GEOTECHNICAL ENGINEERING PROJECT DAY 2018

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

June 14, 2018
At IESL Auditorium

Organised by the
SRI LANKAN GEOTECHNICAL SOCIETY

SLGS

GEOTECHNICAL ENGINEERING
PROJECT DAY – 2018

Message from the President - SLGS

From its inception, Sri Lankan Geotechnical Society has provided a forum for disseminating new knowledge in the field of geotechnical engineering and promoting research. The Project Day competition was commenced in year 2000 with the objective of encouraging the undergraduates conducting projects in the field of Geotechnical Engineering to do good research, publish and make effective presentations. Participants are expected to present their findings in a concise four paged paper and make a 15 minute oral presentation. The best paper and the second paper will receive cash awards and certificates. The competition was held annually uninterrupted to date.

Many winners in the past years have proceeded to do higher studies and established good carriers in the field of Geotechnical Engineering as both academics and practicing engineers.

It is encouraging to note that there are nine papers on a wide variety of topics this year. I thank all the authors for their interest and commitment and hope they will continue with the habit of presenting their research in written form. It is only when one starts to write his findings he would realize the gaps in his work or knowledge and would be able to rectify them.

Currently there are many ongoing projects with wide applications in Geotechnical Engineering. We believe that many research ideas would generate in this background and a large number of research papers would be submitted to the next international conference, ICGEColombo2020 which is to be held in August 2020.

I also wish to convey my sincere gratitude to the panel of evaluators; Emeritus Professor B. L. Tennekoon, Prof. H. S. Thilakasiri and Dr. J. S. M. Fowze.

Prof. Athula Kulathilaka
President – SLGS

CONTENTS

Relationship Between Particle Size and Permeability of Different Soils

A.H.A. Ashrar
Department of Civil Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka.

ABSTRACT: This research paper describes the regression-based model to predict the permeability of compacted soils from particle size distribution. The models incorporate parameter values that adequately represent the distribution of particle sizes. The proposed relationship between particle size and permeability was found with the R^2 value of 0.9988 and it is compared with existing empirical equations. The result suggests that the relationship is better than existing relationships. The developed relationship helps to for example, laboratory measurement of the permeability of some certain soils that may take several days to perform or that may be difficult to prepare samples. It may also be used to give preliminary information about the permeability in a field environment in certain circumstances.

1 INTRODUCTION

Permeability k, is an important soil physical characteristic. Permeability takes importance in relation to geotechnical problems, including settlement computations, seepage losses and stability analysis. But, direct measurement of permeability is time consuming and costly. So, there were some indirect methods were developed to predict permeability from readily available soil properties. Different methods have been proposed to determine permeability from empirical equation for both field and laboratory methods.

In earlier stage some researchers developed, the relationship between permeability and pore sizes. But they realised, pore size distribution was almost difficult to determine. So, pore size distribution was substituted by particle size distribution to develop the relationship.distribution to develop the relationship. Alternatively, methods of estimating permeability from particle size distribution has been used to overcome these problems. There have been attempts to estimate permeability based on particle size distribution. Most of researchers were carried out for specified soil type for last 20 years. But in our research, different soils were used to predict the relationship.

2. LITERATURE REVIEW

Empirical relations have been employed to estimate the permeability from particle size distribution (Hazen 1892; Berg 1970; Alyamani and Sen 1993). Besides empirical method, predictive methods have also been developed to estimate permeability from PSD (Carmen 1937; Kozeny 1927).

In 1892, Hazen proposed a relationship which was commonly accepted and given,

$$k = c.d_{10}^2 \quad (1)$$

Where, k is hydraulic conductivity at 20°C (cm/s); d_{10} (mm) is the tenth percentile particle size by weight and C is a dimensionless constant.

Kozeny (1927) proposed a formula which was again modified by Carman (1937 and 1956). Then, it became as Kozeny- carman equation.

Continuously, Shepard (1989) extended Hazen's relationship by performing regression analysis and find the relationship as given,

$$k = cd^b \quad (2)$$

Where, c is a constant and b is the exponent. The coefficient c ranged from 1014 to 208 818 (gal/day per ft^2) and exponent ranged from 1.11 to 2.05 with an average 1.72. Values of both c and the exponent generally decrease with decreased textural maturity and increased in duration.

Alyamani and Sen (1993), from Wiebenga et al (1970) and utilized their study. They interrelated the relationship between hydraulic conductivity and grain size and given by,

$$k = 1300[I_0 + 0.025(d_{50} - d_{10})^2] \quad (3)$$

Where, I_0 = x intercept of the line formed by connecting d_{50} and d_{10} of PSD curve on paper; units of k as m/day and other parameters in mm.

Continuously, Fred Kofi Boadu (2000), Gupta (2008), Salarashayeri and Siosemarde (2012) developed different type of relationships using soil parameters including particle sizes.
The foregoing discussions clearly indicate that permeability strongly depends on particle size distribution. However, a general correlation equation between permeability for wide range gradation of particle size is not yet available. And also, especially no researchers were carried out in Srilanka on permeability with particle size and most of the studies utilized with specified soil type. So, consider these short comings in this research study different soils were tested, with wide range of gradation.

3. EXPERIMENTAL METHODS

3.1 Sample collection and preparation

Disturbed soil samples were obtained from fresh cuts for highway construction activities at A15 road from Batticaloa to Trincomalee and B10 road from Serunuwara to Kantale at 12 locations which are located in eastern province of Srilanka.

3.2 Laboratory experiments

Standard laboratory procedures (BS 1377:1990) were employed separately for all twelve samples such as mechanical analysis, Proctor compaction test and permeability test.

3.2.1 Mechanical Analysis

All samples were employed to obtain particle size distribution curve from mechanical analysis. The samples had wide range of gradation from fine to coarse particles. So, the wet-sieving method was selected to obtain the result. Wet sieving method consist of two tests, sieve analysis test and hydrometer test. Both tests were carried out according to BS 1377:1990, Part 2.

Sieve analysis test gives the coarse particle distribution and hydrometer test gives the fine particle distribution. Both distribution curves were combined to get total particle size distribution (PSD) of each sample. From PSD curves, soil parameters d_{10}, d_{30}, d_{50}, d_{60} and d_{70} were determined where the soil particle diameter (mm) that 10%, 30%, 50%, 60% and 70% of all soil particles are finer by weight.

3.2.2 Proctor Compaction Test

All samples were involved in the standard Proctor compaction test to establish the compaction curve according to BS 1377:1990, Part 4. Automatic soil compactor was used to carry out compaction test.

Maximum dry density (MDD) and optimum moisture content (OMC) were determined from compaction curve. Soil samples were prepared based on obtained MDD and OMC values for permeability tests. Specific gravity test was carried out to establish zero-void line to confirm the compaction test results.

3.2.3 Permeability test

Falling head permeability test was selected to find permeability because, soil samples had wide range of gradation from fine to coarse particles. Samples were prepared while maintaining 98% compaction level of MDD and OMC. Tests were carried out according to BS 1377:1990, Part 5.

3.3 multiple Regression Analysis

The relationship between permeability and particle size was developed using multiple regression analysis with help of a statistical software called "Minitab 17" and MS excel. Regression analysis is a statistical process for estimating the relationship among more than a variable. In this study, two variables took place such as, permeability and particle sizes.
Minitab has an option to analyze regression as best subsets; this helps to compare all possible models using a specified set of predictors and display the best fitting model.

4. TEST RESULTS

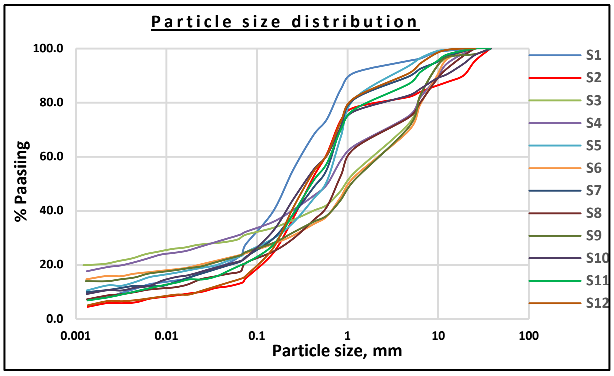

Figure 1: Particle size distribution curves of twelve samples

Particle size distribution curves (PSD) of all twelve samples were generated from results which were obtained from sieve analysis and hydrometer test. PSD curves of all samples are shown above in figure 1.

Parameters d_{10}, d_{30}, d_{50}, d_{60} and d_{70} of samples were determined from particle size distribution curves. The samples represent various different particle sizes. Figure 2 shows the graphical representation of the data.

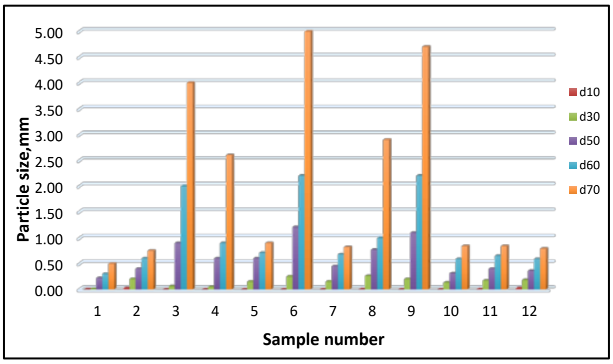

Figure 2: Variation of particle parameters of samples

So, the variation clearly indicates, samples which were used for this study have wide range of particle gradation.

By conducting standard Proctor compaction tests, compaction curves were obtained separately. Compaction curves of all samples are shown in the figure 3.

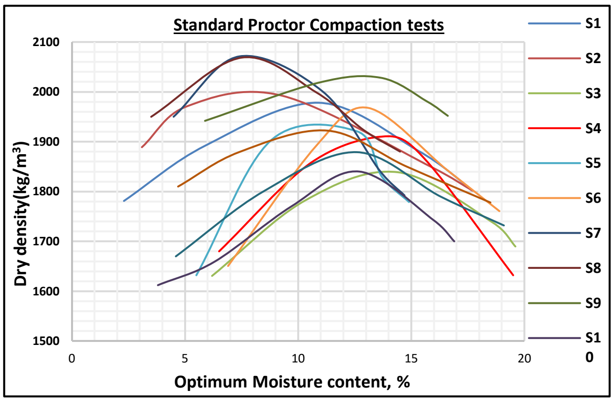

Figure 3: Compaction curves of samples

Maximum dry density (MDD) and optimum moisture content (OMC) of each and every sample were determined by respective compaction curves.

It can be seen that from results, the granular soil has high value of MDD at lower value of OMC compared to finer particle soils.

Variation of time against pressure head curves were plotted from results which were obtained from permeability test and shown in figure 4.

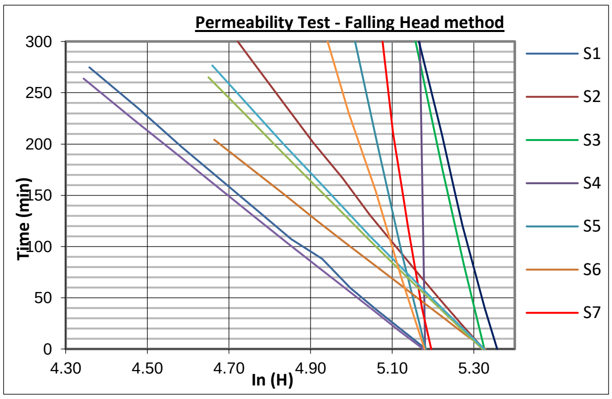

Figure 4: Time vs logarithm pressure head variation

The curves obey the linear equation.

$$\ln(H) = -\left(\frac{A \times k}{a \times L}\right)t + \ln(h_0) \qquad (4)$$

From the gradient of curves, permeability of samples was obtained. Permeability values of samples are given in table 1.

Table 1: Permeability values of samples

Sample Number	Permeability, k (cm/s)
1	1.48E-05
2	9.79E-06
3	2.48E-06
4	1.63E-07
5	2.46E-06
6	1.60E-05
7	1.56E-06
8	3.06E-06
9	1.24E-05
10	1.55E-05
11	1.19E-05
12	3.01E-06

Compare the above results with variation of soil particle diameter, it can be seen that the finer soils have lower permeability compared to coarse particles.

The relationship was developed by multiple regression analysis using Minitab 17. Permeability k (cm/s) and particle parameters, d_{10}, d_{30}, d_{50}, d_{60} and d_{70} are the soil particle diameter (mm) that 10%, 30%, 50%, 60% and 70% of all soil particles are finer by weight were input as variables and regression analysis was carried out using the option called subset regression analysis. It compares all possible models using a specified set of predictors and display the best fitting model. From the comparison, the log scale relationship gave good results. The equation was selected among many equations which has less standard error and high R^2.

And also, for a confirmation, that relationship was also checked using MS excel. That also gave nearly close results what we obtained from Minitab. Using the statistical software, the relationship between particle size and permeability is obtained as:

$$\ln k = 11.39 \ln(d_{30}) + 8.762 \ln(d_{60}) - 13.454 \ln(d_{70}) + 5.072 \ln(d_{70}-d_{60}) - 10.06 \ln(d_{30}-d_{10})$$

The measured k and predicted k are plotted in the below figure 5 and the trend line indicates a good match. So, this equation can be used to determine permeability of different soils at MDD and OMC. This can be used in foundation, dam and pavement design to find k value after compaction and before construction.

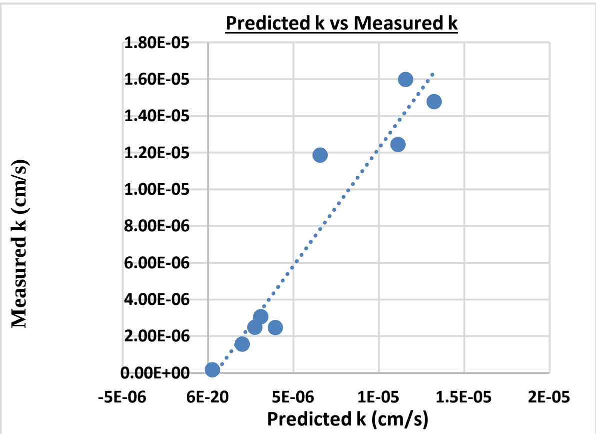

Figure 5: Measured permeability vs predicted permeability

5. CONCLUSIONS

Both field and laboratory procedures employed in the determination of permeability are tedious, expensive and time consuming. Thus, to solve a hydrological or geological problem spanning a large area, economic consideration factor, especially properties of soils may vary very significantly from location to location. So, Prediction method alternates the experimental approach to find permeability from measured soil properties. A relationship is developed to predict the permeability of soils from particle size distribution.

The performance of developed relationship to predict permeability compared with existing relationships. Many relationships only within a particular range of soil particle sizes. But, the newly developed relationship considers whole spectrum of particle size distribution.

The presented relationship of permeability with PSD is suggested as an alternative to laboratory analysis, especially for soils which are hard to prepare soil samples or may take long period to get the measurements of permeability. This relationship may also give first-hand information in certain circumstances in field.

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Salinity Dependent Mechanical Behaviour of Well Cement: An Experimental Study

B. Balinee and P. Disaanth
Department of Civil Engineering, University of Peradeniya, Sri Lanka

ABSTRACT: Integrity of injection well and well cement plays a vital role in successful Carbon Capturing and Storing (CCS) and oil/ gas extraction projects. Certainly, the required zonal isolation is given by the well cement. As usage of Ordinary Portland Cement (OPC) based well cement poses many problems,Geopolymer, which is an inorganic chain structure, can be used as alternativewell cement. In a typical storage reservoir, salinity varies from 0 - 40 % NaCl depending on the geological location. Therefore, aim of this research is to compare mechanical behaviour of geopolymer and class G cement under 0 %, 10 % and 20 % brine concentration after 28 days saturation. Based on the experimental findings, it was noticed that compressive strength of OPC increases with salinity up to 10 % NaCl and it reduces towards 20 % NaCl. On the other hand, compressive strength of geopolymer increases with salinity up to 20 % NaCl. On the whole, it is concluded that geopolymer can be used as an alternative for OPC in high saline environment.

1 INTRODUCTION

Global warming has evolved as a threatening issue in the current generation as well as for the future generation. Carbon dioxide (CO_2) plays a major role in greenhouse gas emission. Hence, reducing carbon dioxide emission is an essential and effective way to reduce global warming. It can be reduced by few means including improving the energy conversion efficiency of fossil fuels, shifting energy production to low carbon sources, enhancing uptake by terrestrial and marine biomass and capturing and storing CO_2 deep underground, according Bruantet al (2002) studies. Among these, Lecolier et al (2007) estimated that capturing and storing CO_2 deep underground is found as an effective way to reduce CO_2 emission with long term durable, safe and less cost. It is important to ensure cement integrity under various down-hole conditions. Well integrity and zonal isolation mainly depends on well cement. Under CO_2 rich environment, Portland cement is unstable and undergoes strength reduction, cement degradation, chemical attack and increase in permeability. On the other hand, it has been found by Giasuddin et al(2013) that geopolymer possess excellent acid resistant, high strength and low shrinkage compared to OPC. Geoploymer is an inorganic chain structure ranging from amorphous to semi crystalline which can be produced by adding alumino silicate and alkaline liquid in a certain proportion. Therefore, the intension of this research is to investigate fly ash based geopolymer as well cement to replace existing OPC cement. It was determined by Bruant et al (2002)that with depth, temperature increases with geothermal gradient of 30 °C/km and pore pressure increases at 10 MPa/km. Salinity of ground water table varies from location to location from 0 % to 40 %. Hence, it is essential to study the effect of salinity on mechanical behaviour of well cement.

Nasvi et al (2012) studied the mechanical behaviour of geopolymer saturated in two concentration of brine (5 % and 15 % NaCl). They observed that strength of geopolymer reduces with curing period and the strength reduction rate reduces with increase of brine concentration as high salt content has more resistance against alkali leaching. Gowthaman et al (2016) studied the mechanical behaviour of class G cement under different salinity level (0 %, 10 %, 20 % and 30 %) for different curing periods (7, 14, 28 and 45 days) at an average temperature of 50 °C. Based on the above study, it was concluded that strength of class G cement reduces with increase in salinity level due to the retardation of hydration and strength increases with curing period due to hydration process. Although, mechanical behaviour of fly ash based well cement and OPC based well cement were investigated under brine condition, there is no studies focusing on the comparison between OPC and geopolymer cement under same curing conditions. Therefore, this work aims to study the effect of salinity on the mechanical behaviour of geopolymer cement and compare it with class G well cement.

2 EXPERIMENTAL METHODOLOGY

Well cement samples were prepared by using sulphate resisting Ordinary Portland Cement (OPC) and fly ash. Both raw materials were purchased from Holcim Lanka Pvt (Ltd). The chemical composition OPC and fly ash used are given in Table 1.

Table 1. Chemical composition of API class G cement and fly ash

Chemical component	Fly ash (%)	API class G cement (%)
SiO_2	52.03	22.91
Al_2O_3	32.31	3.89
Fe_2O_3	7.04	4.75
CaO	5.55	64.70
MgO	1.30	1.80
SO_3	0.07	0.74
K_2O	0.68	0.64
Na_2O	1.00	0.10

Geopolymer samples were prepared by adding alkaline liquid (AL) with fly ash (FA) with an AL/FA ratio of 0.6. A combination of Na_2SiO_3 and NaOH was used as alkaline liquid and the ratio of Na_2SiO_3 and NaOH used in the mix was 2.5. High sulphate resistance cement paste were prepared with w/c of 0.44 (API recommended practice 10 B, for class G) as it has been found that this water cement ratio gives higher strength. Paste was cast in a PVC mould with dimension of 38 mm diameter and 76 mm height cylinder as shown in Figure 1.

Figure 1: Casted geopolymer samples

They were then oven cured at 50 °C, and during oven curing, top surface of samples were covered by polythene to avoid moisture loss. After 24 hours oven curing, samples were transferred to different brine concentration (0 %, 10 % and 20 %). The brine solutions were prepared by mixing NaCl salt with distilled water in mix proportions (BWOW). Samples were cured in brine for 28 days. To ensure reproducibility, three samples were tested for each condition. At the end of the curing period, Uniaxial Compressive Strength (UCS) test was conducted with a stress control loading rate of 0.2 MPa/sec.

Scanning Electron Microscopical (SEM) analysis was conducted at the Faculty of Science, University of Peradeniya, Sri Lanka. Micrographs were obtained using a ZEISS field-emission scanning electron microscope (FESEM). Aim of the SEM test was to trace any microstructural changes in OPC or geopolymer in brine water.

3 RESULTS AND DISCUSSION

3.1 *Variation of UCS and Young's modulus with salinity*

Comparison of compressive strength of geopolymer and OPC with salinity is shown in Figure 2. According Figure 2, it can be noticed that strength of geopolymer increases with salinity level at an increase rate of 9 % as high NaCl content gives more resistivity again weakening under brine water compared with fresh water. Initially strength of OPC increases with salinity level up to 10 % NaCl concentration with a rate of increase of 7 % and the reason for the increment is C-S-H absorbs the NaCl micro crystallites on the surface of the fibrous structure. Then strength reduces with salinity level (beyond 10 % NaCl) with a decrease rate of 22 % as high salt content retards hydration process. Dispersed NaCl in C-S-H and $Ca(OH)_2$ in the form of solid solution or micro-crystallite precipitation lowers the activation energy for decomposition of $Ca(OH)_2$ and causes retardation for hydration process. Therefore, the strength reduces with salinity beyond 10% NaCl salinity.

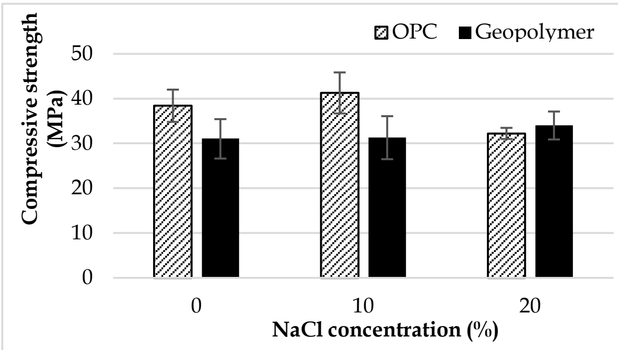


Figure 2: Comparison of compressive strength of OPC and geopolymer

Emilia et al (2009) presented that typical well cement should have a compressive strength of 20 to 50 MPa depending on depth to provide thermal isolation and to avoid mechanical fracturing. Considering these aspects, geopolymer can replace OPC based well cement as the strength of geopolymer fall within the recommended values when cured in saline water.

Average stress - strain behaviour of samples were obtained from compressive strength test. Figures 3 and 4 show the stress strain variation of OPC and geopolymer respectively, at different saline conditions. Based on Figures 3 and 4,Young's modulus values were calculated.Comparison of Young's modulus of geopolymer and OPC after 28 days is shown in Figure 5. It can be observed that Young's modulus of geopolymer increases up to 10 % NaCl concentration with a rate of increase 33.5 %. The introduction of NaCl reduces failure strain and increases Young's modulus. Beyond this range, Young's modulus decreases with a rate of decrease 17.1 % when the salinity is increased as stiffness of geopolymer reduces with high salt content. Young's modulus of OPC reduces with increase in salinity level at a rate of decrease 10.6 %. It means that stiffness decreases with the increase of NaCl content.

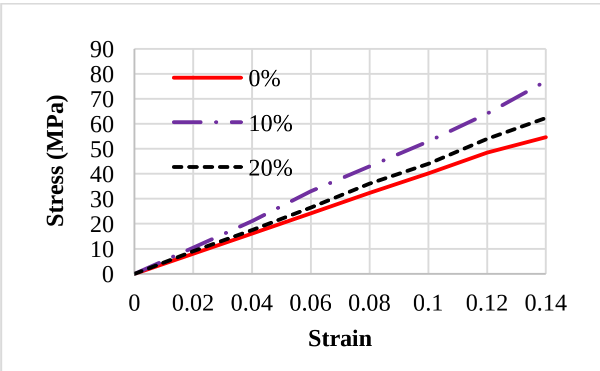


Figure 3: Stress- strain behaviour of OPC samples

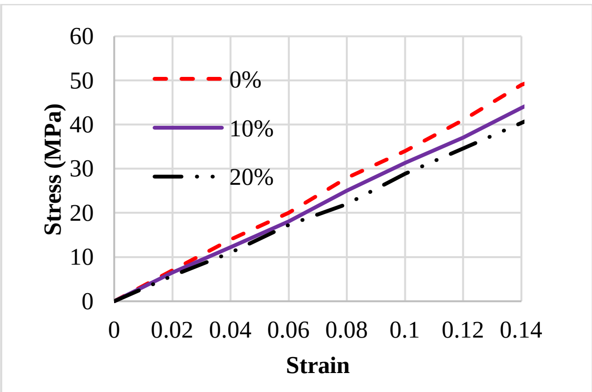


Figure 4: Stress- strain behaviour of geopolymer samples

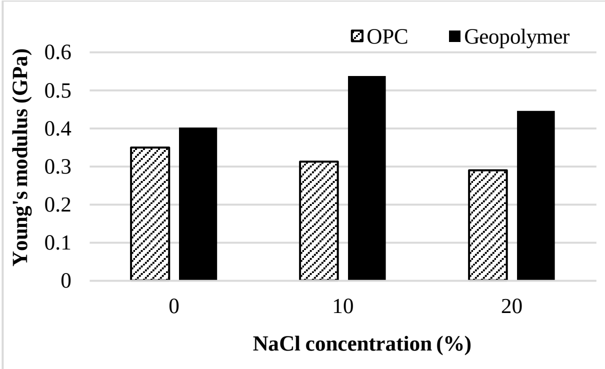
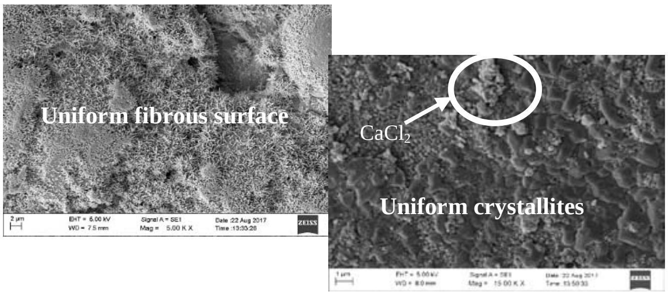


Figure 5: Comparison of Young's modulus of OPC and geopolymer

3.2 *SEM test results*

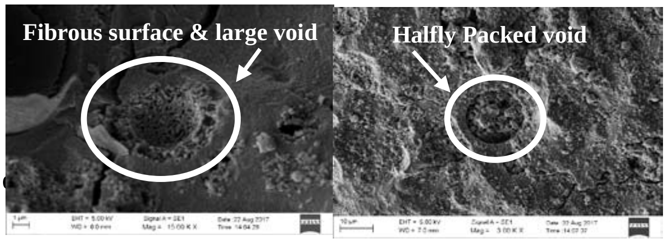
SEM analysis was conducted to identify the microstructural variation and appearance of the samples cured under different brine conditions. Various magnification factors were used to figure out the compositions; and the factors used were 1000, 3000, 5000, 12000 and 15000.

Figures6 and 7 show the SEM images of OPC and geopolymer, respectively, under different salinity conditions. In Figure 6-(a), uniform C-S-H fibrous surface, which is formed due to the hydration process, could be seen under water (0 % NaCl). With the introduction of brine solution, NaCl microcrystallites are absorbed on the fibrous surface (Figure 6-(b)). As a result of it, compressive strength of OPC initially increases. In continuous curing in brine solution, Ca^{2+} ions in Calcium Silica Hydrate (C-S-H) are deposited with Cl^- ions as $CaCl_2$ crystals. It affects the formation of $Ca(OH)_2$. Due to the retardation of degree of hydration and the degradation of C-S-H, strength reduces with salinity level. It was ensured that the white colour crystal forms in Figure 6- (b) are $CaCl_2$ solid deposits. Based on the microstructure of the geopolymer, it is more visible that particles are packed loosely and large voids and fibrous surface (Figure 7-(a)) appear in the structure under water (0 % NaCl). With the increase of salinity level, voids were covered by the geopolymeric products and geopolymer becomes relatively compacted. These scenarios could be seen through Figures7-(a) and 7-(b). As it becomes denser through the increase of salinity, geopolymer gains compressive strength with the increase in salinity.



(a) (b)

Figure 6: Variation of microstructure of OPC: (a) 0 % NaCl and (b) 10 % NaCl



(a) (b)

Figure 7: Variation of microstructure of geopolymer: (a) 0 % NaCl and (b) 10 % NaCl

4 CONCLUSIONS

Compressive strength of OPC initially increases as the NaCl micro crystallites are absorbed by the C-S-H fibrous surface and then it decreases as the high salt content retards hydration. On the other hand, the compressive strength of geopolymer increases with salinity level due to high resistance against alkali leaching in high salt content. Young’s modulus of OPC samples reduces with salinity level, leading to reduced stiffness in brine. Introduction of salt content up to 10% NaCl increases the failure strain in geopolymer and increases Young’s modulus.

However, a further increase of salt content (> 10% NaCl) cause loss of stiffness and reduces the Young’s modulus of geopolymer. When comparing the compressive strength of OPC and geopolymer in high salinity medium, geopolymer gains more compressive strength than the OPC, and geopolymer becomes stiffer than OPC at high salt content concentration. Based on this research outcome, it is concluded that geopolymers can replace existing OPC based cement, especially under high salinity conditions.

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Applicability of Mackintosh Probe Test in Evaluating the Degree of Compaction in Compacted Quarry-dust Backfills

P.A.M. Gomes
Department of Civil Engineering University of Moratuwa, Sri lanka.

ABSTRACT: Mackintosh Probe is a light weight portable penetrometer. It can be handled easily and considerably faster and cheaper. This study aims in investigating the applicability of Mackintosh probe in evaluating degree of compaction in compacted backfills. The depth of exploration should be moderate and soil grading should be monolithic. Experiment was done on uniformly compacted quarry dust backfills. Correlations are developed between Mackintosh Probe result, moisture content and dry density.

1 INTRODUCTION

Backfill compaction evaluation is very critical in construction industry. Backfilling should be reached its required dry density to ensure the adequate bearing capacity. Methods using for degree of compaction evaluation is not optimized in present construction sequence ex: sand cone test. In this research ultimate target is to assess the degree of compaction through Mackintosh probe test.

1.1 *Mackintosh Probe*

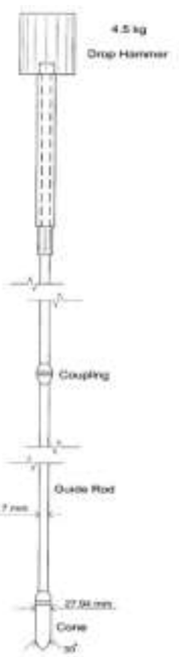

Figure 01 set-up and dimensions of mackintosh probe

The mackintosh Probe is consists of 27.94 mm diameter cone with 30 apex angle, 12.7 mm diameter solid rods and 4.5 kg dead weight with a drop height of 300 mm.

It is a light weight and a portable tool. The cone is penetrated to the soil while blowing the dead weight. Number of blow to penetrate 300 mm is counted as M (Mackintosh Probe blow count)

Research based finding relates to Mackintosh Probe

It is initially developed for investigation of peat. Then it has been used for varieties of soils.

- *Bearing Capacity of soil*

P= (2860 + 550 (M- 40)1/2) x 0.04788 kN/m
(Fakher, Khodaparast, & Jones, 2006)

- *Correlation between Sandard penetration test and mackintosh probe test (for soft soil)*

N= $0.16\ logM^{0.91}$ (Sabtan & Shehata, 1994)

- *Relationship between penetration resistance and relative density (for clean sand)*

$N = \left(17 + 24\frac{\sigma'v}{98}\right)$ Dr^2 (Meyerhof 1957)

N is the SPT blow count,
σ'*v* is the effective overburden pressure in kPa
Dr is the relative density

1.2 *Objectives*

- Applicability of Mackintosh in evaluating the degree of compaction of compacted quarry-dust.
- Develop a relationship between dry density and mackintosh blow count
- Recognizing how the moisture content going to affect the results

1.3 *Methodology*

- Initial soil parameters identification

- Maximum dry density and optimum moisture content were find out through proctor compaction test.
- Dry sieving was carried out to identify the particle size distribution of the soil specimen, to ensure that the specimen is belongs to the category sand.

❖ Research experiments

- Moisture content of the soil sample was fixed(starting with lower value of moisture content)
- Mould was selected in which the soil volume can be measures by the outer dimensions
- Soil was compacted in the mould layer by layer while applying same energy for each layer
- Soil was compacted more than 300mm height
- Dry density of the sample was found out
- Mackintosh Probe test was carried out on the soil specimen and found out the blow count to penetrate 300mm
- While increasing the compaction energy test was carried out to obtain different dry densities

2 SOIL PARAMETERS IDENTIFICATION

2.1 *Observation of maximum dry density and optimum moisture content*

Standard proctor compaction test was carried out. Soil was compacted in the standard mould 101.6mm diameter,115.4 mm height, volume 944 cm^3) in three layers by dropping 2.49 kg rammer with a drop height of 305 mm. 25 blows were applied on each layer.

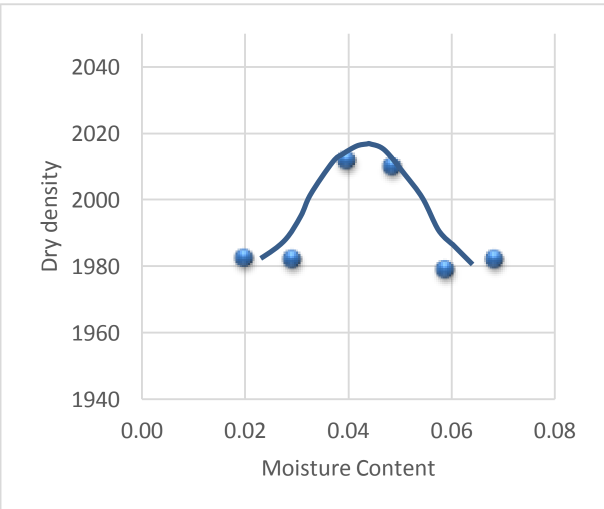

Figure 02 Dry density vs moisture content

Maximum dry density = 2015kg/m^3
Optimum moisture content = 4.4%

2.2 *Observation of particle size distribution*

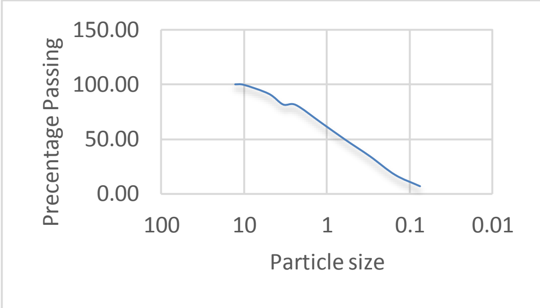

Figure 03 Particle size distribution

D10 = 0.1mm
D50 = 0.7 mm
D90 = 4.8 mm

Cu = 8.9
Cc = 0.8

3 RESEARCH EXPERIMENTS METHODOLOGY

3.1 *Moisture content controlling*

Initially quarry dust was completely air dried. Before starting the experiments moisture content of the sample was measured using an electric hot plate. Measured 80kg of dry soil and adjust the moisture content to the required value by adding the additional amount of water which is further required and proper mixing was done.

Moisture content was measured several times while the experiments are going on and checked moisture content changes due to evaporation. And adjustments was done.

3.2 *Soil compaction*

Apparatus

Figure 04 Compaction mould and rammer

Dimensions of the compaction mould

Width =*300mm*
Length =*300mm*
Height =*600mm*

Dimensions of the dropping Rammer

Width =*250mm*
Length =*250mm*
Weight =*8kg*

Figure 05 Poker vibrator

Compaction of completely air dried soil

The approximate moisture content of completely air dried quarry dust is at around 0.15% it cannot be compacted using a dropping rammer. It was compacted using the poker vibrator. Different dry densities were obtain from small values to large values while changing the compaction time and compaction poker pattern.

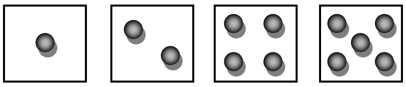

Figure 06 Poker vibrator patterns

Time = 30sec, 60sec, 90sec, 120sec, 150sec

Compaction of moist soil

Moist quarry dust cannot be compacted using the poker vibrator. The vibration does not transfer through soil.

It was compacted layer by layer while applying same compaction energy for the each layer. Different dry densities were obtained from small values to large values by applying different compaction energy.

Compaction was done while changing the initial layer thickness, rammer drop height, number of rammer blows.

Table 01 Changing parameter to apply different compaction energy

Layer thickness	Drop height	Number of Blows
150 mm	100 mm	2
100 mm	150 mm	5
50 mm	200 mm	10
25mm	250 mm	15,20,25,30

Different combinations of layer thickness, drop height, number of blows are used to obtain different soil densities.

Soil was compacted up to a height of approximately 450mm to provide an adequate penetration to the mackintosh probe.

3.3 *Measurement of dry density and Mackintosh probe test*

Calculation of dry density

Wet soil weight is measured before it is compacted in the mould. The cross section area of the mould is known 0.09 m^3. After the compaction height of the soil was measure from all four sides and got the average value.

Soil volume was calculated and obtained the bulk density of quarry dust.

Dry density was measured for the specified moisture content.

Dry Density = Bulk Density / (1 + w)

Mackintosh Probe test

Mackintosh probe instrument was kept on the compacted soil and drop weight blows were applied on the soil to penetrate 300mm. blow count is obtain just to pass 300mm mark.

4 RESULTS AND DISCUSSION

4.1 *Observed relationship of mackintosh blow count with respect to the dry density*

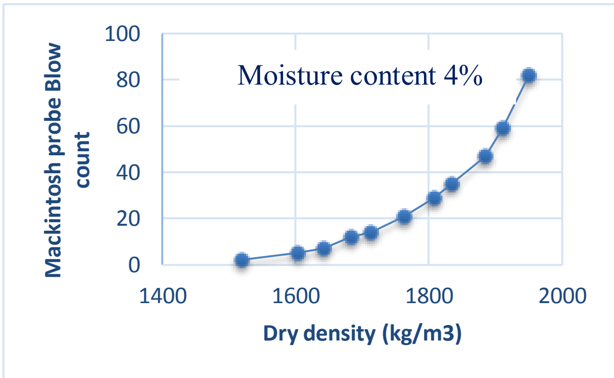

Figure 07 Behavior of MP blow count with respect to Dry density

4.2 *Observations for different moisture contents*

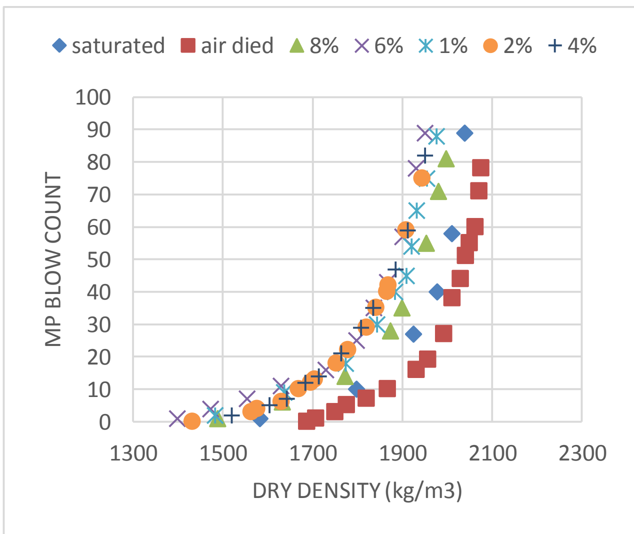

Figure 08 Results for different moisture contents

The MP value behaves exponentially with respect to the dry density to a specified moisture content. Starting from less moisture (0.15%) content to moderate moisture content at around 6% the graph shifts towards y axis. When the moisture content increases beyond 6% graph again tends to move away from the y axis.

For a fixed moisture content it indicates maximum MP blow count falls on the optimum moisture content.

4.3 *Arrangement of the design graph*

Final target of the research is to find out the dry density of compacted backfill through the MP blow count.

Data (MP blows vs MC) has been arranged graphically for different dry density. It views just like the output of proctor compaction test maximum MP blow count falls on the optimum moisture content

Graphs are parallel and similar in shape for different dry densities. Graph rises upward when dry density increases.

In the in-situ condition dry density evaluation can be easily done by marking the point which is represents the moisture content and mackintosh probe blow count. Parallel line can be drawn to incorporated density lines. Existing dry density can be calculated by interpolation.

Design graph

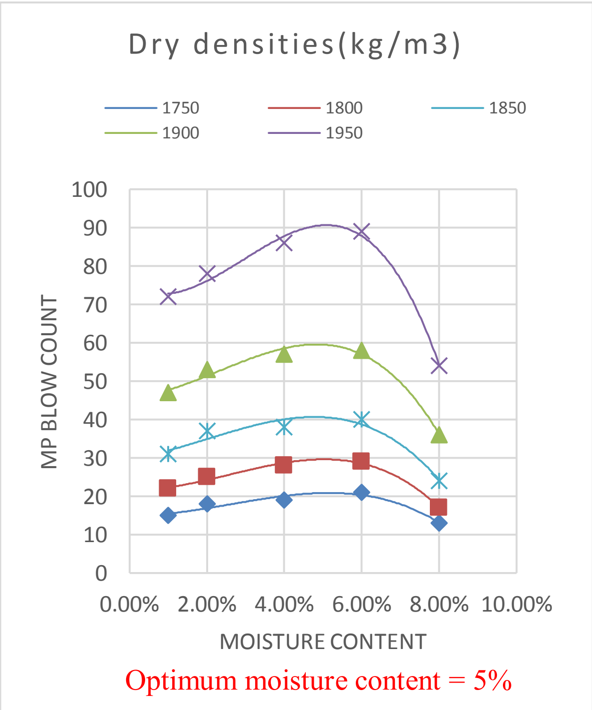

Figure 09 Design graph

5 CONCLUTION

Mackintosh probe is a light weight device, which can be used for investigate and evaluate the bearing capacity of soft soil.

In this research it is discussed that the applicability of mackintosh probe in evaluating degree of compaction in compacted quarry dust backfills. It is efficient to monitor soil dry density through mackintosh probe. This can be done for compacted backfill in which moisture content is known. After find out the mackintosh probe blow count using the design graph dry density can be calculated.

It comparatively better to calculate dry density for shallow depth for compacted soil because

- Less time consuming compared to sand cone apparatus
- No need to get soil bulk weight and volume to calculate the bulk density
- Possibility to reach locations where cannot be reached with sand cone apparatus (ex: Closer to pile caps and ground beams)

6 ACKNOLEDGEMENT

I would like to give my heartiest thanks to my research supervisor Dr. Nalin De silva, all other staff members in the Civil Engineering department and the technical staff of the geotechnical and building materials laboratory for the support given to this research.

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Secondary Consolidation Characteristics of Peaty Clay

J.P.D.C.M. Karunarathne
Department of Civil Engineering, University of Moratuwa, Sri Lanka

ABSTRACT: Peaty Clay present in flood plains of major rivers are with very high water contents, high compressibility and very low shear strength. Numerous ground improvement techniques are to be adopted in constructions done in sites underlain by peaty clay to prevent shear failures during the construction and to ensure the settlements in the final structures are within acceptable limits. Pre consolidation by preloading is one such technique adopted to minimize settlements within the service period in roads done on peaty clay. Peaty clays experience very high secondary consolidation and special attention should be paid to minimize secondary consolidation settlements during service. Previous studies done with Oedometer has shown that reduction of the coefficient of secondary consolidation depends on the over consolidation ratio achieved during preloading. In this research tests are done with Rowe Cell while measuring pore water pressure and identifying the secondary consolidation with the objective of comparing with the results obtained with the Oedometer.

1. INTRODUCTION

Peat is formed by decomposition of vegetation under anaerobic conditions. In the initial stages of decomposition it would be quite fibrous. Thereafter it would be turned into an amorphous stage through a fine fibrous stage. When peat is mixed with deposits of alluvial soil it would be termed peaty clay. Thick deposits of Peaty Clay are encountered in flood plains of Sri Lankan rivers. The organic contents of theses soils were found to be of the order of 40% normally. Peaty clay deposits are with water content of the order of 300-500% and are of very low shear strength and very high compressibility. Major infrastructure development projects such as Colombo-Katunayaka expressway and Southern Expressway were done though areas underlain by such soils. Peaty Clay exhibits high compressibility and very low shear strength.

Numerous ground improvement techniques are to be adopted in these projects to prevent shear failures during the construction and to ensure the settlements in the final structures are within acceptable limits. Peaty clays experience very high secondary consolidation and special attention should be paid to minimize secondary consolidation settlements during service. Pre consolidation by preloading is one such technique adopted to ensure that settlements within the service period are within acceptable limits. Studies done with Oedometer (Kulathilaka 2007, Fernando and Kulathilaka 2007) have shown that reduction of the coefficient of secondary consolidation is related to the over consolidation ratio achieved during preloading. In those tests loading increments of 3 days duration were used due to the very high secondary consolidation in peaty clay the change from primary to secondary consolidation could not be identified. Therefore in each increment which is of 3 days duration considerable amount secondary consolidation have also taken place.

2. METHODOLOGY

In this research in addition to Oedometer, consolidation tests were conducted in Rowe Cells with identical samples. In all tests remoulded samples of peaty clay were used. Within a natural deposit of peaty clay there are intrusions such as pieces of stones, non-decayed timber etc. and conditions are highly non uniform. As such, in a study of this nature it is necessary to prepare a large sample of uniform material from which to obtain specimen for tests at different stages. Mass samples of Peaty clay obtained from the site were mixed for a fixed time with a hand mixer after removal of non-decayed pieces of wood and other impurities such as large gravel and leafs.

Peaty clay for this research was obtained from the Outer Circular highway project. Organic content and water content of this Peaty Clay is quite high compared to other sites as presented in Table 1. Water content was determined by keeping the peaty clay in an oven at a maximum temperature of 60^{o} and verified by keeping a sample at room temperature for about a week until there is no weight loss.

Table 1. Properties of peaty clay used

Properties	Value
Water content (%)	1170
Organic content (%)	74.9
Specific gravity (%)	0.7

Oedometer used in this study was of 20mm in height and 50mm in diameter. Mechanical vertical load was applied to the sample. Two way drainage was permitted through porous disks at the top and the bottom and sample was kept as saturated using the water bath. Only the Settlement can be measured during the Consolidation.

Rowe cell (Rowe and Barden 1966) overcomes many limitations of the conventional Oedometer. Here the samples are loaded hydraulically by water pressure acting on the flexible diaphragm. Therefore pressure distribution is uniform throughout the sample. Large diameter samples can be used making it more representative. Further the drainage can be controlled and pore water pressure can be measured. The Rowe Cell used in this research was with 150mm in diameter and 50mm in height and drainage was only from the top measuring the pore water pressure at the bottom. Rowe cell and pressure transducer was calibrated by application of hydraulic pressure after filling with water.

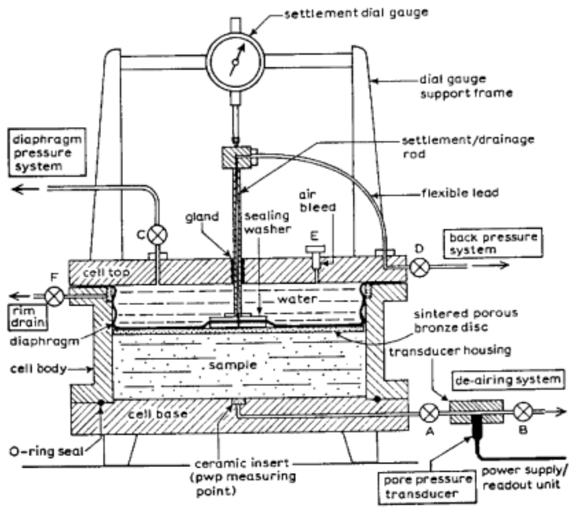

Fig.1 Rowe Cell setup used

3. TEST PROGRAM

Consolidation tests were carried out on remoulded specimen of same water content prepared to the same density in both Oedometer and Rowe Cell. Tests were done with, Loading, unloading and reloading increments. Reloading increments were to simulate the behavior of peaty clay that had been preloaded. Loading steps were decided to provide different over consolidation ratios. Three day loading increments were used for the testing to clearly identify the secondary consolidation phase.

The loading sequence used was 20kN/m^2-40-80-160kN/m^2. Then unloaded through 80kN/m^2- 40 to 20kN/m^2. Thereafter sample was reloaded through 40 kN/m^2, 80, 100, 120, 140, 160 finally to 320kNm^2. These loading steps were used to obtain over consolidation ratios such as 2, 1.5, 1.3 and 1.2.

4. TEST RESULTS AND ANALYSIS

Results of both Oedometer and Rowe Cell tests are presented here. A plot of void ratio vs time for Oedometer test load increment 40-80 kN/m^2 is presented in Figure 2. The reduction of the gradient of the curve after completion of Primary Consolidation usually seen in inorganic soils is not noted here. This is due to high Secondary Consolidation of Peaty Clays.

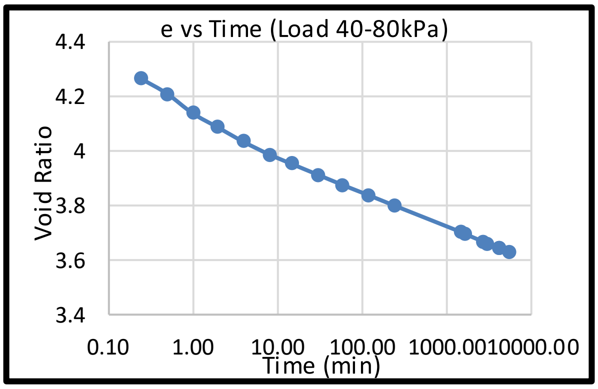

Fig.2 Void ratio variation with time (Oedometer test)

The variation of Void ratio vs time and pore water pressure vs log time obtained from the Rowe cell for the same load increment (40-80kN/m^2) is presented in Figure 3.

The pore water pressure dissipation and accompanying settlement can be clearly identified in Figure 3. Then the starting point of secondary consolidation of the sample can be found. Continuation of settlement even after the dissipation of pore water pressure to a small value is quite evident here. In contrast to the Oedometer sample a slight reduction in the gradient can be identified at the later stages of the thicker Rowe cell sample. From the latter part of the Rowe cell settlement plot the Coefficient of Secondary Consolidation can be calculated. Another observation is that the full increase of the pore water pressure (40kN/m^2) due the increase of stress from 40-80kN/m^2 has not been recorded at the pore pressure transducer which is at the bottom of the sample. Similar observations were made in other load increments also. This may be due to a time lag but need to be further investigated.

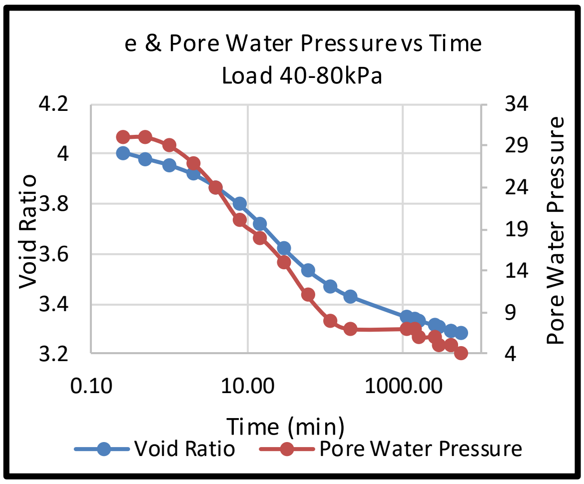

Fig.3 Void ratio and pore water pressure variation with time (Rowe Cell)

The void ratio Vs time plots for some load increments in Oedometer and Rowe Cell are compared in Figure 4. Some differences in the behavior of two specimen could be noted there. The Rowe Cell samples shows a slight reduction in the gradient of the latter part of the curve. This is not seen in Oedometer samples. Also the final void ratios at the end of the increment are slightly different.

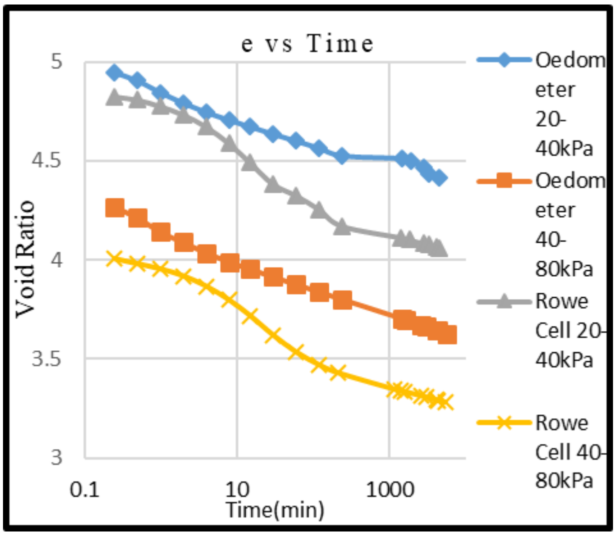

Fig.4Void ratio variation with time (Oedometer and Rowe Cell)

The e vs log (σ) plots for the Oedometer sample and Rowe Cell are presented in Figure 5 and Figure 6 respectively. Using the gradient of void ratio vs log (σ) graph, Compression index of the peat clay samples can be calculated. The values obtained are summarized in Table 2.

Table 2. Compression Index Values

Test	**Cc value**
Oedometer	2.4
Rowe Cell	2.4

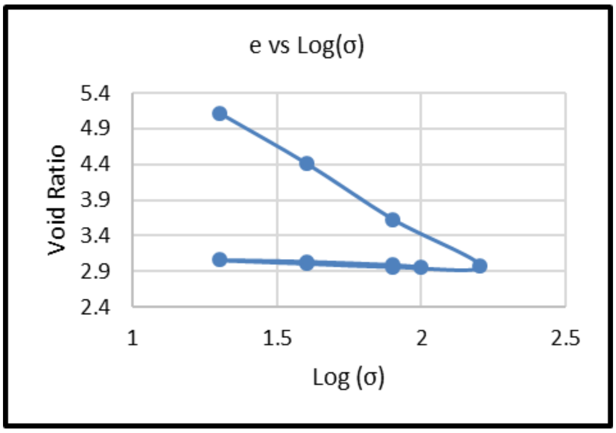

Fig.5Void ratio variation with log (σ) (Oedometer test)

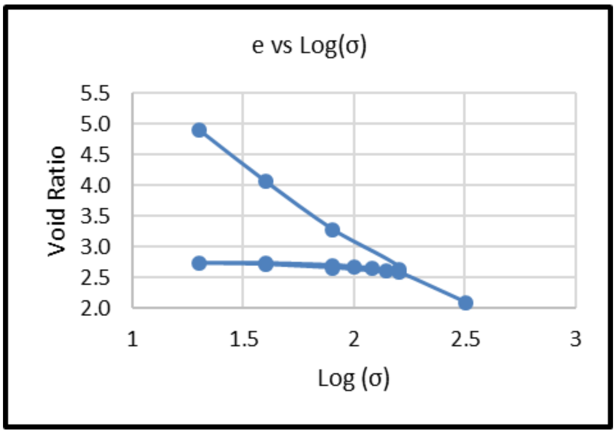

Fig.6Void ratio variation with log (σ) (Rowe Cell test)

Using the void ratio Vs log (time) plots of both samples C_α values were calculated for the loading and reloading increments. The values are presented in Figure 7.

According to the Figure 7 both Rowe Cell and Oedometer tests results show much lower values for C_α in the reloading increments compared to respective loading increments. This is illustrative of the influence of pre consolidation effect in reducing the secondary consolidation. In reloading increment the over consolidation ratio decreases with the loading increment and an increase of the coefficient of secondary consolidation could be observed.

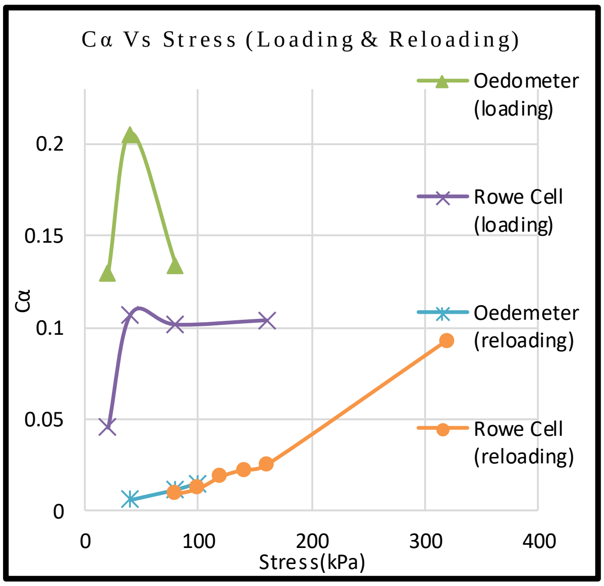

Fig.7Cα variation with stress for loading and reloading

Thereafter the ratio of coefficient of secondary consolidation values in loading (C_{α}) and reloading (C'_{α}) increments were computed and plotted against the over consolidation ratio (Figure 8). The trend shown by both the Oedometer and Rowe Cell samples were similar.

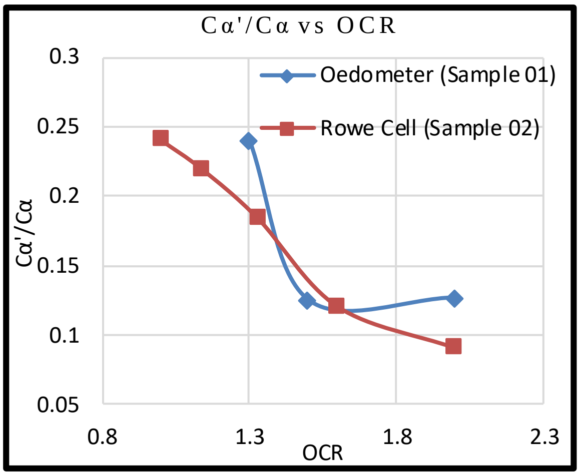

Fig.8 C'_{α} / C_{α} variation with Over Consolidation Ratio

Following the work by Mesri et al (1997) and Mesri (2005) C_{α}/C_c values were estimated for both setups in both loading and reloading stages. In re-loading increments C_r value was used instead of C_C

The ratio of $C\alpha/C_c$ values estimated using the $C\alpha$ values in both loading and reloading increments are summarized in Table 3 & Table 4.

Table 3- C_{α}/C_c comparison (Oedometer)

Increment kN/m^2	loading	Increment kN/m^2	reloading
0-20	0.0542	20-40	0.0352
20-40	0.0857	40-80	0.0517
40-80	0.0558	80-100	0.0529

Table 4- C_{α}/C_c comparison (Rowe Cell)

Increment kN/m^2	loading	Increment kN/m^2	reloading
0-20	0.0189	40-80	0.0389
20-40	0.0446	80-100	0.0518
40-80	0.0422	100-120	0.0795
80-160	0.0432	120-140	0.0954
		140-160	0.1059

5. SUMMARY AND CONCLUSION

The effect of preloading on the coefficient of secondary consolidations was evaluated using both the Oedometer and Rowe Cell with loading, unloading and reloading increment. The results were similar to some extent. Oedometer increments did not clearly showed the end of primary consolidation. But it was evident in Rowe Cell from the dissipation of excess pwp. Also, there was a reduction in the gradient of the settlement graph. Also Rowe Cell sample size is larger than Oedometer sample size would be more representative. Results than Oedometer. In both tests each load increment included significant amount of secondary consolidation. Further tests could be conducted in Rowe Cell by increasing the load soon after the dissipation of pore water pressures.

ACKNOWLEDGMENT

My heartiest gratitude goes to Prof. S.A.S. Kulathilaka for his immense support and guidance throughout the research, Dr. L.I.N.de. Silva for valuable advices during the experimental work and to laboratory staff.

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Investigation on the Applicability of Plate Load Tests in Extrapolating the Bearing Capacity of Large Raft Foundations

O.M.Muthuhewa
Undergraduate, Department of Civil Engineering, University of Moratuwa, Sri Lanka

ABSTRACT: The plate bearing test is usually performed on relatively small size plates such as 1-2 ft^2, because using smaller sized plates is economically more feasible. However, the influence zone of a raft foundation is comparably very larger than the influence zone of plate load test. Therefore, this study is aimed at investigating the applicability of extrapolating the bearing capacity from plate load test results for large raft foundations. In this research, the extrapolated results of bearing capacity of plate load test results have been compared with results of bearing capacity from a theoretical method and computer analysis. Plate load test results have been extrapolated by using two methods called, Tangent-Intersection method and an equation method related to Terzaghi and Peck (1967) method. Theoretical results of bearing capacity of raft foundation have been obtained by using coefficient of volume compressibility. Computer analysis has been done by Plaxis 2D software.

1 BACKGROUND

1.1 *General*

Almost all large structures such as buildings, dams, bridges are located on the soil and loads of them finally transfer to the soil through foundations.

1.2 *Raft foundations*

Raft foundations are large footings that cover entire contact area of the superstructure and soil. These are concrete slabs on the soil.The most important benefit of using raft foundations is reducing differential settlement which causes serious cracks in the superstructure.

1.3 *Plate load test*

Plate Load Test is the most common in-situ test to determine the bearing capacity of soil. According to the American Society for Testing and Materials (ASTM) under Designation D-1194 (ASTM, 1997), circular steel bearing plate with the diameter from 162 mm to 760 mm or square steel plate with the size of 305 mm × 305 mm is used for this test. In Sri Lanka, 300 mm in diameter and 25 mm thick circular steel plates are frequently used for this test.

Generally, the influence zone of plate load test is very small comparing with the influence zone of a raft foundation. Therefore, the extrapolating the results of plate load test for raft foundations is debatable matter in foundation engineering.

Therefore, this research has been conducted to investigate the applicability of extrapolating the bearing capacity of plate bearing test for large raft foundations.

2 LITERATURE REVIEW

2.1 *Terzaghi and Peck (1967)*

Terzaghi and Peck (Terzaghi & Peck, 1967) have proposed a method to extrapolate the plate load test results by using size of plate and footing. The following equation is proposed by them.

$$\delta_{footing} = \delta_{plate} \left(\frac{2B_{footing}}{B_{plate}+B_{footing}}\right)^2 \quad (1)$$

Where, $\delta_{footing}$ is the settlement of the footing,δ_{plate} is the settlement of the plate, $B_{footing}$ isthe diameter of the footing and B_{plate} is the diameter of the plate.

2.2 *Trautmann and Kulhawy (1988)*

Truatmann and Kulhawy (Trautmann & Kulhawy, 1988) have proposed a method to extrapolate plate load test results by graphical method. Load settlement curve is plotted with applied stress in x-axis and corresponding settlement in y-axis. Generally, there is a clear point where the curve has sudden drop down. That point has been identified as the point of failure. It can be identified by the tangent lines drawn from straight portions of beginning and end part of the curve. The applied stress at the point of intersection of those tangent lines has been identified as the ultimate bearing capacity of soil.

2.3 *Priyantha (2009)*

Priyantha (Priyantha, 2009) has conducted a research on "Extrapolation of Plate Load Test Results to Foundations on Sand". Under this research, investigation of the extrapolation of the plate load test results in homogeneous and layered granular soil has been compared with the estimation of the finite element method and a reliable method to estimate the settlement of a foundation on layered soil has been developed using plate load test results. In this study a finite element model has been developed to simulate the plate load test and the loading of the actual footing. The verification of the results obtained from that model and other methods for settlement have been compared with field measurements.

According to this study, the settlement for small foundations in soft soils by Terzaghi and Peck (Terzaghi & Peck, 1967) method has shown overestimated values. When foundation diameter is increasing, the results have been under-predicted. Further, when foundation diameter is increasing, the variation of the results between finite element method and other methods has been gradually become linear. As well as, the results of the parametric study has shown the extrapolation methods has under-predicted the settlement of footing.

3 METHODOLOGY

3.1 *General*

This study has been conducted in mainly three phases. First phase is that plate load test results of an actual site data were extrapolated. Actual data has been taken from the proposed Waste Water Disposal Project at Polwatta near the Gannoruwa Research Centre, Gatambe, Kandy. Plate Load Test have been performed with 300 mm diameter plate. Subsoil condition of the site which is related to all Plate Load Test has been identified as uniform deposit clayey sand. The actual data have been taken from three proposed raft foundations in this site. In this study, those three raft foundations have been represented as three cases as follows.

Case 1: Oxidation Ditch 3&4
Case 2: Sedimentation Tank 1
Case 3: Sedimentation Tank 4

In the first phase of this study, those Plate Load Test results have been extrapolated by using two methods, Tangent-Intersection Method and Equation Method. Second and third phases are based on analysed results of Standard Penetration Tests (SPT) carried out for the site mentioned above. According to the SPT results analysis, the consultant has been recommended raft foundations for the different structures of this project. By using those results, Allowable bearing capacities of raft foundations of different locations have been analysed using a theoretical method by using coefficient of volume compressibility in the second phase.

In third phase, allowable bearing capacities of above raft foundations have been analysed by Plaxis 2D. Finally, extrapolated Plate Load Test values have been compared with theoretical values and results from software analysis.

3.2 *Phase 1: Extrapolation of plate load test results*

In this phase, the graphs have been plotted "Average Displacement vs Vertical Stress" by using plate load test result.

3.2.1 *Tangent-Intersection method*

This method is based on the method proposed by Trautmann and Kulhawy (Trautmann & Kulhawy, 1988). This method has been described in section 2.2.

3.2.2 *Equation method*

In this method, first, the settlement of the plate related to 50 mm settlement of the raft foundation has been found by using the equation 3.1 below. This equation is based on the method proposed by Terzaghi and Peck (Terzaghi & Peck, 1967) is section 2.1.

$$\delta_{plate} = \frac{\delta_{footing}}{\left(\frac{2B_{footing}}{B_{plate}+B_{footing}}\right)^2} (2)$$

Where, $\delta_{footing}$ is the settlement of raft foundation (50mm), $B_{footing}$ is the width of raft foundations (25m) and B_{plate} is the diameter of the plate (300mm).

The corresponding vertical stress for this settlement of plate has been taken by the graph of Average Displacement vs Vertical Stress.

3.3 *Phase 2: Allowable Bearing Capacity of raft foundation by a theoretical method*

3.3.1 *Allowable bearing capacity by using coefficient of volume compressibility*

Allowable bearing capacities of raft foundations have been obtained by considering the allowable settlement of raft foundations as 50 mm. The following equation has been used for this method.

$$\Delta\sigma' = \frac{\delta}{m_v H} \quad (3)$$

Where, $\Delta\sigma'$ is the effective stress, δ is the settlement of the soil layer, m_v is the volume compressibility of soil and H is the thickness of the soil layer.

The effective stress has been taken as the allowable bearing capacity of raft foundation.

3.4 *Allowable bearing capacity of raft foundation by Plaxis 2D*

Plaxis 2D software has been used to simulate the actual site conditions. Assumptions for this simulation are that all soil layers and water table are horizontal, width of raft foundation is 25 m andinfluence depth of raft foundation is two times the width of raft foundation. In the simulation, 50 mm prescribed displacement has been given to the raft foundation as allowable settlement. From output, the effective stress y-y just below the raft foundation has been considered as the allowable bearing capacity of the raft foundation.

4 RESULTS AND DISCUSSION

4.1 *Extrapolation of plate load test result*

The following results have been obtained from Tangent-Intersection method and Equation method. The chart shown in is the extrapolating the PLT results of Case 1: Oxidation Ditch 3&4. Likewise, the charts have been prepared for other two case and summary of the extrapolation of plate load test results have been presented in Table 1.

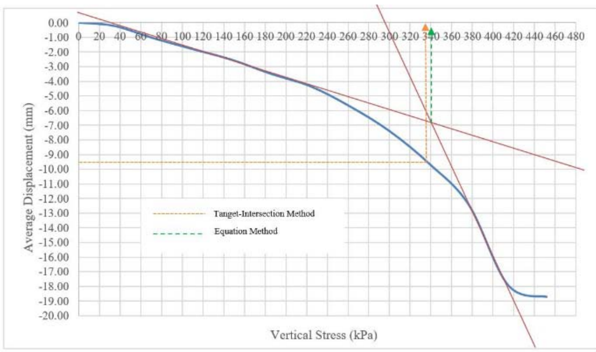

Figure 1 : Extrapolating the PLT results of Oxidation Ditch 3&4

Table 1: Allowable bearing capacities by extrapolating plate load test results

Case	Allowablebearing capacity(kN/m^2)	
	Tangent-Intersection Method	Equation Method
Case 1	340	335
Case 2	445	360
Case 3	296	240

According to results, both Tangent-Intersection method and equation methodshow approximately equal values.

4.2 *Allowable bearing capacities by a theoretical method*

Results of allowable bearing capacities of raft foundations by using coefficient of volume compressibility have been shown inTable 2.

Table 2 : Allowable bearing capacities of raft foundations by using a theoretical method

	Allowable bearing capacity of raft foundation (kN/m^2)
Case 1	232.6
Case 2	229.9
Case 3	151.8

According to the results, the bearing capacities by extrapolating plate load test results are vary larger than bearing capacities found by using this theoretical method

4.3 *Plaxis 2D results*

Vertical effective stress distribution for Case 1 is shown in Figure 3while vertical effective stress diagram just below the raft foundation is shown in Figure 3

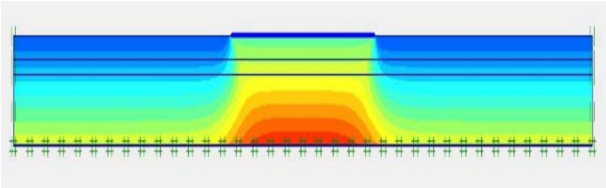

Figure 3: Vertical effective stress diagram of Oxidation Ditch 3&4

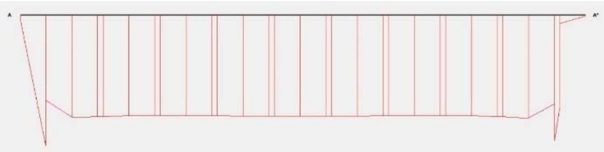

Figure 2 : Vertical effective stress distribution for raft foundation of Oxidation Ditch 3&4

Likewise, Plaxis 2D analysis has been carried out for other two cases and summary of the software analysis has been presented in Table 1.

Table 3 : Summary of Plaxis 2D analysis

Case	Allowable bearing capacity of Raft Foundation (kN/m2)
Case 1	141.8
Case 2	146.7
Case 3	160.8

According to the results,the bearing capacities by extrapolating plate load test results are comparatively larger than software analysis method.

5 CONCLUSION

Because plate load test is commonly used as an in-situ test to obtain the bearing capacity of soil, the investigation on the applicability of extrapolation of plate load test results to find the bearing capacity of raft foundation is very important.

In this study, extrapolation of plate load test results have been conducted by two methods namely, Tangent-Intersection method and equation method. Using results obtained from Standard Penetration Test, the allowable bearing capacities of each case have been calculated by a theoretical methods and using Plaxis 2D analysis.

According to this study, the results for bearing capacities from the theoretical method and software analysis method show comparatively larger values than the results by extrapolating the plate load test results in both methods of extrapolating.

In this study, the influence zone of plate load test is within the top soil layers in every cases and the top soil layer of every case is consist of sandy soil. Therefore the extrapolating results have not been indicated the whole soil layer system.

Therefore, It is shown that the extrapolating plate load test results to obtain the bearing capacity for large raft foundation is an over conservative method with respect to theoretical method using volume compressibility and software analysis method using Plaxis 2D.

ACKNOWLEDGMENT

We are thankful to JFE Engineering Corporation (Pvt.) Ltdfor providing us the required geotechnical data for this study.

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The Compressibility Behavior of Stabilized Peat with Fly Ash using DMM: An Experimental and Numerical Study

M. Nithurshan and R. Nitharshan
Department of Civil Engineering, University of Peradeniya, Sri Lanka

ABSTRACT: The paper presents the findings of a study into the engineering properties of a peat from western coastal area of the Sri Lanka that was mixed with Fly ash (class F) to form a stabilized soil. The study comprised an investigation of the settlement over the time achieved when mixed with Fly ash. A stabilized structure (i.e., stabilized column) was tested in a Rowe cell apparatus. The study showed that the compressibility behavior of peat was considerably improved when mixed with fly ash. The formation of the stabilized column within a Rowe cell chamber significantly reduced the amount of settlement when compared with that interpreted for the untreated soil, and the rate of consolidation was accelerated. A finite element analysis of the recorded behavior in the Rowe cell testing apparatus showed good agreement between the simulated and the experimental behavior.

1 INTRODUCTION

Peat is a type of soil made up of partially disintegrated plant and organic matters under conditions of incomplete aeration and high – water content (Huat et al, 2011). It has low bearing capacity, low specific gravity, medium to low permeability, high compressibility, high natural water content, high water holding capacity and high rates of creep.In Sri Lanka, 2500 hectares of land is covered by peat land along the Western coastal area up to 15 m thick below the ground surface and the coverage of peat land is around 2500 hectares by Bord and Mona(1984). Peat is classified into two main types such as fibrous and amorphous. Von Post (1992) further divided this from H1 (completely fibrous peat) to H10 (completely amorphous peat) based on Von Post scale system (Huat et al, 2011). There are mechanical and chemical methods to stabilize peat. The largest coal power plant in Sri Lanka, called "Lakwijaya power plant", produces about 200 000 metric tons of fly ash (FA) annually. Only about 30% of the total amount produced, is used for cement production, leaving huge amount of FA ends up in landfills. Utilizing this FA for construction application will be a sustainable solution as it will reduce the land pollution.So, this research is to study the compressibility behaviour of peat stabilized with ASTM class F fly ash (FA) using deep mixing method (DMM) in chemical method, an experimental and numerical study.

2 MATERIALS

Undisturbed peat samples of 150 mm diameter and 1000 mm height were collected from Kalutara, Sri Lanka. The samples were collected using Open Drive Thin Wall Tube sampler. Soon after the sampler was withdrawn, the cylindrical tube was sealed with 'paraffin' wax to retain the natural moisture in it. FA used in this study was obtained from Lakwijaya power plant, Sri Lanka. The chemical composition of FA used is shown in Table 1.

Table 1: The chemical composition of FA

Constituents	Percentage / (%)
SiO_2	52.03
Al_2O_3	32.31
Fe_2O_3	7.04
CaO	5.55
MgO	1.30
SO_3	0.07
K_2O	0.68
Cl	1.00

3 METHODOLOGY

This research comprises experimental and numerical study. The detailed methodology for each is explained in the following sections.

3.1 Experimental work

3.1.1 Von Post classification

This was used to classify the peat based on visual observation. A sample was taken, squeezed and compared the result with Von Post (1922) scale classifications.

3.1.2 Atterberg limits (BS 1377: part 2: 1990)

This test was done to determine liquid and plastic limit of undisturbed peat.

3.1.3 Loss of ignition (BS 1377: part 2: 1990)

This test was carried out to find out organic content of the peat.

3.1.4 Specific gravity (BS 1377: part 2: 1990)

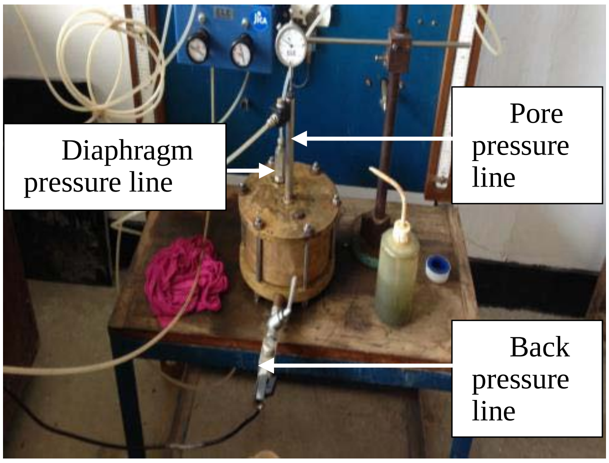
This test was performed to obtain specific gravity of peat using small pycnometer method.

3.1.5Direct shear test (BS 1377: part 7: 1990)

It was conducted to obtain the shear strength parameters [cohesion (c) and angle of friction (Ø)] of raw peat and stabilized peat (peat stabilized with 10% fly ash central column).

3.1.6 Rowe cell test (BS 1377: part 6: 1990)

This test (Figure 1) was performed to determine compressibility characteristics such as coefficient of consolidation (Cv), compression index (Cc) and secondary compression index (Cα). Two types of samples were used: (1) raw peat (2) stabilized peat (Figure 2).



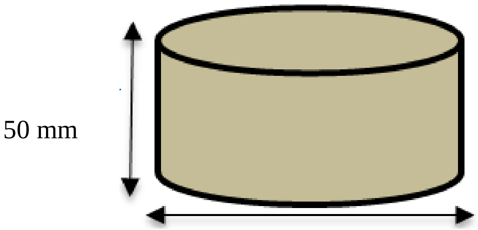
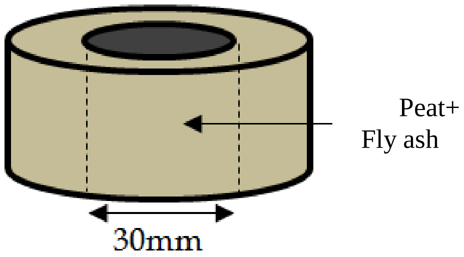


Figure 1 Rowe cell apparatus with connections

150 mm
(a)



(b)

Figure 2 (a) Raw peat and (b) Stabilized peat

3.2 Numerical analysis

Finite element software, Plaxis 2D was used. Axisymmetric model with 75 mm width and 50 mm height was defined (Figure 3). Soft-soil-creep (SSC) and Mohr-Coulomb model were selected for peat and stabilized column. Consolidation condition with stage construction and one-way drainage were used. Interface element reduction factor was defined as 0.65.

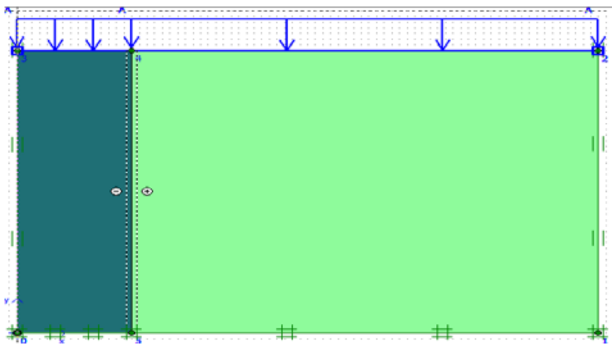


Figure 3. Model geometry for peat – FA stabilized sample

4 RESULTS AND DISCUSSION

Based on Von Post classification, peat falls into sapric amorphous peat (H8).

4.1 Index property test results

Index properties of raw peat is listed in Table 2.

Table 2. Value of index properties

Properties	Values
Initial void ratio	2.03
Bulk density/ (kg/m^3)	1081.7
Moisture content/ (%)	192.1
Specific gravity	1.63
Liquid limit/ (%)	176.5
Plastic limit	Non - Plastic
Organic content/ (%)	62
pH	4.72

4.2 Direct shear test results

Variation of shear stress vs.shear displacement is shown in Figure 4. Peak stresses were obtained for raw peat and FA stabilized peat and then values were plotted against the normal stresses (50, 100 and 200 kPa). Variation of shear stress vs normal stress for peat and FA – peat samples are shown in Figure 5.

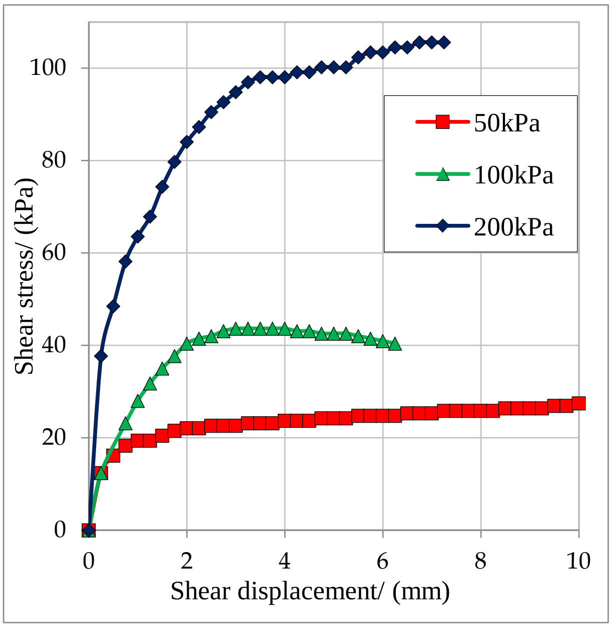


Figure 4 Variation of shear stress vs shear displacement of raw peat

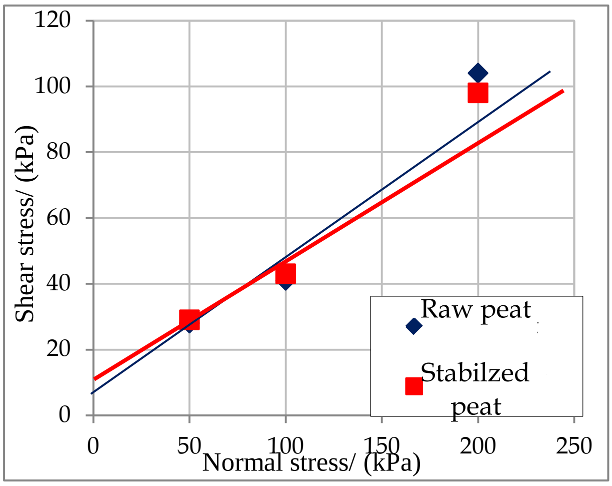


Figure 5 Variation of shear stress vs. normal stress for raw peat and stabilized peat

From the figure 5, c and Ø values were calculated; for raw peat c = 4 kPa and Ø = 24.2º and for stabilized peat c = 7 kPa and Ø = 21.8º

4.3 Rowe cell test results

Variation of settlement vs. time for raw peat and stabilized peat are shown in Figure 6 and Figure 7 respectively.

The values of Cv, Cα and k were calculated and listed in Table 3. The compression index (Cc) for raw peat and stabilized peat are 0.67 and 0.60 respectively. Cv, Cα and Cc decreases with the addition of fly ash because hardened peat-fly ash matrix formed due to hydration, pozzolanic and cation exchange reactions when fly ash comes into contact with water (Huat et al, 2011).

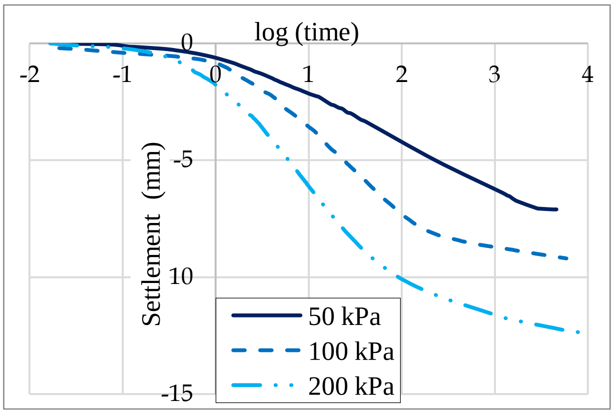


Figure 6 Variation of settlement with logarithmic time of raw peat

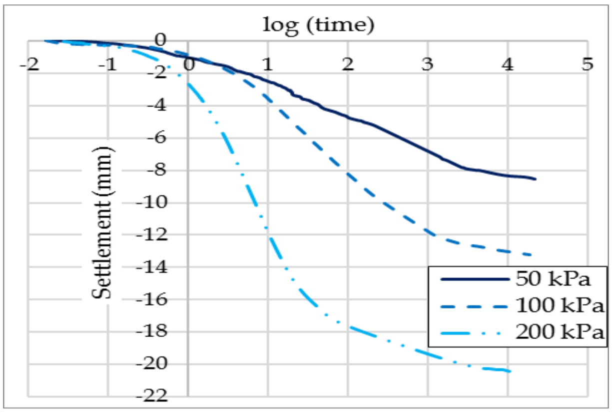


Figure 7 Variation of settlement with logarithmic time of stabilized peat

4.3 Numerical analysis results

Figures 8 shows the experimental and numerical comparison between settlement with time for stabilized peat under 50, 100 and 200 kPa normal pressures.

Based on Figure 8, it can be seen that there is close agreement between experimental and numerical settlement values and hence,Plaxis 2D can be used to model the consolidation behaviour of peat.

Sample	Normal stress (kPa)	Secondary compression index (C_α)	Coefficient of consolidation (C_v) / (m^2/s) (x 10^{-7})	Permeability coefficient (k)/ (m/s) (x 10^{-8})
Raw peat	50	0.014	4.667	1.299
	100	0.038	6.954	1.258
	200	0.054	8.569	1.042
Stabilized peat	50	0.013	3.599	1.204
	100	0.030	4.631	1.202
	200	0.040	7.484	1.198

Table 3 Variation of consolidation parameters

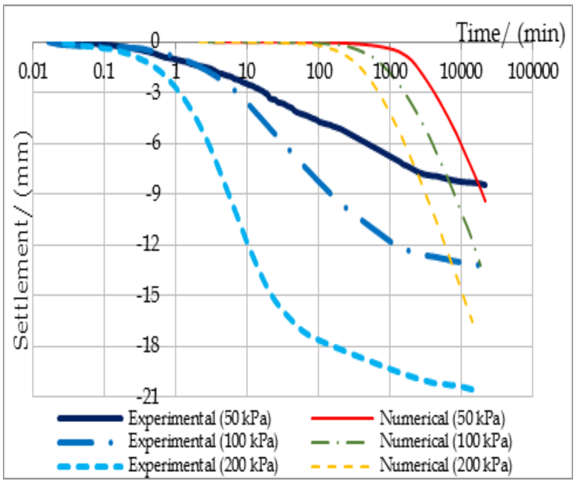


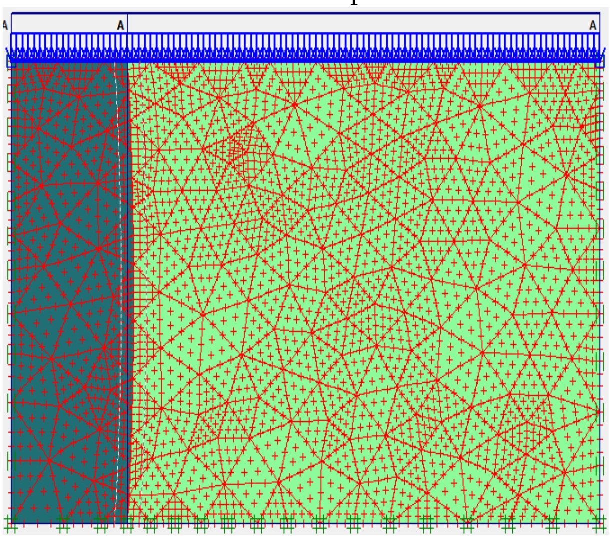
Figure 8 – Settlement – time curve of stabilized peat for 50, 100 and 200 kPa normal pressure

The deformed meshe for 10% FA stabilized peat under 50 kPa normal pressure is shown in Figure 9.

Figure 9 Deformed mesh of stabilized peat for 50 kPa normal pressure

5. CONCLUSIONS

Peat is sapric amorphous peat. Cohesion(c) increases and compressibility parameters (Cc, $C_{\alpha,}$and C_v) decreases with the addition of fly ash.The maximum percentage reduction in Cv, Cα and Cc are 33.4%, 25.9% and 10.5% respectively. There is a good agreement between the settlement value obtained from numerical and experimental works.



On the whole, compressibility parameters reduce significantly with the introduction of FA and this can lead to significant reduction in the settlement and improvement in bearing capacity when peat – FA deep mixing columns are used

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Use of Stone Columns for Settlement Control in Construction of Embankments in Soft Peaty Clays.

S.M.Y.I. Subasinghe

Department of Civil Engineering, University of Moratuwa, Sri Lanka

*ABSTRACT:*Use of stone columns is a cost effective ground improvement technique that can be used when constructing higher embankments on soft thick clay soils. Soft soil reinforced by installing stone columns in grid pattern enhancing the bearing capacity, shear resistance and reduce settlements. They also enhance the stability of embankments and high embankments could be constructed on sites underlain by thick layers of soft soil. This research mainly focusing on the settlement control of embankments using stone columns. Settlement reduction achieved by the use of stone columns in different configurations was studied by modelling with finite element technique and was compared with results given by currently used empirical equations. Finite element analysis was done with PLAXIS 2D software.

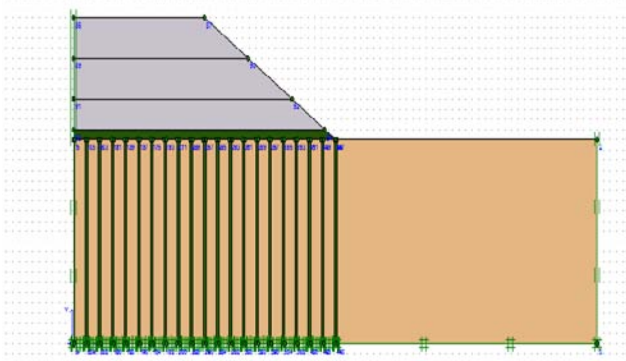
Key words—soft soil; stone column; embankment; finite element

1. INTRODUCTION

Due to the shortage of good quality ground, constructions have to be carried in grounds underlain by soft clays and peaty clays. Structures constructed on soft soils, can fail immediately due to bearing failure, shear failure or excessive settlement. In order to avoid such failures, proper ground improvement has to be appliedduring the construction. Preloading, deep mixing and replacing soft soil with good soil are some methods for ground improvement when constructing embankments. Alternatively, loads can be transferred to deeper soil layers by using piles, which would be of very high cost in constructions such as highways. Use of stone columns is a cost effective technique of reinforcing the soft clay layers. In this method, granular columns (made of; gravel, sand, lime etc.) having higher stiffness and strength are installed in a grid pattern. Stone columns will improve the stability of embankments lopes constructed on improved ground and reduce total and differential settlement. The time rate of settlement is also increased.

2. METHODOLOGY

In this research, settlements of an embankment constructed on a site underlain by soft clay is studied. Two cases were studied. Construction of an embankment without stone columns was the first case and an embankment constructed on soft soil reinforced with stone columns was the second case. Finite element approach was used and geotechnical finite element analysis software PLAXIS 2D was used for the analysis. Due to symmetry, only half of the embankment was modeled. Behavior of soft clay was modeled with coupled consolidation analysis and a parametric study was done varying configuration and properties of stone column material. The soft clay and stone column material were taken as Mohr-Coulomb material. Stage construction procedure with 0.5m per week construction rate was used to represent the actual construction procedure. Stability of embankments were assessed by calculating safety factors for embankments. In PLAXIS, FOS value for embankments cannot be calculated directly and it was obtained through c, ϕ reduction method in the program. In the analysis, FOS was calculated just after the construction of the embankment considering, as the critical scenario. Then embankment was allowed to consolidate until excess pore water pressure falls below 1 kN/m^2.



3. MODEL PARAMETERS

In the model an embankment constructed on 10m thick soft clay later was considered and laterite soil was used as embankment material and sand or gravel

was used for stone column material. Stone Column diameter and grid spacing was taken as 0.5m and 1.0m respectively. Considered embankment heights were 4m, 6m and 8m. Embankment width was maintained at 10m in every scenario and embankment slope was maintained at 1:2 ratio. Model used for 6m height embankment was shown in Figure 1. Updated mesh analysis was performed in the PLAXIS model, due to the occurrence of large deformations in the analysis. Parametric analysis was carried out by changing the stone column material friction angel to 32^0, 36^0 and 40^0. Variation of material stiffness with friction angle (ϕ^0) was also considered to achieve more accurate results (Table 1). A drainage layer of 0.5m was laid as the initial layer to improve the pore water pressure dissipation. Material properties used in the model were given in Table 2.

4. MODEL IDEALIZATION

In reality, the stone column installation is a three dimensional problem. To conduct the analysis in 2D, model has to be idealized to a 2D plain strain model. In the 2D model, cross section of the embankment and the clay layer were directly idealized, but the circular area of a stone columns installed in grid pattern were idealized as strips in the plain strain model (Figure 2). Idealized width of a stone column strip is 0.196m.

Table 1. Variation of stone column material stiffness with friction angle.

ϕ^0	E_{sc} (kN/m^2)
32^0	25000
36^0	32000
40^0	39600

Table 2. Material Properties.

Property	Soft Clay	Embankment	Stone Column
γ_{sat} (kN/m^3)	15	20	20
γ_{unsat} (kN/m^3)	13	18	20
c'(kN/m^2)	1	10	1
ϕ'(^{0})	26	30	32 - 40
E (kN/m^2)	2000	8000	25000 - 39600
ν	0.35	0.3	0.2

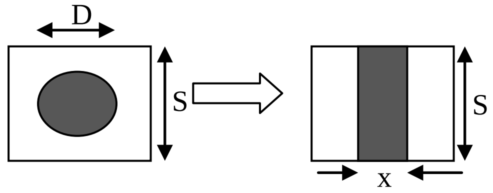


Figure 2. Model idealization from 3D to 2D.

$$\pi \frac{D^2}{4} = xS \quad (1)$$
$$\therefore x = \pi \frac{D^2}{4S}$$
$$\therefore x = \pi \frac{0.5^2}{4x1} = 0.196 \text{ m}$$

Where, D – Stone column diameter, S - Stone column spacing and x – Idealized with of a stone column.

5. COMPARISON OF RESULTS.

5.1 Calculation of stress concentration factor.

In a soft clay layer reinforced with stone columns a greater share of the load is taken by the stiffer stone columns. In the finite element analysis this stress distribution is evaluated based on the assigned material parameters. In the empirical methods a stress concentration ratio as outlined in the preceding paragraph is assigned. Stress concentration effect is lustrated in Figure 3.

Where, a_s = Area replacement ratio, A_s = Cross sectional area of stone column and A = Area of soft soil before installing stone columns

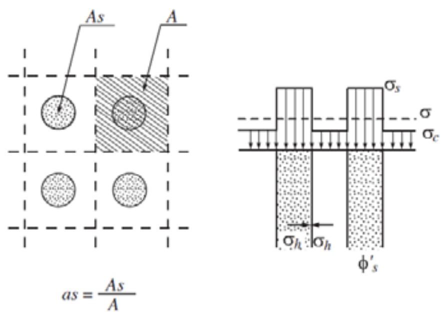
Stress concentration factor (n) is expressed as the ratio

Figure 3. Stress concentration effect.

of stresses in stone columns (σ_s) and soft clay (σ_c)

$$n = \frac{\sigma_s}{\sigma_c} \quad (2)$$

Based on experimental values, an equation has developed to calculate stress concentration factor.



$$n = a + b * \left(\frac{E_S}{E_C}\right) \quad (3)$$

Where,

a = 0.78, b = 0.22

E_S= Modulus of elasticity of stone column material

E_C= Modulus of elasticity of existing ground material (2000 kN/m^2)

Table 3. Empirically estimated stress concentration factor.

ϕ^0	E_{sc} (kN/m^2)	$n = 0.78 + 0.22 * (\frac{E_S}{2000})$
32^0	25000	3.53
36^0	32000	4.3
40^0	39600	5.136

5.2 Comparison of stresses obtained from finite element analysis.

Finite element mesh of the 6m height embankment with stone columns was shown in Figure 4. The stiffness of the stone column material(E) is proportional to the friction angle. The stress concentration factors obtained are presented in Table 4 also. With the empirical methods the stress concentration factor is a constant which do not vary with the embankment height.

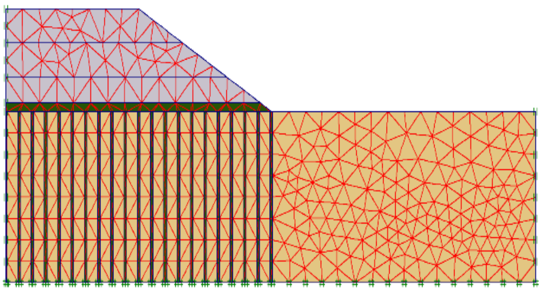


Figure 4. Finite element mesh for 6m height embankment with stone columns.

Table 4. Stress concentration factor from Finite Element analysis results.

Embankment height	φ	Stress concentration factor
	32^0	1.729
4m	36^0	1.737
	40^0	2.520
	32^0	1.773
6m	36^0	2.168
	40^0	2.595
	32^0	2.012
8m	36^0	2.461
	40^0	2.986

5.4. Comparison of settlements of embankments.

Estimated Settlement variation of 6m height embankment using finite element analysis and empirical methods are compared in Figure 6.Estimated settlements using empirical methods have 600mm (Emp_without Sc.), for embankment without stone columns and 268mm(Emp_6m_32), 243mm(Emp_6m_32), 221mm(Emp_6m_32) settlements for embankment with stone column material friction angles of 32^0, 36^0 and 40^0respectively. There is a reasonable agreement with the finite element estimation.

In finite element analysis, highest settlement of 327.7mm occurred in the embankment constructed on unimproved ground. There was a clear reduction of settlements of embankments constructed on ground improved with stone columns. Lowest value of 197.7mm was given under stone column material $\phi=40^0$.Settlement reduction has increased with increase of friction angle, due to the of the increase of stiffness. With the construction rate of 0.5m/week the construction of the 6m high embankment would be completed within 12 weeks. In the embankments supported by stone columns the settlement is almost completed at the end of construction. This is due to the dissipation of excess pore water pressure through the highly permeable stone columns which as placed at a grid spacing of 1.5m. Settlement of unimproved ground has not reached to constant rate even after 27 weeks.

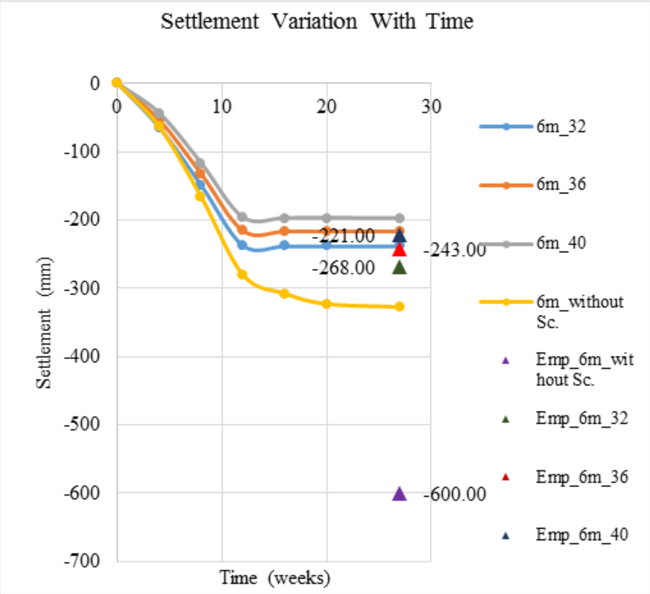


Figure 6. Variation of settlement of clay layer with time under 6m height embankment for finite element and empirical methods.

5.5 Comparison of excess pore water pressure.

A further illustration of this is done by comparison of the excess pore water pressure at different times. Excess pore pressure distribution under 6m height embankment at different times is compared in Table 5. This is graphically presented in Figure 7.

Table 5. Excess pore pressure variation with time of 6m height embankment constructed on improved ground in kN/m^2.

φ	4 weeks	8 weeks	12 weeks	16 weeks
32^0	0.303	0.460	0.508	0.00022
36^0	0.283	0.395	0.452	0.00033
40^0	0.264	0.400	0.438	0.00082

6. CONCLUSION

In this study the focus was on the effect of stone columns in the reduction of settlements constructed in sites underlain by thick layers of soft clays. The results of the analysis clearly illustrated that there is a significant reduction in settlements due to the introduction of stone columns. The reduction increased with the increase of the stiffness of the stone column material. The stress concentration factor which was given a constant value in empirical methods increased with the height of the embankment constructed. The highly permeable stone columns accelerated the consolidation and all the consolidation settlements were completed at the end of construction. When there were no stone columns consolidation continued for a longer time.

However, the ultimate settlements estimated by the empirical methods were reasonably good agreement with those estimated by the finite element method.

ACKNOLEDGEMENT

Author is grateful to Prof. S.A.S. Kulathilaka and Dr. Nalin De silva for their valuable guidance and support on finite element modelling and theoretical information.

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A Study on the Effect of Material Anisotropy on Consolidation Behaviour of Peat

S.M.Gajanayake, R.D.S.S.Jayasooriya, K.G.H.C.N. Seneviratne
Department of Civil Engineering, University of Peradeniya, Sri Lanka

Abstract; Peat is a problematic soil and there can be excessive settlements when the constructions are carried out on peat. As the consolidation behaviour directly deals with the settlement, it is essential to study the factors affecting the consolidation behaviour to implement remedial strategies. Peat shows some unique properties due to its anisotropy and therefore, it is difficult to analyse using conventional approaches. In this study, the consolidation behaviour of peat was discussed in the perspective of cross anisotropy. Stress-strain relationship for peat was modified to account the material anisotropy. Standard laboratory experiments were conducted to examine the consolidation behaviour using horizontal and vertical samples. Considerable anisotropy was observed in the mechanical properties of peat. The main reason behind this anisotropic behaviour is assumed to be the horizontal alignment of fibre. Different models of PLAXIS 2D were used to model the consolidation behaviour. Soft-soil creep model was identified as the optimum model to estimate the final settlement of peat.

Keywords: **Peat, Consolidation, Material Anisotropy, Numerical modelling, PLAXIS 2D**

1 INTRODUCTION

In the field of civil engineering, it is always preferred to have soils with good qualities. But it has become inevitable to carry out the constructions on less favourable soil such as expansive soils, collapsible soils and peat, because of the increase of population and non-availability of land. Low shear strength, high compressibility and high water content can be identified as some problematic properties of soils which make them unfavourable. Adequate countermeasures such as ground improvement, pre-loading and sufficient changes in the construction methodology can be followed to overcome these difficulties.

Settlement of a structure is critical to its stability and due to those undesirable properties, there can be excessive settlements, resulting issues in serviceability. As the consolidation behaviour of a soil directly deals with the settlement of a superimposing structure, it is essential to study the correlations between consolidation and the factors affecting.

Peat can be identified as a problematic soil and it shows some unique properties making it difficult to analyse using conventional theories and approaches. The structural anisotropy of peat is responsible for its unique characteristics and because of that, it has attracted considerable attention of the researchers resulting several approaches. Numerical analysis methods such as finite element and discrete element methods have been used to analyse the behaviour of peat.

In this study, the consolidation behaviour of peat was discussed in the perspective of cross anisotropic linear elasticity. Standard laboratory experiments were conducted to examine the consolidation behaviour of peat. Conventional oedometer test was conducted for peat samples using five load increments to study the consolidation. For the interpretation of oedometer test results, axis-symmetrical conditions were considered. Both horizontally and vertically retrieved samples were used in order to distinguish the anisotropic properties. Considerable anisotropywas observed in the mechanical properties of peat, especially in the coefficient of permeability. The Young's Modulus in the vertical direction was also considerably higher than that of horizontal direction. The variation of the coefficient of permeability with consolidation pressure and the void ratio was also analysed. Themainreason behind this anisotropic behaviour was assumed to be the horizontal alignment of fibre.

Axis-symmetry was adapted for the analyses and using PLAXIS, numerical models were prepared to compare with experimental models. In the numerical simulations, the adequacy of the available models in the PLAXIS was determined to model the consolidation behaviour of peat.

2 METHODOLOGY

The study was conducted in three main phases, namely,

a. Theoretical analysis to account for the anisotropy of mechanical parameters into the consolidation formulations
b. Experimental work to determine the mechanical and index parameters of peat

c. Numerical simulation

2.1 Theoretical Analysis

Stress strain relationships were obtained assuming cross- anisotropic linear elasticity. The plane strain and axis-symmetrical conditions were adopted for the formulations and it was attempted to obtain relevant stress-strain relationships by applying the necessary boundary conditions. The analyses were carried out to identify the mathematical relationships between the stress-strain relationship, consolidation and material anisotropy.

2.2 Experimental Work

Laboratory tests were carried out on peat to obtain the physical properties and material parameters needed for numerical simulations. All the tests were conducted according to BS standards. Horizontally and vertically retrieved samples were tested to identify variation of mechanical properties, using oedometer test and triaxial test. Five load increments were used for the consolidation test and from the consolidation curves, required consolidation parameters (C_v,m_v) were obtained to calculate the coefficient of permeability using Taylor's square root of time fitting method and considering void ratio values. Secant modulus of elasticity was calculated from unconsolidated undrained triaxial test results.

2.3 Numerical Simulation

In order to develop a numerical model to replicate the consolidation behaviour, PLAXIS 2D software was used in the study. To analyse the oedometer test results, axis-symmetric conditions were adopted. Linear elastic model, Soft soil model and Soft soil creep model were used for the analysis and the adequacy of these models were determined to simulate the consolidation behaviour of peat.

3 RESULTS AND DISCUSSIONS

3.1 Experimental work

The physical properties of the peat samples used for the testing can be tabulated as in table 01.

Table 01: Physical properties of the peat sample

Property	Test	Value
Moisture Content	Oven DryTest	150.32%
Liquid Limit	Atterberg Limit Test	175.20%
Organic Content	Loss on Ignition Test	67 %
Specific Gravity	Small Pyknometer Test	1.62
Bulk Unit Weight	-	12.19 kN/m^3
Dry Unit Weight	-	8.02 kN/m^3

Consolidation behaviour of vertically and horizontally retrieved samples were observed and the required consolidation parameters were calculated and their variations were analysed.

3.1.1 Void ratio

Figure 1 shows the variation of the void ratio with the logarithmic value of stress applied. It is apparent that the void ratio values of the vertical sample are higher than that of horizontal sample. The change of void ratio in both samples with the applied stress, seem to be fairly similar for both samples.

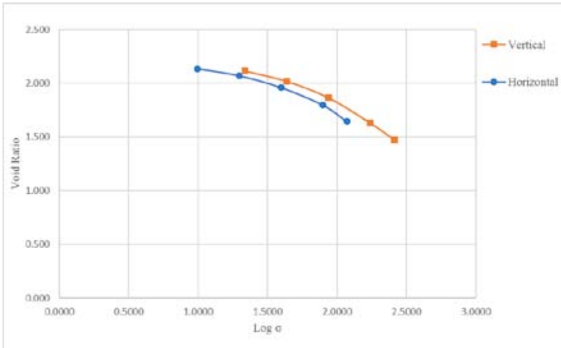

Fig. 1 Variation of void ratio with stress

3.1.2 Consolidation Parameters

The variation of coefficient of consolidation with the stress can be shown as in figure 2. The C_v values of the both samples are decreased when the stress is increased but the rate of decreasing is higher in the horizontal sample. For the horizontally retrieved sample the coefficient of volume compressibility is greater.

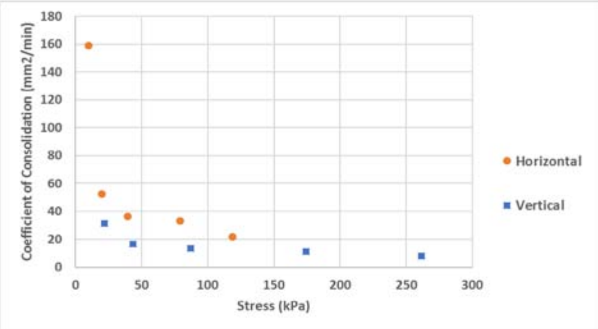

Fig. 2 Variation of coefficient of consolidation

In the figure 3, the variation of the coefficient of volume compressibility with the applied stress is depicted.

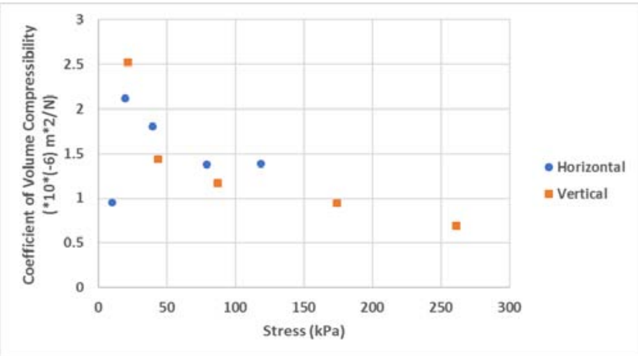

Fig. 3 Variation of Coefficient of volume compressibility with the applied stress

It can be seen that m_v values for vertical sample are lesser than the horizontal sample and in both samples, $m_v$value is decreased when the stress is increased.

3.1.3 Coefficient of Permeability (k)

After obtaining the coefficient of volume compressibility (m_v) and coefficient of consolidation (C_v), the coefficient of permeability was calculated using the following relationship, where γ_w is the unit weight of water.

$k = m_V C_V \gamma_w$ (1)

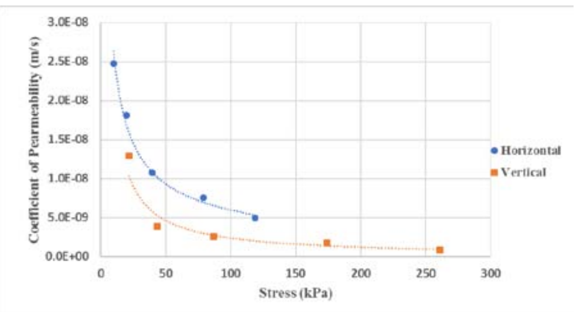

Fig 4. Variation of Coefficient of Permeability with stress

In figure 4, the variation of coefficient of permeability with the stress values is illustrated. It is obvious that the permeability in the horizontal direction is greater. The permeability value is decreased in the both samples when the stress is applied although, the degree of decreasing is higher in the horizontal sample making the k_h/k_v ratio decreased with the stress.

According to the averaged results, the Vertical coefficient of permeability (k_v) is $4.378*10^{-09}$ m/s and the Horizontal coefficient of permeability (k_h) is $1.322*10^{-08}$ m/s. The generalized k_h/ k_v ratio can be obtained as 3.02 for this occasion. The effect of changing the void ratio towards the coefficient of permeability can be shown as in figure 5.

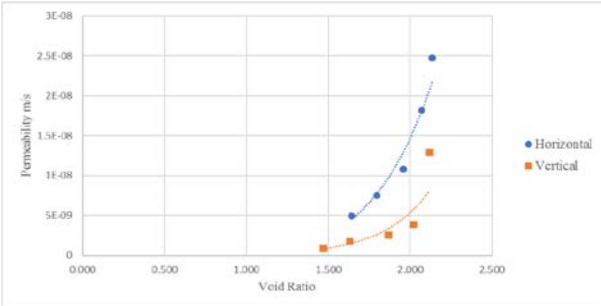

Fig. 5 Variation of Coefficient of permeability with void ratio

A small increment in the void ratio in the horizontal sample yields a considerable increase in the coefficient of permeability when comparing with the vertical sample. The cause behind this is the horizontal alignment of fibre which facilitates the water to flow in that direction easily.

3.1.4 Young's Modulus

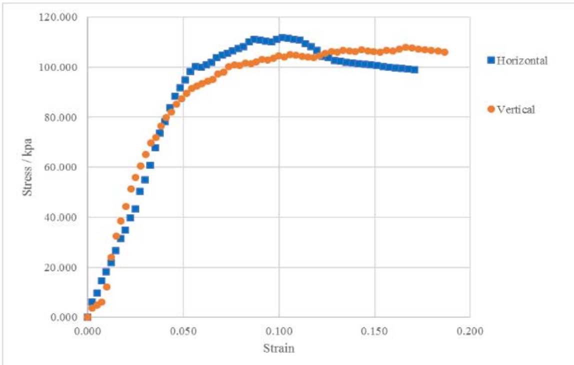

Fig. 6 Stress Strain behaviour of samples

In the study, triaxial test was conducted for horizontally and vertically retrieved samples to obtain the undrained Young's Modulus and consolidation results were used to obtain the drained Young's Modulus. In calculating the undrained Young's Modulus, Secant modulus at the 40 percent of ultimate stress was used. The calculated values for Secant moduli at the 40 percent of the ultimate stress are; 2229.2 kPa for vertical sample and 1818.2 kPa for horizontal sample.

3.2 Theoretical Analysis

It is attempted to account the anisotropy of the material parameters which affect the consolidation behaviour, in the stress strain relationship. For the plane strain case following relationship can be discussed as the resultant.

$$\begin{Bmatrix} \xi_{xx} \\ \xi_{zz} \\ \gamma_{xz} \end{Bmatrix} = \begin{bmatrix} (1-\nu_{hh}^2)/E_h & -\nu_{vh}(\nu_{hh}+1)/E_v & 0 \\ -\nu_{hv}(\nu_{hh}+1)/E_h & (1-\nu_{hv}\nu_{vh})/E_v & 0 \\ 0 & 0 & 1/G \end{bmatrix} \begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \tau_{xz} \end{Bmatrix}$$

It is apparent that Young's modulus and Poisson's ratio in the above relationship have directional differences, hence there is an effect of material anisotropy in this stress-strain relationship.

3.3 Numerical simulation using PLAXIS 2D

The main objective of conducting a numerical simulation was to model the consolidation behaviour and to compare it with the laboratory experimental data to check the adequacy of the available models to simulate the consolidation behaviour of peat. Figure 7 shows the comparison of the experimental data and the results obtained from PLAXIS 2D using different models for the first load increment of the vertical sample.

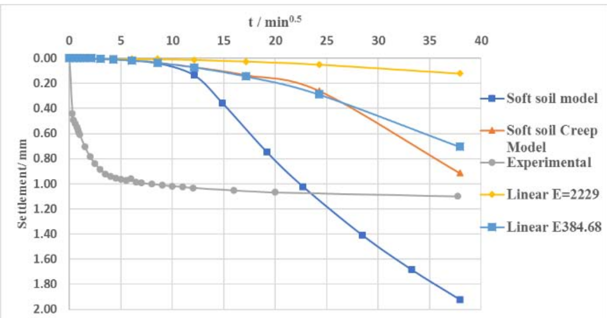

Fig. 7 Comparison of the results generated by PLAXIS 2D with the experimental results

A numerical comparison between these results can be listed as in table 2.

Table 2 Comparison of the test results generated by PLAXIS 2D

Model	Final Settlement (mm)	Deviation from the experimental value (%)
Experimental results	1.100	-
Linear Elastic model using Undrained Young's Modulus	0.122	-88.91
Linear Elastic model using Drained Young's Modulus	0.705	-35.91
Soft-soil model	2.540	+131.91
Soft-soil creep model	0.914	-16.91

4 CONCLUSIONS

It is apparent that the material anisotropy has an effect on the consolidation, since the mechanical parameters of peat which directly related with the consolidation, contains the anisotropy. As discussed in the theoretical analysis; E_v ,E_h , ν_v and ν_h are embedded in the stress-strain relationship and therefore, it is evident that the material anisotropy has an effect on the consolidation.

The horizontal coefficient of permeability was significantly higher than the vertical coefficient of permeability according to the experimental results. This is due to the horizontal alignment of fibre which facilitates the pore water to dissipate easily. Young's Modulus in the horizontal direction was also slightly less than that of vertical direction.

In PLAXIS 2D, Soft-soil creep model is identified as the most suitable model to simulate the consolidation behaviour of peat, even though it cannot replicate the behaviour in the intermediate states.

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Utilization of Soil-cement Columns to Stabilize Soft Peaty Clay

K.K.J.Kodikara
Department of Civil and Environmental Engineering, University of Ruhuna, Sri Lanka

Finding the best soil stabilizing technique to overcome the problematic nature of soft peaty clay is still being the main concern in Sri Lanka. The main drawback in currently using improvement techniques is the requirement of long time duration for achieving improvements. Therefore, the recent attention of engineers has drawn towards deep mixing technique in which soft soil is in-situ mixed with a stabilizing agent. In this research, the improvements that could be achieved in both unconfined compressive strength and compressibility characteristics of soft peaty clay were assessed after mixing soil with different proportions of cement and locally available by products such as rice husk ash, fly ash and quarry dust. Based on the results, it could be concluded that high improvements in soft soil could be achieved by mixing soft soil with 20% cement and 30% of rice husk ash or fly ash in weight.

1 INTRODUCTION

In Sri Lanka, number of new projects such as Katunayaka Expressway, Southern Highway and Outer Circular Highway are going on at locations where there are soft soils with substantial thickness.

Construction on soft soil leads to substantial problems during the construction and long term settlements during the service due to its low shear strength and very high compressibility. Different methods are available for the improvement of characteristics of soft soil. Among them, preloading is the most widely used ground improvement method. However the main drawback in preloading is the necessity of long term duration for consolidation. In addition, replacement method is also commonly used in the Sri Lanka although it is not environmental friendly.

The deep mixing method is a world-wide accepted ground improvement technique today. When soft clay is mixed in-situ with a suitable binder such as cement, the hydration reactions and subsequent pozzolonic reactions will cause a major change in the microstructure and considerable gain in strength and stiffness would be achievable within a relatively shorter period of time such as 28 days.

Therefore attempts were made in this research to study the possibility of using cement and other industrial by products such as rice husk ash, fly ash and quarry dust ash to improve engineering characteristics of peaty clay. The soil sample used for this study was collected from Nilwala flood plain in Matara.

2 METHODOLOGY

The basic properties of natural peaty soil were determined immediately after the collection of soil samples. Then test cubes of 150 mm size were casted by mixing soft peaty clay with different percentages of cement, rice husk ash, fly ash and quarry dust.

The mix proportions of the samples are presented in Table 1. The water cement ratio for all the mix proportions was 0.3.

Table 1. Soil cement mix proportions

Sample No.	Mix proportion
SC10*	Cement 10%
SC20	Cement 20%
SC25	Cement 25%
SC30	Cement 30%
SC40	Cement 40%
SC20+R10	Cement 20% RHA 10%
SC20+R20	Cement 20% RHA 20%
SC20+R30	Cement 20% RHA 30%
SC20+F10	Cement 20%FlyAsh10%
SC20+F20	Cement 20%FlyAsh20%
SC20+F30	Cement 20%FlyAsh30%
SC20+Q10	Cement 20% Quarry Dust 10%
SC20+Q20	Cement 20% Quarry Dust 10%
SC20+Q30	Cement 20% Quarry Dust 10%

*soil 90% + cement 10% mixing

Figure 1 presents the mechanical mixer that was used for mixing.

With the intention of providing similar mixing conditions to all samples, a similar mixing speed and a mixing duration of 10 minutes were maintained.

Fig. 1 Mechanical mixing of soil

The casted test cubes were kept under submerged conditions and unconfined compressive strength was tested after 14 and 28 days using compression testing machine.

Simultaneously, soil samples from each and every mix proportion were taken into consolidation rings to prepare consolidation test specimens. These specimens were also kept under submerged conditions and oedometer test was conducted after 28 days. By using the test results of consolidation test the variation of following consolidation characteristics were identified.

- Void ratio (e) versus effective vertical stress (σ) relationships.
- Variation of co-efficient of compression index (C_c) and modified compression index (C_c') over different percentages of admixtures.
- Variation of co-efficient of volume compressibility (m_v) over different percentages of admixtures.

3 RESULTS AND DISCUSSION

3.1 *Basic properties of soft peaty clay*

The basic properties of soft peaty clay samples collected from Nilwala flood plain in Matara are presented in Table 2. Table 3 presents the basic properties of improved soil samples.

According to the values in Table 2, it is obvious that soft peaty clay in Nilwala flood plain has a high moisture content, low specific gravity and a high initial void ratio. Therefore, finding an effective method of soft soil improvement is a need of the hour.

Table 2. Basic properties of natural soil

Basic property	Value
Moisture Content (%)	202
Bulk density (kN/m^3)	11.4
Specific Gravity (Gs)	1.97
Ph Value	5.3
Organic Content (%)	23
Initial Void Ratio	4.56
Compression index	1.57
Plastic limit (%)	59
Liquid limit	160

Table 3. Basic properties of improved soil

Sample No.	Bulk unit weight (kN/m^3)	Specific Gravity (Gs)	Moisture content after 28 days (%)
SC10	13.5	1.99	61
SC20	14.3	2.06	44
SC25	14.8	2.19	38
SC30	15.4	2.25	36
SC40	16.8	2.36	31
SC20+R10	14.0	1.72	45
SC20+R20	13.9	1.68	61
SC20+R30	13.8	1.57	71
SC20+F10	13.9	1.90	35
SC20+F20	14.1	1.85	37
SC20+F30	14.2	1.77	42
SC20+Q10	14.4	2.24	43
SC20+Q20	14.8	2.27	40
SC20+Q30	15.4	2.33	33

3.2 *Unconfined compressive strength*

Soft peaty clay are of extremely low unconfined compressive strength and prevention of shear failure during constructions has become a major challenge. Therefore, it is essential to increase the confined compressive strength of soft peaty soil prior to constructions.

The unconfined compressive strengths of naturally existing soft peaty clays are extremely low values. The improvements in unconfined compressive strength of soft soil after mixing with different additives are shown in Figure 2.

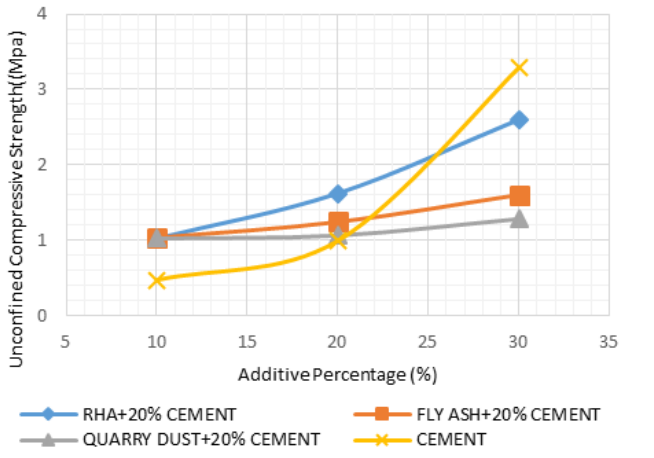

Fig. 2 Variation of UCS with additive percentage

It is obvious that with the increase of the additive percentage, the unconfined compressive strength of soft soil increases. However, mixing of cement more than 30% provides significant improvements. In addition, mixing of 30% rice husk ash and 20% cement provides a considerably high improvement. This improvement can be due to the high silica content present in rice husk ash compared to other additives.

3.3 *Consolidation characteristics*

The assessment of improvement of consolidation characteristics of improved soil is also important same as the assessment of unconfined compressive strength because those values give a clear idea about long term settlement of the structure construct on improved soil.

Variation of e versus log (σ'_v) for cement mixed soil is shown in Figure 3.

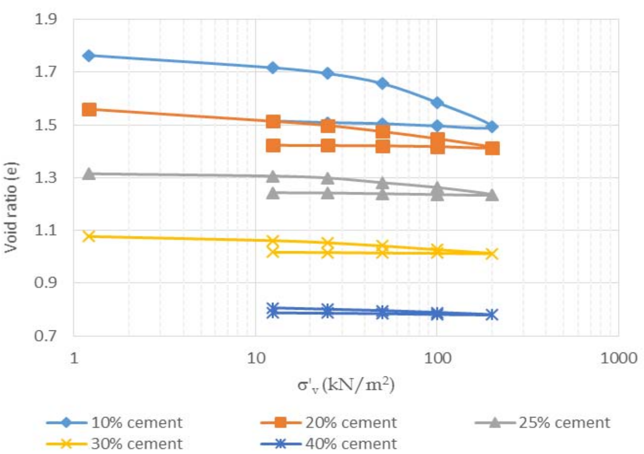

Fig. 3 Changes in e Vs log (σ) relationships with cement percentage

It can be clearly seen initial void ratio of soil-cement mixtures has been reduced with the increase of cement content as shown in Figure 3. This is primary due to the improvement of solidification characteristics of peaty soil under higher cement content.

Variation of e versus log (σ'_v) for soil-cement mixed different additives are shown in Figure 4 to Figure 6.

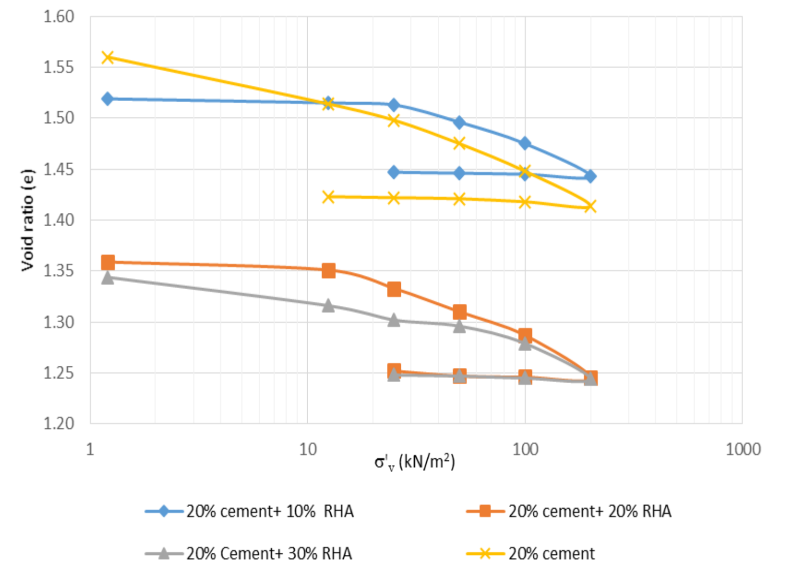

Fig. 4 Changes in e Vs log (σ_v') relationships of soil-cement mixed with RHA

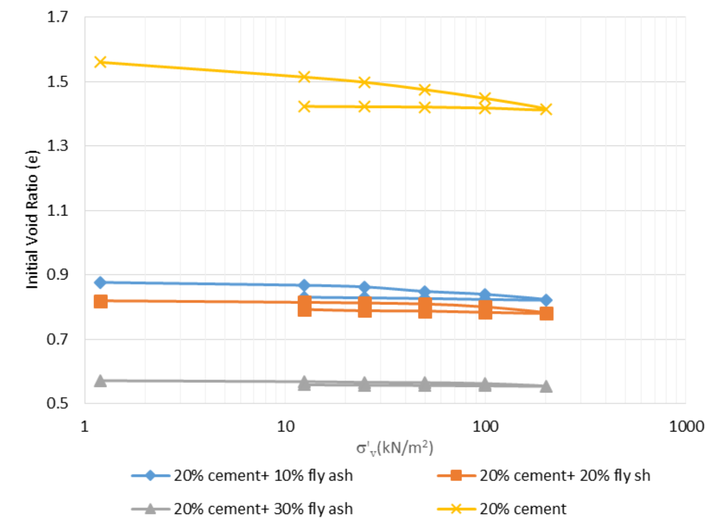

Fig. 4 Changes in e Vs log (σ_v') relationships of soil-cement mixed with Flyash

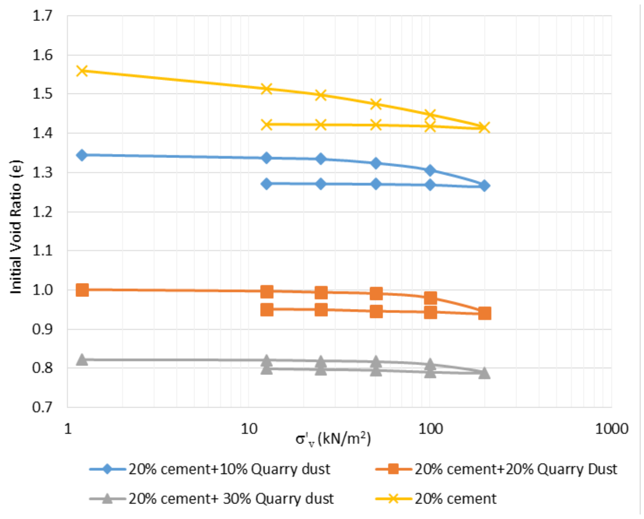

Fig. 5 Changes in e Vs log (σ_v') relationships of soil-cement mixed with Quarry dust

The primary consolidation characteristics that were obtained from e Vs log (σ_v') plots of different improved soil samples are shown in Table 3.

Table 4. Compressibility characteristics of soil-cement mixtures

Sample No.	Initial Void Ratio	Cc	Cc/(1+e$_0$)
SC10	1.76	0.27	0.1
SC20	1.56	0.10	0.04
SC25	1.32	0.09	0.04
SC30	1.08	0.05	0.02
SC40	0.81	0.03	0.01
SC20+R10	1.52	0.10	0.04
SC20+R20	1.36	0.13	0.06
SC20+R30	1.34	0.11	0.05
SC20+F10	0.88	0.06	0.03
SC20+F20	0.82	0.05	0.03
SC20+F30	0.57	0.03	0.02
SC20+Q10	1.35	0.13	0.06
SC20+Q20	1.00	0.12	0.06
SC20+Q30	0.82	0.07	0.04

According to Table 4, it can be clearly seen that the compressibility of soft soil has been reduced with the increase of additive content. After mixing with additives, the settlement of the soft soil has been reduced around 90%. However, when comparing the improvements, it is obvious that mixing of 30% fly ash and 20% cement with soil provides a considerable reduction in compressibility. Other than that, mixing f 30% RHA along with 20% cement also provides a considerable improvement in compressibility characteristics of soft peaty soil.

4 CONCLUSIONS

When studying the results of this study significant improvements in both unconfined compressive strength and compressibility characteristics of soft peaty soil could be achieved by mixing soil with more than 30% of cement in weight. Those improvements were higher than the improvements that could be obtained by mixing 20% cement and 30% additives with soil. However, the mixing of soft soil with higher percentages of cement is not economically feasible. Therefore, after considering the economic concerns, it could be concluded that the mixing of soft soil with 20% cement and 30% fly ash or rice husk ash as the most effective combinations of additives that can be used for deep mixing.

In this study, the additive percentage was varied by keeping the cement percentage constant in 20%. As a future direction, laboratory experiments can be done by reducing the cement percentage and increasing the additive percentage.

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