GEOTECHNICAL ENGINEERING PROJECT DAY 2013

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

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SRI LANKAN GEOTECHNICAL SOCIETY

SLGS

CONTENTS

Message from the President - SLGS

Sri Lankan engineers encounter many challenges in the field of Geotechnical Engineering**.** To overcome those challenges we need to be aware of the current developments in the field and should find ways of applying this new knowledge under our own conditions and make further innovations.

From its inception, Sri Lankan Geotechnical Society has provided a forum for disseminating new knowledge in the field of geotechnical engineering and promoting research.

The Project Day competition is an annual event held among Sri Lankan undergraduates doing projects in the field of geotechnical engineering. It was commenced in year 2001 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.

This year there are thirteen papers on a wide variety of topics and I thank all the authors for their interest and commitment.

I also wish to convey my sincere gratitude to the panel of evaluators; Emeritus Professor B. L. Tennekoon, Mr. K. S. Senanayake and Prof. T. A. Peiris.

Prof. Athula Kulathilaka
President – SLGS

LONG TERM EFFECTS OF $CaCl_2$ SOLUTION ON HYDRAULIC CONDUCTIVITY,PHYSICAL AND INDEX PROPERTIES OF EXPANSIVE CLAY LINER MATERIALS

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Abstract: *The objective of this study was to evaluate the effects of inorganic salt $CaCl_2$ on hydraulic conductivity and some physical, index properties of expansive soil (Soil M) obtained from Moragahakanda area. Also, the effects of $CaCl_2$ on Soil M amended by 5% (Soil M+5%) and 10% (Soil M+10%) bentonite, on hydraulic conductivity and the selected physical, index properties were investigated. The effects on physical, index properties were investigated by carrying out Atterberg Limits Tests, Modified Free Swell Index, Specific Gravity, Shrinkage Limit and Mechanical Analysis before and after samples consolidated under pressures of 50, 100, and 200 kPa were subjected to the permeation of 1M $CaCl_2$ solution under gradually increasing constant head of 50, 100 and 150 kPa, up to replacement of their void volumes by as much as 20 times to simulate long term contact.After long term contact of consolidated soil samples with $CaCl_2$ solution, the liquid limit value of Soil M, Soil M+5% and Soil M+10% is found to be decreased by 22%, 19% and 31% respectively, but only the plastic limit value of Soil M+10% was increased by 12% points. Although the LL and PI values were reduced after contact with $CaCl_2$ solution, the soil classification of Soil M (CH) and Soil M+5% (CH) did not change. However, Soil M+10% was reclassified from Clay of Very High Plasticity (CV) to Clay of High Plasticity (CH). Use of 1M $CaCl_2$ solution as the permeant fluid the hydraulic conductivity requirement was not satisfied in unamended and 5% bentonite amended expansive soils under the consolidation pressure of 50 kPa. Compared to the recommended hydraulic conductivity for Compacted Clay Liners (i.e. 1×10^{-7} cm/s), this all the candidate soils exhibited sufficiently lower hydraulic conductivities except M and M+5% soil under 50Kpa consolidation pressure. These test results indicated that $CaCl_2$ salt affected the plasticity of all the candidate soils even up to the extent of changing the soil classification for the expansive soil amended by 10% bentonite.*

Keywords: *Expansive soil, Bentonite, $CaCl_2$ solution, Hydraulic conductivity, Physical and index properties*

1. Introduction

The most important characteristic of a clay liner material in an engineered landfill is its ability to maintain low hydraulic conductivity during its service life in order to prevent leachate contamination of nearby water sources. Therefore, it is important to examine the hydraulic conductivity of potentialclay liner materialsespecially when subjected to long term contact with leachate. These clay liners are exposed to various types of chemical, physical and biological processes due to contact with leachate. Leachate includes different types of inorganic and organic compounds which affect the physical, index of clay liners that can affect the long-term performance of these clay liners as a barrier.In leachate, the $CaCl_2$ solutions exert the strongest effect on hydraulic conductivity of clay liner materials than anyotherchemicals. In this study, the characteristics of clay liner materials are examined when subjected to contact with $CaCl_2$.

2. Literature review

The hydraulic conductivity properties of the clay liners are greatly influenced by the expansive clay content of the soil. The higher clay content gives rise to higher swelling values and as a result hydraulic conductivity becomeslower and also this could be affected by the interaction with $CaCl_2$ over a long period of time (Jayasekera et al.).The $CaCl_2$solution exerts the strongest effect on hydraulic conductivity of clay liner materialsthan any other chemicals in leachate (QiangXue et al., 2012).To satisfy the required hydraulic conductivity criteria of clay liners,the clay content should be greater than 10%, the maximum particle size should be less than 75 mm,permeability shouldbe less than 1×10^{-9} m/sand plasticity index and liquid limit should bebetween 10%-65% and 35%-90% respectively (Declan O'Sullivan et al.). The properties of expansive soils can be improved when itis mixed with different percentages of bentonite.Laboratory experiments concluded that addition of 10% of bentoniteby weight will yield the most economical soil-bentonite mixture to build clay liners (Abeyrathne et al., 2012).

3. Materials and Methods

Unamended, 5% and 10% bentonite (by mass) amended expansive soil collected from Moragahakandawere selectedascandidate clay linermaterials to investigate their durability under long term contact with $CaCl_2$.

1M $CaCl_2$solution was used as the permeant liquid in hydraulic conductivity tests.

Variation of hydraulic conductivity of clay liner materials was investigated for long term, under different consolidation pressures (50, 100 and 200kPa) and under differenthydraulic pressures (50, 100 and 150 kPa).

Series of laboratory tests were carried out, in orderto examine pH, Electric Conductivity, Specific Gravity, Modified Free Swell Index, Liquid Limit, Plastic Limit, Particle Size Distribution, Swelling Index and Swelling Pressure on the selected clay liner materials before and after contact with $CaCl_2$ solution.

Figure 1: Hydraulic Conductivity Test Arrangement

4. Results and Discussion

The hydraulic conductivity of soil M, soil M+5% and soil M+10% under the consolidation pressure of 50 kPa, increased from 7x 10^{-8} cm/s to 11.8 x 10^{-8} cm/s, varied in between 5x 10^{-8} cm/s 22.5x 10^{-8} cm/s and decreased from 15.6x 10^{-8} cm/s to 1.8x 10^{-8} cm/s respectively. Under 100 and 200 kPa consolidation pressures, under permeation of 1M $CaCl_2$ solution, with all candidate soils satisfying the requirement of hydraulic conductivity (i.e. < 1 x 10^{-7} cm/s) for use in compacted clay liners. The hydraulic conductivity variation of all candidate soil samples are illustrated in the Figures 1, 2 and 3.

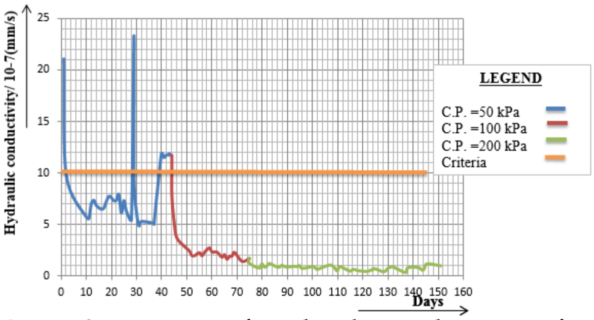

Figure 2: Variation of Hydraulic conductivity of Soil M with duration in days

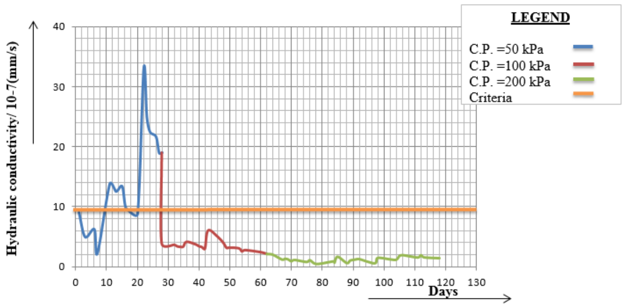

Figure 3: Variation of Hydraulic conductivity ofSoil M+5% with duration in days

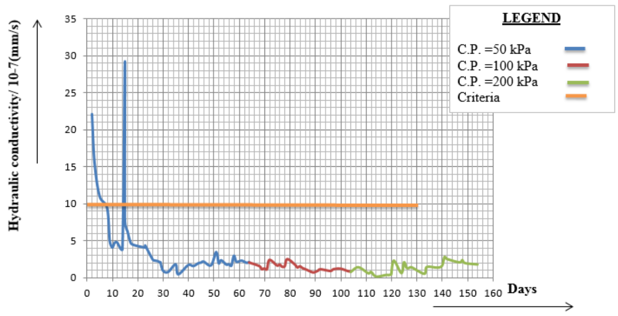

Figure 4: Variation of Hydraulic conductivity of Soil M+10% with duration in days

Figures obviously shows that hydraulic conductivity decreases with increase of bentonite percentage and consolidation pressures as well. Sudden hydraulic conductivity increment was observed in all consolidated samples, when the hydraulic pressure was increased. This may happen because of the voids and flow paths inside the consolidated sample may have been clogged by calcium chloride particles and during hydraulic pressure increment these clogged particles may have been washed out.

After the hydraulic conductivity tests, soil classification tests were carried out following the British Standards. The results of soil classification tests before and after contamination of $CaCl_2$ solution are shown in Table1 and Table 2 respectively.

Table 1: Physical, index of clay liners

Test	Soil M	Soil M + 5% bentonite	Soil M + 10% bentonite
Atterberg limits (%):			
1. LL	68	70	84
2. PL	23	25	25
Specific gravity	2.59	2.46	2.28
M.F.S.I	2.37	2.69	2.88

According to the test results, liquid limit, plastic limit and modified free swell index increases with the increase of bentonite content from 5% to 10%. According to the particle size distribution results, soil M contains 72% of fine and 51% of clay and according to the British soil classification system, soil M can be classified as Clay of High Plasticity (CH). All the clay liner requirements of a landfill (Discussed by Declan O'Sullivan et al.) other than the hydraulic conductivity get satisfied by these candidate expansive soil mixtures itself, before the contamination of $CaCl_2$ solution.

Table 2: Physical, index of clay liners

Test	Soil M	Soil M + 5% bentonite	Soil M + 10% bentonite
Atterberg limits (%):			
1. LL	53	57	58
2. PL	23	25	28
Specific gravity	2.49	2.32	2.39
M.F.S.I	1.74	1.32	1.03

Table 2 obviously shows the physical, index change of all candidate soil mixtures after the contamination of $CaCl_2$ solution. The Liquid Limit value of Soil M, Soil M+5% and Soil M+10% is decreased by 22%, 19% and 31% respectively, but only the Plastic Limit value of soil M+10% is increased by 12% and also the Specific Gravity and Modified Free Swell Index are decreased with the increase of percentage of bentonite up to 10%. According to the particle size distribution results, soil M contains 76% of fine and 68% of silt and according to the British soil classification system, soil M can be classified as Clay of High Plasticity (CH). Because of chemical reactions with $CaCl_2$ solution, soil particles may clogged and formed large particles result silt percentage very high as well as clay percentage very low. All the clay liner requirements of a landfill (Discussed by Declan O'Sullivan et al.) other than the clay content (i.e. Clay content should greater than 10%) get satisfied by these candidate expansive soil mixtures itself, after the long –term contamination of $CaCl_2$ solution.

5. Conclusion

1. Use of 1M $CaCl_2$ solution as the permeant fluid the hydraulic conductivity requirement was not satisfied in unamended and 5% bentonite amended expansive soils under the consolidation pressure of 50 kPa and compared to the recommended hydraulic conductivity for Compacted Clay Liners (i.e. 1 x 10^{-7} cm/s), this all the candidate soils exhibited sufficiently lower hydraulic conductivities except M and M+5% soil under 50Kpa consolidation pressure.
2. All the clay liner requirements of a landfill (Discussed by Declan O'Sullivan et al.) other than the clay content get satisfied by all candidate expansive soil mixtures itself, before and after the long-term contamination of $CaCl_2$ solution.
3. The effectiveness of the soil M as a liner material can be further enhanced by mixing with 5% -10% Bentonite (by mass).
4. Effects of $CaCl_2$ solution on expansive soil bentonite mixtures increase with increase of bentonite percentage.
5. It can be concluded that the expected characteristic and performance of expansive soil and expansive soil bentonite mixtures as clay liners in landfills, won't be affected by the $CaCl_2$ salt significantly.

6. Acknowledgments

This work was supported by the SATREPS project funded by Japanese International Cooperation Agency and Japan Science and Technology Agency.

References

Declan O'Sullivan and Paul Quigley, "Geotechnical Engineering and Environmental Aspects of Clay Liners for Landfill Projects", FehilyTimoney and Co. and Irish Geotechnical Services Ltd, pp. 1-11.

QiangXue, Qian Zhang and Lei Liu (2012), "Impact of High Concentration Solutions on Hydraulic Properties of Geosynthetic Clay Liner Materials", State Key Laboratory of Geomechanics and Geotechnical Engineering, Institute of Rock and Soil Mechanics, Chinese Academy of Sciences, Wuhan, China.

W. K. A. P.Abeyrathne, K. M. T.Pushpakumara, R. K. S. P.Ranaweera, Dr. N. H.Priyankara and Dr. A. M. N.Alagiyawanna (2012), "Development of a landfill clay liner using locally available expansive soil", Civil Engineering Research Exchange Symposium 2012, University of Ruhuna.

The Impact of Surrounding Intra-Plate Earthquakes on Sri Lanka

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Demand for earthquake resistant structures in Sri Lanka has increased with the rapid development and urbanization of the country. However, the lack of basic parameters required for earthquake resistant design considering local seismicity has created difficulties for structural engineers to design such structures according to international standards such as the Eurocode 8. Therefore, the need of establishment of such hazard parameters on a national basis has become an urgent need for seismic risk analysis and design in Sri Lanka. This research is carried out with the objective of studying the seismicity around Sri Lanka and to determine a design peak ground acceleration (PGA) at western coastline of Sri Lanka.

1. Introduction

Sri Lanka had been considered as an aseismic region in the past. Except for some tremors felt from time to time in some areas inside the country, the only recorded event of serious damage was in 1615 in which about 2000 people were reportedly killed in Colombo area due to a nearby earthquake. Sri Lanka is located well inside the Indo-Australian tectonic plate. Therefore, the impact of large inter-plate earthquakes which occur at the active tectonic plate boundaries, are negligible on Sri Lanka due to large distances involved, unless for a resulting Tsunami, which may have a devastating effect like in 2004. However, intra-plate earthquakes which occur at weak zones or active faults within the plate can't be ignored. Even though they are smaller in magnitudes to inter-plate earthquakes they can affect Sri Lanka due to smaller distances involved.

During last few years, the rapid development and urbanization which took place in the country has forced the community to consider disaster mitigation as an important aspect. Seismic risk was one of the major considerations and earthquake resistant designs got more attention of the designers and developers who wanted to ensure that their investments were safe. However, structural designers found difficulties in carrying out proper earthquake resistant designs because of the unavailability of basic seismic design parameters specified for the country. Even though they have managed to proceed with available foreign code of practices, the absence of local seismic risk parameters analysis for engineering purposes has made them to adopt a highly conservative attitude leading to uneconomical designs.

Since the year 2000, Geological Survey and Mines Bureau (GSMB) has started seismic monitoring in Sri Lanka by establishing three monitoring stations at Pallekale (PALK), Hakmana (HALK) and Mahakanadarawa (MALK), for monitoring the bedrock seismic response leading to the development of a database for seismic activity and response in and around Sri Lanka.

This research is carried out to study the seismicity around Sri Lanka and to propose reference peak ground acceleration for engineering designs in Sri Lankan west coastline. The study area was selected to be within 500 km boundary from Sri Lankan coastline (i.e. latitudes: 4^0-12^0N and longitudes: 77^0-85^0E) as the effect from intra-plate earthquakes beyond this boundary may have negligible impact on Sri Lanka due to the large distances involved. The seismic response within the country was considered out of scope of this study as the effects of topographical and geographical variations inside the country cannot be modelled by methodology used.

2. Literature Review

A very limited number of studies have been carried out on seismicity and seismic vulnerability of Sri Lanka. Based on micro-seismic monitoring studies during and after the Kotmale Reservoir Project, Fernando and Kulasinghe (1986) stated that Sri Lanka is not aseismic and seismically active at m=2 level though no definition is given for the scale adopted for definition of m. After, the occurrence of 1993 4.7 magnitude mid-sea earthquake 170 km west of the Colombo Abayakoon (1995) showed that peak ground acceleration (PGA) of 0.15 *g* can be expected in Colombo. In addition, Dissanayake (2005) discussed the possibility of formation of a new plate boundary about 500 km south of Sri Lanka.

3. Methodology

3.1 Earthquake Catalog

Earthquake catalog in the present was prepared using several sources: USGS, Abayakoon (1995), Vitanage (1995) and Fernando and Kulasinghe (1986). Earthquake records before 1900 were omitted in this study due to their uncertainty in measurements and records. Also the number of recorded events before 1900 is significantly less when compared with the same after 1900. Earthquake magnitudes less than 3.5 were not taken into account due to their negligible effect on engineered structures. The selected earthquake catalog is shown in Table 1.

3.2 Probability Analysis

Peak ground acceleration (PGA) for the reference return period is the most significant parameter for an earthquake resistant design in many code of practices (e.g. Eurocode 8). A probabilistic analysis was carried out to find the magnitude of earthquake in the defined seismic region with a 475 year return period which is recommended in Eurocode 8. Gutenberg-Richter (1944) recurrence law was used to determine the 475 year earthquake. Gutenberg-Richter recurrence law assumes a linear relationship between the magnitude of earthquake (M) and the logarithm of its annual rate of exceedance, i.e. the number of earthquakes equal or greater than M per year, λ_M (Eq 1).

$$\text{Log } \lambda_M = a - b\,M \qquad \text{Eq 1}$$

Constants 'a' and 'b' in the equation can be determined graphically using the past earthquake data. Since the inverse of λ_M is the return period, Equation 1 can be used to determine the magnitude of the earthquake for any given return period.

3.3 Ground Motion analysis

PGA values at western coastline for the design 475 year earthquake were obtained by using established attenuation relationships in the literature which gives the variation of PGA with distance from the epicentre. The observed ground motion data (Acceleration time series) inside Sri Lanka were obtained from the GSMB for two past earthquakes of magnitudes 4.7 and 4.2 which occurred in 2011 and 2012 in the study area south west of Sri Lanka. Recorded PGAs were as 0.000028g, 0.000045g, 0.000036g for 2011 event and 0.00015g, 0.000247g, 0.000087g for 2012 event at HALK, PALK and MALK stations respectively. Two attenuation relationships developed by Gitterman et al. (1993), Eq. (2) and Kun-Sung and Yi-Ben (2005), Eq. (3) were used to match the above observed PGA records.

$$\log PGA = -1.02 + 0.249\,M - \log R - 0.00255\,R \qquad \text{Eq 2}$$

$$\ln PGA = -0.852\,\ln X - 0.0071X + 1.027\,M + 1.062 \qquad \text{Eq 3}$$

where, M is the earthquake magnitude, and R and X are the epicentral and hypocentral distances to the site respectively.

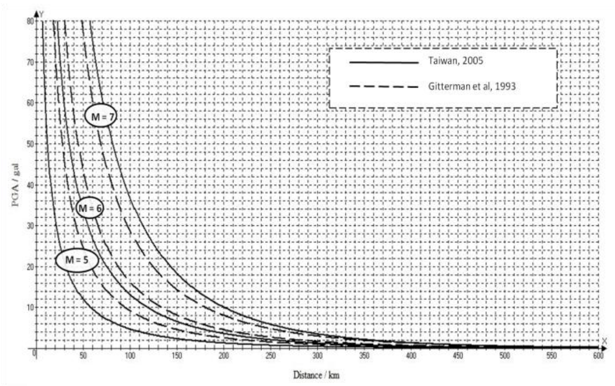

Figure 1: Comparison of Gitterman (Eq 2) and Taiwan (Eq3) attenuation relationships for different magnitudes

Table 1: Earthquake Catalog

	Date			Epicenter		Magnitude	Source
	Year	Month	Date	Latitude (N)	Longitude (E)		
1	1900	2	8	10.70	76.70	6.0	USGS - Abayakoon (1995)
2	1934	1	11	9.45	77.82	4.6	Menon – Jaiswal
3	1938	9	10	7.50	79.00	5.8	USGS - Abayakoon (1995)
4	1939	8	7	4.00	77.50	5.6	USGS - Abayakoon (1995)
5	1944	2	29	9.80	78.10	4.6	Menon - AERB
6	1953	1	29	6.70	82.50	5.0	Fernando & Kulasinghe (1986)
7	1953	2	25	8.73	77.70	4.2	Menon - AERB
8	1956	12	15	6.50	78.00	5.0	USGS - Abayakoon (1995)
9	1957	10	4	10.80	78.60	3.8	Menon - AERB
10	1959	12	17	11.70	78.10	4.3	USGS - Abayakoon (1995)
11	1961	6	13	8.70	83.20	4.0	Fernando & Kulasinghe (1986)
12	1966	1	8	11.60	84.90	5.2	USGS - Abayakoon (1995)
13	1972	7	29	11.00	77.00	5.0	USGS - Abayakoon (1995)
14	1972	11	24	11.67	85.34	5.2	Vitanage (1995)
15	1973	2	23	9.80	83.50	4.7	Menon - AERB
16	1973	8	30	7.10	84.30	5.9	USGS
17	1979	2	16	10.50	77.00	3.8	Menon - AERB
18	1986	10	12	4.00	85.00	4.7	USGS - Abayakoon (1995)
19	1987	10	31	7.20	84.40	4.5	USGS
20	1987	12	18	10.21	82.44	4.4	Menon - AERB
21	1988	6	7	9.81	77.21	4.2	Menon - Jaiswal
22	1993	12	6	6.82	78.30	5.2	USGS
23	1998	11	17	5.51	77.78	4.5	USGS
24	2001	9	25	11.96	80.21	5.2	USGS
25	2005	7	7	4.19	84.78	4.6	USGS
26	2007	7	18	6.47	84.40	5.2	USGS
27	2009	4	15	6.80	82.62	4.5	USGS
28	2010	7	25	6.60	76.78	4.0	USGS
29	2011	11	19	3.92	79.02	4.7	USGS
30	2012	7	6	5.14	77.02	4.2	USGS

The two selected attenuation relationships are plotted in Figure 1. The two relationships give similar estimations for PGA values at higher M (6~7) which is the range of importance for the design. However, as Eq 3 is giving slightly higher estimation than Eq. 2 (Fig 1), it was selected to estimate the PGA values.

In order to determine a maximum PGA value using above relationship, a specific location of the epicentre has to be assumed. This location should be realistic but at the same time be as close as possible to the coastline to produce the highest PGA at the coastline. The closest and most probable location for the 475 year earthquake was assumed as at the location of the closest earthquake of large magnitude in south west area as given by the earthquake catalog. This gives a magnitude of 5.8 based on the earthquake recorded in 1938 at a distance of 90 km away from Negombo coastline.

4. Results and Discussion

Spatial distribution of earthquakes in the study area (Fig 2) indicates an alignment in NE-SW direction. Except the earthquakes in South Indian land mass all the other earthquakes are lying on a NE – SW band. This may be due to the geomorphological history of the area influenced by interplate and intraplate plate tectonics. This is an aspect which should be studied by geologists and seismologists.

Probability analysis performed gave the 475 year earthquake as of 7.4 magnitude for the study area. The reliability of this magnitude calculation is dependent on the regional geology as tectonic activity controls the amount of energy which can be released; this will introduce an upper bound cutoff limit to the maximum possible magnitude. Since this area doesn't have any such established cutoff, for the time being, 7.4 magnitude which is not unusually large in comparison to those given in the literature for similar tectonic regions elsewhere in the world had been chosen for further analysis.

PGA for the 475 year 7.4 magnitude earthquake was estimated up to the south west coast using Eq 3 at 90 km distance and a PGA value of 0.067 g (65 gal) obtained. This value can be used as the design PGA for most of the buildings in Colombo and nearby west coast.

The 475 year return period event is very much dependent on earthquake catalog. Had all the earthquake records back to 1063 A.D. were considered, the design earthquake will have a lesser magnitude such as 6 which can

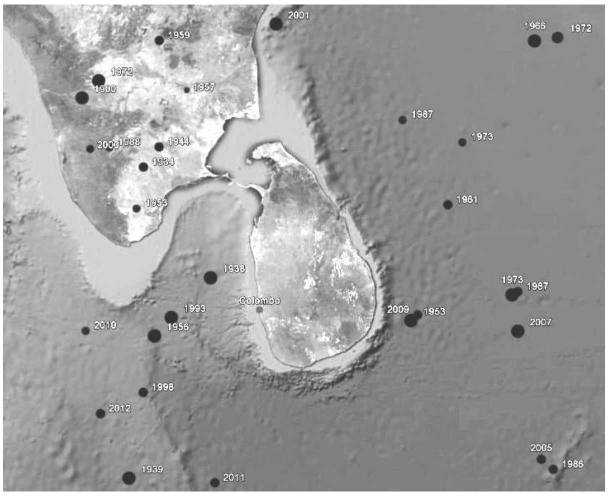

Figure 2: Spatial distribution of the earthquakes

be assumed to take place very close to the boundary of the continental shelf, i.e. about 15 km from the land. Abayakoon (1995) analysed such a scenario to obtain a PGA of 0.15 g for the design PGA.

5. Conclusions and Recommendations

The current study demonstrates that Sri Lanka is not totally safe from seismic hazard, particularly against intra-plate earthquakes. Western and Southern coastal lands are particularly vulnerable. The cities such as Colombo, Negombo, Galle etc. may be exposed to a PGA of 0.067 g considering a 475 year return period earthquake of 7.4 magnitude occurring in sea 90 km away from the western coast. Therefore, proper seismic designs for high rise buildings and post disaster structures would be essential. For structures with lesser importance seismic detailing or use of a static acceleration coefficient may be sufficient. Geological studies about probable earthquakes around Sri Lanka particularly around western and eastern coastline would be helpful in improving the understanding of seismic hazard.

References

[1] Abayakoon S.B.S., 1995. *Seismic Risk Analysis of Sri Lanka*. Journal of the Geological Society of Sri Lanka, Vol 6.

[2] Vitanage P.W., 1995. *Seismicity – Neglected Aspects of Sri Lankan Land Slide Studies*. Proceedings of the Symposium on Landslide Hazard Mapping, Colombo, Sri Lanka, 31-40.

[3] Fernando M.J. and Kulasinghe A.N.S., 1986. *Seismicity of Sri Lanka*. Physics of the Earth and Planetary Interiors, 44: 99-106.

[4] Dissanayake C.B., 2005. *A New Plate Boundary near Sri Lanka; Implications for Future Geo Hazards*. Journal of the National Science Foundation of Sri Lanka, Vol 33(1).

[5] Gitterman Y., Zaslavsky Y and Shapira A, 1993. *Analysis of Strong Motion Records in Israel*. Proceedings of the 17th Regional European Seminar on Earthquake Engineering, Haifa, Israel.109-118.

[6] Kun-Sung Liu and Yi-Ben Tsai, 2005. *Attenuation Relationships of Peak Ground Acceleration and Velocity for Crustal Earthquakes in Taiwan*. Bulletin of the Seismological Society of America, Vol. 95, No. 3, 1045–1058.

[7] Gutenberg B. and Richter C.F., 1944. Frequency of Earthquakes in California. Bulletin of the Seismological Society of America, Vol. 34, No. 4, 1985–1988.

[8] USGS web : http://earthquake.usgs.gov/earthquakes/eqarchives/epic/ (date accessed 28-05-2013)

Development of a Method for Stone Column Design

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Abstract: Stone columns are a method of soft ground improvement where granular columns of greater strength and stiffness are introduced to a soft ground. If cost effective designs are to done various techniques are to be used to incorporate the necessary design concepts with the available software. In this research a method was developed with Geostudio 2004 Slope/W software to simulate the process of design of stone columns. The 'Spencer' method was used to analyse the stability. Auto locate approach was used to locate the critical failure surface since it can accommodate non circular failure surfaces as well. The stone columns that are placed in a grid pattern in plan were idealized as a series of strips in the plane strain analysis. Stone columns will carry a greater portion of the load applied - a phenomenon referred to as stress concentration. A technique was developed in a limit equilibrium framework to incorporate this effect and studies were done both with and without considering the effect of stress concentration.

It was shown that when stone columns were modeled as stiffer strips the factor of safety increased significantly from the analysis done using average shear strength parameters. When the stress concentration effect was accounted, factor of safety increased further. This represents the actual situation more closely and design becomes economical. By performing sensitivity analyses it was shown that the developed models perform appropriately. With very high embankments, the loading will have to be applied in stages and the application of the staged loading with strength gains at every stage was also modeled. The influence of stone columns in the reduction of the settlement was also illustrated.

1. Soft ground Improvement

When it is necessary to construct an embankment fill or similar structure on top of the weaker soil strata one approach is to transfer the load to a deep level by piles. This is very costly especially if the load is applied over a large area as in the case of a highway. In such cases use of a method of ground improvement where the strength and stiffness of the soft soil is improved would be more economical.

In general, soft ground includes soft clay, silty clay, peaty clay and peat which have high moisture contents. They are with very low shear strength and very high compressibility.(Barksdale, R.D. et el.,1983)When large loadings are to be applied on such soils appropriate improvement techniques are to be applied to prevent shear failures during the construction and excessive settlements during the design life. Ground improvement level required will depend on the purpose of utilization.

Following techniques can be used to improve soft clayey sub soil conditions. . (Bergado, D.T. et al.,1996)

- Pre consolidation by preloading with or without vertical drains
- Vacuum consolidation
- Dynamic compaction and replacement
- Deep mixing with a cementing agent

With the application of these ground improvement techniques, soil strength will improve and of total and differential settlements in service will be reduced. It would also reduce the construction time and subsequent maintenance cost.

When the thickness of the soft layer is high and/or load improved by embankment load is high, preloading with vertical drains will take a long time as the load will have to be applied in stages to avoid shear failure during construction.

In such instances, stone columns can be used as a cost effective solution. Since stone columns are done with filling granular material under compacted conditions, it will provide a drainage path for dissipation of pore water pressure, in addition to providing a stiffer intrusion. With the quick dissipation of pore water pressure through reduced drainage paths the time required for construction will be reduced.

With the installation of stone columns, ground becomes a composite section and the applied load will not be shared equally by the soft soil and the stone columns. Due to high strength and stiffness of the granular material in the stone columns, it will take a greater percentage of the applied load. This phenomenon is referred to as stress concentration. Due to stress concentration load on soft soil will be reduced and thus possibility of shear failure will be reduced.

There are difference techniques for construction of stone columns. (Bergado, D.T. et al.,1996)

- Vibro – compaction method
- Top feed vibro displacement method
- Bottom feed vibro displacement method
- Top feed vibro replacement method
- Vibro – composer method
- Cased borehole method

Other than above methods sand columns with internally reinforced with horizontal wire meshes made of plastic, steel and aluminum can be used. (Tahar Ayadat. et el.,2007)

There are different methods for the design and analysis of stone columns in the limit equilibrium approach. Basic objective of this research is to develop a method for analyzing stone columns which closely represent the realistic ground situation accounting for effect such as stress concentration. The method is developed using GeoStudio 2004 SLOPE/W software.

2. Method of Analysis of Stone Columns

In a particular construction of stone columns, they are installed in a grid pattern. Diameter and spacing are the critical parameters. Area replacement ratio is governing by stone column spacing and diameter. Since factor of safety of improved ground surface depend upon the area replacement ratio, appropriate stone column spacing and diameter should be selected depending on the super structure and loading behaviors.

$$a_s = \frac{A_s}{A_s + A_c}$$ (Barksdale, R.D et el.,1983)

Where,

a_s = Area replacement ratio
A_s = Cross sectional area of stone column
A_c = Area of surrounding soil

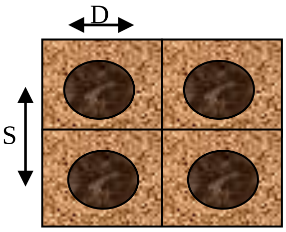

Figure 1 : Plan view of Ground with Stone Columns

$$A_c = S^2 - \frac{\pi D^2}{4}$$

When a load is applied on ground which improved by installation of stone column, they will carry a greater share of the load, a phenomenon referred to as stress concentration.

Stress concentration factor ; $n = \frac{\sigma_s}{\sigma_c}$

Stress concentration factor basically depend on the ratio of modulus of elasticity values of the soft soil and stone column material. In general, stress concentration factor lie in between 2 and 5. It can be calculated by equation;

$$n = a + b\left(\frac{E_s}{E_c}\right.$$

(BBA Technical Note, 2008)

Where, a = 0.78
b = 0.22
E_s = Modulus of elasticity of stone column material
E_c = Modulus of elasticity of existing ground material

Different methods of analysis currently in use are;

- Average shear strength method
- Consideration as composite ground without stress concentration Effect
- Considering as composite ground with stress concentration Effect

In the first approach, averages shear strength parameters (c_{av},ϕ_{av}) were determined for the composite ground based on the area replacement ratio.
$C_{avg} = (1 - a_s)C$

$\phi_{avg} = tan^{-1}(\mu_s a_s tan\phi_s)$

Where, μ_s=$\left(\frac{n}{1+(n-1)a_s}\right.$

In the second approach, the stone columns were idealized to a strip of equivalent width in a plane strain idealization, but without considering the stress concentration effect. In the third approach, the stress concentration effect is also considered.

3. Development of the Excel Program

With the standard procedures available in SLOPE/W, to simulate the construction of an embankment on existing ground, the zone of soil representing the embankment is placed on the existing ground. The composite ground made with installation of stone columns could be idealized in plane strain conditions with a series of strips. The width of a strip depends on the spacing and diameter of stone columns. Thus the coordinates of the zones in the form of strips should be entered into the software. This is a very tedious task and the difficulty was overcome by writing an excel program to derive the necessary coordinates. Then the model in slope/W, was obtained by copying and pasting the coordinated in slope/W data entry phase. (Figure2 and Figure 3)

With the stress concentration effect the loading on strips of stone columns would be greater than the loading on strips of soft clay. This will not be simulated by an embankment of uniform density above the ground. Thus the additional loading intensity on the stone column was simulated by dividing the embankment also into strips. The strips over the stone columns will be assigned a greater density than the strips on soft clay. The density values depend on the stress concentration factor. The values are computed as:

γ of strips on stone columns =$\mu_s\ \gamma$

γ of strips on soft clay =$\mu_c\ \gamma$

Where, μ_s=$\left(\frac{n}{1+(n-1)a_s}\right.$ and $\mu_c = \frac{1}{1+(n-1)a_s}$

The shear strength parameters of embankment material were maintained to be same as the actual shear strength parameters. The embankment thus generated in the slope/w is presented in Figure 4.

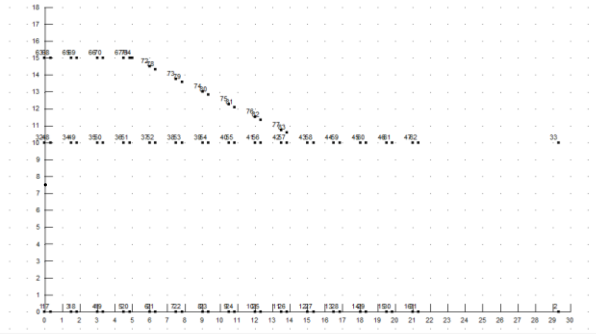

Figure 2- Coordinates obtained from Excel program

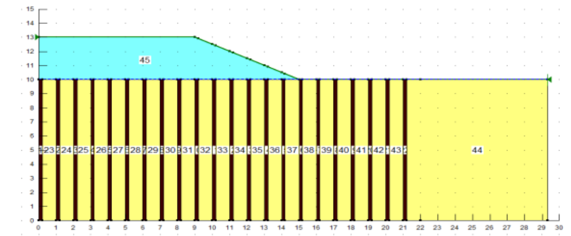

Figure 3- Representation of embankment fill without stress concentration effect

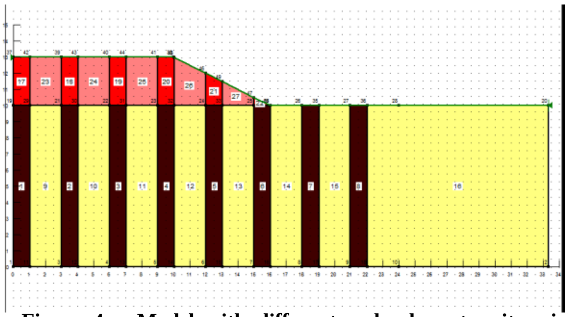

Figure 4 – Model with different embankment unit weights to account stress concentration

4. Results of Analysis Incorporating Stress Concentration

The factor of safety values computed for the construction of a 5m high embankment on a 8.5m thick layer of peaty clay of c_u=10kN/m^2 by the three different approaches; average shear strength method, strips without stress concentration and strips with stress concentration are compared in Table 2. The comparison was done for stone columns of diameter 0.5m done at a spacing of 1.0m. The third method with the incorporation of the stress concentration effects is the one most close to the reality.

The comparison of the results shows that the averages shear strength method grossly under estimate the factor of safety in the construction. It does not take into account the actual mechanism of composite ground behavior. The reduction of FOS is about 50%. When the composite effect of the ground is modeled by strips but without stress concentration the FOS increase. With the incorporation of the stress concentration effect in the manner described the FOS increase further. This is the closest to the reality and the reduction of Factor of Safety for not accounting for stress concentration is around 12%. Atypical failure surface is presented in Figure 5.

Thereafter parametric studies were done with the third approach changing the friction angle of the stone column material and varying the diameter and spacing of stone column material for a stone column material of ϕ =34 for the construction of an embankment of height 5m. The results are summarized in Table2. For each fill height FOS increased with the friction angle of stone column material. Thereafter the effects of the change of the diameter and spacing of the stone columns were studied. The results presented in Table 3 illustrated that the FOS increased with the diameter and decreased with the spacing of the stone columns.

Table 1- Comparison of FOS with Different methods

$\emptyset$	FOS by Avg. Shear strength method	FOS with no stress concentration	FOS with stress concentration (Developed method)
32^0	1.026	1.472	1.649
34^0	1.067	1.525	1.700
36^0	1.103	1.578	1.754
38^0	1.150	1.627	1.811
40^0	1.187	1.681	1.872

Table 2–FOS variation according to fill height and $\emptyset$

Fill Height	Factor Of Safety			
	$\emptyset = 32°$	$\emptyset = 34°$	$\emptyset = 36°$	$\emptyset = 38°$
3m	1.954	2.015	2.078	2.138
5m	1.649	1.700	1.754	1.811
6m	1.541	1.592	1.646	1.703

Table 3- FOS variation according to spacing and diameter of stone columns

Spacing	FOS for	
	0.5m Diameter	0.8m Diameter
1m	1.7	1.748
1.5m	1.489	1.713
1.8m	1.435	1.658

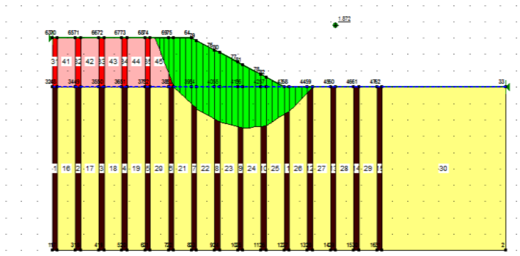

Figure 5 – Typical failure surface for a model

4.1. Simulation of Staged Loading

When the height of the embankment is high it will be constructed in stages. The soft soil will experience some consolidation and gain in shear strength in between the stages of construction. It can be calculated as; $\Delta Cu = U\alpha\Delta\sigma$

Where,

$\Delta Cu = increment\ of\ cohesion$
$U = Degree\ of\ consolidation$
$\alpha = Factor\ in\ between\ 0.2 - 0.3\ for\ most\ of\ soils$
$\Delta\sigma = increment\ of\ stress$

The results of an analysis done with the simulation of construction in stages are presented in Table 4.

Table 4 – Cohesion and FOS variation according to fill height

	Cohesion (kPa)	Fill height	FOS
Stage 1	C_u=10	2m	2.792
Stage 2	C_u=13.43	4m (2m consolidated)	2.096
Stage 3	C_u=16.85	6m (4m consolidated)	1.859
Stage 4	C_u=19.59	6m (6m consolidated)	1.864

4.2. Effect on Settlement due to Stone Columns

With the use of stone columns, stress transferred to soft clay will reduced and the resulting settlement in soft clay is reduced. An analysis done with data obtained from a section of Colombo-Katunayake Expressway (CKE) is presented here. Basic data are as fallows.

Soft clay layer thickness (m) =8.5m

$\frac{Cc}{1+e_0}$ of soft peat clay layer $=\frac{4.156}{1+5.67}$=0.623

E ratio between soft soil and S.C material = 30

Stress concentration factor = 7.38

Area replacement ratio =0.196

(0.5 m diameter Columns at 1.0m spacing)

Embankment height =6.4m

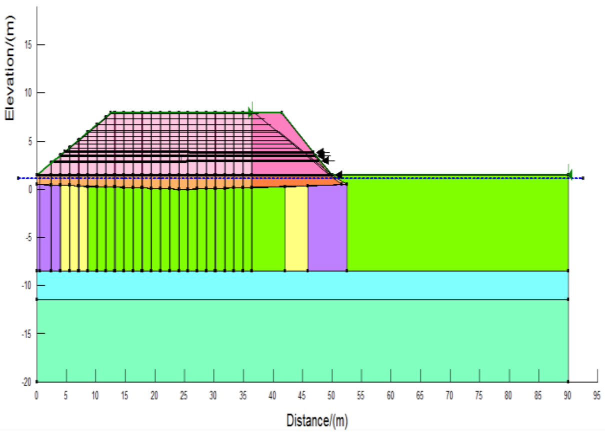

Figure 6 : Section of CKE Project

Settlement with stone columns = 437.9mm
Settlement without stone columns = 1090mm

5. Conclusion

Stone columns can be used as a cost effective construction procedure. With the installation of stone columns ground will exhibit a composite behavior and shear failure is prevented. Post construction settlements will be minimized. Due to higher rate of dissipation of pore water pressure through the drainage paths created by the installing stone columns, the construction speed can be increased.

The analysis of construction of embankment on ground improved by stone column installation could be done using the Finite Element technique simulating all the existing conditions. Commercial software such as PLAXIS can be used in the analysis.

When the analysis is done in a limit equilibrium framework simplified procedures such as average shear strength method is used. In this research a methodology was developed to conduct an analysis with a limit equilibrium approach but closely simulating the real situation, with the Geoslope SLOPEW software. A method was developed to simulate the effect of composite ground and account for the stress concentration effect. Input data necessary for the modeling were generated through an EXCEL program. The model developed was verified through sensitivity analysis.

The results indicated that analyses without accounting for these effects were quite conservative.

Ideally this could be verified by comparing with the results of a finite element simulation.

References

[1]D.T Bergado, L.R Anderson, N. Miura, A.S Balasubramaniam . (1996). *Soft Ground Improvement.* American Society of Civil Engineers.

[2]Tahar Ayadat, A del M. Hanna, Ammer Hamitouche. (2007). Soil improvement by internally reinforced stone columns. *Ground Improvement* , 55-63.

[3]R. D Barksdale, R. C Bachus. (1983). *Design and Construction of Stone Columns Volume ii.* Georgia: School of Civil Engineering.

[4]BBA Technical Note. (2008, April).

[5]Slope/W (2004) Manual.

IMPACT OF MUNICIPAL SOLID WASTE ON GEOTECHNICAL CHARACTERISTICS OF SOIL IN DRY ZONE OF SRI LANKA

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Abstract

Open dumping is the most commonly practiced method by local authorities of Sri Lanka for disposing solid waste, which creates considerable nuisance and environmental problems. Under the open dumping practice, solid wastes are disposed haphazardly. It is believed that due to deposition of solid waste on the ground, geotechnical characteristics of soil may change with time. In order to study the impact of solid waste on geotechnical characteristics of soil, a series of laboratory tests were conducted on undisturbed soil samples collected from Hambantota dump site. Based on the results, it was realized that there is a significance influence of solid waste on the change of geotechnical properties of underneath soil. Further, it was found that the influence of solid waste on soil is decreased with the depth and is increased with the aging.

1. INTRODUCTION

Municipal Solid Waste (MSW) disposal is becoming a major problem in the world today. Especially, developing countries like Sri Lanka, there are no any proper way of disposing solid waste. Open dumping is the most commonly practiced method for disposal of solid waste, which creates considerable nuisance and environmental problems.

MSW is a complex refuse consisting of various materials with different properties. Some of the components are stable while others degrade as a result of biological and chemical processes [1]. Leachate resulting from these solid wastes is a hazardous pollutant for soil underlying and subsequently ground. Leaching of leachate and heavy metals into the soil leads to soil and ground water contamination.

Due to rapid urbanization, there is a scarcity of good lands in the country. As such, there is a trend to utilize former solid waste dump sites to construct buildings in the city centers. Therefore, it is a need of the hour to evaluate impact of solid waste on the geotechnical characteristics of soil underneath in these dumps sites.

2. METHODOLOGY

2.1 Sample collection

Since, Hambantota is a rapidly urbanization area in Sri Lanka, waste dump site at Hambantota municipal council was selected as the study area. The dump site is located along Hambantota-Mattala main road close to newly constructed administrative office complex. The dump site is about 1 acre. The dump site can be categorized into two zones namely old waste and new waste. The old waste is more than ten years old whereas new waste is less than two years old. The site plan of the dump site is shown in Figure 1.

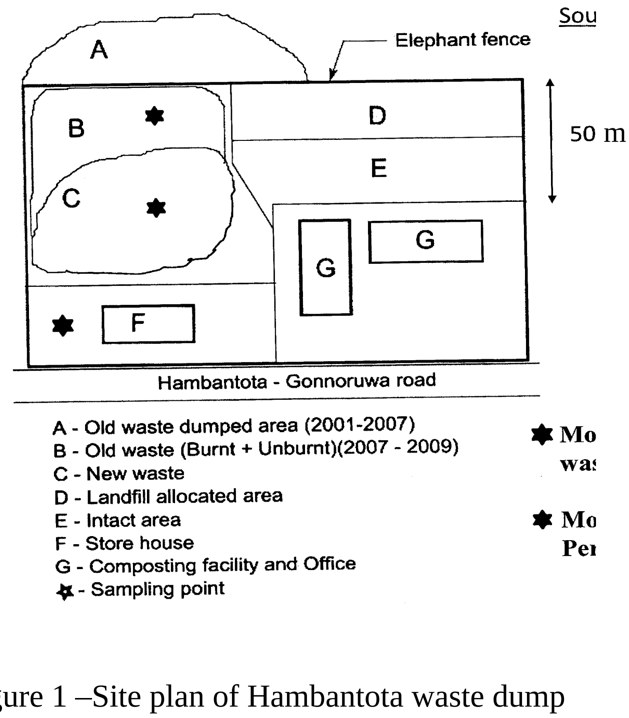

Figure 1 –Site plan of Hambantota waste dump site

To investigate the effect of MSW on soil, undisturbed soil samples were collected from 2 locations at old waste area (Old 1 and Old 2) and 1 location at new waste area (New). The sample size was 300 mm x 300 mm x 300 mm, box type. At each location, box samples were collected at levels of 0.5 m and 1.5 m from bottom of the waste layer. At each level, 2 number of box samples were collected. In addition, to compare the geotechnical characteristics of contaminated soil with that of uncontaminated soil, 2 number of uncontaminated box samples (Intact) were also collected at a depth of 0.5 m as shown in Figure 1. Therefore, altogether 14 number of undisturbed soil samples were collected for this research study.

2.2 Determination of geotechnical properties

Physical properties as well as shear strength parameters of collected soil samples were determined in the laboratory. In addition, in order to investigate the effect of leachate on soil, pH value and Electrical Conductivity (EC) were also determined [2]. All tests were conducted in accordance with BS 1377 (1990) [3].

3. RESULTS AND DISCUSSION

3.1 Physical properties

Variation of physical properties of soil is summarized in Table 1.

Moisture Content

Based on results presented in Table 1, it can be seen that natural moisture content of soil in the old dump area is much higher than that of uncontaminated soil. As intact samples were collected on a rainy day, moisture content of soil in the new dump area is slightly lower than that of uncontaminated soil. It is very clear that, moisture content deceases with depth irrespective of location. Since natural ground is covered with MSW, thereby preventing direct evaporation of moisture from the soil, contaminated soil is generally expected to be damper than that of uncontaminated soil.

Specific Gravity

It can be noted that, generally, specific gravity of contaminated soil is lower than that of uncontaminated soil. Due to contamination of soil with fresh leachate, specific gravity of soil in new dump area is much lower than that of old dump area. Further, the specific gravity of soil has been gradually increased with the depth, which implies that effect of leachate on soil is gradually diminishing with depth.

Table 1- Physical properties of soil

		Moiture Content (%)	Specific Gravity	Plasticity Index	Optimum Moisture Content (%)	Maximum Dry Unit Weight (kN/m^3)	Sand Content (%)
Intact		18.64	2.61	21	16.5	18.34	84.3
New	0.5m	14.18	2.29	35	15.2	18.44	96.3
	1.5m	6.46	2.50	8	13.2	19.35	86.2
Old - 1	0.5m	32.92	2.50	23	15.6	18.36	80.0
	1.5m	19.12	2.51	7	17.0	18.23	78.5
Old - 2	0.5m	21.24	2.67	10	14.3	18.69	82.7
	1.5m	11.56	2.47	5	16.2	17.27	75.6

Particle size distribution

Particle size distribution of all soil samples are presented in Figure 2. Based on the test results, it was realized that soil is basically consists of sand irrespective of contamination. Further, soil at 1.5 m depth in new dump area is almost same as that of intact soil. This implies that the effect of new waste on particle size distribution at greater depth of the ground is negligible. Since MSW in Hambantota consists of large percentage of sand [2], sand content of soil at 0.5 m depth of new dump area is larger than that of others. This implies there is a considerable influence of new MSW on the particle size distribution of soil close to the waste layer. Even though, there is no significant influence of MSW on the fine content of soil, due to decomposition of solid waste, fine content in old dump area is slightly higher than that of new dump area. Since fine particles have more affinity for water, soils in old dump area have higher moisture content than others.

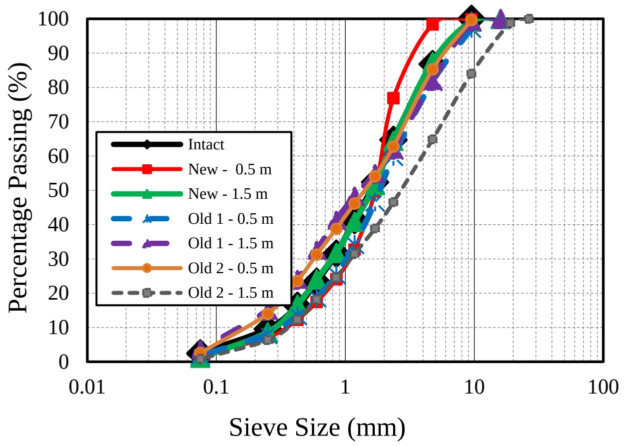

Figure 2 – Particle size distribution of soil samples

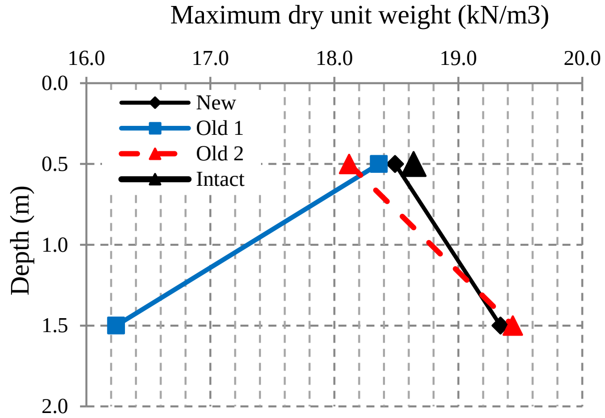

Figure 3 – Variation of maximum dry unit weight with depth

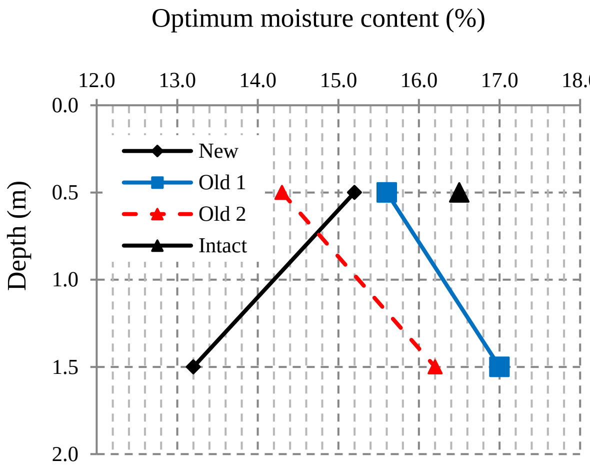

Figure 4 – Variation of optimum moisture content with depth

Consistency limits

As illustrated in Table 1, generally, soil at old dump area has relatively lower consistency limits than that of uncontaminated soil. However, it was noted that soils in old dump area consist of more fine content. Based on these evidences, it is very clear that due to long term effect of leachate interaction, soil becomes more non-plastic. It can be concluded that fines resulting from microbial decomposition are non-plastic.

In contrary, soil at 0.5 m depth of new dump area has higher consistency limits than that of uncontaminated soil. Due to the influence of fresh leachate, soil under the new waste becomes more plastic. It can be seen that soil becomes more non-plastic with depth irrespective of level of contamination.

Compaction characteristics

The variation of maximum dry unit weight and optimum moisture content over depth for all soil samples are illustrated in Figure 3 and Figure 4 respectively. Based on the results, it can be seen that maximum dry unit weight has been decreased with the contamination. Due to long term interaction of leachate with soil, the reduction of maximum dry unit weight of soil in old dump area is greater than that of new dump area. Further, it can be noted that maximum dry unit weight has been increased with depth irrespective of the level of contamination. Similarly, optimum moisture content has been decreased with the contamination.

3.2 Chemical properties

Variation of chemical properties such as pH value and Electrical Conductivity (EC) of soil are summarized in Table 2. It can be noted that contaminated soil has a high EC value than that of uncontaminated soil. Especially, soil immediately under the new waste has a very high EC value than that of other areas. This is a clear indication that ionic concentration in soil immediately under the new waste has significantly increased due to infiltration of fresh leachate. Due to bio-chemical reactions of leachate with time, ionic concentration in soil under the old waste is getting reduced, thus EC value becomes less when compared with that of new waste area. Further, it can be noted that EC value has been reduced with the depth irrespective of level of contamination.

Table 2 – Chemical and shear strength parameters of soil

		Undrained Shear Strength *(kN/m²)*	EC *(μc/cm)*	pH
Intact		187	73	7.53
New	0.5 m	106	634	7.82
	1.5 m	200	434	6.32
Old - 1	0.5 m	59	269	5.93
	1.5 m	116	439	7.90
Old - 2	0.5 m	41	566	7.44
	1.5 m	116	262	7.70

Test results indicate that uncontaminated soil is slightly alkaline (pH>7.0). Even though, there is no any clear relationship between pH value and level of contamination, it can be noted that alkalinity of the soil has been slightly changed to acidic when soil contaminated with leachate.

3.3 Shear strength

Undrained shear strength of soil were determined in the laboratory using triaxial Unconsolidated Undrained (UU) tests and results are summarized in Table 2 and Figure 5. According to the test results, it can be observed that undrained shear strength of soil immediately under the old waste is decreased by about 70% due to long term contamination. And under the new waste, shear strength of soil is decreased about 43% due to contamination with fresh leachate. This is a clear indication of the effect of leachate and decomposition of solid waste on shear strength of natural ground.

As shown in Figure 5, it can be seen that undrained shear strength has been increased with depth irrespective of level of contamination. The shear strength of soil at greater depth in new dump area is almost equal to that of uncontaminated soil.

4. CONCLUSIONS

Following conclusions can be made based on laboratory experiments of this research study.

- Due to leachate contamination and decomposition of solid waste with time, physical properties of soil under the waste has been changed significantly.
- The influence of leachate on soil decreases with depth.
- There is a significant influence of fresh leachate on the change of chemical properties of soil.
- Shear strength of soil has been significantly reduced due to long term soil-leachate interaction.

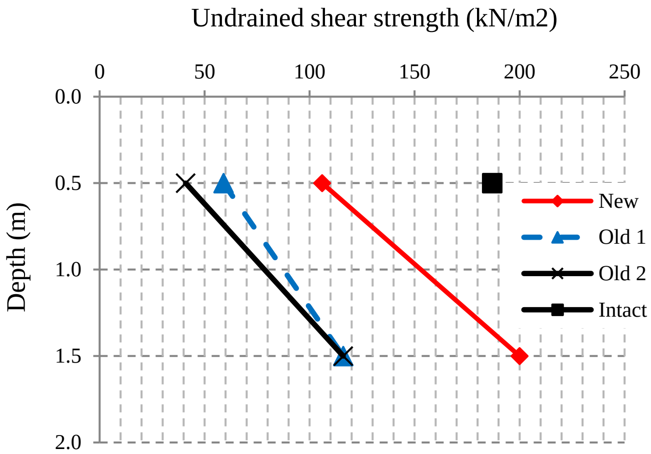

Figure 5 – Variation of undrained shear strength with depth

5. ACKNOWLEDGMENT

This study was supported by SATREPS project funded by Japanese International Cooperation Agency (JICA) and Japan science and Technology Agency (JST).

6. REFERENCES

[1]Alhassan M., “Effect of Municipal Solid waste on Geotechnical properties of soil”, *International Journal of Environmental Science, Management and Engineering Research*, Vol. 1, no 5, pp. 204 - 210, Sept. - Oct., 2012.

[2]Randeniya C. and Fernando S. D. D. S., “Change of Geotechnical Characteristics of Underneath Soil due to Deposition of Solid Waste”, B.Sc Engineering Final year project, University of Ruhuna, 2013.

[3]BS 1377, “Methods of testing soil for Civil Engineering purposes”, 1990.

Possible use of Paddy Husk Ash in Improvement of Engineering Characteristics of Peaty Clay

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Abstract: Preconsolidation by preloading with or without vertical drains or vacuum consolidation has been used to improve the engineering characteristics of peaty clays encountered in highway projects in Sri Lanka. The main drawback in these techniques is the long time duration required in the case of thick deposits of extremely soft peaty clays.

Mixing with cement as an alternate improvement technique had been conducted at University of Moratuwa successfully. It is found that cement percentage of the order of 25% is required to obtain a sufficient level of improvement. Considering the high cost involved in mixing cement, this research attempts to study the use of another pozzolonic material-paddy husk ash (PHA) - that is widely available as a waste product.

In this research, to assess the improvement of strength and compressibility, tests were carried out on samples of ; peaty clay, peaty clay mixed with 20% PHA, peaty clay mixed with 20% of wet weight of peat. Samples have been remoulded at their saturated moisture content and left under submerged conditions for 28 days to allow for the completion of pozzolonic reactions. Improvement of engineering characteristics of the samples was assessed by conducting consolidation tests, unconsolidated undrained triaxial shear tests and laboratory vane shear tests.

Keywords: Peaty clay, PHA, Cement, ground improvement

1. Introduction

Construction industry in Sri Lanka face the challenge of conducting engineering projects in sites underlain with thick layers of peaty clay. Peaty clay gives rise to enormous geotechnical engineering problems due to; high compressibility and low shear strength. In Sri Lanka several soil improvement techniques based on consolidation such as; preloading with vertical drains, vacuum consolidation combined with preloading been developed and used successfully in practice. The major drawback in above methods is the long duration required.

If the peaty clay is mixed in-situ with an appropriate binder such as cement, the hydration reactions and the subsequent pozzolonic reactions will cause a major change in the microstructure and considerable gain in strength and stiffness would be achievable within a relatively shorter period of time such as 28 days.

When pore water in the soil reacts with the cement, hydration occurs rapidly and primary cementitious products such as Calcium Silicates, Calcium Aluminates and Hydrated Lime are formed. These particles bind together to form hardened skeleton matrices which enclose unaltered soil particles. Hydration of cement causes a rise of pH value of pore water and Soil Silica and Alumina will be dissolved in this strong bases. These products react with Calcium ions liberated in the process to form insoluble secondary cementitious products. This secondary reaction is known as the pozzolonic reaction. Since some of the calcium ions liberated will be used to satisfy the high exchange capacity of the organic matter, a greater amount of cement is required to stabilize an organic soil. (Maclean and Sherwood [1]).

Laboratory studies done at University of Moratuwa have shown that a cement content of the order of 20-25% is required (Saputhantiri and Kulathilaka [2], Munasinghe [3]). This is comparable with the cement weights of 200 - 250 kg per cubic meter of treated soils reported in literature (Lahtinen et al [4]). The cost of this treatment would be very high.

Several researchers in recent times have successfully used mixes of cement and various industrial by products such as blast furnaces slag to stabilize both inorganic and organic clays (Jegandan et al [5]). Such industrial by products are not available in Sri Lanka

Paddy husk is an agricultural waste obtained from milling of rice and it is used to burnt bricks and tiles. In Sri Lanka about 2.2 million tons of rice is produced annually and Paddy husk ash (PHA) is a by product of bricks and tiles manufacturing industry. Paddy Husk Ash (PHA) is a pozzolanic material with silica content of 82 – 87 %. The high percentage of siliceous materials in the PHA makes it an excellent material for stabilization.

Therefore attempts are made in this research to identify the possibility of using paddy husk ash to improve engineering characteristics of peaty clay. The soil sample used for this study was collected from Southern highway project area.

2. Preparation of Samples

Samples were prepared by mixing peaty clay with 20% of cement and 20% of 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. As a control sample peaty clay was remoulded and allowed to settle for 28 days. The basic properties of the three samples after 28 days are summarised in Table 1. Undisturbed specimens were obtained from the three samples to conduct the necessary laboratory tests to determine the strength and compressibility characteristics.

Table 1 Sample Properties

Basic Property	Range/Average of values		
	Peat (PT)	PHA 20% +Peat (PHA)	Cement 20% +Peat (CE)
Moisture Content%	250-350	190-200	130-140
Organic Content%	22	9	12
PH Value	4.20	5.12	9.93
Specific gravity	1.77	1.9	2.03
Initial void ratio	5.56	3.96	2.74

3. Compressibility Characteristics

Oedometer tests were conducted to assess the improvements achieved in the compressibility characteristics. Six specimens were tested taking two samples each from; peaty clay, peat clay mixed with 20% PHA and peaty clay mixed with 20% cement.

The settlements in the peaty clays are mainly due to primary consolidation and secondary consolidation. Secondary consolidation settlements are very significant in the peaty clay. As such, the effects on the parameters corresponding to primary and secondary consolidation were analysed separately.

In order to assess the secondary consolidation characteristics the duration of a load increment was increased to 3 days. Tests were done with loading increments of 5, 10, 20, 40, 80 and 160kN/m^2. After unloading to a stress level of 5kN/m^2, specimens were reloaded through increments of 10, 20, 40, 80, 123, 133, and 145 to 160 kN/m^2.

3.1 Changes in e Vs log (σ) Relationship

Changes in the primary consolidation characteristics were illustrated through e Vs log σ plots. The plots for three differently treated peaty clay samples PT, PHA, and CE are presented in Figure 1. It could be seen that with mixing of 20% PHA, the compression index C_c decreased from that of peaty clay indicating an improvement of compressibility. But the level of improvement achieved was much less than that achieved by mixing with 20% of cement. The basic consolidation characteristics are compared in Table 2.

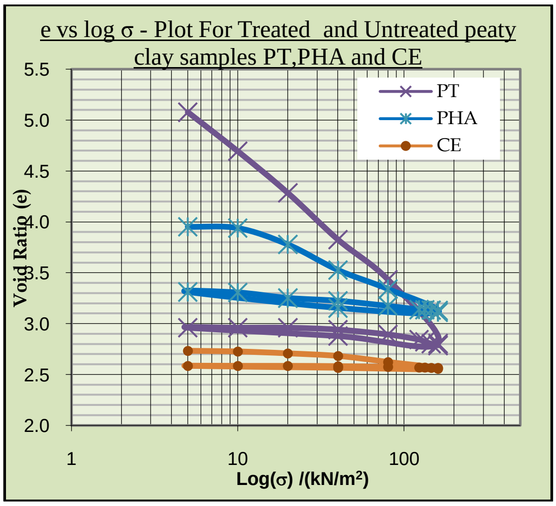

Figure 1– e Vs log(s) plots of PT, PHA and CE

Table 2 Data obtained from e Vs log(σ) plots

Sample	C_c	C_r	p_c (kN/m^2)	$C_c/(1+e_0)$
PT	1.510	0.117	0	0.2485
PHA	0.688	0.121	10	0.1388
CE	0.209	0.013	40	0.0561

3.2 Changes in Coefficient of Volume Compressibility

The coefficient of volume compressibility is an alternative parameter used to estimate the primary consolidation settlements. 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 three differently treated peaty clay samples (Figure 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.

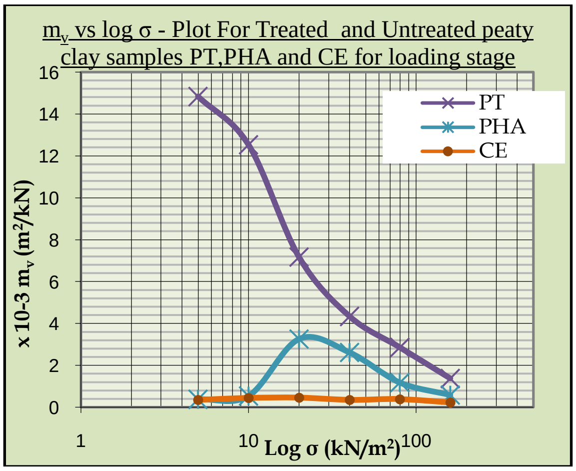

Figure 2 Effects on m_v plots for loading stage

3.3 Changes in Coefficient of Secondary Consolidation (C_α)

Secondary consolidation settlements are generally high in peaty clay and are evaluated through the coefficient of secondary consolidation (C_α). As such, the effect of cement mixing and PHA mixing on the coefficient of secondary consolidation C_α was studied in detail. The C_α value is the gradient of the graph of e Vs log (time) after the completion of the primary consolidation phase. The values of C_α were determined for each stress level for the three differently treated peaty clay samples are presented in Figure 3.

The results show that the secondary consolidation coefficient was considerably reduced due to the mixing with cement or PHA. The reductions achieved were of the same order. In the peaty clay sample the C_α value was increasing with the stress level. This feature was not observed in the samples mixed with either cement or PHA.

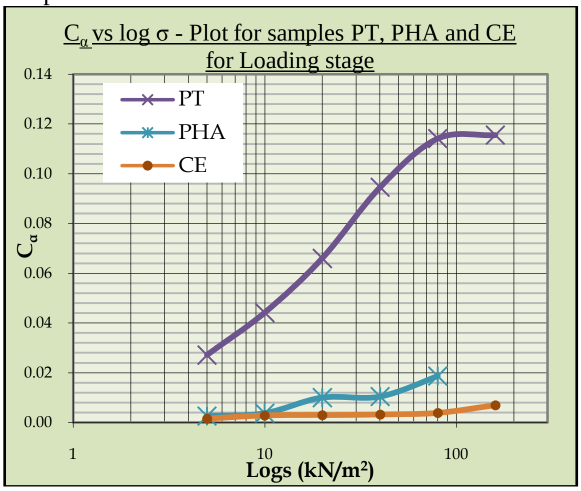

Figure 3 C_α plots of PT, PHA and CE

4. Changes in Undrained Shear Strength

Peaty clays are of extremely low shear strength and possible shear failure during the construction of embankment on them is a major challenge. Moisture content values in excess of 300% and undrained shear strength values as low as 6kN/m^2 (or even lower) had been reported in the infrastructure development projects in Sri Lanka. In these projects the embankment filling has to be done at a very slow rate to prevent shear failures during the construction. The construction may have to be done is stages leaving time gaps in construction to allow pore water pressure dissipation and consolidation to an acceptable level so that there will a sufficient increase in shear strength. As such, undrained shear strength is the most appropriate parameter in the evaluation of stability.

The most appropriate laboratory test to determine undrained shear strength, unconsolidated undrained triaxial was used to evaluate shear strength. However, the samples of natural peat were very soft and sample failed due to its own weight in during the setting up in the laboratory. As such, laboratory vane shear test was used for the peaty clay.

The Mohr circles obtained from the testing of cement mixed peat and paddy hush ash mixed peat are presented in Figure 4 and Figure 5 respectively. The initial moisture contents for the tested specimens of cement mixed and RHA mixed peaty clay are presented in Table 3. The undrained shear strength values obtained from the testing are summarized in Table 4.

Table 2 Sample moisture contents

Sample No	Sample Type	Moisture Content %
1	20% cement+Peat	138.49
2	20% cement+Peat	132.01
3	20% cement+Peat	187.50
4	20% PHA+Peat	184.01
5	20% PHA+Peat	188.91

Table 3 Undrained Shear Strength C_u values

	Sample No	Strain at Failure %	C_u kN/m^2
Cement + Peat	C1	13.44	30.0
	C2	8.96	
	C3	14.94	
PHA + Peat	P1	8.96	6.0
	P2	10.45	

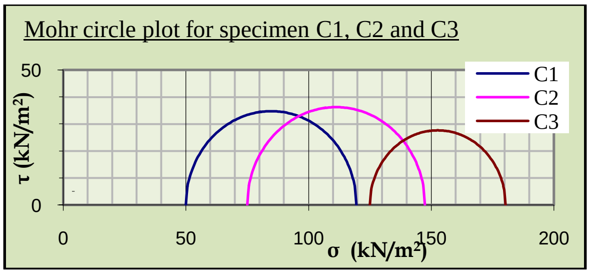

Figure 4 Mohr Circle plot for Cement+Peat samples

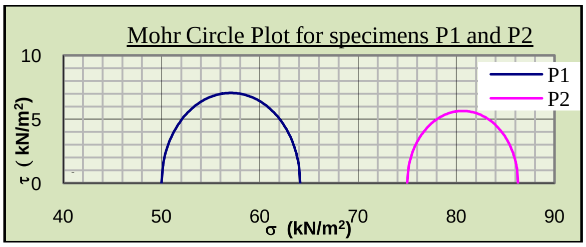

Figure 5 Mohr Circle Plots for PHA + Peat Samples

4.2 Laboratory Vane Shear Test

As it is necessary have a comparison, the undrained shear strength of cement mixed peaty clay and RHA mixed peaty clay were also determined by the laboratory vane shear test. The results obtained are presented in Table 5.

It could be seen that the mixing of PHA and mixing of cement has improved the shear strength. But clearly the improvement achieved by mixing of cement is much greater. Also, it should be noted that for the cement mixed peaty clay PHA mixed peaty clay the shear strength values obtained by the UU triaxial test and laboratory vane shear test are somewhat different.

Table 4 Shear strength, C_u by vane shear test

Sample	Shear Strength (kN/m^2)
Peat only	0.995
20% PHA +Peat	3.318
20% Cement +Peat	40.18

5. Conclusion

Extremely low shear strength and high compressibility of peaty clays impose many construction problems when embankments are to be constructed on sites underlain by such soils. In many infrastructure development projects in Sri Lanka these problems were overcome by pre-consolidation techniques such as preloading with vertical drains and vacuum consolidation. The major drawback associated with these methods is the duration of the treatment.

With the in-situ mixing of the soft peaty clay with an appropriate binder the pozzolonic reactions can be initiated and the engineering characteristic could be enhanced. Research done at University of Moratuwa has shown that mixing of 20%-25% cement would be necessary to achieve a desired level of improvement. Therefore the cost involved would be very high.

Industrial by product such as blast furnace slag had been used together with cement in Europe successfully for the in-situ improvement of soft peaty clay. Such industrial by products are not available in Sri Lanka. As such, attempts were made to use an agricultural by product PHA which is proven to be a pozzolonic material in the improvement of peaty clay.

Samples were prepared by mixing peaty clay with 20% cement and 20% PHA. A controlled sample of remoulded peaty clay was also used. After leaving the samples for 28 days for the pozzolonic reactions to complete, strength and compressibility of the treated peaty clay were evaluated by laboratory testing.

The results showed that that the undrained shear strength improved with the mixing of PHA or cement, but the improvement achieved with the mixing of cement was much greater.

Primary consolidation characteristics as indicated by C_c or m_v were enhanced due to the mixing of cement of PHA. There again the improvements achieved with cement mixing was much greater.
The secondary consolidation characteristics as indicated by C_α decreased significantly due to the mixing with cement or mixing with PHA. The improvements achieved were of similar order. In the cement or PHA mixed samples the C_α value did not increase with the stress level.

The results show that PHA mixing causes some improvements but the level of improvement is not sufficient. Therefore it is suggested that further research should be done mixing different proportions of both of PHA and cement mix with peat so that an economical mix proportion can be achieved.

References

1. Maclenn D J and Sherwood P T (1962), Study of the Occurrence and Effects of Organic Matter in relation to the Stabilization of soils with Cement, Proceedings of the 5th International Conference of Soil Mechanics and Foundation Engineering in Paris.

2. Saputhantiri D.R, Kulathilaka S.A.S (2011), Enhancement of Engineering Characteristics of Peaty Clay due to Mixing With Cement, Institute of Engineers Sri Lanka.

3. Munasinghe W G S (2001), Methods for Improvement of Engineering Properties of Peat – A Comparative Study, Thesis submitted in partial fulfillment of the M.eng in Geotechnical Engineering at University of Moratuwa.

4. Lahtinen P., Jyrava H, Kuusipura K,(1999) Development of Binders for Organic soils, Proceedings of the conference on Dry mix methods for deep soil stabilization, Stockholm.

5. Jegandan S, Liska M, Osma A M , Al-Tabba A, (2010) Sustainable Binders for Soil Stabilization, Proceedings of Institution of Civil Engineers, Ground Improvement.

Durability Improved Geomat Reinforced Vertical Embankment Behaviour

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Abstract: *In this study, the lateral deformation characteristics of a model vertical embankment reinforced with polymer coated coir geomats are compared with the lateral deformation characteristics of the same model embankment reinforced with uncoated coir geomats or geotextiles. For this purpose, a vertical embankment of height 500 mm and length 700 mm was formed by using coated coir geomats of length 605 mm at a vertical spacing of 100 mm as reinforcement in a soil having shear strength parameters of c'=5 kPa and ϕ'=31° compacted to 95% of standard Proctor density. The embankment was initially loaded at its natural moisture content up to a maximum surcharge pressure of 100 kPa through a rigid steel plate and was then unloaded. It reloaded again up to a maximum surcharge pressure of 250 kPa and unloaded. The embankment was then soaked over a period of 48 hours ensuring no erosion of the soil takes place during the process and again the embankment was loaded up to a maximum surcharge pressure of 250 kPa. During the above loading and unloading processes the lateral deformation along the central vertical axis was measured at the mid-height of each soil layer. Based on the laboratory experimental model study, it can be concluded that irrespective of whether the loading is applied under natural moisture content of the fill or soaked condition, the polymer coated coir geomat reinforced vertical wall showed similar lateral deformation characteristics to that of uncoated coir geomat or geotextile reinforced wall, at the same fraction of the allowable surcharge pressure corresponding to each material. This implies that the improvement of the durability of coir geomats by coating a waste polymer material will not affect the lateral deformation characteristics of the coir geomat reinforced wall.*

Keywords: *durability, coir, lateral deformation, model test*

1. Introduction

Use of internally stabilised earth walls has gained popularity due to the possibility of its rapid construction, instead of the conventional reinforced concrete or random rubble masonry walls. When reinforcements are placed at a designed vertical spacing, each reinforcement will carry the force transferred by the soil within its tributary area which can be calculated using Rankine's active earth pressure theory. It is also essential that the lateral displacement of reinforced earth embankments under the design load is kept within a tolerable limit to satisfy the serviceability limit state. Instead of using geotextile and geogrid polymer materials as reinforcement, it is possible to use coir geomats as they possess desirable engineering properties that are comparable with those of polymer reinforcement. However, the suitability of coir geomats also lies in their ability to satisfy durability considerations as nearly 80% of the strength of coir geomat is reduced after one year period. Therefore, it is essential that the durability of coir geomats to be improved suitably before using them as geo-reinforcement. In this experimental study, the lateral deformation characteristics of durability improved coir geomat reinforced vertical embankment are investigated. For this purpose, the constituent natural coir in geomats was coated with waste polythene under controlled conditions.

2. Literature review

When reinforcements are placed at a designed vertical spacing, each reinforcement will carry the force transferred by the soil within its tributary area which can be calculated using Rankine's active earth pressure theory. It is also possible to use coir geomats as the reinforcing material as a cost effective solution to stabilise the vertical face of an embankment making use of its engineering properties (Kurukulasuriya L.C. et.al, 2011). This is in addition to carrying out thorough subsurface investigation to determine strength properties required for settlement evaluation and stability analyses. While taking effective measures to ensure that the loss of tensile strength of reinforcement during the design life of the reinforced wall is kept within acceptable limits, it would be required to assess the performance of the wall with regard to its aesthetic appearance that could be affected due to excessive lateral deformation. Therefore, it is imperative that the designer limits the lateral deformation of reinforced earth walls under service loads. Chai et.al, 2002 had conducted numerical analyses using finite element models and concluded that only when the embankment approaches to failure, the reinforcement has noticeable effect on lateral displacement of the soil. From previous project results, coir geomat can be used as soil reinforcement material. But, its durability should be improved as nearly 80% of the strength of coir geomat is reduced after one year period (Er.Sheela Mary Cherian). Therefore it cannot be used in long run projects.

3. Materials and Methods

For this experimental study the same soil type used by Kurukulasuriya et.al. (2011) was used which makes it possible to carryout comparative studies. The basic soil characteristics are given in Table 1.

Table 1. Wall fill properties

Soil Parameters	Values
Percentage coarse particles	70
Maximum dry density	17.0 kN/m3
Optimum moisture content	16.0 %
Effective cohesion	5 kPa
Effective angle of internal friction	31°

In order to improve the durability of geomats a coating of waste plastic was carried out. For this purpose, the ropes in the same type of geomat were separated and 1 m length of the rope was wrapped using fifteen pieces of 50 mm x 600 mm waste plastic bag material.

Then, geomats were re-woven using the polythene wrapped coir ropes keeping the same rib spacing in the longitudinal and transverse directions, and kept in the oven, at 130°C for5 minutes. The geomats that were used had a rib spacing of 25 mm and 17 mm along the machine and transverse directions respectively and a density of 650 g/m^2 (Fig.1).

Fig.1. Coated Coir Geomat

In order to design the geomat reinforced vertical wall, wide-width tensile tests were carried out on polythene coated geomats to evaluate the ultimate wide-width tensile strength in accordance with BS 6906:Part 1:1987 (Table 2).

Table 2.Properties of coated and uncoated geomats

	Coated Coir Geomat (kN/m)	Uncoated Coir Geomat (kN/m)
Average Maximum load	9.70	8.25
Average Breaking load	9.70	7.40
Average Elastic limit	6.70	5.05

The vertical wall was constructed to have dimensions of 700 mm length, 605 mm width and 500 mm height (Fig.2).

Fig.2.A schematic diagram and a view of the embankment model

Coated coir geomat reinforcement was placed well stretched at a vertical spacing of 100 mm which according to design calculations ensures that the force developed under the surcharge pressure of 250 kPa is below the maximum load obtained along the machine direction.
The soil was compacted to achieve a density of 95% of standard Proctor density at the corresponding water content of the dry side of the compaction curve. Dial gauges were setup along the central axis of the wall at mid-height of each of the 5 soil layers to observe the lateral

deformation of each soil layer. Then the plate was gradually loaded up to 5 tons through a rigid steel plate which produced a contact surcharge pressure of 123 kPa. The pressure was unloaded and it was reloaded gradually up to 10 tons which produced a surcharge pressure of 244 kPa and again the load was unloaded. In the next stage of the experiment, a wet condition was simulated by allowing the wall fill to soak for more than48 hours ensuring that no surface soil eroded away. The loading was again applied up to 10 tons which was then unloaded.

4. Results and Discussion

Fig. 3 gives the comparison of the relationships of lateral deflection ratio (defined as the lateral deflection/height) and the surcharge pressure ratio (defined as the applied surcharge pressure/surcharge pressure that produces breaking load on the geomat) for the two identical walls reinforced with either uncoated coir geomats or coated coir geomats. It is evident that, under the natural moisture condition and soaked conditions, coated geomat reinforced wall had undergone nearly the same lateral deflection corresponding to a particular applied load ratio. Therefore, under natural moisture conditions and soaked conditions, the coated coir geomat reinforced wall had performed in a similar manner to that of uncoated geomats.

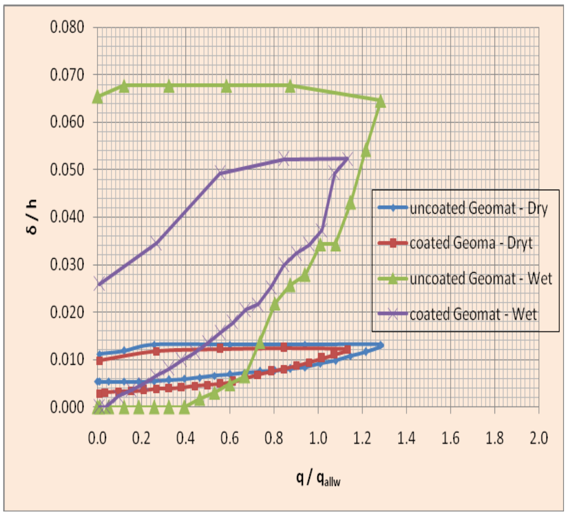

Fig.3. The relationship of Deflection ratio and load ratio for coated and uncoated geomats for the top layer loaded under dry and wet conditions

5. Conclusion

Irrespective of whether the loading is applied under natural moisture content of the fill or soaked conditions, the durability improved coated coir geomat reinforced vertical wall showed similar lateral deformation to that of uncoated coir geomat reinforced wall, at the same fraction of the allowable surcharge pressure corresponding to each material. Therefore, it can be concluded that the improvement of the durability by coating a waste polymer material has not affected the lateral deformation characteristics of the coir mat reinforced wall.

6. Acknowledgments

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References

BS 6906: Part 1: 1987; British Standards Institution.

Chai, J.-C.Zhu, H.-H. "Behavoiur of reinforced embankment over soft subsoil", Proceedings of international seminar on practice and advance in geo technical engineering, shanghai, China.

Er. Sheela Mary Cherian, "Coir Geomats", http://www.associationofengineers.com/pdf/82433554e38aa0f3aa89Coir%20Geo-textiles.pdf, visited the website on 2011/ 09/ 27.

L.C. Kurukulasuriya, N. Mahinthakumar, T. Theenathayarl, T. Krishanthan (2011) "Lateral Deformation Characteristics of Coir Geomat Reinforced Vertical Embankments". Proceedings, International Conference on Structural Engineering, Construction and Management, Kandy, Sri Lanka.

STUDY ON LONG TERM SETTLEMENT BEHAVIOR OF DUMP WASTES IN SRILANKA;
CASE STUDY OF BLOEMENDHAL DUMP SITE

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ABSTRACT

Open dump methods are widely used to dump wastes in developing countries. In SriLanka, most of the dumpsites are open dumpsites. Due to high demand for lands, previously abandoned lands are now utilized for construction. However, due to heterogeneity and variability, construction or performance on these type of lands requires more studies. Settlement of any structure is not desired. Therefore, settlement characteristics of dump waste isneeded to be studied. It has immediate settlement, primary consolidation settlement and long-term settlement. This research paper provides about long-term settlement behavior of dump waste. Secondary compression is due to creep of the waste skeleton and biological decay. In general, secondary compression accounts significant part of total settlement of dump area and it may occur over many years. This research has been carried out to find settlement parameters of Bloemendhal dumpsite. According to Unified Classification System, this sample is classified as well graded sand with silt and gravel (SM-SM). Standard Proctor compaction test reveals that, sample has maximum dry density of 1418 kg/m^3, which is sometimes low compared to traditional lateritic soil in SriLanka. Optimum moisture content is 24.5%. Primary consolidation parameters were evaluated through laboratory experiments. Sample has the coefficient of consolidation in the range of 1 - 4 m^2/year and coefficient of compressibility in the range of 0.05 – 0.1. The sample has lower secondary consolidation creep index, which is in the order of 10^{-3}. However further long term biodegradation studies should be carried out when deals with fresh dumpsite samples.

Key words: long-term settlement, secondary compression index, dump waste, creep of dump waste

1. INTRODUCTION

SriLanka is a rapidly developing country but with limited resource of land. It leads to use the lands, which were abandoned previously or it is required to fill the lower lands in city areas. In city of Colombo the wastage is dumped in sites like Bloemendhal, Madhampitiya, Kolanawa etc.

Dumped waste decays with time and it becomes similar to soil. It may also be used as a fill material. Construction on such lands may have issues due to its nature.Settlement, slope instability, shear failures and adverse chemical reactions are some potential issues,which areneeded to be studied.

In this research, long-term settlement behavior of dump waste is studied. There are three main stages of settlement in waste soil such as initial consolidation, primary consolidation and secondary settlement. Initial settlement occurs immediately when an external load is applied. It is generally associated with the immediate compaction of void space and particles due to a superimposed load. Primary consolidation is due to dissipation of pore water pressure and air from the void spaces. Secondary compression is due to creep of the waste skeleton and biological decay. In general, secondary compression accounts significant part of total settlement of dump area and occurs over many years [1]. In this research, primary consolidation parameters and secondary compression creep parameters of Bloemendhaldumpsite are experimentally evaluated.

2. OBJECTIVE

The main objective of this research is to study about long-term settlement behavior of dump waste. Classification of sample, finding of primary consolidation parameters and finding of secondary compression creeping parameters are identified objectives of this research.

3. SETTLEMENT MODELS

Soil mechanics-based model, empirical models, rheological model and settlement model that incorporates biodegradation are four types of model to analyze the settlement [2]. Under the above categories, number of models had been developed. However, most of them are defined for municipal solid waste or landfill which has proper leachate and gas circulation.

In SriLanka, most of them are open dumps, which do not have proper engineering work to minimize adverse impacts of dumps. Therefore, following models, which could be used for both landfill and open dump waste, are considered [2].Basic soil mechanics model of consolidation to estimate dump wasteis Sowers model [3], wherethe long-term consolidation associated with creep and biodegradation is expressed in terms of the secondary consolidation index. Bjarngard and Edgers model [4]subdivids the secondary compression into two sub phases, through the adjustment of two straight lines. The power creep model [5] is a simple relation for time dependent deformation under constant stress. Hyperbolic equation [6] is to compute settlement at a given time for ultimate settlement of the landfill.

4. METHODOLOGY

Following methodology steps were followed in this research to reach objectives.

- Learning background about research project and the necessity of research was identified
- Literature review was done to accumulate relevant knowledge of this research
- Selection of site for sample collection is done according to how long the site has been abandoned and accessibility to gain sample
- Sample preparation and series of laboratory experiments were performed
- Analysis of results

5. EXPERIMENTAL WORK

To meet the objectives of this research laboratory experiments are required. A series of experiments were conducted according to appropriate guidelines such as BS 1377 and ASTM volume 04.08.Classification of sample and compressibility are two identified objectives.

Classification of the sample was done according to Unified Soil Classification System (USCS). It requires to doing particle size distribution of sample and consistency limit tests.

Chemical test such as pH value, chloride content and sulfite content were found to identify the impacts of the soil to structures and people who they would be working with it.

Proctor compaction test was done to find out the relationship between moisture content and dry unit weight of sample. Then consolidation tests were done as to simulate the compaction that would be done or required when using the sample as a fill material. The secondary consolidation parameters were found from oedometer. Both primary and secondary consolidation parameters were derived from these consolidation tests.

5.1. CLASSIFICATION OF SAMPLE

It is reasonable to expect different particles such as organic components, metal particles, glass particles etc in the sample. Since biodegradation of organic particles have a significant impact on long-term settlement of dump waste, it is necessary to have an idea about availability of organic components in sample.

Therefore, it was decided to perform both air and oven drying. In oven drying process, some particles could be burnt such as organic particles. Particle size distributions were performed for both type of samples.In addition, the particle size distribution was done in both wet and dry sieve methods. Therefore, altogether four different types of procedures were followed in particles size distribution and Atterberg limit tests as follows. Particle size distribution of four cases is shown in Fig.1.

- Air dry – dry sieve
- Air dry – wet sieve
- Oven dry – dry sieve
- Oven dry – wet sieve

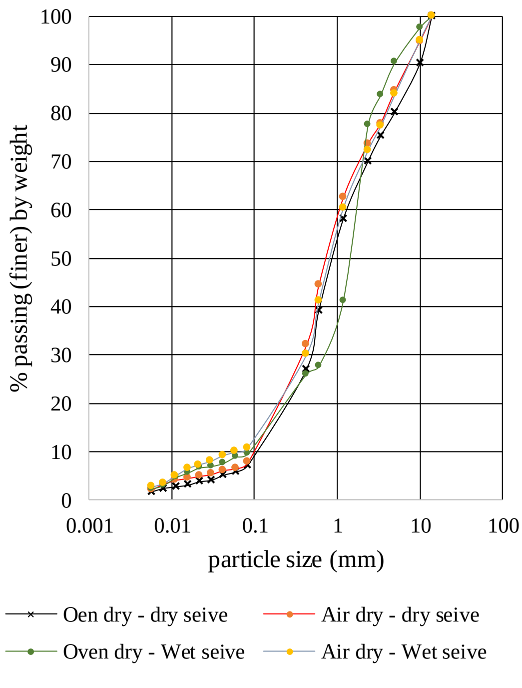


Fig.1 - Particle size distribution

Liquid limitsshall be found from both Casagrande apparatus and penetration method. Initially it was tried with Cassagrande apparatus, but because of lower plasticity of sample, reliable results could not be achieved. Therefore, liquid limit was found by using the penetration method.Table 1 shows the Atterberg limit values of sample.

Table 1 – Plastic and Liquid limits

Type	Liquid limit	Plastic limit	Plastic index
Air dried and dry sieved	45	26	18
Air dried and wet sieved	35	28	7
Oven dried and dry sieved	38	27	11
Oven dried and wet sieved	30	27	3

5.2. CHEMICAL TESTS

Chemical parameters of sample such as pH, chloride content and sulfite content were evaluated according to BS 1377: part 3.

Table 2 – Chemical test results

Parameters	Values
P_H	7
Chloride content	0.02 %
Sulfite content	0.086 %

5.4. COMPACTION AND CONSOLIDATION

Compaction is a type of densification of soil by expelling the air from voids of soil by any compression effort. Since, standard proctor compaction simulate the site procedures, tests weredone to samples, which wereair-dried and dry sieved. From particle size distribution curve, percentage of particles retained on sieve number 4 (4.25 mm) is lesser than 20%. Therefore, method A (ASTM standard D698) was selected to perform. The Maximum dry unit weight is 1418 kg/m^3 obtained at moisture content of 24.5%.

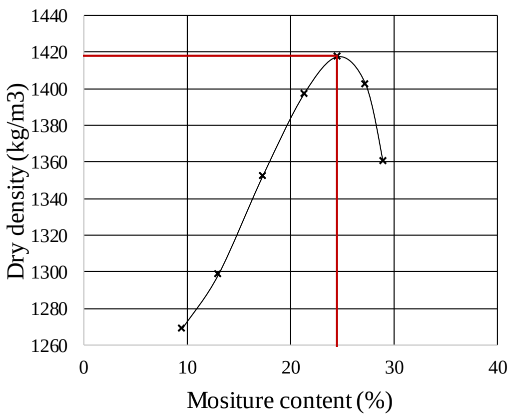


Fig. 2 -Moisture content vs Dry density

Specific gravity value of sample was evaluated according to BS 1377 : part 1, Pycnometer method. Specific gravity of sample is 1.96.Consolidation test was done to the sample, which has taken from the standard Proctor compaction test. Therefore, it can simulate the field effect of compaction when sample is used as a fill material. This test was done for standard compaction of sample with different initial moisture contents. This test was done according to ASTM 2435, test method B.

6. CALCULATIONS

According to Unified Classification System, all considered four types of samples are classified as SW – SM, which is well-graded sand with silt and gravel. Table 3, shows theprimary and secondary creep consolidation parameters

Table 3 – Primary and Secondary consolidation parameters

(%)	Stress (kPa)	C_v (m^2/yr) log time method	C_v (m^2/yr) root time method	Cr	C_c	C_α (x 10^{-3})
15	50	1.78	5.01	0.008	0.17	4.17
	100	0.4	3.03			3.39
	200	0.31	1.26			3.12
24.5	50	0.33	4.27	0.01	0.08	3.2
	100	0.2	1.52			3.17
	200	0.25	1.6			2.47

7. DISCUSSION

Sample has been classified in four different conditions, but all of them has the same classification of well-graded sand with silt. Samples has considerably lower plastic index values, within the considered categories air-dried, dry sieve condition has higher plastic index because of higher liquid limit. However, it can be noticed that, plastic limit is increased when sample is wet sieved, that indicates the availability of smaller amount of fine clay particles, which bonded with coarser particles. It leads to the sample near to the A-line in plastic index vs liquid limit curve.Table 4 [7], shows the comparison of Atterberg limits and C_c of soils.

Chemical tests reveal that, considered sample is almost neutral with pH 7 and does not have significant chloride or sulfite content. pH value indicates the unavailability of significant amount of other adverse chemical compounds.

Primary consolidation parameters such as coefficient of consolidation, primary compression index, recompression index and coefficient of volume compressibility describes that these parameters have lower value in optimum moisture content or maximum dry unit weight. Primary consolidation is lower in optimum moisture content. Secondary compression index has very lower value and lowest value is at optimum moisture content.

Table 4 – Comparison of Atterberg limits and C_c

Soil type	LL	PL	C_c
Boston clay	41	20	0.35
Chicago clay	60	20	0.4
New orleans clay	80	25	0.3
Bloemendhal (Air dry – Dry sieve)	45	26	0.17

Table 5 – Comparison of C_v

Soil	Pressure (kPa)	C_v (x 10^{-4} cm^2/ sec)	
		Logarithm of time method	Square root of time method
Brown soil	50 – 100	3.02	3.77
	100 - 200	2.86	3.4
Illite	50 – 100	1.34	3.13
	100 - 200	2.2	3.18
Chicago clay	50 – 100	1.3.7	17.4
	100 - 200	3.18	4.71
Bloemendhal Dumpsite (ω 15%)	50 – 100	1.27	9.6
	100 - 200	0.98	3.99
Bloemendhal Dumpsite ω(24.5%)	50 – 100	0.634	4.82
	100 - 200	0.79	5.26

Table 5 shows comparison of coefficient of consolidation of different types of soils [7] and Bloemendhal sample.C_v first decreases with increment of pressure then decreases.

8. CONCLUSION

It is mentioned that long-term settlement of dump waste includes creep of soil and biodegradation. Secondary compression index of creep for different conditions are considerably lower in this particular sample.It is essential to consider both biodegradation and secondary compression index for long-term settlement of dump waste.Biodegradation occurs over the period of almost 3500 days (10 years) [8]. However, Bloemendhal dumpsite is almost 30 years old and bottom part is almost decayed to form a soil[9]. Therefore, secondary consolidation due to biodegradation would not be significant in Bloemendhal sample. If bottom part of Bloemendhaldumpsite waste is used as fill material primary consolidation and secondary consolidation creep parameters should be considered. However further studies is required about biodegradation when deals with fresh samples.

9. REFERENCES

[1]El-Fadel, M. and Khoury, R. (2000) Modeling Settlement in MSW Landfills: a Critical Review, Critical Reviews in Environmental Science and Technology, 30(3), 327-361.

[2]NurIrfahMohdPauzi, Husaini Omar, Bujang Kim Huat, Halina Misran (2010) Settlement Model of Waste Soil for Dumping Area in Malaysia

[3]Sowers G.F. (1973) Settlement of waste disposal fills, Proc. 8th Int. Conf. on Soil Mechanics and Foundation Engineering, Moscow, Vol. 1, 2(2), 207-210.

[4]Bjarngard A. and Edgers L. (1990) Settlement of municipal solid waste landfills, Proc. 13th Annual Madison Waste Conf., University of Wisconsin, Madison Wis., 192-205.

[5]Edil T.B., Ranguette V.J. and Wuellner W.W. (1990) Settlement of municipal refuse, Geotechnics of wastefills: Theory and practice, STP 1070, A. Landva and G.D. Knowles, eds, ASTM, West Conshohocken, Pa 225-239.

[6]Ling H.I., Leschchnsky D., Mohri I. and Kawabata T. (1998) Estimation of municipal solid wastelandfill settlement, J. Geotech. Geoenviron. Eng., 124(1), 21-28.

[7] R.G.Robinsion and M.M.Allam (1996), Determination of coefficient of consolidation from early stage of log t plot. Geotechnical testing Journal, 19(3) pp.316-320

[8] Hossain S.M. and Gabr M.A. (2005) Prediction of municipal solid waste landfill settlement with leachate recirculation, Proc. Geo. Frontier, Austin Tex., Vol. 168. ASCE, 50.

[9]U.P. Nawagamuwa, W.D.S.P. Gunaratne, P. Kirubajiny, T. Thiviya and H.K.A. Priyadharshana(2013) Study on the Geotechnical properties of Open Dumps in SriLanka

ESTIMATION OF PRIMARY AND SECONDARY CONSOLIDATION PROPERTIES OF PEAT SOIL USING FIELD SETTLEMENT MONITORING DATA

M.A.D Senanayake[1]

Abstract: This research was conducted with the objective of finding a method to determine primary consolidation **(δ_p)** and secondary consolidation **(δ_s)** properties, C_c primary compression index and C_α secondary compression index of organic peat by using field settlement monitoring data. In this analysis Terzaghi's one dimensional consolidation theory, identification of primary settlement of peat soil by inflection point in Settlement Vs. log time graph and hyperbolic method were used.

It has been attempted to obtain a correlation between modified primary compression index ($\frac{Cc}{1+eo}$) and modified secondary compression index ($\frac{C\alpha}{1+ep}$) instead of determining void ratios "e_o" and "e_p" to calculate C_c and C_α directly, to arrive at a correlation between C_c and C_α. The ratio of modified primary compression index to modified secondary compression index is compared with **(C_α/C_c)** method of compressibility results of soft clay soils by Mesri and Castro (1987) to determine consolidation properties of soil. Hyperbolic method's primary settlement was compared with the actual primary settlement of peat soil. In this research the accuracy of using average modified secondary compression index value, to determine secondary settlement of peat soil with a limited error of ±25mm has been checked.

Keywords: Primary settlement, Secondary settlement, Primary compression index, Secondary compression index, Peat soil, Modified primary and Secondary compression indices.

1. Introduction

Developing a method to determine consolidation of weak soil like peat using field settlement data will be of great value to the construction industry. Since, that kind of a method is both convenient, cost effective and does not require expensive laboratory tests. Obtaining soil parameters of C_c and C_α in weak soil like peat is difficult using conventional methods.

The main objective of the research is to develop a method to find soil consolidation parameters of weak soil (organic peat), using field settlement monitoring data. Also to develop a method to predict primary consolidation **(δ_p)** and secondary consolidation **(δ_s)** of peat soil. The main soil parameters to be determined are C_c (primary compression index) and C_α (secondary compression index) of organic peat.

In previous researches Asaoka method (Asaoka, 1978), hyperbolic method (chin, 1970; Tan, 1971, 1993) and **(C_α/C_c)** method of compressibility (Mesri and Castro, 1987; Mesri and Godlewski 1977) have been used with laboratory settlement data to determine consolidation properties of soil.

2. Problem approach

Colombo Katunayaka Expressway (CKE) project field settlement monitoring data were taken for the research. The predominant organic peat layers of the study area reach about 10m - 15m thickness in critical locations. CKE peat contains with high organic content (38%-mean organic content, calculated in 2003). (Jeff.H, Gunasekara C,Nguyen.V, 2005)

2.1. Data selection

Following criteria were used to select data points for the research.

- Data should exceed the time span of primary consolidation **(δ_p).**
- The field settlement data vs. log time curve should show a well-defined inflection point to identify δ_p and δ_s.
- Existence of a predominant peat layer with borehole data.
- Relatively flatter loading curve, during the time of significant settlement, to obtain an average fill value to calculate "σ_1" and "σ_2" (3.3).

3. Methodology

3.1. Inflection point analysis

Settlement vs. log time curve was plotted and observed for the inflection point location. By inflection point analysis Actual Primary Settlement (APS) and Actual Primary Settlement End Time (APSED) of respective data sets were found. (Fig.3.1)

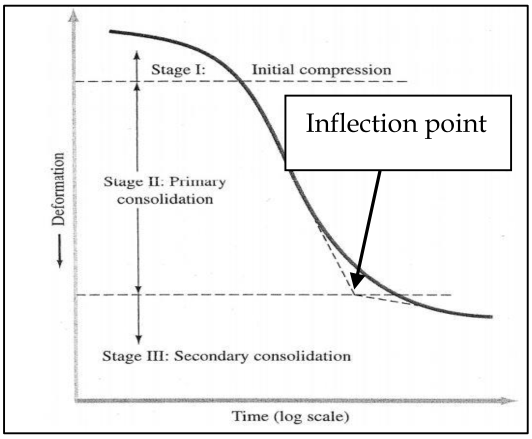


Figure 3.1 .characteristic settlement vs. log time curve

3.2. Assumptions made in analysis

1. One dimensional consolidation.
2. Fill load is applied in a large vicinity of area.
3. Peat layer and fill layer thicknesses to be uniform.
4. Sand layer density of 17 kN/m^3, peat layer density 14 kN/m^3 and fill layer density 18 kN/m^3 under saturated condition.

3.3. Calculation of primary settlement

Theoretical δ_p values were calculated using Eq.3.1, $C_{c'}$ modified primary compression index value is defined in Eq.3.2.

$$\delta_p = \frac{C_c}{1+e_o} H \log\left({}^{\sigma_2}/_{\sigma_1}\right) \qquad [3.1]$$

Where

C_c = primary compression index

e_0 = Initial void ratio

δ_p = Primary settlement of soil

H = soil layer height

σ_2 = Final vertical stress

σ_1 =Initial vertical stress

$$C_{c'} = \frac{C_c}{1+e_o} \qquad [3.2]$$

3.4. Hyperbolic method

Hyperbolic method by Tan (1971, 1994) can be used to develop a relationship with settlement and time, using geotechnical data as follows.

$$\delta = \frac{t}{At+B} \qquad [3.3]$$

Where **t** = time, **A** and **B** are constants.

The Eq3.3 can be rearranged to obtain Eq3.4. Equation 3.4 can be used to obtain hyperbolic curve. The linear portion of the hyperbolic curve can be used to determine ultimate primary settlement.

$$\frac{t}{\delta} = B + At \qquad [3.4]$$

Ultimate primary settlement from hyperbolic method is given by **(1/A).**

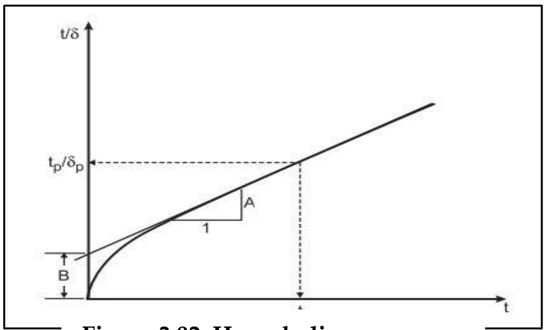


Figure 3.02. Hyperbolic curve.

3.5. Secondary consolidation

Theoretical δ_s values were calculated using Eq3.5. $C_{\alpha'}$ modified secondary compression index value is defined in Eq.3.6.

$$\delta_s = \frac{C_\alpha}{1+e_p} H\log\left({}^{t_2}/_{t_1}\right) \qquad [3.5]$$

Where

$C_{\alpha'}$ = Secondary compression index

e_p = Void ratio at end of primary settlement

δ_s = secondary settlement of soil

H =Soil layer height

t_2 =Final time of secondary settlement

t_1 =Initial time of secondary settlement

$$C_{\alpha'} = \frac{C_\alpha}{1+e_p} \qquad [3.6]$$

4. Analysis and results

4.1. Modified primary compression index by inflection point

After obtaining **(APS)** and **(APSED)** by inflection point analysis (3.1). $C_{c'}$ value (hereafter will be referred as **$C_{c'}$i** value) can be obtained from Eq.3.1 and Eq.3.2 by substituting inflection point analysis settlement results. Average fill height value was used in Eq.3.1 to obtain **σ_2.**

4.2. Modified primary compression index by hyperbolic method

Hyperbolic method primary settlement value can be used in Eq.3.1 and Eq.3.2 to calculate $C_{c'}$ value (hereafter will be referred as **$C_{c'}$h** value).

4.3. Modified secondary compression index by inflection point.

Inflection analysis results can be used in Eq.3.5 to develop a theoretical secondary settlement curve starting from identified δ_p point. The theoretical curve should be fitted best to the actual secondary settlement Vs. log time portion of the curve by adjusting $\mathbf{C_{\alpha'}i}$ value ($C_{\alpha'}$ value from inflection point and curve fitting).

$$\delta_{st} = (C_{\alpha'}\mathrm{i})H\log\left({}^{t_2}/_{t_1}\right) \qquad [4.1]$$

Where δ_{st} is the theoretical secondary settlement.

Total theoretical settlement $\delta_{tt} = \delta_p + \delta_{st}$

Theoretical δ_{tt} **Vs. log time** graph can be plotted starting from δ_p point (Fig 4.1).

4.4. Correlation between $C_{\alpha'}$ and $C_{c'}$

Linear correlation between $C_{\alpha'}$ and $C_{c'}$ values were analysed by $C_{\alpha'}$ **Vs.** $C_{c'}$ graphs. This concept is very similar to (C_α/C_c) concept by Mesri and Castro (1887) and Mesri and Godlewski (1977). $C_{\alpha'}$ and $C_{c'}$ correlation summery is shown in Table 4.1.

4.4.1. Outlier deciding criteria

Histograms and cumulative percentage value graphs were used to identify the distribution of $\mathbf{C_{c'}i}$, $\mathbf{C_{c'}h}$ and $\mathbf{C_{\alpha'}i}$ values to decide acceptable data ranges for graphs.

[1]. $\mathbf{C_{c'}i}$ range was chosen as {0.125< $\mathbf{C_{c'}i}$ <0.425}, range includes 80% of $\mathbf{C_{c'}i}$ data

[2]. $\mathbf{C_{\alpha'}i}$ range was chosen as {0.0200< $\mathbf{C_{\alpha'}i}$ <0.0325}, range includes 70%of $\mathbf{C_{\alpha'}i}$ data.

[3]. $\mathbf{C_{c'}h}$ range was chosen as {0.125< $\mathbf{C_{c'}h}$ <0.450}, range includes 85% of $\mathbf{C_{c'}h}$ data.

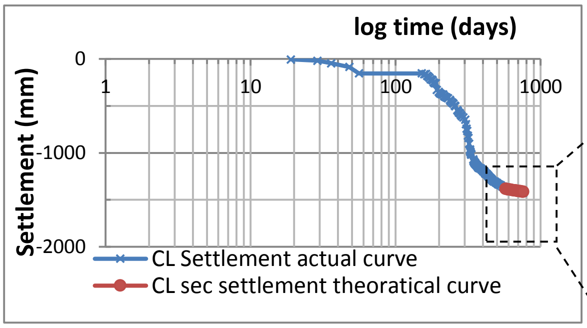
[4]. $\mathbf{(C_{\alpha'}i / C_{c'}i)}$ range was chosen as {0.025< $\mathbf{(C_{\alpha'}i/ C_{c'}i)}$ <0.150}, range includes 90% of $\mathbf{(C_{\alpha'}i / C_{c'}i)}$ data.

[5]. $\mathbf{(C_{\alpha'}i/C_{c'}h)}$ range was chosen as {0.025< $\mathbf{(C_{\alpha'}i/C_{c'}h)}$ <0.125}, range includes 89% of $\mathbf{(C_{\alpha'}i/C_{c'}h)}$ data.

4.5. Actual primary settlement and hyperbolic method primary settlement comparison.

In most of the data points, hyperbolic method over estimates the primary settlement. Shown in Fig.4.2 **(δ_p hyperbolic – δ_p actual)** graph of **CL** (Centre Line) data.

In percentage terms hyperbolic method over estimate nearly 15% of actual primary settlement. Hyperbolic method over estimated primary settlement of 128 data points out of total 140 data points (91% data points, δ_p value has been over estimated by hyperbolic method).



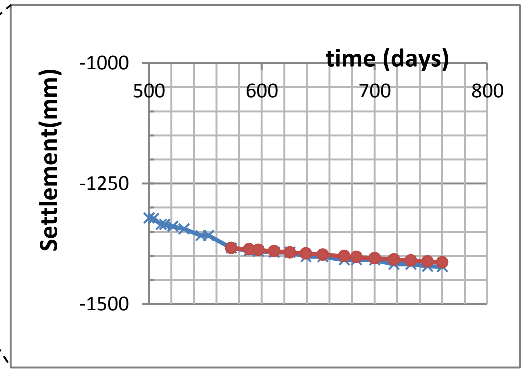


Figure 4.1. K2+200 CL theoretical and actual Settlement Vs log time graph

Table 4.1 Cα′ and Cc′ correlation summery.

	Description of correlation (refer 4.4.1 for outlier criteria [#])	**R^2 value**	**$(C_{\alpha'}/C_{c'})$ from correlation**	**Correlation status**
01	$\mathbf{C_{\alpha'}i}$ Vs. $\mathbf{C_{c'}i}$ **,** no outliers deciding criteria.	0.2	≈0.03	Very weak
02	$\mathbf{C_{\alpha'}i}$ Vs. $\mathbf{C_{c'}i}$, outliers deciding criteria [1] & [2].	0.4910	0.0379	Weak
03	$\mathbf{C_{\alpha'}i}$ Vs. $\mathbf{C_{c'}i}$, outliers deciding criteria [4].	0.5344	0.0318	Weak
04	$\mathbf{C_{\alpha'}i}$ Vs. $\mathbf{C_{c'}h}$ **,** no outliers deciding criteria.	0.2	≈0.03	Very weak
05	$\mathbf{C_{\alpha'}i}$ Vs. $\mathbf{C_{c'}h}$,outliers deciding criteria [2] & [3]	0.5012	0.0302	Weak
06	$\mathbf{C_{\alpha'}i}$ Vs. $\mathbf{C_{c'}h}$, outliers deciding criteria [5].	0.4379	0.03	Weak

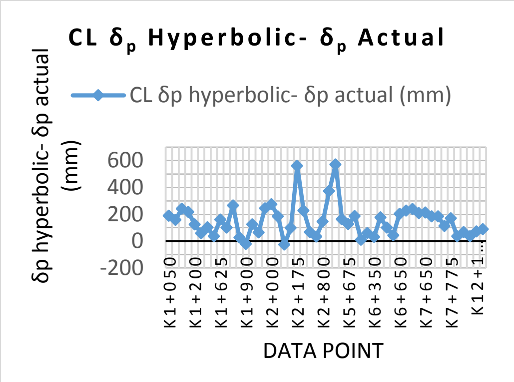


Figure.4.2. δp over estimation by hyperbolic method.

4.6. Secondary settlement prediction by average $C_{\alpha'}i$ value

Assuming that the void ratio does not go through a significant change during secondary settlement (Fig 3.1), Average $C_{\alpha'}i$ value can be used to approximate secondary settlement from Eq.3.5 using field settlement monitoring data. When δ_s estimated by average $C_{\alpha'}i$ by Eq.3.5 , 112 data points out of 140 points, theoretical value of secondary settlement δ_s falls within an error of ±25mm compared to actual settlement. Hence 80% of data points can be approximated in calculating the secondary consolidation of CKE peat under a ±25mm error.

5. Conclusion

Primary settlement value from inflection point analysis can be considered to be a justifiable method of determining the actual primary settlement, since it agrees with the characteristic settlement curve (Fig 3.1).

(C_α/C_c) value of Weak soil was reported as **(0.03-0.05),** using laboratory testing by Mesri and Castro (1987). CKE peat **($C_{\alpha'}/C_{c'}$)** value **(≈0.03)** obtained under weak correlation, using field settlement data agrees with recommended range by Mesri and Castro. Since different outlier deciding criteria did not improve the correlation, there may be an inherent error in the (CKE) field monitoring data set or error in assumptions made in 3.2.

Hyperbolic method over estimates the actual primary settlement. Portion of the secondary settlement may be included in predicted hyperbolic primary settlement. Hyperbolic method, probability of δ_p been over estimated in CKE peat is more than 90%.

By using average value of inflection point analysis $C_{\alpha'}$ value, the secondary settlement of the peat layer can be approximated with a probability of 80%, where data points falling with in the limited error of (± 25mm).

6. Acknowledgment

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7. References

Asaoka.A,1978. Observational procedure of settlement prediction. Soil and foundations.JSSMFF, 18(4), 87-101.

Bujang B.K Hunt, Ng Chung Hoe ,H A Munzir . 2002. Observational methods for predicting embankment settlement. Pertanika Journal of Science & Technology. 12 (1), 116-127

Chin.F.K,1970. Estimation of the ultimate load of piles not carried to failure. Proceeding of the 2nd Southeast Asia Conference on Soil Engineering, 81-90.

Ir.Tan Yen Chin ,2005. Embankment over soft clays – design and construction control. Available: *http://www.gnpgeo.com.my/download/publication/2000_07.pdf. Last accessed 19th July 2013.*

Jeff.H, Gunasekara C,Nguyen.V . 2005. Characteristics of soft peat, organic soils and clays, Colombo Katunayaka expressway Sri Lanka. Ed: Buddhima Indrarathna and Jian Chu ground improvement case histories. 3rd ed.: Elsevier Geo-engineering book series. pp 681-721.

Mesri. G, Stark.T.D, 1997. Secondary Compression of Peat with or without Surcharging. Journal of Geotechnical and Geo-environmental Engineering, 123 (5), 411-421.

Mesri. G, Castro. A, 1987. C_α/C_c concept and K_o during secondary compression, Journal of the Geotechnical Engineering Division, ASCE 113,230-247.

Mesri. G, Godlewski. P.M, 1977. Time and stress compressibility interrelationship. Journal of the Geotechnical Engineering, ASCE 103,417-430.

Mosleh A, AL-Shamrani, 2004. Applying the hyperbolic method and Cά/Cc concept for settlement prediction of complex organic –rich soil formation. *http://faculty.ksu.edu.sa/shamrani/MAS_Articles/ENGE_Hyperbolic.pdf. Last accessed 19th July 2013.*

Tan, S.B., 1971. An empirical method for estimating secondary and total settlement. Proceedings, 4th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, vol. 2, pp. 147–151.

Tan, T.S., 1993. Ultimate settlement by hyperbolic plot for clays with vertical drains. Journal of Geotechnical Engineering, ASCE 119 (5), 950–956.

Tan, T.S., 1994. Hyperbolic method for evaluation of settlement of ground pre-treated by drains and surcharge. Geotechnical Engineering 25 (1), 75–89.

Study on the geotechnical properties of open dumps in Sri Lanka

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Abstract

The focus of the study is to identify the geotechnical properties of MSW (Municipal Solid Waste) with respect to their strength parameters, compaction characteristics and compressibility characteristics. Due to the increase in requirement of land in order to cater the increasing population MSW dumping yards have to be used as construction sites.

This research study has been carried out to determine the geotechnical properties of three dump yards in Sri Lanka such as Bloemandhal, Madampaitiya and Kolonnawa. Particle size distribution, Proctor compaction, direct shear and consolidation tests to find out compressibility characteristics of MSW were carried out. Results of these tests concluded that the geotechnical properties of dump sites were not much different to traditional soils.

Key words: Municipal Solid Waste, Open dump yards, Mechanical properties of MSW

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1. Introduction

Increase in world population and hence the increase in the land requirement have led to the use of MSW dumping yards for construction works, but unidentified variations of the properties of base materials may result in failures of those projects.

Sri Lanka also facing the problem of land scarcity, but results obtained from previous researches carried out in other countries in identifying the geotechnical properties of MSW varies due to variation in composition of solid waste, methods of land fill and climatic conditions etc. Thus there is a need to study the geotechnical properties of Sri Lanka MSW dumping yards. The research study was carried out to identify the particle size distribution, strength parameters, compaction parameters and consolidation properties of the samples collected from three dumping yards of Sri Lanka, which are Bloemandhal, Madampitiya and Kolonnawa.

2. Methodology

2.1. Sample Collection

Samples for research study were collected from three dumping yards of Sri Lanka which are Bloemandhal, Madampitiya and Kolonnawa. Five samples were collected for the study, three from Bloemandhal and one each from Madampitiya and Kolonnawa.

Bloemandhal dumping yard is the oldest one amongst the three, which is about thirty years old and not under operation for about five years, therefore the bottom part is almost decayed to form a soil, but uncontrolled filling has been carried out at the site. Three types of samples are collected which are from bottom part, top part and from the burnt area of the yard.

Madampitiya dumping yard is also not under operation at present, but not as old as Bloemandhal, therefore the waste is partly decayed, layer by layer filling is carried out at the yard.

Kolonnawa is the only functioning dumping yard considered and had fresh dump waste which is not yet bio degraded.

2.2. Experimental Program

2.2.1. Sieve Analysis

Dry sieve and wash sieve analysis was carried out on 500g of each sample to identify the particle size distribution of MSW.

2.2.2. Proctor Compaction Test

Samples were sieved through 4.75mm sieve and 3kg of each samples were oven dried and proctor compaction test was carried and the results were plotted to obtain the maximum dry density and optimum moisture content of the samples.

2.4.3. Direct Shear Test

Direct shear test was carried out parallel in Proctor Compaction Test. Samples for this test, were taken at each moisture addition in Proctor compaction test. With a particular amount of moisture added to the sample, three Direct Shear Tests were carried out under the normal loads of 38.15kN/m^2, 89.925kN/m^2 and 220.725kN/m^2. Therefore, the variation of soil shear strength parameters could be obtained with the varying moisture content for each soil sample.

2.4.5. Consolidation Test

Specific gravity test was conducted for each sample which was sieved through 425μm sieve to obtain a suitable sample to be used with the oedometer. Calculations were performed to find the amounts of oven dried soil and water to be used in order to prepare a sample which is having a same Proctor density. Initially, 50kPa normal stress was applied on the sample and dial gauge readings were taken in increasing time steps. The applied load was increased as 100kPa, 200Kpa and 400kPa and respective readings were taken. Unloading of the sample was also carried out in the steps of releasing the loads of 200kPa, 100kPa, 50kPa and 45kpa and respective dial gauge readings were obtained.

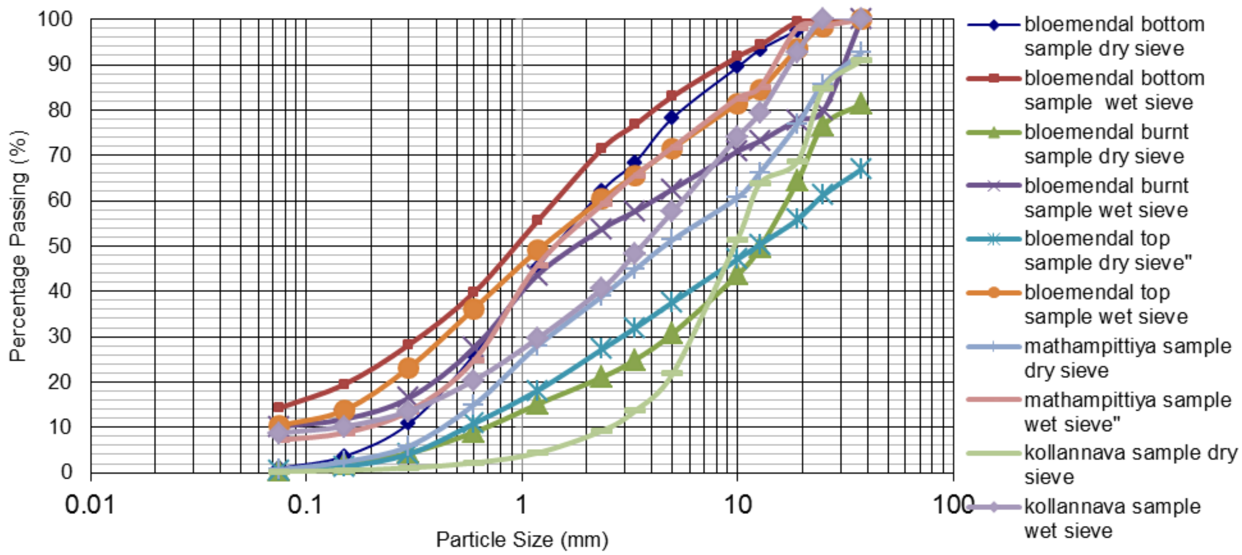

Figure 1. Particle size distribution curves for 5 locations

Table 1: Percentages of each soil type in the samples tested

Sample	Percentage Gravel (76.2mm – 2mm)		Percentage Sand (2mm – 0.075mm)		Percentage Silt & Clay (< 0.075mm)	
	Wash sieving (%)	Dry Sieving (%)	Wash sieving (%)	Dry Sieving (%)	Wash sieving (%)	Dry Sieving (%)
Bloemandhal bottom	33	41	52.7	58.0	14.2	1.0
Bloemandhal burnt	48	92	41.8	7.8	10.2	0.2
Bloemandhal top	43	76	46.6	23.7	10.4	0.3
Madampitiya	55	35	37.8	64.3	7.2	0.7
Kolonnawa	38	8	53.2	91.8	8.8	0.2

3. Results and Discussion

3.1. Particle Size Distribution

A better classification is required for the soil in order to predict possible geotechnical properties of MSW soil. Having its common components of paper/cardboard, plastics, food waste, metal and glasses (Siegel et al., 1990 cited Dixon and Langer, 2006) its classification can vary depending on percentage composition of each of these components, component shape, particle size distribution, component compressibility and degradability. (Dixon and Langer, 2006).

Figure 1 shows the particle size distribution curve for both dry sieve and wash sieve analysis. All samples showed gradual variation of particle size except the burnt sample from Bloemandhal, which is predicted to contain particles stuck together due to burning of materials such as polythene etc, wash sieving gave higher percentage of finer values for all samples considered. Particle size distribution for almost all samples is quite similar to coarse grained soils, with $F_{0.075}$ in the range of 10%- 20%.

3.2. Proctor Compaction Test

Maximum dry density and optimum moisture contents of MSW depend on the maximum size of the particles and also the particle size distribution of the sample (Hettiarachchi et al., 2005; Reddy et al., 2008a). Variation of maximum dry densities and the optimum moisture contents for the tested samples are summarized in Table 2.

Table 2 shows the maximum dry density and optimum moisture content values for collected 5 samples. It can be seen that the maximum dry density for the MSW samples are in the range of 1000 – 1500 kg/m3 which is much low compared to traditional lateritic soils in Sri lanka.

Table 3: Proctor Compaction test results

Sample	Maximum dry density (kg/m^3)	Optimum moisture content (%)
Bloemandhal bottom	1307	22.9
Bloemandhal burnt	985	37.1
Bloemandhal top	1095	34.0
Madampitiya	1443	22.5
Kolonnawa	1337	24.2

Optimum Moisture content of the samples corresponding to those Proctor Compaction tests were lying in a range of 22% - 40%. Further, working range of the moisture content for these samples was wider than traditional soils, which ranged in between 15% to 50% for almost all the cases.

Bloemandhal bottom, Madampitiya and Kolonnawa samples were behaving in a similar manner under the Proctor Compaction whereas top sample and burnt sample from Bloemandhal

Table 2: Results from Direct Shear Test

`	**Description**	**No: 1**	**No: 2**	**No: 3**	**No: 4**	**No: 5**
Bloemandhal bottom	Moisture content (%)	7.69	12.12	22.73	27.78	37.55
	c (kPa)	7.5	16.3	18.8	11.9	20.1
	ϕ (degrees)	23	21	18	20	18
Bloemandhal burnt	Moisture content (%)	14.3	34.6	53.6	63.4	NA
	c (kPa)	24.3	19	12.5	27.2	
	ϕ (degrees)	18	18	21	18	
Bloemandhal top	Moisture content (%)	17.4	27.8	38.9	52.4	NA
	c (kPa)	18	11.6	10.1	8.9	
	ϕ (degrees)	19	19	21	21	
Madampitiya	Moisture content (%)	13.6	20.6	30.3	33.3	NA
	c (kPa)	16.6	8.7	16.3	1.8	
	ϕ (degrees)	23	22	19	17	
Kolonnawa	Moisture content (%)	14.3	28.6	33.3	52.4	NA
	c (kPa)	17.1	14.6	21.2	23.5	
	ϕ (degrees)	21	20	15	20	

Table 4: Compressibility Characteristics of the Samples Tested

Sample	Specific Gravity	Compression Index	Recompression Index	Compression index / Recompression index
Bloemandhal bottom	1.72	0.112	0.0148	7.6
Bloemandhal burnt	1.69	0.14	0.0221	6.3
Bloemandhal top	1.88	0.115	0.037	3.1
Madampitiya	1.71	0.087	0.0083	10.5
Kolonnawa	2.4	0.0343	0.005	6.86

gave comparatively lower maximum dry densities under higher moisture content.

3.3. Direct Shear Test

Friction angle ϕ (degrees) and cohesion c (kPa) values obtained from direct shear tests corresponding to the moisture content values of Proctor compaction test is shown in Table 3. From the graphs obtained for shear stress vs shear displacement it can be seen that almost all the samples were acting as loose soils and both friction angle (ϕ) and cohesion value (c) for all samples varies with the moisture content. Cohesion being varied in a wider range compared to friction angles. The ranges can be observed to be 10kN/m2 to 25kN/m2 for cohesion and 18° to 21° for friction angle.

3.4. Consolidation Test

Compressibility of MSW is an important factor which could increase with the compressible components such as organics, paper, wood, textile etc:

As aging degrades most of these components, compressibility of MSW reduces with time, thus the depth of a MSW fill. (Chen et al., 2009)

Oedometer tests were carried out on all five MSW samples at their Proctor density, from the readings obtained, graphs were plotted for log time vs deformation, square root time vs deformation and log (σ) vs void ratio.

Specific gravity for Bloemandhal and Madampitiya samples lies in the range of 1.65 to 1.9 but the specific gravity of kolonnawa sample is an outlier which has a higher value of 2.4.. Compression index and recompression index for Madampitiya and Kolonnawa were lower, Kolonnawa being much lower than the other. Compression index of MSW is lying in between 3 to 10 times the Recompression index as shown in Table 4.

4.0 Concluding Remarks

Experimental works show that all the selected sites having characteristics very similar to those of traditional Sri Lankan soils. Therefore, construction works on these MSW dump yards could be treated as similar to construction on general soil conditions. However, dump waste could contain a fair amount of organic matters which would decay with time. As some of the dump sites considered in this study had been abandoned for several years, the effect of bio-degradable materials did not influence the results. Long term tests are to be done to assess those properties belong to new MSW yards such as Kolonnawa.

References

- Chen, Y.M., Zhan , T.L.T., Wei, H.Y.H.K. (2009),*'Aging and Compressibility Characteristics of Municipal Waste*'
- Hettiarachchi, C. (2005), *Mechanics of Biofill Lanffill Settlements*, PhD Dissertation, Department of Civil and Environmental Engineering, New Jersey Institute of Technology, Newark, NJ
- Jayasinghe, R, Mushtaq, U., Alyce, T and Baillie, C. (2013) The Garbage Crisis: A Global Challenge for Engineers, Morgan and Claypool publishers.
- Neil Dixon, Ulrich Langer 2006, '*Development of a MSW classification system for the evaluation of mechanical properties*', Waste Management, 26(2006), pp 220-232
- Reddy, K. , Hettiarchchi, H., Parakalla, N. & Gangathulasi, J. 2008a, '*Geotechnical Properties of fresh Municipal Solid Waste at Ochard Hills Land fill, USA*', Waste Management, 29(2008), pp 952-959
- Zhan, T.L.T, Chen, Y.M., Ling, W.A. (2008), '*Shear Strength Characterization of Municipal Solid Waste at the Suzhou Landfill, China*', Engineering Geology, 97(2008), pp 97-111.

STUDY ON THE STRENGTH CHARACTERISTICS OF DUMP WASTES IN SRILANKA;
CASE STUDY OF BLOEMENDHAL DUMP SITE

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ABSTRACT

Shear strength properties are the most important parameters for soils. Due to the heterogeneity, it is important to evaluate those dump waste parameters in dump waste sites too. Dump waste samples were collected from Bloemendhal site for research purposes. The sample was prepared by adding water and then taken for both Proctor compaction test and direct shear test. Direct shear test and unconsolidated undrained tri axial tests were conducted to find the shear strength properties such as cohesion and angle of friction. By adding water, moisture content was increased and optimum moisture content was achieved. For each different moisture content, direct shear test and unconsolidated undrained test were conducted.

Direct shear tests have proven that the angle of internal friction does not vary much with moisture content. But the experimental results of cohesion of dump wastes have proven that the cohesion of dump waste varies with moisture content and the shape of the cohesion vs moisture content graph is same as the shape of the Proctor compaction test. These results indicate that the behavior of the dump wastes is similar to silt soil. Unconsolidated undrained tests have proven that the shape of the cohesion vs moisture content graph is same as the shape of the Proctor compaction test. Because for Unconsolidated undrained test cohesion is the only governing parameter of the shear strength.

Shear strength parameters will vary with the age of the solid waste. So for the future works is to be analyzed the shear strength parameters with the age of the solid waste.

Key words: Dump waste, cohesion, angle of internal friction, optimum moisture content

1. INTRODUCTION

One of the major issues in Sri Lanka is the collection and disposal of dump waste in urban areas. Especially in Colombo city due to the rapid population growth, the amount of waste added to the dump waste areas has been increased.

It is essential to acquire a better understanding of the properties of municipal solid waste to maximize the amount of waste that can be placed in existing and future landfills. So optimization of waste will increase the waste density as well as it will increase the landfill operational life. In addition, to determine that whether this dump waste site can be used as a construction site and, whether the dump waste can be used as a filing material.

The scope of this research is to find out the strength characteristics of the dump waste in Sri Lanka. For that purpose Bloemendhal dump waste site has selected. Bloemendhal dump waste area is the biggest garbage dump area in Sri Lanka. Bloemendhal dump waste area is a desegregated garbage mountain from all sources collected by the Colombo Municipal Council. There are an estimated 58 unmanaged waste dumps in the Western Province, most of which are almost filled to capacity. As these dumps continue burning it creates many health problems too (www.ejustice.lk). So as geotechnical engineers we have to make this land area efficiently. For that purpose sufficient amount of waste sample was collected from Bloemendhal dump waste area and several laboratory tests were conducted.

2. OBJECTIVE

Objective of my research project is to find out the strength characteristics of dump waste in Sri Lanka. Also to determine that whether this dump waste site could be used as a construction site and, whether the dump waste could be used as a filing material.

3. METHODOLOGY

Following methodology steps were followed in this research to reach objectives.

- Learning background about the research project and literature review
- Selection of waste disposal sites to collect samples
- Sample collection for laboratory experiments
- Experimental investigations to find out the shear strength properties of dump waste for collected samples
- Analysis of results obtained from tests

4. EXPERIMENTAL WORK

A series of experiments were conducted according to appropriate guidelines such as BS 1377 and ASTM volume 04.08. Classification of sample and determination of the shear strength characteristics are two identified objectives.

Particle size distribution and consistency limit tests were done to classify the dump waste soil sample. Classification of the dump waste is done with Unified Soil Classification Method.

Chemical parameters of the soil or a fill material will have adverse impacts on structure in that soil such as foundations. Since, dump waste has a high potential to have influence of chemically influenced ingredients, it was decided to perform some general chemical tests such as pH test, sulfite test and chloride test

The proctor compaction test was done to obtain the maximum dry density and optimum moisture content. Then direct shear tests were done to find out the relationship between moisture content and shear strength parameters. Then Unconsolidated Undrained triaxial tests were done to find out the relationship between moisture content and short term shear strength parameters.

5.0 RESULTS AND DISSCUSSION

5.1. PARTICLE SIZE DISTRIBUTION

Particle size distribution is an important physical characteristic of the solid waste. This can be determined by the sieve analysis 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 were bonded together, it will become a larger size particle. Therefore, dry sieve may not provide accurate results in this situation. So wet sieve analysis will give better results of the particle size distribution. In addition, when there are some organic particles present in the waste, which are burnt, to simulate this situation oven dry sieve analysis was done.

Table 1 – Chemical test results

Parameters	Values
P_H	7
Chloride content	0.02 %
Sulfite content	0.086 %

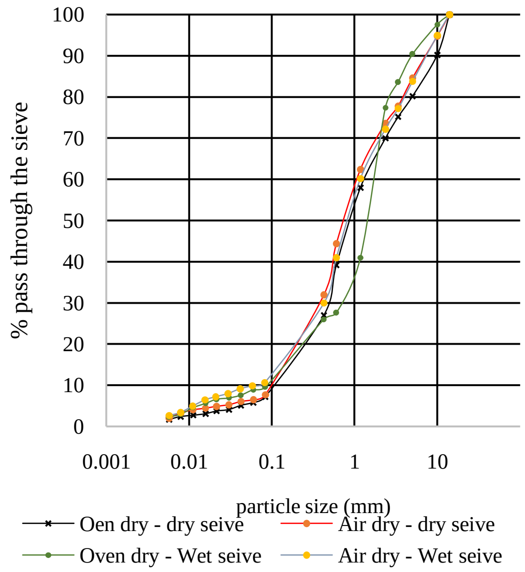

Fig.1 - Particle size distribution

From the above results, for the air dried- dry sieved sample the percentage of clays and silts were 7% and for the air dried –wet sieved sample the percentage of clays and silts were 10%.It means 3 % of particles in the sample were bonded together and when air dried wet sieve analysis were done they were broken into small pieces and they increases the percentage of clays and silts.

From the above results, for the air dried- dry sieved sample the percentage of clays and silts were 7% and for the oven dried –dry sieved sample the percentage of clays and silts were 6%.It means 1% of particles in the sample are burnt in 105^0 temperature when we did oven dried-dry sieved analysis. Classification of soil is done with Unified Soil Classification Method. From the unified classification system, in all situations sample is classified as SW – SM, which is well-graded sand with silt and gravel.

5.2. CHEMICAL TESTS

Chemical parameters of sample such as pH, chloride content and sulfite content were evaluated according to BS 1377: part 3.

Chemical tests reveal that, considered sample is almost neutral with pH 7 and does not have significant chloride or sulfite content. pH value indicates the unavailability of significant amount of other adverse chemical compounds.

5.3 CONSISTENCY LIMIT TESTS

Liquid limit was found by using the penetration method.

Table 2 – Plastic and Liquid limits

Type	LL	PL	PI
Air dry and wet sieve	44.5	26.2	18.3
Air dry and dry sieve	34.5	27.9	6.6
Oven dry and wet sieve	38	27.4	10.6
Oven dry and dry sieve	30	26.6	3.4

From the above results, it can be said that, when the method of Atterberg limit test changes the behavior of the dump waste varies significantly. For example in air dried- wet sieved condition, the soil sample behaves as a highly plastic soil (PI 17-35%). However, in air dried- dry sieved condition the soil behaves as a slightly plastic soil (PI 1-7%).

This is because when the soil sample is wet sieved the clay particles, which were bonded, with larger particles broken into small pieces and increases the plasticity of the dump waste soil.

5.4. PROTOR COMPACTION TEST

Standard proctor compaction test was conducted according to ASTM standard.

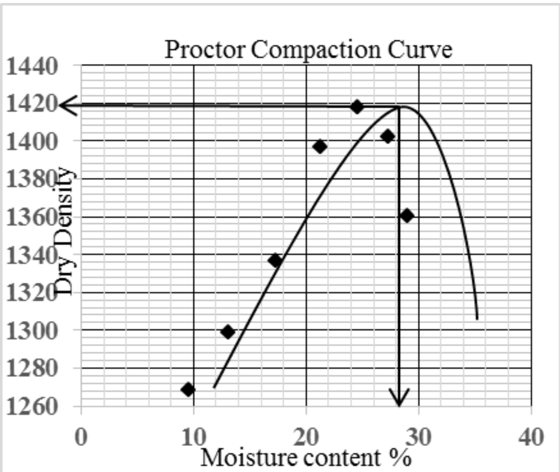

Fig 2. Proctor compaction curve

Maximum dry unit weight is 1418 kg/m^3 obtained at moisture content of 24.5%.

5.5 SPECIFIC GRAVITY

Specific gravity value of sample was evaluated according to BS 1377: part 1, Pycnometer method. Average specific gravity of sample is 1.96.

5.6 DIRECT SHEAR TEST

Firstly, standard Proctor compaction was conducted for different moisture contents. Then for every moisture contents, direct shear test was conducted to find the variation of strength parameters such as cohesion and internal friction angle with different moisture contents.

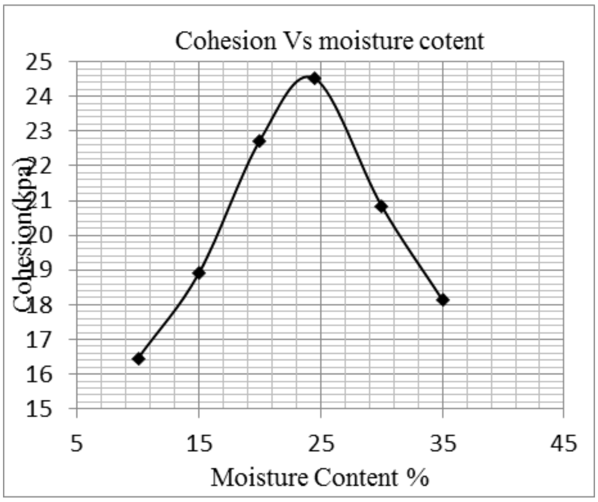

Fig 3.Cohesion variation with moisture content

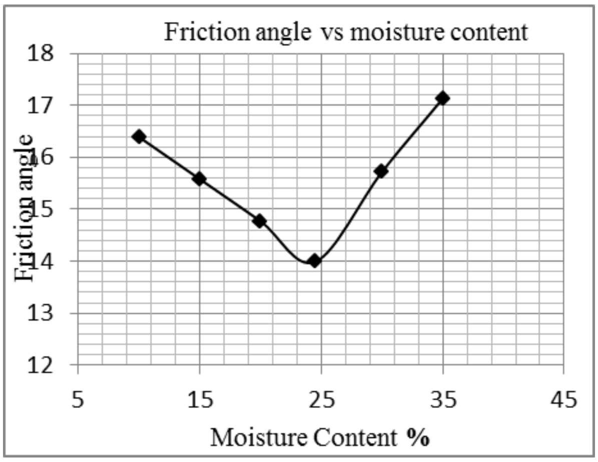

Fig 4.Friction angle variation with moisture content

From the above two graphs, it can be said that the angle of internal friction does not vary much with moisture content (between 14- 17). In addition, behavior of the dump wastes is same as silt soil. However, the experimental work results of cohesion of dump wastes have proven that the cohesion of dump waste varies with moisture content significantly (16.6 kpa- 24.5 kpa).

The below description will be the possible reason for the shape of the above two graphs.

When the dump waste soil is dry of optimum, the soil tends to behave more like cohesion less soil, with a relatively high angle of friction and low cohesion as potentially angular, flocculated structures dominate the shear strength behaviour. As the dump waste soil approaches optimum moisture content the internal angle of friction decreases and the cohesion increases to maximum cohesion at optimum moisture content. This response is due to the breakdown of the flocculated structures, which decreases frictional resistance to shearing and to increasing moisture content, which lubricates the movement of soil particles past each other

5.7 UNCONSOLIDATED UNDRAINED TEST

Unconsolidated Undrained tests were done to find out the relationship between moisture content and short term shear strength parameters of the dump waste.

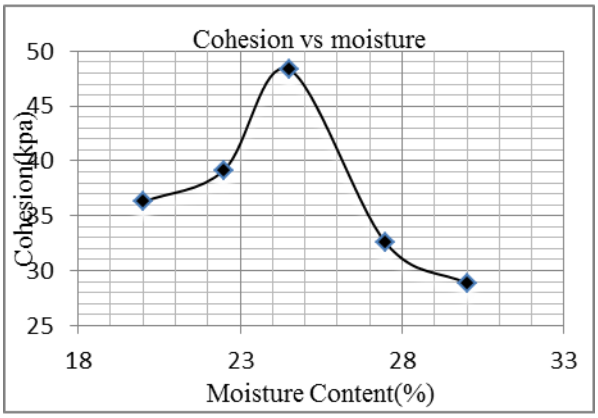

Fig 7.13 Variation of cohesion vs moisture content

The only governing parameter for the shear strength in unconsolidated undrained triaxial test is the cohesion. Because in unconsolidated undrained condition friction angle of soil is assumed to be zero. Therefore, when we plot the graph cohesion vs moisture content, it follows the same graph of Proctor compaction test. In addition, at the optimum moisture content we got the higher cohesion value of 48.35kpa.Obviously it indicates that at the optimum moisture content, the shear strength of the dump waste soil sample is high.

6. CONCLUSION

Geotechnical properties of Bloemendhal dump waste soil were determined through laboratory testing. In particular, particle size distribution, Atterberg limits, chemical properties of soil, specific gravity, and shear strength parameters (friction angle and cohesion) of dump waste were studied. The test results were compared with the relevant published studies. The following conclusions can be drawn based on the results of this study.

Direct shear tests have proven that the angle of internal friction does not vary much with moisture content. However, the experimental results of cohesion of dump wastes have proven that the cohesion of dump waste varies with moisture content and the shape of the cohesion vs moisture content graph is same as the shape of the Proctor compaction test. These results indicate that the behavior of the dump wastes is similar to silt soil. Unconsolidated undrained tests have proven that the shape of the cohesion vs moisture content graph is same as the shape of the Proctor compaction test. Because for Unconsolidated undrained test cohesion is the only governing parameter of the shear strength.

Chemical tests on the dump waste show that Bloemendhal dump waste area is suitable for the construction activities according to those three chemical properties of soil.

Overall, this study utilized that the shear strength parameters significantly vary with the change in moisture content. The variation in shear strength characteristics with moisture content should be properly accounted in the analysis and design of landfills or for the construction activities in existing land.

7. REFERENCES

[1]Zhu et al (2013). Effect of Water Content on the Shear Strength Parameters of Municipal Solid Waste, Vol. 18, Bund.F.

[2]Reddy et al. (2009). Compressibility and shear strength of municipal solid waste under short-term leachate recirculation operations, Waste Management and Research: 27: 578–587.

[3]Cokca et al. (2004). Effects of Compaction Moisture Content on the Shear Strength of an Unsaturated Clay, Geotechnical and Geological Engineering, 22, 2, 285-297.

[4]Bowles, J. E. (1997). Foundation Analysis and Design - Third Edition, Mcgraw-Hill Book Company, Singapore.

[5]Holtz, et al. (1981). An Introduction to Geotechnical Engineering, Prentice Hall, Englewood Cliffs, NJ.

[6]Terzaghi, K., and Peck, R. (1948). Soil Mechanics in Engineering Practice, John Wiley and Sons, New York, NY.

[7] U.P. Nawagamuwa, W.D.S.P. Gunaratne, P. Kirubajiny,T.Thiviya and H.K.A. Priyadharshana (2013) Study on the Geotechnical properties of Open Dumps in Sri Lanka.

[8] www.ejustice.lk

Selection of Suitable Material for Final Cover of an Engineered Landfill in Dry Zone of Sri Lanka

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Abstract

Final cover system is an important component of an engineered landfill to control infiltration of precipitation to minimize leachate generation. Cover soil plays a major role in emission of landfill gases because in Sri Lankan context provision of gas recovery facility is not practiced much. As a result of anaerobic decomposition of waste, generation of methane which has a significant global warming potential, is a severe environmental problem. Therefore improvement of gas ventilation facilities inside the landfill, in order to promote aerobic decomposition, is important in designing landfill final cover, especially in dry zone. The gas exchange through final cover soil is controlled by both diffusive and advective gas transport parameters. Based on laboratory experiments, it was realized that gas transport parameters are significantly vary with the amount of moisture present in the cover material. Further, gas transport parameters can be gradually increased with the increase of air voids present in the cover material.

1. INTRODUCTION

Final cover is an essential part of all municipal solid waste landfill facilities. A well designed final cover controls moisture infiltration from the surface into the closed facilities, limits the formation of leachate, prevent direct exposure of waste to the atmosphere and control gas emission and odor. Regulations have been implemented to design and construct the final cover of the landfill both in local and international context in order to achieve above requirements. Conventional approach has been implemented in most of countries, consisting of number of layers in order to achieve different outcomes [1]. However this design approach is not economical for a country like Sri Lanka and therefore it is important to identify simple cover system to meet the required functioning of the final landfill cover. As the second approach of designing final cover systems, Alternative Final Cover (AFC) system can be identified which equally perform as the conventional approach. The AFC design process is flexible and is predicated on scientific and engineering principles and practices. Geosynthetic Clay Liner (GCL), Capillary barrier, Evapotranspirative cover (ET) are some examples for alternative approach. In order to adapt AFC approach it is required to permit the AFC approach with necessary regulations and guidelines and performance goals [2].

Once the solid waste is placed in a landfill and covered with a soil fill, a complex series of biological and chemical reactions is occurred within the landfill. The anaerobic degradation of organic waste in landfill which is occurred under oxygen limited condition produce mixture of gases that contain methane, carbon dioxide and various other toxic compounds. Since Sri Lanka is a middle developing country, 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 such a way to promote gas (oxygen) exchange between the atmosphere and the waste layer to maintain the aerobic condition and methane oxidation in the waste layer [3].

As infiltration of rain water in dry zone is very less, the generation of leachate in dry zone is comparatively less. Therefore, when selecting material for the final cover of landfill in dry zone, gas exchangeable parameter is the governing factor [4].

Landfill gas emission occurs with two mechanisms, namely diffusion and advection. Diffusion is a principal mechanism in the interexchange of gases between soil and the atmosphere. The soil-gas diffusion coefficient (D_p) governs the gas transport which takes place due to soil-gas concentration gradient. Fick's law is used to calculate the gas diffusion coefficient (D_p) and it can be expressed mathematically as;

$$\frac{\partial C}{\partial t} = \frac{D_p}{\varepsilon}\frac{\partial^2 C}{\partial Z^2} \qquad (1)$$

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. Soil-gas diffusivity (D_p/D_o) is defined as the ratio between soil gas diffusion coefficient (D_p) and gas diffusion coefficient in free air (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 air-filled porosity (ε)[5]. Buckingham (1904) suggested that the soil gas diffusivity follows a power law function of the soil air content such that,

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

where ε is the soil air content and x is an exponent characterizing pore connectivity-tortuosity[6]. The proposed value for x by Buckingham is 2. This model was further modified and the proposed value for x by Marshall is 1.5[7]. Penman (1940) further modified the model [8] proposed by Buckingham and he proposed the relationship of gas diffusivity and soil air content as,

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

Generally the presence of water can significantly affect gas diffusion in soils. In wet soils, water held at bottle necks can potentially create a high tortuous network for gas transport. To consider this effect, Moldrup et al. (2000) developed the water induced linear reduction model for gas diffusivity such as,

$$\frac{D_p}{D_o} = \varepsilon^{3/2}\,\frac{\varepsilon}{f} \qquad (4)$$

where f is porosity of soil [9].

Advection is a gas transport mechanism induced by soil-air pressure gradient. To model the advection flow a parameter called coefficient of air permeability (k_a) is used. Air permeability coefficient is computed based on Darcy's law as;

$$q = \frac{k_a A}{\eta}\,\frac{dP}{dz} \qquad (5)$$

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 x 10^{-5} Pas)[1]. In general 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 can be written as,

$$k_a = \alpha\varepsilon^X \qquad (6)$$

where $\boldsymbol{\alpha}$ is a constant to pore connectivity and x is a power law exponent labeled as a water blocking factor for k_a which can be either 2 (Buckingham type; used by Moldrup et.al, 1998) or 1.5 (Marshall Type)[10].

In this research study, gas transport parameters of a candidate soil to use as landfill final cover material in dry zone were evaluated. Further, results were compared with the existing predictive models for the gas transport parameters.

2. METHOD

The commonly available laterite soil was selected as the candidate material for this research study. The physical properties of the laterite soil are presented in Table 1.

Table 1 – Physical properties of soil

Specific gravity	2.68
Liquid Limit	56 %
Plastic Limit	30 %
Maximum dry unit weight	16.25 kN/m^3
Optimum moisture content	23%

All soil samples were prepared under standard Proctor compaction method by varying moisture content. After completion of each compaction test, 100 cm^3 core samples were collected from the standard Proctor compaction mold. Then D_p and k_a for each sample were measured. Further effect of mixing different percentages of sand with laterite soil on gas transport parameters was studied. Figure 1 and Figure 2 show the schematic diagrams for the air permeameter and the diffusion chamber respectively.

k_a was measured in the laboratory by flowing air at a given inlet pressure through soil samples. Using the pressure gauge atmospheric air pressure and air pressure at the top of the sample was measured. Then k_a was calculated from Darcy's law based on the pressure difference across the soil sample.

The D_p was measured with the diffusion chamber method. Initially, the chamber was flushed with nitrogen in order to remove the oxygen inside the chamber and closed it. Then soil sample was fixed to the apparatus and top of the soil sample is exposed to the atmosphere. Once the chamber was opened, atmospheric air is diffused to the chamber through the soil sample. Oxygen was used as the tracer gas. Change of oxygen concentration was measured as a function of time using the oxygen sensor connected to the diffusion chamber, and then D_p was calculated. In this study, the gas diffusion coefficient of oxygen in free air (D_o) at 20 ^{0}C was taken as 0.20 cm^2/s [11].

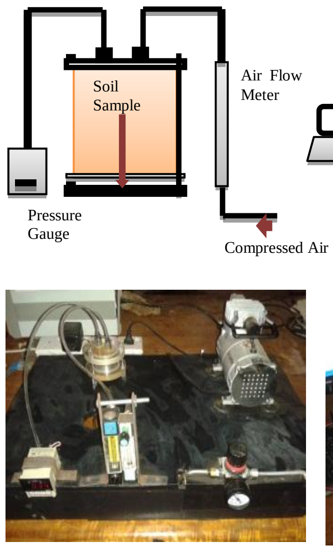

Figure 1- Air permeability Setup

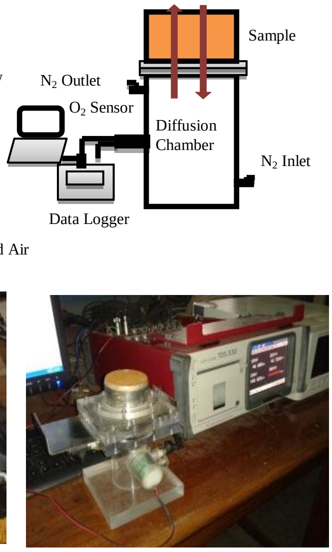

Figure 2 - Gas Diffusion Setup

Falling head method was adapted as the standard test method in determination of saturated hydraulic conductivity since permeability of candidate material is comparatively low. Schematic diagram of saturated hydraulic conductivity test is shown in Figure 3. Laterite soils were prepared with standard Proctor compaction method with different compaction amount and saturated keeping them in a water tank and finally kept on under vacuum for fully saturated. Samples were prepared for both unamended and amended soil of 10% of sand.

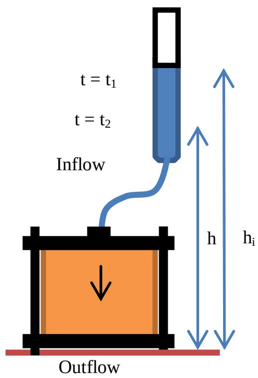

Figure 3- Determination of Saturated Hydraulic Conductivity

3. RESULTS AND DISCUSSION

Figure 4 illustrates the variation of air permeability (k_a) of soil with different percentages of sand as a function of air content (ε). Since samples were prepared with different moisture contents and equal compaction energy was applied, this can be considered as air permeability (k_a) of samples as a function of different compaction amount.. In order to verify the results two graphs of k_a = 250□$^{1.5}$and k_a = 20□$^{1.5}$, existing predictive models for air permeability, are also shown in the graph based on a power law function of soil air content (□), similar to air diffusion. It was selected Marshal type since results of soil-gas diffusivity follows Marshal Model well.

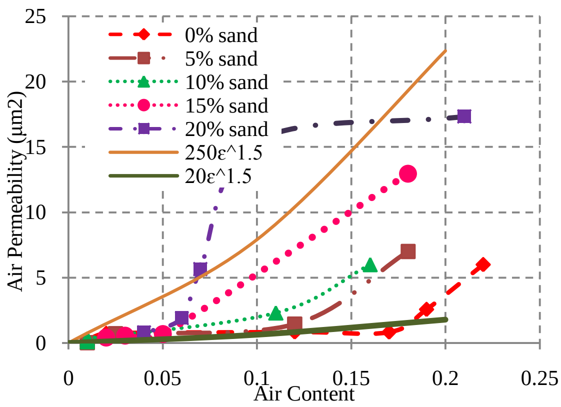

Figure 4- Variation of k_a with ε

It can be noted that k_a value has been significantly increased in the dry side of optimum of soil compaction where as there is no any improvement of k_a value in wet side of optimum of soil compaction irrespective of sand content. Further, when sand content of the cover soil increases, k_a also increases. When sand content is about 20%, there is a significant increase in air permeability. With the increase of sand content, tortuosity of soil has been gradually reduced due to development of higher air content. This implies that air permeability is strongly affected by soil structure.

Figure 5shows the variation of soil-gas diffusivity (D_p/D_0) with air content (ε).

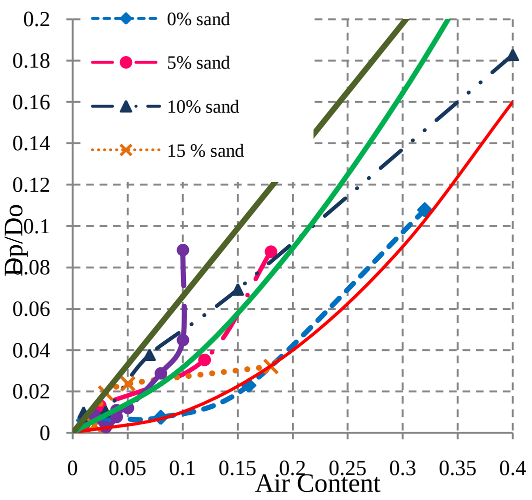

Figure 5 – Variation of $D_p/D_o$with ε

It can be seen that D_p/D_o exponentially increases with the ε irrespective of sand content. However, when sand content of cover material increases, D_p/D_o significantly increases even at low ε. This implies that similar to the air permeability, D_p/D_o is strongly affected by soil structure.

Further, test results are compared with Buckingham, Marshall and Penman models as shown in Figure 4. It can be seen that measured data of unamended soil is well agreed with the Buckingham model where as amended soil with sand is in between Marshall and Penman model when ε <0.15.

Variation of both air permeability and saturated hydraulic conductivity of both unamended and amended soil with 10% of sand with respect to moisture content is shown in Figure 6. The hydraulic conductivity of 1x10^{-7} m/s which is the maximum hydraulic conductivity of landfill final cover soil according to the guidelines of Central Environmental Authority [12] is also shown in the figure.

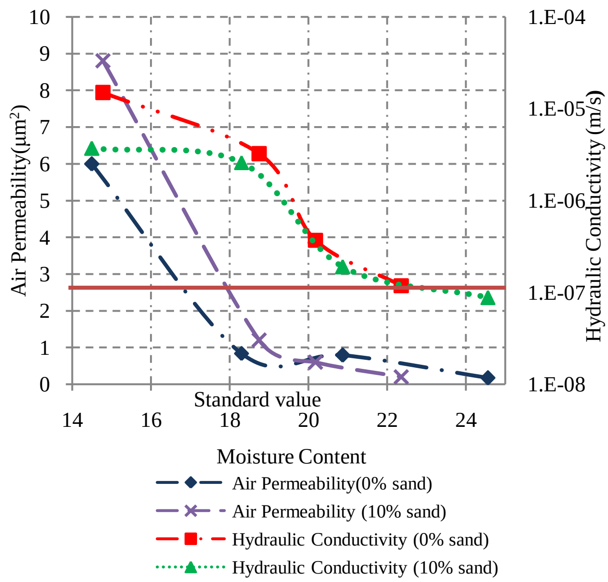

Figure 6 - Variation of air permeability and saturated hydraulic conductivity with moisture content

With the results it can be concluded that both air permeability and saturated hydraulic conductivity is decreased with increment of moisture content. With addition of sand, hydraulic conductivity is getting increased. This increment is significant in dry side of optimum. As a result of increment of coarser fraction, porosity of the material is increased and through the well-connected pore system water is easily passed through the material. However, CEA regulation on landfill cover was satisfied only laterite soil compacted in optimum moisture content and wet side of optimum.

Similarly Figure 7 illustrates the variation of both air permeability and hydraulic conductivity of both unamended and amended soil with 10% of sand with respect to moisture content. The recommended maximum hydraulic conductivity on landfill cover is also shown the figure.

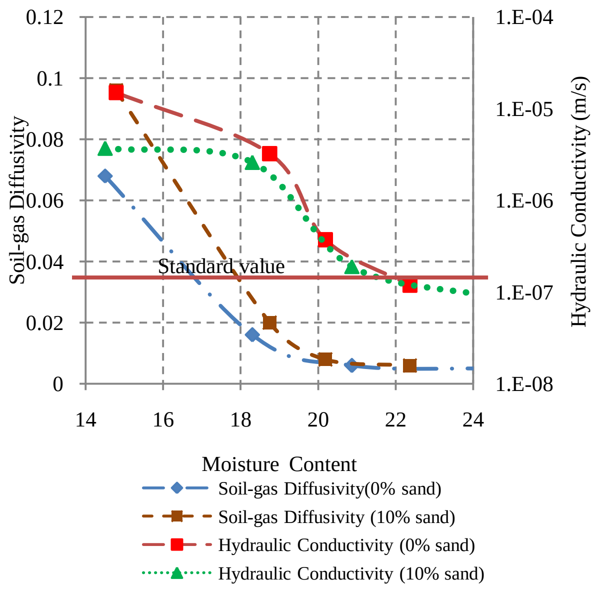

Figure 7- Variation of soil-gas diffusivity and hydraulic conductivity with moisture content

Both soil-gas diffusivity and saturated hydraulic conductivity is decreased with increment of moisture content. Although CEA regulation is satisfied soil compacted in optimum moisture content and wet side of optimum, in this region gas transport parameters are very low.

4. CONCLUSIONS

Series of laboratory experiments were conducted in order to investigate gas transport parameters of cover material. Based on the results, it was realized that both k_a and D_p/D_o are increased with soil-air content. Further, k_a and D_p/D_o are significantly increased in dry side of optimum of compaction irrespective of sand content in the cover material.

According to Sri Lankan regulations, hydraulic conductivity of final cover material should be less than 1 x 10^{-7} m/s and this requirement was satisfied only unamended soil compacted at optimum moisture content and wet side of optimum. However gas transport parameters both air permeability and soil-gas diffusivity is significantly less in optimum moisture content and wet side of optimum.

Therefore if CEA guideline is followed in designing landfill final cover system, highly compacted soil layer should be adapted with an average degree of compaction is more than 95% of maximum dry density. However in this situation, since the volume of air filled pores exit in soil is less, the gas transport through the cover soil is also less. Therefore it can be expected anaerobic condition in the landfill and generation of CH_4 as a result of anaerobic decomposition of waste. There is a severe risk potential of emission of produced landfill gas from waste layer through the loosely compacted points. Therefore it is very important to provide appropriate gas ventilation facilities such as vent pipes or active ventilation systems in order to extract the landfill gases.

When the moisture content of the cover soil material (unamended or amended) is 2-4% less than the optimum moisture content, hydraulic conductivity is slightly higher than the Sri Lankan regulations, however, gas transport parameters are significantly higher in this region. Since, gas transport parameters are the governing factors in dry zone, this region can be identified as the optimum region when designing the landfill cover in dry zone of Sri Lanka.

For economical design of the landfill cover systems, it can be suggested alternative approach for Sri Lanka. Therefore further research is essential to determine the adaptability of suitable alternative approach for dry zone.

REFERENCES

[1] Resource Conservation and Recovery Act, Subtitle D., Subtitle C.US Environmental Protection Agency, 1991

[2] Technical and Regulatory Guidance for Design, Installation,and Monitoring of Alternative Final Landfill Covers(2003),The Interstate Technology & Regulatory Council, U.S.A.(ITRC)

[3] Wickramarachchi P.N.K., Ranasinghe R.H.K.,Nawagamuwa U.P., Kawamoto K.,Hamamoto S., Moldrup P. and Komatsu T. (2010), Measurement of Gas Transport Parameters for Final Cover Soil at MaharagamaLandfill in Sri Lanka, 2010 19th World Congress of Soil Science, Soil Solutions for a Changing World,1 – 6 August 2010, Brisbane, Australia.

[4] Ranasinghe R.H.K., Nawagamuwa U.P, Wickramarachchi P.N.K. and Kawamoto K. (2010), Gas Exchangeable Properties of Maharagama Open Dump Cover Soil, Annual Transactions of IESL, pp 95-102

[5] Suzanne E. Allaire, Jonathan A. Lafond, Alexandre R. Cabralb and Sebastien F. Langea(2008). Measurement of gas diffusion through soils: comparison of laboratory Methods. Journal of Environmental Monitoring,DOI: 10.1039/b809461f

[6] Buckingham, E. 1904. Contributions to our knowledge of the aeration of soils. Bulletin 25. USDA Bureau of Soils, Washington, DC.

[7] Marshall, T.J. 1959. The diffusion of gases through porous media. J. Soil. Sci. 10:79–82

[8] Penman, H.L. 1940. Gas and vapor movements in soil: The diffusionof vapors through porous solids. J. Agric. Sci. (Cambridge) 30:437–462.

[9] Moldrup P. ,Olsen T. , Komatsu T. ,Schjønning P. and Rolsten D.E. (2001). Tortousity Diffusivity and Permeability in the Soil Liquid and Gaseous Phases. Soil Science Society of America Journel 68: pp 750-759.

[10]Kawamoto K, Moldrup P, Schjonning P, Iversen VB, Komatsu T, Rolston DE (2006) Gas transport parameters in the vadose zone: Development and tests of power-law models for air permeability. *VadoseZone Journal* **5**, 1205-1215.

[11]Currie J.A. 1960. Gaseous Diffusion in Porous Media, Part 1- A non-steady state method. British Journel- Applied Physics. 11:314-317.

[12]Technical Guidelines on Solid Waste Management in Sri Lanka, Hazardous Waste Management Unit,Pollution Control Division, Central Environmental Authority

UPLIFT BEHAVIOUR OF SHALLOW FOUNDATION ON COHESIONLESS SOIL

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Abstract: Shallow foundations are widely used in Sri Lanka for communication towers, power transmission towers, silos and wind towers. Several assumptions are made in the evaluation of uplift capacity of such foundations. Therefore, a large factor of safety is applied for the foundation designing, which leads to excessive cost. In view of the above, uplift behavior of shallow foundations on dry sand were investigated in the present study by conducting laboratory model tests and finite element analysis on circular and square type foundation models. Circular and square foundation models were tested at three different depths on dry sand and the uplift force and upward displacement of each model were obtained. Finite element analysis has been done for the above foundation models to verify the experimental results. Uplift force and upward displacement characteristics obtained from numerical analysis was compared with those obtained from experimental investigations. From experimental and theoretical results, it can be concluded that the uplift capacity of square foundation is higher than circular foundation in each depths of embedment and the uplift capacity is increasing with the embedded depth of foundation for both types of foundations. The most important conclusion is that, the angle of the failure plane of soil wedge to the vertical (earth frustum angle) is nearly half of the friction angle of the soil.

Keywords: Uplift capacity, cohesionless soil, earth frustum

1. Introduction

The uplifting force is the most dominant force in the design of tower foundations. It is generally considered that resistance to uplift is provided by both frictional resistance of soil along the failure surface and weight of soil in the failure zone of the foundation.

According to the literature, several theoretical and semi-empirical methods have been developed to predict the ultimate uplift capacity of continuous, circular and rectangular foundations embedded in sand and hardly in clays. Some established theories include Balla's theory (1961), theory of Meyerhoff and Adams (1968), and theory of Vesic. The design equation proposed by Mayerhoff and Adams (1968) considers a rectangular footing only. In general, all the above theories over-estimate the safety of the footing due to the assumptions made in the approximation of the associated failure soil wedge, which is also known as the earth fustrum. Because of these assumptions, there are lots of uncertainties in predicting the behavior of failure soil wedge for different soils and foundations. This requires the application of large factors of safety values, which leads to excessive cost.

Therefore, understanding the earth frustum characteristics of shallow foundations in different soils will be helpful in providing a cost effective design approach.

In the current research, a detailed experimental and numerical analysis was carried out on the failure soil wedge and uplift capacity of shallow foundations on cohesionless soil for square and circular type foundation models.

2. Literature Review

The paper published by Balla (1961) is widely recognized as the pioneer work on tensioned foundations (e.g., Meyerhof and Adams, 1968; Vesic, 1969) which have been followed by a number of researchers around the word.

Stewart (1985), Sutherland (1988) investigated the tensile capacity of layered soils. To account for the inhomogeneity introduced by the compacted backfill, the tensile capacity is controlled by the weaker of the two materials, backfill or surrounding natural soil. If the backfill is weaker than the natural soil, the failure takes place at the vertical interface.

3. Methodology

This study was carried out in three sectios.

1. Experimental investigation
2. Theoretical investigation
3. Numerical investigation

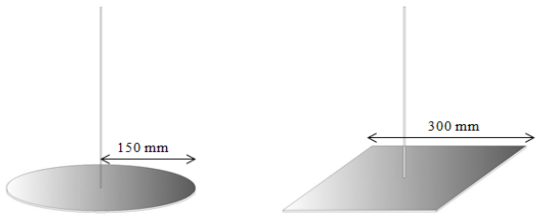
3.1 Experimental Investigation

3.1.1 General

Two types of foundations were used in the investigation, namely flat circular and flat square foundations. Circular and square flat models with the same plan dimensions were tested. The dimensions of all models in plan were kept the same for comparison purposes. Figure 1 shows geometrical configuration of these models. The models were made from steel.

3.1.2 Experimental set up

Internal dimensions of the perspex box were 1300 mmx1200 mmxl000 mm for length, width and height, respectively. The failure pattern



was checked for three different embedded depths of 100m, 200m and 300m. Sand layers were arranged in testing box as shown in Figure 2.

Figure 1: Foundations models

Density of sand in testing box was maintained constant by pouring sand into the box through special bucket, while maintaining a constant falling height of a 1.0m.

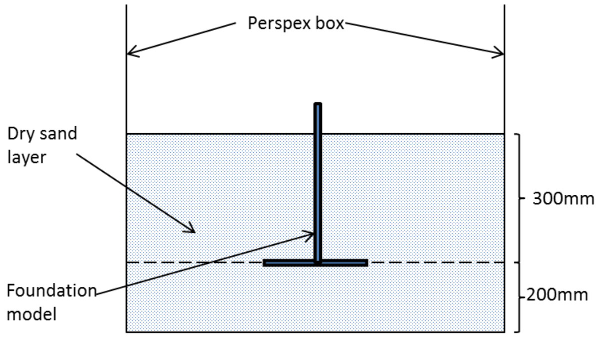


Figure 2: naSd fillsledom noitadnuof eht rednu

The loading system was composed of an drawpu load that generates an puward displacement. A proving-ring of sufficient capacity was connected to the kcaj wercs to measure the applied load. The dial gauge was mounted on the foundation model to measure the displacement of footing during the testing. Refer to Figure 3 for a photo of the loading arrangement.

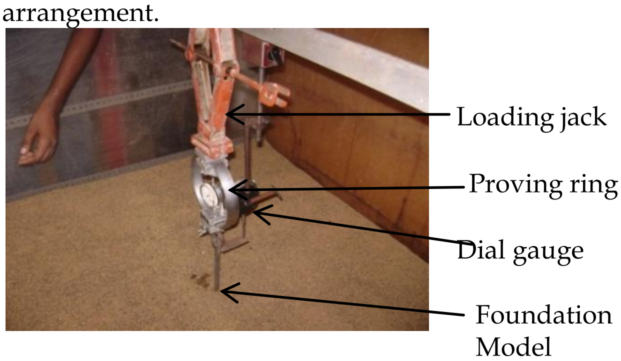


Figure 3: Loading arrangement

3.1.3 Test procedure

After the testing box was prepared, the load application stage was started. The loading was applied under displacement control at a rate of 1 - 2 mm/ min. The proving ring value was measured at every 0.5 mm settlement of foundation model which was measured by dial gauge set on fixed vertical steel rod. At the same time, observations of deformation of surrounding soil were taken into account until the total settlement reached 25 mm.

3.2 Theoretical Investigation

Theoretical investigation was carried out based on formulas proposed by Mayerhof & Adams (1968). The theoretical capacities calculated by following equations.

Equation for circular foundation,

$$T_u = \pi B c_u D + s_f B\gamma \left[\frac{D^2}{2}\right] K_u tan\emptyset + W$$

Equation for square foundation,

$$T_u = 2c_u D(B + L) + \gamma D^2 (2s_f B + L - B) K_u tan\emptyset + W$$

Where,

T_u - Uplift capacity

B - Width of foundation

D – Embedded depth

K_u –$1 - \sin \emptyset$

C_u -Cohesion

W – Weight of foundation + Weight of uplift soil (directly on the foundation)

γ– Density of sand

$\emptyset$ - Friction angle

3.3 Numerical Investigation

Two types of footing models; circular and square were analyzed and compared with obtained experimental results. The finite element software PLAXIS - 2D was used to model circular foundation on sand. The axisymmetric condition and l5-node triangular elements were used for the analysis. Also PLAXIS -3D Foundation software was used to model the square foundation, in 3D. The 3D - parallel planes condition and 15-node wedge elements were used to model the square foundation. In all models, the mesh size was considered to be medium.

4. Results

4.1 Experimental Results

The uplift load and settlement were recorded and plotted for each foundation. Figure 4 shows the settlement versus uplift load curves for square foundations in three different depths which were embedded in dry sand. Figure 5 shows the settlement versus uplift load curves for circular foundations in three different depths which were embedded in dry sand.

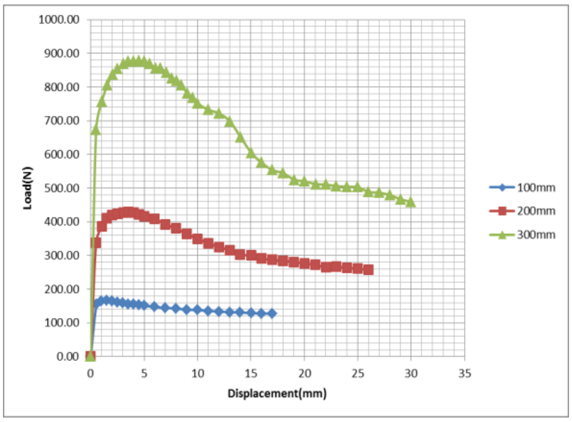


Figure 4- Uplift load-settlement curves for square foundations

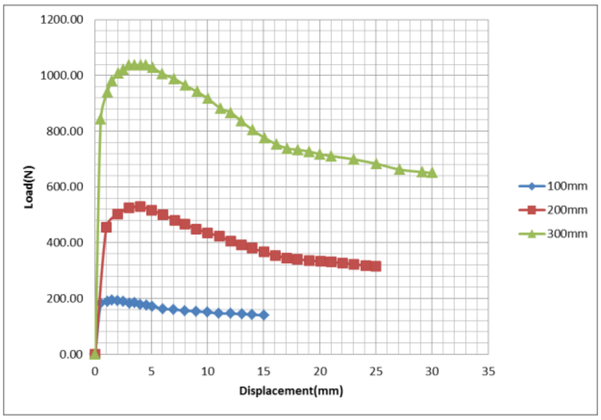


Figure 5- Uplift load-settlement curves for circular foundations

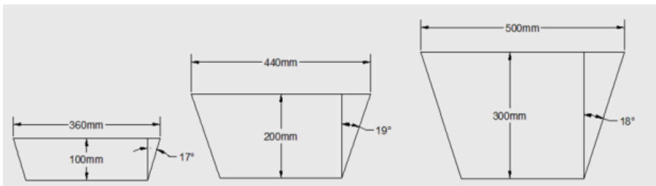
Uplift capacity of each foundation has been tabulated in Table 1.

Table 1: Experimental uplift capacity

Embedded depth (mm)	Uplift capacity of square foundation(N)	Uplift capacity of circular foundation(N)
100	193	166
200	529	428
300	1037	880

Cross section through center of foundation of failure soil wedge has been illustrated in Figure 6 and Figure 7.

Figure 6: Cross section of earth frustum for



square foundation model

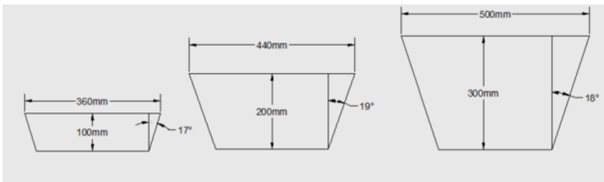


Figure 7: Cross section of earth frustum for circular foundation model

4.2 Theoretical Results

The uplift capacity has been calculated using equations that are given in 3.2. Theoretical uplift capacities has been tabulated in Table 2.

Table 2: Theoretical uplift capacities

Embedded depth (mm)	Uplift capacity of square foundation(N)	Uplift capacity of circular foundation(N)
100	176	139
200	421	332
300	751	591

4.3 Numerical Results

The uplift capacities were found only for the circular foundation models. The values are tabulated in Table 3.The angles of the failure plane were found for the square foundation models using PLAXIS 3D.

Table 3: Numerical uplift capacities of circular foundation model

Embedded depth (mm)	Uplift capacity of circular foundation(N)
100	210
200	578
300	930

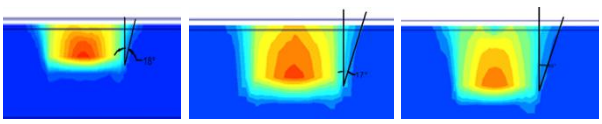


Figure 8: Failure patterns of the soil wedge of square foundation model

The angles of earth frustum obtained from numerical analysis is shown in Table 4.

Table 4: Angles of failure plane of square foundation model

Embedded depth (mm)	Inclination angle to the vertical
100	18^0
200	17^0
300	16^0

5. Results and Discussion

For all the cases the uplifting capacity obtained from the experimental investigation shows higher values than the commonly used theoretical approaches investigated by previous researchers. That is possibly due to the conservative assumptions made in predicting the behavior of earth frustum. From the failure patterns observed during the experiment as shown in the Figure 6 and Figure 7, i is further identified that the angle between vertical plane and failure plane is nearly half of the friction angle of the soil for foundations tested in the current study. Similar observations could be made from the numerical results.

This experimental results are in agreement with the theoretical results obtained from formulas proposed by Mayerhof& Adams. The significant difference could be due to higher weight of soil in pullout zone and frictional resistance of the failure surface of the square foundations.

According to results, uplift capacity of both circular and square foundation models show higher values for higher depths as the weight of the soil wedge is increasing with the embedded depths.

6. Conclusions

The following conclusions are drawn from the results of the current study.

The currently used theoretical formulas for calculation of uplift capacity to design the shallow foundation show significantly a lower value for the uplifting capacity than the experimental value for foundations in cohesionless soils. In order to obtain a highly reasonable value for the uplift capacity that is used for design purpose it is advisable to take the earth frustum angle as half of the friction angle for cohesionless soils.

It can be also suggested that square shape is the most ideal geometrical shape for a shallow foundation in cohesionless soil subjected to uplift forces.

References

Balla, A. (1961). *"The Resistance to Breaking out of Mushroom Foundations for Pylons"*, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, Paris, pp. 569-576.

Mayerhof, G. G. and Adams J. I. (1968). "The Ultimate Uplift Capacity of Foundations", Canadian Geotechnical Journal, Vol. 5, No. 4, pp. 225 – 244.

Sutherland, H. B. (1965). "Model Studies for Shaft Raising through Cohesionless Soils", 6th International Conference on Soil Mechanics and Foundation Engineering, Vol 2, Montreal, pp. 410-416.

SHRINKAGE CHARACTERISTICS OF CLAY LINER MATERIALS

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Abstract: *In many parts of the world compacted clay liner materials (CCL) are widely being used in the municipal solid waste landfills to prevent leachate contamination of nearby water resources. However, during dry season formation of desiccation cracks in the clay liner can result in leachate infiltration during the subsequent rainy season. This paper discusses the shrinkage characteristics of expansive clay materials for use as a landfill liner material in Sri Lanka. Expansive clayey soil from Moragahakanda (North central province of Sri Lanka), water treatment plant sludge from Meewathura (water treatment plant in Kandy Sri Lanka) and which amended by 5% and 10% bentonite were tested to determine the volumetric shrinkage characteristics. Water treatment plant sludge exhibited large amount of cracks at high moisture content during the consolidation process which can lead to crack propagation and subsequent leachate infiltration during a rainy season. Therefore, water treatment sludge was excluded from further investigation of volumetric shrinkage characteristics. In this study volumetric shrinkage limit tests were conducted on the Moragahakanda soil samples consolidated at 50 kPa, 100 kPa and 200 kPa. Variations of volumetric shrinkage limit with consolidation pressure and percentage of bentonite were investigated. It is observed that the volumetric shrinkage increases with the consolidation pressure and the percentage of bentonite. This implies that when the solid waste in a landfill accumulates the shrinkage limit of the clay liner material too increases. Atterberg limits tests also were carried out for samples cured 48 hours at different curing moisture contents. It is observed that the liquid limit decreases and plastic limit increases with the increase of curing moisture content which also resulted in decrease of plasticity index.*
Keywords: *Shrinkage limit, landfill liner, expansive soil, leachate, Bentonite*

1. Introduction

Accumulation of solid waste is one of the biggest challenges that real world finds difficult in overcoming and major problems presented by waste disposal facilities is the formation of leachate. Leachate originates from municipal refuse which contains appreciable amount of contaminants that may endanger public health and the environment, if allowed to percolate into groundwater without protective measures to limit and control its migration. Since most people depend on groundwater for drinking and agricultural purposes, there is need to protect its purity. Therefore, there is need to assess and improve the clay liner materials proposed for liners and protective covers are suitable for use. Expansive soils are used in engineered landfills to prevent leachate infiltration into nearby water resources. Liners made of expansive soils have the advantages of being able to absorb large volume of leachate and selfheal the cracks formed during a long dry period with the subsequent rainy season. However, it is imperative to control the formation of shrinkage cracks during a prolonged dry period, as if not completely healed during the subsequent rainy season, it could affect the intended functionality as an impermeable liner. This paper describes shrinkage characteristics of clay liner materials through experimental investigations. Effects of percentage of bentonite added and the consolidation pressure on shrinkage limit of clay liner materials were investigated through an experimental series and their inter-relationships are presented. The effect of curing moisture content on plasticity index of expansive clayey soils is also investigated in this study

2. Literature review

Expansive soils can swell by more than 100% and shrink by more than 50% of its original volume.Swelling, swelling pressure and compressibility of bentonite mixed sand generally increases with bentonite percentage (Agus Setyo Muntohar, 2003). Drying process of landfill liner develops the capillary suction in the upper layer because of the curved water surface tension effects and this suction makes new arrangement of soil particles as some part of the soil tends to shrink. When the tensile stresses of the soil matrix exceeds the tensile strength desiccation cracks occur on the soil surface (Chao-Sheng Tang et.al, 2011). Desiccation cracks in a clay liner are known to modify its mechanical and hydraulic characteristics such as soil compressibility, consolidation rate, and strength of clay liner

(Morris, 1992).Swelling behaviour of a soil can be expected when the percentage of expansive clay minerals in the soil exceeds 5% by weight (Manosuthikij, 2008).Expansive soil liners have not only the cracking problems but also heaving, soil curling and lift off as identified by numerous researchers (Kodikaraet.al., 2004). Shrinkage cracking of expansive soils during dry environments could lead to enlarged soil heaving in wet conditions as surface shrinkage cracks will allow much more moisture access in to under lying expansive soils and this results in further heaving (Poor, 2004).

3. Materials and Methods

Three candidate soil samples obtained from Moragahakanda L1 (GPS coordinates 7°35′32.7″ N, 80°49′59.8″ E) Moragahakanda L2 (GPS coordinates 7°33′31.0″ N, 80°50′43.7″ E) and Meewathura (Kandy) water treatment plant sludge were used for this investigation. Index properties are listed in Table 1. From the index tests Moragahakanda L1 and water treatment plant sludge were selected for further investigation since they exhibited high plasticity indices.

Consolidation tests were carried out using the Rowe cell apparatus to prepare the specimens required for volumetric shrinkage limit test. During the consolidation process of the water treatment plant sludge, development of large number of cracks was observed at high moisture contents. Since it exhibited large amount of cracks at high moisture content which can lead to formation of cracks and leachate infiltration during a rainy season, this sample was excluded from investigation of volumetric shrinkage characteristics.However, no cracks were observed by naked eye on Moragahakanda L1 sample during the consolidation process.

Finally, shrinkage characteristics were analyzed for Moragahakanda L1 expansive soils amended with different proportions of bentonite. Initially, the effect of curing moisture content was investigated through index tests. The test material was air dried crushed and sieved through 425 µm sieve. Sieved sample amended with 5% and 10% Bentonite by weight and mixed with distilled water at four different moisture contents. Prepared specimens were covered and allowed to cure for 48 hours inmoisture controlled room and index tests were conducted.

Table 1: Physical properties of liner materials

Soil properties	Mora(L1)	W.T.P Sludge
LL(%)	68	158
PL (%)	23	95
Specific gravity	2.72	2.56
Sand (%)	27	4
Silt (%)	21	39
Clay (%)	51	57
Swelling pressure (kPa)	156	-
Swell index (%)	3.16	-
Soil Type(BSCS)	CH	ME

(a) **(b)**

Figure 1: (a) Moragahakanda L1 and (b) water treatment plant sludge consolidated specimens

For the analysis of shrinkage characteristics test material was air dried, crushed, and sieved through 425 µm sieve and mixed with 5% and 10% bentonite by weight. Initially saturated soil slurry was prepared by mixing the soil with distilled water at moisture contents of 65%, 90%, and 110% by weight for unamended, 5%, and 10% bentonite amended samples respectively. Prepared slurry was covered allowed to cure for 24 hours in moisture controlled room. Slurry was consolidated by using the Rowe cell apparatus until the primary consolidation is completed under 50 kPa, 100 kPa, and 200 kPa pressure. From the consolidated samples four specimens each of dimensions 38 m diameter and height between 38 mm and 50 mm were cut and volumetric shrinkage limit test was conducted.

The measurement of volumetric shrinkage limit of consolidated samples was carried out using Mercury method in accordance with the British standard (BS-1377:1990 Part 2; Test Method 6.3). Four specimens were cut from each consolidated samples using 38 mm diameter core cutter. Shrinkage curve was constructed for all specimens by measuring the volumetric shrinkage while slowly drying the samples in ambient conditions over several days. Once the specimen reaches constant volume at subsequent three measurements specimens were dried at 105˚C and the dry weight measurements were taken to calculate moisture contents at various stages. During a prolonged dry period clay liner materials could develop cracks due to shrinkage. It affects the intended functionality of an impermeable liner during the rainy season to follow. Specimen prepared using Moragahakanda L1 sample did not show formation of surface cracks during the experimental process, even after being oven dried at 105˚C .This behavior of Moragahakanda L1 expansive soil can be used as an advantage for use in municipal solid waste landfills, effectively preventing leachate

infiltration through the cracks of landfill liners.

(a) (b)

Figure 2: (a) Test specimen after consolidation (b) Test specimen after being oven dried

4. Results and Discussion

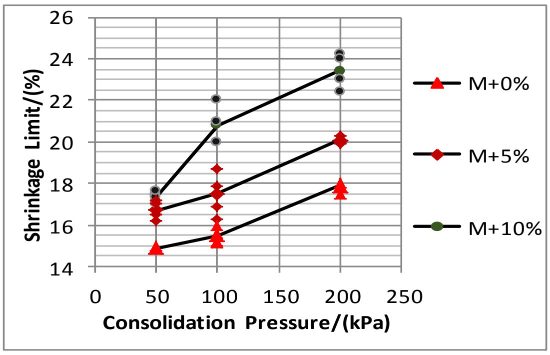


Figure 3: Variation of shrinkage limit with consolidation pressure

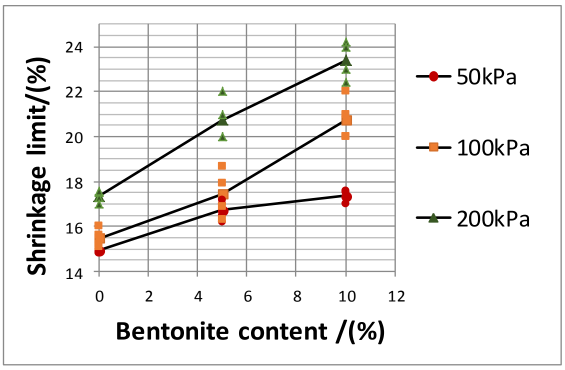


Figure 4: Variation of shrinkage limit with amended Bentonite

Four shrinkage limit tests were carried out for each consolidated sample and the average results obtained for each sample are plotted in figure 3 with the increase of consolidation pressure shrinkage limit also increases and the effect of amended bentonite on shrinkage limit of soil bentonite mixtures also plotted in figure 4 comparing the results of unamended and bentonite amended mixtures shrinkage limit increases with percentage of amended bentonite. Volumetric strain at shrinkage limit (VSS) increased with percentage of bentonite because soils with higher clay content or plasticity index have a greater affinity to water; with the amount of moisture loss volumetric strain also increases.

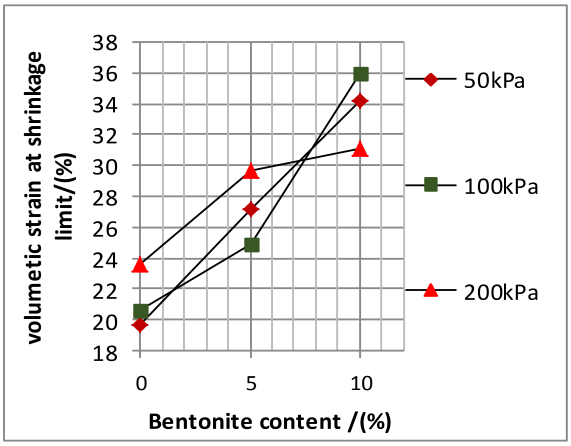


Figure 5: Variation of volumetric strain with Bentonite content

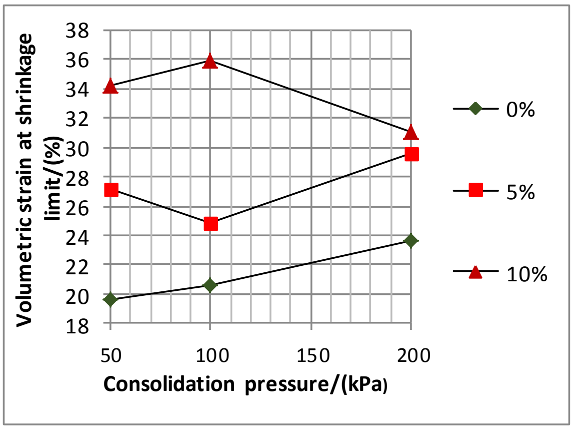
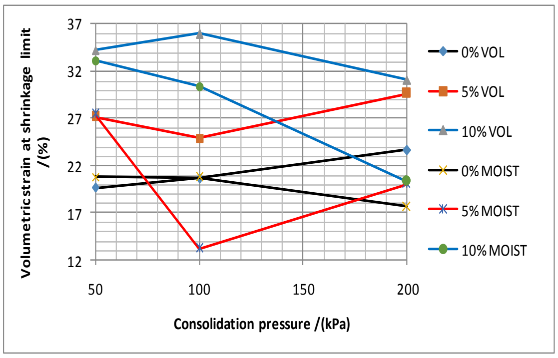


Figure 6: Variation of volumetric strain with Consolidation pressure

Volumetric strain of consolidated specimens during drying mainly depends on loss of moisture and the suction (Surface tension) forces which develop inside the soil matrix. Although, the total moisture loss cannot be associated with the change in volume, the volume change over and above the volume of moisture loss can be attributed to the shrinkage due to particle movement which in turn is an indication of the suction developed. To carry out a comparative study of these two factors; volumetric strains due to (1) moisture loss (2) total volume reduction in the specimens at shrinkage limit are plotted in figure 6. It is observed that difference between these two volumetric strains increases with consolidation pressure and generally the strain caused by moisture loss is lower than the total volumetric strain in the



specimen. It is indicate that the specimens shrink due to the developed suction also.

Figure 7: Variation of shrinkage limit with amended Bentonite

5. Conclusion

Volumetric shrinkage characteristics of clay liner materials under different consolidation pressures were examined in this study. Experiments were performed on an expansive soil and that with Bentonite mixtures under three different consolidation pressures. The following summarizes the major conclusions from this study

1) Shrinkage limit increases with the percentage of added Bentonite

2) Shrinkage limit increases with the increase of consolidation pressure
3) With the increase of curing moisture content plasticity index of soil Bentonite mixtures decreases.
4) Volumetric shrinkage strain (VSS) increases with percentage of amended bentonite
5) Different between Volumetric shrinkage strains (VSS) calculated based on moisture loss and volume reduction increases with consolidation pressure

6. Acknowledgements

This work was supported by the SATREPS project funded by Japanese International Cooperation Agency, Japan Science and Technology Agency and the Department of Civil Engineering. The support given by Mr.N.G.Somapala, Deputy Director, Mahaweli Authority to collect the soil samples from the Moragahakanda project area is gratefully acknowledged.

7. References

AgusSetyoMuntohar (2003). Swelling & Compressibility Characteristics of Soil – BentoniteMixtures. Journal of Geotechnical Engineering,ASCE, 93-98.

Chao-Sheng Tang, Bin Shi, Chun Liu, &I.Inyang, H. (2011). Experimental investigation of the desiccation cracking behaviour of soil layers during drying. Journal of Materials In Civil Engineering,ASCE, 873-878.

Day, R. W., & Fellow. (1994). Swell - Shrink Behaviour of Compacted Clay. Journal of Geotechnical Engineering, ASCE, 618-623.

Kodikara, J.K., Nahlawi, H., and Bouazza, A. 2004.Modelling of curling in desiccating clay. Canadian Geotechnical Journal, 41(3): 560–566. doi:10.1139/t04-01

Manosuthikij, T. (2008)."Studies on Volume Change Movements in High PI Clays for Better Design of Low Volume Pavements" Doctor of Philosophy Dissertation.

Morris, P. H., Graham, J., and Williams, D. J. (1992)."Cracking in drying soils."Can. Geotech. J., 29, 263–277.

Poor, A.R. (1974).''Experimental residential foundation design on expansive clay soils.''Rep. No. TR-3-78, Final Rep., Construction Research Center, The Univ. of Texas at Arlington, Texas.