Asymmetrical Intervention On Dilative Ibeno Residual Soils, Akwa Ibom State, Nigeria

Essien Udo (PhD)
University of Uyo,
Uyo, Akwa Ibom State
Nigeria.
essienau2006@yahoo.com

Ndifreke Udoh
University of Uyo,
Uyo, Akwa Ibom State
Nigeria.
ndiudoh@gmail.com

***Abstract* —Soils' behaviour and structure are affected by the mechanism of transport and deposition to their locations. The phenomenon of dilatancy can be observed in a simple shear test on a sample of dense sand[1]. In the initial stage of deformation, the volumetric strain decreases as the shear strain increases. But as the stress approaches its peak value, the volumetric strain starts to increase. Ibeno residual soil is dilative because of the effects of predominant aluminium sesquioxide and iron compound montmorillonite. These compounds boost the swelling pattern of residual soils if their natural moisture content is allowed to increase. Asymmetrical intervention defines a process characterized by spatial or irregular rearrangement of the soil structure to meet specific target and objective. Composite Stabilization is an asymmetrical process that alters the soils' properties to enhance their physical properties. It can increase the shear strength of a soil and or control the shrink-swell properties of a soil, thus improving the load bearing capacity of a soil to support pavement and foundation structures. This study adopts laboratory investigations to ascertain problems associated with Ibeno dilative residual soils at plain and modified conditions bearing in mind that residual soil parameters have significant effect on the overall performance or non-performance of sub-base and base course formations. Samples from various locations were stabilized mechanically and chemically in the laboratory with lime and quarry dust. At plain condition, it was found that strength development through compaction was related to maximum dry density (MDD), natural moisture content (NMC) and percentage of fines (F). The resultant California bearing ratio (CBR) value did not satisfy the requirement by the code for base course pavement application. Composite stabilization with lime and quarry dust revealed soaked – CBR value ranging from 76% -142% and 87% - 442% for measured and computed values respectively thus suitable for base course application. Unconfined compressive strength (UCS) tests with lime dosages from 2% - 8% and quarry dust content from 10% - 60% were performed to determine the effects of pozzolanic reaction, and flocculation over the development and progress of Ibeno dilatant residual soil composition at 7 and 28 days curing durations. Results revealed that a 2%/40% lime/quarry dust content produced a UCS of 116/135 kPa on a curing duration of 7/28 days respectively. Finally, multiple nonlinear regressed models with correlation coefficients were developed and validity of the correlation was established to confirm agreement with experimental observations. The models considered that CBR depends on lime content, quarry dust content, maximum dry density and optimum moisture content, while UCS depends on lime content, quarry dust content and curing durations. The models thus formed the basis of prediction of results for both CBR and UCS of Ibeno dilatant residual soils at various levels of stabilization.**

Keywords—Dilative soil, Stabilization, Lime, Quarry dust, Regressed models.

I. INTRODUCTION

Unlike most other solid materials, the tendency of a compacted granular material is to dilate (expand in volume) as it is sheared. This occurs because the grains in a compacted state are interlocking and therefore do not have the freedom to move around one another. When stressed, a lever motion occurs between neighbouring grains, which produces a bulk expansion of the material. On the other hand, when a granular material starts in a very loose state it may initially compact instead of dilating under shear. A sample of a material is said to be dilative if its volume increases with increasing shear and contractive if the volume decreases with increasing shear. Because of dilatancy, the angle of friction increases as the confinement increases until it reaches a peak value[2]. After the peak strength of the soil is mobilized the angle of friction abruptly decreases. As a result, geotechnical engineering of slopes, footings, tunnels, and piles in such soils have to consider the potential decrease in strength after the soil strength reaches this peak value. Dilatancy influences almost all aspects of the behaviour of granular material, ranging from shear strength to stress-strain behaviour[3]. However, there is no practical method for estimating the dilatancy angle based on in situ soil properties, although the variables that influence dilatant behaviour are well-known. The volume change behaviour and interparticle friction depend on the density of the particles, the intergranular contact forces, and to a somewhat lesser extent, other factors such as the rate of shearing and the direction of the shear stress[4]. The average normal intergranular contact force per unit area is called the effective stress. If water is not allowed to flow in or out of the soil, the stress path is called an undrained stress path. During undrained shear, if the particles are surrounded by a nearly incompressible fluid such as water, then the density of the particles cannot change without drainage, but the water pressure and effective stress will change. On the other hand, if the fluids are allowed to freely drain out of the pores, then the pore pressures will remain constant and the test path is called a drained stress path. The soil is free to dilate or contract during shear if the soil is drained. In reality, soil is partially drained, somewhere between the perfectly undrained and drained idealized conditions. The shear strength of soil depends on the effective stress, the drainage conditions, the density of the particles, the rate of strain, and the direction of the strain. For undrained, constant volume shearing, the Tresca theory may be used to predict the shear strength, but for drained conditions, the Mohr–Coulomb theory may be used. Two important theories of soil shear are the critical state theory and the steady state theory. There are key differences between the critical state condition and the steady state condition and the resulting theory corresponding to each of these conditions.

II. MATERIALS SELECTED

A. Ibeno Residual Soil

The soils chosen for this research were dug with shovels from four borrow-pits along Eket - Ibeno access road. The soil samples were disturbed and at depths varying from 3.0 meters to 5.0 meters of the profile. The samples were excavated bearing in mind the variability of residual soil in its natural composition. Hence the soil samples were excavated both vertically and laterally and thoroughly blended. The samples were conveyed in four, 50kg nylon bags, carefully tagged for identification purpose and transported to the Civil Engineering Materials Testing Laboratory at Uyo. Physical examination of the different samples revealed coloration ranging from light-brown at 3.0 meters to deep-brown at 5.0 meters. This is a clear indication of predominant iron compound-montmorillonite - a sodium-based smectite or aluminium sesquioxides. Replacing the sodium compounds which may produce such type of cation having less ion exchange capacity and also form a balanced electrical charge in soil structure can reduce the expansion. Replacement of monovalent sodium by calcium ions may lead to a marked reduction in diffused double layer thickness leading to decrease in liquid limit, plastic limit and swelling pressure[5].

B. Lime

Lime helps to arrest the shrinkage and swelling behaviour of soil. This is due to the creation of chemical bonds and aggregation. The use of lime to improve the engineering properties of soil had been in practice for long in many parts of the World. The lime used in this work was purchased from Mbiabong market in Uyo. The primary purpose was to evaluate the behaviour of Ibeno residual soil on application of various percentages of lime and compactive effort on the maximum dry densities and corresponding optimum moisture contents. Lime stabilized soil is an engineered product that must be properly evaluated, proportioned and constructed in order to obtain the good and long-term performance[6]. Generally, lime reduces the plasticity of a highly expansive soil, as well as improving the stress-strain behaviour.

C. Quarry Dust

The quarry dust used in this experiment came from the limestone quarry factory in Akamkpa, Cross River State. This is the by-product or sediments derived from the crushing of limestone. This soil modifying agent has a high percentage of fines, and as expected, the CBR value of quarry dust was the minimum value of all, in that it in fact increases the overall fines content of Ibeno residual soil. The material was purchased from a local supplier at Udo Udoma street depot in Uyo.

III. PREPARATION AND TESTING OF SAMPLES

A. Gradation Test

The samples were air-dried for three weeks. The next step was to sieve through 20mm diameter sieve and any particle retained was broken with rubber hammer or thrown away. With the aid of a riffle box the quantity of material needed or five hundred grams each of the soil samples were extracted and poured into sieve no.200 or 0.075mm diameter sieve and thoroughly washed to remove all clayey materials finer than the 0.075mm diameter. The particles retained were oven-dried, weighed and mechanically sieved in a shaker.

B. Liquid Limit Test

The air-dried samples were quantified through a sample divider – the riffle box – and sieved through 425μm test sieve. 50g of material passing through this sieve was used for the liquid limit test. The sample was put in a flat glass plate, moisturized and thoroughly mixed with a spatula to a thick homogeneous paste. The paste was preserved in air-tight polythene sack for 24 hours to allow water permeate the entire sample, devoid of moisture evaporation. It was then put back into the glass plate and properly mixed for 15 minutes. Finally, the paste was then put into the Casagrande liquid limit apparatus, grooved to V-shape as per specification, to determine the number of blows that will be required to bring the two parts into contact. The range of blows varied from 10-15, 15-20, 21-30, and 31-40 and for various moisture contents.

C. Plastic Limit Test

Sixty grams of samples passing the 425μm test sieve was moisturized and thoroughly mixed in the glass plate until it becomes homogeneous and plastic, enough to be shaped into a ball. The ball was then rolled between the palms of the hand, until the heat of hands dried the sample sufficiently for slight cracks to appear on its surface. It was then rolled continuously forward and backward in between the finger and glass plate until the pressure was sufficient to reduce the diameter of the thread to about 3mm. The procedure was repeated until the thread sheared (crumbled) both longitudinally and transversely. This test determines the lowest moisture content at which the soil is plastic.

D. Plain Mechanical Compaction

The Modified Proctor Compaction tests were conducted for each of the samples. The air-dried material was divided into five equal parts through a riffle box and weighed to 6000g each. Each sample was poured into the mixing plate. A particular percentage of distilled water was poured into each plate and thoroughly mixed with a trowel. An interval of about sixty minutes was allowed for the moisture to fully permeate the soil sample. The sample was thereafter divided into five equal parts, weighed and each was poured into the compaction mould, in five layers and compacted at 61 blows each using a 4.5kg rammer falling over a height of 450mm above the top of the mould. The blows were evenly distributed over the surface of each layer. The collar of the mould was then removed and the compacted sample weighed while the corresponding moisture content was noted. The procedure was repeated with different moisture contents until the weight of compacted sample was noted to be decreasing. With the optimum moisture content obtained from the Modified Proctor test, samples were prepared in the CBR mould and values for the plain mechanical compaction were read for both top and bottom at various depths of penetration. This test was conducted to determine the mass of dry soil per cubic meter and the soil was compacted in a specified manner over a range of moisture contents.

E. Lime – Quarry Dust Composite Stabilization Tests

The percentage of lime ranged from 2%, 4%, 6% to 8%. The percentage of quarry dust ranged from 10%, 20%, 30%, 40%, 50% to 60%. For each lime content the percentage or proportion of quarry dust was varied from 10%-60%. It is an established fact that the measurement of the strength of soil-lime mixture in laboratory and the determination of the parameters which affect it is very important for the estimation of the strength of mixture in-situ[7]. The mixture was thoroughly blended and moisturized and modified proctor compaction test was conducted to establish the OMC and MDD. With the OMC and MDD results, three specimens each were prepared for the CBR test. This procedure meets the provision of clause 6228 design criteria. FMW&H (1997)[8].

F. California Bearing Ratio (CBR) Test

The CBR test [as it is commonly known] involves the determination of the load-deformation curve of the soil in the laboratory using the standard CBR testing equipment. It was originally developed by the California Division of Highways prior to World War II and was used in the design of some highway pavements. This test has now been modified and is standardized under the AASHTO designation of T193. With the OMC and MDD results, three specimens each were prepared for the CBR test. One specimen was tested immediately while the remaining two were wax-cured for 6days and thereafter soaked for 24 hours, and allowed to drain for 15minutes. After testing in CBR machine, the average of the two readings was adopted. This procedure meets the provision of clause 6228 design criteria. FMW&H (1997). CBR gives the relative strength of a soil with respect to crushed rock, which is considered an excellent coarse base material. The main criticism of the CBR test is that it does not correctly simulate the shearing forces imposed on sub-base and sub-grade materials as they support highway pavement.

G. Unconfined Compression Test

Unconfined Compression Test is a triaxial test in which the axial load is applied to a specimen under zero all round pressure. This test is applicable only for testing intact fully saturated soils i.e. only on saturated samples which can stand without any lateral support. By implication the test is applicable to cohesive soils only. The test is an undrained test and is based on the assumption that there is no moisture loss during the test. The unconfined compression test is one of the tests used for the determination of the undrained shear strength of cohesive soils. In this test no radial stress is applied to the sample and the plunger load is increased rapidly until the soil sample fails. The loading is applied quickly so that pore water cannot drain from the soil; the sample is sheared at constant volume.

IV. PRESENTATION OF TEST RESULTS

Table 1: Ibeno Residual Soil Compaction at Plain Condition

Sample No	MDD Kg/m^3	NMC %	Unsoaked CBR, %	Fines %
1	1840	9.8	35	26
2	1850	11.0	29	27
3	1840	10.5	35	30
4	1730	10.8	33	30

Table 2: Lime - Quarry Dust Stabilization Results -Sample Location 1

Lime content, (%)	Quarry dust content, (%)	MDD (kg/m^3)	OMC (%)	Soaked CBR, (%)
2	10	2000	8.0	66
	20	2020	8.5	76
	30	2050	8.8	114
	40	1920	8.7	134
	50	1970	6.8	92
	60	1830	8.1	74
4	10	2010	6.7	64
	20	1920	8.9	78
	30	1940	6.6	96
	40	1960	7.2	118
	50	1990	7.7	99
	60	1790	9.5	60
6	10	1930	11.5	62
	20	2020	11.5	93
	30	2030	8.3	84
	40	2070	9.2	83
	50	2030	10.1	83
	60	2030	8.6	56
8	10	1990	6.7	73
	20	2020	10.3	108
	30	2060	7.8	111
	40	2050	8.4	110
	50	2030	11.5	98
	60	1990	8.2	75

Table 3: Lime - Quarry Dust Stabilization Results - Sample Location 2

Lime content (%)	Quarry dust content (%)	MDD (kg/m^3)	OMC (%)	Soaked CBR (%)
2	10	1990	6.2	65
	20	2000	8.5	69
	30	1910	6.1	85
	40	1940	6.7	87
	50	1960	8.5	99
	60	1990	8.5	60
4	10	1950	9.5	71
	20	2030	10.2	78
	30	2070	12.4	86
	40	2050	9.8	104
	50	2080	10.6	108
	60	2100	9.9	78
6	10	2050	11.8	76
	20	2040	8.3	88
	30	2080	7.9	97
	40	2060	12.5	113
	50	2090	8.5	125
	60	2100	8.4	87
8	10	2080	13.2	99
	20	2080	8.5	105
	30	2080	8.9	114
	40	2110	8.8	126
	50	2050	12.7	133
	60	2120	8.6	78

Table 4: Lime - Quarry Dust Stabilization Results - Sample Location 3

Lime content (%)	Quarry dust content (%)	MDD (kg/m3)	OMC (%)	Soaked CBR (%)
2	10	2090	8.4	66
	20	2040	9.4	72
	30	2050	10.5	76
	40	2060	9.9	87
	50	2070	10.3	128
	60	2080	8.1	68
4	10	2080	9.3	59
	20	2060	9.1	72
	30	2060	10.5	89
	40	2080	9.9	133
	50	2100	10.2	139
	60	2120	10.9	77
6	10	2040	9.8	63
	20	2060	10.8	72
	30	2080	8.2	99
	40	2090	10.8	129
	50	2100	7.9	139
	60	2100	8.1	68
8	10	2070	13.6	66
	20	2070	8.6	78
	30	2100	7.2	121
	40	2090	8.6	129
	50	2040	13.6	138
	60	2120	9.2	76

Table 5: Lime - Quarry Dust Stabilization Results - Sample Location 4

Lime content (%)	Quarry dust content (%)	MDD (kg/m3)	OMC (%)	Soaked CBR (%)
2	10	1820	8.8	58
	20	2040	14.2	66
	30	2030	12.4	79
	40	2040	11.4	118
	50	2050	12.5	122
	60	2060	12.4	71
4	10	2060	13.8	63
	20	2050	10.5	76
	30	2060	12.4	89
	40	2070	9.9	108
	50	2100	10.5	131
	60	2080	10.5	72
6	10	2050	10.3	67
	20	2030	8.6	77
	30	2060	7.7	97
	40	2100	11	119
	50	2080	8.2	141
	60	2100	8.7	78
8	10	2050	14.7	65
	20	2030	6.7	89
	30	2060	6.5	118
	40	2090	6.7	128
	50	2080	12.6	140
	60	2020	6.4	75

Table 6: Lime - Quarry Dust Stabilization UCS Results at 7 days Curing Duration

Lime content (%)	Quarry dust content (%)	Duration (days)	Compressive Strength (KPa)
Sample Location 2			
2	10	7	74
	20	7	79
	30	7	109
	40	7	116
	50	7	124
	60	7	129
4	10	7	86
	20	7	111
	30	7	117
	40	7	149
	50	7	163
	60	7	167
6	10	7	86
	20	7	124
	30	7	127
	40	7	135
	50	7	147
	60	7	158
8	10	7	89
	20	7	103
	30	7	126
	40	7	144
	50	7	156
	60	7	164

Table 7: Lime - Quarry Dust Stabilization UCS Results at 7 days Curing Duration

Lime content (%)	Quarry dust content (%)	Duration (days)	Compressive Strength (KPa)
Sample Location 4			
2	10	7	66
	20	7	73
	30	7	87
	40	7	89
	50	7	96
	60	7	104
4	10	7	101
	20	7	116
	30	7	124
	40	7	135
	50	7	141
	60	7	151
6	10	7	107
	20	7	133
	30	7	149
	40	7	157
	50	7	167
	60	7	176
8	10	7	113
	20	7	124
	30	7	139
	40	7	150
	50	7	177
	60	7	184

Table 8: Lime - Quarry Dust Stabilization UCS Results at 28 days Curing Duration

Lime content (%)	Quarry dust content (%)	Duration (days)	Compressive Strength (KPa)
Sample Location 2			
2	10	28	85
	20	28	93
	30	28	112
	40	28	121
	50	28	128
	60	28	138
4	10	28	85
	20	28	90
	30	28	98
	40	28	118
	50	28	139
	60	28	152
6	10	28	77
	20	28	92
	30	28	97
	40	28	111
	50	28	141
	60	28	175
8	10	28	79
	20	28	89
	30	28	131
	40	28	149
	50	28	178
	60	28	186

Table 9: Lime - Quarry Dust Stabilization UCS Results at 28 days Curing Duration

Lime content (%)	Quarry dust content (%)	Duration (days)	Compressive Strength (KPa)
Sample Location 4			
2	10	28	85
	20	28	88
	30	28	116
	40	28	135
	50	28	148
	60	28	158
4	10	28	88
	20	28	93
	30	28	117
	40	28	137
	50	28	149
	60	28	168
6	10	28	87
	20	28	115
	30	28	131
	40	28	136
	50	28	188
	60	28	192
8	10	28	98
	20	28	139
	30	28	148
	40	28	171

	50	28	198
	60	28	212

V. DISCUSSION OF TEST RESULTS

Table 1 presents the results of Ibeno residual soil compaction at plain condition. Within the four distinct locations the CBR values ranged from 29% to 35%. Tables 2 to 5 present the results of composite stabilization of Ibeno residual soil with lime and quarry dust. The CBR values obtained ranged from 66% minimum to 141% maximum at 2% / 10% and 6% / 50% combination ratios of lime and quarry dust. Tables 6 and 7 present the results of the UCS of Ibeno residual soils subjected to a 7-day curing duration. The UCS values varied from 74kPa to 167kPa at similar combination ratios of lime and quarry dust compared to CBR. Tables 8 to 9 present the UCS results of Ibeno residual soils subjected to a 28-day curing duration. The data obtained varied from 85kPa to 188kPa, equally at same ratios of lime and quarry dust combinations.

VI. MULTIPLE NONLINEAR REGRESSED MODELS

Based on analysis and utilizing multiple non-linear regressed programs, the following models were developed for evaluating the CBR and UCS of Ibeno residual soils at various levels of composite stabilization with quarry dust and lime. The models are often used for the purposes of prediction and optimization to determine for what values of the independent variables the dependent variable is a maximum or minimum.

$CBR_{[1]} = 16.264 + 3.528L - 3.058Q - 1.499D + 1.251M + 1.679L^2 + 0.237Q^2 + 0.853D^2 + 0.333M^2 + 0.848LQ - 0.187LD - 0.116LM - 0.201QD - 0.101QM - 0.984DM$……………………………………………...1.1

Where L = Lime content [%], Q = Quarry dust content [%], D = Maximum dry density [k_{g}/m^3], M = Moisture content [%]

$CBR_{[2]} = 15.198 + 7.524L - 1.611Q + 1.878D + 3.149M + 0.736L^2 + 0.118Q^2 - 0.246D^2 - 0.105M^2 + 0.509LQ - 0.335LD - 0.139LM - 0.102QD - 0.187QM - 0.921DM$………………………………………………..1.2

Where L = Lime content [%], Q = Quarry dust content [%], D = Maximum dry density [k_{g}/m^3], M = Moisture content [%]

$CBR_{[3]} = 14.334 - 1.579L + 2.921Q - 6.922D + 1.865M - 0.388L^2 + 0.156Q^2 + 0.325D^2 - 0.868M^2 - 0.935LQ - 0.729LD + 0.131LM - 0.889QD + 0.154QM - 0.854DM$………………………………………………1.3

Where L = Lime content [%], Q = Quarry dust content [%], D = Maximum dry density [k_{g}/m^3], M = Moisture content [%]

$CBR_{[4]} = 17.054 + 2.339L +0.494Q - 1.585D + 1.707M + 1.067L^2 \; 0.092Q^2 + 0.922D^2 + 0.141M^2 + 0.103LQ - 0.147LD + 0.297LM - 0.309QD + 0.499LM - 0.111DM$………………………………………………1.4

Where L = Lime content [%], Q = Quarry dust content [%], D = Maximum dry density [k_{g}/m^3], M = Moisture content [%]

$UCS_{[7]2} = 22.252 + 0.413L - 0.502Q - 0.195C - 0.812L^2 + 0.087Q^2 + 0.278C^2 - 0.118LQ - 0.591LC - 0.071QC$……….......................................1.5

Where L = Lime content [%], Q = Quarry dust content [%], C = Curing duration [days]

$UCS_{[7]4} = 21.829 + 0.405L - 0.493Q - 0.191C - 0.797L^2 + 0.086Q^2 + 0.273C^2 - 0.115LQ. - 0.579LC - 0.071QC$……………………………………………..1.6

Where L = Lime content [%], Q = Quarry dust content [%], C = Curing duration [days]

$UCS_{[28]2} = 51.261 + 0.433L + 0.178Q + 0.227C - 0.774L^2 + 0.041Q^2 + 0.081C^2 + 0.145LQ - 0.154LC - 0.063QC$...1.7

Where L = Lime content [%], Q = Quarry dust content [%], C = Curing duration [days]

$UCS_{[28]4} = 56.011 + 0.473L + 0.195Q + 0.248C - 0.845L^2 + 0.054Q^2 + 0.089C^2 + 0.158LQ - 0.169LC - 0.069QC$……………………………………………..1.8

Where L = Lime content [%], Q = Quarry dust content [%], C = Curing duration [days]

Sample Location 1					
Lime Content (%)	Quarry Dust Content (%)	MDD (kg/m^3)	OMC (%)	Measured CBR (%)	Computed CBR (%)
2	10	2	8	66	41.402
2	20	2.02	8.5	76	87.828
2	30	2.05	8.8	114	180.199
2	40	1.92	8.7	134	321.353
2	50	1.97	6.8	122	509.064
2	60	1.83	8.1	74	745.305
4	10	2.01	6.7	64	79.372
4	20	1.92	8.9	78	148.420
4	30	1.94	6.6	96	254.816
4	40	1.96	7.2	118	412.869
4	50	1.99	7.7	99	617.188
4	60	1.79	9.5	60	870.830
6	10	1.93	11.5	62	153.082
6	20	2.02	11.5	93	227.675
6	30	2.03	8.3	84	343.848
6	40	2.07	9.2	83	517.247
6	50	2.03	10.1	83	738.064
6	60	2.03	8.6	56	1006.696
8	10	1.99	6.7	73	203.548
8	20	2.02	10.3	108	307.950
8	30	2.06	7.8	111	445.386
8	40	2.05	8.4	111	636.476
8	50	2.03	11.5	98	874.485
8	60	1.99	8.2	75	1161.413

Table 10: Lime - Quarry Dust Stabilization CBR Results - Sample Location 1

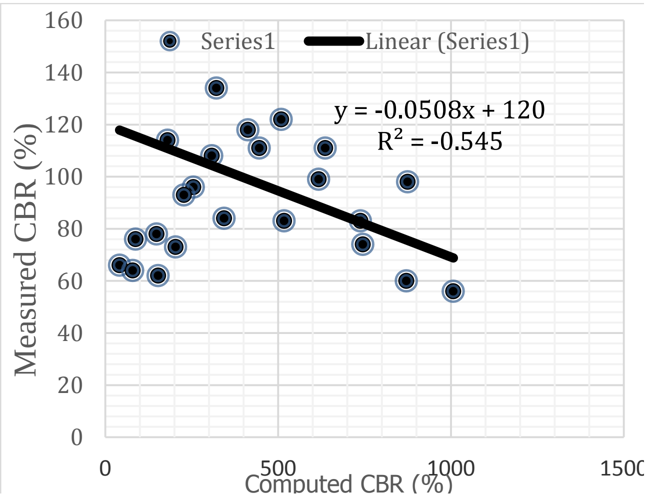

Fig. 1: Cross Plot of Measured Vs Computed CBR Values using equation 1.1

Table 11: Lime - Quarry Dust Stabilization CBR Results - Sample Location 2

Sample Location 2					
Lime Content (%)	Quarry Dust Content (%)	MDD (kg/m^3)	OMC (%)	Measured CBR (%)	Computed CBR (%)
2	10	1.99	6.2	65	29.267
2	20	2	8.5	69	35.252
2	30	1.91	6.1	85	85.820
2	40	1.94	6.7	87	144.208
2	50	1.96	8.5	99	211.965
2	60	1.99	8.5	60	317.529
4	10	1.95	9.5	71	51.575
4	20	2.03	10.2	78	67.013
4	30	2.07	12.4	86	92.550
4	40	2.05	9.8	104	178.095
4	50	2.08	10.6	108	259.039
4	60	2.1	9.9	78	379.717
6	10	2.05	11.8	76	78.024
6	20	2.04	8.3	88	122.788
6	30	2.08	7.9	97	180.907
6	40	2.06	12.5	125	218.953
6	50	2.09	8.5	115	358.233
6	60	2.1	8.4	87	485.547
8	10	2.08	13.2	99	112.672
8	20	2.08	8.5	105	173.593
8	30	2.08	8.9	114	236.260
8	40	2.11	8.8	133	325.207
8	50	2.05	12.7	126	393.622
8	60	2.12	8.6	78	575.642

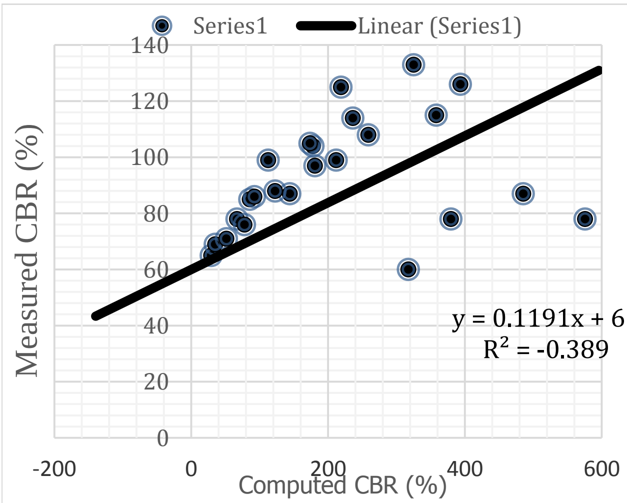

Fig. 2: Cross Plot of Measured Vs Computed CBR Values using equation 1.2

Table 12: Lime - Quarry Dust Stabilization CBR Results - Sample Location 3

Sample Location 3					
Lime Content (%)	Quarry Dust Content (%)	MDD (kg/m^3)	OMC (%)	Measured CBR (%)	Computed CBR (%)
2	10	2.09	8.4	66	-44.377
2	20	2.04	9.4	72	-3.097
2	30	2.05	10.5	76	67.831
2	40	2.06	9.9	126	191.679
2	50	2.07	10.3	128	335.132
2	60	2.08	8.1	68	528.048
4	10	2.08	9.3	59	-83.382
4	20	2.06	9.1	72	-45.804
4	30	2.06	10.5	89	3.238
4	40	2.08	9.9	141	107.605
4	50	2.1	10.2	138	232.597
4	60	2.12	10.9	77	385.688
6	10	2.04	9.8	63	-119.589
6	20	2.06	10.8	72	-117.354
6	30	2.08	8.2	99	-40.249
6	40	2.09	10.8	139	11.061
6	50	2.1	7.9	132	144.242
6	60	2.1	8.1	68	281.691
8	10	2.07	13.6	66	-220.557
8	20	2.07	8.6	78	-141.583
8	30	2.1	7.2	121	-104.529
8	40	2.09	8.6	138	-56.966
8	50	2.04	13.6	133	-16.325
8	60	2.12	9.2	76	147.955

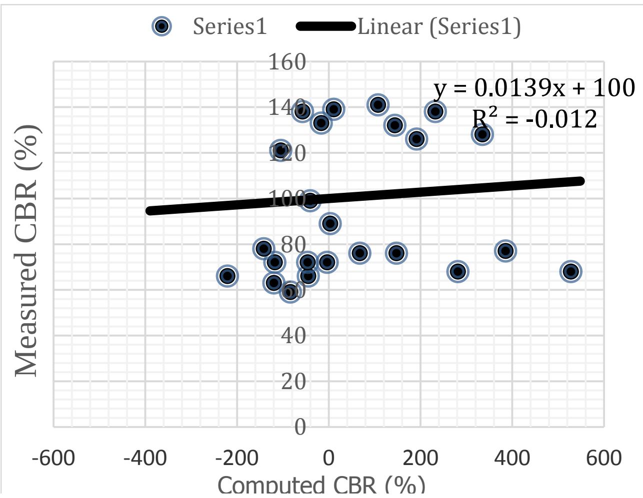

Fig. 3: Cross Plot of Measured Vs Computed CBR Values using equation 1.3

Table 13: Lime - Quarry Dust Stabilization CBR Results - Sample Location 4

Sample Location 4					
Lime Content (%)	Quarry Dust Content (%)	MDD (kg/m^3)	OMC (%)	Measured CBR (%)	Computed CBR (%)
2	10	1.82	8.8	58	109.513
2	20	2.04	14.2	66	263.803
2	30	2.03	12.4	79	344.013
2	40	2.04	11.4	118	445.508
2	50	2.05	12.5	122	619.175
2	60	2.06	12.4	71	779.683
4	10	2.06	13.8	63	187.322
4	20	2.05	10.5	76	233.497
4	30	2.06	12.4	89	374.173
4	40	2.07	9.9	131	438.599
4	50	2.1	10.5	118	591.310
4	60	2.08	10.5	72	747.836
6	10	2.05	10.3	67	182.251
6	20	2.03	8.6	77	239.110
6	30	2.06	7.7	97	314.128
6	40	2.1	11	125	506.545
6	50	2.08	8.2	115	562.609
6	60	2.1	8.7	78	726.848
8	10	2.05	14.7	65	278.811
8	20	2.03	6.7	89	251.905
8	30	2.06	6.5	118	333.838
8	40	2.09	6.7	142	442.262
8	50	2.08	12.6	128	751.356
8	60	2.02	6.4	75	697.156

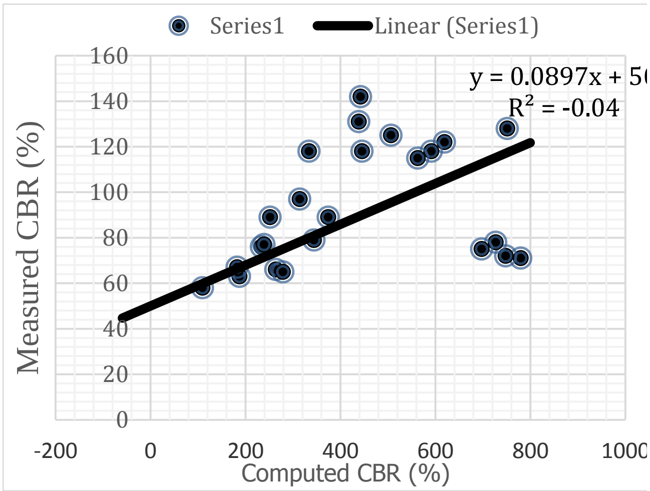

Fig. 4: Cross Plot of Measured Vs Computed CBR Values using equation 1.4

Table 14: Lime - Quarry Dust Stabilization UCS Results at 7 days Curing Duration

SAMPLE LOCATION 2				
Lime Content (%)	Quarry Dust Content (%)	Duration (days)	Measured UCS (KPa)	Computed UCS (KPa)
2	10	7	74	20.163
2	20	7	79	33.913
2	30	7	109	65.063
2	40	7	116	113.613
2	50	7	124	179.563
2	60	7	129	262.913
4	10	7	86	0.611
4	20	7	111	12.001
4	30	7	117	40.791
4	40	7	149	86.981
4	50	7	163	150.571

4	60	7	167	231.561
6	10	7	86	-25.437
6	20	7	124	-16.407
6	30	7	127	10.023
6	40	7	135	53.853
6	50	7	147	115.083
6	60	7	158	193.713
8	10	7	89	-57.981
8	20	7	103	-51.311
8	30	7	126	-27.241
8	40	7	144	14.229
8	50	7	156	73.099
8	60	7	164	149.369

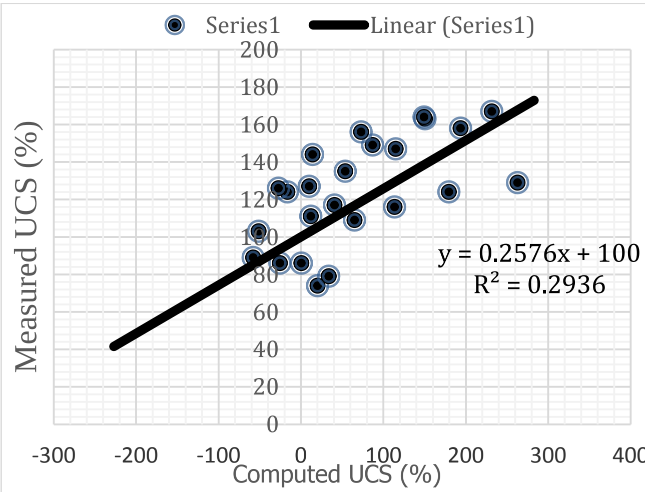

Fig. 5: Cross Plot of Measured Vs Computed UCS Values using equation 1.5

Table 15: Lime - Quarry Dust Stabilization UCS Results at 7 days Curing Duration

Sample Location 4				
Lime Content (%)	Quarry Dust Content (%)	Duration (days)	Measured UCS (KPa)	Computed UCS (KPa)
2	10	7	66	19.785
2	20	7	73	33.385
2	30	7	87	64.185
2	40	7	89	112.185
2	50	7	96	177.385
2	60	7	104	259.785
4	10	7	101	0.625
4	20	7	116	11.925
4	30	7	124	40.425
4	40	7	135	86.125
4	50	7	141	149.025
4	60	7	151	229.125
6	10	7	107	-24.911
6	20	7	133	-15.911
6	30	7	149	10.289
6	40	7	157	53.689
6	50	7	167	114.289
6	60	7	176	192.089

8	10	7	113	-56.823
8	20	7	124	-50.123
8	30	7	139	-26.223
8	40	7	150	14.877
8	50	7	177	73.177
8	60	7	184	148.677

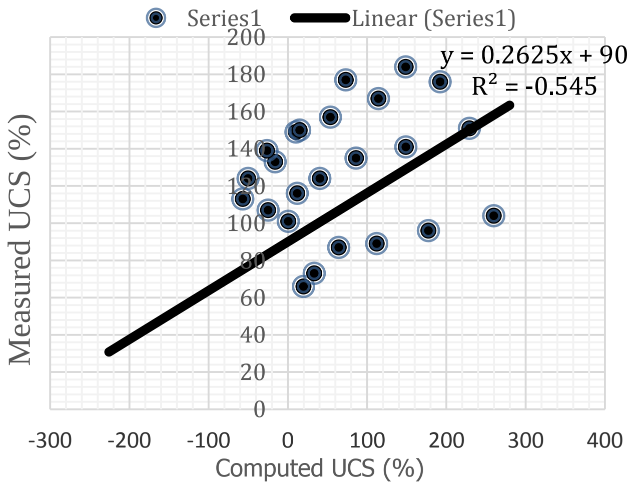

Fig. 6: Cross Plot of Measured Vs Computed UCS Values using equation 1.6

Table 16: Lime - Quarry Dust Stabilization UCS Results at 28 days Curing Duration

Sample Location 2				
Lime Content (%)	Quarry Dust Content (%)	Duration (days)	Measured UCS (KPa)	Computed UCS (KPa)
2	10	28	85	95.607
2	20	28	93	89.147
2	30	28	112	90.887
2	40	28	121	100.827
2	50	28	128	118.967
2	60	28	138	145.307
4	10	28	85	75.661
4	20	28	90	66.301
4	30	28	98	65.141
4	40	28	118	72.181
4	50	28	139	87.421
4	60	28	152	110.861
6	10	28	77	49.523
6	20	28	92	37.263
6	30	28	97	33.203
6	40	28	111	37.343
6	50	28	141	49.683
6	60	28	175	70.223
8	10	28	79	17.193
8	20	28	89	2.033
8	30	28	131	-4.927
8	40	28	149	-3.687
8	50	28	178	5.753
8	60	28	186	23.393

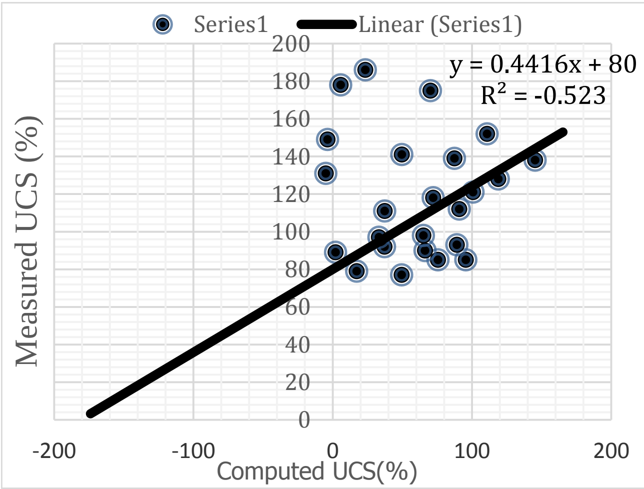

Fig. 7: Cross Plot of Measured Vs Computed UCS Values using equation 1.7

Table 17: Lime - Quarry Dust Stabilization UCS Results at 28 days Curing Duration

Sample Location 4				
Lime Content (%)	Quarry Dust Content (%)	Duration (days)	Measured UCS (KPa)	Computed UCS (KPa)
2	10	28	85	112.023
2	20	28	88	114.013
2	30	28	116	126.803
2	40	28	135	150.393
2	50	28	148	184.783
2	60	28	158	229.973
4	10	28	88	96.525
4	20	28	93	101.675
4	30	28	117	117.625
4	40	28	137	144.375
4	50	28	149	181.925
4	60	28	168	230.275
6	10	28	87	74.267
6	20	28	115	82.577
6	30	28	131	101.687
6	40	28	136	131.597
6	50	28	158	172.307
6	60	28	192	223.817
8	10	28	98	45.249
8	20	28	139	56.719
8	30	28	148	78.989
8	40	28	171	112.059
8	50	28	198	155.929
8	60	28	212	210.599

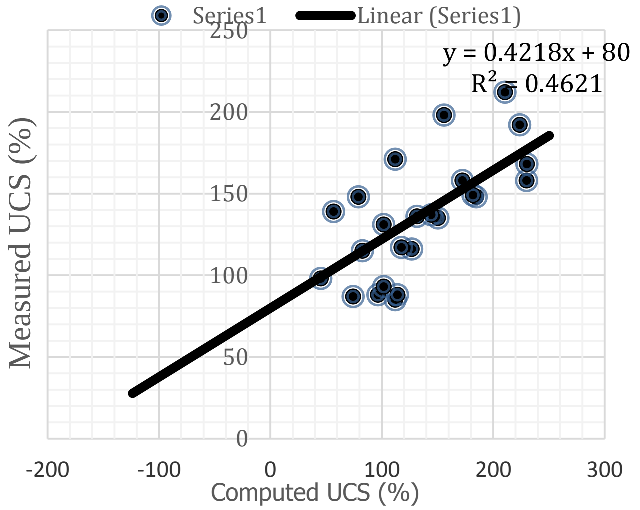

Fig. 8: Cross Plot of Measured Vs Computed CBR Values using equation 1.8

VII. CONCLUSION

Table 10 to 13 present the multiple regressed variables for measured and computed CBR values derived from lime - quarry dust composite stabilization. The values vary from 76% - 142% and 87% - 242% for measured and computed CBR values respectively. Tables 14 to 15 show the results of the UCS values after 7-day curing duration. The data range from 74kPa – 167kPa and 20kPa – 231kPa at optimal level for computed and measured values. Tables 16 and 17 present the UCS values for 28-day curing duration. The values obtained ranged from 85kPa – 192kPa and 95kPa – 223kPa for measured and computed values respectively.

Models 1.1 to 1.4 seem to generate relatively higher computed values of CBR. With a 2% lime content and quarry dust content from 10% - 50%, the measured and computed CBR values ranged from 66% - 122% and 41% - 509% respectively. The computed values though acceptable could further be minimized by subjecting the input variables to some basic iteration. Models 1.5 to 1.8 are considered adequate for this research. With a 2% lime content and quarry dust content from 10% - 50%, the measured and computed UCS values range from 74kPa – 124kPa and 20kPa – 179kPa for a 7-day curing duration with similar input variables. At 28-day curing duration the measured and computed values range from 85kPa – 192kPa and 95kPa – 223kPa respectively.

The accuracy and reliability of the models were checked by comparing the measured and computed values of CBR and UCS and computing the correlation coefficients. The figures I to IV and V to

VIII illustrate the measured and computed values of CBR and UCS based on non-linear regressed models. The straight line in the figure represents the line of perfect equality where the measured and computed values are exactly equal. The correlation coefficients R^2 at 95% confidence interval are 0.545, 0.389, 0.012, 0.04 for CBR and 0.2936, -0.545, -0.523, 0.4621 for UCS at similar stabilization parameters. These values are statistically significant and suggest that the measured and computed values are compatible.

ACKNOWLEDGMENT

The author would like to acknowledge the contribution of Esudo Engineering Ventures for support rendered in the course of this research.

REFERENCE

[1] Reynolds, O., (1886) "Experiments showing dilatancy, a property of granular material, possibly connected with gravitation," Proc. Royal Institution of Great Britain.

[2] Reynolds, Osborne (1885). "LVII. On the dilatancy of media composed of rigid particles in contact, with experimental illustrations". Philosophical Magazine Series 5. 20 (127): 469–481.

[3] Mitchel, J. K., Soga, K. (2005) Fundamentals of Soil Behavior, 3rd Edition, John Wiley & Sons Inc.

[4] Santamanna, J. C. Klein, K. A. Fam, M. A. (2001) Soils and Waves: Particulate Materials Behavior, Characterization and Process Monitoring. John Wiley & Sons Inc.

[5] Breneman, E. J. [2013] EJB´s Soil Stabilization

[6] Butler, R. L. and Cerato, A. B. (2007). Stabilization of Oklahom Expansive Soils using Lime and Class C Fly-Ash. Proc. of Session of Geo Denver. Colorado, USA.

[7] Kamon, M. and Nontananandh, S. (1991). Combining Industrial Wastes with Lime for Soil Stabilization. Journal of Geotechnical Engineering Vol. 117, pp 1-17.

[8] Federal Ministry of Works & Housing Specifications, (1997), Nigeria.