

Predictive Regression Models for Desiccative Degradation of Stabilized Residual Sand Road Courses

¹Agbenyeku Emem-Obong Emmanuel, ¹Aneke Ikechukwu Frank

¹Department of Civil Engineering Science, University of Johannesburg, P.O.Box 524, 2006, South Africa

Abstract

Soil stabilization has gained enormous attention in recent past and efforts by researchers on harnessing industrial by-products as cement additives are investigated. In this study Berea sand (BS) is stabilized by commercially available Lime (L) and Fly ash (FA). BS refers to residual collapsible and weakly bonded sand in portions of the west coastal plain of South Africa. Succession of tests to study the strength and degradation of stabilized BS under desiccative conditions peculiar to tropical and sub-tropical regions were conducted. Regression models were developed to predict the degradation of the stabilized product. Certain indicators were used to track the degradation processes. Batches of BS mixed with 6 and 9%L stabilizers and 0, 6, 12 and 18%FA additives were compacted and hydrated for 7days at relative humidity (RH) of 95-100% and temperature (T) of 22-25°C for the purpose of this work. Hydrated specimens were immersed in distilled water for 12hours and dried at 40°C for 36hours to signify a single wet-dry cycle. 4, 6 and 10cyclic loads were used. Masses of the specimens were measured after designated cycles prior Unconfined Compressive Strength (UCS) tests. Liquid limit (LL) and California bearing ratio (CBR) tests were conducted. L-FA stabilized BS had significant gain in strength and bearing properties. The UCS decreased with increased FA with peak strength from a mix of 6%L+12%FA. For FA content higher than 12%, increased cycles caused significant decrease in UCS from idle FA not engaged in pozzolanic reaction. CBR of the stabilized mixes increased with increased L-FA content and decreased with increased cycles. The cycles caused material loss in specimens and reduction in mass decreased with increased FA content. In the long run, only the 6%L+12%FA mix for 10cycles had adequate strength under the operative drainage conditions to sustain low traffic loadings and may only be suitable for recompacted subgrades and sub-bases of low level rural road. LL was found as the best tracker to the degradation of L-FA stabilized BS as it

had the highest determinant coefficient ($R^2=98.6\%$) in comparison with other indicators.

1. Introduction

Non crystalline or poorly crystalline forms of silica found in flints, sandstones and dolomites are reactive aggregates causing Alkaline silica reaction (ASR). FA and Ground granulated blast furnace slag (GGBFS) are used to prevent ASR. Class-F and class-C FA prevent ASR when used in bulk. Merits of industrial wastes such as GGBFS, FA, and Silica fume (SF) in stabilizing soil have been fairly reported [1, 2, 3, 4]. Power stations in South Africa dump FA either dry or wet in ash dams [5]. Sequence of geotechnical and chemical tests on FA collected from one of the stations showed the presence of significant amounts of free lime, quartz, alumina and iron oxide with high likelihood of pozzolanic action [6, 7] hence, classified as class-C FA. While on the other hand, class-F FA produced by most stations has low pozzolanic potential [8].

BS as recorded by [9] is formed from underlying Calcarene leaching action. It is of a recent formation of unconsolidated red dune ridges along most of the Indian Ocean coastal plain. It is constituted of iron oxide coated quartzite sand weakly bonded by Kaolin clay. BS is collapsible due to its delicate macro structure and heterogeneous stratification. Slopes, embankments, buildings and pavements have been reported to fail from this soil hence; it is grouped as one of the problem soils of Southern Africa [10, 11] whose engineering properties may benefit from stabilization.

FA is an abundant waste generated in South Africa and has paved way for use in road bases with residual sands of Southern African semi arid environments of seasonal rainfall intensity and sunshine. Appreciable performance of base courses of collapsible Durban BS stabilized with L, Bitumen seals and Cement have been reported by [12, 13]. The

northern region of Kwazulu Natal with annual rainfall of 900mm and Weinert number of less than 2 is classified wet for road design purposes. Investigations by [12] have detailed failures of unpaved and gravelled top municipal roads and settlement of old black tops in these regions. Whereas [13] investigated various low traffic volume surfaced road carrying less than 400 vehicles per day. The road was constructed with L and Cement stabilized BS mixed with other aggregates and recorded low relative maintenance cost and higher durability. Furthermore, the advantages of stabilizing residual tropical soils with Cement, Cement and FA, Hydrated Lime (HL), L and Rice Husk Ash (RHA) have been documented. For some residual soils, it was observed that higher strength was developed by the L-RHA mixtures at all stages of the curing period than Cement-RHA mixtures. Stabilization of soils and their behaviour in harsh climatic conditions as well as their durability are the major considerations of civil engineers in recent times [14, 15]. This study however, focused on the prediction of desiccative degradation of L-FA stabilized BS of the Durban bluff in Kwazulu Natal province by certain tracking indicators i.e., LL, UCS, CBR, Cyclic loads, Residual mass and %FA replacements.

2. Experimental Details

Batches of BS were thoroughly mixed with required amount of L, FA and water. Mixtures of 6 and 9%L stabilizer and 0, 6, 12 and 18%FA additives were used for the study. Table 1 shows the chemical constitutions of the BS, L and FA, determined by X-ray diffraction analysis in conformance with the requirements of [16]. The total ($\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$) constitution of the commercially available FA is more than 70% which qualifies it an active pozzolana [8]. Given the low content of pozzolanas in the L, the addition of FA is crucial to the BS-L pozzolanic reaction. Mechanical analysis was conducted on the BS and it was found to be uniformly graded. The grain size curve in accordance with [16] is shown in Figure 1.

The plasticity index of the BS is within 6-8%. Table 2 shows the physical, index and compaction properties of the soil. The thoroughly mixed samples were left to equilibrate for 24hours before compaction in order to properly blend. Maximum Dry Density (MDD) and Optimum Water Content (OMC) for each mix were determined according to the standard AASTHO compaction method [16]. Compacted specimens were cured for 7days at RH of 95-100% and T of 22-25°C in

a curing chamber before compression test was carried out.

Table 1: Constituent of BS, L and FA

	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	MnO	MgO	CaO
BS	62.7	19.5	13.6	0.11	0.30	0.83
L	0.19	0.27	1.13	1.65	93.9	0.00
FA	83.0	8.00	2.65	0.55	0.22	0.18
	Na ₂ O	K ₂ O	TiO ₂	P ₂ O ₅	Cr ₂ O ₃	NiO
BS	0.32	0.85	1.7223	0.013	0.0615	0.0061
L	0.06	0.06	0.0021	0.020	0.0015	0.0000
FA	0.04	0.00	0.2600	0.400	0.0050	0.0000

Permeable hessian bags were used to cover the samples and water was constantly sprinkled on the cover over the seven day period in accordance with the specifications in [16]. The specimens hydrated for 7days were completely immersed in distilled water for 12hours then dried at 40°C for another 36hours; these processes completed an entire single wet-dry cycle.

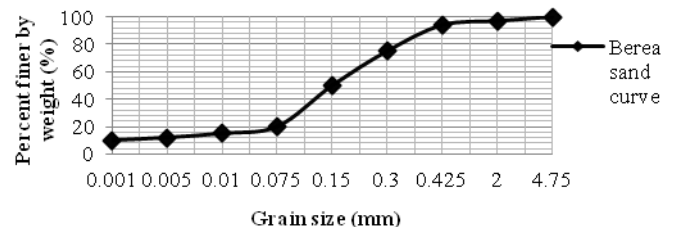


Figure 1: Particle size curve of BS

The selected numbers of cycles initiated in this study were; 4, 6 and 10cycles of wetting and drying. After the designated cycles of wetting and drying, the mass of the specimens were measured before the UCS tests. The LL and CBR tests were also conducted. A 48g of the thoroughly mixed soil fines in accordance with the prescribed standard method was weighed and placed in a porcelain dish in consonance with [16]. Distilled water was added, the material was stirred properly and then transferred to the casagrande device which operated at a speed resulting in two taps per/sec applied to the soil until about 10mm contact was made between the two soil portions. CBR tests were conducted on soils stabilized with combinations of 6 and 9%L and 12%FA only. The samples were compacted at the respective OMC to 95% of the

associated Modified AASTHO MDD, soaked for 7 days and then subjected to respective wet-dry cycles.

Table 2: Physical, index and compaction properties of BS

Specific Gravity	Atterberg Limit	ASSTHO Compaction
$D_{2.0} = 2.68$	PI = 6 - 8%	MDD = 16.8Mg/m ³
$D_{0.075} = 2.71$	SL = 2%	OMC = 10.20%

For the implementation of low-cost and low traffic road projects in developing countries, CBR of naturally occurring materials is the most commonly used design parameter. It remains a simple and direct parameter for the design of roads and can also be correlated with other more complex mechanical pavement parameters. In conjunction with the UCS and other basic physical properties, it has formed the basis of the Catalogue method of Design specified in [17]. The CBR values were determined for all wet-dry cycles of both the 6% and 9%L with 12%FA stabilized soil mixes in accordance with [16]. The soil mixes were compacted to 95% modified AASTHO MDD and OMC and tested after the 7 days curing. This test was carried out on every compacted sample after been soaked for 7 days.

3. Discussion of Result

3.1. Effect of Curing and Wet-Dry Cycles on UCS

In the construction of simple and low cost structures i.e., artificial embankments, small dams, levees, stabilized and earth retaining walls, the UCS test remains the standard and most common strength test used for estimating material strength. The results of specimens cured for 7 days and subjected to cyclic wetting and drying, together with their respective determinant coefficients (R^2) are shown in Figures 2 and 3 for the 6% and 9%L stabilizer respectively. Figures 2 and 3 also show the effects of FA additives on the strength of specimens subjected to 4, 6 and 10 cycles. The general trends show an increment in strength development with the addition of FA until 12%FA is reached for the 6%L mix, above which the strength begins to drop. Generally, the addition of FA from 0–18% yielded a high R^2 value, with the 12%FA showing a strong correlation between UCS and wet-dry cycles to be as high as 81.5% for the 6%L stabilizer. In the case of the 9%L mix, there is a general increase in the R^2

values with the increase in FA up to 18% although; the 12%FA indicated a low R^2 value of 0.06 between UCS and wet-dry cycles.

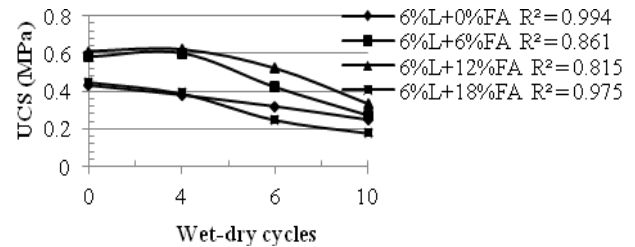


Figure 2: Effect of wet-dry cycles on UCS for 6%L stabilized BS

The UCS results show that improvement in strength in the respective L stabilized soil can be enhanced by adding FA. The characteristic strength of the soil is improved by L stabilization where as the gain in strength comes from the pozzolanic reaction between amorphous silica and or alumina from the soil and L which forms various types of binding agents. Hence, additional amounts of amorphous silica are made available to react with L when FA is added resulting in further increase in strength.

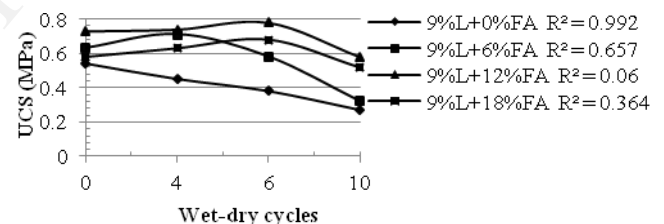


Figure 3: Effect of wet-dry cycles on UCS for 9%L stabilized BS

As recorded by [14] the decrease in strength after the optimum is reached as shown in Figures 2 and 3, occurred due to the introduction of excess amount of FA having relatively lower specific gravity to react with the available L. It is observed that the 9%L mix showed improved strength until the 10 wet-dry cycles, above which a reduced strength for consequent cycles occurred. In the case of the 6%L mix, an increase in strength is also observed and a subsequent reduction right before and at the 10 cycles. Consequently, the number of cycles at which the peak is reached depends on the percentage L in the mix. For the 9%L, the peak is reached at the 6 cycles. This may be due to the development of a significant percentage of maximum strength during the curing stage due to the availability of sufficient quantity of L for the completion of the pozzolanic reaction, while the 6%L stabilizer had a

slight reduction in strength at the 6cycles up to the 10cycles. This reduction may be due to the deterioration caused by the cyclic wet-dry impact.

3.2. Effect of FA additives on UCS for Various Cyclic Loads

The UCS results in Figures 2 and 3 are further illustrated in Figures 4 to 6; showing that improvement in strength in the respective L stabilized soil can be enhanced by the addition of FA. It is seen in Figures 4 and 5 that the general trend is a notable rise in strength development with the addition of FA until an optimum of 12% for both the 6% and 9%L mixes, above which the strength begins to drop.

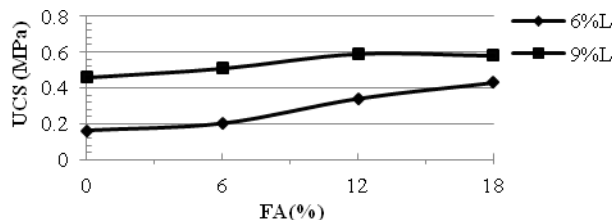


Figure 4: Effect of FA mixes on UCS for 4cyclic loads

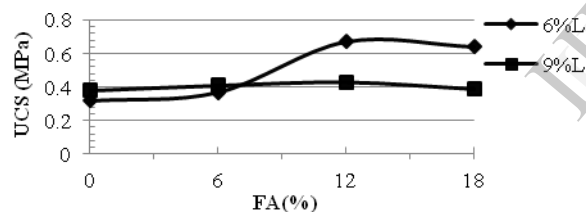


Figure 5: Effect of FA mixes on UCS for 6cyclic loads

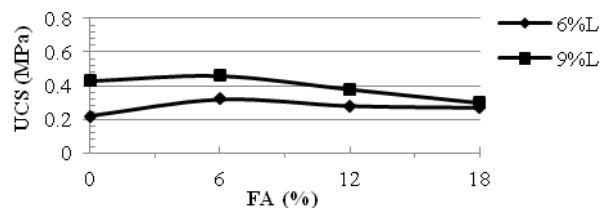


Figure 6: Effect of FA mixes on UCS for 10cyclic loads

Therefore, the changes in strength recorded in Figures 2 and 3 came from the additional amounts of amorphous silica made available to react with L when FA is added which accounts for a further increase in strength. While reduction in strength after the optimum/peak is reached, is as a result of the addition of excess amount of FA

having relatively low specific gravity to react with the available L as noted by [14].

3.3. Effect of Wet-Dry Cycles on UCS with respect to Mass and LL for 6% and 9%L-FA Stabilized Specimens

From Figure 7 it can be drawn that for the 6%L with corresponding FA additives, the number of cyclic loads affects the respective masses which eventually affects the UCS of the specimens. Hence it reveals that the mass of the specimens after the respective wet-dry cycles led to an increase in the UCS.

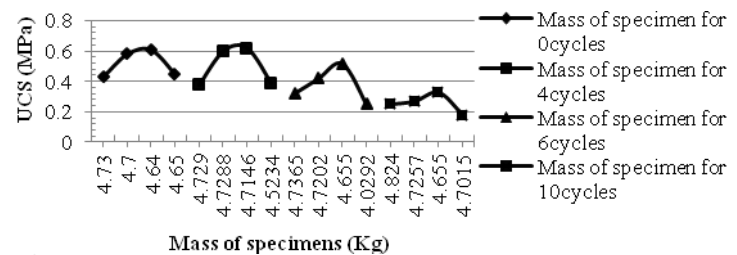


Figure 7: Effect of wet-dry cycles on UCS with respect to masses for 6%L stabilized specimens

Here the 4wet-dry cycles revealed the highest UCS of 0.62MPa for a corresponding mass of 4.7146Kg having the 12%FA. The 9%L with corresponding FA mixes in Figure 8 shows a gradual reduction in the UCS and a corresponding loss in the masses after every cycle. The highest strength value of 0.78MPa for a mass of 4.72Kg was reached at the 6 cycles with 12%FA content. Beyond 12%FA and 10cycles, a general reduction in the respective masses and UCS is recorded. The loss in mass and strength observed may be accounted for by the induced deterioration and material loss caused by the cyclic loading.

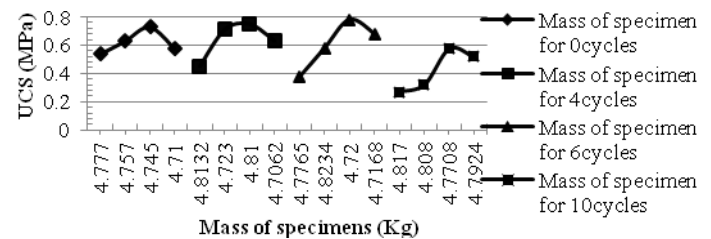


Figure 8: Effect of wet-dry cycles on UCS with respect to masses for 9%L stabilized specimens

There is a general increase in UCS with decreasing LL for all cycles of the 6% and 9%L mixes as shown in

Figures 9 and 10. It remains the case until it reaches the peak at 12%FA after which the strength begins to decline. The reduction in strength after the optimum is reached as stated by [14] can be as a result of the introduction of excessive amount of FA having relatively lower specific gravity needed to react with the available L.

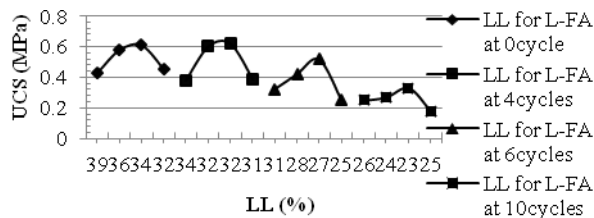


Figure 9: LL-UCS relationship with respect to masses for 6%L stabilized specimens

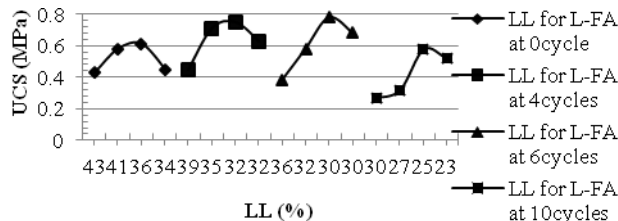


Figure 10: LL-UCS relationship with respect to masses for 6%L stabilized specimens

There is a general increase in UCS with decreasing LL for all cycles of the 6% and 9%L mixes as shown in Figures 9 and 10. It remains the case until it reaches the peak at 12%FA after which the strength begins to decline. The reduction in strength after the optimum is reached as stated by [14] can be as a result of the introduction of excessive amount of FA having relatively lower specific gravity needed to react with the available L.

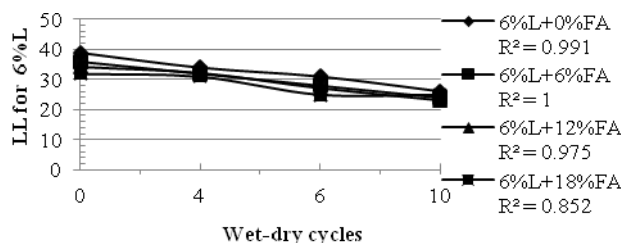


Figure 11: Wet-dry cycles-LL relationship with respect to masses for 6%L stabilized specimens

In terms of UCS in Figures 11 and 12 shows a decrease in the LL with subsequent increase in the wet-dry cycles for both the 6% and 9%L stabilizers

respectively. The trends reveal a situation such that there is a slight reduction in the R^2 value from 0%FA-18%FA with the 6%L+12%FA having a coefficient of determination (R^2) as high as 97.5% while the 9%L+12%FA had a 97.6% R^2 value.

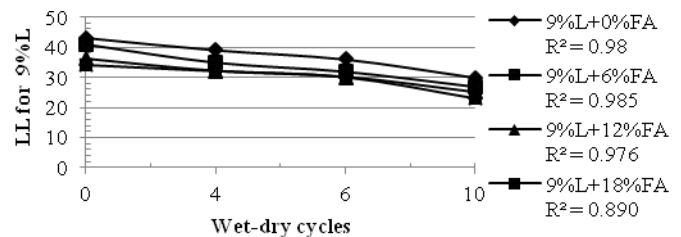


Figure 12: Wet-dry cycles-LL relationship with respect to masses for 9%L stabilized specimens

Hence, it can be said that the LL with relation to wet-dry cycles reduces with increase in L content.

3.4. CBR test after Cyclic Loads on L-FA Mixes in relation to Penetration and LL

From Figures 13 and 14 the relationship established for CBR, penetration depth from all wet-dry cycles, 6% and 9%L-12%FA mixes shows that the CBR increases for an increasing depth of piston penetration for all cycles with an exception of the 4cycles shown in Figure 13; which shows a clear decrease in CBR with increase in piston penetration.

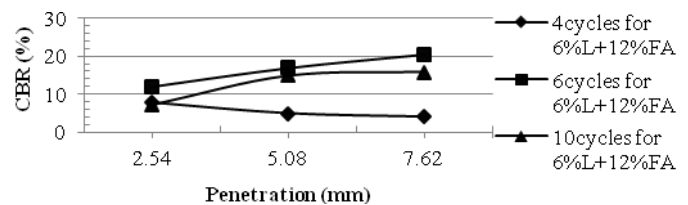


Figure 13: Effect of 6%L+12%FA mixes with respect to cyclic loads on CBR

As recorded in [18] the CBR number is used to rate the performance of soils primarily for use as bases and sub grades beneath airfields and road pavement. it is a measure of shearing resistance of a soil that is laterally confined, under controlled moisture and density conditions.

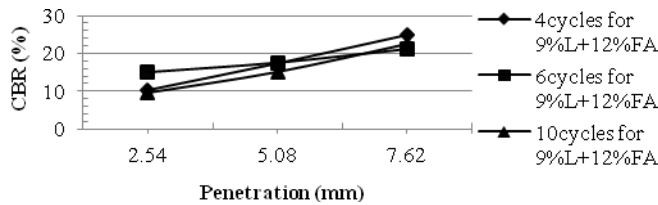


Figure 14: Effect of 9%L+12%FA mixes with respect to cyclic loads on CBR

The CBR test is relatively simple and cheap and can be run on standard loading frames, it is therefore widely used for low cost road projects.

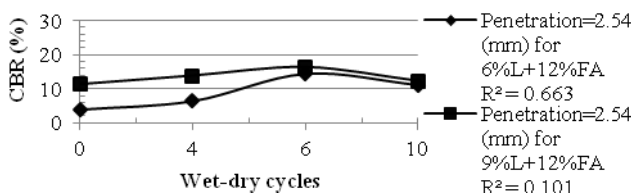


Figure 15: Effect of cyclic loads on CBR for 6% and 9%L-12%FA stabilized specimen

The result of the CBR tests shown in Figures 13 and 14 reveals a pattern of decreasing CBR with decrease in piston penetration and increasing CBR with increase in piston penetration. The considerable reduction in CBR in Figure 13 exhibited by samples mixed with 6%L and 12%FA for the 4cycles is a manifestation of non uniform bulk density resulting from the cyclic loads. Also from Figure 15 the CBR increases from 4cycles to 6cycles and then decreases at 10cycles. The 9%L is seen to have R^2 value of 10.1% while the 6%L had 66.3% direct relationship between the CBR and cyclic loads. It also shows that the average CBR of the 9%L mixes are higher than the CBR of the 6%L mixes. This tendency is true for all the samples regardless of the L and FA mixes. This indicates that the strength development due to the pozzolanic reaction between amorphous silica and or alumina from the soil and L to form various types of binding agents is reliant on the quantity of FA used up in the reaction. In the case of the 9%L mixes, 12%FA supplies adequate additional silica for the reaction while the amount of FA required for optimum reaction in the 6%L mix is less than 12% as such, the excess FA led to the strength reduction.

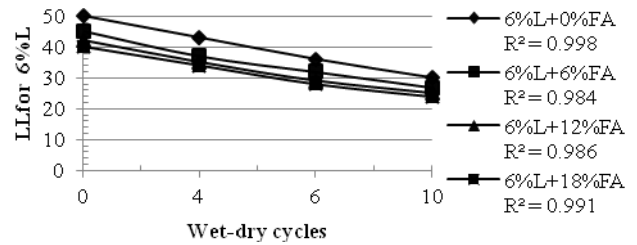


Figure 16: Wet-dry cycles-LL relationship for 6%L stabilized specimens

It can also be deduced from Figures 2 to 5 that the lower percentage of FA i.e. 12% produced stronger and more durable samples than 18%FA irrespective of the quantity of L in the mix. The strength reduction in mixes containing higher amount of FA is also significant. In the long run, defined by 10cyclic impacts, the difference in strength between the two L mixes is also significant. For the 6%L and 12%FA mixes, 4cyclic loads induced similar effect as curing i.e., increased strength development. The results shown in Figures 2 and 3 indicate that the strength of mixes at 10wet-der cycles are very low and can only be used as sub-grades in pavement. However, only the 9%L and 12%FA material have adequate shear strength under the operative drainage conditions to sustain the stresses applied by traffic loadings [17]. A decrease in the LL with increase in the cyclic loads for 6% and 9%L stabilizers are shown in Figure 16 and 17 respectively. The R^2 values are seen to be generally high for both.

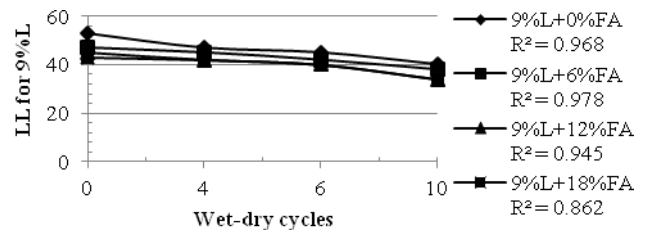


Figure 17: Wet-dry cycles-LL relationship for 9%L stabilized specimens

In the 6%L a slight decrease in the R^2 value was exhibited by the 0%FA to 6%FA and a subsequent increase is observed for the 12%FA and 18%FA content. A similar trend is also established by the 9%L. As such, a reduction in correlation between the LL and cyclic loads was noticed with the increase in L from 6% to 9%. For the 6%L+12%FA a 98.6% direct relationship was gotten between the LL and the cyclic loads, while a 94.5% direct relationship was gotten for 9%L+12FA.

3.5. Water Absorption upon Wet- Dry Cycles and Residual Mass

After every cycle the samples were weighed. Due to the absorption of water and simultaneous loss of materials, the mass of the samples at the end of each cycle changes in relation to specimen constitution, curing programme and number of wet-dry cycles. As shown in Figure 18, there is a general reduction in the mass of the specimens after the wet-dry cycles. Therefore it reveals that the specimen with 6wet-dry cycles caused the highest lose in mass for 6%L+18%FA mix, while the 9%L+18%FA mix suffered the highest mass loss after the 4 and 6cycles as shown in Figure 19.

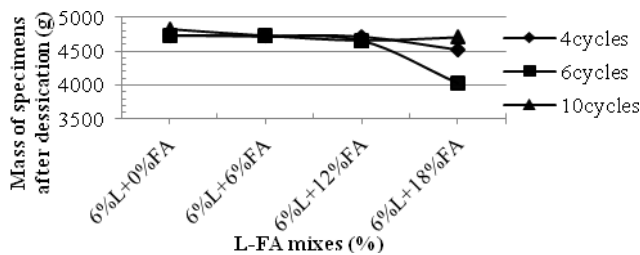


Figure 18: Residual mass-6%L-FA relationship from wet-dry cycles on stabilized specimens

Hence, the general trend of decreasing specimen stability with increasing number of cycles accounts for the loss in mass. The improved sample stability experienced by the specimens with high percentage of FA as shown in Figures 18 and 19 is associated with the changes in the water absorption potentials of the mixes. The samples mixed with 18%FA shows the least amount of water absorption. This trend follows for both 6% and 9%L mixes. It was observed that the moisture absorption at 6%FA was lesser than 0%FA but the strength of 6%FA was higher than 0%FA thus, an increase in strength with decreasing moisture absorption.

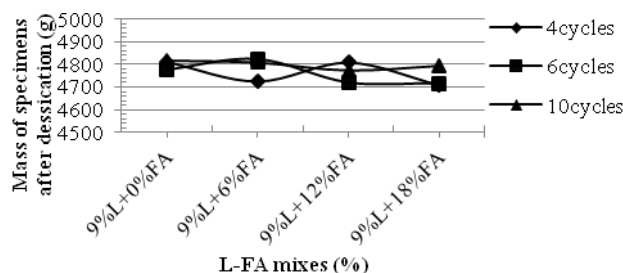


Figure 19: Residual mass-9%L-FA relationship from wet-dry cycles on stabilized specimens

However the trend is not linear as it was also observed that some samples with increased FA content

also absorbed more water than samples without FA. This is reflected in Figures 2 to 4 where the UCS results showed that the strength increases up until 12%FA, after which the strength reduces. However, the 7days curing for mixes containing 6%L did not have a very strong correlation between strength and loss of mass since negligible material loss was experienced.

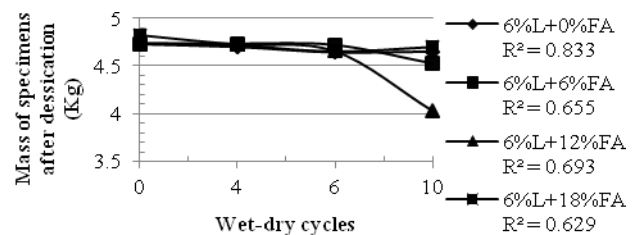


Figure 20: Residual mass-wet-dry cycle relationship from 6%L-FA stabilized specimens

Figure 20 shows a general reduction in the mass of specimens of the various FA mixes just after the 4cycles but with an exception of the 0%FA and 18%FA, which shows slight increment just at the 10cycles. This can be accounted for as a result of water absorption. In the case of the 18%FA, this could have occurred as a result of material loss and a subsequent absorption of moisture. The mass of the 9%L, 12% and 18%FA mixes in Figure 21 show increase at the 4cycles which can be likened to the fact that there was absorption of water and the subsequent reduction in mass of 6%FA; which was due to the increased specimen instability as a result of the increased cyclic loads thereby, leading to a loss in mass.

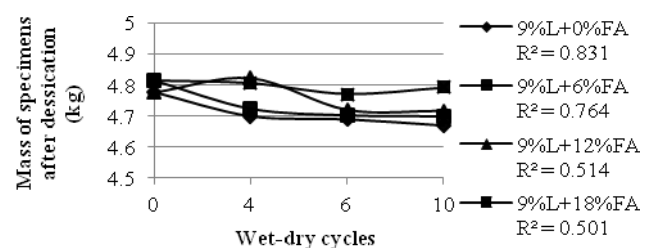


Figure 21: Residual mass-wet-dry cycle relationship from 9%L-FA stabilized specimens

It can be seen that the direct relationship between the mass of the specimens and wet-dry cycles in Figure 20 reduces with the increase in the FA content. As such, the highest R² values were gotten to be 83.3% and 69.3% for the 6%L, 0% and 12%FA mixes respectively. On the other hand, Figure 21 shows that the 9%L exhibits a similar trend. Hence, the highest R²

values were gotten to be 83.1% and 51.4% for the 0% and 12%FA mixes respectively.

In light of these outcomes, it can be drawn that the LL is the best pointer to the resilience degradation pattern of the L-FA stabilized BS specimens. This is simply due to the fact that it had a stronger correlation as compared to the other variables and as such, possessed the highest coefficient of determination, $R^2 = 98.6\%$.

4. Predictive Regression Models

4.1. Relationship between Variables of 6% and 9%L Mixes for UCS and CBR

Statistical models were generated and tested to show the relationships between all the variables for both the 6% and 9% L mixes with respect to UCS and CBR;

Equation (1) shows the model for testing UCS for 6%L mix;

$$[UCS = 0.105x_1 + 0.000033x_2 + 0.55x_3 + 0.024x_4] \quad (1)$$

A value of **0.60MPa** from test data was tabulated for UCS of 6%L and a calculated value of **0.626MPa** from the model was arrived at when tested with the following variables;

$x_1 = 4.7288$ (Kg), $x_2 = 32$, $x_3 = 6\% = 0.06$, $x_4 = 4$, using the generated equation;

$$[UCS = 0.105(4.7288) + 0.000033(32) + 0.55(0.06) + 0.024(4)]$$

$$UCS = 0.626MPa$$

Equation (2) shows the model for testing UCS for 9%L mix;

$$[UCS = 0.070x_1 + 0.00007x_2 + 0.75x_3 + 0.0896x_4] \quad (2)$$

A value of **0.71MPa** from test data was tabulated for UCS of 9%L and a calculated value of **0.736MPa** from the model was arrived at when tested with the following variables;

$x_1 = 4.7230$ (Kg), $x_2 = 35$, $x_3 = 6\% = 0.06$, $x_4 = 4$, using the generated equation;

$$[UCS = 0.070(4.7230) + 0.00007(35) + 0.75(0.06) + 0.0896(4)]$$

$$UCS = 0.736MPa$$

Equation (3) shows the model for testing CBR for 6%L mix;

$$[CBR = 0.0075x_1 + 0.0000085x_2 + 0.087x_3 + 0.0045x_4] \times 100 \quad (3)$$

A value of **6.405%** from test data was tabulated for CBR of 6%L and a calculated value of **6.409%** from the model was arrived at when tested with the following variables;

$x_1 = 4.7146$ (Kg), $x_2 = 35$, $x_3 = 12\% = 0.12$, $x_4 = 4$, using the generated equation;

$$[CBR = 0.0075(4.7146) + 0.0000085(35) + 0.087(0.12) + 0.0045(4)]$$

$$CBR = 0.06409 \times 100 = 6.409\%$$

Equation (4) shows the model for testing CBR for 9%L mix;

$$[CBR = 0.017x_1 + 0.0000026x_2 + 0.16x_3 + 0.0099x_4] \times 100 \quad (4)$$

A value of **13.82%** from test data was tabulated for CBR of 9%L and a calculated value of **14.07%** from the model was arrived at when tested with the following variables;

$x_1 = 4.81$ (Kg), $x_2 = 42$, $x_3 = 12\% = 0.12$, $x_4 = 4$, using the generated equation;

$$[CBR = 0.017(4.81) + 0.0000026(42) + 0.16(0.12) + 0.0099(4)]$$

$$CBR = 0.1407 \times 100 = 14.07\%$$

4.2. Optional Models

Equations (5) and (6) are optional predictive models to determine the CBR values of specimens in cases where the masses of the specimen are not known.

Equation (5) shows the model for testing CBR for 6%L mix;

$$[CBR = 0.00002125x_2 + 0.2088x_3 + 0.0099x_4] \times 100 \quad (5)$$

A value of **6.405%** from test data was tabulated for CBR of 6%L and a calculated value of **6.539%** from the model was arrived at when tested with the following variables;

$x_2 = 35$, $x_3 = 12\% = 0.12$, $x_4 = 4$, using the generated equation;

$$[CBR = 0.00002125(35) + 0.2088(0.12) + 0.0099(4)]$$

$$CBR = 0.06539 \times 100 = 6.539\%$$

Equation (6) shows the model for testing CBR for 9%L mix;

$$[CBR = 0.00000676x_2 + 0.348x_3 + 0.02376x_4] \times 100 \quad (6)$$

A value of **13.82%** from test data was tabulated for CBR of 9%L and a calculated value of **14.14%** from the model was arrived at when tested with the following variables;

$x_2 = 42$, $x_3 = 12\% = 0.12$, $x_4 = 4$, using the generated equation;

$$[CBR = 0.00000676(42) + 0.348(0.12) + 0.02376(4)]$$

$$CBR = 0.14140 \times 100 = 14.14\%$$

These predictive models can be referred to as the Degradation Equations.

The coefficients of determination (R^2) for the respective equations are given below;

$$[UCS \text{ (MPa) for 6\% L} \text{ ---- } R^2 = 0.7594],$$

$$[UCS \text{ (MPa) for 9\% L} \text{ ---- } R^2 = 0.657],$$

$$[CBR \text{ (\%)} \text{ for 6\% L} \text{ ---- } R^2 = 0.8563],$$

$$[CBR \text{ (\%)} \text{ for 9\% L} \text{ ---- } R^2 = 0.7998],$$

$$[\text{Optional model-CBR (\%)} \text{ for 6\% L} \text{ ---- } R^2 = 0.7896],$$

$$[\text{Optional model-CBR (\%)} \text{ for 9\% L} \text{ ---- } R^2 = 0.7969],$$

Where; $x_1 = \text{Mass (Kg)}$

$x_2 = \text{Liquid Limit (LL)}$

$x_3 = \text{Percentage Fly Ash (\%)}$

$x_4 = \text{Number of wet-dry cycles}$

Conclusion

A number of tests to determine the strength properties of stabilized BS were conducted. Influence of the different percentages of L-FA stabilizers and the desiccative effect on the strength behaviour of the stabilized product were investigated. A regression model was generated to predict the degradation of the stabilized product caused by desiccative actions and with certain indicators the degradation was monitored over the test duration. From result and analysis, the following conclusions were reached;

The predicted values from the tested Models were fairly in line with laboratory test data as such can be used to predict degradation patterns.

The UCS (MPa) increased after 4 cycles and decreased as the wet-dry cycles increased.

The UCS (MPa) decreases with increase in FA with a peak strength from a mix of 6%L+12%FA stabilizer. For FA content higher than 12%, increased wet-dry cycles results in significant decrease in UCS (MPa) resulting from idle FA not reacting.

Wet-dry cycles results in material loss in specimens; the reduction in mass decreases with increase in FA content.

The CBR of the stabilized mix increases with increase in L-FA content and decreases with increasing cycles of wetting and drying.

The significant reduction in piston penetration for samples mixed with 6%L+12%FA is a reflection of non uniform bulk density resulting from the cycles of wetting and drying.

In the long term, defined by 10 cyclic loads only the 6%L+12%FA material mix had adequate strength under the operative drainage conditions to sustain traffic loadings.

A strong coefficient of determination (R^2) was shown for the LL thereby, showing high percentage correlation between the variables concerned.

The LL is the best indicator for leaching effect on Lime-FA stabilized Berea soil.

Appendix

TABULATION OF VALUES

CBR with respect to Wet-Dry Cycles				
FA (%)	6%L Mix			
	0cycle	4cycles	6cycles	10cycles
	CBR (%)			
12	3.899	6.405	14.34	11.09
	9%L Mix			
12	11.314	13.82	16.3	12.28
Mass of specimens with respect to Wet-Dry Cycles				
	6%L Mix			
	0cycle	4cycles	6cycles	10cycles
	Mass (Kg)			
0	4.777	4.729	4.7365	4.824
6	4.770	4.7288	4.7202	4.7257
12	4.690	4.7146	4.655	4.655
18	4.67	4.5234	4.0292	4.7015
	9%L Mix			
0	4.73	4.8132	4.7765	4.817
6	4.70	4.723	4.8234	4.808
12	4.745	4.81	4.72	4.7708
18	4.71	4.7062	4.7168	4.7924

LL with respect to Wet-Dry Cycles for UCS				
FA (%)	6%L Mix			
	0cycle	4cycles	6cycles	10cycles
	LL			
0	39	34	31	26
6	36	32	28	24
12	34	32	27	23
18	32	31	25	25
	9%L Mix			
0	43	39	36	30
6	41	35	32	27
12	36	32	30	25
18	34	32	30	23
LL with respect to Wet-Dry Cycles for CBR				
	6%L Mix			
	0cycle	4cycles	6cycles	10cycles
	LL			
0	50	43	36	30
6	45	37	32	27
12	42	35	29	25
18	40	34	28	24
	9%L Mix			
0	53	47	45	40
6	47	45	42	38
12	45	42	40	34
18	43	42	40	34

UCS with respect to Wet-Dry Cycles				
FA (%)	6%L Mix			
	0cycle	4cycles	6cycles	10cycles
UCS (MPa)				
0	0.43	0.38	0.32	0.25
6	0.58	0.60	0.42	0.27
12	0.61	0.62	0.52	0.33
18	0.45	0.39	0.25	0.18
9%L Mix				
0	0.54	0.45	0.38	0.27
6	0.63	0.71	0.58	0.32
12	0.73	0.75	0.78	0.58
18	0.58	0.63	0.68	0.52

Acknowledgement

The Authors appreciate the University of Johannesburg where this investigation was successfully carried out.

References

- [1] Sezer A., Inan G., Yilmaz H.R. and Ramyar K. 2006. Utilization of a very high lime fly ash for improvement of Izmir clay. *Building and Environment*, 41: 150-155.
- [2] Senol A., Edil T.C., Md. Sazzad Bin Shafique, Hector A.A. and Benson C.H. 2006. Soft Subgrades Stabilization by Using Various Fly Ashes. *Resources, Conservation and Recycling*, 46 (4); 365- 376.
- [3] Kalias S., Kasselouri-Rigopoulou V. and Karahalios A. 2005. Stabilization of clayey soils with high calcium fly ash and cement. *Cement and Concrete Composites*, 27: 301-313.
- [4] Basha E.A., Hashim R., Mahmud H.B. and Muntohar A.S. 2005. Stabilization of residual soil with rice husk ash and cement. *Construction and Building Materials*, 19: 448-453.
- [5] Fourie A. and Blight G. 1999. Erosion-resistant crusting of slopes of fly ash dams. *Proceedings 6th International Conference on Tailings and Mine Waste '99*. Fort Collins, U.S.A. pp. 189-195.
- [6] Fourie A., Blight G., Bhana Y., Harris R. and Barnard N. 1997. The geotechnical properties of dry dumped and hydraulically placed power station fly ash. *Proc. 2nd International Conference on Mining and Industrial Waste Management*. Midrand, South Africa. pp. 10-17.
- [7] Fourie A., Bhana Y. and Blight G. 1998. The contributions of matric suction to the stability of ash dump. *Proceedings 2nd International Conference on Unsaturated Soils*, Beijing. pp. 225-230.
- [8] American Standard for Testing Materials. 1978. Specification for fly ash and Raw or Calcium Natural Pozzolana for use as a mineral admixture in Portland Cement Concrete. ASTM C. pp. 618-678.
- [9] Brink A.B.A. 1984. *Engineering Geology of Southern Africa*. Building Publications. Silverton. Vol. 4.
- [10] Jennings J.E. and Knight K. 1975. A guide to construction on or with materials exhibiting additional settlement due to collapse of grain structure. *Proceeding of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering.*, Durban, Vol1, pp.99-105.
- [11] McKnight C.L. 1999. The stratigraphy and engineering geological characteristics of collapsible residual soil on the Southern Mozambique coastal plain. *Proceedings of the 14th African Reg. CSMFE*, Durban, Vol 1, pp. 633-646.
- [12] Paige-Green P. and Gerrys E. 1998. Innovative solution for township roads in Sandy areas. *The Civil Engineering and Building Contractor*. pp. 74 -76.
- [13] Bennett H.E., Ducasse K., Payne G.A. and Sewlal S. 2002. Innovatives in Roads Design and Construction in the Province of Kwazulu Natal, South Africa. *21st Annual South African Transport Conference South Africa*. pp. 21-41.
- [14] Ali F.H., Adnan A. and Chew K.C. 1992. Geotechnical properties of a stabilized chemically stabilized soil from Malaysia with Rice Hush ash as an additive. *Geotechnical and Geological Engineering*. 10:117-134.
- [15] Bagherpour I. and Choobbasti A.J. 2003. Stabilization of fine grained soils by adding microsilica and lime or microsilica and cement. *Electronic Journal of Geotechnical Engineering*, Vol. 8.
- [16] Standard Methods of Testing Road Construction Material. 1996. *Technical Methods for Highways*. TMH1. Vol. 1.
- [17] Structural Design of Flexible Pavements for Interurban and Rural Roads. 1996. Committee of Land Transport Officials, COLTO. TRH4.
- [18] Guide to pavement evaluation and maintenance of bitumen-surfaced roads in tropical and sub-tropical countries. 1988. Department for International Development. Transport Research Laboratory. TRRL. Note 18.