0.001	1.7671	1.769	0.000
0	0.00	6	0.006	3	0.002	1.7671	1.771	0.000
0	0.00	9	0.009	3	0.003	1.7671	1.772	0.000
0	0.00	12	0.012	3	0.004	1.7671	1.774	0.000
0.25	1.27	15	0.015	3	0.005	1.7671	1.776	4.927
0.8	1.44	30	0.03	3	0.01	1.7671	1.785	5.562
1.25	1.58	45	0.045	3	0.015	1.7671	1.794	6.072
1.75	1.73	60	0.06	3	0.02	1.7671	1.803	6.635
2.5	1.97	75	0.075	3	0.025	1.7671	1.812	7.488
2.9	2.09	90	0.09	3	0.03	1.7671	1.822	7.920
3.5	2.28	120	0.12	3	0.04	1.7671	1.841	8.537
4	2.43	150	0.15	3	0.05	1.7671	1.860	9.024
4.6	2.62	180	0.18	3	0.06	1.7671	1.880	9.613
5.2	2.81	210	0.21	3	0.07	1.7671	1.900	10.187
6	3.05	240	0.24	3	0.08	1.7671	1.921	10.970
6.7	3.27	270	0.27	3	0.09	1.7671	1.942	11.623
7.3	3.46	300	0.3	3	0.1	1.7671	1.963	12.150
8	3.67	330	0.33	3	0.11	1.7671	1.986	12.771
8.5	3.83	360	0.36	3	0.12	1.7671	2.008	13.161
8.5	3.83	390	0.39	3	0.13	1.7671	2.031	13.011
8.5	3.83	420	0.42	3	0.14	1.7671	2.055	12.862
8.5	3.83	450	0.45	3	0.15	1.7671	2.079	12.712

Unconfined Compression Test-5 (Test with Jute PVD)

Proj Name:	M.Sc. Thesis	Job No:	5
Location :	Mirpur Cantt	Date:	06/10/08
Sample No:	5	Wt of wet sample:	157.2 gm
Depth:	2 m	Wt of Dry Sample:	113.0 gm

Initial Diameter:	1.5 inch
Initial Area Ao=	1.76714 sq in
Initial Height:	3 inch
Initial Volume Vo=	5.3014 cu inch

Load Dial (.0001 in)	Axial Load P (lb)	Displacement Dial (.001 in)	Tota Displ. ΔL (in)	L (in)	Unit Strain ε=ΔL/L	Ao sq. in	Corr. Area Ac=Ao/(1-ε)	Stress P/Ac (kPa)
0	0.00	0	0	3	0	1.7671	1.767	0.000
0	0.00	3	0.003	3	0.001	1.7671	1.769	0.000
0	0.00	6	0.006	3	0.002	1.7671	1.771	0.000
0	0.00	9	0.009	3	0.003	1.7671	1.772	0.000
0	0.00	12	0.012	3	0.004	1.7671	1.774	0.000
0.25	1.27	15	0.015	3	0.005	1.7671	1.776	4.927
1	1.50	30	0.03	3	0.01	1.7671	1.785	5.802
1.5	1.66	45	0.045	3	0.015	1.7671	1.794	6.370
2	1.81	60	0.06	3	0.02	1.7671	1.803	6.932
2.75	2.04	75	0.075	3	0.025	1.7671	1.812	7.783
3.2	2.18	90	0.09	3	0.03	1.7671	1.822	8.273
3.75	2.36	120	0.12	3	0.04	1.7671	1.841	8.828
4.2	2.49	150	0.15	3	0.05	1.7671	1.860	9.254
4.7	2.65	180	0.18	3	0.06	1.7671	1.880	9.727
5.2	2.81	210	0.21	3	0.07	1.7671	1.900	10.187
5.8	2.99	240	0.24	3	0.08	1.7671	1.921	10.747
6.2	3.12	270	0.27	3	0.09	1.7671	1.942	11.072
6.8	3.30	300	0.3	3	0.1	1.7671	1.963	11.605
7.5	3.52	330	0.33	3	0.11	1.7671	1.986	12.231
7.5	3.52	360	0.36	3	0.12	1.7671	2.008	12.094
7.5	3.52	390	0.39	3	0.13	1.7671	2.031	11.956
7.5	3.52	420	0.42	3	0.14	1.7671	2.055	11.819
7.5	3.52	450	0.45	3	0.15	1.7671	2.079	11.682

Unconfined Compression Test-6(Test with Jute PVD)

Proj Name:	M.Sc. Thesis	Job No:	6
Location :	Mirpur Cantt	Date:	06/10/08
Sample No:	6	Wt of wet sample:	156.8 gm
Depth:	2 m	Wt of Dry Sample:	112.5.0 gm
Initial Diameter:	1.5 inch		
Initial Area Ao=	1.76714 sq in		
Initial Height:	3 inch		
Initial Volume Vo=	5.3014 cu inch		

Load Dial (.0001 in)	Axial Load P (lb)	Displacement Dial (.001 in)	Tota Displ. ΔL (in)	L (in)	Unit Strain ε=ΔL/L	Ao Sq. in.	Corr. Area Ac=Ao/(1-ε)	Stress P/Ac (kPa)
0	0.00	0	0	3	0	1.7671	1.767	0.000
0	0.00	3	0.003	3	0.001	1.7671	1.769	0.000
0	0.00	6	0.006	3	0.002	1.7671	1.771	0.000
0	0.00	9	0.009	3	0.003	1.7671	1.772	0.000
0	0.00	12	0.012	3	0.004	1.7671	1.774	0.000
0	1.19	15	0.015	3	0.005	1.7671	1.776	4.625
0.25	1.27	30	0.03	3	0.01	1.7671	1.785	4.902
0.75	1.42	45	0.045	3	0.015	1.7671	1.794	5.474
1.4	1.63	60	0.06	3	0.02	1.7671	1.803	6.219
1.7	1.72	75	0.075	3	0.025	1.7671	1.812	6.542
2.6	2.00	90	0.09	3	0.03	1.7671	1.822	7.567
3.5	2.28	120	0.12	3	0.04	1.7671	1.841	8.537
4.3	2.53	150	0.15	3	0.05	1.7671	1.860	9.369
5.3	2.84	180	0.18	3	0.06	1.7671	1.880	10.411
6.1	3.08	210	0.21	3	0.07	1.7671	1.900	11.202
6.9	3.33	240	0.24	3	0.08	1.7671	1.921	11.974
7.5	3.52	270	0.27	3	0.09	1.7671	1.942	12.506
8.3	3.77	300	0.3	3	0.1	1.7671	1.963	13.242
8.75	3.91	330	0.33	3	0.11	1.7671	1.986	13.580
8.75	3.91	360	0.36	3	0.12	1.7671	2.008	13.428
8.75	3.91	390	0.39	3	0.13	1.7671	2.031	13.275
8.75	3.91	420	0.42	3	0.14	1.7671	2.055	13.122
8.75	3.91	450	0.45	3	0.15	1.7671	2.079	12.970

Oedometer Test Results for Various Loads (T_{50} Method)

Settlement Reading Time vs Dial Reading														
Time (min)	5 kPa	Time (min)	10 kPa	Time (min)	20 kPa	Time (min)	40 kPa	Time (min)	100 kPa	Time (min)	200 kPa	100 kPa	50 kPa	5 kPa
0.1	300	0.1	305	0.1	436	0.1	547	0.1	735	0.1	972			
0.25	300	0.25	306	0.25	437	0.25	550	0.25	743	0.25	976			
0.5	300	0.5	308	0.5	441	0.5	556	0.5	752	0.5	983			
1	300	1	313	1	444	1	564	1	760	1	992			
2	300	2	335	2	450	2	572	2	766	2	999			
4	300	4	352	4	464	4	584	4	777	4	1015			
8	300	8	368	8	479	8	605	8	799	8	1038			
16	300	16	388	16	495	16	622	16	817	16	1053			
30	300	30	401	30	503	30	637	30	839	30	1069			
60	300	60	409	60	511	60	653	60	864	60	1082			
120	300	120	418	120	517	120	664	120	882	120	1093			
240	300	240	423	240	522	240	670	240	891	240	1100			
480	300	480	426	480	527	480	676	480	900	480	1107			
1440	300	1440	428	1440	530	1440	684	1440	911	1440	1114	1090	1068	983

Calculation of Oedometer Test Results (Time Vs Load) (T_{50} Method)

Settlement (mm) Time vs. load (kPa)															
Time min	5 kPa	Time min	10 kPa	Time min	20 kPa	Time min	40 kPa	Time min	100 kPa	Time min	200 kPa	100 kPa	50 kPa	5 kpa /Initial Reading	
0.10	0	0.10	0.0127	0.10	0.3454	0.10	0.6274	0.10	1.1049	0.10	1.7069				300
0.25	0	0.25	0.01524	0.25	0.348	0.25	0.635	0.25	1.1252	0.25	1.717				300
0.50	0	0.50	0.02032	0.50	0.3581	0.50	0.6502	0.50	1.1481	0.50	1.7348				300
1.00	0	1.00	0.03302	1.00	0.3658	1.00	0.6706	1.00	1.1684	1.00	1.7577				300
2.00	0	2.00	0.0889	2.00	0.381	2.00	0.6909	2.00	1.1836	2.00	1.7755				300
4.00	0	4.00	0.13208	4.00	0.4166	4.00	0.7214	4.00	1.2116	4.00	1.8161				300
8.00	0	8.00	0.17272	8.00	0.4547	8.00	0.7747	8.00	1.2675	8.00	1.8745				300
16.00	0	16.00	0.22352	16.00	0.4953	16.00	0.8179	16.00	1.3132	16.00	1.9126				300
30.00	0	30.00	0.25654	30.00	0.5156	30.00	0.856	30.00	1.3691	30.00	1.9533				300
60.00	0	60.00	0.27686	60.00	0.5359	60.00	0.8966	60.00	1.4326	60.00	1.9863				300
120.00	0	120.00	0.29972	120.00	0.5512	120.00	0.9246	120.00	1.4783	120.00	2.0142				300
240.00	0	240.00	0.31242	240.00	0.5639	240.00	0.9398	240.00	1.5011	240.00	2.032				300
480.00	0	480.00	0.32004	480.00	0.5766	480.00	0.955	480.00	1.524	480.00	2.0498				300
1440.00	0	1440.00	0.32512	1440.00	0.5842	1440.00	0.9754	1440.0	1.5519	1440.00	2.0676	2.0066	1.9507	1.7348	300
t_{50}	80.91		7.24		8.30		6.90		7.13		6.32				
H	25.40		25.07488		24.816		24.42		23.85		23.33	23.39	23.45	23.67	
$H_{Average}$	25.40		25.24		24.95		24.62		24.14		23.59	23.62	23.42	23.56	
ΔH	0		0.32512		0.5842		0.9754		1.5519		2.0676	2.0066	1.950	1.7348	
Δe	0.000		0.029		0.053		0.088		0.140		0.187	0.181	0.176	0.157	
e_o	1.30		1.30		1.30		1.30		1.30		1.30	1.30	1.30	1.30	
e	1.30		1.27		1.24		1.21		1.16		1.11	1.12	1.12	1.14	

Determination of C_c and C_v (T_{50} Method)

p	e	de	dp	$1+e_i$	m_v	t_{50}	H_{avg}	H^2_{avg}	c_v
	1.30								
5	1.30	0.00	5	2.30	0.00	80.91	25.40	645	0.393
10	1.27	0.03	5	2.30	2.56	7.24	25.24	637	4.333
20	1.24	0.02	10	2.27	1.03	8.30	24.95	622	**3.693**
40	1.21	0.04	20	2.24	0.79	6.90	24.62	606	**4.328**
100	1.16	0.05	60	2.21	0.39	7.13	24.14	583	4.023
200	1.11	0.05	100	2.16	0.22	6.32	23.59	556	4.336
100	1.12	-0.01	-100						
50	1.12	-0.01	-50						
5	1.14	-0.02	-45						

logp	logp 1- logp 2	e_1-e_2	C_c
1.00			
1.30	0.30	0.02	0.078
1.60	0.30	0.04	**0.118**
2.00	0.40	0.05	**0.131**
2.30	0.30	0.05	0.155

Dial Reading for Various Loads ($\sqrt{t_{90}}$ Method)

Dial reading

0 kpa	For load 10 kPa	For load = 20 kPa	For load = 40 kPa	For load = 100 kPa	For load = 200 kPa
	304	432	535	718	949
	305	436	547	735	972
	307	437	550	743	976
	311	441	556	752	983
	317	444	564	759	990
	332	450	572	766	999
	355	464	584	777	1015
	379	479	605	797	1036
	395	495	622	817	1053
	405	503	637	840	1069
	407	511	653	864	1083
	418	517	664	882	1093
	423	522	670	891	1100
	426	527	676	900	1107
265	428	530	684	911	1114

Compression Index Determination ($\sqrt{t_{90}}$ Method)

Test No	1	Date:	2-Jun-08	Sent by:	Md Wahidul Islam		
Location :	Mirpur Cantt	B.H. No.	5		Sample no.	UT-3	
Sample description.	Grey clay (soft)				Depth:	2.0 -3.0 m	

Equipment used:	Cell No: 6		Ring No.4		
			Height of ring=	1	in.
Specimen Conditions:			Dia. Of ring =	2.5	in.
Specific gravity, Gs =	2.7		Area of sample=	4.9087	in^2
Vol. Of solids=	2.1370	in^3	Wt. Of ring=	178.98	gm
Ht. Of Solid (2Ho)=	0.4353	in	Wt. Of ring+soil (B.T)=	314.53	gm
Ht. Of Void =	0.5647	in	Wt. Of ring + soil (A.T)=	308.97	gm
Initial void ratio=	1.3000		Wt. Of soil (B.T)=	135.55	gm
	W.C. sample	43.3633	Water Content (B.T)=	43.4	%
Tare weight=	8.26	lbs.	Wt. Of dry specimen=	94.55	gm
			Water Content (A.T)=	37.5	%
Compression Index (C_c)=		**0.12**	Dry Unit wt.	11.5	

Scale load lbs.	Applied load, lbs	Pressure, kPa	Final Dial read, in.	Dial change in.	Sample Ht. (2H), in.	Void Ht. (2H-2Ho)	Void ratio
8.26	0	0	0.0265	0	1	0.5647	1.297
15.38	7.12	10	0.0428	0.0163	0.9837	0.5484	1.260
22.5	14.24	20	0.053	0.0102	0.9735	0.5382	1.236
36.74	28.48	40	0.0684	0.0154	0.9581	0.5228	1.201
79.46	71.2	100	0.0911	0.0227	0.9354	0.5001	1.149
150.66	142.4	200	0.1114	0.0203	0.9151	0.4798	1.102
79.46	71.2	100	0.109	-0.0024	0.9175	0.4822	1.108
43.84	35.58	50	0.1068	-0.0022	0.9197	0.4844	1.113
11.82	3.56	5	0.0983	-0.0085	0.9282	0.4929	1.132

For SP GR.	
W1=	372.1
W2=	340.8
Wbottle=	92.8
Ws	49.7
GwT=	1

For w/c:BT	
Wt. Cont.	7.37
Cont.+wet	26.68
Cont.+dry	20.85

For w/c & Density:sample	
Wt. Cont.(gm)	35.51
Cont.+wet wt.	165.5
Cont.+dry	130.06

Final Dial Reading
265
428
530
684
911
1114
1090
1068
983

Coefficient of Consolidation Determination ($\sqrt{t_{90}}$ Method)

Coefficient of Consolidation						
Pressure, kPa	Av. press. kPa	2H inch.	$H=(H_1+H_2)/2$ mm	t_{90} min.	$C_v=(.44H^2)/t_{90}$ m^2/year	sqrt(t_{90}) min.
0.0		1				
10.0	5	0.9837	12.596	17.64	3.9578	4.2
20	15	0.9735	12.428	29.16	2.3307	5.4
40	30	0.9581	12.266	27.04	2.4481	5.2
100	70	0.9354	12.024	31.36	2.0284	5.6
200	150	0.9151	11.751	28.09	2.1629	. 5.3
						7.84

$H=(H_1+H_2)/2$ mm	t_{50} min.	$C_v=(.104H^2)/t_{50}$ m^2/year
12.596	12	1.3752
12.428	19	0.8455
12.266	17	0.9204
12.024	11	1.3668
11.751	9	1.5956

Degree of Consoli-dation U_r %						Time Factor Tr and Variation of Ur					
	n r_e/r_w 5	10	15	20	25	30	40	50	60	80	100
5	0.006	0.01	0.013	0.014	0.016	0.017	0.019	0.02	0.021	0.032	0.025
10	0.012	0.021	0.026	0.03	0.032	0.035	0.039	0.042	0.044	0.048	0.051
15	0.019	0.032	0.04	0.046	0.05	0.054	0.06	0.064	0.068	0.074	0.079
20	0.026	0.044	0.055	0.063	0.069	0.074	0.082	0.088	0.092	0.101	0.107
25	0.034	0.057	0.071	0.081	0.089	0.096	0.106	0.114	0.12	0.131	0.139
30	0.042	0.07	0.088	0.101	0.11	0.118	0.131	0.141	0.149	0.162	0.172
35	0.05	0.085	0.106	0.121	0.133	0.143	0.158	0.17	0.18	0.196	0.208
40	0.06	0.101	0.125	0.144	0.158	0.17	0.188	0.202	0.214	0.232	0.246
45	0.07	0.118	0.147	0.169	0.185	0.198	0.22	0.236	0.25	0.291	0.288
50	0.081	0.137	0.17	0.195	0.214	0.23	0.255	0.274	0.29	0.315	0.334
55	0.094	0.157	0.197	0.225	0.247	0.265	0.294	0.316	0.334	0.363	0.385
60	0.107	0.18	0.226	0.258	0.283	0.304	0.337	0.362	0.383	0.416	0.441
65	0.123	0.207	0.259	0.296	0.325	0.348	0.386	0.415	0.439	0.477	0.506
70	0.137	0.231	0.289	0.33	0.362	0.389	0.431	0.463	0.49	0.532	0.564
75	0.162	0.273	0.342	0.391	0.429	0.46	0.51	0.548	0.579	0.629	0.668
80	0.188	0.317	0.397	0.453	0.498	0.534	0.592	0.636	0.673	0.73	0.775
85	0.222	0.373	0.467	0.534	0.587	0.629	0.697	0.75	0.793	0.861	0.914
90	0.27	0.455	0.567	0.649	0.712	0.764	0.847	0.911	0.963	1.046	1.11
95	0.351	0.59	0.738	0.844	0.926	0.994	1.102	1.185	1.253	1.36	1.444
99	0.539	0.907	1.135	1.298	1.423	1.528	1.693	1.821	1.925	2.091	2.219

Thickness of DW Twill

Specimen No	Initial Reading (0.01mm)	Final Reading (0.01mm)	Thickness mm	Average Thickness mm
1	318	553	2.35	
2	317	564	2.47	
3	320	557	2.37	
4	321	559	2.38	
5	322	554	2.32	2.38
6	321	560	2.39	
7	328	568	2.4	
8	330	571	2.41	
9	327	563	2.36	
10	340	578	2.38	

Apparent Opening Size(AOS) of DW Twill

Weight of Sand Fraction Taken: 50 gm

No of Specimens: 4

Duration of Shaking: 5 mins.

Size of DW Twill Piece: 280 mm X 280 mm

Sieve No	Sieve Opening, mm	Weight Soil Passing, gm Specimen No 1	2	3	4	Average	% Finer	Sieve Opening, mm
# 16-# 30	1.19-0.6	0.28	0.92	0.75	0.69	0.66	1.32	1.19
# 30-# 50	0.6-0.3	5.11	5.07	5.43	4.97	4.91	9.82	0.6
# 50-# 100	0.3-0.15	48.96	47.93	47.85	48.14	48.22	96.44	0.3
# 100-# 200	0.15-.075	49.23	49.82	49.65	49.55	49.56	99.13	0.15

Permittivity(Cross -Plane Permeability) of DW Twill

Area. A=8490.56 sq

Diameter: 120 mm

Thickness: 69 mm

No of Pieces: 20

$\boxed{Quantity\ of\ Flow = 1000cc = 1X10^6\ mm^3}$

Head Diff. mm	Time. T(Sec) T1	T2	T3	T4	Tavg	Flow Rate. q mm^3/s	$\psi = \frac{q}{(\Delta h)A}$
30	23.56	23.44	23.47	23.27	23.44	42671.2	0.13
51	16.15	16.16	16.22	16.13	16.17	61862	0.11
61	12.49	12.65	12.58	12.52	12.56	79617.8	0.12
92	8.23	8.35	8.38	8.22	8.30	120555	0.12

Transmissivity(In-Plane Permeability of Single DW Twill)

Length: 165mm

Width: 100mm

Temperature: 28°C

Thickness at 2 kPa : 7.0 mm

In-Plane Area A = 700 sq mm

Quantity of Flow = 1.0E-04 cu m

Vertical Area = 165*100 = 16500 sq mm

Vertical Stress = 20 kPa

Head Diff. Δh mm	Hydrau. Grad. Δh/L	Collection Time T1 sec	T2 sec	T3 sec	T4 sec	Tavg sec	Flow Rate. $q = \frac{Total\ Flow}{Time}\ m^3/s$	Trans. $\theta = \frac{q}{iW}$ $m^3/s-m$
175	1.0606	525	526	527	524	525.50	1.903E-07	0.00000179
145	0.8788	625	623	626	627	625.25	1.599E-07	0.00000182
130	0.7879	755	754	753	756	754.50	1.325E-07	0.00000168

Transmissivity(In-Plane Permeability of Jute PVD as a System)

Length: 165 mm — In-Plane Area A = 700 sq mm

Width: 100 mm — Collection Time = 15 min

Temperature: 20.5°C — Vertical Area = 165*100 = 16500 sq mm

Thickness at 2 kPa : 7.0 mm — Head = 168 mm

Vertical Load	Normal Stress	Hydrau. Grad.	Water Collected(cc)				Flow Rate, q	Transmiss. θ = q/iW
			1	2	3	Average		
Kg	kPa						m^3/s	$m^3/s\text{-}m$
3.818	2.2700	1.0182	24245	24270	24255	24256.7	0.0000270	0.000265
23.818	14.1609	1.0182	14812	14825	14830	14822.3	0.0000165	0.000162
33.818	20.1063	1.0182	12175	12183	12165	12174.3	0.0000135	0.000133
43.818	26.0518	1.0182	10985	10995	10983	10987.7	0.0000122	0.00012

Settlement Readings without Vertical Drains

Time, min	Initial reading Divisions I_R	Settlement reading Divisions S_R	Settlement mm $(I_R - S_R)*0.01$
5	3000	2963	0.37
10	3000	2933	0.67
20	3000	2899	1.01
40	3000	2885	1.15
60	3000	2856	1.44
120	3000	2811	1.89
240	3000	2745	2.55
480	3000	2636	3.64
1440	3000	2330	6.7
2880	3000	2150	8.5
4320	3000	2002	9.98
7200	3000	1710	12.9
8640	3000	1550	14.5
10080	3000	1453	15.47
11520	3000	1340	16.6
12960	3000	1250	17.5
14400	3000	1153	18.47
17280	3000	1002	19.98
20160	3000	884	21.16
23040	3000	775	22.25
27360	3000	685	23.15
30240	3000	606	23.94
33120	3000	550	24.5
34560	3000	510	24.9
37440	3000	468	25.32
40320	3000	418	25.82

Water Content Determination for Test without Vertical Drain

Can no	wt of can	can+moist soil	can+ dry soil	wt of water	wt of dry soil	water content %	Avg water content %
701	10.63	22.28	18.85	3.53	8.22	42.59	**43.38%** (Before start of the test)
17	7.37	26.68	20.85	5.83	13.48	43.24	
800	7.69	20.76	16.80	3.96	9.11	43.7	
827	7.69	27.24	21.65	5.59	13.96	40.04	**40.58%** after the test
07	7.06	30.9	23.95	6.95	16.89	41.14	
718	7.22	35.0	26.98	8.02	19.76	40.58	

Settlement Readings (Test with Synthetic PVD)

Time, min	Initial reading Divisions I_R	Settlement reading Divisions S_R	Settlement mm $(I_R- S_R)*0.01$
0	3000	3000	0
5	3000	2962	0.38
15	3000	2890	1.1
35	3000	2800	2
60	3000	2702	2.98
120	3000	2618	3.82
240	3000	2549	4.51
480	3000	2416	5.84
1440	3000	2171	8.29
4320	3000	1768	12.32
5760	3000	1623	13.77
7200	3000	1482	15.18
8640	3000	1369	16.31
10080	3000	1262	17.38
11520	3000	1123	18.77
12960	3000	1005	19.95
14400	3000	937	20.63
17280	3000	742	22.58
20160	3000	612	23.88
21600	3000	561	24.39
25920	3000	369	26.31
30240	3000	269	27.31
31680	3000	205	27.95
34560	3000	169	28.31
37440	3000	112	28.88
40320	3000	72	29.28

Water Content for Test with Synthetic PVD

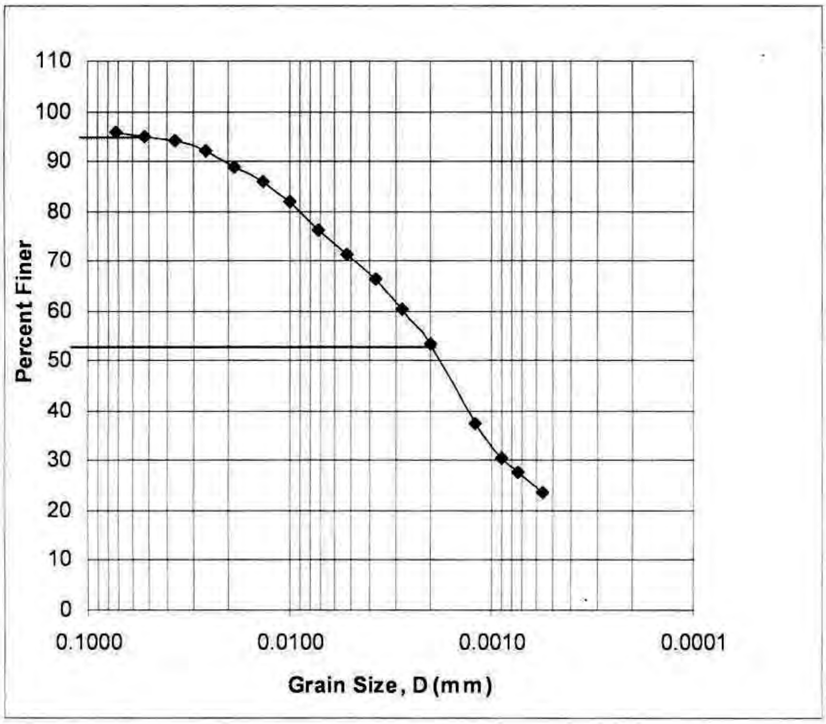
Can no	wt of can	can+moist soil	can+dry soil	wt of water	wt of dry soil	water content %	Avg water content %
078	11.18	25.82	21.38	4.44	10.20	43.53	43.32% (Before start of the test)
726	7.15	25.46	19.92	5.54	12.77	43.38	
145	7.08	26.49	20.65	5.84	13.57	43.04	
219	7.93	26.02	20.84	5.18	12.91	40.12	After the test 39.97%
856	11.48	28.78	23.86	4.92	12.38	39.74	
44	7.40	24.71	19.76	4.95	12.36	40.05	

Dial Reading and Settlement Calculation for Jute PVD

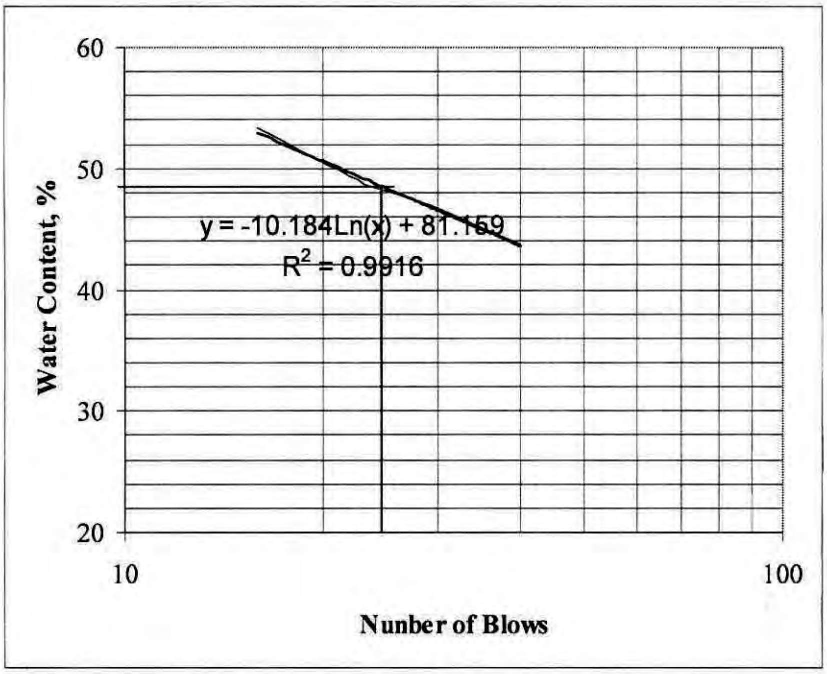
Time, min	Initial reading Divisions I_R	Settlement reading Divisions S_R	Settlement mm $(I_R - S_R)*0.01$
0	4000	4000	0
5	4000	3937	0.63
15	4000	3888	1.12
35	4000	3798	2.02
60	4000	3697	3.03
120	4000	3612	3.88
240	4000	3542	4.58
480	4000	3392	6.08
1440	4000	3129	8.71
2880	4000	2922	10.78
4320	4000	2708	12.92
5760	4000	2545	14.55
7200	4000	2402	15.98
10080	4000	2162	18.38
11520	4000	2031	19.69
12960	4000	1898	21.02
14400	4000	1830	21.7
17280	4000	1628	23.72
20160	4000	1496	25.04
23040	4000	1373	26.27
25920	4000	1248	27.52
30240	4000	1151	28.49
33120	4000	1090	29.1
34560	4000	1047	29.53
37440	4000	996	30.04
40320	4000	938	30.62

Water Content for Test with Jute PVD

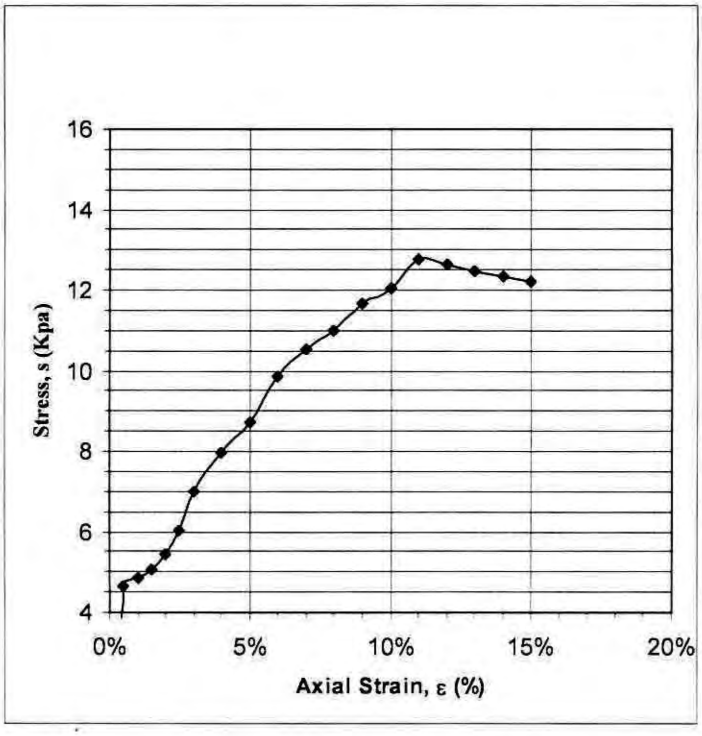
Can no	wt of can	can+ moist soil	can+ dry soil	wt of water	wt of dry soil	water content %	Avg water content %
155	6.98	25.0	19.62	5.38	12.64	42.56	**43.29%** (Before start of the test)
819	10.69	28.45	23.06	5.39	12.37	43.57	
50	6.91	37.40	28.18	9.28	21.21	43.75	
791	6.90	27.53	21.60	5.93	14.7	40.34	**39.82%** after the test
888	6.90	30.59	23.90	6.69	17.0	39.35	
38	7.0	26.15	20.70	5.45	13.7	39.78	



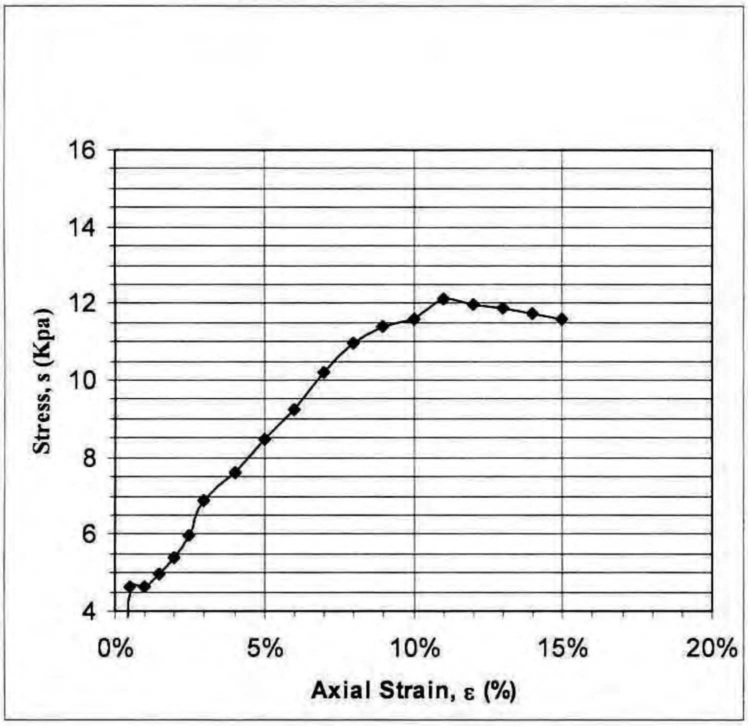
Grain Size Determination by Hydrometer Analysis



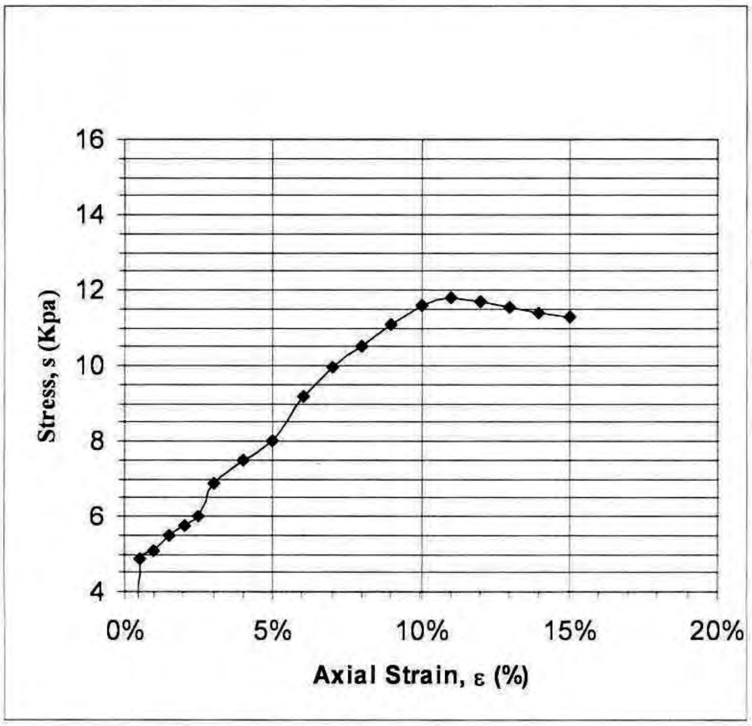
Determination of Liquid Limit.



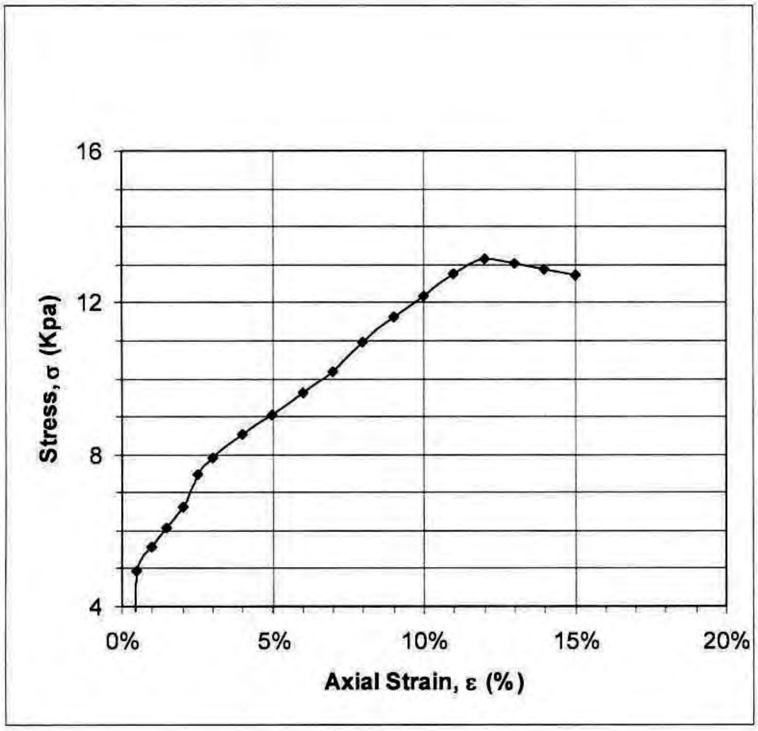
Unconfined Compression Test of Specimen -1 (Test with Synthetic PVD)



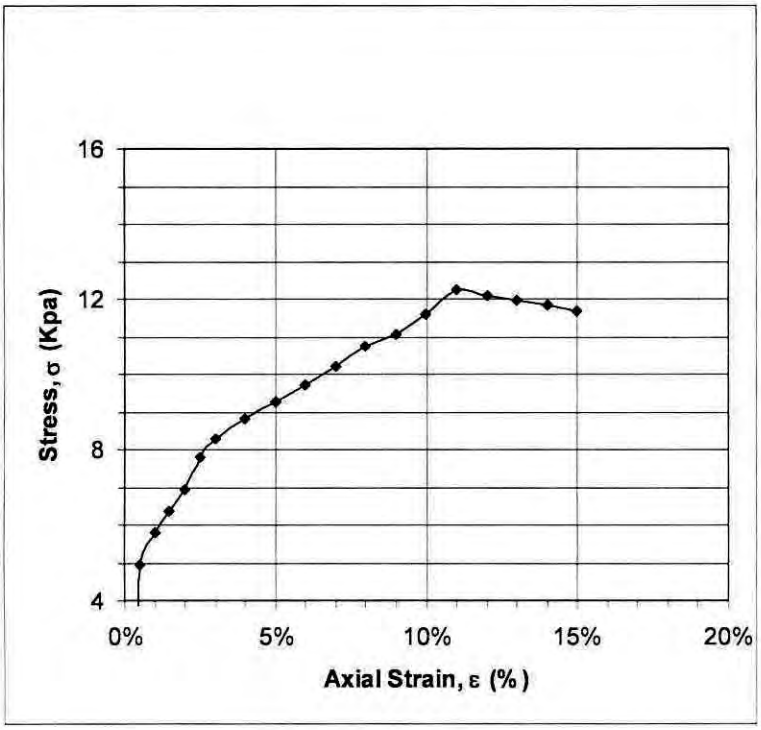
Unconfined Compression Test of Specimen -2 (Test with Synthetic PVD)



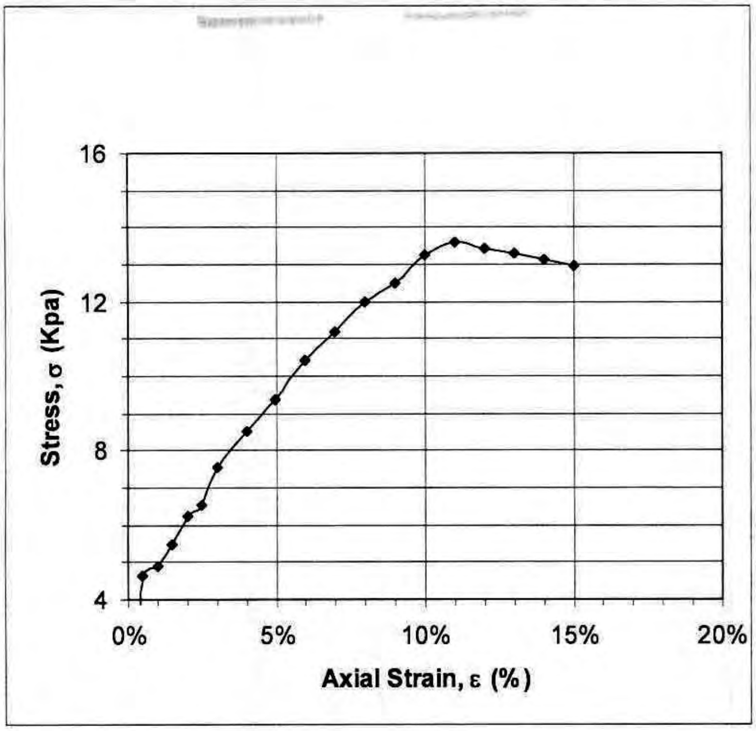
Unconfined Compression Test of Specimen -3 (Test with Synthetic PVD)



Unconfined Compression Test of Specimen -4 (Test with Jute PVD)



Unconfined Compression Test of Specimen -5 (Test with Jute PVD)



Unconfined Compression Test of Specimen -6 (Test with Jute PVD)

Appendix C

Sample Computation of Soil Settlement

(Without Vertical Drains, with Synthetic PVD and with Jute PVD)

Computation of Theoretical (Terzaghi's One-Dimensional) Settlement from Consolidation Parameters

From the oedometer test. following properties were obtained:

Initial void ratio. $e_o = 1.30$

Compression index. $c_c = 0.12$

Degree of saturation. $S_r = \frac{wG_s}{e_0} = \frac{43.38 * 2.70}{1.30} = 90.09\%$

Coefficient of consolidation, $c_v = 4.43\ mm^2/min$

Surcharge load, $\Delta p = 27.406$ kPa

Effective overburden pressure at mid depth:

$$p_0' = \frac{\gamma * H}{2} = (16.51-9.81)*0.825/2 = 2.76 \text{ kPa}$$

So, the Theoretical total settlement: $S_c = \frac{C_c H_c}{1+e_0} \log_{10}\left[\frac{p_0' - \Delta p}{p_0'}\right]$

$$S_c = \frac{0.12 * 825}{1-1.30} \log_{10}\left[\frac{2.76 - 27.406}{2.76}\right]$$

$$= 44.68 \text{ mm.}$$

Time factor for 28 days: $T_v = \frac{C_v t}{H^2}$

$$= 4.43*28*24*60/(825)^2$$

$$= 0.2624$$

So, the **theoretical average degree of consolidation** corresponding to 28 days i.e. U will be 25.73 mm or 57.6%.

Relation between Pore Water Reduction and Reduction in Total Slurry Height (Test without Vertical Drains).

Soil + water = 275.7 kg

Initial Water content = 43.38%

Wt. of Soil Solid (W_s) + 0.4338*W_s = 275.7 kg

W_s = 192.286 kg

Final water content = 40.48%

W_s + 0.4048* W_s = Total Wt. of soil slurry

Total Wt. of soil slurry = 270.123 kg

So. Wt. of water expelled = 275.700-270.123 = 5.5766 kg

= 5576.630 cm^3

Area of the steel tank (Diameter 50.2 cm) = 1979.23 cm^2

So, Reduction in Height of water = 5576.630/1979.23

= 2.817 cm = 28.17 mm

But actual settlement was 25.82 mm.

Combined Vertical and Radial Consolidation Settlement Computations (Test with Synthetic PVD):

Actual settlement in the steel tank: = 29.28 mm.

Actual average degree of consolidation occurred with in-soil synthetic PVD (28 days): = 29.28 44.68 = 65.53%

There are two components in the settlement, namely: vertical (one way) and radial drainage.

Vertical Drainage (28 days): Consolidation for one-way drainage only

$U_v = 57.6\%$

Radial Drainage (28 days):

$C_{vr} = C_v = 4.43$ mm^2/min

Time factor in the radial drainage, $T_r = \dfrac{C_{vr}t}{d_e^2}$

(Here d_e= Diameter of the steel tank=502 mm).

$= 4.43*28*24*60/ (502)^2$

So, $T_r = 0.7088$

The average degree of consolidation due to radial drainage:

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right]$$

Now, $F(n) = \dfrac{n^2}{n^2-1}\ln(n) - \dfrac{3n^2-1}{4n^2}$; here, $n = r_e/r_w$

$r_w = 2(b-t)/2*\pi$

$= (100-3)/\pi$

$= 32.8$ mm

So, $n = 251/32.8 = 7.66$

So, $F(n) = (7.66)^2 \ln(7.66)/(7.66^2-1)-(3*7.66^2-1)/4*7.66^2$

$= 2.071-0.7457$

$= 1.325$

$U_r = 1-\exp(8T_r/F(n))$

$= 1- e^{-8*0.7088/1.325}$

$= 0.9861$

$U = 1-(1-U_v)(1-U_r)$

$= 1-(1-0.576)(1-0.9861)$

$= 0.9941$ or 99.41%

So, in 28 days with synthetic PVD, theoretical average degree of consolidation for simultaneous vertical and radial drainage is 99.41% whereas actual average degree of consolidation occurred 65.53%.

In the first phase, without any use of vertical drain, total settlement occurred in 28 days was 25.82 mm. In the second phase with in-soil synthetic PVD, it took 16.8 days to occur 25.82 mm settlement as obtained. Now let us calculate all parameters for this settlement of 16.8 days.

Actual settlement in the steel tank: = 25.82 mm.

Actual average degree of consolidation occurred = 25.82/44.68

with in-soil synthetic PVD (16.8 days): = 57.8%

There are two components in the settlement, vertical (one way) and radial drainage.

Vertical Drainage (16.8 days):

Time factor, $T_v = \dfrac{C_v.t}{H^2}$

$= 4.43*16.8*24*60/(825)^2$

$= 0.16$

So, the theoretical average degree of consolidation corresponding to this time factor (T_v) i.e. $U_v = 45.54\%$.

Radial Drainage (16.8 days):

It is assumed that coefficient of consolidation in the radial direction (C_{vr}) is same as coefficient of consolidation in the vertical direction (C_v)

$C_{vr} = C_v = 4.43$ mm^2/min

Time factor in the radial drainage. $T_r = \dfrac{C_{vr}t}{d_e^2}$

(Here d_e= Diameter of the steel tank=502 mm).

$= 4.43*16.8*24*60/(502)^2$

So, $T_r = 0.4253$

According to the solution to the problem given by Barron (1948) in respect of equal strain case with no smear ($r_w = r_s$), the average degree of consolidation due to radial drainage:

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right]$$

Now, $F(n) = \dfrac{n^2}{n^2-1}\ln(n) - \dfrac{3n^2-1}{4n^2}$; here. $n = r_e/r_w$

$r_w = 2(b-t)/2*\pi$

$= (100-3)/\pi$

$= 32.8$ mm

So. $n = 251/32.8 = 7.66$

So, $F_n = (7.66)^2\ln(7.66)/(7.66^2-1)-(3*7.66^2-1)/4*7.66^2$

$= 2.071-0.7457$

$= 1.325$

$U_r = 1-\exp(8T_r/F(n))$

$= 1 - e^{-8*0.4253/1.325}$

$= 0.9233$

$U = 1-(1-U_v)(1-U_r)$

$= 1-(1-0.4554)(1-0.9233)$

$= 0.9582$ or 95.82%

So, in 16.8 days with synthetic PVD, the theoretical average degree of consolidation for simultaneous vertical and radial drainage was 95.82% whereas actual average degree of consolidation occurred 57.8%. Therefore, test results of actual settlement in the steel tank differ with the theoretical settlement as calculated with the parameters found from oedometer test and physical properties of steel tank and synthetic PVD.

Relation between Pore Water Reduction and Reduction in Total Slurry Height (Test with Synthetic PVD)

Soil + Water = 274.5 kg

Initial water content = 43.32%

Wt. of soil solid (Ws) + 0.4332*Ws = 274.5 kg

So, Ws = 191.53 kg

Final water content (after 28 days) = 39.97%

Ws + 0.3997*Ws = Total wt. of soil slurry

So, total wt. of soil slurry = 268.084 kg

Wt. of expelled water = 274.5 – 268.084 = 6.4154 kg = 6415.4 cm^3

Cross-sectional area of the steel tank (Diameter = 50.2 cm) = 1979.23 cm^2

So, Reduction in height of water = 6415.4 cm3/1979.23 cm^2

= 3.2413 cm = 32.41 mm

But actual settlement was 29.98 mm

Combined Vertical and Radial Consolidation Settlement Computations (Test with Jute PVD):

Total observed settlement = 30.62 mm

Actual average degree of consolidation = 68.53%

$C_{vr} = C_v = 4.43\ mm^2/min$

Time factor in the radial drainage. $T_r = \dfrac{C_{vr}t}{d_e^2}$

(Here d_e= Diameter of the steel tank=502 mm).

$= 4.43*28*24*60/(502)^2$

So. $T_r = 0.7088$

The average degree of consolidation due to radial drainage:

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right]$$

Now, $F(n) = \dfrac{n^2}{n^2-1}\ln(n) - \dfrac{3n^2-1}{4n^2}$; here. $n = r_e/r_w$

$r_w = 2(b-t)/2*\pi$

$= (90-7)/\pi$

$= 30.8$ mm

So. $n = 251/30.8 = 8.15$

So, $F(n)= (8.15)^2\ln(8.15)/(8.15^2-1)-(3*8.15^2-1)/4*8.15^2$

$= 2.13-0.7456$

$= 1.384$

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right]$$

$= 1 - e^{-8*0.7088/1.384}$

$= 0.9833$ or 98.33%

$U = 1-(1-U_v)(1-U_r)$

$= 1-(1-0.576)(1-0.9833)$

$= 0.9929$ or 99.29%

So, the **theoretical** average degree of consolidation for simultaneous vertical and radial drainage is 99.29% whereas **actual** average degree of consolidation occurred 68.53%. In the first phase, without any use of vertical drain, total settlement occurred in 28 days was 25.82 mm. In the third phase with jute PVD used in-soil: 25.82 mm settlement took place in 14.75 days.

Vertical Drainage (14.75 days):

Time factor, $T_v = \dfrac{C_v t}{H^2}$

$= 4.43*14.75*24*60/(825)^2$

$= 0.138$

So, the **theoretical average degree of consolidation** corresponding to this time factor (T_v) i.e. $U_v = 42.36\%$.

Radial Drainage (14.75 days):

It is assumed that coefficient of consolidation in the radial direction (C_{vr}) is same as coefficient of consolidation in the vertical direction (C_v)

$C_{vr} = C_v = 4.43$ mm^2/min

Time factor in the radial drainage, $T_r = \dfrac{C_{vr} t}{d_e^2}$

(Here d_e = Diameter of the steel tank = 502 mm).

$= 4.43*14.75*24*60/(502)^2$

$$\text{So,} \quad T_r = 0.3734$$

According to the solution to the problem given by Barron (1948) in respect of equal strain case with no smear ($r_w = r_s$), the average degree of consolidation due to radial drainage:

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right]$$

$$\text{Now,} \quad F(n) = \frac{n^2}{n^2 - 1}\ln(n) - \frac{3n^2 - 1}{4n^2} \quad ; \quad \text{here, } n = r_e/r_w$$

$r_w = 2(b+t)/2*\pi$

$= (90+7)/\pi$

$= 30.8$ mm

So, $n = 25.1/3.08 = 8.15$

So, $F_n = (8.15)^2\ln(8.15)/(8.15^2-1)-(3*8.15^2-1)/4*8.15^2$

$= 2.13-0.7456$

$= 1.384$

$U_r = 1-\exp(8T_r/F(n))$

$= 1- e^{-8*0.3734/1.384}$

$= 0.8845$

$U = 1-(1-U_v)(1-U_r)$

$= 1-(1-0.4236)(1-0.8845)$

$= 0.9334$ or 93.34%

So, the theoretical average degree of consolidation for simultaneous vertical and radial drainage is 93.34% whereas actual average degree of consolidation occurred 57.8%.

Relation between Pore Water Reduction and Reduction in Total Slurry Height (Test with Jute PVD)

Soil + water = 274.9 kg

Initial Water content = 43.29%

Wt. of Soil Solid (W_s) + 0.4329* W_s = 274.9 kg

W_s = 191.849 kg

Final water content = 39.82%

After 28 days:

W_s + 0.3937* W_s = Total Wt. of soil slurry

Total Wt. of soil slurry = 268.24 kg

So, Wt. of water expelled = 274.90-268.24 = 6.66 kg

= 6660 cm^3

Area of the steel tank (Diameter 50.2 cm) = 1979.23 cm^2

So, Reduction in Height of water = 6660/1979.23

= 3.3649 cm = 33.65 mm

But settlement was 30.62 mm.