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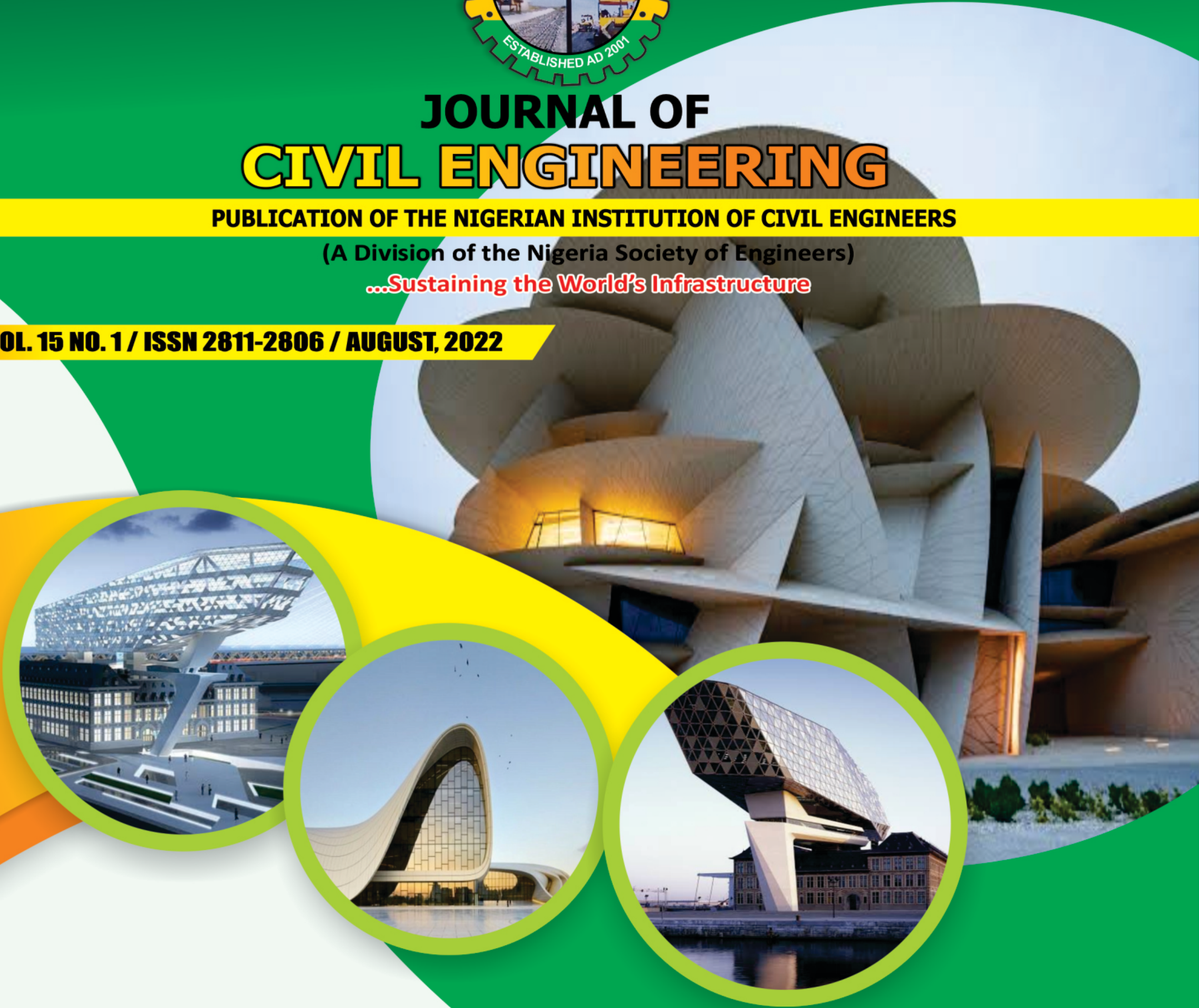

CONTENTS

VISION

Nigerian Institution of Civil Engineers shall be a world renowned body, which will be a pride to all Civil Engineers in Nigeria.

MISSION STATEMENT

To make unique contribution to the development process of the country by constituting itself such that it will be a reservior of Civil Engineers with specialized knowledge, experience and skill, constantly updated through the highest standard of continuing professional development programmes.

To establish and maintain standards and codes for the practice of Civil Engineering in nigeria.

To be constantly in touch with the Government at all levels, for necessary inputs and contributions to policies and matter affecting members and the development of the profession.

Engr. Dr. Jang C. Tanko, FNSE,FINCE
National Chairman

NATIONAL CHAIRMAN'S CORNER

Over the years, Civil Engineering has proven to be the indisputable mother of all engineering professions and by far the broadest of all engineering fields. It focuses in the infrastructure of the entire universe including but not limited to Waterworks, Sewers, Dams, Power Plant, Transmission Towers/Lines, Railroads, Highways, Bridges, Tunnels, Irrigation Canals, River Navigation, Shipping's Canals, Traffic Control, Mass Transit, Airport Runways, Terminals, Industrial Plants, Buildings, skyscrapers etc. Among the important sub divisions of the field are construction Engineering, Geotechnic Engineering, Hydraulic Engineering and Coastal/Oceanic Engineering.

Civil Engineers build the world's infrastructure. By so doing, they quietly but surely shape the history of nation around the world! Life is just one unbearable experience without the numerous contributions of Civil Engineers to the public's health safety and standard of living. Only by exploring Civil Engineers' influence in shaping today's world can we creatively envision the progress of our tomorrow.

It is paramount to me as the National Chairman of NICE at this point of our National Development, that we build an Institution that will continuously strife to make an endearing impact to the growth and development of our nation. It is in this regards that I want to encourage every member of our Institution to get a copy of this journal, which is an integral instrument of professional development to the individual Civil Engineer, and also help to enhance the projection of our innovative ideals to compete both locally and internationally.

Finally, I implore all Civil Engineers to avail themselves of the opportunities presented through the publication of this quarterly Civil Engineering Journal, to come up with technical papers that will improve and promote the practice of Civil Engineering in Nigeria in a positive perspective to the world.

Thank You.

TECHNICAL COMMITTEE CHAIRMAN'S COMMENT

I wish to present a new edition of the Journal of Civil Engineering published by the Nigerian Institution of Civil Engineers. This Journal is conceived to disseminate original contributions from the Academia as well as the Public and Private Sector Practitioners of the profession of Civil Engineering. This year's edition contains scholarly interventions that permeate several branches of the Civil Engineering family profession such as Structural, Geotechnical, Water Resources and Environment, Highway and Transportation Engineering. It is with a great sense of honour and responsibility that I invite you to read and enjoy this new edition.

Engr. Dr. Ezekiel B. Coker, FNSE, FNICE
Chairman Sub-Technical Committee

Determination of Long-Term Concrete Strength Using Accelerated Testing Method by Regression Analysis.

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Abstract

Concrete is a composite material, widely used across the world for construction purposes, consists of fine aggregates, coarse aggregates, cement, and water. When cement is mixed with water a chemical reaction known as hydration occurs leading to the hardening of the concrete. Concrete strength greatly depends on materials used for making it, water-cement ratio, and curing method adopted. The attainment of 28 days compressive strength of any concrete within a shorter period is desirable for good quality control and when construction time is of great concern. This research aimed to generate a regression equation to predict long-term concrete strength based on the accelerated (hot water) method of curing. Laboratory tests were carried out following British Standard on various concrete grades (C25, C37, C50, and C60). The curing ages 3, 7, and 28 days were adopted for normal curing and 24 hours for accelerated curing for the test. From the results, the compressive strength and split tensile strength shows a close range of result between the two curing method adopted. The regression equation generated can be used to predict the long-term concrete strength after 24 hours of accelerated curing.

Keywords: Accelerated testing, regression analysis, predictive model, cement, compressive strength

Introduction

Concrete is a composite material that originated from the mixing of aggregates (fine and coarse), cement, and water at an appropriate mix proportion. Due to the presence of cement and water, hydration takes place leading to the setting and hardening of concrete to form a stone-like material. Hydration is a chemical reaction where ions are released into the mixture thus hardens the concrete. It is a time-dependent reaction. Concrete is a very useful material today in which tons are made of every day. It is used for making pavements, bridges, water tanks, concrete piles, thin shell structures, offshore platforms, nuclear power plants, repair and rehabilitation works, road construction, tunnels, dams, towers, and so on [1]–[3]. These purposes make it the second most used material on earth after water and a multibillion-dollar industry [4].

The quality of concrete such as strength, durability, and resistance to wear are very essential for infrastructural development to fulfill the design requirements. Although, researchers in every century have been working tirelessly toward introducing new concrete types [5]. This has greatly improved the quality of concrete in modern-day design and construction.

The strength of any concrete is determined by the bonding forces within the concrete components and this is achieved by using a good binding material (cement) and water-cement ratio. Concrete strength greatly depends on the type of aggregate used which is a determinate of their parent rock [6]. Concrete strength in the construction industry usually attains 90 percent of its strength at 28days curing age in a moist condition [7], [8]. This is a long time in project management. The required concrete strength may not be achieved when tested after 28 days of curing, meanwhile, other structural members have been erected and loaded while construction is still ongoing. To replace the substandard concrete member(s) will be costlier, time-consuming, and very difficult especially when time is a constraint. Likewise, for a greater concrete strength that is higher than the design requirement, the uneconomical concrete mix will be a waste of costly materials. This necessitates the determination of 28 days compressive strength of any concrete within a very short period of time for a good quality control and construction time management. However, this can be achieved when an accelerated concrete curing method under 24hours is used and the concern concrete sample is taken for testing.

This research was targeted toward establishing regression equation(s) which may be used to predict concrete strength. This was achieved by using a normal curing method (full immersion of concrete in water) and an accelerated curing method. Comparisons of compressive and split tensile strength of concrete were gotten from the laboratory experiments carried out and adequately reported. The regression equation generated will be useful in construction industries of precast concrete, pavement rehabilitation, and control of economic construction schedule.

Literature Review

Concrete usually attains 90% of its characteristic strength at 28 days of production [9]. When concrete target strength is not achieved, it may have experienced water loss due to hot or dry weather condition during the hydration process, this may cause a problem for fresh and hardened concrete, because concrete undergoing such weather condition may lead to the development of tensile strength and thereby resulting to cracks, reduction in concrete strength and durability [4], [7], [10]. During this period, concrete has to be in dampening condition to avert such problems. However, the hydration process is always faster during high temperatures, and therefore concrete gains its strength quickly [7]. Hence, an optimum temperature lesser than the temperature at which water gets evaporated is an advantage if a higher temperature that leads to evaporation of water from the concrete mass will reduce the strength [11]. Therefore, the accelerated concrete curing method makes use of this technique. This is useful in concrete fabrication and precast industry where earlier removal of formwork is paramount for economical purposes. Also, accelerated curing is very useful when maintaining busy road bridges and construction time is of great concern. Various accelerated curing method used to actualize this include; warm water, boiling water, autoclave and microwave method,

Material	Concrete Grade	Accelerated Curing Method	Reference
Concrete	C30	Microwave energy	T.R. Neelakantan, S.Ramsundaram and R. Vinoth (2014)
Concrete	Type 10	**CO_2 Sequestration**	Sormeh Kashef-Haghighi and Subhasis Ghoshal (2010)[12]
Concrete	CEM - 42.5 N	**Infra-Red curing** **Heat coil curing**	
		Halogen curing	

Materials and Methods

Materials

Materials used for the production of concrete specimens tested in this study include; BEM Portland Slag cement (CEM II/B-S) of 42.5R brand, clean water, aggregates (fine and coarse), and Glenium27 additive which conformed BS EN 197-Part 1 [13], BS EN 1008 [14], BS 882 [15] and [16] respectively. All the materials were safely transported to Civil Engineering Department Laboratory, Eastern Mediterranean University, North Cyprus where this experiment was carried out. Before the commencement of the experiment, aggregates were air-dried in the laboratory on a clean surface under room temperature and relative humidity for 4 days. The air-dried specimen in the case of fine aggregate was pulverized with the use of a rubber harmer to disperse the available lumps in the sample.

Methods

Test conducted on the cement used was consistency, setting time, and soundness following BS EN 196 Part 3 [17]. Particle size distribution test was carried out on all in one aggregate (fine and coarse) following BS 812 Part 103.5 [18]. That is, the two aggregates were brought together to form a single aggregate. Graduation curve was plotted using aggregate percentage passing versus sieve diameters. Values of D10 and D60 were obtained from the graph to calculate the aggregate uniformity coefficient (Cu) from equation (1).

$$C_u = \frac{D_{60}}{D_{10}} \quad (1)$$

Where: D60 is the grain diameter at which 60% particles are available in the sample and D10 is the grain diameter at which 10% particles are available in the sample (effective size). This is for soil classification. A soil is said to be classified as well-graded (WS) when Cu is greater than six (Cu>6), otherwise the soil is classified as poorly-graded (SP) [19], [20].

Fineness modulus (FM) is another parameter used to determine the engineering properties of aggregates. This shows how coarse an aggregate is. Aggregate fineness modulus is used together with aggregate gradation when strong and durable concrete is to be produced [19]. This was estimated from equation (2).

$$FM = \frac{\sum Cummulative\ percentage\ retained}{100} \quad (2)$$

The finer the aggregate, the higher the quantity of water required for a good concrete mix, likewise, the higher the fineness modulus, the more coarse the aggregate [19], [21].

A specific gravity test was carried out in the laboratory on the aggregates following BS 1377 Part 2 [22]. Equation (3) was used to estimate the specific gravity of the used aggregate.

$$S_g = \frac{W_1 - W_2}{(W_4 - W_1) - (W_3 - W_2)} \quad (3)$$

Where; S_g is specific gravity, W_1 is empty bottle weight, W_2 is aggregate and bottle weight, W_3 is aggregate, bottle, and distilled water weight, W_4 is bottle and distilled water weight.

Concrete mix design was carried out following BS Concrete Mix Design (DOE method) [23] and BS 5328 Part 2 [24] and a water-cement ratio of 0.5 was used. Concrete batching was carried out by weight. Likewise, concrete was mixed for each concrete grade under laboratory room temperature with the use of a concrete mixer following British Standard DD ENV 206 [25] and BS 1881 Part 125 [26]. The slump test was carried out following BS 1881 Part 102 [27]. Concrete grades worked upon in this research were C25, C37, C50, and C60 respectively. A Glenium27 super-plasticizer admixture of 1% and 3% weight of the cement was added and mixed to produce the concrete grade of C50 and C60 respectively to achieve the higher strength desired. The concrete mixes were thoroughly vibrated using an automatic concrete vibrator in the laboratory that conforms with BS 1881 Part 127 [28] to remove the voids present in the concrete when placing in the molds. Though, caution was taken to avoid concrete bleeding and segregation.

All the concrete grades tested undergone compressive and split tensile testing following British Standard BS 12390 part 3 and Part 6 [29], [30] respectively. The curing age adopted in this research work were 3, 7, and 28 days for normal concrete curing condition (full immersion of concrete sample in water) and 24hours accelerated concrete curing condition (full immersion of concrete in hot water) following British Standard BS 12390 Part 2 and BS 1881 Part 112 [31], [32] respectively. During the accelerated curing, the water was heated to maintain a temperature of 50ºC ± 5ºC. The laboratory experiments were conducted on 12 concrete cubics having 150mm^3 diameter and 12 cylindrical concrete having a diameter of 100mm and 200mm depth for each concrete grade. This makes a total of 48 concrete samples used for the study. Compressive testing machine used for the concrete crushing satisfy the requirement of BS EN 12990 Part 4 [33].

Results and Discussion

Cement test

The result of the test conducted on BEN Portland Slag cement (CEM II/A-S) is presented in Table 1. It shows that the cement satisfies the mechanical and physical requirement of Table 2 in BS EN 197 Part 1 [13] which is suitable to use for construction purposes.

Table 1: Physical Properties of BEM Portland Slag cement (CEM II/A-S)

S/No.	Cement Test	Test Result	BS EN 1961

			Recommendation
1	Consistency	26%	
2	Initial setting time	120mins	≥ 60mins
3	Final setting time	235mins	
4	Soundness	9.23mm	≤ 10mm

Aggregate particle size distribution test

Figure 1 below presents the gradation curve for all in aggregate (fine and coarse) used for the experiment. The curve shows how well distributed the aggregate is which makes it suitable for concrete production as recommended by United State Army Corps Engineering (USACE) [34].

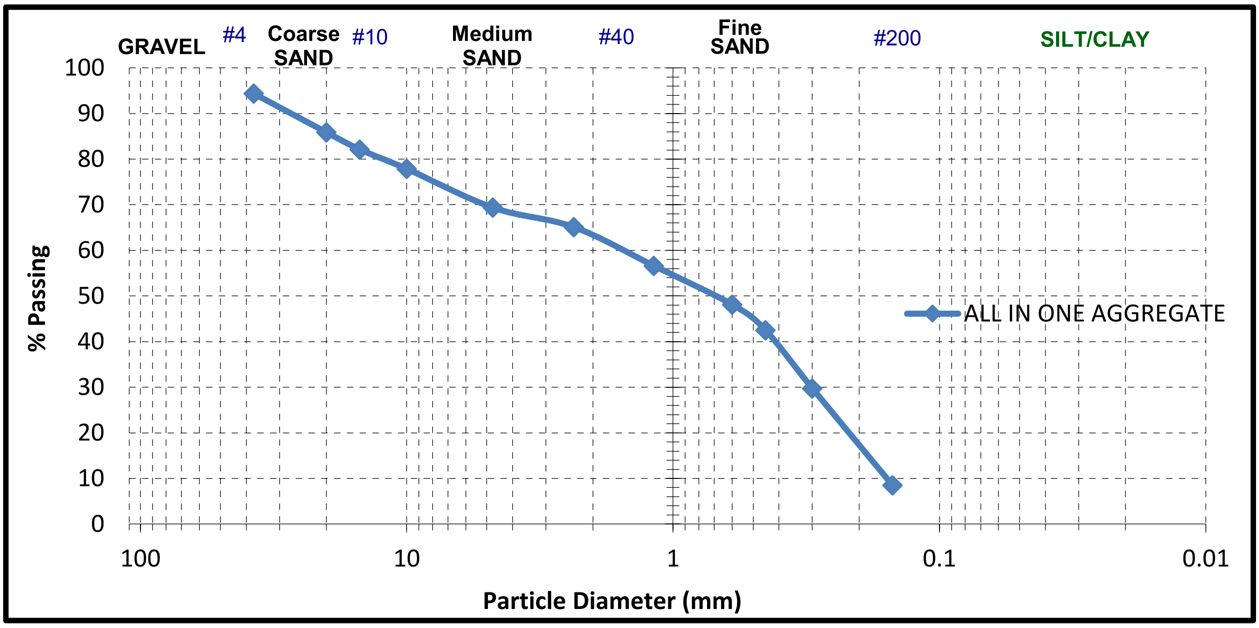

Figure 1: All in One Aggregate Gradation Curve

Table 2 also presents the result extracted from particle sizes distribution carried out in the laboratory and the plotted graph in Figure 1.

Table 2: All in One Aggregate Sieve Analysis Result

S/No	Items	Test Result	ASTM Recommendation	USACE Recommendation
1	Fineness modulus (FM)	5.40		4.00 – 6.75
2	Uniformity coefficient (Cu)	10.30	SP when Cu≥6	
3	Soil Classification	WS		

The fineness modulus of All in one aggregate was found to be 5.4 which fall within the range of United State Army Corps Engineering standard (USACE) [34] and [7]. The uniformity coefficient (Cu) from Table 2 implies that the aggregate is classified as well-graded aggregate haven satisfy the requirement of Neville, 2011 [2], Shetty, 2010 [8] and ASTM D-2487 [20] respectively. Therefore, when such aggregate is used for concrete production, it will produce durable and quality concrete.

Aggregates specific gravity and water absorption test

Table 3: Physical Properties of Aggregates

S/No.	Aggregate Test	Fine Aggregate	Coarse Aggregate
1	Specific gravity saturated surface dry (SSD)	2.73	2.75
2	Specific gravity (Dry)	2.67	2.73
3	Water absorption (%)	2.14	0.85

The specific gravity result ranges from 2.67 to 2.75 as shown in Table 3. The result falls with the acceptable limit of ACI Education Bulletin, 2007 [35] which recommends that, for aggregate suitable for construction, its specific gravity should be range from 2.3 to 2.9. [36], [19] States that, normal-weight aggregates specific gravity (gravels, sands, and crushed stone) ranges from 2.4 to 2.9. [8] Also gave the average specific gravity of rocks to be 2.6 to 2.8, although rocks like basalt and granite could be as high as 3. Therefore, it can be deduced from these analyses that, the specific gravity results obtained from this research can be considered adequate for general construction works.

The water absorption results shown in Table 3 are 2.14% for fine aggregate and 0.85 % for coarse aggregate. This implies that coarse aggregate has lower voids within its particles compared to fine aggregate with higher voids. Therefore, the fine aggregate may require a higher water-cement ratio for the production of concrete due to its water absorption rate.

Slump test

Table 4: Slump Test Result

Concrete Grade	Slump (mm)
C25	80
C37	60
C49	0

C73	0

Slump result presented in Table 4 revealed that concrete grade of C50 and C60 has no slump which shows that, their workability were very poor. This is an indication of higher strength concrete as it is required even though the superplasticizer Glenium27 was used. Meanwhile, concrete mixes of grade C25 and C37 has good workability having a slump of 80mm and 60mm respectively. This is a good workable concrete for construction works.

Compressive strength test

Table 5a: Concrete Compressive Strength Result for Normal Curing Method

Concrete Grade	3 Days (MPa)				7 Days (MPa)				28 Days (MPa)			
	Y_1	Y_2	Y_3	AVG ($\bar{Y}$)	Y_1	Y_2	Y_3	AVG ($\bar{Y}$)	Y_1	Y_2	Y_3	AVG ($\bar{Y}$)
C25	21.80	21.70	21.60	**21.70**	33.60	32,90	32.50	**33.00**	41.50	41.70	40.90	**41.37**
C37	26.00	24.10	23.20	**24.43**	33.40	33.90	33.30	**33.53**	45.50	46.30	43.70	**45.17**
C50	43.80	44.50	43.70	**44.00**	50.60	47.00	46.00	**47.87**	56.20	56.70	56.40	**56.43**
C60	54.90	54.40	54.60	**54.63**	56.60	61.00	58.50	**58.70**	67.60	60.70	63.50	**63.93**

Table 5b: Concrete Compressive Strength Result for Accelerated Curing Method

Concrete Grade	24Hrs. Accelerated (MPa)			
	X_1	X_2	X_3	AVG ($\bar{X}$)
C25	21.30	22.80	22.20	**22.10**
C37	23.40	23.30	20.20	**21.30**
C50	26.40	27.00	26.70	**26.70**
C60	31.70	30.69	31.73	**31.37**

Table 6: Average Concrete Compressive Strength Result

Concrete Grade	3 Days (MPa)	7 Days (MPa)	28 Days (MPa)	24Hrs Accelerated (MPa)
C25	21.70	33.00	41.37	22.10
C37	24.43	33.53	45.17	21.30
C50	44.00	47.87	56.43	26.70
C60	54.63	58.70	63.93	31.37

Table 6 presents the average compressive strength of the tested concrete for each curing age considered in this research work. 3 Days concrete strength ranges from 21.70MPa to 54.63MPa, 7 days range from 33.0MPa to 58.7MPa and 28 days range from 41.31MPa to 63.93MPa respectively. It can be observed that the targeted concrete strengths were achieved despite the slump for concrete grades C50 and C60 were zero. In a short term, all the concrete samples made for this research work satisfy the requirement of Table 8 in BS EN 197 Part 1 [13] regarding the compressive strength for concrete made with the cement used (Portland Slag Cement). The concrete grade of C25 is useful for residential building, concrete grade of C37 is useful for commercial building and sometimes bride construction. Meanwhile, concrete grades of C50 and C60 are usable in a high-rise buildings, skyscrapers, hydropower projects, dams, and bridge construction [37], [38]. Therefore, the method used toward achieving the strengths may be adopted where needed in the construction industry. Figure 2 below shows the graphical representation of the average compressive strength results in Table 6.

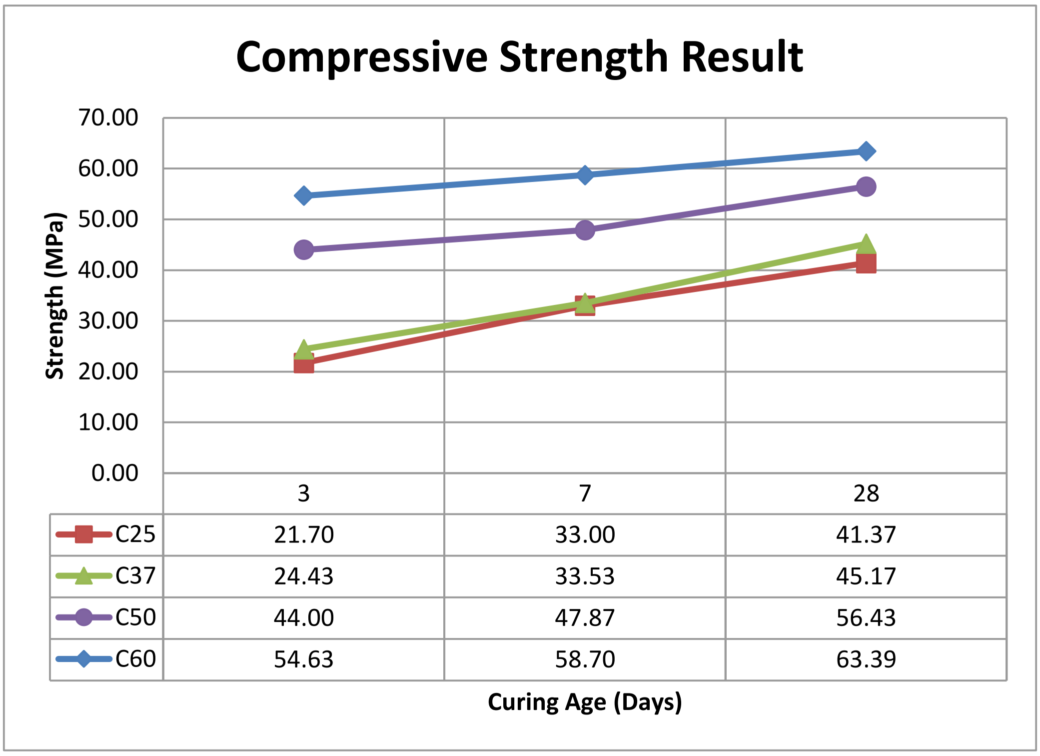

Figure 2: Compressive Strength Result Chart

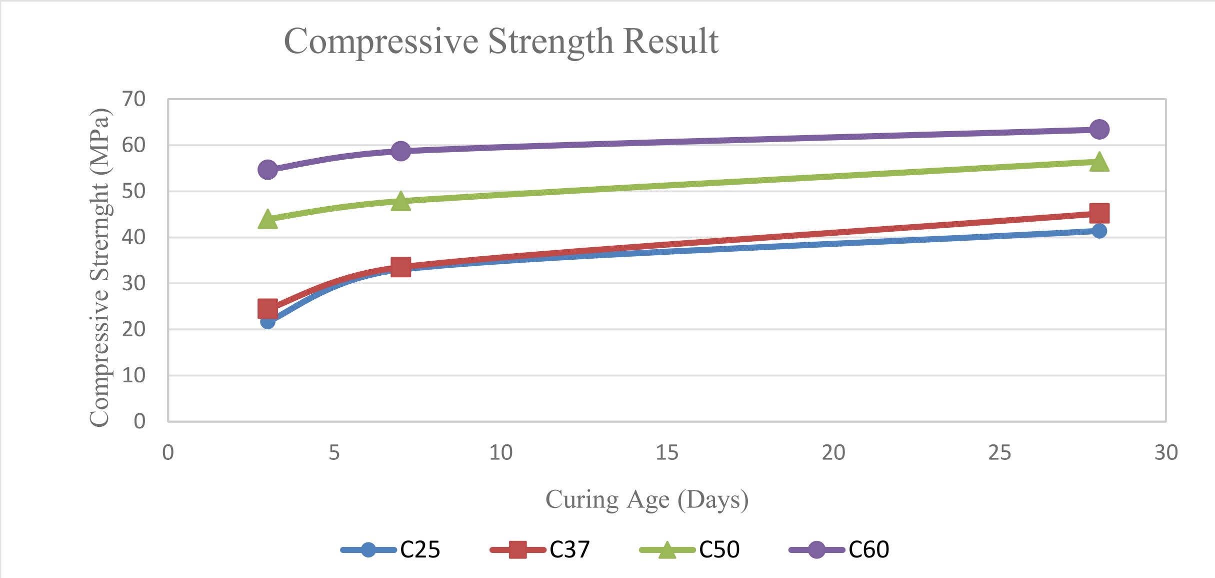

Figure 2: Compressive Strength Result Chart

Tensile split strength test

Table 7a: Split Tensile Strength Result for Normal Curing Method

Concrete Grade	3 Days (MPa)				7 Days (MPa)				28 Days (MPa)			
	Y_1	Y_2	Y_3	AVG ($\bar{Y}$)	Y_1	Y_2	Y_3	AVG ($\bar{Y}$)	Y_1	Y_2	Y_3	AVG ($\bar{Y}$)
C25	7.65	7.90	7.63	**7.73**	10.48	10.52	10.45	**10.50**	15.68	13.99	13.99	**14.45**
C37	14.63	13.24	13.73	**13.67**	12.69	15.69	14.03	**14.14**	20.78	19.91	19.03	**19.91**
C50	15.5	16.12	15.61	**15.73**	18.82	19.20	19.13	**19.05**	22.65	23.20	22.40	**22.75**
C60	35.80	34.25	35.10	**35.05**	34.54	36.50	35.46	**35.50**	34.85	36.90	35.80	**35.85**

Table 7b: Split Tensile Strength Result for Accelerated Curing Method

Concrete Grade	24Hrs. Accelerated (MPa)			
	X_1	X_2	X_3	AVG ($\bar{X}$)
C25	7.17	9.41	4.59	**7.05**
C37	7.60	7.90	7.60	**7.70**
C50	12.15	10.73	9.60	**10.81**
C60	17.55	17.60	15.45	**16.87**

Table 8: Average Split Tensile Strength Result

S/No.	Concrete Grade	3 Days (MPa)	7 Days (MPa)	28 Days (MPa)	24Hrs Accelerated (MPa)
1	C25	7.73	10.50	14.82	7.05
2	C37	13.87	14.14	19.91	7.70
3	C50	15.73	19.05	22.75	10.85

4	C60	35.05	35.50	35.85	16.87

Table 8 presents the average split tensile strength of the tested concrete for each curing age. 3 Days split tensile strength ranges from 7.73MPa to 35.05MPa, 7 days range from 10.50MPa to 35.50MPa and 28 days range from 14.82MPa to 35.85MPa respectively. Percentages of split tensile strength of concrete to their respective compressive strength at 28days curing age ranges from 34.9% to 56.1% for normal curing concrete while 31.9% to 53.7% for accelerated curing concrete. [39]–[42] State that, the tensile strength of any good concrete should not be less than 8 to 10 percent of its compressive strength. This implies that the concrete grades worked on processed good tensile strength that can support steel reinforcement of structural elements when used in civil engineering construction. As such, all the strength results gotten in this study have proven to be adequate for construction purposes.

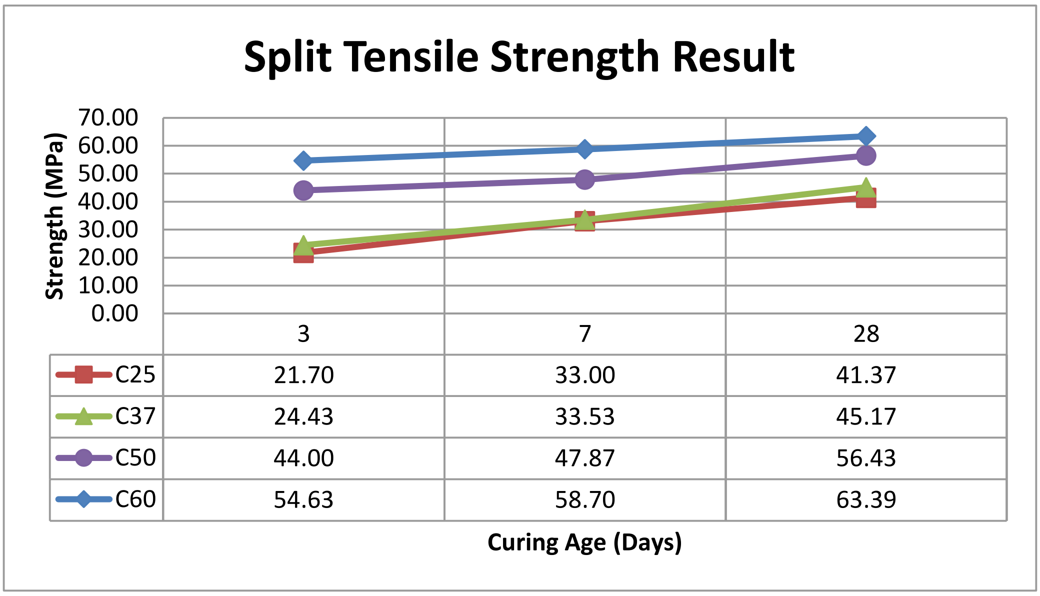

Figure 3: Split Tensile Strength Result Chart

Regression analysis

$$Y = a + bX + S_e \quad (4)$$

$$b = \frac{S_{xy}}{S_{xx}} \quad (5)$$

$$a = Y - bX \quad (6)$$

$$S_{xy} = \sum((X_i - \bar{X})\,(Y_i - \bar{Y})) \quad (7)$$

$$S_{xy} = \sum(X_i - X)^2 \quad (8)$$

$$S_{xy} = \sum(Y_i - Y)^2 \quad (9)$$

$$S_e = \sqrt{\frac{1}{n-2}(Syy - \frac{Sxy}{Sxx})} \quad (10)$$

Where; Y is the dependent variable, X is the independent variable, a is the intercept, b is the slope, S_e is residual standard deviation, $\bar{X}$ is accelerated concrete curing mean, $\bar{Y}$ is normal concrete curing mean.

Equation (5) to (10) was used to estimate the regression for both compressive and split tensile strength and the final regression equation was generated from equation (4).

Compressivestrengthregression analysis

Table 7a and 7b were used to estimate the regression analysis which is presented in Table 9 and 10 respectively. Table 10 also presents the result of the estimated concrete compressive strength from 41.81MPa to 62.29MPa.

Table 9: Concrete Compressive Strength Regression Data

Concrete Grade	Sxx	Syy	Sxy	b (Sxy/Sxx)	a ($\bar{Y}$-b$\bar{X}$)	Se	Slope Equation (Regression Equation)
C25	1.14	0.346667	0.08	0.070175	39.8191228	0.583997116	Y = 39.82 + 0.0701X
C37	6.62	3.546667	1.18	0.178248	41.3733233	1.826563533	Y = 41.37 + 0.1783X
C50	0.18	0.126667	0.15	0.833333	34.18	0.040824829	Y = 34.18 + 0.833X
C60	0.7009	24.0867	0.3407	0.486089	48.6813782	4.890919072	Y = 48.68 + 0.4861X

Table 10; Estimated Concrete Compressive Strength Result

Concrete Grade	3 Days (MPa)3	7 Days (MPa)	14 Days (MPa)	21 Days (MPa)	28 Days (MPa)
C25	40.03	40.32	40.81	41.31	41.81
C37	41.90	42.62	43.87	45.11	46.36
C50	36.68	40.01	45.84	51.67	57.50
C60	50.14	52.08	55.49	58.89	62.29

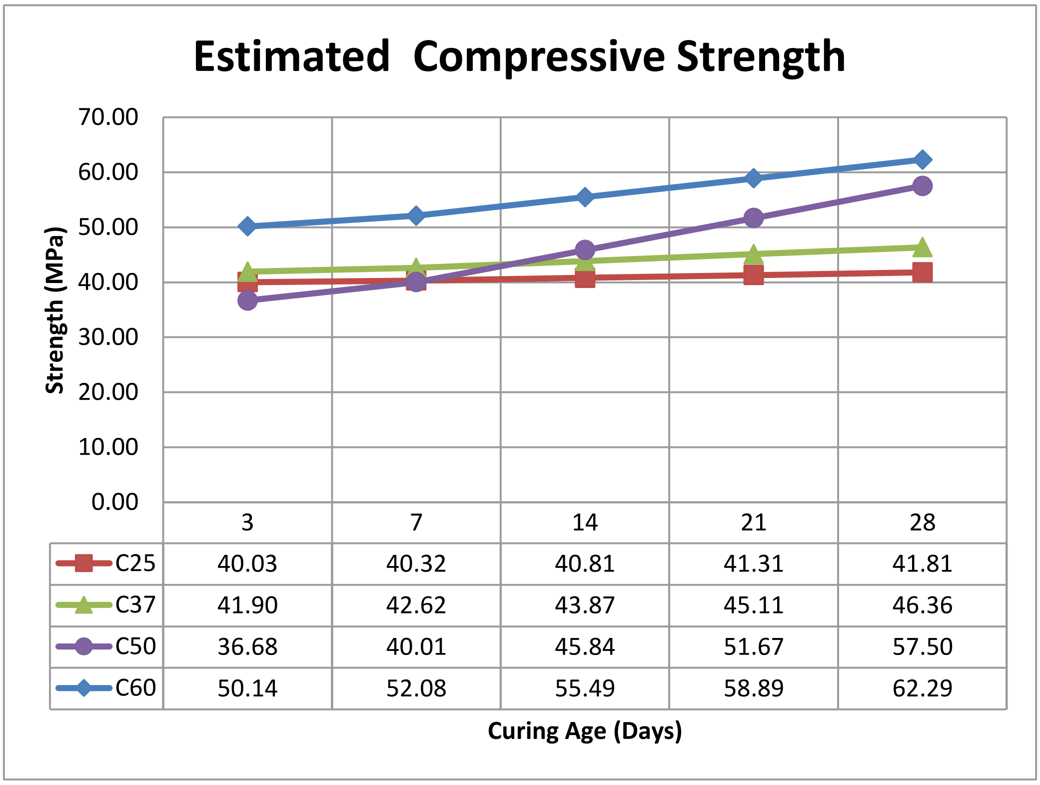

Figure 4; Estimated Compressive Strength Chart

Table 11: Crushing and Estimated Concrete Compressive Strength Result at 28 Days

Concrete Grade	Crushing Strength (MPa)	Estimated Strength (MPa)	Difference (±MPa)	Percentage to Crushing Strength (%)
C25	41.37	41.81	0.44	1.00
C37	45.17	46.36	1.19	2.60
C50	56.43	57.50	1.07	1.90
C60	63.93	62.29	1.62	2.61

The crushing and estimated result for concrete compressive strength at 28 days reveals that the difference between the two results ranges from ±0.44MPa to 1.62MPa. Likewise, the percentage also shows ranges from 1.00% to 2.61%. It can be deduced that both the differences and percentages are negligible. Therefore, the estimation is adequate and can be used to make a good decision in Civil Engineering construction management.

Split tensile strength regression analysis

Table 5a and 5b were used to compute regression analysis which is summarized in Table 12 and 13 respectively. Table 13 also shows the calculated compressive strength result ranges from 14.98MPa to 36.43MPa.

Table 12: Concrete Split Tensile Strength Regression Data

Concrete Grade	Sxx	Syy	Sxy	b (Sxy/Sxx)	a ($\bar{Y}$-b$\bar{X}$)	Se	Slope Equation (Regression Equation)
C25	11.6312	1.9385	0.2428	0.020875	14.392832	1.390478902	Y = 14.393 + 0.02087X
C37	0.06	1.5313	0.001	0.016667	19.7816667	1.237450336	Y = 19.782 + 0.01667X
C50	3.2653	0.335	0.2535	0.077635	21.9092181	0.561533302	Y= 21.909 + 0.0776X
C60	3.0117	2.105	0.1575	0.052296	34.9677657	1.448020502	Y = 34.968 + 0.0523X

Table 13; Estimated Concrete Split Tensile Strength Result

Concrete Grade	3 Days (MPa)3	7 Days (MPa)	14 Days (MPa)	21 Days (MPa)	28 Days (MPa)
C25	14.46	14.54	14.69	14.83	14.98
C37	19.83	19.90	20.01	20.13	20.25
C50	22.14	22.45	23.00	23.54	24.08
C60	35.12	35.33	35.70	36.07	36.43

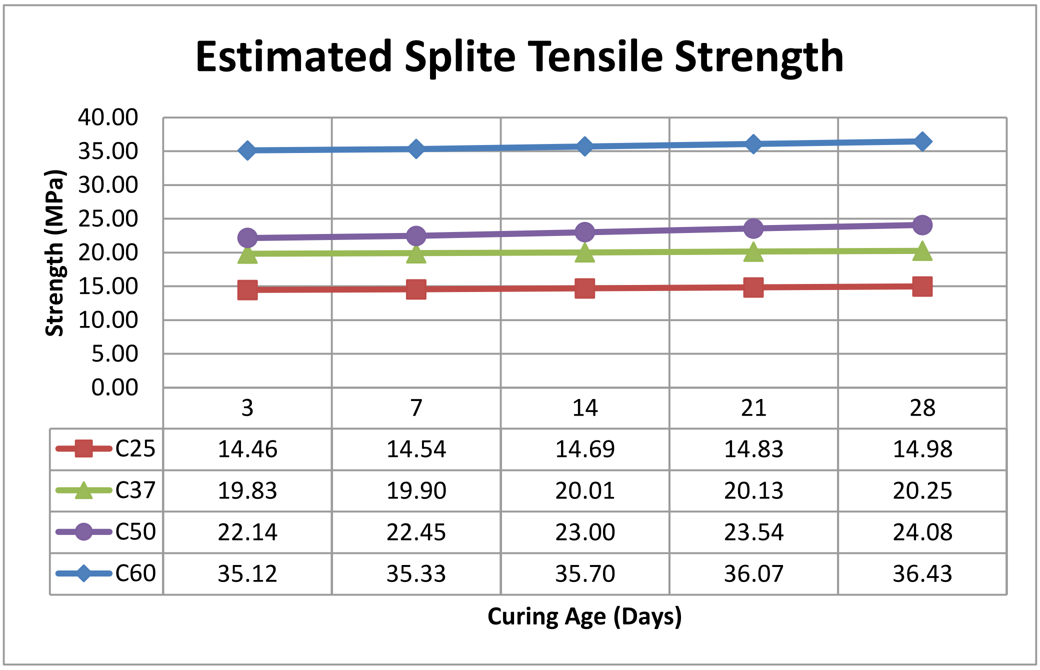

Figure 5; Estimated Split Tensile Compressive Strength Chart

Table 14: Crushing and Estimated Concrete Split Tensile Strength Result at 28 Days

Concrete Grade	Crushing Strength (MPa)	Estimated Strength (MPa)	Difference (±MPa)	Percentage to Crushing Strength (%)
C25	14.82	14.98	0.16	1.08
C37	19.91	20.25	0.34	1.71
C50	22.75	24.08	1.33	5.85
C60	35.85	36.43	0.58	1.62

Table 14 shows the differences between the crushing and estimated result for split tensile strength at curing age 28 days are 0.16MPa, 0.34MPa, 1.33MPa, and 0.58MPa, their percentages are 1.08%, 1.71%, 5.85%and 1.62% for concrete grades C25, C37, C50, and C60 respectively. Their differences and percentages are very much insignificant except for that of C50 that has 5.85% but still within the acceptable statistical data. Therefore, the estimate is a good one and reliable which can be used to make an adequate decision in the construction industry and management.

Summary

This paper generates regression equations for different grades of concrete using the normal concrete curing method and 24 hours accelerated concrete curing method. Generally, four good concrete grades with different mix designs were studied following relevant standards. The findings and conclusion from the laboratory experiment and regression analysis carried out are as follows.

- The laboratory tests conducted on the materials used for this research work reveal that the materials are very okay as they all satisfy the requirements of the standard used for each test. Therefore, they are all suitable for concrete production.
- Compressive strengths targeted were all achieved. That is, at 28 days curing ages, C25, C37 C50, and C60 has an average compressive strength of 41.37MPa, 45.17MPa, 56.43MPa, and 63.93MPa respectively. Although, a Glenium27 superplasticizer admixture of 1% and 3% weight of the cement was mixed with the produced concrete of grade C50 and C60. This is an adequate measure taken toward achieving higher strength concrete and it is recommended for construction purposes.
- The split tensile strength of concrete grade C25, C37, C50, and C60 were found to be 14.45MPa, 19.91MPa, 22.75MPa, and 35.85MPa respectively. This represents 34.9%, 44%,

40% and 56.1% to their respective compressive strength. This is an indication that they all have a good tensile strength that can support concrete reinforcement in tension.

- The regression equations generated were used to estimate the long-term strength (compression and split tensile strength) for various concrete grades. Considering the grades of concrete examine, used mix design, different curing methods, and the regression equations generated, it is evident that, the prediction of long term concrete strength for grade C25, C37, C50, and C60 is made easier after the concrete crushing of any sample subjected to 24hours accelerated curing. This is achieved by using the strength value obtained from the crushing machine which is taken as X in the regression equation generated. Then, Y which is the expected concrete strength at 28 days curing age is calculated. The regression equations generated are presented in Table 9 and 12 for compressive strength and split tensile strength respectively.
- Having achieved the set goals for this research work, it is believed that, this will be very useful in the construction industry. The construction managers can use this study in their respective projects to project what the strength of their concretes will be after 24 hours of accelerated curing age. Effective project management can be achieved as it will help to effectively manage construction time thereby reducing the cost of construction.

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ACCIDENTS ANALYSIS OF MOTORCYCLES IN AKURE, NIGERIA

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Abstract—Motorcycle accident crash injuries are often fatal. In recent years there has been an increase in motorcycling as means of transport in Nigeria and Africa as a whole. It is believed that motorcycle is also a source of employment and income to youth. This increase in motorcycling has been accompanied by an increase in motorcycle crash injuries. This study sought to identify the factors associated with motorcycle crash injuries. The research was a community-based descriptive cross-sectional study carried out among commercial motorcyclists in Akure, Nigeria. The instrument used in collecting data was a semi-structured interviewer-administered questionnaire, Data on demographic factors, accident locations, and types of accidents that occur mostly were recorded. Data were entered into the statistical package for social studies (SPSS) program for clearing, coding, and statistical analysis. A P-value of 0.05 was considered significant. About 62% of the participants had been involved in road traffic crashes since they started to ride a motorcycle while 38% had not been involved in accidents. The majority of motorcyclists were found to be between the age brackets of less than 20 and 25-29 years representing 76% of riders. Also, most of the riders 50% think over-speeding is the major cause of motorcycle accidents. Regression models were also generated to show clearly the significance of a dependent variable on the independent variable.

Keywords—Motorcycle, Accident, Transportation, Public Transport.

INTRODUCTION

Motorcycle-based public transportation continues to prevail in most cities of developing nations and this is attributed to their ability to manoeuvre heavy traffic jams and navigate on poor roads in the countryside and their affordability by all sectors of the society (Peden *et al*, 2002). They are private-sector initiated and can adapt to the needs of the passengers. Motorcycles as a means of mobility have become an issue for urban transport planners especially among developing countries like Nigeria.

In-depth knowledge of motorcycle accident data, unsafe vehicle conditions, and driver behaviours are essential in investigating factors that bring about the occurrence of motorcycle accidents. To use motorcycle accident data to generate solutions, an effective accident data gathering system is therefore needed. The specific need to proffer pragmatic solutions to rising motorcycle fatalities such as what are the vehicle, roadway, and rider-related factors that are associated with motorcycle crashes prompted the need for this research work.
This paper aims to analyze motorcycle accidents in Akure Township. The objectives are.

- To collect data and information on motorcycle accidents from riders, hospitals, and traditional orthopaedic centres using a well-structured questionnaire.
- To analyze collected data using advanced statistics and develop a model that could be used to predict motorcycle accidents.
- To propose pragmatic solutions that may be used to reduce motorcycle accidents.

The study area is Akure, the administrative capital of Ondo State. Akure, the largest city in Ondo State became the state capital of Ondo State in 1976. Akure is a city in southwestern Nigeria with a population of 420,594 people. The town is located within 7^{0} 15′ North of the Equator and Longitude 5^{0} 5′ East of the Greenwich Meridian (Fadairo, 2013). The area towards Ado-Ekiti and Idanre is hilly and studded with large granite formations, rising to 410 meters and 496 meters above sea level respectively (Wikipedia, 2015).

Since conventional transit systems often are not readily available in the area, unconventional para-transit modes such as motorcycles popularly called *Okadas* act as supplements. Akure has 54 motorcycle parks.

Plate 1 is a map showing the various motorcycle parks in Akure.

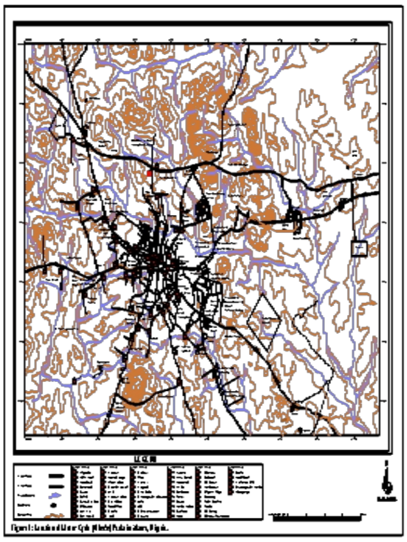

Plate 1: Location of Motorcycle parks in Akure

The selection of a motorcycle as a major mode of transportation can be attributed to its cheap cost of purchase when compared with other means and its low cost of maintenance. Its popularity however has led to numerous researches on it. Several studies have been conducted across the world concerning motorcycle accidents and their causes.

In New Zealand, (Reeder et al, 2009) also observed that 20% of fatalities were motorcycle riders with motorcycles also representing 5% of the licensed vehicle. A study carried out in north-central Nigeria revealed that none of the motorcyclists involved in the study wore helmets (Nwadairo *et al*, 2011) while Ogunmodede *et al* (2012) in his study done in Oyo state found that 33.8% of riders sampled in Oyo state, Nigeria uses crash helmets while driving with less than 10% of the motorcycle riders found to use helmets in Uyo, Nigeria (Ofonime, 2012). However, like most active interventions, the full benefit of protection is dependent on many parameters including the rate of usage, whether standard or approved devices are used and how they are used. In the study carried out by Nwadairo, head injury accounted for the most frequently occurring injury (40.1%) and all mortalities were because of the head injury.

Mu'awiyyah and Sagir (2005) in their study done in Nigeria discovered that the use of motorcycles for commercial purposes was done majorly by males who have not received formal training hence lacking basic knowledge about riding techniques and different protective devices available for use. Naddumba (2005) carried out a study on motorcycle accidents in Uganda and observed that the proportion of motorcycle crashes among road traffic injuries is between 15%-27% while a study done by (Tham *et al*, 2005) in Singapore observed a higher proportion of motorcycle crashes of 49.1% to 62%. Naddumba also observed that most motorcycle crash victims are those at a productive age with males being more involved. He stated from his study that in both developing and developed countries, the peak age for motorcycle-related injury is the late teens and early to late 20s with males more than females while Tham *et al* stated that the mean age of most motorcycle crash victims is 32.5 years with 96.1% being males. Naddumba observed that most crash injury victims were self-employed individuals and lower extremity injuries which affect 32% to 80% of injured riders are the most common outcomes of non-fatal motorcycle crashes. His study also highlighted that the commonest extremity injuries were fractures to the tibia and fibula. He indicated five types of motorcycle collision namely motorcycle and motor vehicle, motorcycle and motorcycle, motorcycle and pedestrian, lone motorcycle, motorcycle, and stationary objects.

From a safety perspective, a helmet is the most important asset for a motorcycle rider. It has been shown that an unprotected rider is 40% more likely to die in a crash than a rider who is wearing a helmet. Studies by (Cervero, 2007) in Yola, Adamawa State, Nigeria showed that 88% of the motorcyclists were aged between 18 and 30 years. The prevalence of motorcycle injuries of 68% was higher than that recorded from the study in western Nigeria, 43.5% (Owoaje *et al*, 2005). Labinjo *et al* (2009) observed that the proportion of motorcycle accidents among road traffic accidents was 54%. In-depth knowledge of motorcycle accident data, unsafe vehicle conditions, and driver behaviours are essential in investigating factors that bring about the occurrence of motorcycle accidents. To use motorcycle accident data to generate solutions, an effective accident data gathering system is therefore needed. The specific need to proffer pragmatic solutions to rising motorcycle fatalities such as what are the vehicle, roadway, and rider-related factors that are associated with motorcycle crashes prompted the need for this research work. This research investigated motorcycle accidents in Akure Township by collecting and analyzing data on motorcycle accidents from riders, hospitals, and traditional orthopaedics.

MATERIALS AND METHODS

The population of the study is made up of mostly motorcyclists, physiotherapists, medical practitioners, and FRSC officials within Akure metropolis Data was collected majorly on Thursdays and Fridays with a minimum of 20 questionnaires administered per week, The research methodology used for this study was based on the distribution of questionnaire throughout motorcycle parks, orthopaedic centres and the State Hospital in Akure and also a collection of data from the Federal road safety corps (FRSC) office. Questionnaires were taken to respondents in their various places of work and the various parks in Akure Town. The questionnaire was used to get four categories of information from the respondents of Biodata of the Respondents, Experience of the Respondents i.e., accident involvements, Respondents' view on motorcycle accident and its causes, and Accident's occurrences and safety measures.

This involves the instruments used to carry out the research. The work of data collection begins after a research problem has been defined and plans on how research is to be carried out have been ironed out. The research instrument used was a set of questionnaires containing series of questions printed or typed in a definite order on a form or set of forms sent to the person(s) concerned with a request to answer and return the questionnaire to gather information. It was divided into two parts, one for data gathering from the motorcycle parks and two, for data collection from the health practitioners. The first contained twenty-six (26) questions while the second had thirteen (13) questions. Data was collected through several methods which included the interview method, observation method.

Under the interview method, the information was gotten by interviewing respondents. The language of communication was the English language, but the translation of each question orally to any respondent who was not literate enough to understand the questions in the language medium of communication was ensured. So, for the illiterate respondent, the person administering the questionnaire helps the respondent in filling the questionnaire himself. Everyone assigned to this project speaks the English language fluently and the native language medium of expression used was Yoruba.

RESULTS

The age distribution of respondents is shown in table 1. It indicates that most of the respondents fall within the 25-29 age category representing 25% closely followed by the 35-39 category at 20%.

Table 1: Age groups of respondents

Age	Frequency	Percent
<20	31	7.75
20 -24	40	10
25 -29	100	25
30 -34	52	13
35 -39	80	20
40 -44	30	7.5
45 -49	33	8.25
50 -54	20	5
55 - 59	11	2.75
>60	3	0.75
Total	400	100

62% of the respondents have been involved in motorcycle accidents while the remaining 38% of the respondents stated that they have not been involved in an accident. Therefore, indicating that most of the respondents have been involved in at least an accident.

The respondents were asked which road user they felt was most likely to cause a motorcycle accident, 52% stated that car drivers were the most likely cause of a motorcycle accident while 24% felt motorcyclists themselves were the most likely cause. 10% went for pedestrians, 6% said truck drivers cause motorcycle accidents the most and 1% went for cyclists. Of the respondents, 7% selected other causes which include the presence of animals crossing etc. as the most likely cause of a motorcycle accident. This is therefore an indication that motorcyclists themselves and car drivers at a combined 76% are the most likely cause of a motorcycle accident. From the study, it was found that motorcyclists are most at risk from accidents arising from collision while overtaking and overshooting bends in the road; this is as indicated by the respondents in figure 1.

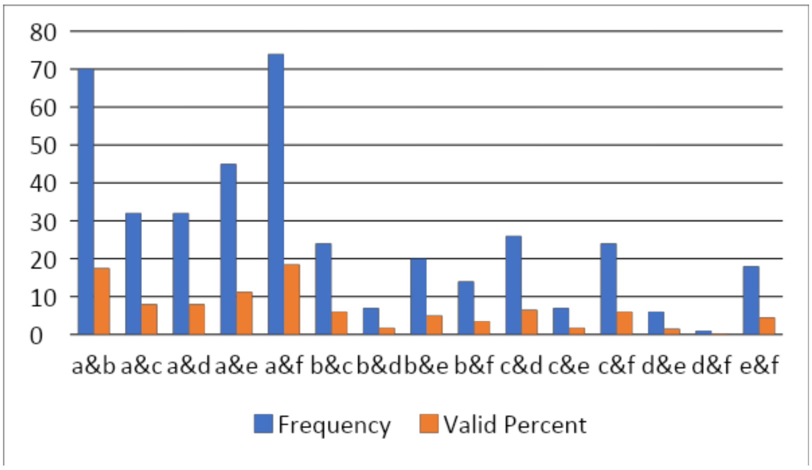

Figure 1: Road users that cause motorcycle accidents

NOTE:
a = Collision while overtaking other road users, b = Being hit from behind by other road users (rear–end shunt), c = Collisions with right-turning vehicles, d = Collisions with left-turning Vehicles, e = Poor riding technique leading to loss of control of the motorcycle and f = Overshooting bends in the road

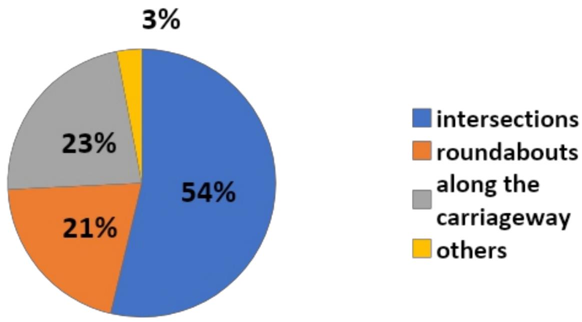

Figure 2: Locations of accidents

From figure 2, Intersections with 54% were identified by the respondents as the point where motorcycle accidents mostly occur on a road followed by those along with the carriageways and roundabouts at 23% and 20% respectively. Other points of the road according to 3% of the respondents accounted for motorcycle accidents.

Over-speeding was identified as the most common cause of motorcycle accidents by 50% of the respondents followed by. 21% of the respondents think bad roads is the major cause of motorcycle while 19% of the respondents think the major cause of motorcycle accidents is recklessness on the part of the rider. According to table 2, alcoholic influence and poor maintenance of motorcycles were the least common cause of motorcycle accidents at 7% and 3% respectively. This, however, indicates that over-speeding is the most common cause of motorcycle accidents.

Table 2: Causes of motorcycle accidents.

Causes of accident	Frequency	Percentage
Over speeding	200	50
Bad roads	85	21.25
Recklessness	75	18.75
Alcoholic influence	30	7.5
Poor maintenance of motorcycle	10	2.5
Total	400	100

For this study, two Multiple Regression Models were developed as described below.
In the first model (model 1), Motorcycle accident death was the dependent variable while the severity of the accident namely, fatal, serious, and minor was the independent variable. The second model (model 2) had the Occurrence of a motorcycle accident as the dependent variable with Over speeding, Bad roads, and Recklessness as the independent variables.
The multiple regression equation takes the form.

$$Y = ß + ß_1X_1 + ß_1X_1 + ß_1X_1$$

Where: Y = Dependent variable, $ß_0$= CONSTANT or intercept, ß1 = coefficients of the 1st independent variable, X1 = 1st independent variable, ß2 = coefficients of the 2nd independent variable, X2 = 2nd independent variable, $ß_3$ = coefficients of the 1st independent variable and X_3 = 3rd independent variable

Table 3 shows the model variables coding.

Table 3: Model variables coding

	MODEL 1	MODEL 2
Dependent variable (Y)	Motorcycle Accident Deaths	Motorcycle Accidents
First independent variable (X1)	Fatal Severity	Over speeding
Second independent variable (X2)	Serious Severity	Bad roads
Third independent variable (X3)	Minor Severity	Recklessness of Rider

MODEL 1

The multiple linear regression analysis was carried out for the 5 years before a general analysis cutting across the five years was done. The results are presented as follows.

Table 4 Model coefficients for 1st year

Model		Unstandardized Coefficients		t	Sig.	95.0% Confidence Interval for B	
		B	Std. Error			Lower Bound	Upper Bound
1	(Constant)	1.168	.670	1.743	.120	-.378	2.713
	X1	.837	.264	3.166	.013	.227	1.447

X2	1.038	.189	5.501	.001	.603	1.473
X3	1.203	.211	5.690	.000	.716	1.691

All the independent variables X's are statistically significant. Therefore, reject the null hypothesis.

Table 5: Model summary for 1st Year

Model	R	R Square	Change Statistics				
			R Square Change	F Change	df1	df2	Sig. F Change
1	.962[a]	.925	.925	32.918	3	8	.000

Tables 4 and 5show the results of the multiple regression analysis for the 1st year. Of the three independent variables, only Serious and Minor accidents had a significant influence on motorcycle accident deaths at the 95% confidence level with $P< 0.05$ while the variable Fatal accident was less significant because the $P> 0.05$.

Therefore, the model for the analysis is:

$$Y=1.168+0.837X1+1.038X2+1.203X3 \ldots\ldots\ldots 1$$

The equation implies that.

- Some factors affect the occurrence of deaths positively by 117%
- If an accident is fatal, the likelihood of death will be increased by 84%
- If an accident is serious, the likelihood of death will be increased by 103%
- If an accident is minor, the likelihood of death will be increased by 120%

The high R-value of 0.962 and R^2 value of 0.925 shows that there is a strong relationship between the dependent and the independent variables and that the dependent variable has been adequately explained by the independent variables.

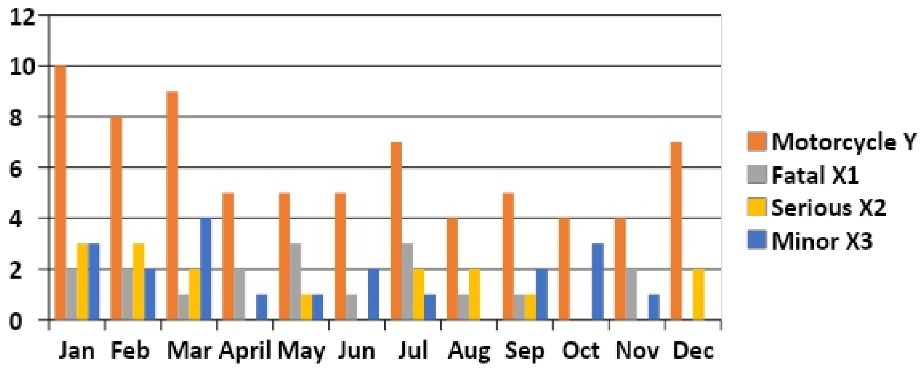

Figure 6: Bar chart for year 1

Model 1: results for 2nd to 5th Year

Similarly, to the analysis of year 1, the varying results of the 2nd to 5th year is summarized as follows.

Model 1: Year 2

$$Y=0.970+1.367X1+1.071X2+1.045X3 \ldots\ldots\ldots\ldots 2$$

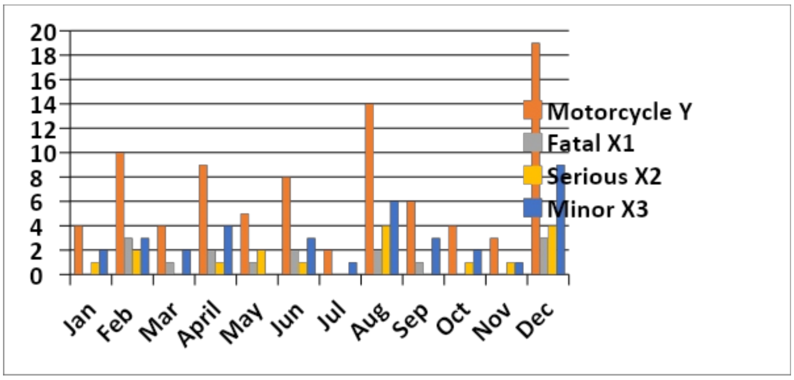

Figure 4.7: Bar chart for year 2

Model 1: Year 3

Y=0.588+1.746X1+1.090X2+0.867X3………….. 3

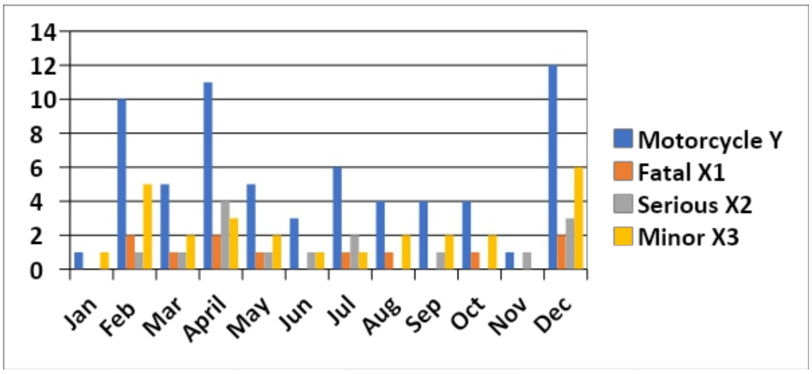

Figure 4.8: Bar chart for the year 3

Model 1: Year 4

Y=0.713+1.448X1+1.087X2+0.931X3 ………….. 4

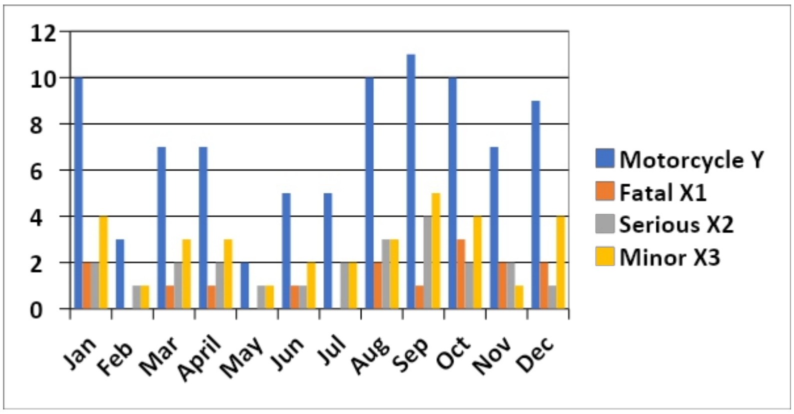

Figure 4.9: Bar chart for year 4

Model 1: year 5

Y=-0.057+1.430X1+0.887X2+1.235X3……….5

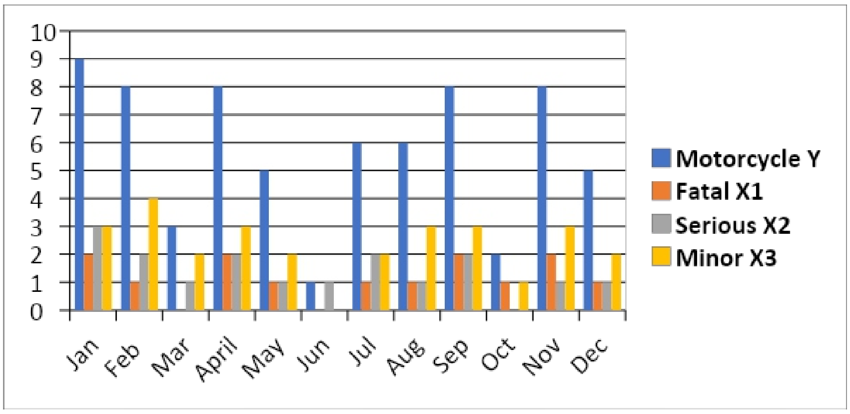

Figure 4.10: Bar chart for year 5

Conclusion: Model 1 year 2 to 5

The commutated result for model one covering 4 years is as follows;

$$Y=1.842+1.325X1+1.216X2+1.060X3 \ldots\ldots\ldots\ldots\ldots\ldots 6$$

Equation 6 implies that.

- There will be a 132% increase in the likelihood of death if a motorcycle accident is fatal.
- The likelihood of a death occurring from a motorcycle accident will be increased by 121% if the accident is serious and
- The likelihood of a death occurring from a motorcycle accident will be increased by 106% if the accident is minor.

MODEL 2

The result presented for Model 1 below is the product of responses from respondents (motorcycle riders) interviewed at various parks in Akure metropolis to determine factors influencing road crashes from the riders.

Table 6: Model coefficients 5 years

Model		Unstandardized Coefficients		t	Sig.	95.0% Confidence Interval for B	
		B	Std. Error			Lower Bound	Upper Bound
1	(Constant)	2.309	.089	14.777	.000	-.435	3.483
	over speeding	.707	.030	2.678	.001	.585	1.061
	Bad roads	1.560	.029	4.007	.000	.561	1.058
	Recklessness	.880	.029	2.282	.001	.205	1.006

Table 7: Model summary 5 years

Model	R	R Square	Change Statistics				
			R Square Change	F Change	df1	df2	Sig. F Change
1	.628	.739	.719	.548	3	396	.000

All the independent variables show a positive relationship with the dependent variable and are all statistically significant with $P<0.05$, so they are all included in the model resulting in the equation below:

$$Y=2.309+0.707X1+1.560X2+0.880X3 \ldots\ldots\ldots\ldots\ldots\ldots..7$$

From equation 7 it can be deduced that.

- A unit increase in over speeding will increase the likelihood of motorcycle accidents by 70%
- A unit increase in the use of bad roads will increase the likelihood of motorcycle accidents by 156% and

An increase in the recklessness of the driver will increase the likelihood of motorcycle accidents by 88%.

CONCLUSION

In the formulated models, Model 1 showed a strong significant relationship between the accident severity and motorcycle accident deaths, could be seen from the results that fatal severity, serious severity, and minor severity increased the likelihood of a motorcycle accident death by 132%, 121%, and 106% respectively. High R and R^2 values of 0.996 and 0.993 have shown that the model is a good fit for the selected variables. Model 2 showed a positively strong relationship between the dependent variable (Motorcycle accident Occurrence) and overspeeding, bad roads, and driver recklessness. The results have shown that overspeeding, bad roads, and driver recklessness increase motorcycle accidents by 70%, 156%, and 88% respectively. This model also displayed high R and R^2 values of 0.628 and 0.739 which shows that the relationship between the variables has been adequately explained. It is also instructive to note here that all the variables both in models 1 and 2 have been significant at the 95% confidence level with $P<0.005$. It can therefore be concluded with the high rate of influences, that over speeding, bad roads, and driver recklessness are major contributors to Motorcycle accidents in Akure.

From the results and discussion, it was observed that motorcycle accident occurrence was increased because of bad roads.

Furthermore, the impact of recklessness and over speeding in increasing motorcycle accidents can be reduced through the following safety measures.

- Strict enforcement of road laws that instructs the riders to undergo the comprehensive pre-riding course.
- It is also recommended that the relevant government traffic agencies should put more effort into educating motorcycle drivers through providing seminars and workshops concerning road regulations.
- More Research and Methodology (R&D) on a motorcycle in public transport should be conducted.
- There should be developed in the level of national security regarding over speeding and non-compliance of traffic rules associated with commercial motorcycles will be curtailed and the pedigree of genuine commercial motorcyclists is protected in Nigeria.

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ASSESSMENT OF THE IMPACT OF PROJECT COMPLEXITY ON CONSTRUCTION COST AND TIME PERFORMANCE

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ABSTRACT

Research in construction management has been skewed towards the causation factors of cost and time overruns with limited studies exploring the impact of project complexity on construction cost and time performance. This study examines the impact of project complexity on construction performance and whether the project overruns rise with complexity of the project.The objectives are to adapt a construction project complexity classification framework and test the hypotheses that cost, and time overruns are not impacted by the project complexity.Cost and time deviations were computed from 246 completed buildings,sourced through the administration of a structured questionnaire to a purposively selected construction professionals in the North-East geopolitical zone of Nigeria. Using an adapted framework into the Nigerian context, the projects were classified into less complex, moderately complex, and largely complex building projects. By comparing the group overrun means,the study found that the more complex a construction project is, the more the cost overrun, whereas time overrun decreases with higher complexities.Evidence of previous construction project team leaders' successes should therefore be the requirements in assigning contract and construction managers. This is recommended as measure for mitigating cost and schedule slippages caused by the complexity contents. The findings of this research on complex projects and cost overruns align with most many studies in other construction environments.

Keywords: Cost Performance, Largely Complex, Moderately Complex, Cost and Time Overruns, Less Complex.

1.0 Introduction

Current models on construction cost and time performance according to Jain & Singh (2012) lack predictions on how cost overruns behave over time regarding the frequency and magnitude. The authors further noted that earlier studies fail to address several important issues like how the cost overruns vary with the size and types of projects. Jain & Singh(2012) subsequently addressed these lapses in their cost overrun theory which states that cost overrun decline over time,adding that overrun is relatively high for procurement involving construction projects compared to procurement of finished products such as machinery within construction projects.Still in the theory, more complex projects experience higher cost overruns compared to less complex, and in contrast to the existing literature on incomplete contracts, an increase in the probability of renegotiation increases the asking price by the bidding contractors.

Shrestha *et al.* (2013) is one of the few studies that responds to the third part of the overrun theory propounded by Jain & Singh (2012),the authors focused on the magnitude of construction cost and schedule overruns in public work projects. Using 363 public construction projects, Shrestha *et al.* (2013) found that large, long-duration projects have significantly higher cost and time overruns than smaller short-duration projects. Apart from Randolph *et al.* (1987) and Odeck (2004) whose earlier discoveries contradict Shrestha *et al.'s* (2013) findings, other researchers aligning with Shrestha *et al.* (2013) are Warne (2005), Lee (2008), Hale *et al.* (2009). They foundthat cost escalations and schedule slippages occur more on more complex projects than inless complex projects. Odeck (2004) observed that overruns increased within a certain range of completion time, after which they decline until a completion time of about 200 Weeks and then fell. Odeck (2004) added that there is a relationship between the location of the projects and overruns noting that cost overrun is higher in projects in mountainous areas than projects located in the plains.

Used informally, complexity as a quality term is the ability to understand a system or an object, itbranches into simple or complex.In terms of quantity,complexity compares two or more items where one is more complex than the other (Standish, 2008). According to Bertelson (2014) the term comes up in science more often, but scientists themselves seem to be a bit in doubt, to the extent that one of the fathers of complexity studies Edward N. Lorenz (1917-2008) did not even include the term in his comprehensive book titled "The Essence of Chaos". The study of complex system is an emerging discipline that has not developed a theoretical framework on par with other theories (Federica *et al.* 2010). Complexity is not a new science, but a way of looking at systems; the studies are about the connection between things, not the things themselves (Lucas, 2000). The use of the term complex is often confused with complicated or difficult which is not necessarily so (Hale, 2018).Complex is the opposite of independent, while complicated is the opposite of simple (Lissack and Johan, 2000). The complexity research journal (Hindawi in partnership with John Wiley and Sons Inc.) emphasized that complex systems are characterized by interactions between their components that produce new information not in either the initial or boundary conditions thus limiting predictability of outcomes.

Scholars and practitioners in the building and construction industry are yet to understand much properly the term 'project complexity'. Project complexity, however, is a critical factor in project management that presents additional challenges to achieving project objectives of quality, cost, and time target (Al-Momani, 2000; Iyer & Jha, 2005; Koushki *et al*, 2005; Ogunsemi & Jagboro, 2006; Le-Hoai *et al*, 2008; Nega, 2008; Kaliba *et al*, 2009; Abdelgawad and Fayek, 2010; Okmen and Oztas, 2010;Olawale & Sun, 2010; Ameh & Osegbo, 2011; Azis *et al*, 2013; Kikwasi, 2012;Dao *et al*., 2016; Hale, 2018). Notwithstanding claim by Wood and Gidado (2008), Azim *et al.* (2010), Federica *et al.* (2010), Xia & Chan (2012) that the study of complex system is an emerging discipline that has not developed a theoretical framework on par with other theories, Kermanshachi *et al.*(2016)summarized existing definitionsof project complexity based on extant literature and discussions with experienced industry experts. They defined project complexity as the degree of interrelatedness between project attitudes and interfaces and their consequential impact on predictability and functionality. The authors used the definition to identify project complexity indicators and management strategies which eventually reduce the undesired outcomes often related to project complexity.

Hale (2018) complex projects are unique, characterized by a high degree of uncertainty which are known and unknowable interdependencies involving multiple teams and stakeholders. Generally, once a project reaches a critical size, timeframe, level of ambiguity and interconnectedness, it becomes complex making traditional management approaches deteriorate rapidly in effectiveness. Other indicators, the author stressed are interconnectedness and interdependent teams, heavy time constraints, extreme volatility, technology, cost, scheduling, political environment, global connectivity and speed of transmission, analysis, and availability of information at the beginning, dynamic visibility across planning, scheduling and execution, need for identification of contingency management plans and critical junctures. As stated earlier, there seems to be no universally accepted definition of the term "project complexity". In addition, no model has been proposed for the quantification of construction project complexity (Brockmann and Kahkonen, n.d). It is on this basis that Gidado (1996) interviewed a few experts and presented the results, saying that professionals see a complex construction project as characterized with the following:

- The project has many different systems that need to be put together and with many interfaces between elements.
- The project involves construction work on a confined site with difficulty of access and requiring many trades to work in proximity at the same time.
- One with a great deal of intricacy, for which it is difficult to specify clearly how long it would take.
- One which requires a lot of details about how it should be executed.
- One which requires efficient coordinating, control and monitoring from start to finish.
- One which requires a logical link, because a complex project usually encounters a series of revisions during construction and without interrelationships between activities it becomes very difficult to successfully update the Programme in the most efficient manner.

Memon *et al.* (2012) in Peninsular Malaysia classified large projects as projects with a minimum contract sum of 5 million Malaysian Ringgit (RM), Altshuler and Luberoff (2003) and Flyvbjerg*et al.* (2004) estimated the minimum contract sum as USD 250 million and USD 500 million. Flyvbjerg*et al.* (2004) argues that the definition of a mega project differs depending on its geographical setting, thus what might constitute a mega project in a more rural area might not be considered as such in a metropolis (Flyvbjerg*et al*, 2004). This argument is also upheld in Montequin *et al.* (2018). Complex projects are composed of multiple interrelated systems where changes in one system require unforeseen changes in the connected systems (Herszon & Keraminiyage, 2014; Shenhar & Dvir, 2007). Though there appears to be a strong indication that complex projects using the construction cost size yield higher cost overruns, Merrow *et al.* (1988) and Hinze *et al.* (1991) stated that the absolute value of cost overruns and schedule slippage increase with the size of projects, putting very large sums of money at risk. Thus, project complexity may cause poor project success.Many researchers as Chen *et al.* (2004) and Bosch-Rekveldt *et al.* (2011), have supported Baccarini's (1996) and Merrow *et al.'s*(1988), view that project success is dependent on the complexity, having a direct effect on the overall project performance.Le-Hoai *et al.* (2008), Nega (2008), Abdelgawad & Fayek (2010), Okmen & Oztas (2010), Olawale & Sun (2010), Ameh & Osegbo (2011), Kikwasi (2012), Azis *et al.* (2013) identified complexity and size of work as impact factor(s) on the construction project cost and time performance. Moreover, heavy time constraints and cost are among the construction project characteristics or indicators listed by Hale (2018).

1.2 Justification of the Study

Unlike other scholars who focused on overrun factors, this study focuses on addressing the issue raised in the latter part of the theory of cost overrun by assessing whether project complexity impact on the cost and time performances. The results will enhance actions among project managers in the control of overruns in less complex projects, moderately complex and largely complex construction projects.In line with the objectives of this study and unlike other scholars who focused on cost overrun factors, they address the last issue raised in the theory of cost overrun by assessing the differences in cost and time performances impacted by the degree of complexity in the construction projects in North-Eastern Nigeria.Primary and secondary data on completed building projects were therefore, sourced from the study area to test the following hypotheses which emerged from the stated research questions.

1.2 Aim and Objectives of the Study

The aim of this study was to assess the impact of project complexity on construction cost and time performance using the Nigeria's North-East geopolitical zone as the study area. The objectives are to adapt a construction project complexity classification framework and test the hypotheses that cost, and time overruns are not impacted by the project complexity.

1.3 Research Hypotheses

$(H_0)_1$: *Thecomplexity ofconstruction project do not impact on thecost performance*

$(H_0)_2$: *The complexity of construction project do not impact on the timeperformance*

1.4 Study Area

Nigeria's North East geopolitical zone comprising six states - Adamawa, Bauchi, Borno, Gombe, Taraba and Yobe. The investigation was situated in the locations randomly selected as representative of continuous construction activities in the geopolitical zone. Public establishments in the study area include secondary and tertiary institutions of learning, military and paramilitary, primary and tertiary health institutions, local, state and federal government ministries. The study population is building projects completed between 2012 and 2017. The survey was conducted between April and October 2017.

2.0 Methodology

2.1 Questionnaire Design

The study population units were completed buildings. Due to the dearth of databases and case studies and limited access, where available in Nigeria, a questionnaire survey was adopted as an alternative. This involved sourcing data directly from professionals who played active roles in the project execution. The questionnaire was therefore designed to extract historical data on completed construction projects. Section one of questionnaire asked question on participant's personal information. Participants were asked in section two to identify a construction project that they had participated activelyin the last six years (2012-2017), and provide information on the building, its owner, initial contract sum, final cost, estimated construction duration and actual construction duration.The overruns were computed. These are differences between the contract sum and actual construction cost, time overrun the difference between the estimated and actual construction duration.

2.2 SampleSize Determination andSampling Method

Sample size (SS) for the primary study was determined considering; sample size of similar past studies, study population size, purpose of study, level of precision, level of confidence and the degree of variability in the attributes being measured (Miaoulis and Michener, 1976) and survey response rate in construction industry research (Idrus and Newman, 2002). The study explored four criteria for determining the sample size (SS) using Equation (1) (Israel, 2003).

$$SS = \frac{Z^2 x\, p\, x\, [1-\rho]}{e^2} \qquad (1)$$

whereZ is the statistical value for the desired confidence level 1.96 found in the statistical tables, ρ is the value of the population proportion, which is being estimated, and
e (0.05) denotes the desired level of precision.

The value of p for unknown population size, taking a conservative value of 0.50, to ensure that a sample size as large as required is obtained. At 95% confidence level for large and unknown population size, the SS for this study is approximated to 384. Secondly, using the Online Creative Research System (1982) sample size calculator for a large and unknown population size at 95% confidence level and 6 % confidence interval, a sample size of 176 was estimated for the questionnaire survey. Thirdly, following earlier and similar studies for example, Chen & Hartman (2000), Emsley *et al.* (2002), Kim *et al.* (2004), Maghraby (2009), Al-Zwainy *et al.* (2012), Aibinu *et al.* (2015), Najafi & Kong (2016) and Fachrurrazi & Munirwansyah (2017) in which data sets of 80, 288, 530, 80, 150, 71, 220 and 40 respectively, the highest data set (530) of Kim *et al.* (2004) was used and combined with the 30% upper bound of Idrus and Newman's (2002) (20% to 30% adequate percentage questionnaire return in the construction industry), the sample size (E) was estimated as, 30/100×SS = 530, therefore SS = 1766.67, which was approximated to a sample size of 1800 cases for the study.

Like Gebrehiwet & Luo (2017), purposive sampling technique was used in collecting the primary data. Secondary data on buildings completed, comprising contract sum and final cost, estimated actual construction durations were obtained from the archives of construction professionals who participated in the survey. This choice of one respondent per project was premised on the fact that site diaries are kept on construction project sites, by representatives of the main contractor and consultants such as the site engineer' and the 'clerk of works' who manage and keep daily records of site operations.

2.3 QuestionnaireDistributio n and Response

The research consent form and samples of the questionnaire were first e-mailed to participants in management positions of some selected public establishments in Bauchi, Damaturu, Gombe, Jalingo, Maiduguri, Mubi and Yola. Others were self-administered to respondents in their offices in the state capitals and to the conference participants at the August 2017 Annual General Meeting/Conference of the Nigerian Institute of Building (NIOB) that held in Bauchi, a state capital in the study area. 1800 questionnaires were distributed (see Table 1). 294 questionnaires (see Table 2) based on projects in the study area (North-East Geopolitical zone) was collated from the total which is 16.33 %. The 294 projects (16.33 %),retrieved for the study area was further separated into, Adamawa, Bauchi, Gombe and Taraba states. Adamawa state was further split into Adamawa North and Adamawa South to achieve similarity in the grouping (largest group/smallest group ratio ≤ 1.50) (Stevens, 1996). The incomplete or incorrectly filled questionnaires amounted to 45 or (2.50%) of the quantity distributed (see Table 2).

Idrus & Newman's (2002) consider a range 20% to 30% response rate adequate for researches in construction management since it is difficult to obtain a high level of response with questionnaire survey in the construction industry (Ankrah, 2007). Chew *et al.* (2008) in China recorded a slightly lower percentage of 13.30% (133/1000) on the core capability and competitive strategy for construction SMEs, while Tan *et al.* (2012) in Hong Kong had a slightly higher response rate of 19.60% (61/312). The research sample 249 as checked with the online sample size calculator (1982) of the creative research systems at 95% confidence level and a range of 5%, 6% and 7% confidence interval for unknown population size (number of completed buildings) results in sample sizes of 384, 267 and 196 respectively. The returned sample of 249 lies in between the lower and upper bounds of the computed and between the 18 and 530 datasets other researchers had used in previous and similar studies, it was, therefore, adjudged adequate for analysis.

2.4 One-Way Analysis of Variance

In this research, the interest is in comparing the mean scores of three groups of complex projects i.e.,to see the steady rises or falls with the means per group of complexity. One-way analysis of variance (ANOVA) was used in this study. It involves one independent variable (overrun) which has many groups (conditions of complexity). Analysis of variance used because it compares the variance (variability in the means) between the different groups (due to project complexity) with the variability within each of the groups (due to chance).

The 246-dataset haven satisfied the assumptions one-way analysis of variance were used in the analysis after further cleansing from outliers. The variance in their cost and time means caused by complexities in the projects were compared using IBM SPSS statistics version 21 one-way between groups ANOVA with a post-hoc test.The means for each group are given in the descriptivetable. The ANOVA table gives between-groups, within-groupsand total sums of squares. The main interests in line with the research objectiveare the means plots that provided an easy way to compare the mean scores for the different groups.

2.4.1 Effect Size

One way that can be used to assess the importance of results of the compared means is to calculate the 'effect size' (also known as 'strength of association between the groups'). This is a statistic that indicates the relative magnitude of the differences between means, or the amount of the total variance in the dependent variable (overruns) that is predictable from knowledge of the levels of the independent variable (complexity) (Tabachnick and Fidell 2007, p. 54). The most used effect size statistic to compare groups are partial eta squared and Cohen's d. Although SPSS does not generate effect size in the analysis, it is possible to determine the effect size for the result usingthe information provided in the ANOVA table to calculate Eta squared using Equation (2) (Pallant, 2010).

$$\text{Eta squared} = \frac{\text{Sum of squares between groups}}{\text{Total sum of squares}} \quad (2)$$

Eta squared values can range from 0 to 1. Large, medium, and small sizes are indicated by 0.138 or 13.8%, 0.06 or 6% and 0.01 or 1% respectively (Tabachnick and Fidell 2007, p. 55).Cohen's d, on the other hand, presents difference between groups in terms of standard deviation units.

2.5 Complexity Framework Adapted in the Study

Currently there is not yet a comprehensive generally accepted definition of complexity (Cooke-Davies *et al*., 2011) nor of a complex construction project (Mabumbulu, 2016). However,the classification adopted in this study is based on the minimum construction cost of \$250 million (Altshuler and Luberoff, 2003) which aligns with Hass' (2016) classification of moderately complex projects, though the classes differ in the construction duration classifications. Altshuler and Luberoff (2003) aligns with Randolph *et al.*'s (1987) who is silent on construction duration. Lack of contextual classification in a developing economy like Nigeria, informs the adoption of Altshuler & Luberoff's (2003) \$250 million as minimum contract sum with four years minimum duration for largely complex construction projects. This aligns with Randolph *et al.* (1987). The translations of Altshuler &Luberoff's (2003) of the United States of America Dollar to the Nigeria Naira in 2018, the year of the study for purpose of classifying the field data was achieved first by converting the 2003 complex construction classification cost bid to total square metre using the [US-DHUDOPDROH] (2005) construction cost per square of \$1077.33/m^2.

A complexity framework was then derived and adopted in this study. \$50 x 10^6 gives 46,411.03m^2i.e. [\$50 x 10^6/\$1077.33/m^2] as upper bound for less complex and 232,055.17m^2i.e. [\$250 x 10^6/\$1077.33/m^2] as the minimum bid cost for largely complex projects. To cater for inflationary rate, consumer price indices (CPIs) which are 45.70 and 14.33 for 2003 and 2018 respectively (National Bureau of Statistics [NBS], 2018) are applied on the Nigerian ₦35, 000.00/m^2 (Windapo, 2005). The conversion process is as follows.

X/35,000.00 = 45.70/14.33

= ₦111, 618.98/m^2 for a year of study in 2018

Thus \$50 x 10^6 = ₦111, 618.98/m^2 x 46411.03m^2

= ₦5.18billion upperbound for less complex *construction projects.*

And \$250 x 10^6 = N111, 618.98/m^2 x 232055.17m^2

= ₦25.90billion lower bound for largely complex construction projects

The complexity framework discussed above was used to classify the surveyed projects. The projects were grouped into categories of complexity shown in the fourth column of Table 4 which translates the Altshuler and Luberoff's (2003), Randolph *et al.* (1987) in United States of America Dollars (\$million) to the 2018 Nigeria version. These are three complexity groups; less complex, medium or moderately complex and largely complex construction projects(see Table 4). The initial contract sums, the estimated construction duration of the identified three groups of complexities as well as their overrunsare analyzed in subsequent sections of this paper for the determination of the impacts of complexity on the out-turn construction project cost and time.

3.0 Results and Discussions

Details of respondents' professions and membership of professional associations, grouping of respondents in stakeholder category and industrial experience are presented in the Table 3. The research participants were construction industry professionals who are, 45.53% registered builders, 25.21% mechanical, electrical, civil, and structural engineers, 21% architects, and 8% quantity surveyors. The respondents are qualified members of their respective professional associations, with satisfactory post-qualification experience, engaged in construction project activities at the time of the survey. They were also employed by stakeholders to the construction industry at the time of the study. Therefore, the information they provided is adjudged reliable for use in this study.

3.1 Impact of Complexity on Construction Project Performance

Complexity impact on construction cost performanceof selected public building projects in the study areawas tested. The three groups of complexity classifications; less complex, medium, or moderately and largely complex construction projects labelled 1, 2 and 3. The following hypothesis were tested, while the results are presented in Figures 1, 2 and Tables 5 and 6.

3.1.1Cost Performance

(H_0)$_1$*: The complexities of construction project do not impact on the cost performance.*

The 246 projects comprised 206 less complex, 30 medium or moderately complex and 10 largely complex constructions. The comparison of the group means of overruns impacted by complexity on the contract sum and estimated construction durations are shown in the SPSS ANOVA in Appendices I and II. The mean cost performance scores of the three groups of complex projects aremean (M) = 47.78,standard deviation (SD) = 63.27 for less complex, M = 36.13, SD = 31.15 for moderately complex and M = 118.90, SD = 67.60 for largely complex projects. As can be seen in Figure1 the curve rises from left to right, meaning that cost overrun rises with the degree of project complexity, though cost impacts initially fall steadily from less complex group to the medium or moderately complex projects, before rising steadily upwards to largely complex projects. The Eta squared found is 0.057 or 5.7%, implying a mediumeffect size.

The suggestions that cost overrun rise with increase in project complexity was first drawn from Merrow *et al.* (1988), supported in Ugulu and Ikwuogu (2011) and recently in (Olaniran *et al*, 2015). However, Randolph *et al.* (1987) and Odeck (2004) observed that cost overrun is inversely correlated with the size of the project, while Jahren and Ashe (1990) found the opposite.

3.1.2Time Performance

Complexity impact on construction time performance of selected public building projects in the study area is tested in this section. The three groups of complexity classifications; less complex, medium, or moderately and largely complex construction projects are labelled 1, 2 and 3 respectively. The estimated construction duration and time overrun data from the three groups of complex projects are tested on the following hypothesis, while the results are presented in Figure 2, Appendices I and II.

$(H_0)_2$: The complexity of construction project do not impact on the time performance.

A total number of 246 completed construction projects were investigated for differentials in the cost and time performance caused by the degrees of complexities in the projects. The projects as earlier stated comprised 206 less complex, 30 medium or moderately complex and 10 largely complex construction projects. The mean time performance scores of the three classes of complexity shown in Appendices I and II are mean (M) = 60.79, standard deviation (SD) = 78.71 for less complex, M = 40.93, SD = 41.41 for moderately complex and M = 36.50, SD = 40.95 for largely complex projects. As shown in Figure 2 the mean of time performance impact falls steadily from less complex construction to largely complex projects. the implications are the more complex a project the less it attracts slippages on the scheduled programme of work in the study area. The Eta squared is computed from information in Table 6 is 0.011 or 1.1% implying a small effect size.

Although this finding is contrary to Shah's (2016) findings in Ghana as well as Merrow *et al.*'s (1988) assertion that schedule slippages rise with project sizes or cost, this finding may imply that large or complex construction project originally are designed with long construction programmes which are more flexible in tracking. Therefore, less time overruns are experienced on largely complex construction projects given thatmore scientific project management strategies are employed by big time foreign contractors engaged normally in Nigeria.

4.0 Conclusions

The paper examined the impact of project complexity on construction cost and time performance The study was motivated to confirm or otherwise a part of the challenges raised in cost overrun theory by an earlier study.The theory states that the more complex construction projects are, thehigher the cost and time overruns they incur. In the literature, the study found that, researchers are yet to focus on the human and machinery resource capacity aspects of construction project complexity which content are embedded in the architectural, services and structural engineering designs of construction projects. These comprise the number, calibre and quality (education and health) of personnel commensurate with the designs and the expected product quality. Primary and secondary data were sourced on 246 completed construction projects in the study area which were grouped into three classes of less complex, moderately complex and largely complex projects. An American based classifications was adapted to suit the Nigerian context since there are no known locally based framework for classifying complexity in construction projects. Hypotheses emerging from the research questions were tested with descriptive statistics. It was found that cost overrun increases with complex nature of the construction project, while the converse was found for time overrun impacted by the project complexity.

The application of the study results in future construction project resource planning, organization and control shall aid construction planners and managers in mitigating construction cost overruns at pre-contract and contract stages.

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LIST OF TABLES, AND FIGURES

Table 1: Questionnaire distribution

Study area & conference Centre	Distribution method		Number distributed and % of total distributed	
	Self-administration across the study area	Electronic mailing	N	%
Adamawa State	322	29	351	19.50

Bauchi State	181	21	202	11.22
Borno State	0	9	9	0.50
Gombe State	157	12	169	9.39
Taraba State	240	9	249	13.83
Yobe State	0	13	13	0.70
Self-administration at Bauchi 2017			807	44.83
	Total number distributed		1800	100.00

Table 2: Research participants' response

Location	Number returned (N)	Percentage of total number distributed (%)	Invalid number & percentage of number distributed		Valid number & percentage of number distributed	
			No	%	No	(%)
Adamawa North	69	3.83	18	1.00	51	2.83
Adamawa South	68	3.78	7	0.39	61	3.39
Bauchi State	62	3.44	15	0.83	47	2.61
Gombe State	46	2.56	0	0.00	46	2.56
Taraba State	49	2.72	5	0.28	44	2.44
Total	294	16.33	45	2.50	249	13.83

Table 3: Background Details of the Respondents

Background Details	Frequency	Percentage (%)
Profession		
Builder	112	45,53
Architect	52	21,14
Mechanical& Electrical Engineer	32	13,01
Civil/Structural engineer	30	12,20
Quantity Surveying	20	8,13
Total	246	100%
Membership of Professional Association		
Member Nigerian Institute of Building	107	43,50
Member Nigerian Society of Engineers	55	22,36
Member Nigerian Institute of Architect	52	21,14
Member Nigerian Institute of Quantity Surveyor	18	7,32

Graduate Members	14	5,69
Total	246	100%

Stake holder Category		
Main contractor	90	36,59
Project Consultants	85	34,55
Client's in-house project team	54	21,95
Sub-contractor	17	6,91
Total	246	100%

Post Qualification Experience		
Up to 5 years	42	17,07
6-10 years	81	32,93
11-15 years	46	18,70
16-20 years	23	9,35
Over 20 years	54	21,95
Total	246	100%

Table 4: Altshuler and Luberoff (2003) construction project complexity classification translations to the 2018 Nigeria Naira value

Project Complexity Group	Conversions to 2018 Nigeria Naira (N billion)	**Altshuler and Luberoff's** (2003) and Randolph ***et al.*** (1987) in United States of America project size Dollar ($million)	No of projects per class
Less complex (1)	< 5.18	< 50	206
Medium or moderately complex (2)	5.18 ↔25.90	50 ↔ 250	30
Largely complex (3)	25.90 >	250 >	10
Total			246

Table 5: Descriptive data for construction projects complexities cost and time performances

		N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean: Lower Bound	95% Confidence Interval for Mean: Upper Bound	Minimum	Maximum
Percentage cost overrun (%)	1	206	47.7816	63.26920	4.40817	39.0904	56.4727	-20.00	439.00
	2	30	36.1333	31.15006	5.68720	24.5017	47.7650	1.00	100.00

	3	10	118.9000	67.59758	21.37623	70.5436	167.256 4	20.00	207.00
	Total	246	49.2520	62.07285	3.95762	41.4567	57.0473	-20.00	439.00
Percentage time overrun (%)	1	206	60.7864	78.71042	5.48401	49.9741	71.5987	-33.00	500.00
	2	30	40.9333	41.41042	7.56047	25.4704	56.3962	-50.00	121.00
	3	10	36.5000	40.94780	12.94883	7.2077	65.7923	-20.00	127.00
	Total	246	57.3780	74.22327	4.73230	48.0569	66.6992	-50.00	500.00

Table 6: Analyses of variance (ANOVA) for construction project complexities on cost and time performance.

		Sum of Squares	df	Mean Square	F	Sig.
Percentage cost overrun (%)	Between Groups	54116.837	2	27058.419	7.389	.001
	Within Groups	889877.537	243	3662.047		
	Total	943994.374	245			
Percentage time overrun (%)	Between Groups	14864.873	2	7432.436	1.353	.260
	Within Groups	1334862.969	243	5493.263		
	Total	1349727.841	245			

Table 7: Template for assessing dimensions of complexity in construction projects

No	The Project (for instance)	Cost dimensions		Time dimensions		Plant and Equipment content		Other dimensions of complexity (energy, mental and creativity, technology)				
		Initial Contract Sum	Project Final Cost	Estimated Constn. Duratn.	Actual Constn. Duratn.	Heavy duty	Light mechanical hand-tools	Creativity or Innovative potentials	Technical dimension	General health dimensions of the construction site operatives	General Health dimensions of the sectional team leaders	Mental health of the building team overall leader
1	Pakistan Hydro-Electric Project											
2	Canadian James Bay											
3	Trans Alaska Pipeline System											
4	Argentina Centro-Oeste project											
5	Columbia Correjon											
6	Stratford platform in North Sea											
7	Australia Cooper Basin Project											
8	Papua New Guinea's huge copper and gold mining											

	complexes											
9	Synthetic fuel plant, South Africa											
10	Denver International Airport	$1.70billion	$4.5billion									

………….

Signed

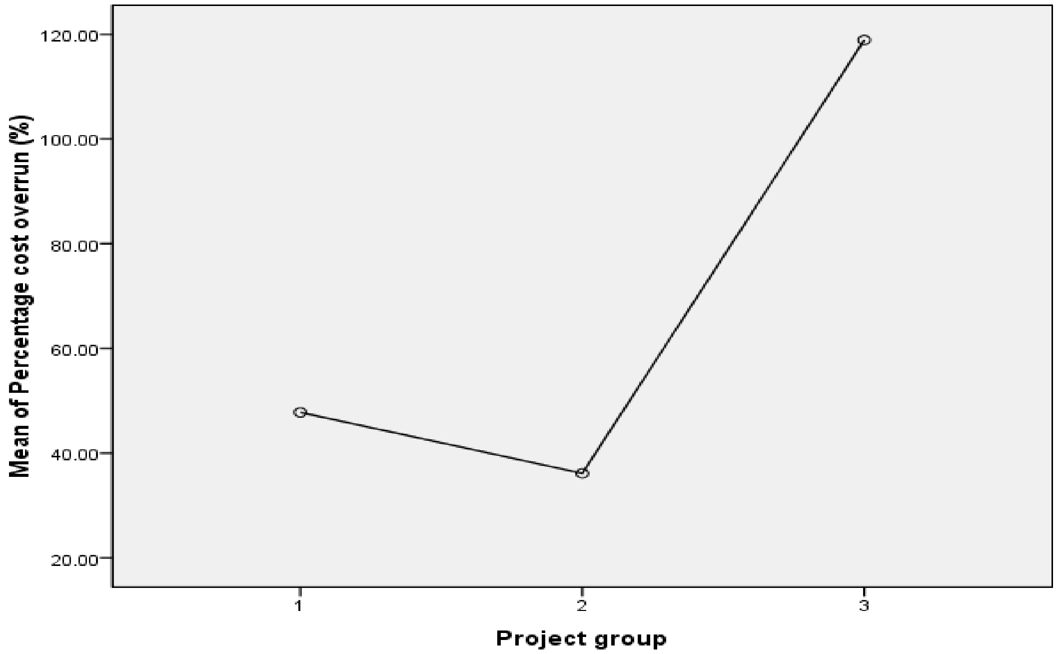

Figure 1: Project complexities mean percentage cost overrun plot

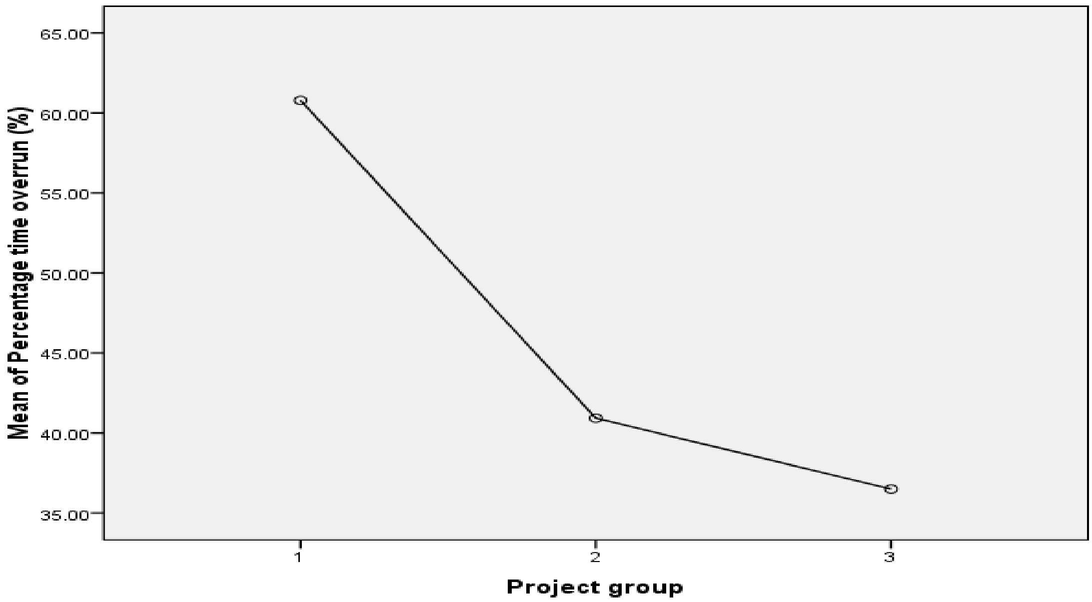

Figure 2: Project complexities mean percentage time overrun plot

EFFECT OF PARTIAL REPLACEMENT OF CEMENT WITH PALM KERNEL SHELL ASH ON COMPRESSIVE STRENGTH OF CONCRETE

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ABSTRACT

Partial replacement of cement by agricultural biomass/waste products is one of the effective means of reducing the cost of construction materials. The study presents the effect of partial replacement of cement with palm kernel shell ash on the compressive strength of concrete. The test procedures carried out in accordance with British Standard Institution guide. The palm kernel shell was obtained by burning palm kernel shell at 900^{0}C for 3 hours. The cement was replaced by palm kernel shell ash at 5, 10 and 15%. The optimum compressive strength of concrete at 28 days curing is 26.41 N/mm^2, which is higher than that of 7, 14 and 21 days. The maximum compressive strength were obtained to be 18.60 N/mm^2, 19.42 N/mm^2, and 20.20 N/mm^2 at 7, 14 and 21 curing respectively. Therefore, the study revealed that the compressive strength of concrete increases with curing age and decreases with increasing percentage of palm kernel shell ash replacement beyond 15%.

Keywords: Cement; Concrete; Compressive Strength; Palm kernel shell ash; Replacement.

1. INTRODUCTION

There has been so much demand in construction industries on the need for construction materials in many countries around the world. Efforts have equally been made by various researchers to reduce the cost of concrete and hence total construction cost by investigating and ascertaining the usefulness of material which would be classified as agricultural and industrial wastes. The adoption of new materials in today's construction market is the result of resource constraint, advances in engineering techniques and cost saving measures. (Tangchirapat, 2009).

The world in general, has turned its focus on environmental effects associated with improper disposal of waste materials which results in excessive accumulation of dirts and pollution. Millions of tons of palm kernel shell are constantly being dumped in the environment through careless disposal and they are mostly resistant to degradability which makes it a problem to the environment (Basri, 1999).

As a result of limited usage of these wastes materials, the rate at which they are disposed as landfill materials are expected to increase consequently leading to potential failure, environmental problems, accumulation, burning and landfill of solid waste disposal which can be expensive and undesirable. When these materials are reused in workable areas such as in the construction industry it is considered as an active area over the entire world which is a current practice (Olowe and Adebayo 2015).

Due to the increase in demand for more building materials and failure of structures (buildings) in recent time across the country (Nigeria), measures has been set up to proffer solutions to this problem especially using locally sourced materials. Cement is one of the vital material in the construction industry therefore its specification must be strictly adhered to. But as a result of the increase in cost of cement which mostly deters construction activities, an alternative choice of Palm Kernel Shell Ash (PKSA) which is likely to be a waste in the environment is being explored towards reduction in construction cost (Joshua et al., 2015). Hence this work is geared towards the possibility of the use of palm kernel shell ash as partial replacement for cement in concrete production.

The word Cement is translated from the Latin word "cementum'' which means the stone and marble powder which is used as binding material for blocks at the time of the Romans. In the European language, the word "cement'' is named as any organic and inorganic binder such as gum, gels, plastics, welding and asphalt alloy and aqueous cement. Cement is an important construction material and an essential binder in the construction industry globally. Its production relies heavily on continuous exploration of non-renewable natural resources leading to depletion of such resources while it contributes about 5% to 7% of total gaseous anthropogenic emission to the atmosphere (Chen *et al.,* 2010).

The palm kernel shell (PKS) is a waste material obtained during the extraction of palm oil by crushing the palm nut in the palm oil mills. They are hard, flaky and of irregular shape. These wastes if properly pulverised has cementitious properties hence making it pozzolanic (Awal and Hussin, 2011). The recycling of these waste into value added products in construction applications will reduce demand on non-renewable natural resources which are fast depleting as well as scarce and costly coupled with the energy required in processing them. This also will further enhance local material research, development, production, utilisation and improvement which will enhance a long term economy by adequately enhancing a cleaner environment and achieving concrete performances with physical tests (Neville, 2011).

2.0 MATERIALS AND METHODS

2.1 Materials

The materials (raw) required for these research work includes: Dangote brand of Ordinary Portland cement (OPC), fine aggregate, coarse aggregate, palm kernel shell ash (PKSA) and clean water. The palm kernel shells used in these research were obtained at Umomi in Kogi State. The Ordinary Portland cement and aggregates were obtained at Albashiri quarry site along Bida – Minna road. The palm kernel shell was burnt in an incinerator using a fabricated furnace for about 3 hours to obtain the ash behind the Civil Engineering Laboratory, Federal University of Technology Minna. After which the ash was sieved using sieve 75um to obtain the required fineness as that of cement. Tap water free from contaminants was obtained from Civil Engineering Laboratory, Federal University of Technology, Minna, and was used for mixing and also curing of the concrete.

2.2 Methods

The production of concrete tests was conducted in Civil Engineering Laboratory, Federal University of Technology, Minna. The materials mentioned in 2.1 above were used, prescribed mix design proportion of 1:2:4 with water cement ratio of 0.6. A total of 48 concrete cubes specimen (150mm X 150mm X 150mm) were cast according to (BS 1881: part 108, 1983), cured according to (BS1881: part 111, 1983) and tested according to (BS 1881: part 116, 1983) at the curing ages of 7, 14, 21, and 28 days respectively.

Tests including sieve analysis according to (BS 812: part 103.1, 1985), specific gravity according to (BS 812: part 107, 1995), bulk density according to (BS 812: part 108, 1995), aggregate impact value test according to (BS 812: part 2, 1995), water absorption test according to(BS 812: part 107, 1995), slump test according to (BS 1881: part 102, 1983) and finally the compressive strength test according to (BS 1881: part 116, 1983) after curing for 7, 14, 21, and 28 days were carried out.

2.2.1 Casting of concrete cubes

After concrete mixing, slump test precedes casting of concrete cubes. The concrete mould of 150mm × 150mm × 150mm dimensions was used. The moulds were rubbed with engine oil so as to allow easy removal of the sample when de-moulding. The moulds were placed on a rigid horizontal surface and filled with concrete in such a way as to remove entrapped air as possible and produce full compaction of the concrete with neither segregation nor laitance. The concrete was poured inside the mould in three layers; each layer being given 25 strokes of the 16mm tamping rod. Each layer is of approximately 50mm deep. The test cube was prepared in accordance to (BS 1881: part 108, 1983).

2.2.2 Curing of concrete cube

Curing follows immediately after de-moulding of the cubes from the mould. The cubes will be submerged immediately in the curing tank for the required curing age of 7, 14, 21, and 28 days which are the ages to be considered for the purpose of this study. The curing of the cubes was carried out in accordance to (BS 1881: part 111, 1983).

2.2.3 Compressive strength test

After curing the concrete specimen, crushing is done. Crushing operation was performed on concrete cubes by applying compressive force on them gradually until the cubes starts cracking having attained its supposed maximum strength limit in a compressive strength testing machine. Compressive strength test was carried out on the concrete cubes at curing age 7, 14, 21, and 28 days respectively, in accordance to (BS 116: part 116, 1963).

Figure 1: Samples after demoulding

Figure 2: Sample undergoingcompressive strength test

Figure 3: Sample undergoing slump test

3.0 RESULTS AND DISCUSSION

The sieve analysis test was carried out on aggregates and the fineness modulus of fine aggregate was calculated and obtained to be 2.60 which conforms with the requirement that aggregate fineness modulus must fall within the range of 2.3-3.1.The specific gravity for the aggregates were obtained as 2.66 and 2.66 for fine and coarse aggregate respectively which falls within the standard range of specific gravity 2.5 – 3.0. The specific gravity of the palm kernel shell ash was obtained as 2.34 which is lesser compared to that of ordinary Portland cement of 3.15 as reviewed in the literature from previous researches and duly referenced. The bulk densities of the material were found to be 1534.11 kg/m^3 and 1660.82 kg/m^3 for un-compacted and compacted fine aggregates respectively, likewise 1481.48 kg/m^3 and 1656.92 kg/m^3 for un-compacted and compacted coarse aggregates which conforms with the standard range of (1500-1700) kg/m^3 and (1300-1800) kg/m^3 for fine and coarse aggregate respectively. Percentage porosity of fine aggregate and coarse aggregate was found to be 7.63 and 10.59 % respectively, and void ratio 0.42 and 0.44 %.

The water absorption was found to be 24.60 % for fine aggregate, 2.8 % for coarse aggregate and 73.24 % for palm kernel shell ash. The aggregate impact value was gotten as 24.40 %.

Slump test was also carried out to determine the workability/consistency of the fresh concrete.

The compressive strength of the cubes after 7 days of curing age with 0 % having the highest compressive strength of 18.60 N/mm^2 followed by 5, 10 and 15 % obtained as 16.02 N/mm^2, 12.21 N/mm^2, and 10.95 N/mm^2 respectively.

The compressive strength for 0, 5, 10 and 15 %, for 14 days curing age and the compressive strength increased than that of 7 days curing ages, the compressive strength obtained are 19.42 N/mm^2, 17.81 N/mm^2, 14.77 N/mm^2 and 11.69 N/mm^2 respectively.

The compressive strength for 0, 5, 10 and 15 %, for 21 days curing age and the compressive strength increased than that of 7 days curing ages and 14 days curing ages, the compressive strength obtained are 20.20 N/mm^2, 19.33 N/mm^2, 15.10 N/mm^2 and 12.29 N/mm^2 respectively.

The maximum compressive strength for 0, 5, 10 and 15 %, at 28 days of curing ages with the compressive strength which is higher than that of 7, 14 and 21 days curing ages, the compressive strength obtained are 26.41 N/mm^2, 20.07 N/mm^2, 16 N/mm^2 and 13.81 N/mm^2 respectively. Figure 5 shows the graph of compressive strength against curing age of concrete specimen.

Table 1. Sieve Analysis of Fine Aggregate (Sand)

S/ No	Sieve sizes (mm)	Weight retaine d (g)	Cumulative weight retained (g)	Cumulative percentage retained (%)	Cumulative Percentage passing (%)
1	5.00	0.21	0.21	0.042	99.96
2	3.35	8.86	9.07	1.814	98.19
3	2.36	27.75	36.82	7.364	92.64
4	2.00	13.72	50.54	10.108	89.89
5	1.18	62.68	113.22	22.644	77.36
6	0.85	57.75	170.97	34.194	65.81
7	0.60	74.93	245.9	49.180	50.82
8	0.43	93.33	339.23	67.846	32.15
9	0.30	79.08	418.31	83.662	16.34
10	0.15	66.76	485.07	97.014	2.99
11	0.08	12.01	497.08	99.416	0.58
12	0.00	2.92	500.00	100.000	0.00

Total weight:500g

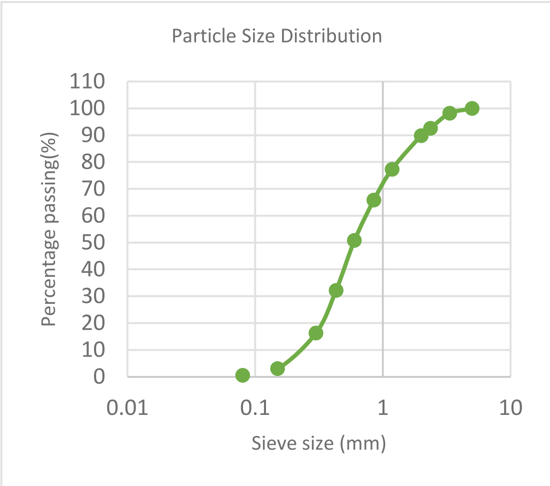

Figure 3: Particle SizeDistribution of Fine Aggregate

Table 2. Sieve Analysis of Fine Aggregate (Sand)

S/No	Sieve sizes (mm)	Weight retained (g)	Cumulative weight retained (g)	Cumulative percentage retained (%)	Cumulative percentage passing (%)
1	20..0	2.50	2.50	50.00	50.00
2	14.00	1.80	4.30	86.00	14.00
3	10.00	0.60	4.90	98.00	2.00
4	6.30	0.10	5.00	100.00	0.00
5	0.00	0.00	5.00	0.00	0.00

Total weight:5kg

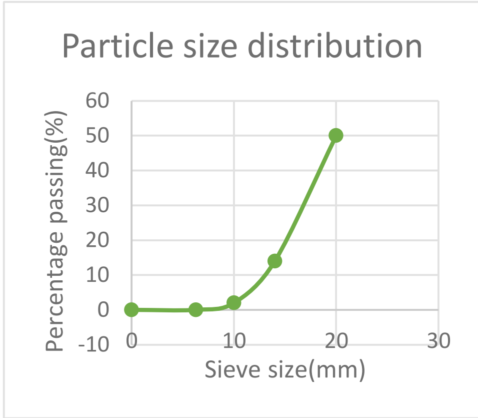

Figure 4: Particle Size Distribution of Coarse Aggregate

Table 3. Specific Gravity of PKSA

Trials	1	2	3
Weight of empty vessel	100.8	100.5	100.6
Weight of sample (g)	106	105	105.4
Weight of vessel + sample + water (B)(g)	216.4	214.1	215.2
Weight of vessel + water only (C) (g)	213.2	211.6	212.6
Specific gravity Gs	2.60	2.25	2.18
Average specific gravity		2.34	

Table 4. Slump Test

(%) PKSA	Slump Value (mm)
0%	46
5%	32
10%	39
15%	56

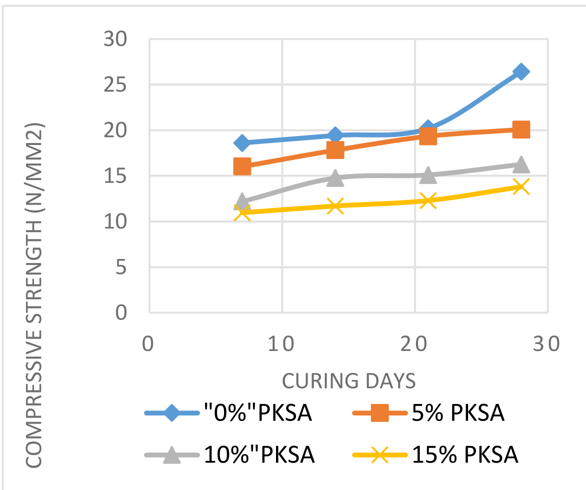

Figure 5: Graph of compressive strengthagainst Curing Days of concrete

4.0 CONCLUSION

From the results obtained from investigation of effect of partial replacement of cement with palm kernel shell ash on compressive strength of concrete, the following conclusions were drawn:

1. Palm kernel shell ash can be used as a material for partial replacement in light weight concrete structures where structural members are not resisting extensive impact load.
2. The specific gravity of the palm kernel shell ash was obtained as 2.34 which is lesser compared to that of ordinary Portland cement of 3.15 as reviewed by previous researches and duly cited. The use of palm kernel shell ash as partial replacement of cement exhibits a lower water absorption rate and slower setting time of concrete. Its use also reduce the volume of cement used in concrete, thereby reducing the cost of concrete production and also minimize the environmental issues arising from the improper disposal of palm kernel wastes.
3. The compressive strength of concrete increase with curing age and decrease with increasing percentage of palm kernel shell ash replacement.
4. The optimum compressive strength of concrete at 28 days curing age is 26.41 N/mm^2, which is higher than that of 7, 14 and 21 days curing age with their maximum compressive strength of 18.60 N/mm^2, 19.42 N/mm^2, and 20.20 N/mm^2 respectively.

5.0 RECOMMENDATION S

From the investigation of effect of partial replacement of cement with palm kernel shell ash on compressive strength of concrete, the following recommendations are made:

1. Curing of concrete with palm kernel shell ash (PKSA) as partial replacement of cement should reach 28 days in order to obtain maximum compressive strength.
2. Further studies should be carried out on 0 – 50% replacement of cement with palm kernel shell ash in order to reveal its possibility or otherwise.
3. Reduced cost of construction arising from the use of locally available agricultural waste materials such as palm kernel shell ash will enhance infrastructural developments.

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A GUIDE FOR THE HYDRAULIC AND STRUCTURAL DESIGN OF A BOX CULVERT

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Abstract— **Culverts are both hydraulic and highway structures. Their main characteristic is the provision of passage for water under any transportation route. They are artificial channels without which washed-out roads, flood, agricultural and environmental damages due to erosion would be very common. For a fully functional culvert that would stand the test of time, both the hydraulic and structural analysis must be carefully carried out. A culvert could be used to connect two areas separated by a water body. Just like other structures, culverts also fail if not well designed and their failure could be fatal. It is therefore of great significance to know how to prevent this failure from happening. Thus, the need to sufficiently design a hydraulically and structurally sound culvert. This paper presents a guide to understanding the engineering of a culvert and how to go about its hydraulic and structural analysis. This paper aims to carry out the basic hydraulic design of a box culvert, design the structural components of the culvert, and check the serviceability of the designed structure. The real-time hydrology and hydraulic data were obtained from NOK Associates Ltd. The hydraulic design was carried out with emphasis on inlet and outlet control conditions and the structural design was in accordance with BS 8110.**

Keywords—culvert, structural design, hydraulic design, highway, waterbody, passage, transportation, serviceability, NOK associates, engineering, civil engineering, structures.

I. INTRODUCTION

Nigeria is a developing country in terms of infrastructure. Every year, millions of Naira is allocated for infrastructural development across the country. Yet, problems of flooding are experienced every year all over the country.

Over the years, Nigeria has witnessed quite an alarming number of floods, causing loss of lives and properties. In September 2018, Nigeria's two infamous rivers; River Niger and Benue, burst their banks and influenced a disaster affecting over 70% of the country [1]. This mishap led to the declaration of a state of emergency in states that were worst hit. In December of 2020, a devastating flood affecting again, over 70% of the country was recorded with lots of deaths and severe injuries inflicted on citizens [2].

As part of the system of ensuring water is of a certain quality before reaching natural bodies, hydraulic structures play an important role. In the past couple of years, the quality of aquatic life has been degrading, rendering several species of fishes and plants at risk of extinction. In recent times, research has shown that our very commercially important and economically valued silver catfish is endangered [3]. The quality of aquatic life is threatened greatly by unsafe breeding grounds, habitat destruction, and water pollution to name a few [4].

One of the most important purposes a culvert serves is to convey surface water across a highway, railroad, or other embankments. They are principal structures in providing natural drainage, embedding traffic over waterways, preventing erosion and providing safe and natural passage for fishes and other aquatic wildlife [5]. Easily dismissed as being a bridge, people rarely give much thought as regards its design or location. However, the main difference between a culvert and a bridge is its span. A culvert spans 6m or less. Anything greater is a bridge. For example, the recently, widely reported bridge collapse in Ozuma Kogi State [6], [7] was a case of culvert collapse.

This technical paper carries out the basic hydraulic and structural design of a box culvert and checks the serviceability of the designed structure.

II. CULVERT OVERVIEW

A. How it Works

Culverts are engineered according to standards and are made of different materials including concrete, steel, and aluminum. The function of a culvert summarized, is to convey all excess water from rainfall and/or water overflowing from riverbanks which could easily lead to flooding. Culverts usually consist of the barrels which pass under the fill; the headwalls and wingwalls at the entrance and end walls at the exit to improve flow conditions (minimize energy loss), prevent embankment scour, and provide debris protection [8] [4].

In addition to the hydraulic function, the culvert must carry construction, highway, railroad, or other traffic and earth loads. Therefore, culvert design involves both hydraulic and structural design considerations. Culverts are available in a variety of sizes, shapes, and materials. These factors, along with several others, affect their capacity and overall performance.

B. Culvert Shapes

There exist quite several culvert shapes. The shape selection should be based on the cost of construction, limitation on upstream water, surface elevation, roadway embankment height, and hydraulic performance [9], [10]. Other factors such as the potential for clogging by debris, the need for a natural stream bottom, or structural and hydraulic requirements may influence the selection of culvert shape [11]. The common culvert shapes are Circular, Elliptical, Box, Arch, and Multiple barrel culverts.

C. Culvert Problems

Problems that can occur with culverts are numerous but can be simplified and broadly classified as strength-related problems or as serviceability problems. The strength-related problems include:

- Cracking of rigid culverts

- Undermining and loss of structural support
- Loss of the invert of culverts due to corrosion or abrasion
- Over-deflection and shape deformation of flexible culverts
- Stress cracking of plastic culverts

The serviceability problems are:

- Sedimentation and blockage of culvert barrel by debris
- Poor flow capacity
- Abrasion and corrosion of metals
- Erosion and scour of embankments
- Deterioration of concrete culverts

III. METHODOLOGY

The major consideration in hydraulic design is to select a structure that will pass the design flow without an excessive headwater elevation. A secondary consideration is the prevention of scours at the culvert outlet. The hydraulic design will help to:

- Determine the type of flow control
- Determine headwater elevation and outlet velocity
- Determine allowable outlet velocity and maximum allowable depth of barrel.

A comparison between the outlet and stream velocities should be carried out to determine the nature of the downstream channel. It should be noted that high outlet velocities may lead to excessive scour of the downstream channel. Thus, the following should be borne in mind:

- For outlet velocities less than 1.3 times the stream velocity, cutoff walls are commonly used.
- For culvert outlet velocities greater than 1.3 times the stream velocity, but less than 2.5 times the stream velocity, armoring riprap is used as a protection against the damaging potential.
- For outlet velocities greater than 2.5 times the stream velocity, energy dissipaters should be used (e.g. hydraulic jump stilling basin, riprap basin).

After the hydraulic design of the box culvert is completed, a final configuration for the culvert will be determined. Box culverts are analyzed and designed as rigid frames with equal bending moments at the end supports [12]. The Ultimate Limit State (ULS) and serviceability Limit States (SLS) are considered. The moment distribution method is generally adopted for the determination of final moments at joints of the frame and the culvert is analyzed for critical loading conditions. The ultimate and serviceability limit states principles are adopted for structural design. ULS is concerned with the safety of the structure as well as the safety of the users. ULS is designed purely elastic and is mainly to limit the amount of stress exerted on the material. SLS, on the other hand, focuses on minimizing movements (deflections and vibrations) of the structure.

A. Data Acquisition

The first step in designing a culvert is to gather all necessary hydrologic, geotechnical, and ecological data, as well as existing site conditions such as information regarding existing overhead and underground utilities on-site, the condition of the roadway, and the existing culvert if applicable[13]. A survey of the site should provide sufficient data for locating the culvert and identifying information on features affected by the installation of the culvert [11]. In situations involving existing culverts, a visual inspection is often conducted to assess structural condition and establish possible rehabilitation options[13].

B. Hydraulic Design

Hydraulically, a culvert is designed to convey the flow for a specified frequency storm.

The amount of flow that can be expected to reach the culvert is dictated by the following parameters:

1. The land type upstream (developed, forested, or agricultural)
2. The topography
3. The soil type, and
4. The storm frequency that must be conveyed.

These are to be determined without causing damage to surrounding property or infrastructure.

C. Structural Analysis

Structural design of a culvert is to ensure that all superimposed dead loads (weight of the surrounding backfill and road structures), and live loads (vehicular traffic on roadways including trains) are being catered for. This will establish that the culvert has adequate strength to be considered structurally sound.

The structural design of culverts will differ depending on if the culvert is rigid or flexible. Rigid culverts such as concrete culverts are designed to withstand the applied external loads internally on their own and are commonly reinforced for enhanced tensile strength, in other words, for crack resistance. Flexible pipe culverts on the other hand resist the applied loads through a composite action with soil [13]. The stability of a flexible culvert is maintained by lateral earth pressure. This is why it is most important during the installation of flexible culverts to compact the bedding and backfill materials with utmost care [13].

D. Inlet and Outlet Control

A culvert will operate under inlet control when the barrels' hydraulic capacity is higher than that of the inlet. Simply put, the flow of water out of the culvert is faster than the flow of water into the culvert. Under this condition, the discharge from the culvert is controlled at the inlet by the inlet geometry, cross-sectional area, and headwater depth. This flow is therefore controlled upstream and is limited to what can enter the culvert [9], [14].

The outlet control is somewhat the opposite of inlet control. In outlet control, the barrels' hydraulic capacity is smaller than that at the inlet.

During outlet control, the flow is always subcritical as it is relatively deep and slower. For this reason, the control of the flow is either at the downstream end of the culvert or further downstream of the culvert outlet.

IV. RESULT

A. Hydraulic Design

TABLE I. HYDRAULIC DESIGN

References	*Calculations*	*Output*
Site Investigation Carried out by NOK Associates Ltd.	**Design parameters:** Discharge, **Site condition:** Natural stream slope = 0.5% Tail water depth, = 0.5m Culvert length, L = 30m Downstream channel = stream of cross-section 7m width x 4.0m height Allowable headwater = **Design assumption:** Two 5m (H) and 3.3m (V) box culverts with 30° beveled edges in a headwall. Rectangular boxes carry equal discharge.	
[8]	**Computing critical depth** **Inlet control headwater** *(HW$_i$)* **Computation of outlet control headwater**	
	Evaluation of controlling headwater	
[8]	**Computation outlet velocity** Outlet velocity, Stream velocity, = 1.71m/s Riprap should be provided.	

B. Structural Analysis

TABLE II. STRUCTURAL ANALYSIS

References	**Structural Analysis**	
	Calculations	*Output*
	Design parameters Height of the culvert, H = 5m Total width of the culvert, w = 7m Width of culvert barrel, B = 3.33m Concrete cover, c = 0.05m = 50mm Soil cohesion, C =18 The angle of internal friction, Unit weight of soil, Unit weight of Concrete, Unit weight of laterite, Unit weight of stone dust, Unit weight of asphalt, Culvert thickness, b = 0.35m = 350mm Characteristic strength of concrete, Characteristic strength of steel,	
Terzaghi table	**Bearing Capacity** 37.2 22.5 20.1 Ultimate bearing capacity of soil: Depth of foundation, m B = 3.33m	

[15]	**CASE ONE: CULVERT EMPTY** **Imposed Load** Stress due to vehicle = 156.3 Stress due to vehicle =146 Design load = 1.5×156.3 = 234.4 Coefficient of earth pressure: = = 0.3333	156.3 146 =234.4 0.3333
[16]	**Top Slab** Wheel load is equivalent to: = Self-weight = = 11.76 Wheel load	= 193 =11.8 = 112.9
[16]	Overburden load (laterite+ stone dust+ asphalt) = Total load = 137.97 Fixed End Moment (F.E.M) = **Walls** Self-weight(as before) =11.76 **Bottom Slab** Top slab load = 138 Walls = 2 Total load on bottom slab = 789.3 Load per meter F.E.M = Stiffness factor for walls = Stiffness factor for slab =	= = 127.52
[16]	**Free Bending Moments** Top slabs, M = 0.125 Walls udl = Triangular = Total moment = 95.3kNm Bottom slab, M = 0.125	= = 95.3kNm
	(see figure 2. for moment distribution)	
[16]	**Shear Force** **Top Slab** **Walls** **Bottom Slab**	
	CASE TWO: CULVERT FULL (water over floods to a maximum of 5m) **Top Slab** Water load = 5 Self-weight (as before) = 11.76 Total load = 91.76 Wall: due to water pressure only At top slab	= =11.76 =

	At bottom slab Bottom Slab: due to top slab and walls Top slab Water on wall wall weight Total Pressure = 771.95kN Unit load	= 771.95kN =192kN/m
[16]	**Reinforcements** Thickness = 350mm Effective depth = 350-50 = 300mm **At top slab** M= 120.17 0.95d = 285mm PROVIDE 242 Y16 @ 125c/c (1610) PROVIDE 57Y16 @125c/c (1610)distribution bars **At bottom slab** M= 196.19) PROVIDE 174 Y25 @ 175c/c(2810) PROVIDE 57 Y16 @ 125c/c (1610) distribution bars	d= 300mm = =
[16]	**For walls** M= 213.55 take z =285mm PROVIDE 174Y25 @175c/c (2810) PROVIDE 57 Y16 @125c/c (1610) distribution bars	=

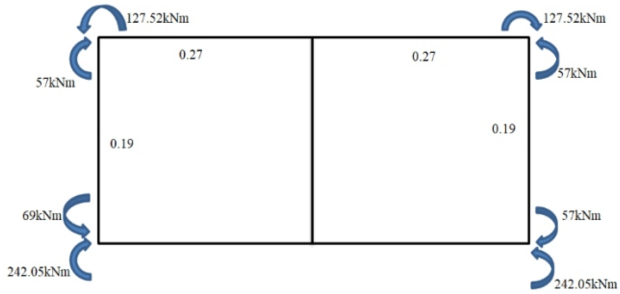
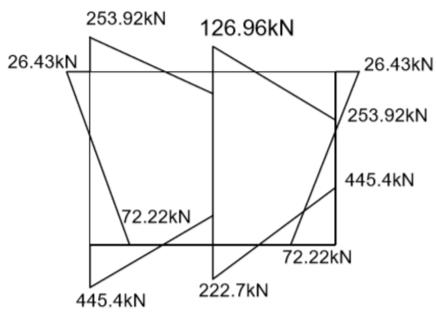


Fig. 1. Box culvert analysis details

Member	AD	AB	BA	BC	DA	DC	CD	CB
Stiffness factor	0.19	0.27	0.27	0.19	0.19	0.27	0.27	0.19
FEM	-57	127.52	-127.52	57	69	-242.05	242.05	-69
Balanced moment	-13.4	-19.04	19.04	13.4	32.88	46.72	-46.72	-32.88
Cross over moment	16.44	9.52	-9.52	-16.44	-6.7	-23.36	23.36	6.7
Balanced moment	-4.93	-7.01	7.01	4.93	5.71	8.12	-8.12	-5.71
Cross over moment	2.86	3.51	-3.51	-2.86	-2.47	-4.06	4.06	2.47
Balanced moment	-1.21	-1.72	1.72	1.21	1.24	1.76	-1.76	-1.24
c/o moment	0.62	0.86	-0.86	-0.62	-0.61	-0.88	0.88	0.61
Balanced moment	-0.28	-0.40	0.40	0.28	0.28	0.40	-0.40	-0.28
Cross over moment	0.14	0.2	-0.2	-0.14	-0.14	-0.20	0.20	0.14
Total	**-56.76**	**113.44**	**-113.44**	**56.76**	**99.19**	**-213.55**	**213.55**	**-99.19**

Fig. 2. Moment distribution result



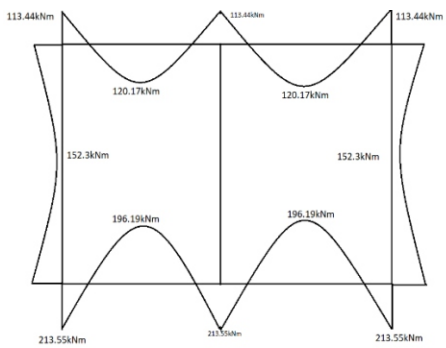


Fig. 3. Bending moment diagram.

DISCUSSION

Culverts are important highway structures. They help in providing safe passage of water below transportation routes without causing traffic or obstructing movement within the area. The failure of a culvert, no matter how small is detrimental, thus the importance to analyze and design a sound culvert.

The hydraulic data was obtained through site investigation by NOK Associates Ltd. The data obtained was studied and a trial configuration of was selected. To accommodate the trial configuration, the culvert has to be a multiple barrel culvert (twin cells), of sizes. Hydraulic analysis was performed and the controlling headwater was established. At the end of the analysis, the stream velocity was compared to the outlet velocity and a provision of rip rap was suggested. This, of course, will be sufficient to break the energy of the water.

The structural design started with the analysis of the dead and imposed loads and the determination of the total design load. This analysis was carried out in two parts. Case one is when the culvert is empty and case two is when the culvert is full. Fixed end moments, stiffness factors, and shear forces were calculated. Bending moment and shear force diagrams were then developed using the output from the calculation. The structural design was concluded by providing sufficient reinforcements to cater for the moments and forces to be developed during service.

CONCLUSION

As established, culverts are no doubt, important structures and a country like ours is indeed in desperate need of as many as can be constructed. Even though their conventional use is for diverting flow, they are utilized in a variety of applications because of their flexibility and accessibility in a wide scope of sizes. This work can serve as a template or guide for the design of Box Culverts for various purposes.

The hydraulic and structural design yielded the summary of design as seen below:

Hydraulic Design

Critical depth	0.44m
Inlet control headwater	0.63m
Outlet control headwater	0.35m
Controlling headwater	Inlet control
Outlet velocity	2.4m/s
Stream velocity	1.71m/s

Structural Analysis

Top slab reinforcement	Y16@125c/c
Walls's reinforcement	Y25@175c/c
Bottom slab reinforcement	Y25@175c/c
Distribution bars	Y16@125c/c

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Influence of Compactive Effort on the Hydraulic Performance of Tropical Red Soil–*Bacillus Coagulans* Mixtures

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ABSTRACT

The influence of compactive effort (CE) on the hydraulic performance of tropical red soil (i.e Lateritic soil) treated with *Bacillus coagulans* (*B.coagulans/microbial dose*) for waste containment use was evaluated. Tests carried out include index, pH, calcite content, hydraulic conductivity test and Micro analysis. Soil samples for pH, calcite content and hydraulic conductivity test were mixed with microbial dose at one-third the pore volume in stepped microbial dose of 0, 1.5 x 10^8, 6.0 x 10^8, 1.2 x 10^9, 1.8 x 10^9 and 2.4 x 10^9cells/ml in that order. Soil samples were prepared at varying moulding water contents, MWCs (i.e -2, 0, +2 and +4%) relative to optimum moisture content (OMC) while Reduced British Standard Light (RBSL), British Standard Light (BSL), West African Standard (WAS) and British Standard heavy (BSH) compaction energies were applied in the compaction test. Cementitious reagent was inoculated into the soil after compaction by gravity until saturation was achieved. Results of pH and calcite content test shows initial increase and subsequently reduced with increase in microbial dose. Values of hydraulic conductivity(k) declined from 1.27 $\times 10^{-6}$, 1.59 $\times 10^{-7}$, 3.94 $\times 10^{-8}$ and 5.42 $\times 10^{-9}$ m/s for the untreated soil to lowest values of 3.69 $\times 10^{-9}$, 1.89 $\times 10^{-9}$, 2.81 $\times 10^{-9}$ and 6.71 $\times 10^{-10}$ m/s for samples prepared at -2% of OMC and compacted with RBSL, BSL, WAS and BSH energy, respectively. Similar decreasing trends were also noted for samples compacted at OMC, +2 and +4 OMC. Statistical evaluation of test result shows significant effect of *B.coagulans* suspension densities on the treated soil. Based on the result obtained, 1.20$\times 10^{-9}$ – 2.40$\times 10^{-9}$cells/ml of the microbial doses compacted with BSH energy at 15.3% moisture content recorded optimal results and is recommended for applications for liners and covers.

INTRODUCTION

The primary environmental issue facing the whole world, in terms of subsurface environment is the protection of good quality groundwater and the clean-up or remediation of already contaminated groundwater resources. Today, highly engineered waste containment facilities, site investigations to describe the bahaviour and level of pollution, and remediation technologies to clean up soil and groundwater, are commonly used (Benson, 1999; Shackelford, 1990). In a developing country like Nigeria where the practice of waste containment is not common, deadly health hazards from soil or groundwater pollution by harmful elements are a reality, especially when the country steps into proper development of it natural mineral resources.

On the other hand, wastes produced are normally predisposed into nearby geological formation like gorges, valleys etc. Conversely, majority of such waste disposal sites are usually near water bodies (which served to provide water for consumption to humans, and animals) into which degenerating wastes and the potential poisonous leachate produced are eroded down when precipitation occurs. More so, the structural and geomechanical uprightness of such geological formations which usually predominately serves as naturally made barriers in such disposal sites, cannot be assured as they may be made up of fissures and cracks, assuming natural weathering situation which helps in propagating the easy flow of leachate down to the underground water and hence contaminating it.

Past and present researches focused on the ways of reducing the transport processes of these contaminants; usually the use of compacted clay liners as essential part of the lining system to serve as liners and covers for landfills has been documented in literatures(Daniel and Benson, 1990; Albrecht and Benson, 2001; Osinubi *et al.,* 2017; 2019). The most substantial factor influencing the performance of compacted clays is permeability (i.e hydraulic conductivity). Majority of regulatory organizations in the world at large oblige that the hydraulic conductivity should not be above $1x10^{-7}$cm/s (Das, 1998), for possible consideration for clay liners. Most soils are not suitable in their natural form for a compacted clay liner applications, and need to be improved. The use of industrially manufactured additives like cement, lime and industrial and agricultural waste having pozzalanic properties has proven to be effective for such purpose (Osinubi *et al.,* 2017). These techniques are either expensive or some of them are not environmentally friendly and therefore not sustainable. A novel sustainable and environmentally welcoming soil enhancement procedure that encompasses the use of soil micro- organisms (*B. coagulans*) to precipitate calcite is termed microbially-induced calcite precipitation (MICP).

MICP is a natural development in which long-lasting carbonates are made as a product of microbial metabolic or enzyme activities. The practice comprises a cementation procedure that joins natural subsurface soils by means of urea hydrolysis to induce calcite precipitation (Dejong *et al.*, 2006). MICP is a desirable field in geotechnical engineering that involves the use of microbial methods for soil improvement. This is a desirable technique for soil improvement for the reason that the calcite precipitation brought about as a product of microbial actions is environmentally friendly. MICP process effectively influences calcite precipitation in the soil by the practice of urea hydrolysis, in that way increasing strength and stiffness and also decreasing water permeability (Whiffin *et al.,* 2007; Harkes *et al.,* 2010; Burbank *et al.,* 2012; Chi *et al.,* 2017).

Engineering measurement is subject to various forms of uncertainties due to systematic and human errors or environmental factors which affect the laboratory results. Therefore, a statistical evaluation of the engineering properties of soils preceding to field application is significant. The concept of statistical analysis has been well employed in geotechnical engineering particularly in strength characterization of soils (Sani *et al.,* 2014, 2018; Yohanna *et al.,* 2017) and compaction properties of iron ore tailings treated black cotton soil (Osinubi *et al.,* 2016) among others.The study is pointed at assessing the influence of compactive effort on the suitability of treated soil for lines and covers. The precise objectives consist of examination of the variability of geotechnical properties especially hydraulic conductivity with changing compactive efforts and microbial doses. Moreover, the statistical evaluation of the test results obtained by laboratory test.

MATERIALS AND METHODS

Soil: The sample of soil used for the research was gotten from Anambra state, Nigeria (68°24'31''N and 27°52'11''E).

Microorganism: *B. coagulans* isolated from the soil was used.

Cementation reagent: Cementitious reagent used encompasses 3g of Nutrient broth, 20 g of urea, 10 g of NH_4Cl, 2.12 g of $NaHCO_3$ and 2.8 g $CaCl_2$ for each litre of distilled water in agreement with that defined by Stocks-Fischer *et al.*, (1999).

Bacteria solution: Solution of the Bacteria used contained 3g of Nutrient broth and 20 g of urea per litre of distilled water.

METHODS

Isolation of the Bacterium Specie: Serial dilution was adopted during isolation of the microbes. After isolation, the isolates were stockpiled at 4°C in nutrient medium preceding classification and characterization.

The Culture medium and growth conditions: The process applied was in agreement with that defined by Stocks-Fischer *et al.*, (1999).

Hydraulic conductivity (k) test: Falling head technique was carried out in accordance with BS 1377-5: 1990 and Head (1992). Soil samples were treated before compaction with microbial doses at one-third (1/3) pore volume (as recommended by Rowshanbakhta *et al.*, 2016) for varying microbial doses of 0, 1.5×10^8, 6×10^8, 1.2×10^9, 1.8×10^9 and 2.4×10^9 cells/ml in that order. Preparation of samples of soils were made at MWCs of -2, 0, +2 and +4 % relative to OMC and compacted using the RBSL, BSL, WAS and BSH Compaction Energy. Cementitious reagent was injected in cycles into the soil (Three cycles of injection) after compaction by gravity until saturation was attained. The compacted samples in the moulds were submerged completely for 24–48 hours in water to ensure complete saturation. Samples were controlled from vertical swelling throughout the saturation period by using metal plates that are perforated at both ends of the samples. Side wall seepage was ignored as recommended by Daniel (1994). k was computed using equation 1.

$$k = \frac{2.303aL}{At}\log\frac{H_1}{H_2} \qquad (1)$$

where k = Hydraulic conductivity(m/s), a = The standing tube cross-sectional area (m^2), A= mould diameter (m), L = specimen length (m), T = time (s)., H_1 and H_2 = heights of stand pipe initial and final water levels (m).

Mass of calcium carbonate measurements

Preparationof the sample and testing

Sample use for calcium carbonate content (CCC) measurement was prepared as discussed for k test. After preparation, samples were obtained from top and bottom of the compacted samples in the mould for each microbial doses of 0, 1.5 x10^8, 6 x10^8, 1.2 x10^9, 1.8 x10^9 and 2.4 x10^9cells/ml, in that order. The samples were at that point opened and dry at temperature of 25 ± 2°C preceding to testing.

The measurement method use is in accordance with that proposed by Mortensen *et al.,* (2011) and Choi *et al.,* (2017) called the acid wash method. In this process, 5g of natural and modified soils were mixed with 20 mL of 1-M hydro chloric acid (HCl) acid to liquefy calcium carbonate. At that moment all the solution and insoluble solid were washed using distilled water on filter paper placed on top of No. 200 sieve and washed for about 10 minutes. This washing method removes all soluble calcium from the soil particles. The solid particles residue on the sieve were dried in oven and weighed. The differences in weight between the original soil sample (A) and post washing sample (B) is the mass of calcium carbonate. The CCC was determined using equation 2

$$CCC = 100 - \frac{B}{A} \times 100 \quad (2)$$

Statistical AnalysisProcedure

Experiments results were obtained through laboratory tests. Measured soil factors include; Coefficient of permeability (hydraulic conductivity) as dependent factor and void ratio (d_1), bulk density(d_2),*B. coagulans*suspension density(d_3), pH(d_4), Compactive effort(d_5),water content relative to optimum(d_6), plasticity index(d_7),viscosity of microbes(d_8) and liquid limit(d_9) as self-determining(independent) factors. Minitab 2014 software and GeneXproTools 5.0 (made of four generations) were used to form multiliear regression equations to predict the k values of treated lateritic soil with varying microbial doses.

RESULTS AND DISCUSSION

5.2 Index Properties

Property	Quantity
Percentage Passing BS 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	
RBSL	1.76
BSL	1.83
WAS	1.86
BSH	1.9
Optimum Moisture Content, %	
RBSL	16.2
BSL	15.3
WAS	14.5
BSH	13.8
Coefficient of hydraulic conductivity, m/s	
RBSL	1.19E-07
BSL	2.83E-09
WAS	2.34E-09
BSH	1.03E-09
Colour	Reddish brown

Preliminary examinations on the natural properties of the soil displayed that the soil is fine-grained, having reddish brown colour. The soil is classified as A-4(2) (AASHTO, 1986) and SC (ASTM, 1992). A brief of the soil properties is given in Table 1.

The grain size curve is shown in Figure .The natural soil recorded a liquid limit of 37.5%, plastic limit of 19.3% and plasticity index of 18.2%. The soil has a maximum dry density values of 1.76, 1.83, 1.86 and 1.9 Mg/m^3 for RBSL, BSL, WAS and BSH compactions, in that order and corresponding optimum moisture content values of 16.2, 15.3, 14.5 and 13.8%, respectively.

Table 1: Characteristics of untreated lateritic soil

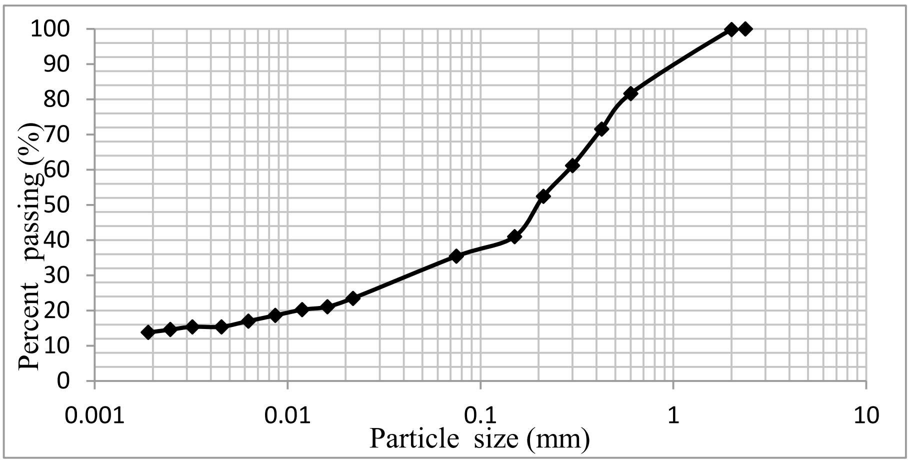


Figure 1: Natural soil grain size curve

Effect of ***B. coagulans*** suspension density on pH

The effect of microbial dose on the pH of the treated soil is revealed in Figure 2. The pH improved from its natural value of 5.97 to greatest value of 7.85 at 1.2 x 10^9cells/ml and thereafter decreased to 6.65 at 2.4 x 10^9 cells/ml. In the case of bottom, pH initially rise from its natural value of 5.97 to highest value of 7.23 at 1.5 x 10^8cells/ml and thereafter decreased to 5.86 at 2.4 x 10^9cells/ml. However, Stocks-Fischer *et al.*,(1999) observed that pH of the treated soil significant affect urease activities in the soil and reported a pH range of 6.0 – 8.0 for optimaly urease activites and calcite formation. It was noticed that the pH values for both top and bottom for the optimally treated soil at 2.4 x 10^9 cells/ml falls within the range recommended by Stocks-Fischer *et al.*,(1999). Several literatures (Mobley *et al*, (1995); Fujita *et al*, (2004); Umar *et al*, (2016) and Muhammed, *et al*, (2018) reported that pH affect MICP processes and reported optimal pH for calcite formation using diffierent micro organisms.

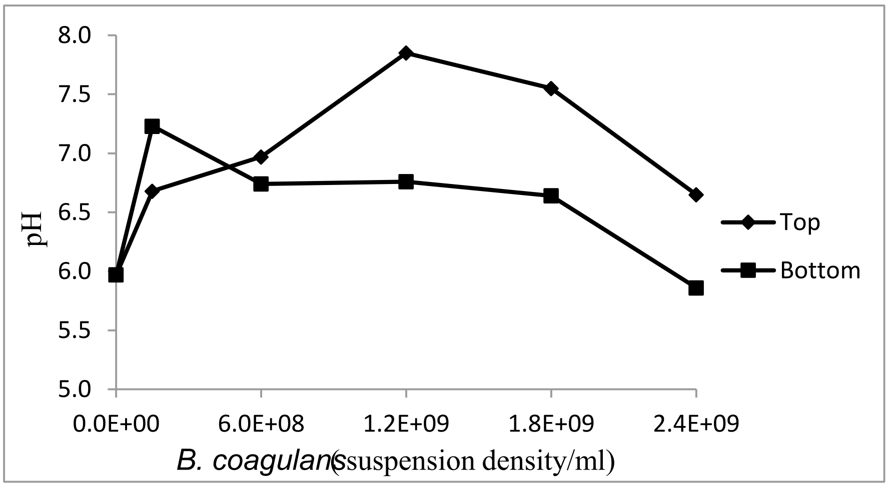


Figure 2: Variation of pH with microbial doses.

Calcium carbonate content(CCC)

One of the guiding principles for soil improvement by using the microbial improvement technique is the formation of soil binding material called calcium carbonate which stiffens the soil and improve its workability. CCC determination was achieved using the acid wash method as contained in Mortensen *et al.,* (2011) and Choi *et al.,* (2017). The variation in calcium carbonate content with *B. coagulans* suspension density is shown in Figure 3. CCC formed within the soil matrix amplified with increased in the population of the microbes from 0cells/ml upto 2.4 x 10^9 cells/ml. Values marginally increased from 3.6 to 3.9%. The increased may be associated with increased in the amount of urease enzymes produced by *B. coagulans*. As the population of the *B. coagulans* increased it is presumed that additional urease enzymes are released by the microbes primarily leading to the rise in the formation of the calcium carbonate. Chi *et al.,* (2017) and Osinubi *et al.,* (2019) in there researches reported that increased bacteria density results to greater enzyme activities because of the surfaces of the microbes serves as a nucleation site to induced calcite precipitation in the soil.

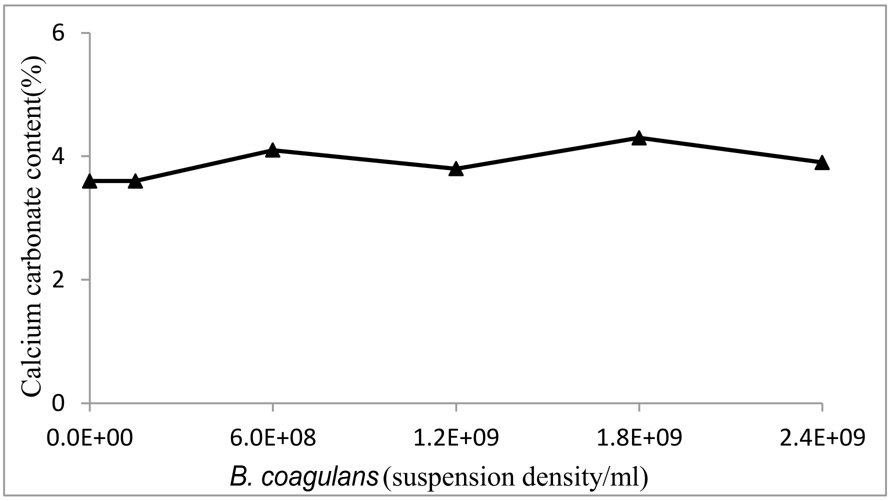
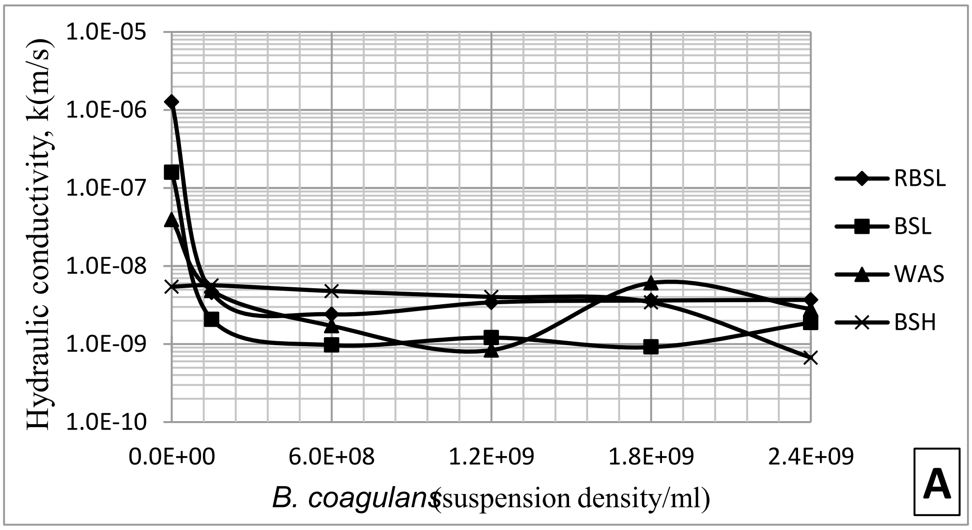


Figure 3: Calcium carbonate content measurement with varying composition of microbial doses

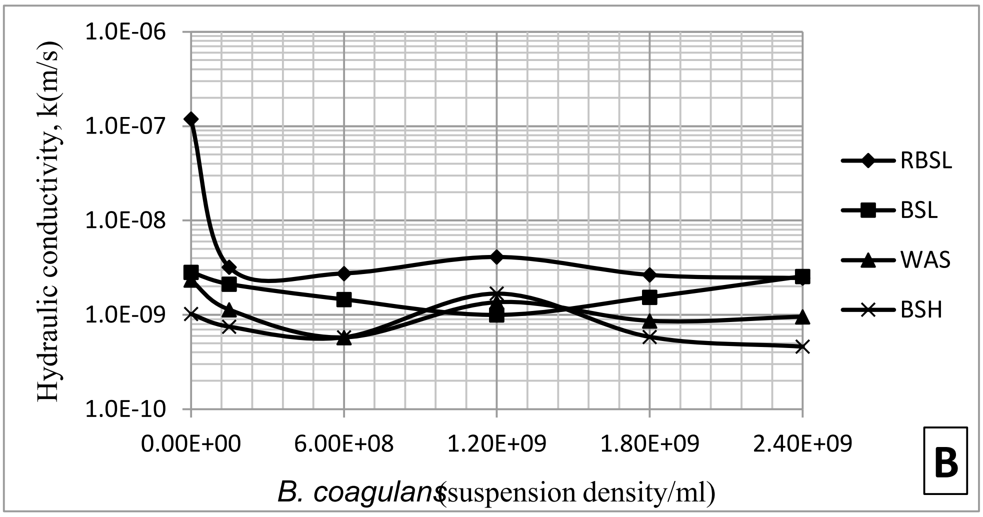
Hydraulic conductivity

Effect ofmicrobial doses on k values

The k values of lateritic soil with varying microbial dose is shown in Figures 4a-d. Generally, the k values of *B. coagulans* treated specimens compacted with the four energies considered (i.e., RBSL, BSL WAS and BSH), marginally reduced with higher microbial dose (see Figure 4a-d). The k values declined from 1.27 $\times 10^{-6}$, 1.59 $\times 10^{-7}$, 3.94 $\times 10^{-8}$ and 5.42 $\times 10^{-9}$ m/s for the natural lateritic soil (see Figure 4a), to least values of 3.69 $\times 10^{-9}$, 1.89 $\times 10^{-9}$, 2.81 $\times 10^{-9}$ and 6.71 $\times 10^{-10}$ m/s (see Figure 4d), when treated with up to 2.4 $\times 10^{9}$ cells/ml microbial dose for samples prepared at -2% of OMC and compacted with RBSL, BSL, WAS and BSH energies, respectively. Similar decreasing trends were also recorded for specimens compacted at OMC, +2 and +4 OMC. The drop in k values may be owing to the ammonia released into the soil that cause a rise in the soil pH, leading to build-up of calcite (Rong and Qian, 2013; Rowshanbakhta *et al.,* 2016). Result of k test satisfy the regulatory lowest k value of 1$\times$ 10^9m/s at 2.4 $\times$ 10^9/ ml microbial dose prepared at OMC and compacted with BSH energy.



Hydraulic conductivity, k(m/s)
1.0E-05
1.0E-06
1.0E-07
1.0E-08
1.0E-09
1.0E-10
0.0E+00
6.0E+08
1.2E+09
1.8E+09
2.4E+09
B. coagulans (suspension density/ml)
RBSL
BSL
WAS
BSH
A



Hydraulic conductivity, k(m/s)
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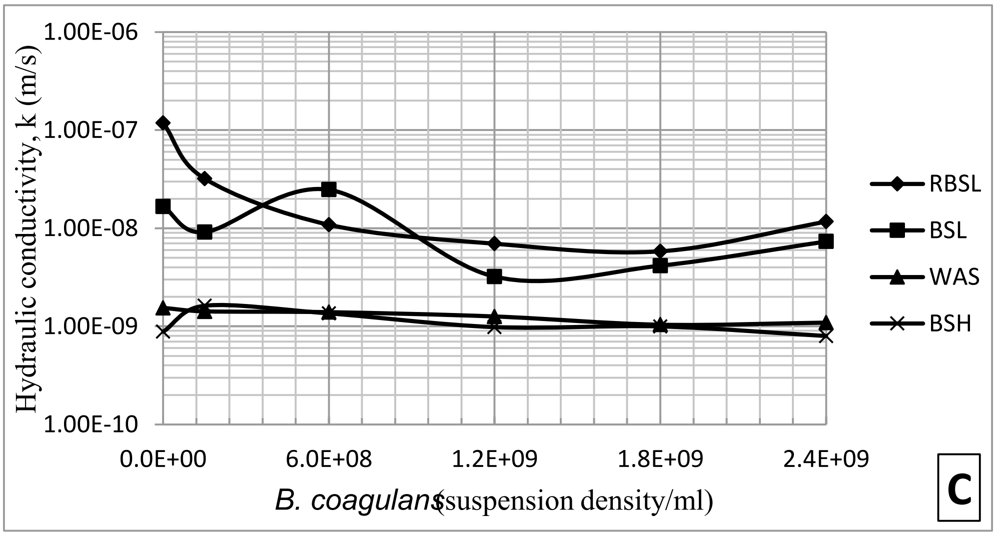
1.0E-07
1.0E-08
1.0E-09
1.0E-10
0.00E+00
6.00E+08
1.20E+09
1.80E+09
2.40E+09
B. coagulans (suspension density/ml)
RBSL
BSL
WAS
BSH
B



Hydraulic conductivity, k (m/s)
1.00E-06
1.00E-07
1.00E-08
1.00E-09
1.00E-10
0.0E+00
6.0E+08
1.2E+09
1.8E+09
2.4E+09
B. coagulans (suspension density/ml)
RBSL
BSL
WAS
BSH
C

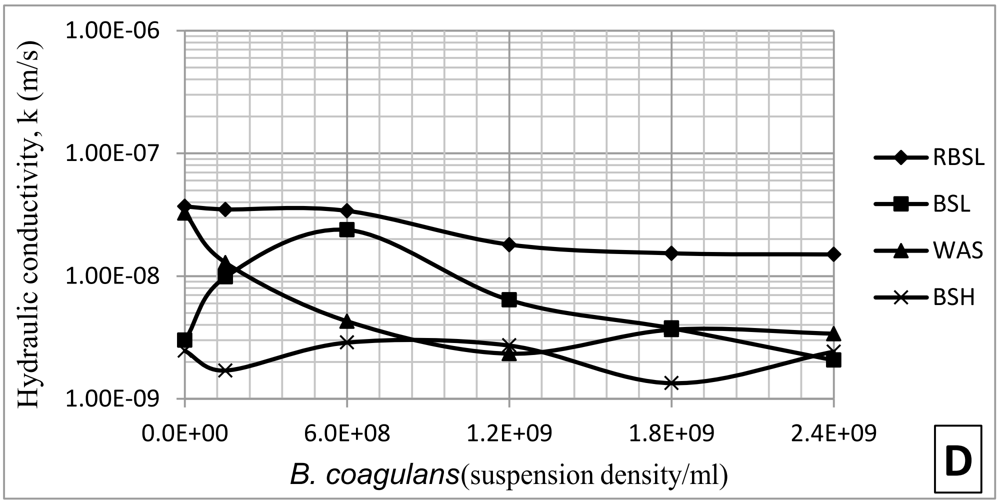
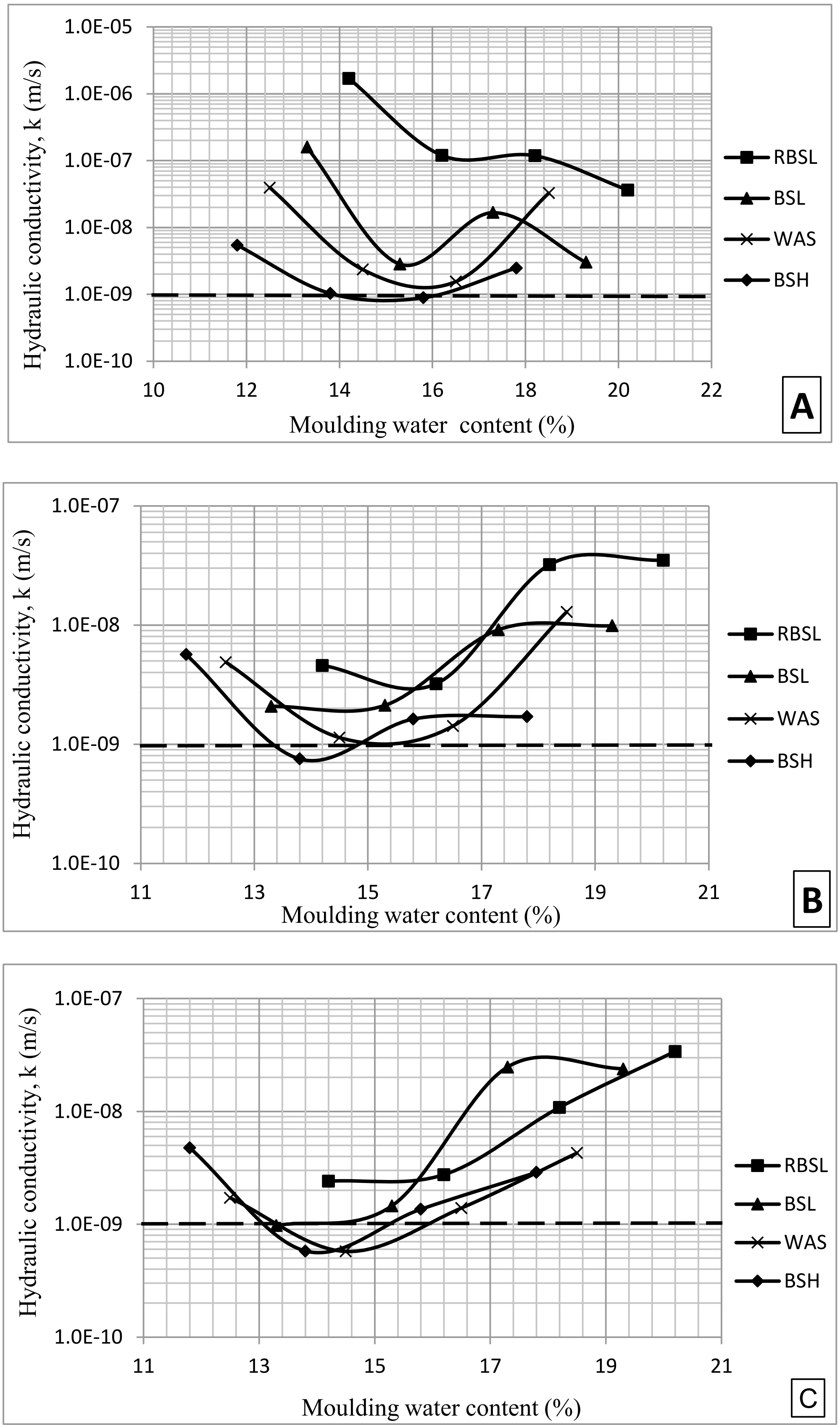


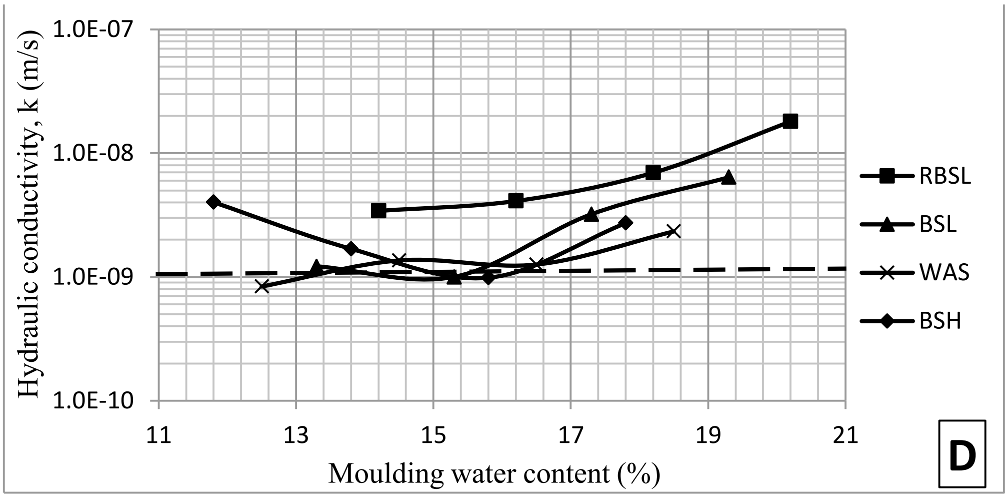
Figure 4: Plot of k values of lateritic soil with microbial doses for samples prepared at: (A) -2 % OMC (B) OMC (C) +2% OMC (D) +4 % OMC.

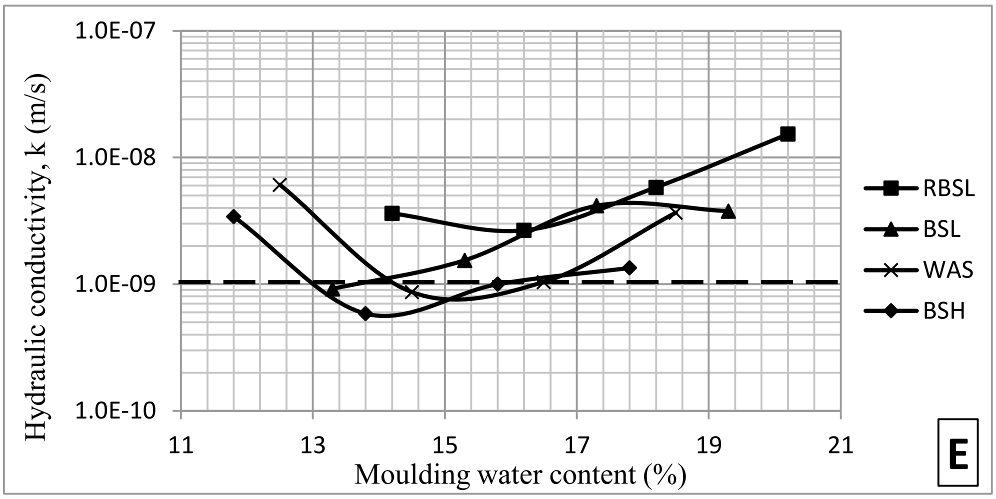
Effect of MWC relative to OMC

MWC greatly influences the k values of compacted soil (Amadi, 2008; 2010; Moses and Afolayan, 2011; Oluremi, 2015). The dissimilarity in k values of lateritic soil with MWC relative to optimum is shown in Figure 5a-f. k initially declined with increase in MWC from -2%OMC to OMC and thereafter increased with higher MWC for the compactive efforts considered. Satisfactory results were recorded for the natural lateritic soil prepared at MWC in the range of 13.8 – 15.8% and compacted with BSH energy only (see Figure 5a). Also, acceptable k values were recorded at optimal 2.4 × 10^9/ ml *B. coagulans* suspension density/ml for specimens prepared at MWCs in the ranges 14.5– 16.5% and 11.8 – 15.8 % for WAS and BSH energies, respectively(see Figure 5f). Other parameters of the WAS and BSH compacted specimens include values in the ranges: for dry density (1.69 – 1.76 and 1.75 – 1.84 Mg/m^3) degree of saturation (94 – 99 and 93 – 99%), void ratio (0.49 – 0.55 and 0.43 – 0.50) and porosity (0.33 – 0.36 and 0.30 – 0.33), for WAS and BSH energies respectively.

The least k values were noted for soil specimens prepared at OMC and compacted with RBSL, BSL, WAS and BSH energies. However, with rise in compactive effort, the k decreased. Natural lateritic soil specimens prepared at MWC between -2 and +4% OMC recorded k values in the ranges 1.27 × 10^{-6} – 3.69 × 10^{-8} ; 1.59 × 10^{-7} – 3.01 × 10^{-9}; 3.94 × 10^{-8} – 3.25 × 10^{-8} and 5.42 × 10^{-9} – 2.46 × 10^{-9} for RBSL, BSL, WAS and BSH energies, respectively (see Figure 5a). The drop in k with increase in compaction energy may possibly be associated with the clods or peds within the soil matrix that were softened and remoulded to eliminate permeable macro-pores in the soil. Result gotten agrees with Rowshanbakhta *et al.* (2016).


1.0E-05
1.0E-06
1.0E-07
1.0E-08
1.0E-09
1.0E-10
Hydraulic conductivity, k (m/s)
10
12
14
16
18
20
22
Moulding water content (%)
RBSL
BSL
WAS
BSH
A
1.0E-07
1.0E-08
1.0E-09
1.0E-10
Hydraulic conductivity, k (m/s)
11
13
15
17
19
21
Moulding water content (%)
RBSL
BSL
WAS
BSH
B
1.0E-07
1.0E-08
1.0E-09
1.0E-10
Hydraulic conductivity, k (m/s)
11
13
15
17
19
21
Moulding water content (%)
RBSL
BSL
WAS
BSH
C





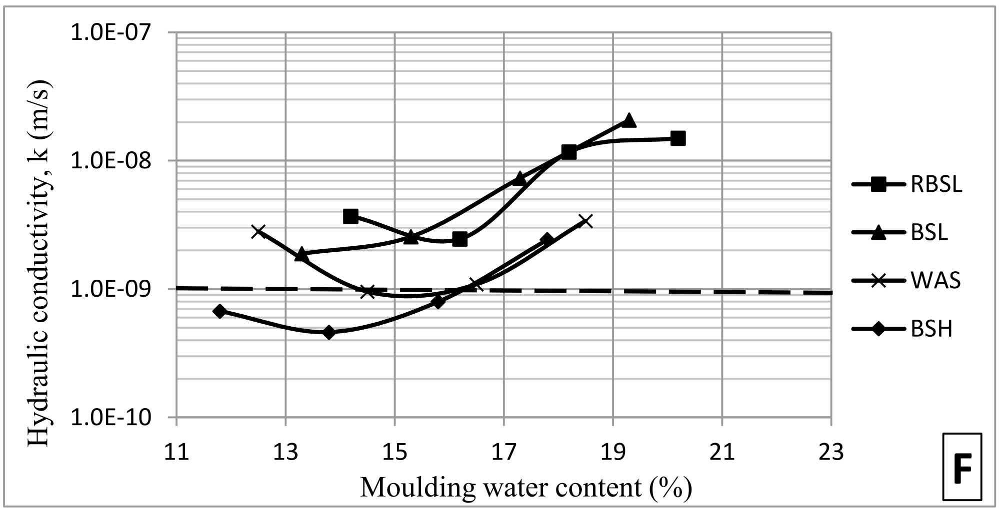
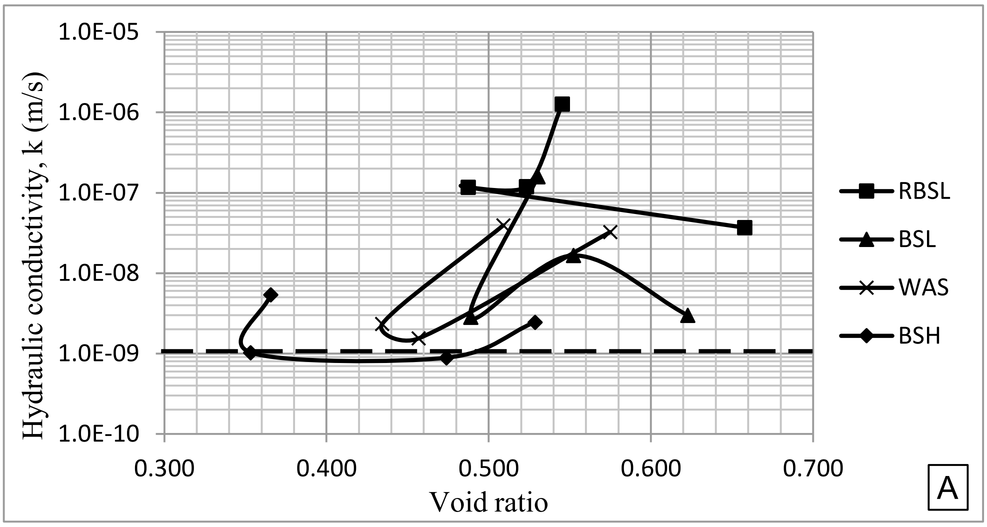
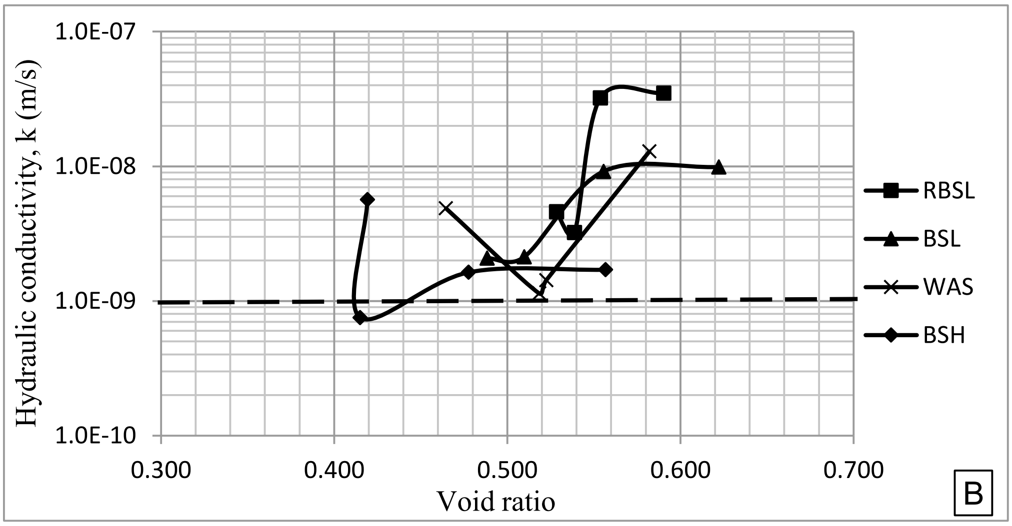


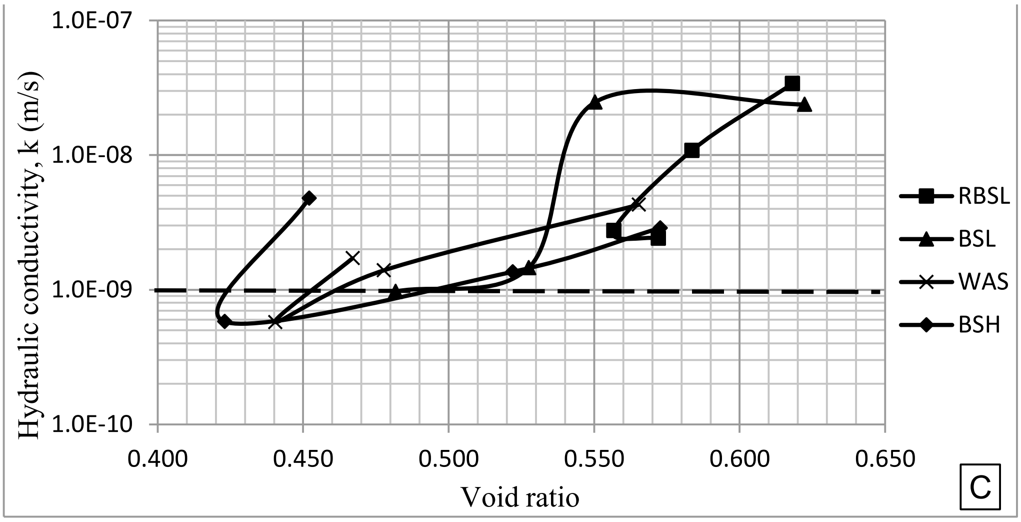
Figure 5: Plot of k values of lateritic soil with MWC for varying microbial dose: (A) 0/ml (i.e., Natural soil) (B) 1.5×10^8 /ml (C) 6.0×10^8/ml (D) 1.2×10^9 /ml (E) 1.8×10^9 /ml (F) 2.4×10^9/ml.

Effect of void ratio on k values of B. coagulans treated lateritic soil

The void ratio shows a dynamic role in influencing the k values of fine grained soils (Yoshikawa *et al.,* 2018). Figures 6a-f shows plot of k against void ratio with varying compaction energy. It will be noticed that k reduced with rise in the compaction energy. Also k reduced with reduction in the void ratio. Al-Moadhen, (2017) reported similar observation on the impact of void ratio on the hydraulic properties of residual soils. As the void ratio reduced the rate of percolation of water via the pores spaces in the soil matrix reduced thereby decreasing the k values of the microbial modified soil. MICP and biochemical processes anchored by the microbes in the soil usually causes stiffening of the soil and reduction in the void spaces. Biocement (calcite) formed which aid in reducing the hydraulic conductivity soil blocks the micro pores(i.e reducing void ratio) within the soil matrix (Nemati and Voordouw, 2003; Dejong *et al.,* 2010; van Paassen *et al.,* 2010; Choi *et al.,*2016).



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1.0E-08
1.0E-09
1.0E-10
Hydraulic conductivity, k (m/s)
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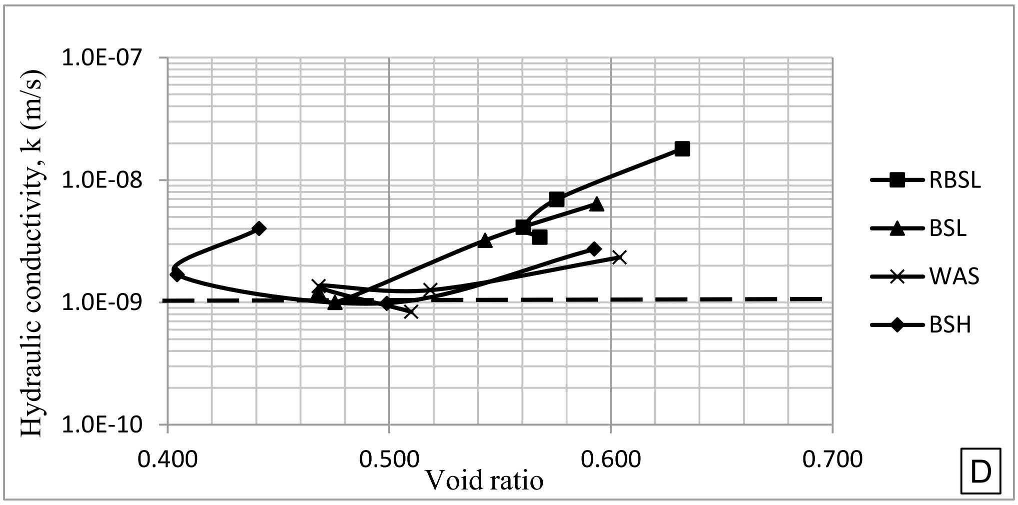
0.500
0.550
0.600
0.650
Void ratio
RBSL
BSL
WAS
BSH
C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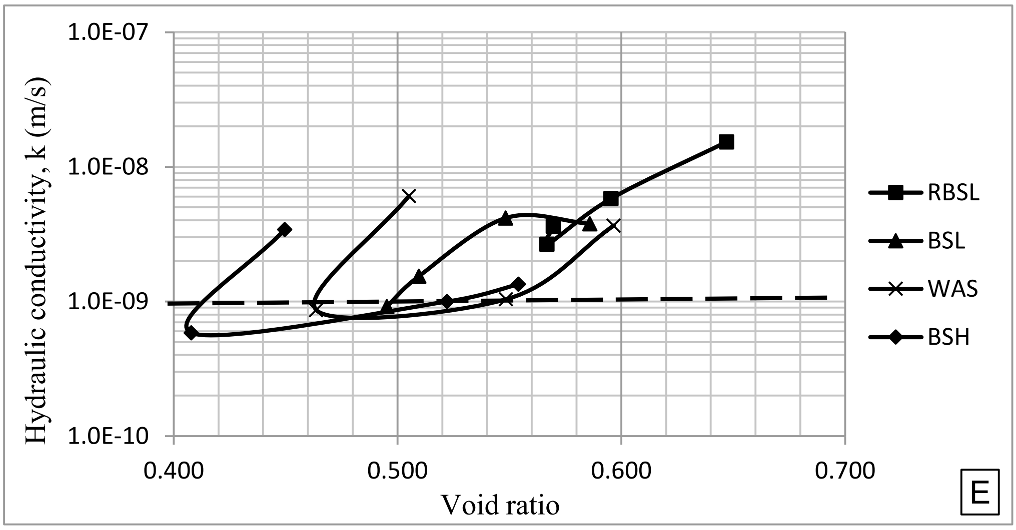
BSL
WAS
BSH
D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1.0E-08
1.0E-09
1.0E-10
Hydraulic conductivity, k (m/s)
0.400
0.500
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Void ratio
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BSL
WAS
BSH
E

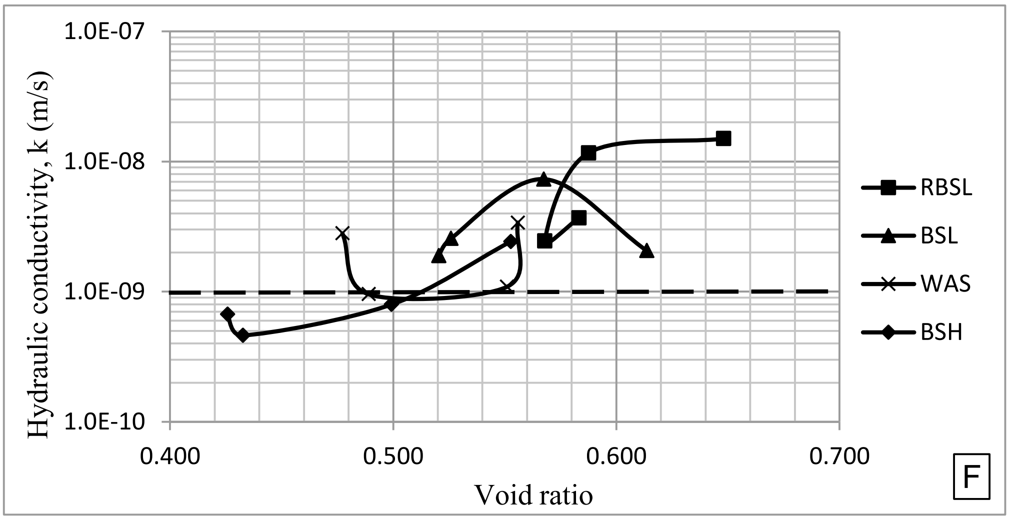
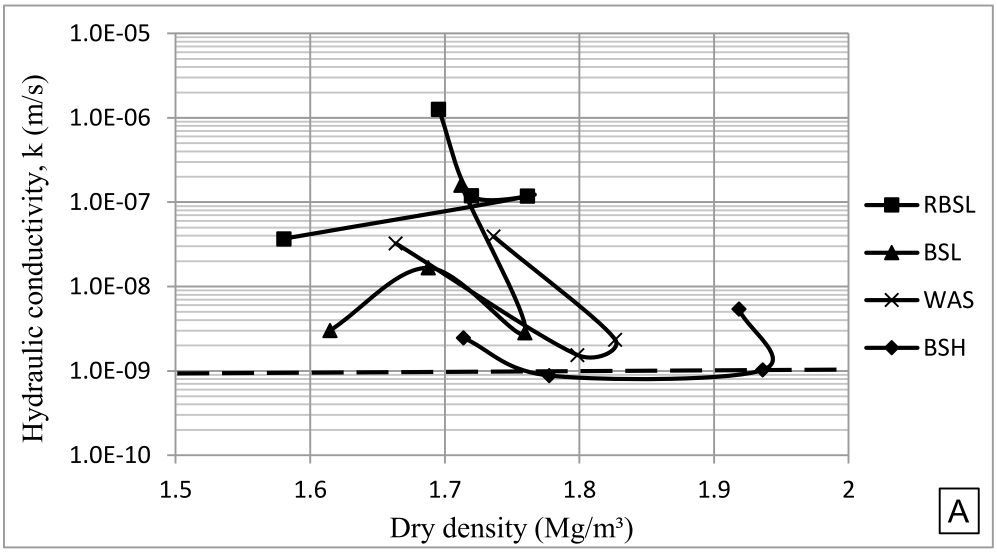
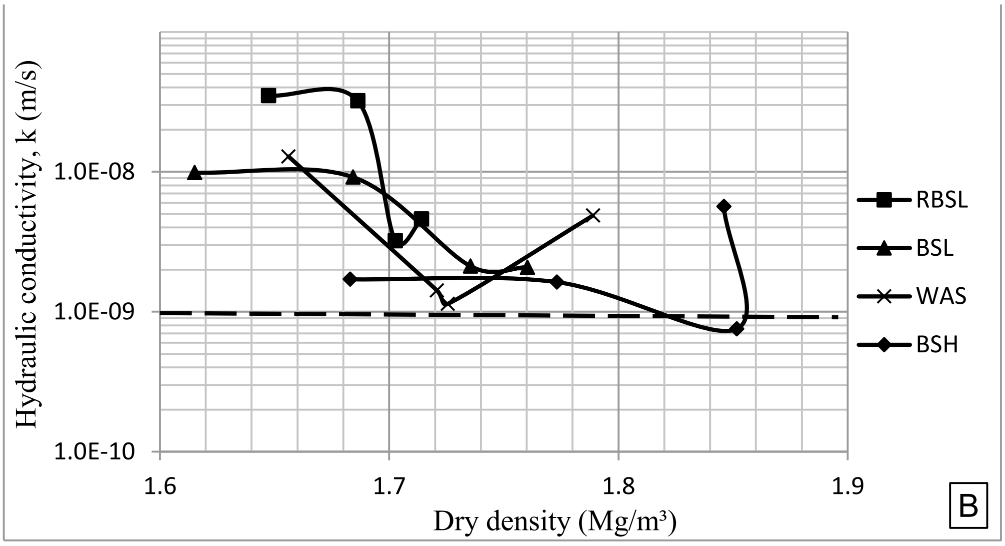


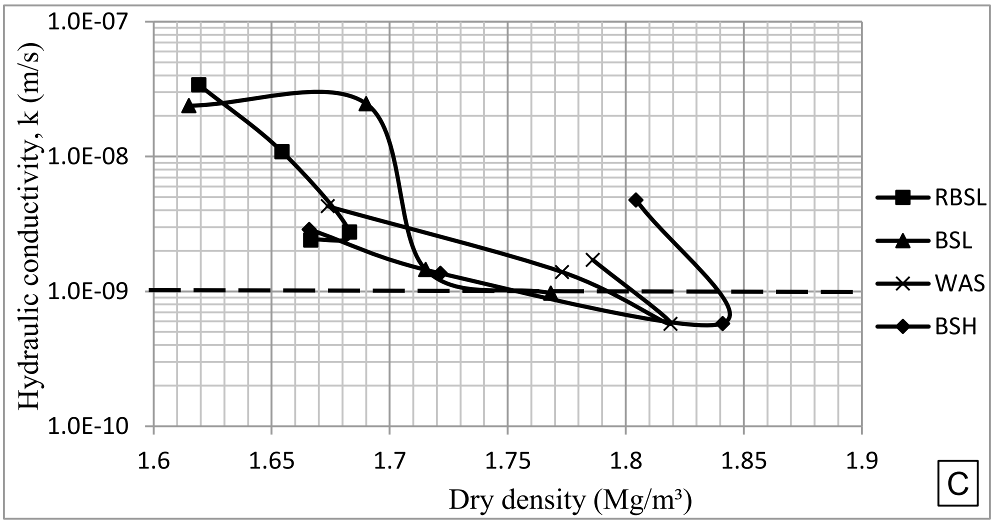
Figure 6: Plot of k values against the void ratio for (A) 0/ml (i.e Natural soil) (B) 1.5 x 10^8/ml (C) 6 x 10^8/ml (D) 1.2 x 10^9/ml (E) 1.8 x 10^9/ml and (F) 2.4 x 10^9/ml microbial doses at varying MWC.

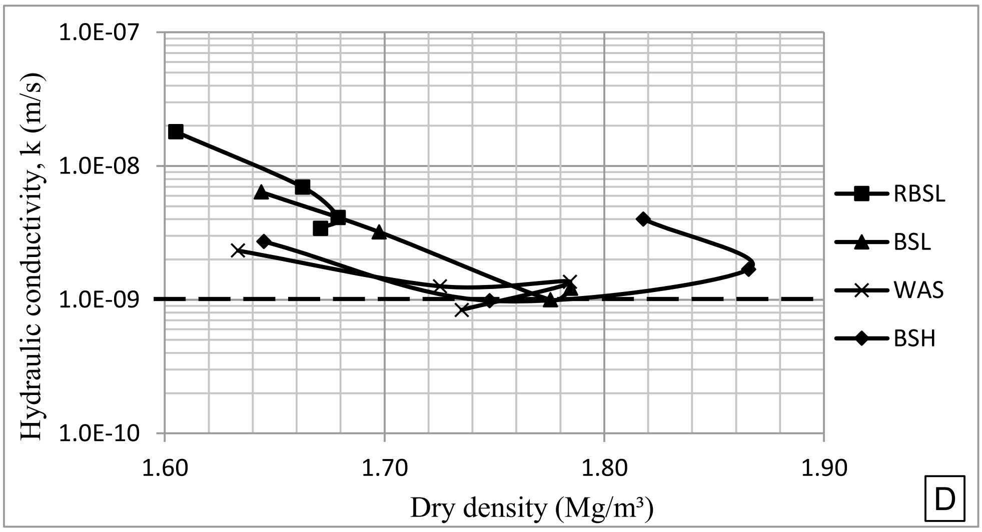
Influence of dry density on k values of B. coagulans treated lateritic soil

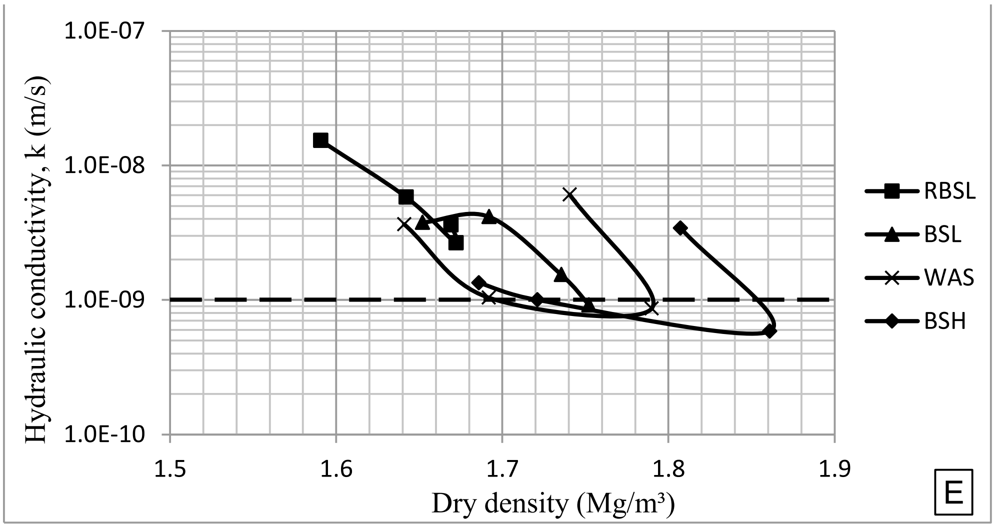
Hydraulic conductivity changes with dry density is shown in Figures 7a-f. Hydraulic conductivity diminished with rise in dry density and compactive efforts. The possible reason for the decline could be based on the packaging of the soil elements due to the effect of the compaction rammer that aid in increasing the density and reducing the voids thus hydraulic conductivity. As the soil becomes denser, the degree of movement of water in the soil reduced because of lessening in the void within the soil structure. The effect of MWC on the k values shows that samples compacted at OMC recorded the least k values with exception in few cases.




Hydraulic conductivity, k (m/s)
1.0E-08
1.0E-09
1.0E-10
1.6
1.7
1.8
1.9
Dry density (Mg/m³)
RBSL
BSL
WAS
BSH
B


Hydraulic conductivity, k (m/s)
1.0E-07
1.0E-08
1.0E-09
1.0E-10
1.6
1.65
1.7
1.75
1.8
1.85
1.9
Dry density (Mg/m³)
RBSL
BSL
WAS
BSH
C


Hydraulic conductivity, k (m/s)
1.0E-07
1.0E-08
1.0E-09
1.0E-10
1.60
1.70
1.80
1.90
Dry density (Mg/m³)
RBSL
BSL
WAS
BSH
D



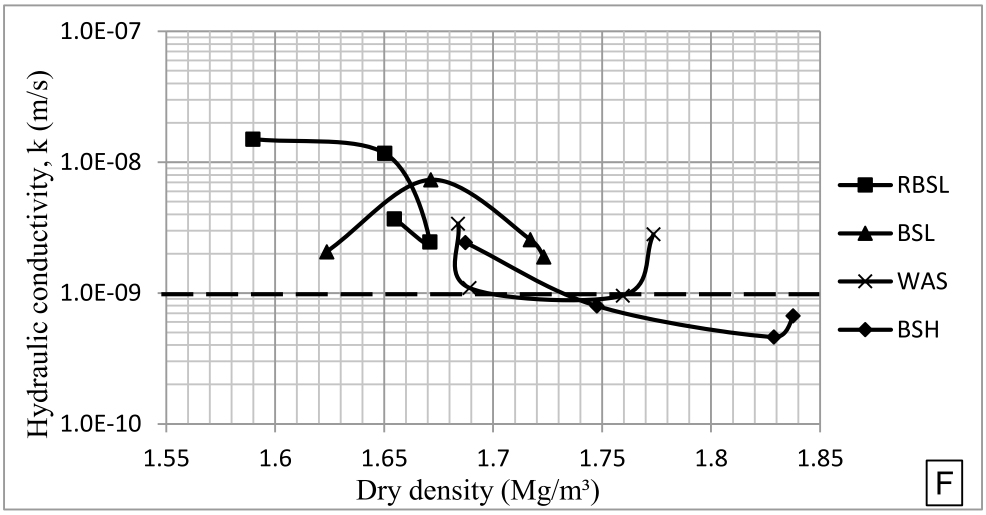


Figure 7: Plot of k values with dry density for (A) 0/ml (i.e Natural soil) (B) 1.5 x 10^8/ml (C) 6 x 10^8/ml (D) 1.2 x 10^9/ml (E) 1.8 x 10^9/ml and (F) 2.4 x 10^9/ml microbial doses at varying MWC.

Statistical Analysis

Analysis of variance

Statistical examination of the test results using two-way analysis of variance for hydraulic conductivity of lateritic soil – microbial doses is shown in Table 2. The effect of microbial dose and Compactive effort on the hydraulic conductivity are not significant ($F_{CAL} = 1.0627 < F_{CRIT} = 2.9013$) for microbial dose and ($F_{CAL} = 1.2043 < F_{CRIT} = 3.2874$) for Compactive effort.

Table 2 Analysis of variance results for k values of lateritic soil – microbial dose

Property	Source of Variation	Degree of Freedom	F_{CAL}	p-value	F_{CRIT}	Remark
Hydraulic conductivity	microbial dose	5	1.0627	0.4188	2.9013	$F_{CAL} < F_{CRIT}$, No Significant effect

Compactive effort	3	1.2043	0.3421	3.2874	$F_{CAL}<F_{CRIT}$, No Significant effect

Regression analysis

The regression model developed using the measured laboratory results shows a strong association between hydraulic conductivity, k as dependent parameters and the independent parameters considered, i.e Void ratio, d_1; Bulk density, d_2; *B. coagulans* suspension density, d_3; pH, d_4; Compactive effort, d_5; water content relative to optimum, d_6 ; Plasticity index, d_7; Viscosity of microbes, d_8 and Liquid limit, d_9 using GeneXproTools 5.0 with regression coefficient, R^2 = 85.5% (see equation 3) . In the case of similar regression model developed using Minitab 2014 software, a fairly poor relationship was established with regression coefficient, R^2 = 56% (see equation 4). It was evident from the regression coefficients that GeneXproTools 5.0 software gives better results and is recommended for prediction of hydraulic performance of a compacted lateritic soil–*B.coagulans* for waste containment application. GeneXproTools 5.0 software uses trigonometrical functions to established better results than ordinary regression model. Table 3 shows results of experimental values found from laboratory experiment and the projected values from the GeneXproTools 5.0 model with absolute percentage error of 0.68–28.62% (see Table 3).

$$k = d_1 - \frac{5.25}{d_9} + 2.52 + d_6 - d_7 + \mathrm{Cos}(\mathrm{Cos}d_1d_6) - 8.14\left[\frac{0.18}{\mathrm{Cos}\left[\mathrm{Sin}\left(\mathrm{Tanh}\left(\frac{d_4 - d_5}{2}\right)\right)\right]}\right]d_7 + \mathrm{Cos}\left[\frac{d_6}{\mathrm{Cos}(d_8 + d_6) + \left(\frac{d_7 + d_2}{d_2}\right)}\right] - d_2 + \mathrm{Tanh}[\mathrm{Sin}\, d_5(\mathrm{Cos}(-5.95d_6) - 3.82d_3)] + d_7 \quad (3)$$

$R^2 = 85.5\%$

$$k = -193 + 23.9d_1 + 19.9d_2 - 0.376d_3 + 10.7d_4 + 3.07d_5 + 4.19d_6 + 0.777d_7 + 1.37d_8 - 1.83d_9 \quad (4)$$

$R^2 = 56\%$

Where;

k= hydraulic conductivity, d_1 = Void ratio, d_2 = Bulk density, d_3 =*B. coagulans* suspension density, d_4 = pH, d_5 = Compactive effort, d_6 = water content relative to optimum, d_7 = Plasticity index, d_8 =Viscosity of microbes, d_9 = Liquid limit

Table 3: Results of Measured and predicted hydraulic conductivity values from Fitness function generated by GeneXproTools 5.

S/No	Measured hydraulic conductivity(m/s)	Predicted hydraulic conductivity(m/s)	Absolute error	% Error

1	5.75E-10	7.48E-10	1.7E-10	30.12
2	5.85E-10	5.47E-10	3.8E-11	6.51
3	7.52E-10	7.47E-10	5.1E-12	0.68
4	8.64E-10	7.37E-10	1.3E-10	14.71
5	9.56E-10	1.18E-09	2.2E-10	23.13
6	9.99E-10	1.19E-09	2.0E-10	19.65
7	1.03E-09	1.02E-09	8.1E-12	0.79
8	1.14E-09	1.33E-09	1.9E-10	16.96
9	1.36E-09	9.72E-10	3.9E-10	28.62
10	1.46E-09	1.42E-09	3.8E-11	2.58
11	2.12E-09	1.85E-09	2.7E-10	12.72
12	2.56E-09	2.36E-09	2.0E-10	7.92

Micro analysis

The micrographs (at 200µm scale and zoomed to 370×) of the microscopic examination by means of scanning electron microscope (SEM) of samples of natural lateritic soil and soil optimally treated with *B. coagulans* suspension density/ml are shown in Plates 1 and 2 respectively. It is practical that the natural soil has a rough textured appearances with black patches of openings or macro pores within the soil matrix. In the case of samples treated with *B. coagulans*, Smooth and needle like surface appearance with micro pores were observed (See plate 2). The smooth appearance could be associated with calcite formed that block the micro pores within the soil matrix or may be due to other geochemical processes (generation of biogas, extracellular polymeric substances formation etc may be responsible for such morphology(Dejong *et al.,* 2010, 2013; Osinubi *et al.,*2019)

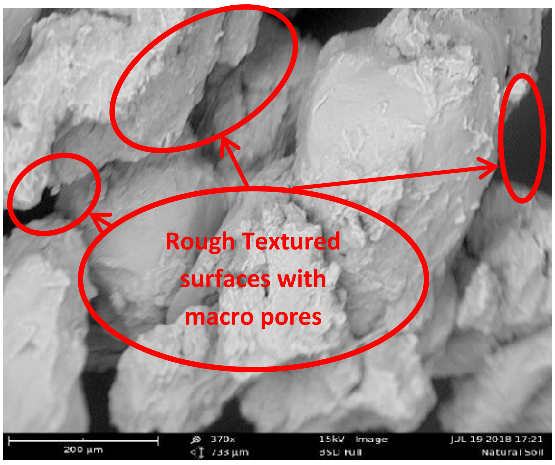


Plate 1.Micrograph of natural lateritic soil

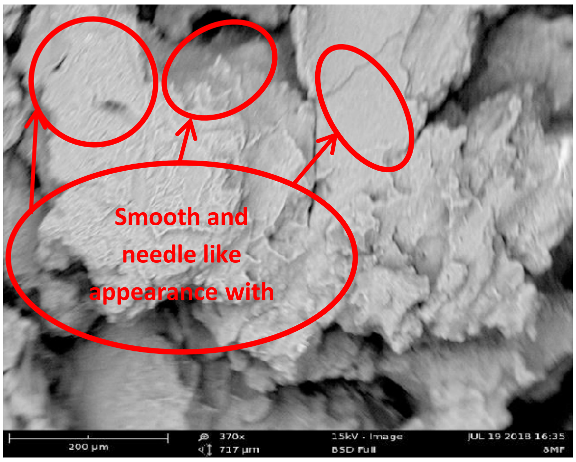


Plate 2 Micrograph of lateritic soil optimally treated with 2.4×10^9/ml *B. coagulans* suspension density.

CONCLUSION

Lateritic soil treated with *B.coagulans* in bio-cementation for use as a hydraulic barrier application was evaluated for varying compaction energy. Tests carried out consist of index test, pH, calcite content test, hydraulic conductivity test and Micro analysis. Results of pH and calcite content test shows initial increase and subsequently decreased with increase in microbial dose. k values declined from 1.27 $\times10^{-6}$, 1.59 $\times10^{-7}$,3.94 $\times10^{-8}$ and 5.42 $\times$ 10^{-9} m/s for the natural lateritic soil to minimum values of 3.69 $\times$ 10^{-9}, 1.89 $\times$ 10^{-9} , 2.81 $\times$ 10^{-9} and 6.71 $\times$ 10^{-10} m/s for specimens prepared at -2% of OMC and compacted with RBSL, BSL, WAS and BSH energies, in that order. Statistical evaluation of test result shows some significant effect of microbial dose on the treated soil. Based on the result obtained, 1.20$\times10^{-9}$ – 2.40$\times10^{-9}$/ml *B.coagulans*suspension density compacted with BSH energy at about 15.3% moisture content recorded optimal results. Therefore, it is recommended that the compactive effort, moisture content and *B.coagulans*suspension density should be considered primarily during design and construction of a lateritic soil compacted clay liner in other to achieve the desired result.

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