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Lateritic Soil Stabilization using Granular Materials; a Comparative Study

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ABSTRACT: Soil mixtures have been used extensively in developed countries to construct great lengths of road when other materials like cement and lime were not either available, developed or widely used. Laterite being readily available in many places can be mixed with sand or quarry dust to improve its strength, stability and other properties for this purpose.This research work involves the improvement of the engineering properties of three samples of laterite soil by stabilization with quarry dust or with sharp river sand. The paper sets out to investigate and compare the improvement in the relevant properties of three samples of laterites when they are mixed with sand or quarry dust and compacted using different compactive efforts (British Standard Light – BSL and British Standard Heavy- BSH). The three samples used were collected at different locations in Awka, Anambra state. The tests carried out include: moisture content, specific gravity, grain size distribution, Atterberg's limits, compaction and unconfined compressive strength tests both before and after adding quarry dust or sharp river sand at varying percentages (0%, 10%, 20%, 30%, 40% and 50%). The results obtained showed that for all types of laterites tested, the increase in the proportion of sand or quarry dust to laterites has the effect of increasing the maximum dry density, specific gravity and reducing the optimum moisture content, the unconfined compressive strength and energy absorption capacity. The optimum sand or quarry dust percentage for maximizing most of the parameters measured was found to be between 30% and 40% of the admixture. Comparison of the results obtained shows that quarry dust has a greater impact in the improvement of the strength of the natural soil than sand.

Keywords: Laterite soils, Consolidation, Admixtures, Moisture content, Specific gravity, Grain size distribution, Atterberg's limits, Compaction and Unconfined Compressive Strength tests

I. INTRODUCTION

Most of the problems encountered by engineers at construction site are often associated with the properties of construction materials being unable to reach the required specification for good engineering performance. The problems faced usually involve the presence of soils with properties that are not suitable for engineering and/or construction purposes. By implication, the project being handled may suffer failure, or have functionality problems due to the presence of such unsuitable earth materials. As it is well known, soils are complex products of nature and possess variable properties due to a number of reasons, and often times, soils that are encountered are unsuited for the requirements of the construction either wholly or partially (Bolarinwa & Ola, 2016). Clays are one of the problematic soils that are encountered in construction practice. Generally, clays exhibit low strength and high compressibility which are often time dependent (Wang et al., 2020). Many are sensitive, in the sense that their strength is reduced by mechanical disturbance(Cotecchia & Chandler, 2000). Hence, the construction over clay soil may experience bearing capacity failure induced by its low shear strength. Therefore clay soil has to be improved and stabilized either with lime or any stabilizing agent before any engineering works can commence (Saidate et al., 2022).

Lime stabilization involves the use of lime as a stabilizing agent to alter the properties of an existing soil chemically to meet the specified engineering requirements based on its application, and to improve the compressibility characteristics of the soil. Soil stabilization aims at adjusting the mechanical and chemical characteristics of soils through the pozzolanic reaction, which improve the engineering properties of the soil. (Onyelowe et al., 2021)

In some cases, chemical stabilization is useful for building the soil systems like dams, canals and river levees.

One of the most important properties of soil in engineering construction is its compressibility. This factor is demanded in large number of geotechnical application such as dams projects, foundation and embankment systems. The compressibility of soil is defined by its consolidation process.

Consolidation is a process in which pore water is dissipated, and the voids in the soil are reduced by implementation of loads in a specific time. The time rate of consolidation is associated with the volume of air and the permeability of the soil. If the soil is not saturated completely, a part of consolidation will happen almost immediately because of expulsion of air or water that remains among the soil particles. The deformation of the soil particles will lead to the settlement of the soil mass. The compressibility of the soil is measured by its compression index (Cc), which is the slope of the line achieved through the void ratio versus the effective vertical stress curve.

II. MATERIALS AND METHODS OF TESTING

2.1 Materials

2.1.1 Soil samples

Three lateritic soil samples were collected from borrow pit at Nachi, Enugu State. These samples collected were tested for their index properties, compaction characteristics and consolidation characteristic. The main work is centered on the compressibility behavior of the soil samples when untreated and treated with lime at different percentage increment. The undisturbed samples of the soil were collected for consolidation test. BS light compactive effort was applied on each sample using standard mould.

2.1.2 Sampling Locality

Three samples (EN01, EN02, and EN03) were collected at two different locations at the site. Samples (EN01 and EN02) were collected at different depth within the same geological coordinate of (lat 6.29106 N and long 7.30581 E). Sample (EN03) was collected at geological coordinate of (lat
6.28601N and long 7.34813E)

2.1.3 Geological Study of the Area

Nachi is a village in Udi Local Government Area of Enugu State. Udi Local Government Area comprises of 23 villages with a population of over 234,002 (2006 census). It has an area of 897km^2 and its headquarters is in the city of Udi on the A232 highway. Its coordinate are 6^{O}19’N and 7O26’E. Nachi is along the boundary between Enugu and Anambra State and because of that, it shares some geographical conditions within the two states. Unlike Anambra state, Nachi lies 300m above sea in a valley plain of the Mamu River. It is situated in a fertile tropical valley but most of the original rain forest has partially been lost due to clearing for farming and human settlement. It is in the tropical rainforest zone of Nigeria and experiences two distinct seasons brought about by two predominant winds that rule the area: the south western monsoon winds from the Atlantic Ocean and the North eastern dry winds from across the Sahara desert. The monsoon wind creates seven months of heavy rain while the dry wind brings five months of dryness. The temp is between 27-30^o Celsius between June and December and rises to 32-34^0 between January and April.

2.2 Methods

2.2.1 Index Properties

Laboratory tests were conducted to determine the index properties of the natural soil and soil- rice husk ash mixtures in accordance with British Standards

- Water Content (Moisture Content)
- Particle Size Distribution, (PSD)
- Atterberg Limits

2.2.2 Computation of Moisture (Water) content

The moisture content of the soil sample, w, is calculated as a percentage of the dry soil mass to the nearest 0.1 %, from the equation:

$$w = \left(\frac{M2 - M3}{M3 - M1}\right) 100\%$$

Where M1 is the mass of container (in g); M2 is the mass of container and wet soil (in g); M3 is the mass of container and dry soil (in g).

2.2.3 Computation of particle size distribution

The test result is presented in a table showing below, to the nearest 1 %, the percentage by mass passing each of the sieves used.

Calculate the percentage by mass of material retained on each test sieve from the equation: $\%Retained = (M_R/M_0) \times 100\%$

Where M_R= mass of soil retained on respective sieve

M_0= total mass of soil sample

Compute the cumulative percentage (by mass of the total sample) passing each of the sieves.

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

Also compute cumulative percentage (by mass of the total sample) retained each of the sieves.

Table 2: Typical table of values for particle size analysis

Mass of soil before sieving: ………………..

Sieve sizes (mm)	Mass retained (g)	% Mass retained	Cumulative % passing	Cumulative % retained
2mm				
400µm				
75 µm				
Total				

Analysis

$$C_U = \frac{D_{60}}{D_{10}}$$

$$C_C = \frac{(D_{30})^2}{D_{60} x D_{10}}$$

Where

C_U = Uniformity coefficient

C_C = Coefficient of curvature

D_{60} = Particle size such that 60% of the soil is finer than this size

D_{10} = Particle size such that 10% of the soil is finer than this size

D_{30} = Particle size corresponding to 30% finer

When $C_U < 2$, the soil is uniform and non uniform when $C_U > 2$.

Coarse grained soil is when more than half is larger than sieve number 200

Well graded sand (SW) is when the value obtained from C_U is greater than 6 and C_U is between 1 & 3.

Poorly graded sand (SP) is when not meeting the entire gradation requirement of well graded sand (SW)

2.2.4 Computation of Atterberg limits (consistency)

The Atterberg limits consists of the liquid limit (LL), the plastic limit (PL) and the shrinkage limit (LS). A value frequently used in conjunction with these limits is the plasticity index (PI). The hydraulic conductivity of soil vary with the amount of water present, and results of the three consistency tests, expressed as moisture contents are arbitrarily used to differentiate between the various states of material.

The liquid limit and plastic limit for the three lateritic soil samples were determined using the procedures described in ASTM D 4318-00.

Plasticity index (PI):

$$PI = LL - PL$$

Where; LL = Liquid Limit

PL = Plastic Limit

2.2.5 Computation of Specific gravity

Table3: Typical table for specific gravity readings and calculation

SAMPLE DESIGNATION	MASS OF DENSITY BOTTLE (g) M1	MASS OF DENSITY BOTTLE +DRY SOIL (g) M2	MASS OF DENSITY BOTTLE +DRY SOIL + WATER (g) M3	DENSITY BOTTLE + WATER (g) M4	SPECIFIC GRAVITY' Gr. Gr $= (\frac{M2 - M1}{(M4 - M1) - (M3}$
SAMPLE 1					
SAMPLE 2					
SAMPLE 3					

$$G = (\frac{M2 - M1}{(M4 - M1) - (M3 - M2)})$$

2.2.6 Computation of Differential free swell

Free swell or differential free swell, also termed as free swell index, is the increase in volume of soil without any external constraint when subjected to submergence in water.

Analysis

$Sf = \frac{Vf - Vd}{Vd} x100$

Vd = Initial Volume of the soil specimen read from the graduated cylinder containing distilled water=10ml.
Vf = Final Volume of the soil specimen read from the graduated cylinder containing distilled water after 24hrs
Sf= free swell

2.3 Consolidation Test
Objective/scope of work

The objective of the oedometer consolidation test is to determine consolidation characteristics of a sample of tropical lateritic soil with low permeability. The test determines the important consolidation parameters of clays, as follows:

i. Coefficeient of volume compressibility (mv)
ii. Compression index (Cc)
iii. Pre-consolidation Pressure (Pc)/Yield stress
iv. Coefficient of vertical consolidation (Cv)

III. RESULTS AND DISCUSSION

Classification of Soil samples: AASHTO Soil Classification System.

The soil samples save less than 35% maximum of their particles passing through the No. 200 BS sieve, they may be classified into group A-2 soils which may be either Inorganic silts with little or no plasticity silt-type materials. From the observations it is deduced that the soil is not plastic. The non-plastic nature of the soil is as a result of higher amount of silt than clay. Therefore the samples can be classified as well graded silty sand (SM)

3.1Compaction characteristics of the natural soil

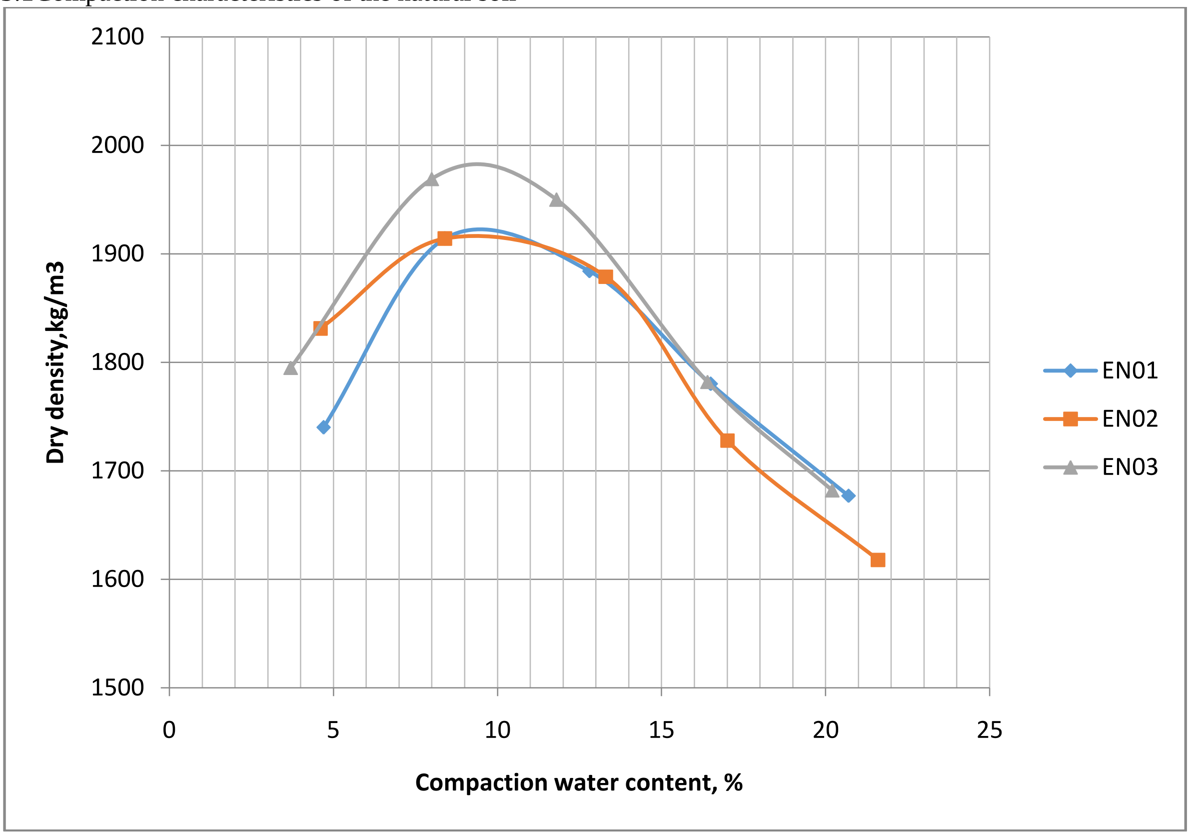

Figure 1: Graph of dry density against water content

3.2 Consolidation characteristics

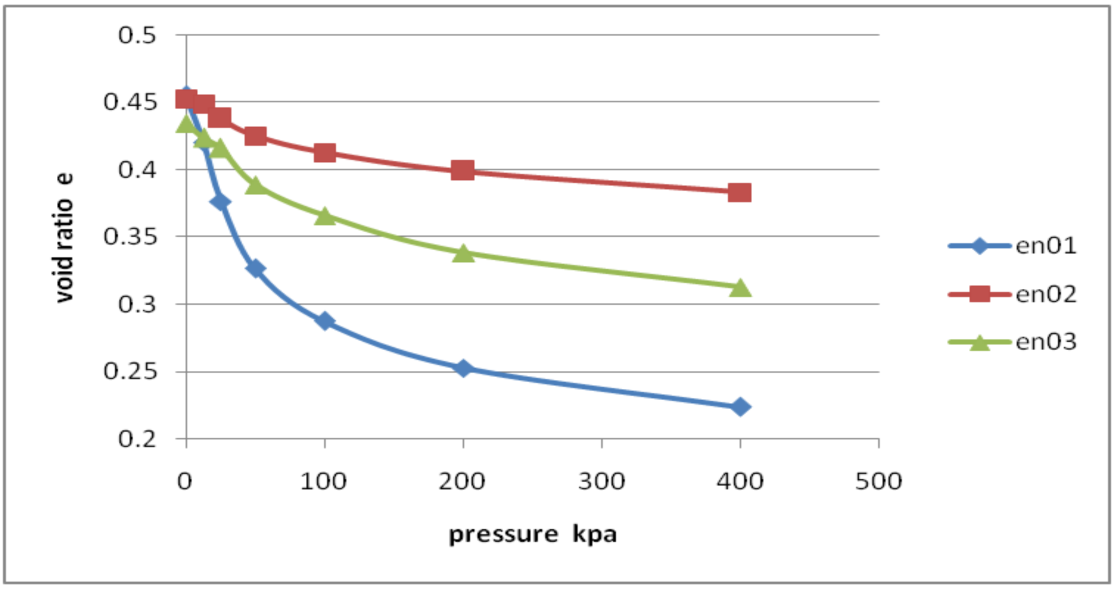

Figure 2:Graph of void ratio against pressure

From the graph above, it can be deducted that void ratio decreases as pressure increases. EN02 has higher range of void ratio followed by EN03 and EN01 of the undisturbed soils.

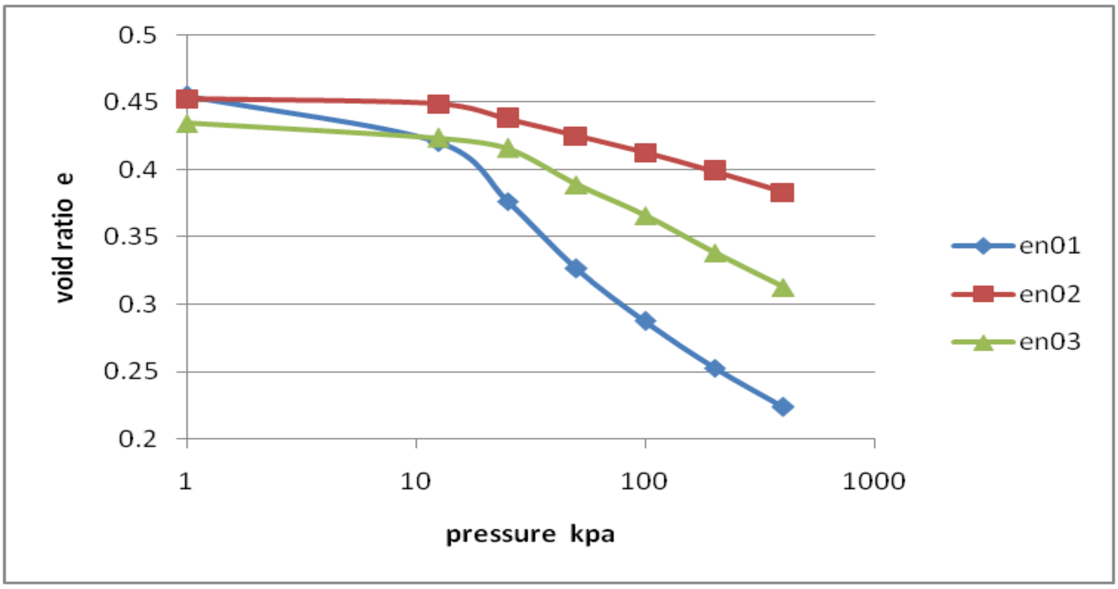

Figure 3:Graph of e against log p (kpa)

From the graph, there was gradual decrease of void ratio as pressure increases. At 10kpa, the samples show sharp reduction of void ratio as the pressure increases to 400kpa. The sharp decrease within (10kpa-400kpa) was due to failure of the bond holding the soils.

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

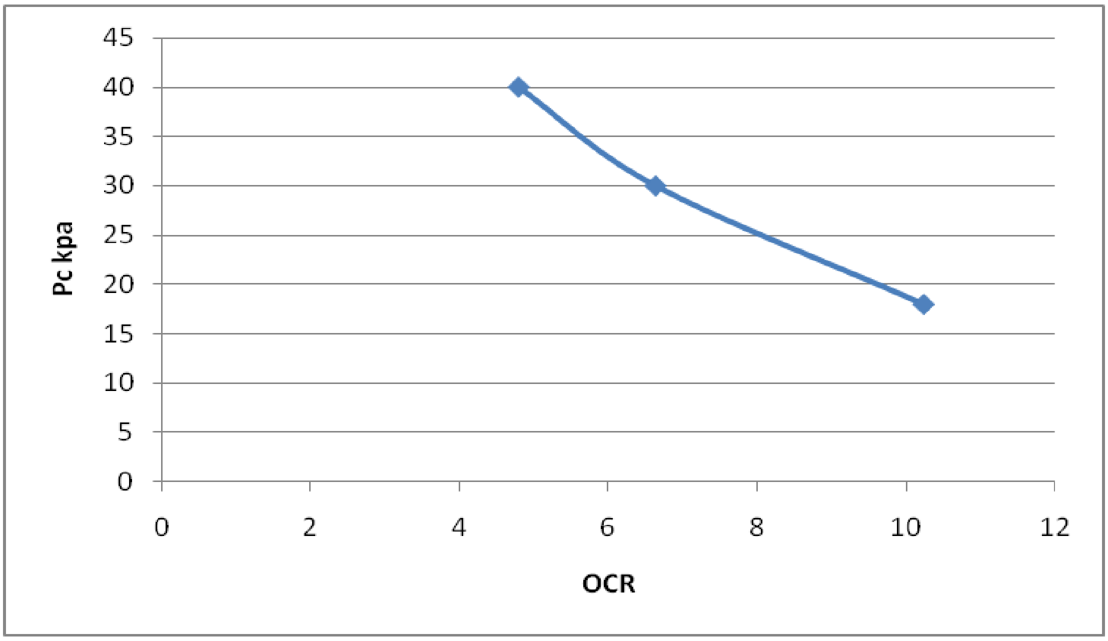

Figure 4: Graph of pre-consolidation pressures against over-consolidation ratio

The graph above shows gradual reduction of pre-consolidation pressure as over-consolidation ratio increases. It can be noted that higher the over-consolidation ratio, lesser the pre-consolidation pressure.

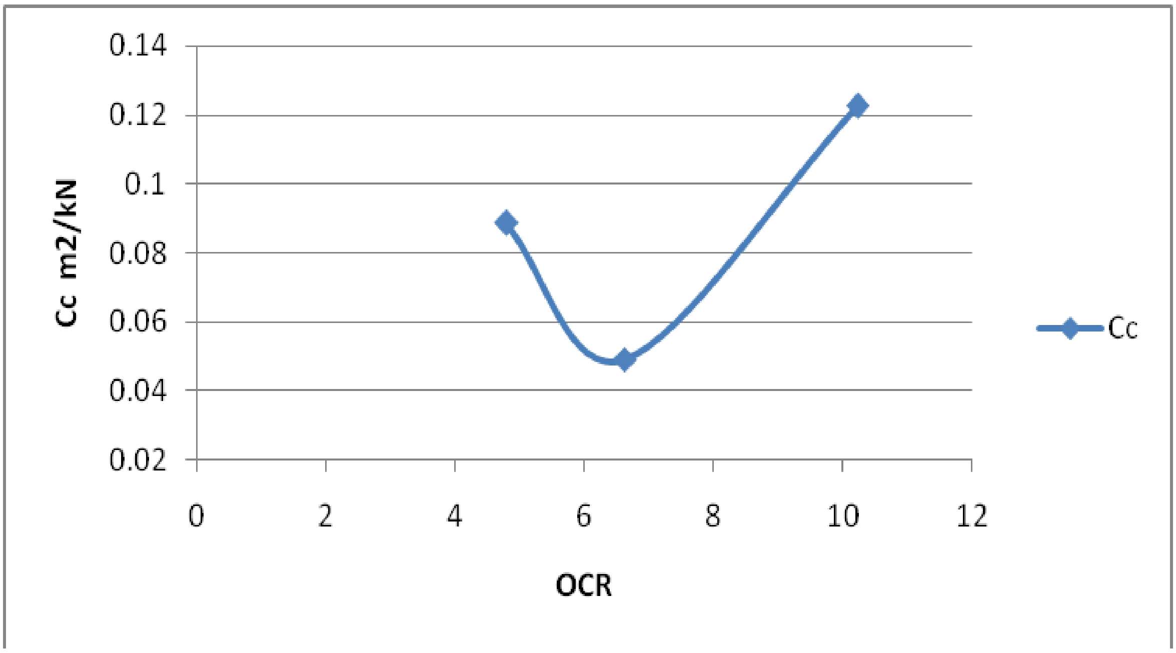

Figure 5: compression index against over consolidation ratio

From the graph above, the curve show decreasing trend initially before increasing at higher over-consolidation ratio and compression index.

International Journal of Advances in Engineering and Management (IJAEM)
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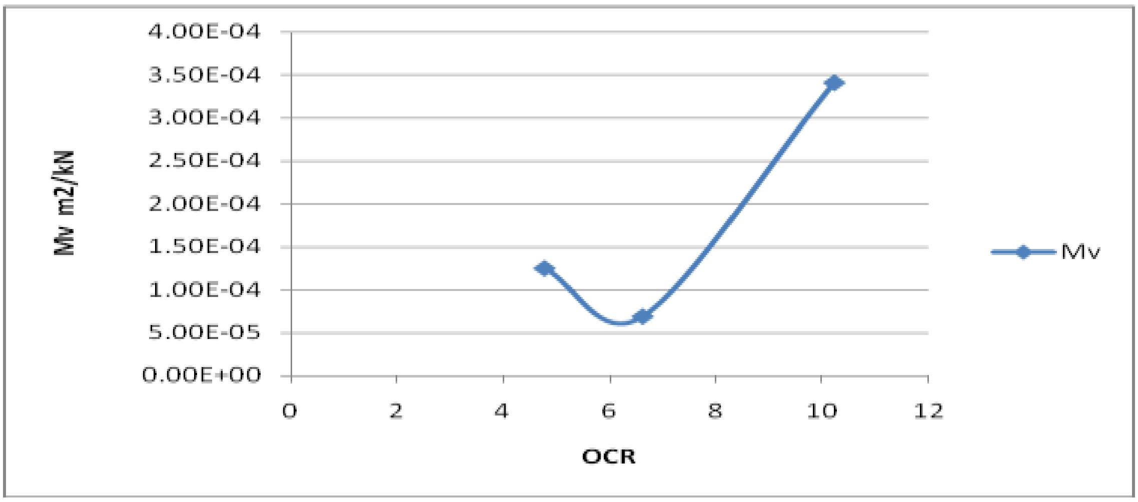

Figure 6:Graph of coefficient of volume change against over-consolidation ratio

From the graph above, the curve show decreasing trend initially before increasing at higher over-consolidation ratio and coefficient of volume change.

From figure 5 and 6, it can be noted that at 6 OCR the soils have minimum values of Cc and Mv.

3.3 Properties of lime-treated soils

3.3.1 Specific gravity

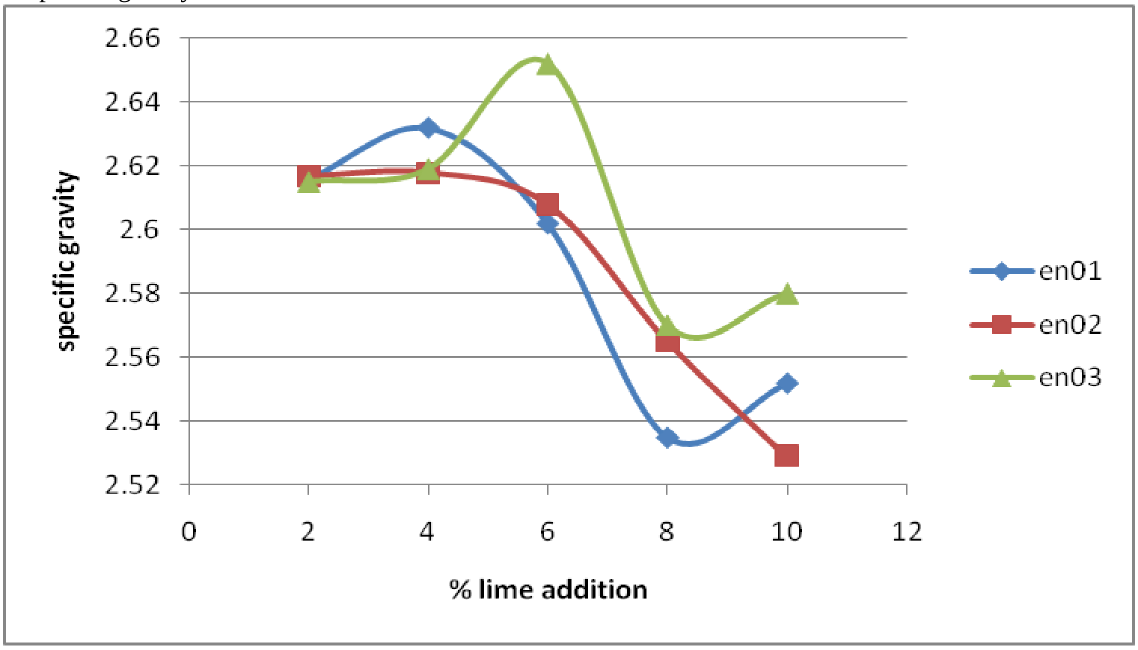

Figure 7: Graph of specific gravity against % lime addition

The specific gravity of sample 1(EN01) ranges between 2.53 and 2.63. From the result, the maximum value of specific gravity is at 4% lime mixtures for EN01. The specific gravity of sample 2 (EN02) ranges between 2.52 and 2.61.From the result, the maximum value of specific gravity is at 4% lime mixtures. The specific gravity of sample 3 (EN03) ranges between 2.57 and 2.65.From the result, the maximum value of specific gravity is at 6% lime mixtures. The specific gravity of lime is 2.36. From observation of the trend in which the specific gravity decreases with lime, it can be deducted that the decrease in specific gravity is due to addition of lime in excess. The specific gravity values are within the range recommended by AASHTO (Bello 2012). Lower specific gravity value indicates a coarse soil, while higher values indicate a fine grained soil.

3.3.2 Free swell

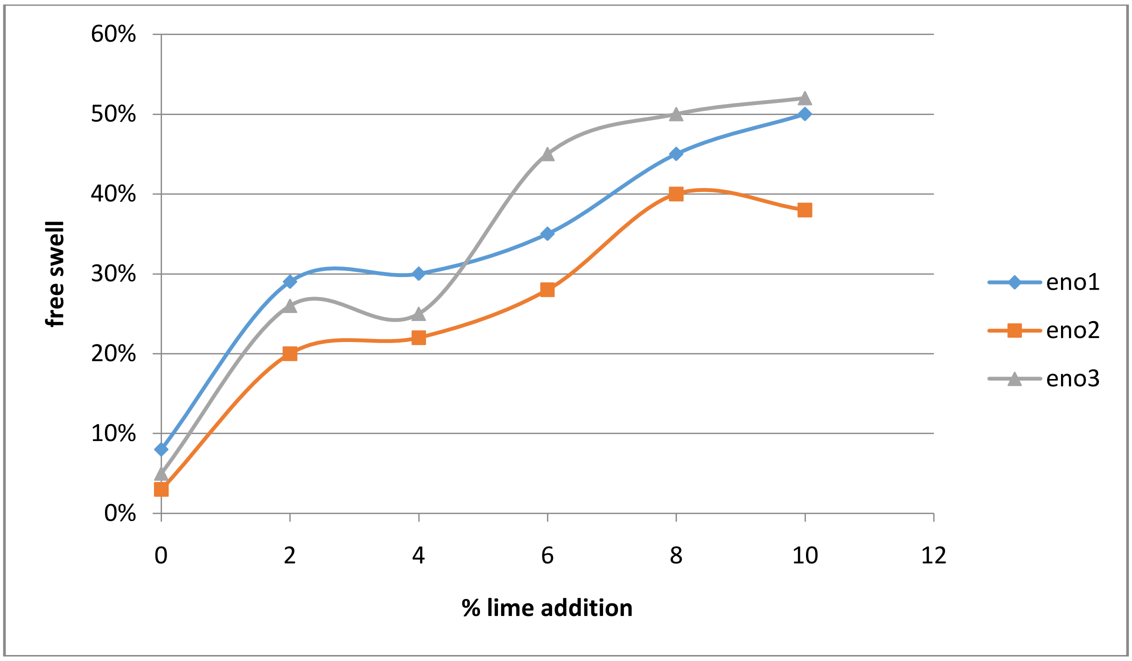

Figure 8: Graph of free swell against % lime addition

All the samples (EN01, EN02, and EN03) show a gradual increase of free swell as the lime addition increases. It can be deduced that the increase in the free swell is due to excess lime addition.

3.3.3 Compaction Characteristic

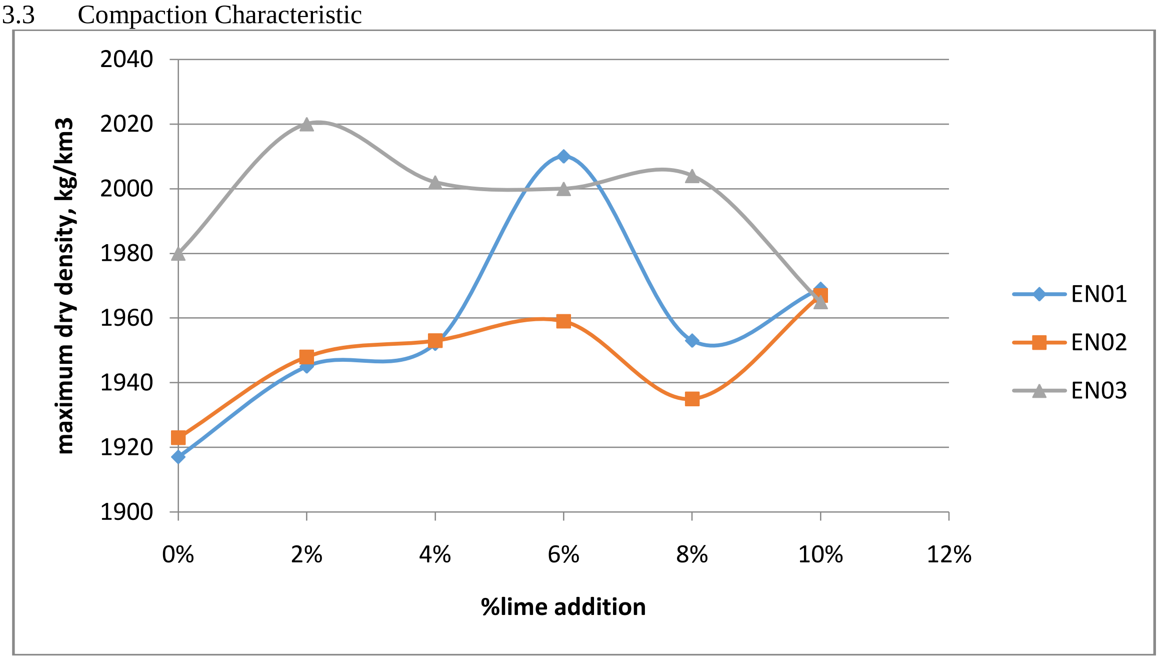

Figure 9:Graph of maximum dry density against lime %

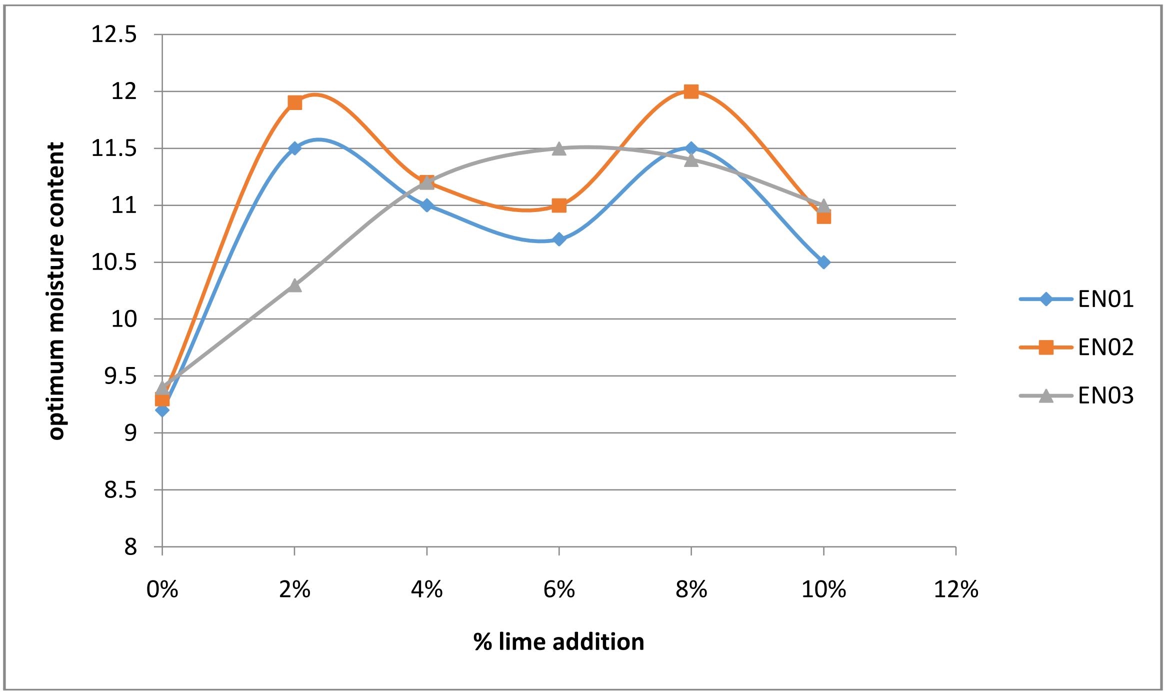

Figure10:Graph of OMC against % lime addition

The graphs above shows the trend in which lime addition to the soil vary with the maximum dry density and optimum moisture content. From the graph, the soil reacts differently as lime is being added to it. The equation on the table can help to determine the value of MDD and OMC at different lime addition.

Table 3.3.3a polynomial equation for MDD and OMC against % lime

	R^2	Polynomial equation
EN01(MDD)	1	$y = 2E+09x^5 - 4E+08x^4 + 3E+07x^3 - 1E+06x^2 + 12020x + 1917$
EN02(MDD)	1	$y = 4E+08x^5 - 1E+08x^4 + 8E+06x^3 - 28687x^2 + 4440x + 1923$
EN03(MDD)	0.995	$y = -2E+07x^4 + 3E+06x^3 - 21828x^2 + 5138.x + 1980.$
EN01(OMC)	0.996	$y = -76822x^4 + 16255x^3 - 11280x^2 + 283.7x + 9.193$
EN02(OMC)	0.996	$y = -76822x^4 + 16255x^3 - 11280x^2 + 283.7x + 9.193$
EN03(OMC)	0.992	$y = -468.7x^2 + 63.44x + 9.346$

Table 3.3.3bThe values of optimum moisture content and max dry density from polynomial equation and graph

BRITISH STANDARD LIGHT COMPACTION							
Soil sample	Lime(%)	R^2	Polynomial equation	OMC from eqn	MDD from eqn	OMC from graph	MDD from graph
(ENO1)	0	1	$y = 0.028x^4 - 1.287x^3 + 16.62x^2 - 60.68x + 1870.$	9.8	1918	9.2	1917
	2	1	$y = 0.054x^4 - 2.513x^3 + 36.64x^2 - 181.1x + 2062$	11.0	1949	11.5	1945
	4	0.998	$y = 0.198x^3 - 10.95x^2 + 163.7x + 1211.$	10.4	1952	11	1952
	6	1	$y = 0.064x^4 - 2.780x^3 + 36.43x^2 - 137.6x + 1874$	10.7	2006	10.7	2010
	8	1	$y = 0.049x^4 - 2.149x^3 + 28.01x^2 -$	11.2	1952	11.5	1953

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

			94.72x + 1747.				
	10	1	$y = 0.142x^4 - 6.334x^3 + 93.14x^2 - 509.0x + 2646$	10.9	1966	10.5	1969
(EN02)	0	0.999	$y = 0.293x^3 - 13.99x^2 + 190.7x + 1124$	9.9	1925	9.3	1923
	2	1	$y = 0.061x^4 - 2.725x^3 + 37.05x^2 - 149.1x + 1839$	11.1	1951	11.9	1948
	4	1	$y = 0.072x^4 - 3.074x^3 + 39.39x^2 - 140.5x + 1765$	10.6	1949	11.2	1953
	6	1	$y = 0.065x^4 - 3.123x^3 + 49.10x^2 - 288.2x + 2391$	11.3	1958	11	1959
	8	1	$y = 0.019x^4 - 0.742x^3 + 4.272x^2 + 77.82x + 1270$	11.3	1934	12	1935
	10	1	$y = 0.066x^4 - 2.891x^3 + 38.92x^2 - 159.7x + 1889$	10.9	1960	10.9	1967
(EN03)	0	0.996	$y = 0.277x^3 - 12.89x^2 + 163.4x + 1351$	8.4	1980	9.4	1980
	2	1	$y = 0.084x^4 - 3.608x^3 + 47.37x^2 - 193x + 1968$	10.4	2009	10.3	2020
	4	0.988	$y = 0.491x^3 - 22.23x^2 + 295.8x + 775.5$	9.9	2002	11.2	2002
	6	1	$y = 0.061x^4 - 2.755x^3 + 36.77x^2 - 134.5x + 1790.$	11	1986	11.5	2000
	8	1	$y = 0.085x^4 - 3.777x^3 + 52.12x^2 - 217.3x + 1866$	11.5	2002	11.4	2004
	10	1	$y = 0.109x^4 - 4.829x^3 + 69.24x^2 - 350.4x + 2264$	11	1956	11	1965

The maximum dry density (MDD) for sample 1(ENO1) ranges between 1917Kg/M^3 and 2010Kg/M^3 (BSL), and their optimum moisture content (OMC) ranges between 9.2% and 11.5% (BSL), MDD increases with increase in lime percentage up to 6% where it is maximum before it starts decreasing, but at 2% and 8% lime increment the MC was at its peak.

The maximum dry density (MDD) for sample 2(EN02) ranges between 1923Kg/M^3 and 1967Kg/M^3 (BSL), and their optimum moisture content (OMC) ranging between 9.3% and 12% (BSL), MDD increases with increase in lime percentage till after 6% where there was partial drop up to 8% but peak increase at 10%. For the OMC it is maximum at 8% lime increment.

The maximum dry density (MDD) for sample 3(EN03) ranges between 1965Kg/M^3 and 2020Kg/M^3 (BSL), and their optimum moisture content (OMC) ranging between 9.4% and 11.5% (BSL), MDD increases at 2% after which it starts decreasing with increase in lime percentage. For the OMC, it is maximum at 6% lime increment.

According to O'Flaherty (1988) the ranges of values that may be anticipated when using the standard proctor test methods are: for clay, maximum dry density (MDD) may fall before 1.44Mg/m3 and 1.685Mg/ m3 and optimum moisture content (OMC) may fall between 20-30%. For silty clay MDD is usually between 1.6 and 1.845Mg/m3 and OMC ranged between 15-25%. For clayey or silty sand, MDD usually ranged between 1.76 and 2.165Mg/m3 and OMC between 8 and 15%. Thus, looking at the results of the soil samples, it could be noticed that they are clayey or silty sand materials.

Since the best compaction can be achieved at OMC, its determination will help to carry out compaction in the field by adding that amount of water to the soil during compaction. This values has given highway Engineers a guide on the amount of water needed to achieve a dry density of those values for laterite from the borrow pits under investigation.

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

3.3.4 Compressibility characteristic of lime-treated soils
Void ratio Vs log p

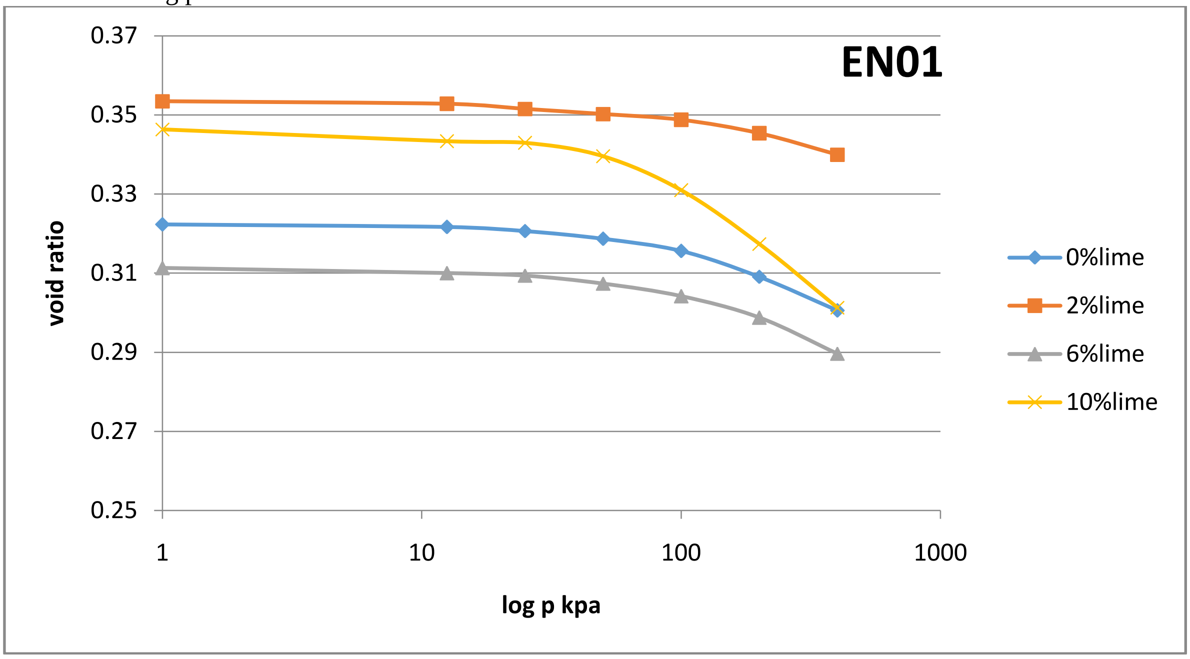

Figure 11:Graph of void ratio against log p

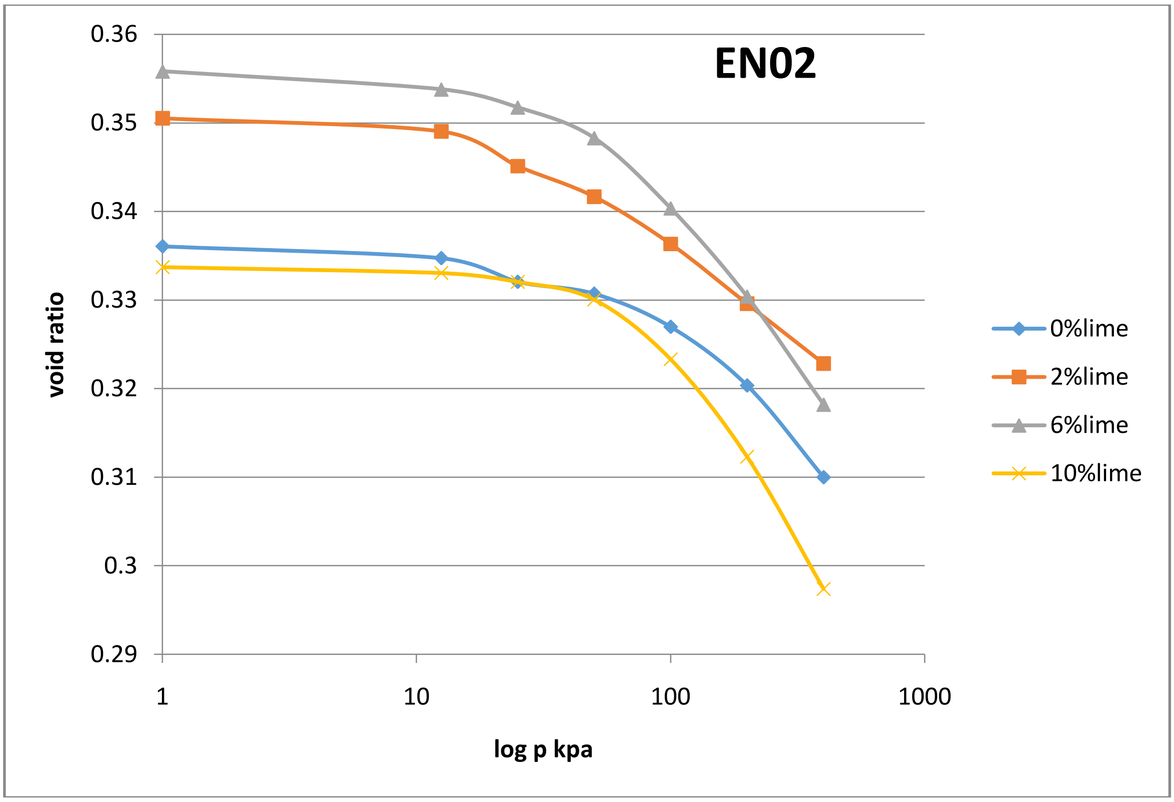

Figure 12:Graph of void ratio against log p

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

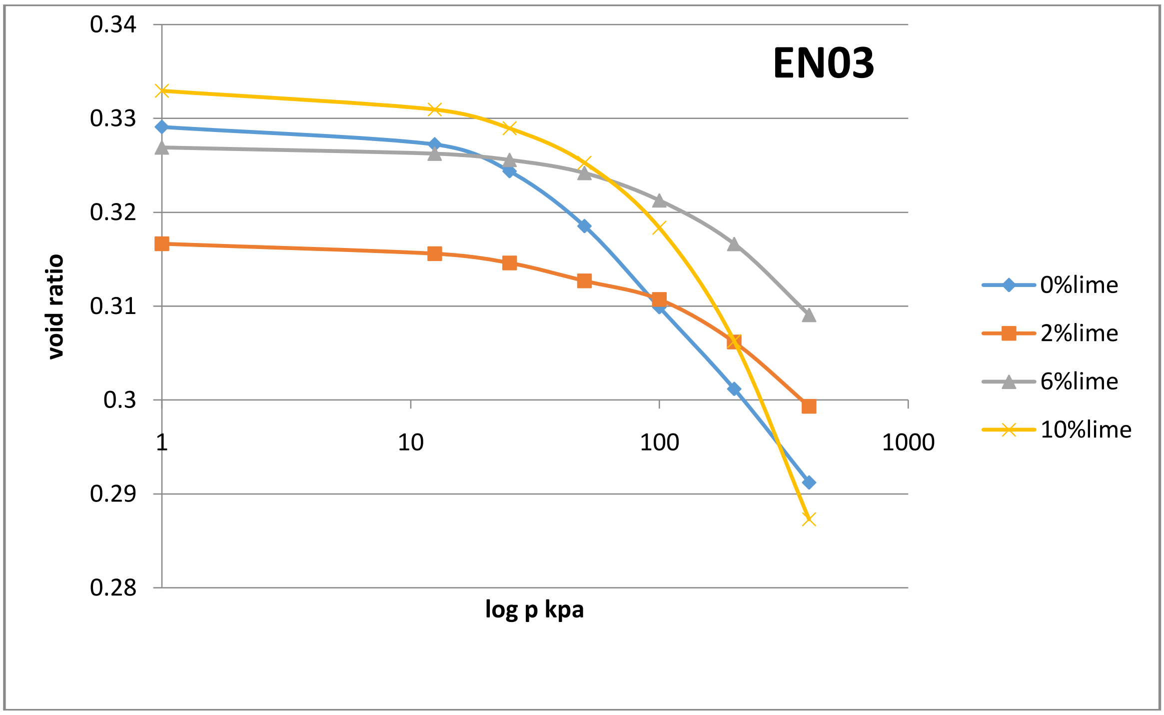

Figure 13:Graph of void ratio against log p

The graphs above show that void ratio decreases as pressure increases. It can be deduced that EN01 has highest void ratio at 2% lime addition, EN02 has highest void ratio at 6% lime addition and EN03 has highest void ratio at 10% lime addition.Higher value of void ratio implies that the soil exhibit a greater resistant to compression. The sharp drop of void ratio as pressure increase above 50kpa was due to effect of structuration.

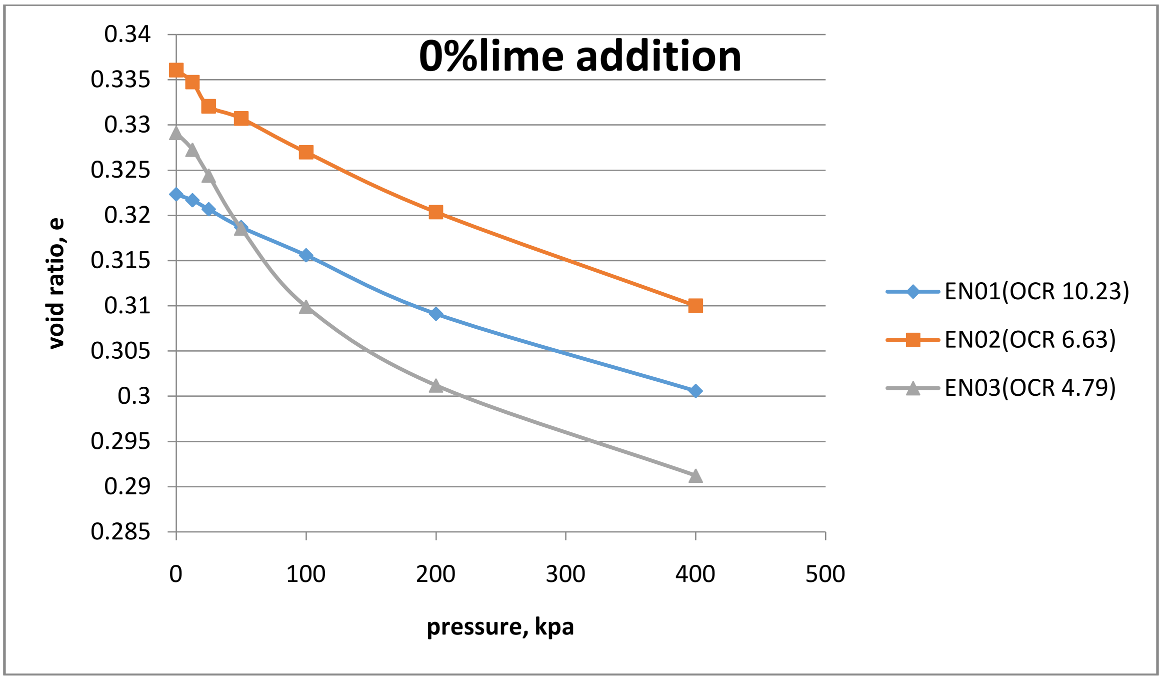

Figure 14-1:Graph of void ratio against pressure

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

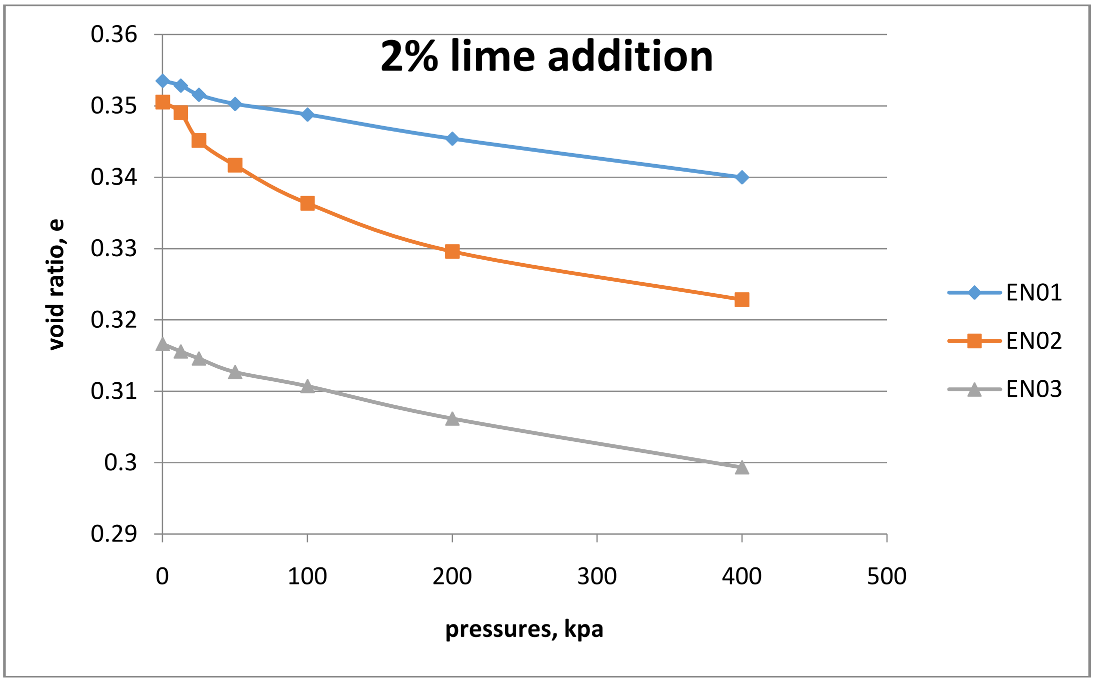

Figure 14-2:Graph of void ratio against pressure

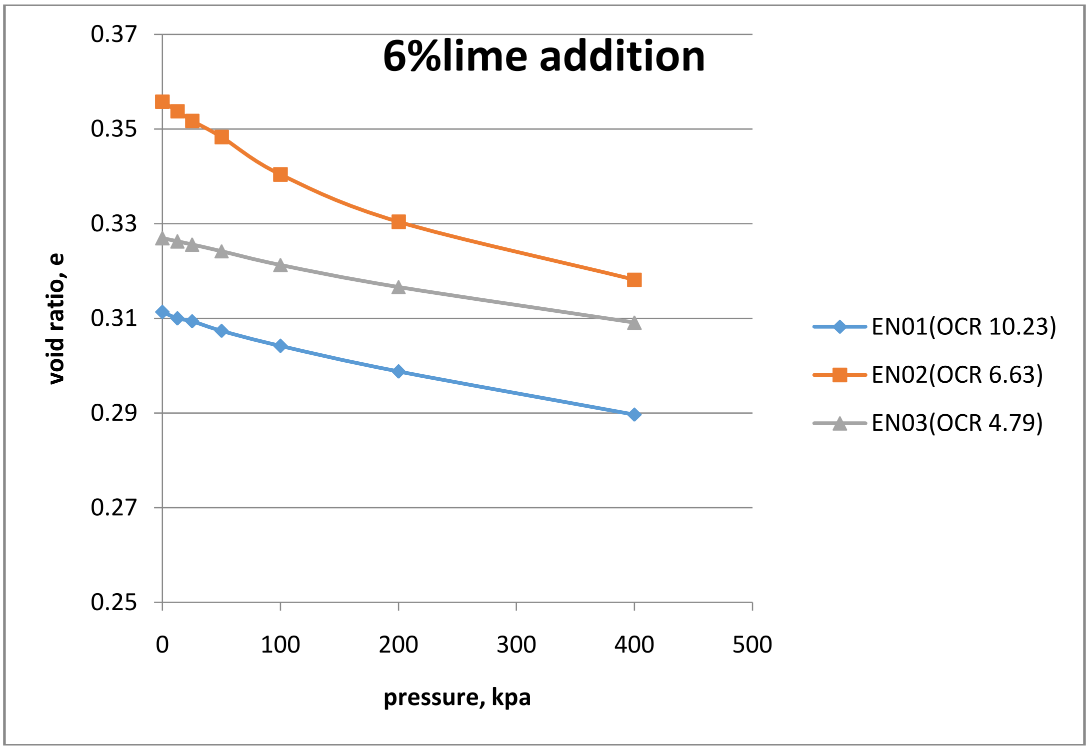

Figure 14-3:Graph of void ratio against pressure

International Journal of Advances in Engineering and Management (IJAEM)
Volume 7, Issue 01 Jan. 2025, pp: 145-164 www.ijaem.net ISSN: 2395-5252

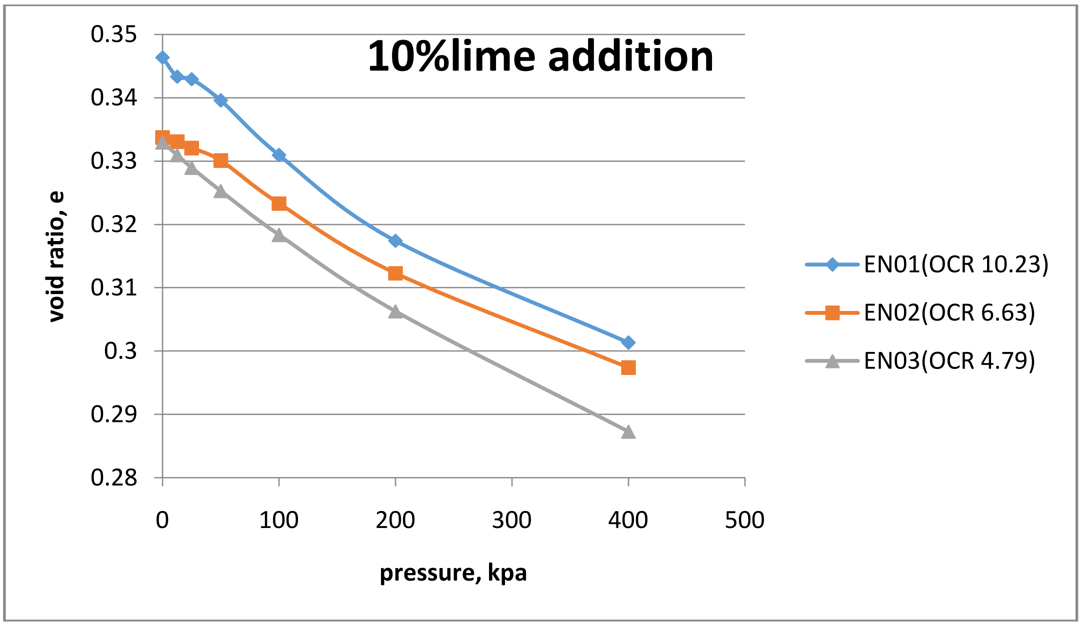

Figure 14-4:Graph of void ratio against pressure

From the graphs above, EN01 has higher value of void ratio at 2%lime and 10%lime while EN02 has higher values at both 0%lime and 6%lime additions to the soil. This implies that samples with higher OCR value should have higher values of void ratio. Generally, void ratio decreases as pressure increases.

Strain (%) Vs log p

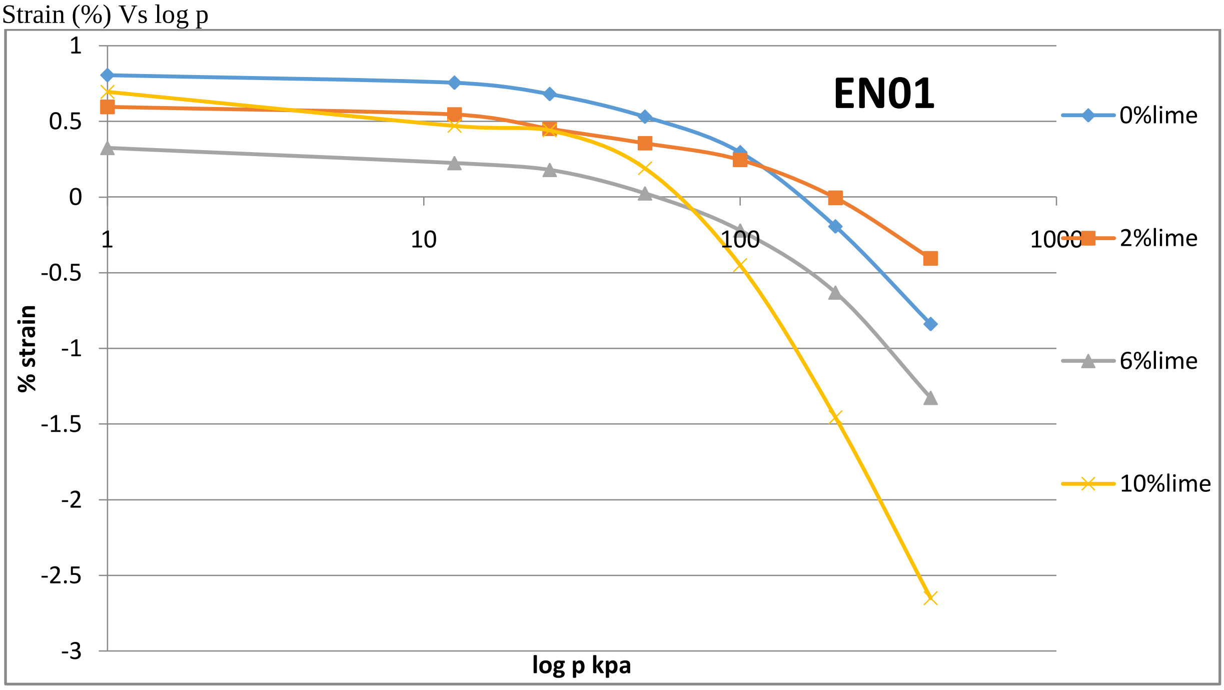

Figure 15:Graph of strain(%) against log P

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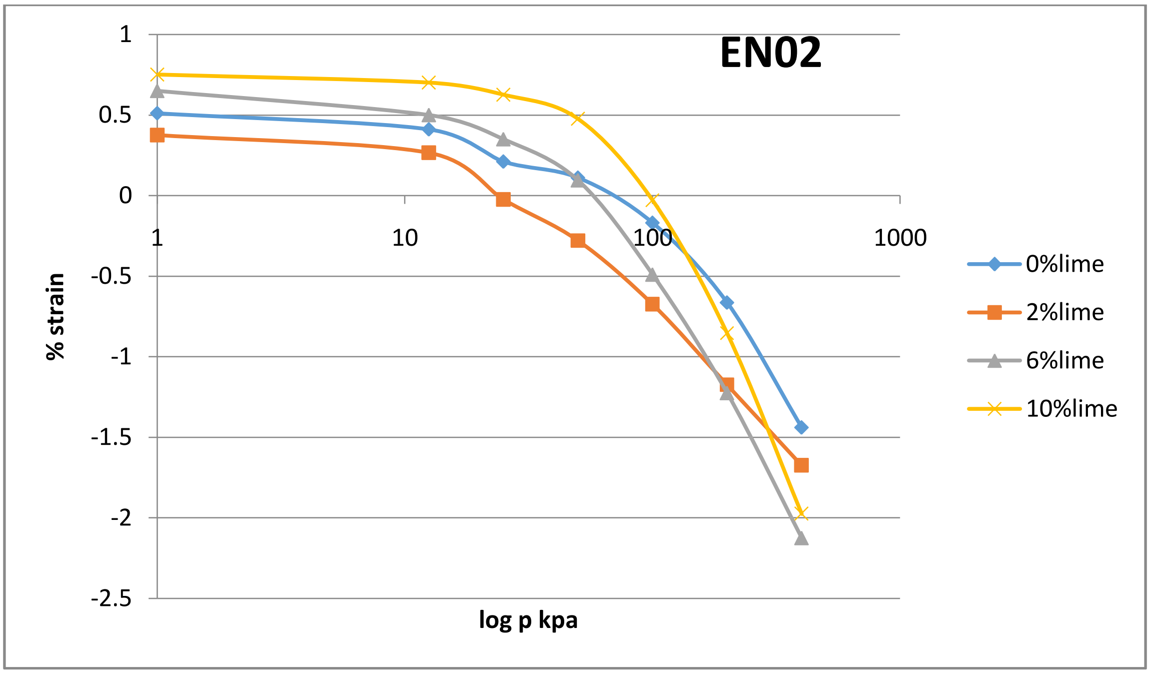

Figure 16:Graph of strain(%) against log P

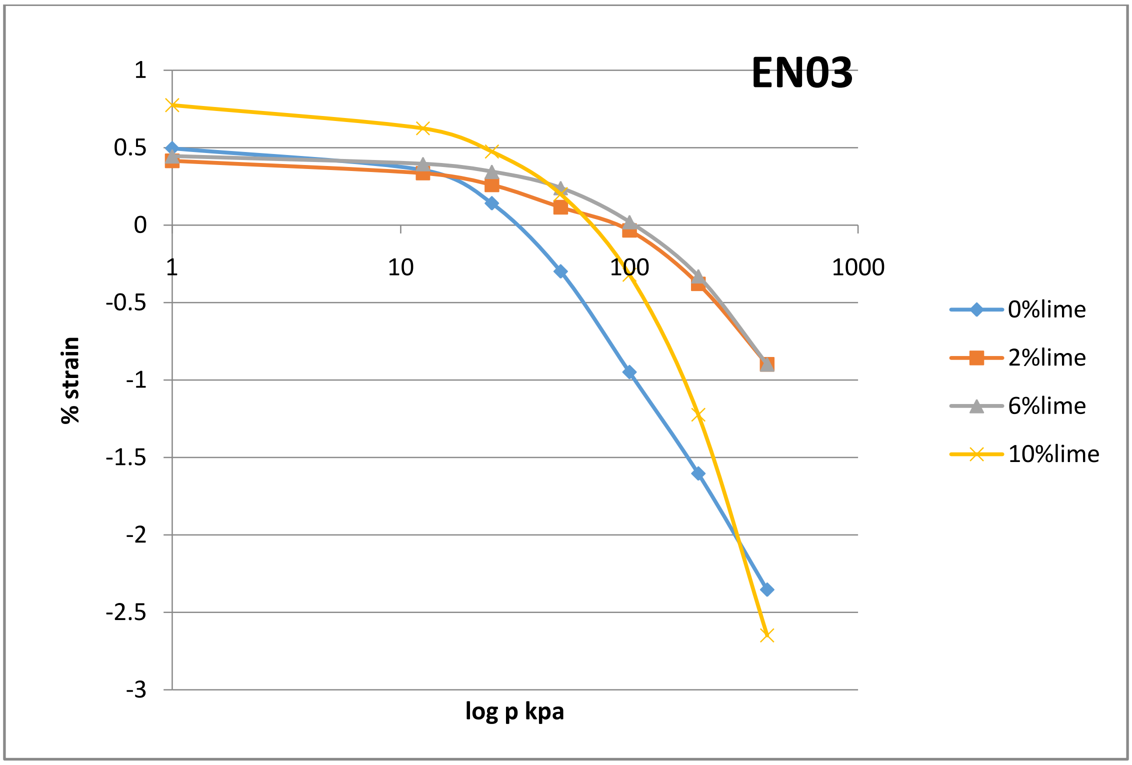

Figure 17: Graph of strain(%) against log P

From the graphs of % strain against log p of the three samples above, it shows a reduction in strain as the pressure increases.EN01 has highest % strain value at 0% lime content.EN02 and EN03 have highest %strain value at 10% lime content. This shows that the soil resisted deformation when the load was applied to it. The sharp reduction of strain was observed from 10kpa to 400kpa. The reduction was due to failure of the bond holding the soils.

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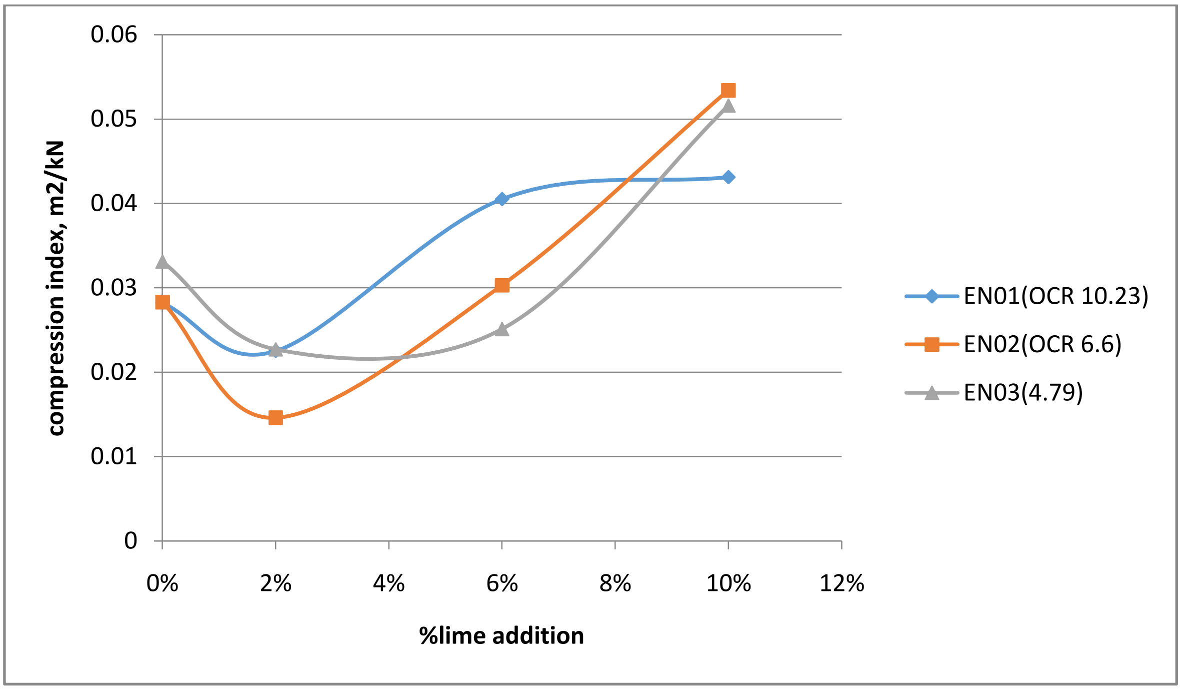

Figure 18:Graph of Compression index against % lime addition

From the graph above, compression index decreases initially from 0% lime to 2% lime before gradual increase as lime content increases. This implies that the compressibility of the soils reduces at 2%lime addition to the the soil.

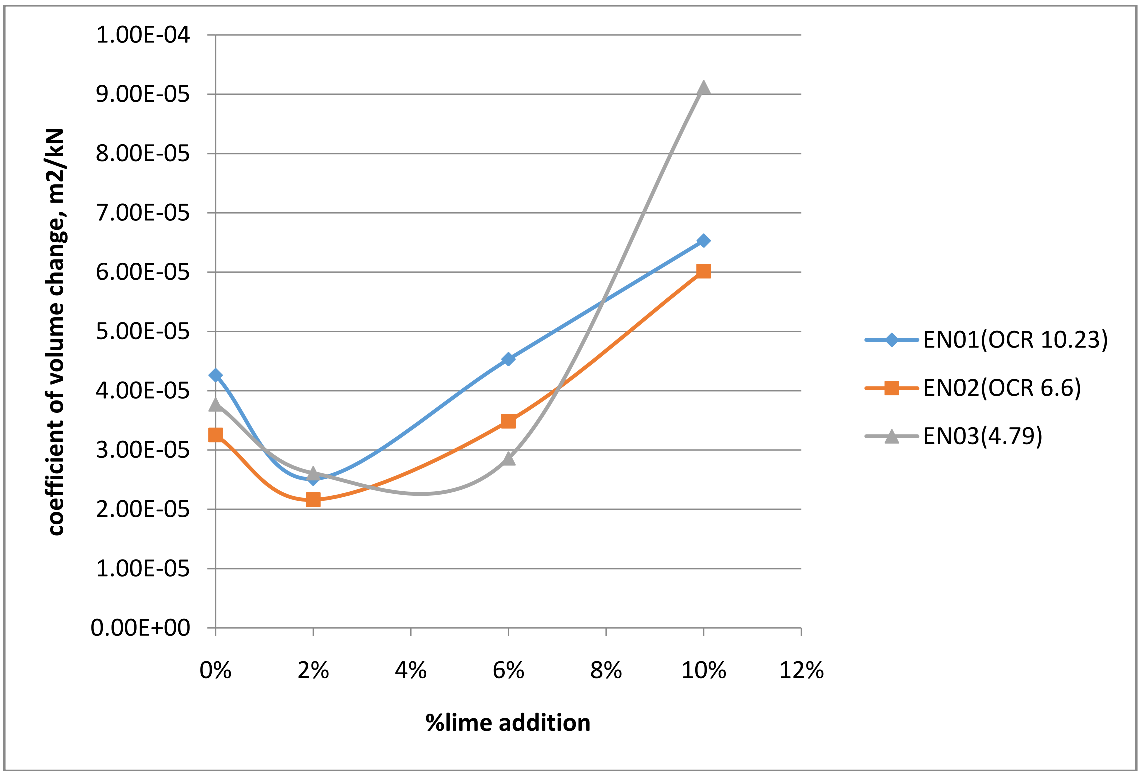

Figure 19: Graph of coefficient of volume change against lime content

From the graph above, coefficient of volume change decreases initially at 0% lime but increases from 2% lime to higher lime content. It can be deducted that coefficient of volume change increases as lime increases.

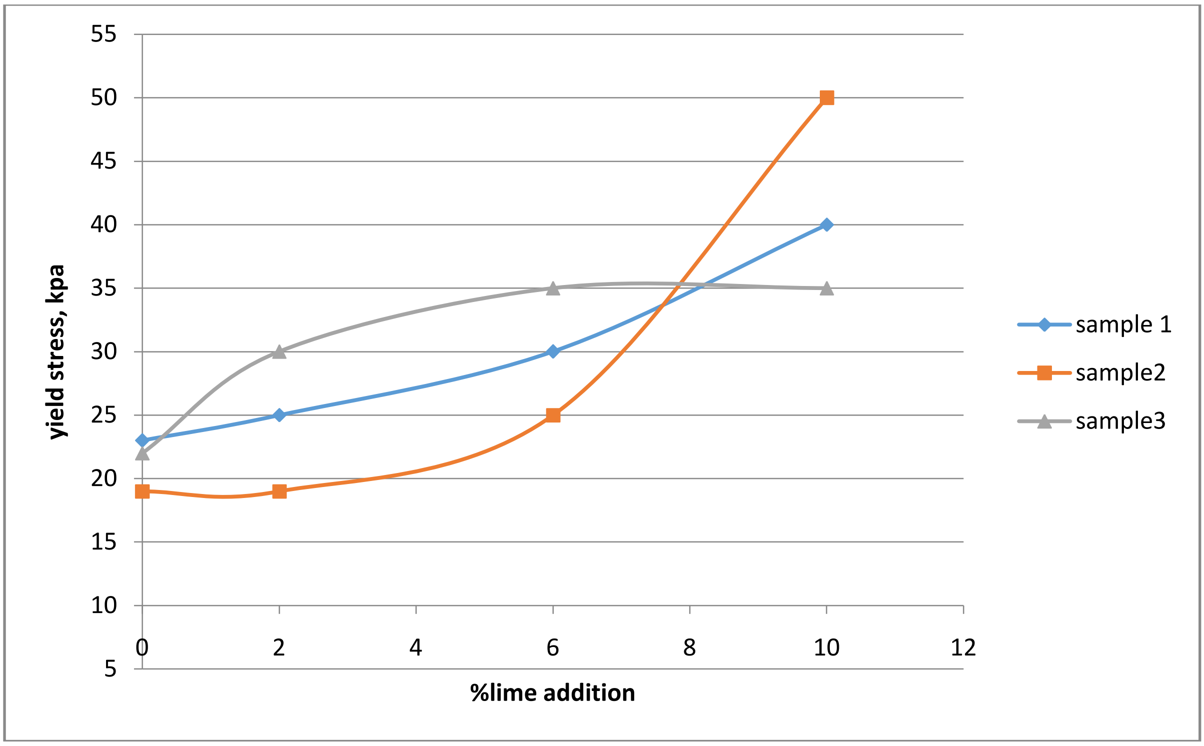

Figure 20:Graph of yield stress against lime addition

From the graph above, yield stress increases as lime content increases. It can be deducted that the gradual increases in yield stress was due to resistance of the bonding material holding the soil. The bonding material increases as lime content increases. From the result, the maximum yield stress is at 10% lime content.
Figure 20:Graph of coefficient of vertical consolidation against lime content
From the graph above, EN02 and EN03 have maximum Cv at 2% lime addition though there was no observable trend as lime % increases while EN01 show gradual increase in Cv as the lime content increases.

3.5 Statistical Analysis
3.5.1 Descriptive Analysis

Tables below represents the Descriptive analysis forSpecific gravity, cumulative percentage passing 75µm, free swell test, maximum dry density, optimum moisture content and consolidation test result of the untreated and treated lime stabilized soil.

Table 3.5a Descriptive analysis of physical properties of the soil

	Specific gravity	cumulative percentage passing BS Sieve No 200	Differential free swell	Maximum dry density from graph	Optimum moisture content from graph
Mean	2.596833	29.60667	0.306111	1966.778	10.86111
Standard Error	0.008023	1.364238	0.035993	7.117848	0.197887
Median	2.6105	29.83	0.295	1962	11

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Mode	2.619	#N/A	0.45	1953	11.5
Standard Deviation	0.034038	2.362929	0.152707	30.19847	0.839565
Sample Variance	0.001159	5.583433	0.023319	911.9477	0.704869
Kurtosis	-0.27643	#DIV/0!	-0.71251	-0.83808	0.046257
Skewness	-0.71363	-0.42152	-0.36987	0.241613	-0.90712
Range	0.123	4.71	0.49	103	2.8
Minimum	2.529	27.14	0.03	1917	9.2
Maximum	2.652	31.85	0.52	2020	12
Sum	46.743	88.82	5.51	35402	195.5
Count	18	3	18	18	18
Largest(1)	2.652	31.85	0.52	2020	12
Smallest(1)	2.529	27.14	0.03	1917	9.2
Confidence Level(95.0%)	0.016927	5.869841	0.075939	15.01735	0.417506
Coefficient of variation=(standard deviation/mean)×100%=	1.31%	7.96%	49.9%	1.54%	7.73%

Table 3.5bDescriptive statistics for the consolidation test

	Compression index Cc (m2/KN)	Average vertical coefficient of consolidation (mm2/min)	Coefficient of volume change Mv (m2/KN)	yield stress (kpa)
Mean	0.032783	394.1	4.26E-05	29.41667
Standard Error	0.00347	18.04546	5.9E-06	2.684011
Median	0.0293	389.35	3.63E-05	27.5
Mode	#N/A	#N/A	#N/A	25
Standard Deviation	0.012021	62.51132	2.04E-05	9.297686
Sample Variance	0.000145	3907.665	4.18E-10	86.44697
Kurtosis	-0.59149	0.363562	1.711878	0.669183
Skewness	0.515155	-0.19944	1.388606	0.970373
Range	0.0388	227.4	6.96E-05	31
Minimum	0.0146	268.6	2.16E-05	19
Maximum	0.0534	496	9.12E-05	50
Sum	0.3934	4729.2	0.000511	353
Count	12	12	12	12
Largest(1)	0.0534	496	9.12E-05	50
Smallest(1)	0.0146	268.6	2.16E-05	19
Confidence Level(95.0%)	0.007638	39.7178	1.3E-05	5.907468
Coefficient of variation=(standard deviation/mean)×100%=	36.7%	15.9%	47.9%	31.6%

3.5.2 Summary of descriptive analysis

According to Michael Sullivan (2007) Descriptive statistics involves collecting, organizing, summarizing and presenting of data arranged in a group. A measure of central tendency (the mean, median and mode) numerically describes the average, the middle and most occurred data value. The standard deviation (i.e. the degree or measure of disparity) depict the spread in the observed laboratory values obtained from the mean value, which give values as shown in the respective tables above, the standard error is the measure of risk in adopting the mean values . The average or mean values obtained from the descriptive statistical analysis of specific gravity, cumulative percentage passing the 75µm sieve, free swell, BSL compaction, and oedometer test, laboratory test values for the lateritic soil samples gotten from Nachi, Enugu state are shown in the statistical table shown above. The respective values of their respective confidence level at 95% in the descriptive analysis are shown on the tables above.

Table 3.5.2a two- factor without replication for Maximum dry density

ANOVA						
Source of Variation	SS	Df	MS	F	P-value	F crit
% lime addition	3776.25	3	1258.75	2.04232	0.209528	4.757063
OCR	3794.667	2	1897.333	3.078421	0.120224	5.143253
Error	3698	6	616.3333			
Total	11268.92	11				

From the table above, since the valueof F< F crit . It can be deducted that lime addition and over-consolidation ratio do not have much difference on the values of maximum dry density.

Table 3.5.2b two-factor without replication for optimum moisture content

ANOVA						
Source of Variation	SS	df	MS	F	P-value	F crit
%lime addition	7.046667	3	2.348889	8.397219	0.0144	4.757063
OCR	0.195	2	0.0975	0.34856	0.7191	5.143253
Error	1.678333	6	0.279722			
Total	8.92	11				

From the table above, since the value of F>F crit . It can be deducted that lime addition has a significant impact on the optimum moisture content of the samples while over-consolidation ratio do not have much difference on the values of optimum moisture content.

Table 3.5.2c two-factor without replication for compression index

ANOVA						
Source of Variation	SS	df	MS	F	P-value	F crit
%lime addition	0.001348	3	0.000449	11.54354	0.006642	4.757063
OCR	8.11E-06	2	4.06E-06	0.104203	0.902638	5.143253
Error	0.000234	6	3.89E-05			
Total	0.00159	11				

From the table above, since the value of F>F crit . It can be deducted that lime addition has a significant impact on the compression index of the samples while over-consolidation ratio do not have much difference on the values of compression index.

IV. RESULTS AND DISCUSSION

The compressibility characteristics of lime stabilized lateritic soil around the geographical location of Nachi, Enugu State, have been investigated in this study. Graphical analyses show that these soils are sensitive to variations in compaction optimum water content, dry unit weight as well as to natural moisture content, free swell, specific gravity, particle size, and consolidation test were properly investigated. Geotechnical test indicates that the lateritic soil found around Nachi, Enugu State is a suitable construction material.

The Particle Size Distribution analysis shows that the percentage passing BS No 200 sieve of the samples from the study area shows greater amount of sand and the specific gravity value for the lateritic soil ranges between 2.52 and 2.65. It implies that the soil can be used as sub-base material in road construction. The results of the investigation carried out shows that the soil samples are better classified as easily compactable silty-sand with good drainage. The samples belong to the A-2 group based on particle size. The soil samples show moderate compressibility on oedometer test but it was almost incompressible when treated with lime. The compressibility behavior of the lime stabilized soils decreases when lime percentage was added as compared to the undisturbed soil. This is because of the effect of bond between the soil and lime. The decrease in the strength of the bond was due to bond failure at higher load. Lime-soil stabilization is a good soil improvement method to reduce the compressibility nature of the soil that is its settlement.

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