www.cafetinnova.org

Indexed in
Scopus Compendex and Geobase Elsevier,
Geo-Ref Information Services-USA, List B of Scientific Journals, Poland, Directory of Research Journals

ISSN 0974-5904, Volume 09, No. 04

International Journal of Earth Sciences and Engineering

August 2016, P.P.1458-1464

Establishing Relationships for Strength Characteristics of Lateritic Soils with Varying Silt Fractions

ANIL KUMAR[1], VARGHESE GEORGE[1] AND SRIRAM MARATHE[2]
[1]*Deptt. of Civil Engrg., National Institute of Technology Karnataka, Mangalore-575025, India*
[2]*NITK, Surathkal, and Assistant Professor, NMAMIT, Nitte, India*
***Email:** anil_nitk@yahoo.com, varghese-g@Lycos.com*

Abstract: Design and construction of highway embankments constitute a major component of highway engineering science. Poor sub-grade strength, overloading due to traffic loads, and seismic vibrations can cause distress to pavement sub-grades and embankments. Inadequate compaction and poor sub soil drainage, in addition to low bearing strength of soils cause failure of embankments especially in submersible regions. The present study is focused on performing investigations on the engineering properties of lateritic and lithomargic soils and the effect of fines on soil strength. Tests such as California Bearing Ratio (CBR), tests for unconfined compressive strength (UCS), and tri-axial tests are carried to study the strength behavior of soil on addition of lithomargic soils. Additionally, the development of regressions will help field engineers in estimating the value of the CBR based on simple laboratory experiments such as Unconfined Compression strength test, and the Tri-axial test

Keywords: *Lateritic soil, Lithomargic soils, California Bearing Ratio, unconfined compressive strength*

1. Introduction

Mangalore is one of the fastest growing cities in India with a major thrust on development of infrastructure, and investment in the industrial sector, aimed at a growth rate that surpasses the projected growth rate of India.

Soil in the region of Dakshina Kannada, and the most parts of Southern peninsular region of India is mainly interspaced with *Lateritic* and *Lithomargic soils* that constitute about 40% of the soil in this region (Rao, 2008). *Lithomargic* or *silty* soils, also locally known as *Shedi* soils, are typically weak soils with high silt content and lesser amounts of clay. Engineers engaged in road construction activities often encounter the need to use *lateritic* and *lithomargic* soils as part of construction of embankments and road sub-grades. It is known that though *lateritic* soils are strong in dry conditions, these soils tend to lose about 30-40% of the strength when exposed to moist climatic conditions. However, *lithomargic* soils tend to lose about 40-80% of the strength on similar conditions. The main objective of this work is to investigate the effect of fines on *lateritic* and *lithomargic* soils present in this region.

1.1. Objective of study

The study focuses on performing investigations on the strength characteristics of *lateritic* and *lithomargic* soils occurring in Dakshina Kannada region of Southern India. The following were the objectives of the study:

- To perform studies on basic properties including Atterberg's limits and grain size distribution of various blends of *lateritic* and *lithomargic* soils (silty);
- To determine the strength characteristics of various soil blends of *lateritic* and *lithomargic* soils using the California Bearing Ratio (CBR) method, and tests for unconfined compressive strength (UCS);
- To analyze variations in stresses due to the effect of fines on various blends of *lateritic* and *lithomargic* soils using the stress-strain curves obtained based on the tests for UCS; and
- To develop correlations between the results obtained based on the tests for CBR, and UCS and the tests using the tri-axial test equipment.

2. A Brief Review of Literature

Many researchers have attempted to study the effect of strength behavior of soil on addition of silt. Important studies related to the effect of fines on soil strength are presented below.

Barksdale (1972, 1991), and Thom and Brown (1988) observe that the effect of increase in percentage of fines content generally increases the magnitude of deformation of soil samples tested. Dodds et al. (1999) confirmed that the addition of 10% of fines resulted in a maximum deformation on aggregates used as sub-grade and sub-base.

Osinubi *et al.* (2012) performed studies on *lateritic* soils to determine index properties, compaction characteristics, strength properties including unconfined compressive strength (UCS), un-drained shear strength parameters, and permeability of reconstituted soils. The results from these studies revealed a reduction in the maximum dry density

(MDD) with respect to an increase in fines content, while the optimum moisture content (OMC) was observed to increase. The UCS and the angle of shearing resistance were found to decrease with increase in fines, while the cohesion increased. Additionally, the permeability was found to improve with higher fines content.

Bayoglu (1995) performed investigations on the effect of various percentages of fines on shear strength of soil samples comprising particles with grain sizes varying between that of sand to silty-clays. The results showed that as the percentage of fines increased, the angle of internal friction angle reduced significantly.

Georgiannou (1988) performed studies on the behavior of clayey sands under monotonic and cyclic loading. The studies revealed that the fines content influenced the stress-strain response of the soil mass significantly. Also, it was observed that an increase in the fines content suppressed the dilatant behavior of the soils.

3. Study Area, Methodology and Basic Properties of *Lateritic* and *Lithomargic* Soil Blends

The present study is conducted on soil that is found to occur most commonly in the coastal regions of the District of Dakshina Kannada, and the peninsular areas of most parts of Southern India.

The soils in this region predominantly comprise *lateritic-lithomarges* and *lithomargic-laterites*. Soil samples with purely *lateritic* characteristics were designated as blend B1 (*100%L+0%S*), and soil samples with purely *lithomargic* characteristics were designated as blend B5 (*0%L+100%S*).

Various blends B2, B3, and B4 were then prepared with 75%, 50% and 25% *lateritic* soil. The basic soil characteristics including the index properties of purely *lateritic* soils (B1) and *purely lithomargic* soils (B5) are compiled in Table 1, while Table 2 provides similar details for soil blends B2, B3, and B4.

The tests for soil strength were performed based on the California bearing ratio (*CBR*), and the unconfined compression strength (*UCS*) for soil samples prepared at molding water contents at the dry-side of optimum (OMC-3%) designated as M_1, at optimum (OMC) designated as M_2, and at the wet-side of optimum (OMC+3%) designated as M_3.

Table 1: *Properties of lateritic and lithomargic soil*

Type of soil used	***Lateritic soil B_1***	***Lithomargic soil B_5***
Specific gravity	2.54	2.30
Atterberg's limit		
Liquid limit (%)	47.2	62.2
Plastic limit (%)	24.4	28.4
Shrinkage limit (%)	20.2	23.4
Gravel (%)	30.0	0.0
Sand (%)	56.0	12.0
Fines (%)	14.0	88.0
IS Soil classification	SM	MH
MDD (kN/cu.m)	18.7	12.8
OMC (%)	17.7	21.4

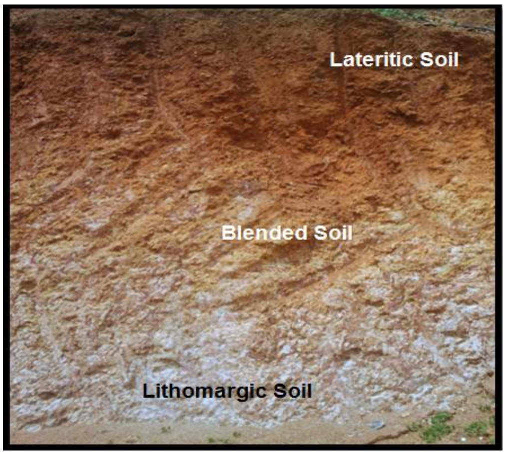

Figure 1 *Soil profile*

Table 2: *Properties of soil blends*

Type of soil used	B_2	B_3	B_4
Specific gravity	2.52	2.44	2.37
Atterbergs limit			
Liquid limit (%)	50.0	55.1	58.9
Plastic limit (%)	25.2	26.0	27.4
Shrinkage limit (%)	20.6	21.1	22.8
Gravel (%)	20.0	18.0	10.0
Sand (%)	38.0	38.0	22.0
Fine (%)	42.0	54.0	68.0
IS Soil classification	SM	MH	MH
MDD (kN/cu.m)	18.0	17.2	16.4
OMC (%)	14.8	16.2	18.8

The tests for CBR values were performed using moulds of 150mm diameter, and 125mm height according to *IS: 2720 Part-16 (1979)* for various blends of soils and for various moisture contents as mentioned above. Investigations were performed on un-soaked soil specimens and on samples soaked for 4 days, and the results of the CBR tests performed are provided in Table 3.

Table 3: *Results of the tests for California Bearing Ratio (CBR)*

Sl. No.	Soil Blends	Water Content	CBR (%)	
			Un-Soaked	Soaked
1	B_1	M_1	36.1	9.7
		M_2	36.0	11.0
		M_3	34.0	10.0
2	B_2	M_1	28.0	9.2
		M_2	29.4	8.4
		M_3	18.0	7.0
3	B_3	M_1	22.0	6.2
		M_2	25.2	7.9
		M_3	15.4	5.7

4	B_4	M_1	17.0	3.0
		M_2	21.0	7.9
		M_3	14.6	2.0
5	B_5	M_1	10.4	4.25
		M_2	11.2	2.0
		M_3	7.0	2.0

The tests for unconfined compressive strength (UCS) were conducted according to *IS: 2720 Part X (1973)* on remolded soil specimens of 38mm diameter with 76mm height for various soil blends and moisture contents. See Figure 2.

Additionally, tri–axial tests were also performed for unconsolidated and un-drained (*UU*-test) soil specimens according to *IS: 2720 Part XI (1981)* and *IS: 2720 Part XII (1981)*.

Tests were performed on various blends of molded soil samples of 38mm diameter and 76mm height at various moisture contents for cell pressures of 0.1MPa (or 1.0 kg/cm^2), 0.15MPa (or 1.5 kg/cm^2) and 0.20MPa (or 1.5 kg/cm^2).

Figure 2 *UCS test setup*

Table 4: *Results of the Tests for Unconfined Compressive Strength (UCS)*

Sl. No.	Soil Blends	Water Content	UCS (MPa)
1	B_1	M_1	0.360
		M_2	0.449
		M_3	0.371
2	B_2	M_1	0.263
		M_2	0.327
		M_3	0.253
3	B_3	M_1	0.205
		M_2	0.303
		M_3	0.201
4	B_4	M_1	0.181
		M_2	0.288
		M_3	0.209
5	B_5	M_1	0.141
		M_2	0.179
		M_3	0.155

Figure 3 provides details on the test setup for the same. The results for the static tri-axial tests are provided in Table 5.

Figure 3 *Tri-axial Test Setup*

Table 5: *Results of the Tri-axial Tests for Various Soil Blends*

Sl. No.	Soil Blend	Water Content	Shear Parameters	
			C (kPa)	Ø°
1	B_1	M_1	33.0	41.2
		M_2	36.0	44.0
		M_3	16.2	30.4
2	B_2	M_1	40.3	26.4
		M_2	38.3	32.7
		M_3	46.0	27.8
3	B_3	M_1	39.0	27.5
		M_2	52.5	27.0
		M_3	80.0	16.3
4	B_4	M_1	67.5	21.4
		M_2	75.0	23.1
		M_3	97.5	14.7
5	B_5	M_1	68.3	13.3
		M_2	88.7	11.5
		M_3	125.0	9.4

4. Development of correlations based on the soil strength determined using the CBR, UCS and the tri-axial tests

This section focuses on the development of relationships between the observations made based on the CBR, UCS and tri-axial tests. Regressions were developed using the statistical package for the social sciences (*SPSS*).

Out of the results compiled in Table 3, Table 4, and Table 5 above, information pertaining to 10 rows of these tables were randomly selected for the development of regressions, while the data in the remaining 5 rows for samples designated as B_1M_1, B_2M_2, B_3M_3, B_4M_1, and B_5M_2, were used for validating the regressions developed.

4.1. Development of a regression between test results for the *CBR* and the *UCS*

A regression between the *CBR* values for un-soaked soils (CBR_u) and the *UCS* was developed as shown in Eq.1 and a scatter plot for the same was obtained as in Figure 4. The R^2 value for the regression was 0.84 and the adjusted R^2 was 0.82. The standard error of estimation (SE) was found to be 4.10, while the values of F-test, and t-test were 42.71 and 6.54 respectively for a significance level of 0.01.

$$CBR_u = 91.88\, UCS - 2.611 \qquad Eq.(1)$$

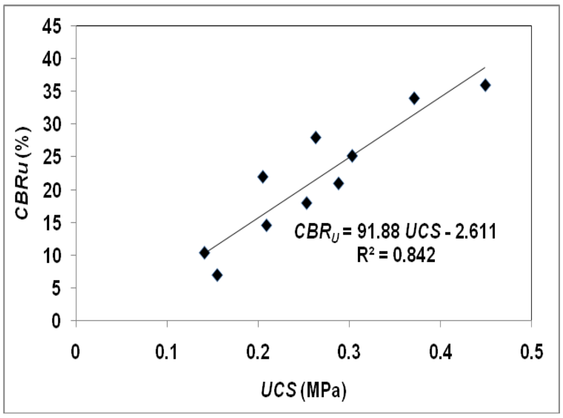

Figure 4 *Correlation between CBR_u and UCS*

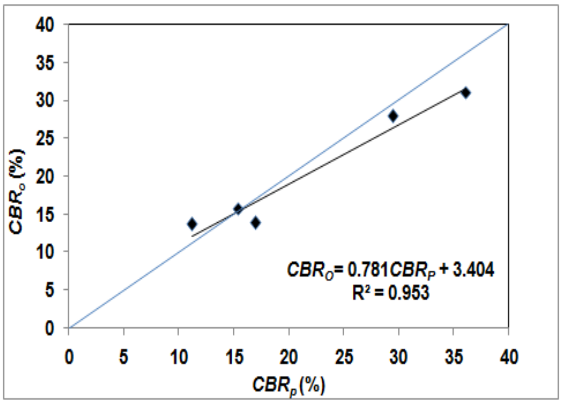

Figure 5 *Observed CBR Vs Predicted CBR based on UCS Values*

Eq.1 was validated for samples designated as B_1M_1, B_2M_2, B_3M_3, B_4M_1, and B_5M_2 as mentioned above. It can be observed from the scatter plot given in Figure 5 that the predicted CBR values (CBR_p), and the observed CBR values (CBR_o) agree with each other. The regression line of the scatter plot satisfied an R^2 value of 0.84 and conformed satisfactorily to the *theoretical line of equality*, at a negligible intercept of 3.044. The results show that the values of CBR of un-soaked blended soils can be effectively predicted using the UCS values.

4.2. Development of a regression between *CBR*, cohesion (*C*) and the angle of internal friction (ϕ)

A regression between the values of the *CBR* for un-soaked soil samples (CBR_u) and the cohesion (*C*) was developed as shown in Eq. 2, and a scatter plot for the same was obtained as in Figure 6. The R^2 value for the regression was 0.70 and the adjusted R^2 was 0.69. The standard error of estimation (SE) was found to be 5.49, while the values of the F-test, and the t-test were 19.15 and - 4.37 respectively for a significance level of 0.01.

$$CBR_u = -\,0.244\, C + 36.10 \qquad Eq.\ (2)$$

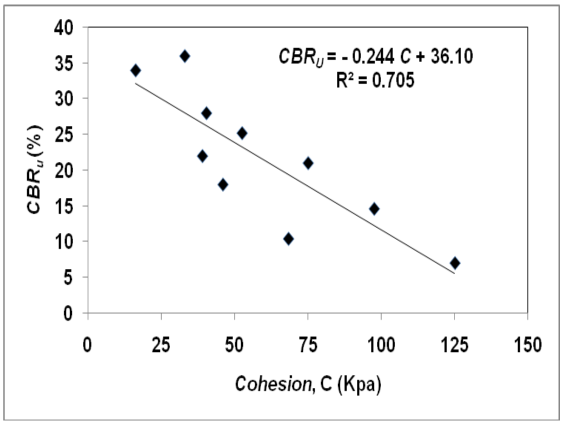

Figure 6 *Correlation between the CBR and the cohesion*

Eq.2 was validated for samples designated as B_1M_1, B_2M_2, B_3M_3, B_4M_1, and B_5M_2 as mentioned above. It can be observed from the scatter plot given in Figure 7 that the predicted CBR values (CBR_p), and the observed CBR values (CBR_o) agree with each other. The regression line of the scatter plot satisfied an R^2 value of 0.97 conforming satisfactorily to the *theoretical line of equality*, at a negligible intercept of 7.78. The results show that the values of CBR of un-soaked blended soils can be effectively predicted using the values of cohesion obtained based on tri-axial tests.

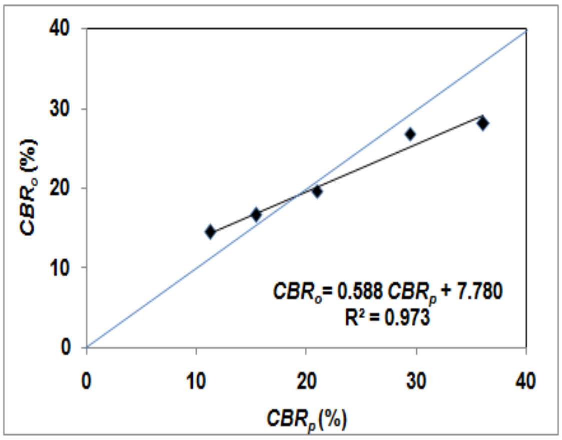

Figure 8 *Correlation between CBR Vs angle of internal friction*

In a similar manner, a regression between the values of the *CBR* for un-soaked soil samples (CBR_u) and the angle of internal friction (ϕ) was developed as shown in Eq. 3, and a scatter plot for the same was obtained as in Figure 8. The R^2 value for the regression was

0.83 and the adjusted R^2 was 0.81. The standard error of estimation (SE) was found to be 4.14, while the values of the F-test, and the t-test were 39.57 and 6.29 respectively for a significance level of 0.01.

$$CBR_u = 0.871\ \phi^o + 0.383 \qquad \textit{Eq. (3)}$$

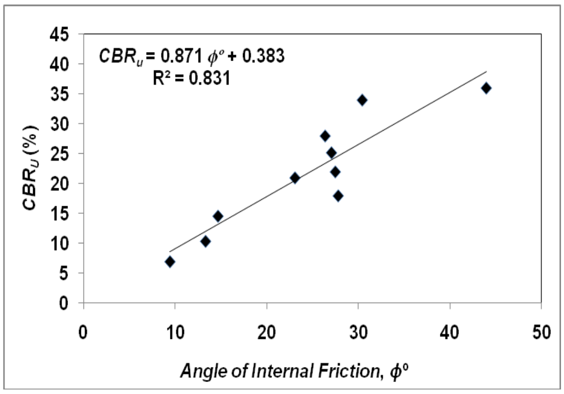

Figure 8 *Correlation between the CBR and the cohesion*

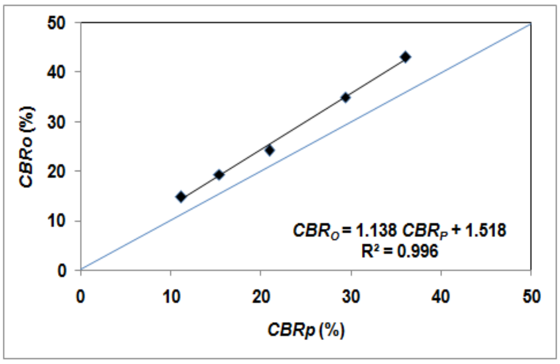

Figure 9 *Plot of Observed CBR Vs CBR Predicted based on the angle of internal friction*

Eq.3 was validated for samples designated as B_1M_1, B_2M_2, B_3M_3, B_4M_1, and B_5M_2 as mentioned above. It can be observed from the scatter plot given in Figure 9 that the predicted CBR values (CBR_p), and the observed CBR values (CBR_o) agree with each other. The regression line of the scatter plot satisfied an R^2 value of 0.99 conforming satisfactorily to the *theoretical line of equality*, at a negligible intercept of 1.52. The results show that the values of CBR of un-soaked blended soils can be effectively predicted using the values of the *angle of internal friction* determined based on tri-axial tests.

5. Discussions on Results and Conclusions

5.1. Discussions and conclusions on general properties of *lateritic* soil blends tested

Based on the results of the tests for index properties, dry-density, and water-content, and also based on the results of the IS heavy compaction (modified Proctor density) test, and the tests for grain-size distribution summarized as in Table 1, and Table 2, the following important observations may be made.

The specific gravity for various blends varied between 2.54 to 2.30 for soil blends with lateritic contents varying from 100%L+0%S to 0%L+100%S. Nanda and Krishnamachari (1958) observe that the specific gravity can vary between 2.2 to 4.6. The lower values of specific gravity observed for the region of Dakshina Kannada indicate that the presence of iron oxides is quite lesser than that observed in other regions.

The tests for Atterberg's limits indicate that the liquid limit varies between 47.2 and 52.2. This reveals that as the fines content increase in the soil blends, the liquid limit, plastic limit and the shrinkage values also increase.

Based on the results of the tests for grain-size distribution, the *lateritic* soil samples were classified as *sandy soils*, since more than 50 percent of the coarse fractions passed through 4.75 mm sieve. Also, since the proportion of soil fractions passing through 75 micron size sieves was greater than 12 percent, it was not possible to classify the soils as well-graded, or poorly graded. Additionally, since the soil fractions passing through 75 micron size sieves were lesser than 50 percent, the *lateritic* soil was classified as *coarse-grained*. *Lithomargic* soils were classified in this study as fine grained, since more than 50 percent of soil fractions passed through 75 microns size sieves.

Based on the tests for OMC and MDD, it can be observed that the MDD varies between 18.7kN/m^3 and 12.8kN/m^3 while the OMC varies between 21.4% and 17.7% for soil blends with lateritic contents varying from 100%L+0%S to 0%L+100%S. Thus it is seen that as the percentage of fines increases, the MDD decreases, while the OMC increases.

5.2. Discussions and conclusions on tests for CBR

The tests for CBR were conducted for un-soaked and soaked soil specimens for all the soil blends at three moisture contents mentioned above. The results for tests on un-soaked and soaked samples were compiled in Table 3. Figure 10 provides details on CBR penetrations at various loads for un-soaked soil samples of various blends. Figure 11 provides similar details for soaked soil samples.

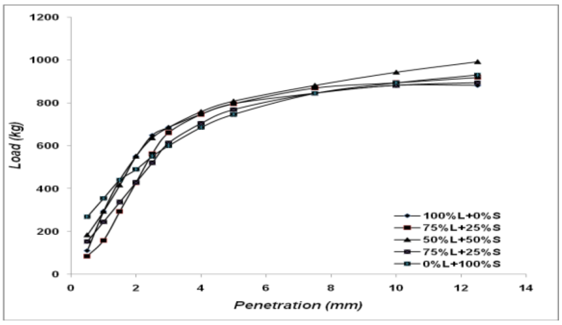

Figure 10 *CBR Charts for Various Blends at Un-Soaked Conditions*

In the case of *lateritic* soil samples of soil blend B1, the CBR value at OMC was found to be 36.0% and 11% for un-soaked and soaked soil specimens respectively, while in the case of soil blend B3, the CBR values at OMC ranged between 15.4% and 5.7% for un-soaked and soaked soil specimens.

Thus, it can be seen that in the case of *lateritic* soils, the soaked soil strength measured using the CBR approach is generally lower than that for un-soaked soils by about 30 to 40%.

Also, the strength at OMC was found to be higher when compared to strengths at other water contents. From tests performed at OMC+3% and OMC-3%, it is seen that the soil strengths at OMC-3% is slightly higher than that at OMC+3%. This is because of the reason that excess water acts as a lubricating agent reducing the friction between the soil particles.

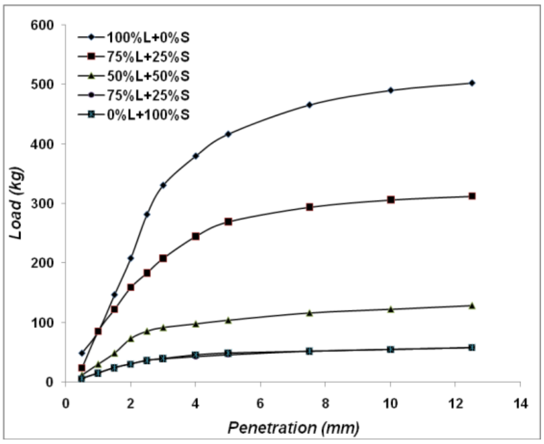

Figure 11 *CBR Charts for Various Blends at Soaked Conditions*

In the case of *lithomargic* soil samples of soil blend B5, the CBR value at OMC was found to be 11.2% and 2% for un-soaked and soaked soil specimens respectively, while in the case of soil blend B3, the CBR values at OMC ranged between 15.4% and 5.7% for un-soaked and soaked soil specimens.

Thus, it can be seen that in the case of *lithomargic* soils, the soaked soil strength measured using the CBR approach is drastically lower than that for un-soaked soils by about 40 to 80%.

5.3. Discussions and conclusions on tests for UCS

From the results for tests for unconfined compressive strength (UCS) compiled in Table 4 and Figure 12, it can be observed that for *lateratic* soils of soil blend B1, the unconfined compression strength is 0.449MPa and for *lithomargic* soils of soil blend B5, it is 0.179 MPa.

The results indicate that *lateritic* soil samples compacted at OMC, are able to withstand higher stresses of up to 2.5 times that resisted by *lithomargic* soils. This is clearly indicated in the stress-strain graphs.

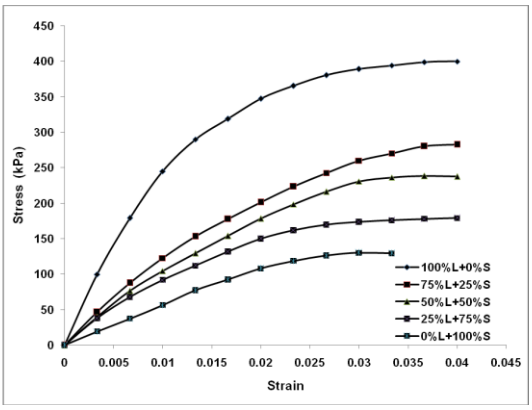

Figure 12 *Stress-Strain Graphs for UCS at Optimum Moisture Content*

It must also be observed in Table 1 that *lateritic* soils comprise more than 50% of sand when compared to *lithomargic* soils which are silty in nature. This has resulted in the mobilization of higher internal friction, and higher stress taken by *lateritic* soil samples.

In the tests for tri-axial strength, it is observed from Table 5 that the values of cohesion (*C*) at OMC range between 36.0 kPa to 88.7 kPa for soil blends B1 to B5. The values for the angle of internal friction range between 44.0 and 11.5 respectively for soil blends B1 to B5.

This reveals that the coefficient of internal friction for *lateritic* soils, is 3.85 times higher than that of *lithomargic* soils. Also, the values of cohesion for *lateritic* soils, is about 60% lesser than that of *lithomargic* soils.

5.4. Discussions and conclusions on correlations

The relationships developed between the values for CBR, and the UCS and the results of the tri-axial tests were highly correlated with R^2 values greater than 0.9 at a significance level of 0.01. The relationships developed were validated and the predicted values were found to tally reasonably well with the observations made. The relationships developed as part of this study can be used effectively in predicting the values of the CBR for un-soaked soil specimens based on the values of the UCS and the tri-axial tests. This will be of special advantage to engineers involved in sub-grade design for pavement.

References

[1] Barksdale, R (1972) “Laboratory evaluation of rutting in base course materials”, Proceedings of the Third International Conference on Structural Design of Asphalt Pavements, London, pp. 161-174.

[2] Barksdale, R. D. (1991) “The aggregate handbook” Washington, DC: National Stone Association.

[3] Bayoglu, Esra. “Shear Strength and Compressibility Behavior of Sand- Clay

Mixtures" M.S. Thesis, Middle East Technical University, Turkey., 1995.

[4] Dodds, A., Logan, T., McLachlan, M., and Patrick, J. (1999). Dynamic load properties of New Zealand base courseTransfund New Zealand Research Report 151.

[5] George, Varghese, Ch., Nageshwar Rao, Shivashankar, R., Investigations on Unsoaked Blended Laterite using PFWD, PBT, DCP, and CBR Tests, *Journal of Indian Roads Congress*, Vol. 70, No.3, pp. 223-233., 2009.

[6] Georgiannou, V. N,. "Behavior of Clayey Sands under Monotonic and Cyclic Loading", Ph.D. thesis, Department of Civil Engineering, Imperial College of Science, Technology and Medicine, London, England .1988

[7] Gidigasu, M.D., Developments in Geo-Technical Engineering -9, Laterite soil engineering, *Elsevier Scientific Publishing Company*, Newyork, pp.553, 1976.

[8] IS: 2720 Part V (1985) Indian Standard Methods of Test for soils, Part 5 Determination of Liquid and Plastic Limit, Compendium of Indian Standards on Soil Engineering, part 1 Laboratory Testing of Soils for Civil Engineering Purposes, *Bureau of Indian Standards*, New Delhi, pp. 109-114.

[9] IS: 1498 (1970) Indian Standard classification and identification of soils for general engineering purposes, Compendium of Indian Standards on Soil Engineering, part 1 Laboratory Testing of Soils for Civil Engineering Purposes, *Bureau of Indian Standards*, New Delhi, pp. 23-40.

[10] IS: 2720 Part III Sec.1 (1964) Indian Standard Methods of test for soils Part 3 Determination of Specific Gravity, SP 36 (Part 1) : 1987, Compendium of Indian Standards on Soil Engineering, part 1 Laboratory Testing of soils for Civil Engineering Purposes, *Bureau of Indian Standards*, New Delhi, pp. 65-67.

[11] IS: 2720 part IV. (1985) Indian Standard Methods of test of soil for soils, grain size analysis, Compendium of Indian Standards on soil Engineering,part1 laboratory testing of soil for civil engineering purpose, Bureau of Indian standards, New Delhi, pp.73-91.

[12] IS: 2720 Part VI (1972) Indian Standard Methods of Test for soils, Part 6 Determination of Shrinkage Factor, Compendium of Indian Standards on Soil Engineering, part 1 Laboratory Testing of Soils for Civil Engineering Purposes, Bureau of Indian Standards, New Delhi, pp. 115-117.

[13] IS: 2720 Part VII (1980) Indian Standard Methods of Test for soils, Part 7 Determination of Water Content- Dry Density Relation Using Light Compaction, Compendium of Indian Standard on Soil Engineering, part 1 Laboratory Testing of Soils for Civil Engineering Purposes, Bureau of Indian Standards, New Delhi, pp. 162-164.

[14] IS: 2720 Part X (1973) Indian Standard Methods of Test for soils, Part 10 Determination of Unconfined Compressive Strength, Compendium of Indian Standards on Soil Engineering, part 1 Laboratory Testing of Soils for Civil Engineering Purposes, Bureau of Indian Standards, New Delhi, pp. 202-2.

[15] K.J. Osinubi et al. "Effect of Fines Content on the Engineering Properties of Reconstituted Lateritic Soils in Waste Containment Application" *Nigerian Journal of Technology*, Vol. 31, No. 3, November 2012.

[16] Thom, N. H., and Brown, S. F. (1988). The effect of grading and density on the mechanical properties of a crushed dolomitic limestone. In Proceedings of the 14th ARRB Conference, Part 7: pp 94–100.