GEOTECHNICAL ENGINEERING PROJECT DAY 2002

A Presentation of Best Geotechnical Engineering Undergraduate Projects in Sri Lankan Universities

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CONTENTS

and it's a major advantage for the construction industry.

3.0 METHODOLOGY

The data were obtained from the boreholes done in Southern Colombo.

The area covered Moratuwa, Rathmalana & Mt. Lavinia.
The logs usually contain details about;

1. Project Information
   Title, client, consultant, cost etc.

2. Exploration Hole
   Hole name/no., geographic location, ground level

3. Strata Descriptions
   Geological material encountered during excavation including depths, thicknesses etc.

4. Ground water level
   This is sufficiently accurate for the preliminary evaluation of type of foundation and planning of dewatering

5. Insitu Test Data
   Primarily concerned with SPT data and permeability test data. Others may include vane shear & CPT data.

3.1 SOIL TYPES

By analyzing about 100 borehole logs, first, various soil types were identified. The log stated only the description of soil types encountered during drilling. These were classified to six major types; alluvial soils, residual soils, organic soils, dense sands, decomposed rocks and earth fills.

Alluvial Soils

These are a variety of soils found in the river valleys and flood plains deposited during the flood season.

Residual Soils

The laterites belong to the class of soils termed as residual soils, which lie above parent rock as a product of the insitu weathering of the rock. Commonly occurs in the hot-wet climates of Sri Lanka, with Colombo being one of them.

Decomposed Rock

These materials are derived from the chemical decomposition of the parent bedrock, and their character depends on the parent rock type. The depth over which decomposed material changes to fresh rock is extremely variable.

Earth Fills

Fill can consist of replaced natural ground, or waste materials of various origins. The uniformity of fill will depend on the degree of control, which has been imposed on the quality of incoming material, placing and compaction.

The soils were classified with a major and a minor soil type. The minor soil type accounts for the various differentiations found in soils within each category given above.

3.2 COORDINATES

The locations of boreholes were shown in the area map. The coordinates of the borehole points were found with relative to a known fixed point. With this known point, the conversion of local coordinates into global coordinates can also be done if required.

3.3 DESIGN OF DATABASE

A database allows quick and efficient retrieval of data. A modern database management system can instantly locate a record in a table with millions of records.
Relational databases are based on relationships among the data they contain. Data are stored in tables, and tables contain related data.
This database has 8 tables. One has the project details and others contain laboratory & insitu test data. SQL is used to formulate relational operations.

- Table 1- Project Details
- Table 2- Borehole Data
- Table 3- Layer Details
- Table 4- SPT 'N' values
- Table 5- Layer ID
- Table 6- Atterberg Test Data
- Table 7- Grain Size Distribution Results
- Table 8- CPT Test Data

The database is completed with entering of all the data of the project. Usually, databases handle a lot of data. Manipulating data in the table format can be tiresome. To eliminate this problem, a user-interface was designed and this enables the user to work in a more pleasing mode. It contains many user-friendly features to minimize the errors that can occur while handling a large amount of data.

4.0 RESULTS

The possibility of assembling the Geotechnical information, which was maintained in paper files so far, into a digital database, has been looked into.
Since there's room for further addition of records, the database can be extended upto various other projects.

The main advantage of this is that the data can be easily queried according to the requirement of the design engineer. Some simple queries are as given below.

- Soil types having the major soil type as Alluvial
- Variation of SPT value of a given soil type (fig. 2)

Query Soil Type : Table

Soil Type	Bore Hole Id	Depth	SPT Values
ALS1	002	2.40	12
ALS1	003	2.70	12
ALS1	004	2.85	11
ALS1	005	2.60	13
ALS1	006	2.45	16
ALS1	007	2.55	13
ALS1	008	2.65	14
		0.00	0

Record: 1 of 7

Fig. 2- Table of queried data

5.0 CONCLUDING REMARKS

It should be noted that the geology of a site is more than that inferred from aerial photography, surface outcrops and sub surface information at the positions of the exploration points. The possibility remains that significant undetected variations or discontinuities can exist.

E.g: Weak compressible soils overlain by a stronger layer; these contribute significantly to the settlement of proposed works.

But these can be identified through a sub surface soil profile. A database with sub surface data is of prime importance here, because the database can be combined with a GIS system to obtain visualizations. Practitioners regard the total GIS as including the data that go into the system. Therefore the DATA plays a vital role in the system.

Also the database can be extended upto an Automated Geotechnical Information and Design System (fig. 1) and this will allow Geotechnical and structural engineers to quickly and economically obtain information and evaluate design alternatives.
The databases can be connected by a user interface application and can be used for performing cross queries, correlations and engineering analyses.

This will allow user to:

- Obtain cost comparisons quickly
- Compare technical sufficiency and economic data on alternative designs
- Locate existing site investigation data and historical load test data rapidly
- Compare various construction methods
- Perform correlations between various geotechnical parameters
- Download data and perform comparative analyses for multiple design alternatives

6.0 ACKNOWLEDGEMENT

A very deep thank to Dr. Ashok Peiris, Senior Lecturer, Department of Civil Engineering, for providing the necessary guidance throughout the project and to Dr. Niranjan Gunawardena, for the kind Corporation in providing us the data for the project.

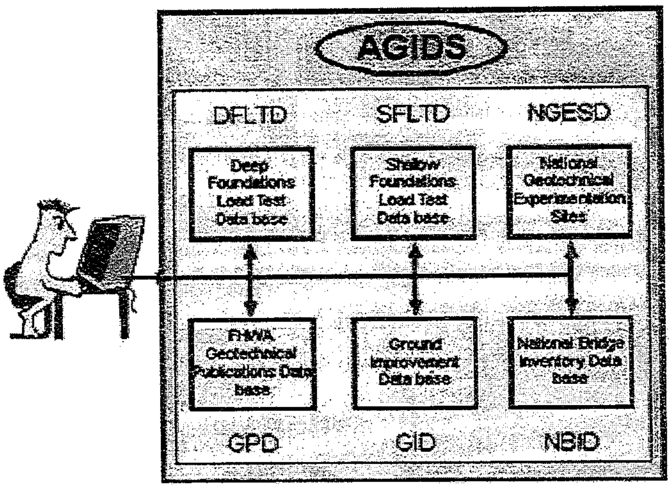

Fig.2- Automated Geotechnical Information and Design System

7.0 REFERENCES

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USE OF LIGHTWEIGHT FILL IN CONSTRUCTION ON SOFT GROUND

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Abstract

The primarily objective of this research was to develop a lightweight fill material to be used in work done on soft clays so that the settlements could be reduced and possible shear failure minimized. The light weight fill material was made by mixing tyre chips with conventional lateritic fill. Different fill proportions were used and the engineering properties of the fill were determined. A construction process was proposed and the problem was modeled using the CRISP finite element package. Advantages of the proposed construction process was highlighted.

1. BACKGROUNDS AND OBJECTIVE

1.1 Difficulties in construction on soft ground

Due to the unavailability of land underlain by competent subsoil conditions, civil engineers are compelled to use land underlain by soft/loose subsoil conditions in the new development projects. Over the years Geotechnical Engineers have experienced many problems due to the construction on soft ground. Two most significant problems associated with the constructions on soft soils are;

1. Excessive settlements and
2. Shear failure

There are large areas of unused land underlain by soft peaty clays in Colombo and its suburbs. Very high primary and secondary settlements and very low shear strength of soils are the major problems associated with construction on such soft ground. Settlement problems are associated with the compressible nature and stability problems are associated with the low shear strength. Therefore, it is very important to develop cost effective methods for handling such conditions.

1.2 Possible solutions

If multistoreyed buildings are to be constructed in areas underlain by soft ground the large structural loads will have to be transferred to an underlying dense layer or rock through a system of piled foundations. However, it would not be economical to transfer the moderate loads imposed by services such as water supply lines and sewerage lines also to an underlying hard stratum through piles. Furthermore, it would not be economical to construct new infrastructure facilities with large plan area such as roads, on piled foundations.

As such, improvement of the engineering properties of soft peaty clay would be a more economical approach in such situations. Pre-consolidation with preloading, vacuum consolidation, dynamic compaction and deep mixing with cement lime are several possible methods for improvement of engineering properties of peaty clays. Numbers of research projects were carried out in recent years at University of Moratuwa to study the level of improvement achievable by above techniques.

Another possible approach to solve this problem is to reduce the load applied on the soft soil. As a result of using a lightweight fill, the load on the soft soil and its consolidation settlement can be reduced and the shear failure may be minimised. There are records of the use of lightweight fill such as; polystyrene blocks, expanded clay etc...(Riordon and J W Seaman) in the construction of embankment done on soft soils in several countries. However, it would be too costly to import such materials to a developing country like SriLanka and it is important to find solutions with local materials.

1.3 Possible local solutions

An attempt was made here to develop an alternate cost effective lightweight fill material by mixing soil with tyre chips obtained by discarded motor vehicle tyres. Tyre chips are non- biodegradable, available in abundance, and are inexpensive. Therefore, the use of tyre chips becomes an attractive option for the development of a lightweight fill material.

The data obtained in this research program indicate that combination of tyre chips and lateritic fill has the potential to be used as a lightweight fill material in the construction of water supply lines and sewage lines or in highway embankments over weak or compressible soils.

2. DEVELOPMENT OF THE LIGHTWEIGHT FILL MATERIAL

Large numbers of used tyres are added annually to the existing stockpiles. Current disposal and stacking methods of waste tyres are not acceptable due to the possibility of fire and health hazards. Therefore discarded tyres should be used for other applications. One possible application is to use shredded tyres to make a lightweight fill material to be used in the construction of embankments over weak or compressible soils.

Lateritic gravelly soils are the most widely used fill material in SriLanka. It contains gravel size particles and also clay sized particles. Material is generally gap graded with a lower percentage of sand size particles. Lateritic fill material is widely available in the Country and is less expensive than other granular fill material such as sand or quarry dust. Therefore it is proposed to mix tyre chips with the lateritic fill material to develop the lightweight fill material. Tyre chips are of loose density around 320kg/m^3 and density of the order of 1800kg/m^3 could be achieved with lateritic fill.

It is necessary to decide on a suitable mix proportion so that the developed fill material will have the desired engineering properties and is also economical. Developed fill should be of sufficient low density, but higher shear strength, and low compressibility. Tyre chips were mixed with lateritic fill in different proportions to get a workable mix of sufficiently low density. After several trials two mix proportions 1:2 (tyre chips: soil) and 1:3 (tyre chips: soil) by weight were used for further testing. Samples with these two mix of proportions were of sufficiently low density and were of reasonable workability.

3. BASIC PROPERTIES OF THE LIGHT WEIGHT FILL

3.1 Particles size distribution

Particle size distribution of the two mixes were analysed using the standard sieve analysis. The results are presented in Figure 1 for the two-mix proportions.

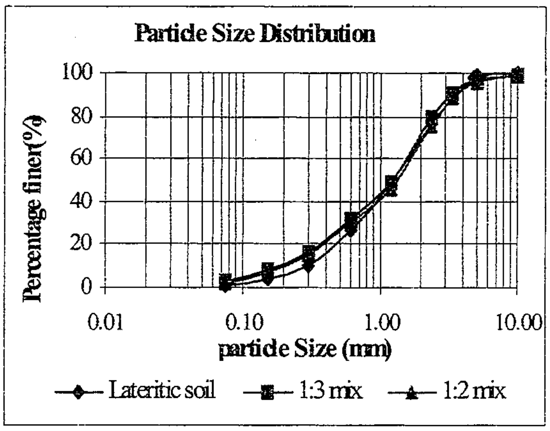

Fig 1 Particles size distribution

3.2 Density analysis

The standard proctor test was done to determine the maximum dry density and the optimum moisture content of the two proposed mixes. The dry unit weight obtained for each trial is plotted against the moisture content. The result of the series of standard proctor compaction tests on lateritic soil with varies percentages of tyre chips are presented in Table 1.

Description of mix	Optimum moisture Content (%)	Maximum dry density (kg/m³)
Lateritic soil	17.55	1776
1:3 mix	17.40	1405
1:2 mix	17.00	1288
1:1 mix	14.20	1173

Table 1-Proctor test results

It is clear from the data of table that the optimum moisture content decrease as the percentage of tyre chips in soil increases. This data also reveals that the dry density of the mix decreases as the percentage of tyre chips increases.

3.3 Compressible characteristic of fill material

In order to determine the compressibility characteristic of the developed lightweight fill a series of consolidation tests were conducted.

Consolidation characteristic of mixes were determined in the laboratory in an oedometer. Properties such as coefficient of consolidation (C_V), coefficient of volume compressibility (m_V), and compression Index (C_c) were determined through that test data.

After saturating the lightweight fill material in the oedometer the consolidation test was done in stages. The e-log σ curves for the developed lightweight fill materials were obtained and the compression index Cc values were computed.

Compression index for tyre chips: soil mixes are;

For lateritic soil	$C_c = 0.084$
For 1:3 mixture	$C_c = 0.148$
For 1:2 mixture	$C_c = 0.151$

Above results indicate that C_c values increased with increased percentage of tyre chips. But the C_c values are still quite low. Study of the shape of the settlement vs. time plot indicated that the major part of the settlement is due to initial elastic compression.

3.4 Coefficient of volume compressibility

Coefficient of volume compressibility is an alternate parameter for estimating the settlement. The m_v values of the mixtures were higher than those for the lateritic soil at lower stress levels but were approximately of the same order at higher stress levels.

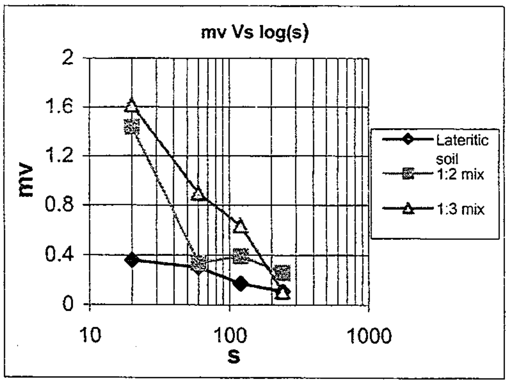

Fig 3 Effect of tyre chip mixing on Mv for lateritic soil

3.5 Shear strength characteristic of fill material

The shear strength characteristics of the lightweight fill for different mixes were measured using the triaxial apparatus. Specimens of 38mm diameter and 85mm height were tested under unconsolidated undrained conditions.

The Mohr-Coulomb failure envelopes for mixes 1:3, 1:2 were obtained. The internal friction angle (Φu) and the cohesion intercept (C_u) values obtained are presented in Table 2.

Soiltype	Cu (kN/m²)	Φu
Lateritic Soil	163	0
1:3 mix Soil	45	20
1:2 mix Soil	40	22

Table 2-Shear strength properties of fill

These results indicate that the lightweight fill obtained by mixing lateritic fill and tyre chips are quite incompressible and have reasonably high shear strength parameters. As such, it is possible to use them in the proposed construction.

4. NUMERICAL SIMULATION OF PROPOSED PROCESS

4.1 Introduction to CRISP

CRISP is a finite element program, which is able to perform, drained, undrained, and time dependent analysis of static problems under monotonic loading/unloading condition.

CRISP needs input data on; information describing the finite element mesh, material properties and insitu stresses for each finite element, and boundary conditions and loading sequences for the analysis.

Other important facility is the CRISP-LOTUS 123 interface program and different modes of programme. CRISP also is able to gives graphical representation of soil behaviour under different conditions.

4.2 Proposed construction process and its Simulation

Lightweight fill material can be used to construct embankments on soft clays, completely replacing the conventional fill. This will improve the factor of safety against shear failure in the soft clays and will decrease the settlements. Lightweight fill also can be used behind the retaining walls to reduce the earth pressure and in the repair of landslides.

However, the problem in replacing the conventional fill completely by the light weight fill is that large quantities of the material will be needed. If the lightweight fill can be produced in large quantities economically then the complete filling would be plausible. This would be ideal in the construction of road embankments on soft soils.

However, in this project a construction procedure was proposed for the laying of service lines over the fill done on soft clays. Here the proposal is to initially fill the sitê with normal material and then allow the peat to consolidate to some level. Subsequently some amount of fill will be removed along the proposal alignment of the service line and will be refilled with the lightweight fill.

Using CRISP each stage of the proposed process is simulated. The stages are;

(1) Insitu stage
(2) Filling stage
(3) Consolidation stage
(4) Excavation stage
(5)Consolidation/ Swelling stage
(6) Refilling stage
(7) Consolidation stage

The construction stages are illustrated in figure 4 and the finite element mesh used for the analysis is shown in Figure 5.

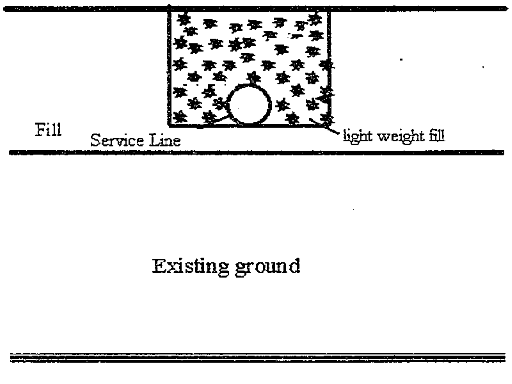

Fig 4(a) Proposed construction process

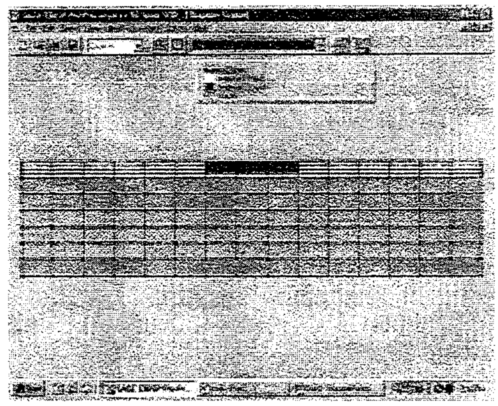

Fig 5 Finite element mesh used

4.3Consolidation Analysis

CRISP program can be used to perform a coupled consolidation analysis effectively. Eight nodded isoparametric elements are used for the consolidation analysis. The element used has excess pore pressure as an additional variable. For nodes at the bottom and at the top of the mesh, pore pressure boundary conditions are to be defined. In this analysis soil is assumed to drain only from the top surface.

In consolidation analysis numbers of increments are selected so that the results converge fairly accurately without any warning messages and errors.

4.4 Material data

Fill material

Fill and refill material were treated as isotropic elastic soil with undrained condition. As the SriLankan lateritic soil and the proposed tyre soil mix are of sufficiently high shear strength and low compressibility the use of an elastic soil model could be justified.

For an isotropic elastic material following parameters are needed by the program.

- Young modulus
- Poison ratio
- Unit weight

Parameters for the lateitic fill and the proposed lightweight fill are found from the laboratory tests conducted on them.

Insitu material

The insitu soft clay layer is modelled by the modified cam clay, which is a well-suited constitutive model for a coupled consolidation analysis. Cam clay model parameters needed were derived using the available laboratory test data on SriLankan peaty clay (Munasinghe 2002). Following cam clay parameters are needed by the CRISP program.

(1) Slope of N.C line
(2) Slope of C.S line
(3) Critical state void ratio
(4) Poison ratio
(5) Shear modulus
(6) Unit weight of material
(7) Permeability

These values were derived based on the lab test data, following the guidance given in the CRISP manual.

5. RESULTS OF THE FINITE ELEMENT ANALYSIS

To analyse the feasibility of the proposed construction process the soft peaty clay was assumed to have C_V values of; $4m^2/yr$, $8m^2/yr$, $16m^2/yr$, $32m^2/yr$. Laboratory determined C_V values were in the range from 4 to 8 m^2/yr. In the field C_V values the order of 10 to $16m^2/yr$ were found. This was the basis for the selection of above values.

Period of consolidation under the initial conventional fill was also varied. Figure 6 presents the settlement behaviour if the soft peaty clay was allowed to consolidate for a period of 1 year under the normal fill. Excavation was done for the service line after this period. After the service lines were laid backfilling was done with the light weight fill.

It could be seen that if the soft peaty clay had a C_V value of $16m^2/yr$, settlements after the placement of service lines are less than 50mm. If the C_V value of peat was $4m^2/yr$ this settlement would be around 200mm.This is due to the greater percentage of consolidation that had taken place when the soil had a higher C_V value. Similar analysis was done for the initial consolidation periods of 6 months and 2 year.

The ultimate settlement of the service lines under these different consolidation periods, the for the different C_V values of the peaty clay are presented in Table 3.

As expected it illustrates that if the coefficient of consolidation of the peaty clay is quite high the in service settlements of the service lines could be considerably reduced by the proposed procedure.

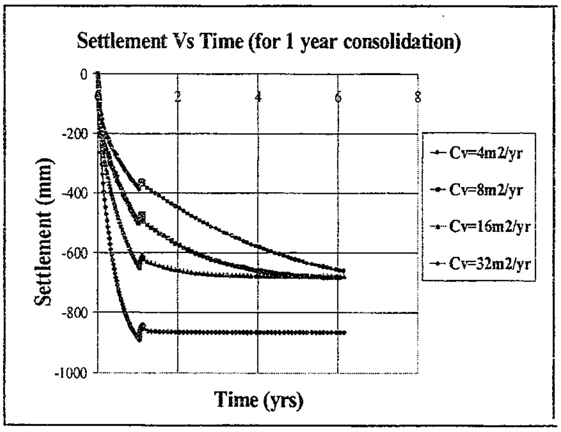

Fig 6 Settlement Vs Time

The effectiveness of the proposed construction procedure was further illustrated by plots the settlement vs time behaviour; with conventional fill and with the proposed process in the same graph.

Figure 7 presents the comparison for the case of $C_V = 8m^2/yr$, where the initial consolidation period under the normal fill was 1 yr. It could be seen that settlement reduction of the order of 100mm in the service lines could be achieved by the proposed.

Further reductions in settlements could be achieved if the depth of refill is increased or if the density of the lightweight fill material can be further reduced.

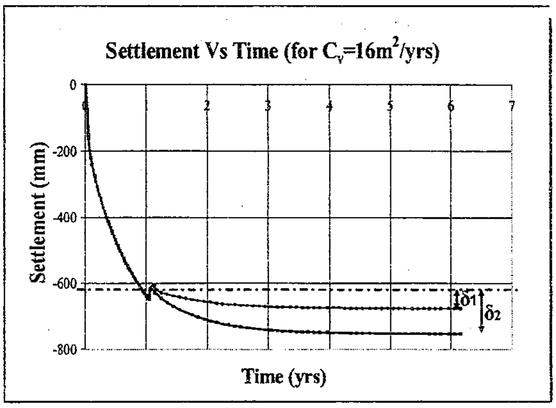

Fig 7 comparison Settlement Vs Time curves for 1-year consolidation time

δ_1 – Settlement of service lines with proposed method.

δ_2 – Settlement of service lines under normal fill.

Consolidation time	C_V=4m^2/yr	C_V=8m^2/yr	C_V=16m^2/yr	C_V=32m^2/yr
6 months	360	300	170	60
1 year	257	192	65	15
2 year	127	59	14	8

Table 3- The amount of settlement in mm after 5 years from the period of laying pipes

6. CONCLUSION

A construction procedure was proposed for the laying of service lines on the fill done on soft peaty clays.

This involves the use of a lightweight fill material. The fill material was developed mixing the conventional lateritic fill with tyre chips to different proportions. It was found that the developed fill material had a lower density, but was of sufficient strength and stiffness.

The proposed construction procedure was numerically simulated using the CRISP finite element package. The modified cam clay model was used to model the soft peat clay and the fill material was modelling as a linear elastic material.

The process of consolidation of peaty clay under the initial fill and the effect of its removal and refilling with lightweight fill was also modelled by the CRISP.

The proposed method was found to be very effective in reducing the in service settlement of the service lines.

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RATE OF LOADING IN UNDRAINED TRIAXIAL TEST; A TECHNICAL NOTE

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ABSTRACT

A residual soil collected from a weathering profile in Kandy was subjected to a set of consolidated undrained triaxial compression tests in a remoulded state. Tests that allowed partial excess pore pressure equalization during the compression stage were aimed at observing the effect of a faster rate of loading than that required for complete dissipation of excess pore water pressure on the state paths and the failure state.

The experimental results were compared with a set of tests carried out on the same soil in the conventional manner allowing 95% excess pore pressure equalization during shearing. Comparisons showed that the deviator stress at failure with partial dissipation of excess pore water pressure was less than that of the corresponding conventional test. However, the effective stress ratio q/p′ at failure remained almost at a constant value in the both instances irrespective of the rate of loading.

INTRODUCTION

Triaxial test is one of the most reliable methods available for determining shear strength parameters of a soil and is suitable for all types of soils. In addition, it has the advantage that drainage conditions can be controlled, enabling saturated soils of low permeability to be consolidated. Pore water pressure can be measured in all stages of a test if required.

In Srilanka, due to physiography of the island residual soils are encountered almost everywhere. So as to obtain reliable shear strength and other design parameters it is recommended by previous researches to test representative specimens of larger sizes than those used in conventional testing (eg; Tennakoon, Thurairajah (1985), Blight (1997)).

When it comes to commercial testing one of the problems in the adoption of larger size specimens is that the longer duration of time needed in performing the test. As a result there may be a tendency of the testing engineer go for faster rates of testing in the determination of relevant parameters.

This project was therefore under taken to study the consequences of a faster rate of loading during the stage of shearing in consolidated undrained triaxial test. Remoulded state of a residual soil was selected in the pilot study as guidance on the determination of the rate of loading is given based on the theories developed for remoulded soils.

OBJECTIVE

Objective of this study is to investigate the stress-strain behavior of residual soils in remoulded state for a faster rate of loading than conventional and to compare these behavior with that for the conventional rate of loading.

SCOPE OF THE STUDY

In order to investigate the effective of rate of loading on the state paths and failure state in triaxial compressive test at least two series of triaxial tests need to be carried out. Considering the time element and the resources available for testing a series of triaxial tests was carried out at a faster rate than that allow 95% excess pore water pressure equalization during loading.

The results were compared with triaxial tests carried out on the same soil allowing 95% excess pore water pressure equalization during shearing reported by Fowze (2002). The selection was made as it has been shown that the stress-strain behavior of the tested soil was well predicted by the modified cam clay model up to the failure state.

Specimens were sheared after consolidating them isotropically to four different effective self pressures; 100kPa, 200kPa, 300kPa, 400kPa rate of loading that allowed partial excess pore water pressure equalization during shearing was approximately thrice the rate that allowed complete excess pore water pressure equalization during shearing.

METHODOLOGY

This research study was carried in the following manner.

1. SAMPLING

Completely decomposed and almost uniform residual material was taken from a weathering profile located in Kalugamuwa, Kandy. Based on visual examinations the parent rock was inferred to be Garnet Biotite Feldspathic GNEISS. According to the British standard soil classification system the soil is classified as sandy CLAY of intermediate plasticity, CIS.

2. REMOULDING

The required amount of sample was placed inside the remoulding apparatus and distilled water was added. The amount of water was nearly added two times the liquid limit of the sample. Then a motorized rotary mixer was operated for a period of 10-12 hours continuously to prepare essential homogeneous slurry.

3. SAMPLE PREPARATION FOR TRIAXIAL TESTS

The remolded sample was spooned into the mould having the diameter of 152.45mm and the height of 178.00mm. Then the sample was consolidated one dimensionally up to 30-40 kPa by which 38mm triaxial sample was prepared after keeping it for 2 weeks.
Four specimens were prepared from a mould, and one of them was tested immediately after preparation.
The other three, after sealing the top and bottom surface of the thin wall tubes, were preserved in a humid environment.

4. TRIAXIAL TESTING

Test was carried out in following manner.

4.1. SATURATION

Although the specimens prepared from the slurry was found to be in a near saturated condition, to have a Skempton's B value greater than 0.95, a backpressure of 90kPa was applied in all cases.

4.2. CONSOLIDATION

After saturation the specimens were brought to the required effective stress state by consolidation. The effective consolidation pressures were 100,200,300 and 400kPa. During the consolidation stage the percentage of pore pressure dissipation vs square root time curve was plotted.

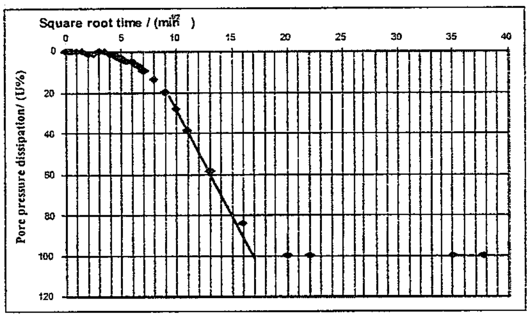

Fig.1. Percentage of excess pore pressure dissipation vs square root time

4.3. SHEARING

From the percentage of pore pressure dissipation vs square root time plot, t_{100} (Time require for 100 % of pore pressure dissipation) was found out and the failure time was calculated from the equation, Significant testing time, $t_f = F * t_{100}$ where F was taken from Bishop and Henkal (1957). By assuming the failure strain as 5% strain rate was calculated as 0.03%/min. In the series of tests performed, strain rate was selected as 0.08%min that allows partial excess pore water pressure equalization during shearing.

RESULTS AND DISCUSSION

Values for critical state constant and friction angle

PARAMETERS	VALUE
M	1.1
ϕ	27.7

TABLE 1

BEHAVIOUR OF SPECIMAN SHEARED IN THE PROPER RATE

Fowze (2002) illustrated the stress-strain and stress path diagram obtained from similar tests to the above carried out on remoulded specimens isotropically consolidated under different effective pressure of 100, 200, 300, and 400kPa. The relationship between deviator stress and effective mean stress at different consolidation pressures were found to be geometrically similar (Ref. Fig.2).

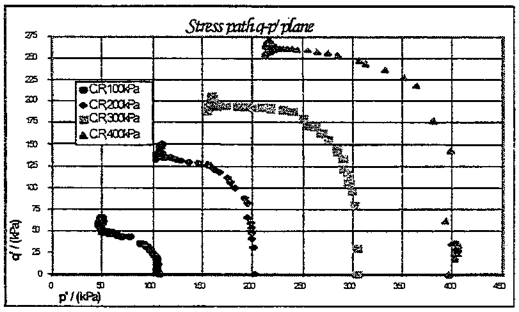

Fig.2. Stress path q vs p′ plane

BEHAVIOUR OF SPECIMAN SHEARED IN THE IN FASTER RATE

In the case of stress path of the above tests, the pore-water pressure buildup is positive and relatively high especially for large effective consolidation pressure (Ref. Fig.3).

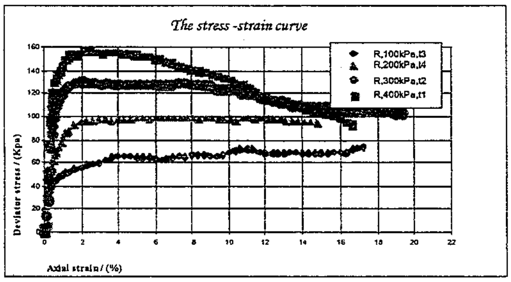

Fig.3. The deviator stress variation with axial strain

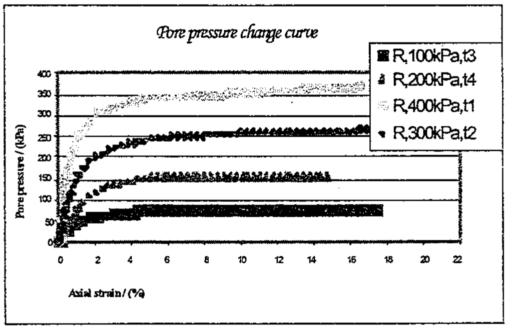

Fig.4. Pore pressure variation with axial strain

The effective stress path in each case leans to the left of vertical throughout the loading. All the stress paths indicate that specimens behave as normally consolidated. However, the relationship between deviator stress and effective mean stress at different consolidation pressures were found to be geometrically dissimilar.

The peak and the ultimate failure points of P′=100 are approximately, same and the strains at the failure and for q/p′ ratio at failure for all cases are nearly the same. (Ref. Fig.5&6).

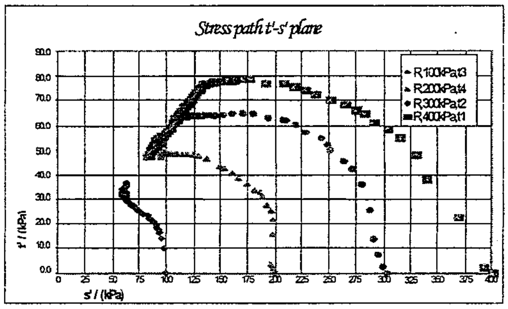

Fig.5. Stress path t′ vs s′ plane

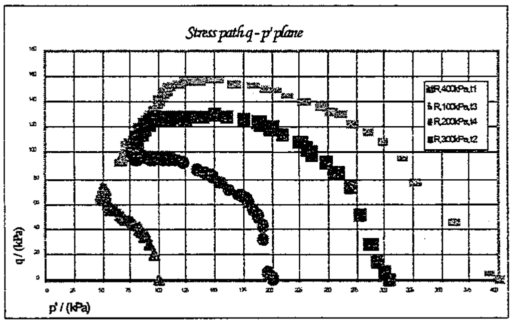

Fig.6. Stress path q vs p′ plane

Some of the special features, observed from the results are discussed below. The deviatoric stress and the effective mean stress of two cases such as P′=300 and 400kPa, decrease after the failure making the stress paths travel down (Ref. Fig.6). In the case of high mean effective stresses, the increment of deviatoric stresses decrease with the constant increment of effective mean stress (Ref. Fig.6). Even though, the stress paths obtained were as the expected trend, they are not geometrically similar. Therefore, the normalization was unsuccessfully (Ref. Fig.7).

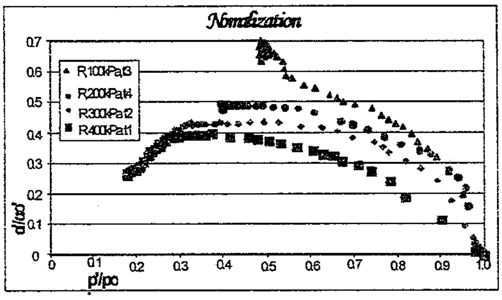

Fig.7. Normalization curve

COMPARISON BETWEEN CONVENTIONAL RATE AND A FASTER RATE

When the results of this study are compared with that of the case of conventional rate of loading for all cases, the deviatoric stresses at failure were less than that of the corresponding conventional test as expected (Ref. Fig.9). The reductions in the deviator stress at failure for different effective consolidation pressures such as, 100, 200, 300, and 400kPa are 2%, 25.0%, 37.5% and 38.5% respectively when the tests are done at higher rate. And the value of q/p' at failure is independent of the rate of loading.

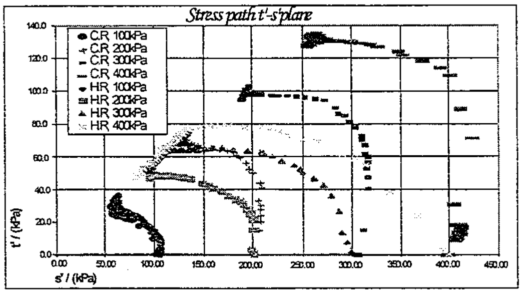

Fig.8. Stress path t' vs s' plane

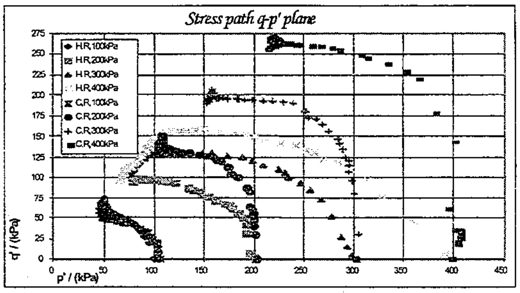

Fig.9. Stress path q vs p' plane

CONCLUSION

A series of consolidated undrained triaxial compression tests in a remoulded state were carried out at a faster rate than that required for complete dissipation of excess pore water pressure on the state paths and the failure state.

The q_f' has found to be less than that of which could obtained by performing at the proper rate. However, an interesting point to note is that q/p' at failure for all tests remained almost at a constant value thus, that is giving a unique critical state line for both cases

RESEARCH SUGGESTION FOR FUTURE

The future research should be carried out on remoulded sample at more different strain rates and for the same effective consolidation pressure. At the same time undrained triaxial tests for undisturbed residual soil should be carried out with different strain rates and for the same effective consolidation pressure.

ACKNOWLEDGEMENT

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FINITE ELEMENT ANALYSIS OF GROUND IMPROVEMENT FOR A SHALLOW FOUNDATION.

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ABSTRACT

A weak soil beneath a shallow foundation may be replaced by a strong soil so that the load coming from the super structure can be safely carried by it without undue settlement. Soil replacement method is a popular ground improvement technique in Sri Lanka. Finding the optimum arrangement for the geometry of the replacement area will be helpful in real practice. It has been decided to carryout a finite element analysis to find the optimum arrangement of soil replacement. Both elastic and elasto-plastic analyses were performed to obtain the variation of foundation settlement with the depth, width and the shape of the fill volume. In the first series of analysis existing soil and the fill materials were modeled as linear elastic materials with fill material stiffer than the existing soil. In the second series of analysis both materials were modeled as Tresca type elasto-plastic materials. A computer software called AFENA5 developed at the University of Sydney, Australia was used for the analyses. The bearing capacity of the foundation obtained from the finite element analysis compared with that derived from upper bound theorem in some instants and they were agreed. Immediate, consolidation and total settlements were analysed in the elastic case. In the elasto-plastic analyses, in addition to the above, the variation of immediate settlement with increasing load was also carried out to obtain the ultimate bearing capacity. A factor of safety of 2.5 was assumed to find the safe bearing capacity. The settlements at the safe bearing pressure, found from the finite element analyses, were used to find the optimum arrangement for soil replacement.

INTRODUCTION

Increasing scarcity of land in urban areas often requires using sites with soft ground for development purposes. In most of the cases, the site can be developed with some kind of ground improvement. Many methods of ground improvement techniques have been developed to suit particular ground condition and / or the purpose of improvement. For a particular site, a combination of the methods available may lead to the best possible ground improvement. Popular ground improvement techniques in Sri Lanka are soil replacement, preloading with vertical drains and compaction.

A properly designed foundation should not settle excessively under the working load. The differential settlement between one part of a structure and another is of great significance to stability of the super structure than the total settlement. Therefore, it is necessary to find displacements, stresses and strains beneath foundations, to select suitable ground improvement method and the extent of ground improvement. This can be achieved by the use of finite element analysis.

The finite method in its application is depended on the skilful use of computer and efficient programming techniques. There are many finite element analysis software available. Here the software called AFENA-version 5.0 is used for analysis.

The stresses within a linear elastic semi-infinite homogeneous isotropic mass due to a point load on the surface, were determined by Boussinesq in 1885. The stresses due to surface loads distributed over a particular area within the same mass can be obtained from the point load solution by integration. The stresses at a point due to more than one surface load are also obtained by superposition. However, the stress – strain behaviour of a soil is known to be non – linear. The relations between stress and strain are much more complicated than assumed from elastic behaviour. Therefore, in order to represent a geotechnical problem realistically some form of non-liner stress strain relationship is more desirable. Since the load deformation behaviour has no one to one correspondence, and different loading sequences give different deformation pattern, principle of super position cannot be used. The problem has to be solved incrementally following the prescribed load path.

METHODOLOGY

IDEALIZATION

The actual problem is idealised as follows: Foundation is flexible strip footing with infinite length, subjected to a uniformly distributed vertical load under plain strain conditions. Existing soil and the replacing soil were modelled using either homogeneous isotropic finite element or elasto-plastic finite element with different stiffness. The gravity effect of the soil is neglected. Determination of stresses and displacements is based on elastic theory / elasto plastic theory. Settlements are undrained settlements or immediate settlements followed by consolidation settlements. Internal friction angle is taken as zero and total stress analysis is carried out with undrained shear strength c_u. Slipping effect is not taken into account in between surface of the footing and the ground surface and between soil layers.

LOADING AND BOUNDARY CONDITIONS

In the elastic analysis a unit distributed load acting on a unit length along the ground surface is used for analysis. However, in the elasto-plastic analysis to find the actual plastic behaviour, incremental loading is used. Loading function available in AFENA is used with a dummy time variable. Load is increased with the increment of (E/1000) units up to $5c_u$, where E/ c_u ratio is used as 50.

In each problem the bottom and side boundaries were chosen at sufficient distance away from the centre of the foundation to ensure the effect of the loading can be neglected at the boundaries. The bottom boundary is restrained against vertical displacement and the side boundary is also restrained for horizontal displacement. Since the problem is symmetric, the nodes along symmetric axis were also restrained for horizontal displacement. Both end of the bottom boundaries were restrained for translation

REPLACEMENT OF SOIL
The existing soil is assumed to have the Young's modulus E=5000 units and poisson's ratio =0.5. Different arrangements of replacements are performed as given in Fig 1. The analysis was carried out for X/B values of 1,2 and 4 with D/B ratio of 1,2 and 4 at each X/B. Where B is the width of the foundation

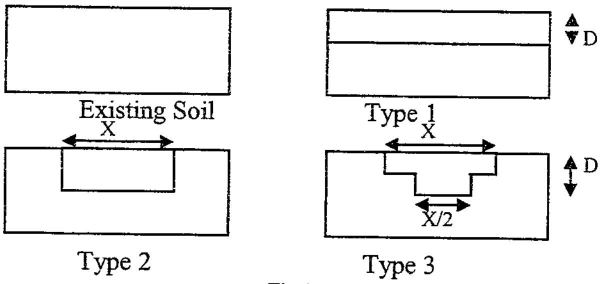


Fig 1

Both elastic analysis and Elasto-plastic analysis were carried out with replacing soil having young modulus five times that of existing soil and the poisson's ratio was assumed to be 0.5 for both soils to represent incompressible behaviour under undrained condition.

MESH GENERATION
Rectangular 8 node 2D elements were used for this finite analysis. In order to select a finite element mesh suitable for the problem and to test the numerical accuracy of the element, structure was analysed by subdividing the mesh until difference of further subdivision become in significant. The results in the elastic case of existing soil without any replacement were compared with the analytical solutions obtained from the classical standard equations available in the literature. The relative displacement respect to the bottom centre of the foundation was calculated for different arrangement of meshes in the finite element analysis and compared with those obtained using the following formula for a strip footing on a single layered soil. (As shown in Fig 2).

$$\rho_z(x,o) - \rho_z(o,o) = \frac{2p(1-v^2)}{\pi E}\{(x-b)\ln|x-b| - (x+b)\ln|x+b| + 2b\ln b\}$$

(Poulos, H,G and Davis, E.H)

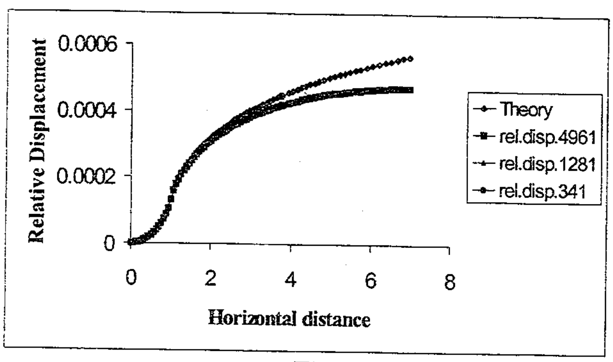


Fig 2

Theoretical calculations differ from values calculated by using AFENA in the region closer to the side boundaries. The reason for that difference is the restrain at those boundaries in the finite element analysis. Since the settlements under the foundation are the most important, the above differences will not affect the comparisons. Taking the above factor into consideration and keeping the number of nodes and elements low, the mesh with 400 elements and 1281 nodes shown in fig. 3 was chosen for further analysis.

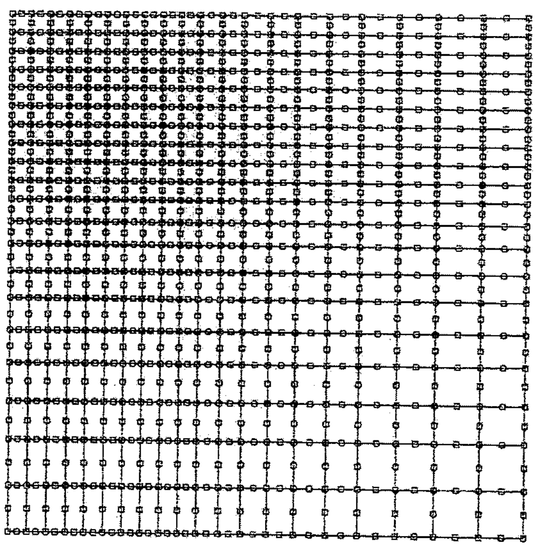


Fig 3

SETTLEMENT
The immediate settlement was found from AFENA programme for the elastic and elasto-plastic cases.
For the multi layered soil, Gibson (1966) has shown that the variation of models with depth has little effect on the distribution of stress, but has a marked effect on the surface displacement, which are concentrated within the loaded area for an incompressible medium. By subtracting the effect of a hypothetical layer above each real layer, settlement of each layer may be found and add up to obtain the total immediate settlement. The results found from the above were used to compare the results from the AFENA. (Fig 4)

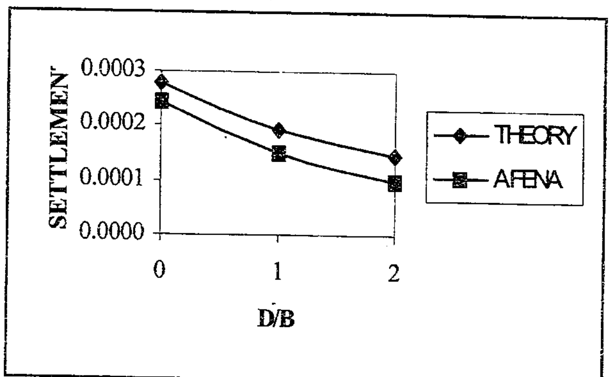


Fig 4

The consolidated settlements were calculated using the vertical stress distribution beneath the centre of the foundation using the classical formula $\delta_c = \int m_v \, \Delta\sigma \, dz$. Here $\Delta\sigma$ values were obtained from the analysed results of AFENA; m_v was taken as $^1/_E$ in both elastic and elasto-plastic analysis.

ULTIMATE BEARING CAPACITY

In the case of plastic analysis for type 2 arrangement ultimate bearing capacity was calculated using upper bound theorem and check with finite element solution. Assumed failure mechanism is illustrated in Fig 5

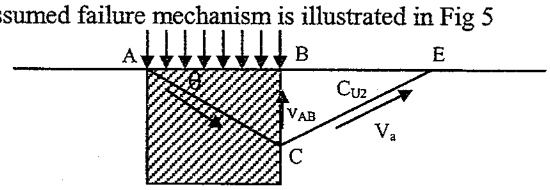
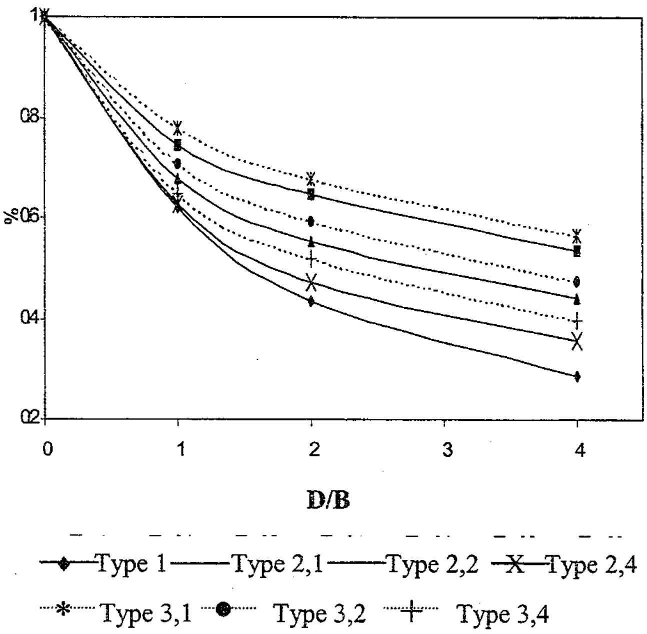


Fig 5

In the above mechanism the corresponding value of θ to get the upper bound for the bearing capacity was invariant for the D/B ratio considered. The ultimate bearing capacity was compared with finite element solution. The Calculated value was 1385 and the AFENA result for the D/B ratio was 1300.

RESULTS AND COMPARISON OF RESULTS

In the analysis for each geometry arrangements of the filling immediate settlement and consolidation settlement were calculated and variation was plotted against each X/B and D/B ratios. Thus total settlement was calculated and the variation was plotted against each X/B and D/B ratios. Then the settlements corresponding to each geometric arrangement were normalized with the settlement of existing soil and given as a percentage in fig 6.



Type N, L means X/B ratio L for type N arrangement

Fig 6

Type 1 arrangement has 70% improvement in settlement and types 2 and 3 arrangement have nearly 50% improvement and shows not much improvement in settlement after X/B = 2 with the increment of D/B ratio. When D/B = 2, improvement in settlement is nearly 50% but for D/B = 4 only a 10% increment in improvement. Therefore considering only improvement in settlement type 1 and both types 2 and 3 corresponding to X/B = 2 and D/B = 2 were better arrangement.

In selecting the optimum arrangement not only improvement in settlement but also the volume of soil needed for replacement also important. Fig 7 shows the percentage of settlement with the volume of soil needed for replacement.

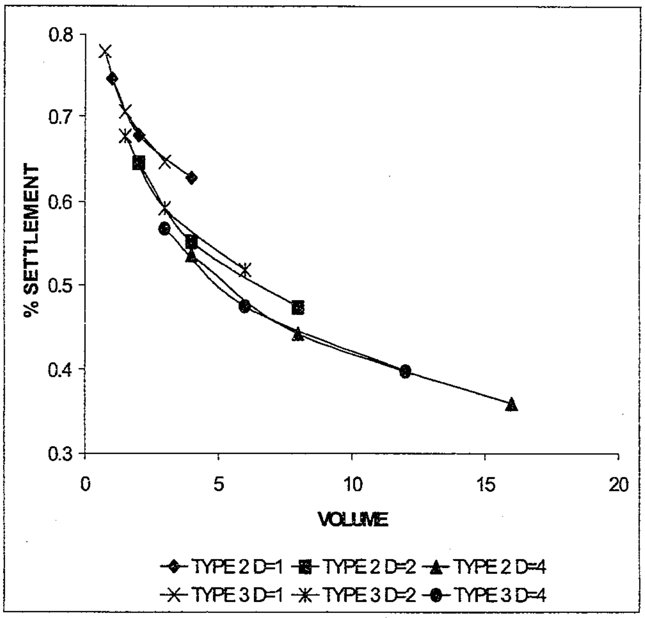


Fig 7

The volume of replacement soil for type 1 is much larger than that of types 2 and 3 arrangements. Therefore type 1 arrangement will not be economical. The volume of replacement soil for type 3 is less than type 2 and the excavation for type 3 is better than type 2. Therefore based on elastic analysis type 3 arrangement with X/B = 2 and D/B = 2 is the best arrangement.

ELASTO-PLASTIC ANALYSIS

In the elasto-plastic analysis, analysis was done for incremental loading with load increment 25. For the different arrangement the graphs of load Vs settlement were plotted. From the above graphs, failure load were determined. By imposing a factor of safety against bearing capacity failure as 2.5, safe working load was calculated. Immediate and consolidation settlement determined for safe working load. Thus total settlement was calculated for the safe working load. For the different geometrical arrangement, the safe working load normalised with the safe working load of existing soil were plotted against the D/B ratio in fig 8.

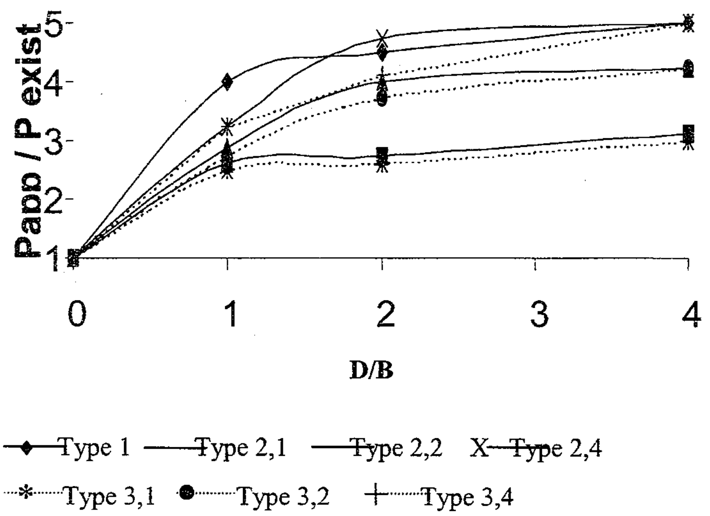


Fig 8

In all three cases no significant improvement after D/B = 2. In the case of types 2 and 3 the improvement is same. It can be clearly seen that improvement after X/B = 2 is not significant when considering the amount of volume of soil needed for replacement.

The volume of replacement soil for type 1 is much larger than that of types 2 and 3 arrangements. Therefore type 1 arrangement will not be economical. The volume of replacement soil for type 3 is less than type 2 and the excavation for type 3 is better than type 2. Therefore based on plastic analysis type 3 arrangement with X/B= 2 and D/B = 2 is the best arrangement.

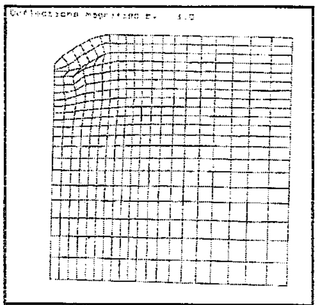


Fig 9

Therefore improvement in type 3 arrangement with X/B = 2 and D/B = 2 can give the optimum arrangement for all cases.

Fig 9 shows the deformed mesh for the optimum arrangement.

The immediate settlement variation for the ultimate load in the optimum arrangement is shown in Fig 10.

It shows that the effects along the boundaries are not significant.

CONCLUSION

As depth of replaced soil increases settlement decreases and no considerable effect after D=2B and with increase on width of the replacement soil settlement decreases no significance effect after X=2B. As overall type 3 arrangement with D=X=2B will be the optimum arrangement.

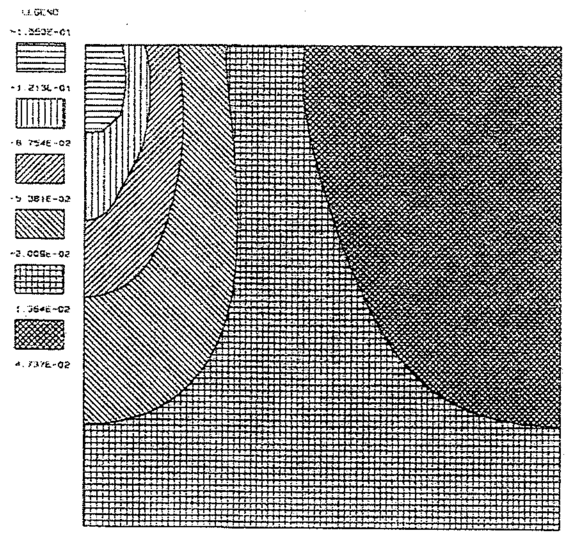


Fig 10

RECOMMENDATION

This project can be continued as a design of the shallow foundation with a particular arrangement of soil replacement method. Here all the necessary analysis was done, and the corresponding displacements are presented. By considering the limits and conditions the allowable settlement can be found, and using trial and error approach the respective loading can be found.

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AN EXPERIMENTAL STUDY ON THE EFFECT OF CEMENT STABILIZATION ON THE PERMEABILITY AND COMPRESSIVE STRENGTH OF A SOIL

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ABSTRACT

An experimental study has been carried out to investigate the variation of permeability, compressive strength and dry density of a stabilized soil sample with different percentage of cement content. Here each specimen of soil was prepared at optimum moisture content. A clayey sand sample was used for the study which had 10 % clay and 90% river sand. Different amount of cement (4%, 8%, and 12% of cement by weight) were added to the soil sample which was then mixed and compacted at optimum moisture content to prepare the specimens. These experiments were conducted in accordance with BS1377: 1990 standards. A special apparatus was designed and made to measure the permeability of the specimen in the direction perpendicular to compaction (horizontal permeability).The conventional falling head apparatus was used to measure the permeability of the specimen in the vertical direction. The permeability was measured after 1 day and 7days of curing time. The uniaxial compressive strength of the specimen was measured after 1-day, 7days, and 28days curing time. The permeability values obtained in the direction of compaction was larger than the values obtained perpendicular to the direction of compaction. The compressive strength of the specimens was in agreement with Duff Abrams law.

INTRODUCTION

As the time goes there are always new technologies developed in the world to challenge the nature. Cement stabilization is a technique used to improve low graded soils so that they become suitable for engineering purposes such as dam and road construction. The cement stabilized soil initially was used in USA for road construction. Since then it has been extensively used for various other purposes around the world. Today it is mainly used in dam, building and road construction.

In dam construction the permeability of the core material in the horizontal direction plays an important role. Therefore, it is necessary to study the variation of permeability of the stabilized soil in that direction, if it is to be used in the field. For road construction the bearing capacity is an important factor. A soil sample compacted at the optimum moisture content will have a considerable bearing strength which reduces when its moisture content is altered. For instance a granular soil in wet condition will have considerable bearing strength become friable when dry. A cohesives Soil has good strength when dry but becomes plastic in the presence of sufficient water.

The objective of soil stabilization is to improve the engineering properties of the soil and maintain it in that condition. There are several materials used to stabilize the soil such as bitumen, lime, cement and resinous materials. In this project cement added soil was studied for its properties.

In general soil cement stabilization involves pulverization, mixing and blending of material in the field with a predetermined amount of cement and moisture. Then it can be compacted to the required thickness. Therefore, as explained earlier, it is necessary to obtain the optimum moisture and cement content for stabilization, normally by conducting experiments in the laboratory. In this project different proportion of cement soil mix was tested for permeability and strength. Then the variation was compared with the cement content in order to obtain the optimum condition for the stabilization.

OBJECTIVE

The objectives of the project are as follows:

Study the variation of optimum moisture content and the dry density with different soil cement mix.

Study the variation of uniaxial compressive strength with different soil cement mix and curing time.

Study the variation of permeability in the directions parallel and perpendicular to the compaction direction with the curing time.

LITERATURE REVIEW

The method of study which was used for experiment, based on several books (Ref 2, 3, 4, 5) and the work of a previous student group (Ref 1). The earlier project (Ref 1) deals with the variation of compressive strength with the cement content where a sandy soil sample was used for the study. However in the current project, an artificially prepared clayey sand sample was used. As clayey sand is more suitable for stabilization compared to sandy soil. The aim of the study was to test the applicability of the stabilized soil in dam construction. Therefore, it was necessary to measure the horizontal permeability variation of stabilized soil with different amount of cement content. To carryout this study, an apparatus (Figure 01) was made based on the research work by M.Livneh and E.shklarsky (Ref 5).

APPARATUS

The illustration of the apparatus with the specimen inside is shown in Figure 1.The apparatus is a modified falling head permeameter. The specimen has a central hole and an outer annulus space, which are filled with a granular material. In the apparatus, only the radial seepage of water through the specimen is allowed. Bentonite clay is used to prevent seepage in direction other than the radial direction. The head is applied through the tube which connects the central hole of the specimen. Then the water flows through the specimen in the horizontal direction as shown in the figure. Finally this water is collected at the outer annulus space. Then this water spills out through the outlet tube. To ensure a horizontal flow the outlet tube was raised a little bit higher than the specimen height as shown in the figure. The permeability of the specimen can be expressed by the following equation using the classified theory of seepage.

$$K = a\ln\left(\frac{h_1}{h_2}\right)\ln\left(\frac{r_1}{r_2}\right)\Big/2\pi HT$$

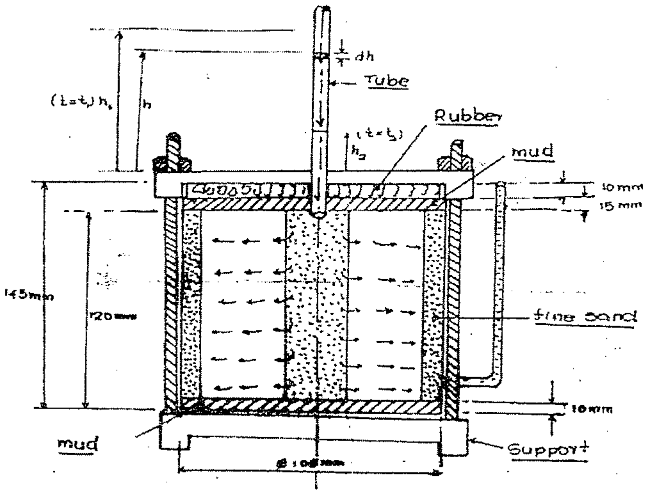
Where
K-permeability
a -cross sectional area of the central hole pipe.
h_1-initial head
h_2-final head
r_2-outer radius of the specimen
r_1-central hole radius
H-sample height
T-time taken for the head change from h1 to h2

METHODOLOGY

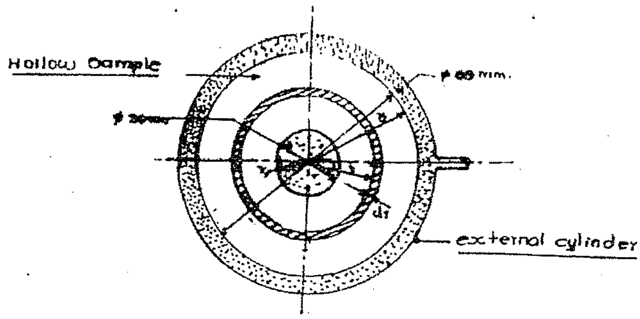
In this project, clayey sand, a soil prepared by mixing 90% of river sand with 10% of clay, was used. First the river sand and clay were mixed in a riffle box to produce a homogenous mix. Then the soil sample was divided in to several 3 kg packets by using the riffle box and these samples were used to prepare the specimens. Experiments carried out were mechanical analysis, Atterberg limit, organic content test, specific gravity test, standard and the modified compaction test, uniaxial compressive strength test and the falling head permeability test .All these tests were done according to BS1377:1990 Standards. The special apparatus shown in Figure 1 was prepared and used to measure the permeability in the direction perpendicular to the compaction.

The samples were prepared with 4%, 8% and12% of ordinary Portland cement (by weight). To measure the uniaxial compressive strength each sample was molded into 100mm diameter*200mm height cylindrical sample by using the standard compaction test. They were tested for compressive strength after 1 day, 7day and 28-day curing time. The universal compressive testing machine was used to measure the compressive strength. The procedure adopted to use the permeability apparatus is as follows.

First the soil cement mix at particular proportion was compacted at optimum moisture content in the standard compaction test apparatus then the specimen was allowed to strengthen for another one day. After that the central hole of 20 mm diameter was drilled using the thin tube cuter; a special cutter prepared for this purpose. Next the sample was allowed to cure for one day. Following day the sample was removed from the mould by using a hydraulic jack. Then the specimen was centrally placed in the apparatus. The annulus space and the central hole of the specimen were filled with a granular soil. To seal leakages, the bentonite clay was applied. Then by connecting the pipe to the apparatus, 1.5m pressure head was applied. To release the entrapped air, the head was applied to the outlet. After the saturation of the soil sample drainage outlets were sealed and any variation in the water level was observed to ensure that there is no leakage. Finally the observations were taken.



SECTIONAL ELEVATION



PLAN

HORIZONTAL PERMEABILITY TEST APPARATUS

FIGURE 01

RESULTS

Figure 2 shows the gradation curve of the soil sample used. The figure reveals that the clay content is 5% and the total fine particles are approximately 8%.Also 60% of the particle are lees than 2 mm in size. Therefore, this soil can be classified according to the BSCS as follows SWC: Well graded, clayey SAND. Figures 3 TO 6 show the variation of permeability and compressive strength for the samples prepared with different cement content. From Figure 2, it is clear that the compressive strength steadily increases with the cement content and the curing time. The rate of increment of one day strength with the cement content is the highest among 1 day, 7days and 28 days strength. Figure 3 shows the dry density increment with the cement content and the variation is marginal. Similar behaviors were reported by the previous group project Ref 1. From the variation of permeability shown in figures 4, 5 and 6, it can be concluded that vertical permeability is always higher than the horizontal permeability and that when the curing time increases the permeability reduces. From Figure6, it can be concluded that at least 5% cement should be added to bring down the permeability to a considerably lower value so that stabilized soil can be used in dam construction.

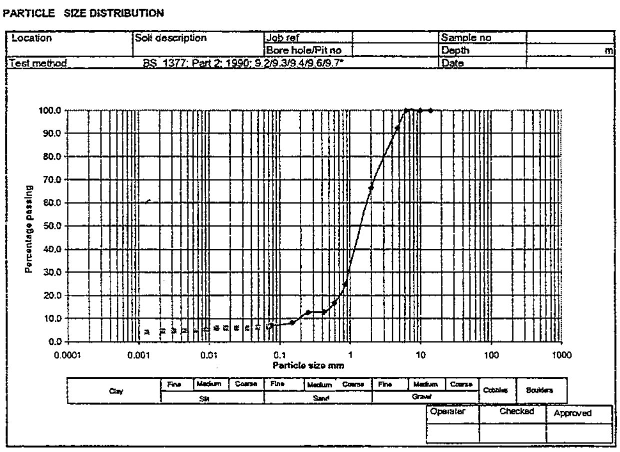


FIGURE 02

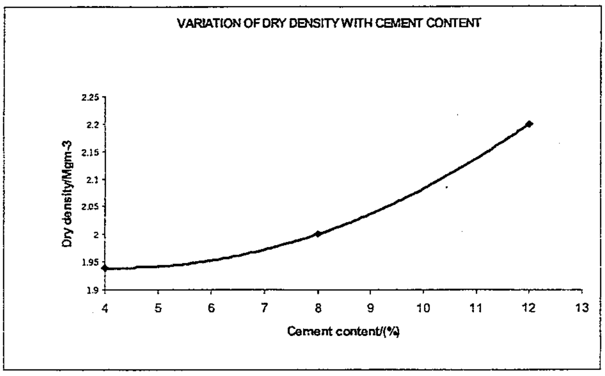


FIGURE 03

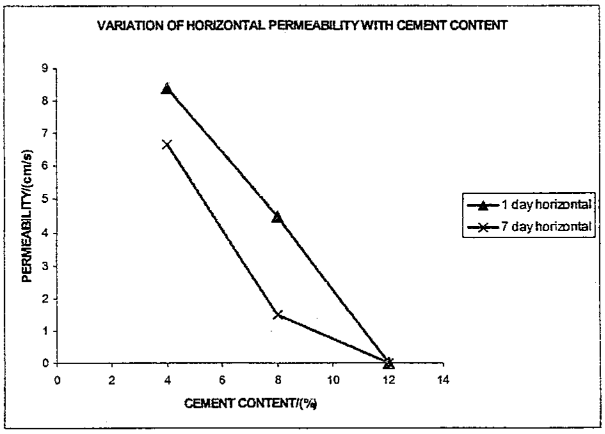


FIGURE 04

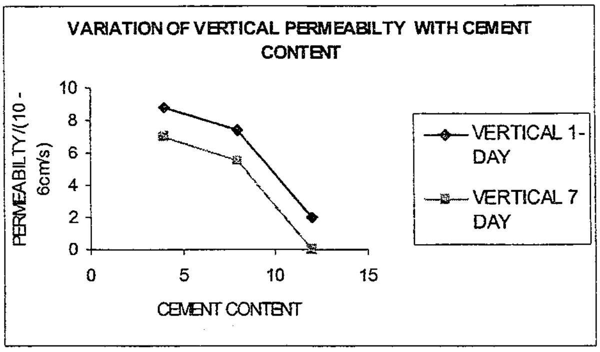


FIGURE 05

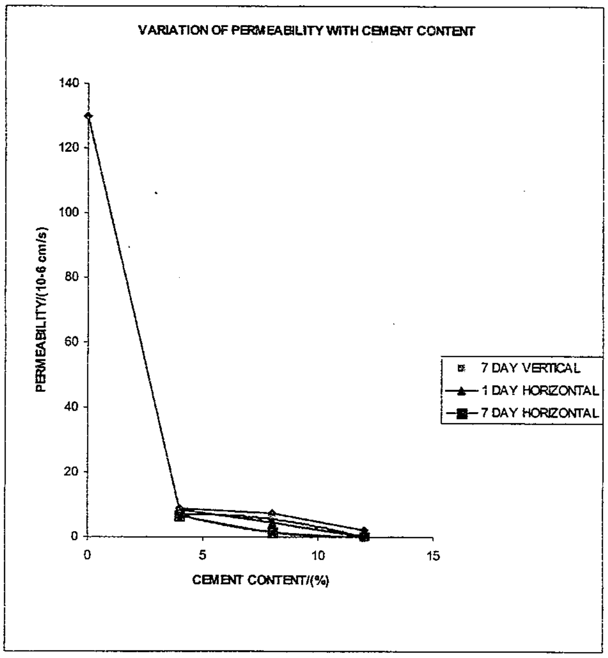


FIGURE 06

CONCLUSIONS

1. Maximum dry density increases with the cement content marginally (Figure 03). However, the optimum moisture content does not vary with cement content.

2. Uniaxial compressive strength increases with cement content (Figure 03) and curing time but the rate of increase are different for different cement content (Figure 03)

3. The permeability, both vertical and horizontal, decreases with increasing cement content (Figures 5 & 6). The variation of vertical permeability with cement content shows similar for both 1day and 7days tests. The variation of horizontal permeability with cement content for 1-day test varies linearly where as the Permeability after 7 days curing has a non-linear pattern. The value of horizontal permeability value always lesser than the vertical permeability.

RECOMMENDATION FOR FUTURE WORK

Further studies can be carried out to investigate the permeability variation with modified compaction, which we omitted due to time restriction. Another important factor is durability of the stabilized soil. There fore it should be studied to select the stabilization parameters. The permeability suddenly reduces by 10 times when the cement content increases from 0 to 5%. There fore the variation in that region should be studied.

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