CONFERENCE ON GEOTECHNICAL ENGINEERING UNDER DIFFICULT GROUND CONDITIONS

Organised by the

SRI LANKAN GEOTECHNICAL SOCIETY

August 15, 2006

At ICTAD Auditorium

SLGS Conference on Geotechnical Engineering under Difficult Ground Condition
Tuesday 15th August 2006 at ICTAD Auditorium

Program

8.45 a.m – 9.00 a.m	Registration
9.00 a.m. – 9.10 a.m.	Welcome Address Mr. D. P. Mallawarachchi President – SLGS
9.10 a.m. – 9.50 a.m.	Construction Problematic Soils Mr. Kirthi Senanayaka Consultant – NBRO
9.50 a.m. – 10.30 a.m.	Effects of Intermediate Permeable Layers on the Consolidation of Soft Clays with Prefabricated Vertical Drains Dr. Udeni Nawagamuwa Senior Lecturer , University of Ruhuna
10.30 a.m. – 10.50 a.m.	Tea
10.50 a.m. – 11.30 a.m.	Improvement of Soft Peaty Clay by Electro Osmosis with Electro Kinetic Geosynthetics Dr. Athula Kulathilaka Senior Lecturer, University of Mortuwa
11.30 a.m. – 12.10 p.m.	Construction with Geosynthetics in Difficult Ground Conditions Dr. G. P. Karunarathna, Formerly at National University of Singapore
12.10 p.m. – 12.30 p.m.	Discussion
12.30 p.m. – 1.30 p.m.	Lunch
1.30 p.m. – 2.10 p.m.	Stabilization of Waste Phosphatic Clay Settlement Areas Using Phoscrete Prof. M. Gunarathna Professor of Civil Engineering University of South Florida - USA
2.10 p.m. – 2.50 p.m.	Development of Negative Skin Friction on Piles Installed Through Compressible Soil Dr. H. S. Thilakasiri Senior Lecturer, University of Moratuwa
2.50 p.m. – 3.10 p.m.	Discussion
3.10 p.m. – 3.30 p.m.	Tea
3.30 p.m. – 5.00 p.m.	General Meeting of SLGS
5.00 p.m. – 6.00 p.m.	Project Day Competition

Sri Lankan Geotechnical Society Conference on Construction on Difficult Ground Conditions

CONTENTS

CONSTRUCTION ON PROBLEMATIC SOILS

Eng. Kirthi Sri Senanayake
Consultant, Geotechnical Engineering Division, National Building Research Organisation,
99/1, Jawatta Road, Colombo 5, Sri Lanka

ABSTRACT

With growing population and rapid development, land hitherto considered as good quality ground for construction is becoming scarce. With the scale of modern infrastructure becoming larger, builders and engineers are faced with the challenge of effectively utilizing the available poor quality land, yet ensuring the safety, stability and economy of the structure.

Difficult ground conditions are generally implied when general weakness or high degree of variability or complexity is encountered in the subsoil that are affected by the structural loads/foundation pressure and/or by the construction operations and could not be easily responded with conventional techniques.

In Sri Lanka, problematic soils with unique engineering characteristics do exist among the highly compressible peat and other organic soil deposits found in the urban and coastal low lying areas, complex lateritic formations found in most parts of the wet zone, expansive soils occasionally observed in some dry zone areas, the residual soils found in the hill country and the unplanned man made earth/waste fills now found almost everywhere, which often present difficult ground conditions that require specific geotechnical engineering solutions.

This paper gives an introduction to some problematic soils, their influence on construction, some ground/structural improvement techniques available and some geotechnical factors to be addressed from planning to construction & maintenance.

1.0 INTRODUCTION

Difficult ground conditions are generally implied when a general weakness and/or a high degree of variability in geotechnical properties is encountered in the ground affected by foundation pressures and/or by construction operations that would pose problems of inadequate strength and/ or excessive deformation in the ground and therefore calls for special attention to ensure safety and stability of the structure and construction environment.

Inferiority of a ground could be due to inherent properties of the constituent soil type/s such as composition and structure, minerals and chemicals present, porosity, bonding of particles, strength and compressibility or how the , high void ratio and occurrence of deep extents of compressible layers. High compressibility may be attributed to the presence of peat, organic clay or soft inorganic clay in the subsoil. Very loose silts and sandy deposits may also lead to high compressibility in the ground.

It is human tendency to take the path of least resistance in reaching a goal. Obviously, structures must be constructed on good quality ground and therefore man had always tried to avoid difficult ground conditions whenever it was possible to select an alternative site with less or practically no problems. But when the easily buildable lands with sound ground conditions are exhausted, the option left would be to build on lands that had been hitherto considered unsuitable for construction purposes either due to high cost of development or due to inherent geotechnical problems.

The geotechnical engineer, first of all, needs to carefully assess the ground conditions of the site and when faced with poor ground, accordingly advise the client on considering following alternatives;

- Avoid the poor ground and change to a new site
- Bypass the poor ground and support on stronger ground
- Remove the poor ground and replace with competent material
- Design the structure to accommodate poor ground behavior
- Treat the ground to improve its properties
- Abandon the project altogether.

In tackling difficult ground situations, the geotechnical engineer may suggest adopting of suitable ground treatment methods or modifications to the structural layout or to design the superstructure and foundations in such a way to tolerate the anticipated ground behavior.

2.0 DIFFICULT GROUND CONDITIONS IN SRI LANKA

In Sri Lanka, with rapid development programmes initiated about three decades ago and prompted by the globalization, urbanization has intensified, thereby generating an increased demand for the use of lands which were hitherto considered unsuitable for development. Geotechnical engineer today is therefore faced than ever, with the challenge of constructing on difficult ground and finding specific solutions with the appropriate foundation systems or ground treatment.

The highly compressible peat and organic clays in the low lying lands, heterogeneous residual and colluvial soils found in the wet zone, expansive soils in some dry zone areas, porous/soluble limestone/coral deposits, man-made fills and reclaimed lands often present difficult ground conditions. Such soils with inherent inferior properties that pose problems in geotechnical engineering may be referred to as "Problematic Soils".

Are soils problematic?

"They behave in a rational manner, certainly more rational than most people. Generally speaking, they behave correctly. It is up to us to understand their behavior and adjust our requirements accordingly or modify their behavior as we do with soil improvement techniques"- S. Loreuiet, University of Laval, Quebeq, Canada.

Difficult ground situations identified in Sri Lanka may broadly be classified into;

- Residual soils and rock with different degree of decomposition and weathering. Laterite formations are often found in the wet zone of the island consist of material in varying stages/degree of decomposition and in different soil composition due to varying rates of leaching of minerals.
- Irregular alluvial or marine soil deposits that are generally associated with estuary and delta deposits and sometimes with shore deposits
- Colluvial deposits with heterogeneous composition of materials having wide variation in particle size.
- Ground with inherent weakness attributed to high compressibility, high void ratio, high moisture content and/or presence of deep compressible or loose layers of peat and clay.
- Special ground conditions attributed to unique soil properties; collapsible soils, dispersive soils and expansive soils for example, found in some parts of the dry zone of Sri Lanka.
- Ground with unique geological structure, faults, cavities, old mines, closed gem pits etc.,

- Reclaimed lands, filled grounds and landfills where filling has been carried out in a controlled/uncontrolled manner using inorganic/organic/waste materials or dredged soils.
- Difficult ground conditions which are attributed to environmental factors such as high or fluctuating water table.
- Loose fine sand deposits below water table that may cause serious problems of liquefaction under seismic conditions which is attracting some speculation in Sri Lanka after the Tsunami event!

3.0 PEATS AND COMPRESSIBLE SOILS

3.1 Occurrence of Peaty Soils

Along the coastal belt and particularly in and around Colombo City earmarked for major infrastructure development activities, a significant feature is the existence of low lying areas with highly compressible substrata mainly consisting of peat, organic clay and organic silt. Peaty soils are also found in many other parts of the island in drowned valleys.

Peat is generally formed by the death and gradual decay of plants in the swampy areas and with the accumulation of such deposits over the years. They could be found in nature at different stages of decomposition from permeable coarse fibrous material to fine amorphous material that resembles soft organic clay. MaFarlane (1969) has identified 17 different types of peat according to their structural characteristics, and broadly classified them into the three groups of (a) Coarse fibrous peat, (b) Fine fibrous peat and (c) Amorphous granular peat. In a similar classification, the Geological Survey Department of Sri Lanka has classified peat in Sri Lanka into three groups; (a) Shrub or tree group – comparable with coarse fibrous peat, (b) Reed and sedge group – comparable with fine fibrous peat and (c) Humus Peat – comparable with amorphous granular peat. Ray et al (1982) and Senanayake (1986) in their geotechnical mapping efforts have developed soil series for the low lying areas in and around Colombo according to the occurrence of the different peat deposits (NBRO 1989).

In Colombo area, the combined thickness of peat and organic clay deposits is found to vary from a few meters to nearly 15 meters (Senanayake, 1986). Peat may appear to exist as thick continuous layers with or without embedded inorganic particles. But peat is sometimes encountered in thin layers of different groups or sandwiched between clay, silt or sand layers as when carefully observed in undisturbed samplings, quite contrary to accepting as a homogeneous thick layer.

3.2 Construction on Peaty soils

Peaty soils pose a challenge to the geotechnical engineer due to their generally low strength, high compressibility and partly unpredictable deformation characteristics.

Settlement occurs in peat deposits due to immediate compression of peat mass with the expulsion of air and gases, gradual expulsion of water from the macro voids in the peat mass and from micro pores in fibrous material, rearrangement of fibrous Skelton, crushing of the fibrous material, lateral spreading, creep and/or in an extreme case by rotational slip and upheaval of the ground. Mechanism of settlement and engineering behavior of peats in Sri Lanka need research taking into consideration of the type and nature of peat encountered.

Filling is done as a common practice in developing low lying land, either to provide a better construction platform over the weak surface layers or to raise the ground above water level. Loads due to the filling and the structures later supported on the fill can cause significant settlement in the underlying compressible layers, often excessive and intolerable in terms of both ultimate and differential settlements.

A sufficiently thick and well compacted fill could serve as a rigid mat in distributing the foundation loads. It can be further strengthened with geotextiles when required. However, the settlement due to fill load sometimes could be significantly larger than the settlement attributed due to the load of a light structure that is to be supported over the fill.

3.3 Geotechnical Solutions

There are many different approaches and techniques available to tackle the problematic nature of peaty soils, depending on the scale of the problem and the associated cost, time, and environmental factors. These may be ground improvement techniques, foundation techniques or structural improvement or their combinations. Some methods have been tried/successfully applied in projects in Sri Lanka.

3.3.1 Soil Replacement

Replacement of weak soil with controlled fill: When the peat deposit is within practically replaceable reach, foundations can be placed over a sufficiently thick, well compacted fill that replaces the weak layer. Light structures may be supported on conventional or suitably stiffened/reinforced strip foundations over adequately thick layer of compacted fill when the thickness of peat layer is not very large.

Replacement with light foundation: As an alternative, the weak layer could be replaced with a lighter hollow foundation to reduce effective pressure on the underlying strata. This could be often achieved by providing a basement where feasible and required, and especially in the urban areas where the land price is high as the additional construction and maintenance cost could be adequately compensated with benefit from the additional space created.

Use of light weight materials: Instead of the earth fill, weak soil can be replaced with lighter inorganic stable materials such as expanded polystyrol/polysterene (EPS), expanded waste glass, slag, cinders. EPS is commonly used to lighten foundation load in buildings and embankment loads in highway/railway projects abroad and the Sri Lankan Geotechnical Society with the collaboration of the industry intends to carry out a field experimental research project on its applicability to local conditions.

A huge volume of light weight non degradable materials are disposed as municipal waste posing a deadly threat to living environment. Though laboratory level studies have been undertaken, it is imperative that further studies must be initiated on the recovery and effective utilization of such resources as filling and/or earth reinforcement material.

3.3.2 Structural Improvements and Appropriate Foundations

Improvement of the stiffness of structure: By improving the stiffness of the superstructure and its foundation it is possible to minimize structural distress by accommodating structural distortion in foundation that would otherwise develop due to differential settlement.

Stiffened foundations: Inverted T type reinforced beam foundations and Vierendeel Girder System are simple types of stiffened foundation when placed over an engineered fill which can be effectively used in low rise buildings to tolerate structural distress. To avoid cracking due to ground settlement, the floors may be suspended.

Raft foundations: Rafts with stiffened beams can be used to spread out the loads and reduce the bearing pressure, provided that anticipated differential settlement is within tolerable limits that would not affect the structural performance. Though rafts are widely used, it is not necessarily a solution to overcome settlement problems.

3.3.3 By passing of the weak soils

Pile foundations: When it is not feasible to support a structure on shallow foundations with or without improvement of the subsoil to attain adequate strength and resistance to deformation, option available would be to adopt pile foundations supported on a suitable bearing stratum. For medium to heavy structures and when the peaty/weak strata extend to higher depths deep foundation is an option.

Under-reamed piles: When a suitable bearing stratum is available within easy reach, say up to about 7m, under-reamed piles prove to be an economical option to bypass the weak layers. NBRO (1989)2 has developed under-reaming tools and the techniques for manual installation of under-reamed piles that shows high potential for use in Sri Lanka particularly for the low to medium rise buildings in the low lying areas.

Cylinder foundations: Concrete cylinders can be adopted as mini-caissons to transmit the structural load to a bearing stratum bypassing the weak layers. Hollow reinforced concrete cylinders/segments are driven to the required depth under its own weight or by loading while removing the soil inside to facilitate sinking of the cylinder. The bottom end is plugged with concrete after reaching the required depth and the cylinder is filled with concrete of design strength.

Granular columns: Granular columns (Stone/Gravel/Sand columns) are easy to install using simple equipment and have been used worldwide since early days to improve weak ground for supporting the structures. Using the energy of a drop hammer, granular material is compacted into the weak ground through a casing while withdrawing the tube to form a compacted column and to partially improve the ground surrounding the column. It would be reasonable to consider granular columns as a composite part of improved ground and not as a rigid structural pile. Its application to peaty soils, where sustained lateral support and relaxation by creep would be questionable, needs to be verified from actual field performance.

3.3.4 Ground Improvement

Excessive, detrimental and prolonged settlement can be experienced in embankments or structures placed over ground with underlying soft clays and peaty soils due to consolidation of deposits that are highly compressible and/or thick. When shallow foundations are not suitable and costly deep pile foundations should be avoided, the option would be to modify the soil characteristics prior to construction. There are many ground improvement techniques available to improve soil strength in order to increase the bearing capacity of foundations, to increase soil stiffness for reduction of total and differential deformations and settlements, to lower soil permeability where required and to achieve general improvement of geotechnical properties in heterogeneous soil deposits.

Preloading: Preloading is a method to pre-compress the soil well in advance of construction by applying a higher load to achieve a given settlement that may be a grater part or whole of the ultimate settlement that is anticipated to take place under the foundation load. It is worthy to explore feasibility of preloading for ground improvement when burrow material for surcharge is economically available and the site is large enough so that surcharge material can be progressively provided in bulk and moved across using earth machinery. Duration of preloading is crucial since on the one hand it should be sufficient to pre-induce the desired settlement and on the other hand it should not be too long to delay the construction activities unduly. Vertical drains, sand drains or any of the many other available types may be used to shorten the drainage path and accelerate the preloading process by facilitating faster dissipation of pore water pressure. In adopting preloading method, like in many other techniques, geotechnical engineers must carefully assess and forecast the anticipated performance in his design and verify the validity and propriety of assumptions and analyses used by systematically monitoring the actual performance in field so that timely precautions could be taken.

Dynamic Compaction: This technique, also known as Dynamic Consolidation or Heavy Tamping, uses the energy of a 2 ~ 30 tons heavy tamping weight repeatedly dropped in free fall of 2 ~ 30 m onto the ground that is to be compacted. The craters formed are filled with granular soil and tamping process is repeated to achieve required improvement of the entire area. Use of this method has been successfully extended to improve soft peaty soils in number of sites including a building project in Sri Lanka at Madiwela (Karunarathna et al 1993).

Lime/Cement Mixing: Lime and cement are used as reagents in in-situ soil mixing to modify the physical or chemical characteristics of the soil without excavating. In deep mixing, the soil in deep layers can be stabilized with the creation of lime/cement columns where as in surface mixing or mass stabilization, the upper part of the soft ground is mixed horizontally as well as vertically. The method is mainly used in soft clays but also in organic clays and clayey silts primarily for reduction of settlements and for improvement of stability.

In Sri Lanka, laboratory studies have being conducted by Kulathilaka et al (2001) on the applicability of lime/cement mixing on amorphous granular peat and fibrous peat, with encouraging results indicating potential application of the method for amorphous granular peat that is close to organic clay. Since the mixing technique at field level needs precision equipment, technology appropriate to local conditions needs to be developed firstly for the surface mixing method.

Electro-osmotic Consolidation: The feasibility of applying electro-osmotic consolidation to improve very soft peaty clays has been studied through laboratory experiments (Kulathilaka et al Results indicate great potential for this method provided

4.0 EXPANSIVE SOILS

4.1 Mechanism of Swelling

Expansive or swelling soils, as their name implies, are soils that swell when subjected to moisture. They typically contain clay minerals that attract and absorb water. When moisture increases in these soils, the water molecules are pulled into gaps between the clay plates forcing them apart and leading to an increase in soil pressure or an expansion of the soil volume. When the clay particles dry out, they can shrink considerably.

It is convenient to declare that any clayey soil is potentially expansive but what matters is the degree of swelling and the intensity of pressures developed thereby. Structures, especially those with lighter loadings, founded on expansive soils could undergo severe distress due to uplift forces caused by the swelling of soil unless special precautions are taken in the design, construction and also in maintenance.

Of the three most important clay minerals; montmorillonite, illite and kaolinite, montmorillonite is the clay mineral that presents most of the expansive soil problems. Expansive soils may derive from montmorillonite and other secondary minerals due to decomposition of the feldspar and pyroxene minerals in the basic igneous rock or from physical breakdown of montmorillonite contained in certain sedimentary rocks.

The clay minerals are formed by a complicated alteration process that includes disintegration, oxidation, hydration and leaching. Formation of montmorillonite minerals is aided in an environment of extreme disintegration, strong hydration and restricted leaching where magnesium, calcium, sodium and iron cations may accumulate in the system. Such conditions are favorable in semi-arid zones having low or seasonal moderate rainfall, with evaporation exceeding precipitation, where enough water is available for the alteration process but without sufficient flush rains to remove the accumulated cations.

4.2 Occurrence of Potential Expansive Soils in Sri Lanka

Expansive soils occur in many parts of the world and they do exist in Sri Lanka. In the early 1980's Soil Testing Laboratory of the Urban Development Authority, (present NBRO) had discovered presence of expansive soil in Murunkan, Mannar. Subsequently, during routine soil investigations conducted by NBRO for various development projects, expansive soils have been identified in various parts of the dry zone including Anuradhapura, Puttalam, Dambulla, Moneragala and Kataragama. Problem of expansive soils was highlighted during the construction of Dambulla Gam Udwa where a number of light buildings suffered structural damage and had to be demolished.

4.3 Identification of Expansive Soils

Recognition of potentially expansive soils and determination of swelling potential of clays is a classical subject that may need different approaches. In the traditional engineering approach the mineralogical content of the clay was highlighted as a key factor in exploring the basic properties of clays. For a given soil mineralogy, the expansion is essentially controlled by three variables: the initial void ratio, the initial suction and the applied confining stress. In another approach, soil index parameters such as activity, plasticity, clay content, dry density and natural water content others, are valuable tools in evaluating the swelling properties (Chen 1975). The third method or the direct measurement of swelling pressure offers most useful data for a practicing engineer and these simple laboratory tests can be easily performed in the laboratory using a conventional one-dimensional consolidometer that could also provide an estimation of the expansion potential. The reliability of current empirical approaches to determine swelling potential of natural soils is still being questioned and it suggests that none of the above approaches should be considered alone.

The presence of capillary stress or negative pore water pressure arising from molecular forces in swelling soils causes available moisture to be absorbed. The vertical confining pressure required to prevent volume expansion from absorbed moisture is defined as the swell pressure.

Swell is determined by subjecting the laterally confined soil specimen to a constant vertical pressure and by giving both the top and bottom of the specimen access to free water (usually distilled) to cause swell. The swell pressure is determined by subjecting the laterally confined soil specimen to increasing vertical pressures, following inundation, to prevent swell.

In the laboratory, testing for Free Swell Index (FSI), i.e. the difference between final and initial volume expressed as a percentage of initial volume, when a known volume of dry soil is allowed to swell in distilled water inside a graduated cylinder for 24 hours without a surcharge, is a very crude yet a useful guide before proceeding with classification based on index parameters such as plasticity index, which are considered to offer further confirmative data. Results of laboratory tests conducted for swelling properties of some soils in the dry zone of Sri Lanka show that there are no clear correlations between FSI.

In the field it may not be an easy task to identify expansive soils. In many cases, expansive soils are buried under the topsoil layer or dense vegetation and cannot be identified at the surface. In some cases, deep running cracks could be observed on ground surface during dry whether.

One practical method of identifying expansive soils in a suspected area is to inquire locally about seasonal behavior of the ground and the condition of existing structures which had experienced a few dry and wet cycles. Observation of cracks in walls, sidewalks, floors etc., shrinkage cracks appearing in the soil during the dry season, high dry strength of soil lumps and high stickiness of soil when wet etc., would be helpful to identify expansive soils.

Cracks on floor slabs due to heaving and diagonal cracks that develop at wall corners, below windows and above doors, cracks in walls or ceilings, diagonal or stair-stepping cracks in brick walls, cracks in ceramic or vinyl tiles, bowed or non-vertical walls, wall separating from the floor, sticking doors and windows (with warped door frames) sagging brick lines and bowed or non-vertical walls, separation of timber joints at corners, tilting of retaining walls, sloping floor surface are strong exterior and interior indicators of swelling movement.

However, such distress should not be unduly blamed on expansive soils as some similar symptoms could be observed under other circumstances too.

However, soils containing a high content of high plastic clays and in a very dry state with moisture content below 15%, possessing high strength and high densities in excess of 1.75 gm/cm^3 are suspected to have expansive properties and therefore should be subjected to confirmative tests. Field penetration resistance exceeding SPT N-value of 15 is often associated with expansive soils.

4.3 Solutions to overcome Problems of Expansive Soil

Slight changes of moisture in expansive soils are sufficient to cause swelling detrimental to the structure. Therefore stable and uniform soil moisture conditions should be maintained beneath and around the foundation. To achieve this, ingress of water must be prevented by proper guidance of surface drainage away from the structure with the provision of moisture barriers with cut-off walls around the building, by removing deep roots intruding into building area from large trees and bushes that grow near the building, and by taking adequate precautions to protect against undetected leakage from underground piping or through poorly backfilled trenches, etc. Where practical, the foundations may be taken to a depth sufficiently below the perennial ground water table so that moisture variation does not cause any further swelling of the soil that supports the structure. However, maintaining constant soil moisture conditions seems to be very difficult in practice, at least in the case of dwellings in Sri Lanka where proper building maintenance is lacking. In such cases, replacement of expansive soil with granular material works as a satisfactory solution in cushioning out or nullifying the swelling effects.

Expansive soils been stiff, offers high ground bearing capacity adequate for some lightly to moderately loaded structures. However, swelling pressures measured in the laboratory can be underestimated and the swelling pressures experienced in field may often exceed the bearing pressures, resulting in an upward movement of the structure. In such circumstances, shallow foundations may be designed to exert high bearing pressures which could withstand the swelling pressures, but at the same time ensuring rigidity of the foundation.

Special waffle or raft foundations which can act as a unit to minimize differential action due to swelling may be considered where shallow spread footings or slab-on-grade construction are required. Shallow foundations may also be adopted after stabilizing the soil with cement, lime or chemicals with potassium and ammonium ions.

If the expansive soil layer is not very thick, the foundation may be supported on a deeper, non-expansive soil layer with adequate care taken against the swelling forces acting on floor slabs, for example, by designing them as suspended floors. Where the expansive soil extends to greater depths, under-reamed piles that could resist the upward swelling forces can be adopted (CBRI 1978). The under ream should be anchored in a non-expansive layer or below the perennial ground water table in a zone of no moisture change.

5.0 LATERITE FORMATIONS

Lateritic soils are widely distributed over the wet zone. Most of the early-built-up urban areas had been developed over highlands or hillocks composed of the so called "lateritic" residual soils locally known as "Kabook". A significant feature of the laterite profile is the presence of a vesicular hard crust of ferrocrete or "ironstone" often seen as outcrops, followed by a hard cellular skeleton of iron oxide with clay-filled cavities. This is followed by highly weathered material generally fall under the soil classifications MH & ML, but CH, CL and SM materials are not uncommon. The boundaries of distribution of these different materials in both vertical and lateral directions are not clear and the properties, both physical and mechanical, often vary widely, thus presenting a variable ground condition. It is not unusual to encounter large and isolated pockets of highly "kaolinized" soft clays among the silty material of different color or even to encounter highly porous clusters of weak ironstone cells devoid of clays. Traces of the structure of parent rock could be often seen in the weathered bedrock.

Hence the interpretation of Standard Penetration Test (STP) results too needs careful attention. The STP N-values observed in lateritic clays generally vary from about 5 to 10 while in somewhat harder materials average values around 15~20 could be observed. It is not unusual to observe very high N-values in the order of 30~50 or refusal to penetration within ironstone clusters and weathered rock.

Many research studies are being conducted worldwide and in Sri Lanka to understand residual soils and other unsaturated soils. However, owing to the peculiar heterogeneous structure of this formation in both lateral and vertical directions, the strength and deformation characteristics and the behavior of lateritic soils are yet to be well understood and may not be easily determined by the conventional field and laboratory testing procedures.

A major problem faced in choosing a foundation type would be the interpretation of subsoil data usually obtained from borehole/s deemed to be representative of the site, typically with results of SPTs and/or laboratory tests conducted often with limited number of samples recovered. Careful assessment of the subsoil profile and the field and laboratory tests results is required to at least partly overcome this problem.

Light foundations for single to three storied buildings have been successfully supported over strong lateritic strata using conventional foundations such as strip footings of rubble or brick masonry for load bearing walls and pad footings for columns. However, heavier buildings are generally supported on wide based strip footings, stiffened footings, raft or pile foundations, when weak or variable soil conditions are encountered. Where severe variations in soil type are observed or anticipated, attention may be necessary to investigate for isolated soft or compressible clay pockets which should either be by-passed with deep foundations or be bridged over using rigid foundations.

6.0 FOUNDATIONS ON FILLS

A fill that supports foundations shall be a controlled fill. Compaction Controlled filling may be used beneath structures for one or more of the following purposes;

- To raise the general grade of the structure
- To replace unsuitable soil beneath a foundation which must be removed
- To provide a relatively stiff mat over a weak subsoil to spread bearing pressure from foundation loads and minimize differential settlement
- To bridge over subsoil with local hard and soft pockets or small cavities in erratic profiles
- As preloading to accelerate subsoil consolidation and eliminate all or part of settlement of the completed structure.

Unfortunately most land fillings in the country, except where done under strict supervision of an engineer, are without any compaction control whatsoever. Situation becomes more problematic to the geotechnical engineer where poor soils, construction debris and refuse etc., are dumped and dressed with a thin cover of "acceptable" earth material which is either simply dumped or surface-compacted ignorant of a client. Uncontrolled fills may consist of organic or inorganic materials or of a mixture of the two.

Rigidity, strength and homogeneity of a fill can be largely increased by controlled compaction with proper equipment and selected burrow material. Foundations resting on controlled fills can be designed in a similar manner as in the case of foundations supported on natural soils.

Foundations supported on filled ground may undergo settlements due to;

- Consolidation of compressible fill under foundation loads transmitted
- Consolidation of the fill under its own weight and
- Consolidation of natural ground beneath the fill under the combined load from the fill and the structure and also under the weight of soil in the natural ground.

Often, the load imposed by a thick fill could be higher than the load imposed by the foundation. Settlement under the fill load is important in the design, particularly for a recently placed fill, as the underlying compressible subsoil's may still be undergoing consolidation. For lightly loaded structures, differential settlement caused by the foundation load and the own weight of fill within a properly compacted fill would generally be insignificant and may not pose serious problems to the engineer. Settlement due to consolidation of a fill under its own weight depends on the thickness and the degree of compaction of filled layer and the type of fill material used.

Uncontrolled fills offer irregular profile and are not suitable for supporting foundations. Especially those containing compressible and decomposable organic matter, hard objects like construction debris etc., with large cavities and loose pockets will exhibit complicated and erratic soil profile and offer misleading information even when subsoil investigations are carried out. In such cases, partial or full replacement with well compacted fill, and/or special foundation types would be required as discussed in the case of peaty ground. Stiffened footings can be considered over a well compacted granular fill which replaces the underlying weak soils beneath the footing to a depth equivalent to more than the width of the footing.

When it is practically not possible to recompact or replace the uncontrolled fill, it is desirable to support the foundation on a natural hard stratum using deep foundation.

7.0 CONSTRUCTION ON HILL SLOPES

Central and south west regions of the island comprise of hilly terrain. Hill slopes were mostly used for the major plantations while the upper areas had been reserved forests. As a result of growth of habitation on the hill slopes and development activities climbing uphill, once well maintained hill slopes have begun to degrade. Provision of dwellings and necessary infrastructural facilities for the families dislocated each year by landslide disasters and for the growing urban and rural population in the hill country has become an urgent need.

Geotechnical conditions of the hill terrain are complex and depend on the geomorphology, geotechnical structure, degree of weathering, topography, type of formation and strength of soils combined with many other factors.

NBRO with the assistance of UNHCS launched a research project on Landslides in Sri Lanka and it has also prepared landslide hazard zonation maps to guide the planners, administrators and developers on optimum use of the hilly terrain free from disasters and environmental problems.

Heavy disturbance and damage to the natural hill slopes due to construction activities involving deep cuts and high fillings etc. could be minimized through preparation of geotechnical guidelines or regulations embedded in the building regulations to be properly enforced by the state and the local government.

Conclusion

Difficult ground conditions and problematic soils are amply available in Sri Lanka which deserve higher attention of the geotechnical engineer for theoretical and practical research and develop appropriate technology.

Selection of the appropriate type of foundation and the method of construction to meet difficult ground conditions require careful assessment of subsoil characteristics based on appropriate investigations and laboratory testing. However, in Sri Lanka performance monitoring construction projects, particularly with respect to behavior of foundations is limited. Due to lack of knowledge of the correlation between predicted and actual performances of the foundation and in the absence of adequate understanding on difficult ground behavior, the foundation engineer may not always be successful in his efforts to find optimum solutions for unpredictable ground conditions.

With the introduction of different construction methods for foundations and ground improvement through large scale infrastructure development projects, geotechnical engineers in Sri Lanka are offered with more opportunities for innovations in geotechnical engineering appropriate for the local conditions. Geotechnical engineer must have the courage to develop new methods try and test them on difficult ground conditions. The academicians, practitioners and industry must get together to achieve this with the support of the state.

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Effects of intermediate permeable layers on the consolidation of soft clays with prefabricated vertical drains

Udeni P. Nawagamuwa
University of Ruhuna

Abstract:
For more than 60 years, vertical drains have been employed to promote more rapid consolidation of relatively thick deposits of fine-grained soils. Although the design of vertical drains is based on Barron's or Hansbo's theory considering the clay layer to be homogeneous, some field data from the improved grounds using the vertical drains have shown the advantages of modifying the commonly used equations to account for the presence of permeable laminations in the clay. As such there are clear advantages in doing a thorough study of the effects of intermediate permeable layers that could lead to an optimized design. In this paper, the effects of the presence of intermediate permeable layers and the effects of smear are discussed using the consolidation equations developed by considering the mass balance and variation of permeability and void ratio. Finite difference analysis has been done for the combined effect of vertical and radial consolidation and the results are given with two defined parameters, namely; $K=[k_s/k_c].[H_s/H_c]$ and $\alpha_{98}=t_{98(2D)}/t_{98(radial)}$. A new design methodology is proposed with a set of graphs and a design example.

1.0 Vertical Drains (in general)

Vertical drains have been employed for more than 60 years to promote more rapid consolidation of relatively thick deposits of soft fine-grained soils. Vertical sand drains were first proposed by Daniel E.Moran in 1925 as a means for deep stabilization. He had received a patent on this process in 1926 (Johnson, 1970). The first installation appears to have been in California in 1934 (Porter, 1936) using 20in. diameter sand drains at 10 ft centers. Until the early 1970s the majority of the vertical drains used were large diameter sand drains. The principal alternative to large diameter sand drains, was the much smaller band-shaped cardboard wicks first employed by Kjellman (1948). These early band drains proved to be susceptible to rotting, particularly in acidic groundwater, and the use of band drains was not common until 1970s. In the 1970s the rising costs of providing large quantities of suitable sand for sand drains and the great technical advances in the manufacture of man-made fabrics led to the development and use of increasing numbers of band drains produced from polyethylene, PVC, polypropylene and polyester etc. (Atkinson and Eldred, 1982). Generally these things consist of a central core, whose function is primarily to act as a free-draining water channel, surrounded by a thin filter jacket, which prevents the soil surrounding the drain from entering the central core but allows free entry to the core of the excess pore water.

1.1 Potential advantages of vertical drains

In general terms the installation of vertical drains into a relatively thick stratum of clay before the application of a load should increase the rate of consolidation of the clay under the load by shortening the drainage path. In addition, in non-uniform soils the horizontal permeability may be considerably greater than the vertical permeability; this anisotropy confers an additional advantage on the use of drains.

The advantages can be threefold.

(a) The increased rate of gain in shear strength of the clay enables the load to be applied more rapidly than would otherwise have been possible, often following a better utilization of construction plant. Furthermore, in the case of embankments, steeper side slopes and the avoidances of the use of berms may also be possible when vertical drains are employed. Thus the total volume of fill required may be reduced and the rate of construction increased. The consequent savings in cost may be appreciably more than the outlay on the vertical drain installation

(b) The increased rate of consolidation of the clay results in a reduction of the time required for primary settlement to take place. Consequently structures can be built on embankments can be put into commission far earlier than would otherwise have been possible, or the subsequent maintenance costs can be greatly reduced.

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

The precompression technique with vertical drains has been used on a wide variety of applications such as highways, airfields, earth dams, warehouse floors, buildings, cellular cofferdams, pile foundations, excavations, quay walls, large scale development of marginal areas, etc for the purposes of increasing stability and decreasing post construction settlements.

1.2 Drainage Blanket

In any vertical drain installation it is normal to provide a granular drainage blanket over the complete area where the drains are to be installed. While this provides a suitable working platform for heavy plant, which may otherwise be unable to gain access to the area, 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 early stage of consolidation. When the drain spacing has been finalized, the total quantities of water being discharged into drainage blanket can be calculated at any degree of consolidation. An assessment can then be made of the adequacy of the thickness of the drainage blanket and its design can be adjusted accordingly.

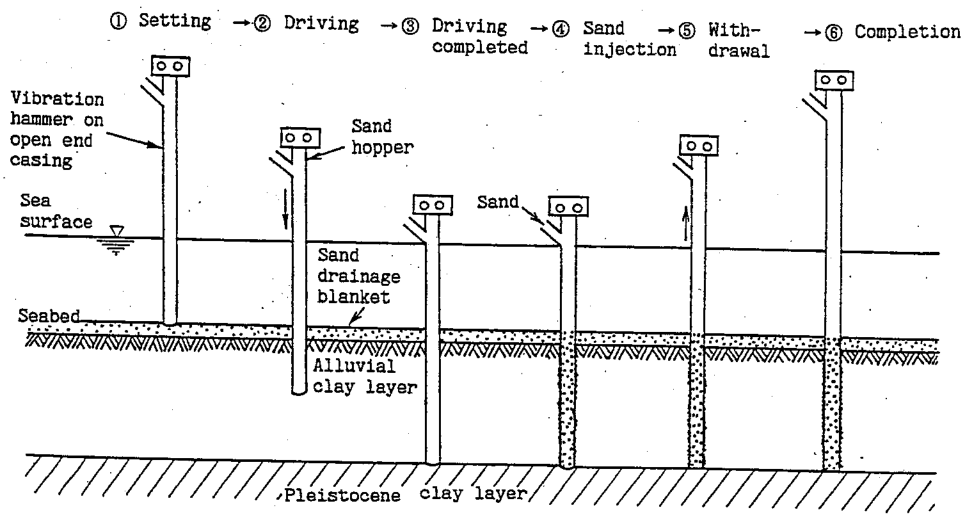


Figure 1.Progress of Sand drain installation in seabed

1.3 Vertical Drain Installation

The sand drain installation procedure is shown in Figure 1. As pore water in the clay layer was discharged to the sand pile by applying a surcharge load, the clay layer decreases in volume and increases in strength. While this method is more economical than others, it requires a longer construction period and careful management.

Some disadvantages of sand drains mentioned by Yeung (1997) are;

(1) The sand to be used for the drains must be carefully chosen to have adequate drainage properties and therefore can seldom be found in the vicinity of the construction site
(2) The drains may become discontinuous due to careless installation or excessive lateral soil displacements during consolidation
(3) Bulking of the sand during its placement in the drain may lead to formation of cavities and collapse on flooding.
(4) The large diameter required for the sand drain may pose a construction problem and/or a budgetary burden
(5) The disturbance to the soil surrounding each drain caused by the drain installation process may reduce its hydraulic conductivity and thereby reduce the flow of water to the drain and the efficiency of the system
(6) The reinforcing effect of sand or gravel drains may reduce the effectiveness of the surcharge loading in consolidating the subsoil.

Various types of prefabricated band-shaped drains that can overcome most of these shortcomings of sand drains are being marketed under different trade names as mentioned by Rixner et al., (1986). Configuration of different prefabricated vertical drains are shown in Figure 2. The prefabricated vertical drains (PVD) shall be installed using a mandrel or sleeve that will be advanced through the soil to the required depth as shown in Figure 3. Constant load or constant rates of advancement methods are the preferred methods.
A vibrator with an eccentric moment

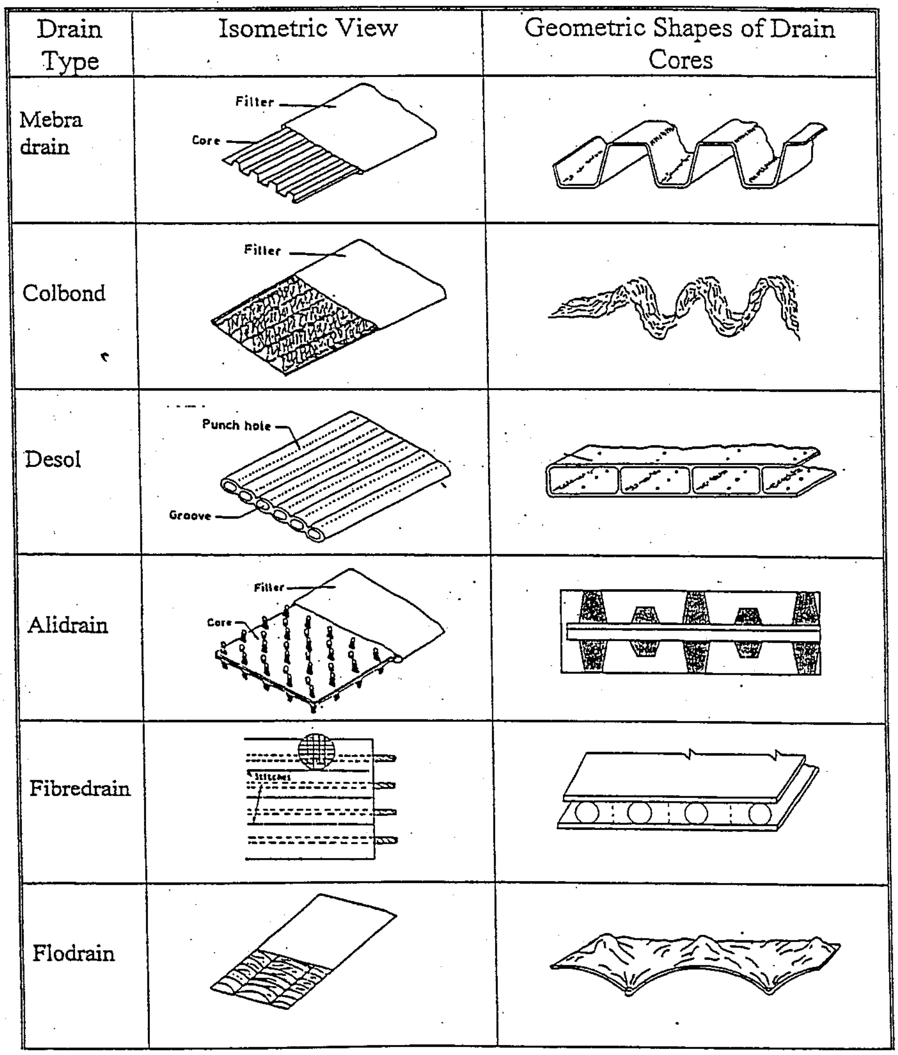
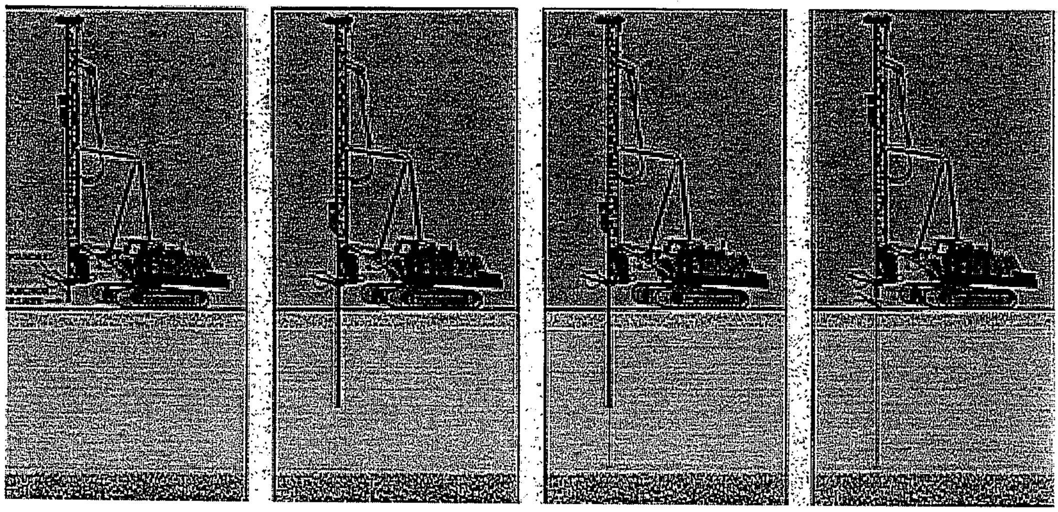


Figure 2. Configuration of different prefabricated vertical drains



(a) (b) (c) (d)

Figure 3. Progress of PVD installation
(a) Installation equipment
(b) Driving mandrel
(c) Extracting mandrel
(d) Cutting drain

shall be available for use in areas where constant load or constant rate of advancement methods cannot install the drains to the design depths. Drains which cannot be installed to design penetration using only static methods must be advanced with the use of the vibrator to be considered for compensation. The vibrator may not be used except in cases where design penetration cannot be achieved by using the full static push force available to the mandrel. Provision must be provided for introducing water into the top of the mandrel, which is rectangular in shape. The mandrel shall protect the prefabricated drain material from tears, cuts, and abrasions during installation and shall be withdrawn after installation of the drain. The main advantage of these PV drains over sand drains is that they do not require drilling, and thus installation is much faster.

1.4 Consolidation mechanism

It has been logically considered the problem of consolidation with vertical drains over a large area to be two dimensional. The flow of water in this case is predominantly in horizontal direction while the availed displacement only in vertical direction. Most of the geotechnical engineers considered these conditions to be valid in the field and design vertical drain system based on the classical theoretical solution developed by Barron in which coefficient of horizontal consolidation (c_h) is assumed to be constant. The value of c_h, which being the combined effect of permeability and compressibility parameters, can be expressed as,

$$c_h = \frac{k_h}{m_v \gamma_w} \qquad (1)$$

where k_h = horizontal permeability
m_v = modulus of volume change
γ_w = unit weight of water

Soil permeability and compressibility decrease with effective stress (σ'), which may balance the value of c_h to be a constant, but Nicholsan and Jardine (1981) revealed an expected decrease in c_h value with effective stress. It had been also reported that the insitu coefficient of consolidation is higher than the laboratory tests results before the range of preconsolidation pressure (p_c') and beyond it approaches to the mean value of laboratory test results. The variation of c_h with effective stress is higher in the stress range before p_c'.

Barry and Wilkinson (1969) examined the effect of c_h critically and proclaimed that the c_h constant is applicable in case of small strain theory only when the value of C_c (slope of e-log σ' curve) is same as C_k (slope of e-log k curve, where k is the coefficient of permeablity). Their analytical solution also argues that if $C_c = C_k$, then the degree of consolidation for a particular time is same irrespective of the load increment ratio, but this values are different if $C_c \neq C_k$. Therefore, it is logical to think twice before using constant c_h value for design purpose which is usually computed from empirical relation.

1.5 Assessment of drain spacing

The main problem of designing a vertical drain scheme is to determine the drain spacing which will give a required degree of consolidation in a specified time for the designed drain type and size in a ground condition that prevail. From the practical viewpoint drain is installed in some square or triangular pattern. The problem is, therefore, no longer axisymetric. Unfortunately, no analytical solution is available for this real problem. Normal practice is to approximate the problem to that of a cylindrical drain placed at the center of the cylinder of soil mass to be considered. The diameter of the equivalent cylinder of soil surrounding each drain D_e is calculated on the basis of equivalent cross-sectional area (as shown in Figure 4).

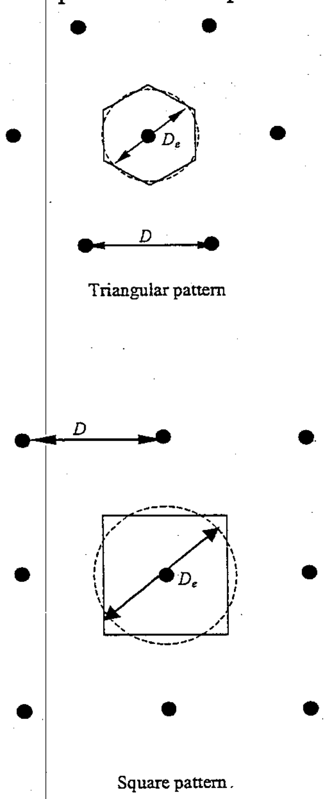


Figure 4. Equivalent diameter of drains in different patterns

For the drains in square grid pattern with drain spacing D;

$$\frac{\pi}{4}D_e^2 = D^2 \qquad (2)$$

$$D_e \cong 1.13D \qquad (3)$$

for a triangular grid pattern;

$$D_e \cong 1.05D \qquad (4)$$

Since the rate of consolidation is approximately proportional to the square of the drainage path, the overall effect is that the average degree of consolidation calculated by means of an equivalent cylinder overestimates the true consolidation. However, for the triangular pattern this effect is smaller as this grid pattern is considerably nearer to the equivalent cylinder. The square pattern has the advantage of easier layout and control, and therefore it is preferred from the viewpoint of utilization, although the triangular pattern provides more uniform consolidation between the drains.

1.6 Depth of drains

The depth of treatment is very often taken simply to be the depth of soft or impermeable soils at a site and for depths of 5-20m vertical drains can often prove to be economic solution to the problem. Beyond 20m depths, the costs of installing the drain rise markedly (McGown and Hughes, 1981). This is essentially due to the rapid increase in the force needed to place the drain: even normally consolidated soils are firm to stiff at 30-40m depths. For example, at Changi, depths of 43m had been treated with band drains. Vane test results at about 22m had indicated undrained shear strengths of just below 60kN/m^2 and the strength-depth relationship suggests strengths of about 70kN/m^2 at 40m in the marine clay (Choa et al., 1979).

The placement of drains to such depths on a continuous production basis requires a large casing entered by a very large, powerful rig, which significantly increases the cost per unit area of site treated. Figure 5 proposed by McGown and Hughes (1981) illustrates this point by considering the cost per unit area treated by 15m deep Sandwicks as the basis cost per unit area. Since it is at shallow depths that most settlement, particularly differential settlement, derives and most chance of shear failure exists, then the cost-benefit of treating depths of 30m or 40m must be questioned. In situations where compressible soils extend down to 30 or 40m the relative merits and costs of more intensive shallow treatments to greater depths should be one of the principal design considerations.

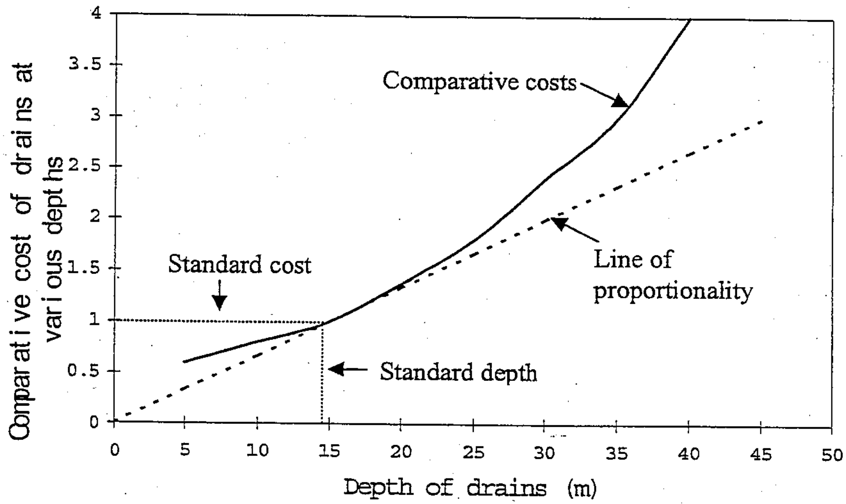


Figure 5. The relationship between costs and depth of band drains taking the cost for 15m depths as the unit cost (McGown and Hughes, 1981)

1.7 Smear and well resistance

The installation will distort and remold the soil in the vicinity of the drain. Thus a zone of smear will be created with reduced permeability and increased compressibility. The effects of smear caused by installing the drain using displacement methods can be largely overcome by the correct choice of the drain filter fabric and the size of the installation lance. The effect of the internal resistance of a vertical drain to the flow of the collected water is defined as well resistance. This resistance has a large effect on the consolidation process, in certain circumstances increasing the time to achieve a particular degree of consolidation by about one order compared with that theoretically obtainable with an infinitely permeable drain (Atkinson and Eldred, 1982).

2.0 Necessity of the Studies on intermediate permeable layers

Vertical drains have been installed successfully in compressible soils to speed up consolidation in many soil improvement and land reclamation projects in the world. Design of vertical drains is usually based on Barron (1948) or Hansbo's (1979) theory although there is evidence that the predicted results of settlements are apparently different from the actual field observations, which cannot be explained by such a simplified approach proposed for a uniform soil. Assumption of drainage taking place only in the radial direction has been considered in the above two theories. However that assumption is not valid in lot of cases, especially for the clayey grounds consist with intermediate permeable layers. Because the assumption of no dissipation of excess pore water pressure in the vertical direction is not justifiable for those layered soils. The existence of the intermediate permeable layers will reduce the drainage path and increase the rate of consolidation thus reduce significantly the waiting time to achieve the required settlement, and in certain cases even the use of vertical drains can be omitted (Gue and Tan, 2001). In most situations the overall degree of consolidation of a 2D model is taken as the combined effect of radial and vertical drainage as defined by Carillo (1942). However, Carrillo's equation presumes the above and beneath layers to be infinitely permeable to calculate the vertical consolidation and in fact it is not practically valid in natural situations with intermediate layers having finite permeability.

Analytical solutions have been developed for radial consolidation for a number of cases of practical interests. There are two widely used methods for the design of vertical drains such as Barron's (1948) solution for ideal situation without smear and Hansbo's (1979) solution including the disturbance during the installation and well resistance. Yeung (1997) developed design curves taking the drain diameter as the independent variable and the drain spacing as the dependent variable to avoid unnecessary iterations and interpolations. Design charts for vertical drains considering construction time have been proposed by Zhu and Yin (2001). Nogami and Li (2003) proposed a design method and design charts for an optimum system of horizontal and vertical drains where horizontal drains in the system were made of multiple thin pervious layers such as sand layers and geotextile sheets, while vertical drains were vertical cylindrical drains. However this method has been used for artificial high permeable horizontal layers, which have a very high coefficient of permeability compared to clay. All these design charts discussed in this paragraph considered a constant c_h value in spite the need of considering the change of c_h value during the consolidation process is a must for a better prediction.

Even though there are lot of field observations on the effects of consolidation with vertical drains due to natural intermediate permeable layers, so far little theoretical studies has been done on that. In this study, 2-dimensional numerical studies have been done considering clay layers having several intermediate permeable layers with finite permeability, using the non-linear void ratio, permeability and effective stress relationships in order to propose a new set of design charts.

2.1 Evidence from field information

A clayey soil medium often contains horizontal thin highly permeable layers that exist either naturally or artificially. Natural ones are typically found in glacial lakes and postglacial clays, which were often formed with a varved or layered structure due to seasonal variation in deposited particle size. Examples of artificial thin layers are found in some reclaimed lands with clay fill (Lee et al., 1987, Karunarathne et al., 1990) and in clay embankments.

The settlement records of Terminal Island Access Freeway stated by Stanton (1948), the test field at Skå-Edeby, Sweden (Hansbo, 1960), which consisted of postglacial varved clay containing thin layers of sand or silt only near bedrock at depths between 9 and 15m and three successive lake deposits due to glacial action in Derwent Reservoir (Ruffle, 1965) are good examples of glacial and post-glacial clays with varved and layered structures, that were met in vertical drain installations.

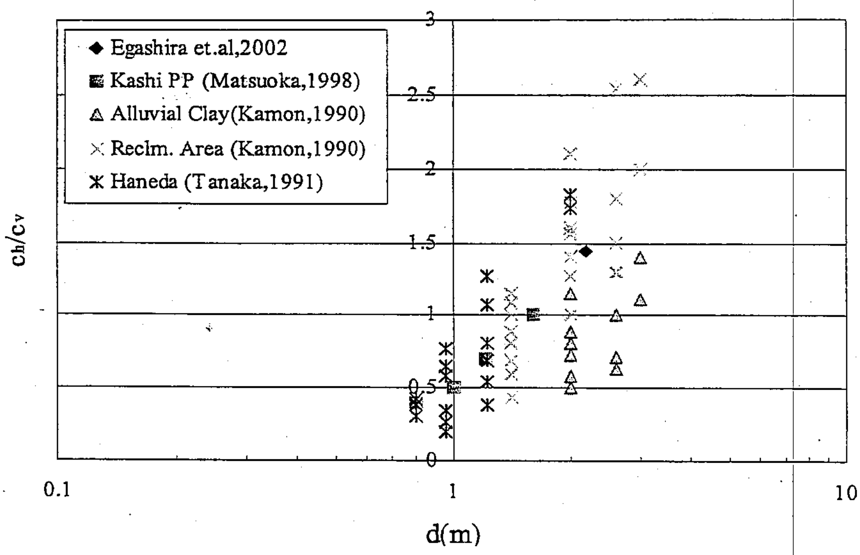


Figure 6. Several field observations (from Egashira et al., 2002)

Several field observations on c_h/c_v (where c_v has been calculated from laboratory tests and c_h from back calculating the field data) and spacing of vertical drains (d) relationship are shown in Figure 6. As it reflects reclaimed lands have higher c_h/c_v values i.e. greater than unity and alluvial marine clay has such values closer to unity. As Win et al., (2001) concluded, the lower c_h values are the result of the significance of the smear effect in soft marine clay due to vertical drain installation while higher values indicate the significant effect of the presence of macro fabric or permeable layers in clay.

Calderon and Romana (1997) discussed the soil improvement process by precharge and prefabricated vertical drains at Tank Group No. 3 site, at the "TOTAL Oil Storage Plant" at Velencia Harbour in Spain. The soil of the site came from dredgings of Velencia Harbour and identified as layers of sandy silt, sandy clay and silty sand. Moreover, into each layer there had been a lot of lenses of sand and clay, of very variable thickness and position. Calderon and Romana

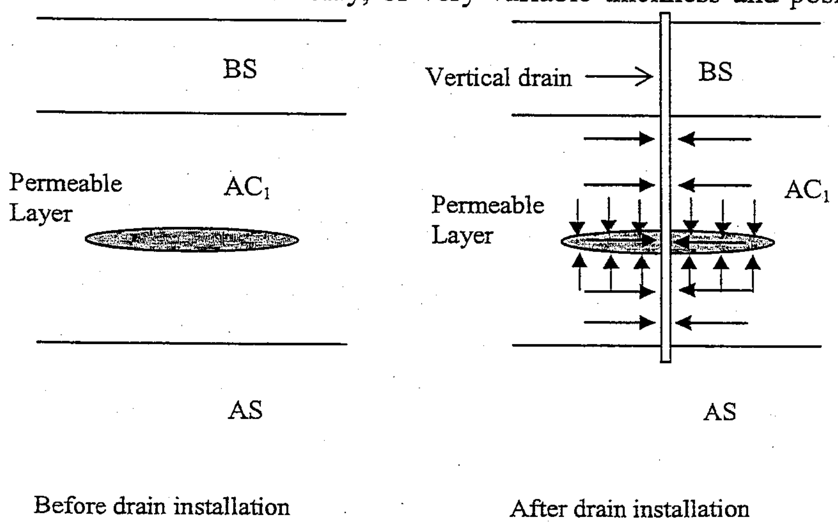


Figure 7. Effect of existence of permeable layer (from Tanaka et al., 1991)

(1997) concluded that the effect of smear is far more important in a soil with a marked difference between vertical and horizontal permeability and suggested the significance of including that effect in designs.

By analyzing Tokyo International Airport (Haneda) site, Tanaka et al., (1991) concluded that the values of c_v obtained from the observed settlements depend considerably on spacing of vertical drains, because it could be due to the existence of permeable layer. An extent of these permeable layers may be not large enough to function as drainage layer for the consolidation of alluvial clay layer AC_I. However, once a vertical drain is installed, these permeable layers become to function as a drainage layer, and the consolidation of AC_I layer is accelerated as shown in Figure 7.

These highly permeable layers provide lateral drainage for the clay medium in the consolidation stage. With sufficient discharge, they modify the hydraulic gradient of vertical flow of pore water in the clay medium. This results in the complex coupled behavior of the consolidation of clay and confined lateral flow of discharged pore water in sand/silt layers. Evaluation of the behavior of such a system requires a rational formulation to take the coupling into account.

3.0 Theoretical background of the study

In order to study the mechanical behavior of a multi-phase mixture, a set of equations that governs its behavior such as mass conservation, momentum balance and flow relation have been used in the derivation of the theory developed by Nawagamuwa and Imai (2005).

In reference to Figure 8, the vertical inflow of water into the small element at time Δt considering the variation or permeability and void ratio in radial and vertical velocities, the following equation can be formulated.

$$\frac{\partial e}{\partial t}=\left(C_1\frac{\partial^2 u}{\partial r^2}+C_2\frac{\partial u}{\partial r}\right)+\left(C_3\frac{\partial^2 u}{\partial z^2}+C_4\frac{\partial u}{\partial z}+C_5\right) \qquad (5)$$

where,

$$C_1=\frac{(1+e)k_r}{\gamma_w}, C_2=\frac{(1+e)}{\gamma_w}\left(\frac{k_r}{r}+\frac{\partial k_r}{\partial r}\right), C_3=\frac{1}{\gamma_w}\frac{k_z}{1+e}$$

$$C_4=\frac{1}{\gamma_w}\left(-\frac{k_z}{(1+e)^2}\frac{\partial e}{\partial z}+\frac{1}{1+e}\frac{\partial k_z}{\partial z}\right), C_5=\frac{\partial k_z}{\partial z}$$

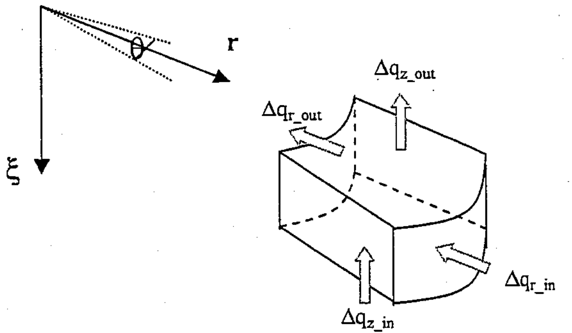


Figure 8. Movement of water in a small clay element

where k_r and k_z are the coefficient of permeability in radial and vertical directions respectively and u is the pore water pressure. Considering $\Delta\sigma'=-\Delta u$ for the consolidation under a constant load, using $-\Delta e=A_I\,.\Delta\sigma'$ and $A_I=0.434C_c/\,\sigma'$ Eq. (12) can be transformed into a normal differential equation in terms of excess pore water pressure.

The following non-linear relationships are considered in the calculations to find out the change in the permeability (k) and the void ratio (e) due to an increase of effective stress σ'.

$$e = N_k + C_k \log k \qquad (6)$$

$$e = \Gamma - C_c \log \sigma' \qquad (7)$$

Since the overall stability and deformation depend on effective stress or pore water pressure of the soil elements, the average degree of consolidation at time t can be considered as a measure of overall improvement of clay layer. It is calculated by,

$$U_t = 1 - \frac{\int_A u(r,z,t)dA}{\int_A u_0 dA} \tag{8}$$

where u_0 and u are the initial and current pore water pressures respectively, and dA is the area of the small element corresponding to a grid point.

3.1 Definition of new parameter K and α_{98}

Various definitions had been defined to discuss the effects of intermediate permeable layers (one dimensional study of Gray (1945), filter efficiency of Gibson and Shefford (1968), layered clay-sand scheme by Tan et.al. (1992), etc) . In the present study, a new parameter K is proposed as;

$$K = \frac{k_s\, H_s}{k_c\, H_c} \tag{9}$$

for vertical drain installation in the clayey soils which have intermediate permeable layers. In this definition, k_s and k_c are permeability of high permeable intermediate layer and clay layers respectively and H_s and H_c are the thickness of those layers respectively.Here the parameter K is the ratio of the products of the horizontal permeability and thickness of the more permeable layer to the less permeable layer as mentioned by Rowe (1964).

The ratio between the time needed for 98% consolidation considering 2D flow ($t_{98(2D)}$) and radial only ($t_{98(radial)}$) , α_{98}, is defined as follows.

$$\alpha_{98}=t_{98(2D)}/t_{98(radial)} \tag{10}$$

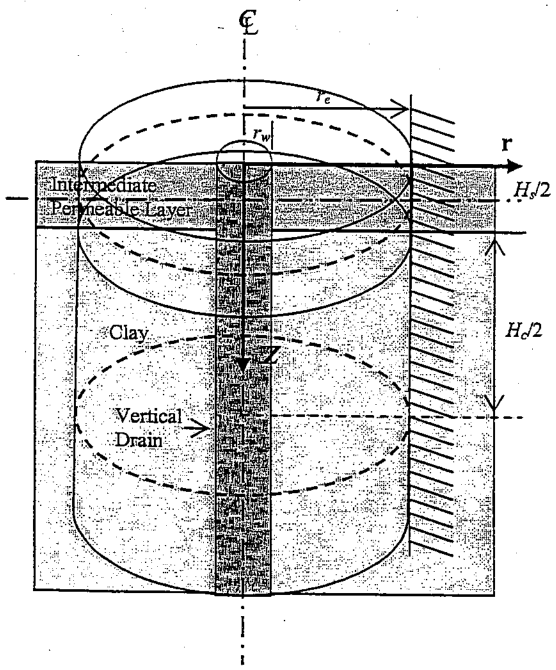


Figure 9. Schematic diagram of clay with vertical drains and intermediate permeable layers

3.2 Numerical Analysis

The problem studied in this research relates to the consolidation of a multi layer system with vertical drains. As shown in Figure 9, due to the symmetrical situation of clay and intermediate permeable layers, half of the selected area is considered for the analysis. In this figure H_s and H_c are the thickness of permeable (sand) and clay layers respectively. Well radius (r_w) and the effective radius (r_e) or the radius of the equivalent cylinder of soil influenced by the drain is also shown in the Figure 9. It is considered that the permeability of the vertical drain as infinite and the permeability in the permeable layer (sand) and clay as k_s and k_c respectively. Due to symmetry, the drainage condition is PTIB, i.e. the top surface is pervious and the bottom is impervious.

As shown in Figure 10, for different K values selected according to the classification proposed by McGown et. al., (1980), the ratio of α_{98} for clay thickness normalized by effective radius (H_c/r_e) is discussed for r_e=140cm. Here $K=\infty$ presents the situation where the intermediate permeable layers have infinite permeability. In this situation, α_{98} is almost zero for very thin clay layer. It can be clearly seen that the effect of K value is significant especially when H_c/r_e ratio is less than 1.0. This indicates that the effect of K value is considerable when clay layer thickness is less than two or three times of the effective radius of the vertical drain. In actual clayey soil, there may be several intermediate permeable layers where the thickness of the clay layer (H_c) between two intermediate permeable layers is small, then this relationship may be useful to analyze actual consolidation behaviors. Using this relationship, it can be observed that lower K values result in higher ratio of α_{98} for smaller thickness of clay layers. However, for thick clay layers, this relationship is not observed. Small α_{98} values for thinner clay layer means that whenever there are closely spaced thin sand layers, vertical consolidation results in a shorter time required for full consolidation.

On the other hand, when there are no closely spaced intermediate permeable layers, i.e. clay layer thickness between two intermediate permeable layers is high, consideration of radial only consolidation is reasonable and there is less effect on consolidation due to the presence of permeable layers since α_{98} value becomes unity.

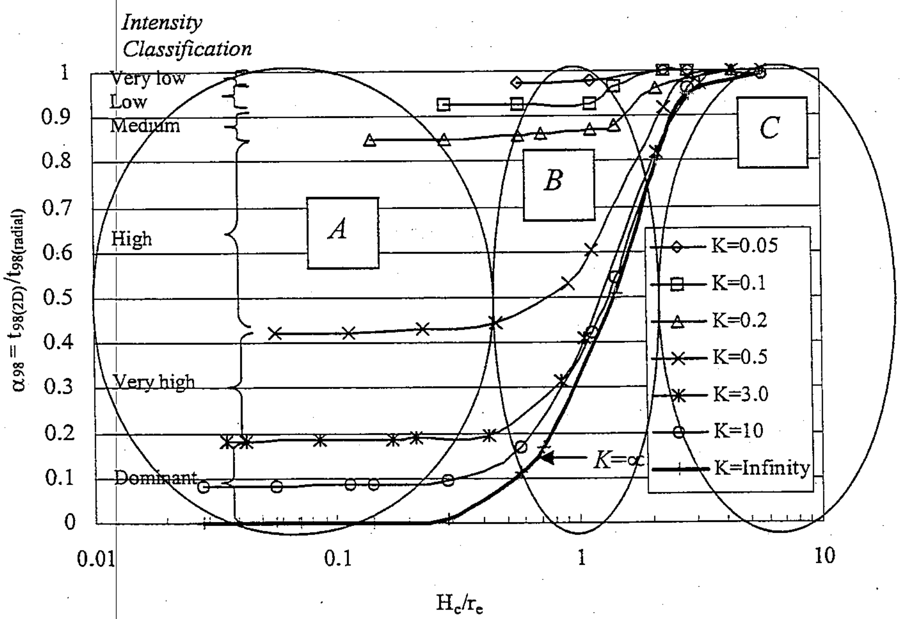


Figure10.Variation of the effect of intermediate permeable layers in 3 different zones; Zone **A**, **B** and **C**

As proposed by Rowe (1964), as a rough guide, for H_c/D<1/20 (where $D=2r_e$), average c_h in many cases would have reached its maximum. This region is equal to H_c/r_e<0.1 of Figure 7 and it can be concluded that the minimum α_{98} value for any respective K case seen in this region will confirm its maximum coefficient of consolidation in this region. Because the minimum constant α_{98} value of any K refers to the maximum effect of permeable layers and it can be referred to the maximum average c_h.

Figure 11 shows a comparison of the consolidation time with respect to no-smear situation in different clay thickness. It also compares the same situation using Hansbo and Barron's solutions. Hansbo's solution represents 1D smear situation and Barron's solution represents 1D no-smear situation. With the following theoretical explanation it clearly describes that the effect of smear is still active even in very high n ($=r_e/r_w$; r_e and r_w have been defined previously) values. However, in reality, the effect of smear should be negligible in very high n values and the present theory gives a better solution for larger n values, especially at the end of the consolidation period.

The following two equations are widely used for vertical drain design in smear and no smear (ideal) situations.

Barron's solution (no smear or ideal situation)

$$U_r = 1 - \exp\left[\frac{-8T_r}{F(n)}\right] \tag{11}$$

where $F(n) = \dfrac{n^2}{n^2-1}\ln(n) - \dfrac{3n^2-1}{4n^2}$ (12)

Hansbo's solution (smear situation without well resistance) $U_h = 1 - \exp\left[\dfrac{-8T_h}{\mu}\right]$ (13)

where $\mu = \ln\left(\dfrac{d_e}{d_s}\right) + \dfrac{k_h}{k_{smear}}\ln\left(\dfrac{d_s}{d}\right) - \dfrac{3}{4}$ (14)

$d_e=2r_e$, $d=2r_w$
k_h – radial permeability in clay
k_{smear} – permeability in smear zone
d_s – diameter of smear zone

Using these equations, for the same degree of consolidation, it can be shown that for larger n values, for the situation discussed in Figure 11, $d_s/d=3.0$ and $k_h/k_s=3.0$

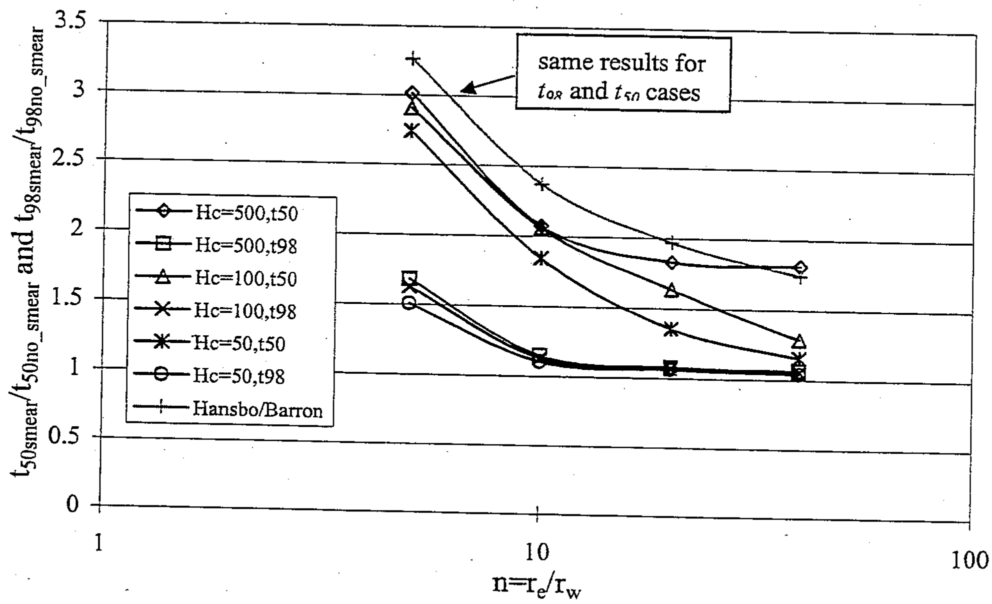


Figure 11. Effect of smear compared to no smear situations for different thick clay layers, K=3, k_r=3.0k_s, r_s=3.0r_w

$$\frac{t_{Hansbo}}{t_{Barron}} = 1 + \frac{2.196}{\ln(n) - \frac{3}{4}} \tag{15}$$

Using Eq. (15), for a larger n value such as 100, t_{Hansbo}/t_{Barron} = 1.57 and this clearly shows a significant deviation from the unity compared to the present theoretical results. Because present theoretical results show a lesser effect of smear in larger n values especially at 98% consolidation and the values are closer to the no smear situation.

Comparing the curves shown in Figure 8, the following conclusions can be made.

1. There is no change between 98% and 50% consolidations in $\text{Hansbo}_{\text{smear}}$ /$\text{Barron}_{\text{ideal}}$ ratio for any n value. However the present theory shows a difference between those types. t_{98} ratio approaches towards unity for higher n values indicating less effect of smear.
2. Effect of smear is severe in low n values.
3. When clay thickness is smaller, it will have fewer disturbances due to smear for higher n values even for t_{50}.
4. Since the Hansbo/Barron solution gives a result which is not closer to the reality, especially regarding the high smear effect even in higher n values, this comparison shows the advantage of considering the different thickness properties into consolidation calculations with smear effect.
5. Richart (1959) concluded that the relationships between the degree of consolidation and time are approximately equal when the radius of the zone of influence (r_e) is greater than approximately 10 times the radius of the drain (r_w) well or when the average degree of consolidation is greater than 50%. These findings too confirm those conclusions.

3.4 Design procedure using the present theory

A new design procedure is proposed here in order to include the effect of intermediate permeable layers and smear. Based on the theoretical approach discussed in the paper with all the definitions and using the model test results of Hird and Mosley (2000), r_s=1.6r_w and k_h/k_s=3 have been applied for the design curves. The following procedure can be applied for the design.

1. Perform detailed subsurface investigation and laboratory soil testing to identify field drainage conditions and to obtain pertinent geotechnical properties of the compressible soils. More investigations are needed to identify the properties of intermediate permeable layers such as thickness and permeability. Using these field and laboratory data, calculate K.
2. Determine the permeability and compressibility relationships as mentioned in Equations (6) and (7). These relationships are different depending on different soil types and different curves should be used for those various relationships. If the attached design curves are used for the design, note that these curves are designed for an average Japanese clay with the following properties, which can be used in those two equations.
   N_k=8.64, C_k-0.72, Γ=3.0 and C_c=0.70
3. Determine the average degree of consolidation U required and the amount of time available for the consolidation process t.
4. Select an appropriate prefabricated vertical drain on the basis of equivalent diameter; discharge capacity; jacket filter characteristics and hydraulic conductivity; and material strength, flexibility and durability; and calculate r_w = $(a+b)/\pi$. If sand drains are going to be installed it can be decided with the available machinery or can be decided at the later stage.
5. Assume a value for spacing and calculate d_e value depending on triangular or square as 1.05S or 1.13S respectively. Calculate n value according to this assumed d_e.

6. Calculate time factor (T_r) using the initial c_h value.

$$T_r = \frac{c_h t}{d_e^2} \tag{16}$$

7. Find the thickness of clay layer (H_c) and categorize the field condition into following three regions. The behaviors in these three regions are similar irrespective of any K and n values. $H_c/r_e<1.0$, $H_c/r_e \cong 1.0$ or $H_c/r_e>1.0$
   These 3 regions can be considered as H_c= 50, 100 and 500 as used in the design curves.
   The different zonation such as **A**, **B** and **C**, is shown in Figure 10.
8. Find the best suitable set of graphs to satisfy the conditions. If exact n value is not found, use interpolation to find the best n value.
9. Match this n value with the previous assumed value and repeat the steps until it finds the best solution. Design the vertical drain scheme as triangular or square system

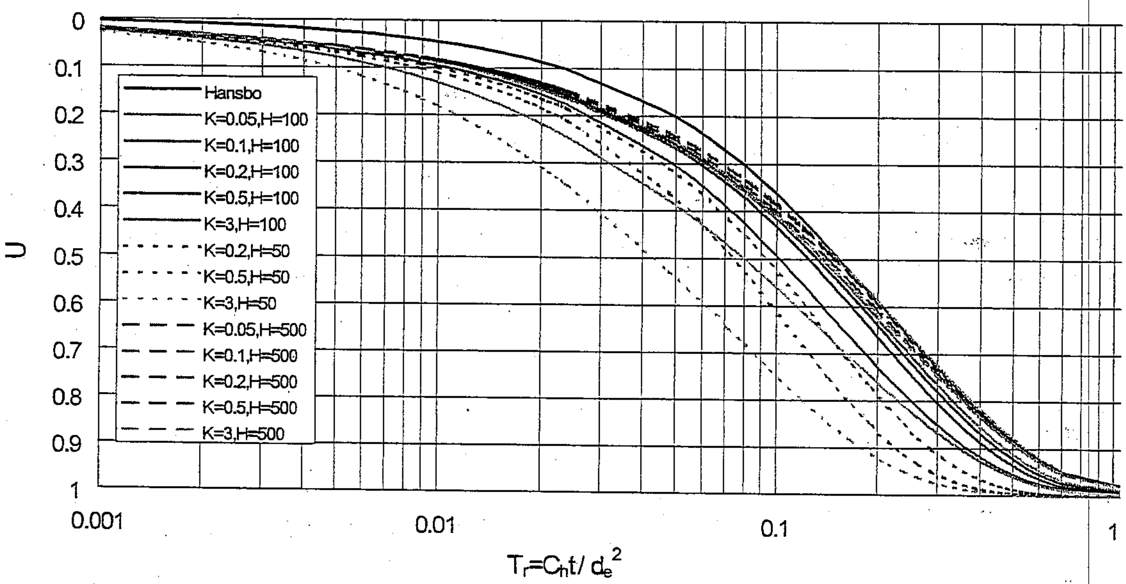


Figure 12. Design Chart 1: Average degree of consolidation U vs. Time factor T_r for n=10; for Prefabricated Vertical Drains (PVD)

3.5 Example of application

As an example calculation, let us consider the case:

H_c= 100 cm, H_s=12.5 cm, k_s/k_c=4.0

Then K= 0.5.It is required that an average degree of consolidation of 70% would be achieved at time t=0.75 years since the start of construction. Assume the initial c_h=0.25m^2/year

PVD parameters are a=90mm, b=4.25mm

Then $r_w = (a+b)/\pi$=30mm =3cm

Using Eq. (27),

T_r=0.25*0.75/(1.2)^2 =0.13

Use the graphs developed for PVD designs.

Using the given K, U find T_r,

Trying n=10 graph (Figure 12), T_r= 0.18

Using Eq. (16),

d_e=1.02

n=17

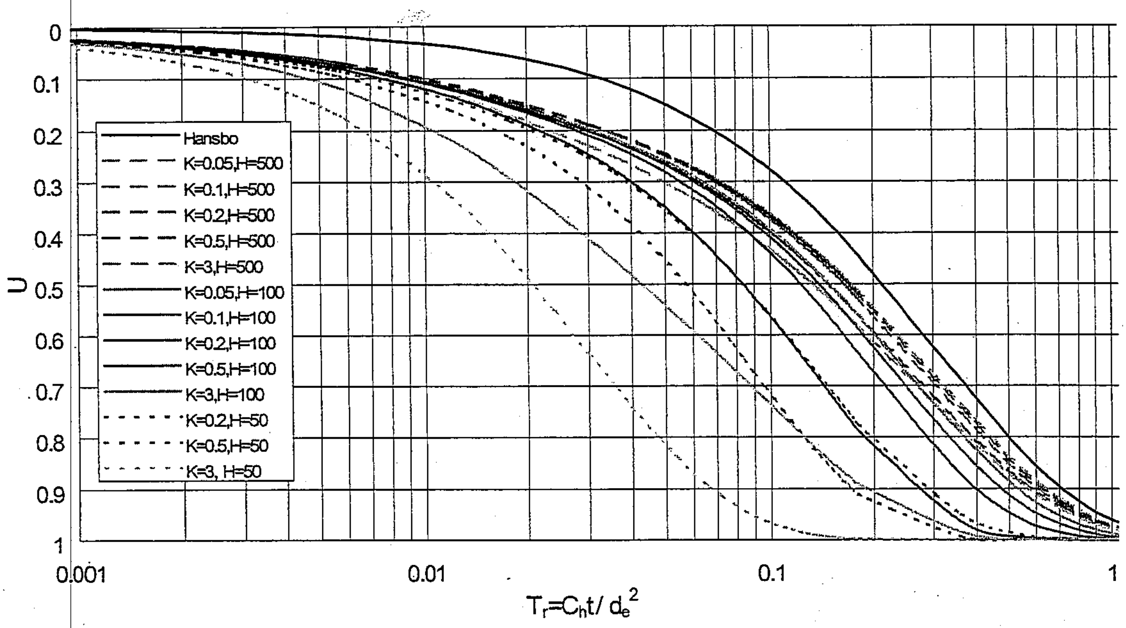


Figure 13. Design Chart 2: Average degree of consolidation U vs. Time factor T_r for n=20; for Prefabricated Vertical Drains (PVD)

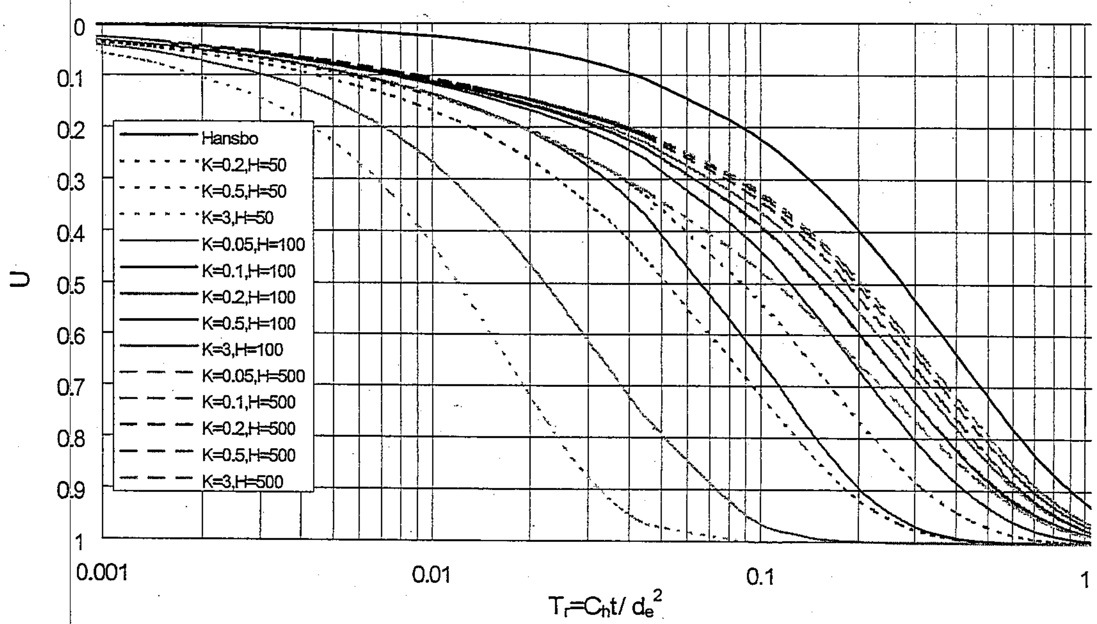


Figure 14. Design Chart 3: Average degree of consolidation U vs. Time factor T_r for n=40; for Prefabricated Vertical Drains (PVD)

Trying n=20 graph (Figure 13), T_r= 0.14
Using Eq. (16),
d_e=1.157
n=19.3

Trying n=40 graph (Figure 14), T_r= 0.122
Using Eq. (16),
d_e=1.02
n=20.66

Using the above 3 calculations, most suitable n value is 20 when the assumed value and the observed value are same.
For triangular scheme d_e=1.05S; S=1.1m
For square scheme d_e=1.13S; S=1.02m
If Hansbo's solution is used for this situation without considering the influence of intermediate permeable layers, the average degree of consolidation will be around 40% during the time of 0.75 years for a design with n=20.

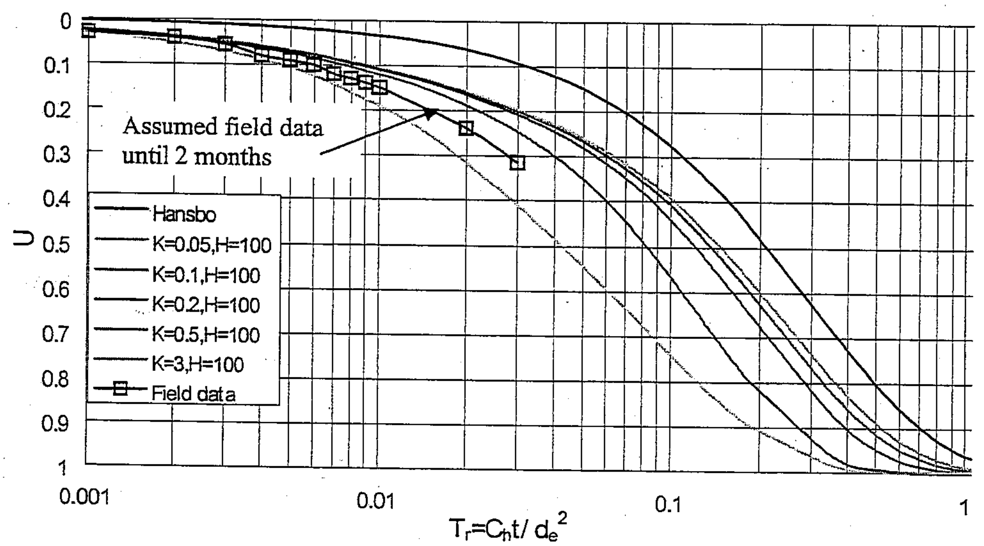


Figure 15. Application of the design charts for back calculations, n=20

3.6 Use of design charts for backward analysis

One of the major problems is the identification of intermediate permeable layers within the clay layers during the investigation stage. Due to inappropriate investigations or inadequate techniques, there will be insufficient data to incorporate a K value in the design as proposed in the proceeding sections. However, after designing the vertical drain installation based on Hansbo or Barron's solution, it may be found that actual degree of consolidation calculated according to the settlement data are quite different from the design predictions. For an example, lets consider a set of assumed field data of degree of consolidation with time as shown in Figure 15. As it shows, Hansbo's solution for n=20, and the field data are very much different. However, when the design graphs are developed using different K values for n=20, a closer solution to the observed situation can be found by proper mathematical interpolations. There after better predictions can be made on the future behavior. Figure 15 shows how the data for a period of 2 months is used to modify the predictions. According to the modified predictions the construction could be completed earlier.

4. 0 Conclusions

By comparing the Barron and Hansbo results for a wide range of n values, it is observed that the present theory gives a better solution for higher n values as well, since time ratio for a particular degree of consolidation with smear to no-smear, should be closer to unity for higher spacing due to the effect of disturbance near the drain wells becomes less with the distance. It can be seen that the effect of smear is severe in low n values and it will be even higher for lower degree of consolidation. This different relationship cannot be found with the Barron and Hansbo solutions mainly because of no consideration of thickness of clay layer in the solutions.

Design curves for prefabricated vertical drains, using the effect of intermediate permeable layers and smear have been developed considering three major regions observed depending on H_c/r_e values. The design procedure is outlined and one example has been discussed. The advantage of considering intermediate permeable layers depending on the permeability and thickness of different layers can be observed in these graphs.

5.0 Acknowledgement

Late Prof. Goro Imai of Yokohama National University, Japan is always remembered for his advices during author's studies with heartfelt thanks. Financial support given by the Ministry of Education, Science, Sports and Culture, Japan is also gratefully acknowledged.

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Improvement of Soft Peaty Clays by Electro Osmosis With Electro Kinetic Geosynthetics

Dr. Athula Kulathilaka - University of Moratuwa

Abstract

Finding economical and speedy methods for the improvement of extremely soft peaty clays is a major challenge ahead of the Sri Lankan geotechnical engineers involved in new infrastructure development projects. Extremely soft consistency of soils encountered in some sites makes it impossible to place any fill without the use of reinforcements and separation layers. Even with that the construction will have to be done in several stage to prevent shear failures. In this context the possibility of consolidating the extremely soft peaty clays by electro osmosis – without application of a physical load was studied. Laboratory studies have indicated that the method would be feasible and the use of non corrosive electro kinetic geosynthetics has given many advantages. These studies should be supplemented by a well instrumented field investigation.

1. Background

Peaty clays are with very high water contents and found in low lying areas. Major Geotechnical problems are associated with constructions done on peaty clays due to very high primary and secondary consolidation settlements and very low shear strengths. Due to unavailability of land with good sub soil conditions, Sri Lankan Engineers are compelled to use land underlain by soft peaty clays for the infrastructure developments such as highways. In number of new proposed highway projects several kilometers are underlain by soft peaty clays of water contents as high as 300-400%. Finding effective, economical and speedy methods to treat them is a major geotechnical challenge.

Preloading had been used in some instances. But due to the very low shear strengths, the required fill had to be placed in several stages and the duration of the project increased considerably. Prefabricated vertical drains may be used to accelerate the consolidation process and reduce the time period. In some sites where Peaty clays of extremely soft consistency were encountered, the placed fill has simply gone into the ground. Geotextiles and geogrids may be needed to overcome these problems. Reinforcement of the soft Peaty clay with the introduction of stone columns or sand compaction piles in a designed grid pattern is another viable option. Technique of deep mixing had been tried out successfully on Sri Lankan peaty clays at the laboratory level. It would be necessary to conduct further research to develop effective and inexpensive field mixing techniques. Under this background, improvement by electro osmosis consolidation is an alternative technique that needs to be investigated. Pioneering research done at University of Moratuwa at the laboratory level has indicated that the engineering properties of Sri Lankan peaty clays could be significantly improved by electro osmotic consolidation.

2. Mechanism of Electro Osmosis

In a clay, particles are platy shaped and are with a negative surface charge. In dry clay, adsorbed cations are tightly held by the negatively charged clay particles. Cations in excess of those needed to neutralise the electro negativity of the clay particles and associated ions are present as salt precipitates. In saturated clay the precipitated ions go into the solution. Because the adsorbed cations produce a much higher concentration near the surface of particles, they try to diffuse away in order to equalize the concentration throughout. However, their freedom to do so is restrained by the negative electric field originating at the particle surface. This results in an ion distribution as shown in Figure 1 adjacent to a clay particle in suspension. The charged surface

Bjerrum(1967) designed and implemented a field application where a 10m thick layer of soft sensitive clay was improved by the electro osmotic treatment given over 120 days. A polarity reversal was adopted for about 8 days during the project. Reinforcing bars of 19 mm diameter and 10 m length were used as electrodes and they were arranged in rows of 2 m apart with a spacing of 0.65 m within a row itself. Small trenches were cut along cathode rows to facilitate the free flow of water expelled. The average shear strength of the sensitive clay has increased to 3.9 t/m^2 from an initial value of 0.9 $t/^2$ and the sensitivity has reduced from 100 to 2. The pre consolidation pressure has increased from 8 t/m^2 to 35 t/m^2.. The liquid limit has also increased and pH value of the soil has increased near the cathodes and decreased near the anodes. The total power consumption was given as 17 $kWhr/m^3$ of the treated clay. Bjerrum et al (1967) claimed this as an improvement from the value of 60 $kWhr/m^3$ quoted by Casagrande(1949).

Lo et al (1991a and 1991b) presented a series of laboratory and field studies conducted to assess the effectiveness of the electro osmotic process in strengthening of soft sensitive clay in a test site at Gloucester, Ottawa, Canada. The average initial moisture content of the soil varied between 60-80% and the undrained shear strength determined by the vane shear apparatus was 12 kPa. After the successful laboratory test program, field studies were done using perforated copper pipes as electrodes. Perorated copper pipes were expected to provide easy passage for the expelled water and the gas generated during the test. The average undrained shear strength increase achieved with the treatment range from 23 – 62 %, with larger percentage increases achieved with closer spacing of electrodes. These were less than the improvements of 60%-172% achieved in the laboratory. Treatment has increased the pre consolidation pressure of the clay, to values in the of 68- 88 kPa from an initial value of 48.5 kPa.

A series of large scale laboratory experiments were conducted at AIT Bangkok on application of the electro osmotic consolidation to improve the properties of soft Bangkok clay. Nayar (1997) used a modified Rowe cell in his study. Chaudhary (1998) did his study using a modified triaxial apparatus with measurement of settlement and pore water pressure at the anode. Patawaran(1998) used a much larger 450 mm diameter 950 mm high cylinder made of 10 mm thick transparent PVC sheets in study. Dinoy (1999) used the same apparatus to conduct further experiments with the installation of four vertical drains that were made electrically conductive. Remoulded Bangkok clay was used in all these studies after some pre consolidation. Shear strengths were increased by electro osmosis and higher strength increases were obtained with higher voltage gradients. Polarity reversal has resulted in more uniform strength increases, with more uniformity achieved with shorter intervals of polarity reversal. Generally, the strength increases were larger at the initial anode.

Micic et al (2003) studied the influence of the electro osmotic process in the enhancement of carrying capacity of skirted foundations in marine clays. Skirted foundations are being used increasingly to support off shore structures. They are expected to resist both the compressive and tensile (pullout) loading. Studies were done in a large model tank of dimensions 1000mmX750mmX700mm, made of polypropylene. A total number of 13 electrodes were installed, some inside the cylinder and some outside. Electro osmotic consolidation was done with the polarity reversal. At an electrode point, a solid stainless steel rod of 16 mm diameter was used as an anode and a 10 mm perforated stainless steel pipe was used as a cathode. The test was done with simulated marine clay of conductivity 5.4 Siemens/m and initially consolidated at a pressure of 15 kPa. There was a control test without any electrical treatment. Electrical treatment was given over 28 days with a polarity reversal after 14 days. The average shear strength of the electrically treated clay was 35 kPa as compared to the value of 12.3 kPa obtained in the control test. The failure load of the model skirted foundation was 7.9 kN in the electrically treated sample as compared to 2.5 kN in the control test. However, the difference in the water contents between the electrically treated soil and the soil in the control test was only1%, clearly indicating the strength increase in electro osmotic consolidation was not simply due to dewatering. The power consumption was very high at 61.5 kWhr/m^3, in this highly conductive marine clay.

3. Further Enhancement needed in the technique to meet new challenges

The effectiveness of the process of electro osmosis in the improvement of engineering properties of soft clays was well demonstrated by the number of studies reviewed in the preceding section. However, the method had not been used very widely to improve soft clays.

The problem of the corrosion of the metal electrodes and its influence on the efficiency of the electro osmotic process as highlighted by some researchers could be a concern. Another major concern affecting the wide acceptability of the electro osmotic technique is the conception that it would be involving high costs on power. In the initial studies done by Bjerrum (1967) the cost of power consumption was 25%, but in the field study done by Lo et al (1991 b) with a better electrode design, the power consumption cost was only 1 % of the total project cost. The power consumption is usually expressed as kWhrs /m^3 – the power required to improve a cubic meter of the treated soft soil. Wide ranges of values were reported in various studies. Some reported values are; Casgrande (1949) in Germany 60 kWhr/m^3; Bjerrum (1967) in Norway, 17 kWhr/m^3; Chappell and Brown (1975) in Singapore 0.5 kWhr/m^3 ; Lo et al (1991) in Canada 2136kWhr/, Chow et al(2004) in Singapore 1.8 kWhr/m^3. The power consumption would increase with the conductivity of the soil. In the marine clays and simulated marine clays used in their laboratory model studies, research group at University of Ontario, Canada has reported power consumptions were in the range of 50-60 kWhr/m^3.

There were few records of the use of non metallic electrodes in the process. Mohomedelhassan and Shang (2001) used Graphite electrodes in their study and have reported that the loss of voltage at the anode was much greater in the graphite electrode compared to a copper or steel electrode. The voltage loss at the cathode was not influenced by the electrode type. The voltage transferred to the soil was defined in terms of an efficiency factor and it was found that the efficiency improves with the applied voltage gradient. Abiera's (1999) attempt to make a conventional PVD conductive by applying a coating of electro conductive paste and wrapping with carbon fibre had been unsuccessful. Chew et al (2004) conducted a field study in Singapore marine clay used a conductive PVD which was patented as EVD. They have been successful to some extent but some deterioration was also reported. Therefore, making electrically conductive drains that are non corrosive and causing minimum voltage loss at the soil/electrode contact is a major challenge the industry is facing at present.

4. Development of new Electro Kinetic Geosynthetic (EKG) Materials and Band Drains with EKG

Band drains with EKG materials were developed as a result of number of years of pioneering research at the University of Newcastle Upon Tyne in 1990's and in collaboration with the Netlon Ltd. An Electrokinetic geosynthetic (EKG) may be defined as a "composite material which may provide filtration, drainage, reinforcement in addition to electrical conduction". The geosynthetic material used in the product was made conductive by the addition of carbon black powder to the conventional polymers.

Organic polymers with all carbon backbones are insulators. If carbon black, carbon fibre or finely divided metals are used as fillers in such organic polymers, then conductive polymers can be created. In these polymers conduction is performed by the fillers (not by the polymer). If carbon black is used as the conductive filler then a loading of between 20% and 30% of weight would be required to produce a suitable conductivity. Carbon black is available in the form of a very fine particulate powder and is formed from the controlled burning of hydrocarbons. EKG band drain consists of an electrically conductive geonet core surrounded by a thermally bonded non woven filter fabric. (Figure 3). The specific design had been patented by Netlon Ltd under the patent application GB2327686A (Netlon Ltd. 1998)

Monofilament wires were located at the centre of alternate ribs to act as current distribution stringers. (Figure 3). The wires were of much higher electrical conductivity than the conductive polymer and this arrangement provides a more efficient distribution of current through the length of the EKG. If the stringers were not included then the electrical current would have taken the path of least resistance, passing majority of the current to the soil at the point of initial contact. This would have caused highly localised current densities with possible adverse implications.

Numerous laboratory studies were done with the EKG band drains developed, with regards to their deterioration with time as during the electro osmotic process and certain initial defects identified were rectified.

5. Basic parameters indicating the feasibility of electro osmosis on Peaty clay

Although numerous studies were done on the application of the electro osmosis technique to improve inorganic clays there were no records in literature of the use of electro osmotic consolidation for the strengthening of organic clays or Peaty clays. With series of one dimensional electro osmotic consolidation studies conducted on Sri Lankan peaty clays Kulathilaka et al (2004), Kulathilaka and Sagarika (2006) have shown that, the primary and secondary consolidation characteristics and undrained shear strength of Sri Lankan peaty clays could be improved by the electro osmotic consolidation.

In the study presented in this paper, initially, the basic indicative parameters on electro osmotic permeability for the Kaolin and Peaty clay will be compared. The technique had been applied vetry successfully in Kaolin. (Hamir et al 2001, Pugh 2002). Remoulded samples of peaty clay obtained from Galle, Sri Lanka in the Southern Highway Project site were used in this study. The basic properties of the peat used are presented in Table 1.

The coefficient of Electro-osmotic permeability k_e, is an important parameter that controls the water flow induced by electro-osmosis and subsequent consolidation. The negative pore water pressure that would develop at the anode in a closed anode configuration and the rate of flow of a permeating fluid in an open anode configuration are related to the electro osmotic coefficient of permeability. The conductivity (ρ) is an indicator of the electric power consumption in the process, with power consumption increasing with the conductivity. The electro osmotic efficiency - the rate of seepage per consumed power given by the ratio k_e/ρ , is another useful indicator.

According to the Helmholtz–Smoluchowski theory, the coefficient of electro osmotic permeability is independent of grain size and varies from $1X10^{-9}$ to $1X10^{-8}$ m^2/sec-V for fine-grained soils (Mitchell, 1993). The coefficient of hydraulic permeability, k_h, however, decreases rapidly with decreasing pore size. Because of its independence of the pore size, electro osmosis would be very effective for water movement in fine grained soils. (Mitchell 1993). To generate electro osmotic consolidation / dewatering effects of engineering significance, the ration of k_e/k_h (m/V) should be greater than 0.1 (Mohomadelhassan and Shang 2001).

A simple electro-osmotic permeability testing cell was designed and fabricated to measure the coefficient of electro-osmotic permeability and conductivity. The apparatus facilitated one dimensional electro osmotic flow under a zero hydraulic gradient. The conductivity of the soil was determined using the same sample, prior to the permeability test. Tests on both Kaolin and Sri Lankan Peaty Clay were conducted on the same apparatus. Tests were also conducted on Bentonite for comparison. The details of the tests are presented in Kulathilaka and Sagarika(2005).

The average values of the coefficient of electro - osmotic permeability determined from a test of 8 hr duration are presented in Table 2. It could be seen that the values obtained for the Peaty clay are comparable with those of Kaolin and lies within the range suggested by Mitchell (1991). The conductivity values obtained are presented in Table 3. The conductivity of Peaty clay was much lower than Kaolin, but lies within the acceptable range 0.002 – 0.2 Siemens/m suggested by Mitchell (1993) and the acceptable range of 0.005 – 0.05 Siemens/m suggested by Pugh (2002). The computed values of electro osmotic efficiency are presented in Table 4. It is evident that the values obtained for the Peaty clay are comparable with those found for Kaolin, where the technique was shown to be effective by other researchers (Hamir et al 2001).

Another indicator used in the assessment of likely success in the electro osmotic process is the k_e/k_h ratio. The hydraulic permeability increases proportional to the square of the pore size and vary significantly with the void ratio. Therefore, it depends on the stress level. With a large scale consolidation model study Kugan (2004) found that the hydraulic permeability of the same peaty clay used in this study would be around 1.2×10^{-8} m/s over a stress level 0 – 20 kN/m^2 and it would reduced to 6.5×10^{-9} m/s for the stress range 20- 80 kN/m^2. Thus the ratio k_e/k_h would be 0.19 and 0.36 for the stress levels 0-20 kN/m^2 and 20-80 kN/m^2 respectively,(Table 4) and greater than 0.1 for both stress level satisfying the suitability criterion for meaningful electro osmotic consolidation. (Mohomadelhassan and Shang- 2001). Pugh (2002) suggests that for efficient treatment by the electro osmotic process, the soil should have a hydraulic permeability sufficiently low to generate a reasonable negative pore water pressure, but should be high enough to give a realistic rate of flow. The suggestion made was that the k_h value should be less than 10^{-8} m/s. The values quoted for Peaty clay in the preceding sections are in this range.

Peat type	Amorphous granular peat
Liquid limit	115.5
Specific Gravity	2.02
pH	4.10
Organic Content	20.8 %

Table 1 : Basic Properties of the Peaty Clay used

Soil type	Moisture Content (%)	Voltage, V	Voltage Gradient, V/m	k_e, m^2/s-v
Peaty Clay	342	3.75	37.5 V/m	2.35×10^{-9}
Peaty Clay	342	7.50	75.0 V/m	1.46×10^{-9}
Kaolin	85.0	7.50	75.0 V/m	6.53×10^{-9}
Bentonite	535.0	7.50	75.0 V/m	7.96×10^{-9}

Table 2: Electro-kinetic permeability values of different soils

Soil type	Moisture Content (%)	Conductivity (S/m)
Peaty Clay	342	0.012
Kaolin	85	0.036
Bentonite	535	0.337

Table 3 – Conductivity values obtained for different soils

Soil	Conductivity (ρ) Siemens/m	Electro Osmotic Permeability (k_e) m/s^2 V	Electro Osmotic efficiency k_e/ρ	K_e/k_h ratio
Peaty Clay	0.012	2.35×10^{-9}	1.96×10^{-7}	0.19 – 0.36
Kaolin	0.036	6.53×10^{-9}	1.81×10^{-7}	0.14 - 2.89
Bentonite	0.337	7.96×10^{-9}	2.36×10^{-8}	***

Table 4 – Comparison of electro – osmotic efficiency and k_e/k_h ratio

6. Electro Osmotic Tests on Peaty Clay with EKG Electrodes

6.1 Arrangement for the Test

The Sri Lankan Peaty clay used in this test was with an initial moisture content of 342.09%, a specific gravity of 2.2 and a pH value of 4.07. The conductivity of the Peaty clay was 0.012 Siemens/m and the electro osmotic permeability was $2.35\text{X}10^{-9}$ m^2/sV. Organic content is 20.88%.

The electro osmotic consolidation test on the peaty clay was conducted in a box made of Perspex with two EKG electrodes with filters (Figure 4). The test was conducted 12 days with polarity reversals. The polarity reversals were not done at regular intervals owing to the response shown by the Peaty clay. Prior to the test on peaty clay several tests were done in a similar setup on Kaolin remoulded at a moisture content of 85%. (greater than the liquid limit of 65%). The test on peaty clay was carried out for a longer duration than with Kaolin to assess the maximum level of improvement that could be obtained. By the time of the termination of the test, 608.3 ml of water (13.93% of initial volume) was expelled from the peaty clay under electro osmosis consolidation. Prior to the start of the electro osmosis, the sample was allowed to consolidate under its own weight for 4 days and 110 ml of water was expelled.

The power consumption was 5.70 $kWhr/m^3$ of soil treated and this was comparable with the power consumption of 4.44 $kWhr/m^3$ obtained for the Test 5 with Kaolin, where a volume reduction of 21.05% was achieved. The power consumption in the test 2 with Kaolin was 6.14 $kWhr/m^3$ over eight days and a volume reduction of 12.14% was achieved. The details on; test duration, power consumption, percentage volume reduction and overall dewatering efficiency for the whole test, are presented in Table 5 for all the tests for the comparison purposes.

Test Detail/No	Test Duration (Days)	Percentage Volume Reduction %	Overall power Consumption $kWhr/m^3$	Overall Dewatering Efficiency $m^3/kWhr$
Kaolin - 1	6	18.65	7.86	0.024
Kaolin - 2	4	12.14	3.95	0.030
Kaolin - 3	8	19.69	6.14	0.032
Kaolin - 4	8	15.07	14.00	0.011
Kaolin - 5	6	21.05	4.22	0.050
Peaty Clay	12	13.93	5.70	0.024

Table 5 - A comparison performance in all EO consolidation Tests

6.2 Electro Osmotic test and observations

The test was commenced with a voltage gradient of 37.5 V/m (Figure 5). A voltage drop of 9.4 V was applied across the electrodes placed at a spacing of 250 mm. As a result of the lower conductivity of Peaty clay,(0.012 Siemens/m as compared with 0.030 Siemens/m for Kaolin), the current developed was much lower. The current and the applied voltage was measured regularly and the variation current density and the voltage gradient were plotted with time in Figure 5. At the latter stages of the test, in order to get a reasonable rate of flow, the voltage gradient had to be increased. A notable difference in peaty clay was that relatively high voltage gradients were needed. However, owing to the lower conductivity, the power consumption will not be unduly increased.

EKG materials offer a very specific advantage in this context. In the three dimensional simulation of the electro osmotic process in a large Perspex box (Kulathilaka and Sagarika 2005), where perforated stainless steel tubes were used as electrodes, when voltage gradients were increased with the objective of enhancing the rate of flow, the stainless steel electrodes corroded rapidly without producing the desired results. It was noted in this test that with the EKG electrodes, high voltage gradients could be used for reasonably long durations, without causing any deterioration in the electrodes.

Another notable difference from Kaolin, witnessed in the behaviour of Peaty clay in the electro osmotic consolidation, is its delayed response to a polarity reversal. This behaviour was witnessed in earlier studies as well. (Kulathilaka et al 2004). When the polarity was reversed after one day, the rate of flow decreased considerably. Therefore, it was decided to double the voltage gradient to 75 V/m at the end of the second day while maintaining the same polarity

(Figure 5). This caused an increase in rate of flow and the same polarity was maintained for 5 days.

When the polarity was re reversed again in the sixth day the rate of flow slowed down considerably Therefore, the voltage was increased to the maximum possible level of the instrument 30 V(Voltage gradient 120 V/m) after one day. The response with an enhanced rate of flow was shown only after two days from this event. Therefore, it appears that when polarity reversals are done in Peaty clay, it is most appropriate to do so with an increase of voltage gradient as well. In one dimensional tests done with Peaty clay (Kulathilaka et al 2004), no further consolidation occurred after the polarity reversal, where same voltage gradient was maintained.

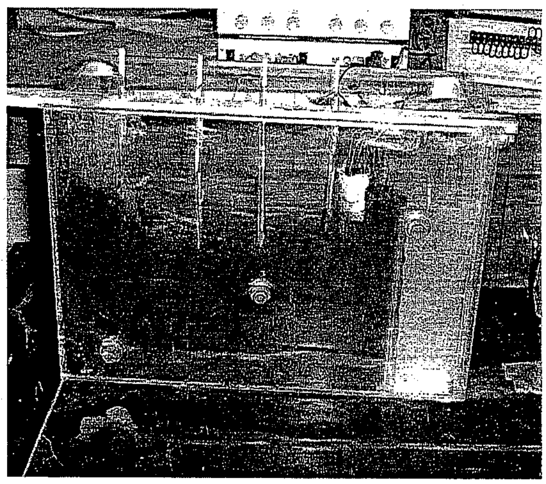


Figure 4 – Arrangement for electro osmotic consolidation of Peaty clay – a front view

Variation of the voltage between the anode and the cathode was measured at four points with the help of voltage probes. In the initial stages the voltage distribution along the sample was linear with similar losses at the anode and the cathode (Figure 6 a). As the test progressed, the voltage distribution along the sample became nonlinear (Figure 6 b). The gradient near cathode was high indicating a region of higher resistance. When the polarity was reversed, the voltage gradient became greater at the vicinity of new anode (former cathode), indicating the existence of a region of high resistance. As time progressed with the new polarity, the voltage distribution changed transferring the zone of high gradient towards the new cathode.

Voltage transferred to the sample was initially around 50%, but varied between 40%-50% as time progressed. With the polarity reversals there was an instant increase of voltage transferred to the sample, but it reduced almost instantly. (Figure 7).

The variation cumulative volume of the water expelled with time is graphically presented in Figure 8. The test was terminated when the rate of discharge became very low.

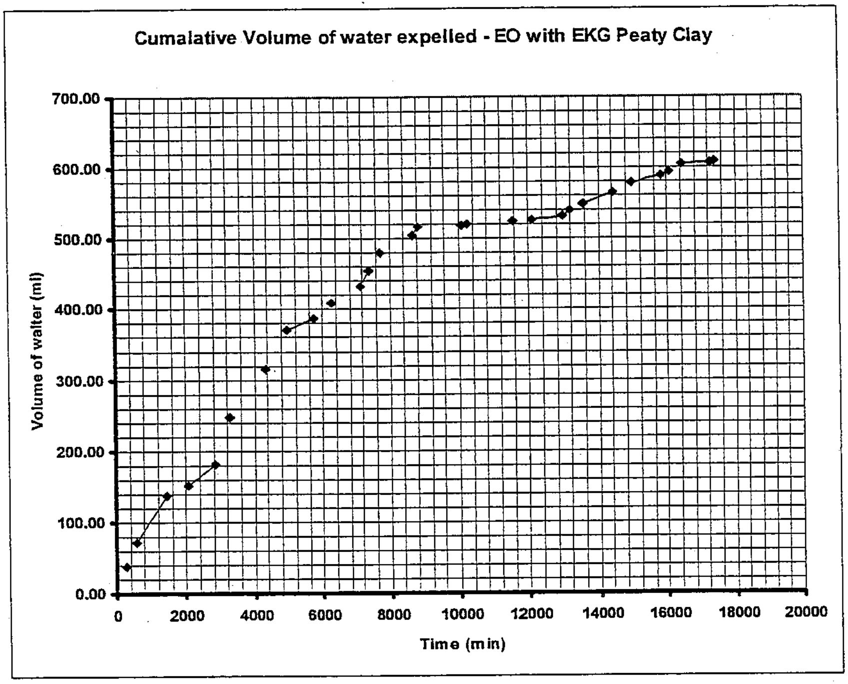


Figure 8 - EO consolidation of Peaty Clay

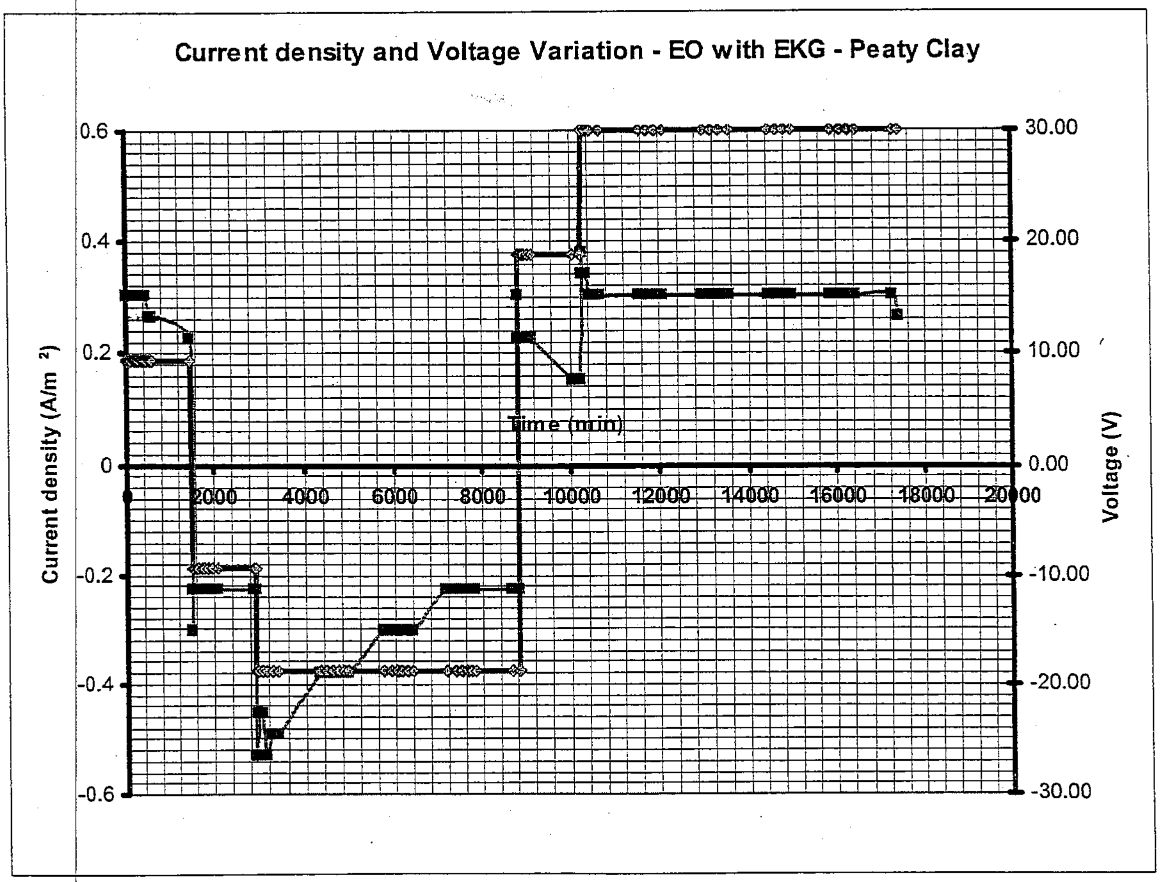


Figure 5 - Variation of current density and voltage - EO consolidation of Peaty Clay

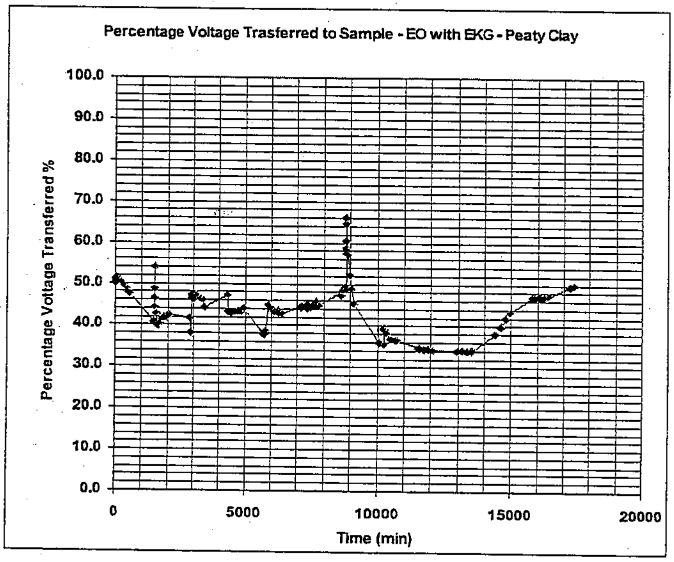
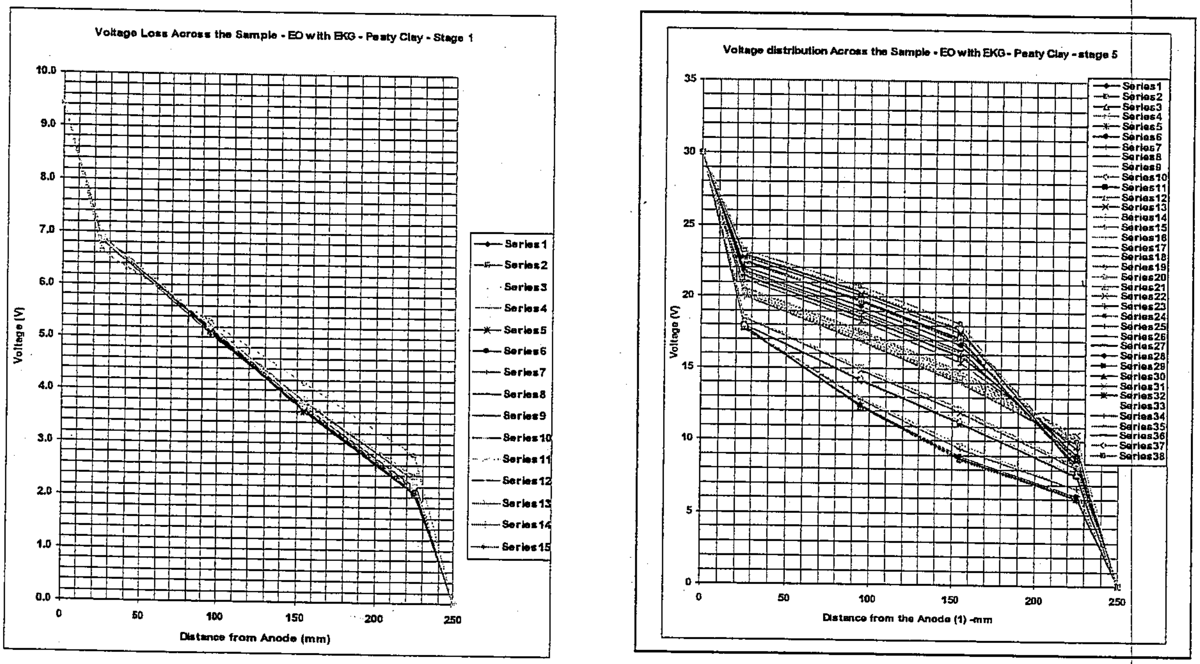


Figure 7 - Percentage voltage transferred to the sample – EO of Peaty clay with EKG



(a) Stage -1 **(b) Stage 5**

Figure 6 - Voltage distribution across the sample - EO of Peaty Clay with EKG

6. 3 The Variation of pH values in Peaty clay during the treatment.

The initial average pH value of the Peaty clay was 4.07. Peaty clays of even lower pH values, in the order of 3.0 are encountered in Sri Lankan sites. The variation of the pH value of the Peaty clay was determined daily from the second day onwards, by scooping out small samples just below the surface. The pH samples were taken close to the location of the voltage probes. Also, the pH values of the peaty clay in contact with the electrodes were measured regularly. The pH value of the water expelled from the cathode was also monitored regularly. At the initial stages pH value of the water expelled was around 7.22 and this value increased to around 9.12 as time progressed. After a polarity reversal the pH values of the water expelled at the new cathode were low (5.26) initially but increased with the time. There was no clear indication of a pH gradient between anode and cathode in peaty clay samples obtained. The pH value of the soil in contact with the anode has reached 2.85 at latter stage. In the tests done with Kaolin there was a clear pH gradient between anode and cathode and the pH of the water expelled has risen to more than 12.0. The lower pH values of the expelled water could be attributed to the low initial pH value of the peaty clay.

7. Improvements in shear strength – Peaty Clay

The undrained shear strength of peaty clay was found to be 1.26 kN/m^2, through the laboratory vane shear apparatus, prior to the electro osmotic treatment. After the electro osmotic treatment undrained shear strength was determined at the location shown in the Figure 9. The test locations were around 100 mm below the surface at the time of treatment. Two undisturbed samples were also obtained for the consolidation tests and those locations are also shown in the same Figure. The Vane shear tests indicated some strain softening after the peak value. The shear strength values obtained at the moisture content at the corresponding locations are presented in Table 6. The strength increases differ with the locations but considerable strength increase have occurred everywhere. Even the minimum increase obtained was greater than 400%.

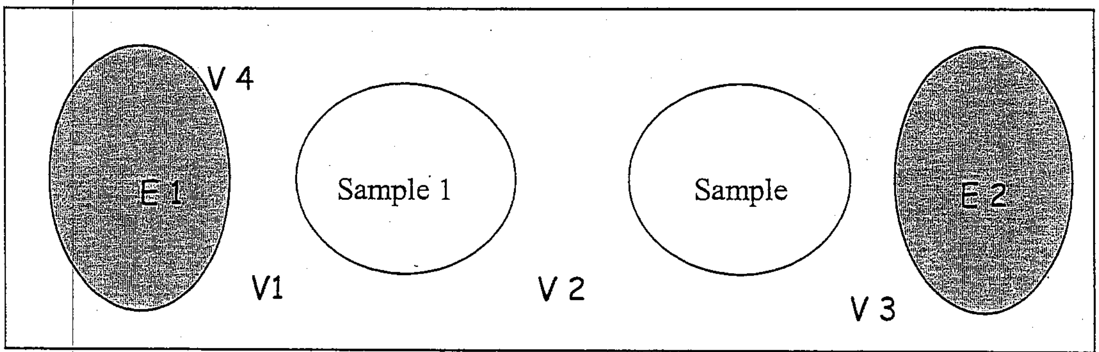


Figure 9 - Locations of Vane shear tests and Consolidation samples – Peaty clay

Location	Undrained shear strength kN/m^2	Moisture content %
V 1	25.14	214.45
V2	9.43	233.19
V 2 - Deep	10.69	233.35
V 3	6.91	235.94
V 4	5.03	211.76

Table 6 - Undrained shear strength and moisture contents of treated Peaty clay

8. Consolidation behaviour of Treated Samples of Peaty clay

Two consolidation samples taken from the treated peaty clay were subjected to the Oedometer test with load increments 5 kPa, 10 kPa, 20 kPa, 40 kPa and 80 kPa. In view of the importance of secondary consolidation characteristics, loading increments of 3 days duration were used. The water content at the start of the tests were 232.39% and 225.38% for the sample 1 and sample 2 respectively. This is a reduction of more than 100% from the initial moisture content of 342.09% at the commencement of the electro osmotic consolidation. The locations of the two samples were quiet close to each other and the electro osmotic consolidation was done with several polarity reversals (Although not regular). This could be the reason for having very close moisture contents at the end of the treatment. The locations of the samples were given in Figure 9.

For the comparison purposes, a sample of untreated Peaty clay was taken through the load increments 5kPa, 10 kPa, 20 kPa to 40 kPa and then unloaded to 5 kPa and reloaded back though, 10 kPa, 20 kPa, 40 kPa to 80 kPa. All the loading, unloading and reloading increments were of 3 days duration.

For the purpose of establishing the improvements achieved in primary consolidation characteristics, the e Vs log (σ) plots of untreated peat, preloaded peat (to 40 kPa), treated peat sample 1 and treated peat sample 2 are presented in Figure 10. Similarly the m_v values are compared in Figure 11 and the C_α values of the samples are compared in Figure 12.

The e vs log σ plots of both electrically treated samples were quite similar indicating the uniformity of the treatment received. The test was done with polarity reversals. The electrically treatment samples appear to have a preconsolidation pressure of around 30 kN/m^2. The e vs log σ plot of the sample preloaded to 40 kN/m^2, clearly indicated a preloading pressure of 40 kN/m^2

The initial void ratios of the electrically treated samples were reduced to values lower than that of the preloaded sample. But the gradients of the e vs log σ plot of the electrically treated samples upto the preconsolidation pressure, were somewhat greater than that of the preloaded sample. Similar observations were done in the tests conducted with Kaolin.

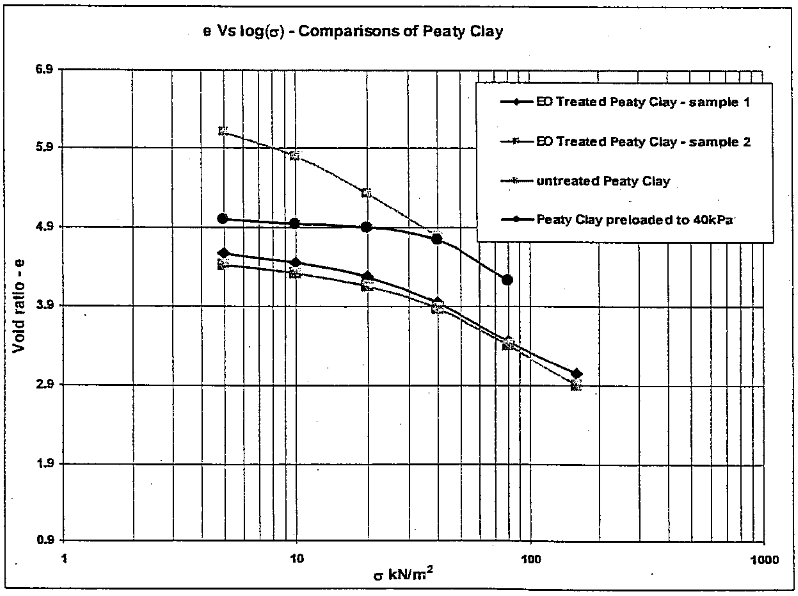


Figure 10 - comparison of e Vs log (σ) plots of Peaty Clay

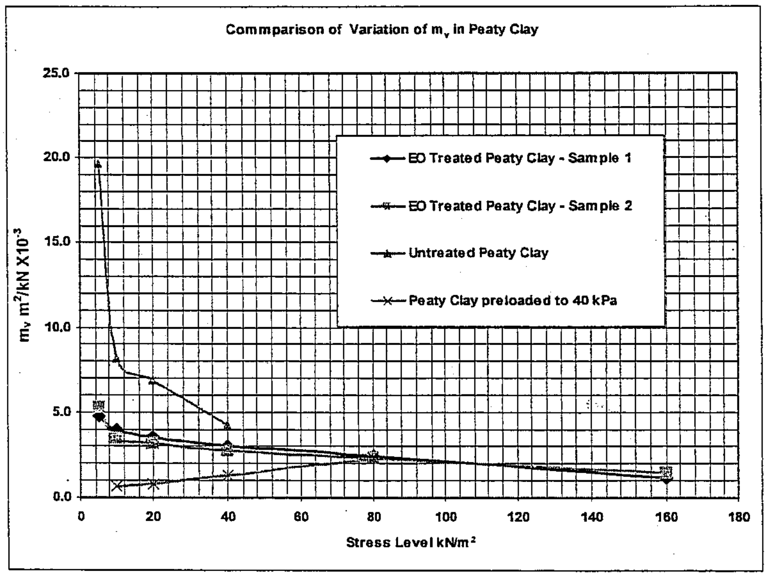


Figure 11 - Comparison of m_v plots in Peaty clay

The m_v values of the two electrically treated specimen were significantly reduced from the values of the untreated sample, for the similar stress levels, as illustrated in Figure 11. The m_v values of the samples preloaded to 40 kN/m^2 were much smaller, for the same stress levels. The values of coefficient of secondary consolidation ($C\alpha$) were also considerably reduced in the electrically treated samples upto a stress level of 40 kN/m^2 (Figure 12). The $C\alpha$ values of the electrically treated sample have not reduced beyond a stress level of 40 kN/m^2. The $C\alpha$ values of the sample preloaded to 40 kN/m^2 were much smaller than those of the electrically treated samples, at the same stress levels.

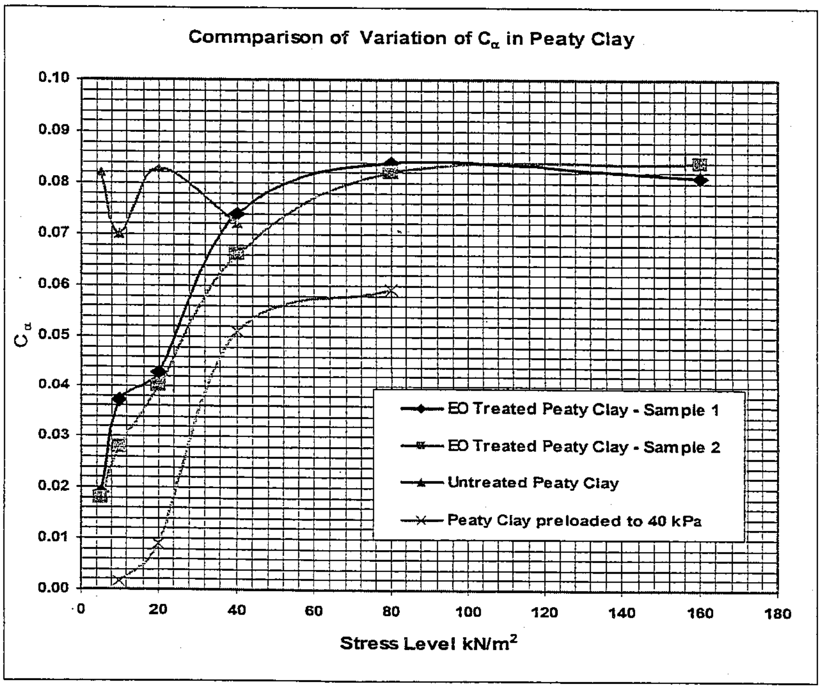


Figure 12 - Comparison of C_α plots in Peaty clay

9. Conclusions

Improvement of the engineering properties of peaty clay, effectively and economically within a relatively short period of time is a major challenge faced by the Sri Lankan geotechnical engineers involved in number of infrastructure development projects such as highways. Techniques of preloading and preloading with vertical drains have been applied in the field on successfully on several occasions.

However, with extremely soft peaty clays the required preload will have to be placed in several stages. In some cases it would not be possible to place any fill due to the extremely soft water like consistency of the soft peaty clay. Getextiles and geogrids would be required under those circumstances.

Using electro osmotic treatment, the consolidation and the improvement of shear strength of the extremely soft peaty clays could be effected without the application of a physical load. In this background, the process of electro osmotic consolidation offers number of advantages. The basic electro kinetic parameters of the peaty clay such as electro osmotic permeability, electro osmotic efficiency and the results of the laboratory simulations have clearly indicated that extremely soft Sri Lankan peaty clays could be improved by the technique.

Specialised machinery will not be needed to drive the electrodes into the soft peaty clay in a desired grid arrangement. A modified light crane can be used for the said purpose. The necessary power could be provided with a DC generator. Furthermore, in most of the sites the Peaty clay is present at the surface level and there would be no loss of current and voltage through short circuiting in a surface layer.

Owing to the lower conductivity of Peaty clay, it was necessary to use high voltage gradients in some situations to get the water drained out. However, the lower conductivity kept the power consumption at values comparable with Kaolin. For the electro osmotic strengthening of Peaty clay, EKG materials will offer a special advantage as the process could be continued for a longer duration at high voltage gradient, without any evidence of deterioration of the electrodes. The stainless steel electrodes used in previous studies corroded quickly at high voltage gradients.

Research reported in this paper clearly shows that under laboratory conditions, the strength and stiffness of the extremely soft Sri Lankan peaty clays can be significantly improved by the process of electro osmotic consolidation. The next stage of the study should be to conduct a well instrumented field study. Aspects such as voltage losses, short circuiting etc. may be more dominant in the field. Currently, further laboratory studies are conducted to decide on an optimal drain arrangements and polarity reversal.

In a field application, electro osmotic consolidation need not be continued for a long time. Sufficient improvements in undrained strength of the extremely softy peaty clay may be achieved within a short period so that the conventional filling operations can take over.

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CONSTRUCTION WITH GEOSYNTHETICS IN DIFFICULT GROUND CONDITIONS

G P Karunaratne

1. INTRODUCTION

An exciting new chapter in engineered materials has emerged since 1970s for the civil engineering community, and the rapidity at which the related products are being developed and used is nothing short of amazing.

- They are quality control manufactured in a factory environment.
- They can be installed rapidly.
- They generally replace raw material resources.
- They generally replace difficult designs using natural materials.
- Their timing is very appropriate.
- Their use is required by law in some cases.
- They are generally cost competitive against the natural soils that they replace.
- They are being actively marketed and are widely available.

The professional groups most strongly influenced are geotechnical engineering, transportation engineering, and environmental engineering, although all soil-, rock-, and groundwater-related activities fall within the general scope of the various applications. This being the case, the term *geosynthetics* seems appropriate. *Geo,*of course, refers to the earth. The realization that the materials are almost exclusively from human-made products gives the second part of the name-*synthetics*. The American Society for Testing and Materials (ASTM) has defined "geosynthetic" in D4439 Terminology as follows:

> Geosynthetic: a product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system.

The materials used in the manufacture of geosynthetics are almost entirely from the plastics industry, i.e., they are primarily polymers, although rubber, fiberglass, and natural materials (which are also polymers) are sometimes used.

2. BASIC DESCRIPTION OF GEOSYNTHETICS

The initial attempts to reinforce soils are lost to history; the adding of materials possessing properties that would enhance the behavior of the soil itself was no doubt done long before our first historical records of it. It seems reasonable to assume that first attempts were made to stabilize swamps and marshy soils using tree trunks, small bushes, and the like. These soft soils would accept the fibrous material until a mass was formed that had adequate properties for the intended purpose.

The specific families of geosynthetics are
Geotextiles, Geogrids, Geonets, Geomembranes, Geosynthetic clay liners, Geopipes, Geocomposites and "Geo-others" (an assortment of new and emerging geosynthetics)

<u>Geotextiles</u> form one of the two largest groups of geosynthetics. Their growth during the past 30 years has been nothing short of awesome. They arc indeed textiles in the traditional sense, but consist of mainly synthetic fibers and a little of natural fibres such as cotton, coconut fibre, wool, or silk, in special applications. Thus biodegradation is not a problem with the former but acceptable with the latter. These fibers are made into a flexible, porous fabric by standard weaving machinery or are matted together in a random, or nonwoven, manner. Some are also knit. The major point is that they are porous to water flow across their manufactured plane and also within their plane, but to a widely varying degree.

<u>Geogrids</u> represent second rapidly grown segment within the geosynthetics area. Rather than being a tightly woven, nonwoven, or knit textile (or even a textile-like) fabric, geogrids are plastics formed into a very open, gridlike configuration (i.e., they have large apertures). Geogrids are either stretched for improved physical properties or made on weaving machinery by unique methods.

Geonets, sometimes called geospacers, constitute another specialized segment within the geosynthetic area. They are usually formed by a continuous extrusion of parallel sets of polymeric ribs at acute angles to one another. When the ribs are opened, relatively large apertures are formed into a netlike configuration. Their design function is completely within the drainage area, where they have been used to convey fluids of all types.

Geomembranes represent the other largest group of geosynthetics, and in dollar volume their sales are probably larger than that of geotextiles. Their growth has been stimulated by governmental regulations originally enacted in 1982. The materials themselves are impervious thin sheets of rubber or plastic material used primarily for linings and covers of liquid or solid-storage facilities. Thus the primary function is always as a liquid or vapor barrier. The range of applications, however, is very great, and at least 30 individual applications in civil engineering have been developed.

Geosynthetic clay liners (or GCLs) are a new subset within geosynthetic materials. They are rolls of factory-fabricated thin layers of bentonite clay sandwiched between two geotextiles or bonded to a geomembrane. Structural integrity is maintained by needle punching, stitching, or adhesive bonding. They are used as a composite component beneath a geomembrane or by themselves as primary or secondary liners. GCLs used as hydraulic barriers

Perhaps the original geosynthetic material still available today is buried plastic pipe. This is used in all aspects of geotechnical, transportation, and environmental engineering with little design and testing awareness. This is probably due to a general lack of formalized training. The critical nature of leachate collection pipes coupled with high compressive loads makes geopipe a bonafide member of the geosynthetics family. The function is clearly drainage.

A geocomposite consists of a combination of geotextile and geonet; or geogrid and geomembrane; or geotextile, geogrid, and geomembrane; or anyone of these four geosynthetic materials with another material (e.g., various soils, deformed plastic sheets, steel cables, or steel anchors). This exciting area brings out the best creative efforts of the engineer, manufacturer, and contractor. The application areas are numerous and growing steadily. The major functions encompass the entire range of functions listed for geosynthetics discussed previously: separation, reinforcement, filtration, drainage, and liquid barrier. These composite functions, tied intimately to their respective application areas.

Geo-Others: The general area of geosynthetics has exhibited such innovation that many systems defy categorization. For want of a better phrase, geo-others describes items such as threaded soil masses, polymeric anchors, and encapsulated soil cells. As with geocomposites, their primary function is product dependent and can be any of the five major functions of geosynthetics.

3 POLYMERIC MATERIALS

The vast majority (well over 95%) of the materials are made from synthetic polymers.

The molecular weight of the polymer is the degree of polymerization (ie., the number of times a repeating unit occurs) times the molecular weight of the repeating unit. The average molecular weight and its statistical distribution are very important in the resulting behavior of the polymer, since

1. Increasing *average* molecular weight results in increased textile strength, elongation, impact strength, stress crack resistance, heat resistance, and decreased flow behavior, and process ability , while
2. Narrowing the molecular weight *distribution* results in increased impact strength, and decreased stress crack resistance, flow behavior, and processability.

While most of the polymers used in the manufacture of geosynthetics are from one type of monomer, and thus are called homopolymers, there are other possibilities. A polymer made from two repeating units in its chain is called a copolymer. The manner of linking or joining the repeating units is important, and can be random, alternating, block, or branch (graft).

4 FUNCTIONS, PROPERTIES and APPLICATIONS

Geosynthetics have six primary functions:

(a) filtration, (b) drainage, (c) separation, (d) reinforcement, (e)fluid barrier and (f) protection

In function - protection, the geosynthetic acts as a stress relief layer. Temporary geosynthetic blankets and permanent geosynthetic mats are placed over the soil to reduce erosion caused by rainfall impact and water flow shear stress. A protective cushion of non-woven geotextiles is often used to prevent puncture of geomembranes (by reducing point stresses) from stones in the adjacent soil or drainage aggregate during installation and while in service. Erosion control Geotextiles in Riprap Revetments and other Permanent Erosion Control Systems, and Temporary Runoff and Sediment Control can also be addressed with geosynthetics.

In addition to the primary function, geosynthetics usually perform one or more secondary functions. The primary and secondary functions make up the total contribution of the geosynthetic to a particular application. It is important to consider both the primary and secondary functions in the design computations and specifications. Performance tests are not normally used in specifications; rather, geosynthetics should be pre-selected for performance testing based on index values, or performance test results should be correlated to index values for use in specifications.

Brief descriptions of some of the basic properties of geosynthetics as presented below:

<u>Mass per Unit Area</u>: The unit weight of a geosynthetic is measured in terms of area as opposed to volume due to variations in thickness under normal stress. This property is mainly used to identify materials.

<u>Thickness</u>: Thickness is not usually required for geotextiles except in permeability-flow calculations. It is used as a primary identifier for geomembranes. When needed, it must be measured under a specified normal stress of 2 kPa for geotexti1es and 20 kPa for geogrids and geomembranes.

<u>Tensile Strength</u>: To understand the load-strain characteristics, it is important to consider the complete load-strain curve, the nature of the test and the testing environment. With most materials, it is usual to use stress in strength and modulus determination. However, because of the thin, two-dimensional nature of geosynthetics, it is awkward to use stress. Therefore, it is conventional to use force per unit length along the edge of the geosynthetics material. Then, strength and modulus have units of P/L (i.e., kN/m).

Specific geosynthetic specimen shapes and loadings all give different results.

The plane-strain test represents the loading for many applications, but because it is complicated to perform, it is not a practical test for many routine applications. Therefore, it is approximated by a strip tensile test. Since many narrow strip geosynthetic specimens neck when strained, most applications use wide, short specimens (100mm long and 200mm wide). This is called a wide strip tensile test (ASTM D4595).

Geosynthetics may have different strengths in different directions. Therefore, tests should be conducted in both principal directions.

The *<u>grab tensile test</u>* is typically used in the specification of geotextiles but is an unusual test. It is widely used and almost universally misused. The grab test may be useful in some applications, but it is difficult, if not impossible, to relate to actual strength without direct correlation tests. The grab tensile test normally uses 25 mm jaws to grip a 100 mm specimen. The strength is reported as the total force needed to cause failure - not the force per unit width. It is not clear how the force is distributed across the sample. The effects of the specimen being wider than the grips depend on the geotextile filament interaction. In non-woven geotextiles, these effects are large and in woven geotextiles, small.

The *<u>burst test</u>* is performed by applying a normal pressure (usually by compressed air) against a geosynthetic specimen clamped in a ring. The burst strength, given in pascals, is the normal stress of the geosynthetic at failure. It depends on the strength in all directions and is controlled by the minimum value. Burst strength is a function of the diameter of the test specimen; therefore, care must be used in comparing tests.

<u>Creep</u> is a time-dependent mechanical property of strain at constant load. Creep tests are most frequently performed on a wide strip specimen by applying a constant load for a sustained period. Creep tests are influenced by the same factors as tensile load-strain tests - specimen length to width ratio, temperature, moisture, lateral restraint, and confinement.

Short-term strain is strongly influenced by the geosynthetic structure. Geogrids and woven geotextiles have the least; heat-bonded geotextiles have intermediate; and needled geotextiles have the most. Longer-term creep strains are controlled by structure and polymer type. Polyester has lower creep rates than polypropylene. The *creep limit* is the most important creep characteristic. It is the load per unit width above which the geosynthetic will creep to rupture. The creep limit is controlled by the polymer and ranges from 20% to 60% of the material's ultimate strength.

<u>Friction</u>: Soil-geosynthetic and geosynthetic-geosynthetic friction are important properties. It is common to assume a soil-geotextile friction value between 2/3 and 1 of the soil angle of friction. For geogrid materials, the value approaches the full friction angle. Caution is advised for geomembranes where soil-geosynthetic friction

angle may be much lower than the soil angle of friction. For important applications, tests are justified.

The direct friction test is simple in principle, but numerous details must be considered for accurate results. Recent procedures proposed by ASTM indicate a minimum shear box size of 300 mm by 300 mm to reduce boundary effects. For many geosynthetics, the friction angle is a function of the soils on each side of the geosynthetic and the normal stress; therefore, test conditions must model the actual field conditions.

Durability Properties: Other properties that require consideration are related to durability and longevity. Exposure to ultraviolet light can degrade some geosynthetic properties. The geosynthetic polymer must be compatible with the environment chemistry. The environment should be checked for such items as high and low pH, chlorides, organics and oxidation agents such as ferruginous soils which contain Fe_2S0_3, calcareous soils, and acid sulfate soils that may deteriorate the geosynthetic in time. Other possible detrimental environmental factors include chemical solvents, diesel, and other fuels. Each geosynthetic is different in its resistance to aging and attack by chemical and biological agents. Therefore, each product must be investigated individually to determine the effects of these durability factors. The geosynthetic manufacturer should supply the results of product exposure studies, including, but not limited to, strength reduction due to aging, deterioration in ultraviolet light, chemical attack, microbiological attack, environmental stress cracking, hydrolysis, and any possible synergism between individual factors. Unless otherwise provided, AASHTO (1992) recommends that the allowable geosynthetic strength be determined with a factor of 2.0 to account for durability. AASHTO also recommends a minimum reduction factor of 1. 1 is used in all cases.

Hydraulic Properties: Hydraulic properties relate to the pore size of the geosynthetic and correspondingly its ability to retain soil particles over the life of the project while allowing water to pass. Hydraulic properties may also be affected by chemical and biological agents. Ionic deposits as well as slime growth have been known to clog filter systems (granular filters as well as geotextiles).

The ability of a geotextile to retain soil particles is directly related to its apparent opening size (AOS) which is the apparent largest hole in the geotextile. The AOS value is equal to the size of the largest particle that can effectively pass through the geotextile in a dry sieving test.

The ability of water to pass through a geotextile is determined from its hydraulic conductivity (coefficient of permeability, k), as measured in a permeability test. The flow capacity of the material can then be determined from Darcy's law. Due to the compressibility of geotextiles, the permittivity, (permeability divided by thickness), is often determined from the test and used to directly evaluate flow capacity.

The ability of water to pass through a geotextile over the life of the project is dependent on its filtration potential or its ability not to clog with soil particles. Essentially, if the finer particles of soil can pass through the geotextile, it should not clog. Effective filtration can be evaluated through relations between the geotextile's pore size distribution and the soil's grain size distribution; however, such formulations are still in the development phase. For a precise evaluation, laboratory performance testing of the proposed soil and candidate geotextile should be conducted.

5 SPECIFICATIONS

Specifications should be based on the specific geosynthetic properties required for design and installation. Standard geosynthetics may result in uneconomical or unsafe designs. To specify a particular type of geosynthetic or its equivalent can also be very misleading. As a result, the contractor may select a product that has completely different properties than intended by the designer.

For small projects, the cost of ASTM acceptance/rejection criterion testing is often a significant portion of the total project cost and may even exceed the cost of the geosynthetic itself. In such cases, a manufacturer's product certification specification requirement or an approved product list type specification may be satisfactory.

All geosynthetic specifications should include: general requirements, specific geosynthetic properties seams and overlaps, placement procedures, repairs, and acceptance and rejection criteria

General requirements include the types of geosynthetics, acceptable polymeric materials, and comments related to the stability of the material. Geosynthetic manufacturers and representatives are good sources of information on these characteristics. Other items that should be specified in this section are instructions on storage and handling so products can be protected from ultraviolet exposure, dust, mud, or any other elements that may affect performance. Guidelines concerning on-site storage and handling of geotextiles are contained in ASTM D 4873, Standard Guide for Identification, Storage, and Handling of Geotextiles. If pertinent, roll weight and dimensions may also be specified. Finally, certification requirements should be included in this section.

Specific geosynthetic physical, index, and performance properties as required by the design must be listed. Properties should be given in terms of minimum (or maximum) average roll values (MARVs) along with the required test methods. MARVs are simply the smallest (or largest) anticipated average value that would be obtained for any roll tested. This average property value must exceed the minimum (or be less than the maximum) value specified for that property based on a particular test. Ordinarily it is possible to obtain a manufacturer's certification for MARVs.

If performance tests have been conducted as part of the design, a list of approved products could be provided. The language *or equal* and *or equivalent* should be avoided within the specification, unless equivalency is spelled out in terms of the index properties and the performance criteria that were required to be included on the approved list. Approved lists can also be developed based on experience with recurring application conditions. Once an approved list has been established, new geosynthetics can be added as they are approved. Manufacturers' samples should be periodically obtained so they can be examined alongside the original tested specimens to verify whether the manufacturing process has changed since the product was approved. Development of an approved list program will take considerable initial effort, but once established, it provides a simple, convenient method of specifying geosynthetics with confidence.

6 APPLICATIONS IN DIFFICULT GROUNDS

There are many applications in difficult ground conditions, some of which are to increase the bearing capacity and the others to limit settlement or accelerate settlement. Figures 1 to 13 show how several difficult situation are addressed with the incorporation of geosynthetics. Important of these situations are those related to highways and railway constructions.

Both locations and expansion of available space for roadways can be achieved easily and economically with geosynthetics. Walls or slopes can be built and repaired with them without using linear foundations as for reinforced concrete walls. Since reinforced soil-structures can take considerably large settlement, they are more adaptable than masonry or concrete structures on soft ground or relatively unstable terrains.

In soft ground conditions, water occurs in excess amounts, which needs to be expelled prior to building a stable highway embankment. Perhaps prefabricated vertical drains (PVD or band drains) may be needed if soft clay exists in the ground. PVDs are another geosynthetic product manufactured normally with a plastic core wrapped around with a non-woven plastic filter. They enable excess water pressure to dissipate in the soft soil thereby making it stiff.

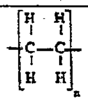
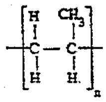
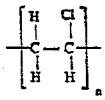
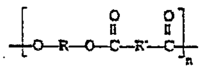
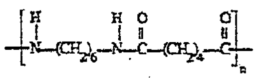
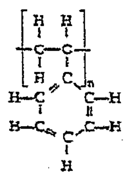
7 CONCLUSIONS

Geosynthetics have many advantages in construction on difficult ground conditions. Important aspects of using them include identifying their properties, behaviour and proper application in the individual designs. Advantages of their use include cost saving, speed of construction and durability over several other types of materials. It is also important for designers to keep the limitations in mind. Among the latter are damage during installation, effect of UV and sun light in weakening the original properties such as strength and permeability.

8 ACKNOWLEDGEMENTS

The author wishes to acknowledge, with gratitude and appreciation, consultation of text books of the following eminent geotechnical and geosynthetics professors and engineers with whom he had many valuable discussions and associations in the development of geosynthetic technology, research, conferences and publications during the last two decades: Robert M. Koerner (Drexel University, Philadelphia, Penn, USA), Robert D. Holtz (University of Washington, Seattle, USA), Barry R. Christopher, Ryan R. Berg and Colin J. F. P. Jones (University of Newcastle, UK).

Table 1. Polymers in the manufacture of geosynthetics

Polymer	*Repeating Unit*	*Types of Geosynthetics*	
Polyethylene (PE)		•Geotextiles •Geogrids •Geonets	•Geomembranes •Geopipe •Geocomposites
Polypropylene (PP)		•Geotextiles •Geogrids	•Geomembranes •Geocomposites
Polyvinyl chloride (PVC)		•Geomembranes •Geopipe	•Geocomposites
Polyester (polyethylene terephthalate) PET		•Geotextiles	•Geogrids
Polyamide PA (nylon 6/6)		•Geotextiles •Geogrids	Geocomposites
Polystyrene (PS)		•Geocomposites	

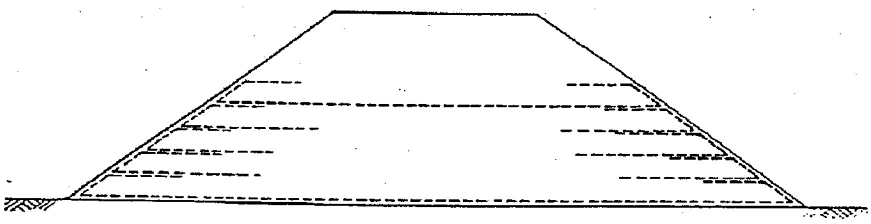


Figure 1. Geosynthetics as stabilizing reinforcement for an embankment on soft ground

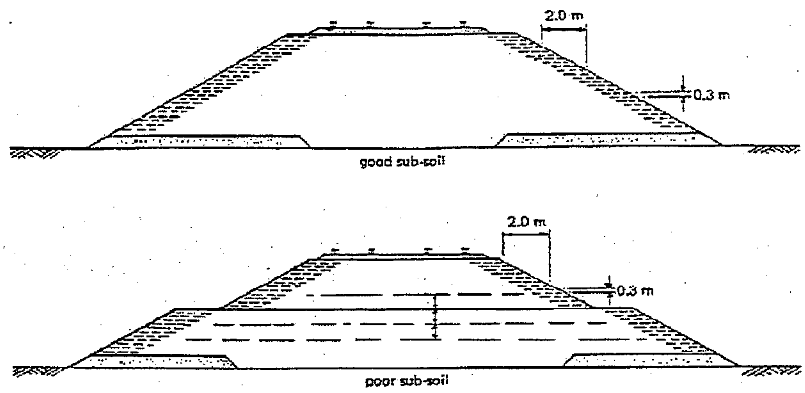


Figure 2. Railway embankments, (Japanese National Railway) in which geosynthetics are used for erosion control and stabilizing slopes, as well as base reinforcement

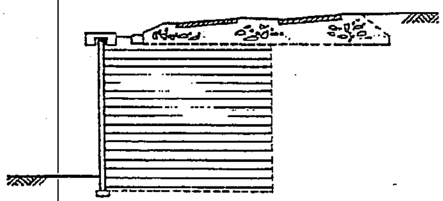


Figure 3. Railway line on elevated reinforced soil walls

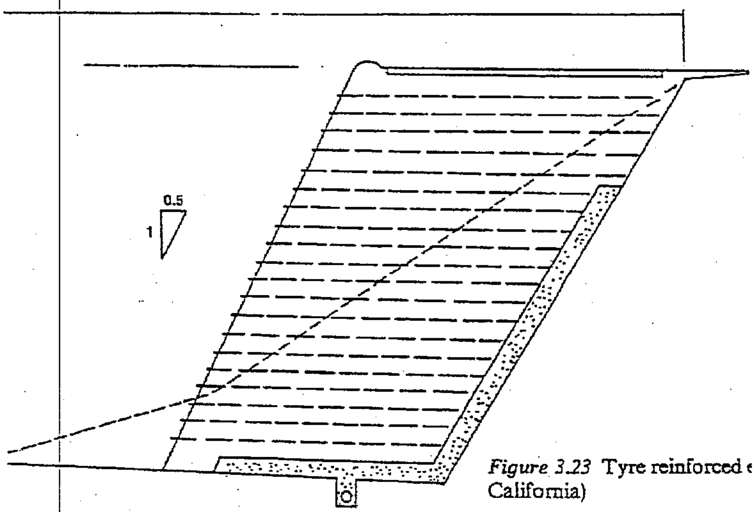


Figure 4. Tyre –reinforced embankment slope in California

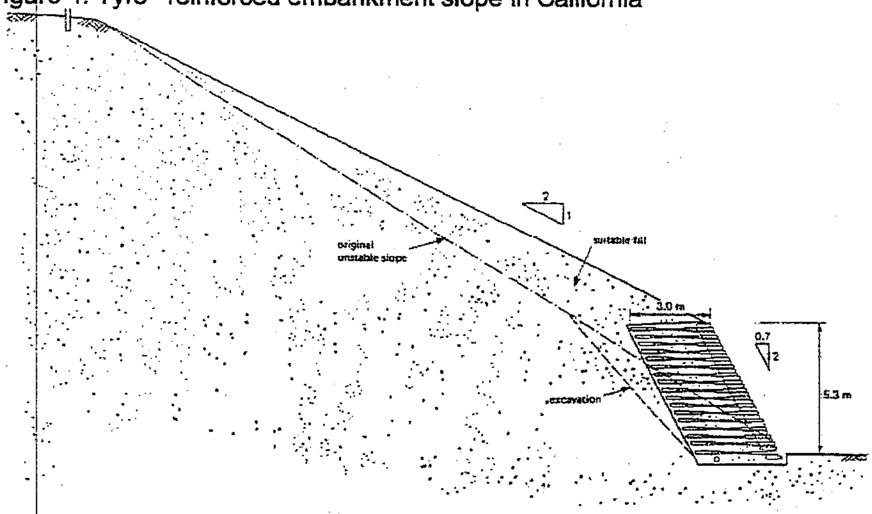


Figure 5. Tyre wall repair of slope failure

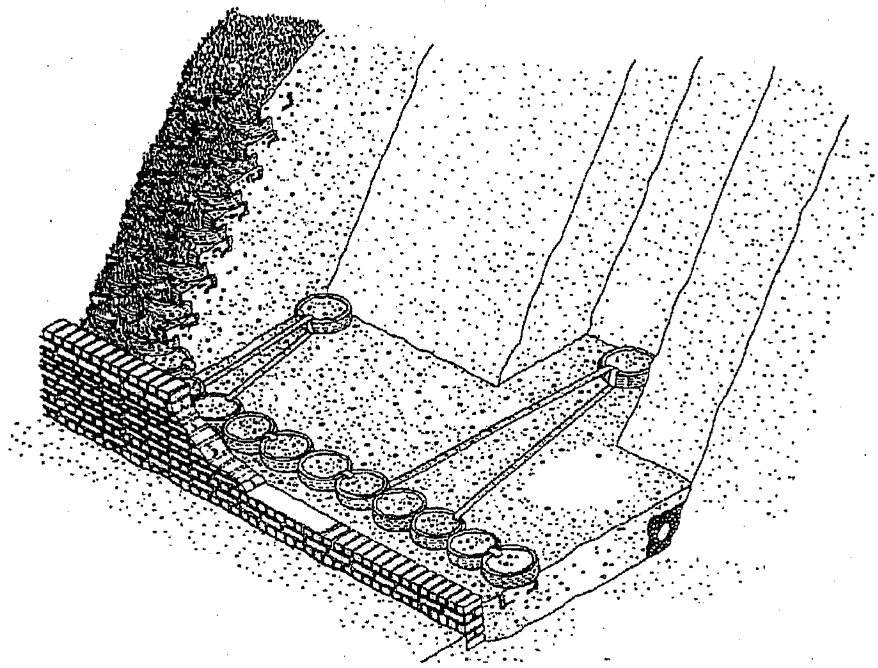


Figure 6. Tyre /geotextile composite

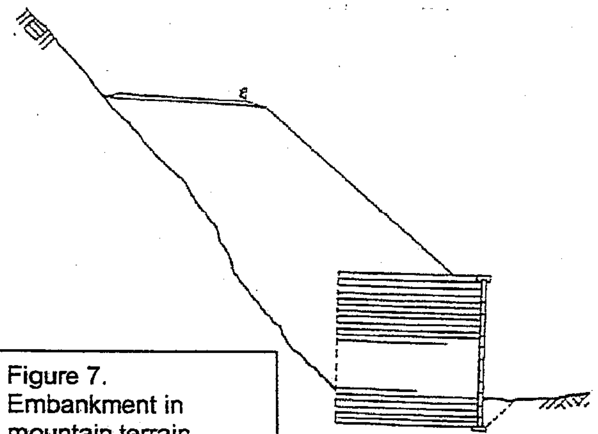


Figure 7.
Embankment in
mountain terrain

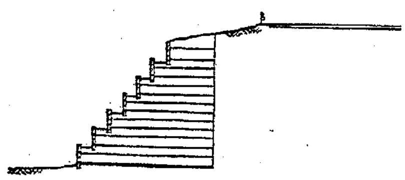


Figure 8. Stepped embankment

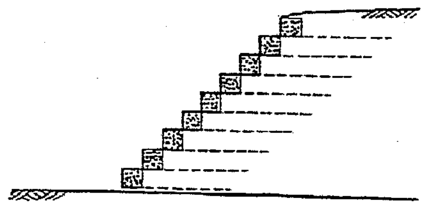


Figure 9. Gabion faced reinforced soil structure

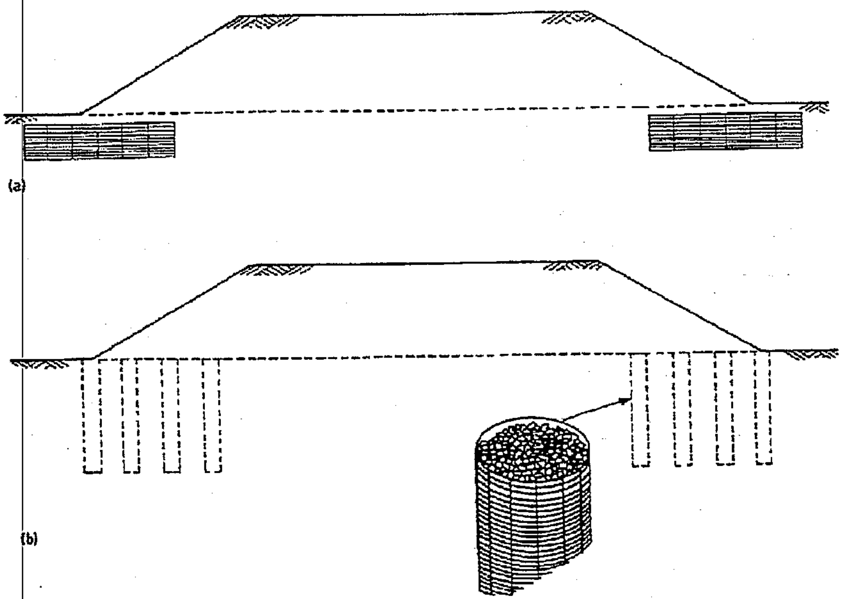
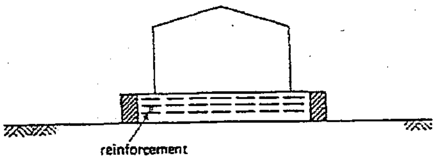
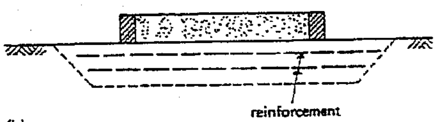


Figure 10. Geogrid reinforcement (a) and stone column formed geogrid tubes (b) beneath embankment



(a)

Figure 3.16(a) Reinforced foundation



(b)

Figure 11. Reinforced foundations

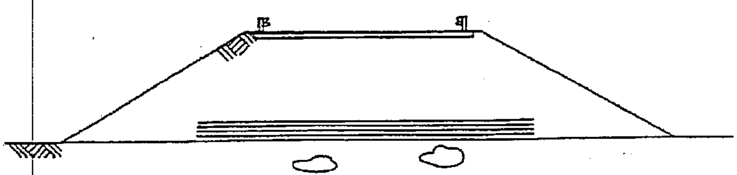


Figure 12. Reinforced slab over cavities

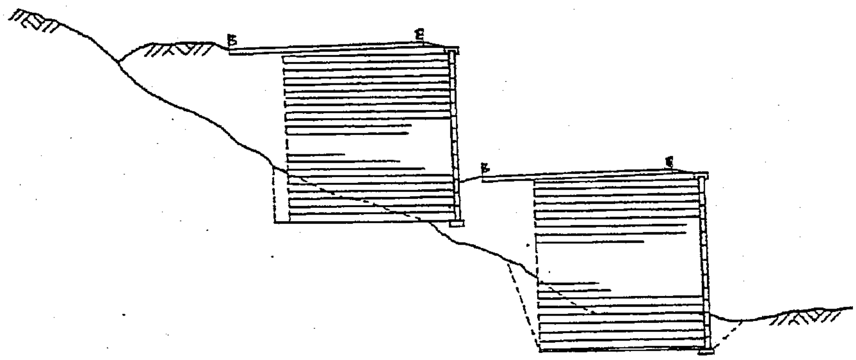
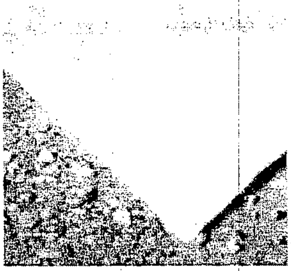


Figure 13. Stepped highway structures

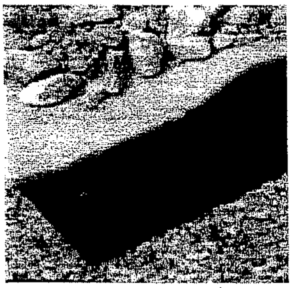
TENCATE

Polyfelt



Polyfelt TS Non Woven Geotextiles

- High mechanical properties due to endless fibre manufacturing process
- High water flow and good soil retention property due to special needling technique
- High UV stability and durability

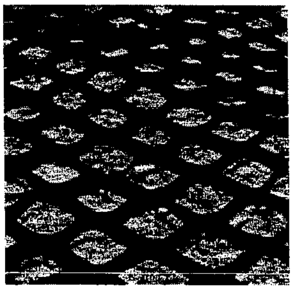


Enviromat GCL / Heavy Duty Polyfelt TS Geotextiles / Geonets / Megadrain

- High puncture resistance geotextile for geomembrane protection
- Low permeability GCL as primary and secondary liner system
- High chemical and UV resistance
- High permeability drainage of leachate and rainwater run off

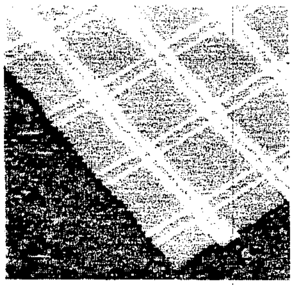
Polyfelt PEC / GX Geogrids / WX

- Suitable for granular and poor draining soils
- High tensile modulus at low strain
- Low tensile creep property for design > 100 years
- Quality product - British Board of Agrement (BBA) certified



Envirocell / Polymat EM Series

- Retention of soil against steep slopes
- Soil erosion protection
- Enhance vegetation growth
- Vegetation on hard surface



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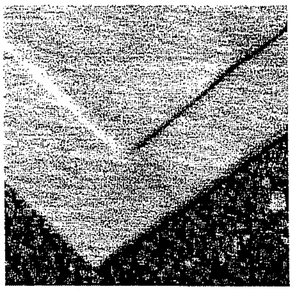
- Excellent sealing property for permanent adhesion of new pavement
- High tensile strength for pavement reinforcement



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STABILIZATION OF WASTE PHOSPHATIC CLAY SETTLEMENT AREAS USING *PHOSCRETE*

By
Catalina Carvajal, Kalyani Jeyisanker and M.Gunaratne
Department of Civil and Environmental Engineering
University of South Florida
Tampa, Florida, USA

Abstract

Florida (USA) is one of the largest phosphate producing areas in the world and hence phosphate mining is a principal industry in Central Florida. The phosphate extraction process has produced thousands of acres of phosphatic waste during the past few decades creating vast areas of highly compressible clays. Several stabilization methods have been used on phosphatic clays including vertical drains with surcharges, lime-columns, replacement and other mixing methods. This paper describes an innovative method that uses a mixture of quicklime and other pozzolans called *Phoscrete* to stabilize phosphatic clay at a desired location. This is a relatively rapid and cost-effective method compared to deep foundations, excavating and replacing, and surcharges. The study involved three investigative phases; 1) laboratory investigation of compressibility and shear strength properties of phosphatic caly, 2) numerical investigation of the optimal field implementation based on the finite element method, and 3) field settlement monitoring of a *phoscrete* treated clay site and a control site. The results certainly indicate the effectiveness of the new stabilization method.

Introduction

The primary use of phosphate is in agriculture as a fertilizer. In order to obtain a water-soluble fertilizer, phosphate is extracted from sands and clays. This process that uses large amounts of water produces approximately 1 part of phosphate for 2 parts of sand and 1 part of clay. After the phosphate is extracted, the remaining waste sand is used in reclamation of the mining areas while the waste clay suspended in water is pumped into settling areas across Central Florida. This industry had produced more than 100,000 acres of phosphatic waste during the last decades, creating areas of highly compressible clays.

The objective of this research is to evaluate effectiveness of *Phoscrete* in stabilizing phosphatic clays. Three different tasks were set in order to achieve the research objectives. The first phase of the research program was laboratory testing, specifically consolidation and Triaxial tests. Pre-treated and post-treated samples were tested to compare strength and compressibility properties. Secondly, a computer program called *GeoStudio* was used to model the settlement under different loading conditions. The results of the modeling were used to determine the type of optimal treatment configuration for different types of structures. Finally, a field demonstration was used to monitor settlement of pre-treated and post-treated clay.

Laboratory Investigation

Triaxial Testing: Figures 1(a) and 1(b) illustrate the results of triaxial tests on untreated and treated clay respectively. It is seen that the failure stress is increased significantly due to *Phoscrete* treatment.

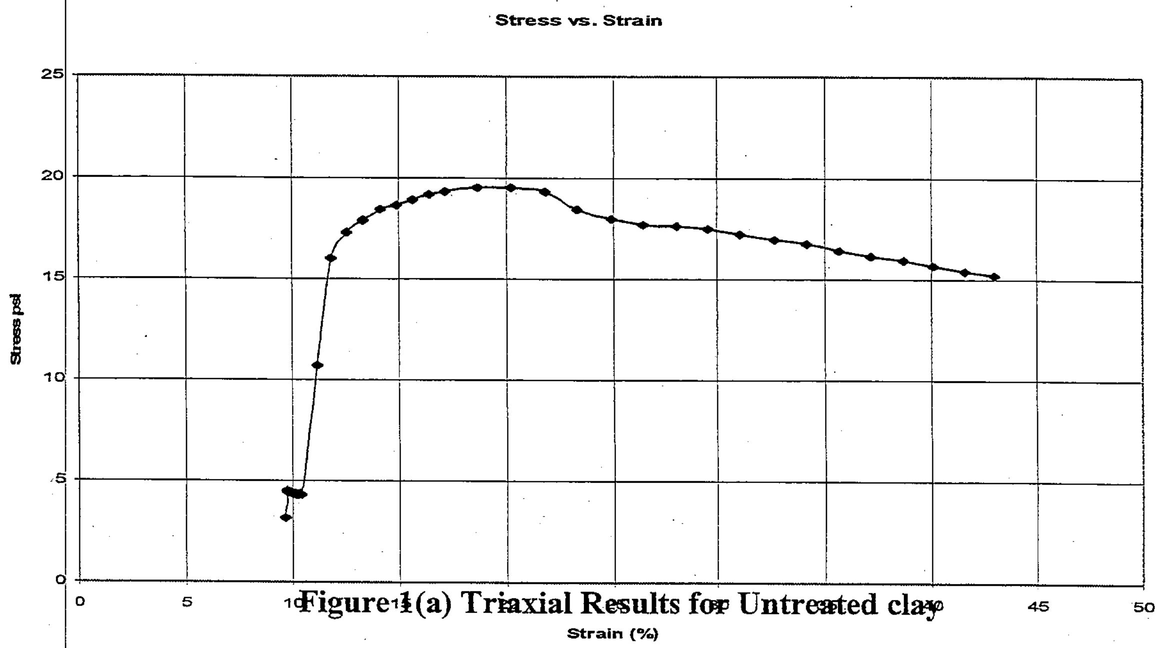


Figure 1(a) Triaxial Results for Untreated clay

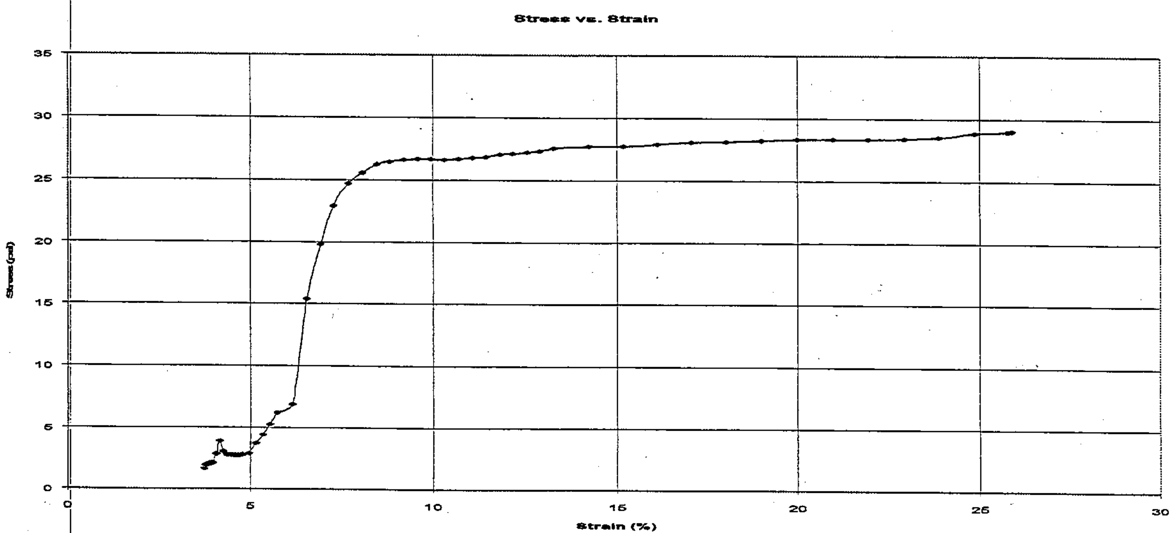


Figure 1(b) Triaxial Results for Treated clay

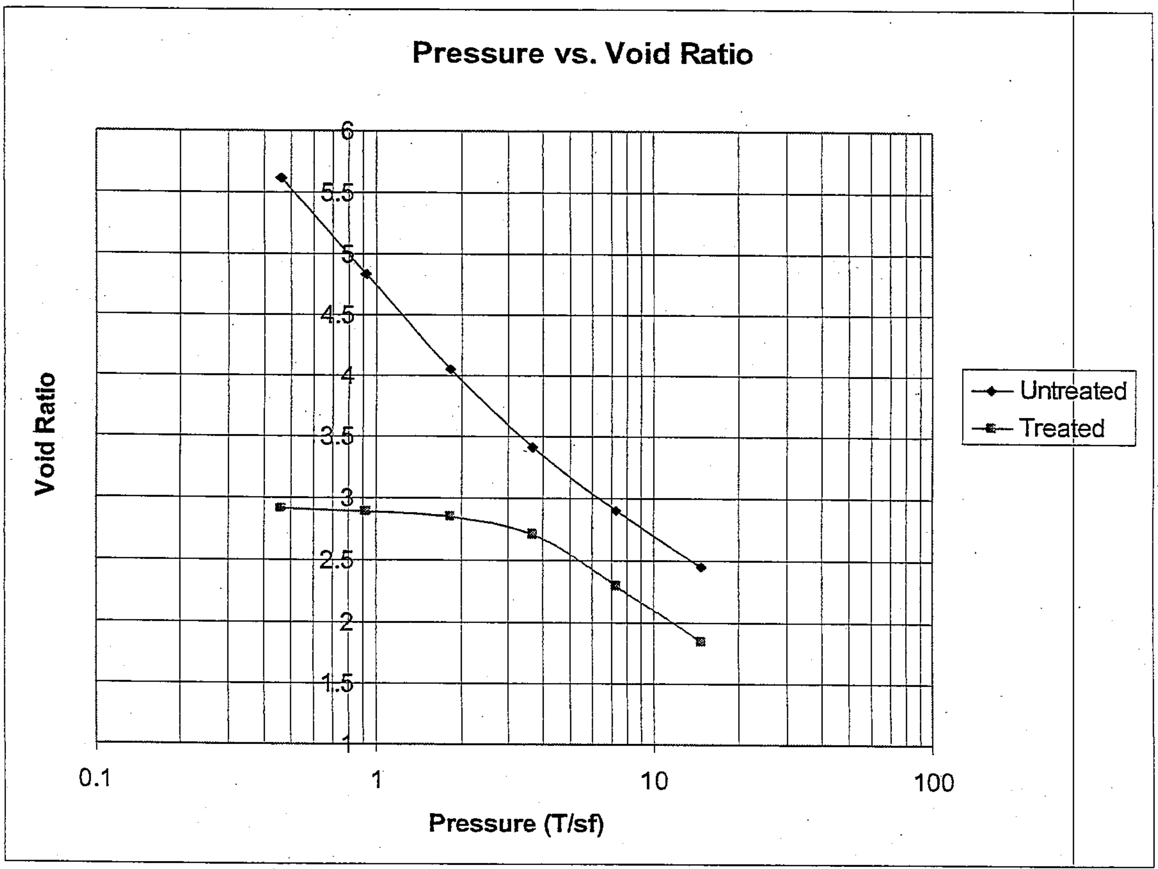


Figure 2 Comparison of the laboratory consolidation curves for treated and untreated phsophatic clay

Consolidation Testing: Figure 2 shows that *phoscrete* treated phosphatic clays clearly show the cementation effects introduced by the chemical bonding that takes place among the *phoscrete* constituents and the phosphatic clay. Fig. 2 also shows that although this cementation reduces the settlements under relatively small loads, the treated clay behaves very much like the untreated clay once bonding is destroyed. The laboratory testing results are summarized in Table 1.

Table 1 Summary of Laboratory Test Results

Soil Property	Elastic modulus (psi)	Cohesion (psf)	Angle of friction (deg)	Compression index
Untreated	700	500	15	0.2
Treated	1,000	720	20	0.03/0.2

Numerical Modeling

The Sigma/W component of the Geostudio finite element program was used to model the field implementation of the *Phoscrete* treatment process. For this purpose, the treated and untreated clay properties summarized in Table 1 were used. While Fig. 3 shows the general finite element modeling configuration, the results of modeling are illustrated in Figures 4-6.

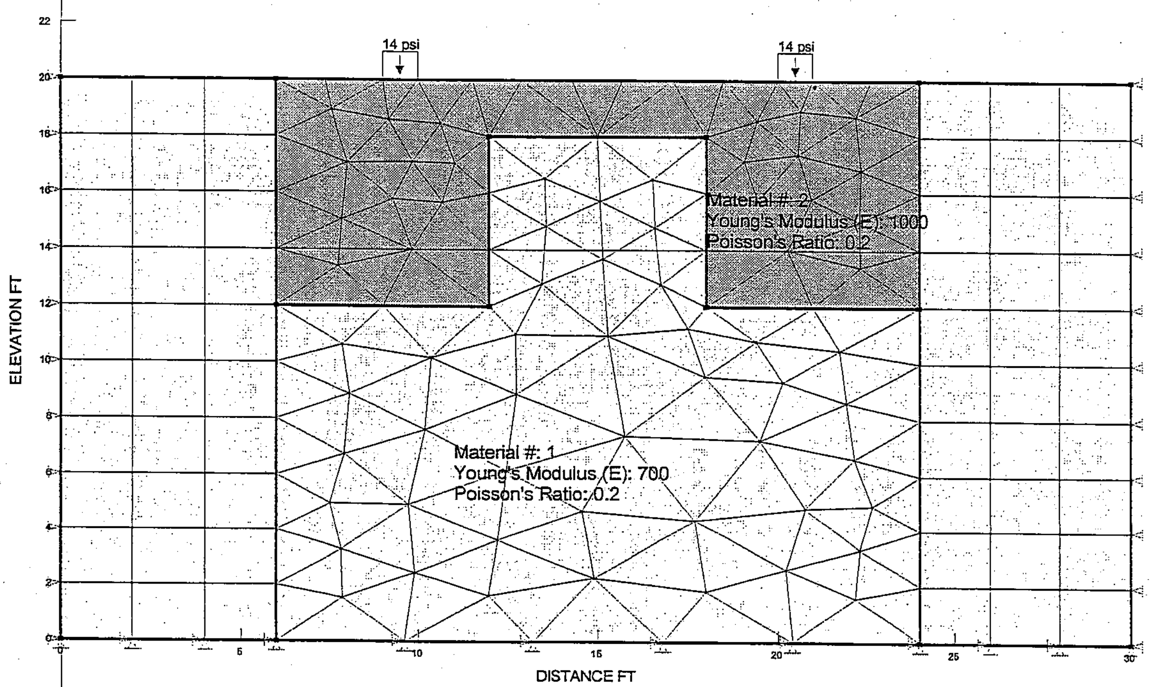


Figure 3 General Finite Element Configuration

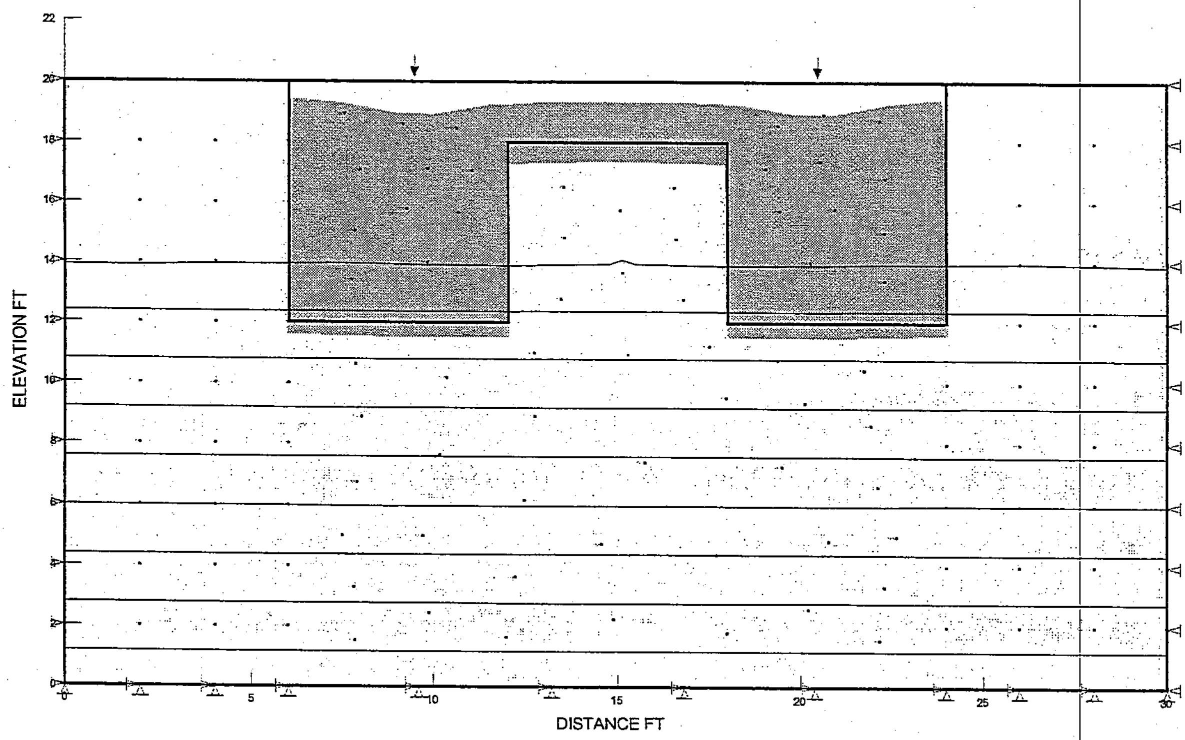


Figure 4 Deformed Finite Element Configuration

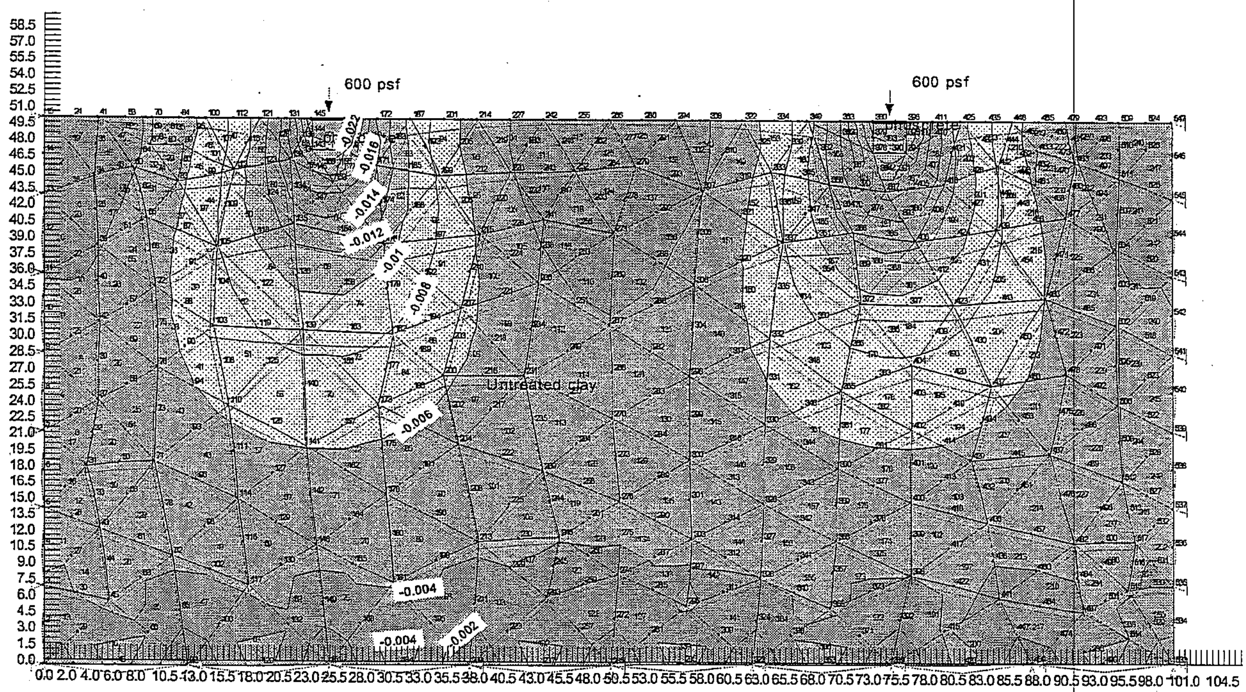


Figure 4 Contours of strain

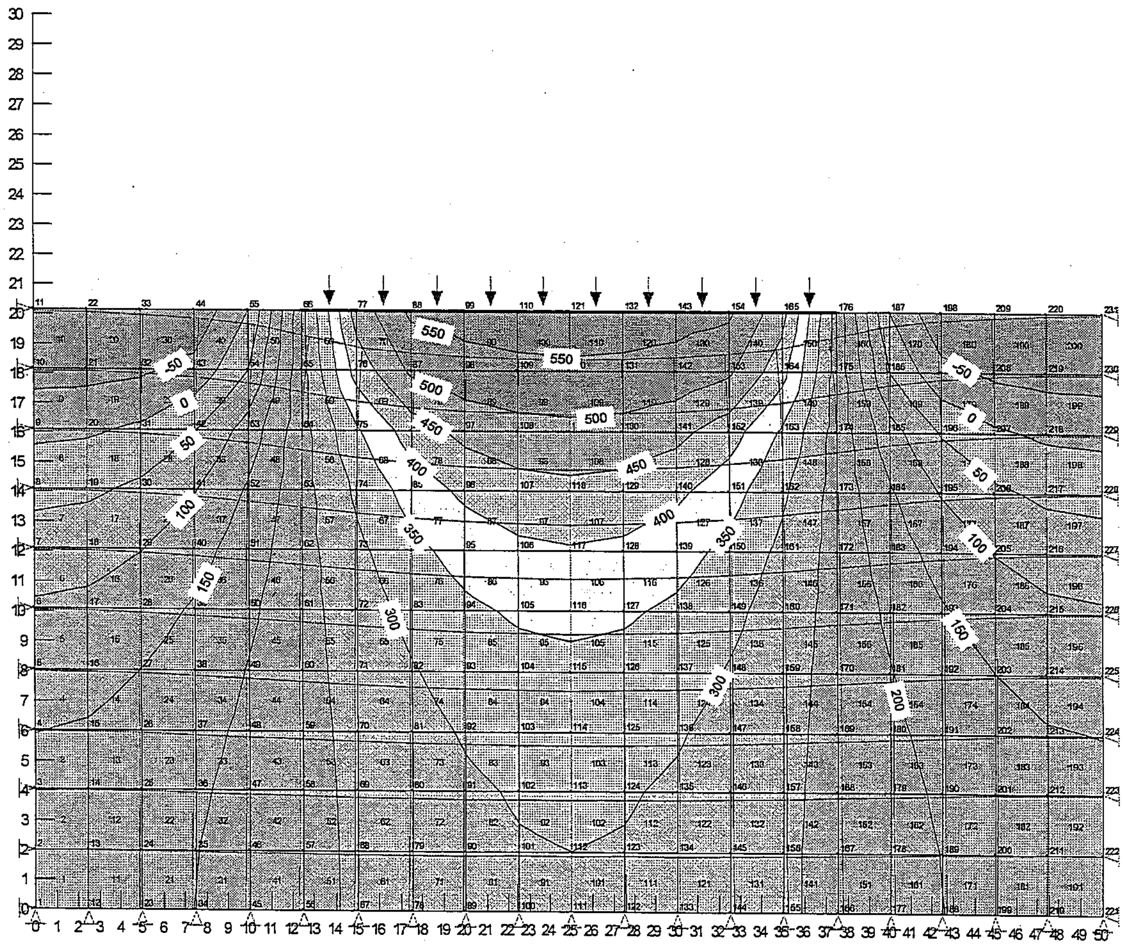


Figure 6 Pore water pressure distribution at the end of elastic settlement

It can be seen how one can determine the magnitude of the elastic settlement of the treated soil under the given loading condition based on the strain contours shown in Figure 5. On the other hand the Sigma/W program also predicts the pore pressure induced at any point based on the Skempton's pore pressure parameters determined from trixial testing. The authors used the above pore pressures as an effective stress increase induced in the influence area of the foundation and a trial pseudo-elastic modulus to predict the settlement caused by the dissipation of pore pressures, i.e. consolidation settlement. The appropriate modulus was selected in this exercise by calibrating the settlement against the settlement computed from the conventional consolidation equation as seen in Fig. 7.

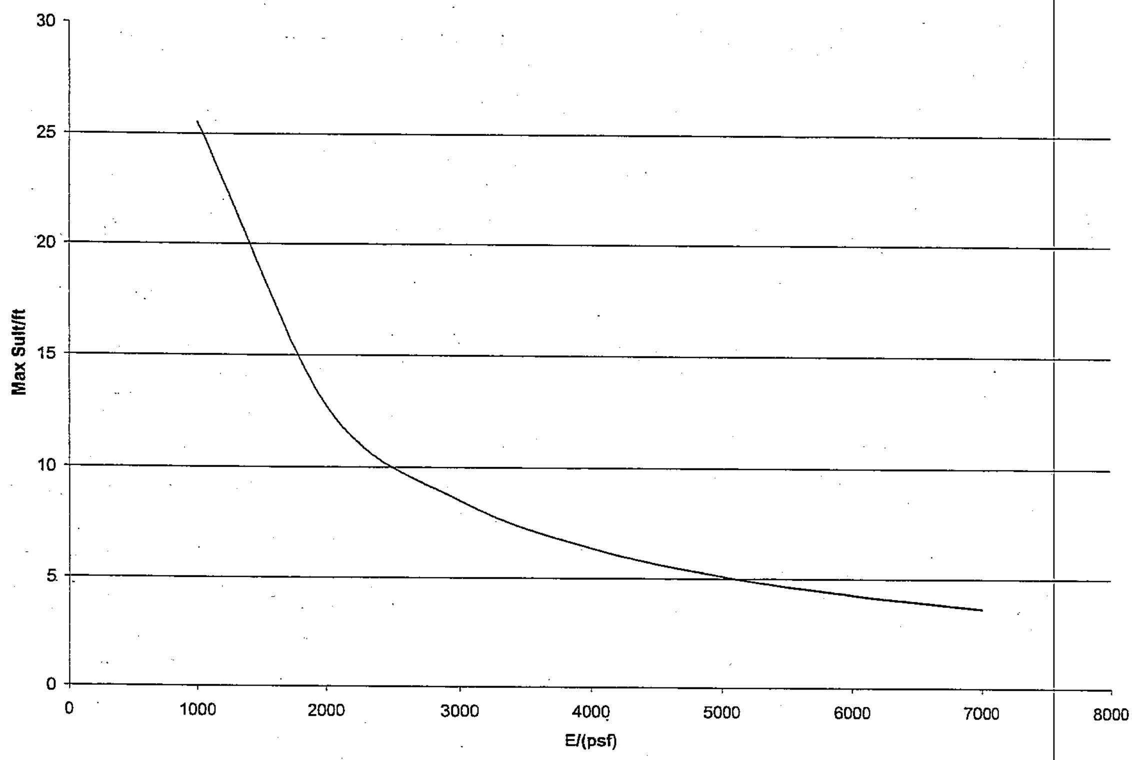


Figure 7 Plot of S_{ult} versus E

Field Testing

The general field *phoscrete* treatment configuration is indicated in Figure 8. A 20 ft x 20 ft area stabilized as shown in Fig. 8 and a control site of similar size were loaded with a 5 foot fill and the settlement were monitored using settlement plates.

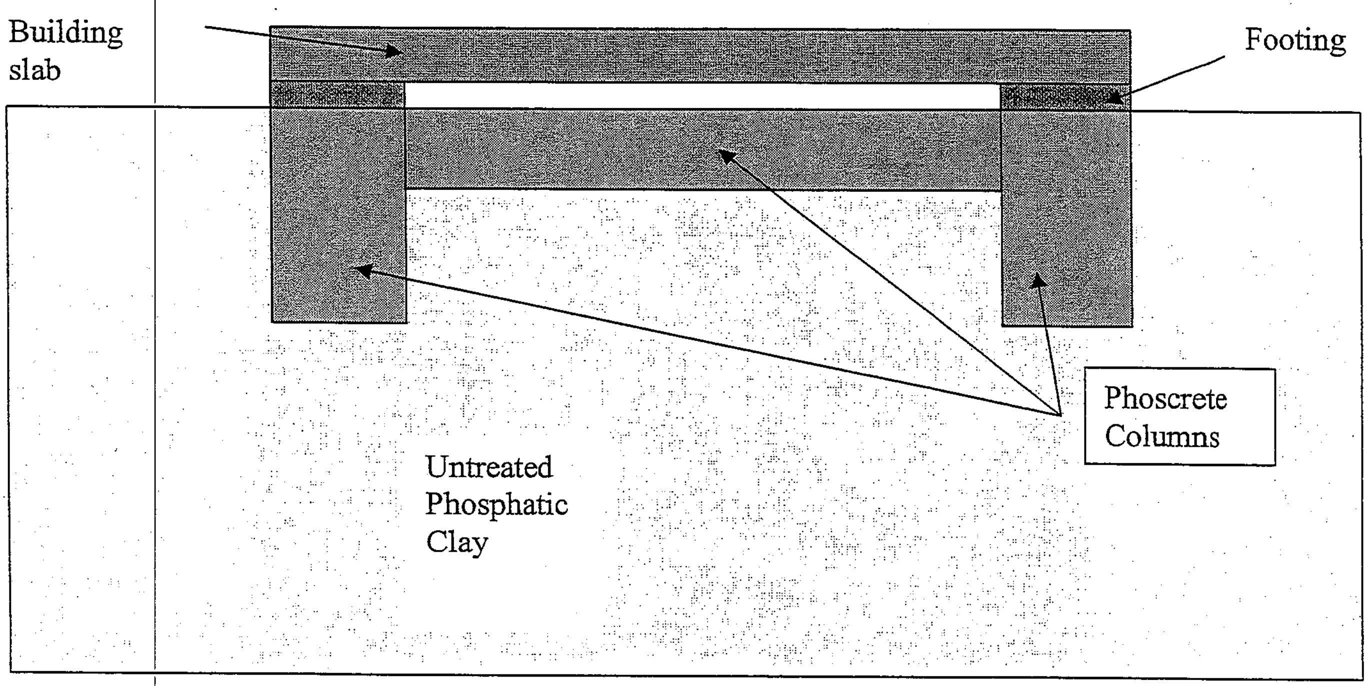


Figure 9 illustrates the settlement pattern for the two areas where it is seen that *phoscrete* treatment has been effective in reducing the settlement of phosphatic clay.

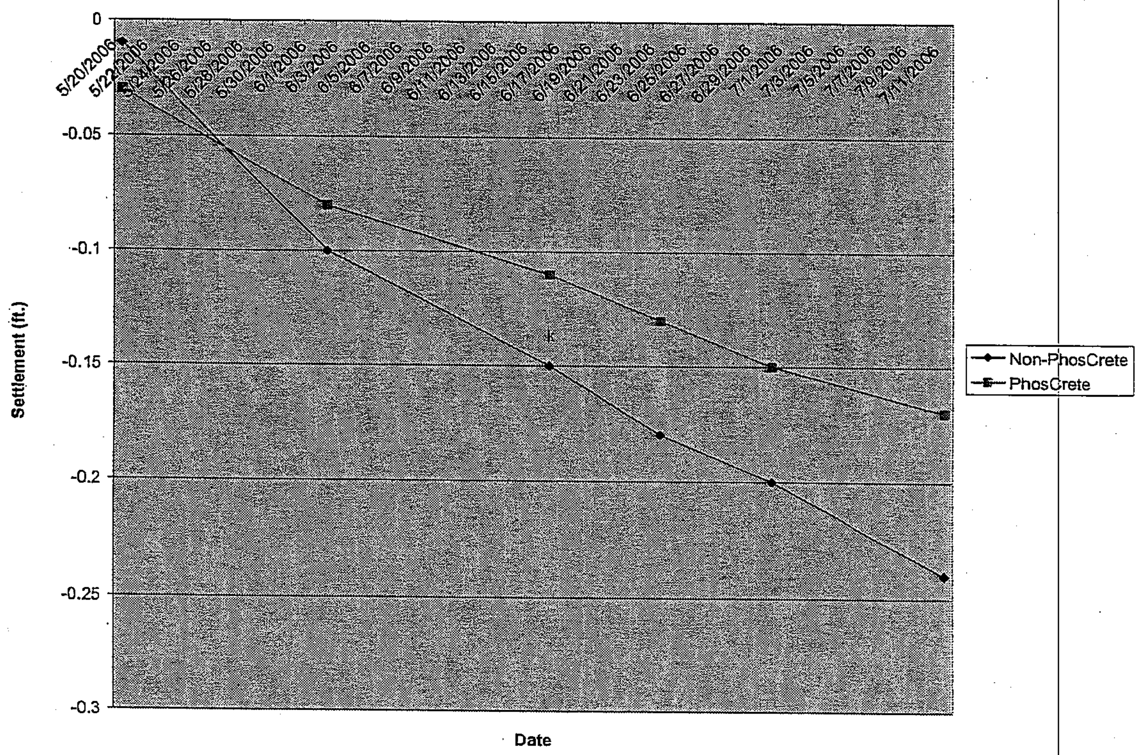


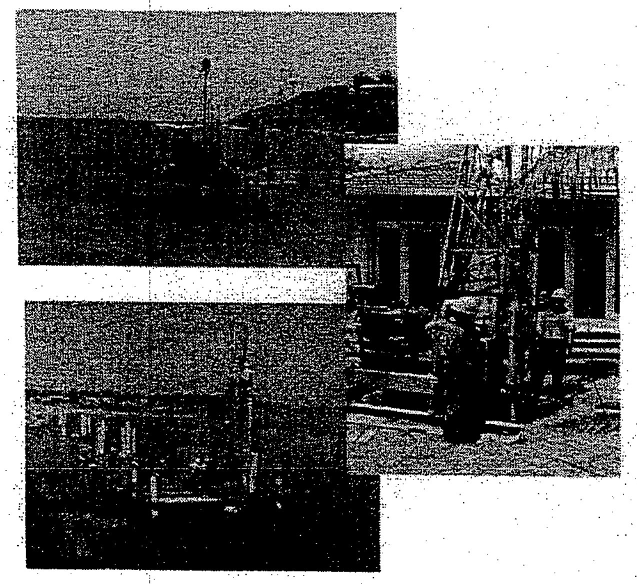
Figure 9 Comparison of field settlements at the treated and the Untreated Sites

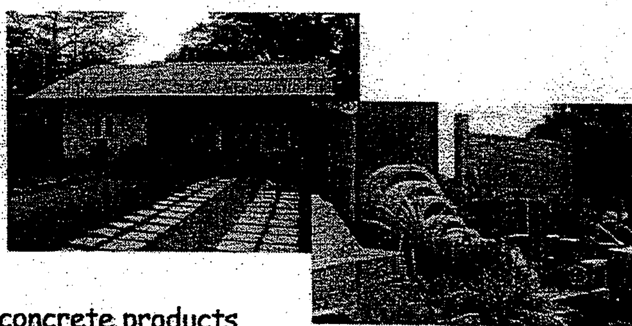
Conclusion

The effectiveness of *phoscrete* as a stabilization method of phosphatic clay was demonstrated. The preliminary laboratory testing program indicated that strength, elastic compressibility and long-term compressibility properties of this clay are improved significantly as a result of *phoscrete* treatment. An existing finite element program (Geostudio) was used in determining the optimal treatment plan for a given loading situation. The effectiveness of this stabilization method was also evidenced during the subsequent field testing program. Calibration of the finite element program using the updated compressibility properties will certainly enhance its predictive ability for other desired loading scenarios.

Acknowledgment

The authors are grateful to Mr. Larry Madrid and Mr. Patrick Ford of Madrid Engineering Company, Bartow, Florida for financial support and other field testing services, respectively.





Development of Negative Skin Friction on Piles Installed through Soft Compressible Soils

By. Dr. H. S. Thilakasiri, Department of Civil Engineering, University of Moratuwa.

Abstract

Utilization of grounds with soft compressible layers near the ground surface for development projects requires the use of pile foundations to support the structural loads. Therefore, in most cases the piles are installed through the soft compressible layers, which under go consolidation settlement due to placing fills, lowering of the ground water table, pile driving etc. As a result of the consolidation of the surrounding soil, a frictional drag force commonly referred to as the 'negative skin friction' is developed on the pile shaft. The drag force is due to the skin frictional resistance between the pile and the soil layers above the consolidating layer. The concepts of the drag force, drag-down force and the neutral plane are discussed in this paper with the help of some case histories of measured negative skin friction acting on steel piles. The mechanism of development of drag force is also discussed to investigate the effect of the drag down force on the estimation of the carrying capacity of end bearing bored piles and floating pile foundations consisting precast concrete piles. Finally, the use of the new research findings in estimation of the negative skin friction in the design and testing of piles in Sri Lanka is presented.

Introduction

The load carrying capacity of piles come from skin friction and end bearing. The development of the skin friction and the associated deformation of the soil and the pile could be simply illustrated as shown in Figure 1 using a typical pile element with the corresponding soil elements in contact with it. Figure 1(a) shows the equilibrium position of the pile and the corresponding soil elements before loading the pile. Figure 1(b) shows the location of the pile and soil elements after application of an axial force of P_1.

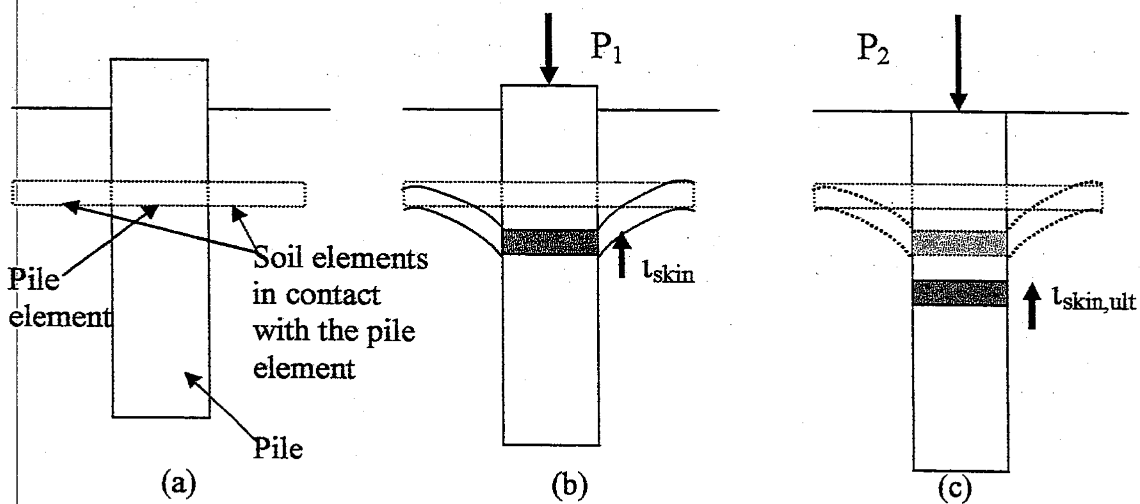


Figure 1. Development of ultimate skin friction: (a) pile at rest condition with one pile element in contact with the adjacent soil element; (ii) During loading of the pile; and (c) slipping of the pile and soil elements after development of the ultimate skin friction.

Figure 1(c) shows the location same elements after slipping between the pile and the soil. When the slipping between the pile element and the soil element occurs, the frictional resistance acting on the pile has reached the ultimate value ($\tau_{skin,\ ult}$). The shear deformation required to mobilize the ultimate skin friction is considered to be relatively small. Since the direction of the skin friction is opposite to the direction of the applied axial load, the frictional resistance is referred as skin frictional resistance (or positive friction). The skin friction is generated due to the relative deformation between the pile and the soil elements.

If the soil surrounding the pile moves in the downwards direction relative to the pile, the direction of the frictional force acting on the pile is in the downward direction. As a result, the direction of the frictional force is same as that of the applied load. Therefore, such frictional force acts as a load on the pile. This phenomenon is referred to as the negative skin friction.

The surrounding soil can move in the downward direction as a result of the consolidation settlement of the surrounding soil due to: (i) lowering of the ground water table; (ii) surcharging of a clay layer as a result of placing a fill layer; and (iii) increased pore water pressure in the vicinity of a driven pile. A portion of the pile must be fixed against the vertical movement for the development of a significant negative skin friction on the pile. If the pile moves with the consolidating soil, there is no negative skin friction developed. In Sri Lanka, bored piles are end bearing and driven piles are driven to a strong bearing layer. Therefore, significant negative skin friction can be developed. Most piling sites in Sri Lanka with a soft surface soil layer are filled prior or after installation of the piles. Such soil deposits, which are normally consolidated undergoes large amount of consolidation settlement over a period of time after placing such a fill. The time taken for the completion of the major portion of the settlement depends on the thickness and the consolidation properties of the compressible soft soil layer. Another significant feature of the ground condition in Sri Lanka is a presence of a relatively residual soil formation generally consisting of thick weathered rock layer overlying the bedrock. Therefore, a typical subsurface condition with a possibility of development of negative skin friction consists of a fill layer underlain by a normally consolidated soft soil layer followed by a residual weathered rock layer on the bedrock. A typical subsurface is shown in Figure 2.

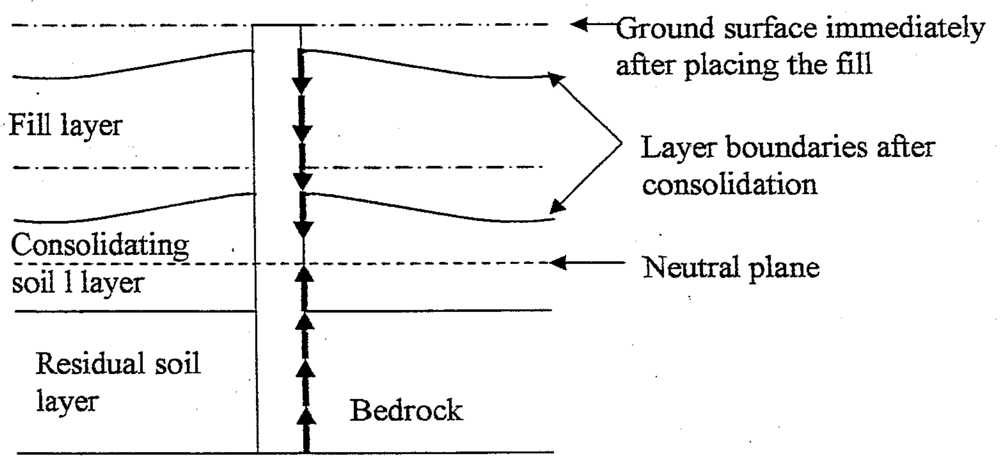


Figure 2. Typical subsurface profile and development of negative and positive skin friction on end bearing piles

The soil upto the bottom of the consolidating soft soil layer is moving in the downward direction. However, the amount of downward movement is maximum at the ground surface and zero at the bottom of the soft soil layer. If it is assumed that the pile is restrained from moving at the bottom, the downward movement of the surrounding soil generates a drag force at the top of the pile upto a certain depth where the relative movement of the surrounding soil with respect to the pile is in the downward direction. Beyond that depth, the movement of the surrounding soil is in the upward direction with respect to the pile and, therefore, positive friction is developed on the pile. The plane separating the positive and negative skin friction is referred to as the 'neutral plane'. However, if the pile is free to move at the bottom, the neutral point is also moving in the downward direction and the neutral point is established at the location where the relative downward movement of the surrounding soil with respect to the pile is zero. In such situations, the drag force becomes a dragdown force and the pile settles under the negative skin friction. Results of two case studies are presented here to support the existence of a neutral plane along the pile shaft.

Bjerrum et al. (1969) presented one of the early experiments done to confirm the development of the negative skin friction and the existence of the neutral plane. A 300mm diameter closed-toe steel pile instrumented with telltales was driven through a 20 m thick silty clay layer upto the bedrock. A 10 m thick fill was placed on top of the original ground surface and the settlement of the telltales were recorded. A cross sectional view of the site with one of the test piles are re-produced in Figure 3(a). Figure 3(b) shows the axial force, estimated from the telltale settlements, 18 months after placing the fill.

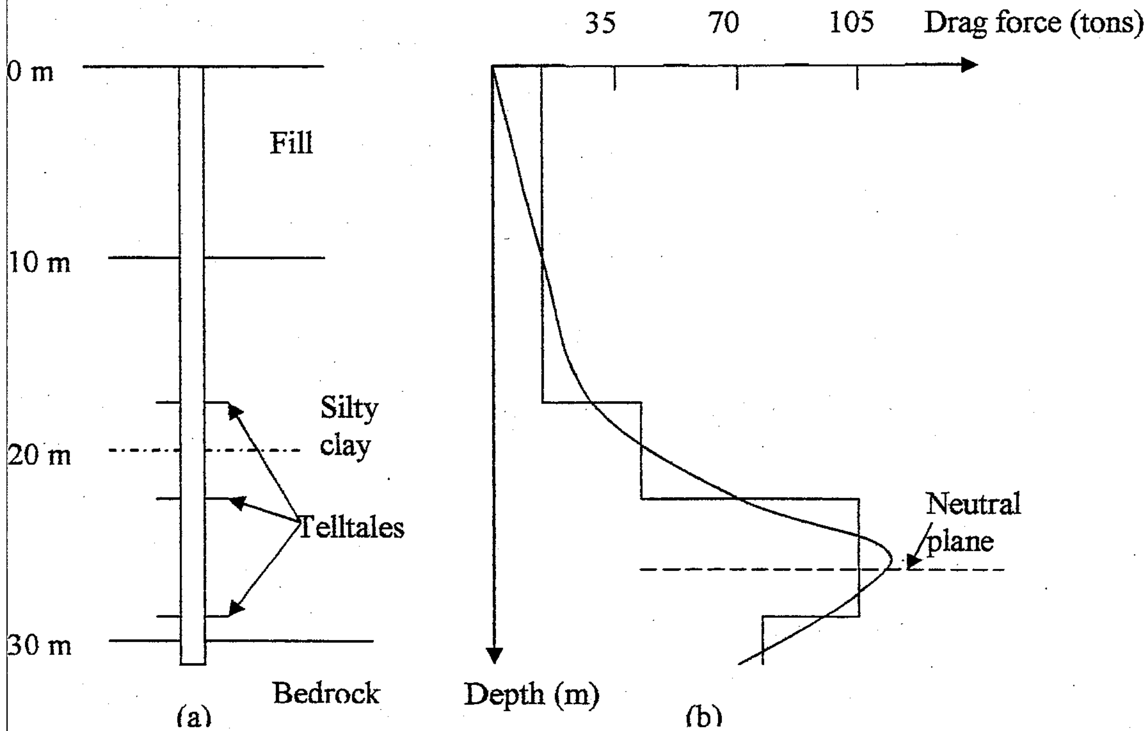


Figure 3. (a) Cross section of the subsurface with the test pile and the location of the