GEOTECHNICAL ENGINEERING PROJECT DAY 2015

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

December 09, 2015
At IESL Auditorium

Organised by the
SRI LANKAN GEOTECHNICAL SOCIETY

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GEOTECHNICAL ENGINEERING
PROJECT DAY – 2015

SRI LANKAN GEOTECHNICAL SOCIETY

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Message from the President - SLGS

From its inception, Sri Lankan Geotechnical Society has provided a forum for disseminating new knowledge in the field of geotechnical engineering and promoting research. A very successful international conference ICGE-Colombo-2015 was held in August 2015. There were five keynote lectures by very eminent personnel in the field of Geotechnical Engineering. Further, 135 technical papers were presented on 12 different themes in three parallel sessions. Although the conference was a success in many aspects such as participation of local and overseas academics and practitioners one disappointing aspects was that only 23 papers originated from Sri Lanka, despite the canvassing for papers done over more than 3 years.

The Project Day competition is an annual event held among Sri Lankan undergraduates doing projects in the field of geotechnical engineering. It commenced in year 2000 with the objective of encouraging them to do good research and publish. Participants are expected to present their findings in a concise four paged paper and make a 15 minute oral presentation. The best paper and the second paper will receive cash awards and certificates.

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

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

I also wish to convey my sincere gratitude to the panel of evaluators; Emeritus Professor B. L. Tennekoon, Mr. K. S. Senanayake and Dr. H. G. P. A. Ratnaweera.

Prof. Athula Kulathilaka
President - SLGS

CONTENTS

Shear Strength Characteristics of Unsaturated Residual Soils

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ABSTRACT : The tropical climate conditions cause the residual soils near ground water level to be in an unsaturated state generally. Matric suction enhance the shear strength of the soil. In tropical countries like Sri Lanka where the soil formation is mainly residual, this added shear strength is no longer reliable due to frequent rainfall periods. With rainfall infiltration, the matric suction reduces and the shear strength diminishes making the slopes vulnerable. Therefore, it is necessary to study how the moisture content of soil is affected after the rainfall and how it can be related to shear strength. As an initial step, the variation of apparent cohesion with degree of saturation was investigated in the present study for residual soil obtained from the site of the failed slope at Southern Transport Development Project in Welipenna. Direct shear tests was preferred in the study over triaxial test because of the shorter time taken for the pore water pressure to reach equilibrium.

1 INTRODUCTION

Sri Lanka, which has a climate condition of tropical wet and dry, has residual soils that developed mainly from metamorphic rocks. Ground water table is generally low in these conditions. The soils are therefore generally unsaturated and possess negative pore water pressures. Climatic changes greatly affect the soil properties, negative pore water pressure and shear strength of unsaturated soils.

The matric suction plays a vital role in enhancing the shear strength of the soil. In tropical countries like Sri Lanka where the soil formation is mainly residual, this added shear strength is no longer reliable due to frequent rainfall periods. With rainfall infiltration, the matric suction reduces and the shear strength diminishes making the slopes vulnerable. Therefore, it is necessary to study how the moisture content of soil is affected after rainfall and how it can be related to shear strength. As an initial step, the variation of apparent cohesion with degree of saturation has been investigated in the present study for residual soil obtained from failed slope of Southern Transport Development Project (STDP) in Welipenna. This study is based on direct shear tests performed on undisturbed residual soil specimens in unsaturated state.

The results of this research will be useful for future studies on slope stability analysis. In the design of rainwater drainage systems, the results can be used to determine the apparent cohesion which must be maintained in the soil with similar history of formation as that of Welipenna. By maintaining sufficient apparent cohesion in the soil, the slope stability shall be developed internally with lower scale or absence of stabilization techniques such as retaining wall and anchors.

2 OBJECTIVES

The aim of this research is to determine the variation of unsaturated shear strength parameters of residual soil obtained from failed slope of STDP in Welipenna. To accomplish this aim, soil classification, determination of shearing rate required for 90% consolidation, determination of consolidated drained shear strength parameters and derivation of the relationship between the apparent cohesion and degree of saturation of soil were identified as main objectives.

3 SHEAR STRENGTH OF UNSATURATED SOILS

The mechanical behaviour of unsaturated soil depends on two independent stress state variables, namely the net normal stress, $(\sigma - u_a)$, and matric suction, $(u_a - u_w)$. Fredlund et al. (1993) proposed the equation 1 for interpreting the shear strength of unsaturated soils, which have planar failure surface, in terms of the two stress state variables.

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan \phi' + (u_a - u_w)_f \tan \phi_b \qquad (1)$$

where, τ_{ff} is the shear stress on the failure plane at failure, c' is the effective cohesion intercept, $(\sigma_f - u_a)_f$ is the net normal stress at failure, u_{af} is the pore air pressure at failure, ϕ' is the angle of internal friction associated with net normal stress state variable, $(u_a - u_w)_f$ is the matric suction on failure plane at failure, and ϕ_b is the angle indicating the rate of increase in shear strength relative to the matric suction $(u_a - u_w)_f$. The extended Mohr-Coulomb envelope defines the shear strength of an unsaturated soil. It can be either planar or curved. Unlike a curved failure envelope, planar failure envelope is assumed to have a constant slope angle, ϕ_b.

The soil-water characteristic curve which is one of the important unsaturated soil properties of residual soils, defines the relationship between the water content and matric suction in the soil. The general trend of soil- water characteristic curve is that the water content decreases as soil matric suction increases.

4 METHODOLOGY

The methodology followed in this research to accomplish the objectives is as listed below.

- Background study and clarifications about the research
- Literature review
- Acquisition of sample and preparation of sample for testing
- Performance of laboratory tests
- Analysis of results

5 EXPERIMENTAL WORKS

A series of experiments required to find out essential parameters were performed following the guidelines given on BS 1377.

Soil classification was done according to unified classification system. Basic tests such as particle size distribution and consistency limit tests were performed for the soil classification. It was required to know the time needed for 90 % consolidation of soil in order to come up with an appropriate shearing rate for sample, which ensures complete dissipation of excess pore water pressure. Consolidation test was conducted for this purpose prior to the commencement of primary tests.

In order to find out the consolidated drained shear strength parameters, series of direct shear tests were conducted on undisturbed specimens by varying the degree of saturation.

5.1 *Soil classification tests*

Residual soils possess variability in its nature. Hence, it was expected that the Welipenna soil can have range of soil particles. In particle size distribution test, wet sieving was done for coarser particles as soil was found to be cohesive and hydrometer test was conducted for finer particles as their percentage in the soil was greater than 10%. Consistency limit tests were carried out to identify the dominating type of finer particle, either silt or clay. Fig. 1 shows the particle size distribution in the soil.

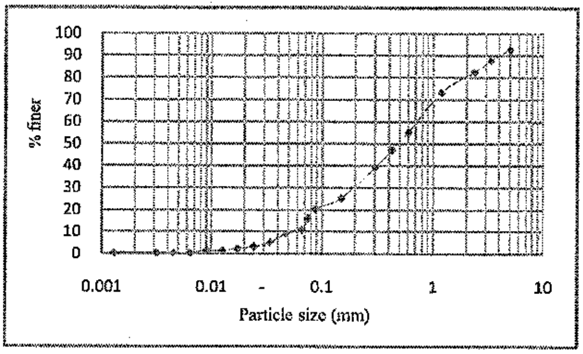

Fig. 1 Particle size distribution

The liquid limit and plastic limit of soil were 39.6% and 35.17% respectively. The plasticity index of 4.47% indicated that the soil is slightly plastic. With that, the soil was classified as SM (silty sand) according to the unified classification system.

6 DIRECT SHEAR TESTS

The direct shear test was chosen over other tests for this study because of the shorter drainage path of soil sample and thus less time taken to dissipate the excess pore water pressure. Test was carried out on undisturbed residual soil specimens in order to determine the shear strength parameters of soil in its natural state. The purpose of the direct shear tests was to find out the variation of drained shear strength parameters at different degree of saturation. The degree of saturation of test specimens was changed by adding different amount of water to natural sample. Degrees of saturation achieved were 30%, 45%, 63% and 78%. For each degree of saturation, specimens were tested under net normal stresses of 50 kPa, 100 kPa, 150 kPa and 200 kPa with a shearing rate of 0.05 mm/min. The details of the testing program for Welipenna soil are presented in Table 1.

6.1 Curve fitting

Shear stress versus normal stress relationship of soil was established mathematically. From that, the friction slope of unsaturated soil specimen associated with net normal stress variable was found to vary between 39° and 45°. The slope, ϕ' is constant for specific soil. Hence, manual curve fitting was performed for the test results by varying the slope angle between 39° and 40°. The analysis expressed the friction angle of soil as 39.3°.

In manual curve fitting, consideration was given to the variables such as dry density, moisture content and observed sample variability such as presence of large stones and local shear failure. Because the dry density of specimens showed variation, insitu density of samples was taken to be 1250 kgm^{-3} which is the average density of samples. The actual shear strength and the shear strength of soil predicted from manual curve fitting are presented on Table 2. Fig.2 shows the variation of actual and predicted shear strength. The summary of curve fitting is presented in Table 3.

Table 1. Testing program

Test number	S_r (%)	w (%)	γ_d (kg/m^3)
1	32.3	17.89	1065.0
	30.8	15.49	1125.6
	26.7	14.01	1098.9
2	43.4	21.82	1165.5
	41.7	16.78	1306.7
	52.3	19.67	1206.9
3	63.1	27.63	1216.1
	64.1	30.58	1160.3
	63.0	18.80	1463.5
	60.5	19.75	1406.8
4	77.9	34.18	1214.1
	79.0	30.91	1288.9
	77.7	27.70	1343.9
	78.1	38.83	1133.7

Table 2. Actual shear strength and predicted shear strength

S_r	Normal stress (kNm^{-2})	Actual shear strength (kNm^{-2})	Predicted shear strength (kNm^{-2})
30%	50	83.13	109.82
	150	159.75	190.07
	200	232.29	230.19
45%	50	82.36	84.62
	150	214..81	166.47
	200	192.15	207.39
63%	50	94.84	92.82
	100	155.94	130.94
	150	186.31	169.07
	200	222.42	207.19
78%	50	66.08	79.62
	100	182.00	120.54
	150	180.52	161.47
	200	241.80	202.39

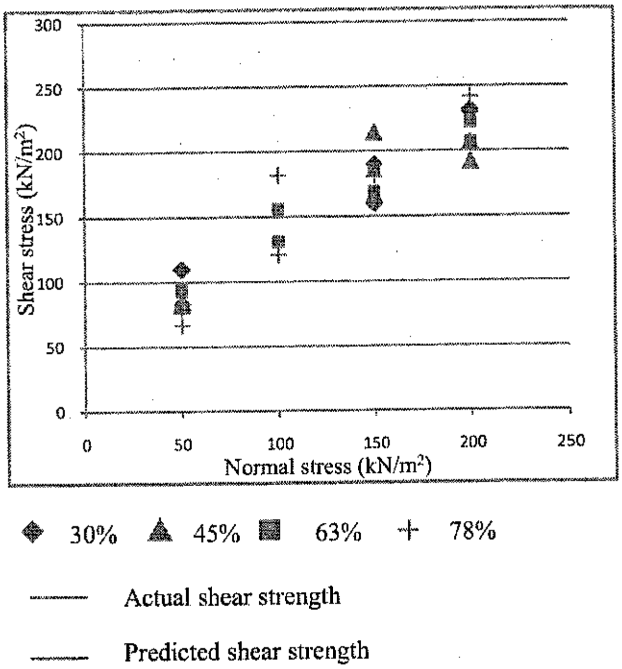

Fig. 2 Actual shear strength and predicted shear strength

The summary of curve fitting is presented on Table 3.

Table 3. Apparent cohesion of soil

Degree of saturation (%)	Apparent cohesion (kPa)
30	69.69
45	43.69
63	54.69
78	38.69

7 DISCUSSION

The results of direct shear test conducted on undisturbed residual soil samples obtained from Welipenna site have been analyzed according to shear strength theory of unsaturated soil (Fredlund and Rahardjo,1993). The results indicate that the soil has a friction angle of 39.3°. The basic tests such as particle size distribution test and consistency limit test performed on the soil confirms such higher value by indicating that the soil is well graded and considerable percentage of finer particles present in the soil making a bond with coarser particles which eventually makes the soil slightly plastic.

The apparent cohesion of the soil can be expressed as,

$$c_a' = c' + (u_a - u_w) \tan \phi_b \qquad (2)$$

The shear strength of the tested soil can be expressed as,

$$\tau_f = c_a' + (\sigma_f - u_a)_f \tan 39.3° \qquad (3)$$

The variation of apparent cohesion c_a' with the degree of saturation is presented in fig. 3. The decreasing trend of apparent cohesion with increasing degree of saturation can be clearly seen. But with the limited data, it is not feasible to obtain an accurate curve fitting.

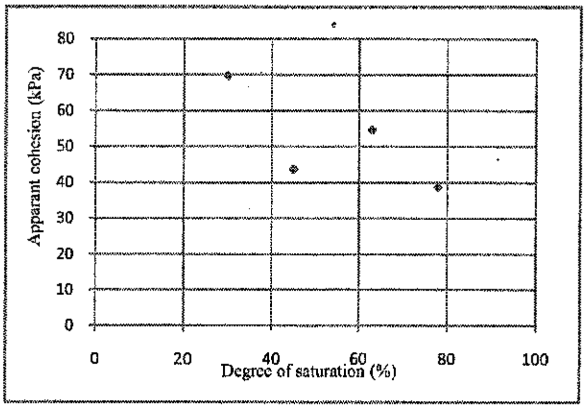

Fig. 3 Apparent cohesion versus degree of saturation

With the increasing degree of saturation, the matric suction, $(u_a - u_w)$, will decrease and approach zero at 100% saturation. There were not enough samples to conduct direct shear tests at 100% saturation. Experiments to develop the soil-water characteristic curve are currently underway but could not be completed.

If the matric suction, $(u_a - u_w)$, is measured during the tests, the ϕ^b value could be obtained.

8 CONCLUSIONS

The aim of this research is to derive the relationship between the apparent cohesion and degree of saturation of residual soil obtained from Welipenna site. A direct shear test was performed on undisturbed residual soil samples under net normal stress ranging from 50 kPa to 200 kPa by varying the degree of saturation of soil specimens. The direct test was chosen over other tests because of shorter drainage path of sample and thus lesser time taken for the pore water pressure to reach equilibrium. The test results were analyzed according to unsaturated shear strength theory proposed by Fredlund and Rahardjo (1993). These results convey that the soil has a friction angle of 39.3° associated with net normal stress axis. Also, the apparent cohesion of tested soil decreases with increasing degree of saturation.

The limitations of this study are,

(i) Matric suction could not be measured during the tests.

(ii) Soil- water characteristic curve could not be obtained for the tested soil.

9 RECOMMENDATIONS

The scope of this research is limited to derivation of relationship between the apparent cohesion and degree of saturation of soil. From a practical point of view, monitoring the matric suction is easier than monitoring the degree of saturation of residual soil. In order to produce the relationship between shear strength and matric suction, soil- water characteristic curve for particular soil must be developed. It can be done either by taking field measurements of matric suction using the tensiometer or performing laboratory tests on pressure plate apparatus.

Soil obtained from shallower depth (0.5 m below the ground level) was investigated for its shear strength in the present study. In actual situation, the unsaturated soil properties vary with depth as degree of weathering changes which will ultimately affect the shear strength of soil. Hence, the study must be extended to assess the strength parameters of soil at various depths.

Natural heterogeneity of residual soils raised uncertainty about the similarity of specimens tested in this study. Therefore, repeatability of tests should be done in future to verify the test results.

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A Study of Settlement Behaviour of Raft Foundation with Vertical Flaps

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ABSTRACT: Shallow foundation in soft soils faces many problems due to low bearing capacity and excessive settlement. Although deep foundations and soil improvements can be used as alternatives, they are often uneconomical with high time and the cost involved. This has led researchers to consider innovative shallow foundation systems for construction on soft soils. "Cakar ayam foundation" and "akar foundation" which are popularly used in Malaysia and Indonesia are two such foundations types. In the current study two vertical flaps attached to the underneath of a raft were used as a modified foundation system to investigate whether it is capable of reducing the settlement when built on soft soil. The physical models of the modified foundation were built by varying the flap length and then the observed model behaviour was analysed using finite element software. The results obtained from physical modelling and finite element analyses showed that the flaps can be used to reduce the raft settlement on soft soil. The raft settlement reduces with the increasing flap length.

1 INTRODUCTION

Today, shallow foundation construction in soft soil faces many problems due to low bearing capacity of soft soil that results in excessive settlement. Deep foundation is a solution to this problem, but considering the costs and the time involved, the approach is proven to be uneconomical. This has led many researchers to design innovative shallow foundation systems for construction on soft soils, with adequate bearing capacity while minimizing the settlement.

The aim of the study was to perform analytical and experimental study on the use of flexible flaps in improving the performance of raft foundations. In this study, it was expected to study settlement behaviour of a raft foundation to which two flexible flaps are attached underneath, using physical testing and finite element analysis. The study was limited to identifying only the influence of the flap length and spacing between the two flaps, to the behaviour of the foundation, while keeping all the other parameters such as raft dimensions, connectivity between flaps and the raft, fixed.

2 LITRATURE REVIEW

Countries like Malaysia have extensive deposits of peat and organic soils, which makes development on such areas a challenging task for civil engineers. "Cakar Ayam foundation" and "Akar foundation"are two shallow foundation systems for construction on soft soils, designed through the researches.

Dr Ir. Sedyatmo (1961) from Indonesia, proposed "Cakar Ayam" or "Chicken Feet" foundation system, that consisted of a reinforced concrete slab resting on a number of reinforced concrete pipes. The passive soil pressure creates a stiff condition of slab-pipe system, enabling the thin concrete slab to float on the supporting soil with the concrete pipes kept vertical. The "Akar Foundation", literally translates as "Root Foundation, is a lightweight platform supported by a group of pipes. This foundation collectively exerts a strong grip on the soft soils and spread the imposed structural load evenly into the subsoil, avoiding excessive and non-uniform settlements. Research conducted has shown the effectiveness of both foundations depending on several parameters such as compatibility of the pipe spacing, individual pipe lengths, raft dimensions and the load applied.

3 MATERIALS AND METHODS

3.1 *Sample preparation*

First task of the project was to identify a location with soft soil with suitable soil parameters and collect samples of that soil for physical model testing. For this task, four locations in Panideniya town, Kandy were identified. Vane shear tests were then carried out in those four locations to find the in-situ shear strength of the soil at depths 0.5m and 1m from the ground level. The location with the least shear strength was identified as the most suitable location and soil samples were obtained at a depth of 0.5m from the surface. The collected soil samples were protected and preserved with wax at the laboratory, to find soil properties of the selected soil. Finally laboratory tests were carried out to identify required soil parameters (Table 1).

Table 1. Properties of the soil

Property	Value
Natural moisture content	51%
Liquid limit	45%
Plastic limit	24%
Organic content	6.8%
Specific gravity	2.58
Undrained shear strength	14KPa
Young's modulus	1800KPa

3.2 *Dimensions of the model*

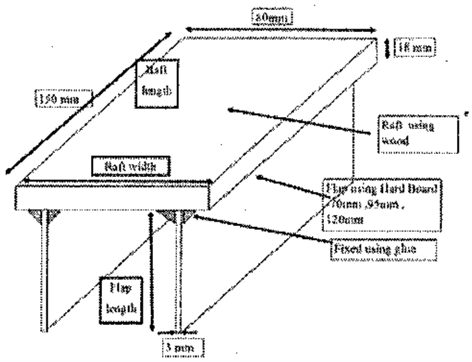

Fig. 1 Proposed model

3.3 *Finite element model analyses using Plaxis*

The Plaxis analysis was performed for 2-D model of the proposed foundation using Mohr-Coulomb model under plane strain conditions. The boundary was the dimensions of the soil box and the boundary conditions were fixed. 15 node triangular elements were used in developing the mesh. The input parameters for the soil such as Young's modulus, unit weight, cohesion used in the Plaxis analysis were the results from the soil tests. The Poisson's ratio of the soil was used as 0.495 assuming incompressibility of soil. The rigid material was assigned with the material properties of wood and the flexible material were assigned with material properties of hardboard.

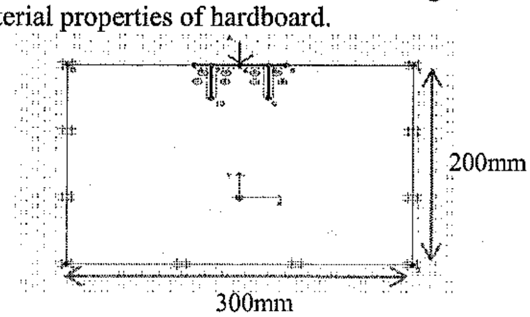

Fig. 2 The foundation model used in the analyses

3.4 *Physical model testing*

Four models were prepared which included three physical models of the proposed foundation (rigid raft to which two flexible flaps are attached underneath) with flap lengths 70mm, 95mm, 120mm and the control model (a rigid raft). Hardboard was used for the flap and wood was used for the raft. The two flaps and the raft were joined together with glue.

First, the experimental model was placed in the soil sample such that the raft was seated on the top soil surface. In order to ensure the visibility of these vertical edges of the flaps, they were painted with white paint prior to inserting into the soil. A dial gauge with a least count of 0.002mm was placed on the horizontal bar of the loading arrangement, to measure the mid span vertical deflection of the raft. Then, the load was applied at a rate of 0.05mm/min to the model, until the ultimate load of 5.76 kN/m was achieved. At the end of the test, the deflected shapes of the flaps were marked on the Perspex wall before withdrawing it from the soil sample. The same procedure was repeated for all the other models.

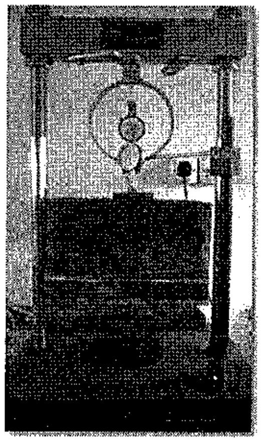

Fig. 3 The physical model testing

4 RESULTS AND DISCUSSION

4.1 ***Physical model testing***

Variation of the mid-point vertical displacement of the raft with the load applied for the rigid raft-flexible flap model with different flap depths and the rigid raft are shown in Figure 4.

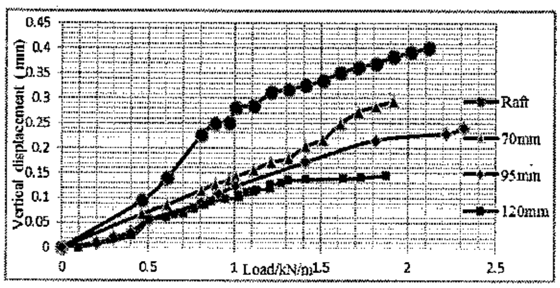

Fig. 4 Variation of mid-point displacement of the raft with the load applied

The variation between the mid-point deflection of the raft and the load applied was obtained for rigid raft-flexible flap foundation model through physical model testing as depicted in Figure 3. It is clear from the results obtained, that when the flap length increases, the mid span deflection of the raft reduces.

4.2 *Finite element analyses results*

The mid-point vertical displacement of the raft of rigid raft - flexible flap model with different flap lengths 50mm, 95mm, 120mm, 175mm and 225mm, at different loads applied (1kN, 2kN, 3kN,4kN, 5kN and 5.6kN) were obtained as follows. The model had a fixed spacing between the two flaps of 50mm. The behaviour of the raft was only analysed here.

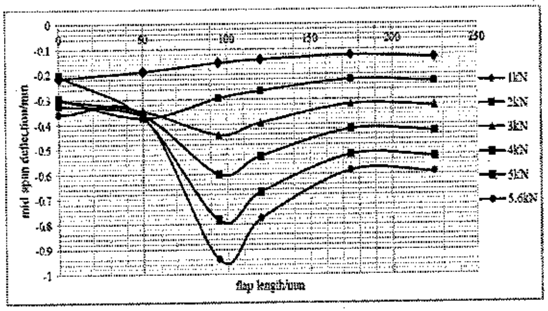

Fig. 5 Variation of the mid span deflection with the flap length for different applied loads

4.3 *Comparison between physical testing and finite element analyses*

Figures 6 to 9 indicate the individual comparison between physical test results and Plaxis analysis result for rigid raft- flexible models with flap lengths 70mm,95mm,120mm) and rigid raft.

1. Rigid raft

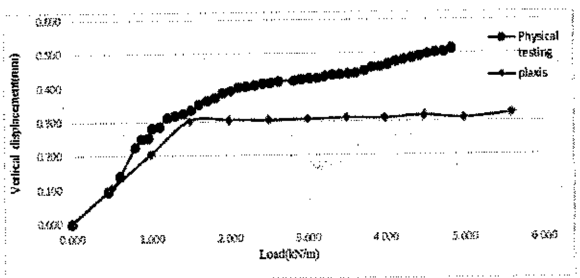

Fig.6 Variation of the vertical displacement of the mid-point of the raft with the load applied for the rigid raft

2. *Rigid raft-flexible flap model with flap length 70mm*

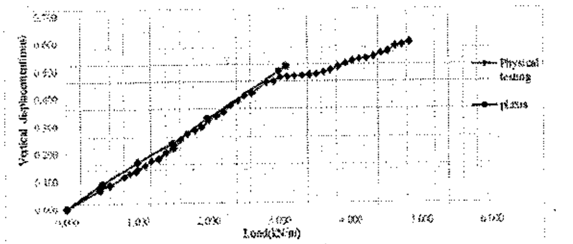

Fig. 7 Variation of the vertical displacement of the mid-point of the raft with the load applied

3. *Rigid raft-flexible flap model with flap length 95mm*

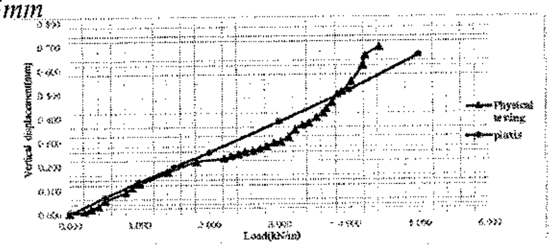

Fig. 8 Variation of the vertical displacement of the mid-point of the raft with the load applied

4. *Rigid raft-flexible flap model with flap length 120mm*

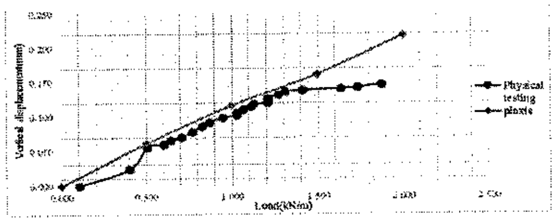

Fig. 9 Variation of the vertical displacement of the mid-point of the raft with the load applied

Although the values obtained for the vertical displacement of the raft at a specific load applied, is nearly equal in both physical testing and Plaxis analysis, there are deviations between the physical testing and Plaxis analysis results.

4.4 *Behaviour of the flap*

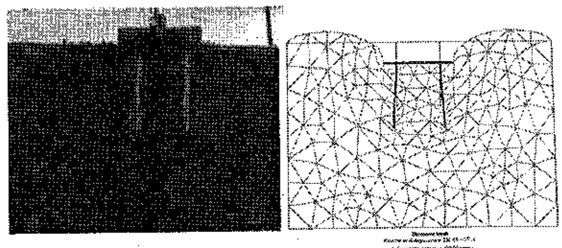

Fig. 10 Comparison of the behaviour of flap in physical testing and PLAXIS analyses for the rigid raft-flexible flap model with flap length 70mm

The similarity in the behaviour of the flap in physical testing and Plaxis analysis is shown in Figure 10 for the model with 70mm flap length. Other models 90mm and 120mm flap length models also showed a similar pattern.

4.5 *Behaviour of the soil*

During the physical model testing, the soil on either side of the raft exhibited a bulging behaviour outwards, with the increase in the load application. This behaviour is clearly shown in Figure 11.

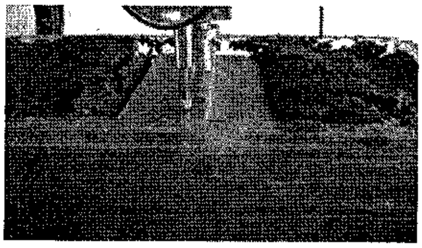

Fig. 11 The bulging behaviour exhibited by the rigid raft-flexible flap foundation model under applied loads

The same behaviour is shown by the model analysed using Plaxis and thus can be explained using plaxis. Figure 12 shows the total displacement of the soil in the model. According to this figure, the soil underneath the raft moves vertically downwards. This movement causes the soil near to the flap to move laterally and ultimately vertically upward, away from the raft this results in the soil bulging.

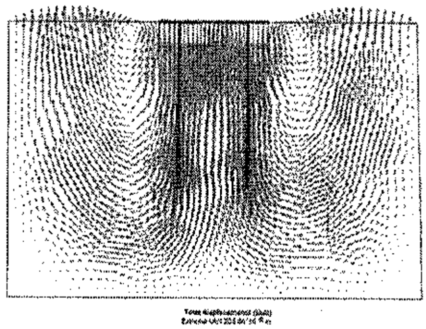

Fig. 12 The bulging behaviour exhibited by the rigid raft-flexible flap foundation model under applied loads

4.6 *Theoretical calculations*

According to the theory of elastic settlement of foundations by Steinbrenner in 1934, elastic settlement, $Si = C_s\, q\, b\, (\frac{1-v^2}{E_s})$.

Using above equation elastic settlement of the raft was calculated for further verifications. Substituting the data, Es = 1800 KPa, v= 0.495, C_S= 0.50 and q= 17.5 KPa (Elastic failure under raft foundation – found from Plaxis) for the equation, elastic settlement at the mid span of the raft can be found as 0.294mm. This compares favourably with those obtained from experiments and finite element analyses (Table 2).

Table 2.Comparison of results

Method	Midpoint vertical Displacement(mm)
Theoritical calculation	0.294
Physical testing	0.316
Finite element analtses	0.287

5 CONCLUSIONS

- For a rigid raft-flexible flap model, with the increase in flap length and spacing between the two flaps, the vertical displacement of the raft reduces.
- Soil underneath the raft undergoes a vertical movement when load is applied. To keep the volumetric strain constant, soil moves in the lateral direction which results in horizontal deflection of the flaps. Thus, the flap and the raft mutually influence each other's' deflection behaviour.
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ACKNOWLEDGEMENT

We would like to thank our project supervisor Prof: K.G.H.C.N. Seneviratne for supervising and guiding us to make our project a successful one and the panel members, Prof. U. de S. Jayawardana and Dr. (Mrs.) D. de S. Udakara for evaluating and instructing us.

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Comparison of Different Ballast Standard Gradations Based on Shear Strength

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ABSTRACT: There are different standard gradations are used in all over the world and Sri Lanka railways also specified a gradation limits in the selection of ballast for rail tracks. Current standard is closely resembled to the Indian rail track ballast specifications. However, there is no examination conducted to evaluate the performance of rail track ballast used in Sri Lanka. The primary purpose of the study to investigate the shear behaviour of current Sri Lankan ballast gradation compared to selected ballast specifications from other countries based on shear strength. Parallel gradation technique was used to model the sample as it is difficult to handle large size ballast. Direct shear tests were conducted on different ballast gradations including current Sri Lankan specification. The results showed that the Current Sri Lankan ballast gradation specification which is the same as Indian standard gradation has the highest shear strength compared to other ballast gradations tested.

1 INTRODUCTION

Ballast is the selected graded granular media placed as the top layer of substructure of a railway track. Main functions of ballast are to resist vertical, lateral and longitudinal forces, provide sufficient resiliency, energy absorption, facilitate drainage, reduce the pressure from sleeper to an acceptable stress and distributed to the subgrade (waters 1994). Different countries use different ballast material depending on the availability. The ballast material should be angular, crushed, hard, uniformly graded, free of dust and dirt, and not prone to cementing action for better performance. In Sri Lanka gneiss rock is currently commonly used for ballast. Kaya (2004) stated that, method of selection of ballast has been based on the physical testing of representative specimens to ensure that materials are of the suitable rock type, with no inherent planes of weakness such as foliation and cleavage (Petrographic Analysis), grain shape and size distribution, adequate wearing resistance (Los Angeles Abrasion), and weathering resistance (freeze-thaw, wetting and drying, and absorption) tests. Among all, shear strength of ballast plays an important role in ballast selection as the shear strength of ballast will be affected by the gradation. Previous researchers Kaya, Asadzadeh (2004) have analyzed the effect of gradation on shear strength of ballast. It is expected that, if the density of ballast is increased by using well graded aggregates, the inter particle contact stresses will be reduced by the increased contact area of the particles, resulting in the reduction of particle breakage. There are a number of problems associated with employing a very well-graded distribution in track. Two of the most important are size segregation and a reduction in drainage. However, it is obvious that the shear strength is higher for the well graded materials (Indraratna, 1991).

The gradation is specified by the relevant countries' rail authority. European standard gradation, American standard gradation (AREMA), Australian standard gradation, Indian standard gradation and British standard gradation, etc. are some of the ballast specifications (Alemu, 2001). Sri Lankan Railway currently uses a ballast gradation specification similar to the Indian standard gradation yet no proper study has been conducted to evaluate the engineering behavior. Therefore, it is necessary to investigate the engineering properties of ballast and optimize the gradation to suitable for Sri Lankan conditions. The shear strength properties are important since ballast is a granular medium. Therefore, this study is focused to study the shear behavior of different ballast gradations and thus evaluate the performance of current Sri Lankan ballast gradation in the aspect of shear strength.

The size of the ballast is large hence it is difficult to conduct the direct shear test for the ballast and scaling down of the particle size is needed. For scaling down, parallel gradation technique can be used. Lowe (1964) originally presented the framework for the parallel gradation modeling technique. The only difference between prototype and model sample is the difference in size of particles. The model sample should closely duplicate the behavior of the larger prototype. As a result of using this technique, there will be a small decrease in shear strength (Cambio, nd), but it has no significance for the comparison of shear behavior of standard gradations. This study mainly focused on finding the optimum standard gradation for Sri Lanka based on shear strength.

2. MATERIALS AND METHODS

European standard gradation, American standard gradation (AREMA No. 4), Indian standard gradations and British standard gradation were used for the study. The parallel gradation technique was used to scale down the original gradation to a gradation with smaller particle sizes could be tested in conventional direct shear apparatus (Cambio, nd). In order to maintain the properties of rock the same with prototype ballast in rail tracks the specimens were prepared from the material obtained from a quarry which supply ballast to Sri Lanka Railways. Considering the worst conditions, the lower limits of the standard gradations were taken for the study. The Fig. 1 shows the particle size distribution curves used for the study.

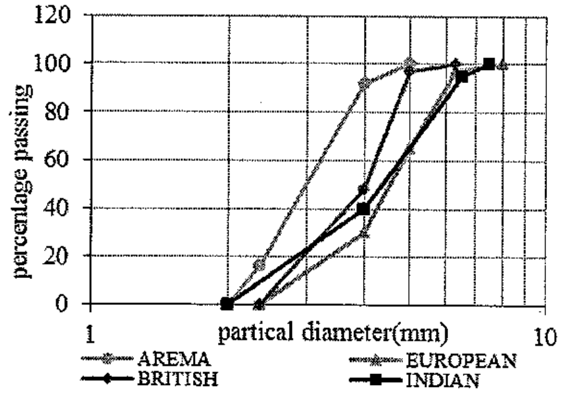

Fig. 1 Different ballast gradations used for the study.

The physical properties of ballast material were investigated first. The specific gravity, aggregate impact value (AIV), aggregate crushing value (ACV) and water absorption was found using relevant standards and results are tabulated in table.1. Available sieves to BS, ASTM standards were used to prepare the modeled ballast samples to follow the selected gradations.

Table 1. Physical properties of ballast

Properties	Values
Specific Gravity	2.684
AIV	35%
ACV	28.5%
Water absorption	0.194%

The direct shear tests were used to investigate the shear behavior of different ballast specimens as it is the most efficient method to study the shear behavior of granular material and low cost (Fityus, nd). 100 mm diameter standard direct shear device was used for the current study.

Each test sample was compacted in three layers inside the shear box applying the same compacting effort. The tamping was done using 8mm diameter steel rod with 25 blows per layer in shear box. Depending on the gradation, initial density of the specimen were not the same. The initial specimen densities are given in table 2. All the direct shear tests were conducted at the same shearing rate of 0.2mm/s under three normal pressures of 15 kPa, 30 kPa and 90 kPa (BS part7,1990).

Table 2. Density, Uniformity coefficient (Cu) and Coefficient of curvature (Cv) of tested standard gradations

Gradations	Cu	Cv	Density (kg/m^3)
European	1.68	1.14	14.52
British	1.57	1.01	14.09
AREMA	1.50	0.97	15.12
Indian	1.96	1.00	14.74

3. RESULTS AND DISCUSSION

Fig. 2 shows the variation of shear stress to the shear displacement under all the normal pressures for tested ballast gradations.

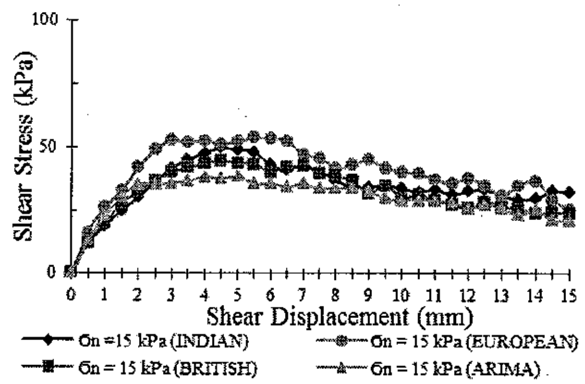

Fig. 2(a)

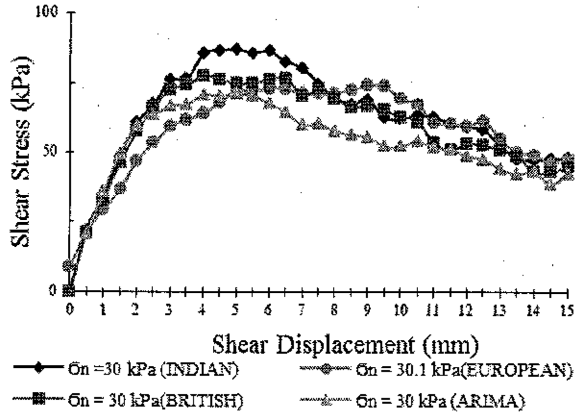

Fig. 2(b)

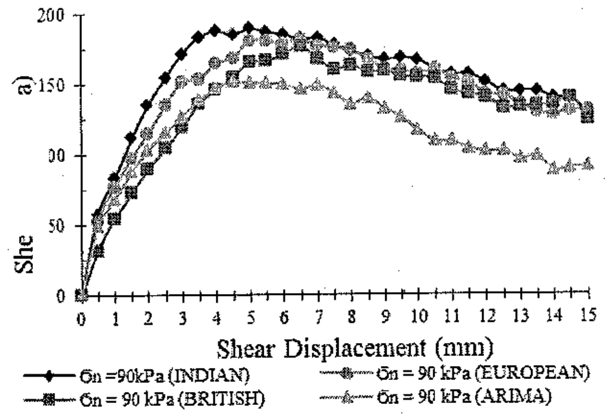

Fig.2(c)

Fig. 2 Shear stress-Shear displacement relationships for normal pressures of (a)15 kPa (b) 30 kPa and (c) 90 kPa.

From Fig. 2, it is seen that Indian standard gradation has the highest shear strength at high normal stress and European standard gradation has the highest shear strength at low normal stress compared to that of other standard gradations tested.

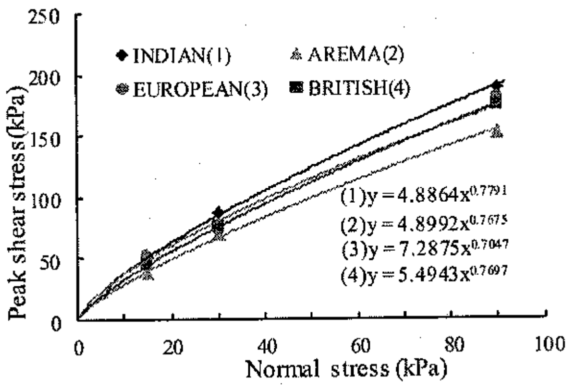

Fig. 3 Relationships of peak shear strength and normal stress

It was observed that for all ballast samples, the shear strength envelope at failure is better resembled to nonlinear behavior. Fig. 3 shows the variation of peak shear strength with normal stress ballast assuming nonlinear Mohr Coulomb failure criteria. It has been explained that this nonlinear behavior was due to particle interlocking of their highly angular nature. The results of the study shows, Indian standard gradation has higher shear strength than the other gradations. European and British standard gradations show intermediate shear strength while AREMA No.4 ballast gradation has the lowest shear strength.

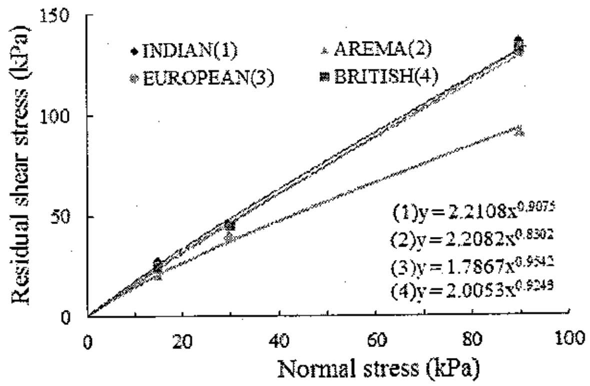

Fig. 4 Variation of residual shear stress with normal stress

Fig. 4 shows the variation of residual shear stress to the normal stress. Indian, British and European ballast gradations has comparatively similar residual shear strength and American standard gradation has shown a lower residual shear strength compared to the other gradations.

Table 3: Peak and Residual friction angle of tested standard gradations.

Gradations	Cu	peak friction angle(deg)	residual friction angle(deg)
European	1.68	60.0	54.6
British	1.57	59.8	55.3
AREMA	1.5	55.9	42.1
Indian	1.96	61.1	55.5

It is further seen that, the friction angle is decreasing with the increase of normal stress due to the nonlinear behavior. Also it was observed an increasing trend of friction angle with the increase of uniformity coefficient for the ballast tested The observation indicate that most compacted and well graded aggregates have better ability to resist loads than uniformly graded aggregates.

4. CONCLUSION

Indian standard gradation shows highest shear strength than that for other standard gradations.

The friction angle is decreasing with the increase of normal stress for the ballast used

Shear strength increases with uniformity coefficient for the ballast used in this study and Indian standard gradation has highest uniformity coefficient than others.

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Use of Paddy Husk Ash as a Binder in Improvement of Soft Peaty Clay

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ABSTRACT: Ground improvement in soft soils through pre-consolidation by preloading with or without vertical drains or vacuum consolidation has the main drawback of time consumption. Deep mixing with the cement is an alternative solution that has been studied and it was found that cement percentages of 20-30% are required to obtain a sufficient level of improvement. In view of the high cost involved the study of the use of paddy husk ash (PHA) also known as Rice Husk Ash (RHA)-another pozzolonic material which is a waste product of rice production has been conducted. Improvement through mixing natural peat with different proportions of cement and PHA was studied in this project. Engineering properties of samples comprise with different mix proportion of cement and PHA were compared with samples of untreated peat. Improvement in compressible characteristics and shear strength were evaluated through performing one dimensional consolidation test and undrained triaxial test.

1 INTRODUCTION

Rapid development and need to provide good infrastructure facilities has caused a high demand for land. Most of the lands underlain by competent soils are occupied already and it has become necessary to use lands with poor sub soil conditions to meet the demand. Improvement of the existing poor soil conditions to an adequate level or use of deep foundations to transfer the loads to an underlying competent soil layer are the two options available. With the development of roads that occupy a large area in plan improvement of the insitu ground condition becomes the more viable option.

Mechanical and chemical stabilization methods were used for improving the soil strength and stiffness to an adequate level. Mechanical methods involve driving out of excess water from the soil through consolidation. Alternatively, microstructure of the soil is changed by mixing with an appropriate binder in the chemical stabilization methods. Cement and lime are the binders used in the early stages. With high cation exchange capacity in Peaty clay high percentages of the order of 30% were required to improve the peaty clays to an adequate level. (Munasinghe 2001, Saputhantiri and Kulathilaka 2011, Lahtinen et al 1999) Research was directed at using industrial by products along with cement to reduce the excessive costs. Blast furnace slag- a byproduct from steel industry has shown very promising results in European countries. (Jegandan et al 2001, Makusa 2012).

The need for stabilizing soft peaty clay is a critical problem in Sri Lanka and in the absence of proven by products such as blast furnace slag it is necessary to identify commonly available byproducts in the country. Hence, the attention was given to paddy husk ash (also known as rise husk ash). Paddy husk ash is a by product of rice generation and paddy hush ash is a byproduct of the burning of paddy husk in manufacturing of tiles and bricks from clay. Paddy husk is burned at temperatures around 500 C° in this process.

Paddy husk ash is proven to be a pozzolonic material and used in improvement of engineering properties of soils. (Vinh 2012)

A study by Chandrasiri, Priyankara, and Madhusanka (2012) showed that mixing peaty clay with 20% cement and 20% PHA, primary consolidation characteristics as indicated by C_c or m_v were enhanced and secondary consolidation characteristics as indicated by C_α decreased significantly. The improvements achieved with paddy husk ash were much less compared to those achieved with cement.

Therefore attempts were made in this research to identify the possibility of using paddy husk ash by in combination with cement to improve engineering characteristics of peaty clay.

2 PREPARATION OF SAMPLES

Samples were prepared by mixing peaty clay with cement and PHA with a hand mixer in the laboratory. In order ensure a uniform level of mixing in all cases, similar mixing speeds and mixing durations were used and mixing was done with small quantities. Mix was placed in buckets and left for 28 days under submerged conditions and under surcharge of $10kN/m^2$. As a control sample peaty clay was remoulded and kept to settle for 28 days under similar surcharge. The basic properties of the five samples after 28 days are summarized in Table 1. Undisturbed specimens were obtained from the five samples to conduct the necessary laboratory tests to determine the strength and compressibility characteristics. Five number of samples were prepared mixing different percentages of cement and PHA.

Table 1. Description of samples

Sample No	Mix proportion
S1	Cement 30%
S2	Cement 20% PHA 10%
S3	Cement 10% PHA 20%
S4	PHA 30%
S5	Peat only

Table 2. Sample properties

Basic property	Moisture content %	Specific gravity	Initial void ratio	pH value at 25^0C
S1	117	1.68	1.96	9.4
S2	143	1.67	2.39	8.8
S3	142	2.00	2.84	8.2
S4	198	1.51	2.99	3.6
S6	188	1.98	3.72	2.1

3 COMPRESSIBILITY CHARACTERISTICS

The Oedometer testing apparatus was used to determined compressibility characteristics of treated and untreated peaty clay. Laboratory consolidation tests were conducted by loading, unloading and reloading with different stresses increments. The effects on the parameters corresponding to primary and secondary consolidation were analyzed separately.

Loading/ unloading was done once in three days specially to concentrate on secondary consolidation characteristics. Tests were done with loading increments of 5, 10, 20, 40, 80, 160, 320 and 640kN/m^2. After unloading to a stress level of 10kN/m^2, specimens were reloaded through increments of 20, 40, 80, 160, 320, and 640 kN/m^2.

3.1 *Changes in e vs log (σ) relationship*

Table 3. Data obtained from e Vs log (σ) plots

Sample	Cc	Cr	Cc/1+e$_0$
1	0.111	0.046	0.038
2	0.219	0.056	0.065
3	0.296	0.064	0.078
4	0.334	0.099	0.083
5	0.387	0.056	0.054

Changes in the primary consolidation characteristics were illustrated through e vs log σ plots. The plots for all samples are presented in Fig. 1. The basic consolidation characteristics are compared in Table 3. It could be clearly see in Figure 1 that initial void ratio is reduced with the mixing of cement or PHA. Mixing with 30% of cement, S1 provided the maximum improvement. Mixing with 30% PHA alone, S4 provided the least improvement. Mixing of a smaller amount of cement together with PHA to compensate in S2 and S3 provided better improvements than mixing PHA alone, but less improvement than the use of 30% cement.

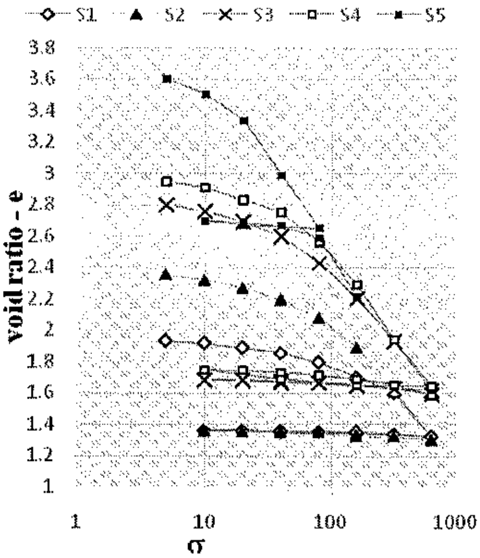

Fig. 1 e vs log σ plot for treated and untreated peaty clay samples

3.2 *Changes in coefficient of volume compressibility*

Coefficient of volume compressibility is an alternative parameter for estimating settlement. It is evaluated at different stress levels. The influence of mixing with cement or PHA on this parameter is illustrated by the comparison of the parameter for differently treated peaty clay samples in loading stage in Fig. 2.

This result also confirmed that the coefficient of volume compressibility decreased due to mixing with PHA, but the reduction due to mixing with cement was much greater.

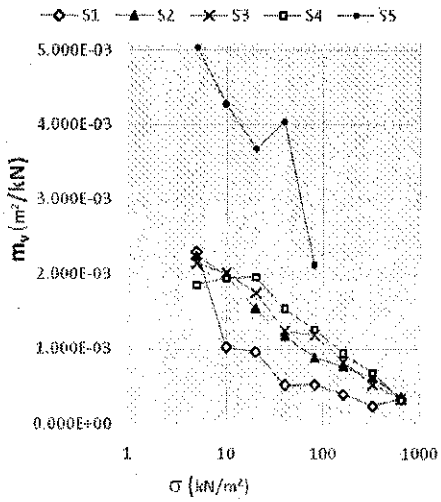

Fig. 2 Effects on m_v plots for loading stage

3.3 *Changes in coefficient of secondary consolidation (C_α)*

Secondary consolidation is continued creep after complete dissipation of pore water pressure and soft organic soils show large amount of secondary consolidation. The Coefficient of secondary consolidation $C\alpha$ values were obtained from gradient of the secondary consolidation part of void ratio vs. (log) time curve and presented in Fig. 3.

The results showed that the secondary consolidation coefficient was considerably reduced due to the mixing with cement or PHA. Here also the improvement achieved with mixing of cement was much greater. Use of PHA also caused some improvement but to a lesser extent.

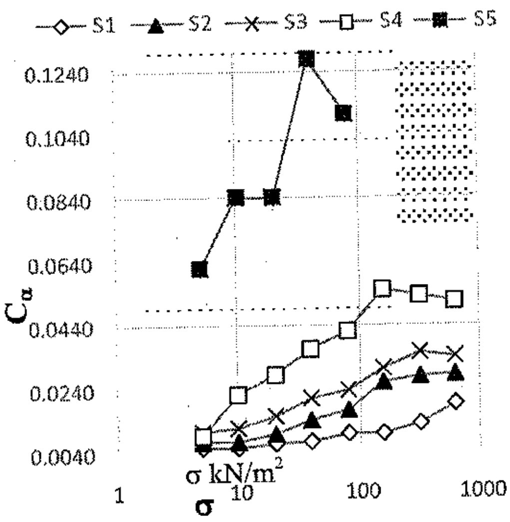

Fig. 3 C_α plots for samples in loading stage

4 CHANGES IN UNDRAINED SHEAR STRENGTH

The other most undesirable engineering characteristic of peaty clay is low shear strength. In construction projects the soft subsoil has to be loaded at a practically acceptable rate to prevent shear failure. In such cases undrained shear strength is the most appropriate parameter in evaluation of stability. Cell pressures of 50, 100 and 150 kN/m^2 were used for testing three specimens from each sample. Continuous increment of axial stress was maintained till failure of the sample occurred. Stress – strain relationship and Mohr circles were plotted for each sample and undrained shear strength parameter (C_u) was evaluated.

Table 4. Values obtained for shear strength

Sample No	Sample description	Average C_u (kN/m^2)	
1	Cement 30%	46.67	15
2	Cement 20% PHA 10%	18.67	
3	Cement 10% PHA 20%	19.33	
4	PHA 30%	11.00	
5	Peat only	5.36	

The maximum improvement of undrained shear strength was achieved with mixing of 30% cement. Mixing of smaller amount cement and PHA to compensate the reduction, achieved less improvements.

5 CHANGES IN MICROSTRUCTURE

The changes in the microstructure of peaty clay due to mixing of cement can be illustrated by the comparison of Figure 4 and Fig. 5. The formation of structure can be seen in Fig. 5 where 30% is used With the 30% PHA only similar structure formation was not seen (Fig. 6).

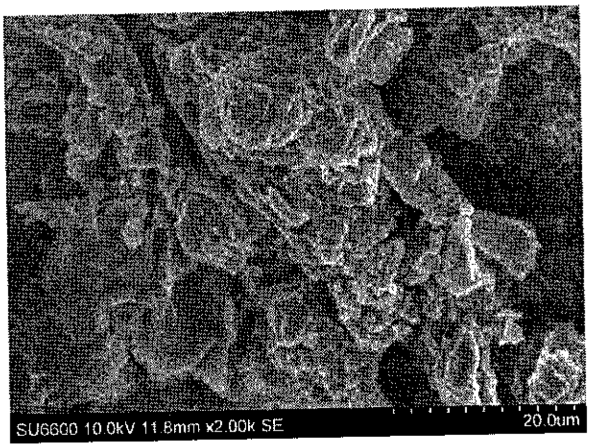

Fig. 4 Microstructure of the natural peat sample

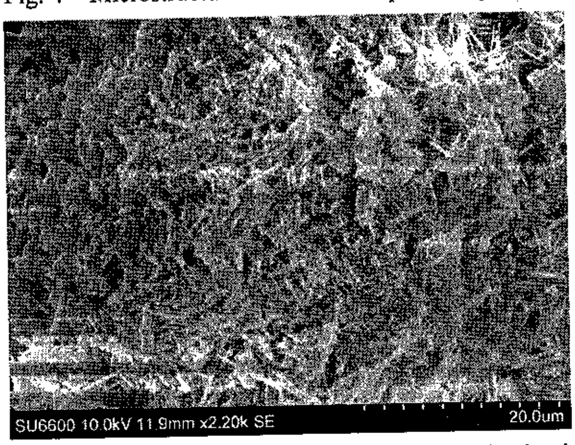

Fig. 5 Microstructure of the peat sample mixed with 30% cement

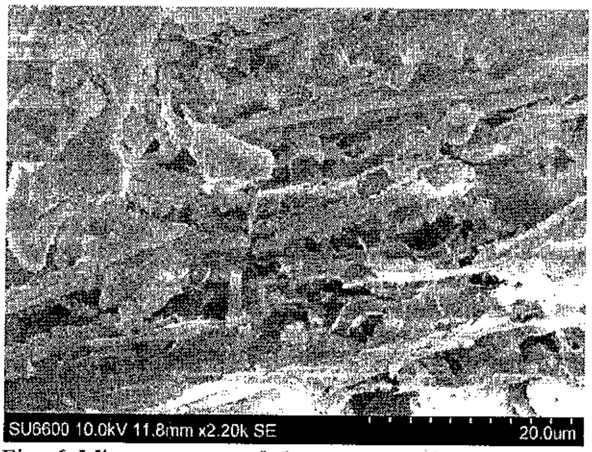

Fig. 6 Microstructure of the peat sample mixed with 30% PHA

6 CONCLUSIONS

Preconsolidation by preloading is the most widely used technique for improvement of soft peaty clay. Even with the use of prefabricated vertical drains the process takes a considerable time. In situations where there are time constraints other techniques such as insitu mixing with cement or other binders have been used. In order to achieve desired level of improvements a cement content of around 30% will have to be used in the improvement of peaty clay. In this research an attempt was made to study whether PHA can replace some amount of cement.

Different percentages of PHA added to peat samples were compared with sample of untreated peat and sample with added cement percentage. As consolidation is the most dominant factor for soft soils Oedometer test was conducted to evaluate compressible characteristics. Undrained Triaxial tests were carried out to evaluate shear strength parameters.

The results of the study showed that the primary consolidation and secondary consolidation characteristics are clearly improved by mixing with PHA and PHA + cement mix. But the improvement achieved through mixing with cement is much greater. Mixing only with PHA did not result in significant improvement.

Improvement in shear strength can be obtained through treating the natural peaty clay. The addition of 30% cement provided the best improvement.

The results showed that to achieve a significant improvement of strength and compressibility a cement percentage of around 20% is needed. PHA percentage of 10% may be added to enhance it further. PHA alone did not cause much improvement.

ACKNOWLEDGMENTS

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Analysis of Gas Transport Parameters of Landfill Final Cover Soil in Sri Lankan Dry Zone

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ABSTRACT: Landfill final cover plays a vital role in separating waste from the surrounding. Therefore landfill final cover should able to minimize infiltration of moisture and to control harmful gas emissions through it. In dry zone of Sri Lanka, due to less precipitation, infiltration is less. Thus more attention should be given regarding harmful greenhouse gases, produced within the landfill as a result of anaerobic decomposition. Therefore it is important to enhance Oxygen (O_2) transfer through final cover and hence to allow aerobic decomposition of refuse. The gas exchange process through final cover soil is controlled by both diffusive and advective gas transport properties. Based on laboratory experiments it was realized that gas transport parameters are high in dry side of optimum of compaction. And gas transport parameters were gradually increased with the air content. Further, by combining both hydraulic performances and gas transport parameters final cover thickness was decided.

1 INTRODUCTION

Final cover is an essential part of all municipal solid waste landfills. A well designed final cover controls moisture infiltration, limits the formation of leachate, prevents direct exposure of waste to the atmosphere and controls gas emission and odor. Regulations have been implemented to construct the conventional final cover with several layers (USEPA, 2005) to meet above requirements.

However a complex layer system is not economical for a country like Sri Lanka. Thus a simple cover system should be identified which equally performs as the conventional approach. Alternative Final Cover (AFC) system, is one such system predicated on scientific and engineering principles and practices(Wickramarachchi et al, 2010).

The anaerobic degradation of organic waste in a landfill produces a mixture of gases that contains methane, carbon dioxide and various other toxic compounds (IPCC, 2007). But in Sri Lanka recovery of landfill gases is not in practiced. As a result, landfill gases are emitted to atmosphere through final cover soil which creates numerous environmental problems. Therefore, it is economical to design the landfill final cover to promote oxygen exchange between the atmosphere and the waste layer to maintain the aerobic condition in the waste layer (Wickramarachchi et al, 2010).

Nevertheless, when selecting the final cover soil for landfill, a special care should be given not only to minimize generation of greenhouse gases also to minimize leachate production. Therefore, it is necessary to select high gas exchangeable and less water permeable material for final cover of a landfill (Wickramarachchi et al, 2010). However, finding such a material with two opposite properties is not an easy task.

Since rain water infiltration through soil in dry zone is very less, the leachate generation is comparatively less. Thus, in this research study, a suitable material for an alternative final cover in dry zone of Sri Lanka was selected, taking gas exchangeable properties as the dominant factor. Further, results were compared with the existing predictive models for the gas transport parameters and final cover thickness was decided according Sri Lanka Central Environmental Authority (CEA) regulations.

1.1 *Numerical models for gas transport parameters*

1.1.1 *Diffusion*

Diffusion is a principal mechanism in the interexchange of gases between soil and the atmosphere due to a concentration gradient, which is governed by the soil-gas diffusion coefficient (D_p). Fick's law is used to calculate D_p and it can be expressed mathematically as;

$$\varepsilon \frac{dC}{dt} = D_p \frac{d^2C}{dz^2} \qquad (1)$$

where C is the gas concentration, Z is the elevation difference of the sample, t is time period and ε is the soil-air content. Generally diffusion is represented as a ratio of gas diffusion coefficient of soil (D_p) with respect to gas diffusion coefficient in free air (D_o). This is termed as soil-gas diffusivity (D_p/D_o).

Since determination of the gas diffusion in laboratory is time consuming and difficult, researchers have introduced mathematical formulae to determine the gas diffusivity as a function of soil type and the soil-air content (ε). Buckingham(1904) suggested that the soil-gas diffusivity follows a power law function of the soil-air content as,

$$\frac{D_p}{D_0} = \varepsilon^x \qquad (2)$$

Where x is an exponent characterizing pore connectivity-tortuosity (Buckingham, 1904). For this simple model Buckingham (1904) suggested $x = 2$ whereas Marshall (1959) suggested $x = 1.5$. Penman (1940) further modified the model proposed by Buckingham and proposed a relationship between soil-gas diffusivity and soil-air content as,

$$\frac{D_p}{D_0} = 0.66\varepsilon \qquad (3)$$

1.1.2 *Advection*

Advection is a gas transport mechanism induced by soil-air pressure gradient which can be modeledby coefficient of air permeability (k_a). It is computed based on Darcy's law as,

$$q = -\frac{AK_a}{\eta}\left(\frac{dP}{dz}\right) \qquad (4)$$

where dP/dz represents the pressure gradient across the sample, A is cross sectional area of the soil sample, η is the dynamic viscosity of the air (1.86×10^{-5} Pas) (Wickramarachchi et al, 2010). Existing predictive models for air permeability are also based on a power law function of soil-air content (ε), similar to air diffusion. The generalized form of such model proposed by Buckingham (1904) can be written as,

$$K_a = \alpha\varepsilon^x \qquad (5)$$

where α is a constant to pore connectivity and x is a power law exponent labeled as a water blocking factor for k_a. Buckingham (1904) suggested $x = 2$ and Marshall (1959) suggested $x = 1.5$ based on several laboratory experiments, Kawamoto et al (2006) suggested that α =700.

2. MATERIALS AND METHODOLOGY

Laterite soil, commonly available in dry zone of Sri Lanka was selected as the candidate material. After checking all the physical properties of laterite soil, for sample preparation 2.36 mm sieved soil was used and compaction was done in 100 cm^3 molds. By adding different sand percentages by 5% increments (5%, 10%, 15%, 20%, 25% and 30% by soil weight) to natural laterite soil, different soil sand mixtures were prepared. Then compaction curves were developed for each and every soil sand mixture. The compaction curves of soil-sand mixtures are depicted in Fig 1.In order to check gas transport parameters, three samples from dry side and three samples from wet side were taken for a particular soil sand mixture.

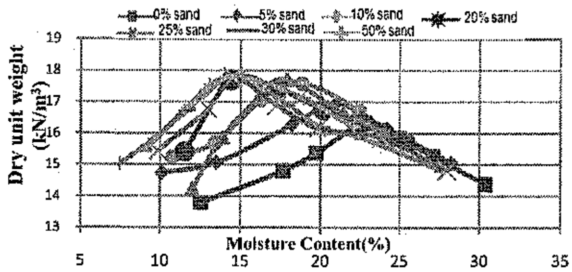

Fig. 1 Compaction curves for different soil mixtures

2.1 *Determination of gas transport parameters*

k_a was measured in the laboratory using air permeability setup illustrated in Fig. 2, in which air is flowing at a given inlet pressure through soil samples .Using the pressure gauge, pressure difference between top and bottom of the sample was measured to calculate k_a value based on Darcy`s law (Curie, 1960).

D_p was measured using the diffusion chamber as illustrated in Fig. 3.Initially the closed chamber was flushed with N_2 and all the O_2(used as the tracer gas) inside the chamber was removed. Then the soil sample was fixed to the apparatus, keeping the top surface of the sample open to the atmosphere. Once the chamber was opened, atmospheric air is diffused to the chamber through the soil sample. Change of O_2 concentration was measured as a function of time using the O_2 sensor connected to the diffusion chamber through a data logger, and then D_p was calculated (Wickramarachchi et al, 2010). The gas diffusion coefficient of oxygen in free air (D_o) at 20^0 C was taken as 0.20 cm^2/s(Curie, 1960).

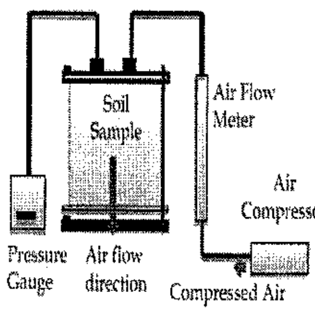

Fig. 2 Air permeability setup

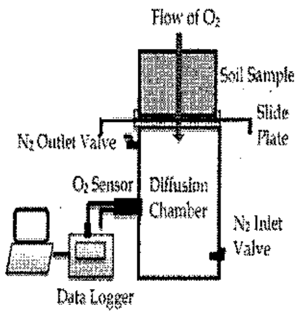

Fig. 3 Gas diffusion setup

3. RESULTS AND DISCUSSION

3.1 *Effect of soil air content on soil gas diffusivity*

The measured diffusivity (D_p) was normalized with respect to diffusivity in free air (D_o) and variation of D_p/D_o over air content (ε) is illustrated in Fig.4.

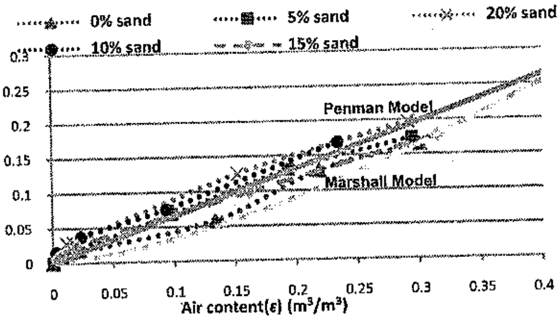

Fig. 4 Soil-gas diffusivity vs. air content

It can be seen that D_p/D_o increases with ε irrespective of sand content. However, when sand content of cover material increases, D_p/D_o increases. This implies that, D_p/D_o is having an effect of by soil structure. When test results were compared with numerical models natural laterite soil is well agreed with Marshall model where as amended soil with sand is agreed with Penman model.

It is well understood that moisture content is significantly affected on the air presence in the compacted soil. Compacted soil in the dry side of OMC has higher air content than that of wet side of OMC. The variation of D_p/D_o with moisture presence in both dry and wet side of OMC is illustrated in Fig. 5.

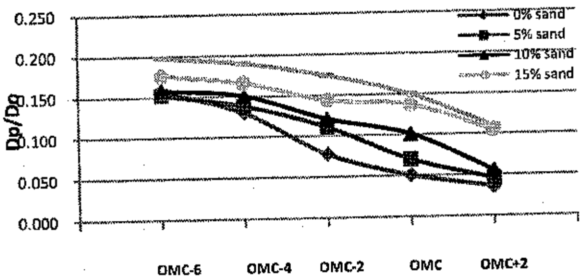

Fig. 5 Soil-gas diffusivity vs. OMC

5.2 *Effect of Soil-Air Content on Air Permeability*

The variation of coefficient of air permeability (k_a)of different soil sand mixtures as a function of soil-air content (ε) is illustrated in Fig. 6. Further, results have been compared with Marshall Model (Wickramarachchi et al, 2010).

It can be noted that $k_a$value has been increased in the dry side of optimum, when ε values are high, irrespective to the sand content. Especially when sand content is increased beyond 20% a significant increase can be noted.

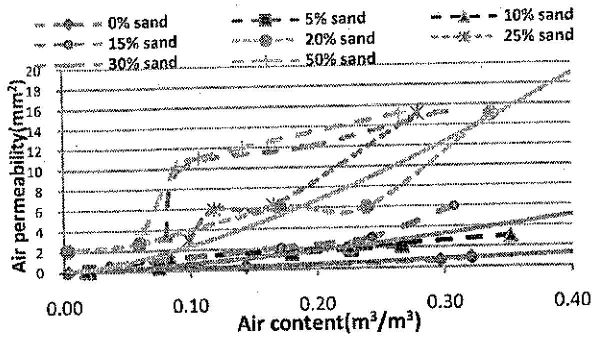

Fig. 6 Air permeability vs. Air content

This is due to the reduction of tortuosity of soil with addition of sand (Curie, 1960). This implies that k_a is highly affected by soil structure.

Based on the test results, it can be seen that k_a has been significantly increased with the addition of sand. Even though air content of the soil increases, there is not any improvement in k_a in all samples up to a certain extent of the soil-air content.

After this particular point, k_a of all soil samples has been significantly increased. This particular point is identified as OMC of the particular soil sample. Based on these results, it is very clear that there is no any noticeable improvement in k_a in wet side of OMC whereas there is a significant improvement in dry side of OMC with the increase of air content. Further, this behavior is clearly shown in Fig. 7, where variation of air permeability over moisture content is presented.

It can be observed from the graph that air permeability in dry side of OMC is significantly higher than that in wet side of OMC. With the addition of sand, total porosity of the material is increased, which enhance the air permeability. This implies that air permeability is strongly affected by soil structure.

Further, test results were compared with Marshall model (Equation *(5)*) in Figure 6. It can be seen that natural laterite soil is well agreed with Marshall model when $\alpha = 5$ whereas amended soil with sand (5 %, 10 % and 15 %) is agreed with Marshall model when $\alpha = 20$.

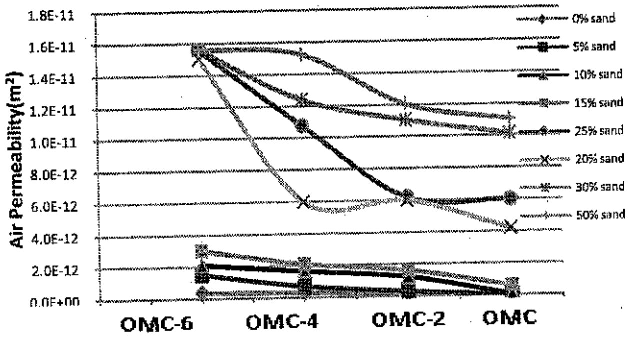

Fig. 7 Variation of k_a with OMC

3.3 *Relationship between gas transport parameters and saturated hydraulic conductivity*

Although gas transport parameters are significant in dry side of OMC, according CEA guidelines (CEA, 1992), landfill cover materials should be satisfied the hydraulic conductivity (k) requirements (k< 1 x 10^{-7} m/s). Thus a relationship has been developed between gas transport parameters and hydraulic conductivity, in order to select the most suitable landfill cover material.

Variation of D_p/D_o and hydraulic conductivity over moisture content is shown in Fig. 8, whereas variation of k_a and hydraulic conductivity over moisture content is shown in Fig. 9. Further, effect of addition of sand on gas transport parameters and hydraulic conductivity is presented. The landfill cover material requirement with respect to hydraulic conductivity (i.e. 1 x 10^{-7} m/s) are also presented in Fig. 8 and Fig.9.

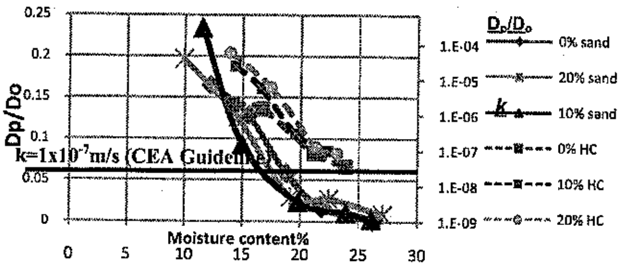

Fig. 8Variations of D_p/Do and k with Moisture Content

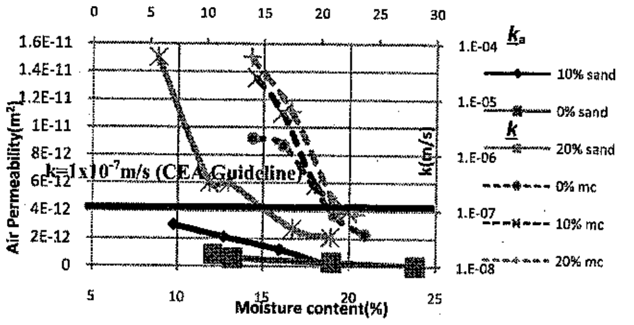

Fig.9 Variations of k_a and k with Moisture Content

It is clear that both gas transport parameters and k values have been decreased with increase of moisture content. With the addition of sand to cover material, both saturated hydraulic conductivity and gas transport parameters have been increased.

4 CONCLUSIONS

Based on the results, it was realized that both k_a and D_p/D_o are increased with soil-air content and they are significantly increased in dry side of OMC irrespective of sand content in the cover material.

In order to satisfy CEA guidelines unamended laterite soil should be compacted in OMC and wet side of optimum. However, gas transport parameters are significantly high in dry side of OMC.

Based on the analysis, it was identified that, the range of 2-4% less than OMC, is the optimum region when designing the final cover in dry zone considering both gas transport parameters and hydraulic performances. If gas transport parameters were not matched with the required hydraulic conductivity standards, cover thickness has to be increased. Hence the head loss can be increased and hydraulic conductivity can be reduced as required.

ACKNOWLEDGMENTS

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Development of a Capping Material for an Engineered Landfill in Wet Zone of Sri Lanka

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ABSTRACT: I Capillary Barriers (CB) which consist of coarse sand overlain by a low permeable fine soil are low cost alternatives used in capping systems for controlling the infiltration in a landfill. Hydrophobicity (water repellency) of CBs containing coarse sand mixed with Oleic Acid (OA) was investigated using Water Drop Penetration Time (WDPT) test. Tested capping layer was consisted of a layer of coarse sand mixed with an optimum amount of Oleic acid which enhances water repellent characteristics, minimising the infiltration, overlain by a clayey soil layer mixed with biochar and compost. WDPT increased sharply with increasing HA content and reached a peak value of 4080 s at 3gkg-1 OA content and thereafter decreased gradually. In addition, the effect of moisture content of coarse sand particles on its hydrophobicity was studied. This decrease is less significant for the optimum value of OA content of 3gkg-1 within the range of moisture content tested. At the optimum OA content, the influence of moisture content on the water repellency was insignificant for the range of moisture content tested. Irrespective of the OA content, an increase in moisture content decreased the water repellency. Stability of the capping layer was assessed by carrying out slope stability analysis at the interface between the oleic acid mixed sand layer and the clayey soil layer. For this purpose, interface shear strength parameters between the two layers was determined by carrying out direct shear tests for different mix proportions of compost and biochar. Factor of Safety (FOS) was determined using infinite slope theory under wet and dry conditions. Landfill capping slope can be increased up to 15 degree to have required slope stability in safe manner for the materials used in this study

INTRODUCTION

An Engineered Landfill allows final disposal of solid waste in a secure manner by minimizing the impacts on the Environment. To enhance this, the final cover or the capping layer should be selected in such a way that it should minimize infiltration and assist gas emissions. The leachate generation is caused by precipitation, percolating through waste deposited in landfill. Leachate from landfill varies widely in composition depending on age of the landfill and the type of waste it contains. Through the effective capping layer development, the leachate production will be limited and greenhouse gas effects can be minimized.

In this study, a capping layer is to be developed for the wet zone of Sri Lanka. Though modernengineering capping systems such as geomembranes, geotextiles, and geo-synthetic clay liners are technically effective (Koerner et al, 1997, [1]), they are not suitable for waste landfills in developing countries such as Sri Lanka due toeconomic and technical constraints (Subedi et al, 2012, [2]). Therefore, development of a capping layer using locally available materials is appropriate.

The study considered only the aspect of infiltration control of a capping layer developed using locally available materials. A low permeable layer was introduced at the top of the waste for infiltration control. A possible technique to enhance the impermeable properties of this layer is to make water repellent grains by mixing the earthen cover material with a hydrophobic agent (HA) (Subedi et al, 2012, [2]). Capillary barriers consisting of a layer of fine grains underlain by a layer of coarse grains are commonly used. Coarse particle are mixed with a Hydrophobic Agent (HA) to introduce water repellence properties which would result in less infiltration of water into the underlying waste (Subedi et al, 2012,[2]).

There can be methane, carbon dioxide and Volatile Organic Compounds (VOC) present in the gas emissions in landfills. The Global Warming Potential (GWP) of CH_4 is 25 times compared to the GWP of CO_2 over the 100 year horizon (IPCC, 2007, [3]).

To minimize greenhouse effects, methane has to be oxidized. Compost covers were identified to be effective in oxidizing CH4 into CO_2(Stern et al., 2007). It has been found that mature sewage sludge compost (SSC) exhibits in principle optimal ambient conditions for methanotrophic bacteria to enhance methane oxidation (Marlies et al, 2011, [4]). Also, for effective VOC removal "Biochar" is used. Biochar is proven to be an absorbent of Volatile Organic Compounds (VOC). The capillary barrier along with an efficient gas removal layer has to be implemented in a landfill.

In addition, stability of slopes is an important aspect in a landfill. There should be enough shear resistance between capillary barrier and gas removal layer to prevent sliding of capping layer.

The objectives of the study are therefore, to determine the optimum oleic acid content that has to be mixed with coarse sand to obmaximum water repellence in capillary barrier system, to determine the optimum mix proportion of low permeable soil, compost and biochar in gas removal layer which has minimum infiltration rate and to determine shear strength characteristics of the interface of capillary barrier layer and gas removal layer for establishing stability of the slopes.

2 MATERIAL AND METHODS

2.1 *Sample preparation for Water Drop Penetration Time (WDPT) test*

River sand was sieved using sieves of sizesranging from 0.6mm to 2 mm in order to separate the coarse particles.

OA was used as the HA and the sample was prepared using mixing-in method (Subedi et al, 2012,[2]) where sand is mixed thoroughly with liquid OA in a plastic bag and stored under 20°C climate controlled condition to equilibrate. Air dried grains were mixed with different contents of Oleic Acid varying from 1 g/kg to 10 g/kg. These were known as "dry hydrophobized grains"

From the WDPT results of the dry hydrophobized sand, 3 samples were selected to investigate the effect of moisture content on the Hydrophobicity. For this purpose, a hydrophobized coarse sand specimen was prepared by mixing with an OA content corresponding to the peak value of WDPT and two specimens corresponding to an HA content from either side of the peak. Distilled water was added to adjust the water content to predetermined values. These water added samples were mixed well and stored in plastic bags under 20°C climate controlled condition. These specimen were known as "wet hydrophobized grains".

2.2 *Water repellence measurement by WDPT test*

The coated sample was packed into a 15 cm diameter, 2cm high cylindrical ring to have a dry density of 1.6 g/cm^3 and the ring. A drop of distilled water of volume 50μl was placed on the surface of the sample using a pipette as shown in Fig. 1 and the time taken for the water drop to infiltrate (WDPT) was recorded. This procedure was performed for both dry and wet hydrophobized grains.

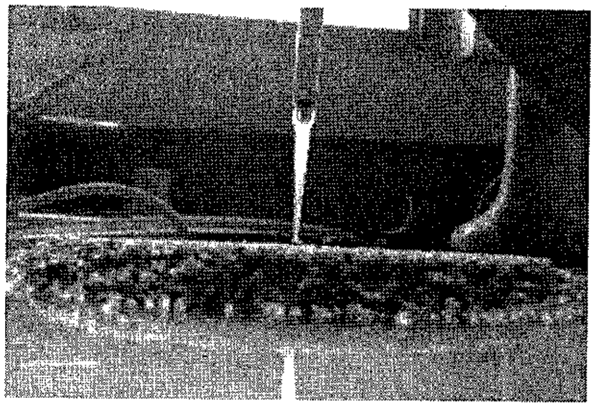

Fig. 1 WDPT Test

2.3 *Sample preparation for permeability test*

The samples which consist of low permeable soil (clay), compost and biochar were selected to test the permeability in gas removal layer. Compost and biochar were sieved using 2 mm and 1.18 mm sieves. Four samples with different proportions were prepared for initial studies. The proportions were differentiated by increasing the amount of compost and biochar. The samples are clay (100%), clay(94%) + compost (5%) + biochar(1%), clay(88%) + compost (10%)+ biochar(2%) and clay(82%)+ compost (15%)+ biochar(3%). Falling head method was used to determine the permeability of the layer consisting of the layer consisting materials mixed under these proportions.

2.4 *Sample preparation for direct shear test*

The combination that gives the minimum infiltration was obtained. The interface shear strength parameters were determined between capillary barrier layer and the gas removal layer which has minimum permeability using the direct shear apparatusunder saturated and dry conditions.

The tests were done at a shearing rate of 0.2 mm/min. Infinite slope theory was applied to determine the Factor of Safety (FOS) for different levels of watertable as well as for different slopes

($_\beta$) using the shear strength parameters that were obtained.

3 RESULTS AND DISCUSSION

3.1 Capillary barrier system

The variation of WDPT with different HA contents is shown inFigure 2.

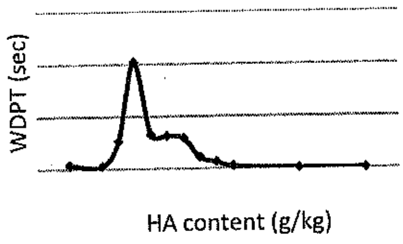

Fig. 2 Variation of WDPT with HA content

Water repellency of samples initially increased sharply with increasing OA content and reached a peak value and thereafter decreased with increasing HA content, the grain surface tends to become more hydrophobic, however, the measured WDPT values decreased after reaching a peak value at 3g/kg, as high amount of OA will reduce the contact angle at the grain water surface due to the multilayer coverage of the grain surface in which a hydrophobic end may be facing outside due to the excess OA content.The peak value of 4080 seconds was obtained at a HA content of 3g/kg.

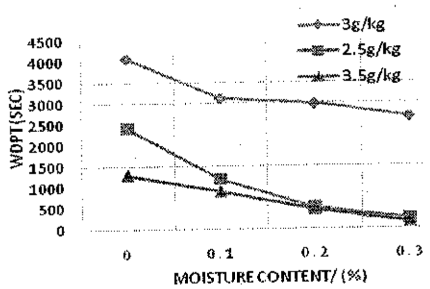

Fig. 3 Variation of WDPT with moisture content

Variation of water repellence with moisture content is given in Fig. 3. It shows that irrespective of the OA content an increase in moisture content has caused the water repellence to decrease. However, this decrease is less significant for the sand grains containing the optimum OA content for the range of moisture content tested.

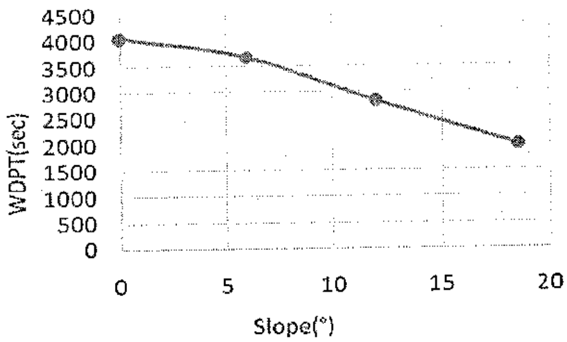

Fig. 4 Variation of WDPT with surface slope

The variation of WDPT with surface slope is shown in Fig. 4. Results showed decreasing behaviour in water repellence when slope increases. It was observed that water drop was spreading, since the surface tension effect reduces, the WDPT time decreased.

3.2 Infiltration characteristics in gas removal layer

The permeability values obtained for the different combinations are shown in table 1.

Table 1. Permeability values obtained for the different samples

Sample number	Sample	Permeability $*10^{-7}$m/s
1	Clay (100%)	1.15
2	Clay (94%), compost (5%), bio-char (1%)	5.76
3	Clay (88%), compost (10%), bio-char (2%)	0.86
4	Clay (82%), compost (15%), bio-char (3%)	0.58

Sample 4 which has the low permeability was further analysed to determine the interface shear strength characteristics under both unsaturated and saturated conditions.

3.3 Shear strength characteristics

Interface shear strength in both dry and wet condition were tested at a shearing rate of 0.2mm/min.

The Factor of Safety (FOS) for different water level change as well as for different slopes ($_\beta$) shown in table 2.

Table 2. FOS of the layers correspondingto different slope

β (deg)	Z	FOS (Dry condition)	FOS (Wet condition)
5	0.30	11.9	3.59
10	0.30	5.95	1.78
15	0.31	3.92	1.17
18.5	0.32	3.14	0.94
20	0.32	2.91	0.86
25	0.33	2.30	0.67
30	0.35	1.87	0.54

The FOS values for wet condition is lower than that for dry condition. FOS values drastically reduced with the increasing slope but the change in FOS for wet condition is not much significant than dry condition.

Table 3. FOS for varying slopes and water table levels

m \ θ	5	10	15	18.5	20	25	30
0	7.98	3.96	2.60	2.09	1.92	1.50	1.21
0.25	6.89	3.42	2.25	1.80	1.65	1.29	1.04
0.50	5.79	2.87	1.89	1.51	1.39	1.09	0.88
0.75	4.70	2.33	1.53	1.23	1.13	0.88	0.71
1	3.61	1.79	1.18	0.94	0.87	0.68	0.55

m: the ratio between the height of water level above the interface and the thickness of the layer above the interface.

4 CONCLUSIONS AND RECOMMENDATIONS

A capping system is the final component in the construction of engineered sanitary landfills. It is possible to use locally available materials as a capillary barrier that can be developed as a low cost alternative to use of geosynthetics. From the results, it can be concluded that use of Oleic acid as a Hydrophobized Agent with a mix ratio of 3g of HA per 1 kg of coarse sand in the capillary barrier, water repellence is developed and it is caused to minimize the water penetration into the waste. At the optimum OA content, the influence of moisture content on the water repellency was insignificant for the range of moisture content tested. Irrespective of the OA content, an increase in moisture content decreased the water repellency.

The use of a layer consists of clay, compost and biochar can be used to minimize the water penetration into the waste. Further the tests can be extended to find out an optimum combination for minimum infiltration. The influence of saturation of layers does not show significant changes comparing the dry condition. When considering the wet and dry conditions of the capping system, factor of safety of layers corresponding to different slopes has been analysed. Landfill capping slope can be increased up to 15 degree to have required slope stability in safe manner for the materials used in this study.

ACKNOWLEDGEMENTS

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Reliability Analysis for Evaluation of Municipal Solid Waste Landfill Slope Stability

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ABSTRACT: The most frequently adopted method in disposal of municipal solid waste (MSW) in Sri Lanka is open dumping and unplanned disposal of waste has resulted in catastrophic shear failures in landfills. This study aims to determine strength parameters of MSW which can be used in proposing stable slope angles for design of MSW landfills in the dry zone of Sri Lanka. Waste samples excavated from Hambantota dump site, representing the dry zone were remoulded and tested by direct shear apparatus. The cohesion of the specimens ranged from 14kPa to 74kPa while the friction angle ranged between 25° and 48°. Failure probability of various landfill slopes were evaluated by using Reliability Analysis via response surface method. For slopes ranging from 1:3 to1:0.5 failure probabilities below 2% were observed. The methodology used in this study can be adopted in the design of Engineered MSW landfills in the dry zone of Sri Lanka.

1 INTRODUCTON

Generation of Municipal Solid Waste (MSW) has exponentially increased in the last few decades. A shocking 3.6 million tons of MSW per day is disposed by 3 billion global city residents according to Hoornweg et al (2013) based on the World Bank review of solid waste management. Additionally, the fraction of city dwellers is expected to increase by 70%~80% in the next few decades resulting in a further expected increase in the current global daily per capita waste generation of 1.2 kg.

Negligence in solid waste disposal in developing countries had lead to unregulated disposal of solid waste primarily in urban and suburban areas. According to World Bank statistics in 2012 Sri Lanka is among the largest MSW producing countries in terms of daily per capita generation. Open dumps are the primary mode of disposing MSW in the country due to the lack of technical knowledge and financial constrains. Deficiency in sufficient land has lead to dumping waste in high mounds and in steep slopes resulting in catastrophic failures in MSW slopes. One example for such failure can be taken as the Payatas Landfill in Manila, Philippines which was subjected to a shear failure in the year 2000 causing 230 reported deaths and over 800 people reported missing (Kölsch, n.d.).

An example for one such recent failure in Sri Lanka can be taken from the 80 foot high Bloemendhal open dump in Maligawatta which took place in mid 2009. The shear failure of the dump resulted in a considerable amount of property damage to nearby dwellings. Evaluation of MSW landfill slope stability in Sri Lanka has not been done previously and it is a vital component which needs to be fulfilled in the design of engineered landfills.

Unlike soil, shear strength parameters of MSW tend to change over time and with location due to bio-chemical degradation taking place within the waste body (Zhan et al, 2008; Reddy et al, 2009). As a result evaluation of slope stability of MSW landfills has proven to be a tedious and unreliable task if done with conventional factor of safety (FOS) methods. Considering the high variability of shear strength parameters of MSW landfills a probabilistic analysis method was adopted in this study. Waste samples were collected from Hambantota waste dump site, in the dry zone of Sri Lanka and direct shear tests were conducted to determine the shear strength parameters of MSW of the dump site prior to evaluating the slope stability of the landfilled solid waste.

Objective of this paper is to propose a methodology which can be adopted in determining slope stability of MSW landfills in the dry zone of Sri Lanka while incorporating the uncertainties involved in determining shear strength characteristics of landfilled MSW.

2 TEST SITE AND SAMPLE COLLECTION

Waste dump site at Hambantota Municipal Council, was selected as the study area to obtain representative samples of a landfill in the dry zone of Sri Lanka. Schematic diagram of Hambantota Municipal Council waste dump site is depicted in Fig. 1. Old dump areas are indicated by 'A' and 'B'. The old waste dump area indicated by 'A' is about 13 years in age and was not selected for sampling as it is outside the fence. The old waste area indicated by 'B' is less than 10 years in age. The new waste area indicated by 'C' is less than 2 years in age. The area indicated by 'D' is allocated for future landfill and intact area 'E' is assumed to be uncontaminated with leachate. Samples were collected from trial pits at varying depths of 0.5m, 1.0m and 1.5m in waste (Fig. 1). In the parent stratum under the waste, soil samples were obtained at 0.5 m and 1.5 m depths starting from ground level.

As waste consists of particles like polythene and plastics it is difficult to obtain undisturbed well representative samples using boreholes and other conventional techniques. Thus, a large scale 300 mm × 300 mm × 300 mm box was used to extract samples. First ground surface was smoothed and the steel box driven in to waste. Then a trench around the box was carefully excavated and the sample was cut.

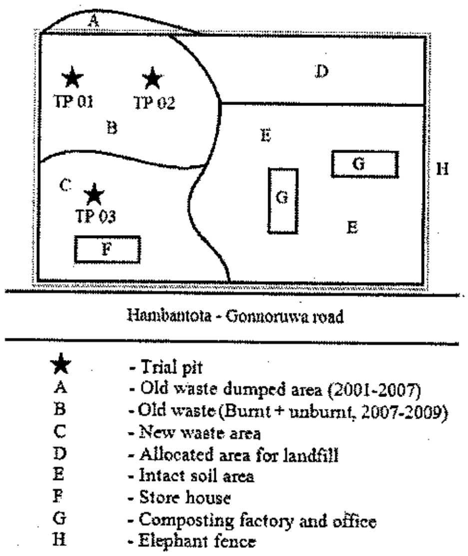

Fig. 1 Sampling points of dump site

3 LABORATORY TESTING

3.1 *Composition of waste*

The undisturbed MSW samples collected were used to conduct laboratory tests. Initially, waste composition was established for each waste sample location and bulk unit weights of the samples were calculated by measuring the weight of the samples obtained.

In this process, initially MSW components were sorted as bio-degradable and non bio-degradable by visual observation. The solid waste consisted of a higher percentage of fine particulate matter which was indistinguishable. Thus, in advance to sorting the solid waste sample was sieved through a 4.75 mm standard sieve to separate the fine component. A portion of intact samples from box sampling was oven dried at 60°C to determine the natural moisture content. Shown in Fig. 2 is the variation of composition of MSW with respect to location and age.

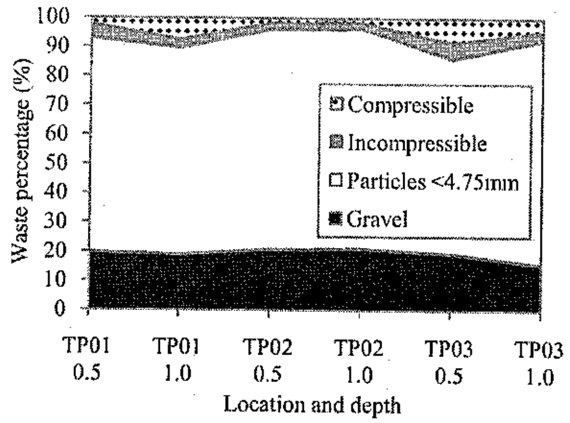

Fig. 2 Variation of MSW composition with location

3.2 *Direct shear test*

Shear strength parameters for MSW samples collected was conducted using a direct shear test apparatus. Because the Shear box had a diameter of 100 mm and a depth of 45 mm particles had to be shredded and recompacted in to the mould in order to be sheared. Similar methodology was adopted by Reddy et al. (2009) and Zekkos et al. (2010). Specimens were prepared by air drying the waste samples until a constant weight was observed and shredded into particle size no more than 4.75 mm. Air drying was adopted to prevent deformation of particles which would have resulted if oven drying was used. The dried waste samples were prepared with varying moisture contents.

All direct shear tests were conducted under drained condition to obtain effective shear strength parameters at vertical stresses of 25kPa, 50kPa and 75kPa. For locations where soil was present under waste layers, undisturbed samples were extruded from a circular cutter for testing. Where samples showed no peak shear stress, the stress at 15 % shear strain was obtained to produce the Mohr-Coulomb failure envelope. The concept of defining shear strength parameters at strain levels considered critical was further mentioned by Canizal et al. (2011) and Zekkos et al. (2010).

Fig.3 illustrates the failure envelope fluctuations for various samples tested and results were significantly larger than the values in literature (Dixon and Jones, 2005). The effective cohesion ranged from 14kPa to 74kPa whereas the friction angle fluctuated between 25° and 48°. This erratic behavior of samples results due to the heterogeneity of MSW and thus, conventional method in defining factor of safety for MSW landfill slope stability is not reliable and causes in false interpretation of results.

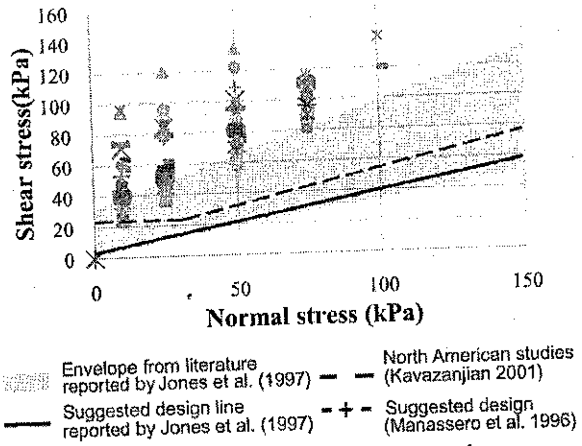

Fig. 3 Shear envelopes of tested MSW samples

4 SLOPE STABILITY ANALYSIS

4.1 *Reliability analysis*

The results obtained by laboratory testing was used to evaluate the slope stability of landfills in varying slopes of 1:0.5, 1:1, 1:2 and 1:3. Reliability Analysis via Response Surface Method proposed by Xu and Low (2006) for embankments was adopted for the purpose of evaluation. Finite Element Modeling was used to evaluate the slope stability of each design slope angle.

The probability of failure of an embankment is given by,

$$p_f = P[G(X) \leq 0] \quad (1)$$

Where ***G(X)***, the performance function defined by,

$$G(X) = F(X) - 1 \quad (2)$$

Where, ***F(X)*** is the function for the factor of safety. An approximate 2nd order polynomial function is defined for the performance function ***G(X)***. Thus, the probability of failure can be calculated when the Factor of Safety is equal to one. Reason for defining a second order polynomial function and the procedure for reliability analysis is further elaborated by Low (1998) and Xu and Low (2006). The second order polynomial function which defines the approximate performance function can be written as,

$$G^{'}(X) = l + \textstyle\sum_{i=1}^{r} m_i x_i \sum_{i=1}^{r} n_i x_i^2 \quad (3)$$

Where ***l***, m_i, n_i, are coefficients that need to be determined to define the performance function and x_i are the random variables incorporated into the simulated problem (cohesion, friction angle, density, etc.). If the number of random variables in a problem are n, then the required sampling points to determine the coefficients of the approximate performance function is given by (2n+1), indicating that (2n+1) simulations are required to be run in order to produce sufficient equations to be solved.

4.2 *Method of analysis*

First, the random variables concerned with the analysis of slope failure were identified. For the evaluation of slope stability of the landfill a two layered model was created using PLAXIS finite element modeling software. As indicated in Fig. 4 a waste layer of 5 m and a soil layer of 6m were considered. This selection was done based on site observations where the parent stratum was to be homogeneous to a depth of 6m and the waste layer with the current composition is not likely to reach beyond 5m in height. The cohesion, friction angle and the bulk density of each layer, altogether 6 parameters (Fig.4) were considered as random variables which govern the factor of safety. Thus, the performance function for the factor of safety consisted of 13 (2x6+1) terms which meant 13 unknown coefficients had to be determined to define the performance function ***G'(X)***.The input data set and the corresponding FOS obtained for each slope angle using PLAXIS was used to construct the response surface ***G'(X)***. Using the performance function defined the reliability index was calculated by using the First Order Reliability Method as explained by Low (1996).

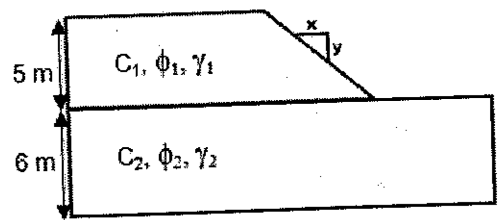

Fig. 4 Two layer slope model for landfill

4.3 *Reliability index and probability of failure*

The reliability index is defined as the minimum distance from the point of mean values of the random variables to the boundary of the limit state function in units of directional standard deviations. The reliability index β, is given by,

$$\beta = min_{X=F} \sqrt{\left(\frac{X_i - m_i}{\sigma_i}\right)^T (R)^{-1} \left(\frac{X_i - m_i}{\sigma_i}\right)} \quad (4)$$

Where, $\boldsymbol{R}$ = correlation matrix, m_i = mean of the random variable and σ_i=standard deviation of random variable x_i. The reliability index is calculated based on the limit state function, $G(X) = 0$. Microsoft Excel's built-in Solver optimization tool can be used to minimize the reliability index (Low, 1996). Based on the reliability index the probability of failure, P_f can be obtained by,

$$P_f = 1 - \sigma(\beta) \tag{5}$$

Where σ = Cumulative Distribution Function (CDF) of the standard normal variable. The failure probability used here means the probability that the performance function $G(X) = F(X) - 1$ is equal to 0.

Summarized in Table 1 are the values obtained for the reliability index and corresponding probability of failure for each slope angle. According to the results a clear trend can be observed where the failure probability is increased from 0.001% to 1.995% with the increasing slope angle.

Table 1. Results of reliability analysis

Slope	FOS mean	FOS Standard deviation	Reliability index, β	Failure probability, P_f(%)
1:3	5.67	0.9	4.27	0.001
1:2	4.64	0.64	4.19	0.004
1:1	3.45	0.47	2.14	1.626
1:0.5	2.71	0.46	2.05	1.995

5 CONCLUSIONS

Based on the results of the study, following conclusions can be made.

- Shear strength parameters showed erratic behaviour for samples tested while direct shear test results proved to be very high due to high content (about 60% by dry mass) of fine particles (<4.75 mm) and gravel (about 20% by dry mass)
- Slope stability of MSW landfills must be evaluated by adopting a probabilistic approach. Variation in shear strength parameters of the waste layer and uncertainties involved in conducting tests are not accounted for in conventional FOS methods. By using the reliability analysis via response surface method, the uncertainties involved with the spatial distribution of landfilled MSW shear strength parameters can be accounted for.
- The method can be extended for analysis in the wet zone of the country incorporating pore water pressure within the waste mass as another random variable, enabling the evaluation of stability of a solid waste slope under saturated conditions.

ACKNOWLEDGEMENT

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Shear Strength Characteristics of Waste of an Open Dump-site in Evaluating Stability of its Slope

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ABSTRACT: Open dumping is the most commonly used method adopted in Sri Lanka as solid waste management. However, slope failures of open dump sites lead to environmental pollution as most of the open dumps are located near water bodies. Therefore, analysing the stability of open dumps is important in implementing mitigatory measures where required. Abandoned Udapalatha open dump site which is located near Gampola, Sri Lanka was considered as a case study to analyse the stability of its slopes consisting of old and new waste representing different degrees of decomposition. Shear strength parameters of the waste samples of the old and new waste sites were determined using box samples at different depths with particle size less than 9.5 mm. Specific gravity test, Oedometer test and Standard Proctor compaction test were performed to obtain Gs, primary and secondary consolidation parameters, maximum dry density and the optimum moisture content. In addition, direct shear test was carried out to determine the shear strength parameters of the fill. Slope stability analysis was carried out using Slope/W and Plaxis-2D software considering Mohr Coulomb and soft soil creep models respectively for waste material. Consideration of primary and secondary consolidation settlement within the landfill in the Plaxis-2D analysis resulted in an increase in the Factor of safety (FOS). Therefore, FOS values obtained from the slope stability analysis of the old site, was higher than that in the new waste site.

1 INTRODUCTION

Municipal solid waste removal is a major problem all over the world. There are some methods available to remove solid wastes such as engineering landfill, open dumping, recycle and reuse. An open dumping is defined as a land disposal site at which solid wastes are disposed of in a manner that does not protect the environment and susceptible to open burning, and exposed to the elements, scavengers, etc. There were number of landfill failures occurred in world with high loss of lives and properties. If the open dump sites undergo failures they can fall down to nearby water bodies and make bad condition to them which are used to get drinking water. Therefore analysing stability of this landfill is very important. Research aim is to evaluate the stability of slope of old site with the spatial variation of cross section of an open dumpsite in Udapalatha, SriLanka.

2 LITERATURE REVIEW

Slope stability of a MSW landfill mainly depends on the geotechnical properties of MSW, such as unit weight and shear strength. Although it is common to perform stability analysis with uniformshear strength parameters for MSW, it should be noted that the MSW properties vary spatially due to heterogeneous nature, overburden pressure, and degradation. Thus, spatial and temporal variation in geotechnical properties of MSW should be properly considered in the landfill slope stability evaluations (Babu et al (2014)). There are various researches and tests done to obtain shear strength characteristics of municipal solid waste (Strak et al (2000); Dixon and Jones (2005); Zhan et al(2008); Reddy et al (2011); Canizal et al (2011); Jafari et al(2013)).They are laboratory tests, field measurements and back calculation of shear strength.

In laboratory testing methods, in most of the cases direct shear test, unconfined compression test and triaxial compression test are proposed to obtain shear strength parameters. Vane shear test, standard penetration test and cone penetration test are used as field testing methods and for back calculation analysis plate load test is also used (Dixon and Jones (2005)).However, it is common to use either direct shear test or triaxial test to determine shear strength parameters (Stark et al(2000)).

3 METHODOLOGY

The MSW samples were collected at Udapalatha using a hydraulically operated rotary drilling machine. Box samples were also obtained in the landfill site at various depths.MSW is a heterogeneous material containing paper, polythene, plastic, clothes, organic matters, food particles, etc. It is difficult to analyse the geotechnical properties of MSW which contains materials such as polythene, plastic, etc. Normally fibrous materials present in the waste give additional strength to MSW in a sloped landfill. Therefore, fibrous materials were removed from box samples which were obtained at 0.5m, 1.5m, and 2.5m depths below the subgrade under the dump site. The maximum particle size selected for laboratory testing was selected as 9.5 mm as larger size particles, if present would induce a scale effect on direct shear test specimens.

In the laboratory, several parameters were measured in order to characterize its stability of slope and spatial variation of consolidation and shear strength parameters. These parameters areeffective cohesion(c′), effective friction angle(ϕ'), Optimum moisture content (OMC), secondary consolidation parameters(λ^*-Modified compression index, K*-Modified swelling index, μ^*-Modified creep index) and unit weight(γ).

These parameters were evaluated by carrying out following tests;

1) Direct shear test-To determine c' &ϕ'
2) Consolidation test-To determine λ^*, K*, μ^*
3) Compaction test-To determine omc
4) Specific gravity test-To determine G_s

Slope stability analysis was done using different numerical software (Plaxis-2D and Slope/W) using different material models. Plaxis-2D with soft soil creep model and Slope/W with Mohr Coulomb model were used.Soft soil creep model is suitable for MSW materials because of its heterogeneity and the presence of more organic matters. This model tends to over predict the range of elastic soil behavior. Other material models do not take creep effects into account.

4 RESULTS AND DISCUSSION

Results were obtained from the Udapalatha old site samples. The optimum moisture content and the maximum dry density were obtained from Proctor compaction test.

Table 1. Maximum dry density and omc at different depths

Sample depth /m	0.5	1.5	2.5
Maximum dry density/ ρ_d (g/cm^3)	1.10	1.12	1.13
Optimum moisture content /w(%)	39.0	39.9	38.9

The secondary consolidation parameters were obtained from consolidation test. It was carried out in several load increments from 0.5 lbs to 64 lbs including the unloading stage. Fig. 1 shows the consolidation test results of 1.5m depth sample.

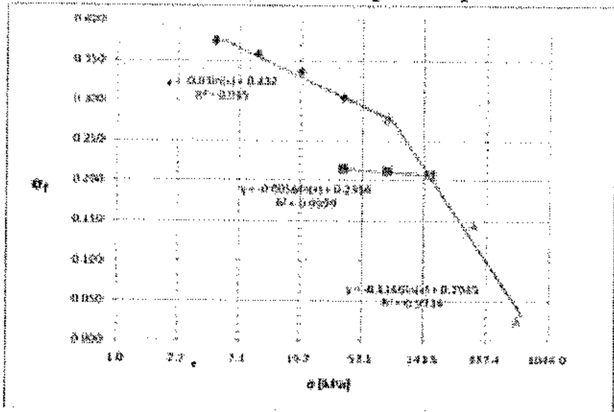


Fig. 1 Variation of void ratio with σ

The modified compression index (λ^* = 0.1166) was found from Figure 2 loading part and the modified swelling index (κ^*=0.0056) was found from the unloading curve. Fig. 2 shows the variation of void ratio with time from that graph.

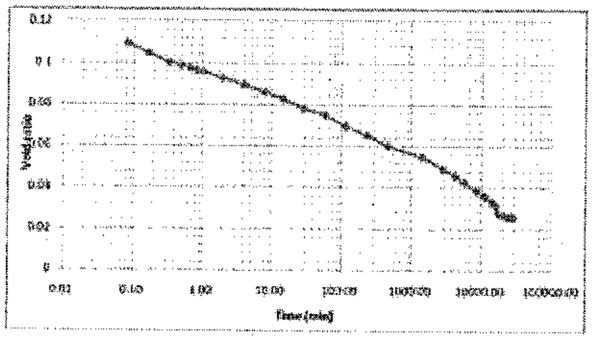


Fig. 2 Variation of Void ratio with time

Secondary compression index ($C\alpha$) was obtained using the void ratio- time relationship beyond the primary consolidation. $C\alpha$ was found to be 0.008 from Fig. 2. Modified creep index (μ^*=0.0024) was found using $C\alpha$ and initial void ratio (e). The shear strength parameters of the waste samples obtained at 0.5 m, 1.5 m and 2.5 m depths were obtained using direct shear test. Fig.3 shows the results of the direct shear test for sample obtained atdepths of 0.5 m. A displacement of 20mm was taken as ultimate strength and corresponding shear stress and normal stress were used to draw the graphs. The shear strength parameters obtained from the direct shear test are listed below in Table 2.

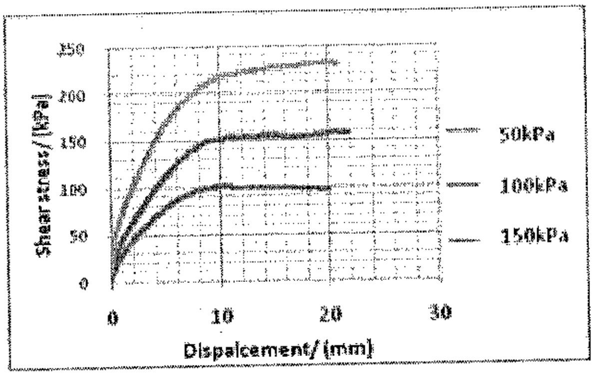


Fig. 3 Variation of shear stress with shear displacement for 0.5m depth sample

Based on the shear stress vs shear displacement curve, shear stress vs normal stress curves were prepared as shown in Fig.4 and the values for cohesion and angle of friction were found based on the graphs.

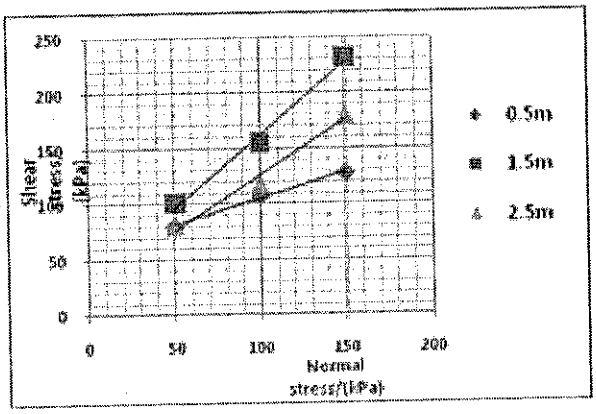


Fig. 4 Variation of shear stress with normal stress

The cohesion and angle of friction values obtained for each depth from the direct shear tests are given in Table 2.

Table 2. Results of direct shear test

Depth (m)	c′ (kPa)	ϕ′ (°)
0.5	57.9	25.1
1.5	30.9	52.8
2.5	26.9	44.7

Fig.5 and 6 show the variation of effective cohesion and the angle of friction with depth. The angle of friction obtained for the 1.5m depth sample is higher than that of the 2.5 m depth samples. This may be obtained due to the heterogeneity and decomposition with ageing.

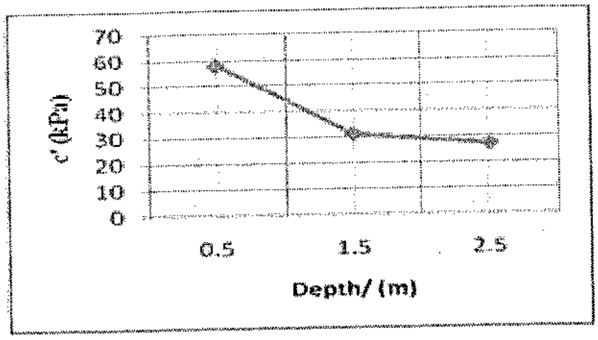


Fig. 5 Variation of effective cohesion with depth

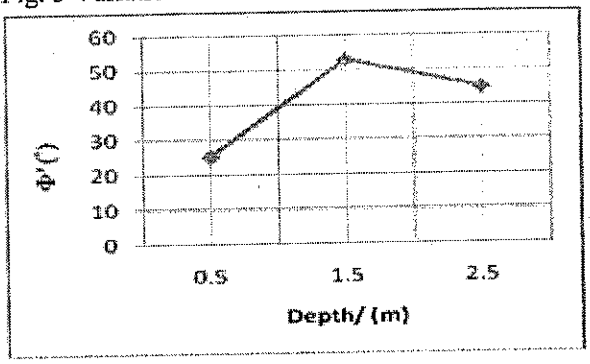
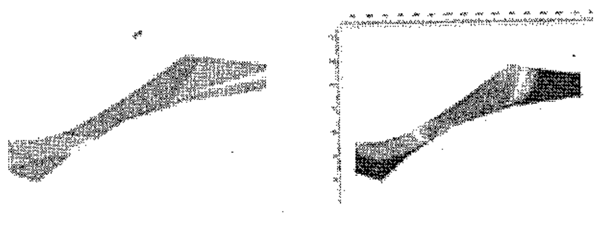


Fig. 6 Variation of friction angle with depth

The stability analyses were carried out using Plaxis-2D and Slope/W software. The FOS values obtained from both software are different from each other due to the use of different material models and the results are shown in Table 3. The critical failure surfaces are shown in Fig. 7. The watertable is well below the ground surface at the Udapalatha open dump site. Therefore, the stability analyses were carried out without considering the presence of watertable.

Table 3. FOS values obtained from different software (Old site)

Software	Depth (m)		
	0.5	1.5	2.5
Plaxis-2D	2.954	2.799	2.554
Slope/W	2.585	2.924	2.659



(a) (b)

Fig. 7 Failure surface obtained using properties at 0.5m depth (a) Slope/w (b)Plaxis-2D.

However, the possibility of rising of watertable to a reasonable height above the bottom level of the waste layer was also considered in the analysis. Table 4 shows the reduced FOS values obtained from Slope/W software under the above condition.

Table 4. FOS values obtained from Slope/W with watertable

Software	Depth (m)		
	0.5	1.5	2.5
Slope/W	2.384	2.698	2.452

The values of FOS given in Table 3 is compared with those given in Table 5 which gives the FOS values for the site with new waste analysed using the waste properties obtained from three different boreholes. The FOS values reported in Tables 3 and 5 reflect the effect of ageing on the stability of the slopes.

Table 5. FOS values obtained from the new site samplesafter, Prathapan et al (2015)

Borehole	Effective Cohesion (kPa)	Effective Friction Angle (°)	FOS	
			SLOPE /W	PLAXIS-2D
PBH1	15.4	31.7	1.184	1.139
BH02	46.7	19.8	1.594	1.382
PBH2	50.2	13.9	1.722	1.428

5 CONCLUSIONS

A research study was conducted to evaluate the stability of the slope of an open dump site located in Udapalatha, Sri Lanka. Results revealed that the stability of the slope is in safer range from the obtained results using the two software and the factor of safety values obtained using Slope/W is larger than the values obtained from Plaxis-2D. Presence of ground watertable within the landfill decreases the stability of the slope as expected. Consolidation settlement inside the landfill changes the FOS which is reflected in the Plaxis-2D analysis. Stability of the landfill is increased after the secondary consolidation. Based on the comparison of FOS values of the site with old waste with that of the site with new waste, it can be concluded that the site with old waste is safer which is due to the ageing effect.

ACKNOWLEDGMENTS

SATREPS project is gratefully acknowledged.

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Desiccation Cracking Behaviour of Landfill Clay Liner Materials in Dry Zone

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ABSTRACT: It is a common natural phenomenon that the formation of shrinkage cracks in landfill clay liners in dry zone of Sri Lanka and it leads to infiltrate of leachate into ground during the rainy season. Therefore, in this research study, laboratory desiccation plate tests were conducted to evaluate the shrinkage behaviour of expansive soil available in dry zone of Sri Lanka, which is used to develop compacted clay liners. Digital image processing technique has been used to determine the Crack Intensity Factor (CIF), which is the ratio of crack area to total surface area. The results show that higher desiccation rate was observed for smaller thickness of soil specimen and higher CIF was recorded for the bentonite and oleic acid amended soil. In addition, shrinkage cracks can be significantly controlled by amending soil with coconut coir fibers.

1 INTRODUCTION

Solid waste, especially Municipal Solid Waste (MSW), is a growing problem in urban areas of Sri Lanka and management of waste, both liquid and solid has become a critical environmental concern. The absence of engineered methods for disposing waste and the open dump approach adapted has created major environmental and social problem of waste within most of the cities. Especially, the attention given to the solid waste management in dry zone of the country is very less due to the fact that almost all major cities in Sri Lanka are situated in wet zone. However, solid waste management in dry zone is very important as people in this area depend very much on ground water for their drinking purposes and therefore, the contamination of ground water especially by the leachate generated in waste disposal sites should be kept at a minimum by following engineered waste disposal methodologies.

Engineered landfilling is one of the best options to overcome the problems associated with contamination of ground water with leachate. The liner system in an engineered landfill acts as a barrier for leachate and prevents the transportation of contaminants to the surrounding pollution prone environment. Hence liner system in a landfill becomes one of the critical design considerations. A landfill liner is intended to be a low permeable barrier which is generally involves with the application of clay or synthetic material layer. Since, synthetic materials are very expensive, Compacted Clay Liners (CCL) is the most suitable liner system for developing countries (Priyankara et al, 2013).

However, it was realized that a lot of cracks have been developed during the dry season in most of the clay liners in dry zone of Sri Lanka. This is basically due to the shrinkage behaviour of landfill liner material, which leads to infiltration of leachate into the ground during the rainy season. Thus, there is a possibility of contamination of ground water in this area due to development of shrinkage cracks in compacted clay liners.

Shrinkage cracking in soil is a complex phenomenon. It is a natural process involving weathering, chemical and biological changes. Occurrence of shrinkage cracks may due to several factors, such as clay content, mineralogy, soil thickness, surface configuration, rate of drying and total drying time etc (Alvis and Marcelo, 2011). And the primary requirement for the formation of desiccation cracks is restrained shrinkage leading to tensile stress development exceeding the tensile strength of soil (Kodikara and Costa, 2013). Shrinkage cracks significantly affect the soil performance. Cracks create zones of weakness in soil and reduce its overall strength and stability.

Even though a lot of shrinkage cracks have been recorded in lándfill clay liners in dry zone of Sri Lanka, so far no research has been conducted in order to control the shrinkage cracks. As such, an attempt was made in this research study to study the shrinkage behaviour of clay which is commonly available in dry zone of Sri Lanka. Laboratory desiccation plate test was used to study the shrinkage behaviour of clay and by applying digital image processing technique, geometric and kinematic characteristics of the surface crack patterns were described quantitatively.

2 MATERIALS AND METHODOLOGY

2.1 *Materials*

Clay soil samples were collected from Buttala area in order to study the shrinkage behaviour of clay available in dry zone of Sri Lanka. The physical properties of the soil are presented in Table 1. It was found that, soil is medium swelling potential material and rich of illite mineral, which makes the soil highly attracted to absorb water causing expansion of the material. These kinds of soil develop significant change of volume during drying due the evaporation of absorbed water.

Table 1. Physical properties of soil

Property	Value
Liquid Limit (LL)	35 %
Plastic Limit (PL)	20 %
Plasticity Index (PI)	15 %
Linear Shrinkage (LS)	3.9 %
Sand Content	58 %
Silt Content	14 %
Clay-content	28 %
Maximum Dry Unit Weight	17.5 kN/m^3
Optimum Moisture Content	15.5 %
Specific Gravity	2.56

2.2 *Desiccation plate test*

5 mm, 10 mm, 20 mm and 50 mm thick, four number of circular desiccation moulds with diameter of 20 cm were prepared using Perspex sheets. The bases have been grooved in circular shape to avoid soil sliding at the base. Initially soil has been air dried and sieved through 0.425 mm sieve to prepare the sample. The test sample was prepared by mixing sieved soil with water in such a way that initial water content of the soil is slightly higher than the liquid limit of the soil. Then samples have been manually placed into the moulds using a spatula by ensuring that air is not entrapped within the soil layers. Then samples were allowed to air-dry under room temperature. The weight of the samples was recorded at predefined intervals (1 hourly). At the same time, high resolution digital camera has been mounted at top of the setup to capture pictures at predefined intervals (1 hourly). Hence, crack initiation time, crack propagation information and crack pattern were investigated.

Priyankara et al (2013) were found out that engineering properties of soil can be improved by the addition of commercially available bentonite to build a low hydraulic conductivity barrier as liner in engineering landfills. Therefore, a series of tests have been carried out with unamended soil and soil amended with 5 % and 10 % of bentonite in order to study the shrinkage behaviour of proposed liner materials with different bentonite percentages. Furthermore, the effects of oleic acid and coconut coir fibers on the control of shrinkage behaviour were studied using desiccation plate tests. The proposed soil and bentonite amended soils were mixed with oleic acid (1g of oleic acid per 1kg of soil) and with coconut coir fibers (5% by volume) in this research study.

3 RESULTS AND DISCUSSION

3.1 *Soil-water evaporation process*

The variations of moisture content of soil with time for different sample thicknesses are presented in Fig. 1, in order to study the soil-water evaporation process. The experimental results indicate that all samples have similar initial moisture content, which is slightly above the liquid limit of the soil. It can be seen that moisture content has been decreased with time irrespective sample thickness. A significant reduction of moisture content can be observed in thinner samples (5 mm and 10 mm). Further, it can be noticed that thinner samples have been reached to constant moisture content with time within shorter time period and thicker sample takes longer time period to reach the constant moisture content. The 5 mm thick sample has been reached to constant moisture content before that of other samples. This behaviour can be further explained using soil-water evaporation rate.

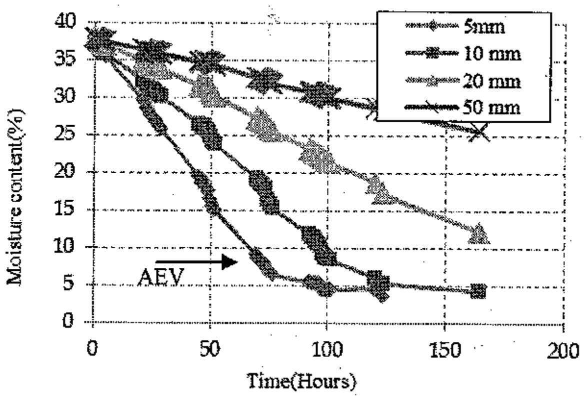

Fig. 1 Variation of moisture content with time with respect to different sample thicknesses

The variation of soil-water evaporation rate with time for different sample thicknesses are illustrated in Fig. 2. Based on the data presented, two evaporation stages can be easily identified irrespective of sample thickness, namely;

(a) Initial constant evaporation stage
(b) Falling evaporation stage

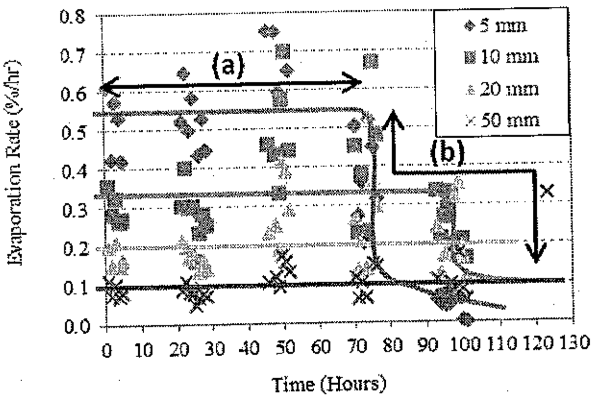

Fig. 2 Variation of evaporation rate with time

This evaporation behaviour can be explained with respect to the heat and mass flow between soil and air (Tang et al, 2011). Based on the test results as shown in Fig. 2, it can be noted that thinner sample has a higher soil-water evaporation rate than that of others. The moisture in the deeply seated pores can be easily drawn to surface when the sample is thinner; thus higher evaporation rate can be expected. In addition, soil-water content has been gradually diminished with time (Fig. 1) and soil suction has been increased under constant evaporation rate. The constant evaporation rate period has come to an end once the Air Entry Value (AEV) is reached. It can be seen that all samples have the same AEV (9%) irrespective of the sample thickness as AEV is a property of the material. However, when the sample is thicker, it takes longer time to reach to the AEV. At this point, air starts to penetrate into the soil pore spaces due to increase of suction, as a result soil transition from a saturated state to an unsaturated state.

3.2 *Quantitative analysis of shrinkage cracks*

In order to quantitatively evaluate the development of shrinkage cracks during drying, a term Crack Intensity Factor (CIF) was defined, which is the ratio of crack area to total surface area. The variation of CIF with time for unamended soil is depicted in Fig. 3. It can be seen that CIF has been increased with time in thinner samples. However, there was no shrinkage cracks have been developed in 20 mm and 50 mm thick samples during the test period. It can be concluded that development of shrinkage cracks are highly depended on the sample thickness based on these observations.

In order to study the effect of moisture content on development of shrinkage cracks, CIF has been plotted against moisture content as shown in Fig. 4. The figure clearly illustrated that CIF has been increased with the decrease of moisture content irrespective of sample thickness. It can be noted that under particular moisture content, CIF is same for both 5 mm and 10 mm samples. This implies that even though thinner samples have higher desiccation rate, crack area is directly proportional to the moisture content of soil. Based on these observations it can be concluded that cracks have been developed irrespective of the Compacted Clay Liner (CCL) thickness. However, if the CCL is thicker, the crack development rate is less. Also by controlling the moisture present in the soil, crack development rate can be controlled.

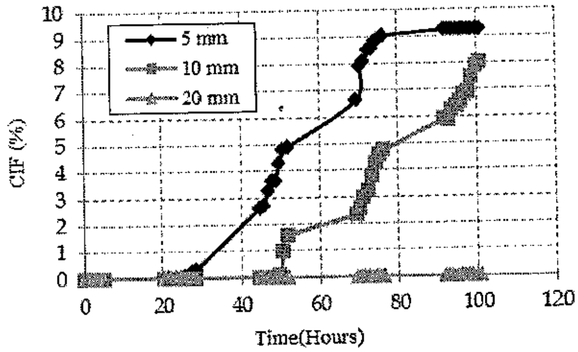

Fig. 3 Variation of CIF with time for unamended soil

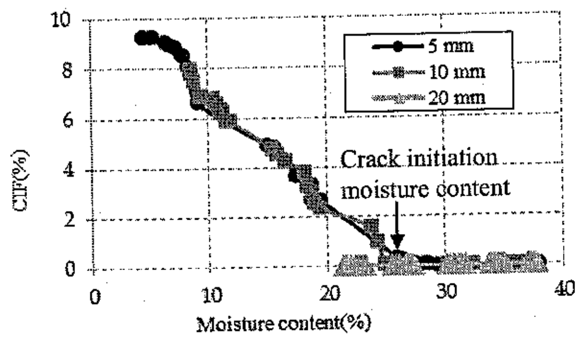

Fig. 4 Variation of CIF with moisture content

3.3 *Effect of Bentonite on shrinkage behaviour*

Since bentonite has been successfully utilized to improve hydraulic characteristics of CCL material (Priyankara et al, 2013), it is very important to investigate the effect of bentonite on shrinkage behaviour of clay liner material. The variation of CIF with time for 5 mm thick sample is presented in Fig. 5. It can be seen that CIF has been increased with increasing bentonite content. With the increase of bentonite, which mainly consists of montmorillonite mineral, the diffused double layers surrounding the clay particles are getting thicker. The diffused double layers create repulsive forces along the sides of the clay particles making it difficult for individual clay particles to stay closer to each other. As a result, cracks can be developed.

Further, based on image analysis, it was noted that when a new crack is propagating close to an existing crack, the new crack is attracted by the existing crack towards it, and new crack is at right angle to the existing crack. As a result, the final crack pattern is mostly square shaped clods.

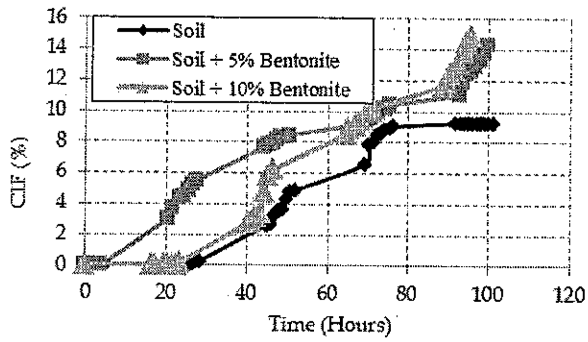

Fig. 5 Variation of CIF with bentonite content

3.4 *Effect of Oleic acid and Coconut coir fibres on shrinkage behaviour*

In order to reduce the shrinkage behaviour of clay liner material, oleic acid and coconut coir fibres were used in this research study. Oleic acid is a product of olive oil, and which is an environmental friendly product. The effect of oleic acid and coconut coir fibres on shrinkage behaviour is presented in Fig. 6. Even though oleic acid was added to control the shrinkage behaviour, it can be seen that with the addition of oleic acid, shrinkage behaviour of soil has been significantly increased. However, it is very clear that development of shrinkage cracks have been significantly reduced with the addition of coconut coir fibres. Coir fibres act as reinforcements and resist the tensile stresses to develop during soil-water drying process, as a result crack formation can be reduced. In other words, due to inclusion of coconut coir fibres into soil, plasticity characteristics of soil gets reduced controlling the volume change behaviour of soil.

4 CONCLUSIONS

Based on laboratory experimental results, it can be concluded that, the soil-water evaporation process composed of two stages namely initial constant evaporation stage and falling evaporation stage. Thinner samples have higher soil-water evaporation rate than that of others. During soil-water evaporation process, the constant evaporation rate period comes to an end once the moisture content of the soil reaches to AEV of the soil, at which soil transition from saturate state to unsaturated state. All samples have the same AEV irrespective of the sample thickness. However, when the sample is thicker, it takes longer time to reach to the AEV.

Desiccation cracking highly depends on the moisture presence in the soil. Formation of desiccation cracking is increased with the decrease of moisture content irrespective of sample thickness. This indicates that development of desiccation cracks in CCL can be minimized by increasing the liner thickness. In addition, coconut coir fibres can be effectively used to control the desiccation cracking behaviour of CCL.

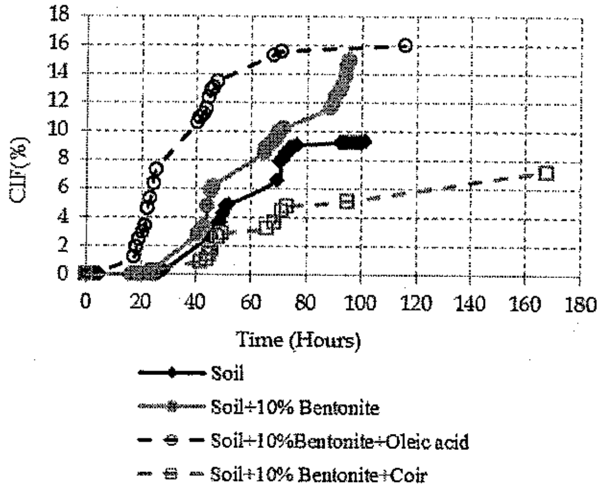

Fig. 6 Effect of oleic acid and coir on CIF

ACKNOWLEDGEMENT

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Identification of Local Soils for Development of Cricket Pitches

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ABSTRACT: Most of the Cricket batsmen in Indian subcontinent face a great difficulty in batting against fast bowlers on English and Australian fast and bouncy wickets. The lack of having fast and bouncy pitches in Indian subcontinent has led to this problem. It had been discovered that the pace and bounce of a cricket pitch is governed by clay content, clay mineralogy and grass content of the cricket pitch. Local pitches were found to have high silt content, low clay content and low plasticity due to a difference in clay mineralogy. In this research "Grumusol" clay was discovered in Murunkan with a comparatively high clay content which is fulfilling most of the required clay properties of a fast and bouncy wicket. Typical Sri Lankan clay sample used to prepare wickets, Clay sample from Bangalore and the local "Grumusol" sample from Murunkan were tested for the clay properties. Results showed that "Grumusol" is most suitable for constructing a fast and bouncy wicket. Other than the clay as the basic material grass plays a vital role in binding the basic material together and maintaining the required moisture levels in the wicket preventing the wicket from fracturing during the game period. Here, Crouch grass was selected as the most suitable type of grass to achieve the desired output of the fast and bouncy wicket.

1 INTRODUCTION

The sport Cricket has conquered every nook and corner of the world continuing its legacy on different pitches in different countries situated at different subcontinents. In this context players encounter different pitches on which their performances are questioned.

Cricket pitches have been categorized according to their ball behaviour. Players have learnt to adapt to how the pitches behave in order to perform better. Of these general categories of pitches, the most common are the "fast" and "slow" pitches (James, et al., 2005). "Fast" pitches quite commonly are "Bouncy" pitches as well while "slow" pitches are "low and spinning" pitches. Therefore the pitches are of two kinds which are known as fast and bouncy pitches, Slow and Low pitches (Nawagamuwa, et al., 2009).

All Asian countries have comparatively slow and low pitches when compared to the pitches in other subcontinents. Australia and England are popular for having fast and bouncy pitches (Carre, et al., 1999) where Asian batsmen struggle to score big numbers and the fast bowlers dominate the game.

Sri Lanka as well as most Asian countries are trying to create fast pitches in their home lands in order to enhance the player performance. Reasons for the lack of 'Pace and Bounce' in Sri Lankan pitches has been a topic of debate for a long time in the cricket arena (Nawagamuwa, et al., 2009).

To create fast pitches it is required to either import clay from foreign country which is a costly operation or to find a locally available clay which has the ideal characteristics. This research addresses the problem directly and has come up with some valuable results in order to make fast and bouncy pitches using local soil.

2 OBJECTIVES

Objective of this research project is to identify locally available soils that have the soil properties of a fast and bouncy wicket and a grass type to support its playing characteristics.

3 METHODOLOGY

Following steps were followed in this research to reach objectives.

- Study the background of the research project through literature review
- Selection of local soils that are likely having the soil property requirements and soils, which

are currently being used in Sri Lanka and India to prepare cricket wickets.

- Sample collection for laboratory experiments
- Experimental investigations to find the soil properties of the collected samples
- Analysis of results obtained from tests

4 EXPERIMENTAL WORK

A series of experiments was conducted according to appropriate guidelines such as BS 1377 and ASTM volume 04.08. Soil classification of three samples was done by performing particle size distribution using wet sieve analysis and Hydrometer test, Atterberg limit test, specific gravity test and Ash content test.

Particle size distribution and Atterberg limit tests were done in order to classify the selected soil samples. Rather than using the ordinary sieve analysis here a wet sieve analysis is done because the soil is nearly clay.

The primary objective of wet sieve analysis test was to separate the particles greater than 0.075mm and doing the sieve analysis for the greater size particles.

Liquid limits, Plastic limits and Plasticity Index were found by Atterberg limit tests.

Ash content test was used to determine the organic matter content of the selected clayey soil samples. Classification of the soils was done using the Unified Soil Classification Method.

5 RESULTS AND DISSCUSSION

5.1 *Particle size distribution*

Particle size distribution shows the amount of finer particle (which are usually clay and silt) content of a soil and the soil grading. This can be determined by the sieve analysis test and Hydrometer test. There are four methods to analyze the particle size distribution. They are air dry-dry sieve, air dry - wet sieve, oven dry-dry sieve and oven dry-wet sieve. When the particles are bonded together, it will become a larger size particle. Therefore, dry sieve may not provide accurate results in this situation where the accuracy should be high in finding the clay content of these soils since the clay content will be the key factor to produce the pace and bounce of a wicket. So wet sieve analysis gave better results for the particle size distribution above 0.075mm range.

Hydrometer test gives a better picture of the finer content, which are below 0.075mm. Combining both test results, the ultimate particle size distribution graphs were drawn.

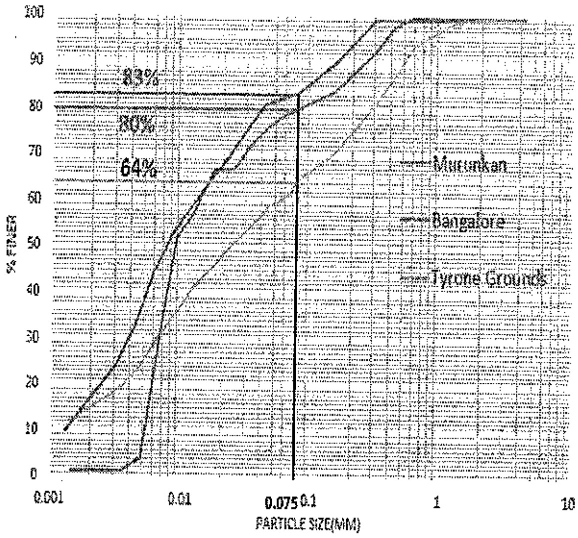


Fig. 1 Comparison between Particle size distributions

From the results of particle size distribution it is obvious that Bangalore soil sample has 83% finer particles. This implies that the Bangalore soil sample is rich with finer particles than the "Grumusol" sample and the sample from Tyrone Fernando Stadium. "Grumusol" sample and the Sample from Tyrone Fernando Stadium have 80% and 64% respectively. Since the Bangalore sample was taken from a developing fast pitch it may have much finer particles. However, the "Grumusol" soil has little lower value than the Bangalore sample (value of 80%) which can be identified as a soil of similar high clay and silt content. In contrast the currently used soil in SL pitches gives 64% which is comparatively a lower value for the finer particles, confirming high sand and organic content in that soil. These components definitely lead to the unexpected breaking of the wicket surface (Wicket may turn in to an uneven surface including coverts and dust). Moreover the friction development by sand reduces the speed as well as the bounce of the ball. This scenario makes SL pitches slow and low when compared to fast and bouncy Australian pitches which have much low sand content (6%) (Nawagamuwa, et al., 2009). In addition when the soil grading of 3 samples are considered, "Grumusol" is more poorly graded at finer particle range. Poorly graded soil may lead to less binding. Therefore, it must be prevented by adding other soils to fill in the gap ranges along with a suitable grass.

5.2 *Atterberg Limit Tests*

Primary objective of the Atterberg limits test is to find the Plastic and liquid limits and the find the plasticity index. The test was done using Casagrende's instrument.

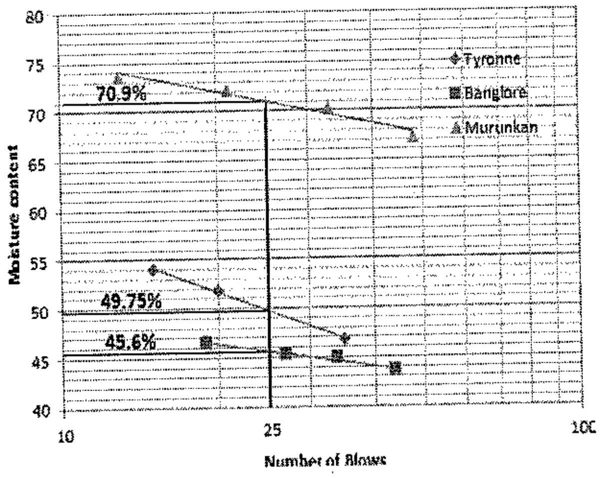


Fig. 2 Liquid limit plot

Atterberg test results shows that the "Grumusol" sample taken from Murunkan gives comparatively higher liquid limit (70.94%) and higher Plastic Index (42.8%) which indicates that it consists of a comparatively higher clay content than the designed fast pitch in Bangalore. In contrast the prevailing soil in SL pitches gives low liquid limit and lower Plastic Index which indicates the lower clay content which again gives the reason of being slow and low.

Usually Australian fast pitches have a clay content of 51%- 82% which is a higher value. Therefore this test results indicates that the "Gumusol" sample from Murunkan matches with the clay requirement of a fast pitch.

Table 1. Plastic and Liquid limits

Soil Sample	LL%	PL%	PI%
Tyrone Grounds	49.75	30.48	18.27
Bangalore	45.6	18.6	27
Murunkan	70.94	28.14	42.8

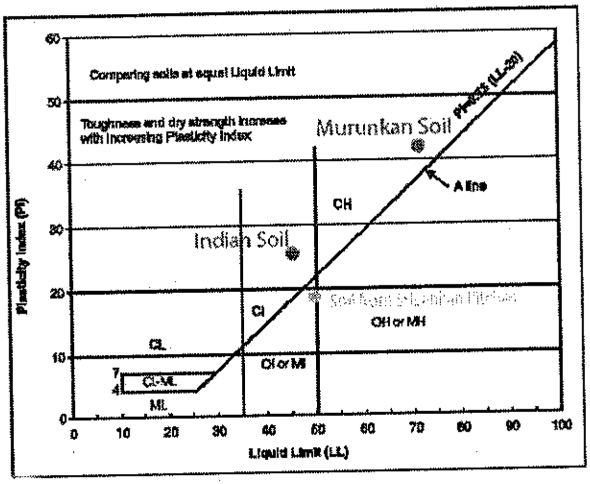
5.3 *Specific Gravity Test*

Specific gravity value of sample was determined according to BS 1377: part 1, Pycnometer method.

Table 2 Specific Gravity of the samples

Soil Sample	Specific Gravity (G)
Tyrone Grounds	2.35
Bangalore	2.53
Murunkan	2.52

5.4 *USCS Classification*



PLASTICITY CHART FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS.

Fig. 3 USCS Classification Chat

USCS classification clearly separates 3 samples into 3 categories. Sample taken from Tyrone Fernando Stadium is in OH/ MH range which are Inorganic silts, fine sandy, silty soil or elastic silts. This leads to the untimely cracking of the Sri Lankan pitches into dust.

Bangalore sample is in CL range which is inorganic clays with low to medium plasticity, and it meets the required standards to some extent.

"Grumusol" sample falls in CH range which is inorganic clays of high plasticity, Fat clays. Then, "Gumusol" sample from Murunkan corresponds to the clay type of a fast pitch.

5.5 *Selection of Grass*

Warm-season grasses such as couch grass (Cynodon dactylon) and kikuyu grass (Pennisetum clandestinum) are more efficient in using water and have better drought tolerance than the cool season grasses such as ryegrass (Lolium perenne), fescue (Festuca sp.) A saving of approximately 40 to 50% in water use can be achieved with warm-season grasses compared to cool-season grasses (Michael, 2012). Therefore variety of couch grass types will be suitable for hot and humid conditions in Sri Lanka.

Another advantage of warm-season grasses is that without an adequate supply of water (e.g.under water restrictions), sports surfaces consisting predominantly of cool-season grasses will lose cover, become tufty, have excessive hardness and often become unsafe. Warm-season grasses have a greater capacity to maintain good density and have a creeping growth habit that is less likely to become tufty during periods of extreme heat and low rainfall (Michael, 2012). Therefore warm season grass will be ideal for an equivalent situation of 5 day test cricket match without any water throughout the game.

"Santa Ana" which is a variety of couch grass and it can be identified as the most suitable grass for Sri Lankan cricket wickets considering its Quality, Colour retention/Dormancy, Cover under wear and Cover Density. For further proceedings of this research a locally available grass having same properties of "Santa Ana" should be identified.

6 CONCLUDING REMARKS

Cricket is an unpredictable game which can turn its momentum completely upside down in several seconds. Though the main role is acted by the bat and the ball the pitch/wicket plays a huge hidden role which is a key factor of deciding the day's champion team.

Due to the soil properties of the wicket, the playing characteristics of one pitch differs from others.

Australian and England pitches are considered as "Fast and Bouncy" pitches while the Indian subcontinent pitches are considered as "Slow and Low" pitches. This nature is inherited by a pitch due to aforementioned soil properties of wicket itself.

Since the importing of soil is a costly operation, finding a reliable source which meets the requirement of a soil in fast and bouncy wicket is a great advantage for a developing country like Sri Lanka.

In this research project three samples were tested for the soil properties. The "Grumusol" sample from Murunkan shows its higher potential for a fast pitch soil, while the soil from Bangalore meets the requirement to some extent. In contrast the currently used soil in Sri Lanka has higher silt, sand and organic impurities which is the reason of being slow and low.

Moreover, to keep the clay particles together while maintaining the strong bond and reducing the crack openings in extreme hot temperature and humid conditions *Cynodon dactylon* (Crouch grass) is ideal (Michael, 2012). This situation could be considered as equivalent to Sri Lanka when considering the tropical hot weather conditions.

"Grumusol" will be the solution for the local soil requirement in order to develop a fast and bouncy wicket in Sri Lanka.

Further model testing should be carried out in order to find the friction and the bounce generated by pure/mixed clay samples having almost the same soil properties.

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Tunnel Route & Tunneling Method Selection for Broadlands Hydropower Project

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ABSTRACT: The purpose of this study was to select the optimal tunnel route and tunneling method for the head raise tunnel for Broadlands Hydropower Project (BHP) based on geological, geotechnical and economical aspects, with the help of a multiple factor decision making approach (Analytical Hierarchy Process (AHP)). Engineering geological assessments were performed along two proposed tunnel routes using borehole data, feasibility reports and outcrop data. Two rock mass classification systems (Q & RMR) were adapted to classify surrounding rock masses. The overall Q and RMR values revealed that both the rock masses are of poor quality. The results of AHP revealed that the route B is the optimal tunnel route and combination of Drill & Blast and New Austrian Tunneling Method is the most appropriate tunneling criteria along the selected tunnel route.

1 INTRODUCTION

1.1 *The background of the BHP*

The Broadlands Hydropower Project (BHP) is one of the major ongoing construction projects in Sri Lanka, which is located in the central highlands; approximately ninety kilometers east of Colombo city. This will be the last hydropower project of the Kelani River basin.

This study is focused on a part of the tunnel route which connects the intake and the surge chamber. This part of the tunnel has two proposed route alternatives, Route A and Route B. The lengths are 1.2 km and 1.5 km respectively.

1.2 *Aim*

This study was aimed to select the most suitable tunnel route and tunneling method for BHP, based on area geological and geotechnical conditions as well as economical aspects.

1.3 *Objectives*

The main objectives were to recommend the most suitable tunnel route and tunneling method using Analytical Hierarchy Process (AHP). A comprehensive engineering geological study and a cost analysis for the proposed tunnel routes had to be performed additionally align with above objectives.

2 METHODOLOGY

2.1 *Data gathering*

The Information on existing rock types, occurrence of deferent rock strata, details of existing structural geological features of the area and various topographical features were recorded during data gathering. Rock outcrop and the borehole data were the major data sources.

2.2 *Engineering geological assessment*

An Engineering Geological Assessment has been carried out to study the geology of two tunnel routes. For this purpose, preparation of an engineering geological map of the site and development of longitudinal sections along two tunnel routs were carried out. Figs. 2, 3 and 4 present the results of the engineering geological assessment.

2.3 *Rock mass classification*

The rock mass classification was performed along two tunnel routes using RMR and Q methods to examine the quality of the surrounding rock masses in a systematic manner and to classify rock masses into different rock classes. The results are summarized in Tables 1, 2, 3 and 4.

2.4 *Rock wedge stability analysis*

When carry out underground excavations in hard rock, failures frequently occur due to presence of discontinuities such as faults, shear zones, bedding planes and joints. In this regards, the Stereonet analysis, which is a graphical technique, was performed considering major joint sets in the area. Fig. 7 presents the results of the rock wedge stability analysis.

2.5 *Cost analysis*

The tunnel construction cost per meter run of the two route alternatives were evaluated for the purpose of tunnel construction cost comparison. The construction cost includes excavation and supporting cost. The rates were obtained from Kukuleganga and Upper Kothmale Hydropower Projects in Sri Lanka. The results are summarized in Table 5.

2.6 *Analytical Hierarchy Process (AHP)*

There are many factors involved in tunnel route and tunneling method selection. Therefore, a multiple factor decision making approach was needed. In this regards the AHP method, which was introduced by Prof. Thomas L. Satty in 1980 was employed.

In selecting optimal tunnel route, the identified main criterion were, surrounding rock type, rock slope stability, tunnel overburden, tunnel geometry, geomorphology, tunnel construction safety, socio-environmental impacts, cost (drilling, supporting), construction duration and length of the tunnel route. The alternatives were Route A and Route B.

In selecting suitable tunneling method, the identified factors were rock mass properties, tunnel geometry, tunnel overburden, ability of mechanization, ground water level and expected water inflow, tunnel construction safety, socio-environmental impacts, surface disturbances, time/cost considerations and local experience. These factors were evaluated over three alternative tunneling methods namely drill and blast, (D & B) cut and cover (C & C) and New Austrian Tunneling Method (NATM). The Tunnel Boring Machine (TBM) was not considered since it is uneconomical for short length tunnels.

To identify the main criterion to be incorporated the AHP model, the importance of above mentioned factors has to be assessed. For this a survey was conducted among 23 experts from different functional levels in local tunneling industry.

For the assessment, a questionnaire with a five-point scale for each of the criterion had been prepared. The respondents were asked to rate each factor according to five-point scale, based on the priority that should be placed for each factor.

A weight had been assigned to each alternative through a pair wise comparison based on the studied geological, geotechnical and relevant conditions. The pair wise comparison matrices were made with the aid of the scale of relative importance defined by Satty, 2008. The relative priorities of alternatives were obtained by this comparison and hence the relative suitability of the available alternatives to serve the objective was defined. The results are summarized in Tables 1, 2, 3 and 4.

2.7 *Specimen calculation*

Step 1:

Pair wise comparison of route A and B against "Surrounding rock type"

Factor	Factor weighting score			Factor
	More importance than B	Equal	Less importance than A	
Route A	9 7 5 3	1	3 5 7 9	*Route B*

According to rock mass classification, the average importance of route A than route B in terms RMR and Q values is nearly 25%. It is almost 1/4th of the scale. Therefore, surrounding rock type of route A is weakly important than route B. Referring to the table of relative importance defined by Satty, the intensity of importance is 3. This is marked on the Satty's scale of relative importance as follows.

Step 2:

Build up pair wise comparison matrix

According to the rules of AHP, the built up pair wise comparison matrix is as follows.

Normalized principal eigen vector (also called priority vector) (w) is given by;

Surrounding rock type	Route A	Route B
Route A	1	3
Route B	1/3	1

$$w = \begin{bmatrix} 0.75 \\ 0.25 \end{bmatrix}$$

The priority vector provides relative weights of the two route alternatives against "surrounding rock type". The weight of Route A is 75% and the weight of route B is 25%.

Step 3:

Check for consistency

When many pair wise comparisons are performed, some inconsistencies may typically arise. Therefore consistency of judgment is checked as follows.

$$CI = \frac{\lambda max - n}{n - 1} \tag{1}$$

Where ***CI*** is the consistency index, ***n*** is the size of the matrix and λ_{max} is the principal eigen value.

For this specimen,

$n = 2$

$\lambda_{max} = 4/3 \times 0.75 + 4 \times 0.25 = 2.0$

Hence, CI = (2 – 2) / (2 – 1) = 0

According to AHP theory, if $\lambda_{max} = n$, then the judgments have turned out to be consistent.

The composite weight of each alternative against all selected criterion can be determined using the decision matrix. Hence the relative suitability of the available alternatives is defined to serve for the objective. The obtained composite weights are summarized in Tables 6 and 7.

3 RESULTS

3.1 *Engineering geological map*

Using boreholes and outcrop data, geological map of the area was prepared with the help of regional topography maps and satellite photo images.

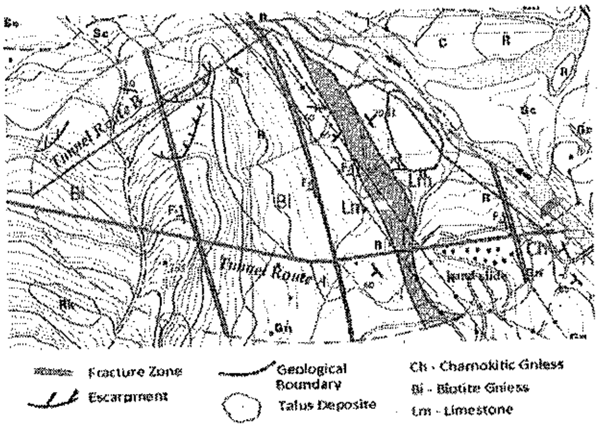


Fig. 2 Engineering geological map

3.2 *Cross sectional profiles along tunnel routes*

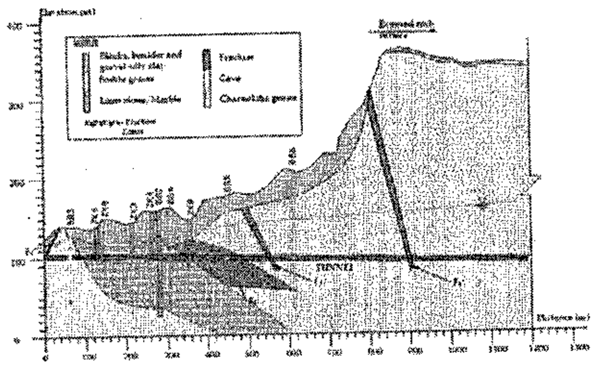


Fig. 3 Cross sectional profile along route A

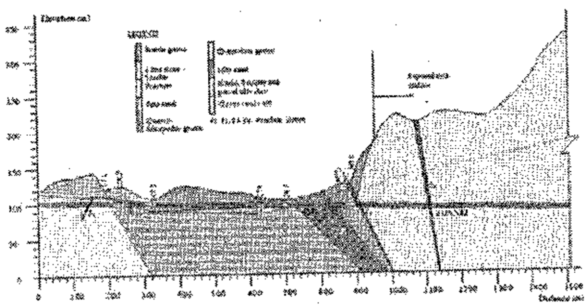


Fig. 4 Cross sectional profile along route B

3.3 *Rock mass classification*

Table 1. RMR and rock mass quality along Route A

Tunnel section (m)	RMR	Rock mass quality
0-8	44	Fair
8-323	35	Poor
323-508	17	Very Poor
508-564	29	Poor
564-1200	43	Fair
Overall	**37.5**	**Poor**

Table 2. RMR and rock mass quality along Route A

Tunnel section (m)	RMR	Rock mass quality
0-40	41	Fair
40-360	42	Fair
360-860	-1	Very poor
860-1500	43	Fair
Overall	**29.7**	**Poor**

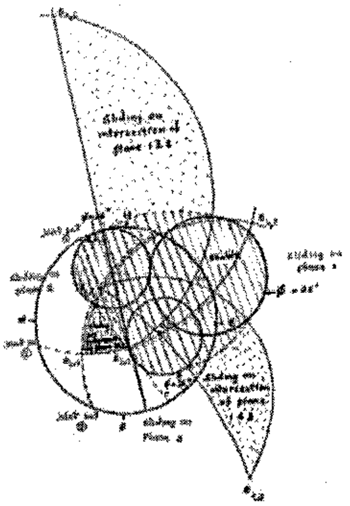
Table 3. Q value, rock mass quality and supporting system along Route A cccording to Q charts

Chainage (m)	Q	Rock mass quality	Supporting system category*	
			Crown	Wall
0-8	4.750	Fair	1	1
8-100	3.770	Fair	4	1
100-220	1.430	Poor	4	4
220-323	0.530	very Poor	5	8
323-400	0.002	Exceptionally poor	8	7
400-508	0.067	Extremely poor	7	5
508-1200	5.000	Fair	3	1

Table 4. Q value, rock mass quality and supporting system along Route B

Chainage (m)	Q	Rock mass quality	Supporting system category*	
			Crown	Wall
0-200	0.9	Very poor	5	4
200-275	1.6	Poor	4	4
275-860	0	Cut and cover tunnel		
860-1500	5.6	Fair	3	1

3.4 *Stereonet analysis*



3.5 *Cost analysis*

Table 5 Cost per meter run

Route	Cost per meter run (LKR)
A	1,245,000.00
B	985,000.00

3.6 *Results of AHP approach*

Table 6 Weights of alternative tunnel routes

Alternative	Weight (%)
Route A	48.7
Route B	51.3

Table 7 Weights of tunneling methods for route B

Section of route B (m)	Weight (%) D & B	C & C	NATM
0-200	25.7	35.2	39
200-940		49.4	50.5
940-1500	50.7	NA	49.2

4 CONCLUSIONS

The Engineering geological assessment revealed that the surrounding rock masses belong to the poor rock class. The final result of the AHP has been given that, the route B is the optimal tunnel route. According to the AHP analysis, chainage 0 to 940 m, excavation has to be carried out using NATM whilst Chainage 900 to 1500 m, it is preferred to use Drill and Blast method.

ACKNOWLEDGMENTS

First and foremost we would like to thank our supervisors of this project, Eng. Prof. T. M. Pallewatta and Eng. M. N. C. Samarawickrama for the valuable guidance and advice.

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We thank profusely the senior lecturers of the Civil Engineering Department of the Open University of Sri Lanka, all the project staff of CEB and CECB for their kind help and co-operation throughout our study period.

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Effect of Construction Sequence on the Behaviour of Gravity Type Retaining Wall

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ABSTRACT: Gravity retaining walls derive their capacity to resist lateral movement through the dead weight of the wall. The design methodologies proposed by standards do not take into account the construction sequences that simulate the process by which the soil and retaining wall are brought together. However, in reality, at least during the backfilling process, the retaining wall undergoes many displacements that are not so far considered in the design. In this investigation, effect of construction sequences in the gravity retaining walls with different shapes is investigated with the help of finite element method. Two different construction sequences, namely the backfilling after wall construction and the backfilling parallel to wall construction, are compared for different wall shape models. Lateral displacement of the bottom and the top of the wall is plotted for each model and construction sequence with construction stages. Bearing pressure distribution, lateral earth pressure and failure wedge angle are summarized and compared with design values. Each wall showed different behavior for each of the construction sequences. Back filling after wall construction minimizes the sliding failure and bearing pressure. Overturning failure could be reduced by backfilling parallel to wall construction. However, it was observed that, comparatively, backfilling after wall construction is more effective than backfilling parallel to wall construction, suggesting that proper selection of construction method also may reduce negative effects on the wall stability.

1 INTRODUCTION

To ensure stability of retaining structures, they shall be designed to withstand lateral pressures due to soil and water, the effects of surcharge loads, self-weight of the wall, and earthquake loads. In addition, earth-retaining systems shall be designed to provide adequate structural capacity with acceptable movements, adequate foundation capacity with acceptable settlements, and acceptable overall stability of slopes adjacent to walls. These are the serviceability requirements. The tolerable levels of lateral and vertical deformations are controlled by type and location of wall structure and surrounding facilities.

Gravity retaining walls derive their capacity to resist lateral loads through the dead weight of the wall. In the construction process of retaining walls, back fill is done after the construction. This is the traditional method usually used. However, often construction sequence is not taken into account in the design methodology of the retaining walls. Overall the stability design is believed to be reliable and accurate, because the safety factors have been allowed in design calculations. However, would the design calculations be adequate against the disturbances during the construction sequence? Would different construction sequences influence the stability of gravity retaining walls? With respect to construction sequence, which is the most suitable shape for gravity retaining wall? These are the main questions that would be addressed in this research.

Researches on influence of compaction behind the retaining walls were carried out by Transport and Road Laboratory-UK and Kulathilaka (1990). Ahmed (2012) explored the effect of construction sequences on the behavior of a backfilled retaining wall. In his investigation, the influence of the construction sequences on the behavior of an L shaped stiff retaining wall was investigated with a numerical model. His observations highlighted the fact that rotations and translations of the wall occur simultaneously during the staged backfilling process, which better simulate the real construction process.

However, often the design methodology does not take into account the construction sequences that simulate the process by which the soil and the gravity retaining wall are brought together. There is little research which addresses the effect of construction sequences of gravity type retaining walls. Possible construction sequences are backfilling after wall construction and backfilling parallel to wall construction. This research will compare both of these construction sequences for different shapes of gravity retaining walls.

2 OBJECTIVES

The main objectives of the study are,

1) Carry out numerical analysis on the effects of construction sequence on different shapes of gravity retaining walls.
2) Investigate the effects of construction sequences on bearing pressure distribution and failure wedge of gravity retaining walls.

3 RETAINING WALL DESIGN

In order to construct the finite element model for this study, retaining walls were designed based on BS 8002 design guide. Three different shapes with constant height and cross sectional area were selected and trial method was used to get proper stable retaining wall based on BS 8002.

In the design procedure, first force exerted on the retaining wall was estimated by considering the statical equilibrium on the soil wedge bounded by the wall, the failure surface and the surface profile. Calculations were based on Coulomb's method of analyse and wedge method.

Optimal base sizes were calculated for three walls by considering overturning, sliding, and bearing capacity. Cross section area and height are maintained as constant. The dimensions were calculated considering the safety against self-weight failure. All dimensions (in mm) of three retaining wall models are shown below.

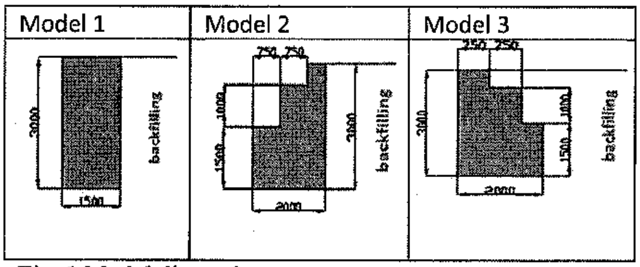


Fig. 1 Model dimensions

4 PARAMETERS FOR FEM ANALYSIS

Performance of an earth retaining system depends on many factors, in particular, successive stages of construction. The conventional design methods using design guidelines are not capable of evaluating the yield information on likely displacements in the system. The finite element analysis, which is widely used in design practices today, can be used to model complex soil-wall interaction problems.

Numerical analysis was carried out in plane strain and 15-nodes triangular elements. Movement of the wall is the major consideration in determining the wall deflection. Hence fine mesh was used in the model. Soil was modeled using Mohr-Coulomb model and concrete wall model as linear elastic model. The utilized soil modeling parameters and concrete retaining wall modeling parameters are presented in Table 1 and Table 2.

Table 1. Concrete properties

Parameters	Name	Concrete	Unit
Material model	model	Linear elastic	-
Type of material behavior	type	Non-porous	-
concrete unit weight-Grade 40	γ_{bulk}	24	kN/m^3
Permeability	k_x, k_y	0	m/day
Young's modulus	E_{ref}	26,000,000	kN/m^2
Poisson's ratio	v	0.15	-
Strength reduction factor	R_{inter}	-	-

Table 2. Dense sand properties

Parameters	**Name**	**Dense sand**	**Unit**
Material model	model	M-C model	-
material behavior	type	drained	-
Soil unit weight	γ_{bulk}	18	kN/m^3
Permeability	k_x, k_y	0.36	m/day
Young's modulus	E_{ref}	20,000	kN/m^2
Poisson's ratio	v	0.3	-
Cohesion	C_{ref}	0.1	kN/m^2
Friction angle	φ	32	°
Dilatancy angle		2	°
Strength reduction factor	R_{inter}	1	-

5 CONSTRUCTION SEQUENCES

In order to investigate the effect of the construction sequences, the backfill soil was divided into 6 layers of 0.5m thick each that yield the total initial height of 3m.

5.1 *Backfilling after wall construction - (construction method-1)*

Calculations for the multi-phases numerical analysis were performed using the stage construction procedure. The calculations were executed in 8 phases including the surcharge loading, starting from the initial state where the wall is constructed parallel to, each phase corresponding to a single loading of 0.5m of backfilling, yielding a total of 6 layers (phases), and ending with the state where all finite element model components, including surcharge loading, were activated. Calculation progressed until the prescribed ultimate state is fully reached.

5.2 *Backfilling parallel to wall construction - (construction method-2)*

The calculations were executed in 7 phases including the surcharge loading, starting from the initial state where the wall is constructed parallel to each phase corresponding to a single loading of 0.5m of backfilling, yielding a total of 6 layers (phases), and ending with the state where all finite element model components, including surcharge loading were activated.

6 FEM ANALYISIS AND RESULTS

Development of the lateral deformation at the top of the wall with the progress of backfill for each type of retaining wall is presented in Fig. 2 through Fig. 5.

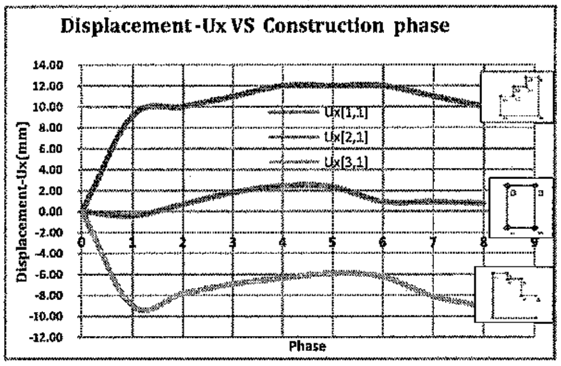


Fig. 2 Construction method 1 - (reference to top edge)

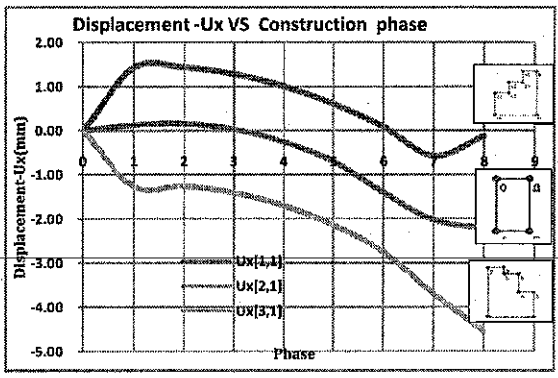


Fig. 3 Construction method 1 (reference to bottom base)

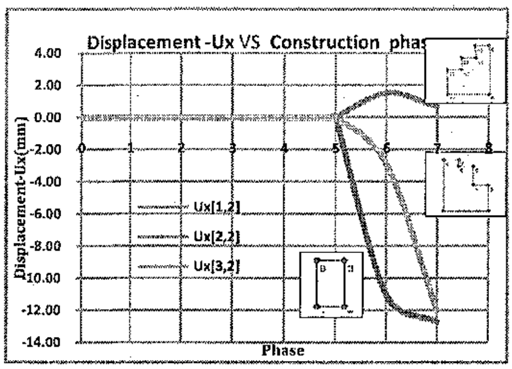


Fig. 4 Construction method 2 - (reference to top edge)

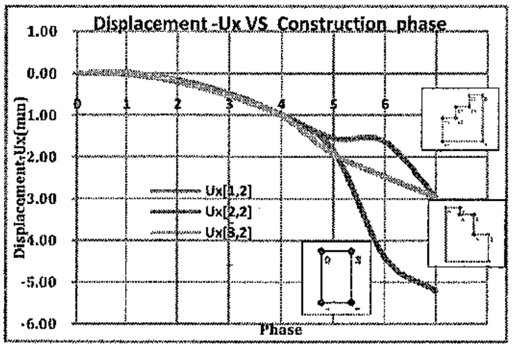


Fig. 5 Construction method 2 - (reference to bottom base)

6.1 *Final displacement analysis in a view*

Table 3 briefly explains the final displacement vector with magnitude and direction.

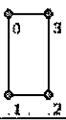
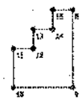
Table 3. Final displacement

Model	Construction sequence	Horizontal Displacement diagram(backfilling right hand side)	Vertical displacement	Comments
Model 1	1	[illegible]mm; 2.21mm	14.5mm	Ux_{tp}=+2.98mm
	2	12.67mm; 5.21mm	10.7mm	Ux_{tp}=-7.46mm
Model 2	1	10mm; 0.12mm	13.2mm	Ux_{tp}=+10.12mm
	2	0.64mm; 2.96mm	10.7mm	Ux_{tp}=+3.6mm
Model 3	1	9.106mm; 4.544mm	13.2mm	Ux_{tp}=-4.544mm
	2	12mm; 2.96mm	14.5mm	Ux_{tp}=-9.04mm

6.2 *Bearing pressure distribution*

Table 4 briefly compares the results obtained from FEM and manual.

Table 4. Bearing pressure distribution

Model	Construction sequences	Maximum bearing pressure(kN/m²)		Pressure distribution
		FEM	MANUAL	
	1	93.87	195.84	Non uniform
	2	103.32		Non uniform
	1	81.68	100.01	Non uniform
	2	81.76		Non uniform
	1	104.46	201.64	Non uniform
	2	123.3		Non uniform

7 DISCUSSION AND CONCLUSION

Often the design methodology of retaining walls does not take into account the construction sequences which simulate the process by which the soil and the retaining wall are brought together. In the present investigation, two different construction sequences were employed to evaluate the effect of the construction methods. Out of the three types of walls considered, the third type is found to have the lowest stability. It shows high bottom and top displacement outward the backfilling. Both sliding and overturning are in the same direction. Bearing pressure is 201.64kN/m^2 (BS 8002). When considering wedge failure, the wedge starts from under the base. The wall is likely to fail due to above critical reasons. In addition the centre of gravity of the wall is toward the outward face of wall. This is the reason for high rotation in anti-clockwise direction, which is negative in this instance. For these reasons, we suggest that wall type-3 is not preferable in stable construction of high walls.

Other two gravity walls show stability against backfilling. When we consider the wall type-1, it shows unfavorable horizontal displacement in top and bottom of wall for construction method 2. Both sliding and overturning are outward of the backfilling. Construction method 1 leads to smaller top and bottom displacement in opposite directions, however in clockwise direction, which is positive in this instance. Bearing pressure is within the limit. Significant (2.21mm) sliding has increased the stability of the wall. For these reasons, the construction sequence of method 1, i.e., backfilling after wall construction, is preferable for wall type 1.

Wall type 2 appears to be the most preferable among all three types of walls. In construction method-1, even though overturning is significantly high, it is toward the backfilling, which is a desirable direction. Centre of gravity of wall is toward the backfilling face. Stability has increased by this. Construction method 2 shows a small sliding and overturning tendency. However, its failure wedge angle is smaller than construction method 1. Therefore, both construction (methods) sequences are preferable for wall type 2.

Finally with this examination, it could be concluded that the construction sequence is a critical factor to be considered in the design stage of gravity type walls, because these observations clearly demonstrate that the construction sequences influence the stability of the wall both during and after wall construction.

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Effect of Temperature on Mechanical Behavior of Geopolymer Well Cement used for Oil and Gas Wells

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ABSTRACT: Carbon capture and storage (CCS) technique is found as a best solution to reduce the emission of CO_2 to the atmosphere. In this technique, the CO_2 emitted from large industries is captured, and pressurized, and finally injected into deep underground reservoirs. In a geological sequestration project, integrity of injection well play an important role. It means the well cement is a key factor that affects the well integrity. In typical injection wells, Ordinary Portland cement (OPC) based cement is used as well cement and it has been found that it undergoes degradation in CO_2 rich environment. Geopolymer can be a good alternative to existing OPC based well cement as it has been found that geopolymer possess high strength and durability compared to OPC. Geopolymer is a binder produced through the process called geopolymerization of alumino- silicate materials and alkaline activators. In the sequestration wells, well cement is exposed to different curing temperatures with a geothermal gradient of 30°C/km. Therefore, it is important to study the mechanical behaviour of well cement with curing temperatures expected deep under the ground. Therefore, this research aims to study geopolymer as well cement and its mechanical behaviour at different curing temperatures (25, 40, 50, 60, 70, 80 °C). In addition, effect of ageing on the mechanical behaviour was also studied. The OPC samples were tested for the comparison of results with geopolymer. The results showed that the optimal curing temperature for higher strength of geopolymer and OPC are 60 °C and 50 °C respectively. Geopolymer possess highest strength at elevated temperatures whereas OPC possess higher strength at ambient temperatures. Moreover, at elevated temperature curing, geopolymer develops ultimate strength within short curing period and it does not gain significant strength with further ageing.

1 INTRODUCTION

Emission of greenhouse gases is a major problem in the world. Of all greenhouse gases, CO_2 is responsible for 64% of the greenhouse gas effects [1]. There are so many ways to reduce the emission of greenhouse gases; such as minimizing fossil fuel consumption in industries and vehicles, increasing the energy conversion efficiency of fossil fuels, switching energy sources in to renewable energy sources such as wind energy, wave energy and solar radiation and capturing and storing carbon dioxide in deep under the ground [1, 2]. Of all the proposed methods, carbon capture and storage (CCS) technique is found as a good solution to reduce CO_2 emission to atmosphere [3].
The lifetime of CCS projects depends on many factors and within these well integrity plays an important role. Well cement is the major factor that affects well integrity and, in the injection wells, Ordinary Portland Cement (OPC) based cement (class G, H) is used as well cement. According to previous studies [4, 5], OPC undergoes degradation in CO_2 rich environment due to the reaction with dissolved CO_2 in brine. Kutchko et al [4], found that when OPC based well cement exposed to a CO_2 rich environment, it undergoes carbonation followed by cement degradation. In addition, various other researchers [5, 6, 7] have also found that OPC based well cement experiences degradation exposed to CO_2 rich environment.

This paper examines geopolymer as well cement since, geopolymer possess high strength, excellent acid resistance characteristics and high durability [8, 9]. Davidovits, [10] proposed that an alkaline liquid could be used to react with the silicon (Si) and the aluminium (Al) in a source material of geological origin or in by-product materials such as fly ash and rice husk ash to produce binders and he termed these binders as geopolymers. A generalized formula for geopolymer is as follows:

$$M_n[-(SiO_2)_z-AlO_2]_n.wH_2O$$

Where z is 1, 2 or 3; M is an alkali cation, such as potassium or sodium, and n is the degree of polymerization [10, 11].

A typical underground well is constructed from ground level to the required depth depends on the injection reservoir level and it may vary from 800 m to 2 km. As the temperature is varying with depth with a geothermal gradient of 30 °C/ km [2], the well cement is exposed to different temperatures varying up to approximately 80 °C. Therefore variation of mechanical behaviour of geopolymer at different down-hole temperature conditions need to be studied in order to predict the behaviour of geopolymer cement during the life time in the down-hole conditions. This paper investigate geopolymer as well cement and its mechanical behaviour with curing temperature from ambient level (27 °C) to 80 °C.

2 MATERIALS AND METHODOLOGY

2.1 *Materials*

Geopolymer paste samples were prepared using fly ash as the alumino-silicate material and combination of NaOH and Na_2SiO_3 as alkaline activator. The ASTM class F fly ash (low calcium) which is produced at Nuraichcholai coal power plant, Puttalam, Sri Lanka, was obtained from Holcim Lanka (Pvt) ltd.

The mix compositions of fly ash and OPC was obtained from X-Ray Diffraction (XRD) test and the results are shown in Table 1.

Table 1. Compositions of fly ash and OPC

Constituents	Percentage (%)	
	Fly ash	OPC
SiO_2	52.03	20.38
Al_2O_3	32.31	4.79
Fe_2O_3	7.04	3.26
CaO	5.55	64.4
Mgo	1.3	0.98
SO_3	0.07	2.21
K_2O	0.68	0.04
Cl	1	0.01

8 M NaOH solution was mixed with Na_2SiO_3 with a ratio of Na_2SiO_3 to NaOH of 2.5 to obtain higher strength [12]. The ratio of alkaline activator to fly ash used was 0.4 for all the mix design. In addition, sulphate resistant OPC samples was tested to compare the results. Sulphate resistance OPC was obtained from Holcim Lanka (Pvt) Ltd. For the mix of OPC samples, a w/c ratio of 0.44 was used as it is found to be the optimum to achieve higher strength [13].

2.2 *Sample preparation and experimental methodology*

Geopolymer paste was prepared by mixing fly ash with alkaline activator in above proportions. The NaOH pellets was mixed with distilled water to prepare 8 M NaOH solution. This was mixed with Na_2SiO_3 with above proportions and the alkaline activator was prepared. PVC pipes with 50 mm diameter were cut into 100 mm height pieces to make the casting moulds. Fly ash was mixed with alkali solution using a mechanical concrete mixture for 3 minutes and the mixture was poured in to the prepared mould in three layers. Then the samples was placed on the vibrating table for 2 minutes in order to remove any air voids present in the sample.

This work includes two types of curing to study the effect of curing temperature and ageing time. To study the effect of curing temperature all the samples were cured at different curing temperatures (27- 80 °C) for 48 hours and then they were allowed to cool at room temperature (RT) for another 24 hours before testing. Fig. 1 shows the oven cured samples. Based on this test, optimum temperature for high strength was observed. Effect of ageing was studied by exposing samples to two different temperatures (27 °C and the optimum temperature) for prolong curing periods of 2, 7, 14, 28 and 45 days.

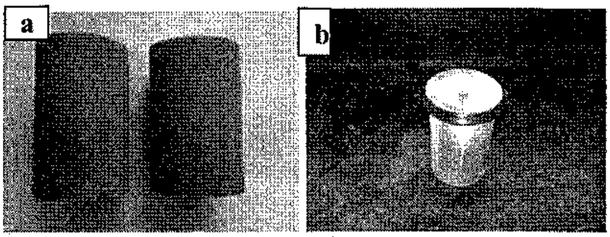

Fig. 1 (a) Oven cured geopolymer samples, (b) sulphur capped sample

UCS test was conducted on the samples with a stress controlled loading rate of 0.2 MPa/ s. For each data point, two samples were tested to ensure reproducibility. To study the microstructural behaviour of geopolymer cement with temperature variations and the curing duration, SEM analysis was performed.

3 RESULTS AND DISCUSSIONS

3.1 *Effect of curing temperature*

Well cement is exposed to different temperature conditions (from ambient level to 80 °C). Therefore, the failure stress of geopolymer paste and OPC motar was tested at different curing temperature for 48 hours of curing. Fig. 2 shows the strength variation with curing temperature for both geopolymer and OPC.

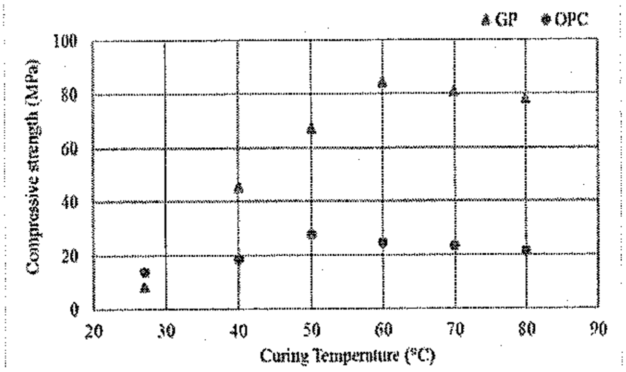

Fig. 2 Variation of UCS with curing temperature

According to Fig. 2, at room temperature, the strength of the geopolymer is considerably low and it is because of the poor rate of geopolymerization process. The rate of geopolymerization is high at elevated temperatures [8, 9]. The optimum temperature for high strength for geopolymer and OPC are approximately 60 and 50 °C respectively.

Geopolymer gains strength with curing temperature as Si and Al from the source material readily dissolves with the increase in curing temperature up to 60 °C. After that the strength decreases with the temperature. However, some of the researches [12, 14] found that the optimum strength is between 70- 80 °C. The optimum temperature may vary depends on the source of fly ash, type of curing, sample compositions and the mix compositions [12].

For geopolymer, strength is decreasing beyond 60 °C. This may be due to the weakening of microstructure at elevated temperatures or the formation of micro cracks [9]. When the behaviour of OPC and geopolymer is compared, it can be seen that at room temperature conditions, OPC has higher strength compared to geopolymer. This is because of the poor geopolymerization rate for geopolymer at room temperature. On the other hand, at elevated temperatures, geopolymer possesses high strength compared to OPC. Rate of strength increment of geopolymer from room temperature to optimum temperature (60 °C) is 90 % while the rate of increment of OPC is 49 %. The increment rate is much high for geopolymer than OPC.

3.2 *Effect of ageing*

To study the effect of ageing, OPC and geopolymer samples were cured at room temperature and optimum temperature (60 °C) for different curing periods. Fig. 3 shows the variation of UCS of OPC and geopolymer with the ageing.

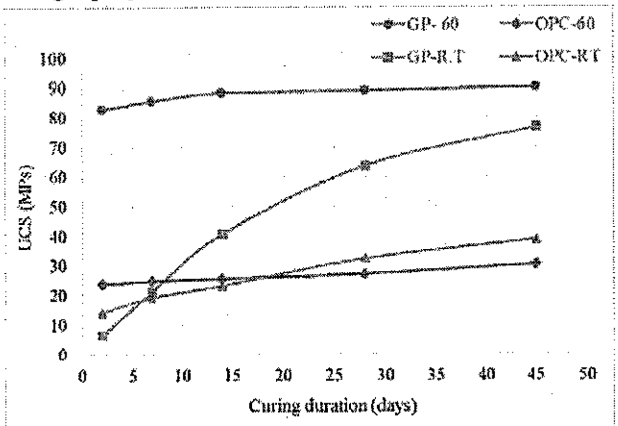

Fig. 3 Variation of UCS with ageing

According to Fig. 3, UCS of geopolymer and OPC increases with the ageing time. This is because of the geopolymerization process of geopolymer and hydration process of OPC with ageing. The strength gaining of the geopolymer cured at room temperature is higher than that of OPC cured at same Conditions.

At room temperature curing the rate of strength increment of geopolymer in 2- 45 days is 92 % while that for OPC is 63 %. This shows that even at low temperatures geopolymer develops higher strength compared to OPC. For geopolymer cured at 60 °C, the rate of increment in strength is low compared to the samples cured at room temperature. For geopolymer, the geopolymerization process is almost finished within 48 hours of curing for elevated temperature. Hence, geopolymer will not develop significant strength increment with further ageing when cured at elevated temperatures.

3.3 *SEM results*

Micrographs of fly ash based geopolymer were obtained using a ZEISS field-emission scanning electron microscope (FESEM). Fig. 4 shows the SEM images of fly ash and geopolymer samples cured at different conditions.

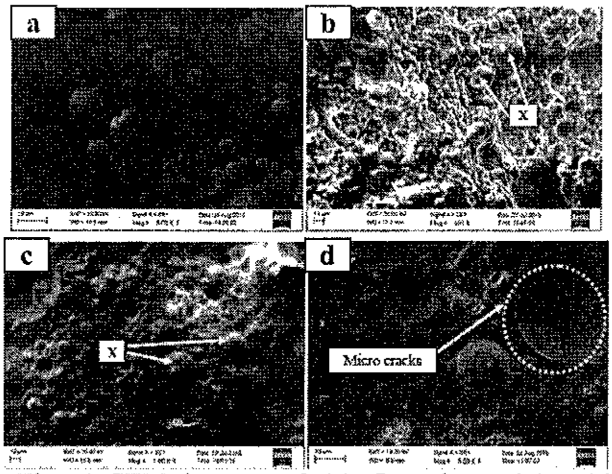

Fig. 4 SEM images of (a) fly ash particles (a), geopolymer cured at (b) RT, (c) 60 °C and (d) 80 °C

Fig. 4 clearly shows that the different geopolymerization rate between samples cured at R.T and 60 °C. But there is no significant variation of unreacted particles between samples cured at 60 °C and 80 °C. Based on this, it can be concluded that the strength reduction beyond 60 °C is not due to the variation in rate of geopolymerization. In Fig. 8 (d), some micro-cracks can be observed. Hence, it can be concluded that the strength reduction beyond 60 °C is due to the formation of micro-cracks at elevated temperatures.

4 CONCLUSIONS

The following conclusions are drawn based on the outcomes of this research.

1. The optimum curing temperature for fly ash based geopolymer is 60 °C and there is no considerable strength gain after optimum temperature.
2. UCS and Young's modulus of geopolymer increases with curing temperature up to 60 °C and beyond that it decreases.
3. At lower curing temperatures (below 40 °C), OPC possess higher strength than geopolymer, whereas at elevated temperatures geopolymer possess higher strength.
4. At low temperature curing, both OPC and geopolymer develop strength with ageing and the rate of strength gaining is high for geopolymer compared to OPC.
5. On the whole, geopolymer is suitable for temperature of above 40 °C, whereas OPC can be used at shallow depths where temperature is low (< 40 °C).

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Mechanical Behavior of Well Cement Cured in Saline Water: an Application to Oil and Gas Wells

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ABSTRACT: Geo sequestration of carbon dioxide (CO_2) has been found to be one of the best solutions to reduce greenhouse effect. The injection wells and well cement plays a vital role in well integrity. The loss of well integrity is the major issued and that may be explained by the degradation of cement as well as the down hole curing conditions. Salinity is an important factor that varies from 0-30% NaCl depending on the geological location. Therefore, aim of this research is to analyze the mechanical behaviour of cement cured in different salinity levels with various curing periods. Testings such as uniaxial compressive strength (UCS) test, X-Ray Diffraction (XRD) and Scanning Electron Microscopical (SEM) observation were conducted to obtain the behavior of the cement cured in different conditions. Based on the outcome, it was observed that the strength of the OPC based well cement decreases with increasing salinity levels and with ageing in time.

1 INTRODUCTION

The CO_2 is the gas that plays a major role in greenhouse effect that leads to environmental pollution and dispersion of sustainability of the globe. According to the report of Energy information administration 2003, annual emission of CO_2 is 6.5GT/year and it is predicted that 2000 million tons of CO_2 increment in 2030 (Nasvi et al. 2012)). Therefore, it is very much important to understand how the CO_2 reaches the atmosphere. The major CO_2 emissions are due to the increasing usage of coal powered plants, petroleum usage and oil recovery. Using low carbon energy sources are eco solutions for this problem but it cannot satisfy the energy requirement in our present situation. Therefore Lecolier et al. (2007) and Bachu and Bennion (2009) concluded that storing carbon dioxide (CO_2) in underground wells is the most effective way for long-term, safe and low-cost CO_2 sequestration.

In Sri Lanka, a coal power plant (Lakvijaya coal power plant, Norochcholai) has been constructed and it is in operation from 2011. But in the future, we might go for more coal power plants to fulfil our energy requirements. However, there will be a problem related to emission of CO_2 due to the combustion of coal. Hence, carbon capture and storage will be useful in this context. Coal seams, saline aquifers and depleted oil and gas reservoirs, deep saline aquifers and unminable coals beds are used for storing captured CO_2 (Bruant et al (2009))

Injection wells are widely used not only for CO_2 injection but also for the oil and gas extraction projects. Therefore, it is important to ensure cement integrity under various down-hole conditions. The integrity and zonal isolation of the injection wells depends on many factors and the major factor is well cement that is used as primary cement for the construction of these wells. Well cement must be carefully designed to meet demanding requirements such as predictable thickening time, high sulphate resistance, high durability, fluid loss control, consistency, low viscosity, low free fluid, and strength (Luke et al. 2012). This well cement is categorized according to the American Petroleum Institute (API) specification standards, which has identified 8 classes of cement according to the depth of the well, the temperature and pressure (Runar 2010)). Generally, API class G & H cements are commonly used for the construction of injection wells. However, class G is most preferable for construction (Runar 2010). At present, OPC based well cement is used for more than 80% of the well construction in oil & gas field (Barlet et al. 2009; Arina and Sonny 2012)). This primary cement acts as a sealant and prevents fluid communication between the various underground fluid-containing layers where the salinity level (0-36%) varies in place to place and also varies with depth (Nasvi et al. 2012;, Barlet et al. 2009). Therefore, the well cement in the injection/production well is exposed to different salinity level, and this issue affects the actual strength of the OPC based well cement. Hence, mechanical behaviour of the well cement depends on the curing under various salinity levels.

To date, there are few studies focusing on oil well cement behaviour in saline water. Lecolier et al. (2007) studied the strength variation of the API class G cement cured in brine for one year at 80°C. Based on the above study, the strength of the cement decreased with ageing time due to the degradation of C-S-H. Similarly, Xingshan et al.

(1996) studied the mechanical behaviour of class G cement cured in different NaCl brine concentrations at two different curing temperatures (93 °C & 160 °C) and they observed that the strength increased with curing time (up to 72 hours) at both temperatures because of the degree of hydration. In addition they also observed that the strength decreases with the brine concentrations due to the retardation in hydration process in brine. However, there are no studies relevant to the mechanical behavior of OPC based well cement in brine at average down-hole temperature (50°C). Therefore, major aim of this research is to study the mechanical behavior of OPC based well cement cured under different salinity levels for different ageing time. Uniaxial compressive strength (UCS) test, X-Ray Diffraction (XRD) test, Scanning Electron Microscopical (SEM) observation and Energy Dispersive X-ray (EDX) spectroscopy analysis were conducted and results were compared.

2 EXPERIMENTAL METHODOLOGY

Well cement samples were prepared using Ordinary Portland Cement (OPC) with sulphate resistance. OPC was purchased from Holcim Lanka (Pvt) ltd, Colombo, Sri Lanka. The composition of the OPC with sulphate resistance was obtained from X-Ray Diffraction (XRD) test and the results are given in section 3.1. The cement paste was prepared with a water/cement ratio of 0.44 as it is found to be the optimum based on Nasvi et al. (2012), Lecolier et al. (2007) and Nadine et al. (2009) . Based on the mix design, 50 mm diameter and 100 mm height samples were prepared using PVC moulds. Then they were oven cured at 50 °C and during the oven curing, top side of the samples was covered with polythene to avoid excessive moisture loss. After 24 hours of curing, the samples were allowed to cool down for another 6 hours in the atmosphere.

The samples were then transferred to the saturation containers having different brine concentrations (0, 10, 20 and 30% NaCl) except the control samples that were used to compare the results. The brine solutions were prepared by mixing NaCl salt and distilled water in due mix proportions. All the samples were saturated for 7, 14, 28 and 45 days prior to the testing. For each conditions three samples were tested to ensure reproducibility. UCS test was conducted by using the UCS testing machine shown in Fig. 1 with the stress control loading rate of 0.2 MPa/sec. Samples were machine ground both sides and top surface was capped with sulphur prior to the UCS testing to apply uniform uniaxial loading. In addition, Scanning Electron Microscopical (SEM) test was conducted by using Oxford EVO LS15 SEM apparatus at Faculty of Science, University of Peradeniya, Sri Lanka. For the SEM observation, samples were gold coated to remove the voids prior to the observation.

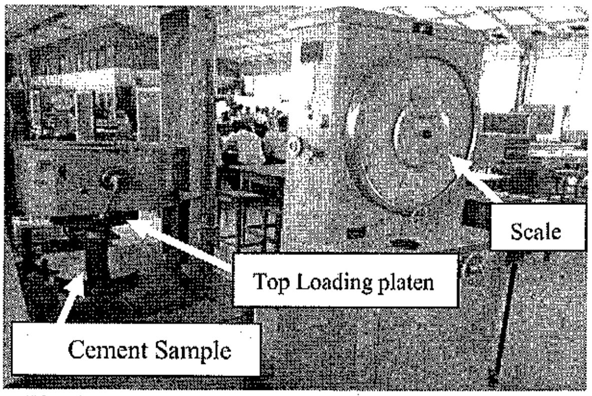


Fig. 1 UCS testing machine used for the experiment

3 RESULTS AND DISCUSSION

3.1 *XRD Test results*

The composition of the OPC based well cement (OPC with sulphate resistance) was obtained from XRD test. Table 01 shows the composition of OPC based well cement obtained from XRD test and typical composition of API class G cement. For the construction of injection wells, API class G cement is most preferable and widely used due to its suitability for high depths, high pressure/temperature conditions and high sulphate resistance.
Based on Table 1, it can be seen that the composition of OPC based well cement is similar to API class G cement. Therefore, it can be concluded that OPC can be used instead of API class G cement for the construction of injection wells.

Table 1. Composition of OPC and typical class G cement

Constituents	Percentage (%)	
	OPC	**Class G**
CaO	64.40	64.7
SiO_2	20.38	22.91
Fe_2O_3	3.26	4.75
Al_2O_3	4.79	3.89
MgO	0.98	1.8
SO_3	2.21	0.74
K_2O	0.04	0.64

3.2 *Variation of UCS and Young's modulus with salinity and ageing*

The variation of average UCS of OPC based well cement with curing period is shown in Fig. 2.

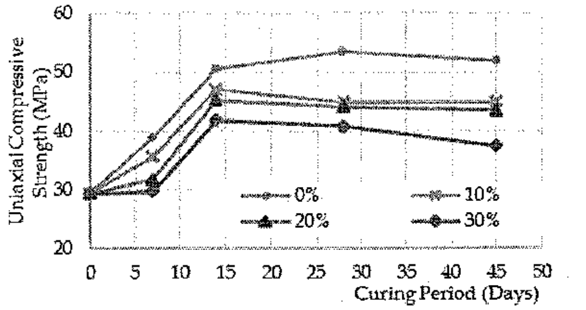


Fig. 2 Variation of Uniaxial compressive strength with curing period

It can be seen that initially the strength of the OPC cement in brine concentrations increases with the ageing period up to 14 days and then it decreases towards 45 days. The UCS of dry sample was 29.4 MPa. The increment varies from 42% - 60 % depending on the salinity up to 14 days. This initial increment is because of the dominant hydration process (Lecolier et al. 2007). However, after a certain period (approximately 14 days), continuous ageing in brine solution causes the reduction of strength and the reduction percentage varies from 4.7% - 10.6% for different salinity levels. This is because of the Ca^{2+} ions in Calcium Silica Hydrate (C-S-H) get precipitated with Cl^- ions i.e. in continuous aging C-S-H degrades in brine solution (Xingshan et al. 1996). On the other hand, samples saturated in 0% NaCl (pure water) does not show any significant amount of decrement in compressive strength as strength variation is caused only due to hydration process.

The variation of UCS with salinity level of the samples saturated for different curing periods is shown in Fig. 3.

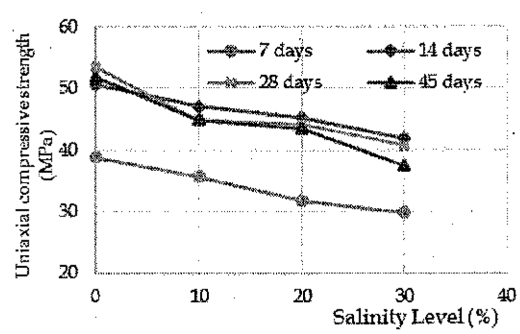


Fig. 3 Variation of Uniaxial compressive strength with salinity level

Based on Fig. 3, it is clear that the uniaxial compressive strength decreases with salinity level for all the curing periods. This strength decrement is because of the retardation in the degree of hydration. When the brine concentration increases, there is a retardation in hydration process i.e. salt content affects the $Ca(OH)_2$ formation. In addition, C-S-H absorbs the NaCl micro crystallites and Na^+ on the surface of the fibrous structure and this affects the interface bonding between $Ca(OH)_2$ and CSH. (Lecolier et al. 2007, Xingshan et al. 1996). As a result, uniaxial compressive strength of well cement tends to reduce. In addition, it can be seen that 7 days UCS for all the brine contents (0-30%) is much lower than that of other curing periods (14, 28 & 45 days). This might be due to the partial hydration process i.e. high strength can be obtained only after the complete hydration process.

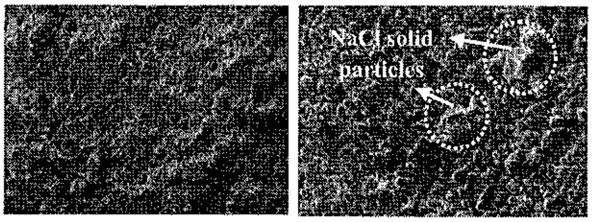
Further, average stress-strain behaviour of the brine saturated samples were obtained from UCS testing and Young's modulus of the cement samples were calculated based on these stress strain plots. Stress – strain plots were derived from the load – displacement relationship obtained from UCS test apparatus. The behavior of Young's modulus of OPC based well cement with ageing period and salinity levels are also shown the similar variation of UCS. Young's Modulus initially increases in the brine concentrations and then decreases with the saturation periods. In each duration period, Young's modulus decreases with salinity level. This means stiffness of the cement decreases with the increase of brine content and with ageing time.

3.3 *SEM Test results*

SEM observation was conducted to identify the microstructural variation and appearance of the samples cured under different curing conditions.

Fig. 4 shows the microstructural variations and appearance of the samples cured under different conditions. Samples cured under different conditions shows different microstructural characteristics. Generally, the microstructure of the samples varies due to hydration process.

Presence of NaCl affects the microstructure of the cement and behaviour of the cement as well (Fig. 4). However, the effect of NaCl is mostly dominant exposed at long term curing as can be seen from Fig. 4(b). With the help of Energy-Dispersive X-Ray Spectroscopy (EDX) analysis, it was ensured that the white colour crystal forms indicated in Fig. 4(b) are NaCl solid deposits. It shows that in continuous curing in brine solution, NaCl particles penetrates through the cement samples and get deposited in between and causes the degradation of the cement samples.



(a) (b)

Fig. 4 Microstructure of the samples saturated (a) 7 days (b) 45 days in 30% NaCl brine

Because of this degradation process by NaCl, the compressive strength reduces with the increasing of brine concentration and increasing of curing time as well.

4 CONCLUSIONS

Well cement plays a vital role in the integrity of the carbon capture storages and oil and gas extraction projects. The loss of well integrity may be explained by the degradation of cement as well as the down hole curing conditions. Therefore, an experimental research conducted to analyze the mechanical behavior of OPC cured in different salinity levels. A series of uniaxial compressive strength (UCS) tests was conducted on OPC based well cement to study the effect of salinity (0% - 30% NaCl) and ageing (7 - 45 days) on the mechanical behavior of well cement.

The X-Ray Diffraction test was carried out to compare the chemical composition of the OPC samples with API Class G cement. From this test results it is ensured that OPC with sulphate resistance satisfies the requirement of API Class G cement. Therefore, OPC with sulphate resistance can be used instead of API class G cement for the construction of injection wells. Based on the findings, strength and the Young's modulus of OPC samples reduce with increase in salinity levels and this is due to the retardation in hydration process with increase in salinity. In addition, the strength and Young's modulus of the cement samples cured in brine concentrations initially increases for a short duration (14 days) but then decreases beyond 14 days. This is because initially the hydration process is dominant but in continuous ageing the samples get degraded because of the presence of NaCl and this dominant factor reduces the strength and Young's Modulus.

This result was further supported by observing the microstructural variations by using Scanning Electron Microscope (SEM) and ensured that the above behavior of OPC is due to the presence of NaCl in brine saturated samples.

Based on this research outcome, concentration of brine solution plays an important role in physical and chemical changes in well cement. Therefore, effect of saline water is a very important factor that should be considered seriously for the construction of CO_2 injection wells and oil and gas wells.Conclusion

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