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CONTENTS – PROJECT DAY 2004 - 1

IMPROVEMENT OF STRENGTH AND STIFNESS CHARACTERISTICS OF PEATY CLAY BY ELECTRO-OSMOSIS

Perera H.A.C., Weerawardana W.J., Kumari A.T.K.T.D.
University of Moratuwa, Sri Lanka

Abstract

Electro-Osmosis (EO) Consolidation had been applied successfully for the improvement of soft inorganic clays. This project looks at the possibility of using the technique on Sri Lankan Peaty Clays. In the studies done in previous projects it was found that the EO treatment could improve the engineering properties of peaty clay. The EO technique has been tested in the laboratory on remoulded clay samples. The objective was to demonstrate the efficiency of the EO method and to understand the most effective voltage gradients and duration of treatments for Sri Lankan peaty clays. It was planed to find out variation in the improvements achieved in the clay properties due to the process such as polarity reversals.

1. Background

Improvement and stabilization of soft peaty clay remains a challenge facing Sri Lankan geotechnical engineers today. Excessive settlement and shear failure upon loading are the main difficulties associated with peaty clay. Therefore, some improvement of peaty clay is required in construction projects such as highways. Even though various methods for achieving improvement of peaty clay such as; preloading, deep mixing and vacuum preloading are available they have different disadvantages. EO consolidation process can overcome some of the difficulties encountered in the above methods. Furthermore EO has consolidation process does not cause shear failures and does not need any special machinery. (Housmann; 1990). It could be very helpful with extremely soft peat clays.

2. Previous Research Done at University of Moratuwa

The effectiveness of the EO technique in the improvement of engineering properties of peaty clay was studied by Sagarika et-al (2002). Although many studies were done with inorganic soils (Casagrande1948, Ling 2000,Lo et al 1991, Mitchel & Van 1997) there were no records of the use of this technique on organic clay or peat elsewhere in the world. Sagarika et-al (2002) conducted several test series to study different behaviours of EO process under different conditions.

Series 1: EO and Pre-Loading treatment were given to identical samples in the same frame to compare the improvements achievable in two techniques.
Series 2: Electrically treated samples (with and without polarity reversals) were subjected to Oedometer tests and improvements achieved were compared.
Series 3: Compared the effectiveness of different electrode types (Stainless steel/Copper).

Through these tests, it was found that both the stainless steel and copper electrodes are equally effective and that polarity reversal is not that effective. EO treatments caused considerable reductions in m_v, C_C, and C_α. It resulted in improvement in shear strength as well. Further tests were planned in this project to understand the; influence of duration of treatment and the effect of voltage gradient given during the EO treatment.

3. Basic Properties of Peat Used

A peat obtained from Colombo Katunayaka Expressway site was used in the study. The initial soil 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		1.69
pH		3.16
Organic Content		30.5%

Table 1: Basic Properties of the Peat Used

4.0 New Test Series

4.1 Stages of in Series 4

The difference in the level on improvement achieved in two samples; one tested for a shorter duration only till the rate of settlement diminishes and the other sample treated for a long period was compared, in this test Series.
In test series 4, three specimens were tested; two with Electro Osmosis treatment and the other with conventional consolidation with loading, unloading and reloading. Data of conventional consolidation test was used to compare the improvements achievable with pre-loading and EO. Table 2 presents the load increments used.

Series 4	Specimen 1	Initial consolidation with $10kN/m^2$ EO supplied with 10V D.C. for 7 days Polarity reversal done and test continued for 9 days
	Specimen 2	Initial consolidation with $10kN/m^2$ EO supplied with 10V D.C. for 7 days
	Specimen 3	Test with load increment of 10, 20, 40, 80 then unload to 10 and reload to 20, 40, 80, $160kN/m^2$

Table 2: Stages of Test Series 4

4.2 Stages of Test Series 5

This test series was planned with the objective of finding out the influence of the voltage gradient applied on the achieved improvements. Same electrodes were used in this test series as in the series 4. Three samples were prepared and different voltage differences, namely; 4V, 8V and 12V were applied and specimen was allowed to consolidate until dial gauge reading showed a constant value (i.e. till the rate of settlement diminishes)

Applied Potential Difference	Duration (Days)
Electro Osmosis consolidation under 4V	5
Electro Osmosis consolidation under 8V	11
Electro Osmosis consolidation under 12V SP1	4
Electro Osmosis consolidation under 12V SP2	13

Table 3: Stages of Test Series 5

5.0 Analysis of Series 4 Results

5.1. Reduction in Void Ratio, e

The data obtained from the loading increments of the conventional test and loading increments of the electrically treated peaty clay tested in the Oedometer are shown in Table 4.

Consolidation Pressure	Void ratio		
	Conventional	Electro osmosis SP 1	Electro osmosis SP 2
5	-	-	2.883
10	3.56	3.02	2.791
20	3.25	2.96	2.681
40	2.67	2.89	2.485
80	2.43	2.76	2.068

Table 4: Variation of Void Ratio in Specimens

The improvement achieved in primary consolidation characteristics can be also illustrated by the gradients in the e Vs log (σ) plot in Figure 1 for different specimens in Series 4. The electrically treated Peat has smaller gradients. The Specimen treated for a long time had the smallest gradient. The apparent Pre-Consolidation pressure is also greater for specimen1.

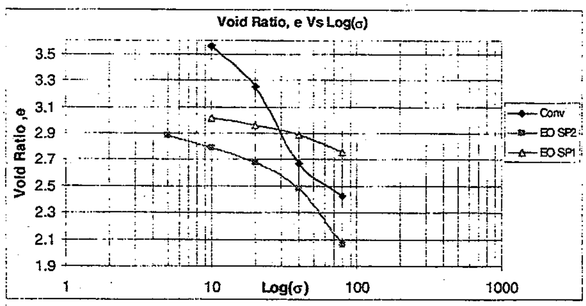

Figure 1: Comparison of Void Ratio Vs Log (σ) plots in Series 4

5.2 Reduction of Coefficient of Volume Compressibility, m_v

Table 5 presents the comparison of m_v values from treated and untreated samples and Figure 2 presents it graphically. It could be seen that the m_v values of EO primary treated peaty clay are much less than that of untreated clay. This indicates that the compressibility had been reduced due to the EO treatment.

Consolidation pressure (kN/m^2)	m_v (m^2/kN*10^{-3})			
	Conventional Consolidation Test-Series4 Conv SP3		Oedometer Test on Electrically Treated specimens	
	Load	Reload	Series4 EO SP1 Load	Series4 EO SP2 Load
10	19.22	-	0.38	1.45
20	6.56	0.798	0.43	0.88
40	6.21	1.043	0.28	0.78
80	1.32	0.342	0.25	0.83
160	-	0.72	-	-
C_c	0.9559	0.257	0.1361	0.4404
$C_c/(1+e_0)$	0.1751	0.0471	0.0335	0.1107

Table 5: Comparison of m_v and C

This specimen 1 that was treated for a long time showed a better improvement than specimen 2.

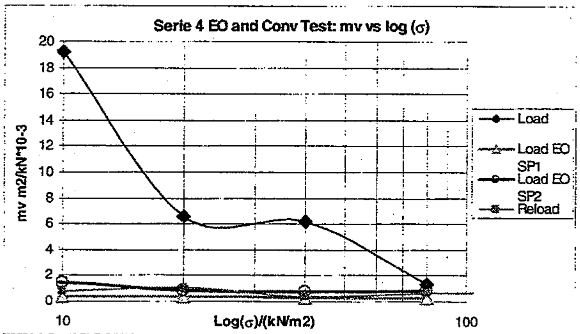

Figure 2: Comparison of m_v of Electro Osmosis and Preloading

The reduction in the primary consolidation settlements further illustrated through the C_c and $C_c/(1+e_0)$ values in the table 5.

5.3 Reduction in the coefficient of Secondary Consolidation, C_α

The Coefficient of Secondary Consolidation, C_α values for both treated and untreated clays are presented in table 6. The values graphically presented in figure 3.

Loading Stages In (kN/m^2)	C_α (m^2/kN) - Series 4			
	Conv SP3		EO SP1	EO SP2
	Loading	Reloading	Loading	Loading
10	0.11	-	0.004	0.03
20	0.1	0.01	0.005	0.015
40	0.05	0.02	0.007	0.02
80	0.05	0.06	0.009	0.002
160	-	0.06	-	-

Table 6: Results of Coefficient of Secondary Consolidation, C_α

It is evident that EO treatment has reduced the C_α values. The comparison of data from reload increments of conventional tests and loads increments of electrically treated clay indicate that level of improvements achieved in C_α are greater in the EO treatment than in the preloading.

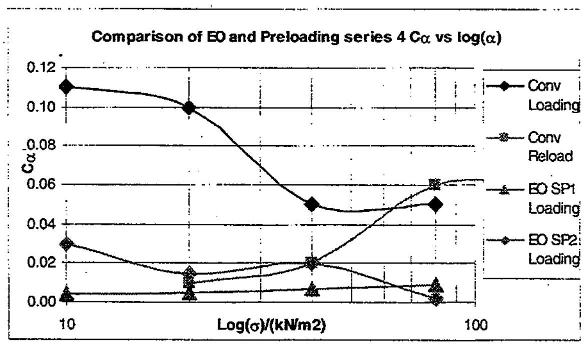

Figure 3: Comparison of Cα of Electro Osmosis and Preloading

5.4 Improvements in Undrained Shear Strength, C_U.

Improvements achieved in the undrained shear strength characteristics are presented in Table 7. Due to the smaller sample size only the Tor Vane test could be conducted. It was seen later that Tor vane test over estimated the shear strength improvement.

Series 4		Conv SP 3	EO SP 1	EO SP 2
Initial moisture content %		263.18	247.09	251.75
Final moisture content %	Top	128.0	173.54	138.45
	Bottom	179.8	188.27	212.95
	Average	153.87	180.9	175.7
Initial C_U (kN/m^2)		3.5	3.5	3.5
Final C_U (kN/m^2)	Top	153.4	108.53	91.87

Table 7: Undrained Shear Strength Values in Series 4

6.0 Analysis of the results of the Series 5

Identical specimens of peaty clay were used for Series 5 as well. This test series was conducted to study the effectiveness of different voltage gradients. Specimens obtained from the electrically treated samples with voltage difference 4V, 8V and 12V were subjected to consolidation testing in a conventional oedometer and data on e, mv, and C_α were gathered. Two specimens were tested for 12V voltage difference due to some inconsistencies in the results. In each test, loading is done to 80kN/m^2 from 10kN/m^2

6.1 Reduction in Void Ratio, e

The variation of void ratio observed in the tests are given in Table 8 and graphically presented in Figure 4. It could be clearly seen that higher voltage gradients gave lower void ratios and gradients in the graph.

Consolidation pressure (kN/m2)	Void Ratio, e			
	4V	8V	12V SP1	12V SP2
10	3.532	3.377	2.872	2.996
20	3.365	3.246	2.819	2.988
40	3.050	3.950	2.676	2.978
80	2.566	2.828	2.501	2.856

Table 8: Comparison of Void Ratio,e

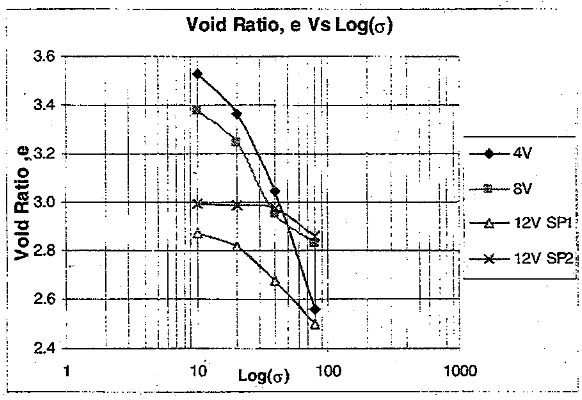

Figure 4: Comparison of Void Ratio Vs Log (σ) plots in Series 5

6.2 Reduction in Primary Consolidation Settlements

The variation of m_v with stress levels is summarized in table 9 and graphically illustrated in in Figure 5.

Consolidation pressure (kN/m2)	m_v (m^2/kN*10^{-3})			
	4V	8V	12V SP1	12V SP2
10	6.52	3.4	2.89	2.07
20	3.44	2.89	1.33	1.03
40	3.25	3.26	1.79	0.13
80	2.49	0.68	2.49	0.75
C_C	1.07	0.6082	0.4102	0.1029
$C_c/(1+e_0)$	0.2206	0.1342	0.4537	0.0381

Table 9: Comparison of m_v and C_C – Series 5

Data in Table 9 showed that the m_v values are varying according to the voltage gradient applied during treatments. Lower m_v values could be achieved with higher voltage gradients. That implies that better improvements could be achieved by providing higher voltage gradients. Similar observations were made by Ling et al (2000) with soft inorganic clays. The reduction in the primary consolidation settlements are further illustrated by compression index, C_C values and compression ratio, $C_C/(1+e_0)$values in Table 9.

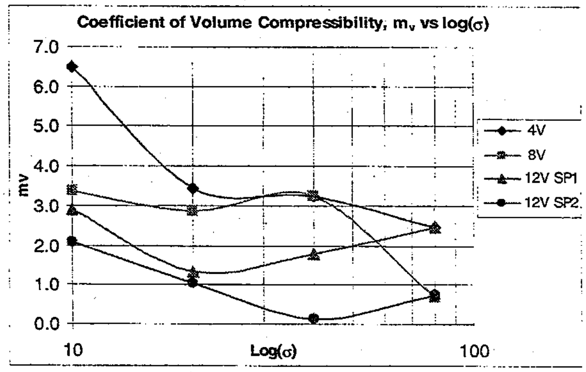

Figure 5: Comparison of m_v Vs Log (σ) plot in Series5

6.3 Reduction in, Cα

The variations of Cα with stress levels obtained in series 5 are presented in the Table 10 and Figure 6. It is clearly seen that the Cα values have decreased with the increase of applied voltage gradient. Therefore the data indicate that the improvements achieved in C_α increases with the applied voltage gradient.

Consolidation Pressure (kN/m^2)	C_α ($m^2/kN*10^{-3}$)			
	EO SP1 Series 4	EO 4V	EO 8V	EO 12V SP2
10	4.0	8.2	5.0	5.2
20	5.0	9.6	3.0	8.0
40	7.0	11.2	4.2	1.0
80	9.0	13.0	3.0	9.6

Table 10: Summery of C_α – Series 5

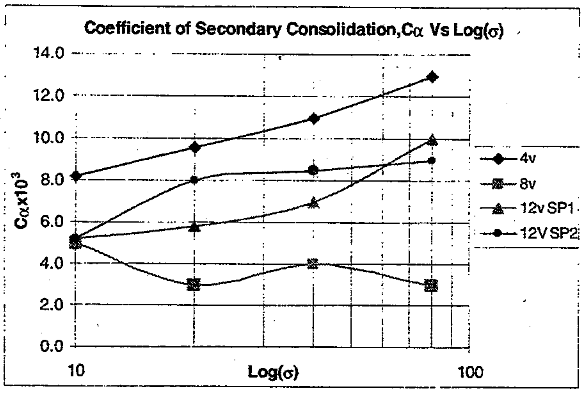

Figure 6. Variation of C_α With Different Voltages

6.4 Improvements in Undrained Shear Strength, C_u

In the Series 5, Shear Strength of the treated sample were evaluated by performing the Lab Vane Shear test on the consolidated specimens. Variations of undrained shear strength of the tests conducted under series 5 are presented in Table 11. Lab vane shear apparatus gave more reliable values of C_u than the Tor Vane.

Series 5		EO-4V	EO-8V	EO-12V SP1	EO-12V SP2
Initial Water Contents %		224.27	223.94	270.49	275.88
Final Water Contents%	Top	201.92	145.02	175.64	179.34
	Bottom	211.75	184.08	177.02	183.01
	Average	206.84	164.55	176.33	181.17
C_U (kN/m^2)	Initial	3.1	3.8	4.2	4.4
	Final	51.4	67.2	63.5	71.5

Table 11: Summery of C_α – Series 5

7. Conclusions

The test results obtained in this study clearly showed that the EO process would be successful for Sri Lankan peaty clays. The EO treated peaty clay experienced a significant decrease in compression index. The coefficient of volume compressibility has also decreased by EO treatment. Hence it is concluded that primary consolidation characteristics can be improved by providing an EO treatment

It further showed that due to the EO treatment the secondary consolidation coefficient also has decreased. The improvements achieved were greater than that achieved by preloading

The magnitude of settlements and the level of improvements appear to increase with the applied voltage gradient.

It is also found that EO caused a significant improvement in Undrained Shear Strength of peaty clay. The improvements achieved in Shear Strength by electro-osmosis are much greater than that achieved due to Preloading.

Based on this evidence it could be suggested that EO treatment be applied initially to improve extremely soft peaty clays encountered in various road projects in Sri Lanka. Even if the treatment is not applied for a long period it could be used to improve the peaty clay to a suitable level so that other methods such as preloading could be done without much of a problem.

8. Acknowledgments:

We are immensely grateful to our project supervisor Dr. S.A.S. Kulathilaka, Senior Lecturer, University of Moratuwa for the guidance provided through the project. Special thanks go to Miss D.K.N.S. Sagarika, post graduate student, Department of Civil Engineering, University of Moratuwa for her help in various stages in the project. The staff of the Soil laboratory and Work shop are also acknowledged for there support in various activities.

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FINITE ELEMENT MODELING OF SKIN FRICTION IN PILES

M.M.M.RINOOS, B.SASI KUMAR, J.H.S.S. JAYAMAHA
Department of Civil Engineering, University of Moratuwa

ABSTRACT

A vertically load pile in a homogeneous and a layered media is considered, and the load transfer from the pile to the soil is analyzed using the finite element method. An interface element is used to model the pile-soil interface, and the analysis is carried out assuming linear elastic behaviour in axi-symmetry. The shear stress distribution at the interface along the length of the pile and the axial load carried by the pile, as predicted by the finite element analysis program, are presented.

INTRODUCTION

Interaction between a pile and surrounding soil should be analyzed, if it is to be in the most comprehensive sense, by considering the deformability of both media (pile and soil) under load. Depending on the ability of the selected constitutive models to represent the behavior of the media concerned and the accuracy of the material parameters selected, this type of analysis will provide the most realistic solution to problems of soil-pile interaction.

With the advent of the computers and the Finite Element Method (FEM), solutions of complicated soil-structure interaction problems became possible. This numerical approach made it possible to solve interaction problems involving complicated geometries as well as non-linear constitutive models.

The problem of a vertically loaded pile in a layered medium is of significance to geotechnical engineers. The analytical approach to this problem resolves the resistance offered by the soil to the load on the pile into two parts; friction at the pile-soil interface and end bearing at the tip of the pile. When the layered soil medium has widely varying properties (e.g. harder sandy soil layers and very soft clay or peaty clay layers) the behavior of the skin friction (friction at the pile–soil interface) may show significant variation. This problem can most conveniently be solved using the FEM. In this work, soil and pile material are all considered as elastic materials (represented by appropriate equivalent elastic parameters) and the load transfer problem from as axi-symmetric pile to the surrounding layered soil is investigated using FEM.

OBJECTIVE AND SCOPE

The main objective of the project is to investigate the behavior of the load transfer through interface shear from pile to the surrounding soil medium.

The finite element program FEAP (Zienkiewicz, 1977; Kodagoda and Puswewala, 2002) is used for carrying out a parametric study. The program was verified against a problem where the analytical solution is known. For a range of interface skin friction values load transfer of an axially loaded pile in soil is considered. The shear stress distribution along the pile length (on the pile-soil interface) is derived and presented in a graphical form.

The finite element analysis is carried out using the property of axi-symmetry of the actual problem; thus there are no simplifying assumptions regarding the geometry of the actual problem, except that a fixed, far boundary is considered. However, material properties are taken as linear elastic.

The numerical study of this project is for demonstration purpose only, since a full parametric study would involve a large amount of analysis and results; this is because the individual layer thickness and material properties of each layer in the soil medium could vary over a wide range. Here one typical layered arrangement is considered and a limited parametric study conducted using this configuration.

FEATURES OF FEAP

The program FEAP can be utilized to solve one, two and three dimensional finite element analysis problems which may be linear or non linear. The program is written in FORTRAN computer language; it follows a modular concept, by which flexibility to change or modify various modules of the program without affecting the rest of the program is allowed. This enables the user to incorporate various additional features in the program, in the form of new algorithms or element subroutines. Another feature of the program is the macro programming language; under this, specifying a series of Macro commands performs the various stages of a finite element analysis. Each Macro command instructs the program to access a certain set of subroutines in order to perform a certain task.

Figure 1 represents the schematic structure of FEAP. The program can be basically divided into three parts as main program, system subroutines and the element subroutine library. Main program controls the overall memory allocation and the start and end of the analysis. System subroutines constitute the body of the finite element algorithms and control the input (data) and the output (results) of the program. Element subroutine library contains a number of element subroutines, each subroutine for a specific type of finite element. For example, subroutine ELEM14 (Element No. 14) is for plane stress / strain element under isotropic linear elasticity, ELEM13 is for 3 dimensional (solid) element under isotropic linear elasticity and ELEM15 is for axi-symmetry in isotropic linear elasticity. This arrangement allows the users to write their own element subroutines to solve their specific problems.

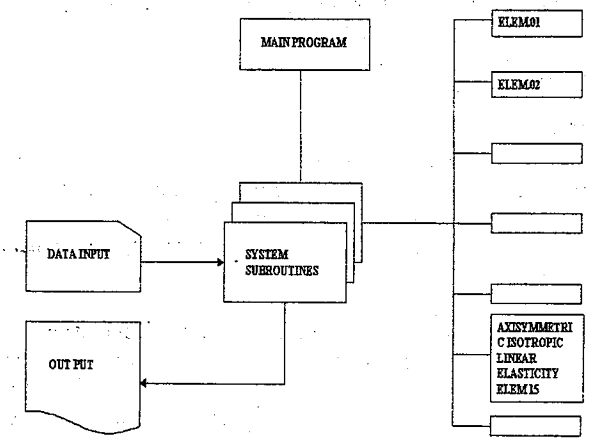

Figure 1 Schematic structure of FEAP

INTERFACE ELEMENT

An interface element had been introduced by Kodagoda and Puswewala (2002). This interface element is schematically shown in Figure 2 and is introduced along the pile-soil interface. An interface element has 4 nodes. Nodes 1 and 2 connect with the material on one side of the interface, and nodes 3 and 4 with that on the other side of the interface. Initially, the node pairs (1, 3) and (2, 4) will have identical coordinates.

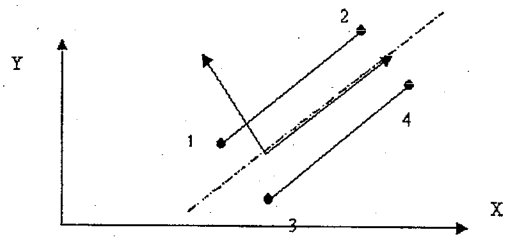

Figure 2 Schematic representation of the interface element

The constitutive model for the zero thickness interface element was based on a stress-relative displacement relationship (rather than a stress-strain relationship employed in the continuum elements). The relative displacement at a given location on the interface is defined as the difference in displacement undergone by two corresponding points on the two surfaces of the interface, which initially had identical co-ordinates (Kodagoda and Puswewala, 2002)

The elastic interface behaviour is defined as,

$$\sigma_s = C_s d_s$$

Where σ_s is the shear stress on the bond interface (stress parallel to the interface), d_s is the relative displacement parallel to he bond interface, and C_s a bond modulus for the adhesive strength in units of N/mm^2 /mm.

Stress-displacement relationship normal to interface can similarly be defined, but in the present elastic analysis, it will have no bearing on the results. The interface element was able to yield exact force balance in a simple finite element analysis problem where a steel rod was pulled out of some embedding material surrounding it. Thus its performance was verified.

FEM MESH, LOADINGS, BOUNDARY CONDITIONS AND MATERIAL PARAMETERS

First, a homogeneous soil medium was considered to be around a pile of diameter 0.2 m, and the problem was analyzed using a FE mesh consisting of 224 elements and 255 nodes, as shown in Figure 3. And then by using the same FE mesh geometry, a problem involving a layered soil medium around the pile was analyzed. This mesh is shown in Figure 4.

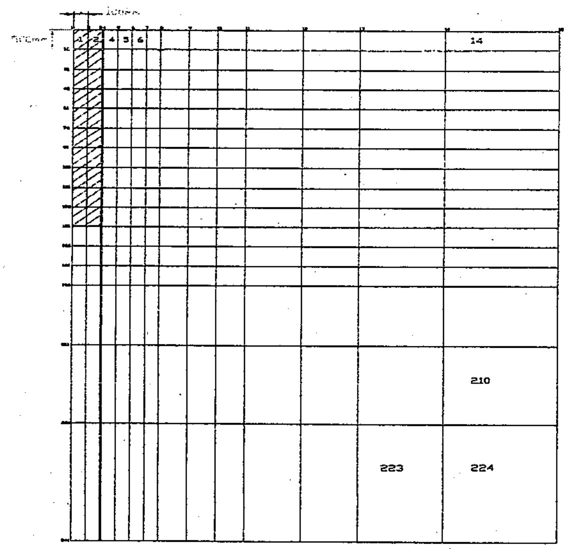

Figure 3- for the pile around with Homogeneous soil

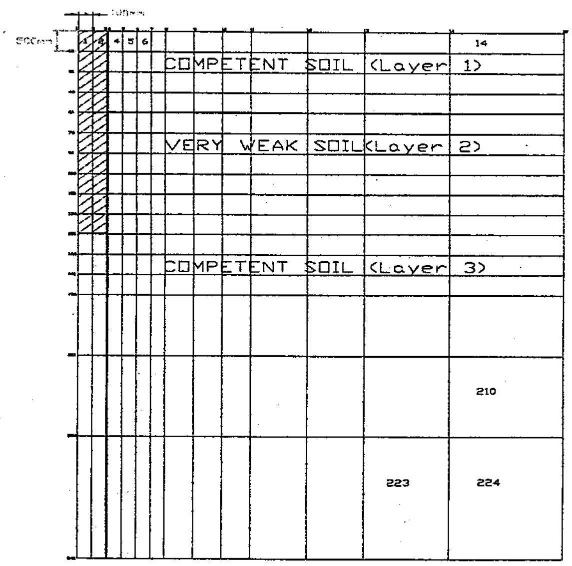

Figure 4- FEM for pile around with layered soil medium

The pile was always loaded by a uniform vertical (downward) pressure of 1MPa.

Following material parameters (E- Young's modulus; v- Poission Ratio) were used. The parameters are selected following Selvedurai (1979).

For homogeneous soil;

Ec (Concrete)	= 2.1E+04 KN/m^2
υ_c (Concrete)	= 0.20
Es (Soil)	= 2.1E+04 KN/m^2
υ_s (Soil)	= 0.25

For layered soil;

Ec (Concrete)	= 2.1E+04 KN/m^2
υ_c (Concrete)	= 0.20
E_{s1} (Soil layer 1)	= 7.5E+01 KN/m^2
υ_{s1} (Soil layer 1)	= 0.30
E_{s2} (Soil layer 2)	= 2.0E+00 KN/m^2
υ_{s2} (Soil layer 2)	= 0.20
E_{s3} (Soil layer 3)	= 1.2E+02 KN/m^2
υ_{s3} (Soil layer3)	= 0.30

Cs value is varied from .001 to .1 N/mm^2/mm

NUMERICAL RESULTS

Figure 5a illustrates the variation of axial force along the pile and Figure 5b illustrates the variation of shear stress along the inter-face (pile-soil), of the homogeneous soil media under the action of a vertical load, as the Cs value of interface elements is varied while all other soil parameters are kept constant.

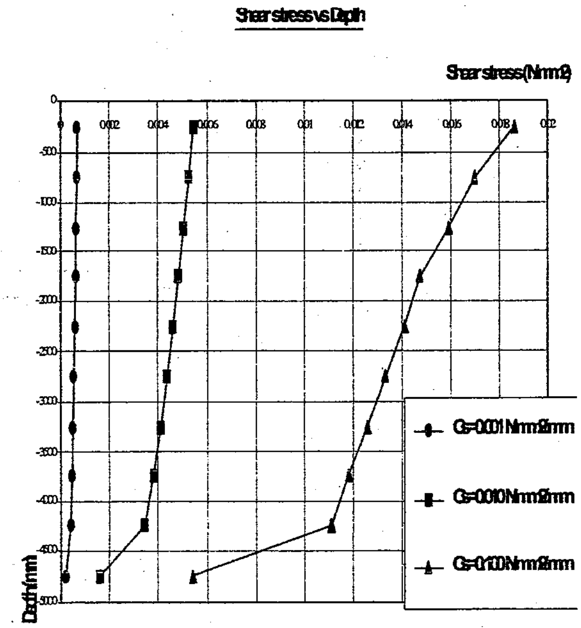

Figure 5b; Shear Stress Vs Depth for homogeneous soil

Same procedure was repeated for the layered soil media. For various Cs values of interface material (soil-pile), axial force and shear force were found using FEAP for a layered medium.

Variation of the axial force is presented in Figure 5c and the variation of shear stress is presented in figure 5d along the interface. FE mesh for layered soil medium is shown in Figure 4

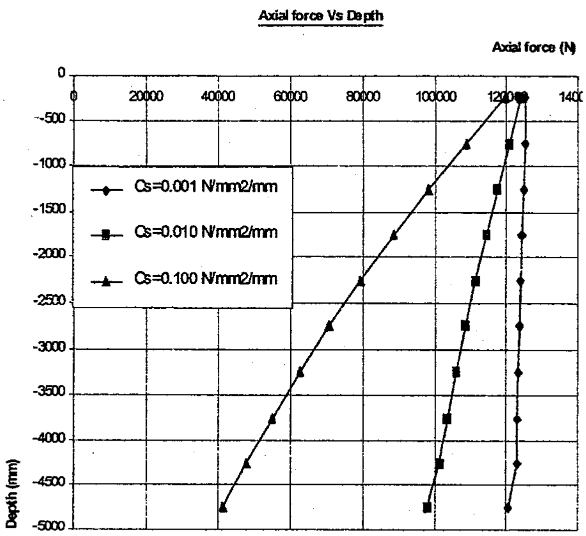

Figure 5a; Axial Force Vs Depth for homogeneous soil

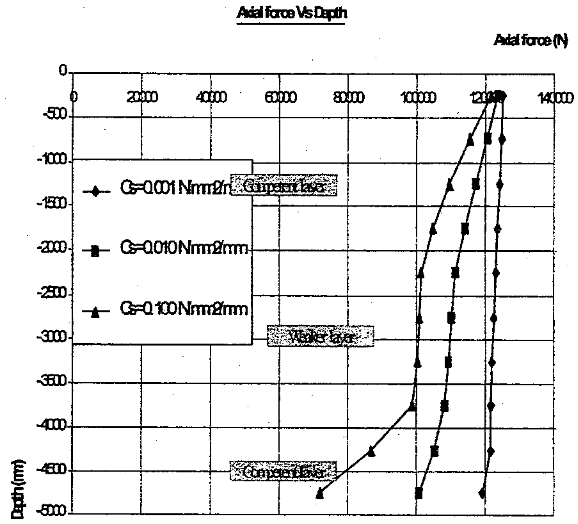

Figure 5c; Axial Force Vs Depth For Layered Medium

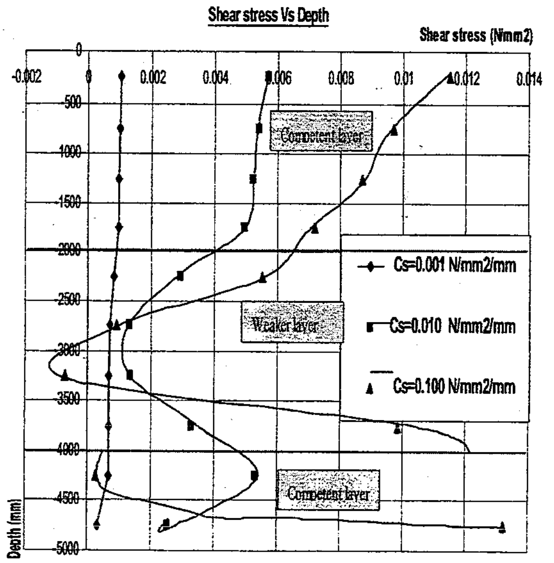

Figure 5d; Shear Stress Vs Depth for Layered Medium

DISCUSSION

The analytical solutions have been derived considering the influence of the surface loads on a semi-infinite soil medium (i.e. boundaries of the influence region are at the infinity). But in the case of finite element analysis, boundary has to be fixed at a known distance away from the pile. Considering smaller elements and a lager influence region will minimize deviations in the numerical solution.

As mentioned before, analysis here is limited to linear elastic materials, and effects of consolidation, secondary creep, etc. are not included. Thus the results are a good indicator for the initial stress distribution and its consequences in the soil medium. As long as a good estimate of secant modulii can be made for the pile construction materials and soils, the predicted results here could be used in a wider perspective.

Figure 5a and 5b show that, in the homogeneous soil medium, Cs value of 0.001 N/mm^2/mm represents a smooth pile-soil interface, whereas for Cs = 0.100 N/mm^2/mm, the interface is rougher. For Cs = 0.001 N/mm^2/mm almost the entire applied load is transferred to the tip as base resistance (Figure 5a) and very little shear stress is developed on the interface. When Cs is increased to 0.100 N/mm^2/mm, only about 30% of the applied axial load is transferred to the base and larger shear stresses are developed at the interface.

The numerical results for the layered medium are presented in Figures 5c and 5.. There a weaker soil layer is sandwiched between two competent layers at top and bottom of the pile. Figure clearly shows a drop in the shear stress resistance at the interface in the middle region corresponding to the weaker layer. For the ease of Cs =0.100 N/mm^2/mm, Figure shows a negative shear stress developing in the weaker layer. This is a demonstration of the negative skin friction, which develops due to the large settlements of weaker layers, and exerts an additional downward force on the pile. For a smooth interface (Cs = 0.001 N/mm^2/mm), these are very little shear resistance developed on the interface. In Figure 5c, for the case of Cs = 0.001 N/mm^2/mm, the axial force remains nearly constant in the central part corresponding to the thickness of the weaker soil layer.

CONCLUDING REMARKS

Numerical results obtained indicate the load-transfer behaviour of a smooth soil-pile interface and a rougher interface. A large proportion of the applied load if transferred to the tip as base resistance in the case of the smooth interface, while this proportion is only about 30% for a rougher interface considered in the study, for the particular pile-soil system analysed. For a layered medium, the interface element is able to demonstrate the varying shear stress development on the pile wall and the development of negative skin friction when the pile is in contact with layers of weaker soil.

Only a limited parametric study was undertaken in this work, in order to demonstrate the versatility of the interface element used. Also, materials are considered as linear isotropic elastic. A more detailed investigation involving a wider range of material parameter is preferable to further study the behaviour of piles in layered media. In this aspect, comparison with some available test results is also preferable.

ACKNOWLEDGEMENT

We are greatly indebted to our project supervisor, Dr.U.G.A.Puswewala for his invaluable advice, guidance, criticism and suggestions given in carrying out the project to successful completion. His ready available and easy accessibility at all the time was a great encouragement to us.

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COMPUTER AIDED LEARNING IN SOIL MECHANICS LABORATORY EXPERIMENTS

K.Pratheepan, S.J.Pratheepan and D.De.S.Udakara
Department of Civil Engineering, University of Peradeniya.

ABSTRACT

A program was developed according to BS1377 (1990) using Visual Basic 6.0 to computerize the learning of soil mechanics experiments in a user friendly environment. This has a main menu from which the required experiment can be selected, and from the experiment menu introduction, theory, apparatus, input/data observation, calculation, graphs, results and the application windows can be browsed. Using these windows graphs and results can be obtained very quickly and accurately. Since it is in windows format any one can easily understand and use this program. Therefore a person can perform many experiments and obtain the results quickly. This will lead to good designs. This project deals not only the calculations but also with other. learning concepts of the experiment. Therefore a student or any one interested in soil mechanics experiments himself can refer this and study about the experiments. A completely developed program for all experiments can be up loaded to the web and can be made accessible. Since the programs were developed using a user friendly programming language of Visual Basic 6.0 a person who refers this can also understand the program codes easily and can make improvements to the programs.

INTRODUCTION

The Computer Aided Learning in Soil Mechanics is a computer program specially developed to assist with the teaching of Geotechnical Engineering to university students at under graduate level. This has been designed in such a way users are made to work interactively and are often required to provide a numerical input. In this way the users are involved in the solution process, ensuring that the concepts are reinforced. A help menu in provided which guides the user through the program.

As an initial step our predecessors developed a program for the soil classification tests. Here a program is developed for direct shear test and our successors are developing programs for compaction and consolidation. Which would enable to perform the calculations and graphs in a short period of time and accurately. If such programs are developed to all the experiments and link them all together to a main form will be very helpful to students and to those who are interested in the site investigation tests. The program was developed using Visual Basic 6.0 programming language.

PROGRAM DEVELOPMENT

In order to create an efficient program it is decided that this should meet the following criteria.

- It should be easy to use and should be user friendly.
- It should have adequate help facilities to encourage the students to

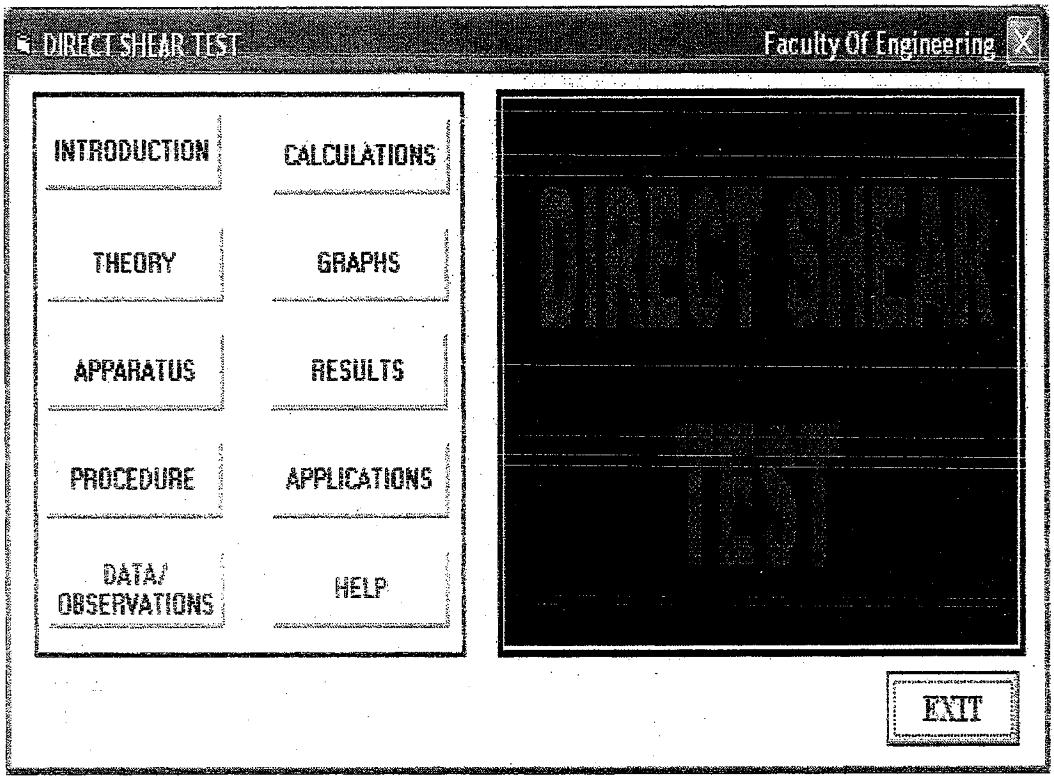

Fig1. Main menu window for the Direct Shear Test

use the soft ware in their own and to minimize the supervision by others.

- Since the program has elements of analysis associated with the tests the primary aim should be to enhance the students understanding of fundamental geotechnical Engineering problems.
- The program should be user interactive; i.e., the student should be encouraged to participate in the solution process.

To meet these requirements in the program for every test a general introduction of the test, figures of several types of apparatus, test procedure according to the BS1377(1990), calculation section, application of the results and the completed example tutorial are included. Also the program is developed in such a way that when a wrong action is performed it would be indicated and the right action would be displayed.

DISCUSSION

This project enables a student or any one dealing with soil tests to study about the tests and to perform relevant calculations easily and quickly. Since it is computerized in a user friendly environment students will tend to use this program. Once the observations of the laboratory tests are entered the program gives the calculations, graphs and results. Since it is in windows format it is easy to use by any person and the help menu also will help in using the program. Since the computer is used for calculations the results will be more accurate than manual calculations. Therefore usage of this program will be time saving and lead to more accurate designs. Since the programs were developed by a user friendly programming language; a student who has programming ability can improve this program or develop programs for the other experiments also and make it as a very useful software for geotechnical Engineering students.

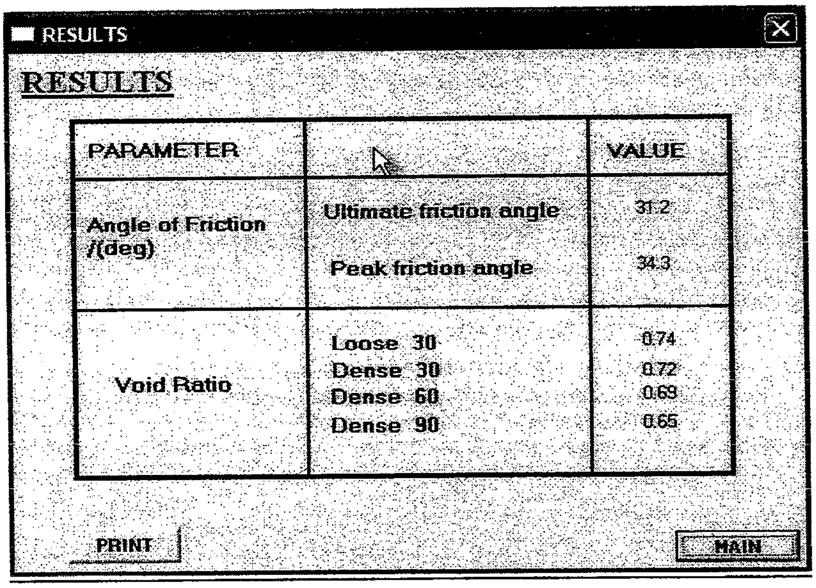

Fig2. Results window of the Direct Shear Test

CONCLUSIONS

This program will help to students, people interested in soil tests and even to Geotechnical Engineers to learn about the soil tests and to determine the appropriate tests according to the requirements. To perform the calculations quickly to obtain the accurate results and to come up with excellent designs.

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CONSTANT HEIGHT BEHAVIOUR IN DIRECT SHEAR APPARATUS

P.RAJEEV, M.UMAHANTHAN and H.N.SENEVIRATNE

Department of Civil Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka

ABSTRACT

A research was carried out to find the behaviour of shear strength of a soil sample when dilation was controlled. Shear strength of the soil, which can be found by different ways, is one of the most important properties. Although there are many tests, the Direct Shear Test is simple and easy to find the shear strength of a soil. When the direct shear test is carried out the volume of the soil sample will change with shear displacement. However in the case of Constant Height Behaviour of Direct Shear test, the volume of the soil sample will be remaining constant when shear displacement is given to the soil sample. In the usual direct shear apparatus height of the sample varies and therefore a suitable mechanism should be found to keep the height of the sample constant during shear. After modifying the apparatus, the behaviour of dense fine sand was found at constant height and this behaviour was compared with the results of conventional direct shear testing.

INTRODUTION

The term soil is used by the engineer and the agriculturist to refer to natural materials made up of separate grains not cemented together into solid rock. The engineer is interested in those properties which affect its ability to support loads in structures. These properties for soils are the same as for any other materials; i.e. Time-stress deformation relationships, ultimate strength, permeability etc. since the engineer usually cannot control the extent or the properties of these materials, he must control the loading condition.

Generally, the properties of soils can not be determined with as great a degree of accuracy as those of most other construction materials. Even though the properties were known for one sample of soil beneath an area, the properties of the entire soil affected would be only vaguely known, because material may varies over wide range in small area. Experience, imagination and judgement are of considerably greater important than mathematical analysis in the design of earth structures.

Even though many properties exist in soil one of the most important and complex engineering properties of soil is its shearing strength or its ability to resist sliding along internal surfaces within a mass. Shearing strength is the property, which enables soil to maintain equilibrium on a sliding surface, such as a natural hillside, back slope of a highway or railway cut, or the sloping sides of an embankment. One must understand the nature of shearing resistance in order to analyze soil stability problems such as bearing capacity, slop stability and lateral pressure on earth-retaining structure. Shear strength of a soil is not a function only of the material but is also a function of the stresses applied to it as well as the manner in which these stresses are applied.

$$\tau = f(\sigma)$$

Where τ-Shear stress

σ-Normal stress

The shear strength of a soil is usually determined experimentally by one of the following methods

- Direct shear test
- Laterally confined compression, known also as the tri-axial compression test and
- Unconfined compression test

The simplest and easiest of the plane shear test is the direct shear test. In this test the volume of the soil sample is changing with shear displacement and failure plane is predetermined.

The shear strength, which is measured by using the modified apparatus, is useful, when we consider the shearing occurred within materials that are in between the rock joints. That is why constant volume shear test is very important for engineers when they are working with huge masses undergoing shearing.

The current research involves two major components; (1) a study of a suitable mechanism to control the dilation in direct shear apparatus, (2) the evaluation of the characteristics and the magnitude of shear strength of soil sample under such condition.

EXPERIMENTAL SETUP

Initially the maximum load that should be needed to keep the soil sample in constant height has been found by using some trial tests in conventional direct shear apparatus. Then according to the maximum normal load all the components have been designed. Also all required design calculations have been done to select the proper material and size for each component. Thereafter all the designed components have been assembled as shown in the figure-1.

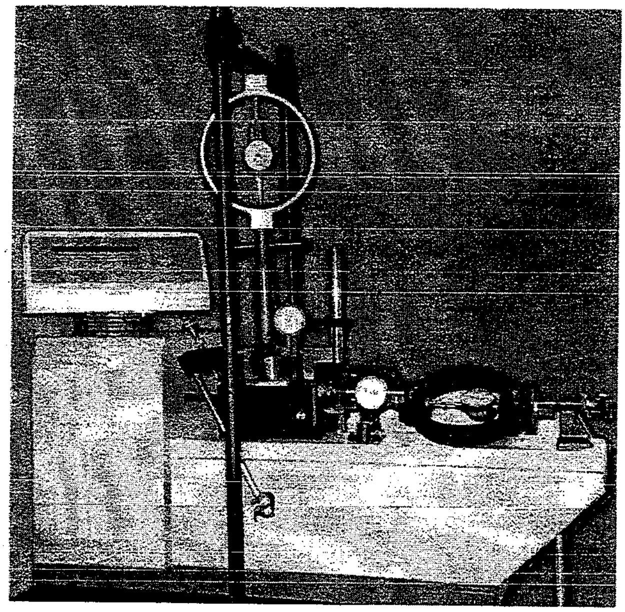


Figure 01

The direct shear apparatus consists essentially of a box divided into two sections. The lower section is placed on roller bearings, while the upper section is placed on top of other. Shearing will occur at the mid horizontal plane of the shear box that has area of 59.9*59.9 mm ^{2}. The movement of the shear box is measured by use of dial gauges.

The proving rings which are used to measure the normal forces and shear force have the proving ring constants 2.49 N/Div and 2.05N/Div respectively

SAMPLE PREPARATION AND TESTING PROCEDURE

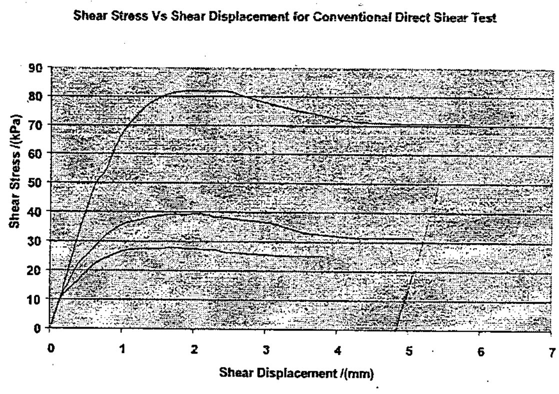
The preparation procedures depend on the type of the soil that will be tested. According to the BS 1377:Part7: 1990, the size of the largest particle shall not exceed one-tenth of the height of the specimen. Sieve analysis has to be done to get the fine sand.

In the case of Dry sand, prepare a quantity of soil somewhat larger than that required for the test specimen. Place the sand evenly into the shear box and apply a controlled amount of tamping with the square-ended tamper to achieve the desired density.

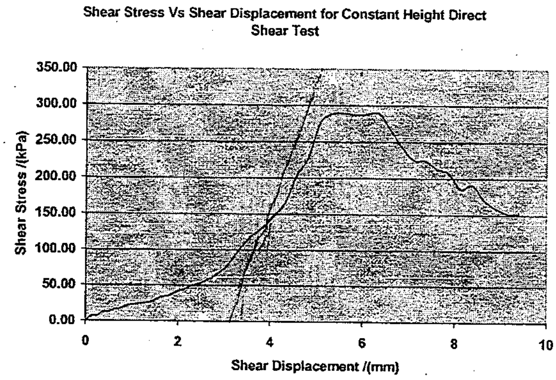
After preparing soil sample as specified it is placed in the direct shear apparatus. After that shear displacement will be applied to the shear box at the rate of 0.2 mm/min. When the soil sample is shearing vertical displacement will occur, that will be observed by using normal dial gauge. When the vertical dial gauge reading started to increase by using the screw jack that vertical moment will be controlled.

This procedure will be repeated until the failure stage of the soil sample. And also, the vertical load that was used to control the dilation, shear displacement and shear force will be recorded.

RESULTS



Graph 01

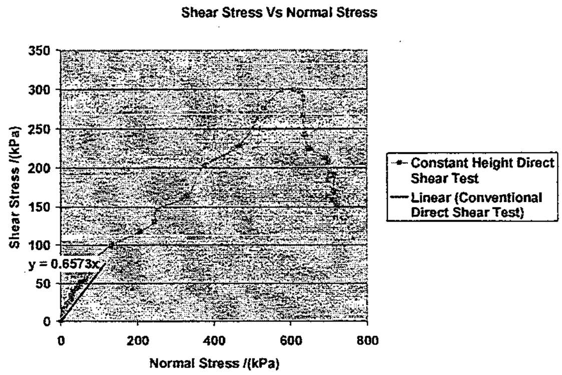


Graph 02

Over all result

State of apparatus		Failure shear strength /(kPa)	Shear displacement at failure /(mm)	Maximum normal stress /(kPa)	Void Ratio
Modified direct shear apparatus		289.05	6.35	722.9	0.524
conventional direct shear apparatus	30kg	27.8	1.78	39.12	0.615
	60kg	39.52	2.03	76.89	0.567
	90kg	82.05	2.16	114.66	0.556

Table 01



Friction angle (ø): Peak = 33.32°

Cohesion (C) = 0

DISCUSSION

Form the result it is clear that the failure shear strength of the soil sample depends on the normal load. When the normal load is high, the shear strength is also high. The shear strength of the soil is 39.12kPa, 76.89kPa & 114.66kPa when the normal load is 30lb, 60lb & 90lb respectively. In the case of constant height test failure shear strength is 289.05kPa.

The failure shear strength of the soil sample in the case of the constant volume is very high. Also the shear displacement at failure in constant height direct shear test is greater than the normal direct shear test.

ACKNOWLEDGEMENT

We wish to sincerely express our gratitude and thanks to our project supervisor Prof.K.G.H.C.N.Seneviratne for giving this project to us and for his interest, advice, guidance and encouragement. Next we like to thank Dr.H.J.Edirisinghe,the coordinator, providing instruction and necessary materials to complete the project. And we would like to thank Dr.Kamal Dissanayake for giving advice and encouragement to fulfil the project.

This project arrangement mainly depended on mechanical design. So that most of the works carried out at the workshop. Therefore we wish to thank workshop Engineer and technicians who are working in the Engineering Faculty workshop.

Then our thanks go to all the technicians of Geotechnical laboratory who helped us in doing the receptive tests and for doing all necessary jobs regarding the project.

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FINITE ELEMENT ANALYSIS OF GROUND IMPROVEMENT BY SOIL REPLACEMENT AT SHALLOW DEPTHS

Raveendiraraj A, Ravindran S and Seneviratne H.N.
Department of Civil Engineering, University of Peradeniya, Sri lanka

ABSTRACT

Every super structure is borne ultimately by the Earth. So the soil, beneath a superstructure should be able to carry the stress exerted on it. In case of weak soil, beneath the foundation, it should be improved through what ever the methods of ground improvement suitable. Here Ground improvement is done by replacing the stiffer soil (than existing one) of different fill of volume. The effect of ground disturbance due to replacement is modeled using a smear zone. To represent the smear zone an intermediate layer was considered. This paper discusses an analytical project conducted to find the optimum fill of volume and configuration of ground replacement carried out by replacing part of an existing soil by stiffer soil. The computer software AFENA-5 is used to find the optimum arrangement of soil replacement. Basically, three shapes of fill volume were considered varying the breadth to depth ratio. In the first series of analysis existing soil, replaced soil and mixing layer were modeled as an elastic material. In the second series, all three layers were modeled as elasto- plastic materials. In both cases settlement behavior with the depth and width and shape of fill volume were analyzed and plotted. In the case of elasto-plastic analysis, bearing capacities were found and with a factor of safety of 2.5, the settlement analysis was carried out and the optimum arrangement was found out. Optimum configuration of fill volume for elastic analysis is X/B = 2 and D/B = 1 and for elasto-plastic analysis X/B = 2 and D/B = 2. Here B = Width of continuous footing, D and X are the depth and width of the replaced soil volume.

INTRODUCTION

Generally, all civil engineering structures are supported by the earth. So, soil must be capable of carrying the loads from any engineering structure placed upon it without shear failure and with resulting settlement being tolerable for that structure. Due to the increasing ground usage, engineers have to improve the existing soil for the large scale construction on weak soil. A significant increasing in bearing capacity may be achieved by altering the soil properties Φ, C and γ. The improvement of soil parameter may be done in number of ways, such as compaction, drainage, preloading, grouting, chemical stabilization, use of geotextiles and soil replacement. Soil replacement is the most popular method in Sri Lanka.

LITERATURE REVIEW

According to the 'Terzaghi' (M.J.Tomilson, 1995) definition,

D = Foundation depth from the bottom footing

B = Width of the footing

If D/B ≤ 2 then that will be the *'Shallow foundation'*.

If D/B ≥ 4 then that will be the *'Deep foundation'*.

In the case of shallow foundations , the mean of support is usually either a footing , which is often simply an enlargement of the base of column or wall that supports or a mat or raft foundation in which number of column are supported by a single slab.

Footings may be classified as Individual, Combined, Continuous and Strap footing

The type considered here is continuous footing.

The foundation is designed on the basis of two essential characteristics known as bearing capacity and settlement.

Bearing capacity refers to the ability of a soil to support a foundation and structure. The ultimate bearing capacity of a soil refers to the loading per unit area that will just cause shear failure in the soil. There are number of mechanisms of failure available to work out the ultimate bearing capacity of the foundation. Basic mechanisms are two triangular and three triangular mechanisms. Terzhagi developed expressions for the bearing capacity for several types of footings. The bearing capacity for continuous footing is;

$q_{ult} = CN_c + \gamma D_f N_q + 0.5\gamma B N_\gamma$

q_{ult} – Ultimate bearing capacity

C - Cohesion of soil

N_c, N_q, N_γ – TERZHAGI'S bearing capacity factors

γ - Effective unit weight of soil

D_f - Depth of footing or distance from ground surface to the base of footings

B - Width of continuous or square footing

R - Radius of circular footing

$N_q = e^{(\pi\tan\Phi*(\tan(45+\Phi/2))^2}$

$N_c = \cot\Phi*(N_q-1)$

TYPE OF ANALYSIS

The structure, built on soil induces stress beneath the foundation as a result of that, the soil is subjected to some strain. This is called the settlement. The settlement should be tolerable for any structures.

The total settlement comprises three components.

1. Immediate settlement (δe)
2. Primary consolidation settlement (δc)
3. Secondary consolidation settlement (δs)

The calculation of δ_c based on the assumption, that the settlement is the function of net pressure increase and it can be found from the formula given below;

$\delta_c = \int m_v \Delta\sigma dz$ m_v=coefficient of volume compressibility

In elastic analysis, the material is considered to be isotropic elastic. The load is proportional to the displacement, i.e. material obeys the Hooke's law. The elastic stress-strain behavior may be linear or non- linear. Two elastic properties are enough to define the material and there will be unique solution for a particular load. The principle of superposition can be used in the analysis. If the solution for a point load is known, then stress or strain of a point due to more than one surface load, moment etc can be obtained by super position.

In plastic analysis, the material is assumed to be deformed elasto-plastically. The stress- strain relationship is non-linear and irrecoverable. It is necessary to analyze the model using an incremental loading. The strain increment direction is pre defined (In the direction of principal stresses) in the plastic analysis. In the case of incremental loading, the stress and displacement at any time can be found using equilibrium equation and the constitutive equations considering the boundary conditions. On this basis, a global stiffness matrix can be formed using finite element concepts. This stiffness matrix is used to calculate stress, strain and other parameters for next incremental loading. This procedure considers the yield function, used by the computer program.

METHOD OF ANANLYSIS

Engineering problems are analyzed in three main ways known as analytical, numerical and simulation methods.

Numerical method is widely used nowadays to solve complicated engineering problems. Finite element and finite difference methods are the main methods used.

In finite element technique, the domain within which the particular problem is to be investigated, is subdivided into a number of discrete areas of elements and within each of these elements, it is assumed that certain material properties will be constant and certain variablcs vary in predefined fashion. Adjacent elements are interconnected at nodal points. In the case of an analysis to determine displacement and stresses within a domain, displacements are assumed to vary within each element according to a displacement function which satisfies the requirement for compatibility of displacement. Using this displacement function and the physical properties of the material bounded by the element, it is possible to compute expressions which relate displacements and force at the nodal points of the element.

A feature of the finite element method is that when modeling the non-linear stress-strain behavior of soils and including a yield criterion such as Mohr- Coulomb, it can indicate the progressive development of yield zones and so represent the progressive onset of failure.

DEFINITION OF PROBLEM

Soil was modelled as elastic and elasto-plastic material and finite element analysis is performed by using a computer program called AFENA – 5 to find the optimum configuration for the soil replacement. The replaced soil and existing soil are the two layers. In addition to that, an intermediate thin layer of thickness 0.25 units to represent smear zone between two soils also considered in this analysis. For that stress and displacement in single/multi layered soil beneath the foundation were determined. Three types of geometric shape (type I,II and III) of shallow strip flexible foundation with infinite length (fig 2),subjected to uniformly distributed vertical load is taken for analysis by replacing soil stiffer than existing soil and incorporating smearing zone introducing a intermediate layer. Type I and II were modified introducing smear zone. Analysis was carried out in undrained condition with Poison ratio of 0.49, internal friction angle zero neglecting the gravity effect of soil. Soil is modelled using homogeneous isotropic elements. In elastic analysis, existing soil, replaced soil and soil representing smear zone are modeled as linear elastic material with different stiffness. In the case of elasto-plastic analysis, three layers of soil are modeled as elasto-plastic material with different stiffness. The deformation of stress and displacements is based on elastic/ elasto- plastic theory. Settlements are immediate and consolidation settlement.

Rectangular eight nodded elements are used for finite element analysis. Symmetric behavior along central axis is taken. The thickness of intermediate layer is taken to be constant for all position and shape of fill volume.

CONFIGURATION OF THE SOIL REPLACEMENT

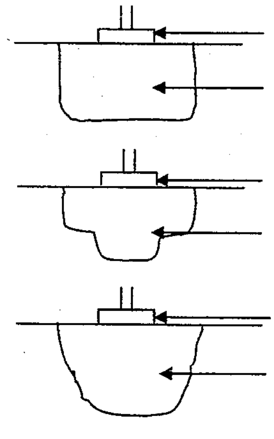

Fig 1

MODELLING OF THE SOIL REPLACEMENT

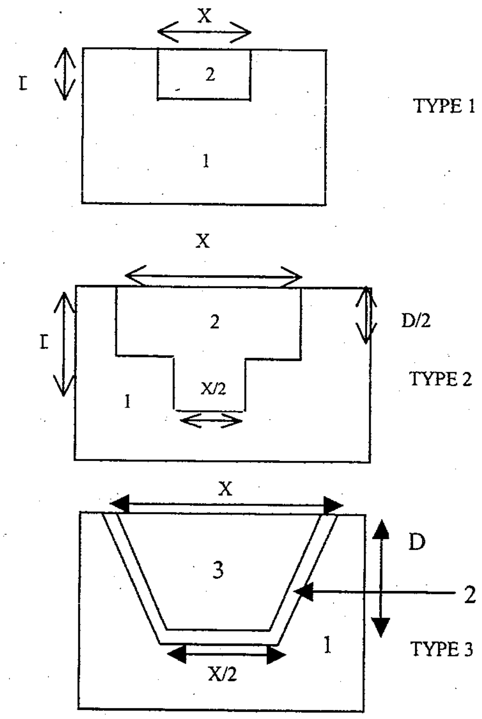

Fig 2

AFENA PROGRAM

The program AFENA (J.P.Carter and N.P.Balaam, 1990) is an algorithm for the solution of boundary and initial value problem using the finite element techniques. It is a general finite element program. The useful future is the incorporation of a macro instruction language, which can be used to construct specific algorithms. Features of the finite element module that is particularly relevant to geotechnical engineering networks.

Non – linear material model such as ideal solutions that obey both Mohr-Coulomb or Tresca failure criteria and that deform plastically and may flow according to associated or non- associated flow rules, fully drained, undrained and

consolidation analysis of soils that are represented by the modified cam- clay model, can be analysed.
FELPA, a post processor of AFENA is used to visualize the result graphically.
It is essential for economic computer processing kind to obtain a reliable solution to provide a good model of the physical problem. In finite element analysis this normally involves three phase known as Idealization, Discretization Node, element and numbering

FINITE ELEMENT ANALYSIS

Finite element solution will be more reliable, when the mesh is made finer and finer. However, we have to consider the time taken to solve the linear equation the memory of the computer and the mathematical instability. The elements subjected to much concentrated stress are subdivided to finer elements. This procedure is repeated until the solution obtained converges to a value. Here, the settlements at the bottom centre of the foundation are analyzed to get the suitable mesh. Ultimately mesh (Fig3) with 1600 elements and 4961 nodes were selected for this analysis. Since the problem is taken to be axis symmetric, only the half of the section was taken for this analysis.
For type 1and 2 automatic mesh generations, that available in the program, was used and for type 3, manual mesh generation was needed.

FINITE ELEMENT MESH USED FOR THE ANALYSIS

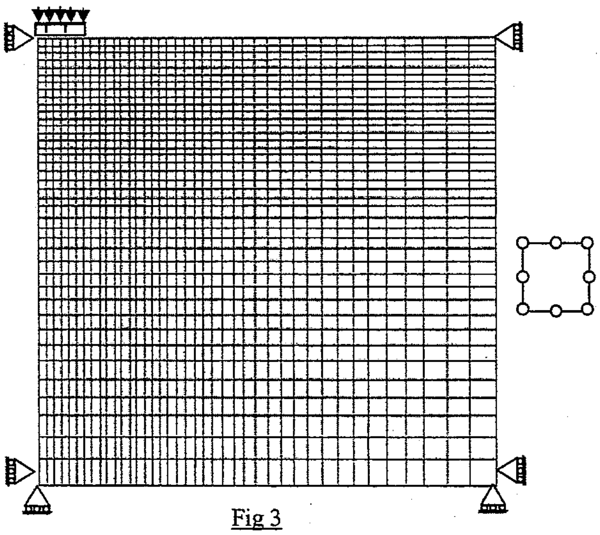

Fig 3

For the elastic analysis, a unit distributed load acting on a unit length of foundation is used for analysis. In the case of elasto- plastic analysis, loading increment of (E/200) units up to 5Cu is used to analyze elasto-plastic behaviour of the soil. Since the loading increment also affects the stress-strain behaviour of elasto–plastic material, for different increment loading was analyzed and plotted. Then for different arrangement of soil, the loading increment is applied and failure working load is found out and incorporating a factor of safety 2.5 settlement analyses was carried out.
The boundary was located so that the effect of the loading can be neglected near to the boundary regions. The bottom nodes were fixed from moving downwards and sideward. The far end side nodes were restrained from moving side wards and since the problem is taken to be axis symmetric, the nodes along the symmetric axes also restrained from moving sideward.
Three basic types of shape of fill volume of soil were taken for analysis as shown in fig.1.The existing soil was assumed to have 5000 units of young modulus and for replaced and smearing layer 25000 and 15000 units respectively. Since the condition is undrained, Poisson ratio is taken to be 0.49 for all the layers.
The analysis was carried out by varying X/B as 1, 2 and 4 and D/B as 1, 2, 3 and 4.

ELASTIC ANALYSIS

Since it is an elastic settlement, in analysis using AFENA-5 computer program, immediate settlement just beneath the foundation on the symmetric axis was taken as the settlement to the foundation.
Consolidation settlement can be calculated for elastic and elasto plastic analysis using the output file of AFENA program and the equation given below.
$\delta_c = \int m_v \Delta\sigma dz$
$\Delta\sigma$ & dz can be calculated, m_v can be taken as 1/E
The settlements relative to the settlements of existing soil are found at the bottom center of the foundation and plotted with varying D/B and X/B.

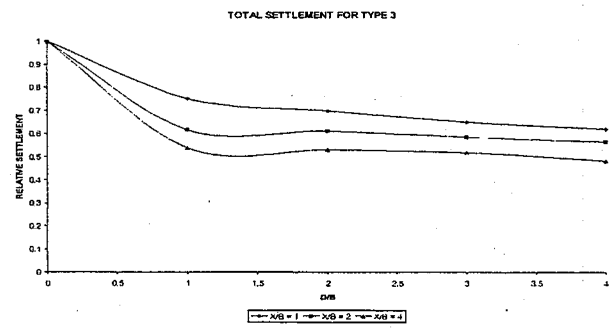

Fig 4

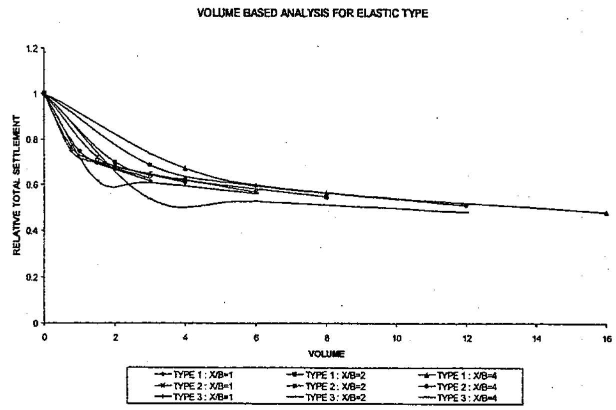

Fig 5

ELASTO PLASTIC ANALYSIS

In the elasto-plastic case, existing, replaced and mixed layers are assumed to be elasto-plastic material.Elasto plastic analysis was carried out by applying incremental loading of magnitude 25 units and the variation of P_{app}/P_{exist} vs. D/B were plotted varying the X/B as well as the configuration of the volume of fill. From the graph, failure load is found out by imposing a factor of safety 2.5 the working load is determined. The immediate, consolidation and total settlement were determined by applying corresponding working load to each type.

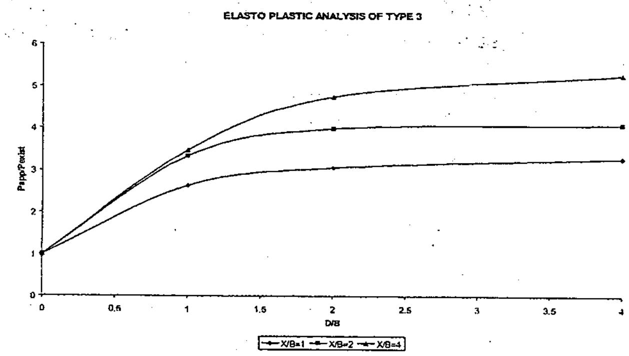

Fig 6

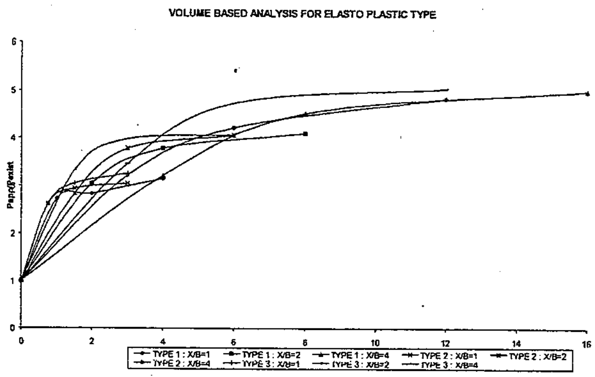

Fig 7

RESULTS AND DISCUSSION OF ELASTIC ANALYSIS

Type 1 (modified), type 2 (modified) and type 3 improves the settlement of soil varying from 25% to 52%.when X/B =1, their improvement differ from 25% to 33%.

From D/B =1 to D/B =2 7% of increment in improvement of settlement is achieved.

From D/B = 2 to D/B = 3 only 4% of increment is achieved. So D/B =2 could be selected as an optimum value for Type 1 (modified), type 2 (modified) since the increment of improvement is settlement decays when D/B increases.

For Type 1 (modified), type 2 (modified), the increment in the improvement of settlement after X/B =2 is not significant.

So X/B =2, D/B =2 could be taken as optimum arrangement for Type 1 (modified), type 2 (modified)

Fig 4 shows the variation of total settlement with D/B ratio for type 3.In this case, for X/B =2, X/B =4 the increment in improvement of settlement after D/B =1 is not significant.

At X/B= 2, 40% of improvement in settlement of soil is achieved and for X/B =4 only 6% increment could be achieved.

So X/B =2, D/B =1 could be taken as optimum arrangement for type 3

If we refer fig 5,the volume of fill needed for Type 1 (modified) is 1.5 times of type 2 (modified) .but the improvement in the settlement of soil differs only by 2-3%.so Type 1 (modified) is not an optimum arrangement.

Type 2 (modified) and type 3 needs same fill volume for a particular arrangement. But improvement in the settlement of soil for X/B =2 to X/B =4 are insignificant. Type 3 improves the settlement 10% more than type 2 (modified) having the same fill of volume at D/B =1.

RESULTS AND DISCUSSION OF ELASTO – PLASTIC ANALYSIS

In all cases there is no significant improvement in the relative bearing capacity after D/B =2.

In all cases there is no significant improvement after X/B =2. So X/B =2, D/B =2 could be concluded as optimum arrangement.

Although Type 1 (modified) improves soil better than Type 2 (modified), it needs more fill volume than Type 2 (modified) .so Type 2 (modified) is better arrangement than Type 1 (modified).

The fig 6 shows the variation of relative bearing capacity with D/B ratio for type 3. If we refer fig 7, type 3 improves the bearing capacity of the soil almost equally (sometimes more than Type 1 (modified)) to Type 1(modified).

So type 3 arrangement with X/B =2, D/B =2 can give an optimum improvement.

NOTATIONS

B	Width of continuous footing
C	Cohesion of soil
D	Depth of replacing soil
E	Young's modulus
m_v	Coefficient of volume compressibility
N_c, N_q, N_γ	Terzhagi's bearing capacity factors
Q	Point load
q_{ult}	Ultimate bearing capacity
R	Radius of circular footing
X	Width of the replacing soil
V	Poison ratio
δ	Settlement
δe	Immediate settlement
δc	Consolidation settlement
δs	Secondary settlement
σ	Stress
$\delta_z(x, y)$	Vertical displacement
$\Delta\sigma$	Increase in effective stress

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CONTENTS – PROJECT DAY 2004 - 2

A comparative study of measurement of soil matric suction using different methods

A. G. Jayasinghe, K. A. S. B. Kumaraarachchi, B. N. S. Fernando

ABSTRACT

Several experimental setups and procedures are being employed all over the world for the measurement of soil matric suction and developing soil water characteristic curve. This research deals with the results of an experimental study carried out on unsaturated soil samples to measure the soil matric suction with two different experimental setups, namely, the modified tri-axial apparatus (After Rathnajothy and Puswewala, 2001) and the pressure plate extractor (5 bar, Soil Moisture Corporation, California) with the intention of comparing results and identifying reasons for any deviations. For both testing procedures, samples of the same soil obtained from Pussellawa landslide area were used.

1. INTRODUCTION

1.1 Background

Over a number of years it has been observed worldwide, that the shear strength properties of slopes, which are stable in the dry season, tend to go down drastically in the rainy season resulting in landslides with unbearable damages to the properties and death to people recorded. In the dry season, the unsaturated zone present is associated with capillary movement of water up through the pores in the soil (soil matric suction). This results in a meniscus to form at the air water interface and negative pore water pressures to exist in the soil structure. The tensile stresses that develop along the meniscus results in an additional compression on the soil structure while the negative pore water pressure causes the soil particles to attract each other and make the soil structure stable as a whole. In the rainy season as the soil saturates, the capillary effect exists no longer and the structure becomes unstable.

Normal practice in geotechnical engineering is to work with saturated soil strength parameters. In areas where the water table is deep, use of saturated shear strength parameters may lead to underestimation of actual shear strength of the soil and the solutions would be uneconomical. Therefore it is important to understand the behavior of unsaturated soil and incorporate unsaturated soil strength parameters into geotechnical engineering designs where applicable. In the context of slope stabilization projects it is important to develop an understanding as to the range within which the FOS (Factor of safety) varies as the soil condition varies from fully saturated to dry condition. This enables the designers to use a low FOS while making sure that other precautions are taken to prevent saturation and to increase the rate of seepage.

1.2 Objective and the scope of the report

Several experimental setups and procedures are being employed in many countries worldwide for the measurement of soil matric suction. This research was carried out with the intention of studying some of those different methods available for the measurement of soil suction, and to develop the unsaturated soil shear strength function and the soil water characteristic curve (SWCC) for soil obtained from Pussellawa landslide site. For this purpose, the modified tri-axial apparatus (After Rathnajothy and Puswewala, 2001) and the pressure plate extractor (5 bar, Soil Moisture Corporation, California) were used. The results obtained from the two methods are compared and an attempt is made to identify the reasons for deviations.

2: COLLECTION OF SOIL SAMPLES, CLASSIFICATION TESTS AND INDEX TESTS

2.1 Collection of soil samples

The samples were collected from Pussellawa landslide located between road structures numbered 37/10 and 382 on the Kandy-Nuwaraeliya road.

2.2.1 Classification tests and Index test

Number of tests was carried out on collected soil samples from Pussellawa landslide

Maximum Dry Density (kg/m^3)	1605
Optimum Moisture Content (%)	20.5
Liquid Limit	37.80
Plastic Limit	27.43
Plasticity Index	10.37
Specific Gravity	2.65

Consolidated drained direct shear test results:

C'	20 kN / m^2
ϕ'	33°

3. MATRIC SUCTION MEASUREMENTS USING MODIFIED TRI-AXIAL APPARATUS

3.1 Modification of the Tri-axial Testing Apparatus (After Rathnajothy & Puswewala, 2001)

The existing standard tri-axial apparatus was modified appropriately by Rathnajothy and Puswewala (2001) to conduct tri-axial shear testing of the selected soil samples under unsaturated condition by introducing two new features, namely:

Pore-water pressure control:

A porous, ceramic disk that allows the passage of water but prevents the flow of free air placed at the bottom of the sample on the base pedestal.

Pore-air pressure control:

A controlled pore-air pressure line connected to the loading cap of the conventional tri-axial cell.

3.2 Sampling and testing procedure

In this research single stage tri-axial testing was used. Three test specimens of the dimensions of 50 mm in diameter and 112 mm in height were tested each under a predetermined set of stresses. Each specimen (soil passing No. 5 sieve) was compacted in the mould at 100% of the maximum dry density and optimum moisture content (as determined by Proctor compaction test). Two rubber membranes were placed around the specimen. The high air entry disk was sealed properly by applying vacuum grease along its periphery. The vacuum grease ensures that water flows only through the high air entry disk. In order to ensure that the suction applied to the specimen for testing was greater than the initial suction, moisture content of specimens was increased by adding water to the top of the specimen. Once the specimen was sufficiently imbibed with water, it was mounted in the tri-axial cell and O-rings were placed over the membrane to fit around the loading cap and the bottom of the pedestal. Each sample was allowed to reach equilibrium consolidated drained conditions under a predetermined set of stresses.

At the end of consolidation, the specimen was axially loaded at a constant displacement rate of 0.0025 inches / min., until failure by shearing, while maintaining the applied pressure σ_3, u_a and u_w at their selected magnitudes.

σ_3 (kPa)	u_a (kPa)	u_w (kPa)	$(u_a - u_w)$ (kPa)	$(\sigma_3 - u_a)$ (kPa)
250	100	50	50	150
300	150	50	100	150
400	250	50	200	150

Table 3.2.1: The set of stresses for tri-axial test

3.3 Tri-axial Test Results and Their Interpretation for Pussellawa landslide

The relationship between matric suction and apparent cohesion (Ho and Fredlund, 1982) for Pussellawa soil is shown in Fig.3.3.3. The average angle of friction with respect to matric suction (ϕ_b) is found to be 11.35 degrees.

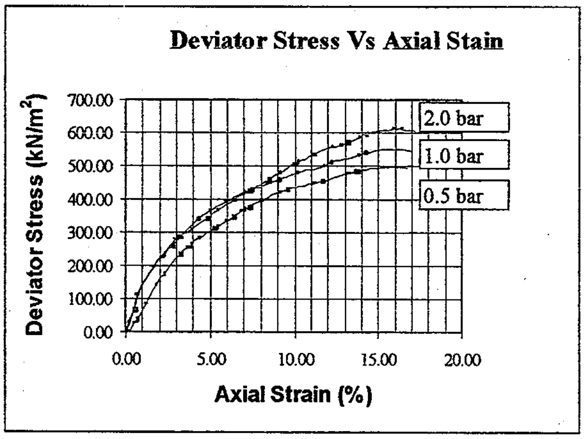

Fig. 3.3.1: Deviator Stress Vs Axial Strain

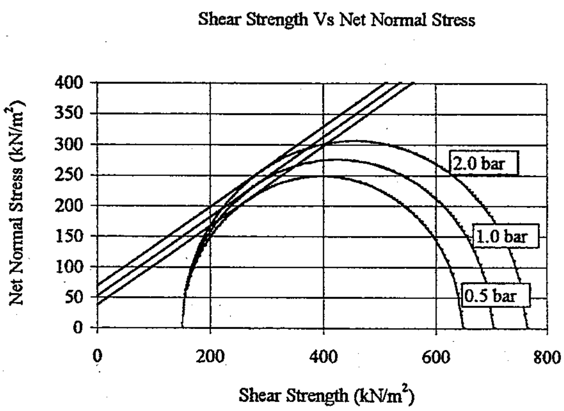

Fig. 3.3.2: Shear Strength Vs Net Normal Stress

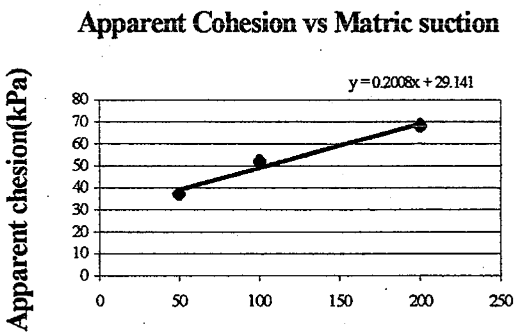

Fig. 3.3.3: Apparent Cohesion Vs Matric Suction

4. MATRIC SUCTION MEASUREMENT USING PRESSURE PLATE EXTRACTOR

4.1 The pressure plate apparatus

The 5 bar pressure plate extractor (Model 1600, Soil Moisture Corp, California), was used to measure the soil matric potential in the range of 0 to 5 bars.

The essential components of the pressure plate extractors are the following:

(a) The pressure vessel

(b) Ceramic pressure plate cell

(c) The pressure supplies and control system

4.2 Sample perpetration

1.18 mm sieve was used for the sample preparation. A group of five samples was tested for each matric suction value selected. To receive the soil sample retaining rings (10 mm height, 50 mm diameter) were used. The porous ceramic plate together with the samples were then allowed to stand at least 16 hours with an excess of water on the plate to make sure that the plate and the samples had reached saturation. At the end of the saturation period, the excess water from the ceramic plate was removed and the plate and the samples were then mounted in the extractor to proceed with the extraction process.

4.3 Testing procedure

- Dry weights of the retaining rings were obtained before the sample preparation.
- Samples for testing were prepared as described above.
- The saturated ceramic pressure plate cell and the set of samples held in position by the retaining rings with filter papers placed underneath the soil were mounted in the pressure vessel.
- The 6-inch length of rubber tube attached to the outflow stem of the ceramic cell was then connected to the outflow fitting of the pressure vessel.
- Pressure vessel lid was then placed in position and clamped with the special bolts and the wing nuts.
- The air pressure inside the vessel was then set to the appropriate value so that the desired matric suction value can be obtained.
- The pressures were maintained until the equilibrium was reached. A burette was connected to the outflow fitting of the pressure vessel with a piece of small diameter tubing to collect extracted water. The burette was read periodically so that the approach to equilibrium could be monitored. No measurable amount of change in burette reading over a period of many hours indicates the equilibrium state.
- The end of the outflow tube was sealed before releasing the air pressure in the extractor to prevent backflow of water to the samples after pressure is released.
- The extractor lid was removed and the samples were weighed as quickly as possible for the wet weight of the soil + filter paper + retaining ring.
- Part of each soil sample was collected to a container to determine the moisture content after oven drying at 105^0 C.
- Weight of the corresponding wet filter papers was also noted down.
- Volumetric water content versus matric suction was then obtained for the selected range of matric suction values.

5. COMPARISON OF TEST RESULTS AND CONCLUSION

5.1 comparison of test results and conclusion

Unsaturated soil shear strength function for Pussellawa soil obtained from modified tri-axial test is:

$\tau = 20 + (\sigma_n - u_a) \tan 33^0 + (u_a - u_w) \tan 11.35^0$

The soil water characteristic curves (the relationship between volumetric water content and matric suction) obtained from both testing procedure is compared below in Fig. 5.1.1.

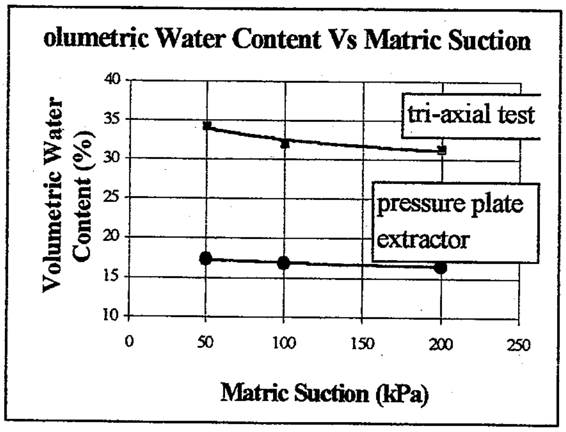

Fig. 5.1.1: Comparison of Test Results

The results obtained from both methods reveal a decrease in the volumetric water content when the matric suction is increased. In other words, the shear strength capacity of the soil increases as the matric suction in soil increases. However, the two sets of results deviate from each other considerably. For example, the volumetric water content corresponding to 0.5 bar matric suction is 34.24% when measured with the modified tri-axial apparatus, while it is 17.28 when measured with the pressure plate extractor. Number of factors can be considered to account for this difference in test results, and some important ones are listed below.

1. The difference in the degree of compaction of the samples
2. The method used to obtain the final volume of the samples
3. Errors introduced by the assumptions made etc.

For the tests carried out with the modified tri-axial apparatus, samples compacted at optimum moisture content (such that the maximum dry density could be achieved) were used while no mechanical compaction was done in preparing the samples for the tests carried out with the pressure plate extractor. Also, for the testing with modified tri-axial apparatus, soil passing through No. 5 sieve was used while soil passing No. 16 sieve was used for the tests carried out with the pressure plate extractor. Since the particle size distribution of the soil sample governs the pore sizes in the structure, and hence the capillary action, the two sets of samples may result in two different SWCCs.

To obtain the volumetric water content, the final volume of the soil sample was required to be found. In the case of modified tri-axial test samples, the sample was immersed in a known volume of water after applying grease and the displaced volume of water was taken as the volume of the sample. This method results in appropriately accurate volume calculation. In the case of the pressure plate extractor samples, the method does not provide any proper means of obtaining the final volume of the sample. Therefore an assumption was made that the compression occurred in the samples were negligible and the volume of the sample retaining rings were taken to be equal to the volume of the sample. However, this assumption brings an error in the calculations.

For the tests carried out with the pressure plate extractor, the atmospheric pressure was assumed to be equal to 1 bar. This assumption too introduces an error in to the calculations.

5.2 Suggestions

It is suggested that the matric suction measurement using the pressure plate extractor to develop the SWCC for a particular soil should be carried out with a more accurate method of sample preparation. It is further suggested to carry out research on the effects of the initial degree of compaction of soil samples on soil water characteristic curve.

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IMPROVEMENT OF PEATY CLAYS BY DEEP MIXING AND CORRELATIONS FOR SHEAR STRENGTH

S.Jegandan, T.Jothivel, K.Jegatheesan

Abstract

Peaty clays have very low shear strength and very high compressibility characteristics. Land areas that are underlain by peaty clay posses enormous problems to civil engineers. They may have to specify the use of pile foundations in the case of construction of heavy buildings. In the case of construction over a large area such as highways, pile foundations are not use of appropriate ground improvement technique could be more economical there. This paper presents laboratory study to study the improvements achievable by deep mixing in primary and secondary consolidation characteristics and shear strength properties in peaty clays obtained from Colombo- Katunayake Expressway project and Southern Highway project. The improvements achieved were compared with those achieved through preloading.

1. Background

A developing country demands a substantial amount of new infrastructure and buildings. These buildings and infrastructure may have to be built on whatever the existing land where the sub soil conditions may not be very sound. In an around Colombo and suburbs most of the land with good sub soil conditions have been already used and most of the new infrastructure facilities and buildings will have to be erected on areas underlain by soft soils such as peat.

Peat contains the remains of dead vegetation and organic matter in various stages of decomposition. Thus peat posses very high water contents, high compressibility and very low shear strength. Due to these inherent properties, peaty soils do not provide any favorable conditions for constructions on them.

If any constructionloads are to be transmitted to a peaty soil, the above mentioned undesirable properties would have to be improved to prevent potential disasters. The shear strength will have to be increased and the compressibility will have to be reduced. Alternatively, the structural loads could be transmitted to an underlying stronger stratum with the help of piled foundation. However, for infrastructure facilities such as roads occupying a large plan area and for moderately loaded buildings, provision of plied foundation will not be an economical solution. Srialnkan geotechnical engineers are compelled to find appropriate methods of improving the engineering properties of peat.

Research was carried at university of Moratuwa over the last five to six years on the application of preloading, deep mixing and electro-osmosis on the improvement engineering properties of peat. . It was found that deep mixing was successful in some amorphous peats, but was not effective with fibrous peats. As such it was necessary to see whether the technique was applicable to peat at different levels of decomposition found from different locations. The level of improvements achieved by the process of preloading, deep mixing and electro-osmotic consolidation were found to be at some order. This project continues those studies using peat samples obtained from katunayke expressway project and Southern Highway project.

2. Experimental procedure.

Peaty clay obtained from Colombo- Katunayake expressway project was mixed with stabilizer (from Korea) by weight percentages of 10%, 15% and 20% also with cement in percentages of 10% and 20%. The original water content of peat was 180.1%. Peaty clay obtained from Southern Highway project was mixed with cement by weight percentages of 10%, 15% and 20%. The original water content of this particular peat was 171.4%.

Peaty clays obtained from both sites were also remoulded without any addition of cement. All the samples were left to harden for six weeks in buckets under submerged conditions. Specimens obtained were subjected to consolidation tests to check the compressibility. In consolidation test samples were loaded through 10kN/m^2, 20kN/m^2, 40kN/m^2, to 80kN/m^2 in loading cycle and unloaded through 40kN/m^2 to 10kN/m^2. Then the samples were reloaded up to 160kN/m^2 through the same stress levels as in the loading cycle.

The purpose of reloading cycle is to simulate the behaviour of a preloaded peat. The behaviour in loading cycles of cement or stabilizer mixed peat, will illustrate the improvement by that technique. Reloading increments in those specimens illustrate the behaviour of peaty clay subjected to both types of improvements.

To study the improvement of shear strength in peaty clay due to deep mixing Unconsolidated Undrained Triaxial Tests were carried out. It was possible to do those tests on treated samples only. Untreated peaty clay samples were too soft and sample could not stand in the triaxial apparatus. UU triaxial tests were conducted at three cell pressures to check repeatability. Since all the softer samples could not be tested through UU triaxial tests, other shear tests such as laboratory vane shear test and Torvane tests were used to determine the shear strength both the treated and untreated peat. It is anticipated that the depth of penetration in the fall cone Liquid limit device and the number of blows required in the Casagrande's liquid limit device could have a relationship with the undrained shear strength of peaty clay. Therefore with the objective of finding correlations between the C_u values determined through UU triaxial test and other simple and indirect tests wherever possible these simple and indirect tests were conducted on treated peat. However, peats mixed with higher percentages of stabilizer where very stiff and lab vane shear and Torvane shear test could not be conducted on them.

3. Improvement of compressibility characteristics.

3.1 Parameters to be evaluated

In the study of the improvements achieved in consolidation characteristics due to deep mixing, the major characteristics to be evaluated are; Compression index (C_c), compression ratio ($C_c/1+e$) and coefficient of volume compressibility (m_v) in the case primary consolidation. Similarly the improvement in secondary consolidation could be assessed by the coefficient (C_α).

3.2 Improvement of primary consolidation characteristics

It was an evident from the results of the consolidation tests (Table1 & Table2) that both Compression index (C_c) and compression ratio ($C_c/1+e$) were reduced due to deep mixing. Void ratio vs log σ plots obtained during the simulated testing for pure peat obtained from Colombo- Katunayake Express way project and treated peaty clay (Peat +20% cement and peat+20% stabilizer) are presented in Figure 1.

The gradients of the loading curve of treated peats and reloaded curve of peat are of the similar order. This indicates that the primary consolidation characteristics were improved to similar levels by the two methods of treatment

The variation of coefficient of volume compressibility (m_v) with stress levels in loading cycle for different samples is shown in the Figure 2.

	Peat	Peat+ 10% stabilizer	Peat+ 15% stabilizer	Peat+ 20% stabilizer	Peat +10% cement	Peat + 20% cement
C_c	1.2469	0.7816	0.1320	0.1136	0.1709	0.1651
$C_c/1+e$	0.0304	0.0191	0.0074	0.0052	0.0042	0.0040

Table 1. Primary consolidation characteristics of peaty clay from Katunayake expressway project

	Peat+ 5% cement	Peat+10% cement	Peat+15% cement	Peat+20% cement
C_c	1.5108	1.3341	0.9942	0.4890
$C_c/(1+e)$	0.3905	0.3645	0.3005	0.1602

Table 2. Primary Consolidation Characteristics of peaty clay from Southern highway project

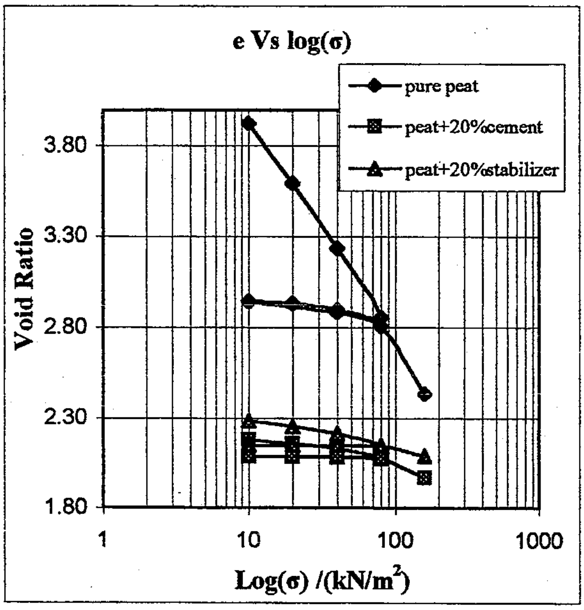

Figure 1. e vs log σ plots

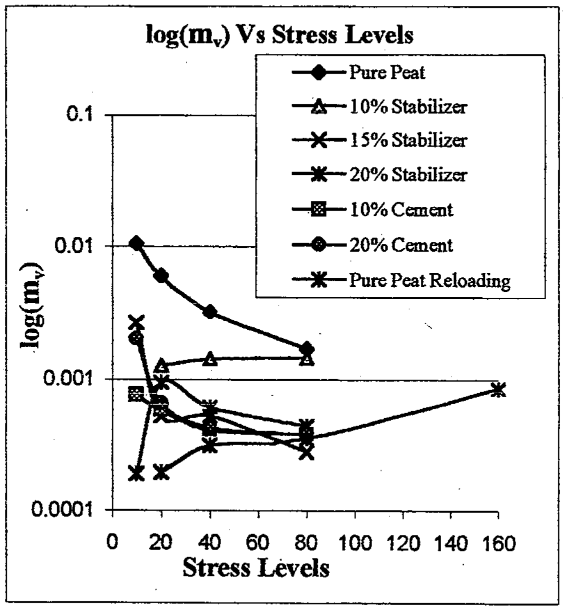

Figure 2. Variation of m_v with stress levels

3.3 Improvement of secondary consolidation characteristics

Comparison of the variation of the secondary compression index – Cα with the stress levels highlights the improvements achieved in secondary consolidation characteristics. Since the peats are known for very high secondary compression settlements, the coefficient of secondary compression index– C_α is an important property to be assessed. The Cα values can be obtained from e vs log t curve. Variation of secondary compression index C_α is shown in figure 3.

4. Improvement of shear strength characteristics

Undrained shear strength values (C_u) obtained from the treated samples of peat from Colombo Katunayake Express way project are presented in Table 3. The results of the samples obtained from Southern Highway project are presented in Table 4. It could be seen that with the mixing of the stabilizer and cement the Undrained shear strength of the peat improved considerably. The strength gains achieved with the stabilizer is much greater than that achieved with cement.

	Pure peat	Stabilizer			Cement	
Weight percentage	0	10	15	20	10	20
C_u by vane shear test (kNm^{-2})	4.50	16.65	Not possible	Not possible	14.64	Not possible
C_u by Torvane test (kNm^{-2})	3.81	8.72	Not possible	Not possible	8.16	Not possible
C_u by UU test (kNm^{-2})	Not possible	12.5	101.0	249.0	34.0	27.0
Number of blows	4	13	17	24	12	26

Table 3. Undrained shear strength peaty clay from Katunayake Expressway project

Percentage of cement	0	5	10	15	20
C_u by vane shear test (kNm^{-2})	3.60	3.73	5.50	8.00	8.90
C_u by Torvane test (kNm^{-2})	2.72	4.58	6.98	Not possible	Not possible
C_u by UU test (kNm^{-2})	Not possible	13.35	20.45	29.50	37.10
Number of blows	3	6	9	11	14
Depth of penetration (mm)	Not possible	Not possible	15.1	8.8	6.2

Table 4. Undrained shear strength peaty clay from Southern Highway project

5. Correlations for undrained shear strength

It can be seen that the Undrained cohesion obtained by the UU Triaxial tests and vane shear tests not identical. The results in Table 3 and Table 4 indicate that there appear to be a relationship between undrained cohesion C_u and the data from other tests. By carrying out more tests good correlations may be produced. The variation of C_u with number of blows in Casagrande's apparatus is shown in figure 4.

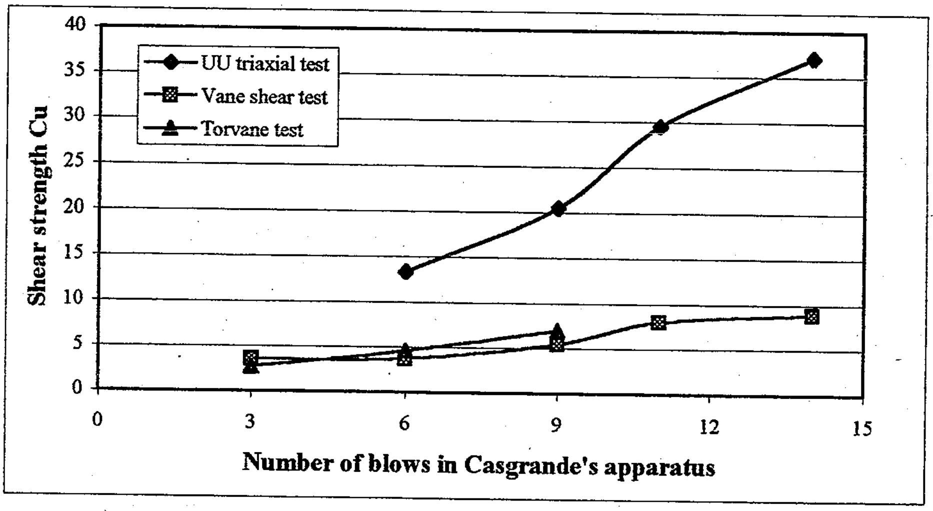

Figure 4 Variation of C_u with no of blows

6. Conclusion

It can be concluded that deep mixing process causes significant improvements in both primary and secondary consolidation properties of peat and undrained shear strength. Cement percentages at the order of about 20% by weight are required. The levels of improvement achieved are comparable with those achieved by preloading. The important advantage is that the improvements achieved within 4 to 6 weeks.

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IMPROVEMENT OF SLOPE STABILITY THROUGH BIO-ENGINEERING TECHNIQUES

Athavan.S, Kalyani. B,Ramanan.M
University of Moratuwa, Srilanka.

Abstract

Slope Failures and landslides are a Major concern in Srilanka. Most of these landslides are triggered by excessive rainfall. The stabilization by external means such as subsurface drains and soil nailing are quite costly. On the other hand if the other the stabilization of slopes can be achieved through vegetation it would really attractive and environmentally friendly and cost effective. Vegetation Influences on Stability by enhancing the cohesion of soil and providing reinforcement through it's root system. The root system could also be effective in reducing the pore water pressure. The paper reviews the current state of art knowledge in the influence of vegetation on slope stability and make suggestions for Srilankan Applications.

1. Introduction

Bio technical and soil bio engineering methods greatly expand the range of options for stabilizing and protecting slopes against erosion and shallow mass movements.

Vegetation that may be growing on a slope has traditionally been considered to have an indirect or minor effect on stability, and it is usually neglected in stability analysis. This assumption is not always correct, and for certain forested slopes with relatively thin soil mantles, it has been shown to be significantly in error. This has hampered the efforts to quantify them in stability analysis.

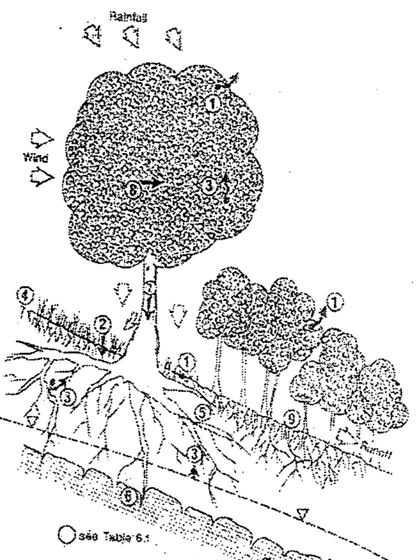
Vegetation slope interactions are complex. Vegetation provides a protective layer or buffer between the atmosphere and the soil. The effect of vegetation on slope is mainly two fold. It could have a mechanical effect, improving the cohesion and providing a reinforcing action through it's roots. On the other hand it could have a hydrological effect reducing the pore water pressure and inducing negative pore water pressure at times.

The physical principles underlying the effects of vegetation can be combined in either analytical procedures based on theory, or a series of general empirical relationships, based on observations and experiences.

The method of using vegetation to enhance stability of slope is cost effective, visually attractive and environmentally friendly. It had been used at number of sites, in providing economical and environmentally attractive structures. In this Project the current state of the art on application of vegetation to the improvement of slope stability is reviewed and suggestions were made for possible Srilankan applications.

2. The Role of vegetation in stability

Vegetation influences on slope stability may be broadly classified as either hydrological or mechanical in nature (Anderson 1990). Under the hydrological factors, factors like interception, infiltration, transpiration and desiccation cracking should be considered. In the mechanical factors mainly, root morphology and strength are significant. Roots reinforce the soil due to their tensile strength and frictional or adhesion properties.



Slope –vegetation interaction influencing stability. (After Anderson &Richards 1990)

2.1 Hydrological mechanisms

1) Foliage intercepts rainfall, causing absorptive and evaporative losses that reduce rainfall available for infiltration.
2) Roots and stems increase the roughness of the ground surface and the permeability of the soil, leading to increased infiltration capacity.
3) Roots extract moisture from the soil, which is lost to the atmosphere via transpiration, leading to lower pore- water pressures.
4) Depletion of soil moisture may accentuate desiccation cracking in the soil, resulting in higher infiltration capacity.

2.2 Mechanical mechanisms

5) Roots reinforce the soil, increasing soil shear strength.
6) Tree roots may anchor into firm strata, providing support to the upslope soil mantle through buttressing and arching.
7) Weight of the vegetation surcharges the slope, increasing normal and downhill force components.
8) Vegetation exposed to the wind, transmits dynamic forces into the slope.
9) Roots bind soil particles at the ground surface, reducing their susceptibility to erosion and enhancing the cohesion.

3. Modes Of Slope Instability And Vegetative Influence

3.1 Shallow Translational Slide

Considering a shallow translational slide with water table at an intermediate level, the factor of safety expression can be derived as;

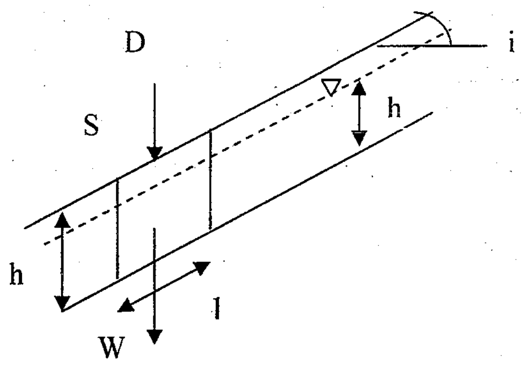
$0 < m < 1$

$$FOS = \frac{C' + [(1-m)H\gamma + mH(\gamma_{sat} - \gamma_w)] \cos^2 \iota \tan \phi}{[(1-m)H_\gamma + mH_{\gamma sat}] \sin \iota \ \cos \iota}$$

Bache and Mac skill (1984) studied the effect of vegetation on a shallow translational slide and modified the expression for FOS as;

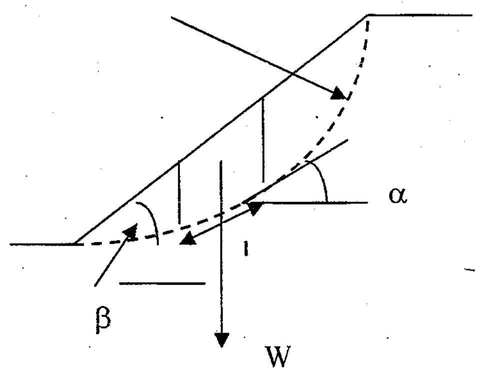
$$F = \frac{(C' + C'_R)\, l + [(W + Sw) \cos \iota - u\, l] \tan \phi}{(W + Sw) \sin \iota + D}$$

Where C'_R is the improvement of cohesion due to the roots and is the surcharge due to the vegetation. W is the weight of the soil Per unit width.



A parametric study was performed by the authors on a 30° slope with C= kN/m^2 and ϕ= 34.7. The value of C'_R was changed from 5 kN/m^2 15 kN/m^2. This is a typical range considering the values reported in literature. With this change The FOS changes from 1.187 – 1.578. It simply illustrates the usefulness of vegetation.

3.2 Circular Rotational slides



The factor of safety for a rotational slide under the ordinary method of slices is given by;

$$FOS = \frac{\sum_{I=1}^{n} [C' \Delta l_i + (W_i \cos\theta_i - u_i \Delta l_i) \tan \phi']}{\sum_{I=1}^{n} (W_i \sin\theta_i)}$$

Expression For FOS for Circular sliding with vegetative influence is given by Greenwood (1983);

$$\frac{\Sigma\{C'_0 b\sec\alpha + [(w_0 - u_0)b)\cos\alpha - D\sin(\alpha - P) + T\sin\theta]\tan\phi + T\cos\theta\}}{\Sigma\, [w_0 \sin\alpha + D\cos(\alpha - \beta)]}$$

Where

$w + sw = w_0$

$C' + C'_R = C'_0$

$u - u_v = u_0$

D = weight of Vegetation

Authors performed a parametric study on vegetative effect, by changing the value of C'_R in the range2.5

kN/m^2 to 7.5 kN/m^2, Factor of safety change in the range 1.439 to 1.797 was obtained.

4. Case Histories of use of Vegetation

1) The study along a 10km stretch of road along the Sacramento River in California concluded that woody Vegetation did not adversely affect the structural integrity of a levee. The presence of plant roots reinforced the soil and measurability increased the shear strength of the surface layers. (" Value of Vegetation " Robin Soffit- Ground Engineering Supplement, March 2001 –page 08)

2) To quantify the possible benefits of vegetation, a Bio-engineering field trial, sponsored by CIRIA was set up in 1993 way at Longham Wood cutting, Near Maid stone, Kent. Result indicated that over the five to six year trial period, soil moisture changes due to the vegetation tended to be masked by the large seasonal moisture changes in the Gault clay. Similarly the seasonal water table fluctuations were not noticeably influenced by the presence of vegetation. The main benefit from the vegetation was the enhanced shear strength developed by the root-reinforced soil. ("Rooting for Research ", John Greenwood Ground Engineering Supplement - March 2001–page 18)

5. Selection of a Suitable Type of Vegetation

Planting of vegetation on slopes had been done in several different ways. They are:

1. Seeding
2. Container or Bare Root Planting
3. Live staking
4. Contour Wattling
5. Brush Layering

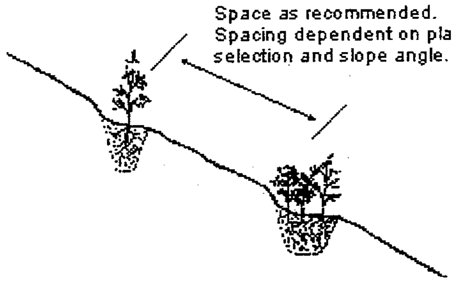
5.1 Seeding

Seeding Involves the application of grass, forbs, and woody plant seed mixes to slope areas. Seeds may be applied to slopes by broad casting seed mixes onto the slope by hand or by placing seed into small holes placed into the slope. Seeding can be quickly applied to slopes, materials are expensive and Technique is compatible with many slope situations are advantages.
Disadvantages being, Seeding does not readily self-repair eroding slope areas. Seeding is not adequate to de used alone for highly eroded areas or for shallow seated land sliding stabilization.

5.2 Container or bare Root Planting

Bare root plating is mostly used to plant grasses. It involves placing single or bunches of rooted plant into excavated holes on the slope. This method can be used for woody plants or for non-woody plants that will eventually spread into uniform root coverage.

Material should be placed on the slope during the fall or spring in a country with a temperate climate. When placing material in holes, make sure that the root ball is loosened and roots are not bent upwards in the holes.



Bare root planting (After Anderson 1990)

Advantages are: Well-developed rooted plant materials installed for faster slope stabilization. Typically higher plant success, minimal slope disturbance using planting holes.
Relative cost of materials, Hard to Install into some mulching systems, initial watering requirements are the detriments.

Other methods are not widely used to plant grasses on slopes.

6.Use of Vetiver grass for stabilization

"VETIVER" Grass can be recommended as a suitable type of vegetation For Srilankan conditions as most of the weather and other conditions are favorable. And also some Asian countries like China, Hong Kong, Thaiwan and Malaysia are using this grass mainly for improving slope stability.

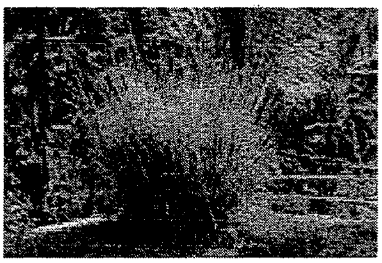
There are twelve known varieties of vetiver grass the most important variety is "Vetivera zizaniodes ", which can be grown in Asia. There are other types such as V- nigratana & V-nemoralis and are grown in other parts of the world. In modern times the sugar industry used Vetiver grass quite widely for contour conservation hedge sand for the stabilisation of roadsides and embankments.

6. 1 Special Physiological characteristics

- Tolerance to extreme climatic variation such as prolonged drought and extreme temperature from 10°c- 50° c. Growth is rapid in the absence of Shade.
- Wide range of soil PH (3.0- 10.5)
- Highly tolerant to toxic levels aluminum, copper, chromium, manganese, arsenic, cadmium, mercury nickel, lead and Zinc.
- Survive total drought, but normally requires a wet season of at least three months. Well spreaded monthly rainfall is ideal. Rainfall above 700 mm is preferable.
- Grows best on deep sandy loam soils, however it will grow on most soil types ranging from black cracking vertisoils.

6.2 Planting process

Planting of hedgerows should take place early in wet season. 2-3 slips should be planted at each " Station ". Each station should be 10-15 cm apart. Distance between hedgerows should be at a vertical interval of about 2m. In flatter land this may be reduced to 1m. Care should be taken to select good quality slips and they should be planted with in 3 days of lifting from nursery.



Vetiver Grass

Planting slips should not be allowed to dry off and should be protected from sun. From 2000-3000 planting slips are required per 100m of hedgerow. Under very dry conditions, it is better to plant Vetiver slips in small " v " ditch to enhance moisture availability at time of planting.

The influence of Vetiver root system on the Cohesion of a soil, the strength of the roots etc should be quantified through appropriate laboratory testing. Vetiver should not be planted under shade and removal of shade will enhance the rate of growth. Fertilizers such as NPK, DAP can be used to optimize the growth rates.

8. Conclusions

Parametric study showed that all modes of slope failures like translational slides circular slides and Non-circular slides can be improved through Bioengineering techniques.

In Srilanka, Vetiver can be used to stabilize slope, since the morphological and philological characteristics like climate and soil type are prevailing here. It is safe to use for erosion control in agricultural lands and the protection of environmentally hazardous areas. It is a plant, which the people of Sri Lanka are familiar with. They have been cultivating vetiver in their home gardens, for medicinal purposes and to repel insect pests, from time immemorial. Vetiver is adoptable to a wide range agro –climatic regions, soil types and existing cropping systems.

Vetiver grass is used as a soil and water conservation measure in Sri Lanka especially in the tea, vegetable and tobacco cultivation areas. It is remarkably adaptable to a wide range of soil and climatic conditions. Direct shear tests can be conducted to determine the improved cohesion of a soil with roots of Vetiver plant. From the improved cohesion values the improvements possible in FOS can be computed. The tensile strength of the Vetiver root system also can be incorporated into analysis.

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FINITE ELEMENT SIMULATION OF SETTLEMENTS IN THE CKE EMBANKMENT

R.S. Dinesh, A.T.S. Epage, P.M. Sirithunga.

ABSTRACT

Currently most major Sri Lankan cities face land problems for infrastructure development due to lack of land with good sub-soil condition. Thus many proposed or ongoing highways are planned to have their routes over weak sub-soils. One major highway infrastructure projects where very soft peaty clay layers exist in the sub-soil is the Colombo-Katunayaka Expressway (CKE). Geotechnical problems are associated with construction done on peaty clays due to very high consolidation settlements. Generally application of preloading (surcharge) is done in such cases

This project was done to investigating the ability of using a simple linear elastic constitutive model used with the FEM to simulate the behaviour of very soft peaty clay layer underlying the CKE project when loaded by a sand embankment construction. Investigations are conducted on behavioral patterns at different loading stages, stress development in the soil and the change in soil properties with load increments. The deformations at each node and the stresses developed at each element are predicted by the finite element analysis program FEAP.

1 Introduction

The analysis of deformation of layered soil media under load should include the aspects of displacement of points inside the soil media (settlements etc.) and migration of water (permeability and seepage characteristics). This process is known as "consolidation" in geotechnical engineering field.

A theory for one-dimensional consolidation was first proposed by Terzaghi. Biot extended these theories to three-dimensional consolidation. These theories can be used to obtain analytical solution to problems involving simple geometries. When the problems become complicated, such as a layered formation involving different material properties, obtaining closed form analytical solution become intractable.

A more simple way to approach deformation analysis of layered media is to consider it as elastic media. However, selection of appropriate elastic parameters to represent the behaviour of each layer is essential. Numerical methods like the finite element method offer convenient tools to solve complicated deformation and stress analysis problems in layered elastic media.

2 Objective and Scope

The main objective of the project is to investigate the suitability of the simple linear elastic constitutive model used with the FEM to simulate the behaviour of very soft peaty clay layer underlying the CKE project when loaded by a sand embankment construction. Investigations are conducted on behavioral patterns at different loading stages, stress development in the soil and the change in soil properties with load increments.

The stress and settlement analysis is conducted using the finite element method. Materials are assumed as linear elastic. Material properties for each layer are selected based on the data available in borehole reports and by comparison of the numerical results obtained from the finite element analysis with the actual readings observed at site.

The deformations at each node and the stresses developed in each element are predicted by the finite element analysis program FEAP. Using these results, the final deformed shape of the embankment is plotted and settlements calculated.

3 Elastic analysis idealization

The stress strain relationships are the mathematical description of the mechanical properties of the soil. A complete stress strain relationship for a soil will represent the stresses and strains in a soil medium at any particular time under any given loading condition.

In the particular situation under consideration here, the results depend on instantaneous deformation, consolidation, and creep settlements. Thus, the complexity of the soil behavior in this particular situation has increased. Here the soil media are idealized to behave as isotropic linear elastic material. The elastic analysis will be based only on the material parameters Young's modulus and the Poisson's ratio.

4 Finite Element Program FEAP

The analysis in this project was carried out using the Finite Element Analysis Program (FEAP) originally developed by professor R.L. Taylor of University of Berkely, California. Chapter 24 of Zienkiewicz (1977) describes the initial format of the program. The program was later expanded at the Asian Institute of Technology by Professor Worsak Kanok Nukulchai (1984). This program has been modified by incorporating new element subroutines, reducing its code length and adjusting the number of element subroutines, and installed in Pentium microcomputers with limited RAM (Random Access Memory) at the University of Moratuwa by Dr. U.G.A.Puswewala.

5 Numerical analysis of the problem

For the peaty soil at the location considered the Young's modulus(E) was taken according to selvadurai (1979); see Appendix D. The peat was considered as three different layers by considering the SPTN values of the borehole at the location considered. Thus three different E values were taken for the analysis.

Material	Soil type	E (KN/m^2)	Poison's Ratio
Layer 1	Sandy clay	0.20×10^5	0.30
Layer 2	Soft clay	0.75×10^3	0.20
Layer 3	Very soft clay	0.22×10^3	0.20
Layer 4	Loose sand	0.15×10^5	0.25

Table 1. Material parameters

The following boundary conditions were adopted.

Nodes at the bottommost row - Fixed in both X and Z directions
Nodes along the symmetric axis – Fixed only in X direction

The peat was loaded vertically with the sand embankment and the nodal loads were applied to the nodal points.

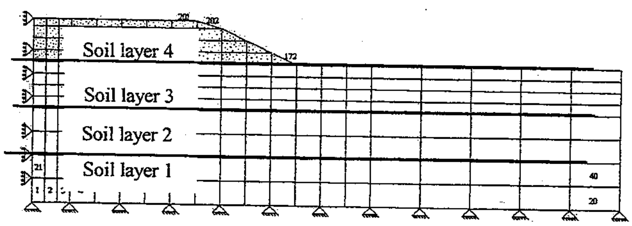

Figure 1. Finite element mesh

6 DISCUSSION

Finite element simulations were conducted using FEAP (assuming all materials as isotropic linear elastic) for the embankment section considered in the study.

The E values for peat were selected by carrying out three finite element simulations and adjusting the E values for peat until the settlement at the top of peat layer predicted by FEAP matched with the settlement measured in the field.

The settlement (recorded and predicted) at the top of peat layer at center of

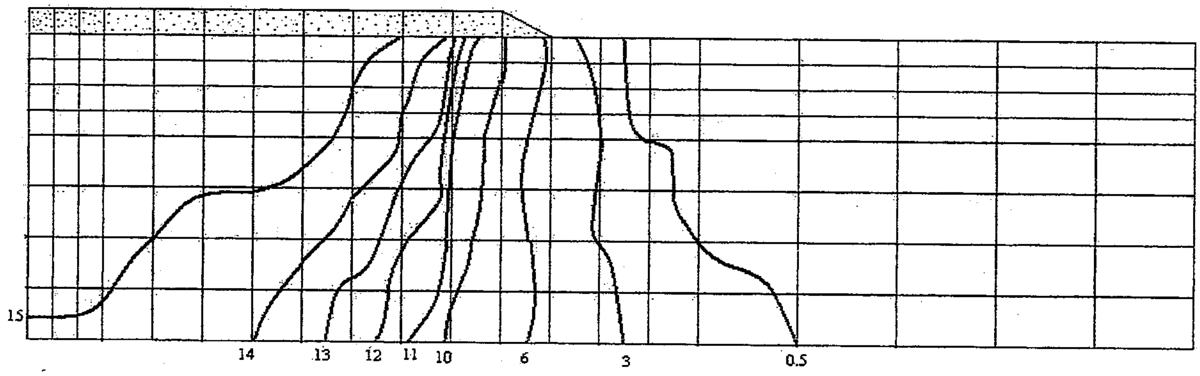

Figure 2 Stress contours for the loading stage 1

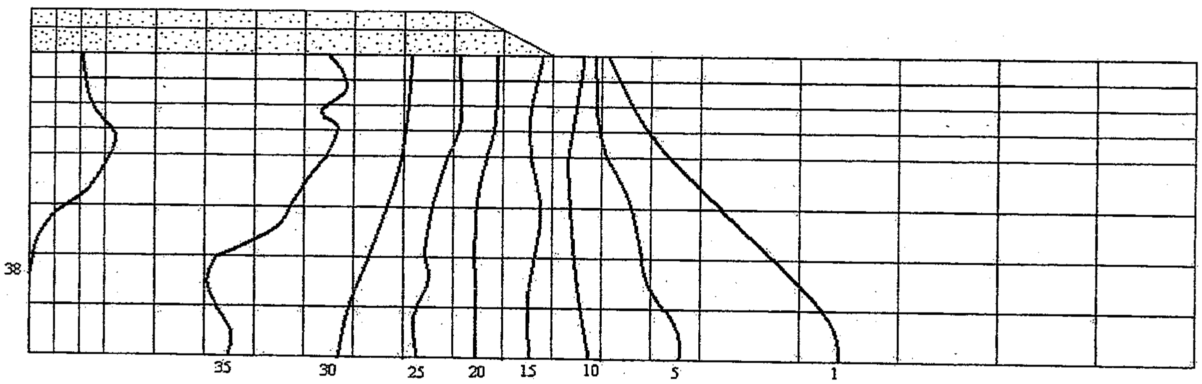

Figure 3 Stress contours for the loading stage 2

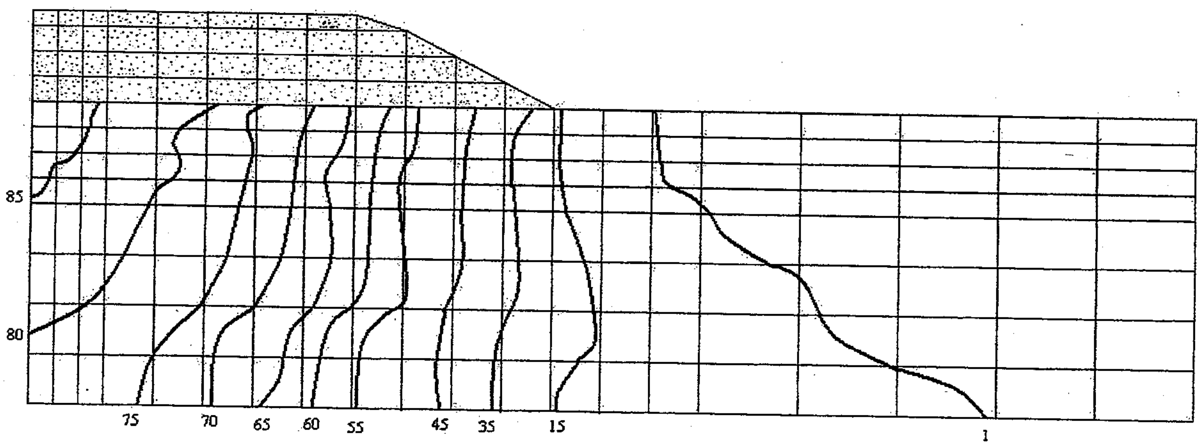

Figure 4 Stress contours for the loading stage 3

embankment was used for comparison of measured and predicted values.

Following table summarizes comparison procedure followed.

Fill thickness	Measured settlement	Predicted settlement	E_{peat} used for prediction
1m	0.661m	0.66m	150 KN/m^2
2.5m	1.553m	1.663m	150 KN/m^2
5.5m	2.814m	2.753m	220 KN/m^2

Table 2. Comparison of settlements

The comparison of E values for the stress ranges or 10 KN/m^2– 20 KN/m^2, 20 KN/m^2-40 KN/m^2 and 40-80KN/m^2 as calculated from Oedometer tests of kugan (2003) and as used in finite element analysis are presented in Table 3.

Stress range KN/m^2	E value from tests of kugan (2003) KN/m^2	E_{peat} value used in FE analysis KN/m^2
10 – 20	135.15	150
20 – 40	182.6	150
40 -80	363.38	220

Table 3.Comparison of E values

7 Conclusion & Recommendations

The value of E in the second and third column of Table 3 do not agree within practically acceptable ranges for the stress interval of 20 KN/m^2- 40 KN/m^2 and 40 KN/m^2- 80 KN/m^2.

This observation confirms that a simple isotropic elastic analysis of the CKE embankment is not always in agreement within ranges of practical applicability.

With the increase of loading there is a considerable amount of gain in the Young's modulus according to this elastic analysis.

It is also very important to take the 3-D extension considered in the results in to account where, the 3-D nature of deformation in the field, as compared to 1-D nature of settlement in Oedometer test.

More complex simulations which will take consolidation effects seepage, creep etc. into account using a computer software such as SIGMA/W, SEEP/W can be recommended for more accurate and more detailed results. It is also very important to consider the comparison of 3-D nature of deformation in the field with the 1-D nature of settlement in Oedometer test.

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PILE LOAD TESTING USING PILE DRIVING ANALYZER

E.A.B.Edirisooriya, A.K.Y.B.Sumanasinghe, J.A.C.Prabash.

Abstract

Pile Driving Analyzer (PDA) is the most sophisticated method that exists for the determination of ultimate carrying capacity of a pile. It considered the wave propagation through the pile in addition to the damping effect of the soil. In PDA, it uses Case Global method, to obtain initial value for the carrying capacity at the field. Then using CAse pile Wave Analysis Program (CAPWAP) we take more accurate value. While using case method we assume value for damping factor of soil. And also during CAPWAP analysis we should assume initial toe quake and skin quake values. Statistical analysis was carried out to find the suitable initial damping factor for case method according to pile dimensions. This paper discusses variation of CAPWAP E value with pile dimensions. At last it was checked suitability of normal assume value for skin and toe quake values.

Introduction

Piles are used to transmit foundation loads to deeper soil or rock strata having a high bearing capacity through soil strata of low bearing capacity. Although this is the main utilization of pile, they are also used in normal ground conditions to resist heavy uplift forces or in poor soil conditions to resist lateral (horizontal) loads. All these piling construction works carried out at subsurface conditions. Therefore we need some quality assurance method to ensure the quality of pile after construction. Although in case of driven piles we can check quality of pile before driving, after driving there should be a reliable method to check caring capacity of piles. Especially for non displacement piles lot of defects can be formed during construction.

Pile Testing methods

When designing the pile foundation system, engineers have a wide range of choices, including the ultimate load per pile and pile size (type, length and diameter). The required pile capacity depends on the applied loading from the super structure, the testing method for verification of the pile capacity, and the frequency of testing. The ultimate pile capacity must exceed the applied loads by sufficient margin. To reduce the risk and prevent foundation failures, safety factors are assigned to compensate for uncertainties. Deep foundations require adequate quality assurance for successful service life. A well planned test program allows the design engineer to assure adequate bearing capacity while at the same time minimizing the cost of the foundation, for example through reduced factors of safety. Various methods have been developed to estimate the ultimate caring capacity of pile. These methods can be divided in to mainly two groups; namely static methods and dynamic methods. Dynamic methods are further sub divided as follows: 1. Pile driving equations(Hiley, Janbu, Danish, etc) 2.Load testing using Pile Driving Analyzer (PDA). Each method has some inherit advantages and disadvantage. There are many empirical formulae for the evaluation of the bearing capacity of piles. These are generally based on the following assumptions: 1.The pile is regarded as a rigid body during driving. 2. Static and dynamic soil resistance is taken to be equal. Neglect the damping effect of soil.3.Analysis results is independent from time. 4. Some formulae are considered no energy loss during driving.

There are many limitations and drawbacks in pile driving equations. Some can be uprooted by using other methods. But basically these methods are simple and get an idea about ultimate carrying capacity of pile relatively quickly. Some values are quite deviated from actual values so either dangerously under predicted or uneconomical. In most cases these difficulties are dependent on following 2 factors: 1.Modeling of hammer, driving system, pile, and soil. They are not applicable to most hammer types used on today's sites, long elastic piles, or cohesive impermeable, compressible soils.2. There are no provisions for the estimation of strength gain with time, of driven piles in the normal pile driving equations and these equations are adopted for capacity estimation at the time of pile driving only. After driving there may be regaining of soil particles and ultimate carrying capacity will go high.

Foundation engineers searched for the better methods to simulate the actual pile behavior under any driving condition as much as possible. As a result of these researches, it was recognized that the pile driving is a phenomenon better approximated by wave propagation theories. This analysis takes account of the fact that each hammer blow produces a stress wave that moves down the length of pile at the speed of sound, so the entire length of the pile is not stressed simultaneously, as assumed in the conventional dynamic formulas. The wave equation approach is primarily used to yield a relationship between ultimate pile load and set, although the stresses set up in the pile during driving are also calculated. In addition, this analysis enables to take various factors such as pile characteristics, hammer characteristics, and cushion stiffness. Therefore it provides convenient method to simulate the pile and driving system.

Because of the complications involved in practical piling problems, analytical solutions to the above equation are not feasible. A convenient numerical method has been described by Smith (1960).

Smith Idealization

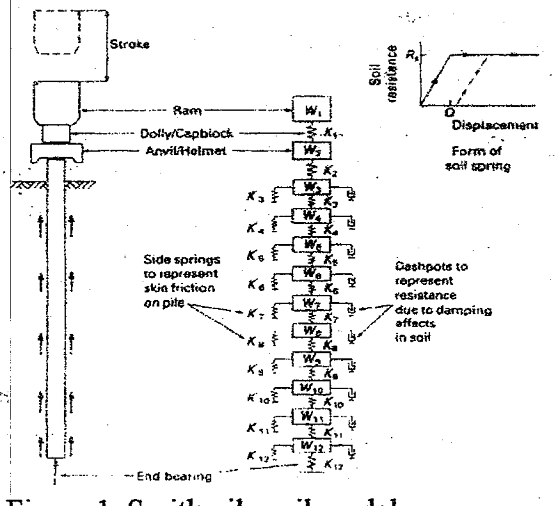


Figure 1. Smith pile soil model

The ram, cap block, pile cap, cushion block and pile are represented by discrete weight springs. The frictional resistance on the side of the pile and the point resistance are represented by single springs and dashpots. So analysis part was tried to computerize from the beginning.

History of PDA

Although now PDA is fully computerized method initially it was not so. First engineers wanted to simulate the actual pile behavior during pile driving by satisfactory mathematical model. This was done by E.A.L Smith in 1954-1960 and named as wave equation method. In 1972, pile dynamics cooperation introduced its first commercial device called "Pile Capacity Computer". After that there was significant contribution to this technology by three persons, named Goble, Rauche and Likins in 1974. They developed more powerful computer program called GRLWEAP to solve wave equation method. In addition to that, first use of the name "pile driving analyzer" came with the field method called case method developed by Case Western University under supervision of Prof. Goble.

Pile Driving Analysis (PDA) is a testing method, which determines static pile carrying capacity using a dynamic impact on a pile. The PDA measures strain and acceleration under the pile driving hammer or a relatively small drop weight (typically 1 to 1.5 % of maximum ultimate load. The variation of the acceleration and the strain with the time are measured by the pairs of accelerometers and the strain transducers attached at a location about 1.5 times pile diameter below the pile top. Analog signals from transducers are convey to the Pile Driving Analyzer, which is computer. PDA applies case method to pile force and velocity data in real time between hammer blows after providing signal conditioning, amplification, filtering, and calibration to measured signals and data quality assessment

Objectives of research

1. Recommend a suitable damping factor for initial PDA test according to pile dimensions.
2. Find any relationship between CAPWAP E value and other pile dimensions
3. Check the validity of existing soil parameters for PDA test, such as Toe quake, Skin quake.
4. Check the validity of theoretical co relationship between actual concrete grade and measured velocity at actual Sri Lankan conditions.

During the testing and analyzing, damping factor should be assumed. Normally this is used as 0.5. Then we tried to derive qualitative relationship CAPWAP E value with other pile parameters. At last it was inquired about the validity of theoretical co relationship between actual concrete grade and measured velocity at actual Sri Lankan conditions. The PDA computes some 40 different dynamic variables according to the case method after each hammer blow. The most interesting values are: 1.Maximum energy transferred to the pile 2.Maximum dynamic pile compressive and tensile stresses 3.A structural integrity assessment factor 4.Pile driving resistance 5.Static bearing capacity.

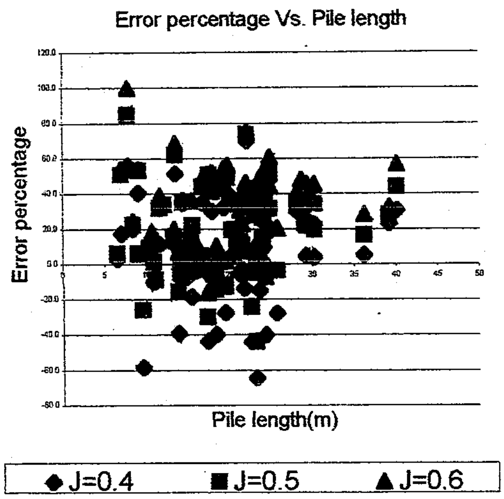
CAPWAP is an analytical method that combines measured field data with pile wave equation type procedures, to predict the pile's static bearing capacity and soil resistance distribution along the pile shaft. Measured force and velocity data is directly input from the PDA. Based on the measured velocity data, the program computes force required to induce the imposed velocity. Both measured and computed forces are plotted as a function of time and the iterative analysis is continued till there is good agreement between both the curves. If the agreement is not satisfactory, the soil resistances at the pile point and along the pile are adjusted until a good match is obtained. This gives better estimates of the actual static pile capacity measured during field-testing, and also the friction and end bearing Pile top force, and acceleration are measures and velocity is obtained by integrating the accelerations

Data analysis

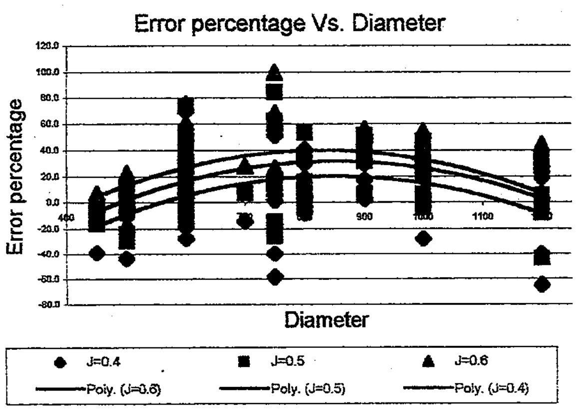
Firstly, the PDA test results were collected and a database was created. These PDA test data were not from same area of country. Test results are obtained from sites, which are situated at Colombo, vicinity of Colombo, Galle and Kandy areas. All together 73 PDA records were collected. But out of 73 pile records only 67 records were taken for a research. Four piles at Galle site were failed in PDA test and two at Kandy site were failed in pile integrity test. Diameter, Length from ground level, Wave speed, Pile location, Hammer details (Ram weight, Observed height) and PDA data and CAPWAP results were collected.

If CAPWAP capacity is correct, it is possible to define value as error percentage as follows.***Error percentage = (CAPWAP Capacity-RMX Capacity) X 100/ (CAPWAP Capacity)***

If this value is positive value our initial RMX value given by PDA method is conservative. Observed error percentage Vs pile length and error percentage vs. pile diameter variations are as follows.



Graph 1 Error percentages vs. Pile length (m)

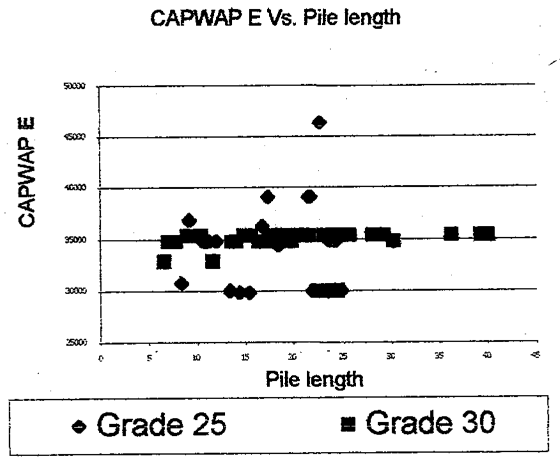


Graph 2 Error percentages vs. pile Diameter (mm)

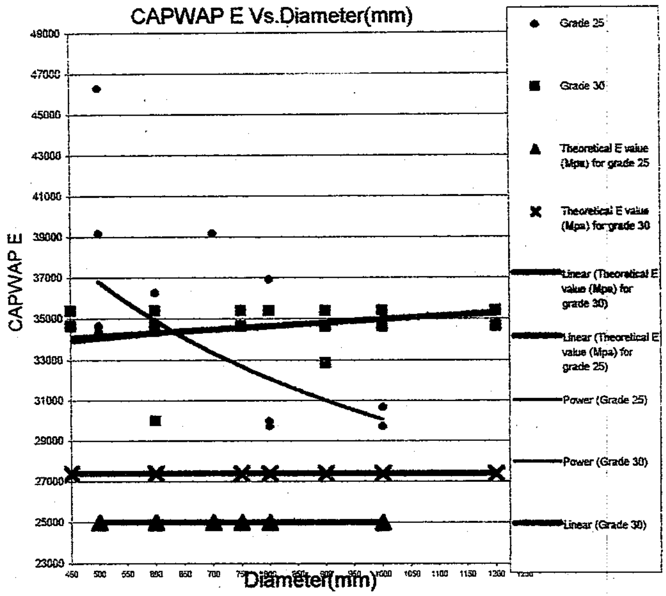
After doing CAPWAP analysis, it was given some E value (young's modulus for concrete). Theoretical value is given by following equation.

E value= $5000\ \sqrt{Grade}\ \ (MPa)$ (1)

During this research observations wee done with respect to pile physical dimensions and expected concrete grade at site .All data collected were belonging to grade 25 and grade 30 strength concrete.



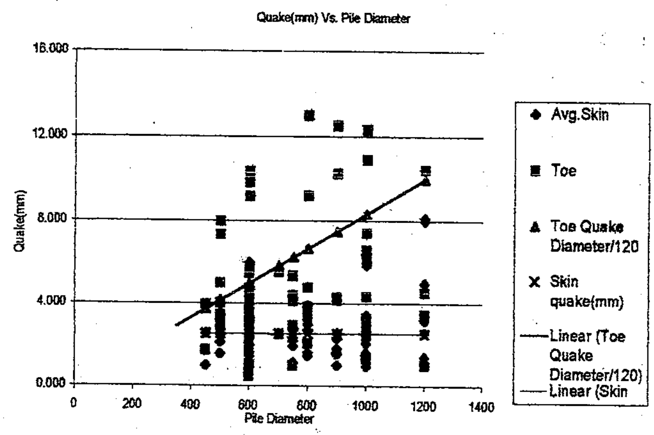
Graph 3 CAPWAP E vs. Pile length



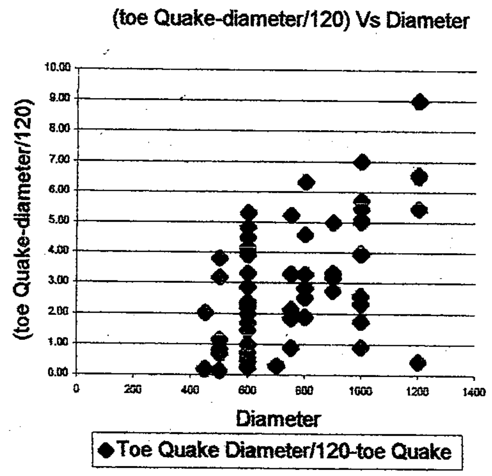
Graph 4 Diameters vs. CAPWAP E

With increasing of diameter, variation of "CAPWAP E "values, become less. This result is more correct for lower grade (e.g. grade 25). For grade 30, CAPWAP E value is independent from their diameter and pile length. But for grade 25 CAPWAP E value change with their dimensions (i.e. Diameter and Length). All observations give higher E value than theoretical values {CAPWAP E vs. Diameter}.

Normally for PDA test initial toe Quake is taken as Pile diameter/120. Research was conducted to check its validity when pile diameter is actually changed.



Graph 5 Quake vs. Pile length



Graph.6 Absolute value of (toe quake-diameter/120) vs. Pile diameter (m)

At the site, 0.1 inch (2.5mm) is used for shaft quakes. This value is reasonable value. Measurements have demonstrated that toe quakes can be much lager than 2.5mm.

Conclusion

For more lengthy piles, by assuming higher damping factor (e.g. 0.6), the estimate capacity is conservative. (i.e. RMX < Actual CAPWAP). That does not mean error percentage becomes less. So initially predicted capacity value at the site is lower than the actual capacity. Therefore our assumptions are conservative. When consider the range of length 5m to 30m, there is higher possibility of becoming non-safe due to the incorrect assumption of damping factor. {Error percentage Vs Length}. Variation of the error percentage with the diameter is similar for any damping factor. {Error percentage Vs. Diameter}. So error percentage variation with diameter does not depends on damping values

During the research it was shown that with increasing of diameter, the variation of "CAPWAP E values become less. This may be due to the following reasons: Actual grade of the concrete may be higher than what we expected. ~~With~~ increasing of diameter self-compacting of concrete may be more and it's density become more. As a result of this overall quality of pile ~~goes high~~. When we consider higher-grade concrete effect of diameter to E value becomes less. And also CAPWAP analysis bases on one-dimensional wave propagation through the pile. Actually there will be some three dimensional effect.

If we take initial toe quake as diameter/ 120, this assumption may not be accurate for the higher diameter (For 1000mm, 1200mm). This factor can be clearly shown in above graphs (Toe Quake-Diameter/120 vs. Pile diameter). Usually for skin quake 0.1 inch is used. This value has been extensively used with good correlation.

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QUALITY CONTROL & ASSESMENT OF BORED CAST IN-SITU PILING PRACTICES IN SRI LANKA

Jayasekera V.C.M., Galapaththi L.P., Pathirage L.P.D.

University of Moratuwa, Sri Lanka.

Abstract

Detailed verification of the integrity of bored cast in-situ foundation piles after installation is time-consuming and expensive. The objective of this research is to provide a summary description of the piling practices carried out during the construction process, and the difficulties experienced by the contractors in carrying out the said process, including observed deviations from recommended practice. During the approximately ten month study period, a survey was carried out on the existing piling sites within Sri-Lanka, in order to study the identified practices being carried out on bored cast in-situ piles. Special attention was paid to recognize the common difficulties in controlling the quality within the said process. Every effort was made to obtain a representative data set of common practices and procedures of piling contractors in Sri-Lanka and the survey was carried out on nearly all known, reputed piling contractors. Finally the conclusions and suggested recommendations with suggestions and comments, summarises our experience. And from the observations made during the survey, an attempt is made to recommend additional specifications which can be incorporated into the current codes of practice and specifications in use.

1.0 Aim and Objectives

The main aim of the study is to investigate the quality control measures that are practiced in the construction of "bored and cast in-situ concrete piles" in Sri Lanka and compare them with the accepted standards, with the ultimate objective of displaying the most important parameters to control the quality and integrity of "bored cast in-situ foundation piles" directly during the production process and to propose a protocol for quality assurance and documentation.

The rationale for such a study stems mainly from the fact that a detailed verification of the integrity of bored cast in-situ foundation piles after installation is time-consuming and expensive.

Insufficient studies have been carried out specifically in this area of piling construction work, and the study hopes to provide a better understanding of the deficiencies of the current piling practices carried out in Sri-Lanka and aims to identify the true process involved in "cast in-situ" piling carried out by piling contractors in Sri Lanka. In addition to identify the effectiveness of the piling operation, quality control measures adopted, difficulties experienced, problems involved and rectification works incurred.

1.1 Method of Analysis

During an approximately ten month study period a survey was carried out on the existing piling sites within Sri-Lanka to study the common practices of the piling process identified as being carried out on bored cast in-situ piles. Special attention was paid to identify the common difficulties in controlling the quality within the said process.

Every effort was made to obtain a representative data set of common practices and procedures of piling, carried out by piling contractors in Sri-Lanka. The survey was thus carried out on nearly all identified reputed piling contractors. Attempts were made to survey at least two sites of each contractor to obtain a common protocol of practice by each.

Limitations to the survey included the fact that it concentrated on piling sites in operation within the greater Colombo area. However as the process carried out by individual contractors rarely changed from location to location this limitation was considered to bear insignificant relevance to the results of the survey.

A questionnaire was used in the survey as the "field documentation" of the work carried out and provided a step by step walkthrough to identify the piling process and quality control procedures adopted in the construction industry. Data was collected under three main sections of preliminary preparation for piling work, the actual piling process and post piling work carried out

The current codes of practice and specifications in use were analysed, not only in Sri-Lanka but also around the world, to identify additional relevant material which are currently not available in the local specifications. The analysis was carried out to identify the differences and additional data existing in these literature with the more commonly used ICTAD specifications of Sri-Lanka. Other relevant material which has been analysed include;

- o British Standards – BS 8004: 1986
- o Indian Standards - IS 2911 Part I Section 2: 1979
- o European Standards – BS EN 1536: 2000
- o California Department of Transportation – "CalTrans Foundation Manual" (Which include Specifications for Slurry Displacement Piles)

2.0 Bored Cast In-Situ Piles – Quality Control & Documentation

2.1 Soil Investigation

The investigation of ground should be carried out in accordance with BS 5930. The objective of the investigation is to examine the sub soil condition of the site and to recommend a suitable type of foundation for the proposed structure and safe bearing capacity of the ground. Thus "*Boring should reach depths adequate to explore the nature of the soil both around and beneath the proposed pile, including all strata likely to contribute significantly to settlement*". (BS 8004) The Soil Investigation Report is the final result of the ground investigations carried out and it is the initial documentation through which the quality control procedures required for the piling process can be established. The identified details such as the type of soil present as well as the ground water level can forewarn a contractor of the potential problems present at site.

However as the codes of practice used worldwide illustrates the information to be provided on *the number description and location of boreholes together with the required depths and sampling depths* is on the onus of the specification writer. i.e. Engineer/Client Due to these unclear specifications in site investigation, the survey carried out indicated no consistent use of soil investigation procedure. Inconsistent use of boreholes in different sites, ranging from only two boreholes used at one site to a maximum of twenty in another were observed although both sites were of roughly the same dimensions.

BS 8004 states that *Chemical analysis of samples of the groundwater and soil should be undertaken to assess the necessity for special precautions.* However during the survey it was observed that no chemical analysis was carried out on the soil on any occasion and no additional requirements were specified for the use of concrete, cover to reinforcement etc. due to adverse conditions. (eg. In circumstances where the piles were built near water or the sea.)

2.2 Locating Pile Points

The most important aspect of the piling process is the initial location of pile points. Correct location of pile points ensure minimum eccentricity and thus avoids undue moments developing in the structure.

A pile cut off at or above ground level the maximum permitted deviation of the pile centre from the centre point shown in drawing shall be within 75mm in any direction. An additional tolerance for a pile head cut below ground level is permitted in accordance with the verticality requirement where the maximum permitted deviation of the finished pile form the vertical is 1 in 75. (ICTAD/DEV/16 - clause 1.06.2 & 1.06.3)

Further according to ICTAD/DEV/16 clause 1.06.5 *if one or more piles deviate from the allowable tolerances, the Engineer may decide that additional piles shall be placed and/or the foundation construction shall strengthened or extended.*

The survey observed that on average although the required tolerance limits were met 18 % of the time, setting out was not carried out accurately and on average pile centres often deviated (within the required tolerances) 82 % of the time. However none of the contractors were required to place additional piles for those which exceeded tolerance limits. The remedial measures were to strengthen / lengthen the pile caps with additional reinforcement to enable it to carry additional eccentricities.

2.3 *Verticality*

The maximum permitted deviation of the finished pile form the vertical is 1 in 75. (ICTAD/DEV/16 – clause 1.06.3) and the survey identified several methods through which the incline of the pile was measured during the ongoing boring process by contractors. However it was also noted that very few contractors (22 %) adopted the practice and the few that did carry it out did so under the specific requirement and the constant supervision of the Engineer/Client. Most contractors relied on eye estimation, especially to check the verticality of the Kelly Bar when the method of drive by Auger was used in boring. The methods if adopted, were simple whereby for boring by winch and chisel included simply measuring the incline of the DMC (Direct Mud Circulation) rods and in the case of drive by Auger verticality of Kelly bar is checked at-least to eye level or by use of plumb bob.

2.4 Bentonite

Bentonite as supplied to the site and prior to mixing shall be in accordance with DFCP 4 and *shall be mixed thoroughly with clean fresh water to make a suspension which will maintain the stability of pile excavation until place of concrete and complete construction.* Also *where saline or chemically contaminated groundwater occurs special precaution shall be taken to modify the bentonite suspension or pre-hydrate the bentonite in fresh water.* (ICTAD/DEV/16) However as highlighted in section 2.1inadequate chemical analysis carried out, indicates that this is not carried out in practice.

Control test shall be carried out on the bentonite suspension using suitable apparatus. The density of freshly mixed bentonite suspension shall be measured ***daily*** *as a check on the quality of suspension being formed.* (ICTAD/DEV/16) However only 56 % of contractors carried this out in practice and relied instead on estimation and experience to decide on the density levels of the suspension.

Bentonite slurry is returned to the surface for sedimentation and de-sanding. Cleaned bentonite slurry may be re-used, with adjustment of bentonite content by the addition of new bentonite or water as necessary. After several re-uses, the bentonite slurry may become too contaminated for effective cleaning, and should then be discarded.

Before placing concrete, measures shall be taken to ensure that there is no accumulation of slit or other material and heavily contaminated drilling fluid, (which could impair the free flow of concrete from the pipe of the tremie) at the base of the boring. A sample of the drilling fluid shall be taken from the base of the boring using an approved sampling device. If the specific gravity of the suspension ***exceeds 1.25*** *the placing of concrete shall not proceed. Then the drilling fluid must be modified or replaced to meet the specification.* (ICTAD/DEV/16) However this was not carried out in actual practice.

Concreting is usually carried out by the tremie method, using concrete with a slump in excess of 150 mm. With clean bentonite slurry in the borehole, the suspension is effectively scoured and displaced by the fluid concrete. Inadequate cleaning of the borehole will result in bentonite and/or mud trapped within the reinforcement cage and/or concrete resulting in defects such as weakening of the concrete mix due to excess water from the slurry, reduction in pile diameter and exposure of reinforcement cage due to build up of filter cake, contamination of concrete etc. The survey observed one site where piles had to be hacked and built up due to contamination resulting from improper cleaning methods before concreting.

2.5 Reinforcement Cage

According to ICTAD/DEV/16 – clause 3.05, *no heating of reinforcement bar shall be permitted for bending of the bars. All intersection bars shall be tied together with approved wire.* Also *Reinforcement in the form of a cage shall be assembled with additional support, such as spreader forks and lacings necessary* *to form a rigid cage* and *Shall be clean and free from loose rust and loose mill scale at the time of fixing in position and subsequent concreting.* 90% of sites packed all reinforcement used for cage fabrication upon timber and were adequately covered to avoid rusting. Cleaning of the cage is specified so as to be devoid of any mud or dirt which in turn affects its bonding capacity. This can be done manually, as well as by aiming a powerful jet of water at the cage just before lowering.

However there are no specification in Sri Lanka of minimum amount of reinforcement to be used and clear distances between longitudinal bars and consideration should be given to specifications given in BS EN 5366 and IS 2911 to be adopted in the Sri-Lankan context. Some of the notable recommendations include;

- Minimum clear distance between two adjacent main reinforcement should *normally be 100mm* for full depth of cage. The diameter or spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handling and installation.
- Minimum diameter of links or spirals shall be 6mm and the spacing of the links or spirals shall be ***not less than 150mm***
- The concentric positions or the cage in the bore, and shall be provided by spacers (unless the position and cover are otherwise provided) and arranged symmetrically around the cage with ***at least 3*** numbers at each level, at level intervals of not more ***than 3.0m*** with sufficient tolerance to the inner wall of casings or the wall of the pile bore to allow safe installation and avoid damage to the bore wall.
- Reinforcement cages shall be suspended or supported so as to maintain their correct position during concreting.
- Pile should be ***50mm in*** to the pile cap concrete.

2.6 Concreting

Good quality concrete in bored piling sense means that the properties and characteristics of the concrete are suitable for the process of work and subsequently meet requirements of the finished product.

Characteristics of Concrete and Concrete Mix

Concrete mix for bored piles is designed according to concrete pouring process and mechanical properties required and needs to be specially mixed having cohesiveness with high workability (high slump/excellent fluidity) which is not prone to segregation and retain it workability as far as possible throughout the tremie placing operation for the complete pour. In addition to those characteristics, compaction under self-weight, resistance to harsh environment, resistance to leaching, and strength are essential.

In general terms concrete suitable for constructing cast-in-place piles should be free flowing yet cohesive; the mixes designed to be self-compacting. The slump test is quite adequate to assess the former property; however, cohesiveness or resistance to segregation is difficult to define. No delay in pouring should occur due to field test. Adding of water to the concrete with very low slump on site to increase the workability can have detrimental effect of reducing the strength, compactability and impermeability of the concrete. The results of adding water could cause a significant change in the characteristics of mix and the possibility of segregation as the pour is made. Segregation of concrete during pouring can also lead to increase in permeability of concrete, especially at the top section of piles due to upward migration of water in the concrete mix. Adding of water to the concrete on site must not be allowed unless specified.

Concreting

The quality of piles depends on a good pouring procedure as well as on good quality of concrete. The tremie pipes in suitable lengths having a diameter of minimum 6 times of coarse aggregate size and watertight joints should be lowered up to the bottom of the bore.

After the completion of tremie pipe setting, Bentonite slurry should be pumped under high pressure through tremie pipes to the bottom of the hole. This is to ensure that the rock particle, sand and sludge sediment at the bottom of the bore is brought up together with Bentonite slurry. It is very important to continue the circulation of Bentonite slurry until the arrival of concrete.

Prior to charging the tremie pipe with concrete, the bottom of tremie needs to be sealed and a plug is usually inserted at the top of tremie. There are two potential problems that are associated with the initial charging of tremie with concrete which include firstly that the concrete can segregate during placement and secondly that the air in tremie will prevent the complete filling of the tremie. These problems can be avoided if the tremie is filled slowly. Faulty initial charging of tremie during concreting can cause entrapment of mud within the concrete.

Excessive initial lifting of tremie can result in possible distribution of leached concrete caused by concrete falling through the slurry. The bottom of tremie must stay well below the top of the column of fresh concrete all the time. *The pipe shall be at all time penetrate the concrete which has previously been placed and shall not been placed and shall not be withdrawn from the concrete until withdrawn from the concrete until completion of concreting.* (ICTAD/DEV/16)

The tremie must not be lifted and lowered rapidly to avoid the cause of contamination of concrete with slurry. It is also suggested that the tremie pipe must not be lifted and lowered rapidly to start or restart the flow of the concrete (Reese & Neil 1988). This was a common practice carried out in all the piling sites visited during the survey. However Xanthakos (1994) suggested that if the concrete is not deposited easily the tremie pipes may be moved up and down with movement not exceeding 300 mm.

The concrete should be supplied in sufficient quantity to ensure that the concreting of each pile proceeds without interruption. (BS 8004) Continuous concrete pouring is mandatory in piling and it is sometime disrupted by blockage of segregated or prematurely set concrete mix in the tremie pipe. Early setting of concrete after pouring in bored hole can also cause discontinuities in pile by accidental lifting of set concrete during extraction of the temporary casing. However it was observed that although 90% sites order concrete in sufficient quantity to carry out the piling process continuously the dispatch of concrete from the batching plants are in irregular intervals and 33% sites face problems due to arrival times between trucks varying from 0.5 – 2 hours.

Ready mixed concrete for bored piles usually specified as self-compacting concrete and its "compacting factor" is very important for achieving required strength, especially at the top section of pile which usually carries the maximum portion of transferred load. *The concrete will need to have adequate workability so that it can flow against the wall of the shaft, and into every cavity.* (BS 8004)

In the area of high ground water level, the concrete must be deposited above the external water table before the casing is withdrawn. The hydrostatic pressure in the concrete column should be greater at all time than the pressure in any column of fluid outside the casing.

3.0 Conclusions

Soil investigation is a very important part in the effort towards the construction of a good quality pile. This is because it is the starting point of the piling construction process and is the basis of all planning. Also for projects, which are near sea, water and rivers, variable soil characteristics and ground water conditions can be encountered along the project area. In such cases, concrete mixes need to be designed to suit these conditions. However, no standard procedures are available as to a minimum required number of observation boreholes to be carried out, insufficient chemical analysis etc.

The common defects of piles are cold joints, zone of segregated or contaminated concrete, trapping of bentonite mud and cavities. These defects occur mainly due to problems in concrete and concreting. Foundation designers and concrete suppliers should pay more attention to workability and many other important factors required for cast in-situ tremie concrete than strength alone. It is essential to design a better quality mix than the concrete for other structural works in some aspects, considering the process of work. It is concluded that appropriate concrete mix and casting practice is essential in bored piling work to achieve good quality piles.

4.0 Suggested Recommendations

Soil Investigation is of utmost importance to save costs and properly plan out the construction process. Special considerations to be noted are;

- To have an adequate number of boreholes, showing consistency on the usage of the number of boreholes. It is recommended to propose at least one borehole per designated maximum area of plot to provide a consistent practice in the industry.
- At-least some boreholes should ideally be located in positions where load transfer to the ground is envisaged.
- Chemical analysis should be carried out on the ground water and soil and the necessity for special precautions analysed.

There is no mention in the specifications commonly used in Sri Lanka of minimum amount of reinforcement to be used and clear distances between longitudinal bars. However clear specifications are given in BSEN 5366 and IS2911 which can be adopted in the Sri-Lankan context. Some of the notable recommendations are included in section 2.5.

Other specifications given in BS EN 5366 and IS 2911 which can be adopted in the Sri-Lankan context for concreting include:

- During the concreting the volume placed and the level of concrete inside the bore shall be checked & recorded.
- The method and the sequence of the checking and recording shall suit the dimensions and type of the pile and shall be agreed prior the beginning of the work.
- The level shall be checked at least once after every pour or before or after a temporary casing is lifted.
- For piles of diameter ***less than 0.6m*** it may be sufficient to record the concreting of the first ***10 piles*** of a sites & a percentage of the remaining piles.

Formation of cold joints is a problem in Sri Lanka, due to inefficiencies in part of the concrete suppliers. The method of resuming concreting should be regulated as the current practices carried out are in direct contrast to recommendations in foreign codes of practice (eg: IS 2911: Clause 7.5.2)

- Concreting of piles should be uninterrupted.
- In the exceptional case of interruption of concrete but which can be resumed within 1hr or 2hr, the ***tremie shall not be raised and lowered slowly***, from time to time to prevent the concrete around the tremie from setting.
- Concreting should be resumed by introducing a little richer concrete with a slump of about 200mm for easy displacement of the partly set concrete.
- If the concreting cannot be resumed before final set of concrete already placed the pile so cast may be rejected or accepted with modifications.

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ESTIMATION OF ULTIMATE CARRYING CAPACITY OF BORED PILES

Sutharshan.S,Thivaharan.O,Rupan.N
(University of Moratuwa,Sri Lanka)

Abstract

This paper describes a project carried out to estimate the ultimate carrying capacity of bored piles, which are widely used for the pile foundation in Sri Lanka. In practice only end bearing capacity is considered in the design neglecting the skin friction from the soil layers. The actual skin frictions of bored piles were obtained from testing of piles using the Pile Driving Analyzer (PDA). The skin friction capacity was estimated using commonly used methods such as: ICTAD, Kulwathy, Broms and Burland and, then, the estimated capacities were compared with actual skin frictional capacities mobilized during dynamic testing of bored pile using PDA.

1. OBJECTIVES

In Sri Lanka, when the ultimate carrying capacities of bored pile are estimated, the skin frictional capacity is mostly neglected due to shallow bed rock and use of bentonite slurry in drilling for the bored piles. But neglecting the skin friction become very costly if the bedrock is deep and therefore the pile becomes unnecessarily longer.

In this research project, the ultimate skin frictional carrying capacity of bored piles was estimated using several methods such as; SPT-ICTAD method, Kulwathy method, Broms method,and Burland method. Until recently, no measured capacity of skin friction and end bearing were available for comparison with the estimated capacities. But with the availability of Pile Driving Analyzer (PDA) measured skin friction and end bearing capacities are available. Estimated total skin friction was compared with measured skin friction to investigating the accuracy of different methods.

2. PILE FOUNDATION

Piles are long and slender members, which transfer the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity. Based on the load carrying mechanism, piles can be mainly divided into two types:

1 End bearing pile; and
2 Skin friction piles,

2.1 End Bearing Piles

These piles transfer their load mainly on to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile. The pile behaves as an ordinary column and should be designed as such. Even in weak soil a pile will not fail by buckling and this effect need only be considered if part of the pile is unsupported, i.e. if it is in either air or water. Load is transmitted to the soil through friction and/or cohesion.

2.2 Skin Friction Piles

Carrying capacity is derived mainly from the adhesion or friction of the soil in contact with the shaft of the pile. These piles transmit most of their load to the soil through skin friction. The pile is driven far enough into the lower material to develop adequate frictional resistance.

3 ESTIMATION OF SKIN FRICTIONAL CAPACITY

In pile foundations, the load is carried both by skin friction and end bearing. Therefore, from a static analysis, the ultimate carrying capacity of a single pile can be obtained as:

$P_u = q_u . A_b + f_u . A_s$

Where P_u = applied load at ultimate Carrying capacity of pile;

q_u = ultimate value of end resistance Per unit area of base;

f_u = ultimate value of skin friction coefficient per unit surface are of shaft;

A_b, A_s= area of base, and surface area of shaft respectively.

The β method

Burland (1973) proposed the following equation to estimate the skin friction, f_u at a given depth z:

$f_u = Kq \tan\delta$

Where q - effective overburden pressure

K_o - Lateral earth pressure coefficient

δ - Angle of friction between soil and the pile material

β is defined as, $\beta = K \tan\delta$

Estimation of f_u in clays

$$f_u = \alpha C_u$$

Where the coefficient **α** is a function of the stress history of the clay and taken from empirical graph.

Estimation of f_u based on SPT N Value

ICTAD Method

It is recommended to use

$$f_u = 1.3 * N \quad \text{kN/m}^2$$

Limiting value of f_u=100 kN/ m^2

Estimation of f_u in rocks

ICTAD Method

This estimated using SPT value N. It is recommended to use $f_u = 2 * N$ kN/m^2

With the further condition that f_u should not exceed 200 kN/m^2.

Estimation of f_u based on angle of internal friction Φ

ICTAD Method

1 $f_u = K_s * p(D) \tan\delta$

Where K_s = coefficient of lateral earth pressure on shaft;

p(D) =vertical effective stress at any depth;

δ = angle of friction between concrete and sand

A) Estimation of K_s as per ref(1)

For bored and cast in-situ piles .K_s is related to the K_0 value as follows:

$$K_s / K_0 = 0.7 \text{ To } 1$$

B)Estiation of p(d)

In the case of p (d), the recommendation made, is that for practical design purpose that

1) P(d) increases linearly with depth up to pile lengths of 10 or 15 diameters; and
2) Below the pile lengths given above, p(d) is constant with depth.

C) Estimation for δ

Values of δ are related to the friction angle Φ,Gives the recommendations of 2 researchers;

Kulwathy: $\delta / \phi = 1.0$

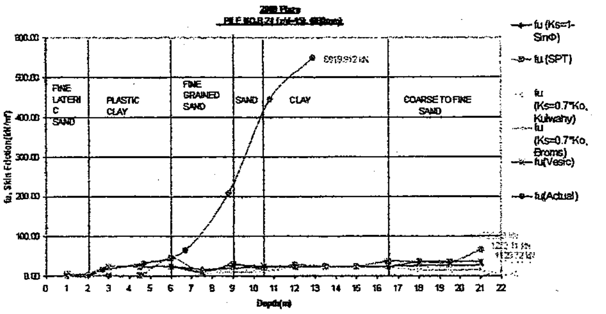
Broms: $\delta / \phi = 0.75$

4 ANALYSES

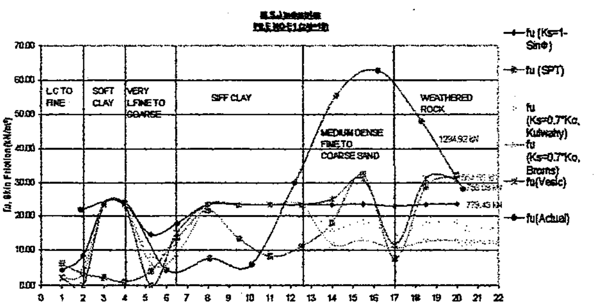
Four sites were considered in this analysis with total no of twenty three piles. In this analysis the following data were considered for calculations.

- Soil investigation report
- CAPWAP analysis report
- Site plan with Bore Hole locations
- Tested pile locations
- SPT N value with Borehole

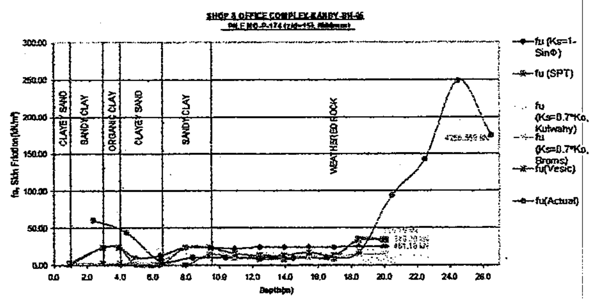
Soil parameters and test results are in the soil investigation report. The SPT values which are taken from the soil investigation report are used to find the parameters from the empirical graphs. Before finding out the parameters, the fields SPT Values were corrected using Energy method (Bowles 1996). Some typical measured and estimated capacities are given in figure 1.



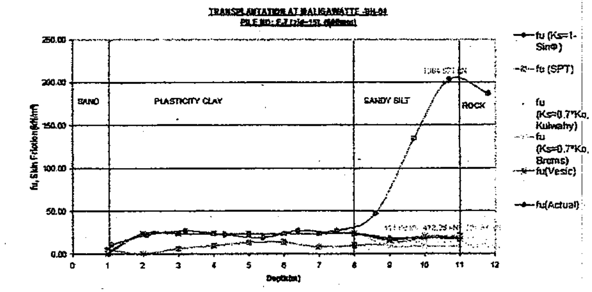
(a) 2000 plaza site



(b) MSJ industries site



(c) Kandy site



(d) Malligawatte site

Figure 1: Estimated and measured skin friction capacities.

P#	(a)	(b)	(c)	(d)	(E)
2000 Plaza					
B-24	1314	1130	1327	1015	885
K-13	669	531	593	489	429
T-28	1556	428	596	388	324
C-3	2432	779	555	660	569
D-3	2131	779	555	660	569
E-1	1295	779	555	660	569
Transplantation At Maligawatte					
E7	1365	413	325	381	359
A1	1993	1213	1223	1019	840
B5	6039	1209	1030	1115	1006
RW13	2765	875	1055	808	735
RW6	2700	760	541	681	634
C3	4026	1213	1223	1019	840
D561	2202	673	966	615	583
KANDY AND SHOP COMPLEX					
P 103-104	1593	508	145	444	409
P 174	1257	461	118	397	368
P 40-41	1788	461	118	397	368
P 45	413	768	382	735	698
P 51	154	613	201	613	613
P 59-60	540	1426	374	987	701
P 69-71	713	522	109	487	473
P 73	768	447	135	385	350
P 79	1450	768	382	735	698
P 85	1495	768	382	735	698

Table1.Total skin friction (kN),(a)measured (b)Burlandmethod(c)ICTADmethod (d)Kulwathy and (e)Broms

5 DISCUSSIONS

Since only few boreholes were available per site, the nearest borehole was considered for the tested pile location in estimating the skin frictional capacity. Due to this reason the predicted value may differ from the measured value. It the actual soil layer distribution will get on that location were obtained using the 3D software of Arcview, a better agreement between the measured and the actual skin frictional capacity may be expected.

In case studies considered, soft to medium and fractured rock mainly comprising Biotite Feldsbar bands or Feldshatic Gneiss with weathered bands are generally observed in the upper layers of the weathered rock stratum. Though RQD (Rock Quality Designation) values of this rock often lie well above 50%, this rock appears to be moderately strong.

When the total measured skin frictional capacities are compared with the predicted capacities. It clearly seen that there is a very significant amount of skin friction is developed and all the methods under predicts the actual total skin frictional capacity.

When the measured skin frictional capacities are compared with the estimated capacities, it is clearly seen this prediction methods under predicts the capacity, especially, in the weathered rock layer.

6 CONCLUSIONS

Based on the results of this study, it is concluded that:

It is clearly seen that significant amount of skin friction is developed in bored piles and pile design can be made more economical by considering skin frictional capacity in the design.

Total skin friction was under predicted in most of the pile by all methods. Skin friction value was highly under predicted by all methods in weathered rock region. Estimated skin friction is about 5-6 *N in weathered rock region, where as ICTAD method considers 2*N.

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DEVELOPMENT OF CORRELATIONS FOR CONE PENETROMETER RESULTS FOR SRI LANKAN SOILS

MOHANARAJ, R.T., NANTHAGOPAN, P., SIVASITHAMPARAM, N., ABAYAKOON, S.B.S.
UNIVERSITY OF PERADENIYA, SRI LANKA

ABSTRACT

In Sri Lanka the geotechnical designs based mainly on the standard penetration test (SPT) results. However countries in Europe and America are to carry out the cone penetrometer test (CPT) results. A study is carried out to find the correlation between CPT & SPT results in some Sri Lankan soils. SPT & DCPT tests were carried out using PAGANI TG-63 cone penetrometer which has a 100 kN capacity. These two tests were carried out at close proximity so that the change in underground stratigraphy is negligible. Some useful correlations have been developed and presented.

GENERAL BACKGROND

Prior to designing any civil engineering structure, a proper knowledge of the strength and deformation characteristics of the underlying soil strata is very important. To determine the relevant physical properties of the soil and the variation of these properties within a site, usually a subsurface exploration program is carried out. This has led to the development of considerable number of different in-situ test techniques. standard penetration test (SPT) has become the most widely used soil test in sri lanka. However, its reliability is somewhat variable in deposits other than those make of fine sands.

The Cone Penetration Test (CPT) is a simple logging test that is becoming very widely used in lieu of Standard Penetration Test (SPT) for soft and fine to medium coarse sands. CPT can be divided into two categories;

- Dynamic Cone Penetrometer test (DCPT)
- Static Cone Penetrometer test (SCPT)

DCPT is a continuous test in which an impacting weight drives a rod into the ground. Number of blows per 200 mm penetration is recorded. The rod is usually equipped with a cone tip with a 10 cm^2 cross sectional area, although considerable variation exist world wide. The falling height and the impacting weight of CPT are 760mm and 63.5kg respectively which are usually same as for the SPT.

SCPT test carried out continuously uses a cylindrical cone with a projected base area of 10 cm^2 and an apex angle of 60 degree, which is pushed vertically into the ground at a constant rate of 10 to 20 mm/sec. During penetration, measurements are made of the variation of cone resistance (q_c), and the skin friction (q_s) with the depth. The measurements are made and recorded using electronic devices; the frequency of the readings provides a detailed picture of the variation of the measured parameters with penetration depth.

There is a significant difference in the quality of site investigation practices between developed countries and Sri Lanka. The Sri Lankan practice of in situ site investigation is still concentrated around the use of SPT. However despite continued efforts to standardize the SPT procedure there are still problems associated with its repeatability, reliability and comparability between different regions and different countries. The need to use new or improved methods of site investigation by using good sampling and laboratory testing or by employing new equipment/techniques in in-situ testing is extremely important. CPT test is a quick, more reliable, easy to carry out and a repeatable technique. In SPT and DCPT number of blows to drive the rod for certain depth is recorded. This project concentrates on the correlation of DCPT/SPT.

The PAGANI TG-63 100kN is one of the latest cone penetrometer which can carry out SCPT/DCPT quite efficiently. Fig 1 shows the equipment which is portable & easily moveable with a reasonably large area.

Using Pagani TG 63-100kN penetrometer following test can be carried out with addition of proper accessories.

- Dynamic Cone Penetration Test (DCPT)
- Static Cone Penetration Test (CPT)
- Standard Penetration Test (SPT)
- Soil sampling using thin wall tube sampler]

AIMS AND OBJECTIVES

1. Development of DCPT/SPT correlations for Sri Lankan soils.
2. Determine sub-surface stratigraphy and identify materials present.
3. To estimate geotechnical parameters (C,ϕ etc.).

METHODOLOGY

Throughout this study the SPT & the DCPT were carried out using the TG 63-100KN at a site. These two tests were carried out at close proximate so that the change in underground stratigraphy is negligible

- Pagani TG 63-100 kN penetrometer was taken to site and securely parked on a nearly horizontal ground
- The stem was elevated to the almost vertical position Pagani TG 63-100 kN
- Using three hydraulic legs machine was leveled
- Two side anchors were driven into ground to maintain the stability of machine
- Depending on the test required accessory (Begmann cone or Split sampler) was fixed with the rod and attached to the machine

TESTING PROCEDURE

SPT

The split sampler was attached to a rod and it was connected to the machine. Then using a 63.5 kg hammer dropping form a height of 760mm an impact force was given to the rod. Number of blows to penetrate 150 mm was recorded. After penetrating to 450mm depth the sampler removed from the ground and sample was collected. The cleaned sampler was connected again with the rod attached to the machine and pushed to the previous penetrated level and the test was preceded.

DCPT

The Begmann cone was attached to a rod and it was connected to the machine. Then using a 63.5 kg hammer dropping form a height of 760mm an impact force was given to the rod. Number of blows to penetrate 150 mm was recorded. After penetrating to 1m depth another rod was connected again and the test was preceded.

OBSERVATION AND RESULTS

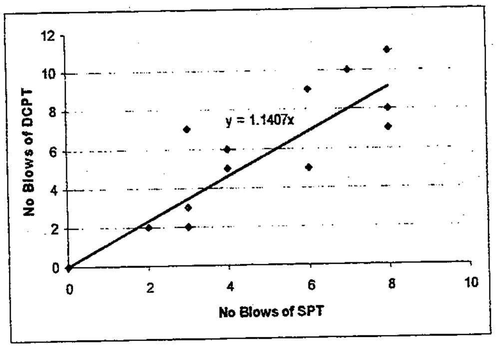

Fig 1: The graph shows DCPT Vs SPT variations for accumulated data

RESULTS

1. Based on analysis done above, it has been found that No. of blows of DCPT is equivalent to 1.1 that given by SPT per 300mm penetration with Pagani TG 63-100 kN geotechnical equipment.

DCPT = 1.1 SPT

2

2. Internal angle of friction Vs DCPT N-values

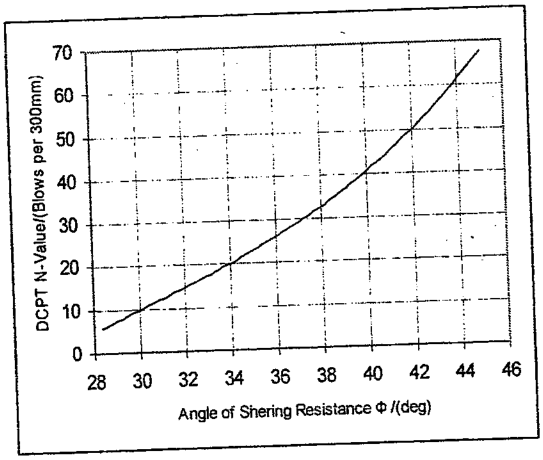

Fig 2: The graph shows correlation of DCPT-N values with Φ values.

Note: Above graph has been modified from correlations obtained by Peck, Hanson and Thornburn for SPT N values with Φ value.

3. Relative Density of Soil

DCPT N-value	Relative Density of Soil
0--4	Very loose
5--9	Loose
10--26	Medium dense
27--44	dense
>44	Very dense

Table 1

Note: Above table has been modified from correlations obtained by Peck, Hanson and Thornburn for SPT N values with Relative Density of Soil.

DISCUSSION

In our study DCPT and SPT N values per 300mm penetration were taken to make comparison easy. But the rod of this machine has divisions of 200mm and according to which divisions the tests will be done in future for easiness. Therefore to use the correlation which we have found above directly in the following conversion should be done.

$$(DCPT)_{150} = .75\,(DCPT)_{200}$$

DIFFICULTIES

Static Cone Penetration Test

While doing the Static Cone Penetration Test the following difficulties were observed at certain sites. Due to the plugging of soil with the sleeve. This restrains the relative movement between the sleeve and the cone thus the data obtained were not reliable. And other major difficulty is that the machine tends to lift when the vertical force exceeds 75 kN.

Dynamic Cone Penetration test

The main problem at sites with soft soils in using DCPT is the cone penetrating into ground without any blows been applied (with only self weight of rods plus the hammer weight acting on cone). Steel rods further bend when the strata are very stiff.

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EVALUATION OF SHEAR VELOCITY OF SOIL USING SURFACE WAVES GENERATED BY DIFFERENT SOURCES

Thurairajah A, Thirumaran N, Sutharsanan S and Abayakoon S.B.S
Department of Civil Engineering, University of Peradeniya, Sri Lanka.

ABSTRACT

A project was carried out to evaluate the shear wave velocity of soil using surface waves generated by impact source and continuous vibrating sources and to compare the results. Shear wave velocity is one of the most important parameters used in dynamic analysis of the soil profile. This parameter is used in many field applications especially in geotechnical earthquake engineering analysis and vibration of foundations. Although there are laboratory tests available to evaluate shear wave velocity of a soil profile, here seismic method was used to determine the shear wave velocity, as seismic methods have the advantage of not being affected by sampling disturbance and insertion effects.25kg and 67kg weights were used as the impact sources to generate surface waves, which are sufficient to determine shear wave velocity of soil profile up to 10m.Surface waves were generated by the movement of trains and road vehicles also taken into consideration and the data is analysed.

INTRODUCTION

Shear wave velocity is one of the most important parameters used in dynamic analysis of a soil profile. This parameter is used in many field applications especially in geotechnical earthquake engineering analysis and vibration of foundations.

There are many methods available to determine the shear velocity. In the laboratory this can be determined by means of cyclic triaxial test etc. However field methods are superior because Seismic methods have the advantages of not being affected by sampling disturbances and insertion effects. Here the surface wave is measured based on the relationship between shear waves and the Rayleigh waves. In this analysis Rayleigh wave is developed along the ground surface using an impact or vibration on the ground.

When a hammer strikes the ground, basically two types of elastic waves will be generated. They are body waves and surface waves (Rayleigh waves). Out of these two wave types, surface wave can be detected at the ground surface. Therefore we have chosen to use surface waves for our analysis.

There are many types of sources such as hammer impact, sinusoidal vibrators and impact due to the heavy vehicles such as trains, containers, trucks etc having the potential of developing the surface waves.

Basically there are two methods available to analyse the surface waves. They are spectral analysis of surface wave (SASW) and continuous analysis of surface waves (CSW). SASW uses a hammer blow as an energy source and CSW uses a steady state vibrator as an energy source. In the case of SASW lack of frequency control and resolution is the major limitation. CSW method, which is relatively costlier than SASW method, has the advantages of frequency control and frequency resolution.

METHODOLOGY

EXPERIMENTAL PROCEDURE

Surface waves developed by an impulsive hammer or by a continuous vibrator were detected by geophones fixed on the ground. The analogue signals were converted into time domain signals using A to D converter. By using mat lab software these signals were converted into frequency domain signals.

INSTRUMENTS USED

An impulsive hammer or heavy vehicle or train- To generate surface waves along the ground.

1. Geophones - To detect the surface waves.
2. A to D converter - To convert analogue signals to digital signals.

EXPERIMENTAL SET-UP

In this evaluation different types of experimental setups were carried out. Those consist

1. different type of Geophone configurations
2. different types of impact sources.
3. different weights for impact hammer (25 kg and 67 kg).

DATA ANALYSIS

Data collected were converted into frequency-based signals by using a mat-lab programme. Plots for variation of Coherence and Phase difference with the frequency were obtained by using the above-mentioned programme. Also graphs for variation of Amplitude with Frequency for both collected data and noise were taken into consideration in determining the frequency range suitable for the evaluation.

In selecting the frequency range amplitudes of both collected data and noise were compared and the range having amplitude of about ten times higher than that of the noise was selected. This frequency range was further narrowed in so that that all the data have a coherence (coherence is a function with values between 0 and 1 that indicates how well the input corresponds to the output) greater than 0.9. Subsequently phase differences between signals for selected frequency range were obtained from the plots and graph for the variation of shear velocity with depth was derived.

1

RESULTS AND DISCUSSION

Some configurations used are presented below; CH indicates the slot of signal receiving channel connecting A to D converter and geophone.

Configuration-1

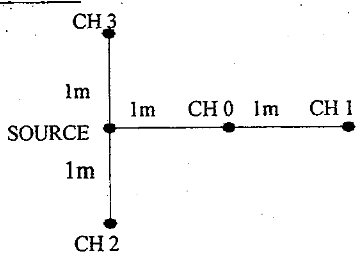

Fig 1

Configuration-2

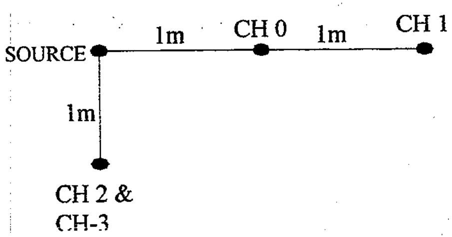

Fig 2

Configuration-3

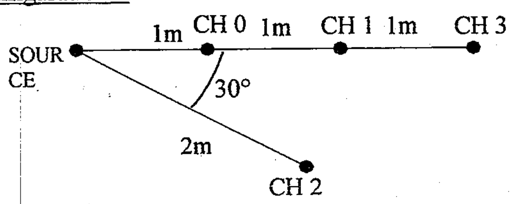

Fig 3

Configuration-4

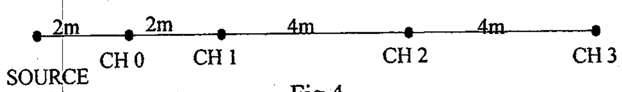

Fig 4

Configuration 5

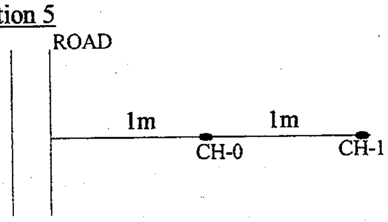

Fig 5

Configuration-6

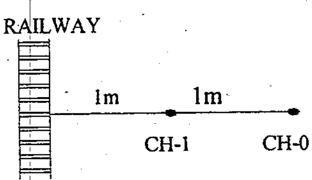

Fig 6

Configuration-7

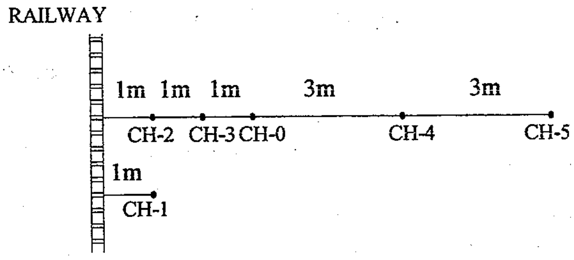

Fig 7

Figure 8 was generated using the results from the Figure 2.

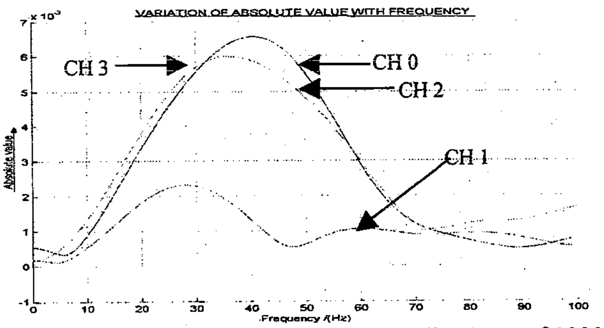

Fig 8 Absolute Value Diagram at Sampling Rate of 1000 Hz

It has been observed that the geophone in the 1m distance receives useful signals with high signal to noise ratio for a significant frequency interval, and the geophone at 2m distance receives signals with high signal to noise ratio for a short frequency interval. Further it has been observed that the geophones at the same distance from the source show the same behaviour.

Figures 9 and 10 present the results obtained using fourth configuration (Fig 4) with two hammer weights 25kg and 67 kg respectively dropped from different heights.

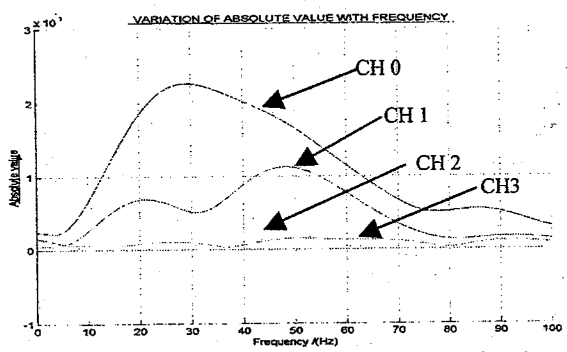

Fig 9 Amplitude Diagram for the Weight of 25 kg Dropped from 1m Elevation

It is clear from figure 9 that the geophone at 2m distance from the source receiving the signals with high signal to noise ratio for a shorter frequency range and the geophones at 8m and 12m are almost not receiving useful signals.

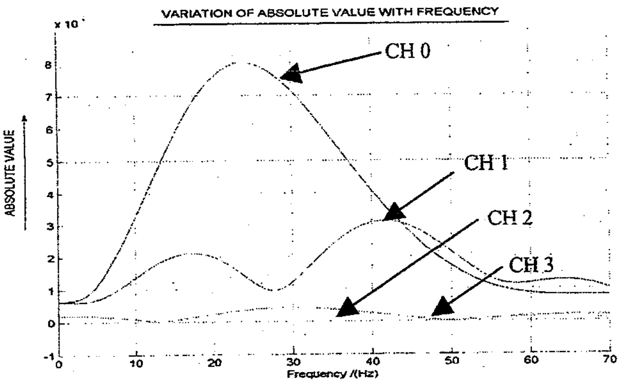

Fig 10 Amplitude Diagram for the Weight of 67 kg Dropped from 2m Elevation

It has been clearly observed from Fig 10 that the variation of height in the variation in the amplitude of two closer geophones from the source. Once again the geophone located 2m away from the source receives much better signals than any other for a wide range of frequency and the one located 4m away from the source receives quality signals only for a short frequency range.

In the cases given in Figures 6 and 7 respectively, train is used as the source of vibration. Higher amplitudes were obtained for the fast moving train taken at Kattuppatiya, Gampola than the slow moving train taken near the Penideniya station as shown in Figures 11 and 12 respectively.

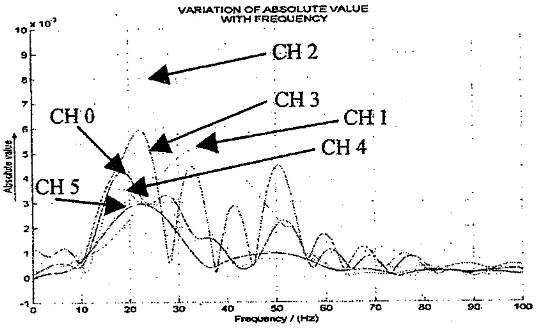

Fig 11 Amplitude Diagram for Fast Moving Train

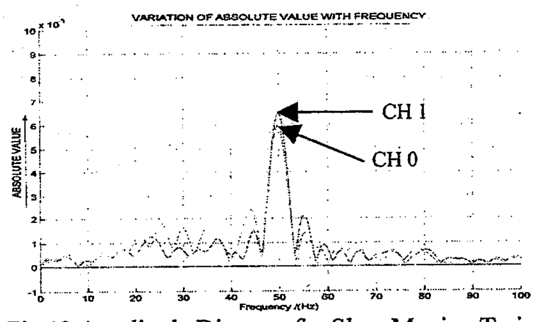

Fig 12 Amplitude Diagram for Slow Moving Train

In the case of fast moving train, a large frequency range can be picked where the amplitude is considerably high. This is not the case when the train is moving slowly.

The observations with the trains do not show smooth variations of phase difference with the frequency as shown in figures 13 and 14. Although we were quite successful in capturing the signals, we have not been able to analyse this results due to the inconsistency in variation of phase difference with frequency.

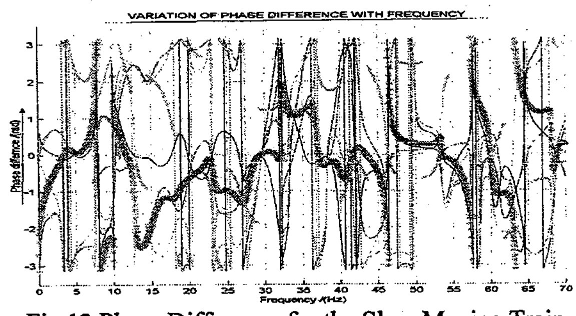

Fig 13 Phase Difference for the Slow Moving Train

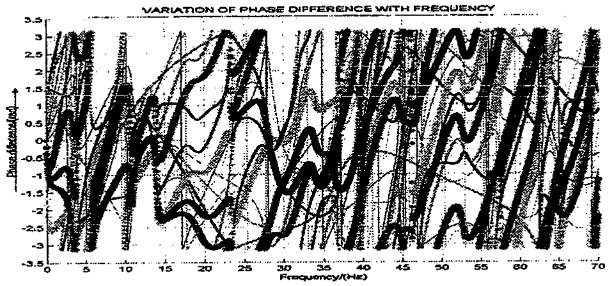

Fig 14 Phase Difference Variation for the Fast Moving Train

Figure 15 presents, for comparison purpose, the phase difference obtained when a hammer is used as the impact source.

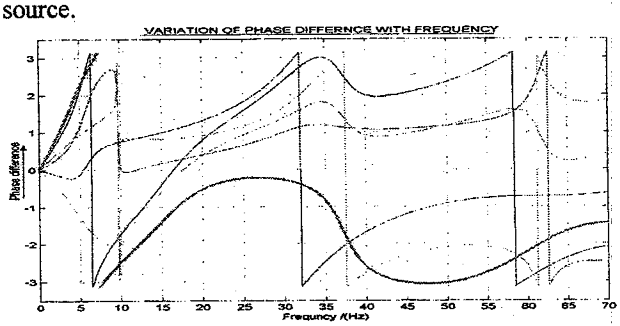

Fig 15 Phase Difference Generated from Impact Source-Given for the Comparison Purpose

When the train moves along the rail track the impact given to the rail track will be transferred to the ground, as impulse energy develops the surface waves in the surrounding soil mass. Hence surface waves will be generated by a set of moving points load distributed along the railway tracks.

Therefore there may not be good correlations between the geophone signals generated by these surface waves. However the intention of the current work is to check any useful results, which can be derived from the surface wave data generated by a moving train.

Fifth configuration, (Fig 5) was used to analyse road vehicles as the source of vibration, but as expected it has not been able to capture useful signals during this analysis. When a bus moves along the road energy given to the road will be reduced on the road base. So as the

3

results, no effective impulse will be transferred to soil mass, just as in the trains.
Therefore there may not be correlations between the geophone signals generated by these surface waves.

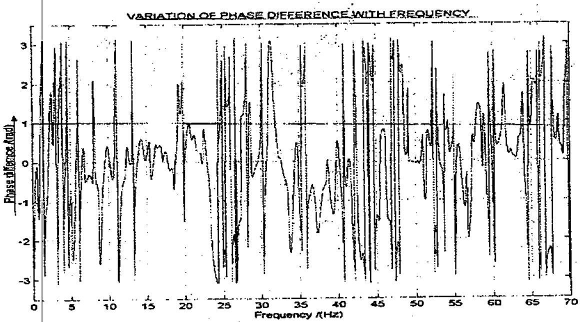

Fig 16 Phase Difference for the Bus

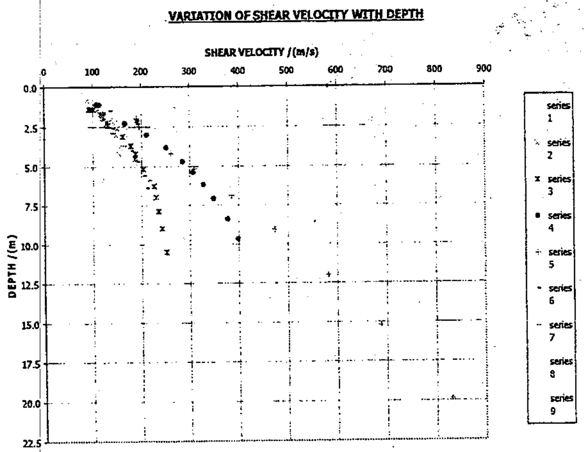

Fig No 17

Figure 17 presents variation of Shear Wave Velocity of the soil with the effective depth based on energy transferred by impact sources. From this figure it is obvious that sources with high energy can be used to determine shear wave velocity at deep. Also figure shows that different configurations give variation in shear wave velocity at deep and in high wave velocity. So it is necessary to consider about the configuration setup also.

LEGEND

SERIES	CONFIGURATIONS	SIGNAL RECEIVED BETWEEN GEOPHONES CHANNELS
01	1	1,2
02	3	2,3
03	3	0,2
04	6(large weight, 1m)	1,0
05	6(large weight, 2m)	1,0
06	6(small weight, 2m)	1,0
07	6(small weight 1m)	1,0
08	2	1,2
09	2	1,3

Table-1

CONCLUSION

1) Impact sources (25 kg and 67 kg with potential energies of 245.25J, 490.5J, 657J, 1314.5J) generate surface waves of good quality which can be used to determine the shear velocity down to 10m of the in-situ soil profile.
2) Fast moving train produces high strength surface waves and the slow moving train produces the surface waves with the low strength.
3) Since there is no linear trend of wave variation, the analysis based on moving train as well as road vehicle as energy source has shown the non-linear phase variation between the geophone signals and therefore these methods cannot be used for the determination of shear wave velocity.

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STRENGTH CORRELATION FACTORS FOR SOME SRI LANKAN RESIDUAL SOILS FOR THE ANALYSIS OF LANDSLIDES

NANTHINI.V, THULASIMATHY.S, DISSANAYAKE.A.K.
University of Peradeniya.

ABSTRACT

This research study presents the correlations of laboratory shear strength parameters and field mobilized shear strength parameters for the analysis of landslides. The laboratory shear strength parameters are based on the triaxial test results and the field shear strength parameters are obtained by analyzing the actual failure of slopes using the back analysis technique.

The increasing population of the island demands that new engineering structures to be located on sloping ground and / or excavation, which increase the slope angle, thus posing many problems.

Due to this heterogenic property, the engineering properties of small samples of residual soils are mostly non-representative of the mass, thus rendering conventional method of field testing, laboratory testing, and analysis procedure for slope stability become unsatisfactory. There are no realistic testing and analytical models available to represent residual soils in the geotechnical engineering aspects.

In order to find field shear strength, three landslides were reassessed and analyzed to obtain the field shear strength parameters using the back analysis technique. On comparison with laboratory shear strength parameters obtained from triaxial tests, correlation factors were ascertained. This correlation factor may be used to modify laboratory shear strength parameters to obtain fairly actual field shear strength values of residual soils for design purposes.

INTRODUCTION

In SriLanka, landslides occur, more frequently, in the central highlands, particularly during the monsoonal rainy periods. So, several parts of the central highlands of SriLanka frequently suffer from landslides, earth slips, and other types of mass movement of earth and rock. Although the areas so affected are relatively small, these earth slips cause a great extent of damage and inconvenience, and the problems arising from them have to be solved by geologists and engineers, working in conjunction, in a relatively short space of time.

The estimation of stability of slopes is thus an important consideration in natural slopes and in the design of man-made slopes. The failure of slopes in hilly areas is attributed to a number of factors, mainly depending on the slope angle, the soil water content, the type of earth material involved and geological structure and agricultural practice and local environmental factors, such as rainfall, earthquakes, ground temperature, etc. In most instances the major factors causing landslides are geological. Although the immediate cause which triggers off a landslide in SriLanka is generally heavy and continuous rainfall. In Sri Lanka rain induced landslides are the most common ones.

The areas in SriLanka susceptible to landslides are located in the central hilly regions, where the soils belong to the "Residual Soil" category. Residual soils are considered problematic due to their heterogeneous and anisotropic nature. Some slopes expected to be stable (F.O.S >1) fails and some others expected to be unstable (F.O.S<1) be stable for a long time. These field experience shows that the shear strength parameters obtained from the laboratory or field tests does not satisfactorily represent the shear strength parameters developed in the field. So it is necessary to find some alternative means to reflect the field mobilized shear strength parameters. Back analysis technique has been used for this purpose.

Residual Soils

There is no universally accepted definition for residual soils. The residual soils are products of weathering igneous, sedimentary and metamorphic rocks, the degree of weathering and the extent to which the original structure of the rock mass is destroyed varying with depth from the ground surface. Residual soils found to some degree in most countries of the world. Any rock sufficiently decomposed so that the principles of soil mechanics are applicable is referred to as residual soils; transported residual materials like colluvium which usually consist of gravels, cobbles and boulders in a matrix of soil are also included in the residual soil category.

In Sri Lanka over 90% of the land is made up of highly crystalline rocks belonging to the south Indian shield; residual soils formed by weathering of these rocks are found almost everywhere. The problems associated with residual soils are mainly due to their inhomogenity, anisotropy and, non-availability of good models or methods to describe or to understand their behaviour. The difficulties of sampling and testing of residual soils, is also a major problem and thus has rendered minimum research in this area.

Back analysis

Stability analyses of slopes are normally carried out to find the factor of safety using known shear strength parameters, but on the other hand, by analyzing the actual failure, the field mobilized shear strength can be determined. This technique is called as back analysis. The back analysis became important because some slopes expected to be stable fails and some others expected to be unstable be stable for a long time. To achieve the back analysis, it is necessary to perform an analysis in reverse from a known factor of safety, taken to be 1 at the instant of failure. This procedure is commonly termed back analysis (Chandler 1997; Singh et al. 1979).

To initiate a backward analysis of failure, factors such as environmental conditions prevalent at the instant of failure (rainfall, wind effects, etc.), the actual causative and triggering factors of the particular failure or land slide, and the actual failure mechanism (progressive or retrogressive or single block failure) should be studied and interpreted carefully.

Shear strength correlation factor.

The difference in values of field and laboratory shear strength parameters of residual soils forming the SriLankan hill slopes is attributed to their complex nature. The actual field shear strength parameters are assessed by employing the technique of back analysis, and can thereupon be correlated with the laboratory shear strength parameters by a factor termed the strength correlation factor. Therefore, for a given laboratory testing method, stress path, and method of analysis, the strength correlation factor can be used to predict the field shear strength, thus improving design and construction in residual soil slopes.

When back analysis is performed for a known slip surface for which the factor of safety is unity, the shear strength along the slip surface at failure can be determined. For "c" and "ϕ" soils, this shear strength can be found for different combination of the soil strength parameters, "c" and "ϕ". A sound engineering judgment is needed to select the appropriate and more suitable soil strength parameters out of the varies combination of c and ϕ, which contribute to the same failure strength.

SLOPE/W Software

SLOPE/W is a software that uses limit equilibrium theory to compute the factor of safety of earth and rock slopes. The comprehensive formulation of SLOPE/W makes it possible to analyze both simple and complex slope stability problems using a variety of methods to calculate the factor of safety. SLOPE/W has many applications in the analysis and design for geotechnical, civil and mining engineering projects.

SLPOE/W allows stability analysis using the following methods: Ordinary (or Fellenius) method), Bishop simplified method, Janbu simplified method, Spencer method, Morgenstern-Price method, Generalized Limit Equilibrium (GLE) method, and Finite Element Stress method. Furthermore a variety of interslice side force functions can be used with the more mathematically rigorous Morgenstern-Price and GLE methods. The availability of many analytical methods in SLOPE/W provides to select best suited method for the problem.

AIM AND OBJECTIVES

To determine the strength correlation factors for some residual soil of SriLanka on comparison with laboratory strength parameters obtained from triaxial tests and those obtained using back analysis, for the analysis of landslides.

SCOPE OF STUDY

1. Find the stability factor (Factor of Safety) for some selected slopes before the failure for existing slip surface.
2. Find the critical slip surface and compare with existing one.
3. Verify the actual failure mechanism (Progressive or Retrogressive) using existing failure surface.
4. Find the strength parameters developed (expected) in the field using back analysis technique.
5. Correlate the field shear strength parameters obtained from back analysis with those obtained from laboratory tests.

METHODOLOGY

1. Data collection

The data for some post landslides were collected from National Building and Research Organization. The following details were collected for each slide,

- Subsoil profile.
- Shear strength (laboratory and field) parameters for each layer.
- Bulk density of each layer.
- Existing (actual) failure surface.

2. Soil profile was plotted and analyzed for different conditions using SLOPE/W software. The stability analysis was done for the following conditions.

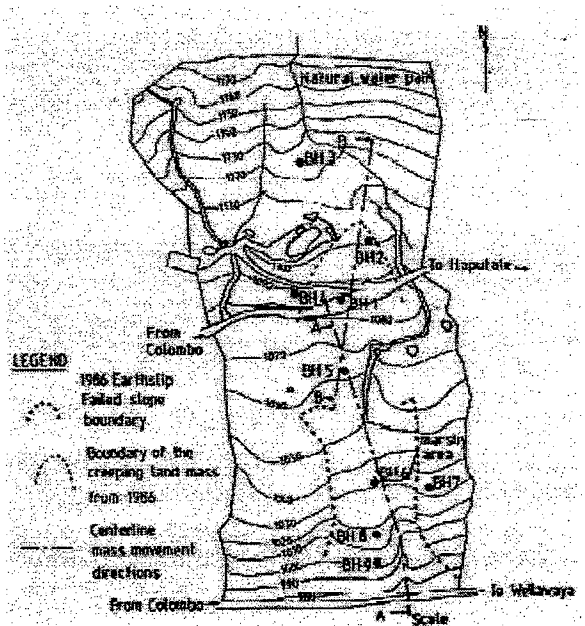
- Stability for normal ground water level for different slip surfaces.
- Stability for higher ground water level for different slip surfaces including the actual failure surface for progressive failure.
- Stability analysis for retrogressive failure.

3. The mechanism of failure (progressive or retrogressive) was found using previous analysis and the critical failure surface identified and then the factor of safety along that surface was obtained.
4. Using different sets of modification factors for cohesion and angle of friction, the different combinations were obtained for unit factor of safety along critical failure surface.
5. Among these combinations, suitable set was selected as the strength correlation factors using the concept that when the shear strength obtained using these factors is reapplied to the soil profile should give the minimum factor of safety along the critical surface and the value should be 1.0 also.

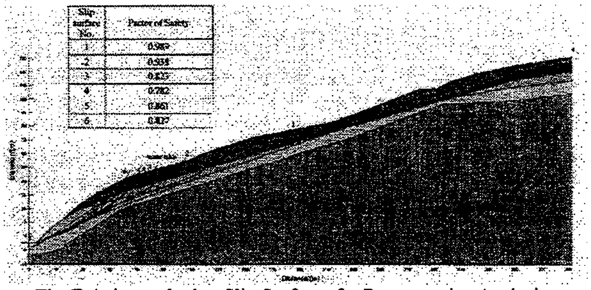
CASE HISTORIES

1. BERAGALLA LANDSLIDE

In 1986, following heavy rainfall, a major landslide occurred in the slope above highway A4 at 168 km along the Colombo-Wellawaya highway, passing the southern mountain slope of the Idalgashinna-Haputale ridge, in the central highlands of Srilanka, which rises to an elevation of 1788m above MSL. The slide deposited debris on the highway on the slope below. Furthermore, the upper slope of the hill has since been creeping continuously, badly affecting the Beragala-Hali-Ela (A16) highway, passing the same slope, but at higher elevation.



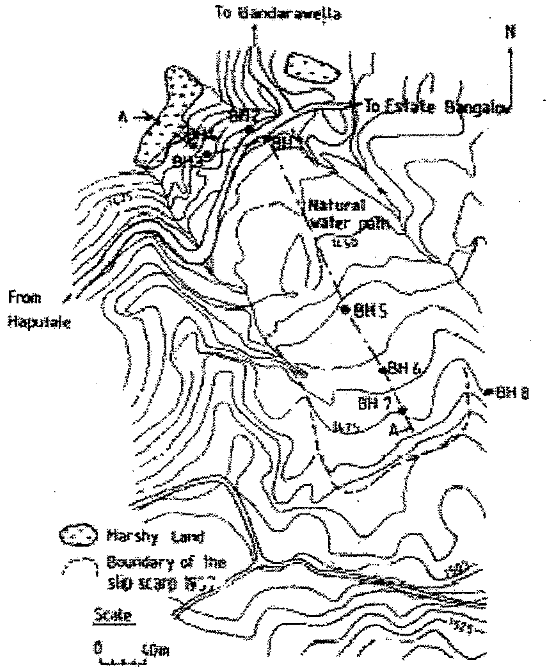
Beragala Earthslip, Postslide Topography (1988)



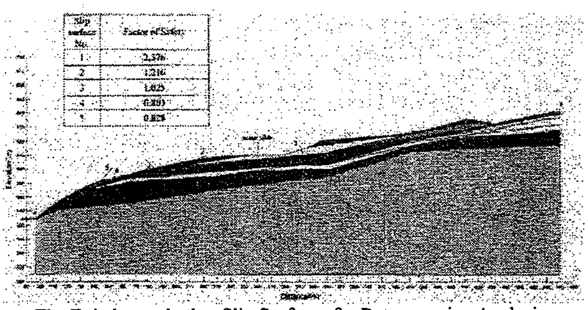
The Existing and other Slip Surfaces for Retrogressive Analysis

2. KEHAGALLA LANDSLIDE

The Kehagalla earthslip is located between culvert numbers 11/3 and 11/6 along Beragala-Hali-Ela highway (A16), in the central hills of SriLanka at an elevation of 1430m above MSL. The slide is in western slope of the mountain range running northeast from Haputale. The aerial extent affected by the slide is around 4.5ha. The material within the slide area consists of colluvium deposits and in-situ weathered residual materials. The Kehagalla earthslip occurred in 1957. The soil mass had moved at the toe. Subsidence at the scarp had left a 3m deep soil scarp. Therefore, to identify the slip surface and to construct a sub-surface profile, a detailed geotechnical investigation was carried out.



Kehagalla Earthslip, Postslide Topography (1988)

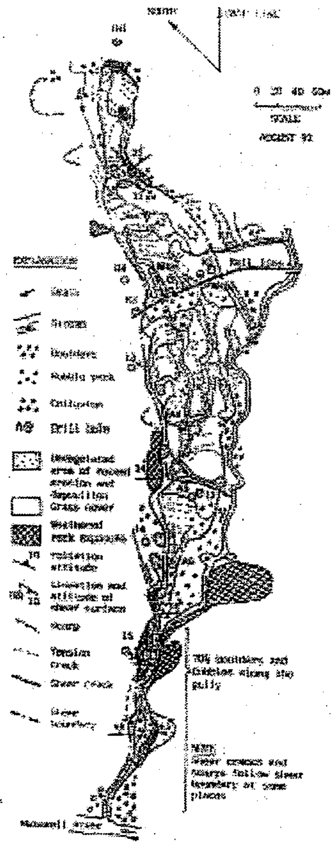


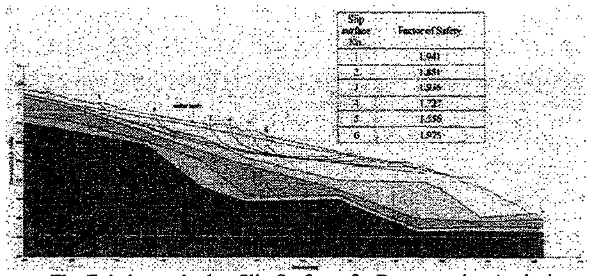
The Existing and other Slip Surfaces for Retrogressive Analysis

3. WATAWALA EARTHSLIP

The Watawala area is located in the western part of the central highlands of Sri Lanka. It occurs within the wet zone of Sri Lanka and in fact, experiences the highest rainfall in the island. Rocks of the Watawala area belong to the highland group.

The upcountry railway connecting Colombo and Badulla via Peradeniya runs across the Watawala slide. In addition it provides services of transporting thousands of passengers up and down, this rail track still remains the principal mode of transporting bulk quantities of fuel and food stuffs to the up country, and upcountry products to Colombo.


Watawala Earthslip (1993)



The Existing and other Slip Surfaces for Retrogressive Analysis

RESULTS AND DISCUSSION

Three past landslides were analyzed using slope/w software to find the field shear strength. Initially, the actual mechanism of failure was analyzed to perform back analysis. It was found that retrogressive mode of failure occurred in residual soils at Beragala, Kehagalla, & Watawala earth slips.

The first slip in the retrogressive mode of failure was identified and used for back analysis. Stability analysis was performed for various combinations of cohesion and angle of friction (the laboratory soil strength parameters multiplied by correlation factors) to get field strength to provide corresponding factors of safety. The factors of safety were plotted against cohesion correlation factors for various values of friction correlation factors. The correlation factor corresponding to a unit factor of safety was then determined from the plot. It was observed that the correlation factors k_c and k_Φ give a linear relationship. Therefore final correlation factors were chosen doing further analysis and by using engineering judgment.

For Beragala, & Kehagalla according to Loganathan et.al, (1992), the engineering judgment was used that the shear strength parameters were found for a particular sample under similar testing condition. Therefore, the influence of sample disturbances, assumptions made in modeling and other phenomena equally affect both the cohesion (intercept) and the friction angle, as a result of which it can be assumed that $k_c = k_{\Phi}$. But these values were checked using newly developed technique. But for Watawala, the combinations of c' & Φ' obtained from back analysis of the actual slip surface were used to calculate a set of safety factor for the slope without the slip plane present; that is, for the intact slope. And k_c & k_{Φ} were obtained.

In Beragala & Kehagalla, the strength correlation factor obtained for residual soil slope is greater than unity. This shows that the field shear strength of residual soil is greater than the laboratory shear strength value of the soil mass. On the other hand, in Watawala the correlation factor obtained for residual soil slope is less than unity. This shows that the field shear strength of residual soil is less than laboratory shear strength value. As the cohesion correlation factor is very small (0.21), the developed cohesion can be taken as zero. So the resistance is only developed by friction (Φ'). Moreover the literature (NBRO, 1993) specify that after failure the direct shear test done on the surface sample produced c'=0 and Φ'=16. This proves the reliability of back analysis.

CONCLUSION AND RECOMMENDATIONS

Detailed data collection was performed on three landslides, which were already occurred. Subsurface soil profiles were constructed and analyzed using slope\W software. Stability analysis of the slope was performed using the modified Janbu method of analysis, which is capable of analyzing irregular slip surfaces.

To perform back analysis the actual failure mechanism was studied. The study showed that a retrogressive mode of failure occurs on residual soil slopes.

It was proved that the effective field shear strength parameters of those three landslide areas can be obtained by multiplying the laboratory shear strength parameter c_{lab} and ϕ_{lap} by some factors called correlation factors found from back analysis.

Final results of correlation factors for three landslide areas.

Landslide area	From analysis and calculations		From literature review	
	k_c	k_ϕ	k_c	k_ϕ
Beragala	1.240	1.240	1.205	1.205
Kehagalla	1.225	1.225	1.273	1.273
Watawala	0.210	0.710	-	-

SUGGESTIONS FOR FUTURE WORK

- Consider the rainfall data and use correct (actual) water table at the instant of failure.
- Consider the seepage forces.
- Use the programme for back analysis with auto searching of minimum factor of safety slip surface.

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USE OF GEOGRIDS IN IMPROVING SOIL REPLACEMENT METHOD

Jayasekara V.P.I., Nabadawewa M.C.B., Samarasingha S.V.P.A., Kurukulasooriya L.C.
Department of Civil Engineering, University of Peradeniya.

ABSTRACT

The objective is to obtain the advantage of use of geogrid to reduce the soil replacement, where the soil has low bearing capacity. A study was undertaken to investigate the bearing capacity of a strip footing on geogrid reinforced sand by performing laboratory model tests. The stress strain behavior of sand was investigated with various depths of sand layer for both with the geogrid reinforcement and without the geogrid reinforcement.

INTRODUCTION

Reinforced soil or mechanically stabilized soil is a construction technique that consists of soil that has been strengthened by tensile elements such as metal strips, geotextiles or geogrids. In 1960's the French road research laboratory conducted extensive research to evaluate the beneficial effects of using reinforced soil as a construction technique. During last forty years many civil engineering constructions such as retaining walls and embankments were done all over the world using reinforced soil and they have performed very well.

The construction of a reinforced soil foundation to support a shallow footing has considerable potential as a cost effective alternative method. In this fill material are placed beneath the footing to create a composite material with improved performance characteristics. One or more layers of a geosynthetic reinforcement and compacted fill material are placed beneath the footing to create a composite material with improved performance characteristics.

The metallic strips that are used for reinforced soil are usually galvanized steel strips. However the galvanized steel strips are subject to corrosion, hence depending on the projected service life of a given structure allowances must be made for the rate of corrosion.

Geotextiles and geogrids are the best alternative solution for the above problem, as they are non biodegradable materials. They are made from petroleum by-products such as polyester, polyethylene and polypropylene.

What is geogrid?

Geogrids are made by tensile drawing of polymer materials such as polyethylene and polypropylene. They are relatively stiff material compared to geotextiles. They have large apertures which allow interlocking with surrounding soil to perform the function of reinforcement. There are two types of geogrids; they are biaxial geogrids and uniaxial geogrids.

OBJECTIVES

1. To determine how the depth of excavation could be reduced with the use of geogrid reinforcement.
2. To determine optimum depth of sand replacement with the use of geogrid reinforcement.

METHODOLOGY

Experimental Setup

Model: A Series of model loading tests were conducted inside a cubical steel tank of 400 mm by 86 mm in plan and 300 mm in depth.

Model of foundation: A model of strip foundation 85 mm x 25 mm x 12 mm was made using mild steel. Hemi-spherical hole was made at the center of foundation to apply the load through a steel ball.

Plate to consolidate soil: A perforated mild steel plate was made to consolidate the clay slurry in the model.

Geogrid reinforcement: A geogrid was prepared by cuting a geo membrane (Fig.01).

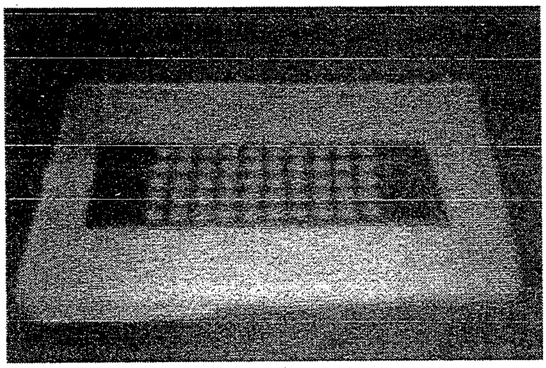


Fig. 01: Geogrid reinforcement cut from a geomembrane

Physical Properties

Tests on the soft clay (Table 1).

1. Atterberg Limits Tests.
2. Mechanical Analysis.

Table 1: Characteristics of soft clay

Liquid limit / (%)	50
Plastic limit / (%)	31
Plasticity Index / (%)	19
Type of soil	Silty sand

Tests on sand used as fill

- Direct Shear Test. (Table 2)

Table 2: Characteristics of sand

Dry density of the fill /(kN/m^3)	16
Angle of friction / (°)	30.9

Tests on geogrid reinforcement

- Tensile Test. (Table 3)

Table 3: Characteristics of geogrid reinforcement

Tensile Strength /(kN/m^2)	13.9
Thickness / (mm)	2.3
Aperture size /(mm)	10

Experimental Procedure

Consolidation of soil

- Preparation of soft clay layer.

Soil was sieved through a 2mm sieve to remove large particles, to eliminate the scale effect.

- Preparation of the clay slurry.

10 kg of soil was weighed and the moisture content of soil was found. De-aired water was added so that the initial moisture content of the slurry was kept at two times the liquid limit. The slurry was then transferred into the model box ensuring that no air entered during the process.

- Consolidation of soil

The clay slurry was consolidated up to a maximum pressure of 50 kPa. The Fig.02 shows the consolidation of soft clay.

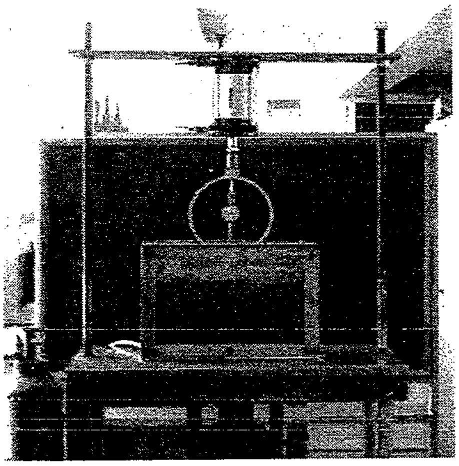


Fig. 02: Consolidation of soft clay

Test on soft clay to simulate existing ground.

The model of foundation was placed at the center of top surface. A dial gauge was attached to the model to monitor the settlement of foundation. A proving ring was attached to measure the axial load.

The axial load was applied by applying a rate of displacement of 0.1 mm/min. Dial gauge readings and proving ring readings were recorded simultaneously.

Test on sand only to simulate maximum soil replacement.

Initially sand was compacted to a uniform density of 17 kN/m^3. Then foundation model was placed on the top surface and same experiment was repeated described above

Tests on a layer of sand on soft clay layer to simulate soil replacement.

Two tests were carried out as by varying the thickness of sand. Thicknesses of sand layer were 2 times and 3 times of width of the strip foundation respectively for the two tests. Fig. 03 shows the experimental setup of the test.

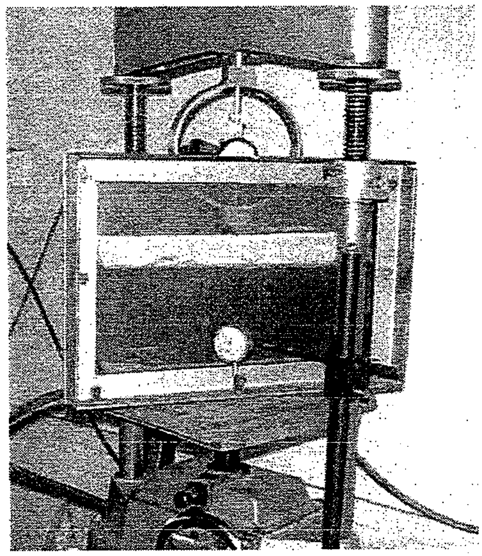


Fig. 03: Test on sand and clay

Tests on sand reinforced with a geogrid on the soft clay layer to simulate reinforced sand fill.

Two tests were carried as described above. Thickness of sand layers were 2 times and 3 times of width of the strip foundation. A geogrid was placed 1 cm above the sand –clay interface for both cases as shown in fig.04.

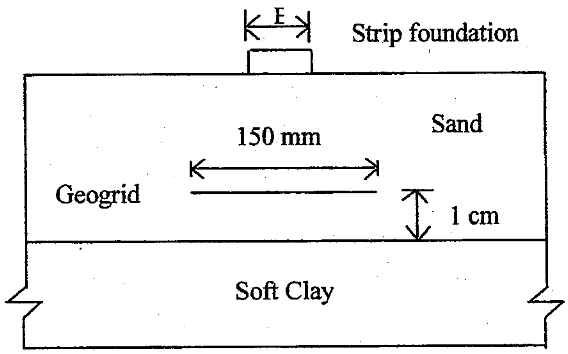
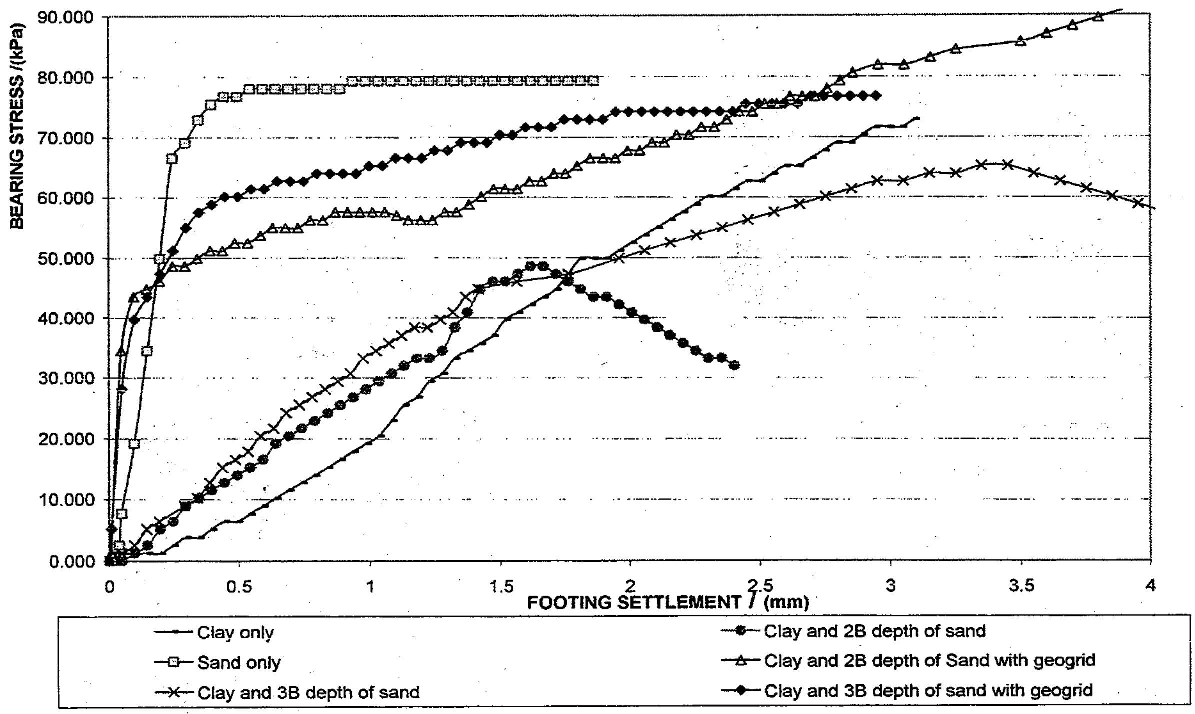


Fig. 04: Schemetic diagram of testing model

RESULTS

GRAPH NO. 1: BEARING STRESS Vs FOOTING SETTLEMENT



DISCUSSION

The ultimate bearing capacity can be taken either peak value of bearing stress or that corresponds to 1.5 times foundation width of settlement.

According to the ultimate stress value at 1.5 times foundation width of settlement, it gives clear indication of advantage of use geogrids to reduce the soil replacement.

CONCLUSION

According to the bearing stress- settlement variation (graph no 1), geogrid with 2B depth of sand gives large bearing capacity higher than that for an unreinforced sand layer of thickness 3B.

According to the tests carried out, it is clear that, with the use of geogrids, amount of depth of sand replacement can be largely reduced

Suggestions for future work

- By changing the depth of sand layer with geogrid reinforcement, and doing the same experiment is repeated.
- Same test can be extended by changing the number of geogrids within the fill.

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