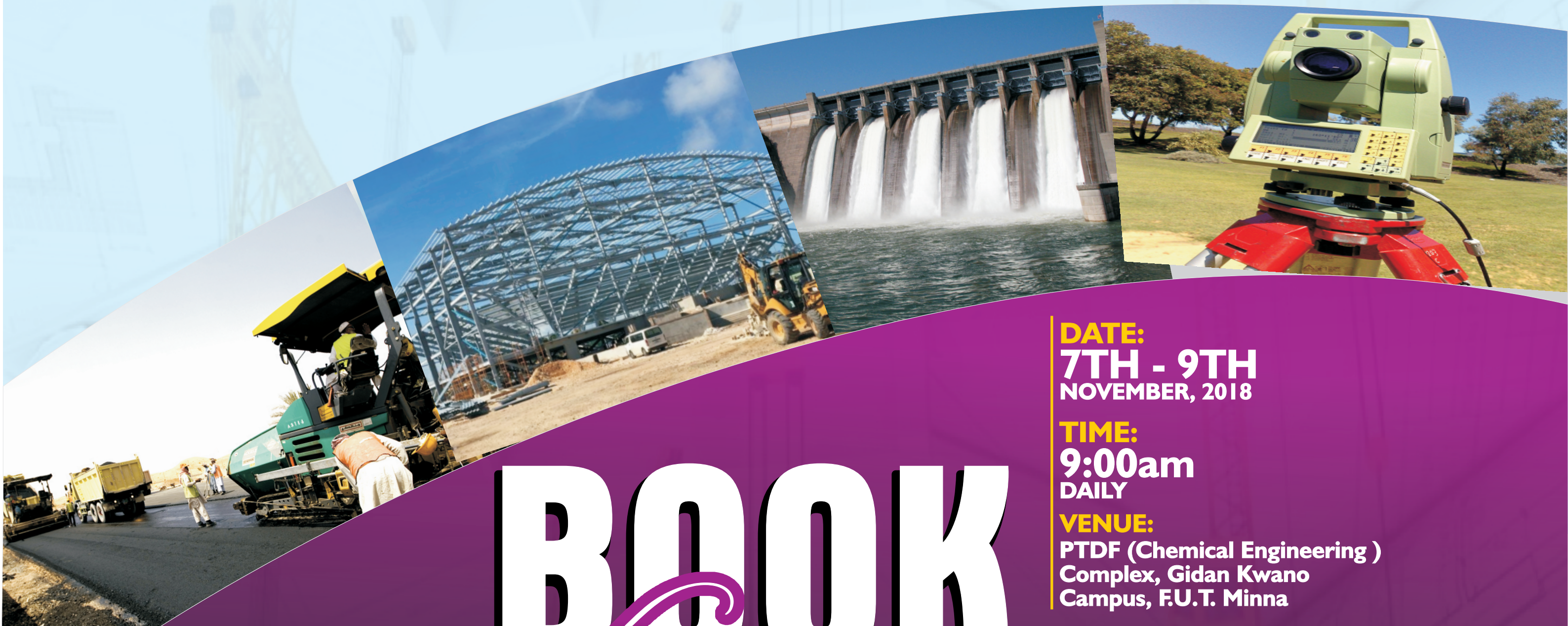
DEPARTMENT OF
CIVIL ENGINEERING
FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA

1st INTERNATIONAL CIVIL ENGINEERING CONFERENCE (ICEC)

Theme:

INFRASTRUCTURE DEVELOPMENT IN THE CONTEXT OF CONTEMPORARY ECONOMIC CHALLENGES



DATE:
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NOVEMBER, 2018

TIME:
9:00am
DAILY

VENUE:
PTDF (Chemical Engineering)
Complex, Gidan Kwano
Campus, F.U.T. Minna

BOOK of PROCEEDINGS

EDITED BY:

- Engr. Prof. A.A. Amadi
- Engr. Dr. T.W.E. Adejumo
- Engr. Dr. A.R. Adesiji
- Engr. M. Abubakar
- Engr. H.O. Aminulai
- Engr. A. Yusuf

FOREWORD

The First International Civil Engineering Conference (ICEC) being held in the Main Campus of the Federal University of Technology, Minna, Nigeria on 7th – 9th of November, 2018 derives from the necessity to provide a suitable forum for the interaction of a wide spectrum of stakeholders including the academia and practitioners in the industry for the purpose of advancing the frontiers of knowledge in the Civil Engineering profession and allied sciences and technology. It is sheer truism to state that economic vicissitudes are rapidly becoming the major determinant in the rate and size of infrastructure development in any modern nation. By implication, therefore, the practice of civil engineering is subject to the vagaries of the nation's economy with particular reference to Nigeria. Accordingly, the Conference theme, "Infrastructure Development in the Context of Contemporary Economic Challenges" has been carefully chosen to address myriads of problems that are re-defining the scope and solution techniques applicable in the contemporary practice of civil engineering.

In order to ensure a wide coverage of the domain of civil engineering, we broke the theme down to eight sub-themes encompassing such areas as: (a) Structural Engineering Practice (b) Transport Systems including Planning, Development, Operation and Maintenance (c) Water Resources Management (d) Geo-Sciences and Geo-Engineering (e) ICT in Infrastructure Development (f) Energy (g) Engineering Materials, and (h) Project Management. Thrillingly, responses were received in all the sub-themes.

Manuscripts received were subjected to blind peer-review, carried out by researchers who have in-depth knowledge and experience in the specialization, to ensure that our threshold minimum standard was met – the standard being consistent with any other similar international academic fora. Thus, out of the total eighty submissions received, sixty-five (representing 81% of the total submissions) were adjudged acceptable while the remaining fifteen could not meet the minimum acceptable quality level and therefore rejected. However, in the long run sixty of the accepted sixty-five registered for the conference. Interestingly, going by the accepted papers and subsequent registration, the participating countries include Nigeria, Niger Republic, India and Malaysia.

We, the entire members of the Conference Organizing Committee (COC), heartily welcome all participants to the ICEC and trust that you would maximally utilize the opportunity to peer-interact and establish contacts for possible future research collaborations and linkages. We also seize this opportunity to express our profound gratitude to all – authors, reviewers, supporting individuals and agencies, and above all the Management and staff of the Federal University of Technology, Minna, Nigeria, whose efforts have culminated in successfully holding the conference.

As the maiden edition of the biennial conference, this edition places us on the learning curve. We are therefore open to suggestions and constructive criticisms that would improve our future outings. We wish you all happy and fruitful academic and professional interactions.

Engr. Prof. S. Sadiku

Chairman (Conference Organizing Committee)

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Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

DISCRETE-CONTINUOUS CONFIGURATION OPTIMIZATION OF TRUSS STRUCTURE USING THE HARMONY SEARCH ALGORITHM

Jibrin I. Jibrin and Aaron Aboshio**
Civil Engineering Department, Bayero University, Kano
** Corresponding Author: aaboshio.civ@buk.edu.ng

ABSTRACT

The majority of the gradient-based optimization methods which are frequently used to solve structural optimization problems assume that cross-sectional areas (i.e., the sizing variables) are continuous. In most practical structural engineering design problems, however, the design variables are discrete due to the availability of components in standard sizes. This paper proposes efficient structural optimization method based on the harmony search (HS) heuristic algorithm that treats integrated discrete sizing and continuous geometric variables on a 37-bar simply supported truss structure, with stress and displacement constraints in a unified design space for structural weight minimization. Harmony Search method implemented in MATLAB code that uses a stochastic random search instead of a gradient search, obtained an optimal mass of 360.72kg on the example solved in this paper. This is better when compared with results reported by other researchers using different optimization methods. This result indicates that Harmony Search optimization algorithm studied in this research is as efficient as or better than other optimization algorithms.

Keywords: Optimization; Harmony Search Method; Truss Design; Matlab code; Discrete Size Optimization.

1 INTRODUCTION

The field of structural optimization is subject to rapid changes in its methods and goals (Borges at al., 2013). Until recently, there was a large imbalance between the huge amount of literature on the subject, and the lack of its applications in practical design problems (Geem, 2010). This imbalance is being corrected gradually, because there are many applications of methods for structural optimization in civil engineering and other engineering fields. As a result of the growth rate of these applications, studies on structural optimization methods are increasingly being driven by real problems (Geem, 2010).

Typically, structural optimization problems involve searching for the minimum of a stated objective function, usually the structural weight (Lee et al., 2011). This minimum design is subjected to various constraints with respect to performance measures, such as stresses and displacements, and also restricted by practical minimum cross-sectional areas or dimensions of the structural members or components. If the design variables can be varied continuously in the optimization, the problem is termed “continuous”; while if the design variables represent a selection from a set of parts, the problem is considered “discrete” (Lee and Geem, 2004).

Traditionally, mathematical gradient-based optimization techniques that depend on the designer’s prior experience and some common rules of thumb can be helpful, although they may not necessarily result in efficient and economical designs. However, these gradient-based methods are not suitable for discrete-type design variables such as ready-made cross sectional area of structural members (Geem, 2010). In most practical structural engineering design problems, however, sizes have to be chosen from a list of discrete values due to the availability of components in standard sizes and constraints caused by construction and manufacturing practices (Lipson and Gwin, 1977). This leads to discrete optimization problems, which are somewhat difficult to solve. Although conventional mathematical methods can consider discreteness by employing round-off techniques based on continuous solutions, the rounded-off solutions may yield results that are far from optimum, or they may even become infeasible as the number of variables increases (Lee & Geem, 2004.). Although both the traditional and the optimum design processes are iterative in nature they differ in the way the iterative process is terminated. In traditional design, the iterative process is terminated when the structure satisfies the necessary specifications. On the other hand, the process in the case of the optimum design approach is terminated only when a different termination or convergence criteria related to optimality is achieved (Arora, 2004). Recently, researchers have turned their interest into metaheuristic optimization techniques because they are able to efficiently handle discrete design variables. Harmony search (HS), as a metaheuristic algorithm concerted from music improvisation, has been vigorously applied to various structural design problems, obtaining good results (Geem, 2010).

The best solution of an optimization process is obtained from combining size and configuration optimization into a single stage optimization process because of the potential for much larger savings (Zhou et al., 2004). This research demonstrate the effectiveness and robustness of the HS meta-heuristic algorithm method for solving truss discrete size and continuous configuration optimization problems as compared to current optimization techniques, such as Particle Swarm Optimization (PSO) and the genetic algorithm (GA).

In this paper, Harmony Search was implemented in MATLAB code, as well as, a subroutines developed by the authors for truss analysis and optimization. The method changes cross sectional areas (Ai) and nodal coordinates (Ri), which are the design variables, looking for the minimum structural mass (f(x)), subject to the design constraints.

Discrete size and continuous configuration optimization methods based on the HS heuristic algorithm are introduced. The size and configuration optimization of structural systems involves arriving at optimum values for member cross-sectional areas A and continuous nodal coordinates R that minimize an objective function f(A, R) (i.e., the structural weight W).

Both minimum designs must satisfy q inequality constraint functions that limit the design variable sizes and the structural responses. The constraints imposed (member stress and nodal displacement) assumed to come from the set of admissible ranges (upper and lower limits). Thus, the problems can be stated mathematically, as minimizing the objective function (structural weight) subject to the design constraints:

$$Minimize \qquad f(A,R) = \gamma \sum_{i=1}^{n} L_i A_i \qquad 1$$

$$Subjected\ to \quad G_j^L \leq G_j(A,R) \leq G_j^U, \qquad j = 1,2,3,\ldots q \qquad 2$$

where, f(A, R) is an objective function (i.e., the structural weight), A = (A1, A2, … An) is the sizing variable vector that consists of the cross-sectional areas, γ is the material density of each member, R = (R1, R2, ..., Rm) is the continuous nodal coordinate variable vector and Ai and Li are the cross-sectional area and length of the ith member. Gj(A, R) shown in Equation (2) is the inequality constraints (stress and displacement) for the size and shape optimizations, G_j^l and G_j^u are the lower and the upper bounds on the constraints.

2. HS ALGORITHM-BASED STRUCTURAL OPTIMIZATION PROCEDURE

Figure 1 shows a detailed procedure of HS algorithm-based method used in this study as described in the following steps:

i. Step 1: Formulate optimization problem and initialize the Harmony memory (HM).
ii. Step 2: Improvise a new solution
iii. Step 3: Update the HM
iv. Step 4: Repeat Step 2 to Step 3 until a preset termination criterion is met.

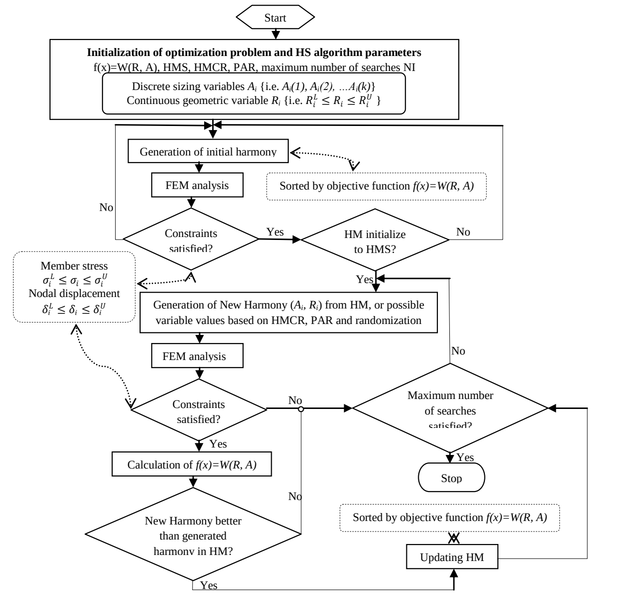

FIGURE 1: OPTIMIZATION PROCEDURE OF THE HARMONY SEARCH ALGORITHM.

Step 1: Formulate optimization problem, specify HS Parameters and initialize the Harmony memory HM.

The first step is basically expressing the objective function and the constraints mathematically as shown in Equation 1 and 2. The HS algorithm parameters that are required to solve the optimization problem are also specified in this step. These include the Harmony Memory Size (HMS), Harmony Memory considering rate (HMCR), pitch adjusting rate (PAR), bandwidth (BW), termination criterion (maximum number of iteration NI) and design variable bounds.

Subsequently, the Harmony Memory (HM) is initialized. HM consists a matrix of m rows and n columns, where each column represents candidate values that can be assign to the ith design variable. The initial HM consists of a certain number of randomly generated solutions from the possible available value set for the optimization problems under consideration. These sets are equal to the size of the HM (i.e., HMS). For an n-dimension problem, an HM with the size of n can be represented as follows:

$$HM = \begin{bmatrix} x_1^1,\ x_2^1, \ldots\ x_n^1 \\ x_1^2,\ x_2^2, \ldots\ x_n^2 \\ \vdots \quad \vdots \quad \vdots \\ x_1^{HMS}, x_2^{HMS}, \ldots x_n^{HMS} \end{bmatrix} \qquad 3$$

where, $[x_1^i, x_2^i, \ldots x_n^i]$ (i = 1,2,3,…HMS) is a solution candidate for the design variables. HMS is typically set to be between 10 and 50. Here, an initial HM is generated based on the FEM structural analysis results, subject to the constraint functions (Eq. 2), and sorted by the objective function values (Eq. 1).

$$HM = \begin{Bmatrix} x_1^1\ x_2^1\ \cdots\ x_n^1 \\ x_1^2\ x_2^2\ \cdots\ x_n^2 \\ \vdots \quad \vdots \quad \vdots \\ x_1^{HMS}\ x_2^{HMS} \cdots x_n^{HMS} \end{Bmatrix} \begin{matrix} \to f(x^1) \\ \to f(x^2) \\ \to \quad \vdots \\ \to f(x^{HMS}) \end{matrix} \qquad 4$$

Step 2. Improvise a new solution $X^i = [x_1^i, x_2^i, \ldots x_n^i]$ from the HM. Figure 2 shows the flowchart of the HS improvisation method for discrete and continuous design variables. Each component of this solution x_j^i, is obtained either from the initially generated Harmony Memory HM (with probability equal to HMCR) or generated randomly from the set of admissible range of values (with probability equal to 1-HMCR).

$$x_j^i = \begin{cases} x_j^i \in \{x_j^1, x_j^2, \ldots, x_j^{HMS}\} & w.p.\ \ HMCR \\ x_j^i\ \in\ X^i & w.p.\ (1-HMCR) \end{cases} \qquad 5$$

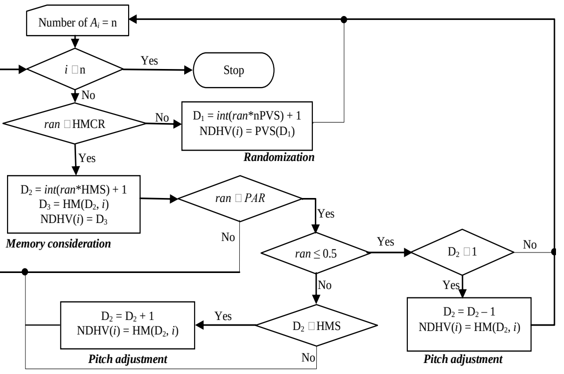

FIGURE 2(A). A NEW HARMONY IMPROVISATION FLOWCHART FOR DISCRETE SIZING VARIABLES

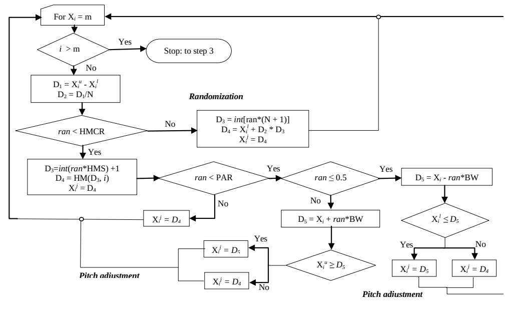

FIGURE 2(B): NEW HARMONY IMPROVISATION FLOWCHART FOR CONTINUOUS VARIABLES

If the component's value comes from the Harmony Memory, it is further tested to see if it should be mutated according to the Pitching Adjust Rate (PAR). The PAR determines the probability of a candidate from the HM to be mutated. The pitch adjustment process is different for discrete and continuous design variables.

$$x_j^i \leftarrow \begin{Bmatrix} Yes & w.p. & PAR \\ No & w.p. & (1-PAR) \end{Bmatrix} \qquad 6$$

The pitch adjusting process is performed only after a value has been chosen from the HM. The value (1-PAR) sets the rate of doing nothing. A PAR of 0.4 indicates that the algorithm will choose a neighboring value with 40% × HMCR probability. If the pitch adjustment decision for x_j^i is 'Yes' x_j^i is assumed to be $x^i(k)$, i.e., the k-th element in X^i, the pitch adjusted value for discrete and continuous design variable $x^i(k)$ is

$$\begin{matrix} x_j^i = x^i(k+c) & for\ discrete\ variable \\ x_j^i = x_j^i + BW * \mu & for\ continuous\ variable \end{matrix} \qquad 7$$

where c is the neighboring index, $c \in \{-1, 1\}$, BW is an arbitrary distance bandwidth for the continuous design variable, and μ is a uniform distribution between -1 and 1. The HMCR and PAR parameters introduced in the Harmony Search help the algorithm find globally and locally improved solutions, respectively.

Equations 5 to 7 represent the different ways a New Harmony can be improvised. These are summarized in Equation 8. The first case in Equation 8 represents picking values randomly from the HM without pitch adjusting. The second and the third cases represent picking random values from HM with pitch adjustments. And finally the last case represents picking a value randomly from the admissible set for the design variable X^i.

$$x_j^i = \begin{cases} HM[j,i], & P = HMCR \times (1 - PAR) \\ HM[j,i] + (ran[0,1].bw), & P = 0.5HMCR \times (PAR) \\ HM[j,i] - (ran[0,1].bw), & P = 0.5HMCR \times (PAR) \\ x_{random}, & P = 1 - HMCR \end{cases} \quad 8$$

As we can see, the improvisation of $[x_1^i, x_2^i, \ldots x_n^i]$ is rather similar to the production of offspring in the Genetic Algorithms GAs with the mutation and crossover operations [9]. However, the GA creates new chromosomes using only one (mutation) or two (simple crossover) existing ones, while the generation of new solutions in the HS method makes full use of all the HM members.

Step 3. Update the HM. The new solution from Step 2 is evaluated in terms of the objective function value using a FEM structural analysis method based on the constraint function. If the new improvised Harmony is better than any of the harmonies in the HM, then it goes into the HM replacing the worst Harmony. Otherwise, the improvised Harmony is neglected. A better Harmony is one that has better objective function values and less constraint violations. The HM is then sorted by the objective function value.

Step 4. Repeat Step 2 to Step 3 until a preset termination criterion is met (which in this case is the maximal number of iterations).

3. 37-BAR SIMPLY SUPPORTED PLANE TRUSS

Simultaneous discrete size and continuous shape optimizations were performed on the simply supported 37-bar plane truss with initial configuration shown in Figure. 3.

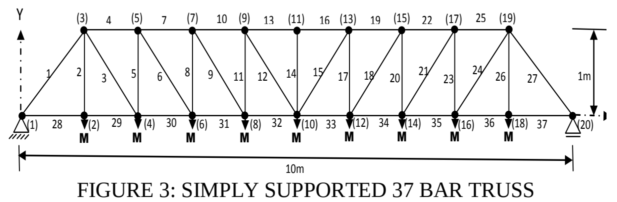

FIGURE 3: SIMPLY SUPPORTED 37 BAR TRUSS

i = Element number

(i) = Node number

Due to its simple configuration, this structure has been used as a benchmark to verify the efficiency of various optimization methods. The structure was initially modeled by Wang *et al*. (2004) and studied this problem using the Evolutionary Node Shift Method; Lingyun *et al*. (2005), solve the truss using the Niche Hybrid Genetic Algorithm (NHGA); Gomes (2011), using the Particle Swarm Optimization (PSO); Borges *et al*. (2013), using Harmony Search method and Kaveh *et al*. (2015), apply Dolphin Echolocation (DE) algorithm for optimum design of the truss structures. All these researchers studied the problem with a continuous design variable and with frequency constraints. In this research, Harmony Search (HS) is used to solve size and shape optimization of the truss using discrete sizing variable with stress and displacement constraints.

Loading condition considered in this research was a set of vertical mass M of 10 kg attached at each of nodes on the lower chord (2, 4, 6, 8, 10, 12, 14, 16, 18), which remain fixed during the design process. The material properties for equal leg angles (L) section S275 steel are Young's modulus E = 200x 10^9 N/m^2, Specific mass γ = 7800 kg/m^3. The HS design variables are the cross sectional areas of the bars *(A_i)* and the coordinates of the nodes *(R_i)*. The Harmony Search method change the cross sectional areas and nodal coordinates, looking for the minimum structural mass (*f(x, R)*), subject to stresses (σ) and displacements (δ) constraints. The members were subjected to stress limitations of 138 MPa and displacement limitations of 50.8mm.

The input parameters of the HS algorithm were set as follows: HMS = 20, HMCR = 0.8, PAR = 0.3, BW = 0.2, maximum number of iterations NI = 10,000. The HS parameter values were arbitrarily selected based on the empirical findings by (Yang, 2009), which determined that the HS algorithm performed well with $10 \leq HMS \leq 50$, $0.7 \leq HMCR \leq 0.95$, and $0.05 \leq PAR \leq 0.7$. An initial HM is generated based on the finite element FE analysis of the truss system developed in MATLAB code.

All members on the lower chord (28-37) have fixed cross-sectional areas of 4 x 10^{-3} m^2 and the others are to be chosen from the discrete values. Discrete values for the cross-sectional areas were taken from the set D $\in$ {0.645, 0.716, 0.910, 1.000, 1.129, 1.265, 1.290, 1.387, 1.613, 1.774, 1.981, 2.258, 2.523, 2.581, 2.852, 3.632} (cm^2), which has twenty discrete values.

In the optimization process, nodes on the upper chord can be shifted vertically. In addition, nodal coordinates and member areas are linked to maintain the structural symmetry. This was as contained in (Wang *et al*., 2004). Thus, only five shape variables (nodes) and fourteen sizing variables (areas) were redesigned for optimization, and are shown in Table 1. Each member and node is subjected to stress limitations and vertical displacement.

TABLE 1: NODAL COORDINATES AND BAR GROUPS OF THE SIMPLY SUPPORTED 37 BAR TRUSS

DESIGN VARIABLES	
Node Coordinates	
$Y_3 = Y_{19}$	
$Y_5 = Y_{17}$	
$Y_7 = Y_{15}$	
$Y_9 = Y_{13}$	
Y_{11}	
Cross Sectional Areas	**Elements**
Group 1	$A_1 = A_{27}$
Group 2	$A_2 = A_{26}$
Group 3	$A_3 = A_{24}$
Group 4	$A_4 = A_{25}$
Group 5	$A_5 = A_{23}$
Group 6	$A_6 = A_{21}$
Group 7	$A_7 = A_{22}$
Group 8	$A_8 = A_{20}$
Group 9	$A_9 = A_{18}$
Group 10	$A_{10} = A_{19}$
Group 11	$A_{11} = A_{17}$
Group 12	$A_{12} = A_{15}$
Group 13	$A_{13} = A_{16}$
Group 14	A_{14}

The HS algorithm-based method found 20 different solution vectors (i.e., values of the 19 design independent variables) after 10,000 searches. The best solution vector obtained from the HS algorithm with an objective function value *f(x)* = 360.75kg was the best of all the methods reported in the literature.

The optimal design obtained in this study and the comparisons with results of the other researchers cited above are shown in Table 2. The final configuration that provides minimum weight obtained in this research is shown in Figure 4.

TABLE 2: OPTIMUM DESIGN FOR THE 37-BAR SIMPLY SUPPORTED TRUSS WITH VARIOUS METHODS

RESEARCHERS			**Wang *et al.*, (2004)**	**Lingyun *et al.*, (2005)**	**Gomes, (2011)**	**Borges *et al.*, (2013)**	**Kaveh *at al.*, (2015)**	**Present study**
ALGORITHM			Evolutionary Node Shift	NHGA	PSO	HS	DE	Harmony Search
Design variables	**Nodal coordinates (m)**	$Y_3 = Y_{19}$	1.2086	1.1998	0.9637	0.9561	1.04	0.9559
		$Y_5 = Y_{17}$	1.5788	1.6553	1.3978	1.3331	1.40	1.3006
		$Y_7 = Y_{15}$	1.6719	1.9652	1.5929	1.5716	1.64	1.5616
		$Y_9 = Y_{13}$	1.7703	2.0737	1.8812	1.7741	1.74	1.7430
		Y_{11}	1.8502	2.3050	2.0856	1.8569	1.84	1.8657
	Cross sectional areas (cm^2)	$A_1 = A_{27}$	3.2508	2.8932	2.6797	2.7878	2.70	2.8520
		$A_2 = A_{26}$	1.2364	1.1201	1.1568	1.1194	1.00	0.910
		$A_3 = A_{24}$	1.0000	1.0000	2.3476	1.1428	1.00	1.000

$A_4 = A_{25}$	2.5386	1.8655	1.7182	2.2458	2.40	1.981
$A_5 = A_{23}$	1.3714	1.5962	1.2751	1.1426	1.20	1.129
$A_6 = A_{21}$	1.3681	1.2642	1.4819	1.1541	1.20	1.265
$A_7 = A_{22}$	2.4290	1.8254	4.6850	1.9163	2.20	2.258
$A_8 = A_{20}$	1.6522	2.0009	1.1246	1.4539	1.30	1.387
$A_9 = A_{18}$	1.8257	1.9526	2.1214	1.5773	1.90	1.613
$A_{10} = A_{19}$	2.3022	1.9705	3.8600	2.5871	2.20	2.581
$A_{11} = A_{17}$	1.3103	1.8294	2.9817	1.6016	1.30	1.290
$A_{12} = A_{15}$	1.4067	1.2358	1.2021	1.5072	1.40	1.774
$A_{13} = A_{16}$	2.1896	1.4049	1.2563	2.4911	2.50	2.523
A_{14}	1.0000	1.0000	3.3276	1.1166	1.00	1.000
Mass (kg)	**366.50**	**368.84**	**377.20**	**361.35**	**361.03**	**360.72**

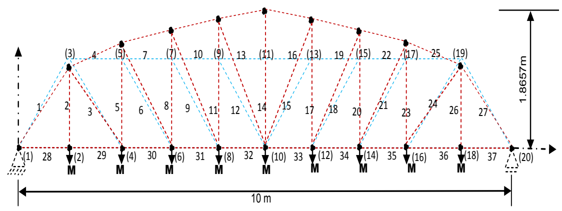

FIGURE 4: FINAL CONFIGURATION DESIGN OF 37-BAR SIMPLY SUPPORTED TRUSS

This paper described the Harmony Search meta-heuristic algorithm-based approach for truss optimization problems with discrete-continuous design variables. 37-bar benchmark was presented as a reference for comparison to demonstrate the effectiveness and robustness of the HS algorithm compared to other optimization methods, especially meta-heuristic algorithms already established in the literature. The result obtained using the HS algorithm on this example revealed that, optimization of weight using HS for the truss system considered in this research compares well and better than submissions in the literature based on other algorithms on similar structure, with a significant reduction in the total mass of the structure. This study, therefore, suggests that the HS is a global search algorithm and a powerful optimization technique in terms of both the obtained optimal value and the convergence capability that can be easily applied to truss optimization problems with stress and displacement constraints imposed.

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EFFECT OF OPERATIONAL CONDITIONS ON ADSORPTION EFFICIENCY OF POLLUTANTS FROM LIVESTOCK WASTEWATER

Anijiofor, S.C.[1,2]; *Nik Daud N. N.[1]; Idrus S[1].; & Che Man, H.[3]
[1]Department of Civil Engineering, Engineering Faculty, Universiti Putra Malaysia, Malaysia
[2]Department of Civil Engineering, Federal Polytechnic Birnin Kebbi, Nigeria
[3]Department of Biological and Agricultural Engineering, Engineering Faculty, Universiti Putra Malaysia, Malaysia
*Corresponding author email: niknor@upm.edu.my

ABSTRACT
Livestock wastewater treatment has become a growing challenge due to the varying nature and composition of wastewater from different livestock farms. As a result, effective and affordable livestock wastewater treatment methods has been the focus of recent studies. Notably, adsorption has become a widely studied wastewater treatment option due to availability of cheap adsorbents. This study investigates the use of stabilized solid wastes also known as "Aged Refuse" as adsorbent to treat wastewater from different livestock farms. Aged refuse are solid wastes deposited in the landfill which become stabilized after a period of time. The results revealed effects of operational conditions such as; pH, adsorbent dosage and contact time on removal efficiency of pollutants from Dairy Farm and Chicken Slaughterhouse wastewater. The optimum conditions were 4 g/100 mL dosage, 20-180 min treatment time, pH 6, room temperature, agitation speed 250 rpm and particle size ≤ 2mm. The maximum removal rates were (89 to 91 %) TSS, (82 to 91 %) TDS, (95 to 97 %) Colour, (91 to 94 %) COD, (85 to 94 %) BOD and (78 to 88 %) NH_3-N, at varying equilibrium times. Optimizing operational and environmental conditions are paramount for maximum removal efficiency of pollutants from such wastewaters. Utilization of solid wastes as treatment media will provide an alternative simple and affordable wastewater treatment option for livestock wastewater as well as reuse potentials for such wastes.

Keywords: *adsorption sites, aged refuse, operational parameters, removal efficiency, treatment time*

1 INTRODUCTION

Adsorption is a physical or chemical reaction where atoms, ions or molecules from a substance which can be either gaseous, liquid or solid, adhere to an adsorbent surface due to forces of attraction. During such process, the van der Waals forces attract adsorbed molecules and the solid surface resulting in reversible adsorption in some cases. Adsorption does not require a chemical compound during the process (Guellide U.de Souza et al., 2008). Adsorption isotherms are usually used to describe the phenomenon which governs the release of a substance from the aqueous state to a solid phase at a constant pH and temperature (Allen et al., 2004; Limousin et al., 2007). Tran et al., (2017) stated that "the isotherm curve is vital and explains the relationship that exists between the adsorbent and adsorbate at constant temperature, and this aids the design of adsorption systems". This analysis is significant because it describes the interactions between the adsorbent surface and adsorbate molecules. The physiochemical parameters and thermodynamic assumptions are essential features which explains the adsorption mechanism. Furthermore, once the adsorbate and adsorbent has contacted at suitable time, equilibrium is established (Kumar and Sivanesan, 2007). Also, the relationship between the equilibrium data and theoretical/practical equations is used to interpret and predict the degree of adsorption. Several adsorption models are available in literature; Henry, Langmuir, Freundlich, Hill, Redlich-Peterson, Dubinin-Radushkevich, Flory-Huggins, Tempkin, Sips, Toth, Koble-Corrigan, MacMillan-Teller Brunauer-Emmett-Teller, (Foo and Hameed, 2010) and several others.

Numerous studies have been carried out on the adsorption of different organic pollutants by different biomass in conventional biological treatment processes. Such studies have continued to dominate research, but developing an economically viable and efficient adsorbent remain a challenge. Genovese and Gonzalez (1998), achieved only 26–28% reduction in total solids from fisheries wastewater when treated with chitosan while Sakar et al., (2006), used coagulation by Chitosan followed by adsorption with Powdered Activated Carbon (PAC), to achieve 60-68% COD, and 40-44% TDS removal from dairy wastewater treatment with influent concentration of 1532 and 730 mg/L for COD and TDS respectively. Kushwaha et al., (2010), reported 68.7% COD reduction from synthetic dairy wastewater by adsorption onto activated carbon and Wang et al. (2011) reported 90.77, 76.10 and 70.13% removal efficiency for NH_4-N, TP and COD from livestock wastewater using immobilized microcystis aeruginosa. Shao and Zhang (2012), achieved 73 and 88.9% removal for NH_3-N and COD with influent concentrations of 2060 and 27520 mg/L respectively from livestock wastewater using Bentonite coated Chitosan as adsorbents. However, cost and future availability of adsorbents may possibly hinder future applications of some of these adsorbents.

The “Aged Refuse” (AR), is a typical example of unused, available and natural resources with various characteristics for adsorption. Chai and Zhao, (2006), reported that adsorption was very rapid with maximum adsorption of phenolic pollutants achieved within the first during the adsorption of phenolic compounds by AR. Wang et al., (2012), also studied sorption behaviour of aged refuse using batch experiments for chromium removal from wastewater. It was discovered that most of the chromium adsorbed on the aged refuse were in the trivalent form, and the main processes involved were surface adsorption and Cr (iv) reduction to Cr (iii) by the organic matter in the treatment media. However, under operating conditions, the maximum adsorption capacity of the adsorbent is a function of the amount adsorbed at equilibrium time as stated in several works (Azouaou et al., 2010; Brahmaiah et al., 2015), and several others. Such operating conditions include the pH which is dependent on the basic and acidic functional groups on the adsorbate surface, initial wastewater concentrations, adsorbent dosage for increased percentage removal, adsorbent surface, hence, dosage of adsorbent determines the capacity for removal. Also, adsorption is heat dependent as implied in Le Chatlier’s principles and increase in temperature increases the extent of adsorption and vice versa. The Surface area of the adsorbent also affect rate of adsorption**,** adsorption increases as particle size decreases as a result of fluctuating surface area characteristics. Such effect is perhaps due to the failure of the large ions to penetrate all the initial pore structure of the adsorbent. Therefore, adsorbents with smaller particle sizes favour adsorption process due to large external surface which removes more metal ions than the larger sizes. Agitation speed also control the rate of adsorption, adsorbents such as the AR with physical properties similar to soil, will require high agitation speed. However, most results obtained during adsorption studies are usually difficult to compare because the adsorbents have different properties (surface area, porosities etc), and as such different adsorption capacities in relation to experimental conditions and wastewater characteristics.

This study determines the effect of operational conditions such as pH, adsorbent dosage and contact time on the removal efficiency of pollutants from varying livestock wastewater concentration using landfilled aged refuse. To achieve this effectively, livestock wastewater sources such as dairy farm and slaughterhouse were investigated. Studies on livestock wastewater treatment by adsorption is very limited in literature, and such adsorption effects by aged refuse has not been previously investigated in Malaysia. Therefore, considering the compositional differences of solid wastes from region to region and the huge organic content of solid wastes generated in developing countries, there is a justification for this study as alternative cheap and abundant resource for wastewater treatment. Moreover, the study highlights pollutant configurations in different livestock wastewaters and the need for proper selection of operational conditions as a function of maximum removal efficiency.

2 METHODOLOGY

2.1 SAMPLING PROCEDURE

The adsorbent used for this investigation was obtained from a closed landfill in Selangor, Malaysia. The aged refuse was air-dried and sorted out and particle size of ≤ 2 mm was used for the batch adsorption studies. Livestock wastewater was collected from UPM dairy farm (GPS coordinate: 2.9992 ºN, 101. 7078 ºE), and a chicken slaughterhouse in Serdang Selangor, Malaysia (GPS coordinate: 3.0333 ºN, 101.7167 ºE), which is very close to the UPM (Figure 1).

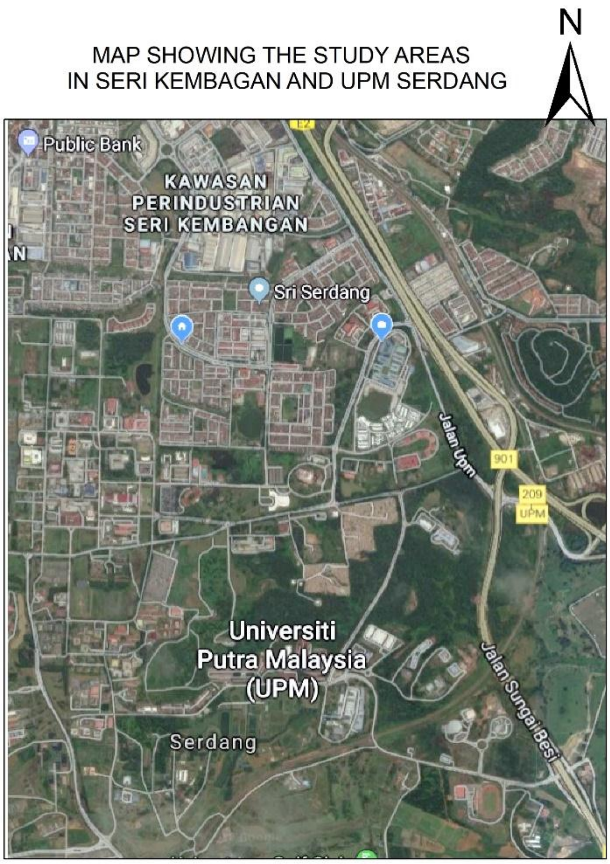


FIGURE 1: LOCATION OF DAIRY FARM (LADANG 16) UPM AND CHICKEN SLAUGHTERHOUSE SRI KEMBANGAN

Influent and effluent concentrations of BOD, COD, TSS, TDS, Colour and NH_3-N were examined in line with Standard Method for Examination of Water and Wastewater guidelines (APHA, 2005).
The vacuum filtration method was used for TSS and TDS determination, while apparent color was measured in terms of Platinum-Cobalt (Pt-Co) by Spectrophometric method. The COD which measures the percentage of the organic matter removed, was determined by digesting the wastewater samples by procedures laid down in the closed reflux method using $K_2Cr_2O_7$, H_2SO_4, ferrion indicator and Ferrous ammonium Sulphate (FAS), as titrant. The BOD measures the quantity of oxygen consumed by the microorganisms and Azide Modification method was used for BOD_5 incubated at 20 ºC for 5 days. The DO was determined using $MnSO_4$, Alkaline-Iodide Azide reagent, H_2SO_4 and $Na_2S_2O_3$ titrant with starch indicator.
For Ammonia nitrogen (NH_3-N) determination, the Salicylate method using ammonia salicylate reagent powder pillow and ammonia cyanurate reagents, while the PhosVer 3 with Acid Persulfate Digestion method was used for TP determination at 150 ºC for 30 minutes.
Both influent and effluent concentrations of the wastewater samples were determined in triplicates or duplicates as the case may be and average results have been reported, with several dilutions where necessary.

2.2 BATCH EXPERIMENTS

The batch study was carried out in 250 ml flasks by means of a DAIHAN Scientific Digital orbital shaker, (SHO-1D), at room temperature. The effect treatment time, pH and adsorbent dosage was studied and constant agitation speed of 250 rpm for uniformity.
The effect of pH was carried out by agitating 4g/100 mL for SHW and DFW at different levels of pH 4 – 8, using 0.1 N NaOH or H_2S0_4, to adjust the pH to the desired level. Several studies including (Devi et al., 2008; Wang et al., 2012; Sivakumar, 2013), have reported pH within this range as suitable for biological wastewater treatment. The experiment was monitored at room temperature, agitation speed of 250 rpm, ≤ 2 mm adsorbent size and 40 min contact time.
The influence of AR dosage on pollutants reduction efficiency was determined by varying the dosage from 2 to 10 g in 100 ml of sample, while other conditions; pH 6, adsorbent size ≤ 2 mm, agitation speed 250 rpm, treatment time of 40 min and temperature were constant.
For effect of contact time, 4g/100 mL adsorbent dosage, pH 6, adsorbent size ≤ 2mm, agitation speed 250 rpm and ambient temperature conditions were constant while time varied. For SHW, agitation was 180 min at interval time of 30 min while DFW was at an interval time of 20 min up to 180 min.

3. RESULTS AND DISCUSSION

The parameters investigated for the adsorption studies were; COD, BOD, Colour, TDS, TSS and NH_3-N. The wastewater characteristics are shown in Table 1.

TABLE 1: LIVESTOCK WASTEWATER CHARACTERISTICS

Parameter	**Unit**	**DFW**	**SHW**	***MEQR, (2009)**
pH	-	7.08	7.17	6.0-9.0
Temp.	ºC	24.9	27.2	40
Colour	Pt Co	11,933	10,600	300
TSS	mg/L	1260	312	100
TDS	mg/L	1900	702	1000
COD	mg/L	3300	2720	200
BOD_5	mg/L	1250	960	50
COD:BOD_5	(average)	2.6	2.8	2.0:1
NH_3-N	mg/L	92	76	5

DFW-Dairy Farm Wastewater; FPW-Fishpond Wastewater; SHW-Slaughterhouse Wastewater; *MEQR - Malaysia Environmental Quality (Sewage) Regulation, (2009).

The sorption capacity of the adsorbent is related to the type and concentration of surface groups responsible for interactions with the pollutants which gives rise to different sorption capacities for different adsorbents. Therefore, porous nature of the AR allows penetration to the sorption sites. The temperature for the experiment was kept constant fluctuating at ambient conditions (25 - 30 ºC), which is within favourable design standards and previously reported temperature conditions of less than 40 ºC conditions. The nature of the adsorbent which is soil-like required high agitations to ensure a rapid and favourable mix, therefore agitation speed was maintained at 250 rpm.

3.1 EFFECT OF PH ON POLLUTANT REMOVAL EFFICIENCY

The pH of the adsorption medium influences the adsorption capacity due to the surface charge distribution of the adsorbent. Therefore, the degree of adsorption depends on the varying functional groups on the adsorbate. Also, the pH affects the degree of ionization of the wastewater as well as the surface property of the adsorbent which also affects the adsorption capacity (Adelaja et al., 2011; Ramaraju et al., 2014). The removal efficiency was calculated from (1).

$$Removal\ Efficiency\ (\%) = \frac{C_0 - C_e}{C_0}\ x\ 100 \qquad (1)$$

C_o, C_e = initial and final wastewater concentration (mg/L).
Therefore, the effect of pH is demonstrated in Figure 2.

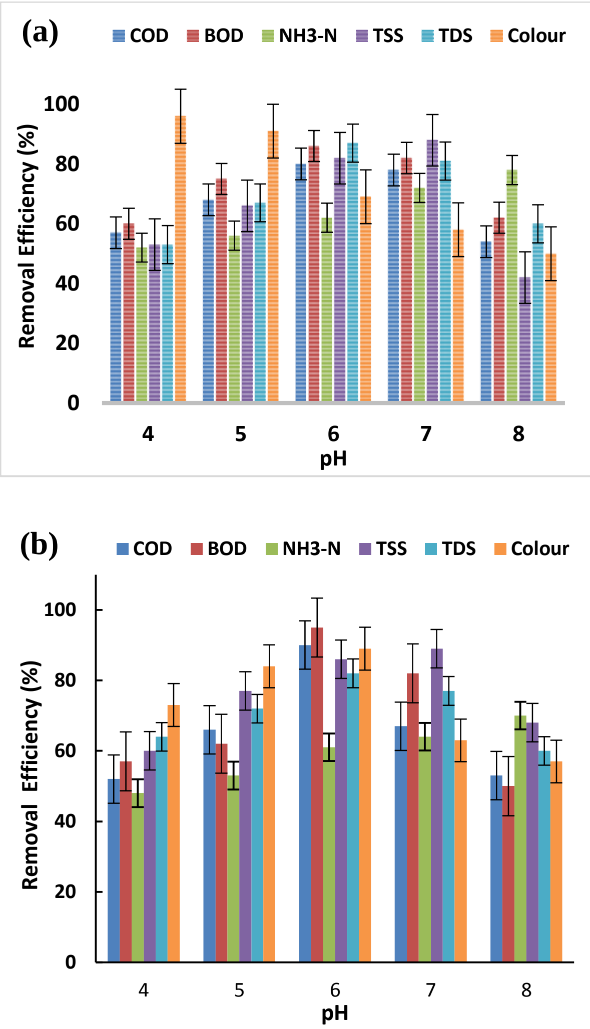


FIGURE 2: EFFECT OF PH ON POLLUTANTS REMOVAL EFFICIENCY (A) DAIRY FARM AND (B) SLAUGHTERHOUSE WASTEWATER

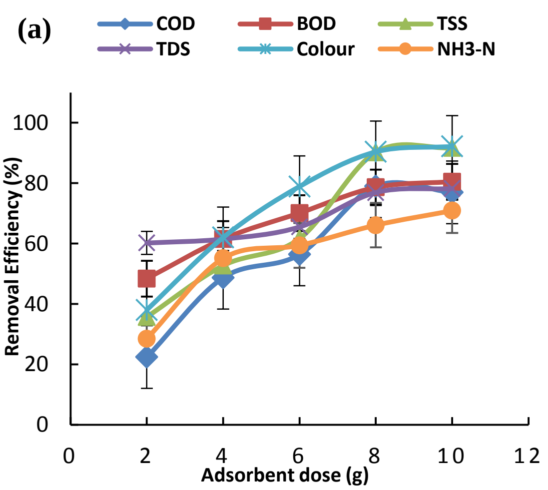
The optimum pH value for most of the parameters in this investigation lies between pH 6 and 7 due to the neutral nature of the adsorbent and wastewater. The removal efficiency of most of the pollutants increased rapidly at first as the pH was increased and then decreased afterwards at pH 8. The high removal efficiency for various pollutants at low pH is as a result of the presence of H+ ions which enhances adsorption capacity (Baral et al., 2006; Andal and Charulatha, 2014). These results obtained for the pH were similar to those reported in (Chai and Zhao, 2006; Devi et al., 2008; Mulu, 2013; Kaur et al., 2016), for removal of organics from wastewater. Similarly, the decreased adsorption efficiency at higher pH could also be as a result of abundance of OH^- ions which reduced adsorbent capacity and as a result caused increased hindrance to diffusion of organics. For ammonia nitrogen removal, increased pH resulted in increased adsorption efficiency due to the volatile nature of ammonia. Van der Stelt, (2007) and Zhang et al., (2009), obtained similar pH range for ammonia removal.

The maximum percentage removal for DFW (Fig. 2a), were; 80% COD, 86% BOD_5, 88% TSS, 87% TDS, 96% Colour and 78% NH_3-N. The colour of the wastewater was reduced drastically from darkish brown to a colourless and odourless effluent. Maximum efficiencies for SHW (Fig. 2b), were within the range of 90 % COD, 95% BOD_5, 89% TSS, 82% TDS, 89% Colour and 70% NH_3-N.

Temperature fluctuations occurred during the treatment, but maintained a suitable range of 25 to 30 °C for all the wastewaters. Most adsorption processes are governed by exothermic processes. Higher temperature enhances adsorbate capacity because increased temperature increases the adsorbents kinetic energy and expands the available pores and active sites (El-Naas et al., 2010; Kaur et al., 2016).

3.2 EFFECT OF DOSAGE ON POLLUTANT REMOVAL EFFICIENCY

The effect of dosage on adsorption efficiency was examined by varying adsorbent dosage from 2-10 g as shown in Figure 3, while other conditions were kept constant. Since adsorption is a surface phenomenon where adsorbate molecules occupy specific sites on the adsorbent, increasing the adsorbent dosage would definitely result in changes in the configuration and consequently on the removal efficiency. For this investigation the contact time was fixed at 40 minutes, agitation speed of 250 rpm, media size of ≤ 2mm with constant influent concentrations and pH of 6. A decrease in pollutant concentration as the adsorbent dosage was varied from 2 to 10 g, was observed during treatment. This result indicates that an increase in adsorbent dosage resulted in an increase available sites for adsorption. Increased adsorbent dosage favours organic pollutants removal from highly polluted wastewater within a reasonable time. This effect is demonstrated in Figure 3.



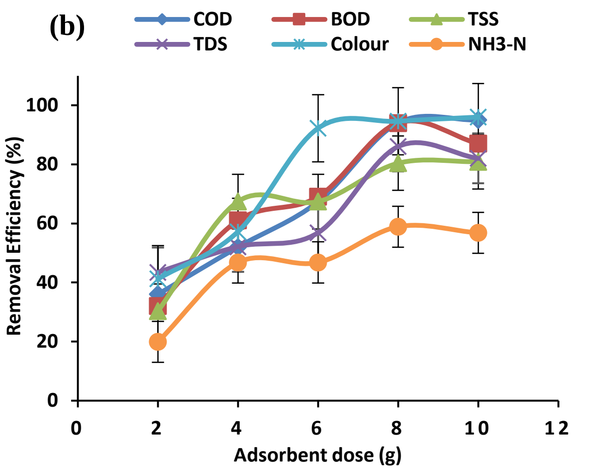


FIGURE 3: EFFECT OF DOSAGE ON REMOVAL EFFICIENCY (A) DAIRY FARM AND (B) SLAUGHTERHOUSE WASTEWATER

Removal efficiency for the DFW Figure 3a, and SHW Figure 3b, increased as the dosage was increased up to 8 g and thereafter further increase resulted in a slight difference in the removal efficiency up to 10 g. Maximum removal was (95 and 79) % COD and (94 and 80.5) % BOD_5 for SHW and DFW respectively. This is due to unsaturated adsorption sites and an increase in dosage creates availability for exchangeable sites. This is because having obtained saturation, the adsorption sites were filled up and as the dosage increased no further adsorption of the pollutants occurred. Meanwhile, even at as low as 2 g, 63 and 81 % efficiency was observed for COD and BOD removal. For the physical parameters Colour, TDS and TSS, removal efficiency increased with increased dosage and maximum removal was achieved between 8-10 g for SHW and DFW. Maximum removal were (91.7, 78.2 and 92.2%) DFW, and (80.9, 86, and 96%) SHW respectively.

For NH_3-N removal, increased dosage at a fixed time of 40 min, resulted in increased removal efficiency from (28.5 to 70.9%) for DFW, and (19.9 to56.8%) for SHW. The livestock wastewater samples varied in their compositions, and such variations determine the rate of pollutant reduction based on their ionic properties. The presence of blood in the SHW and other toxic compounds from manure in the DFW could be responsible for favoured adsorption when large dosages were used as observed. These results are similar to results reported in literature (EI-Said et al., 2010;; Ramaraju et al., 2014; Brahmaiah et al., 2015; Anijiofor et al., 2018), on the increasing rate of pollutant reduction with increasing adsorbent dosage. Large adsorbent dosage is required for COD and BOD optimal removal, however, as removal efficiency increases due to increased dosage, adsorption capacity decreases.

3.3 EFFECT OF CONTACT TIME ON REMOVAL EFFICIENCY

Increase in the contact time resulted in an increased removal efficiency as demonstrated in Figure 4. This occurrence is due to the nature of the adsorbent as well as its pore spaces which are determinant factors on the time needed for equilibrium to be reached.

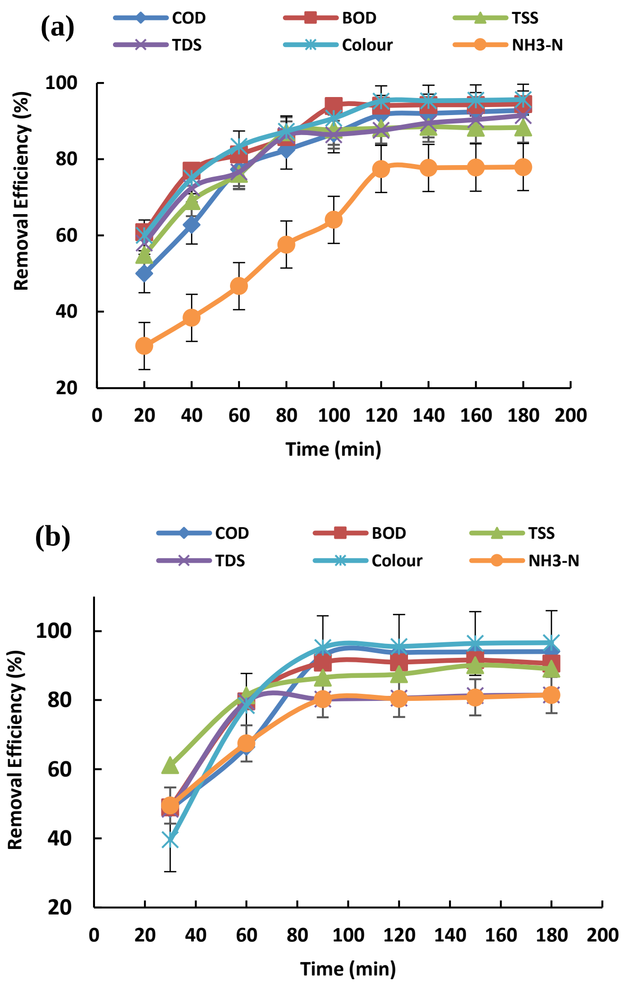


FIGURE 4: EFFECT OF CONTACT TIME ON POLLUTANT REMOVAL EFFICIENCY (A) DAIRY FARM AND (B) SLAUGHTERHOUSE WASTEWATER

Contact time relates to the amount of pollutants adsorbed as a function of time. However, treatment efficiencies and optimal conditions for treatment highly depends on the characteristics of the adsorbate and adsorbent.

For DFW Figure 4a, the removal efficiency for all the parameters increased with increasing contact time. COD and NH_3-N removal maintained saturation at 120 minutes of contact time with COD efficiency increasing gradually from 50% to 92%. Initial and final concentrations were 3399 and 238 mg/L respectively. On the other hand, 77% reduction rate was observed for NH_3-N from initial and final concentration of 92 and 20.3 mg/L respectively.

BOD_5 removal had an initial reduction of 60.8% which increased further to 93.9 % at a saturation time of 100 minutes with maximum removal of 94.4 % and residual concentration of 70 mg/L from 1250 mg/L initial concentration. TSS and TDS reached saturation at 80 minutes with removal rates of 87.0 and 86.2 % and residual concentrations of 38 and 163 mg/L respectively while colour reduction was maximum at 95.6 % at saturation time of 120 minutes and residual concentration of 520 from 11933 Pt Co influent concentration. Meanwhile pH and temperature increased during treatment from initial values of 6.00 at 25.5 ºC to 6.42 at 30.8 ºC after180 minutes. The increase was steady and gradual but within the favourable range for biological wastewater treatment and did not affect the removal process considering the high removal rates obtained

For the SHW Figure 4b, a similar trend was observed with the removal efficiencies of all the parameters increasing as the contact time increased, and attained equilibrium corresponding to 90 minutes of treatment time. Ammonia nitrogen removal rate was higher than in the DFW with up to 81.5 % removal. Residual concentrations were 160.2, 90, 34, 360, 14.1 mg/L and 130 Pt Co from initial concentrations of 2720, 960, 312, 702, 76 mg/L and 10600 Pt Co for COD, BOD_5, TSS, TDS, NH_3-N and Colour respectively. Slight increases in pH was observed from 6.00 to 6.23 and temperature from an initial 25.1 to 29.6 ºC.

The pore filling mechanism for adsorption onto AR was observed due to its permeable nature, as well as other characteristics which include; cation exchange, surface precipitation hydrogen bonding, and so on. These factors also depend on the adsorbent characteristics and wastewater composition, (Anijiofor at al., 2018). Similarly, the mass transfer of molecules from liquid to solid state is dependent on the influent composition of the wastewater and also the treatment time. This phenomenon describes the adsorbent-adsorbate affinity resulting from increasing contact time and the saturation and equilibrium affinity of the available sites. Therefore, contact time determines equilibrium after which no significant change in the adsorption rate can occur which is in line with other reported results in literature. Due to the varying degrees of concentration and composition of the different livestock wastewater, differences exist in the adsorption capacities and saturation times observed during the experiment.

COD and BOD_5 removal rates were lower with shorter time because removal of carbonaceous material is highly time dependent. Ammonia nitrogen adsorption increased also as the contact time increased as a result of more contact time between the adsorbent and the adsorbate. These results are similar to various results reported in the literature in which removal efficiency of pollutants increased as the contact time increased (EI-Said et al., 2010; Mulu, 2013; Ramaraju et al., 2014).

4. CONCLUSION

Wastewater treatment technologies seem complex and expensive, hence the need for the utilization of cheap, simple and affordable treatment options to curtail increasing effect of disposal of unwanted livestock wastewater into the environment. Adsorption effect of pollutants onto aged refuse investigated in this study has been very successful and maximum removal rates ranging from (89 to 91%) TSS, (82 to 91%) TDS, (95 to 97%) Colour, (91 to 94%) COD, (85 to 94%) BOD and (78 to 88%) NH_3N, at varying equilibrium time was achieved. Therefore, optimization of environmental and operational conditions are valuable treatment strategies for enhanced pollutant removal efficiency. Moreover, the aged refuse is an available and cheap adsorbent which can be utilized for wastewater treatment and consequently enhance its recycling potentials for solving landfill congestion problems. This technology suits the “waste to wealth” initiative currently explored in solid waste management. In future applications, an aged refuse biofilter could be employed to experiment its biodegradation and sedimentation effects.

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WEIGHT OPTIMISATION OF A LABORATORY STOOL USING ANSYS WORKBENCH

***Aisha Muhammad[1,2], Ibrahim Haruna Shanono[3]**
[1]Department of Mechatronics, Faculty of the Engineering, Bayero University Kano, Nigeria
[2]Mechatronics Engineering, Faculty of Engineering, University Malaya, Malaysia
[3]Department of Electrical, Faculty of the Engineering, Bayero University Kano, Nigeria
*Corresponding author email: ayshermuhd@gmail.com

ABSTRACT
In this paper, failure analysis and weight optimisation of a laboratory stool were carried out by the Finite Element Method (FEM) using ANSYS workbench. The optimisation is performed on a stool structure made of structural steel material under different loading and design constraints. The marginal and optimised masses for the different load cases are analysed and shown on a scaler plot. Convergence plot was also obtained for the static analysis solution after a maximum number of 10 refinement loops. The structural design was carried out in SolidWorks software and then imported into the ANSYS workbench for analysis

Keywords: *ANSYS, Deformation, Finite Element Analysis, Laboratory stool, Von-Mises Stress, weight optimisation.*

1 INTRODUCTION

A stool is primal sitting furniture introduced by the Varangian guard in Byzantium (Macquoid, 1988), (Chinnery, 1979). It is similar to a chair but had no arms and back. The early days developed stools are made up of a single person seat on a base of three or four legs. Some latest developed stools have a backrest. Being one of the primitive wooden furniture, the emergence of stools is lost in time (Chinnery, 1979). During the middle age around the 11th to 14th-century seating is done on stools, benches and the throne-like chairs which signify and represents royalty and higher status. Farmers also use the three-legged stools during cow milking (Wikipedia, 2018). A large box-like joined stool through the joining of nails and long spindles together was later developed in the 17th century using timber (Chinnery, 1979)

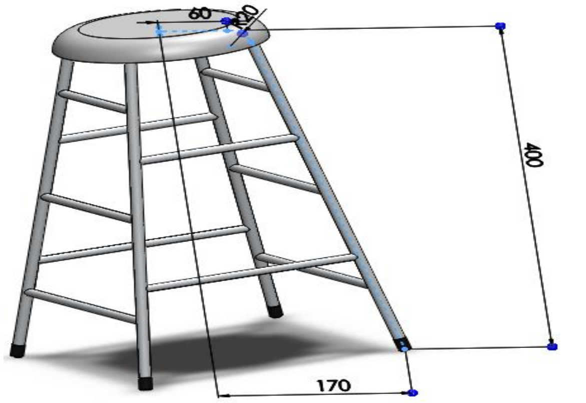
Stools are made from different type of materials and in different designs or models. Some are made of metal or wooden materials including stainless steel and chrome style. They are constructed with or without back, armrest and padding on the seat top. The designs also differ from simple ones too much more complex designs. Some are extra tall or short, swivel and floor mounted (those that are immovable and as such cannot be stolen or moved to another location), with adjustable and non-adjustable height, and can be used indoor or outdoor (Wikipedia, 2018).

For an effective, safe and relaxing studying environment, a perfect lab seating is required. Key importance in lab seating is finding the ideal seating stool by taking into consideration the working condition such as duration of work and concentration. Examining the seating requirements needed in the laboratory will significantly help in choosing the correct material, minimising cost by avoiding the need for replacement and manufacture as well as improving the working condition (Rewade, 2016). The need for thorough consciousness is needed in the design of school laboratory seating. Safety precautions should also be taken into consideration (Pinnacle, 2015)

Weight reduction is needed in laboratory stools in order to reduce the weight that is needed in construction which minimises the cost by saving the amount of material. In this paper, failure analysis is carried out together with weight optimisation by Finite Element Method using ANSYS workbench. Different loads and design constraints are applied to the structure and analysis carried out. The marginal and optimised masses for the different load cases are shown on a scaler plot.

Many kinds of literature have worked on weight optimisation. Gaikwad in his paper modifies a roller conveyor by performing weight optimization after carrying out static analysis on the roller conveyor (Gaikwad, 2013). In another paper by Rajratna (2014), the weight of a roller conveyor was reduced thereby saving the materials under specific load constraints using finite element method.

In this paper, a laboratory stool of structural steel material is used for the analysis with dimensions as shown in figure 1



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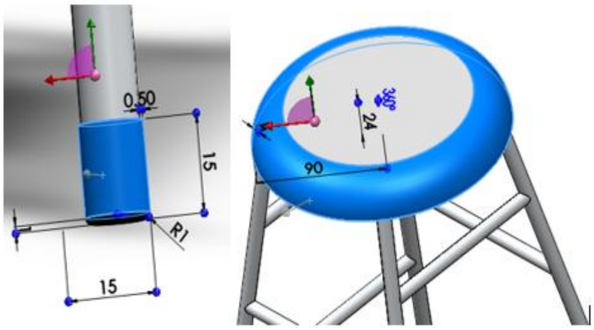

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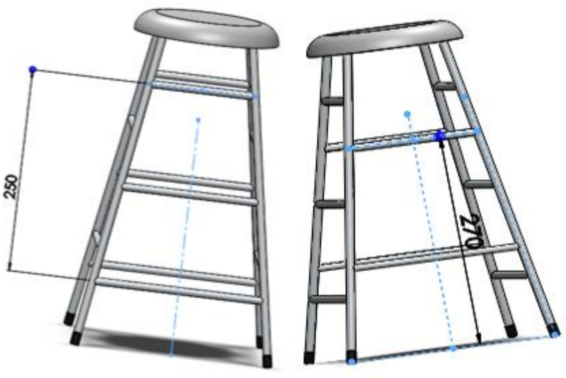


(c)

FIGURE 1 (A) STOOL OVERALL HEIGHT (B) GLIDES/BASE AND SEAT (C) FOOT REST HEIGHT

2 MATERIALS AND METHODS

2.1 Finite Element Methods

Finite Element Method is a partial differential equation that shows the exact numerical methods problem solution (Chen and Liu, 2014). Sub-dividing a complex structure into smaller components is the crucial idea in Finite Element Analysis (FEA). It is a very powerful tool among the CAE computational tool as engineering designs can be design based on the design criteria within a shorter period. The analysis that includes Stress and dynamic response under different constraints are solved using Finite Element Method. The design of objects or structures that cannot be done manually is done in Computer-aided design software. ANYSYS Workbench software platform is used as it is easy to understand. It also gives two-way access to computer-aided design software. Figure 2 below depicts the FEM procedure and steps used to solve problems.

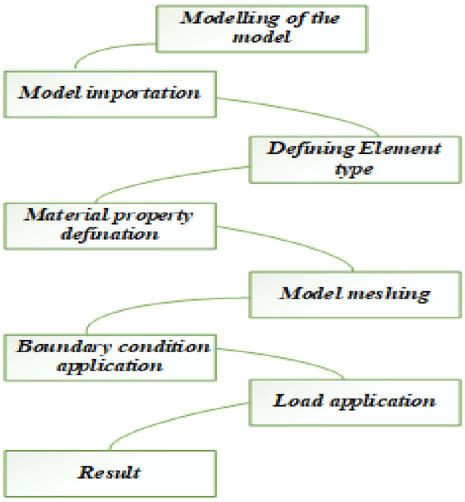


FIGURE 2: FEA PROCESS FLOWCHART

2.2 Methodology

In this paper, the material used for the laboratory tool is structural steel. The setting of the material property is done accurately as it affects as well as determines the analysis result that will be obtained (Talikoti, 2016). These material properties are defined based on the type of simulation analysis to be performed. For an efficient and qualitative analysis of material, the material properties need to be correctly and carefully entered. These materials which can be either linear or nonlinear, isotropic or orthotropic, constant or temperature dependent needs to be defined correctly. Depending on the aim of the analysis, some mechanical properties such as density, strength and coefficient of thermal expansion definition is optional (Barbero, 2014). Knowing and declaring the correct value of the material property is very useful for design analysis purpose. The different types of material indicated by different density will as well show different analysis result. The Young's modulus of a material alternatively called modulus of elasticity is a numerical constant that describes the elasticity and measures the capability of a solid to withstand changes when subjected to tension or compression in a particular direction. The higher Young's modulus, the stiffer (i.e. how it deflects under load) is the material which will require a much higher amount of load to deform. Poisson’s ration which is the ratio of compression to the expansion of material together with Young's modulus (ratio of stress to strain) defines the strength and nature of how a material structure deforms based on a certain constraint. Material deformation due to uniform volume and opposing forces are described by the bulk and shear modulus respectively. Two other vital properties that determine when the material losses its elastic behaviour and the maximum stress a material can undergo are the yield and tensile strength respectively (Nipun, 2015). Despite the limitations of steel in terms of rust-prone and heaviness, it is chosen as the material for this analysis due to some of its advantages that include: stiffness, long lasting and high strength (Madiwal, 2014). Table 1 below shows some of the material properties.

TABLE 1: PROPERTIES OF MATERIAL

Material/ Property	Structural steel
Young's Modulus (Pa)	2E+11
Density (kgm^-3)	7850
Poisson's ratio	0.30
Bulk modulus (Pa)	1.6667E+11
Shear Modulus (Pa)	7.6923E+10
Tensile strength (Pa)	2.5E+08
Ultimate shear strength (Pa)	4.6E+08

Analysis of any structure begins with the geometry definition which is defined based on the type of simulation analysis that is to be carried out. The stool structure that was designed using SolidWorks software is imported through the interfacing between the ANSYS workbench and SolidWorks (Kim, 2007) (Janq, 2002) (Xi, 2002). There are two ways in which the solid geometry can be imported into the ANSYS workbench:

a) The designed structure in SolidWorks should be saved in IGES (Initial Graphics Exchange Specification) format before its imported
b) Accessing the ANSYS as the structure is being designed (Guanzhu, 2012).

The figure below shows the importation on the laboratory tool into the ANSYS workbench.

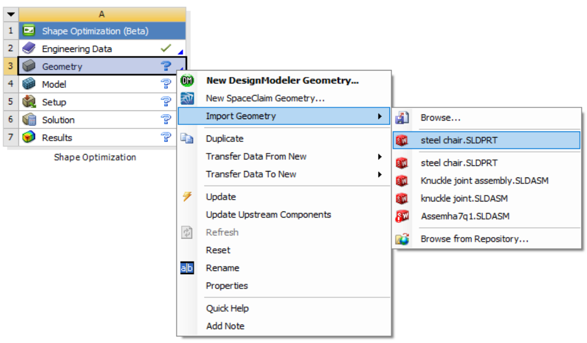


Figure 3: Geometry importation

After the geometry importation, meshing is done before analysis is carried out. Meshing in FEA is the breaking down of the model into smaller elements that can be separately analysed (Talikoti, 2016). It is a discrete realisation of the structure, which helps in solving the structure into a number of exact solutions. The accuracy of the analysis result is proportional to the meshing size. Finer meshing size gives a more accurate result but with a longer computational time (Qiongying, 2014). ANSYS has a great control tool for meshing. (Baomin, 2005). Meshing tools are categorised into five, which include:

1. Unit size control
2. Level control of the intelligent division
3. Thinning grid control

Taking into consideration the setup time and computational expense, with the speed and ease of use, a free mesh type is used. The default meshing control is used having a relevance value of +100 with a medium smoothing number of iteration. Figure 4 below shows the meshed structure.

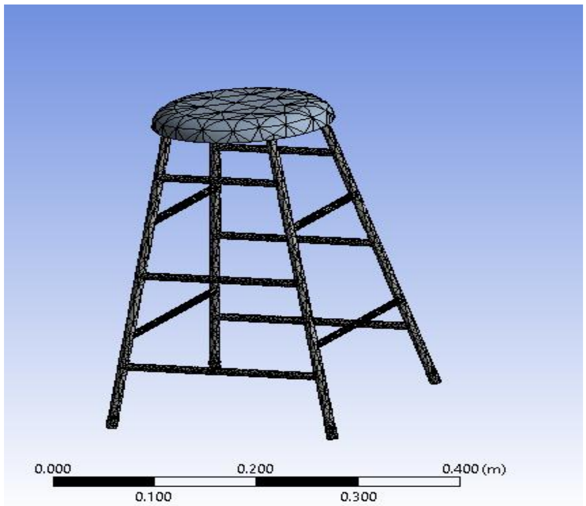
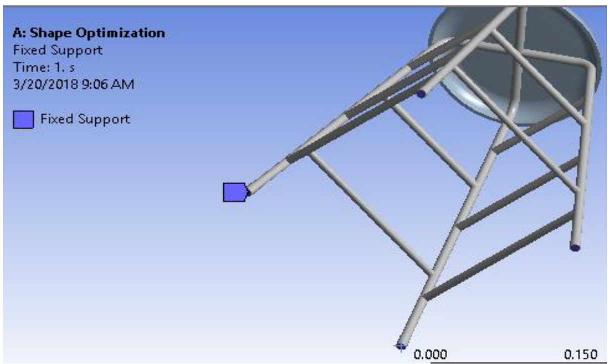


FIGURE 4: MESHED STRUCTURE

Setting the boundary conditions and constraints is an essential step in Finite Element Analysis. The key step before starting analysis is the setting of the boundary conditions (Talikoti, 2016). The constraints set in a way that fits the actual conditions (Guanzhu, 2012). The laboratory stool is fixed at the base legs, and a force of 1000N is applied on the sitter as shown in the figures below:



(a)

(b)

FIGURE 5: (A) FIXED SUPPORT APPLICATION
(B) LOAD CONSTRAINTS APPLICATION.

2.3 STRENGTH ANALYSIS

Engineering components and structures (Machines) are prone to failure. Conducting analysis becomes very important in order to ascertain the safety and certainty of these structures and components. Failure occurs as a result of the unanticipated breakdown of the structure due to deformation caused by overload. Among the various criteria used in predicting failure of designs to a given load condition, Von Mises-Hencky distortion-energy theory is widely used by designers mostly in ductile materials. The expression of Von-Mises is as follows (Mathew, 2012).

$$\sigma_v = \left[\frac{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}{2}\right]^{\frac{1}{2}} \quad (1)$$

Based on this theory, failure of design is subject to the maximum Von Mises stress value being more than the material strength. Therefore, keeping the Von Mises stress below the yielding strength keeps the design safe (Moaveni, 2008). Hence, a failure condition is given by the expression below;

$$\sigma_v \geq \sigma_y \quad (2)$$

$$\left[\frac{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}{2}\right]^{\frac{1}{2}} \geq \sigma_y \quad (3)$$

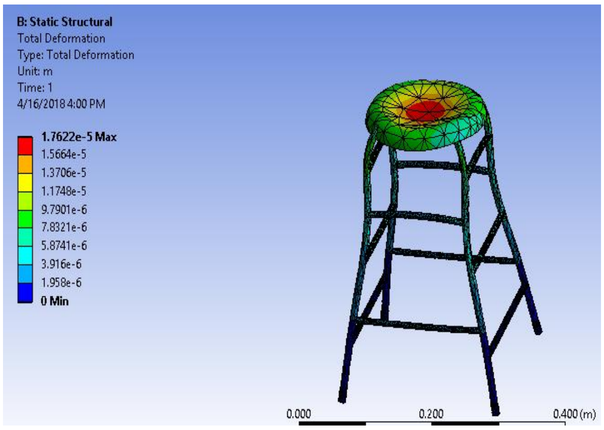
Where σ_y is the obtained material yield strength from a tension test.

The relationship between Von-Mises stress, yielding stress and the factor of safety (F.S) is given by:

$$\sigma_v = \frac{\sigma_y}{F.S} \quad (4)$$

3 RESULTS AND DISCUSSION

This section presents the analysis result performed. Figure 6a shows the deformation property of the material having 1.5664e-005m as the maximum deformation occurring in the middle of the sitter. Figure 6b shows the stress distribution in the material with the maximum stress of 1.4002e+007Pa. Figure 6c shows the safety factor plot, which indicates that the material strength can withstand the maximum applied stress, making it safe to be used in the said application.



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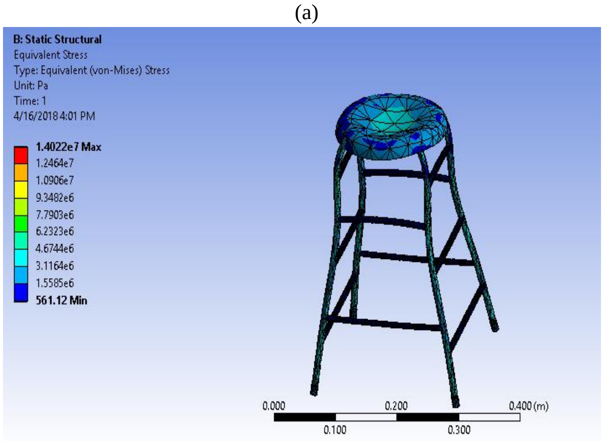

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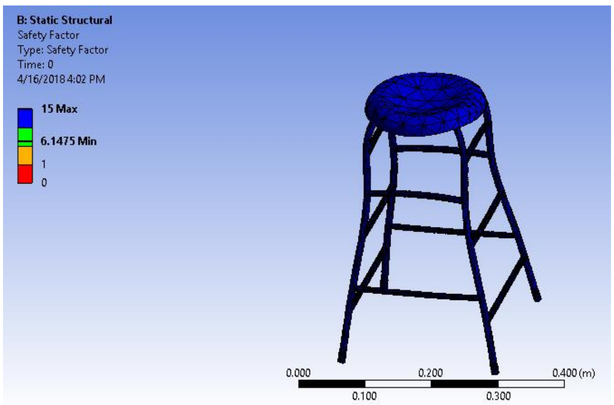

(c)

FIGURE 6: (A) TOTAL DEFORMATION.
(B) EQUIVALENT STRESS.
(C) SAFETY FACTOR.

For a load of 785N the following result was obtained for a weight reduction of 40, 50, 60, 70, 80, and 90% respectively.

FIGURE 7: 785N LOAD CONSTRAINTS.

FIGURE 8: 40% WEIGHT OPTIMIZATION

FIGURE 9: 50% WEIGHT OPTIMISATION

FIGURE 10: 60% WEIGHT OPTIMIZATION

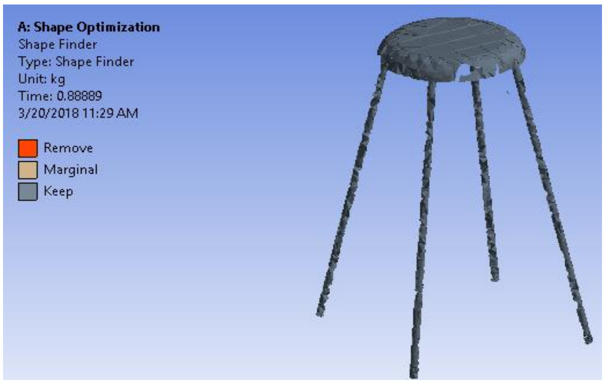

FIGURE 11: 70% WEIGHT OPTIMIZATION

FIGURE 12: 80% WEIGHT OPTIMIZATION

3.1 MASS COMPARISON

The weight optimisation result for the various percentages is summarised in the table below. A scalar representation is also shown in figure 13.

TABLE 1: % WEIGHT REDUCTION TABLE.

Weight Reduction	Marginal Mass (Kg)	Optimized Mass (Kg)
40%	9.52E-03	2.192
50%	1.32E-02	1.9884
60%	3.25E-02	1.7896
70%	2.53E-02	1.6044
80%	3.40E-02	1.4061
90%	1.93E-02	1.2027

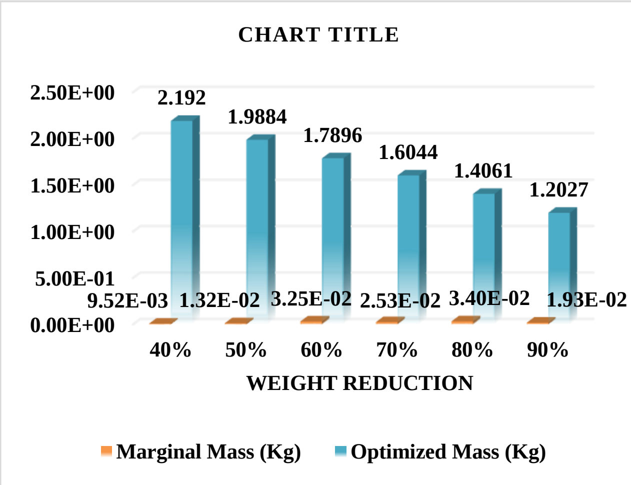

FIGURE 13: MASS COMPARISON

3.2 CONVERGENCE HISTORY

Further repeated refinement of the FEA structure into much smaller elements results in the convergence of the solution to an exact solution of the mathematical problem. Some of these refinement types formulated in FEA which converges the solution to an exact solution of the mathematical model include: h-refinement (with h referring to the element size), p-refinement (preferring to the polynomial highest order), r-refinement (which rearrange the nodes in the mesh) and hp-refinement (which combines h and p-refinement) (Chen and Liu, 2014). In this analysis, the convergence criteria are based on the total displacement with 1% tolerance limit. Convergence was obtained for the static analysis solution after a maximum number of 5 refinement loops as shown in figure 13. Maximum total deformations that result at different mesh iteration are displayed and shown in table 3 below.

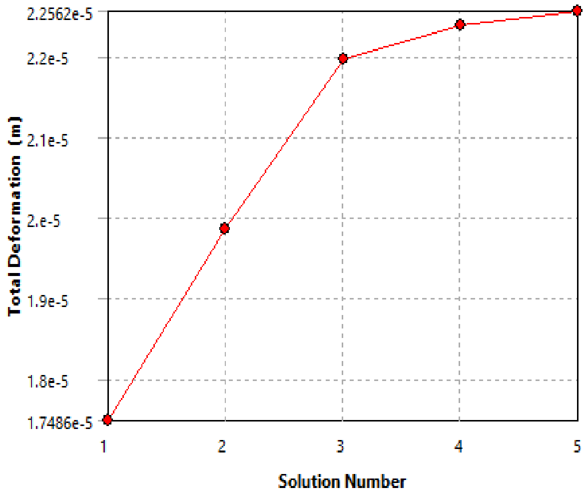

Figure 14: Convergence plot

TABLE 2: ITERATION TABLE

	Total Deformation (m)	Change (%)	Nodes	Elements
1	1.7486e-005		29162	13757
2	1.9858e-005	12.704	37163	18241
3	2.1973e-005	10.11	64653	35545
4	2.2399e-005	1.9203	104640	62653
5	2.2562e-005	0.72572	114608	69733

4 CONCLUSION

Finite element method is seen as an effective method in the modelling as well as analysis of structures. In this paper, failure analysis is carried out on a laboratory stool together with weight optimisation through Finite Element Method using ANSYS. The process was performed under different structural constraints with the analysis carried out with the marginal and optimised masses. Convergence was also obtained for the static analysis solution after a maximum number of 5 refinement loops. The result shows that structural steel material is suitable for the said structure when used within the design loading condition.

It can be concluded that ANSYS software can be employed by production companies to minimise material wastages and maximise profits at the same time maintaining product quality and reliability.

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A POLICY FRAMEWORK FOR EFFICIENT AND SUSTAINABLE ROAD TRANSPORT SYSTEM TO BOOST SYNERGY BETWEEN URBAN AND RURAL SETTLEMENTS IN DEVELOPING COUNTRIES: A CASE OF NIGERIA

***Adeke, P. T.[1]; Atoo, A. A.[2] & Joel, E.[3]**
[1,2] Department of Civil Engineering, University of Agriculture Makurdi, Benue State, Nigeria.
[3]Department of Economics, Federal University Wukari, Taraba State, Nigeria.
*Corresponding author email: adeke.pt@outlook.com

ABSTRACT

This study presented a policy framework for achieving an efficient and sustainable road transport system to boost synergy between urban and rural settlements in developing countries considering the case of Nigeria. The policy framework aims at creating an efficient and sustainable transport system that connects people and places to support economic growth, the environment and social well-being of the citizens in both urban and rural areas of developing countries like Nigeria. A transport development strategy plan was proposed in line with Nigeria's peculiar characteristics for the attainment of a prosperous economy, healthy, safe and secured environment with social equity among citizen. The interrelationship between proposed transport strategy elements and contributory policy instruments were examined, then an integration matrix built to access the strength of the proposed transport policy instruments. The key objective and performance indicators were identified with their corresponding operationalization as well as measure of successes and justification for the proposed framework. It was logically satisfactory that the proposed policy framework was reliable as it agreed with findings of other researchers, hence recommended for adoption for urban and rural area design and development to improve the synergy between urban and rural settlements so as to boost sustainable development in Nigeria.

Keywords: *Performance indicators; policy objectives; policy strategy; strategy elements; sustainable transport system and transport policy framework.*

1 INTRODUCTION

The transport system of every city has significant impact on all its facets ranging from socio-cultural, economic and environment (Schiller *et al.,* 2010; Efobi and Anierobi, 2014; Gudmundsson *et al.,* 2016). In line with modern societal developmental policies, sustainable transport system is crucial for every society, especially our fast growing cities (Olanke, 2013). An efficient and sustainable transport system aims at providing livable cities and rural areas associated with hitch free flow of people and activities for optimum performance and better living condition over time (Mitiku, 2009; Schiller *et al.* 2010; Olievschi, 2013; Gudmundsson *et al.,* 2015). Many urban and rural settlements of developing countries are faced with several transport challenges ranging from one kind to the other, which have adverse effects on their economic growth and well-being of the citizens; in the urban areas, these challenges in most cases are manifests of congestion problem also known as performance indicators which could hamper the safety, health, security, environment and economic activities of a city. A good transport system facilitates economic and social-cultural activities and improve the livability of every city for improved productivity (Aderamo, 2010; Hine, 2014; Beck *et al.,* 2017).

Though rural settlements of most developing countries such as Nigeria are characterized by poor or total absence of rail or road transport facilities which disconnects the rural and urban areas thereby subjecting rural dwellers to untold hardship since cities are generally referred to as the engine of economic growth, center of cultural diversification and technological hub (Efobi and Anierobi, 2014). The negative impact of such disconnection are suffered through transportation of agricultural produce to markets located in the cities, maternal mortality cases that call for urgent medical attention, educations (especially girl child), etc. which affect to a great extent the livability of the city and overall gross domestic product (GDP) of the country (FDEAER, 2015). Therefore, rural and regional integration and globalizations should be encouraged through effective rural transport system development (Mitiku, 2009; Banjo *et al.,* 2012; Hine, 2014).

Also, previous researchers have established that the fast growing populations of cities and the high rate of private car use in most cities constitute the major transport problem in form of congestion described as increased link flow, hence generate pollution (air, noise, and hazardous particle), affects bus services (exclusion and delay), requires more parking spaces which claims lands meant for other developments such as housing, recreational centers, shopping and industries etc. (Cullinane, 2001; Cullinane and Cullinane, 2003; Solanke, 2013). This retards economic activities in the city and affects the environment and the social well-being of the citizens (Transport for London, 2006; South-Yorkshire, 2014; Efobi and Anierobi, 2014). Consequently, the increased number of vehicular accidents and casualties in cities threatens human lives as it is associated with traffic congestions, poor road infrastructures as well as road network which fails to adequately accommodate excessive car use demand for to-and from movement mostly from suburb to the central or traffic attraction zones for purposes of education, hospital, shopping, work, leisure, recreation etc. This assertion is not doubtful since traffic behaves like gas which claims every available space in a vacuum (Schiller *et al.,* 2010). The planning, design and development of urban cities as well as their surrounding rural or suburb areas to address their present, near and future or foreseen transport problems is essential to guarantee an efficient and sustainable transport system for a growing city characterized by equity, safety, healthy environment, economic boost and social-wellbeing for the dwellers (South-Yorkshire, 2014).

In line with sustainable developmental goals, most developing countries in recent times have made transport development policy a priority with the aim of achieving growth and developmental goals through outcomes and broad framework for the transport sector and sets objectives for provision of safe, affordable and reliable access to the rural population and improving multi-modal transport services as a priority. This calls for improvement of the transport network and increasing the efficiency of the existing transport system (Mitiku, 2009; Aderamo, 2010; Banjo *et al.,* 2012).

The current and forecasted transport problems in urban areas of Nigeria which have direct negative impact on the surrounding rural areas include traffic congestion in cities which have been identified as a major transport problem as its affects the free movement or link flow from one location to another regardless of the mode used (Schutte, 2008; Schiller *et al.,* 2010). Indicators of congestion in most cities at present reveal a trend characterized by steady increase in traffic volume over time, which consequently leads to increased queue lengths along routes and high density of traffic at trip attraction points (Cullinane, 2001; Cullinane and Cullinane, 2003), the probable reason for this is the increase in human population with corresponding private cars use on the road. Also, if truly traffic behaves like gas as asserted by Schiller *et al*. (2010), then high concentration of traffic at the city centers would not allow vehicles from the suburb to enter city centers, thereby leading to issues of inequity, retraction of economic activities involving rural produces, human right infringements, etc.(Hine, 2014).

In most developed cities, mass transit is easily achieved using efficient and reliable public transport services. Private car ownership affects the percentage of trips made by cars as compared to those made using other modes. Also the increase in number of people using public parking spaces mostly on-street parking for long periods as its being practiced in Nigeria has significant impact on transport systems performance in terms of roadway capacities. These parking spaces are associated with peak hour car trips made to work and for educational purposes. The problem of congestion is due to the fact that most travelers use private cars rather than public transport due to poor and unreliable public transport services in some cities of countries like Nigeria. The present state of public transport services in major cities of Nigeria such as Abuja, Lagos, Kano, Port-Harcourt, Calabar, Ibadan, etc is appalling. It is characterized by increased rate of auto crashes with high number of casualties, prolonged travel time, high level of passenger discomfort, intensive air and noise pollution caused by high traffic volume with private cars being the major stream composition, unregulated bus fares, etc. This highly congested scenario constitutes serious environmental, health, socio-economic, etc nuisance to the public (Schiller *et al.,* 2010; Airey, 2014; Gudmundsson *et al.,* 2016).

Disconnection between the urban, suburb and rural areas of most states has been a common situation caused by poor town planning or negligence by the experts involved in the act. Road facilities connecting the urban and rural settlements are usually left in dilapidated state, thereby frustrating trips from rural to urban areas. This creates significant difference in the standard of living between both dwellers which promotes inequity among citizens and infringes human rights laws (Schiller *et al.,* 2010; Wakefield-Council, 2014; Redcar&Cleveland, 2014; Gudmundsson *et al.,* 2016).

In order to address this ugly situation existing between the urban and rural areas of most developing countries like Nigeria, a transport development strategy plan is proposed by this study in line with Nigeria's peculiar characteristics for the attainment of a prosperous economy, healthy, safe and secured environment with social equity among citizen, this transport strategy hopes to create a sustainable transport system that connects people and places to supports economic growth, the environment and social well-being of the citizens in both urban and rural areas of Nigeria. Towards this end, some policy instruments and strategy elements are employed to actualize the set goal.

This transport strategy aims at promoting the use of public transport system thereby reducing car use which is responsible for congestion that leads to excessive noise, high level of greenhouse gases emission that are harmful to human health, high travel costs (travel time and fuel), as well as ensuring even distribution of development facilities to discourage unnecessary long distance travel and possibly rural-urban migration. This can be achieved by providing reliable and affordable public transport system and discouraging car use through strict or unfavourable policies as well as approval of development in suburb and rural areas. A conducive living condition (good roads and facilities with proper maintenance) at the city center and its suburb zones to create social equity (Christie *et al.,* 2013). Generally, in terms of operations of transport system, an efficient and sustainable city is targeted towards attaining the following objectives;

A more livable City for all citizens - a livable city offers high level of wellbeing, health and security for all its dwellers. An efficient transport system contributes towards achieving this by providing to the inhabitants a reliable and safe access to social services. People are able to travel on different transportation modes as "they are fairly certain of reaching their destinations without delay". The interrelated or integrated flow of traffic ensures a smooth flow of movement (Mitiku, 2009). The successes of this policy are measured through levels of accessibility of transport services to all zones within the district, reduction in accident causalities, road pavement condition and, availability and optimum frequency of bus services (Beck *et al.,* 2017). Elimination or minimization of transport pollution in the form of noise, air pollution in form of small particles, nitrogenous and carbon gases emitted from exhaust pipes, etc. Also, to carter for the abled and disabled commuters in accessing transport facilities with sufficient supportive road infrastructures such as walkways, pedestrian crossing, traffic control signals, parking bays, off-street parking facilities, etc. (Wakefield-Council, 2014).

To Support a vibrant economy - for a vibrant economy, the presence of reliable transport systems and infrastructure are important for sustainability. Commercial operators rely on the regional connectivity provided by transport systems for conveying goods and services and to promote growth in the economy (Hine, 2014; BrockleBank, 2014). A reliable transport system would go a long way in addressing issues of congestion that have been seen to negatively affect the economic growth (Mitiku, 2009; Schiller *et al*., 2010). The attainment of this objective is measured by indicators such as annual turnover, amount of new retail floor space and new households established within the urban area. This could be estimated as the number of newly established supermarkets, establishment of small and large market squares, amount of small scale business setup within the shortest time frame, etc. The increase in traffic volume on routes and vehicle occupancy over time (FDEAER, 2015; Pinard, 2015; Nogales, 2015).

Tackle Environmental and Social inequity - this objective aims at creating a balance between the positive and negative effects of transport on society and the environment, giving equal opportunities to all group of individuals, not being genda sensitive in transport accessment, selective service pattern, etc. the adoption of dispersed settlement parten is essential in distributing the negative externalities of transport systems to various settlements. This can be done by improving air quality, "access to public transport" and addressing noise levels due to industrialization (Mitiku, 2009; Banjo *et al.,* 2012; Hine, 2014).

Reduced contribution of transport to climate change - the effects of transport operations on climate change intensify with increased vehicular flow. Personal cars use carbon based fuels and as such more carbon dioxide emissions are expected with more cars on the road. Changes to climate can be mitigated by "encouraging more sustainable modes of transport" such as bus, bicycle, walking, etc (Mitiku, 2009; Hine, 2014; Runji, 2015). Developed societies have measured success in this area through changes in modal split to favour public transport and walking as well as decreased link flows to limit congestion (Dargay, 2000; Nogales, 2015). Road pricing has been identified as another strategy to discourage private car use (Walker, 2011).

Integration of Strategy Elements and Measures

In transport planning and policy development, policy instruments are tools used for measuring the outcome of a strategy action, while strategy elements are the actions or methods adopted to improve part of the transport system. A strategy element is a collection of policy instruments aimed at improving transport systems; while a collection of logically related transport related strategy elements constitute a transport strategy (Gudmundsson *et al*., 2015).

Generally, a reliable and cheap bus service discourages car use (Cullinane, 2001). This could be achieved by the provision of relatively cheap, clean, efficient and reliable bus service which attracts more patronage, through reduced fare, increased frequency and bus lane in the major cities. Revenues generated from road pricing, parking charges (aimed at discouraging car use) and taxations shall be used for bus subsidization, road facility construction or maintenance, safety campaign and calming measures etc.(Cullinane, 2001; Cullinane and Cullinane, 2003; Gudmundsson *et al.,* 2015). Good public transport system minimizes congestion and its related problems (high travel cost, pollution, accidents etc.), hence ensures travel time reliability, reduced pollution as well as connects people and places easily, and creates social equity among citizens. Inadequate and expensive car parking facilities frustrates car users (Cullinane and Cullinane, 2003).

On the other hand, road pricing, reduced parking space required, while increased parking charges for both long and short stays discourage car use and promote mode shift, generate revenues for infrastructural development and subsidization of bus services as well as improve land use, hence reduce congestion, promote public transport and walking which have economic and health benefits due to available walkable destinations (Transport For London, 2006). The strategy intends to promote active modes, integration of transport and land use by encouraging development (mostly) in local areas to discourage long distance travels, ensure accessibility and improved safety facilities for pedestrians and motorists alike to reduce accidents and casualties, promote walking and guarantees safety; which creates a livable city with social equity among its dwellers. Also, even-distribution of infrastructural development across entire city extending to the suburb with improved road network creates equity, car users and non-car users bear approximately the same travel cost for daily travel demands. Reducing distance travelled by situating industries, shopping and commercial centers, administrative offices within walkable distances (in local zones) prevents unnecessary long distance travel to central area for work, education, shopping, hospitals, recreation etc., hence reduce congestion and consequently accelerates economic growth, creates healthy and safe environment, and social well-being of the dwellers, generates revenue (taxation) and encourages walking, therefore discourage car use hence reduce congestion and risk of vehicle accidents and casualties (Dargay, 2000; BrockleBank, 2014). Also, improved road facilities and network ensures safety for both motorist and pedestrians. Based on guidelines proposed by KonSULT (2014), the integration of strategy elements and contributory policy instruments required for achieving the set goal of this study are as shown in Table 1;

TABLE 1: INTERRELATIONSHIP BETWEEN STRATEGY ELEMENTS AND CONTRIBUTORY POLICY INSTRUMENTS

Strategy Elements	**Policy instrument**									
	Increase Bus Frequency	Reduce Bus Fare	Decrease off-street Parking Spaces	Improve road facilities	Approve Dev. in local area	Invest in road maintenance	Invest in New Projects at the city center	Assign Bus lane	Increase Taxation	Introduce Road Pricing
Improved Public Transport	•	•	-	•	•			•	•	-
Reduced Car use	•	•	•		-			-		•
Promote active Mode			-	•	•	•	•			-
Reduce travel distance			-		•		•			•
Improved Road Network					-			•	•	•
Key: • Major Contributor - Minor Contributor										

Table 1 showed that, there is a fair distribution of the influence of strategy elements over the policy instruments. There are 20 points of major and 8 points of minor contributions correlating between the policy instruments and proposed strategy elements as highlighted by the study. Since the number of major contributors are relatively higher, the influence of the strategy elements towards achieving the set objective for the transport policy are said to be stronger and more promising.

For a sustainable and integrated transport planning strategy, synergy and complimentary benefits of policy instruments are expressed in a matrix form showing the interrelationship between policy instruments based on benefit reinforced, financial barrier reduced, compensation for losses and political impact as shown in Table 2. According to KonSULT (2014), a policy integration matrix is used to show the interrelationship between policy instruments.

TABLE 2: POLICY INTEGRATION MATRIX

Policy Instruments	**Increase Bus Frequency**	**Reduce Bus Fare**	**Decrease Parking Spaces**	**Increase Parking Charge**	**Improve road facilities**	**Approve Dev. in local area**	**Invest in road maintenance**	**Invest in New Projects**	**Create Bus lane**	**Introduce Taxation**	**Introduce Road Pricing**
Increased Bus Frequency			+ •	* + •		•				+ * •	+ * •
Reduced Bus Fare			• +	• + *						+ * •	+ • *
Decreased Parking Space	+ •	+ •							+ •		
Increased Parking Charges	+ • *	+ * •			* •		• *	* •	+		
Improved Road Facilities				• *			+			+ • *	*
Approved Development in Local Area	+	+	+	+ • *			•	+ •	+	+ *	+ *
Invest in Road Maintenance				+ *	+					* + •	+ *
Invest in New Projects				* •		+				* •	+ •
Create Bus Lane	+		+ •	+ * •		+				• + *	+ • *
Introduce Taxation	+ • *	• + *			• + *	•	+ • *	+ • *	+ *		
Introduce Road Pricing	• *				• + *		• + *	+	• +	• +	

Key: + Benefits Reinforced ***** Financial Barriers reduced • Compensation for Losses

Table 2 revealed the correlation among policy instruments as they affect the success of the proposed transport strategy.

2 APPRAISAL FRAMEWORK

A proposed appraisal framework for achieving a sustainable transport system that connects people and places to supports economic growth, the environment and social well-being of the citizens living in urban and rural areas of Nigeria measure for initial period of Do-nothing to a future time when the policy is fully implemented after 2 -5 years is as shown in Tables 3 and 4;

TABLE 3: APPRAISAL FRAMEWORK FOR SET OBJECTIVES

Objective	Indicators	Operationalization	Measure of success	Justification
To create a more liveable City for all citizens	Vehicle user casualties per kilometre travel	Number of vehicle user casualties in accident reports	Decrease	Road safety is important as people travel daily to meet their needs. Reduced accidents indicate increased safety.
	Bus headway or frequency	Bus frequency as reported by public transport services	Increase	Reduced waiting time by increasing bus frequency reduces average cost and increases demand as well as reliability.
	Number of destinations reachable within short travel time by non-car users living at the suburb	Calculate average number of destinations for each zone from land use report	Increase	Transport improves the quality of life by giving people access to various destinations. Access is lowest among non-car users living in zones further from the city centre.
To support a vibrant economy	Commercial Taxation	Financial value of current account	Increase	Increase at uniform tax rate indicates more sources of revenue resulting from vibrant economy.
	Retail floor space at central business districts (CBD)	Count amount of retail floor space in CBD of each zone	Increase	Indicates growth in employment opportunities.
	New development in urban and rural areas	Read number of New households from land use data	Increase	An increase in new households is an indicator of economic growth and improvement in the housing market.
To tackle environmental and social inequity	Distribution of Noise levels in the districts	Difference in noise levels between central and peripheral zones	Decrease	Noise levels should be within acceptable limits within all zones to ensure equality in environmental conditions.
	Level of disturbance particles (NO_x, CO_2, CO, etc)	Difference in level of disturbance particles between the central and peripheral zones	Decrease	The amount of disturbance particles should be minimal at all zones to achieve quality in air condition.
	Number of home-based trip origins made by car and none-car commuters	Difference between home-based trip origins made by car and no-car commuters	Decrease	All social groups have access to transport services regardless of the mode for social inclusion and equity.

TABLE 4: APPRAISAL FRAMEWORK FOR SET OBJECTIVES CONTINUED

Objective	Indicators	Operationalization	Measure of success	Justification
Reduce the contribution of transport to climate change	Number of vehicles parked at public parking spaces	Average number of parking period	Decrease	Reduced parking spaces for reduced car use and consequently less carbon dioxide emissions. Free space used as green zones to improve air quality.
	Percentage of car trips	Assessment of difference in Modal Split	Decrease	Reducing % of trips by car helps in reducing Carbon dioxide emissions.
	Link flow for inbound cars	Average difference from flows on all links.	Decrease	A decrease in traffic flow means higher speed, less travel time and reduced congestion as well as emissions.

3 STRATEGY IMPLEMENTATION

The proposed strategy could be implemented by any tier of government (local, state or federal) with the aim of achieving the set objectives based on measures highlighted. In order to attract patronage for public transport at the inertial year, the inherited bus fare should be reduced with increased frequency (Gudmundsson *et al.,* 2015). This operation and others such as road maintenance practice, reduced parking space with increased charges (for city center zones), and traffic calming and safety decisions required huge investment. Due to insufficient funds and high cost incurred by the transport authorities for improving poor transport situation, ongoing projects within the district should be shelved so as to gain financial stability as the aforementioned policies required huge amount of money for successful implementation.

The approved development for local areas should be relatively higher than city centers and environs; this is to encourage sustainable rural development. Increase in both domestic and commercial tax for more revenue generation and transfer of funds from the current to capital accounts in order to balance it and avoid denier of subsequent capital expenditures by the policy makers/operators.

This policy instrument promotes walking mode since most destinations shall be within walkable distances of the zones; hence increase health benefits associated with walking. Also, improved road facilities to ensure safety for both motorist and pedestrians is essential to reduce rate of accidents and casualties. In order to ensure safe and good road network, there shall be consistent investment in road maintenance over time. Revenues shall be generated for the strategy's sustainability over time, they shall be steady increase in domestic taxes for some time, while commercial tax shall be on increase. They shall be increase in parking charge in order to reduce car usage which constitutes congestion (Cullinane, 2001; Cullinane and Cullinane, 2003; Transport for London, 2006; Olanke, 2013; Salau, 2015).

Generally, conditions of congestion could easily lead to increased air and noise pollution creating an unhealthy environment (Transport For London, 2006; Sheehan, 2010). A decline in the economy is also likely due to reduced productivity among frustrated and weary workers using the unreliable bus services. The establishment of new residential developments in zones furthest from the city is a land use pattern contributing to congestion in the city-centre as people still have to travel to the centre to access essential services.

Based on the responsive indicators identified earlier in this study, schematic of a city strategy used for achieving an integrated sustainable road transport system using coherent strategy elements and policy instruments to boost the synergy between urban and rural areas using the set objectives in Nigeria are as shown in Figure1;

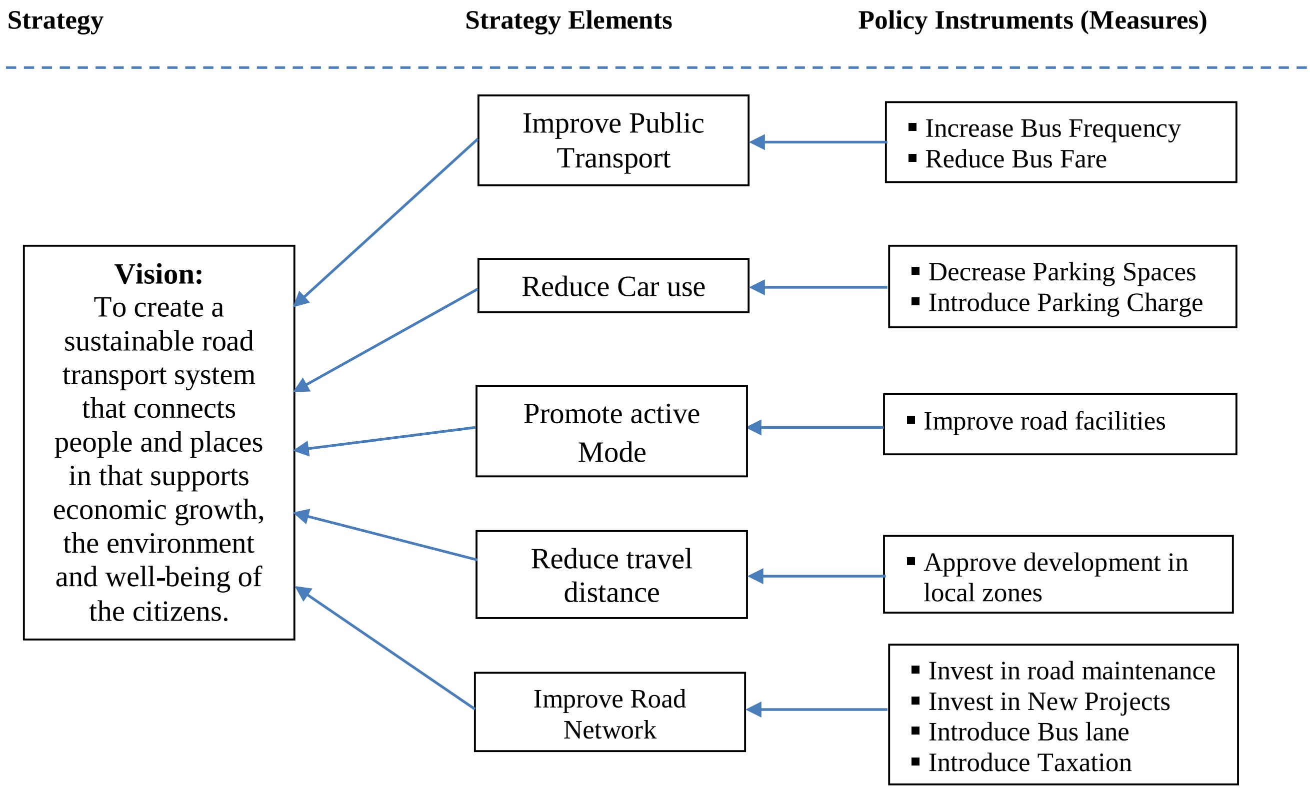

FIGURE 1: SCHEMATIC OF A CITY STRATEGY (KONSULT, 2014)

4 CONCLUSION

This study highlighted strategy elements and policy instruments aimed at creating an efficient and sustainable road transport system that connects people and places to supports economic growth, the environment and well-being of the citizens. The objectives of the study included; to plan for a more livable city for all citizens, to support a vibrant economy, to tackle environmental and social inequity among the dwellers and to reduced contribution of transport to climate change. The proposed policy framework highlighted relevant indicators required with their operationalization, measure of success and justification for achieving the set goals. It is therefore recommended that; policy enforcement agencies should device means to discourage private car use through toll charges (road pricing), parking charges, etc. to boost road transport facilities development and rehabilitations in order to promote rural and urban transport system and the implementation and thorough monitoring and maintenance of road infrastructures.

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EFFECT OF COMPACTION DELAY ON THE CEMENT KLIN DUST STABILIZED LATERITIC SOIL

*Idris, M. H.[1]; Khalid, A. A.[2]; Zango, M.U.[3]
[1]Department of Building Technology, Jigawa State Polytechnic, Dutse
[2]Department of Civil Engineering, Kano University of Science and Technology, Wudil
[3]Department of Civil Engineering, Kano University of Science and Technology, Wudil
*Corresponding author email:mhidris@jigpoly.edu.ng

ABSTRACT
This research investigated effect of 0 to 3 hours compaction delay with an hour intervals, on Soil-Cement Klin Dust (CKD) mixtures 2, 4, 6 and 8percent CKD contents by weight of dry soils were used. Tests carried out on the CKD stabilized soils were the Compaction test (Standard Proctor) and California Bearing Ratio (CBR). Maximum dry densities (MDD) of the mixtures were found to range between 1.82Mg/m^3 for 0% CKD content at 0-hour delay and 1.76Mg/m^3 for 8% CKD contents at 3-hours delay and Optimum Moisture Contents (OMC) were found to be between 17.8% at 0-hour delay and 14.2% at 3-hours delay for 0% and 8% CKD contents respectively. Therefore, MDD decreased linearly and OMC decreased linearly with the increase in CKD content. CBR tests were conducted on the soil-CKD mix (2%, 4%, 6% and 8% CKD) by delaying the compaction time from 0, 1, 2 and 3 hrs. It was observed that time delay and the increasing percentage of CKD decrease the CBR values. Soils with 4, 6 and 8% CKD content were sufficient with compaction delay up to 2 hours. They are good in stabilized form for sub-base. While delay for 2% CKD content must not be more than 2 hours for stabilized sub-grade.

Keywords:*CBR, Cement Kiln Dust (CKD), Compaction delay, MDD and OMC.*

1.0 INTRODUCTION

The increasing rate of human population and the development of construction industry necessitate improvement in current surface soils for construction purposes. Therefore, an essential issue for geotechnical engineers is to achieve required knowledge and information about various methods of surface soil improvement. The technology of soil improvement is one of the most trustable, practical, and cheapest ways of enhancing the resistance, strength, and permeability of soil (Latifi*et al.* 2013). According to Houben (1994) the term Laterite is derived from the Latin word "later", meaning brick. It was first used in 1807 by Buchanan to describe a red iron-rich material found in the southern parts of India. Laterites are widely distributed throughout the world in the regions with high rainfall, but especially in the inter-tropical regions of Africa, Australia, India, South-East Asia and South America, where they generally occur just below the surface of grasslands or forest clearings. Their extension indicates that conditions were favorable for their formation at some point in time in the history of the world, but not necessarily simultaneously in all regions. Laterite is the product of a humid tropical weathering process, current or past, which has the following effects: 1. the parent material is chemically enriched with iron and aluminum oxides and hydroxides (sesquioxides) 2. The clay mineral component is largely kaolinite 3. The silica content is reduced (Netterberg 2014).

The above processes usually produce yellow, brown, red or purple materials, with red being the predominant colour. Patrick *et al* (2011) reported that Podwojewski (1996) and Houben (1994) stated that laterites vary greatly in structure, but can be reduced to the following three structural patterns: (a) The indurate elements form a continuous, coherent skeleton; (b) The indurate elements are free concretions or nodules in an earthy matrix; (c) The indurate elements cement pre-existing materials.

Moses *et. al.* (2012) stated from Liman (2009) that Cement Kiln Dust (CKD) is an industrial waste from cement production. The quantities and characteristics of CKD generated depend upon a number of operational factors and characteristics of the inputs to the manufacturing process. Although the relative constituent's concentrations in CKD can vary significantly, it has certain physical characteristics that are relatively consistent. When managed on site in a waste pile, CKD can retain these characteristics within the pile, while developing an externally weathered crust, due to absorption of moisture and subsequent cementation of dust particles on the surface of the pile. The ability of the CKD to absorb water stems from its chemically dehydrated nature, which results from the thermal treatments it receives in the system. The action of absorbing water (rehydrating) releases a significant amount of heat from non-weathered crust, a phenomenon that can be exploited in the stabilization of poor engineering material. Typical oxide composition of the cement kiln dust is shown in Tables 1.

A delay between mixing the stabilizers with soil and compaction of the mixture leads to a decrease in both density and strength for a fixed compactive effort (Sahaya*et al,* 2009). Most of the time, the time delay is unavoidable, because of any of the followings: sudden raining, delaying of compaction equipment's after mixing, insufficient workers, poor transportations. These make the compaction process as a delayed one (Sahaya*et al*, 2009). These delaying hours considerably affects the strength of stabilized soils (Sahaya*et al*, 2009). Among the most various chemicals used for stabilizing the expansive soils are lime which is very popular. In addition, CKD, fly-ash which is a waste product from cement production and thermal power station are also used for stabilizing expansive soil. Hence study of compaction delay effects is needed, to find the effects of delay between the mixing and compaction on the engineering properties of expansive soil stabilized with cement kiln dust. According to Osinubi (1998) compaction delays have been shown to affect some properties of lime stabilized soil but much have not been stated regarding the influence of compaction delay on CBR and UCS of cement stabilized yellowish brown tropical soil. The objective of this study is to show the effects of compaction delays on the properties of CKD stabilized reddish brown tropical soil.

TABLE 1: BASIC PROPERTIES AND TYPICAL OXIDE COMPOSITION OF THE CEMENT KILN DUST

Oxide	Cao	Al_2O_3	SiO_2	Fe_2O_3	Mn_2O_3	Na_2O	K_2O	pH	Gs
Concentration (%)	50.81	4.71	-	1.92	0.002	0.001	1.32	11.2	2.22

Source Moses et.al. (2012)

2.0 MATERIALS

Soil: The soil sample used was collected from borrow pit of Dantata and Sawoe Construction Company located along Kano-Maiduguri Road (Gaya Road along Kademi-Saya-Saya Road with the coordinate 11^0 48' 35''N and 8^0 54' 28''E.)

Cement kiln dust: The cement kiln Dust sample used in this study was collected from ASHAKA CEMENT COMPANY NIGERIA, located at Ashaka city, Funakaye, Gombe State, Nigeria with latitude of 10^0 53' 20'' and longitude of 11^0 30' 59''

2.1 METHODS

Standard proctor compaction was used. An Air dried soil samples was passed through British Standard sieve with 4.76 mm aperture mixed with 0, 2, 4, 6 and 8% cement kiln dust by weight of dry soil were used. British Standard Light compactive effort was carried out in accordance with BS 1377 (1990).

To find the effect of compaction delay on the various properties of the stabilized soil, it was mixed with the required percentage of CKD. Predetermined amount of water was added to the mix to achieve the water content of the mix. The wet mix was left undisturbed for a period of 0hr (no delay), 1, 2 and 3hrs. During the delay period, care was taken to avoid evaporation (loss) of water. After the required period of delay, CBR test was prepared, by applying Standard Proctor Compactive effort.

3.0 RESULTS AND DICUSSION

3.1 INDEX PROPERTIES

Laboratory tests were conducted to determine the index properties of the natural soil and soil-CKD mixtures in accordance with BS 1377 (1990). A summary of the soil index properties is presented in Table 2.

TABLE 2. CHARACTERISTICS PROPERTIES OF THE NATURAL LATERITIC SOIL

INDEX PROPERTIES OF LATERITIC SOIL	Quantity
Liquid Limit (%)	32.00
Plastic Limit (%)	25.23
Plasticity Index (%)	6.77
Linear Shrinkage Limit (%)	10.00
Specific Gravity	2.54
Natural Moisture Content (%)	13.30
% Passing sieve NO. 200	0.58
AASHTO Classification	A-2-4 (0)
USCS Classification	SW
Group Name	Well graded sand
MDD (Mg/m^3)	1.82
OMC (%)	14.5
CBR (Unsoaked)	35.8
CBR (24-hrs Soaked)	15.5

3.2 SPECIFIC GRAVITY TEST

The specific gravity of the soil depends on the amount of sand and also depends on their mineral constituents and mode of formation of the soil. According to specification, a good lateritic material should have specific gravity ranging from 2.5 to 2.75 Olafusi*et al.* (2012). The sample has specific gravity of 2.54.

3.3INDEX PROPERTIES

The soil is classified as A–2–4 sub-groups according to AASHTO classification system and SW under USCS classification system. The liquids limit decreased from 32.0 to 30.5% when cement kiln dust was increased from 0 to 8% of the samples. This can be considered to be as a result of the addition of cement kiln dust, which has less affinity for water and yields a decrease liquid limit. The results of the Atterberg's limits test (Liquid Limits LL, Plastic Limits PL and Plastic Index PI) on the sample are shown in Figure 1. The LL, PL and PI of the natural soil samples are 32, 25.23 and 6.77% respectively. According to Whitlow (1995), liquid limit less than 35% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity and greater than 90% extremely high plasticity. This shows that our sample has low plasticity. Addition of 2, 4, 6 and 8% CKD to the sample caused changes in the liquid limits and plastic limits, which are shown in Figure 1. The plasticity indices of samples decreased from 6.77 to 6.51 at 8% of CKD and increased from 6.77 to 6.99, 7.15 and 7.90 at 2, 4 and 6% of CKD respectively. These reductions in plasticity indices are indicators of soil improvement

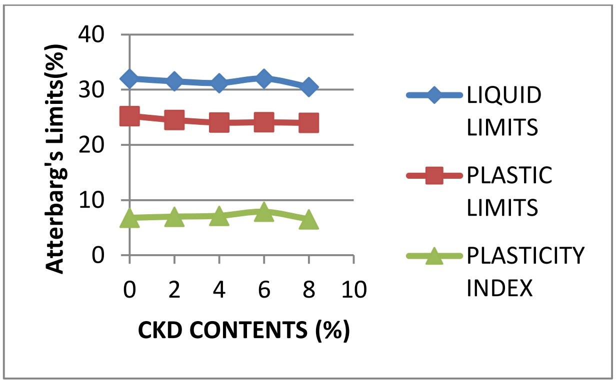

FIGURE 1: ATTERBERG'S LIMITS

3.4 MAXIMUM DRY DENSITY

Figure 2 shows the relationship between Maximum Dry Density (MDD) of the Cement kiln dust stabilize lateritic soil and compaction delay after mixing. It was showed that the MDD decreased with increase in elapsed time after mixing. As soon as the water was added to the soil-CKD mix, hydration reaction kicks off. This resulted in the bonding of the particles in the loose state and disruption of particles was required to increase the density of the soil. Consequently, some of the hardening effect of the cement will be lost and in addition, part of the compactive effort was directed in overcoming the cementation. Thus, the MDD decreased with increase in compaction delay and it is dependent on the rate of hydration of the CKD.

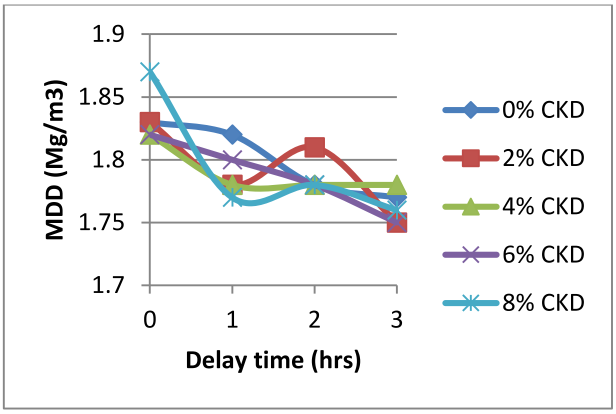

FIGURE 2: VARIATION OF MDD WITH DELAY TIME

According to Shetty (2005), the rate of hydration is very rapid at the initial stage but reduces with time. The MDD decreased from 1.83Mg/m^{3}at 0% CKD content at 0-hour delay to 1.76Mg/m^3 at 8% CKD at 3-hours respectively; these indicate that for a given level of compaction, the stabilized soil has lower MDD than that of the unstabilized soil (Sherwood 1993). The reduction in the MDD can be attributed to the reaction between CKD and the fine fraction as pozzolanic components in which they form clusters like coarse aggregates. These clusters occupied larger spaces thus increasing their volume and in consequent decreasing the MDD.

3.5 OPTIMUM MOISTURE CONTENT

The relationship between Optimum Moisture Content (OMC) of the CKD stabilized lateritic soils with compaction delay after mixing is shown in Figure 3. It was observed that OMC of the lateritic soils decreased steadily with increase in compaction delay from 17.8 to 14.5% at no CKD content and from 19.9 to 13.5% at 2% of CKD. It was also observed OMC of the lateritic soils decreased steadily with increase in compaction delay. This was because the hydration reaction kicks off rapidly on the addition of water and continues to decrease with time, thus the demand for water in the system also reduced continuously. If there is no free flow of water within the soil-CKD mix, the hydration reaction uses up the water until too little will be left to lubricate the soil particles and relative humidity decrease within the mix (Osinubi, 1998).

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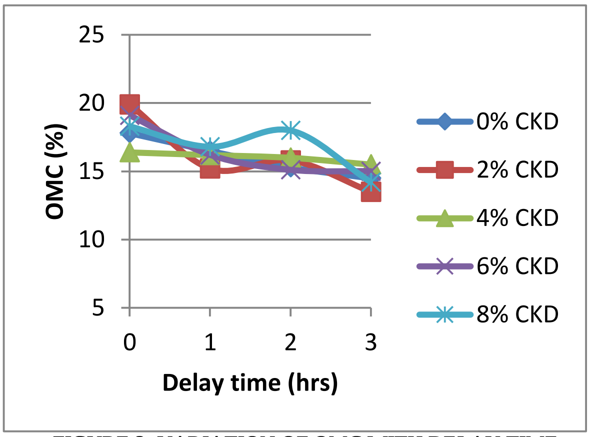

FIGURE 3: VARIATION OF OMC WITH DELAY TIME

This decrease in OMC can also be probably due to self – desiccation in which all the water was used, resulting in low hydration. When no water movement to or from CKD – paste is permitted, the water is used up in the hydration reaction, until too little is left to saturate the solid surfaces and hence the relative humidity within the paste decreases. The process described above might have affected the reaction mechanism of CKD treated lateritic soil (Osinubi and Stephen 2007, Moses (2008).

3.6 CALIFORNIA BEARING RATIO

The variation of California bearing ratio (CBR) with CKD content for soil–CKD mixtures and compaction delay after mixing is shown in Figure 4 and Figure 5. The Unsoaked CBR of lateritic soil rose from 35.8% at no CKD content to 7.5% at 3-hours delay and also that of the lateritic soil with 2% of CKD rose from 38.1% to 7.8% and from 39.6% to 7.5% at 4% CKD content at 3-hours delay time respectively. It also increase from 39.8 to 7.8% at 6% CKD with delay of 3-hours and that of 8% CKD increase from 41.8% to 7.5% by delay of 3-hour respectively

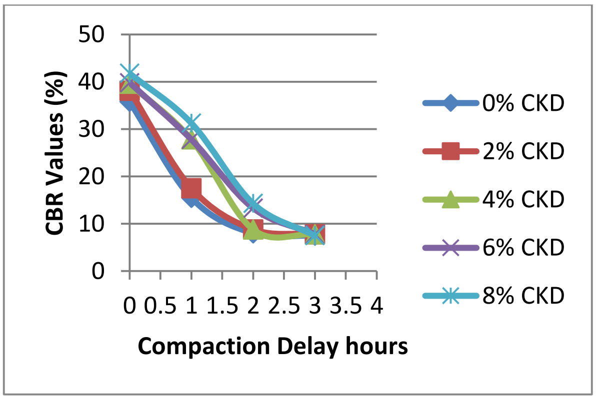

FIGURE 4: VARIATION OF CBR (UNSOAKED) WITH DELAY TIME

Therefore, the Soaked CBR values shown in Figure 5 also decrease by increasing of the delay time, it rose from 15.5 to 1.8% at no CKD content, 16.5 to 2.2% at 2% CKD content, 17.3 to 3.0% at 4% CKD content, also from 17.5 to 2.7% and 19.3 to 2.1% at 6% and 8% CKD contents up to 3-hours delay respectively. It is obvious that there was an appreciable improvement in strength properties of lateritic soil at a little addition of CKD.

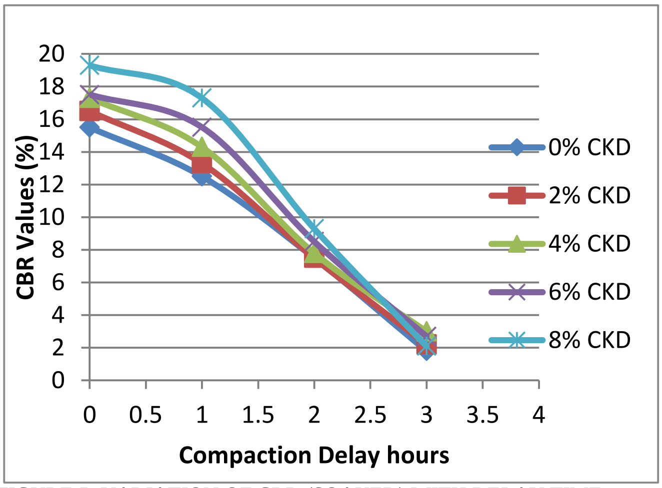

FIGURE 5: VARIATION OF CBR (SOAKED) WITH DELAY TIME

Figure 4 and 5 show that strength properties decreased with increase in compaction delay for lateritic soil. This was because, as soon as water was added to the soil-CKD mix, the cement began to hydrate and it was therefore desirable to compact as soon as mixing was completed. When this was not done, not only that some of the hardening effects of the cement properties in the CKD would be lost but in addition extra compactive effort will be required to break down the cemented bonds that have been formed. Both of these effects together will lead to serious loss in strength.

4.1 CONCLUSION

The following conclusions were drawn from this research:

1. The natural Lateritic soil was classified as A – 2 – 4 or SW in the AASHTO and Unified Soil Classification System (USCS), respectively. Soils under these groups are of excellent to good engineering benefit.
2. The optimum moisture content decreased with increasing Cement Kiln Dust content.
3. The maximum dry density decreased with increasing Cement Kiln Dust content.
4. The compaction and strength characteristics decreased with increase in compaction delay.
5. Compaction delay affects the CBR value for all the percentages of CKD addition. The CBR value reduces with increase in time delay. However, the rate of reduction in CBR value decreases as the CKD content increases.
6. 4, 6 and 8% cement kiln dust content will be adequate for lateritic soil.

4.2 RECOMMENDATION

The minimum conventional values of CBR, is judged between 7% to 20% and from 0 to 7% as recommended by the asphalt institute (1962) for highway sub-base and sub-grade material, respectively. Therefore, based on this, lateritic soil with 4, 6 and 8% CKD with elapse time not more than 1-hours is a good sub-base material and lateritic soil with 2% CKD with elapse of 3-hours can be used as sub-grade.

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PREDICTING SOIL MOISTURE AND SOIL TEMPERATURE IN A TROPICAL PEATLAND USING WATER TABLE DEPTH, SURFACE TEMPERATURE AND RAINFALL

Adesiji, A.R.[1], Saidu, M.[1], Musa, J.J[2]
[1] Department of Civil Engineering, Federal University of Technology, Minna
[2]Department of Agricultural and Bioresources Engineering, Federal University of Technology, Minna
*Corresponding author email: adrichard01@yahoo.co.uk

ABSTRACT
Degradation of tropical peatland largely depends on water table depth and other soil physical properties like soil moisture and soil temperature. Empirical studies indicate positive relationships between water table depth, soil moisture and soil temperature. This paper aims to further investigate the relationship between soil moisture and soil temperature variability in time as a function of soil depth and their relationship with water table depth. Linear models for prediction of soil moisture and soil temperature at 70 cm using surface soil moisture and soil temperature measurements, rainfall and surface temperature are examined. In all the four plots used in the study, seasonality of soil moisture and soil temperature was confirmed as they strongly correlate with rainfall. Both soil moisture and soil temperature at 5 cm, as a result of storm events, experienced more fluctuations compared to the values recorded at 70 cm. The linear models give better prediction capabilities as predicted and observed values give good fits and coefficients of determination, R^2, is mostly above 0.5 with agreements within 0.05 – 0.1.

Keywords: *Peatland degradation , Soil moisture, Soil temperature, Tropical peatland, Water table depth*.

1 INTRODUCTION

Peatlands, according to Parish et al., (2008), have been known to be long-term carbon stores storing this natural resource for thousands of years. But keeping these ecosystems functioning as carbon stores is of high importance to global environmental sustenance and also depends on certain factors like water table depths, soil temperature and soil moisture (Carlson *et al.*, 2015), among other factors. Peats have been formed as a result of microbial transformation of soil organic matters to mineral forms which, according to Suseela *et al.* (2012), Preston *et al.* (2011) and Hagedorn and Joos (2014) has been largely influenced by soil temperature and soil moisture. According to Lakshmi *et al.* (2003), for the proper understanding of land surface- atmosphere interactions, soil moisture and temperature estimation is of critical importance. They further stated that soil moisture plays major roles in water and energy cycle which serves as one of the crucial variables of peatland water balance. In water cycle, soil moisture helps in determining parts of precipitation partitioned into other water cycle parameters like runoff, surface storage and infiltration. The role of solar energy in peatland water balance cannot be underestimated. Lakshmi *et al.* (2003) further stated that fluxes of outgoing longwave, sensible, and peatland heat surface temperature is attributed to surface temperature. With the points raised above, soil moisture and soil temperature are known to be interrelated.

Peatlands are known to be waterlogged which makes them unsuitable for agricultural activities except the water table is lowered to enhance their use for agriculture and for the purpose of peat extraction (Maljanen et al., 2012; Querner *et al.*, 2012). In order to achieve this, drainage systems are put in place which controls the water table depths (Macrae *et al.*, 2013; Potvin *et al.*, 2015) and peat compaction in order to aerate the crop root zone while increasing the peat soil bulk density (Melling et al., 2012). Excessive lowering of water table as a result of peatland drainage has been known to have its own negative influence not only on the peatlands management (Adesiji *et al.*, 2015) but also on peatland soil moisture (Couwenberg 2009). Soil moisture has been identified as a control on peat carbon loss (Jauhiainen *et al.*, 2008; Couwenberg 2009). This is because it mediates the volume of peat substrate exposed to oxygen, thereby influencing microbial activity and decomposition (Carlson *et al.*, 2015; Husen *et al.*, 2014).

Influence of water table depth in peatland biogeochemical roles has been studied. It has been established that drainage systems in the peatlands control its water table depths (Hefting et al., 2004; Katimon et al., 2013) and that with excessive lowering of water table, soil carbon, methane and other GHGs are lost further fuelling the environmental degradation (Strack *et al.*, 2008; Bhullar *et al.*, 2013; Reddy 2015). Also the excessive drying of peats as a result of low or change in water table depths by compression and oxidation results in irreversible drying of peats and its change in volume, otherwise known as subsidence (Whittington and Price

2006; Couwenberg and Hooijer, 2013). Salm *et al.*, (2012) and Carlson *et al.*, (2015) attributed the emission of carbon and methane from peatland surface to depth to groundwater table and soil temperature. It has also been established that lowering the water table depths changes peatlands from carbon sinks to carbon sources, by largely reversing the C flux into net CO_2 emissions, while CH_4 emissions decrease (Furukawa et al., 2005; Couwenberg 2011). Therefore, the variation in environmental conditions and soil properties like soil temperature and soil moisture that drive these emissions and changes in peatland water table depth become important factors. Therefore, the need to study these variations for proper understanding of peatland biogeochemical activities and groundwater table fluctuation is essential. Wen et al., (2006) reported the importance of temperature and soil moisture in determining emissions of CO_2 from ecosystems. The reports concluded that seasonal fluctuation of terrestrial ecosystem respiration is accounted for by variation in temperature and soil moisture. Bellisario *et al.*, (1999) also attributed the variation in soil temperature and soil moisture to the peatland water table fluctuations which in turn determine the emissions of methane and other GHGs from the peat surface.

Thus, in this study, variability and relationships between water table depth and soil moisture and soil temperature at two different depths were considered to ascertain the dependence of one over the other. The daily rainfall measurement, water table depth, and soil temperature at two different depths of 5 cm and 70 cm were also used in predicting soil moisture and soil temperature at 70 cm. In achieving the aim of this study, therefore, the objectives will be:

(1) to examine the hypothesis that peatlands soil moisture and soil temperature are seasonal and are statistically significant irrespective of soil depth.

(2) to predict, using a model, the peatland soil moisture (at a specific depth) using soil temperature, water table depth and rainfall as independent variables or predictors;

2 MATERIALS AND METHODS

2.1 STUDY AREAS

The study area is located in Sepang, the state of Selangor, Malaysia at Kuala Langat South Forest Reserve area, between latitude 02° 43'N and longitude 101° 39'E bounded to the West by Straits of Malacca. The study area experience tropical climate and high humidity with an annual rainfall between 2500 – 3000 mm. The monthly air temperature ranges between 32 °C and 36 °C, with the highest value recorded in May each year. For the 2014 water year under study, the lowest rainfall recorded was in February as 7.0 mm and highest in April as 437 mm (Figure 1). The average annual rainfall for the study year, 2014 was 2348 mm from the Malaysian Department of Irrigation and Drainage (DID) gaging station 2918101 in the study area. The study ran from June, 2014 to December, 2014, with both months inclusive. During this study period, the highest rainfall depth observed was in October as 338.5 mm as shown in Figure 1. Properties of peat soil available in the study area as shown in Table 1. The vegetation of the study area is characterized with oil palms since the peat swamp forests were converted to oil palm plantation in 1978.

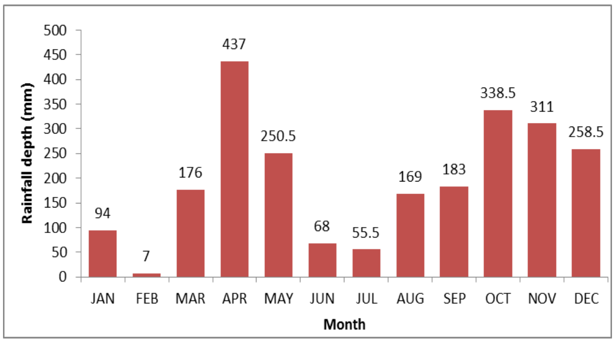


FIGURE 1: MONTHLY RAINFALL DEPTH FOR 2014 IN THE STUDY AREA

TABLE 1: PROPERTIES OF PEAT SOIL IN THE STUDY AREA

Index properties	Range	Average
Natural moisture content (%)	150-600	200
Peat pH.	3.10 - 3.82	3.65
Peat carbon content (%)	11.66 - 51.22	45.32
Peat nitrogen content (%)	0.24 - 1.93	1.45
Peat sulphur content (%)	0.07 - 0.22	0.136
pH of groundwater	3.09 - 4.4	3.42
Bulk density (g/cm^3)	0.18 - 0.397	0.276

2.2 SITE SELECTION

The study area of 10,000 ha was divided into four different plots, each plot named according to the age of plantation, such as; 2000, 2002, 2006, and 2010/1978. The plots were further sub-divided into sampling points and labeled as; 2000-A2, 2000-B1, 2000-B2, 2000VP-1, 2000VP-3, 2000VP-5, 2002-1, 2006-1, 2006-2, and 2010-1, making 10 sampling points in the entire study area (6 sampling points in 2000-Year of plantation, 2 in 2006-Year of plantation, and 1 sampling point each in 2002 and 2010 years of plantation).

2.3 EXPERIMENTAL DESIGN AND SET UP

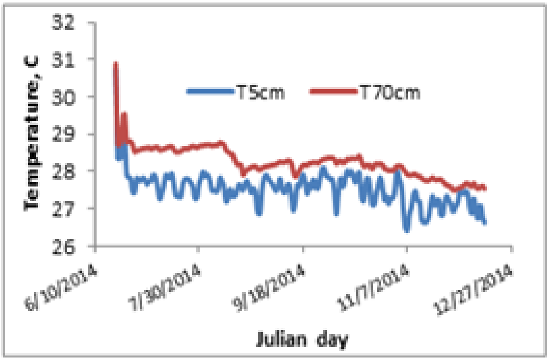
Surface and subsurface measurements were carried to determine water table depth, soil moisture and soil temperature which are in turn used as predictors with daily rainfall to predict the daily soil moisture. For soil moisture determination, 10 cm-long, 10HS soil moisture probes with 1 litre area of influence that estimates the volumetric water content VWC of the soil within that volume of influence were installed in undisturbed soil samples at two separate depths of 5 cm and 70 cm in all the 10 study sites.

The soil moisture probes and soil temperature sensors were installed by burying them into the soil. The installation began by excavating the soil to the depth of 80 cm below the ground surface in all the chosen stations. Two depths were chosen for the installation; 5 cm and 70 cm. The depths of 5 cm and 70 cm were chosen so as to check the influence of soil moisture and soil temperature with soil depth at near soil surface and deep beneath the soil. At the 70 cm depth, the soil moisture probe and soil temperature sensor were both inserted into the undisturbed soil, making sure the prongs are fully buried up to the black overmolding. The sharpened tip of each prong is in place in order to ensure easier pushing of the probe into the soil mass. It was ensured that there was a good soil-to-sensor contact at both chosen depths before the trench was backfilled ensuring that the bulk density of the backfilling materials was the same as that of the surrounding soil. Connecting sensor cables were well protected with a flexible PVC casing above the ground surface before being connected to the terminal block situated close to the installed sensors. The connecting sensor cables were well insulated with PVC in order to prevent them from environmental hazards. Water table depths were measured with the aid of pressure transducers inserted through the tube wells into the groundwater. Soil temperature and soil moisture at both 5 cm and 70 cm are compared to check their variations with soil depth and what influences their changes. Surface air temperature (SAT) which is the maximum daily air temperature in the study area was obtained from Malaysia Meteorological Department (MMD) station 48650 in Sepang KLIA adjacent to the study area. Statistical analysis (IBM SPSS statistics 21) was used to analyze the patterns of correlations among the variables.

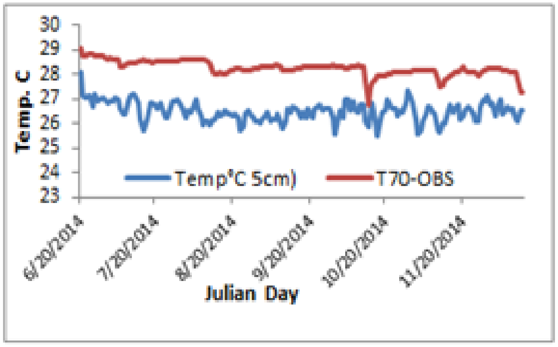
3 RESULTS AND DISCUSSION

3.1 SOIL TEMPERATURE

Seasonal variations in soil temperature at 5 cm and 70 cm in the selected study plots covering the study area are shown in Figure 2 (a-d). In all the study plots, the average daily soil temperature recorded at the surface, 5 cm, ranged from 22.80 ºC to 30.21 ºC 33.11 ºC while at 70 cm, the average daily soil temperature ranged from 23.20 ºC to 34.50 ºC (Figure 2 a-d). A seasonal variation of soil temperature as shown in Figures 2(a-d) revealed that soil temperature seems to become more constant at 70 cm compared to soil temperature at 5 cm which shows the correlation of soil depth with soil temperature ($P < 0.05$, $n = 178$). In other words, soil temperature at 5 cm exhibited prolonged fluctuations compared to soil temperature at 70 cm.



(a)



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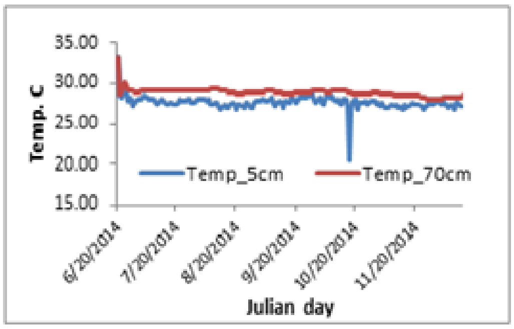

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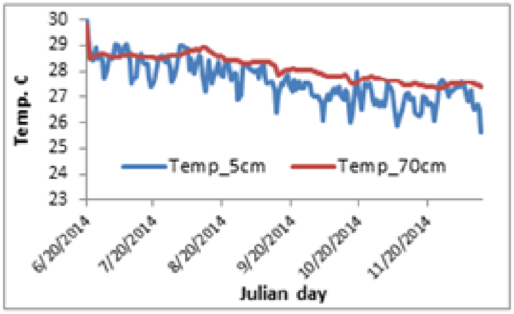
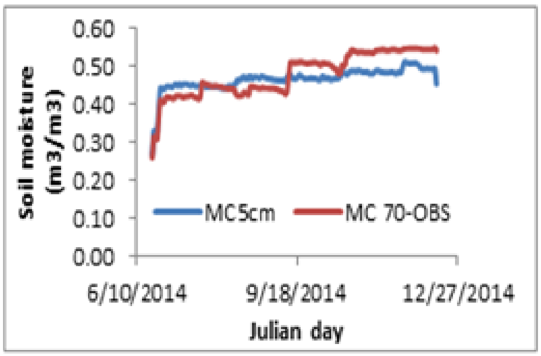


(d)

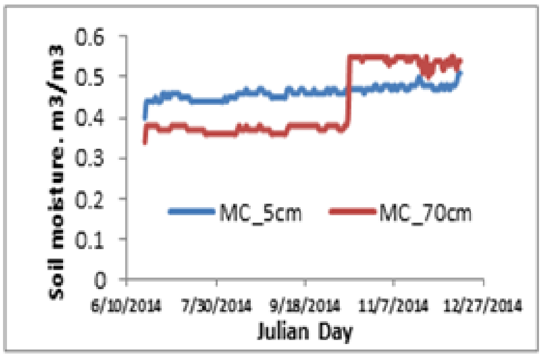
Figure 2a-d: Seasonal Variations of Soil Temperature at 5cm and 70 cm Depths from Selected Study Plots

3.2 SOIL MOISTURE

Soil moisture at 5 cm experienced much fluctuation compared to the soil moisture at 70 cm, varying from 0.46 m3/m3 to 0.54 m3/m3 while the average soil moisture at 70 cm varied between 0.51 m3/m3 to 0.53 m3/m3 (Figure 3 a-d). Soil moisture at both depths showed similar patterns with soil temperature. Soil moisture at 5 cm, like soil temperature at 5 cm, is lower than that of 70 cm, with the exception of period with sizeable amount of rainfall as represented with the intercept between the two graphs in 2002 study plot (Figure 3b). Like the soil temperature at 70 cm, soil moisture at 70 cm was also observed to be more constant compared to the soil moisture at 5 cm which could be due to the contribution from groundwater body or base flow.



(a)



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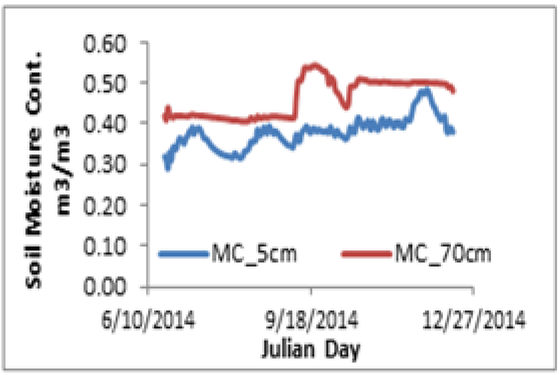

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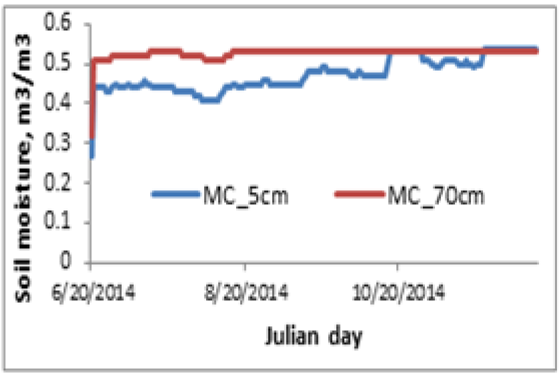

(d)

FIGURE 3A-D: SEASONAL VARIATIONS OF SOIL MOISTURE AT 5CM AND 70 CM DEPTHS FROM SELECTED STUDY PLOTS

3.3 PREDICTION OF SOIL MOISTURE AND TEMPERATURE AT 70 CM

Water table depth, soil moisture at 5 cm and temperature (at 5 cm and 70 cm) with rainfall and surface temperature were used in models to predict soil moisture and soil temperature at 70 cm soil depth in all the sampling points. In this approach, all the six study areas under age '2000' were grouped together as a single plot. The same went for site '2002', '2006' and site '2010'. Thus, 4 equations (one from each of the 4 study areas of 2000, 2002, 2006, and 2010) predicting the values of soil moisture and soil temperature were developed for prediction. From the models developed in Tables 2 and 3, the empirical equations predicting these variables will take the form of Equations 1 and 2;

$$MC_{70} = [K]_{MC} + [A]\mathrm{TEMP}_5 + [B]\,\mathrm{TEMP}_{70} + [C]\mathrm{MC}_5 + [D]\mathrm{RAINFALL} + [E]\mathrm{WATB} + [F]\mathrm{SAT} \quad [1]$$

$$TEMP_{70} = [K]_{MC} + [A]MC_5 + [B]\,\mathrm{TEMP}_5 + [C]\mathrm{MC}_{70} + [D]\mathrm{RAINFALL} + [E]\mathrm{WATB} + [F]\mathrm{SAT} \quad [2]$$

Where MC70 represents soil moisture at 70 cm in m3/m3; TEMP70, Temperature at 70 cm in oC; WATb, Water table depth in cm, TEMP5, Temperature at 5cm in oC;; MC5 represents soil moisture at 5 cm in m3/m3; SAT, Surface air temperature in oC and a, b, c, d, & e are coefficients. The intercept K corresponds to the theoretical soil moisture, soil temperature or water table depths of the study area depending on which parameter is being considered while the rest of the coefficients represent the expression of the independent variables.

For soil moisture at 70 cm as a dependent variable, soil moisture at 5 cm, soil temperatures at 5 cm and 70 cm, rainfall depth, water table depth and surface air temperature, (SAT) are the environmental parameters accounting for its variance (Table 2, equations 1-4). Thus the model is a combination of in situ data (water table depth, soil moisture and soil temperature at 5 cm and 70 cm) and meteorological data (daily rainfall and surface air temperature, SAT) which were used as observed data in predicting and computing accurately the daily soil moisture and temperature at 70 cm. The models utilized these data collected for prediction within the study period. The models predicting these variables (soil moisture and temperature at 70 cm) are as shown in Tables 2 and 3.

The results of the comparison of soil moisture and soil temperature for observed and predicted values using 45° line of best fit for all the four study plots are as shown in Figures 4 and 5 respectively. From the figures, it can be deduced that there is a very reasonable agreement between the observed and predicted soil moisture and temperature in all the study plots.

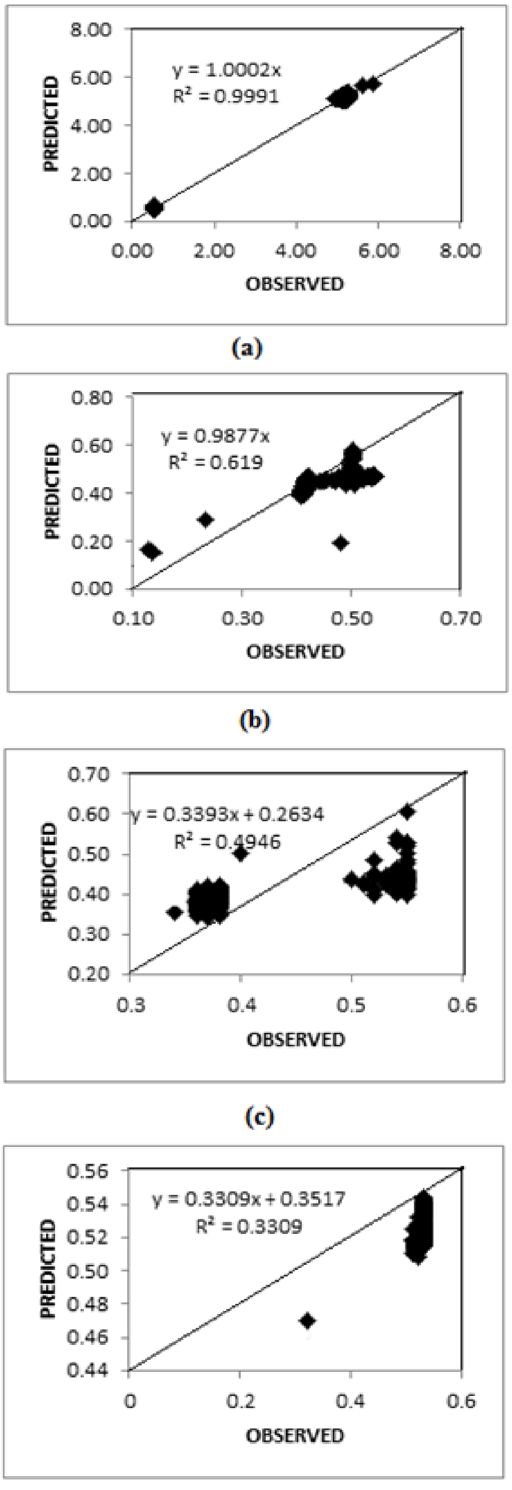

FIGURE 4: COMPARISON OF THE OBSERVED AND PREDICTED SOIL MOISTURE AT 70 CM FOR THE STUDY PLOTS. A) 2000, B) 2002, C) 2006, AND D) 2010

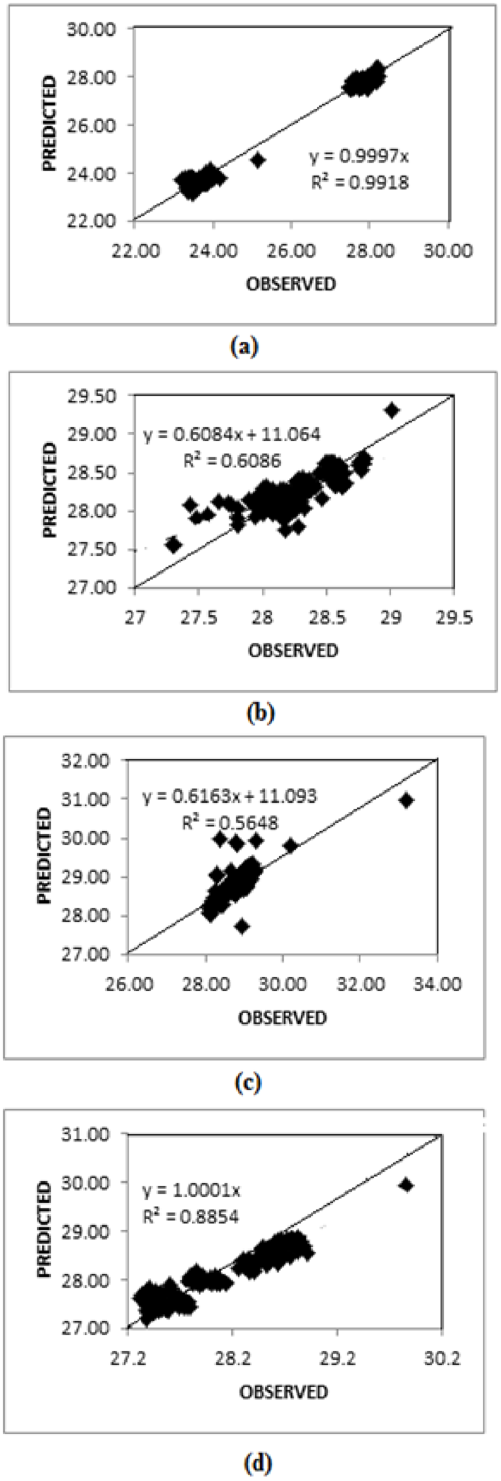

FIGURE 5: COMPARISON OF THE OBSERVED AND PREDICTED SOIL TEMPERATURE AT 70 CM FOR THE STUDY PLOTS. A) 2000, B) 2002, C) 2006, AND D) 2010

3.4 STATISTICAL ANALYSIS

Pearson's measures of correlation were used in investigating the strengths of association between rainfall and soil temperature at both depths. The result showed a negative significant difference ($P<0.01$, $n = 178$) between rainfall and soil temperature, a result indicating percolating rainfall cools the soil temperature both at the surface and at the deep layers. This explains the reasons behind the downwards trends in the soil temperature at both depths with high storm events. There is also a strong Pearson's correlation coefficient between soil temperature at 5 cm and soil temperature at 70 cm depth ranging from 0.719 – 0.813 ($P<0.01$, $n = 178$).

Positive significant correlation between water table depths and all other parameters like soil moisture and soil temperature at 5 cm and 70 cm, and rainfall ($P<0.01$, $n = 178$, ANOVA), except surface air temperature (SAT) was observed in plot '2000'. All other study plots, however, showed positive correlation ($P<0.01$, $n = 178$, ANOVA) between rainfall and water table depths. It further revealed that rainfall and SAT are significantly correlated with soil surface temperature and soil moisture at 70 cm in all the plots ($P<0.01$, $n = 178$, ANOVA). This, therefore, implies that surface air temperature, SAT could serve as a proxy of the soil temperature beneath the soil surface. However, Water table depth shows statistical significance with all the parameters ($P<0.01$, $n = 178$, ANOVA), except with SAT at both 2000 and 2002 study plots.

4 DISCUSSION

In this study, it was observed that soil moisture and soil temperature at 5 cm below the peat surface experienced much fluctuation compared to the observed values at 70 cm. Fluctuations experienced by soil moisture at the surface could be attributed to the environmental effect, water holding capacity of the soil at oil palm root zones (Hökkä *et al.*, 2013) or the moisture accumulation at the root zones which is dependent of storm events and soil bulk density. On the other hand, fluctuations experienced by soil temperature at peat surface could be due to precipitation and varying surface air temperature which has little or no effects on deep soil temperature (Feng and Hu, 2005). Soil temperature at the surface has been observed to be the source of deep (terrestrial) temperature (Feng and Hu, 2005). The seasonality of soil moisture and temperature was therefore enhanced by their correlation with precipitation (Feng and Hu, 2005; Dong et al., 2007; Cong and Brady, 2012) and some other factors like SAT, rainfall intensity, soil physical properties and groundwater levels which is consistent with Schlosser and Milly (2002) and Carretero and Kruse (2012).

The positive significant correlation between soil moisture and soil temperature at 5 cm and 70 cm with water table depths and rainfall means that water table depth is influenced by storm events with significant effects on soil moisture and temperature at both depths (Moore et al., 2012; Yi et al., 2015). Also the strong significance change in rainfall ($P < 0.05$, $n = 178$) with water table level means the rise in water table level in the study areas was largely as a result of primary maximum rainfall that occurred between October and November, 2014. Mean water table depth of 90.80 cm below the peat surface was observed in April as against 27.16 cm below the peat surface in December which largely reflects the influence of storm events on water table depth.

The positive correlation between rainfall and water table depths could also be attributed to the contribution of baseflow to the groundwater recharge in the event of rainfall which is consistent with Brixel (2010). In other words, higher soil moisture recorded at 70 cm in all the study plots could be due to the nearness to groundwater table. However, the decline in soil temperature at both depths with increased rainfall (Feng and Hu, 2005; Cong and Brady, 2012) could largely be attributed to the cooling effects as a result of advent of storm events which cools the soil temperature both at the surface and at the deep layers.

This study establishes linear relationships between soil moisture and soil temperature at 70 cm (predicted and observed) using in-situ measurements of soil moisture and soil temperature at 5 cm with meteorological data; rainfall and surface air temperature. The use of these linear relationships in all the study plots for prediction gives a better prediction capability of soil moisture and temperature as coefficients of determination, R2, is mostly above 0.5 and the agreements are within 0.05 – 0.1 of each other (i.e 95% and 99% confidence intervals). A conclusion should review the main points of the paper and should state concisely the most important propositions of the paper. It should state the author's views of the practical implications of the results.in addition to the deductions that can be made from the results. Do not replicate the abstract as the conclusion. A conclusion might also elaborate on the importance of the work or suggest applications and extensions.

5 CONCLUSION

The relationship between soil moisture, soil temperature, rainfall and water table depth in a drained tropical peatland in Southeast Asia was evaluated. Seasonal variation in soil moisture and soil temperature was observed. The two soil properties, irrespective of depth, were observed to be correlated significantly with rainfall. Negative significant correlation between soil temperature and soil moisture in the two depths examined was also observed. In other words, inverse relationships were observed between soil moisture and soil temperature at both depths considered which means following a rainfall event, increase in soil moisture results in decrease in soil temperature. The soil moisture and soil temperature at the surface (0-5 cm) were observed to be influenced by storm events which also explain why there are fluctuations in the surface soil moisture and soil temperature. Rainfall percolating into the soil was also observed to influence both the surface and deep soil temperature.

In conclusion, peatland soil moisture and temperature have been observed to be seasonal which majorly are influenced by storm events. Linear relationships between them at 70 cm below peat surface have also been observed. These linear relationships, which predict the estimation of both the soil moisture and soil temperature at 70 cm yields good results and give better prediction capacities for their estimation irrespective of soil depth.

ACKNOWLEDGEMENTS

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TABLE 2: PREDICTION OF SOIL MOISTURE @ 70 CM USING SOIL PROPERTIES AND METEOROLOGICAL DATA (X0=TEMP@5CM; X1=TEMP@70CM; X2=SOILMOIST@5CM; X3=RAINFALL; X4=WATERTABLE; X5= SAT)

Equation Nos	Study Plot	Dependent Variables (y)	Independent Variables (x)	R^2	Equations of prediction	n
1	2000	Soil moisture @ 70 cm	WaterTableLev; Rainfall; Temp@5cm; Temp@70cm; WaterTab@5cm	0.999**	y= *3.067 - (0.105 * x0) + (0.00850 * x1) + (0.900 * x2) - (0.000322 * x3) - (0.00268 * x4) - (0.00409 * x5)*	178
2	2002	Soil moisture @ 70 cm	WaterTableLev; Rainfall; Temp@5cm; Temp@70cm; WaterTab@5cm	0.0848**	*y = 3.910 + (0.0104 * x0) - (0.139 * x1) - (0.477 * x2) - (0.000352 * x3) + (0.000379 *x4) + (0.0111 * x5)*	178
3	2006	Soil moisture @ 70 cm	WaterTableLev; Rainfall; Temp@5cm; Temp@70cm; WaterTab@5cm	0.772**	y = *-0.566 + (0.00213 * x0) + (0.0186 * x1) + (1.011 * x2) + (0.0000545 * x3) + (0.000489 * x4) + (0.00104 * x5)*	174
4	2010	Soil moisture @ 70 cm	WaterTableLev; Rainfall; Temp@5cm; Temp@70cm; WaterTab@5cm	0.331**	y = *0.279 - (0.00421 * x0) + (0.00797 * x1) + (0.344 * x2) - (0.000139 * x3) - (0.000144 * x4) - (0.000379 * x5)*	178

*** significant at 0.05% level of probability

TABLE 3: PREDICTION OF SOIL TEMPERATURE @ 70 CM USING SOIL PROPERTIES AND METEOROLOGICAL DATA (X0=TEMP@5CM; X1=SOILMOIST@5CM; X2=SOILMOIST@70CM; X3=RAINFALL; X4=WATERTABLE; X5=SAT)

Equation Nos	Study Plot	Dependent Variables (y)	Independent Variables (x)	R^2	Equations of prediction	n
1	2000	Soil Temp.@ 70 cm	Temp@5cm; MoisContent@5cm, MoisContent@70cm; Rainfall; WatTab	0.992**	*y= 12.996 + (0.536 * x0) - (0.543 * x1) + (0.0620 * x2) + (0.000916 * x3) - (0.00462 * x4) + (0.0268 * x5)*	178
2	2002	Soil Temp.@ 70 cm	WaterTableLev; Rainfall; WaterTab @5cm;Temp@70cm; WaterTab@70cm	0.608**	*y = 31.870 + (0.114 * x0) - (14.636 * x1) - (0.297 * x2) - (0.000704 * x3) + (0.00132 * x4) + (0.00571 * x5)*	178
3	2006	Soil Temp.@ 70 cm	WaterTableLev; Rainfall; WaterTab @5cm;Temp@70cm; WaterTab@70cm	0.661**	*y = 25.458 + (0.166 * x0) - (3.762 * x1) + (1.117 * x2) + (0.00238 * x3) - (0.00767 * x4) - (0.00288 * x5)*	174
4	2010	Soil Temp.@ 5 cm	WaterTableLev; Rainfall; WaterTab @5cm;Temp@70cm; WaterTab@70cm	0.897**	y = *26.802 + (0.192 * x0) - (6.761 * x1) + (1.084 * x2) + (0.00131 * x3) - (0.00337 * x4) - (0.0346 * x5)*	178

*** significant at 0.05% level of probability

RELIABILITY ASSESSMENT OF DEFLECTION FOR CRANE RUNWAY BEAM

***Idris, M. H.[1]; Yusuf, I. M.[2];**
[1] Department of Building of Technology, Jigawa State Polytechnic, Dutse
[2]Department of Statistic, Jigawa State Polytechnic, Dutse
*Corresponding author email:mhidris@jigpoly.edu.ng

ABSTRACT

This research presents a study on reliability study of runway crane beam in accordance with Eurocode 3 (2007). The analysis is based on First Order Second Moment Integration Technique (FOSMIT) using FORM 5. Limit state equation for the runway crane beam is derived and evaluated safety levels. A load ratio of 0.2 to 1.2 was used to determine the safety levels using the electric wheel load percentage increase of 0 to 25%. A steel simply supported beam of the 6m span with electric wheel load of 170kN as an external load (live load) is used. The results show that Safety index decreases with increase in wheel load percentage increase. Also, the results show that at the load ratio of 0.2 to 0.6 gives a safe β - the value of 3.1 to 4.23 from minor to the large consequence of failure (JCSS, 2001) for a wheel load percentage increase of 0 to 25%.

Keywords: Deflection, Electric wheel load percentage, Load ratio, Reliability index and Runway crane beam.

INTRODUCTION

Industrial buildings commonly house manufacturing processes which involve heavy items being moved from one point to another during assembly, fabrication or plant maintenance. In some cases, overhead cranes are the best way of providing a heavy lifting facility covering virtually the whole area of the building. These cranes are usually electrically operated and are provided by specialist suppliers. The crane is usually supported on four wheels running on special crane rail. These rails are not considered to have significant bending strength, and each is supported on a crane beam or girder (Morris and Plum, 1996).

Before the Eurocodes were introduced, BS 2853 covered design and testing of overhead runway beams. Following a revision in 2011, BS 2853 now only provides guidance on testing overhead runway beams. BS EN 1993-6:2007 (EC3-6) covers the design of steel crane supporting structures, which includes overhead runway beams, while guidance on determining actions induced by cranes is given in BS EN 1991-3.

Research carried out by Ma'aruf et.al, 2016 shows that bending, shear, and shear connection are safe and conservative for the composite steel beam while the deflection is critical at some load ratios considered in term of its reliability.

Crane support structures need to be reliable in order to ensure and continuous working of the cranes that run upon them. Consequences of failure can include loss of life, damage to property as well as large financial losses due to downtime. The design of these support structures

should therefore always require a ''best practice'' approach in order to avoid any downtime or worse consequences (Dymond, 2005).

1.1 THEORETICAL CONCEPT

Description of deflection

It is stated in Table 7.2 of EC3-6 (2007) that the vertical deflection of the runway crane beam is given as

$\delta_Z \leq \frac{L}{600}$ and $\delta_Z \leq 25$.

The vertical deformation should be taken as the total

deformation due to vertical loads, less the possible pre-camber, as for δ_{max} in figure AI. 1 of EN 1990.

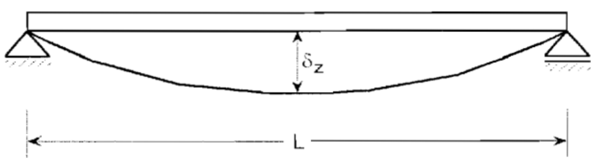

FIGURE 1: VERTICAL DEFLECTION OF RUNWAY CRANE BEAM

Loads on crane beams

Crane beams are subjected to:

(1) Vertical loads from self-weight, the weight of the crane, the hook load and impact; and

(2) Horizontal loads from crane surge.

Cranes are classified into four classes in BS 2573: *Rules for Design of Cranes*, Part 1: Specification for Classification, Stress Calculations and Design Criteria for Structures. The classes are:

Class 1—light. The safe working load is rarely hoisted;

Class 2—moderate. The safe working load is hoisted fairly frequently;

Classes 3 and 4 are heavy and very heavy cranes.

The dynamic loads caused by these classes of cranes are given in BS 6399: Part 1, Section 7. The loading specified in the code is set out below.

The following allowances shall be deemed to cover all forces set up by vibration, shock from slipping of slings, the kinetic action of acceleration and retardation and impact of wheel loads:

(1) For loads acting vertically, the maximum static wheel loads shall be increased by the following percentages:

For electric overhead cranes: 25%

For hand-operated cranes: 10%

(2) The horizontal force acting transverse to the rails shall be taken as a percentage of the combined weight of the crab and the load lifted as follows:

For electric overhead cranes: 10%

For hand-operated cranes: 5%

(3) The horizontal force acting along the rails shall be taken as 5 percent of the static wheel loads for either electric or hand-operated cranes (Lam, et.al, 2004).

2.0 METHODOLOGY

Design of Crane Runway Beams will involve the deflection mode of failures.

Based on this, the mode of failures which are in conformity with the Eurocode 3 (2007) was developed to cater for this failure. A computer program for calculating reliability indices of crane runway beam using the mode of failure equation was developed and run using a reliability software FORM5 (Gollwizter, 1988). This program is used to estimate the probability of failure or safety index (β) of structures.

2.1 FIRST – ORDER RELIABILITY METHOD (FORM)

If R is the strength capacity and S is the loading effect of the structural system which are random variables, the main objective of reliability analysis of any system or component is to ensure that R is never exceeded by S, in practice, R and S are usually functions of basic variables, in order to investigate the effect of the variables on the performance of the variables on the performance of a structural system, a limit state equation in term of the basic design variables is required. The failure function is expressed as:

$$g(x) = g(X_i) = R - S \tag{1}$$

Where X_i for i = 1, 2,… n, represent the basic design variables R and S are the resistance and load effect respectively. The basic variables are normalized using the expression:

$$X_i' = \frac{X_i - \mu_i}{\sigma_i}, (i = 1,2, \dots, n) \tag{2}$$

Substituting for X_i in the limit state equation using equation (1)

We have:

$$g(\sigma_1 X_1' + \mu_1, \sigma_2 X_2' + \mu_{2,} \dots, \sigma_n X_n' + \mu_n = 0 \tag{3}$$

Whereμ and σ are the means and standard deviations of the design variable.

The minimum distance from the origin describing the variable space to the line representing the failure surface can be obtained by minimizing the distance. The shortest distance equals beta, β and is therefore given by equation (4):

$$\beta = d = \frac{-G^{*t}X}{(G^{*t}G^{*})^{1/2}} \tag{4}$$

Where, G* gradient vector at the most probable failure point (MMP) and value of β is the measure of the safety of any given design under uncertainties in the decision variables. The statistical characteristics of the design variables were obtained in literature.

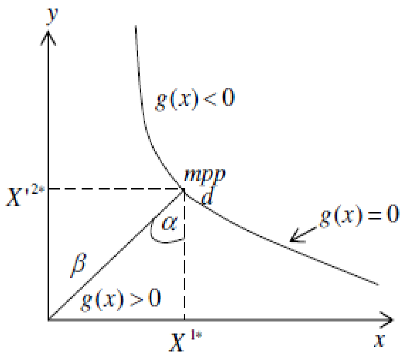

Figure 2: Formulation of safety analysis in normalized coordinates

Graphically, $g(X) > 0$Corresponds to a favourable (safe, intact, acceptable) state while $g(X) = 0$ denote the failure boundary and $g(X) < 0$defines the failure (unacceptable) domain. This is shown in Figure 2.

2.2 DERIVATION OF SAFE DESIGN PARAMETERS

In order to resist the loads, the resistance properties must be carefully chosen. The selection may not only revolve around the derivation of ultimate strength equation of crane runway beam but also aid in deriving the limit state expression of the various failure modes considered in its loading.

Therefore, limit state equation for the idealized strength and load analysis are derived from the respective failure modes.

2.3 LIMIT STATE FAILURE MODES

The limit state equation considered in this reliability analysis is to check the mode of failure vertical deflection of runway crane beam as given below:

$$g(x) = \frac{L}{600} - \frac{5}{384EI}(1.35\alpha + 1.5)Q_{ki}L^4 \qquad (5)$$

Where,

L is the length of the runway crane beam,
Q_{ki} is the wheel load on the beam,
$\alpha = \frac{G_{ki}}{Q_{ki}}$ is the dead-live load ratio and
EI is the Flexural rigidity

TABLE 1: PARAMETERS OF STOCHASTIC MODEL

S/N	Physical meaning	Type	Code	Mean	COV	Std. Dev
1	Young modulus, E	Weibull	9	210,000N/mm2	0.1	21,000N/mm2
2	Second moment of area, I	Log-normal	3	$2.87\text{x}10^7\text{mm}^4$	0.1	$2.87\text{x}10^6\text{mm}^4$
3	Span, L	Normal	2	5000mm	0.02	100mm
4	Imposed load, Qk	Gumbel	7	$2.5\ \text{x}10^{-3}$N/mm2	0.12	$0.35\ \text{x}10^{-3}$N/mm2

4.0 RESULTS AND DISCUSSION

The result obtained is presented in Figure 3, 4 and 5;

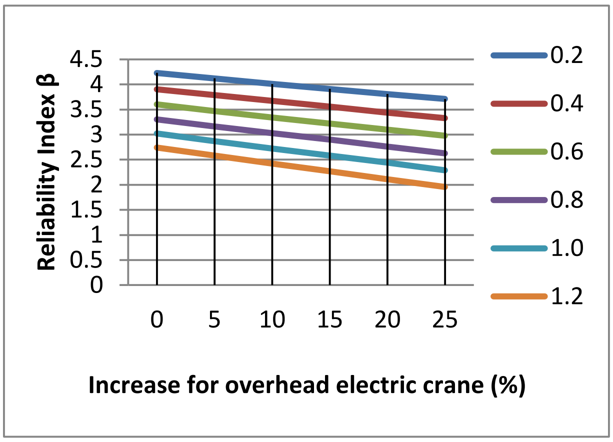

FIGURE 3: RELIABILITY INDEX AGAINST PERCENTAGE INCREASE FOR ELECTRIC OVERHEAD CRANE

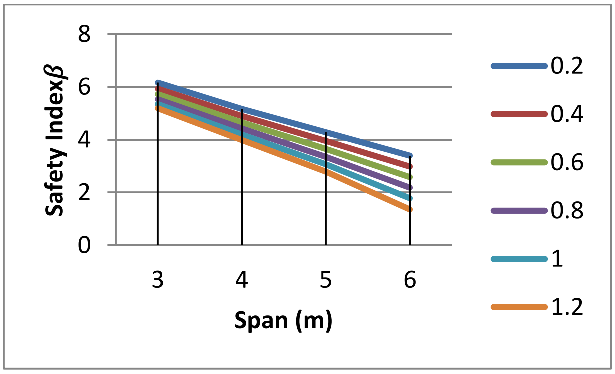

Figure 4: Variation of Safety Index against Span for Electric Overhead Crane

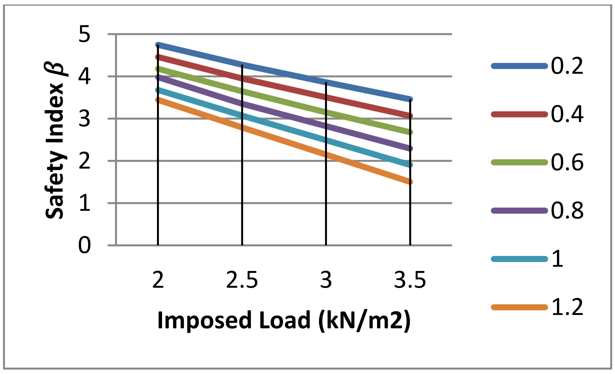

Figure 5: Variation of Safety Index against Imposed Load for Electric Overhead Crane

The modeling of runway crane beam shows that as the load percentage increase, the reliability decrease with increase in the load ratio.

The result shows that at the increased wheel load percentage of 0% the reliability index (β – value) range from 2.74 to 4.23 at the load ratio of 0.2 to 1.2. Therefore, it shows failure at a load ratio of 1.2 which gives 2.74 β – value.

At load ratio of 0.2 and 0.4, it shows that the reliability indices range from 3.33 to 4.23 which is safe at large consequences of failure (JCSS, 2001) for the wheel load percentage of 5% to 25%.

Consider a load ratio of 0.6 the β – value range from 2.98 to 3.6 at wheel load percentage of 0% to 25%. It shows that it is safe up to 20% increase in wheel load having a failure in the wheel load percentage of 25% which give a β –the value of 2.98.

For a load of 0.8, the β – value is safe at 5% with a value of 3.16 and fail at 10% to 25% wheel load percentage increase with β – the value of 2.63 to 3.03.

At load ratio of 1.0 and 1.2 it shows that β – value is unsafe for the wheel load percentage of 0% to 25% which gives a β – the value of 1.96 to 3.02 from minor to large consequences of failure (JCSS, 2001).

Figure 4 and 5 show the sensitivity analysis for the crane runway beam for keeping the all the other variables constant and varying one variable. The analysis show that for span and imposed load, the safety index increase as the span and imposed load increase.

For the span from figure 4, it shows that the safety index is safe at 5m from 3.07 to 4.28 at the load ratio of 0.2 to 1.0.

Also for the imposed load from figure 5, it shows that is safe at 3kN/m2 for safety index of 3.15 to 3.86 at the load ratio of 0.2 to 0.6.

5.0 CONCLUSION

The following conclusions were drawn from this research:

1. Modeling of runway crane beam is feasible using Eurocode 3-6 (2007).
2. The analysis shows the safety indices and the load ratio affects the increase percentage of wheel load of 0 to 25%.
3. The load ratio of 0.2 to 0.6 can be adopted as it gives a safe β – the value of 3.1 to 4.23 for 0 to 25% wheel load percentage increase.

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PRODUCTION OF CEILING BOARD USING WASTE MATERIALS

***Kolo S. S.[1], Jimoh Y. A.[2], Adeleke O. O.[3], Oyelade O. A.[4], and Yusuf I. T,[5]**
[1, 4] Civil Engineering Department, Federal University of Technology, Minna, Nigeria
[2,3,5]Civil Engineering Department, University of Ilorin, Ilorin Nigeria.
*s.kolo@futminna.edu.ng, bukysayo123@yahoo.com

ABSTRACT
The Production of Ceiling Board from waste Materials was carried out basically using waste paper and sawdust materials which was sourced locally. These raw materials were found to be in abundance and constitute environmental hazard. The raw materials used for the production of the ceiling board includes, waste paper, sawdust, cement, calcium carbonate ($CaCO_3$), Kaolin, starch and water. The quantity of materials used, was weighed out sawdust fibre (40%), Cement (10%), Kaolin (10%), $CaCO_3$ (15%), starch (10%) and water (15%) while for waste paper material composition of fibre (40%), Cement (35%), Kaolin (10%), $CaCO_3$ (10%), starch (10%) and water (15%) was used to obtain the best mix. The deflection and fire resistance test shows that the produced ceiling boards are better in terms of rigidity and fire resistance ability. It was then recommended that the used of local materials in the production of ceiling board should be encourage to create job opportunities for Nigerians and to sanitised the environment from waste material.

Keywords-: Ceiling board, waste materials, cement, $CaCo_3$, kaolin, starch.

1 INTRODUCTION

Engineering infrastructures are said to be combination of fundamental system that support a community, region or country (Hayes, 2005*)*. Civil infrastructures system involves the design, analysis and management of infrastructures supporting human activities, including e.g electric power, oil and gas, water and waste water communication, transportation and the collections of building that make up urban and rural communities (Huler, 2010). Civil infrastructures system emphasizes on, how different structures behave together as a system to serves community's need. It can be defined generally, as set of interconnected structural elements that provide a frame work supporting an entire structure of development, aiming at making life better for the people. The engineers are usually responsible for design, construction and maintenances of building infrastructure. A building is a structure that has a roof and walls and stands more or less permanently in one place. The fundamental purpose of buildings is to provide man with a comfortable working, living space and protection from extreme of climate. The components of building are element which uses industrial products that are manufactured as independent units capable of been jointed with another element are beam, column, roof e.t.c. The other finished building components are doors and windows, floors, covering of rough walls and ceilings.
A ceiling board is horizontal slab covering the upper section of a room or internal space. A ceiling board is generally not structural element but its shade (covering), concealing the details of the structure above. Provision of ceiling covering is an essential stage in the building process. The use of asbestos ceiling is been discouraged because of its hazardous effect on human. Though demand for ceiling board and other panel products have been on the increase in recent time due to increased activities in the building industries.
Recent research effort is focused on how to use the waste generated from paper and wood products for ceiling board production, these waste has been a major source of solid waste problem in Nigeria and only a small percentage of the discarded waste paper e.g newsprint are kept in the achieves as reference material and a small portion of sawdust is been used and difficult to dispose.
The Production of Ceiling Board from waste Materials was carried out basically using waste paper and sawdust materials which was sourced locally and could serve as a very good fibre (Oladele, *et al*, 2014). These raw materials are found to be very much in abundance and constitute environmental hazard (Callister, 2001).
It is aimed that this waste paper product and sawdust can be reuse into a more permanent usage in the production of ceiling board for building and hence be able to solve the disposal problem of waste generated from these papers and wood, these will in turn reduce the cost of building construction and make the material for the production of ceiling to be locally sourced.

2. METHODOLOGY

The Production of Ceiling Board from waste Materials was carried out using the following raw materials: waste paper, cement, calcium carbonate ($CaCO_3$), Kaolin, starch and water, all are sourced locally. These raw materials are found to be very much in abundance.

2.1 Methods Adopted for Study

The under listed method was adopted for this research work in order to meet the set of aim and objectives.

i. Material sourcing
ii. Equipment used

iii. Production of ceiling board / manufacturing process
iv. Test on produced ceiling and available ceiling

2.2 Material Sourcing

The materials in Table 1, were used in the production of the ceiling board

TABLE 1: SHOW THE SOURCE OF MATERIALS USED AND THEIR FORM.

S/No	Materials	Form	Source
1	Waste paper	Plain and Mostly white	Minna
2	Sawdust	Powdery and Brown	Minna
3	Cement	Grey and Powdery	Minna
4	Calcium carbonate	Powdery	Ibadan
5	Kaolin	Powdery	Minna
6	Starch	White powder	Minna

2.3 Equipment Used

The underlisted equipment was in the production of the ceiling board

i. Measuring Cylinder
ii. Thermometer
iii. Weighing Balance
iv. Reactor
v. Stirrer
vi. Mould
vii. Cellophane
viii. Bucket
ix. Roller

2.4 Production of Ceiling Board/ Manufacturing Process.

There are different ways ceiling boards can be manufactured and this depends on the availability of raw materials used for the production. This study will be based mainly on the use of cement and different fibres for the production of ceiling board. The manufacturing processes to be discussed include:

1. The manufacturing of ceiling board from sawdust
2. The manufacture of ceiling boards from paper.

The production processes for the above are similar, the different only depends on the fibre type (paper or saw dust) been included as the raw material especially the type of re-enforcement that will be used but they are all cement composite ceiling boards**.**

2.5: Ceiling Board Production process

The Production of the ceiling was group into seven (7) major steps.

Step1: Material Collection

Waste paper and saw dust were collected from different places. e.g printing houses, schools, hotels, saw mill e.t.c.

Step 2: Sorting

Sorting involves the selection of waste paper from other solid waste in order to prevent contamination that could prevent the recycling of this waste paper, while the sawdust was also sieve to remove every large wooden particles.

Step 3: Soaking

After sorting out some few coated and strong paper that cannot be easily re-pulped by the beater from the collected sample, some quantity of these paper was soaked in enough volume of water for 2 days. The essence of soaking these papers is to allow easy blending. While enough quantity of the sawdust was soaked in enough volume of water for 24hrs (1days).

Step 4: Beating

The paper was then blended with the grinding machine to achieve a smooth finish. The used paper was from the wet paper which was soaked. The sawdust needs not to be blended.

Step 5: Mixing

A total of 5kg of various components for the production of ceiling board were then mixed. These components include; (i) Water (ii) Waste paper (iii) sawdust (vi) Calcium carbonate ($CaCo_3$), (v) Kaolin and (iv) cement in various percentages. They were all mixed together thoroughly with the machine to enhance proper blending. The chemical additive ($CaCo_3$) of 3% concentration in water dilution was used for each specimen, and the water containing the additives was added to the mixture while blending in the mix. The ceiling board production was divided into two (2) Categories (i) The paper fibre and (2) The Sawdust Fibre. The fibre (paper and sawdust) was used because it was the structural back bone of the product. It contributes to the strength, optics, stiffness and smoothness of the ceiling board. Cement and $CaCo_3$ was added to improve the texture and opacity, brightness of the board. It also contributes to the smoothness and more uniform surface of the board. Starch was added to increase the forces required to tear or rupture a board. Finally, the addition of water enables easy mixing of the slurry and to make the mixture mouldable.

(a) The paper Fibre

Three different composition was chosen for paper ceiling production as stated in Table 2 and the best selected as the final production mix.

(b) The sawdust Fibre

Three different composition was also chosen for sawdust ceiling production as stated in Table 2 and the best selected as the final production mix.

TABLE 2: WASTE PAPER AND SAWDUST FIBRE COMPOSITION

Material composition	Sawdust (%)			Waste Paper (%)		
	Sample A	Sample B	Sample C	Sample A	Sample B	Sample C
Fibre	40	40	40	40	45	40
Cement	5	15	10	10	10	5
Kaolin	10	10	10	10	10	15
$CaCO_3$	15	10	15	10	10	15

Starch	15	10	10	15	10	10
Water	15	15	15	15	15	15

Step 6: Mould
The mixed component was then poured into a 600mm by 600mm square mould designed for the production of ceiling board. The mix component was then spread across the mould in order to take the shape of the mould. It must fill every part of the mould, vibrated and the levelled using a trowel. It was then compress together to remove some water content. This process is also known as expression. It was used to separate liquid from a mixture of liquid and solid. This was done by compression under the condition that permits liquid to escape while the solid is retained between the compressing surfaces. pressure of $1.23Nm^2$ to form the required thickness of 1inch for a period of 24 hours setting was applied. The boards were removed from press and from the mould.

Step 7: Drying
The board was then allowed to set and to dry in the mould for about two days after which is removed from the mould and dried naturally in conducive environment usually it is dried with solar energy. They are stacked under a controlled laboratory environment to allow for 7 days curing.

Step 8: Trimming
The sheets formed from the mould undergo trimming to get rid of the rough edges.

3. RESULT AND DISCUSION

3.1 Material Composition

After careful observation of the mix composition for various samples sample A of waste paper was selected while sample C for sawdust was selected as the best mix composition. The two selected mix were outstanding in their performance against the common ceiling board in the market. Table 3 shows the selected composition in Kilogramme (Kg) for 5kg production quantity.

TABLE 3: SELECTED PRODUCTION COMPOSITION

Material composition	Sawdust (Kg)	Paper (Kg)
Fibre	2	2
Cement	0.5	0.5
Kaolin	0.5	0.5
$CaCO_3$	0.75	0.5
Starch	0.5	0.75
Water	0.75	0.75

3.2 Sample drying period

Table 4, present the rate of drying for each sample composition. It was observed that the composition of waste paper for sample A dries faster than others while in the composition of sawdust, sample C dries faster than others after 24hrs of drying in an open air, the curing continues after then.

TABLE 4: DRYING PERIOD PER SAMPLE

Duration	Waste Paper (Days)			Sawdust (Days)		
	Sample A	Sample B	Sample C	Sample A	Sample B	Sample C
Number of the days	4	7	5	5	4	3
Rate of drying	Fast	Very Slow	Slow	Very Slow	Slow	Fast

After drying, sample B and C did not bind properly as a result there were lots of cracks. More starch was thus used to produce a latter ceiling board. For drying the ceiling board, solar energy was used in place of even drying. This is because when high temperature is used for drying, the surface of the board dries up while the inner part is still wet as a result; there were lots of cracks on the board. This was notice when the board was dried using an oven. Therefore, drying is better when done at a low temperature to avoid the development of micro-cracks due to quick setting of cement. During the drying process, it was notices that sample A dries faster than all the others. It took 4days for it to dry totally while it took sample B seven days to dry and sample C it took 5days for it to dry totally.

After drying, it was noticed that sample B has the smoothest surface, next to it is sample A then finally is sample C. This is because, the Starch is minimal. The colour of sample B was found to be ash colour, this is because of the percentage of cement which is 5%. sample A and C however are result obtained from analysis, of the ceiling board, it was noticed that the dry weight of the ceiling board from at (sample C) was higher than others, the dry weight was found to be 15.43g, next to it is sample B with dry weight of 11.6g is sample B with dry weight of 11.6g and finally is sample A with dry weight of 11.00g.

3.4 Load Bearing Test

The results load test in Table 5, shown that all the samples resisted more load than the existing ceiling board of same size. Indicating a better deflection ability when suspended. Sample A has the highest bursting strength which means that it has a higher tensile strength than others. Therefore, it has a better quality, it has a bursting strength of 42.50 N, next to it is sample B with bursting strength of 37.50 N then sample C of bursting strength of 56.20 N.

TABLE 5: LOADING BEARING TEST

Loading	Waste Paper (Days)			Sawdust (Days)			EXISTING CEILING BOARD
	Sample A	Sample B	Sample C	Sample A	Sample B	Sample C	
Applied mass(kg)	4.33	3.82	5.76	2.05	3.3	2.4	2.09
Force (N)	42.5	37.5	56.2	20.11	32.37	23.72	20.5

3.5 Fire Resistance Test

All the samples were subjected to fire test, it was clear that both the waste paper ceiling and the sawdust ceiling board has higher fire resistance than the existing ceiling board. Table 6 present the results of the fire resistance test.

TABLE 6: BURNING RESISTANCE TEST

Burning Time Interval (min)	Waste Paper (Time)			Sawdust (Time)			EXISTING CEILING BOARD
	Sample			Sample			
	A	B	C	A	B	C	
1	No effect	No effect	No effect	No effect	No effect	No effect	No effect
2	No effect	No effect	No effect	No effect	No effect	No effect	No effect
3	No effect	No effect	No effect	No effect	No effect	little effect	little effect

3.6 Costing

The costing carried out on this project is a rough estimate of the cost of production. That is it depends on or based on cost of materials used in production not including the equipment cost. This is to show the economic viability of the production of these ceiling boards here in Nigeria.

In this project, three different types of ceiling board were produced from different quantity of the samples. The difference being in the quantity of the material used for the sample of ceiling board produced from cellulose fibre. The different raw materials used include fibre, cement, CaCo3, kaolin, starch and water, which all had their function in the ceiling board.

Finally, from the costing it was observed that, sample B is the cheapest which cost ₦145.38 next is sampling C which cost ₦149.94 and finally is sample A which cost ₦154.94. Table 7 showing Specification of the Overall Cost of the Production of ceiling board.

TABLE 7: COST OF PRODUCTION

		Cost of materials		Cost of material used	
S/N	Materials	Quantity(kg)	Cost (#)	Quantity (kg)	Cost (#)
1	Waste paper	20	0.00	2	0.00
2	Cement	50	1850.00	0.5	200.00
3	Kaolin	4	1000.00	0.6	90.00
4	$Caco_3$	10	150.00	0.5	7.50
5	Starch	2	250.00	0.5	31.25
6	Cellophane yield	_	150.00	_	150.00

TABLE 8: SHOWING SPECIFICATION OF THE TOTAL COST OF EACH SAMPLES PRODUCED

Samples	Cost of materials	Transportation Cost (#)	Total Cost (#)
A	22.88	132.06	154.94
B	13.38	132.06	145.44
C	17.88	132.06	149.94

4. CONCLUSION

It was observed that the paper fibre ceiling of sample A has a better quality and properties and its composition was chosen as best for production while Sample C, for sawdust was chosen for production. The selected composition for paper and sawdust as in Table 3, shows superiority over the available ceiling board in the market in term of strength, fire resistance ability, cost and been able to sanitise the environment after used. Finally, our waste instead of dumping them should be recycled and re-used like in this case; paper and sawdust should not be dumped but recycled and used for the production.

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Federal University of Technology, Minna, Nigeria

A CONTRIBUTION TO THE BUCKLING ANALYSIS OF STIFFENED RECTANGULAR ISOTROPIC PLATES

***Ibeabuchi, V. T.[1]; Ibearugbulem, M. O.[2]; Ezeah, C.[3]; & Ugwu, O. O[4]**
[1, 3&4] Department of Civil Engineering, Alex Ekwueme Federal University Ndufu Alike, Ikwo
[2]Department of Civil Engineering, Federal University of Technology, Owerri
*Corresponding author email: ibeabuchi.victor@funai.edu.ng

ABSTRACT

In this research paper, buckling analysis of stiffened rectangular isotropic plates elastically restrained along all the edges (CCCC), using work principle approach that is based on polynomial function were carried out. The stiffeners are assumed to be rigidly connected to the plate, such plates are widely used in civil engineering, marine, and aeronautic structures. Analysis for critical buckling of stiffened plates possessing different aspect ratios, varying stiffness properties and varying number of stiffeners were carried out. The governing differential equation for the stiffened plate system was obtained by super position principle. Polynomial functions were used in this study and the present analysis was carried out only for uniaxially stiffened plate, where longitudinal stiffeners are presented parallel to inplane load of the plate. Effects of the number of stiffeners, aspect ratios, boundary conditions, stiffener parameters upon the buckling coefficients, K of the stiffened plates were investigated. The results were obtained considering the bending displacements of the plate and the stiffener for all edge clamped conditions presented. Maximum percentage increase in buckling coefficients of stiffened plates for the case of one and two stiffeners is 20.586%. For the case of one and three stiffeners, the maximum percentage increase in buckling coefficients recorded is 48.2257%. Several numerical examples were presented to demonstrate the accuracy and convergence of the current solutions.

Keywords: *Buckling; Governing Differential Equation; Polynomial Function; Uniaxially Stiffened Plate; Work Principle Method.*

1 INTRODUCTION

Stiffened plates have wide applications in civil engineering, aerospace and marine structures. They are used in box girders, plate girders, ship hulls and wing structures. Research in Stiffened plate construction has gained attention in recent years as a result of its economic and structural benefits. The advantage of stiffening a plate lies in achieving an economical, light weight design.

Wutzow and De Paiva (2008) defined Stiffeners as linear elements, nearly always of negligible thickness and the analysis of such plates has received the attention of many researchers. Several methods have been suggested for the prediction of the global buckling load of stiffened plates. Energy approach had been extensively used by different researchers for buckling analysis of flat plates under different load conditions. The analytical solutions for buckling are presented by (Timoshenko and Gere, 1961; Ventsel and Krauthammer, 2001 and Eccher et al., 2009). Both single and double Fourier series were used as deflection functions to compute the values for buckling coefficients, but no theoretical solutions exist for more complicated boundary conditions of stiffened plates (Nildem, 2010).

However, in the analysis of stiffened plates by Energy method, it is a common knowledge that only few cases of edge supports had been covered in the literature. Various researchers employed energy approach which involves substituting the assumed shape function into energy solution. Other scholars have used trigonometric series in this approach. Both single and double series can be used to formulate approximate shape function for a stiffened plate, whose four edges are simply supported. It can also be used for a plate whose opposite edges are clamped and the other opposite edges are simply supported. However, it is difficult to apply trigonometric shape function in analysing stiffened plates with non uniform edge. Such stiffened plates whose solutions had been presented using Energy approach includes, SSSS stiffened Plates and CSCS stiffened Plates (Timoshenko and Gere, 1961; Bulson, 1970; Iyengar, 1988; Szilad, 2004; Ibearugbulem et al. 2014). The main objective of this work is to derive the solution of all edge clamped stiffened rectangular isotropic plates using work principle and polynomial function.

2 METHODOLOGY

The governing equation was derived from first principle by using the equations and principles of theory of elasticity. The Buckling solution was formulated from the governing equations (for plate and beam) combined by super position principle. The substitution of boundary conditions for CCCC rectangular stiffened plates into the governing differential equation derived gave the Buckling solutions for the case of one stiffener, two stiffeners and three stiffeners.

2.1 GOVERNING DIFFERENTIAL EQUATION

In this study, stiffeners were considered as line continuum. From the principles of the theory of elasticity, the governing equation for stiffened rectangular isotropic plates was given by Ibeabuchi (2014) as;

$$\frac{1}{P^4}\frac{\partial^4 w}{\partial R^4}+\frac{2}{P^2}.\frac{\partial^4 w}{\partial R^2\partial Q^2}+\frac{\partial^4 w}{\partial Q^4}+\frac{1}{P^4}.\sum_{i=1}^{n}\gamma_i\left(\frac{\partial^4 w}{\partial R^4}\right)_{Q=ci}+\frac{b^2}{P^2}.\frac{N_x}{D}.\frac{\partial^2 w}{\partial R^2}\frac{b^2}{P^2}.\frac{N_x}{D}.\sum_{i=1}^{n}\delta_i\left(\frac{\partial^2 w}{\partial R^2}\right)_{Q=ci}=0 \quad (1)$$

$Q\ and\ R\ are$ non dimensional parameter

where;

$$\mathrm{Q}=\frac{y}{b}\quad \text{that is } \mathrm{y}=\mathrm{bQ}$$

$$\mathrm{R}=\frac{x}{\mathrm{a}}\quad \text{that is } \mathrm{x}=\mathrm{aR}$$

$$\gamma_i=\frac{EI_i}{Db}=\text{Ratio of bending stiffness rigidity of the Stiffiners to the plate}$$

$$\delta_i=\frac{A_i}{bh}=\text{Ratio of cross}-\text{sectional area of the}$$

Stiffeners to the plate

2.2 WORK PRINCIPLE

Work is expressed mathematically as the product of average force and distance travelled by the force. Hence, for the combined action of work done by the compressive and resistive force on the stiffened system through a distance w, equation (1) becomes;

$$\frac{1}{2}\left[\frac{A^2}{P^4}.H\frac{\partial^4 H}{\partial R^4}+\frac{2A^2}{P^2}.H\frac{\partial^4 H}{\partial R^2\partial Q^2}+A^2\frac{H.\partial^4 H}{\partial Q^4}+\frac{A^2}{P^4}.\sum_{i=1}^{n}\gamma_i\left(\frac{H.\partial^4 H}{\partial R^4}\right)_{Q=ci}+\frac{b^2}{P^2}.\frac{N_x A^2}{D}.H\frac{\partial^2 H}{\partial R^2}+\frac{b^2}{P^2}.\frac{N_x}{D}.A^2\sum_{i=1}^{n}\delta_i\left(\frac{H.\partial^2 H}{\partial R^2}\right)_{Q=ci}\right]=\mathrm{ei} \quad (2)$$

Where; Deflection function, W = AH; A = coefficient; H is the shape function: ei is the introduced error, "i" is the number of points on the continuum. Integrating equation (2) twice with respect to R and Q gave;

$$\Pi=\frac{\mathrm{A}^2}{2}\int_0^1\int_0^1\left[\frac{H}{P^4}.\frac{\partial^4 H}{\partial R^4}+\frac{2H}{P^2}.\frac{\partial^4 H}{\partial R^2\partial Q^2}+\frac{H.\partial^4 H}{\partial Q^4}+\frac{1}{P^4}.\sum_{i=1}^{n}\gamma_i\left(\frac{H.\partial^4 H}{\partial R^4}\right)_{Q=ci}\right]\partial R\partial Q+\frac{\mathrm{A}^2 b^2}{2P^2}.\frac{N_x}{D}\int_0^1\int_0^1\left[\frac{H.\partial^2 H}{\partial R^2}+\sum_{i=1}^{n}\delta_i\left(\frac{H.\partial^2 H}{\partial R^2}\right)_{Q=ci}\right]\partial R\partial Q \quad (3)$$

$\prod$ is the Total Work error Functional. Minimizing and making N_x the subject of equation 3 gave;

$$N_{x(cri)}=\frac{D\int_0^1\int_0^1\left[\frac{H}{P^2}.\frac{\partial^4 H}{\partial R^4}+2\mathrm{H}.\frac{\partial^4 H}{\partial R^2\partial Q^2}\right]\partial R\partial Q}{-b^2\int_0^1\int_0^1\left[\frac{H.\partial^2 H}{\partial R^2}+\sum_{i=1}^{n}\delta_i\left(\frac{H.\partial^2 H}{\partial R^2}\right)_{Q=ci}\right]\partial R\partial Q}+\frac{D\int_0^1\int_0^1\left[\mathrm{P}^2\frac{H.\partial^4 H}{\partial Q^4}+\frac{1}{P^2}.\sum_{i=1}^{n}\gamma_i\left(\frac{H.\partial^4 H}{\partial R^4}\right)_{Q=ci}\right]\partial R\partial Q}{-b^2\int_0^1\int_0^1\left[\frac{H.\partial^2 H}{\partial R^2}+\sum_{i=1}^{n}\delta_i\left(\frac{H.\partial^2 H}{\partial R^2}\right)_{Q=ci}\right]\partial R\partial Q} \quad (4)$$

2.3 SHAPE FUNCTION FOR CCCC STIFFENED PLATES

The boundary conditions in dimensionless coordinate system for stiffened plates with all edge clamped and dimensionless R-Q axes as shown in Figure 1.0., are given below;

Ibearugbulem (2012) formulated the general shape function for rectangular plates from Taylor-McLaurin's series as;

$$w=\sum_{m=o}^{4}\sum_{n=o}^{4}a_m b_n R^m Q^n \quad (5)$$

CCCC stiffened plate has the following boundary conditions;

$$w(R=0)=\frac{\partial w}{\partial R}(R=0)=0 \quad (6)$$

$$w(R=1)=\frac{\partial w}{\partial R}(R=1)=0 \quad (7)$$

$$w(Q=0)=\frac{\partial w}{\partial Q}(Q=0)=0 \quad (8)$$

$$w(Q=1)=\frac{\partial w}{\partial Q}(Q=1)=0 \quad (9)$$

Applying these boundary conditions in equation 5 gave;

$$w=A(R^2-2R^3+R^4)(Q^2-2Q^3+Q^4) \quad (10)$$

2.4 BUCKLING EQUATION FOR CCCC PLATES WITH ONE STIFFENER

Consider Figure 1, the stiffener divides the plate into two equal parts.

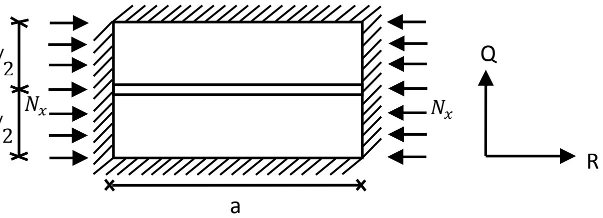

FIGURE 1: PLATE WITH ONE LONGITUDINAL STIFFENER

FOR THE STIFFENER /RIB

When there is only one stiffener for $1 \le Q \le 1$; $1 \le R \le 1$, we have;

$$(H)_{y=b/2}=(H)_{Q=1/2}$$

$$=(R^2-2R^3+R^4)(Q^2-2Q^3+Q^4)$$
$$=0.0625\,(R^2-2R^3+R^4) \quad (11)$$

$$\int_0^1\int_0^1\left(H.\frac{\partial^2 H}{\partial R^2}\right)_{Q=1/2}=-0.7440*10^{-4}\,;$$
$$\int_0^1\int_0^1\left(H.\frac{\partial^4 H}{\partial R^4}\right)_{Q=1/2}=0.003125 \quad (12)$$

FOR THE PLATE ELEMENT

$$\int_0^1\int_0^1 H.\frac{\partial^4 H}{\partial R^4}\partial R\partial Q=0.0012698\quad;\int_0^1\int_0^1 H.\frac{\partial^4 H}{\partial Q^4}\partial R\partial Q$$
$$=0.0012698 \quad (13)$$

$$\int_0^1\int_0^1 H.\frac{\partial^4 H}{\partial R^2\partial Q^2}\partial R\partial Q=0.00036281;$$
$$\int_0^1\int_0^1 H.\frac{\partial^2 H}{\partial R^2}\partial R\partial Q-3.0234*10^{-5} \quad (14)$$

Substituting equations (12), (13), (14) into (4), gave;

$$K=\frac{[4.2554+2.4317P^2+4.2554P^4+10.4726\,\gamma]}{\mathrm{P}^2[1+2.46088\delta]} \quad (15)$$

2.5 CASE OF TWO STIFFENERS

Consider Figure 2, two stiffeners divides the plate into three equal parts.

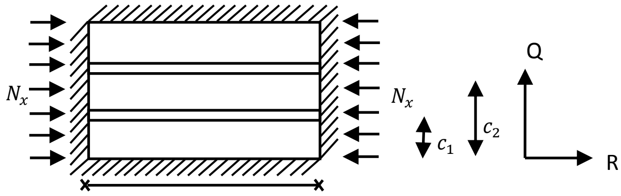

FIGURE 2: PLATE WITH TWO LONGITUDINAL STIFFENERS

Where C_1 and C_2 are the distances of the stiffeners from the edge y =0.
From Figure 2, assuming stiffeners are symmetrical, hence;

$$\delta_1=\delta_2=\delta \text{ and } \gamma_1=\gamma_2=\gamma:\ C_1=\frac{1}{3},\ C_2=\frac{2}{3}$$

Following the same procedure in section (2.4), we obtain the equations as follows;

$$(H)_{Q=C1}=0.04938(R^2-2R^3+R^4)\quad;\quad(H)_{Q=C2}=0.04938(R^2-2R^3+R^4) \quad (16)$$

$$\int_0^1\int_0^1\left(H.\frac{\partial^2 H}{\partial R^2}\right)_{Q=C1}\partial R\partial Q=\int_0^1\int_0^1\left(H.\frac{\partial^2 H}{\partial R^2}\right)_{Q=C2}\partial R\partial Q=-0.46445*10^{-4} \quad (17)$$

$$\int_0^1\int_0^1\left(H.\frac{\partial^4 H}{\partial R^4}\right)_{Q=C1}\partial R\partial Q=\int_0^1\int_0^1\left(H.\frac{\partial^4 H}{\partial R^4}\right)_{Q=C2}\partial R\partial Q=0.0019507 \quad (18)$$

Substituting equations (12), (13), (17), (18) into (4), gave;

$$K=\frac{[4.2554+2.4317P^2+4.2554P^4+13.0745\,\gamma]}{\mathrm{P}^2[1+3.0724\delta]} \quad (19)$$

2.6 CASE OF THREE STIFFENERS

Consider Figure 3, the stiffeners divides the plate into four equal parts;

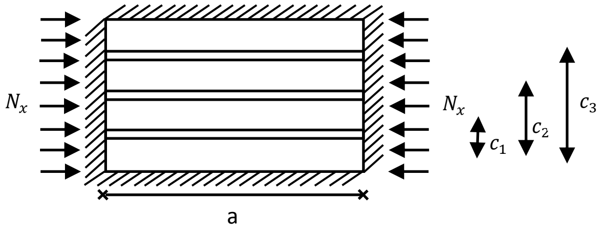

FIGURE 3: PLATE WITH THREE LONGITUDINAL STIFFENERS

$$\delta_1 = \delta_2 = \delta_3 = \delta; \quad \gamma_2 = \gamma_2 = \gamma_3 = \gamma : C_1 = \frac{1}{4}, \quad C_2 = \frac{1}{2}, C_3 = \frac{3}{4}$$

Following the same procedure in section (2.4), we obtain;

$$K = \frac{[4.2554 + 2.4317P^2 + 4.2554P^4 + 17.1000\gamma]}{\mathrm{P}^2[1 + 4.0180\delta]} \quad (20)$$

3 RESULTS AND DISCUSSION

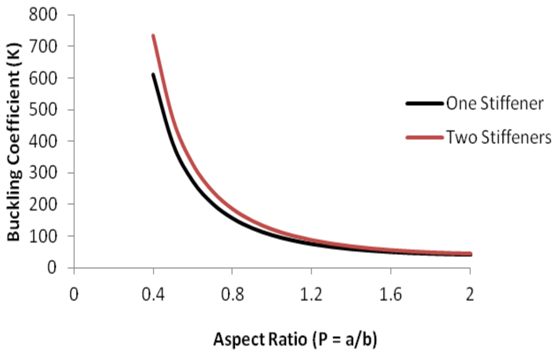

FIGURE 1: BUCKLING COEFFICIENT VS ASPECT RATIO FOR CCCC STIFFENED PLATE USING DATA FROM TABLE A1, FOR Γ =10 , =0.05.

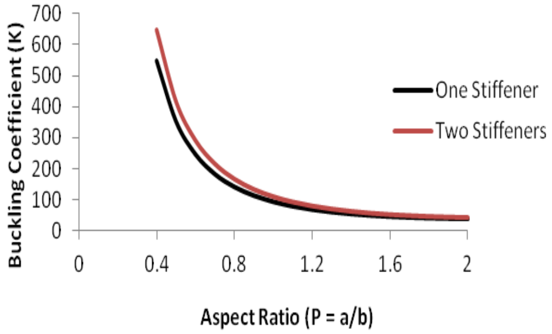

FIGURE 2: BUCKLING COEFFICIENT VS ASPECT RATIO FOR CCCC STIFFENED PLATE USING DATA FROM TABLE A1, FOR Γ =10, □=0.10.

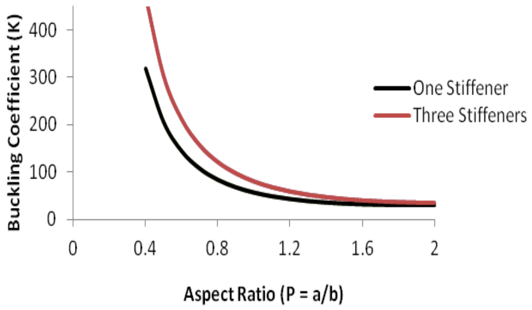

FIGURE 3: BUCKLING COEFFICIENT VS ASPECT RATIO FOR CCCC STIFFENED PLATE USING DATA FROM TABLE A2, FOR Γ =5,□ =0.05.

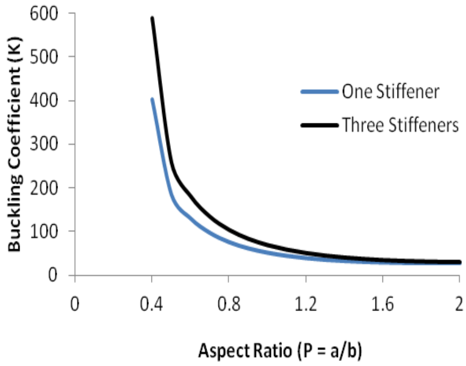

FIGURE 4: BUCKLING COEFFICIENT VS ASPECT RATIO FOR CCCC STIFFENED PLATE USING DATA FROM TABLE A2, FOR Γ =5 ,□ =0.10.

Substituting the aspect ratio in the range of $0.1 \leq P \leq 2.0$ into equations (15) and (19) gave the numerical values presented in Table A1. From the behavior of the graphs in Figure 1 and 2, the curves show that, at constant value of $\gamma = 5$, buckling coefficient decreases with increase in ⍰ from 0.05 to 0.1. Percentage increase in the buckling coefficient is found to be maximum at P = 0.1 which is 19.698 and minimum at P = 2 which is 3.111. In Figure 3 and 4, the curves show that, at constant value of $\gamma = 10$, it is observed that increase in the number of stiffener increases the buckling coefficients rapidly. However, increase in aspect ratio causes decrease in K value.

4 CONCLUSION

Based on the result of this research, the new solution for stiffened plates using work principle and polynomial function proposed is computationally efficient in solving buckling problems of stiffened plates with complicated boundary conditions such as CCCC stiffened plates.

Polynomial function is satisfactory in approximating the deformed shape of rectangular stiffened plates of various boundary conditions.

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APPENDIX

TABLE A1: COMPARISON OF BUCKLING COEFFICIENT, K FOR CCCC STIFFENED PLATES OBTAINED IN EQUATIONS (15) AND (19), A CASE OF ONE CENTRAL LONGITUDINAL AND TWO LONGITUDINAL STIFFENERS HAVING, $\gamma = 10$

P	⍰ = 0.05		Percentage increase	⍰ = 0.10		Percentage increase
	ONE ST	TWO ST		ONE ST	TWO ST	
0.1	9796.346	11704.470	20.586	8787.925	10329.020	18.074
0.2	2428.352	2927.838	20.569	2188.573	2583.773	18.057
0.3	1080.744	1302.699	20.537	974.030	1149.612	18.026
0.4	609.280	734.094	20.485	549.119	647.827	17.976
0.5	391.278	471.123	20.406	352.643	415.759	17.898
0.6	273.089	328.501	20.291	246.124	289.897	17.785
0.7	202.066	242.738	20.128	182.113	214.213	17.626
0.8	156.218	187.318	19.908	140.792	165.305	17.410
0.9	125.039	149.569	19.618	112.692	131.992	17.126
1.0	102.996	122.820	19.247	92.826	108.387	16.763
1.1	86.950	103.285	18.787	78.364	91.147	16.312
1.2	75.012	88.686	18.230	67.605	78.264	15.767

1.3	65.990	77.586	17.573	59.474	68.469	15.124
1.4	59.103	69.044	16.819	53.267	60.930	14.386
1.5	53.820	62.418	15.974	48.506	55.083	13.559
1.6	49.772	57.263	15.050	44.858	50.534	12.654
1.7	46.694	53.261	14.063	42.084	47.002	11.687
1.8	44.393	50.178	13.030	40.010	44.281	10.676
1.9	42.726	47.841	11.972	38.507	42.219	9.640
2.0	41.582	46.119	10.909	37.476	40.699	8.599

ST means Stiffener(s)

TABLE A2: COMPARISON OF BUCKLING COEFFICIENT, K FOR CCCC STIFFENED PLATES OBTAINED IN EQUATIONS (15) AND (20), A CASE OF ONE CENTRAL LONGITUDINAL AND THREE LONGITUDINAL STIFFENERS HAVING, $\gamma = 5$

P	⍰ = 0.05		Percentage increase	⍰ = 0.10		Percentage increase
	ONE ST	THREE ST		ONE ST	THREE ST	
0.1	5043.733	7476.071	48.225	4545.707	6404.633	40.894
0.2	1262.699	1870.699	48.148	1138.018	1602.573	40.821
0.3	562.676	832.790	48.005	507.117	713.438	40.685
0.4	317.867	469.718	47.771	402.340	588.481	40.463
0.5	204.774	301.871	47.417	184.554	258.608	40.126
0.6	143.572	210.912	46.903	129.395	180.685	39.638
0.7	106.910	156.292	46.190	96.354	133.892	38.960
0.8	83.364	121.074	45.235	75.132	103.722	38.052
0.9	67.47	97.167	44.003	60.813	83.241	36.881
	6					
1.0	56.370	80.309	42.467	50.804	68.799	35.421
1.1	48.416	68.081	40.618	43.635	58.324	33.663
1.2	42.632	59.030	38.464	38.423	50.570	31.616
1.3	38.401	52.238	36.035	34.601	44.752	29.307
1.4	35.314	47.103	33.382	31.827	40.352	26.786
1.5	33.098	43.216	30.570	29.830	37.022	24.112
1.6	31.559	40.292	27.671	28.443	34.517	21.356
1.7	30.561	38.127	24.759	27.543	32.663	18.589
1.8	30.002	36.574	21.903	27.040	31.332	15.874
1.9	29.810	35.521	19.158	26.866	30.430	13.265
2.0	29.926	34.884	16.568	26.971	29.885	10.803

ST means Stiffener(s)

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Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

MODEL FOR PREDICTING THE QUANTITY OF PLASTERING WASTE IN BUILDING CONSTRUCTION WORKS IN ABUJA, NIGERIA

*Umar, A.[1] & Saidu, I[1]
[1] Department of Quantity Surveying Federal University of Technology, Minna
*Corresponding author email: everlyumar2000@yahoo.com

ABSTRACT

Construction waste has been identified as global environmental problem that have great effects on the progress of projects. Statistical data relating to the quantities of material waste for building works have not been well documented, and there is no known model for predicting the quantities of onsite material waste for different materials, such as plastering mortar. This study was aims to develop a model for predicting volume of plastering waste in building works. The population for the study consisted 1345 residential construction project sites within Abuja, Nigeria. The sample frame constituted a total of twenty 3-bedroom bungalow buildings and twenty 3-bedroom duplex buildings using stratified random sampling method. The volume of the mortar wasted and mortar used are determined by using bucket as the most commonly used on construction sites. Linear-regression analytical tool was used for the analyses. The study revealed relationships between variables considered are statistically significant in this study. The study concludes that relationship between volume of plastering mortar to be used and volume of plastering waste are statistically significant in both bungalow and duplex works. This means that any change in any of the variables would bring a similarly change in the other variables. Based on these, the research recommends the use of these models by the building construction professionals at the early stage in order to have an ideal record of likely waste.

Keywords: *Model, Nigeria, Plastering waste, Used plaster.*

1 INTRODUCTION

Construction waste has been identified as a global environmental problem which can have great effects on time, cost, quality and sustainability, as well as the progress of projects (Nagapan et al., 2012; Hassan et al., 2012; Saidu, 2016; Saidu et al., 2017).

Material wastage has become a serious challenge all over the world, which needs immediate action in the construction industry and it has affects the completion of many projects (Adewuyi and Otali, 2013);(Saidu et al., 2017). Studies from around the world have revealed that material waste from the construction industry contributes to higher percentage of the production costs. Hence, inadequate management of materials and waste leads to an increase in the total cost of construction projects (Ameh and Itodo, 2013). During the construction process extra construction materials are purchased as a result of material wastage (Ping et al., 2009). The issue of material waste all over the nation remains unresolved in the construction industry. For instance, in the United Kingdom (UK) 10% of the materials supplied to construction sites end up as waste that may not be explained (Osmani, 2011). Ameh and Itodo (2013) stated that in Nigeria in every 100 houses built; there is sufficient waste material to build another 10 houses. Adewuyi and Otali,(2013); Saidu, (2016) discovered that during the process of preparing an estimate for a construction project the quantity of material waste generated on some construction sites is over the 5 percent allowance for material wastage on construction sites. Thus, construction cost estimation methods play crucial roles in the completion of projects (Oyedele, 2015). Masudi et al. (2012);Saidu and Shakantu (2017) noted that while quantification of material waste is very important for construction waste control, accurate calculation can be accomplished by developing waste quantification model that is applicable to national and regional construction waste generation.

Material wastage on construction sites can have impact on the quantity of materials delivered/used but objective researches to provide evidence of such impact of its cost are suboptimal (Saidu and Shakantu, 2017). In Nigeria, not all materials supplied to site are used during construction process; the leftover may remain as waste that may not be explained (Ameh and Itodo, 2013; Saidu, 2016). This is despite the fact that construction professionals allow some percentage of wastage figures during the process of pricing a bill of quantities, but sites experience has revealed that wastage are often more than the amount allowed in the bid document, if site management is not tight (Saidu, 2016). In the last decade, management of waste generation has received less attention in the Nigerian construction industry (Wahab and Lawal, 2011). This could be as a result of the low level or lack of proper quantification of material waste and this is evident in the amount of waste generated at construction sites; low level of awareness of the construction workers; a low level of available means of waste disposal; or the slow adoption of environmentally sustainable practices (Saidu and Shakantu, 2017). Yuan and Shen (2011) highlighted that the insufficient attention given to material-waste generation in developing nations during the past decades has meant that the statistical data on the quantity of material-waste generation are not readily available. This is supported by Babatunde (2012), who believes that the situation is not any different in the Nigerian construction industry and these are important for the construction professionals to properly manage their disposal thereof. The statistical data relating the quantities of material waste for projects have not been well documented in the construction industry (Saidu, 2016; Saidu and Shakantu, 2017). There is no known model for predicting the quantities of onsite material waste for different materials, such as plastering mortar, block works and so forth; and also statistics on the waste generated are minimal in the Nigerian Construction industry (Babatunde, 2012).

Adewuyi and Otali (2013) argue that despite the allowance of 5 percentages usually made to take care of material waste when preparing estimate for project is inadequate because waste is found in several ways in project in Nigeria. Several researches evidence has revealed that previous studies from around Nigeria centered mostly on waste-control in the industry; as well as the necessary means for their control. Nonetheless, these studies have failed to develop a model for predicting material waste in building construction works. Hence, this study seeks to develop a model for predicting the volume of plastering waste in building works. To achieve this aim, null hypothesis was set as: there is non-statistically relationship between the volume of plastering material to be used and the volume of plastering waste recorded.

1.1 MATERIAL WASTE IN BUILDING WORKS

Construction waste is a well-known issue internationally and has adverse impact on overall progress of a project as well as the building society and nature (Nagapan et al., 2012). As result of this construction activity during the acquisition of raw materials has contributes to the generation of waste in the construction site (Saidu, 2016). In the cost of carrying out these construction activities during construction works has pose negative effects to the environment and the generation of waste, changes in living environment, sewage, reduction in environmental resources and energy usage (Mahayuddin et al., 2013).Due to the fact that the majority of this waste has not been properly controlled, thus causing considerable health and environmental challenges (Imam et al.,2008) and affecting the progress of many projects in Nigeria (Adewuyi and Otali 2013); (Saidu and Shakantu 2016).

According to Al-Hajj and Hamani (2011) construction waste is described as 'the difference between materials supplied to construction site and those materials placed for use on construction projects'. Nagapan et al. (2012) therefore, confirm that waste is any surplus or unwanted material constantly causing environmental difficulties.

On other hand, Construction waste was viewed by many scholars as construction process that generate waste but creates no value, such as rework , incorrect choice, programme of work and poor constructability that leads to waste generation in construction sites (Ma, 2011; Nagapan et al., 2012; Nagapan, Abdul Rahman and Asmi, 2012; Chikezirim and Mwanaumo, 2013 and Saidu, 2016).

Nugroho et al. (2013) also noted that construction waste is leftover material as the residue of construction activities and is caused by many factors, such as over production, handling error, accidents. Baldwin et al. (2010) further considered construction waste as the difference between the materials requested and those materials placed on construction projects. Gulghane and Khandve, (2015)later considered that construction waste involved unnecessary material produced directly or incidentally by the construction or industries which leads to waste generation in the construction sites.

Kwan et al. (2001) and Swinburne et al. (2010) contended that construction waste can be categories into; waste generated due to design and specification, and waste generated during construction activities. Formoso et al. (1999) and Swinburne et al. (2010) also contended that waste can be unavoidable (or natural waste), in which the necessary time to its reduction is higher than the economy produced; and waste can be avoidable when the cost of waste is greatly considerable higher than the cost to curb the waste .Construction material waste was also divided into cutting waste, application waste, transit waste and theft and vandalism (Muhwezi et al., 2012) and (Iqbal and Baig, 2016).

The plaster waste causes environmental damage that results from generating waste material and the economic and social aspects of waste that have an effects on the construction sites (Alencer et.al., 2010).

Ameh and Itodo (2013) also identified mortar from plastering/rendering as the most wasteful materials on construction sites. Eze et al. (2017) concluded that waste from mortar has the most wasteful materials on construction sites.

Material waste can also have a great effect on the progress of a construction project, since it precisely has a great effect on the construction costs (Nagapan et al., 2012). Madhavi et al. (2013);Gulghane and Khandve(2015) noted that , if management of material in construction projects is not well done it will produce a great project cost difference which also leads to waste generation on construction sites. Ameh and Itodo (2013) also identified waste from mortar as the highest cost production to the project cost and waste from concrete cost ten times the cost of mortar. Babatunde (2012) indicates that mortar from plaster has the highest cost of materials wastage on site. Babatunde (2012) further concluded that mortar used for plaster contributed to an average of 15.32% cost in the construction sites in Nigerian.

3 METHODOLOGY

This research employed the use of field study design approach by collecting quantitative data. The data were generated from the direct measurement of the on-site plastering waste volume /measured plastering volume to be used for 20 number 3-bedroom bungalow and 3-bedroom duplex respectively, all converted to cubic metre (volume). The table containing these details is presented in Appendix 1of this research.

The population for the study consisted 1345 3-bedroom residential bungalow and duplex construction project sites within Federal Capital Territory (FCT) area of Nigeria.

In this research, total twenty (20) 3-bedroom bungalow buildings and twenty (20) 3-bedroom duplex buildings were sampled. These were the active 3-bedroom construction projects as at the time of collecting the research data and to which access was made easier.

3-bedroom bungalows and duplexes were selected, because they were the most convenient forms of residential buildings for average Nigerians today.

In order to guarantee equal representation for each of the identified groups/strata in the population, stratified random sampling method was adopted. The respondents were first categorized into two different strata (3-bedroom bungalow and 3-bedroom duplex) before they were selected and randomly sampled accordingly.

For this research, primary sources were used to generate data.

This study collected primary data through quantitative research approaches which included the use of onsite site observation, measurements of quantity of plastering waste and recording on site was employed. This research also concentrated on mortar waste for plaster only as material considered in this research.

3.1 Field inspections:

The volume of the mortar waste and mortar used are determined by using bucket as the most commonly used on construction sites.

3.2 Method of data analyses

The research adopted the use of inferential methods of analysis to analyze the data and the results were presented in Tables.

Regression analyses are used to show the statistical relationship between one dependent variable and one or more independent variables data. They are also used as a basic predictive analysis. This study was conducted between January 2018 to June 2018.

4. RESULTS AND DISCUSSION

This section presents the relationship between volume of mortar to be used for plaster and volume of plastering waste by using linear - regression analyses and its discussions.

Also before running the regression analyses, test for normality was performed to ensure that the data were normally distributed using the Shapiro Wilks Test and the results revealed a normally distributed data and this allows for further regression analyses to be conducted.

4.1 Relationship between the volume of mortar to be used for plaster and the volume of plastering waste

The two analyses in Table 1 show the result of linear regression analyses performed between the volume of plaster waste and the volume of plaster to be used on 3-bedroom bungalows and duplexes. The results depict a linear and a strong correlation with the R-square values of 83.10% and 95.10% respectively. The probability values (0.000) were less than the 5% significance level; and the hypotheses were conducted at the 95% confidence level. Therefore, relationships are statistically significant; and the null hypotheses are rejected. The results show that any change in the either of the variables (X and Y) would lead to a corresponding change in the other.

TABLE 1: RESULTS OF REGRESSION ANALYSES BETWEEN VOLUME OF PLASTER TO BE USED AND VOLUME OF PLASTERING WASTE FOR 3-BEDROOM BUNGALOWS AND DUPLEXES

s/n	variables		Type of model	observation			inference		
	X	Y		Regression Equation (Y=a +bx)	R^2	Probability value	Strength of relationship	Remarks	Action on Hypothesis
1	Volume of mortar to be used for plaster	Volume of mortar waste for plaster	Linear regression	Plaster waste =2.35+ 3.205 plaster to be used Bungalow	83.10%	0.000	Very strong	Statistically significant	Reject Ho
2	Volume of mortar to be used for plaster	Volume of mortar waste for plaster	Linear regression	Plaster waste =1.334 + 4.276 plaster to be used duplex	95.10%	0.000	Very strong	Statistically significant	Reject Ho

Therefore, to predict the volume of plaster waste using the 3-bedroom bungalows and duplexes will be determined by: adding the constant value (2.35 and

1.334 respectively) to the coefficient value of the plaster used in volume (3.205 and 4.276 respectively), and multiplied by the volume of plaster to be used for building works.

These results corroborate with the findings of Saidu and Shakantu (2017) who observed that increase in the volume of materials used would leads to increase on the quantity of material waste and would also increases the cost of materials waste for project. Also, Ameh and Itodo (2013); Teo et al. (2009); Saidu and Shakantu (2016) observed that final cost of a building project also increases as results of material wastage on construction sites. Which more so means that as more materials are wasted, more is needed, thus affecting the final project costs.

5. CONCLUSION AND RECOMMENDATIONS

Construction waste has been identified as global environmental problem that have great effects on time, cost, quality and sustainability, as well as the progress of construction projects. Statistical data relating to the quantities of material waste for building works have not been well documented, and there is no known model for predicting the quantities of onsite material waste for different materials, such as plastering mortar, block works and so forth. This study was aims to develop a model for predicting plastering waste volume in building works. The study concludes that relationship between volume of plastering mortar to be used and volume of plastering waste are statistically significant in both bungalow and duplex works. This means that any change in any of the variables would bring a similarly change in the other variables. The model was developed from the linear -regression analysis. Based on these, the research recommends the use of these models by the building construction professionals at the early stage in order to have an idea on the likely volume of waste to be recorded, so that adjustment could be made in the areas of management and supervision of project at hand.

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Appendix1

DATA COLLECTED USING TABLE PROFOMA

Plastering Mortar 3-BED BUNGALOW			Plastering Mortar 3-BED DUPLEX		
S/n	Volume of materials used (m^3)	Volume of waste (m^3)	S/n	Volume of materials used (m^3)	Volume of waste (m^3)
1.	14.31	3.43	1.	35.52	8.53
2.	34.59	9.55	2.	32.28	7.73
3.	21.72	5.40	3.	46.89	11.27
4.	32.79	8.15	4.	36.60	8.80
5.	16.68	4.61	5.	91.62	22.00
6.	44.49	11.06	6.	23.68	5.28
7.	9.54	4.05	7.	21.52	4.81
8.	23.06	9.54	8.	31.26	6.99
9.	23.06	7.29	9.	24.40	5.47
10	14.31	2.25	10	61.08	13.66
11.	14.34	3.96	11.	69.84	16.77
12.	14.40	3.58	12.	83.31	20.00
13.	19.14	4.75	13.	25.16	6.03
14.	9.56	2.64	14.	60.02	14.40
15.	14.55	4.07	15.	46.56	10.4
16.	9.70	2.67	16.	47.94	10.71
17	9.60	2.98	17	16.77	4.41
18	12.76	3.54	18	40.01	8.94
19	12.76	3.98	19	69.84	12.14
20	14.55	2.02	20	69.84	13.88

Source: Researcher's field survey, 2018.

INFLUENCE OF AGGREGATES SIZES IN CONCRETES SUBJECTED TO HIGHER ELEVATED TEMPERATURES

M. M. Goga[1], S. Dahiru[2], A. Ishaq [3], Jummai I. Zailani [4]

[1,2,3,4] Civil Engineering Department, Nuhu Bamalli Polytechnic - Zaria, Kaduna State, Nigeria
Corresponding author: mohgoga@gmail.com

ABSTRACT
The effects of aggregates sizes in concrete subjected to various elevated temperature was experimented. Concrete mixes were made from aggregates of 10 mm, 14 mm and 20 mm sizes. Water content – cement ratio of 0.45 and mix ratio 1: 1 ½: 3 was adopted. The workability test was carried out on the concrete mixes also concrete cubes (150×150× **150** mm) were cast and subjected to 1, 3, 7, 14, 21 and 28 days of curing. The specimens casted were subjected to varying temperature of 100˚C, 300˚C, 500˚C, 700˚C and 900˚C before compressive strength test was conducted. The results obtained were recorded and analyzed. It was observed from the results that the residual compressive strength decreases with increase in temperature and the specimens made from Combined aggregate was found to have higher residual compressive strength than those from 10 mm, 14 mm or 20 mm aggregates at both room temperature and elevated temperatures.

Keywords: *Aggregates, Residual compressive strength, Temperature, Workability.*

1 INTRODUCTION

Concrete is the most widely used material in the world. It plays an important role in infrastructure and private buildings construction. Understanding the basic behaviour of concrete is essential for civil engineering students to become civil engineering professionals. Subjecting concrete to high temperatures leads to transformations and reactions that cause the progressive breakdown of cement gel structure and consequent lost in load bearing capacity.

Several process have been identified for the deterioration of concrete due to high temperature. These include decomposition of calcium hydroxide with time and water, expansion of lime on re-hydration, destruction of gel structure, phase transformation in some type of aggregate and development of micro-cracks due to thermal incompatibility between cement paste matrix and aggregate phase (Mohammed et al, 2009 and Chandramouli, 2011). Concrete composite material consisting of steel and concrete, its strength (compressive and bending strengths) decreases as the temperature increases. Heating considerably changes the strength and physical properties of concrete structural components such as beams, column, wall and slabs (Gruz, 1966).

2 MATERIALS AND METHODS

2.2 Cement

The cement used in the study was ordinary Portland cement, Dangote brand obtained from Obajana plant. The cement has an Initial setting time of 104 min. and Final setting time of 270 min. also the fineness and soundness of the cement was determined to be 0.036 and 2.26 respectively, which is in compliance with BS 197-1(2000).

2.3 Fine Aggregates

The fine aggregate used in this study consist of naturally (river) sand which was obtained from river bank in Zaria. The specific gravity of the fine aggregate was determined to be 2.665 which is in compliance with BS 812 (1975).

2.4 Coarse Aggregates

The coarse aggregate are crushed rock obtained from three different rock quarries with aggregate sizes ranging from 10 mm, 14 mm and 20 mm. The specific gravity of the fine aggregate was determined to be 2.660, 2.672 and 2.663 respectively. While the particle size distribution was found to be in conformity with BS 812 (1975). The Aggregate crushing value (ACV) and Water absorption of the aggregates was also determined to be 18.7% and 1.43%.

2.5 Preliminary Tests

Preliminary tests were conducted in the materials (Cement, Fine and Coarse aggregate). Concrete mixes were made from fine aggregate and coarse aggregates of 10mm, 14mm and 20mm sizes. Water content – cement ratio of 0.45 and mix ratio 1: 1 ½: 3 was adopted for the concrete mix design.

2.6 WORKABILITY TEST

Workability test was carried out on the concrete mixes made from varying sizes of coarse aggregates ranging from 10mm, 14mm and 20mm. The test was in accordance with BS EN 12350-2 (2009).

2.7 HEAT APPLICATION

The specimens were heated using an electric furnace with 1000°C maximum capacity. At first the furnace was operated to the required temperatures and then the specimens were put inside. The same procedure was repeated for 1 hours period. After the specified time duration, the specimens were taken out and cooled at room temperature for 24 hours.

2.8 COMPRESSIVE STRENGTH TEST

Compressive strength test on concrete cubes produced was carried out. Five Hundred and Four (504) cube specimens were produced from concrete mixes namely; M_1, M_2, M_3, and M_4 where; M_1, M_2, M_3, and M_4 stands for Concrete made from Combined aggregates, 10 mm, 14 mm and 20 mm aggregate sizes respectively, using steel moulds of size 150 x 150 x 150 mm; they were weighed to determine the density then subjected to varying temperatures of 100˚C, 300˚C, 500˚C , 700˚C and 900˚C before tested for compressive strength at 1, 3, 7, 14, and 28 days in accordance with BS EN 12390-3 (2009).

3 RESULTS AND DISCUSSION

The tests from both preliminary as well as the major tests on the heated concrete specimens were discussed for reasonable conclusion and recommendation.

3.1 SLUMP TEST

The workability test of different concrete mixes was carried and M_1 concrete was found to be more workable, which can be attributed to its proper particles distribution, the results were tabulated in Table 1 below.

TABLE 1. WORKABILITY TEST RESULTS

Test	Slump (mm)	Overall Average (mm)	ASTM 143, EN12350-2 1997
M_1	62		
M_2	64		
M_3	57	61	20 – 80
M_4	59		

3.2 COMPRESSIVE STRENGTH TEST

The concrete cube specimens were tested in accordance with ASTM C297. The results were presented in Figure 1 to 7 below.

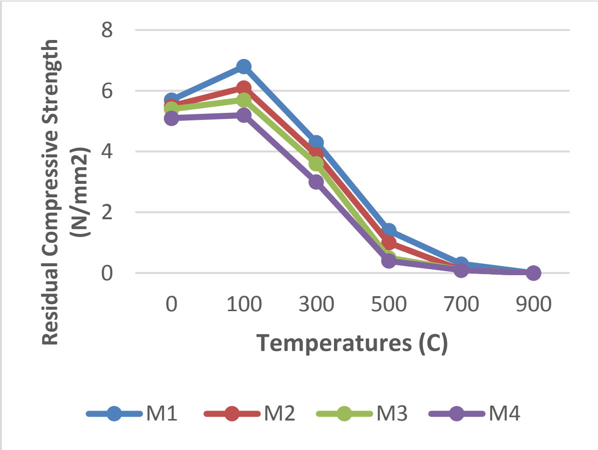

FIGURE 1: RESIDUAL COMPRESSIVE STRENGTH @ 1 DAYS AGE, ELEVATED TEMPERATURES FOR 30 MINUTES

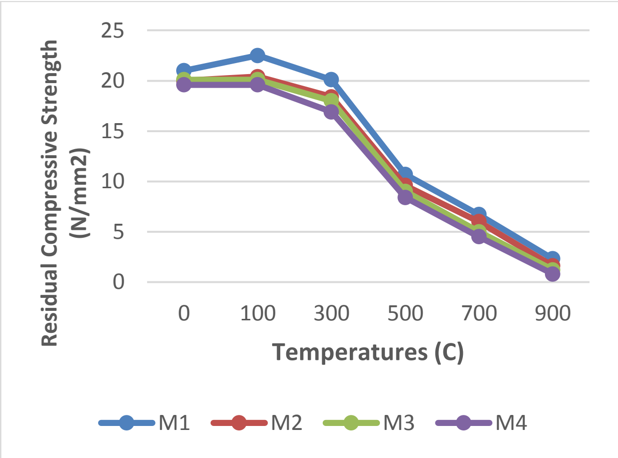

FIGURE 2: RESIDUAL COMPRESSIVE STRENGTH @ 3 DAYS AGE, ELEVATED TEMPERATURES FOR 30 MINUTES

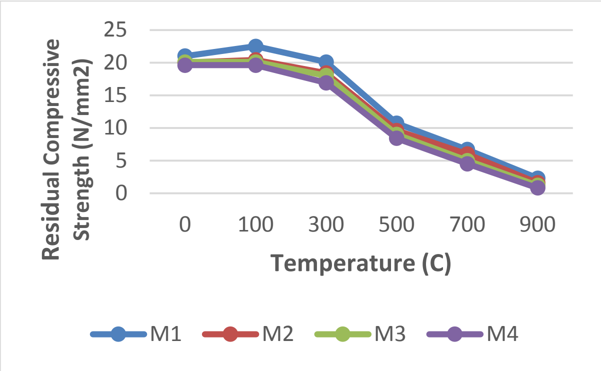

FIGURE 3: RESIDUAL COMPRESSIVE STRENGTH @ 7 DAYS AGE, ELEVATED TEMPERATURES FOR 30 MINUTES

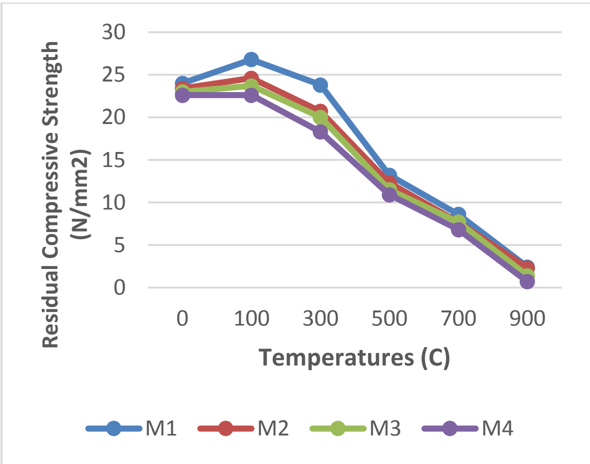

FIGURE 4: RESIDUAL COMPRESSIVE STRENGTH @ 14 DAYS AGE, ELEVATED TEMPERATURES FOR 30 MINUTES

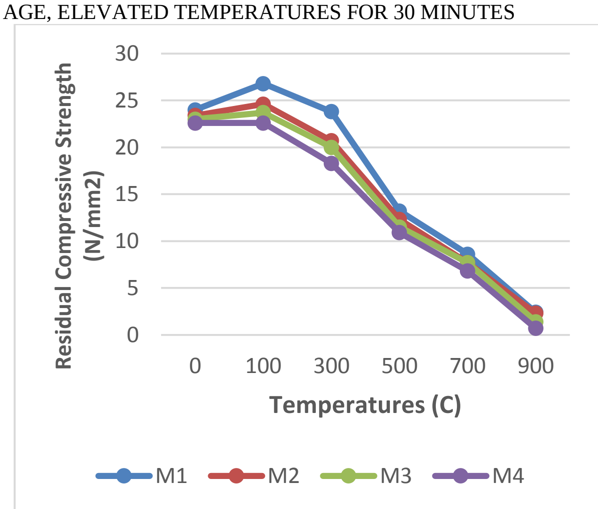

FIGURE 5: RESIDUAL COMPRESSIVE STRENGTH @ 28 DAYS AGE, ELEVATED TEMPERATURES FOR 30 MINUTES

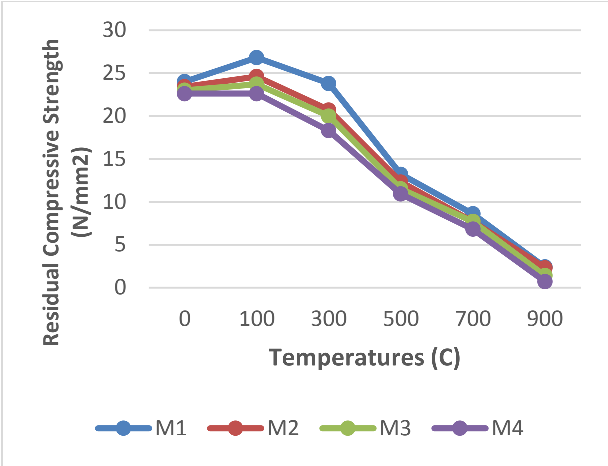

FIGURE 6: RESIDUAL COMPRESSIVE STRENGTH @ 14 DAYS AGE, ELEVATED TEMPERATURES FOR 60 MINUTES

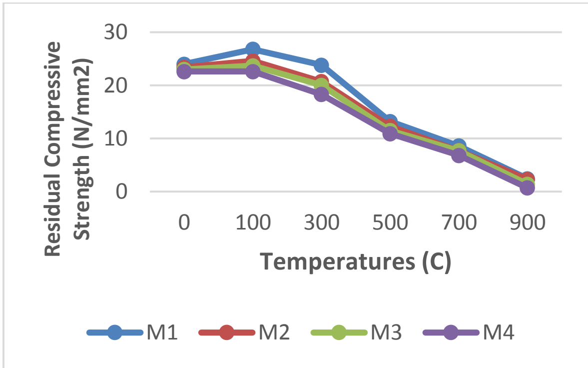

FIGURE 7: RESIDUAL COMPRESSIVE STRENGTH @ 28 DAYS AGE, ELEVATED TEMPERATURES FOR 60 MINUTES

4 CONCLUSION

From the analysis of the results presented so far in this research, generally the following Conclusion and Recommendation can be made.

1. The Cement used is of good quality and complied with the BS 197-1(2000) requirement.
2. The fine and coarse aggregates used are as well of god quality and in conformity with BS 812:1975
3. The concrete produced from Combined aggregate was recorded to have higher compressive strength than others at both room temperature and elevated temperatures, as a result of proper distribution of the aggregate particles.
4. The critical Temperature beyond which the residual compressive strength drops below 75% of normal strength was found to be 600˚C and above as it was also proven by Ashok at el (2015).

5. Rapid increase in residual compressive strength amounting to 18% above the strength of the unheated specimens was recorded at 100˚C. This increase was due to the higher acceleration of hydration and drying of concrete at high temperature.

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ASSESSMENT OF THE EFFECT OF CHICKEN FEATHER ON THE BIOGAS PRODUCTION OF HORSE DUNG

A. Ishaq,[1] S. Dahiru[1], I. Garba[1], and M. I. Alfa*[2]
[1]Department of Civil Engineering, Nuhu Bamalli Polytechnic, Zaia, Nigeria
[2]Department of Civil Engineering, University of Jos, Jos, Nigeria
*Corresponding Author: E-mail: aleeruks4real@gmail.com, Tel: +234080351688008

ABSTRACT
This study assessed the effect of Chicken feather (CF) on the biogas production potentials of horse dung (HD) co-digested anaerobically. The study was carried out simultaneously in eight (8) experimental designs using eight (8) identical digesters. The first two (2) digesters contained 100%HD (6kg) and 100%CF (6kg) respectively, the next three (3) digesters contained 75%HD, 50%HD and 25%HD (4.5kg, 3kg and 1.5kg) respectively while the last three (3) digesters contained 75%HD:25%CF, 50%HD:50%CF and 25%HD:75%CF respectively. The Cumulative biogas production from the respective set ups were described using the Modified Gompertz equation. Paired t-test carried out on the daily biogas production using Minitab 14.12 software showed significant difference for 75%HD and 50%HD but no significant difference for 25%HD at 95% Confidence level. Chicken feather had an overall negative effect of 11.82% on all parameters examined. The study concluded that chicken feather has negative effect on the biogas production from horse dung.
Keywords: ***Biogas, Chicken feather effect, Horse dung, Gompertz model.***

1 INTRODUCTION

The fact that energy is a basic tool for development is no longer news especially amongst developmental experts but the exploration and exploitation of energy sources that will serve as a safe, renewable and environmentally sustainable alternative to those of fossil origin is the main challenge confronting researchers (Budyono *et al.,* 2001; Anushiya, 2010; Alfa *et al.*, 2014a; Owamah, 2014a). Biogas has since been recognized as one of such alternative energy that will be a major catalyst towards the attainment of the sustainable development goals of the United Nations (United Nations, 2015; Alfa *et al.,* 2013a; Dahunsi *et al*, 2017a).

While significant progress has been made in biogas production research, exploration into various substrates for biogas production is receiving serious attention among researchers. This has been necessitated especially by the need to develop various alternative methods of handling organic waste and the consistent need for energy for meeting the increasing demand for safe, renewable and environmentally sustainable energy globally (Dahunsi *et al.,* 2017b). The focus therefore has been the exploration and exploitation of various substrates ranging from Animal wastes, agricultural residues amongst others. Extensive studies have therefore been carried on the production of biogas from various substrates either singly digested or co-digested (Ojolo *et al.,* 2007; Ojolo *et al.,* 2012; Owamah *et al.,* 2014a; Dahunsi *et al.,* 2017a; Dahunsi *et al.,* 2017b). Besides the advantage of biogas production from these substrates, the technology has also proven to be an alternative and eco-friendly method for handling organic waste (Alfa *et al.,* 2013b; Owamah *et al.,* 2014b).

The ancient city of Zaria is experiencing an increasing population of horses owing to its traditional connotation as a cardinal part of the northern part of Nigeria (Useh *et al.*, 2005; Mshelia *et al.*, 2016). As a result, there is an attendant increase in the dung produced by these animals which if not managed will continue to constitute a major environmental hazard. Similarly, with the significant increase in consumption of Chicken in Zaria especially, handling the feathers has become a major challenge (Damisa and Hassan, 2009; Otalu *et al.*, 2011; Olagunju *et al,* 2012). Indiscriminate disposal of the feathers which usually is the last resort for the slaughterhouses has constituted a major environmental challenge which has made a need for research into alternative means of handling these waste very urgent.

This study was conducted therefore to investigate the potentials of biogas production from Chicken feather and horse dung and assess the effect of the co-digestion of chicken feather on the production of biogas from horse dung.

2 METHODOLOGY

2.1 COLLECTION AND PREPARATION OF BIOMASS

Horse dung was collected fresh and free from impurities from the ancient Zaria city and was transported in sacks to the research ground in the Department of Water Resources and Environmental Engineering, Ahmadu Bello University Zaria. The dung was grounded to reduce the surface area and stored prior to use. The Chicken feather (CF) on the other hand were obtained from the poultry slaughterhouse of the Zaria city market. They were washed with clean water to remove sand and other particles, stacked into sacks and transported to the Laboratory for pre-treatment. The washed chicken feathers were placed in dryer boxes at 45ºC (Ishaq *et al.*, 2016) after which they were grinded into smaller particles of sizes not greater than 4mm using a hammer mill. They were also stored prior to use.

2.2 Experimental Design, Substrates Preparation and Anaerobic Digestion

The study was carried out in eight (8) experimental designs using eight (8) digester set ups labelled A-H. Digester A and B contained 100%HD (6kg) and 100%CF (6kg) respectively (controls), Digester C-E contained 75%HD, 50%HD and 25%HD (4.5kg, 3kg and 1.5kg) respectively while Digesters F-H contained the same content of digesters C-E but respectively co-digested with 25%CF, 50%CF and 75%CF (1.5kg, 3kg and 4.5kg) respectively. Each substrate combination was mixed with clean water in ratio 1/1 (weight/volume) and mixed to form slurry after which they were fed into the respective digesters occupying about two-third of the digester space.
The eight identical digesters used in this study were 25-litre in size each with a separate air-tight gas collection system comprising of a water jacket and an inverted gasholder. The digester contained an in-built mechanical stirrer for substrate mixing (Alfa *et al.*, 2012; Dahunsi and Oranusi, 2013; Alfa *et al*., 2014a; Dahunsi *et al*., 2016).
The slurries respectively prepared for Digesters A-H were charged into each of the digestion tanks through an inlet pipe (Alfa *et al.*, 2012; Alfa *et al*., 2014a; Dahunsi *et al.*, 2016). The batch anaerobic digestion was carried out for a retention period of 37 days under mesophilic conditions (Owamah *et al.*, 2014a).

2.3 Measurement of Operational Parameters and Gas Production

Evaluation of operating parameters was carried out periodically so as to assess the efficiency and stability of the anaerobic digestion process. The parameters include daily measurement of gas production, daily measurement of ambient and digester temperatures (twice daily) and daily measurement of slurry pH. The temperature measurement was carried out using 2/1ºC thermometers inserted into the temperature probes on the respective digesters, while the pH of the digester content was measured at ambient temperature using a pH meter model pHS-2S, SHANGHAI JINYKE REX, CHINA. The measurement of gas production on the other hand was carried out using the calibration on the gasholder as described previously in Alfa *et al.* (2014a) and Owamah *et al.* (2014a).

2.4 Modelling of the Cumulative Biogas Production

The cumulative biogas productions from the respective digesters were described using the modified gompertz model in order to study the kinetics of biogas production (Yusuf *et al.*, 2014; Alfa *et al.*, 2016). The modified Gompertz model is presented in Eq. (1).

$$Y(t) = Aexp\left\{-\exp\left[\frac{\mu_m e}{A}(\lambda - T) + 1\right]\right\} \qquad (1)$$

Where, *Y(t)* is the cumulative biogas produced (m^3) at any time (*t*), *A* is biogas production potential (m^3), λ is Lag phase period (days), which is the minimum time taken to produce biogas or time taken for bacterial to acclimatize to the environment, *t* is the cumulative time for biogas production (days) and *e* = mathematical constant (2.718282) while μ is the maximum biogas production rate. The constants *A*, μ_m and λ were determined using the nonlinear regression approach with the aid the solver function of the Microsoft excel tool pack (Budiyono *et al*., 2010; lay *et al*., 1996; Matheri *et al*., 2015, Alfa *et al*., 2016).

2.4 Statistical Analysis

The daily production from the 75%HD, 50%HD and 25%HD singly digested were respectively compared with their co-digestions with CF (75%HD:25%CF, 50%HD:50%CF and 25%HD:75%CF) using paired t-test at 95% confidence level. A *P-value* less than 0.005 was taken as statistically significant. The 95% confidence interval (CI) was also used to buttress this. The t-test was carried out using MINITAB Statistical Software version 14.12.0.

3 RESULTS AND DISCUSSION

The daily biogas productions from the respective combinations are presented in Figure. 1, while the results of the comparison of the daily biogas production from the respective single digestions of HD and their corresponding co-digestion with varying proportion of CF are presented in Table 1.

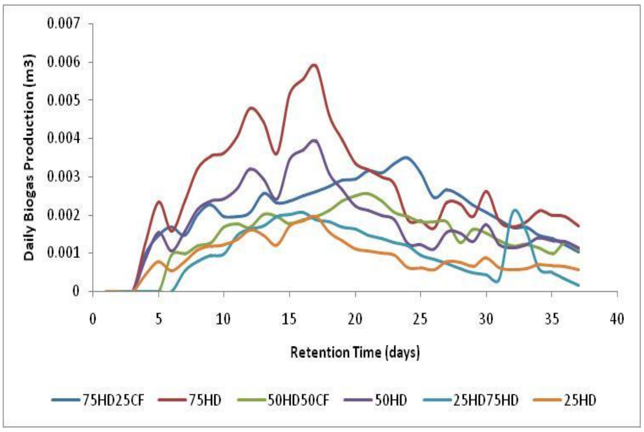

FIGURE 1: DAILY BIOGAS PRODUCTION FROM THE RESPECTIVE CO-DIGESTIONS

Table 1 shows that the difference between the daily biogas production from 75HD25CF and 75HD was statistically significant at 95% Confidence level (*P = 0.001;* 95%CI = -0.001052, -0.000281). Similarly, the difference between the daily biogas production from 50HD50CF and 50HD was statistically significant at 95% Confidence level (*P = 0.004;* 95%CI = -0.000606, -0.000125). On the contrary, the difference between the daily biogas production from 25HD75CF and 25HD was not statistically significant at 95% Confidence level (*P = 0.348;* 95%CI = -0.000078, 0.000219).

TABLE 1: TABLE 1: RESULTS OF THE T-TEST CARRIED OUT ON THE DAILY BIOGAS PRODUCTION

Substrates	Mean ± SD	*P-Value*	95% Confidence Interval
75HD25CF	0.002032±0.000886	-0.001052, -0.000281	0.001
75HD	0.002699±0.001453		
50HD50CF	0.001433±0.000718	-0.000606, -0.000125	0.004
50HD	0.001799±0.000969		
25HD75CF	0.000970±0.000698	-0.000078, 0.000219	0.348
25HD	0.000900±0.000484		

These results show that the co-digestion of Chicken feather with Horse dung possibly has an inhibitive effect on the biogas production from Horse dung. The reduction in biogas yield after co-digestion with CF could better be explained by the results of the digestion of 100%HD and 100CF presented in Figure. 2.

Figure 2 shows that, while the maximum daily biogas production was 7.86 x 10-3 m3 for 100%HD, it was 7.86 x 10-5 m3 for 100%CF. More so, gas production for the 100%HD started on the 4th day of set up and continued for till the end of the experiment, it started on the 13th for the 100%CF and stopped on the 17th day.

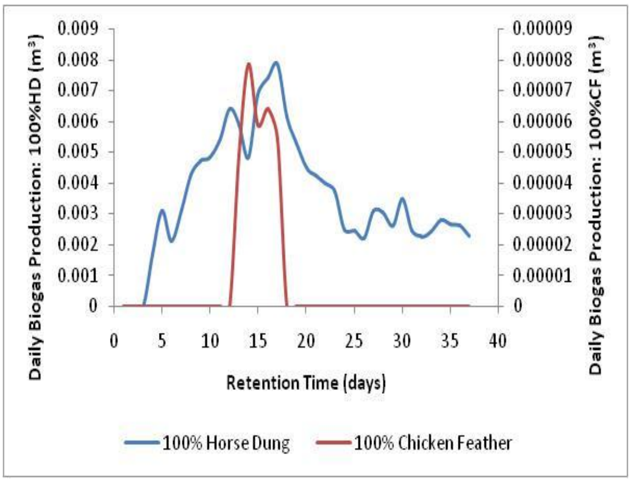

FIGURE 2: DAILY BIOGAS PRODUCTION FROM 100%HD AND 100%CF

This probably indicates that the digestion environment may not be conducive for the microbial profile that can degrade CF. The study of Nam *et al.* (2002) explains this. They reported that *F. islandicum AW-1* was able to degrade chicken feather at a temperature of 70ºC which is within the thermophilic range. Throughout the experiment, the average ambient temperature of Samaru was 37±2.76ºC while the average digester temperatures for all digesters ranged between 32±3.54ºC and 34±4.23ºC. Since this study was conducted within the mesophilic temperature range, this could account for the low biogas production from chicken feather and the consequent inhibitive effect. This organism is rarely implicated in other similar studies carried out in the mesophilic temperature range (Alfa *et al.*, 2014b; Owamah *et al.*, 2014b; Alfa *et al.*, 2017). The extremely low gas production from the chicken feather could also be attributable to the slurry pH which ranged between 5.3 and 6.3 (Figure 3). Meanwhile, in a study by Suntornsuk and Suntornsuk (2003), *Bacillus sp* which has been previously implicated in related studies (Alfa *et al.*, 2014b; Owamah *et al.*, 2014b; Alfa *et al.*, 2017) was only able to degrade chicken feather at a pH of 9. This probably accounts for the low production and the inhibitive effect since most methanogens are acid lovers (Karki *et al.*, 2005).

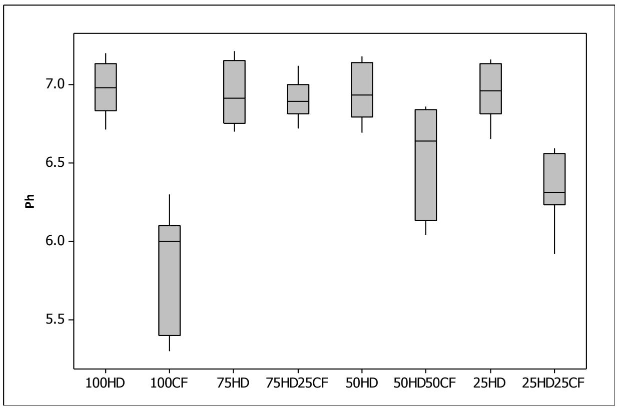

FIGURE 3: BOX PLOT OF PH FOR THE RESPECTIVE SUBSTRATES COMBINATIONS

The cumulative biogas productions from the respective substrates combinations (both measured and estimated by model) are presented in Figure. 4. The sum of square error (SSE) and the coefficients of determination (R^2) obtained for the curve fitting (Table 2) shows that the model was able to effectively describe the cumulative production, thus the kinetics parameters are reliable**.**

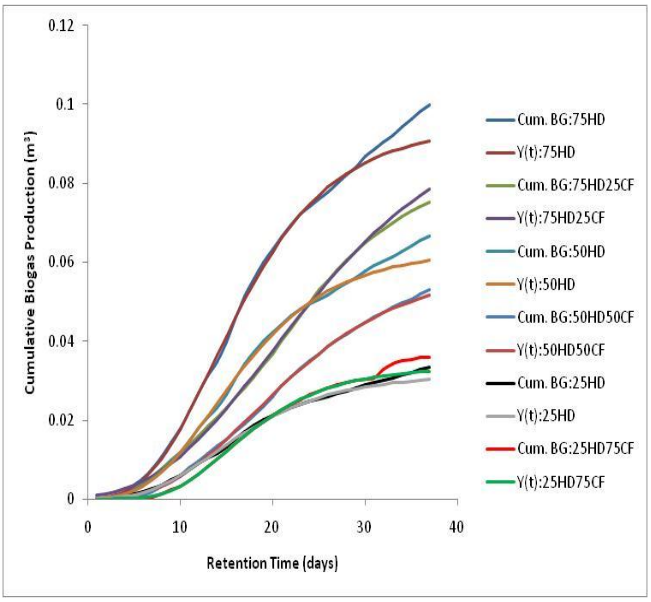

FIGURE 4: CUMULATIVE BIOGAS PRODUCTION MODELLED USING THE MODIFIED GOMPERTZ EQUATION

TABLE 2: SUM OF SQUARE ERROR AND COEFFICIENTS OF DETERMINATION FOR THE MODEL FITTING

Substrates	Sum of Square Error (SSE)	Coefficient of Determination (R^2)
75HD	1.80E-05	0.9985
75HD25CF	1.85E-05	0.9987
50HD	8.02E-06	0.9993
50HD50CF	5.53E-06	0.9996
25HD	2.00E-06	0.9998
25HD75CF	3.48E-07	1

Similarly, the total biogas production estimated using the modified gompertz models as well as other kinetic parameters are presented in Table 3. In addition to this the estimation of the percentage differences between 75HD, 50HD, 25HD and their respective co-digestions are also presented in Table 3. These percentage differences were used as the indicators of the effects.

The total biogas production and the average daily biogas production showed similar trends. Chicken feather had a negative effect of 13.29% on the total biogas production from 75HD, a negative effect of 14.39% on the total biogas production from 50HD and a positive effect of 7.01% on the total biogas production from 25HD. Similarly, chicken feather had a negative effect of 38.06% on the average daily biogas production from 75HD, a negative effect of 29.16% on the average daily biogas production from 50HD and a positive effect of 22.70% on the average daily biogas production from 25HD. The trend was however different for the maximum biogas potential. It was positive (6.36%) for 75HD, negative (-5.84%) for 50HD and positive (6.07%) for 25HD. Lastly, table 3 shows that the co-digestion with chicken feather increased the lag phase periods, that is the time required for bacteria to acclimatize with the slurry environment thus delaying the time for biogas production by 15.53%, 30.44% and 37.29% respectively.

Furthermore, Table 3 shows that chicken feather had an average negative effect of 6.89%, 14.84% and 27.75% on the total biogas produced; average daily biogas production and lag phase respectively, an average positive effect of 2.20% on the maximum biogas potential and an overall negative effect of 11.82% on all parameters examined. These variations are further expressed in Figure. 5

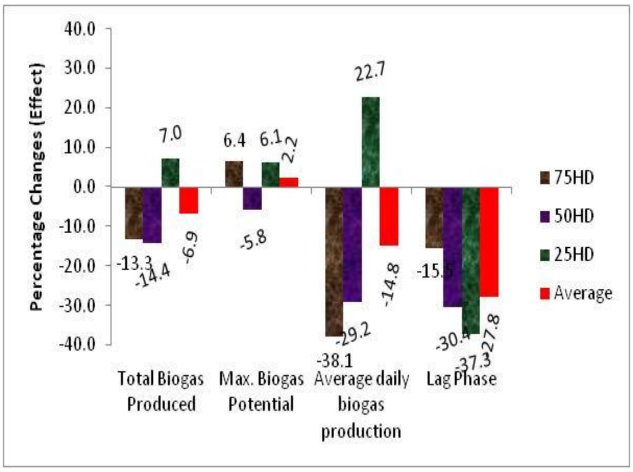

FIGURE 5: PERCENTAGE CHANGES IN TOTAL BIOGAS PRODUCTION, MAXIMUM BIOGAS POTENTIAL, AVERAGE DAILY BIOGAS PRODUCTION AND LAG PHASE

4 CONCLUSION

The study concludes that although chicken feather could be biodegradable, the pH range of the digester does not permit easy degradation at mesophilic temperature range. Thus, chicken feather co-digested with horse dung had an overall negative (inhibitive) effect of 11.82% on all kinetic parameters of the biogas production. Meanwhile, the single digestion of horse dung at varying proportions demonstrated that horse dung is a viable biomass for biogas production. The study therefore recommends further biomass conversion techniques at neutral pH that can be suitable for the conversion of chicken feathers into energy under mesophilic temperature range and serve as an alternative waste management technique.

5 CONFLICT OF INTERESTS

NONE DECLARED

6 ACKNOWLEDGEMENT

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EFEECT OF BACILLUS *COAGULANS*-INDUCED PRECIPITATE ON SOME PROPERTIES OF LATERITIC SOIL AS A ROAD CONSTRUCTION MATERIAL

[1]Osinubi, K. J., [1]Eberemu, A. O., [1]Ijimdiya, T.S., [1]Yohanna, P and [2]Sani, J.E.
[1]Department of Civil Engineering, Ahmadu Bello University, Zaria, Kaduna State, Nigeria.
[2]Department of Civil Engineering, Nigerian defence Academy, Kaduna, Kaduna State, Nigeria
Corresponding Author's Email: paulyohanna45@yahoo.co.uk

ABSTRACT
The study evaluated the potential of microbial-induced calcite precipitation (MICP) using 'beneficial/ bacteria (Bacillus *coagulans)* suspension density on some engineering properties of lateritic soil for road construction application. Soil samples were treated with B. *coagulans* at one-third (1/3) pore volume in stepped suspension density of 0, 1.5 x 10^8, 6.0 x 10^8, 12.0 x 10^8, 18.0 x 10^8 and 24.0 x 10^8/ml. Index and unconfined compression tests were carried out on specimens prepared at optimum moisture content (OMC) and compacted with reduced British Standard light (RBSL) energy. Cementitious reagent containing 3 g of Nutrient broth, 20 g of urea, 10 g of NH_4Cl, 2.12 g of $NaHCO_3$ and 2.8 g $CaCl_2$ per litre of distilled water was injected by gravity into the specimens after compaction. Statistical analysis was carried out on results obtained using analysis of variance (ANOVA) with the Microsoft Excel Analysis Tool Pak Software Package to determine the levels of significance of effect of B. *coagulans* on the properties of the soil. Results obtained show that liquid limit and plastic limit values increased, while plasticity index and linear shrinkage values decreased with higher B. *coagulans* suspension density. The unconfined compressive strength (UCS) value of specimens increased with higher B. *coagulans* suspension density. Statistical analysis of results obtained show that B. *coagulans* had significant effect on plasticity index, linear shrinkage and UCS values of the treated specimens. An optimal 24.0 x 10^8/ml B. *coagulans* suspension density significantly improved the engineering properties of the treated soil and met the requirements of the Nigerian General Specifications of not more than 35 % passing sieve No. 200, maximum plasticity (PI) index of 30 % and maximum liquid limit (LL) of 50 % for it to be used as subgrade material in road construction.

Keywords: : *Atterberg limits; B. coagulans;Microbial-induced calcite precipitation; Optimum moisture content; Unconfined compressive strength.*

1 INTRODUCTION

Soil improvement techniques via microbiological treatments rely on the capacity of soil microorganisms to precipitate substances with stabilizing characteristics, in particular calcium carbonate (Morales *et al*., 2011). The use of industrially manufactured chemical additives like lime and cement has increased the cost of improving deficient soils (Neville, 2000). Production of cement has been found by the nature of its chemistry to produce large quantities of carbon (IV) oxide (CO_2) for every tonne of its final product (Sear, 2005) thereby increasing greenhouse effect and global warming. Open burning of industrial and agricultural wastes (e.g., rice husk, sugarcane bagasse, locust bean pods, etc.) to obtain ashes with pozzalanic properties for use in soil improvement equally emit carbon (II) oxide (CO) to the environment. Replacement of this approach with more environmentally friendly techniques like microbial-induced calcite precipitation (MICP) will reduce the overall negative impact of the conventional stabilization process.

Boquet *et al*. (1973) reported that almost all bacteria are capable of precipitating calcium trioxocarbonate ($CaCO_3$. Studies by Stocks-Fischer *et al*. (1999) and Dick *et al*. (2006) on carbonate precipitation by bacteria were carried out using ureolytic bacteria which are able to influence precipitation of calcium carbonate by the production of a urease enzyme through a process termed microbial-induced calcite precipitation (MICP). This enzyme catalyses the hydrolysis of urea ($(NH_2)_2$) to carbonate (CO_3^{2-}) and ammonium ($2NH_4^+$), resulting in an increase in pH and carbonate concentration in the bacterial environment as shown in eq. (1)(Stocks-Fischer *et al*. 1999; Morales *et al*., 2011):

$$CO(NH_2)_2 + 2H_2O \xrightarrow{Urease} 2NH_4^+ \ CO_3^{2-} \quad (1)$$

In the presence of calcium ions supplied by the calcium source, the solution can become supersaturated with respect to calcium carbonate leading to its precipitation as shown in eq. (2):

$$2CO_3^{2-} + 2Ca^{2+} \rightarrow 2CaCO_3 \quad (2)$$

Precipitation of calcium carbonate crystals occurs by heterogeneous nucleation on the bacterial cell wall, once super saturation is achieved. Castanier *et al.* (1999), Rowshanbakhta *et al*., *et al*. (2016) and Karim *et al.* (2016) suggested that the production of carbonate by hydrolysis of urea can be easily controlled, and it allows for the production of high concentrations of carbonate within a short amount of time.

There are two major approaches to *in-situ* soil microbial-induced calcite precipitation (MICP), namely, (i) bio-augmentation, in which specific bacteria is added with or without growth media to the treated site; (ii) bio-stimulation, in which indigenous soil bacteria are provided with a growth media in order to stimulate $CaCO_3$ precipitation as reported by Gat *et al*. (2011). *In-situ* bio-augmentation requires the introduction of large amounts of monoclonal bacterial cultures into an existing micro-fauna. Bio-stimulation encourages the growth of a certain guild of the native soil micro-fauna through the introduction of specific growth conditions (Gat *et al*. (2011). Harkes *et al.* (2008) and Paassen *et al.* (2009) reported improvement in unconfined compressive strength with microbial treatment due to calcite formed that stiffened the soil by binding the soil particles through calcite precipitation process. Based on the study carried out by Ismail *et al.* (2002), the amount of calcite required to significantly modify geotechnical properties of soil may depend on initial soil density, grain shape, and rate of calcite precipitation.

Lateritic soils are formed in tropical and sub-tropical regions; very rich in iron and aluminium (Townsend, 1985). These soils are essentially the products of tropical or sub-tropical weathering. The suitability of these types of soils for use in bio-cementation is based on its large pore throat size which enables free movement of the microbes within the soil to enhance bio-cementation and bio-clogging of the soil. Rebata-Landa (2007) reported that the optimum range of grain size for the bio-cementation process is between 50 and 400 µm as bacterial activity cannot take place in very fine soils, while large amounts of calcites are required to promote effective improvements in very coarse soils. Bacteria with size ranging from 0.3 to 2 µm can move freely within sandy soil with particle size of 0.05 to 2.0 mm (Maier *et al*., 2009). Thus, this study focused on the evaluation of the effect of soil microbes (B. *coagulans*) on some engineering properties of compacted lateritic soil. The objectives of the study include the evaluation of environmentally friendly compacted lateritic soil – B. *coagulans* mixture as a road construction material

2 MATERIALS AND METHODS

2.1 MATERIALS

Soil: The soil sample used for this study was obtained by the method of disturbed sampling from an erosion site in Abagana (68°24'31''N and 27°52'11''E), Njikoka Local Government Area of Anambra state, South East, Nigeria.
Microorganism: Urease positive bacteria B. *Coagulans.* American Type Culture Collection (ATCC) classified the microorganism as ATCC 8038 was used in the study. It is a rod shaped gram positive bacterium.
Cementation reagent: The cementation reagent used contains 3 g of Nutrient broth, 20 g of urea, 10 g of NH_4Cl, 2.12 g of $NaHCO_3$ and 2.8 g $CaCl_2$ per litre of distilled water in accordance with that described by Stocks-Fischer *et al*. (1999).

2.2 METHODS

Isolation of the bacterium specie

The bacterium was isolated from the soil by serial dilution. Serial dilution entails adding 1 ml of this medium(i.e., original bacterial culture) to 9 ml of sterile water which makes a 1:10 dilution; adding 1 ml of the 1:10 dilution to 9 ml of sterile water makes a 1:100 dilution; and so on. The concentration of bacteria per millilitre decreased by 9/10 for each dilution. Further dilutions were made in ratio of 1:1000, 1:10000, 1:100000, 1:1000000, or even 1:10000000 if the original culture contained an extremely large number of organisms. The isolates were stored at 4°C in nutrient medium prior to classification and characterization.

The culture medium and growth conditions

The procedure adopted was in accordance with that described by Stocks-Fischer *et al*. (1999). B. *coagulans* ATCC 8038 was used throughout the study Medium (Tris ±YE). The stock and pilot cultures contained the following ingredients per litre of distilled water: Tris±HCl, 130 mM (pH 9.0); $(NH_4)2SO_4$, 10 g; and yeast extract, 20 g; to which 1.5% agar was added to obtain a solid medium for the stock culture. $CaCO_3$ precipitation experiments was carried out in liquid medium (urea±$CaCl_2$) containing the following per litre of distilled water: nutrient broth (Bacto), 3 g; urea, 20 g; NH_4Cl, 10 g; and $NaHCO_3$, 2.12 g (equivalent to 25.2 mM). The pH of the medium was adjusted to 6.0 with 6 N HCl prior to autoclaving. 10 ml of filter sterilized solution containing 2.80 g $CaCl_2$ was added afterward and the final pH of the medium was measured. B. *coagulans* was grown at 30°C under aerobic conditions for stock and pilot cultures. Broth cultures were incubated in a water bath shaker (Lab-line, Model 3540) operated at 200 rpm. Cell concentrations were determined by viable cell counting on Tris±YE plates.

Index properties

Laboratory tests were performed on the natural and treated soils in accordance with British Standards BS 1377 (1990) and BS 1924 (1990), respectively. Samples for Atterberg limits were passed through No. 40 sieve (425µm aperture). The soil specimens were first mixed each with the bacteria solution and cementition reagent at optimum moisture content (OMC), then air-dried at room temperature before tests were carried out on the respective treated soil specimens. The optimal growth temperature for Bacilus species is 38°C beyond which the potency for urease activity decreases. Specimens were air-dried at room temperature (25 ± 2°C) which is within the limit for optimal performance of the microbes.

Compaction: Tests to determine the moisture – density relationships were carried out in accordance with BS 1377 (1990) using the reduced British Standard light (RBSL) energy. The RBSL compaction energy involves 3 layers each receiving 27 blows in 1000 cm^3 compaction mould.

Unconfined compressive strength: The unconfined compressive strength (UCS) tests were performed in accordance with BS 1377; 1990 part (7). Specimens were treated before compaction with B.*coagulans* suspension at one-third (1/3) pore volume as recommended by Rowshanbakhta *et al*., (2016) in stepped suspension density of 0, 1.5 x 10^8, 6.0 x 10^8, 12.0 x 10^8, 18.0 x 10^8 and 24.0 x 10^8/ml, respectively. Rowshanbakhta *et al*., (2016) reported that reduction of the volume of injected voids to up to one third of the pore volume did not significantly affect the improvement performance of the treated soil which is economically viable for engineering applications. Specimens were prepared at OMC and compacted with RBSL energy. Cementitious reagent containing 3 g of Nutrient broth, 20 g of urea, 10 g of NH_4Cl, 2.12 g of $NaHCO_3$ and 2.8 g $CaCl_2$ per litre of distilled water was injected into the soil after compaction by gravity (i.e Cementation reagent containing 3 g of Nutrient broth, 20 g of urea, 10 g of NH_4Cl, 2.12 g of $NaHCO_3$ and 2.8 g $CaCl_2$ per litre of distilled water was poured on the compacted specimen and allowed to flow by gravity until saturation was achieved. The procedure was carried out in three cycles at 6 hours interval. The specimens were cured for 48 hours and thereafter placed in a load frame machine driven strain controlled at 0.02 rev/sec until failure occurred. The UCS of the specimen was determined at the point on the stress-strain curve at which failure occurred. The UCS was calculated using eq. (3):

$$\text{Uncomfined compressive strength} = \frac{\text{Failure load} \times \text{Load ring factor}}{\text{Surface area of specicmen}} \, (\text{kN}/m^2) \quad (3)$$

with failure load is in kN, load ring factor (has no unit) and surface area in m^2

3 RESULTS AND DISCUSSION

Index Properties

The natural lateritic soil classifies as A-4 (2) soil in the AASHTO classification system (AASHTO, 1986) and SC in the Unified Soil Classification System (USCS) (ASTM, 1992). A summary of the properties of the natural soil is given in Table 1. The particle size distribution curve for the natural soil is shown in Figure 1.

TABLE 1: PROPERTIES OF THE NATURAL LATERITIC SOIL

Property	Quantity
Percentage Passing No. 200 Sieve	35.4
Natural Moisture Content, %	11.3
Liquid Limit, %	37.5
Plastic Limit, %	19.3
Plasticity Index, %	18.2
Specific Gravity	2.62
AASHTO Classification	A-4 (2)
USCS	SC
Maximum Dry Density, Mg/m^3	1.76
Optimum Moisture Content, %	16.2
Unconfined Compressive Strength, kN/m^2	197.8
Colour	Reddish brown
Dominant Clay Mineral	Kaolinite

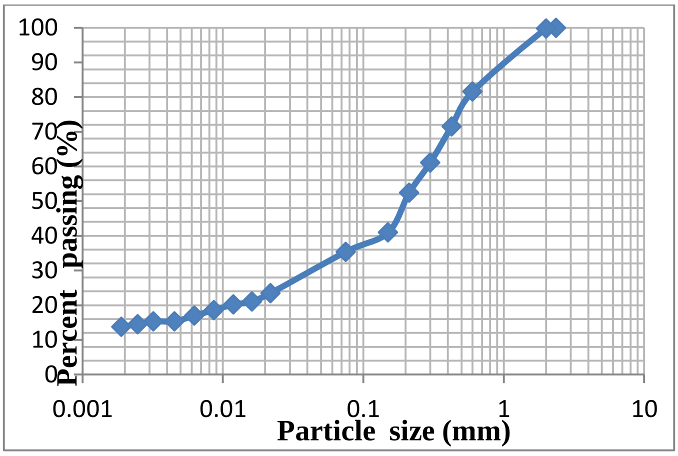

FIGURE 1: PARTICLE SIZE DISTRIBUTION CURVE FOR THE NATURAL LATERITIC SOIL

Atterberg Limits

Liquid limit: The variation of liquid limit of lateritic soil with B. *coagulans* suspension is shown in Fig. 2. It was observed that the liquid limit marginally increased with higher B. coagulans suspension from a value of 34.4 % for the natural soil (i.e., 0.0 x 10^8 /ml) to 36.4 % at 2.4 x 10^8 /ml B. *coagulans* suspension. The increase in liquid limit of the soil is consistent with the findings reported by Osinubi *et al.* (2017) for Bacillus *pumilus*-induced calcite precipitation improvement of lateritic soil. The increase recorded was suggested to be caused by the calcite precipitate formed when the soil was mixed with both the bacteria solution and the cementition reagent. The product of urea hydrolysis in the presence of urease enzyme produced by B. *coagulans* and subsequently calcite formation in the presence of calcium chloride in the mixture may be responsible for the changes in the liquid limit. As B. *coagulans* suspension density increased, more urease enzymes were added to the soil thereby resulting in the observed increase in the amount of calcite formed within the soil matrix. Analysis of variance (ANOVA) test on liquid limit test results shows that B. *coagulans* did not have a statistically significant effect (F_{CAL} = 6.15 x 10^{9} < F_{CRIT} = 9.04 x 10^{17}) on the lateritic soil . Detailed result is given in Table 2.

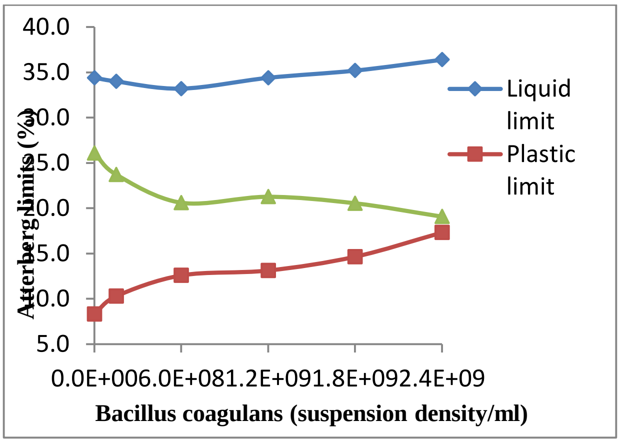

FIGURE 2: VARIATION OF ATTERBERG LIMITS OF LATERITIC SOIL WITH BACILLUS *COAGULANS* SUSPENSION DENSITY

TABLE 2: ANALYSIS OF VARIANCE FOR LATERITIC SOIL TREATED WITH B. *COAGULANS*

Property	Source of Variation	Degree of Freedom	F_{CAL}	p-value	F_{CRIT}	Remark
Liquid limit	B. *coagulans*	1	6.15E+09	1.03E+09	9.04E+17	F_{CAL}<F_{CRIT} NS
Plastic limit	B.*coagulans*	1	6.15E+09	1.03E+09	9.04E+17	F_{CAL}<F_{CRIT} NS
Plasticity index	B. *coagulans*	1	6.975103	0.02469	4.964603	F_{CAL}>F_{CRIT} SS
Linear shrinkage	B. *coagulans*	1	6.975104	0.02469	4.964603	F_{CAL}>F_{CRIT} SS
Unconfined compressive strength	B. *coagulans*	1	6.975095	0.02469	4.964603	F_{CAL}>F_{CRIT} SS

SS = Significant effect; NS= No Significant effect

Plastic limit: The variation of plastic limit of lateritic soil with B. *coagulans* suspension is shown in Fig. 2. Generally, plastic limit increased with higher B. *coagulans* suspension density from a value of 8.3 % for the natural soil (i.e., 0.0 x 10^{8} /ml) to 17.3 % at 24.0 x 10^{8} /ml. The increase could be due to microbial urease hydrolysis of urea which produced dissolved ammonium and inorganic carbon, and carbon dioxide (CO_2). The ammonia produced and released into the soil specimens increases pH, leading to accumulation of insoluble calcium trioxocarbonate IV ($CaCO_3$) in a calcium rich environment (Rong and Qian, 2013). Analysis of variance (ANOVA) test on plastic limit test results shows that B.*coagulans* suspension did not have a statistically significant effect (F_{CAL} = 6.15 x 10^{9} < F_{CRIT} = 9.04 x 10^{17}) on the lateritic soil. Detailed result is given in Table 2.

Plasticity index: The variation of plasticity index of lateritic soil with B. *coagulans* suspension is shown in Fig 2. Plasticity index decreased with increase in the Bacterial suspension density from a value of 26.1 % for the natural soil (i.e., 0.0 x 10^{8} /ml) to 19.1 % at 2.4 x 10^{8} /ml. The observed trend was probably because the insoluble calcite produced from urea hydrolysis bridged the soil particles to form soil clods by the agglomeration of the smaller soil particles into larges large clods. Similar findings were reported by Karim *et al.* (2016). Analysis of variance (ANOVA) test on plasticity index test results shows that B. *coagulans* had statistically significant effect (F_{CAL} = 6.975103 > F_{CRIT} = 4.964603) on the lateritic soil. Detailed result is given in Table 2.

Linear Shrinkage

The variation of linear shrinkage with B. *coagulans* suspension density is shown in Fig 3. Linear shrinkage generally decreased with higher bacteria density. A similar trend was reported by Osinubi *et al.* (2017) for Bacillus *pumilus*-induced calcite precipitation improvement of lateritic soil. Linear shrinkage value decreased from 8.7 % for the natural soil (i.e., 0.0 x 10^{8} /ml) to 7.2 %.at 24.0 x 10^{8} /ml suspension density. The decrease in linear shrinkage could be due to bonding between the soil particles caused by the products of calcite precipitate formed within the soil matrix. Analysis of variance (ANOVA) test on linear shrinkage test results shows that B. *coagulans* had statistically significant effect (F_{CAL} = 6.975104 > F_{CRIT} = 4.964603) on lateritic soil. Detailed result is given in Table 2.

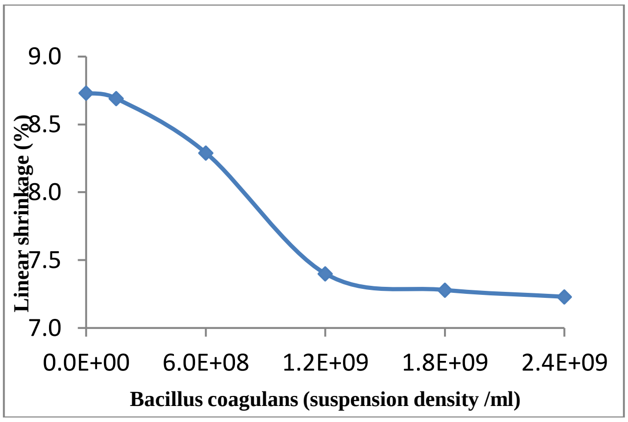

FIGURE 3: VARIATION OF LINEAR SHRINKAGE OF LATERITIC SOIL WITH B. *COAGULANS* SUSPENSION DENSITY

Unconfined Compressive Strength

The variation of unconfined compressive strength (UCS) of lateritic soil with B. *coagulans* suspension density is shown in Fig. 4. Generally, UCS values increased with higher B. *coagulans* suspension density from a value of 197.78 kN/m^2 for the natural soil (i.e., 0.0 x 10^8 /ml) to 820.69 kN/m^2 at 24.0 x 10^8 cells/ml. The increase in UCS value can be attributed to the increased bacteria cells that precipitated higher quantity of rhombohedral calcium carbonate from urea hydrolysis (Abo-El-Enein *et al.*, 2012). As the concentration of B. *coagulans* increased the larger quantity of calcite formed brings soil particles together by the clogging of pore spaces within the soil structure. Similar behaviour was reported by Abo-El-Enein *et al.* (2012) who utilized microbial-induced calcite precipitation for sand consolidation and mortar crack remediation. Analysis of variance (ANOVA) test on UCS test results shows that B. *coagulans* had statistically significant effect (F_{CAL} = 6.975095> F_{CRIT} = 4.964603) on the lateritic soil. Detailed result is given in Table 2.

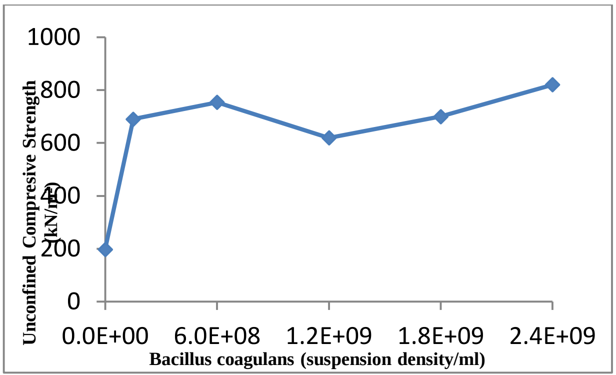

FIGURE 4: VARIATION OF UNCONFINED COMPRESSIVE STRENGTH OF LATERITIC SOIL WITH B. *COAGULANS* SUSPENSION DENSITY

4 CONCLUSION

The potential of B. *coagulans*-induced precipitate as a lateritic soil improvement technique was evaluated. Specimens of the lateritic soil classified as A – 4 (2) were prepared at OMC and compacted with RBSL energy. Results obtained showed increases in liquid limit and plastic limit values, while the plasticity index and linear shrinkage values decreased with higher B. *coagulans* suspension density. On the other hand UCS value increased with higher B. *coagulans* suspension density. Statistical analysis of variance of tests results shows that B. *coagulans* had significant effect on the treated soil. An optimal 24.0 x 10^8/ml B. *coagulans* suspension density significantly improved the engineering properties of the treated soil and met the requirements of the Nigerian General Specifications of not more than 35% passing sieve No. 200, maximum plasticity (PI) index of 30% and liquid limit (LL) of a maximum of 50% for use as a subgrade material is therefore recommended for use in the construction of low volume roads.

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FLOOD INUNDATION MAPPING AROUND LOKOJA CONFLUENCE AREA, KOGI STATE, NIGERIA

Muhammed M[1]., Shar J. T[2], *Odekunle M. O[1], Touraki A. M[3] and Pankaj R Dhote[4]

[1]Department of Geography, Federal University of Technology Minna, Nigeria
[2]National Space Research and Development Agency (NASRDA) Nigeria.
[3]Directorate of National Laboratory of public work and building, Ministry of Equipment Niger
[4]Water Resources Department, Indian Institute of Remote Sensing Dehradun, India
Corresponding email: odemary@futminna.edu.ng (+2348035957159)

ABSTRACT

Flood is characterized as a batch of water spreading over an extent that was mostly dry. Flooding results into a lot of damages and the extent of damage varies from place to place. Lokoja and environs witness flood almost every year with tremendous loss of lives, properties and degradation of the environment. The aim of this research is to evaluate flood in Lokoja and the environs using less complex geo-spatial flood inundation mapping tool. The assessment was carried out using HAND (Height Above Nearest Drainage) tool to identify the probable extent of flood and vulnerable communities at different levels of flood. Datasets used in the study were SRTM based DEM, settlement map and topographic map. Stream was digitized using topographic sheet in polyline format and later converted into raster format in GIS environment using ArcGIS toolbox. HAND is a hydrological terrain analysis approach which uses DEM and stream network as the only input for producing flood inundation maps. HAND value is defined as difference between elevation of a given cell and elevation of nearest cell along stream where it drains. Thus, all cells on the landscape that have a HAND value smaller than the specified stage (water level) were treated as inundated. Different flood inundation scenarios were generated based on varying thresholds and corresponding vulnerable communities were identified. It was concluded that the HAND has a reasonably good prediction skills, and is a low-complexity flood models that can be considered as a suitable alternative for fast predictions in large-scale hyper-resolution operational frameworks, without completely overriding hydrodynamic models' efficacy.

Keywords: DEM, Flood, HAND, Mapping.

1 INTRODUCTION

The globe has suffered disasters in many different regions depending on location and variation in the climatic zones or conditions. These include major natural disasters such as, flood, cyclone, drought, earthquake, sunstroke and minor natural disasters: cold wave, snow fall, thunder storm to mention just a few. Man-made disasters are not left out. UNDP (2004) estimated that about 190 million people in about 90 countries or more are vulnerable to flood risk and estimated 170,000 deaths from this globally from 1980 to year 2000. This statistics illustrate that flooding is a major disaster in many regions of the world. IFRC (2001) compared flood disasters with other natural disasters around the globe for a period of ten years, 1993 to 2002 and concluded that flood disasters had impacted more people than the others.

Flood is as water spreading over an extent that was mostly dry (Olajuyigbe, 2012). Flooding results into a lot of damages and the extent of damage varies from place to place. Vulnerability is the degree of fragility of a natural or socio economic, community system towards natural hazards. It is a set of conditions and processes resulting from physical, social, economic and environmental factors, which increase the susceptibility of the impact and the consequences of natural hazards. It is determined by the potential of a natural hazard, the resulting risk and the potential to react to and/or to withstand it, i.e. its adaptability, adaptive capacity and/or coping capacity. The concept of vulnerability originated in research communities examining risks and hazards, climate impacts and resilience (Donelly, 2008). Poorer communities adaptation capabilities are less in terms of disasters resilience compared with developed societies.

African continent with regions comprising mostly low in-come people is worst hit by this disaster. Nigeria as an entity also has its fair piece of natural and man-made disasters depending on the geopolitical zone one is looking at. Prominent amongst is flood. This phenomenon is a function of a set of active processes as heavy rainfall, over flow of river banks, steepness of slope or flood plains, soil types as well as anthropogenic activities. Low in-come people lived in densely populated areas with their assets, in most cases farms around them. When flood struck, many lives are lost along with their assets. Lokoja, the confluence area is one of such settlements. The vulnerability of these communities to flood disaster could be attributed to factors such as the location of large parts of the area in a lower terrain (Niger valleys and Plains) along the largest rivers in Nigeria, that is, River Niger, making the land and communities that are located in this terrain area prone to annual flooding Muhammad *et al*., (2013). This process degrades the environment by disrupting and dislocating the social and economic structure of the society. Many lives, properties, physical and biological strata of the environment are lost. Urgent need is therefore required to manage this phenomenon to reduce its impact on the society and the environment.

Advancements in remote sensing technology and Geographic Information Systems (GIS) help in real time monitoring, early warning and quick damage assessment of flood disasters. Its repetitive coverage, spectral and spatial resolution, near real time and real time data provision characteristics makes it viable for its integration with GIS capabilities of data capture, storage, manipulation and analysis for identification of the people and features that are vulnerable along this axis.

With important action oriented Zheng (2016) acknowledged flood mapping as a vital practice that generates images with many useful information which include levels of flood risk of a given area. Such inundation maps can provide communities knowledge, which serves to educate, inform and better prepare such communities for the possible risks associated with natural disasters such as flood. According to Zheng (2016) efforts to inform and educate the general communities on varying levels of flood risks using inundation maps should be made a top priority. On the other hand, the quality of such type of information used to generate such inundation map must be well improved. As a result of this the need for a model such as HAND (Height Above Nearest Drainage) becomes paramount so as to generate the inundation map.

HAND is a hydrological terrain analysis approach, which has been tested for reasonable functionality in producing flood inundation maps (Rodda, 2005; Renno et al., 2008). The HAND method as described by Nobre et al., (2015) uses the Local Drainage Directions (LDD) and its drainage network extracted from a Digital Elevation Model DEM) of the study area. The LDD with the drainage network were then used to generate the closest drainage map in a way that each drainage cell is spatially associated with all DEM cells that drains into it. The HAND tool can be used as a predictor of flood potential and extent and as an original static assessment of floodwater across the landscape. The HAND contour method could be used to map flood hazards in areas with poor information and could promote the development of new method for predicting hydrological hazards (Anthonio et al, 2015). Irrespective of the configurations, the low-complexity models like HAND can be able to produce inundation extents similar to HECRAS 2D (Shahab et al., 2017).

Accordingly Nobre et al., (2010), posited that the application of HAND model provides the possibility of capturing and examining heterogeneities in local environment in a qualitative and widely comparable manner provided that the drainage network density is accurately represented in the HAND model. Representation of local soil draining potential could be replicable for any type of terrain in which there is digital elevation data, irrespective of geology, geomorphology, or soil complexities. Shahab et al., (2017) noted that based on the reasonably good prediction skills, low-complexity flood models like HAND can be considered as a suitable alternative for fast predictions in large-scale hyper-resolution operational frameworks, without completely overriding hydrodynamic models' efficacy. But also stated that for inundation depth, the low-complexity models may show an overestimating tendency, especially in the deeper segments of the channel.

Lokoja confluence area and the environs witness flood almost every year, sometimes in between are flash floods. There is always tremendous loss of lives, properties and degradation of the environment. According to Anunobi (2013) historically, Nigerian floodplains have always attracted settlements especially in the northern part of Nigeria where the population is mostly agrarian. Those involved in the fishing sector of the agrarian industry normally settle along river banks and tributaries which are naturally flood prone. These farmers because of fishing and the rich deposit of alluvial sediment that makes the soil very fertile refused to be relocated even with the Federal Government pressure on them to do so. Government therefore keeps spending huge amounts of money in-vain. With climate change and global warming worst scenarios are being expected. Thus there is need of detailed space based research to ascertain flood parameters that addresses the vulnerability of the area.

The aim of this research is to evaluate flood inundation in Lokoja and the environs using geo-spatial technique, this was achieved using the following sets of objectives:

1. To identify the probable extent of flood in the study area using 'HAND' (Height Above Nearest Drainage)
2. To identify vulnerable communities based on varying HAND thresholding

2 METHODOLOGY

2.1 THE STUDY AREA AND DATA USED

Kogi state is situated in the North central Nigeria, located between latitude 6° 31´49.26''N and 8° 43´53.682''N and between, longitude 5°20´16.407''E and 7°53´7.2333''E. State has a total land area of 29,833 km^2 and has a population of 3,595,789 in the year 2005

(2006 census). River Niger and Benue meet at Lokoja the capital of the state (Figure 1).
The data used are Shuttle Radar Topographic Mission (SRTM) based Digital Elevation model (DEM), administrative map, topographic map sourced from NASRDA and HAND tool sourced from the Department of water resource Indian Institute of Remote Sensing.

such as boundaries from Administrative map of Nigeria to facilitate clipping out the study area.

2.4 HAND TOOL FOR WATER FLOW AND FLOOD ANALYSIS

With HAND operation, the nearest drainage of the area was generated, in this way each drainage cell is

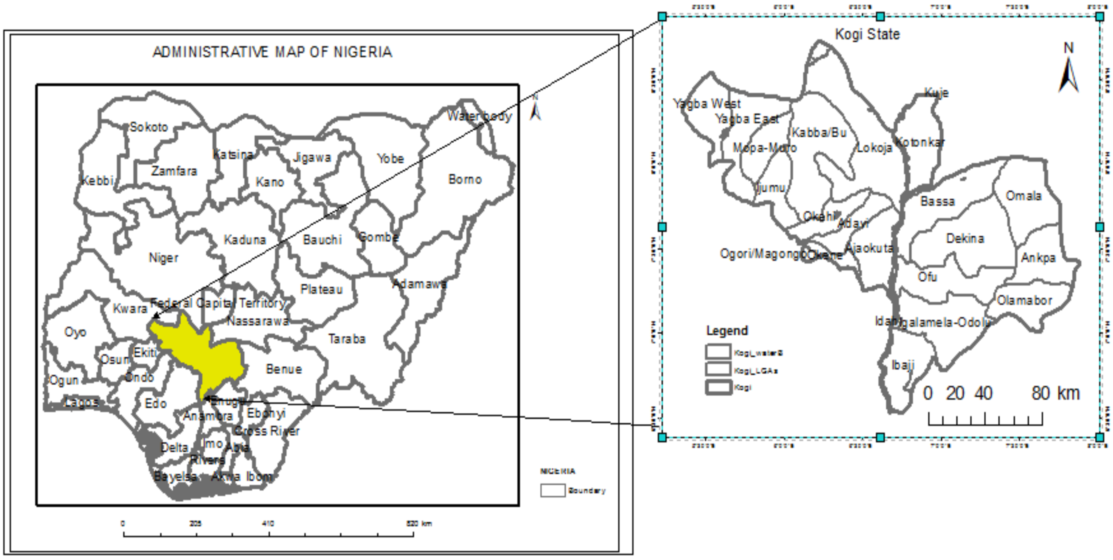


FIGURE 1 THE STUDY AREA

This research employed different hydrological approaches and procedures to come up with adopted methodology which is aimed at achieving the objectives set up for the study. Overall methodology is explained in Figure 2.

2.2 SATELLITE BASED SRTM DEM

Shuttle Radar Topographic Mission (SRTM) based Digital Elevation model (DEM) data with a resolution of 30 meters was used (Figure 3) in the study. It is one of the parameters used for topographic feature based Height Above Nearest Drainage (HAND) method for flood inundation mapping

2.3 STREAM FEATURE GENERATION

Stream generation was done by digitization using topographic map of the study area (Figure 4). The data was converted from polyline to raster format using Arc Toolbox in ArcMap interface. Ancillary data was used,

spatially associated with all ground elevation cells that drains into it (Figure 5).

The height of the corresponding drainage outlet elevation cell was subtracted from the height of each hill slope elevation cell and the nearest drainage information obtained; signifying subtraction of pairs producing the HAND model. What HAND model does is that each cell height represents the difference in level to its respective closest drainage cell.

With this, the HAND value was obtained. All cells on the landscape that have a HAND value smaller than the specified stage (water level) are treated as inundated. HAND is entirely raster-based and defines the flooded areas by a corresponding river segment. To produce different levels of flood maps; 2 meters, 5 meters, 8 meters and 10 meters thresholding values were taken. The settlement map of the study area was then overlaid to identify communities that would be submerged at various levels or height of inundated water.

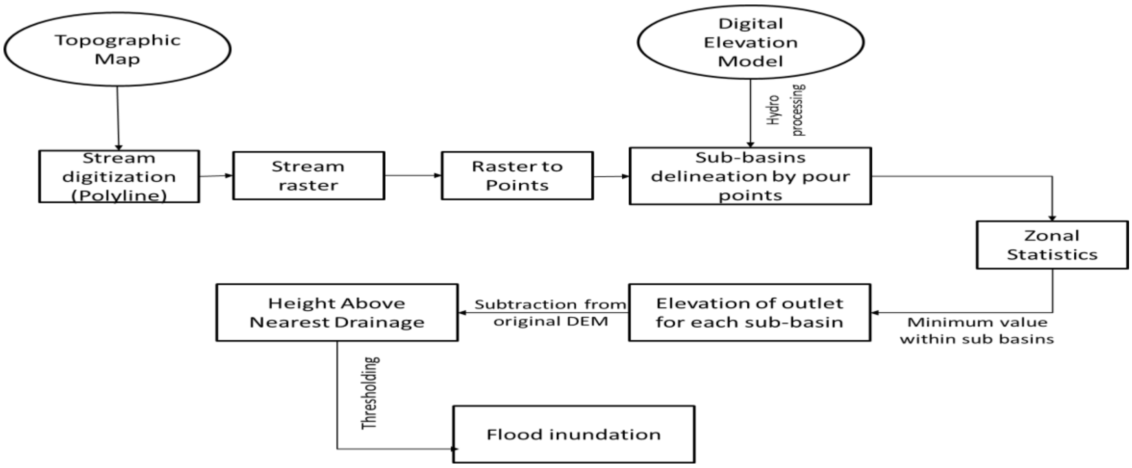


FIGURE 2 OVERALL METHODOLOGY OF THE STUDY

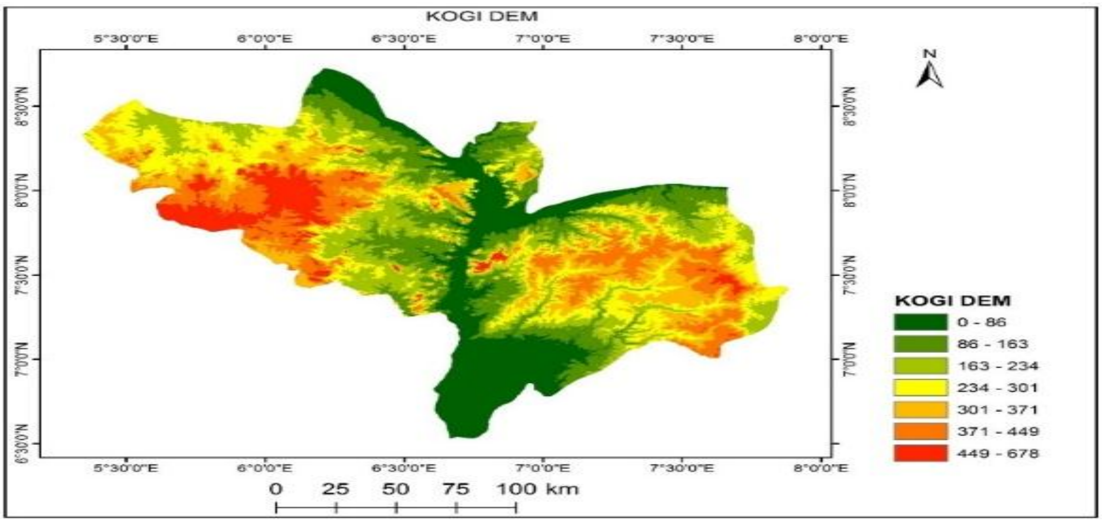


FIGURE 3 DIGITAL ELEVATION MODEL

3 RESULTS AND DISCUSSION

The results for two meters threshold is presented in Figure 6. At this level, two settlements: Lokoja the confluence town as well as the state capital with another large settlement called Idah are inundated.

At five meters (5m) threshold, in addition to lokoja and area would be inundated. The higher the level of water the more the damage and the lower the less. Shahab et al., (2017 in Comparing new generation low-complexity flood inundation mapping tools with a hydrodynamic model concluded that based on such reasonably good prediction skills, low-complexity flood models can be considered as a suitable alternative for fast predictions in large-scale hyper-resolution operational frameworks,

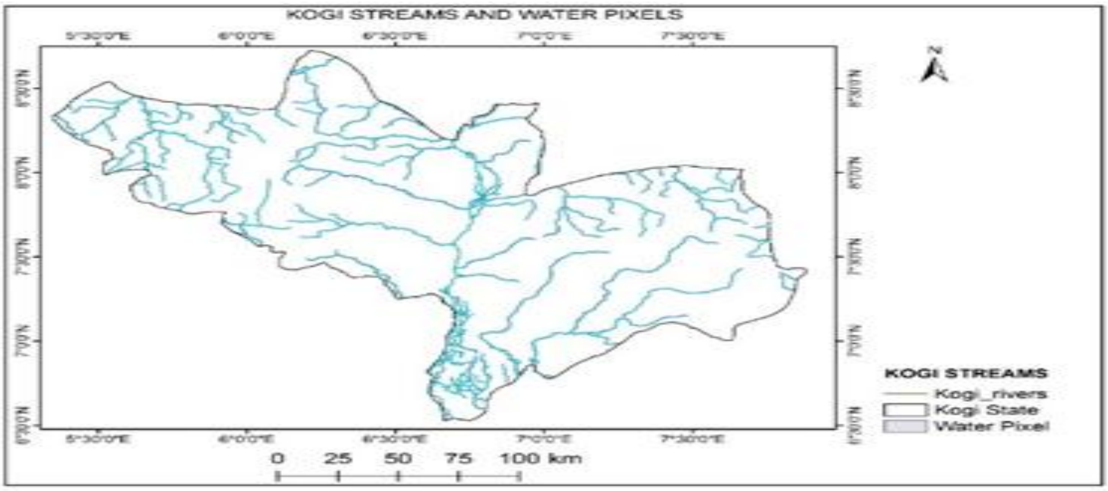


FIGURE 4 STREAM NETWORK

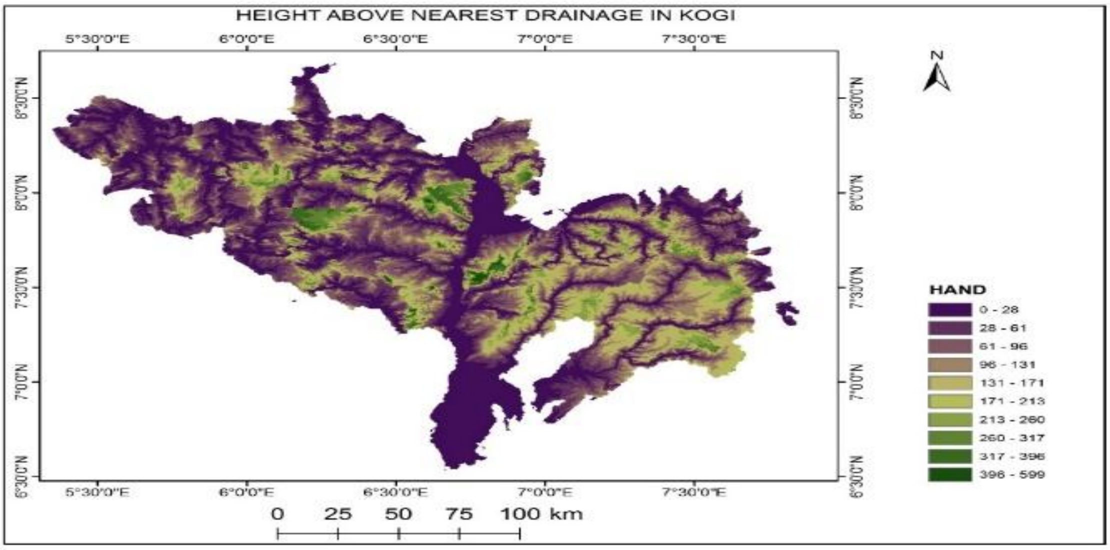
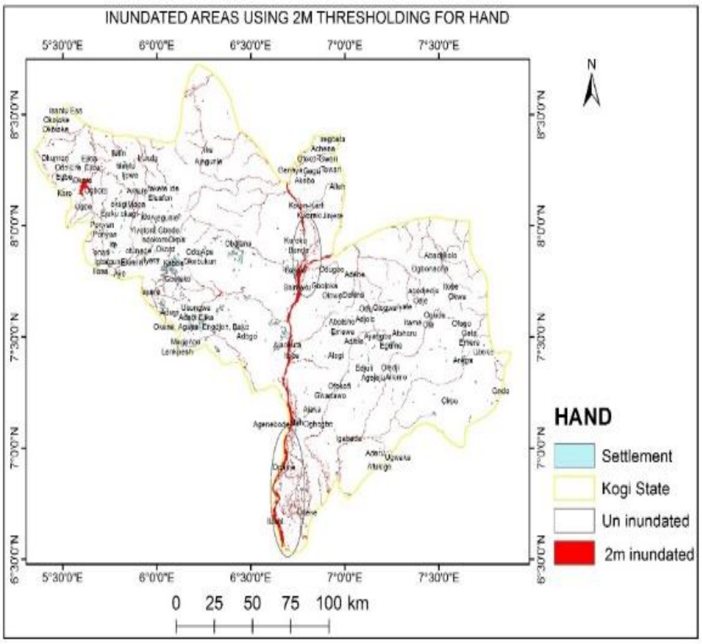


FIGURE 5 HEIGHT ABOVE NEAREST DRAINAGE (HAND)

Idah two more communities would be inundated which are Ajaokuta and Itobe (Figure 7). For thresholds of eight (8m) and ten meters (10) More communities would be inundated as presented in Figures 8 and 9. The probability of flood hazard to occur at different level of height of water would determine how much area or communities would be inundated. Areas with a two meters threshold would have low risk and fewer communities at risk while areas covered by ten meter of water would have high risk and more communities and without completely overriding hydrodynamic models' efficacy. This result unveils the high level risk typical of the community that justifies the need for immediate proactive (environment-friendly and structural) action for enhanced resilience across the confluence communities. There is a need to identify the risk in flood-prone areas to support decisions for risk management, from high-level planning proposals to detailed design (Balica et.al 2013).



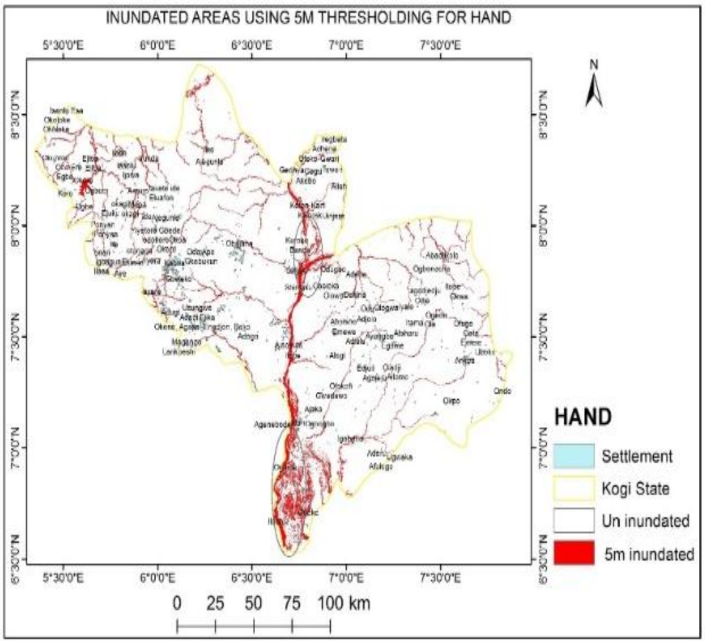
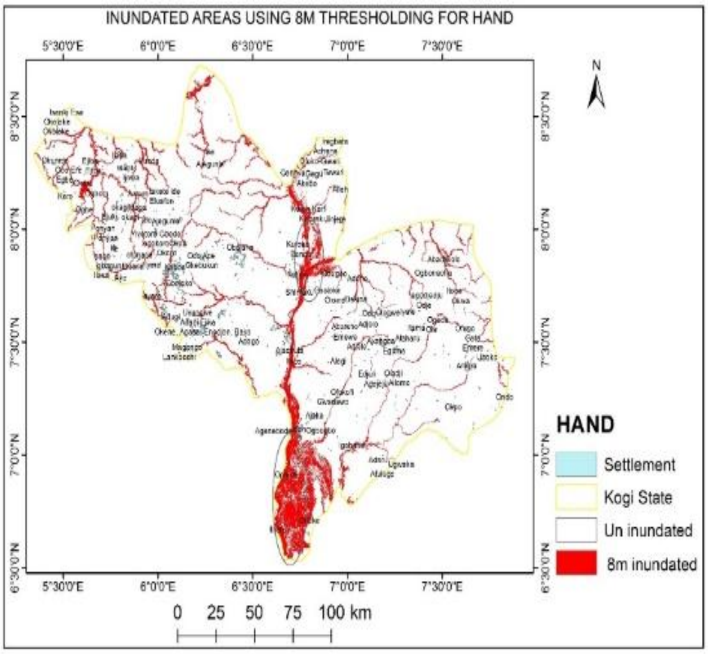


FIGURE 6 : INUNDATED AREAS AT 2 METERS

FIGURE 7 NUNDATED AREAS AT 5 METERS

4 CONCLUSION

The study was able to identify the probable extent of flood in the study area and identify vulnerable communities based on varying HAND thresholds. HAND has been used to identify flood inundated areas at different thresholds from 2 meters, 5m, 8m and 10 meters. It has a reasonably good prediction skills, and is a low-complexity flood models that can be considered as a suitable alternative for fast predictions in large-scale hyper-resolution operational frameworks, without completely overriding hydrodynamic models' efficacy. It is found that this method is less time consuming and adequate for mapping inundated areas at different levels of thresholds.



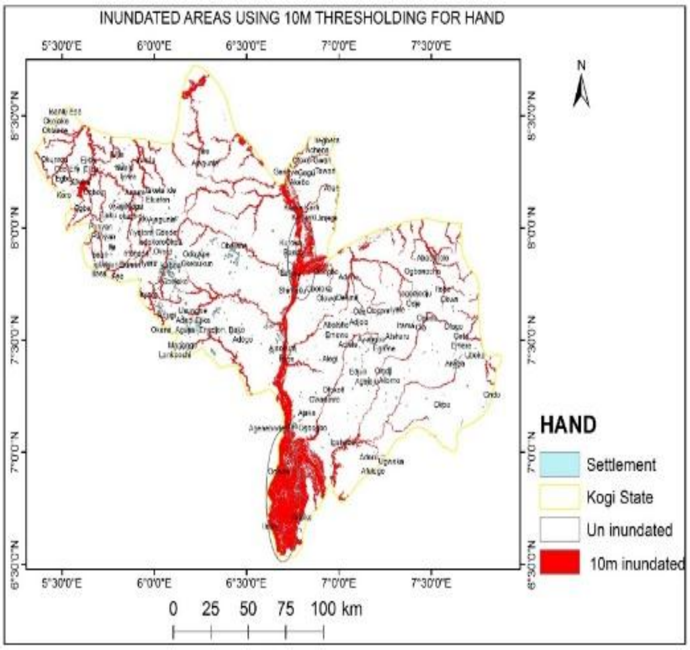


FIGURE 8 INUNDATED AREAS AT 8 METERS

FIGURE 9 INUNDATED AREAS AT 10 METERS

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BENEFITS OF IMPLEMENTING PRECISION AGRICULTURE TECHNOLOGIES IN NIGERIAN AGRICULTURAL SYSTEM: A REVIEW

***Kutigi, I. B.[1], Musa, J. J.[2], Adeoye P. A.[2], Adesiji, R.[3] and Obasa, P.[2]**

1. Niger State Fadama Development Project, Off Minna-Paiko Road, Behind Farm Centre, Minna.
2. Department of Agriculture and Bioresources Engineering, Federal University of Technology, P. M. B. 65, Minna
3. Department of Civil Engineering, Federal University of Technology, P. M. B. 65, Minna

Corresponding Author email: johnmusa@futminna.edu.ng

ABSTRACT

Precision Agriculture (PA) or Information-based Management of Agricultural Production System (IMAPS) came into existence since the mid-1980s as the process through which the right treatment is given to the agricultural process at the right time. PA was employed mainly for fertilizer application to various soil conditions across agricultural fields at the onset. Since then, different use of PA has been engaged in other areas of agriculture such as farming vehicles and implements, autonomous machinery and processes, product traceability, on-farm research, and software for the overall management of agricultural production systems. To describe precision agriculture, most simplified way is the examination of the five "R"s which are the right time, right amount, right place, right source and proper manner of agriculture inputs like – water, fertilizer, pesticide, etc. The primary constraint of agricultural development in Nigeria is the use of inadequate methods of data and information acquisition on agrarian land potential, crops condition, and farming activities. It can, therefore, be concluded that the lack of decision support systems to be a significant barrier to the adoption of PA. Farmers need decision support systems that enable effective decision making based on accurate and timely data.

Keywords: *Agriculture, Precision, GPS, GIS, Tools.*

1 INTRODUCTION

Availability of food and other agricultural products in adequate supply and quality under environmentally safe conditions and the sustainability of the various resources involved is of paramount importance to researchers (Gebbers and Adamchuk, 2010). Precision Agriculture (PA) or Information-based Management of Agricultural Production System (IMAPS) came into existence since the mid-1980s as the process through which the right treatment is given to the agricultural process at the right time. Sensitization on the variation in soil and crop conditions combined with the various forms of technology such as global navigation satellite systems (GNSSs), geographic information systems (GISs), and microcomputers, serve as the primary drivers. PA was employed mainly for fertilizer application to various soil conditions across agricultural fields at the onset. Since then, different use of PA has been engaged in other areas of agriculture such as farming vehicles and implements, autonomous machinery and processes, product traceability, on-farm research, and software for the overall management of agricultural production systems.

Aside from crop production, PA technology is employed successfully in viticulture and horticulture, including orchards, and in livestock production, as well as pasture and turf management (Gebbers and Adamchuk, 2010). They further stated that application of PA ranges from the tea industry in Tanzania and Sri Lanka to the production of sugar cane in Brazil; rice in China, India, and Japan; and cereals and sugar beets in Argentina, Australia, Europe, and the United States. PA is in three folds that is to optimize the use of available resources to increase the profitability and sustainability of agricultural operations. Second, to reduce negative environmental impact. Third, to improve the quality of the work environment and the social aspects of farming, ranching, and relevant professions.

The idea of precision agricultural practice started several decades ago most especially in developed countries like the State of Israel, the United States of America, Canada, and the Western Europe countries (Junk *et al*. 2013). Precision agriculture is the application of established technologies and principles to manage all areas of an agricultural system for improved quality production of farm products and the environment at large (Pretty, 2008). In the last three decades, this has not only matured but has moved to all other developing countries of which Nigeria is not left out. The connection of PA to the improvement and increment in the level of civilization of humanity and the quest for improved gross domestic product and national cannot be overemphasised. To describe precision agriculture, most simplified way is the examination of the five "R"s which are the right time, right amount, right place, right source and proper manner of agriculture inputs like – water, fertilizer, pesticide, etc. The success of PA is seen through the productivity and profitability of the process. Precision agriculture is the ability of the farmer to manage the differences that may occur during the agricultural process for profitability and sustainability of the farming activities. PA further manages the low input, high yield, and environmentally sustainable conditions. The effect of these variabilities according to Bongiovanni *et al*., (2004) is grouped into yield variability, field variability, soil variability, crop variability, variability in anomalous factors, and management variability.

Every section of the farmland or zones of the country has different production capability, which is connected to the soil type's environmental conditions and other geographic conditions like- such as the slop of the farmland and topography (van Ittersum *et al*. 2013). This makes the lands to respond differently to the crops grown. Yield capacity difference can also mean yield variability. The difference in the field topography (elevation, slope, closeness to a water source) can be termed to be the field variability. The variation for nutrients in the field is termed to be soil variability (Araus and Cairns, 2014). Soil texture to an extent could be called as a variable factor. The plant growth rate and duration are termed crop variability. Other factors affecting the life cycle of the crops include weed infestation, insect infestation, disease condition, and wind damage. These factors are known to affect the performance of the agricultural process. It is therefore essential to understand the variability of these factors and their effect on agriculture. The presence of these factors and the corrective measures are taken to reduce their impact on the farm's aids in achieving the objective of PA. Thus, the introduction of PA helps to make maximum yield and profit from the sales of farm products in spite of yield, field, soil and all another variabilities. The practice of PA is accomplished through the introduction of mitigative measures that control the variabilities.

2. PRECISION AGRICULTURE IN NIGERIA

Agriculture in Nigeria since independence has played a significant role in the country's development at both the food security level and income generation. This sector is an essential occupation of the average Nigerian as about 70% of the population depends on it. The agricultural sector provides bulk employment, income, and food for the rapidly growing population as well as agro-based industries. As a nation, Nigeria is still not self-sufficient in agrarian products as most food crops are imported from other countries. Several governments of Nigeria have introduced different farming programmes and policies targeted at improving agricultural productivity to meet the growing demand by the population and agro-industries (Olaoye, 2014). However, the hydra problems of agriculture in Nigeria and food insecurity have taken great contrary positions because the rate of growth in the area of agricultural production has not in any way met the growing demands of the population. Two significant areas have been identified from the agrarian policies of 2016 which are the inability of products to meet the domestic demand and the failure to achieve the quality required for the foreign market (Lee *et al*. 2012). These policies and programmes are connected to the poor farming methods employed and the lack of essential farming inputs such as fertilizer, herbicides, improved seeds, irrigation crop protection and necessary support from the various agricultural schemes (Toenniessen *et al*. 2008). The advent of industries and urbanization has drastically reduced the farmlands and farming resources largely. Population increase and limited agrarian support resources have in recent times opened up issues like productivity, sustainability, and profitability of agriculture system. Currently, Nigeria is rated as one of the leading producers of grain crops such as rice, maize, sorghum, etc. in Africa. Thus, there is the need for improvement using the conventional process of agriculture and the adoption of technology.

Many challenges oppose the implementation of PA in developing and underdeveloped countries (Scott, 2008). These challenges do not only include the non-availability of the much need technology but many other factors which consist of the lack of electricity supply, insufficient water supply, land allocation methods to farmers, knowledge of PA among the farmers and the government policies (Mustapha *et al*. 2012). With all these challenges it becomes almost impossible for modern agricultural technologies to strive. As one of the developing countries, Nigeria has not started adopting the practice of PA.

Recent developments have shown that PA system when introduced to farmers said that they are farming processes for the future while some others have keyed into it and are making ways regarding the quality and quantity of agricultural products. Precision farming involves the use of specific soil and crop data for improved yield. This process is targeted at optimizing returns on investment by matching farming practice more closely to the crop needs (Okorie, 2018). Nigeria is the most populous black nation and is faced with the adaptation of PA in practice due to cultural and financial reasons. Some of these challenges are due to the types of instruments that are involved which includes Global Positioning System (GPS), Remote Sensing (RS), on the go sensors, etc. Most of the agricultural systems in Nigeria are made up of the so-called hard PA.

3. CHALLENGES OF PRECISION AGRICULTURE

The challenges facing food insecurity, poverty, disease and hunger in Nigeria and many other nations have called for research in this area to forestall these menaces. The primary constraint of agricultural development in Nigeria is the use of inadequate methods of data and information acquisition of agrarian land potential, crops condition and farming activities (Harris and Orr, 2014). The effect of this is the imperfect knowledge and unreliable data acquisition for farm planning and policy formulation. For instance, unguided use of land whose consequence is often the misuse of prime farmland. This has a significant set back on agrarian development. It is worthy of note that substantial agricultural production takes place under traditional systems. This is highly dependent upon natural forces and processes for the maintenance of yield and the quality of produce. Detection, identification, measurement, and monitoring of agricultural phenomena predicated the assumption that agrarian landscape features (such as crops, livestock, crop infestation, and soil anomalies) have consistently identifiable signatures on the type of remote sensing data. These identifiable signatures are a reflection of crop type, state of maturity, crop density, crop geometry, crop vigor, crop moisture, crop temperature, and soil moisture as well as soil temperature (Chong *et al*. 2017). PA-based on the incorporation of information and communication technologies into machinery, equipment, and sensors in agricultural production systems, allows a large volume of data and information generated are inputted into the automation system for processing (Rodrigues, 2013).

Demographic trends, including aging populations and continued migration of people from rural to urban areas, have attracted the attention of researchers, because labor issues may become a scarcity factor in agriculture. According to the UN data, the world's urban population is poised to surpass the rural total for the first time in history. By the time this happens, more than half of the rural population in Nigeria will be living in cities. This kind of population growth is observed mainly in low and middle-income nations like India, China, Nigeria and Brazil (Prince *et al*. 2013). China and India have occupied the first and second positions in the list of countries with the fastest growing 100 cities while Nigeria is not left out in Africa (Seto, 2011). The implications of such dramatic shifts for economic development, urbanization, and energy consumption are immense. In addition to these trends, the intensification of climate change will continue to alter growing conditions, such as temperature, precipitation, and soil moisture, in less predictable ways (Erwin, 2009). PA tools can help reduce these impacts, keep them constant or reduce production costs in agricultural activities, and they can assist in minimizing environmental constraints (Chen, and Yada, 2011).

To meet the growing food grain demand in Nigeria and with the increasing challenge of biotic and abiotic stresses experienced by crops, the introduction, and adoption of modern technology in Nigerian agriculture is inevitable. Agriculture, like other industries, has made entry into the knowledge-based era, leaving its previous resource-based nature in recent times through the various policies governing the importation of agricultural products (Andersen, 2012). Future agriculture will be severely competitive, knowledge-intensive and market driven (Holt-Giménez and Altieri, 2013). Identifying how science frames PA over time, countries and targeted research can help drive new study with the objective of covering areas that have received less attention; this will develop new approaches to understand PA better and illuminate new applications. Some of the challenges in PA includes low technological development; inconsistency and inept implementation of government policies; the level of investment; crop Inputs; farm size management practices; Optimal size zone for soil sampling. Other technical problems include farmland holdings, monocropping system and market imperfection which is regarded as the most important to the farmers as they do not have control over the market forces.

These challenges are real, and they constitute a significant roadblock to the implementation of PA in Nigeria agricultural development. Concerted efforts and careful planning are required to cover these problems. The most significant challenge is perhaps the acquisition of relevant space technology. Remote sensing, Geographic Information Systems, and Global Positioning Systems are expensive tools and are currently very scarce in Nigeria. However, with the successful launch of an earth observation satellite, NigeriaSat-1 in March 2003, by the Nigerian government brought this a step towards the application of space technology to solving some of the socio-economic problems in the country, including the agricultural sector. The satellite will improve the efficiency and reliability of agrarian data collection. Several researchers have expressed in various ways the capabilities and relevance of NigeriaSat 1 in Nigerian agricultural development. Rilwani and Gbakeji (2009) have demonstrated the ability of NigeriaSat 1 in farm planning and management. He integrated data from NigeriaSat 1 with existing soil and topographical map in a Geographic Information System environment to assess the current and potential agricultural land use in the Kadawa sub-sector of the Kano River Irrigation Project.
An alternative to satellite remote sensing that could be adopted in Nigeria is Airborne Videography. This technology provides higher levels of spatial details (between 0.25m and 4m pixel size) than current satellite technology (Woodget *et al*., 2015; Matese *et al*. 2015). This advantage in addition to the flexibility in the frequency and time of coverage make it ideal for the site-specific management of soil and crop conditions.

Some of the tools used in PA includes the yield monitors which have the capability of indicating yield (kg/ha), total kg, ha/hour; hectare worked, and grain moisture content. They are attached to crop harvesting equipment providing information on crop yield; global positioning system (GPS) is a network of 24 satellites orbiting the earth which gives exact satellite time and location to ground receivers. Differential Global Positioning System (DGPS) which is a way of improving the GPS accuracy. This uses the pseudo-range errors measured at a known location to adjust the measurements made by the other GPS receivers within the same general geographic area. Geographical Information System (GIS) are computer soft and hardware's which use feature attributes and location data to produce maps. Remote sensing is a useful tool for collecting lots of information simultaneously from a distance (Zook *et al*. 2010). Remotely-sensed data provide a mechanism for evaluating crop health (Mandal and Maity, 2013). Variable Rate Applicator which has variable rate applicators for the three components which include a control computer, locator and actuator (Chopra *et al*., 2008; Yuan *et al*., 2010; Schumann, 2010).

4. PROCESSES OF PRECISION AGRICULTURE

This is divided into two parts which include:

i. ***Identification and Assessment of Variable components***: This is further broken down into the following segments:

a. Grid soil sampling: Grid soil sampling uses the same principles of soil sampling but increases the intensity of sampling compared to the traditional sampling. Soil samples collected in a systematic grid also have location information that allows the data to be mapped (Morvan *et al*., 2008). The goal of grid soil sampling is to generate a map of nutrient/water requirement, called an application map.

b. Yield map: Yield mapping is the first step to determine the precise locations of the highest and lowest yield areas of the field, and to analyze the factors causing yield variation (van Ittersum *et al*., 2013). One way to determine yields map is to take samples from the land in a 100m x 100m grid pattern to test for nutrient levels, acidity and other factors (Mandal and Maity, 2013). The results can then be combined with the yield map more effective yet more economical placement that produces higher crop yields (Lobell *et al*., 2009). Researchers at Kyoto University recently developed a two-row rice harvester for determining yields on a micro-plot basis (Dixit *et al*. 2014).

c. Crop scouting: In-season observations of crop conditions like weed patches (weed type and intensity); insect or fungal infestation (species and concentrations) crop tissue nutrient status; also can be helpful later when explaining variations in yield maps (Huseth *et al.* 2018).

d. Use of precision technologies for assessing variability: Faster and in real time assessment of variability is possible only through advanced tools of precision agriculture (Zhang and Kovacs, 2012).

ii. ***Variability Management***: The management processes of precision agriculture include:

a. Application rate: Grid soil samples are analysed in the laboratory, and an interpretation of crop input (nutrient/water) needs is made for each soil sample. Then the input application map is plotted using the entire set of soil samples. The input application map is loaded into a computer mounted on a variable-rate input applicator. The machine uses the input application map and a GPS receiver to direct a product-delivery controller that changes the amount and kind of input (fertiliser/water) according to the application map.

b. Yield monitoring and mapping: Yield measurements are essential for making sound management decisions. However, soil, landscape and other environmental factors should also be weighed when interpreting a yield map. Appropriately used, yield information provides essential feedback in determining the effects of managed inputs such as fertiliser amendments, seed, pesticides and cultural practices including tillage and irrigation. Since yield measurements from a single year may be heavily influenced by weather, it is always advisable to examine yield data of several years including data from extreme weather years that helps in pinpointing whether the observed yields are due to management or climate-induced.

c. Quantifying on-farm variability: Every farm presents a unique management scheme. Not all the tools described above will help determine the causes of variability in a field, and it would be cost-prohibitive to implement all of them immediately. An incremental approach is a wiser strategy, using one or two of the tools at a time and carefully evaluating the results and then proceeding further.

d. Flexibility: Small-scale farmers often have highly detailed knowledge of their lands based on personal observations and could already be modifying their management accordingly. Appropriate technologies here might make this task more accessible or more efficient. Larger farmers may find the more advanced technologies necessary to collect and properly analyse data for better management decisions (Baumgart-Getz *et al*. 2012).

5. Benefits of Precision Agriculture (PA)

Precision agriculture within farmland can be managed using different levels of inputs depending on the yield potential of the crop in that particular area of land. Wang *et al.,* (2006) stated that the benefits of precision farming are in two folds:

i. Reduction in the cost of producing a given crop in an area of land and

ii. the risk of environmental pollution from agrochemicals applied at levels higher than those required by the plant can be reduced.

The following are some of the benefits of PA to the farmers:

i. Efficient use of equipment: Information on soil characteristics and weather can be used to plan and improve scheduling of operations, which can increase machinery utilisation rates and lower per- acre costs. Also, GPS based guidance systems can allow farm machinery operators to achieve greater field efficiency under challenging conditions. They can reduce overlap and missed applications of inputs (e.g. spraying), helping fatigued operators maintain higher field efficiency.

ii. Risk reduction: At the field level, PA provides site-specific management that can point out problems with growing conditions, thereby reducing variability in net returns. At the farm level, PA information can be used to improve variety choice, crop rotation, and other agronomic practices that minimise risk. As well, information on crop growth during the season can help you make more informed market decisions.

iii. Management of different products: In the future, precision technology may help farmers differentiate their production within a particular field. For example, you might segregate higher protein wheat for marketing in more rewarding channels. Also, PA technology will allow the additional control that is required when you are managing the production of differentiated products as opposed to the output of regular bulk crops. It will allow documentation of crops conditions and control of inputs to meet the particular requirements of these crops.

Farmers are constantly making important decisions that impact their business success. PA uses a continuous cycle of data collection, data analysis and application to maximise farm profits and protect the environment by managing land and livestock changes over time. The data collection process identifies areas of interest and records the required data. The data analysis process organises, queries and reports on the collected data. The farmer can make effective decisions based on the reports generated (Hochman *et al*. 2009).

iv. Increased farm profitability is an essential benefit of PA for farmers. Some researchers Reviewed some studies of Precision Agriculture published between 1988 and 2005 and found PA to be profitable for 68% of the cases studied (Stoate *et al*., 2009; Ahumada and Villalobos, 2009; Crosson *et al*., 2011; Zhang and Kovacs, 2012). Arable farmers have traditionally used a whole field approach when planting seeds, applying fertiliser and spraying pesticides. PA allows the arable farmer to break a field down into smaller management zones based on crop yield rates and crop production factors such as pest presence, soil types and soil acidity levels. Farmers can use the knowledge gained from management zones to develop management plans and implement processes that ensure the best use of resources to maximise output and profits (Shiferaw *et al*., 2009).

v. Dairy farmers can use PA applications to enhance profitability by monitoring only livestock and making interventions at the right time to optimise outcomes. Sensors can record critical aspects of livestock fertility and alert farmers when an animal is ready to reproduce. Dairy farmers can maximise the number of calves, produce more milk, save time and reduce artificial insemination costs by monitoring their livestock (Hamadani and Khan, 2015).

vi. Time and labour savings are achieved through the automation of repetitive farming tasks. In the dairy sector, robotic milking can record valuable data on milking performance, save time for farmers, reduce the need for external labour, encourage greater production, better animal health and higher quality milk. Auto-steer systems on tractors and harvesters can reduce driver fatigue by automating the navigation of fields with satellite positioning. Automated feeding systems can provide livestock with feed at regular intervals and reduce the workload for farmers. Animals are less stressed with automatic feeding, and lower ranking animals have more access to feed (Grothmann *et al*. 2010).

vii. PA applications can continuously monitor animal health in real-time and alert farmers when intervention is required. Sensors can monitor livestock and their environment to detect changes in livestock positioning, feeding patterns, temperature, humidity and sounds. Pig farmers can monitor the health of their herds by reviewing the sounds produced by the crowd. Early detection of coughing sounds can reduce disease transmission in the group and save money on antibiotic purchases and veterinary fees. Feeding patterns can be monitored for individual animals, and farmers can be alerted when particular animals are eating or drinking less (Banhazi and Black, 2009).

6. APPLICATIONS OF PRECISION AGRICULTURE

Agricultural farmlands are of diverse environments with variable topology and microclimates. Crops grow and produce at different rates depending on factors such as soil quality, access to water and nutrients, altitude and temperature. PA applications are used to increase quality, record production levels, generate yield maps and identify zones requiring additional irrigation or fertiliser (Hochman *et al.*, 2013). Drones and satellite imagery are used to analyse the health of the crops using Normalised Difference Vegetation Index (NDVI) to identify areas that require attention. The NDVI uses the visible and near-infrared bands of multispectral imagery to display plant health information. Nutrients are applied to specific areas using the analysed data to reduce water usage and the cost of fertilisers.

Arable farmers in developed countries use high precision positioning systems such as the Variable-Rate Technologies (VRT) and Controlled Traffic Farming (CTF) to drive efficiencies for crop production and protect the environment (Pedersen and Lind, 2017). High precision positioning systems enable the accurate positioning of a farmer’s tractor in a field and facilitate the precise seeding of crops, higher planting density and the efficient application of pesticides, nutrients and herbicides. VRT allows farmers to vary the use of fertiliser on specific areas of the field according to the needs of the crop. CTF enables farm vehicles to accurately navigate fields which result in reduced operator fatigue and minimised crop damage (Kroulík *et al*., 2011). Tractors and combine harvesters are large vehicles with the capacity to damage crops with poor operator direction (Shearer *et al*., 2010).

Livestock farmers are using Precision Livestock Farming (PLF) to monitor their herds and environment, detect diseases at an early stage, record growth, food intake and milk production (Meen *et al*. 2015). Farmers can review the variation in performance within their herd and make the necessary input changes to achieve optimal results. Alerts can be set up to notify a farmer when a cow is going to calve. Time savings and better outcomes are obtained by applying technology to herd management (Sarac *et al*. 2010). Horticulture farmers are using machine vision methods to record the size, shape, colour, visible defects, sugar content and acidity of their products (Kondo, 2010).

PA is used in forestry to monitor growth, produce biomass estimates, identify diseased or infested trees, classify different species of trees and determine areas ready for harvesting. Remote sensing imagery captured by satellites and drones are analysed in geospatial systems at regular intervals to produce data that drives planning and decision making (Matase *et al*. 2015). NDVI maps can be used to identify tree health in specific areas. Harvesting machines fitted with high precision positioning systems can record their location and harvesting yields to ensure that a forest is managed appropriately (Suprem *et al*., 2013).

Real-time information from PA applications will lead to changes in the monitoring and trading of crops. Government agencies and the financial markets will be aware of crop yields during the growing season rather than at the end of the season. The pricing for crop markets will become more dynamic with fluctuations occurring as data is received during the growing season (Verchot *et al*., 2007; Peltonen-Sainio *et al*., 2010). Government agencies will be able to forecast crop yields more accurately with the increased volumes of crop performance data (Challinor, 2009; Lin, 2011).

7. REQUIREMENTS FOR IMPLEMENTING PRECISION AGRICULTURE

Key components that can improve the implementation of PA amongst most farmers includes scalability, low cost, support, integration and interoperability with the utilisation of open data standards, rule-based workflows, automated and intuitive data processing methods, user control over analysis and processing functions, systems customised to meet farmer needs and an easy to user interface (Janssen *et al*. 2015). Farmers need systems that can grow over time as more PA applications come to bear. Low-cost systems are required as farmers are not willing to take a risk on expensive applications that may not deliver the expected benefits. Farmers need systems and applications with interfaces that integrate with legacy, current and future operations.

Farming is a diverse industry, and PA applications must be customised to suit the particular needs of the farmer. Specific modules of PA applications can be supplied to the farmers based on their requirements. Rule-based workflows allow farmers to deploy their business knowledge into a PA application. Effective communication is ensured as regards standards and operability of the various forms of technologies. Usability and automated data processing methods help the farmer manage the large volume of data generated by PA applications (Lee *et al.*, 2014; Fountas *et al*., 2015).

The educational status of the farmers as regards PA has in the recent times been emphasised to enable them to understand the potential benefits of the PA technologies and practices (Fountas *et al.*, 2015). There are six learning processes which has been identified for stakeholders to improve their agronomic knowledge, information management skills and understanding of PA. These processes include the experience of the idea of spatial data management, spatial variability and maps; the second is that the stakeholders gain an understanding of sensors and how sensors can be used for benefit in farming. Such systems that use sensors were described as GPS, Yield Monitoring Systems, Remote Sensing and VRT systems. The relevant stakeholders are thought IT skills at appropriate levels and become familiar with GIS technology. The fourth step for the stakeholders is the creation of awareness as regards the factors that enable the identification of flexible yield influence elements (Akhtar- Schuster *et al*., 2011). Here, they learn how to analyse yield maps, yield variation patterns and understand the difference between natural and management-induced variation. The final step shows stakeholders how to carry out strategic sampling and on-farm trials to test PA technologies and practices on their farms (Kutter *et al*., 2011; Mariano *et al*., 2012; Eastwood *et al*. 2012).

8. THE ADOPTION OF PRECISION AGRICULTURE

With the adoption of new technologies and its practice, agriculture develops rapidly to meet the competitive demand for its products (Hatanaka *et al*. 2005). The rate and diffusion of PA technology adoption determine the impact on farm production levels. Factors such as the farmer profile, farm type, economic conditions, complexity and cost of the technology influence the diffusion and speed of PA adoption (Aubert *et al*. 2012). Farmers go through a five-stage decision-making process when adopting PA technologies. In the Knowledge stage, the farmer learns about the new technology and its applications. At the Persuasion stage, the farmer develops an opinion on the latest technology. The farmer chooses to adopt the innovation at the Decision stage. The Implementation stage is where the farmer puts the technology into use on their farm. The Confirmation stage is the final stage where the farmer seeks to validate the decision to adopt the technology (Mackrell *et al*. 2009).

Five significant stages of adoption of agricultural technology were identified as the innovators, the early adopters, the old majority, the late majority and the laggards (Läpple and Van Rensburg, 2011). The innovators are adventurous farmers who discover new techniques and pay a premium to evaluate the technologies. Innovators are a small but essential part of a market. Early adopters are influential leaders who observe the innovators' findings and find practical usages for the new technology. They communicate the benefits of the technology to a broader audience. The early majority adopt technologies when they are confident that the product will be useful on their farm and there will be a good return on their investment. The late majority are doubtful of new technology and wait until the technology has achieved widespread adoption before deciding to invest. The laggards are happy to continue farming in the old way and adopt new technologies reluctantly.

PA has not achieved widespread adoption in Nigeria due to high start-up costs, complexity, stakeholder awareness and training, data management issues and the size and diversity of farm structures. The average Nigerian farm is less than 4 hectares, and many farmers cannot afford large investments in technology products. Nigeria with its large arable regions and intensive farming have higher prospects of PA usage (Seck *et al*. 2012). Limited research and investment are on-going to develop PA in Nigeria to ensure higher adoption rates going forward.

9. FACTORS INFLUENCING THE ADOPTION OF PRECISION AGRICULTURE

Research shows the primary driver of PA adoption to be increased profitability and cost to be the primary barrier to PA adoption (Aubert *et al*. 2012). Secondary adoption drivers were environmental compliance, availability of improved information for better decision making and risk reduction. Nigerian farmers have expressed frustration that PA was not a "turn-key" technology as there are many complex interactions to be interpreted to derive the benefits from PA. Current research works should focus on low cost, robust and easy to use PA technology to drive increased adoption (Tey and Brindal, 2012). In their study, they studied the adoption factors for PA and classified the elements found into seven categories; socioeconomic factors, agro-ecological factors, institutional factors, information factors, perception factors, behavioural factors and technological factors. Socioeconomic factors that influence the adoption of PA were found to be the farmer's age, education, farming experience, attitude to risk, market conditions and access to information. Older farmers are less likely to adopt new technologies that require training and investment. Farmers with higher levels of education are more likely to take PA technologies as they often have a more excellent knowledge of best practice farming practices. The risk associated with every investment and the risk-averse farmer is more likely to continue farming traditionally. Market conditions influence the adoption of PA and farmers are more likely to invest in new PA technologies and equipment when market conditions are stable, and the return on investment is high (Tey and Brindal, 2012).

Agro-ecological factors that influence adoption decisions include farm size, income, land tenure, environmental compliance and crop type. Larger farms with steady incomes are more likely to invest in PA. Farmers who are renting land are unlikely to significantly invest in PA technology due to uncertainty regarding future control of the area. Farmers growing crops planted in rows such as corn, cotton and soybeans were more likely to adopt PA than farmers growing vegetables, fruits and minor crops. Environmental compliance is becoming an increasingly important adoption factor as farmers need to meet strict environmental protection measures.

Institutional factors were found to be government organisations and policies, distance from fertiliser and equipment suppliers and the farm's location. Government organisations have a significant role to play in training and educating farmers on the technologies driving PA and the possible PA applications for their farms. Well informed farmers who understand the benefits of PA are more likely to adopt the technologies. Distance from fertiliser and equipment suppliers is another adoption factor as farmers located far from suppliers will be in less contact with sales personnel that can inform farmers of the availability of new PA equipment and possibly convince the farmer to invest in the latest technologies (Tey and Brindal, 2012).

Information factors included the use of consultants and access to information sources. Farmers who work with consultants receive information on the best practices for their farm and are more likely to adopt PA. Access to information sources such as industry and government publications allows a farmer to keep informed of the latest developments with farming. Perception factors were the farmer's view on the importance of PA and the profitability of PA. The farmer's attitude toward PA is crucial as ultimately the farmer is the decision maker who adopts the appropriate technologies for their farm. A farmer who had a bad experience with early PA technologies may be reluctant to invest in new technologies. Behavioural factors included the farmer's behavioural profile and intentions (Tey and Brindal, 2012).

Technological factors found to be essential adoption influences were the complexity of the PA technology, the type of technology to be adopted, farm irrigation structure and the usage of computers on the farm (Tey and Brindal, 2012). Technologies need to be understandable and usable to achieve widespread adoption by farmers. Many farmers are reluctant to adopt complex technologies due to the time and training required for usage. Farmers with previous experience of working with information technology are more likely to take PA technologies as they are familiar with computers. The type of technology influences adoption decisions as there are varying costs associated with different techniques and some technologies may be more familiar to farmers.

Ex-post adoption factors include the farm size, quality of the farm's soils, farmer income, farmer education, access to information, costs savings, desire for higher profitability, land tenure and IT experience. The typical PA adopter was found to be an educated farmer seeking a competitive advantage through better agricultural practices on their large fertile farm. The primary ex-post driver for PA adoption was found to be farm size. Large farms with over 500 hectares can benefit from economy of scale when adopting PA. A secondary driver was the farmer's confidence with technology. Farmers with good technological skills were found to be more likely to take PA. Other ex-post drivers for PA adoption were a high income, the farm's location and the farmer's education (Kassie *et al*. 2011; Paustian and Theuvsen, 2017).

10. CONCLUSION

The adoption of PA has been constrained by some barriers such as cost, complexity and weak or non-availability of rural broadband infrastructure. The accessibility and speed of rural broadband will need to be improved to enhance internet connectivity between farm systems and external providers. PA applications use remote sensing data to identify crop health and development patterns. Remote sensing data is delivered in large files which require fast broadband connections for effective communication. It can, therefore, be concluded that lack of decision support systems to be a major barrier to the adoption of PA. Farmers need decision support systems that enable effective decision making based on accurate and timely data.

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Federal University of Technology, Minna, Nigeria

ANALYSIS OF A KNUCKLE JOINT USING DIFFERENT MATERIALS

***Aisha Muhammad[1,2], Ibrahim Haruna Shanono[3]**
[1]Department of Mechatronics, Faculty of the Engineering, Bayero University Kano, Nigeria
[2]Mechatronics Engineering, Faculty of Engineering, University Malaya, Malaysia
[3]Department of Electrical, Faculty of the Engineering, Bayero University Kano, Nigeria
*Corresponding author email: ayshermuhd@gmail.com

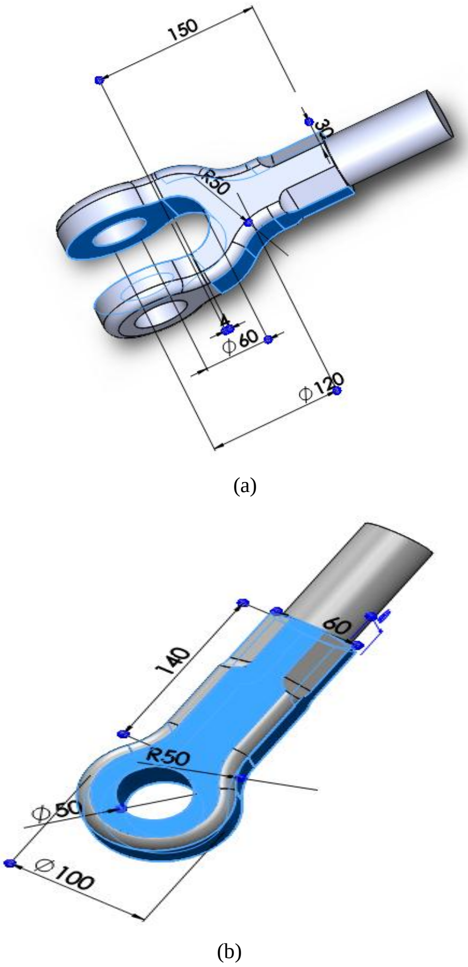
ABSTRACT
A knuckle joint is a device used to secure two loaded rods together. It is applicable in various structures and systems, hence making it design, analysis and production vital to the field of engineering. This paper aims at performing the deformation, stress and strain analysis of a knuckle joint using ANSYS workbench. Three different materials: Aluminium, Copper alloy, and structural steel are used with five different load values. The results obtained were examined, and the best material capable of withstanding the subjected load is identified. From the results obtained for the three different cases, it appears that Aluminium alloy has the highest deformation level, followed by Copper alloy, then Structural steel. This indicates that Structural steel is having the least deformation when subjected to the same loading condition. The 3D model of the knuckle joint was designed in SolidWorks and then exported to ANSYS software for analysis.

Keywords: *ANSYS Workbench, FEA, knuckle joint, Static analysis.*

1 INTRODUCTION

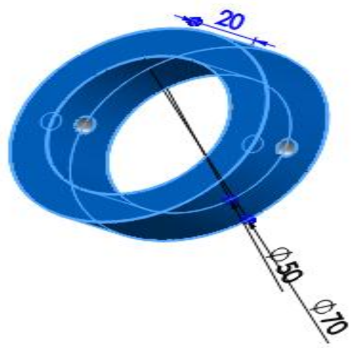
A knuckle joint is used in joining two rods lying on the same plane with the axis intersecting or coinciding. It allows little movement between the rods along the pin axis while transmitting a force (Jha, 2016). Static analysis can be performed in ANSYS software after the application of loads in brake torque, traction, vehicle weight and steering. (Sharma, 2014), (Yadav, 2016) in their papers used computer Aided Engineering to design a steering knuckle joint and also analyse the model using ANSYS. For the connection of rods together to give a little amount of movement under specific loads, a knuckle joint is used. It can also be utilised for compressive load under certain instructions. However, the connection can be easily removed for corrections or adjustments (Patil, 2016).

It is a mechanical joint for joining two fitting under loads with a flexible amount of movement (Rao, 2017). Some examples of knuckle joints include roof truss rod joint, roller chain link, etc. A knuckle joint consist of five main parts: a knuckle pin, an eye end, fork end, Collar and a taper pin as shown in figure 1.

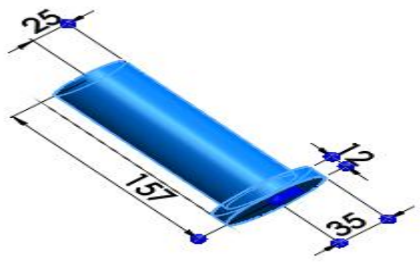


(a)

(b)



(c)



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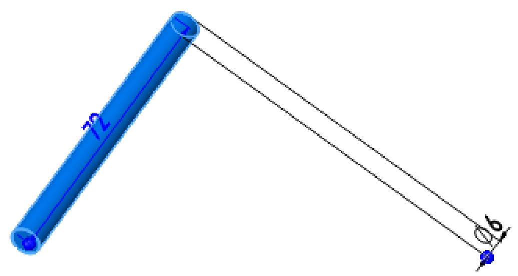


(e)

FIGURE 1 (A) FORE END (B) EYE (C) COLLAR (D) KNUCKLE PIN (E) TAPPER PIN

The two ends of the rod are formed into eye and fork respectively, with the fork having eyes in each of its legs. Having secured the eye inside the fork, the knuckle pin is placed inside them after the alignment of the holes in the fork and eye. A knuckle joint has one of its ends tightened with a taper pin and collar; the other end has to have a head (Patil, 2016). It has a simple design as it can be constructed and reconstructed again when the need requires. A knuckle joint has a range of applications such as in robotics, reciprocating engine valve, fulcrum, suspension bridge, etc. (Bhandari, 2012). For turning the front wheel of a vehicle, the joint used is a steering knuckle which applies circular forces on the assembly as it turns the vehicle (Rao, 2017).

A study conducted on the joint of a coupling system shows that failure in knuckle joint is caused by torsional overload (Pantazopoulos, 2007). However, several studies have proved that severe friction which causes the wearing of the material results in delamination wear (Pantazopoulos, 2004), (Psyllaki, 2002). Jones (1993) in his book states that a common failure in most engineering structures is due to shear stress caused by torsional forces. Any of the following modes can result in the failure of a knuckle joint: the crushing of pin, shear failure of the pin and the tensile failure of the flat end bar (Patil, 2016).

In this paper, a finite element analysis of a knuckle joint assembly is done with different design constraints and materials using ANSYS workbench. The 3D-model is designed and modelled in SolidWorks. Before the analysis is done, the meshing of the imported model is done, and design constraints are set (Gupta, 2005).

2 MATERIALS AND METHODS

2.1 FINITE ELEMENT METHOD

Partial differential equations that represent the approximate exact solution of solving a numerical method problem is described as FEM (Chen, 2014). To solve the engineering problem, FEM is used whereby the structure is subdivided into much smaller element. Using this method, complex structures can be solved by making FEM a desirable approach. Figure 2 shows the FEM procedure and steps used in solving the problem.

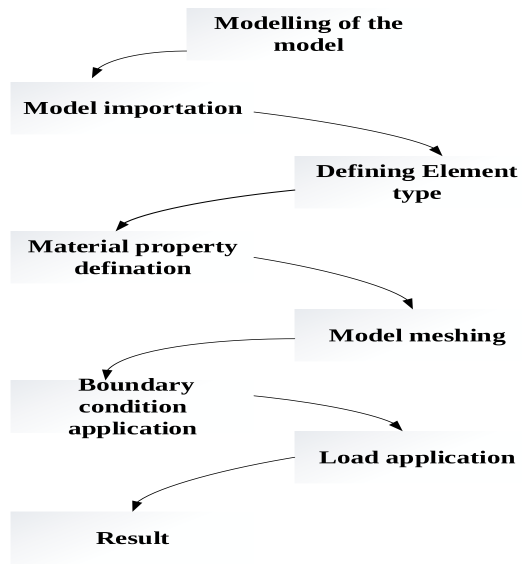


FIGURE 2: FEA FLOW CHART

2.2 KNUCKLE JOINT MODELLING IN CAD.

To find the dimensions in a knuckle joint, failure analysis of its various sections are looked into to make sure that the resulted stresses in these parts are less than the permissible stress (Rao, 2017). Strength equations for these failures as well as empirical relations are used in determining the dimensions of the knuckle joint. The rods and knuckle joints are separate from each other with the rod welded to the eye and fork. There is no motion available between the pin and eye. In a situation where little movement exists, bearings are needed to reduce wear. In such cases, the pin is secured in the eye with a screw, and then roller or plain bearings are added to the fork. The knuckle joint 3D model was designed using SolidWorks.

2.2 FINITE ELEMENT ANALYSIS USING ANSYS

A numerical method which uses a finite element that involves dividing a system into smaller and simpler parts to find an approximate solution to a boundary value problem while minimising any related error function is known as Finite Element Method (Jha, 2016). Analysis of any structure begins with the geometry definition which is defined based on the type of simulation analysis that is to be carried out. Since our study focuses on stress and deformation, the exact FEA model used is a substructure. To introduce a 3D structure for analysis in ANSYS software can be achieved in either saving it in an Initial Graphics Exchange Specification(IGES) format and then imported into the ANSYS workbench or can be created in the ANSYS workbench entirely (Guanzhu et al., 2012) (Kim et al., 2007) (Janq, 2002) (Xi et al., 2002). In this paper, the analysis is performed via geometry cell by importing the geometry from a CAD in the IGES format into the software.

The simulation analysis that is carried out determines the material property definition. For an efficient and qualitative analysis of material, the material properties need to be correctly and carefully entered. Depending on the aim of the analysis, some mechanical properties such as density, strength and coefficient of thermal expansion definition is optional (Barbero, 2014). Knowing and declaring the correct value of the material property is very useful for design analysis purpose. Results vary based on the different types of materials having different densities. The Young's modulus of a material is a numerical constant that shows the elasticity and capability of a solid to withstand changes when subjected to tension in a particular direction. The higher Young's modulus, the stiffer the material will require an amount of load to deform. Poisson's ration and Young's modulus describes the strength and nature of how a material structure deforms based on a particular constraint. Two other essential properties that determine when the material losses its elastic behaviour and the maximum stress a material can undergo are the yields and tensile strength respectively. (Nipun, 2015). After importing the geometry, the definition of an element and material properties is carryout. As outlined earlier, in choosing the material property of the structure, the yield strength is used as a standard. Since the pin is being exposed to both shear and bending stress, strength becomes a standard for the pin material selection (Jha, 2016). Knuckle joint is made from different materials, but in this study structural steel is the material used. The table below shows the material property of the various materials used for the knuckle joint analysis.

TABLE 1: PROPERTIES OF MATERIALS

Material/ Property	Structural steel	Aluminium alloy	Copper alloy
Young's Modulus (Pa)	2E+11	7.1E+10	1.1E+11
Density (kgm^-3)	7850	2770	8300
Poisson's ratio	0.30	0.33	0.34
Bulk modulus (Pa)	1.6667E+11	6.9608E+10	1.1458E+11
Shear Modulus (Pa)	7.6923E+10	2.6692E+10	4.1045E+10
Tensile strength (Pa)	2.5E+08	2.8E+08	2.8E+08
Ultimate shear strength (Pa)	4.6E+08	3.1E+08	4.3E+08

Knuckle joint are made from different types of materials such as grey, white cast and ductile iron (Sharma, 2014). After the importation of the 3D model into the ANSYS software, meshing process is performed. Meshing is the breaking down of the structure into smaller elements to perform analysis on each of these components (Talikoti et al., 2016). It is a discrete realisation of the structure, which helps in solving the exact model solutions. The quality and accuracy of the result depend on the meshing size. Finer or smaller meshing size results in a better and accurate result with a higher computational time (Sharma, 2014), (Qiongying, 2014). ANSYS is a great control tool for meshing (Baomin, 2005). Meshing tools in ANSYS are classified into: Unit size control, Level control of the intelligent division, Thinning grid control, Shape settings of meshing, and Grid partition. (Guanzhu et al., 2012). In this analysis, free mesh type is used due to the setup time and computational expense, with the speed and ease of application. The default meshing control is used having a relevance value of +50 with a medium smoothing number of iteration. Using Table 1 above, the model properties are set in the software followed by dividing the structure or model into finer number of nodes and elements. Figure 3 shows the meshed structure of the knuckle joint.

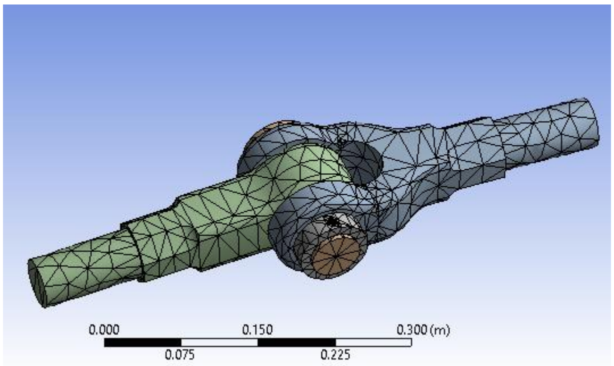
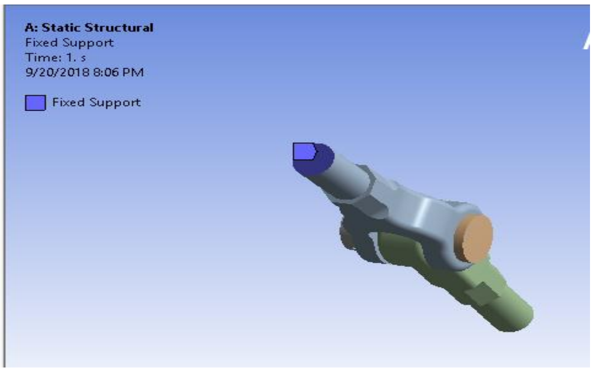
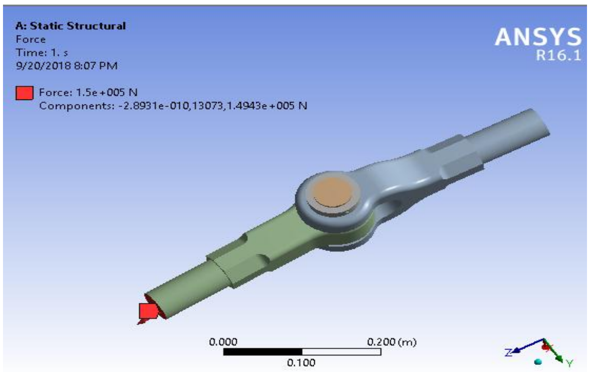


FIGURE 3: MESHED MODEL IN ANSYS

To determine the maximum (Von-Mises) stress and deformation of the knuckle joint. The constrains needs to be carefully applied based on the prevailing conditions (Mahesh P. Sharma, 2014). Constraints such as fixed support and forces are applied to the model after the meshing is done, this is very important and serves as a primary step required in the analysis (Talikoti et al., 2016). Constraints are set in a manner conforming to the real-life situations (Guanzhu et al., 2012). One end of the knuckle pin is fixed while a static force of values 1.5E05N is applied on the other end as shown in figure 4 below.



(a)

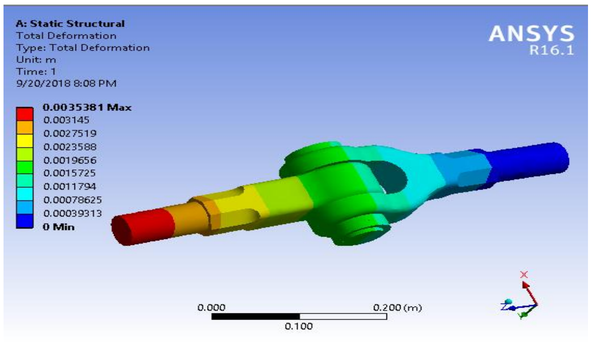


(b)

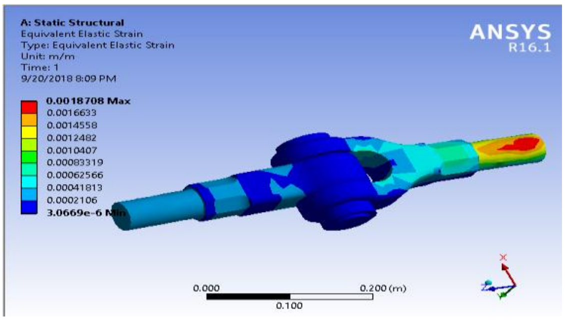
FIGURE 4: (A) BOUNDARY CONDITION APPLICATION (B) LOAD APPLICATION

3 SIMULATION RESULT AND DISCUSSION

To determine the stress, strain and deformation results, the knuckle joint is subjected to the design constraints and the static analysis is carried out. The maximum Equivalent (Von-mises) stress result retrieve from the analysis result based on the applied constraints is developed at the fork end. The deformation and stress of a knuckle joint are analysed. The result of the analysis as seen in table 2 shows the maximum and minimum values of Von-mises stress of 3.73Mpa and 0.508MPa respectively.



(a)



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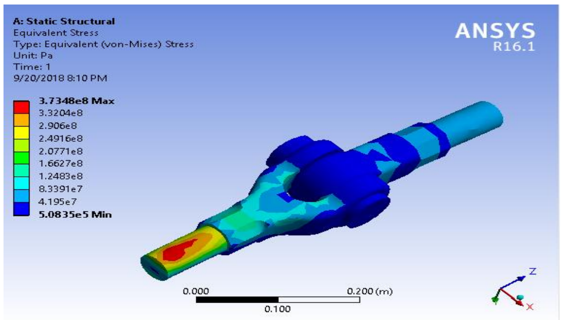
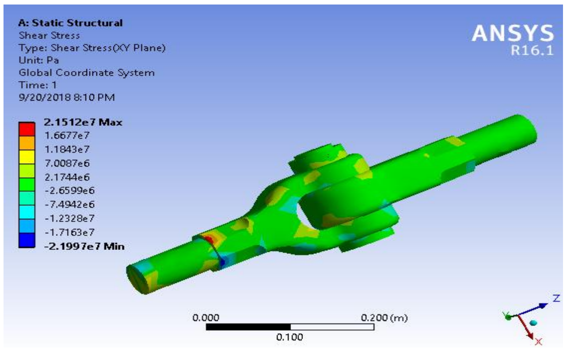
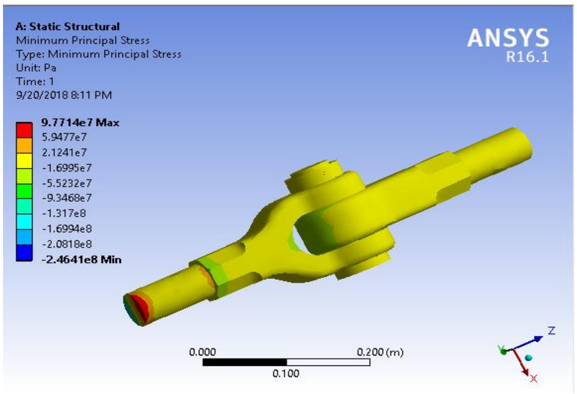


(c)



(d)



(e)

FIGURE 5 (A) TOTAL DEFORMATION (B) EQUIVALENT ELASTIC STRAIN (C) EQUIVALENT STRESS (D) SHEAR STRESS (E) PRINCIPAL STRESS

TABLE 2: ANALYSIS RESULT

Analysis	Minimum	Maximum
Total Deformation (m)	0	3.54E-03
Equivalent Elastic Strain (m/m)	3.07E-06	1.87E-03
Equivalent Stress (Pa)	5.08E+05	3.73E+08
Shear Stress (Pa)	-2.20E+07	2.15E+07
Minimum Principal Stress (Pa)	-2.46E+08	9.77E+07

The maximum deformation, stress and equivalent strain results obtained using structural steel, Aluminum alloy and copper alloy material for an applied load force of 150000N to 350000N in an increment of 50000N are tabulated as shown table 3, 4 and 5 respectively.

TABLE 3: RESULT USING ALUMINUM MATERIAL

Force(N)	1.50E+05	2.00E+05	2.50E+05	3.00E+05	3.50E+05
Total Deformation (m)	8.96E-03	1.19E-02	1.49E-02	1.79E-02	2.09E-02
Elastic Strain (m/m)	4.97E-03	6.63E-03	8.29E-03	9.94E-03	1.16E-02
Equivalent Stress (Pa)	3.52E+08	4.70E+08	5.87E+08	7.05E+08	8.22E+08

TABLE 4: RESULT USING STRUCTURAL STEEL MATERIAL

Force(N)	1.50E+05	2.00E+05	2.50E+05	3.00E+05	3.50E+05
Total Deformation (m)	3.19E-03	4.26E-03	5.32E-03	6.39E-03	7.45E-03
Equivalent Elastic Strain (m/m)	1.76E-03	2.49E-03	2.93E-03	3.52E-03	4.11E-03
Equivalent Stress (Pa)	3.52E+08	4.69E+08	5.86E+08	7.03E+08	8.20E+08

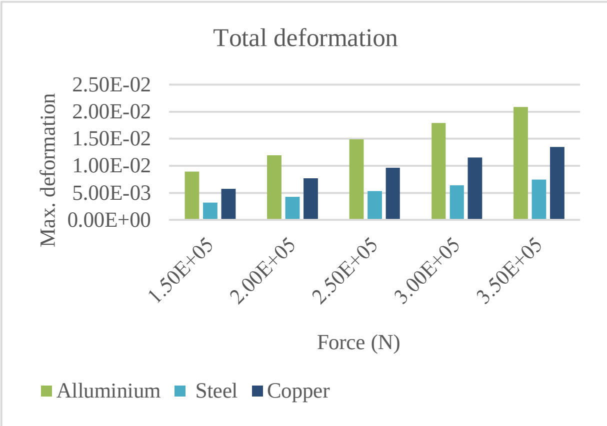
TABLE 5: RESULT USING COPPER MATERIAL

Force(N)	1.50E+05	2.00E+05	2.50E+05	3.00E+05	3.50E+05
Total Deformation (m)	5.77E-03	7.70E-03	9.62E-03	1.15E-02	1.35E-02
Equivalent Elastic Strain (m/m)	3.21E-03	4.28E-03	5.35E-03	6.42E-03	7.49E-03
Equivalent Stress (Pa)	3.53E+08	4.70E+08	5.88E+08	7.05E+08	8.23E+08

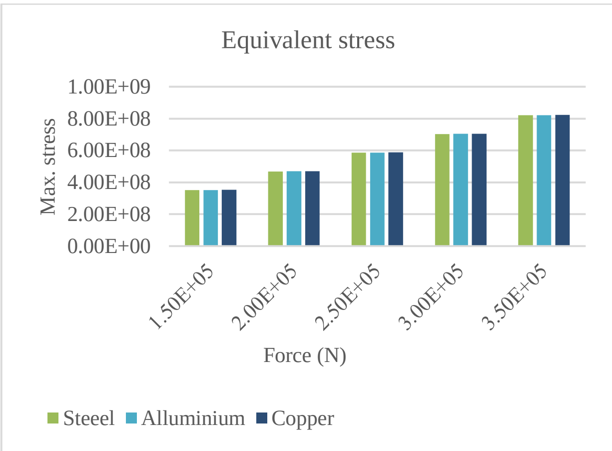
The analysis using various material (Aluminum alloy, structural steel and carbon) as shown in tables 3, 4 and 5 gives the results obtained from the static analysis using aluminium, steel and copper materials respectively. Comparison of the result is conducted and shown using histograms in figure 6.

3.1 Material Comparison

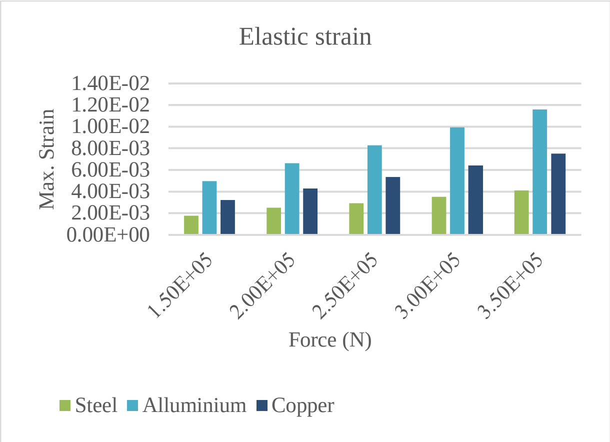
The deformation, Von-mises equivalent stress, and elastic strain for the three sets of materials using five different static force are compared and shown graphically in the figure 6a, 6b and 6c below.



(a)



(b)



(c)

FIGURE 6 (A) TOTAL DEFORMATION
(B) MAXIMUM EQUIVALENT STRESS (C) ELASTIC STRAIN

4 CONCLUSION

The paper explores the strength analysis of a knuckle joint using ANSYS workbench. Based on the results obtained for the three different cases, it appears that Aluminum alloy has the highest deformation level, followed by Copper alloy, then Structural steel. This indicates that Structural steel is having the least deformation when subjected to the same loading condition. Based on the Von-mises stress analysis obtained, Aluminum has the highest value of maximum stress, followed by Copper, then Structural Steel. All the maximum Von-Mises stress in the material structures are less than the material tensile and ultimate shear stress, indicating that the applied pressure is less than the material yield point value.
Lastly, structural steel is the best choice based on the results obtained. The materials are too excessive, and this shows the wastage of resources and cost. Therefore, it is recommended that the structure should be optimised.

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EFFECT OF REMOULDED DENSITY ON CREEP OF BLACK COTTON SOIL

Jibrin, R.; Adejumo, T. W. E. and Alhaji, M. M.
Department of Civil Engineering, Federal University of Technology, Minna
Corresponding author: ramatumustapha90@gmail.com

ABSTRACT
A black clay soil collected using the method of disturbed sampling, from Bako village, along Gwagwalada road, Abuja, was characterized and tested for mineral composition. The clay was compacted at four different compaction energy levels (Reduced Standard Proctor, Standard Proctor, West African Standard and Reduced Modified Energy Levels). The resultant compaction characteristics were used to remold the clay for 72 hours loading consolidation test which was used to evaluate the consolidation and creep parameters as well as the magnitude of creep. Results showed that the clay classified as Clay of High Plasticity (CH) according to unified Soil Classification System and composed predominantly of secondary minerals including montmorillonite. The recompression index was observed to reduce from 0.0522 at reduced standard proctor energy level through 0.0472 at standard proctor energy level to 0.0243 at West African standard energy level which ended up with 0.0143 at reduced modified energy level. The compression index however, reduced from 0.489 at reduced standard proctor energy level to 0.398 at standard proctor energy level through 0.305 at West African standard energy level after which the value increased to 0.324 at reduced modified energy level. The trend of creep shows increase from 248.6 mm at reduced standard proctor to 350.6 mm at standard proctor compaction energy level after which the value reduced to 226.5 mm at reduced modified compaction energy level.
Keywords: *Compaction Energy, Creep, Coefficient of secondary consolidation, One-Dimensional consolidation.*

1 INTRODUCTION

Consolidation and creep of clayey soils are aspects of geotechnical engineering that has been and has continued to be under intensive study. Terzaghi (1941) was the first to evolve an organized and classical mathematical model to represent the progress of consolidation for soils which the author defined as a decrease in the water content of a saturated clay soil without replacement of the water by air. Consolidation was believed to be completed when all the water in the soil pore have dissipated. Laboratory experimental result had shown the author that deformation continued after the completion of consolidation.

Buisman (1936) was the first to study in to detail the deformation that follows the Terzaghi's one-dimensional consolidation model. The author evolved a mathematical model:

$$z_t = \alpha_p + \alpha_s \log_{10} t \tag{1}$$

where α_p and α_s are primary and secondary compression index simultaneously. The author took the parameter α_p and α_s to be linearly proportional to the load p. Grey (1936) explain the secondary time effect and observed that it occurs in all soils but more strong in organic clay soils. Taylor and Merchant (1940) are one of the first to evolve a mathematical model combining creep with consolidation:

$$\frac{de}{dt} = \frac{\partial e}{\partial t} + \frac{\partial e}{\partial p}.\frac{dp}{dt} \tag{2}$$

Where e is the void ratio and p is the vertical effective stress. Taylor (1942) termed the first theory as theory A and presented a new theory B. Theory B pointed out that secondary compression in the previous loading steps has significance influence in the next load increment. This theory also considers the resistance against deformation which is assumed to depend on the rate of deformation. The author evolved a mathematical model as:

$$e = e_{Eop} - C_\alpha Log \frac{t}{t_{100}} \tag{3}$$

where e_{Eop} is the void ratio at the beginning of secondary consolidation and C_α is the slope of void ratio versus time in a semi-logarithmic space.

Koppejan (1948) changed Buisman's mathematical model to resemble Terzaghi's logarithmic compression law by making the parameter to depend on logarithm of vertical effective stress. The author developed a model for direct evaluation of magnitude of creep:

$$\varepsilon = \frac{1}{C_p}\ln\left(\frac{\sigma_v^1}{\sigma_{vo}^1}\right) + \frac{1}{C_s}\ln\left(\frac{\sigma_v^i}{\sigma_{vo}^i}\right)\log t \tag{4}$$

where ε is the magnitude of creep, C_p is the coefficient of primary consolidation, C_s is coefficient of secondary consolidation, σ_{v0} is the surcharge pressure and t is time.

Suklje (1957) criticized Taylor's work pointing out that theory A does not use logarithmic creep law and the stress component in theory B have no clear physical meaning. Both theories cannot be applied to thick clay layers. The author assumed parabolic excess pore pressure distribution and determine the rate of strain which was put equal to the rate of pore water outflow. The stress-strain path was extrapolated to bigger clay thickness by approximation. According to Lambe (1958), the dissipation of pore fluids from the micro voids is the reason for secondary compression. Similarly, Berry and Xu, (1972); De jong, 1968 and Nakaoka *et-al.,* (2004) concluded that secondary compression results from a local mass transfer of water between macro-pores and micro- pores. Gibson et-al (1967) improved Terzaghi's equation by removing the small strain limitation and allowing variable compressibility and permeability of grain skeleton. The effect of the self-weight of the consolidating layer was shown. The resulting equation is giving as:

$$\pm\left(\frac{\rho_s}{\rho_f}-1\right)\frac{d}{de}\left[\frac{k(e)}{1+e}\right]\frac{\partial e}{\partial z}+\frac{\partial}{\partial z}\left[\frac{k(e)}{\rho_f(1+e)}\frac{d\sigma^1}{de}\frac{\partial e}{\partial z}\right]+\frac{\partial e}{\partial t}=0 \quad (5)$$

where k is coefficient of permeability.

Leroueil *et-al.,* (1985) conducted four different types of one-dimensional odometer tests (constant rate of strain test, controlled gradient test, multiple stage loading test and creep test) on five sites located within the Champlain sea clays. Results showed that the rheological behavior of these clays is controlled by two curves ($\sigma'_p-\varepsilon_v$) and ($\frac{\sigma'_v}{\sigma'_p}-\varepsilon_v$). Formula for calculating creep strain is given by Smoltczyk, (2002) as

$$\varepsilon_{cr}=\frac{C_\alpha}{1+e_0}\log_{10}\left(\frac{t}{t_0}\right) \quad (6)$$

Where e_0 is the initial void ratio, t_0 is initial time at which creep is assumed to start; Cα is the secondary compression which is the slope of e-logt plot of odometer test. The author concluded that Cα changes with change in over-consolidation ratio with its maximum occurring at OCR=1. Meanwhile, Mesri and Godlewski (1977) had proposed that Cα is approximately 0.04 times the compression index C_c for normally consolidated clays.

Alexandre (2006) developed a creep model which was observed to be more general of the earlier models:

$$\sigma_d=\sigma_{df}(\varepsilon)+K(e).\varepsilon^n \quad (7)$$

Considering specified stress ranges, equation 7 can be rewritten as

$$\sigma_d=E\dot{\varepsilon}+K\dot{\varepsilon}^{-n} \quad (8)$$

The solution of the equations is given as:

$$\varepsilon=\left(\frac{\sigma_d}{E}\right)-\left(\frac{K}{E}\right).\frac{1}{\left[\left(\frac{K}{\sigma_d}\right)^{\frac{(1-n)}{n}}+\left(\frac{1-n}{n}\right)\frac{E.t}{K}\right]^{\left(\frac{n}{1-n}\right)}} \quad (9)$$

$$\dot{\varepsilon}=\ .\frac{1}{\left[\left(\frac{K}{\sigma_d}\right)^{\frac{(1-n)}{n}}+\left(\frac{1-n}{n}\right)\frac{E.t}{K}\right]^{\left(\frac{1}{1-n}\right)}} \quad (10)$$

The effect of loading time on clay soils was studied through oedometer tests considering double and single drainage (Halder *et-al.,* 2017). The author concluded that increased loading time was observed to affect consolidation rates, coefficient of consolidation C_v and secondary compression index Cα. This was attributed to time dependent plastic adjustment of soil fabric. Maria and Maria (2018) presented an experimental evidence which the author believed could contribute to the understanding of the viscous behavior of soft soils. The author presented equation of primary one-dimensional consolidation, including soil viscous resistance and compressibility of water with analytical solution. The general equation for one-dimensional viscous consolidation considering the compressibility of water may be written as:

$$\frac{\partial e}{\partial t}=-C_k\frac{\partial}{\partial t}\left(e-a_v.\sigma'_v\right)+C_v\frac{\partial^2}{\partial z^2}\left(e-a_v.\sigma'_v\right) \quad (11)$$

Where $C_k=\frac{e}{k.a_v}$ and $C_v=\frac{k(1+e)}{\gamma_w.a_v}$

Using some simplifying assumptions, the equation was mathematically analyzed to give:

$$e=e_0-\sigma.a_v\left\{1-\sum_{m=0}^{n}\left[\frac{(Bm_2-c_c)}{M\sqrt{A_m}}\ell^{-\frac{Bm_1-t}{2C_nh^2}}-\frac{(Bm_1-c_c)}{M\sqrt{A_m}}\ell^{\frac{-Bm_2-t}{2(c_n-h^2)}}\right]Sin\left(\frac{m.z}{h}\right)\right\} \quad (12)$$

$$u=\sigma\left\{\sum_{m=0}^{n}\left[\frac{\left(Bm_2-c_c\right).\left(c_k+h^2-\tfrac{1}{2}\dfrac{Bm_1c_vt}{c_v}\right)}{M\sqrt{A_m}c_{kt}h^2}\ell^{-\tfrac{1}{2}\frac{Bm_1-t}{c_{kt}-h^2}}-\frac{\left(Bm_1-c_c\right).\left(c_{kt}h^2-\tfrac{1}{2}\dfrac{Bm_2c_{vt}}{c_v}\right)}{M\sqrt{A_m}c_{kt}h^2}\ell^{-\tfrac{1}{2}\frac{Bm_2-t}{c_{kt}-h^2}}\right]Sin\frac{m.z}{h}\right\} \quad (13)$$

These two solutions gave the relationship in terms of void ratio and the pore water pressure.

It has been shown by many authors that variation in compaction energy level affects significantly the geotechnical properties of soils (Daniel and Benson, 1990; Daniel and Xu 1993; Lara *et al.,* 2014; Mada *et al.,* 2013; Singh *et al.,* 2015). But, it is worth to note that all the creep models reviewed above do not consider remolded clay soils specifically; this work is therefore aimed at studying the effect of remolded densities on the black cotton soils of Nigeria.

2 METHODOLOGY

The material used in this study is mainly black cotton soil collected from Bako village along Gwagwalada road, Federal Capital Territory, Abuja, Nigeria. The clay was collected at depth between 0.5 – 1.2m using the method of disturbed sampling. The clay was immediately transferred to the laboratory, air-dried and pulverized using the method highlighted in part 1 of BS 1377 (1992).

The method involves carrying out index properties test (Liquid limit test, plastic limit test, mechanical sieve analysis, hydrometer analysis test and specific gravity tests) on the black cotton soil so as to classify the clay and determine the percentage composition of the silt and clay sized particles contained in the clay soil. X ray diffraction test was carried out on the clay sample. A crushed sample was collected and crushed and small amount of ethyl alcohol was added to stop the hydration reaction. About 10g of the grounded clay was immersed in ethyl alcohol and kept in desiccator at room temperature. The powdered sample was loaded on a sample holder. The powdered samples were scanned with an X ray powder diffractometer using copper and potassium.

Compaction test was then carried out on the clay soil at four different compaction energy levels. The Reduced Standard Proctor energy level (Daniel and Benson, 1990; Daniel and Wu, 1993), Standard Proctor energy level (Proctor 1933), West African Standard energy level (Nigeria General Specification for Road and Bridge Works, 1992), Reduced Modified Standard energy level. This is to allow for the use of four different standard compaction energy levels to obtain wide range of densities which is necessary to achieve the aim of this study.

Reduced Modified Standard proctor is included as a new method which has not been specified in literature. The method involves the use of 4.5kg rammer falling through the height of 46 cm in to a standard compaction mold with volume of 944 cm^3. The soil was placed in the mold at five layers with each layer receiving 15 blows. This is lower than the modified compaction which has 25 blows per layer but higher than West African Standard which has 10 blows per layer. The Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) were evaluated for each of the compaction energy levels. These compaction characteristics were used to mold specimen for consolidation tests.

The consolidation test was conducted using an odometer device and a transparent cell within which a mold was placed. The mold which has a diameter of 50 mm and height of 20 mm, was covered top and bottom with a porous stone (double drainage) was always lubricated with silicon grease to prevent friction between the clay and the mold during consolidation test. Filter papers were placed between the porous stone and the clay contained in the mold to avoid clay soil particles migrating in to the pores of the porous stone to block the pores. Each loading was applied for 72 hours before addition of further loads. This would allow the deformation to enter secondary stage. Five loadings whose present load doubled the previous one were applied before unloading for further 72 hours. The parameters obtained from the consolidation tests were used to evaluate the magnitude of creep from the model developed by (Smoltczyk, 2002).

3.0 RESULTS AND DISCUSSION

The result of the physical properties of the test soil used for this study found to be almost exact, if not exact to that of black cotton soil is shown in Table 1. From the Table, the clay is classified under clay of high plasticity (CH) and is very prone to creep settlement.

TABLE 1: PHYSICAL PROPERTIES OF TEST SOIL

Description	Quantity
Sand (%)	18.4
Silt (%)	28.9
Clay (%)	53.0
Liquid Limit (%)	64.3
Plasticity Index (%)	35.9
Specific Gravity	2.66
Unified Soil Classification	CH

The result of the mineral composition of the clay soil is shown in Table 2 while the graphical representation of the XRD result is also shown in Figure 1.

TABLE 2: MINERALOGICAL COMPOSITION OF CLAY

Description	Quantity
Quartz (%)	50.00
Ankerite (%)	6.25
Calcium Silicide (%)	6.25
Montmorillonite (%)	6.28
Anorthite (%)	8.33
Sodium Aluminium Silicate Hydrate	6.25
Anothoclase	8.33
Orthoclase	10.07

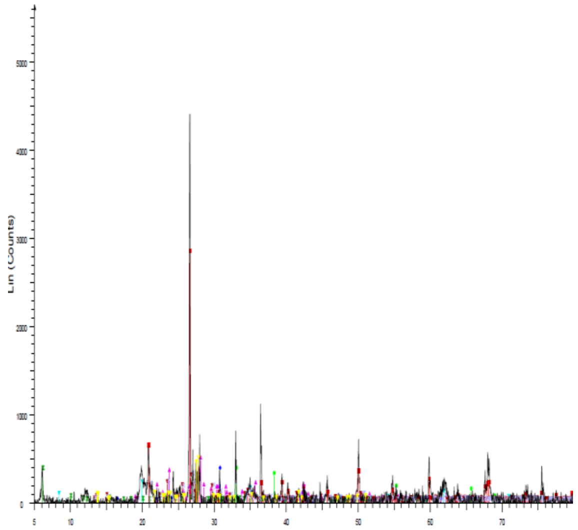

FIGURE 1: XRD RESULT OF TEST SOIL

From Table 2 and Figure 1, the soil contained substantial composition of secondary minerals including troublesome montmorillonite mineral. This will increase the activity of the clay and consequently, the creep response of the clay.

2.1 COMPACTION CHARACTERISTICS

The result of the compaction characteristics of the clay soil at varied compaction energy level is shown in figure 2 and 3. Figure two show the variation of maximum dry density (MDD) with compaction energy level while figure 3 shows the variation of optimum moisture content (OMC) with varied compaction energy level.

Each compaction test result shows increase in dry densities with increase in moisture contents down to optimum moisture content after which the dry densities decreases. These trends are in agreement with Lambe (1958) and Yusoff *et al*. (2016) who attributed the variation to soil structure and electrical double layer theory.

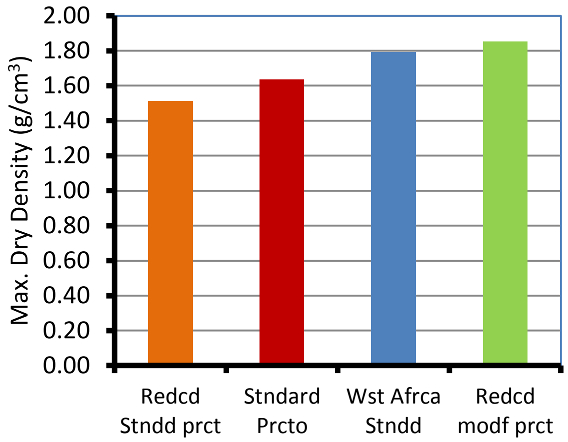

FIGURE 2: VARIATION OF MDD WITH COMPACTION ENERGY LEVELS

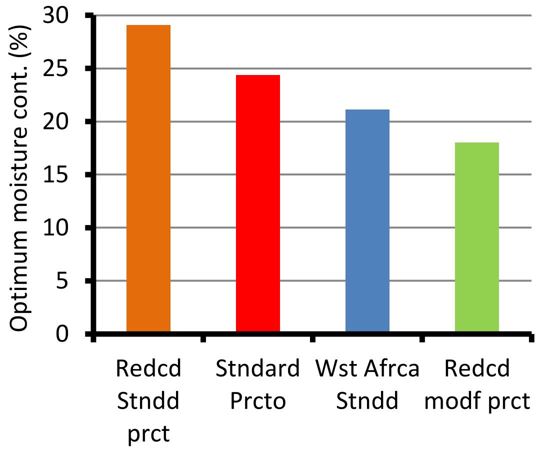

FIGURE 3: VARIATION OF OMC WITH COMPACTION ENERGY LEVELS

Clay soils usually possess attractive Van der Waals' forces between two clay particles and a repulsive force which is due to double layers of adsorbed water trying to come in to contact with each other. The attractive force tends to remain the same while the repulsive force increases with increase in water. If the net force between the particles is attractive, flocculated structure will result and if the net force is repulsive, dispersed structure will result. Addition of small amount of water will reduce the activity of the double layer which will encourage the clay particles to be in a flocculated form thus resulting in to soft soil mass. Increase in water will reduce the action of the double layer around clay particles thus resulting in particles of dispersed form which are dense in structure.

Figures 2 shows increase in MDD with increase in compaction energy level from 1.512g/cm^3 at reduced standard proctor compaction energy level through 1.635g/cm^3 at standard proctor energy level to 1.793g/cm^3 at West African Standard energy level which ended with 1.851g/cm^3 at reduced modified standard proctor energy level. The OMC however, reduces with increase in compaction energy level from 29.1% at reduced standard proctor compaction energy level through 24.4% at standard proctor energy level to 21.1% at West African Standard energy level which ended in 18.0% at reduced modified standard proctor energy level. This trend is in agreement with Daniel and Benson, 1990; Daniel and Xu (1993), Lara *et al.*, (2014), Mada *et al.*., (2013), and Singh *et al.*, (2015).

2.3 CONSOLIDATION CHARACTERISTICS

The result of consolidation tests carried out on the clay soil at four different compaction energy levels are shown in figure 4. It was observed that recompression index reduced from 0.0522 at reduced standard proctor energy level through 0.0472 at standard proctor energy level to 0.0243 at West African standard energy level which ended up with 0.0143 at reduced modified energy level.

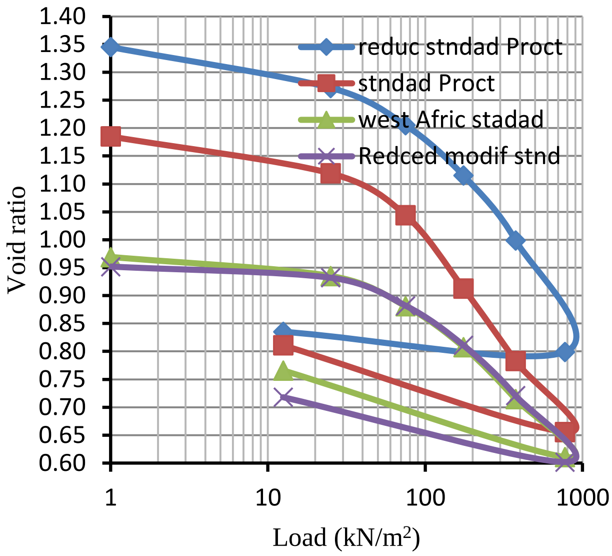

FIGURE 4: VOID RATIO-EFFECTIVE STRESS RELATION FOR VARIED COMPACTION ENERGIES

The compression index however, reduced from 0.489 at reduced standard proctor energy level to 0.398 at standard proctor energy level to 0.305 at West African standard energy level after which the value increased to 0.324 at reduced modified energy level. These trends can be attributed to the rate of water absorption of the clay during the 24 hours soaking with a seating load, before the application of the subsequent loads and the rate at which water dissipate out of the clay during consolidation.

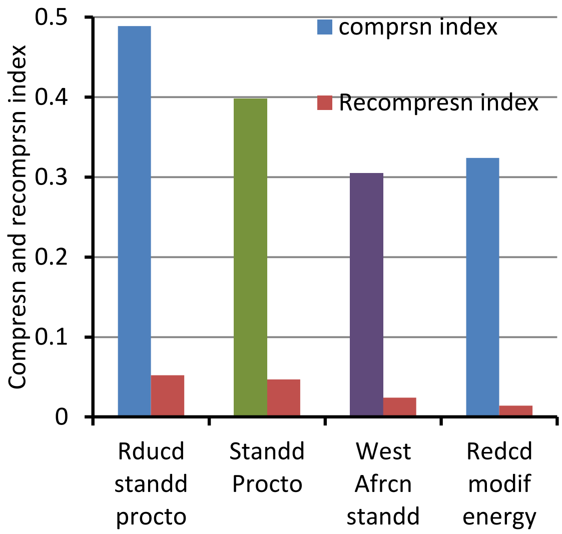

FIGURE 5: COMPRESSION AND RECOMPRESSION INDEX WITH VARIED COMPACTION ENERGIES

At lower energy level (reduced standard proctor), the molding moisture was higher while the dry density was lower. This allowed only little more water to be absorbed in to the clay mineral structure and the void spaces during the initial 24 hours soaking. During the consolidation process, only outward dissipation of water will occur and will continue until all water contained in the voids are dissipated. This process will be free and fast since there is no further absorption of water by the clay particles thus, resulting in higher compression index. At higher compaction energy level (reduced modified energy level), there was still more affinity for water absorption by the clay mineral structure which will reduce outward dissipation of water during consolidation. This is probably responsible for the increase in compression index at reduced modified energy level.

2.4 CREEP CHARACTERISTICS OF CLAY

Each of the loadings during consolidation test was allowed for 72 hours before increment of next load. This is to allow for evaluation of secondary consolidation properties which can be used to evaluate the secondary compression index and subsequently the magnitude of creep.

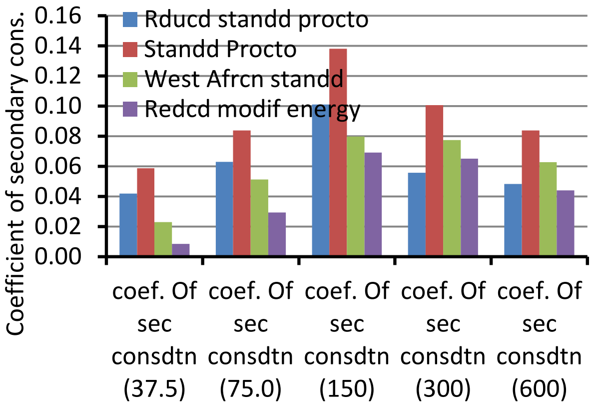

FIGURE 6: COEFFICIENT OF SECONDARY CONSOLIDATION FOR VARIED COMPACTION ENERGY AND AT VARIED LOADING

The result of coefficient of secondary consolidation for the varied load increments and at varied compaction energy levels is shown in Figure 6. The trend for all the loading increments showed increase from the first loading (25.0 kN/m^2) to its maximum at the loading of 175 kN/m^2 after which the values reduced. Variation of coefficient of secondary consolidation with change in loading has been observed by Larsson (1986), but the work did not give any specific trend in their relationship. The trend observed in Figure 6 probably resulted from the time resistance concept (R) as observed by Janbu (1969). Since excess pore pressure is zero during creep, time can be taken as an action and creep strain as a reaction.
The magnitude of creep for three years was evaluated using Smoltczyk (2002) model for each compaction energy level at the loading of 375 kN/m^2 and the results are shown in Figure 7.

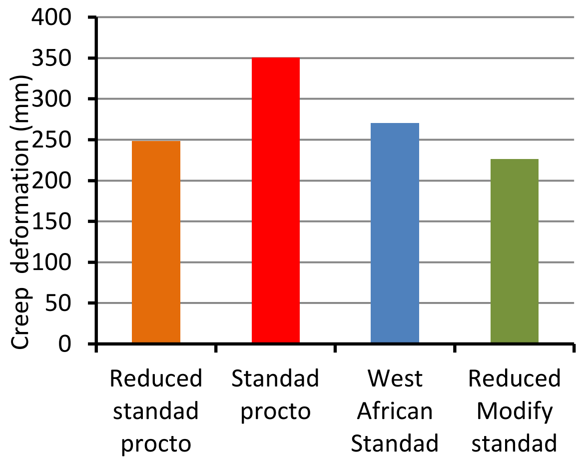

FIGURE 7: MAGNITUDE OF CREEP IN 3 YEARS FOR VARIED COMPACTION ENERGIES

The trend shows increase from 248.6 mm at reduced standard proctor to 350.6 mm at standard proctor compaction energy level after which the value reduced to 226.5 mm at reduced modified compaction energy level. The low value observed in the reduced standard proctor results from the excessive initial void ratio.

3 CONCLUSION

1. The soil used is classified as clay of high plasticity according to Unified Soil Classification system and A-7-6 according to AASHTO soil classification system.
2. The compression index decreased from 0.489 at reduced standard proctor down to 0.305 at West African compaction energy level after which the value increased to 0.324 at reduced modified compaction energy level.
3. The coefficient of secondary consolidation for all the compaction energy levels was observed to increase from loading of 25 kN/m^2 to its maximum at loading of 175 kN/m^2 after which the values dropped.
4. The magnitude of creep was observed to increase from 248.6 mm at reduced standard proctor compaction energy level to maximum of 350.6 mm at standard proctor compaction energy level after which the values dropped to 226.5 mm at reduced modified compaction energy levels.

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SUITABILITY OF CARBIDE WASTE AS PAVEMENT MATERIAL

*Isa, N.[1]; & Joel, M.[2]
[1] Department of Civil Engineering Ahmadu Bello University, Zaria
[2]Department of Civil Engineering, Federal University of Agriculture, Makurdi
*Corresponding author email: nuhamz@gmail.com

ABSTRACT
Calcium carbide waste is generated in large quantity globally. Because it is practically non-biodegradable, its indiscriminate disposal, is becoming a problem. One economic and effective way of disposing calcium carbide waste is by its use as pavement material. This paper is a review of some research works carried out on the suitability of calcium carbide waste as pavement material. It was discovered that calcium carbide waste can be used solely or in combination with rice husk ash as a flexible pavement material. The results obtained showed good performance in terms of strength and durability. The paper recommends that designers should specify its use in their designs to popularize it, especially in light traffic roads. Also the performance characteristics of carbide waste modified pavement block needs to be assessed.

Keywords: *Calcium Carbide, Pavement, Suitability, Waste*

1 INTRODUCTION

With development of infrastructure and increasing transportation demands, the development of new modified paving materials and use them in construction results in high performance pavements to meet the needs of the communities. While developing these new modified paving materials, attention should be paid to using industrial and by product waste materials effectively in construction to address environmental and economic concerns. Waste material recycling into useful products has become a main solution to waste disposal problems. Numerous organizations are leading a wide range of studies and research ventures concerning the plausibility, ecological reasonableness, and execution of utilizing reused items in roadway development.

Nwaobakata and Agunwamba, (2014) reported that palm kernel shells ash as filler on hot mix asphalt concrete reduced moisture susceptibility, improved permanent deformation characteristics and fatigue. Waste shingles used as an additive in hot mix asphalt enhanced the rutting resistance and Marshall stability of asphalt mixes.Burak and Ali, (2004). There was an observed increase in resistance against permanent deformation when waste high-density polyethylene was used as a bitumen modifier in asphalt concrete mix as a result of high marshall quotient and stability (Sinan and Emine, 2003).. Aisien, et al, (2006) researched the use of ground scrap tire rubbers in asphalt concrete, the investigation demonstrated that the rubber treated asphalt concrete blend has much preferable mechanical properties over the customary blend.

Calcium carbide waste is a by-product from the acetylene gas production. This gas is used around the world for lighting, welding, metal cutting, and to ripen fruit. The calcium carbide residue is produced by a simple process, which is obtained from a reaction between calcium carbide (CaC_2) and water (H_2O) to form acetylene gas (C_2H_2) and calcium hydroxide ($CaOH_2$), as shown by the following equation:

$$CaC_2 + 2H_2O \rightarrow - C_2H_2 + CaOH_2 \quad (1)$$

Calcium carbide residue mainly consists of calcium hydroxide, ($CaOH_2$) in a slurry form. The sludge has a pH of 12.2 and contains Cu, Pb, Fe, Mn, Ni and Zn ions Bogner et al., (2002).

The Nigerian Automobile Technicians Association (NATA) with a mean membership of twenty-five thousand panel beaters are scattered everywhere throughout the nation. Each panel beater produces an average of 30 kg of carbide waste daily (Chukwudebelu et al, 2013). This waste has accumulated over the years. The sludge are dumped indiscriminately resulting in environmental problems and health hazard such as adverse effects on the fertility of land, as these wastes are deficient in plant nutrients, drainage problems, contamination of water resources. It also causes air pollution as the dry powder becomes air borne on windy days, wastage of valuable resources of calcium hydroxide (slaked lime) and handling and disposal problem World Bank, (2003).

2 USE OF CARBIDE WASTE FOR CONSTRUCTION

Using recycled materials for construction purpose should be one of the most viable options in solid waste disposal. this is because of the large quantity of materials required and the relatively low quality requirements of materials used in construction compared to so many other industries (Lakshmi and Nagan, 2001). Also, there is usually a large volume of construction works in progress at any given time; in fact, there is no time that construction work is completely at a standstill.

Calcium carbide waste also find useful application in other areas of construction. Ndububa and Omeiza (2016) reported that calcium carbide waste concrete (CCWC) setting times increased with increase in percentage replacement levels of CCW and could make a better load bearing concrete at 5% CCW replacement level over plain concrete. Also, CCWC will do better in mass concreting and in hot climate in view of its higher setting times. Orgok and Ibrahim (2017) studied the properties of cement paste and concrete containing calcium carbide waste as additive and came to a conclusion that calcium carbide waste addition increased the consistency, but decreased drying shrinkage and setting times of cement paste, and could be used as an accelerator. CCW addition also slightly increased the workability of concrete. Compressive strength model of CCW-concrete with R^2 value of 0.830 could be used to predict concrete strength. Hungfang, *et al*, (2015) investigated the properties of chemically combusted calcium carbide residue and its influence on cement properties by synthesizing CCW and silica fume through a chemical combustion technique to produce a new reactive cementitious powder (RCP). It was reported that the compressive strength at the age of 45 days for RCP mortar mix was found to be higher than that of OPC mortar and OPC mortar with silica fume mix by 10% and 8%, respectively. Therefore, the synthesized RCP was proved to be a sustainable active cementitious powder for the strength enhancement of building materials. Nattapong et al, (2010) investigated the effects of calcium carbide residue-fly ash Binder on mechanical properties of concrete and reported that that the hardened concretes produced from calcium carbide residue – fly ash mixtures had mechanical properties similar to those from normal portland cement concrete.

3 USE OF CARBIDE WASTE IN PAVEMENT

The incorporation of carbide waste as a pavement material could be an effective way of disposing carbide waste in an economic and profitable manner considering the research findings of Joel *et al*, (2014) who investigated the stabilization of Ikpayongo lateritic soil with cement and calcium carbide waste and observed that calcium carbide waste can partially be used in the stabilization of Ikpayongo lateritic soil. Du *et al*, (2011) investigated the strength and California bearing ratio properties of natural soils treated with calcium carbide residue and reported that calcium carbide residue can be adopted as an alternative binder to treat over-wetted soils being used as highway embankment filling materials. Edeh *et* al, (2016) examined rice husk ash-carbide waste stabilization of reclaimed asphalt pavement (RAP) and came to a conclusion that The compaction characteristic was affected by the proportions of RAP + CW + RHA in the mixes. The maximum dry density (MDD) increased as the optimum moisture content (OMC) decreased with increased CW content and decreased RHA at any fixed RAP content in the mixes. The optimum RAP + CW + RHA mix, is durable as highway sub-base construction material with insignificant water absorption.

3.1 PROPERTIES OF CARBIDE WASTE

All the percentage passing sieve 30, 50 and 200 of both Portland cement and calcium carbide waste conformed with the range specified by ASTM D242; Portland cement and calcium carbide waste are non-plastic (NP), with specific gravity of Portland cement being 3.15 and that of calcium carbide waste 2.42. The mineral filler properties reported in Table 1 are within the ranges specified by ASTM D242 which makes the result satisfactory.

TABLE 1: PROPERTIES OF MATERIALS

Sieve (mm)	% Passing		ASTM D242
	Portland Cement	Carbide Waste	
No. 30 (0.59)	100	100	100
No. 50 (0.297)	100	100	95-100
No. 200 (0.074)	94	92	70-100
Plasticity Index	NP	NP	< 4
Specific Gravity	3.15	2.42	----

Source: Isa *et al*, (2018)

The oxide composition of the carbide waste (CW) and rice husk ash (RHA) are presented in Table 2. For the CW, calcium oxide (CaO) content is 63.14 % and the silicon oxide (SiO_2) content is 5.20 %. The CaO/SiO_2 ratio, which is indicative of cementing potential, is 12.14, $SiO_2 + Al_2O_3 + Fe_2O_3$ = 7.13 %. Loss on ignition (LOI), which is the indication of the amount of unburned carbon in the CW is 26.73%. According to ASTM C618-92a specification for coal fly ash, the CW used for this study falls under class C and like fly ash, is self-cementing.

TABLE 2: OXIDE COMPOSITION OF CALCIUM CARBIDE WASTE

Oxide	CaO	SiO_2	Al_2O_3	Fe_2O_3	Ag_2O	SO_3	TiO_2
Concentration (%)	63.14	5.20	1.60	0.33	1.40	0.35	0.14
Oxide	**BaO**	**V_2O_5**	**Nd_2O_3**	**Tm_2O_3**	**Y_2O_3**	**Lu_2O_3**	**LOI**
Concentration (%)	0.10	0.01	0.06	0.42	0.20	0.07	26.73

Source: Edeh *et al*, (2016)

Joel *et al*, (2013) stated that the optimum percentage of replacement of lime with calcium carbide waste showed increase in CBR value with CCW and lime content. CBR value of 40% and 60 % obtained with the treatment of laterite with 10 % CCW and lime respectively, qualifies the material for use as sub base material based on CBR value of 40 % for lime treated soil, recommended by Osinubi (1999). The plot of CBR against lime/CCW content is as shown in Figure 1.

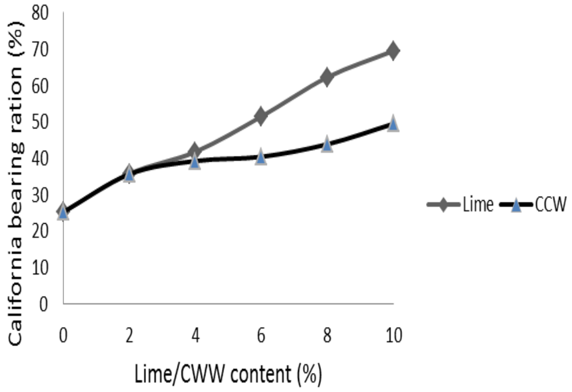

FIGURE 1: VARIATION OF CALIFORNIA BEARING RATIO WITH LIME/CCW CONTENT. SOURCE: JOEL *ET AL*, (2013)

Isa *et al*, (2018) stated that Calcium Carbide Waste (CCW) can be used as an alternative to traditional Portland cement mineral filler in hot mix asphalt concrete to rid its disposal problem at a percentage of 40% replacement. Positive stability results were obtained with the replacement of Portland cement with CCW because they were above the Nigerian General Specification of roads and Bridges (1997) standard of not less than 3.5 kN as shown in figure 2

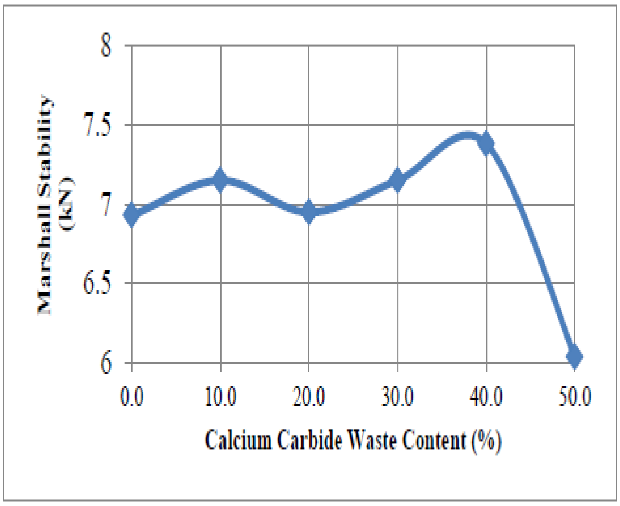

FIGURE 2: VARIATION OF CALIFORNIA BEARING RATIO WITH LIME/CCW CONTENT. SOURCE: ISA *ET AL*, (2018)

4 CONCLUSION

As shown earlier, carbide waste form an appreciable proportion of solid municipal waste with its attendant disposal problem. It has been shown that in one form or another , it can be used as a pavement material. The values for the strength as a base or surface material in pavement are within acceptable limits, with or without additives. In addition to strength consideration in the resulting pavement component, the grain size distribution was also found to be of acceptable level. A large quantity of carbide waste can be eliminated from our environment by its use in pavement construction which will result in cleaner and healthier environment.

5 RECOMMENDATIONS

i. Government should put in place a separate carbide waste disposal method to enhance its easy collection from automobile workshops.
ii. Designers should take advantage of these research findings to incorporate carbide waste in pavement design materials especially for low traffic roads.
iii. Further work may be encouraged to assess the performance characteristics of carbide waste modified concrete pavement blocks.

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EVALUATION OF COMPACTION CHARACTERISTICS OF IRON ORE TAILINGS TREATED WITH BENTONITE

***Okoro, I. J.[1]; Amadi, A. A.[2]; Alhassan, M[3] & Agbese, E.[4]**
[1,2,3,4] Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: isiuwaokoro@gmail.com

ABSTRACT

This paper reports the evaluation of compaction parameters (optimum moisture content and maximum dry density) of Iron Ore Tailings mixed with bentonite (IOT-bentonite mixtures) as a potential material for landfill liner purposes. Different percentages of bentonite (20, 25, 30, 35 and 40%) by dry weight of IOT were added to IOT and the mixtures evaluated in this study. Laboratory tests to evaluate the index properties and compaction characteristics of the mixtures were conducted. The index properties of the mixtures were also seen to improve with increase in bentonite content. The liquid limits of the mixtures containing 25 to 40% were seen to increase from 31 to 34.5% while their plasticity index also increased from 12.1 to 19.8%. These values satisfy the liquid limit and plasticity index requirements for landfill liners as reported in some regulatory codes. The mixtures were compacted using three compactive efforts namely British Standard Light (BSL), West African Standard (WAS) and British Standard Heavy (BSH) which simulates compaction efforts expected in the field. Findings show an increasing trend in the Optimum Moisture Content (OMC) and decreasing trend in the Maximum Dry Density (MDD) of the mixtures with increase in bentonite content. The OMC increased from 13.0 to 16.9% for BSL, 11.0 to 12.8% for WAS and 8.8 to 10.1% for BSH. While their MDDs decreased from 2.34 to 2.14 g/cm^3 for BSL, 2.47 to 2.38g/cm^3 for WAS and 2.5 to 2.42g/cm^3 for BSH..

Keywords: *Bentonite, Compaction characteristics, Iron ore tailings, Landfill liner.*

1 INTRODUCTION

Iron ore tailing (IOT) is the mining waste left over from the iron ore industry after the process of separating the valuable fraction from the worthless fraction of an ore. Increase in mining activities in the world has brought about tremendous generation of tailings which causes environmental pollution and also require a large portion of land for disposal. The composition of tailings is directly dependent on the composition of the ore and the process of mineral extraction used on the Ore (Adedeji and Sale, 1984).

Recent research works have been geared towards effective utilization of iron ore tailings as a partial replacement of other materials in construction and engineering works such as, landfilling, brick production and concrete production. In concrete production, it has been used as a partial replacement for other materials by several authors. Results obtained met the required design and in some cases more economical compare to the conventional materials used (Shetty *et al*., 2014; Ugama *et al*., 2014; Bakulamba *et al*., 2015 and Tiwari *et al.*, 2017). IOT mixed with other materials for land fill liner have also been studied by a few authors with their compaction characteristics reported (Manjunatha and Sunil, 2013; Umar *et al*., 2015).

The use of IOT alone as landfill liner is not feasible as a result of its properties especially its permeability which is the primary function of a landfill liner. Therefore, other materials such as bentonite with low permeability can serve as suitable admixture to enhance and complement the IOT for landfill liner application.

Bentonite is a montmorillonite type of clay having relatively small pore spaces occupied by water which leads to reduction in hydraulic conductivity. Hence, it is preferred as blended soil and it is widely used for landfill liner due to its low permeability (Nithi *et al.*, 2017).

Many engineers in a bid to predict the hydraulic conductivity of land fill liner carry out compaction test to study the interpretation of moisture content-dry density relationship. The relationship between dry density, moisture content and hydraulic conductivity is correctly interpreted for specific soil type in order not to mislead the engineers by the scattered values and manner of presentation (Wright *et al.*, 1996). It is also important to determine the index properties of landfill liner material in order to ascertain its suitability for the purpose. The liquid limit of landfill liner material is required to be between 30 to 60% and the plasticity index should be between 12 to 30% (Daniel, 1993).

Even though IOT-bentonite mixture is to be used as landfill liner, compaction by mechanical means is required to modify and improve its engineering properties. The main objective of compaction is to increase the soil density and reduce the air voids between the soil particles. OMC and MDD are compaction parameters required to evaluate the compaction characteristics of the mixture. These parameters are also important in other construction projects such as road and railway embankments, earth dams and backfills of retaining structure.

Most compacted earthworks, different molding moisture content often result in different dry densities under the same compaction energy. A constant value of energy applied to a particular type of soil, at optimum moisture content, leads to a maximum dry density. The MDD and OMC are unique for various types of soils and vary with the type of soils and the compaction energy (Jesmani *et al*., 2008). According to Umar *et al.* (2016), hydraulic conductivity generally decreases with increase in compactive effort.

Results obtained from several authors in the use of soil mixed with bentonite as liner materials has shown that increase in bentonite content in their mixtures resulted to an increase in optimum moisture content (OMC) and consequently decreases the maximum dry densities (MDDs) of the mixtures (Akgun *et al*. 2006; Amadi and Eberemu, 2012; Vijayan *et al*. 2016; Jaskiran and Sanjay, 2017; Nithi *et al.* 2017).

Manjunatha and Sunil (2013) carried out a research using polymetallic iron ore tailings, with lime and fly ash added as candidate material. The specific gravity of the mixture with the highest fly ash content of 40% was seen to be the least. The result obtained from the compaction characteristics of the mixtures containing IOT and flyash showed that the MDD decreased as the OMC increased with increase in fly ash content of the mixtures and this was attributed to the low specific gravities of fly ash as an additive blended with the tailings.

Khalid and Mukri (2017) stated that MDD and OMC were affected by the compaction energy applied to soil sample whether the soil was mixed or not with addition of bentonite. It was seen from their study that the increment in percentages of bentonite resulted in reduction of MDD and a consequent increase in OMC.

Jaskiran and Sanjay (2017) in their research on compaction characteristics of sand-bentonite mixtures reported a decrease in MDD as the OMC increases with increase in bentonite content. It was also reported that decrease in MDD may be attributed to the decrease in specific gravity, while the increase in OMC to the higher clay content requiring more water to hydrate the soil particles in the mix.

Jesmani *et al*., (2008) reported from their study that an addition of clay content (beyond 20%) to soil leads to decrease in MDD. This decrease in MDD was said to be due to increase in fine grains which causes the coarse grains to be away from each other and this changes the soil state from semi-buoyant to buoyant. Also, the OMC was said to increase due to the increase in clay content of the mixtures, since generally for obtaining MDD by increasing clay content, larger optimum water content is needed.

From the foregoing, several authors have carried out research on the compaction characteristics of soil-bentonite mixtures to determine their compaction parameters as well as the use of IOT as partial replacement of other materials for construction purposes. To the authors' knowledge there is no information about the index properties and compaction characteristics of IOT-bentonite mixtures with IOT as the major material treated with bentonite.

This paper therefore presents the study of index properties and compaction characteristics of bentonite added at different percentages by dry weight of IOT to iron ore tailings in geo-environmental engineering. The effect of compactive efforts on the IOT-bentonite mixtures was also analysed.

Iron ore tailings (IOT): The IOT used in this study was locally sourced from the National Iron Ore Mining Company (NIOMCO), Itakpe in Ajaokuta, Kogi state, Nigeria. The Itakpe Iron Mining Project is a ferruginous quartzite deposit of geological reserve amounting to 200 million tonnes (Olubami and Potgieter, 2005; Elinwa and Maichibi, 2014; Umar *et al.*, 2015). The waste material from the processing was approximately 64% which translated into 3,072 tonnes per day (Ajaka, 2004). The mineralogical characterization of Itakpe iron Ore shows that it contains mainly hematite, magnetite and quartz (Adedeji & Sale, 1984; Olubami and Potgieter, 2005; Nwosu *et al*., 2013).

Bentonite: Bentonite used in this study was Na-bentonite and it is the same as those used by Amadi and Osinubi, (2017). It was locally sourced from commercial suppliers in Kaduna, Kaduna state, Nigeria.

2 METHODOLOGY

Sieve analysis was carried to determine the particle size distribution of the IOT, specific gravity tests, atterberg limit tests and compaction test of the IOT-bentonite mixtures were also carried out.

2.1 PARTICLE SIZE DISTRIBUTION ANALYSIS

The IOT was washed with water using sieve size of 0.75μm in order remove the fine portion of the sample after which the mass retained in the sieve was dried in the oven for 24 hours before carrying out the sieve analysis.

2.2 INDEX PROPERTIES

Index properties including consistency limits and specific gravities of the IOT-bentonite mixtures were determined in accordance with BS1377 standard procedures. The Atterberg limit test for determining the liquid limit was carried out using the cone penetrometer.

2.3 COMPACTION TEST

Pulverized air dried bentonite and iron ore talings was used to prepare the samples by adding 20, 25, 30, 35 and 40% bentonite by dry weight of IOT to IOT portion. The samples were first mixed in dried state; 3000g of each mixture was measured for the compaction test with water is added and mixing done simultaneously. Properties of the compaction test is summarised in Table I, with the different compaction energy produced from each compactive effort.

The five mixtures were compacted using British Standard Light (BSL), West African Standard (WAS) and British Standard Heavy (BSH) compactive efforts. The BSL (standard proctor) and BSH (modified standard proctor) compaction were carried out in accordance with the British Standards (BS1377: Part 4:1990), while the WAS was carried out in accordance with the Nigerian General Specification, (1997). A total of 15 samples were prepared for determination of moisture-density relationships of the mixtures.

Characteristics	Concentration
V_2O	0.02%
MnO	0.05%
Fe_2O_3	76.84%
pH	13.1%

Adapted from Umar *et al*., (2016).

TABLE 1 PROPERTIES OF COMPACTION

Type of compaction	Weight of rammer (kg)	Height of fall (m)	Number of blows	Number of layers	Compaction energy (KNm/m)
BSL	2.5	0.300	27	3	596.4
WAS	4.5	0.450	10	5	993.9
BSH	4.5	0.450	27	5	2683.7

The five mixtures were compacted using British Standard Light (BSL), West African Standard (WAS) and British Standard Heavy (BSH) compactive efforts. The BSL (standard proctor) and BSH (modified standard proctor) compaction were carried out in accordance with the British Standards (BS1377: Part 4:1990), while the WAS was carried out in accordance with the Nigerian General Specification, (1997). A total of 15 samples were prepared for determination of moisture-density relationships of the mixtures.

3 RESULTS AND DISCUSSION

The physio-chemical properties of IOT and bentonite are shown in Table 2 and 3. Their specific gravities were obtained as 3.1 and 2.34 respectively in accordance with BS 1377 and the specific gravities of the IOT-bentonite mixtures with bentonite in percentages of 20, 25, 30, 35 and 40 % gave values of 2.60, 2.67, 2.64, 2.62 and 2.55. This decrease in specific gravities was attributed to the increase in bentonite content as a result of its low specific graavity (Manjunatha and Sunil, 2013).

TABLE II: CHEMICAL CHARACTERISTICS OF IOT

Characteristics	Concentration
SiO_2	20.2%
K_2O	0.30%
CaO	0.53%
TiO_2	0.18%

TABLE III: CHEMICAL COMPOSITION OF BENTONITE

Chemical composition	Value (%)
SiO_2	58.14
Al_2O_3	21.73
Fe_2O_3	2.46
TiO_2	1.86
CaO	0.86
MgO	2.42
P_2O_5	0.119
Cr_2O_3	0.007
K_2O	0.52
Na_2O	2.08
MnO	0.07

Adapted from Amadi and Osinubi, (2017).

3.1 PARTICLE SIZE DISTRIBUTION

The result obtained from the sieve analysis shows that the IOT contains 98.8% sand and 0.6% gravel. The particle size distribution of IOT is shown in Figure 1.

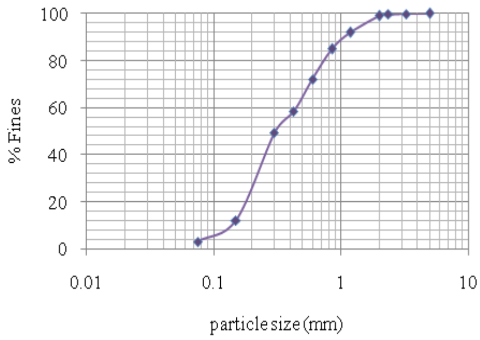

FIGURE 1: PARTICLE SIZE DISTRIBUTION CURVE (IOT)

3.2 INDEX PROPERTIES

The result obtained from the atterberg limit test shows that the liquid limit and plastic index increased with an increase in bentonite content. The liquid limit ranged from 27.0 to 34.5%, while the plastic index ranged from 8.5 to 19.48%. All other mixtures except that with 20% bentonite met the requirement of landfill liner according to Daniel, (1991) in the literature review. The liquid limit of mixtures containing 25 to 40% bentonite ranged from 31 to 34.5% and their plasticity index from 12.1 to 19.45%. Figure 2 shows the variation of consistency limits with bentonite content.

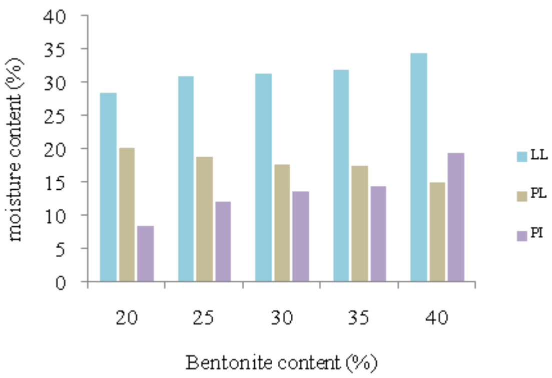

FIGURE 2: VARIATION OF CONSISTENCY LIMITS WITH BENTONITE CONTENT

3.3 COMPACTION CHARACTERISTICS

Figure 3 and 4 shows the effect of bentonite content on the MDD and OMC of the mixtures. It can be seen that the MDD decreased with higher bentonite content while the OMC increased with increase in bentonite content. The MDD ranged from 2.34 to 2.14g/cm^3 at 20 to 40% bentonite content for BSL, 2.47 to 2.38g/cm^3 % for WAS and 2.50 to 2.42 g/cm^3 for BSH. While the OMC ranged from 13 to 16.9% at 20 to 40 % bentonite content for BSL, 11.0 to 12.8% for WAS and 8.80 to 10.1% for BSH. This trend is similar to that found by Akgun *et al*. (2006); Jesmani *et al*. (2008); Vijayan *et al*. (2016); Amadi and Eberemu, (2012); Jaskiran and Sanjay, (2017) as well as Nithi *et al.* (2017). Jaskiran and Sanjay, (2017) explained that the decrease in MDD is attributed to the decrease in specific gravities of the mixtures as a result of increase in bentonite content which is also seen in the present study. The increase in fine grain fraction from bentonite led to a decrease in MDD, while the OMC of the mixtures increased with the increase in clay content as a result of more water needed to provide good lubrication. This is also similar to the report found by Jesmani *et al*., (2008).

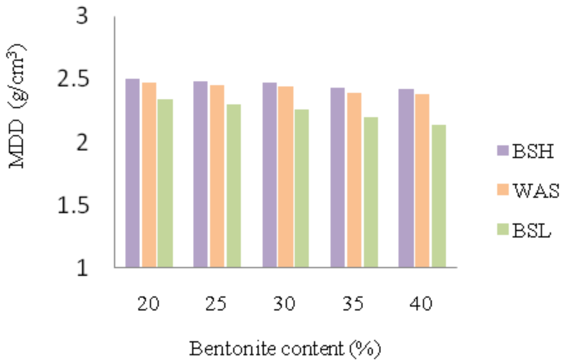

FIGURE 3: EFFECT OF BENTONITE CONTENT ON MDD OF MIXTURES.

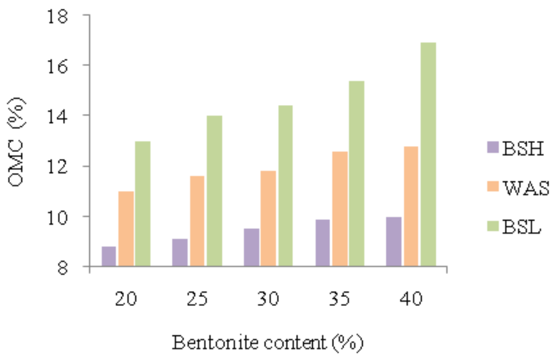

FIGURE 4: EFFECT OF BENTONITE CONTENT ON OMC OF MIXTURES

The effect of compaction energy on the MDD and OMC of the mixtures are presented in Figures 5 and 6. The result showed an increase in MDD and a decrease in OMC as compactive effort increased. The OMC ranged from 8.8 to 16.9% for the three compactive efforts used, while their MDD ranged from 2.14 to 2.50g/cm^3. This trend is consistent with the results reported in the literature review (Khalid and Mukri, 2017; Manjunatha and Sunil, 2013).

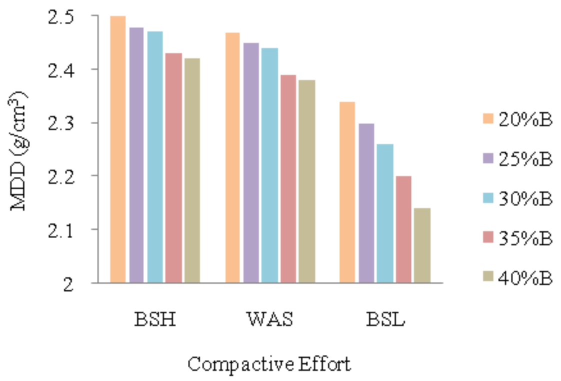

FIGURE 5: VARIATION OF MDD WITH THE DIFFERENT COMPACTIVE EFFORTS AT DIFFERENT PERCENTAGE OF BENTONITE CONTENT.

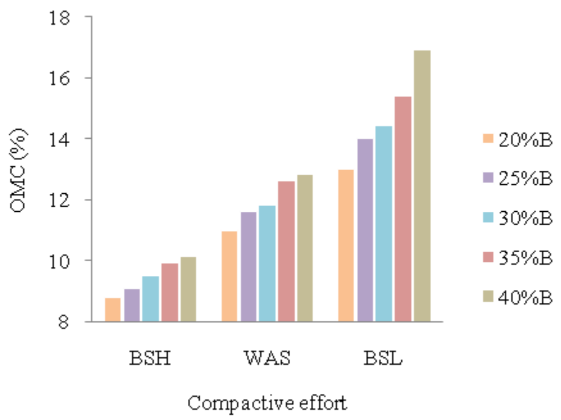

FIGURE 6: VARIATION OF MDD WITH THE DIFFERENT COMPACTIVE EFFORTS AT DIFFERENT PERCENTAGE OF BENTONITE CONTENT

4 CONCLUSION

The present study evaluated the compaction parameters of IOT treated with varying percentages of bentonite from the study, the following conclusions were drawn.

1. The liquid limit of IOT- Bentonite mixtures with 20 to 40% bentonite content is seen to increase gradually between 28.5 and 34.5%, while the plasticity index increases from 8.4 to 19.48%. All other mixtures except the mixture containing 20% bentonite content satisfied the requirements for landfill liner.
2. The specific gravities of the mixtures decrease with increase in bentonite content and this is attributed to the clay content found in bentonite.
3. With bentonite content of 20 to 40% by dry weight of IOT added as blending material to IOT, an increasing trend is found in the OMC while the MDD decreases. The increase in OMC was attributed to increase in bentonite content as more water is needed to lubricate the mixture. The MDD also decreased as a result of reduction in specific gravities with increase in bentonite content.
4. An increase in compactive effort increased the MDD while a decreasing trend was achieved for OMC. The decrease in MDD as a result of higher compactive efforts is said to give a lower hydraulic conductivity for landfill liner.

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BARRIERS TO THE ADOPTION OF BUILDING INFORMATION MODELING IN NIGERIAN CONSTRUCTION INDUSTRY

Kolo, D.N[1]., Tsado, T. Y[1]., Bala, A[1]., Adinoyi, S. A[1]., and Kolo, D.N[2].
[1]Department of Civil Engineering, Federal University of Technology, Minna.
[2]Physical Planning and Development Unit, Federal University of Technology, Minna.
Email: daniel.kolo@futminna.edu.ng / 08038255859

ABSTRACT

Building Information Modeling (BIM) involves the development and use of computer generated n-D models to simulate, plan, design, construct, adapt, operate, maintain, renovate, and ultimately beneficially deconstruct a building at the end of its life cycle. BIM represents a new paradigm in construction, it encourages the integration of roles of construction stakeholders enabling them to visualize the project to be built and further identifying potential issues that may occur during the operational phase of buildings. This study assessed the barriers to the adoption of BIM in the Nigerian construction industry and further highlighted ways to improve its adoption. A total of 50 questionnaires were administered to construction professionals in Abuja, Nigeria. Forty (40) retrieved questionnaires were analysed using Statistical Package for social Sciences (SPSS 21) and used for this study, it was revealed that there was generally a low awareness on the use of BIM among construction professionals. The major barrier to using BIM was lack of skilled personnel while the major means of ensuring its adoption was Provision of basic BIM infrastructure. These problems can be effectively tackled by increased support from government and construction industry stakeholders for its use; stressing the benefits derivable, training and retraining of key construction professionals taking into consideration peculiarities to the Nigerian construction industry.

Keywords: *Building Information Modeling, construction, Nigeria.*

1 INTRODUCTION

The construction industry in Nigeria is growing rapidly as a result of increasing population and demand for infrastructure. Recent performance studies have revealed an alarming rate of clients' dissatisfaction concerning time and cost overrun in construction projects. According to Mbachu and Nkado (2004) it is as a result of the use of inefficient procurement and project delivery, poor project management, incomplete documentation, discord between the design and construction teams and frequent design changes. This has led to the adoption of Building Information Modeling (BIM) by some countries around the world. However, the level of awareness and adoption of BIM in Nigeria's private, public sector and amongst different building professionals has been very slow despite the numerous benefits enjoyed by the various countries that have adopted it fully (Alufohai, 2012).

A BIM is a digital representation of the physical and functional characteristics of a facility, it represents a shared knowledge resource or a process of sharing vital information about a facility. This forms a reliable basis for taking decisions during the life cycle of the facility. Some definitions portray BIM as "Process", "Product", "Technology", "Innovation", or a "Strategy". It is however a digital representation of the physical and functional characteristics of a facility, its major goal is to produce complete and detailed model of a facility in a digital environment with the sole aim of providing a collaborative platform through which Building information can be managed throughout the facility's life cycle (Ibrahim and Abdullahi, 2016). The promotion of BIM's adoption among key construction industry stakeholders in Nigeria by increasing public awareness on the techniques, the tools employed and the benefits associated with its use becomes timely and justifiable. Hence, this study is conducted to investigate the barriers to the successful implementation of BIM and suggest ways of promoting its adoption within Nigerian Building Construction Industry.

Building Information Modeling (BIM) involves the use of computer-generated models to simulate planning, design and construction of projects. According to Autodesk (1999) "Building information modeling is a design methodology that maintains a single database of information about a building design. All information for a building design, from geometry to construction data is stored in a project file. This information includes components used to design the model, views of the project, drawings of the design and related documentation". Internationally, the building industry is transforming rapidly with the introduction of BIM. It is changing the process of design and construction of buildings.

Globally BIM usage is transforming the construction industry, it is changing the process of design and construction of buildings. Onungwa and Uduma-Olugu (2016) opined that BIM entails a seven dimensional process: the 3D modeling process extends to scheduling and sequencing (4D), cost estimating (5D), sustainable design also termed Green Design (6D) and facility management (7D). It explores the benefits, costs, risks and rewards associated with BIM, interoperability and integration.

In order to understand the benefits of BIM to the construction industry, we must explore some of the global benefits. Globally one of the great advantages of BIM is its ability to create an accurate model which is useful throughout the entire life of the building, from initial design through occupancy and operation. A BIM ideally is created in the early stages of the design and updated as the design is refined and used by the construction team, it is refined continuously during the course of constructing the facility. Post-occupancy, the BIM is used by the owner and owner's maintenance team to improve understanding of the facility operation and to make adaptations, renovations, additions and alterations to the building faster and for less cost than through the conventional traditional processes (Conover *et al .*, 2009).

The power of BIM is in its ability to allow the entire building to be optimized in lieu of optimizing individual components. Each discipline and trade benefits through integration and optimization within a BIM and becomes more efficient by providing parametric responses to single discipline changes through the use of consistent data sets for calculation and decision making. The work of the Heating, ventilation and air conditioning (HVAC) industry has an impact on every other design and construction discipline and trade including the following: architecture, electrical engineering, lighting design, roof and envelope consultation, fire protection, civil engineering, structural engineering, security consultants, acoustical engineering and others. BIM can benefit these associated and complimentary disciplines and trades through precise interdisciplinary coordination using parametric geometric modeling (Conover *et al.*, 2009).

According to Conover *et al.* (2009) the benefits of BIM are evident in its ability to capture, organise, integrate, maintain and grow the vast amount of knowledge, data and information required to conceive, plan, design, construct, adapt, operate, maintain, renovate, maintain and, ultimately, beneficially deconstruct a building at the end of its life cycle.

BIM enabled Software's

According to Conover *et al.* (2009) commercially available application software's are presented on Table 1. However, the list is not an exhaustive one because the concept of BIM is still evolving as such improvements and development keeps occurring. The choice of the best software to use is dependent on several criteria ranging from production practices, interoperability, functional capability of the design organization to undertake particular types of projects (Ibrahim and Abdullahi, 2016).

TABLE 1: COMMERCIALLY AVAILABLE BIM SOFTWARE'S (CONOVER *ET AL.*, 2009)

Organization	Product
Autodesk Inc	Revit MEP AutoCAD MEP Auto desk Navis Works Manage Auto desk Green Building Studio Auto desk Ecotect Auto desk Buzzsaw Auto desk Constructware
Bentley Solutions	Bentley Architecture MicroStation Auto Pipe AutoPlant Bentley building Mechanical systems Hevacomp simulator V8i Hevacomp M&E Designer V8i Bentley Tas Ambeins CFD Bentley Tas Simulator V8i Bentley Building Electrical systems
Bentley Systems	Architecture Structural Civil Mechanical Electrical HVAC Instrumentation and Wiring Geospatial (GIS) and Facilities
Graphisoft	ArchiCAD 11
Granlund	RIUSKA Integrated Building Solutions
CADPIPE	ArTrA BIM
Wrightsoft	Right-Suit Universal

Global adoption of BIM and Implementation

The concept of BIM and its implementation is on the rise globally, many countries have realized this procedural and technological evolution in the construction industry (Gerges *et al*., 2017). In the USA, UK and some other developed countries, BIM has become mandatory. It is a criteria for evaluation in the industry. European countries like Finland, Denmark, Norway and Sweden are the leaders in BIM implementation (Arayici, 2012). According to Chan (2014) BIM is still in its early stages of implementation in Hong Kong although moving rapidly. The clients are gradually getting to discover the benefits and advantages of BIM: they can conduct different tests on BIM models, generate different design options and early detection of design faults in order to forestall changes or renovations in the future. Africa has witnessed a generally low acceptance and implementation of BIM, in South Africa the concept faces huge challenges attributed to personal inadequacies in terms of education, training and skills development, contractual issues and population growth. Nigeria however experiences a very low level of implementation of both process and technology due to some peculiarities to the country's construction industry (Ogwueleka, 2015).

2 METHODOLOGY

The research was conducted in Abuja, Nigeria's Federal Capital Territory; a well structured questionnaire was used in soliciting data from the respondent in this study. A total of 50 questionnaires were issued to construction professionals on the sites visited. 40 questionnaires representing 80% response rate were retrieved and analysed using Statistical Package for Social Sciences (SPSS) version 21. The response rate is deemed adequate according to Ahmadu (2014) who opined that a response rate of 30 was adequate for construction industry studies. The mean scores of each factor were computed from the analysis of the ratings provided from a five point likert scale.

3 RESULTS AND DISCUSSION

Characteristics of Respondents

The characteristics of respondents were assessed under their profession, duration in construction and type of project executed. Table 1 shows that majority of the respondents were Engineers. Majority of the construction professionals had 1-5 years (52.5%) experience in the industry, followed closely by 6-10 (25%) years experience, 11-15 years (15%) and over 21 years (7.5%) experience. Information about the type of project executed by these respondents also revealed that they engaged mainly in all types of construction (62.5%), the remaining engaged in residential (27.5%), commercial (2.5%) and other (7.5%).

TABLE 2: PROFESSION OF RESPONDENTS

Profession	Frequency	Percent (%)
Architect	5	12.5
Engineer	12	30
Quantity Surveyor	6	15
Builder	4	10
Surveyors	6	15
Others	7	17.5
Total	**40**	**100**

Use of BIM software

33 (82.5%) of the respondents use BIM software in their practice while 7 (17.5%) indicated to have never used any BIM software in their practice.

Industry Clarity on the use of BIM

Since most of the stakeholders testified to be conversant with the available BIM software it is safe to assume that assume that the industry would be very clear on the concept of BIM however, result revealed the stakeholders Agreed (45.9%) the Nigerian construction industry is not yet clear on the workings and operation of BIM. 21.6% Strongly agreed, 16.2 % were neutral, while 10.8% and 5.4% Disagreed and Strongly Disagreed respectively that the industry was clear about BIM. This result shows that although many stakeholders use BIM software in their practice, nevertheless, the industry is not yet clear enough of what BIM actually is and how to maximize the components of BIM as a tool to ensure efficient project performance.

Impact of BIM on construction Performance

The mean score ranking was used to asses the impact of BIM on construction performance. Table 2 shows the mean scores and rankings of the identified construction performance assessment variables. From the Table it can be inferred from the relatively high mean scores that the listed factors have a significant impact on construction performance. BIM has a greater impact on construction programming ranking first with a mean score of 4.32 from the analysis of the data followed by collaboration with other consultants and Quality of Completed Jobs which ranked 2nd and 3rd respectively with mean scores of 4.18 and 4.03. Cost estimation ranked 4th while Project completion time ranked 5th. Job supervision, Safety and Energy efficiency ranked 6th, 7th and 8th respectively with mean scores of 3.49, 3.48 and 3.41 respectively. This shows that the respondents believe if efficiently utilized, BIM would improve all the listed performance criteria most importantly improving on construction programming activities.

TABLE 3: ASSESSMENT OF IMPACT OF BIM ON PROJECT PERFORMANCE

Construction Performance Variables	Mean Score	Rank
Construction programming	4.23	1
Supervision of Jobs	3.49	6
Quality of Completed Jobs	4.03	3
Energy Efficiency	3.41	8
Project completion time	3.87	5
Collaboration with other Consultants	4.18	2
Cost Estimation	3.88	4
Safety	3.48	7

Barriers to the successful adoption of BIM in Nigerian construction industry

The mean score ranking was adopted in analysing the barriers to the successful implementation of BIM in the construction industry. As seen from Table 4, Lack of Skilled personnel emerged first with mean score of 4.11. Poor awareness of technology emerged 2nd while Inadequate design process was 3rd with mean scores of 3.79 and 3.16 respectively. Lack of BIM object libraries, Fear of Change, inadequate planning and budgetary all tied in the 4th rank with mean score values of 3.05, 3.05, 3.05 and 3.05 respectively. These were closely followed by Cost, Power Failure, Stakeholders reluctance to use BIM, Lack of contractual documents on BIM and Poor internet connectivity with mean scores of 3.08, 3.00, 2.92, 2.85 and 2.68 respectively.

TABLE 4: BARRIERS TO ADOPTION OF BIM

Perceived Barrier	Mean Score	Rank
Lack of Skilled Personnel	4.11	1
Poor Internet Connectivity	2.68	12
Stakeholders reluctance to use BIM	2.92	10
Lack of BIM Object Libraries	3.05	4
Poor Technological Awareness	3.79	2
Cost	3.08	8
Power Failure	3.00	9
Lack of Contractual BIM documents	2.85	11
Fear of Change	3.05	4
Inadequate design process	3.16	3
Fragmented nature of Construction industry	3.05	4
Inadequate planning and Budgetary provisions	3.05	4

Solutions to improve BIM adoption in Nigerian Construction Industry

Table 5 gives a summary of perceived solutions to the implementation of BIM in the Nigerian Construction industry. Provision of Basic BIM infrastructure was perceived as the most effective measure to improve its adoption, ranking 1st with a mean score of 1.84. The government and industry stakeholders should key into this and provide the necessary infrastructure for BIM implementation. Sensitization and Adaptation to change ranked 2nd with a score of 1.64 followed by research on construction methods that will encourage the use of BIM, Seminars, lectures, demonstrations on the use of BIM and Training and awareness with mean scores of 1.63, 1.58 and 1.55 respectively.

TABLE 5: PERCEIVED SOLUTIONS TO BIM ADOPTION

Perceived Solution	Mean	Rank
Research on Construction methods that will encourage the use of BIM	1.63	3
Seminars, lectures, demonstrations on the use of BIM	1.58	4
Provision of basic Infrastructure	1.84	1
Training and awareness	1.55	5
Sensitization and adaptation to change	1.64	2

4. CONCLUSION

The concept of BIM is clearly gaining momentum as it evolves with the possibility of better interoperability between the various software systems. It is an expansive domain of knowledge in the design and construction industry, useful to both the Architects and Construction Engineers. It is slowly gaining acceptance in the Nigerian construction industry, based on the result of this study it can be concluded that there is generally low awareness of the concept of BIM as it is still evolving and gaining grounds in Nigeria. The major barrier identified as hindering its adoption is lack of available skilled personnel. As such the duty of training construction professionals by the government and key industry stakeholders on the operation and benefits of the various commercially available BIM software becomes timely. The major way to enhance its adoption is the Provision of basic infrastructure for practicing construction professionals. The future of the design and construction industry lies in the utilization of technology and BIM is expected to shape this effectively.

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Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

EXAMINING THE IMPACTS OF INFORMATION COMMUNICATION TECHNOLOGY ON QUANTITY SURVEYORS IN ABUJA, FCT

* BODE-BADAKI, O[1]; SAIDU, I.[2]
[1] Department of Quantity Surveying, Federal University of Technology, Minna
[2] Department of Quantity Surveying, Federal University of Technology, Minna
*Corresponding author email: [1]femibadaki@gmail.com, [2]ibr,saidu@futminna.edu.ng

ABSTRACT

The research work examined the overall impacts of Information communication technology on Quantity Surveyors residents in the Federal Capital Territory, Abuja. The research was carried out using a mixed method approach (quantitative and qualitative). The research population for the quantitative data was the 569 registered in the professional regulatory body (QSRBN) register. Out of which 230 as the sample size was gotten using Glenn (2013) formulae. While the population for the qualitative data were the principal Quantity Surveyors in the construction industry. Using a stratified random sampling method, the collected data from the questionnaire were analysed using Relative Importance Index (RII) and Ranking method and the qualitative analysis was done using thematic and deductive analysis. The research found that the benefits of ICT to Quantity Surveyors are speedy exchange of information, providing an easy access to needed data, faster response to clients enquires, increase productivity through automated quantities and cost calculations and improving job presentation while the following were the negative effects on Quantity Surveyors: eye strain, neck pain, watery and dry eyes, blurred vision and headache Body pains, job distraction, headaches, and feelings of been overwhelmed from work overloads, the threat to project confidentiality, mistakes by the younger or inexperienced quantity surveyors. Keywords: *Effects of ICT on Quantity Surveyors, Information Communication Technology in the Construction industry, Impact of Information Communication Technology on Quantity*.

1 INTRODUCTION

The construction industry may be viewed as that sector of the economy that, through planning, design, construction, maintenance and repair, and operation, transforms various resources into constructed facilities (Ezeokoli *et al.,* 2016). The industry contributes to the economic development of any country, in Nigeria for instance; at the fourth quarter of 2016, it contributed about 3.41% to the total real GDP (National Bureau of Statistics, 2017). A construction process relies on the collaborative participation and contributions of different interdependent parties hence the great need to communicate. This makes the industry one of the most information-intensive industries (Chidebere, 2012).

The growth of the Information and Communication Technology (ICT) has had unquantifiable impact on business systems and processes, with the construction industry inclusive; this is showed by the yearly increase of ICT users (Oyediran & Odusami 2005, Akinnagbe & Adelakun 2014). The need to increase staff productivity, gaining competitive advantage, reducing the burden of data processing, storage and dissemination as highlighted by Ezeokoli *et al.* (2016), as led to the gross acceptance of ICT in the industry.

Monideepa *et al.* (2014) noted that the implementation of ICT leads to effects that have a "dual nature", the benefits on a side and negative reactions on the other side. The benefits were indicated to be enhancing productivity, improving accuracy and documentation, enhancing effective delivery and improving time, cost and quality performance (Gambo, 2017). The negative reactions inferred from studies could be grouped into psychological, emotional, health and social. Thomee (2012) also noted decrease in learning, creativity, and critical thinking (Gerardi, 2017). Social: Personal invasion, uncertainties, information overload (Bucher *et al.,* 2012).

2 STATEMENT OF PROBLEM

The rise in the use of ICT in the workplace has undoubtedly produce favorable results as shown by several studies, but at the same time, there come challenges with the shift in paradigm because of lack of a proper adoption strategy. Studies revealing these side effects are emerging in other fields of human studies notably; health, social science, the communication profession, human and personal management, and administrations, with the construction industry not left out.

Tagurum *et al.* (2017) noted that there is a growing perception that rapid advancement in technology are responsible for inducing stress in our lives, thereby affecting performance and wellbeing. Akinbinu and Mashalla (2014) also observed that an increase in dependence on computers in the developed has led to the prevalent of Computer vision syndrome. Thomee (2012) highlighted that intensive ICT use has impacts on the mental health of its users, reporting sleep disturbance and symptoms of depression. Paul (2011) remarks the summation of these effects as the human brain under a serious threat from the modern world.

Rimmington *et al.* (2015) studied the effects of ICT on the social environment of construction project teams and the project outcome, and confirmed the existence of tensions and conflicts in the human-electronic and human-human communication, the eroding of the social skills in the work environment, the decrease in the social interaction among team members required to increase team performance, the loss of understanding of information passed due to the speedy and spontaneous nature of ICT tools and lastly the tendencies to be distracted from the job at hand, proposing that the increasing use of ICT will occur at the expanse of soft system communication. The aim of this research therefore is to examine the overall impacts of ICT on Quantity Surveyors.

3 JUSTIFICATION

Several studies have been concerted on the adoption of information and technology tools, its impacts/benefits, and barriers to its full implementation; they include surveys conducted in New Zealand in 1997 (Doherty, 1997), Scandinavia in 1998 (Howard *et al.*, 1998), Canada in 2003 (Rivard *et al.*, 2004), Australia in 2005 and 2009 (Peansupap and Walker 2005, Brewer and Gajendran, 2009), in Construction Small and Medium enterprise in ,developing countries in 2005 (Lee and Egbu 2007), Turkey in 2009 (Isikdag *et al.*, 2009), Brazil in 2010 (Michaloski and Paula 2010), Sweden in 2007 (Samuelson, 2008), Malaysia in 2011 (Kareem and Bakar 2011), Singapore in 2003 (Hua, 2004), Taiwan in 2008 (Chien and Barthorpe 2010). All of these studies observed the high and increasing usage of ICT as against perceptions that the construction industry is slow to the ICT revolution, although they noted that the ICT was majorly used for accounting, documentation and information transmission, and not necessarily for professional practice, though the use of CADs and 3D is prevalent in the industry.

Summarily, studies on ICT carried out in the Nigeria construction industry, has largely been focused on assessment, implementations and innovations of ICT usages and also the impediments or reasons in the approach towards a full adoption of ICT in work practice, little however exists on the overall impacts on Quantity Surveyors, this indicate a gap in knowledge.

Significance of Filling the Gap

This research creates the awareness and provides understanding and guidance on the impacts ICT has on Quantity surveyors with a view to encourage the use and mitigate the negative effects, it will also improve the performance and skills of ICT users in the industry.The recommendations of the study, if properly implemented, would benefits the Quantity Surveying profession and the construction industry at large.

4 RESEARCH METHODOLOGY

A survey design approach was employed in this research with the quantitative data gathered from the respondents using a questionnaire and the qualitative data through a semi-structure interview. The population for this research work was the 569 registered quantity surveyors residents in the Federal Capital Territory, Abuja. This number was arrived at through a formal online consultation from the Quantity Surveying Registration Board of Nigeria (QSRBN). The sample frame consisted of Registered Quantity surveying firms, Government Ministries and department of works, registered construction firms and consultant construction firms in Abuja.

Abuja was chosen as a case study in this context because it is the capital city of Nigeria, and with a high presence of construction activities.

In order to guarantee equal representation for each of the identified groups in the population, stratified random sampling method was adopted. The respondents were first categorised into different strata then randomly sampled.

Using Glenn (2013) formulae, a sample size of 230 quantity surveyors respondents was addressed based on the figure calculated using Glenn's (2013) formulae.

$$\frac{N}{1+Ne^2}$$

Where:

n = Sample size

N = Population size in the sample unit

e = Level of precision which is +5% (0.05)

Table 1 shows the that 27% of the quantity surveyors worked in the Federal ministries, 18% worked in the various departments of works, 20% in private construction firms, 16% in Quantity Surveying firms, 19% private consultant construction firms under the questionnaire section, while the interview aspect showed that 20% of the quantity surveyors worked in the Federal ministries, 20% worked in the departments of works, 20% in private construction firms, 20% in Quantity Surveying firms, 20% private consultant construction firms.

TABLE 1.0: QUESTIONNAIRE RESPONDENTS' DEMOGRAPHIC

Respondents Place of Work	Questionnaire	Interview
Federal Ministries	55	2
Department of works	36	2
Private construction firms	40	2
Quantity Surveying firms	33	2
Private Construction consultant firms	38	2
Total	**202**	**10**

Table 2 shows that 26% of the quantity surveyors had 1-5 years working experience, 27% had 5-10 Years' experience, 20% had 10-15 years in experience and 27% had 15 years and above under the questionnaire section. While the interview section revealed that 40% of the quantity surveyors had 10-15 years working experience and 60% had 15Years' and above experience.

TABLE 2.0 YEAR OF EXPERIENCE

Years of experience	Questionnaire	Interview
1-5 years	26%	
5-10 years	27%	
10-15 years	20%	40%
15 years & above	27%	60%

Table 3 shows that 19% of the quantity surveyors had 1-5 years working experience with ICT, 43% had 5-10 Years' of experience, 26% had 10-15 years in experience and 12% had 15 years and above under the questionnaire section, while the interviewed respondents' had 5- 10 years working experience with ICT, 60% had 10-15 Years' of experience and 10% had 15 years and above.

TABLE 3.0 YEAR OF ICT EXPERIENCE

Years of ICT experience	Questionnaire	Interview
1-5 years	19%	
5-10 years	43%	30%
10-15 years	26%	60%
15 years & above	12%	10%

4 RESULTS AND DISCUSSION

QUESTIONNAIRE ANALYSIS

Positive impacts of ICT on Quantity Surveyors

All of the listed positive impacts were highlighted as having a strong positive effect on the performance of Quantity Surveyors. Although Speedy exchange of information was ranked first with the RII of 0.903, followed by provides an easy access to needed data (0.892), faster response to client's enquires (0.891), increase productivity through automated quantities and cost calculations (0.891) and the least was increasing project value (0.810).

TABLE 4.0 POSITIVE IMPACTS OF ICT ON QUANTITY SURVEYORS

S/N	Positive Impacts of ICT	RII	RANK
1	Speedy exchange of information	0.903	1
2	Provides an easy access to needed data	0.892	2
3	Faster response to client's enquires	0.891	3
4	Increase productivity through automated quantities and cost calculations	0.891	4
5	Improves job presentations	0.885	5
6	Effective collaboration and communication between project team members	0.883	6
7	Enhances innovation in work practices	0.874	7
8	Effective material procurement management	0.822	20
9	Effective decision making	0.821	21
10	Improve forecasting and control	0.814	22
11	Improving Client's satisfaction	0.813	23
12	Effective material management	0.812	24
13	Increasing project value	0.810	25

Source: Researcher's own field survey (2018).

Negative effects of ICT experienced by Quantity Surveyors

Eye strain was ranked was the most prevalent negative effect experienced with the RII of 0.810. Neck pain, Watery eyes, dry eyes, blurred vision, headache, stress induced from loss of information due to virus attacks, ICT addictions, sleep disturbance, less time spent with the family, double vision, muscle weakness, wrist pains, information overloads were all ranked as negative effects often experienced with the respective RII of 0.746, 0.741, 0.740, 0.729 and 0.708. The following were indicated as negative effects experienced sometimes; burn out syndrome (0.586), occasional clumsiness (0.596), work overload (0.580), invasion of personal/ privacy time (0.576), numbness (0.553), feelings of being overwhelmed with tasks (0.551) Job distractions (0.531), making professionals redundant (0.527), weak grip (0.519), reducing professionals interpersonal skills (0.513), anxiety (0.484), tendency to drop things(0.484), feelings of job uncertainty (0.481), eroding proper communication skills (0.480), frustration (0.479) and the least ranked was dissatisfaction with job (0.437).

TABLE 5.0 NEGATIVE EFFECTS OF ICT EXPERIENCED BY QUANTITY SURVEYORS

S/N	Negative effects of ICT	RII	RANK
1	Eye strain	0.810	1
2	Neck pain	0.746	2
3	Watery eyes, dry eyes,	0.741	3
4	Blurred vision	0.740	4
5	Headache	0.729	5
6	Stress induced from loss of information due to virus attacks	0.708	6
7	ICT addictions	0.689	7
8	Sleep disturbance	0.649	8
9	Less time spent with the family	0.648	9
10	Double vision	0.647	10
11	Burning sensation	0.633	11
12	Muscle weakness	0.630	12
13	Wrist pains	0.625	13
14	Information Overload	0.610	14
15	Burn out syndrome	0.586	15
16	Occasional clumsiness	0.596	16
16	Work overload	0.580	17
17	Invasion of personal/privacy time	0.576	18
18	Numbness	0.553	19
19	Feelings of being overwhelmed with tasks	0.551	20
20	Job distractions	0.531	21
21	Making professionals redundant	0.527	22
22	Weak grip	0.519	23
23	Reducing professionals Interpersonal skills	0.513	24
24	Anxiety	0.484	25
26	Tendency to drop things	0.484	26
27	Feelings of job uncertainty	0.481	27
28	Eroding proper communication skills	0.480	28
29	Frustration	0.479	29
30	Dissatisfaction with job	0.437	30

Source: Researcher's field survey (2018).

INTERVIEW ANALYSIS

According to all the interviewed respondents (PQS 1-10), the introduction of ICT into the construction and profession is a welcomed development, paramount to the sustenance and growth of the profession. Specifically PQS-01 iterated that ICT is a good innovation to the profession, makes the work professional, and brings out the best. PQS-06 particularly reinforced that ICT is the lifeblood of any private Quantity Surveying firm. "*One of the best things that have happened to the profession*" says PQS-3

Positive benefits of ICT in the profession

All the interviewed respondents reported that ICT has increased the speed of doing works, thereby saving time and costs and making work easier. PQS-4 particularly is of opinion that ICT reduces errors, makes knowledge sharing faster. PQS-7 also opined that it reduces corruption while making work appear neat. PQS-8 stated that it gives quick analysis to trending issues in the construction industry. PQS-10 reported that it creates an easy access to contractors and consultants, while reducing paper wastages.

Experienced negative effects of ICT in the profession

All of the interviewed respondents with the exception of PQS-7 attested to experiencing and observing negative effects of ICT. PQS-1 states body pains, job distraction, headaches, and feelings of been overwhelmed from work overloads. PQS-2 says " *Our profession deals with lots of confidentiality and secrecy, the use of ICT in the* *profession has made keeping secrets difficult, even when the files are deleted, someone with the know-how can still retrieve it*" this align with what PQS-6's observation. PQS-3 complained of the younger or inexperienced quantity surveyors been prone to mistakes "*Computer cannot do everything, there are some specific areas software cannot handle, younger quantity surveyors don't know this, and are quick to believe the computer is the answer to everything*" PQS-4 noted that some of these software are not user friendly, the illiteracy level is low among professionals, getting trainers to train is hard and expensive PQS-8 also stated the same problem of getting experts to train the younger quantity surveyors. PQS-5 complained of the pain of losing important files due to system break down due to virus attack, "*there was a time my system didn't work, all my files, bills of quantities were lost, till date I haven't been able to get some back, it's really painful*" furthermore it was observed that the workload is increasing, bulky jobs are expected to be done faster while also rendering some worker jobless "*What used to take 5 workers to do, is now been done by 1 person, the rest just sit and look*". PQS-9 berated the lack of training affecting the output of workers using ICT, also adding the cost implications of acquiring and train is burdensome. PQS-10 stated the problem of documents failing to open and the pain of re starting the whole work over again, also adding the security risk of hacking the documents.

5 CONCLUSION

The research examined the impacts of ICT on Quantity Surveyors. The specific objectives included: to examine the impacts: positive and negative. The research concludes that speedy exchange of information, providing an easy access to needed data, faster response to clients enquires, increase productivity through automated quantities and cost calculations and improving job presentation are the major benefits of ICT to a Quantity while eye strain, neck pain, watery and dry eyes, blurred vision and headache Body pains, job distraction, headaches, and feelings of been overwhelmed from work overloads, the threat to project confidentiality, mistakes by the younger or inexperienced quantity surveyors, software not user friendly, the illiteracy level among professionals, the pain of losing important files due to system break down due to virus attack, increase in workload, bulky jobs are expected to be done faster while also rendering some worker jobless, not forgetting to add the cost implications of acquiring and train is burdensome are the negative effects experienced.

6 RECOMMENDATIONS

i. Quantity surveyors are to endeavor to train themselves and learn relevant software tools related to the practice.
ii. The younger professionals are to be vast in relevant all ICT tools and to also know all the software, particularly, more research work has to be done.
iii. All quantity surveyors should be up to date with the new technologies.
iv. The Cost of ICT equipment procurement should be reduced and subsidized by the company, government and the institute.
v. Trainings and retraining should be encouraged by the professional institution and organizations.
vi. A central backup and database should be created and maintained to ensure data are not lost due to virus attacks or system breakdown
vii. Adequate power supply should be provided by the management and the government as most ICT tools depend on power
viii. Organizations, firms, and government should teach ergonomics (the proper and health way to use computers) to employees in order to safe guard the health of the employees who use these tools
ix. Original and updated softwares should be installed to ensure smooth work

Recommendations to the QS Professional Bodies (NIQS and QSRBN)

The Nigerian Institute of Quantity surveyors (NIQS National bodies (QSRBN and NIQS) should champion the campaign for planning, regulating and implementing a world class standard educational framework including effective teaching strategies such as softwares knowledge, adequate teaching and learning facilities for outstanding performance and relevance in the construction industry and the world at large.

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CEMENT STABILIZATION OF BLACK COTTON SOIL USING RICE HUSKASH AND PROMOTER

*A.I. Dogo[1], M. M. Alhaji[1], T.W. E. Adejumo[1], J.I. Aguwa[1]

[1]Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author: aydogo2000@gmail.com

ABSTRACT

Black cotton soils exhibit high swelling and shrinkage on the absorption or depletion of moisture and are the main cause of the many problems such as pavement failure, foundation cracks, and excessive settlement associated with the soil. Although poor and undesirable for engineering purposes, black cotton soil could be improved to meet engineering specifications by stabilization processes. The aim of this paper is to evaluate the effect of RHA and promoter on the compressive strength of black cotton soil. Natural black cotton soil was stabilized with 0% cement, 0% RHA, 0% promoter; 0% cement, 1% RHA, 0.3% promoter; 0% cement; 2% RHA, 0.6% promoter; 0% cement, 3% RHA, 1% promoter continuously up to 1% and 2% cement replacement and cured for a period of 1, 7, 14, 28, and 60 days. To evaluate the strength of the natural soil, various tests were carried out in the laboratory such as moisture content, sieve analysis, Atterberg limit, specific gravity and compaction (OMC and MDD).
The experimental results showed that unconfined compressive strength (UCS) values increased with higher compactive effort, RHA and promoter content. The UCS increased from 340KN/m2 to a maximum strength value of 582kN/m2 at 3% cement, 3% RHA and 1% promoter for specimens compacted at the West Africa Standard energy level at 60 days curing period, which implies that stabilization of black cotton soil can be improved with increased amount of RHA and promoter.

Keywords: Black Cotton Soil, Promoter, Rice Husk Ash, Stabilization, Unconfined Compressive Strength.

1.0 INTRODUCTION

Black cotton soils are expansive and inorganic clays of medium to high compressibility characterized by high shrinkage and swelling properties which form a major soil group in Nigeria (Adeniji, 1999). They exist majorly in arid and semi-arid regions of the tropical/temperate zones marked with dry and wet seasons; and with low rainfall, poor drainage and exceedingly great heat. The name black cotton is derived from the fact that cotton plant thrives on it.

In Nigeria, black cotton soils are vastly available in the North-Eastern part lying within the Lake Chad Basin and the upper Benue trough which covers a wide area extending North-East of the Jos Plateau. They are problematic soils (Adesunloye, 1987).

a. Dry state b. Wet state
PLATE I: BLACK COTTON SOIL IN DIFFERENT FORMS

Black cotton soils are susceptible to detrimental moisture and appear firm in its dry state and demonstrate tremendous swelling when wet. Cracks measuring about 70 mm wide and over 1 m deep have been observed and may extend up to 3 m or more in cases of high deposits (Adeniji, 1999).

PLATE II: HORIZONTAL CRACK ON A PAVEMENT CONSTRUCTED ON BLACK COTTON SOIL

Black cotton soils are greyish to blackish in colour, possess pure clay particles with about 85-100% passing through sieve 75µm and are rich in montmorillonite which is responsible for their shrink-swell behaviour (Adesunloye, 1987). The high shrinkage and swelling of these soils are especially troublesome to pavement subgrades and has also caused a lot of problems for engineering works such as foundation cracks, severe structural damage, heaving and cracking of sidewalk (Osinubi, 1997). Although poor and undesirable for engineering applications, black cotton soil can be improved to meet standard specification by stabilization processes (Ola, 1983).

Soil stabilization is the improvement of the original soil properties to meet specific engineering requirements (Arora, 2011). Objectives of soil stabilization include: improvement of the strength of the soil and bearing capacity, reduction of compressibility and volume instability, decrease permeability and water absorption, and to increase the durability under varying moisture content. Soil stabilization can be mechanical or chemical. Mechanical stabilization is achieved by altering the properties of a soil and includes controlled grading of the coarse aggregate, fine aggregate, silt and clay correctly proportioned and fully compacted. Chemical stabilization depends on the chemical reaction between stabilizers such as cement, lime, bitumen, Rice Husk Ash or other agents and soil minerals to achieve the desired result. Brooks (2009) stabilized black cotton soil with rice and husk ash and fly ash and concluded that unconfined compressive strength increase by 97% with increase Rice Husk Ash.

For decades, the stabilization of soil with cement, lime and bitumen have successfully been experimented and used extensively in the USA, Europe, India and Africa. However, chemical conditions of black cotton soil inhibits the normal hardening of cement, lead to loss of durability and high construction cost, while bitumen and lime have significant effect on vegetation and the environment (IRC, 1973). This phenomenon has led to the growing research on the use of cheap and readily available industrial and agricultural waste materials such as rice husk ash and promoter for the stabilization of soils.

In Nigeria, about 2 million tons of rice is produced while about 96, 600 tons was produced in Niger State in the year 2000 alone (Oyetola and Abdullahi, 2006). The rice husk generated does not degrade easily and their disposal has had a devastating effect on the natural environment. Rice Husk Ash (RHA) is an agro-waste obtained from burning the protecting outer cover of rice husk which is about one-fifth (20%) by weight of the harvested rice. It contains about 50% cellulose, 25-30% lignin, and 85-90% of silica which makes it a superb replacement of silica because they contain lime-pozzolana particles (Aparna, 2014). The high angularity and friction angle (up to 530) of RHA contribute to excellent stability and load bearing capacity (Raj, 2016). When rice husk is burnt, rice husk ash is generated. Rice husk ash when used for cement-based stabilization increases calcium silicate hydroxide and decreases calcium hydroxide which leads to high strength, reduced efflorescence, reduced sulphate and chemical attacks. Promoters are chemical additives that speed up the rate of chemical reaction in the stabilization process and lead to improvement of mechanical strength. In other to improve the strength, durability and other engineering properties of soil-cement, small quantities of promoter have always been used. The addition of 1% to 4% by weight of hydroxides and various salts would greatly increase compressive strength (Lambe and Moh 1958). It is worthy to note that most promoters and agricultural wastes posses pozzolanic properties, that is, having cementitious tendencies on exposure to moisture (O'Flaherty, 1988). Pozzolanas are, therefore, siliceous and aluminous material which themselves possess little or no cementitious value but, will, in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties (Robert, 1993). Sodium silicate has often been used as a pozzolana for replacement of rice husk ash. However, sodium silicate is expensive and difficult to handle which is why a cheap and easy to handle promoter (calcium chloride and sodium hydroxide) was explored in this research as replacement for rice husk ash chemically react with calcium hydroxide at ordinary temperature, to form compounds possessing cementitious properties. Sodium silicate has often been used as a pozzolana for replacement of RHA. However, sodium silicate is expensive and difficult to handle, which is why a cheap and easy to handle promoter (calcium chloride and sodium hydroxide) sorted and used in this research as replacement for RHA.

2.0 MATERIALS AND METHODOLOGY

2.1 Materials

The materials used in this research are black cotton soil, rice husk ash, promoter (calcium chloride and sodium hydroxide) and distilled water

2.1.1 Black Cotton Soil

The black cotton soil sample was collected from a borrow pit in Gwagwalada, FCT, Abuja. Gwagwalada is located at an elevation of 210m above sea level and has a latitude of 8º56□29□□ N and a longitude of 7º 5□31□□ E on the Nigerian geographic map (FCDA, 2015). It was collected by method of disturbed sampling in conformity with BS 1377 (1992) using the hand carved method at depths of 0.50 to 1.0m below the ground surface to avoid organic matter. The sample was then wrapped in polythene bags to avoid loss of moisture and transported to the Civil Engineering Laboratory, Federal University of Technology, Minna. At the laboratory the natural soil sample was air–dried and pulverized using a hammer and classification tests such as natural moisture content, sieve analysis, Atterberg limit and compaction were conducted.

2.1.2 Rice Husk Ash

The Rice Husk Ash was obtained locally from Minna town. Raw husk of parboiled rice was collected from local milling plants and burnt for 7 days in an open place without the control of heat (mass burning and ashing). The ash was then transported to the laboratory and sieved through sieve 75μm size and then stored in air-tight polythene bags to avoid any form of hydration. Dangote cement brand of grade 32.5 was used as the main binder in conformity with BS EN 197-1:2000.

2.1.3 Promoter

The promoter (NaOH and $Cacl_2$) was sourced locally in Minna town. The promoter was obtained in a solid form and dissolved in 1000ml of water to produce 1.0 molar concentration of calcium chloride and 1.5 molar concentration of sodium hydroxide.

2.1.4 Water

The water used was clean and portable water in accordance with BS EN 1008:2002, and was obtained at the permanent site of the Federal University of Technology, Gidan Kwano, Minna. Clean and portable water is of great importance in the stabilization process because the presence of impurities in water can affect the cementitious process and reduce the compressive strength and durability of the stabilized soil.

2.2 Methodology

The laboratory test performed on the soil in order to determine its engineering properties were in accordance with BS 1377 (1990) and BS 1924 (1990). The following tests were performed:

i. Natural moisture content
ii. Specific gravity
iii. Particle size distribution
iv. Atterberg limit (Liquid limit and Plastic limit)
v. Compaction (Standard Proctor and West Africa Standard Energy levels)
vi. Unconfined compressive strength
vii. X-ray Fluorescence
viii. X-ray Diffraction
ix. Screening electron microscopy

The unconfined compressive strength (UCS) was determined in accordance with BS 1377 (1990). A dried soil, mixed with different percentages of the stabilizers by using predetermined weight of soil obtained from the density-volume relationship, was compacted in a 1000 cm^3 cylindrical mould using West Africa Standard and Standard Proctor energy levels after the addition of water at optimum moisture content. The compacted samples were then collected from the mould and placed in sealed transparent plastic bags and cured inside moist river sharp sand for 1, 7, 14, 28, and 60 days. The unconfined compressive strength of the cured samples were determined by placing them in platens of a universal testing machine and then crushed to failure.

3.0 RESULTS AND DISCUSSION

3.1 Geotechnical Properties of Black Cotton Soil

The results of the preliminary tests conducted for identification and the determination of the properties of the natural soil are presented in Table I while the particle size distribution curve is shown in Figure 1. The soil is classified by AASHTO as A-7-6 (13) or CH in the unified soil classification system is greyish brown in colour and its geotechnical properties falls below the standard recommended for most civil engineering construction works especially highway construction (Osinubi and Medubi, 1997).

TABLE I: GEOTECHNICAL PROPERTIES OF NATURAL BLACK COTTON SOIL

Property	Quantity
Percentage passing BS No 200 sieve	68.5
Moisture Content (%)	35.06
Liquid limit (%)	63.00
Plastic limit (%)	28.37
Plasticity Index (%)	34.63
Specific Gravity	2.66
AASHTO Classification	A-7-6 (13)
USCS	CH
NBRRI Classification	High swell potential
Maximum Dry Density (g/cm^3) Standard Proctor	1.6258
Maximum Dry Density (g/cm^3) West Africa	1.7928
Optimum Moisture Content (%) Standard Proctor	24.5
Optimum Moisture Content (%) West Africa	20.95
Colour	Greyish black
Dominant clay mineral	Montmorillonite

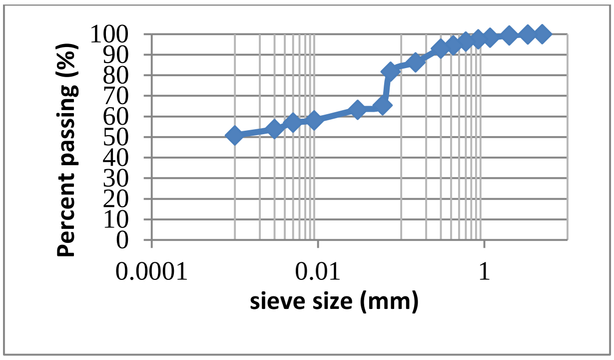

FIGURE 1: PARTICLE SIZE DISTRIBUTION CURVE OF THE NATURAL BLACK COTTON SOIL

3.2 Chemical Composition of RHA

The oxide composition of the RHA used in this study was obtained through a Compact Energy Dispersive X-ray Spectrometer Method (Mini Pal), designed for elementary analysis of a wide range of samples. The test was carried out at the Department of Civil Engineering, Kaduna Polytechnic. The chemical composition is as shown in Table 3.2

TABLE II: CHEMICAL COMPOSITION OF THE RICE HUSK ASH

Constituent	Composition%
SiO_2	68.6
Al_2O_3	4.9
Fe_2O_3	0.95
Ca	1.36
MgO	1.81
Loss in Ignition (LOI)	17.78

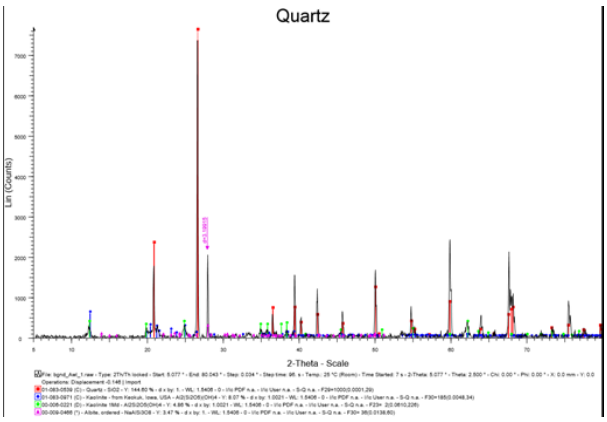

FIGURE 2: XRF OF THE BLACK COTTON SOIL

The percentages of SiO_2, Al_2O_3, and Fe_2O_3 combine is more than 70 thus, the Rice Husk Ash falls under class C according to ASTM C618 classification of Pozzolanas, which implies it is a good pozzolan and would help mobilize the CaOH in the soil to form cementitious compounds.

3.3 Unconfined Compressive Strength (UCS)

The main test recommended for use to determine the required amount of additive to be used in the stabilization of soils is the unconfined compressive strength test (Singh, 1991). A total of 128 samples mixed with different proportions of cement, rice husk ash and promoter were tested. The effects of the different mix and curing period of 1, 7, 14, 28 and 60 days on the soil samples compacted at the Standard Proctor and West Africa Standard energy levels were studied and their unconfined compressive strength variations are shown in Figures 2 to 8.

The Variation of UCS with promoter at various percentage of RHA for 1 day curing period is shown in Figure 2 to 11. From the figures, it is observed that there is an increase in UCS as the percentage content of promoter and RHA increases, while UCS gradually decreases in value as the percentage content of cement increases. For specimens compacted at the Standard Proctor and WAS energy levels, UCS values increased from 20 KN/m^2 to peak value of 233 KN/m^2 at 0.3% promoter and 1% RHA and 102 KN/m2 to peak value of 389 KN/m2 at 1% promoter and 3% RHA respectively. The increases in UCS can be attributed to ion exchange at the surface of clay particles as the Ca^{2+} in the RHA and promoter reacts with the lower valence metallic ions in the clay microstructure which results in agglomeration and flocculation of the clay particles which contribute to the increase in strength (Osinubi (2001). The reduction in UCS at 2 % cement content may be due to excess Ca^{2+} that could not be neutralized therefore forming weak bonds between the soil and the cementitious compounds formed. Generally, UCS values were relatively significant for specimens compacted at WAS energy levels.

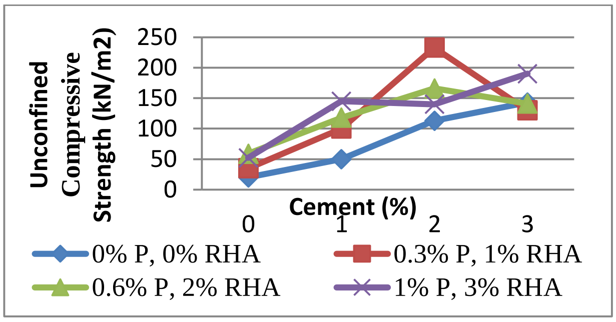

Figure 2: Variation of UCS with cement at various percentages of RHA and promoter for 1 day curing period (Standard Proctor Compaction)

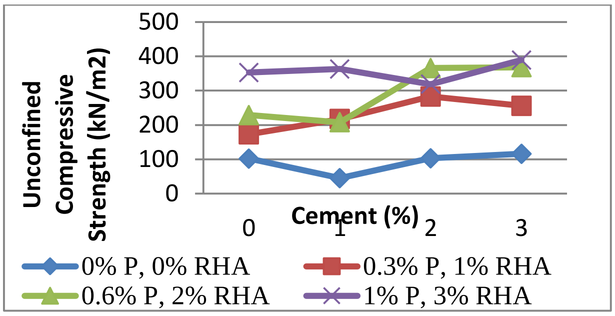

FIGURE 3: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 1 DAY CURING PERIOD (WAS COMPACTION)

Variations of UCS with promoter at various percentage of RHA for 7 days curing period are shown in Figure 4 and 5. From this figures, it was observed that UCS values gradually decreases with increase in promoter, increases with reduction in RHA. UCS decreased from 170 KN/m2 to 127 KN/m2 at 1% cement content for specimens compacted at Standard Proctor energy level and from 450 KN/m2 to 440 KN/m2 at 2% cement,1% promoter and 3% RHA content for specimens compacted at WAS energy levels. This trend in the variation of UCS values can be attributed to the reason advanced in the case of 1 day curing period.

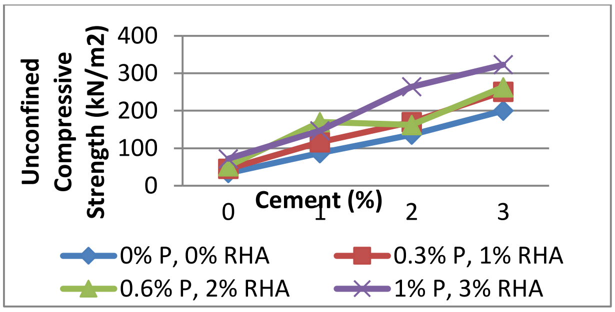

FIGURE 4: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 7 DAYS CURING PERIOD (STANDARD PROCTOR COMPACTION)

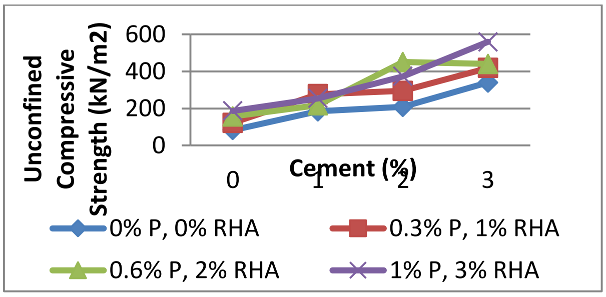

FIGURE 5: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 7 DAYS CURING PERIOD (WAS COMPACTION)

Figure 6 and 7 shows the variation of UCS with cement at different percentages of RHA and promoter for 14 days curing period. From the figures, it was observed that UCS gradually decreases as percentage of cement increase, and increases with increase in promoter and RHA content. UCS reduced from 244kN/m2 to 224kN/m2 at 2% cement and increased from 117kN/m2 to a peak value of 552kN/m2 at 3% RHA and 1% promoter. Generally, UCS increases with increase in compactive effort. The increase in UCS can be attributed to cationic exchange reaction which led to flocculation and a subsequent increase in strength while the reduction was due to the deflocculation induced by the disequilibrium in the cationic exchange reaction (Butcher and Sailie, 1984)

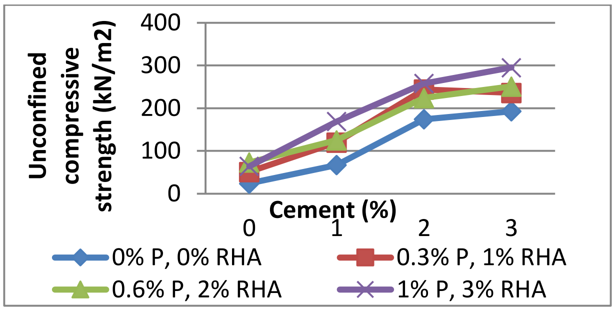

FIGURE 6: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 14 DAYS CURING PERIOD (STANDARD PROCTOR COMPACTION)

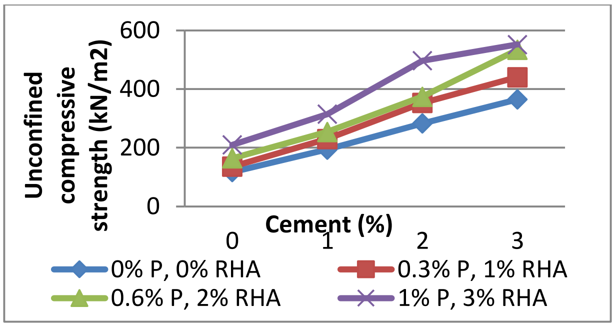

FIGURE 7: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 14 DAYS CURING PERIOD (WAS COMPACTION)

The UCS of specimens compacted at the energy levels of Standard Proctor and WAS and cured for 28 days are shown in Figure 8 and 9. Generally, UCS values increased with higher compactive effort and RHA and promoter content. However, it was noted again that the increase in UCS was relatively significant for specimens compacted at WAS which recorded a sharp increase from 330KN/m2 to a peak value of 560kN/m2 at 2% cement, 3% RHA and 1% promoter. Subsequent treatment of the black cotton soil with higher dosages of cement, RHA and promoter will decrease UCS.

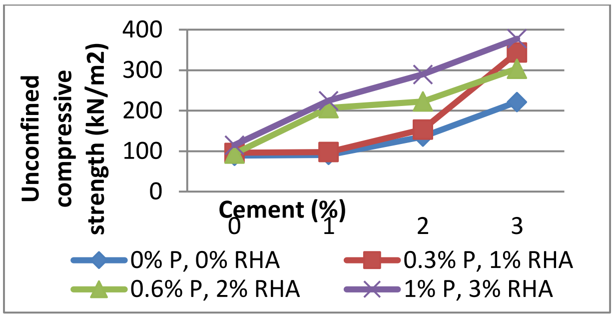

FIGURE 8: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 28 DAYS CURING PERIOD (STANDARD PROCTOR COMPACTION)

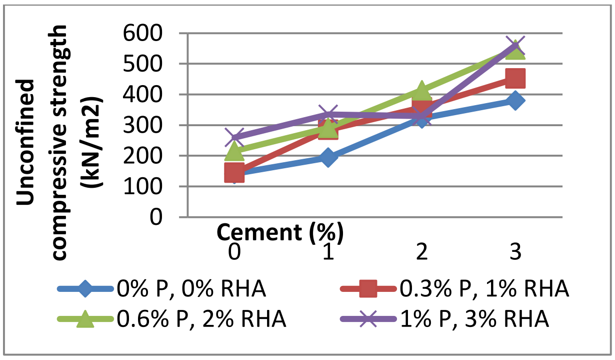

Figure 9: Variation of UCS with cement at various percentages of RHA and promoter for 28 days curing period (WAS Compaction)

The results of the unconfined compressive strength (UCS) test conducted on specimens compacted at the energy levels of Standard Proctor and WAS and cured for 60 days are shown in Figure 10 and 11. Generally, UCS values increased with higher compactive effort and RHA and promoter content. However, it was noted again that the increase in UCS was relatively significant for specimens compacted at WAS which recorded a sharp increase from 340KN/m2 to a peak value of 582kN/m2 at 3% cement, 3% RHA and 1% promoter. For specimens compacted at Standard Proctor energy level, UCS from 200kN/m2 to a peak value of 353kN/m2 at 3% cement, 3% RHA and 1 % promoter content. The increase in unconfined compressive strength is due to the production of CSH from the pozzolanic reaction between the stabilizers which is greatly enhanced by longer curing periods.

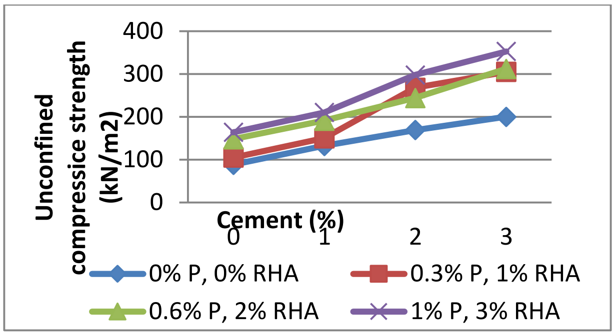

FIGURE 10: VARIATION OF UCS WITH CEMENT AT VARIOUS PERCENTAGES OF RHA AND PROMOTER FOR 60 DAYS CURING PERIOD (STANDARD PROCTOR COMPACTION)

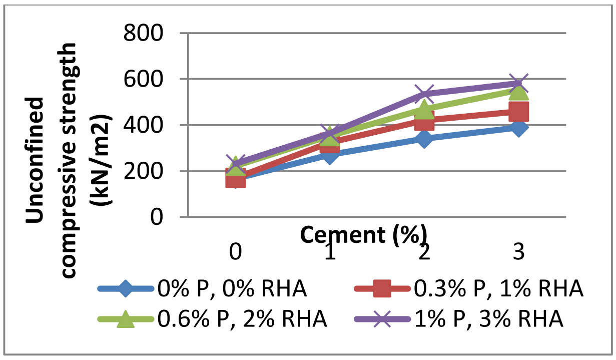

Figure 11: Variation of UCS with cement at various percentages of RHA and promoter for 60 days curing period (WAS Compaction)

4.0 CONCLUSION

The results of the laboratory investigation indicated that the inclusion of RHA and promoter in the soil improved the UCS of the soil.

1. Cement was used as a chemical agent and the values of UCS gradually decrease as the proportion of cement increase and increased with increase in RHA and promoter.
2. UCS increased from 340KN/m2 to a maximum strength value of 582kN/m2 at 3% cement, 3% RHA and 1% promoter for specimens compacted at WAS energy levels. For specimens compacted at Standard Proctor energy level, UCS increased from 200kN/m2 to a maximum strength value of 353kN/m2 at 3% cement, 3% RHA and 1 % promoter content.
3. Maximum compressive strength is achieved for a black cotton soil stabilized with 3% cement, 3% RHA and 1% promoter. Also, UCS increases with higher compactive effort.
4. The unconfined compressive strength of black cotton soil also increases with increase in curing period.

5. Also, it can be concluded that increase in Rice Husk Ash and promoter increases the unconfined compressive strength and changes the mode of failure of the soil from brittle to ductile.
6. Agricultural waste materials such as rice husk ash could be used effectively in civil engineering construction for soil stabilization.

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PHYSICOCHEMICAL CHARACTERIZATION OF LIME STABILIZED IRON ORE TAILINGS (IOT) USING BENTONITE AS ADMIXTURE

* Muhammad, A.O [1], Amadi, A. A. [2] & Alhaji, M.M [3]
[1,2,3]Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email:elmumyn@gmail,com

ABSTRACT
In this study, the physicochemical characteristics of iron ore tailings (IOT) – bentonite mixtures stabilized with lime were evaluated. The evaluation program consisted of soil characterization tests including (specific gravity, particle size distribution, Atterberg limits), pH and electrical conductivity (EC) tests. The representative IOT was treated with 20,25,30,35 and 40% bentonite content and stabilized with 6% lime content. Results of this study showed that the liquid limit (LL) increased from 28-35.5% and plastic index (PI) increased from 2-20.1% , with the addition of bentonite contents. The pH values for the various lime treated IOT-Bentonite mixtures range from 9.25-10.42 and the EC values range from 1220-1730 μScm^{-1} . The results obtained from the study showed that the mixtures containing 30,35 and 40% bentonite content were adjudged suitable for landfill liner based on their physicochemical properties.

Keywords: *Bentonite, Iron ore tailings (IOT), Landfill liners, Lime*

1 INTRODUCTION
Material selection for landfill construction is usually based on locally available materials such as iron ore tailings (IOT) and clay soil. IOT are the materials left over after the beneficiation process of separating the valuable fraction from the worthless fraction of an iron ore. However, its gradation does not favour its use as liner because of its porosity and permeability properties. IOT contain some sulphide minerals and heavy metals, and there is a possibility that these minerals may come into contact with other oxidizing agents such as oxygen and this will lead to the production of some acidic contents (Thomas and Gupta 2013). When the IOT made liners are exposed to these extreme conditions, disintegration of the liners can occur due to acid attack. Also the ferrous content of the IOT is subject to corrosion which could also have negative effects on the long term durability of the liners. Alkalinity and acid resistance test is therefore a means to check the long term durability of the tailings liners against acid attack and corrosion. If the pH of the resulting solution is above 7 it means it is in alkaline condition and there is a potential that acid attack and potential for corrosion will be very low (Thomas and Gupta 2013). Soil barriers containing appreciable clay minerals are materials used extensively in constructing liners and covers for waste containment units due to their low hydraulic conductivity. (Umar *et al* 2016). Umar *et al.*, (2016) reported that compacted soil treated with 5–20% IOT yielded hydraulic conductivity values that are lower than those obtained for the natural soil. The steady decrease in hydraulic conductivity with increasing IOT content was probably due to the high concentration of Fe_2O_3 in IOT which resulted in improvement in the mechanical properties of the soil. According to Shadfan *et al.* (1985), the interaction of Fe_2O_3 with clay minerals depends on the environments' pH. At low pH, oxides precipitate on the surface of clay minerals and, once formed, these coatings are stable at high pH values. To improve on its impermeability, IOT are sometimes treated with other materials such as bentonite to form a thicker slurry that slows the release of impacted water to the environment.

Bentonite are generally classified according to their dominant exchangeable interlayer cation, either sodium, calcium or magnesium. When sodium predominates, a large amount of water can be absorbed in the interlayer, resulting in the remarkable swelling properties observed with hydrating sodium bentonite. The presence of sodium facilitates the almost unlimited absorption of oriented layers of water molecules. Bentonite is a type of material with smectite as its main composition and also having its physical properties to be dictated by the smectite minerals. It is a montmorillonite and hygroscopic clay which is characterized by an octahedral sheet of aluminum atoms being infixed between two tetrahedral layers of silicon atoms. Bentonite also has great cation exchange capacity, bonding capacity, plasticity and strong tendency to react with organic compounds. Their physical and chemical properties (swelling ability, plasticity, cation exchange capacity) vary typically within and between deposits due to the differences in the degree of chemical substitution within the smectite structure and nature of exchangeable cations present, and also due to the type and amount of impurities present (Asad et al., 2013).

In landfill liners beyond a given bentonite content the strength starts decreasing when bentonite is further added, therefore, it can be stabilized by the addition of small percentages, by weight, of lime. Its efficiency lies in the low quantity of lime addition and the related ecological advantage because it uses the soil already in place without requiring soil replacement.

The chemical reactions between clay particles and lime can be categorized into two forms of improvement, short term reaction (modification) and long term reaction (stabilizations). In the first reaction the process of ion exchanges makes the clay minerals flocculate and agglomerate leading to a reduction in plasticity, swell and moisture content. The second reaction (pozzolanic reaction) accomplishes over a period of time creating cementing products that cause long-term strength gain (Asma and Darius, 2013). Improvement of geotechnical parameters of geometrical by lime is mainly due to physicochemical changes generated in the material. Therefore, it is important to study the physicochemical characteristics of the mixtures as it provides insight into the properties and behaviour as well as the competence of the material for the purposes of landfill liners.

2 Materials and METHODS

2.1 Materials Used

Iron Ore Tailings (IOT)

The IOT used in this work was collected from an open dump in National Iron Ore and Mining Company Itakpe, Kogi State, Nigeria. The physical and chemical characteristics of the IOT are shown in Table 1.

Bentonite

The bentonite used for this work is in powdered form obtained from a major supplier. It is a typical class of bentonite suitable for engineering construction purposes. The chemical composition is shown in Table 2

Lime

The lime used in this work is quicklime which is Calcium Oxide(CaO) was obtained from a major supplier in Kaduna, Nigeria.

2.2 METHODS

The following tests were carried out in accordance to British Standard (BS) specifications for fined-grained soils.

- Particle size distribution (Sieve analysis) test; BS 1377(1990) Part 2
- Atterberg limits (Cone Penetrometer) test 1(A); BS1377 (1990) Part 2
- Specific gravity; BS 1377 of 1990 test B for fine-grained soils
- Electrical conductivity and pH tests were conducted following American Society for Testing and Materials ASTM D 4972.
- All the tests conducted is for IOT, 6% lime and bentonite content (20,25,30,35 and 40%)

3 RESULTS AND DISCUSSION

3.1 MATERIAL CHARACTERIZATION

Table 1 shows the physical and chemical characteristics of the IOT. The specific gravity and the average moisture content of IOT is 3.5 and 0.22 %. respectively. The specific gravity is greater than 2.7 which is the minimum value for liner materials (CGRM 2007).

TABLE 1: PHYSICAL AND CHEMICAL CHARACTERISTICS OF IOT

Characteristics	**Concentration**
Moisture content (%)	0.22
Bulk density (Mg/m^3)	1.65
Specific gravity	3.5
pH	13.11
SiO_2 (%)	20.2
$FeO_{3\,(\%)}$	76.84
Al_2O_3(%)	Not Detected
CaO(%)	0.53
MnO(%)	0.05
TiO_2(%)	0.18
K_2O(%)	0.30
V_2O (%)	0.02
Au_2O(%)	1.88

Source: Umar *et al.*, (2016)

The particle size distribution curve for the IOT is shown in Figure 1

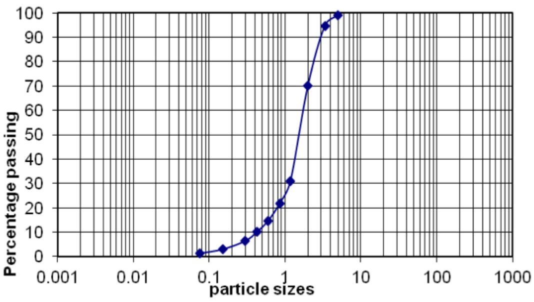

Fig 1: Particle size distribution (IOT)

In Table 2, the chemical composition of the bentonite is shown

TABLE 2 CHEMICAL COMPOSITION OF BENTONITE

Chemical	Composition Values(%)
SiO_2	58.14
Al_2O_3	21.73
Fe_2O_3	2.46
TiO_2	1.86
CaO	0.86
MgO	2.42
P_2O_5	0.119
Cr_2O_5	0.007
K_2O	0.52
Na_2O	2.08
MnO	0.07

Source: Amadi and Osinubi (2017)

3.2 Atterberg Limits of IOT-Bentonite -Lime mixtures

From figure 2, it was observed that, with the increase in bentonite content of the mixtures the liquid limit increased from 28-35.5% and the plastic index increased from 2-20.1% this is as a result of the high percentage of montmorillonite content which tends to absorb more water due to its swelling property. The figure 2, also showed that the mixtures containing 25,30,35 and 40% bentonite content had the liquid limit of 30,31,34 and35.5% respectively were of adequate liquid limit based on liquid limit value of greater or equals 30% as specified by CGRM, 2007. Also, the mixtures containing 30,35 and 40% bentonite content had the plastic index of 10.2,15.5, and 20.1% respectively which met the specifications for liner requirements. However, the plastic index of mixture containing 25% bentonite content was inadequate as it was below 10% in the specifications. A minimum plastic index of 10% is normally required as soils with a lower plasticity index are unlikely to achieve a sufficiently low permeability Teha 2004, CGRM,2007.

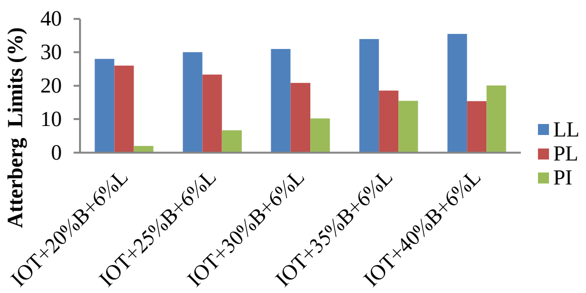

FIGURE 2 VARIATION OF ATTERBERG LIMITS OF IOT TREATED WITH 6% LIME AT VARYING BENTONITE CONTENT

3.3 pH of the IOT-Bentonite –Lime mixtures

In figure 3, it was observed that the mixtures containing bentonite content from 20-30% had a decreasing pH values from 10.42 to 8.5 but further increase of the bentonite content from 35-40% increased the pH values from 8.87 to 9.25. The pH results showed that, all the modified IOT mixtures were having alkaline characteristics with the lowest value at the mixture containing 30% bentonite content.

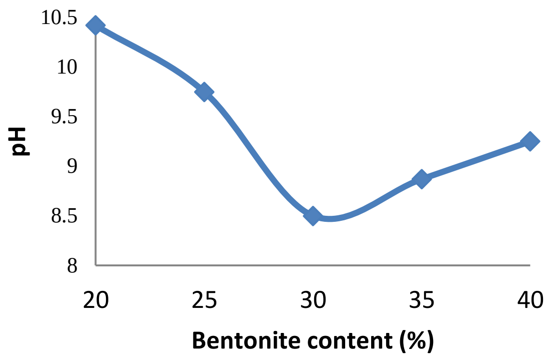

FIGURE 3 VARIATION OF PH VALUES OF IOT TREATED WITH 6% LIME AT VARYING BENTONITE CONTENT

3.4 Electrical Conductivity of the IOT-Bentonite-Lime mixtures

From figure 4 ,it was clearly observed that the electrical conductivity increased, from 1220 to 2680 μScm^{-1} at the mixtures containing 20,25 and 30% bentonite mixtures while the further increment of bentonite content decreased the electrical conductivity from 1740 to 1730 μScm^{-1}. The maximum value of the electrical conductivity was at the mixture containing 30% bentonite content.

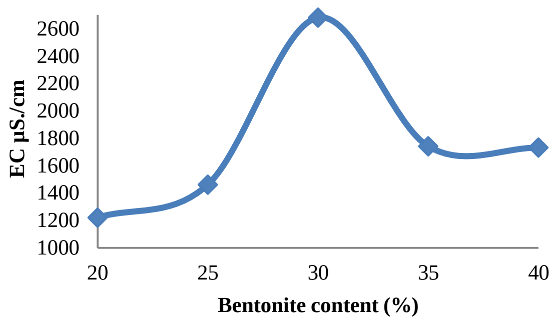

FIGURE 4 VARIATION OF ELECTRICAL CONDUCTIVITY OF IOT TREATED WITH 6% LIME AT VARYING BENTONITE CONTENT

4 CONCLUSION

Some significant variations were observed from some of the results such as in the index properties, pH and electrical conductivity of the mixtures. The geotechnical parameters of the geomaterial needs to be improved due to the physicochemical changes generated in the material. Therefore, it is important to study the physicochemical characteristics of the mixtures as it provides insight into the properties and behaviour as well as the suitability of the material for use as landfill liners. From the results of pH tests, the alkanity of the mixtures favoured its use as liners.

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Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

EVALUATION OF RISK FACTORS IMPACTING ON CONSTRUCTION PROJECT PERFORMANCE IN ABUJA

Abdullahi, M.S[1]; & Anifowose, O.M[2]
[1&2] Department of Quantity Surveying Federal University of Technology, Minna
sanusiabdullahi1@gmail.com

ABSTRACT
Nigerian construction industry plays an important role in terms of meeting the country's infrastructure and economic development. Construction is been considered to be also a risky business due to its complexity and position to national growth domestic product of any country- especially developing countries. Risk in construction industry is perceived to be an occurrence that has impacts on major objectives of any project with respect to cost, time and quality. This has led to an investigation to discover the risks factors impacting on construction project performance with a view of suggesting strategies for mitigating the risk factors. The study employed quantitative research technique and relies on questionnaires survey to understand the perception of stakeholders and their mitigating strategies of each risks factors as an instrument for data collection. A statistical tools (means items scores and standard deviation) were used to identified and ranked the most critical risk factors that has effect on construction project performance. The top major risk factors that impact on cost are finance, credit facilities and economy policy. While those that top the time are: Finance, Security and design; Design, political and economic are for quality. For each of the identified risks, practical mitigation measures were provided and evaluated. It is been suggested that when mitigating a specific risk, the measures with higher effectiveness should be given a higher priority. Taking into account the higher criticalities of higher risk hierarchy levels, the mitigation measures should also be prioritized by the higher risk hierarchy level.

Keywords: *Construction Industry, Construction Project Performance, Risk factor, and Risk management process*

1 INTRODUCTION

Construction project has to do with process that consists of tangibly assembling an infrastructure or building. Construction projects incorporate numerous activities, such as fabrication, plumbing, masonry, carpentry and joinery (Ghaffari, 2013). According to Sharma and Swain, (2011) a construction project is not a single activity. Construction starts with planning, designing, costing and financing. It continues until project is completed and ready for use. Furthermore Ghaffari,(2013) stated that construction projects performance is affected by many complex and dynamic factors such as political, economic, social, and cultural risks, term as external as well as internal risks from within the project. A project is term to be successful when it achieves its objectives, (time, cost and specification) as expected without overrun (Sharma and Swain, 2011). Abd El-Karim, *et al*. (2015) is of the view that the basic objectives of any construction project is to complete it within the cost target, agreed time and the required quality, safety and environmental limits. Hence, construction projects are characterized a poor performance which leads to small profit margin (Ghaffari-2013).

Construction is been considered to be highly risky business due to its complexity and position to national growth domestic product of any country- especially developing countries (Renuka, *et al*,2014). Risk is defined as the likelihood of a determinant event occurring to the project Therefore, risk can be seen as a measurable uncertainty positive or negative event that can occurred as result of certain decision or event on project performance (Baloi and Price, 2003). It is pertinent to understand that there is the need to evaluate risk factors that impact on the construction projects performance with a view of suggesting strategy for mitigating the risk factors.

Construction industry as a whole faces a lot of challenges and suffers costing risk ends up or bulk sum called the contingency sum which sadly has been arbitrary misconstrued by many construction professional to be 10% (Alabo, 2015). The level of project risk contingency in estimates has a major impact in their financial outcomes for clients. If a contingency is too high, it might encourage poor cost management and subsequently caused the project to be uneconomical and aborted. On the other hand, if the contingency is too low, it may be too rigid and set unrealistic financial environment, resulting in unsatisfactory performance. Yet identification, analysis and response of risk factors by various researchers need further study on the strategy for dealing with these risk factors.

Given the above scenario and observations, the aim of this study is to evaluate risks factors impacting on construction project performance in Abuja with a view of suggesting strategies for mitigating risk factors.

Furthermore in order to achieve the above aim, the following objectives were formulated;

(i) To identify and examine the risk factors that has effect on construction project performance in terms of cost, time and quality.

(ii) To suggest strategies for mitigating risk factors of Construction projects.

1.1 Risk and Construction Industry in Nigeria

According to Dantata & Tenaga, (2008), the Nigerian construction industry plays an important role in terms of meeting the country's infrastructure and economic development. About 70% of nation's capital goes to construction industry. But its contribution to gross domestic product (GDP) has been dropped in recent years. The industry also has potential for generating activity and employment in other sectors of the economy such as mining, manufacturing, and transportation). The average growth of the Nigerian construction industry was 18% in 2010 – 2012. Its contribution to the gross domestic product (GDP) in 2010 was 2.78% (Bandupe, 2015). It can be concluded that construction industry in Nigeria generates growth and development by providing public facilities which then avail the citizenry and thereby increase productivity and in turn generate county's GDP. (NBS, 2017).

However, construction industry in Nigeria is facing chronic problems including poor performance of time, cost, construction waste and poor productivity. Many constructions companies especially some indigenous contractors; were successful for some years, unfortunately, it was confronted with many operational challenges, including under estimation of projects and their inability to meet performance target. (Chileshel and Yirenlayi – Fianko, (2012) observed that, despite the use of common form of contract, called JCT form of contract includes Liquidated and Ascertained damages (LAD) clause which guarantee that a predetermined amount will be paid to the client in the event of project delays for which contractor is responsible, traditionally, the contractor bears the risk of completing the work based on agree cost and quality

. Risk in construction industry is perceived to be an occurrence that has impacts on major objectives of any project; namely cost, time and quality. Risk can be seen from two angles; that is – the external risk (the environmental) and the other internal risks which exists in projects itself (Ismail, 2014). Compared to other industries, construction is subject to more risk due to its unique features of the construction activities such as completed procedures, financial intensity, environment long dedication and dynamic organization structure. Studies have revealed that construction industry has a poor track record of risk analysis as compared to other business (Mishra and Mishra, 2016).

1.3 Risk Management Process

Risk management process (RMP) involves the drawing up of plans on how to combat risk. The overall goal of the risk management process is to maximise the opportunities and minimize the consequences of a risk event. This includes Risk identification, assessment/ analysis, register and monitoring. .The risk management process aims to identify and assess risk in order to enable the risk to be understood clearly and managed effectively

1.3.1Identification of Risk Factors Causing Cost Overruns Time Overruns and Quality Problems in Construction Projects.

Risk identification relies mostly on past experience on projects executed to be used for proposed projects. risk identification has to do with obtaining a list of potential risk to be managed in construction project (Gajewaska and Ropel, 2011). Risk identification involves the following techniques; Brainstorming, Industrial checklist., Interview with key projects participant, Examination of historical data from previous similar projects., experts' evaluation, Delphic technique. Risk factors can occur in any kind of construction project. Construction project risks include business, contract relationship, cost, funding, management, political and schedule risk.

Various risk factors that impact on construction projects performance are identified by various researches and are categorized, amongst are; Ghaffari (2013) identified forty six (46) risk factors impacting on construction projects. According to Soyingbe *et al* (2014) identified thirty three (33) risk factors that affects cost and time performance of construction projects..Alabo (2015) identified thirty (30) risk factors. Tipili and Yakubu (2016) have identified nineteen (19) risk factors. Ajator (2017) listed twenty one (21) likelihood or risk in a construction projects.

1.4 Project Performance Scope

Chan and Chan (2004) defined project success as "the set of principles or standards by which favorable outcomes can be accomplished within a set specification". Project success means different things to different people. Therefore, in this research, the following variables are used to determine project performance: (i) Cost, (ii) Time, and (ii) Quality. These variables form a system that must remain in balance for a building construction project to be achieved. Because they are so important to the success or failure of the project. Each of these variables is discussed individually in the subsequent sub section.

Cost is a major consideration throughout the project management life cycle. According to Wysocki (2009), the first consideration occurs at an early and informal phase in the life of the construction project

Time frame or deadline date is being specified by the client within a construction project (Wysocki, 2009). To an extent, time and cost are inversely related to one another. For example, the time a construction project takes to be completed can be reduced but cost increases as a result,

According to oxford English dictionary, defined quality as the standard of something when measured against the thing of similar kind, the degree of excellence of something. Quality is a comprehensive concept and its definition differs from different people. According to Wysocki (2009), a project is said to be of a good quality if the output; meets the specification, meets the customer's requirements and satisfies the customer

2 METHODOLOGY

In order to meet the research objectives, two research approaches was adopted; First, the research work started with extensive literature scan for past researchers concerning risk management, risk factors and construction project performance containing some thesis, construction and engineering journals, and academic published papers. The literature review was used to assist in developing the initial risk factors list and mitigating measure. Similar to this step, a pilot survey was considered to validate these identified risk factors and further add to existing one.

The second approach is the used of questionnaire. Data collection was carried out using questionnaire survey to understand the perception of the stakeholders, to the risk factors and mitigation strategies. The questionnaire was designed based on knowledge obtained through the reviewed of literature and content validity. A questionnaire survey was conducted between June and August, 2018, and the target respondents were the Architects, Builders, Quantity Surveyors, Electrical Engineers, and Mechanical Engineers, that formed the population for the study.

Stratified sampling technique was used to determine the number of the responses. This is a form of probability sampling that classified people into groups according to their characteristics. For each risk factor, expert was asked to assign an impact based on scale as it relate to cost, time and quality. Furthermore, percentage was used to assess the sharing of risk between the client, consultant, and contractor. The survey was distributed to 30 Experts within the Federal Capital Territory - Abuja (30 valid responses were received out of the 32 distributed questionnaires)

The data gathered from the survey were analyzed statistically using two of the techniques of Statistical Package for Social Sciences (SPSS) they are Means Item Score (MIS) and Standard Deviation (SD)i.e equation (1) and (2) respectively to determine most significant and ranking of risk factors that impact on construction project performance. The same approached was adopted by many researchers to analyze their data collected from survey. The mean item score and standard deviation are calculated with the following formulae

$$. \text{MISi}= \frac{\sum(f \times S)}{N} \quad (1)$$

where;

S= score given to each risk factor by the despondence,

f= Frequency of response to each score for each risk factor

N= Total number of response in the respective groups for the respective risk factor, and

i= Respective risk factor

While for standard deviation (SD) is as follows;

$$\text{SD}=\frac{\sqrt{\sum(x-\hat{x})^2}}{\text{n-1}} \quad (2)$$

Standard deviation б = √variance; and variance is

$$Variance=\frac{(n-mean)+\cdots\ldots(n-mean)}{N-1}$$

3 RESULTS AND DISCUSSION

3.1 Demographic result

A total of 32 questionnaires were distributed; A response rate was 100% but, the valid questionnaire were 30 out of 32, means 93% valid questionnaire were analyzed.

Table II: RESPONSE RATE

S/No	Description	No. of Questionnaires Distributed	No. of Valid Responses	No. of Invalid Responses	Percentage of Valid Responses
1	Respondents	32	30	4	93%

Source; Authors' Field Survey(2018)

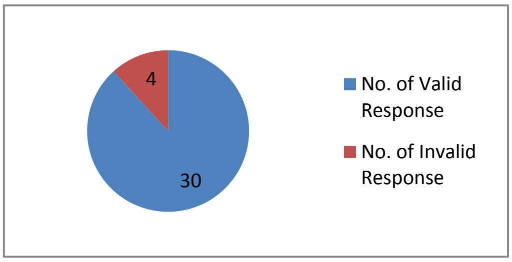

FIGURE I : RATE OF RESPONSES

To validate the study, the sample size adopted for this study was thirty (32), selectively distributed based on stratified sampling. The chosen number was based on recommendation of Abd Karim (2012), that a sample size larger than 30 and less than 500 are appropriate and efficient for most research. Thirty two (32) questionnaires were given and all returned making 100% return rate; 30 were properly completed. Abd Karim (2012), stated that a respondents range between 25-100 is appropriate for a study test. The response rate are categorized based on their educational and professional qualifications and area of specialization; year of experience in the construction industry.

From the analysis 14 out of 30 had first degree or Higher National Diploma (HND), 5 with Post Graduate Diploma (PGD) and 11 with Master's degree. On professional qualifications, 90% of them are registered with their professional bodies. On years of experience; 12 of the respondents are 1-15 years of experience, 6 are those between 16-25 years , 12 are those 25yaers and above.

On the Response rate 23% response are Architects, Builders, Electrical Engineers and Mechanical Engineers are all 17% each, while Quantity Surveyors 26% response. Table 2 and Figure 2 show the categorized response rate.

Table III: CATEGORIES BY AREA OF SPECIALIZATION

S/No	Status	No. of Responses	Percentage	Cumulative Percentage
1	Architect	7	23	23
2	Builders	5	17	40
3	Electrical Engineer	5	17	57
4	Mechanical Engineer	5	17	74
5	Quantity surveyors	8	26	100
	Total	**30**	**100**	

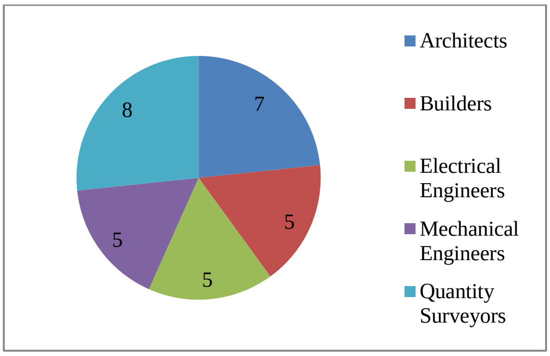

FIGURE II: CATEGORIES BY AREA OF SPECIALIZATION

Identified and categorized risk factors impacting on Construction projects

The identified risk factors from literature scan and response from the questionnaires are hereby categorized. Ninety four (94) risk factors have been identified and categorized into eighteen (18); Viz a) Design risks, b) Planning & Delay Schedule, c) Technology risk, d)Financial risk, e) Procurement-contractual risk, f) Political risk, ,g) Environmental risk, h) Social risk, i) Economic risk:, j) Credit risk:, k) Standards and regulation risk ,l) Physical Factor, m) Legal Risk ,,n) Construction (Project execution), o) Project Management , p) Political, q) Resources (*Material, Labour,Plants& Machinery, r)* Contractual (*Technical Procedure & procurement).*

Ranking study of risk factors

The five most important risk factors that impact on construction project Cost from table 2 are; Finance, Credit facilities, Economic policy Contract/procurement system and Planning & delay Schedule, they are ranked as 1st, 2nd, 3rd, 4th, and 5th respectively.. While those risk factors that impact on Time are; Finance, Security, Design, Contract/procurement and Project management 1st, 2nd , 3rd , 4th , and 5th respectively. Those that affect quality are; Design, Political, Economic, Resources and Finance

In attempt to rank them, Mean Item Score and Standard deviation was used. The MIS with highest value is considered, but where MIS value are sane with a different independent variable, the lowest value of SD is used to rank the most effective

TABLE IV; RANKING Of RISK FACTORS

S/No	Risk Factors	Cost			Time			Quality		
		MIS	SD	Rank	MIS	SD	Rank	MIS	SD	Rank
1	Design	3.9643	0.9222	6th	3.9643	1.8852	3rd	4.2857	0.8668	1st
2	Environment	3.5357	0.8599	14th	3.3929	0.9940	15th	3.2143	0.7868	9th
3	Physical Factor	3.5000	0.3848	15th	3.3571	1.1616	18th	3.1071	1.2573	15th
4	Finance	4.2500	0.5853	1st	4.1428	0.5909	1st	3.4285	1.2301	5th
5	Economic	4.1785	0.6118	3rd	3.8214	0.8630	8th	3.5714	1.1362	3rd
6	Contract/Procurement	4.1428	0.5909	4th	3.9228	0.6627	4th	3.2142	1.1975	11th
7	Resources	3.7857	0.8759	10th	3.8571	1.4584	6th	3.4286	0.8789	4th
8	Construction	3.6428	0.8698	13th	3.5000	1.7533	17th	3.2142	0.5443	10th
9	Safety	3.4286	1.7305	17thh	3.6429	2.0224	13th	3.1071	1.1001	14th
10	Legal	3.7143	0.7629	12th	3.6786	0.6696	11th	2.9286	0.9786	17th
11	Planning & Delay schedule	3.9643	0.7444	5th	3.6786	2.0558	12th	3.3214	1.3486	7th
12	Standard & Regulation	3.7500	0.9280	11th	3.8214	0.6118	7th	3.1785	0.8181	13th
13	Security	3.9285	1.2745	7th	4.0000	0.5443	2nd	3.1786	0.9833	12th
14	Credit	4.2142	0.7382	2nd	3.7142	0.7228	10th	3.0714	1.0338	16th
15	Social	3.4642	0.8381	16th	3.3928	0.7859	16th	2.9285	1.0157	18th
16	Technology	3.8214	1.0203	9th	3.6428	1.0261	14th	3.4285	1.5494	6th
17	Political	3.8928	0.6853	8th	3.7500	0.7005	9th	4.000	0.8165	2nd
18	Project management	3.1071	1.6631	18th	3.8571	0.6506	5th	3.2500	0.7515	8th

Source; Authors" Field Survey

Risk Mitigation Measure

Based on the outcome of preliminary investigations and a review of relevant literature, a generic model of the payment risk sources and the response measures was developed to provide a holistic insight into the key constructs underlying the internal and external sources of risks and their associated mitigation measures The model helps to put the scope of the study in the context of the wider facets of the subject matter. The proposed strategies for responding to risks that may have negative impacts on expected outcomes are risk avoidance/elimination, risk minimization/mitigation, risk retention and contingency, and risk transfer-and an improvement on the three strategies provided in the Project Management Body of Knowledge (PMBOK) (Project Management Institute, 2004); the key point of difference is the inclusion of the risk retention and contingency on Table 4 shows the details of response strategies.

Table IV: RISK RESPONSE STRATEGIES- MITIGATING MEASURES

S/No	Risk factors	Mitigating measures
1	Design	- Ensure the design are done properly - All design before commencement must be stamped - Proper coordinates of drawings (Structure Mechanical and Electrical) - Adopt design & built option
2	Site safety	- Ensure that construction and operation are as per examination - Get third party insurance for compensation - Study and implement the local accident regulations
3	Construction	- Adopt proper quality control procedures - Undertake probability analysis
4	Legal	- Maintain good relation with development control unit of government higher official - Obtain good guarantee to adjust tariff or extend concession period (for BOT) - Obtain support of in termination monitory institution like world bank, ADB against discrimination and harassment
5	Political instability	- Develop over contingency plan for possibly instability - Seek incorporation of termination or delay clause in contract - Obtain insurance for political risk from international finance
6	Social	- Undertake comprehensive negotiations & agreement with local government & partners - Provide dispute settlement clauses in the contract - Have an understanding with the host community
7	Resources	- Sign formal employment contract with every staff - Offer better enumeration/incentives packages to staff - Office training to new and existing staff - Ensure good material are chosen
8	Contract/procurement	- Examine the target company's financial viability, technical and management competence - Insist on having trustworthy people on key places - Have a clear contractual terms and conditions agree on one accounting standard - Pay careful attention to contract translation - Conduct market survey
9	Environment	- Insure all of the insurable force majeure risk - Include delay clause for contingency plan in contract
10	Technology	- Limit the deviation of technology transfer - Negotiate on amount and speed of technology transfer - Intellectual property right's training of all key employees
11	Standard and regulation	- Ensure compliance with both local and intermediate laws
12	Project management	- Hire competent project management team - Clear definition of staff duties and responsibilities - Employ local staff with bilingual ability because of communication gap
13	Physical factor	- Ensure an archeological survey is done before commencement of project - Visit the site and study the environment before commencement
14	Credit/Economic	- Get letter of credit form government - Obtain payment & performance bond - Adopt alternative to contract payment e.g. land development rights, resources swap - Client to secure standby financing- (i.e. more 100% financing commitment when needed)
15	Finance	- Use dual-currency contract with portion to be paid in local currency & others in foreign currency - Obtain government guarantee of exchange rate and convertibility e.g. fixed rate for long period
16	Corruption	- Enter into contract with government authorities to prevent corruption - Set aside a budget for unavoidable spending - Try to work with the business connection; i.e. avoid hire broker or middle men - Obtain all necessary approval in time to minimize chance for corrupt individual to obstruct work.

Source; Authors' Field Survey (2018)

4 CONCLUSION

Ninety four (94) risks factors associated with construction projects in Abuja were identified and categorized into three hierarchy levels (Country, Market and Project). Of which some were evaluated as Critical or Very Much Critical based on a 5-degree rating system (using likert scale). This paper presents research results obtained through questionnaire surveys conducted in Abuja. It is pertinent to note that, key risk factors that have impact on cost, time, and quality performance of construction projects were based on comprehensive assessment of their index score comprising the likelihood of occurrence and magnitude of consequence (Impact). The top major risk factors that impact on cost are finance, credit facilities and government economy policy. Similarly, finance, security and design top the time, while design, political and economic were the top ranked risk factors against quality.

Apart from identifying the top major risk factors/and their effect, it also suggest the strategies for responses that suit each of the identified risk. For each of the identified risks, practical mitigation measures were provided and evaluated. Almost all of the mitigation measures were perceived by the respondents to the survey as effective

The findings of this study have a lot of contributions to the body of knowledge. Some of these contributions are: a) Risk factors can be avoided if not fully eliminated if the findings of this research work are adopted and implemented, b) The study can also serve as a basis for future work and sources of literature for other research work.

Recommendations

It is suggested that when mitigating a specific risk, the measures with higher effectiveness should be given a higher priority. Taking into account the higher criticalities of higher risk hierarchy levels, the mitigation measures should also be prioritized by the higher risk hierarchy level, i.e. the risks at higher hierarchy level should be mitigated first with higher priority with their respective more effective mitigation measures. This paper provides a number of recommendations for decision makers on construction industry in Abuja.

(i) The Federal ministry of finance, National Planning, Investment and Power, works and housing in conjunction with professional bodies registered board/council to provide early warning or broadcasting of risk factors facing different fields of projects

(ii) Nigerian universities and other institutes of higher learning be directed to includes risk knowledge into their curriculums of studies especially in field of construction and engineering.

(iii) Improve the role of insurance companies to help eliminating or minimizing a wide range of risk factors

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PRELIMINARY ASSESSMENT OF IRON ORE TAILINGS STABILIZED WITH QUARRY FINES AND CEMENT FOR PAVEMENT SUBBASE

[*1]Igbadumhe O. A., [2]Amadi A.A., [3]Alhassan M.
[1,2,3]Department of Civil Engineering, School of Infrastructure and Process Engineering, Federal University of Technology, Minna, Niger State Nigeria.
*Corresponding author email:tonyoshio@gmail.com

ABSTRACT
This study examined the preliminary assessment of Iron Ore Tailings (IOT) stabilized with Quarry Fines and cement for road pavement construction. As a result, index properties and compaction tests of the IOT were prepared with 0,5,10,15 and 20% Quarry fines and 0, 3, 6, 9 and 12% cement and evaluated. The results indicate that the materials fall short of standard in gradation, the IOT and Quarry Fines were found to have specific gravity of 3.68 and 2.53 respectively. The mixtures were found to be non – plastic (NP). From compaction, it was observed that the optimum moisture content reduces with increase in cement from 18 to 6% whereas the maximum dry densities increased from 2.07 to 2.42g/cm^3 with increase in cement content.

Keywords:, *IOT, Quarry Fines, Subbase Pavement, Cement*

1 INTRODUCTION

Tremendous quantities of materials are used annually in the construction and maintenance of highway pavements at all levels of the roadway network. A subbase is that layer of aggregate material laid on a subgrade on which the base course layer is located. Rising energy costs and depletion or shortages of conventional quality paving materials in many areas have contributed to a helical financial burden on highway agencies (Yellishetty, 2009). As a result, there is accretive interest in making optimal use of available resources (resources include money, materials, manpower, etc.). Optimal use of available resources can imply the development of “new” materials and/or the use of hitherto considered “marginal or unacceptable” locally available materials.

In this context, Marginal materials can be defined as materials which do not in their present form possess quality levels as defined by current highway standards sufficient for their use as various pavement structural components including surfaces, bases, and/or subbases. Waste materials such as iron ore tailings belong to this category. Iron ore tailings are waste materials obtained as an end product of mining activities of iron ore (Kuranchie, 2014).

Soil stabilization is a general term for any physical, chemical, biological or combined method of changing a natural soil to meet an engineering purpose. Improvements arising from this include increasing the weight bearing capabilities, tensile strength, and overall performance of in-situ subsoils, sands, and other waste materials in order to strengthen road surfaces. The use of stabilization means that a wider range of soils can be improved for bulk fill applications and for construction purposes. The most common method of stabilization involves the incorporation of small quantities of binders, such as cement, lime, fly ash, to the aggregate (Pandey 2017).

Cement is a binder used to improve geotechnical properties of soils for engineering purposes. Cements used in construction are usually inorganic, often lime or calcium silicate based and can be considered hydraulic or non – hydraulic depending on its ability to set in the presence of water. Cement bound materials comes in multiple of types ranging from soil cement through lean mix to mass and reinforced concrete. Cement modified soils contains relatively small proportion of Portland cement the result is caked or slightly hardened material similar to soil but with improved mechanical properties such as lower plasticity, increased bearing ratio and shear strength and decreased volume change (Venkatramaiah, 2006).

MATERIALS

Iron Ore Tailings (IOT). The IOT was obtained from the Nigeria Iron Ore Mining Company, Itakpe, Kogi State, Nigeria.

Quarry Fines; Quarry Fines were obtained from Maitumbi quarry, Minna, Nigeria. Only fraction passing through BS. Sieve No. 4 (4.76mm) was used in the study.

Cement; Ordinary Portland cement (OPC) type – 1 (Dangote Cement from Obajana) was locally sourced within Minna Metropolis to comply with BS12: 1996; specification for Portland cement

Water; Clean portable drinking water was used in this work.

2 METHOD

Index Properties

Particle size distribution, Atterberg limit and specific gravity tests were conducted in accordance with BS 1377 and BS 1924 (1990).

Compaction Tests

Compaction tests were carried out on the IOT – Quarry fines mixtures stabilized with cement using the BS Heavy compactive effort in accordance with BS 1377 (1990) and BS 1924 (1990).

3 RESULTS AND DISCUSSION

Particle Size Distribution

The particle size distribution curves presented in Figure 1a and 1b shows that the IOT and Quarry Fines contains about 21.9 and 16.58% fines respectively. This indicates that the IOT and Quarry Fines may be classified as A – 3 in accordance with AASHTO classification or Poorly Graded Sand (SP) under the USCS. In view of this, it was observed that from local specifications (Nigeria General Specification (NGS)), the IOT do not meet the minimum requirements of fines of less than 20% which makes it in the natural form unsuitable for use as subbase material (NGS).

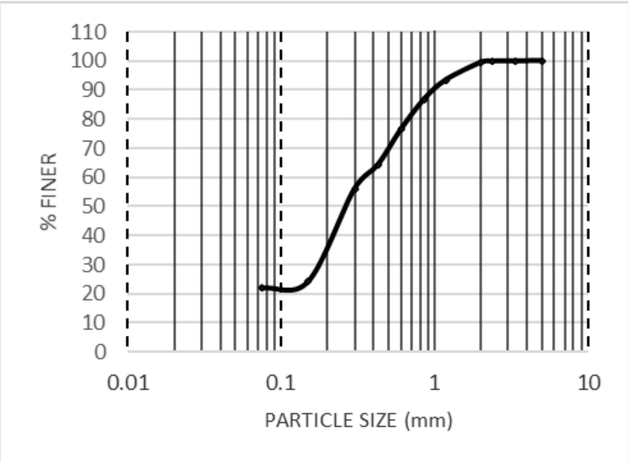

FIGURE 1A: PARTICLE SIZE DISTRIBUTION OF IOT

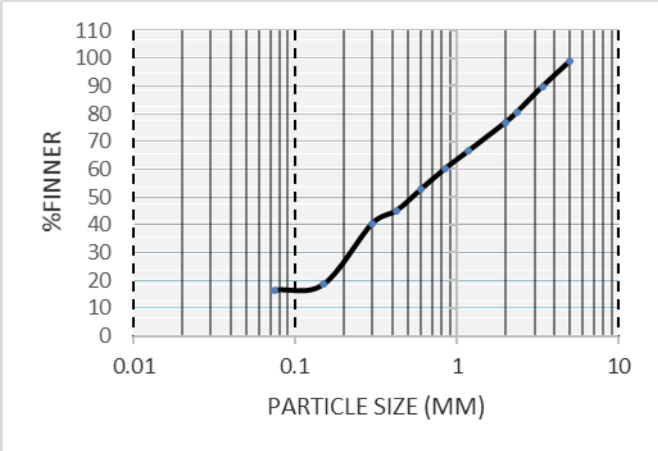

FIGURE 1B: PARTICLE SIZE DISTRIBUTION OF QUARRY FINES

Atterberg Limit

The mixtures were observed to be non – plastic nevertheless, IOT was found to fall short of liquid limit requirement of not more than 35% for subbase as outlined in NGS. Introduction of quarry fines initially increased the liquid limit to 60% at 10 % QF and then reduced then liquid limit 27% at 20% QF. With introduction of cement, it was discovered that the liquid limit further reduced to 26% at 12%cement content of the mixtures. Thus liquid limit of the mixture satisfies the requirement for pavement subbase and base as outlined in the specification NGS (1997).

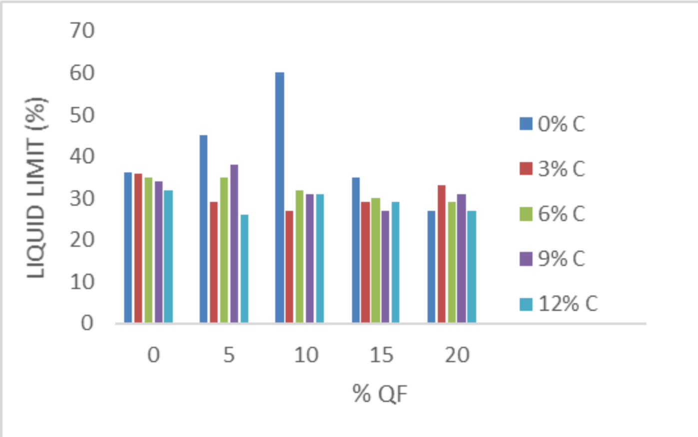

FIGURE 2: ATTERBERG LIMIT

Compaction Characteristics

The Maximum Dry Density and Optimum Moisture Content compacted with BS Heavy effort were evaluated. Dry densities are found to be on the increase with addition of quarry fines from 2.07 to 2.29 g/cm^{3}at 20% quarry fines.

Effect of Quarry Fines and Cement on MDD

The introduction of quarry fines – IOT mixture resulted in an increase in the maximum dry density from 2.24g/cm^3 at 5% QF to 2.29g/cm^3 at 20% QF. The introduction of cement to the mixture further resulted in an increase up to 2.42g/cm^3 at 20% QF 12% Cement. In soils stabilized with non-plastic quarry fines or similar materials, the introduction of fines usually causes increase in angle of internal friction consequently decreasing cohesion but with increase in quantity in excess of the required amount to fill the voids, lesser strength properties results (Amadi, 2011).

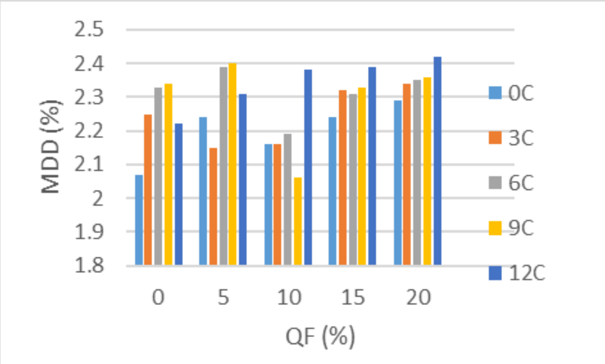

FIGURE 3: EFFECT OF QUARRY FINES AND CEMENT ON MAXIMUM DRY DENSITY

Effect of Quarry Fines and Cement on Optimum Moisture Content

From the fig 4, it was observed that the moisture content increased with increase in quarry fines and decreased with increase in cement content from 12% at 0% cement to 6% at 12% cement content. Less amount of water is usually required for a reduced efficient surface to attain optimum moisture content. This can be attributed to the continuous decrease in optimum moisture content with increase in cement content (Meshiba and Akanbi 2007).

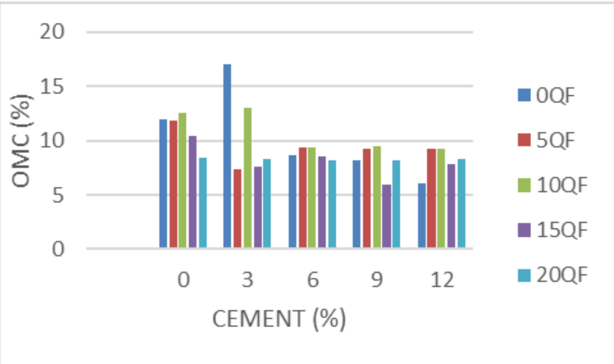

Fig. 4. Effect of Quarry Fines and Cement on Optimum Moisture Content

4 CONCLUSION

This investigation was carried out in an attempt to study the compaction characteristic of IOT – Quarry Fines and Cement mixture using the BS Heavy energy. IOT was modified with quarry fines in proportions (0 – 20%) strided at 5%. These mixtures were evaluated for Atterberg limits, specific gravity, as well as compaction. The results obtained shows that all the mixtures were non – plastic (NP), the dry density increased with increase in quarry fines from 2.07 to 2.29 at 20% as well as increase in cement content up to 2.42 at 12% while the optimum moisture content (OMC) decreased from about 13% to 6%. The IOT was found to have specific gravity of 3.68 and the quarry fines 2.53.

In terms of compliance with specifications as stipulated by local standards, the gradation, liquid limits and plastic limits for the mixture gratifies specification requirements for use of cement stabilized soils for road pavement construction.

Data from this study could provide baseline information for highway Engineers in the use of this material for stabilization of IOT for effective delivery of serviceable roads in rural areas.

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CORRELATION BETWEEN GEOTECHNICAL INDEX PROPERTIES AND COMPACTION PARAMETERS OF CEMENT STABILIZED IRON ORE TAILINGS–BENTONITE MIXTURES

Akinwande B.O, Amadi A.A, Alhaji M.M, Sanni A.
Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email:akinwande.pg718093@st.futminna.edu.ng

ABSTRACT
This work focuses on correlation between selected geotechnical index properties (Liquid limit, Plastic limit and Plasticity Index) and compaction parameters (Optimum Moisture Content and Maximum Dry Density) of cement stabilized Iron ore tailings-bentonite mixtures. IOT, an industrial by-product from mining industry was treated with varying percentage of bentonite (20, 25, 30, 35,and 40%) and stabilized with 8% cement. Compaction was carried out using three compaction energy levels (British Standard Light (BSL), West African Standard (WAS) and British Standard Heavy(BSH). Experimental results showed an increase in both Plasticity index (from 8.2% at 20% bentonite to 17.5% at 40% bentonite) and liquid limit (from 29.2% at 20% bentonite to 36% at 40% bentonite). It was also observed that a decrease in Maximum dry density (from 18.9kN/m^3 at 20% bentonite to 15.5kN/m^3 at 40% bentonite was recorded

Keywords: *Bentonite, Iron ore tailing (IOT), Optimum moisture content, Maximum dry density.*

1 INTRODUCTION

Suitability of soil in it's natural state for supporting an engineering structure at a construction site can be a serious challenge as soil is adjudged heterogeneous in nature. One of the most important activities in the early stages of construction is soil compaction and this is carried in other to make soil properties fit for engineering purposes by improving the bearing capacity, decrease the expected settlement under working loads and increase it's shear strength.(Bharathan *et al.,* 2017). The liquid and plastic limits are especially extensively used for the identification, description, and classification of fine-grained soils and as a basis for the preliminary assessment of their mechanical properties where data on in-situ parameters are difficult to obtain.

In practical field, it is often difficult to carry out all types of laboratory tests due to lack of time and cost considerations. Empirical relationships between properties of soil are generally sought for preliminary design and planning of quality control programme to be implemented on the construction site. Compaction characteristics of a soil can be related to different characteristic properties, such as the Atterberg Limits (Prasanna *et al,* 2017). Field compaction of fine-grained soils usually involves different equipment with compaction energy varying significantly. Hence the compaction characteristics need to be obtained at different compaction energies. Knowledge of compaction behavior and its characteristics of fine-grained soils at different compaction energy levels assume great importance from the view point of practical significance.

In recent decades, intensive research and development efforts have been directed towards finding cost-effective and eco-compatible solutions for minimizing and utilizing the waste produced in iron-ore mining operations (Bandopadhyay et al.,2002; Johnson et al., 1992. Ruiyingbai *et al,* (2011),stated that the fine particles less than 75μm in iron ore tailing sand are beneficial to the reduction of expansion induced by alkali-silica reaction(ASR) in concrete and mortar. (Ayrton and Adriana, 2010), described IOT to be innocuous based on the quantity of dissolved silica and reduction in alkalinity of the mix.

Iron ore tailings are the reminant of rock mineral debris after the useful metals such as iron are removed from the raw iron ore which goes through the beneficiation process of crushing, screening, grinding, grading while mining (Chen *et al.,* 2012). Up till now, the most common methods of utilizing iron ore tailings as resource are recycling element, making it the backfill material for backfilling mining or the raw material for cement production, producing floor or building bricks, being the filler in road construction (Huang, 2013). Tailings contain all other constituents of the ore but the extracted metal, among the heavy metals that are either added to the tailings in the milling process or available with the ore before(ICOLD, 2003; Mahmood and Mulligan, 2007) .

Recently, research works have focused on effective utilization of iron-ore tailings as partial replacement in construction works which includes, brick and concrete production and backfilling. Mangalpady (2012) studied the suitability and reliability of Iron ore tailings in manufacture of paving blocks. He prepared five reference mixes using cement, jelly dust and baby jelly with different mix ratios and by using sand and Iron ore tailings as fine aggregates. The results showed that compressive strength of tailing based mix was higher than the respective reference mix. This is controlled by introduction of montmorillonite minerals such as bentonite. Sujing *et al.,* (2013) studied the possibility of using Iron ore tailings to replace natural aggregate to prepare ultra – high performance concrete (UHPC). It was found that the 100% replacement of natural aggregate by Iron ore tailings significantly replaced the workability and compressive strength of UHPC. Sridharan (2004) studied the compaction behavior and characteristics of fine-grained soils with reference to compaction energy. They reported that index properties namely plastic limit, liquid limit and shrinkage limit can be related to the compaction characteristics of the fine-grained soils. Iron ore tailings cannot be used alone for landfill application due to it's gradation and high permeability

The physical characteristics of bentonite are based on the characteristics of smectite minerals. These characteristics are: high swelling, large cation-exchange capacity, low hydraulic conductivity, and large specific surface area (Gleason et al. 1997). Bentonite is available in two major types; Sodium (Na) and Calcium (Ca) depending on the type of external cation. Sodium bentonite is more used in the engineering practices than Calcium bentonite because sodium bentonite has lower hydraulic conductivity and higher swelling (Alther 1982, 1987; Reschke and Haug 1991). Mesri and Olson (1971) stated that at the same void ratio, a calcium-dominated smectite was about 1,000 times more permeable than a sodium-dominated smectite. Bentonite is utilized in different engineering practices, such as barriers in landfill, geosynthetic clay liners, and vertical cutoff walls (Gleason et al. 1997). Smectite minerals in bentonite have mainly montmorillonite in its structure. However, in landfill liner application, bentonite reduces compressive strength thus giving rise to introduction of cement to add to compressive strength.

2 METHODOLOGY

2.1 Materials

Iron ore tailings (IOT)

The iron ore tailings (IOT) was collected from National Mining Ore Company Itakpe, Ajaokuta Local Government Area of Kogi State, geographically located in North Central Nigeria. The IOT, which was old, and exposed to sun and rainfall was from tailing heaps formation in Itakpe

Bentonite

The bentonite used for this study is in powdered form from a major supplier in Nigeria. It is a representative of typical bentonite available for construction purposes.

Cement

The Ordinary Portland cement (OPC) used in this study is from the open market in Minna, Nigeria.

2.2 Testing Methods

The following tests were conducted in accordance to British Standard (BS) specification for fined-grained soil viz;

- Particle size distribution(Sieve analysis) test; BS 1377(1990) Part 2
- Atterberg limits (Cone Penetrometer); Test 1 (A) BS 1377 (1990) Part 2
- Specific gravity (BS 1377 of 1990 test B for fine-grained soils
- Compaction tests (according to BS 1377(1990) Part 4

3 RESULTS AND DISCUSSION

From the results of chemical characteristics of IOT ob in Table 1, It is seen that the main crystalline phases were quartz (SiO_2) and hematite (Fe_2O_3). The total percentages of SiO_2, Al_2O_3 (ND) and Fe_2O_3 phases present in the IOT is 97.04 % which is greater than the minimum (70 %) specified in ASTM C-618

TABLE 1 CHEMICAL CHARACTERISTICS OF IRON ORE TAILINGS

Characteristics	Concentration(%)
Al_2O_3	Not determined
SiO_2	20.2
K_2O	0.30
CaO	0.53
TiO_2	0.18
V_2O	0.02
MnO	0.05
Fe_2O_3	76.84
Au_2O	1.88
Ph	13.1

Adapted from Umar *et al.,* (2015)

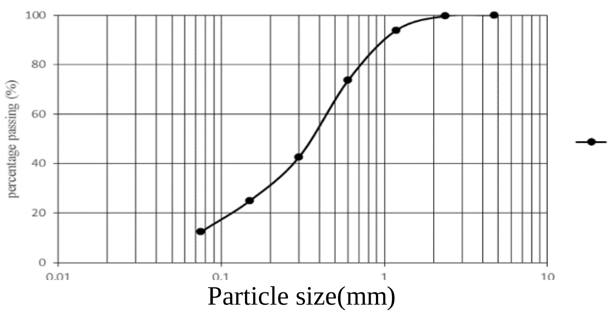

FIGURE1 PARTICLE SIZE DISTRIBUTION CURVE (IOT

TABLE 2 CHEMICAL COMPOSITION OF BENTONITE

Chemical Composition	Values(%)
SiO_2	58.14
Al_2O_3	21.73
Fe_2O_3	2.46
TiO_2	1.86
CaO	0.86
MgO	2.42
P_2O_5	0.119
Cr_2O_3	0.007
K_2O	0.52
Na_2O	2.08
MnO	0.07

Adapted from Amadi and Osinubi(2017)

The result obtained from the Atterberg limits and percentage bentonite content blended in shows that addition of bentonite to IOT stabilized with 8% cement increased the liquid limit. From the results obtained in figure 2, it was observed that as bentonite content increased, liquid limit increased with a decrease in plastic limit and an increase in plasticity index. This is attributed to high swelling characteristics of bentonite in the mixtures, also plastic limit has tendency to decrease with increasing water content caused by liquid limit as bentonite content increased (Ahmet S.Z and Temel Y.,2011, Amadi and Eberemu,2013).

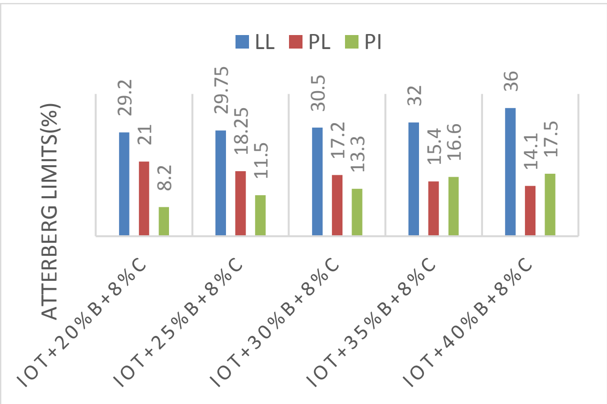

FIGURE 2 VARIATION OF ATTERBERG LIMITS OF IOT TREATED WITH 8% CEMENT AND VARYING BENTONITE CONTENT

From the results obtained in Figure 3, it was observed that addition of bentonite content in the IOT treated with 8% cement increased the optimum moisture content in all the three energy levels. The increase in optimum moisture content with higher bentonite content is connected with the increasing demand for more moisture for hydration reaction arising from the increase (Gleason et al. 1997; Amadi and Eberemu,2013).

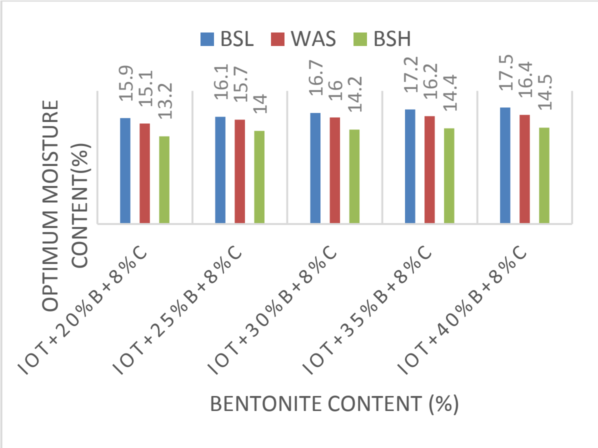

FIGURE 3 VARIATION OF OPTIMUM MOISTURE CONTENT WITH IOT TREATED WITH 8%CEMENT AND VARYING BENTONITE CONTENT.

In Figure 4, it is clearly seen that increase in bentonite content decreases the MDD. This is as a result of void created between particle to particle contact of IOT. The decrease in maximum dry unit weight with increase in bentonite content may be as a result of high swelling characteristics of bentonite that could form a gel around the IOT particles. When gel forms around the soil particles, their effective size increases, which causes increase in void volume, and thus decreased dry unit weight (Amadi and Eberemu,2013).

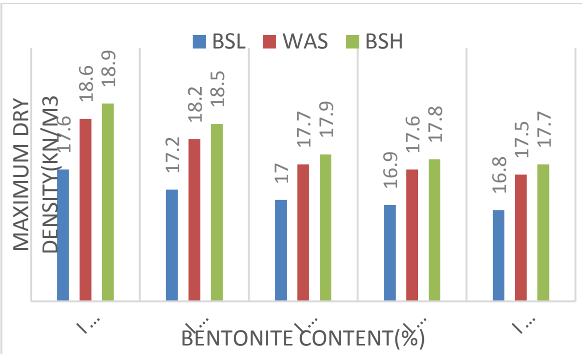

FIGURE 4 VARIATION OF MAXIMUM DRY DENSITY OF IOT TREATED WITH 8%CEMENT AND VARYING BENTONITE CONTENT

Discussion and interpretations model is based on the coefficient of correlation, r and coefficient of determination R^2. Value of coefficient of determination ,R^2 indicates the representativeness of the model. The model is more representative of R^2 which is closer to one. The linear coefficient of correlation, r is a measure of correlation strength between variables x and y. According to Vukadinovic, if $r \leq 0.3$, there is no significant correlation, if $0.5 < r < 0.7$correlation is significant, when $0.7 < r < 0.9$ correlation is strong, in the case where $r > 0.9$ very strong correlation (Vukadinović S,1990). So if the correlation coefficient is closer to one,the correlation is stronger. From the coefficient of determination, R^2

obtained in Figure 5, the value of R2 is 0.8706 which is closer to one, and the coefficient of correlation, r , gives 0.932 which depicts that the coefficient of correlation is strong

From the result in Figure 5, it was clearly seen that an increase in liquid limit increased the moisture content for both the measured and predicted values. Coefficient of determination, R^2 for both measured and predicted values was 0.8111 and 0.7282 respectively which agreed with (Vukadinović 1990). The difference in the values is minimal showing that the representativeness of the predicted values may be closer.

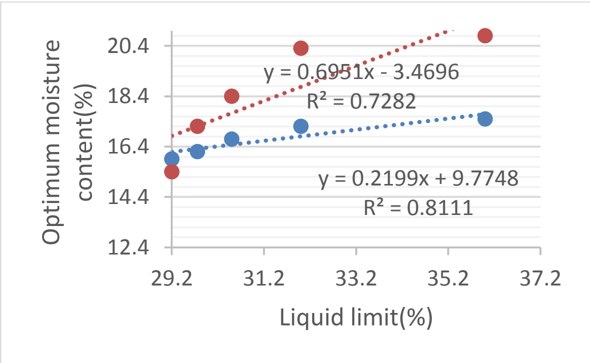

FIGURE 5 GRAPH OF OPTIMUM MOISTURE CONTENT AND LIQUID LIMIT OF IOT TREATED WITH 8%CEMENT AND VARYING BENTONITE CONTENT

The results of the graph in Figure 6 showed the variation of optimum moisture content with plasticity index. Both slopes of measured and predicted values give rise to almost from the same intercept as their R^2 is 0.9716 and 0.9993 respectively. From the coefficient of determination R^2, It can be concluded that, the equations proposed by the authors After (Draper and Smith, 1994, Vukadinović S,1990) can be adopted for the determination OMC for the energy levels (BSL, WAS, and BSH)

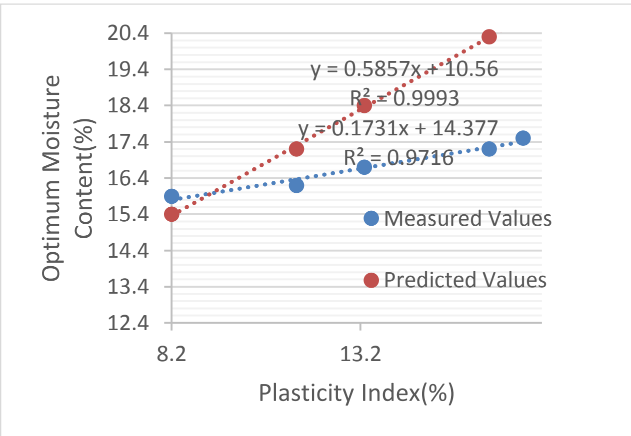

FIGURE 6 GRAPH OF OPTIMUM MOISTURE CONTENT AND PLASTICITY INDEX OF IOT TREATED WITH 8%CEMENT AND VARYING BENTONITE CONTENT.

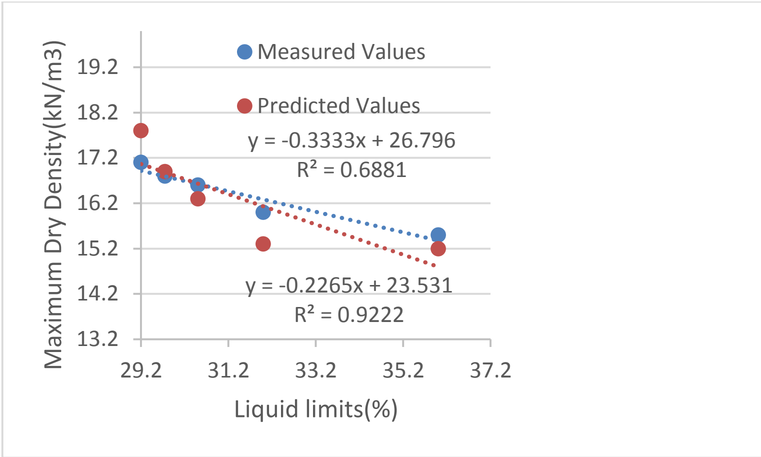

FIGURE 7 GRAPH OF OPTIMUM MOISTURE CONTENT AND LIQUID LIMIT OF IOT TREATED WITH 8%CEMENT AND VARYING BENTONITE CONTENT.

Graph of maximum dry density with liquid limit shown in Figure 7 depicted a decrease in MDD as liquid limits increased This may be as a result of high swelling characteristics of bentonite present in the mixtures (Amadi and Eberemu,2013). The representativeness of predicted values using Equation (After Draper and Smith, 1994), R^2 gives strong values, reliable correlation.

The decrease in maximum dry density with increase in plasticity index recorded in Figure 8 was attributed to the prensence of bentonite in the mixtures with high swelling characteristics that could form a gel around the IOT particles. When gel forms around the soil particles, their effective size increases, which causes increase in void volume, (Amadi and Eberemu,2013). Also the representativeness of predicted values of R^2 gives 0.9204.

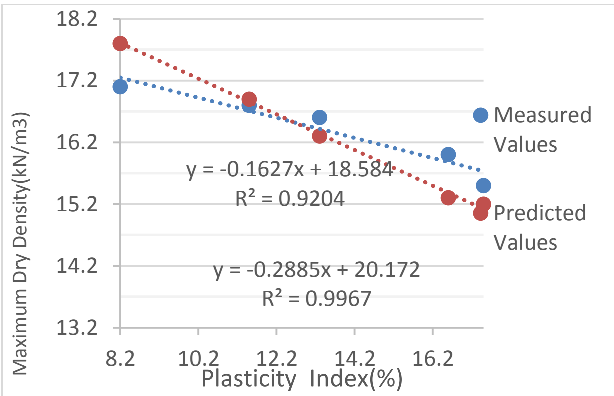

FIGURE 8 GRAPH OF MAXIMUM DRY DENSITY AND PLASTICITY INDEX OF IOT TREATED WITH 8%CEMENT AND VARYING BENTONITE CONTENT.

4 CONCLUSION

It is clearly observed that predicted compaction characteristics correlates well with the laboratory values. It can be concluded that, can be adopted for the determination OMC and MDD for the energy levels

(BSL,WAS,and BSH) for a soil having percentage Liguid limit ranging from 29.2% to 36% and plasticity index from 8.2% to 17.5%. From the obtained laboratory results and the predicted values, the following conclusions were drawn;

1. The compaction characteristics can be correlated with liquid limit and plastic limit for IOT-Bentonite stabilized with cement
2. The study highlights that there is a need for careful determination of geotechnical index properties(Atterberg limits) when IOT is used as major material which can be effectively correlated with compaction characteristics for various energy levels
3. Compaction characteristics can be effectively predicted using geotechnical index properties of soils, thus saving materials,time and money. Hence, these empirical relations would be useful for the engineers in the field to satisfy the quality control as well as for preliminary design and estimation.

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SOIL PROPERTIES OF RESIDUAL SOIL MIXED WITH AGRICULTURAL WASTE ASHES

*Nik Daud, N. N.[1]; Sadan, N. A.[2]; Adesiji, R.[3]; & Anijiofor, S. C.[4]
[1,2]Department of Civil Engineering, Engineering Faculty, Universiti Putra Malaysia, Malaysia
[3]Department of Civil Engineering Federal University of Technology, Minna, Nigeria
[4]Department of Civil Engineering, Federal Polytechnic Birnin Kebbi, Nigeria
*Corresponding author email: niknor@upm.edu.my

ABSTRACT
Agricultural waste ashes have been investigated worldwide by researchers for their ability to be used as an additive in soil stabilization process. To improve the stability of problematic soil, thus ensuring their long term performance especially in foundation purposes is usually a challenge. This paper describes the effect of palm oil fuel ash (POFA) and rice husk ash (RHA) combined with ordinary Portland cement (OPC) as additives to improve the strength properties of residual soil. Results show an improvement in physical and strength properties of stabilized soils for both additives compared to the original residual soil. The optimum condition for both additives is 10% for each mixture with fix 10% OPC.

Keywords: *Palm oil fuel ash; Residual soil; Rice husk ash; Stabilization; Strength.*

1 INTRODUCTION

Construction of any structures on weak or soft soil is difficult without any soil improvement due to their poor shear strength and high compressibility. Several studies on ground improvement techniques using agricultural waste and ashes to stabilize soil indicate that the soil properties such as shear strength and permeability characteristics can be improved (Bayshakhi et al, 2017, Nik Daud et al, 2016a; Nik Daud et al, 2016b).

Subgrade layer is important in road construction. It is the in-situ material upon which the pavement structure is placed. Subgrade layer provides an adequate support and stability to the road pavement. However, there are many types of defects which occur in road pavement such as crack, corrugation and ruts, due to the construction of a road on problematic soil. Previously, traditional soil stabilization using lime or cement is well established. However, despite the fact that this type of stabilization is very famous and has been successful in the past, there is the need to look for other solutions or alternative technologies which are more environmental friendly and economical.

Nowadays, the used of lime or cement are gradually being replaced by industrial or agricultural waste by-product which proved sustainable and provides cost-effective methods to improve the engineering properties of low load bearing or problematic soils. So it is shown that soils stabilized with industrial waste materials have been extensively tested and do not have any adverse environmental impact and consequences. Consequently, in order to reduce the cost of construction of stabilized road subgrade, the use of natural waste such as rice husk ash (RHA) and palm oil fuel ash (POFA) as an alternative additive would be the best solution, towards reducing the environmental hazards caused by cement (Aparna, 2014).

Rice husks are the shells produced during de-husking operation of paddy, which varies from 20% (Mehta, 1986) to 23% (Della et al. 2002) by the weight of paddy. The husk is a waste material and is disposed of either by dumping or burning in the boiler for processing paddy. According to Nair (2006), this ash known as RHA contains silica as a major constituent whose quality (percent of amorphous and unburnt carbon) is influenced by the type of burning process. The RHA is pozzolanic in nature because of its high amorphous silica content (Mehta, 1986). However, RHA cannot be used individually for the soil stabilization due to lack of cementitious properties (Ali et al. 1992). Therefore, it is used along with a binder, for example; cement, lime, lime sludge, and calcium chloride for the stabilization of soil (Ali et al. 1992; Muntohar and Hantoro, 2000; Basha et al. 2005; Sharma et al. 2008; Brooks, 2009).

Palm oil fuel ash (POFA) is one of the most abundantly produced waste materials in tropical regions which has a strong potential to treat physicochemical characteristics of soft soils due to its amorphous nature and high silica content. POFA is widely produced by the oil palm industry through the burning of empty fruit bunches, fiber and palm oil shells as fuel to generate electricity and the waste, collected as ash, becomes POFA. Hypothetically, the large amount of amorphous silica in POFA potentially contributes to the pozzolanic reaction during hydration, which results in the cementitious compound called calcium aluminate hydrates and calcium silicate hydrates. These compounds are responsible for improving the engineering characteristics of soils that increase over time as the pozzolanic reaction develops. However, rice husk ash

(RHA) and palm oil fuel ash (POFA), can only be used as a partial replacement for the more expensive additive Ordinary Portland Cement (OPC) because it has inadequate cementation property.

Therefore, this study focusses on the effect of residual soil stabilization using agriculture waste ash (RHA and POFA), using ordinary Portland cement (OPC) as the binding agent. The specific areas of interest include;

a) To determine the physical and compaction properties of residual soil mixed additives.
b) To investigate and analyse the mechanical properties of residual soil mixed additives.

2 METHODOLOGY

Soil samples were collected within the Universiti Putra Malaysia, Serdang Selangor, Malaysia (GPS: 2°59'19.1"N 101°43'46.8"E). The location is shown in Figure 1.

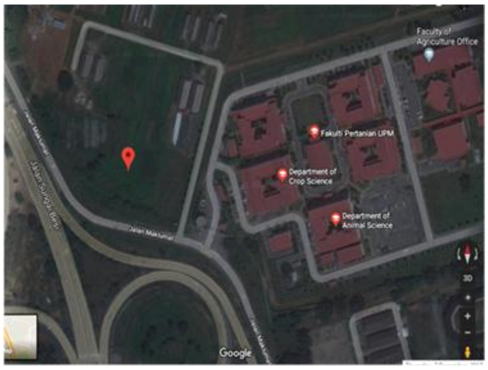

FIGURE 1: LOCATION OF RESIDUAL SOIL SAMPLES COLLECTED FOR THIS STUDY

From observation, the soil sample is categorized as residual soil type. Residual soil is a soil which has been formed in situ by decomposition of parent material, which has not been transported to any significant distance.

2.1 ADDITIVES

The rice husk ash (RHA) sample (Figure 2) was collected from Kilang Bernas Seri Tiram Jaya which is located in Tanjung Karang, Selangor, Malaysia.

FIGURE 2: RICE HUSK ASH (RHA) SAMPLES USED FOR THIS STUDY

Open burning is the process used to form RHA because based on the field observation, RHA is characterized by black color. Meanwhile for this sample, burning was under controlled temperature below 800 °C.

Palm oil fuel ash (POFA) sample (Figure 3) was collected from Tenaga Sulpom Sdn Bhd Lot 3115, Batu 34 Jalan Banting, Dengkil Selangor, Malaysia. The factory used the incineration process for the oil palm waste which resulted in fly ash and bottom ash as their by-product. The fly ash samples were used in this study.

FIGURE 3: PALM OIL FUEL ASH (POFA) SAMPLES USED FOR THIS STUDY

As an extra additive, Ordinary Portland Cement (OPC) was used as a binding agent in a small amount and combined with soil-RHA and soil-POFA sample respectively.

The soil sample was air dried and sieved through 4.75mm sieve aperture in preparation for the test. The soil basic property and soil mechanical property tests were carried out according to BS 1377: 1990. On the other hand, the POFA and RHA samples were kept in sealed plastic bag to protect it from outside moisture content and was subsequently oven dried at 110^{o}C to remove extra moisture. The batch of each sample with and without additive was prepared by adding 5, 10 and 15% of additives.

For each mixture, the compaction tests using standard proctor compaction technique were carried out to determine its maximum dry density and optimum moisture content. The amount of additives as percent of dry soil by weight was mixed thoroughly to produce a homogenous additive-soil mixture. Since there is no standardized protocol for testing additive-soil mixtures, there is a report that compaction of samples should be done two hours after blending to simulate field compaction delay (Acosta et al., 2003). Then, the optimum condition of each soil mixed additive was used to test the strength behaviour.

3 RESULTS AND DISCUSSION

3.1 BASIC PROPERTIES

Basic properties of residual soil sample used in this study are shown in Table 1.

TABLE 1: BASIC PROPERTIES OF RESIDUAL SOIL USED IN THIS STUDY

Parameter	Unit	Value
Initial moisture content, *wc*	%	128
Specific Gravity, G_s	-	2.52
Liquid Limit, *LL*	%	68.7
Plastic Limit, *PL*	%	56.6
Plasticity Index, *PI*	%	12.1
Percent passing BS No. 200 sieve	%	5
Coeff. of Curvature, *Cc*	-	2.5
Coeff. of Uniformity, *Cu*	-	4.44
pH	-	5

The initial moisture content of the soil is about 128% which is very high for normal residual soil due to the rainy season. Meanwhile, the specific gravity is 2.52 which fall in the range of 2.52-2.66, for clay inorganic type of soil. Thus, it can be summarised that the initial moisture content will increase due to the clay content present in it. The percentage of liquid limit, plastic limit, and plasticity index is 68.7%, 56.62% and 12.08%, respectively. According to plasticity chart of Unified System, it indicates that the soil fall in high in silt (MH) or organic (OH) type of soil. The percent of particle which passed through no. 200 sieve is 5% and the Cc and Cu values are 2.5% and 4.44%, respectively. It indicates that the soil is well graded soil, and the Cc fall in the range of 1 to 3. The pH value of about 5, show that the soil sample is moderately acidic.

3.2 COMPACTION CHARACTERISTICS

The characteristics of compaction for residual soil with and without additives were presented in this section. Figure 4, 5 and 6 showed the behaviour of compaction varied by additive with 5, 10 and 15% content; ordinary Portland cement (OPC), palm oil fuel ash (POFA) and rice husk ash (RHA), respectively.

For OPC additive, the sample mixed with 10% showed the optimum condition of the mixture of all samples (Figure 4). The maximum dry density and optimum moisture content for the optimum condition of residual soil mixed 10% OPC is 1.51 Mg/m^3 and 23%, respectively.

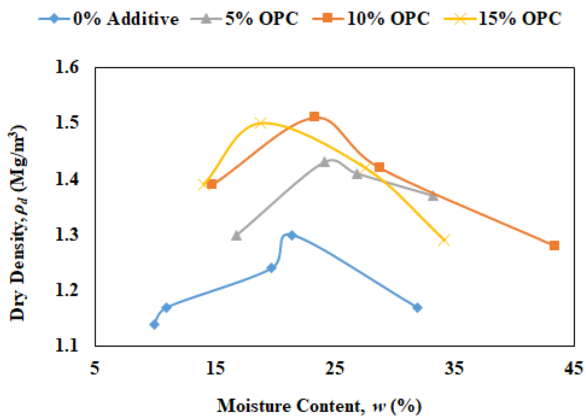

FIGURE 4: COMPACTION CHARACTERISTICS OF OPC VARIED BY PERCENTAGE

Figure 5 shows the maximum dry density and optimum moisture content for the optimum condition of residual soil mixed 10% POFA is 1.54 Mg/m^3 and 16.9%, respectively.

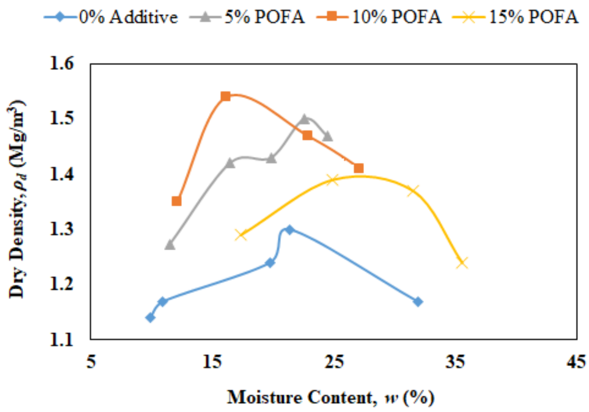

FIGURE 5: COMPACTION CHARACTERISTICS OF POFA VARIED BY PERCENTAGE

Figure 6 shows the maximum dry density and optimum moisture content for the optimum condition of residual soil mixed 10% RHA is 1.5 Mg/m^3 and 21%, respectively.

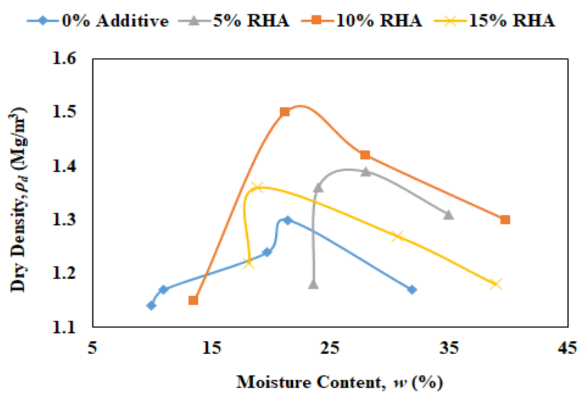

FIGURE 6: COMPACTION CHARACTERISTICS OF RHA VARIED BY PERCENTAGE

Table 2 shows the detailed values of maximum dry density and optimum moisture content for each mixture of different additives. According to the results, in general, the values of maximum dry density for each additive-soil mixture showing an increment pattern if it is compared to the original soil. However, in contrast, the values of optimum moisture content showing a decrement pattern in additive-soil mixture.

TABLE 2: COMPACTION PROPERTIES OF RESIDUAL SOIL WITH AND WITHOUT ADDITIVES

Parameter	0% additive	POFA (%)			RHA (%)			OPC (%)		
		5	10	15	5	10	15	5	10	15
MDD (Mg/m^3)	1.3	1.5	1.54	1.39	1.39	1.5	1.36	1.43	1.51	1.5
OMC (%)	21.42	22.56	16.19	17.35	28.02	21.19	18.82	24.11	23.32	18.8

*MDD – Maximum Dry Density; OMC – Optimum Moisture Content

3.3 STRENGTH PROPERTIES

The strength behavior of residual soil with and without additives at their optimum condition are discussed in this section. The direct shear box test was used to determine the shear strength properties of the samples.

TABLE 3: SHEAR STRENGTH PARAMETERS OF SAMPLES WITH AND WITHOUT ADDITIVES

Parameters	Original Soil	Soil + 10% POFA + 10% OPC	Soil + 10% RHA + 10% OPC
Cohesion, c (kN/m^2)	5	13	9
Internal friction angle, ϕ (^{o})	29	35	32

The values of both shear strength parameters show an increment once mixed with additives.

4 CONCLUSION

This investigation has been successful and shows that both waste materials are suitable for soil stabilization. The optimum condition for both additives is 10% content and the amount of ordinary Portland cement for each mixture is fixed by 10%. The addition of rice husk ash (RHA) and palm oil fuel ash (POFA) with 10% ordinary Portland cement (OPC) indicated an improvement in physical properties and strength properties of residual soil. The RHA and POFA can alter the engineering properties of residual soil.

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INVESTIGATION INTO THE GEOTECHNICAL PROPERTIES OF SELECTED LATERITIC SOILS FROM MINNA AS PAVEMENT MATERIALS

Ob'lama, E.J.[1,2], Amadi, A.A.[2], Alhaji A.A.[2], Alhassan, M.[2], Adejumo, T.W.E.[2]

[1.] Department of Civil Engineering, The Federal Polytechnic, Bida
[2.] Department of Civil Engineering, Federal University of Technology, Minna

Abstract

This study presents an evaluation of some geotechnical properties of selected lateritic soil samples around Minna, Niger State for pavement application. Sixteen (16) lateritic soil samples were collected from five borrow pits within the study area. The samples were subjected to physical, mineralogical, index, compaction and strength properties tests. The samples were further characterized according to AASHTO and USC systems. The natural moisture content values of samples ranged between 2.03-34.24%.Specific gravity values were in the range of 2.47-2.70 while bulk density ranged between 1.76-2.09g/cm^3.The particle size analysis showed that ten(10) of the lateritic soil samples were gap-graded while six(6) were poorly graded. Eleven (11) of the samples had more than 35% of their constituent passing sieve No.200 (0.075 mm), while five (5) of the samples have less than 35% fines content. The compaction relationship of samples compacted at British Standard Light energy gave values between 1.52-1.96g/cm^3 and 9.2 - 20.6% for MDD and OMC respectively. Strength property evaluation of samples revealed an average unsoaked CBR value of 11.78% with a range of 0.1–5 kPa for unconfined compressive strength. Results from XRD showed that all samples except PG3 contain Kaolinite mineral in varying proportion. Quartz mineral were found in the range of 20.83-76.45%.Clay minerals were generally less than 7%. Based on plasticity index and liquid limit requirements, five (5) of the test samples are suitable for base course application, while only one (1) samples met the requirements as sub-grade material. Ten (10) of the samples will require some forms of treatment to improve or remedy their engineering deficiencies in pavement or other applications.

Keywords: *Index Property, Lateritic soils, Marginal materials, Pavements, Strength*

1. INTRODUCTION

The need for cost effective and efficient transportation facilities in any economy cannot be over emphasized. Functional transportation facilities play a vital role in fast tracking socio-economic growth, poverty alleviation and development in general. For developing economies like Nigeria, roads are the most accessible forms of transportation. Sharma (2012) compared roads to the arteries of the human body which are vital for survival. This is because they act as essential catalysts for initiating and sustaining meaningful goals and agenda aimed at development. For a country like Nigeria an approximate land area of 924,763 km^2, huge networks of roads are required to effectively serve the people.

Considering the high cost of building new roads and the frequent structural failures associated with existing roads, development of cost effective standard durable all-weather roads is a major challenge to engineers. Amu *et al. (*2011) noted that though many reasons account for structural failures of many Nigerian roads, the principal cause is attributed to the use of sub-standard material. Eze-Uzoamaka (1981) observed that most aggregates used in pavement construction lack the requisite engineering quality for use. Materials which lack adequate strength cannot withstand traffic stresses especially under varying moisture conditions. The use of such materials gradually leads to their deterioration under working load. These subsequently lead to pavement failures which require frequent maintenance.

The design and construction of durable pavement components with the capacity to withstand loads is a serious challenge to stakeholders in the road industry. This is due to the high cost of road construction using high quality conventional materials. The use of conventional materials are no longer a viable option but the adaptation local materials using relevant techniques which satisfy both traffic and maintenance needs at minimal cost.

Laterites are highly weathered and altered residual soils formed by in-situ weathering and decomposition of rocks under tropical conditions (Elarabi, Taha and Elkhawad; 2013).Lateritic soils are surface soil formations enriched with iron, Aluminum, manganese with some titanium developed by intense and long lasting weathering of parent rock(McFarlane,1976).They are usually reddish, brown or yellowish in colour. In Nigeria, these soil groups are adjourned primary construction materials especially in road construction (Faluyi and Amu, 1989; Uche, 2007; Mttalib, 2008; Mu'azu, 2009; Omotosho and Eze-Uzoamaka, 2008; Amadi, 2010; Ogunribo, 2011; Amu *et al.*, 2011; Osinubi, 1998).

In this vein, diverse researches aimed at exploiting and adopting local available materials like Lateritic soils for construction are ongoing (Osinubi*et al.*, 2017; Ayeni,2016;Manasseh & Joseph, 2015;Dada and Faluyi,2015;Bello *et al.*,2015; Mustapha *et al.*, 2014;Ogunribido,2011;Bwalya,2009).According to Amadi (2010) consideration of lateritic soils for use as construction materials should be based on a thorough evaluation and understanding of their inherent characteristics necessary to predict their engineering performance. Results from such researches possessing the requisite engineering quality will be adopted aimed at providing cheaper, stable and more durable pavements.

Pavements are made up of superimposed components of carefully selected and processed materials. These components are expected to bear and distribute imposed loads in a manner that the natural ground formations are not unduly stressed. For this reason, materials which make up these components are therefore expected to possess minimal engineering specifications. According to Singh (2002), pavements are relatively stable crust of material over the natural soil formation for the purpose of supporting and distributing wheel loads while providing an adequate wearing surface. The designs of pavement are aimed at attaining a structure that is stable, durable and free from any forms of defects. Osinubi (1998) observed that low cost roads are not of poor quality but cost effective and capable of being maintained at low recurring cost.

Lateritic soils are often referred to as *marginal* or *non-standard* materials due to their peculiar behaviour in construction. Marginal materials are usually not wholly in conformity with the specification in use in a country or region for normal road materials. Faluyi and Amu (1989) noted however that most Nigerian lateritic soils contain Kaolinite as predominant clay mineral. They are therefore not out rightly *problematic* soils as portrayed in certain literatures. The most viable option in their use is to alter such materials with the aim of improving specific engineering properties relevant for their use. Such materials can be used successfully with project adaptation (Cook and Gourley, 2002).

Road Note 31 (1977) encourages the use of available local materials with emphasis on reliable results from performance evaluation studies while incorporating any special feature for their satisfactory usage. The key to successful innovative solution is to challenge conventional assumptions with regards to design and construction where locally available materials are used (Cook and Gourley, 2002).

2. MATERIALS AND METHODS

The lateritic soil samples used in this study were obtained from five (5) selected borrow pits around Minna; NigerState**.**Minna is located in North-central Nigeria between latitude 9^0 36' 50" North and longitude 6^0 33' 25" East. Geologically, the study area lies within the Northern central basement complex of Nigeria. The area is characterized by migmatite gneiss complex, older granite and schist (Ajibade, Rahaman and Egezi; 1988).The maximum rainfall per year is between 1000 to 1500mm drained by several rivers which are tributaries of the river Niger (Alhassan and Mustapha, 2012).

A total of sixteen (16) lateritic soil samples were collected by method of disturbed sampling during the dry season. Samples were collected from five (5) different axis around Minna, the capital of Niger State Samples were collected between 0.5-1.0m below the natural ground level. Samples were put in air tight bags, marked and sealed to preserve their natural moisture. Sample labels indicated date of sampling; locations and depth of sample collection. Samples were immediately transported to the Geotechnical Laboratory of the Civil Engineering Department of the Federal University of Technology; Minna (Gidan Kwano campus).Natural moisture content tests were performed on soil samples immediately on arrival at the laboratory. Other tests were conducted in the Soil Mechanics Laboratory of the Federal Polytechnic, Bida, Niger State, Nigeria.

Samples MK1, MK2 and MK3 were collected from active borrow pits at Maikunkele at the northern axis of Minna. Samples labeled JT1, JT2, JT3 were collected from a relatively active borrow pits at Jatai, a village along Minna-Sarkin Pawa road on the eastern axis of Minna. On the Minna-Suleja road axis, three samples labeled PG1, PG2 and PG3 were collected from active borrow pits located at Poggo.Samples LG1, LG2 and LG3 were collected from active borrow pits at Lapia Gwari near Talba farm estate. Four samples; GK1, GK2, GK3 and GK4were collected from the Gidan Kwanu campus of the Federal University of Technology; Minna along Bida-Minna road axis.

2.1 Laboratory evaluation of lateritic soil samples

Laboratory tests were performed in accordance to BS 1377(1990): Methods of test for soil for civil engineering purposes. Physical and geotechnical investigations conducted on collected lateritic soil samples included:

1. Natural moisture content determination of test soil samples.
2. Bulk density determination of soil test samples.
3. Particle density (specific gravity) determination of soil test samples.

4. Particle size distribution of soil test samples (Dry and Wet sieve analysis)
5. Determination of Atterberg limits values of soil samples.
   i. Determination of Liquid Limit, (LL) (Cone penetration method).
   ii. Determination of Plastic Limit, (PL).
   iii. Determination of Plasticity Index, (PI).
   iv. Determination of linear shrinkage values of soil test samples.
5. Determination of density/moisture relationship by the British Standard Light (BSL) method.
6. Determination of California Bearing Ratio (CBR) values for soil samples.
7. Determination of the Unconfined Compressive Strength (UCS) values of soil samples.

3. RESULTS AND DISCUSSION

Results of physical, index, strength and mineralogical investigations produced results discussed as follows. Natural moisture content average value was 11.15% from a range of 2.03 – 34.24 % was obtained. Specific gravity values were within 2.47 – 2.70 with mean value of 2. 61. Bulk density values were between 1.76 – 2.09 g/cm^3 with an average value of 1.89g/cm^3. Details of these test results are summarized in Table I.

3.1Consistency Properties of Lateritic Soil Samples

Consistency limits describe the behaviour of fine grained soils in the presence of moisture. The presence of clay minerals in soils allows them to be remolded in the presence of moisture without crumbling. Consistency or Atterberg limits are defined by Liquid limits, LL; plastic limits, PL and shrinkage limit. Liquid limits, LL and Plastic Limits PL provide a significant way of identifying fine grained cohesive soils. The difference between the LL and PL known as plasticity index, PI.

TABLE 1: PHYSICAL PROPERTIES OF LATERITIC SOIL SAMPLES

Test	Natural moisture content, w (%)	Specific Gravity,(G_S)	Bulk density, ρ_b(g/cm^3)	Colour
MK1	4.75	2.56	2.09	Dark Reddish Grey
MK2	22.34	2.62	2.76	Reddish Grey
MK3	34.24	2.47	2.75	Grey
JT1	2.20	2.50	2.03	Light Grey
JT2	2.08	2.59	1.98	Reddish Yellow
JT3	3.24	2.65	1.96	Redish Brown
PG1	13.9	2.61	1.79	Dusty Red
PG2	12.43	2.62	1.87	Light Red
PG3	11.99	2.63	1.80	Light Reddish Brown
LG1	21.35	2.70	1.86	Light Grey
LG2	20.0	2.65	1.82	Red
LG3	9.16	2.63	1.98	Red
GK1	3.12	2.63	1.96	Reddish Brown
GK2	10.95	2.60	1.83	Reddish Yellow
GK3	6.69	2.58	1.91	Light Reddish Brown
GK4	2.03	2.55	2.02	Yellowish Brown

Osinubi, Eberumu, Bello and Adzegah (2012) observed that soils with too high LL were susceptible to desiccation cracking. Soils with high PL were less workable linear shrinkage values are useful for establishing likely conditions of expansion on wetting of soils hence are useful for quantifying the amount of shrinkage likely to be experienced by clay soil materials. Liquid Limits (LL) values from consistency tests were variable.LLvalues were between 17.7 and 62.2% with mean value of 35.59%. Plastic limits (PL) values of the lateritic soil samples ranged from non-plastic (NP) to 31.6%, and an average value of 17.28%. Plasticity Index values were between 1.2 and 30.6% with average value of 18.3%.The federal ministry of works and housing FMWH(1997) specified a maximum LL value of 35% and 30% respectively for sub-grade and sub-base materials. Corresponding PI values of 12% and 10% is given for sub-grades and sub-bases. For base courses, maximum values of 30 and 10% are specified for LL and PI respectively. Investigations for linear shrinkage reveal an average value of 8.18% within a range of 1.0 – 12.0%.Consistency values are presented in Table II.

TABLE II: **CONSISTENCY VALUES OF LATERITIC SOIL SAMPLES**

Sample No	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI (%)	Linear Shrinkage (LS)	Remarks on Plasticity
MK1	43.6	25.9	17.7	11.5	Med Plasticity
MK2	51.7	30.1	21.6	10.0	High Plasticity
MK3	62.2	31.6	30.6	14.5	High Plasticity
JT1	18.0	16.8	1.2	3.5	Low Plasticity
JT2	25.2	15.7	9.5	6.5	Med Plasticity
JT3	17.8	NP	17.8	3.0	High Plasticity
PG1	30.0	16.5	13.5	11.0	Med Plasticity
PG2	34.8	21.9	12.9	9.0	Med Plasticity
PG3	39.1	24.6	14.5	11.0	Med Plasticity
LG1	35.1	NP	35.1	1.0	High Plasticity
LG2	41.4	11.4	30.0	10.0	High Plasticity
LG3	54.0	24.7	29.3	12.0	High Plasticity
GK1	17.7	NP	17.7	3.0	High Plasticity
GK2	44.6	26.5	18.1	9.0	High Plasticity
GK3	26.3	16.2	10.1	7.0	Med Plasticity
GK4	28.0	14.5	13.5	9.0	Med Plasticity

NP= Non-Plastic

3.2 Gradation Characteristics of Lateritic Soil Samples

The aim of any gradation analysis is to determine the particle sizes distribution of soil samples. The resultant grading curves are graphical representation of the particle size distribution. They are useful means of describing a soil. They are the basis for soil classification and a means of predicting their behaviour. Figure 1 shows the obtained grading curves from the particle size analysis test. Twelve of the soil samples had more than 35% of their constituents passing the BS 200 sieve. The gradation curves show that samples were either poorly graded or gap graded as noted by Amu *et al* (2011). Samples MK_1, JT3, LG1 and GK_1 had less than 35% of their constituents passing the 75µm sieve. SamplesMK_1, JT3, LG1 and GK_1 had fines content of 24.5%, 30.1%, 31.3% and 32.65% respectively. Sample MK_3 is gap graded with the most fines content of 90.1%.

TABLE III:FINES CONTENT OF LATERITIC SOIL SAMPLES

Sample Label	Percentage Passing BS sieve No 200(%)	Remarks on Grading Curve	Predominant constituent Materials
MK1	24.5	Gap graded	Sandy gravel
MK2	76.8	Gap graded	Sandy Gravel
MK3	90.1	Gap graded	Clayey soil
JT1	36.4	Poorly graded	Sandy gravel
JT2	41.5	Poorly graded	Sandy gravel
JT3	30.1	Poorly graded	Sandy gravel
PG1	54.5	Gap graded	Sandy soil
PG2	61.1	Gap graded	Sandy soil
PG3	70.1	Gap graded	Sandy soil
LG1	31.3	Gap graded	Sandy soil
LG2	66.7	Poorly graded	Sandy soil
LG3	63.3	Gap graded	Sandy soil
GK1	32.6	Gap graded	Gravel Sand
GK2	64.3	Poorly graded	Sandy gravel
GK3	56.0	Gap graded	Gravel Sand
GK4	40.4	Poorly graded	Gravel Sand

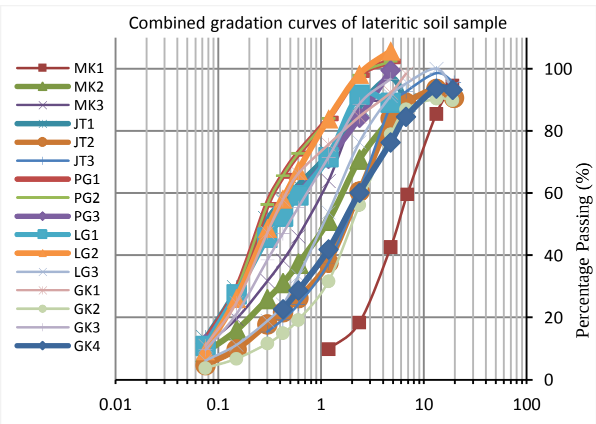

FIGURE 1: COMBINED GRADATION CURVES OF TEST SAMPLES

3.3 Compaction Characteristics of lateritic soil samples.

Compaction is an indication of the state of stability of a soil for construction. Compaction tests are used to establish the dry density/moisture content relationship of soils under controlled conditions used as a standard for comparison in field condition. The compaction characteristics of the individual lateritic soil samples were determined in the laboratory by the British Standard Light (BSL) method. The compaction characteristics also referred to as the moisture density relationship included maximum dry density (MDD) and optimum moisture content (OMC).The range of MDD values were between 1.52 – 1.96g/cm^3 with mean value of 1.77g/cm^3.The mean optimum moisture content was 14.68% from a range values of 9.2% – 20.6% as shown in Table III.

TABLE III: COMPACTION PROPERTIES AND STRENGTH VALUES OF LATERITIC SOIL SAMPLES

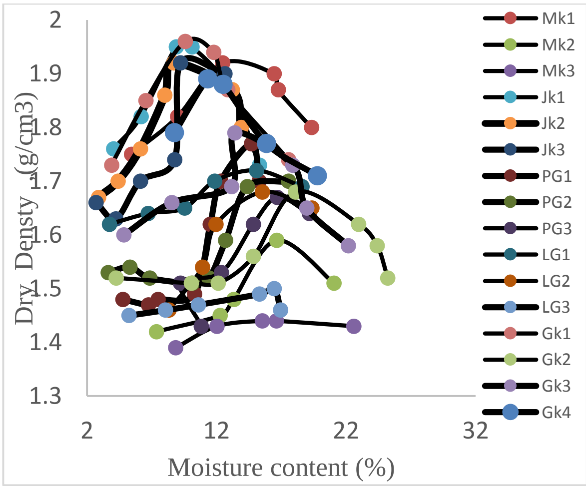

FIGURE 2: COMBINED PLOTS OF MOISTURE/DENSITY RELATIONSHIP OF LATERITIC SOIL SAMPLES

3.4 Strength characteristics of test samples

Strength is an important property of any soil. Two strength characteristics were evaluated. They include California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS).

The California bearing ratio (CBR) test is a semi-empirical test used in the estimation of the strength of or bearing capacities of pavement components. It measures the resistance which is equivalent to the difficulty to penetration under specified density and moisture conditions. The CBR rating is proportional to the resistance to penetration. The average unsoaked CBR value is 11.78% from a range value of 1.12-26.02%. These are below the FMWH Specification of 30% for non-dry soil purpose for use as sub- base material.

The maximum value of compressive force per unit area which a soil specimen can sustain is referred to as the unconfined compressive strength of the soil. The method adopted in this study is that of the definitive load frame. Values of Unconfined Compressive Strength were in the range of 0.1 – 5 KPa.

Table IV: COMPACTION AND STRENGTH PROPERTIES RESULTS

Sample Identity	Compaction Characteristic Tests		Strength Property Tests	
	MDD(g/cm^3)	OMC (%)	CBR (%)	UCS (kPa)
MK1	1.92	14.6	12.84	4.71
MK2	1.61	20.6	1.12	3.92
MK3	1.52	17.0	1.12	0.10
JT1	1.93	9.2	20.70	1.37
JT2	1.93	10.0	15.08	3.82
JT3	1.96	9.6	26.02	5.00
PG1	1.75	14.7	10.99	3.92
PG2	1.70	17.0	12.90	3.82
PG3	1.68	17.6	6.55	2.35
LG1	1.72	15.0	12.40	1.08
LG2	1.69	16.0	19.84	3.14
LG3	1.62	20.3	12.71	3.14
GK1	1.96	9.6	20.83	2.16
GK2	1.68	18.0	15.22	3.33
GK3	1.79	14.0	2.81	2.45
GK4	1.89	11.6	11.12	2.45

3.5 Sample Identification and classification

The Lateritic soil samples were classified according to both AASHTO and Unified Soil Classification System (USCS).Three (3) of the soil samples; MK1, JT3 and GK1 can be classified as A–2–6 soils. These groups of soil according to USC can be classified as GC or SC soils. Only one test sample, LG1 fell into A-3 group of soils in accordance to AASHTO, corresponding to SP on USCS. Two (2) soil samples; JT2 and GK3 were grouped as A–4 soils class based on AASTO classification which corresponds to MH or OH on the USCS. Four soil samples; JT1, PG1, PG2 and GK4were classified as CL and A – 6on the USCS and AASHTO classification schemes respectively. Based on AASHTO classification schemes, samples MK2 and MK3 can be classified as A–7–5 which corresponds to OH or MH in the USCS. Four samples, samples JT3, LG2, LG3 and GK2 are A–7–6 which according to USCS can either be GC or SC as shown in Table V. Classification and pavement rating of test lateritic samples are shown in Table VI.

TABLE V: THE LATERITIC SOIL SAMPLES ACCORDING TO THEIR CLASSES AND PAVEMENT RATING

Sample Label	Soil Classification	
	AASHTO	**USCS**
MK1	A-2-6	GC,SC
MK2	A-7-5	OH,MH
MK3	A-7-5	OH,MH
JT1	A-6	CL
JT2	A-4	MH
JT3	A-2-6	OH
PG1	A-6	GC,SC
PG2	A-6	CL
PG3	A-6	CL
LG1	A-3	SM
LG2	A-7-6	CH,CL
LG3	A-7-6	CH,CL
GK1	A-2-6	ML,OL
GK2	A-7-6	CH,CL
GK3	A-4	ML,OL
GK4	A-6	CL

3.6 Mineralogical Composition of Lateritic Soil Samples

Results from X-ray diffraction, (XRD) gave a wide range of mineralogical constituents in varying proportion. These minerals include; Kaolinite, Montmorillonite Albite, Albite (calcian low), Muscovite, Halloysite, Illite, Potassium Aluminum Silicate Hydrate, Sodium Magnesium Aluminium Oxide, Beidellite, Sepiolite, Silicon and quartz. Quartz being the predominant mineral with a range value of 20.0% -93.2% as indicated in Table VII. The quantity of Kaolinite for the samples were in the range of 4.17-7.78%.Sample LG2 contained 8.33% Montmorillonite clay mineral. Sample PG3 contained Illite and Halloysite clay minerals each in 2.08% proportion. Sample MK3, a highly plastic gap-graded clayey soil with a natural moisture content equivalent to 34.24% and a fines content of 90.1% contain 1.45% muscovite mineral.

TABLE VI:AASHTO CLASSIFICATION AND RATING OF TEST SAMPLES

Sample Label	Soil Class	Typical Soil description of soil class	General Rating as Sub-grade Materials
MK1,GK1,JT3	A-2-6	Silty Clay materials(More than 35% of total sample passing No,200 sieve	Excellent to good materials
LG1	A-3	Fine sands with silty or clay fines or small amount of non-plastic silt	Excellent to good materials
JT2.GK3	A-4	Non-plastic or moderately plastic silty soils	Fair to Poor materials
PG1,PG2,PG3	A-6	Plastic clays having 75% or more fines.	Fair to poor materials
JT1,GK4 MK2,MK3	A-7-5	Clay materials with moderate PI in relation to LL.	Fair to poor materials
LG2,LG3,GK2	A-7-6	Materials have high PI in relation to LL.	Fair to poor materials

TABLE VII: MINERALOGICAL COMPOSITION OF SELECED LATERITIC

Name of Mineral	Quantities (%)					
Sample name	MK3	JT2	PG3	LG1	LG2	GK3
Quartz SiO_2	70.45	50.40	20.83	70.45	76.45	50.0
Kaolinite $Al_2(Si_2O_5)(OH)_4$	6.38	4.20	-	6.38	6.38	4.17
Kaolinite IMD $Al_2Si_2O_5(OH)_4$	4.77	1.57	-	2.39	4.77	1.56
Silicon	2.08	--	-	--	-	-
Albebite $NaAlSi_3O_8$	-	4.20	10.42	25.0	3.1	4.17
Halloysite	-	-	2.08	-	-	-
Illite	-	-	2.08	-	-	-
Potasium Aluminium Silicate Hydrate K10OA	-	-	8.33	6.08	-	-
Sodium Magnesium Aluminium Oxide				3.12		
Montmorillonite					8.33	
Muscuvite	-	-	-	-	-	1.45
Beidellite	-	-	-	1.21	-	-
Sepiolite	-	-	-	2.51	-	

4 CONCLUSIONS

1. The lateritic soil samples studied included soils of A-2-6, A-3, A-4, A-6, A-2-5 and A-7-6 soil groups on the AASHTO classification system. These correspond to GC, MH, CL, SP, CH and SC according to Unified Soil Classification Scheme (USCS).
2. Based on plasticity index and liquid limit requirements, five (5) of the test samples are suitable for application as pavement components. Only sample JT3 met the requirements for use as base course material. Ten (10) of the samples will require some forms of treatment to improve or remedy their engineering deficiencies.
3. The average moisture content value for the lateritic soil samples is 11.5% Values of specific gravity ranged 2.47-2.7 while that for bulk density is between 1.76-2.09g/cm^3.Other Index property values were 17.7-62.25% for LL.. Plasticity ranged from low to high plasticity with linear shrinkage value of 8.18 % on the average.
4. The mean Optimum Moisture Content (OMC) value for compacted samples is 14.7% average Maximum Dry Density value of 1.77g/cm^3. Strength properties evaluation showed that the lateritic soil samples had average unsoaked CBR values of 11.78% with unconfined compressive strength in the range 0.1–5 kPa.

5. Mineralogical content evaluations from X-ray diffraction (XRD) analysis for the test samples revealed clay minerals such as Montmorillonite,Halloysite,Muscuvite and Illite.Only sample LG2 contained Montmorillonite mineral at a value of 8.33%.The quartz content for the soil samples were in the range of 20.83-76.45%.
6. Ten (10) samples, MK2, MK3, JT2, PG1,PG2,PG3,LG1,LG2,LG3 and GK3 did not meet the requirement for use as pavement materials. For effective utilization, they need to be treated to meet minimum specifications for use as pavement components.

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EFFECT OF LOCUST BEAN POD EPICARP ASH (LBPEA) ON THE COMPRESSIVE STRENGTH OF REVIBRATED CONCRETE

Kabiru Adebayo[1] and Samuel M. Auta[2]
[1,2]Department of Civil Engineering, Federal University of Technology, Minna, Nigeria
E-mail: bayo997@gmail.com

ABSTRACT

The effect of locust bean pod epicarp ash (LBPEA) on the compressive strength of re-vibrated concrete is presented. The LBPEA used was obtained by controlled incineration of locust bean pod epicarp in an enclosed furnace. The result of the chemical analysis carried out on the LBPEA showed that it is a very good pozzolana with its SiO_2, Al_2O_3, and Fe_2O_3 content percentage summation at 77.81%. Using a concrete mix ratio of 1:2:4 and a water-binder ratio of 0.6, a total of 168 cubes were cast and cured for 7, 14 and 28 days: 42 for control (0% LBPEA + 100% OPC); and 21 cubes for each other different percentages (5%, 10%, 15%, 20%, 25%, and 30%) replacements of OPC. Each category of cubes was initially vibrated and then re-vibrated at an interval of 10 minutes up till 1 hour. Compressive strength of the concrete cubes were determined at 7, 14 and 28 days curing age. The compressive strength increased gradually with curing age and a maximum compressive strength of 28.44 N/mm^2 at 60 minutes revibration was obtained at 28days for 5% of LBPEA. At 14 days curing, a maximum compressive strength of 24.76 N/mm^2 at 60 minutes revibration was obtained for 0% of LBPEA. At 7 days curing, a maximum compressive strength of 25.29 N/mm^2 at 60 minutes revibration was obtained for 0% of LBPEA. This suggests that 5% replacement of OPC with LBPEA can be adopted for structural concrete production.

Keywords: locust bean pod epicarp ash, pozzolana, revibrated concrete, compressive strength.

1 INTRODUCTION

The over reliance on the usage of concrete produced from cement (as binder) for buildings have kept their cost high and this has deterred average man in underdeveloped nations of the world from building houses for their local dwellers that make up the greater percentage of their population and most often are agriculturally dependent (Aguwa *et al.*, 2016). As a result of the high cost of acquiring the required materials to construct functional and stable houses, greater proportion of the country's population cannot afford the cost (Aguwa, 2009). An easy way out of this, points at suitable means of replacing a proportion of cement with cheap and easily obtainable pozzolanic materials. ASTM C618 (2012) defined pozzolana as "siliceous or siliceous and aluminous material which in themselves have little or no cementitious properties but in finely divided form and in the presence of moisture, react with calcium hydroxide which is liberated during the hydration of Portland cement at ordinary temperatures to form compounds possessing cementitious properties". The global increase in human population has triggered an increase in the demand for scarce and expensive building materials for building construction. Also, measures suitable to safely dispose huge agricultural wastes produced by man from involvement in farming for food are demanded, measures such as disposal of these agricultural wastes on land; causes land pollution, when disposed into water bodies; pose dangers to aquatic animals and consequently man suffers from low aquatic animals available for consumption, when the option of burning the wastes pulps up; man also is seen to be affected by air pollution and from depleting ozone layer.

Concrete according to Anzar (2015), is that pourable (when fresh) composite mix of binder (cement), water, fine aggregates (mostly sand), and coarse aggregates (gravel) that hardens or sets (when dry) into a super-strong material for building. When concrete is re-vibrated, it momentarily liquefies again. The primary chemical process that occurs in the first 2 hours after concrete is placed is the formation of calcium hydroxide, which typically makes up 15 to 25 percent of ordinary Portland cement concrete. The other major product of hydration is calcium silicate hydrate, which usually constitute about 50% of OPC concrete which gives the concrete its hardness and durability. Initial vibration of concrete may not eliminate defects such as honey comb and voids causing reduction in strength and performance. But re-vibration can eliminate such defects (honey comb and voids) and thereby increasing bond, improving concrete quality, better impermeability, reduction in shrinkage and creep, increasing the compressive strength of the concrete, reduction in surface and other voids as well as cracks in fresh concrete and so on (Krishna *et al.*, 2008). Re-vibration can be done usually at any time as long as the running internal vibrator can sink by its own weight into the concrete before the final setting time of the concrete is reached [(Krishna *et al.*, 2008), (Auta, 2011)]. Re-vibration time lag is one of the major factors that can affect the compressive strength of concrete. Krishna *et al.*, (2008) Recommended a minimum re-vibration time lag interval of 30 minutes to 4 hours for different w/c ratio, while in another separate work Auta (2011) reported that the effect of re-vibration on the strength of concrete was dynamic. A research conducted suggests that re-vibration enhances the strength of concrete once done when the concrete is still plastic. Also it was clear that flexural strength of Rice Husk Ash (RHA) concrete increases at the early stage of re-vibration, the flexural strength of RHA concrete decreases from 30 minutes to 60 minutes of re-vibration almost for all percentage replacement level of RHA. Further observations showed that re-vibration on early age retarded concrete in which the result shows that the maximum compressive strength was achieved when the concrete was re-vibrated and cured at older age, thus re-vibration improves many of the quality of hardened concrete [(Auta, 2011), (Auta *et al.*, 2011), (Kassim, 2012)]. Another investigation questioned the effect of re-vibration on compressive strength of concrete and concluded that re-vibration resulted in improvement of compressive strength when carried out within the initial setting time confirming that good re-vibration within standards increases compressive strength [(Krishna *et al.*, 2008), (Auta, 2011), (Dunham *et al.*, 2007)]. The effect of re-vibration on the compressive strength of 56 days aged RHA-cement concrete has been investigated and presented. Re-vibration was seen to increase the compressive strength of OPC and OPC-RHA concrete by completely eliminating void, provided it is carried out within the setting time of concrete [(Krishna *et al.*, 2008), (Auta *et al.*, 2011)]. The compressive strength of the re-vibrated OPC-RHA at 56 days was generally observed to be higher than the re-vibrated and non-re-vibrated OPC concrete at 28 days curing. It is thus recommended to re-vibrate the concrete up to 20minutes and with reasonable replacement of up to 10% RHA for cement.

Pozzolana is classified into two groups (Neville and Brooks, 2002); natural and artificial. Though the natural and artificial classes have similar pozzolana activity, they differ slightly in chemical and mineralogical constituents, but greatly from one another in origin. A number of biomass residues are increasingly being identified as pozzolanas. Utilization of Rice Husk Ash is particularly well improved in the rice producing regions of the United States and Thailand; rice husk is first used as fuel in the rice parboiling plants, with further utilization of the ash residue as pozzolana in making special quality cement concrete (Chungsangunsit, 2009). Other biomass residues which have been identified as having pozzolanic properties include Bamboo Leaf Ash (Dwivedi *et al.*, 2006), Palm Fruit Ash (Colonnade, 2010), Locust bean pod epicarp ash (Adama and Jimoh, 2011) and Corn Cob Ash (Adesanya and Raheem, 2009). Low-cost agro-wastes are also being investigated as partial OPC replacement materials because of their pozzolanic properties. These materials might also warrant further investigation in the development of regional hydraulic-lime pozzolana concretes (HLPCs): rice-husk ash (Chao-Lung *et al.*, 2011), sugar cane bagasse (Cordeiro *et al.*, 2008), saw dust ash (Elinwa and Mahmood, 2002), corn cob ash (Adesanya and Raheem, 2009), coconut husk ash (Ettu *et al.*, 2013), Locust bean pod ash (Adama and Jimoh, 2012), cassava waste ash (Ettu *et al.*, 2013), olive waste ash (Cruz-Yusta *et al.*, 2011) and periwinkle, oyster and snail shell ash (Etuk *et al.*, 2012). Aguwa and Okafor (2012) reported that the locust bean pods epicarp (Figure 1) were spread over mud walls and as soon as rain begins to fall on the pods, the leachate percolates down the wall. These buildings and fence walls have been found by the natives to withstand over a long period of time under varying weather conditions such as rains, wind and heat. The African locust bean (Parkia biglobosa) has a wide distribution ranging across the Sudan and western coast of Africa in Senegal. Concentrated locust bean pod extract is used to impart water resiliency to floors, walls and ceramics pot. The tannins present in the epicarp act to bind the soil by their polymeric nature and render the surface impervious to water, sealant to pot and creates a dark, mottled surface.

FIGURE 1: LOCUST BEAN POD EPICARP

When rice husk is burnt under controlled conditions, the RHA is highly pozzolanic, but when in an uncontrolled manner, it was concluded that the ash which is essentially silica is converted to crystalline forms and become less reactive (Ogunbode *et al.*, 2011). However, when blended with cement to produce concrete, it is observed to be highly pozzolanic. Ndububa and Uloko (2015) obtained LBPEA by incinerating locust bean pod peel up to 600ºC using a kiln fueled by kerosene. The ash was allowed to cool, then grinded and sieved through sieve 0.250 ASTM micro sieve. Cement was replaced with 5 to 25% LBPEA and the result showed that the higher the ash content, the less the compressive strength value. Ndububa and Uloko (2015) concluded that the decrease in strength was due to the percentage of (SiO_2 + Al_2O_3 + Fe_2O_3) content of the ash (63.57%) which was less than 70% as required for any material to behave as a pozzolana although they later maintained that the compressive strength of the cement-LBPEA concrete is adequate enough for use as a load bearing building material. According to Auta *et al.*, (2011) the use of RHA as a partial replacement for cement in concrete is not recommended when re-vibration is not envisaged as it results in concrete having very low compressive strength at all ages of curing. An average duration of 90 minutes is recommended for re-vibration of concrete with 20 % RHA as cement replacement for optimal or higher compressive strength. An investigation was conducted on the usage of wood waste ash as a partial cement replacement material in the production of structural grade concrete and mortar, assessment of the fresh concrete properties of self-compacting concrete containing SDA, and it was evident that ash from timber waste was a material capable of replacing cement (Elinwa *et al.*, 2008).

Much work has not been reported on the behavior of re-vibrated concrete produced from Locust bean pod epicarp ash (LBPEA) as partial replacement for cement. As such, the aim of this study is to examine the effect of LBPEA on compressive strength of re-vibrated concrete cubes made from 5%, 10%, 15%, 20%, 25% and 30% replacements of OPC.

2 MATERIALS AND METHODS

Materials which were used in the laboratory experiments were subjected to tests of their physical, chemical and engineering properties with the ultimate aim of determining the compressive strength of re-vibrated concrete using 5% to 30% of LBPEA as a replacement for OPC.

2.1 MATERIALS

The materials for this research work include:

OPC: The binder used for this work was Ordinary Portland Cement grade 43 (Dangote Cement), obtained from a cement depot at Mobile area of Minna, Niger State, conforming to BS 12:1996 was used;

Fine aggregates: Clean sharp sand of maximum size 5mm collected from a nearby building material dealer in Minna, Niger State conforming with the provisions of BS EN 12620: 2013 was used.

Coarse aggregates: Coarse aggregates of maximum size 20mm collected from a nearby building material dealer in Minna, Niger State conforming with the provisions of BS EN 12620: 2013 was used.

Locust bean pod epicarp ash (LBPEA): Clean dry Locust bean pod epicarp (Figure 1) collected from trees at Kangi village in Bida local government area and around Gidan kwano campus of the Federal University of Technology-Minna, both in Niger state. The pods were incinerated using a furnace at the Civil Engineering laboratory of the Federal University of Technology, Minna. The ashes (Figure 2) obtained, were passed through sieve 75μm. The chemical analysis of the LBPEA (Figure 2) was conducted at chemistry laboratory and the result presented in Table 1.

Water: Potable tap water from the borehole provided near the Civil Engineering laboratory of the Federal University of Technology, Minna (Gidan Kwano campus), Niger state was used.

FIGURE 2: LOCUST BEAN POD EPICARP ASH (LBPEA)

2.2 METHODS

For the purpose of classification and checking compliance, the following laboratory tests were carried out on the aggregates: Specific gravity test; Bulk density, porosity and void ratio test (BS 812-2:1995); Moisture content test (BS 812-109:1990); Particle size distribution (sieve analysis) test.

Component materials for concrete were mixed using the Absolute Volume Method and the concrete were cast into moulds of dimension 150 × 150 × 150mm. The fresh concrete in the moulds were vibrated immediately and re-vibrated at 10 minutes' intervals for 1 hour. Slump test was conducted on the fresh concrete to check workability. Water curing method was used to cure the concrete produced. Compressive strength of the concrete were obtained using a compressive testing machine at ages of 7, 21 and 28days of curing.

2.3 CONCRETE PRODUCTION

A thin layer of mineral oil was applied to the inside surfaces of the moulds in order to prevent bond between the concrete and the mould. Concrete were then poured in the mould in 3 layers, each layer being compacted by 25 strokes of 16mm diameter steel rod with a bullet end. Thereafter, the top surface was finished using a trowel, the compacted cubes were vibrated and re-vibrated on the vibrating machine. The cubes were then properly stored and allowed to harden. The hardened cubes were de-moulded after 24 hours.

2.4 VIBRATION AND RE-VIBRATION OF CONCRETE SAMPLES

The time lag used for the study was 60 minutes. The duration of each initial vibration was 40 seconds and subsequent re-vibration lasted for 20 seconds each with intervals of 10 minutes between successive re-vibrations. 42 numbers of 150 x 150 x 150mm cubes were cast using

Ordinary Portland Cement alone as binder (control), representing 2 cubes per test for 7-time lag intervals namely: 0 min, 10 min, 20 min, 30 min, 40 min, 50 min and 60 min, for three ages (7, 14, 21 days) for crushing. Additional 21 cubes were cast using a binary blend of Ordinary Portland Cement and LBPEA.

A Humboldt 800mm x 400mm 55-C0160/H vibrating table fitted with a clamping device and waterproof pedal switch was used for both initial vibration and subsequent re-vibrations. It is capable of vibrating two 150 mm cube moulds simultaneously.

2.5 CURING OF CONCRETE CUBES

Considering that the cement requires time to fully hydrate before it acquires strength and hardness, concrete must be cured once it has formed and achieved initial setting. Curing is the process of keeping or storing concrete under a suitable specific environmental condition until hydration is relatively complete. The environment promotes hydration after the moulds are removed in a process. Although before striking, it is assumed that the concrete should have formed, hardened and set after 24 hours of casting. The concrete specimens were cured for 7, 14 and 28 days after which they are removed for crushing.

2.6 COMPRESSIVE STRENGTH TEST

The weights of the samples were taken before the compressive strength test was conducted. The cubes (both with and without LBPEA vibrated and re-vibrated from zero to one hour) were removed from the curing tank after 7, 14 and 28 days of curing consecutively. On the designated days for crushing, twenty-eight (28) sample cubes (i.e. 14 without LBPEA and another 14 with LBPEA) were crushed. The maximum load carried by each specimen before failure was recorded. The maximum load divided by the net surface area of the specimen gave the compressive strength of the concrete.

The test result for the compressive strength of hardened re-vibrated concrete for the different percentages of replacement of OPC with LBPEA at ages 7, 14 and 28 days of curing are shown in Tables 2, 3 and 4

3 RESULTS AND DISCUSSIONS

Result of the chemical analysis conducted on the LBPEA is presented in table 1.

TABLE 1: LABORATORY CHEMICAL ANALYSIS OF LBPEA

Chemical	Content (%)
Na_2O	0.926
K_2O	2.626
MgO	3.659
Pb_2O_5	3.449
Fe_2O_3	3.365
Al_2O_3	9.785
CaO	7.416
SiO_2	64.659
SO_3	1.990
Cl	0.791
TiO_2	1.068
Cr_2O_3	0.002
Mn_2O_3	0.183
ZnO	0.021
SrO	0.060

The result of the chemical analysis carried out on the LBPEA showed that it is a very good pozzolana with its SiO_2, Al_2O_3, and Fe_2O_3 content summing to 77.81% as required by ASTM C618 (2012). As such, the usage of the LBPEA in this work is justified.

The ash content of the LBPE was obtained as 2.83% showing that only that much percentage of the burnt LBPE is useful for this work as it is the portion that passed sieve 75µm as required for cement fineness.

Results of the specific gravity test; Bulk density, porosity and void ratio test; Moisture content test; Particle size distribution (sieve analysis) test actually demonstrated that the aggregates were standard within specifications. However, the moisture content of the LBPEA was 21.07% indicating that it requires to be dried out before incinerating.

The sieve analysis result of the fine aggregates showed that it is chiefly composed of sand passing sieve 5.0mm and retained on sieve 0.15mm. For the coarse aggregates, bulk of the materials were seen to pass sieve 20.0mm and retained on sieve 10.0mm.

TABLE 2: EFFECT OF LBPEA ON THE COMPRESSIVE STRENGTH OF RE-VIBRATED CONCRETE AT DAY 7

Revibration time interval (mins)	Compressive strength, N/mm^2						
	0%LBPEA+ 100%OPC (Control)	5%LBPEA + 95%OPC (5L95C)	10%LBPEA + 90%OPC (10L90C)	15%LBPEA+ 85%OPC (15L85C)	20%LBPEA + 80%OPC (20L80C)	25%LBPEA + 75%OPC (25L75C)	30%LBPEA + 70%OPC (30L70C)
0	18.87	17.33	17.07	11.51	5.78	5.33	4.93
10	18.60	18.89	13.87	10.84	6.49	5.87	5.24
20	20.40	17.51	12.89	11.47	7.38	6.40	5.33
30	19.93	18.76	13.33	11.69	7.33	6.67	5.56
40	18.11	17.07	12.84	11.78	7.42	6.98	5.78
50	23.98	18.84	14.09	12.67	8.00	6.89	5.47
60	25.29	21.11	15.64	12.62	7.82	7.56	6.13

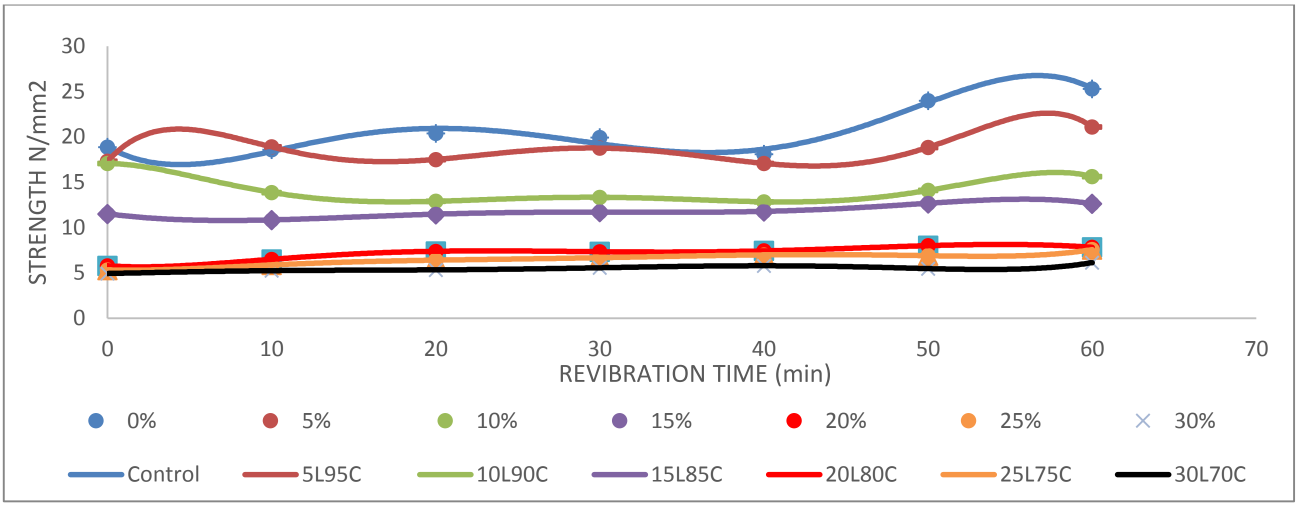

FIGURE 3: EFFECT OF LBPEA ON THE COMPRESSIVE STRENGTH OF RE-VIBRATED CONCRETE AT DAY 7

The trends of the effect of LBPEA on the day 7 compressive strength of re-vibrated concrete at various re-vibration time are shown in Figure 3. There was a sinusoidal behavior in the strength of the concrete at different re-vibration time. The maximum strength at day 7 was obtained at the control stage to be 25.29 N/mm^2 at 60 minutes re-vibration time. Closest to this value is that obtained at 5% ash replacement at 60 minutes re-vibration gotten as 21.11 N/mm^2. Other replacement percentages were seen to have relatively low strength compared to the control and 5% replacement at almost all the re-vibration time.

TABLE 3: EFFECT OF LBPEA ON THE COMPRESSIVE STRENGTH OF RE-VIBRATED CONCRETE AT DAY 14

Revibration time interval (mins)	Compressive strength, N/mm^2						
	0%LBPEA + 100%OPC (Control)	5%LBPEA + 95%OPC (5L95C)	10%LBPEA + 90%OPC (10L90C)	15%LBPEA + 85%OPC (15L85C)	20%LBPEA + 80%OPC (20L80C)	25%LBPEA + 75%OPC (25L75C)	30%LBPEA + 70%OPC (30L70C)
0	19.27	18.58	12.36	12.09	6.31	6.04	5.11
10	18.80	20.89	14.00	12.44	7.47	7.11	5.24
20	21.84	18.76	13.73	12.80	7.42	7.24	5.38
30	20.40	20.27	12.44	12.36	7.20	7.07	5.24
40	19.33	20.49	13.24	13.51	7.33	7.16	6.00
50	24.71	21.42	13.02	13.11	7.56	7.6	5.69
60	24.76	21.60	12.67	13.16	7.73	8.04	6.40

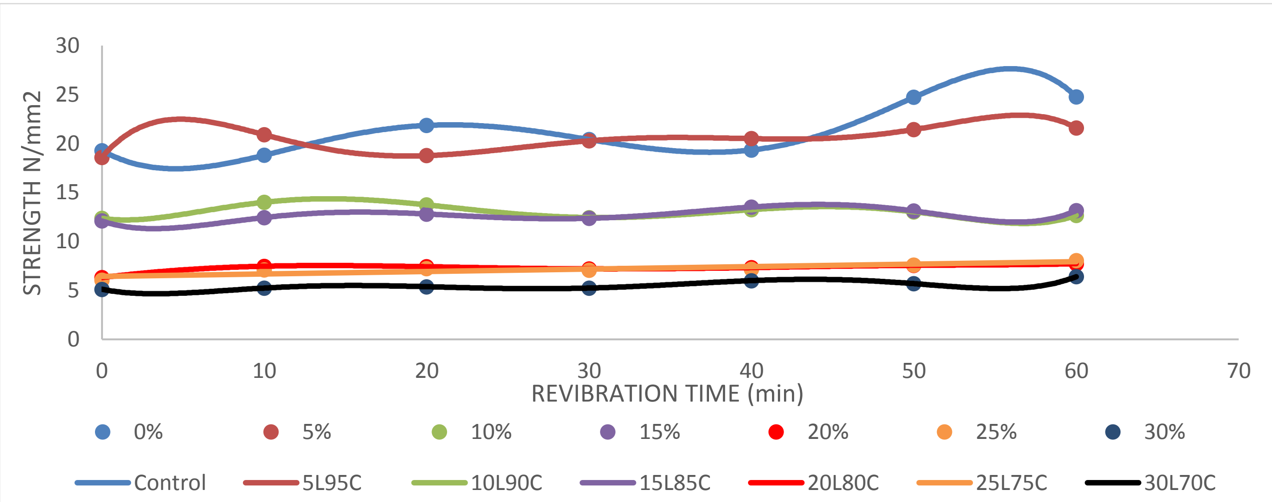

FIGURE 4: EFFECT OF LBPEA ON THE COMPRESSIVE STRENGTH OF RE-VIBRATED CONCRETE AT DAY 14

The trends of the effect of LBPEA on the day 14 compressive strength of re-vibrated concrete at various re-vibration time are shown in Figure 4. There was a sinusoidal behavior in the strength of the concrete at different re-vibration time. The maximum strength at day 14 was obtained at the control stage to be 24.76 N/mm^2 at 60 minutes re-vibration time. Closest to this value is that obtained at 5% ash replacement at 60 minutes re-vibration gotten as 21.60 N/mm^2. Other replacement percentages were seen to have relatively low strength compared to the control and 5% replacement at almost all the re-vibration time.

TABLE 4: EFFECT OF LBPEA ON THE COMPRESSIVE STRENGTH OF RE-VIBRATED CONCRETE AT DAY 28

Revibration time interval (mins)	Compressive strength, N/mm^2						
	0%LBPEA + 100%OPC (Control)	5%LBPEA + 95%OPC (5L95C)	10%LBPEA + 90%OPC (10L90C)	15%LBPEA + 85%OPC (15L85C)	20%LBPEA + 80%OPC (20L80C)	25%LBPEA + 75%OPC (25L75C)	30%LBPEA + 70%OPC (30L70C)
0	22.27	20.36	13.78	12.27	6.71	6.31	5.69
10	22.58	20.67	15.78	13.42	7.16	7.02	5.87
20	22.78	19.33	14.31	13.16	7.56	7.38	6.22
30	21.34	22.22	15.29	14.22	7.33	7.24	6.44
40	20.65	21.78	15.38	15.20	7.64	7.56	6.22
50	24.09	24.44	13.87	13.33	8.44	7.82	6.13
60	22.27	28.44	16.09	15.56	8.00	8.18	6.93

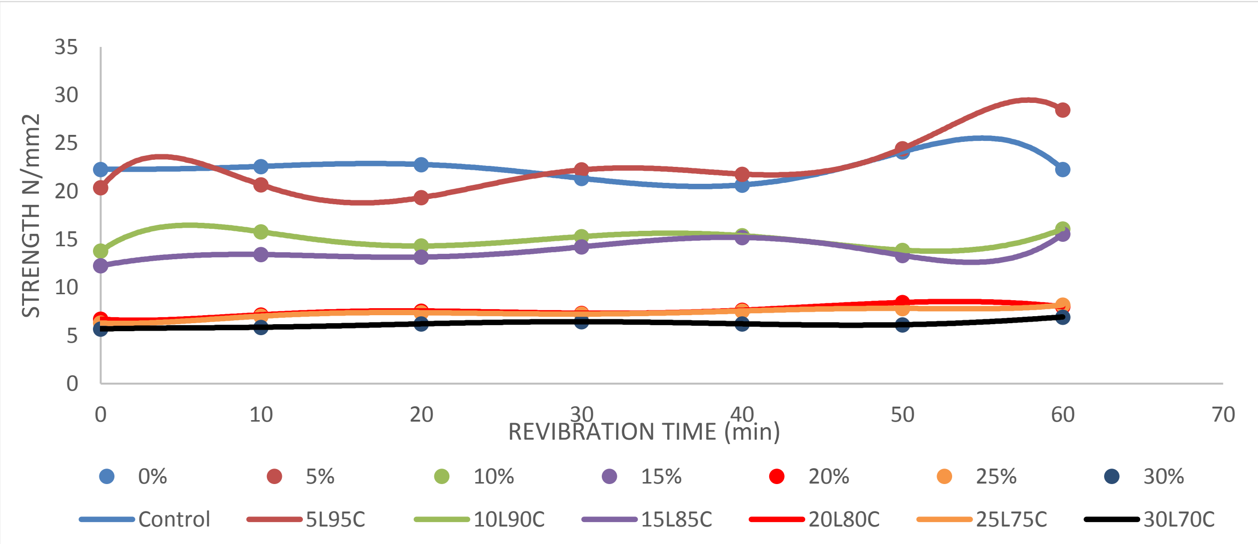

FIGURE 5: EFFECT OF LBPEA ON THE COMPRESSIVE STRENGTH OF RE-VIBRATED CONCRETE AT DAY 28

The trends of the effect of LBPEA on the day 28 compressive strength of re-vibrated concrete at various re-vibration time are shown in Figure 5. There was also a sinusoidal behavior in the strength of the concrete at different re-vibration time. The maximum strength at day 28 was at 5% ash replacement at 60 minutes re-vibration and gotten as 28.44 N/mm^2. Nearest to this percentage is that obtained at the control stage to be 24.09 N/mm^2 at 50 minutes re-vibration time. This day 28 strength for the 5% replacement cubes indicates about 18.1% in the peak value for the control. Other replacement percentages were seen to have relatively low strength compared to the control and 5% replacement at almost all the re-vibration time.

4 CONCLUSIONS

Effect of LBPEA on the compressive strength of re-vibrated concrete has been investigated and the results have been presented, analysed and discussed. From the study, the following conclusions were drawn:

LBPEA used is a good pozzolana with its SiO_2, Al_2O_3, and Fe_2O_3 content summing to 77.81% (ASTM C618);

The compressive strength at 28 days curing of re-vibrated concrete for 0% and 5% LBPEA replacement were all greater than 20 N/mm^2 which showed an advantage in compressive strength due to LBPEA incorporation as binder in concrete mix. Thus, 5% replacement of OPC with LBPEA can be adopted for structural concrete production. However, addition of LBPEA beyond 5% only weakens the concrete.

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VOLUMETRIC SHRINKAGE OF COMPACTED LATERITIC SOIL TREATED WITH *SPOROSARCINA PASTEURII*

K.J. Osinubi[1], *E.W. Gadzama[2], T.S. Ijimdiya[1] and A.O. Eberemu[1]

[1]Department of Civil Engineering, Ahmadu Bello University, Zaria, Nigeria
[2]Department of Civil Engineering, Modibbo Adama University of Technology Yola, Adamawa State Nigeria
* corresponding author email: gadzymo@gmail.com

ABSTRACT

This paper evaluates the volumetric shrinkage of an A-4(3) lateritic soil obtained from Abagana in Anambra state, Nigeria. The soil was mixed with stepped *Sporosarcina pasteurii* (S. *pasteurii*) suspension density of 0, 0.5, 2.0, 4.0, 6.0 and 8.0 McFarland Standards (i.e. 0, 1.50 x 10^8, 6.0 x 10^8, 1.20 x 10^9, 1.80 x 10^9 and 2.40 x 10^9 cells/ml, respectively). Samples were prepared at moulding water content from -2 to 4 % relative to optimum moisture content (OMC) and compacted with British Standard light (BSL) energy. The compacted specimens were permeated with cementation reagent in three cycles to initiate microbial-induced calcite precipitation (MICP) process. The treated soil was cured for 24 hours at a temperature of 24 ± 2ºC before extrusion from the compaction moulds. The weights, heights and diameters of the extruded samples were measured at five days' interval up to a maximum of thirty days. Results showed that the lowest and the highest volumetric shrinkage strain (VSS) values were recorded for specimens prepared at -2 % OMC and +4% OMC when treated with S. *pasteurii* suspension densities of 2.40 x 10^9 cells/ml and 0 (i.e., natural soil), respectively. The results also revealed that in the first 10 days, the loss in weight of both the natural and treated soil specimens was significant. A linear relationship was established between VSS values and moulding water content regardless of the S. *pasteurii* suspension density considered.

Keywords: Cementation reagent, Compactive effort, Lateritic, *Sporosarcina pasteurii*, Volumetric shrinkage strain.

1 INTRODUCTION

The traditional and conventional methods of soil improvement involve the use of chemical additives which may not be without some environmental issues. Karol, (2003) and Qian *et al*. (2018) reported that apart from the use of sodium silicate, all other chemical grouts used for soil improvement for over a century have been questioned when their sustainability is evaluated with regard to environmental related issues associated with them. However, in recent times, a relatively new and sustainable approach to soil improvement emerged known as microbial-induced calcite precipitation (MICP). MICP is a novel, innovative and environmentally friendly soil improvement method that has been appraised as a sustainable method that could possibly substitute the current conventional soil improvement method (Moravej *et al*., 2018)

Gomez and Dejong (2017) reported that MICP is an environmentally conscious soil improvement technique that improves the geotechnical properties of soils through calcite precipitation. The product responsible for the soil improvement is calcite which is obtained from the reaction between a urease producing microorganism which hydrolyzes urea into ammonium and carbonate, and in the presence of calcium source, calcium carbonate is precipitated as summarized in the equations (1) and (2):

$$CO(NH_2)_2 + 2H_2O \rightarrow 2NH_{4^+} + CO_3^{2-} \quad (1)$$

$$Ca^{2+} + CO_3^{2-} \rightarrow CaCO_3 \downarrow \quad (2)$$

According to Al-Qabany *et al*. (2012), at higher input rates, the efficiency of calcite precipitation decreases because the rate of bacterial urea hydrolysis is slower than the input rate. Also, the precipitation pattern of calcite at the pore scale level is affected by the liquid media concentration. Laterite is a residual soil with no specific definition because of differences in opinions by different researchers. The definition reported by Alexander and Cady (1962) that "Laterite is a highly weathered material, rich in secondary oxides of iron, aluminum, or both. It is
void or nearly void of bases primary silicates, but may contain large amounts of quartz and
kaolinite" has been modified by Ola (1983), CIRIA (1988), Osula (1991), Ijimdiya (2012), Oyelami and Van Rooy (2016), etc. This tropical soil has featured as an engineering material in different engineering structures including landfill liners and covers.

For materials to be considered for possible use in liners and waste containment facilities, they are required to have volumetric shrinkage strain not exceeding 4% as recommended by Daniel and Wu, (1993), they reported that a maximum VSS of 4 % would not undermine the stability of the liner in the process of drying. This study evaluates the volumetric shrinkage strain of a lateritic soil treated with different suspension densities of a commonly found indigenous soil microorganism *S. pasteurii* for use as a landfill liner material in waste containment application.

2. MATERIALS AND METHODS

2.1 MATERIALS

Soil: The lateritic soil used in this study was obtained by method of disturbed sampling at depths in the range 0.5 – 3.0 m from an erosion prone site located in Abagana (Latitude 6º10'15" N and Longitude 6º58'10" E), Anambra state, Nigeria.

Bacteria: The species of bacteria used in this study is S. pasteurii, which was screened and cultured from the soil sample.

Cementation reagent: The following are the composition of the cementation reagent used in this study: 20g of Urea, 10g of NH4Cl, 3g of Nutrient broth, 2.8g of CaCl2 and 2.12g of NaHCO3 per litre of de-ionized water, which has been used in several studies (e.g., Stocks-Fischer et al., 1999; Dejong et al., 2006; Al Qabany et al., 2011; Kim and Youn, 2016; Tirkolaei and Bilsel, 2017, etc.). The other reagents were autoclaved after which the urea was added aseptically to avoid it decomposition by heat.

2.2METHODS

Bacteria culture/growth medium: The identification of the bacteria was carried out using the conventional procedure described by Cheesbrough (2006). Bacteria were cultured in Ammonium-Yeast Extract media described by Mortensen et al. (2011) and Feng et al. (2014). The bacteria S. pasteurii was isolated from soil inoculated on media containing the following: 20 g yeast extract, 10 g ammonium chloride, 2 g urea 0.1 g nickel all in 1litre distilled water and NaOH was used to adjust the pH of the media to 9.0. The media was poured in an aliquot amount of 10 ml per culture bottle corked with foil paper and placed in an autoclave set at 121ºC per 1.1 kg pressure for a period of 15 min. 1 g each of the soil samples were inoculated on each of the culture bottle and incubated at 37ºC for a period of 24-48 hours for identification and characterization of S. pasteurii as an isolated urease producing organism. Biochemical confirmatory tests were conducted on the cultured microorganisms.

Bacteria cell suspension density: The *S. pasteurii* suspension densities used were varied as follows: 0, 0.5, 2.0, 4.0, 6.0 and 8.0 McFarland Standards (i.e., 0, 1.5 x 10^8, 6.0 x 10^8, 1.20 x 10^9, 1.80 x 10^9 and 2.40 x 10^9 cells/ml, respectively). The maximum volume of microorganisms mixed with the soil was 1/3rd of its pore volume as recommended by Rowshanbakht et al. (2016). Individual pore volumes were determined for each compactive effort that was used.

Characterization of calcite precipitate in cementation reagent and treated soil sample:

3ml each of *S. pasteurii* suspension densities 1.50 x 10^8, 6.0 x 10^8, 1.20 x 10^9, 1.80 x 10^9 and 2.40 x 10^9 cells/ml were each mixed with 7 ml of the cementation reagent in three cycles. The suspensions were placed in sealed test tubes for 24 hours at a temperature range of 24 ± 2ºC. The precipitates resulting from the reactions shown on Plate 1A, were separated using a filter paper and oven-dried at a temperature of 105ºC before being characterized using X-Ray Diffraction (XRD) analysis.

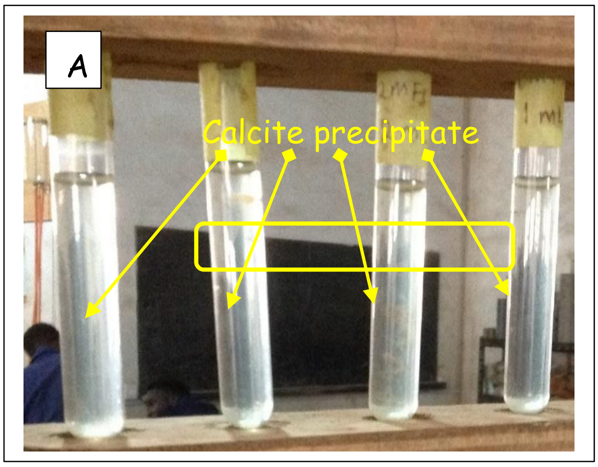


PLATE 1: (A) CHARACTERIZATION OF CALCITE PRECIPITATE

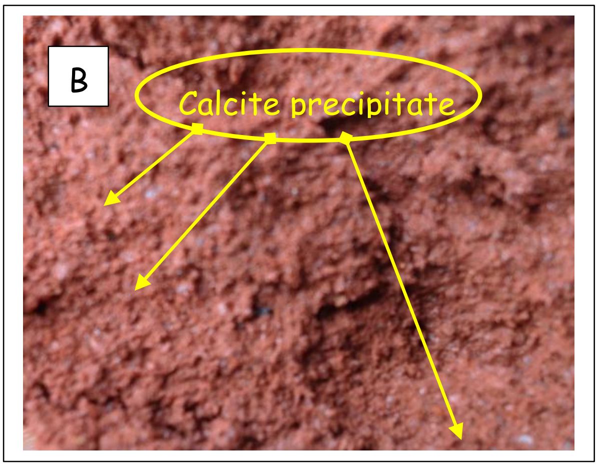


PLATE 1: (B) CRUST FROM THE CENTER OF THE COMPACTED TREATED SOIL COLUMN SHOWING NON PREFERENTIAL CALCITE CRYSTAL PRECIPITATION.

Index properties: Natural moisture content, specific gravity, and sieve analysis were carried out in accordance with procedures outlined in BS 1377: 1990.

Atterberg Limits: Soil sample passing through BS No. 40 sieve (425 µm aperture) was used for Atterberg limits (i.e., liquid limit, plastic limit and plasticity index) tests. The soil samples were initially mixed with varying *S. pasteurii* suspension densities at the optimum moisture content (OMC) for British Standard light (BSL) compaction. Thereafter, the mixture was treated with the cementation reagent in three cycles each with 2/3rd pore volume within a 24-hour period before being air-dried at room temperature of 24 ± 2°C prior to Atterberg limits tests in accordance with BS 1377: 1990.

Compaction characteristics: Compaction of specimens was carried out in accordance with the method outlined in BS 1377 (1990) to obtain the OMC for BSL energy derived from 2.5 kg rammer falling through 30 cm onto three layers each receiving 27 blows.

Volumetric shrinkage: The compacted specimens were extruded from the mould air-dried for 30 days at an average temperature of 24 ± 2°C. Measurements of weights, diameters and heights were made every five days with the aid of an electronic weighing balance and a Vernier caliper with a precision of 0.01mm. Average readings obtained were used for the computation of the volumetric shrinkage of all the specimens.

TABLE 2: OXIDE COMPOSITION OF THE NATURAL LATERITIC SOIL

Oxide	SiO_2	Al_2O_3	CaO	TiO_2	V_2O_5	Cr_2O_3	Fe_2O_3	MnO	CuO	ZrO_2	LOI	Total
Concentration (%)	56.5	19.00	0.33	2.89	0.061	0.051	15.41	0.075	0.056	0.290	4.54	99.20

Preparation of specimen: 3000 g of the crushed air-dried soil sample passing through BS No. 4 sieve (4.76 mm aperture) was mixed thoroughly at moulding water contents between -2 % and +4 % relative to OMC each containing 1/3rd of its pore volume as the bacterial densities. The mixture was placed in sealed polythene bags and cured for 12 hours at an average temperature of 24 ± 2°C for the microorganisms to be properly distributed and held unto the soil surface before compaction using the BSL energy. Compacted specimens

were permeated in three circles with 2/3rd of it pore volume each under gravity with the cementation reagent described earlier to initiate the MICP process.

3. RESULTS AND DISCUSSION

Index properties of the natural soil: The index properties of the soil are summarized in Table 1. Only 36.0 % of the soil passed BS No. 200 sieve with silt and clay contents of approximately 22 % and 14 %, respectively. The soil is classified as A-4(3) and SC according to AASHTO (1986), and ASTM (1992), respectively. XRD analysis of the natural soil showed that the dominant clay mineral is kaolinite, which is a relatively stable and non-expansive clay mineral. While Table 2 shows The oxide composition of the soil summarized in Table 2 indicate that the soil has a silica - sesqueoxide {i.e., Si/(Al + Fe)} ratio value of 1.64 which lies between 1.33 and 2.00 for lateritic soils as recommended by Joachim and Kandiah (1941).

TABLE 1. PHYSICAL PROPERTIES OF THE NATURAL SOIL

Property	
Natural moisture content (%)	11.3
Percentage Passing No. 200 Sieve (Wet Sieve)	36.0
Liquid Limit (%)	44.0
Plastic Limit (%)	21.6
Plasticity Index (%)	22.4
CEC (Meq/100g)	5.50
Specific Gravity	2.62
AASHTO classification	A – 4(3)
USCS	Sandy clay
Colour	Reddish brown
Dominant Clay Mineral	Kaolinite

Atterberg Limits: The variation of Atterberg limits (liqiud limit (LL), plastic limit (PL) and plasticity index (PI)) with *S. pasteurii* suspension density is shown in Fig. 1. It was observed that the liquid limit increased from 44 % for the natural soil to 49.5 % when treated with *S. pasteurii* suspension density of 1.50 x 10^8 /ml. At higher *S. pasteurii* suspension density of 1.20 x 10^9 /ml, the plastic limit increased to a peak value of 50.5 %. Plastic limit remained constant at 49.5 % with higher *S. pasteurii* suspension densities of 1.80 x 10^9 /ml and 2.40 x 10^9 /ml but decreased from a value of 21.6 % for the natural soil to 14.8 % when treated with *S. pasteurii* density of 1.50 x 10^8 /ml. However, plastic limit gradually increased to a peak value of 17.37 % when treated with *S. pasteurii* density of 2.40 x 10^9 /ml. The Atterberg limits results show that the influence of treatment with varying *S. pasteurii* suspension density is of the order: plastic limit > plasticity index > liquid limit. It implies that treatment of the lateritic soil with *S. pasteurii* and cementation reagent improved the engineering properties of the soil through the reduction in plastic limit probably due to the formation of calcites during MICP process. Similar trend was reported by rresearcher (Eberemu, 2011; Eberemu, 2013; Eberemu *et al*., 2013; Ijimdiya, 2017; Ijimdiya and Musa, 2017; Osinubi e*t al*., 2017; 2018) for different soil improvement techniques.

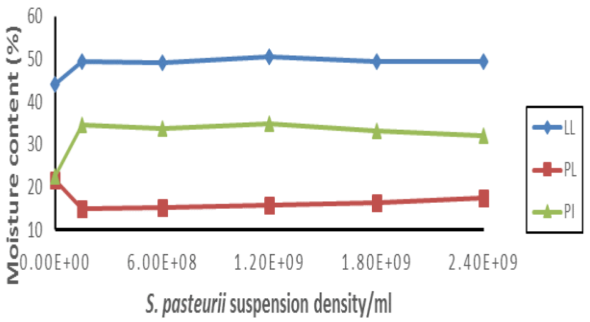
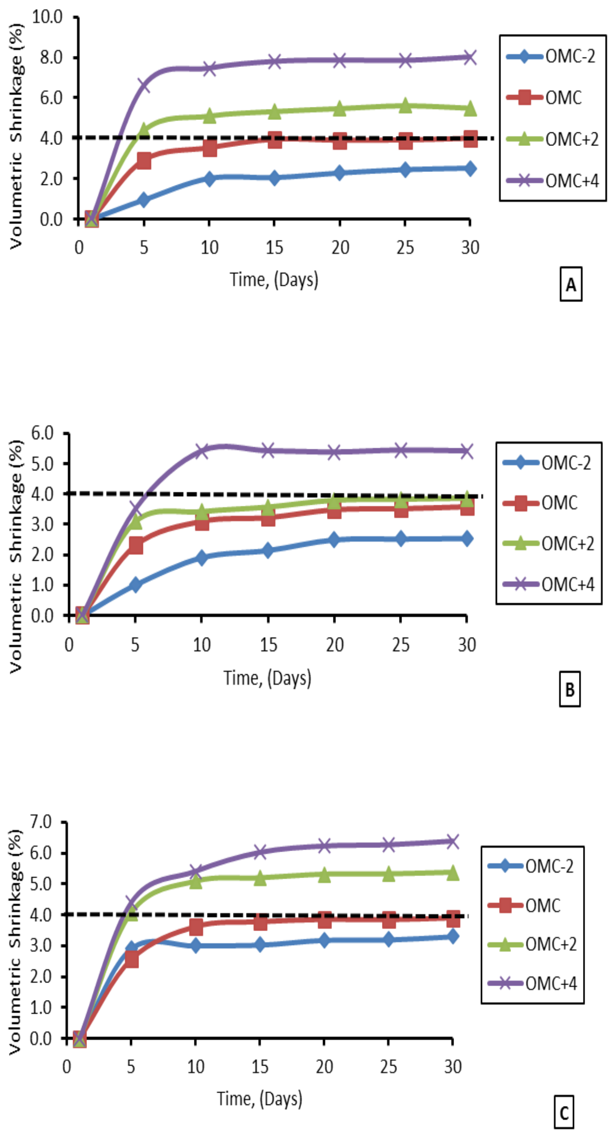


FIG. 1: VARIATION OF ATTERBERG LIMITS WITH *S. PASTEURII* SUSPENSION DENSITY



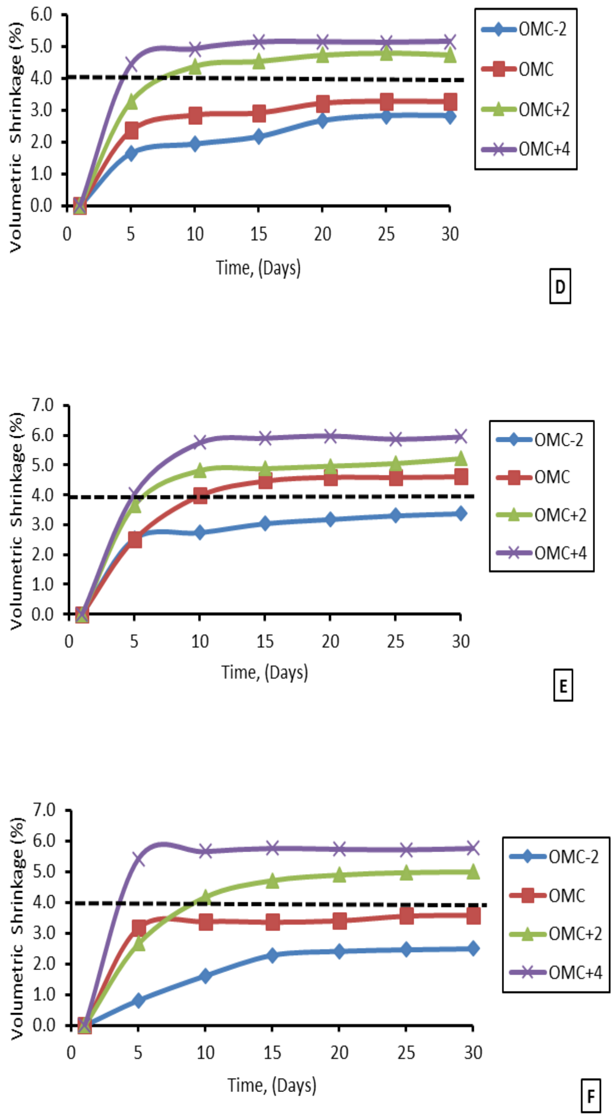
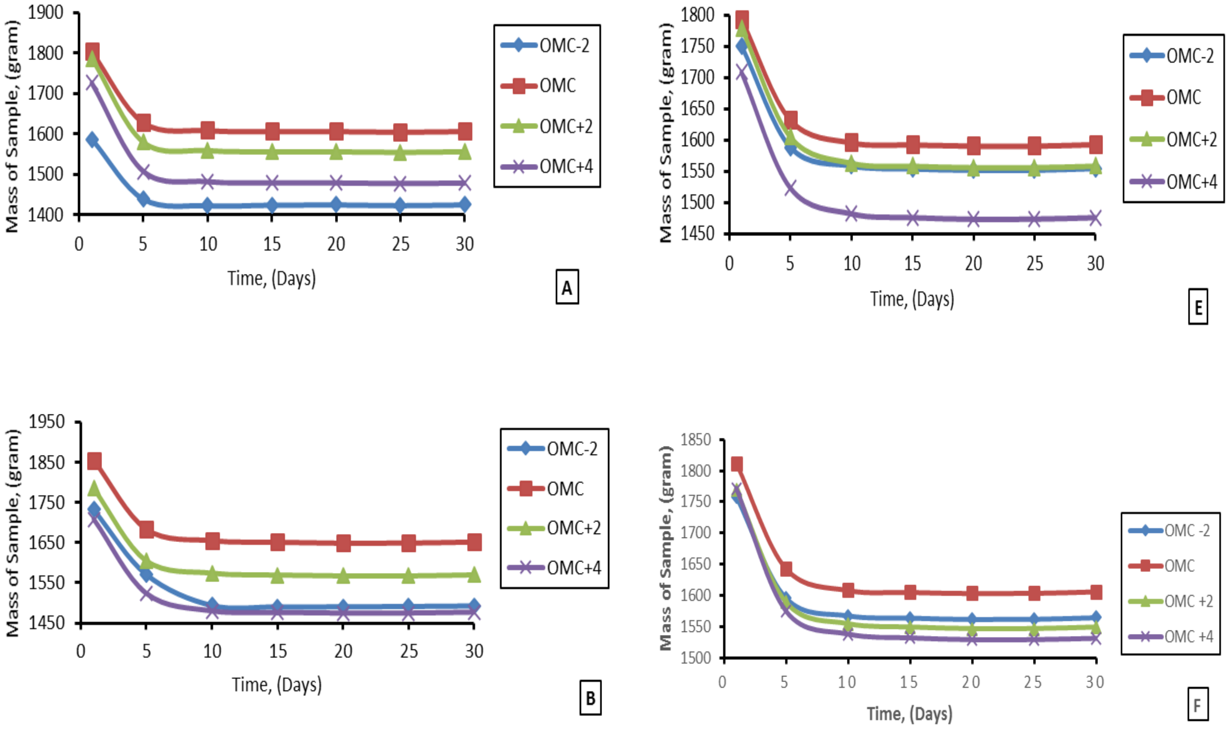


FIG. 2: VARIATION OF VOLUMETRIC SHRINKAGE STRAIN OF COMPACTED LATERITIC SOIL WITH TIME FOR VARYING MOULDING WATER CONTENT RELATIVE TO OPTIMUM FOR *S. PASTEURII* SUSPENSION DENSITY: (A) 0 /ML (I.E., NATURAL SOIL) (B) 1.5 X 10^8 /ML (C) 6.0 X 10^8 /ML (D) 1.2 X 10^9 /ML (E) 1.8 X 10^9 /ML (F) 2.4 X 10^9 /ML

Effect of air-drying on the mass of compacted specimens

The effect of air-drying on the mass of compacted specimens for *S. pasteurii* suspension densities considered is shown in Fig. 3a-f. Loss in mass was significant within the first 5 days and thereafter remained relatively constant for the duration of the test. For the natural soil, the peak loss in mass of 175 g was recorded within 5 days on the specimen treated at OMC while lowest weight loss of 146.9 g was recorded for specimen prepared at moulding water content -2% OMC within the same period. For the MICP treated soil the highest and lowest losses in mass of 187.4 g and 158.6 g were recorded for specimens treated with 6.0 x 10^8 cells/ml *S. pasteurii* suspension density within the same period at water contents of +4% OMC and OMC, respectively. The trend observed for the natural soil was due to the unsaturated nature of the specimens that dried based on their moulding water content. The trend in the treated soil that were saturated with the same volume of cementation media prepared at +4% OMC recorded loss in mass due to their higher moisture content than specimens prepared at OMC. This therefore explains why the specimens prepared at the OMC retained higher masses upon drying when compared to specimens prepared at other moulding water contents. For the natural soil, the order of volumetric shrinkage values based on the moulding water content at which specimens were prepared is: OMC > +2% OMC > +4% OMC > -2% OMC, which is different from OMC > +2% OMC > -2% OMC > +4% OMC recorded for the treated specimens. The volumetric shrinkage results of the natural soil suggest that the highest density was achieved at the OMC, followed by +2% OMC > +4% OMC > -2% OMC. This trend could be due to the condition for achieving maximum density during compaction, which is directly related to the moulding moisture used but, thereafter it becomes inversely proportional resulting in decreased density with higher moulding water content. After the maximum density is achieved, soil particles were displaced by excess moulding water content. The recorded result has also shown that regardless of the displacement of soil particles by the excess moulding water, higher densities were recorded for specimens prepared at +2% and +4% OMC than those prepared at -2% OMC. For the treated soil, the densities recorded at -2% OMC were all higher than those recorded for the natural soil probably due to the precipitation of calcite in the voids that made the specimens denser when compared to the natural soil. Similar trends were reported by Albrecht and Benson (2001); Eberemu (2011); Osinubi and Eberemu (2011); Moses and Afolayan, (2013) as well as Moses *et al*. (2016). Treatment of lateritic soil with *S. pasteurii* resulted in increase in density of the soil especially for specimens prepared at -2% OMC which recorded a steady increase from 1.425 Mg/m^3 for natural soil to 1.492 Mg/m^3, 1.547 Mg/m^3 at 1.50 x 10^8 and 6.0 x 10^8 cells/ml before decreasing to 1.528 Mg/m^3 at 1.20 x 10^9 cells/ml before finally increasing to a maximum of 1.554 Mg/m^3 at *S. pasteurii* densities of 1.80 x 10^9 cells/ml and 2.40 x 10^9 cells/ml respectively.



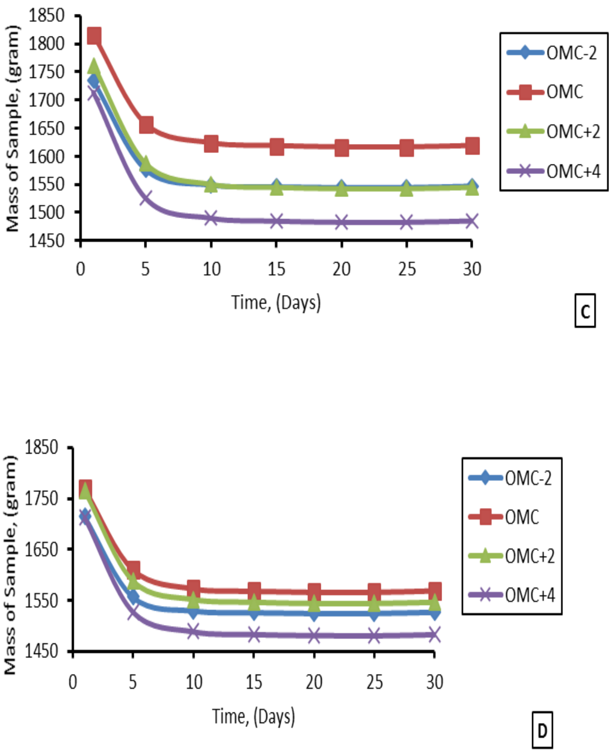


FIG.3 VARIATION OF MASS OF COMPACTED LATERITIC SOIL WITH TIME FOR VARYING MOULDING WATER CONTENT RELATIVE TO OPTIMUM AND *S. PASTEURII* SUSPENSION DENSITY: (A) 0 /ML (I.E., NATURAL SOIL) (B) 1.50 X 10^8 /ML (C) 6.0 X 10^8 /ML (D) 1.20 X 10^9 /ML (E) 1.80 X 10^9 /ML (F) 2.40 X 10^9 /ML

Effect of moulding water content on volumetric shrinkage strain

Fig.4 shows The variation of VSS with moulding water content for varying *S. pasteurii* suspension densities is shown in Fig. 4. Generally, VSS value increased non-linearly with higher moulding water content. All the treatments at 13.3% moulding water content met the maximum VSS design criterion of 4 % for use in waste containment facilities. At the OMC (i.e., 15.3 %) only specimen treated with *S. pasteurii* suspension density of 1.80 x 10^9cells/ml recorded VSS value that is higher than the maximum 4 % design criterion. At 17.3 %, moulding water content only specimen treated with *S. pasteurii* suspension density of 1.20 x 10^9 cells/ml satisfied the design criterion. At 19.3%, moulding water content all the treatments considered recorded VSS values higher than the design criterion. The calcite precipitated in the voids may have been responsible for the lower VSS values recorded therefore higher volumetric shrinkage is expected to be significant in the specimens prepared at higher moulding moisture content than at lower moulding water content in agreement with the findings reported by Osinubi and Nwaiwu, (2008); Cheng *et al*. (2013) and Mujah *et al*. (2017). Based on the results recorded, it implies that VSS values are linearly related to moulding water content irrespective of the *S. pasteurii* suspension density considered.

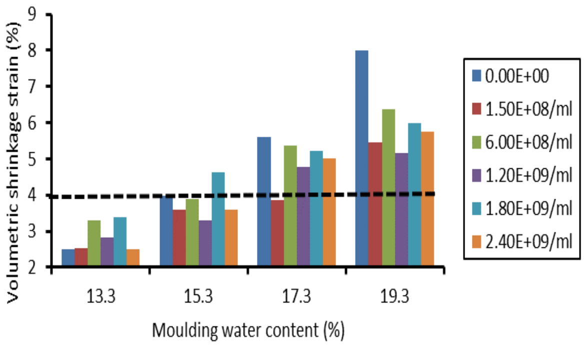


FIG.4 VARIATION OF VOLUMETRIC SHRINKAGE STRAIN OF LATERITIC SOIL WITH MOULDING WATER CONTENT AT VARYING *S. PASTEURII* SUSPENSION DENSITY.

Effect of *S. pasteurii* suspension density on volumetric shrinkage strain

The variation of volumetric shrinkage strain of lateritic soil with *S. pasteurii* suspension density is shown in Fig. 5. It was generally observed that the VSS is linearly related to moulding water content relative to optimum irrespective of the *S. pasteurii* suspension density considered. At -2% and +4% moulding water contents relative to optimum, the lowest and highest VSS values, respectively, were recorded for the *S. pasteurii* suspension densities considered. Treatment of lateritic soil with *S. pasteurii* suspension density of 2.40 x 10^9 cell/ml recorded the lowest VSS value of 2.5 % while the natural soil recorded the highest VSS value of 8.01 %. On the other hand, treatment with *S. pasteurii* suspension density of 1.50 x 10^8 cells/ml was observed to be the optimum because it is the only treatment for which specimens prepared at +4% OMC did not meet the maximum 4 % VSS value design criterion. The results presented suggest that regardless of the *S. pasteurii* suspension density used, moulding water content relative to optimum influenced the VSS of the soil. Similar results have been reported by Osinubi and Nwaiwu, (2008); Moses and Afolayan, (2013); Widomski *et al.* (2015); Moses *et al*. (2016) and Osinubi *et al.* (2018).

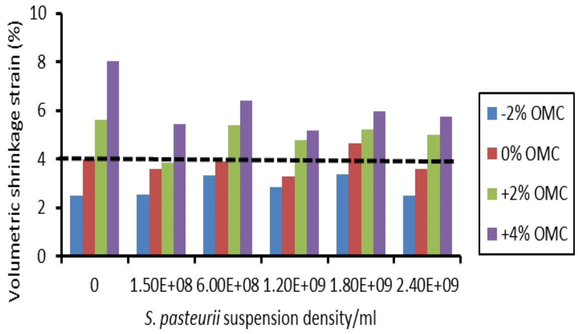


FIG.5: VARIATION OF VOLUMETRIC SHRINKAGE STRAIN OF LATERITIC SOIL WITH *S. PASTEURII* SUSPENSION DENSITY AT VARYING MOULDING WATER CONTENT RELATIVE TO OPTIMUM.

4 CONCLUSION

From the results presented, the following conclusions can be drawn:

1. A linear relationship exists between moulding water content relative to optimum and volumetric shrinkage strain of lateritic soil regardless of the *S. pasteurii* suspension densities considered.
2. The order of volumetric shrinkage strain values lateritic soil treated with *S. pasteurii* suspension densities considered prepared at varying moulding moisture content relative to optimum is: -2% OMC < OMC < +2% OMC < +4% OMC.
3. Treatment of lateritic soil with *S. pasteurii* increased the density of the soil especially for specimens prepared at -2 % OMC which recorded a significant increase from 1.425 Mg/m^3 for the natural soil to a maximum of 1.554 Mg/m^3 at *S. pasteurii* densities of 1.80 x 10^9 cells/ml and 2.40 x 10^9 cells/ml respectively.
4. Based on the results recorded lateritic soil can be treated with an optimum *S. pasteurii* suspension density of 1.50 x 10^8 cells/ml when used in municipal solid waste containment applications.

5 RECOMMENDATION

In order to satisfy the maximum 4 % volumetric shrinkage strain design criterion for materials to be used as barriers in municipal waste containment facilities, it is recommended that the lateritic soil used in the study be treated with *S. pasteurii* suspension density of 1.50 x 10^8 cells/ml and compacted with british Standard light (BSL) energy at moulding moisture content between -2% and +2% OMC.

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PHYSICAL AND MINERALOGICAL CHARACTERISTICS OF OVERBURDEN ON THREE COMMON BASEMENT COMPLEXES

*Abdullahi, Y. M.[1]; Alhaji, M. M.[1]; & Alhassan, M.[1]
[1]Civil Engineering Department, Federal University of Technology, Minna, Nigeria
*Corresponding Author; ymozay@gmail.com

ABSTRACT

Disturbed soil samples taken at depths 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, and 5m/to bedrock from three (3) trial pits, of common basement complexes around Minna, of granite, gneiss and schist parent rocks, were subjected to physical tests, while X-ray diffraction (XRD) method was employed for the mineralogical analysis. Variation of physical and mineralogical properties with depth of the weathering profiles was made. Index properties results for the three (3) location shows, they are clay and silt of intermediate and low plasticity with liquid limit and plasticity index ranging between 27.80 to 50.50% and 4.74 to 40.00% respectively while percentage of fines ranging between 24.67 to 63.70%. The x-ray diffraction results show the presence of quartz in all locations, while the major minerals for each of the location were identified to be kaolinite, albite; a non-clay mineral and illite in combination with other minerals. The presence of kaolinite in combination with other clay minerals, such as montmorillonite is responsible for intermediate plasticity of samples of granite parent rock.

Keywords: *Basement Complexes, Bedrock, Index Properties, Plasticity, Weathering Profile, X-Ray Diffraction.*

1 INTRODUCTION

All soils are derived from rock by weathering process; the relationship between rock types and soil types is an integral aspect of the study of soils, especially on tropically weathered soils, this process brings about changes in chemical and mineralogical composition of parent rock (Irfan & Dearman 1978) and are dictated primarily by the minerals that constitute the soil particles and their parent rock (Braja 2010). Five factors that have an influence on the weathering of rock and the subsequent formation of soil minerals are; parent rock - composition and texture, climate - principally the variation in temperature and rainfall, topography, vegetation; in particular the products of decomposition, and time (Grim 1968), this factors are optimized in the tropics (Townsend 1985), as they affect the variation of mineralogical and engineering properties of soils with depth. Mechanical process disintegrates these rocks into smaller fragments; and chemical action which depends on temperature, surface area and amount of water (Gard 2012), reduces these small fragments, rearrange the elements into new minerals, and thus decompose them.

Chemical weathering (decomposition) can transform hard rock minerals into soft, easily erodible matter. The principal types of decomposition are hydration, oxidation, carbonation, desilication and leaching. These processes occur when oxygen and carbon dioxide which are always present in the air readily combine with the elements of rock in the presence of water (especially if it contains traces of acid or alkali) resulting to a change in the mineral form of the parent rock (Murthy 2012) and consequently resulting in some minerals disappearing partially or fully, and new compounds formed, with maximum intensity in the humid and tropical climate, (Venkatramaiah 2006).

With overburden soils of the tropics receiving relatively little attention because of their location in areas of underdeveloped economies (Lambe and Whitemen 1969), most soil workers assume weathering products are uniform with depth conducting only a single or few soil tests at the beginning of excavation to represent soils at deeper depth borrow pits. They treat weathering profile as uniform with depth, not considering different formation factors, the composite and complex nature of the weathering materials (Adekoya, 1987), but it is common to find differences in the results of investigation even within short distances and/or depth. Therefore ignoring the variability of soil properties in a weathering profile with depth is denying the different formation factors of these soils and may lead to misleading results and consequently, serious failure to engineering structures erected on them.

Soil mineralogy plays an important role in forming the character of a soil, such that the key features employed to differentiate soils at the highest level depend on mineralogy (Uehara and Gillsman, 1981), this property is responsible for all the engineering properties such as specific gravity, shear strength, Atterberg limits, petrophysical properties and soil classification (Shafique et al., 2012) and when chemical and mineralogical analysis are conducted on soils in addition to the geotechnical characteristics, it proves to be useful in understanding soil behavior for engineering construction and has assisted significantly in its utility ((Mahalinger-Iyer and Williams 1997), but unfortunately, analysis to determine the mineral composition of soils is not usually explored in soil investigation, partly because the techniques and the equipment used are beyond the resources of the ordinary soil testing laboratory in Nigeria.

Soil minerals are composed of primary and secondary minerals. Primary are of the same minerals with the parent rocks; sand and gravel, which are products of mechanical weathering, are composed of this minerals, with no cohesive and plastic characteristics, while secondary minerals are found in silts and clay, they are products of chemical weathering; they are further classified as non-clay mineral and clay minerals. Non clay minerals are amorphous and impart little or no cohesion and plasticity characteristics to soil. However, particles defined as clay on the basis of their size are not necessarily clay minerals. Clays are geological materials having particle size of less than 2 microns and the family of minerals having similar chemical compositions and common crystal structural characteristics (Velde, 1995) they are generally plastic at appropriate water contents and will harden when fired or dried. Clays are very abundant at the earth's surface; they form rocks known as shales and are a major component in nearly all sedimentary rocks. The small size of the particles and their unique crystal structures give clay materials special properties, including cation exchange capabilities, plastic behavior when wet, catalytic abilities, swelling behavior, and low permeabilities.

Clay minerals can only be seen with high power electron microscope. They are responsible for many of the soil's most important and characteristic (Schulze, 2005), because of their tendency to develop plasticity when mixed with water, they influence basic properties, strength properties as well as chemical properties especially if they are prone to swelling (Jworchan, 2006), however the basic properties found in the clay fraction of soils that form the basis for their distinction are the constant surface charge and the constant surface potential (Uehara and Gillsman, 1981). Clay minerals can be categorized into four subgroups: (1) kaolinite; (2) smectite (montmorillonite, saponite); (3) mica (illite), and (4) chlorite (Shichi and Takagi, 2000; Nayak and Singh, 2007; Burhan et al., 2010), they are determined by their chemical composition, layered structure and size. The highest volume changing clay, exibiting high swelling and shrinkage potentials is the montmorillonite group (smectites). These volume changing clays are generally present all over the world being the cause of most natural hazards like foundation problems causing damage to structures, roads, services and rock instability causing landslides. The illite group has the same structural arrangement with the montmorillonite but presence of potassium as the bonding materials between units makes the illite minerals swell less, while the kaolinite is stable and water cannot enter between the sheets to expand the unit cells due to strong hydrogen bonding.

Basement complex rocks have been exposed to varying degrees of weathering over the years which led to the formation of clay deposits which are widely distributed in the study area (Ajayi 1981). In the north of Nigeria, these rocks are composed of gneisses, migmatites and granites, with extensive areas of schists, phylites and quartzites (Oyawoye, 1972; Rahaman, 1989). A better understanding of the nature of soils formed from these rocks and their mineralogical composition would make possible a more precise prediction of behavior, and many of the problems encountered in engineering practice would be avoided (Ademila and Adebanjo 2017). Therefore the aim of this study is to establish the variability in physical and mineralogical characteristics of overburden with depth derived from three (3) basement complexes of granite, schist and gneisss, as data from this study can aid in assessing the soils for classification and engineering performance.

2 METHODOLOGY

Basement complex weathered rocks of schist, gneiss and schist origin were identified at Tudun Fulani, dam site, and Birgi Village, along Talba Farm road, Bosso Local Government and in Kateregi Village adjacent Minning site, Katcha Local government of Niger State.

Three (3) trial pits were dogged manually to bedrock or 5m depth, during which disturbed soils samples were collected at 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, and 5.0m, with the soil profile been inspected visually. Samples were collected in polythene bags, sealed, labeled and taken to the Laboratory. Collected samples were air dried immediately after removing samples for the determination of natural moisture content

The samples were air dried and sieved, then subjected to clay mineral analysis using X-Ray Diffraction (XRD) conducted in South Africa. X-ray diffraction method is suitable for identifying and quantifying the minerals present in soils. Tests including Natural Moisture Content (NMC), washed sieved analysis, Liquid Limit (LL), Plastic Limit (PL), and Specific Gravity (SG) were carried out on each of the samples. All these tests were done in Civil Engineering Laboratory of Federal University of Technology, Minna in accordance with B. S. 8110 (1990) with modifications where necessary.

3 RESULTS AND DISCUSSION

3.1 Physical properties

Results for Natural Moisture Content (NMC), Liquid limit (LL), Plastic limit (PL), Plasticity Index (PI), Percentage passing sieve No. 200, AASHTO classification and Plasticity of all sampless are contained in table I, II and III.

The natural moisture content for samples from weathered granite increased from 10.30% at 0.5m to 24.10% at 2.0m depth, it then decreased along the profile, and then increased between 4.5m to 5m depth, while samples from weathered gneiss rock increased from 5.7% at 0.5m depth to 6.5% at 2.5m after which it finally dropped to 5.26% at 4m depth. Consequently samples obtained from weathered schist recorded values ranging between 4.4 to 8.88%. Natural moisture content is not a constant property of soils, these values are consistent with the fines contents of the clays (Ademila et al 2017), it increases with increasing clay content due to the ability of clay particles to absorb water and consequently increase with depth because of the exposure of the ground top to the sun (Asmaa 2012).

The weathered gneiss recorded the highest value of 2.92 as specific gravity at depth 0.5m. The weathered granite recorded values ranging between 2.54 to 2.77, while samples of weathered schist show an increment from 2.58 at 0.5m to 2.65 at 2m depth, it then decreased to 2.57 at 3m depth and finally increased to 2.66 at depth 3.5m. Specific gravity is an important property in the identification and evaluation of aggregate parameters for construction purposes; the higher, the better for construction purposes (Gidigasu 1976), it has also been shown to be closely linked with the mineralogy and/or the chemical composition of the soil (Ramamurthy and Sitharam 2005).

Samples of granite parent rock recorded the highest values of liquid limit, ranging between 46.02%- 50.50%, while sample from weathered gneiss recording between 29%-30.80%, consequently the liquid limit value for schist basement increased from 36% at 0.5m depth to 40% at depth 2m, and finally decreased to 39.9% at 3.5m depth. Liquid limit less than 30% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, between 50% and 70% high plasticity, between 70% and 90% indicates very high plasticity and greater than 90% indicates extremely high plasticity (Whitlow, 1995). Soils with high liquid limit are generally preferable for liners because of their low hydraulic conductivity (Ige, 2010). Liquid limit of soil use for barriers lining should be less than 90% (Declan and Paul 2003) and should have a lower limit of at least 20% (Benson et al 1994). Standard for road works recommend liquid limits of 50% maximum for sub base and base materials (FMWH, 1997). Variation in the liquid limit may be due to the mineralogical composition of the samples. Thus all soils studied are can be used for sub base material and base material, because they fall below the 50% threshold of liquid limit, with improvement where necessary. The plastic limit for weathered granite recorded the highest, with values ranging between 27.75 to 37.75%, with weathered gneiss rock recording between 0 to 27.37%, while samples from weathered schist recorded no plastic limit. Soil with plastic limit ranging between 10 – 60% is recommended for use in ceramic production (Grimshaw 1971), weathered granite samples and some from the gneiss weathered rock fall within this specified limit.

Plasticity index is the range of moisture content over which a soil is in plastic condition, and is influenced by the amount clay fraction and the type of clay minerals present, since the amount of attracted water held in a soil is influenced by clay minerals. The plasticity Index for the weathered granite ranges between 9.24% to 20.66%, samples from weathered gneiss recording values between 1.63% and 30.8%, while weathered schist samples recorded the highest values, between 35.01% and 39.9%. The difference in the plasticity index may be due to the presence of montmorillonite which is responsible for great volume change potential (Ademila and Adebanjo 2017).

Low plasticity index and liquid limit could an indication of the presence of high amount of kaolinite and absence or very low content of montmorillonite (Duruojinnaka et al 2016), however small value of plasticity index such as 5% indicates that a small change in moisture content will change the soil from a semi-solid to a liquid condition, while plasticity index, such as 20% shows that considerable water can be added before the soil becomes liquid (Salome et al 2013).

According to Unified Soil Classification System (USCS), samples from granite basement, on the plasticity chart are classified as silt and clays of intermediate plasticity with percentage passing sieve No. 200(0.075mm) ranging between 35.47% to 63.70%, those taken from weathered gneiss falls into the category of silts and clays with low plasticity, with percentage passing sieve No. 200(0.075mm) between 24.67% to 34.00% while samples of schist basement origin falling under clays of intermediate plasticity with percentage passing sieve No. 200(0.075mm) between 22.17% to 40.77%. Soils with amounts of fines less than 50% are expected to possess better engineering properties while those with amounts of fines greater than 50% are expected to pose field compaction problems when used either as sub-base or sub-grade materials (Oyediran 2010), while recommended specification for general filling for highway construction should contain fines ≤ 35 % (FMWH 2000). Therefore on the basis of the recommendations almost all samples are suitable for general filling for highway construction; hence, the materials qualify as general filling materials.

TABLE I: PHYSICAL PROPERTIES WITH DEPTH OF WEATHERED GRANITE

BIRGI SAMPLES										
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**
Natural moisture content (%)	10.30	16.90	20.50	24.10	22.40	21.80	22.60	22.70	25.90	26.30
Specific gravity Gs	2.71	2.64	2.77	2.68	2.54	2.59	2.66	2.69	2.68	2.66
Liquid limit ''LL" (%)	46.02	49.20	49.60	50.50	46.20	46.00	48.80	46.00	48.20	46.20
Plastic limit "PL"(%)	27.76	28.54	36.5	36.07	36.07	35.02	37.75	35.71	36.93	36.96
Plasticity Index "PI" (%)	18.26	20.66	13.10	14.43	10.13	10.98	11.05	10.29	11.27	9.24
Percentage passing sieve No. 200 (0.075mm)	35.47	53.40	56.97	52.00	48.10	49.30	47.90	53.37	62.60	63.70
AASHTO classification	A-7-6	A-7-6	A-7-6	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5
Plasticity	CI	CI	CI	CI	MI	MI	MI	MI	MI	MI

TABLE II: PHYSICAL PROPERTIES WITH DEPTH OF WEATHERED GNEISS

KATEREGI SAMPLES								
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**
Natural moisture content (%)	5.70	4.81	4.50	6.10	6.50	5.90	5.32	5.26
Specific gravity Gs	2.92	2.68	2.62	2.58	2.69	2.68	2.71	2.71
Liquid limit ''LL" (%)	29.00	30.80	30.50	30.00	29.00	27.80	28.60	30.80
Plastic limit "PL"(%)	0.00	21.12	25.76	24.19	27.37	0.00	0.00	0.00
Plasticity Index "PI" (%)	29.00	9.68	4.74	5.81	1.63	27.80	28.60	30.80
Percentage passing sieve No. 200 (0.075mm)	27.87	34.00	29.30	30.80	28.87	24.57	27.63	24.67
AASHTO classification	A-6	A-5	A-4	A-4	A-4	A-6	A-6	A-4
Plasticity	CL	ML	ML	ML	ML	CL	CL	CL

TABLE III: PHYSICAL PROPERTIES WITH DEPTH OF WEATHERED SCHIST

TUDUN FULANI							
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**
Natural moisture content (%)	5.96	5.57	8.88	6.35	8.22	4.60	4.44
Specific gravity Gs	2.58	2.62	2.60	2.65	2.62	2.57	2.66
Liquid limit ''LL" (%)	36.00	35.01	35.50	40.00	39.10	39.00	39.90
Plastic limit "PL"(%)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Plasticity Index "PI" (%)	36.00	35.01	35.50	40.00	39.10	39.00	39.90
Percentage passing seive No. 200 (0.075mm)	40.77	34.80	22.17	30.73	37.23	32.23	29.90
AASHTO classification	A-6	A-2-6	A-2-6	A-2-6	A-6	A-2-6	A-2-6
Plasticity	CI	CI	CI	CI	CI	CI	CI

3.2 Mineralogical Composition

The mineralogical compositions of the studied samples are shown in table IV, V and VI and figure 1, 2 and 3 showing the XRD diffractogram. The XRD results show the presence of clay and non-clay minerals with high presence of quartz in all the samples.

The X-Ray pattern for weathered granite indicates the presence of clay minerals of kaolinite as the dominant mineral, montmorillonite, mica called phlogopite; the trio showing presence from the top to the bottom of the profile, saponite between 2.5 to 5m depth and mica called annite occuring only at depth 5.0m, all showing a decreasing trend from the top of the profile to 4.5m depth, with with their presence increasing at 5m depth.

However, Potassium Aluminum Silicate, Magnisium Silicate Hydroxide, Magnisium Aluminium Oxide, and Potassium Aluminium Hydride was introduced into the profile at 2.5m, with an increasing trend to 5.0m. Chemical weathering can results in some minerals disappearing partially or fully, and new compounds formed (Venkatramaiah 2006).

The clay mineral that is mostly responsible for expansiveness belongs to the montmorillonite group (Murthy 2002) therefore the presence of kaolinite combined with monmorilonite may be responsible for the intermediate plasticity of samples collected from this location.

TABLE IV: MINEROLOGY OF WEATHERED GRANITE

Granite										
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**
Quartz	96.46	46.77	46.77	56.13	35.45	35.48	35.45	35.45	12.61	171.73
Kaolinite	37.84	37.84	37.84	37.84	26.83	22.71	16.52	16.52	24.23	131.25
Montmorillonite	11.01	11.01	11.01	11.01	11.01	11.01	11.01	11.01	11.01	50
Phlogopite	4.10	4.10	9.84	5.26	5.25	2.68	2.68	2.68	2.68	41.67
Anorthoclase	4.17	-	-	-	-	-	-	-	-	-
Saponite	-	-	-	-	2.54	2.54	2.18	2.18	2.18	9.90
Annite	-	-	-	-	-	-	-	-	-	22.92

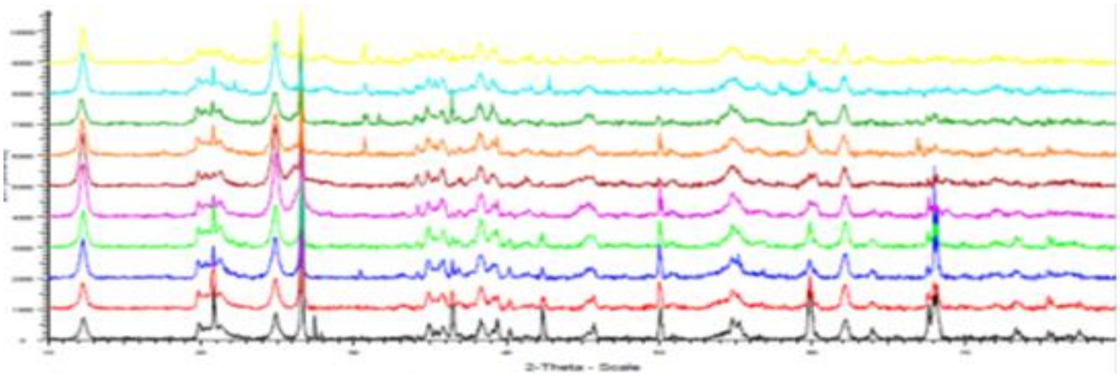

FIGURE 1: XRD DIFFRACTOGRAM OF WEATHERED GRANITE

The XRD result for weathered gneiss shows presence of non-clay mineral such as albite of the feldspar group; existing all through the profile, Magnesioarfvedsonite, and Gregoryite; identified between 0.5m to 2.5m depth, Arfvedsonite and Hydrobiotite, while clay minerals such as Phlogopit and Biotite of the mica group shows presence only at 3.0m depth. This location records the least plasticity, this may due to the dominance of non-clay minerals or as a result of the age of weathering.

TABLE V: MINEROLOGY OF WEATHERED GNEISS

Gneiss								
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**
Quartz	44.70	83.19	77.67	100.91	57.64	117.81	151.78	151.78
Albite	22.30	14.58	38.75	43.84	25.04	14.93	31.25	31.25
Magnesioarfvedsonite	6.92	12.89	12.03	13.82	9.72	-	-	-
Gregoryite	3.60	6.70	6.25	-	-	-	-	-
Arfvedsonite	-	6.95	4.74	-	-	8.48	6.33	6.33
Hydrobiotite	-	-	7.81	-	-	-	-	-
Carbon	-	-	-	-	-	23.17	48.50	48.50
Phlogopite	-	-	-	-	-	2.08	-	-
Biotite	-	-	-	-	-	2.08	-	-

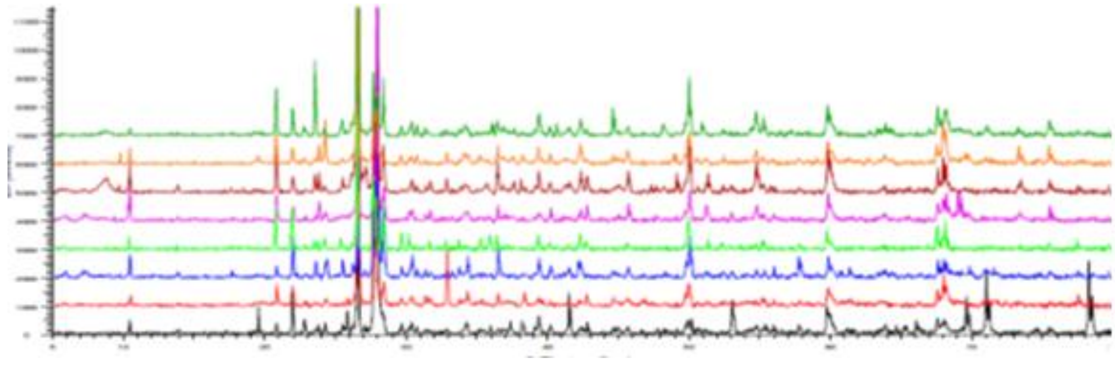

FIGURE 2: XRD DIFFRACTOGRAM OF SAMPLES FROM WEATHERED GNEISS

The XRD pattern for samples from weatherd schist, shows illite as dominant clay mineral, with other minerals such as albite; a non-clay mineral, showing presence from depth 0.5m to the bedrock at depth 3.5m, with kaolinite showing presence at depth 0.5, 1.0, 1.5, 3.0 and 3.5m, phlogopite of mica group appearing between 1.5 and 2.5m depth, biotite of mica group while nontronite of smectite group showing presence at depth 2.5m and 3.0m respectively. The presence of illite, combined with other clay minerals may be responsible for the low plasticity of samples collected from this location

TABLE VI: MINEROLOGY OF WEATHERED SCHIST

Schist							
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**
Quartz	164.67	91.29	173.25	120.50	50	153.22	90.87
Illite	32.01	5.26	44.93	4.46	4.17	44.56	9.92
Albite	68.91	38.20	72.50	5.04	5.04	85.48	50.70
Kaolinite	12.82	4.74	9.00	-	-	8.94	5.30
Phlogopite	-	-	27.10	10.42	10.42	-	-
Biotite	-	-	-	-	4.17	-	-
Nontronite	-	-	-	-	-	4.17	-

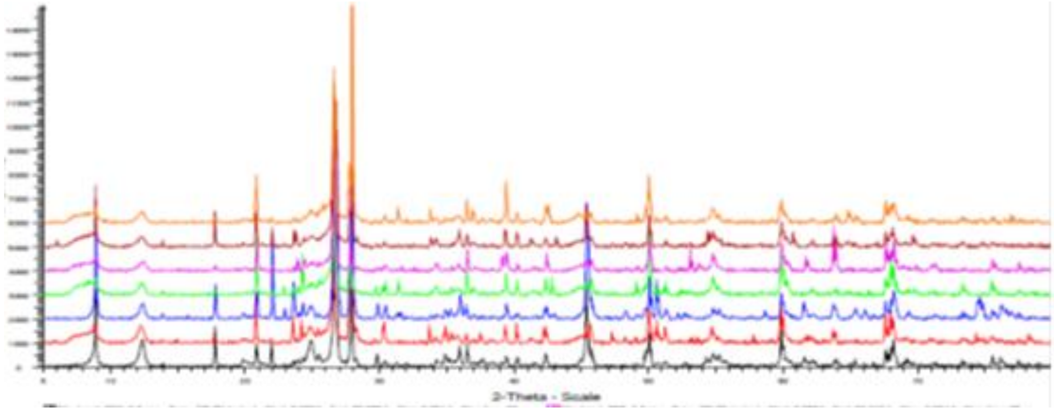

FIGURE 3: XRD DIFFRACTOGRAM OF SAMPLES FROM WEATHERED SCHIST

4 CONCLUSION

The mineralogy of soils influences its performance such that the fundamental understanding of their engineering behavior is provided by investigating their physical and mineralogical properties from different locations and depth.

The high quartz content revealed by the X-ray diffraction accounts of all locations studied is responsible for the high percentage of sand size fraction observed.

The mineralogical investigations indicate the presence high swelling clay montmorillonite on only samples from the granite basement; this location indicates the presence of clay minerals such as kaolinite, montmorillonite, phlogopite saponite and anite, while samples of weathered schist showing presence of clay minerals such as illite, kaolinite, phlogotite, biotite and nontronite which could be responsible for their intermediate plasticity, and that of weathered gneiss show the presence swelling clays of biotite and phlogotite only at 3m depth, This could also be responsible for their low plasticity of this soils. This result is in agreement with previous finding reported on minerology of lateritic soils derived from weathered granite in Minna by Alhassan and et al (2012) where kaolinite was found to be the dominant mineral.

From the foregoing, conclusion could be drawn that the presence of clay minerals derived from these basement complex weathered rocks is responsible for a major contributory factor to the behaviour of soils. The results of this project should be adopted as a basis for further studies and investigation of schist, granite and gneiss weathered rocks in the study area.

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Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

EVALUATION OF PHYSICAL AND GEOTECHNICAL CHARACTERISTIC OF RESIDUAL PROFILE WITH DEPTH OF THREE DIFFERENT BASEMENT COMPLEXES IN NIGER STATE

*Ibrahim, A.[1]; Alhassan, M.[1]; & Alhaji, M. M.[1]
Civil Engineering Department, Federal University of Technology, Minna, Nigeria
*Corresponding Author.agwai2010@gmail.com

ABSTRACT
Disturbed soil samples were collected from three different trial pit at depths of 0.5m interval, to bedrock or 5.0m. at three different basement complexes of (Granite, Schist and Gneiss) physical and geotechnical properties test were carried out on the disturbed samples, test carried out includes index properties, compaction and California Bearing Ratio (CBR), the compaction and California Bearing Ratio (CBR) test of all the soil samples were prepared using British Standard Heavy (BSH) .Evaluation of the variation of physical and geotechnical properties of the weathered basement profiles with depth were made. The result shows that soil sample of Granite and Schist has intermediate plasticity, while that of Gneiss has a low plasticity. The compaction properties of Granite, Schist and Gneiss has maximum dry density (MDD) of 1.9701kg/cm3,2.430kg/cm3 and 2.450kg/cm3 respectively and the Optimum Moisture Content (OMC) of 15.40%, 18.00% and 9.80% accordingly. While that of the California Bearing Ratio (CBR) for both Granite, Schist and Gneiss has the maximum values of 40.42%, 25.85% and 21.60% respectively.

Keywords: *Basement complex, California Bearing Ratio (CBR), Geotechnical, Gneiss, Schist, Residual profile.*

1 INTRODUCTION

Soils are aggregates of mineral particles, and together with air and/or water in the void spaces, they form three-phase systems. A large portion of the earth's surface is covered by soils, and they are widely used as construction and foundation materials. (Braja, 2007). To civil engineers 'soil' means, the loose unconsolidated inorganic material on the earth's crust produced by the disintegration of rocks, overlying hard rock with or without organic matter. Foundations of all structures have to be placed on or in such soil, which is the primary reason for our interest as Civil Engineers in its engineering behavior (Venkatramaiah 2006). Many soil workers in Nigeria may treat laterite weathering products and other residual soils as uniform with depth. This is the main reason why single or few soil test results carried out on soil from trial pit at the beginning of excavation is used to represent soils at deeper depth. The composite and complex nature of the weathering materials and the variation in the morphological, geotechnical, mineralogical and chemical properties of the materials with depth of the weathering profile is usually not taken into consideration. (Adekoya, 1987).

Soils as construction materials are completely different from materials of structural mechanics. The reason is that soils are aggregation of particles, formed by the weathering of rocks, and the behavior of soils is a legacy of natural processes, from their origin to the actual state. This gives soils a character of inhomogeneity and anisotropy, and basic parameters, such as strength, stiffness and hydraulic conductivity, need to be measured instead of being specified and may vary over a wide range. The discrete particles that make up soils are not strongly bounded together, they are free to move relatively among themselves and, when a soil element deforms, the overall deformation is essentially the result of relative sliding between particles and rotation of particles. Therefore, it is not surprising that soil behavior is highly non-linear and irreversible.

Furthermore, it must be realized that the voids or pores between particles are filled with water, or there may be more than one fluid, typically water and air at near surface depths, but there could be water and liquid and gaseous hydrocarbon in certain circumstances. It follows that soils are multi-phase materials, their behavior being influenced by the interaction between solids and fluids (Lancellotta 2009).

Soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and/or air. Weak cementation can be due to carbonates or oxides precipitated between the particles or due to organic matter. If the products of weathering remain at their original location, they constitute a residual soil. If the products are transported and deposited in a different location they constitute a transported soil, the agents of transportation being gravity, wind, water and glaciers (Craig 2004).

Soils which are formed by weathering of rocks may remain in position at the place of region. In that case these are 'Residual Soils'. These may get transported from the place of origin by various agencies such as wind, water, ice, gravity, etc. In this case these are termed ''Transported soil''. Residual soils differ very much from transported soils in their characteristics and engineering behavior. The degree of disintegration may vary greatly throughout a residual soil mass and hence, only a gradual transition into rock is to be expected. An important characteristic of these soils is that the sizes of grains are not definite because of the partially disintegrated condition. The grains may break into smaller grains with the application of a little pressure.

The residual soil profile may be divided into three zones: (i) the upper zone in which there is a high degree of weathering and removal of material; (ii) the intermediate zone in which there is some degree of weathering in the top portion and some deposition in the bottom portion; and (iii) the partially weathered zone where there is the transition from the weathered material to the unweathered parent rock. Residual soils tend to be more abundant in humid and warm zones where conditions are favorable to chemical weathering of rocks and have sufficient vegetation to keep the products of weathering from being easily transported as sediments (Venkatramaiah 2006).

1.1. Residual Soils and Transported Soils

Residual soils are formed by the in-situ physical and chemical weathering of underlying rock, while sedimentary soils are formed by a process of erosion and transportation followed by deposition and consolidation under their own weight. In addition, the latter may undergo further alteration after deposition due to processes such as secondary consolidation, leaching and thixotropic effects (Bjerrum 1967a). Unloading processes may produce over consolidated clay's. Sedimentary soils may also be subjected to the development of inter particle bonds as well as other post deposited effects (Bjerrum 1967 b). As bonds develop with time in residual soils, hardening occurs. The reverse will be normal trend, as bonds and cementation are broken down by the weathering process.

The behavior of a soil, weather residual or sedimentary, can be considered to depend on two factors; firstly, the nature of the soil particles themselves (i.e., their size, shape, and mineralogical compositions); and, secondly, the particular state in which these particles exists in the soil in its undisturbed condition. For convenience, these factors will be referred to respectively as composition and structure. The term structure will be used here to refer to those aspects of the soil that are peculiar to the soil in its undisturbed state, such as inter particle bonding or cementation, and that are eliminated or destroyed by remolding the soil. With residual soils, the structure results directly from whatever in-situ physical and chemical processes have taken place in altering the parent rock to become a residual soil. With sedimentary soils, the picture appears more complex, as a variety of factors have been involved in the formation of the final structure covering the deposition process, the loading and unloading history, and the post-depositional processes mentioned. The important factors, however, generally lead to a degree of homogeneity and predictability with sedimentary soils that is absent in residual soils (Vaughan, 1988). These are:

a) The sorting process that takes place during erosion, transportation and deposition of sedimentary soils tends to produce homogeneous deposits.

b) Stress history is generally a dominant factor in influencing the behavior of sedimentary soils, and leads to the well-known division of these soils to normally consolidated, and over consolidated materials. The absence of these factors with residual soils means that, in practice, structural effects may be generally more complex and important with residual soils than sedimentary soils. In addition to structural effects, the behavior of residual soil may be markedly influenced by the presence of clay minerals not found in sedimentary soils. Halloysite and allophane in particular are common in volcanic residual soils and have quite different properties from the minerals normally found in sedimentary soils. Hence, both composition and structure should be taken into account in seeking to explain the distinctive aspects of residual soil behavior. The above discussion terms out that there are clear differences between the factors influencing the transported and residual soils. In transported soils the particles are "pre-formed, delivered by some transporting agency and deposited in a certain way.

The soil is then subjected to an increase in effective stress due to increasing burial (normal consolidation) and, sometimes, a subsequent decrease due to removal of overburden (over-consolidation). In the special case of clay deposited from suspension En water, this stress history wholly determines porosity and particle packing. Classical soil mechanics has been developed for particular materials with properties wholly arising from initial porosity and subsequent stress history (Vaughan, 1988).

According to Zonn (1986), all tropical and subtropical soils can be grouped in terms of their profile as:

Soils whose profile depends on textural or structural differentiation.

Soils whose profile are mainly differentiated by texture and

Soil that can further be differentiated by the morphology of the individual generic horizons.

1.2. Soil Formation

Soil are formed by disintegration (technically called weathering), of rocks. The disintegrated or weathered materials may either be found deposited at its own place of origin, or may get transported by agents like water, wind ,ice etc. before deposition . In the first case, the resultant soil is called residual soil; and in the second case is called transported soil. More over depending upon whether the sediments are transported y water, ice, or wind, the soil are called alluvial, glacial, or aeolin respectively. Mechanical weathering disintegrate a pre-existing rock into smaller fragment, while chemical weathering act on this small fragments, rearranged the element into new minerals and thus decomposed them.(Garg 2012).

Soil is defined as a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water. By contrast rock is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces. The process of weathering of the rock decreases the cohesive forces binding the mineral grains and leads to the disintegration of bigger masses to smaller and smaller particles. Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition. In mechanical weathering the expansive forces of freezing water in fissures, due to sudden changes of temperature or due to the abrasion of rock by moving water or glaciers. Temperature changes of sufficient amplitude and frequency bring about changes in the volume of the rocks in the superficial layers of the earth's crust in terms of expansion and contraction. Such a volume change sets up tensile and shear stresses in the rock ultimately leading to the fracture of even large rocks. This type of rock weathering takes place in a very significant manner in arid climates where free, extreme atmospheric radiation brings about considerable variation in temperature at sunrise and sun set, and the chemical weathering (Murthy 2012).

Composition, structure and properties of a natural soil element are the result of its geological history. This history includes weathering, transportation, deposition and post-depositional changes. The actual state of homogeneity and anisotropy of any soil deposit is related to this formational history and to subsequent changes, summarized in Figure 1.1 and discussed in detail in the sequel (Lancellotta 2009).

This work is therefore aimed at determining the variation of the physical and geotechnical properties of residual soils derived from three different basement complexes of weathered schist, granite and gneiss at Tudun Fulani dam site and Birgi village all in Minna and Kateregi mining village in Kacha local government area all in Niger state respectively.

2 MATERIALS AND METHODS

Three locations were identified on the basement complexes of weathered schist, granite and gneiss origin. Three trial pits were then dug manually on each of the identified location. Both disturbed and undisturbed soil samples were collected from 0.5m to bedrock or 5.0m depth at interval of 0.5m and profile inspected manually. The sample were carefully labeled in sample bags and then taken to the laboratory in sealed polythene bags to prevent contamination and loss of moisture.

Physical properties tests including, natural moisture content (NMC), sieve analysis, liquid limit, plastic limit, specific gravity (S.G) were conducted while the geotechnical properties tests include compaction – British standard heavy (BSH) and California bearing ratio(CBR) which was conducted at a predetermined optimum moisture content (OMC) and maximum dry density (MDD). All these tests were conducted according to the procedure highlighted in BS8110 (1990) with some modifications where necessary.

3 DISCUSSION AND RESULTS

3.1. Physical properties

The physical properties of studied sample are shown in table 1, 2 and 3, the natural moisture content (NMC) of granite basement increased between 10.30%, 16.90%, 20.50%, 24.10% at 0.5m to 2.0m where it decreased again between 22.40%, 21.80%, 22.60%, and 22.70% at 2.5m to 4.0m and subsequently increased to 25.90% and 26.30% at 4.5m and 5.0m, with that of weathered gneiss ranges from 9.70%, 10.81%, 13.78% at 0.5m -1.5m then decreased to 12.97%,11.49% 8.31% and 7.62% at 2.0m-3.5m and increased to 7.90% at 4.0m, while that of weathered rock from schist from 5.96% to 5.57% at 0.5m -1.0m and increased to 8.88% at 1.5m then decreased to 6.35%, at 2.0m increase to 8.22%, at 2.5m before decreasing to 4.60% and 4.44% at 3.0m and 3.5m respectively. Natural moisture content is the function of void ratios and the specific gravity of the samples. Although it is not a constant property of soils. This values are consistent with the fine content of the clays (Ademila *et al.*, 2017).

The liquid limit for Granite basement increased from 46.02% at o.5m to 50.50% at 2.0m depth from where it decreased to 46.20% at 2.5m and increased to 48.80% at 3.5m and falls to 46.00% at 4.0m and increased to 48.20% at 4.5m before finally decreased to 46.20% at 5.0m, with that of gneiss basement increased from 29.0% at 0.5m to 30.80% at 1.0m and continuous decreasing from 30.50% at 1.5m 30.00% 2.0m, 29.00%, at 2.5m, 27.80% at 3.0m, and increased to 28.60% and 30.80% at 3.5m and 4.0m. While that of rock from schist basement decreased from 36.005 at 0.5m to 35.01% at 1.0m and increased from 35.50% to 40.00% at 1.5m and 2.0m then falls to 39.10% at 2.5m decreased to 39.00% at 3.0m and finally increased to 39.90% at 3.5m. Liquid limit less than 30% indicate low plasticity, between 35% and 50% are intermediate plasticity. Between 50% and 70% high plasticity, between 70% and 90% indicates very high. plasticity and greater than 90% indicates extremely high plasticity (R. Whitlow 1995).

On this basis the granite and schist basement are termed to be of intermediate plasticity, while that of gneiss basement show low plasticity. Liquid limit of soil use for barriers lining should be less than 90% (Declan *et al.*, 2003). Therefore, all the samples studied can be used as barriers liners hence they are less than 90%. Plastic limit for granite basement decreased from 37.75% at 3.5m depth to 27.75% at 0.5m depth and that of gneiss basement varies from 0.00% at 0.5m to 27.37% at 2.5m. While that of schist basement was 0.00% for all depths. This shows that the granite basement is suitable for production of ceramic clay (Grimsha, 1971) prescribed a range of 10-60% for clay used in ceramic production.

The value of plasticity index (an indicator of soil plasticity) of granite basement ranged from 18.26% at 0.5m depth and 9.24% at 5.0m depth and that of gneiss basement ranged from 30.80% at 5.0m and 4.74% at 1.5m while that of schist basement ranges from 40.00% at 2.0m depth and 35.01% at 1.0m depth. The schist basement has the highest plasticity index of 40.0% and gneiss having the lowest plasticity index, the schist basement has a high swelling and high compressibility characteristics. The difference in the plasticity index may be due to the presence of clay minerals in the mineralogy of the soil samples (Rowe et al., 1995).

Specific gravity for granite basement ranged from 2.77 at 1.5m depth to 2.54 at 2.5m depth and that of gneiss basement ranged from 2.92 at 0.5m depth to 2.58 at 2.0m depth, while that of schist basement ranged from 2.66 at 3.5m depth and 2.58 at 0.5m depth. Specific gravity is an important property in the identification and evaluation of aggregate parameters for construction purposes. The higher the specific gravity of the soil, the better it is for construction purposes (Ademila *et al., 2017).*

TABLE I: PHYSICAL PROPERTIES OF WEATHERED GRANITE WITH DEPTH

BIRGI SAMPLES										
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**
Natural moisture content (%)	10.30	16.90	20.50	24.10	22.40	21.80	22.60	22.70	25.90	26.30
Specific gravity Gs	2.71	2.64	2.77	2.68	2.54	2.59	2.66	2.69	2.68	2.66
Liquid limit ''LL" (%)	46.02	49.20	49.60	50.50	46.20	46.00	48.80	46.00	48.20	46.20
Plastic limit "PL"(%)	27.70	28.54	36.5	36.07	36.07	35.02	37.75	35.71	36.93	36.96
Plasticity Index "PI" (%)	18.26	20.66	13.10	14.43	10.13	10.98	11.05	10.29	11.27	9.24
Percentage passing sieve No. 200 (0.075mm)	35.47	53.40	56.97	52.00	48.10	49.30	47.90	53.37	62.60	63.70
AASHTO classification	A-7-6	A-7-6	A-7-6	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5
Plasticity	CI	CI	CI	CI	MI	MI	MI	MI	MI	MI

TABLE II: PHYSICAL PROPERTIES OF WEATHERED GNEISS WITH DEPTH

KATEREGI SAMPLES								
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**
Natural moisture content (%)	9.70	10.81	13.78	12.97	11.49	8.31	7.62	7.90
Specific gravity Gs	2.92	2.68	2.62	2.58	2.69	2.68	2.71	2.71
Liquid limit ''LL" (%)	29.00	30.80	30.50	30.00	29.00	27.80	28.60	30.80
Plastic limit "PL"(%)	0.00	21.12	25.76	24.19	27.37	0.00	0.00	0.00
Plasticity Index "PI" (%)	29.00	9.68	4.74	5.81	1.63	27.80	28.60	30.80
Percentage passing sieve No. 200 (0.075mm)	27.87	34.00	29.30	30.80	28.87	24.57	27.63	24.67
AASHTO classification	A-6	A-5	A-4	A-4	A-4	A-6	A-6	A-2-6
Plasticity	CL	ML	ML	ML	ML	CL	CL	CL

TABLE III: PHYSICAL PROPERTIES OF WEATHERED SCHIST WITH DEPTH

TUDUN FULANI							
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**
Natural moisture content (%)	5.96	5.57	8.88	6.35	8.22	4.60	4.44
Specific gravity Gs	2.58	2.62	2.60	2.65	2.62	2.57	2.66
Liquid limit ''LL" (%)	36.00	35.01	35.50	40.00	39.10	39.00	39.90
Plastic limit "PL"(%)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Plasticity Index "PI" (%)	36.00	35.01	35.50	40.00	39.10	39.00	39.90
Percentage passing sieve No. 200 (0.075mm)	40.77	34.80	22.17	30.73	37.23	32.23	29.90
AASHTO classification	A-6	A-2-6	A-2-6	A-2-6	A-6	A-6	A-2-6
Plasticity	CI	CI	CI	CI	CI	CI	CI

3.2. Geotechnical properties

3.2.1. Compaction

The maximum dry density (MDD) for granite weathered basement varied between 1.9701 Kg/ cm3 and 1.8753 Kg/cm3 at depth 3.5m and 5.0m.while the optimum moisture content (OMC) ranged between 15.40% and 8.54% at depth of 1.5m and 4.0m (Table 1. Also the maximum dry density (MDD) for gneiss basement varied between2.450kg/cm3 and 2.260kg/cm3 at depth of 3.5m and 1.0m while the optimum moisture content (OMC) ranged between 9.80% and 7.00% at depth of 1.0m to 4.0m. (Table ii. Also that of schist basement varied between2.430kg/cm3 and 1.910kg/cm3 at depth 2.0m and 3.5m and the optimal moisture content varied from 18.00% to 9.00% at 1.0m and 3.0m depth. Table iii. The observed results for all the samples show that the higher the MDD, the lower the OMC. The results of the MDD and OMC show that the samples can be used as filling and embankment materials because they fall within the specification (FMW&H.2000).

TABLE I: GEOTECHNICAL PROPERTIES OF WEATHERED GRANITE WITH DEPTH

BIRGI SAMPLES										
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**
Maximum dry density (kg/cm3)	1.954	1.927	1.875	1.902	1.950	1.950	1.970	1.969	1.929	1.892
Optimum moisture content (%)	12.54	14.16	15.40	12.94	12.10	12.10	10.32	8.54	10.46	11.83
California bearing ratio (CBR)	31.63	25.5	40.42	29.92	25.61	32.02	14.43	27.10	19.50	15.80

TABLE II: GEOTECHNICAL PROPERTIES OF WEATHERED GNEISS WITH DEPTH

KATEREGI SAMPLES								
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**
Maximum dry density (kg/cm3)	2.274	2.260	2.360	2.350	2.340	2.450	2.450	2.400
Optimum moisture content (%)	9.68	9.80	7.60	8.40	9.70	7.40	7.40	7.00
California bearing ratio (CBR)	5.2	0.40	0.62	4.48	12.4	20.5	16.2	21.60

TABLE III: GEOTECHNICAL PROPERTIES OF WEATHERED SCHIST WITH DEPTH

TUDUN FULANI SAMPLES							
Depth (m)	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**
Maximum dry density (kg/cm3)	2.108	2.080	2.080	1.910	2.100	2.090	2.430
Optimum moisture content (%)	10.20	18.00	17.50	17.00	11.00	9.00	11.50
California bearing ratio (CBR)	6.08	0.51	5.50	0.80	17.90	25.85	24.00

3.3. California bearing ratio (CBR)

The California bearing ratio (CBR) is often used in estimation of the bearing capacity of soil, which are used as highway sub-base, sub-grade and base course materials, the result obtained from these work shows the unsoaked CBR value for granite basement rock ranged from 40.42% at 1.5m and 14.43% at 3.5m depth, while that of gneiss basement rock ranged between 21.60% at 4.0m and 0.40% at 1.0m, also the schist basement rock CBR values ranged between 25.85% at 3.0m and 0.51% at 1.0m with these results none of the samples have meet the required value of 80% for unsoaked and 30% for soaked samples as recommended for highway sub-base and sub grade materials by federal ministry of works and housing (FMW&H, 2000).

4 CONCLUSION

It was observed that all the soil samples studied possess liquid limits within the range considered suitable for use in landfill liner systems. The compaction values of all the soils are considered good, if 100% of the MDD and OMC are attained during field compaction. The relatively good values of compaction properties possessed by these soils makes them good for engineering construction materials.

The CBR test shows that the entire studied soil did not meet up with the general specification requirement for roads and bridges recommended by federal ministry of works, which is 80% for unsoaked and 30% for soaked but can be used as filling materials.

The granite and schist basement samples are granular materials with plasticity index greater than 12% which may be considered suitable for barrier as notable increase in permeability is not expected, and can be used to make structural blocks, bricks.

It is observed that all physical properties of residual soils studied varied with depth and results obtained for soils at one layer should not be used to represent the result of soils at other layers for the avoidance of in accurate design of soil structures which may lead to its failure on application of the first load.

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COMPACTION AND CONSOLIDATION CHARACTERISTICS OF A-7-6 SOIL

*Otene, G.O.; Adejumo, T.W.E.; Alhaji, M.M.
Department of Civil Engineering Federal University of Technology, Minna, Niger State.

ABSTRACT

This study presents an investigation of the compaction and consolidation characteristics of black clay soil, which belongs to A – 7 – 6 soil class. The soil samples were collected at depths 0.5, 1.0, 1.5 and 2.0 metres respectively from selected active pits of clay deposits in Bako, a village in Gwagwalada Area Council of the Federal Capital Territory, Abuja, Nigeria. Compaction and consolidation characteristics of collected samples under axial and compressive loads were investigated. Maximum Dry Density and Optimum Moisture Content as well as recompression/compression index were calculated from the compaction and consolidation results respectively. The values of compaction and consolidation characteristics from the experimental results were then compared with those obtained using different empirical correlations from literature. The Liquid Limit for the soil sample is 64.29 % while Plasticity Index is 34.29 %. The compression index (*Cc*) values vary slightly with the applied compaction energy levels. The Modified Proctor energy level recorded the highest *Cc* of 0.2989 which reduces to 0.2823 at the Standard proctor compaction energy level. The West Africa Standard compaction energy level gave a value of 0.227. The lowest value of 0.1531 was obtained for the Reduced Standard Proctor compaction energy level. The soil's recompression index (*Cr*) varies from 0.0529 at the Standard proctor compaction energy level to 0.0229 at the Reduced Standard Proctor energy level while its value was maintained at 0.0059 for both the West Africa Standard and Modified Proctor energy levels. The soil's Maximum Dry Density (MDD) has its highest and lowest values obtained to be 1.99 g/cm^3 and 1.51 g/cm^3 at the Modified Proctor and Reduced Standard Proctor compaction energy levels respectively whereas the highest and lowest values of its Optimum Moisture Content (OMC) were established to be 29.1% and 21.1% at Reduced Standard Proctor and Modified Proctor energy levels respectively.

Keywords: *Compression Index (C_c), Maximum Dry Density (MDD), Optimum Moisture Content (OMC).Recompression Index (C_r)*

1.0 INTRODUCTION

A – 7 – 6 soils are Silt-Clay soils of high plasticity whose plasticity index is equal or greater than 30 and its liquid limit is in excess of 60; this is in accordance with AASHTO method of soil classification. The soil samples obtained from selected pits in Bako, Gwagwalada area of the Federal Capital Territory, Abuja Nigeria was classified as A – 7 - 6, dark grayish coloured clay, by using the established results of Atterberg limit test conducted in the Geotechnics departments' laboratory of the Federal University of Technology Minna, Nigeria. In Nigeria, this soil type is predominantly found in the North-Eastern part, along the Lake Chad basin, and partly within the Benue trough (Ola, 1981; NBBRRI, 1983; Osinubi *et al.*, 2009).

This clay type of soil is established to pose construction problems to engineers owing to its behaviour with varying moisture content. With rapid development in soil improvement, construction technique and social needs, various constructions of structures are taking place. The possibility of good construction sites founded on clay formations to build structures is difficult due to its poor strength and deformation characteristics when wet (Fulzele *et al*., 2016). This study focused on black clay soil, an **A – 7 - 6** soil by classification, and its compaction and consolidation characteristics.

The compaction or densification occurs as a direct result of the application of mechanical or axial loading, and are essentially targeted at rearranging the soil particles and reducing its void ratio. Volume changes are incurred as a result of reduction in the quantity of air voids; water content remaining constant. Since it is impractical to squeeze out all the air, the as-compacted condition is a partly saturated one.

While compaction is densification, the achievement of high unit weight is not the direct objective. Rather, the intent is to produce a soil structure which will exhibit and retain a requisite level of integrity throughout its designed service life. The properties which must be imparted to the soil vary with the study, but such descriptors as strength, compressibility, and flexibility are commonly involved and this discussion focuses primarily upon the behavior and or characteristics of compacted **A – 7 - 6** soil and its corresponding consolidation characteristics (Altschaeffl *et al.*, 1983).

Consolidation on the other hand is the process that involves a decrease in volume by the expulsion of pore water under long term static loads from a saturated soil without replacement of the pore water by air. This process involves a gradual compression occurring simultaneously with a flow of water out of the soil mass with the gradual transfer of applied pressure from the pore water to the soil mineral skeleton. When a saturated clay-water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores resulting thereby in an excess pore water pressure. If drainage is permitted, the resulting hydraulic gradients initiate a flow of water out of the clay mass resulting in the compression of the mass thereby transferring a portion of the applied stress to the soil skeleton which in turn causes a reduction in the excess pore pressure. The water that is dissipated in a soil sample when load is applied is called free water (Murthy, 2009).

Casagrande was the first to develop a graphical method of determining the preconsolidation pressure of a clay deposit using a semi-logarithmic graph to evaluate its void ratio and effective stress. The graph consists of a recompression curve with its slope referred as recompression index C_r, and a virgin compression curve whose slope is also known as the compression index C_c. These indices are very essential in the evaluation of the magnitude of consolidation settlement (Casagrande, 1936).

The primary settlement of the clay stratum was determined using equation 1.2.

$$C_c = \frac{\Delta e}{\Delta \log \sigma'} \qquad (1.1)$$

where C_c is the compression index, Δe is the change in void ratio, $\Delta\sigma'$ is the change in effective stress.

$$S = \frac{CcHo}{1+e0} Log \frac{Po+\Delta p}{Po} \qquad (1.2)$$

where S is consolidation settlement, Cc is the compression index, H_0 is clay layer thickness, e_o is the initial void ratio, P_0 is the average effective pressure before the application of new load, Δp is the average pressure increase on the clay layer due to the application of new load.

If the soil has been previously loaded in the past by episodes such as a glacier, embankment, and structure, the C_c term is replaced with the recompression index (C_r) term to compute the anticipated settlement up to the estimated preconsolidation pressure. The C_r and C_c values as obtained from the consolidation test (ASTM D2435) and the void ratio is plot against the introduced pressure in increments on the clay soil sample using logarithmic scale. Figure 1 shows a typical consolidation curve with the various portions of the curve described. The C_r of the consolidation curve is the reloading curve portion while C_c is the virgin portion of the consolidation curve.

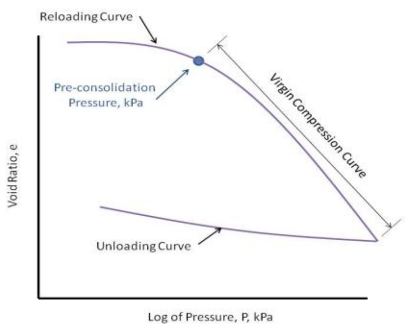

FIGURE 1: TYPICAL CONSOLIDATION CURVE (ASTM D2435: 2003).

2.0 METHODOLOGY

2.1 Compaction Characteristics of A – 7 - 6 Soil Five different compaction energy level tests were conducted on the air dried and pulverized disturbed samples collected at depths 0.5 m to 1.9 m from selected pits in Bako, Gwagwalada area of Abuja in the geotechnics laboratory of the Federal University of Technology Minna according to part1 of BS 1377 (1990). The tests were conducted to determine the soil's Optimum Moisture Contents (OMCs) and Maximum Dry Densities (MDDs) as represented in figure 2.1, using five different compaction energy levels. The Reduced Standard Proctor Compaction energy level (Daniel and Benson, 1990) which involved the use of 2.5 kg rammer falling through 300 mm height with 15 blows and 3 different layers in a 944 m^3 compaction mould, Standard Proctor energy level (Proctor, 1993) involves the use of 2.5 kg rammer falling through 300 mm height, 25 blows, 3 layers in a 944 m^3 mould, West African Standard compaction energy level (Nigeria General Specification for Road and Bridge Works, 1992) employs the use of 4.5 kg rammer with falling height of 300 mm and 10 blows on each of 5 layers in a 944 m^3 mould, Reduced Modified Standard energy level involved the use of 4.5 kg rammer falling through 450 mm with 10 blows, 5 layers in a 944 m^3 mould and the Modified Proctor compaction energy level that employs the use of 4.5 kg rammer falling through 450 mm height with 25 blows, 5 layers in a 944 m^3 mould.
This research work made use of both the light and heavy compaction procedures so as to obtain a wide range of MDDs and OMCs in order to achieve the aim of this study. The sample was thoroughly mixed and the mixture is allowed to equilibrate for 24 hours in a sealed container. After equilibration, the sample was placed in the compaction mould in the respective layers and the specified number of blows required for the various compaction energy levels applied to each layer. The

sample was then extruded, the top side leveled off and the weight and moisture content measured in order to compute the dry density and moisture content.

In the West Africa Standard Proctor and Modified Proctor compaction tests (4.5 kg rammer method), the preparation of samples was similar to that of the Reduced and Standard Proctor compaction energy level tests. But the compaction is done in five layers, 10 and 25 blows, for the West African and Modified Proctor compaction energy levels respectively, applied per layer from a controlled drop height of 450 mm. In both methods, each dry density was plotted against the corresponding moisture content and a smooth curve drawn through the points in order to compute the optimum water content (OMC) and maximum dry density (MDD). The dry density (ρ_d in Mg/m^3) corresponding to a certain moisture content (w in %) can be calculated from equation 1.3.

$$\rho_d = \left[\frac{100}{(100+w)}\right] \rho \tag{1.3}$$

where ρ is the bulk density of each compacted specimen.

2.2 Consolidation Characteristics of A-7-6 Soil

Consolidation tests were carried out using Wykeham Farrance WF24001 One Dimensional Oedometer apparatus for investigating the consolidation characteristics of the A-7-6 (black cotton) soil in accordance with BS 1377 (1990).

After compaction, using the required energy level and its corresponding Optimum Moisture Content (OMC) established from the various compaction energy levels, the sample was placed in the oedometer apparatus with a transparent cell that encased the consolidation mould of 20 mm and 50mm height and diameter respectively. The mould was then covered with a double drainage porous stone which was always lubricated after each consolidation test to prevent friction. The loading beam and hanger was checked to be balanced without weights. The oedometer cell was then filled with water to ensure that the sample remains fully saturated for tests on fully saturated samples (reconstituted samples). In the case of partly saturated samples (intact samples) the water was added at a specified loading level in order to observe swelling and collapse potentials. Each loading increment was maintained for 24 hours and loads were doubled each day. Seven different loadings were applied for seven consecutive days before unloading for a further 24 hours at the eight day. Readings of time and displacement were recorded during each loading stage of the test according to BS 1377 (1990), this is to obtain required consolidation parameters used in the evaluation of consolidation.

3.0 RESULTS AND DISCUSSION

3.1 PHYSICAL PROPERTIES

Table 3.1 gives the range of physical properties of an A – 7 – 6 soil according to Fulzele *et al*, 2016 while table 3.2 contains the physical properties of the study sample.

Table 3.1a: Physical Properties of Clay Soil

Properties	Values
Liquid Limit (LL) %	(40-120) %
Plastic Limit (PL) %	(20 - 60) %
Specific Gravity (G)	2.60 - 2.75
Fine (<75 μm)	(70 - 100) %
Soil Classification BS	CH or MH Clay/Silt of high plasticity

Table 3.1b: Physical Properties of Study Sample

Properties	Values
Liquid Limit (LL) %	64.29 %
Plastic Limit (PL) %	34.29 %
Specific Gravity (G)	2.60
Fine (<75μm)	81.67 %
Soil Classification BS	CH of high plasticity

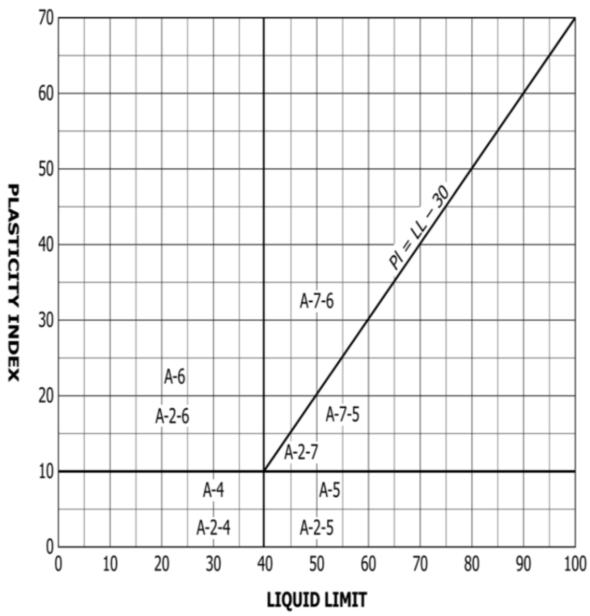

FIGURE 1.2: LIQUID LIMIT AND PLASTICITY INDEX FLANGES FOR SILT-CLAY MATERIALS (AASHTO)

The study sample has a Liquid Limit of 64.29 % and Plasticity Index of 34.29 %, using the chart in figure 1.2, this soil is established to be an **A – 7 - 6** clay soil.

3.2 COMPACTION CHARACTERISTICS

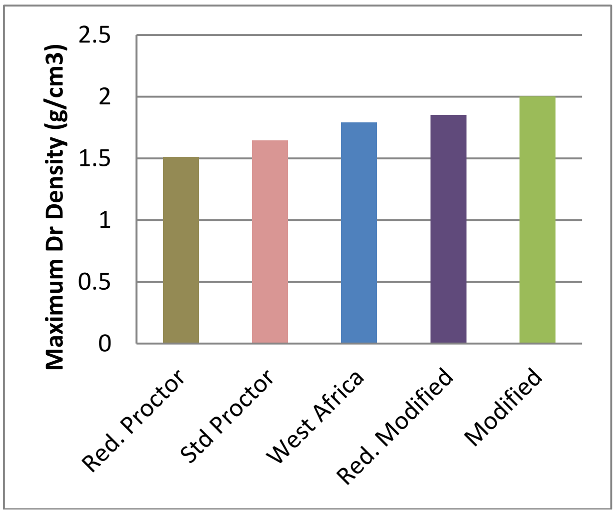

FIGURE 3.1: VARIATION IN MDD RELATIVE TO VARIOUS COMPACTION ENERGY LEVELS

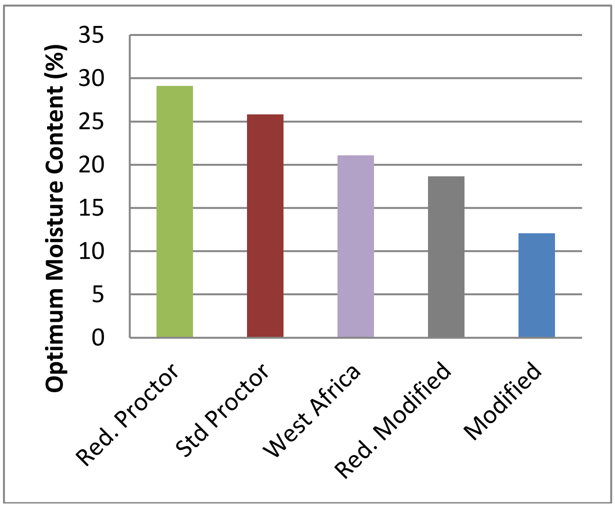

FIGURE 3.2: VARIATION IN OMC RELATIVE TO VARIOUS COMPACTION ENERGY LEVELS

The Maximum Dry Densities (MDDs) and Optimum Moisture Contents (OMCs) obtained from the five different compaction energy levels are as summarized and shown in figures 3.1 and 3.2 respectively.

The compaction test results are in consonant with the findings of Lambe (1958), Daniel and Wu (1993), Daniel and Benson (1990), Lare *et al.*, (2014), Sigh *et al.*, and Mada *et al.*, (2013) as the trend indicates that there is an increase in the dry densities from its lowest value at 1.5116 g/cm^3 using the reduced energy level to its highest value at 1.9997 g/cm^3 using the Modified Proctor compaction energy level as in figure 3.1. The MDD values are in reverse order of the optimum moisture content of the test sample as it decreases in value from 29.14 % using the reduced energy level to its minimum value of 12.09 % in the Modified Proctor energy level.

3.2 CONSOLIDATION CHARACTERISTICS

The various recompression (C_r) and compression (C_c) index results of the consolidation tests conducted on the clay soil sample using different compaction energy levels are as shown in figures 3.3 and 3.4 respectively. Anaylysis of these results indicates that the West Africa Standard and Modified Proctor energy levels have the lowest recompression index of 0.0059 each whereas the highest recompression index was observed to be 0.0529 in the Standard Proctor energy level while the Reduced energy level have an intermediate recompression index of 0.0229. The compression index inturn was observed to increase from 0.1531 at Reduced energy level to 0.2271 at West Africa Standard energy level after which the value decreases from 0.2989 at Modified Proctor energy level to 0.2823 at Standard Proctor energy level.

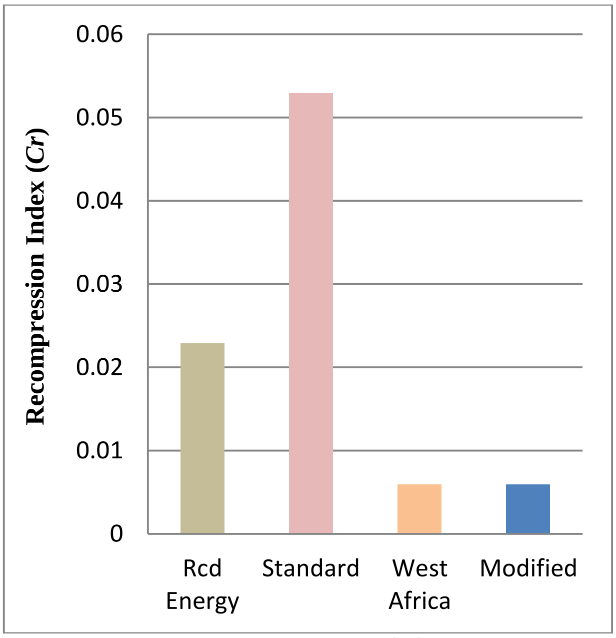

FIGURE 3.3: RECOMPRESSION INDEX (C_R) WITH DIFFERENT COMPACTION ENERGY LEVELS

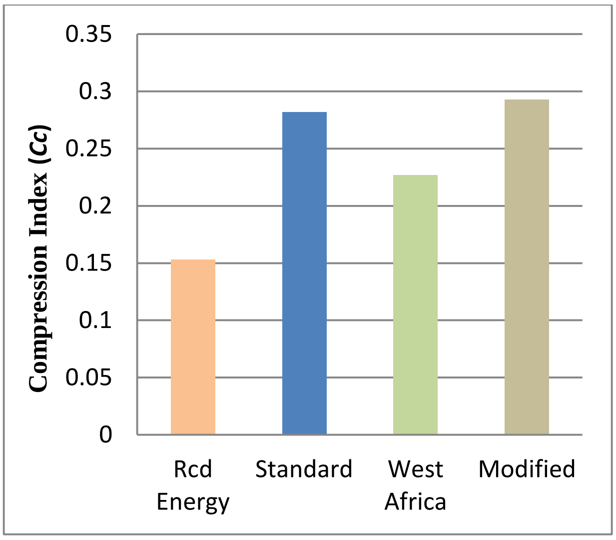

FIGURE 3.4: COMPRESSION INDEX (C_C) WITH DIFFERENT COMPACTION ENERGY LEVELS

The varying C_r and C_c values can be attributed to the water absorptive tendencies of the clay soil under 24 hours sustained loading, compaction efforts, saturation and rate/channel of dissipation of water during consolidation before the addiction of further loads.

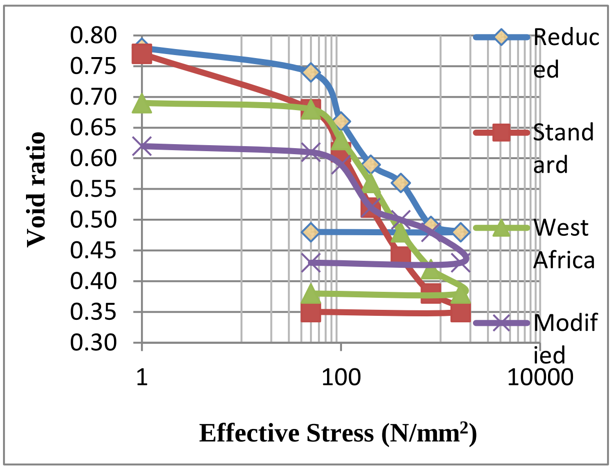

FIGURE 3.5: RELATIONSHIP BETWEEN VOID RATIO AND EFFECTIVE STRESS WITH VARYING COMPACTION ENERGY LEVELS

The graph in Fig 3.5 shows the corresponding relationship between void ratio and effective stress in a semi-log graph for the various compaction energy levels. For each compaction energy level, the relationship is in two phases comprising the recompression phase which collapses to the compression phase when the applied load exceeds the maximum load to which the soil ever experinced in the past. The recompression index (C_r) is obtained from the recompression phase and the compression index (C_c) is obtained from the compression phase. The various indices are applied in evaluating the soil's settlement with respect to the stress history of the soil as to whether the soil is normally consolidated or overconsolidated.

Inferred also from the varying consolidation characteristics (C_r and C_c) with respect to different energy levels are the possibilities of the effects of the various Maximum Dry Densities and Optimum Moisture Contents associated with the respective energy levels.

4.0 CONCLUSION

1. The soil sample from selected pits in Bako, Gwagwalada was classified using AASHTO soil classification system as A-7-6 clay soil.
2. The values of Maximum Dry Densities (MDD) for the soil increases from 1.5116 g/cm^3 at the Reduced Standard proctor energy level to its highest value of 1.999 g/cm^3 at the Modified Proctor energy level
3. The Optimum Moisture Content (OMC) behaves differently in the reverse order of the MDD as it decreases from 29.14 % at the Reduced energy level to its lowest value of 12.09 % at the Modified Proctor energy level
4. The soils recompression index (C_r) varies from 0.0529 at the Standard Proctor compaction energy level to 0.0229 at the Reduced Standard Proctor energy level while its value is maintained at 0.0059 for both the West Africa Standard and Modified Proctor energy levels.
5. The compression index (Cc) values vary slightly with the applied compaction energy levels. The Modified Proctor energy level recorded the highest Cc of 0.2989 which reduces to 0.2823 at the Standard Proctor compaction energy level. The West Africa Standard energy level gave a value of 0.227. The lowest value of 0.1531 was obtained for the Reduced Standard Proctor compaction energy level.

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INVESTIGATING THE USE OF CORN COB ASH AS PARTIAL REPLACEMENT FOR CEMENT IN CONCRETE PRODUCTION

***Abdulai, F. A, Aguwa, J.I., & Tsado, T. Y**
Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: alabayo@yahoo.com

ABSTRACT
This paper investigates the pozzolanic properties of corn cob ash for partial replacement of cement in concrete production. Tests were carried out to determine the chemical composition of CCA in comparison with Ordinary Portland Cement (OPC) to check its pozzolanic properties. Concrete cubes of size 150mm x 150mm x 150mm were produced with partial replacement of cement with CCA at varied levels of 0, 6, 12, 18, 24, 30, 36 and 42% respectively and tested at 7, 28, 56, and 90 days. The results showed that concrete with 0 % CCA recorded the highest compressive strength at all the curing ages. At 7 days, a decrease in strength from 18.81 N/mm^2 for 0% to 7.96 N/mm^2 for 42 % CCA replacement was observed. Similar trend was observed at 7, 28, 56, and 90 days. In conclusion, it was also observed that the compressive strength of the concrete cubes always increases with curing ages and decreases as the amount of the percentage combination of CCA increased. A curing of up to 120 days should be studied by which pozzolanic activity of ashes would have been concluded.

Keywords: *Cement, Compressive Strength, Concrete, Corn-Cob Ash, Pozzolanas*

1 INTRODUCTION

Concrete is one of the oldest and the most widely used construction material in the world and this has made its constituent materials to be of high demand. Hence, the overall cost of concrete production depends largely on the availability and cost of its constituents. The three basic constituents of concrete are cement, coarse aggregates (or gravel) and fine aggregates (or sharp sand). Apart from cement which can only be produced by industrial processes, other aggregates like fine and coarse aggregates can be sourced locally (naturally) or by manufacturing them. For natural sources, gravel comes from pits, river deposits and rock quarries, granites (crushed stones) are produced using explosives to break igneous rock formation into boulders which are in-turn crushed into required size distribution, shape and texture ranging from stone dusts to hardcore. Fine aggregates can be sourced from the rivers or sand pits. Manufactured aggregates on the other hand are lightweight aggregates which can be sourced from slag waste from iron and steel mills, expanded shales and slate, foamed slag, artificial cinders and Styrofoam beads while natural lightweight aggregates include pumice, scoria, volcanic cinders, saw dust, rice husk, sugarcane bagasse, guinea corn and maize cob (corn cob). All these can be used to produce lightweight concrete which is now widely utilized by modern technology because of its lower density, lower thermal conductivity and high insulation properties.

Corn cob is an agricultural waste product obtained from maize or corn. It is the hard-cylindrical center stalk of maize which bears the grains or kernels that usually finds inadequate final disposal of either being burnt or buried. Dwivedia *et al*., (2006) and Mehta (1997) identified the temperature range of 500ºC to 700ºC as optimum reactive ash formation before being used in the concrete making as they noted that these agricultural wastes become reactive at these temperature range. It comprises three natural parts: the chaff, the pith forming the light part and the woody ring which forms the hard part of the cob. Ash is the residue of burned plant parts like; bark, wood, sawdust, leaves, woody debris, pulp, husk, hulls, fronds, and other plant debris. Hence Corn Cob Ash is obtained from the residue of combusted Corn cobs. Many works have been carried out with successes recorded on the use of these materials as partial replacement of the concrete. Adesanya and Raheem (2009) showed that compressive strength of concrete using CCA up to 10 % replacement of OPC increased significantly after preparing the ingredients from open-air burning to produce the ash (Okamura and Ouchi, 2003). Okamura and Ouchi (2003) thus concluded in their study that Concrete strength increases with curing age and decreases with increasing percentage of corn cob ash. They also concluded that CCA-made concrete does not attain their design strengths at 28 days. This was also buttressed by Olafusi and Olutoge (2012) who concluded that the strength of CCA–Cement concrete increases with curing age and decreases with increasing percentage of corn cob ash. They, however, concluded that corn cob ash concretes do not attain their design strengths at 28 days and should be allowed to cure beyond this curing age by which the pozzolanic activity of ash would have been concluded which is consistent with Okamura and Ouchi (2003). Olafusi and Olutoge (2012) further suggested that succeeding studies should be done on with partial replacement ranging from 0 to 40 % of cement with corn cob ash following the steps of 5 %.

The current study seeks to further investigate the impact of partial replacement of corn cob ash (CCA) for cement in concrete production. The main objectives of the research are (1) to determine the chemical properties of CCA and engineering properties of fine and coarse aggregates and (2) to determine the compressive strength of the cubes made of concrete made with partial replacement of cement with CCA at 7, 28, 56 and 90 days curing ages with steps of 6 % ranging from 0% to 42%.

2 METHODOLOGY

The corn cobs used for this study were obtained from Minna, in Talba farm about 5 km from the permanent site of Federal University of Technology, Minna, Niger State of Nigeria. They were obtained in dry form and sundried for 1 week and air-dried for a few days. The collected samples were then burnt separately into ash by open burning in a metal container at the control temperature of 700 ^oC. The burnt corn cob was then grounded separately after cooling to smaller sizes of about 4mm diameter using mortar and pestle and the burnt ashes sieved separately through BS sieve of 75µm (Tumba *et al*, 2018) while the residue was thrown away (Adesanya and Raheem, 2009). The cement used, ordinary Portland cement (Dangote Cement) was obtained at cement seller in the State capital, Minna, Niger State of Nigeria. River sand and granite with maximum size of 19 mm were used as both fine aggregates and coarse aggregates respectively. The both fine and coarse aggregates used were obtained from local suppliers within Minna metropolis.

2.1 SAMPLES COLLECTION AND EXPERIMENTAL METHOD

The corn cobs used for this study were obtained from Minna, in Talba farm about 5 km from the permanent site of Federal University of Technology, Minna, Niger State of Nigeria. They were obtained in dry form and sundried for 1 week and air-dried for a few days. The collected samples were then burnt separately into ash by open burning in a metal container at the controlled temperature of 700 ^oC with a thermometer attached to the burner. The burnt corn cob was then grounded separately after cooling to smaller sizes of about 4mm diameter using mortar and pestle and the burnt ashes sieved separately through BS sieve of 75µm (Tumba *et al*, 2018) while the residue was thrown away (Adesanya and Raheem, 2009). The cement used, ordinary Portland cement (Dangote Cement) was obtained at cement seller in the State capital, Minna, Niger State of Nigeria. River sand and granite with maximum size of 19 mm were used as both fine aggregates and coarse aggregates respectively. Both fine and coarse aggregates used were obtained from local suppliers within Minna metropolis.

2.2 LABORATORY AND CHEMICAL ANALYSIS

After the burning of the corn cob ash and eventual production of CCA, the Chemical analysis was carried out on CCA at the central laboratory of Ahmadu Bello University Zaria in accordance with ASTM Standard (ASTM C311 - 77). This was done by using XRF (X-Ray Fluorescence) Spectrometer, which is a non-destructive analytical method of determining the chemical/elemental composition of materials. The outcome of the XRF result shown in Table 1 exhibited the percentage concentration of Silicon Oxide (SiO_2), Aluminum Oxide (Al_2O_3) and Iron Oxide (Fe_2O_3) in the CCA sample. This acted as a guide in classifying the sample in accordance with ASTM C619 (1992).

The CCA obtained were used to replace ordinary Portland cement at 0, 6, 12, 18, 24, 30, 36 and 42% by weight of cement. Control cubes were cast using concrete cubes with 0% of CCA (pure cement with no CCA replacement). The mix ratio used was 1: 2: 4 (cement – binder, sand and granite) which was adopted after design mix was performed on the constituent materials, with water to binder maintained at 0.5. Table 2 showed the batching information for each percentage combination of CCA and used to replace cement for the concrete cubes cast. Concrete cubes were cast using 150mm x 150mm 150 mm cube steel moulds. The cube steel moulds were assembled prior to mixing and properly lubricated with engine oil for easy removal of hardened concrete cubes. Each mould was then filled with prepared fresh concrete in three layers and each layers was tamped with tamping rod using thirty – five (35) strokes uniformly distributed across the seldom of the concrete in the mould. The top of each mould was smoothened and levelled with hand trowel and then the outside surfaces cleaned. The moulds and their contents were left in the open air for 24 hours. The concrete cubes were demoulded after 24 hours of the concrete setting under air and later kept in storage curing tank measuring 2.0m x 6.0m filled with tap water only for periods of 7, 28, 56 and 90 days.

TABLE 1: CHEMICAL COMPOSITION OF CORN COB ASH

Constituents	% Composition
Aluminum Oxide (Al2O3)	8.245
Silicon dioxide (SiO2)	54.006
Lead Oxide (P2O5)	5.967
Calcium Oxide (CaO)	12.412
Manganese Oxide (Mn2O3)	0.141
Iron Oxide (Fe2O3)	3.617
Titanium Oxide (TiO2)	1.252
Potassium Oxide (K2O)	6.471
Chloride (Cl)	0.889
Chromium Oxide (Cr2O3)	0.005
Sulphate (S03)	1.415
Sodium Oxide (Na2O3)	0.938
Mangnesium Oxide (MgO)	4.497
Zinc Oxide (ZnO)	0.112

TABLE 2: BATCHING INFORMATION FOR CCA CONCRETE

% Combination of CCA Replacement	Cement (kg)	Corn cob Ash (kg)	Sand (kg)	Granite (kg)	W/C Ratio
0	20.83	0	41.1	76.59	0.5
6	18.51	1.11	41.1	76.59	0.5
12	18.51	2.22	41.1	76.59	0.5
18	15.17	3.33	41.1	76.59	0.5
24	14.06	4.44	41.1	76.59	0.5
30	12.95	5.55	41.1	76.59	0.5
36	11.85	6.66	41.1	76.59	0.5
2	10.76	7.77	41.1	76.59	0.5

3 RESULTS AND DISCUSSION

3.1 CHEMICAL COMPOSITION

Table 3 presents the compressive strength of the concrete cubes for different percentages combination of CCA with other concrete components ranging from 0% to 42 % with different curing ages (7 to 90 days). This was also presented graphically as shown in Figure 1. The table and figure show that compressive strength generally increases with curing ages and decreases in compressive strength observed as the percentage of CCA increase. This is attributed to hydration of cement and the fact that ash possesses little cementing properties when compared to a Portland cement. The highest rate of early strength development of concrete cubes was observed with 0% combination of CCA with other concrete components which served as the control. In other words, concrete with 0 % CCA recorded the highest compressive strength (Table 4 and Figure 1). At 7 days, a decrease in strength from 18.81 N/mm^2 for 0% to 7.96 N/mm^2 for 42 % CCA replacement. Similar trend was observed at 28, 56, and 90 days as shown in Figure 1. These results show that concrete containing CCA that are pozzolanic materials gain strength gradually at early curing age which is consistent with Hossain (2005), Adesanya and Raheem (2009) and Raheem *et. al*, (2012). The continuous increase in compressive strength for all the percentages of ashes was observed at 56 days with values ranging from 22.23 N/mm^2 for the 0% to 9.9 N/mm^2 for 42 % CCA replacement of concrete. This is also in consistent with BS 8110 (1985) which specifies that a grade 20 concrete of 1 : 2 : 4 mix design without any partial cement replacement requires a strength of 13.5 N/mm^2 within the first 7 days of wet curing and 20 N/mm^2 within 28 days.

TABLE 3: COMPRESSIVE STRENGTH (N/MM^2) OF CONCRETE CUBES FOR DIFFERENT PERCENTAGES OF COMBINATION OF CCA WITH CURING AGES

% Combination of CCA Replacement	Curing Age (Days) 7	28	56	90
0%	18.81	21.27	22.23	23.21
6%	12.44	12.55	14.3	15.2
12%	11.67	12.4	13.34	14.1
18%	11.3	12.07	12.31	12.9
24%	10.03	11.3	12.1	12.5
30%	8.9	10.3	11.5	12.2
36%	8.45	9.6	11.2	11.4
42%	7.96	8.99	9.9	10.0

The results at 56 days indicated that pozzolanic action had commenced as evident from the higher percentage increase in compressive strength by CCA concrete over that of the control. The percentage increase with respect to the 28 days strength for control was 4.32 % while it was 12.23%, 7.05%, 1.95 %, 6.62%, 10.43%, 14.3% and 9.2 % for 6%, 12%, 18%, 24%, 30%, 36% and 42% CCA replacements. The increase in compressive strength can be attributed to the reaction of CCA and with calcium hydroxide [Ca(OH)2] liberated during the hydration of cement. The strength gain can also be attributed to the cementitious products formed as a result of hydration of cement and those formed when lime reacts with the pozzolan (Balendran and Martin – Buades, 2000). As could be seen from the Figure 1, there is a general decrease in compressive strength as the percentage of CCA content increases. Since all the concrete cubes meet the minimum strength of 6 N/mm^2 after 28 days of curing recommended by BS 5224 (1976) for masonry cement, CCA concrete could be used for general concrete works where strength is of less importance such as in floor screed, mortar and mass concrete.

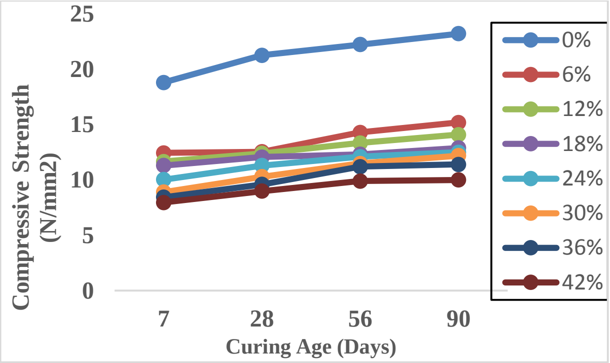

FIGURE 1: EFFECT OF CURING AGE ON THE COMPRESSIVE STRENGTH OF DIFFERENT PERCENTAGES OF CCA CONCRETE

Figure 2 illustrates the inter-relationship between each percentage of cement replacement with CCA against the compressive strength for different curing days.

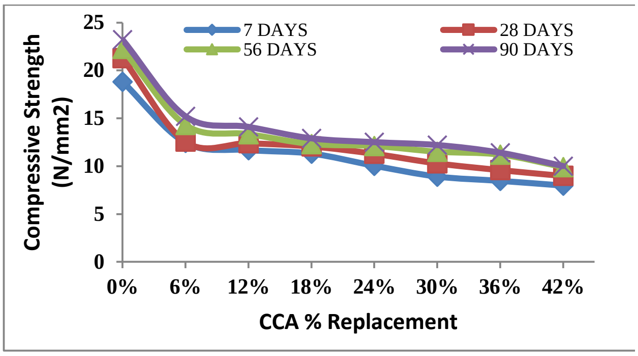

FIGURE 2: COMPRESSIVE STRENGTH OF CCA AGAINST PERCENTAGE REPLACEMENT

The results indicate that maximum compressive strength was attained by the concrete at 0% CCA replacement with concrete. This in other words means that compressive strength of the concrete reduces with increase in percentage of CCA in the concrete. The sharp decrease in concrete compressive strength was observed between 0% and 6% CCA replacement as observed in Figure 2 before it became steady from 6% to 42%.

4 CONCLUSION

The use of corn cob ash as a partial replacement for cement in concrete production has been investigated. From the results obtained, it was observed that CCA as partial replacement of cement in concrete production are suitable materials for use as a pozzolan, since it possesses pozzolanic properties having a combined (SiO_2 + Al_2O_3 + Fe_2O_3) of more than 50% for Class C of pozzolanas indicating that the CCA is a good pozzolanic materials in accordance with the requirements in ASTM C 618 (1991). It was also observed that the compressive strength of the concrete cubes always increases with curing ages and decreases as the percentage of partial replacement increases. This, therefore, means that, though the strength of CCA concrete was lower than that of the control, it can be used for concrete works where strength is of no significance such as mortar and mass concrete and in lightweight concrete. A step of 3% percentage replacement level would be recommended and a curing of up to 120 days should be studied by which pozzolanic activity of ashes would have been concluded.

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ASSESSMENT OF THE PROPERTIES OF HIGH STRENGTH CONCRETE MADE USING QUARRY DUST AS FINE AGGREGATE

Mohammed T.A.[1]; *Yusuf A.[2] & Abdullahi A.[2]
[1] Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author email: yusuf.abdul@futminna.edu.ng

ABSTRACT
The study was carried out to assess the properties of High-strength concrete made using Quarry dust as Fine aggregate. Preliminary tests were carried out on the aggregates to determine their suitability for use in High- strength concrete. The aggregates were well graded and other properties met the requirements of aggregates to be used for High-strength concrete. Superplasticizer and Rice husk were used as chemical and mineral additives to improve the workability and other properties of the concrete. Based on the properties of the aggregates, a mix ratio of 0.23:0.51:0.044:1.05:0.7:0.2, representing cement: fine aggregate: coarse aggregate: superplasticizer: water: rice husk was used. Concrete cubes of 100x100x100 mm were cast, cured and the 28-day compressive strength determined. An average 28-day compressive strength of 63 N/mm^2 was obtained. This can be categorized as class I high strength concrete and is applicable in the construction of high rise buildings. The Study therefore concludes that quarry dust can be used as total replacement for fine river sand in high strength concrete production.

Keywords: *Rice Husk Ash, High-strength Concrete, Natural River Sand, Quarry Dust.*

1 INTRODUCTION

Concrete is a construction material made up of cement, fine aggregate, coarse aggregate, water and mineral and/or chemical admixtures. Concrete is a widely used material in the Nigeria and the world at large for various building and civil engineering works. Based on global usage, it is placed at second position after water (Azhagarsamy**,** 2017). Based on compressive strength, concrete can be categorized as normal strength and high strength. High strength concrete are characterized with a compressive strength in excess of 62.5 N/mm^2 (Caldarone, 2009). Because of the superior mechanical properties and the significant economic savings offered by high strength concrete, the use of it has accelerated far beyond the current status of research on the material. Up to now, the major application of high strength concrete has been for cast-inplace columns in tall buildings. However, the use of high strength concrete in the precast, prestressed concrete industry can offer several benefits. For instance, the use of high strength concrete, in general, can shorten turnover times of casting beds and speed up construction time. Also, the increased concrete strength would allow for the use of more slender members, thus reducing the dead load of sections. It would also allow for an increase in the length of members, limited only by stability and transportation concerns (Fahim and Esko, 2015).

Fine aggregate is an essential component of concrete. The most commonly used fine aggregate is Natural River or pit sand (Azhagarsamy**,** 2017). The global consumption of natural sand is very high due to the extensive use of concrete as a result of rapid infrastructural growth. In order to lessen the use of the natural river sand, the construction industry of developing countries have been saddled with the responsibility of identifying alternative materials to reduce or eliminate the demand for natural sand. This research is therefore aimed at assessing the suitability of quarry waste as fine aggregate in high-strength concrete production.

Quarry dust, stone dust or crusher dust as it is variably called, is a by-product of the cutting and crushing process of stones. Quarry dust are produced as a result of mostly controlled blasting operation. The primary object of quarrying is to obtain coarse aggregate of various sizes to be used for various construction purposes. Quarry dust has been identified over the years as a substitute material for fine aggregate in concrete. Quarry dust can be used in building works, which would decrease the cost of construction and the construction material would be saved and the natural resources could be used properly. Quarry dust have been used for different purposes in the construction industry, such as building materials, road construction materials, fine aggregates, bricks and tiles.

The world around us is rapidly evolving and so is the world of infrastructure (Agrawal, *et al*., 2017). The use of the resources required to achieve the construction of these infrastructures are also in rapid use. Fine aggregate, one of the major component of concrete (which is the most common material used for infrastructural development) is being mined at an exponential rate. The sand mining from pits and river bed is a direct cause of erosion. The physical impact of sand mining include;

1. Downstream erosion due to increased carrying capacity of stream, downstream changes in patterns of deposition and the changes in channel bed and habitat type
2. Upstream erosion as a result of an increase in channel bed slope and changes in the flow velocity
3. The loss of adjacent land and/or structures
4. The undercutting and collapse of river banks (Saviour and Stalin, 2012).

Sand mining is regulated by law in many places, but it is still often done illegally (Kadi, *et al*., 2012). Quarry dust, a by-product of stone blasting in quarries constitute air pollution and disposal problem. However, the better geotechnical properties that quarry dust possess makes it strong candidate for the replacement of fine aggregate in concrete. This study was carried out to determine the suitability of quarry dust as fine aggregate in High Strength Concrete (HSC).

Due to the need to replace river sand in convectional and high-strength concrete so many research work have been carried out to this effect. Anzar (2015) attempted to improve the properties of concrete using quarry dust as replacement for natural sand, he found out that quarry dust improved the mechanical properties of concrete. He concluded that the optimum compressive strength is achieved at the proportion of coarse to fine of 40:60. Radhikesh *et al*. (2010) examined the suitability of stone crusher dust as a fine aggregate in concrete paving blocks. Some of the physical and mechanical properties of paving blocks with fine aggregate replaced by various percentages of stone dust were investigated. The experiment results showed that the replacement of fine aggregate by stone dust up to 50% by weight had negligible effect on the reduction of any physical and mechanical properties of the concrete. He also found out that there was 56% saving in money. Arivumangai and Felixala (2014) examined the strength and durability properties of granite powder concrete. The main parameter investigated was M39 grade concrete with replacement of sand by granite powder by 0, 25 and 50%, the cement was partially replaced by superplasticizer, slag, fly ash and silica fume. The test result indicated that use of granite powder and admixtures in concrete improved the performance of concrete durability and compressive strength. Anitha *et al*. (2013) investigated the use of quarry dust as replacement of river sand alongside chemical admixture in concrete. They found that as the replacement percentage increased, there was appreciable increase in flexural strength of the concrete. They also found out that a superplasticizer dosage of 1% increased the flexural strength compared to conventional concrete. They concluded that quarry dust can be used as an alternative material to natural river sand with a considerable increase in flexural strength reduction in disposal problem and economy in construction. Subramanian and Kannan (2013) experimented on the usage of quarry dust as partial replacement for sand in concrete and mortar. They reported that it is possible to replace river sand in conventional concrete with quarry dust. They found out that washed quarry dust gave better results due to more silicon (iv) oxide (SiO_2) and iron (iii) oxide (Fe_2O_3) and less amount of fines of size up to 150 microns. They recommended that trial casting with quarry sand proposed to be used in order to arrive at the water content and mix proportion to suit the required workability levels and strength requirement should be done. They also expressed the importance of removing excess fines of size up to 150 microns by washing. According to Nur *et al*. (2018) fifty percent of quarry dust gave optimum workability in concrete mix, beyond this percentage, the workability of the concrete was greatly affected. They concluded that realization of quarry dust from quarrying industry is a sustainable approach in order to comply with future need of the environment and concrete technology. From the test conducted by Sivakumar and Prakash (2011), it was inferred that quarry dust may be used as an effective replacement material for natural sand. The increase in cement content in the mortar phase showed an increase in strength. They added that fine quarry dust tends to increase the amount of plasticizer needed for the quarry mixes in order to achieve the rheological properties. When the river sand was replaced 100% with quarry dust they found out that the compressive strength was higher by 11.8% than the controlled cement mortar cube. The elastic modulus of the “quarry dust” concrete also increased. They concluded that “though there is an appreciable increase in strength gain of concrete when river sand (fine aggregate) is replaced with quarry dust at 100%, but the fines present in quarry dust increased water demand". With the addition of 15% of fly ash and 15% of quarry dust, the compressive strength increased to 22% than control mix M40 grade at day 28 (Arfat *et al*., 2016). Arfat *et al*. (2016) found out that using fly ash and quarry dust, more durable and sustainable concrete can be produced by evaluating optimum content of both. Ukpata and Ephraim (2012) identified the flexural and tensile strength properties compared with those for normal concrete. They found out that concrete proportion of lateritic sand and quarry dust could be used for construction provided the mixture of lateritic sand content is reserved below 50%. Both flexural strength and tensile strength were increased with increase in lateritic content. Ganesan (2009) reported that volume fraction of steel fibre to be used are 0.5, 1.5, 2.0%. For M60 compaction factor ranges from 0.88 to 0.92. Raman *et al.* (2007) indicated that quarry waste did not significantly affect the non-destructive properties of the concretes except initial surface absorption. Dynamic modulus of elasticity, ultrasonic pulse velocity, and initial surface absorption varied linearly with compressive strength. Moreover, dynamic modulus of elasticity and ultrasonic pulse velocity were well-correlated. Ilangovan and Nagamani (2006) reported that natural sand with quarry dust as full replacement in concrete as possible with proper treatment of quarry dust before utilization. Quarry waste fine aggregate was used in presence of silica fume. The overall test results revealed that quarry waste fine aggregate can be utilized in concrete mixtures as a good substitute of natural sand. It is found that the compressive, flexural strength and durability studies of concrete made of quarry rock dust are nearly 10% more than the conventional concrete. Sahu, *et al.* (2003) reported significant increase in compressive strength, modulus of rupture and split tensile strength when 40 percent of sand is replaced by quarry rock dust in concrete. Nagaraj (2000) studied that the consumption of cement content, workability, compressive strength and cost of concrete made with quarry rock dust. The mix design proposed showed the possibilities of ensuring the workability by wise combination of rock dust and sand, use of super plasticizer and optimum water content. Hudson (1997) reported that the strength of quarry rock dust concrete is comparatively 10-12 percent more than that of similar mix of conventional concrete. Also the result of this investigation showed that drying shrinkage strains of quarry rock dust concrete were quite large to the shrinkage strain of conventional concrete. However, at the later age, showed equal strain than conventional concrete. Durability of quarry rock dust concrete under sulphate and acid action was higher inferior to the Conventional Concrete Permeability Test results clearly demonstrated that permeability of quarry dust concrete was less compared to conventional concrete. Nagaraj *et al.* (1996) produced concrete using the rock dust as an alternative to natural sand. They studied the effect of rock dust on the strength and workability of concrete.

2 METHODOLOGY

High strength concrete (HSC) was prepared by selecting suitable materials, good quality control and proportioning. The materials conformed to British Standard requirements. The materials used in this research include;

1. Cement: The Ordinary Portland cement used in this study conform to BS EN 197-1 (2000). The specific gravity of cement was 3.15. The initial and final setting times were found as 30 minutes and 120 minutes respectively. Standard consistency of cement was 31%.
2. Superplasticizer: The superplasticizer was sourced from Armosil Manufacturing Incorporation, a supplier of different kinds of concrete admixtures. Hydroplast 300, a light blue colourless liquid with specific gravity of 1.175 ± 0.005 at 20^0C was used. Hydroplast 300 is a high performance water reducing superplasticizer formulated to comply with ASTM C-94 type F and EN 943, part 2.
3. Quarry stone dust: The quarry stone dust used as the fine aggregate in this research was sourced from Abuja, Nigeria. The specific gravity, fineness modulus and compacted density are 2.63, 4.39 and 1435.75 kg/m^3 respectively. The quarry dust passing through sieve 5 mm and retrained on sieve 150 µm were used.
4. Coarse aggregate: A well graded crushed granite with a specific gravity of 2.64 and compacted density of 1582.87 kg/m^3 was used as coarse aggregate in this research.
5. Rice Husk Ash (RHA): The RHA used was sourced from Gidan Kwano village, burnt at a controlled temperature of 600^{o}C and sieved. The chemical composition of RHA used is given in Figure 1. The RHA possess major oxides in excess of 70% as recommended by standards.

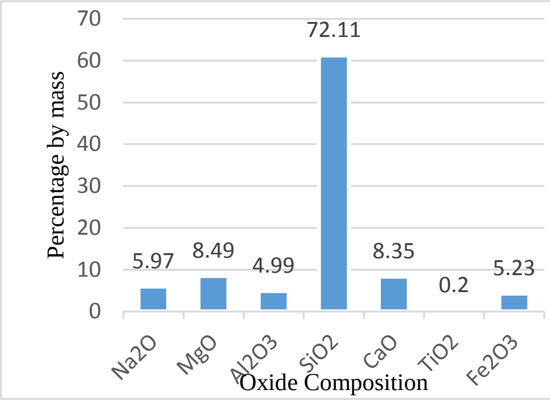

FIGURE 1: CHEMICAL COMPOSITION OF CHEMICAL ADMIXTURE

2.1 TEST ON AGGREGATE

The various tests conducted on the aggregates used in this research are sieve analysis, specific gravity test; and compacted density test. The tests were carried out in accordance to BS EN 12620 (2008).

2.2 CASTING AND CURING OF TEST SPECIMENS

The mix ratio adopted was 0.23:0.51:0.044:1.05:0.7:0.2 representing cement: fine aggregate: coarse aggregate: superplasticizer: water: rice husk. 100x100x100 mm moulds used were cleaned, assembled and oiled. The concrete was cast in moulds in three layers. Each layer was compacted using a tampering rod to remove entrapped air. The concrete surfaces were levelled by trowel, and the specimens were covered with nylon sheets to prevent evaporation of water for 24 hours. The specimens were demoulded after 24 hours and cured in a curing tank.

2.3 SLUMP TEST

This test was carried out to determine the workability of concrete mixture according to BS EN 12350:5 (2009) by using standard slump cone. The average slump obtained was 120mm.

3.0 RESULTS AND DISCUSSION

3.1 PARTICLE SIZE DISTRIBUTION

Table 1 shows the result of particle size distribution analysis conducted using quarry dust size sample. Total mass of dry sample used was 500g, but summing the masses of the retained sand we have 499.9g. The reduction is due to losses mainly from small quantities of sand that gets stuck in the meshes of the sieves.

TABLE 1: PARTICLE SIZE DISTRIBUTION OF FINE AGGREGATE (QUARRY STONE DUST)

Sieve sizes (mm)	**Weight of empty sieve (g)**	**Weight of sieve + Sample (g)**	**Weight of sample retained (g)**	**Percentage weight retained**	**Cumulative percentage retained**	**Percentage passing**
5.00	475.4	489.2	13.8	4.6	4.6	95.40
3.35	468.0	501.0	46.8	11.6	16.2	83.80
2.36	434.0	462.2	28.2	9.4	25.6	74.40
2.00	416.9	428.9	12.0	4.0	29.6	70.40
1.18	385.2	416.9	31.7	10.57	40.17	59.83
850	352.5	371.0	18.5	6.17	46.34	53.66
600	467.9	492.0	24.1	8.03	54.37	45.63
425	435.0	457.1	22.1	7.37	61.74	38.26
300	384.7	398.9	14.2	4.73	66.47	33.53
150	420.6	501.5	80.9	26.97	93.44	6.56
75	383.1	402.0	18.9	6.3	99.74	0.26
Pan	298.1	300.7	2.6	0.87	100.0	0
Total			**499.9**			

Cumulative percentage retained from 150mm sieve size and above:

93.44+ 66.47 + 61.74 + 54.37 + 46.34+ 40.17 + 29.60 + 25.60 + 16.20 +4.60 = 438.53

Finest Modulus = 4.39

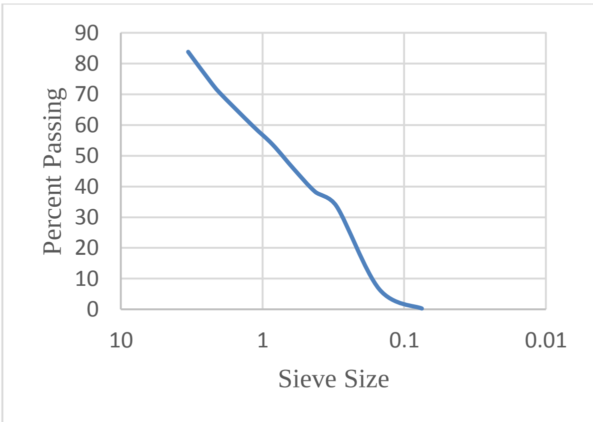

FIGURE 1: SIEVE ANALYSIS OF QUARRY DUST

3.2 SPECIFIC GRAVITY

Table 2 shows the result for the specific gravity of fine aggregate (Quarry stone dust). The specific gravity of fine aggregate obtained is 2.63, which is appropriate for use in High-strength concrete BS EN 12620 (2008).

TABLE 2: SPECIFIC GRAVITY OF FINE AGGREGATE

Trial	**1**	**2**	**3**
Weight of Cylinder: M_1 (g)	162.4	164.1	152.1
Weight of Cylinder + Dry Sample: M_2 (g)	334.5	338.8	331.0
Weight of Cylinder + Dry Sample + Water: M_3(g)	306.5	309.1	308.9
Weight of Cylinder + Water: M_4 (g)	116.6	116.6	116.6
M_2-M_1 (g)	45.8	47.5	35.5
M_4-M_1 (g)	189.9	192.5	192.3
M_3-M_2 (g)	172.1	174.7	178.9
G_S	2.57	2.67	2.65
Average G_S		**2.63**	

3.3 WATER ABSORPTION

Table 3 shows the result of water absorption test of Fine aggregate (Quarry stone dust). A mean value of 21.71% was obtained which is adequate for use in High-strength concrete.

TABLE 3: WATER ABSORPTION OF FINE AGGREGATE

Trial	**1**	**2**
Weight of empty can: M_1 (g)	76.3	79.8
Weight of can + Dry sample: M_2 (g)	315.5	284.8
Weight of can + Sample + Water: M_3 (g)	367.3	329.4
Moisture weight M_3-M_2 (g)	51.8	44.6
Weight of dry sample: M_2-M_1 (g)	239.2	205.0
%Water absorption	21.66	21.76
%Mean water absorption		21.71

3.4 BULK DENSITY

Table 4 shows the results of compacted bulk density of fine aggregate (Quarry stone dust). The test result shows that the bulk density of the dust is 1435.75 kg/m^3, which is well above the average value required for use in High-strength concrete.

Table 4: Bulk Density of Fine Aggregate

Trial	**1**	**2**	**3**
Weight of empty mould: M_1 (kg)	1.08	1.08	1.08
Weight of empty mould + sample: M_2 (kg)	3.68	3.62	3.65
Weight of loose sample: $M_3 = M_2$-M_1 (kg)	2.60	2.54	2.57
Volume of mould: V (m^3)	0.00179	0.00179	0.001179
Bulk Density = M_3/V (kg/m^3)	1452.51	1418.99	1435.75
Mean Bulk Density (kg/m^3)		**1435.75**	

3.5 COMPRESSIVE STRENGTH OF CONCRETE

The compressive strength of the hardened HSC as obtained in the laboratory are presented in table 4.9. The test was conducted in accordance with BS EN 12690:3 (2009). The cured concrete samples were crushed at day-3, day-7 day-21 and day-28. There was about 13% increase in strength from day 3 to day 7, about 50% increase in strength from day 7 to 21 and about 22% increase in strength from day 21 to 28. Overall, there was about 100% increase in strength from day 3 to day 28. The accelerated gain in strength is an attribute of HSC.

TABLE 5: COMPRESSIVE STRENGTH OF CONCRETE SPECIMENS

S/N	**Curing Age (Days)**	**Dry density (kg/m^3)**	**Weight density (kg/m^3)**	**Specific area (mm^2)**	**Crushing loads (N)**	**Compressive strength ($\frac{kN}{m2}$)**
1	3	2490	2520	10000	310000	31.00
2	7	2560	2590	10000	350000	35.00
4	21	2640	2660	10000	520000	52.00
5	28	2660	2670	10000	630000	63.00

5.0 CONCLUSION

The physical and mechanical properties of aggregates were determined and found to be adequate for high strength concrete production. The strength of the concrete obtained falls within class I high strength concrete and can be used to construct high rise buildings.

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ANALYSIS OF THE USE OF UNMANNED AERIAL VEHICLE (UAV) FOR PHOTOGRAMMETRIC SURVEY – NILE UNIVERSITY OF NIGERIA AS STUDY AREA

***Mogbo, O. N.**[1]**; Mambo, A. D.**[2]**; Wali, S.**[3]
[1-3] Department of Civil Engineering, Nile University of Nigeria, Abuja
*Corresponding author email: mogbo.nwabueze@nileuniversity.edu.ng

ABSTRACT
In the 21st century, the use of UAVs has greatly risen. For the purpose of surveying in construction work, the use of drones is still new and it is a fast and efficient way of carrying out survey work. While carrying out this project, the feasibility of UAVs for photogrammetry survey as an alternative to conventional survey methods is tested. Aerial photographs were captured to generate a 3D model of Nile university of Nigeria, along with a DSM and orthophotos for accurate measurements in performing volume and area calculations for the proposed Reservoir Area at Nile University of Nigeria. At the end of the project, a topographical map of Nile university was made and the UAV proved to be a better alternative for photogrammetry than the use of satellite images. A key component in the accuracy of aerial photography is the flying altitude from which the photos are taken. The study shows that aerial photographs taken at a lower flying altitude using a UAV provides a more accurate result in representing the Ground Sampling Distance.

Keywords: *Photogrammetry, Unmanned Aerial Vehicle, Surveying, 3D Modelling, Topography*

1. INTRODUCTION

The goal of UAV photogrammetry is generally to achieve high resolution aerial imagery and the production of a topographic map and 3D model from the photographed area. The resolution in this case is referred to as the ground sample distance (GSD) which is essentially a measure of how big of a footprint or portion of land on the ground is represented by one pixel in the orthophoto. For example, aerial maps collected by a small drone may have a GSD of as precise as 1 cm (that is, one pixel in the image represents an area of the ground that is 1 cm x 1 cm square), whereas a manned aircraft photogrammetry survey may produce imagery with a GSD of around 15 cm or more or less depending on the type of UAV and the camera specifications and calibrations. Satellite imagery can range from around 30 cm on the high-end (very expensive) to 30 m which is more typical and seen like the google earth satellite. ((Delair, 2017a).

Another critical aspect of evaluating the quality of a dataset is in assessing the accuracy of it. In this case, accuracy can be either relative or absolute. In the geospatial sense, relative accuracy suggests that if you were to take the location of one feature in the image and measure its distance from another feature, that the distance and relative position between the two features would be correct. However, the actual geographic coordinates of both locations could be far off from actual. Which is why when georeferencing, the appropriate coordinate system is overlapped with the photographs.

Absolute accuracy in the geographic sense suggests that for any given location within the dataset, it can be relied upon that the calculated coordinates of that position are within a given measurement from the true coordinates, if one were to know that position with absolute certainty. With the use of UAVs accurate measurements are obtained with lower flying altitude providing a higher resolution of the photographs with accuracy of up to 1cm/pixel. (Delair, 2017a)

Aerial imagery collected from a drone may generally be highly precise but it is not always highly accurate. Experienced surveyors or geospatial professionals will employ the use of ground control points (GCPs) or will use drones equipped with dual-frequency global navigation satellite system (GNSS) receivers that are post processed kinematic (PPK)-enabled for highly accurate georeferencing of the aerial data. PPK-enabled drones may be able to achieve as good as 2-3 cm accuracy horizontally and around 5 cm vertical accuracy. (Delair, 2017a)

The scope of the project will determine what is truly needed and the equipment selected along with the operator's experience will determine the accuracy that can truly be achieved. Ultimately, the photogrammetry dataset will become input data for analysis which will ultimately yield the geospatial intelligence being sought after, and the output can be limited to just the scope of the project assigned. In the planning stage the accuracy can be determined after the scope has been established.

UNNAMED AERIAL VEHICLES (UAVS)

The UAV is an acronym for Unmanned Aerial Vehicle, which is an aircraft with no pilot on board to fly inside it. UAVs can be remote controlled aircraft (flown by a pilot at a ground control station) or can fly autonomously based on a pre-programmed flight plans or more complex dynamic automation systems (UAV, n.d.).

Some early UAVs are called drones because they are no more sophisticated than a simple radio-controlled aircraft being controlled by a human pilot (sometimes called the operator or controller) at all times. More sophisticated versions may have built-in control and guide systems to perform low level human pilot duties such as speed and flight path stabilization, and simple prescribed navigation functions such as waypoint following. However, most of the functionalities can be set before the flight mission. The flying altitude, flight speed, flight direction, and overlay can be chosen and the area to be surveyed can also be assigned with its boundaries.

From this perspective, most early UAVs are not autonomous in their function (Everaerts, 2008). In fact, the field of air vehicle autonomy is an emerging field, whose economics is largely driven by the military to develop battle ready technology for the war fighter. (UAV, n.d.)

In this project the UAV used is a DJI Phantom 3 Quad copter drone.

RECONNAISSANCE STUDY

A reconnaissance survey was carried out, in which the size and location of the reservoir was established and also easy access point. The reservoir area is not very easy to access and is surrounded by trees and shrubs within it. Manual survey could not be carried along the entire area because of its nature. The boundary points and span location were established on google earth as shown in (Figure 1).

FIGURE 1: SATELLITE IMAGE OVERVIEW OF SITE BOUNDARY POINTS AT NILE UNIVERSITY OF NIGERIA

Where the points A, B, C and D are the boundary points Table 1 while X and Y are the top and bottom edge points.

TABLE 1: BOUNDARY POINTS

Boundary Points

Points	Latitude N	Longitude E
A	9º0'58.11''	7º23'53.08''
B	9º1'8.07''	7º23'51.14''
C	9º1'8.35''	7º23'53.88''
D	9º0'58.73''	7º23'55.36''
X	9º1'7.36''	7º23'52.37''
Y	9º0'59.41''	7º23'54.18''

A pre-flight test was carried out to ensure proper connection between the UAV and receiver and tests its functionality. Also, a video was made recording the entire reservoir area and seeing the entire area on the system. The time of day was also taken into consideration, ensuring that they were no shadows for high resolution pictures. During the day the shadows forms at different lengths at different times affecting the output of pictures. In carrying out the snapshots evening time was used, where ones' shadow is directly under them to increase the resolution of pictures.

IMAGE ACQUISITION

Generally, spending some time planning your shot session might be very useful. For acquiring the images on the UAV, it was a two-man job involving a pilot operating the UAV charged with the take-off and landing, the battery status and a co-pilot monitoring the connection between the UAV and stationed computer system. The software DJI GO was used for viewing the drone camera and instructing the drone. It provides information about the total estimated flight time from the calibration and allows for calibration of the altitude of the drone, overlay, the flight speed and flight direction. A total of 197 images for the topographic map design and 66 images for the volume and area calculation were captured in the drone from the flight and used for the image processing and rendering. For the software number of blind-zones were minimized since PhotoScan is able to reconstruct only geometry visible from at least two cameras (PhotoScan, n.d.).

In case of aerial photography, the overlap requirement can be put in the following figures: 60% of side overlap + 80% of forward overlap (Soni, 2016). Each photo should effectively use the frame size: object of interest should take up the maximum area. In some cases, portrait camera orientation should be used (Hanke, Grussenmeyer, & Streilein, 2002). Good lighting is required to achieve better quality of the results, yet blinks should be avoided. It is recommended to remove sources of light from camera fields of view. In case of aerial photography and demand to fulfill georeferencing task, Agisoft PhotoScan is able to complete the reconstruction and georeferencing tasks without ground control points (GCP)s.

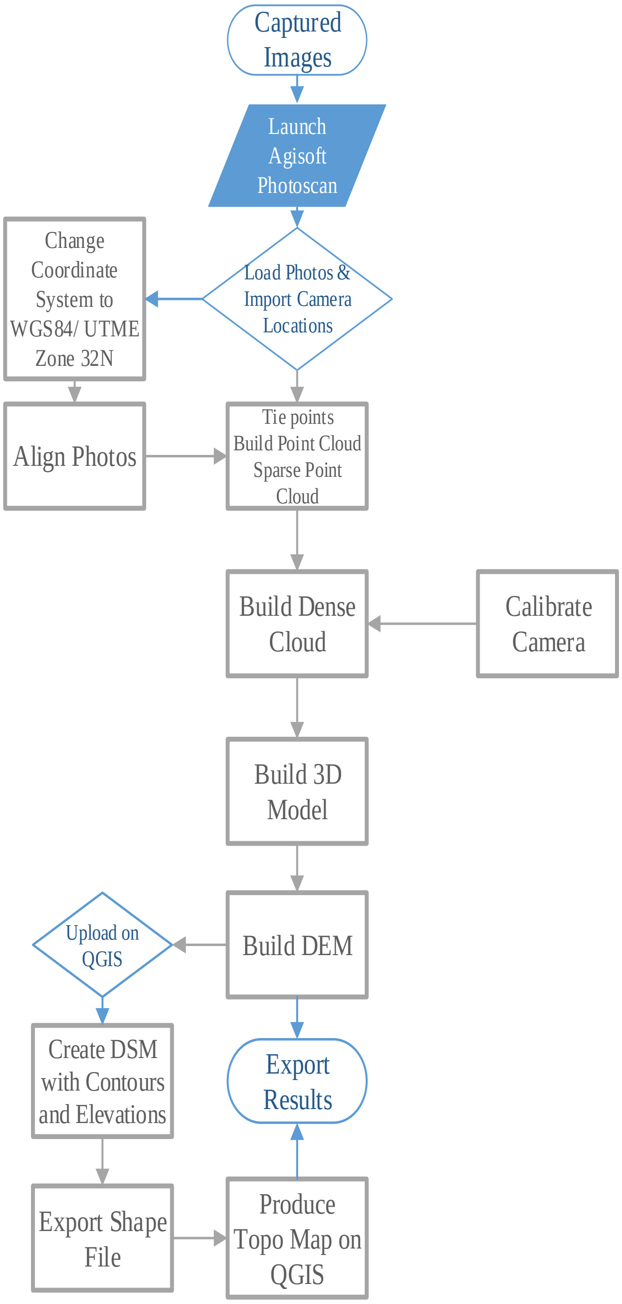

FIGURE 2: PROCESS WORK FLOW CHART OF PROJECT

For the study area the two main software used are Agisoft Photoscan for the image processing and 3D model generation and QGIS for generating a DEM and creating a topographical map. While for the overlap 70% side overlap and 60% front overlap was used. The process of the work flow (Figure 2) shows the steps involved for the image acquisition to the final output of the project.

3D MODEL

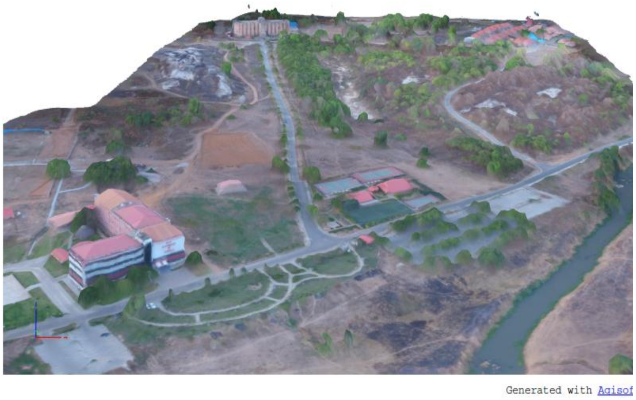

FIGURE 3: 3D MODEL OF NILE UNIVERSITY OF NIGERIA

FIGURE 4: 3D MODEL OF NILE UNIVERSITY OF NIGERIA RESERVOIR AREA

The 3D models for both the 197 photos and 66 photos were made on Photoscan and can be seen in Figure 3 and Figure 4.

DIGITAL SURFACE MODEL (DSM)

The digital surface model (DSM) generated from Agisoft Photoscan, and the shapefile was uploaded on QGIS providing a more comprehensive elevation data. It can be seen in (Figure 5), that the reservoir height falls within the range of 468.2-483.1m which is a difference of 14.9m from the highest point to the lowest point. The legend has a class interval of 8.

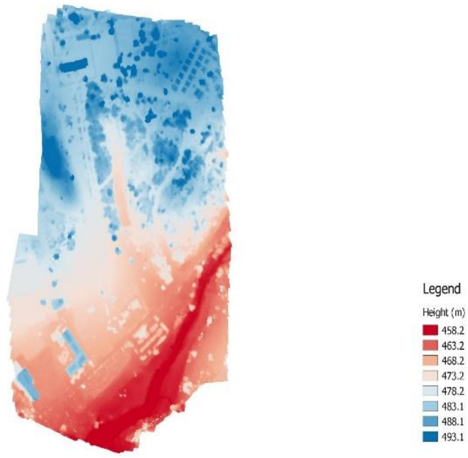

FIGURE 5: DSM SHOWING THE STUDY AREA

From the digital surface model, we can see that the stream is at a much lower level compared to the rest of the features around it.

CONTOURS

The contours generated from QGIS can be seen in figure, with a contour interval of 1m for maximum detailing. In (Figure 6), we can see that the heights range from 458.2-493.1m. The difference in height from the highest point to the lowest point is 34.9m within the photographed area. Areas with high elevation are coloured blue and areas with lower elevation are coloured red.

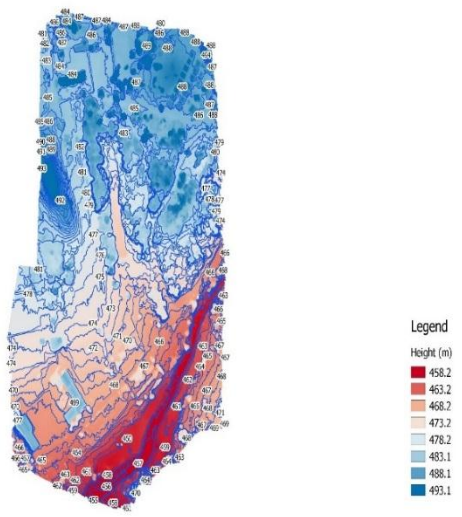

FIGURE 6: DSM WITH CONTOUR LINES OF THE STUDY AREA

For the topographical map, the contours generated from the digital surface model (DSM) (Figure 7) was used in the production of the topographical map.

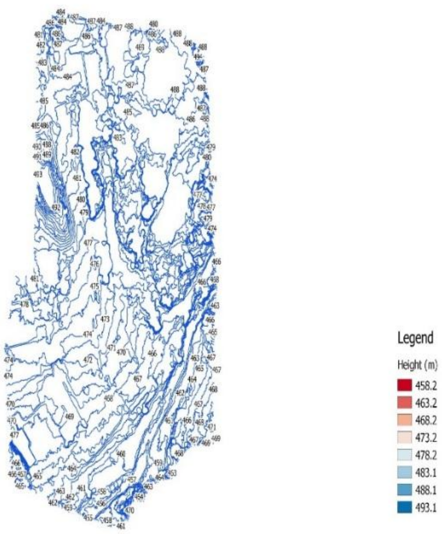

FIGURE 7: CONTOURS GENERATED FROM DSM OF THE STUDY AREA

VOLUME AND AREA CALCULATION

On the orthophoto created, the volume and area (Figure 8) calculation were made using line and polygon tool. The line tool provides data on the perimeter including the profile along the line drawn. Volume (Figure 9) and area were measured by creating a polygon around the boundary of the reservoir. In Figure 8, it can be seen that the measured perimeter is 520.107 m and the area 5,489.546 m^2. The volume is 41,309.049 m^3 from Figure 9.

Using the line tool, the cross-sectional area cut across the reservoir provides the profile at that particular point. With profile it can be seen that there is need for levelling and the right side needs to be excavated for a gentler slope. In Figure 10, the profile of the dissected area shows an elevation difference of approximately 3.4m.

The vertical profile along the centre of the reservoir shows that Figure 9 the highest point is at point X is 444.677 and lowest point Y is 430.215. The points were referenced from Table 1 and can be seen in Figure 3 boundary points.

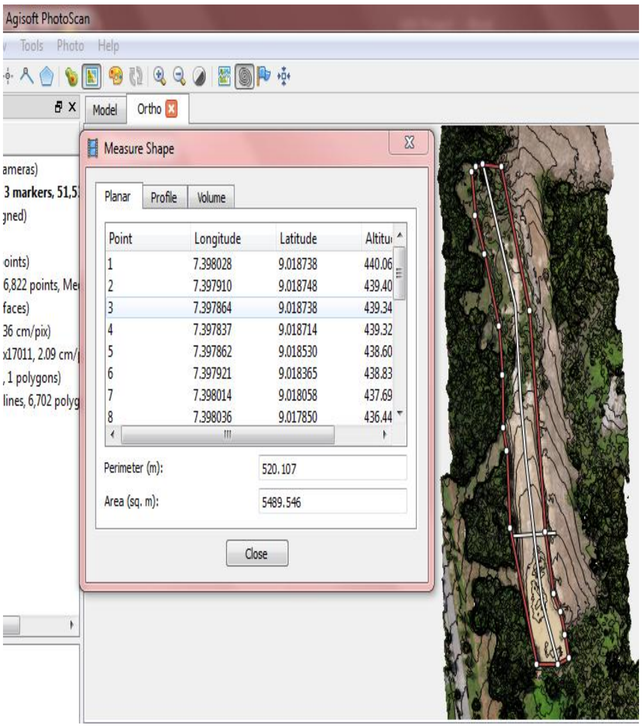

FIGURE 8: ESTIMATED PERIMETER AND AREA OF RESERVOIR AREA

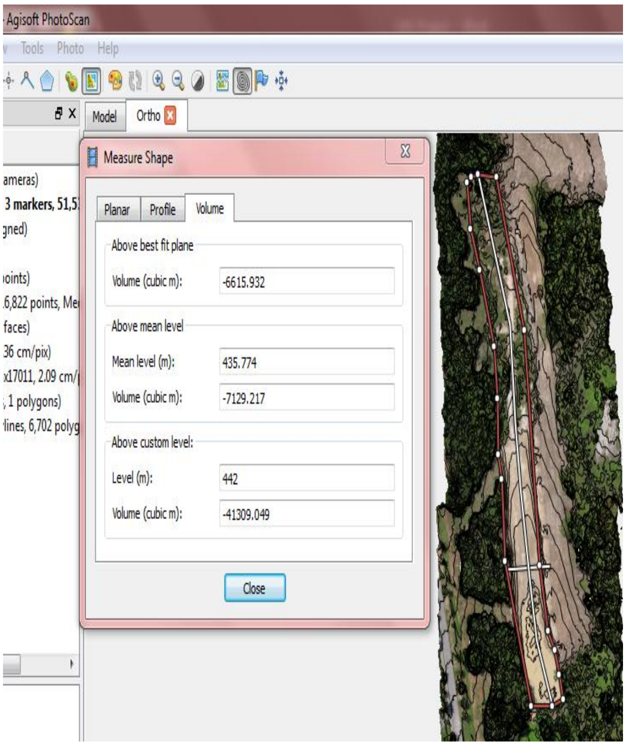

FIGURE 9: ESTIMATED VOLUME OF RESERVOIR AREA

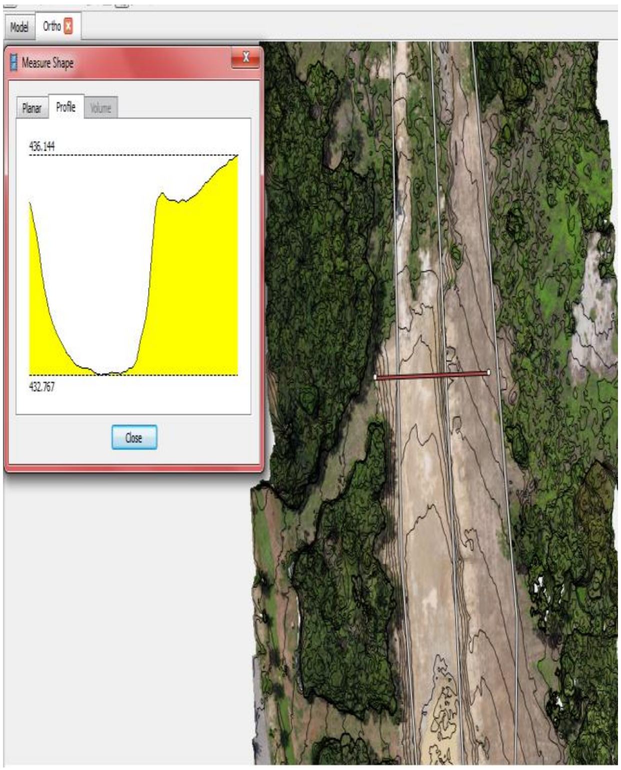

FIGURE 10: CROSS-SECTIONAL PROFILE OF DISSECTED AREA

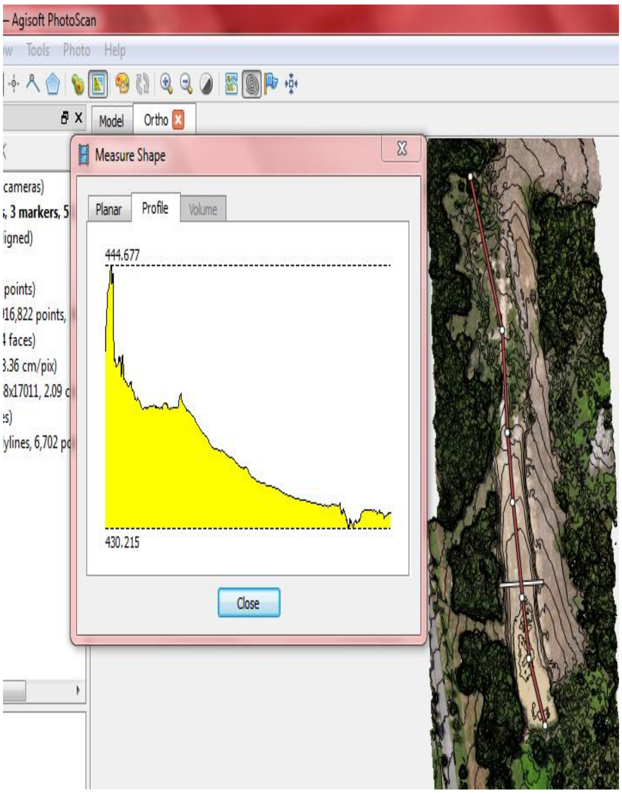

FIGURE 11: PROFILE ALONG THE CENTRE OF RESERVOIR AREA

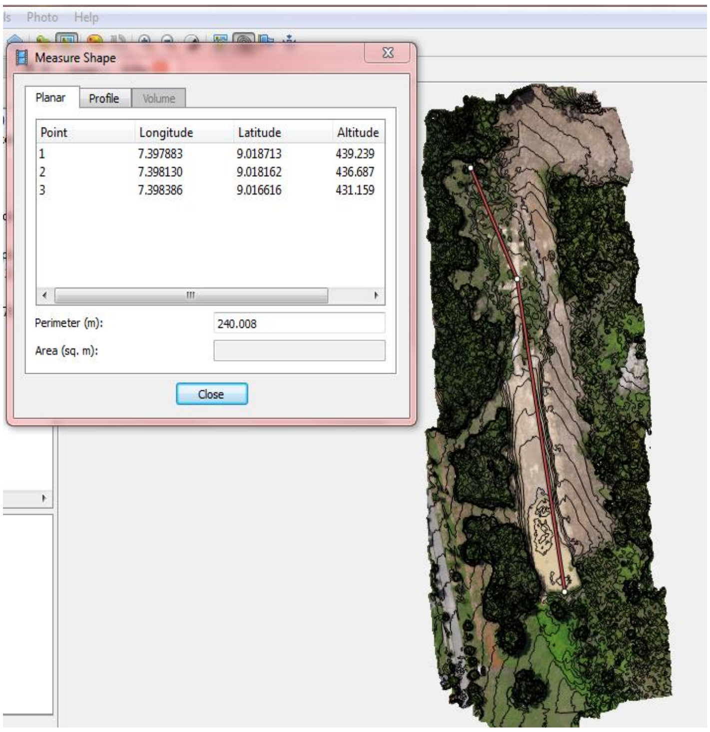

FIGURE 12: ESTIMATED PERIMETER ALONG THE CENTER OF THE RESERVOIR AREA

At the first point also known as point X the elevation there is 439.239 m and the last point (point Z) has an elevation of 431.159 m as shown in Figure 12.

TOPOGRAPHICAL MAP

In the production of the topographical map, the use of the satellite (Figure 13) and contours shapefile (Figure 7) were used to trace the exact points of the features within the photographed area and overlaying them to produce the topographical map (Figure 14), (Wali, S. 2018). The map scale used was 1:5,793 with a contour interval of 0.5m.

Note: Map Units in Meters.

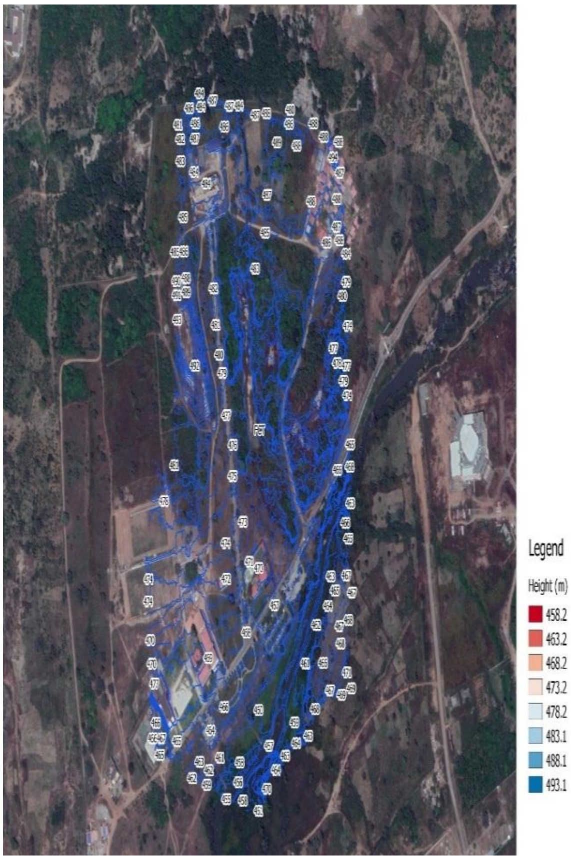

FIGURE 13: CONTOUR LINES OVERLAID ON THE PHOTOGRAPHED AREA

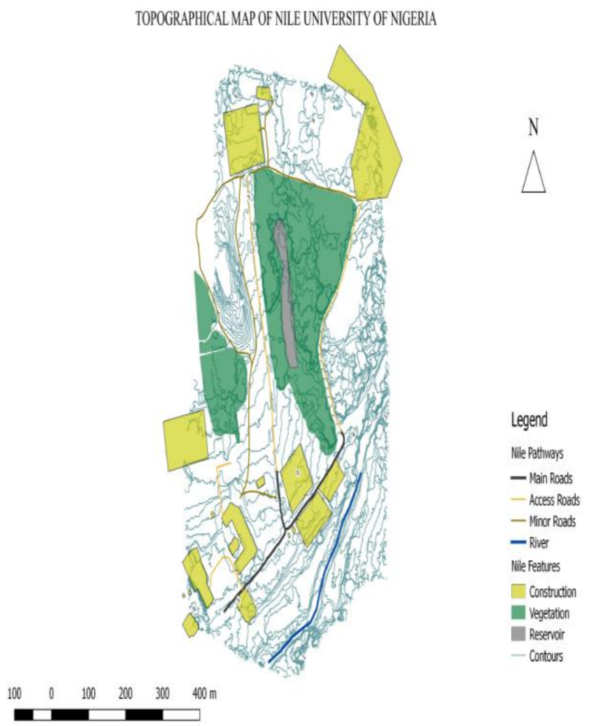

Figure 14: Topographical Map of Nile University of Nigeria Showing Features

UAV AND SATELLITE

In Figure 16 the points selected from google earth along the centre of the reservoir, for the generation of the digital elevation model (DEM) can be seen. Figure 17, shows the DEM with the points as they are on google earth while with an interval of 10m, Figure 18 shows the contours generated from the DEM itself. The contours have an interval of 0.5m as the difference in elevation is 9m which is small compared to the difference in elevation from the DSM with a difference in elevation of 14.5m., that is a gap of 5.5m and the difference and accuracy is from the images taken at closer range, resulting into a more in depth and precise data.

FIGURE 15: GOOGLE EARTH POINTS ALONG THE CENTRE OF RESERVOIR

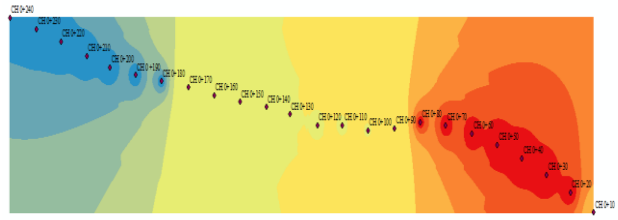

FIGURE 16: DEM 10M POINTS FROM ARCMAP

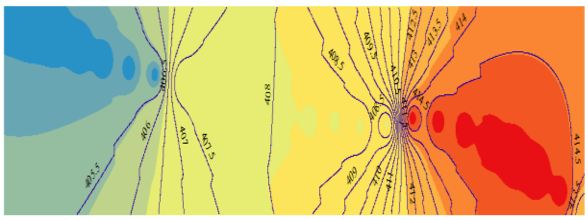

FIGURE 17: CONTOURS FROM DEM ON ARCMAP

DISCUSSION OF RESULT

At the end of the analysis it can be seen that, volume and area calculations can be obtained using photogrammetry survey. The use of aerial photographs not only allows for the generation of a 3D model but also, the generation of digital surface model and contours. Orthophotos allow accurate measurements to be performed. For creating topographical maps, digital surface models are a key element as they allow the generation of contours which are included in the map.

Volume and area calculation can be achieved from orthophotos. The estimated volume for the area of interest is 41,309.049 m^3 and 5,489.546 m^2 as seen in Figure 8 and Figure 9.

The shapefile from the contours made overlaid with satellite image, enabled the creation of the topographical map. The guide for accuracy shows that a photo scale of 1:5,000 yields a horizontal/vertical accuracy of 0.120 m. (Wali, S. 2018) Also, (Wali, S. 2018) suggests the recommended use for such scale and interval of 0.5 m to be used for planning/rural design, which can be used by the school for their own design. For this topographical the scale used was 1:5,793 and contour interval of 0.5 m.

Using satellite images provides fairly accurate data but, the accuracy of coordinates and altitudes have a huge difference with UAVs and this is simply because of the difference in altitude from the captured distance.

The percentage difference from the DSM and DEM can be seen below:

[1] $$\frac{14.5-9}{\frac{14.5+9}{2}} x\ 100\% = \frac{5.5}{11.75} x 100\% = 46.8085 \cong 47\%$$

From the survey carried out the difference in accuracy is up to 47%, and it can be concluded that UAVs are far more accurate the satellite.

In Summary, **Photogrammetry is the science of making measurements from photographs.** The input of photogrammetry is simply to capture photographs, and the output is represented in form of a map, a DEM, a drawing, or a 3D model of some real-world object or land. The goal of UAV photogrammetry is generally to achieve high resolution aerial imagery and the production of a topographic map and 3D model from the photographed area.

In Photoscan, the aerial photographs uploaded are stitched together to form a mosaic allowing pictures of the same face from different angles to form in order to map out every contour. The attainable accuracy of a photogrammetric product depends on two main factors. The first is the scale of the photographs from which the product is derived and the second is related to errors in the photogrammetric process.

From the research questions in this research;

(a) Is photogrammetry an alternative for other surveying methods?

The answer would be yes because, in conventional survey methods data on elevation, area, distance and volume is acquired. The use of photogrammetry provides a relatively accurate data, which can be used for making measurements and design purposes.

(b) How can volume and area calculation be achieved using photogrammetry?

The calculation of volume and area using photogrammetry by making orthophotos from the aerial photographs, enables one to make area and volume calculation from the orthophoto taken and making measurements on them using a software and establishing points on the orthophoto in which measurements are desired to be made. Generally, orthophotos allow one to make accurate measurements and photogrammetry provides this.

(c) What is the accuracy of a UAV compared to satellite data?

The accuracy of a UAV compared to satellite data varies with flying altitudes and captured elevation. Since satellite image provides a ground sampling distance (GSD)of 30m per pixel while, the UAV used here provides a ground sampling (GSD) of 2cm per pixel that alone tells one that the accuracy of the UAV data is far better and precise than the satellite image. In this project the difference in accuracies from that of UAV and satellite was 48%. A UAV will almost always have a better accuracy in terms of resolution than a satellite because of the flying altitudes never exceeds a height that will provide a 30m GSD. The accuracy however, will vary depending on flying altitude. A lower flying altitude will give a greater difference and a higher altitude will give a smaller difference for the accuracies.

CONCLUSION

The Survey carried out was successful, the outcomes were achieved. A 3D model of Nile University of Nigeria was generated along with a Topographical Map of the area, the DSM and contours were generated on Agisoft Photoscan including an orthophoto of the reservoir area which was used to perform volume and area calculations. The map created could be used for planning of rural design. For more accurate GSD the use of UAVs at a lower altitude is recommend, the use of satellite image for survey data provides a lower GSD.

Using a UAV provided to be a great and easy alternative for conventional survey method. The volume and area calculations were done on Agisoft Photoscan by, creating an orthophoto from the aerial photographs taken by the drone. A topographical map of Nile University of Nigeria was made using the contours generated on QGIS with an overlay of the satellite image of the area.

In terms of accuracy, the UAV proved to be a better alternative than google earth satellite. This is because the GSD of the satellite is higher and provides a lower resolution of the pixels while, the UAV has a lower GSD and provides a higher resolution of pixels representing the ground surface more accurately.

RECOMMENDATIONS

For the purpose of carrying out survey using a UAV, one should ensure to plan out the flight beforehand in order to make the process faster, by establishing the perimeter that is to be surveyed on a map. And also establishing of ground control points for getting precise locations.

The time of day in which the UAV is to be used should be considered and the weather also, as it affects the output of the images. It is best recommended to take pictures when the sun is directly on top of ones' shadow and the clouds are still. During the image processing the effects can be seen in the outputs, making it have a higher resolution and uniformity.

A flying altitude of less than 30m gives an accuracy of about 1cm/pixel of the ground sampling distance. Lower altitudes should be use for more accurate results. Before sending the drone on a mission, careful and detailed flight plan should be made to save time and for effectiveness.

In designing a topographical map, the desired use for the map and accuracy should be considered before deciding on the photo scale to be used as well as the contour interval.

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1st International Civil Engineering Conference (ICEC 2018)
Department of Civil Engineering
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Soil Slope Stability Analysis Using Limit Equilibrium Method for a Proposed Water Reservoir at Nile University of Nigeria Abuja

Mambo A.D, Mogbo O. N, Dayyabu A., Ozsoy U, Albadawi M. M.
Department of Civil Engineering Nile University of Nigeria, Abuja
*Corresponding author email: abdulhameed.mambo@nileuniversity.edu.ng

ABSTRACT
Geotechnical and hydraulic properties of soil materials differ considerably from place to place. It is therefore important to do slope stability analysis to determine how stable a slope will be especially for a major reservoir project. This paper is an investigation of slope stability at the site of a proposed natural water reservoir at Nile university of Nigeria Abuja. The purpose is to determine whether simple slope stabilization is sufficient to guarantee safe operation. Soil Samples were collected from six trial pits and laboratory tests were conducted using standard test methods such as the grain-size distribution test and the Atterberg limits test to determine the permeability as well as the unit weight and specific gravity of the soil. Stability analysis was performed using slope stability analysis software Oasys Slope 19.1 based on limit equilibrium approach for creating slope models. Circular failure surfaces through the slope were analyzed for static loading conditions. Factors of safety were calculated using the "Bishop," "Janbu," "Swedish," "Felleniu's," "Sarma," and "Spencer" methods. These methods yielded very similar results regarding the relative sensitivity of the assumed factor of safety. The average of the six methods was chosen as a factor of safety for the slope. Stability analysis in which critical circular slip surfaces have been located showed that the critical slip surface exits the toe of the slope at angles very similar to what would be expected based on passive earth pressure theory.

Keywords- *Factor of Safety; Slope stability; Soil tests; Water Reservoir; Direct Shear Test*

1. INTRODUCTION

Slope stability is the potential of soil-covered slopes to withstand movement. A previously stable slope may be initially affected by preparatory factors, making the slope conditionally unstable. Triggering factors of a slope failure can be climatic events, which can then make a slope unstable, leading to mass movements. Mass movements can be caused by increase in shear stress, such as loading, lateral pressure, and transient forces. Alternatively, weathering may decrease shear strength, changes in pore water pressure, and organic material (Lin & Chang, 2004).

The geometry of the slope also has direct correlation with soil mechanical properties, underground water level and also with slope stability and liquefaction ability, the liquefaction ability at any earthen structure or area is specific (Varnes, 1978).

Many authors have carried out extensive literature reviews of landslides in reservoirs as well as detailed case studies (Alonso & Pinyol, 2011; Crozier, 2010, Gutiérrez, et-al 2010). In particular deep-seated rock slides have been monitored and investigated intensively, and monitoring results have been reported over the last decades (Saurer, et-al, 2013). Moreover, process-based investigation methods and monitoring measures have been developed (Zangerl, et-al, 2010, Baroň, et-al, 2014, Jomard, et-al, 2010). The issue of slope stability, in particular for reservoir safety, during rapid drawdown loading and has been studied widely (Fredlund, et-al, 2011; Paronuzzi, et-al, 2013), The fundamental requirement for stability of slopes is that the shear strength of the soil must be greater than the shear stress required for equilibrium (Corporation & Manual, 2015). Given this basic requirement, it follows that the most fundamental cause of instability is that, for some reason, the shear strength of the soil is less than the shear stress required for equilibrium (Safety, 1978). This condition can be reached in two ways: through a decrease in the shear strength of the soil or through an increase in the shear stress required for equilibrium (Duncan, et-al, 2014).

When considering slope stability problems in geotechnical engineering, an early step consists of correctly determining the level of safety of a slope. A correct determination of level of safety should properly handle the three-geotechnical fundamentals that control slope stability: geometry, pore pressures, and strength. Nearly all the methods commonly used to integrate these fundamentals into determination of a safety factor provide similar answers (Silva, et-al, 2008).

Limit equilibrium method first defines a slip surface, then it analyses the slip surface to obtain the factor of safety, which is defined as the ratio between forces (moments or stresses) causing stability of the mass and those that resisting stability (disturbing forces).

Two-dimensional sections are normally analyzed assuming plain strain conditions. The assumption for these methods is that the linear (Mohr-Coulomb) or non-linear relationships between shear strength and the normal stresses on the failure surface regulate the shear strengths of the materials in the direction of the potential failure surface.

Functional slope design determines the critical slip surface where the factor of safety is found to be of last value. Computer programs help locate failure surface using optimization techniques. The program analyzes the stability of different layered slopes, different embankments, and structures. Fast optimization of different slip surfaces (circular & non- circular surfaces) gives the lowest factor of safety. External forces (Earthquake effects, external effects by loading, groundwater conditions, and stabilization forces) can be included. The software uses method of slices to decide the factor of safety (Pereira, et-al, 2016).

With regard to the serviceability limit state design of slopes, Euro code Geotechnical design standards (EN 1997-1 11.6-1) P states that the design of slopes shall show that the deformations of the ground will not cause serviceability limit state in structures and infrastructure on or near the particular ground (Bond, et-al, 2009).

For simple slopes it is possible to estimate the location of the critical slip surface relatively well. For example, for a homogeneous slope composed of dry cohesion less soil with a constant friction angle (linear strength envelope), the critical slip surface is a plane coincident with the face of the slope. The factor of safety for most cases the critical slip surface must be determined by trial and error (Gedeon, 2003).

Analysis methods all calculate a factor of safety for the slope, but they differ in how they calculate the forces acting upon the slope. In Janbu's method, the driving force is the K factor (0.0 – 1.0) that is needed to place the soil mass into limiting equilibrium. Whereas, the other methods compute the ratio of the total resisting force to the total driving force for the soil mass being evaluated.

The Simplified Bishop method defines the factor of safety as the ratio of the available shear strength of the soil to that required to maintain equilibrium. The Spencer-Wright analysis method is a limiting equilibrium technique, which balances vertical force, horizontal force, and moment equilibrium. This contrasts with the Bishop method, which satisfied only vertical force and moment equilibrium. In Sarma's method the K factor is determined explicitly by solving the equations for equilibrium for each of the constraints specified by the user. In the other methods, the safety factor is determined for each failure configuration by varying the geometry of the failure surface by changing the left and right ending points and the radius of the failure surface.

This requires a hunting algorithm for the other methods to vary the geometry, compute the safety factor, keep track of minimum safety factor within the confines of the constraints placed on the search area (Duncan et al., 2014).

The mathematics of slope failure calculation became more complicated around 1920, when the Swedish Geotechnical Commission presented their findings that slope failures tend to occur along a curved surface in soils and unconsolidated material. Slope failures in consolidated material (bedrock) tend to occur along fractures or bedding planes as a non-circular surface. Since then, engineers and scientists have developed several methods of mathematical iteration to more accurately resolve all of the forces acting upon a slope. Some methods are more suitable than others, depending upon geometry and slope material properties.

The aim of this project is to analyze basic slope failure and design suitable slope and stability measure with a systematic technique based on studies and site investigations for the stability of the proposed water reservoir slope at Nile university of Nigeria.

2. PROJECT DESCRIPTION

The proposed water reservoir is located within the university campus (figure1). The reservoir has dimensions of length of 230m, a width of 32m, and depth of 2m from ground surface, and the reservoir will have a capacity of about 10 million liters of water. In addition to serving as water storage facility, the proposed reservoir will be recreational facilities with beautiful landscape design around the reservoir. The reservoirs main source of water will be from a nearby stream through a specifically designed underground pipe flow channel.

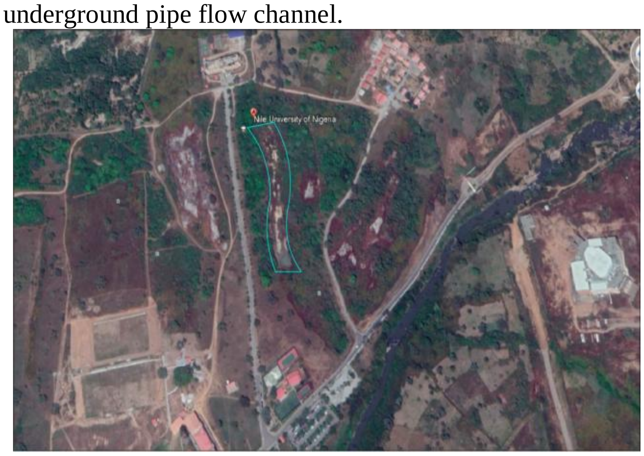

Figure 1: Satellite view of the reservoir

3. MATERIALS AND METHODS

The method comprises of site visitation and a series of soil investigations carried out according to technical specifications of geotechnical design manuals, suitable slope stability measures were designed, detailed geometry of the slope was determined and basic slope failure modes and failure analysis were carried out. Also, geotechnical design and slope stability software, computer aided design programs were used for information modeling, stability analysis, and detailing.

The process used in this project was auger boring to collect disturbed samples. Rotating a soil auger while pressing it into the soil advances the hole and as the auger gets filled with soil, it is taken out and the soil sample collected. Augers were hand-operated and not power-driven. Samples were taken from six trial pits at a depth of 2.0m at an interval of 40m along the reservoir.

The laboratory tests were conducted in accordance with American standards for testing and materials (ASTM) standards.

For the stability analysis according to limit equilibrium theory, method of slice was applied by subdividing the slope into slices of constant width and by assuming circular slip surface. The stability analysis of the slope was performed using slope stability analysis software (Oasys Slope 19.1) based on the limit equilibrium approach. For creating slope models the average soils angle of friction (Φ), unit weight (γ) and cohesion (C) have been chosen. The slope models was developed by six deferent methods of analysis; (Felleniu’s, Bishop, Spencer, Swedish Janbu and Sarma.), based on geometry, and mixed soil mechanical properties were developed.

4. RESULT AND DISCUSSIONS

4.1. ATTERBERG LIMIT TEST

Table 1: Summary result from Atterberg Limit Test

Liquid Limit (LL)	*25.97%*
Plastic Limit (PL)	18.27%
Plasticity Index (PI)	7.24%

The Unified soil classification (USCS) chart was used to determine the composition of the soil. The soil was classified as fine-grained with low compressibility, putting it in the ML group Silt with low plasticity.

4.2. SIEVE ANALYSIS TEST

Effective size of soil on D60, D30 and D10

D60 = 0.571
D30 = 0.233
D10 = 0.065
Coefficient of Uniformity Cu = 1.465
Coefficient of Curvature Cc = 8.785
% Gravel- 10.66
% Sand - 80.23
% Clay - 9.11
This indicates that the sample is well-graded.

4.3. DENSITY AND WATER CONTENT

Table 2: Proctor compaction test graph

Optimum moisture content	*13%*
Maximum Dry Density	2.15 g/cm^3
Average dry density (ρd)	2.00 g/cm^3
Bulk density	2.18 g/cm^3
Specific Gravity of Soil (Gs)	2.69

From table 2, soil can be classified as silty sand based on the specific gravity of 2.69.

4.4. COHESION AND ANGLE OF FRICTION

The cohesion of the soil and the angle of friction of the soil were determined using the Direct Shear Method. The angle of friction is the angle of the linear line produced (line’s slope). The different individual samples were tested separately and data obtained from each sample were used to determine the average Cohesion and angle of friction of the soil in site by plotting a graph of normal stress against shear stress in figure 5.

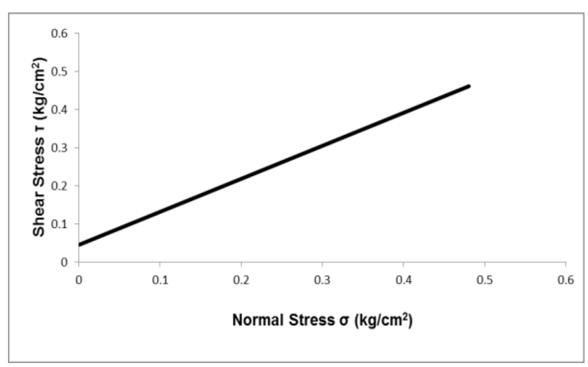

Figure 2: Graph of shear stress against normal stress

From Figure 2, the angle of the gradient of the graph is 30°. The intercept on the y-axis is at the value of 0.04 KN/m^2. Therefore experimentally, cohesion of the soil, c

= 0 KN/m^2. Theoretically, the cohesion C, of the sample used should be approximately zero because the samples are silty and clayey sand grains previously classified.

4.5. SLOPE GEOMETRY

4.5.1. Existing Geometry

The existing height of the subject slopes was about 1.5 - 2 m from the top of the slope to the toe of slope. See figure 3. The steepness of the slopes ranged from 2.7 H: 1V to 3.5H: 1V. The slope is generally covered with grasses and surrounded with tress and understory vegetation. No slope slumping or tension cracks were observed on the slope. Slight erosion features such as scour, rills or bare spots were present at the time of our site visit. Slope surface shows no presence of rapid drawdown of the existing slope from the ground surface. No excess water seepage was observed on the slope surface or at the toe of slope area.

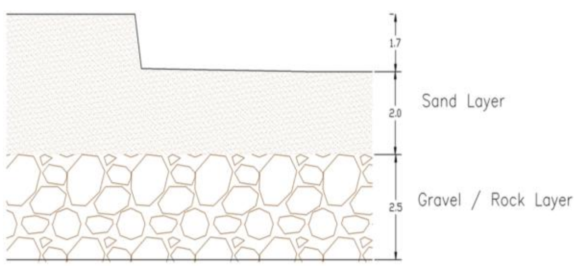

Figure 3: Existing cross section profile

4.5.2. Proposed Geometry

The proposed design for the slope will be 2m in height from top of the slope to the toe as shown in figure 4. The steepness of the slope is determined to be 1.5V: 3H. This geometry was calculated to give an angle of repose of 23° in which the safety factor is calculated for the stability of the slope in relation with the geotechnical properties of the soil.

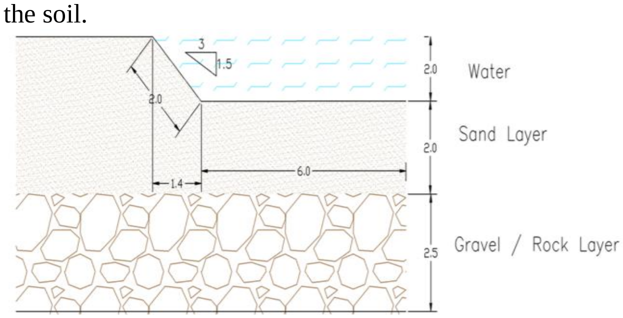

Figure 4: Proposed vertical cross section

The slope geometry has direct correlation with soil mechanical properties, underground water level and also with slope stability and liquefaction ability, the liquefaction ability at any earthen structure or area is specific.

4.6. STABILITY ANALYSIS

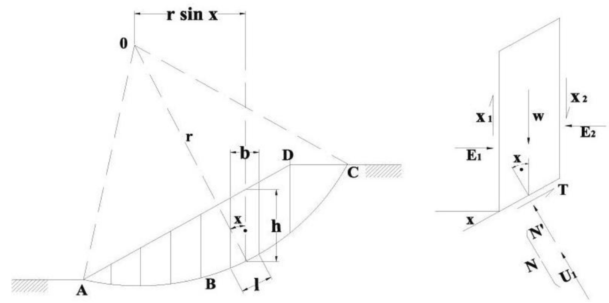

Figure 5: Schematics diagram Showing method of slice

Figures presenting the results of stability analysis under static and loading conditions using the limit equilibrium method by the various methods of analysis are presented. In these figures, the geotechnical parameters of the encountered formations are also included. The critical slip circular surfaces are showed corresponding to the factor of safety assumed by the analysis.

4.6.1. Bishops Method Analysis

The Bishop's Method assumed a factor of safety of 2.512, which is above the minimum allowable factor of safety for liquid equilibrium state analysis. The yellow circular shows a slip surface of a factor of safety above 2.4. However, the minimum factor of safety assumed by the method was at a range of 1.5 – 0.5 indicated in red and orange in the figure 5.4 The maximum factor of safety was assumed to be above 4, which shows a high factor of safety against a large critical slip surface.

4.6.2. Janbu's Method Analysis

The Janbu's Method assumed a factor of safety of 2.513, which is approximately the same with bishop's method. The orange colure slip surfaces show a factor of safety above 2.2. However, the minimum factor of safety assumed by the method was at a range of 1.1 – 0.5 indicated in red and orange in the figure 5.5. The maximum factor of safety was assumed to be above 2.7, indicated by green slip surfaces, which shows a high factor of safety against a large slip surface.

4.6.3. Felleniu's Method Analysis

The Felleniu's Method assumed the highest factor of safety of 2.869, which is also above the minimum allowable factor of safety for liquid equilibrium state analysis. However, the minimum factor of safety assumed by the method was 2.472 indicated in red. The maximum factor of safety was assumed to be above 5, which shows a high factor of safety against a large critical slip surface.

4.6.4. *Sarma's Method Analysis*

The Sarma's Method assumed a factor of safety of 2.585, which is above the minimum allowable factor of safety for liquid equilibrium state analysis and close to Janbu and spencer's method result. The maximum factor of safety was assumed to be also above 5, which shows a high factor of safety against a large critical slip surface.

4.6.5. *Spencer's Method Analysis*

Spencer's method also assumed a factor of safety of 2.517. For large critical slip surface, the assumed factor of safety was assumed to be above 3, which is the minimum of all the methods. Likewise, a minimum factor of safety of 0.587 was assumed.

4.6.6. *Swedish Method Analysis*

Swedish method assumed the lowest factor of safety of 2.060. The highest assumed safety factor for critical slip surface was 2.6, much lower than the other methods. However, the minimum critical slip surface factor of safety assumed was close to that of other methods.

4.7. ASSUMED FACTOR OF SAFETY

Factors of safety calculated using the methods commonly identified as "Bishop," "Janbu," "Swedish," "Felleniu's," "Sarma," and "Spencer", yielded very similar results regarding the respective sensitivity of the calculated safety factor to each input parameter. Therefore, for simplicity, the average of the six assumed factors of safety by the methods used in the analysis was chosen as a factor of safety for the slope. Table 2 shows the factor of safety obtained from all the methods.

Table 3: Factors of safety obtained by analysis methods

Method	*Felleniu*	*Bishop*	*Janbu*	*Swedish*	*Spencer*	*Sarma*
Factor of Safety	2.869	2.512	2.513	2.060	2.517	2.585
Average			2.509			

The results of slope stability analysis with the limit equilibrium method indicate acceptable safety factor values for static loading conditions. The minimum factor of safety assumed was 2.060 calculated by Swedish method and the maximum factor of safety assumed was 2.869 calculated by Felleniu's method. Choosing the average factor of safety assumed by all the six methods shows a safe slope design, which is above the minimum safety of factor recommended by limit equilibrium theory. By observing the results of the analysis depicted in table 4, it is concluded that the resulting factors of safety correspond to slip circles passing through the main part of the slope. Thus, the most unfavorable conditions correspond to internal slope stability.

It should be mentioned that in the above analysis, the drained shear strength is used for the sand layer (Layer I), whereas only the value for the angle of internal friction $\Phi = 30°$ is given as an input parameter where the cohesion $c = 0$. As already mentioned, in slope stability analysis under static loading conditions the shear strength parameters of the encountered formations are reduced through the soil parameters partial factors.

However, with regard to stability analysis of slopes, Euro code Geotechnical design standards (EN 1997-1 §11.5.1-4) states that the mass of soil or rock bounded by the failure surface should normally be treated as a rigid body or several rigid bodies moving simultaneously (Bond et al., 2009).

5. CONCLUSSIONS

Four basic factors determine the stability of a slope: Unit weight of the Soil, Slope angle, Cohesion of the slope material and Angle of Internal Friction.

Before the slope stability analysis, laboratory tests were carried out to determine the geotechnical properties of the soil. These tests yielded results used for the slope geometry design and stability analysis.

Using the grain-size test and the Atterberg limits test the classification of the soil samples was achieved. To determine the liquid and plastic limit of a soil sample, the Atterberg test was used, and based on the data that was collected, the water content of the sample was determined from this result, the liquid limit and plastic limit was obtained. This determined the soil type, which corresponded with the sieve analysis method indicating a silty sandy soil. It can be readily observed that the values of optimum moisture and max dry unit weight for this sample are closest to those of ML. This suggests that a large part of the sample remains to be comprised of this type of soil. The soil's low plasticity causes it to achieve a relatively low value of optimum moisture content.

Permeability of this sample is moderate. This is because the porosity of sand and silt is high or moderate where by water can flows through the soil with less resistance. It can drain water easily but hardly can retain any water.

The cohesion of the soil and the angle of friction of the soil were determined. The angle of friction is the angle of the linear line produced (line's slope). The angle of the gradient of the graph is 30° which is the angle of friction of the soil and relatively, the cohesion of the soil, C = 0 KN/m^2.

The stability analysis of the slope was performed using slope stability analysis software (Oasys Slope 19.1) based on the limit equilibrium approach for creating slope models. Circular slip failure surfaces through the slope were analyzed for static loading conditions. The circular lines in the developed models show the path of the slip surface. The colours indicate the factor of safety for each assumed critical circular failure surface. Factors of safety were calculated using the methods commonly identified as "Bishop," "Janbu," "Swedish," "Felleniu's," "Sarma," and "Spencer". The six methods yielded very similar results regarding the relative sensitivity of the calculated factor of safety to each input parameter. Therefore, for simplicity, the average of the six assumed factors of safety by the methods used in the analysis was chosen as a factor of safety for the slope.

Slope stability calculations in which critical noncircular slip surfaces have been located show that the critical slip surface exits the toe of the slope at angles very similar to what would be expected based on passive earth pressure theory. Many computer programs that search for critical noncircular slip surfaces contain provisions for limiting the steepness of the slip surfaces where they exit the slope.

6. ACKNOWLEDGEMENT

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STABILIZATION OF A-6 LATERITIC SOIL USING RICE HUSK ASH AND PROMOTER

* D. Amodu[1], M. Alhassan[1] and M. M. Alhaji[1]

[1] Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: engramodudaniel@gmail.com

ABSTRACT

Lateritic soil sample classified as A-6, taken from a borrow pit at Birgin Gwari, Minna, was stabilized with 0 to 6% Rice Husk Ash (RHA), at 2% variations. Each of the soil-RHA mixture was admixed with 0 to 2% promoter (calcium chloride and sodium hydroxide), at 0.5% variations. Unconfined Compressive Strength (UCS) test samples of the soil-RHA-promoter mixtures were prepared at Maximum Dry Density (MDD) and Optimum Moisture Content (OMC), cured for 1, 7, 14, 28 and 60 days, before testing. The results indicated marginal increase in UCS with increase in curing period, but no defined trend in the UCS values was observed as curing period increases constant promoter content. The results also indicated that, at constant percentage content of promoter, increase in percentage of RHA has more pronounced effect on the strength (UCS) of the stabilized soil than curing age. The UCS values peak at 6% RHA/0.0% promoter and 28 days curing period.

Keywords: *Lateritic soils, Rice husk ash, Promoter, Maximum dry density, Optimum moisture content, Unconfined compressive strength.*

1 INTRODUCTION

Soil improvement could either be by modification or stabilization, or both. Soil modification is the addition of a modifier (cement, lime, etc.) to a soil to change its index properties, while soil stabilization is the treatment of soils to enable their strength and durability to be improved such that they become totally suitable for construction beyond their original classification (Alhassan 2008).

Lateritic soils are generally used for road construction in Nigeria. Some of these soils in their natural state generally have low bearing capacity and low strength due to high content of clay. When lateritic soil contains a large amount of clay materials its stability and strength cannot be guaranteed under load in presence of moisture (Alhassan, 2008). When lateritic soil consists of high plastic clay, the plasticity of the soil may result to cracks and damage on pavement, roadways, building foundations or any civil engineering construction projects. Improvement in strength and durability of lateritic soil in recent times has become imperative; this has geared researchers towards using stabilizing materials that can be sourced locally at a very low cost (Bello *et al*., 2015). Where in most cases sourcing for alternative soil may prove economically unwise, to improve the soil by way of stabilizing it to meet the desired objective becomes a viable option (Mustapha 2005, Osinubi, 1999). These local materials can be classified as either agricultural or industrial wastes *Amu et al.* (2011a). The ability to blend the naturally occurring lateritic soil with some chemical additives to give it better engineering properties in both strength and water proofing is very essential (Amu *et al*., 2011b; Amu and Adetuberu, 2010; Bello *et al*., 2014).

Over the years, cement and lime have been the two major materials used for modifying or stabilizing soils. The price of these materials has rapidly increased due to the sharp increase in the cost of energy and high demand. This has therefore, prevented third world countries Nigeria from providing good road for its citizen particularly rural dwellers. Amu *et al*., (2011b), Bello *et al*. (2014) and Sear (2005) showed that Portland cement, by nature of its chemistry, produces large quantities of CO_2 for every ton of its final product which contributes to the melting of the ozone layer covering the earth surface. Therefore, replacing cement in soil stabilization with agricultural waste material, like Rice Hush Ash (RHA) and little promoter for stabilization of soils will reduce the overall environmental impact of the stabilization process.

Rice husk is an agricultural residue gotten from paddy rice. World rice production (2016) indicated that the global rice production by 2015/2016 season was 472.09 million tons, while for 2016/2017, it was estimated to be 483.26 million tons, representing an increase of 11.17 million tons (2.37%) rise in the production. This translate to about 157.2 and 160.9 million tons of rice husk for 2015/2016 and 2016/2017 respectively, with corresponding increase of 2.37% over these two seasons (Alhassan and Alhaji, 2017). According to Oyetola and Abdullahi (2006), about 2.0 million tons of rice is produced annually in Nigeria. Rice production in Nigeria for 2016/2017 season was projected by Wailes and Chavez (2012) to stand at 3.120 million metric tons.

Partially burnt husk from the milling plants when used as a fuel, also contributes to pollution and efforts are being made to overcome this environmental issue by utilizing it as a supplementary cementing material (Chandrasekhar *et al*, 2006). The chemical composition of rice husk is found to vary from one sample to another due to the differences paddy type, production year, climate and geographical conditions. Chandrasekhar, *et al*. (2003) and Zhang, *et al*. (1996) stated that burning rice husk under controlled temperature below 800°C can produce ash with silica, mainly in amorphous form. Nair *et al.*, (2008) reported an investigation on the pozzolanic activity of RHA by using various techniques in order to verify the effect of incineration temperature and burning duration. They stated that the samples burnt at 500 or 700 °C and for more than 12 hours produced ashes with high reactivity with no significant amount of crystalline material. The short burning durations (15 – 360 minutes), resulted in high carbon content for the produced RHA, even with high incinerating temperatures of 500 to 700 °C. A state-of-the-art report on RHA was published by Mehta (1992), which contains a review of physical and chemical properties of RHA, the effect of incineration conditions on the pozzolanic characteristics and a summary of the research findings from several countries on the use of the ash as a supplementary cementing pozzolanic material. Alhassan and Alhaji (2017), in a view, chronicled utilisation of RHA for improvement of deficient ioils in Nigeria. Use of RHA and promoter in soil stabilization is relatively a new area.

Promoters are chemical waste that increase available surface area of stabilization against crystal growth and lead to improvement of mechanical strength. In other to improve the strength, durability and engineering properties of soil-cement, small quantities of promoter have been used. Addition of 1.0 and 4.0% by weight of hydroxides and various salts, greatly increase compressive strength (Lambe *et al*., (1979).. O'Flaherty (1988) noted that most promoters and agricultural wastes possess pozzolanic properties that have cementitious tendencies on exposure to moisture. Robert (1993) defined pozzolanas are siliceous and aluminous material which themselves possess little or no cementitious value but, will, in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature, to form compounds possessing cementitious properties. Sodium silicate has often been used as a pozzolana for replacement of RHA. However, sodium silicate is expensive and difficult to handle, which is why a cheap and easy to handle promoter (calcium chloride and sodium hydroxide) sorted and used in this research as replacement for RHA.

2 MATERIALS AND METHODOLOGY

2.1 Materials

The materials used in this research are A-6 lateritic soil, rice husk ash, promoter (calcium chloride and sodium hydroxide) and distilled water

2.1.2 Lateritic soil

The disturbed lateritic soil sample used for this study was collected from borrow pit at Birgin Gwari, a suburb of Minna town in Niger State, Nigeria. Before the soil samples were taken, the top soil was removed to depth of 0.5 to 1m. The sample was collected, sealed in plastic bags and placed in sacks to avoid loss of moisture during transportation. The soil sample was then air dried in Civil Engineering laboratory, Federal University of Technology, Minna. Before commencing laboratory tests of the soil sample, it was pulverized and sieved through BS No. 4 sieve (4.76mm).

2.1.3 Rice husk ash

The Rice Husk Ash (RHA) used for this study was obtained after burning rice husk collected from a local rice milling plant in Minna. The raw husk of parboiled rice was burnt for 2 days in open place without controlling the temperature (mass burning/ashing). The ash was then transported to laboratory and sieved through sieve 75µm and then stored in air-tight polythene bags to avoid any form of hydration.

2.1.4 Promoter

The promoter used for this study is calcium chloride and sodium hydroxide, which was obtained from chemical and agro products sellers in Ibadan, Oyo state, Nigeria. The promoter was obtained in a solid form and converted to 1.0 molar concentration of calcium chloride and 1.5 molar concentration of sodium hydroxide.

$$Ca\ Cl + NaOH\text{-----}\ Ca(OH)2 + NaCl \qquad (1)$$

2.1.5 Distilled Water

The water used for this research was distilled water. This was obtained in a chemical equipment shop in Bosso, Minna. Distilled water is of great importance in the stabilization process involving promoter, because presence of impurities in water can affect the cementitious process and reduce the compressive strength and durability of the stabilized soil.

2.2 Methodology

Laboratory test were carried out to determine the index properties of the natural soil, which include moisture content, specific gravity, sieve analysis, Atterberg limit, compaction characteristics. These tests were conducted accordance with BS 1377 (1990). XRF and XRD tests were also carried out to determine the chemical and mineralogical properties of the lateritic soil and RHA.

The natural lateritic soil was then thoroughly mixed with RHA, varied from 0 to 6% at 2% variations. Each of the soil-RHA mixture was then admixed with 0, 0.5, 1.0, 1.5 and 2% promoter. Compaction test was carried on the natural and the soil-RHA-promoter mixtures in accordance with BS 1377 (1990) and BS 1924 (1990) respectively. The Maximum Dry Density (MDD) and Optimum Moisture Content (OMC), obtained from the compaction tests, carried out at British Standard Light (BSL) energy level, were then used to mould cylindrical samples that were eventually used for Unconfined Compressive Strength (UCS) test. Molded samples for UCS tests were properly sealed and cured for 1, 7, 14, 28 and,60 days before testing in with BS 1377 (1990) and BS 1924 (1990) respectively for unstabilized and stabilized samples respectively.

3 RESULTS AND DISCUSSION

3.1 Index Properties of the natural soil

Results of index properties tests conducted on the natural lateritic soil are presented on Table I, while Figure 1 shows XRF of the soil. From Table I, the soil is an A--6 and CL (clay of low plasticity) according to AASHTO and Unified Soil Classification System (USCS) respectively. It falls below the standards recommended for most geotechnical construction works and would therefore require stabilization (AASHTO, 1986; Alhassan and Alhaji, 2007).

TABLE I: INDEX PROPERTIES OF THE NATURAL LATERITIC SOIL

Property	Quantity
Percentage Passing BS sieve No 200 (%)	57.5
Natural Moisture Content (%)	5.1
Liquid Limit (%)	39.36
Plastic Limit (%)	24.42
Plasticity index (%)	14.94
OMC (%)	12.30
MDD (Mg/m^3)	2.1634
Specific Gravity	2.6
AASHTO Classification	A-6
Unified Soil Classification System	CL
Unconfined Compressive Strength (kN/m^2)	146.9
Colour	Reddish Brown

TABLE II: CHEMICAL COMPOSITION OF THE RHA

Constituent	Composition%
SiO_2	67.3
Al_2O_3	4.9
Fe_2O_3	0.95
Ca	1.36
MgO	1.80
Loss in lgnition (LOI)	17.78

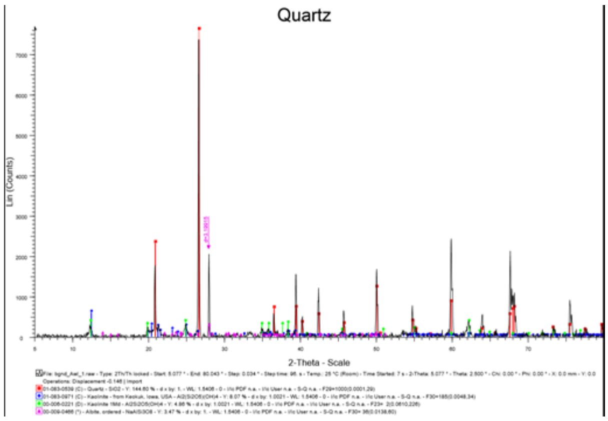

FIGURE 1: XRF OF THE LATERITIC SOIL.

3.2 Effect of Treatment with RHA and promoter

3.2.1 Unconfined Compressive Strength

Unconfined compressive strength (UCS) is the most common and adaptable method of evaluating the strength of stabilized soil. It is the main test recommended for the determination of the required amount of additive to be used in stabilization of soil (Singh and Singh, 2008). Variation of UCS with increase in RHA from 0 to 6% and promoter from 0 to 2% at British Standard Light energy level and after 1, 7, 14, 28 and 60 days curing period are presented in Figures 2 to 6.

The Variation of UCS with promoter at various percentage of RHA for 1 day curing period is shown in Figure 2. From the figure, it is observed that, at constant percentage content of RHA, UCS gradually reduces as percentage of the promoter increase. The figure also shows that as the percentage content of RHA increases, UCS increases. The UCS reduce from 146.9 to 43.2 kN/m^2 at 1.5 % promoter and increase from the natural value of 146.9 kN/m^2 to peak value of 268.5 kN/m^2 at 0.5% promoter/6% RHA. This subsequent increase in the UCS value is attributed to the formation of cementitious compounds between the CaOH, present in the promoter and RHA and the pozzolans present in the RHA. The decrease in UCS values, after 1.5% promoter may be attributed to the excess promoter introduced to the soil and therefore forming weak bonds between the soil and the cementitious compounds formed.

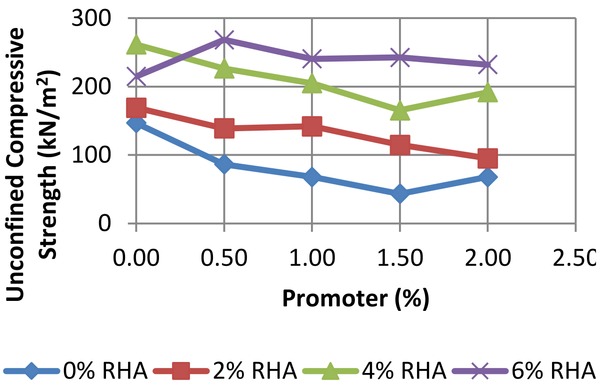

FIGURE 2: VARIATION OF UCS WITH PROMOTER AT VARIOUS PERCENTAGE OF RHA FOR 1 DAY CURING PERIOD.

Variations of UCS with promoter at various percentage of RHA for 7 days curing period are shown in Figure 3. From this figure, it is also observed that UCS gradually values decreased as percentage content of the promoter increases, while increase in UCS values are observed as percentage of RHA increased. UCS reduced from 117.2 to 47.5 kN/m^2 at 1.5 % promoter/0% RHA. Increase in UCS values is observed for the natural soil from 117.2 kN/m^2 to peak value of 300 kN/m^2 at 0.5% promoter/6% RHA. This trend in the variation of UCS values is attributed the reason advanced in the case of 1 day curing period.

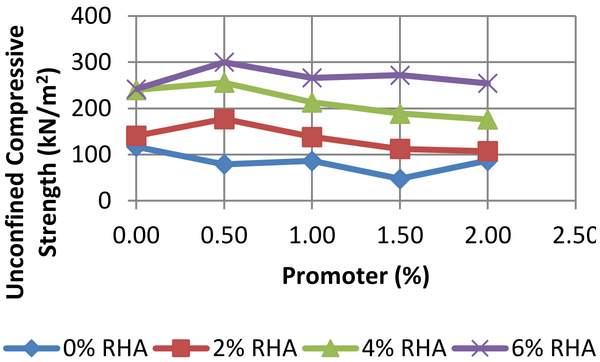

Figure 3: Variation of UCS with promoter at various percentage of RHA for 7days curing period.

Figure 4 presents variation of UCS with promoter at various percentage of RHA for 14 days curing period. Similar trend in variation of UCS values is also observed for the 14 days curing period. UCS reduced from 166.7 to 40.1 kN/m^2 at 1.5 % promoter/2%RHA. For the natural soil, it increased from 166.7.2 kN/m^2 to peak value of 288.9 kN/m^2 at 0.5% promoter/6% RHA. The trend in the variation of UCS values for this curing period is attributed the reason advanced for the case of 7 days curing period.

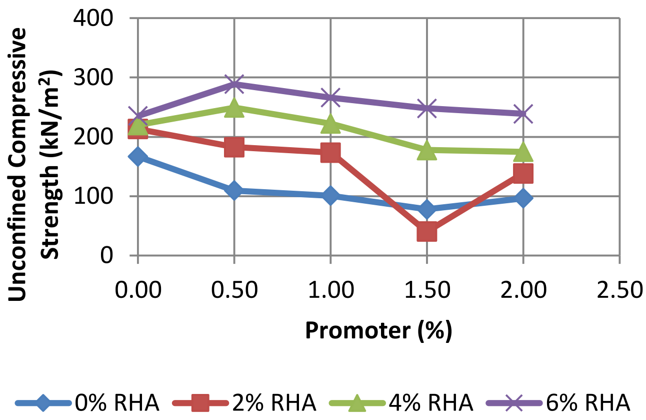

FIGURE 4: VARIATION OF UCS WITH PROMOTER AT VARIOUS PERCENTAGE OF RHA FOR 14 DAYS CURING PERIOD.

Figure 5 presents variation of UCS with promoter at various percentage of RHA for 28 days curing period. From the figure, it is also observed that UCS gradually reduced as percentage of the promoter increased, and increases with increase in percentage of RHA. UCS value reduced from 118.5 to 43.2 kN/m^2 at 2 % promoter/0%. It increase from 118.5 kN/m^2 at 0.5% promoter/0%RHA to peak value of 336.4 kN/m^2 at 0% promoter and 6% RHA. The maximum UCS value recorded was 293 and 295kN/m2 at 6 and 8% RHA contents respectively, after 28 days curing period.

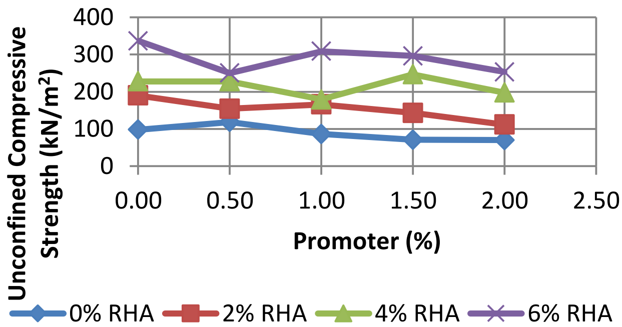

FIGURE 5: VARIATION OF UCS WITH PROMOTER AT VARIOUS PERCENTAGE OF RHA FOR 28 DAYS CURING PERIOD.

Figure 6 presents variation of UCS with promoter at various percentage of RHA for 60 days curing period. From the figure, it is observed that UCS reduces with increase in percentage of promoter. UCS reduces from 132.7 to 74.1 kN/m^2 at 2 % promoter/0%. It increased from 132.7 kN/m^2 at 0.5%promoter/0%RHA to peak value of 268.5 kN/m^2 at 0.5%promoter and 6% RHA. From the figure, as percentage content of RHA increases, the UCS of the stabilized soil increased. This increase in the UCS is attributed to the formation of cementitious compounds between the CaOH present in the promoter and RHA and the pozzolans present in the RHA. The observed decrease in UCS values, after 1.5% promoter may be due to the excess promoter introduced to the soil, which resulted to the formation of weak bonds between the soil and the cementitious compounds, formed. This is in conformity with reason advanced by Alhassan (2008) and Alabi1 *et al*. (2015).

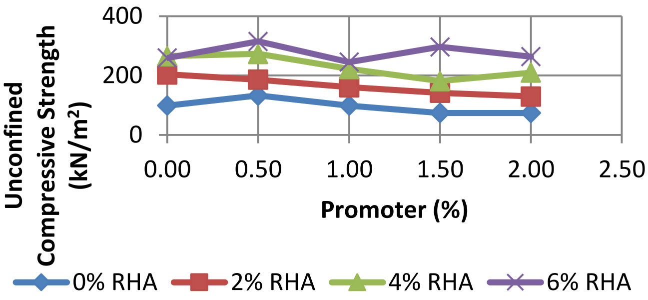

FIGURE 6: VARIATION OF UCS WITH PROMOTER AT VARIOUS PERCENTAGE OF RHA FOR 60DAYS CURING PERIOD.

3.2.2 Effect of curing age on UCS of the stabilized soil

Variation of UCS of the stabilized soil with curing period and at constant content of promoter are presented on Figures 7 to 10 for 0, 2, 4 and 6% RHA content respectively.

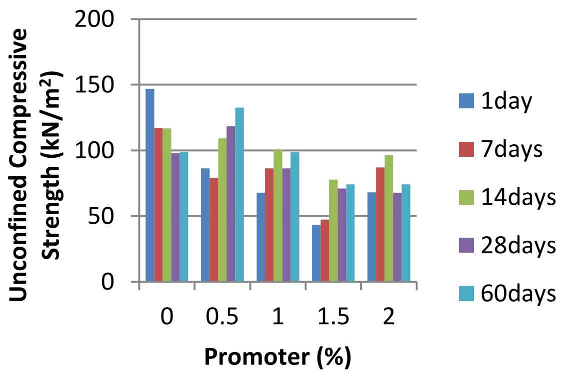

FIGURE 7: VARIATION OF UCS WITH PERCENTAGE OF PROMOTER AND CURING DAYS AT 0% RHA.

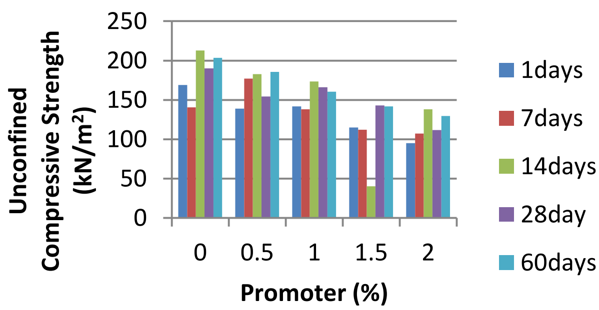

FIGURE 8: VARIATION OF UCS WITH PERCENTAGE OF PROMOTER AND CURING DAYS AT 2% RHA.

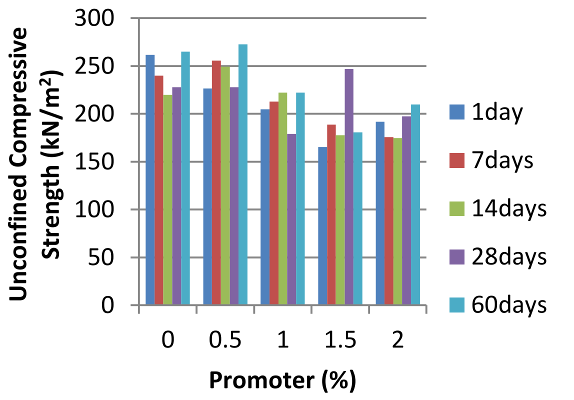

FIGURE 9: VARIATION OF UCS WITH PERCENTAGE OF PROMOTER AND CURING DAYS AT 4% RHA.

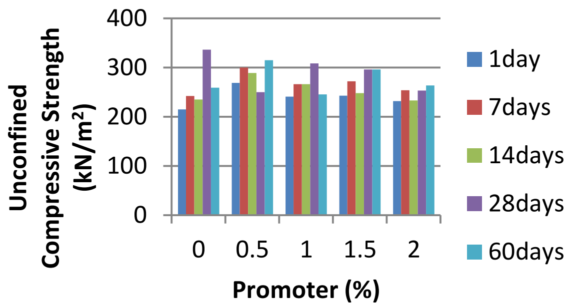

FIGURE 10: VARIATION OF UCS WITH PERCENTAGE OF PROMOTER AND CURING DAYS AT 6% RHA.

Observation of Figures 7 to 10 showed that, although, there is a marginal increase in UCS with increase in curing period, no defined trend in the UCS values was observed as curing period increases at constant content of promoter. The figures showed that increase in percentage of RHA has more pronounced effect on the strength (UCS) of the stabilized soil than curing age, at constant percentage of promoter.

4 CONCLUSION

From the results of this study, the following conclusions are drawn:

The lateritic soil used for the study was identified to be an A--6 and CL (clay of low plasticity) according to AASHTO and Unified Soil Classification System (USCS) respectively.

There was marginal increase in UCS with increase in curing period, but no defined trend in the UCS values was observed as curing period increases at constant content of promoter.

At constant percentage content of promoter, increase in percentage of RHA has more pronounced effect on the strength (UCS) of the stabilized soil than curing age. The UCS values peak at 6% RHA/0.0% promoter and 28 days curing period.

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PRECONSOLIDATION STRESS OF RESIDUAL SOILS IN NORTH-CENTRAL NIGERIA

***Abdulkareem M. F[1]; Alhassan M.[1], and Alhaji M. M.[1]**
[1]: Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author email: engrfatihu@yahoo.com

ABSTRACT
Preconsolidation stress of residual overburden soil in North-central Nigeria was studied with depth variation. The results indicate variability of maximum stress on the undisturbed deposit which it has experienced in its geologic past. While at some location, the preconsolidation pressure increased with depth at some other points, the pressure decreased with depth of the residual soils. The observed trend in variability of the preconsolidation pressure conformed with other studies carried out on similar residual profile from other regions.

Keywords: *North-central Nigeria, Preconsolidation pressure, Residual profile, Undisturbed soil deposit.*

1 INTRODUCTION

Residual soils overlie parent rocks in most parts of Nigeria. The parent rocks could be either crystalline rocks (from Basement Complex) or of sedimentary origin (from Sedimentary Basins). These soils are widely employed as construction and foundation materials in most parts of the country. In Nigeria, sub-surface conditions at building sites, in some cases (especially within Sedimentary Basins) are such that bedrock is far beneath the ground surface, while at other sites (especially within Basement Complex Terrain), it can be close to the ground surface. In either of these cases, residual soils still constitute material on which the foundations are sited. Although, the most common residual soil profile in the tropics (Nigeria inclusive) is the lateritic weathering profile, they could also be laterite or non-lateritic soils, consisting of clay-sized particles. Depending on their mineralogical compositions, clay-sized particles are known to pose challenges to civil engineers during and after construction (Adebisi and Adeyemi, 2012). This is because they exhibit plasticity and compressibility characteristics. Compressibility is the ability of a clay soil to reduce in volume on application of structural load over a period of time, resulting in settlement. Compressibility of a soil deposit is influenced by preconsolidation stress of the deposit.

Preconsolidation stress of a soil deposit deals with stress history of the deposit. Stress history of soil is concern with the pressure an undisturbed deposit has ever experienced in its geologic past. This load (stress) has significant influence on the compressibility of a particular soil deposit, especially under structural loads (Das, 1999; Punmia et al., 2005; Ranjan and Rao, 2005). A soil deposit can be classified as under-consolidated, if the existing surcharge load is higher than the load the soil has ever experienced in its geologic past; normally consolidated, if the existing surcharge load is equal to the load the soil deposit has ever experienced in its geologic past, and over-consolidated or pre-consolidated, if the existing surcharge load is less than the load the soil has ever experienced in its geologic past. Nishimura et al (1999) states that, estimation of volume change and evaluation of shear strength of soils require information on the present in-situ stress state in a soil mass and possible future changes to the stress state. This needed information, can be gotten from the study of stress history of soil deposit.

Solanki and Desai (2008) defined preconsolidation pressure as the maximum vertical overburden stress that a particular soil deposit has sustained in the past (Das, 1999; Punmia et al., 2005; Ranjan and Rao, 2005). The ratio of this pressure and the present overburden pressure is known as overconsolidation ratio (Alhaji and Alhassan,2013). Based on overconsolidation ratio, soils deposits are classified as normally consolidated, over consolidated or under consolidated.

Preconsolidation pressure can be utilized in the determination of the maximum overburden pressure that can be exerted on a soil without irrecoverable volume change. This is important in understanding shrinkage behavior, crack and structure formation and resistance to shearing stresses of soils. Previous stresses and other changes in a soil's history are preserved within the soils structure. If a soil is loaded beyond this stress, it is unable to sustain the increased load and its structure breaks down. This therefore, makes it an important parameter in geotechnical engineering. Selection of consolidation parameters, such as compression index (Cc), recompression index (Cr) or coefficient of volume change (mv), used for computing consolidation settlement, are often based on overconsolidation ratio (Solanki and Desai, 2008).

Preconsolidation pressure is not usually measured directly, but estimated using a number of indirect methods from laboratory data. The stress history of a soil is commonly and classically determined from one-dimensional compressibility test on undisturbed samples (Józsa, 2003). In 1936, Casagrande (1936) evolved a graphical method for determination of preconsolidation stress of a clay deposit, using results of one-dimensional oedometer (compressibility) test, represented on a semi-logarithmic graph (Figure 1). Grozicet al. (2003), Grozicet al, (2005), Clementino (2005) and Boone (2010) presented detailed and graphical summaries of these approaches. The graph consists of a recompression curve with a slope called recompression index Cr, and a virgin compression curve whose slope is known as the compression index Cc. These indices are very essential as they are used mainly in the evaluation of magnitude of consolidation settlement, which accounts for most of the settlement in clay soils.

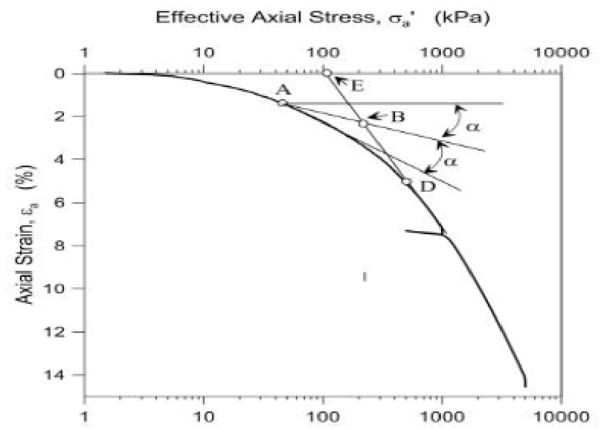

FIGURE 1: CASAGRANDE METHOD OF EVALUATING PRECONSOLIDATION PRESSURE P_C (SOURCE: HOLTZ AND KOVACS, 1981)

Although, many studies have been carried out on the properties of soils in Nigeria (Solanki, 2008; Józsa, 2013; Ola, 1987; Farrington, 1983; Ajayi, 1983; Madedor and Lal, 1985; Adesunloye, 1987; Omange and Aitsebaomo, 1989; Abolarinwa, 2010; Nwaiwu and Nuhu, 2006; Mustapha and Alhassan, 2012; Adebisi and Adeyemi, 2012), much attention has not been given to the study of preconsolidation pressure of these soil deposits. This work presents preconsolidation pressure of some residual soil deposits within North-central Nigeria, and hoped to serve as a preliminary guide for choice of design parameters for foundations of buildings and other structures with the studied area.

Casagrande (1936) method is still considered as the most commonly used method of determining preconsolidation pressure of soil deposits (Strokova, 2013). The method involves using empirical geometrical construction from the void ratio (e) vs logarithm of vertical effective stress, бv', curve to determine preconsolidation pressure (Figure 1). The method is therefore, employed in this study.

2 METHODOLOGY

To collect undisturbed soil samples for the study, sampling through boring was adopted. Six boreholes, drilled to bedrock, at six different locations, were used for the study. The Boreholes (BH) were tagged BH1, BH 2, BH 3, BH 4 BH 5 and BH 6, and were respectively sited as shown on Table I. From BH 1, samples were taken at 1.5, 3.0 and 4.5m, while from BH 2, samples were taken 1.5, 3.0, 4.5 and 6.0m. From BH 3, samples were taken at 1.5, 3.0, 4.5 and 6.0m, while from BH 4, samples were taken at 1.5, 4.5, and 6.0m. From BH 5, samples were collected at 1.5, 3.0, 6.0, and 9.0m, while from BH 6, samples were taken at 1.5, 3.0, 4.5 and 7.5 m. Collection of samples at the stated depths was informed by changes in strata of the soils in the respective bore holes. All the collected soil samples were carefully placed in labelled polythene sample bags, and transported to geotechnical laboratory of Civil Engineering, Federal University of Technology, Minna.

Although, preconsolidation stress, from one-dimensional consolidation test result is the main focus of this study, specific gravity and index properties tests were carried out. All the tests were conducted in accordance with BS 1377 (1990). From the resulting one-dimensional consolidation plot of void ratio (e) vs logarithm of vertical effective stress, бv, preconsolidation pressures for each of the tested samples were determined using Casagrande construction method. The method essentially involves (as shown on Figure 1): (i) choosing, by eye, the point of minimum radius (or maximum curvature) on the consolidation curve (point A in Figure 1); (ii) drawing a horizontal line from point A; (iii) drawing a line, tangent to the curve at point A; iv) bisecting the angle made by steps (ii) and (iii); (v) extending the straight-line portion of the virgin compression curve up to where it meets the bisector line obtained in step (iv). The point of intersection of these two lines is the most probable preconsolidation stress (point B of Figure 1). The maximum possible preconsolidation stress is shown as point D, while E represents the minimum possible value of the preconsolidation stress.

TABLE I: LOCATION OF BOREHOLES

Coordinate	Borehole					
	BH1	**BH2**	**BH3**	**BH4**	**BH5**	**BH6**
m E	274206	284829	285920	287974	288749	314080
m N	976278	1044052	1048996	1063179	1066811	1136448

3 RESULTS AND DISCUSSION

Figures 2 to 7 show variations of preconsolidation stress with depth for soils at BH I to BH 6. From the figures, it is observed that for BH 1 (Figure 2), within the studied overburden, maximum preconsolidation stress of 120 kN/m2 occurred at 1.5m depth. This stress was also experienced by soil at 3.0m depth. Soil at 4.5m depth, in this borehole, has 100 kN/m2 as preconsolidation pressure. For BH 2 (Figure 3), soil at 1.5m depth has preconsolidation pressure of 85 kN/m2. At 3.0 and 6.0m depth, the preconsolidation pressure reduces to 65 kN/m2 and 52 kN/m2 respectively. At 7.5m depth, the value increased to 220 kN/m2.

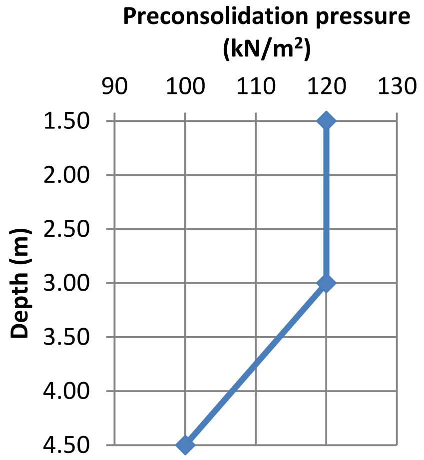

FIGURE 2: VARIATION OF PRECONSOLIDATION PRESSURE WITH DEPTH AT BH 1

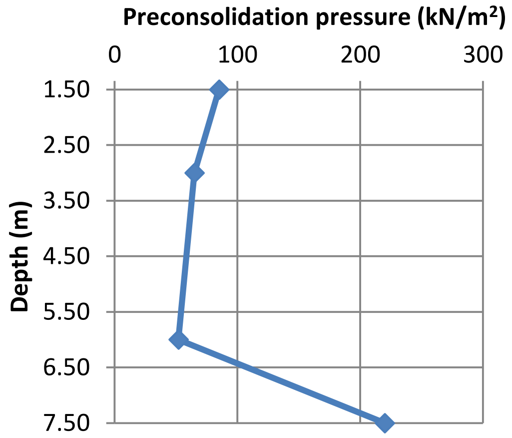

FIGURE 3: VARIATION OF PRECONSOLIDATION PRESSURE WITH DEPTH AT BH 2

Also from the results, it is observed that for BH 3 (Figure 4), within the studied overburden, minimum preconsolidation stress of 85 kN/m2 occurred at 1.5m depth. This stress increased to 130 and 170 kN/m2 at 3.0m and 4.5m respectively. The results from this borehole indicate gradual increase in preconsolidation pressure as depth of the soil increases. For BH 4 (Figure 5), the pattern of variation, observed from BH 1, where maximum preconsolidation stress was observed at the top layer, and increasing with depth, is also observed in this borehole.

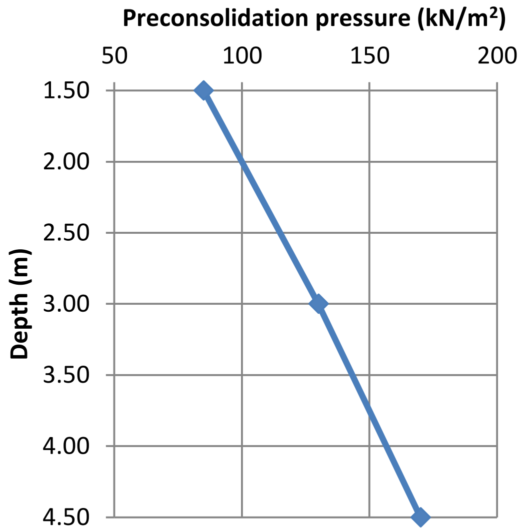

FIGURE 4: VARIATION OF PRECONSOLIDATION PRESSURE WITH DEPTH AT BH 3

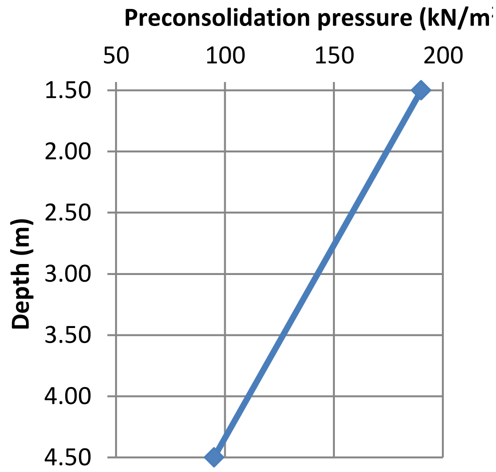

FIGURE 5: VARIATION OF PRECONSOLIDATION PRESSURE WITH DEPTH AT BH 4

Result from BH 5 (Figure 6) shows preconsolidation pressure varying from 102 to 150 kN/m^2 and 120 to 180 kN/m^2 at 1.5 to 3.0m and 6.0 to 9.0m depth respectively. For BH 6 (Figure 7), soil at 1.5m depth has preconsolidation stress of 180 kN/m^2, which reduces to 80 kN/m^2 at 3.0m depth. The preconsolidation pressure increased to 102 kN/m^2 and 104 kN/m^{2}at 4.5 and 7.5m depth.

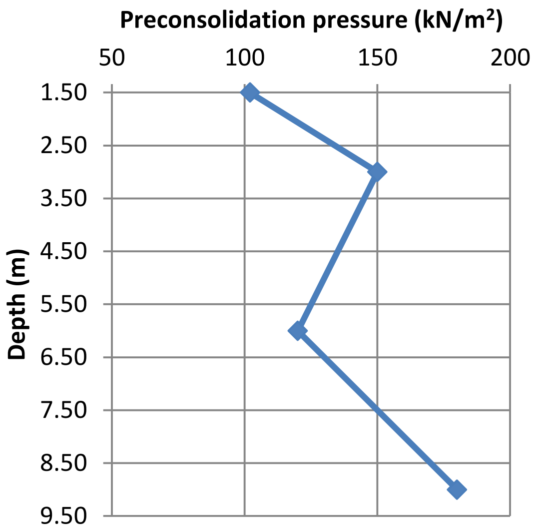

FIGURE 6: VARIATION OF PRECONSOLIDATION PRESSURE WITH DEPTH AT BH 5

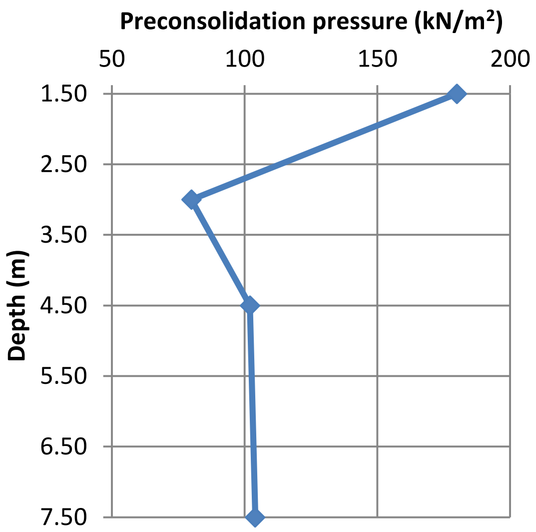

FIGURE 7: VARIATION OF PRECONSOLIDATION PRESSURE WITH DEPTH AT BH 6

The trend in variation of the preconsolidation stress with depth, observed in this studied, relatively conformed with study carried out by Alhajiand Alhassan (2013) on overconsolidation ratio of some selected soil deposits in Nigeria. Adebisi and Adeyemi (2012), in their study on assessment of compressibility characteristics of residual laterized soils in southwestern Nigeria, also reported variability of compressibility characteristics with depth. Carreón-Freyre*et al.*, (2015), in reporting analysis of the variation of the compressibility index of volcanic clays and its application to estimate subsidence in lacustrine areas, presented reported curves of void ratio (e) vs logarithm of vertical effective stress, which evidently indicated similar variation of preconsolidation pressure with depth.Józsa (2013) and Sohail*et al.*, (2012) also reported similar trend with overconsolidation ratio.

4 CONCLUSION

Preconsolidation stress of residual overburden soil in North-central Nigeria was studied with depth. The results indicate variability of maximum stress the undisturbed deposit has ever experienced in its geologic past. While at some location, the preconsolidation pressure increased with depth, at some other point, the pressure decreased with depth of the residual soil. The observed trend in variability of the preconsolidation pressure conformed with other studies carried out on similar residual from other regions.

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APPLICATION OF VALUE MANAGEMENT TO ENHANCE CONSTRUCTION OF RESIDENTIAL HOUSING FOR FEDERAL CIVIL SERVANTS IN NIGERIA

***Umesi, R. O.[1];& Ganiyu, B. O[2]**
[1] Department of Quantity Surveying Federal Polytechnic, Bida
[2]Department of Quantity Surveying, Federal University of technology, Minna
*Corresponding author email:ruthomozokpia@gmail.com

ABSTRACT
Value management techniques have been successfully applied to all types of construction projects from buildings to offshore oil and gas platforms, and for all types of clients from private industry to governmental organizations worldwide. One of the Sustainable Development Goals objectives is increasing access to new technologies to support sustainable development; this has led to the application of value management to residential housing to ensure sustainable development of affordable housing. In Nigeria, access to affordable housing has largely remained an unfulfilled dream to the vast majority most especially, the middle and the lower class of the society. The gap between the need for housing and the capacity to acquire the desired housing type has led to a demand crisis for affordable housing in Nigeria. In this paper, the concept and benefits of the application of Value Management was explored to enhance affordability of sustainable residential housing for civil servants in Nigeria. The quantitative approach employed was to understand the perception on the benefit of the application of value management on residential project in Niger state. This was done by the use of well structured questionnaire. A great number of innovative ideas are brainstormed during the Value Management process.

Keywords: *Affordable housing; construction; sustainability; sustainable residential housing; value management.*

1 INTRODUCTION

Accommodation is one of the basic need of man and has no doubt a widespread impact on the health, welfare and productivity of the individual (Akintunde, 2008; Akinyode and Tareef, 2014). Adapted and affordable housing arrangement has over the years been the necessity of most countries, especially the developing ones, given that it is one of the three most notable basic necessity of mankind –others been food and clothing. Accommodation (Shelter) is basically one of the requirements of man. It is classified second after food in the hierarchy of man's needs but as emphasized by Ebie (2009) it is the first and most expressive of all rights. As such, the supply of housing along with the fact that dwelling involves more than mere shelter since it encompasses all social services and utilities that make a society or neighbourhood a livable environment, is now a right.

Precedent Nigerian government have made invested efforts in housing delivery through various policies and scheme either as a provider in the 70's and 80's, and as facilitator and enabler in recent time (Aminu and Rukazat, 2013). Public-Private Partnership in housing provisions was commended as a means of addressing this problem. An investigation carried out on thirteen government agencies in some selected zones in Nigeria shows that although the agencies aim is to focus on the availability of access to land and the regulatory structure for housing growth, a great multitude of Nigerians have not benefited from Public-Private Partnership arrangement (Eziyi, 2010). More exertion is required to deal with this uncertainty.

The quality and quantity of useful dwelling unit in any country is also a well acknowledge indices of a country's level of growth and quality of life. Nations, therefore, pay special attention to the availability of affordable housing, for its residents. Statistics signify that investment in shelter accounts for 15 –35% of total investment worldwide compared to only 0.4% in Nigeria. Furthermore, shelter represents 15 – 40% of monthly expenditure of families globally. The present government in effect keyed in to the earlier "visions" that is vision 2010 and then inaugurated the vision 20:2020. The vision is designed to place Nigeria among the largest economies in the world by the year 2020. This means that in Africa, Nigeria must rise from our current 3rd position with a GDP of $294.8b to surpass

Egypt with a GDP of $432.9b and South Africa ($467.6b), within the same period (www.nigerianstat.gov.ng). There has been tremendous growth in the nations GDP over the last five years. The nation's GDP grew in the third quarter of 2017 by 1.4%. This growth is 3.74% rate higher in the third quarter of 2016. Presently, Nigeria is singled as the most developed country in Africa. Services in the largest distinctive part of the economy, accounts for 50% of total GDP. Information and telecommunication which together attribute for 10% of the total output is one of the fastest growing segments in services. Agriculture, former biggest sector now accounts for 23%, Industry and Construction accounts for 16% of GDP (www.tradingeconomics.com).

The democratic administration, which began in 1999, brought revived vigour and opportunities in the housing sector. More financing options opened for local and international investors coupled with the influx of overseas construction firms, who are flocking in to benefit from this enormous opportunities existing in this sector. This has resulted in various state governments presently executing one housing project or the other either directly through their state housing corporations or in partnership with the private sector. For example, the Federal Mortgage Bank of Nigeria (FMBN) has invested about =N=1.423 billion Naira on 402 housing units in Niger state. The housing unit are FMBN/Sutas Estate in Zuba with 92 units, FMBN/Jedo ministerial Pilot housing scheme Estate in Suleja with 102 units, FMBN/Niima Shelter limited with 75 units and the FMBN/Sea mountain Estate in Minna with 133units (leadershipngr, july 19,2017).

In spite of all these attempts, it is regrettable to note that the federal civil servants are yet to be provided for because the available houses are not affordable. As a result there is severe reliance on rental housing which in itself is grossly insufficient both in qualitative and quantitative terms leading to enormous rents. Currently, the average worker spends as much as 40% – 50% of his allowance on rental. As a result very little is therefore saved at the end of the month. This result is the inability of the low-income earners in particular to benefit from the various housing scheme. It is compelling to identify initiatives that can exclude the extra cost of construction (hidden- costs that do not contribute to value) for the future visibility of affordable residential housing units. To eliminate the extra cost element caused by this aspect of inefficiency, a cost model must aim at improving the peculiarity of the decisions made throughout the life cycle of the construction projects. Hence, it is consequential to focus on the value of the construction project throughout the project duration. This can be achieved through value management as shown by the definition of Institute of Civil Engineers (1996):"Value Management addresses the value process during concept, definition, implementation and operation phases of a project.

Whilst there have been many studies on housing needs, demands and supply, housing delivery, housing policies and programmes in Nigeria, less attention has been given to affordability of housing (Akintunde, 2008; Ebie, 2009; Amao and Ilesanmi, 2013; Aminu and Ruhizal, 2013; Akinyode and Tareef, 2014). Recent literature tend to focus on effective housing policy and sustainable development and challenges to housing development and delivery (Omoniyi and Jiboye, 2011; Celestine and Fidelis, 2013). The use of value management to enhance affordability of residential housing in Nigeria has not been studied in any detail.

AFFORDABLE AND SUSTAINABLE HOUSING

Housing, literally is defined as buildings or other shelters in which people stay, a living, and to Countries an important element in social and economic structure. Housing represents one of the most basic human desires. To most groups housing means shelter however to others it means extra because it serves as one of the nice indicators of a person's standard of dwelling and her place in the society (Nubi, 2008). It's far a concern for the attainment of living preferred and it is important to both rural and urban areas. These attribute make demand for housing to recognize no certain as population boom and urbanization are booming very swiftly and the gap between housing want and supply becomes widen. Cultural elements together with choices and values or social repute, taste and financial resources, additionally have an impact on a house physical characteristics.

Nigeria is perhaps the fastest urbanizing country in the African continent. One of the maximum crucial challenges facing the country is the supply of low cost housing. As more Nigerians make towns and cities their homes, the resulting social, economic, environmental and political challenges need to be urgently addressed (Raji, 2008).

Low priced housing is housing that is reasonably good enough in standard and location for a lower or middle-income family and does not cost so much that this family is unlikely to meet other simple dwelling costs expenses on a sustainable basis (National Summit on Housing Affordability, 2006). Stone (2005) notes that affordability is not a function of housing per se, rather, it is a relationship between housing and people that relies on answering three questions: Low price to whom? On what standard of affordability? For how long?

Sparks (2007) defines green low cost housing as "housing that is better designed and constructed, more long lasting, not significantly more expensive, less expensive to operate, healthier, more environmentally sound, and less risky" (Sparks 2007 in Arman et al., 2009). Global Green USA (2007) also talks about green low cost housing and adds that such housing "forges a sturdy link between social justice and environmental sustainability, and connects the wellbeing of people with

the wellbeing of the environment, thus building on the core social and monetary values of affordable housing".
There are many economic and social determinants of affordability (including costs of running a home and associated cost of maintenance), the most widely used measure in Australia (use by, for example, Australian Government, 2008; Beer et al., 2007; Berry et al., 2004; Disney, 2007; Gurran et al., 2008; Yates et al., 2007; Yates et al., 2008) is the '30/40 split' which indicates that housing costs have to no longer exceed 30% of household income for the bottom 40% of income groups. Knowing average incomes, it's far then possible to calculate a low cost house cost in terms of purchase price and rent and such figures also determine eligibility for certain low cost housing schemes
Arman et al. (2009) reviewed a spread of definitions of affordability, low cost housing, sustainability and sustainable housing and arrived at a conceptual definition of inexpensive and sustainable housing, housing that meets the needs and demands of the present generation without compromising the capability of future generations to meet their housing desires and demands. Inexpensive and sustainable housing has strong and inter-related economic, social and environmental components (Arman et al., 2009). They counseled that unique standard may be required to ensure that affordability and sustainability in housing are sincerely realized. To this end, Arman et al (2009) arrived at ten 'traits' of cheap and sustainable housing. These broad traits sought to mirror literature on affordability (traits 1-4) economic sustainability (traits 5), social sustainability (traits 6) and environmental sustainability (traits 7).

1.1 FEATURES OF SUSTAINABLE AFFORDABLE DWELLING.

	Dwelling Features	Source
1	Adequate in standard and location and does not cost so much	National Summit on Housing Affordability, (2006)
2	A product where the rent or mortgage repayments do not exceed 30% of household incomes for the bottom 40% of income groups.	Beer et al, 2007;Gurran et al, 2008., Yates et al., 2008
3	A product that is of a suitable size and quality for its occupants.	Stone(2005)
4	A product that does not increase the incidence of housing stress over the lifecycle of the house.	Sparks (2007) in Arman et al., (2009)
5	Meets the need and demands of the present generation without compromising the ability of future generations on affordability and sustainability.	Arman et al (2009)
6	A product that is socially acceptable and does not increase social exclusion or polarization.	Global Green USA (2007)
7	A product that encompasses the following environmental features; Energy efficiency; Passive solar design; sun shading; water conservation, appropriate waste management during construction, occupation and deconstruction.	Arman et al. (2009)

Source: author summary from Literature review

2 BENEFITS OF VALUE MANAGEMENT

Over the past few decades, the economy has modified hastily and increasing competition has positioned an significant call on increased efficiency, effectiveness and value for money (Rangelova and Traykova, 2014). Value Management addresses these three facets sucessfully and directly. The Institute of Value Management (2008) and The department of Housing and Works (2005) additionally observed that apart from acting as a cost reduction tool, the most glaring benefits arising out of the application of Value Management encompass: higher business decisions by providing decision makers a legitimate basis for their desire; enhanced competitiveness via facilitating technical and organizational innovation; a common value culture, thus enhancing every member's understanding of the organization's dream; improved products and services to external customers by clearly understanding, and giving due priority to their real desires; improved internal communication as well as common knowledge of the main success factors for the organization; simultaneously enhanced communication and efficiency through developing multidisciplinary and multitask teamwork; decisions which can be supported by the stakeholders; time savings through focus of attempt; aid to the briefing and approvals process; enhancement of danger control measures; improved quality; improved sustainability; and promotion of modern service delivery techniques. These benefits according to Oke and Ogunsemi, (2011) are available and update to providers and consumers in all sectors of the society.
Following the Society of American Value Engineers (2008) definition of Value Management being a systematic, multi-disciplinary attempt directed towards analysing the functions of projects for the purpose of achieving the best value at the lowest overall life cycle cost. The premise is that some unnecessary costs are inevitable in any building design; Value Management sets out to identify and eliminate these unnecessary

costs, resulting in cost savings. Value Management should not be confused for cost control. Value Management focuses on value in relation to the function while cost control focus on cost of construction.

Noor, Kamruzzaman and Ghaffar (2015) observed that in Malaysia, Value Management has been diagnosed by the authorities as a strategic planning tool and it has been practiced ever since as an appropriate mechanism to deliver sustainable construction project. The application of Value Management during project development phase may be utilized to improve building sustainability. Therefore, the appropriate approach of sustainable development as a process will be to balance and integrate social, economic and environmental sustainable values in construction.

In area of production, Yekinni et. al (2015) found out that there is a conceptual synergy between Value Management and Sustainable product and service design that leads to achieving best value in terms of quality and cost of a product/service. Thus, Value Management can be said to be a reliable tool in providing sustainable products.

In practice, at various stages of a Value Management workshop, the Value Management team tries to analyze each characteristic and look for better alternatives. Certain questions are asked and this includes questions like: What an element is? What does it do? What else can it do? What does it cost? What is its value? When these questions are answered, several alternatives are drawn and the best alternative is developed. In doing this, the Value Manangement crew try to identify unnecessary cost which can be in; use of unnecessary materials which less expensive materials would have been able to replace and do the job satisfactorily or failure to identify opportunity cost. This being the case, since cost savings is one of the major objectives of sustainable development from the economic point of view (Agenda 21, 1992, as citied in Romiguer, 2011), and Value Management sets out to achieve “value for money”, it therefore follows that Value Management is the appropriate mechanism for selling the objective of an economic sustainable development. Hence Value Management plays a vital role in the delivery of economic sustainable construction.

According to research carried out by SAVE, Value Management methodology can increase customer satisfaction and add value to an organization's investment in any business or economic setting (www.value-eng.org). Value Management practitioners apply Value Management methodology to products and services in industries such as the following: corporations and manufacturing, construction, transportation, government, health care and environmental engineering. Similarly, from the studies carried out they found out that Value Management methodology easily produces financial savings of 30 % of the estimated cost for manufacturing a product, constructing a project or providing a service. The return on investment that public and private organizations derive from implementing Value Management programs averages 10 to 1. That is, for every dollar invested in a Value Management study, including participants' time and implementation costs, 10 dollars in net saving results.

The following are some of the results of Value Management application by some agencies

Benefits of Value Management highlighted by design consultants included (Come de Leeuw 2001): evidence that the initial design was indeed the best; the owner receives good value for money; an introduction of higher quality products; best up-to-date technology introduced at lowest cost; and a clear focus on project objectives as well as several alternatives for the design being considered.

2.1 BENEFITS OF VALUE MANAGEMENT

BENEFITS		SOURCES
1	Cost reduction tool	Yekinni et al (2015), IVM (2008), Romiguer (2011), SAVE (2008), DHW(2005)
2	It enables better business decisions based on choice	Come de Leeuw (2001), SAVE(2008), IVM (2008)
3	It enhances competitiveness based on technical and organizational innovation	Come de Leeuw (2001), IVM (2008)
4	A common value culture, every member in the team understand organizational goal	Come de Leeuw (2001), IVM (2008)
5	Improved products and service	Yekinni et al (2015), IVM (2008)
6	Improved internal communication	IVM (2008)
7	Strategic Planning tool	Noor, Kamuzzaman and Ghaffer (2015), IVM (2008)
8	Develops multidisciplinary and multitask teamwork	SAVE (2008), IVM(2008)
9	Time saving	IVM (2008)
10	Aid to the briefing and approvals process	Come de Leeuw (2001), IVM (2008)
11	Enhance risk management measure	IVM (2008)
12	Increased quality	Come de Leeuw (2001), IVM (2008), Yekinni et al (2015)
13	Improved sustainability	Noor, Kamuzzaman and

		Ghaffer (2015), IVM (2008), Romiguer (2011), Yekinni et al (2015)
14	Promote innovative service delivery process	Come de Leeuw (2001)

Source: author summary from Literature review

3 METHODOLOGY

One of the methods of survey employed in this study is the cluster and simple random sampling. This was done by sending out questionnaires to some professionals and stakeholders in the building industry in Niger state and the interview of key actors in the industry.

Analysis of data was done using both descriptive and inferential statistical methods. Descriptive statistic was carried out to reveal difference in demographic attributes of the respondents. A summary of the benefits of value Management for residential projects was analyzed. Benefits were categorized into Planning and Design Stage and Construction Stage. Respondent's opinions were ranked from the opinion that was very significant on to the one not sure about. Inferential statistic allows the use of samples of mean and standard deviation to make generalization about the population from which the sample were drawn.

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4 RESULTS AND DISCUSSION

TABLE 1: PLANNING AND DESIGN STAGE

BENEFITS OF VALUE MANAGEMENT	Mean	Standard Deviation
Better business decision based on choice	4.63	.554
A common value culture, every member in the team understand organizational goal	4.54	.691
Enhanced competitiveness based on technical and organizational innovation	4.43	.635
Promote innovative service	4.37	.699
Aid to the briefing and approval process	4.37	.895
Strategic planning tool	4.33	.702
Develops multidisciplinary and multitask teamwork	4.33	.731
Time saving	4.24	.875

Source: Researcher's fieldwork (2018)

The benefits of Value Management in Planning and Design Stage as shown in table 1.1 revealed that all eight benefits had a mean score above 4.0. This implies that the respondents strongly agreed the benefits are achieved through Value Management. The first benefit is that Value Management enables better business decision based on choice. This had a mean score of 4.63. This implies respondent strongly agreed to the finding of Come de Leeuw (2001) and the Institute of Value Management (2008). Value Management enables a common value culture and enhance competitiveness were the second and third benefits with a mean score of 4.54. These findings is in agreement with Come de Leeuw (2001), Rangelova and Traykoya (2014) and the Institute of Value Management (2008) findings. The least benefit is Time saving with a mean score of 4.24. All respondents agreed to the finding of the institute of Value Management.

TABLE 2: CONSTRUCTION STAGE

BENEFITS OF VALUE MANAGEMENT	Mean	Standard Deviation
Cost reduction tool	4.42	.862
Improved product and service	4.37	.539
Increased quality	4.35	.631
Increased sustainability	4.30	.732
Improved internal communication	4.24	.733
Enhance risk management measures	4.17	.850

Source: Researcher's fieldwork (2018)

The benefits of Value Management in the construction stage are revealed in table (1.2). All six benefits had a mean score above 4.0. The first benefit of Value Management in construction stage is Value Management is a cost reduction tool with a mean of 4.42. This indicates that respondent agreed to the findings of Yekinni et al (2015), Insitute of Value Management (2008), Romiguer (2011) and SAVE (2000) who believes a cost reduction tool for sustainable development is achieved by Value Management mechanism. SAVE(2000) discovered the return on investment that Public and Private organization derive from implementing Value Management programs. The second benefit of Value Management on construction stage is improved product and service with a mean score of 4.37. This supports the findings of Yekinni et al (2015) and Institute of Value Management (2008). Increased quality is the third benefits of Value Management in construction stage with a mean of 4.35. Come de Leeuw(2001) stated that Value management introduces higher quality products while Yekinni et al (2015) is of the opinion that there is a conceptual synergy between forth point, increased sustainability with a mean score of 4.30. This result is inline with the findings of Noor, Kamuzzamam and Ghaffer(2015) and Romiguer(2011). The fifth and sixth benefits are increased internal communication and enhance risk measures. There mean scores are 4.24 and 4.17. This findings support the findings of Institute of Value Management (2008).

5 CONCLUSION

Research on benefits of Value Management for Residential housing outlines numerous benefits both in the planning and design stage and also the construction stage. These will enable affordable, sustainable, innovative residential housing for civil servant. This research has outline define ways in which Value Management contributes to a successful delivery of economic sustainable construction. A great number of innovative ideas are usually brainstormed during the Value Management process. This enables affordability. There should be communication between all parties in the project, from the professionals down to the end user – civil servant so as to achieve an affordable residential building. Value Management should be encouraged.

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PERFORMANCE EVALUATION OF CONSORTIA ON BUILDING CONSTRUCTION PROJECTS IN LAGOS

***Olorundare, F. P, [1]; & Anifowose M. O, [2]**
[1&2] Department of Quantity Surveying Federal University of Technology, Minna
*Corresponding author email: olorundarefemi7@gmail.com

ABSTRACT
Construction industry is one of the key industries in every nation, being a key contributor to country's economy that supports economic growth and is an instrument to achieve economic goals. One of the main objectives of embarking on a building construction projects by any building owner is to get satisfaction at the completion of the project. This has resulted to the emergent of professional consortia in building construction to offer a joint professionalism. In Lagos the satisfaction level of client on building construction is still far to attain it goal. The gap between the need to improve client satisfaction on building projects is yet to be filled. Data were collected through questionnaire survey and analysed using descriptive analysis. Results shows that completing the project within the projected estimated cost, Client assessment on quality of materials and Employer's involvement during construction phase are the most effective strategies that improve client satisfaction on building construction projects. The study concluded that using the listed strategies by the consortia for a building project will improve client satisfaction. Recommended that the established strategies should be applied on building construction in order to improve client satisfaction.
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Keywords: Building projects, Consortia, client satisfaction, Lagos

1 INTRODUCTION

The construction industry is multifarious in its nature because it comprises large numbers of group as owners (clients), contractors, consultants, stakeholders, and regulators, despite this complexity, the industry plays a major role in the development and achievement of society's goals also It is one of the largest industries and contributes to about 10% of the total national product (GNP) in industrialized countries (Navon 2005). The construction industry is one of the most important industries in every country (Aziz and Abdel-Hakam, 2016), being a major contributor to country's economy (Alaghbari *et al*., 2007) that supports economic growth and is an instrument to achieve economic objectives. It ensures common benefits to all group involved by creating a collective situation, besides creating efficient teamwork. However, evaluation of the performance of partnering projects is still inconclusive. In construction industry today, the construction associating has become one of the major managerial forms utilized in important projects (Lin and Ho 2012). Due to the growing measure and complexity of construction projects, as well as technological innovations, groups have begun to set up associating to develop partner resources (Famakin *et al*. 2012; Zhao *et al.* 2012). Joint venture formation between construction companies has become one of the most commonly adopted methods in both developed and developing countries.
Popular building construction projects are those projects completed on time, within budget, in agreement with specifications and to shareholders' contentment (Chua, 2011). Research was conducted to examine factors impacting on project operation in developing countries. Shortage of skills and He further observed that the evaluation of performance has been a challenge for the construction industry for decades. The Architects and contractors expect more on profits while the client are more interested in completing their projects on time and on budget (Heywood and Smith, 2006; Meeampol and Ogunlana, 2006). However, the contractor may have roles; for example, extra costs are importance when overtime is required, to complete a project within the tendered time structure (Risner, 2010).
On this theme, agreeing to Memon *et al*. (2014), instability in the prices of materials owing to increase is the most vital factor that affects construction cost performance. Any mistake or deviation of information relayed from the client to the Architect team may cause revise and generate unnecessary costs and schedule overruns to construction projects (Lopez *et al*., 2010). Also, different viewpoints and know-how among several subcontractors require close communication and management in a construction project (Ye *et al*., 2014). It is common awareness that the execution of the construction project in the industry is usually go together with with time delay and cost increase as well as client dissatisfaction (Hafez, 2001). Majority of construction clients are attracted in the cost of execution of their projects as the most usual question asked are "what is the cost of the project?" and followed by "can there be a drop in cost?" (Cunningham, 2014).

2 STRATEGIES USE BY CONSORTIA TO IMPROVE PERFORMANCE IN ENSURING CLIENT SATISFACTION

According to Basu (2004), quality professionals have different observations in defining quality such as fitness for purpose, right first time, what the customer wants conformance to standard, value for money and right thing at the right time and others. Therefore, quality is fundamental to high-performing organizations and organizations should focus on the quality of goods or services. Moreover, the organizations should emphasize the quality concept in the management practices of the organization (Evans *et al*., 2008). Quality in the construction environment comprise with doing the job in time, achieved the requirement requirements and getting the job done within the budget given. The core factors involve with quality issues are the use of quality standard, administration commitment, communication, activities during project and planning and relationship between construction actors (Kandeil *et al* ., 2010). Quality and quality improvement have been receiving growing attention worldwide.

The need for doing quality of the final product in the construction environment is equal important to other industry. However, related to other nations, construction activities always related to discontinuous, dispersed, diverse and distinct. Therefore, the location of quality is more challenging to implement and improvement in quality is hard to reach (Albert *et al*., 2003). The project quality is the key to success and the quality level of a project reflects the level of technology and management. The quality is divided into step taking, survey, design, construction, getting and application of process (Jiang, 2010). Moreover, the quality should also take into consideration of the financiers and social responsibility so as to keep viable advantage.

Additionally, a major quality task in the construction environment is relating the principles on the job site. Construction bids much more patchiness; each project represents a distinctive formula of plan, place, personnel, materials, weather, cost, and time. Idoro (2010) specify that the excellence of the project in the construction atmosphere are power by usual of workmanship, charge by the user on the quality of construction materials, level of faulty works and upkeep running costs of the project. Furthermore, the security of every construction edifice and the satisfaction of the investors hinge on the construction value. In Malaysia construction environment, the actual benefits of having quality are to improve the functionality and achieve a certain level of user satisfactions. Among quality matters involved in the construction environment are plan management practices, financial management and project success (Din *et al*., 2011).

Quality task in Nigeria edifice atmosphere is related by those truly doing the work, offsite and on site events, project running, construction process, training and education, collaboration, supplier enterprise, policies and recognitions (Sodangi *et al* ., 2010: Wan Mahmood *et al*., 2006). Additionally, issues such as understandable and applicable plan, conformity of plan with the required requirement, finances of construction atmosphere, ease of setup and maintenance and energy productivity need to be measured in construction quality. The practices of quality in the construction atmosphere are also comprising the human resource management, supplier association, management assurance and information and analysis (Abdul Rahman *et al*, 2010). Che Ali *et al.* (2010) highlight that the main issues in construction quality are the participation of the contractors in the construction process, optimise resources to emerge the final constriction products, meeting the condition requirement and application of formal quality scheme. Quality task in Malaysian construction atmosphere is related by those actually doing the work, offsite and on site activities, project administration, construction exercise, training and education, collaboration, supplier corporation, policies and respects (Sodangi *et al*., 2010: Wan Mahmood et al., 2006). Guerrero De los Rios (2012) proposed a collective model to introduce learning skilled competence in project management for sustainable development.

The theoretical study of Mishra *et al* (2011) foretold that the ethics approach would end in sustainability of projects, as it would upturn satisfaction and client loyalty, form agreement, union, trust, values and ethics among the team associates. Using literature analysis and dialogue survey methods, Hwang and Ng (2013) examined the critical awareness and expertise of project managers in deliver green construction. They believed these abilities were necessary for project managers to respond to project-related, plan interconnected, client-related, project team related, labour-related and external tasks.

Based on studies on hand drive projects, Baraki and Brent (2013) revealed the reason for project miscarriage was a lack of organized and viable awareness sharing practices among consortium. They suggested that the process and upkeep, awareness management and project life phase management style were crucial in order to establish a workable organized support system through a public private enterprise. Pietrosemoli and Monroy (2013) evaluated the connection between awareness management and possible construction and their control to reach sustainable objectives. The list of critical success influence (CSF) can help the project leaders to measure project performance and outcomes and correctly assign project resources (Chua, 1999; Cox *et al*, 2003; Yu and Kwon, 2011).

2 RESEARCH METHOD

Some approaches use in the building construction projects to increase buyer were well-defined through a thorough literature evaluation. The strategies were tabulated into a questionnaire form. Then the draft questionnaire was discussed with three experts in construction industry to the questionnaire is divided into two main parts. Part I is related to general facts for the company. The surveyed consultant and contracting professional were bidden to answer questions pertaining to their experience in construction industry. Part II includes the list of the recognized strategies in building construction projects.

4 DATA COLLECTION AND ANALYSIS

120 professionals in consortia working on building construction projects were successfully questioned. The questionnaire gave each respondent a chance to identify variables that they perceived as likely to contribute to consortia performance by responding on a scale from 5 (very high) to 1 (very low). Participants then rated the frequency of occurrence for each variable on project that they have experiences on an ordinal scale: very high (5), high (4), moderate (3), low (2), or very low (1). The mean value from respondents, frequency rating, standard deviation. Frequency and descriptive analysis were identified using Table. Finally, some of the variables of strategies were identified using Figure.

5 TARGET POPULATIONS

The target population in this study was all professionals in the built environment practicing in both contracting and consulting organization as well in consortia.
The professionals include, Architect, Quantity Surveyor and Builders. These are the participant who engages in building construction and majorly form built environment consortia.

6 SAMPLE SIZE

The population of this research was 150 and the sample size used for this research was 120. The percentage of sample size to the population was 80% after which 150 questionaire were given out and 120 were retrieved from the respondent

7 DATA COLLECTION

7.1 DATA COLLECTION INSTRUMENTS

The data collection was by quantitative techniques in which the researcher used primary data that were collected through questionnaires. The questionnaires were given to the respondents by the researcher himself and retrieved after been filled by the respondents.

7.2 DATA PROCESSING AND ANALYSIS

The data for this study were analyzed quantitatively using percentages, frequencies and using linear. Statistical software called Statistical Package for Social Sciences was used to execute frequency and descriptive analysis. The results were presented using table's charts for ease of understanding. This allowed the researcher to interpret the findings and also generate recommendations from the findings.

8 RESULTS AND DISCUSSION

TABLE 1: POSITION OF THE RESPONDENT IN THE ORGANIZATION

Position	**Freq.**	**Perce**	**Val. Perc**	**Cum. Percent**
Junior Staff	17	14.0	14.2	14.2
Senior Staff	43	35.5	35.8	50.0
Management Staff	22	18.2	18.3	68.3
Principal/CEO	36	29.8	30.0	98.3
5	2	1.7	1.7	100.0
Total	120	99.2	100.0	
Missing	System	1	.8	

Source: Researcher's field work, 2018

The analysis of the questionnaire showed that, (14%) 17 out of 120 are junior staff, (35.5%) 43 out of 120 respondents were senior staff position in the company, (18.2%) 22 out of 120 were holding managerial position level and (29.8%) 36 out of 120 respondents on Principal/CEO level. This provides reliable data because these individuals are in the best position in the company to answer questionnaires as they are involved in everyday business activities of the company as far as procurement is concerned.

TABLE 2: YEARS OF EXPERIENCE OF THE RESPONDENT

Year		Frequency	Percent	Valid Percent	Cumulative Percent
Valid	0 - 5	32	26.4	26.7	26.7
	6-10	35	28.9	29.2	55.8
	11-15	20	16.5	16.7	72.5
	16-20	14	11.6	11.7	84.2
	21-25	14	11.6	11.7	95.8
	above 25	5	4.1	4.2	100.0
	Total	120	99.2	100.0	
Missing	System	1	.8		
Total		121	100.0		

Source: Researcher's field work, 2018

As far as study questionnaires were concerned, the student wanted to know the strength of manpower in terms of years of experience, which gives sustainability of performance, (29.2%) 35 out of 120 respondents on range of 6-10 years, (26.4%) 32 out of 120 on range of 0-5 years, (16.5%) 20 out of 120 on range of 11-15 years, (11.6%) 14 out of 120 on range 16-20 years, , (11.6%) 14 out of 120 on range 21-25 years and (4.1%) 5 out of 120 respondents are above 25 years. It shows maturity of manpower for willingness to take tasks and responsibility for benefit of the company.

TABLE 3: DESCRIPTIVE ANALYSIS OF THE STRATEGIES USED BY THE CONSORTIA FOR A BUILDING PROJECT TO MEET CLIENT SATISFACTION.

Strategies	N	Min	Max	Mean	Std. D
Completing the project within the projected estimated cost	120	2	5	3.27	.896
Client assessment on quality of materials	120	1	5	3.26	.815
Completing the project within the contract period	120	1	5	3.24	1.037
Adherent to quality target	120	1	5	3.22	.791
Written approvals promptly	120	2	5	3.13	.898
Sustainable knowledge sharing practice	120	1	5	3.13	.809
Involvement of competent professionals throughout the stages of project	120	1	5	3.12	.842
Applying ethnic approach	120	1	5	3.11	.868
Clear and thorough project brief	120	1	5	3.10	.834
Thorough detailing of design	120	1	5	3.08	.866
Value engineering at conceptual phase	120	1	5	3.06	.892
Comprehensive documentation of variation order	120	1	5	3.03	.777
Appointment Project manager from an independent firm to manage the project	119	1	5	3.03	1.045
Effective leadership	120	2	5	2.98	.855

Employer's involvement during construction phase	120	1	5	2.97	.840
Management commitment	119	1	5	2.96	1.077
Variation logic and justification	120	1	5	2.91	.953
Effective good communication	120	1	5	2.90	.854
Clarity of Variation Order procedures	119	1	5	2.90	.858
Constant training and education of the consortia	120	1	5	2.87	.916
Valid N (listwise)	119				

Source: Researcher's field work, 2018

The figure 3. shows the result of the descriptive analysis of the factors that determine the client satisfactions of the projects undertaken by the consortia in Lagos metropolis. It revealed that Completing the project within the projected estimated cost has 120 respondents that means all the participant actually rated it and it has total mean of 3.27 and standard deviation of .896, Client assessment on quality of materials also had 120 respondent that rated it and it has mean of 3.26 and standard deviation of .815, Completing the project within the contract period has total respondent of 120 and it has a mean of 3.24 with standard deviation of 1.037, Adherent to quality target has total respondent of 120, mean of 3.22 and standard deviation of .791, Written approvals promptly has a total respondents of 120, with mean of 3.13 and standard deviation of .898, Sustainable knowledge sharing practice has a mean value of 3.13 and standard deviation of .809, Involvement of competent professionals throughout the stages of project has a mean of 3.12 and standard deviation of .842, Applying ethnic approach has 3.11 and standard deviation of .868, Clear and thorough project brief has 3.10 and standard deviation of .834, Thorough detailing of design has 3.08 and standard deviation of .866, others has lower means and lower standard deviation.

TABLE 4: COMPLETING THE PROJECT WITHIN THE PROJECTED ESTIMATED COST

Rating		**Frequency**	**Per cent**	**Valid Per cent**	**Cumulative Percent**
Valid	Low	21	17.4	17.5	17.5
	Moderate	61	50.4	50.8	68.3
	High	23	19.0	19.2	87.5
	Very High	15	12.4	12.5	100.0
	Total	120	99.2	100.0	
Missing	System	1	.8		
Total		121	100.0		

Source: Researcher's field work, 2018

Table 4. shows that completing the project within the projected estimated cost has 50.4% at moderate level, shows that completing the project within the projected estimated cost has 19.0% at high level, shows that completing the project within the projected estimated cost has 17.4% at low level, shows that completing the project within the projected estimated cost has 12.4% at Very high level.

Completing the project within the projected estimated cost has 50.4% at moderate level which was ranked highest which indicate that is one of the strategies which influence the performance of consortia on a building project in order to satisfy the client which is align with Gunduz *et al* (2013) mentioned that if the project meets the time target, stays within the estimated cost, is in accordance with specifications, and achieves stakeholder satisfactions, it is regarded as a successful construction project.

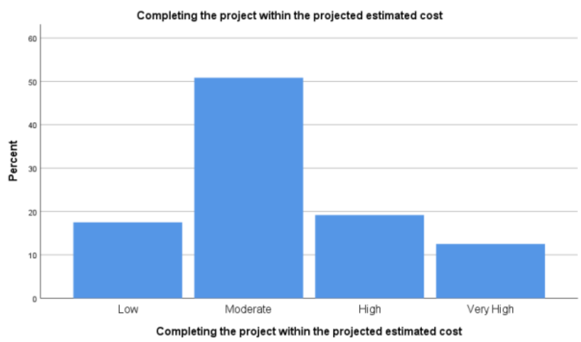

FIG.1. SHOWING PERCENT.

TABLE 5: CLIENT ASSESSMENT ON QUALITY OF MATERIALS

Rating		Frequency	Percent	Valid Percent	Cumulative Percent
Valid	V. Low	2	1.7	1.7	1.7
	Low	14	11.6	15.8	20.8
	Moderate	63	52.1	52.3	65.8
	High	33	27.3	27.5	93.3
	Very High	8	6.6	6.7	100.0
	Total	120	99.2	100.	

Source: Researcher's field work, 2018

Table 4.6 shows that Client assessment on quality of materials has 52.1% at moderate level, Client assessment on quality of materials 27.3% at high level, Client assessment on quality of materials has 11.6% at low level, Client assessment on quality of materials 6.6% at Very high level, Client assessment on quality of materials 1.7% at Very high level.

Client assessment on quality of materials has 52.1% at moderate level which was ranked highest which indicate that is one of the strategies that must be used to improve on performance of consortia on a building project in order to satisfy the client which is align . Idoro (2010) specify that the excellence of the project in the construction atmosphere are power by usual of workmanship, charge by the user on the quality of construction materials, level of faulty works and upkeep running costs of the project.

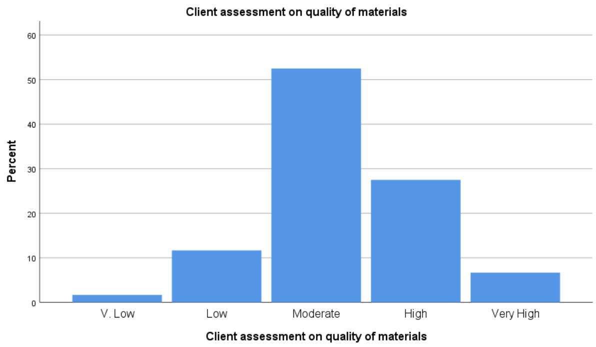

FIG.2. SHOWING PERCENT.

TABLE 6: VALUE ENGINEERING AT CONCEPTUAL PHASE

Rating		Frequency	Percent	Valid Percent	Cumulative Percent
Valid	V. Low	6	5.0	5.0	5.0
	Low	19	15.7	15.8	20.8
	Moderate	64	52.9	53.3	74.2
	High	24	19.8	20.0	94.2
	Very High	7	5.8	5.8	100.0
	Total	120	99.2	100.	

Source: Researcher's field work, 2018

Table 4.5 shows that Value engineering at conceptual phase has 52.9% at moderate level, Value engineering at conceptual phase 19.8% at high level, Value engineering at conceptual phase has 15.7% at low level, Value engineering at conceptual phase 5.8% at Very high level, Value engineering at conceptual phase 5.0% at Very high level.

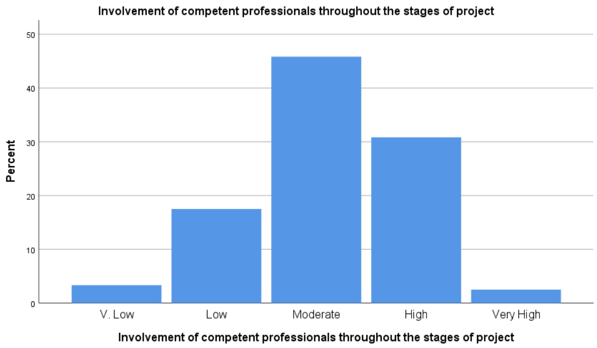

FIGURE 3. SHOWING PERCENT.

TABLE 7: APPLYING ETHNIC APPROACH

Rating		Frequency	Percent	Valid Percent	Cumulative Percent
Valid	V. Low	5	4.1	4.2	4.2
	Low	20	16.5	16.5	20.8
	Moderate	56	46.3	46.3	67.5
	High	35	28.9	28.9	96.7
	Very High	4	3.3	3.3	100.0
	Total	120	99.2	100.	
	Missing	1	.8		
	Total	121	100		

Source: Researcher's field work 2018,

Table 4.5 shows that applying ethnic approach has 46.3% at moderate level; applying ethnic approach 28.9% at high level, applying ethnic approach has 16.5% at low level, applying ethnic approach 4.1% at Very high level, applying ethnic approach 3.3% at Very high level.

applying ethnic approach has 46.3% at moderate level which was ranked highest which indicate that is one of the strategies that must be used to improve on performance of consortia on a building project in order to satisfy the client which is align with the theoretical study of Mishra *et al* (2011) foretold that the ethics approach would end in sustainability of projects, as it would upturn satisfaction and client loyalty, form agreement, union, trust, values and ethics among the team associates.

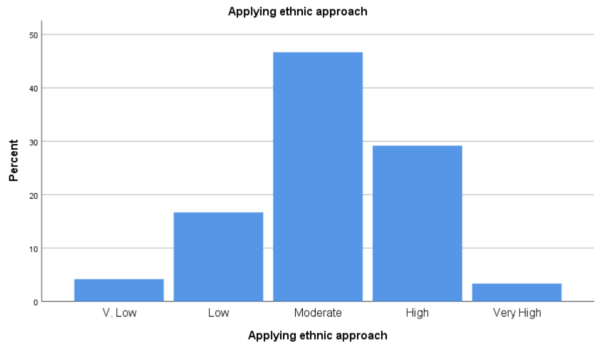

FIGURE 4. SHOWING PERCENT.

8 Conclusion and Recommendations

8.1 Conclusions

Effective use of performance strategies for building project by consortia at very great level will improve the level at which client will have confidence and accept the building project concede by the consortia and not only that but also would be satisfy with their products . The study concluded with an illustration using descriptive table and also with bar chat for clear and easy understanding of the importance of application of performance strategies using by the consortia.

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MODIFICATION OF BITUMEN WITH POLYETHYLENE VINYL-ACETATE FROM THE SOLE OF FLIP-FLOPS

* **Murana, A. A.[1]; Okpanachi, V. U.[2]; Laraiyetan, E.T.[3]**
[1,2]Department of Civil Engineering, Ahmadu Bello University Zaria, Kaduna state, Nigeria.
[3]Civil Engineering Department, School of Engineering, Kogi State Polytechnic, Lokoja, Kogi State, Nigeria
*Corresponding author email: fatinoyi2007@gmail.com

ABSTRACT
Polymer-modified bitumen is used to combat different pavement distresses and to increase the life span of pavement. Conventional bitumen cannot perform better within the range of extreme minimum and maximum pavement temperatures. The polymers commonly used to modify the bitumen are ethylene vinyl acetate (EVA) and styrene butadiene styrene (SBS). This research seeks to assess the effect of EVA from the soles of flip-flop as a modifier of bitumen. The flip-flops were obtained from waste bins, refuse dumps and where ever they were kept after usage. They were washed and milled then sieved through 300μm mesh sieve size. The FTIR chemical analysis performed on the flip-flop showed familiar transmittance peaks at various wavenumber (cm^{-1}) which confirmed the presence of EVA in the soles of flip-flop. The flip-flops were added to the bitumen at 2, 4, 6, 8, and 10 % by weight of bitumen by gradual mixing until temperatures between 160ºC – 165ºC were attained. The results showed that viscosity and softening point temperature increased while penetration, specific gravity and ductility drops with increasing percentage of flip-flop content. This shows that flip-flop improved the temperature susceptibility and rutting resistance of the bitumen. The FTIR comparative chemical analysis showed that the wave lengths of some certain transmission peaks remained constant for all percentages of the modified bitumen although the value of their respective transmittance was observed to have dropped for 2% modifier by weight of bitumen and increased from 2% up to 10%. Some peaks were also observed to have appeared while some disappeared. This confirms that a chemical interaction between the EVA and bitumen is responsible for the modification process. Statistical analysis (ANOVA) on pure and modified bitumen shows that the modifier has a significant effect on the bitumen.

Keywords: *Bitumen, Ethylene vinyl-acetate, flip-flops, FTIR analysis, Rutting and Temperature susceptibility*.

1 INTRODUCTION

Countries around the world face challenges to maintain their existing road networks at the time of increasing traffic volume, higher axle loads and increased tire pressure. Such country in the world like Malaysia in which most of the major road network, is paved with dense graded asphalt (Mustafa and Sufian, 1997).

The development of the Nigerian economy demands for sophisticated and technical satisfactory bituminous mixture or binder to reduce rutting at high temperature and thermal cracking at low temperature that mostly causes weather induced stresses on roads, airport runways and parking lots (Adegoke *et al* 1980, Adegoke *et al*, 1982).

The behaviour of bitumen to stress depends on temperature and loading time. At low temperatures, and short loading times, bitumen behave predominantly elastic. At high temperature and long loading times bitumen behaves like a liquid (viscous behaviour). For typical pavement temperatures and load conditions, bitumen generally exhibits both viscous and elastic properties. These properties of bitumen can be modified (improved) by adding modifiers to the bitumen (Akiije and Moyinwin, 2013).

Polymer modification of bitumen is the incorporation of polymers in bitumen by mechanical mixing or chemical reaction (Lu, 1997). Polymer modification is aimed at improving the stiffness and the elasticity of the asphalt bitumen at high pavement temperatures and reducing stiffness and elasticity of the bitumen at low temperatures by the incorporation of modifiers which include organic and inorganic materials such as styrene butadiene (SB), styrene-butadiene-styrene (SBS), ethylene-terpolymer, Ethylene-vinyl-acetate (EVA), Poly-ethylene (PE), Styrene-isoprene-styrene (SIS) and polyphosphoric acid (Baumgardner *et al*, 2005, Masson, 2008).

For the purpose of this research, poly-ethylene-vinyl-acetate (EVA) from waste soles of flip-flops, would be used as a polymer modifier of bitumen.

One of the greatest world challenges today is the disposal of industrial waste. For this reason, great efforts have been made by a number of developed countries to dispose industrial waste by means of putting them into domestic use or by using them as a modifier on the properties of bitumen in hot mix asphalt (Tariq, *et al*., 2014). From an environmental and economical point of view, we may envisage the possibility of disposing of troublesome waste plastics within road bitumen (Fawcett, *et al*, 1999). Incorporating recycled polymers may show similar performance to those which contain virgin polymers (Martinez-Boza *et al*, 2000).

Flip-flops, cheap and disposable, are the footwear of choice for the poor in developing countries and seaside tourists everywhere. “Flip-flops are one of the simplest products to produce. It’s a three-step casting process, versus a typical shoe, which is made in 100 steps When it comes to rubber flip flops, the cheaper they are, the faster they break. Consequently, millions upon millions of these shoes end up either discarded in landfills or worse. Each year, more and more flip-flops add to the mountainous tide of plastic waste polluting our oceans and killing the marine wildlife. (Watcher, 2009).

EVA is a polymer which results from the copolymerization of ethylene and vinyl acetate, is probably the thermoplastic copolymer that is most widely used to modify bitumen for roadworks. The properties of an EVA copolymer are controlled by its molecular weight and vinyl acetate content. The lower the molecular weight the lower the viscosity and, hence, the stiffness. The greater the vinyl acetate content the more ‘rubbery’, i.e. flexible, the material. (Nicholls and Evatech, 1994)

EVA has been used for various applications, including flip flops, shoe soles and midsoles, sports equipment and insulation because the material is highly durable, light, and very comfortable because of its high foaming capabilities (Reyes-Labarta, *et al*., 2006)

EVA (ethylene and vinyl Acetate) which is a thermoplastic crystalline polymer (plastomers) has been used for the modification of the bitumen binder from more than 30 years (Hameau and Druon, 1976).

The conventional penetration, softening point, Fraass breaking point, ductility and high temperature viscosity tests have demonstrated the increased stiffness (hardness) and improved temperature susceptibility of the EVA modified bitumen (Airey, 2002).

Panda and Mazumdar (1999) reported that the penetration, ductility, and specific gravity of the EVA modified binders decreases as compared with the neat bitumen while the softening point temperature and viscosity increases.

2 METHODOLOGY

The materials used for this study are; 1. Bitumen 60/70 penetration grade. 2. Modifier (soles of flip-flops). The bitumen was 60/70 penetration grade bitumen obtained from MOTHER CAT Construction Company Ltd, Zaria, Kaduna state, Nigeria. The modifier and waste material used consists of the commonly worn plastic flip-flop in the northern part of Nigeria, and was sourced from waste bins, refuse dumps and where ever they were kept after usage. The Flip-flops were manufactured by the Asian Plastic Industry of Nigeria (APIN) and the Asian Standard Plastics (ASP) whose predominant manufacturing component for plastic flip-flops is Ethylene Vinyl Acetate (EVA); both are located in Kano state.

2.1 METHODS

The methods employed in this study includes:

i. Sample preparation of the modifier (soles of flip-flops) and the modified bitumen;
ii. Laboratory tests conducted on modifier (soles of flip-flops) and the modified bitumen
iii. Statistical analysis of the modified bitumen using Analysis of Variance (ANOVA).

2.1.1 Sample preparation

2.2.1.1 *Modifier*

The soles of flip-flops (modifier) were obtained from waste dump sites in market areas, residential areas, and also from retailers within Zaria and its environs. These was washed thoroughly and dried for a period of 48 hours. Then, they were cut down to smaller sizes of approximately 1cm^2 in area with a very sharp scissors in-order to aid in easy feeding of the flip-flops to the milling machine. With the aid of the Thomas Wiley milling laboratory machine model 4, fitted with a sieve of mesh size 0.5mm, the flip-flops were further broken down into smaller sizes in order to increase contact surface area when mixed with the bitumen; after which they were sieved through ASTM sieve No. 50 (300μ).The flip-flop particles that passed through the 300μ sieve was used as the modifier. (Kaymanesh *et al*, 2017; Oyedepo and Oluwajana, 2014; Akinpelu *et al*, 2013).

2.2.1.2 *Modified Bitumen*

600g of bitumen was placed in five different metal containers and (2, 4, 6, 8, and 10%) of the modifier by wieght of the bitumen was then added and gradually heated with low heat with the aid of a seven liter camp gas cooker. The heat was supplied gradually untill a temperature of 160ºC – 165ºC was attained which is within the temperature range of hot mix asphalt production. The modifier was then added (with respect to each percentage (%) replacement by weight of bitumen to each container as stated above) gradually to the bitumen and stirred manually while maintaining the range of temperatures until homogenous mix was achieved.

2.2.1 Laboratory tests

Modifier

The following tests were conducted on the modifier, Fourier Transform Infra-Red spectrum (FTIR) analysis (ASTM-E168) and Specific gravity test (BS1377).

The Fourier Transform Infra-Red spectrum (FTIR) analysis (ASTM-E168)

The Infra-Red spectrum analysis is mainly used to determine the structures of molecules with the molecules' characteristic absorption of infrared radiation and can also be used to confirm the existence of a chemical interaction between two substances by observing the FTIR spectra before and after the combination. . The analysis was conducted using an Agilent Cary 630 FTIR Spectrometer. The wave length of the equipment is between 4000 – 650 cm-1.

Specific gravity test (BS1377 (1990))

The specific gravity test was conducted using a pycnometer and an ultra-precision weighing scale to nearest 0.001g of a material (milled flip-flop) with respect to water is a ratio of the mass of a given volume of that material at 25ºC to that of an equal volume of water at the same temperature.

Pure / modified Bitumen

The following consistency tests were conducted on the **pure** and the **modified bitumen** are as follows: Penetration test (ASTM D5, 2005); Softening point of bitumen (ASTM D36/D36M, 2009); Ductility test (ASTM D113, 2007); Viscosity test (ASTM D 4402. 2015); Specific gravity test (ASTM D2170, 2010). The Fourier Transform Infra-Red spectrum (FTIR) analysis was also conducted (ASTM-E168) (Ojeyemi *et al*, 2016). The flash and fire point of bitumen (AASHTO-48., 2004 and ASTM D92, 2005), and the Solubility test (AASHTO-T4403) were only conducted only on the pure bitumen (Panda and Mazumdar 2009, Kumar *et al*, 2013, Mohammed and Patil 2014, Kaymanesh *et al*, 2017, Ojeyemi *et al*, 2016).

Penetration test (ASTM D5, 2005)

The penetration test is used for determining the grades of bitumen. It measures the consistency or hardness of bitumen and determines the grade of the bitumen which translates to its stability and temperature susceptibility.

Ductility test (ASTM D113, 2007)

The ductility test was conducted using a Ductilometer. The ductility measures the distance a standard asphalt sample will stretch without breaking under a standard testing condition (5 cm/min at 25°C). The significance of the ductile property of bitumen is an indication of the ability of bitumen to form a thin ductile film around the aggregate and facilitate the binding action between aggregates.

Softening point of bitumen (ASTM D36/D36M., 2009)

The softening point measures the temperature at which a bitumen sample will no longer support the weight of 3.5g steel ball. Its significance is to determine the temperature at which the bitumen softens so as to mitigate bleeding and rutting action that may to occur at certain environmental temperatures at and above the softening point.

Viscosity test (ASTM D 4402. 2015)

The viscosity test is a consistency test done to determine the measure of the resistance of bitumen to flow at certain temperatures. For this test the temperature is $60+0.1^0$C, and the viscosity is measured as the time it takes 50ml of bitumen to flow through the orifice of a viscometer cup inserted in a water bath whose temperature is constantly maintained at $60+0.1^0$C.

Specific gravity test (ASTM D2170, 2010)

The specific gravity test was conducted using a pycnometer and an ultra-precision weighing scale to nearest 0.001g of a material (bitumen / modified bitumen) with respect to water is a ratio of the mass of a given volume of that material at 25ºC to that of an equal volume of water at the same temperature.

The Flash and fire point of bitumen (ASTM D92. 2018)

The flash point and fire point test was conducted using a Cleaveland cup apparatus. The **flash point** test of bitumen is done to determine the temperature at which its vapors will ignite instantaneously in the presence of an open flame; the temperature of the flash point is usually lower than the temperature at which the material will burn.

The Solubility test (AASHTO-T4403)

This test is carried out in other to determine the degree of purity of bitumen in relation to the possibility of contamination by foreign materials. The test is carried out by dissolving a known quantity of the material in a solvent (such as carbon-tetrachloride (CCL_4) or carbon-di-sulphide (CS_2)) and then filtering it through a Gooch crucible.

Analysis of Variance (ANOVA)

ANOVA is statistical analytical tool that uses the F-distribution to compare the means of two groups usually a dependent and independent variable at particular significant level (0.05), and determines if the independent variable has a significant effect on the dependent variable. This is established by determining if there is no significant difference between the means of the two groups as stated in the null hypothesis H_o ; if it happens that their means are equal and rejecting the null hypothesis by accepting the alternate hypothesis H_1 if the means of the two groups are unequal.

3 RESULTS AND DISCUSSION

3.1 MODIFIER

The Fourier Transform Infra-Red spectrum (FTIR) analysis (ASTM-E168)

The chemical formula of Ethylene vinyl-acetate (EVA) is $[C_2H_4]n$ $[C_4H_6O_2]m$, and its chemical structure is as illustrated in Figure 1.

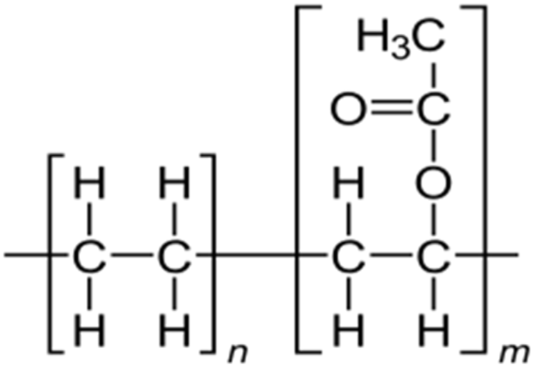

FIGURE 1: THE CHEMICAL STRUCTURE OF EVA

A correlation of the chemical structure of the EVA, the standard chart of Characteristic Infrared Absorption Bands of Functional Groups (CIABFG) and the FTIR test results of the modifier as shown in Fig. 2; was used to determine the presence of EVA in the modifier. From the CIABFG as shown in Table 1, and the FTIR test results, it can be observed that the very sharp peaks at 2847 cm^{-1} and 2914.8 cm^{-1} wave numbers indicate a strong presence of (C - H) group bond stretching of CH2 group and CH3 group from both the ethylene and vinyl acetate present in the EVA molecule. Also peaks observed at 1736.9 cm^{-1} and wave numbers are a strong indication of the carbonyl group bond stretching (C = O) from the vinyl-acetate in the EVA molecule. The peak observed at 1375.4 cm^{-1} is an indication of the presence of methyl (CH_3) group bending from the vinyl acetate in the EVA molecule. The peak observed at 1028.7 cm^{-1} is an indication of the (C – O – C) group bond twisting from vinyl-acetate in the EVA molecule. The peak observed at 1237.5 cm^{-1} is an indication of the presence of C-C (O)-C stretching in the acetate .The peak at 1461.1cm− and the one at 719.4 cm^{-1} observed at the finger print region is an indication of methylene (CH2) group bond bending group from both the ethylene and vinyl acetate present in the EVA molecule. From this analysis, it can be inferred that the EVA molecule is present in the modifier.

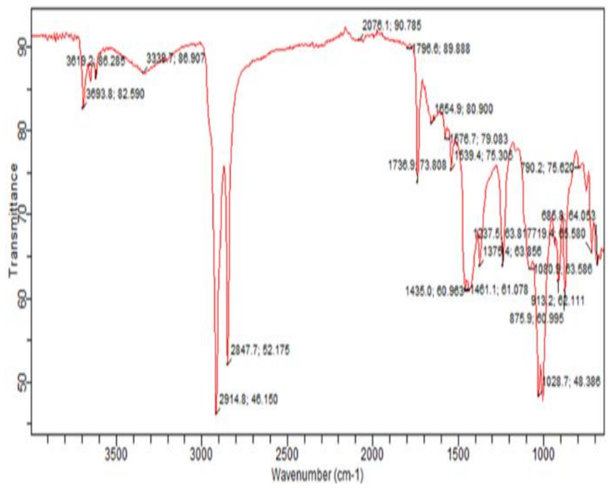

FIGURE 2: FTIR SPECTRUM OF THE MODIFIER (FLIP-FLOP CONTAINING EVA)

TABLE 1: CHARACTERISTIC INFRARED ABSORPTION BANDS OF FUNCTIONAL GROUPS (CIABFG)

S/N	Frequency range (cm^{-1})	Bond type	Bond name (Family name)	Molecular Motion
1	3000 - 2850	C – H	methylidyn (Alkanes)	Stretching
2	1745 - 1725	C = O	Carbonyl (esters)	Stretching
3	1150 - 911	C – O	Carbon monoxide (esters)	Twisting
4	3400 -3300	O – H	Hydroxyl	stretching
5	~ 1375	CH_3	Methyl (esters)	Bending
6	1260 - 1230	C- C O -C	NIL (Esters)	Stretching
7	~720 and ~1465	CH_2	Methylene (alkanes)	Bending
8	1640-1500	N-H	Imidogen (amines)	Bending
9	1570-1515	N-H	Imidogen (amides)	Bending
10	~880	C-H	Methylidyn (aromatics)	Bending (meta)
11	~910	C-H	Methylidyn (Alkenes)	Bending
12	785-540	C-Cl	Alkyl chloride (alkanes)	Stretching
13	~815	C-H	methylidyn (Alkenes)	Bending (trisubtituted)
14	1430-1290	C-H	methylidyn (Alkenes)	Bending (in-plane)

Specific gravity test (BS1377, 1990)

The result of specific gravity test conducted on the modifier (sole of flip-flop) according to BS1377 was determined to be 0.38 which signifies that the modifier material is approximately 2.6 times lighter than the weight of water.

3.2 PURE BITUMEN

The test results of the consistency and other general tests conducted on the pure bitumen are in line with the Nigerian General Specification for Road and Bridges (NGSRB, 2016), for 60/70 penetration grade bitumen as shown in Table 2.

TABLE 2: ONE-WAY ANOVA ON THE PROPERTIES OF PURE AND MODIFIED BITUMEN

S/N	Test Conducted	Unit	Result	Specification
1	Penetration	0.1mm	67.9	60 -70
2	Softening point	ºC	48.9	48-56
3	Ductility @ 25 ºC	cm	119	100 (Min)
4	Specific gravity	NIL	1.013	1.01-1.06
5	Flash-point	ºC	256	250 (Min)
6	Fire-point	ºC	221	NIL
7	Solubility in C_2S	%	100	99 (Min)
8	Viscosity @ 60 ºC	Secs	1544	NIL

3.3 MODIFIED BITUMEN

The results of the consistency and general tests conducted on the modified bitumen are as illustrated in Fig. 3, 4, 5, 6 and 7.

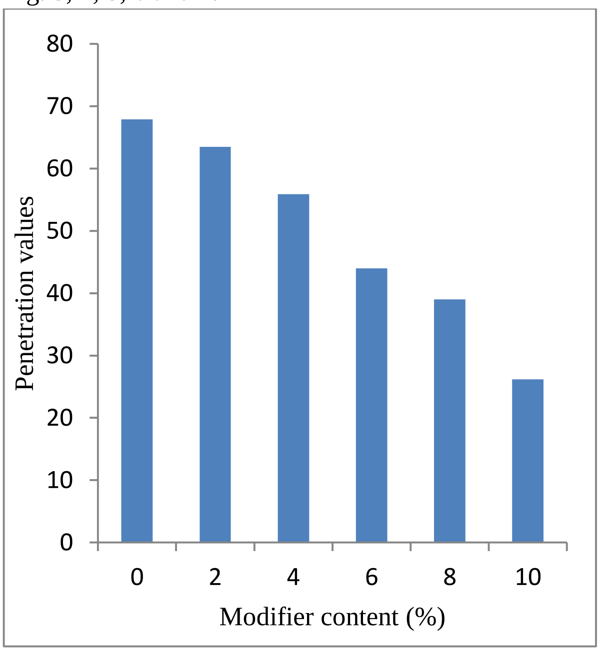

FIG. 3: A GRAPH OF PENETRATION AGAINST MODIFIER

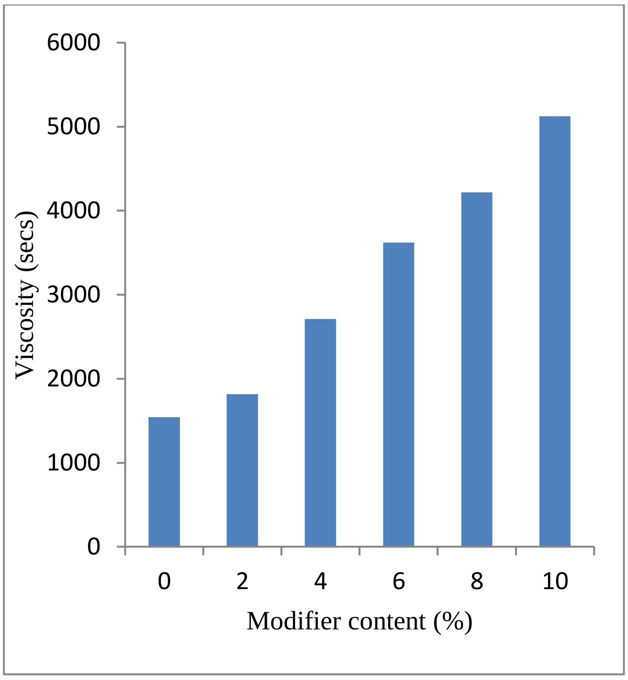

FIG. 4: A GRAPH OF VISCOSITY AGAINST MODIFIER

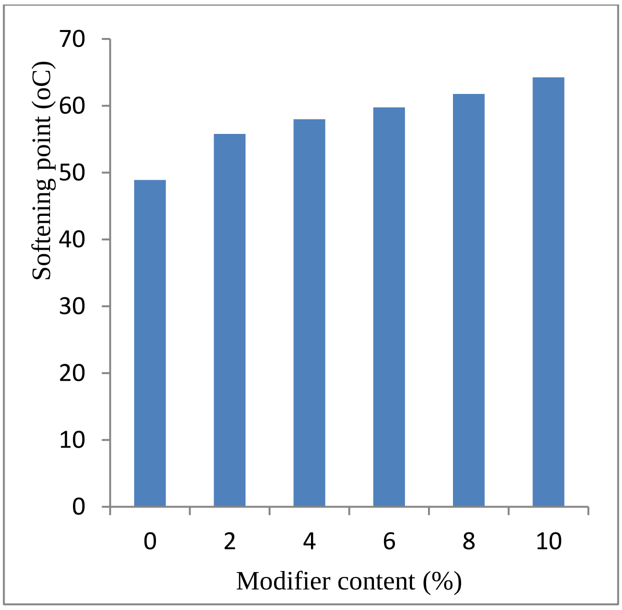

FIG. 5: A GRAPH OF SOFTENING POINT AGAINST MODIFIER

From the graphs illustrated in Figures 3 – 7, it can be seen that with an increase in the percentage of the modifier content, the viscosity, softening point tends to increase in value while the penetration and specific gravity and ductility tends to drop. This trend is in concordance with the virgin EVA modified bitumen as recorded by the works of Mohammed and Patil, 2014, Pander and Mazumder, 1997, Kumar *et al*, 2013.

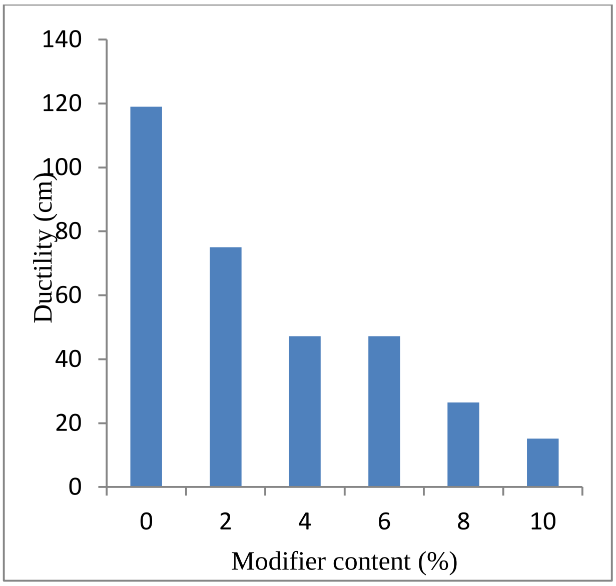

FIG. 6: A GRAPH OF DUCTILITY AGAINST MODIFIER

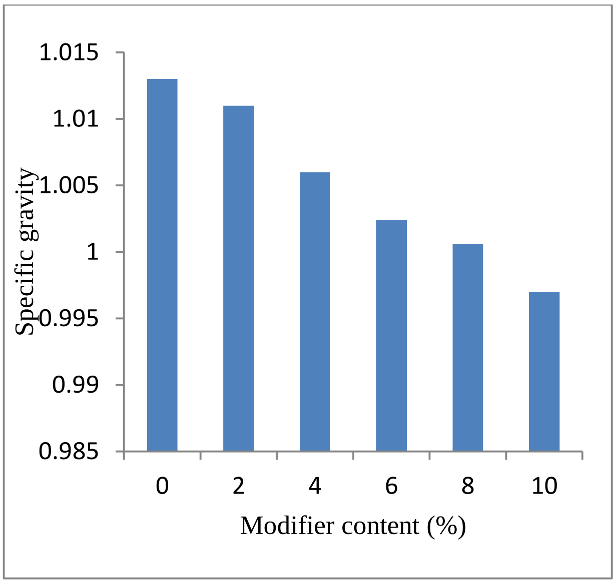

FIG. 7: A GRAPH OF SPECIFIC GRAVITY AGAINST MODIFIER CONTENT

The phenomenon responsible for this trend can be attributed to the fact that at temperatures above 160^{0}C polymers are in melt condition, they absorbs some oil and release low molecular weight fraction into the bitumen, which increases the viscosity of modified bitumen. By the end of mixing process, and by the time it cools a harden mixture is formed, which gives rise to lower penetration value (Noor *et al*, 2011). Airey (2002) also reported that this modification process of bitumen is achieved through the crystallization of rigid three-dimensional networks within the bitumen. Conventional penetration, softening point, ductility and high temperature viscosity tests have demonstrated the increased stiffness (hardness) and improved temperature susceptibility.

The Fourier Transform Infra-Red spectrum (FTIR) analysis (ASTM-E168)

The results of FTIR tests conducted on both the pure and modified bitumen are as illustrated in Fig. 8, 9, 10, 11, 12 and 13.

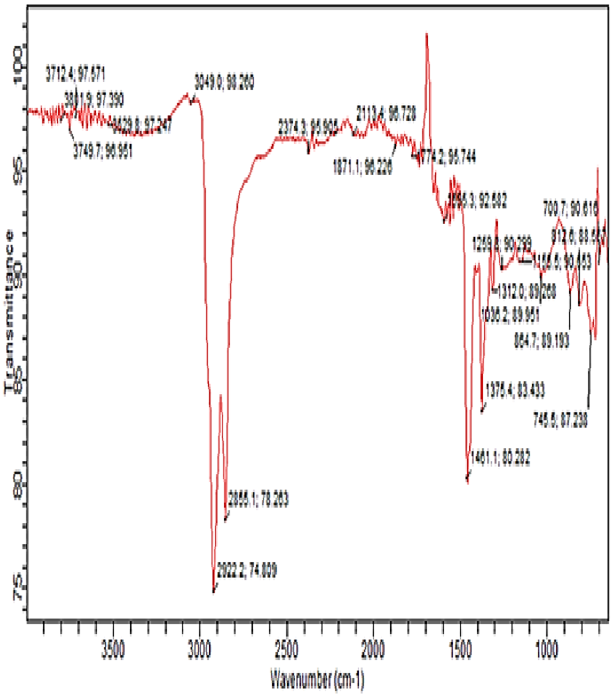

FIG. 8: FTIR SPECTRUM OF THE PURE BITUMEN (0% MODIFIER BY WEIGHT OF BITUMEN)

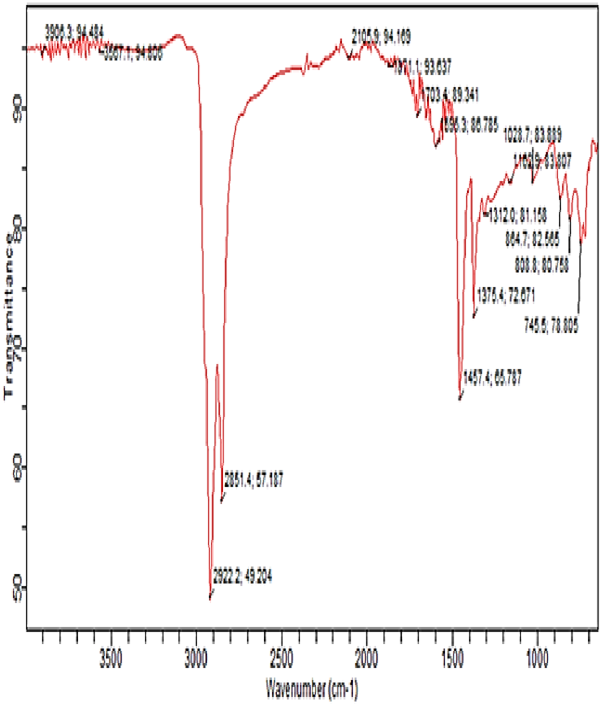

FIG. 9: FTIR SPECTRUM OF 2% MODIFIER BY WEIGHT OF PURE BITUMEN

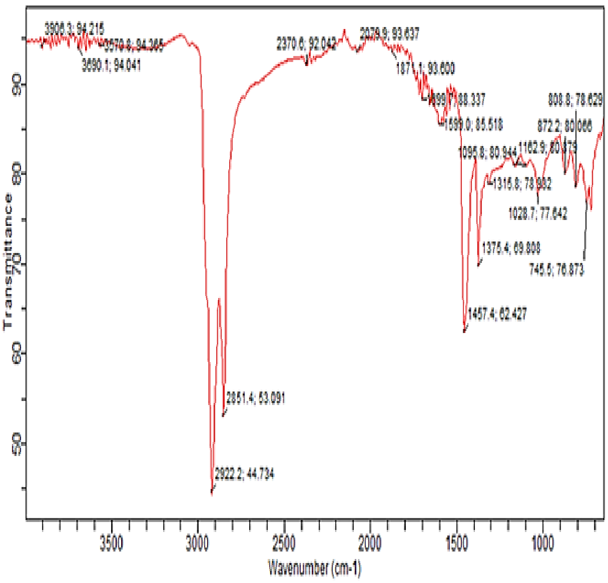

FIG. 10: FTIR SPECTRUM OF 4% MODIFIER BY WEIGHT OF PURE BITUMEN

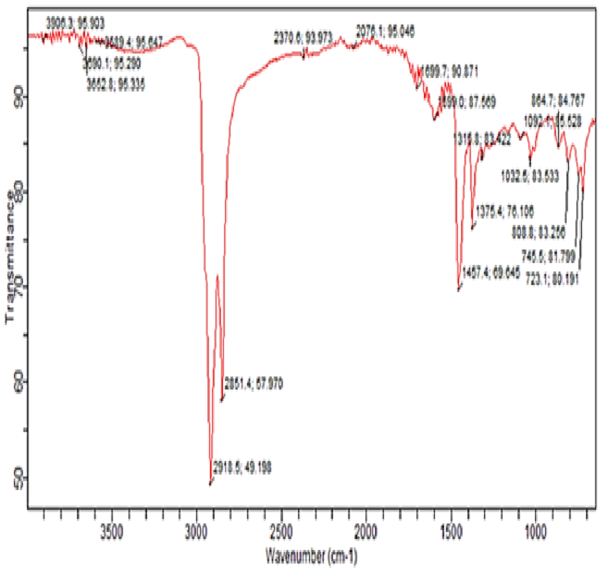

FIG. 11: FTIR SPECTRUM OF 6% MODIFIER BY WEIGHT OF PURE BITUMEN

From Fig. 8, it can be seen that FTIR spectrum of the pure Bitumen has absorption peaks at 2922 cm−1, 2855.1 cm−1, 1699 cm−1, 1595 cm−1, 1461 cm−1, 1375.4 cm−1 and 1036.2 cm−1 and peaks in the region 864.7 cm−1, 812.6 cm−1, 745.5 cm−1, and 700.7 cm−1 appear as shoulder. The infrared absorption peaks are similar to the submission of La Montagne *et al*, 2001 and Kamal *et al*, 2017, whom identified absorption bands related to pure bitumen to include 2922 cm^{-1} (υasCH2 CH3), 2862 cm^{-1} (υsCH2 CH3), 1601 cm-1(υ C═C), 1455 cm^{-1}, 1376 cm^{-1}, 1031 cm^{-1}(υ SO2), 868 cm^{-1},813 cm^{-1} (C═C), 747 cm^{-1}, and 722 cm^{-1}). Assignment of functional groups in the pure bitumen is based on some previous studies.

A comparative analysis done between the FTIR spectra of the pure bitumen and different percentages modifier added to bitumen by weight of pure bitumen as shown in Fig 9, 10, 11, 12 and 13, shows that the Infra-red spectrum of the pure bitumen is similar to that of the modified bitumen with Transmittance peaks at 745.5 cm^{-1}, 1375.4 cm^{-1}, 1312.0 cm^{-1}, 2922 cm^{-1}, 2855.1 cm^{-1}, 1871.1 cm^{-1}, 3049 cm^{-1} remaining constant for the pure bitumen and through all percent of the modified bitumen. Although it was observed that the value of their transmittance decreased at 2% modifier content but went on to maintain an increasing fashion from 4% up to 10% increase in modifier content. The disappearance of some peaks such as 700.7 cm−1,864.7 cm−1, 1259.8 cm−1, 2113.4 cm−1, 2374.3 cm−1, 3529.8 cm−1, 3712.4 cm−1, 3749.7 cm−1, 3801.9 cm−1 were also observed and appearance of some peaks such as 3906.3 cm−1, 3609.1 cm−1, 3570.8 cm−1, 2370.6 cm−1, 2027.9 cm−1, 2370.6 cm−1, were observed as a result of addition of the modifier to the bitumen. All peaks remain the same for all percentage addition of modifier to the bitumen. The only notable difference between various percentages of modifier addition to the pure bitumen is the value of the transmittance which assumed an increasing fashion as stated earlier. All these changes signify that there was indeed a chemical reaction that resulted from the interaction between the bitumen and the modifier; chiefly as a result of the chemical interaction between EVA in the modifier and the bitumen. This was as a result of microstructural changes related to the development of a polymer-rich phase that occurs in the bitumen as polymer concentration increased. These changes in microstructure have a significant influence on the flow behaviour of the binder (Morales and Partal, 2004).

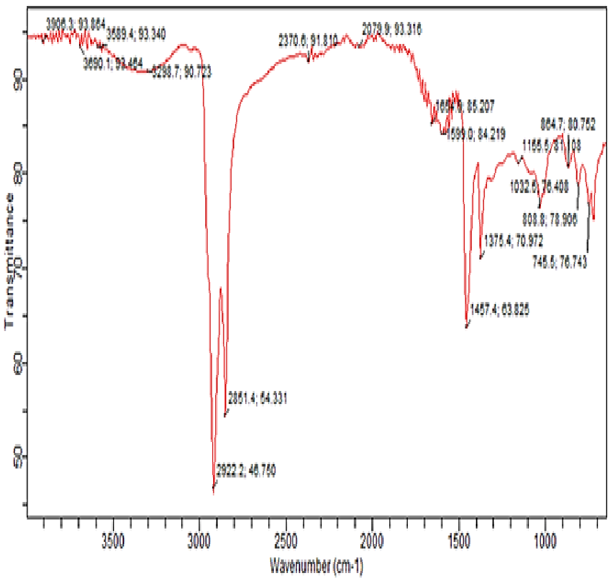

FIG. 12: FTIR SPECTRUM OF 8% MODIFIER BY WEIGHT OF PURE BITUMEN

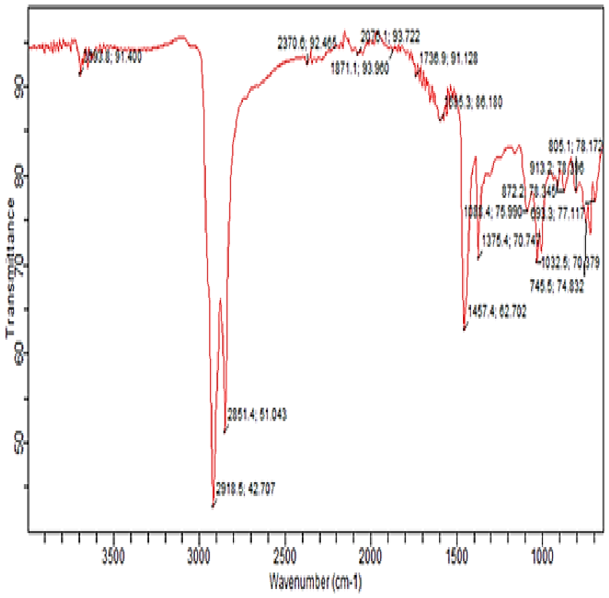

FIG. 13: FTIR SPECTRUM OF 10% MODIFIER BY WEIGHT OF PURE BITUMEN

STATISTICAL ANALYSIS (ANOVA)

The results of the ANOVA are as presented in Table 2. The result of the statistical analysis carried out on the result of each of the consistency test of the pure and modified bitumen at a 95% confidence level and 5% significance level shows that the value of Fcalculated is greater than Fcritical. Therefore there is a significant effect of the modifier on the bitumen with respect to all the consistency tests carried. It can also be observed from difference between the Fcalculated and the Fcritical of all the consistency tests carried out on the modified bitumen, the measure of significant effect of the modifier is highest for the softening point test and lowest for the specific gravity test.

TABLE 2: RESULTS OF THE ANOVA

Modifier Content (%)	Penetration (mm)	Ductility (cm)	Viscosity (secs)	Softening Point (ºC)	Specific Gravity
0	67.9	119	1544.0	48.9	1.0130
2	63.5	75	1817.0	55.8	1.0110
4	55.9	65	2712.7	58	1.0600
6	44	47.2	3621.3	59.8	1.0024
8	39	26.5	4218.3	61.8	1.0006
10	26.2	15.2	5124.0	64.3	0.9970
Fcalculated	44.53805	11.91234	30.707242	6.839992	393.5072
Fcritical	4.96403	4.96403	4.96403	4.96403	4.96403
P value	5.54×10^{-5}	6.211×10^{-3}	2.247×10^{-4}	2.247×10^{-4}	2.32×10^{-9}
Comment	Significant effect	Significant effect	Significant effect	Significant effect	Significant effect

4 CONCLUSION

1. The FTIR spectra analysis carried out on the sole of flip-flops (modifier) confirmed the presence of EVA in the flip-flop.
2. The use of EVA from the sole of flip-flops (thermoset material) showed similar trend as the virgin polymer (thermoplastic material) when used as a modifier of bitumen which is an indication of improved rutting resistance and temperature susceptibility.
3. The results of the FITR of the pure and modified bitumen show that the modification process was as a result of chemical interaction between the pure bitumen nd the EVA in the flip-flops.
4. The results of the statistical analysis (ANOVA) show that the modifier had a significant effect on the bitumen.

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Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

DESIGN AND IMPLEMENTATION OF AN INTEGRATED ELECTRONIC SECURITY SYSTEM

***Abubakar, I. N.[1]; Yusuf, H. M.[2]; Ambafi, J. G.[3]; &Ajagun, A. S.[4]**
[1,2,3,4] Department of Electrical and Electronics Engineering, Federal University of Technology, Minna
*Corresponding author email: is.abubakar@futminna.edu.ng

ABSTRACT
Insecurity can lead to loss of lives and properties in our society. Consequently, the microcontroller deployment to improve security and control took centre stage when threats to engineering equipment such as machines, telecommunication devices, electronic devices and ammunition warehouses became imminent.This paper deals with design analysis and implementation of an integrated electronic security system to aid in securing infrastructures. The objectives are: to optimise a security lock with minimal cost when compared with mechanical locks, design and implement a locking mechanism which gives genuine users the privilege to have access to their premises and also provides an avenue for password changing whenever an admin password is providedusing a microcontroller (ATMEGA 16). In achieving these objectives,the design was categorised into the software and the hardware section. Atmel Studio 5 and Proteus ISIS software were used for coding (C-Language) and simulation for the software section whereas; the hardware section comprises of a keypad, relay, buzzer and the microcontroller. The control unit either activate the pin in which the relay is connected to grant access to the user or activates the pin to which the buzzer is connected when the wrong password is used. The system is designed to activate the relay when the correct password is inserted; however, it activates the buzzer when the wrong pin is inserted depicting that it is an intruder. For flexibility, an admin password is provided which grantsthe owner access to an interface to change password whenever the need arises. The results presented show that the system responded as designed.

Keywords: *ATMEGA 16*, Buzzer, *Keypad, Liquid Crystal Display, Relay*.

1 INTRODUCTION

Due to the need to improve the security of life and properties in our habitations (Sadeque and Farzana et al., 2015), choice of security systemsaremadein apeculiar mannerminding who access a given location anddistinctive personal trait of the user (Abel, 2017). As we all know security is one of the primary concern of the present day(Diptanil, 2015) and the first thing that should be desired when you look at your family and home should be safety (Sudhir *et al*., 2016). It also serves the function of decoding or detecting intrusion. From time past until present traditional techniques of alarm based security have gained much popularity (Raqibull *et al*., 2015), security locks usually includes mechanical engineering components made up of forged metal, i.e. simple lock and bolt (Mehek *et al*., 2015), the door chain, pin tumbler lock, the jam lock and padlock. Other recently innovated security devices are gadgets such as laser beam detectors (Liu Ying-nan et al., 2017), motion or presence detectors (Jer-vui *et al*., 2013), metal detectors and magnetic card readers. Most recent of these gadgets are an offshoot of biometric engineering. They include voice recognition systems, fingerprint readers, retina eye scanners etc. The principal objectiveof security devices is to prevent an intruder from gaining access to a designated location. Most of these devices can be compromised which at the end of the day gives an unauthorised persons access to where they are barred.The simple jam lock and padlock can be forced openor their keys easilyduplicated byillegal persons.In biometric devices,physical changes of the individual concernedcan fail in authenticating user's identity for granting access. Hence they cannot provide the maximum security that is needed.

Furthermore, these devices are costly; its use is restricted to only a few individuals or organisations that can pay the price. Systems such as motion detectors; light detectors among others are vulnerable to be triggered by wrong signals such as sound impulses, whenever its sensitivity increased. There suitable for most outdoor security protection is limitedbecause they donot possess high discriminative capability during operation and are relatively expensive. Therefore It is imperative to provide a locking mechanism which is efficient and reliable; with high discriminative capacity, non-reliance on the physical quality of the individual concerned, which when forced open triggersan alarm and they are less expensive than their counterparts. The facts presented have led to the design and implementation of an electronic security system. It comprises using correct personal identification numbers to activate the locking device and conditioning the access created on the formation of the security device thereby granting access to the user with the right password access. The electronic security system finds application in homes, banks, in the field of military use, i.e. ammunition warehouse, Academic libraries (Odaro, 2011), households (Prity,*et al*., 2016) industries, (Nandeesh,*et al*., 2014) ministries, and government offices etc.

2 METHODOLOGY

The electronic security system is a user-defined password system that allows only authorised access to users. This section of the paper contains the processesinvolved to get the system working. This section mainly deals with the designed aspect of the hardware and the analysis of individual part that made up the device.Figure1 shows a block representation of the electronic security system.

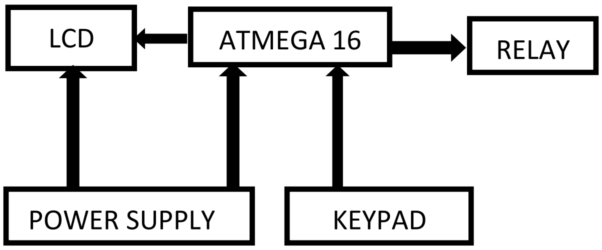

FIGURE 1:BLOCK DIAGRAM OF THE ELECTRONIC SECURITY SYSTEM

2.1 POWER SUPPLY SECTION

The AC source was stepped down from 220Vac to 15Vacusing a transformer. The low AC voltage was then rectified to DC and then filtered by a capacitor to clear ripples. The dc was further regulated to 5Vdc using a regulator (LM7805) and 12Vdc using anotherregulator (LM7812) to power the microcontroller and relay respectively (Abubakar Isah Ndakara *et al*., 2017).

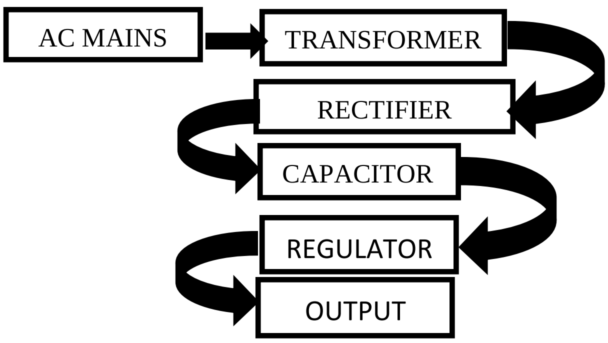

FIGURE 2: BLOCK DIAGRAM OF THE POWER SUPPLY

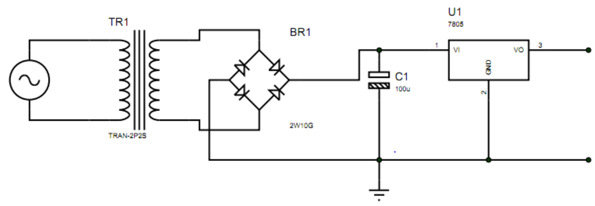

FIGURE 3: THE CIRCUIT DIAGRAM OF THE POWER SUPPLY

2.2 THE CONTROL SECTION

The control section uses a microcontroller ATMEGA 16 which is equipped with inbuilt memory (both ROM and RAM) that allow both temporary and permanent storage of codes. The EEPROM of the controller was used to store the password. The controller was programmed to communicate with the LCD, buzzer and relay connected to the load which is responsible for the switching.

2.3. LCD SECTION

This sectionwas achieved by the use of a liquid crystal display (LCD). Instructions from the datasheet are employed in interfacing the LCD with the controller. The different control signals that could initialise the system were specified. All these instructions were adhered to for success in the prototype made. The symbolic representation of the device is shown in Figure4.

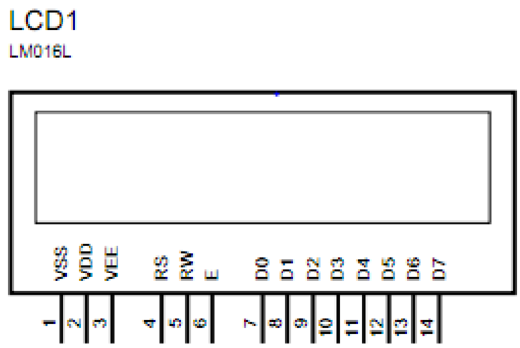

FIGURE 4: SYMBOLIC REPRESENTATION OF AN LCD

2.4RELAY SECTION.

The unit function is switching between the user's load when the correct password is used and vice versa. The diagram in Figure4 is the relay circuit.

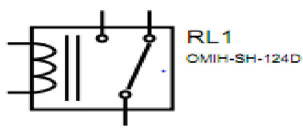

FIGURE 5: RELAY CIRCUIT

2.5 KEYPAD SECTION.

The design uses a 4 by 4 matrix keypad which allows for alphanumeric entries. It consumes no power. It has 8 pins, one for each of the four rows and four columns of buttons. The connections of the 8 pins keypad were to the 8 pins of the microcontroller's port B. The pins representing the four columns are also connected to ground each through a 10kohm resistor. A digital multimeter was used to test for continuity in determining which pin is connected to each row and each column. When theswitch is pushed on, a connection is established between the input pin for the row and the input pin for the column in which the switch is located. Suppose the input pin for the second column is grounded and a voltage is supplied at the input pin for the first row. If no button is pressed, no current will flow, through any row input pin. If however button 0 is pressed, a connection is made between the two input pins and current flows through them thus representing continuity indicated on the multi-meter. Current will not flow between these two pins when any other switch is pressed.

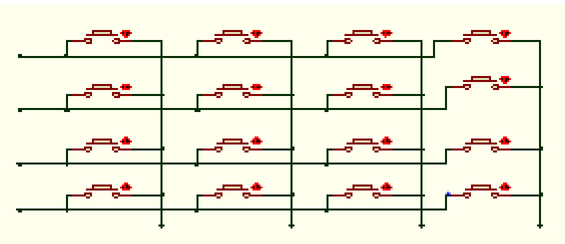

FIGURE 6: KEYPAD

2.6. LOAD CONTROL SECTION

In this paper, the load used was a bulb driven by a relay. The relay is an electromechanical device that enablesa small signal to be used to control a very high current and voltage. The relay is of two made parts which are:the electrical part and the mechanical part. When a logic high was sent to the transistor, since the relay cannot be driven directly by the microcontroller,the amplification transistor isintroduced to boost the current to control the relay more effectively. The resulting magnetic field set up in the relay coil attracts the armature and results to movement of the movable contact to either make or break a connection with a fixed contactor. If the set of contactswere closed when the relay was de-energised, then the movement opens the contacts and breaks the link and vice versa. If the current into the coil is withdrawn, the armature returns forcefully, roughly half the strength of the magnetic force to its stress-free position. Typically, this force is provided by a spring, except in industrial motor starters that commonly use gravity. Most relays are manufactured to operate quickly. In a low voltage application, this is to reduce noise. In a high voltage or high current application, this is to reduce arcing. When dc energises the coil, a diode is placed across the coil which dissipates the energy from the collapsing magnetic field at deactivation. Also,it generates a voltage spike dangerous to circuit components. Some automotive relays already have a diode intheir relay case. Alternatively,the surge may be absorbed by a series connection of a resistor and a capacitor which forms a protection network. If ac is to energise the coil by design, end of the solenoid should be crimped with a small copper ring.

This "shading ring" produces a small out-of-phase current, which raises the minimum pull on the armature during the ac cycle.

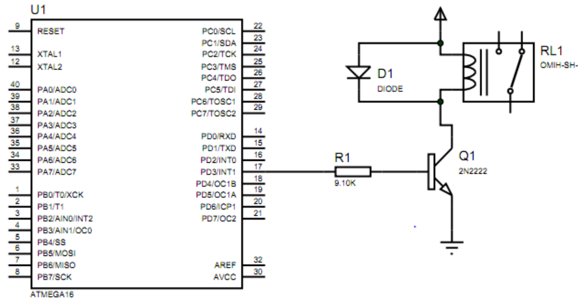

FIGURE 7: LOAD CONTROL SECTION

2.7 OVERALL CIRCUIT DIAGRAM OF THE ELECTRONIC SECURITY SYSTEM

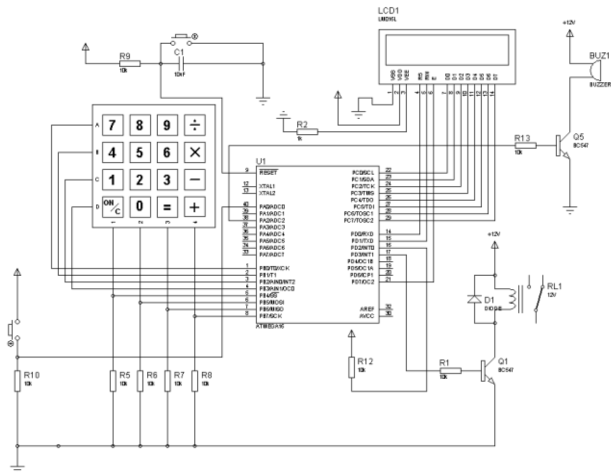

FIGURE 8: CIRCUIT DIAGRAM OF THE OVERALL DESIGN

2.8 IMPLEMENTATION OF THE INTEGRATED ELECTRONIC SECURITY SYSTEM

The casing for the electronic security system was locally constructed using plastic in four corners to form a box. The casing built is shown in Plate1.

PLATE I:CASING OF THE ELECTRONIC SECURITY SYSTEM

3TESTING AND RESULTS

The testing of components for the prototype system was carried out before the implementation was made.

3.1 POWER SUPPLY TEST

The testing of the system started with the power supply unit. The output of the power supply was first tested using digital multimeter before connecting it to the main circuit. The output was measured to be constant 5V at all the time. The input ac supply fluctuation does not affect the magnitude of the output due to the 7805 voltage regulator that was used. The output waveform was observed under oscilloscope to be a straight line without ripples due to the filtering capacitor used at the output of the supply.

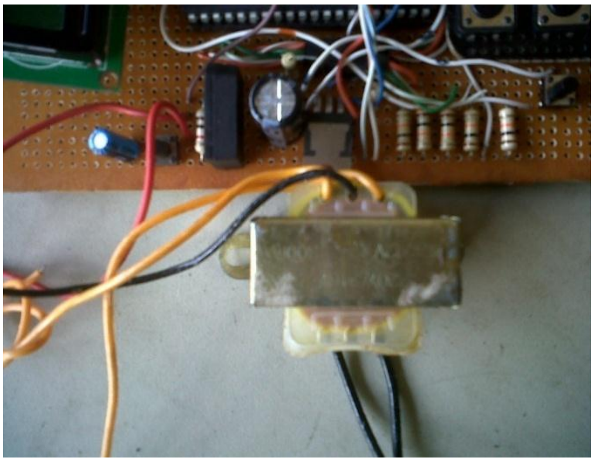

PLATE II. POWER SUPPLY TEST.

3.2 KEYPAD, LCD, RELAY & ICTEST

The keypad was tested by measuring the continuity between one row and its corresponding column, e.g. row1 and column1. When any button is press, the corresponding row and column will be short-circuited of which it enables the signal from the row to be passed directly to the column. The keypad was constructed using 16 pieces of a soft-touch button. This button creates a path for the current to flow whenever it is press and has immediate de-bounce back effect whenever it is released. The keypad host numbers from 0 to 9 and proceeded with the alphabet from **a** to **f**which was written using AVR studio4 software in **C** languagewas debugged so many times to make it an error-freeprogram. Afterwards, it was burned into the memory of the Atmega16 microcontroller. The microcontroller was tested by interfacing it with keypad, LCD and the relay. The LCD (liquid crystal display) provides the visual display of all the operation carried out with the system. The LCD was tested by powering the circuit, it backlight LED comes ON immediately and the character written to the memory of the IC displayed showing that both the program and the LCD are OK. The relay was tested separately by connecting it across a 12V battery and its energies. It means that if the relay should receive a signal from the microcontroller, it will activate any device connected across the **common** and **normally open.**

PLATE III. KEYPAD TEST.

TABLE 1: POWER SUPPLY TEST

Point of taking readings	Multimeter reading
Output terminals of the transformer	11.5Vac
The output of rectified DC	12.07
The voltage across the capacitor	12.05
The output of the voltage regulator	4.95

TABLE 2: KEYPAD/LCD TEST

Test	Result
2222 entered (correct password by default)	LCD shows “ACCESS GIVEN’’
2848 (Admin password)	LCD shows ‘’ENTER OLD PASSWORD’’

3.3 DISCUSSION OF RESULT

Reliability and efficiency of the system were achieved through the authentication of a password before access can granted to any user. The password entered was compared with the one stored in the EEPROM of the microcontroller before granting or denying access to the user when there is a match or mismatch respectively. This makes the system attain optimisation. The use of Atmega 16 microcontroller is advantageous to the system in the sense that it is compact as it is having internal storage of 512 Bytes thereby eliminating the need of an external memory which would have contributed to system bulkiness. The LCD used in the design enhances users interaction with the system by displaying appropriate commands on its screen. The idea makes the system more user-friendly and less tedious to operate. The model also offers an extra level of flexibility by making it possible for the user of the system to change the system password at will atan infinite number of times sequel to the entering of the admin password. This feature could come in quite handy in situations where the user suspects the system's password may have been compromised. The user may then change the password to a unique password known to him alone.Figure8 shows the action on the LCD as simulated using Proteus ISIS.

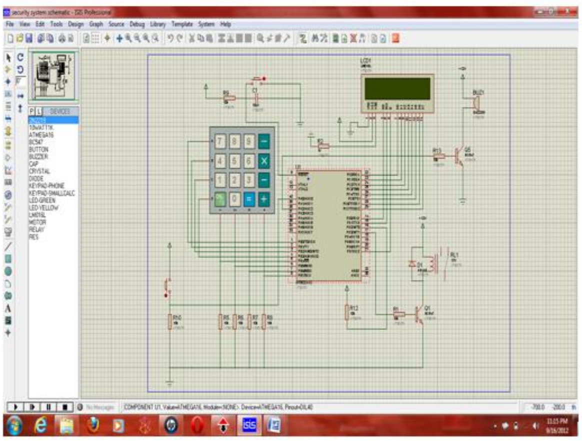

FIGURE 9: CIRCUIT DIAGRAM SIMULATION

4 CONCLUSION

In this paper, the Integrated Electronic Security System was designed and implemented, considering the additional feature of the ATMEGA 16 microcontroller,i.e. electrically erasable programmable read-only memory (EEPROM), password manipulation and storing was executed. The relay was activated when the correct password was insertedand access was given to the user. However,a case of wrong password access was denied by the user and the alarm was triggered by signifying an intruder. The electronic security system finds application in homes, banks, in militaryammunition warehouses, industries, ministries, and government offices etc.

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EFFECTS OF PARTIAL REPLACEMENT OF RIVER SAND WITH STONE DUST IN CONCRETE PRODUCTION

Eze, O. O; Aguwa, J. I. and Auta, S. M.

Civil Engineering Department, Federal University of Technology, Minna, Nigeria
* ezeogbonna38@gmail.com

ABSTRACT

This paper investigates the effects of partial replacement of conventional river sand with crushed rock dust in concrete production. To achieve this, different preliminary tests were performed on the two sources of aggregate (river sand and crushed rock dust) to determine their physical properties before using in the production of concrete cubes and beams to determine their compressive and flexural strengths. Hence, the percentage replacement of conventional river sharp sand with crushed rock dust at 0%, 10%, 20%, 30%, 40%, 50%, and 100% to evaluate the optimum compressive and flexural strengths of concrete produced. The water-cement ratio used is 0.5 while the mix ratio is 1: 1.397:3.419. To determine the compressive and flexural strengths, concrete cubes and beams of sizes (150 x 150 x 150) mm and (100 x 100 x 500) mm respectively were prepared and cured for 7days and 28days and its compressive and flexural strengths determined. The result shows that concrete produced with 0% replacement of conventional river sand with crushed rock dust attain the highest compressive strength at 7 and 28days curing ages while concrete produced with crushed rock dust has a lower compressive strength. Also, the optimum compressive strength was achieved at 40% replacement of conventional river sharp sand with crushed rock dust while the concrete attains its maximum flexural strength at 30% replacement level. At 28day, the concrete produced with river sharp sand attain its optimum compressive and flexural strengths of 31.72% and 6.72N/mm^2 respectively, while the maximum compressive and flexural strengths was achieved at 40% replacement level.

Keywords: *Compressive strength, Concrete, Crushed rock dust, Flexural strength and River Sharp sand.*

1 INTRODUCTION

One of the major characteristics of a developing nation is rapid growth in infrastructural development. Concrete plays a major role in any infrastructural development. One of the basic constituents of concrete is aggregate as it constitutes approximately 75% of total volume of concrete. Aggregate can either be coarse or fine, both been naturally occurring rock materials. In major cities in Nigeria, the most widely used fine aggregates are conventional river sand and crushed rock dust. While conventional river sand is traditionally derived from natural sources in the form of river sand, Crushed rock dust is a process controlled crushed fine aggregate produced from quarried stone by crushing or grinding and classification to obtain a controlled gradation product that completely passes the 4.75 mm sieve. However, in recent years, the scarcity of clean river sand has led to a sharp increase in the use of the readily available crushed stone dust as fine aggregate. While some companies involved in concrete production replace the conventional river sand with crushed rock dust at arbitrary percentage. Others use solely crushed rock dust as fine aggregate in concrete production. Therefore, it is very important to investigate the effect of partial replacement of river sand with crushed rock dust in concrete production. Moreover, there is the need to determine the optimum percentage replacement of conventional river sand with crushed rock dust (stone dust). Hitherto, different researchers have examined the properties of concrete using Natural River sand and Crushed Rock dust. (Ilangovana, Mahendrana, & Nagamanib, 2008), crushed rock dust is residue, tailing or other non-voluble waste material after the extraction and processing of rocks to form fine particles less than 4.75mm. These researchers carried out a research on Strength and durability properties of concrete containing quarry rock dust as fine aggregate and concluded that the strength of Crushed Rock Dust concrete is comparatively 10-12 percent more than that of similar mix of Conventional Concrete. (Sanjay, Sindhib, Vinay, Ravindra, & Vinay, 2016), investigated Crushed rock sand-An economical and ecological alternative to natural sand to optimize concrete mix and concluded that partial replacement up to 30% leads to decrease in slump value, a significant improvement in compressive, flexural strength and impact resistance. Hence, the properties of concrete (Compressive and flexural strength) made with partial or full replacement with crushed rock (stone dust) are comparable to natural sand results. (Rameshwar & Shrikant, 2017), carried out a research on Replacement of Natural Sand by Crushed Sand in the Concrete and concluded that concrete with crushed sand performed better than concrete with natural sand as the property of crush sand is better than that of natural sand. They further stated that different Crushed sand gives different results for compressive strength depending on different quarries and from study of different research paper at 40% to 50% replacement of crushed sand the maximum compressive strength is obtained. (Lalit & Arvinder, 2015) examined the strength of concrete using crushed stone dust as fine aggregate concluded that the compressive strength, flexural strength and split tensile strength of concrete for grade M25 and M30 with stone dust as fine aggregate were found to be comparable with the concrete made with the river bed sand. They further concluded that increase in compressive strength of concrete with 20% replacement and 50% replacement of fine aggregate with stone dust is found to be 8 to 10%, hence, stone dust can effectively be used in plain cement concrete in place of fine aggregate. (Chijioke, Igwegbe, Ibearugbulem, Okoye, & Oke, 2015) on their research stated that the gradual shift from conventional river sand to Crushed Rock Dust as an alternative and suitable source of fine aggregate is as result of scarcity in conventional river sand. They compared the compressive strengths of concrete made with river sand and quarry dust as fine aggregates and concluded that Crushed Rock Dust can effectively be used to replace river sand and reduce the negative impact this causes our environments due to constant plunging of our rivers and coastal areas in the name of extracting river sand for construction purposes.

This present research work seeks to further examine the effect of percentage replacement of conventional river sand with crushed rock (stone dust) in other to obtain a satisfactory compressive, and flexural strength. The aim of this research work is to evaluate effects of partial replacement of river sand with stone dust in concrete production. The main objectives of the research are (1) to determine the physical properties of fine aggregate (river sharp sand and crushed rock dust) and Coarse aggregate (2) to evaluate the percentage replacement of conventional river sand with crushed rock (stone dust) in other to obtain a satisfactory and optimum compressive and flexural strengths of concrete. The compressive and flexural strength at 7 and 28 days will be determined while the percentage replacement will be at 10% intervals.

2 MATERIALS AND METHOD

2.1 MATERIALS

The materials used for this research work for the production of concrete (cubes, beams and cylinders) include; fine aggregates (river sand and crushed rock dust), coarse aggregate (granite), water and cement. The river sharp sand was sourced from Chanchaga River, while crushed rock was obtained from Maikunkele Quarry. Both purchased locally from Minna, Niger state. It shall be tested in accordance with BS 882 (1992) requirements. The coarse aggregate used for the concrete production is crushed aggregate (granite) with its maximum size of 20mm. The coarse aggregate was locally sourced from within Minna metropolis in Niger state and complies with the requirements of BS882 (1992).The quality of water that used in the concrete production is portable water free from impurities, organic matter, dissolved salts or any form of chemical impurities in compliance to BS EN 1008 (2002) requirements. The source of water is Federal University of Technology, Minna water tap. This is to ensure that the concrete meets its specification. The type of cement used for the production of concrete is Ordinary Portland Cement (OPC) and of the brand of Dangote cement. The physical properties of this cement conform to the requirements of Ordinary Portland Cement as prescribed in British Standard, BS EN 197-1: 2000.

2.2 METHODS

The methods adopted in this research work before the concrete production and casting of the concrete cubes and beams includes; sieve analysis, specific gravity and natural moisture content. These methods were effectively carried-out at the structural and material laboratory of the Civil Engineering Department, Federal University of Technology Minna, Niger state. In order to achieve the primary objectives of this research work, the methods adopted was carried-out in accordance to British Standard codes. For the purpose of this research work, the mix design method adopted in the concrete mix design was the Standard British Method of concrete mix design popularly known as (DOE) first published in 1975 and revised in 1998. This method was used in batching out the entire ingredient used in concrete production and this is done in compliance with the relevant British Standards codes. The steps outlined in this method for the calculation of the constituent materials were carefully followed to arrive at the results.

Crushed rock dust were used to replace conventional river sharp sand at 0%, 10%, 20%, 30%, 40%, 50%, and 100%. Batching by weight was used as shown in Table 1. The British Standard Method of Mix Design was adopted and used to carry out mix design on the constituent materials after preliminary tests to determine the physical properties of constituent materials. Using the British Standard Method, a mix ratio of 1: 1.397:3.419 and water cement ratio of 0.5 was used in producing the concrete. Concrete cubes and beams were cast using 150mm x 150mm x 150mm and 500mm x 100mm x 100mm steel moulds. Prior to mixing of the fresh concrete, the concrete cube and beam moulds were assembled, firmly secured with bolts and nuts, well lubricated with engine oil for easy removal of the concrete cubes and beams after it must have hardened. The concrete was mixed using a concrete mixer. Part of the mixing water was introduced into the mixer, followed by fine aggregate and cement in their respective quantities by weight and allowed to mix homogenously before putting the coarse aggregate and the remaining water. The mixer was allowed to rotate severally until a homogenous mix is obtained. Afterwards, each mould was filled with the fresh concrete in three layers of about 50mm thick with each layer tampered with a 16mm tamping rod using thirty five (35) strokes uniformly distributed across the concrete mould. The top of each mould was smoothened and leveled with hand trowel and then the outside surface cleaned. After 24hours after casting the concrete, when the concrete cubes and beams must have set, they were removed and kept in a curing tank filled with fresh tap water for 7 and 28days curing respectively.

3 RESULTS AND DISCUSSION

3.1 PHYSICAL PROPERTIES

Table 1 shows the results of physical properties of the fine and coarse aggregates used. These physical properties includes; Specific gravity, Sieve analysis, Bulk relative density and Natural moisture content. The specific gravities of Natural river sand, crushed rock dust and coarse aggregate were found to be 2.65, 2.69 and 2.66 respectively. Their specific gravities falls within the range of specific gravities as contained in (Neville, 2011) and in accordance with the requirements in British Standard (BS) 1377(1990). The sieve analysis results for both natural river sand and crushed rock dust shows that both fine aggregates are in Zone I and also satisfies the overall grading limit and also the medium grading in compliance with BS EN12620: 2002. The coarse aggregate also satisfies the fineness modulus requirement for concrete works which ranges from 2.5 to 3.5 as stated by (Akinboboye, Adegbesan, Ayegbusi, & Oderinde, 2015). Moreover, the coefficient of uniformity (Cu) is > 4 and the coefficient of curvature (Ccr) is more than 1.0, the two types of fine aggregate used in this research work are well graded sand. The graphical representation is shown in figure 1.

TABLE 1: PHYSICAL PROPERTIES OF CRUSHED ROCK DUST AND NATURAL RIVER SAND.

Property	Natural River Sand	Crushed Rock Dust	Coarse Aggregate	Test Method Used
Specific gravity	2.65	2.69	2.66	(ASTM) C 127-04
Bulk relative density (kg/m^3)	1720.00	1715.93	-	ACI (1999)
Water Absorption (%)	23.88	23.48	0.32	
Natural Moisture content (%)	3.97	0.49	0.59	BS 812-109 (1990).
Sieve analysis (Fineness modulus)	Zone I (2.81)	Zone I (3.07)	2.94	BS 410-1 and 2:2000
Coefficient of Uniformity (Cu)	5.67	6.50	-	(ASTM) D 2487 and
Coefficient of curvature (Ccr)	1.42	1.63	-	AASHTO

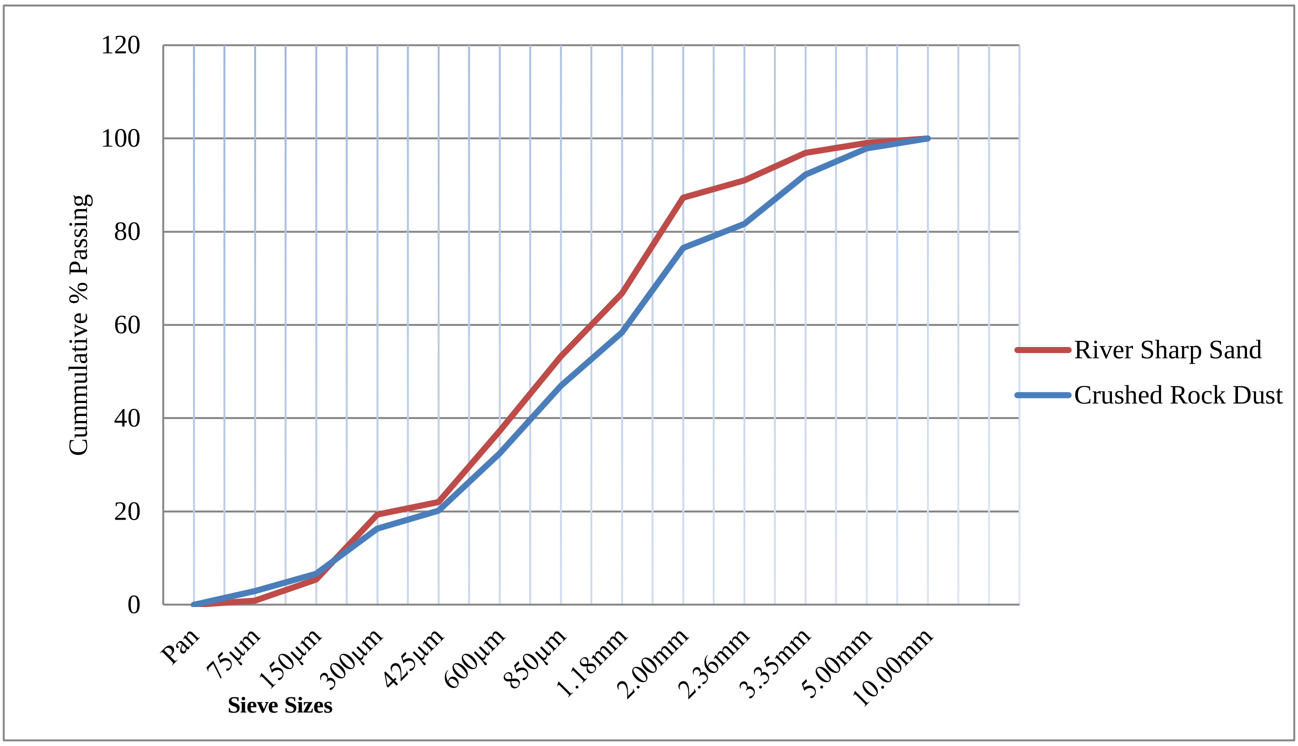

FIGURE 1: SIEVE ANALYSIS GRAPH FOR CRUSHED ROCK DUST AND RIVER SHARP SAND

3.2 COMPRESSIVE STRENGTH

The compressive strength of the concrete cubes at 7 and 28 curing days for different percentage replacement of conventional river sand with crushed rock dust are presented in Tables 2 and 3 respectively. The graphical presentation is shown in the figure 2. The result shows that the concrete cube produced with conventional river sand (0% replacement) attains the highest compressive strength of 28.80N/mm^2 and 31.73N/mm^2 at 7 and 28days curing age respectively while concrete produced with crushed rock dust (100% replacement) attains its maximum compressive strength of 24.03N/mm^2 and 28.49N/mm^2 respectively. This shows 13.35% decrease in compressive strength of concrete produced with crushed rock dust when compared with the compressive strength of concrete produced with natural river sand. This is consistent with (Thushar & Balakrishna, 2015). The result also shows that at 40% replacement of natural river sand with crushed rock sand the optimum compressive strength of the concrete cubes was obtained. This is in agreement with (Rameshwar & Shrikant, 2017). Moreover, it was observed that the lowest compressive strength was obtained at 10% replacement of conventional river sand with crushed rock dust. The tables and figure is shown below.

TABLE 2: COMPRESSIVE STRENGTHS FOR DIFFERENT PERCENTAGE REPLACEMENTS AT 7DAYS.

S/No	Sample No	Weight of sample (kg)	Area of sample (mm^2)	Crushing load (N)	Compressive strength (N/mm^2)	Average Strength (N/mm^2)
7days compressive strength for 0% replacement						
1	X_{0-1}	9.63	22500	580	25.78	
2	X_{0-2}	9.01	22500	680	30.22	
3	X_{0-3}	9.57	22500	610	27.11	28.80
4	X_{0-4}	8.60	22500	690	30.68	
5	X_{0-5}	9.05	22500	680	30.22	
7days compressive strength for 10% replacement						
1	A_{10-1}	9.00	22500	380	16.89	
2	A_{10-2}	9.26	22500	420	18.67	
3	A_{10-3}	9.20	22500	450	20.00	20.18
4	A_{10-4}	9.30	22500	550	24.44	
5	A_{10-5}	9.28	22500	470	20.88	

		7days compressive strength for 20% replacement				
1	B_{20-1}	8.42	22500	580	25.78	
2	B_{20-2}	9.11	22500	540	24.00	
3	B_{20-3}	9.14	22500	490	21.78	24.09
4	B_{20-4}	9.13	22500	510	22.68	
5	B_{20-5}	9.25	22500	590	26.22	
		7days compressive strength for 30% replacement				
1	C_{30-1}	8.73	22500	540	24.00	
2	C_{30-2}	9.31	22500	510	22.67	
3	C_{30-3}	8.97	22500	610	27.11	23.38
4	C_{30-4}	9.34	22500	480	21.33	
5	C_{30-5}	9.42	22500	490	21.78	
		7days compressive strength for 40% replacement				
1	D_{40-1}	8.70	22500	530	23.56	
2	D_{40-2}	8.91	22500	540	24.00	
3	D_{40-3}	9.18	22500	620	27.56	25.37
4	D_{40-4}	9.22	22500	570	25.33	
5	D_{40-5}	8.96	22500	590	26.40	
		7days compressive strength for 50% replacement				
1	E_{50-1}	9.23	22500	380	16.89	
2	E_{50-2}	9.33	22500	390	17.33	
3	E_{50-3}	9.28	22500	570	25.33	21.33
4	E_{50-4}	9.26	22500	570	25.33	
5	E_{50-5}	9.32	22500	490	21.78	
		7days compressive strength for 100% replacement				
1	Y_{100-1}	8.73	22500	570	25.33	
2	Y_{100-2}	8.95	22500	530	23.56	24.03
3	Y_{100-3}	8.19	22500	590	26.40	
4	Y_{100-4}	9.23	22500	460	20.44	
5	Y_{100-5}	8.58	22500	550	24.44	

TABLE 3: COMPRESSIVE STRENGTHS FOR DIFFERENT PERCENTAGE REPLACEMENTS AT 28DAYS.

S/No	Sample No	Weight of sample (kg)	Area of sample (mm^2)	Crushing load (N)	Compressive strength (N/mm^2)	Average Strength (N/mm^2)
		28days compressive strength for 0% replacement				
1	X_{0-6}	9.61	22500	610	27.11	
2	X_{0-7}	8.89	22500	850	37.78	
3	X_{0-8}	9.54	22500	610	27.11	31.73
4	X_{0-9}	9.37	22500	680	30.22	
5	X_{0-10}	8.92	22500	820	36.44	
		28days compressive strength for 10% replacement				
1	A_{10-6}	8.85	22500	590	26.22	
2	A_{10-7}	8.66	22500	670	29.77	
3	A_{10-8}	9.17	22500	720	32.00	27.39
4	A_{10-9}	9.21	22500	490	21.77	
5	A_{10-10}	9.15	22500	600	26.67	
		28days compressive strength for 20% replacement				
1	B_{20-6}	8.82	22500	670	29.78	
2	B_{20-7}	8.64	22500	650	28.89	
3	B_{20-8}	8.99	22500	640	28.44	27.73
4	B_{20-9}	9.22	22500	480	21.33	
5	B_{20-10}	9.22	22500	680	30.22	
		28days compressive strength for 30% replacement				
1	C_{30-6}	8.89	22500	550	24.44	
2	C_{30-7}	9.32	22500	660	29.33	

3	C_{30-8}	9.34	22500	700	31.11	27.73
4	C_{30-9}	9.09	22500	610	27.11	
5	C_{30-10}	8.84	22500	600	26.67	
28days compressive strength for 40% replacement						
1	D_{40-6}	8.92	22500	640	28.44	
2	D_{40-7}	9.51	22500	760	33.78	
3	D_{40-8}	9.40	22500	690	30.67	29.60
4	D_{40-9}	9.45	22500	520	23.11	
5	D_{40-10}	9.29	22500	720	32.00	
28days compressive strength for 50% replacement						
1	E_{50-6}	9.18	22500	600	26.67	
2	E_{50-7}	9.06	22500	670	29.78	
3	E_{50-8}	8.99	22500	620	27.56	27.82
4	E_{50-9}	8.89	22500	560	24.89	
5	E_{50-10}	9.16	22500	680	30.22	
28days compressive strength for 100% replacement						
1	Y_{100-6}	9.00	22500	640	28.44	
2	Y_{100-7}	9.32	22500	530	23.56	
3	Y_{100-8}	8.55	22500	770	34.22	28.49
4	Y_{100-9}	9.02	22500	680	30.22	
5	Y_{100-10}	9.02	22500	540	24.00	

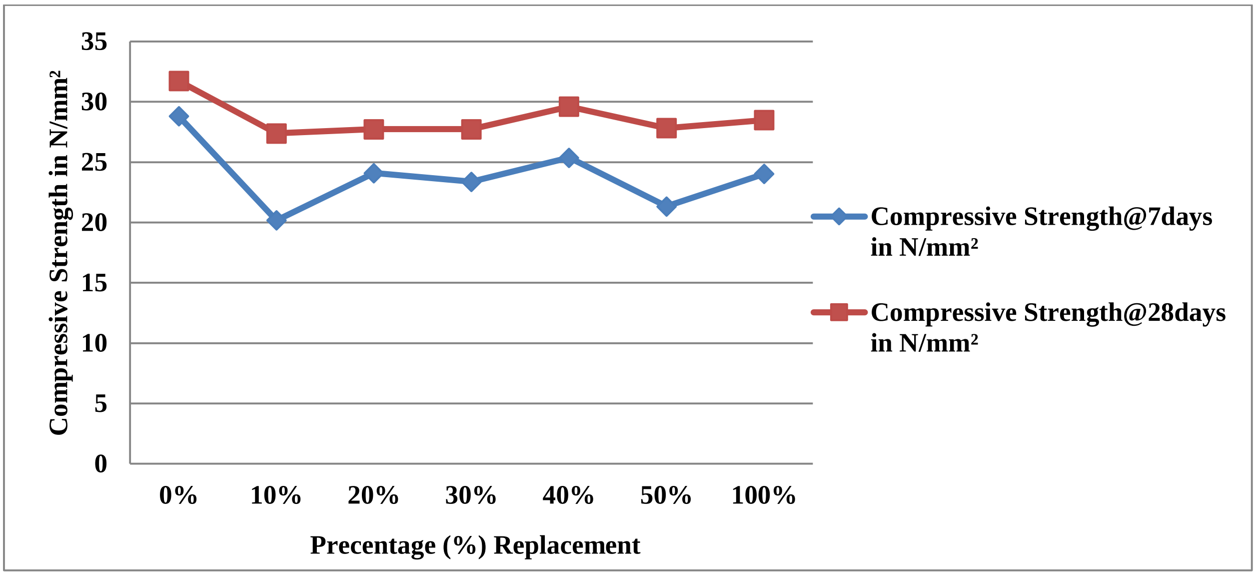

FIGURE 2: COMPRESSIVE STRENGTH OF THE PERCENTAGE REPLACEMENT AT 7 AND 28DAYS CURING DAYS.

3.3 FLEXURAL STRENGTH

Tables 4 and 5 shows the results of the flexural strength of the concrete beams at 7 and 28days curing ages for varying percentage replacement of conventional river sand with crushed rock dust. The result shows that the concrete beams produced with conventional river sand (0% replacement) attains the highest compressive strength of 4.13N/mm^2 and 6.72N/mm^2 at 7 and 28days curing age respectively while concrete produced with crushed rock dust (100% replacement) attains its maximum compressive strength of 3.71N/mm^2 and 5.98 N/mm^2 respectively.

This shows an average of 0.58% decrease in flexural strength of concrete produced with crushed rock dust when compared with the compressive strength of concrete produced with natural river sand. The result also shows that at 30% replacement of natural river sand with crushed rock sand the optimum flexural strength of the concrete beams was obtained. The tables and figure are shown below.

TABLE 4: FLEXURAL STRENGTHS FOR DIFFERENT PERCENTAGE REPLACEMENTS AT 7DAYS.

S/No	Sample No	Weight of sample (kg)	Area of sample (mm^2)	Crushing load (N)	Flexural strength (N/mm^2)	Average Strength (N/mm^2)
7days compressive strength for 0% replacement						
1	X_{0-1}	13.26	10000	77	3.85	
2	X_{0-2}	12.98	10000	90	4.50	
3	X_{0-3}	13.45	10000	81	4.05	4.13
4	X_{0-4}	13.07	10000	86	4.3	
5	X_{0-5}	12.76	10000	92	3.95	
7days compressive strength for 100% replacement						
1	A_{10-1}	12.96	10000	84	4.20	
2	A_{10-2}	13.23	10000	66	3.85	
3	A_{10-3}	12.78	10000	77	3.35	3.64
4	A_{10-4}	12.32	10000	67	3.50	
5	A_{10-5}	13.95	10000	70	3.30	
7days compressive strength for 20% replacement						
1	B_{20-1}	13.98	10000	73	3.65	
2	B_{20-2}	13.04	10000	72	3.60	
3	B_{20-3}	13.88	10000	66	3.33	3.59
4	B_{20-4}	12.97	10000	75	3.75	
5	B_{20-5}	12.77	10000	73	3.65	
7days compressive strength for 30% replacement						
1	C_{30-1}	12.54	10000	72	3.75	
2	C_{30-2}	12.89	10000	68	3.85	
3	C_{30-3}	13.08	10000	76	4.05	3.84
4	C_{30-4}	13.77	10000	68	4.05	
5	C_{30-5}	12.90	10000	66	3.5	
7days compressive strength for 40% replacement						
1	D_{40-1}	14.02	10000	75	3.40	
2	D_{40-2}	13.88	10000	77	3.80	
3	D_{40-3}	13.67	10000	81	3.40	3.53
4	D_{40-4}	12.98	10000	81	3.60	
5	D_{40-5}	13.65	10000	70	3.45	
7days compressive strength for 50% replacement						
1	E_{50-1}	13.45	10000	62	3.10	
2	E_{50-2}	12.79	10000	60	3.00	
3	E_{50-3}	12.89	10000	68	3.40	3.24
4	E_{50-4}	13.76	10000	61	3.05	
5	E_{50-5}	13.93	10000	73	3.65	
7days compressive strength for 50% replacement						
1	Y_{100-1}	13.56	10000	76	3.80	
2	Y_{100-2}	12.98	10000	71	3.55	
3	Y_{100-3}	12.54	10000	78	3.90	3.71
4	Y_{100-4}	13.22	10000	61	3.05	
5	Y_{100-5}	13.55	10000	85	4.25	

TABLE 9: FLEXURAL STRENGTHS FOR DIFFERENT PERCENTAGE REPLACEMENTS AT 28DAYS.

S/No	Sample No	Weight of sample (kg)	Area of sample (mm^2)	Crushing load (N)	Flexural strength (N/mm^2)	Average Strength (N/mm^2)
28days compressive strength for 0% replacement						
1	X_{0-1}	12.78	10000	122	6.10	
2	X_{0-2}	12.54	10000	142	7.10	
3	X_{0-3}	11.98	10000	128	6.40	6.72

4	X_{0-4}	12.66	10000	135	6.75	
5	X_{0-5}	11.77	10000	145	7.25	
28days compressive strength for 10% replacement						
1	A_{10-1}	12.56	10000	120	6.60	
2	A_{10-2}	13.07	10000	113	5.25	
3	A_{10-3}	13.28	10000	123	6.10	5.98
4	A_{10-4}	12.70	10000	108	5.35	
5	A_{10-5}	12.90	10000	134	5.55	
28days compressive strength for 20% replacement						
1	B_{20-1}	12.86	10000	132	5.80	
2	B_{20-2}	12.65	10000	105	5.70	
3	B_{20-3}	12.95	10000	122	5.25	5.70
4	B_{20-4}	13.08	10000	107	5.95	
5	B_{20-5}	13.11	10000	111	5.80	
28days compressive strength for 30% replacement						
1	C_{30-1}	13.78	10000	114	5.95	
2	C_{30-2}	13.29	10000	108	6.10	
3	C_{30-3}	12.25	10000	120	6.40	6.07
4	C_{30-4}	12.48	10000	108	6.40	
5	C_{30-5}	13.35	10000	105	5.50	
28days compressive strength for 40% replacement						
1	D_{40-1}	12.08	10000	116	5.70	
2	D_{40-2}	13.80	10000	114	5.40	
3	D_{40-3}	12.78	10000	105	6.00	5.69
4	D_{40-4}	13.72	10000	119	5.40	
5	D_{40-5}	12.95	10000	116	5.95	
28days compressive strength for 50% replacement						
1	E_{50-1}	13.78	10000	119	4.95	
2	E_{50-2}	13.85	10000	122	4.80	
3	E_{50-3}	12.75	10000	128	5.40	5.16
4	E_{50-4}	12.92	10000	128	4.85	
5	E_{50-5}	13.95	10000	111	5.80	
28days compressive strength for 50% replacement						
1	Y_{100-1}	13.90	10000	99	6.00	
2	Y_{100-2}	13.79	10000	96	5.65	
3	Y_{100-3}	12.85	10000	108	6.15	5.98
4	Y_{100-4}	13.76	10000	97	5.40	
5	Y_{100-5}	12.54	10000	116	6.70	

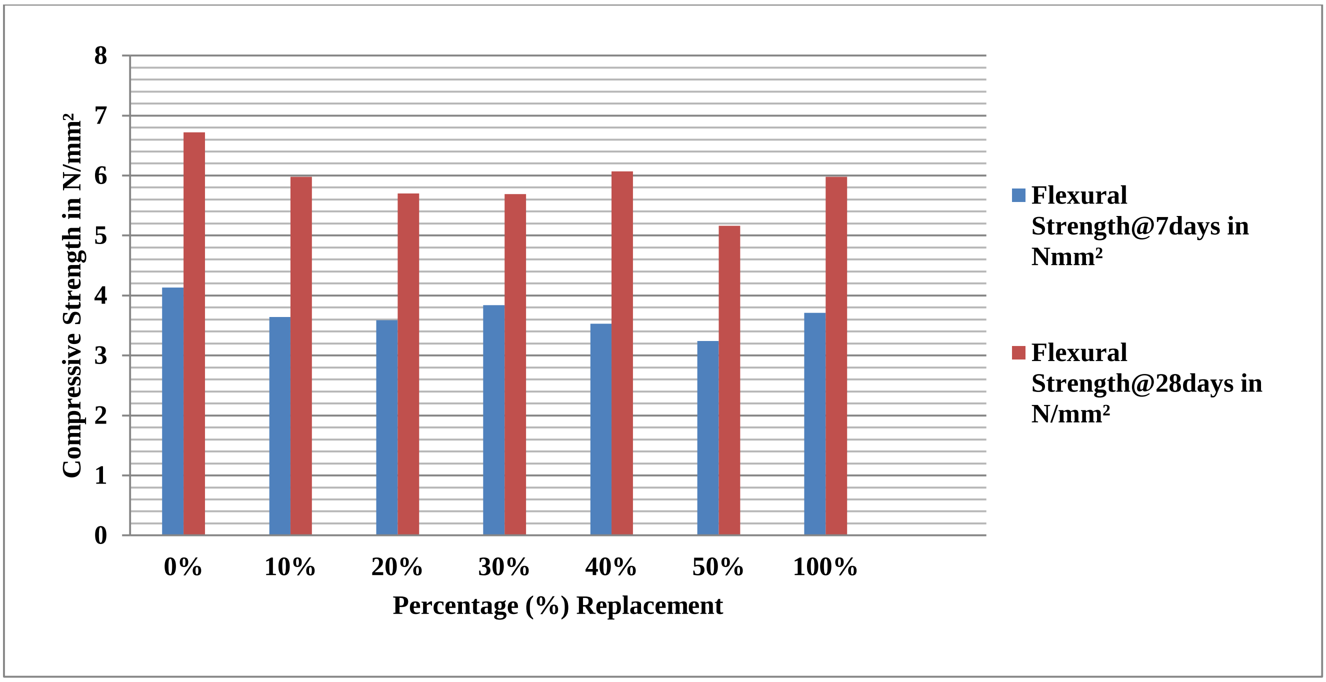

FIGURE 3: FLEXURAL STRENGTH OF THE PERCENTAGE REPLACEMENT AT 7 AND 28DAYS CURING DAYS.

TABLE 5: BATCHING INFORMATION.

S/No	% replacement of crushed rock dust with river sand	Cement (kg)	Natural river sand (kg)	Crushed rock dust (kg)	Coarse aggregate (kg)	Water (kg)	water-cement ratio(w/c)
1	0	7.65	10.67	0	26.18	3.83	0.5
2	10	7.65	9.605	1.065	26.18	3.83	0.5
3	20	7.65	8.535	2.134	26.18	3.83	0.5
4	30	7.65	7.47	3.201	26.18	3.83	0.5
5	40	7.65	6.402	4.268	26.18	3.83	0.5
6	50	7.65	5.335	5.335	26.18	3.83	0.5
7	100	7.65	0	10.67	26.18	3.83	0.5

4 CONCLUSION

The effect of partial replacement of conventional river sand with crushed rock dust has been investigated and the results obtained. From the results, it was observed that the physical properties (Table 1) of River sharp sand sourced from Chanchaga River and the Crushed Rock Dust sourced from Maikunkele Quarry are nearly same and satisfies the requirements of the Standard Codes Provision for the physical properties of fine aggregate for normal weight concrete production. However, Crushed rock dust when solely used as fine aggregate in concrete production gives a satisfactory result in terms of compressive and flexural strengths when compared with concrete produced with the conventional river sand.

Conversely, the conventional river sand when used as fine aggregate gives higher compressive and flexural strengths in the production of normal weight concrete. Moreover, each percentage replacement of conventional river sand with crushed rock dust gives a different compressive and flexural strength with the optimum compressive and flexural strengths obtained at 40% replacement respectively. It was also observed that the rate of strength gain increases as the age of concrete curing increases.

5. ACKNOWLEDGEMENTS

From this research work, the following recommendations are made:

That the physical properties of the fine and coarse aggregates be tested before being used to determine their suitability in concrete production.

For optimum compressive and flexural strengths, 40% replacement level of conventional river sand with crushed rock as fine aggregate in the production of normal weight concrete should be adopted.

Full percentage replacement is also recommended in area where there is scarcity or limited supply of the conventional river sharp sand.

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COMPUTER AIDED ANALYSIS OF A POWER TRANSMISSION TOWER SUBJECTED TO GROUND ACCELERATION

*Alufa, S. O.[1]; & Sadiku, S.[2]
[1,2] Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: alufasamson@hotmail.com

ABSTRACT

Power transmission tower is a relatively light weight structure with high flexibility and low circular or natural frequencies. Towers are usually sensitive to horizontal loadings especially those induced by explosions, earth tremors and wind. These categories of disturbances might lead to long term fatigue or eventual failure of the power transmission tower thus impeding power supply, causing damage to properties within reach and risk of electrocution. This paper presents the computer aided analysis of a self-supporting tower using STAAD pro and Matrix Laboratory (MATLAB) software. Based on the finite element method, the tower was modelled as a 3D structure with discrete finite elements and subjected to a constant ground acceleration of 0.3g, (where g = 9.81 m/s^2, is acceleration due to gravity). The tower considered had 264 elements, 79 nodes and 237 degrees of freedom. The tower members were assumed to be axially loaded, bending and twisting moments were assumed to be negligible hence, not considered. The natural frequencies, various mode shapes and the response of the system were presented. The maximum displacements obtained as a result of the base excitation in the X, Y and Z directions were 89.142 mm, 88.915 mm and 23.44 mm respectively. While the maximum stress was 613.53 N/mm^2 (compressive). The study showed that the effect of ground motion leads to large displacements and stresses, hence the effect of earth tremors need to be taken into consideration when designing towers in areas that are exposed to mining and other earth disturbing activities and not just areas exposed to earthquakes alone.

Keywords: *Consistent matrix, Eigenvalue problem, Ground acceleration, Modal analysis*

1 INTRODUCTION

Power transmission towers are elevated lattice or tubular structures comprising of majorly steel members used for supporting electrical components like conductor subsystems, insulator subsystem and ground wires, at a certain height above ground level (Punse, 2014). The structural integrity of such lines is crucial in the prevention of power supply failure. Such failures have potentially huge impacts on society, with both locals and commercial operations affected, potentially leading to large economic consequences and huge disruption in day to day life. Mechanical supports of transmission line represent a significant portion of the cost of the line and play an important role in the reliable power transmission. They are designed and constructed in wide variety of shapes, types, sizes, configurations and materials. The supporting structure types used in transmission lines generally fall into one of the three categories: lattice, pole and guyed.

The executive director, Association of Nigeria Electricity Distributors, Mr. Sunday Oduntan reported that Nigeria must generate at least 180,000 megawatts of electricity to have adequate and stable power supply (Awoyinka, 2018). Also, the disposition of the primary resources for electrical power generation such as gas, wind and solar, is quite uneven or not properly managed, thus adding to the transmission requirements (Punse, 2014).

As Nigeria is not located in the seismic region (no risk of earthquake), most mast or tower were designed to the British standard which did not take into account the effect of earthquake or ground motion. In recent time, there has been a number of earth tremors in several locations, one occurred in Shaki, Oyo state some two years ago, one in Oyi, Kaduna State and in some other locations. This year it was in Mpape, FCT, Abuja (Nnodim, 2018). These occurrences and the possible impacts on civil engineering structures is the major motivation why this study was carried out.

The incessant blasting of rocks by miners, extraction of crude oil in the south eastern part of the country and the uncontrolled borehole drilling are some of the possible operations that could lead to ground excitation.

The aim of this study is to carry out the computer aided analysis and design of a power transmission tower subjected to ground acceleration.

Due to the importance of the power transmission line system, so many researchers have carried out the analysis and design of the power transmission tower.

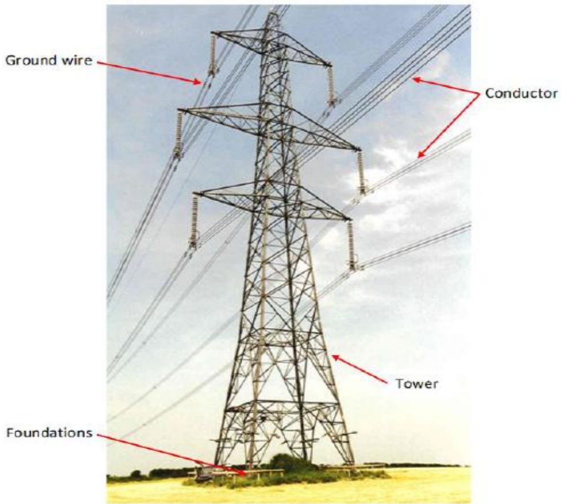


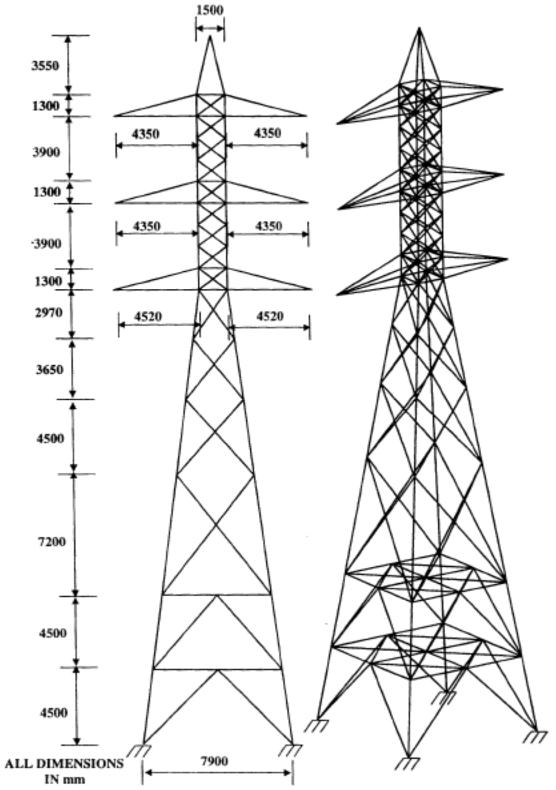
PLATE I: COMPONENTS OF A POWER TRANSMISSION TOWER. **SOURCE**: (ALAA & AHMED, 2013)

Aziz and Ghoborah (1996) worked on the nonlinear dynamics of the transmission lines. The towers were modelled by three-dimensional truss elements while the cables were modelled by two node elements that accounted for their geometric nonlinearity. The nonlinear analysis showed that the displacement of the cable when subjected to vertical and transverse ground motions could be substantial especially for earthquakes with relatively small acceleration to velocity ratio. Li *et al.* (2012) presented a method for evaluating the dynamic characteristics of a power transmission tower coupled with power line under wind load the evaluation criteria and dynamic response was developed and applied to the finite element analysis of a power transmission tower line system under wind load. The final result indicated that the proposed energy evaluation could be effectively utilized in the examination of structural dynamic performance. Liang *et al.* (2014) carried out the dynamic analysis of a power transmission tower collapse under wind load. The tower was modelled as an assembly of beam-column elements four-node isoparametric curved element and prestressed pole unit simulation insulator using bilinear restoring force model to formulate the material nonlinearity. They found out that compressive failure occurred due to the downburst wind at the lower part of the tower. Sonowal *et al.* (2015) analyzed and design a 220 kV transmission tower line by the application of a conventional method of analysis and Indian code base design. The tower was design in wind zone- v with base with 1/5th the total height of the tower. They reported that the 220 kV transmission was chosen with a view to optimize the existing geometry and then the tower was subsequently analyzed as a 2-D structure. The structure was made determinate by excluding the horizontal members and axial forces were calculated using method of joint resolution and designed in accordance to IS 800: 2007. Pasupuleti and Ganesh (2016) carried out the analysis of electrical transmission tower using finite element technique, they found out that the static response of the transmission tower, like deformation and rotation in all direction and corresponding stress resultant like bending moment and shear force in corresponding direction due to static transverse ad vertical loads applied on electrical transmission tower system were within allowable or safe limit. Oglan *et al*. (2017) carried out the dynamic response analysis of masonry minaret of Yögüç Paşa Mosque subjected to artificially generated blast induced ground motion by using a three dimensional finite element model. In order to model the blast-induced ground motion, peak acceleration and time envelope curve function of ground motion acceleration were obtained from the distance of the explosion centre. The result of the study indicated that the masonry minaret was affected substantially by effects of blast-induced ground motion.

2 METHODOLOGY

In order to carry out the analysis, the following steps were followed

1. The tower was modelled with STAAD pro as a three dimensional structure with a total of 264 pin connected elements and 237 degrees of freedom with 225 free nodal displacements.
2. The tower nodes and members connectivity were extracted and subsequently used as an input data in the written MATLAB program.
3. The dynamic analysis of the tower was carried out using the modal analysis technique.



2.1 GOVERNING EQUATIONS

The differential equation of a multi-degree of freedom system subjected to some dynamic load may be represented as

$$[M]\,\{\ddot{u}\} + [C]\,\{\dot{u}\} + [K]\,\{u\} = [F\,(t)] \quad (1)$$

where

$[M]$ represents the mass matrix
$[C]$ represents the damping matrix
$[K]$ represents the stiffness matrix
$[F(t)]$ represents the vector of externally applied force
$\{u\}$ represents the absolute nodal displacement
$\{\dot{u}\}$ represents the absolute nodal velocity
$\{\ddot{u}\}$ represents the absolute nodal acceleration

If the system has m degrees of freedom, the dimensions of $[M]$, $[C]$and $[K]$ will be an $m \times m$ matrix. Equation (1) is a set of second-order differential equations expressing the assembled finite element model of a structure subjected to a general (nonharmonic) forcing function.
In this study, damping of the system was ignored.
For a free undamped vibrating body, (1) reduces to;

$$[M]\,\{\ddot{u}\} + [K]\,\{u\} = \{0\} \quad (2)$$

For this simple, second-order differential equation, the solution may be obtained by assuming a trial solution

$$u_i = A\sin(\omega t) \quad (3)$$

Or

$$u_i = B\sin(\omega t) \quad (4)$$

Where 'A' and 'B' are constants depending on the initiation of the motion while ω is a quantity denoting a physical characteristic of the system.
Substitution of (3) or (4) into (2) gives

$$(-[M]\omega^2 + [K])A\cos\omega t = \{0\} \quad (5)$$

If this condition is to be satisfied at any time, the factor in parenthesis must be equal to zero, and since is an eigenvalue problem, the determinant must also be equal to zero

$$|(-[M]\omega^2 + [K])| = 0 \quad (6)$$

Equation (6) is the *frequency equation* of the system which is a polynomial of order n in the variable ω^2. The solution of the Equation results in N natural frequencies ω_j. In this research, consistent mass matrix was used and is represented here as (7). The stiffness matrix of each element used was that of a three dimensional truss element as presented in (8).

$$[M_C] = \frac{\rho A L}{6}\begin{bmatrix} 2 & 0 & 0 & 1 & 0 & 0 \\ 0 & 2 & 0 & 0 & 1 & 0 \\ 0 & 0 & 2 & 0 & 0 & 1 \\ 1 & 0 & 0 & 2 & 0 & 0 \\ 0 & 1 & 0 & 0 & 2 & 0 \\ 0 & 0 & 1 & 0 & 0 & 2 \end{bmatrix} \quad (7)$$

$$[k_o] = \alpha\begin{bmatrix} c_x^2 & c_{xy} & c_{xz} & -c_x^2 & -c_{xy} & -c_{xz} \\ c_{xy} & c_y^2 & c_{yz} & -c_{xy} & -c_y^2 & -c_{yz} \\ c_{xz} & c_{yz} & c_z^2 & -c_{xz} & -c_{yz} & -c_z^2 \\ -c_x^2 & -c_{xy} & -c_{xz} & c_x^2 & c_{xy} & c_{xz} \\ -c_{xy} & -c_y^2 & -c_{yz} & c_{xy} & c_y^2 & c_{yz} \\ -c_{xz} & -c_{yz} & -c_z^2 & c_{xz} & c_{yz} & c_z^2 \end{bmatrix} (8)$$

where

$$c_x = Cos\,\theta_x = \frac{x_2^o - x_1^o}{L} \quad (9)$$

$$c_y = Cos\,\theta_y = \frac{y_2^o - y_1^o}{L} \quad (10)$$

$$c_z = Cos\,\theta_z = \frac{z_2^0 - z_1^0}{L} \quad (11)$$

$$L = \sqrt{(x_2^o - x_1^o)^2 + (y_2^o - y_1^o)^2 + (z_2^o - z_1^o)^2} \quad (12)$$

$$\alpha = \frac{AE}{L}$$

A is the cross sectional area of the element
E is the elastic modulus and
ρ is the mass per unit volume of each element

2.1 MODAL ANALYSIS OF A STRUCTURE SUBJECTED TO BASE EXCITATION

The equation of motion of an undamped system subjected to base excitation may be represented by;

$$[M]\{\ddot{u_r}\} + [K]\{u_r\} = -[M]\{1\}\ddot{u}_s(t) \quad (13)$$

$\{\ddot{u_r}\}$ and $\{u_r\}$ represent acceleration and displacement relative to the base of structure, $\ddot{u}_s(t)$ is the applied acceleration at the foundation of the structure and {1} is a vector with all its elements equal to 1.

The system of differential equation can be uncoupled through the transformation;

$$\{u_r\} = [\emptyset]\{z\} \tag{14}$$

where $[\emptyset]$ is the modal matrix obtained in the solution of the corresponding Eigenproblem of (6).

The substitution of (14) into (13) followed by the premultiplication by the transpose of the ith Eigenvector, $\{\emptyset\}_i^T$ (the modal shape), results in;

$$\{\emptyset\}_i^T[M][\emptyset]\{\ddot{z}\} + \{\emptyset\}_i^T[K][\emptyset]\{z\} = -\{\emptyset\}_i^T[M]\{1\}\ddot{u}_s(t) \tag{15}$$

Upon the introduction of orthogonality property of the normalized Eigenvectors, the modal equation is obtained.

$$\ddot{z}_i + \omega_i^2 z_i = \Gamma_i \ddot{u}_s(t) \tag{16}$$

(i = 1,2,3,…….,N)

And Γ_i is the modal participation factor.

The solution to (16) is of the form

$$z_i = \frac{p_i}{\omega_i^2} \tag{17}$$

(Mario & William, 2004).

The nodal displacements are obtained from

$$\{u_i\} = [\emptyset_i]\{z_i\} \tag{18}$$

The maximum possible displacements at the nodal coordinates may then be estimated as the summation of the absolute values of the coefficients in (18).

2.2 STRESSES AND STRAINS

The dynamic strains and stresses were calculated using (19) and (20) respectively.

$$\{\boldsymbol{\varepsilon_d}\} = \frac{1}{L}\left[-c_x \ -c_{xy} \ -c_{xz} \ \ c_x \ c_{xy} \ c_{xz}\right]\{\mathbf{u_m}\} \tag{19}$$

The stress due to dynamic load $(\boldsymbol{\sigma_d})$ calculated for each element from

$$\{\boldsymbol{\sigma_d}\} = \frac{E}{L}\left[-c_x \ -c_{xy} \ -c_{xz} \ \ c_x \ c_{xy} \ c_{xz}\right]\{\mathbf{u_m}\} \tag{20}$$

2.3 MATLAB CODE

An excerpt of the written MATLAB code for the dynamic analysis is shown here.

```
[X,lm] = polyeig(K(isol,isol),-M(isol,isol)); % Eigenvalues
Angular_Frequency = sqrt(lm); %Angular frequency in rad/sec
Natural_Frequency = Angular_Frequency/(2*pi); %Natural frequency in cycle/sec
Period = 1./Natural_Frequency; % period in seconds
Eigen_Value = lm;
A1=(X)'; %transpose of the Eigenvector
I = ones (225,1); %Influence matrix %225 is the number of free nodes
Ground_acc = 0.3 * 9.81; %0.3 is the value of the ground acceleration g=9.810 m/s^2
G = Ground_acc*I;
P = -A1*M (isol,isol)*G;% force matrix
```

3 RESULTS AND DISCUSSION

The results of the analysis of the tower performed with the aid of MATLAB using the method of modal analysis are presented in this section.

Table 1 shows the Natural frequencies, Periods and Eigenvalues of the first 6 modes. It can be seen from the table that the period is inversely proportional to the frequency and vice versa. The first six modes are shown because in design, only the frequencies and periods of the first few modes are significant.

TABLE 1: RESULT OF NATURAL FREQUENCIES AND PERIODS

MODE	PERIOD (S)	FREQUENCY (RAD/S)	EIGENVALUE (RAD/S)^2
1	0.5839	10.7602	115.7813
2	0.5795	10.8426	117.5615
3	0.2782	22.5851	510.0880
4	0.1475	42.5869	1813.6
5	0.1404	44.7394	2001.6
6	0.1072	58.5889	3432.7

Table 2 presents the nodal displacements of the first five nodes. Node one is the topmost part of the tower and it has the maximum displacement because it's the farthest from the 'cantilevered' support.

TABLE 2: NODAL DISPLACEMENTS DUE TO GROUND EXCITATION

NODE	X-DISP (MM)	Y-DISP (MM)	Z-DISP (MM
1	89.142	88.915	1.037
2	73.769	73.718	7.448
3	73.769	73.717	4.640
4	73.766	73.717	4.635
5	73.766	73.718	6.440

Table 3 presents the strains and stresses induced in the specified members. The maximum compressive stress obtained was 613.53 N/mm^2 in member 97, while the maximum tensile stress obtained was 542.04 N/mm^2 in members 64 and 67.

TABLE 3: STAINS AND STRESSES

MEMBER	STRAIN	STRESS (N/MM^2)	MEMBER	STRAIN	STRESS (N/MM^2)
63	-0.0027	-538.34	97	-0.0031	-613.53
64	0.0027	542.04	113	0.0027	545.26
66	- 0.0026	-529.35	114	-0.0027	-541.72
67	0.0027	542.04	116	-0.0026	-528.58
68	0.0026	529.33	117	0.0027	537.16
69	- 0.0026	-512.72	118	0.0027	533.12
71	0.0025	506.96	119	-0.0026	-516.45

4 CONCLUSION

At the end of this study, the deformations and stresses in the tower members were obtained and tabulated as shown in section 3. The maximum displaced obtained in the X, Y and Z directions were 89.142 mm, 88.915 mm and 23.44 mm respectively. The maximum compressive stress and maximum tensile stress were 613.52 N/mm^2 and 542.04 N/mm^2 respectively. It is evident that the impact of the ground excitation of 0.3g which approximates the impact of the earth tremor is capable of producing large displacements in high rise structures, most especially, towers. In the analysis of a tower, the possibility of excessive deformation due to base excitation must be checked for the structure to be designed effectively without the risk of collapse.

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CORN HUSK ASH AS PARTIAL REPLACEMENT FOR CEMENT IN LATERITIC INTERLOCKING BLOCKS

*Oladipo, J. O.[1]; Raheem, A. A.[2]; Ogunjobi, S. O.[3]; & Ajayi, V. A.[4]
[1] Department of Civil Engineering Federal University of Technology, Minna
[2,3,4] Department of Civil Engineering Ladoke Akintola University of Technology, Ogbomoso
*Corresponding author email: oladipojamiuconsult@gmail.com

ABSTRACT
This study looked into the possibility of replacing the cement content of lateritic interlocking blocks with Corn Husk Ash (CHA). Production of lateritic interlocking blocks was carried out using locally fabricated steel mould of size 250 mm x 130 mm x 220 mm with variation in the percentage of cement and CHA in proportion of 2%, 4%, 6%, 8% and 10% with 0% being the control. Laterite used was obtained from Olomi area in Ogbomoso. The mixing ratio used for laterite and cement was 9:1. Blocks produced were cured and tested for abrasion, water absorption and compressive strength. Results showed that percentage mass abraded away increased from 0.92% to 28.28% as the percentage of CHA replaced increased from 2% to 10%. Also, 4% CHA absorbed 4.90% of water which is higher than 3.91% and 1.85% absorbed by 2% and 6% CHA respectively. The compressive strength increased from 0.95 N/mm^2 to 2.89 N/mm^2, 0.85 N/mm^2 to 2.62 N/mm^2, 0.86 N/mm^2 to 2.26 N/mm^2, 0.84 N/mm^2 to 1.78 N/mm^2 and 0.84 N/mm^2 to 1.67 N/mm^2 for 2%, 4%, 6%, 8% and 10% CHA lateritic interlocking blocks respectively, between the curing ages of 3 to 90 days. In conclusion, 4% CHA replacement was recommended for use in lateritic interlocking blocks since it met the minimum requirement for non-load bearing wall.

Keywords: *Cement, Compressive Strength, Corn Husk Ash (CHA), Lateritic Interlocking Blocks*

1 INTRODUCTION

Demand for cement in the developed countries is ever-increasing, which has caused a deficiency of cement on the world market causing elevated prices (PCA, 2004). Meanwhile, cement production generates approximately 7% of the total worldwide carbon dioxide emissions which contributes to global warming (Olutoge *et al*, 2010).

There have been various supplementary cementitious materials (SCMs) that are used in concrete including commonly used fly ash, blast furnace slag, and silica fume, while other SCMs are agricultural by-products such as rice husk ash, saw dust ash etc. Since cement is the most expensive fraction in concrete and difficult to obtain for third-world countries, other ash materials have been used to help extend cement supplies and allow more development. The use of saw dust ash along with naturally occurring metakaolin type clay has shown promise for low-cost concrete production in Africa (Elinwa *et. al.*, 2005). One study from Cuba used another waste material, this time from the burning of sugar cane husks leftover from sugar production. The results showed that sugar cane husks burned in the open-air produced a highly reactive pozzolanic material (Hernardez *et. al.*, 1998). The use of corn husk ash as a supplementary cementitious material would allow corn growing areas of the world to increase their building construction without corresponding increase in the cost of cement. Thus, there is need for engineering consideration for the use of cheaper and locally available materials to meet desired need to enhance self-efficiency, and lead to an overall reduction in construction cost for sustainable development.

Pozzolanic materials are siliceous and aluminous materials which are not cementitious in themselves, but when finely ground, contain some properties which at ordinary temperatures will combine with lime in the presence of water to form compounds which have a low solubility character and possess cementitious properties. Lateritic interlocking block is one of the products that Nigerian Building and Road Research Institute (NBRRI) introduced into the construction industry due to the fact that laterite is readily available in Nigeria and that it requires a very small quantity of cement, Raheem *et. al.*, (2012). Corn Husks are the outer membranous or green envelopes of an ear of corn. Corn Husk Ash is the ash residue generated from burning or heating of corn husks whether in an open air or under a controlled temperature in the oven.
There had been various research efforts on the use of agricultural wastes ashes and other pozzolan as replacement for cement in concrete. kevern *et. al.*, (2010) researched into corn husk ash as a supplementary

cementitious material in concrete and found out that mortar with up to 10% high-silica corn husk ash replacement for Portland cement produced comparative compressive strength values to mortar made with 100% Portland Cement and significantly higher compressive strength values (~20%) than samples produced using the regular corn husk ash. Pushpakumara *et. al.,* (2012) studied characteristics of masonry blocks manufactured with rice husk ash and lime and concluded that RHA lime based cement sand blocks have greater compressive strength compared to standard requirement and can be used for load bearing walls. Oyelade (2011) investigated into coconut husk ash as a partial replacement of cement in sandcrete block and concluded that agriculture wastes such as coconut husk ash does not show good pozzolanic property in the production of sandcrete blocks as the increment in the percentage of coconut husk ash content in the mix led to appreciable decrease in compressive strength value of 0.06N/mm^{2}at 30% coconut husk ash content. Olafusi *et. al.,* (2012) studied the Strength Properties of Corn Cob Ash Concrete and discovered that corn cob ash concretes do not attain their design strengths at 28 days. The strengths of corn cob ash concrete are dependent on its pozzolanic activities. This research work investigated into using Corn Husk Ash (CHA) as partial replacement for cement in lateritic interlocking block.

2 METHODOLOGY

Dried corn husks were collected from research farm, burnt into ashes at controlled temperature in a large cylindrical vessel using thermocouple to monitor temperature variation. It was ensured that the corn husks were totally burnt into fine ash for better reactivity. The resultant products was sieved, particles that passed through 45microns sieve size were used so as to meet up with fineness property for earlier pozzolanic reaction which in turn helps in early strength development as measured by standardized tests such as ASTM C-618/C-109 (Karen, 2006); while other ones retained on the sieve were discarded.

CHA samples were taken for pozzolanicity test at the West African Portland Cement Company (WAPCO), Sagamu, Ogun State. The chemical composition analysis was carried using X-ray fluorescent spectrometer in order to verify the suitability of the material as a pozzolan. Other physical properties such as colour, texture, fineness and specific gravity were also determined and detailed results were presented in table 4.1.

The laterite sample was air-dried for seven days in a cool, dry place. Air drying was necessary to enhance grinding and sieving of the laterite. After drying, grinding was performed using a punner and a hammer to break the lumps present in the soil. Sieving was performed to remove oversized materials from the laterite samples using a wire mesh screen with an aperture diameter of approximately 6 mm, as recommended by Oshodi (2004). Fine materials that passed through the sieve were collected for use, whereas those retained were discarded.

Cement used for this project work was Dangote Ordinary Portland Cement and it was procured from a cement vendor along Under G. LAUTECH road, Ogbomoso.

2.1 PRODUCTION OF LATERITIC INTERLOCKING BLOCKS

(i) Batching

The materials used for the production of lateritic interlocking blocks were measured by weight in accordance with the percentages of stabilization and percentage replacement (0%, 2%, 4%, 6%, 8% and 10%). The optimum moisture content was determined on the field according to what was stated in National Building Code (2006). The 0% CHA stabilized samples represented the control.

(ii) Mixing

The mixing was carried out on an impermeable surface free of all harmful materials which could alter the properties of the mix. The required quantity of laterite sample was measured and spread using a shovel to a reasonably large surface area. CHA was then be spread evenly on the laterite and mixed thoroughly with the shovel. The dry mixture was spread again to receive water which was added gradually while mixing, until the optimum moisture content of the mixture is attained. The optimum moisture content (OMC) of the mixture was determined by progressively wet the soil and take handful of the soil, compressed it firmly in the fist, then allow it to drop on a hard and flat surface from a height of about 1.10m. When the soil breaks into 4 or 5 parts, the water was considered right (National Building Code, 2006).

(iii) Casting/Compaction

The steel moulds were thoroughly cleaned before being coupled together and oiled to enhance the demoulding of the blocks. The wet mixture were filled into the mould in 3 layers, with each layer being compacted with 35 blows of 4.5 kg rammer on a level and rigid platform. The excess mixture was scraped off using a straight edge. Identification marks were inscribed on the blocks to allow easy referencing. The mould and its content were taken to the demoulding site for proper demoulding (Raheem *et. al.,* 2010).

(iv) Curing of the blocks

The blocks were first allowed to air dry for 24 hours under a cover of polythene sheet. Thereafter, water was sprinkled on the blocks in the morning and evening and the blocks were covered with polythene sheet for minimum of two weeks to continue the curing process and prevent rapid drying of the blocks, which could lead to shrinkage cracking. The blocks were later stacked in rows and columns with a maximum of five blocks in a column as recommended by Raheem *et. al.,* (2010) until they were ready for strength and abrasion test.

2.2 TESTING OF LATERITIC INTERLOCKING BLOCKS

1. Abrasion Test

The durability of the blocks was determined through abrasion testing. After the interlocking blocks attained the age of 28 days, three blocks were selected at random and weighed in the laboratory; their weights were recorded. The block samples were placed on a smooth, firm surface and all the surfaces were wire-brushed in a back-and-forth motion 50 times, where one back and forth motion was considered a single stroke. After being brushed, the blocks were then re-weighed to determine the amount of materials or particles abraded (Raheem et. al., 2012). This procedure was repeated for all the block samples produced with various cement - CHA content.

The percentage of block abraded away was estimated using the formula:

$$Wa = \frac{Wb - Wf}{Wb} \times 100 \quad (1)$$

Where: Wa = percentage of mass abraded away

Ws = weight of block sample before abrasion

Wd = weight of block sample after abrasion

2. Water absorption tests

Three block samples were randomly selected for each percentage replacement at 28 days of age and weighed on the weighing balance. These blocks were immersed completely in water for 24 hours after which they were removed and re-weighed (Raheem et. al., 2010).

The percentage of water absorbed by the block samples were estimated using the expression below:

$$Wa = \frac{Ws - Wd}{Wd} \times 100 \quad (2)$$

Where: Wa = percentage moisture absorption.

Ws = weight of soaked block.

Wd = weight of dry block.

3. Compressive strength test

Compressive strength test was performed to determine the load-bearing capacity of various block samples. The test was carried out at 3, 7, 21, 28, 56 and 90 days respectively for block samples of various CHA content. The weight of each block samples was first measured and recorded thereafter, the blocks were placed onto the compression testing machine one after the other, such that the top and bottom, as moulded, lied horizontally on a flat metal plate; the recesses filled with a metal plate of the exact size to prevent shearing of the block during testing. The blocks were crushed, and the corresponding failure loads were recorded (Raheem et. al., 2012). The compressive strength was obtained by dividing the crushing load by the sectional area of the block samples.

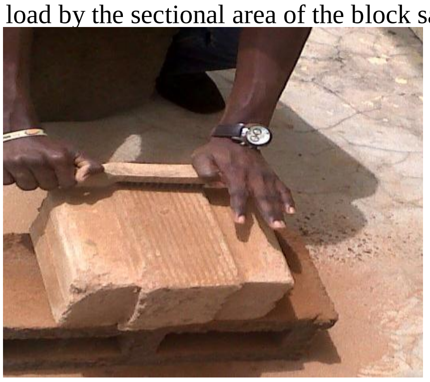

PLATE 1: ABRASION TEST FOR LATERITIC INTERLOCKING BLOCK

3 RESULTS AND DISCUSSION

1. Chemical Composition of Corn Husk Ash

The result of the pozzolanicity test carried out on the CHA sample used is shown in Table 4.1. The percentage composition of SiO_2, Al_2O_3 and Fe_2O_3 as shown in the table is 72.15%, 2.22% and 0.76% respectively, the sum of which gives 75.13% which satisfied the minimum requirement, specified in ASTM C-618 (2008), for a good pozzolan. The result is less than 87.55% obtained in Al-Khalaf et. al., (1984) for rice husk ash; Also, the result is higher than the value obtained for volcanic ash as it can be said to range between 63.74% as reported by Lar and

Tsalha (2005) and 67.14% by Hassan (2006). When compared with 73.07% obtained for saw dust ash in Raheem et. al., (2012) CHA is a better pozzolan. Thus, CHA is a more pozzolanic than volcanic ash and saw dust ash but less pozzolanic compared to rice husk ash.

TABLE 1: CHEMICAL COMPOSITION OF CORN HUSK ASH (CHA)

Chemical Constituents	Percentage Composition (%)			
	Sample 1	Sample 2	Sample 3	Average
SiO_2	72.20	71.15	73.10	72.15
Al_2O_3	2.20	2.15	2.30	2.22
Fe_2O_3	0.75	0.78	0.74	0.76
CaO	2.96	2.85	2.70	2.84
MgO	3.58	3.45	3.80	3.61
SO_3	0.58	0.73	0.50	0.60
K_2O	9.22	8.95	9.35	9.17
Na_2O	0.21	0.23	0.22	0.22
Mn_2O_3	0.09	0.11	0.08	0.09
P_2O_5	2.59	2.53	2.62	2.58
TiO_2	0.14	0.17	0.13	0.15
$SiO_2 + Al_2O_3 + Fe_2O_3$	75.15	74.08	76.14	75.13

2. Water Absorption Test

The results of water absorption of lateritic interlocking blocks are presented in Table 4.2. The results showed that block samples with 4% CHA absorbed the highest percentage of water among the samples tested. The 4% CHA block samples absorbed 4.90% which is higher than 4.16%, 3.91% and 1.85% absorbed by 0%, 2% and 6% CHA block samples respectively. When compared with the maximum value of 12% recommended in Nigerian Industrial Standard (2004), 0%, 2%, 4% and 6% CHA block samples satisfied the requirement. 8% and 10% CHA block samples were unable to absorb reasonable percentage of water as they dissolved in water before 24 hours.

TABLE 2: RESULT OF WATER ABSORPTION TEST FOR LATERITIC INTERLOCKING BLOCK SAMPLES

Percentage of CHA replaced	Average Mass before immersion in water (Kg)	Average Mass after immersion in water (Kg)	Average Water absorbed (%)
0	14.749	15.364	4.16
2	13.579	14.109	3.91
4	13.568	14.233	4.90
6	14.775	14.030	1.85
8	11.947	-	-
10	11.163	-	-

3. Abrasion Test Result

The abrasion test results for the block samples produced are shown in the Table 4.3. The result showed that percentage mass abraded away increased from 0.92% to 28.28% as the percentage of CHA replaced increased from 2% to 10% for the block samples tested which is an indication of decrease in durability of block samples tested up to 10% CHA. It can also be observed that the percentage mass abraded away in 2% CHA block samples has the least value of 0.92% out of all block samples tested including the control. It is an indication of higher durability of 2% CHA block samples when compared to that of control.

TABLE 3: RESULT OF ABRASION TEST FOR LATERITIC INTERLOCKING BLOCK SAMPLES

% CHA replacement	Average Mass before abrasion (Kg)	Average Mass after abrasion (Kg)	Average Mass abraded (%)
0	15.312	15.250	1.14
2	13.729	13.686	0.92
4	13.610	12.850	5.59
6	14.464	13.108	9.38
8	11.748	9.447	18.42
10	11.357	7.947	28.28

4. Compressive Strength Test

The results of the compressive strength of CHA lateritic interlocking block samples are plotted in Fig. 4.1. It was observed that the compressive strength increased from 0.95 N/mm^2 to 2.89 N/mm^2, 0.85 N/mm^2 to 2.62 N/mm^2, 0.86 N/mm^2 to 2.26 N/mm^2, 0.84 N/mm^2 to 1.78 N/mm^2 and 0.84 N/mm^2 to 1.67 N/mm^2 for 2%, 4%, 6%, 8% and

10% CHA lateritic interlocking blocks respectively between the curing age of 3 days to 90 days. The range of the compressive strength for the control is 1.39 N/mm^2 to 3.72 N/mm^2 during the same period. The minimum 7 days compressive strength for 5% cement stabilized blocks of not less than 1.60 N/mm^2 as recommended by National Building Code (2006) was not satisfied by the blocks tested except the control. Also, 2.00 N/mm^2 compressive strength for 5% cement stabilized manually produced blocks at 28 days as recommended by Nigerian Building and Road Research Institute (NBRRI, 2006) for non-load bearing walls was satisfied by 2% and 4% CHA block samples with compressive strength value of 2.03 N/mm^2 and 2.26 N/mm^2 respectively. It could be concluded that 4% CHA replacement for OPC is the optimum from compressive strength viewpoint.

Figure 4.1 showed the effect of curing ages on the compressive strength of CHA lateritic interlocking blocks. Generally, it can be observed that the compressive strength increases with increase in curing ages for all blocks tested. When compared with 28 days compressive strength of 1.53 N/mm^2 obtained for 10% cement stabilization in Raheem et. al., (2012), 4% CHA replacement compressive strength is higher. Also, compared to 1.25 N/mm^2 compressive strength recorded in Raheem et. al., (2010) for 10% lime stabilization at 28 day, 4% CHA value is higher. At 28 day, 2.16 N/mm^2 was obtained for 5% coconut husk ash as replacement for cement in Oyelade (2011), this value is less than what was obtained for 4% CHA replacement for the same period.

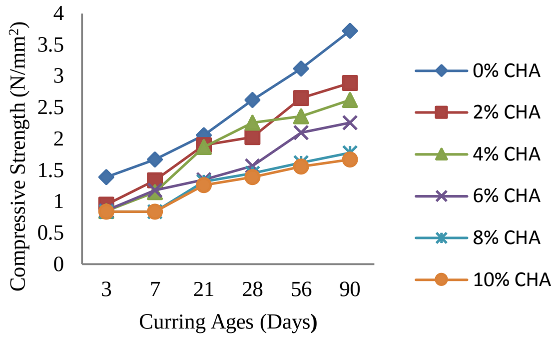

FIGURE 4.1: EFFECT OF CURING AGES ON THE COMPRESSIVE STRENGTH OF CHA LATERITIC INTERLOCKING BLOCKS

4 CONCLUSIONS AND RECOMMENDATION

Conclusions

Based on the findings in this study, the following conclusions were drawn:

1. Corn husk ash has the pozzolanic property required to replace cement in lateritic interlocking blocks.
2. 4% CHA replacement for cement in lateritic interlocking blocks is adequate as it shown high value of water absorption.
3. 4% CHA replacement for cement is suitable in lateritic interlocking blocks for non-load bearing walls.
4. 2% CHA replacement produced more durable lateritic interlocking blocks compared to the control.

Recommendations

1. 4% CHA lateritic interlocking blocks can be used as non-load bearing wall.
2. Further study should be carried out considering when hydraulic machine is used using same specification.
3. Study should be carried out on cost benefit analysis and labour required in production of lateritic interlocking blocks with CHA replacement compared to that of sandcrete blocks.

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INVESTIGATING SOME GEOTECHNICAL PROPERTIES OF OVERBURDEN SOIL WITHIN ABUJA IN NORTH CENTRAL NIGERIA

B. Gomna[1], M. Alhassan[1], and M. M. Alhaji[1]
[1]: Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author email: babagomna6@gmail.com

ABSTRACT
Boreholes were drilled to bedrock at ten different locations, along a stretch path, within the study area. Tests including, natural moisture content (NMC), specific gravity (SG), Liquid Limit (LL), Plastic Limit (PL) and sieve analysis were carried out with the aim of classifying the soils according to Unified Soil Classification System (USCS). Result of the study showed that the soil along the studied path is characterized by a cap of silty organic top soil, followed by clayey soil, which in some cases formed lenses along the path. Below the clayey soil layer is stratum of weathered silty soil, which extends to the parent rock.

Keywords: *Geotechnical properties, Overburden soil, Soil classification, Bedrock.*

1 INTRODUCTION

Almost all civil engineering structures are founded on soil. The stability of such structures therefore depends on the stability of the foundation soil. The stability of foundation soils for civil engineering structures depends on the geotechnical properties of the soils. Geotechnical properties of soils are appraised through geotechnical investigation. Investigating geotechnical properties of soils at sites proposed for civil engineering structures is therefore, an important prerequisite to successful design and construction of the structures.

Soil is overburden weathered material on parent rock. Overburden soil consists of loose, silt, sand and clay, which overlies the parent material (bedrock). It is uncemented or weakly cemented accumulation of mineral particles, which are formed by weathering of rocks, and contain void spaces between particles, which are filled by water and air (Craig, 1998). The geological formation of soil is based on rock weathering which occur either chemically, when minerals of rock are altered through a chemical reaction with rain water, or mechanically through climatic effects such as freeze-thaw and erosion (Gidigasu, 1976).

Geotechnical soil investigation is necessary because it provides the needed information on geotechnical characteristics, which help civil engineers to understand the strength and mechanical properties of soil in generating relevant data for design, assessment and construction of foundations for proposed structures (Nwankwoala and Warmate, 2014). It helps in adopting soil with properties that can be safely and economically used to avoid future construction errors. For successful geotechnical investigation of soil, it is necessary to be mindful of composite and complex nature of the weathering material and the variation in the morphological and geotechnical properties, both in depth and horizontal spans. This is most appreciated through knowledge of geology and geomorphology of the area.

2 GEOLOGY AND GEOMORPHOLOGY OF NIGERIA

Nigeria is geologically bounded on the south by the gulf of Guinea and on the north by the southern edge of the Sahara desert. The climate is characterized by hot tropical condition, which is humid in the south and semi-arid in the north. Seasonal rainfall results from the influence of the wet south westerly monsoon winds from the sea and the hot dry dusty north east trade wind from the Sahara, known locally as the hamattan (Alhassan, 2016).

According to Durotoye (1983), Rahman (1983), McCurry (1989), Rahaman (1989), the geology of Nigeria is dominated by sedimentary and crystalline Basement Complexes formations (Figure1), which occur in almost equal proportions all over the country. The sediment is mainly Upper Cretaceous to recent in age while the basement complex rocks are thought to be Precambrian.

According to Malomo (1983), products of weathering in Nigeria are generally grouped into four main basic classes (Figure 2): Ferruginous soils, Ferrallitic soils, weakly developed soils, and Vertisols, which is localized to the North-eastern part of the country. Due to the climatic conditions in the tropical region, soil formation from the parent rocks (igneous, sedimentary or metamorphic), is mainly by process of chemical weathering. Rahaman (1980) however, asserted that, mechanical process of weathering is also encountered within the tropics. The weathered material is usually residue of both chemical and physical process of rock weathering. The most important end products are clays and the resistant minerals, quartz. Other end products depend very much on the type of rock, however, in Nigeria, iron oxides are also common residues. They contain the reddish, brownish and yellowish coloration on the weathering residues, which are generally referred to as laterite (Durotoye, 1983). Most available residual soils in Nigeria are mainly ferruginous and ferrallitic tropical soils (Malomo, 1983).

The existing soil map of Nigeria only depicts soil types in different regions of the country. Many studies have been carried out on the properties of soils in Nigeria (Solanki, 2008; Ola, 1987; Farrington, 1983; Ajayi, 1983; Madedor and Lal, 1985; Adesunloye, 1987; Omange and Aitsebaomo, 1989; Abolarinwa, 2010; Nwaiwu and Nuhu, 2006; Mustapha and Alhassan, 2012; Adebisi and Adeyemi, 2012; Alhaji and Alhassan, 2013). Variations of the geotechnical properties of these soils with depth over a region (location) of the country have not been given much attention in the literature. This study therefore, focused on variation of geotechnical properties of soil with depth within Abuja in North Central Nigeria.

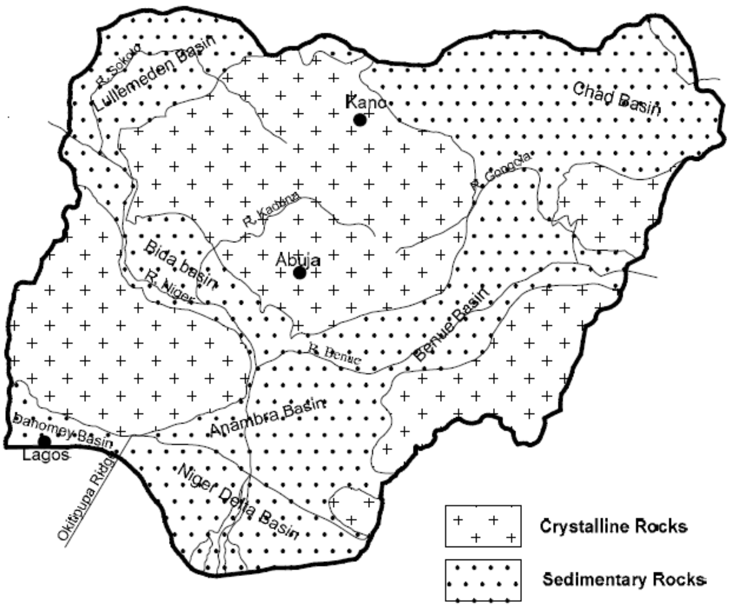

FIGURE 1: GEOLOGICAL FORMATIONS OF NIGERIA (ALHASSAN *ET AL.*, 2012)

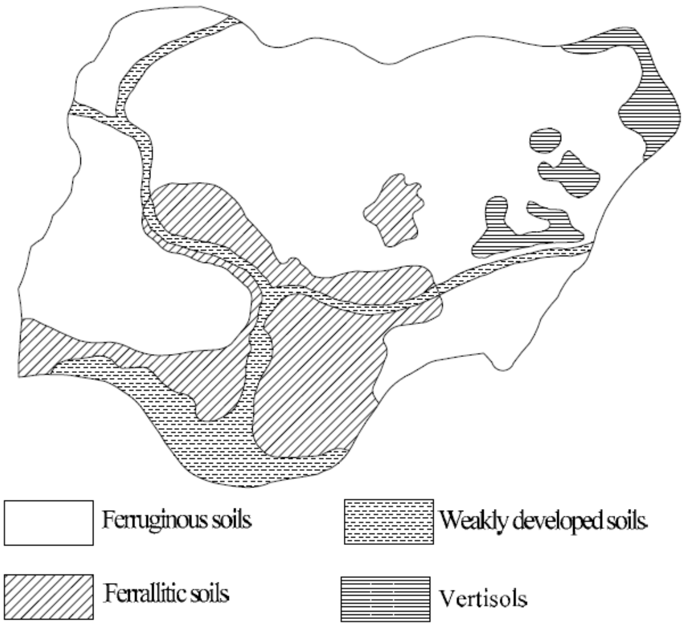

FIGURE 2: SOIL GROUP IN NIGERIA (ALHASSAN, 2016)

3 DESCRIPTION OF THE STUDY AREA

Abuja is Nigeria's capital territory, and is within North Central Nigeria. It lies between latitude 8.25° and

9.20° ***north*** of the equator and longitude 6.45° and 7.39° east of ***Greenwich Meridian***, with 536m as average elevation above sea level. The study line cut through the territories of Kwali and Gwagwalada area councils (Figure 3). Generally, major part of Abuja is geologically located in the basement complex terrain of Northern Nigeria, precisely belonging to the central Northern Block of Nigerian Basement Complex. In terms of rock type, the area is underlain by igneous and metamorphic rocks such as gneisses, Migmatites, Brotites, Granite etc. These rocks are of Precambrian age and are capped by sand and lateritic crust. Laterite is a residual soil found in tropical area with heavy rainfall and high temperatures. Soil map of Nigeria (Figure 2) indicates that the study area is dominantly characterized by ferruginous and partly ferrallitic soils. The study area experiences dry and rainy seasons, with mean annual rainfall of between 2000 to 2500mm. Rainfall in the area reaches its peak in mid-August/September. Temperature ranges between 20°C and less during the harmattan period to about 38°C in the dry season. The wind pattern is strongly from north east/east sector during the harmattan period and at onset/cessation of rainy season.

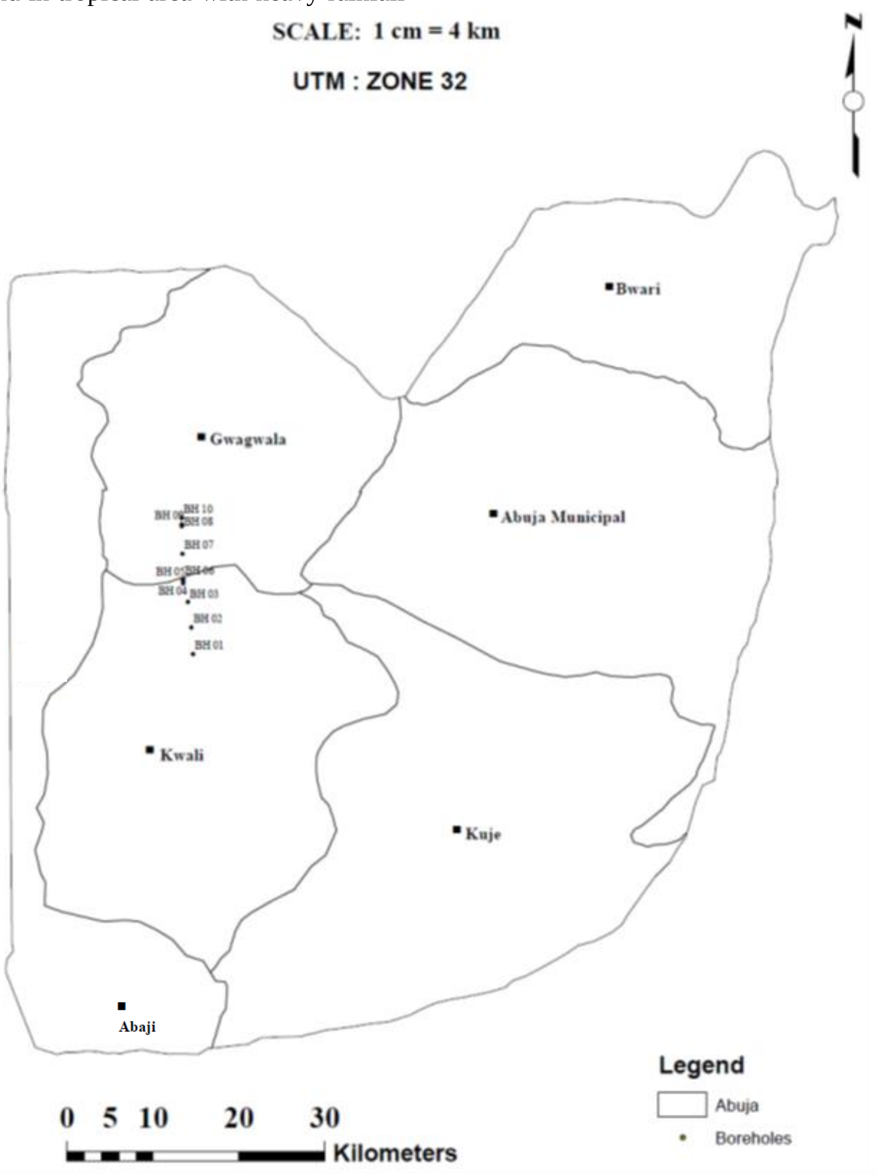

FIGURE 3: MAP OF ABUJA SHOWING BOREHOLES LOCATIONS

4 METHODOLOGY

Ten Boreholes (BH) were drilled to bedrock, at different locations, but along a stretched path, within the study area. Global Positioning System (GPS) was used to define the coordinates (Table I) and height above sea level for each location. Soil Samples were collected at 0.25 to 10m for the locations: BH 01, BH 02, BH 03, BH 04, BH 05, BH 06, BH 07, BH 08, BH 09, and BH 10. Tests including, Natural Moisture Content (NMC), Liquid Limit (LL), Plastic Limit (PL), mechanical and hydrometer sieve analysis, Specific Gravity (Gs) were carried out in Civil Engineering laboratory, Federal University of Technology, Minna in accordance with BS 1377 (1990). These tests were conducted with the aim of classifying the soils.

TABLE I: LOCATIONS OF THE BOREHOLES

Coordinates for boreholes (N° and E°)				
S/N	**Depths (m)**	**Boreholes**	**N°**	**E°**
1	10.00	BH01	274206	976278
2	5.25	BH02	274013	979375
3	5.25	BH03	273611	982304
4	10.00	BH04	273063	984553
5	6.75	BH05	27339.999	984899
6	8.25	BH06	273039	984987
7	5.00	BH07	272969	987977
8	5.00	BH08	272909	991128
9	3.00	BH09	272895	991469
10	5.00	BH10	272855	992114

5 RESULTS AND DISCUSSION

Results of the geotechnical properties tests, conducted on the collected soil samples are presented on Figures 4 to 7 and Table II.

From Figure 4, it is observed that the specific gravity of the soil within the studied area varies with depth at a particular borehole point, and also along the horizontal stretch of the studied path. The observed trend is as a result of geological processes of soil formation, which could have resulted to heterogeneity of the soil. Similar pattern of variation is also observed with Atterberg limits (Figure 5 to 7) of the soil in the suited area.

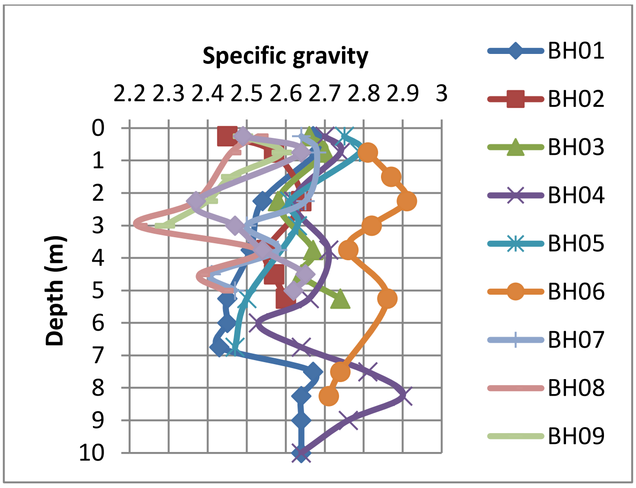

FIGURE 4: VARIATION OF SPECIFIC GRAVITY WITH DEPTH

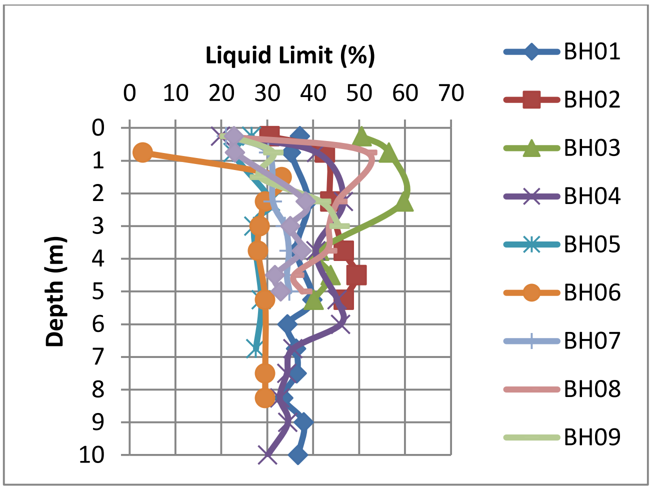

FIGURE 5: VARIATION OF LIQUID LIMIT WITH DEPTH

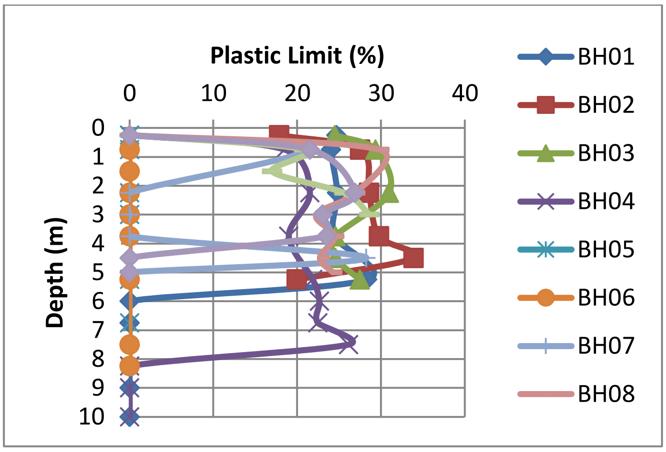

FIGURE 6: VARIATION OF PLASTIC LIMIT WITH DEPTH

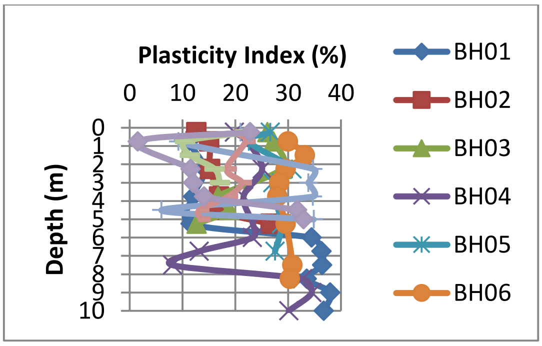

FIGURE 7: VARIATION OF PLASTICITY INDEX WITH DEPTH

From Figure 8, it is observed that the percentage of fines in soils within the studied area also varies with depth at a particular borehole location, and also along the horizontal stretch of the studied path. General observation of the variations showed fine content to be more within 2 to 5m depth, after which the percentage fine content gradually reduces down. This trend is as a result of soil typical formation process in the tropics, which have highly weathered rock, at the top, down through partially weathered to unweathered parent rock.

TABLE II: VARIATION OF SOIL CLASSIFICATION WITH DEPTH OF BOREHOLES

BH01		BH02		BH03		BH04		BH05		BH06		BH07		BH08		BH09		BH10	
Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS	Depth	USCS
0,25	SC	0,25	ML	0,25	MH	0,25	SM	0,25	SM	0,75	SM	0,25	SM	0,25	SM	0,25	SM	0,25	SM
0,75	GC	0,75	CL	0,75	ML	0,75	SC	0,75	SM	1,5	SM	0,75	GC	0,75	MH	0,75	SC	0,75	SC
2,25	CL	2,25	CL	2,25	MH	2,25	SC	2,25	SM	2,25	SM	2,25	SM	2,25	CL	1,5	CL	2,25	CL
3,75	CL	3,75	GC	3,75	ML	3,75	ML	3	SM	3	SM	3	SM	3	CL	2,25	SC	3	SC
5,25	CL	4,5	CL	4,5	SC	5,25	SC	5,25	SM	3,75	SM	3,75	SM	3,75	CL	3	CL	3,75	ML
6	GM	5,25	CL	5,25	SC	6	GC	6,75	SM	5,25	SM	4,5	CL	4,5	CL			4,5	SM
6,75	ML					6,75	SC			7,5	SM	5	SM	5	CL			5	SM
7,5	SM					7,5	SM			8,25	SM								
8,25	SM					8,25	SM			9	SM								
9	GM					9	GM			10	SM								
10	GM					10	GM												

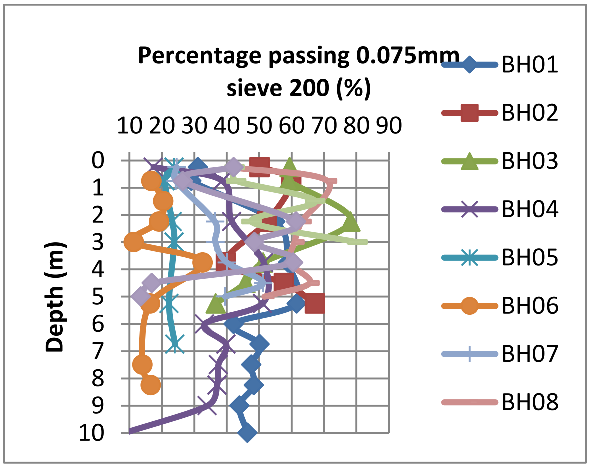

FIGURE 8: VARIATION WITH DEPTH OF PERCENTAGE PASSING 0.075MM SIEVE

Table II presents classifications of the soils according to Unified Soil classification System (USCS). From the table, with exception to BH01, which showed clayey sand, it is observed that, at 0.25m, the soil along the studied path is generally silty. Between 0.75 to 6.0m, soil at BH01, BH02, BH04, BH08 and BH09 ranges from clayey gravel, clayey sand and clay of low plasticity, while soils at BH05, BH06 and BH07 are generally silty sand. In most of the boreholes, from 6.75m downward, the soil was generally silty.

From the foregoing, it was observed that the soil along the studied path is characterized by a cap of silty organic top soil, followed by clayey soil, which in some cases formed lenses along the path. Below the clayey soil layer is stratum of weathered silty soil, which extends to the parent material.

6 CONCLUSION

The study revealed that the soil along the studied path is characterized by a cap of silty organic top soil, followed by clayey soil, which in some cases, formed lenses along the path. Below the clayey soil layer is stratum of weathered silty soil, which extends to the parent rock.

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ASSESSMENT OF DISPUTES RESOLUTION TECHNIQUES IN BUILDING CONTRACTS IN ABUJA, NIGERIA

* **Nwonumara – Paul**, O.[1]; **Saidu**, I.[2]
[1] Department of Quantity Surveying Federal University of Technology, Minna
Department of Quantity Surveying Federal University of Technology, Minna
*paulnwonumara@gmail.com[1], ibr,saidu@futminna.edu.ng[2]

ABSTRACT
The study aimed to assess disputes resolution techniques in Nigerian building contracts with the view to determining the most economical resolution methods. Adopting a survey research design to achieve the objectives of examining the common disputes encountered in building contracts and determining the most appropriate strategies for minimizing the disputes. Mean item score (MIS) was used to analyses the common disputes encountered in building contracts while relative importance index was used to rank most appropriated strategies for minimizing dispute in building contract. Financial related disputes and design related disputes were found to be the predominant dispute encountered while the best possible strategies for minimizing disputes are good communication between the parties involved in building contract, client clarity of objectives and promptness of payment, nipping disputes in the bud as soon as they emerge, effective documentations, cost and schedule control, quality management and constructability. Using the questionnaire as the instrument of data collection, the sample population were the registered construction professionals resident in Abuja, FCT, which totaled at 3399, with the sample frame consisting; Quantity surveyors, Architects, Engineers, Contractors through which the sample size of 345 was arrived using Krejcie and Morgan's table of sampling.
Keywords: *Assessing disputes resolution techniques, Construction disputes resolution methods, Disputes resolutions in building contracts, Effects of disputes in building.*

1 INTRODUCTION

The construction sector generally, in the world contribute to the realization of about 50 percent of the total capital, being the second employer in the country, it is also an engine for technology innovation and overall development (Ling, 2012). However, the construction industry is faced with complex issues of fact, and unexpected events (Leong, 2012), one of which is construction disputes. The disputation nature of the construction industry is not peculiar only to Nigeria but a universal problem. For instance, the construction industry in Gaza strip suffers from the misunderstanding of dispute resolution management. Over the years there has been a break down in the relationship between parties involved in the construction process which affected the development and expansion of construction sector (Guiene *et al*., 2013).
Dispute arises naturally from the construction process largely due to complexity of the project where players involved must coordinate their work in all stages of the design and development (Harris, 2013). The high cost of litigation and the lengthy time it takes to resolve dispute had made disputing parties seen to alternative dispute resolution to consideration as an alternative means in resolving disputes. Over the past decades, construction practitioners have tried to develop and implement the right contractual method which fit the best approach of their needs and minimizing dispute in construction project (Aibinu and Odeyinka, 2006).
Razaq (2010) have identified dispute as one of the major setbacks in the Nigerian construction industry. Consequently, resolving disputes has become an inevitable part of project delivery in today's complex and highly competitive construction industry. The complex nature of construction provides the catalyst for dispute in the construction industry. Construction projects are complex as they involve many human and non-human factor and variables. They characterized by long duration, diverse uncertainties and complex relationship (Whitield, 2012).
The nature of dispute in the construction industry is complex that if not properly managed, it can reduce productivity and escalated to prolong litigation (Okuntade, 2014). Dispute affects the success of the project and time over run, it drags a success process to failure. At the same time dispute may damage business between the parties and ultimately their future careers (Whitield, 2012).

2 STATEMENT OF PROBLEM

Large sums of money are spent annually by companies, government, corporations even government ministries on litigation (Mitkus and Mitkus, 2014).
Consequently, resolving disputes has become an inevitable part of project delivery in today's complex and highly competitive construction industry.
Construction disputes affect the interest of many stakeholders in connection with huge investments by reducing profit and are expensive. If the wheel of the vehicle of contractual and commercial transactions must move forward, Mitkus and Mitkus (2014) submitted that disputes must be resolved.

The major construction disputes still suffer in the hands of litigation there by resulting to wasting of materials, resources and abandonment of the projects. There is also problem of selecting the most appropriate resolution technique that can fit in the nature of dispute and best satisfy the disputing parties' needs nowadays, as the approaches are expensive.

Dispute, which remain unsolved may prevent completion of the project on time and at a determine time and quantity. In addition, disputes may results in project participants becoming inefficient and eventually lead to the failure of the project. The consequences of construction disputes have given rise to economic downturn in the construction sector due to lack of formal contractual arrangements and bad payment practices in the sector (Kilazsh, 2013). Delay payment, non-payment and condition payment namely 'pay when paid' and `pay if paid' have and continue to cripple the construction industry (Kilash, 2013).

The financial problems affecting some construction companies have in turn affected sub-contractors and suppliers, further downstream along the value chain. This affects the quality of work and livelihood of many people.

Whitified (2012) asserted that disputes are seen to be inevitable in the construction industry due to high differences of interest among the participants of construction projects. Due to arguments and counter arguments, distraction sets into the running of the project and eventual construction works. This leads to apathy on the part of parties to the contracts thus delay sets in.

3 JUSTIFICATION

Numerous studies have been carried out on disputes in construction contract. For instance, Okuntade (2014) conducted a research on the causes of disputes in construction contract. From the study, the main causes of dispute in construction contract are as a result of the actions, or inactions, of the client, the contractor the various consultants. Mitkus and Mitkus (2014) hypothesized that the true cause of construction related disputes is unsuccessful communication between the participants in a construction project.

In another study, Isah (2012) examined the causes of disputes in construction industry and identified that the major causes was delay. He used survey method to determine other causes. His findings showed that improper planning, lack and shortage of supply also contributed to delay in construction projects.

$$= \frac{3264.3996}{9.4554}$$

Yee and Abdul-Rahmman (2010) identify that clients are major cause of disputes in construction industry by deliberately delay the payment to the constructor for their own financial benefits especially in releasing the retention monies to contractor.

Adebayo *el-al.* (2009) stated that construction disputes originated from variety of sources. Ranging from unrealistic schedules and expections to changes in the economic situation.

Significance of filling the gap

This research will create awareness and provides guidance on the effective planning and efficient resolution of disputes in construction contracts by professionals.

The recommendation of the study, if properly implemented and used in construction contracts would give the best value for money to client. There would be minimum level of dispute in construction contracts.

4. RESEARCH METHODOLOGY

A survey design approach was employed in this research with the quantitative data gathered from the respondents using a questionnaire and the qualitative data through a semi-structure interview. The populations for this research work were the total 3399 registered construction professionals in Abuja, FCT. The sample frame consisted of the (Quantity surveyors = 569, Architects = 800, Engineers = 2000, and contractors = 30). Through which the sample size of (345) was arrived at, using Krejcie and Morgan table of sampling (at 95% confidence level) shown in question 3.1

According to Krejcie and Morgan (1970)

$$\mathbf{s = X^2 NP (1 - P) \div d^2 (N - 1) + X^2 P (1 - P)} \quad \text{...................................... 3.1}$$

Where;

s = sample size

X = based on confidence level 1.96 for 95% confidence was applied in this study

d = Precision desired, expressed as a decimal (i.e. 0.05 for 5% adopted for this study

P = Estimated variance in Population as a decimal (i.e. 0.5 used)

N= total number of population, 3399

$$s = \frac{1.96^2 \text{ x } 3399 \text{ x } 0.5 \text{ x } (1\text{-}0.5)}{(0.05^2 \text{ x } (3399 - 1) + 1.96^2 \text{ x } 0.5 \text{ x } (1\text{-}0.5))}$$

$$= \frac{3264.3996}{(8.495 + 0.9604)}$$

; $s = 345.242$

Therefore, s = 345

.

Abuja was chosen as a case study in this context because it is the capital city of Nigeria, and with a high presence of construction activities.

In order to guarantee equal representation for each of the identified groups in the population, stratified random sampling method was adopted. The respondents were first categorised into different strata then randomly sampled.

The value (345) was subjected to Krejcie and Morgan table of sampling (at 95% confidence level) shown in question 3.1

Demographic Characteristics of Respondents

Category	Classification	Freq.	%
Gender	Male	130	76.92%
	Female	39	23.08%
	TOTAL	**169**	**100.00%**
Years of experience	1 - 5 years	45	26.63%
	6 - 10 years	78	46.15%
	11 - 15 years	25	14.79%
	15 years and above	21	12.43%
	TOTAL	**169**	**100.00%**
Academic Qualification	OND	9	5.33%
	NCE	0	0.00%
	HND	49	28.99%
	Bachelor Degree	68	40.24%
	Master degree	41	24.26%
	PhD	2	1.18%
	TOTAL	**169**	**100.00%**
Experienced disputes in building contract before	Yes	167	98.82%
	No	2	1.18%
	TOTAL	**169**	**100.00%**

Source: Researcher's analysis (2018)

The result shows that most of the respondents sampled are male (76.92%) and females are (23.08%). A further look at the table shows that only 26.63% of the population have their year of working experience to fall with the 1 to 5 range, while 46.15% and 14.79% falls between the range of 6 to 10 and 11 to 15 years respectively. Also 12.43% of the population falls above 15 years. However, the average years of working experience of the respondents is calculated as approximately 7 years. This implies that these respondents have considerable numbers of years within the built environment, hence, should be able to give response to the research questions based on experience.

4 RESULTS AND DISCUSSION

Common Disputes in Building Contract

S/N	Variables	MIS	SD	Rank	Overall Rank
	Design related disputes				
1	Design error	4.32	0.941	1	3
2	Inadequate time for design	3.59	1.246	4	20
3	Inadequate/incomplete specifications	4.02	1.220	2	9
4	Availability of design related information	3.46	1.107	5	23
5	Inexperienced designer	3.73	0.911	3	17
	Financial related disputes				
6	Payment delays	4.54	0.715	1	1
7	Excessive claim made by contractors beyond client financial positions	3.50	1.230	5	22
8	Inadequate contractual financial provision to meet contract condition	4.24	0.750	3	6
9	Unnecessary bureaucracy in the payment process on the client's side	4.45	0.932	2	2
10	Delays originations from evaluation process of the contractors by the consultants	3.96	1.231	4	12
	Contractual claim related				
11	Variations	3.98	1.170	3	11
12	Errors in drawing	4.31	0.577	1	4
13	Incomplete tender information	4.20	0.760	2	7
14	Change of scope	3.89	1.008	4	14
15	Site condition	3.71	1.192	5	18
	Different interpretations of the contract provision related disputes				
16	Unforeseen changes	3.79	1.144	3	15
17	Errors in Specifications	3.79	1.128	3	15
18	Use of incomplete design during tender	3.89	0.939	2	13
19	Excessive contract Variations	3.57	1.335	5	21
20	Errors in drawings	4.01	0.768	1	10
	Change of scope related disputes				
21	Alterations to standard	3.23	1.024	4	24
22	Site conditions	3.64	0.997	3	19
23	Poor standard of workmanship	4.20	1.256	2	7
24	Design insufficient	4.25	0.756	1	5
25	Changes and distinction requirement	3.16	1.315	5	25

Source: Researcher's analysis (2018)

In assessing the common disputes in building contracts in Abuja, a total of twenty-five of the them were identified from literature and respondents were asked to rank these disputes based on their frequency of occurrence. Result in Table 4.3 shows the ranking of the identified disputes by the respondents. Result reveals that the top ranked disputes under each category of area as; for the design related disputes, design errors is ranked 1st with (MIS = 4.32) and this is followed by Inadequate/incomplete specifications in the 2nd position (MIS=4.02). Under the financial related disputes; payment delays (MIS=4.54) and Unnecessary bureaucracy in the payment process on the client's side (MIS=4.45) was ranked 1st and 2nd respectively. Errors in drawing (MIS=4.31) and Incomplete tender information (MIS=4.20) were ranked 1st and 2nd respectively under the contractual claim related disputes. For the Different interpretations of the contract provision related disputes; Errors in drawings (MIS=4.01) and (MIS=3.89) were ranked 1st and 2nd respectively. Similarly, Design insufficient with MIS=4.25 occupies the 1st position, and this is followed in the 2nd position by Poor standard of workmanship with MIS=4.20.

Strategies To minimize disputes in building contracts

S/No	Variables	RII	Rank
1	Mutual respect, Trust and team work	0.734	10
2	Good communication between the parties	0.888	1
3	On site dispute resolution and avoidance mechanism	0.850	6
4	Regular site meetings	0.852	5
5	Dispute resolution efficiency cycle	0.716	15
6	Promoting good working relationship	0.682	17
7	General management track of project progress	0.656	19
8	Trust and reputation among principal and agent	0.727	12
9	Spirit of partnership with a mutually agreed goals and objectives	0.703	16
10	Trust ethnics and cooperation in solving the problem of opportunism cost or agency costs	0.679	18
11	Collaboration between clients and the main contractors	0.727	12
12	Disputes avoidance approaches	0.762	9
13	Mutual goals and objectives of the parties to the contracts	0.615	20
14	Nipping disputes in the bud as soon as they emerge	0.862	3
15	Procurement and related processes to manage relationships so as to avoid disputes	0.725	14
16	Effective documentations, cost and schedule control, quality management and constructability	0.854	4
17	Prevention techniques prior to construction in order to minimize the impact of disputes that may arise	0.783	8
18	Regular interaction between the contractor and the project supervision on routine construction issues relating to correct interpretation of the project drawings, project specifications and other contract documents	0.830	7
19	Client clarity of objectives and promptness of payment	0.884	2
20	The construction clients should allow sufficient time for the design phase, so that designers are able to produce a comprehensive design.	0.730	11

Source: Researcher's analysis (2018)

The respondents were asked to rank the possible strategies for minimizing building contract disputes. Result in table 4.6 shows that the top strategies for minimizing construction contract disputes are good communication between the parties involved in building contract (RII = 0.888), client clarity of objectives and promptness of payment (RII = 0.884), nipping disputes in the bud as soon as they emerge (RII= 0.862), effective documentations, cost and schedule control, quality management and constructability (RII = 0.854), and regular site meetings involving all the stakeholder in a given project (RII = 0.852).
The least strategies according to the respondents are Trust ethnics and cooperation in solving the problem of opportunism cost or agency costs, General management track of project progress, and Mutual goals and objectives of the parties to the contracts.

However, a critical examination of the RII values showed that they are above 0.50. This implies that these strategies are effective when applied in building construction contract in preventing or minimizing the effects of disputes in contracts.

CONCLUSION AND RECOMMENDATION

Conclusion

This study set out to assess disputes resolution techniques in Nigerian building contracts with the view to determining the most economical resolution methods. The study utilized a survey design approach the study was able to ascertain the most common contract dispute, the economical dispute resolution techniques within building contracts in Abuja; and was able to determine the prominent disputes at various stages of the contract.

The area were disputes occur most in building contract Financial related disputes, design related disputes and Contractual claim related disputes. The prominent disputes in building contracts are Payment delays, unnecessary bureaucracy in the payment process on the client's side, design error, errors in drawing, and design insufficient these disputes have the tendency of influencing the performance of a construction contracts in terms of time, cost and quality. The most frequency of occurrence of disputes at planning, design and tender stages is the design related disputes. While for the estimating and construction stages is the financial related disputes; and for the Materials procurement and final account stages is different interpretations of the contract provision related dispute. The best possible strategies for minimizing disputes are good communication between the parties involved in building contract, client clarity of objectives and promptness of payment, nipping disputes in the bud as soon as they emerge, effective documentations, cost and schedule control, quality management and constructability, and regular site meetings involving the entire stakeholder in a given project.

Recommendation

From the findings and conclusion, the study makes the following recommendation

i. Designs should be completed prior to commencement of work on site, this will ensure that parties are aware of their respective financial commitment to avoid cash flow problems.
ii. The processes of getting money approved by the client or consultant should be reduced or minimized to ensure faster approval of valuations and payment
iii. Litigation should never be used in resolving any disputes in a contract. The use of mediation, negotiation among other less cost and time-consuming strategies should be used.
iv. The parties should be proactive in ensuring sources of disputes are eliminated prior to their emergence. This will be achieved by ensuring effective and efficient information flow, proper documentations of meetings and instructions, making discussions an important part of every meeting, early payment of valuation certificates, and clearness of objectives.

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EXPERIMENTAL AND FIELD EVALUATION OF A-6 LATERITIC SOIL STABILIZED WITH RECLAIMED ASPHALT PAVEMENT

*Umar, A.T.[1]; Alhaji, M.M.[2]; Adejumo, T.W.E.[3]
[1, 2, 3] Department of Civil Engineering, Federal University of Technology, Minna, Niger State.

ABSTRACT
Stability and strength are major properties investigated for materials used in road construction. Hence it is important for tests to be conducted on these materials before use. This paper evaluates the compaction characteristics and strength properties of Reclaimed Asphalt pavement mixed with lateritic soil (RAP-soil mixture) obtained from tests conducted in the laboratory and on the field. The tests were carried out on samples prepared by adding 20, 40, 50, 60, 80, 100, 120 and 140% RAP by dry weight of soil to lateritic soil and a clean sample with 0% RAP was also used as the control. The compaction test was carried out in the laboratory using the British Standard Heavy (BSH) compaction type. OMC obtained from the laboratory compaction was found to decrease from 12.3% at 0% RAP content to 6.72% at 120% RAP content. Although it further increase to 7.97% at 140% RAP content. The MDD obtained was found to increase from 2.163g/cm^3 at 0% RAP to 2.252 g/cm^3 at 120% RAP content after which it decreases to 2.231 g/cm^3 at 140% RAP content. The optimum RAP content of 120% obtained was used for the field experimentation and in-situ density obtained was found to be 2.282 g/cm^3 28 days after compaction. The CBR determined from the laboratory is 18.3%, while the value obtained on the field 2 days after compaction was 19.28%. This result yielded a strong correlation of 0.95 to validate the study's findings.

Keywords: *California Bearing Ratio, Compaction, Lateritic Soil, Reclaimed Asphalt Pavement, Stabilization*

1.0 INTRODUCTION

The use of quality and cost-effective materials for road construction with the aim of achieving the required design is of paramount importance to engineers. Stability of soil by adding industrial waste have been investigated by several authors in order to establish to what degree it can be manipulated so as to determine the required strength design. Mechanical stabilization improves soil properties by mixing other soil materials with the target soil to change the gradation and therefore change the engineering properties. It is also done to improve the load bearing capacity of a subgrade to support pavements and foundations. According to Bessa *et al*. (2015), the use of recycling technique in the rehabilitation of old pavement have become an important tool for pavement engineering practice.

Reclaimed Asphalt Pavement (RAP) is defined as pavement materials containing asphalt and aggregates which have been removed and reprocessed. These materials are generated when asphalt pavements are removed for reconstruction, resurfacing, or to obtain access to buried utilities. When properly crushed and screened, RAP consists of high-quality, well-graded aggregates coated by asphalt cement (Jirayut *et al.*, 2014). Pradyumna *et al*., (2013) concluded from their research that Asphalt mixes with 20% RAP performs better than virgin Asphalt mixes as it improves the properties of bituminous mixes.

Several authors have used RAP in stabilization of soils and their properties were reported to improve. According to Jirayut *et al*., (2014) an increase in RAP content in laterite soil- RAP mixture decreases the OMC to an optimum soil/RAP ratio of 50/50. Results obtained from compaction tests carried out by some authors have also shown that increase in RAP content increases the MDD of RAP- soil mixtures while a decrease it decreases their OMC (Edeh *et al*., 2012; Mustapha *et al*; 2014; Ochepo, 2014)

Ochebo, (2014) reported from his research on Stabilization of laterite soil using reclaimed asphalt pavement and sugarcane bagasse ash for pavement construction that addition of RAP to laterite soil reduces the Optimum Moisture Content (OMC) and increases the Maximum Dry Density (MDD) as compared with the natural soil.

Laterite soils can be used as base course for roads, in some cases, without any improvement. These soils may contain substantial amount of silica in the form of clay silicate minerals and could affect its strength and stability. Sourcing for alternative suitable soil for road construction maybe too expensive in areas where deposits of these laterites exist. Therefore, stabilizing the available laterite to meet the desired strength and stability may be necessary (Mustapha *et al.*, 2014).

Numerous research works have also been carried out on stabilization of laterites with other industrial wastes such as Bagasse ash and sugar cane straw ash.

2.0 MATERIALS AND METHODS

2.1 MATERIALS

2.1.1 Lateritic Soil

The lateritic soil sample used for this study was collected from a borrow pit at Gidan kwanu main campus of Federal University of Technology Minna, Niger State, Nigeria, along Minna-Bida Road. Disturbed samples were collected from the borrow pit. The soil was air dried, and index properties test was conducted on the natural soil collected Plate I.

PLATE I: PREPARATION OF LATERITIC SOIL SAMPLE

2.1.2 **Reclaimed Asphalt Pavement** (RAP)

The reclaimed asphalt pavement material used in this study was obtained from a scarified discarded pavement surface along Suleja-Minna Road (47Km from Suleja) in Niger State, Nigeria on latitude 9˚23′30" N and longitude 6˚58′0" E. The RAP was milled using milling machine. Substantial amount of RAP was collected, pulverised and sieved through sieve 5.0mm, Plate II.

Plate II: Pulverized Reclaimed Asphalt Pavement

2.2 METHODOLOGY

2.2.1 Sample Preparation

The soil was air dried in the laboratory and the mix were prepared by adding 0, 20, 40, 50, 60, 80, 100, 120 and 140% RAP by dry weight of the lateritic soil to the laterite. The samples were thoroughly mixed and a total of nine samples were gotten with the sample containing 0% RAP as the control mix for the study.

2.2.2 Compaction Test

The optimum moisture content (OMC) and maximum dry densities (MDDs) of the mixtures were obtained by carrying out compaction tests in the laboratory in accordance with BS 1377 (1990). The control mix specimen containing only the lateritic soil material and the remaining seven specimens were compacted using British Standard Heavy (BSH) compactive efforts. The BSH or modified proctor compactive effort utilized 27 blows of the 4.5 kg rammer falling freely from a height of 450mm onto 5 layers of the soil in a 1000 cm^3 mould. The dry densities and moisture contents were plotted on a graph to obtain the OMC and MDD of the specimens.
Compaction is also carried out on field using a roller. The compaction is carried out at optimum RAP-laterite content which corresponds to the mixture with the highest OMC. The in-situ density of the mixture is obtained on the first day of compaction and at different intervals of 28, 60 and 90 days. Results obtained from field in-situ density are hereafter compared to the MDD of the optimum mixture obtained from the laboratory test conducted Plates III – VI.

PLATE III: FIELD COMPACTION OF LATERITE-RAP MIX


PLATE IV: FIELD COMPACTION OF LATERITE-RAP MIX


PLATE V: IN-SITU DENSITY TEST OF LATERITIC SOIL SAMPLE


PLATE VI: IN-SITU DENSITY TEST OF RAP

2.2.3 California Bearing Ratio

It involves penetration of the moulded soil sample (optimum mixture) with a cylindrical plunger at a constant rate of 1 mm/min. The force corresponding to penetration of 5.0mm is computed and then compared to the standard force attained from the field using the dynamic cone penetration test. The CBR from the field was carried out at periodic intervals of 2, 7, 14, 28, 60 and 90 days for comparison with that obtained from the laboratory Plate VII.


PLATE VII: FIELD DYNAMIC CONE PENETRATION TEST

3.0 RESULTS AND DISCUSSION

Preliminary tests were conducted in the laboratory in order to determine the index and strength properties of the natural A-6 Lateritic soil. Results obtained are shown in Table I. Sieve analysis was also carried out for RAP to determine its particle size distribution. Results obtained showed it contains 24% sand and 68% gravel particles Figure 1.

TABLE 1: INDEX AND PHYSICAL PROPERTIES OF LATERITIC SOIL AND RAP

Properties	RAP	Lateritic soil
Natural moisture content (%)	0.27	3.43
Liquid limit (%)	-	39.36
Plastic limit (%)	-	24.42
Plastic index (%)	-	14.94
Specific gravity (%)		
Percentage passing BS 200 sieve (%)		48.04
AASHTO classification	-	A-6
USCS classification		CL
Maximum dry density (Mg/m3)		2.163
Optimum moisture content (%)		12.3
California Bearing Ratio (%)		
Colour	Black	Brownish

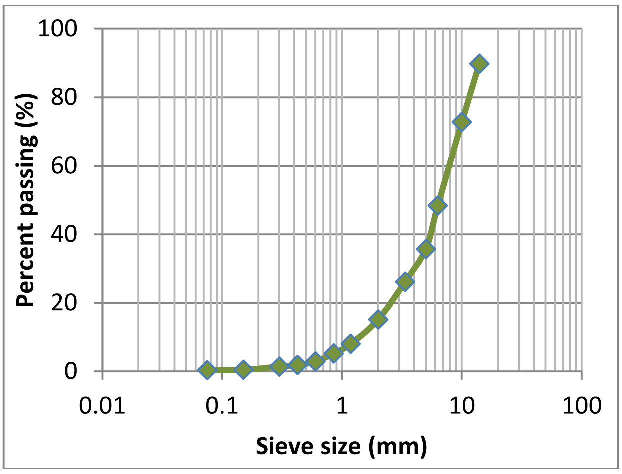

FIGURE 1: PARTICLE SIZE DISTRIBUTION OF RAP

3.1 Compaction Characteristics

Results obtained from the laboratory compaction test are shown on Figures 2 and 3. The OMC is seen to decrease from 12.3% at 0% RAP content to 6.72% at 120% RAP content and hereafter increases to 7.97% at 140% RAP content. While the MDD increases from 2.163g/cm^3 at 0% RAP content to 2.252g/cm^3 at 120% RAP content and then decreases to 2.231g/cm^3 at 140% RAP content. Optimum RAP content is attained at 120% RAP with the highest MDD value of 2.252g/cm^3 and the least OMC value of 6.72%.

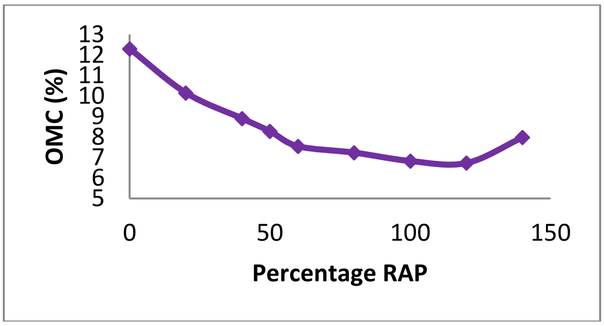

FIGURE 2: VARIATION OF OMC WITH RAP CONTENT

The decrease in OMC is as a result of increase in RAP content of the mixture. This was attributed to the increase in coarse particles from the RAP, and thereby needing less water to lubricate the mixture. The increase in MDD of the mixtures was attributed to fine particles of the laterite soil filling the void space among the coarse particles of the RAP, which resulted in the formation of a denser matrix (Ochepo, 2014).

The in-situ density obtained from field at various intervals as shown in Table II was also compared to the MDD corresponding to the optimum mixture. Result showed that the in-situ density with a value of 2.282g/cm^3 obtained after 28 days of compaction was closer to the MDD of the mixture obtained in the laboratory with value 2.252g/cm^3.

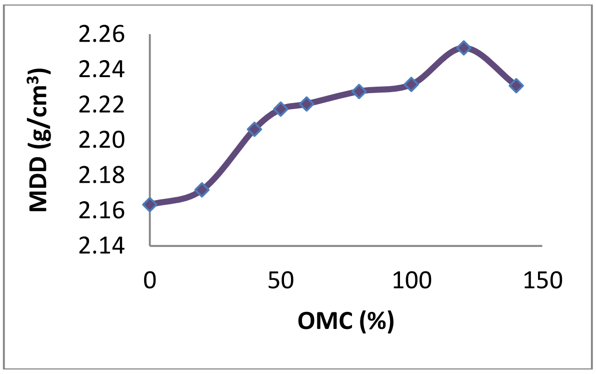

FIGURE 3: VARIATION OF MDD WITH RAP CONTENT

TABLE II. IN-SITU DENSITY OBTAINED AT VARIOUS INTERVALS

Number of days	In-situ density(g/cm^3)
1	2.037
28	2.282
60	2.116
90	2.007

3.2 Strength Properties

The CBR obtained from the laboratory at a penetration of 5.0mm was found to be 18.3% and this was compared to that obtained at a depth of 25mm on the field after compaction. The CBR result obtained from the field at various intervals is shown in Figure 4. The result obtained from the field CBR after 2 days with value 19.28% was found to be closer to that obtained in the laboratory. The CBR value decreases further for the first 28 days after which it begins to increase till 65 days and then decreases till 90 days.

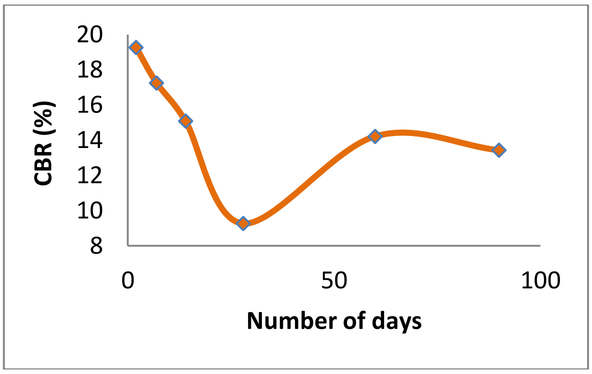

FIGURE 4: VARIATION OF CBR VALUES WITH NUMBER OF DAYS.

4.0 CONCLUSION

The following conclusions have been reported from this research work.

1. The increase in RAP content of RAP-soil mixtures increases the MDD from 2.163g/cm^3 at 0% to 2.252g/cm^3 at 120% RAP and decreases to 2.231g/cm^3 at 140% RAP. The OMC decreases with increase in RAP content from 12.3% at 0%

RAP to 6.72% at 120% RAP and then increases to 7.97% at 140% RAP. The optimum mixture used in the field was found to be the mixture containing 120% RAP with MDD value of 2.252g/cm^3, while the in-situ density obtained in the field 28 days after compaction with value 2.282g/cm^3 was found to be closest to that obtained from the laboratory.

2. The CBR value of 19.28% obtained in the field 2 days after compaction was found to be the closest to that obtained in the laboratory with value 18.3%.

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EFFECT OF INOCULUM ON CO-DIGESTION OF CHICKEN DROPLET AND FOOD WASTE FOR BIOGAS PRODUCTION

Saidu. M[1*]., Ibrahim, A. M[1]., Adesiji, A. R[1], and Gbongbo, J[2].
[1]Department of Civil Engineering, Federal University of Technology Minna, Nigeria.
2 Department of Civil Engineering, Niger State Polytechnic, Zungeru, Niger State, Nigeria.
*Corresponding email: mohammed_saidu@futminna.edu.ng

ABSTRACT
Increased in Population brings about high demand of energy, the available source of energy is seriously reducing. The need for alternative sources of energy is vital. This paper looked at Chicken Droplet (CD) and Food Waste (FW) as sources of renewable energy. The best system of utilizing this waste is by anaerobic digestion of the substrate. In this study, Three (3) no of six (6) liters batch anaerobic reactors are fed with Chicken Droplets (CD), Food Waste (FW) and Inoculum (I) of Rumen fluid at various mixtures (100%CD, 70%CD: 30%FW & 100%I, 30%C: 70%FW & 100%I). The digestion process took place at room temperature for a thirty (30) day period, the lag phase for these reactor were six (6) days for 100%CD and four (4) days for both 70%CD: 30%FW & 100%I and 30%C: 70%FW & 100%I. This anaerobic digestion process produced biogas in all the three (3) with 70%CD: 30%FW & 100%I having highest volume of 0.1litres. The final value of Total solids for 100%CD was 682mg/L, 70%CD: 30%FW & 100%I was 633mg/L 30%C: 70%FW & 100%I was 660mg/L. Volatile solids for 100%CD was 340mg/L, 70%CD: 30%FW & 100%I was 447mg/L, 30%C: 70%FW & 100%I was 462mg/L, Total Nitrogen for 100%CD was 3.2%, 70%CD: 30%FW & 100%I was 3%, 30%C: 70%FW & 100%I was 1.7%, Organic Carbon content for 100%CD was 10%, 70%CD: 30%FW & 100%I was 9.8%, 30%C: 70%FW & 100%I was 13.55% these values were reduced because of anaerobic digestion compared to their initial value before anaerobic digestion. From the result analysis anaerobic co-digestion improved the volume of biogas produced where reactor D2 of Chicken Droplet (CD) 70%: Food Waste (FW) 30% & Inoculum (I) 100% and D3 30%CD: 70%FW & 100%I produced more biogas volume than mono digestion. Thus, Chicken droplets and food waste are good sources of biogas production.

Keywords: *Anaerobic, Chicken, Food waste, Rumen fluid*

1 INTRODUCTION

Increase of municipal solid waste in developing countries due to population increase has raised concern with respect to the best system of disposal and treatment of such waste. Water pollution and adequate energy resources affect economic human development and environmental health (Mshandete and Parawira, 2009).

Population increase brings about high demand of energy, and the wastes generated are good alternative sources of energy which is renewable. The best system of utilizing these wastes is by anaerobic digestion of the substrate. Anaerobic digestion of agricultural waste produces a by-product called biogas. Anaerobic digestion is a biochemical process whereby organic matter is first decomposed by acidic bacteria and the process is completed by bacteria which thrive in slightly alkaline environment to produce methane and other by products in the absence of oxygen (Chukwuma, 2012).
Biogas is a clean source of energy used for various purposes such as: heating, cooking, transport and power generation. Biogas usually contains about 55-65% methane, 30-35% carbon dioxide, and trace amount of hydrogen, nitrogen and other impurities (Ojikutu and Osokoya, 2014). Biogas is produced from biological breakdown of organic matter (kitchen waste, dead animals, human excretal etc.) in the absence of oxygen (Ojikutu and Osokoya, 2014). In this research, food waste and chicken droplet were used as anaerobic digestion substrate.
The aim of this study determined the effect of using Rumen fluid as inoculum on the rate of biogas production from anaerobic digestion of Chicken Droplet and Food Waste.

2 MATERIALS AND METHODS

Chicken Droplets was collected from the Farm of Niger State Ministry of Agriculture and livestock Bosso, Minna, Niger state, while Food wastes was collected from Federal University of Technology (F.U.T) Minna Main Campus food canteen and the Rumen fluid (Inoculum) was taken from Minna abattoir and were taken immediately to Water and Fishery Technology (WAFT) laboratory at F.U.T. Main Campous, Minna, for analysis.
Samples from Farm of Ministry of Agriculture and livestock Bosso of fresh poultry manure (Chicken Droplets) were taken from 5-cage layers house, to get representative result during analysis.

The following parameters were determined in the laboratory using standard method; total solid (TS), volatile Solid (VS), Biological Oxygen Demand (BOD), Total Nitrogen, Organic Carbon and Ammonia- Nitrogen Content. pH using standard method.

This research was conducted for a thirty (30) days. Three (3) no of six (6) liters batch tank reactors are fed with Chicken Droplets (CD) and Food Waste (FD) in the following manner;

First digester named D1 contained 1500g Chicken droplets (CD) weighed with a weight balance which equals 100% was mixed with one thousand grams (1000g) of water to ensure homogeneity of mixtures which made a total of one thousand seven hundred grams (1700g) volume in the reactor; this was ascertained using a S.PYREX 1000 milliliters (ml) Measuring cylinder. The temperature and pH reading was taken before substrate was charged into the reactor.

The second digester named D2 contained 1100g Chicken droplets (CD) also weighed which equals 70%, Five hundred gram (500g) food waste (FW) which is 30% and five hundred gram (500g) of Inoculum (Rumen fluid) which is 100% these were mixed with one thousand five hundred grams (1500g) of water forming a total of two thousand five hundred gram (2500g) volume in the reactor. The temperature and pH reading was taken before substrate was charged into the reactor.

Third digester named D3 contained five hundred gram (500g) chicken droplets (CD) which is 30%, one thousand one hundred (1100g) of food waste (FW) which is 70% and five hundred (500g) grams of Inoculum (Rumen fluid) which is 100%. These were mixed with two thousand gram (2000g) of water forming a total of two thousand eight hundred gram (2800g) volume in the reactor as shown in table 2.1. The temperature and pH reading were taken throughout the study period of thirty days (30) after substrates were charged into the reactor. Readings were taken at two (2) days interval to ascertain pH nature and temperature range at which the reactor is digesting substrates to know if the reactor was at optimal condition.

TABLE 2.1 DIGESTION DESCRIPTION IN REACTORS

Digester	Chicken Droplets (kg)	Food waste(kg)	Inoculum(Liter)	Volume in Reactor (Liter)
D1	**1.5**	**0**	**0**	**1.7**
D2	**1.1**	**0.5**	**0.5**	**2.5**
D3	**0.5**	**1.1**	**0.5**	**2.8**

3 RESULTS AND DISCUSSION

The lag phase of various mixtures was different because of different composition of the substrates shown in the Table 3.1. The table describe the lag phase of various mixture and the control.

TABLE 3.1: LAG PHASE OF SUBSTRATES

Mixing Ratio	100%CD	70%CD, 30%FW & 100% I	30%CD, 70%FW 100% I
Period(Days)	**6**	**4**	**4**

3.2 Characterisation of Substrates

Total solids (TS), volatile solids (VS), Biological Oxygen Demand, Total Nitrogen, Organic Carbon and Ammonia Nitrogen (NH_3-N) were carried out using the standard methods (APHA., 2005). From these result volatile solids had the highest reduction percentage as shown in Table 3.2 this showed that volatile solids gives a measure of organic matter available for biogas production. Dupade and Pawar, (2013) reported result that treatment of food influent using micro-organisms from anaerobic digestion produced useful bi-product, biogas with a considerable rate of decrease in the values of COD, BOD, pH, acidity and alkalinity, through the successful anaerobic digestion inside their reactors for 90days.

TABLE 3.2: RESULTS OF SUBSTRATES CHARACTERISATION FROM WATER AND FISHERY TECHNOLOGY LABORATORY.

Test Conducted	Chicken Droplet (CD 100%)influent	Chicken droplet (70%) & Food Waste (30%) Influent	Chicken droplet (30%) &Food Waste (70%)Influent	Chicken Droplet (CD 100%)Effluent	Chicken droplet (70%) & Food Waste (30%)Effluent	Chicken droplet (30%) &Food Waste (70%)Effluent
Total Solids (mg/L)	**975**	**1055**	**944**	**682**	**633**	**660**
Volatile Solids (mg/L)	**680**	**746**	**834**	**340**	**447**	**462**

Total Nitrogen (%)	**4.06**	**3.55**	**2.88**	**3.2**	**3**	**1.7**
Organic Carbon (%)	**19.95**	**19.66**	**19.25**	**10**	**9.8**	**13.55**
Ammonia-Nitrogen Content(NH_3-N)mg/L	**1.4**	**1.19**	**0.91**	**1.2**	**0.8**	**0.5**
Biological Oxygen Demand(mg/L)	**15**	**13.2**	**10.8**	**9**	**6.6**	**7**
pH	**7.64**	**6.85**	**7.64**	**5.5**	**5.9**	**5.0**
Temperature (°C)	**32.1**	**31.2**	**30.5**	**36.5**	**33.8**	**38.2**

Substrates influents of 100% CD: 70% CD & 30% FW: 30%CD & 70% FW had reductions in total solids, volatile solids, ammonia nitrogen content and biological oxygen demand in mg/L after anaerobic digestion. Similarly total nitrogen and carbon content of substrates influents of 100% CD: 70% CD & 30% FW: 30%CD & 70% FW were reduced after anaerobic digestion process. This is similar to study conducted by Velmurugan and Alwar, (2011) that anaerobic digestion of vegetable waste which is a food waste had percentage reductions of 60.77% and 68.6% in total solids and volatile solids respectively.

Oliveira and Doelle, (2015) result also reported that volatile solids reduce by 81% at twenty-eight (28th) day of anaerobic digestion. Tamrat *et al*, (2013) result from Co-digestion of cattle manure with organic kitchen waste to increase biogas production using rumen fluid as inoculums also proved anaerobic digestion reduces total solids, volatile solids.

The figure 1 shows production of biogas from 100% chicken droplet, started on the sixth (6th) day which rose slowly to 0.062 litres on the twenty-second (22) day, reduced to 0.058 litres on the twenty-fourth (24) day and increased to 0.06 litres on the twenty-sixth (26) day. Volume of the biogas declined gradually as number of days increased. This could be as a result of exhaustion of volatile solid content present in the substrates which is a measure of organic matter available for production of biogas (Sidik, *et al* 2013). It was observed that daily biogas production from 100% chicken droplet was lower than both 70% chicken droplet and 30% food waste and 70% food waste and 30% chicken droplet. Molinuevo *et al*, (2013) reported chicken manure digested singly produced lower biogas volume compared to co-digestion with food waste. This study showed pure chicken droplets (Controlled) produced lower volume of biogas when compared to the other reactors where co-digestion was achieved with presence of inoculum.

Biogas produced from 70% chicken droplet and 30% Food Waste started on the fourth (4th) day, a reduced time compared to mono digestion without presence of inoculum. This volume rose slowly to 0.1 liters. Production of biogas decreased slightly till 0.035litres, this volume of 0.1 liters biogas which is the highest of all three (3) reactors could also result from inoculum presence or adequate proportion mixture with water during preparation. Previous study from Ravi *et al*, (2013) confirms this where one of the reactors had a mixture of eight kilograms (8kg) of substrate to sixteen (16) liters of water digested singly without inoculum, with methane percent of (48%). Hence this study proved inoculum improves percentage of methane in biogas because methane percent in 70% chicken droplet and 30% food waste co-digestion had fifty-five (55%) percent methane.

Lag phase for biogas generation in reactor D2 (70% Chicken Droplet & 30% Food Waste) and D3 (30% Chicken droplet & 30% Food Waste) was shorter compared to reactor D1 (100% Chicken Droplet), this could result of inoculum added to both reactor. It is possible the presence of this inoculum contributed to the increase in biogas volume of reactor D2 and D3 which had the highest volumes. However, it should be noted that the digestive system of the animal will influence the amount of organic matter in its faeces (Ahmadu *et al*, 2009). Biogas production from 70% chicken Droplet & 30% Food Waste could also be attributed to the available nutrients in both substrates as well as co-digestion as shown figure 3.1.

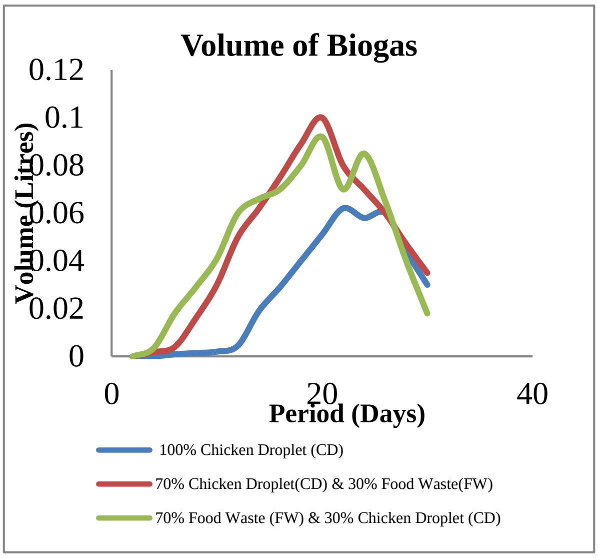

FIGURE 3.1: VOLUME OF BIOGAS PRODUCED

4. CONCLUSION

In this study inoculum (rumen Fluid) reduced lag phase for biogas production from Chicken Droplet and Food Wastes of different mixtures, compared to the control. Therefore, Co-digestion improved the volume of biogas produced. And reactor D2 of Chicken Droplet (CD) 70% & Food Waste (FW) 30% had the highest biogas volume of 0.1litres within the study period.
This study showed rumen fluid has an effect on the rate of biogas produced at different mix ratio. Therefore, using the rumen fluid as inoculum would be of high benefit.

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EFFECT OF WATER CEMENT RATIO ON COMPRESSIVE AND FLEXURAL STRENGTHS OF CONCRETE CONTAINING COARSE AGGREGATE PARTIALLY REPLACED WITH PERIWINKLE SHELLS

*Onehi, J. S.[1]; Tsado, T. Y[1] & Abdullahi, M[1]
[1] Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: onehi_josephshelleng@yahoo.com

ABSTRACT
The quest for the replacement of expensive concrete constituent materials coupled with the growing problem of periwinkle shells deposits associated with risk to public health has led to the use of the shells for replacement of coarse aggregate. To determine strength characteristics of concrete with partial replacement of coarse aggregate with periwinkle shells, this study investigated the effect of water cement (w/c) ratio on compressive and flexural strengths of the concrete. A total of sixty (60) concrete cubes and sixty (60) beams with dimensions of 150 mm x 150 mm x 150 mm and 100 mm x 100 mm x 500 mm were cast and cured for 7 days and 28 days respectively. The study adopted percentage by weight of gravel to periwinkle shells as coarse aggregate maintained at 30% inclusion of periwinkle shells and at fixed mix ratio of 1:2:4 with the variation of w/c ratio between 0.40 and 0.60 for the mix design. The physical properties of the aggregates and mechanical properties of the concrete were determined. The slump of the concrete increases with increase in w/c ratio. Both compressive and flexural strengths of the concrete cured for 28 days showed an upward shift compared to 7 days. The maximum compressive strengths of 20.89 N/mm^2 and 21.48 N/mm^2 and maximum flexural strengths of 1.63 N/mm^2 and 2.13 N/mm^2 were obtained at w/c ratio 0.55 for 7 days and 28 days respectively indicating that w/c of 0.55 is the optimum value for the mix design.

Keywords: *Coarse aggregate, Compressive strength, Flexural strength, Periwinkle Shells, Water cement ratio.*

1 INTRODUCTION

The high rate of construction of building structures as a result of ever increasing population globally has led to building materials being rapidly consumed. Concrete is arguably the most important building material and a major component of most of infrastructural facilities in modern society, playing a part in all building structures. Its virtue is its versatility, i.e. its ability to be moulded to take up the shapes required for the various structural forms. Concrete can be used for all standard buildings ranging from single storey to multistorey and for containment and retaining structures and bridges (Macginley and Choo, 2003). Concrete is a composite material; its basic constituents are cement, fine aggregate (sand), coarse aggregate (granite chippings) and water (Bharathi *et al.*, 2016). The rapid consumption of concrete material poses a great threat of possible depletion in the near future. Consequently, there are ongoing research activities to fully or partially replace some of these materials with new ones (Oyedepo, 2016; Macginley and Choo, 2003). Thus, this has led to the exploration of alternative materials or locally accessible materials. The utilization of locally accessible materials has attracted lots of attention in the recent years for the production of light weight concrete (LWC) (Oyedepo, 2016). Numerous achievements have been made in these regards due to its functional benefit of waste reusability and sustainable development. Research has in the recent times focused on the locally available solid waste to be used as concrete constituent.

Solid wastes are substances and masses resulted by the various human activities that have to be dumped. Solid waste materials usually include industrial waste, medical waste, agricultural waste as well as domestic waste. In continuing quest for more cost-efficient and environmentally acceptable materials, recently, there has been a growing interest in the use of agricultural wastes as aggregate. Some of the aggregates of agricultural origin include palm kernel shells (Olutoge, 2010; Ndoke, 2006), coconut shell (Olanipekun, 2006) and periwinkle shells (Taiwo *et al.*, 2018;; Etim *et al.*, 2017; Job *et al.*, 2017; Ekop *et al.* 2013).

Periwinkle Shells (PS) are agricultural waste which is discarded when the edible Periwinkles have been removed. The waste is stockpiled in open fields thereby causing negative impacts on the environment, and managing it has become a big challenge because of high generation of this waste on daily basis. Accordingly, Ohimain *et al.* (2009) stated that one of the ways to dispose the waste would be utilization of some of it into constructive building materials as this would help to prevent the depletion of natural resources such as granite and gravel and to maintain ecological balance. Moreover, the quantities of periwinkle shell waste mostly in the riverine areas in Nigeria have been increasing significantly without being recycled increasing the risk to public health due to their unpleasant odour and unsightly appearance in open-dump sites located at strategic places as well as scarcity of land area. This growing problem of periwinkle shell waste in the riverine areas can be alleviated if new disposal options other than landfill can be found. There has been an increasing significant interest in the development of concrete mixes with periwinkle shell, besides, using periwinkle shell as an aggregate is effective for environmental conservation and economical advantage.

Furthermore, concrete will only become a quality material for construction when the ingredients are properly sourced and selected as well as when it is manufactured under a regulated standard and practice procedure. The issue of materials soundness is very important during selection. If concrete materials contain deleterious substances the product will not meet its design strength and so can fail. Also, even when the materials are properly selected but are not produced to the specified mix proportion, they will eventually fail when it cannot provide the required designed strength that will sustain it when loads are imposed. In the light of the above, it is expedient to determine strength characteristics of concrete with partial replacement of coarse aggregate by periwinkle shells which is a common practice among many Nigerians especially in the riverine areas. Since concrete strength is the most important factor when creating structural building materials (Osarenmwinda and Awaro, 2009), it is necessary to investigate the suitability of the replacement, thus to avoid building collapse and ensure good practice without violating rules. Thus, this study investigates the effect of water cement ratio on compressive and flexural strengths of concrete partially replaced with periwinkle shells as coarse aggregate.

2 METHODOLOGY

2.1 MATERIALS

The materials used during the study include fine aggregate, coarse aggregates, periwinkle shells, cement and water. Naturally occurring river sand in Minna, Nigeria was used as fine aggregate, gravel was used as coarse aggregate and periwinkle shells aggregate (PSA) was obtained from local sellers at Dutse market in Abuja and used to partially replace coarse aggregate (gravel) in the mixture. The Periwinkle shell aggregate (PSA) was thoroughly washed and sundried to remove impurities, and hand-picking of further impurities was done before taking for laboratory tests. Portable water from the Civil Engineering Laboratory of the Federal University of Technology, Minna was used throughout the production of the concrete as well as washing of the dirty periwinkle shells. Dangote Cement, an Ordinary Portland Cement (OPC), was used for the experiment.

2.2 CASTING, CURING AND CRUSHING

Prior to the concrete production, some tests were conducted on the aggregates. Gradation test (sieve analysis) was conducted on the periwinkle shells, fine and coarse aggregates according to BS EN 12620:2002. The specific density of fine aggregate (FA), coarse aggregate (CA) and periwinkle shell aggregate (PSA) were obtained according to BS 1377-2:1990. Bulk density of both fine aggregate (FA) and coarse aggregate (CA) was obtained according to BS 812-2:1995. The moisture contents of the fine aggregate (FA), coarse aggregate (CA) and periwinkle shell aggregate (PSA) were determined in accordance with the procedure enumerated by ASTM C128-12:2012.

Batching operation by absolute volume method was adopted in the study. The weights of the constituent materials were measured out using weighing scale. The concrete materials were mixed in a concrete mixing machine. A mix ratio of 1:2:4 (cement: fines: coarse) was adopted using water/cement ratio of 0.40, 0.45, 0.50, 0.55, and 0.60 respectively. The coarse aggregate was replaced with periwinkle shells at 30 %wt (percentage by weight). Cast iron mould of size 150 mm x 150 mm x 150 mm; and 100 mm x 100 mm x 500 mm prismatic mould were used for the production of the samples. The moulds were assembled prior to mixing and properly lubricated for easy removal of hardened concrete cubes or beams. The specimens were made in accordance to BS 1881.

Slump test was conducted on the fresh concrete mixes according to BS EN 12350-2:2009 before it was filled in cube and beam moulds. The molded concrete cubes and beams were given 24 hours to set before demoulding. They were immersed into curing tank, to promote increase in strength, hydration, eliminate shrinkage and absorb heat of hydration until the age of test. The prepared cubes and beams were cured for 7 days and 28 days respectively. The specimens were weighted before testing and the densities of the specimens at different time of testing were measured. Prior to testing, the specimens were brought out of the curing tank, left inside in the open air for about 2 hours before crushing. The compressive and flexural strengths of the cubes and beams were tested in accordance to BS EN 12390-3:2009 and BS EN 12390-5:2009 respectively.

3 RESULTS AND DISCUSSION

3.1 PROPERTIES OF AGGREGATES

The particle size distribution of sand, gravel and periwinkle shells from sieve analysis results are shown in Figure 1. For the sand (fine aggregate), the percentage by mass passing sieves 5.00mm, 3.35mm, 2.36mm, 1.18mm, 600μm, 300μm, and 150μm were 91%, 73%, 68%, 46%, 27%, and 12% respectively. It shows that the fine aggregate is well graded suitable for all purpose, since the percentage by mass passing 2.36mm sieve was 68% > 60% as stipulated by BS 882 (1992). For gravel, the percentages by mass passing sieves 20.0mm, 14mm, and 10mm were 75%, 24%, and 6% respectively. These values showed that the aggregates belong to grade 20mm to 5mm as provided by BS 882 (1992).

Periwinkle Shells aggregate, the percentages by mass passing sieves 20.0mm, 14mm, and 10mm were 100%, 83%, and 2% respectively. These results showed that the shells are well graded aggregate and also fit into grades below 20mm as provided by BS 882 (1992) which is suitable for gravel replacement for concrete production.

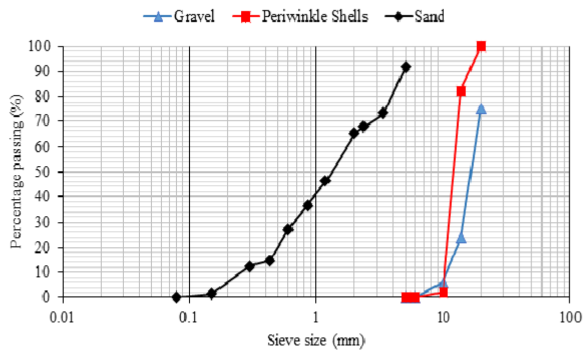

FIGURE 1: PARTICLE SIZE DISTRIBUTION OF AGGREGATES

Table I shows specific gravity, bulk density, moisture content and water absorption of sand, gravel and periwinkle shells aggregate (PSA). The specific gravity of sand, gravel and PSA were 2.65, 2.68 and 1.21 respectively. It was deduced that the PSA has a lower specific gravity when compared to that of sand and gravel. The obtained bulk density of sand, gravel and PSA were 1753.60 kg/m^3, 1563.5 kg/m^3, and 551.07 kg/m^3 for uncompacted and 1898.84 kg/m^3, 1798.45 kg/m^3, and 653.60 kg/m^3 for compacted specimen respectively. The Bulk density of PSA is lower than lower that of sand and gravel. The moisture content of sand, gravel and PSA were 3.89 %, 0.59 % and 2.48 % corresponding to 23.23, 0.33, and 2.92 water absorption respectively. The moisture content of PSA is lower than that of sand but higher than that of gravel.

Slump test

The workability of concrete containing coarse aggregates replaced with 0% and 30% periwinkle shells were studied for each water cement (w/c) ratio of 0.40, 0.45, 0.50, 0.55 and 0.60 (Table II). For concrete with 0% PSA content, the slump was 80 mm, 110 mm, 175 mm, 220 mm and 225 mm for w/c ratio of 0.40, 0.45, 0.50, 0.55 and 0.60 respectively. The slump increases with an increase in the in w/c ratio. For concrete with 30% PSA content, the slump was 50 mm, 80 mm, 150 mm, 175 mm and 200 mm for w/c ratio of 0.40, 0.45, 0.50, 0.55 and 0.60 respectively. The slump also increased with an increase in w/c ratio. Comparatively, the slump decreases when coarse aggregate was replaced by 30% PSA.

TABLE 1: PROPERTIES OF AGGREGATES

Aggregate	Specific Gravity	Bulk Density (kg/m^3)		Moisture Content (%)	Water Absorption
		Uncompacted	Compacted		
Sand	2.65	1753.60	1898.84	3.89	23.23
Gravel	2.68	1563.5	1798.45	0.59	0.33
PSA	1.21	551.07	653.60	2.48	2.92

TABLE II: SLUMP TEST

Water cement (w/c) Ratio	Slump value for 0% PSA (mm)	Slump value for 30% PSA (mm)
0.40	80	50
0.45	110	80
0.50	175	150
0.55	220	175
0.60	225	200

This means that the concrete becomes less workable (stiff) with the introduction of PSA, the concrete becomes more workable with increase in w/c ratio as revealed in the table. This is in agreement with the finding of Adewuyi and Adegoke (2008); Taiwo *et al.* (2018) and Aboshio *et al.* (2018). The decrease in workability observed could be attributed to the possibility of water molecules trapping in the pores of periwinkle shells.

3.2 COMPRESSIVE STRENGTH

The compressive strength of concrete containing coarse aggregates replaced with 0% and 30% periwinkle shells was studied. For each water cement (w/c) ratio of 0.40, 0.45, 0.50, 0.55 and 0.60, twelve cubes each were prepared. The cubes were tested at age 7 days and 28 days respectively. For 7 and 28 days, three cubes were crushed for each curing age and the average strength taken. Figure 2 compares the effects of water cement ratio on the compressive strength of concrete containing 0% (control) and 30% PSA content cured at 7 days and 28 days respectively. Generally, the comprehensive strengths of the control specimen are higher than those 30% PSA and also the strengths increased with the increase in curing age from 7 days to 28 days with the maximum values at w/c of 0.5, beyond this, the strength decreases.

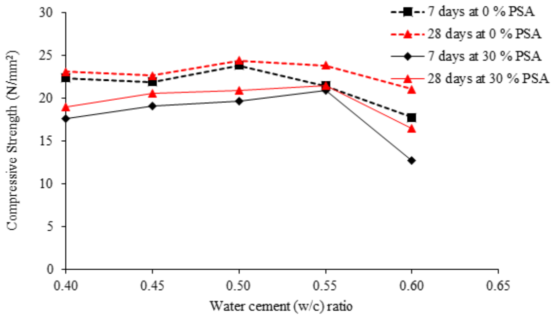

FIGURE 2: VARIATION OF COMPREHENSIVE STRENGTH WITH W/C RATIO

For 0% PSA concrete, the maximum compressive strengths of 23.85 N/mm^2 and 24.44 N/mm^2 were obtained at w/c ratio of 0.50 for 7 days and 28 days respectively. For concrete with 30% PSA content, the variation of comprehensive strength with w/c ratio observed from the charts indicates that the compressive strengths increased with increase in w/c ratio and reached the maximum value at w/c ratio of 0.55. It is also observed that there is an upward shift when the curing age increased from 7 days to 28 days. The maximum compressive strengths for the 30% PSA were 20.89 N/mm^2 and 21.48 N/mm^2 at w/c ratio of 0.55 for 7 days and 28 days respectively. This indicates that w/c ratio of 0.55 is the optimum value to obtain the maximum compressive strength for the mix design.

3.3 FLEXURAL STRENGTH

Similar to compressive strength, for each water cement (w/c) ratio of 0.40, 0.45, 0.50, 0.55 and 0.60, twelve beams each were prepared comprising the beams with coarse aggregates replaced with 0% and 30% periwinkle shells. The beams were tested at age 7 and 28 days respectively. For 7 days and 28 days, three beams were crushed for each curing age and the average strength taken. The effects of water cement (w/c) ratio on the flexural strength of concrete containing coarse aggregate replaced with 0% and 30% PSA cured at 7 days and 28 days are respectively presented in Figure 3. The maximum flexural strengths of concrete containing 0% PSA concrete were 2.38 N/mm^2 and 2.75 N/mm^2 obtained at w/c ratio of 0.50 for 7 days and 28 days respectively. The strength of the concrete increased with increase in curing age.

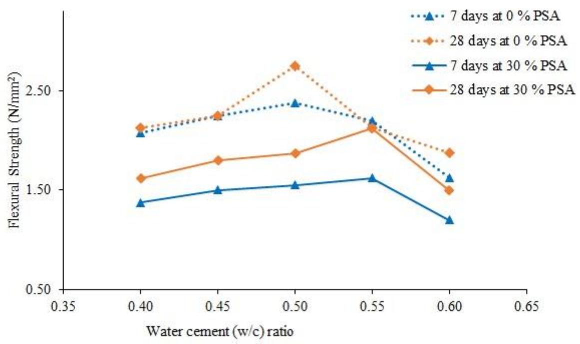

FIGURE 3: VARIATION OF FLEXURAL STRENGTH WITH W/C RATIO

For 30% PSA concrete, the strength of the concrete is lower than their corresponding 0% PSA concretes (control). The maximum strength values of 1.63 N/mm^2 and 2.13 N/mm^2 were obtained at w/c ratio of 0.55 for 7 days and 28 days respectively. The results show that flexural strength increased as the curing age increased from 7 days to 28 days and in the meantime from 0.4 to 0.55 w/c ratio but decline beyond 0.55. It can be deduced that w/c ratio of 0.55 is the optimum value to obtain the maximum flexural strength for the mix design with 30% PSA. This can suggest that the demand for more water to maintain the same workability level as the control (0% PSA) could be responsible for the shift from 0.50 to 0.55 of w/c ratio optimum value.

4 CONCLUSION

Strengths of concrete with 1:2:4 mix ratio containing 0% and 30 % of coarse aggregate partially replaced with periwinkle shells cured for 7 days and 28 days respectively were determined based on the properties of the aggregate at different water cement (w/c) ratio. The percentages by mass passing sieves values of gravel belong to grade 20 mm to 5 mm as provided by BS 882 and Periwinkle Shells aggregate (PSA) were determined to fall within the range of grade values of 20 mm to 5 mm, this implies that it is a suitable replacement for coarse aggregate (gravel) for the production of light weight concrete due to its low specific gravity of 1.21.

The slump increased with an increase in w/c ratio for both 0% and 30% PSA concrete. Comparatively, the slump decreases when coarse aggregate was replaced by 30% PSA. This implies that the decrease in workability was as a result of water molecules trapping in the pores of periwinkle shells. The compressive strengths of both 0% and 30% PSA concrete cured for 28 days showed higher continuous strength development comparable with that of the concrete cured for 7 days. The flexural strength showed similar characteristics as that of compressive strength. Both compressive and flexural strengths have their maximum at w/c of 0.55 for 7 days and 28 days respectively. Thus, w/c of 0.55 is the optimum value to obtain the maximum compressive and flexural strengths for the mix design. In contrast, the demand for more water to maintain the same workability level as the control (0% PSA) may be responsible for the shift from 0.50 to 0.55 of w/c ratio optimum value. The optimum w/c ratio of 0.55 recorded at 28 days with 1:2:4 of compressive strength of 23.85 N/mm^2 for 30% PSA content satisfied ASTM C 330 (2017) minimum requirement of 17 N/mm^2 for lightweight concrete, hence Periwinkle shells is suitable for the production of lightweight concrete.

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STIFFENED SLAB ANALYSIS USING FINITE STRIP METHOD

*Ike, V. I.[1]; Sadiku, S.[2]; & Abdullahi, M.[3]
[1 2 3] Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: ike.vincente@gmail.com

ABSTRACT

A rectangular slab stiffened by ribs oriented perpendicular to the support of the slab as structurally equivalent to an orthotropic plate is analyzed using the finite strip method. The analytic formulation is obtained from the governing equation of motion of plate using the classical plate theory to an arbitrary boundary and initial condition. The stiffness formulation from total internal energy is considered. The force-displacement equation technique by applying stiffness and uniformly distributed loading is used to compute the nodal displacement, rotation and hence, displacement at an arbitrary point of the plate by a summation of results for all the assumed harmonic function. Approximate moment at the nodal line can be computed as the mean values from moment yielding from adjacent strips.

Keywords: *Finite Strip Analysis; Simply Supported; Stiffened slab; Stiffness matrix.*

1 INTRODUCTION

Discontinuity in the thickness of slab causes singularities in the differential equation of the problem. Obtaining analytical solution involves solving very highly discontinuous partial differential equation which proves prohibitive and most often inapplicable to practical engineering problems involving rib slabs.
Treatment of this problem which considers the parent plate as distinct from the ribs does not reflect the true mechanical behaviour of ribbed civil engineering slabs in practical monolithically cast reinforced concrete ribbed slabs. It is common knowledge that in monolithic structures, the framing beams and slabs act together as one structure. This approach of treating a ribbed slab as a combination of strips of different thickness and hence bending rigidities via finite element considerations preserves the monolithic behaviour of the parent rib or slab.
Finite Strip method (FSM) is the method of semi-numerical and semi-analytical nature. It is suitable for the analysis of rectangular plates and plane-stress elements or structures being the combination of both. The finite strip method enables a three-dimensional structure to be treated as a two-dimensional problem, or a two-dimensional structure (such as a plate) to be treated as a one-dimensional problem. Such a reduction in the size of the problem and hence in the amount of computational effort is generally possible only in situations where the geometrical and material properties do not vary along one coordinate direction (Cusens, et al., 1975).
The finite strip method, pioneered in 1968 by Y. K. Cheung, is an efficient tool for analyzing structures with regular geometric platform and simple boundary conditions. Originally, the finite strip method was designed for rectangular plate problems similar to Levy’s solution by Timoshenko and Woinowsky-Krieger in 1971. Later, the finite strip method was extended to treat folded plates, and box girders (Cusens, et al., 1975), curved plates (Dey, 1980), skewed (quadrilateral) plates and parabolic cylinders (K. H. Miiller; Commission of the European Communities; Joint Research Centre, 1988).
Unlike the standard finite element method, which uses polynomial displacement functions in all directions, the finite strip method calls for use of simple polynomials in some directions and continuously differentiable smooth series in the other directions, with the stipulation that such series should satisfy the prior at the boundary conditions at the ends of the strips. The accuracy of the results obtained using the FSM depends on two parameters: the number of strips and the number of the harmonic functions m.

2 PROBLEM FORMULATION

The flexural motion of the rectangular stiffened isotropic slab is examined, with solution for each plates and ribs in the form of strips of uniform dimensions.

The flexural motion of the stiffened slab is governed by the following differential equation:

$$D\nabla^4 w = q(x, y) \tag{1}$$

The notations appearing have the following meaning: w is the flexural deflection of the plate, D is the bending rigidity of the plate in both directions, x and y are the spatial coordinates, q is the intensity of loading. The plate is simply supported on two parallel edges; the other two edges are arbitrary or on elastic support.

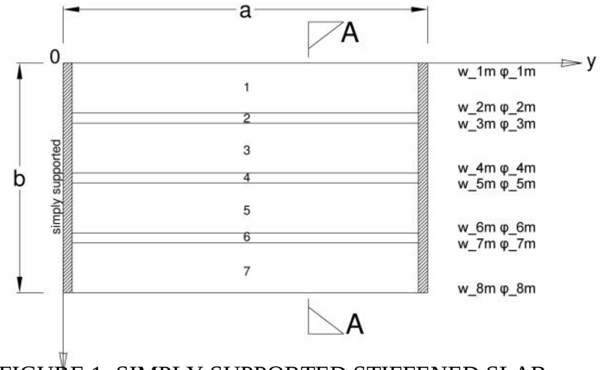

FIGURE 1: SIMPLY SUPPORTED STIFFENED SLAB

The deflection component to (1) is

$$w(x,y) = \sum_{m=1}^{r} w(x) \sin\frac{m\pi y}{a} \tag{2}$$

$$k_m = \frac{m\pi}{a}, \qquad m = 1,2,\ldots,r$$

2.1 STRIP ANALYSIS

The approximate displacement function for the points on a single strip combine the sine harmonic series in the longitudinal direction, y (analytical aspect) and the polynomial function $F_m(x)$ in the transverse direction, x (numerical aspect). The deflection of the strip may be written in the transform of two uncoupled function

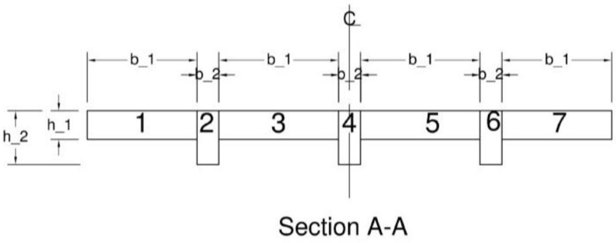

Figure 2.0: Section representation of stiffened slab

$$w(x,y) = \sum_{m=1}^{r} F_m(x) \sin\frac{m\pi y}{a} \tag{3}$$

$$F_m(x) = C(x)\delta_m(x) =$$

$$\left[1 - \frac{3x^2}{b^2} + \frac{2x^3}{b^3} \quad x\left(1 - \frac{2x}{b} + \frac{x^2}{b^2}\right) \quad \frac{3x^2}{b^2} - \frac{2x^3}{b^3} \quad x\left(\frac{x^2}{b^2} - \frac{x}{b}\right)\right] \begin{Bmatrix} w_{im} \\ \varphi_{im} \\ w_{jm} \\ \varphi_{jm} \end{Bmatrix} \tag{4}$$

The strains ε_B can be written in terms of appropriate derivatives of shape functions, C_m, and the nodal displacements, with respect to each half-wave number, m.

$$\varepsilon_B = \begin{Bmatrix} -\frac{\partial^2 w}{\partial x^2} \\ -\frac{\partial^2 w}{\partial y^2} \\ 2\frac{\partial^2 w}{\partial x \partial y} \end{Bmatrix} = \sum_{m=1}^{r} [B]_m \begin{Bmatrix} w_{im} \\ \varphi_{im} \\ w_{jm} \\ \varphi_{jm} \end{Bmatrix} = \sum_{m=1}^{r} [B]_m \{\delta\}_m \tag{5}$$

The elastic stiffness matrix can be readily derived from the total internal energy.

$$U = \frac{1}{2}\int_0^a \int_0^b \varepsilon_B{}^T [D]\, \varepsilon_B \, dxdy \tag{6}$$

Where, $[D] = \begin{bmatrix} D_x & D_1 & 0 \\ D_1 & D_y & 0 \\ 0 & 0 & D_{xy} \end{bmatrix}$

since the unstiffened plate is isotropic and also that the stiffener offers negligible torsional resistance, then one can put

$$D_x = D_y = D = \frac{E_x t^3}{12(1 - \vartheta_x \vartheta_y)}, = \frac{E_y t^3}{12(1 - \vartheta_x \vartheta_y)}$$

$$D_1 = \vartheta D, \qquad D_{xy} = \frac{1 - \vartheta}{2} D$$

the strain energy of a strip be defined as

$$U_s = \frac{1}{2}\int_0^a \int_0^b \delta^T B^T [D]\, B\, \delta \, dxdy \tag{7}$$

the potential energy in terms of external surface loads is

$$U_w = -\int_0^a \int_0^b \delta^T \{C\}^T q(x,y) \sin k_m y \, dxdy \tag{8}$$

$$[P] = \int_0^a \int_0^b \{C\}^T q(x,y) \sin k_m y \, dxdy \tag{9}$$

and total potential energy is the sum of the elastic strain energy and the work potential of each strip, thus

$$W = U_s + U_w \tag{10}$$

by minimizing the total potential energy, we set

$$\left\{\frac{\partial W}{\partial\{\delta\}}\right\} = \{0\} \tag{11}$$

so that the differentiation gets

$$\left\{\frac{\partial W}{\partial\{\delta\}}\right\} = \int_0^a\int_0^b \{B\}^T[D]\{B\}\,\delta\,dxdy - \int_0^a\int_0^b \{C\}^T q(x,y,t)\sin k_m y\,dxdy = \{0\}$$

and,

$$[K]\{\delta\} - \{P\} = \{0\} \tag{12}$$

the exact stiffness is derived from the solution of strain energy as

$$K_{mn} = \int_0^a\int_0^b B^T[D]\,B\,dxdy \tag{13}$$

stiffness matrix becomes

$$[K] = [D]\int_0^a\int_0^b \begin{bmatrix} B_m^TB_n & B_m^TB_n & B_m^TB_n & B_m^TB_n \\ B_m^TB_n & B_m^TB_n & B_m^TB_n & B_m^TB_n \\ B_m^TB_n & B_m^TB_n & B_m^TB_n & B_m^TB_n \\ B_m^TB_n & B_m^TB_n & B_m^TB_n & B_m^TB_n \end{bmatrix} dxdy$$

where $[K]$ is the elastic stiffness matrix corresponding to half-wave numbers m.

Furthermore,

$$\{B\}_m = \begin{bmatrix} -C''_{m1}\sin k_m y & -C''_{m2}\sin k_m y & -C''_{m3}\sin k_m y & -C''_{m4}\sin k_m y \\ k_m^2C_{m1}\sin k_m y & k_m^2C_{m2}\sin k_m y & k_m^2C_{m3}\sin k_m y & k_m^2C_{m4}\sin k_m y \\ 2k_mC'_{m1}\cos k_m y & 2k_mC'_{m2}\cos k_m y & 2k_mC'_{m3}\cos k_m y & 2k_mC'_{m1}\cos k_m y \end{bmatrix}$$

Since opposite ends of the finite strip are simply supported in the transverse direction, the basic function and its first and second derivatives are involved in the evaluation of the stiffness matrix.

$$Y_m = \sin k_m y,\; Y'_m = k_m\cos k_m y,\; Y''_m = -{k_m}^2\sin k_m y$$

Thus, we involve the following expressions

$$\int_0^a \sin\frac{m\pi y}{a}\sin\frac{n\pi y}{a}dy = \begin{cases} 0 & \text{for } m \neq n \\ \frac{a}{2} & \text{for } m = n \end{cases}$$

$$\int_0^a \cos\frac{m\pi y}{a}\cos\frac{n\pi y}{a}dy = \begin{cases} 0 & \text{for } m \neq n \\ \frac{a}{2} & \text{for } m = n \end{cases}$$

and

$$\int_0^a Y'_mY'_n dy = \begin{cases} \left(\frac{m\pi}{a}\right)\left(\frac{n\pi}{a}\right)\int_0^a \cos\frac{m\pi y}{a}\cos\frac{n\pi y}{a}dy = 0 & \text{for } m \neq n \\ \frac{m^2\pi^2}{2a} & \text{for } m = n \end{cases}$$

$$\int_0^a Y_mY''_n dy = \begin{cases} -\frac{m^2\pi^2}{b^2}\int_0^a \sin\frac{m\pi y}{a}\sin\frac{n\pi y}{a}dy = 0 & \text{for } m \neq n \\ -\frac{m^2\pi^2}{2a} & \text{for } m = n \end{cases}$$

Corresponding to simply supported rectangular strips and nodal lines displacement components in the y direction with expressions for the definite integrals solution

$$I_1 = \int_0^a Y_mY_n dy,\; I_2 = \int_0^a Y''_mY_n dy,\; I_3 = \int_0^a Y_mY''_n dy,$$
$$I_4 = \int_0^a Y''_mY''_n dy,\; I_5 = \int_0^a Y'_mY'_n dy$$

so that the explicit form of the stiffness matrix for a single strip in the m and n-th harmonic function is

$$[K]_m = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} \\ & K_{22} & K_{23} & K_{24} \\ & & K_{33} & K_{34} \\ \text{sym.} & & & K_{44} \end{bmatrix} \tag{14}$$

with:

$$K_{11} = \frac{13ab}{70}k_m^4D + \frac{12a}{5b}k_m^2D_{xy} + \frac{6a}{5b}k_m^2D_1 + \frac{6a}{b^3}k_m^2D$$

$$K_{12} = \frac{3a}{5}k_m^2D_1 + \frac{a}{5}k_m^2D_{xy} + \frac{3a}{b^2}D + \frac{11ab^2}{420}k_m^2D$$

$$K_{22} = \frac{ab^3}{210}k_m^4D + \frac{4ab}{15}k_m^2D_{xy} + \frac{2ab}{15}k_m^2D_1 + \frac{2a}{b}D$$

$$K_{13} = \frac{9ab}{140}k_m^4D - \frac{12a}{5b}k_m^2D_{xy} - \frac{6a}{5b}k_m^2D_1 - \frac{6a}{b^2}D$$

$$K_{23} = \frac{13ab^2}{840}k_m^4D - \frac{a}{5}k_m^2D_{xy} - \frac{a}{10}k_m^2D_1 - \frac{3a}{b^2}D$$

$$K_{33} = \frac{13ab}{70}k_m^4D + \frac{12a}{5b}k_m^2D_{xy} + \frac{6a}{5b}k_m^2D_1 + \frac{6a}{b^2}D$$

$$K_{14} = -\frac{13ab^2}{840}k_m^4D + \frac{a}{5}k_m^2D_{xy} - \frac{a}{10}k_m^2D_1 + \frac{3a}{b^2}D$$

$$K_{24} = -\frac{3ab^3}{840}k_m^4 D - \frac{ab}{15}k_m^2 D_{xy} - \frac{ab}{30}k_m^2 D_1 + \frac{a}{b}D$$

$$K_{34} = -\frac{11ab^2}{420}k_m^4 D - \frac{a}{5}k_m^2 D_{xy} - \frac{3a}{5}k_m^2 D_1 - \frac{3a}{b^2}D$$

$$K_{44} = \frac{ab^3}{210}k_m^4 D + \frac{4ab}{15}k_m^2 D_{xy} + \frac{2ab}{15}k_m^2 D_1 + \frac{2a}{b}D$$

The finite strip which has two degrees of freedom per nodal line in bending, hence, the load matrix under uniform external loading

$$[P]_m = \int_0^a \int_0^b \{C\}^T q(x,y) \sin k_m y \, dxdy \qquad (15)$$

Also to dispense with the integration,

$$[P]_m = q(x,y)\begin{Bmatrix} \frac{b}{2} \\ \frac{b^2}{12} \\ \frac{b}{2} \\ \frac{b}{2} \end{Bmatrix} \frac{1}{k_m}(1 - \cos k_m a) \qquad (16)$$

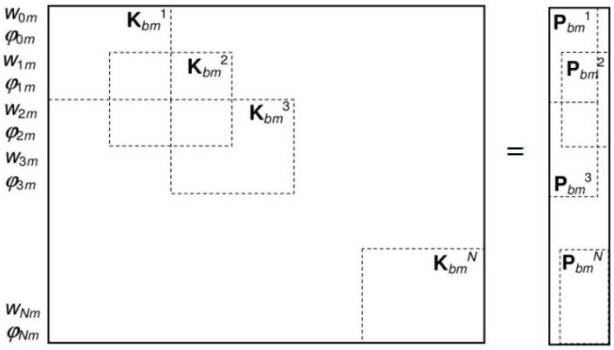

Figure 3.0: Assembled force-deflection solution under arbitrary boundary and initial condition

The bending and twisting moments for a strip are:

$$\begin{Bmatrix} M_x \\ M_y \\ M_{xy} \end{Bmatrix} = [D] \sum_{m=1}^{r} [B]_m \{\delta\}_m \qquad (17)$$

2.2 BOUNDARY CONDITION

Boundary conditions can be applied in the same way as we apply in the finite element method. The penalty method is generally used to apply the boundary conditions to suppress the particular degree of freedom. The procedure for applying the boundary conditions is summarized as follows:

Free (F) at nodal line: no penalty is applied

Simply supported (SS) at nodal line: w_{im} vanishes or is replaced with zero.

3 NUMERICAL EXAMPLE

The table below show the design description of a slab.

TABLE 1: PARAMETERS OF THE DESIGN

Parameter	Description	Value
q	UDL on slab	25 kN/m^2
E	Young modulus	35.82*$10^6 kN/m^2$
v	Poisson's ratio	0.15
a	Length of slab	12.0 m
b	Breadth of slab	7.5 m

divide into 7-strips of ribs and plates

TABLE 2: FINITE STRIPS DETAIL

Strip	a (m)	b(m)	h (m)
Plate – 1	12	1.5	0.3
Rib – 2	12	0.5	0.8
Plate – 3	12	1.5	0.3
Rib – 4	12	0.5	0.8
Plate – 5	12	1.5	0.3
Rib – 6	12	0.5	0.8
Plate – 7	12	1.5	0.3

Plate strip:

$$[D] = 10^4 \begin{bmatrix} 8.245 & 1.237 & 0 \\ 1.237 & 8.245 & 0 \\ 0 & 0 & 3.504 \end{bmatrix}$$

$$[K] = 10^9 \begin{bmatrix} 2.250 & 0.022 & 0.681 & -0.265 \\ & 0.145 & 0.267 & 0.098 \\ & & 2.251 & -0.459 \\ sym. & & & 0.145 \end{bmatrix}_{m=1,2,..,8}$$

$$[P] = \begin{bmatrix} -13.316 \\ -3.329 \\ -13.316 \\ -13.316 \end{bmatrix}_{m=1,2,..,8}$$

Rib strip:

$$[D] = 10^6 \begin{bmatrix} 1.564 & 0.235 & 0 \\ 0.235 & 1.564 & 0 \\ 0 & 0 & 0.665 \end{bmatrix}$$

$$[K] = 10^9 \begin{bmatrix} 18.67 & 0.528 & 0.313 & -0.232 \\ & 0.275 & 0.682 & 0.057 \\ & & 18.22 & -0.737 \\ sym. & & & 0.275 \end{bmatrix}_{m=1,2,..,8}$$

$$[\mathrm{P}] = \begin{bmatrix} -4.439 \\ -0.37 \\ -4.439 \\ -4.439 \end{bmatrix}_{m=1,2,..,8}$$

TABLE 3: DEFLECION AT CENTRE OF STRIP AND MOMENT ALONG NODAL LINE

Strip (x,y)	Nodal line	Deflection($\boldsymbol{m}$) & Moments ($\boldsymbol{kNm}$) of strip			
		w	M_x	M_y	M_{xy}
1 (0.75,6)	w_1(0,6)	$-3.1 * 10^{-9}$	-0.0166	-0.0747	0.018
	w_2(1.5,6)		0.0095	-0.0020	-0.1997
2 (1.75,6)	w_2(1.5,6)	$-1.0 * 10^{-6}$	-3.942	-39.14	-74.50
	w_3(2,6)		-14.56	-115.30	-151.62
3 (2.75,6)	w_3(2,6)	$-1.9 * 10^{-7}$	-0.019	-0.194	-0.423
	w_4(3.5,6)		-0.349	-2.463	-1.953
4 (3.75,6)	w_4(3.5,6)	$-1.3 * 10^{-5}$	-96.73	-673.90	-460.17
	w_5(4,6)		-122.05	-844.93	-534.85
w_5, w_6, w_7, w_8 conform to the conditon of symmetry with w_4, w_3, w_2 & w_1 respectively.					

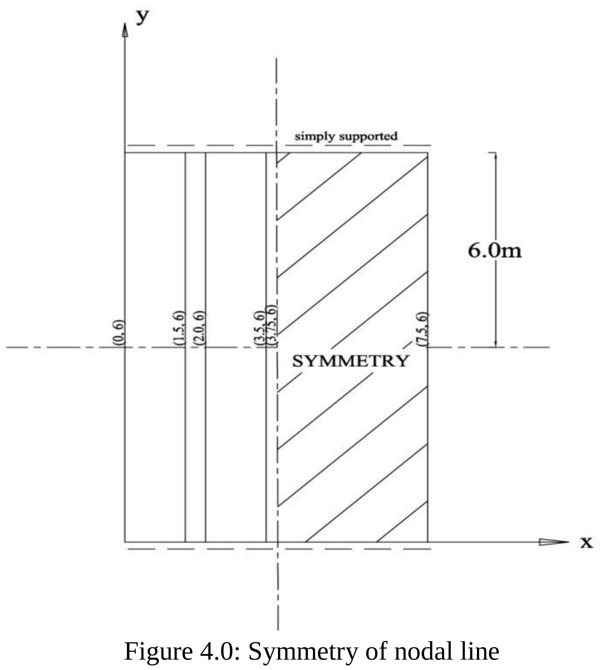

Figure 4.0: Symmetry of nodal line

4 CONCLUSION

The Finite Strip method is applicable in dealing with singularities of an orthotropic plate. This semi-numerical model requires less computational effort, hence saving time. The approach divides the stiffened slab into strips that preserves the monolithic behaviour, in terms of deflections and moments along the nodal lines. Results obtained have been adequate base on the numerical example to indicate the method as efficient and accurate.

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COMPUTER AIDED ANALYSIS OF HIERARCHICAL TRUSS STRUCTURES BASED ON THE METHOD OF SUBSTRUCTURING

***Musa, A[1]; Sadiku,S.[2] ; Abdullahi,M.[3]**
[1,2,3] Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author email: alfiyyamusa@gmail.com

ABSTRACT

The efficiency of using substructuring method(SM) for the static analysis of hierarchical truss structure (HTS) is examined, by using MATLAB R2015a (the Mathworks Inc, 2015) to program an algorithm for the analysis. HTS are widely used in space transport technology and also aeronautical engineering but not utilised in civil engineering as much as it should. The HTS was modelled in AUTOCAD 2015 version 20 **(**Autodesk Inc**,** 2015**),** the structure was then subdivided into 2 substructures, the degrees of freedom (dof) were generated and elements connectivity, the member element properties were specified that is the cross-sectional area and young's modulus, the stiffness matrix K was assembled for each substructure and then the boundary stiffness matrix and hence, the boundary and interior displacements were also calculated. The boundary and interior displacements derived from the substructuring method were then compared to the unconstrained displacement derived by classical FEM of the whole structure. The member forces derived by substructuring method and conventional FEM were also compared. The compared results showed similarities and hence the substructuring method is a viable alternative to FEM approach when analyzing HTS.

Keywords: *computer-aided, hierarchical truss structure, MATLAB, Substructuring*

1 INTRODUCTION

Hierarchical structures contain structural elements which are similar to the main structure (Fazli, et al, 2011), therefore, hierarchical truss structures (HTS) are trusses that have some or all of its members being trusses**.** A major advantage of Hierarchical truss structure is its strength in the absence of bracing, secondly smaller structural components are assembled together in the truss form to replace very large structural sections necessary to withstand applied loads therefore HTS are structures aimed at minimizing the use of structural materials.

Hierarchical structures are of interest due to several advantages that they offer such as the improved strength and stiffness of the whole structures (Fazli, et al, 2011).

For quite some time now the use of HTS has gained some prominence in mechanical and aeronautical structures, while it is not altogether strange in civil engineering structures, its use has been fairly minimal and checkered in civil engineering structures, with the advent of modern communication necessitating the erection of tall lattice towers, the need to introduce hierarchical truss forms into civil engineering practice is becoming increasingly inevitable. It is therefore important that the analysis of hierarchical truss structures in the context of civil engineering be properly articulated for use by field engineers.

(Murphey & Hinkle, 2003) investigated some performance trends in HTS as it applies to astronautics; the performance trends in linear truss structures was investigated as a function of self-similar hierarchy order and of loading conditions. The investigations show the order of structural hierarchy resulting in a lightest weight self-similar four longeron (longitudinal member) solid element truss-column is 2nd (a truss made from trusses) for requirements typical of space structures.

Substructuring is a process of analyzing a large structure as a collection of (natural) components. The FE models for these components are called substructures.

Substructuring is the splitting of a structure into a number of parts (called substructures) whose assembly forms the original structure (Onate, 2009).

Substructuring was invented by aerospace engineers in the early 1960s to carry out a first-level breakdown of complex systems such as a complete airplane, One obvious advantage of this idea results if the structure is built of several identical units (Ziaei-Rad, 2012).

Structural Partitioning corresponds to division of the whole structure into a number of substructures, the boundaries of which may be indicated arbitrarily, however, for convenience it is better to make structural partitioning correspond to physical partitioning (Przemieniecki,1968).

2 METHODOLOGY

The substructuring method is utilized; these calculations are made easier by computer aided analysis using MATLAB to write the program for the analysis that will produce the boundary and interior displacements and also the member forces.

The analysis carried out is on a two-dimensional HTS (Figure 1) of height 6000 millimetres and one horizontal member with hierarchy, attached to one node to simplify the analysis, an assumed load of 50KN was applied at the boundary nodes (where two or more substructures meet when the structure is whole) Young's Modulus was taken as 200KN/mm^2 for all steel sections. In actual construction the joints are more rigid than it is assumed to be (assumed to be pinned), this might affect accuracy of the results and also the scope does not include the action of secondary stresses.

Substructure one – is a truss tower with members of steel section L90 by 90 by 8, cross sectional area 1376mm^2, 6 nodes, 7 members and 12 degrees of freedom. The nodes were labeled from top to bottom, left to right. An assumed load of 50KN was applied at opposite ends of the boundary nodes (nodes 3 and 12)

Substructure two – is a truss with steel section L90 by 90 by 8, cross sectional area 1376 mm^2, 10 nodes, 18 members, and 20 degrees of freedom.

The two substructures are both treated separately, the substructure stiffness matrices $K^{(1)}$ and $K^{(2)}$ were derived and the boundary stiffnesses, $K_b^{(1)}$ and $K_b^{(2)}$ were found using equation (1) and (2), the combination which yielded the boundary stiffness of the whole structure K_b. The resultant boundary force, S_b was derived from equation(4), and subsequently the boundary displacement, U_b for the whole structure and the interior displacements of both substructures interior and boundary were derived from equation (5,6 and 7) respectively.

2.1 SUBSTRUCTURING

The equations used in the substructuring method of finite element method analysis of the structure after the stiffness matrix has been found for each substructure, the next step is to find the boundary stiffness of each substructure, The boundary stiffness for the two substructures;

$$K_b^{(1)} = K_{bb}^{(1)} - K_{bi}^{(1)}(K_{ii}^{(1)})^{-1} K_{ib}^{(1)} \qquad (1)$$

$$K_b^{(2)} = K_{bb}^{(2)} - K_{bi}^{(2)}(K_{ii}^{(2)})^{-1} K_{ib}^{(2)} \qquad (2)$$

Combining $K_b^{(1)}$ and $K_b^{(2)}$ to form the boundary stiffness for the entire structure, K_b, the resultant boundary forces, S_b was determined,

$$S_b = P_b - R_b \qquad (3)$$

Where;

P_b is the external forces on the substructure boundary forces,

R_b is substructure boundary reactions,

$$S_b = P_b - K_{bi}^{(1)}(K_{ii}^{(1)})^{-1} P_i^{(1)} \qquad (4)$$

P_i is the interior forces in the substructures, in this case it was noted to be zero, therefore $S_b = P_b$.

To determine boundary displacements, U_b

$$U_b = (K_b)^{-1} S_b \qquad (6)$$

To determine interior displacement,

$$U_i^{(1)} = (K_{ii}^{(1)})^{-1} P_i^{(1)} - (K_{ii}^{(1)})^{-1} K_{ib}^{(1)} U_b^{(1)} \qquad (7)$$

$$U_i^{(2)} = (K_{ii}^{(2)})^{-1} P_i^{(2)} - (K_{ii}^{(2)})^{-1} K_{ib}^{(2)} U_b^{(2)} \qquad (8)$$

To determine member forces, f, from boundary and interior displacements,

$$f_1 = \frac{A \times E}{L} \left[C_x \; C_y \; -C_x - C_y\right] \times \{U_1 \; U_2 \; U_3 \; U_4\}^T \quad (9)$$

2.2 FIGURES

All the dimensions are in millimeters, the node numbers are circled, an external force of 50KN was applied at the nodes 3 and 12 before the displacements were measured;

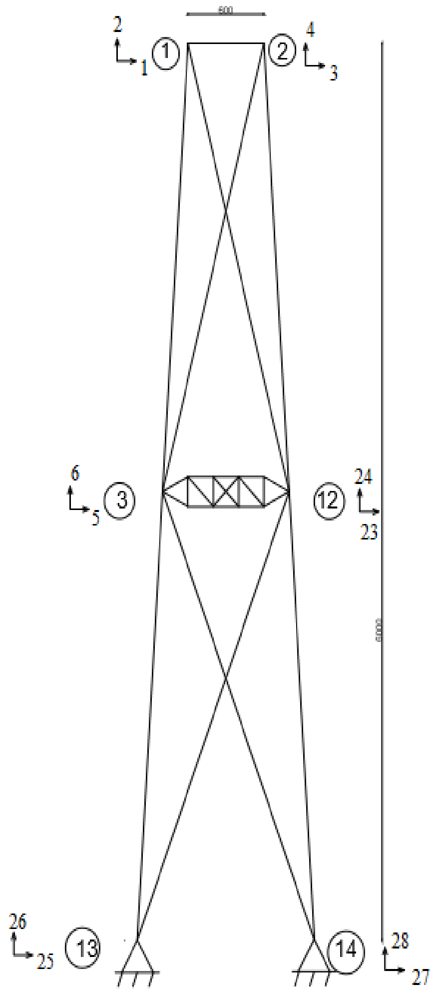

FIGURE 1: THE TWO-DIMENSIONAL HTS MODEL.

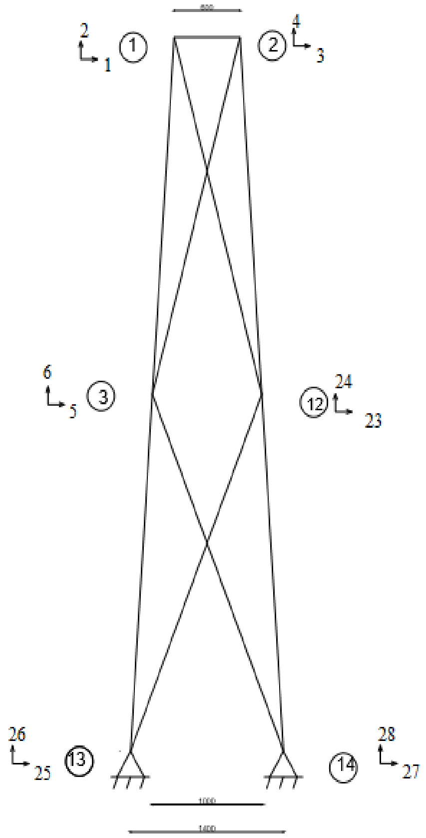

FIGURE 2A: SUBSTRUCTURE ONE

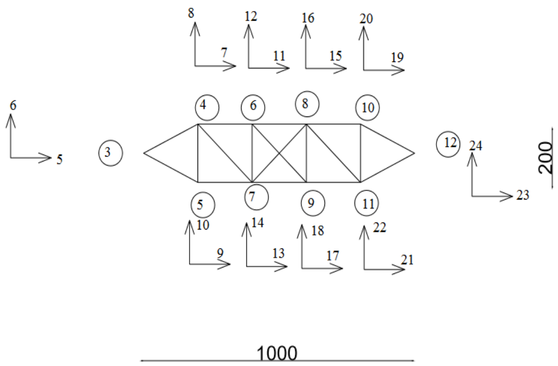

FIGURE 2B: SUBSTRUCTURE TWO (HORIZONTAL MEMBER WITH HIERARCHY).

3 RESULTS AND DISCUSSION

The displacements results derived from the substructuring method (SM) and FEM are all in millimetres (mm). The unconstrained displacement, D_f derived from the classical FEM is similar to the boundary and interior displacements derived by the substructuring method this is evident in Table 1. The member forces are all in Newtons, the member forces derived from SM and FEM are compared in Table 2 and show great semblances.

The results show that both results for substructuring technique(SM) and finite element method (FEM) are comparable. Substructuring method is a viable alternative to FEM approach.

TABLE 1: COMPARISON OF DISPLACEMENTS DERIVED BY SM AND FEM.

SM (*mm*)	**FEM (*mm*)**
0.0071	0.0061
0.0280	0.0261
0.0280	0.0268
0.0163	0.0148
0.0536	0.0558
-0.0018	-0.0025
0.0280	0.0224
-0.0233	-0.0190

TABLE 2: COMPARISON OF MEMBER FORCES DERIVED BY BOTH SM AND FEM.

SM (*N*)	**FEM (*N*)**
-169.0649	-157.08
692.82	591.64
238.73	218.55
-181.10	-157.08
-238.73	-231.66
733.51	758.43
627.02	591.64

4 CONCLUSION

A method of simplifying the analysis of hierarchical truss structure through the means of substructuring method is being proposed and demonstrated. The method has been interpreted into a Matlab program for ease of field application. A sample truss has been analyzed using the proposed approach and the results compare very well with a classical matrix structural analysis approach. For the displacements the percentage error was ranged from 28% to 3% while that of member forces ranged from 17% to 3%.

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1st International Civil Engineering Conference (ICEC 2018)
Department of Civil Engineering
Federal University of Technology, Minna, Nigeria

STRESS ANALYSIS OF CONTINUOUS SLAB ON ELASTIC FOUNDATION USING THE FINITE DIFFERENCE METHOD

*Ukonu, P. O.[1]; Sadiku, S.[1]; Aguwa, J. I.[1]
[1] Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author email: ukosonwa@yahoo.com

ABSTRACT
In this paper, finite difference method is used to determine the deflections, moments and stresses on a continuous plate resting on elastic foundation and subjected to uniformly distributed load for selected types of subgrades. The Winkler model is used to model the soil-structure interaction. The modified plate equation was then solved using the finite difference method. The results obtained for the various subgrades were compared. The comparisons show good agreement with the model.

Keywords: *Continuous plate, Elastic foundation, Finite Difference Method, Subgrade Modulus, Winkler model*

1 INTRODUCTION

Foundation, the basic and lowest part of a structure, is a structural element that transmits the load of the structure safely to the soil beneath. For soils of very low bearing capacity or for structures carrying excessive loads, mat or raft foundation is necessary. The raft slab is generally reinforced and sometimes where excessive loads from columns are expected, inverted beams are cast monolithically with the raft slab.

The inverted beam-slab arrangement of a mat foundation, foundation for railway tracks, and airport runway pavement present a model of continuous plates directly supported by the soil medium. They are quite often treated as resting on elastic foundation because the integral nature of the slab and soil is complicated by the complexity of the soil properties. Unlike the reinforced concrete materials of the slab, soil is heterogeneous, anisotropic, linearly inelastic and difficult to adequately sample. Therefore, it is necessary to make some simplifying assumptions in order to model the soil-structure interaction. One of such models is the Winkler model introduced by Winkler in 1867. It is a one-parameter model of the soil-structure interaction as obeying Hooke's law.

If the slab is a thin plate resting on the Winkler foundation, application of the classical theory of thin plates already available in literature leads to the non-homogeneous bi-harmonic governing differential equation.

Analytical approach to the solution of governing equation for continuous plates is, however, quite formidable. A purely mathematical procedure will involve equilibrium equations, constitutive relations, compatibility considerations as well as complex boundary conditions. Due to this mathematical complexity, researchers have developed and continued to develop more practicable computational techniques from numerical methods such as the Finite Difference Method (FDM), Finite Element Method (FEM), Boundary Element Method (BEM), and the Gridwork Method.

Timoshenko and Woinowsky-Krieger (1959), and Dolic´anin *et al* (2010) applied finite differences equations to the bending of thin plates of various edge conditions. Straughan (1990) analysed plates on elastic foundations using the method of finite differences. He developed a computational technique from a three-parameter model of the plate equation. Ventsel and Krauthammer (2001), and Szilard (2004) also used the finite difference method to develop finite difference stencils from the plate differential equation. They used the module to find sufficiently approximate deflection values for a continuous rectangular plate. They highlighted the advantages of the finite difference method as being straightforward to understand and apply, sufficiently universal, and well suited for computer application. Ali and Mahyar (2014) observed that the "finite difference method as one of the existing numerical methods is relatively strong method for numerical solution of the plate equations with different loading and support solutions conditions." For regions of complex shape, inaccuracies that may arise can be eliminated by coordinate transformation (Sadd, 2005).

It is, however, quite laborious to manually obtain and solve the finite difference equations for relatively fine mesh widths and multiple spans (Otto and Denier, 2005). This paper, therefore, attempts to use the finite difference method to automate the stress analysis of continuous slabs on elastic foundation.

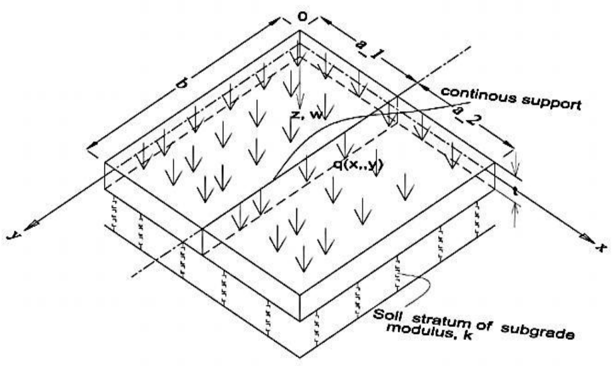

FIGURE 1: CONTINUOUS PLATE ON ELASTIC FOUNDATION

2 METHODOLOGY

2.1 PROBLEM FORMULATION

The Winkler model is based on the assumption that given a homogeneous, isotropic and elastic soil, the foundation reaction, $q^*(x,y)$, is directly proportional to the soil deflection w. That is,

$$q^*(x,y) = \mathrm{K}w \tag{1}$$

Where K represents the subgrade modulus of the soil reaction.

The application of the classical theory of thin plates already leads to the non-homogeneous bi-harmonic governing differential equation of the form

$$\mathrm{D}\left(\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4}\right) = q(x,y) \tag{2}$$

Where $q(x,y) =$ the surface load on the slab.

D = the flexural rigidity of the slab

D, the flexural rigidity of the slab, is given by

$$D = \frac{Et^3}{12(1-v^2)} \tag{3}$$

Where E =Elastic modulus of the slab material

$t =$ thickness of the slab

$v =$ Poisson ratio of the slab

If the plate is resting on an elastic foundation, then the external load acting in the lateral direction consists of the surface load on the plate $q(x,y)$, and the foundation reaction $q^*(x,y)$. Thus, the governing differential equation of the plate becomes

$$\mathrm{D}\left(\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4}\right) = q(x,y) - q^*(x,y) \tag{4}$$

Substituting (1), into (4), yields the governing differential equation of a plate on an elastic foundation written as

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} + \frac{\mathrm{K}}{D}w = \frac{q(x,y)}{D} \tag{5}$$

Stress analysis of the continuous slab resting on elastic foundation requires the solution of (5).

2.2 THE FINITE DIFFERENCE METHOD

The finite difference method is a computational technique that involves the replacement of the governing differential equation and the associated boundary conditions with finite difference equations. The finite difference quotients represent approximately the derivatives of the differential equations.

Consider the case of the continuous plate function $w(x,y)$ and let the plate's middle surface be approximated by a mesh with the mesh widths of Δx and Δy in x and y directions respectively as shown in figure 2.

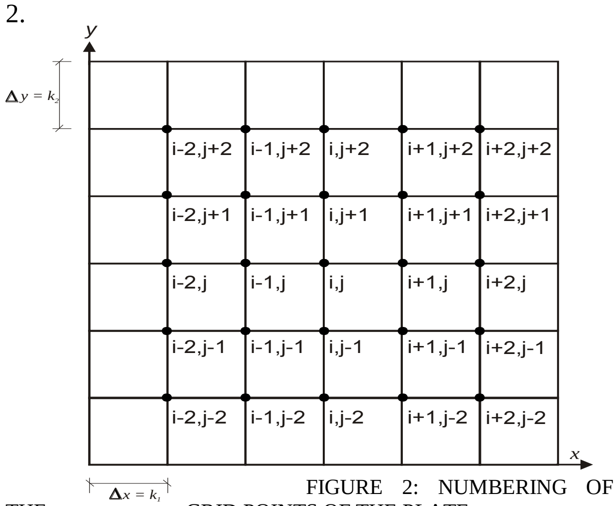

FIGURE 2: NUMBERING OF THE GRID POINTS OF THE PLATE

Take $\Delta x = k_1$, (6a)

and $\Delta y = k_2$ (6b) Also,

let $\alpha = \frac{\Delta x}{\Delta y}$ (7)

$$\therefore\ k_2 = \frac{k_1}{\alpha} \tag{8}$$

$$\left(\frac{\partial w}{\partial x}\right)_{i,j} = \frac{w_{i+1,j} - w_{i-1,j}}{2k_1} \tag{9a}$$

$$\left(\frac{\partial w}{\partial y}\right)_{i,j} = \frac{w_{i,j+1} - w_{i,j-1}}{2k_2} \tag{9b}$$

$$\left(\frac{\partial^2 w}{\partial x^2}\right)_{i,j} = \frac{w_{i+1,j} - 2w_{i,j} + w_{i-1,j}}{{k_1}^2} \tag{11a}$$

$$\left(\frac{\partial^2 w}{\partial y^2}\right)_{i,j} = \frac{w_{i,j+1} - 2w_{i,j} + w_{i,j-1}}{{k_2}^2} \tag{11b}$$

$$\left(\frac{\partial^3 w}{\partial x^3}\right)_{i,j} = \frac{w_{i+2,j} - 2w_{i+1,j} + 2w_{i-1,j} - w_{i-2,j}}{2{k_1}^3} \tag{12a}$$

$$\left(\frac{\partial^3 w}{\partial y^3}\right)_{i,j} = \frac{w_{i,j+2} - 2w_{i,j+1} + 2w_{i,j-1} - w_{i,j-2}}{2{k_2}^3} \tag{12b}$$

$$\left(\frac{\partial^4 w}{\partial x^4}\right)_{i,j} = \frac{w_{i+2,j} - 4w_{i+1,j} + 6w_{i,j} - 4w_{i-1,j} - w_{i-2,j}}{{k_1}^4} \qquad (13a)$$

$$\left(\frac{\partial^4 w}{\partial y^4}\right)_{i,j} = \frac{w_{i,j+2} - 4w_{i,j+1} + 6w_{i,j} - 4w_{i,j-1} - w_{i,j-2}}{{k_2}^4} \qquad (14b)$$

Also,

$$\left(\frac{\partial^4 w}{\partial x^2 \partial y^2}\right)_{i,j} = \frac{1}{{k_1}^2 {k_2}^2}\{2w_{i+1,j+1} - 4w_{i+1,j} + 2w_{i+1,j-1} + 2w_{i-1,j+1} - 4w_{i-1,j} + 2w_{i-1,j-1} + 4w_{i,j+1} - 8w_{i,j} + 4w_{i,j-1}\} \qquad (15)$$

The finite difference operators are better expressed in computational modules.

$$\left(\frac{\partial w}{\partial x}\right)_{i,j} = \frac{1}{2k_1}\left\{ 1 - 0 - 1 \right\}[w]$$

$$\left(\frac{\partial w}{\partial y}\right)_{i,j} = \frac{1}{2k_2}\left\{ \begin{array}{c} 1 \\ | \\ 0 \\ | \\ 1 \end{array} \right\}[w]$$

$$\left(\frac{\partial^2 w}{\partial x^2}\right)_{i,j} = \frac{1}{{k_1}^2}\left\{ 1 - (-2) - 1 \right\}[w]$$

$$\left(\frac{\partial^2 w}{\partial y^2}\right)_{i,j} = \frac{1}{{k_2}^2}\left\{ \begin{array}{c} 1 \\ | \\ -2 \\ | \\ 1 \end{array} \right\}[w]$$

$$\left(\frac{\partial^3 w}{\partial x^3}\right)_{i,j} = \frac{1}{2{k_1}^3}\left\{ -1 - 2 - 0 - (-2) - 1 \right\}[w]$$

$$\left(\frac{\partial^3 w}{\partial y^3}\right)_{i,j} = \frac{1}{2{k_2}^3}\left\{ \begin{array}{c} 1 \\ | \\ -2 \\ | \\ 0 \\ | \\ 2 \\ | \\ -1 \end{array} \right\}[w]$$

$$\left(\frac{\partial^4 w}{\partial x^4}\right)_{i,j} = \frac{1}{{k_1}^4}\left\{ 1 - (-4) - 6 - (-4) - 1 \right\}[w]$$

$$\left(\frac{\partial^4 w}{\partial y^4}\right)_{i,j} = \frac{1}{{k_2}^4}\left\{ \begin{array}{c} 1 \\ | \\ -4 \\ | \\ 6 \\ | \\ -4 \\ | \\ 1 \end{array} \right\}[w]$$

$$\left(\frac{\partial^4 w}{\partial x^2 \partial y^2}\right)_{i,j} = \frac{1}{{k_1}^2 {k_2}^2}\left\{ \begin{array}{ccccc} 2 & - & 4 & - & 2 \\ & & | & & \\ -4 & - & -8 & - & -4 \\ & & | & & \\ 2 & - & 4 & - & 2 \end{array} \right\}[w]$$

FIGURE 3: SOME FINITE DIFFERENCE MODULES FOR $w(x, y)$

Applying the foregoing finite difference operators on the governing differential equation for slab on elastic foundation as given in (1), together with (3), and (4), the molecule shown in figure 4 is obtained for interior grid points.

$$\frac{1}{{k_1}^4}\left\{ \begin{array}{ccccccccc} & & & & \alpha^4 & & & & \\ & & & & | & & & & \\ & & 2\alpha^2 & - & -4\alpha^2 - 4\alpha^4 & - & 2\alpha^2 & & \\ & & & & | & & & & \\ 1 & - & -4 - 4\alpha^2 & - & 6 + 8\alpha^2 + 6\alpha^4 + \beta & - & -4 - 4\alpha^2 & - & 1 \\ & & & & | & & & & \\ & & 2\alpha^2 & - & -4\alpha^2 - 4\alpha^4 & - & 2\alpha^2 & & \\ & & & & | & & & & \\ & & & & \alpha^4 & & & & \end{array} \right\}[w] = \frac{q(x, y)}{D}$$

Where $\beta = \frac{K{k_1}^4}{D}$

FIGURE 4: FINITE DIFFERENCE MODULE FOR INTERIOR GRID POINT

The finite difference modules for the bending moments (M_{xx},M_{yy}), and twisting moment (M_{xy}) at any point on the plate can also be obtained.

$$(M_{xx})_{i,j} = -D\left[\left(\frac{\partial^2 w}{\partial x^2} + v\frac{\partial^2 w}{\partial y^2}\right)\right]_{i,j} \qquad (16)$$

$$(M_{xx})_{i,j} = -\frac{D}{{k_1}^2}\left\{ \begin{array}{ccccc} & & v\alpha^2 & & \\ & & | & & \\ 1 & - & (-2 - 2v\alpha^2)_{i,j} & - & 1 \\ & & | & & \\ & & v\alpha^2 & & \end{array} \right\}[w]$$

$$\left(M_{yy}\right)_{i,j} = -D\left[\left(\frac{\partial^2 w}{\partial y^2} + v\frac{\partial^2 w}{\partial x^2}\right)\right]_{i,j} \qquad (17)$$

$$\left(M_{yy}\right)_{i,j} = -\frac{D}{{k_1}^2}\left\{\begin{array}{ccc} & \alpha^2 & \\ \nu & (-2\nu - 2\alpha^2)_{i,j} & \nu \\ & \alpha^2 & \end{array}\right\}[W]$$

$$\left(M_{xy}\right)_{i,j} = -D(1-v)\left[\frac{\partial^2 w}{\partial x \partial y}\right]_{i,j} \qquad (18)$$

$$\left(M_{xy}\right)_{i,j} = -\frac{D(1-v)}{4k_1k_2}\left\{\begin{array}{ccc} 1 & & -1 \\ & 0_{i,j} & \\ -1 & & 1 \end{array}\right\}[W]$$

2.3 BOUNDARY CONDITIONS

Solution of the governing plate equation, (5), by the finite difference method requires proper finite difference representation of the boundary conditions. Fictitious nodes outside of the plate are introduced to enable the evaluation of the boundary derivatives relying on, as may be applicable, zero deflection at the boundaries.

On a fixed boundary, there is zero deflection and slope. That is,

$$w_{i,j} = 0 \quad \text{and} \quad \left(\frac{\partial w}{\partial x}\right)_{i,j} = \frac{w_{i+1,j} - w_{i-1,j}}{2k_1} = 0 \qquad (19)$$

It follows that,

$$w_{i+1,j} = w_{i-1,j} \qquad (20)$$

If the boundary is simply supported, there is zero deflection and the moment. That is,

$$w_{i,j} = 0 \quad \text{and} \quad \left(\frac{\partial^2 w}{\partial x^2}\right)_{i,j} = \frac{w_{i+1,j} - 2w_{i,j} + w_{i-1,j}}{{k_1}^2} = 0 \qquad (21)$$

Hence,

$$w_{i+1,j} = -w_{i-1,j} \qquad (22)$$

These finite difference modules together with the boundary conditions are used to obtain a set of linear equation at every nodal point on the plate. This system of equations is solved simultaneously to obtain the deflection of the slab due to the lateral forces at these nodal points. The associated moments and stresses can also be obtained once the deflections are known. The MATLAB programming software is used to write and run the codes that automate these processes.

3 RESULTS AND DISCUSSION

3.1 NUMERICAL EXAMPLES

Given the values in Table 1, the finite difference method is used to analyse the continuous slabs on elastic foundation shown in figure 2 and figure 3 for various soil classifications.

TABLE 1: PARAMETERS OF THE DESIGN

Poisson ratio	0.3
Uniformly distributed load, q_0(kN/m)	100
Modulus of Elasticity, E (GPa)	30
Slab thickness (mm)	50
Support Condition	All edges simply supported
Mesh size ($\Delta x = \Delta y$)	0.25

3.2 EXAMPLE 1

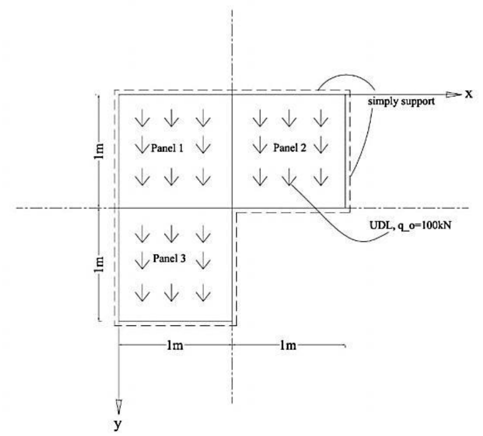

FIGURE 2: THREE SPAN SLAB CONTINUOUS

3.3 EXAMPLE 2

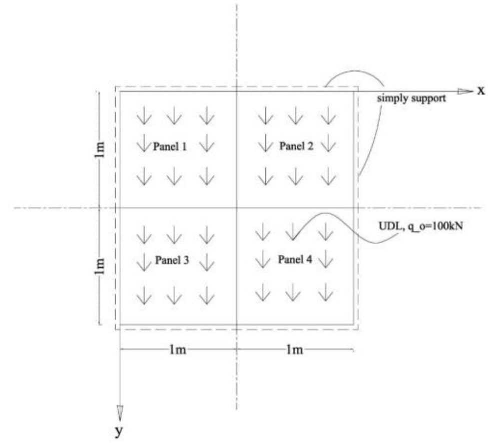

FIGURE 3: FOUR SPAN SLAB CONTINUOUS IN BOTH DIRECTIONS

3.4 RESULTS

In example 1, the centres of panels 2 and 3 experienced equal and maximum deflection. This is expected due to their symmetry, and to their having only one continuous edge. In example 2, maximum deflection occurred at the centre of each of the four panels. This is also expected due to their symmetry in loading and support conditions.

There is a marked increase in maximum deflection of the slabs as the soil stability reduced from very well graded gravel to low compressible organic soil (see figures 4 and 5) as a result of the decreasing subgrade resistance offered by the soil. In addition, figures 6 and 7 show an almost linear relationship between the subgrade modulus and the stress on the continuous slab. These phenomena are in agreement with the Winkler model adopted in this paper.

TABLE 2: DEFLECTION, MOMENT AND STRESS VALUES FOR VARIOUS SOIL CLASSIFICATIONS (EXAMPLE 1)

Soil Type	Subgrade Modulus ($\times \mathbf{10^3}$kN/$\boldsymbol{m^3}$)	$\boldsymbol{w_{max}}$ ($\boldsymbol{mm}$)	Moment at $\boldsymbol{w_{max}}$ (kN$\boldsymbol{m}$)			Stress at $\boldsymbol{w_{max}}$ (MPa)	
			M_{xx}	M_{yy}	M_{xy}	σ_{xx}	σ_{yy}
GW	163	0.4343	1.6813	1.5681	0.002515	4.0351	3.7634
GP	122	0.5032	1.9870	1.8553	0.002952	4.7688	4.4527
SW	95	0.5615	2.2462	2.0997	0.003389	5.3909	5.0393
SP	62	0.6531	2.6547	2.4854	0.004045	6.3713	5.9650
ML	49	0.6979	2.8555	2.6753	0.004483	6.8532	6.4207
CL	43	0.7206	2.9570	2.7711	0.004701	7.0968	6.6506
OL	27	0.7891	3.2621	3.0605	0.005139	7.8290	7.3452

KEY:
GW: GRAVEL WELL GRADED
GP: GRAVEL POORLY GRADED
SW: SANDY WELL GRADED
SP: SANDY POORLY GRADED
ML: MODERATE FINE SAND, SILT LOW TO MEDIUM COMPRESSIBILITY
CL: CLAY LOW TO MEDIUM COMPRESSIBILITY
OL: ORGANIC LOW TO MEDIUM COMPRESSIBILITY

TABLE 3: DEFLECTION, MOMENT AND STRESS VALUES FOR VARIOUS SOIL CLASSIFICATIONS (EXAMPLE 2)

Soil Type	Subgrade Modulus ($\times 10^3$kN/m^3)	w_{max} (mm)	Moment at w_{max} (kNm)			Stress at w_{max} (MPa)	
			M_{xx}	M_{yy}	M_{xy}	σ_{xx}	σ_{yy}
GW	163	0.3901	1.5382	1.5382	-0.007762	3.6917	3.6917
GP	122	0.4442	1.7875	1.7875	-0.009293	4.2900	4.2900
SW	95	0.4883	1.9914	1.9914	-0.01093	4.7794	4.7794
SP	62	0.5551	2.3008	2.3008	-0.0130	5.5219	5.5219
ML	49	0.5865	2.4470	2.4470	-0.01378	5.8728	5.8728
CL	43	0.6022	2.5193	2.5193	-0.01432	6.0463	6.0463
OL	27	0.6481	2.7337	2.7337	-0.01585	6.5609	6.5609

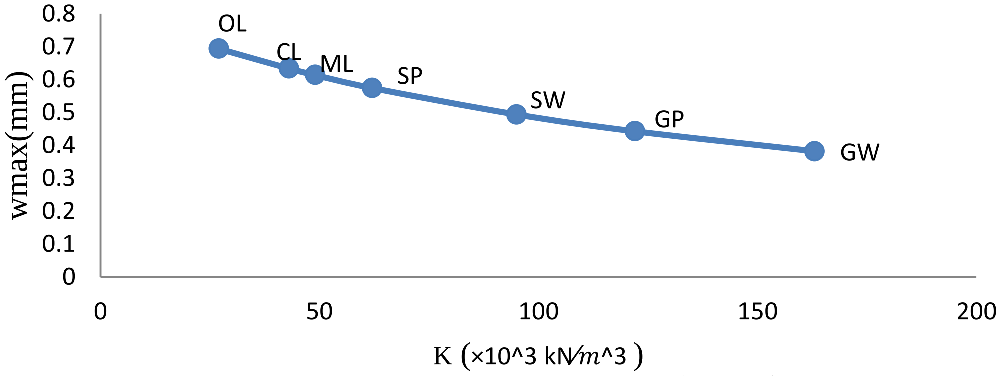

FIGURE 4: MAXIMUM DEFLECTION (WMAX)-SUBGRADE MODULUS (K) EXAMPLE 1

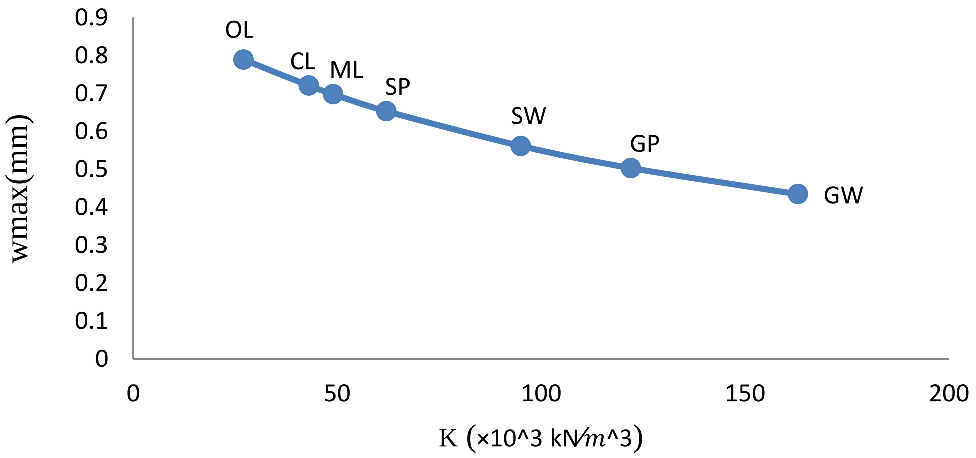

FIGURE 5: MAXIMUM DEFLECTION (WMAX)-SUBGRADE MODULUS (K). EXAMPLE 2

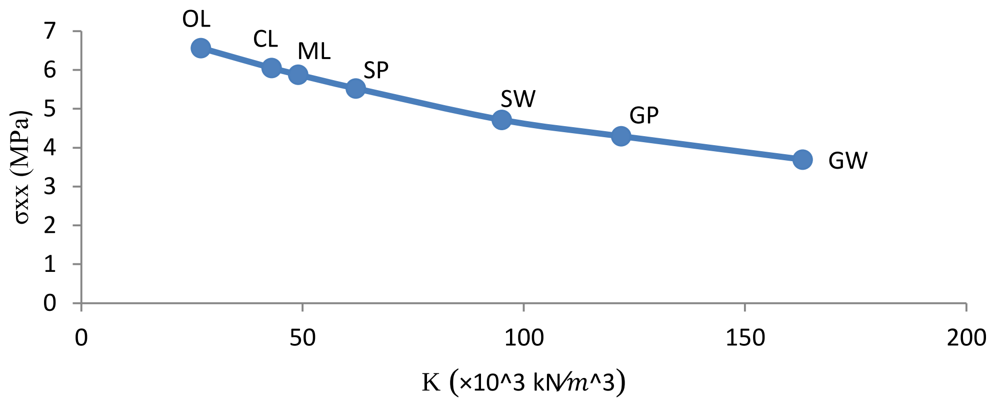

FIGURE 6: STRESS (ΣXX) AT POINT OF MAXIMUM DEFLECTION-SUBGRADE MODULUS (K). EXAMPLE 1

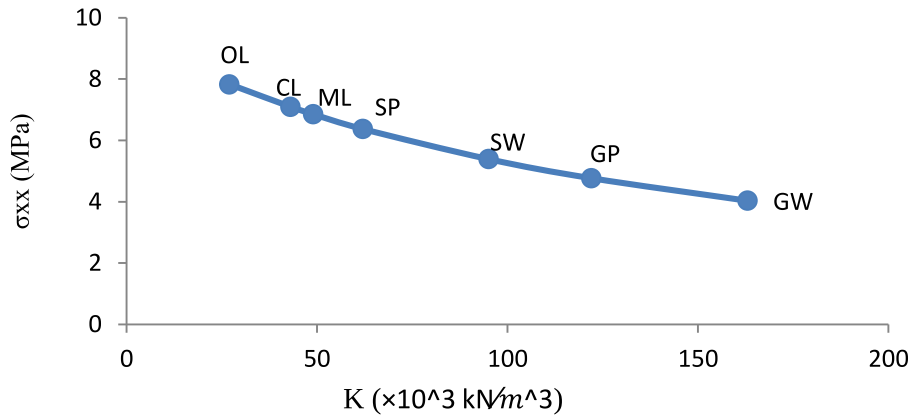

FIGURE 7: STRESS (ΣXX) AT POINT OF MAXIMUM DEFLECTION-SUBGRADE MODULUS (K). EXAMPLE 2

4 CONCLUSION

The Winkler model is sufficiently accurate in modeling the complex soil-structure interaction. This is because for a raft slab, compressive forces are dominant. The finite difference method is quite robust in application and programming as it involves the digitization of the slab, replacing, at the grid points, the governing differential equation with their equivalent finite differences. The results obtained are in good agreement with the fact that there exist an inverse proportion between deflection and the subgrade modulus for a constant lateral force as proposed in the Winkler model.

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COMPUTER-AIDED ANALYSIS OF REINFORCED CONCRETE WAFFLE BRIDGE DECK USING METHOD OF GRILLAGES

***Adamu, H. N.[1]; S. Sadiku [2]; B.Alhaji.[3];**
[1,2,3] Department of Civil Engineering Federal University of Technology, Minna
*Corresponding author email: h.adamu@futminna.edu.ng

ABSTRACT
This paper aims to analyse a reinforced concrete waffle bridge deck using method of grillages where the topping and ribs are analysed as a monolithic unit in contrast to the conventional methods where the slab and beams are analyzed differently. In addition, the grillage approach accounts for the torsion that is usually lost in the conventional approach. The slab loading is in line with the HA loading of the BS 5400 part 2 for lightly loaded (accommodation) bridges. One of the strength of this approach is that it is amenable to computer application which has been demonstrated by using a code written in Matrix Laboratory (MATLAB) software and therefore easy for field use by practitioners. For the purpose of rendering this approach amenable to computer application, a program was used to determine the displacements, bending moments and torsional moments in the bridge deck. It is observed that the values of bending moments obtained from grillage analysis are lower than the moments from conventional beam-slab analysis carried out manually. Bending moments and other responses generated by conventional beam-slab approaches are usually exaggerated thereby reducing the anticipated benefits of waffle slab

Keywords: *Computer-aided, Grillage analysis, MATLAB, waffle bridge deck.*

1 INTRODUCTION

Waffle slabs are structural elements with a combination of top slab and a system of spaced longitudinal and transverse beams. They are efficient in resisting lateral loads than flat slabs, and are suitable for large spans. They can withstand heavier load and cover large span as they exhibit higher stiffness and smaller deflections. The waffle slab system is an evolution of the solid slab that results from the elimination of concrete below the neutral axis which allows an economic increase on the total thickness of the slab with the creation of voids in a rhythmic arrangement. As a result, the self-weight of the structure is reduced (Schwetz *et al,* 2009).

For quite some time in Nigeria, the use of conventional reinforced concrete system was so prevalent that no serious attention was paid to ribbed slab. However, in recent years, there has been a sudden increase in the use of waffle slabs. That however, makes it necessary to examine new ways in which it can be used in construction.

Principally, static analysis of waffle slabs aim to determine the amount and distribution of shear forces bending moment and torsional moments acting on the structure.

Over the years, researchers have analysed waffle slabs substantially based on conventional methods; both analytical and numerical but less research have been carried out on the use of grillage analysis for waffle bridge decks. Other methods available in literature include plate analogy by Timoshenko (Halkude and Mahamuni, 2014) Rankine Grashoff method (Mohammed *et al,* 2013). However the direct stiffness gives more accurate results as concluded by (Halkude and Mahamuni, 2014).

Up until now, waffle slabs are found more in number in building construction than in bridge construction. An argument against this is that loads are distributed in two orthogonal directions in waffle slabs as against the one-way loading system in bridges. As a result, engineers deem it incompatible with bridges as loads are transferred in one way only in bridges. However, technical reasoning has shown that when loads are transferred to bridges in one way only, large twisting moments are produced, the orthogonal rib system in a waffle slab provides an efficient means of resisting these twisting moments by incorporating large bending moments in the two orthogonal directions (Kennedy and Bahkt, 1983).

(Vaignan and Prashad, 2014) analysed voided and cellular deck slab using MIDAS civil and concluded that rectangular shaped cellular decks withstand more load than voided slabs.

For this purpose, serious attention needs to be given to the analysis of waffle slabs as bridge decks. Several methods have been used in the analysis of bridges, in each, the three dimensional structure is simplified based on assumptions on geometry, materials and relationship between components. The accuracy of analysis is dependent on the method used.

Bridge decks have been analyzed using several methods such as finite element grillage analogy, orthotropic plate theory as seen in past literatures (Schwetz *et al,* 2009).

Grillage method of analysis involves representing the bridge deck as a 2 by 2 system of interconnected beams intersecting each other. It is a numerical approach in analyzing bridge decks. It is easy to use and comprehend (Shreedar and Kharde, 2013).

According to (Shreedar and Kharde 2013) Lightfoot and Sawko made this method of analysis using grillages open to computer programming

As structures become complex and large, several methods of simplifying their analysis have been developed, among this is the computer aid. Computer aided analysis is a way of solving continuous system problems by dividing them into discrete elements thus simplifying analysis taking into consideration of compatibility and boundary conditions. A waffle slab is a three dimensional and complex structural element whose analysis requires very cumbersome calculations, hence the use of computer program for analysis. In this paper, the analysis of a waffle bridge deck using method of grillages was performed using direct stiffness method and MATLAB software as tools for writing the program as well as the analysis.

In the grillage analysis, the structure is represented by a plane grillage of discrete but interconnected beams. Almost any arrangement in plan is possible, so skew, curved, tapering or irregular decks can be analysed. But the usual layout is sets of parallel beams in two directions by assuming the plane of the grillage to be horizontal.

In a simple form of grillage analysis, each beam is assigned a torsional stiffness and flexural stiffness in the vertical plane. Vertical loads are applied only at the intersections of the beams. The matrix stiffness method of analysis is used by the existing software, to find the rotations about two horizontal axes and the vertical displacement at these nodes, and hence the bending and torsional moments and shear forces in the beams at each intersection. Warping stresses and shear lag are neglected in the analysis.

Location of grid lines

1. Grid lines should be adopted along line of strength.
2. The longitudinal gridlines run in parallel direction to the edge of the deck that is free. For longitudinal direction, it may be along the longitudinal webs, centre line of girders or edge beams etc.
3. Where isolated bearings are present, the grid line may be along the line joining center of bearing.
4. For transverse direction, it should be considered as one of each end connecting the center of bearing and along the center line of transverse beam (Surana And Agrawal, 1998).

Number and spacing of grid lines.

1. Where possible, odd numbers of gridlines should be chosen in both longitudinal and transverse directions.
2. The ratio of spacing of transverse grid line to those of longitudinal grids may be taken as 1 to 2.
3. As regards to the depth of slab, the minimum distance between longitudinal grid lines is limited to two to three times of the slab depth and the maximum separation of longitudinal members should not be more than one fourth of the effective span (Pandey and Maru, 2015).

A typical output gives the external reactions at each support. The bending and torsional moments will, in general, show a discontinuity at each joint. For an orthogonal grillage, each change in bending moment is equal to the change in torsional moment at that joint in the member at right angles to the one considered. Similarly, the change in torsional moment equals the change in bending moment in the perpendicular member.

Approximately one half of the local load can be distributed over the eight nodes of the vicinity to get correct results, even near the loaded point. An appropriate idealization for a continuous structure must be carefully selected. Each T-section of the longitudinal and transverse sides of a waffle slab is represented by a grillage beam. The transverse grillage members should extend to the edge of the real slab and their ends should be attached to longitudinal grillage beams, even if the real slab has no significant edge stiffening.

2. METHODOLOGY

Problem Formulation

TABLE 1: BRIDGE DATA

Bridge Deck Properties	
Width	7.3m
Width of Notional Lanes	3.65m
Thickness of Slab Topping	0.075m
Depth of Bridge Deck	0.37m
Width of Grid Beam	0.15mm
Depth of Asphalt Overlay	0.05m
Grade of Concrete	M30
Grade of Steel	E460
Rib spacing	1.2m

The loading considered is the self-weight, wearing course. The live load on the floor is HA loading as given in BS 5400-part 2 (1987) Clause 6.2.1. Load combination 1 of the BS 5400 part 2 is used. In this, eleven transverse members and five longitudinal members have been modeled. The grillage model has 77 nodes, 136 members and 231 degrees of freedom.

$$E = 3.61 \times 10^{7} kN/m^{2}$$
$$G = 1.57 \times 10^{7} kN/m^{2}$$
$$I = 1.93 \times 10^{-4} m^{4}$$
$$J = 1.063 \times 10^{-3} m^{4}$$

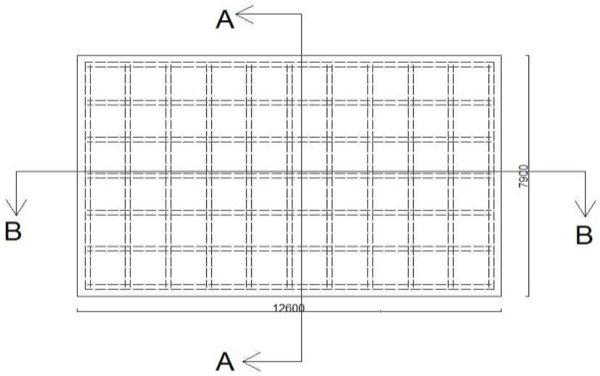

FIGURE1: PLAN OF WAFFLE BRIDGE DECK

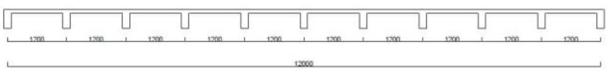

FIGURE 2: TRANSVERSE SECTION OF WAFFLE BRIDGE DECK.

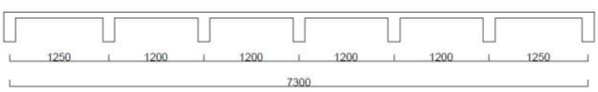

FIGURE 3: LONGITUDINAL SECTION OF THE WAFFLE BRIDGE DECK.

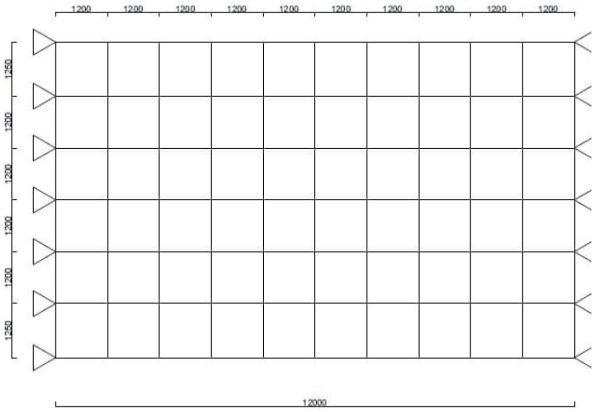

FIGURE 4: IDEALIZED GRILLAGE MODEL OF BRIDGE DECK.

Analysis using MATLAB 2015a software (stiffness method)

1. Defining the nodal coordinates.
2. Numbering of numbers.
3. Defining the connectivity of elements
4. The length and angle of orientation
5. Material properties are modulus of elasticity and rigidities are defined.
6. For each element, the stiffness matrix computed the software.
7. The stiffness matrix for a grid member is a 6 by 6 matrix.
8. First the degrees of freedom at each node are identified and numbered; two perpendicular rotational displacement and one translational displacements$\Delta_1, \theta_2, \theta_3$.
9. The structures stiffness matrix for two nodes (one element) becomes;
   The global stiffness matrix is obtained by combining all the element stiffness matrices.
10. Assignments of boundary conditions.

Formulation of stiffness matrix

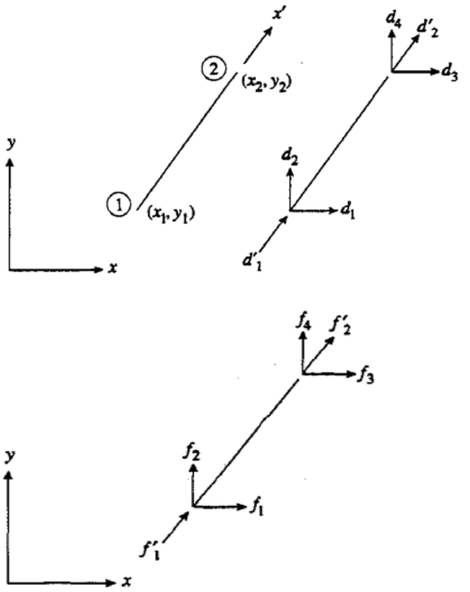

FIGURE 5: NODAL DEGREES OF FREEDOM FOR A GRID ELEMENT

Mathematical Model;

$$EI\frac{d^4v}{dx^4} = q \tag{1}$$

$$EI\frac{d^2v}{dx^2} = M \tag{2}$$

$$EI\frac{d^3v}{dx^3} = F \tag{3}$$

$$EI\frac{d^4v}{dx^4} = 0 \tag{4}$$

Integrating

$$EIv = a_0 + a_1x + a_2x^2 + a_3x^3 \tag{5}$$

The rotational degree of freedom

$$\frac{dv}{dx} = 0; \tag{6}$$

Applying boundary conditions

Solving for coefficients,

$$x = 0: \frac{dv}{dx} = 0;\ v = 1; \Rightarrow a_0 = 0\ and\ a_1 = 1 \tag{7}$$

$$x = L: \frac{dv}{dx} = 0;\ v = 0 \tag{8}$$

$$\frac{dv}{dx} = 0; \Rightarrow 2a_2 + 3a_3l \tag{9}$$

$$v = 1 + a_2l + a_3l^3 \tag{10}$$

$$\Rightarrow a_2 = \frac{3}{l^2}\ and\ a_3 = \frac{2}{l^3} \tag{11}$$

Equation (5) becomes

$$v = 1 - \frac{3x^2}{l^2} + \frac{2x^3}{l^3} \tag{12}$$

$$-EI\frac{d^3v}{dx^3} = F \Rightarrow -EI\left(\frac{12}{L^3}\right) = \frac{12}{L^3}EI \tag{13}$$

$$EI\frac{d^2v}{dx^2} = M \Rightarrow M_{x=0} = -\left(\frac{6}{L^2}\right) \tag{14}$$

$$k_{11} = -F_{x=0} = EI\left(\frac{12}{L^3}\right) \tag{15}$$

$$k_{21} = -M_{x=0} = EI\left(\frac{6}{L^2}\right) \tag{16}$$

By imposing a twisting moment at node 1, giving a rotation θ and applying boundary conditions the constant of integration found to be

$$T = \frac{GJ}{L}\theta \tag{17}$$

Therefore,

$$k_{33} = \frac{GJ}{L} \tag{18}$$

The remaining forces acting on the grid beam can be determined by applying unit displacement corresponding to translation and rotation at the two nodes of the beam.

$$\begin{bmatrix} k_{11}k_{12}k_{13}k_{14}k_{15}k_{16} \\ k_{21}k_{22}k_{23}k_{24}k_{25}k_{26} \\ k_{31}k_{32}k_{33}k_{34}k_{35}k_{36} \\ k_{41}k_{42}k_{43}k_{44}k_{45}k_{46} \\ k_{51}k_{52}k_{53}k_{54}k_{55}k_{56} \\ k_{61}k_{62}k_{63}k_{64}k_{65}k_{66} \end{bmatrix} = \begin{bmatrix} \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{-12EI}{L^3} & \frac{6EI}{L^2} & 0 \\ \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{2EI}{L} & 0 \\ 0 & 0 & \frac{GJ}{L} & 0 & 0 & \frac{-GJ}{L} \\ \frac{-12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} & 0 \\ \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} & 0 \\ 0 & 0 & \frac{-GJ}{L} & 0 & 0 & \frac{GJ}{L} \end{bmatrix}$$

3. RESULTS AND DISCUSSION

Analysis of a waffle bridge deck have been carried by using grillage analogy method by simulating full the HA loading. The displacements are shown in table (2) bending moments are shown in table (3). The bending moments are estimated from the summation of member forces of adjacent members.

TABLE 2: DISPLACEMENTS AT EDGE AND MIDDLE LONGITUDINAL RIBS

Node numbers	y-translation(m)	x-rotations(radians)	Z-rotations(radians)
1	0	-0.0022	0
11	0	0.0020	0
12	0	-0.0057	0
13	-0.0059	-0.0045	-0.0066
14	-0.0104	-0.0034	-0.0123
15	-0.0139	-0.0029	-0.0169
16	-0.0170	-0.0025	-0.0204
17	-0.0190	-0.0001	-0.0255
18	-0.0200	0.0019	-0.0242
19	-0.0173	0.0028	-0.0209
20	-0.0135	0.0041	-0.0158
21	-0.0078	0.0059	-0.087
22	0	0.131	0
34	0	-0.0155	0
35	-0.0212	-0.0158	0
36	-0.0388	-0.0115	0
37	-0.0512	-0.007	0
38	--0.0593	-0.0047	0
39	-0.0627	0.0001	0.0000
40	-0.0592	0.0047	0
41	-0.0511	0.0078	0
42	-0.038	0.0115	0
43	-0.0211	0.015	0
44	0	-0.0000	0

The rotation in the vertical direction which causes torsion is higher towards the edge of the bridge deck. And then reduces towards mid-span of the deck. There is no rotation in the vertical direction at midspan in the longitudinal direction. This can be attributed to the symmetry in the grillage layout of the deck. This results in zero twisting moment in the longitudinal rib.

TABLE 3: COMPARISON BETWEEN VALUES OF BENDING MOMENTS (LONGITUDINAL RIB)

Node numbers	Bending moments(grillage analysis) kN-m	Bending moments(manual analysis) kN-m
Internal Longitudinal rib		
13	64.43	386.2
14	52.73	686.7
15	39.11	901
16	24.93	1030
17	1.46	1073
18	-23.93	1030
19	-39.47	901
20	-53.2	686.7
21	-45.19	386.2

TABLE 4: COMPARISON BETWEEN VALUES OF BENDING MOMENTS (TRANSVERSE RIB)

Node numbers	Bending moments(grillage analysis) kN-m	Bending moments(manual analysis) kN-m
Internal transverse rib		
6	3.63	132
17	43.97	113
28	27.07	177
39	1.48	198
50	-24.14	177
61	-46.36	113
72	-3.63	132

The values of bending moments obtained in the grillage analysis are lower than those obtained manually with beam line analysis, it is as a result of the neglect of twisting moments in the beam line analysis that is, twisting of beam is neglected. The negative values of bending moments is caused by the effect of support conditions adopted in the analysis. The stepped or saw-tooth values of bending moments in grillage analysis is as a result of discontinuity at the nodes and difference in the values of bending moments in adjacent beams which results from the values of displacements of member elements. The bending moments are higher at the supports to resist the large twisting moments developed at the edges and reduces gradually towards the middle at there is no twisting moment.

4. CONCLUSION

The static analysis of a waffle slab bridge deck was carried out using grillage analysis which is computer amenable and compared with manual beam line analysis. Based on the comparative study of bending moment of longitudinal and transverse rib, more economical designs can be obtained using the grillage method. The negative bending moments developed in the slab can be resisted by making the supports solid. The method also makes it possible for the slab to be analsed as an entity instead of the conventional slab-beam analysis. Large twisting moments developed at the support and edge of a bridge deck can be minimized using waffle slabs for bridge decks.

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A GENETICALLY OPTIMIZED MODEL IN DETERMINING APPLICATIONS FOR DESIRED STREAM CIPHER

* Joshua Mamza Edward, John Kolo Alhassan and Joseph Adebayo Ojeniyi

Department of Cyber Security Science, Federal University of Technology Minna

*Corresponding author email: ojeniyija@futminna.edu.ng

ABSTRACT

Encryption plays a very vital role in the field of stream cipher application. Most of the stream ciphers keys are use for encryption of data and it changes randomly to generate a cipher text that is not possible to break it mathematically. The intention of this research examine why stream ciphers cannot provide authentication and integrity. The research also aimed at to determine a model in which a stream cipher would be desirable by providing authenticity and Integrity of encrypted messages and decrypted messages. Genetic Algorithm Flowchart was used on the flow chat and genetic algorithm was obtain. We proposed model for determining the desirability of a stream cipher, the two security pillars that are authentication and integrity was achieved from gene formation to reproduction with the help of genetic algorithm which identify the processes of evaluation and selection of chromosome to the best chromosomes.

Keywords: *Stream Cipher, Encryption, Gene algorithm, Cipher Text, Plaintext text, authenticity and integrity*

1 INTRODUCTION

Encryption is basically is the application of key to run a message such that that message cannot be read by someone but only by the holder of the correct key. The use of computers now has become part of our life, and most importantly the need to protect these information on these computers, encryption usage has expanded most especially in email and instant messages applications (Kamesh & Sakthi Priya, 2014). Steam cipher is defined as a symmetric key encryption where all bits of data are encrypted with each bit of key. The key is called a " Crypto Key" which is used for the encryption of the data and changes randomly to generate a cipher text that is impossible to break it mathematically(Kamesh & Sakthi Priya, 2014). Also, stream cipher is symmetric cipher which adds the key stream to the plaintext generating a cipher text. Stream ciphers are divided into two types, synchronous stream ciphers. Stream ciphers are commonly classified as synchronous or self-synchronizing. Streams ciphers also consist of two parts; a key scheduled algorithm and key generator (Crainicu, 2017). Symmetric key cryptography is an encryption in which both the sender of the message and the recipient of the message have same key (Krishna, Ravi, & Bhattu, 2018). Some of the advantages of symmetric key encryption are (i) it is faster than public key (ii) Uses fewer amount of computational resources (iii) it is quite simple and easy to perform encryption (iv) requires less memory (Krishna et al., 2018). If a key lost, the message between the sender and the recipient are influenced. Figure 1 shows the Diagram of a stream cipher.

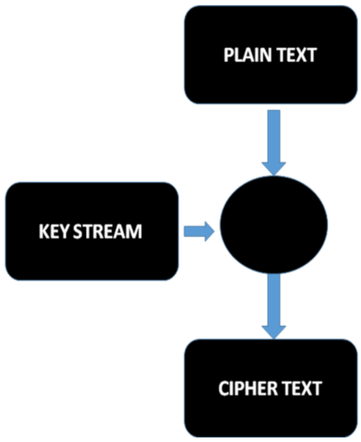

FIGURE 1: BLOCK DIAGRAM OF STREAM CIPHER

In Addition, a stream cipher uses a standard key generator on the key so that the key matches the length of the plain text and then encrypts the plain text. Stream ciphers takes plaintext which produces cipher text using pseudorandom generated keystream.(Ramesh, Mishra, & Nayak, 2016). In stream cipher, bit operation of plaintext which the matching key stream digit gives a ciphertext.(Ramesh et al., 2016).

1.2 RELATED WORK

Encryption is defined as the change of plain message form into cipher text message which cannot be understood by anyone without decrypting the encrypted message form. Decryption is the reverse of encryption. (John & Manimurugan, 2012)(Bharath, Rakshith, Karunakar, Nayak, & Deepashri, 2015).

Gutha Jaya, Vadlamani R. and Ravi S. Nagesh introduces key generation for plain text in a stream cipher via Bi-objective evolutionary computing based key generation. Here, evolutionary computing algorithm was used to generate keys. The evolutionary algorithms have fitness function based on which keys are generated. In the proposed work the fitness function has two objectives. Both objectives have to be minimized. The both objective functions, ASCII code values between 33 and 77 were utilized. These values then become the lower and upper bounds of solution vector generated from optimization algorithms (Krishna et al., 2018). Lu Zhou, Jiageng Chen, Yidan Zhang and Anthony James investigated and carried out a task on building an automatic encryption scheme based on neutral networks. It started from symmetric key model; they investigated the security of the scheme on several statistical models to show the original proposed scheme is not secure. They eventually extended the original model by investigating other powerful adversaries. They proposed encryption schemes are more strong and flexible in resisting against various attacks. Future work include how to further optimized the neutral network to make the legal party communication more efficient given less training steps and how to other security solutions with rich functionality by using network is worth investigation (Zhou, Chen, Zhang, Su, & Anthony James, 2018). Ashan VC proposed WG stream cipher used an innovation function block, which increases the security of private data. Involution function takes only a small amount of hardware. With the integration of involution function in the stream cipher, the randomness property of WG cipher increased. The proposed WG stream cipher is developed using Xilinx and implemented on FPGA (Ashan, 2016). Cryptanalysis of chaotic stream cipher has been presented by Skrobek Andrian (Skrobek, 2007) which sated that a chaotic system are characterized by the properties which are promising for the design of stream ciphers. However, he suggested an improvement of the cryptanalysis the following weaknesses of the encryption algorithm was observed: (i) One part of the key depends on the other (ii) The first two blocks are always enciphered with the same key. (iii) a number of the keystream bits are predictable. (iv) Systems are vulnerable for running out of control (Skrobek, 2007). Qingchun Zhao and Hongxi Yin proposed a setup of high speed physical layer stream ciphers whose key is generated by chaotic semiconductor laser. The encryption and decryption processes are numerically simulated. The future topics of this field are how to use TRNs (True Random Numbers) generated by chaotic LD (Laser Diode) in block cipher and public key cipher (Zhao & Yin, 2013).

1.3 PROBLEMS STATEMENT

The purpose of this research work is to examine why stream ciphers cannot provide authentication and integrity.

2.2 OBJECTIVE OF THIS RESEARCH

The objective of this research is to determine a model in which a stream cipher would be desirable by providing authenticity and Integrity of encrypted messages and decrypted messages.

2 METHODOLOGY

Let $x = (x_1, \ldots, x_n)$ denotes a binary vector. For notational simplicity we restricted the discussion to binary variables $x_i \in \{0,1)$. We use the following C Capital letters X_i denotes variables, small letter x_i assignments.

DEFINITION 3.1. Let a function $f : X \quad R \geq 0$ be given as. We consider the optimization setback

$$X_{opt} = \operatorname{argmax} f(x) \quad . \quad . \quad . \quad 3.1$$

DEFINITION 3.2 Consider two strings **a** and **b** be given. In one point crossover the string **c** is created by randomly choosing a crossover point $0 < l < n$ and setting $c_i = a_i$ *for* $i \leq l$ and $c_i = y_i$ for $i > l$. In uniform crossover, c_i is randomly chosen with equal probability from ($\{a_i$ *and* $b_i\}$

DEFINITION ***3.3*** *Let p(x,t)* denote the probability of x in the probability of x in the population at the generation t. Then $p_i(x_i, t) = \sum_x, x_i = x_i \, P(x,t)$. Note that if one generation is discussed we always write $p_i(x)$

The fitness of the population and variable is given by $f(t) = \sum p(x,t) f(x)$

$$V(t) = \sum p(x,t) \quad (\dot{f}(x-))^2$$

The response of selection R(t) is usually define as stated below

$$R(t) = \dot{f}(t+1) - \dot{f}(t) \quad . \quad . \quad . \quad 3.2$$

3.4 THE PROPORTIONATE SELECTION

The proportionate selections changes the probability as stated below:

1. $p(x,t+1) = p(x,t) f(x)/f(t) \quad . \quad . \quad 3.3$

Lemma 3.1 For proportionate selection the response is given by

$$R(t) = \quad V(t)/f(t) \quad . \quad . \quad . \quad 3.4$$

Proof: We have

$$R(t) = \sum p(x,t) \frac{f2}{f(t)}) - f(t) = \frac{V(t)}{f(t)} \quad . \quad . \quad 3.5$$

Therefore, with the proportionate selection the average fitness never decreases. This is certainly true for every rational selection scheme.

3.5 RECOMBINATION

The analysis of combination, we introduced a special distribution.
Definations 2.4 *Robbins proportions are given by the distribution π*
$\pi(x,t) := \prod_{i=1}^{n} p_i \ (xi, t)$ *...3.6*

A populations in Robins proportional is also called LINKAGE EQUILIBRIUM.
Geiringer [Gei44] has clearly revealed that all reasonable recombination scheme leads to the same limit of distribution.

Theorem 3.1 (Geiringer). *Recombination does not change the univariate marginal frequencies i.e* p_i *(xi, t+1) =* p_i *(xi, t). The limit distribution of any complete recombination scheme is Robbins proportional π (x).*
A complete combination means that for every subset of S of { 1, …,n), the probability of an exchange of genes by recombination is greater than zero. The convergence also to the limit of distribution is very fast. This is an important fact. We take the uniform distribution as an example. Here linkage equilibrium is given as $p(x) = 2^{-n}$. This value is obtainable if the size of the population N is substantial larger than 2^n!. For a population of N = 1000, the minimum DSQ_{min} for Robbins Proportions is already achieved after four generations, then DSQ slowly increases due to the stochastic fluctuation of a ***genetic drift.*** Finally, the population will then be made up of one genotype only. Genetic drift was analyzed by Asoh & Muhlenbein [AM94B] and it will be considered here.

3.6 SELECTION AND RECOMBINATION

We have clearly shown that the average $\dot{f}$ *(t)* never decreases after selection and that any complete recombination scheme moves the genetic population to Ribbons proportions. The question is: **What happens if recombination is applied *after* selection**? The answer would be very difficult. Therefore, the problems still puzzles populations' genetics. The equation can be written as: Let a recombination distribution R be given. $R_{x,y,z}$ denotes the probability that *y and z* produces *x* after recombination. Therefore

$$P(x, t+1) = \sum_{y,z} \mathrm{R}\ \mathrm{x,yz}\ p^s(\boldsymbol{y}) p^s(\boldsymbol{z}) \quad .3.7$$

p^s (x) represent the probability of string of x after selection.
For *n* loci the recombination distribution R consists of $2^n * 2^n$ parameters. A mathematical analysis of the mathematical properties of *n* loci system is difficult. For a problem of size *n we have* 2^n equations. Furthermore the equations depend on the recombination operator that is used. If the gene frequencies remain in linkage equilibrium, then only n equations are needed for the marginal frequencies. Therefore the most important question is: Does the optimization process gets worse because of this simplification? No. This shows that the univariate marginal frequencies are the same for all recombination scheme if applied to the same distribution p(*x*,*t*).

Theorem 3.2 *For any complete recombination/crossover scheme used after proportionate selection the univariate marginal frequencies are determine by*

$$p\ (xi, t) = \sum_{x|Xi=xi} \frac{p(x,t)f(x)}{\bar{f}\ (t)} \qquad 3.8$$

Proof: After selection the univariate marginal frequencies are given by

$$p^s\ (xi, t) = \sum_{x|Xi=xi} p^s(x,t) = \sum_{x|Xi=xi} \frac{p(x,t)f(x)}{\bar{f}\ (t)}$$

Now the selected individuals are randomly paired as shown in the equation below:
p_i (x_i, t+1) = p_i^S(x_i , t).

3.7 Schema Analysis Demystified

Theorem 2.2 can be formulated in the terms of Holland's Schema theory. Let $H_{(xi)}$ = (*,…,*,x_i , *,…,*) be first order schema at locus *i*. This schema includes all the strings where the gene at locus is fixed to x_i . The univariate marginal frequency p(x_i, *t)* is clearly identical to the frequency of the schema $H_{(x_i)}$. The fitness of the schema at generation *t is given by*

$$f\ (H(x_i), t = \frac{1}{p_i\ ((x_i), t)} \sum_{x|Xi=xi} \frac{p(x,t)f(x)}{\bar{f}\ (t)}. \quad .. \qquad 3.9$$

From Theorem 2,2 we have:
Corollary 2.1 ***(First – order Schema Theorem)****. For a genetic algorithm with proportionate selection using any complete recombination the frequency of first-order schemata changes according to*

$$p_i\ (x_i, t+1) = p_i\ (x_i, t)\ \frac{f(H((x_i, t)}{\bar{f}}$$

(Mühlenbein & Mahnig, 2002)
Hence, Fitness function is $(f_t) = \frac{1}{1 + objective\ function}$
The Objective function is obtained from the Mean Square Error of the Genetic Output. Therefore,

$$\text{Minimize} \quad (f_t) = \frac{1}{1 + \frac{1}{N}\sum_X^N \quad (DES_X - ACT_X)^2)} \qquad .\ 3.10$$

Subject to the following constraints:
a ≤ $DES_X \le b$
$c \le ACT_X \le d$
n > 0

Where N = is the size of the entire training *dataset*
DESx is the desired password outpout from using the *training dataset number* X,

ACTx is the password output when using the *training dataset number* X, as an input.

4.0 METHODOLOGY

4.1 Genetic Algorithm Flowchart

Based on the flowchart in Figure 2, the procedures of obtaining the genetic algorithm are
i. Objective function formation ii. Encoding iii. Gene formation iv. Chromosome development v. population selection vi. Cross over vii. Mutation viii. Reproduction ix. Next generation until fitted v. Decoding

- Gene Formation: 0's and 1's are stream cipher bits that form byte.
- Byte now chromosome (That's group of bytes)
- Each chromosome will then form different parent.
- Parents form population. These identify options for the population of the genetic algorithm. Population types identify the type of the input that will be provided to fitness function.
- Crossover occurs when parent meet together. Crossover combines two or more individuals, or parents, to form a new individual child that form the next generation
- Mutation makes random small changes in individual based on the population. This occurs when you pick the gene formation at random and change it that is 0 to or 1 to 0.
- Reproduction is production of an offspring. Reproduction verifies how the genetic algorithm creates children at each new generation.

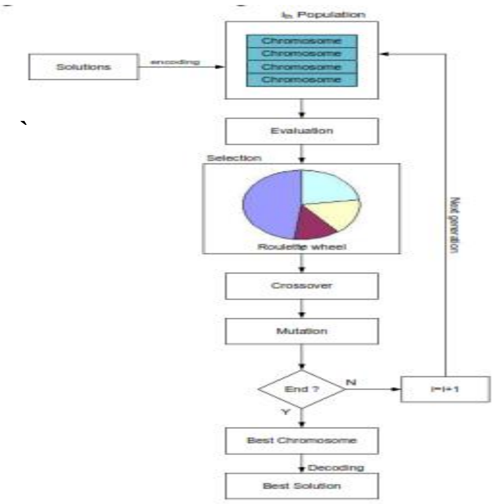

FIGURE 2: GENETIC ALGORITHM FLOWCHART

5.0 RESULT AND DISCUSSION

Given our variables =2 and our bounds $\begin{matrix} 1 & 999 \\ 2 & 1000 \end{matrix} = \begin{matrix} a & b \\ c & d \end{matrix}$ as stated to minimized fitness function subject to the following constraint as stated in equation 3.10 above. After running the optimizing tool, the following result was generated.

5.1 PLOT FUNCTIONS

The results of the proposed genetically optimized model are presented from Figure 3 to Figure 7. Fitness values across the generations are shown in Figure 3. The best value was attained at 0.000108759 at mean value of 0.000108952. The first variable gives the best individual consideration. Average distance between individuals across generation and fitness scaling is shown in Figure 4. The average distance decline drastically across generations with stably decline in expected scaling in fitness. The genealogy of the individuals shows an evenly distributed range over the generations as in Figure 5. Normal diversity score was recorded in fitness of each individual as shown in Figure 6. The selection function was evenly distributed even though poor in stopping criteria as presented in Figure 7.

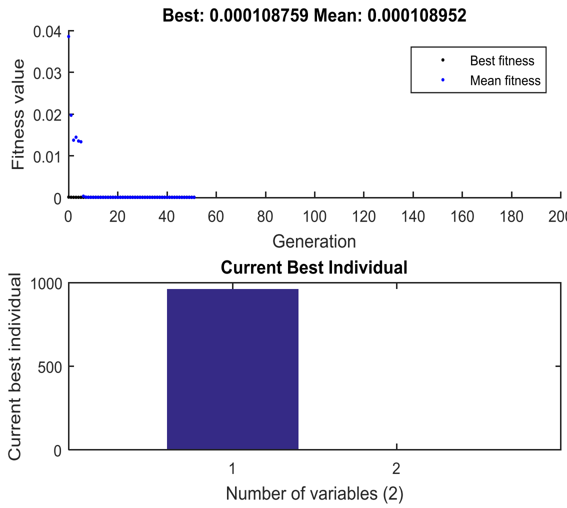

FIGURE 3: BEST FITNESS AND BEST INDIVIDUAL PLOT FUNCTION

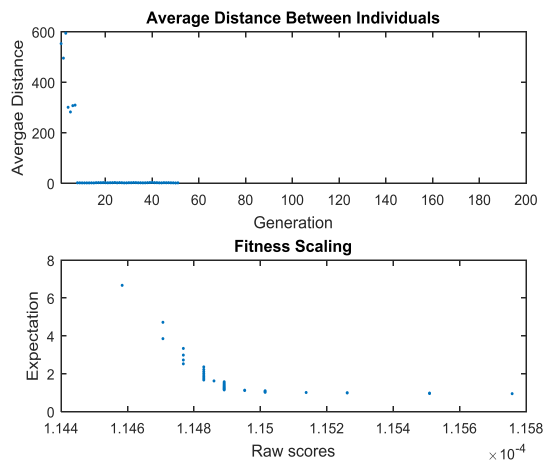

FIGURE 4: DISTANCE & EXPECTATION PLOT FUNCTION

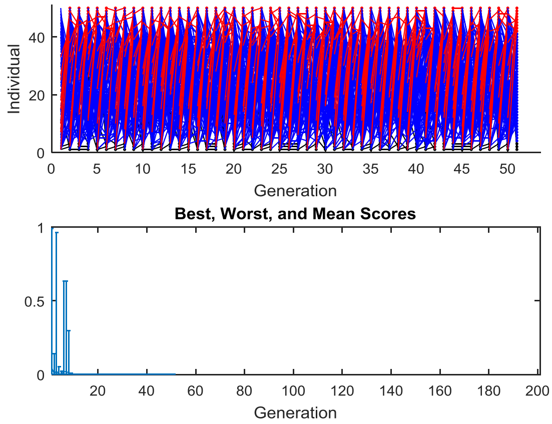

FIGURE 5: GENEALOGY & RANGE PLOT FUNCTION

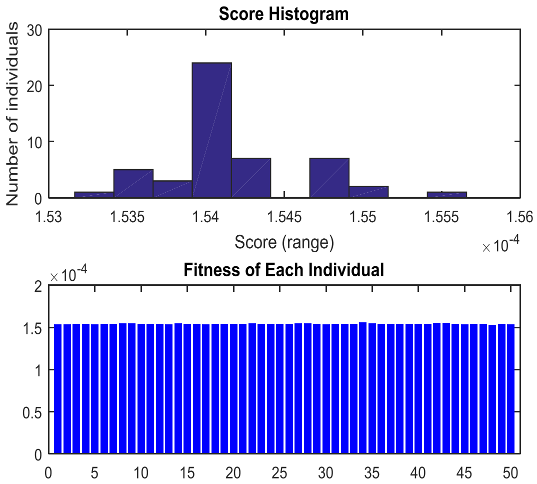

FIGURE 6: SCORE DIVERSITY AND SCORE PLOT FUNCTION

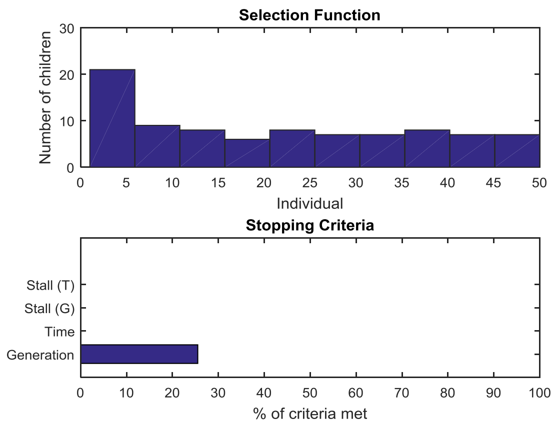

FIGURE 7: SELECTION AND STOPPING PLOT FUNCTION

6.0 CONCLUSION

With the proposed model for determining the desirability of a stream cipher, in this work we have demonstrated that it is possible to achieve the two security pillars that are authentication and integrity from gene formation to reproduction with the help of genetic algorithm and to identify the processes of evaluation and selection of chromosome to the best chromosomes.

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PRELIMINARY EVALUATION OF PAGO CLAY IN PRODUCTION OF INTERLOCKING BRICKS.

Ibe, P. C; Alhaji, M. M & Amadi, A. A
Civil Engineering Department, Federal University of Technology, Minna, Nigeria
*engchucksibe@gmail.com

ABSTRACT
The rising choice of clay bricks as an affordable and environmental friendly building material led to the evaluation of Pago clay that is mostly used in the production of bricks. Based on the clay sample that was collected during the rainy season which could increase the moisture rate, the natural moisture content is an average of 11.5%. Considering the particle size distribution, the percentage gravel is 2.83%, the percentage coarse sand is 6.33% while the percentage medium sand is 25.84%. The percentage fine sand is 14.96% and the percentage silt plus clay is 50.04%. The result also shows that the percentage sand is 47.13%. In determining the Atterberg limits, the Liquid limit is 40% and the Plastic limit is 31.9%. This gives the value of the Plasticity Index as 8.1%. Soil Classification according to their Plasticity indices shows that the soil is Medium Plastic (Between 7-17). Also Classifying the clay soil using the Unified Soil Classification System (ASTM D-2487), the soil is classified as "CL" which is Inorganic clays of low to medium plasticity.

Keywords: *clay, classification, interlocking brick, pago.*

1 INTRODUCTION

Housing is a basic human need and owning a house becomes a life long struggle as majority of the people see housing costs to be very expensive. The problem becomes even more worrisome when considering the low income families who constitute a greater percentage of the population.

Bricks are the major components for walling in buildings, and walling materials in any building contributes about 22% of the total building cost (Raheem *et al.,* 2012). There is need to reduce the cost of housing and make it affordable for the increasing population.

The common building material in Nigeria and other countries of the world are cement driven. It is common knowledge that, apart from the cost of cement due to energy required in its production, the carbon (IV) oxide (CO_2) emitted during production is a major hazard to the environment which must be reduced. In cement production, energy consumption is not only an environmental concern but also a major economic concern. Atmospheric emissions from the combustion of fuels (typically natural gas) for heat, reinforce the idea that emphasis should be placed on reducing energy consumption and also reducing the environmental impact (Turgut, 2012).

Clay bricks have been important components of building construction in Nigeria before the advent of sandcrete blocks and have been acknowledged as alternative to Sandcrete Blocks that are so expensive due to the high cost of cement. The use of clay bricks for buildings is on the rise because it is environmentally friendly and is expected to substitute the use of cement in the long run. A Common building brick is made of a mixture of clay that is subjected to several processes, differing according to the nature of the clay material, the method of manufacture and the character of the finished product. A good soil for brick making should have clay content of between 20-35%, liquid limit of 25-38%, Plastic Index of 7 to 16 % and volumetric shrinkage of 15 to 25% (Mueller *et al,* 2008, Kiptum et al, 2014).

Variations in the firing temperature and the firing time have important effects on quality of bricks. The bricks, when fired gradually to temperatures between 900 to 1100ºC (Karaman *et al,* 2012), show no cracks, but change their colour to brick red from the natural gray-brown hue. The red colour is indicative of the presence of iron (Tse, 2012). One of the most widely used was an open clamp, in which clay bricks were placed on a fire beneath a layer of dirt and used bricks. As the fire dies down after operating for several weeks, the clay bricks become fired. Such methods gradually became obsolete after 1865, when the Hoffmann kiln was invented in Germany. Very suitable for the production of large numbers of bricks, this kiln contained a series of compartments through which stacked bricks were transferred for pre-heating, burning, and cooling (Altayework, 2013). Traditionally, coal and Low Pure Fuel Oil (LPFO) are used as fuel for firing of clay bricks in the kilns (Ghauri *et al.*, 2009, Lakho and Zardari, 2016). The resulting clay bricks have a reduced final size of about 2mm on each side due to the loss of humidity during burning. Houses built with these bricks are reported to be cooler than those built with cement blocks since burnt bricks are poorer conductors of heat than cement bricks, giving rise to good insulation properties that contribute to greater thermal comfort. In addition to being fire resistant because of the high temperatures of firing, they also have higher aesthetics than unpainted cement blocks. The manufacturing of fired clay bricks is an art. It starts with the selection of the best clay, which has to be free of any organic compounds.

The Standards Organization of Nigeria stipulates that the compressive strength of bricks for building should be a minimum of 2.5N/mm^2 (NIS: 74, 1976). Also, the British Standards Institute (BS 368, 1976) specifies 2.75 N/mm^2 and 1.38 N/mm^2 respectively as minimum values of crushing strength for bricks to be used in 2-storey buildings and non-load bearing walls.

2 MATERIALS AND METHODS

2.1 SOIL SAMPLE COLLECTION

The clay soil is collected from the soil pit of Shelter Clay Products Ltd situated at Pago Town, opposite NNPC Depot along Minna-Suleja Road, Minna Niger State, using disturbed sampling method.

2.2 EXPERIMENTAL METHODS

The tests carried out during this research are the index properties of soil such as natural moisture content, sieve analysis, liquid limit and plastic limit. All these tests are done in accordance with BS: 1337 (1992).

3 RESULTS AND DISCUSSIONS

3.1 DETERMINATION OF NATURAL MOISTURE CONTENT

The moisture content *M C* of a soil mass is the ratio of the mass of water *W2 – W3* in the voids to the mass of solids *W3 –W1*. That is, $MC = \frac{W2-W3}{W3-W1}X100$

The clay soil sample was collected from the pit during the rainy season which could increase the moisture rate. Three trials were carried out during the test which is on dry basis and an average natural moisture content of 11.5% was determined.

3.2 PARTICLE SIZE DISTRIBUTION OF THE SOIL SAMPLE

The curve of the particle size distribution is shown in figure 1.

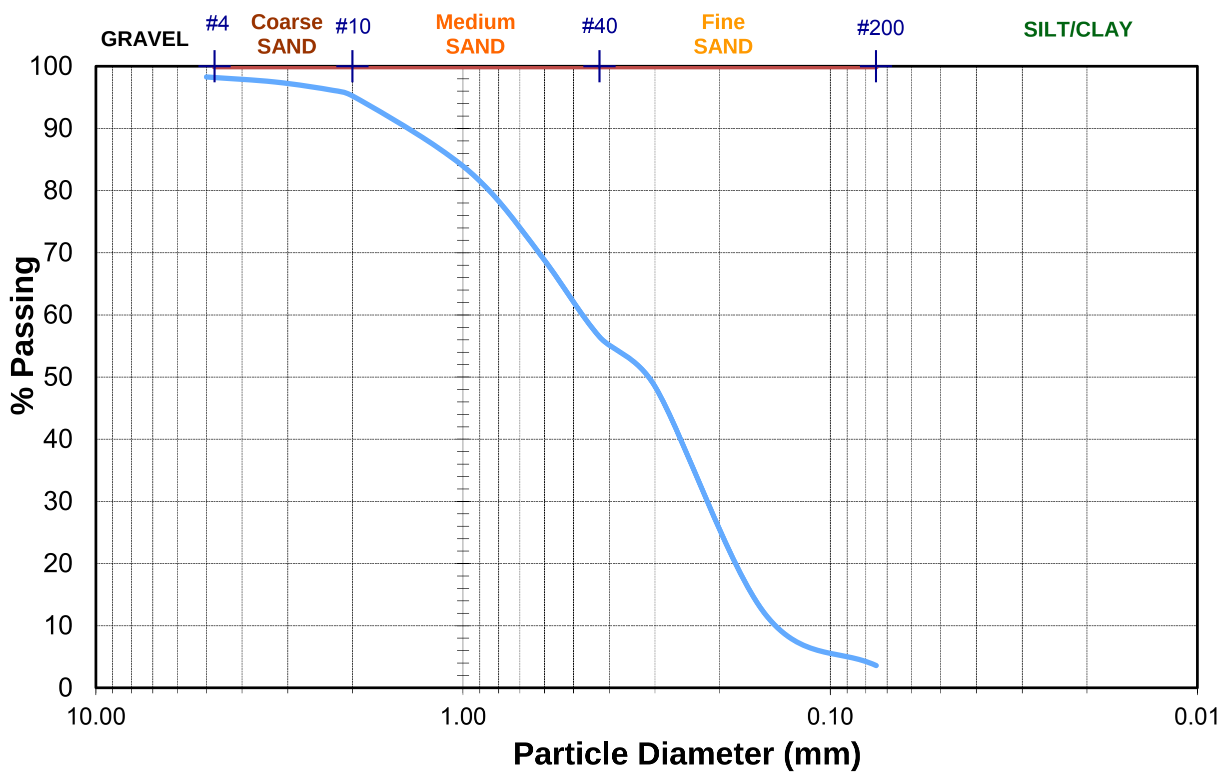


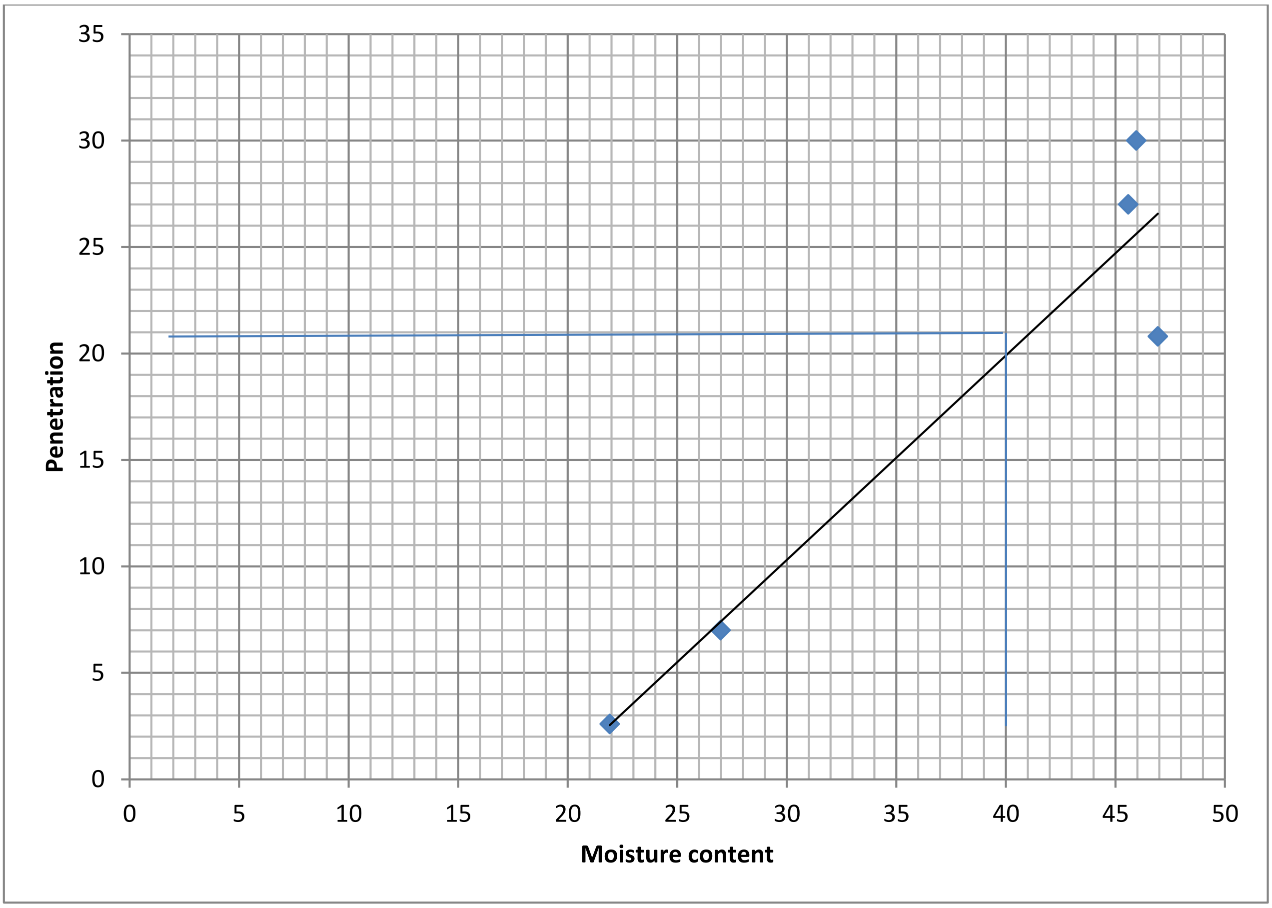
FIGURE 1 THE PARTICLE SIZE DISTRIBUTION CURVE.

From the particle size distribution curve
Percentage gravel = 100 – 97.17 = 2.83%
Percentage coarse sand = 97.17 – 90.84 = 6.33%
Percentage medium sand = 90.84 – 65.0 = 25.84%
Percentage fine sand = 65.0 – 50.04 = 14.96%
Percentage silt = 50.04%
Percentage sand = 6.33 + 25.84 + 14.96 = 47.13%

3.3 ATTERBERG LIMITS OF CLAY SOIL SAMPLE

The Static Cone Penetrometer was used to determine the Liquid limit and the Plastic limit.
The plotted graph for the determination of the Atterberg limits which has the penetration on the vertical and the moisture content on the horizontal is shown in figure 2.

Figure 2 Graph for the determination of the Atterberg limits.



From the Graph, the Liquid limit is 40% and the Plastic limit was determined from the average as 31.9%
Plasticity index$= 40 - 31.90 = 8.1\%$

3.4 SOIL CLASSIFICATION

Atterberg soil classification according to their plasticity indices shows that the soil is MEDIUM PLASTIC (Between 7 – 17). Also, classifying the soil using the Unified soil classification system (ASTM D-2487), the soil is classified as "CL" which is INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY.

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ASSESSMENT OF THE HYGROTHERMAL PROPERTIES OF MORTAR USING QUARRY DUST

Okafor A.[1]; *Yusuf A.[2]; & Abdullahi A.[3]
[1]Civil Engineering Department, Federal University of Technology, Minna, Nigeria
*yusuf.abdul@futminna.edu.ng

ABSTRACT
Assessment of the hygrothermal properties of mortar using quarry dust as fine aggregate was studied. The material used include Ordinary Portland Cement, water and quarry dust. Preliminary test such as particle size distribution, bulk density, moisture content and specific gravity were conducted on the aggregate in accordance to BS EN 1097:6. The mortar was prepared using a mix ratio of 1.4 and cured for 28 days. Water absorption, porosity and sorptivity test were the properties measured. Fineness modulus of 2.94, specific gravity of 2.42 and water absorption of 1.4% was recorded. The porosity and sorptivity were within the limit set by standard but the water absorption capacity was slightly higher that the limit specified. The study concluded that quarry dust can be used in the production of structural mortar.

Keywords: *Hygrothermal Mortar, porosity, quarry dust, sorptivity, water absorption.*

1 INTRODUCTION

Mortar is one of the important component of a building. It has been extensively utilized in masonry works, plastering and repair of damaged structural elements (Skoulikari, 2007). Mortar is defined as a mixture of cement with inactive materials of small granulometric gradation and with treatment liquid, which is usually water (Skoulikari, 2007). Mortar can also be defined as a workable paste used to bind construction blocks together and fill the gaps between them. The word comes from Latin mortarium, meaning crushed. Mortar may be used to bind masonry blocks of stone, brick and cinder blocks (Mehulkumar, 2015). This implies that mortar is made up of two major components; aggregate which offers resistance and cement paste which provides binding properties. Aggregates desirable for mortar production are usually sand with bigger diameter of grain size 4 mm. The attributes of mortar depend on the type of cement used, the type of aggregates and the mix proportion as well as the type of additives and the way of condensation of mortar.

Natural river sand has been conventionally used in the production of mortar for masonry works. The over exploitation of natural river sand has led to degradation of rivers leading to environmental defects such as bank erosion and destruction of aquatic habitat (Appukutty, 2009). The function of fine aggregate is to assist in enhancing workability and uniformity in a mortar mixture. River deposits are the most common source of fine aggregate in Nigeria today. Now-a-days the natural river sand has become scarce and very costly in some parts of Nigeria. Hence the need for alternative materials. Quarry dust has been used in place of river sand fully or partly in the production of concrete and mortar (Mahzuz *et al.*, 2011; Mayank *et. al.*, 2017; Chandana *et. al.*, 2013; Subramanian and Kannan, 2013; Vishal *et. al.*, 2017). This study is therefore aimed at assessing the hygrothermal properties of structural mortar made using quarry dust as aggregate.

The durability of mortar is a very fundamental factor that depends upon the transportation of water and how gases enters and move within it. Moisture transport characteristics has been employed in different construction practices such as the production of damp-proof basement or flat roofs, but the same transport characteristics may also adversely affect the thermal performance and durability of a structure. Moisture transport is always coupled with heat transfer, especially when vapour diffusion and drying processes are involved giving rise to the assessment of the hygrothermal property of mortar used in construction (Lawrence *et al.*, 2004). The basic hygrothermal properties or parameters are porosity, permeability, capillary action, absorption and sorptivity. Permeability is a property that measures the flow of water under pressure. It measures the ability of concrete to move water more concisely with both mechanisms that controls the absorption and transportation of liquid and gaseous substances in concrete (Pitroda and Umrigar 2013). Sorptivity is the ability of material to absorb and transmit water in it by capillary suction (Pitroda and Umrigar, 2013).

2 MATERIALS AND METHODS

2.1 MATERIALS

Materials used in this research and their functions are:

CEMENT: The most widely used cement that is readily available is the Ordinary Portland Cement (OPC). The type of OPC used in this work is the Dangote Portland cement of 42.5R grade. It is manufactured by Dangote

Cement Company Plc. and is in accordance with BS 12 (1996) and ASTM 150 (1994).

FINE AGGREGATE (QUARRY DUST)
Quarry dust was obtained from Usmani Quarry site in Abuja. The quarry dust was passed through sieve size 4.75mm. Organic substances were screened from the dust to obtain a fine grain.

WATER
Water is an important ingredient of mortar as it participates in the chemical reaction with cement. Potable, clean water free from deleterious substances was used as mixing water. The water was obtained from Civil Engineering laboratory and it conforms with BS 3148 (satisfies the required specification in the production of mortar according to BS 3148.

2.2 METHODS

2.2.1 PARTICLE SIZE DISTRIBUTION

The test was done according to BS 1097 (2000). The sieves were arranged in decreasing order of their size. Fine aggregate of air dried sand was introduced into the top sieve and shook vigorously so that finer materials less than 5mm passes through. The mass of each sieve plus its content was determine and the retained sample mass was determine.

2.2.3 SPECIFIC GRAVITY TEST

The test was carried out in order to obtain the specific gravity of the fine aggregate (river sand) according to BS 1097 (2000). The specific gravity was calculate using equation 3.1

$$Gs = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)} \qquad (3.1)$$

where

M_1 mass of empty flask
M_2 mass of flask and sample
M_3 mass of flask, sample and water
M_4 mass of flask and water

2.2.4 BULK DENSITY TEST

Bulk density test was carried out on the fine aggregate according to BS EN 1079 (2000). The British Standard recognises two degree of compaction which are the loose (uncompacted) and dense (compacted) degrees. The test was performed in a metal cylinder container of 1 litre capacity. The bulk density was estimated using equation 3.2

$$\text{Bulk density} = \frac{Weight\ of\ Material}{Volume\ of\ Cylinder} \qquad (3.2)$$

2.2.5 WATER ABSORPTION TEST

The test was carried out in order to measure the rate at which the fine aggregate absorbs water. The water absorption was calculated using the formula.

$$\text{Water Absorption} = \frac{M_3 - M_4}{M_2 - M_1} \times 100\% \qquad (3.3)$$

where M_1 mass of can
M_2 mass of can and sample
M_3 mass of can, sample and water
M_4 mass of can, sample and water (after 24hours)

2.2.6 PREPARATION OF SAMPLES

From the preliminary test that was carried out, selection of the right proportion of mortar constituents was done. For this research, mix design ratio of 1:4 was used. Mixing was carried out manually and a total of 30 50x50x50 mm mortar cubes were cast. The mould were oil smeared on the inside to avoid sticking, compaction was done manually using tamping rod to remove entrapped void and the mortar was left in the mould for 24 hours after casting before there were removed from the mould. The mortar cubes were prepared for porosity, water absorption and sorptivity test. The mortar cubes were cured in a water curing tank for 7, 14, 21, and 28 days respectively.

2.2.7 WATER ABSORPTION COEFFICIENT DUE TO CAPILLARY ACTION

The water absorption coefficient due to capillary action is measured using mortar cubes specimens under prescribed conditions at atmospheric pressure. Coefficient of water absorption due to capillary action was calculated from formula

$$water\ absorption = \frac{M2 - M1}{M1} x100\% \qquad 3.5$$

2.2.8 POROSITY TEST METHOD

This method covers the determination of the porosity of hardened mortar according to ASTM, C 20 (2000). The porosity was calculated using the formula given by:

$$P = \left[1 - \frac{\frac{WD - WS}{pW}}{VT} \right] \times 100 \qquad (3.6)$$

where
P = Porosity in %
W_D = Oven dried weight (g)

W_S = Submerged weight (g)
pW = Density of water (g/cm^3)
V_T = Volume of the cube

2.2.9 SORPTIVITY TEST METHOD

Sorptivity (S) is a material property which characterizes the tendency of a porous material to absorb and transmit water by capillary suction (Pitroda, & Umrigar 2013). The cumulative water absorption (per unit area of the inflow surface) increases as the square root of elapsed time (t). Sorptivity was calculated using equation 3.7.

$$S = \frac{I}{t^{1/2}} \quad (3.7)$$

where,
S = sorptivity
t = elapsed time (minutes)
I = Aw/Ad
Aw= W_2- W_1 (change in weight)
W_1 = Oven dried weight of the cube (g)
W_2 = Weight of the cubes after 30 minutes capillary suction of water (g)
A= Surface area of the cube
d= Density of water
t= Elapsed time

3 RESULTS AND DISCUSSION

3.1 PARTICLE SIZE DISTRIBUTION

Figure 3.1 shows the result of particle size distribution analysis. Total mass of dry sample used was 500g, but summing the masses of the retained sand we have 499.8. The reduction is due to losses mainly from small quantities of sand that gets stuck in the meshes of the sieves. The fineness modulus of fine aggregate was 2.94. It means that the average value of aggregate is between 2nd and 3th sieves. Thus the fineness modulus obtained suggested that the aggregate used had an approximate average size of 0.15mm to 0.30mm. Therefore, the quarry dust is fine in nature and adequate for use in mortar production (Vishal *et al.,* 2004).

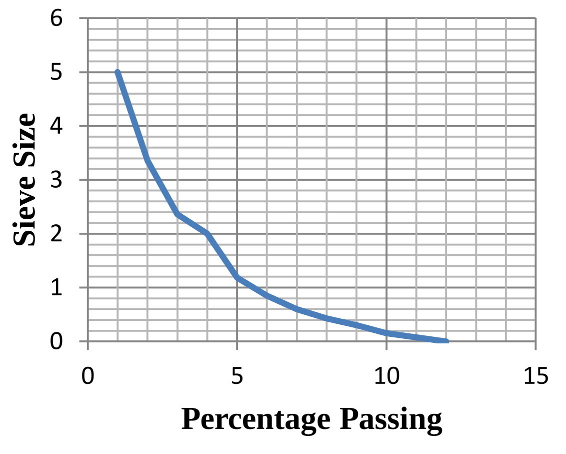

FIGURE 3.1: GRAPH OF PARTICLE SIZE DISTRIBUTION

3.2 SPECIFIC GRAVITY

Table 3.1 shows the result for the specific gravity of quarry dust. The specific gravity obtained fro the quarry dust was 2.42. This indicates that the aggregate used for the study was within the accepted specified values of 2.0 to 2.6 in accordance to BS-EN 1097 -6 (2000).

TABLE 3.1: SPECIFIC GRAVITY OF FINE AGGREGATE

Trial	1	2	3
Weight of Cylinder: M_1 (g)	116.7	116.7	116.7
Weight of Cylinder + Dry Sample: M_2 (g)	171.8	211.0	197.0
Weight of Cylinder + Dry Sample + Water: M_3(g)	344.3	342.4	351.7
Weight of Cylinder + Water: M_4 (g)	309.0	303.9	300.1
M_2-M_1 (g)	55.1	94.3	80.3
M_4-M_1 (g)	192.3	187.2	183.4
M_3-M_2 (g)	172.5	131.4	154.7
G_S	2.7828	1.6899	2.7979
Average G_S		2.42	

3.3 BULK DENSITY

Table 4.3 shows the results of compacted and uncompacted bulk density of quarry dust. From the result obtained it shows that the quarry dust has a high bulk density of 1471.70 kg/m^3 as compared to standard for the uncompacted bulk density and a low bulk density of 1671.25kg/m^3 as compared to standard for compacted bulk density.

3.4 WATER ABSORPTION

Table 3.2 shows the result of water absorption testWater absorption is the ability of a porous material to retain water. The water absorption for the sand used was 1.4%, which is below the specification limit of 2% according to BS 882. This explains that, much of the water used for mixing the mortar will be absorbed by the aggregate to keep it at the saturated surface-dry state and the rest of the rest for mixing the mortar and hydration of cement.

TABLE 3.2: WATER ABSORPTION OF FINE AGGREGATE

Trial	1	2	3
Weight of empty can: M_1 (g)	24.1	23.6	24.7
Weight of can + Dry sample: M_2 (g)	124.7	120.8	130.0
Weight of can + Sample + Water: M_3 (g)	194.7	195.7	195.7
Weight of can + Sample + Water(after 24 hours):M_4 (g)	192.6	193.7	193.8
Decrease in mass: M_3-M_4 (g)	1.01	2.00	1.19
Weight of initial dry sample: M_2-M_1 (g)	100.6	97.2	105.3
% Water Absorption	1.00	2.06	1.13
% Mean Water Absorption		1.4	

3.5 HYGROTHERMAL PROPERTIES TEST RESULTS

3.5.1 POROSITY OF MORTAR

The result of mortar porosity is shown in table 3.3. The result shows clearly that the mortar is highly porous in nature, thus this mortar cannot be subjected to water and any other adverse weather condition, because mortar with larger pores permits larger water absorption in saturated condition and larger evaporation of water in drying process accordingly. This shows consistent result with previous research work carried out on mortar, for every increase in the w/c ratio (additional water content) from 0.45 to 0.60, porosity goes up to 150% and compressive strength is reduced.

TABLE 3.3: POROSITY

Trails	Dry Weight W_D (g)	Submerged Weight W_S (g)	Bulk Porosity (%)
Specimen 1	245.6	257.7	100.00
Specimen 2	228.3	239.9	100.00
Specimen 3	258.3	274.6	100.00
Specimen 4	268.9	285.8	100.00
Average			100.00

3.5.2 WATER ABSORPTION

Table 3.4 shows the result of water absorption, the difference in the value of percentage water absorption is due to the variation in mass of each specimen. The result obtained indicates that the mortar has large void spaces having a finest modulus of which are interconnected that allow rapid ingress and flow of water through it. From the average result (9.78%) obtained it shows that the fine aggregate used is of lightweight (low density river sand) as low density relatively have high absorption capacity, thereby affecting the durability of the mortar. According to BS 8002, it is stated that the higher (>8%) the water absorption by mortar the less durable it becomes.

TABLE 3.4: WATER ABSORPTION

Trails	Dry Weight W1 (g)	Wet Weight W2 (g)	Water Absorption (%)
Specimen 1	226.3	247.8	9.50
Specimen 2	247.9	270.8	9.16
Specimen 3	265.9	283.4	6.58
Specimen 4	256.6	292.2	13.87
Average			9.78

3.5.3 SORPTIVITY

Table 4.7 shows the result of sorptivity of cement mortar cured for 28 days. For determining the sorptivity of the specimens, it was decided to base the observation on the first 30 minutes of elapse test time. For all specimen tested, this duration of time produces a linear relationship which ranges from 0.051 to 0.058. The average value obtained correlates with the minimum range of 0.0358 $g/cm^2/min^{1/2}$.

TABLE 3.5: SORPTIVITY

Trails	Dry Weight W1 (g)	Wet Weight W2 (g)	Sorptivity Value ($g/cm^2/min^{1/2}$)
Specimen 1	261.5	269.5	0.058
Specimen 2	213.6	220.6	0.051
Specimen 3	260.1	267.8	0.056
Specimen 4	269.6	277.1	0.055
Average			0.055

4 CONCLUSION

Hygrothermal properties of mortar produced using quarry dust was investigated. The properties of the quarry dust were determined and found to conform with specifications. The hygrothermal properties measured were within the specified limit recommended by several standards except for water absorption which was high that the specified limit. The study therefore concluded that quarry dust can be used to produce mortar for structural use

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Statistical Modeling of Compressive Strength of Ordinary Portland Cement concrete with Rice Husk Ash

*Mustapha, G. A.[1]; Aguwa, J. I.[2]; Bala, A.[3]
[1] Department of Civil Engineering, Federal University of Technology, Minna
[2]Department of Civil Engineering, Federal University of Technology, Minna
[3]Department of Civil Engineering, Federal University of Technology, Minna
*Corresponding author email: mgabdul1@yahoo.com

ABSTRACT

The study aimed at developing a model function to analyse the statistical data of the compressive strength of an ordinary Portland cement concrete with 15% replacement of cement with RHA. A total of twenty (20) concrete cubes were produced and cured for 28 days and the compressive strengths were plotted against cube weights. Regression analysis was then used to analyse the experimental values. R^2 values between 31-44% were recorded from the different models. It was observed that the polynomial function has the highest coefficient of determination to the experimental values. The linear function shows the least statistical values of 1.39 for standard deviation and 0.15 for coefficient of correlation. The mathematical models developed are in conformity with experimental values with minimal variability.

Keywords: *Aggregate, Cement, Compressive strength, Mean Strength, Rice husk.*

1 INTRODUCTION

Concrete is a mixture of water, cement, aggregate and admixtures, which are used to improve certain properties of concrete in its fresh or hardened state. In every concrete or composite construction, the strength of concrete is unavoidably an integral design factor. Though, durability due to environmental conditions, flexural and tensile strength ability are also important factors. BS 8110: Part 1specified strength requirement in terms of characteristic strength with a certain level of probability of the strength falling less than it. Typically, 5 per cent or 1 in every 20 chance of cube strength is expected to fall below the characteristic strength at 28 days of curing in water (Jackson and Dhir, 1996).

Concrete is an essential civil engineering material, its strength is dependent on various properties of its constituent materials, construction methods adopted, loading and environmental conditions to which it will be subjected to during its life time (Deepa et al, 2010). The production of concrete to meet certain requirements is not void of certain level of variability in the construction method as well as materials involved, which consequently affect the desired outcome in terms of strength variations for a concrete mix. Therefore, the need for quality control measures aim at limiting as much as possible the variability inherent in it. Statistical quality control methods provide a scientific approach to understand the variability of materials and processes with regards to specifications with proper tolerance to cater for unavoidable variations.

The use of artificial pozzolans as Supplementary Cementitious Materials (SCMs) in concrete production has necessitated the need to study the strength of pozzolanic concrete using statistical modeling to determine the effects of its constituent materials. The coefficient of determination R^2 is defined as the proportion of the total variation in Y "explained" by the regression of Y on X. it ranges from 0 when estimated regression model explains none of the variation in Y to 1 (when all points lie on the regression line) (Mahmoud, 2012).

Abdullahi et al. (2017) adopted a linear polynomial model in their study of modified water-cement ratio law for compressive strength of rice husk ash concrete to examine the age long water-cement ratio law of Ordinary Portland Cement (OPC) concrete to cater for concrete with rice husk ash. They conducted test on one hundred and fifty (150) concrete cubes focusing on the water-binder ratio at six (6) different replacement levels (5%, 10%, 15%, 20%, 25%, and 30%) of OPC with RHA. It was concluded from their studies that the model fitted adequately into the experimental data with an adjusted coefficient of determination of 73.0%.

Ettu et al (2016) conducted a research on the tensile strengths of concrete containing rice husk ash using different incineration methods. Three different percentage replacement level (5%, 10% and 15%) of OPC with RHA at curing ages of 28, 90 and 150 days were adopted for the model analysis. Regression analysis of OPC-RHA concrete confirms the 95%

adequacy of the model prediction of the split tensile strength.

Mahmoud (2012) in his study presented statistical modeling and prediction of compressive strength of concrete containing different matrix mixtures at fixed age or at different age of 1, 3, 7, 28, 56, 90 and 180 days. The parameters of the mixture examined were; time, water, cement, metakaolin, silica fume, aggregates and super plasticizer on the compressive strength of concrete. It was concluded that the predicted model has high correlation to the experimental results for the concrete compressive results.

Tanwani and Memon (2016) study the relationship between weight and concrete strength at 28 days of curing using trend line analysis. It was observed that the trend line fit by power function gives lowest deviation from mean strength with minimum error of 0.01% and maximum error of 10.33% with regards to other functions. Which best represent the relationship between weight and compressive strength observed.

Rachna et al. (2015) conducted a research work to propose the statistical model for predicting concrete strength using linear regression analysis. The regression model was developed for fly ash replacements at 0 and 15% and curing ages of 28, 56 and 91 days. Four variables were considered in predicting the strength, namely; water-binder ratio, fine aggregate-binder ratio, coarse aggregate-binder ratio and binder content.

Pozzolans such as rice husk ash (RHA) when blended with ordinary Portland cement behave as a cementitious material and give rise to hydrated calcium silicate (CSH) as a results of pozzolanic reaction of the silicate oxide (SiO_2) with the Calcium Oxide (CaO) liberated during hydration (Neville, 2011). Literature has shown that Rice husk ash contains more than 80% silicon oxide (Siddique, 2008) and other compounds which influence the desired output of a normal or light concrete. Pozzolanic blended cement has low heat of hydration compared to ordinary Portland cement (Mostafa and Brown, 2005). Rodrigues et al., (2006) observed that the incorporation of RHA in the composites could cause an extensive pore refinement in the matrix and in the interface layer, thereby decreasing water permeability. Furthermore, low heat development and resistance to sulfate attack are attributed to pozzolans as the hydration of cement results in great heat liberation, and the differential temperature between the initial setting time and hardening of cement causes shrinkage cracks.

This study is aimed at achieving a correlation between experimental data and mathematical models by analyzing the statistical values of the compressive strength relative to the weight of the cubes to determine some confidence level and degree of variability from targeted mean strength.

2 METHODOLOGY

The preliminary analysis of constituent materials; fine aggregate, coarse aggregate and Rice husk ash to determine some physical and chemical properties were conducted. Physical properties such as; specific gravity, bulk density, water absorption, particle size distribution of aggregates and rice husk ash were determined at the civil engineering laboratory of the Federal University of Technology Minna. Ten trials were conducted each for the specific gravity, bulk density, moisture content and water absorption tests and the mean values were recorded as shown in Table 1.Thus, the chemical analysis for the oxide composition for the Rice husk ash was conducted at the chemistry department laboratory of Ahmadu Bello University, Zaria.

A concrete mix design using the British "DOE" method was adopted and a mix ratio of 1:1.6:2.7 (Cement: Fine Aggregate: Coarse Aggregate) at a constant water-cement ratio of 0.5 with a target-mean strength of 25 $N/mm^2$was designed as shown in Table 1. The cement content batched was replaced with 15% Rice husk ash as a supplementary cementious material. ASTM C618 (2008) specified a maximum replacement of cement at 20% for a normal quality and economy concrete at 28 days of curing. A total number of 20 concrete cubes of 150 mm x 150 mm x150 mm dimension were prepared and the compressive strength values at 28 days of curing in water in accordance with BS 1881:part 116(1983). The concrete cubes were weighed after 28 days of curing and the corresponding compressive strength were computed. The experimental compressive strength and its corresponding mass were used to plot a scattered graph in Microsoft excel and analysed using trend line functions and the corresponding coefficient of determination values derived. The statistical data of the experimental compressive strength were then compared with the predicted mathematical models to check the level of conformity which are presented in Table 8 and Figure 4.

2.1 MATERIALS

Cement: Commercially available Ordinary Portland cement was used for this purpose.

Rice husk ash: Rice husk was locally sourced from available milling plant and incinerated under controlled temperature of 500-600oC.

Aggregate: Crushed granite and river bed sand were used as coarse aggregate and fine aggregate respectively.

Water: Potable water was used for the experiment.

TABLE 1: MATERIALS PER CUBIC METER OF CONCRETE

Material	Proportion	Weight (Kg)
Cement	1	420.00
Fine Aggregate	1.6	654.90
Coarse Aggregate	2.7	1115.10
W/C ratio	0.50	210.00

TABLE 2: PHYSICAL PROPERTIES OF THE CONSTITUENT MATERIALS

	Properties	Cement	Fine Aggregate	Coarse Aggregate	Rice Husk Ash
1	Specific Gravity	3.15	2.63	2.7	1.94
2	Bulk density: Compacted Loose	- -	1892.86 kg/m^3 1769.83 kg/m^3	1814.05 kg/m^3 1589.13 kg/m^3	476.19 kg/m^3 397.0 kg/m^3
3	Moisture content	-	0.14%	0.16%	-
4	Water absorption	-	23.67%	1.6%	-

3 RESULTS AND DISCUSSION

3.1 Physical Properties of the Constituent Materials

Table 2 shows the physical properties of the constituent materials. The mean-specific gravity of the fine and coarse aggregate was 2.63 and 2.70, the mean value for the crushed granite aggregate used falls within the range of 2.6 and 3.0 reported by Neville (2011). The mean specific gravity for RHA obtained was 1.94, a value less than what was obtained by Oyetola and Abdullahi (2004), but within the range of 1.9 and 2.4 specified for pulverized fuel ash (PFA) as reported by Neville (2011). Bui et al., (2005) and De Sensale (2006) reported that the low specific gravity of RHA relative to cement specific gravity of 3.15 will results in concrete of a less density as percentage replacement increases with respect to the cement.

The mean compacted and loose bulk densities derived for the coarse aggregate are 1814.05 kg/m^3 and 1589.13 kg/m^3 respectively, while that of fine aggregate were 1769.83 kg/m^3 and 1892.86 kg/m^3 as shown in Table 2. The ratio of the loose bulk density to the compacted bulk density for the coarse and fine aggregate are 0.88 and 0.93 respectively, which lies within the range of 0.87 and 0.96 stated in Neville, (2011). The bulk density is directly related to how densely the aggregate is packed, it measures the volume the aggregate will occupy in concrete which is a factor of particle size and shapes. For a coarse aggregate of given specific gravity, a higher bulk density means that there are fewer voids to be filled by fine aggregate and cement paste (Neville, 2011).

3.2 PARTICLE SIZE DISTRIBUTION

The particle size gradation of the fine and coarse aggregates are as shown in Table 3 and 4 and Figure 1 and 2 respectively. From the test result, the cumulative percentage of fine aggregate passing 600 μm sieves is not less than 84%. This according to BS 882 (1992) satisfies the requirement for fine grading, and will influence the workability of concrete mix.

The coarse aggregate has particles completely passing sieve 50 mm and 37.5 mm meeting the requirement of BS 882 (1992), and mostly retained on sieve sizes 20 mm to 10 mm with less fine particles. Neville (2011) stated that when crushed rock coarse aggregate is used in concrete production, a slightly higher proportion of fine aggregate is required than with gravel aggregate in order to compensate for the lowering of workability by the sharp angular shape of the crushed particles.

TABLE 3: SIEVE ANALYSIS OF FINE AGGREGATE

Sieve Size (mm)	Weight Retained (g)	Percentage Retained	Percentage Passing	cumulative % passing
5.00	22.40	4.5	95.5	95.5
3.35	10.20	2.0	98.0	93.5
2.36	22.70	4.5	95.5	88.9
2.00	15.20	3.0	97.0	85.9
1.18	79.70	15.9	84.1	70.0
0.85	57.90	11.6	88.4	58.4
0.60	76.70	15.3	84.7	43.0
0.43	83.30	16.7	83.3	26.4
0.30	32.40	6.5	93.5	19.9
0.15	87.60	17.5	82.5	2.4
0.08	9.90	2.0	98.0	0.4
Pan	1.90	0.4	99.6	0.0

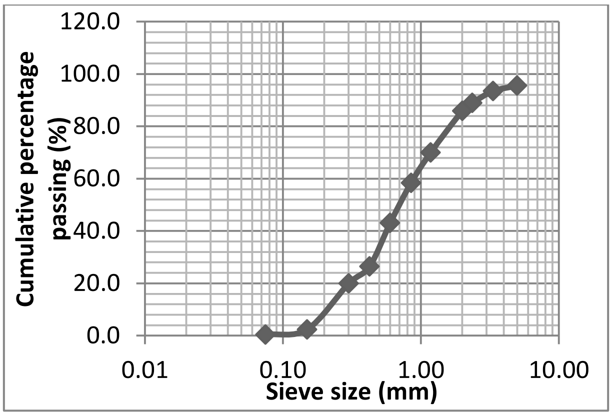


Figure 1: Particle size distribution of fine aggregates

TABLE 4: SIEVE ANALYSIS OF COARSE AGGREGATE

Sieve Size (mm)	Weight Retained (g)	Percentage Retained	Percentage Passing	Cumulative % Passing
50	0.00	0.00	100.00	100.00
37.5	0.00	0.00	100.00	100.00
20	1194.20	59.74	40.26	40.26
14	643.40	32.18	67.82	8.08
10	144.00	7.20	92.80	0.88
6.3	15.40	0.77	99.23	0.11
5	0.80	0.04	99.96	0.07
Pan	1.30	0.07	99.93	0.00

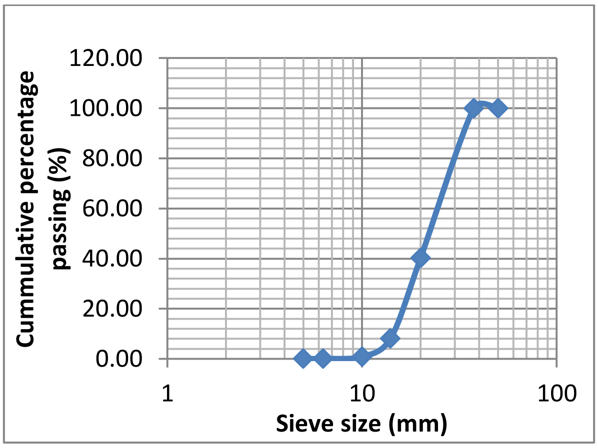


Figure 2: Particle size distribution of coarse aggregate

3.3 CHEMICAL COMPOSITION OF RHA

The oxide composition of the rice husk ash used is shown in Table 5. It can be observed that the rice husk ash has high silica content above 88% which is a measure of its reactivity. Silica is the compound responsible for strength development in concrete (Nair *et al.*, 2008). Also, the total percentage composition of Aluminum Oxide (Al_2O_3), Silicon Oxide (SiO_2) and Iron Oxide (Fe_2O_3) was found to be 91.92%, it exceed 70% minimum value for Class F fly ash (ASTM C618, 2005). This is an indication of a high pozzolanic reactivity of the rice husk ash sample and a suitable pozzolans for cement partial replacement in concrete.

TABLE 5: CHEMICAL COMPOSITION OF RHA

Elemental Oxide	Percentage Composition
Na_2O	0.0%
MgO	0.67%
Al_2O_3	2.47%
SiO_2	88.77%
P_2O_5	3.65%
SO_3	1.28%
K_2O	0.88%
CaO	1.05%
TiO_2	0.33%
Cr_2O_3	0.00%
Mn_2O_3	0.13%
Fe_2O_3	0.68%
ZnO	0.04%
SrO	0.01%

3.4 DENSITY OF HARDENED CONCRETE

The density range of the concrete samples as shown in Table 6 was 2491.85 kg/m^3 and 2702.22 kg/m^3; this is above the ranges of RHA concrete from the findings of Adenuga et al. (2010) and Yuzer et al. (2013) for normal weight concrete, that is, 2200 and 2550 kg/m3 as per ACI Committee 213 (2003). The experimental values of the densities indicate that concrete produced with 15% RHA cement replacement can be regarded as a normal weight concrete.

3.5 COMPRESSIVE STRENGTH

The experimental compressive strength values of the cubes specimens at 28 days of curing are shown in Table 6. The minimum compressive strength was 23.11 $N/mm^2$and the maximum of 30.89 N/mm^2. The mean value of the compressive strength was 26.76 N/mm^2, the standard deviation derived was 2.48 and the coefficient of variation was 9.27%. The inbuilt function of trend line analysis in Microsoft excel package has options with linear, polynomial, logarithm, exponential and power functions. The experimental values of the compressive strength and the corresponding weight were plotted on scattered graph in Figure 3(a-e) and fitted with trend line to develop mathematical models of the various trend line functions and R^2 as shown in Table 7. The R^2 value is an important parameter in regression analysis as it gives the percentage variation of fitted data. From the predicted models, the polynomial function was found to have the highest value of R^2 of 43.93%. The equations in Table 7 represent the relationship between the compressive strength (y) and the mass (x) of the concrete cubes with regards to the model functions. The model functions were used to re-evaluate the compressive strength as a function of weight of concrete cube and the statistical data of the experimental compressive strength compared with the predicted models is given in Table 8.

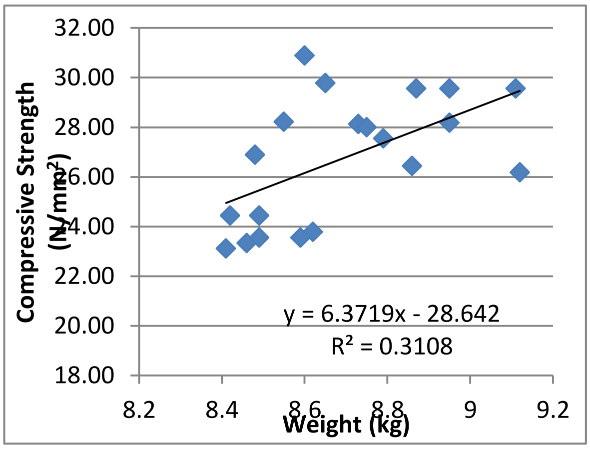
From Table 8, the linear function has the minimum standard deviation of 1.39 compared to other functions; it gives lowest deviation from the mean strength and a least coefficient of correlation of 0.15. Therefore, the linear function can best represent the relationship between weight and compressive strength of concrete cubes.

TABLE 6: CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS CURING AGE

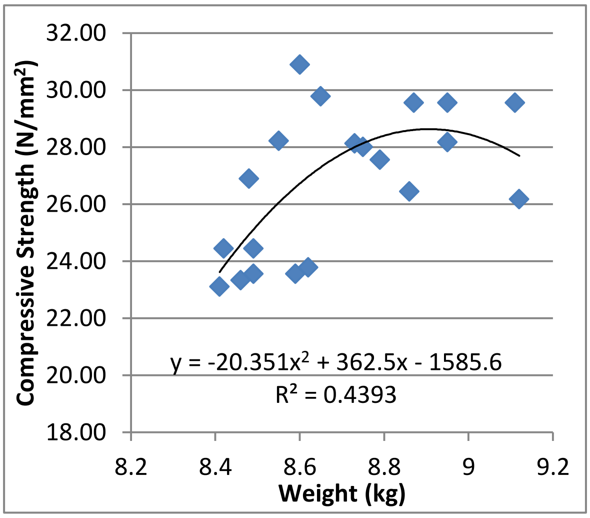
Sample	Mass of Cube (Kg)	Crushing Load (KN)	Density of Cube (Kg/m^2)	Compressive Strength (N/mm^2)
1	8.6	695	2548.15	30.89
2	8.87	465	2628.15	20.67
3	8.62	535	2554.07	23.78
4	9.11	465	2699.26	20.67
5	9.12	589	2702.22	26.18
6	8.49	550	2515.56	24.44
7	8.95	434	2651.85	19.29
8	8.73	433	2586.67	19.24
9	8.75	430	2592.59	19.11
10	8.46	525	2506.67	23.33
11	8.55	635	2533.33	28.22
12	8.48	605	2512.59	26.89
13	8.79	620	2604.44	27.56
14	8.49	530	2515.56	23.56
15	8.59	530	2545.19	23.56
16	8.41	520	2491.85	23.11
17	8.42	550	2494.81	24.44
18	8.95	465	2651.85	20.67
19	8.86	495	2625.19	22.00
20	8.65	670	2562.96	29.78

TABLE 7: TREND LINE MATHEMATICAL MODELS FOR COMPRESSIVE STRENGTH

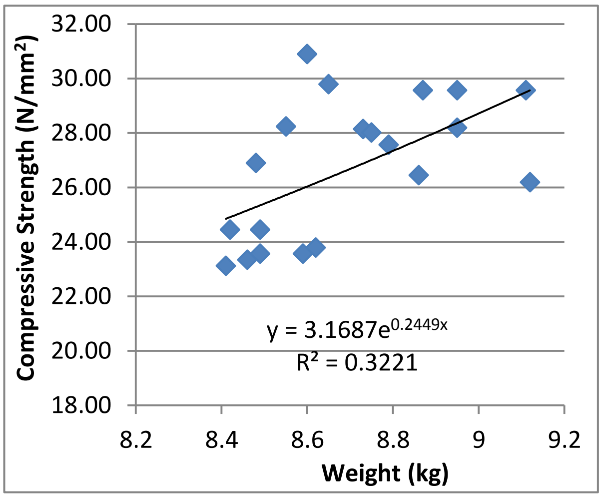
Function Type	Mathematical Model	R^2
Linear	$y=6.3719x-28.642$	0.3108
Polynomial D-2	$y=-20.351x^2+362.5x-1585.6$	0.4393
Exponential	$y=3.1687e^{0.2449x}$	0.3221
Logarithm	$y=56.134\ln(x)-94.625$	0.3153
Power	$y=0.251x^{2.1571}$	0.3268



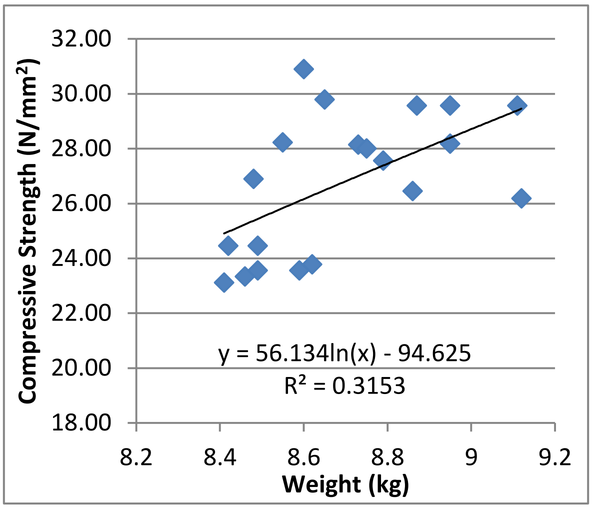
(a): Linear Function



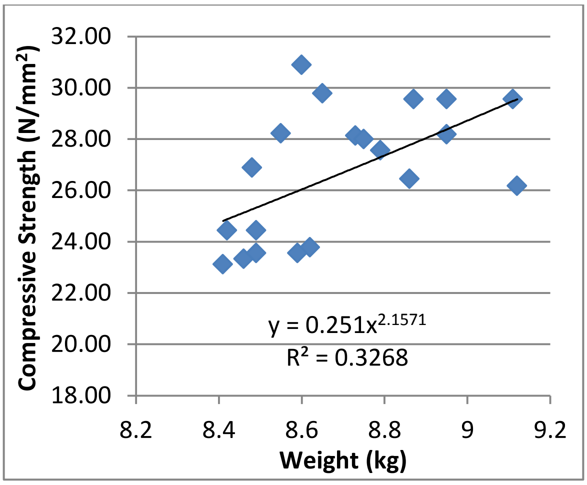
(b): Polynomial Function



(c): Exponential Function



(d): Logarithm Function



(e): Power Function

Figure 3(a-e): Experimental Trend line Functions

TABLE 8: STATISTICAL PARAMETERS OF EXPERIMENTAL AND PREDICTED MODELS OF COMPRESSIVE STRENGTH

Parameter	**Experiment**	**Linear**	**Polynomial**	**Exponential**	**Logarithm**	**Power**
Minimum	23.11	24.95	23.64	24.85	24.91	24.81
Maximum	30.89	29.47	28.62	29.56	29.46	29.54
Mean	26.76	26.76	27.77	26.68	26.76	26.67
Standard Deviation	2.48	1.39	1.65	1.44	1.40	1.45
Correlation Coefficient		0.15	0.18	0.15	0.15	0.16

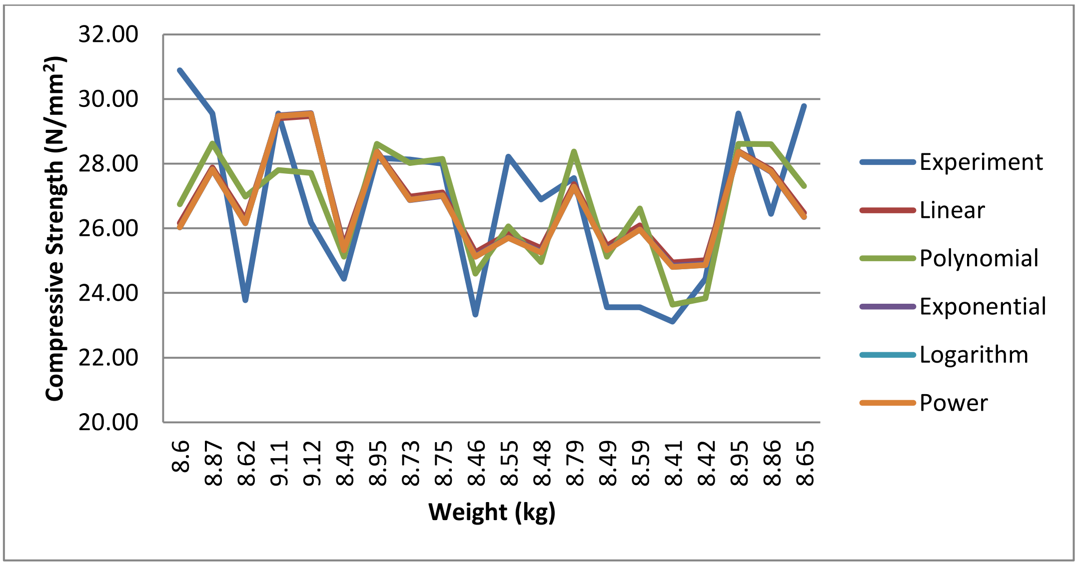


Figure 4: Compressive strength/weight experimental and model functions

4 CONCLUSION

It can be deduced from the outcome of the study that a good quality control was ensured as the derived mean strength of 26.76 $\boldsymbol{N/mm^2}$ was higher than the targeted mean strength of 25 $\boldsymbol{N/mm^2}$ used for the design mix. It can also be implied that, the properties of the constituent materials in the concrete mix affected positively the desired outcome of the compressive strength, and a mix ratio of 1: 1.6: 2.7 with a 15% replacement of OPC with RHA will yield an adequate strength in terms of normal weight concrete if proper quality assurance is observed.

The predicted mathematical models show conformity with the experimental values of the concrete strength. Therefore, it leads to the conclusion that mathematical functions can adequately represent compressive strengths given the weight as variables.

More so, sustainable environment can be guarantee if industrial and agricultural waste can be transformed into economic benefit and by so doing reducing environmental challenges associated with the built environment.

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ROAD PAVEMENT SETTLEMENT DETERMINATION

Ekuje E. O[1]*Kolo S. S.[2],., Ladan M[3]., Aba J.[4], Elogie P.[5], and Alhaji U.[6],
[1,2,3,4,5,6] Civil Engineering Department, Federal University of Technology, Minna, Nigeria.
*s.kolo@futminna.edu.ng, bukysayo123@yahoo.com

ABSTRACT
Settlement is a type of pavement defect that occurs majorly due to induced stresses from vehicular wheel loads repetitions on the pavement. Though pavement settlement after construction cannot be avoided it is a continuous process but the degree at which it settles, that take the center stage. In this work the factors that can cause settlement where spelt out and analysed, the response of the pavement to these factors was observed using the dumpy level to take the differential settlement of the pavement on periodic basis of two weeks, a settlement of 5mm, 9mm and 13mm respectively was observed for the duration of the study, from the observed result it can be said that the pavement was subjected to mild stresses because there are less trucks and buses using the road way. The small settlement noticed shows that the pavement design satisfies the traffic load and environmental condition.

Keywords-: Settlement, traffic load, wheel load, pavement.

1 INTRODUCTION

Road transportation is one of the most common modes of land transportation. Roads in the form of tracks, human pathways etc. were in use long before now, but many improvements has been made to this mode of transportation make it safe and comfortable. Thus road construction became an inseparable part of civilization and growth of empires. (Carpenter, 1993).

A road is therefore said to be paved or unpaved track or path on land to ease transportation of people and materials between two points. It can also be defined as a route, thoroughfare or way on land between two places, which typically has been paved or improved to ease transportation (Hart-Davis and Adam, 2001).

The paved surfaces of the road are commonly constructed of asphaltic materials. It consists of mineral aggregates bound together with bitumen laid in compacted layers, which has been in use since the eighteenth century facilitated by the advancement in road construction technology call pavement (Abaza, and Abu-Eisheh, 2003).

Pavement types include flexible, rigid and composite pavement. The flexible pavements also known as asphaltic pavement has its surface layer made of asphalt which is laid on underlying strata, made up of the sub-grade, sub-base and base layers. Contrarily; the rigid pavement are made of concrete while the composite pavements have their bases stabilized with asphaltic materials and the top layer bonded with cement or pozzolanic materials (O'Flaherty, 2010).

Asphalt paving mixes are usually prepared at an asphalt mixing plant. There are three types of asphalt mixes: Hot-Mix, Warm-Mix and Cold-Mix (www.wikipedia.com). Hot-Mix asphalt (HMA or HMAC) is more commonly used especially in developing countries that has little or no access to cold asphalt technology, while the use of Cold-Mix asphalt has replaced the use of Warm-Mix asphalt (WMA) and Hot Mix Asphalt (HMA) in developed countries. Cold-Mix asphalt (generally made with emulsified or cut back bitumen) is used for light to medium traffic secondary roads, or for remote locations or maintenance uses. Hot mix asphalt (HMA) pavement as it is more commonly called, refers to the top layers of a flexible pavement structure. For most applications, asphaltic concrete is placed as HMA, which is a mixture of coarse and fine aggregate, and bitumen binder.

The term Hot-Mix is derived from the process of mixing the aggregate and bitumen under elevated temperatures of about 150 and 166 degree centigrade for virgin and polymer modified asphalt respectively to obtain sufficient fluidity and remove water from the aggregates for proper mixing and easy laying.

The determination of the settling rate of cold asphalt is one of the methods used to determine the suitability of cold mix asphalts for pavement work. This can be done either by subjecting the asphalt to traffic and atmospheric conditions or by laboratory analysis. (Kolo, 2014)

Asphaltic settlement generally occurs due to traffic loading and environmental changes resulting in the volumetric change of the asphaltic thickness, soil mass reduction. Settlement of asphaltic pavement is the differential variation of the pavement thickness due to the load or usage of the pavement, which occurs when the asphalt pavement is subjected to variations in traffic loading and atmospheric conditions such as rainfall, temperature etc.

Pavement settlement is a form of road defect amidst others like rutting, corrugation, raveling etc, it is said to be gradual downward movement (sinking) of the asphalt after construction in relation to its initial level on the road surface when it was newly constructed (Kings, 2012). They result from the surface stresses that the pavement is subjected to.

Pavement stresses are observed on structural response model, which is said to be the model that defines the response of the asphaltic pavement to loadings in terms of stresses, deflections and strain, how pavement respond to applied stresses determine their structural behavior (Kolo, 2017). The surface layer of the entire pavement is looked upon for its responses to load bearing

The surface layer of the newly constructed Bahago road situated in Minna; (lat. 9.61524 and longitude 6.54776) with an estimated population of 321,687, which is a dual carriage road with sidewalks linking Bahago round-about. The construction project cost is about ₦1.4bn is a 3.4km road intended to reduce the cost and time of motorists between Minna and Maitunbi (Alhaji Muazu Bawa, 2014) former Niger state commissioner of works.

2. METHODOLOGY

The method adopted for this research work is in to form (a) traffic count (b) settlement determination and (c) Temperature Determination.

(a) Traffic count was conducted on the study area to estimate the load on the pavement per day the manual method of count was adopted, from 6am – 6pm daily for a period of one week on both carriages. This gives the estimated volume of traffic weekly and the vehicle classifications passing the roadway. The vehicle classification considered were passenger cars, trucks and buses.

(b) Settlement determination: In determining the rate of settlement with traffic load the dumpy level was used, the road was marked from chainage 0+00 to 0+500 at 25m intervals (total of 20 points) with measurement taken with the odometer wheel tape and the points being marked with paint for proper identification and accuracy.

The steps taken in determining the settlement rate of the pavement are outlined below.

i. The distance between each chainages were measured as 25m and marked
ii. The Temporary bench mark was established
iii. The leveling instrument was set up at the mid-point of the entire distance for easy coverage.
iv. The staff was placed at the temporary benchmark and the level at that point was recorded as back sight
v. The staff was moved to the first point 'chainage 0+00' the reading at this point was recorded as intermediate sight.
vi. The staff reading for the next point was also taken as the backsight.and so on to the last reading termed foresight

The levelling process was repeated at interval period of about 2 weeks at the established points; this was done to determine the different in level of the road with respect to traffic and time.

(c) Temperature determination: the daily temperature of the environment was obtained on daily basis, this accounts for the air or Environmental temperature, this is done wth the aid of the Microsoft weather application software. (weather version 4.3.193.0), the highest temperature for the day is considered as the air temperature for the area, with asphaltic pavement absorbing more of the air temperature and cooling off very slowly the environmental (air) temperatures obtained were added to 24^0C, since asphaltic pavement absorbs the environmental temperature to an estimated temperature of 24^0C (Meizhu Chen et al, 2014)

3. RESULT AND DISCUSION

The summary of traffic census conducted on the selected road is display in Figure 1, it was discovered that in a week 2,038 passenger car, 149 Buses while truck is 160 moves on the road which then translate to 97,824 veh/year passenger car, 7,152 veh/year Buses and 7,680 veh/year truck per year. According to Kolo et al (2015) Equivalent factor (E.F) for car is 0.00033, bus is 0.00864 while truck is 111.68. Resulting to 857,796.47 kN loading on the experimental road per year resulting in 13mm initial settlement for one week which is in conformity with FHA (1979) stipulation of an initial settlement between 6mm to 12.5mm as can be seen in Figure 3.

Figure 2, show the settlement with time on experimental road, the height of the pavement is seen to have settled a little from the initial height of the pavement in week one. With the initial height represented as blue and the subsequent heights represented as pink and grey, the pink line shows the week one settlement with an averages settlement of 5mm along the pavement, while the grey line shows the week two settlement which averages 9mm along the pavement length. The road can be said to have settle for about 13mm in week three.

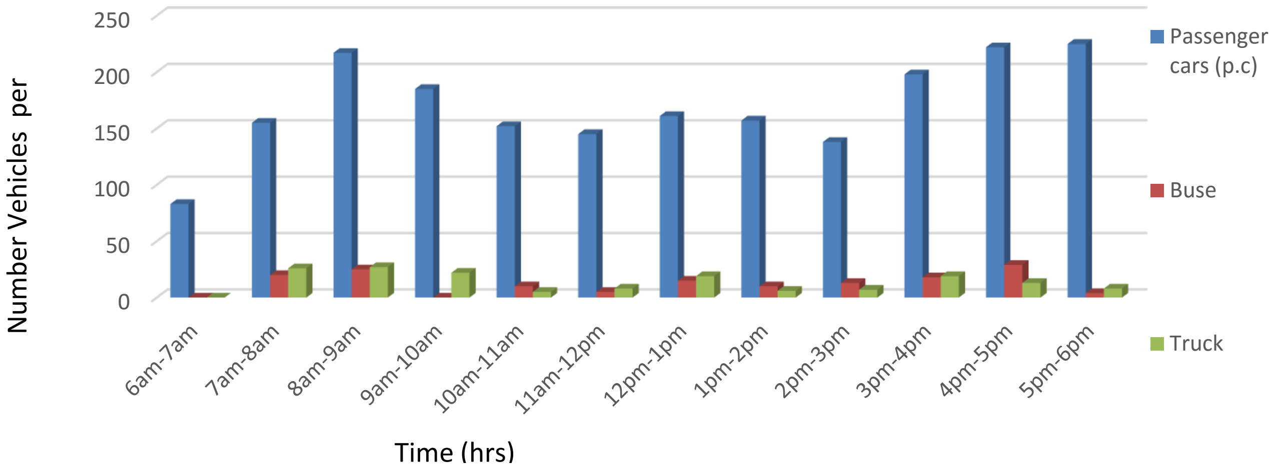

Figure 1: Proportion of passenger cars, buses and trucks on experimental road

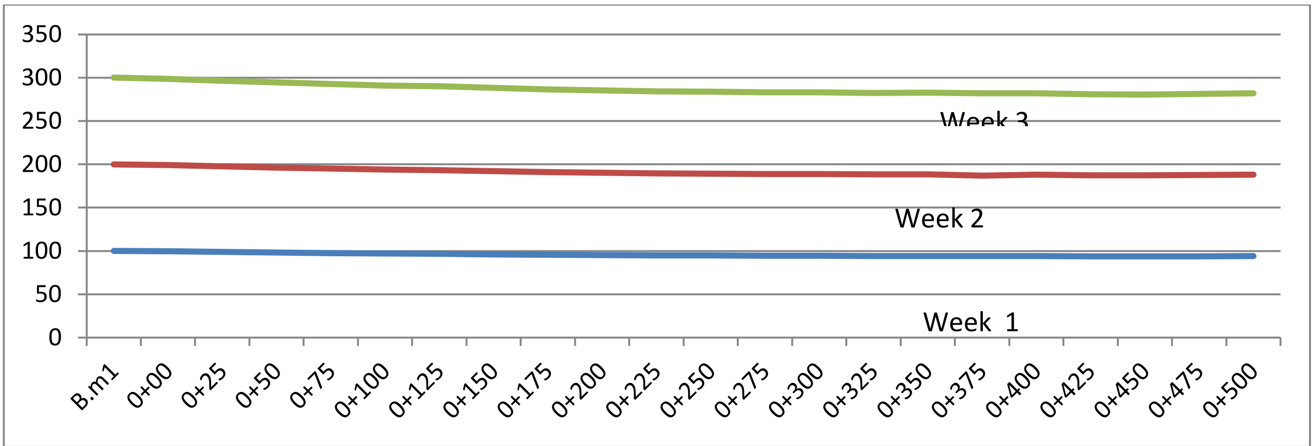

Figure 2: Pavement settlement

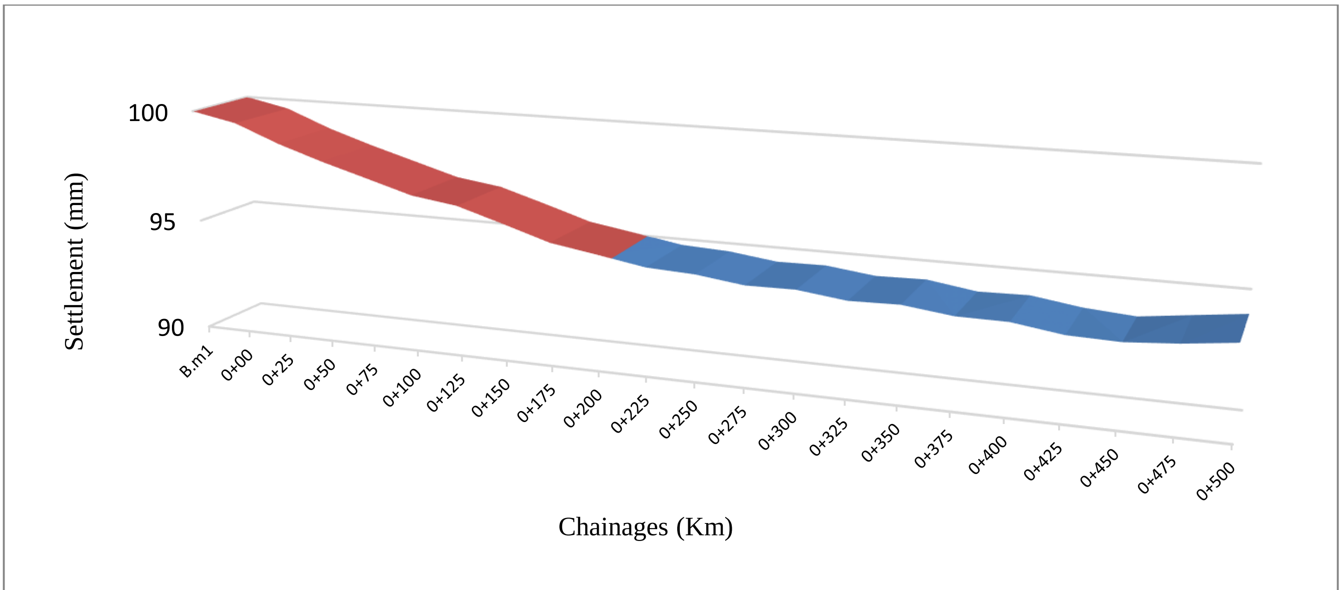

Figure 3: Pavement settlement

4. CONCLUSION

It can be seen that asphaltic pavement has the ability for settlement. The pavement was seen to have a little settlement that averages 5mm, 9mm and 13mm respectively for the first 3weeks; this value is negligent or not obvious to cause the pavement to fail with the predominant traffic load.

The imposed traffic load was seen to be mild, as they are less trucks and buses passing the route.

5. Recommendation

Based on the study conducted the pavement design; with respect to its thickness, satisfies the environmental condition and the traffic load. So the pavement thickness is okay to the traffic loading on the road.

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Properties and Microstructure of Concrete Containing Iron ore tailings

*Oritola, S. F.[1]; Saleh, A. L.[2]; & Mohd Sam, A. R[2]
[1] Department of Civil Engineering Federal University of Technology, Minna
[2]Deparment, of Structures and Materials, Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Johor Bahru, Malaysia.
*Corresponding Author email: sfaoritola@futminna.edu.ng

ABSTRACT
Successful utilization of waste materials such as agro, urban and industrial waste in concrete depends on its use being economically competitive with the alternate natural material. In this study, Iron ore tailings (IOTs), an industrial waste product, generated during the production of iron ore, is used in its natural state as fine aggregate to partially replace sand, for the production of normal strength concrete. The use of this waste material brings about conservation of declining natural resources, the utilization of valuable land for more profitable use and economic advantage in comparison to the conventional material. The physical properties of the Iron ore tailings were determined and compared with that of natural sand. Fresh and hardened properties of concrete were evaluated. Field emission scanning electron microscopy (FESEM) images of fine aggregate materials and the hardened concrete produced, were also studied. The microstructure of IOTs concrete samples shows a tighter interface between the cement gel and the aggregate when compared with those of the control concrete sample having no IOTs. The outcomes of mechanical properties tests and the microstructure analysis reveals that the IOTs was able to improve the strength and denseness of concrete. Based on findings from this research, it can be concluded that IOTs can be used to partially replace sand as fine aggregate in concrete, in order to improve the mechanical properties and the pore structure of the concrete.

Keywords: *Concrete properties, Industrial waste, Iron ore tailings, Normal strength concrete, Pore structure*

1 INTRODUCTION

By visual examination, the Iron ore tailings resembles natural sand and X-ray fluorescence reveals that for most Iron ore tailings, the major component is silicon dioxide (SiO_2) (Oritola et al., 2015) as shown in Figure. This suggests the need for research into how best the Iron ore tailings can be utilized in concrete. By physical examination, the fineness and angular nature of the Iron ore tailings also suggest that it can improve the denseness of concrete. In severe climates, the surfaces of concrete sidewalks, parking decks, bridges, canals, dams and other structures deteriorate progressively due to different kinds of causes (Jahangir et al., 2014). Due to the fineness of Iron ore tailings, it can also be considered as a promising material for the repair and maintenance of these concrete structures.

Previous research revealed that Iron ore tailings was utilized as fine aggregate to produce ultra-high performance concrete (Zhao et al., 2014). Iron ore tailings from different origins do not have the same geotechnical behaviour. The tailings may even show similar grading, but the parameters cannot be generalized for mines in terms of mineralogy or beneficiation process (Oritola et al., 2015). The IOTs was also used to produce green engineered cementitious composites (GECC) (Haung et al., 2013) and it was mentioned that mortar's compressive strength was improved due to tighter interface between aggregate and hard cement paste and the enhanced structure related to the finer nature of the Iron ore tailings compared to the reference sand (Yu et al., 2012). Toxicity leaching procedure tests for concrete sample also revealed that the hydraulic binder arrests metal mobility from the iron ore mine wastes (Yellishetty et al., 2008).

The sustainability of the construction industry would be enhanced if the utilization of IOTs in concrete is fully established based on outcomes of research. Plant covers can be saved thereby promoting the greenness of civil infrastructure. This industrial waste can therefore be turned into a major economic gain.

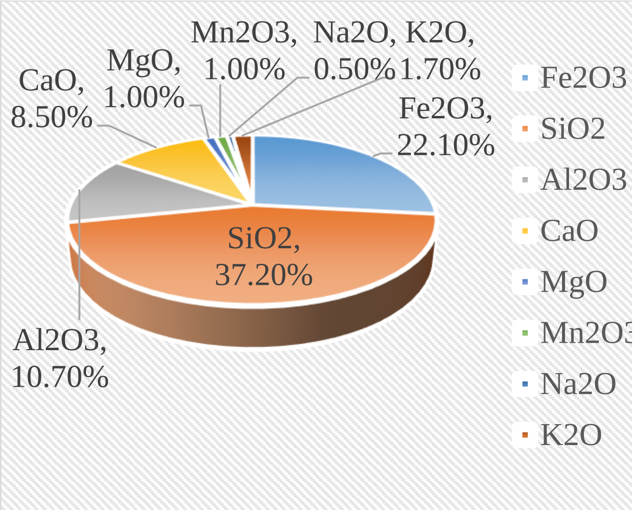

Figure 1: Chemical Composition of Iron ore tailings

2 METHODOLOGY

Materials used for Production of Concrete Samples.

The ordinary Portland cement brand with strength class of 42.5 in accordance with the British standard was used as binder for preparing the concrete samples. The natural sand used as fine aggregate and granite used as coarse aggregate were obtained from a local quarry in Johor. The research focused on the use of three different types of iron ore tailings as fine aggregate serving as partial replacement for the natural sand. These tailings were obtained from ZCM Minerals Kota Tinggi, Landas Seketa Mines Kota Tinggi and Honest Sam Development, Batu Pahat, all in Johor state, Malaysia. The tailings were denoted as ZIOT, LIOT and HIOT respectively.

The experimentally determined physical properties of the aggregate are given in Table 1 while the particle size distribution for all the fine aggregates is shown in Fig. 2. The iron ore tailings HIOT 30% partial replacement of sand in concrete produced the highest compressive strength, the microscopic image of sand is therefore compared with that of HIOTs at 500µm as revealed in Fig. 3 and Fig. 4 correspondingly. The energy dispersive x-ray spectroscopy of sand and HIOTs are shown Figure 5 and Figure 6 respectively.

Table 1, Physical properties of fine aggregate

Physical properties	Fine aggregates			
	Sand	ZIOT	LIOT	HIOT
Size Passing 600µm %	44	95	96	93
Coef of uniformity	3.7	4.7	4.0	3.9
Coef of curvature	0.02	0.01	0.01	0.01
Porosity %	14	12.1	12.4	11.4
Specific gravity	2.65	2.91	2.74	2.79
Fineness Modulus	3.2	1.4	1.3	1.4
Loose unit wt kg/m^3	1459	1598	1554	1629
Compacted unit wt kg/m^3	1696	1817	1774	1839

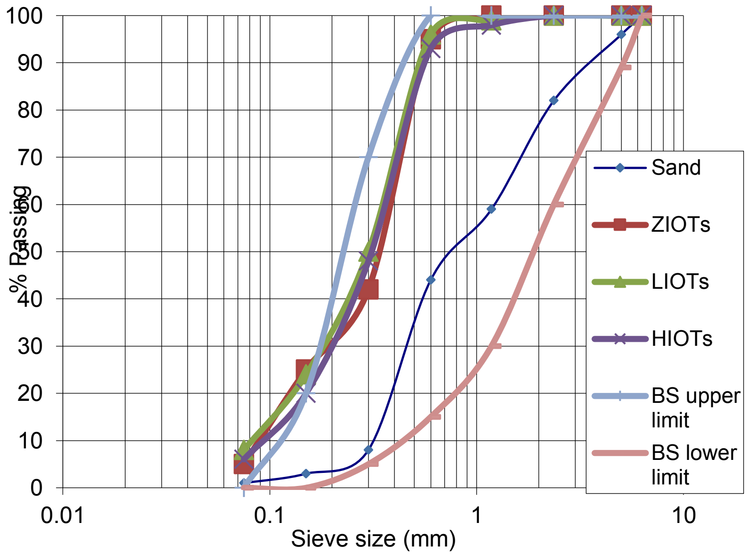

Figure 2. Particle size distribution of fine aggregates

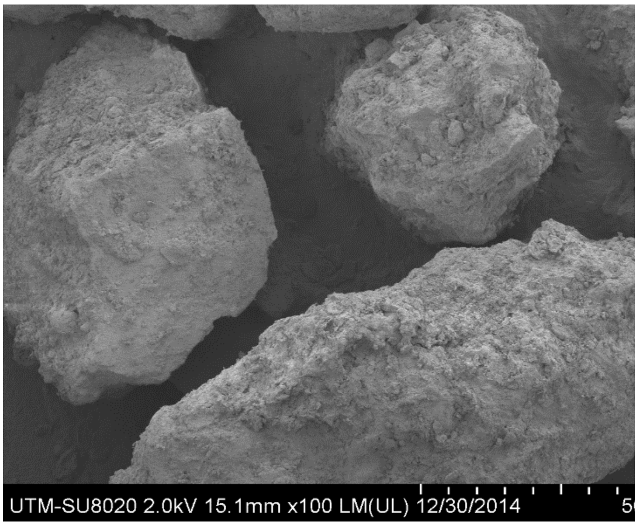

Figure 3: Microscopic image of sand at 500µm

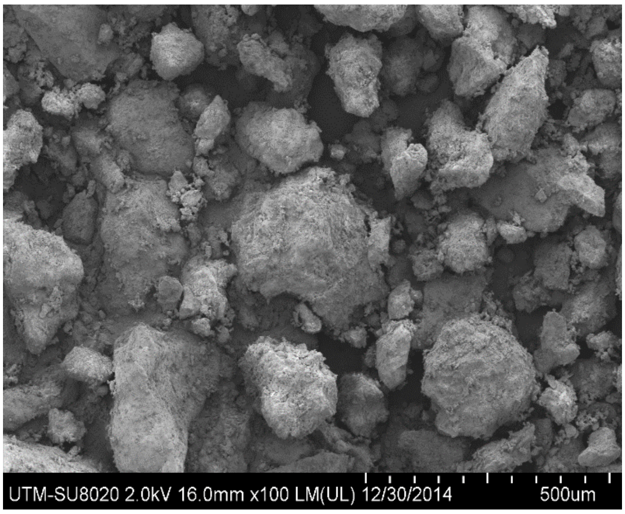

Figure 4: Microscopic image of HIOTs at 500µm

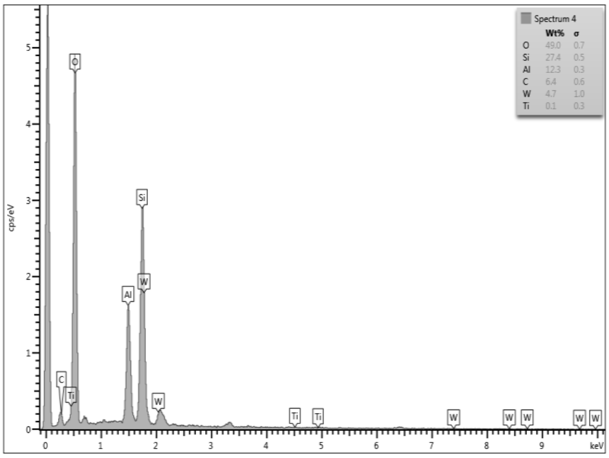

Figure 5: Energy dispersive x-ray spectroscopy of sand

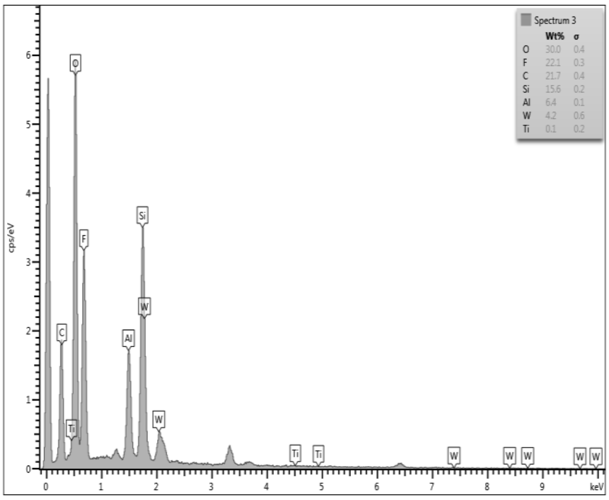

Figure 6: Energy dispersive x-ray spectroscopy of HIOTs

Mix proportion and preparations of concrete samples.

Uniform concrete can be obtained only through proper quality control of all operations from selection and production of materials through batching, mixing, transporting, conveying, placing, consolidation, finishing, and curing (Lambert & James, 2013). The British method for the design of normal strength concrete made with Portland cement as outlined by the Building Research Establishment was used to design normal strength concrete.

Based on the procedure of the concrete mix design and using the appropriate design tables and figures, a normal strength concrete with water content 250 Kg/m^3, cement content 463 Kg/m^3, fine aggregate content 769 Kg/m^3 and coarse aggregate content of 868 Kg/m^3 was designed using water-cement ratio of 0.54. The reference mix adopted is that, which contain sand as the only fine aggregate. For each of the IOTs collected from three different mines, the sand replacement level among the concrete samples that gave the highest compressive strength were selected for further comparison with the control sample. These selected concrete samples are denoted as CZT30 (concrete containing 30% ZCM iron ore tailings as fine aggregate), CLT40 and CHT30 accordingly. The reference concrete sample with 0% tailings is denoted as CTO.

3 RESULTS AND DISCUSSION

The summary of test results, revealing the properties of the fresh concrete is shown in Table 2. There was significant decrease in the workability of the IOTs concrete samples, because of its much greater particle surface area, but concomitantly there was improvement in cohesiveness. This result is in agreement with the findings of Zhao et al., (2014).

There are several reasons for choosing compressive strength as representative index for concrete. First, concrete is used in a structure to resist compression force. Second, the measurement of compressive strength is easier and lastly, other properties of concrete can be related to it (Oritola et al., 2014). The incorporation of IOTs improves concrete compressive strength up to 30% optimum level for the CZT and CHT concrete samples. This result is in line with the findings of Yunfen, (2014) and Zhang et al., (2014).

The flexural strength test is a strong indicator of how porous or dense a concrete sample is, and it's very sensitive to defects in the microstructure, like micro-cracks in the concrete, than compressive strength and splitting tension tests. Similar trend of results was obtained for the flexural strength test, as it was with the compressive strength and splitting tension tests. Table 3, gives the results of the mechanical properties of hardened concrete samples.

The ultrasonic pulse velocity (UPV) test checks the uniformity of concrete samples. The values of pulse velocity for all the concrete samples falls within the range 3.5 – 4.5 Km/s which is considered good. Table 4, indicate the UPV test results for the concrete samples.

The FESEM morphology of materials at magnification of 500µm as shown in Figure 3 and Fig. 4 clearly reveals the particle size effect between iron ore tailings and sand. Within the same area, fewer particles of sand were seen compared with those of IOTs. This implies that the iron ore tailing has larger particles per surface area and can therefore combine effectively with cement to reduce the pores within the cement gel. This will also, drastically reduce the formation of capillary cavities. The microstructure of the concrete samples revealed that more pore space can be seen in concrete with no IOTs compared with those with IOTs. Figure 7 shows the FESEM morphology of concrete sample with no IOTs compared with that containing 30% iron ore tailings, in Figure 8. The structure of this concrete is characterized by the structure of the cement paste and by the structure of the aggregate. Since IOTs is the only variable in the concrete samples production, the crystalline nature of the tailings in terms of void less crystals or fragments of crystals, would have been responsible for the reduction in concrete pores and dense structure of the concrete samples containing iron ore tailings.

TABLE 2: PROPERTIES OF THE FRESH CONCRETE

Fresh Properties	**Concrete Samples**			
	CT0	CZT30	CLT40	CHT30
Slump [mm]	81	59	55	57
Compacting Factor	0.92	0.9	0.89	0.9

TABLE 3: PROPERTIES OF HARDENED CONCRETE

Concrete Samples	**Compressive Strength N/mm^2**		**Flexural Strength N/mm^2**		**Tensile Strength N/mm^2**	
	7	28	7	28	7	28
	(Days)					
CTO	27.2	35.4	3.0	4.5	2.8	3.5
CZT30	31.8	43.7	3.6	4.8	3.4	3.9
CLT40	33.5	42.5	4.2	5.2	3.5	3.6
CHT30	34.5	45.0	4.2	5.3	3.0	3.5

TABLE 4: ULTRASONIC PULSE VELOCITY OF CONCRETE

Property	**CT0**	**CZT30**	**CLT40**	**CHT30**
Ultrasonic pulse velocity Km/s	4.35	4.39	4.37	4.41

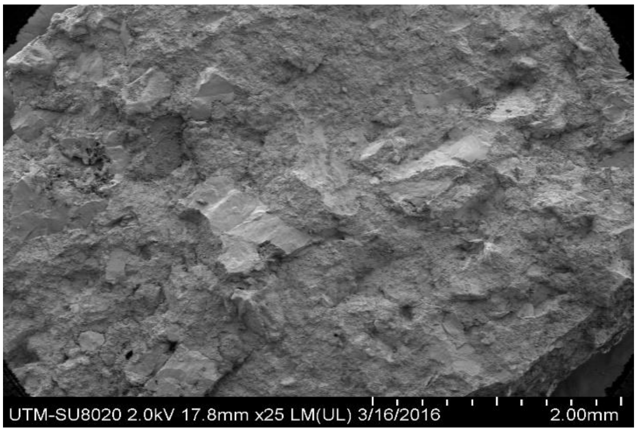

Figure 7: FESEM Morphology of conventional concrete

Figure 8: FESEM Morphology of HIOTs concrete

4 CONCLUSION

This study has brought to focus the utilization in concrete, of three types of iron ore tailings sourced from different locations. Based on the outcome of study, the tailings improved the mechanical properties of concrete. This is an indication that iron ore tailings can be used as fine aggregate in concrete for structural applications. Therefore the material rather than being discarded can be effectively utilized in making concrete.

Considering the performance of concrete containing iron ore tailings in terms of modulus of rupture and splitting tensile strength tests results, this research has revealed that, the tailings can be used in concrete to improve the tensile behaviour of concrete.

The field emission scanning electron microscopy morphology of the concrete samples further confirms the intimate combination between the aggregate interface and the cement paste, due to no transition zone and no feasible cracks around the aggregate in concrete containing the tailings. This suggest that, this concrete can find applications where dense and water resisting concrete are required, such as dam, swimming pool and water retaining tanks.

The pore structure of concrete samples containing iron ore tailings, also indicate that, the material is capable of satisfying the aesthetic requirement of concrete as well as improving the strength.

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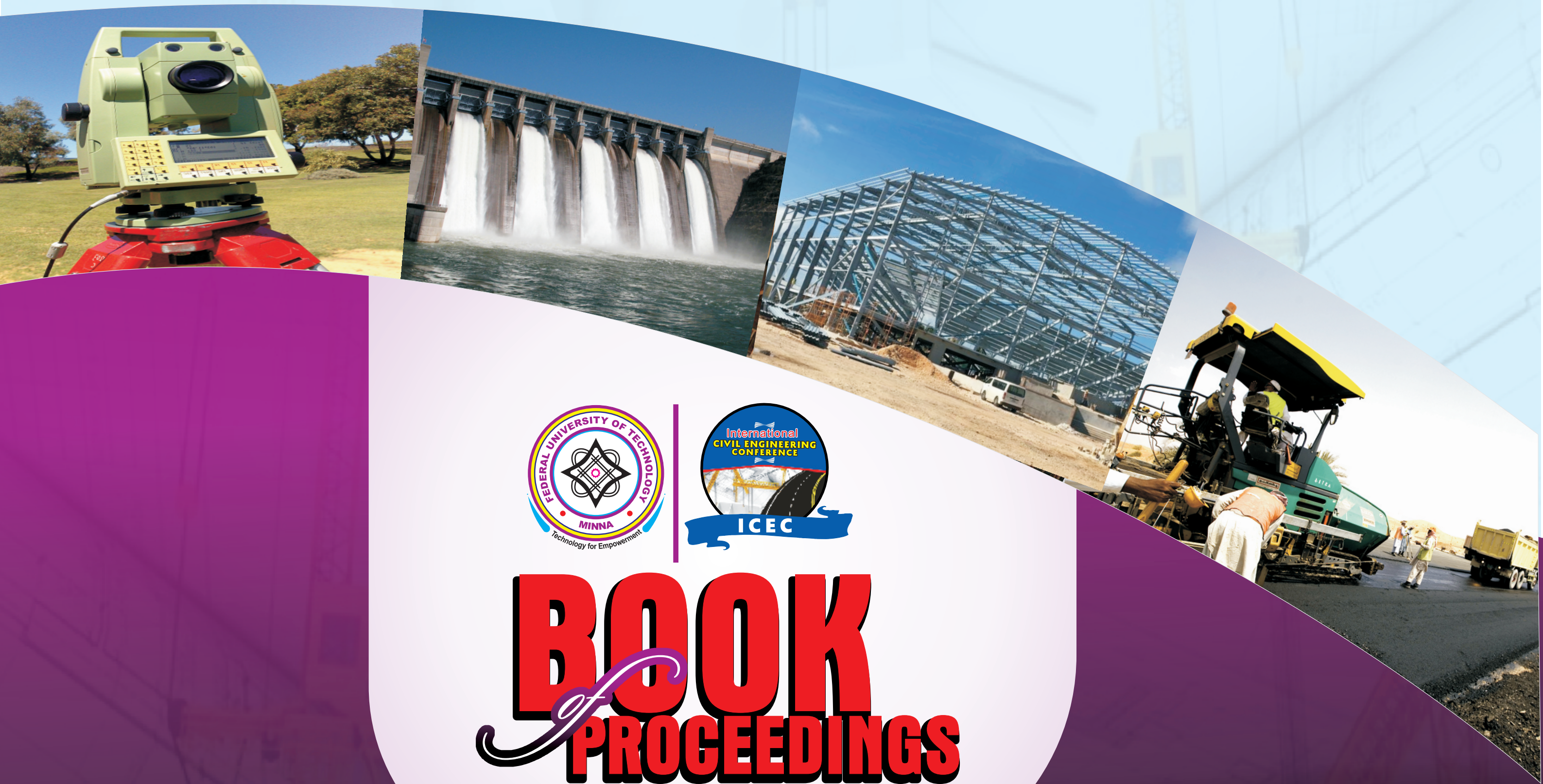

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