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A STUDY OF SEISMIC STATUS OF GREATER COLOMBO AREA

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ABSTRACT

A study has been undertaken to evaluate the local site effects upon earthquake damage in the Greater Colombo Region. It was demonstrated during many past earthquakes that the characteristics and the magnitude of the damage are greatly influenced by the dynamic properties of local soil profiles. The current study consists of a series of analysis in designing of earthquakes having different magnitudes, determination of the soil dynamic parameters, and the evaluation of the characteristics and the magnitude of local site response. The last characteristic is an indicator of the vulnerability of the structures under a future earthquake in the region. Local seismic amplification and soil liquefaction are the two different characteristics of the local-site-response considered in the analysis.

INTRODUCTION

In the past there had been many seismic events in the region, which were felt in Sri Lanka. Even though some of these are very small in magnitude and others had epicenters far away from the country, the necessity to study the seismic effects in Greater Colombo has been come to increasing attention of engineering professionals.

Colombo is a densely populated city and further expansion of the city is expected to accommodate modern development schemes leading to further increasing population. Abayakoon (1998) analyzed the seismic response of low lying areas in Colombo and demonstrated that Colombo is more vulnerable to any future seismic events than any other part of the country. However, the city is not yet sufficiently prepared for a seismic hazard.

Past seismic history, distance from any active fault, and the local site conditions have great influence in the magnitude and the characteristics of the damage caused by an earthquake. The first two factors have little influence in the current study as all the analyses were carried out for designed, imaginary earthquakes having the characteristics of the motion most likely to occur. The current study primarily deals with the influence of local site conditions upon earthquake damage. In many past earthquakes it was demonstrated that sites underlain by deposits of soft soils experienced peak ground acceleration 2 to 3 times greater than the nearby sites on stiff soils or rock. During 1989 Loma Prieta earthquake in the USA the portions of the highway I-80 (Cypress Viaduct) collapsed while adjacent sections of the same design founded on stiffer soils did not collapse. Another well known example for this soft deposit amplification effect was experienced during 1985 Mechoacan Earthquake; Mexico city which is 350 km away from the epicenter experienced severe damage than the cities closer to the epicenter (Celebi et al. 1987). It has been observed in many instances that structures at a particular locations suffered same damage during successive earthquakes at a given region. Therefore, it may be possible to predict the typical damage due to any seismic event at a particular location considering the local soil stratigraphy.

Local seismic amplification and the soil liquefaction are two of the seismic effects, which cause potential damage in many past earthquakes. Abayakoon (1998) analyzed the seismic responses of sub-series in low-lying areas in Colombo and found that one of these sub series is more likely to show some seismic amplification and two other series tend to damp down the effects of base rock motion by about 50%. However, more detailed analysis was recommended for further study. During the current study sand profiles were observed in most of the Colombo municipal area and some other urban areas along the Galle road, some of these locations having sand deposits with small SPT values which are more likely to experience soil liquefaction in case of an earthquake.

The current research involves four major components; (1) a study of the seismic history of the region, (2) an analysis of the local soil stratigraphy and the determination of the soil dynamic parameters, (3) designing of an imaginary earthquake having the characteristics of the motion most likely to occur and (4) the evaluation of the characteristics and the magnitude of possible local seismic effects.

Results of this analysis will be an indicator of the seismic performance of Greater Colombo and it will provide necessary knowledge to the structural engineers to make appropriate allowance in their structural design practice.

SEISMIC HISTORY OF THE REGION

On 7-12-1993 at 2.24 am, a seismic event was felt in most parts of Sri Lanka, triggered by a 4.7 Richter magnitude earthquake with epicenter located 170 km west of Colombo. Further, in the same year, ten thousand people died in Latur, Maharashtra, India in a supposedly Stable Continental Region (SCR) from an earthquake of magnitude 6.2 in Richter scale. It is also very important to point out about two very recent seismic events in Southern India with magnitudes 5.0 and 4.3 occurred on 12-12-2000 and 29-01-2001 respectively. The first one was felt in most parts of Southern India. One of another very recent event felt in Sri Lanka was an earthquake of magnitude 5.9 in Richter Scale and the epicenter located at 0^0 55' 48" N, 82^0 30' 36"E, about 725 km South South East of Colombo (USGS), which occurred on 21-09-2001 and the earth tremors were felt in Kalutara, Panandura, Moratuwa, and other southern coastal regions.

Vitanage (1995) described an earthquake, which occurred on 10-09-1938 at 22hrs 23 minutes and 36, 45, 57 seconds respectively, at latitude 6.00 N and longitude 77.04 E. This had a Richter magnitude of 5.6 and the epicentre was less than 75 km from the west coast of Sri Lanka.

Abayakoon (1998) used the data obtained from United States Geological Survey (USGS) and some other locally recorded events and pointed out ten earthquakes with their epicenters within the island area of Sri Lanka and seven of these were close to Colombo. From the analysis it was stated that Colombo would be more vulnerable to a future earthquake than any other part of the country. Two previous seismic events in 1615 and 1814, which have caused severe destruction in Colombo and Batticaloa respectively, have also been mentioned.

The one which cause damage in Colombo Fort (1615 April 14, 7.00 pm) had killed thousands of people and destroyed hundreds of houses; this first documented earthquake in Sri Lanka, had an estimated magnitude of 6.5 on Richter scale (Vitanage 1995).

Considering on the above information the current study was focused on detailed analysis of an earthquake of magnitude 6.0 in Richter scale.

LOCAL SOIL STRATIGRAPHY OF THE STUDY AREA

About 250 boreholes of 130 locations situated within Greater Colombo are used to determine the stratiagraphy of the study area. From the borehole log analysis it was found that the depth of the top soil layers is in the range of 5 m to 25 m and based on the dominant soil type the locations can be classified into three main groups as locations dominated by sand profiles, clay layers and peat deposits.

Sand profiles were observed in most part of the Colombo Municipality and some other adjoining urban areas along the Galle Road up to Moratuwa and the clay deposits were observed as isolated pockets within Colombo Municipality except near Borella where sand as well as clay layers have been observed. Rajagiriya, Malabe, Nugegoda, Nawala, Athurugiriya, Orugodawatta, Negombo, J-ela, Etul-kotte, Boralesgamuwa, Wattala, Kalniya are some other locations where sand and clay profiles were observed. The peat deposits were generally observed in the low-lying areas of Greater Colombo (some of these areas are now reclaimed) namely Peliyagoda, Orugodawatta, Blomendhal, Yakbedda, Nawala, Battramulla, Maligawatta, etc. (Senenayake 1996)

Further, based on the SPT values the areas dominated by sand deposits can be identified as loose and dense profiles and clay deposits as soft, stiff and hard profiles (Das 1990).

Other than the 3 dominant soil types mentioned, gravel was observed rarely in some of the locations within the study area.

Strata	Thickness	Shear wave Velocity/ ms^{-1}	Unit weight / KNm^{-3}
Weathered Rock	15m	750	22
Rock	-	2500	25

Table 01

In most of the locations the soil profile end up with the weathered rock for which not enough details found in Sri Lankan literature. Table 01 presents parameters used to characterize the bed rock, are assumed based on the typical values given in the literature (Morochik et al. 1998, Barnes 1995)

DETERMINATION OF RELATIVE DENSITY OF SOIL PROFILES

The density of the soil profile is one of the key input parameters required for the equivalent linear surface response analysis, generally determined form the tube samples taken throughout the borehole log or by means of field density tests.

One of the alternative ways is to use the SPT values with empirical relationships to determine the relative density, for sand profiles.

For the case of clayey soil Das (1990) relates the SPT value with the consistency of the clay while stating it as an approximate correlation. Beck et al. (1974) reports a correlation between the SPT values and the relative density of clay, whilst pointing out the relationship as rather unreliable.

Considering the extend of the study area and the number of data points required, it was decided to use the SPT for clayey soils as well as sandy soils. Therefore, results from the analysis will be more reliable for the locations dominated by sand profiles than clay profiles.

Some of the other soil data determined from the borehole log information are given in Table 02.

Parameter	Sandy soil		Gravel		Clayey soil		
	Min	Max	Min	Max	Min	Max	Avg
Unit Weight / kNm^{-3}	16	22	16	22	16	22	-
Void Ratio	-	-	-	-	0.3	2.0	-
Plasticity Index	-	-	-	-	-	-	15

Table 02

DETERMINATION OF SOIL DYNAMIC PROPERTIES

Maximum shear modulus values and the modulus reduction and damping curves are the most important parameters required for the analysis of local seismic effects. There are many field and laboratory techniques available for the determination of the above-mentioned parameters, all these require special equipment, which are not available in Sri Lanka. In the current study, the empirical relationships proposed by Seed et al. (1986) has been used to determine the maximum shear modulus values for sand and gravel and another empirical proposed by Hardin and Drnevich (1972) and later Das (1992) has been used for clayey soils assuming that the clay profiles are normally consolidated.

The in-situ densities were calculated by interpolation using relative density values obtained using SPT values.

For peaty soil at low strain (10^{-4}), G/S_U has been extrapolated from Seed et al. (1972) relationship for peat. Su has the range of 6 to 12 kNm^{-2}, corresponding SPT range is 2 to 5 (Aziz 1986) and the unit weight was averaged from the data given by Ray et al. (1986).

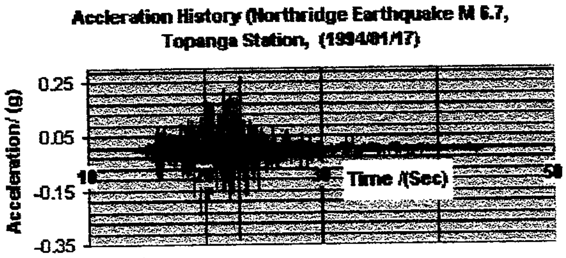
The modulus reduction and damping curves used for different strata are given in Table 03

Strata	Modulus Reduction Curve	Damping Curve
Sand / Clay	Seed et al. (1970) sand average	Seed et al. (1970) sand Average
Gravel	Seed et al. (1986)	Seed et al. (1986)
Peat	Bounglar et al. (1998) Sherman Island Peat	Bounglar et al. (1998) Sherman Island Peat

Table 03

DESIGN EARTHQUAKE

The earthquake motion (Graph 01) recorded at Topanga station during the 1994 Northridge Earthquake in USA has been modified to earthquake of magnitude 6 in Richter scale having epicentral distances of 20 and 40 kms.



Graph 01

Peak ground acceleration (Seed et al.1969), predominant period (Seed et al. 1969), bracketed duration (Karmer 1996) and number of significant cycles (Kramer 1996) are some of the basic parameters, which characterizes the earthquake motion based on the magnitude and the epicentral distance.

M	Epicentral Distance (D) / (km)					
	20			40		
	t_B	t_P	a_{max}	t_B	t_P	a_{max}
6	7	0.24	60	3	0.24	37

Table 04

Legend
M - Magnitude /(Richter Scale
t_B - Bracketed duration/(Sec)
t_P - Predominant Period/(Sec)
a_{max} - Peak Ground Acceleration/(gal)

The attenuation relationship proposed by Seed et al. (1969) has been used to determine the peak ground acceleration at the local site. The summaries of the characteristics of the design earthquake motions have been given in Table 04.

SURFACE RESPONSE ANALYSIS

The equivalent linear response computer program ProShake (EduPro Civil systems,Inc 1998) which has similar performance of that of Shake91 (Idriss.I.M et al. 1967) a computer program for conducting equivalent linear seismic response analysis of horizontally layered soil deposits, has been used to analyse the surface responses of the design earthquake motion input at the bed rock.

Thickness of the layer, unit weight, maximum shear modulus, modulus reduction curve, damping curve, depth of water table and the bedrock motion (acceleration vs. time) are the input parameters for the program.

Based on the type of the dominant soil strata, average SPT value of the location, and the total thickness of the soil profile, twelve locations having different soil stratigraphy were selected to carry out surface response analysis. Table 05 summarizes the soil strata details at the selected locations.

No	Location	Depth to bed rock/ (m)	Dominant Soil Profile
01	Dr. Wijewardene Mawatha , Col-10	12	LS, SC
02	George R de silva Mawatha, Col-13	06	VL
03	Ananda Mawatha, Col-05	13	P, LS, SC
04	Wije wardene Mawatha Col-02	12	VL
05	Dharmapala Mawatha, Col-03	15	LS, SC
06	Near Duplication Road and Krulapone, canal cross, Col-06	21	VD, DS
07	Near Musaeus College, Col-07	22	DS, SF
08	Reid Avenue Col-07	16	DS, SF
09	Near Seewalli Maha Vidayala Col-08	12	SF
10	Near Kalani tissa power station,	20	P, SF
11	Near Colombage Mawatha, Nawala	16	MF
12	Paliyagoda	22	LS, P, SF

Legend

LS	- Loose Sand	VL	- Very Loose Sand
DS	- Dense Sand	VD	- Very Dense Sand
SC	- Soft Clay	SF	- Stiff Clay
P	- Peat	MF	- Medium Stiff Clay

Table 05

RESULTS

No	D= 20 km				D= 40 km			
	a_{max}/ (gal)	A(a)	t_P	t_B	a_{max}/ (gal)	A(a)	t_P	t_B
03	21	0.35	1.7	-	14	0.38	1.6	-
04	27	0.45	1.1	-	21	0.57	1.1	-
12	41	0.68	1.6	-	27	0.73	1.6	-
05	63	1.05	0.4	3.9	44	1.19	0.7	-
02	72	1.20	0.1	3.2	57	1.54	0.1	0.2
10	82	1.37	0.5	7.2	66	1.78	0.5	0.5
11	83	1.38	0.8	4.8	60	1.62	0.2	2.3
01	94	1.57	0.4	3.9	66	1.78	0.4	1.9
09	117	1.95	0.5	6.5	85	2.30	0.5	3.9
08	130	2.17	0.2	6.3	88	2.38	0.6	3.7
06	160	2.67	0.2	8.2	103	2.78	0.2	3.9
07	169	2.82	0.5	8.2	135	3.65	0.5	4.6

Legend : A(a) - Amplification ratio of input acceleration.

Table 06

JUSTIFICATION OF THE RESULTS

As mentioned earlier the results will be more reliable for locations dominated by sand profiles. However, as sand deposits dominate the study area, the results can be interpreted as accurate, while the assumptions made.

DISCUSSION :

From the analysis it clear that both motions have been modified by a similar trend in all the locations. Four Locations, 7, 6, 8, 9, having stiff and dense soil profiles amplified the motion by 100% to 200%. Three other locations, 3, 4, 12 having loose and soft soil profiles damped down the motion by 30% to 70%.

The seismic motions of maximum acceleration more than 100 gal are identified as having potential to cause damage. Therefore, the structures around locations 7, 6 and other locations having similar soil strata in the study area, are more vulnerable to any seismic events of magnitude 6.0 in Richter scale and having less than 40 km epicentral distance and for locations 8 and 9 the potential zone reduces to an epicentral distance of 20 km. The predominant period of the predicted motion in locations 6 and 8 is around 0.2 sec, is close to the natural period of 1 to 3 story buildings and that of locations 7 and 9 are close to the natural period of 4 to 6 story buildings which may be subject to resonance vibration leading to total destruction of the structure.

All the four typical locations considered above are heavily dense or stiff soil profiles. Therefore, it can be predicted that there will be no soil liquefaction for any earthquake of magnitude 6 or less and the epicentral distance greater than 20 km.

CONCLUSIONS:

The locations which are more vulnerable to any future seismic event if happen, were identified using an approximate analysis for a design earthquake of magnitude 6 in Richter scale and having epicentral distances of 20 km and 40 km.

This will be guidance for the structural engineers and other professionals in the field to make necessary allowance for seismic forces in structural design practices considering the underneath soil stratigraphy.

It is recommended to develop field techniques to measure dynamic parameters of the soil, which will give more representative values of shear modulus and damping factors.

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USGS – United States Geological Survey,
http://www.usgs.gov/

A DCP/CBR Correlation for Sri Lankan Residual Soils

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ABSTRACT : Dynamic Cone Penetrometer (DCP) is an instrument that can be used to evaluate California Bearing Ratio (CBR) value of road pavement subgrade. It is an insitu test that has many advantages over the traditional CBR test, which is laborious, expensive and slow. On the other hand, the test results obtained from a laboratory tests may be questionable because they do not represent the site conditions. Dynamic Cone Penetrometer is simple to use and inexpensive. It also can be used to measure the pavement layer thicknesses. As the DCP results depend on the soil type and the condition, this study tried to develop a correlation between DCP and CBR for Sri Lankan residual soils using a locally produced DCP.

Tests were carried out for two rural road projects under the Central Provincial Council and within the Peradeniya Engineering Faculty premises. The DCP tests were carried out for the selected sections and soil samples were collected from the same locations for laboratory tests, with one undisturbed sample for undisturbed CBR test. Undisturbed unsoaked CBR, disturbed unsoaked CBR, disturbed soaked CBR, moisture content test, particle size distribution test and compaction test were carried out as laboratory tests.

Regression analysis for the results shows that there is a significant correlation between DCP and CBR for Sri Lankan soils. The data was analyzed with linear, logarithmic, exponential and power (log/log) models. Out of the four models, power model is the best model to describe this relationship. Three correlations were established between DCP and disturbed unsoaked CBR, DCP and disturbed soaked CBR and DCP and undisturbed unsoaked CBR for clayey or silty sand. All three equations showed good correlations between DCP and CBR with coefficient of determination (R^2) more than 0.6.

1.0 INTRODUCTION

The most commonly used test to measure the bearing capacity of soils for highway projects is the California Bearing Ratio (CBR) test. It is widely accepted as a measure of material stability through out the world. Many pavement design methods are based on CBR value. Besides the CBR test, various other test methods are also available to evaluate the road condition during as well as after the construction. However, most of these methods are expensive and time consuming. A developing country like Sri Lanka cannot afford these tests for every road project especially for the rural and low traffic road projects. In Sri Lanka most of the rural roads are constructed and maintained by local authorities, which cannot spend a large amount of money for the laboratory and field tests that should be carried out prior to a road construction project as well as in the maintenance work. However a less expensive test can be carried out with an instrument called Dynamic Cone Penetrometer (DCP).

2.0 DYNAMIC CONE PENETROMETER

Dynamic Cone Penetrometer consists of 8 kg weight dropping through a height of 575 mm and a 60^0 cone having a base diameter of 20 mm as shown in Figure 01. The top rod is a 16 mm of diameter stainless steel shaft, which is calibrated to ensure the drop of the hammer is always 575 mm. The bottom rod that is made of stainless steel is 915 mm in length and 16mm in diameter. A steel cone that is attached to one end of the DCP, which can be driven into the soils. The angle of the cone is 60 degrees and the diameter of the base of the point is 20 mm. The diameter of the cone is larger than that of the rod to ensure the resistance to penetration is exerted on the cone without skin friction on the rod.

The penetration of the cone is measured using a calibrated scale. It is possible to measure up to 800 mm depth without an extension rod and up to 1200 mm depth when fitted with an extension rod. It needs three operators, one to hold the instrument, one to raise and drop the weight and another to record the results. The results of the DCP tests are recorded on a field data sheet. Later it can be plotted by hand or processed by a computer. A graph of depth (penetration) versus total number of blows is plotted. Then the plot is divided in to ''best fit'' straight lines. The changing points of the graph show the changing of the soil layers. The gradient of each straight line in mm/blow gives the DCP value of the relevant layer. The CBR value of that layer can be obtained using the DCP/CBR relationship. Transportation and Road Research Laboratory (TRRL), The South African National Roads Agency (CSIR) and various authorities through out the world have developed various computer soft wares for DCP data analysis.

3.0 OBJECTIVE OF THE STUDY

The main objective of the study is to develop a generalized correlation between DCP and CBR for Sri Lankan residual soils using a locally produced DCP and compares it with the available correlations. It is clear that the correlation between DCP and CBR depends on the soil type and the conditions. Sri Lankan soils are described as residual soils, which derived from insitu weathering of parent rocks. No research has been carried out about the validity of the available correlations between the DCP and CBR for Sri Lankan soils. So this study tried to develop a correlation between DCP and CBR for disturbed unsoaked CBR, disturbed soaked CBR and undisturbed unsoaked CBR.

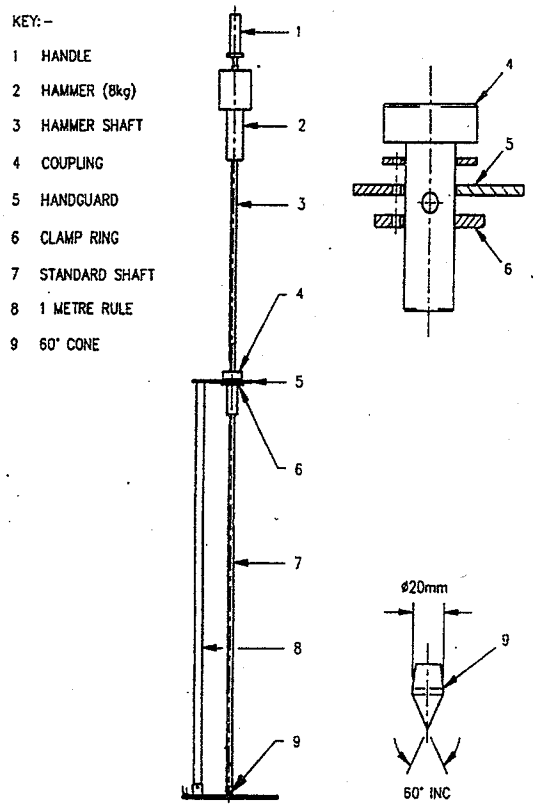

FIGURE 01: DYNAMIC CONE PENETROMETER

4.0 METHODOLOGY

Two rural road projects under the central provincial council were selected for carrying out the tests. They are Katapitiya – Adiyathenna road project and Yatihalagala – Yahalathenna road project. Some tests were also carried out within the Peradeniya Engineering Faculty premises. For this study a Dynamic Cone Penetrometer was produced in the faculty workshop according to the TRRL DCP specifications. The DCP test was carried out for the selected sections of the roads. Soil samples were collected from the same sections for laboratory tests. Undisturbed unsoaked CBR, disturbed unsoaked CBR, disturbed soaked CBR, moisture content test, particle size distribution test and compaction test were carried out as laboratory tests. Undisturbed CBR samples were collected to the standard CBR mould at the site and carefully transported to the laboratory for testing. Precautions were taken to minimize the disturbance on the samples. Disturbed unsoaked CBR samples were prepared at optimum moisture content corresponding to proctor compaction test. The disturbed soaked CBR samples were prepared as the same way and allowed to soak four days in a water bath prior to testing. All CBR tests were carried out according to the BS 1377:Part 4: 1990 specifications. About 30 sets of tests were carried out.

5.0 TEST RESULTS AND DATA ANALYSIS

Simple regression method was used to analyze the data. Following four models were considered.

- Linear
- Logarithmic
- Exponential
- Power (log/log)

The DCP value was used as the independent variable and CBR value was used as dependent variable. Mean Square Error (MSE) and Coefficient of Determination (R^2) were used to find out the goodness of fit. Mean square error (MSE), which is a measure of how well the model fits the data, should be minimum. Coefficient of Determination (R^2) is a standardized measure of the goodness of fit and should be highest. The coefficient of determination (R^2) ranges from 0 to 1. If R^2 is grater than 0.5, the determination is considered as acceptable.

Analysis was carried out to find out the following relationships.

- DCP and Disturbed Unsoaked CBR (DUCBR)
- DCP and Undisturbed Unsoaked CBR (UUCBR)
- DCP and Disturbed Soaked CBR (DSCBR).

The results of statistical analysis with comparison of four models for relationships of DCP/DUCBR, DCP/UUCBR and DCP/DSCBR are shown in Table 01.Figure 02 to 05 show the models graphically for DCP/DUCBR relationship.

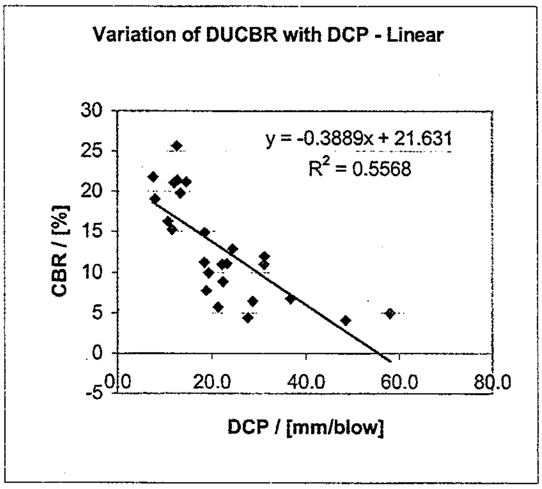

FIGURE 02 : VARIATION OF DUCBR WITH DCP – LINEAR MODEL

TABLE 01: SUMMARY OF STATISTICAL ANALYSIS

CORRELATON	MODEL	EQUATION	MEAN SQUARE ERROR (MSE)	COEFFICIENT OF DETERMINATION (R^2)
DCP/DUCBR	Linear	CBR=-0.3889 DCP+21.631	18.597	0.56
	Logarithmic	CBR =-10.009Ln(DCP) + 42.730	14.042	0.67
	Exponential	CBR = $24.896e^{-0.0351DCP}$	0.117	0.62
	Power	Log CBR = 2.169 – 0.861 Log DCP	0.019	0.68
DCP/UUCBR	Linear	CBR = -0.046DCP + 6.245	0.970	0.48
	Logarithmic	CBR = -1.641 Ln (DCP) + 10.18	0.695	0.63
	Exponential	CBR = $6.317e^{-0.0098DCP}$	0.402	0.50
	Power	Log CBR = 1.145 – 0.336 Log DCP	0.055	0.61
DCP/DSCBR	Linear	CBR = -0.1366DCP + 12.477	11.060	0.41
	Logarithmic	CBR = -5.0985 Ln (DCP) + 24.718	7.400	0.61
	Exponential	CBR = $12.504e^{-0.0169DCP}$	0.135	0.46
	Power	Log CBR = 1.702 – 0.593 Log DCP	0.019	0.61

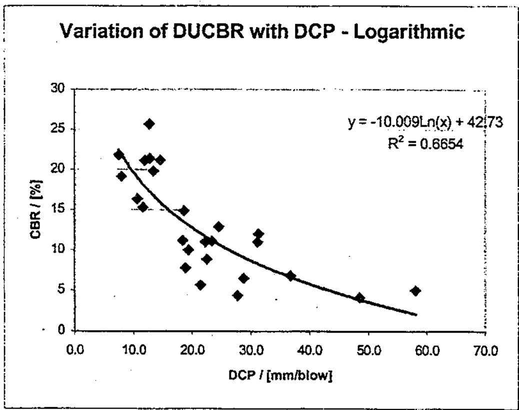

FIGURE 03 : VARIATION OF DUCBR WITH DCP - LOGARITHMIC MODEL

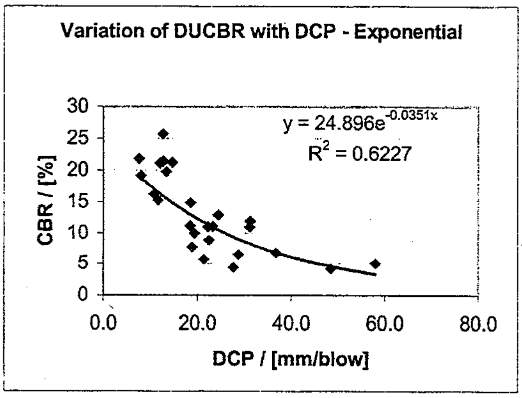

FIGURE 04 : VARIATION OF DUCBR WITH DCP - EXPONENTIAL MODEL

Power model has the smallest MSE and the highest Coefficient of determination (R^2) for the relationship between DCP and DUCBR and the relationship between DCP and DSCBR.

Therefore, out of four models the best-fit model was the power model. For the relationship between DCP/UUCBR, the R^2 value is higher in logarithmic model than the power model by 0.02. But the Mean Square Error is higher in logarithmic model than the power model by 0.64. Therefore, It is also considered that the best-fit model was the power model.

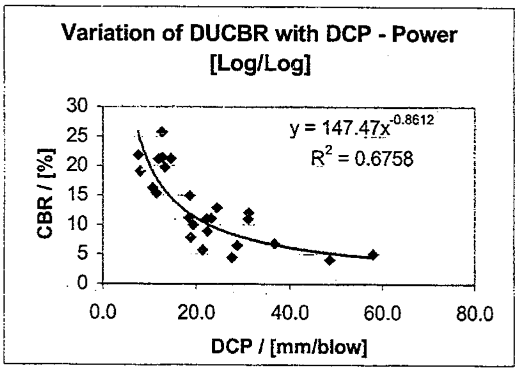

FIGURE 05: VARIATION OF DUCBR WITH DCP – POWER MODEL

The equations derived from the test results are,

- Log CBR = 2.169 – 0.861 Log DCP Equation A, For DCP/DUCBR
- Log CBR = 1.145 – 0.336 Log DCP Equation B, For DCP/UUCBR
- Log CBR = 1.702 – 0.593Log DCP Equation C, For DCP/DSCBR

Where DCP in mm/blow.
The data limit of the equation derived are,

$2 \text{ mm/blow} < DCP < 75 \text{ mm/blow}$

$3 < CBR < 26$

The soil types are Clayey or Silty sand and very clayey or silty sand. The relationships are graphically illustrated in the Figure 06 with both axis in log scale.

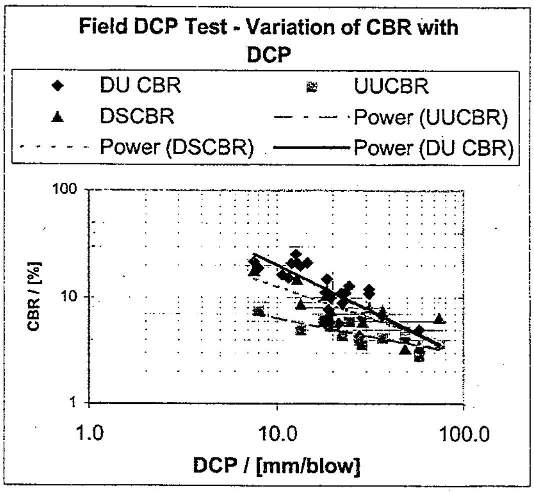

FIGURE 06: VARIATION OF CBR WITH DCP – DERIVED EQUATIONS

6.0COMPARISON OF THE RELATIONSHIPS

The results were compared with the available correlations. There are about 22 equations available for various soil types and derived with DCPs that have various specifications. This study conceded the following seven equations for comparison.

1. log CBR = 2.810-1.320logDCP- Harrison (1986)
2. log CBR = 2.631 - 1.280logDCP - Kleyn (1975)
3. log CBR = 2.390 - 1.260logDCP - Do -
4. log CBR = 2.605 - 1.269logDCP - Do -
5. log CBR = 2.317 - 0.858logDCP - TRRL , 1986
6. log CBR = 2.480 - 1.060logDCP - TRRL, 1987
7. log CBR = 2.580 - 1.310logDCP - Belgium,1980

The equations are use for all soil types. The specifications of the DCP used to derive those equations are the same as the DCP that used for this study. i.e. with 60^0 cone, 8 kg Hammer and 575 mm drop.
The comparison shows that the relationship between DCP/ DUCBR for DCP values in between 25 – 100 mm/ blow, closed to the TRRL equation derived in 1987. The relationship between DCP/UUCBR for DCP values in between 25 – 40 mm/blow closed to the relationship derived in Belgium in 1980. However, for the correlation between DCP/DSCBR could not be find close equations from the available relationships. Anyway the relationship between DCP/UUCBR and DCP/DSCBR equations shows approximately equal values for the DCP values range from 60 – 100 mm/blow. Figure 07, shows the comparison of derived equations A, B and C with the existing correlations for all soil types. Equation 1 to 7 represents the above seven equations respectively.

7.0 CONCLUSION

Regression analysis for the results shows that there is a significant correlation between DCP and CBR for Sri Lankan soils. Out of four models, power model is the best model to describe this relationship. But a better correlation could be obtained by carrying out more tests for wide range of soil types while taking into account the effect of other soil properties like moisture content, dry density etc. This study will extend to find out such a relationship.

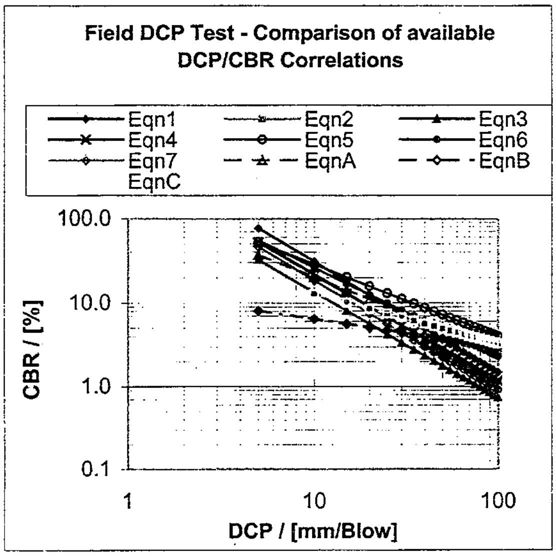

FIGURE 07: COMPARISON OF DERIVED RELATIONSHIPS WITH THE AVAILABLE RELATIONSHIPS

8.0 ACKNOWLEDGEMENT

The two funds provided by the Asian Development Bank (ADB) and the National Science Foundation (NSF – Grant No : RG/2001/E/05) for this research and support provided by Central Provincial Highway Department are gratefully acknowledged.

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UNDRAINED SHEAR CHARACTERISTICS OF A SRI LANKAN RESIDUAL SOIL IN THE UNDISTURBED AND REMOULDED STATES

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ABSTRACT

A series of undrained triaxial compression tests was performed on a residual soil collected from a weathering profile in Kandy to investigate the essential differences in shear behaviour of a residual soil in the undisturbed and remoulded states. All specimens were sheared after isotropic consolidation to different cell pressures. The consolidation tests on undisturbed specimens showed that the soil was overconsolidated in the field to an OCR of about 2.8. The experimental behaviour was predicted by a point element analysis using the Modified Cam-Clay model.

The experimental behaviour in triaxial compression tests illustrated that the residual soil in the undisturbed state behaves as if heavily overconsolidated showing dilatory behaviour, whilst the remoulded soil which were tested after normal consolidation exhibits a contractive behaviour. It is also observed that the remoulded soil produces geometrically similar undrained stress paths with a unique state boundary surface. In contrast, stress paths of undisturbed specimens were not geometrically similar. The stress ratio q/p' at failure remain almost the same for specimens irrespective of the disturbance and consolidation pressure thus giving a unique critical state line for the soil.

The experimental behaviour of remoulded specimens was predicted very well by the Modified Cam-Clay model up to the critical state. However, the characteristics of the remoulded soil after reaching the critical state and the stress- strain behaviour of the undisturbed specimens are not well predicted by the Modified Cam –Clay model. A numerical model which may account for breakage of structure and plastic shear behaviour below the yield surface may be required to improve these predictions.

INTRODUTION

Residual soils, derived from in-situ weathering of rock, develop their fabric, grain structure, and grading which may inherit characteristics from in-situ parent material. They are fundamentally different from transported soils, which develop their fabric as a result of their mode of deposition. In both cases the stress history after formation or deposition may influence the subsequent behaviour.

In Sri Lanka, more than 90% of the land is made up of highly crystalline rocks belonging to the South Indian Shield (Cooray, 1967). The tropical weathering environment accelerates the formation of residual soils which are found almost everywhere in the island. However, these local residual soils have received little attention of the researchers partly because of their inhomogeneity, anisotropy, and difficulties of sampling and testing.

Pannila (1990) carried out a series of percolated drained and undrained triaxial tests on undisturbed samples of a lateritic soil and concluded that the results were in agreement with the critical state concepts. The results also showed the existence of a unique state boundary surface for the drained tests. However, the observed stress- strain behaviour was not well predicted by the Cam-clay model. There is a possibility of partial saturation in the experiments carried out by Pannila due to the limitations of equipment used for testing. Pannila does not give the degree of saturation of the specimens tested and percolation alone is insufficient for complete saturation of a fine-grained soil. Small size samples used in the experiments, 37 mm in diameter, was another limitation of her work.

Seneviratne (1996) indicated that shear strength characteristics of residual soils exhibit dilatory behaviour of heavily overconsolidated samples and shear compression of normally or lightly overconsolidated specimens. Mohr-Coulomb criterion is found satisfactory in describing the failure of residual soils though Hvorslev surface was indicated in failure at low stress levels. Seneviratne (1996), also reported the shear strength parameters obtained from percolated drained triaxial tests carried out on eight, 71mm diameter undisturbed specimens and their classification according to the British Soil Classification System. In the light of the results obtained, Seneviratne points out the differences in the observed shear strength parameters and what can be expected from the soil classification based on the literature, and suggests the use of a classification system based on the degree of weathering to describe the behaviour of residual soils.

The past studies described above indicate that the investigations on shear characteristics of local residual soils were carried out using undisturbed samples. However, it is rational as well as advantageous to study the behaviour of natural soils by using the behaviour of remoulded soils as a frame of reference. (Burland, 1990, Liu and Carter, 1997). The work in the present study has concentrated on triaxial compression tests on undisturbed as well as remoulded specimens, consolidated normally and isotropically. The observed experimental behaviour was compared with the prediction made using the Modified Cam- Clay constitutive model.

EXPERIMENTAL PROCEDURE

A series of triaxial compression tests with measurements of pore pressure was performed on undisturbed and remoulded specimens of a residual soil from a weathering profile located in Kalugamuwa, Kandy. Based on visual examinations the parent rock from which the residual soil is derived was inferred to be Garnet Biotite Feldspathic GNEISS.

The physical properties of the residual soil tested are given in Table 1. Particle size distribution tests were carried out during preparation of undisturbed specimens. They showed almost uniform particle size distribution, According to British Soil Classification System the soil is named as Sandy CLAY of Intermediate Plasticity, CIS.

Table 1.Physical properties

G_s	2.61
$w_L(\%)$	46
$I_P(\%)$	21

The diameters of the undisturbed and remoulded specimens tested in triaxial compression were 50 mm and 38 mm respectively. A length to diameter ratio of specimens was 2.0 in both cases. Larger sized specimens were employed in the case of undisturbed specimens so that some account could be taken of the cementation effects of soil fabric.

Undisturbed specimens were prepared from block samples extracted with extreme care from the site so that "research class" undisturbed specimens were obtained.

Remoulded specimens were prepared as follows; a sample of the required soil mass was mixed to form a slurry with distilled water utilising a motorised rotary mixer for a period of 10-12 hours. The water content of the slurry was approximately twice the liquid limit of the soil. Thereafter, the slurry was poured into cylindrical cells having a diameter of 150 mm and a height of 125 mm. Consequently, they were one-dimensionally consolidated to an effective pressure of 45 kPa, which was approximately the overburden pressure in existence at the depth from which the undisturbed samples were extracted. (Head, 1985)

The specimens were saturated by applying a back pressure. In undisturbed specimens saturation was carried out under a back pressure of 300 kPa until a Skempton's B value exceeding 0.95 was obtained. Remoulded specimens extracted from the moulds were also saturated in the same manner though they were found to be in a near saturated condition.

After saturation the specimens were brought to the required effective stress state by consolidation. The shearing of the specimens were carried out in a strain-controlled manner, with an axial strain rate of 0.08%/min and 0.04%/min for undisturbed and remoulded specimens, respectively. The adopted rates were obtained generally following the standard procedure described by Bishop and Henkel (1957) to ensure a uniform pore pressure distribution throughout the specimen. Pore pressure was monitored at the bottom of the specimen using a pressure transducer to an accuracy of ± 0.5 kPa.

UNDRAINED MONOTONIC LOADING TESTS ON UNDISTURBED SAMPLES

Triaxial compression tests were carried out in a monotonic loading condition on undisturbed specimens, which were isotropically consolidated under effective cell pressures (p_c') of 100, 200, 300 and 400 kPa. The stress-strain and stress path diagrams obtained in the tests are presented in Figs. 1 and 2. In the figures, the effective mean stress is defined by $p'=(\sigma_1'+2\sigma_3')/3$, and the deviator stress by $q=(\sigma_1-\sigma_3)$. Two specimens were tested at each pressures of 100, 200, 300 and 400 kPa and the results were found to be almost identical.

As can be seen from the Figs. 2 stress paths of the soil in the undisturbed state are leaning to the right of vertical showing dilatory behaviour. The deviation from the vertical reduces with the increase of the isotropic consolidation pressure with the reduction of 'over consolidation ratio'. Similar characteristics have been observed by other researchers in Sri Lanka as well as in other countries. eg. Seneviratne (1996), Brenner et al (1997)

UNDRAINED MONOTONIC LOADING TESTS ON REMOULDED SPECIMENS

Figs. 3 and 4 show the stress-strain and stress path diagrams obtained from similar tests to the above carried out on remoulded specimens isotropically consolidated under p_c' of 100, 200, 300 and 400 kPa. The relationship between deviator stress and axial strain at different consolidation pressures were found to be geometrically similar as described by Atkinson and Bransby (1977).

In contrast to the undisturbed specimens remoulded specimens show contractive behaviour. This of course is expected for isotropically normally consolidated specimens. The stress ratio q/p' at failure of specimens remained at a constant value.

MODEL PARAMETERS

Five parameters M, λ, κ, ν and e_{cs} are needed to define a Modified Cam – Clay model for a given soil. In this model the behaviour below the yield surface is assumed to be elastic with bulk modulus varying in proportion to elastic mean normal stress.

M, the gradient of the failure line was obtained from the triaxial test results. It could be observed from Fig.5 that the critical state line for all specimens to be almost unique. Strain parameters λ and κ were derived from Oedometer test data conducted on 75 mm diameter undisturbed and remoulded specimens. In the case of undisturbed specimens, Oedometer test results showed a 'virtual pre consolidation pressure' of 175 kPa which corresponds to a yield stress p_c' of 152 kPa with a K_o value of 0.79 for Modified Cam - Clay. In the elastic response of Modified Cam – Clay model, shear modulus was calculated using bulk modulus and a constant Poisson ratio. Poisson ratio ν was varied in the analysis to obtain the best predictions. The voids ratio at unit stress on the critical state line (e_{cs}) was obtained from the undrained test data. Model parameters for undisturbed and remoulded samples were calculated separately and are given in Table 2.

Table 2. Model parameters

	Remoulded	Undisturbed
M	1.1	1.3
λ	0.142	0.125
κ	0.016	0.010
ν	0.39	0.39
e_{cs}	1.34	1.23

COMPARISON OF EXPERIMENTAL AND ANALYTICAL BEHAVIOUR

A single point analysis was carried out with Modified Cam-Clay model. The analysis was performed using a computer program specifically written for this purpose. Several sub - routines from **A F**inite **E**lement program for Numerical Analysis, AFENA (Version 5.0,1995) developed at the University of Sydney, Australia were used in the program.

Figs. 6 and 7 illustrate the comparison between observed and predicted behaviour of remoulded specimens. It is clear from the results that both predictions and observations show similar behaviour except after critical state, where the deviator stress seems to either increase or decrease. In other areas the agreement between the observed and predicted behaviour is very good in most instances. The characteristics after reaching the critical state may be due to either the effect of softening due to swelling or hardening due to drainage as described by Schofield and Wroth (1968). These characteristics may be numerically simulated as described by Asaoka et al (1994).

Figs. 8 illustrates the comparison between observed and predicted behaviour of the undisturbed specimens in the triaxial tests. It can be seen from Fig.8 that the behaviour of the undisturbed specimens is not well predicted by the analyses. Stress paths at 300 and 400 kPa consolidation pressures are reasonably acceptable at low stress levels. However, in all cases the behaviour at high stress level is totally unsatisfactory. Instead of the dialatory behaviour exhibited by the experimental specimens the analytical results show compressive behaviour.

There are several reasons for the above disagreements. It is well reported in the literature (Asaoka, 2000) that collapse of structure and particle realignment of a natural soil will cause hardening with volume expansion due to the remaining overconsolidation. Even at low stress levels the concept of virtual pre - consolidation pressure seems to be inadequate in describing the experimental behaviour particularly at high OCRs. A numerical model, which may account for breakage of structure and plastic shear behaviour below the yield surface may be required to improve these predictions.

CONCLUDING REMARKS

A series of undrained triaxial compression tests was performed on undisturbed and remoulded states of a residual soil from Kandy area. All specimens were isotropically normally consolidated before shearing. The conclusions made from this study are summarised below.

- The residual soil tested in the undisturbed state behaves as if heavily overconsolidated showing dilatory behaviour, whilst the remoulded soil exhibits contractive behaviour.

- Remoulded specimens produce geometrically similar undrained stress paths as explained by the critical state concepts with the uniqueness of state boundary surface.

- The stress ratio q/p′ at failure remain almost the same for the residual soil irrespective of the disturbance and consolidation pressure.

- The behaviour of residual soil in the remoulded state is well predicted by the Modified Cam-Clay Model up to the critical state. However, the stress-strain characteristics of the remoulded soil after reaching the critical state and the behaviour undisturbed soil are not well predicted by Modified Cam –Clay model.

ACKNOWLEDGEMENTS

The work described in this paper forms part of an overall research programme investigating the stress-strain behaviour of Sri Lankan residual soils. The author would like express his sincere gratitude to Prof. Nimal Seneviratne for his valuable guidance and untiring advice offered during the course of study. The author also extends his heartfelt gratitude to Dr. Sarath Fernando for his valuable advice concerning the experimental programme. He is also grateful to Dr. Chandana Kurukulasuriya for his kind assistance at any time it was sought for. Financial assistance received through Science and Technology Personnel Development Project, ADB Loan No. 1535 - SRI (SF) is gratefully acknowledged. The author also wish to acknowledge the help given by the technical staff of the Geotechnical Laboratory, Department of Civil Engineering, University of Peradeniya for their support in conducting the experiments.

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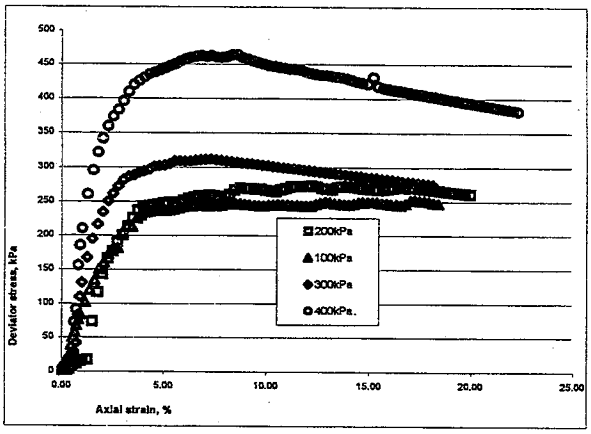

Fig.1. Stress strain behaviour of undisturbed specimens

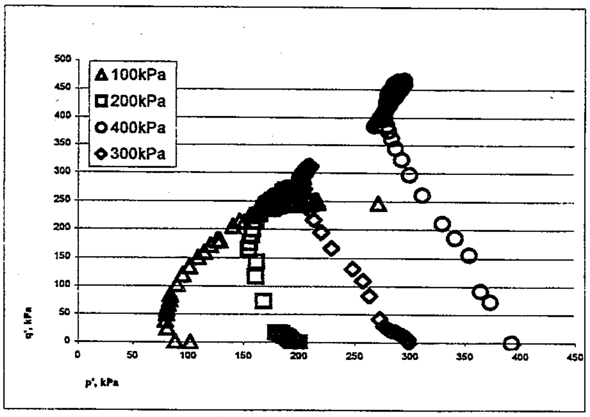

Fig.2. Stress paths obtained for undisturbed specimens

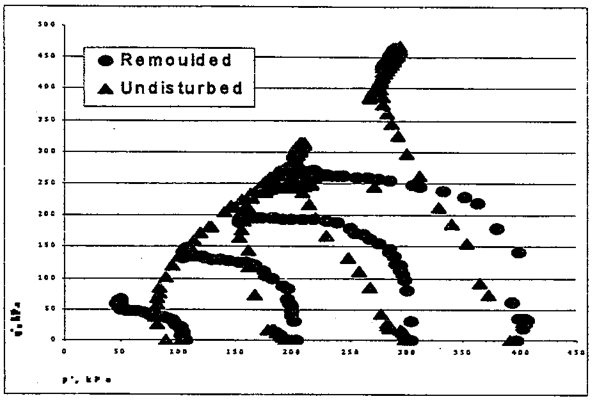

Fig.5. Stress paths obtained for both undisturbed and remoulded specimens

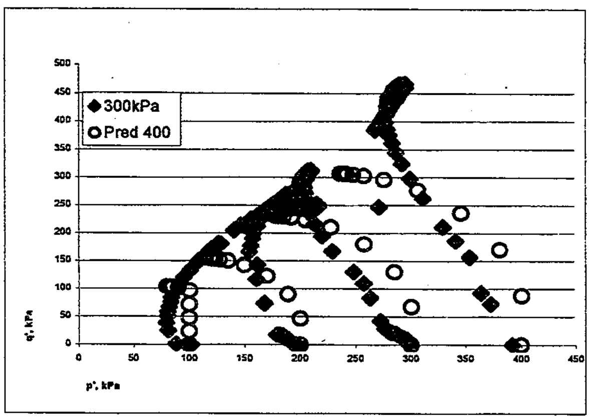

Fig.8. Experimental and predicted stress paths of undisturbed speceimens

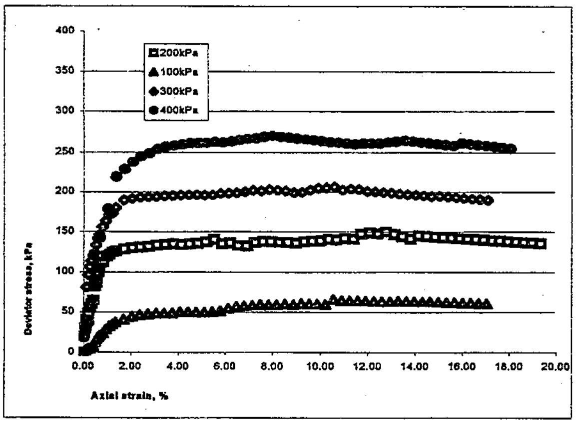

Fig.3. Stress strain behaviour of remoulded specimens

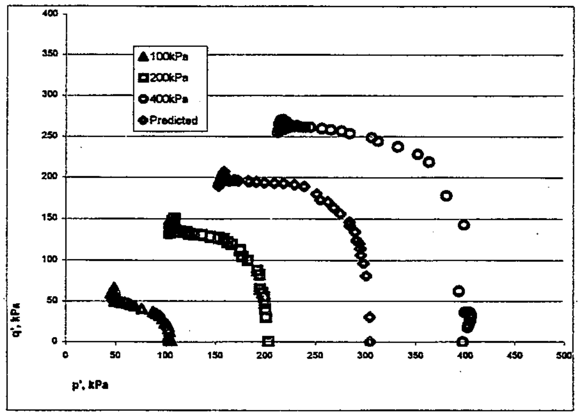

Fig.4. Stress paths obtained for remoulded specimens

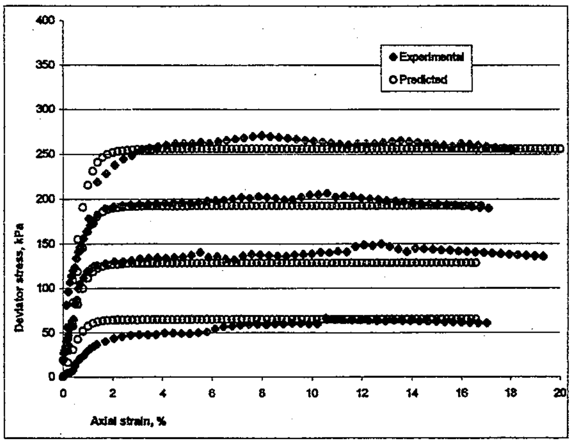

Fig.6. Experimental and predicted stress-strain curves of remoulded speceimens

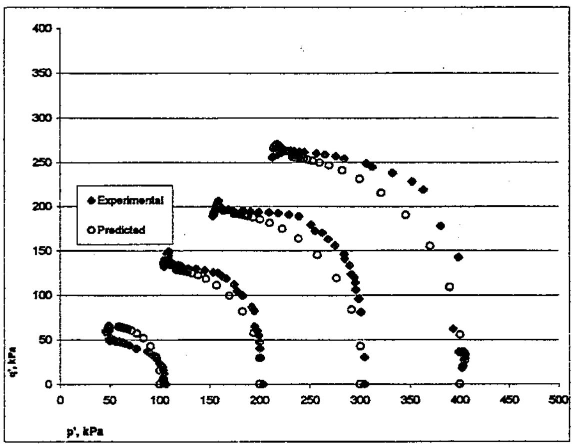

Fig.7. Experimental and predicted stress paths of remoulded speceimens

A STUDY TO DEVELOP A CLASSIFICATION SYSTEM FOR INSITU WEATHERING PROFILES OF GNEISSIC ROCKS IN SRI LANKA

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SYNOPSIS

The literature published for field identification of weathering profiles in tropical humid conditions is scarse. Many of the available classification systems are not applicable in identifying Sri Lankan weathering profiles. Therefore, a good classification system to recognise Sri Lankan weathering profiles is important for field identification and site investigation purposes.

A project was undertaken to find a suitable classification system for Wet Zone weathered rock profiles in Sri Lanka. At initial stage only Gneissic rocks in Wet Zone were investigated. Among the sixteen classification systems studied Little's classification (1969) was selected for field identification of different weathering grades prior to the detailed study. The visual properties of different decomposition grades were then be compared with Little's classification. The study showed that with some modifications specially in Completely Decomposed, Highly Decomposed and Moderately Decomposed grades Little's Classification may be used to classify local Gneissic weathering profiles.

INTRODUCTION

Engineering classification of weathered rock profiles is important and useful for geotechnical engineers and geologists in solving problems associated with many engineering applications. A proper investigation of field material is essential for formulation of design problems. A number of classification systems for field identification of weathering profiles have been developed and are available in the literature. In many cases classifications developed for field identifications are based on colour change, discontinuity patterns and excavation methods. Some others were described with the aid of index properties of soil/rock. Among the classification systems only a few systems were intended for specific rock types.

Classification Systems

Many classification systems were developed in the past few decades by geological, geotechnical and agricultural based organisations and researchers. Some of them were developed for special engineering projects thereby narrowing down their general applicability. There are many classification systems published to recognise weathering profile of igneous and sedimentary rocks. However for identification of metamorphic rock profiles only a few classification systems are available in the literature. Most of the classification

systems are divided into six grades of weathering namely Residual Soil (RS), Completely Weathered (CW), Highly Weathered (HW), Moderately Weathered (MW), Slightly Weathered (SW) and Fresh rock (FR).

Moye(1955) was the first investigator who developed and proposed a grading system for the degree of weathering rock profiles found in granite at the site of Snowy mountains scheme in Australia. Ward (1968) developed a classification for Middle Chalk at Munford by dividing it into five grades with their characteristic properties. In 1969 Little published a weathering classification of tropical residual soil, which was the only successive system available for tropical climatic zone. However, due to poor description of boundaries along the profile and also due to difficulties in identifying decomposition grades limit the use of Little classification for applications in Sri Lanka. Fookes & Horsewill (1969) divided the classification into four catogaries namely soils (and soft rocks), rocks under chemical weathering, rocks under physical weathering and carbonate rocks. This system cannot be used to clearly identify weathering profiles in tropical humid climatic regions. Hobbes (1969) published a classification system by describing engineering properties of weathering grades. However, there is little use of Hobbes system in field identification. Dearman (1974) published an ideal profile of weathering which is irrespective of rock types. However this classification is not descriptive and does not represent profiles found in Sri Lanka. Anon(1981) , Hencher and Martin(1982), Bell and Petiga (1984) , Rolando (1985) , Lawrence (1985) and professional organisations/standards namely IAGE (1981) , Geotechnical control office (1984) and BS 5930(1981) give weathering classifications in field identification of different weathering grades. However these classification systems do not represent the weathering profiles in Sri Lanka.

The following common facts were noted among the weathering grades in different classification systems. The difference between residual soil (grade VI) and completely weathered rock (grade V) is the original texture. In many classifications residual soils are described with the term 'original texture destroyed'. However, the original texture in completely weathered rock is preserved. The percentage of soil content in highly weathered rock is less than that of moderately weathered rock. The discolouration can be seen in slightly weathered rock along discontinuities but they cannot be seen in fresh rock. The discontinuities can be present in fresh rock but in micro scale.

Bell (1983) stated that local conditions and particular rock types have a marked influence on the development of weathering profiles. Therefore a good classification system to recognise local weathering profiles for different rock types is important. With the scarcity of the literature on weathering classification of metamorphic rocks in humid tropical climates and also due to poorly define grades of decomposition of them, this study is proposed to develop a suitable classification system to identify local Gneissic profiles in the Wet Zone.

GEOLOGY AND GEOMORPHOLOGY OF THE AREA

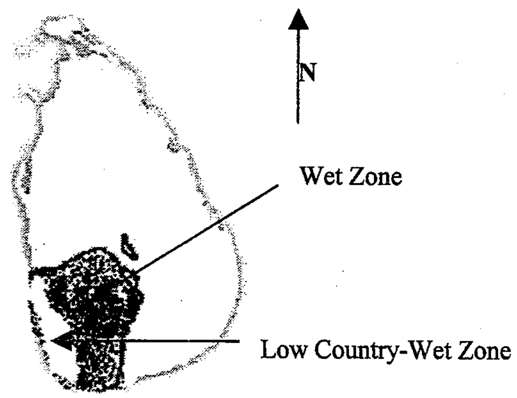

Fig 1: The Study Area

Sri Lankan rocks are part of the Indian shield which is a portion of most of the stable part of the earth crust. Most of these rocks are highly crystalline and nonfossiliferous. Main rock formations of the island belong to Quaternary, Tertiary, Cretaceous, Jurassic and Precambrian age. About 90% of Sri Lankan rocks are metamorphic and can be found in Highland series, Vijayan complex and Southwestern group (Cooray1984). These three groups are during the Precambrian period. The Highland series, so called because these rocks underlie most of the higher terrain of the island, is composed of interbanded metamorphosed sediments (quartzites, marbles, and garnet-sillimanite schists) and charnockite gneisses of volcanic-origin. Both of the above general rock types can be found intermixed throughout the Highland series.

The present study was carried out in Wet Zone of Sri Lanka (Fig 1), where many of the rocks are Gneisses - the banded high grade metamorphosed coarse-grained rocks and metasediments. Hornblende Biotite Gneiss, Garnet Biotite Gneiss, Garnet Sillimanite Gneiss, Migmatite Gneiss, Quartzo Feldspathic Gneiss and Charnockitic Gneiss are the most dominant metamorphic rocks in the study area. The chemical weathering is more dominant than physical and biological weathering process in the Wet Zone. A Series of geological uplifts, some strikingly large peaks and escarpments mark the topography of the Wet Zone. Some individual rock types have a notable influence on the geomorphology of the area. The areas where laterites occur specially in low country were not incorporated in the proposed classification system.

METHOD OF STUDY

Sixteen classification systems developed for field identification and engineering properties were reviewed for this study. Among those classification systems, the classification published by Little (1969) was selected as an initial identification tool as it has been developed for rocks in tropical humid climatic regions. About hundred sites along roads and quarries were investigated for visual properties such as colour, texture, vegetation, geological structures, mineral bands, profile slope etc. Factors, which influence the weathering and stability of weathered materials, were also taken into consideration. Eroded areas, topsoils and organic matters were ignored when considering the profile in the study. The observed properties were then be compared with the Little's (1969) classification system. However the profiles of Charnockitic Gneisses were not included in the proposed classification system due to the difference of their origin from other Gneisses present in the study area.

VISUAL OBSERVATIONS

Careful examination of collected data shows that the weathering patterns of different rock types vary both climatically and topographically within the study area. Their profile thickness, weathering patterns and also engineering properties show significant variation with climatic changes. Most of the Gneissic profiles include all grades of decomposition and many of them are overlain by weathered layers of impure quartzites. Well-foliated rocks show comparatively thicker profiles than faintly foliated rocks. However in contrast, Quartzo Feldspathic Gneisses show deep profiles due to abundance of cracks in the rock mass, through which water percolates easily leading to an extensive weathering. As can be expected Profiles near mountainous crests are thinner than those found in valleys.

A Careful examination of residual soil from different sites shows no signs of its origin. Also Colour of residual soil is not unique for specific rock type and instead varies largely with climate. Completely decomposed rock of Gneissic profiles have preserved their original texture. Highly decomposed rocks have comparatively higher soil content than moderately decomposed rocks. Slight discolouration can be observed in slightly decomposed rock. Bands are visible in all grade of decomposition except residual soil and they are clear towards the fresh rock. Major minerals like Biotite, Hornblende, Hypersthene, Quartz, Feldspar and Garnet can be observed in fresh state in Gneissic fresh rocks.

RESULTS AND DISCUSSION – MODIFIED LITTLE'S (1969) CLASSIFICATION

The study reveals that out of the classification systems considered Little's system is the most suitable for describing local weathering profiles. However based on the field observations certain modifications are proposed to the Little's system to identify Sri Lankan weathering profiles in the Wet Zone. Differences between the original Little's classification and the proposed system for gneisses are given below.

General: The proposed classification system intended for the weathering profiles of gneissic origin where as Little's classification was based on rocks of granitic origin. Also any descriptive mineralogical and textural properties are not incorporated in Little's classification. The following observations, which are not present in Little's classification, have been identified in the proposed system. Alternative bands are visible in all grades of decomposition except the grade of residual soil. Partially weathered mica is more common on RS, CD and HD grades and generally quartz particles are visible within these grades.

Residual Soil: As stated in all most all classifications "Residual Soil" is defined in similar words: "original rock texture is completely destroyed"
This is true for Sri Lankan residual soils. Residual soils of gneisses are mainly composed of silt and fine sand. They can be easily crumbled by hand.

Completely Decomposed Rock: it has been observed that the original texture of local rocks of gneissic origin is preserved as in the case of rocks of granitic origin. This is completely a soil mass (major fraction is composed of silt and sand) and they can be easily crumbled by hand.

Highly Decomposed Rock: This grade of rock is discoloured from parent rock and soils contain coarse to medium sand with less fine fraction. Highly decomposed rocks can be easily broken along foliation planes.

Moderately Decomposed Rock: Bonding of the alternative layers is fairly weak and therefore it is not necessary to use explosives to break the rock mass. Undisturbed cores may be recovered by careful drilling of the rock mass.

Slightly Decomposed Rock: Slight discolouration can be easily observed along foliation planes. Garnet appears in pale violet (slightly discoloured) in Garnet bearing Gneisses.

Fresh Rock: Similar as other classifications.

Table 1 gives the proposed modified Little's (1969) classification system for Sri Lankan Gneissic weathering profiles.

CONCLUSION & RECOMENDATIONS

a). Among the classification systems reviewed the system proposed by Little (1969) is suitable than others to represent the weathering profiles in tropical humid climatic conditions found locally.

b). With certain modifications Little's classification system can be used to identify different weathering grades of Gneissic rocks in Sri Lanka.

c). The study reveals that it is a necessity to develop different classification systems to recognise weathering grades of different rock groups for different climatic zones in Sri Lanka.

d). Further studies are to be carried out to incorporate engineering properties to the proposed classification system for different grades of decomposition.

ACKNOWLEDGEMENTS

The advice given by Mr. Udeni Amarasinghe, Department of Geology and Mr. G.S. Gurusinghe, Department of Civil Engineering, both of University of Peradeniya through out the research is gratefully acknowledged. Also the authors would wish to acknowledge the fund support given by Asian Development Bank through the Science and Technology Personnel Development Project-loan no 1535-SRI (SF).

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Table 1 :Classification of Weathered Gneissic rocks in Sri Lanka

Grade	Degree of decomposition	Field Recognition
VI	Residual Soil (RS)	Surface layer contains roots and organic matter. No recognisable rock texture or structure. Totally discoloured from parent rock. Most of the primary minerals fully weathered (Feldspar decomposed to clay minerals). Sometimes partially weathered mica can be observed. Silt fraction is very high. Can be easily crumbled by hands. Finger pressure is enough to penetrate into constituent grains.
V	Completely Decomposed (CD)	Original rock texture is preserved. Rock completely decomposed to soil. Totally discoloured from parent material. Bands are visible. Feldspars completely decomposed to clay minerals. Unweathered mica and Quartz gravel can be observed. Black Biotite flakes are visible in most of the Biotite bearing gneisses. Can be excavated by hand but cannot be extracted as pieces. Easily pick by geological hammer. At moist condition finger pressure is just enough to penetrate into constituent grains.
IV	Highly Decomposed (HD)	Original Rock texture is preserved. Discoloured from fresh rock. Both soil and rock are composed. Bands are much clear .Can be excavated by means of a geological hammer. Fairly large pieces can be extracted carefully. Can be crumbled by applying a force by hand.
III	Moderately Decomposed (MD)	Original Rock texture is largely preserved. Discoloured from fresh rock. Contain very little amount of soils. Considered as a rock. Bands are very clear. Large pieces can be broken. Can be excavated means of a geological hammer but cannot be crumbled by hand. When hammered make poor sound.
II	Slightly Decomposed (SD)	Original Rock texture is largely preserved. Contains totally rock fraction though there may be very little amount of soil along fracture planes. Discoloured from fresh rock. Cannot be broken by geological hammer except to chip like pieces. Need explosives to break. Strength approaches that of fresh rock. When hammered make ringing sound.
I	Fresh rock (FR)	No discolouration. Mineral can be observed as in fresh state. Minerals bands are visible. Need explosives to break. When hammered make ringing sound.

Deformation Analysis in Model Soil Slope Studies

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ABSTRACT

Determination of deformations that occur at various locations of the soil body during failure of soil slopes is significant for geotechnical engineers and a method developed to determine such deformations can be applied in many model soil slope studies. A method developed for the above purpose with the help of a rubber membrane is discussed in this paper.

A rubber membrane, on which a grid pattern was initially drawn, was placed in between soil and the Perspex sheet of the model box, ensuring the movement of membrane with soil as the slope failed under applied load. The deformations that occurred in the membrane were captured with a digital camera on photographs at different loading increments and by digitizing the photographs, coordinates of the nodes were found. Having converted these coordinates into actual coordinates on the model relative to a selected base, they were used to find the deformations of node points; i.e. the deformations of the corresponding points on the soil slope. Deformation patterns were then plotted to observe the magnitudes and the directions of the displacements of nodes, between different loading stages. Strains occurring in the soil body were evaluated.

1. Introduction:

In soil slope studies, most of the time, geotechnical engineers are interested in determining the failure plane and the shear stress developed along the failure plane. It is also important for engineers to establish the strains developed and the deformation patterns at various locations of the soil body as the slope fails.

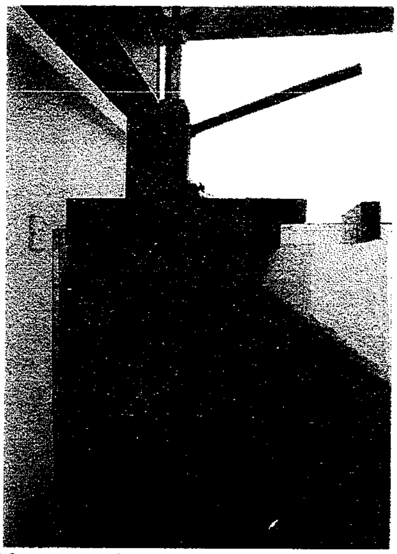
2.Background:

Strain is a measurement of deformations. In classical solid mechanics, the normal strains and shear strains are defined (Timoshenko and Goodier, 1970). These definitions are incorporated in conventional finite element programs, along with the concept of displacement interpretation functions to define the distribution of displacement over the described domain. For the determination of strains, a convenient technique is to use the strain displacement relationships already incorporated in finite element analysis. A finite element program originally developed by Prof: R.L.Taylor (Zienkiewicz, 1977) and later modified and installed in personal computers by Dr. U.G.A. Puswewala at Moratuwa University was used to derive strain distribution within the soil mass, when displacements at nodal points were input as data.

3.Methodology:

A model soil slope was prepared in a box made up of transparent, perspex sheets. As the soil slope was constructed, a rubber membrane was placed between soil mass and a wall of the box. Prior to placing the membrane inside the box, a grid pattern was drawn on it, and the inward face of the membrane touching the soil mass was roughened, by pasting sand. The purpose of roughening the membrane was to ensure the movement of membrane with the soil, and the grid was used to determine the displacements at each point of the soil mass, as the slope failed. The grid pattern on the membrane represented the elements and node points require to discretise the soil domain.

Loading the soil slope was done, by jacking against a loading frame. The setup of the soil slope and the loading arrangement are shown on the Photograph 3.1.



Photograph 3.1: Set up of the soil slope and Loading Arrangement

As the load was increased, the deformations that took place within the soil mass could be seen by the distortion of the grid pattern drawn on the membrane. The distortion of the membrane was captured on photographs taken with a digital camera, at different instances while increasing the load. The camera was stationed at the same point for all photographs.

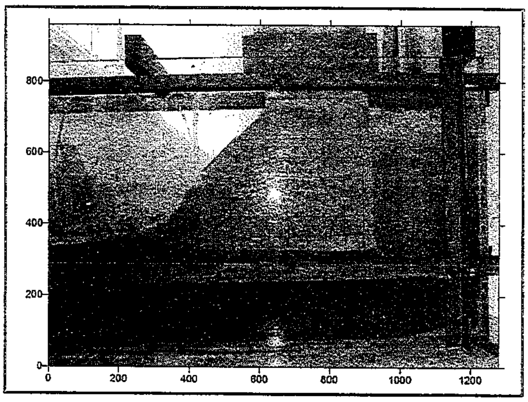
The photographs so taken were digitized, by using a computer software, to obtain coordinates of the grid points relative to a fixed point. In this manner, the coordinates of the grid points at several stages of loading were found.

Because coordinates given by the software while digitizing were relative to a coordinate system determined by the software, they had to be converted to actual coordinates on the soil slope. For that three fixed points, which appeared on each photograph as corners of a triangle and that could easily be identified on the model box, were selected. The coordinates of those three points were found with the software and they were used to calculate the distances between those points on the photograph. Then the actual lengths between those three points on the model box were measured. A scale for converting the coordinates given by the software to actual coordinates on the slope was selected, after ensuring that the ratios of the length between two points on the photograph to that on the model box were the same for all three lengths between selected points. Those actual coordinates were used to find the relative movements or the displacements of the grid points; i.e., the movement at various points of the node points of the soil body as the slope failed.

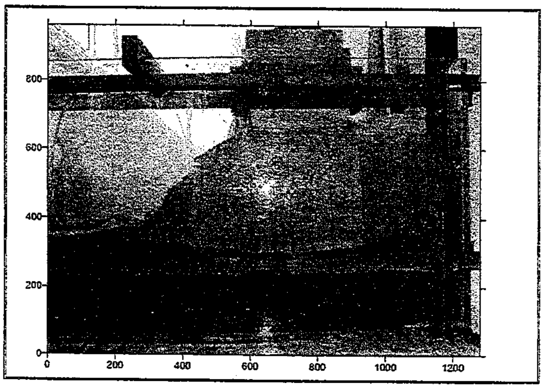
4.Results:

4.1 Test Results

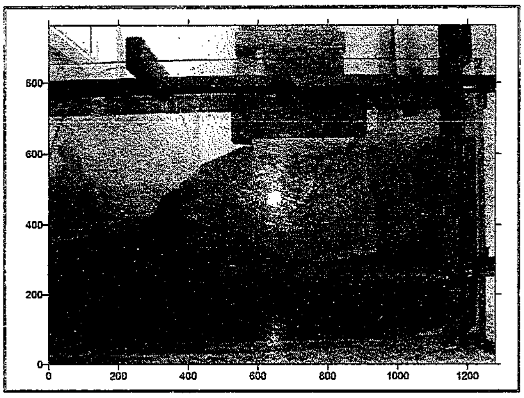
In this paper, the results obtained by analyzing three photographs taken at three different stage of loading, are discussed. Photograph 4.1 was taken before applying any load. Photograph 4.2 and Photograph 4.3 show the pictures at an intermediate stage of loading and at the final stage of loading respectively. The movement of the membrane with the soil body can be seen in Photograph 4.2 and photograph 4.3.



Photograph 4.1: Model before applying load



Photograph 4.2: Intermediate stage of loading



Photograph 4.3: After final stage of loading

4.2 Displacement Analysis

Table 4.1 below gives some coordinate values given by the program while digitizing the photographs taken at three different stages of loading and table 4.2 gives the same coordinates relative to a selected base point after converting to actual coordinates. The points given here are as in Figure 4.3.

	Coordinates given by program					
	Stage1-2		Stage 2-3		Stage1-3	
Point	x	y	X	Y	X	Y
1	286.72	321.28	267.52	313.81	234.88	313.81
2	340.05	322.34	319.79	310.82	287.57	305.49
3	393.39	324.48	372.06	306.77	340.05	293.12
4	446.72	326.82	428.37	300.80	396.59	273.71
5	500.27	326.82	486.83	296.74	460.16	258.56
6	555.73	327.89	546.99	294.61	530.77	250.24
7	609.07	330.24	607.36	305.70	602.66	286.08
8	661.33	332.37	664.75	316.80	666.24	304.21
9	714.67	336.85	721.07	322.77	725.76	316.59

Table 4.1: Coordinates given by program After digitizing photographs

	Actual coordinate					
	Stage1-2		Stage2-3		Stage1-3	
point	x	y	X	y	x	y
1	215.04	240.96	200.64	235.36	176.16	235.36
2	255.04	241.76	239.84	233.12	215.68	229.12
3	295.04	243.36	279.05	230.08	255.04	219.84
4	335.04	245.12	321.28	225.60	297.44	205.28
5	375.20	245.12	365.12	222.56	345.12	193.92
6	416.80	245.92	410.24	220.96	398.08	187.68
7	456.80	247.68	455.52	229.28	452.00	214.56
8	496.00	249.28	498.56	237.60	499.68	228.16
9	536.00	252.64	540.80	242.08	544.32	237.44

Table 4.2: Actual coordinates of grid points relative to a selected origin

Actual coordinates thus obtained were used, to plot the displacements of each node point.

Figure 4.1 shows the displacements of the nodes when load was increased upto the intermediate loading stage.

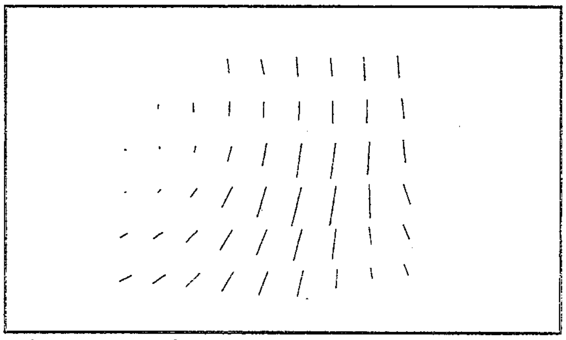


Figure 4.1: Displacements of the node points from loading stage 1 to 2

There are several interesting features appearing that can be observed in this plot. Displacements of nodal points at the top left corner are relatively small. The vertical lines to the right of the plot show some settlements of nodes vertically downwards. Longer lines at the middle region of the plot indicate that large displacements have taken place there. One reason for this occurrence may be the loading arrangement selected. Another reason is due to the development of the failure plane there. There appears a tendency of the middle lines to show a form of a circle. This is evidence to say that the failure surface has developed in that region.

Figure 4.2 shows the displacement pattern from loading stages 2 to 3 and Figure 4.3 shows the same from loading stages 1 to 3.

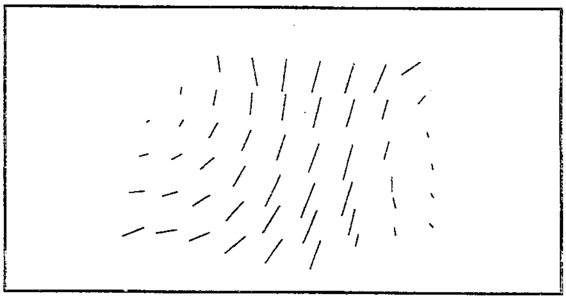


Figure 4.2: Displacement pattern from loading stage 2 to 3

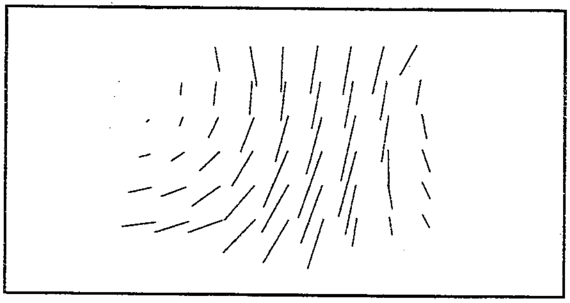


Figure 03: Displacement pattern from loading stage 1 to 3

4.3 Strain Analysis

The actual coordinates together with the displacements were used in the finite element program mentioned in section 2 to find the principal strains developed in each element of the soil body. The FE mesh was defined by the grid pattern drawn initially on the membrane. The program facilitates the entry of displacements as the input loading when boundary conditions are given accordingly.
The program gives results after analyzing the problem and the strains were obtained for all elements. As an illustration, Table 4.3 gives strains obtained for nine elements shown in Figure 4.3.

Shear strains developed in some elements			
Loading stage 1-2			
Element	Coordinates		Maximum Shear Strain
	X	Y	
1	220.6425	255.7200	7.05E-02
2	259.6475	253.2800	6.93E-02
3	299.6050	248.6125	1.63E-01
4	341.8825	243.6925	1.91E-01
5	285.8050	240.0400	2.80E-01
6	430.6425	240.8000	5.15E-01
7	475.5225	248.9200	5.10E-01
8	519.6425	257.0000	1.77E-01
9	223.6850	299.2400	5.71E-02

Table 4.3: Shear strains of nine elements shown in Figure 4.3

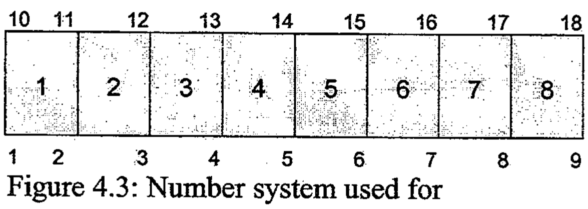


Figure 4.3: Number system used for elements and node (part of mesh)

In Table 4.3, the coordinates indicated are those corresponding to some Gauss point in each element.

5.1.Concluding Remarks

The methodology developed seems suitable for experimental determination of deformation pattern in model soil slope failures. Some problems were encountered as the rubber membrane wrinkled at higher loads/deformations. Certain improvements need to be incorporated to overcome the experimental drawbacks.

The investigation here is a part of an overall study of the effect of unsaturated soil shear strength parameters on the failure of soil slopes.

6. Acknowledgement

My sincere thanks are due to Dr. U.G.A.Puswewala and Dr. T.A.Peiris in the department of civil engineering, University of Moratuwa, for the guidance given for testing and writing this paper.

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Improvement of Engineering Properties of Peat by Preconsolidation -A Comparison of Field and Laboratory Test Results

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Abstract
Preconsolidation by preloading can be used to improve the Engineering properties of peat. Simulations of the field condition in the laboratory samples indicate the level of improvements that can be achieved.
With a highly none homogeneous material like peat it is essential to verify these findings with appropriate field measurements. This paper presents an attempt to compare the improvements in primary and secondary consolidation characteristics and shear strength properties obtained through laboratory tests, with those obtained from a monitored fill. Improvements observed in the field are seen to be comparable with those achieved in the laboratory.
Field consolidation due to the gradual placement of the fill was modeled using the Finite Difference method and comparisons were made with the observed Pore water Pressure and Settlements. The laboratory determined c_v value had to be increased to match the field behavior.

1. BACKGROUND AND OBJECTIVE

Presence of soft, highly compressible peat in layers of large thickness is a major problem encountered by Geotechnical Engineers involved in new infrastructure and housing development. Peat layers of thickness ranging from 5m to 13m are present in some of the lands available for development in the city of Colombo and its suburbs. Therefore, it is very important to develop cost effective methods for handling such conditions.

If multistoried buildings are to be constructed in areas underlain by thick layers of peat, the large structural loads will have to be transferred to an underlying dense layer or rock through a system of piled foundations. However, it would not be economical to transfer the moderate loads imposed by services such as water supply lines and sewerage lines also to an underlying hard stratum through piles. Furthermore, it would not be economical to construct new infrastructure facilities with large plan area such as roads, on piled foundations. It is most economical to transfer such moderate loads to the soft peat layers, after improving their strength and stiffness to an appropriate level.

Numbers of different approaches are in use around the world for the improvement of soft peat deposits. Pre-consolidation by preloading of the peat with a surcharge is one such method that could be carried out without the help of any special machinery.
Laboratory tests conducted to assess the improvement of strength and stiffness of peat through pre-consolidation had shown that the above-mentioned properties could be significantly improved through preconsolidation. With a highly non-homogeneous formation like peat, it is extremely important to verify the laboratory findings with field measurement by constructing a trial fill where a large mass of peat is subjected to a significant increase of stress. Such a study was done in the Fill Area 2, of the Madiwela Government project.

2. INITIAL SUBSOIL CONDITION AT THE SITE

A comprehensive subsoil investigation program was carried out in order to establish the relevant soil parameters. The subsoil condition at the instrumented fill area was investigated with 3 boreholes and 8 auger holes. The vertical soil profile deduced from the borehole investigation is presented in Figure 1. It revealed that there exists a compressible peat layer underneath and the thickness of this layer was approximately 5m to 6m. The average physical properties of this layer were indicated in the Figure 1.

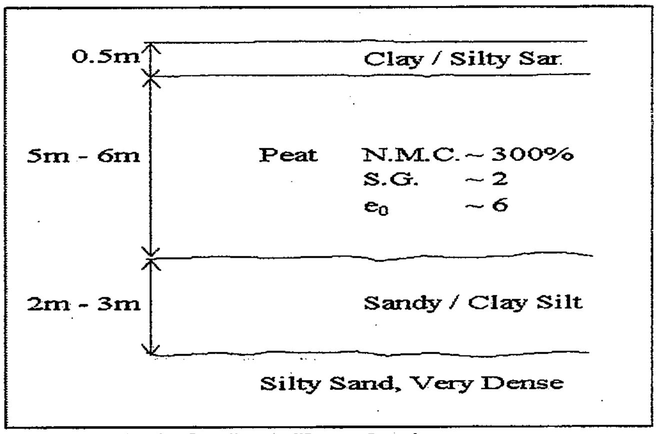

Figure 1. Typical soil profile at the site

Standard Penetration Tests were carried out at 1m intervals in order to obtain a continuous strength profile of the subsoil. The zero S.P.T value indicated the existence of soft soil up to about 5m. Field vane shear tests were carried out at number of locations at different depths to establish the initial shear strength. Based on the data of vane shear tests an average undrain cohesion (c_u) in the peat layer is found to be around 10 kN/m^2.

3. IMPROVEMENT ACHIVED IN THE ENGINEERING PROPERTIES.

Undisturbed soil sample were taken prior to any filling and specimens were subjected to consolidation tests and Unconsolidated Undrained triaxial tests under different cell pressures in order to established the initial Engineering properties. Test procedures were developed, for the laboratory simulation of the consolidation of the peat due to the weight of the fill. Subsequently, once the filling was completed and once the peat has consolidated under the weight of the fill, undisturbed soil samples were taken again and were subjected to a similar series of tests. Improvements achieved under laboratory condition by simulated testing were compared with the properties established by the samples obtained after the consolidation of the peat in the field.

(a) Improvements in Compression Index

Preloading process was simulated by conducting the consolidation tests with the loading/unloading and reloading increments in the laboratory. The initial loading increments simulated the natural peat and the reloading increments up to maximum previous consolidated pressure simulated the preloaded peat.

Improvements achieved in primary consolidation were compared using compression index (c_c) and coefficient of volume compressibility (m_v) in each phase. It was evident that both parameters were reduced up to around 10% due to the preloading . A void ratio vs. log (σ) plot obtained during the simulated testing of a sample obtained before the filling is presented in Figure 2.

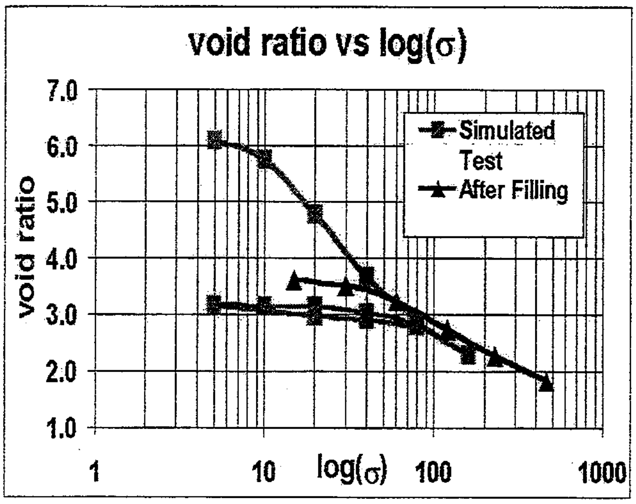

Figure 2.Graph showing reduction of the compressibility due to preloading

In the field the peat layer was consolidated under the weight of the fill placed. Thus, consolidation tests were conducted on the undisturbed samples obtained from the consolidated peat. This sample is reffered to here as the "after filling sample". The gradient of the void (e) vs. log (σ) plot up to the preconsolidated pressure Pc of the "after filling sample" should provide an assessment of the compressibility of the preloaded peat. This should be compared with the gradient of the reloading curve obtained during the simulated testing. Figure 2 compares the void ratio (e) vs. log (σ) plot for the above two cases. It could be seen that the gradient of the reloading curve in the simulated testing and the initial gradient from the test on the "after filling sample" are similar. The two gradients are 0.265 and 0.397 respectively. Similar behavior was observed in the samples taken from other locations as well.

(b) Improvements in Coefficient of Volume Compressibility

Coefficient of volume compressibility (m_v) obtained during the reloading phase of the simulated testing were compared with the values of coefficient of volume compressibility observed in the loading increments up to the preconsolidated pressure of the "after filling sample". (Figure 3)

It shows that the observed coefficients of volume compressibility up to the preconsolidated pressure are approximately equal to the values obtained in the reloading phase of the simulated testing. (3rd and 4th points in the two graphs)

The values were reduced to about 10 % of the untreated peat .

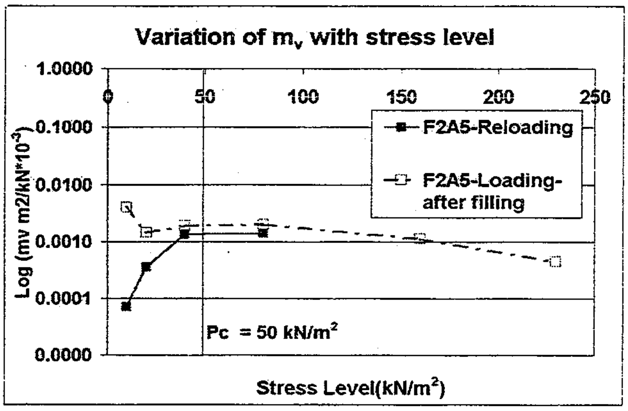

Figure. 3 Graph showing predicted and observed Coefficient of volume compressibility

(c) Improvements in Coefficient of Secondary Consolidation

Results of the simulated tests indicated that the coefficient of secondary consolidation (c_α) obtained during the loading have reduced about 10% of in the reloading stages. Results are presented in Figure 4. Also it can be seen that the observed c_α values up to the preconsolidated pressure of "after filling sample" and reloading phase of the simulated tests are also of same order.

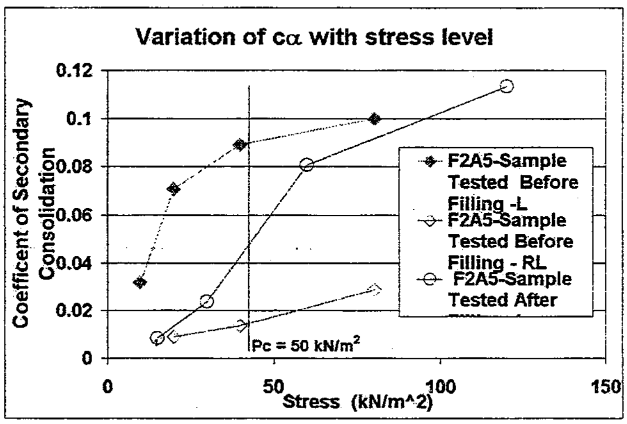

Figure 4. Graph showing predicted and observed Coefficient of Secondary consolidation values

(d) Improvements in Undrained Shear Strength

Shear strength gain due to consolidation of peat was studied through consolidated undrained tests. Several identical specimens were isotropically consolidated at different cell pressures and subjected to deviator loads under undrained conditions. The change in deviator load at failure due to the change in consolidation pressure was used to obtain the $\Delta c_u/\Delta\sigma$ ratio and it is around 0.21 for this site. Similar results were reported in Kulatilaka (1999).

Undrained shear strength obtained from the samples tested after filling was in the rage of 18 kN/m^2 to 21 kN/m^2, an average increase of around 10 kN/m^2 which is equal 0.2 times the weight of the fill (46 kN/m^2). This is approximately equal to the predicted laboratory improvement.

4.0 Monitoring of Field Behavior

Traditionally, the rate of consolidation in the field is estimated with the use of coefficient of consolidation c_v determined through laboratory tests. Numerous research publications (for a example Wojeieoh et al (1988)) have reported that the field rate of consolidation can be much

faster than the rate predicted through the laboratory results and therefore the field rate of consolidation should be obtained by independent field monitoring of the pore pressure and the settlements. Pizometers were installed in the middle of the peat layer and the pore pressure development in the peat layer with the placement of the further fill and the subsequent dissipation with time was monitored daily. Also, settlement gauges were placed just above the peat layer after the placement of a nominal fill of thickness 300mm. Settlement of the peat during further filling and during the consolidation phase was captured through these settlement gauges. The observed pore water pressure and settlement due to filling are presented in Figure 5.
The monitoring of both the settlement and pore pressure independently provided important information about the secondary consolidation characteristics of peats.

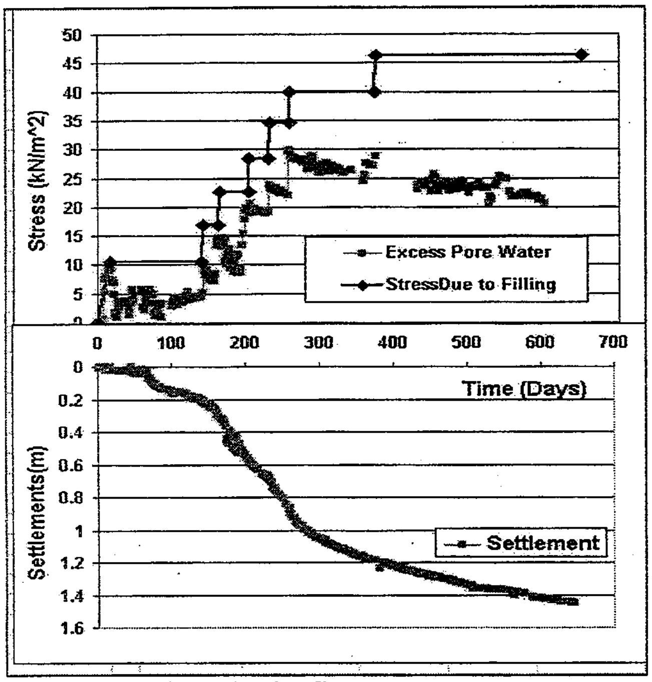

Figure. 5. Field monitoring Data

The fill was placed and compacted in layers under the controlled conditions. Fill material was placed in layers not exceeding 300mm thickness in loose state and was compacted at each lift to not less than 95% of the standard proctor density.

5.0 Modeling field behavior and comparison with actual observation

The prediction of the field rate of consolidation is a very important part of a preloading project. The prediction of the field behavior is mainly based on the coefficient of consolidation determined from the laboratory consolidation test. Often the actual observed rate of settlement is much higher than the rate predicted on the basis of the values obtained using undisturbed samples. The possibility of 2D or 3D consolidation in the field and the small soil sample being unrepresentative, might be the reasons for this difference. Also, the time required to construct the compacted fill to its final height is fairly long and may be comparable with the time required for the consolidation of the soil layer. Thus, during the gradual building up of the embankment, consolidation occurs simultaneous with the increase of the fill height. Therefore, it is necessary to model this real behavior to obtain a good prediction of the field settlements and the pore pressure response.
In this research project a finite difference model was developed for the prediction of field behavior.
Also, the predicted field behavior was compared with the actual rate of consolidation, which was observed through the independent measurement of pore pressure, and the cumulative measurement of settlements in the filed. Finally, when the predicted field behavior using lab values was found to be quite different from the actual observed behavior, fresh prediction were made by changing the lab parameters till the observations are well matched.

5.1 Modeling of Field Behavior using Finite Difference Method.

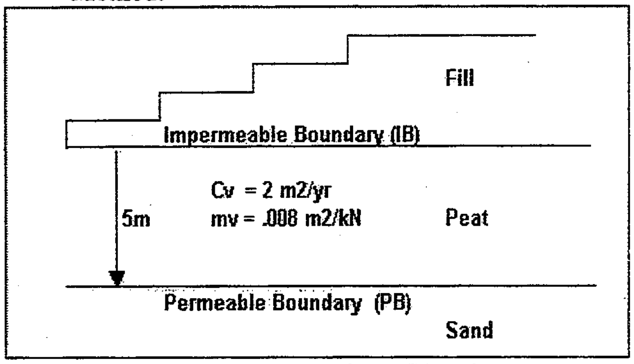

Figure 6. Idealized section used for F.D.M

The idealized section of the sub soil condition deduced from the boreholes investigation is presented in Figure 4.The parameters required for analysis was obtained from the lab tests.

Modeling the pore pressure response

It is done by using the following equation describing practical situation in which the external stresses causing consolidation progressively changes with time is

$$c_v \frac{\partial^2 u}{\partial y^2} = \frac{\partial u}{\partial t} - \frac{\partial \sigma}{\partial t}$$

where
u = excess pore water pressure y = distance to the point
t = time σ = external pressure
c_v = coefficient of consolidation

A Finite Difference model was developed with an appropriate mesh and excess pore pressures are calculated for different boundary conditions, following the procedures outline in Lee et al. (1982)
The top boundary is taken to be undrained and the bottom boundary is taken to be drained.

For permeable boundary

$u_{i+1,j}=u_{i,j}+ c_v\delta t/\delta H^2(u_{i,j+1}+ u_{i,j-1}-2u_{i,j})+(\sigma_{i+1}-\sigma_i)$

For impermeable boundary

$u_{i+1,j}=u_{i,j}+ \beta(2u_{i,j+1}-2u_{i,j})+(\sigma_{i+1}-\sigma_i)$

Where $u_{i,j}$ = pore pressure at time $i\delta t$ at depth level j.

Modeling of the Settlement behavior

Then effective stress at time $i\delta t$ at depth level j can be expressed as

$\sigma_{i,j} = \sigma_{i,j} - u_{i,j}$

where $u_{i,j}$ - pore pressure at time $i\delta t$ at depth level j
$\sigma_{i,j}$ - total stress at time $i\delta t$ at depth level j

Using the Terzaghi consolidation Theory settlement ' δ ' for each layer is calculated as follows.

$\delta = m_v \Delta\sigma \Delta h$

where, m_v – coeff. of volume compressibility
Δσ - effective stress increase in the soil mass
Δh - thickness of the compressible layer considered

Then the total settlement due to a time varying load can be evaluated by summing the settlements of each layer.

This can be mathematically expressed as follows;

$$s_{i,j} = m_v*\delta h*\left(\frac{\sigma_{i,0}}{2}+\Sigma\sigma_{i,j}+\frac{\sigma_{i,n}}{2}\right)$$

where the for the summation part j= 1 to n-1
(assume soft layer is divided to n nos. of layers)

To compare the accuracy of the finite difference method, the settlements predicted using the finite difference method were compared with the standard Terzaghi solution by considering the instantaneous loading (Figure 7). Model was also verified with the stage loading case for different c_v values and also for different boundary conditions. The ultimate primary settlement was seen to be same irrespective of the rate of consolidation, confirming the validity of the finite difference model.

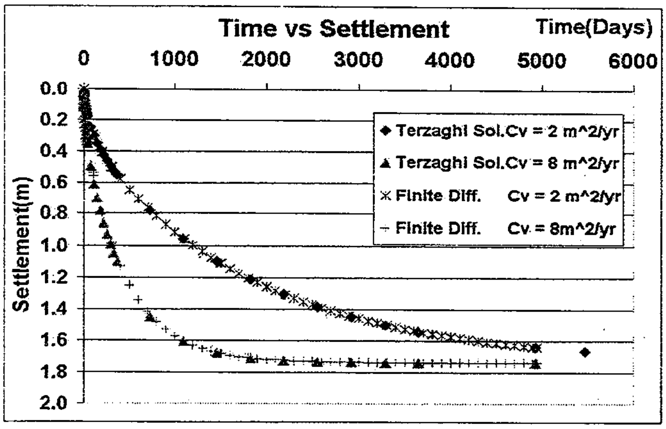

Figure 7. Comparison of F.D.M. with Terzaghi Model

Comparison of Pore Water Pressure Dissipation.

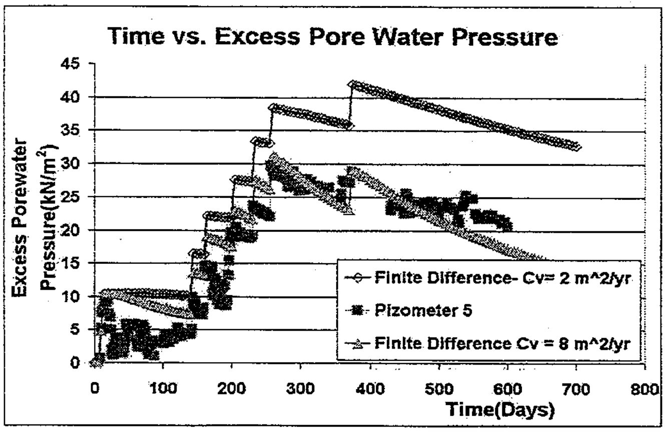

Figure 8. Comparison of p.w.p. For different c_v values

The predicted excess pore water pressure dissipation was compared with the actual pore water pressure dissipation measured from the pizometer installed in the field, namely; pizometer 5. The results are presented in figure 8.

It was clear that the actual pore water pressure dissipation is much faster than that predicted with the laboratory determined c_v value of 2 m^2 /yr. As such, the pore water pressure dissipation was predicted using higher values of c_v. The prediction was found to be closer to actual observed behavior when c_v equal to 8 m^2/yr was used.

Comparison of predicted and actual settlements

The predicted settlements were compared with the actual cumulative settlements measured from the settlement gauges installed in the field, namely; settlement gauge 3 and 5.

It has been observed that actual settlement is much higher than the settlements predicted with the laboratory-determined coefficient of consolidation $2m^2$/yr. It is an indication that the field rate of consolidation is much faster than the prediction done using laboratory parameters. Therefore settlements were predicted using a for c_v value of 8 m^2/yr and results are is given in figure 9. It should be noted that a reasonable prediction of excess pore water pressure was also obtained with the use of a c_v value 8 m^2/yr. Assumed boundary conditions appear to be reasonable.

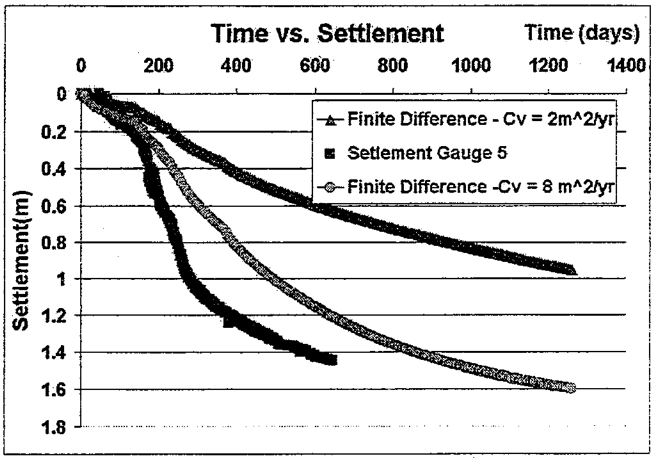

Figure 9.Comparison of settlement for different c_v

6. Conclusion

Improvements of primary and secondary consolidation characteristics shown by simulated testing, and the results obtained from the samples collected after a field consolidation, were in same order. Also, improvement shown in the shear strength with consolidated undrained tests are comparable with the results obtained in the sample collected after the field consolidation and field vane shear tests.

Field behavior was modeled with the assistance of the finite difference method. It was shown that the dissipation of pore water pressure in the field was much faster than the rate suggested by the lab c_v value. Therefore, prediction was done using the higher c_v values and it was found to be closer to the observed behavior when the ratio between c_v field /c_v lab = 4. Field settlements are still higher than that predicted by the Finite Difference model. This might be due to the secondary consolidation happening together with the primary consolidation.

Acknowledgements.

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Modeling the Consolidation Behavior of Peat and Improvements

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Abstract
Construction on peaty soils is problematic and it is necessary to find cost effective methods for improvement of peat. Therefore, this research was aimed at studying the response of peats with different levels of humification on the improvement techniques such as preloading and deep mixing with cement and lime. It showed that preloading caused improvements in all types of peat and Deep mixing was effective only for Amorphous peat that are of higher degree of humification.

Also, this study has compared the observed and predicted consolidation behavior of peat using Terzaghi's One Dimensional Consolidation theory. A new laboratory set up was developed to observe the simultaneous settlement and pore water pressure. Results showed that the level of agreement depends on the method of obtaining the C_v value.

1. Background

Very high primary and secondary settlements and very low shear strength are the major problems associated with peaty soils, which are often found in low-lying areas of Colombo and other suburbs. With the increase of population, demand for land goes up and engineers are compelled to use them for future development projects.

Construction on peaty soils leads to stability problems due to its very high compressibility and very low shear strength. Although heavy structures can be built on piled foundations, use of shallow foundations on improved grounds would be more appropriate for light structures such as service lines, light buildings and roads occupying a large plan area.

The most widely used method of improvement for peat is the pre-consolidation by preloading. But, it requires a rather long time period. Deep mixing method mixes the existing soil with a cementiteous material using mixing shafts and nozzles over the full thickness of the layer. Although this method has been successfully used in Japan and Sweden for soft inorganic clays, not much information is available on application in Peaty soils except for few publications in late 1990's from Finland (Huttunen et al 1996) and Sweden. Therefore, study of the possible improvement of consolidation behavior of peat by deep mixing is of vital importance.

This paper presents an attempt to study the consolidation behavior of peat by mixing different percentages of cement or lime with peat at different levels of humification. Peats can be grouped as Amorphous and Fibrous Peat. Amorphous peat has a higher degree of humification. The level of humification can vary within a short distance even in the same locality.

This paper compares the improvements of primary and secondary consolidation achieved by cement and lime mixing against the improvements achieved by preloading.

Also, an attempt was made to model the consolidation behavior of both the natural peat and peat improved by cement mixing using the Terzaghi's one dimensional consolidation theory (Terzaghi 1925). The prediction of settlement and pore water pressure behavior done using the Terzaghi's theory were compared with the experimental observations.

2. Comparison of methods for Improvement of Compressibility of Peat

2.1 Laboratory simulation of pre-loading and Deep Mixing

In order to simulate the preloading process; loading, unloading and reloading increments were applied in the Oedometer. Loading increments represent the untreated peat and the reloading increments represent the Preloaded Peat. Improvements achieved at similar stress levels were compared. Load increments of two-week duration were used in view of the dominant secondary consolidation in Peat.

To simulate the site condition in the deep mixing method, peat was remoulded in the laboratory by mixing with cement and lime using an electrical hand mixer over the same time period, at the same rotating speed and they were left to harden in buckets for a period of four weeks under water. Consolidation tests were conducted on undisturbed samples obtained from the treated peat.

With types of peat shown in Table 01, mixing was done with the addition of 5%, 10%, and 15% cement and 15% lime. Above mix proportions were decided based on the results of some initial tests.

Place	Type	Natural Moisture Content	Organic Content	PH Value	Specific Gravity	Initial Void Ratio
Wattala	Amorphous Granular Peat	387.00	29.00	2.60	2.23	8.63
Madiwela	Fibrous Peat	297.00	34.80	2.99	1.87	5.55

Table 01: Basic properties of selected Peat

2.2 Improvement of Primary consolidation characteristics of Peat

Improvement of primary consolidation characteristics were studied by comparing the parameters; coefficient of volume compressibility - m_v, compression index - C_c, recompression Index - C_r, compression ratio $C_c/(1+e_o)$ and recompression ratio $C_r/(1+e_o)$.

The effect of preloading and deep mixing with cement and lime on the coefficient of volume compressibility m_v is illustrated in Figure 1 and 2 for Wattala Amorphous Peat and

Madiwela Fibrous peat respectively. Both peats possessed similar organic contents and void ratios but the degree of humification was much higher in Wattala Peat. It is evident from Figure 1, that due to the mixing of 5% cement by weight, Wattala amorphous peat experienced a significant reduction of m_v, that was of the same order as the improvement achieved by preloading. With the fibrous Madiwela peat, although the preloading has caused a reduction in m_v, even the mixing of 15% cement or 15% lime has not caused much improvement in the coefficient of volume compressibility as illustrated by Figure 2.

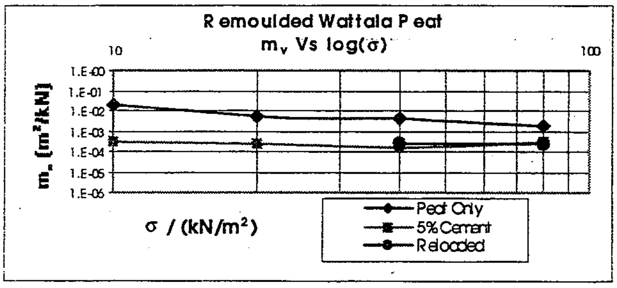


Figure 01:Effect on m_v for Wattala Amorphous Peat

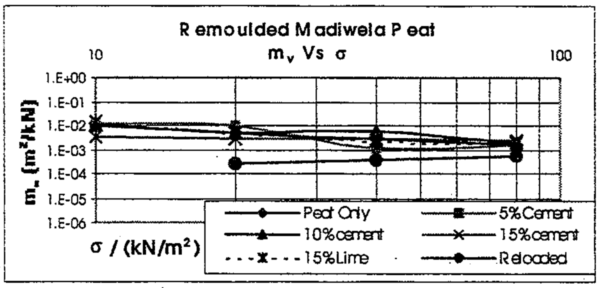


Figure 02: Effect on m_v for Madiwela Fibrous Peat

Alternatively improvements achieved in primary consolidation characteristics can be illustrated with e vs log σ plot. It is evident from e vs log σ graphs corresponding natural peat for Wattala Amorphous peat (Fig. 03) and Madiwela Fibrous Peat (Fig. 04), that re-compression index C_r is much smaller than the (less than 8%) compression index C_c. Thus, it is clear that the preloading can cause a significant improvement in primary consolidation characteristics in both types of peat. The C_c value corresponding to 5% cement mixed peat in figure 03 is of the same order as the C_r for the preloaded peat for Amorphous Wattala peat thus indicating that similar levels of improvements were achieved. However, as illustrated by figure 04 mixing of different percentage of cement and lime has not caused much of a reduction in the C_c in the Madiwela Fibrous peat.

2.3 Improvement of secondary consolidation characteristics of peat

The effect of the improvement methods on secondary consolidation characteristics was assessed by comparing the coefficient of secondary consolidation-c_α and its variation with time and the stress level.

The variation of C_α values with stress level is plotted for Wattala amorphous peat and for Wattala peat mixed with 5% cement in Figure 05 and it is evident from these plots that the mixing of 5% cement has caused a significant reduction in C_α. This reduction is of the same order as the reduction achieved through preloading.

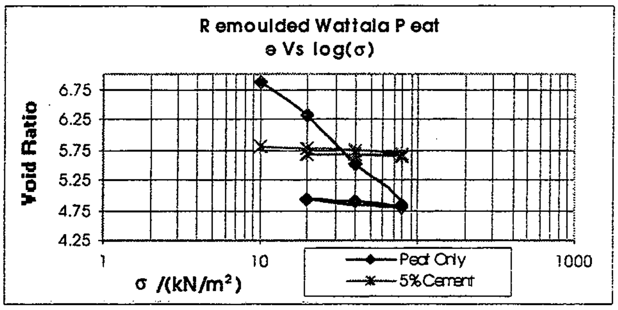


Figure 03:Comparison of e vs log σ plots Wattala Peat

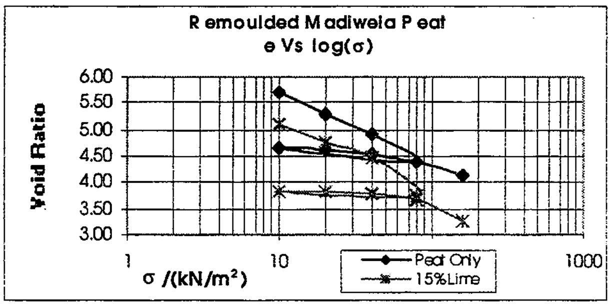


Figure 04:Comparison of e vs log σ plots Madiwela Peat

Test results presented in figure 06 show that mixing of cement or lime has not caused much improvement in the C_α values of Madiwela fibrous peat while the C_α values for the reloading increments were much smaller. Thus, it is clear that the preloading will cause an improvement in secondary consolidation characteristics even in a fibrous peat.

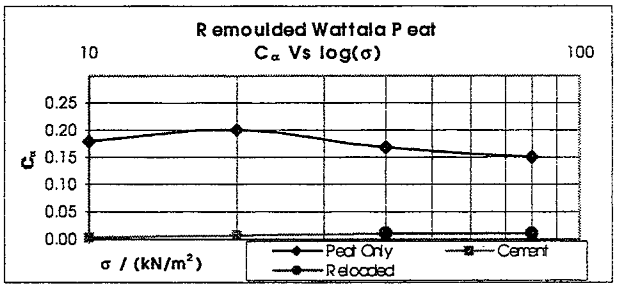


Figure 05: C_α vs log σ plot for Wattala Peat

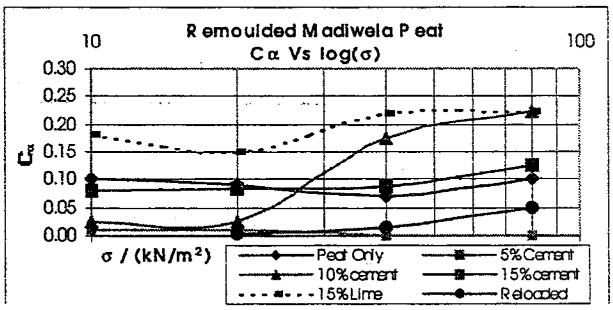


Figure 06: C_α vs log σ plot for Madiwela Fibrous Peat

3.0 Modeling the Consolidation Behavior of Improved and Natural Peat

3.1 Importance of measuring pore water pressure

Since the primary consolidation settlements are accompanied by the dissipation of excess pore water pressure, any laboratory consolidation tests conducted to model the consolidation behavior of a soil should be ideally done with the measurements of both the settlements and the pore water pressures. In the conventional laboratory consolidation test, only the settlements are measured. A laboratory setup was developed to conduct the consolidation tests with pore water pressure measurements.

3.2 Development of a Laboratory test setup for simultaneous measurement of pore water pressure and settlement

In the new laboratory setup, a GI pipe of 69.5mm diameter and the height of 85mm was used as the consolidation ring. Drained conditions were provided at the upper boundary of the sample while undrained conditions were established at the bottom boundary. A plate containing four holes to facilitate drainage was used on top of the sample with a porous plate and the bottom plate consist of one hole at the center to be connected to the pore water pressure measuring arrangement. Top plate and the top porous plate were made to a smaller diameter facilitating the free movements inside the cylinder freely and the bottom plate was made water tight by fixing two rubber "O" rings around. The bottom plate has a threaded end to facilitate fixing to the triaxial base. The pore water pressure measurement was done through the diaphragm system used to measure pore water pressure in the triaxial setup. In these tests, vertical loading was applied directly on the sample using a loading frame.

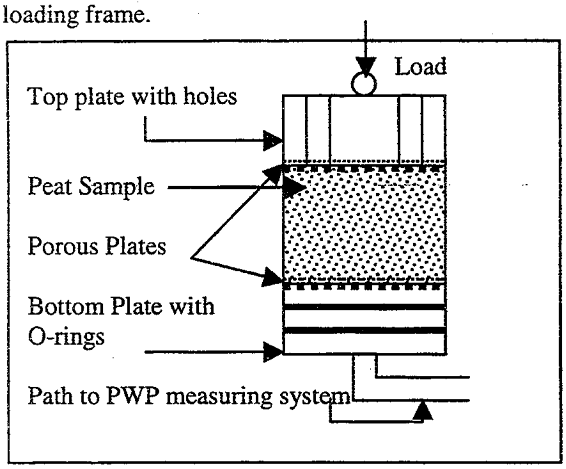
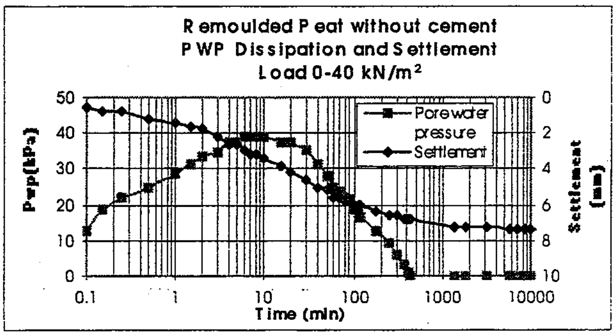


Figure 07: New Laboratory setup

3.3 Testing Procedure and Results

The sample was subjected to a vertical stress of 40 kN/m^2 using the loading hanger. Two soil specimen; specimen of Madiwela Peat remoulded with 10% cement and a specimen of Madiwela peat remoulded without addition of any cement were used for the tests.

The major shortcome seen with pore water pressure measuring system was the slow response time. It was minimized as far as possible by reducing the length of the connecting tubes. However, with all that efforts the maximum pore water pressure was developed only after 4 minutes. Figure 08 present the graphs for remoulded natural Madiwela peat. The graph for the peat with 10% cement was also of similar shape. Results shows that the excess pore water pressure has dissipated completely within a 200-400 minutes period. Therefore, it can be deduced that primary consolidation has finished by that time. But the settlements continued even after the 100% pore water pressure dissipation. This must be due to the secondary consolidation.



water pressure for Madiwela Peat without cement

3.4 Back analysis of the test results using Terzaghi model

Although there are some limitations, Terzaghi's one dimensional consolidation theory was used to model the consolidation test. Under the boundary conditions prevailing in the test setup, the pore water pressure at the bottom of the sample is given by;

$$u = \sum_{n=0}^{\infty}\left[\frac{2u_0}{M}.Sin\frac{MZ}{H}.e^{(-M^2T_v)}\right]$$

Where $M = \frac{\pi}{2}(2n+1)$ $\qquad T_v = \frac{C_v t}{H^2}$

The above expression is evaluated by adding up the values in the series from n = 0 to 10 using an EXEL worksheet and the variation of the pore water pressure u with time is computed. The average degree of consolidation of the specimen can be expressed in terms of the pore water pressure by;

$$U = \frac{\left(u_0 - \bar{u}\right)}{u_0} = 1 - \frac{\bar{u}}{u_0} = 1 - \sum_{n=0}^{\infty}\frac{2}{M^2}e^{(-M^2T_v)}$$

The settlement δ_t at a given time t can be expressed as $\delta_t = \delta^*U$, where δ is the ultimate settlement at, 100% pore water dissipation. Another EXEL spreadsheet was developed to obtain δ_t variation with time; by adding first ten terms of the expression. Details are presented in Munasinghe (2001).

The C_v value required to model the test results was found by two methods. In the first method value of C_v was obtained using the time taken for 50% consolidation settlement of the sample. The settlement at 100% excess pore water pressure dissipation was found and time required for 50% of that settlement was taken as t_{50}. Thereafter, C_v was calculated using the time factor of 0.197 at 50% consolidation. This yielded a C_v value of 15.64 mm^2/min.

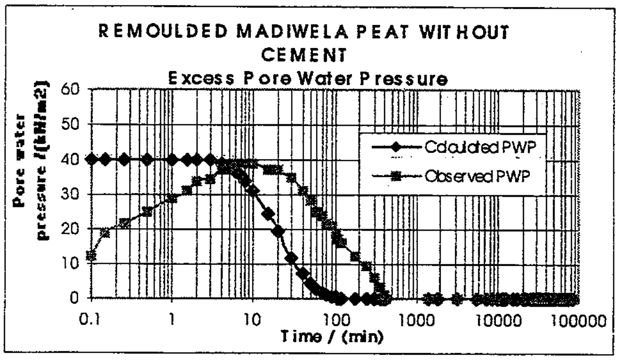


Fig 09:Calculated and Observed PWP using C_v -method 1

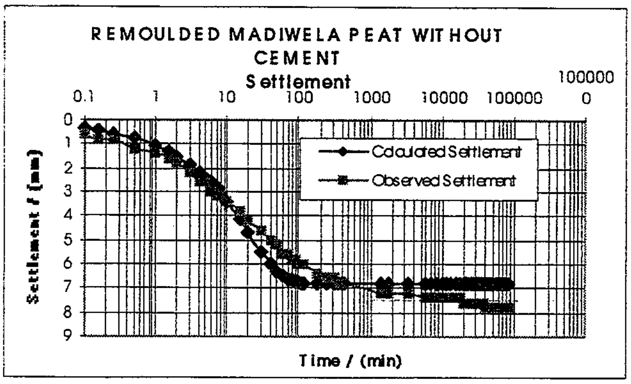


Fig 10:Calculated and Observed δ using C_v-method 1

The pore water pressure and settlements computed are compared in fig. 09 and Fig. 10 respectively for Madiwela untreated peat. It shows that the settlement behavior up to end of primary is reasonably well predicted. As expected the secondary consolidation settlements were not predicted by the Terzaghi model computations. Madiwela peat treated with 10% cement also exhibited a similar behavior.

In the second method, C_v was obtained using the time taken for 50% excess pore water pressure dissipation. The C_v value obtained using isochrones was 3.02 mm^2/min for Madiwela untreated peat. Fig. 11 and Fig.12 show the comparison of pore water pressure and settlement thus computed and measured values respectively for Madiwela fibrous peat treated with 10% cement. As expected, the pore water pressure behavior is reasonably predicted. From Fig. 12 also it can be clearly shown that the Terzaghi's one dimensional consolidation theory does not include the secondary consolidation settlement. Also, at the initial stages, it is seen that the measured settlement is higher than that computed. Similar observations were made by Mesri et al (1997). This behavior may attribute to the secondary consolidation happening simultaneous with the primary consolidation.

4.0 Conclusions

It can be concluded that preloading causes significant improvements in both primary and secondary consolidation properties on all types of peat that are at different levels of humification. Deep mixing with cement or lime has an ability to develop the stiffness within the period as short as 4 weeks. However, this method was successful only for Amorphous peat that are with a high level of humification. Improvements achieved were of the same order as that achieved during preloading.

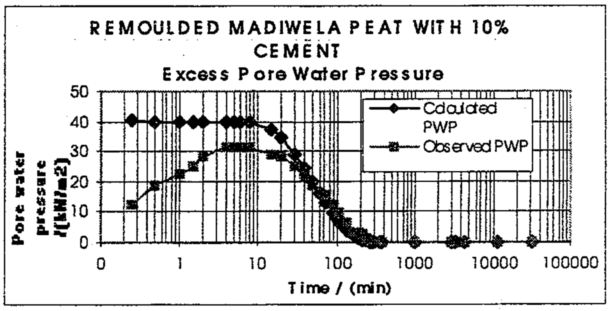


Fig 11:Calculated and Observed PWP using C_v-method 2

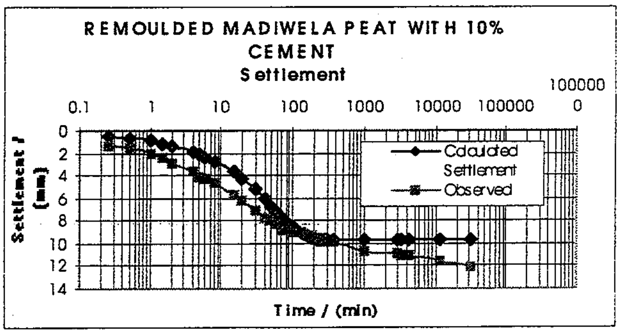


Fig 12:Calculated and Observed δ using C_v-method 2

Consolidation tests conducted with pore water pressure measurements revealed that settlement continue even after the full dissipation of excess pore water pressures. It is clearly shown that the secondary consolidation settlements were not predicted by the Terzaghi Model. Back analysis of Terzaghi's theory showed that the prediction of settlement or pore water pressure depended on the method of computation of Cv value. Thus the Terzaghi Model is not ideally suited to model the consolidation behavior of peat.

Acknowledgements:
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DEVELOPMENT OF ANALYTICAL MODELS FOR PROBABILISTIC SLOPE STABILITY ANALYSIS

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ABSTRACT
Probabilistic approach to slope stability evaluation recognises the uncertainties involved in the determination of key geotechnical parameters and considers them as random variables. It is not readily used in practice due to the fact that most engineers are not familiar with probabilistic concepts and due to the misconception that it require large amount of data. However, probabilistic analysis can be performed with the same amount of data that is required for the deterministic analysis, and also using some simple analytical models. This paper discusses the development of analytical models for probabilistic slope stability analysis. Results of the application to two hypothetical problems and the rehabilitated watawala slope are presented, and discussed.

1.0 CONCEPT OF FACTOR OF SAFETY

The stability of a slope is evaluated in conventional geotechnical engineering design problems using the "Factor of Safety", which is defined as the ratio between the shear strength and the mobilized shear stress. Generally, it is common to adopt a design value of allowable factor of safety for a given application without giving much regard to the degree of uncertainty involved in the determination of parameters used in the calculation. Thus the factor of safety alone does not reflect the actual situation.

2.0 CONCEPT OF PROBABILITY OF FAILURE

Concept of "Probability of Failure" formally recognises the various uncertainties involved in geotechnical parameters by treating them as random variables. Parameters that carry uncertainty may be listed as; shear strength parameters, pore water pressure, soil unit weight and slope geometry. However, it is most common to use soil strength parameters and pore water pressures as probabilistic parameters that carry a certain degree of uncertainty. Other parameters such as density and geometry are considered as deterministic parameters.

Probability of failure may be defined using two approaches, viz. factor of safety and safety margin, which is the difference between the resisting forces and the disturbing forces. Uncertainties of basic parameters gives rise to an uncertainty of either factor of safety or safety margin, and the probability of failure is defined as the "probability of factor of safety being less than one" or the "probability of safety margin being less than zero". Since, the factor of safety is the more familiar parameter to most geotechnical engineers, the definition that uses it is used ahead of the one that uses the safety margin.

3.0 APPROPRIATE PROBABILITY DISTRIBUTION FUNCTIONS

Various probability distribution functions can be assumed to account for the variability of the factor of safety. Potential distribution functions discussed in literature are; normal distribution, log normal distribution and the beta distribution [Smith (1986), Chowdhury (1992)].

The normal distribution has a bell-shaped probability distribution function as shown in figure 1(a). The range of probable values of factor of safety is from -∞ to +∞, and that has created doubt in the mind of some researchers, since the factor of safety cannot be a negative value [Chowdhury (1992)]. When the normal distribution is standardized, a parameter called "Reliability Index, β" can be defined as;

$$\beta = \frac{\overline{F}-1}{\sigma_F} \quad ....(1)$$

where $\overline{F}$ is the mean FOS and σ_F is the standard deviation.

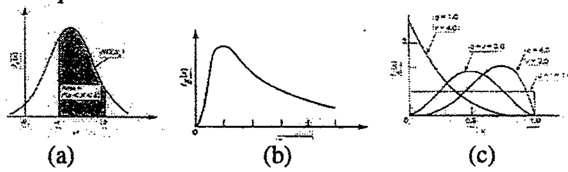
When the reliability index is used, the reliability (opposite of probability of failure), can be directly read from standard normal distribution tables, and the probability of failure = 1 – reliability.

The factor of safety is said to have a lognormal distribution, where the logarithms of factors of safety are normally distributed. The shape of the distribution, which has possible values from 0 to +∞, is shown in figure 1 (b). When the lognormal distribution is standardized, the reliability index is given by;

$$\beta_{LN} = \frac{\ln\left(\dfrac{\overline{F}}{\sqrt{1+V_F^2}}\right)}{\sqrt{\ln(1+V_F^2)}} \quad ....(2) \text{ [Duncan (2000)]}$$

where $\overline{F}$ = mean factor of safety, and V_F = coefficient of variation of factor of safety. Reliability can again be read from the standard normal distribution tables [Duncan (2000)]. In his papers, Duncan has assumed a lognormal distribution for factor of safety.

The beta distribution, which has the distribution function as shown in figure 1(c), is another possible distribution function, but its use has been limited due to complexity of the equations.



(a) (b) (c)

Figure 1: Various probability distribution functions: (a) normal distribution; (b) lognormal distribution; (c) various shapes of beta distribution

4.0 DIFFERENT METHODS OF QUANTIFYING THE VARIABILITY OF FACTOR OF SAFETY

Computation of "Probability of Failure" involves computation of the "most likely value (or mean, $\overline{F}$)" and the variance (σ_F^2) of the factor of safety. Most likely value of factor of safety is calculated using the mean values of the parameters, and can be mathematically expressed as;

$F = f(x_1, x_2, x_3, \ldots)$ ....(3)
where $x_1, x_2, x_3, \ldots$ are the parameters on which the value of F depends.

The variance (also called second moment of the distribution) can be estimated by expanding equation (3) in a Taylor series and truncating to the first terms. Hence, the method is called "First-order second moment (FOSM) method".

If the x_is are uncorrelated, the variance is expressed as;

$$\sigma_F^2 = \sum_{i=1}^{k}\left(\frac{\partial F}{\partial x_i}\right)^2 \sigma_{x_i}^2 \quad \text{.......(4)}$$

where σ_{xi} is the standard deviation of i th parameter.

Different methods can be adopted in calculating the variance, and they are;

1) direct differentiation
2) approximate calculation using divided differences [Christian *et.al.* (1994)]

Apart from this method, Monte-Carlo approach also gives a probability distribution of the factor of safety. In Monte-Carlo method, a stability problem is analysed a large number of times changing the soil strength parameters over a wide range (using a selected deterministic model) and the resulting factors of safety are used to obtain the probability distribution (p.d.f.) of the FOS. This p.d.f. is then used to estimate $\overline{F}$ and σ_F [Kulathilaka (1999)].

5.0 DEVELOPMENT OF THE MODELS

Two probabilistic models were developed, one using the Bishop's simplified method and the other using the Janbu's simplified method. Each model calculates the probability of failure in five different methods, which has different levels of complexity as illustrated in the subsequent sections. Parameters considered to carry uncertainty are; soil strength parameters and pore water pressures.

5.1 Model using Bishop's simplified method

The most probable value of factor of safety is calculated using the usual equation;

$$F = \frac{\sum\{(c'\Delta x_i + (W_i + Q_i - u_i\Delta x_i)\tan\varphi')/M_i(\theta)\}}{\sum W_i \sin\theta_i} \quad \text{......(5)}$$

The variance of factor of safety is calculated based on the equation (4) using the following different methods to calculate $\frac{\partial F}{\partial x_i}$.

5.1.1 Direct Differentiation assuming $M_i(\theta)$ is deterministic

Since the $M_i(\theta)$ is almost close to unity, the assumption made to consider $M_i(\theta)$ as deterministic seems reasonable. Furthermore it reduces the complexity of the equations and is more useful in practical situations. The derived partial derivatives are;

$$\frac{\partial F}{\partial c'} = \frac{\sum\left(\frac{\Delta x_i}{M_i(\theta)}\right)}{\sum W_i \sin\theta_i} \quad \text{........(6)}$$

$$\frac{\partial F}{\partial \varphi'} = \frac{\sum\left\{(W_i + Q_i - u_i\Delta x_i)\frac{\sec^2\varphi'}{M_i(\theta)}\right\}}{\sum W_i \sin\theta_i} \quad \text{........(7)}$$

$$\frac{\partial F}{\partial u} = \frac{\sum\left(-\frac{\Delta x_i \tan\varphi'}{M_i(\theta)}\right)}{\sum W_i \sin\theta_i} \quad \text{........(8)}$$

5.1.2 Direct Differentiation assuming $M_i(\theta)$ is probabilistic

Although the differentiation process is somewhat complex, this model was also developed as a tool for comparison of results. Here, $M_i(\theta)$ is assumed to be a function of ϕ'. Hence, the partial derivative with respect to ϕ' is modified to give a more complex equation.

5.1.3 Approximate Differentials using Divided Differences

This does not evaluate the exact differential, but uses the approximation;

$$\frac{\partial F}{\partial x_i} \approx \frac{\Delta F}{\Delta x_i} \quad \text{.......(9)}$$

Three different methods were adopted to evaluate these partial derivatives.

a) Assumes $\Delta F = F(x_i+\sigma_{xi}) - F(x_i-\sigma_{xi})$, and $\Delta x_i=2\sigma_{xi}$. This method, which uses central differences is suggested by Duncan (2000).
b) Christian (2001), in his discussion to Duncan (2000) paper suggested that $\Delta x_i=2\sigma_{xi}$ to be a much broader limit, and wants to make it narrow. Hence $\Delta F = F(x_i+\Delta x_i/2) - F(x_i-\Delta x_i/2)$, and Δx_i can be any small value. This also uses the central differences.
c) Next approach uses the forward differences with $\Delta F = F(x_i+\Delta x_i) - F(x_i)$, with any small value for Δx_i.

5.2 Model using Janbu's simplified method

The most likely value of the factor of safety is computed with the usual equations;

$$F_0 = \frac{\sum\{(c'\Delta x_i + (W_i + Q_i - u_i\Delta x_i)\tan\varphi')/n_i(\theta)\}}{\sum W_i \tan\theta_i} \quad \text{......(10)}$$

$$F = f_0 F_0 \quad \text{...........(11)}$$

where f_0 is the Janbu's modification factor that depends on the slope geometry.

The variance of initial factor of safety is calculated based on equation (4) using the following different methods to calculate $\frac{\partial F_0}{\partial x_i}$. The variance of final factor of safety is obtained as;

$\sigma_F^2 = f_0^2 \sigma_{F0}^2$ ....(12)
by considering that f_0 is deterministic.

5.2.1 Direct Differentiation assuming $n_i(\theta)$ is deterministic

Assumption to consider $n_i(\theta)$ as a deterministic parameter was made to simplify the calculations as discussed in section 5.1.1. The derived partial derivatives are;

$$\frac{\partial F_0}{\partial c'} = \frac{\sum \left(\frac{\Delta x_i}{n_i(\theta)} \right)}{\sum W_i \tan\theta_i} \quad \text{.........(13)}$$

$$\frac{\partial F_0}{\partial \varphi'} = \frac{\sum \left\{ (W_i + Q_i - u_i \Delta x_i) \frac{\sec^2 \varphi'}{n_i(\theta)} \right\}}{\sum W_i \tan\theta_i} \quad \text{.........(14)}$$

$$\frac{\partial F_0}{\partial u} = \frac{\sum \left(-\frac{\Delta x_i \tan\varphi'}{n_i(\theta)} \right)}{\sum W_i \tan\theta_i} \quad \text{.........(15)}$$

5.2.2 Direct Differentiation assuming $n_i(\theta)$ is probabilistic

As discussed in section 5.1.2, another analysis was done assuming that $n_i(\theta)$ is probabilistic.

5.2.3 Approximate Differentials using Divided Differences

These models were developed using the same concept as discussed in section 5.1.3, but using Janbu's simplified method.

6.0 APPLICATION TO A CIRCULAR FAILURE SURFACE

6.1 Example Details

A hypothetical problem that had been used in Giam (1989) was selected for the analysis. This problem was one of the several problems used in a survey conducted in Australia to evaluate the state of the practice of slope stability evaluation in the country. Hence, the problem had been analysed by number of different well-established complex programs and the factors of safety and the critical failure surface are accurately known.

The slope has a non-homogeneous soil profile with three soil types of somewhat different strength parameters. According to the strength parameters and geometry, a near circular failure surface can be expected. The section of the slope and the soil strength parameters are presented in figure 2. Uncertainties were quantified by the author, guided by some published values [Duncan (2000), Lee *et.al.* (1983)].

6.2 Results of the analysis

The slope was analysed using all five models developed for Bishop's simplified method. For the initial calculations total number of 8 slices were used and the factor of safety was slightly different from the referee values given in Giam (1989). But the analysis of the same slope using SLIDE and SLOPE/W computer programs gave values very close the referee values. Those programs, by default, consider 30 slices per a failure surface. Therefore, trials were made after increasing the number of slices and the results of the analysis where 12 slices were considered is presented. The results are presented in figure 3.

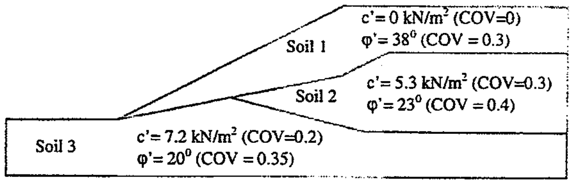


Figure 2: Geometry of Example 01

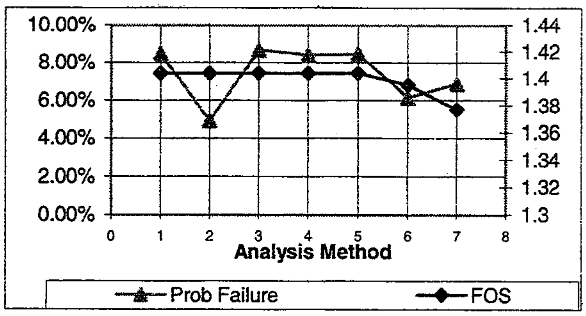


Figure 3: Results of Example 1

In figure 3, first five analyses correspond to the methods illustrated in 5.1.1, 5.1.2 and 5.1.3 (a) to (c) respectively. The penultimate analysis is the Monte-Carlo analysis using Bishop's method, and the final result correspond to Monte-Carlo analysis with Spencer's method. The FOS achieved from SLIDE program is 1.40, and the referee value is 1.39.

6.3 Discussion of Results

Results show that, as the number of slices is increased, the factor of safety converges to the referee value. Therefore, it can be justified that our probabilistic analysis had been performed for the critical failure surface.

It can be seen that, the probability of failure (p_f) values obtained from the procedures given in section 5.1.2 and the Monte-Carlo analyses are very close to each other. The analysis described in section 5.1.2 performs the exact evaluation of partial derivatives, and hence it can be concluded that it is a reasonable approach. All other four models developed here gave slightly higher (therefore conservative) probabilities of failure (p_f). But all the p_f values agree within 3-4%. Therefore, simple models developed can be used in practice to provide reasonable results.

7.0 APPLICATION TO A NON-CIRCULAR FAILURE SURFACE

7.1 Example Details

This example (figure 4) is also obtained from Giam (1989). The geological feature of a weak layer controls the position and shape of the critical surface and most critical failure surface would be non-circular. As such Janbu's simplified method was used in the analysis. Bishop's analysis was also performed to show its unsuitability.

7.2 Results of the Analyses

Analysis was performed with five models developed using the Janbu's simplified method. Monte-Carlo analyses were also performed using SLOPE/W program. When Janbu's analysis is performed in SLOPE/W, the final result gives only F_0 and σ_{F0}. Hence, the Janbu's modification factor f_0

was manually obtained and included in the final result to find out F and σ_F.

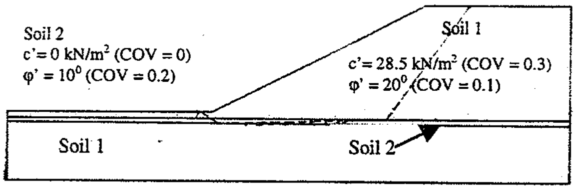


Figure 4: Slope Geometry of Example 2

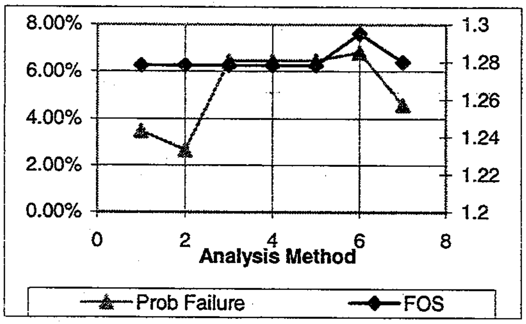


Figure 5: Results of Example 2

In figure 5 the first five analysis methods correspond to the developed models as discussed from 5.2.1 to 5.2.3. Sixth analysis is the Monte-Carlo analysis using Janbu's simplified method and the last one is the Monte-Carlo analysis with Spencer's method. The referee value of factor of safety is 1.27-1.29. When the slope was analysed using the Bishop's simplified method with a circular failure surface, the factor of safety obtained was 1.50 and the probability of failure was only 0.1%.

7.3 Discussion of Results

The results show that the probabilities of failure obtained from different methods are in agreement within 3% provided that appropriate failure mechanism and the critical failure surface is selected. The probability of failure achieved from Bishop's analysis, for slope geometry where critical failure surface is unlikely to be circular, is in gross error.

8.0 APPLICATION TO REHABILITATED WATAWALA EARTHSLIP

Watawala earthslip is one of the heavily studied landslides in Sri Lanka. Major part of the investigations had been carried out by the National Building Research Organization (NBRO), and the details for our study were extracted from Rajaratnam and Bhandari.(1994), and Rajaratnam (1995). Figure 6 shows the details of the selected failure surface. Analysis was performed using residual soil parameters c'=0 and φ'= 16^0. Janbu's analysis was performed and the factors of safety values obtained by the author were 0.87 for highest piezometric condition (prevailing from May to October) and 1.44 for the lowest piezometric condition (prevailing from November to April). This clearly shows the instability of the slope during wet condition.

Then the uncertainty was introduced into the analysis and cov[φ] = 0.2, and $\sigma(u)$ = 20 kN/m^2 were considered. When the same failure surface was analysed in probabilistic terms the resulting values of probability of failure are 71.99% for highest piezometric condition, and 8.75% for the lowest piezometric condition. This result clearly shows the large risk associated with the slope when the piezometric line is at its maximum. Also, it is seen that this probability of failure is remarkably reduced when the piezometric line is lowered. Surface and subsurface drainage measures adopted in the rehabilitation program has lowered the phreatic surface and the slope is safe to date.

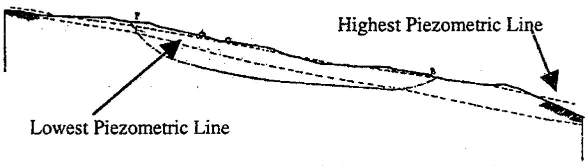


Figure 6: Sectional view of Watawala Earthslip

9.0 CONCLUSIONS

Probabilistic Analysis can be applied to the slope stability analysis by incorporating some rigorous computations as well as by using some simplified approximate methods. Results of approximate analyses are also not much different from those of the rigorous methods.

If the appropriate published values of coefficients of variation are used to quantify the variability, probabilistic analysis does not require any additional data than what is required for the deterministic analysis.

Identification of an analytical model appropriate for the prevailing geotechnical conditions is of key importance in the probabilistic analysis as well.

Probability of Failure is a good index to quantify the stability of slopes when the parameters carry certain degree of uncertainty, and it is proposed that probability of failure should not be viewed as a replacement for the factor of safety, but as a supplement. However, still there is no universal agreement about the acceptable values of probability of failure.

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