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CONTENTS

ERODABILITY AND DISPERSIVENESS OF SOME SRILANKAN SOILS

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This paper describes a project carried out to investigate the dispersive properties of some Sri Lankan soils. In earth dams the presence of dispersive soil may cause to high erodability leading to even complete collapse. However, still in Sri Lanka unlike in the developed countries dispersiveness of local soils is not tested before dam construction. Soils from three different local dam sites were used for the present study. The dispersiveness of the soil samples was found using pinhole test, crump test and double hydrometer test. From the study it was found that all three soil samples have non-disperive characteristics, however, they in the proximity of dispersive region in the dispersive/non-dispersive soil chart. Continuation of this work by experimental investigations further of great is local importance.

INTRODUCTION

Soils are erodible by nature and some fine grained soils" are highly erodible than others are called "*dispersive soils*. These dispersive soils erode by a process called dispersion in which the individual clay particles go into suspension. The dispersibility of a soil is defined as the susceptibility of the soil to colloidal erosion.

The existence of dispersive clays was first generally recognized by engineers after pioneering investigations of eroded clay dams in Australia in the early 1960's. More than twenty year earlier, soil scientists have known that sodium clays are responsible for many problems related to erosion of earth structures and agricultural land. These sodium rich soils are termed as "*dispersive soils*" in the soil science literature in the mid – 1930's. By 1970's studies have identified wide spread presence of dispersive clays throughout the world. In the United States, a rough criterion for identifying dispersive clays was introduced in 1973 using a laboratory test called the *Pinhole test* in which the colloidal erosion was measured directly by passing water through a hole in a compacted soil specimen.

Severe erosion damage of embankments by rainfall in the form of deep gullies has been reported by *Sherard et al* (1976). He found that certain soils in nature are highly erodible and recommended the use of four tests for identifying highly erodible soils. These four identification tests are the Soluble Salts Test, the Soil Conservation Service (SCS) Laboratory Dispersion Test, the Crumb Test, and the Pinhole Test. By these tests, a soil can be identified as being dispersive or non-dispersive. Dispersive soils are reported to be highly erodible (*Sherard et al* 1976).

The principal difference between dispersive clays and ordinary resistant clays (Non-dispersive clays) is the natural cation in the pore water. Dispersive clays have a higher content of sodium cations, whereas ordinary clays have a small amount of calcium and magnesium cations in the pore water.

The dispersion occurs when repulsive forces (*electrical surface forces*) between individual clay particles exceed the attractive forces (*Vander-walls attraction*) so that when the clay mass is in contact with water, individual clay particles are progressively detached from the surface and goes into suspension. The tendency towards dispersion is usually caused by a decreasing of the dielectric constant of hydrated ion, pH and high sodium percentage (greater than two to ten). The same soil can be made dispersive or non dispersive depending upon chemistry of eroding water, i.e. eroding water with pH less than four or greater than eleven may cause change from dispersive to non-dispersive behavior.

The use of dispersive clays in earth filled dams may cause serious problem. Unless the dispersive nature of the soil is identified and due consideration given to this fact at the design stage, problems may develop sometime after construction and the remedial measures may be extremely difficult. The problems may occur suddenly and may lead to catastrophic failures. Also many of the hydraulic structures and highways containing dispersive soils or on dispersive soils have suffered by any or combination of forms of piping, jugging, gully erosion, tunnels and surface erosion, because they disperse easily and rapidly. Therefore, the knowledge and identification of dispersive soils is very important in the design and construction of such structures.

In recent years the problem of dispersive or erodible clays has been recognized by the Engineering profession and some investigations and research in this area have been conducted by various agencies and individuals. Many erosion and piping problems with earth embankments, which were previously unexplained, have recently been attributed to this relatively new phenomenon.

There are three tests commonly used to identify the highly erodible soils i.e. dispersive soils, namely Soil Conservation Service (SCS) Laboratory Dispersion Test sometimes called Double Hydrometer Test, Crumb Test, and Pinhole Test. However, there is no direct method of estimating the erosion rate of dispersive soils in the laboratory.

Now in developed countries the soils used for the construction of dams are tested for dispersive characteristics before the construction stage. However, in Sri Lanka this is still not practiced.

The objective of this study is to find out the dispersive characteristics of soils in some existing Sri Lankan dam sites and attempt to relate that property with other basic properties of soils. For this, Double hydrometer test, Crumb test and Pinhole test were carried-out for soil samples from three existing Sri Lankan dam sites and other basic tests for soil such as Mechanical analysis, Liquid limit test, Plastic limit test, Shrinkage limit test and Compaction test were carried-out for these soil samples.

METHODOLOGY

1. Soil Conservation Service (SCS) Laboratory dispersion Test

In this method a hydrometer sedimentation test is carried out on two identical portions of the soil sample, one with and one without the use of a dispersion agent (Sodium hexametaphosphate) and mechanical shaking or stirring. The ratio between the measured clay fractions provides a measure of the dispersibility of the clay.

2. Crumb Test

In this method, dispersive clay soils are identified by observing the behavior of a few crumbs of soil placed in a dilute solution of sodium hydroxide.

TABLE 1. Identification of Dispersive soil by Crumb Test

OBSERVATION	GRADE	SOIL TYPE
No reaction	Grade 1	Non-dispersive
Slight reaction	Grade 2	Non-dispersive
Moderate reaction	Grade 3	Dispersive
Strong reaction	Grade 4	Dispersive

3. Pinhole Test

In this test distilled water is caused to flow through a one-millimeter diameter hole formed in a specimen of compacted clay under a controlled hydraulic head. The resistant to erosion of the clay is judged visually by the presence or absence of turbidity in the water when emerging and from the measurements of rates of flow and final hole diameter.

Since the laboratory has no pinhole apparatus, it was made according to the standard specifications (BS 1377).

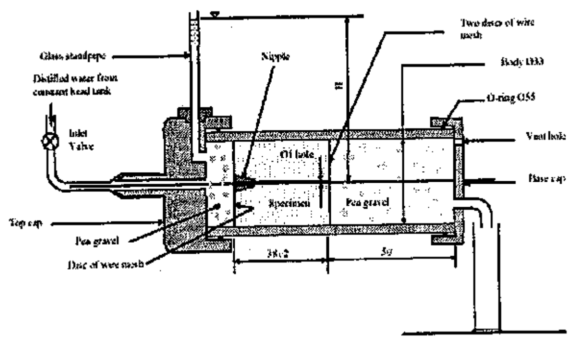
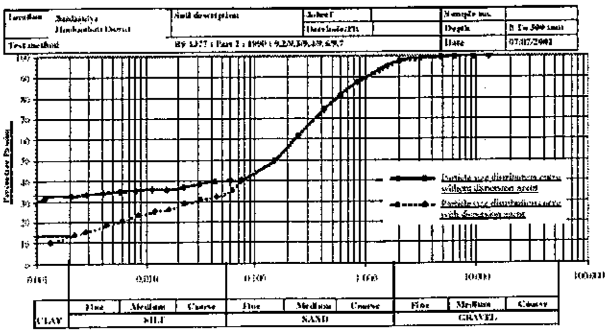


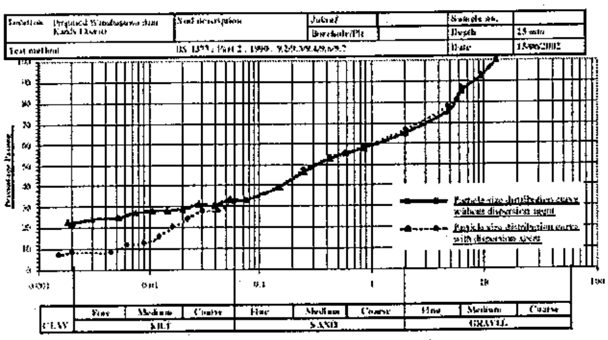
FIG. 2. Experimental arrangement of pinhole test

EXPERIMENTAL ANALYSIS

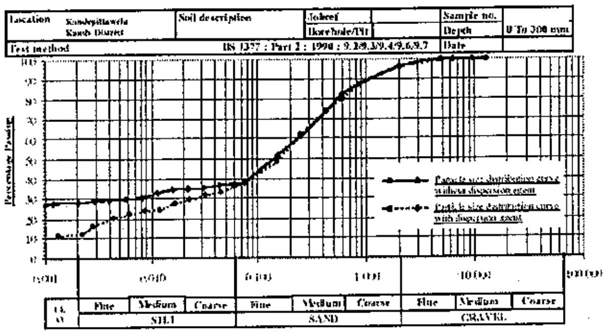
Since the time for the undergraduate project is limited, only three samples were tested for the clarification of dispersive soils. Samples were taken from existing Sri Lankan dam sites; Kandepittawela, Wanahapuwa and Bandagiriya. These samples were tested using above methods.



(a) Bandaragiriya sample



(b) Wanahapuwa sample



(c) Kandepittawela sample

FIG. 1. Particle size distribution curves with and without the use of dispersion agent

RESULTS

TABLE 2. Dispersive Properties

Sample Location	SCS Dispersion Test	Crumb Test	Pinhole Test
Kandepittawela	Non-Dispersive (D=43%)	Non-Dispersive (Slight reaction- Grade 2)	Non-Dispersive (Slightly dark-ND_3)
Wanahapuwa	Non-Dispersive (D=36%)	Non-Dispersive (Slight reaction- Grade 2)	Non-Dispersive (Clear-ND_2)
Bandaragiriya	Non-Dispersive (D=40%)	Non-Dispersive (No reaction- Grade 1)	Non-Dispersive (Clear-ND_2)

*D-Percentage Dispersion

TABLE 3. Other Properties

Parameters	Sample Location		
	Kandepittawela	Wanahapuwa	Bandaragiriya
Soil particles less than 0.06 mm(fines) /(%)	38	35	40
Plastic Limit/(%)	20	27	15
Liquid Limit/(%)	34	38	31
Plasticity Index/(%)	14	11	16
Soil Classification	Gravelly CLAY of low Plasticity	Gravelly SILT of Intermediate plasticity	Gravelly CLAY of low Plasticity
Shrinkage Limit/(%)	7	9	5
Optimum Moisture Content /(%)	20	19	14
Natural moisture content/(%)	4.5	4.4	4.5

DISCUSSION

FIG. 1. shows the particle distribution curve with and without the use of dispersion agent. In mechanical analysis both tests give approximately same curve. However, in hydrometer analysis, when the dispersion agent is used, the individual soil particles go into suspension and settle individually. Therefore the curve with the use of dispersion agent differs from the curve without the use of dispersion agent. Percentage dispersion is calculated by dividing the percentage of clay in the soil sample with the use of dispersion agent by that without the use of dispersion agent.

The dispersive properties of soil samples from Kandepittawela, Wanahapuwa and Bandaragiriya are tabulated in TABLE 2. From this table it is clear that the all three samples are Non-dispersive. The percentage dispersion found by Dispersion test is also shown in TABLE 2. The calculated percentage dispersion differs between the three samples. Also it could be noted in result TABLE 2., the dispersion characteristics obtained by Dispersion test, Crumb test and Pinhole test are not same. From SCS dispersion test, the dispersibilty of Wanahapuwa soil is lesser than other two. However, from Crumb Test, dispersibilty of Bandaragiriya soil is lesser than other two.

Only SCS Dispersion test gives a numerical result for dispersiveness of soil samples and other two tests give only visual observation as a result.

The basic properties of soil samples from Kandepittawela, Wanahapuwa and Bandaragiriya are tabulated in TABLE 3. All of the tests were done in accordance with BS 1377. Sample from Bandaragiriya has higher percentage of fines than other two and it has a low value of shrinkage limit; which is closer to its natural moisture content. In Badaragiriya soil sample more cracks appeared than other two samples during the shrinkage. Wanahapuwa sample has higher value of shrinkage limit than other two samples and it has the lowest percentage dispersion among the three samples (TABLE 2.).

The classification of soil samples is shown in TABLE 2.The soil samples were classified according to British Soil Classification System (BSCS). Kandepittawela and Bandaragiriya soils are Gravelly CLAY of low plasticity and Wanahapuwa soil is Gravelly SILT of Intermediate plasticity. From the classification it is clear that the Wanahapuwa soil sample has the lowest percentage of fines among the three samples. Therefore, it has the lowest percentage dispersion (TABLE 2.).

CONCLUSION

The tested soil samples from three dam sites are non-dispersive, but dispersibility of Kandepittawela soil sample is larger than the other two, but it is within the non-dispersive region. All three soil samples are safe to use in the earth dam construction when considering the dispersive problems. However, they are in the proximity of dispersive region.

SUGGESTIONS

From the study it is clear that the tested soil samples are non-dispersive but they are closer to the dispersive region. Therefore in Sri Lanka also like in the other developed countries the soils for the dam construction should be tested for dispersive characteristics before the construction stage. Further study should be carried out with large number of soil samples to find a definite relationship between the basic properties and dispersive properties of local soils.

APPENDIX. – REFERENCES

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IMPROVEMENTS OF PEATY CLAYS BY ELECTRO-OSMOTIC CONSOLIDATION

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Abstract

Laboratory studies were conducted to study the effectiveness of electro-osmotic consolidation to improve the strength and stiffness characteristics of Sri Lankan peaty clays. Results showed that electro-osmotic treatment of Sri Lankan peaty clay caused significant improvements in both the primary and secondary consolidation characteristics and shear strength. It was also found that the effectiveness of Electro-osmotic consolidation increases with the frequent polarity reversals.

1 Background And Objectives

Major geotechnical problems are associated with constructions done on peaty clays due to very high primary and secondary consolidation settlements and very low shear strength. Due to unavailability of land with good sub soil conditions, Engineers are compelled to use land underlain by soft peaty clays in and around Colombo for the infrastructure developments such as highways. In such cases, it is more economical to have them on improved ground rather than to use pile foundations. Hence, it is of utmost importance to develop economical and rapid methods for the improvement of peaty clays.

Electro-osmosis process had been applied successfully on several occasions to improve the Engineering properties of soft inorganic clays. Consolidation is achieved by providing an electrical voltage difference across the soil and allowing the water to be drained at the cathode. The process of electro kinetic consolidation was found to be much faster than hydrodynamic consolidation. (Hausmann (1990), Mitchell & Wan (1977), Morris et al (1985)).

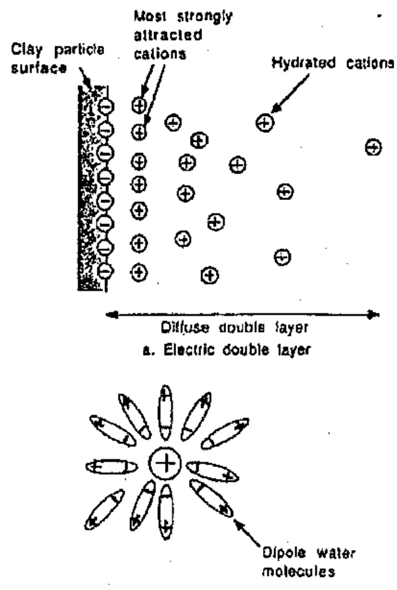
As there is no need of applying a surcharge, process will not cause shear failures even in extremely soft clays

There were no reports in literature on the application of this technique to organic clay or peat. Sri Lankan peaty clays are encountered close to the ground surface and the organic content is relatively low. As such, electrical losses associated with the Electro-osmosis process will be minimum and the process could be quite successful with the Sri Lankan peaty clays. Furthermore, the process can be carried out in the field without any special machinery. In this project, an attempt was made to study the improvements achievable in Sri Lankan peaty clays by Electro-osmosis, under laboratory conditions.

2.1 Theory of Electro-Osmosis

Reuss was the first person who observed that the water flow in soil as a results of a direct current passing through it (Reuss 1809). The process of ground water flow in response to an electric potential difference is known as electro-osmosis. In clays, the surfaces of particles possess a net negative charge and there is a diffuse double layer of cations (positive ions), such as K^+, Na^+ or $Ca^{+,}$ surrounding the particles (Figure 1).

Water molecules are bipolar, with a positive charge at one end and a negative charge at the other.Bipolar water molecules are attracted both by the negative charged surface of the clay particles and by the cations in the double layer. Ion concentration changes with the distance from the clay particle surface. Beyond the diffuse double layer the ion concentration is equal to that of free water and ions are free to move. Bipolar water molecules surround the positive ions in the diffuse double layer around a clay particle and these positive ions are called hydrated cations. Applications of an electric potential to the saturated clay, causes the hydrated cations in the diffused double layer to migrate towards the cathode to gain electrons and thereby become discharged. As the cations move, they carry free water with them. The larger the particle surface area, the more soil moisture transfer will occur. If the water is drained out at the cathode, but not replaced at the anode, then the process of Electro-osmosis is capable of achieving improvements in soft sensitive clays through consolidation.



a. Electric double layer

b. Simplified representation of hydrated cation

2 Methodology

2.1 Development Of The Experimental Set-up

An experimental setup was developed to carry out the electro-osmotic consolidation of peaty clay. The level of improvements achieved in the peaty clay by Electro-osmosis process has to be compared with the level of improvements achieved by the other conventional procedures such as preloading. As such, a loading frame was fabricated so that it can accommodate four consolidation samples to be tested at one time. In all the four test setups, facilities were provided to conduct conventional consolidation tests. Two of the setups were provided with the facilities to conduct the electro-osmotic consolidation. Specimens were kept inside a stainless steel cylinder of diameter 72.5 mm and height 90mm .In the specimen subjected to electro-osmotic consolidation, two electrodes were used at the top and bottom directly touching the soil sample and a DC voltage difference was provided. The voltage supply to both the samples was done using separate voltage regulators. Each regulator can control voltage from 0 to 12V. By using a transformer, Voltage of the main electrical supply is reduced down to 12V. The 12V AC is converted into 12V DC and connected to the regulators. A 12V battery was also connected to supply the power in the case of a power failure. A relay is used to control the power supply to the regulator from battery and the main supply. Electrodes were made of stainless steel or brass plates and holes were made through to facilitate drainage. The loading arrangement, cross sectional view and electrodes and the electrical circuit are shown in Figure 3, Figure 4, Figure 5 & Figure 6 respectively.

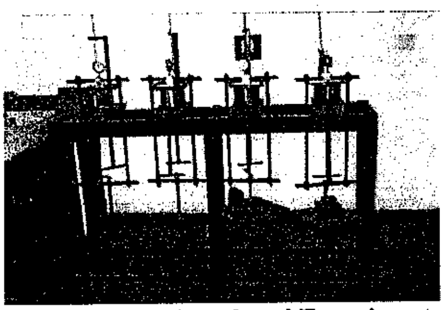


Figure.3 Newly Developed Experimental Set-up

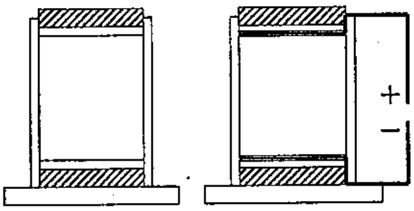


Figure 4 cross sectional view

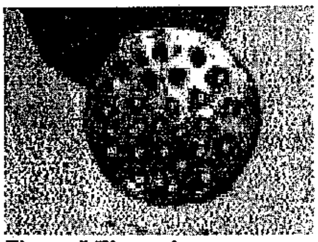


Figure 5 Electrode

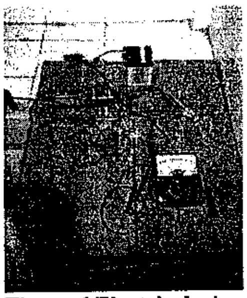


Figure 6 Electrical circuit

2.2 Electro-Osmotic Consolidation Testing

An Amorphous peat obtained from Wattala was used in the study. The basic properties of the peat used are presented in Table 1.

Water Content		263.6 %
Atterberg Limits	Liquid limit	130.6 %
	Plastic limit	108.0 %
	Plasticity index	22.6 %
Specific Gravity		1.777
pH		3.165
Organic Content		30.5 %

Table 1. Basic properties of peat used

Remoulded samples of peat were subjected to testing in the apparatus. Several series of tests were done. In the test series1, four identical samples were used. All four samples were initially consolidated at a load of 10 kN/m^2. Two samples were subjected to Electro-osmotic consolidation thereafter. The other two samples were taken through load increments 20kN/m^2, 40kN/m^2, 80 kN/m^2 and then unloaded to 10 kN/m^2 and reloaded through 20kN/m^2, 40kN/m^2, 80kN/m^2 to160 kN/m^2. The electrically treated samples were loaded through 20kN/m^2, 40kN/m^2, to 80 kN/m^2. A typical settlement vs time plot during the Electro-osmotic consolidation is presented in Figure 7.

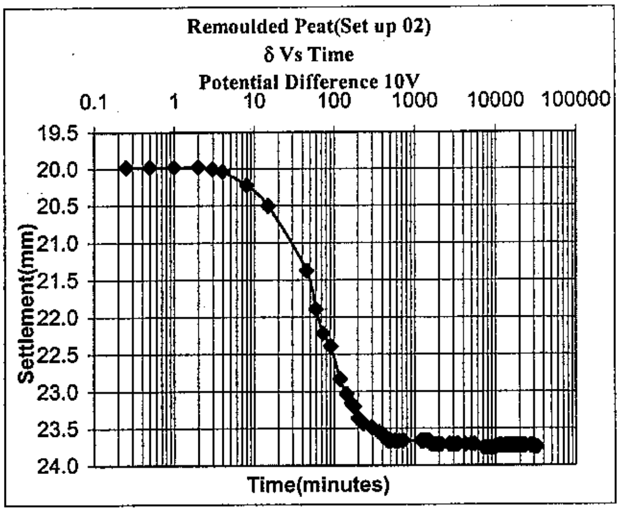


Figure 7.Graph of settlement vs. time

3 Analysis Of Results

3.1 Results from test series 1

3.1.1Reduction of volume compressibility, m_v

The results of the tests are presented in table 2.Column 2,presents the data of the loading increments of the conventional test and column 3 presents the data of the reloading increments of the same test. Column 4 presents the data from the sample, which was subjected to Electro-osmotic consolidation after 10kN/m^2. Thus the column 3 is presenting the case of a peat treated with preloading and column 4 is presenting the case of a electrically treated peat. The data are graphically presented in Figure 8.It could be seen that the Electro-osmotic treatment has reduced the m_v values considerably. The reduction was of similar order as that achieved by preloading .The reduction in the primary consolidation settlements are further illustrated by the C_c and $C_c/(1+e_0)$ values in the table 2.

Consolidation pressure (kN/m^2)	m_v ($m^2/kN*10^{-3}$)		
	Conventional loading	**Conventional test (Reloading)**	**Test with Electro-osmosis**
10	16.8	-	14.9 EO
20	5.69	0.41	0.3
40	5.45	0.41	0.22
80	0.80	0.40	0.18
C_c	1.34	0.4609	0.1046
$C_c/(1+e_0)$	0.23	0.1695	0.019

Table 2. Comparison of m_v and Cc -series1.

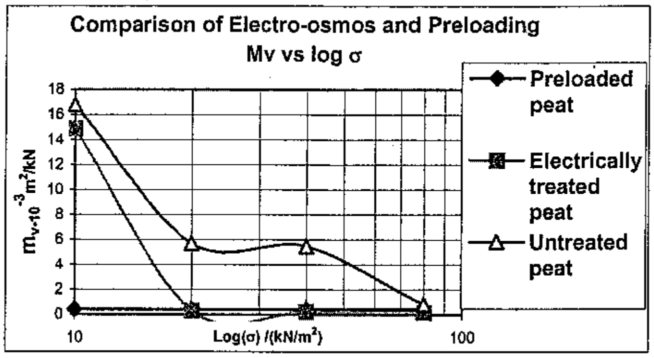


Figure 8. Comparisons of m_v vs. log(σ)–series 1

3.1.2 Reduction in the coefficient of secondary consolidation C_α.

Data in Table 3, illustrates that the electrically treated peaty clay has C_α values much less than that of untreated peaty clay. The heading of the columns are similar to those in Table 2.The data indicates that the improvements achieved in C_α by Electro-osmosis are much greater than those achieved by preloading.

Consolidation pressure (kN/m^2)	C_α ($m^2/kN*10^{-3}$)		
	Conventional loading	**Conventional test (Reloading)**	**Test with Electro-osmosis**
10	0.15	-	0.16 EO
20	0.19	0.02	0.06
40	0.22	0.02	0.008
80	0.37	0.01	0.006

Table 3. Comparison of Cα -series1

3.1.4 Improvements in undrained shear strength.

Undrained shear strength of treated samples as determined by the Torvane apparatus were considerably increased from the initial values. (Table 4). This improvement is much greater than that achieved by the preloading.

	Initial	Treated sample
Moisture Content %	263.6	153.7
P^H	3.16	4.17
C_u, kN/m^2	4.29	101.9
Specific gravity	1.78	
Organic content %	30.5	

Table 4 - Results – series1-electrically treated samples

3.2 Results from test series 2

Second test series was conducted to study the effectiveness of different electrode types. Specimen obtained from the electrically treated samples with Brass and Stainless steel electrodes were subjected to consolidation testing in a conventional oedometer and data on m_v and C_α were gathered. Behaviour of the samples treated with Brass and Stainless steel electrodes were found to be similar. As such, only the results obtained from the samples done with Brass electrodes are presented.

3.2.1 Reductions in primary consolidation settlements

The variations of m_v obtained from the test with brass electrodes are given in Table 5 and are graphically presented in Figure 9. It could be clearly seen that the electrical treatment has caused a reduction in the m_v values.

Consolidation pressure (kN/m^2)	m_v ($m^2/kN*10^{-3}$)		
	Conventional loading	**Conventional test (Reloading)**	**Electrically treated sample**
10	16.8	-	19.47
15	-	-	0.8
20	5.69	0.41	-
30	-	-	1.35
40	5.45	0.41	-
60	-	-	1.24
80	0.80	0.40	-
Cc	1.34	C_s 0.4609	0.370
$Cc/(1+e_0)$	0.23	0.1695	0.1236

Table 5. Comparison of m_v –series2

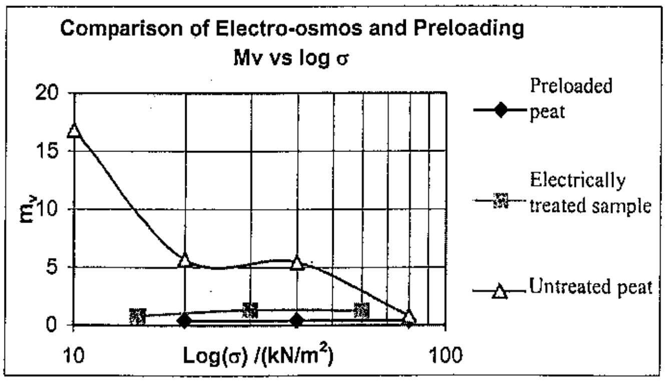


Figure 9. Comparisons of m_v vs. log(σ)–series2-(Brass electrode).

Improvement in primary consolidation can also be verified by comparing the void ratio vs log(σ) plot. The comparison of e vs log σ plots obtained from the untreated peat with loading/unloading and reloading increments (series 1) and Electrically treated peat (series 2)-tested in the conventional oedometer, are shawn in Figure 10.It can be clearly seen that there is a reduction in void ratio and the compressibility, due to the electrical treatment.

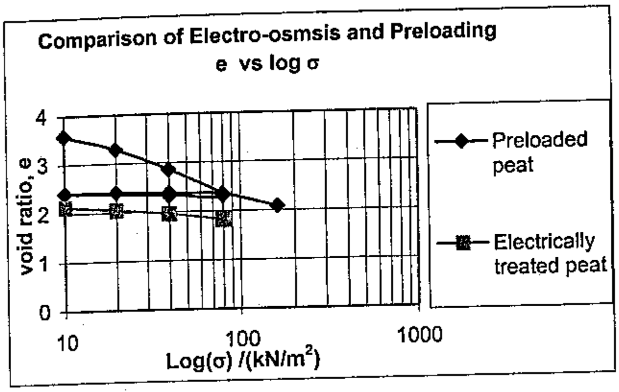


Figure 10. Comparisons of e vs. log(σ)–series2-(Stainless steel electrode).

3.2.2 Reductions in C_α

The variation of C_α with stress levels obtained in series 2 done with Brass electrode are presented in Table 6 and Figure 11.It is clearly seen that C_α values of the electrically treated samples are much lower than that of natural peaty clay. Reductions achieved were of same order as that achieved by preloading.

Consolidation pressure (kN/m²)	C_α (m²/kN)		
	Conventional loading	Conventional test (Reloading)	Electrically treated sample
10	0.15	-	0.1133
15	-	-	0.03
20	0.19	0.02	-
30	-	-	0.01
40	0.22	0.02	-
60	-	-	0.02
80	0.37	0.01	-

Table 6. Comparison of C_α –series2 –(Brass electrodes)

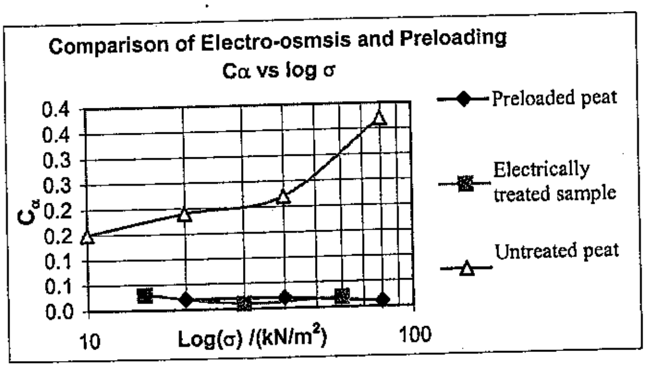


Figure 11. Comparison of C_α vs. log(σ) – series 2- (Brass electrode)

3.2.3 Improvement of the undrained shear strength C_u

Undrained Shear strengths of both the samples treated using two types of electrodes were increased from the initial values as shown in Table 7.

	Stainless steel Electrode		Brass electrode	
	Initial	Final	Initial	Final
Moisture Content %	263.6	116.6	265.94	109.5
p^H	3.31	5.45	3.31	7.49
C_u, kN/m²	3.3	80.4	3.3	112.6
Specific gravity	1.79		1.79	
Organic content %	30.5		30.5	

Table 7 - Results – electrically treated samples – series2

3.2.4 Variation of Water Content and pH

Water contents have reduced from the original values in both cases. In the Electrically treated peat the moisture content was lowest at the middle and higher at the two ends. That could be due to the two-way drainage that has taken place in the soil with polarity reversals. Reduction in void ratio due to Electro-osmotic treatment has also been observed by Lo et al.(1991).

In both cases the pH value had been increased during the electro-osmosis treatment. Similar observations were made by Lo et al (1991) and Shang et al (1996). A greater increase of pH value was recorded with the brass electrodes. This has to be verified with further tests.

4 Conclusion

It can be concluded that the electro-osmotic treatment of Sri Lankan peaty clay has improved both the primary and secondary consolidation characteristics and shear strength. Improvements achieved were of the same order or somewhat better than that achieved through preloading. These finding should be verified with the field-testing.

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Model Testing For Soil Slope Stability Using Sand

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ABSTRACT

Soil slope failures or landslides are a major concern in Sri Lanka, which has hilly terrains in the central part and several other locations in the country. A factor that has not received much attention in the study of landslides in the country is the effect of apparent cohesion generated in unsaturated residual soils by negative pressure in pore water above the water table. A method developed to investigate the effect of matric suction for soil slope stability using sand is discussed in this paper.

It is difficult to carry out model testing of residual soil slope failures, mainly due to scaling problems inherent in soil model tests, Hence sand is used as the soil medium for the model test. Colored nail heads were embedded in the sand mass so that deformation fields could be easily identified by capturing the movements of nail heads using a digital camera. The sand slope was gradually saturated by introducing water and deformation fields recorded by digital photographs at different stages of the deformation of the sand slope were analyzed to get deformation vectors. The coordinates given by the program were converted to actual coordinates by using a factor. A program was developed to calculate the best-fitting factor by checking each coordinate and by minimizing standard deviation. Displacement and strain fields occurring in the sand mass were evaluated and presented in diagrams.

1. INTRODUCTION

Most soils in our hilly areas are residual soils, which exist in unsaturated state in the dry seasons, with the water table located deep. Due to the matric suction, defined as the difference of pore air pressure and (negative) pore water pressure, these soils develop apparent, additional shear strength, which contribute to a higher factor of safety against slope failure. With the onset of rains the wetting front may reach downwards into the soil mass, and saturate these unsaturated layers. Then the matric suction vanishes and the factor of safety against slope failure may decrease to a low value, triggering these landslides. Most of the time Geotechnical Engineers analyse for slope stability using saturated parameters although the exiting slope is unsaturated. Then the analysis may indicate that the slope is unstable although it is stable in real nature. Of course it may not be the sole mechanism by which all landslides are trigged once the rainy season commences, but is a major contributory factor.

2. BACKGROUND

Model testing of soil slope failures has been carried out by some research workers in the past. An experimental program to study slope failures in local residual soils through model testing had been conducted recently at University of Moratuwa (Ratnasiri et.al. 2002). However, mainly due to scaling problems inherent in soil model tests, it has been found that many corrections need to be applied to the deformation field observed, in order to derive the deformation field that would result if the soil slope failed solely due to saturation of the initially unsaturated soil. Also it was difficult to get a clear failure surface in soil mass under gradual increments of load from top of the slope. Due to such difficulties this testing program was envisaged to demonstrate the triggering of slope instability due to the gradual saturation of an initially unsaturated soil mass. Sand was used as the soil medium. Sand has no cohesion, and can be made to a steep slope in an unsaturated state.

3. METHODOLOGY

A model sand slope was prepared in a box of dimensions 1.5m length * 0.15m width * 0.6m height made up of transparent Perspex sheets; the box was prepared watertight. Sand was wetted by mixing with water to maintain constant moisture content at 5.75% and this enabled the construction of a slope with a slope angle greater than its angle of repose in the dry state. A wet sand slope could be constructed at 48^0 to the horizontal, which was higher than the free slope angle of the sand (i.e., 36^0). Each sand layer was slightly compacted to the same degree by maintaining the same number of blows and height of fall of a rammer, so that each compacted layer was roughly 40mm in thickness. Nails 50mm in length, with the head painted, were used to represent some selected node points of the sand mass for the purpose of monitoring the movements. Nails were placed with the body of the nail inside the soil mass, so that the nails could be assumed to move with the neighboring sand masses. Black and white colored sand layers were placed alternatively and the nails were placed roughly 40mm apart along the centerline of each sand layer.

Water table was raised gradually by a continuous supply of water introduced through a perforated tube connected to a tap, and buried in the sand below the toe of the slope. The rate of water supply was carefully maintained (at a sufficiently low value) so that failure of the slope was not caused by soil movement by the water pressure at the water outlet (perforated tube) inside the soil mass. The perforated tube was prepared by using a one-foot length of 12.5 mm PVC conduit pipe. It is laid horizontally 75 mm above the bottom of the box and inside the sand mass. Figure 3.1 shows the typical arrangement of the model sand slope.

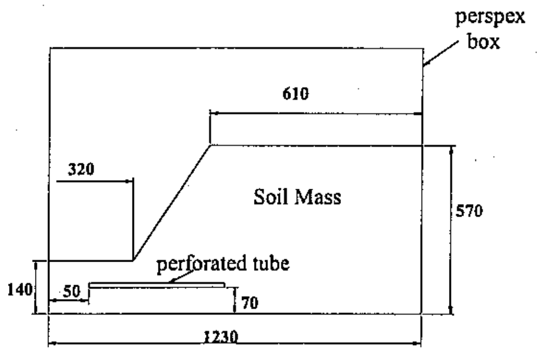

Figure3.1:Typical arrangement of the model sand slope (All dimensions in mm)

4. TEST PROCEDURE

The deformations that took place within the soil mass due to decrease in matric suction could be seen by the distortion of the nail pattern placed inside the sand layers. The displacements of the nail positions were captured on photographs taken with a digital camera, at different instances while the water table rose. The camera was stationed at the same point for all photographs.

The photographs taken during the test were digitized, by using computer software SURFER, to obtain coordinates of the grid points relative to a fixed point. Because coordinates given by the software while digitizing were relative to a coordinate system determined by the software, they had to be converted to actual coordinates on the soil slope. For this three fixed points, which appeared on each photograph as corners of a triangle, which could be easily identified on the model box, were selected. The coordinates of those three points were found with the software and they were used to calculate the distances between those points on the photograph. Then the actual lengths between those three points on the model box were measured. A scale for converting the coordinates given by the software to actual coordinates on the slope had to be selected, after ensuring that the ratios of the length between two points on the photograph to that on the model box were the same for all three lengths between selected points.

The above-calculated three ratios may have a slight deviation from each other, instead of being identical. A programme was developed to get the best-fitting ratio by checking each coordinate and minimizing the standard deviation. This scale was used to convert the coordinates given by the software to actual coordinates on the slope. Those actual coordinates were used to find the relative movements or the displacements of the nail points.

5. TEST RESULTS

Particle size distribution of the sand was analysed using the standard sieve analysis test. The results are presented in Figure 5.1.

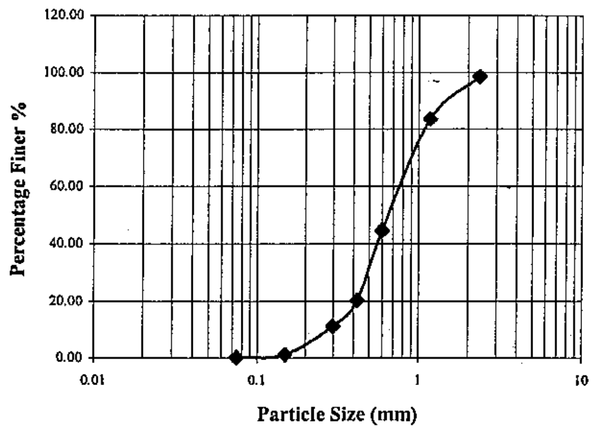

Figure 5.1 Particle size distribution of the sand used

For this paper, results obtained by analyzing three digitized photographs taken at different stages of deformation due to rise of water table, are discussed. Figure 5.2 shows model soil slope before the test was commenced (before introducing water through the buried perforated pipe). Figure 5.3 and Figure 5.4 show the photographs at an intermediate stage of deformation, and at the stage of slope failure, respectively.

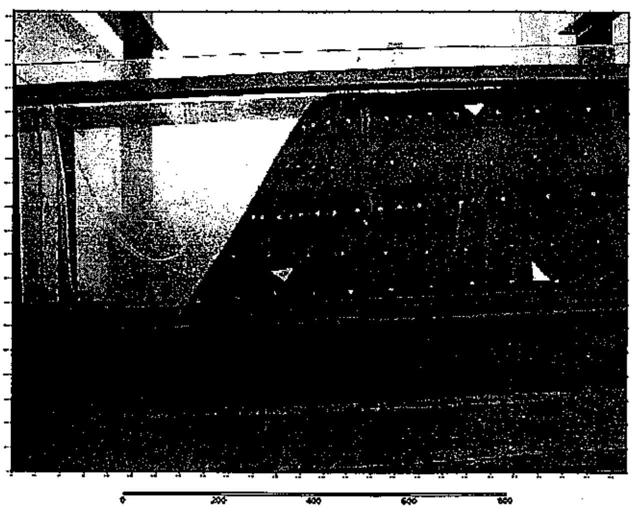

Figure 5.2 Model before raising the water table

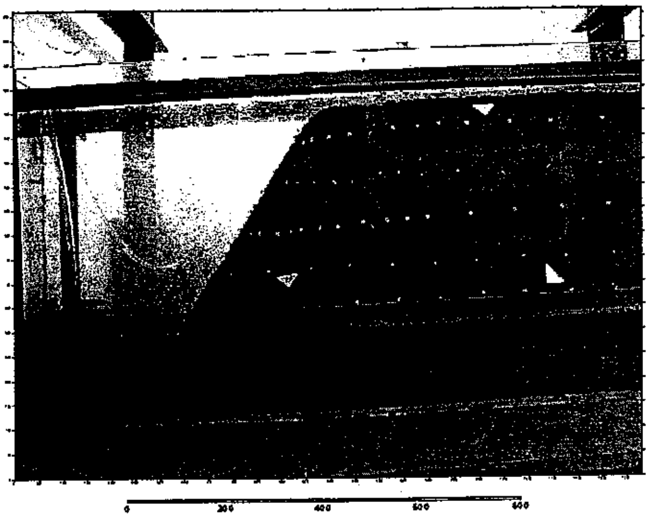

Figure 5.3 intermediate stage of deformation

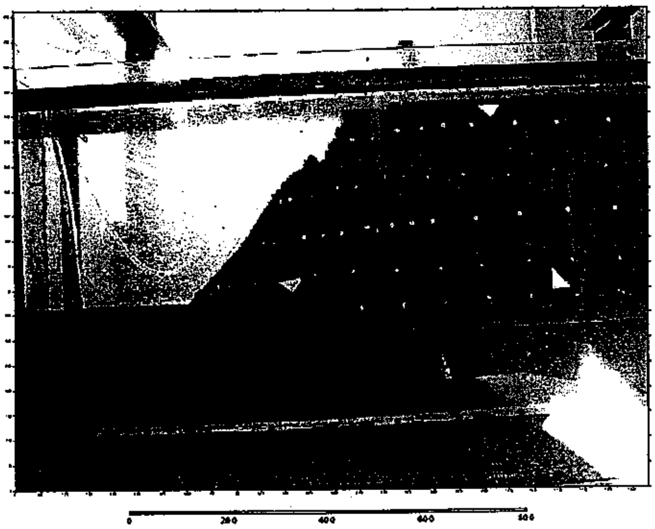

Figure 5.4 Final stage of failure

Moisture contents were measured at different locations on the slope face, after the failure of the slope. These results are shown in Table 5.1.

Table 5.1 Moisture content after slope failed

Location (Height above toe of slope) mm	Moisture content
100	22.8
200	11.6
300	6.3

6. DISPLACEMENT ANALYSIS

The coordinates of node points can be used to derive the displacement fields over the deforming sand mass (plane strain condition assumed). Figure 6.1 shows the displacement field as the water table is raised to a certain value. Figure 6.2 shows displacement field at the stage of failure of the sand slope.

Displacements of nodal points at the top corner of the slope in Figure 6.2 are relatively large. The vertical lines to the right of the plot show settlements of nodes vertically downwards.

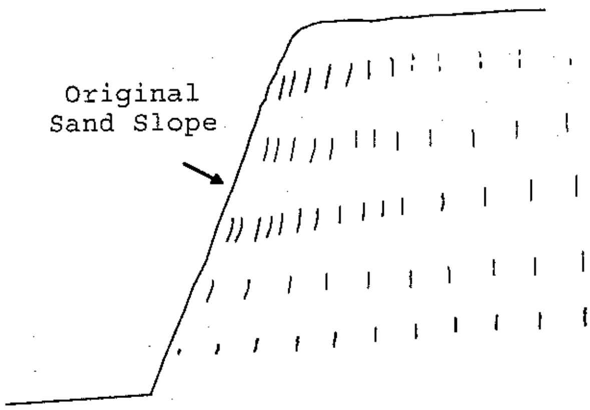

Figure 6.1 Displacement vectors of the node points at an intermediate stage of model test

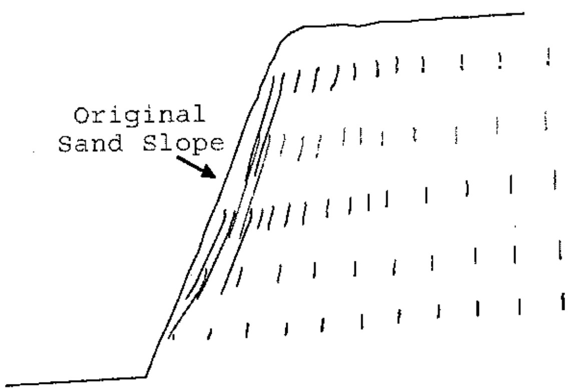

Figure 6.2 Displacement vectors of the node points at the stage of failure in the model test

With reference to figure 6.2, longer lines close to the sand slope surface of the plot indicate that large displacements have taken place there. Reasons for this occurrence may be the loss of matric suction and zero cohesion of the sand. Another reason is the development of a failure surface there. The lines in that region appear to indicate the formation of linear paths. This is an indication that a translational failure surface is developing in that region.

However, the nodes in the central part of Figure 6.2 show a predominantly vertical settlement. This may be due to the particular sand slope used and the rate of change in water table of the sand.

7. STRAIN ANALYSIS

The actual coordinates together with the displacements were used in the finite element program FEAP (Zienkiewcz, 1977), later modified and developed by Puswewala (Puswewala, et al. 2000). This was for the purpose of finding the principal strains developed in each element of the soil body. The FE mesh was defined by the nail pattern initially on the model. (The program facilitates the entry of displacements as the input when boundary conditions are given accordingly).

The program calculates the strains for each element after analyzing the problem.

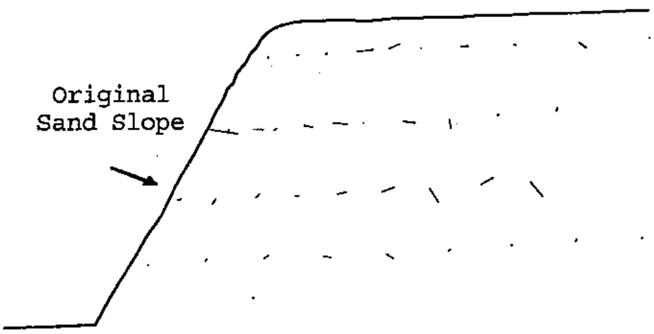

Figure 7.1 Principle Tensile Strain Field

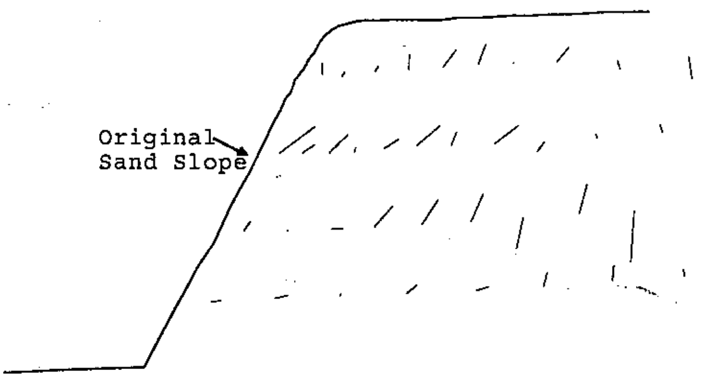

Figure 7.2 Principle shear Strain Field

8. CONCLUDING REMARKS

In conclusion, following points can be highlighted:

1. The devised model test successfully demonstrated the reduction of shear strength of the sand as the moisture content was gradually increased.
2. The sole reason for the translational type of sand slope failure was the increase in moisture content. The test successfully eliminated other effects (such as artificial surcharges) on soil slope failure.
3. The derived displacement fields and strain fields indicate the deformation characteristics and pattern of a sand slope failure.

9. ACKNOWLEDGEMENT

We would like to express my profound gratitude to Dr. U. G. A. Puswewala for his valuable guidance and would like to extend my sincere thanks to Dr. Ashok Peiris for giving instruments and instructions to do the test. My special thanks go to Mr. R.M. Rathnasiri who gave us valuable instructions about references, test procedure, test results analysis (about the software), theoretical aspects. etc.

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INFLUENCE OF PARTIAL SATURATION OF RESIDUAL SOILS ON SLOPE STABILITY AT PUSSELLAWA LANDSLIDE

Fernando K.P.M., Niroshantha M.A.J. and Range I.R.

Abstract

Unsaturated soil mechanics is becoming increasingly popular in the world of Geotechnical Engineering because of the additional shear strength that unsaturated soils possesses compared to saturated soils and specific problems that are associated with unsaturated soils. In this paper, effects of unsaturated shear strength properties on the stability of slopes are discussed for Pussellawa landslide using unsaturated shear strength parameters obtained from laboratory tests.

The factor of safety of the slope, at Pussellawa landslide site is evaluated by using the saturated parameters applicable when the water table is above the failure surface, is compared with the factor of safety values against failure along the same failure surface evaluated by using the unsaturated parameters applicable when the water table is below the failure surface.

Also the relationship between the factor of safety and the effective percentage of matric suction is investigated for above cases.

The effect of the location of the water table below the failure surface, on the factor of safety against failure is investigated for each case. For this the slope is analysed assuming different water table contours.

Introduction

The landslide is located between the road structures numbering 37/10 & 38/2 On Peradeniya- Badulla- Chenkalladi Road (Kandy- Nuwara Eliya Road).

In 1976, the lower portion of the hill slope below the road had been cut and converted to a public playground. For a period of nearly ten years since 1979, this section of roadway of length of about 40m has been subjected to slips and subsidence from time to time during periods of high rainfall (Mallawaratchie, 1994). Whenever the road subsided, highway authorities had been compelled to fill this road with tunnel muck and allow traffic on the road. This however further endangered the road due to extra loading caused by tunnel muck at the head of the landslide.

In order to determinate the causes of failure and to propose the remedial measures, the Research & Development Division of the Road Development Authority (RDA) carried out the following detailed investigation from January to March 1989:-

(a). Topographical survey

(b). Geotechnical investigation

1. 07 boreholes were sunk upto 3m depth in bedrock.
2. Undisturbed samples were taken and triaxial tests were performed.
3. Piezometers were installed in all the boreholes

(c). Geophysical survey

(d). Geological survey

The landslide has been caused by the removal of toe support and blocking the drainage by the cutting, filling and leveling operations for the construction of the playground at the toe area of the present landslide.

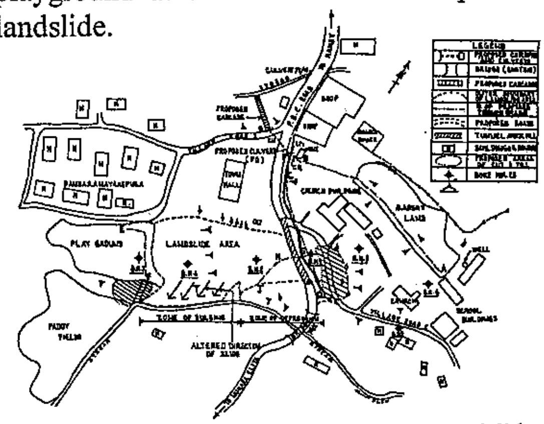

Fig. 1: Sketch showing the Pussellawa Landslide

Field Observations & Analysis of data

According to the data obtained from a survey carried by Research & Development Division, RDA (1993) some of the properties corresponding to the landslide are given below.

Slope category to degees 10-20 moderately sloping

Slope cover – grassland
Drainage – satisfactory
Type of bedrock – garnet biotite guiss
Classification of landslide – compound slide
Depth of landslide – shallow

Cross section of the slope were analysed, as illustrated in figure 2.

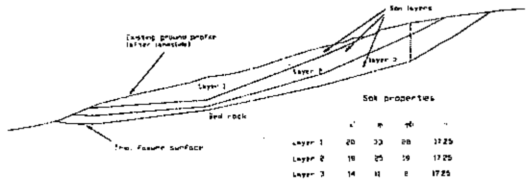

Fig. 2: Cross section of the slope

A method for the stability analysis of Pussellawa slope with negative pore water pressures involves the following steps.

1. a detailed study of the survey of the elevation of the ground surface on a selected section
2. the advancement of several boreholes to identify the stratigraphy and obtain undisturbed soil samples. The following SPT values were obtained using those borehole data.

Layer	1	2	3	4
Average SPT No	11	18	7	28

Table 1: SPT values of selected cross section

3. suitable shear strength parameters for each stratigraphic unit (i.e. C', ϕ' ,and ϕ^b parameters) were provided by Rathnasiri, M. (2002) from his past laboratory testing.
4. different levels of water table were assumed.

Those steps provide the input data for performing a stability analysis. However the location and shape of the most critical slip surface is an unknown.
In design, the shape the unknown surface is assumed while its location is determined by trial and error procedure. Here both circular and non-circular failure surfaces were considered for the analysis.

Stability analysis of slope

Analysis of Non-Circular failure surface

After the detailed study of borehole data, a weak soil layer which has the lowest SPT value among the others was identified as a failure surface. Using these data the section was drawn using AutoCad and it was divided into several vertical slices to perform analysis. Using this scaled drawing, geometric properties of each slice was obtained and the analysis was done using an Excel worksheet based on the Janbu's Simplified method.
Here slope stability analysis was performed to assess the effect of variation of matric suction on the factor of safety. Latter analysis was done by varying matric suction using several percentage values for a constant level of water table, as illustrated in fig. 3.

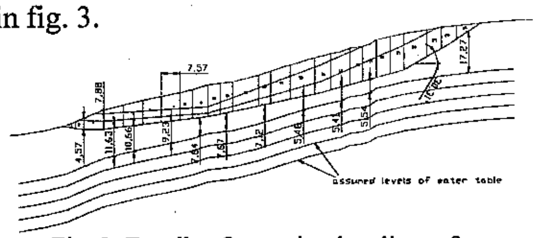

Fig. 3: Details of non-circular slip surface

Also, a parametric analysis was conducted using various levels of water table and the factor of safety was obtained.

Analysis of Circular failure surface

This circular failure surface was obtained using the SLIDE computer program. In that program a grid center were selected and the radius varied at each center providing coverage of all possible conditions as shown in figure 4.

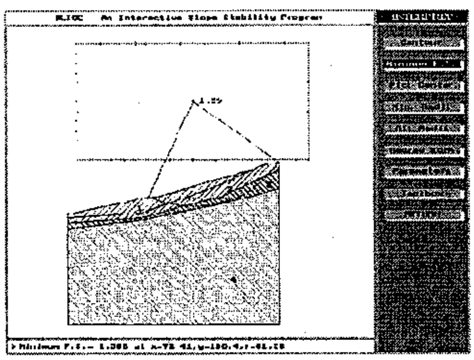

Fig. 4: Typical output from SLIDE program

Then the selected circular slip surface was drawn using the AutoCAD package. Calculations for the stability of a slope are performed by dividing the soil mass above the slip surface into vertical slices. Those slices were selected to have more or less uniform soil properties. (see figure 5)

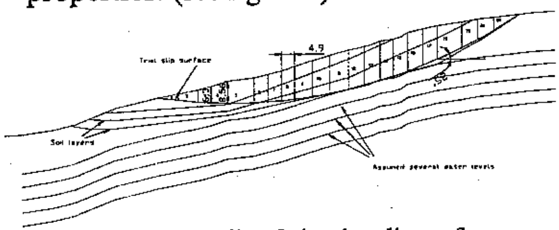

Fig. 5: Details of circular slip surface

Slice properties like slice width, heights and angles required to analyse the slope were extracted from the scaled drawing. Spreadsheets were written to analyse the slopes with saturated parameters and unsaturated parameters. Analysis was done by varing the water level of the selected section of the landslide and factor of safety was observed.
Also factor of safety was obtained by considering variuos percentages of the matric suction for a given water table positions.

Results of analysis

Effects of level of water table

Varying depth to the water table will affect the pore water pressure and hence the stability of the slope.

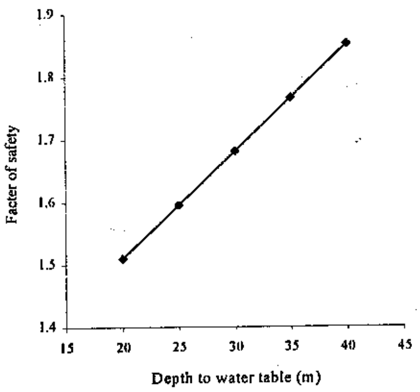

Fig. 6: FOS vs. Depth of water table for non-circular slope failure

From the results shown in the figures 6 and 7, it is clear that safety factors drop when the water table starts to rise. The lowest factor of safety is observed at the saturated conditions. When the water table goes down factor of safety increases.

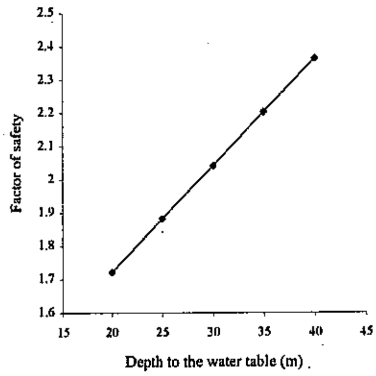

Fig. 7: FOS vs. Depth of water table for circular slope failure

Effect of matric suction

The suction profiles in the slope are affected by the position of the ground water table and the climate (Leong and Rahardjo, 1997a). The deeper the ground water table, the higher is the suction at the ground surface, which can increase to even higher values due to evaporation and evapo-transpiration with rain water infiltration. The upper part of the ground may loose part of or in the extreme case all of its suction. Such changes in the suction profile have been observered in Singapore residual soil slopes. (Lim et al 1996, Rahardjo et al, 1998)

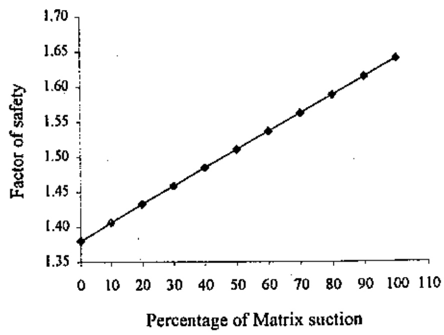

Fig. 8: FOS vs. percentage of suction for non-circular slope failure

Since the matric suction capacity is not constant, even for a constant depth of water it is better to consider several percentages of suction values with analysis.

Although the ground water table is constant, factor of safety will vary with matric suction value. When the matric suction capacity increases, stabhility of slope also increases.

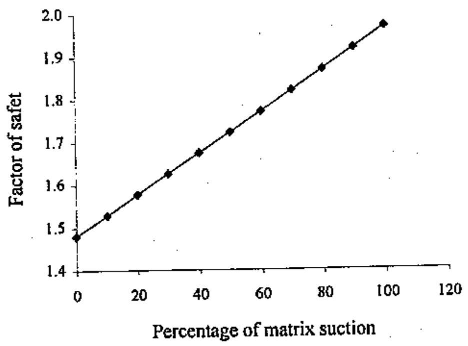

Fig. 9: FOS vs. percentage of suction for circular slope failure

CONCLUSION

From the studies carried out, the following conclusion can be summarized.

- Factor of safety of the slope against failure along the selected surface increases when the water table is lowered.
- Factor of safety of the slope against failure increases when the percentage of matric suction is increased.

However the negative pore water pressure values used in this analysis are hypothetical. Methods are available in other countries for insitu measurement of soil suction. With such measurements, the method discussed in this paper can be used to find out the actual factor of safety of natural slopes, as the location of the water table changes. The insitu suction depends on both the location of the water table and the moisture content of soil. The soil moisture content can vary due to heavy and/or prolonged rainstorms. Therefore to find the actual factor of safety of a slope incorporating unsaturated shear strength parameters, it is essential to record the insitu suction with depth above the water table at different periods of the year and consider a reasonable suction value for calculating the factor of safety of the slope.

This type of analysis is also helpful to understand how the slopes fail upon saturation during rainy seasons. Most of the rain-induced landslides occur due to the strength reduction that takes place when the soil suction vanishes upon saturation.

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Undrained shear strength characteristics of Residual soils

Thilakarathna H.M.I., Jayanthi H.W., Upashantha A.K.M.

ABSTRACT

In Sri Lanka, except few parts of the northern areas of the island, more than 90% of the land is made up of Metamorphic Rocks. Most of the time this soil type is considered as a heavy load bearing soil. Residual soil is derived from insitu weathering of soil due to different courses of environmental actions. Many of the landslides take place in the hill country during periods of heavy rain, and the failure surfaces pass through residual soils and colluviums deposits. The investigation of these slopes for classical slope stability analysis requires information on the actual or probable failure surface, shear strength parameters and the pore water pressures developed on the failure surface. In order to investigate the property of the residual soil, fissures and joints as well as the particle sizes varying from clay size to boulder size need to be considered.

1. Introduction

We often found these soil types under the construction of roads and buildings. Also we found that most of the landslides, which occur in Sri Lanka, have their failure surface passing through residual soils and colluviums deposits. On the other hand the soil mass insitu may display a sequence of materials ranging from a true soil to a soft rock depending on the degrees of weathering.

Residual soil derived from insitu weathering of rocks due to different course of actions of the environmental conditions of the related areas shows different characteristics from its parent material. So it is not possible to extrapolate the properties of the parent materials to predict the properties of residual soils. There for in each situation requires individual consideration.

2. Objectives

The main objective was to find out the variation of strength characteristics of residual soil with the change of density.

At the same time we aware to measure the pore water pressure for different densities as well as for different cell pressures.

3. Scope of the study

Tri-axial test is one of the most reliable methods available for determining shear strength characteristics. In the case of residual soil, a higher attention should be paid for testing methodology because of its anisotropy and difficulties of sampling and testing.

To prepare our test specimen we restrict the maximum particle size to 5mm. The scope was to find out a lower boundary for the shear strength characteristics of the residual soils based on the suggestion that strength is increase with the percentage of larger particles.

On the other hand we realized that effect of breakage of the soil particle was minimized under the compaction effort and failure was occurred along a more ordinary surface.

We do not apply a backpressure the idea behind was create the site condition that is unsaturated and pore pressure developed under the gravity.

This research was carried out using 100mm diameter samples and we believe that the accuracy expected was achieved.

4. Material Properties

One of the important objectives of investigation and testing of residual soil is to determine the shear strength parameters for purpose of analysis. But even in this case there are decisions to be taken on the type of strength parameters required depending on the age and the conditions at failure. ie;

-Total strength parameters (c_u, ϕ_u)
-Effective strength parameters (c', ϕ') or
-Residual strength parameters (c_r^1, ϕ_r^1)

Material obtained from Beragala and Kahagalle landsides were used for this research.

4.1 The effect of Pre-drying and method of mining

Soil to be used in this test should be at its natural moisture content. Pre-test drying of the soil should be avoided, as this tends to reduce the measured specific gravity as compared with natural moisture content samples.

Drying can cause partial or complete dehydration of the clay minerals and can change them and their properties. Therefore effect of air drying specimens prior to carrying out the Atterberg limit tests, rather than testing starting at natural moisture content has been observed to result in a decrease in a liquid limit and plastic index.

In Generally, the greater the duration of mining the larger the resulting liquid limit and to a lesser extent, the larger the plasticity index.

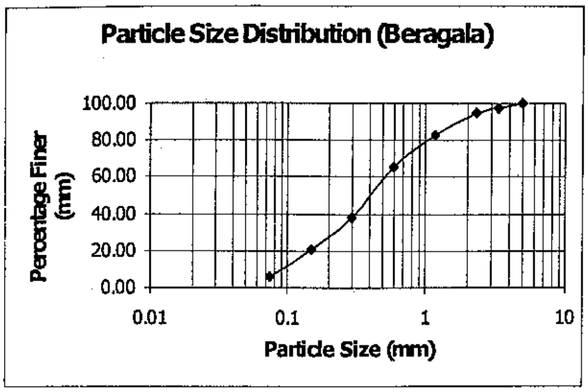


Fig: 1 Particle size distribution

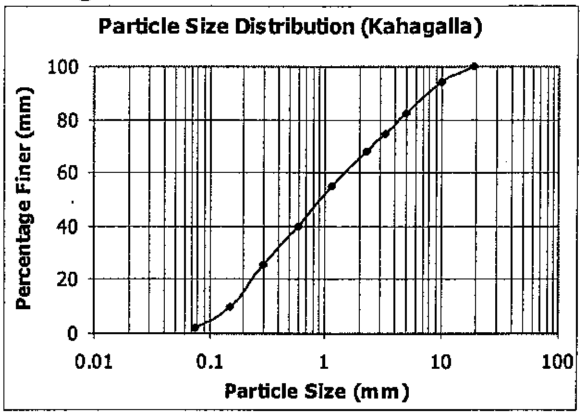


Fig: 2 Particle size distribution

Source	Beragala	Kahagalle
Liquid limit	51	47
Plastic limit	31	44
Specific gravity	2.66	2.65
Max. dry density (kN/m^3)	14.85	14.2
Optimum moisture (kN/m^3)	24.4%	29.0%

5. Tri axial Test

There are two genetic types of testing methods commonly used for the shear strength testing of soil in the laboratory, the direct shear test and the tri-axial test.

Both tests have their advantages and disadvantages, but certain field conditions may be simulated well by one type than by the other.

Tri-axial strength testing is now internationally accepted, and the common types of test for the determination of the effective stress strength parameters are the consolidated untrained test with pore pressure measurement, and the consolidated drained test. Conventional tri-axial testing was developed for 37.5 mm diameter samples, but these may be too small for the testing of residual soils with large particles.

Here we used 100 mm diameter samples for tri-axial testing in these soils. To measure the shear strength parameters we used consolidated un-drain test with pore water pressure measurements.

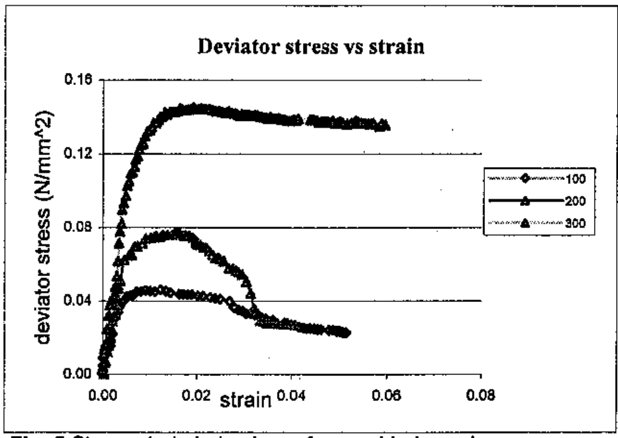
6.0 Methodology

6.1 Sampling: Composition of the residual soil often varies from gravel to slit or clay, because these soils are the end product of a weathering process. Therefore to improve the accuracy of the test results we have to conduct the test on many more samples. Also the accuracy of the test results can be improved by using larger diameter samples. Therefore through out our investigation we use 100mm diameter samples. To investigate the failure of slopes it would be desirable to use undisturbed samples. But due to the lack of resource to obtain larger diameter bore holes at different depths and the failure plane is not precisely identified, there may be some errors arising. But those can be accommodating by remolding the samples with smaller particles passing through a smaller sieve.

We obtained a known amount of soil and added a measured amount of water so that the sample can prepare to a known density. Here we limit the time of mixing of the sample after adding the water to a 10 min to minimize the further breakage of the bonds between clay clusters and generate more small particles. Then we mould the sample to four equal layers by applying a constant load (blow) on each layer. Here we use the particle sizes passing through the 5mm sieve and the soil was nearly well graded.

6.2 Saturation: CU test was conducted with the hope of measure the pore water pressure of the sample. We did the flushing of sample under gravity flow of water. We could achieve 90% as an average 'B' value.

6.3 Consolidation: At the next stage of testing we consolidated the sample under constant cell pressure. After 16 to 18 hr of continuous consolidation seem to the sample be achieved its full consolidation. Specimens were sheared



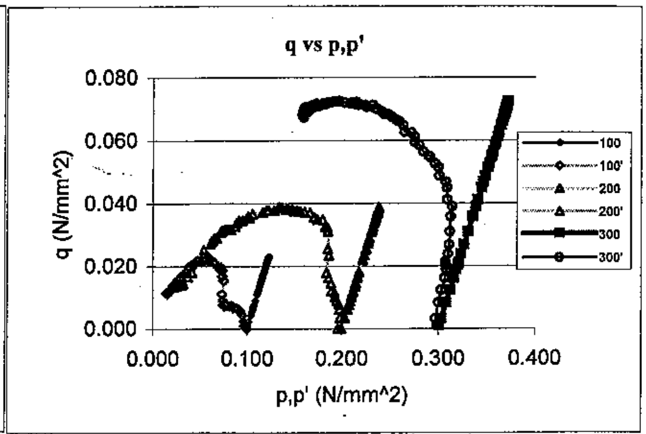
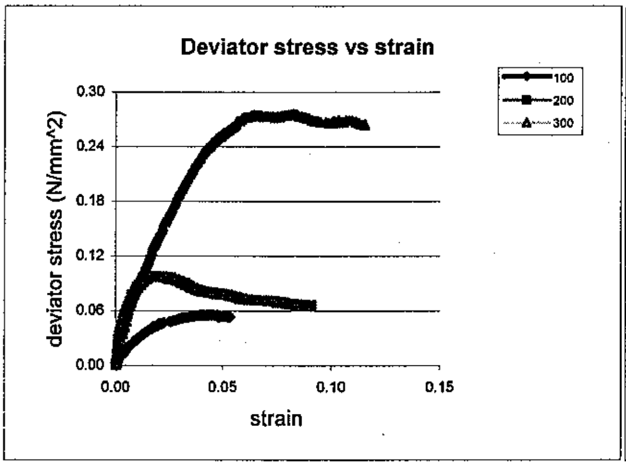


Fig. 5 Stress strain behaviour of remoulded specimens
Beragala (density =14.8 KN/m^3)

Fig.6 Stress paths obtained for remoulded specimens
Beragala (density =14.8 KN/m^3)



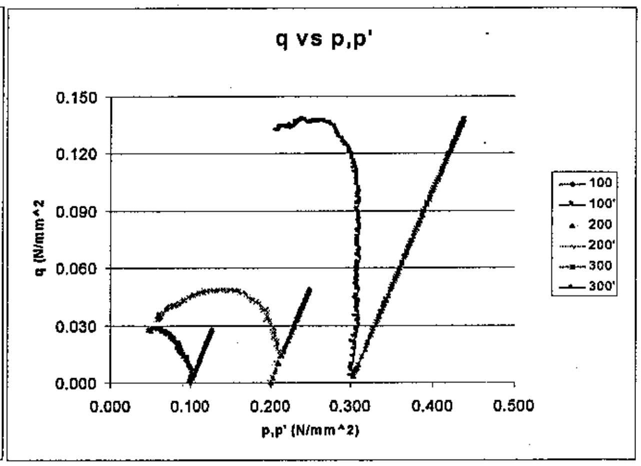
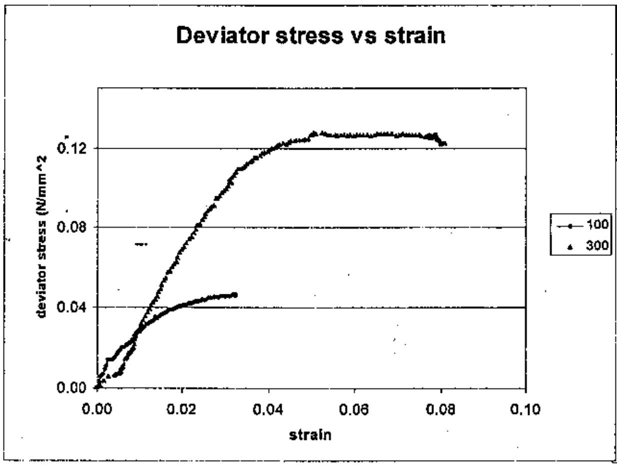


Fig. 7 Stress strain behaviour of remoulded specimens
Beragala (density =13.85 KN/m^3)

Fig.8 Stress paths obtained for remoulded specimens
Beragala (density =13.85 KN/m^3)



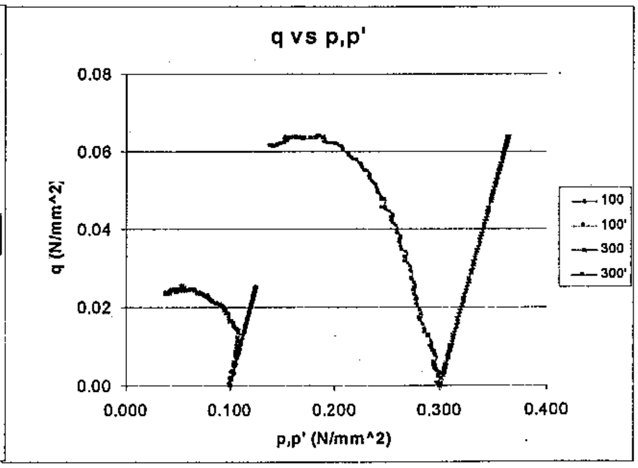


Fig. 9 Stress strain behaviour of remoulded specimens
Beragala (density =11.51 KN/m^3)

Fig.10 Stress paths obtained for remoulded specimens
Beragala (density =11.51 KN/m^3)

after consolidating isotropically to three different cell pressures namely 100,200 and 300kPa.

6.4 Shearing: Then we test the sample under the same cell pressure after closing the drainage value. The strain rate is about 0.05 mm/min. we did the manual application of the load even though the machine has the facility to automatic loading. The expectation was to allocate the time to spread the pour water pressure evenly throughout the sample and effect of the application of load, distribute throughout the sample.

7. Results and Discussion

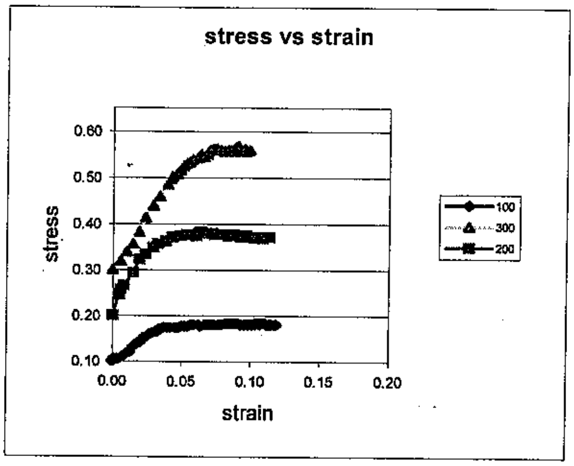


Fig 3: Beragala (Density = 13.85 kN/m^3)

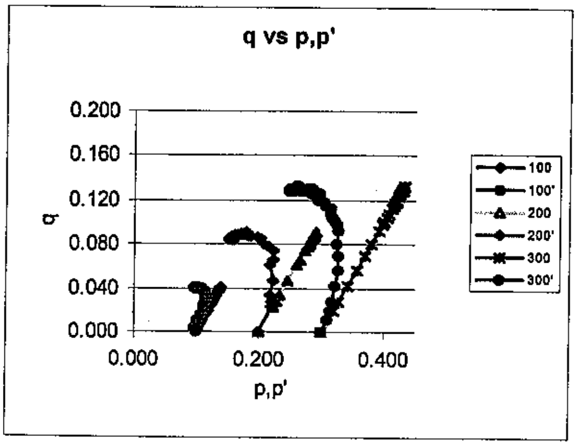


Fig 4: Beragala (Density = 13.85 kN/m^3)

Source	Beragala			Kahagalle
Density (KN/m^3)	11.50	13.25	14.85	14.2
ϕ	9.93	10.94	11.01	20.33
ϕ^1	28.09	32.24	31.34	39.23
Water content	0.158	0.215	0.240	0.280

Cohesion = 0

8.0 Recommendations

8.1 General

To identify the variation of the particle size distribution along the failure plane we can conduct a sieve analysis in the vicinity of the failure plane if it can be identified.

We noted that it is important to measure the pore water expelled from the sample while it gets consolidated. At the same time if there is an equipment to measure the volume change of the sample, it will be more helpful to conclude the test results more accurately.

8.2 Method of testing.

Here new specimen is set up for each and every test. So there could be exist some form of differences of densities and amount of saturation from one sample to the other.

To address this problem we can use multi-stage testing. By conducting multistage tri-axial test, the shear strength parameters c' and ϕ' are determined by testing a single specimen.

9. Research suggestions for Future

The future research could be carried out on remolded samples by varying the particle size distribution of the test specimens and using Multi stage testing method.

10. Acknowledgement

We wish to express our sincere thanks to Prof. D. C. H. Senerath, project co-coordinator, for his guidance and advice. We wish to express our heartfelt gratitude to the project supervisor Dr. Ashok Peiris for his valuable guidance and motivation to carry out the research. We would like to extend our special thanks to Dr. Saman Thilakasiri for his valuable support.

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Undrained shear strength anisotropy of undisturbed and remolded Over-consolidated clayey soils

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ABSTRACT

In order to find out the anisotropic behavior of undrained shear strength of an over-consolidated clayey soil, consolidated undrained triaxial tests were carried out in undisturbed and remolded samples. The undisturbed samples showed a minimum undrained shear strength when compressed at an angle of 45° to the consolidation plane and the anisotropy in undrained shear strength becomes greater as the OCR is increased. The remolded samples showed a minimum undrained shear strength when compressed at an angle of 15° to the consolidation plane suggesting a lesser influence of induced anisotropy on remolded over-consolidated clayey soils.

INTRODUCTION

Soil particles, especially platy clay particles tend to align their faces perpendicular to the direction of deposition or K_0- consolidation so that the micro-structure becomes inherently anisotropic. This inherent micro-structure can change during shear which is called the induced anisotropy. As a result the undrained shear strength of over-consolidated clayey soils has been found to be anisotropic. The importance of shear strength anisotropy in engineering problems has been demonstrated by Bjerrum (1973). Different types of variations of undrained shear strength categorized into three groups have been identified, when the direction of compression is changed from a direction parallel to the plane of consolidation to that normal to the plane of consolidation (Kurukulasuriya et al,1999). They are 1. Monotonically increasing variation 2. Monotonically decreasing variation 3. A variation with a minimum value when sheared at about 30° to the consolidation plane. (Duncan and Seed,1966a, Davis and Christian,1971)

In order to explain the existence of different trends in undrained shear strength of clays from a micro-structural point of view, an evolution rule has been incorporated in formulating an anisotropically hardening model (Kurukulasuriya et al,1999) which describes the anisotropy using a fabric tensor which changes during shear.

An evolution rule the form of which was suggested by Oda (1993) and which is modified is incorporated into the Ohta-Sekiguchi model in simulating the fabric changes during plastic deformation and is given as follows.

$dF_{ij} = a[(\ {}_2^F) - (J_2^F)]\ dS_{ij}$ where,

dF_{ij} = Incremental fabric tensor
dS_{ij} = Incremental deviatoric stress tensor
${}_2^F$ = A limiting value (a saturation value) of second invariant of fabric tensor (J_2^F)
a = A constant which influences the rate of change of fabric tensor

METHODOLOGY

Site selection
Soil samples were extracted from a location having high clay content, in order to prepare samples under undisturbed and remolded states. The site was selected in Hindagala after carrying out a mechanical analysis and Atterberg limit tests on soil samples from several different locations. The soil was classified as CLAY of extremely high plasticity (CE) according to the British Soil Classification system.

Undisturbed Samples

The samples were extracted from the above site at different directions to the plane of consolidation (0°,30°,45°,60°,90°) using a special apparatus made for the purpose (Fig.1). The exact location of sampling was selected such that the plane of consolidation was apparently horizontal based on visual observation of bedding planes of weathered rock. Consolidated undrained triaxial compression tests (CU) were carried out on the undisturbed samples under a cell pressure of 50, 100 and 150 kPa after ensuring that the value of pore pressure coefficient, B is greater than 0.95. Consolidation tests were carried out to determine the pre-consolidation pressure using the Casagrande's construction which was found to be 275 kPa. Accordingly, the over-consolidation ratios were found to be 5.5, 2.75 and 1.8 under cell pressures of 50, 100 and 150 kPa respectively.

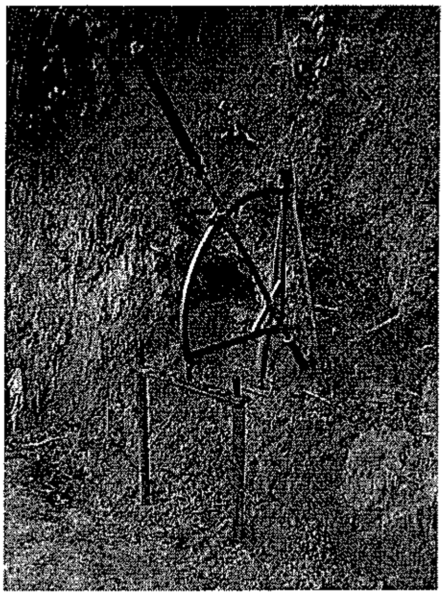

Fig.1 Apparatus used for extraction of undisturbed samples at a desired direction

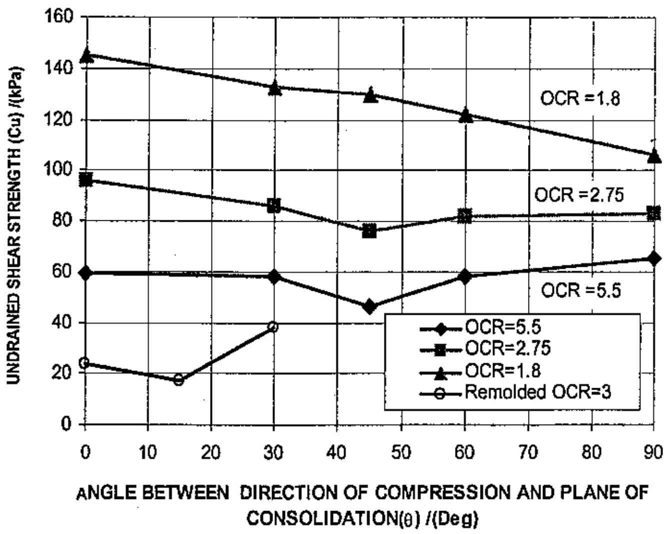

Fig.2 The variation of undrained shear strength with the angle between the direction of compression and the plane of consolidation for undisturbed and remolded samples

Remolded samples

A slurry was prepared by depositing clay in de-aired water and samples consolidated to a uniaxial pressure of 150 kPa were prepared in a steel mould. Samples were extracted having an inclination of 0°, 15° and 30° to the consolidation plane using the special apparatus described above (Fig.3). Three samples were extracted from each direction of sample extraction. The selection of above directions for sample extraction facilitates to observe the changes of orientation of fabric structure during shear deformation as the inherent fabric structure can be expected to change noticeably during compression for the above. The samples were isotropically consolidated in a triaxial cell under a pressure of 50 kPa to maintain an over-consolidation ratio of 3. The three samples extracted having a particular inclination were saturated and compressed under triaxial conditions until a) failure strain, b) strain corresponding to 60% of peak deviator stress, and c) strain corresponding to 90% of peak deviator stress, was reached. Specimens of these samples were collected for microscopical analysis for future studies.

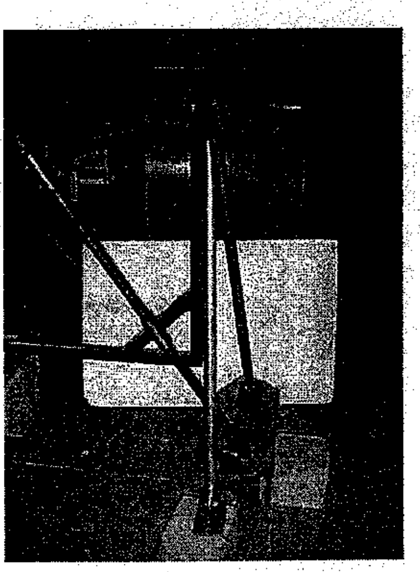

Fig.3. Sample extraction from remolded samples at a desired angle using the special apparatus

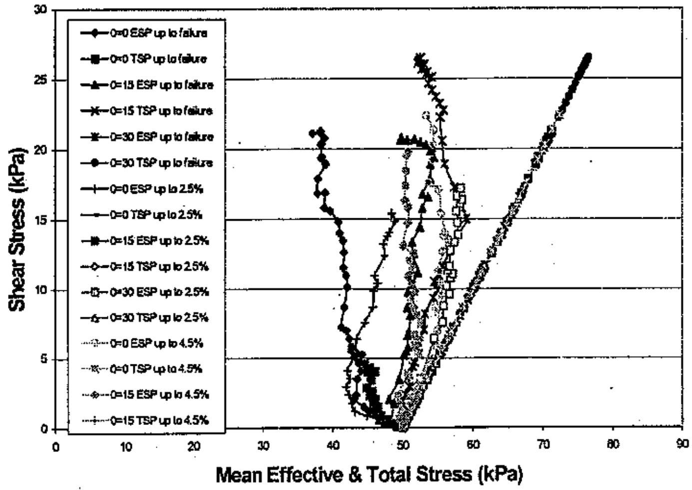

Fig.4. Stress path for remolded samples sheared up to (a) failure stress, (b) an axial strain corresponding to 60% of peak deviator stress and (c) an axial strain corresponding to 90% of peak deviator stress

RESULTS

The variation of the observed undrained shear strength when the angle between the direction of compression and the plane of consolidation is varied from 0° to 90° for tests on undisturbed samples is shown in Fig.2. Based on the above results, it could be seen that the anisotropy of undrained shear strength at high OCR values shows a variation showing a minimum value of undrained shear strength when compressed in a direction inclined at 45° to the consolidation plane. The Fig.2 also shows the undrained shear strength variation of the remolded samples. The total and the effective stress path of the samples sheared on remolded samples are shown in Fig.4.

CONCLUSIONS

Based on the consolidated undrained triaxial tests on undisturbed and remolded samples of Hindagala over-consolidated clay, it can be concluded that the Hindagala clay has a variation of undrained shear strength showing a minimum value when compressed at 45° to the plane of consolidation at high OCR. The remolded samples showed a minimum undrained shear strength value at an angle of 15° to consolidation plane. This suggests a lesser influence of induced anisotropy on the undrained shear strength for remolded over-consolidated Hindagala clay. However, justification of this conclusion requires further study.

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