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Organised by the
SRI LANKAN GEOTECHNICAL SOCIETY

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Improvement of Soft Peaty Clay by Electro Osmosis Consolidation

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Abstract

Finding cost effective and speedy methods to improve the soft peaty clay is a major challenge faced by Sri Lankan geotechnical engineers today. Electro Osmosis Consolidation is a method that can be used very effectively for these types of extremely soft organic clays. Already, the research conducted at the University of Moratwa has shown that the primary and secondary consolidation characteristics and undrained shear strength of peaty clays can be significantly improved by Electro Osmosis Consolidation. In this research the field application of Electro Osmosis Consolidation was simulated with the prefabricated vertical drains (EVD) made of conductive plastics and copper plates. The process was affected by the corrosion of electrodes and cracks developed on the surface of the sample. Moisture contents were reduced and undrained shear strength was improved, but compressibility characteristics were not improved to anticipated levels.

1. Background

Improvement and stabilization of soft peaty clay still remains a challenge facing Sri Lankan geotechnical engineers. Due to unavailability of lands with good sub soil conditions, engineers are compelled to use land underlain by soft peaty clays in infrastructure development projects such as highways. This is one of the major problems faced during the constrictions of southern expressway project. In some locations peat layers of thickness in the range of 8m-10m were encountered.

Excessive settlements and shear failure upon loading are the main problems encountered. Pre loading had been used in some instances, but due to the very low shear strengths, the required fill has to be placed in several stages and the duration of the project increased considerably. Prefabricated vertical drains may be used to accelerate the consolidation process and reduce the time period. In some sites where peaty clays of extremely soft consistency were encountered, the placed fill has simply gone in to the ground. Geo textiles and geogrids were used only with limited success.

The process of excavation and removal was used in Southern expressway project. This has many adverse environmental effects.

As such, development of a method to improve the extremely soft peaty clay in situ, is of utmost importance. In electro osmotic consolidation water is drawn out of the soil without application of a physical load. As such, no shear failure would occur.

2. Studies on Electro Osmosis Consolidation at University of Moratuwa on Peaty clays

There were no previous records of application of electro Osmotic Consolidation to peaty clays anywhere else in the world. Pioneering studies on Electro osmosis consolidation of peaty clay were done at the University of Moratuwa. Measurement of basic electric properties such as electro osmotic permeability, conductivity and efficiency indicated that process would be successful in peaty clays.

Studies done at the laboratory level (Kulathilaka et al 2003, 2004) using an experimental setup fabricated to provide one dimensional electro osmotic consolidation, showed that the moisture content, undrained shear strength, compression index, coefficient of volume compressibility and coefficient of secondary consolidation were significantly improved by this technique.

Subsequently, (Kulathilaka and Sagarika 2005) studies were done in a large Perspex box using perforated stainless steel tubes placed in grids as electrodes, to simulate a field application of electro osmotic consolidations. Some rows were selected as anodes and others as cathodes. However, anticipated improvements could not be achieved due to the corrosion of electrodes.

Therefore, tests were done with electrodes made of conductive plastics named electro kinetic geosynthetics (EKG). In these tests, high voltage gradients could be used for reasonably long durations, without causing any deterioration in the electrodes. There were problems due to development of excessive cracking during the process. Once these cracks were closed manually, the process could proceed without any hindrance. Tests were done with regular polarity reversals and uniform level of improvements were achieved (Kulathilaka 2007).

Tests were done with different arrangement of EKG electrodes (Kulathilaka 2008). It was found that significant improvements in strength and stiffness could be achieved when the initial water content was around 400%.When it is increased to 600%, the process was not so effective.

In this research project Electro Osmosis Consolidation process was tried out on peaty clays with a different type of electrodes named as Electrical Vertical Drain (EVD) which is made with conductive plastics. Conductivity was further enhanced by the use of a copper plate in between the plastics as shown in Figure 1.

Figure 1: Electrode Made from Prefabricated Vertical Drain

3. Experimental Procedure

3.1. Preparation of Peat Sample

Peat samples were obtained from southern highway project and sample was prepared to a uniform consistency. Since peat contained lot of partially decayed timber pieces, sand etc, these were removed. Thereafter, the sample was mix thoroughly with a cake mixer after adding some water to maintain the moisture content required.

Remoulded peat sample was placed in a box of dimensions 600 × 600 × 600 mm made of 5 mm thick transparent Perspex sheets up to a height of 300mm and samples were obtained for the consolidation test and initial moisture content determination, at the same time.

Electrodes were inserted in to the prepared sample while special care was takes to install them vertically. The tips of the electrodes were plugged with a copper shoe to prevent peaty clay entering to electrodes. Four electrodes were used in two lines and two of them were used as cathodes while the other two were act as cathodes. Spacing between anode line and cathode line is kept as 250mm. Electrode configuration is shown in Figure 2.

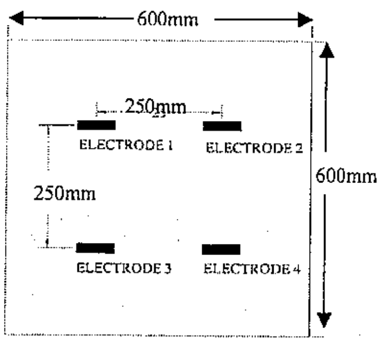

Figure 2 : Electrode Configuration

3.2. Consolidation under self-weight and application Voltage

The remoulded peaty clay sample was allowed to consolidate under its own weight for some time. During this time water came out through all the vertical drains. The collected water volume and pH value of the removed water were measured. When the water ceases to come out to the surface it could be assumed that the process of self weight Consolidation has been completed. The electrical treatment after this stage.

3.3. Application of Voltage Difference

Voltage difference between anode and cathode is the driving force in the electro-osmotic consolidation. Hence, it is necessary to measure the pattern of variation of voltage between anode and cathode as the electrical treatment progresses. In order to achieve this, voltage probes were installed at 50mm intervals in between anode and the cathode. Arrangement of electrodes and voltage probes are illustrated in Figure 3.

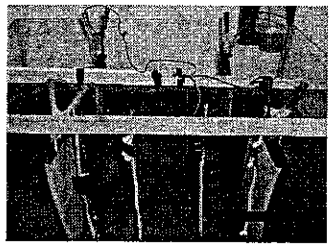

Figure 3: Arrangement of Electrodes

Electrical treatment was started by providing a voltage difference of 7.5V and it was continued until the water ceased to come out to the surface. After that polarity reversal was done while increasing the voltage difference to 15V. In the next stage voltage was increased to 30V and in the last stage to 45 V. Polarity was reversed at each increase. In all stages, the volume of water collected at cathode and the pH value of the water were measured. The variation of cumulative volume of water with time is given in Figure 4

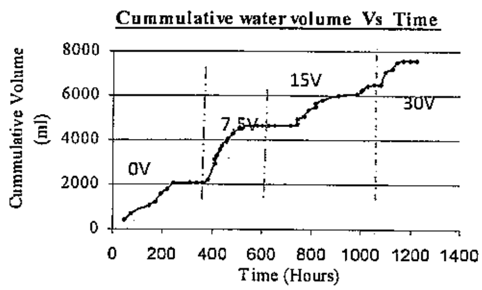

Figure 4: Cumulative Water Variation

The pH value of the water discharged was presented in Figure 5.With a given polarity, the pH value of the water discharged near the cathode increased gradually up to a value of 10-12.When polarity was reversed, water starts to discharged from the former anode where hydrogen ions had been discharging. Thus the pH value of the water discharged is low initially. As time progresses, with hydroxyl ions releasing at this new cathode, pH value keeps on increasing. This behaviour confirms the progression of the electro-osmosis consolidation.

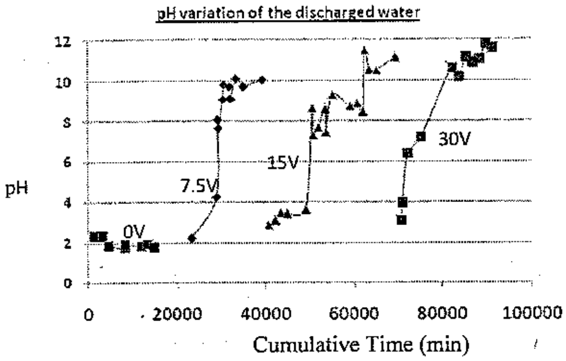

Figure 5: pH Variation with Time

4.4. Measurement of Voltage Gradient

Variation of the voltage between anode and cathode was measured at four points with the help of voltage probes. Graphs of voltage variation across the sample were plotted and presented in Figure 6. Voltage distribution along the sample was not uniform. The gradient near the cathode was high indicating the presence of a zone of high resistance.

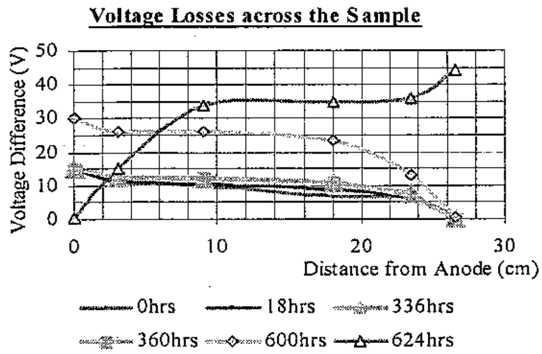

Figure 6: Variation of Voltage with Distance from Anode

4.5 Obtaining the Undisturbed Samples

At the end of the electrical treatment process, undisturbed samples were taken to establish the improvements achieved. Before obtaining these samples, a layer of thickness of about 50mm was removed from the surface to minimise the effect of adding water in to the surface to prevent the surface drying. Four undisturbed samples were taken to the steel tubes and sealed until the undrained triaxial tests are carried out. At the same time samples were taken to determine the moisture content test at different locations and at different depths. Another three samples were taken to conduct the consolidation test.

5. Assessment of Improvements

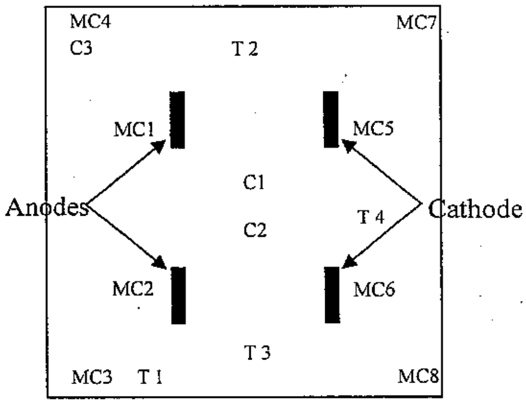

Figure 7: Locations of Samples Taken

5.1. Moisture Content (MC)

The locations selected for the determination of moisture content after the test are Moisture Content to MC8 (Figure 7). The obtained moisture content values are presented in Table 1.

Electrodes adjacent to Location 1 and 2 acted as anodes for the final voltage gradient and the MC of those locations are considerably lower when compared to the other locations. A marginal reduction of moisture content could be seen with the depth.

Location	Moisture Content (%)	
	75mm depth	200 mm depth
MC 1	103.5	96.01
MC 2	104.53	95.68
MC 3	155.65	141.75
MC 4	176.7	150.13
MC 5	234.39	219.18
MC 6	210.38	195.44
MC 7	208.79	130.91
MC 8	192.15	138.74

Table 1: Results of the Moisture Content Test

5.2. Shear Strength

The main problem associated with soft peaty clay was its extremely lower shear strength. It is not possible to carry out the tri axial test on untreated sample as it failed due to its own weight. Thus the Vane shear test was carried out to find out the initial strength of the untreated peat sample. It was found to be about 4 kN/m^2. After the treatment undrained tri-axial tests were conducted on the undisturbed samples taken from Location T1, T2, T3 and T4 in Figure 7. The undrained shear strength values obtained are shown in Table 2. It could be seen that consider able improvements of shear strength had been achieved.

Sample Location	% Initial M.C.	% Strain at failure	Deviator Stress at failure (kN/m²)	C_u (kN/m²)
T1	207	8.0	28	14
T2	183	8.4	66	33
T3	302	12.0	46	23
T4	254	14.0	38	19

Table2: Results Obtained from Tri-axial Test

5.3. Compressibility Characteristics

5.3.1 Reduction in Void Ratio

When electrical treatment was performed pore water is drained out from the cathode. As a result the voids in the soil mass will be reduced. The e Vs logσ graphs for three treated samples are shown in Figure 8 along with that from an untreated sample.. Samples 1 and 2 were taken from the middle of the electrodes while the sample 3 was taken from a place outside the anode at the final polarity reversal. Tested locations are shown as C1, C2, and C3 in Figure 7.

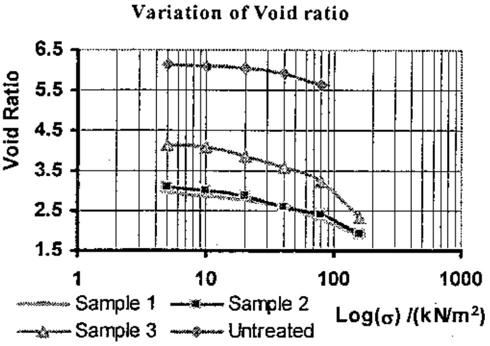

Figure 8: Comparison of Void Ratios

5.3.2. Reduction in Coefficient of Volume Compressibility, m_v

Coefficient of volume compressibility is an alternate parameter used to calculate the settlement due to stress increment imposed on a soil sample. The variation of coefficient .of compressibility with stress level is presented in Figure 9, for the three treated samples and the untreated sample. It could be seen from Figure 9 due to that that there is no significant improvement in mv values to the electrical treatment.

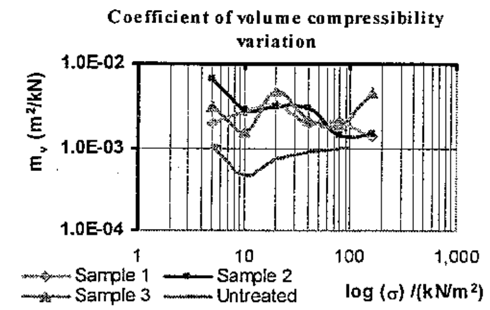

Figure 9: Comparison of Coefficient of Volume Compressibility

5.3.3 Reduction in coefficient of Secondary Consolidation

The values of coefficient of secondary consolidation ($C\alpha$) in electrically treated samples have not reduced from that of untreated samples as anticipated. Comparison of $C\alpha$ with treated and untreated samples are presented in Figure 10.

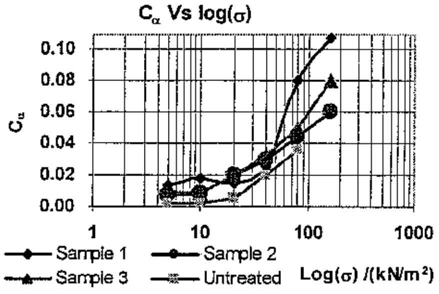

Figure 10: Comparison of $C\alpha$

6. Conclusions

Based on the results of this research, the following conclusions may be drawn.

- The EO Process achieves considerable reduction in the water content. It was maximum near anodes and minimum near at cathodes.
- The level of improvement achieved in compressibility characteristics such as C_c, m_v and $C\alpha$ in three-dimensional condition was much less than that achieved under one dimensional study done earlier.
- The improvement of shear strength due to the electrical treatment was much higher at anode.
- pH of water expelled at cathode increased with time due to EO. This confirms the process
- The amount of current that passed through the sample was very low and it was not measurable.
- There is a significant voltage loss at the anode and the cathode, the one at the cathode being the larger of the two. Voltage loss is uniform over the middle part of the sample.

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Rain Induced Slope Failures in Unsaturated Residual soils

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Abstract:

Rainfall-induced slope failure is a common geotechnical problem in the tropics where residual soils are abundant. Residual soils are formed due to the weathering of rocks and lie at the location of the parent rock. They are characterized by the significant variations in the level of weathering and the composition of weathered product over short distances.

Quite often, the ground water table is low and these soils are in an unsaturated state. The shear strength of the unsaturated soils is enhanced by the presence of matric suction. With the infiltration of rainwater the matric suction will be depleted for some depth. As the rain progresses, the depth of depletion increases. This aspect was studied in this research with an infiltration model. The model was applied initially to a slope with a homogeneous soil and later to a slope where two layers of significantly different levels of weathering exists.

Thereafter, the variation of the safety margins of the slope as the rainfall progressed was assessed using the limit equilibrium approach. Bishop's simplified method and Spencer's method were used for possible circular and non-circular modes of slope failures. Subsequently, the variation of the probability of failure with time was evaluated by the Monte-Carlo approach.

***Key Words**: Slope stability; Matric Suction; Rainfall; Unsaturated soil; Probabilistic analysis; Infiltration*

Background

Rainfall-induced slope failures constitutes one of the most common geotechnical hazards in tropical regions such as Sri Lanka. Landslides in residual soils are often triggered by prolonged extensive rainfalls. Conditions leading to these failures are caused by the loss of matric suction and rise in pore-water pressure as a result of infiltration of rainwater. Infiltration into soils depends on several factors such as soil structure, antecedent soil moisture, soil exchangeable medium, infiltrating water quality, and the status of the soil air.

The objective of this research is to study how the infiltration of rainwater changes the pore pressure regime in a slope and how it will affect the stability of the slope consequently. Safety was assessed in terms of both the factor of safety and the probability of failures.

Methodology

The analysis was performed on two typical cut slopes in the southern expressway project. Considering the highly variable conditions encountered, two cases with different sub soil conditions were studied in the project. The case 1 was a slope made of a uniform residual soil. In case 2 a layer of highly weathered rock is underlying the residual soil. The second layer is with greater shear strength and lower permeability. This is a simplification of a typical situation seen in many slopes in the country.

Infiltration of the rainwater through the unsaturated soil slope and the resulting changes in the pore pressure regime were analyzed by the SEEPW-2007 computer package. Thereafter, the stability of the slope was analyzed using the SLOPE/W-2007 package incorporating the pore water pressures derived from the SEEP/W analysis. Stability analyses were carried out for different time steps ranging from 1 day to 5 days, for rainfalls of intensities 5mm/hr, 20mm/hr and 40 mm/hr respectively. Initially, the values of factor of safety were computed and subsequently the probability of failure was evaluated.

Considering the possibility of occurrence of both circular and non-circular modes of failure, Bishop's simplified method and Spencer's method were used in the analysis. The probabilistic analysis was done with the Monte-Carlo formulation.

Saturated and Unsaturated Soils

A soil above the ground water table would have both water and air occupying the void spaces. The pore air pressure (u_a) is usually equal to the atmospheric. The pore water pressure (u_w) above ground water table is less than atmospheric and the variation is presented in a simplified form in full line in Figure 1. Providing a limit to the practically possible maximum matric suction, a more realistic distribution is presented in broken lines in Figure 1. (In the analysis of unsaturated soils, the two stress variables σ-u_a termed net normal stress and (u_a-u_w) termed matric suction are used.)

Soil Water Characteristic curves (SWCC)

SWCC which shows the variation of the matric suction with the volumetric water content is the most fundamentally important feature in the analysis of the behavior of an unsaturated soil.

The volumetric water content is denoted by θ and is defined as;

$$\theta = \frac{V_w}{V_s}$$

V_w- Volume of water
V_s- Volume of soil

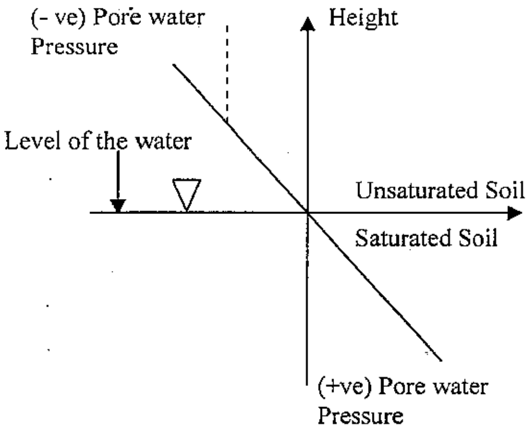


Figure 1: Pore water pressure distribution

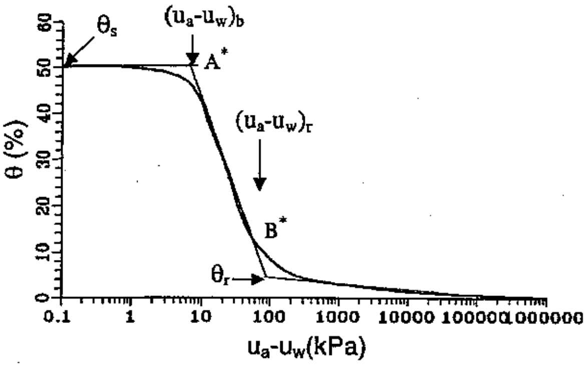


Figure2: Idealized Soil-Water Characteristic Curve

Figure.2 shows an idealized SWCC with two characteristic points A* and B*.Point A* corresponds to the air-entry value $((u_a\text{-}u_w)_b)$, and B* corresponds to the residual water content (θr). As shown in Figure 2, prior to A*, the soil is saturated or nearly saturated, and can be treated as a saturated. Beyond B*, there is little water in the soil, so the effects of water content or negative pore-water pressure on soil behavior may be negligible.

What is of great concern in an unsaturated soils is the stage between A* and B*, in which both air and water phases are continuous or partially continuous, and hence the soil properties are strongly related to its water content or negative pore-water pressure.

Hydraulic Conductivity Function

In consideration of flow through an unsaturated soil, the major difference from the saturated case is the existence of a storage term. This storage term is not a constant but depends on the matric suction. There will be no overall volume change during the infiltration process.

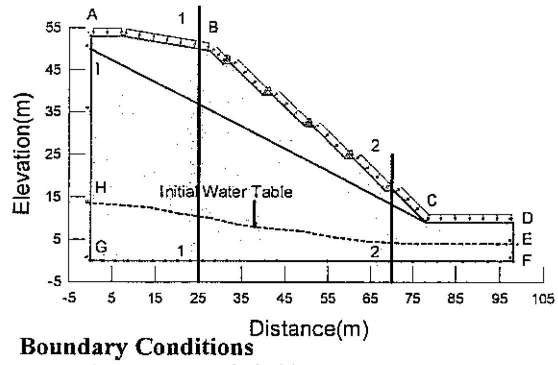
Water flows only through the pore space filled with water (air-filled pores are not conductive to water). As such, the percentage of the voids filled with water (i.e., degree of saturation) is an important factor.

For an unsaturated soil, the water coefficient of permeability depends on the degree of saturation or negative pore-water pressure of the soil. It can be estimated from the saturated permeability and the characteristics of the SWCC such as air-entry value, desaturation rate, saturated, and residual volumetric water contents *(Fredlund, and Rahardjo 1993).*

Infiltration modeling using SEEP/W-2007

The geometry of a typical cut slope and the boundary conditions utilized for the transient seepage analysis are shown in Figure.3. In the homogeneous soil (case 1) the entire slope is made of residual soils and in the case 2, where weathered rock is underlying a thin layer of residual soil, the boundary between the two soils is shown by line IC in Figure 3.

A boundary flux, *q*, equal to the desired rainfall intensity, *Ir*, was applied to the surface of the slope. The nodal flux, *Q*, was taken to be zero at the sides of the slope above the water table and at the bottom of the slope to simulate a no flow zone (Figure 3). Equal total heads, *ht*, were applied at the sides of the slope below the water table *(Rahardjo et.al-2007).*



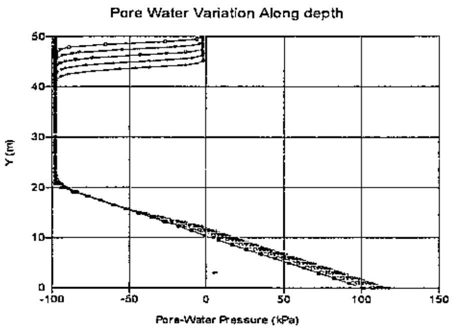
Boundary Conditions
AB, BC, CD= Ir (Rainfall intensity)
AH, DE, FG=Q=0m3/s (No flow Boundary)
EF, GH=ht (Total head at sides)

Figure 3: Cut Slope Geometry, Selected Sections and Boundary conditions

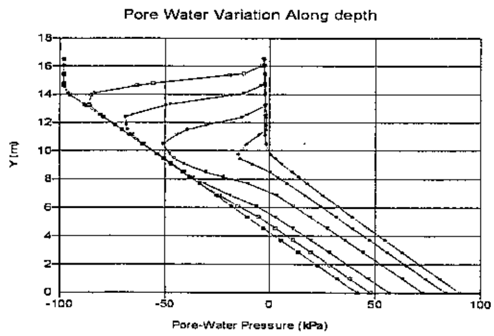
As the soil hydraulic properties, the accepted parameters normally appropriate for the residual soils were taken from the available literature *(Sun et.al-1998)*. The saturated hydraulic conductivities used are $1\times10^{-5}m/s$for residual soils and $1\times10^{-7}m/s$ for highly weathered rock. The saturated volumetric water contents used are *0.45, 0.40* respectively.

The pore water pressure changes in the slope were computed using a finite element formulation through SEEP/W program taking an initial time step of 0.5 hr, which was varied in an exponential manner as time progressed. In the analysis of this transient flow condition a steady state was assumed within a time step. Using the pore pressure values computed at nodal points, the distributions can be drawn for any section to visualize the effect. The pore water pressure distributions for Section 1-1 and Section 2-2 for rainfall intensities of 5mm/hr and 20mm/hr are presented in Figure 4 and Figure 5 respectively.

It could be seen that for a 5mm/hr rainfall, as rain progressed the negative pore water pressures near the ground surfaces approached zero at the top level (Section 1-1) and water table has arisen at the lower levels (Section2-2) (Figure 4 (a) and (b)). With a 20mm/hr rainfall, not only the matric suctions were lost but also positive pore water pressures were developed at the top level (Figure 5 (a)).At the lower levels, the development of the positive pore water pressure and the rise of the ground water table with the progression of the rain could be seen (Figure 5 (b)). When a highly weathered rock layer is present, the downward movement of water is hampered and water gets accumulated at the boundary. This leads to the development of a perched water table at the boundary and increased pore water pressure closer to the surface (Figure 6).

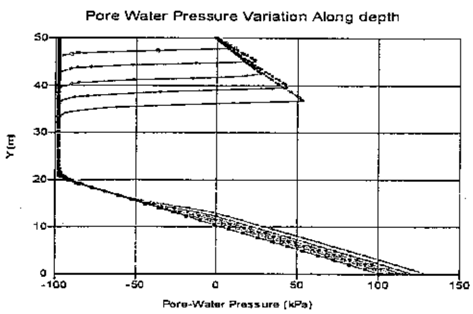


(a) For Section 1-1

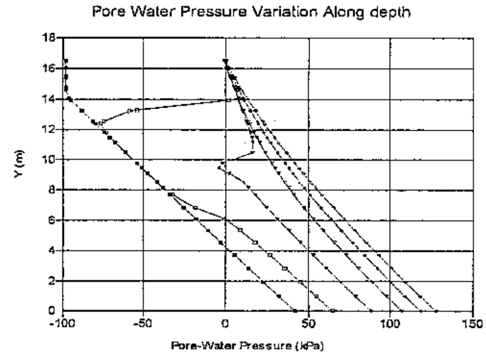


(b) For Section 2-2

Figure 4 –pore water pressure distribution for 5mm/hr rainfall



(a) For Section 1-1



(b) For Section 2-2

Figure 5 –pore water pressure distribution for 20mm/hr rainfall

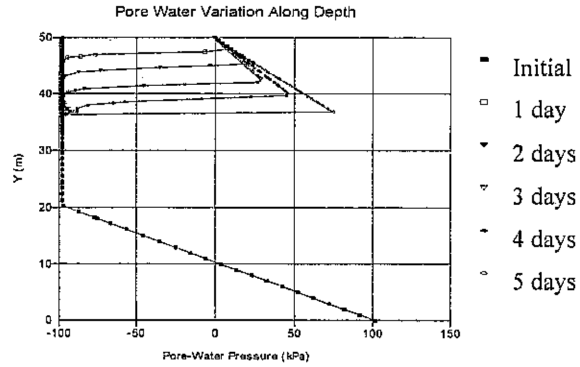


Figure 6 –pore water pressure distribution for 20mm/hr rainfall for Section 1-1 (Two layer system)

Slope Stability Analysis

With the loss of matric suction and development of positive pore water pressures over shallow depths, the potential failure surface could be non-circular. This aspect was considered in the analysis of the stability of the slope that was done through the limit equilibrium approach. Circular modes of failure were analyzed by the Bishop's simplified method and non-circular modes of failure were analyzed by the Spencer's method.
The factor of safety (*F*) is defined as;

$$F = \frac{\tau_f}{\tau_m} \qquad \text{-07}$$

Where τ_f=Shear Strength

$\tau_f = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b$ *(Fredlund, 1993).*

τ_m =Shear strength mobilized for equilibrium.

The minimum factor of safety (FOS) and the corresponding most critical failure surface at different times into the rainfall event were obtained using the SLOPE/W program. Mean value of Soil strength parameters and standard deviation used in the slope stability analysis are given in Table 1.

Soil type	(γ_{eff}) kN/m³	(c') kPa	(ϕ')	(ϕ_b)
Residual Soil	19±0.5	10±2.5	34 ° ±2.5	30 ° ±2.5
Highly weathered rock	20±0.5	25±2.5	40 ° ±2.5	38 ° ±2.5

Table 1: Soil strength parameters used for stability analysis and standard deviations

Results of Stability Model

The reduction of the factor of safety as rain progressed is summarized in Figure 8, for both cases of homogeneous soil and the two-layered system.

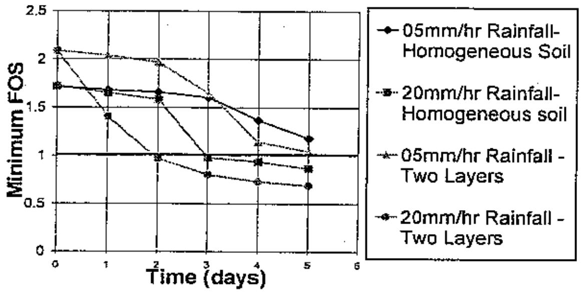


Figure 8: Plot of Minimum factor of safety VS time

It could be seen that in both cases the FOS reduced with time. In homogeneous soil the FOS is just above 1.0 after five days due to the 5mm/hr rainfall. But with the 20mm/hr rainfall the FOS approached 1.0 in the third day.

When there is a layer of weathered rock underneath the residual soil, the FOS was greater initially but reduced rapidly as the rain progressed (Figure 8). For the 5mm/hr rainfall the FOS approached unity after 5days and with the 20mm/hr rainfall FOS was less than 1.0 within two days due to the positive PWP developed. It could be seen that the critical failure surfaces are quite shallow (Figure 9).

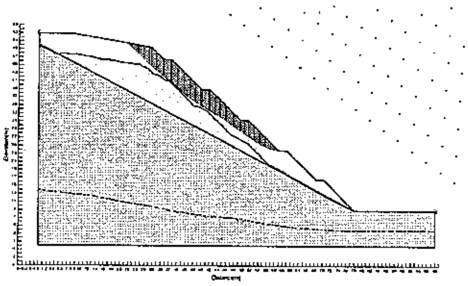


Figure 9: Shape of typical failure mass of two layers of soil slope

Probabilistic Analysis

Uncertainties in the shear strength parameters and pore water pressures used could be accounted for in a probabilistic analysis. The mean values and the standard deviations given in Table 1 are used in the analysis. In this research probabilistic Analysis was performed using the Monte-Carlo approach, a facility available in the SLOPE/W program. Here the stability analyses were conducted thousands of time varying the parameters within a range based on the mean value and the standard deviation. Using the large number of FOS values computed in this analysis, the probability the FOS would be less than 1.0 is computed and referred to as the probability of failure. Values obtained are given in Table 2 (to two decimals)

If the probability of occurrence of 5mm/hr or 20mm/hr rainfall can be obtained using available data, an annual probability of failure, A p(f) could be computed by;

A p(f) = p(f-SLOPEW) × p(occurrence of rainfall)

Decision regarding (remedial measures to be adopted) can be taken based on the annual probability of failure.

Time/days	0	1	2	3	4	5
5mm/hr (Uniform)	0	0	0	0	0	0.15
20mm/hr (Uniform)	0	0	0	64.7	68.6	99.0
5mm/hr (2 -layer)	0	0	0	0	0.7	18.6
20mm/hr (2 -layer)	0	0	67.6	100	100	100

Table2: Probability of Failure as a percentage

Conclusions

Failures in slopes made of residual soils are often triggered by rainfall. This behavior was clearly illustrated by modeling the infiltration process and analyzing the stability of the slope after inputting the resulting pore water pressures. The parametric study clearly indicated the effect of the duration and intensity of the rainfall.

If the probability of occurrence of a typical rainfall can be estimated with the help of available data, an annual probability of failure could be estimated. This parameter could be used for risk analysis purposes and in the deign of appropriate drainage measures to maintain acceptable level of safety.

Acknowledgements

We express our heartfelt gratitude and admiration to Prof. S.A.S Kulathilaka. His valuable commitment and dedication was a great encouragement for us.

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Development of Computer Programme Using MathCAD to Analyse Shallow Foundations.

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ABSTRACT

This programme using MathCAD software was developed to analyse shallow foundations, i.e finding out Factor of Safety, Ultimate bearing capacity and Settlement for given dimensions and to obtain minimum width, length and depth of the foundations that are satisfying allowable bearing capacity and allowable settlement. Foundation on sandy and clayey soils, different loading conditions and water table variation were considered. Programme was written for Spread, Rectangular combined and Mat footing. In the preliminary design, with the certain assumptions and column layout, user will be directed to select the type of footing. Soil parameters, column loads, loading and base conditions are the input. Dimensions will be increased by small increment and ultimate bearing and settlement will be calculated and compared with allowable values. The output dimensions are limited to maximum corresponding values. If the output values exceed the limiting values, an error message will be displayed and user can go for next type of foundation which may be more suitable. Factor of safety, Ultimate bearing capacity settlement, Dimensions of the footing and Depth of the footing are the output of the programme.

INTRODUCTION

This involves the study of different types of shallow foundation and develops programme using MathCAD to determine the bearing capacity and settlement calculations for those foundations. When the problems of shallow foundations are dealt, the complicated and repetitive calculations may be encountered. The above problem can be effectively solved using a computer programme. MathCAD, which consists of many distinguish features to solve engineering problems, has been used in this project.

All Civil Engineering constructions resting on the earth must be carried by some kind of interfacing element called foundation. The foundation is the sub structure which transmits the loads from the super structure to the underlying soil. The super structure brings the loads to the soil interface using columns-type members. The loads must be spread to the soil in a manner such that its limiting strength is not exceeded and the resulting deformations are tolerable. Shallow foundations accomplish this by spreading the loads laterally whereas the deep foundation distributes the loads vertically rather than horizontally.

In more general sense, shallow foundations are those foundations that have a depth of embedment to width ratio approximately less than four, when the depth of embedment to width ratio of a foundation greater than four, it may be classified as a deep foundation. Selection of foundation types mainly depends on soil conditions, load and total settlement. Foundations are designed such that allowable strength is not exceeded and resulting settlement is within to tolerable limits.

Shallow foundation has least width generally greater than the depth beneath the ground surface. The contribution from the shear strength (not the weight) of the back fill soil above the bottom of the foundation to stability is small and therefore is usually is ignored. Generally shallow foundations are treated as a separate category within foundation engineering but they are subdivided into a number of types according to their shape, size and general configuration, namely Strip foundation, Pad foundation and Raft foundation.

AIM

Develop a computer programme using Math CAD to analyse shallow foundation.

OBJECTIVES

- Study of different type of shallow foundations.
- Find out optimum dimensions for foundation with acceptable value of factor of safety and settlement.

SCOPE

The programme was developed to determine the minimum dimensions which are satisfying allowable bearing pressure and settlement and to determine the factor of safety, ultimate bearing capacity and settlement for a known foundation.

1. The following kind of foundations were analyzed,

- Pad foundation
- Strip foundation
- Raft foundation

2. Foundation was analysed under different type of soils, particularly,

- Sandy soil
- Clayey soil

3. Following different loading condition were considered,

- Vertically centrally load
- Vertically eccentric load
- Inclined load

4. Water table effect was also considered.

5. Ground level was assumed as horizontal.

SOFTWARE USED

Math CAD (version14.0) was used to develop a programme to design and analyse shallow foundation in an user friendly environment. Math CAD has been selected for the following reasons such as performing large amount of arithmetic calculation at a time with higher accuracy which lead to avoid human errors due to repetitive and complicated calculations, variables and functions which are included in the programming can be defined according to our requirements, use of unit system helps to derive the output with the required unit, etc. Unlike other program languages, here as language of mathematics has been used so that this software programming can be learnt easily without complications.

METHODOLOGY

1. Selection of a Foundation

Foundation is selected using the soil type, column loads, spacing between columns and available area. According to the soil type given, the bearing capacity of soil will be assumed and trial dimensions will be calculated and compared with spacing. If there is enough spacing available, pad footing can be selected otherwise combined footing can be selected. In combined footings there are two options i.e, strip foundation and raft foundation. If the footing area covered by pad footing or strip footing greater than fifty percentage of the total area raft foundation will be selected.

2. Soil Parameters

In foundation design, long term stability is considered. In the Design of footings in sandy soils soil parameter friction angle (Φ) is taken into account since sand is the cohesion less soil and is obtained from the standard penetration test (SPT), then field value will be corrected and weighted average of SPT N value is used in defining the friction angle (Φ). When clay is considered, soil parameters shear strength (c') and friction angle (Φ') are considered and obtained from the tri axial, consolidated drained test.

3. Bearing Capacity

Hansen's bearing capacity equations are used for calculating bearing capacity.

General:

When $\Phi = 0$,

4. Factor of Safety

Factor of Safety (FOS) =

Factor of safety 2.5 will be is selected. Factor of safety used here is higher when compare with design of super structure due to inhomogeneity and anisotropy of the soil.

5. Settlement

In sandy soils immediate settlement only is considered and the maximum allowable settlement is the 25mm. In clayey soil immediate and consolidation settlements are there. But only consolidation settlement is considered. Because of the immediate settlement is small when compare with the consolidation settlement and the immediate settlement is possible to occur when during the construction.

In clay, soil under foundation is divided by layers up to a depth at which stresses due to the load is negligible(10% of the applied pressure),and settlement of each layer is calculated and added up, hence final consolidated settlement is obtained.

6. Flow Charts

The main objective of this programme is achieved by representing the design in logical flow charts, which used to demonstrate appropriate analysis and design equations. The flow charts are efficiently used to develop program in MathCAD.

Selection of Foundation

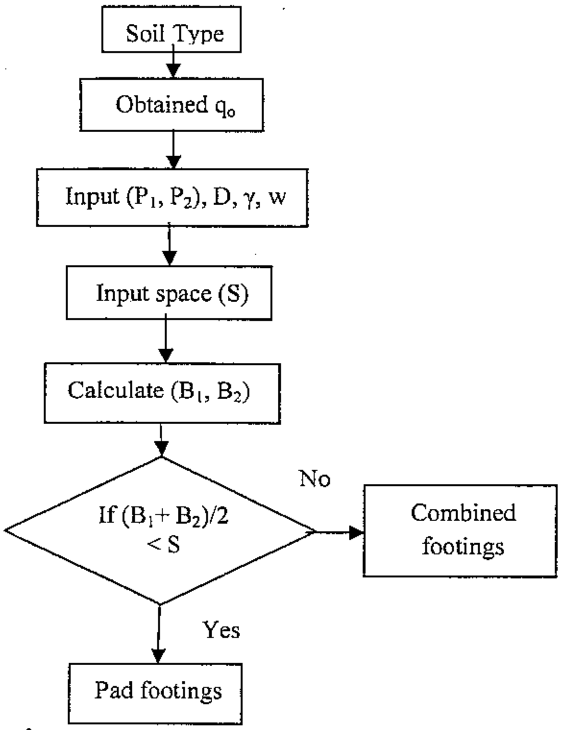


Chart1: Flow chart for Selection of Foundation

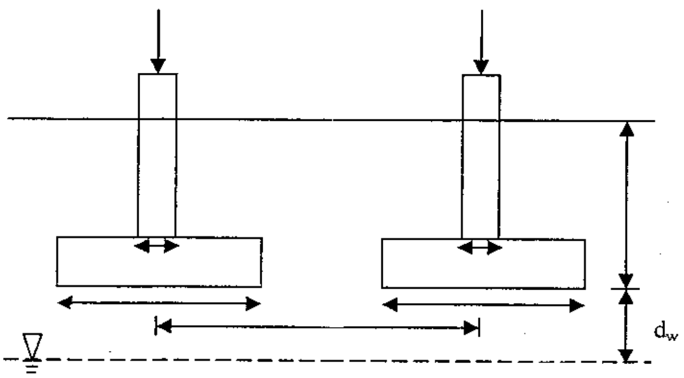


Figure 1: Typical layout of the adjacent foundations

Correction for SPT 'N' Value

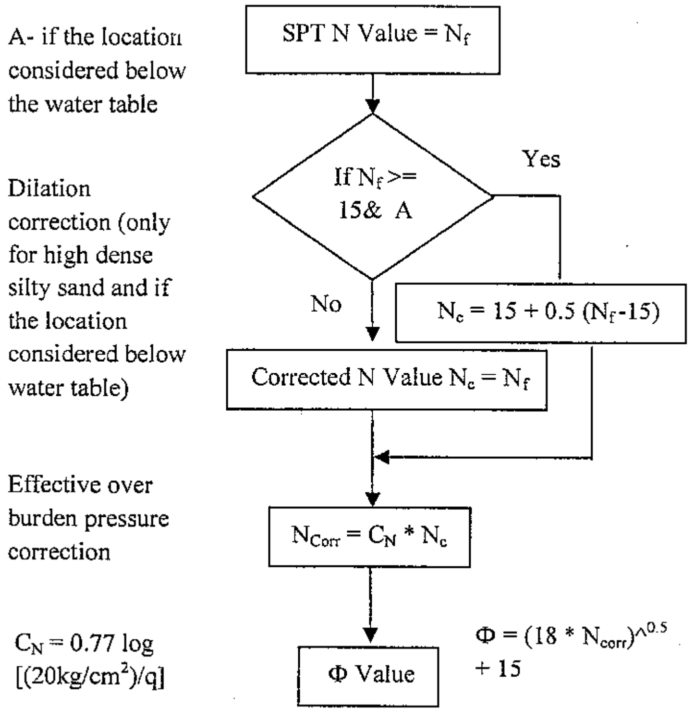
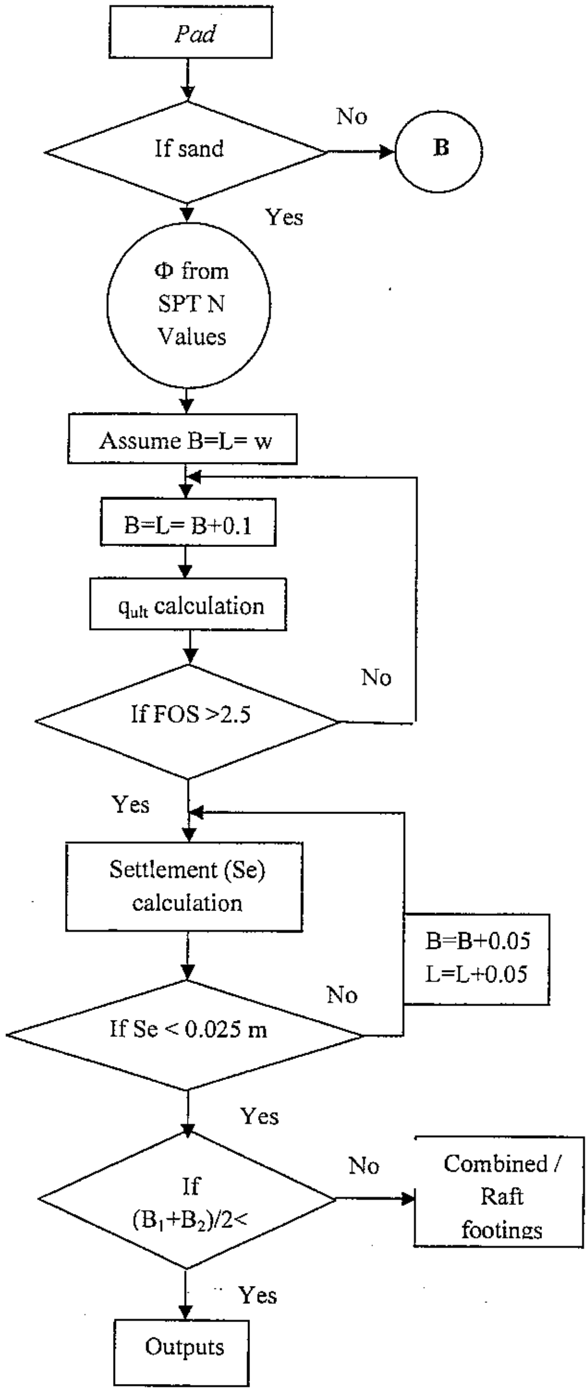


Chart 2: flow chart for SPT 'N' value correction

CONCLUSIONS

since our study is a software developing project, we realized the importance of writing proper flow charts before writing programme and converting the flow chart into programming is quite to achieve. More consideration is given to develop user friendly input screens, options and output screens, more importantly, we have tried to move away using computers to do the routine calculations and to do the programming in some intelligent way, that is through optimize design.

We think, we have achieved quite success in our objectives, and we are now confident that we can programme any Engineering related designs.



B- Flow chart for analysis of Foundation

under clayey soil

Chart 3: Flow chart for pad foundation

ACKNOWLEDGEMENT

We would like to express our gratitude to our project supervisor, Dr.(Mrs).D.De S.Udakara for encouraging us to reach some knowledge and presentation skills in a project work, and for helping us to get lot of ideas to work out our project. We also would like to thank other panel advisers, Prof.S.B.S.Abayakoon, Dr. L.C.Kurukulasuriya, Dr.G.S.Gurusinghe, Dr.Vasantha Wickramasinghe and Dr.U.de S.Jayawardhana , for encouraging us to reach some knowledge and presentation skills in a project work and for giving some ideas about our project. We also would like to thank all of the members of our panel for helping and encouraging us at the panel time and out of that time. Finally, we thank everyone who helped us in various ways to move our project work.

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Development of the Computer Programme Using MATHCAD to Analyse Retaining Walls

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ABSTRACT

Gravity retaining walls and cantilever retaining walls are designed by developing the programme with using MathCAD (version 14). The programme is used to find the effective dimensions of retaining walls, factor of safety for overturning and sliding and maximum and minimum pressure under the foundation by using Rankine's earth pressure theory. The programme is developed by considering geotechnical aspect of retaining wall design. Each type of retaining wall's programme has two parts. It depends on what type of backfill is used. Those are horizontal backfill and slope backfill. In case of horizontal backfill is applicable for any type of soil but in case of slope backfill is applicable for only cohesion less soil. Surcharge load and line load which are applicable to programme. In programme height and material properties of retaining wall, External loads and soil properties are input data and effective dimensions of retaining walls, factor of safety for overturning and sliding and maximum and minimum pressure under the foundation are calculated.

1.0 INTRODUCTION

1.1 General Background

A retaining wall is a structure that holds back soil or rock from a building, structure or area. Retaining walls prevent down slope movement or erosion and provide support for vertical or near-vertical grade changes. Cofferdams and bulkheads, structures that hold back water, are sometimes also considered retaining walls. Retaining walls are generally made of masonry, stone, brick, concrete, vinyl, steel or timber. There are several types of retaining walls, including the traditional gravity, semi gravity, cantilever, and counter fort retaining wall that made of plain and concrete. Retaining wall can also be constructed from other material, such as gabion, reinforced earth, and sheet piles.

Among today's competitive business environment Engineers have to aware about the recent development in engineering applications and should always use innovative, cost and time. The time factor takes the main role in the recent environment. Therefore it is a great challenge to be thought of time.

While designing, any type of retaining wall, more routing and complex calculations has to be done in order to satisfy all the standard requirements. If it is going to be design manually it will be inefficient and tedious work. Therefore manual design procedure cannot be adopted effectively for today's competitive world. Even though there are so many soft ware's available in the market for designing and analyzing purpose of retaining walls. But using with MathCAD software is creating a design in the least amount of time,

Most of the engineering problems can be solved with the help of computer each engineer should know to solve the engineering problem with help of computer in future to develop his carrier and to be good position.

1.2 Objective and Scope of the project

To develop programme using MathCAD for retaining wall analyze and retaining wall such as,

- Geotechnical design of gravity retaining walls
- Geotechnical design of cantilever retaining walls

To get the results by input the soil parameters, height and material properties of retaining wall and allowable bearing pressure under its foundation. The design part is done by based on Rankine's earth pressure theory. The design is done under the two types of backfill conditions with surcharge and line loads.

2.0 LITERATURE REVIEW

2.1 Stepped Gravity Retaining Wall

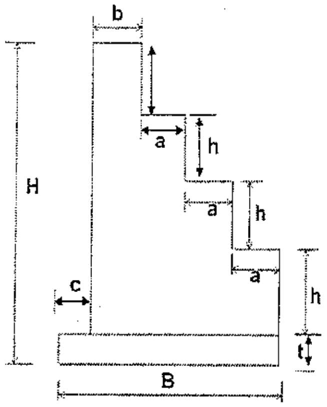


Figure1: stepped gravity retaining wall

Dimensions of retaining wall should be B>H/3, a>0.15m, b>0.30m, c>0.50m, h>0.60m and t>0.15m.

2.2 Plain Gravity Retaining Wall

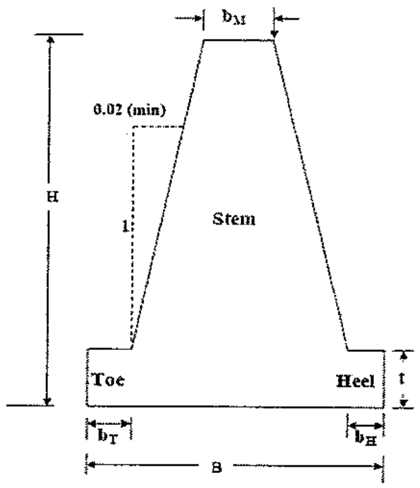


Figure2.Plain gravity retaining wall

Dimensions of retaining wall should be B= (0.4H-0.7H), b_{um}>0.3m,b_T=b_H= (0.12H-0.17H) and t= (0.12H-0.17H).

2.3 Cantilever Retaining Wall

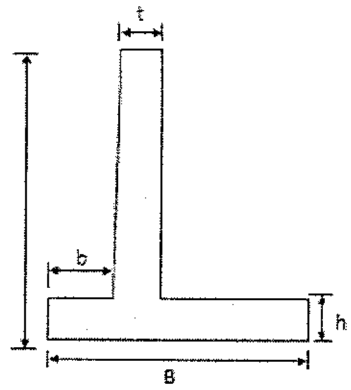


Figure3: Cantilever retaining wall

2.4 Theory

The fundamental theory for calculating lateral earth pressure is Rankine's earth pressure theory. There are two types of lateral earth pressure to be needed for retaining wall design which are active and passive lateral pressure. Lateral earth pressure coefficients to used in retaining wall design as shown below,

Horizontal back fill,

$Ka=\tan^2(45-\phi/2)$, $Kp=\tan^2(45+\phi/2)$

Slope back fill,

$$Ka = \cos\beta \cdot \frac{\cos\beta - \sqrt{(\cos\beta)^2 - (\cos\Phi)^2}}{\cos\beta + \sqrt{(\cos\beta)^2 - (\cos\Phi)^2}}$$

$$Kp = \cos\beta \cdot \frac{\cos\beta + \sqrt{(\cos\beta)^2 - (\cos\Phi)^2}}{\cos\beta - \sqrt{(\cos\beta)^2 - (\cos\Phi)^2}}$$

Where Ka, Kp, β and ϕ are respectively active lateral, passive lateral earth pressure coefficients, slope angle and soil internal friction angle.

3.0 METHOD OF ANALYSIS

In geotechnical analysis of retaining wall with lateral pressure known, the structure as whole is checked for stability. The structure is examined for possible overturning, sliding and bearing failure.Dimensions of retaining wall should be B=(0.4H-0.7H), h=t=(H/12-H/10) and b=B/3

Check for Overturning

The factor of safety against overturning about the toe,

$$FOS = \frac{\sum M_R}{\sum M_O} \geq 2.0$$

Where, □M_R = sum of the moments of forces tending to overturn about toe

□M_O = sum of the moments of forces tending to resist about toe

If check is not satisfy, while increasing base width of retaining wall to achieve the satisfy requirements.

- **Check for Sliding along the Base**

Passive zone is not considered as the risk of cutting a drain is present. Then only resistance for sliding is provided between the soil and the foundation therefore friction between the foundation and soil should be known.

The factor of safety against sliding,

$$FOS = \frac{F_R}{F_S} \geq 1.5$$

Where, F_R = force resisting to sliding
F_S = forces causing to sliding

If check is not satisfy, there are two option to achieve the satisfy requirements which are to widening the base width of retaining wall and provide the key under the foundation to increase passive resistance under the foundation.

- **Check for Bearing failure**

Due to the forces are acting on the wall there will be a pressure distribution developed upwards to resist the resultant forces vertical and horizontal as shown figure 4

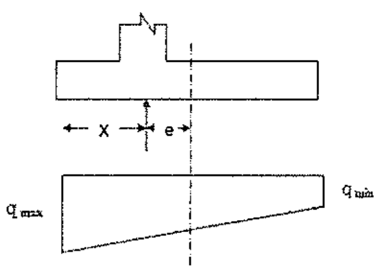


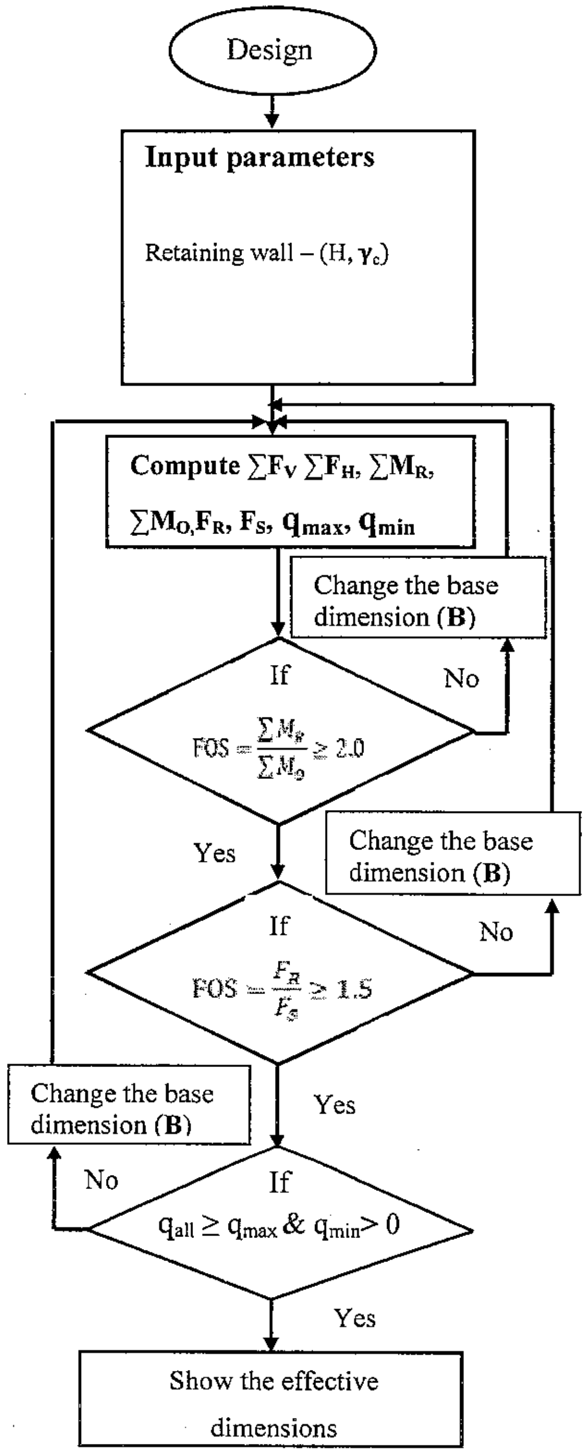
Figure.4.Disturbution of pressure

If, $q_{all} \geq q_{max}$ and $q_{min} > 0$ (No tension developed under the base) check is ok.

Otherwise to widening the base width of the retaining wall to be avoided the bearing failure.

4.0 PROGRAMME DEVELOPMENT

Flow Chart for Design of Retaining wall



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

- It should be easy to use and should be user friendly.
- The programme should be user interactive
- The programme should be effective i.e. producing the intended results from programme

To meet these requirements in the programme for every types of retaining wall design, inside the programme structure initially assumptions which is made for design the retaining wall, next input parameters such as retained soil and base soil parameters, retaining wall parameters and loads. The programme is developed in such a way that when a wrong action is performed it would be indicated and right action would be displayed, therefore from that advantages programme errors can be minimize.

5.0 CONCULSION

In this project have developed computer programme using MathCAD which can be used for design and analysis of two types of retaining walls. They are gravity retaining wall and cantilever retaining wall. Project have given most of our consideration to write programme to find the effective dimensions of retaining wall and also to develop user friendly input screens and output screens.

Through our programme can able to find out effective dimensions of retaining wall, factor of safety for overturning and sliding and maximum and minimum pressure under the foundation if the following parameters are feed to the programme. Those are the length and material properties of retaining wall, External loads and soil properties. This programme was designed using Rankine's earth pressure theory.

In a design of a retaining wall initially was assigned a value to the base width and while increasing the base with checked the adequacy of the retaining wall with the governing parameters. Through this can achieve suitable and economical dimensions for the retaining wall.

6.0 RECOMMENDATIONS

The following development could be recommended to the programme as extension.

- Developments are to be done to Gabion wall, Reinforced earth wall and sheet pile wall.
- Modifications are to be done to finding the reinforcement while inputting the structural design to a programme.
- Modifications are to be done to finding the suitable dimensions of retaining wall when the existing soil having several layers of different soil properties.
- Development is to be done with consider Coulomb's method of analysis for used to retaining wall design.

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ASSESSMENT OF MINIMUM AND MAXIMUM DENSITY OF A SPHERICAL FRICTIONAL PARTICLE ASSEMBLY USING DISCRETE ELEMENT METHOD (DEM)

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ABSTRACT

Interpretation of tests on real granular media, such as sand, is difficult because the stresses inside the sample cannot be measured and must be estimated from the boundary conditions. A Numerical model allows simulating the mechanical behaviour of cohesionless soils, which is a discontinuous media, using simple shapes of discrete elements. The interaction of the particles is monitored contact by contact and the motion of the particles modeled particle by particle. In this study, the effects of particle size distribution and inter particle friction on compaction characteristics of a spherical particle assembly are investigated using DEM. An assembly is modelled numerically starting from a particle size distribution curve and relevant laboratory tests are simulated to evaluate limit densities of that assembly. Numerical results are then compared with results obtained from relevant laboratory tests.

1. INTRODUCTION

An indication of the state of compaction of a cohesionless soil is obtained by relating its dry density to its maximum and minimum possible values.

In BS 1377: part 4, two methods are described for the determination of maximum density; one for sands and one for gravelly soils. In both tests soil is compacted under water with a vibrating hammer. Likewise, to determine minimum density, two tests are described. One is simple dry shaking test for sands and the other is dropping test for gravelly soils.

A granular medium is composed of distinct particles which displace independently from one another and interact only at contact points. The discrete character of the medium results in a complex behaviour under conditions of loading and unloading. Interpretation of tests on real granular media, such as sand, is difficult because the stresses inside the sample cannot be measured and must be estimated from the boundary conditions. In addition the conventional methods for determination of limit densities are generally costly and require relatively large sample volumes.

Numerical model based on the Discrete Element Method as proposed by Cundall & Strack (1979) allows simulating the mechanical behaviour of cohesionless soils, which is a discontinuous media, using simple shapes of discrete elements. The method is based on the use of an explicit numerical scheme in which the interaction of the particles is monitored contact by contact and the motion of the particles modeled particle by particle with states of equilibrium developing whenever the internal forces balance.

Aim of this study is to investigate the effects of particle size distribution and inter-particle friction on compaction characteristics of a spherical particle assembly using DEM. Objective is to create an assembly numerically starting from a particle size distribution curve and simulating relevant laboratory tests to evaluate limit densities of the assembly.

This study is limited to sand particles of spherical (rounded) nature. The effects of gradation and inter particle friction on the compaction characteristics are investigated. For that, well graded and uniform graded assemblies are analyzed numerically to obtain their limiting densities and the results are compared with results obtained from relevant laboratory tests.

2. PARTICLE FLOW CODE (PFC^{2D})

PFC^{2D} is a software capable of implementing DEM in 2-dimensions. PFC^{2D} is governed by the law of motion acting on an assemblage of particles and the Force-Displacement law acting on a contact between particles.

Simulation of 2D model

In order to set up a model to run a simulation with PFC^{2D}, three fundamental components must be specified:

1. An assembly of particles
2. Contact behavior and material properties
3. Boundary and initial conditions

The geometrical data describe the positions and orientations of the straight rigid boundaries (walls) and the positions and radii of the particles (balls), with respect to a global co-ordinate system. The boundary conditions are of the strain-controlled type. Motion of the wall is specified rather than the forces applied to it. Some specified physical properties are disc radius, density, cohesion, inter-particle friction coefficient, shear stiffness and normal stiffness. A Coulomb-type friction law is incorporated.

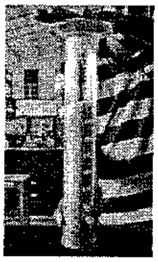
3. METHODOLOGY

3.1 Laboratory determination of minimum density

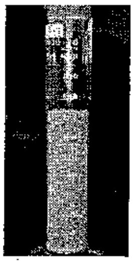
As per test described in BS 1377- Part 4,

* A representative test sample of 1kg was obtained.
* The dry sand was shaken in a 1 L glass measuring cylinder and allowed to fall freely, there by entrapping air and forming a grain structure enclosing the maximum possible volume of voids.

* A cover was fitted to the cylinder and was shaken to loosen the sand and inverted few times.
* The cylinder was turned upside down (fig 3.1), paused until all the sand was at rest, and then it was quickly turned right way up. Then the cylinder was stood on flat surface (fig 3.2) without jarring.



3.1: Inverting



3.2: Reading volume

* Volume (V) at the mean level of the top surface of sand was recorded to the nearest 10 ml. Now minimum dry density of the sand can be calculated by, $\rho_{min}=1000/V$

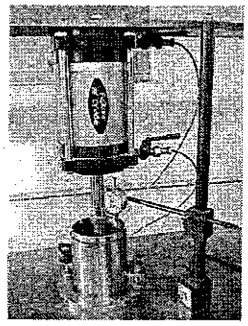
3.2 Laboratory determination of maximum density

Even though a dynamic compaction method is instructed in BS 1377-part 4 to determine the maximum dry density, a static compression method (BF cylinder apparatus was used to apply a static pressure) was used for the determination of maximum density in the laboratory.

* BF cylinder had been calibrated
* Using the calibration chart, the pressure values to be applied to the apparatus in order to achieve the required rod force can be calculated.
* Obtained the weight of the mould and lubricated before filling the soil, to reduce the effect of wall friction. Mould was filled with 1/3 of sand sample and top surface was leveled before applying the rod force.

3.3: Mould was filled with 1/3 of the sample and the top surface was leveled

* Mould was set under the BF cylinder and desired pressure was applied to the top balloon while keeping the pressure of the bottom balloon unchanged at 20 kPa.
* Likewise sample was compacted in 3 layers. At the mean time a dial gauge was set to the top of the sample and monitored to ensure the end of compaction as shown in figure 3.4.

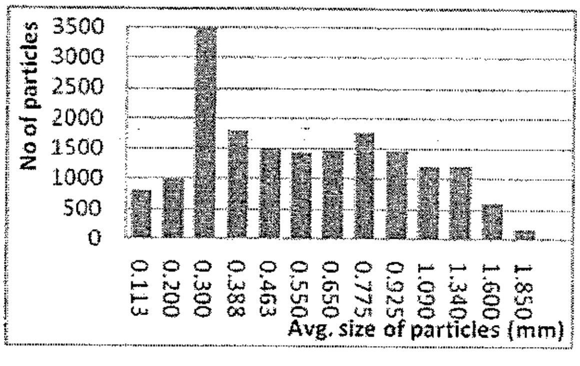


3.4: Soil sample under static pressure

* After end of compaction of the 3 layers, the mean height of the sample and weight of the sample with the mould was measured.
* The same procedure was repeated for the pressure values of 100, 300, 500 and 700 kPa and the corresponded values for density of sample were obtained.

3.3 Determination of minimum density using DEM Numerical simulation

Particle Size Distribution (PSD) curve of the same soil sample can be drawn. Now diameter of particles in a selected class can be considered as the mid value of that class. Then area and mass of one particle in each class can be calculated. Once the total mass and % passing are known, number of particles in each size can be obtained.
In laboratory, a cylinder of size 0.065 m diameter and 0.385 m height was used. A cube of unit thickness (1 m) and having a cross-section of 0.065 m x 0.385 m, is considered as the container for the PFC^{2D} simulation. According to the volume ratio, 1 kg of sample in the cylinder corresponds with a mass of 21.1 kg in the cube.



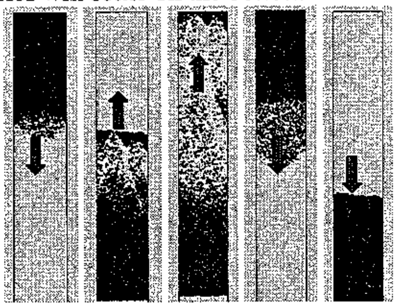
3.5: Distribution of particles for numerical simulation

Obtaining the minimum density using PFC^{2D}

- Create a beaker consisting of boundaries (walls)
- Create the particle assembly inside the beaker (balls)
- Set downward and upward acceleration alternatively to particles
- Display all balls and walls
- Get the height of assembly resting on bottom wall

Simulating particle free fall and beaker inversion

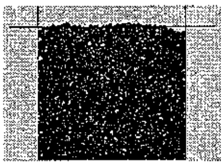
Gravity is set in downward direction to simulate the falling of assembly. Certain number of cycles is given and particles will fall downwards and retain on the bottom wall. In order to simulate the inverting effect as done in laboratory experiment, gravity can be set to act in the opposite direction and particles will move upwards and settle on the top wall. Repeat this twice and finally let them fall downwards (Fig 3.6). Now all the particles will settle on the bottom wall.



3.6: Intermediate plots in PFC^{2D}

Getting minimum density value

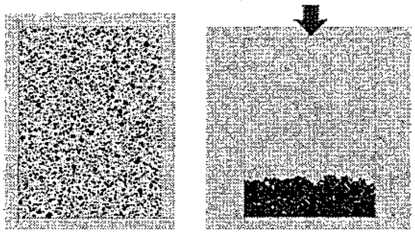
In order to get height, the top wall can be set to move downwards. Some trials have to be carried out to select the best line to represent the top surface. When the wall best fits the top surface of particle assembly which is resting on bottom wall, the density value can be obtained. (Fig 3.7)

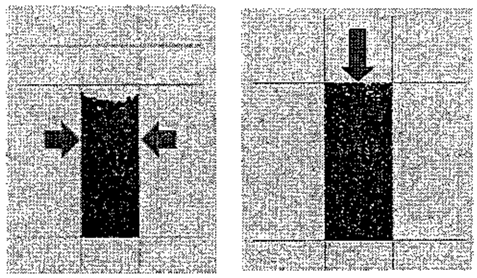


3.7: Getting minimum density value

3.4 Determination of maximum density using DEM

As simulated in minimum test, particle assembly is created within boundaries and let to fall downwards under gravity. Then assembly is compressed by servo controlled walls moving inwards. At the densest state, maximum density is obtained. Intermediate steps are interpreted in fig 3.8





3.8: Intermediate steps in max. density test

In the PSD curves drawn in fig 3.9, Lab1 and Lab2 represent laboratory tests and PSD 1-6 were simulated in numerical tests.

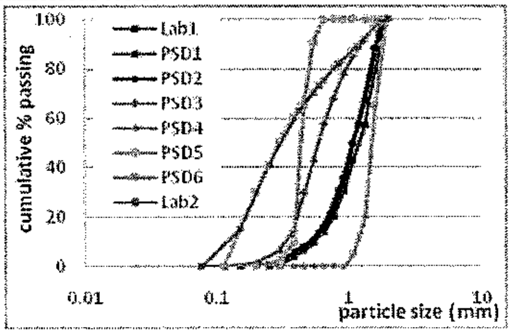


Fig 3.9: PSD curves used for tests

4. RESULTS

4.1 Minimum density (laboratory)

Results of the investigation of the effect of initial sample weight on the minimum density obtained using the test procedure described in BS1377-part4, are tabulated in table 4.1. Sample having distribution as Lab1 shown in figure 4.3 was used in this test.

Table 4.1: Minimum density values from lab. test

4.2 Maximum density (laboratory)

Weight of Sample/(g)	*Volume of sample (ml)*			*Selected volume*	*Density (g/cm^3)*
	Trial 1	*Trial 2*	*Trial 3*		
200	*150*	*160*	*150*	*150*	*1.333*
400	*300*	*300*	*300*	*300*	*1.333*
600	*440*	*440*	*440*	*440*	*1.364*
800	*590*	*590*	*590*	*590*	*1.356*
1000	*740*	*730*	*740*	*740*	*1.351*

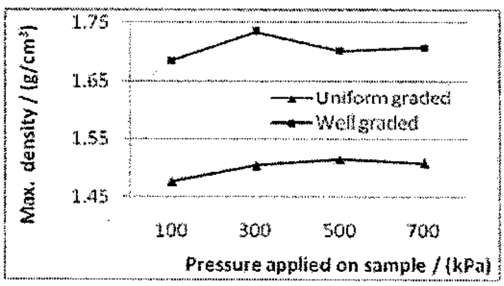


Fig 4.1: Lab results for maximum density

4.3 Minimum density (numerical)

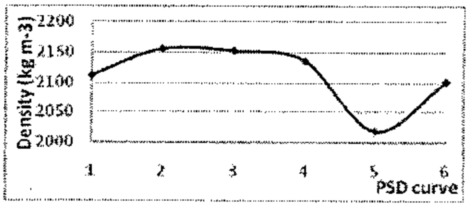


Fig 4.2: Variation of minimum density with PSD

4.4 Maximum density (numerical)

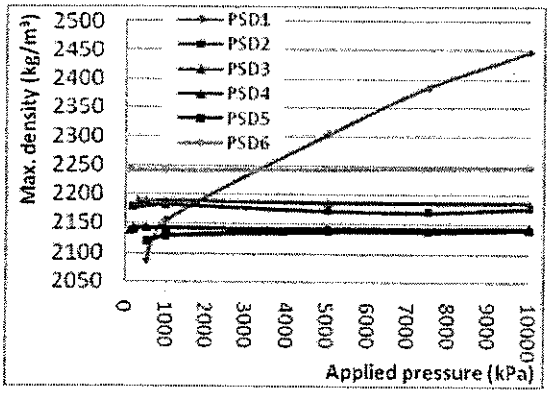


Fig 4.3: Variation of max. density with pressure

4.5 Effect of friction on maximum density

For PSD3, numerical test was done for maximum density, with different inter particle friction values at 10000 kPa pressure.

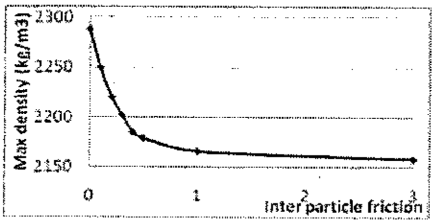


Fig 4.4: Variation of maximum density with friction

Discussion

In the laboratory test for minimum density, obtained values vary between 1333 and 1364 kg/m^3 for different weights of sample as tabulated in table 4.1. Variation of density with weight is not significant.

Considering laboratory test result for maximum density (Fig 4.3), variations in maximum density value for uniform graded sample and well graded sample are not significant. However, maximum density value for well graded sample is nearly 1700 kg/m^3, which is higher than of uniform graded sample for which maximum density is nearly 1500 kg/m^3.

Variation of numerical minimum density value for various PSDs is shown in fig 4.2. Among the six values for minimum densities, PSD5 gives the lowest minimum density value. Maximum density varies with applied pressure for various PSDs as shown in fig 4.3. No significant variation is observed except for PSD1. Maximum density for PSD1 increases with increasing pressure.

5. CONCLUSIONS & RECOMMENDATIONS

5.1 Conclusions

Values for minimum density and maximum density obtained from laboratory tests are lower than numerical test values. For both laboratory and numerical tests, well graded samples have higher values of maximum density.

To evaluate minimum density in laboratory, sample having distribution as lab1 shown in fig 4.3 was used. PSD2 is the numerical simulation for this. From results, laboratory minimum density is 63% of numerical value. PSD2 used in numerical test is the same distribution as lab1 (uniform graded) used in laboratory test. From the results, maximum density from lab1 is of 68% of the value of PSD2 for 200 kPa. Similarly, PSD6 and lab2 (well graded) are same. Here, maximum density from lab2 is of 75% of the value of PSD6 for 100 kPa.

Above verification between laboratory results and numerical results is primarily qualitative. DEM simulation was done taking all the particles as circular tubes. However assembly used for laboratory test contains some angular and elongated particles also. Variation of maximum density with inter particle friction is plotted in fig 4.4. In numerical simulation, inter particle friction was used as 0.4. Therefore if actual value of inter particle friction is different, density values also will deviate much.

If PSD curve only is available, minimum and maximum density values can be evaluated using PFC^{2D}. In addition, soil sample can be improved by changing the number of particles of selected sizes which changes the distribution. Finally it can be concluded that, Discrete Element Method is a useful tool for investigating effects of particle size distribution on compaction characteristics.

Recommendations

In this study, effects of particle size distribution on compaction characteristics, was analyzed. With some more results, any correlation between numerical and laboratory results can be derived. Furthermore the effects of inter particle friction and damping can be analyzed for the assembly using PFC^{2D}. Particles were taken as spherical for numerical simulation in this study. If the particles are modelled as of different shapes (using clump logics available in PFC^{2D}, elongated particles can be modeled), it may approach the actual assembly.

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Faster and Bouncier Cricket Pitches using Locally Available Clays

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Abstract

Reasons for the lack of 'pace & bounce' in Sri Lankan pitches has been a topic of debate for a long time. In order to address this problem from an engineering perspective, soil samples from Test venue in Colombo were gathered, analyzed and then compared to available data from other countries. Local pitches were found to have high silt content, low clay content and low plasticity due to a difference in clay mineralogy. Small scale model pitches were built and then tested to find the effect of adding commercially available clays on the pitch characteristics.

1. Introduction

Sri Lanka (as well as most of the Indian Sub-continent) is home to "slow and low" pitches and even though it has been attempted, the creation of fast & bouncy pitches in Sri Lanka has eluded us. This paper deals with the problem of creating a fast and bouncy pitch by investigating how some of the fundamental physical characteristics of a cricket pitch varies with the soil used to make the pitch and attempting to modify the playing character of the pitches by altering the composition of the soil.

2. Methodology

2.1 *Soil data from foreign pitches*

The first step in this study was to gather data on the make up of soils used in foreign pitches. Soil data from England and Australia was obtained through Sri Lanka Cricket (SLC) and it was decided to use Australian pitches as the frame of reference based on the following reasons.

- Australian pitches are considered to be among the fastest and bounciest pitches in the world
- The climate in Australia is comparable to that in Sri Lanka so the effect of climatic differences on pitch preparation and behaviour was reduced.

2.2 Soil data from local pitches

Soil samples were extracted from pitches at the SSC and NCC grounds and then analyzed to find their grading and make up. Soil samples which were directly from the suppliers were also tested and analyzed. Most of the soil samples were tested at the Industrial Technology Institute (ITI) using laser particle analyzing and the results were confirmed by hydrometer particle size analysis tests carried out at the University of Moratuwa.

The soil data from local pitches were then compared to the soil data from Australian pitches and the major differences were identified. The local soil was then modified by the introduction of local clay in order to make it more similar to the Australian soils.

2.3 Construction of pitch models

Once the necessary modifications to the local soil were made, it had to be verified whether the expected improvements in playing character would be seen in the pitches. The construction of an actual sized pitch was neither economically viable nor provided the necessary flexibility to test different soil mixes. Thus, small pitch models were constructed in 30cm x 25cm x 25cm boxes and then tested. The coefficient of restitution and the coefficient of friciton of these models were measured using two simple tests. These two properties were then used in a mathematical model which could predict the playing character.

2.4 Newtonian mathematical model

Daish (1972) first proposed the applicability of a Newtonian impact model for cricket and his work was later developed by Carre et al. (2000). This Newtonian impact model allows for no deformation of either the surface or the ball and calculates the ball rebound dynamics using a combination of friction and restitution measurements.

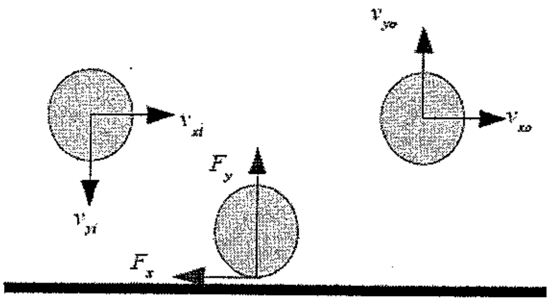

Figure 1: Impact between ball and surface

F – force v – velocity
μ – coefficient of friction
e – coefficient of restitution

In figure 1, subscripts 'x' and 'y' denote the direction while 'i' and 'o' denote incoming and outgoing velocities.

Governing equations for the above model,

$$\uparrow \quad \int F_y dt = m(v_{yo} + v_{yi}) \quad \text{-----------} \quad (1)$$

$$\rightarrow - \int F_x dt = m(v_{xo} - v_{xi}) \quad \text{-----------} \quad (2)$$

$$F_x = \mu F_y \quad \text{-----------} \quad (3)$$

$$v_{yo} = e v_{yi} \quad \text{-----------} \quad (4)$$

From the above equations,

$$v_{xo} = v_{xi} - \mu v_{yi}(1+e) \quad \text{----------} (5)$$

From equations 4 and 5 the outgoing velocity components can be found if 'μ' and 'e' are known. If v_{xo} and v_{yo} are known, the complete path of the ball can be defined and thus the behaviour of the ball can be predicted. (James et al. 2005)

2.5 Analysis and comparison of soils

Several probable reasons for the lack of pace and bounce in local pitches were identified by comparison of local soil data to those from Australian pitches. The most important among these were:

- Silt content was too high
- Clay content was too low
- Plasticity of soil was low due to different clay mineralogy

Atterberg limits tests were conducted on the soil samples obtained from the SSC and NCC pitches. The plasticity indices of these soils were found to be in the range of 12 to 16. This together with Skempton's "Activity" index implied that the predominant clay mineral in local soils was kaolinite as opposed to smectite in Australia.

Soils with smectite clays bind together stronger and produce harder surfaces at low moisture levels. Kaolinites on the other hand are much weaker and brittle.

Local pitches tend to crumble and dust up faster than Australian pitches and have much lower bounce. The above factors fit in well with this sort of behaviour.

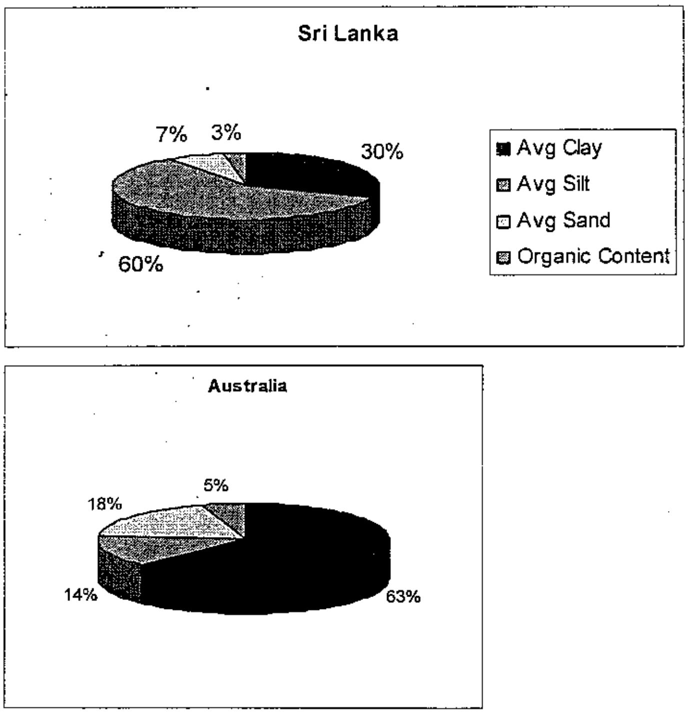

Figure 2: Soil components of pitch soil *(NZSTI Guide to cricket pitch preparation)*

2.6 Modification of soil

Once the probable shortcomings of local soils currently been used were identified, there were two alternatives to solve this problem.

1. Modify available soil
2. Find new soil with appropriate characteristics

The better alternative for constructing actual pitches was to find sites from where highly plastic clays could be mined but, to test whether the hypothesis developed by the authors was correct it was decided to modify the available soils to attain the necessary characteristics and then build small scale models of pitches to be tested. So, soil obtained from SLC was modified by adding bentonite which is a commercially available clay belonging to the smectite family. Enough bentonite was added so that the total clay content would increase to 50%, thereby improving the plasticity of the soil and reducing the silt percentage. The authors had to be careful not to adversely affect the ability of the soil to facilitate the growth of grass by adding too much bentonite, 50% clay was chosen because it was an intermediate value which at the same time was expected to be high enough to indicate whether the pitch characteristics had improved.

2.7 Tests

Once the models were constructed, two types of tests done on them at different moisture contents to evaluate their characteristics.

1. Bounce test – A cricket ball was dropped on the model from a height of 4m (to simulate a fast bowler) and the rebound height was measured. The coefficient of restitution was calculated from the results.
2. Friction test – A heavy metal disc was placed on the model and the model was tilted up from one end until the disc slid. The angle of tilt at which the disc began to slide was used to calculate the coefficient of friction.

3.0 Results and Discussion

The pitch models constructed using this modified soil showed improved bounce as compared to a control model constructed using the original soil. Figure 3 shows how the addition of bentonite to the original soil had modified the grading of the soil. Particles smaller than 0.025mm representing clay had increased to 45% from 25% while the silt content had decreased to 45% from 65%.

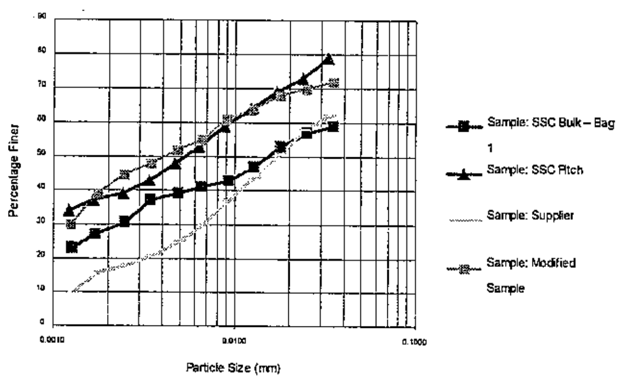

Figure 3: Soil grading

Figure 4 shows the results of the bounce test for the control pitch model and the modified pitch model. The modified model shows higher 'e' values at lower moisture levels implying that such a soil will be well suited to produce bouncier pitches in a warm climate like that in Sri Lanka.

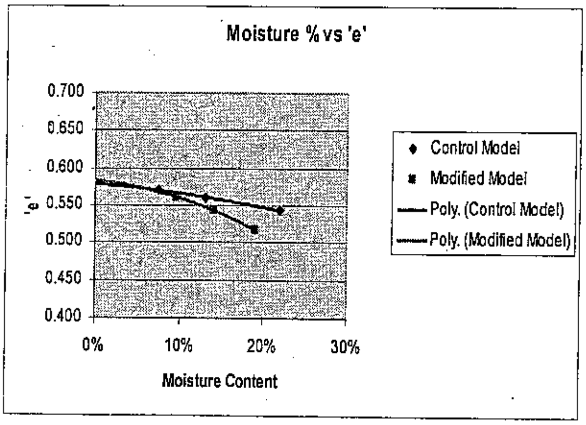

Figure 4: Moisture Content vs Coefficient of Restitution for the models

It can be argued that the improvements are too small to be of consequence as the *'e'* values in the two soils are virtually identical at a moisture level of 5% which the approximate moisture level of soil in pitch ready to be played on. However, the authors are confident more substantial improvements can be made if the clay content is further increased. This can be done as Australian pitches have clay contents of upto 80% (NZSTI Guide to Cricket Pitch Preparation).

The modified pitch model also showed better resistance to crumbling and powdering during testing which means the bentonite is helping to binder the soil together and thus would produce longer lasting pitches.

The results from the friction test are not included in this paper as they were found to vary with the extent of grass cover on the surface.

4.0 Conclusion

The above data shows that cricket pitch behavior depends primarily on the type of soil and that by using soils with the stipulated properties, faster and bouncier pitches can be produced. Furthermore, it also gives substantial evidence that the creation of such pitches using locally available soils is certainly in the realm of possibility. The results obtained from the use of the soil modification method have been encouraging enough to justify construction of full scale test pitches prepared with naturally occurring highly plastic local clays as the next step to this study. The author are planning to investigate the effect of grass and the method of pitch preparation on pitch playing characteristics in addition to the soil type in future studies.

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AN ARTIFICIAL NEURAL NETWORK MODEL TO PREDICT THE RAIN INDUCED CUT SLOPE FAILURES

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ABSTRACT

Slope failure is very common phenomenon in hilly regions. It is hazardous because of the accompanying progressive movement of the slope-forming material. In order to minimize the cut slope failure effects, slope stability analysis model was developed by using neural network techniques.

The influence of different factors on the occurrence of slope failure is not equal and therefore each factor must be weighted. Initially, a simple model was developed by using linear relationships of each causative factors and weighted factors. Due to shortcomings of that simple model, a model for slope failure prediction was formed by using neural network theories which can handle multivariable with nonlinear relationships.

INTRODUCTION

Cutting into existing ground disturbs the mechanics of the surrounding area, which can be resulted in masses of rock, earth and debris to flow down a slope during periods of intense rainfall. With the growth of population, there is an increasing demand for land that is suitable for housing, necessary infrastructure and other services such as health care and education. Therefore, in a country where 20% of the total land area is mountainous, utilization of land on hill slopes is unavoidable and it is extremely important to predict the cut slope failure, in order to ensure the safety of the people and to select the suitable land for development.

In this paper a cost effective prediction model of cut slope failure using Artificial Neural Network is mainly discussed.

OBJECTIVES

1. To identify the factors and its magnitudes for the occurrence of slope failures.
2. To develop a new methodology for prediction of slope failures based on the analysis carried under 1 above.
3. To analysis the stability of selected slopes based on their physical properties and to compare the results with the Neural Network model.

SITE INVESTIGATION

In this research 70 sites in Southern province of Sri Lanka were investigated. The study area, near the city of Deniyaya, is situated in the wet low and wet medium climatic zones and South-Western geological group in the Sri Lanka. It is located in mountainous areas adjoining highways. The majority of the land lies below 600 meters, and a quarter of the land lie between 300 and 600 meters. This study site was composed of charnockite, charnockitic gneisses, weathered rocks and formed soils.

Visual observations such as surface slope angle, cutting height, angle of cut, type of rock, mountainous location, soil profile, dip angle, vegetation cover, mountainous position and height to water table were recorded in each sites. Due to time limitation and complexity, some factors could not be considered for this research such as joint intensity, fracture plane, mineralogy, etc.

DEVELOP THE PREDICTION SYSTEM

In general, one factor does not normally cause a slope failure but many factors intertwined and contributed to the cause. The influence of different factors on the occurrence of slope processes is not equal. Therefore each factor must be weighted as represent their individual contribution for slope failure. Initially weights were assigned and formulated a function for slope stability. For the instability of slope,

$S = f(H, L, R, \beta, \gamma, D, \mu, h, q, x, V, \alpha, W)$

Where;

L: width of slope
H: embankment slope height
R: Rainfall condition
β: embankment slope gradient
γ: density of soil
D: Depth
μ: pore water pressure
h: Ground water level
W: Topographical condition
V: Vegetation cover
α: Dip angle
x: mountainous position

Since this study is mainly focused on geologically and climatically similar area, some of the factors such as rainfall and rock type. After reveling of literature we deduced our problem to following parametrs.

$$S = f(H, \beta, h, \alpha, x, \theta, W, L, V)$$

Then a simple multivariable analysis was carried out and found a mathematical relationship among above weighted parameters considering their linear relationship. Non failure and failure sites cannot be clearly identified by using this simple model.

A neural network is a non-linear system consisting of a large number of highly interconnected processing units, nodes or artificial neurons. Eight causative factors of each site were considered as input variables. Each input signal is multiplied by the associated weight value *wi* and summed at a neuron. The result is put through an activation function to generate a level of activity for the neuron. This activity is the output of the neuron. When the weight value at each link and the connection pattern are determined, the neural network is trained. This process is accomplished by learning from the training set and by applying for certain learning rule. 47 sites were used for training purpose. The trained network can be used to generalize for those inputs that are not including in the training set. A flow diagram of this model is shown in Figure 1.

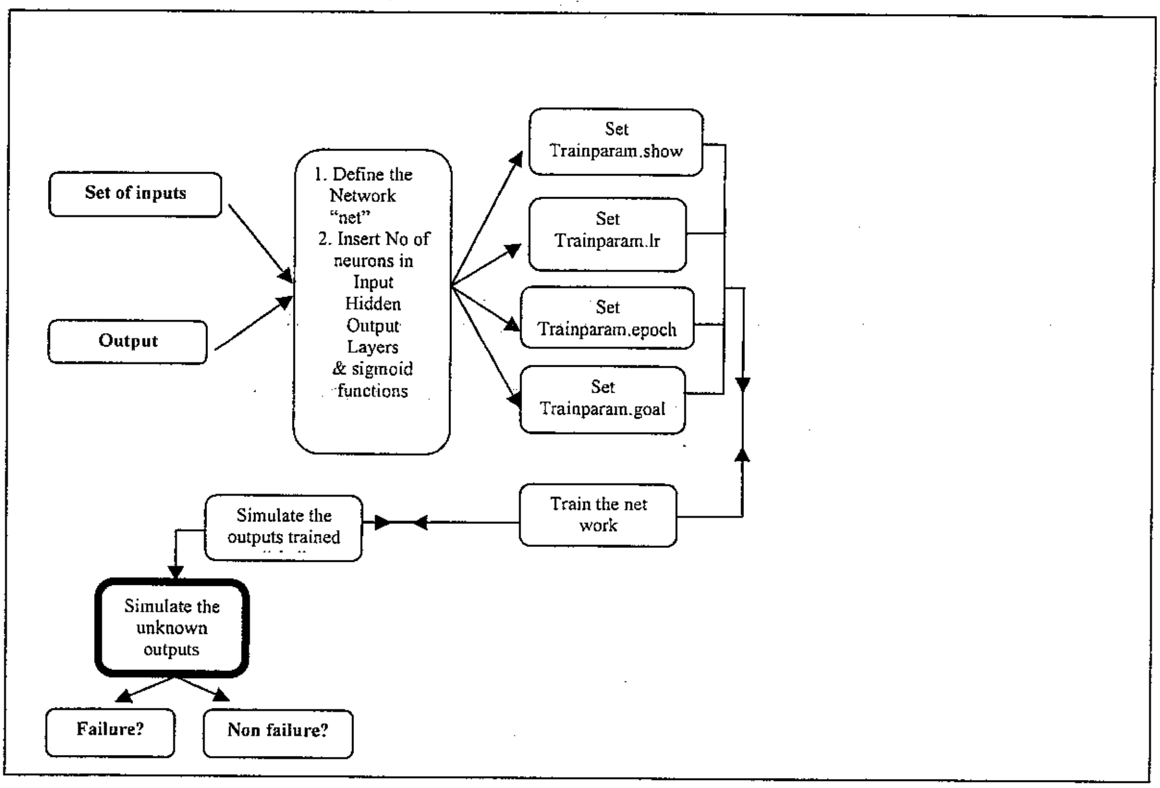

Figure 1: Flow chart of the Artificial Neural Network model

RESULTS AND DISCUSSION

To assess stability of cut slopes, an Artificial Neural Network model was developed and validated successfully. Figure 2 shows some of the results of the developed model after several trails. It has been shown that the failure slopes and non-failure slopes were assessed successfully by the well trained program. The trained network with the slightly change weights can be used for the prediction of new slopes. Figure 3 shows the developed interface of the program.

CONCLUSIONS AND RECOMMENDATIONS

- A neural network model was developed to evaluate the stability of cut slopes in Galle-Deniyaya highway. Results revealed that this model can be successfully used to predict the slopes in the region.
- This model can be extended to predict the slopes in other regions and to further extend to incorporate remedial actions for the unstable slopes.

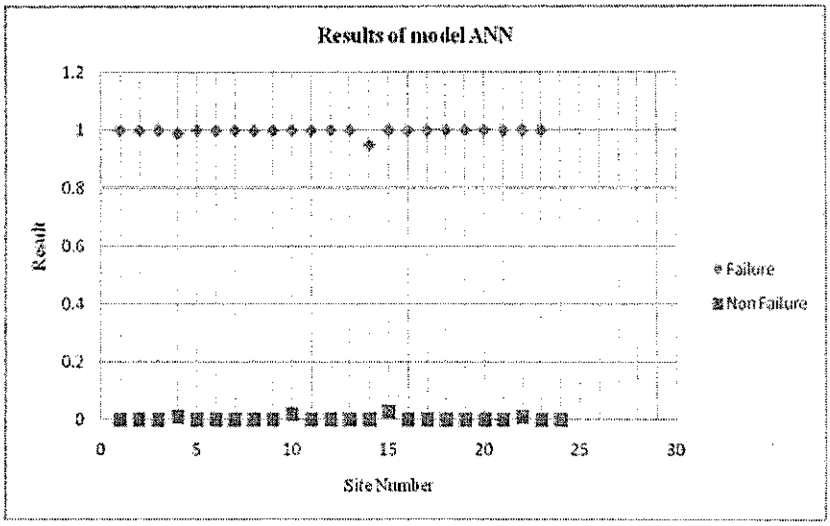

Figure 2: Results of the Program

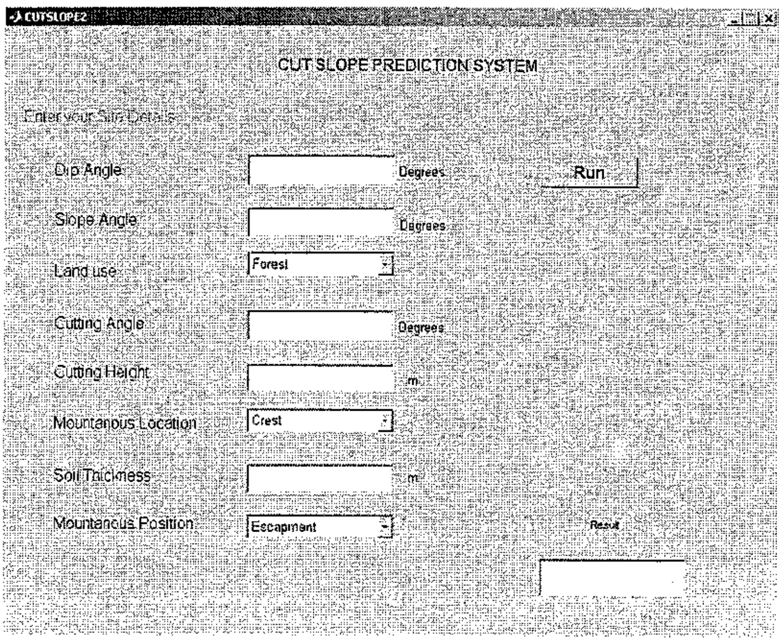

Figure 3 Interface for ANN model

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