This Book is a report of detailed experimental work carried out on Strength Characteristics of Volcanic Ash Blended Laterized Concrete. It offers data on chemical and physical properties of volcanic ash, effects of curing age and percentage replacement of cement with volcanic ash (VA) on the compressive, tensile splitting and flexural strengths characteristics of laterized concrete. It further examined the existing relationships between compressive, tensile splitting and flexural strengths for granite concrete and its applicability to VA-blended cement laterized concrete with a view to ascertaining the suitability of VA as a pozzolanic material in the production of laterized concrete. It is a must have for every concrete practitioner interested in utilization of alternative binders in concrete.

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Olawuyi, Olusola

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STRENGTH CHARACTERISTICS OF VOLCANIC ASH BLENDED CEMENT LATERIZED CONCRETE

BY

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ABSTRACT

The study investigated the chemical and physical properties of volcanic ash, determined the effects of curing age and percentage replacement of cement with volcanic ash (VA) on the compressive, tensile splitting and flexural strengths characteristics of laterized concrete. It further examined the existing relationships between compressive, tensile splitting and flexural strengths for granite concrete and its applicability to VA-blended cement laterized concrete with a view to ascertaining the suitability of VA as a pozzolanic material in the production of laterized concrete.

A 4 x 4 x 6 x 3 factorial experimental arrangement was used for the study. The volcanic ash was obtained from Dutsin Dushowa, Kerang in Jos Plateau. Four levels of sand replacement by laterite (LAT), 0%, 10%, 20%, and 30%; four levels of cement replacement with volcanic ash, 0%, 10%, 20%, 30% and six levels of curing ages, 3, 7, 28, 56, 90 and 120 days were adopted with British Method of mix design for 28-day target strength of 25N/mm^2. This was taken as control. A total of 288 cubical, 288 cylindrical and 288 rectangular prisms were cast and tested for compressive, tensile splitting and flexural strengths. The test specimens were cured by complete immersion in water. Data obtained were fitted into existing models. Inferential and descriptive statistics were used for data analysis.

The result revealed that the chemical composition of the VA sample from the study area met minimum standard requirement for pozzolan. It had SiO_2 content of 41.13%; total SiO_2 + Al_2O_3 + Fe_2O_3 content of 70.99%; and Loss on Ignition value of 8.6. The VA-blended cements up to 30% VA content satisfied the physical requirements of NIS 439:2000, BS EN 196 – 6:2005 and ASTM C618:2008 for pozzolan. The strength properties (compressive, tensile splitting and flexural) of the VA-blended cement laterized concrete increased with

increase in curing age but decreased as the VA and LAT contents increased. The optimum replacement level was 20%LAT/ 20%VA. At this level, compressive, tensile splitting and flexural strengths increased with cutting age at a decreasing rate beyond 28days. The target compressive strength of 25N/mm^2 was achieved for this mixture at 90days of curing. The relation between tensile and compressive strengths for VA-blended cement concrete was similar to that of granite concrete. The result further showed a strong correlation between compressive and tensile (splitting and flexural) strengths of laterized concrete at various laterite contents 0% ($R^2 = 0.9558$ and 0.7139, $p \leq 0.5$), 10% ($R^2 = 0.9895$ and 0.7894, $p \leq 0.5$), 20% ($R^2 = 0.7456$ and 0.8970, $p \leq 0.5$), 30% ($R^2 = 0.9895$ and 0.7894, $p \leq 0.5$). LAT content; VA content and curing age had significant ($p \leq 0.5$) effects on the compressive, tensile splitting and flexural strengths of the VA-blended laterized concrete.

The study concluded that 20%LAT/ 20%VA laterized concrete was suitable for low cost housing, non-reinforced and low heat concrete works.

1 INTRODUCTION

1.1 BACKGROUND TO THE STUDY

Housing has been classified as one of the basic human needs. This need formerly seen in form of shelter is in modern use generally known to be more than shelter. It involves not just the structure, but also the infrastructures provided and the environmental condition of the building, the social services and other qualities of the environment that contribute to making a community a liveable environment (FRN National Housing Policy, 1991; Olusola *et al*, 2002). Housing is therefore an important element in the set of factors making visible contribution to the development of a nation and her citizens. However, in Nigeria like other developing countries and also to a certain extent in industrialised ones, housing shortage problems according to Falade (1999) is assuming increasing dimensions because of the consistent increase in building materials cost.

One of the major aspirations of human beings in life is to own a house. This is however frustratingly becoming a goal unattainable in Nigeria as in most other developing countries. This can be attributed to factors such as the present global economic recession, the disabled purchasing power, diminishing national income, lack of soft loans for housing finance, the rapidly expanding population of the nation, failed government policy, high cost of land, astronomical increases in the cost of conventional building materials, especially sand, cement and other ‘concrete’ components and lack of government or private sector in serious investment in building materials research development, mass production and patronisation (Olateju, 1991a; Anthonio, 2002; Olusola and Adesanya, 2004). The worst hit in this trend is the low-cost housing sector, while history reveals that man made his home from locally available material using the technology at his disposal. The return to the true principle of local material utilisation and familiar technology as it was in Africa before colonisation therefore may hold the key to the dream of housing for all.

Basic conventional building materials like cement and sand are becoming increasingly expensive to obtain because of high cost incurred in cement production, sand excavation process, pre-treatment and transportation. A 50kg bag of cement, which sold for #280 and #480 in December 1994 and April 1995 respectively (Olawuyi, 1995), now sell for #1850 as of September, 2008 in Minna market. Umoh (1990) reported that in spite of the large cement factories in Nigeria, the yearly supply does not match the demand for cement. To worsen the situation, most of the factories do not produce at full installed capacity and because the importation of cement is economically inadvisable, the difference between demand and supply invariably has an effect on the cost of cement. Similarly, the cost of sand has also impacted negatively on the cost of possible, but not widely utilised alternatives like laterite. This situation can be reversed if the potentials of laterite is fully exploited or popularised in addition to the advantage of its wider geographical spread and local availability on nearly every building site.

The provision of housing is governed by the need for shelter among other factors and according to Fitch and Branch (1960), the need for shelter must be met by materials that the environment can afford. Such materials must therefore be widely and readily available, appropriate to the environmental demands, thermally efficient and socially acceptable (Olusola, 2005). Besides, the building system derived from such materials must allow participation from the community and thereby improving the cash economy of that community. This is what Adegoke and Ajayi (2003) referred to as appropriate technology. Examples of such locally available building materials that fit into these descriptions are cement replacement materials such as rice husk ash, corncob ash, sawdust ash, volcanic ash and conventional sand replacement materials such as erosion sand and laterite. This study focuses on partial replacement of cement with volcanic ash in laterized concrete. Laterized concrete is concrete in which the sand component has been partially or wholly replaced with laterite.

Research trends on sourcing, development and the use of alternative, non-conventional materials have been concentrating either on purely partial or total replacement of cement in concrete on one hand and the replacement of sand with laterite on the other. Job (1998) reported the efforts made by researchers like Neville (1992), Talero (1990), Smith (1987) and Popovics (1986) to practically substitute cement with locally available materials called pozzolanas. "Pozzolan" is used to describe naturally occurring and artificially siliceous or siliceous and aluminous materials, which in themselves possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compound possessing cementitious properties (Neville, 2006; Shetty, 2004; Neville and Brooks, 2002). In the words of Matawal (2005), the application of use of various ashes as potential replacement of cement in mortar and concrete production has attracted the attention of researchers because of its potential to

a. reduce or totally eliminate the classification of ashes as waste materials polluting the environment, and

b. reduce the quantity and consequently the cost of cement applied in concrete works.

Matawal (2005) further highlighted that recent researches in Nigeria and abroad have shown that pozzolanas can produce concrete with close characteristics as normal concrete at ages beyond 28 days. Experimental studies have thereby been carried out on variety of waste ashes and materials with pozzolanic potentials such as fly ash, rice husk ash, sawdust ash, Acha dust ash, sugarcane fibre (Bagasse) ash, pulverized fuel ash, groundnut husk ash, blast furnace slag, mining tailings and volcanic ash (Hassan, 2006).

Laterite or laterized concrete on the other hand has attracted the attention of many authors and researchers. Gidigasu (1976) as cited in Olusola (2005) defined Laterite as a term used to describe all the reddish residual and non-residual tropically weathered soils, which generally form a chain of materials ranging from decomposed rock through clay to sesquioxides ($Al_2O_3 + Fe_2O_3$) - rich crusts, generally known as carapace. Laterite, either in raw form or improved form is commonly used both in rural and urban areas for housing construction (in form of masonry units). The Federal Low-Cost Housing Scheme at Satellite town, Ojo, Lagos is built of stabilized laterite blocks. So also are Low-Cost Housing Schemes in some States like Kebbi, Ekiti and others built with Hydra form (i.e. interlocking stabilized laterite) blocks. However, it has not been widely utilized to an equal level as sandcrete blocks and concrete, especially for structural works (Olusola and Adesanya, 2004). The reasons for this had been given as uncertainties as to their reliability, lack of knowledge of their physical properties and strength characteristics prior to use, inadequate knowledge of the actual performance of structures made from it under varying climatic condition and problems of quality control (Osunade, 1994; Olusola and Adesanya, 2004). The public believed that for laterite to be used on a wider scale, it should be improved at the technical level. Research investigations have thereby shown that stabilized laterite (laterite mixed with a certain quantity of cement $\leq 10\%$ by weight) can be advantageously used for the production of masonry units and that laterite holds promise as a partial replacement for sand in structural and non-structural concrete constructions.

Various research works on the production process, physical properties of laterite, its chemical interactions with cement, the effects of its incorporation on the strength and serviceability properties of hardened concrete have been carried out in Nigeria (Adepegba, 1975a; 1975b; 1977; Balogun and Adepegba, 1982; Lasisi and Osunade, 1984; Lasisi *et al*, 1990; Falade, 1991a; Ata, 1995; Olusola, 2005).

This research is therefore a further work on laterized concrete with the addition of a local pozzolanic material-volcanic ash as partial substitute for cement. This is in a bid to source for, develop and use of alternative material to replace sand and cement in the continuous effort towards reduction in the cost of building materials for provision of affordable houses.

1.2 STATEMENT OF THE RESEARCH PROBLEM

The housing shortage problem being faced and the global economic depression, low growth rate, severe shortage of foreign exchange earnings, increased debt repayment burden, rapidly increasing population and increasing cost of building materials (Ezem, 1997 as cited in Hassan, 2006; Olusola, 2005) has geared research efforts over the past few years on partial substitution of cement with locally available pozzolanic materials on one hand, and partial or whole substitution of sand with laterite on the other. A gap still exists on effect of introduction of local available pozzolanic materials on strength properties of laterized concrete. It is in this regard that this research work focus on the utilisation of volcanic ash (a locally available pozzolanic material) as a partial replacement of cement in concrete made with lateritic soils (i.e. laterized concrete). The abundant deposit of Basalt formations (the parent material of volcanic ash) in Nigeria (Salau, 2008; Lar and Tsalha, 2005) informed this study on the potential of volcanic ash as a useful component in laterized concrete. The study basically sought to proffer answers to the following questions

- Does the volcanic ash in question possess the required properties of a pozzolan?

- What effect will curing age and percentage replacement of cement with volcanic ash have on strength characteristics (compressive, tensile splitting and flexural) of laterized concrete?

- Will existing relationship between compressive, tensile and flexural strengths for concrete be applicable and valid for volcanic ash blended cement laterized concrete?

1.3 AIM AND OBJECTIVES

This research aimed at investigating the strength properties of laterized concrete using volcanic ash blended with ordinary Portland cement with a view to ascertaining the suitability of volcanic ash as a pozzolanic material in the production of laterized concrete.

The specific objectives are to

1. investigate the chemical and physical properties of volcanic ash in the study area and hence, determine its suitability as a pozzolan.
2. determine the effects of curing age and percentage replacement of cement with volcanic ash on the compressive, tensile splitting and flexural strength characteristics of laterized concrete; and
3. examine the relationships between compressive, tensile splitting and flexural strengths for normal concrete and its applicability to volcanic ash blended cement laterized concrete.

1.4 JUSTIFICATION OF THE STUDY

The global economic recession, the high cost of most conventional concrete materials, the need to increase the existing housing stock thereby alleviating or at least minimising the lack of affordable housing and the global search for alternative building materials coupled with the need to use locally available materials as a means of cost reduction have necessitated research works into the utilisation of laterite as a partial substitute for fine aggregate in concrete production for building purposes. So also are efforts being made to practically substitute cement (wholly or partially) with locally available pozzolanic materials like volcanic ash, rice husk ash, sawdust ash, millet husk ash, pulverized fuel ash, bagasse ash and others in concrete! They are all efforts centred at the search for alternatives. Laterite has been identified as a possible material for partial replacement of sand in concrete to produce what has been called laterized concrete, while studies have been carried out on effects of laterite incorporation in strength and serviceability properties of fresh and hardened concrete. This research is therefore a further work on laterized concrete with the addition of a local pozzolanic material (volcanic ash) as partial substitute for cement. It will further enrich knowledge in the effective structural utilisation of laterized concrete in building construction.

1.5 SCOPE AND LIMITATION OF STUDY

The study was limited to laterized concrete containing volcanic ash as pozzolan. The volcanic ash content was of varying amount up to 30% of the weight of cementitious material, while granite concrete having an average 28-day compressive strength specification of 25N/ mm^2 in accordance to BS 8110 (1985) now BS EN 1992 - 1 – 1:2004 for structural works was used as control specimen. The study is limited to compressive, tensile splitting and flexural strengths tests.

2 LITERATURE REVIEW

2.1 LATERITE

2.1.1 Definition and nature of laterite

Laterite is defined by Encarta English Dictionary (2008) as red tropical soil: a reddish mixture of clayey iron and aluminium oxides and hydroxides formed by the weathering of basalt under humid, tropical conditions.

Numerous definitions have been given to Laterite depending on the professional inclination of the authors. While some are purely morphological, some are purely physical and some others are purely chemical.

The term “laterite”, according to Hamilton (1995), was first used by Buchanan in 1807 to describe a ferruginous (high iron content), vesicular (contain small cavities), unstratified and porous material with yellow archers caused by its high iron content, and occurring abundantly in Malabar, India. It was used for weathering materials from which blocks are cut, that after drying are used as building bricks. Hence the word “laterite” was derived from the Latin word “later” which means brick or tile. Laterite has been recognized as the alteration or in-situ weathering products of various materials including crystalline igneous rocks, sediments detrital deposit and volcanic ash. The degree of weathering to which the parent materials have been subjected influences greatly the physical and chemical composition of Laterite soils (Olusola, 2005).

The first to establish the chemical concept of the definitions of Laterite was probably Mallet (1883) as quoted in Osunade (1984), Owoshagba (1991) and Olusola (2005). He established the ferruginous and aluminium nature of lateritic soils. Fermor (1981) defined various forms of laterite soils on the basis of the relative contents of the so called laterite constituents (Fe

Al, Ti, Mn) in relation to Silica. A chemical definition base on the (S-S) Silica Sesquioxides ratio (SiO_2 / Al_2O_3+Fe_2O_3) had been proposed, the conclusion being an s-s ratio $\leq$ 1.33 implies a true laterite; an s-s ratio between 1.33 and 2.0 refers to a lateritic soil; and an s-s ratio $\geq$ 2.0 indicates a non-lateritic typically weathered soil.

Based on its morphological properties, a basic definition has been given by Pendleton and Sharasuvana (1994) who viewed laterite soil as a profile in which there are immature horizon of laterite from which a true laterite horizon develop if appropriate conditions prevail for long enough.

A chemical-mineralogical definition of laterite is given by Olusola (2005) quoting Tietz (1997). He stated that a distinction should be made between the highly weathered (laterite) and the less strongly weathered residual rock (saprolites). The chemical-mineralogical reactions which characterize the weathering of rock into saprolites and laterite are thereby listed as:

1. Kaolinization of Al-Si-bearing minerals;
2. Formation of Fe oxides from Fe-containing minerals;
3. Formation of Al hydroxides by incongruent solution of Kaolinite minerals (incongruent solution is a dissolution accompanied with decomposition);
4. Congruent dissolution of Kaolinite minerals, i.e. dissolution without change in composition;
5. Dissolution of quartz.

According to Olusola (2005), reactions 1, 2, 3 & 5 causes an enrichment of iron and aluminium in the weathering residue; while reaction 4 increases the iron content. Hence laterites are commonly composed of a mixture of Goethite ($\propto$- FeOH.OH), yellowish brown to red; Hematite ($\propto$- Fe_2O_3), mainly red ochre's; Alhydroxides (gibbsite) [$Al(OH)_3$] white, pale pink, green, grey, light brown; Kaolinite minerals and Quartz (SiO_2), colourless, white, variable: black, purple, green, etc.

Laterite was thereby defined as "products of intense sub-aerial rock weathering". They consist predominantly of mineral assemblages of **goethite, hematite, aluminium hydroxides, kaolinite minerals and quartz.**

The weathering ratio, r_w (SiO_2; Al_2O_3 + Fe_2O_3) of a laterite must be lower than that of the kaolinized parent rock in which all the alumina of the parent rock is present in the form of kaolinite, all the iron in the form of iron oxides and which contains no more Silica than is fixed in the kaolinite plus the primary quartz.

Olusola (2005) stated in summary that Laterites:

Are rocks, in the widest use of the term; belongs to the group of residual rocks; are classified on the basis of their mineral content.

On basis of physical properties, Lateritic soil have been defined as an igneous rock, tropically weathered in-situ which has decomposed partially or totally with the concentration of iron or aluminium sesquioxides at the expense of silica. However, going by the range of soils the term laterite covers, its definition should not specify the type of composition of the original or parent rock. The fact that Si-Al-bearing minerals are in the majority (and this is true for the average composition of igneous rocks) does not necessarily infer that every parent rock has to be the igneous type; the physical definition can therefore not hold.

In the view of Maignien (1996) and Campbell (1977), purely morphological, purely physical or purely chemical definitions of lateritic soils are unrealistic for practical application (especially in engineering). Campbell further states that chemical analysis might not be sufficient to reveal the composition, nature and origin of laterite soils. Research works and practical experience have revealed that Laterites obtained from the same location or near distances apart may differ in physical properties, chemical composition and strength (Lasisi and Ogunjimi, 1984 as cited in Olusola, 2005) or in one of these.

Those with similar physical properties may differ in chemical composition and vice versa. This implies that the word "Laterite" is used to describe a general and a wide variety of tropical soils. It continues to be used to refer to lateritic soils of different physical, chemical and generic formations.

Gidigasu (1976) gave a broad-based definition of Laterite which may be more appropriate for engineering applications. He states that the word laterite should be used to describe "all the reddish residual and non-residual tropically weathered soils, which genetically form a chain of materials ranging from decomposed rock through clays to sesquioxides (Al_2O_3 + Fe_2O_3) rich crust, generally known as cuirass or carapace". Cuirass stands for the upper layer of laterite accumulation zone and is particularly enriched in iron oxide minerals. Carapace on the other hand stands for the lower part of laterite accumulation zone. Miller (1999) also describes laterite as heavily leached tropical subsoil which is not fertile and comprises mainly iron and aluminium oxides and kaolinite-clays.

Olusola (2005) quoting Tietz (1997) outlined the following as conclusion on the term laterite: It covers a wide variety of tropical soil formation; signifies products of intense sub-aerial rock weathering; belongs majorly to the group of residual rocks; is an "engineering" soil material near the earth surface whose chemical and mineralogical composition are largely determined by their parent rocks; has its genesis centred around the two common sesquioxides, Al_2O_3 and Fe_2O_3, the main and characteristic compounds of most laterite accumulation zones; and has the presence of iron considered the most important factor that influences its engineering properties.

Laterite therefore signifies a highly weathered material of wide varieties and rich in secondary oxides of iron and aluminium, with iron having a greater influence on the engineering properties. It consist mainly sand and clay fractions. It is either hard or capable of hardening on exposure to wetting and drying.

2.1.2 Formation of Laterites

Laterite genesis or mode of formation has been identified as one of the most important factors influencing the geotechnical characteristics and field performances of Lateritic soils (Lasisi and Ogunjimi, 1984). The combined effects of the pedogenic factors (i.e. parent material, climate, vegetation etc) determine in turn the weathering system. Other factors affecting engineering performance of Laterite already identified by most researchers are degree of weathering, morphological characteristic and their chemical and mineralogical composition.

Laterite is most especially found extensively in the tropical regions of Africa, Australia, India, South-East Asia and South America. Figure 2.1 and 2.2 presents the global distribution of laterite and pedological soil map of West Africa respectively.

Olusola (2005) quoting the works of Gidigasu (1976), Campbell (1977), Jeje (1980), and Tietz (1997) presented a list of factors or parameters usually responsible for formation of Laterite as follows:

a) A high average ambient temperature (25 – 40^{0}C) and sufficient organisms and decaying organic matter to make the abundant percolating rainwater a chemically and physically active fluid. The amount of vegetation is also equally important.

Figure 2.1: Generalised Map Showing the Distribution of Laterite Soils
Source: Gidigasu (1976) Fig. 2.1 pp. 3

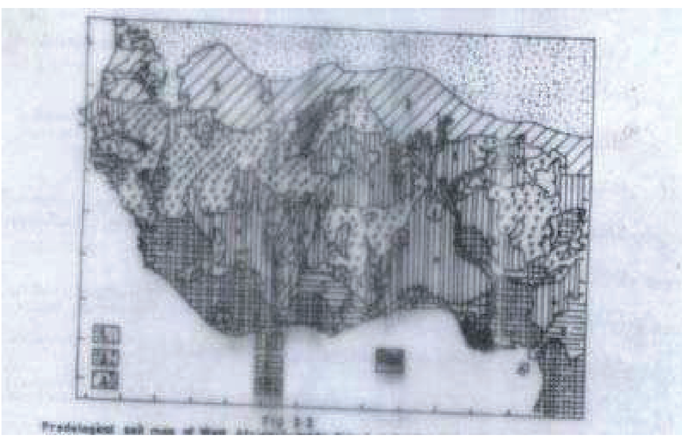


Figure 2.2: Pedological Soil Map of West Africa

1 = mainly desert and sub-desert soils; 2 = mainly weakly developed soil rocky areas (including young soils lithosol and soils in iron pan crusts); 3 = mainly brown and reddish brown soils of arid *and semi-arid areas; 4 = mainly ferruginous tropical soils; 5 = mainly ferrisols; 6 = mainly ferrallitic soils; 7 = hydromorphic soils mainly developed in alluvium; black = mangrove soils; D = vertisoils; H = eutropic brown soils; M = halomorphic (saline) soils; R = volcanic soils, Note: 3 – 6 are laterite soils.*

Source: Gidigasu (1976), Fig. 31 p.76

b) A high supply rate of rain (solvent) to promote sufficient leaching (removal of a soluble material from an otherwise insoluble solid phase by dissolution in a liquid solvent).

c) A permeable profile that will ensure high rate of percolation of the water in order to evacuate the leachate (solution obtained by leaching and which contains substances from the original material leached) and bring fresh water (solvent) in contact with, as yet unleashed matter.

d) A sufficient topographic height above the local and/or regional base level of erosion to promote a continuous high rate of percolation of water and transport of leachate.

e) A parent rock or material which:

   i. Has a fabric (structure or texture) that allows a continuous and pervasive flow of water and which provides sufficient reaction surfaces to promote leaching, i.e. not too fine-grained to become too tight for percolation and not too coarse-grained to reduce the area of active solution surfaces,

   ii. Contains high concentration of water-soluble iron and aluminium salts in silicates forms which can form a skeleton of newly formed hydroxides and oxides of Al and Fe. During wet periods, iron is mobilized and held in solution in its more mobile ferrous form (Fe^{2+}- goethite) within the weathering profile. The iron is mobilized either by percolating water in an acidic medium or by anaerobic weathering in the saturated zone. In the succeeding dry period, when the water table is lowered, iron is oxidized to the ferric (less soluble and less mobile form) compounds, Fe^{3+}. This remains through the next period of high water table to be added to in the following period of desiccation. Since Fe^{3+} is less mobile than aluminium, the latter will migrate downward to accumulate in bauxite deposit below an iron crust;

   iii. Allows for removal through solution of large parts of its mineral content.

iv. Contains an underlying barrier which inhibits the downward departure of ground water. This ensures that the water table is close to the surface. A fluctuating water table is essential especially where the topography is flattish and of savannah type.

v. Contains a certain content of coarse quartz to promote continuous porosity by providing a non-collapsible skeleton to prevent collapse of the structure and resulting decrease of porosity and rate of percolation.

In the words of Olusola (2005), variation in the level of involvement of these factors account for the wide varieties of laterite found in various parts of the world.

In the West Africa, a hard, pervious laterite crust is usually the climax of soil development. When a layer of laterite is removed by erosion, a new soil is exposed which is subject to another cycle of laterisation process.

2.1.3 Composition, properties and classification of Laterite

2.1.3.1 Composition

The chemical composition of typical lateritic soils as presented in Table 2.1 reveals high quantities of silica and iron oxide and relatively low aluminium content. Where aluminium content is presented in a relative large quantity, the laterite is referred to as bauxite or aluminous laterite. Where the iron content is higher and the aluminium content is not too low, that is, the iron content fairly predominates; such is referred to as ferruginous laterite. It can therefore be inferred from Table 2.1 that the chemical composition of laterite soils varies from one location to another. Laterite could also be identified visually by its colour, besides its chemical composition. The basic macroscopic characteristics of laterite are summarised below (Tietz, 1997).

a) **Hardness** – Highly variable, both within and between laterite deposits plastic, brittle, sectile and breakable between the fingers to difficult to break with a hammer.

b) **Colour** – Highly variable, although mostly reddish, reddish brown, brownish to yellow brown, but black, greyish or purplish blue and green may also occur.

Table 2.1: Chemical Composition of Typical Laterite Soils

Type	Mineral Constituents										
	Location	SiO_2	Al_2O_3	Fe_2O_3	FeO	MgO	CaO	Na_2O	K_2O	TCO_2	H_2O
Ferruginous bauxite	India	0.9	26.3	-	-	-	-	-	-	1.6	14.4
Bauxite detrital laterite	Madras	34.8	6.7	-	-	-	-	-	-	-	10.7
Laterite	Nigeria	26.5	19.9	36.7	-	-	-	-	-	1.1	-
Laterite	Australia	2.5	16.4	60.6	4.0	-	-	-	-	1.3	-
Laterite	Australia	2.6	4.3	80.0	-	-	-	-	-	-	6.1
Laterite or Genesis	Sudan	34.9	30.7	12.5	-	1.2	0.7	0.9	0.6	1.0	1.0
Laterite Over Ytachy-andesite	Jawa	37.0	28.8	8.9	-	0.7	0.4	2.9	-	1.0	13.4
Ferruginous Laterite	Ghana	23.9	16.7	43.6	0.3	0.2	0.2	-	-	1.1	13.6
Aluminium Laterite	Ghana	21.9	15.7	43.1	-	0.3	0.3	-	-	2.2	13.6
Bauxite Laterite	Ghana	0.7	59.7	8.5	-	-	-	-	-	3.3	27.9

Source: Maignien, 1966.

Table 2.2: Physical Components of Laterite

Component	% Composition
Gravel	5
Sand	48
Silt	12
Clay	35

Source: Adepegba, 1975a

c) **Grain size** – In crystalline Laterites, the grain size varies between < 0.1 and 2 mm; however, particle size for lateritic gravel can be greater than 25.4 mm (Gidigasu, 1976). Laterite may have coarse, medium and fine-gained texture.

d) **Fabric (structure / texture)** – Highly variable, from massive to even-gained and layered, but also with vermiform, scoriaceous, columnar and root-like structures.

e) **Chemical composition** – Highly variable, with Fe_2O_3 content between 1 and 60% and Al_2O_3 content between >60% (bauxite) and <10%.

f) **Mineralogical composition** – Gibbsite, goethite, hematite, maghemite, kaolinite, secondary quartz etc.

g) **Clay minerals** – May occur, but are not an essential component, kaolinites predominate.

h) **Others** – Unweathered rock-forming silicates (feldspars, hornblende, biolite etc.) may occur as relict minerals of the original parent rock.

According to Olusola (2005), the five chemical-mineral logical reactions stated in the definition of laterite as offered in section 2.1.1 do imply a high variability in the mineral composition and in the rock fabric of many laterites.

2.1.3.2 Other physical properties

Apart from colour and size distribution, other physical properties of laterite soil included Specific Gravity and Atterberg limits. Typical values of these properties obtained from the research work on geotechnical classification of lateritic soils in parts of Ile-Ife by Abidoye (1977) are listed as follows:

- Specific gravity 2.53 – 3.04
- Moisture content 12.2% - 43.5% when well compacted to soft silt clay.
- Atterberg limits
    - i) Liquid limit: 30.40 – 42.00
    - ii) Plastic limit: 11.84 – 28.57
    - iii) Plastic index: 8.77 – 24.14
- Dry density: 1.388 – 1.642
- Void ratio: 0.717 – 0.969
- Porosity: 0.432 – 0.475

Laterite, on basis of physical components, consists of gravel, sand, silt and clay at various percentages. A typical analysis carried out by Adepegba (1975a) on samples collected from about 32 kilometres outside Lagos yielded the results in Table 2.2

2.1.3.3 Classification

According to Gidigasu (1974), laterites are classified (for engineering purpose) as either "sensitive" or "stable". The sensitive laterites are generally found in the regions of recent volcanic activity; evaluation of their properties is unreliable. Consequently sensitive laterite is unstable for engineering purposes. Stable laterites are amenable to standard laboratory test and yield reproducible test values (Gidigasu, 1974; Adepegba, 1975a).

Laterites could also be classified as fine laterites and rock or quarry laterites in terms of particles sizes. In terms of chemical composition, it could be classified as normal laterite, ferruginous laterite, aluminous laterite and specialized laterites (e.g. those rich in manganese or nickel).

On morphological basis, lateritic soils could be classified as follows (Olusola, 2005):

a) **Massive Laterite:** These possess homogenous hard, not visible internal fabric and are divided into :
   i. **Vascular laterites** – Those containing cavities which are predominantly tabular.
   ii. **Cellular laterites** – Those containing cavities that are appropriate rounded or bubble-shaped.
b) **Soft Laterites:** These contains clay with high iron enrichment and hardens on exposure to air, this action could be reversed due to the actions of wetting and drying.
c) **Ferruginised rock:** These are laterites whose rock structures are visible with substantial isomorphous replacement by iron.

d) **Recemented laterites:** These contain massive laterite of ferruginous rock, loose, wholly or partially cemented.

e) **Nodular laterites:** A lateritic nodule is an irregular shaped rounded mass or lump, or a mineral or mineral aggregate, normally having a warty or knobby surface and no internal structure and essentially from the surrounding matrix in which it is embedded. The nodules are separated from one another by hardened ferruginous, or by sandy, clayey matrix. A nodular laterite may also contain a few lateritic pisoliths (a ferruginous lateritic practice resembling a pea in shape and ≤ 2 mm in diameter) and voids.

2.2 STABILIZED LATERITE: POSSIBILITIES AND LIMITATIONS

2.2.1 The possibilities

The term soil stabilization implies improvement in strength and durability of soil; this means that a less stable soil, after treatment, improves its strength and resistance to erosion and abrasion by a mass of water (Olusola, 2005). It can be of mechanized stabilization or chemical stabilization, the latter being the oldest and most widely used technique.

Research efforts in this area reveals attempts on the addition of different admixture called "chemical stabilizer" such as cement which is found to be the most effective (Ola, 1974; Ola, 1983), lime, fly ash, brick wastes, rice husk ash, bitumen for water proofing, corncob ash and combinations of these (Matawal, 2005; Ikpong & Okpala, 1992; Olateju 1991a).

Stabilisation with 8 to 12% cement had been recommended (Lasisi, 1977; Mesida, 1978; Aderibigbe *et al.,* 1983; Folagbade, 1998, Olusola & Folagbade, 2000). However, this level of cement consumption used to make the stabilised lateritic blocks competitive against sandcrete blocks. However, if in addition to using a chemical stabilizer, mechanical compaction is adopted; lateritic blocks of higher strength at lower percentages of cement stabilization are produced. Madedor and Dirisu (1991) as cited in Olusola (2005) reported the efforts made at Nigerian Building and Road Research Institute (NBRRI) and recommended 5% cement stabilization at 1.0 N/mm^2 compaction effort using NBRRI developed machine, with 7 days of curing. Olateju (1991a) used combination of cement, fly ash, lime and brick wastes to produce standard laterite blocks that met the Nigerian Standard Organization (NSO) minimum 28-day's strength requirement of 2.1 N/ mm^2.

Adesanya (2001) also discovered that the use of $\leq 5.8\%$ by weight of blended corncob ash cement as fillers in stabilized laterite for production of blocks improved their strength and thermal performance. Olateju (1991b) and Iyagba (1985) had previously used stabilized laterite to produce fibre-reinforced corrugated roofing sheets.

It is therefore obvious that stabilised laterite is a suitable material for production of masonry units, ceiling and roofing sheets. Stabilised laterite has also been reported to be used in construction of bungalows in Lagos State (FESTAC Town, Satellite Town, Ojo), Borno State (Maiduguri) and Uyo in Akwa Ibom State; by the Federal Housing Authority (for Lagos State) and the respective States Housing or Development Authority's (Omange, 1994). The more recent development in this regards is the use of Hydra-from laterite blocks in Kebbi, Ekiti and Kwara States for Housing Estates across the States.

2.2.2 The limitations

In spite of the breakthrough in the use of stabilized laterite, a missing gap is noted to exist from a critical review of the research works and reports. There is disparity in the range of strengths reported for different types of lateritic soils. Some researchers found a laterite / cement ratio of 3:1 as optimum for strength requirements while some other concluded that a ratio of up to 10:1 was optimum.

These limitations arise mainly from the varying factors: nature of parent rock material, degree of permeability of soil profile, amount of vegetation cover, degree of leaching, topography, differing external climates etc responsible for formation of laterite. Though cement has been found to be a good and effective stabilizer for laterite, the amount to be used will depend on the source, iron oxide content and grading of the laterite. In the works of Osunade (1984) and Lasisi and Osunade (1984), it was found that the finer the grain sizes of lateritic soils, the higher the compressive strength obtained. They further established that the possible formation processes form a factor in the strength determination, and that the compressive strength of lateritic soils is a function of the source(s) from where they were collected. Lasisi and Ogunjide (1984) hold the same view on the effect of grain size on the strength characteristics of cement stabilized lateritic soils.

Some works have been done in recent pasts on the effects of the parameters' enumerated above on Laterized Concrete (Osunade, 1994; Olusola, 2005; Ata *et al*., 2005; Ata, 2007). The studies reveal that laterite could produce concrete of higher grades than 10 MPa (10N/ mm^2) as opposed to submission by Neville (1995).

Olusola (2005) worked intensively on factors affecting compressive strength and elastic properties of Laterized concrete while Ata (2007) reports on the effects of varying curing age and water/cement ratio on the elastic properties of Laterized concrete. Not much has actually been done on effect of introduction of pozzolan (such as volcanic ash) on the strength properties and the behavioural properties of laterized concrete, this being the focus of this study.

2.3 POZZOLANAS AND THEIR CHARACTERISTICS

According to ASTM C125 – 05, a pozzolan is defined as a siliceous or siliceous and aluminous material, which in itself, possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (Neville, 2006). It is essential that pozzolan be in a finely divided state as it is only then that silica can combine with calcium hydroxide (produced by the hydrating Portland cement) in the presence of water to form stable calcium silicates which have cementitious properties. Neville (2006) further stated that the silica has to be amorphous, that is glossy, because crystalline silica has very low reactivity. Hossain (2005) submitted that these pozzolanic materials can improve the durability of concrete and rate of gain in strength thus reducing the rate of liberation of heat, which is beneficial for mass concrete.

The resurgence of the ancient concept of combining different materials in a composite material is a result of numerous and diverse requirements imposed on construction materials in a bid to satisfy the diverse user requirements (Raheem, 2006). The materials mainly used in this connection are those possessing pozzolanic properties (Dias and Thaumaturgo, 2005). The use of pozzolanic materials is as old as the art of concrete construction. This was due to the fact that the use of suitable pozzolan in appropriate quantity modifies certain properties of fresh and hardened mortar and concrete.

Ancient Greeks and Romans used volcanic ash or tuff found near Pozzuoli (Italy) in the construction of aqua ducts, arch, bridges etc. It was observed that the long-term strength and durability of concrete containing slag exceeds that of normal Portland cement concrete (Ramezanianpour and Malhotra, 1995; Khatib and Hibber, 2005).

Gbrahgm, *et al.,* (2003) stated that with only a five percent addition of silica fume substituted for Portland cement, the concrete produced thereof is rendered impermeable to harmful chemicals thereby increasing the life span of the concrete. The use of Metakaolin as a pozzolan was observed by Qian and Li (2001) to increase resistance of concrete to alkaline-silica reaction. Silica fume (SF) and Metakaolin (MK) pozzolan was also found by Mohammed and Sayed (2006) to cause an increase in the compressive strength of blended cements with temperature increase up to 400^0C and that the replacement of Portland cement by 15% MK and 5% SF in cement pastes increase the thermal shock resistance by about 10 times. Siddique (2004) was also quoted by Raheem (2006) to have reported that class F fly ash can be suitably used up to 50% level of cement replacement in concrete for use in precast elements and reinforced concrete construction.

Raheem (2006) thereby deduce that the use of pozzolan in optimum proportion with Portland cement enhanced the following qualities of concrete:

a) Reduction in heat of hydration and thermal shrinkage
b) Increase in water tightness
c) Improved resistance to chemical attack
d) Lower susceptibility to dissolution and leaching
e) Improved workability and
f) Reduction in cost.

Although the addition of pozzolan to Portland cement does not contribute to the compressive strength of concrete at early ages, strengths similar to those of ordinary Portland cement can be expected at later ages provided the cement is cured under moist conditions for a sufficient period (Bhanja and Senguptab, 2002a).

2.3.1 Chemical composition of pozzolan

Pozzolanas are a class of material that combines with calcium hydroxide and water to produce calcium silicate hydrate (CSH), which is the glue in Portland cements. The chemical composition and pozzolanic activities of those materials vary depending on the source (Pekmezci and Akyuz, 2004). However, there is a common denominator for any material to qualify as a pozzolan.

Syagga *et al.*, (2001) cited in Raheem (2006) states that the Kenya Standard (KS-02-1261) recommends that a good pozzolan for manufacture of pozzolanic cement should have a combined $SiO_2 + Al_2O_3$ of at least 70%. The ASTM C618-2008 on the other hand requires that a good pozzolan should have a combined percentage of $SiO_2 + Al_2O_3 + Fe_2O_3$ of more than 70%. Similarly, the Indian Standard 1344: 1968 also stipulated that a pozzolan should have a combined silica (SiO_2), alumina (Al_2O_3) and iron oxide (Fe_2O_3) composition of not less than 70% of the entire constituents with silica alone not having a composition of less than 35% (Shetty, 2004). Table 2.3 shows the chemical composition of a pozzolan according to Indian Standard while Table 2.4 also presents the physical and chemical requirement for pozzolanas according to American Standard (ASTM C618 - 2008).

Table 2.3: Chemical Composition of Pozzolan according to Indian Standard

S/ N	Characteristic	Requirement
i)	Silicon dioxide (SiO_2) plus aluminium oxide (Al_2O_3) plus iron oxide (Fe_2O_3) percent by mass, Min	**70.0**
ii)	Silicon dioxide (SiO_2), percent by mass, Min	**35.0**
iii)	Magnesium oxide (MgO), percent by mass, Max	**5.0**
iv)	Total sulphur as sulphur trioxide (SO_3), percent by mass, Max	**2.75**
v)	Available alkalis, as sodium oxide (Na_2O), percent by mass, Max	**1.5**
vi)	Loss on ignition, percent by mass, Max	**12.0**

Source: Shetty, 2004

Table 2.4: Physical and Chemical Requirement for Pozzolan as offered by ASTM

S/N	Property of Pozzolan	ASTM Requirement (%)
i)	Water-soluble fraction	10.0
ii)	Fines: Amount retained when wet sieve	

	Number 30 Sieve(600μm) Max% Number 200 Sieve(75μm) Max%	2.0 30.0
ii)	Drying shrinkage (Max %)	0.15
iii)	Increase in drying shrinkage of Portland pozzolan cement mortar bars at 28days (Max %).	0.03
iv)	Water requirement (Max % of control)	115
v)	Silicon dioxide(SiO_2) + Aluminium oxide (Al_2O_3) + Iron oxide (Fe_2O_3)	70
vi)	Pozzolanic activity index with Portland cement at 28days (Max. % of control)	75
vii)	Magnesium Oxide (MgO) Max. %	5.0
viii)	Loss of ignition (LOI) Max. %	12.0

Source: ASTM C618-2008

It can therefore be observed from above discussions, that for any material to qualify as a pozzolan, it must have these main components of Silica (SiO_2); Alumina (Al_2O_3) and Iron Oxide (Fe_2O_3) whose combined composition should not be less than70% of the entire constituents. Also, silica has the highest composition of at least 35%.

Thus, an essential quality of a pozzolan is that it must contain large amounts of Silica and Alumina in a suitably reactive form, so that it can react with Calcium Hydroxide (Knofel, 1983). This study considers the determination of the chemical composition of volcanic – ash in order to ascertain its compliance with the requirements above.

2.3.2 Classification and uses of pozzolan

Pozzolanic materials can be classified into two groups:

a) Natural pozzolanas

b) Artificial pozzolanas

Natural pozzolanas are of volcanic origin with volcanic ash referred to by Neville (2006) as the original pozzolan. Others include pumicite, tuff, trass, opaline shale and cherts, calcined diatomacceous earth and burnt clay; they are described by ASTM C 618 – 2008 as class N. According to Neville (2006), some natural pozzolanas may create problems because of their physical properties e.g. diatomaceous earth, because of its angular and porous forms, requires high water content. He further stated that certain natural pozzolanas improves their activity by calcinations in the range of 550 to 1100^{o}C, depending on the material.

Artificial pozzolanas are mainly products obtained by heat treatment of natural materials. Examples are fly ash (PFA), Blast furnace slag, Silica fume, Metakaolin, Rice husk ash (RHA), Saw dust ash (SDA), Acha husk ash (AHA), Bagasse ash, Groundnut husk ash (GHA) and Corn cob ash (CCA) as mentioned in Matawal (2005), Alabadan *et al.,* (2006), Raheem (2006) and Neville (2006).

Pozzolan is mainly used as an admixture in concrete or as a constituent of blended or pozzolan cement. As an admixture, the specific pozzolan is incorporated into the concrete materials as percentage replacement of ordinary Portland cement during the process of concrete production. As a constituent of blended or pozzolan cement, the particular pozzolan is interground with Portland cement clinker during the cement manufacturing process. This research adopts mixing the volcanic ash thoroughly with Portland cement before the cementitious material is mixed with other constituents of the Laterized concrete.

According to Matawal (2005), whatever is the process of obtaining the pozzolan, coalition of research studies (Neville, 1992; Ikpong, 1990; Swamy, 1987; Okpala 1987) indicates that pozzolanas produce concrete with similar characteristics at ages beyond 28 days. Pozzolanic concrete are said to also exhibit better resistance to Sulphate attack and they reduce permeability which consequently improves water tightness (Dunstan, 1984 and Mehta, 1993). Pozzolanic materials may however result in concrete of lower strength in the early ages (Popovic, 1986).

Hassan (2006) quoting Matawal (2005) discussed the reasons for employment of ashes to clinker in cement under three headings:

i) Technological
ii) Economical and
iii) Environmental

Technologically, it can modify the properties of cement by increasing or decreasing its durability and resistance to aggressive agents as well as to lime (Talero, 1990; Gaspar and Sagrera, 1987). Improved behaviour is a function of the activity of the additions and this varies from one pozzolan to another.

Economically, active additions reduce the quantity of cement required, while environmentally, employing such additions, utilizes waste materials.

Other advantages of using pozzolanic material as partial replacement of cement highlighted by Matawal (2005) are as follows:

- Improved placeability or workability: a vital consideration in the assessment of fresh concrete;
- Improved sulphate resistance particularly in marine environment;
- Improve resistance to freezing and thawing in temperate environment;
- Increased cohesiveness or bonding strength of the concrete;
- In a few instance, there is an increased long-term strength;
- A reduction in the water content of mortar and concrete mixes resulting in less shrinkage and cracking;
- A reduction in the heat of hydration: a particularly potent advantage in hot weather concreting;
- Decreased permeability and water tightness;
- High resistance to alkaline-aggregate reactions.

These advantages vary from one pozzolanic material to another; a detailed discussion on the pozzolan of concern to us (volcanic ash) is thereby of great importance.

2.4 VOLCANIC ASH

Volcanic ash is a finely fragmented magma or pulverised volcanic rock, measuring less than 2 mm in diameter, that is emptied from the vent of a volcano in either a molten or solid state. The most common state of ash is vitric, which contains glassy particles formed by gas bubble busting through liquid magma (Encarta, 2008).

In the words of Shoji, *et al.*, (1993), volcanic ash comprises small jagged piece of rock minerals and volcanic glass that was erupted by a volcano. Volcanic ash is opined not to be a product of combustion like soft fluffy material created by burning wood, leaves or paper. Volcanic ash is hard, does not dissolve in water and is extremely abrasive, mildly corrosive and conducts electricity when wet. In their opinion, the average grain size of rock fragment and volcanic ash erupted from an exploding volcanic vent varies greatly among different eruption. Heavier and large size rock fragment typically fall back to the ground or close to the volcano while smaller and lighter fragments are blown farther from the volcano by wind.

2.4.1 Compaction and Density of Ash Deposits.

Shoji, *et al.*, (1993) argues that ash particles will compact close together after they fall to the ground. The compaction will increase the bulk density of an ash deposits sometimes as much as 50% within a few weeks of eruption. The thickness of ash deposited may correspondingly decrease slightly over time.

Volcanic ash is made of different particles i.e. pumice fragments, volcanic glass shards, crystals and minerals and other rock fragment; the density of the particle are as give in Table 2.5.

Table 2.5: Density of Individual Ash Particle

Types of particle	Density of particle
Pumice fragment	700 – 1200 kg/m^3

Volcanic glass shards	2350 – 2450 kg/m^3
Crystal and minerals	2700 – 3300 kg/m^3
Other rock fragments	2600 – 3200 kg/m^3

Source: Shipley and Saran-Wojcicki, 1982

The density of any ash fall deposits can be variable, with reported dry bulk densities of newly fallen and slightly compacted deposit ranging from 500 and 1500 kg/m^3; bulk density of wet ash ranges between 1000 and 2000 kg/m^3 (Shoji, *et al.*, 1993) as cited in Matthew (2007).

2.4.2 Volcanic Rocks in Nigeria

Wright (1970) observe that although a significant proportion of Nigeria's volcanic rock are found in the Jurassic younger granite province, the tertiary to quaternary phase of volcanism (the process by which molten rock or magma rises from interior of the earth to or toward its surface and by which associated gases are released to the atmosphere) was most mid spread and voluminous in Nigeria. There are also other volcanic episodes which observe wider publicity. A summary of the spread is as provided in Table 2.6 as cited in Hassan (2006).

Table 2.6: Summary of Volcanic Rocks in Nigeria

Approximate Age	Petrographic Affinity	Approximate Distribution (cf. fig 2.3)
Jurassic (150Ma)	Alkaline to Tholeitic	Basaltic half of Runka (1), also Gazamma (G), Kinberlite of Kafur (K) and clays at Kankara (K)
Cretaceous (100Ma)	Alkaline to Calc Alkaline	Basic to intermediate laxias and proclactic and minor intrusive of Benue trough (2)
Lower Cenozoic (70 - 60Ma)	Rock too altered	Fluvio-volcanic series or laterized older basalts of Jos Plateau region (3)
Upper Cenozoic	Alkaline to per Alkaline	Basalts phlomolites, trachytes of Jos Plateau, Benue valley and Manbilla Plateau (4)

Source: Wright, 1970

Salau (2008) also outlined the spread of Basalt formations (the parent material from which volcanic-ash forms) in Nigeria. According to Salau (2008), basalt formations are found in the

South and West of Biu Plateau, Namu, Gindiri, Pankshin and Runka areas and also in Jos Plateau in Plateau State.

They also occur in Rabah, Gwaini, Wurno and Sokoto Plateau of Sokoto State. Traces of basalt can also be found in the Yoruba Plateau (Salau, 2008). This study hereby focuses on the Jos Plateau Volcano.

2.4.3 The Geology of Jos Plateau Volcano

The Jos Plateau lies precisely within the North Central Basement Complex of Nigeria (Fig.2.3). The Basement Complex rocks of the lower Palaeozoic to Precambrian ages underlie about half of its entire landmass. These rocks are represented by gneiss-migmatites and intrusive into these Basement rocks are the Pan-African granites and the predominant Jurassic non-organic alkaline Younger Granites (Turner, 1976).

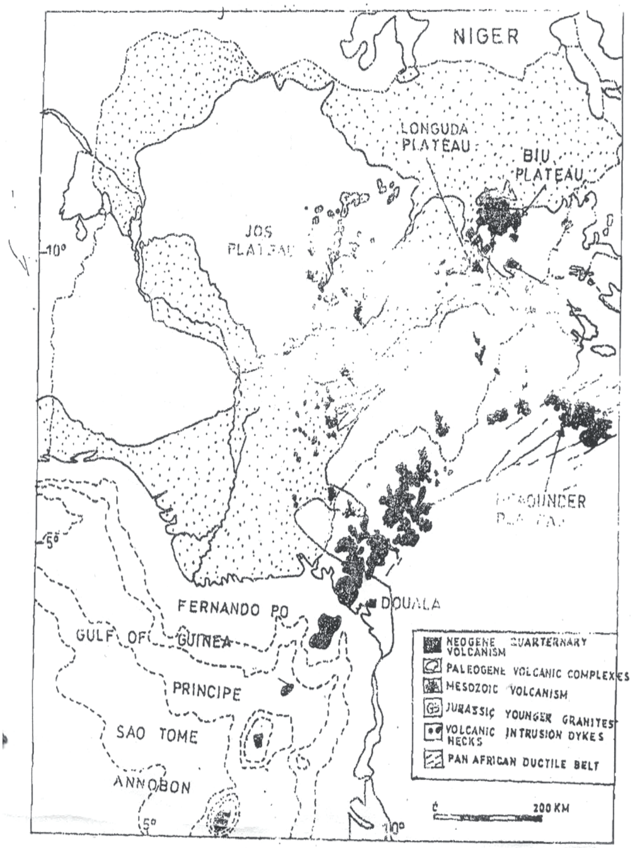


Figure 2.3: Map of Location Cenozoic Volcanism Showing Location of Jos Plateau

Source: Lar and Tsalha, 2005

Tertiary and Quaternary basaltic volcanoes are the youngest rocks in the area and overlie directly in the basement and in places of the Younger Granites (Wright, 1970). Two main basalt subtypes have been distinguished based on these periods of replacement and textural differences. They are the Older (Tertiary) and the Newer (Quaternary) basalts (MacLeod *et al.*, 1971) as quoted in Lar and Tsalha (2005).

The Newer basalts occupy nearly 150 km^2 in the western and southern Jos Plateau. They also extend towards the Kafanchan area and Southwards down to the Shemankar valley. They occur as cones and lava flow characterised sleep-sided central craters rising a few meters above their surroundings.

The Newer Basaltic cones are aligned in NNW-NNE direction, corresponding to the trend of dolerite dykes (MacLeod *et al.*, 1971). They are mainly built of basaltic scoria and pyroclastics, with the vesicles filled with a variety of inclusions (olivine, Iherzolite, websterite etc).

Partly decomposed basaltic boulders, plugs or dome-like out-crops represent the Older Basalts. They are very visible from the Werram valley southward of Jos extending to the Keffi-Abo area to Rukuba, Ganawuri, South Ropp, Mbar and Mangu. The laterized basalt represents the product of weathering of mainly the Older Basalts (MacLeod *et al.*, 1971)

2.4.4 Chemistry of Volcanic Ash

Volcanic Ash chemistry is directly related to the chemistry of the source magma. Volcanic glass is relatively high in silica compared to mineral crystals, but relatively low in non-silica elements (especially Mg and Fe). Both glass and most minerals almost always contain Si, Al, K, Na, Ca, Mg, & Fe (Shoji *et al.*, 1993).

Lar and Tsalha (2005) present the result of chemical analysis of the Jos Plateau Basalts as shown in Table 2.7 with the SiO_2 content ranging between 39.8 to 46.49 wt.%, a total SiO_2 + Al_2O_3 + Fe_2O_3 content ranging between 53.13 to 71.07 wt.%. The sample taken from Kerang environments (KG1) has a total SiO_2 + Al_2O_3 + Fe_2O_3 content of 63.74 wt. % by the analysis. Hassan (2006) on other hand present a report of analysis of sample taken from Kerang having total SiO_2 + Al_2O_3 + Fe_2O_3 content of 67.14 wt. % as shown in Table 2.8. This study is thereby investigates the chemical constituents of the sample from Kerang and assess its suitability as a Pozzolan.

Table 2.7: Major Elements (w %) Abundances in Basaltic Rocks from Jos Plateau

Wt.	KG1	HP2	AM1	AH1	RY2	RY1	VM1	KS1	HP1	RH1	GS1	APW1	KS2	KS3
SiO_2	39.64	40.9	45.89	44.94	46.38	46.49	42.37	38.85	40.43	42.58	29.97	39.08	40.46	42.75
Al_2O_3	11.18	13.11	12.4	14.4	14.15	13.99	14.1	15.85	13.45	14.04	14.28	14.44	13.67	13.53
Fe_2O_3	12.92	9.93	10.36	9.75	10.28	9.79	12.66	12.68	13.85	13.86	12.88	13.63	12.37	12.18
TiO_2	2.52	2.43	2.71	2.37	2.62	2.84	2.36	2.36	2.39	2.39	2.89	2.56	2.51	2.47
CaO	10.43	0.77	8.57	9.72	8.53	8.64	9.71	9.71	9.78	8.78	8.85	10.92	10.29	10.66
MgO	18.79	21.66	17.82	16.3	14.98	15.3	15.87	15.87	16.48	16.48	28.44	15.88	17.56	15.33
MnO	0.08	8.07	0.09	0.07	0.07	0.07	0.02	0.09	0.07	0.07	0.08	0.07	0.08	0.08
K_2O	1.64	1.64	0.97	1.42	1.67	1.47	1.96	1.86	1.96	1.84	0.87	1.84	1.50	1.55
P_2O_5	0.48	0.62	0.57	0.48	0.48	0.46	0.35	0.76	0.68	0.61	0.44	0.71	0.54	0.54
SO_3	0.02	0.01	0.04	0.01	0.02	0.02	0.01	0.05	0.02	0.02	0.12	0.02	0.11	0.08
V2O	0.00	0	0.02	0.02	0	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.00	0.02
Na_2O	0.95	0.62	0.32	0.77	0.76	0.85	0.36	0.88	0.76	0.63	0.86	0.63	0.75	0.85
Cr_2O	0.04	0.03	0.02	0.03	0.03	0.03	0.03	0.03	0.02	0.02	0.03	0.20	0.03	0.03
Total%	98.69	99.79	99.78	100.28	99.97	99.97	99.82	99.01	99.91	101.34	99.73	100.00	99.87	100.07

Legend: KG1=Kerang; HP1&2=Heipang; AM1=Amper; AH1=Assop Hausa; RY1&2=Riyom; VM1=Vom; KS1,2&3=Kassa; RH1=Richa; GS1=Gumshir; APW1=Ampang West

Source: Lar and Tsalha (2005)

Table 2.8: Chemical Composition of Volcanic Ash from Kerang

Elements	**% Composition by weight**
SiO_2	48.75
Al_2O_3	16.26
Fe_2O_3	2.13
CaO	11.67
MgO	4.24
K_2O	5.71
Na_2O	3.83
P_2O_5	0.81
L.O.I	2.71
Total SiO_2+Al_2O_3+Fe_2O_3	67.14

Source: Hassan (2006)

2.5 CONCRETE AS A STRUCTURAL MATERIAL

Concrete in the broadest sense, is defined as any product or mass made by the use of a cementing medium (Neville and Brooks, 2002). This medium is referred to in general as the product of reaction between hydraulic cement and water. Its basic composition are Ordinary Portland Cement (OPC), water and aggregates (fine aggregate (sand) and conventional coarse aggregates such as gravel and granite). According to Abdullah, *et al.,* (2006) continuous research has resulted in production of many types of concrete known in various name; each having unique characteristic to fulfil the current Construction Industry demand. Today other types of cement, fine aggregates and coarse aggregates are being used. New cement types in the form of blended cements are mixtures of normal cement and cementitious materials like fly ash, Blast furnace slag, Rice husk ash, Corn cob ash and other pozzolanas (Olusola, 2005). Attempt to improve one or more properties of concrete have resulted in emergence of additives like admixtures of different kinds, polymers and fibres in conventional concrete. Fibre-reinforced concrete is an example of this modification.

Substitute to conventional sand as a fine aggregate in concrete include use of laterite and quarry granite grains among others. Hence the emergence of laterized concrete; that is concrete in which sand has been partially or wholly replaced by laterite as fine aggregate. This study examines the effect of volcanic-ash's (a natural pozzolan) introduction on the strength characteristics of laterized concrete.

The satisfactory performance of concrete in structures presupposes that it possess adequate characteristics in both the fresh and hardened state. In its fresh state it implies:

i. The mix is cohesive enough to be transported and placed without segregation.

ii. The consistence of the mix ensures that the concrete can be adequately compacted by the available means on the site.

In the hardened state, satisfactory performance has to do with the concrete having a satisfactory compressive strength, an indicator of the quality of the concrete. Thus Neville (2006), states that compressive strength of concrete is an easy way of ascertaining compliance with the specification. It was thereby argued that compressive strength enjoys prominence because other properties of concrete e.g. density, tensile strength, durability, resistance etc- are related to it. This study therefore investigates the following properties of volcanic ash blended cement laterized concrete:

i. Workability for fresh state and
ii. Compressive strength, flexural strength and Tensile Splitting strength, in the hardened state.

2.5.1 Workability Characteristics

Workability is one of the most important characteristics of fresh concrete. It is a term used to describe qualitatively, the ease with which concrete can be mixed, placed, compacted and finished. Workability of concrete is said to be intimately related to:

a) **Morbidit**y, which is the property that determines how easily the concrete can flow into the moulds and around the reinforcement.

b) **Stability**, The property which determines the ability of the concrete to remain stable and coherent mass during handling and vibration and

c) **Compatibility**, the property which determines how easily it can be compacted to remove air voids (Kong & Evans, 1987).

The various factors that has been identified in literature affecting the workability of concrete include water content, aggregate type and characteristics including grading, aggregate /

cement ratio, presence of admixtures, length of time of mixing, delay of casting after a fixed mixing duration and other ambient conditions such as humidity and wind velocity (Kayyali, 1984; Kakizaki, 1992; Domone, 1994; Jackson and Dhir, 1996; Neville and Brooks, 2002; Shetty, 2004).

Workability can be measured using two point tests (Domone, 1994) and single point test; the latter, though less efficient are more popular and commonly used. The single point test includes Slump test, Compacting Factor test, Vebe test and Flow Table test. The procedures are fully discussed respectively in the following British Standards BS EN 12350, Parts 2, 3, and 4 (2000). This research adopts the use of the Slump test only due to limitations posed by equipment availability.

2.5.2 Compressive Strength of Hardened Concrete

Strength of a hardened concrete usually gives an overall picture of the quality because it is directly related to the structure of the hydrated cement paste. The strength of concrete is invariably a vital element of structural design for compliance purpose (Neville, 2006). The strength of concrete is defined as the maximum stress it can carry. As the strength of concrete increases, its other properties usually improve (Jackson and Dhir, 1996).

Once concrete has hardened, it can be subjected to a wide range of tests to prove its ability to perform as planned or to discover its characteristics if its history is unknown. For new concrete this usually involves casting specimens from fresh concrete and testing them for various properties as the concrete matures. The “concrete cube test” is the most familiar test and is used as the standard method of measuring compressive strength for quality control purposes. Concrete beam specimens are cast to test for flexural strength and cast cylinders can be used for Tensile Splitting strength. Specimens for many other tests can be made at the same time to asses other properties, e.g. drying shrinkage, thermal coefficient, and modulus of elasticity. Test on existing concrete samples on the other hand can be by cutting smaller samples from the structure (smaller precast units can be tested as found) or the Non – destructing testing (NTD) approach.

The "strength" of hardened concrete is its ability to resist strain or rupture induced by external forces. The resistance of concrete to compressive, tensile and bending stresses is known as compressive strength, tensile strength, and bending (or flexural) strength respectively. The resistance of concrete to repeated stresses is called fatigue strength. Strength is expressed in terms of N/ mm^2 or MPa. Compressive strength test results are primarily used to determine that the concrete mixture as delivered meets the requirement of the specified strength in the job specification (CIP, 2003). The standard test requirement and the procedure for laboratory determination of compressive strength of concrete cube specimens are listed in BS EN 12390: Part 3 (2000). The standard cube size re commended is 150 mm; though 100 mm cube size could be used (Neville & Brooks, 2002; Shetty, 2004; Neville, 2006).

The compressive strength is taken as the maximum compressive load it can carry per unit area. This is calculated as follows:

$$\sigma_c = P_{\max} / A \text{ .... (2.0)}$$

Where

$\sigma_c =$ Compressive Strength (N/ mm^2)

$P_{\max} =$ Magnitude of the load that causes breaking (N)

$A =$ Cross-section area of the specimen (mm^2)

If a cube is used, the cube compressive strength f_{cu} is given by

$$\sigma_c = P_{max} / A \quad \text{.......................... (2.1)}$$

If a cylindrical specimen of diameter d is used, the cylinder compressive strength f'_c is given by

$$\sigma'_c = 4P_{max} / \pi d^2 \quad \text{................................ (2.2)}$$

Concrete incorporating pozzolanic materials have been known to gain strength gradually, especially during the early ages. This is because pozzolanic reaction at room temperature is slow; therefore a long-curing period is needed to observe its positive effects. This study therefore investigates the curing of laterized concrete cube specimens for up to 120 days, so as to allow more time for pozzolanic reaction.

2.5.3 Tensile Strength Characteristics

The tensile strength of concrete is of importance as it has a great influence on the serviceability limit state of cracking. The knowledge of tensile strength is of value in estimating the load under which cracking will develop. Cracking problems occur when diagonal tension arising from shearing stresses develops, especially, as a result of destined shrinkage and temperature gradients (Neville, 2006). The absence of cracking is of considerable importance in maintaining the continuity of a concrete structure and in the prevention of corrosion of the embedded reinforcement due to ingress of water.

The incorporation of pozzolanic materials into concrete has been identified as a means to enhance its tensile properties. Previous studies by Bhanja and Senguptab (2002b) and Almusallam *et al.,* (2004) indicated that the incorporation of silica fume in concrete results in significant improvements in the tensile strengths of concrete. Chaowat (2001) also stated that partial replacement of ordinary Portland cement by rice husk ash (RHA) increase the tensile strength of concrete.

The tensile strength of concrete can be experimentally determined using the following three methods:

a) Uniaxial tensile test i.e. direct tensile
b) Split cylinder test i.e. Tensile Splitting and
c) Beam test in flexure or third – point loading test.

The first method is referred to as direct test for determining tensile strength. The direct measurement of tensile strength is rarely carried out because the direct application of a pure tension force, free from eccentricity, is very difficult and is further complicated by secondary stresses induced by grips or the embedded studs (Luong, 1990; Exadaktylos *et al.,* 2001; Osunade 2002 and Neville, 2006). The second and third methods are called indirect tests and are the ones commonly used to estimate tensile strengths.

The Tensile Splitting was developed by a Brazilian engineer hence the tag "Brazilian test". The main advantage of the test according to Rocco *et al.*, (2001) is that only external compressive loads are required. A cylinder or prismatic specimen is compressed along two diametrically opposed generators so that a nearly uniform tensile stress is induced in the loading plane. To prevent local failure in compression at the loading generators, two thin strips, usually of plywood are placed between the loading platens and the specimen to distribute the load. The Tensile Splitting strength (Fsp) is given by Zhou *et al.*, (1998) and Osunade *et al.*, (1990) as:

$$Fsp = 2P/\pi dl \qquad (2.3)$$

Where:

P = compressive load on the cylinder

d = diameter of the cylinder and

l = length of the cylinder.

If cubes, BS EN 12390 – 6:2000 gives

$$Fsp = 2P/\pi a^2 \quad (2.4)$$

Where 'a' represents the cube's side

The beam test in flexure sometimes called 'Third – Point Loading System' is mainly used for rectangular beam specimens. The tensile strength (modulus of rupture, fbt) is given by Neville (2006) and Shetty (2004) as:

$$Fbt = Pl/bd^2 \quad (2.5)$$

Where:

P = maximum total load on the beam

l = length of the beam (span)

b = width of the beam

d = depth of the beam.

If the fracture occurs within the middle third: But the test should be discarded if fracture takes place outside the middle one-third (Neville & Books, 2002 quoting BS 1881: Part 117, 1983).

The splitting test is simple to perform and as stated by Neville (2006), gives more uniform results than other tension tests; this study adopts both the Tensile Splitting strength and the flexural tensile strength tests. The outcome will be used to establish a relationship between the concrete's Compressive Strength and Tensile Strength values obtained.

2.5.4 Relation between Compressive and Tensile Strengths of Concrete

The compressive strength of concrete is its property commonly considered in structural design but for some purposes; the tensile strength is of interest. Even though it is expected that these strengths be related, there is yet no direct proportionality, the ratio of the two strengths depending on the general level of strength of the concrete. That is, as the compressive strength, f_c, increases, the tensile strength, f_t, also increases but at a decreasing rate. Factors affecting the relation between the two strengths are:

(a) The effect of crushed coarse aggregate on the flexural strength (beneficial).
(b) The properties of fine aggregate also influence the f_t/f_c ratio.
(c) Grading of the aggregate.
(d) Age; beyond one month, the f_t increase more slowly than the f_c so that f_t/f_c decreases with time (Neville, 2006).

The tensile strength of concrete can be measured by radically different tests namely: flexure, direct tension and splitting, and resulting value strength are not the same. Consequently, the numerical value of the ratio of strength is not the same. The tensile strength of concrete is more sensitive to inadequate curing than the compressive strength (Neville, 2006) possibly because the effects of non-uniform shrinkage on flexure test beams are very serious.

Lightweight concrete conforms broadly to the pattern of the relation between f_t and f_c for ordinary concrete. A number of empirical formulae connecting f_t and f_c have been suggested, many of them of the type

$$f_t = k\,(f_c)^n \quad \text{.......} \quad (2.6)$$

k and n are coefficients. Values of n between ½ and ¾ have been suggested while k varies from 6.2 for gravels to 10.4 for crushed rock while average value is 8.3 (Shetty, 2004). Probably, the best according to Neville, 2006 is given by

$$f_t = 0.3(f_c)^{2/3} \quad \text{.........} \quad (2.7)$$

f_t = splitting strength; f_c = compressive strength of cylinders.

The expression used in BS 8007: 1987 is

$$f_t = 0.12(f_c)^{0.7} \quad \text{.........} \quad (2.8)$$

bearing in mind that the f_c is determined on cubes (MPa); f_t represents the direct tensile strength.

Also, since crushed coarse aggregate seems to improve tensile strength more than it does compressive strength, the ratio of split tensile to compressive strength (f_{ct}/f'_c) also depends on the type of aggregate. In general, this ratio ranges from 0.08 to 0.14. The actual relationships between tensile and compressive strengths vary widely and exhibit significant scatter (Mindess *et al.*, 2003). The differences in aggregate surface texture influence the paste-aggregate bond strength; this seems to control the overall tensile strength (Cetin and Carrasquillo, 1998). Also, the overall Tensile Splitting strength of concrete tends to increase as the Tensile Splitting strengths of the aggregates increase (Wu *et al.*, 2001). However, such relationship is scarce in literature for volcanic ash blended cement laterized concrete.

2.5.5 Factors Affecting Concrete Strength

There are many factors affecting the strength of concrete. Some of the most important are as follows:

a) **Concrete porosity:** voids in concrete can be filled with air or with water. Air voids are an obvious and easily-visible example of pores in concrete. Broadly speaking, the less porous the concrete, the stronger it will be as measured by compressive strength. Probably the most important source of porosity in concrete is the ratio of water to cement in the mix, known as the "water to cement" ratio. This parameter is so important and will be discussed separately below.

b) **Water/cement ratio:** this is defined as the mass of water divided by the mass of cement in a mix. For example, a concrete mix containing 400 kg cement and 240 litres (=240 kg) of water will have a water/cement ratio of 240/400=0.6. The water/cement ratio may be abbreviated to "w/c ratio" or just "w/c". In mixes where the w/c is greater than approximately 0.4, all the cement can, in theory, react with water to form cement hydration products. At higher w/c ratios it follows that the space occupied by the additional water above w/c=0.4 will remain as pore space filled with water, or with air if the concrete dries out.

   Consequently, as the w/c ratio increases, the porosity of the cement paste in the concrete also increases. As the porosity increases, the compressive strength of the concrete will decrease.

c) **Soundness of aggregate:** it will be obvious that if the aggregate in concrete is weak, the concrete will also be weak. Rocks with low intrinsic strength, such as chalk, are clearly unsuitable for use as aggregate.

d) **Aggregate-paste bond:** the strength of the bond between the paste and the aggregate is critical. If there is no bond, the aggregate effectively represents a void; as discussed above, voids reduce the strength of concrete.

e) **Cement-related parameters:** many parameters relating to the composition of the individual cement minerals and their proportions in the cement can affect the rate of strength growth and the final strengths achieved. These include:

(i). Alite content
(ii) Alite and belite reactivity
(iii) Cement sulphate content

Since alite is the most reactive cement mineral that contributes significantly to concrete strength, more alite should give better early strengths ('early' in this context means up to about 7 days). However, this statement needs to be heavily qualified as much depends on burning conditions in the kiln. It is possible that lighter burning of a particular clinker could result in higher early strength due the formation of more reactive alite, even if there is a little less of it. Not all alite is created equal!

For particular cement, there will be what is called an “optimum sulphate content”, or “optimum gypsum content”. Sulphate in cement, both the clinker sulphate and added gypsum, retards the hydration of the aluminates phase. If there is insufficient sulphate, a flash set may occur; conversely, too much sulphate can cause false-setting.

A balance is therefore required between the ability of the main clinker minerals, particularly the aluminates phase, to react with sulphate in the early stages after mixing and the ability of the cement to supply the sulphate. The optimum sulphate content will be affected by many factors, including aluminates content, aluminates crystal size, aluminates reactivity, solubility of the different sources of sulphate, sulphate particle sizes and whether admixtures are used.

If this were not already complicated enough, the amount of sulphate necessary to optimize one property, strength for example, may not be the same as that required to optimize other properties such as drying shrinkage. Concrete and mortar may also have different optimum sulphate contents.

In addition to the compositional parameters considered above, physical parameters are also important; particularly cement surface area and particle size distribution.

The fineness to which the cement is ground will evidently affect the rate at which concrete strengths increase after mixing. Grinding the cement more finely will result in a more rapid increase in strength. Fineness is often expressed in terms of total particle surface area, e.g. 400 square meters per kilogram. However, of as much, if not more, importance is the particle size distribution of the cement; relying simply on surface area measurements can be misleading. Some minerals, gypsum for example, can grind preferentially producing cement with a high surface area. Such cement may contain very finely-ground gypsum but also relatively coarse clinker particles resulting in slower strength development.

2.6 2.6 STRENGTH CHARACTERISTICS OF LATERIZED CONCRETE

Laterized concrete is defined as concrete in which stable laterite fines replace sand wholly or partially, whole replacement is also referred to as terracrete (Olusola, 2005). Neville (2006) reported that laterite when used to wholly replace sand in concrete can rarely produce concrete stronger than 10 MPa (10 N/mm2). Report of studies by Osunade (2003), Ata (2003) and Olusola (2005) has proved this not to be true; they submitted that Laterite can produce concrete of higher grades.

Adepegba (1975a) was the first to consider the possibility of replacing sand in concrete with laterite in Nigeria. He studied the effect of using laterite fines instead of sand in relation to the density, compressive strength, tensile strength, modulus of elasticity and resistance to exposure to high temperature. He concluded that their properties fared well in comparison with those of normal concrete, thereby offering that Laterite fines in place of Sand can be used for structural members.

Research works by Balogun and Adepegba (1982) discovered that the most suitable mix of Laterized concrete for structural propose is 1:1.5:3 using batching by weight with a water/cement ratio of 0.65, provided that the Laterite content is kept below 50% of the total aggregate content. The w/c used conforms to the recommendation of Lasisi and Ogunjide (1984) who obtained a linear relationship between the optimum w/c ratio (X) and the laterite-cement ratio (Y). The equation was given as

$$Y=-0.9+3.85X \qquad (2.9)$$

Chandrakaran *et al.*, (1996) also reported that for fully laterized concrete, the compressive strength is 50% of that of ordinary concrete. Lasisi *et al.*, (1990) also revealed that the durability of laterized concrete and laterite/cement mortar specimens can be enhanced by the low permeability characteristics of the lateritic soil contents of such specimens. A study on the effect of mix proportion and reinforcement on the anchorage bond stress of laterized concrete by Osunade and Babalola (1991) established that both mix proportion and the size of reinforcement have a significant effect on the anchorage bond stress of laterized concrete specimens. They also assert that the anchorage bond stress between plain and round steel reinforcement and laterized concrete increase with increase in the size of reinforcement used. Osunade (1994), in another study found that increase in shear and tensile strength of laterized concrete was obtained as grain size ranges and curing age increased. Greater values of shear and tensile strength were obtained for rectangular specimens than those obtained for cylinders.

A study by Lasisi and Ogunjimi (1984) on source and mix proportions as factors in the characteristics strength of laterized concrete presented the average characteristic strength for laterized concrete as 27 N/mm^2 for 1:1:2, 17 N/mm^2 for 1:2:3 and 16 N/mm^2 for 1:2:4. A comparison with Adepegba (1975b) results shows that the differences in strength arise due to different chemical composition, method of compaction and difference in maximum size of aggregate used. They discovered that the source of lateritic soil, grain size, the mix proportion and age are highly significant to strength achieved by laterized concrete as they are in normal concrete.

Adepegba (1975a) and several other authors maintains that laterized concrete would require slightly more cement than normal concrete to obtain a mix which would yield the same compressive strength as normal concrete. Influence of duration of curing and mix proportions on the compressive strength of laterized concrete has also been researched on (Falade, 1991a). Water curing was found to give the highest strength values, while air-cured specimens gave the lowest strength values. Compressive strength was observed to be increasing with cement/aggregate ratio and curing period.

Rai *et al.*, (1987) reveals that water absorption characteristics of laterized concrete were higher than that of ordinary concrete. The water requirement increases enormously for the workability of concrete with laterite fines. Workability of concrete for a given water-cement ratio decreases with increasing replacement levels of sand with laterite as fine aggregate.

In the works of Osunade (2002) on the effect of replacement of lateritic soils with granite fines on the compressive and tensile strengths of laterized concrete; the results showed that for different mix proportions (1:1:2; 1:1.5:3; 1:2:4 and 1:3:6) maximum compressive strength values were obtained for laterized concrete containing 50% granite fines. The addition of granite fines in laterized concrete resulted in a decrease in tensile strength. The report concludes that laterized concrete containing laterite fines can be used in the construction of buildings and rural infrastructures. According to Olusola (2005) the need for further research work thereby arises to ascertain cost implications, availability and affordability of this type of laterized concrete since one of the major components which is granite fines may be quite expensive.

Lasisi *et al.*, (1990) reported the result of short-term studies on the durability of laterized concrete. On a short-term basis, the resistance of laterized concrete specimen to chemical attack, like that from magnesium sulphate solution, was found to be good and produces no detrimental effect on the compressive strength of laterized concrete.

Olusola (2005) in his bid to fill the gap existing on available data on size effects on compressive strength of laterized concrete discover in his study that the compressive strength of laterized concrete is strongly affected by the ratio of the specimen size to the diameter of the maximum coarse aggregate size. The phenomenon of size effects, he said, has to do with change in the indicated limit strength due to change in both specimen and maximum aggregate sizes. The shear and flexural strength of laterized concrete have also been investigated. Adepegba (1975a) established that the moment of resistance of wholly laterized concrete beam was 15800kgcm (15.8Nm) which was less than that of normal concrete; a value of 188000kgcm (188.0Nm). Adepegba (1975a) thereby noted that this observation may disqualify the use of laterite as the sole aggregate in structural concrete. A pool of research results from other studies on shear strength of laterized concrete beams led to a conclusion that the nominal shear strength of laterized concrete beam is comparable with that of the corresponding normal concrete of the same mix (Salau and Balogun, 1990; Falade, 1991b and Osunade et al., 1990). Salau and Balogun (1990) further asserted that this is true if the percentage of laterite in concrete is not greater than 25%.

The study of shear strength of laterized concrete beams has led to investigations covering different situations namely:

i) when they are unreinforced sections;

ii) when they are reinforced with consideration given to effect of only two variables – span/effective depth ratio and amount of longitudinal reinforcement and

iii) without shear reinforcement provided (Salau and Balogun, 1990; Falade,1991b and Osunade *et al.*, 1990).

In a recent study by Ata, et al, (2005), it was discovered that Poisson's ratio of laterized concrete ranges between 0.25 and 0.35 and increases with age at decreasing rate. Methods of curing, compaction method and water/cement ratio have little influence on Poisson's ratio; the Poisson's ratio of laterized concrete is said to increase as the mix becomes less rich. Ata (2007) offer on the "effects of varying curing age and water/cement ratio on the elastic properties of laterized concrete" in conclusion as follows:

- The modulus of elasticity of laterized concrete lies between 7000 and 9500 MPa (N/mm^2), while that of deformability lies between the range of 5000 and 6000 MPa (N/mm^2).
- Modulus of elasticity and deformability of concrete increase with an increase in curing age.

- The value of modulus of elasticity of laterized is always higher than its corresponding modulus of deformability. The richer the mix; the higher the modulus of elasticity and deformability of laterized concrete.
- The stronger the laterized concrete; the higher the two moduli.
- Any water/cement ratio which gives laterized concrete a high strength will lead to an increase in its modulus of elasticity and modulus of deformability.

3 RESEARCH METHODOLOGY

3.1 MATERIALS AND LABORATORY ANALYSIS OF SAMPLES

3.1.1 Summary of Research Method

This research work was based on laboratory tests conducted on laterized concrete with a concrete mix of mean **28-day** compressive strength of **25 N/ mm^2** adopted as control. It involved casting concrete specimens with partial substitution of Sand with Laterite and the Ordinary Portland Cement with Volcanic Ash at various percentages (10%, 20% and 30% respectively). The Volcanic Ash was obtained as a solid mass from the foot of Dutshin Dushowa (a hill) at Kerang in Mangu Local Government in Plateau State, pounded and grinded at Minna, Niger State. Hence laboratory tests were carried out on samples of volcanic ash, fine aggregate and coarse aggregate to determine their respective properties. The proportioning of the constituent for mix varied according to the requirement of the specific objectives of the research works in line with mix design.

The general experimental procedures, materials and instrumentation are as discussed in the subsequent sub-sections.

3.1.2 Materials Collection

The aggregate types used for the research were laterite and sharp sand (5 mm maximum size) as fine aggregate and crushed granite (19 mm maximum size specified) as coarse aggregate. These were procured at the various deposits within Minna, Niger State. The cement used was obtained from the building materials market in Minna and was that produced by the Obajana factory of Dangote Cement whose properties conform to the requirements of BS EN 197-

1:2000 (which replaces BS 12 (1991)) for Ordinary Portland Cement. The Volcanic Ash used was obtained from the foot of Dutshin Dushowa (a hill) at Kerang in Mangu Local Government Area of Plateau State. This was dug from the foot of the volcanic deposit as solid mass, pounded and grounded to very fine particles and sieved with 75μm before use in Minna.

3.1.3 Instrumentation

The laterized concrete specimens tested were 100 mm x 100 mm x 100 mm cubes, 150 mm x 300 mm cylindrical prisms and 100 mm x 100 mm x 500 mm rectangular prisms in accordance to specifications. The steel moulds for cylindrical prisms were gotten in the Department of Building Laboratory, Federal University of Technology (FUT), Minna, while the cube and rectangular prisms steel moulds were fabricated. The use of these moulds was employed in the tests conducted as required by the second and third objectives of this research work. The compressive and Tensile Splitting strength tests were determined using an ELE 2000 KN compression testing machine available in the Building Department of FUT Minna while the flexural strength was done using a hydraulically operated universal testing machine of the Civil Engineering Laboratory, FUT, Minna for a three point loading arrangement. The chemical analysis of Volcanic Ash and Laterite was carried out at the Sagamu Works Department of Lafarge Cement (West African Portland Cement Company - WAPCO) via an X-ray Fluorescent Analysis using a Total Cement Analyser model ARL 9900 XP. The pounding and grinding of the Volcanic Ash was carried out in the Department of Building laboratory, FUT, Minna and at a local shop in Minna. Furthermore all mass measurements were taken on weighing balances available in the various Laboratories of the Federal University of Technology (FUT), Minna.

3.1.4 Laboratory Analyses of Samples

The preliminary tests carried out on the fine aggregate samples (sand and laterite) are sample grading (sieve analysis), moisture content determination, specific gravity; Atterberg limits determination and chemical analysis of the laterite soil to determine the basic oxides. Chemical analysis of the volcanic ash was also carried out, while the sieve analysis, specific gravity and moisture content tests also included the volcanic ash and granite samples. Also determined are the physical properties of the binder volcanic ash/Ordinary Portland Cement (VA/OPC) such as fineness, consistency, setting time and soundness.

a) Sample grading (Sieve analysis of dry aggregate samples)

The Sieve analysis is the method used for the determination of the relative proportions of the different grain sizes that make up a given soil/material mass. Particles were allowed to pass through stack of sieves with openings of known sizes by shaking for 10 minutes as recommended by Bowles (1992), using a mechanical test sieve shaker. The sieves were thereafter removed from the shaker and the weight of each sieve with the sample retained was taken to the nearest 0.1 g. The mass sample (fine or coarse aggregate) retained on each sieve was obtained by subtracting the respective mass of each sieve. This test was carried out for all aggregates according to standard procedure (BS EN 933 Pt. 1: 1997).

The percentage passing and the cumulative percentage of the soil/material retained was calculated using the expression below:

$$\% \, retainedonanysieve = \frac{weightofsoil/materialretained}{totalweightofsoil/material} \times 100 \quad \ldots (3.1)$$

$$= \frac{w2 - w1}{w} \times 100$$

where: w1 = weight of sieve

w2= weight of sieve + soil/material in grams

Percentage passing = 100 – cumulative percentage of soil/material retained … (3.2)

The results of the sieve analysis was presented in a tabular form and plotted on a grading curve. The results enabled us to determine whether the aggregates meet the grading requirements of BS EN 12620:2002.

Vandevelde (2008) stated that the shape of the particle size distribution curve for any soil sample can be expressed approximately by a Coefficient of Uniformity (C_u) and also Coefficient of Curvature (C_C)given by the expressions below:

$$C_u = D_{60} / D_{10} \quad \text{...(3.3)}$$

and $$C_c = (D_{30})^2 / (D_{60} \times D_{10}) \quad \text{...............................(3.4)}$$

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

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

D_{30} = Particle size such that 30% of the soil is finer than this size.

Well graded requirements were thereby presented as:

$C_u \geq 4$ for gravel; $C_u \geq 6$ for sand and C_c= 1 to 3 for all type of soil. Soils having a $C_u < 2$ are classified as uniformly graded. It was stated that both C_u and C_c indicates the soil classification, Vandevelde (2008).

Lambe and Whitmann (1969) however has a broad classification based on C_u as follows:

i. $C_u > 5$, soil is **<u>well graded</u>**, that is; it has a particle size distribution extending evenly over a wide range of particle sizes, without excess or deficiency of any particle size.
ii. C_u between 1.0 and 5.0, soil is **<u>uniformly graded</u>**, all particles in the soil are more or less of the same size.
iii. $C_u < 1.0$, soil is **<u>poorly graded</u>**, that is, it has a particle distribution containing an excess of some particle sizes and a deficiency of others.

Atkinson (1993), Neville and Brooks (2002) and Shetty (2004) presents the term "Fineness Modulus" as a ready index of coarseness or fineness of an aggregate. It is an empirical factor obtained by adding the cumulative percentages of aggregates retained on each of the standard sieves ranging from 80 mm to 150μm and dividing this sum by an arbitrary number 100 (Shetty, 2004). The larger the Fineness Modulus value, the coarser the material.

Shetty (2004) presents the following limits to be taken as guidance for Fineness Modulus (F.M.) of Sand for concrete works.

Fine Sand, F.M.: 2.2 – 2.6

Medium Sand F.M.: 2.6 – 2.9

Coarse Sand F.M.: 2.9 – 3.2

A sand having F.M. >3.2 will be unsuitable for making satisfactory concrete.

The fineness moduli, the coefficient of uniformity (C_u) and the coefficient of curvature (C_c) of the aggregates were calculated from the data listed in Tables A.1 to A.3 and Figure 4.1 respectively.

b) **Atterberg Limit/Consistency Limit Tests**

This test was used to determine the liquid and plastic limit and hence the liquidity and plasticity of laterite. According to BS EN 1377 - 2 (1990), "the liquid limit (L.L) is the empirically established moisture content at which a soil passes from the liquid state to the plastic state". This implies the moisture content at which the soil stops acting as a liquid and starts acting as a plastic solid. The plastic limit (P.L) is the empirically established moisture content at which a soil becomes too dry to be plastic (BS EN 1377 - 2, 1990).This is the water content at which soil begins to crumble when rolled into threads of specific size. The plastic limit is used together with the liquid limit to determine the plasticity index. The tests were carried out for Laterite only in accordance to BS EN 1377, - 2 (1990).

The test processes was as presented below:

Material: Laterite and Distilled water

Apparatus: cylinder (of known volume), weighing balance, cone penetrometer, glass plate, 425 μm sieve and spatula.

i) **Liquid limit**: Using the cone penetrometer (the preferred method), the soil to be tested was air dried and thoroughly mixed. 200g of the soil was sieved through a 425μm sieve and placed on a glass sheet. The soil was then mixed with distilled water into a paste.

   A metal cup approximately 55 mm in diameter and 40 mm deep was filled with the paste and the surface struck off level. The cone was next placed at the centre of the smoothed soil surface and level with it, the cone was released so that it penetrates into the soil and the amount of penetration measured.

   The test was now repeated by lifting the cone clear, cleaning it and filling up the depression in the soil's surface by adding a little more of the wet soil.

   BS EN 1377-2 (1990) state that if the difference between the two measured penetrations is less than 0.5 mm then the tests can be considered valid. The average penetration was noted and a moisture content determination was carried on the soil tested. The procedure was repeated at least four times with increasing water content. The amount of water used throughout was such that the penetrations obtained lie within a range of 15 to 25 mm.

   To obtain the liquid limit, the variation of cone penetrations (plotted vertically) to moisture content (plotted horizontally) was drawn out (both scale being natural) as presented in Fig 4.2 with the best straight line drawn through the experimental points and the liquid limit taken to be the moisture content corresponding to a cone penetration of 20 mm (expressed as a whole number).

ii) Plastic limit: About 20 g of soil prepared as in the liquid limit test was used. The soil was mixed on the glass plate with just enough water to make it sufficiently plastic for rolling into a ball, which was then rolled out between the hand and the glass to form a thread. According to BS EN 1377-2 (1990), the soil is said to be at its plastic limit when it just begins to crumble at a thread diameter of 3 mm. At this stage a section of the thread was removed for moisture content determination. The test was repeated at least once.

$$\text{Plasticity Index (PI)} = L.L - P.L \qquad (3.5)$$

$$\text{Liquidity Index (L.I)} = \frac{W - P.L}{P.I} \qquad (3.6)$$

Where W = moisture content of soil.

These values as presented in Table A.4 serve as a measure of its resistance to deformation and a measure of its plasticity and compressibility.

c) **Determination of Moisture Content of Aggregate Samples**

This involves oven-drying known weights of the aggregate sample for 24 hours at a temperature above 110^{o}C (e.g. 115^{o}C). Their weights were taken after drying to determine the weight of water evaporated and that of the dry sample. This test was carried out on both the Laterite and Sand samples in accordance to BS EN 1097 -5 (1999).

The moisture content was then calculated as follows:

$$\text{Moisture content} = \frac{\text{Initial weight of sample} - \text{Dry weight of sample}}{\text{Dry weight of sample}} \qquad (3.7)$$

Table A.9 presents the result of the moisture content test.

d) Determination of Specific Gravity for Material Samples

Neville and Brooks (2002) defines specific gravity quoting ASTM C127 – 93 as "the ratio of the mass (or weight in air) of a unit of volume of material to the mass of the same volume of water at the stated temperature. BS EN 1097 – 6 (2000) uses the term particle density, expressed in kilograms per cubic meter. Thus particle density is numerically 1000 times greater than specific gravity.

The absolute specific gravity and the particle density refer to the volume of the solid material excluding all pores, whilst the apparent particle density refer to the volume of solid material including the impermeable pores, but not the capillary ones (Neville and Brooks, 2002).

The test procedure for the respective sizes of aggregates are well spelt out in BS EN 1097 – 6 (2000) and this was properly followed to determine the particle density (specific gravity) on a saturated surface dry basis for the cement, volcanic ash, laterite, sand and granite respectively. The procedure as adopted for each aggregate is as presented in detail below:

i) Specific Gravity of Binder (Cement/Volcanic Ash)

Materials: pycnometer, weighing balance.

Procedure: The pycnometer was weighed empty. The sample was filled to 1/3 of the pycnometer and weighed. Water was then added slightly above the level of the volcanic ash. The pycnometer was shaken vigorously to expel air but carefully to avoid spilling of the mixture. The pycnometer was finally filled with water to the level marked and was weighed. The content of the pycnometer was poured out and the vessel thoroughly washed. The pycnometer was filled with water alone to the brim and then weighed.

ii) Specific Gravity of Fine Aggregate (Sand and Laterite)

Material: sand/laterite and water

Apparatus: density bottle (100 cm^3), electronic weighing balance, measuring cylinder, rag (towel), tray and spatula.

Procedure: The density bottle was weighed empty. The bottle was then filled with sample of sand/laterite using spatula up to an appreciable volume; the bottle together with its content was weighed. Then water was added slightly above the level of sand. The bottle was covered properly and vigorously shaken to expel air mixture, but this time to the brim (level mark) and was covered with glass stopper. The bottle together with its content including the glass stopper was weighed. The content of the bottle was poured away and the bottle thoroughly washed. The same bottle was filled with water only to the mark level (brim) covered with the glass stopper and weighed. The submerged weight was obtained by subtracting the weight of the bottle plus water from the weight of bottle plus sand and water. A second test was conducted with the same procedure and the average of the two was taken as the specific gravity.

iii) Specific Gravity of Coarse Aggregate

Material: granite and water

Apparatus: pycnometer, tray, scoop, drying cloth, weighing balance, measuring cylinder.

Procedure: About 400 g of coarse aggregate was weighed and soaked in water for 24 hours. The soaked aggregate was removed from the water and cleaned with a piece of cloth to bring to saturated surface – dry (ssd) condition.

The cap of the pycnometer was removed and the jar was filled with water until the water over flowed and was free of air bubbles. The pycnometer and its content were weighed and the water poured out. Again the pycnometer was filled with water to one – third (1/3) of its volume. The saturated surface dry sample of the coarse aggregate was weighed and added to the jar, the pycnometer was filled with water until it overflowed and was free from air bubbles. The weight of pycnometer plus aggregate plus water was determined. Thus the specific gravity was determined.

Tables A.4 to A.6 presents the results of the specific gravity tests.

e) Bulk Density of Aggregates (Granite and Laterite)

Bulk density of a material is the average weight of material held by container of a unit volume when filled or compacted under defined condition. The bulk density of an aggregate is affected by factors such as the amount of compacting effort used in filling the container and the amount of moisture present.

The material for the test were crushed granite, laterite and water; while the apparatus used are cylinder (of known volume), weighing balance, flat metal plate and tampering rod.

The test was carried out separately for granite and also for laterite for the compacted and loose bulk density in accordance to BS EN 1097-3: 1997 as follows:

i) Compacted Bulk Density

A 300 m^3 cylinder was filled in three layers, being tampered 25 times with standard tampering rod (16 mm) the last layer was allowed to overflow the container and the surplus being struck off with a straight edge. The container plus aggregate was then weighed (w_{cc}). The compacted bulk density was calculated from the formulae as follows:

Compacted bulk density = w_{cc}-w_{ec}/v_c

Where w_{cc} = weight of compacted aggregate + cylinder

w_{ec} = weight of empty cylinder

v_c = volume of cylinder

ii) Loose/Uncompacted Bulk Density

Again the container (300 mm^3 cylinder) was filled to overflow by dropping the aggregates from a height of about 50 mm from the top of the container with a scoop. The surplus was struck off and levelled gently from the top of the container using straight edge as before and the container with the aggregate was weighed (w_{el}). The uncompacted bulk density was calculated from the formulae as follows:

Uncompacted bulk density = $w_{el} - w_{ec}/v_c$

Where w_{el} = weight of loose aggregate + cylinder

w_{ec} = weight of empty cylinder

v_c = volume of cylinder

The results of the bulk density tests are presented in Tables A.7 and A.8 of the Appendix.

3.2 EXPERIMENTAL DESIGN

The main experimental designs involved the following laboratory experiments.

i. Determination of chemical composition of Volcanic Ash and Laterite.

ii. Determination of physical properties of volcanic ash blended cement.

iii. Determination of the underlisted characteristics of the volcanic ash blended cement laterized concrete produced:

a) Compressive strength

b) Tensile Splitting strength and

c) Flexural strength.

Experiment (i) was carried out at Sagamu Works Department of Lafarge Cement (West African Portland Cement Company, WAPCO) while (ii) and (iii) was carried out partly in Building Department Laboratory and Civil Engineering Laboratory of F.U.T, Minna.

3.3 EXPERIMENTAL PROCEDURES

The detailed description of the various laboratory experiments carried out is as discussed in subsequent sections.

3.3.1 Determination of Chemical Composition of Volcanic Ash and Laterite

The volcanic ash and laterite sample were prepared in F.U.T, Minna and then taken to WAPCO, Sagamu Works for analysis. About 150 g of each prepared sample was involved. The volcanic-ash sample was pounded, ground and sieved using a 75μm sieve before the 150 g was packaged in small nylon bag. The laterite sample was also air dried and sieved using a 4.25 mm sieve before the 150 g was packaged as in the case of volcanic ash.

The determination of the chemical composition at WAPCO involves: drying, grinding, pressing and analysing. The materials were dried in an oven at 100 $\pm 10^0$C for about two hours until a constant weight (±0.01 g) was obtained after which the sample was placed in a desiccator to cool for about 30 minutes before grinding commences. In order to aid grinding and to prevent sticking of the sample to dish, 0.8 g of stearic acid was weighed into sample dish before adding 20.0 g of the material (laterite/volcanic ash sample) into it. Grinding was done on a gyro-mill grinding machine (Model HSM 100H, Serial Number MA 11566-5-1, 2004), which stops automatically after grinding for a pre-set time of 3 minutes. The sample was then ready for pressing.

The ground sample plus 1.0 g of stearic acid to ensure adequate binding, was used to fill the pellet cup to the brim. The pellet cup was then centrally placed in an automatic hydraulic operated press (Model TP 40/2D), pressed at 20 tons load and 30 seconds hold time. On completion of pressing, the pressed pellet was carefully removed from the cylindrical pressing die and transferred into the X-ray analyser sample holder ready for analysis.

The analysis was carried out using X-Ray Fluorescent Analyser called Total Cement Analyser (Model ARL 9900 XP), which is connected directly to a computer system. The pressed pellet was loaded in the sample port of the analyser and the assembly left for about three minutes after which the values of elements concentration were displayed on the monitor. The computer automatically prints the result of the analysis. The result of the chemical analysis for both the volcanic ash and laterite sample is as presented in Table 4.6 and 4.5 respectively.

3.3.2 Determination of Physical Characteristics of Volcanic-Ash Blended Cements

The following physical characteristics of the volcanic ash blended cements were considered:

i. Fineness (Sieving Method)
ii. Consistency
iii. Soundness and
iv. Initial and final setting times.

a) Fineness (Sieving Method)

The fineness of cement is measured by sieving it on standard sieves. The proportion of cement of which the grain sizes are larger than the specified mesh size is thus determined (BS EN 196 – 6:1992).

This test was carried out to determine cement residue as specified in BS EN 196 – 6:1992 using a 45µm sieve since the volcanic ash sample being used are those passing 75µm sieve. The sample to be tested was agitated by shaking for 2 min in a stoppered jar to disperse agglomerates. After waiting for 2 min the resulting powder was stirred gently using a clean dry rod in order to distribute the fines throughout the cement. Hence the tray was fitted under the sieve and approximately 10 g of cement to the nearest 0.01 g was weighed and placed in the sieve, being careful to avoid loss. Agglomerates were then dispersed and the lid fitted back over the sieve. The sieve was agitated using a sieving machine at a pre-set time to 5 minutes. The residue was then removed and weighed and this mass expressed as a percentage, $R1$, of the quantity first placed in the sieve to the nearest 0.1 %. Brushing all the fine material off the base of the sieve into the tray, the whole procedure was repeated using a fresh 10 g sample to obtain $R2$. The residue of the cement R was then calculated as the mean of $R1$ and $R2$ as a percentage, expressed to the nearest 0.1 %.

b) Consistency Test

The consistency of standard cement paste was determined using Vicat apparatus with a 10 mm diameter plunger as specified in BS EN 196 – 3:1995. 500 g of cement sample (also the blended cement) was weighed and spread out on steel plate; in case of the volcanic-ash blended cement the appropriate percentage of replacements was noted and weighed as required with everything thoroughly mixed together. Using the measuring cylinder, 125 g of clean tap water was added and mixing with trowel was done for 4 ± 0.25 minutes to give a paste. The paste was then transferred into the Vicat mould which had earlier been cleaned and lightly oiled. The top of the mould was levelled and the mould with the paste placed under the Vicat apparatus with the plunger gently lowered to contact surface of the paste and quickly released to allow it sink into the paste. Under the action of its weight the plunger will penetrate the paste, the depth of the penetration depending on the consistency. When the plunger penetrates the paste to a point 6 ± 1 mm from the bottom of the mould, the water content of the standard paste is expressed as a percentage by mass of the dry cement, the usual range of values between 26 and 33 percent.

c) Initial and Final Setting Times

The setting times tests were carried out using the Vicat apparatus. The temperature of the test room was kept at 27 ± 5^0C. Cement paste of standard consistency as described above and the two types of setting time tests (Initial and Final) were carried out on the cement pastes of the four different levels of percentage replacement of cement by volcanic ash (0%, 10%, 20% and 30% respectively) in accordance to BS EN 196 – 3:1995 as discussed below:

- **Initial setting time**: - For the determination of the initial set, a round needle with a diameter of 1.13± 0.05 mm was used. The needle, acting under a prescribed weight, was used to penetrate the paste of standard consistency placed in the Vicat mould. When the paste stiffens sufficiently for the needle to penetrate no deeper than to a point 5± 1 mm from the bottom, initial set was recorded. Initial set is expressed as the time elapsed since the mixing water was added to the cement.
- **Final setting time**: - Final set was determined by a similar needle fitted with a metal attachment hollowed out so as to leave a circular cutting edge 5 mm in diameter and set 0.5 mm behind the tip of the needle. Final set is said to have taken place when the needle gently lowered to the surface of the paste, penetrates it to a depth of 0.5 mm but the circular cutting edge fails to make an impression on the surface of the paste. The final setting is reckoned from the moment when mixing water was added to the cement.

d) Soundness Test

The soundness or cement expansion test was performed using the Le-Chatelier apparatus. A cement paste of standard consistency was prepared and used to fill the expansion mould on a glass plate, keeping the split of the mould gently closed. The top of the mould was smoothened and levelled and a glass top end applied. The assembly was then placed in water at 27 ± 5^0C with a small weight placed on the top end plate. The mould was removed after 24hours and the distance between the two points (i.e. the split opening) was measured to the nearest 0.5 mm (say A mm)

The mould was then placed in a heating bath and the temperature raised to boiling point within 15 minutes and then allowed to boil for 1 hour. The mould was thereafter removed from the bath and allowed to cool for 1 hour after which the distance between the two pointers was measured again to the nearest 0.5 mm (say B mm). The difference between the two measurements (B – A) was recorded as the expansion of the cement.

3.3.3 Determination of Laterized Concrete Characteristics

In an effort to determine the effect of volcanic ash on laterized concrete, other mix design variables like quality of ingredients, mixing procedures, curing condition and testing procedures were kept constant.

The properties investigated on the laterized concrete covers both the fresh and hardened concrete. The major experiments here involved the compressive strength test, the Tensile Splitting strength test and the flexural strength test. In line with requirements for pozzolan, this required the specimens to be cured for six different curing ages (3, 7, 28, 56, 90 and 120 days).

Table 3.1 shows the details of specimen samples for various tests in a bid to answer the questions posed by the research objectives

-Cube specimen required for compressive strength test.

- Cylindrical Specimen for Indirect Splitting = 288 Cylinders

- 100 x 100 x 500 mm Beams for Flexural Strength = 288 Beams

TABLE 3.1

Table 3.1: Cube Specimens Required for Compressive Strength Test

	Volcanic Ash			
Laterite	0%	10%	20%	30%
0%	3	3	3	3
10%	3	3	3	3
20%	3	3	3	3
30%	3	3	3	3

= 16 x 3 x 6 curing ages = 288 Cubes

Hence, a total of 288 Cubes, 288 Cylinders and 288 Beams were cast for the experimental work. The discussion of the procedures involved in this aspect of work can then be placed under the following sub-heads

i. Proportioning and Mixing of Constituents

ii. Workability Test

iii. Compressive Strength Test

iv. Tensile Splitting Strength Test Specimen

v. Flexural Strength Test.

a) Proportioning and Mixing of Constituents

The mix-proportioning involved the British Mix-Design (a.k.a. D.O.E) approach for 28-day target strength of 25 N/mm^2 for the Normal Concrete (i.e. Control (0% laterite, 0% volcanic ash)) and the water/cement requirement of the mix-design for the requisite workability was adhered to. The partial substitution by weight of sand by laterite and cement by volcanic ash was then calculated for the 10%, 20% and 30% respectively as required for both materials. Tables B.3 and B.4 shows the details of the material proportioning.

The ingredients were mixed mechanically and re-mixed manually on a neat platform with the pre-determined amount of water. The proportioning by weight was done according to the outcome of calculations from the mix-design of 25 N/mm^2, 28-day target strength as control mix.

The laterite and sand was thoroughly mixed, so also was the volcanic ash and cement on one side before been loaded into the concrete mixer. Some quantity of water was the first to be loaded into the mixer, followed by granite, the volcanic ash blended cement and the sand/laterite mix. The whole mixture was thoroughly mixed before additional quantity of water was added. The mixture was then discharged from the mixer and the "working" process of gradual addition of water to the dry mixtures and the continuous stirring/agitation with the trowel or/and shovel continues. Mixing will only be assumed to be completed when a homogeneous mix has been obtained.

b) Workability Tests

Slump tests were carried out to determine the workability of each mix. The laterized concrete was made with different percentage replacements of cement with volcanic ash (0%, 10%, 20% and 30% respectively) and also sand with laterite (0%, 10%, 20% and 30%).

The tests were carried out in all cases in accordance with the requirement of BS EN 12350 – 2:2000 for Slump Tests. The slump test was performed using a standard slump cone mould. The internal face of the mould was thoroughly cleaned, free from hardened concrete. The mould was placed on a smooth, horizontal surface with the mould firmly held in position while it is filled. The mould was filled with concrete in three layers, each layer being tamped 25 times with a standard 16 mm diameter steel rod, ensuring that the strokes are distributed in a uniform manner over the cross section of the mould. The top surface of the concrete was struck off by means of rolling motion of the tamping rod and damp cloth used to wipe the outside of the cone and the base plate clean. The mould was then slowly and carefully lifted in a vertical direction and the unsupported concrete allowed to slump. The mould was turned upside down and placed next to the slumped concrete. The tamping rod was placed on top of the empty inverted cone, projecting over the highest point of the slumped concrete. The distance from the top of the slumped concrete to the underside of the rod represents the slump in millimetre.

c) Compressive Strength Test

The compressive strength was determined using 100 mm concrete cubes. A total of 288 cubes were cast for the four levels of volcanic ash replacements of cement, four laterite replacement levels of sand and the six curing durations as outlined in Table 3.1.

The 100 mm x 100 mm were fabricated in a welding shop in Minna using a 4 mm thick grade 55 steel sheets ensuring they conform to BS EN 12390 – 1:2000 specifications. The moulds were thoroughly cleaned and coated with mould oil before casting to ensure easy demoulding and smooth surface finish. The wet mixture was cast into moulds, immediately after mixing with hand trowel. The moulds were filled in two layers of 50 mm each, compacted using the compaction rod (25 mm diameter steel rod), the minimum of 25 strokes uniformly distributed over its surface during casting as stipulated by the requirements of BS EN 12390 –2 & 3:2000. The top of each mould was smoothened and levelled and the outside surfaces cleaned. The mould and their contents were kept in the curing room at temperature of 27 ± 5^0C and relative humidity not less than 90% for 24 hours. Demoulding of cubes took place after 24 hours and the specimens were transferred into a water bath maintained at 27± 5^0C in the curing room. Compressive strength was determined at curing age 3, 7, 28, 56, 90 and 120 days in-line with the code specification.

d) Tensile Splitting Strength Test

The Tensile Splitting strength was determined using 150 mm x 300 mm concrete cylinders. A total of 288 concrete cylinders were cast for the four levels of volcanic ash replacements of cement, four laterite replacement levels of sand and the six curing durations. The cylindrical moulds available in Building Laboratory of F.U.T Minna were assembled in conformity with BS EN 12390 – 1:2000 specifications, thoroughly cleaned and coated with mould oil before casting to ensure easy remoulding and smooth surface finish .The specimens were cast in steel moulds by filling each mould in three layers, each being compacted manually by evenly distributing 35 strokes of 25 mm tamping rod across the cross section of the mould. They were demoulded after 24hours and cured in curing tank until testing age 3, 7, 28, 56, 90 and 120 days. On the testing age, three specimens for each replacement were brought out of the curing tank, allowed to drain and dry in open air for one hour before crushing on an ELE compression machine (maximum capacity 2000KN, Model No JYS 2000A CLASS 1Serial No 16). The cylindrical prisms specimens were compressed along two diametrically opposed generators lying horizontal. To prevent multiple cracking and crushing at the point of loading, two thin plywood strips (25 mm thick) were placed between the loading paten and the specimen to distribute the load while a special appliance fabricated was used to hold the cylindrical prisms in place to avoid tilting or rolling under load. The induced stress caused the specimen to fail by splitting into two halves across the loading plane as shown in Figure 3.1. This test was carried out in accordance to the provision of BS EN 12390 - 6:2000.

e) Flexural Strength Test

The flexural strength test was determined using 100 mm x 100 mm x 500 mm concrete beams since the maximum aggregate size is 25 mm. A total of 288 concrete beam prisms were cast for the four levels of volcanic ash replacements of cement, four laterite replacement levels of sand and the six curing durations. The moulds were fabricated in a welding shop in

Figure 3.1: Test set-up for Tensile Splitting Test Using Cylindrical Prism

Figure 3.2: Flexural Strength Test Result Being Taken

Minna using a 4 mm thick grade 55 steel sheets ensuring they conform to BS EN 12390 – 1:2000 specifications. They were then thoroughly cleaned and coated with mould oil before casting to ensure easy demoulding and smooth surface finish. The specimens were cast in steel moulds by filling each mould in two layers, each being compacted manually by evenly 125 (i.e. 5 x 25) strokes of 25 mm rod in conformity with BS EN 12390 – 5:2000. The specimens were demoulded and cured as explained in the two earlier tests. On the testing age the beams specimens were tested on their side in relation to the as-cast position, in moist condition, using the hydraulically operated universal testing machine Model No: C90; S/No: E518 – 95; 150KN capacity available in the Civil Engineering Laboratory of F.U.T Minna conforming to BS EN 12390 – 4:2000. Excess moisture was wiped from the surfaces of the specimen before placing in the testing machine. The bearing surfaces of the testing machine were wiped clean while loose grit or other extraneous material was removed from the surfaces of the specimen that will be in contact with the rollers. The device for load application conforms to the specifications of BS EN 12390 – 5:2000, consisting of two supporting rollers and two upper rollers carried by an articulated cross member which divides the load applied by the machine equally between the two rollers. The upper rollers were kept at 100 mm spacing, while the supporting rollers were kept at 300 mm spacing. The test adopted the three point loading arrangement (also referred to as two point load in BS EN 12390 – 5:2000) while the fracture were noted to have occurred within the outside rollers. The maximum crushing load was recorded to 0.1 KN (as shown in Plate 2), with result of the test properly analysed and discussed in chapter 4.

The weight of the various specimens at the age of testing was noted in all the three major tests discussed above and density calculated in conformity to BS EN 12390 – 7:2000.

3.4 METHOD OF DATA COLLECTION AND ANALYSIS

In all cases full factorial experimental design approach and graphical illustrations were adopted. According to Johnson (1994), the statistical basis is the factorial experiment. The statistical approach aims at determining which of the variable has significant effects on the measured parameter being considered under each objective. The Analysis of Variance and Regression Analysis were employed for objectives 2, while Microsoft Excel was used for the compilation and analysis of the various tables of data generated and the plot of the requisite graphs where necessary. F-distribution table as well as the ANOVA-table which resulted from the analysis of variance was used to test for the significance of each factor and its interactions. Individual means was also compared. The Regression Analysis and ANOVA were done using the Statistical Analysis Software (SAS, 2002) while objective 3 adopted the use of Matrix Laboratory (MATLAB) Software.

The treatments consist of appropriate combinations of the levels of the various factors. In general, if two factors **A** and **B** are to be investigated at 'a' levels and 'b' levels respectively, these are **a.b** experimental conditions (treatments) corresponding to all possible combinations of the levels of the two factors and resulting experiment is referred to as **a x b** factorial experiment (Johnson, 1994). Results obtained from tests was scientifically analyzed and conclusion drawn. The specific methods of data analysis employed are discussed under each objective as stated below:

i) Objective one

Data collected were the chemical and physical properties of volcanic ash and the VA/OPC cementitious mixture. These was determined for the volcanic ash at natural state

(a) Oxide Contents, at three levels, that is, SiO_3, Al_2O_3 and Fe_20_3.

(b) Other Chemical Contents, at five levels, that is, Ca0, Mg0, $S0_3$, Na_20 or K_20 and the Loss of Ignition (L.O.I).

(c) Determination of Soluble Salts, at two levels, that is, water soluble alkalis and water soluble materials.

(d) Physical Properties, at three levels, that is, Specific Gravity, Fineness, Consistency, and Soundness.

(e) Determination of Setting Times of the Cementitious Mixture, at two levels, that is, Initial and Final. Here Microsoft Excel is to be used for data compilation and analysis.

ii) Objective number two and three

Using three replicates a total of 288 cubes, 288 cylindrical prisms and 288 rectangular prisms were cast to determine the effects of curing age and percentage replacement of cement with volcanic ash on the compressive, Tensile Splitting and flexural strength characteristics of laterized concrete;

(a) Specimen sizes are at three levels 100 mm cube, 150 mm x 150 mm cylinders and 100 mm x 100 mm x 500 mm beams.

(b) Mix proportions, was at one level, 28-day design strength of $25N/\ mm^2$.

(c) Percentage replacement of Sand with Laterite, at four levels, that is 0, 10, 20, and 30 by weight.

(d) Percentage replacement of Cement with Volcanic Ash, at four levels, that is 0, 10, 20 and 30 by weight.

(e) Curing age, at six levels, that is 3, 7, 28, 56, 90 and 120 days.

Variations of compressive, tensile and flexural strength of volcanic ash blended cement laterized concretes specimens (cubes, cylinders beams) with percentage replacement of Laterite and Volcanic Ash respectively were expressed in graphical form and a theoretical explanation given for observed variations. The 3 x 4 x 4 x 6 factorial experimental arrangement was used to examine the relationship between these strength characteristics via regression analysis. Applying the MATLAB 7.8 to examine the relationship between compressive strength and tensile strength for the VA-blended cement laterized concrete requires simulating the generalised equation (Eq.2.6) of existing relationship between compressive strength and tensile for normal concrete into the data obtained; this was done discussed below.

iii) Curve fitting/parameters estimation

To solve the nonlinear Eq. 2.6, estimates of *n* and *k* are required. Curve fitting of the experimental data was used to estimate these parameters. The equation was log-transformed to linearize it as follows:

$$ln\,(f_t) = ln\,[k(f_c)^{\,n}] \quad \ldots \quad \ldots \qquad (3.6)$$

$$ln\,(f_t) = ln\,(f_c)^n + ln\,(k) \quad \ldots. \quad \ldots \qquad (3.7)$$

$$ln\,(f_t) = n\,ln\,(f_c) + ln\,(k) \quad \ldots \quad \ldots \qquad (3.8)$$

The original tensile splitting strength and flexural strength data was then transformed using the left side of Eq. 3.8 to generate a new dataset on *Y*:

$$Y = mX + C \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \qquad (3.9)$$

Where:

$m = n$: slope of the line,

$C = ln\ (k)$: intercept of the line, and

$k = \exp\ (C)$

iv) Tests of goodness of fit

The goodness of fit of the model was evaluated using the *R*-squared (R^2) statistic, which is a popular indicator of goodness of fit in regression analysis. The R^2 was calculated from the variance statistics that are reported for the regression, using the equation:

$$R^2 = \frac{SS_{Regression}}{SS_{Total}} \qquad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \qquad (3.10)$$

A value of R^2 close to unity indicates a good fit whereas a value close to zero indicates a poor fit.

Standard deviation, which is the average deviation of the residuals from zero, is another important indicator of the goodness of fit of a nonlinear model. A residual is the difference between the observed and the predicted values for a given data point. Student's *t*-test was used to evaluate the observed and predicted data based on the deviation, with the null hypothesis that the overall mean of the residuals did not differ significantly from zero at $p \leq 0.05$. If the resulting *p*-value of the test is greater than 0.05, it implies that the predicted values closely approximate the observed values.

4 DATA ANALYSIS AND DISCUSSION OF RESULTS

4.1 PHYSICAL AND MECHANICAL PROPERTIES OF AGGREGATES

4.1.1 Sieve Analysis of the Aggregates Used

The result of the sieve analysis of the Sand, Laterite and Granite used were as presented in Tables A.1 – A.3 of the Appendix while Figure 4.1 shows the grading curves with the summary of the analysis as presented in Table 4.1.

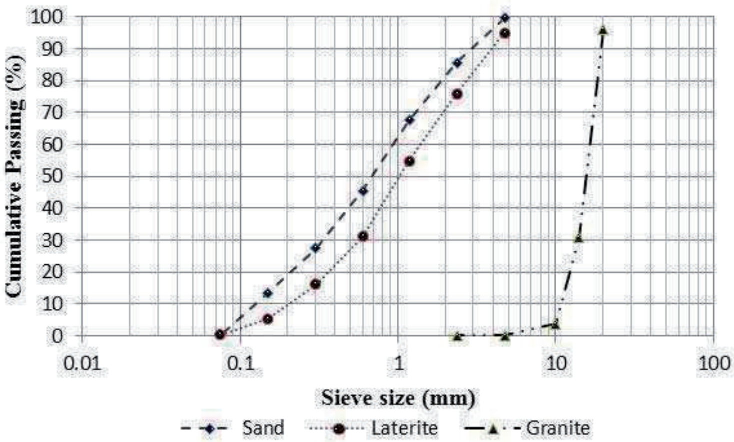


Figure 4.1: Grading Curve for the Aggregates used for the VA-Blended Laterized Concrete

The Fineness Modulus of Sand (2.6) indicated a medium grading, while Laterite reflected a coarse grading (3.2). The coefficient of uniformity (C_u) for both Sand and Laterite was greater than 6.0, while coefficient of curvature (C_c) fell between 1.0 and 3.0, this implied both materials used as fine aggregate were well graded and are therefore very suitable for making good concrete.

Table 4.1: Summary of Results of Sieve Analysis

Item	Sand	Laterite	Granite
Fineness Modulus	2.6	3.2	
D_{60}	0.90	1.45	16.15
D_{30}	0.34	0.58	15
D_{10}	0.13	0.22	12.06
C_u	7.3	6.6	1.34
C_c	1.05	1.05	1.15

The values of C_u (1.34) and C_c (1.15) for the Granite indicates a uniformly graded coarse aggregate, but is still within the limits required for suitable coarse aggregate for good concrete.

4.1.2 Other Physical Properties

The results of other physical properties carried out on the aggregates used are as compressed into Table 4.2 while Tables A.4 – A.8 in Appendix shows the details.

The physical properties of the constituent materials as shown Table 4.2 fell within the limits of the codes requirements for materials suitable for making good concrete.

Table 4.2: Summary of Results of Physical Properties of Aggregates

Material	Volcanic Ash	Cement	Laterite	Sand	Granite
Specific Gravity (kg/m^3)	2.65	3.21	2.68	2.59	2.66
Loose Bulk Density (kg/m^3)	-	-	1263	-	1452
compacted Bulk Density (kg/m^3)	-	-	1907	-	1580
Moisture Content (%)	-	-	14.15	3.67	-

4.2 CHEMICAL COMPOSITION OF THE LATERITE AND VOLCANIC ASH

4.2.1 Atterberg Limits (Liquid and Plastic Limits) of Laterite Sample

The result of Liquid and Plastic Limit are shown in Table 4.3 and 4.4 while Fig.4.2 shows the plot of the Liquid Limit.

Table 4.3: Liquid Limit of Laterite Sample Used

	LIQUID LIMIT				
Penetration (mm)	15	17	19.5	22.5	24.5
Can Number	A	B	C	D	E
Weight of Can (g)	24.1	24.3	24.6	23.9	25.4
Weight of Can + wet Soil (g)	29.6	29.9	30.1	30.2	31.7
Weight of Can + dry soil (g)	28.5	28.6	28.8	28.4	29.5
Weight of wet soil (g)	5.5	5.6	5.5	6.3	6.3
Weight of dry soil (g)	4.4	4.3	4.2	4.5	4.1
Moisture Content (%)	25.0	30.2	31.0	40.0	53.7

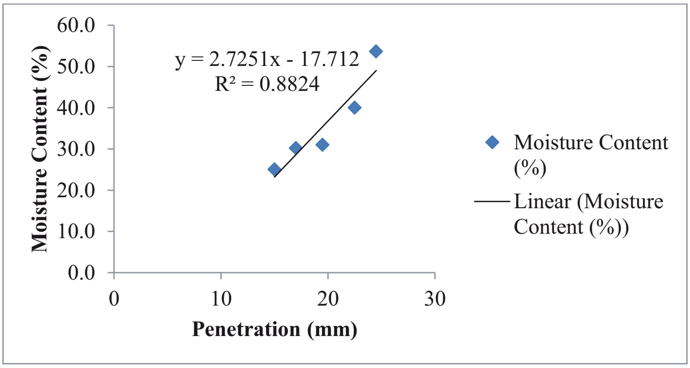


Figure 4.2: Plot of Liquid Limit of Laterite Sample Used

Using the equation of the line of best fit given as **$y = 2.725x - 17.71$ and R^2 = 0.882**

Hence Liquid Limit (L. L .i.e. Moisture Content at 20 mm penetration) = **36.79.**

Table 4.4 present the Plastic Limit=**26.11,** while the Plastic Index = L. L – P. L =**10.68**.

Table 4.4: Plastic Limit of Laterite Sample Used

	Plastic Limit	
Can Number	20	10
Weight of Can (g)	24.9	24.3
Weight of Can + wet Soil (g)	26.2	25.4
Weight of Can + dry soil (g)	25.9	25.2
Weight of wet soil (g)	1.3	1.1
Weight of dry soil (g)	1.0	0.9
Moisture Content (%)	30.0	22.2
Average	26.11	

The result shows the laterite sample has Atterberg limits conforming to the range as specified by the findings of Abidoye (1977).

4.2.2 Chemical Analysis of Laterite

The result of the chemical analysis carried out on the Laterite sample as shown in Table 4.5.

Table 4.5: Result of Chemical Analysis of Laterite Sample

Elements	% Composition by weight	Others	Values
SiO_2	40.95	Cl^-	0.00
Al_2O_3	20.38	L.O.I	
Fe_2O_3	21.95	SUM	83.76
CaO	-0.65	LSF	-0.34
MgO	-0.62	SR	0.97
K_2O	0.32	AR	0.93
Na_2O	0.23	C_3S	-487.34
P_2O_5	0.03	C_2S	-481.23
TiO_2	1.14	C_3A	16.92
Mn_2O_3	0.16	C_4AF	36.45
SO_3	-0.14	$Al_2O_3+Fe_2O_3$	42.33
Total $SiO_2+Al_2O_3+Fe_2O_3$	83.28		

It reflects Silica – Sesquioxide (s-s) Ratio tagged SR in the Table, as 0.97 implying a **true laterite.**

The laterite sample is noted to be reddish brown in colour and have a high quantity of Silica (SiO_2 = 40.95%), average Iron Oxide and Aluminium content (Fe_2O_3 = 21.95% and Al_2O_3 = 20.38%) and can be classified to be ferruginous but not bauxite in line with Tietz (1997) classification since the Iron content is higher than the Aluminium content.

4.2.3 Chemical Analysis of the Volcanic Ash Sample

The result of Chemical analysis of the Volcanic Ash sample is as shown in Table 4.6.

This reflects a Silicon Dioxide content of 41.13% which is greater than BS EN 197-1(2000) minimum requirement of 25.0% and a total Silicon Dioxide, Iron Oxide, and Aluminium

Oxide (SiO_2+Fe_2O_3+Al_2O_3) content of 70.99% which is slightly higher than the values gotten in earlier studies 63.74% by Lar and Tsalha (2005) and 67.14% by Hassan (2006).

Table 4.6: Result of Chemical Analysis of the Volcanic Ash Sample

Elements	% Composition by weight
SiO_2	41.13
Al_2O_3	18.36
Fe_2O_3	11.5
CaO	6.57
MgO	4.24
SO_3	-0.13
K_2O	1.12
Na_2O	1.29
Mn_2O_3	0.29
P_2O_5	1
TiO_2	3.56
Cl-	0
SUM	88.92
LSF	4.64
SR	1.38
AR	1.6
C_3S	-430.78
C_2S	439.08
C_3A	29.21
C_4AF	34.95
L.O.I	8.30
SiO_2+Al_2O_3+Fe_2O_3	70.99

This new value is noted to be slightly above the code (ASTM C618 – 2008) requirement of 70% minimum for a pozzolan. The SO_3 content is -0.13 which is below the maximum value of 4.0% as specified for Class N pozzolan to which it belongs; in ASTM C618-2008. The loss on ignition (8.60) though higher than the value (2.71) gotten in earlier study by Hassan (2006), is also below the maximum allowable (10.0) specified. The Volcanic Ash sample from Mangu, in Kerang Local Government Area of Plateau State, Nigeria; which was used for the research work can then be said to be a pozzolan on basis of Chemical composition.

4.2.4 Physical Properties of the VA-Blended Cement

The physical properties of the VA-Blended cement are presented in Table 4.7.

Table 4.7: Summary of Physical Properties of VA-Blended Cement

Parameters	Percentage Replacement by VA			
	0	10	20	30
Fineness (% Residue on 75μm Sieve)	12.5	11.0	8.5	7.0
Fineness (% Residue on 53μm Sieve)	52.0	39.0	34.5	33.5
Soundness (mm)	1.5	2.5	3.5	4.5
Consistency (%)	30.0	30.0	31.0	31.5
Water Requirement (% of control)	100.00	100.2	104.2	104.8
Initial Setting Time (min)	50	75	83	105
Final Setting Time (min)	135	165	175	180

The Fineness Test (residue on75μm and 53μm sieve) shows that the blended cements were finer than the control (Dangote - Obajana) cement. Both 75μm and 53μm sieve was used since the VA had been made to pass a 75μm, hence using a 90μm as specified by BS EN 196 – 6:2005 will be in-appropriate and a 45μm was not available in the laboratory in F.U.T Minna. The higher the VA content in the blended cement the lower the residue observed in both cases. The Table also reveals that the soundness of the cement ranges between 1.5 and 4.5 for replacement levels of 0% to 30%. These values are far lesser than the 10 mm limiting value recommended by both NIS 439:2000 and BS EN 197 – 1:2000. Hence the blended cement does not show any appreciable change in volume after setting.

The consistency increases from 30.0% to 31.5% as VA substitution increases from 0% to 30%. The water required for a standard consistency was noted to increase as the VA content increases, although this was noted to be within the limit of 115% as specified for Class N pozzolan in ASTM C618:2008.

The initial and final setting times increased from 50 to 105 minutes and 135 to 180 minutes respectively when percentage VA replacement increased from 0% to 30%. All the cement satisfy the NIS 439:2000 and BS EN 197 – 1:2000 requirements of 45minutes minimum initial setting time and maximum of 10 hours final setting time as spelt out by NIS 439:2000 and the 375 minutes maximum specified for final setting time by ASTM C150. BS EN 197 -1:2000 was however silent on the maximum final setting time. The variation of setting times with percentage VA replacements shows that both initial and final setting times increased as the percentage VA increased. As a result, the hydration process is slowed down in consonance with the views of Hossain (2003). The slow hydration means low rate of heat development which is one of the notable characteristics for which pozzolanic cements are known. This is of great importance in mass concrete construction where low rate of heat development is very essential as it reduces thermal stress.

A plot of the initial setting time against the final setting time as shown in Figure 4.3, indicates a very strong linear relationship between the parameters as the coefficient of correlation was calculated to be 0.944 (square root of R^2).

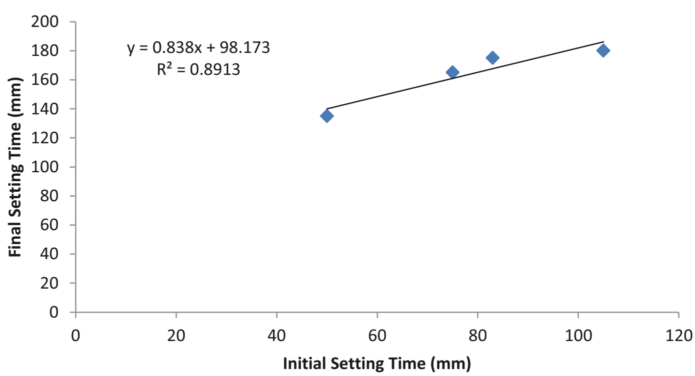


Figure 4.3: Relationship between Initial and Final Setting Times of VA-Blended Cement

As stated by Johnson (1994), a strong relationship exists between two variables when 0.5< /r/ <1. Thus an estimate of the final setting time can be calculated from equation 4.1 when an initial time has been obtained.

$$y = 0.838x + 98.71 \qquad (4.1)$$

Where: y = final setting time

x = initial setting time

4.3 CHARACTERISTICS OF VA-BLENDED CEMENT LATERIZED CONCRETE

The characteristics of the Laterized concrete produced using the VA – blended cement investigated were Workability, Compressive Strength, Tensile Splitting Strength and Flexural Strength. The British Method (i.e. D. O. E method) was adopted with 28day target strength of 25 N/mm^2 taken as control. Table B.1 of the Appendix shows the mix design detail while Table B.2 presents the breakdown of its application for the various replacement levels of laterite and volcanic ash. Adjustment was made for the moisture content of the laterite only as the percentage replacement varies to ensure the slump range of 10 to 30 mm is maintained all through, for this is the first precaution at arriving at the target strength on the 28day.

4.3.1 Workability

The result of the slumps indicating the workability of the VA-blended cement laterized concrete is shown in Table 4.8. It indicates that the concrete slump decreased as the VA content increased. The adjustment made to the water used to mix based on the water requirement of the ash as established in the consistency test can be said to have help maintain the slump values of all the various proportions within the limit of the mix design i.e. 10 to 30 mm, this infers that the control mix ought to achieve the target strength by the 28day. Implying an ideal mix range for strength properties comparison exists.

Table 4.8 presents the slump values in the groupings of the laterite content in the concrete and hence the mixing water content. Despite the adjustment made to the mixing water in relation to ash content the trend of slump still shows decrease as the laterite content increased, indicating an increase in requisite water requirement in Laterized Concrete, this is in consonance with all the previous studies on laterized concrete.

Table 4.8: Slump Values for VA-blended Cement Laterized Concrete

Concrete mix		Slump value (mm)
Lat Cont. (%)	VA Cont. (%)	
0	0	26
0	10	22
0	20	18
0	30	16
10	0	24
10	10	20
10	20	18
10	30	16
20	0	22
20	10	18
20	20	14
20	30	12
30	0	20
30	10	17
30	20	13
30	30	11

4.3.2 Compressive Strength

The mean compressive strength (i.e. average of the triplicate) of VA-blended cement laterized concrete and the effects of curing age and percentage replacements of cement with volcanic ash presented in Table 4.9, while Figures 4.4 to 4.7 shows graphically the effect of this variables at the various levels of laterite content (0%, 10%, 20% and 30% respectively). The details of the strength calculation for respective curing ages are shown in Table B.3 to B.8 of appendix B.

The mean compressive strength for each curing age, replacement level and its percentage of the 28day strength of the control are shown in Table 4.9, while the figures present graphically the effect of these variables on the compressive strength of the laterized concrete at the curing ages of 7, 28, 56, 90 and 120days as selected for this research. The compressive strength generally increased with curing age and decreased with increased percentage of volcanic ash in the mix.

Table 4.9: Summary of Compressive Strength of VA-blended Cement Laterized Concrete

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Compressive Strength (N/mm^2) 3 Days	7 Days	28 Days	56 Days	90 Days	120 Days
A	0	0	11.02 (42.43)	18.5 (71.24)	25.97 (100.00)	27.44 (105.66)	28.47 (109.63)	29.14 (112.21)
B	0	10	8.61 (33.15)	12.39 (47.71)	21.97 (84.60)	21.97 (84.60)	25.14 (96.80)	26.34 (101.42)
C	0	20	7.38 (28.42)	10.79 (41.55)	20.07 (77.28)	21.74 (83.71)	22.19 (85.44)	22.57 (86.91)
D	0	30	6.45 (24.84)	9.82 (37.81)	18.96 (73.43)	18.96 (73.01)	20.97 (80.75)	22.02 (84.79)
E	10	0	8.67 (33.380	11.55 (44.47)	21.55 (82.98)	22.21 (85.52)	26.78 (103.12)	27.33 (105.24)
F	10	10	8.03 (30.92)	9.77 (37.62)	19.07 (73.43)	21.67 (83.44)	24.05 (92.61)	25.09 (96.61)
G	10	20	7.04 (27.11)	8.62 (33.19)	18.38 (70.77)	20.98 (80.79)	23.52 (90.57)	23.65 (91.07)
H	10	30	5.11 (19.68)	6.80 (26.18)	18.2 (70.08)	19.63 (75.59)	20.97 (80.75)	21.07 (81.13)
I	20	0	8.89 (34.23)	9.97 (38.390	21.06 (81.09)	24.19 (93.15)	26.69 (102.77)	27.06 (104.20)
J	20	10	8.52 (32.81)	9.49 (36.54)	19.26 (74.16)	21.76 (83.79)	24.16 (93.03)	24.96 (96.11)
K	20	20	7.24 (27.88)	7.65 (29.46)	18.4 (70.85)	21.53 (82.90)	24.43 (94.07)	24.52 (94.42)
L	20	30	5.19 (19.98)	6.98 (26.88)	17.72 (68.23)	20.75 (79.90)	21.75 (83.75)	22.95 (88.37)
M	30	0	8.46 (32.58)	9.74 (37.50)	20.59 (79.28)	22.93 (88.29)	25.26 (97.27)	26.19 (100.82)
N	30	10	7.13 (27.45)	8.24 (31.73)	18.9 (72.78)	21.97 (84.60)	25.00 (96.26)	25.89 (99.69)
O	30	20	6.64 (25.570	7.63 (29.38)	18.03 (69.43)	20.9 (80.48)	23.66 (91.11)	23.77 (91.53)
P	30	30	5.02 (19.33)	6.28 (24.18)	17.46 (67.23)	20.46 (78.78)	23.46 (90.34)	23.69 (91.22)

Note: Value in parenthesis refers to percentage of 28day strength of the control (0%Lat/0%VA).

Up to the 20%Lat/20%VA replacements, the VA-blended cement laterized concrete has a minimum of 70% strength of the control at the 28day, at ages beyond 28days the VA-blended cement laterized concrete shows strengths comparative to that of the control. Specifically at the 120days, the lowest strength was 23.69N/ mm^2 (91.22% of 28day of control); this is in consonance with the code's (ASTM C618:2008) expectation of pozzolanic cement and in line with Matawal (2005) postulations.

The 28day strength of 20%Lat/20%VA sample (70.91%); though is a bit lower than the code requirement for pozzolanicity test, this sample can still be seen as the limit of replacement with the hope that the 75% requirement can be met with little treatment on the volcanic ash to improve its chemical composition.

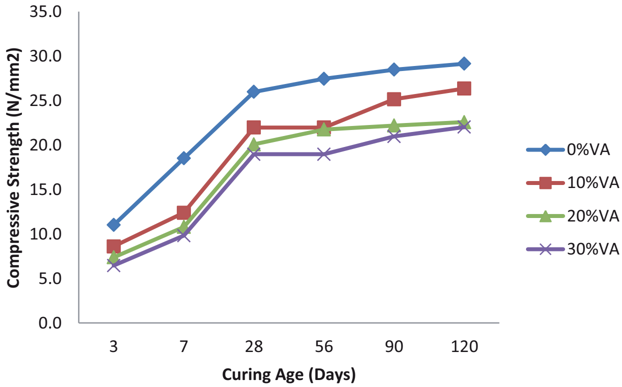


Figure 4.4: Compressive Strength of VA-blended Cement Laterized Concrete at 0% Laterite

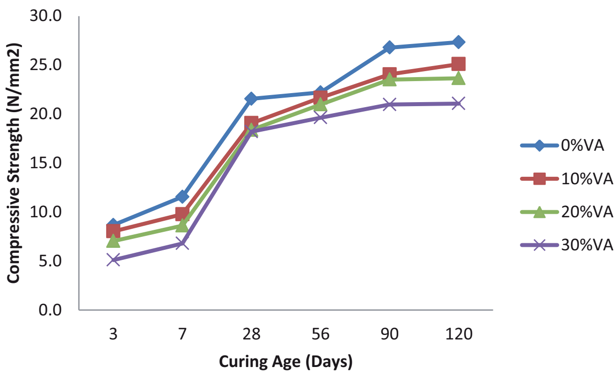


Figure 4.5: Compressive Strength of VA-blended Cement Laterized Concrete at 10% Laterite

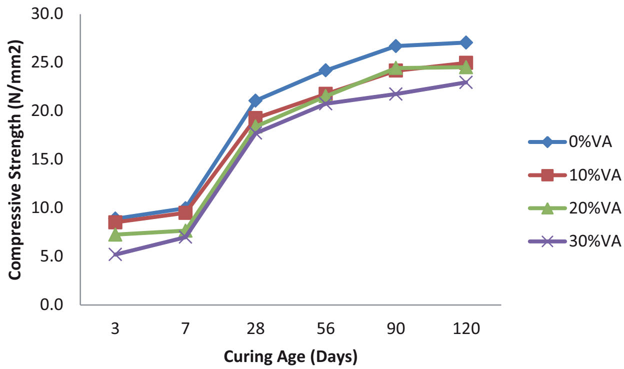


Figure 4.6: Compressive Strength of VA-blended Cement Laterized Concrete at 20% Laterite

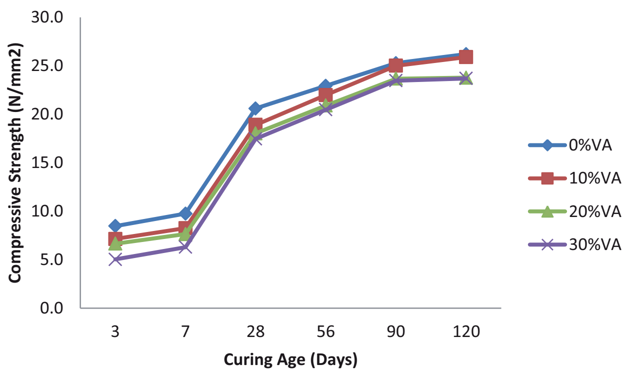


Figure 4.7: Compressive Strength of VA-blended Cement Laterized Concrete at 30% Laterite

The results as shown in Fig. 4.4 to Fig.4.7 reflects that the rate of strength development of VA-blended cement laterized concrete was slow at early curing ages but faster at later ages, unlike the strength development of the control (i.e. 0%Lat/0%VA – normal concrete) which accelerates at the initial stage and then decelerates after 28days. These results corroborate earlier findings on pozzolan cement concrete (Raheem, 2006; Hassan, 2006; Antiohos *et al.*, 2005, and Kim *et al.*, 2003). This implies that VA-blended cement laterized concrete is not advisable for use when early strength is required; rather it is mostly applicable for structures requiring long term strength development. Thus, it could be concluded that the strength characteristics of VA-blended cement laterized concrete is a function of the curing age and percentage VA content.

The influence of laterite content, volcanic ash and curing age (called independent variables) on the compressive strength (called dependent variable) was statistically analyzed using Analysis of Variance (ANOVA) and the result is presented as shown in Table 4.10

The analysis was aimed at determining which of the factors considered had significant effect on the compressive strength of the concrete. The results of the statistical analysis as shown in the table, indicated that all the independent factors, when considered individually and collectively (two and three factors interactions) had significant effects on the compressive strength of the concrete at 95% confidence level ($\alpha = 0.05$). This indicates that whenever any of the factors is varied, the compressive strength of the concrete changes and the degree of the variation is proportional to the magnitude of the change. The coefficient of determination (adjusted R-Square value) obtained from the analysis was 0.986 (98.6%). This implies a strong statistical association among the three independent variables and the dependent variable. The independent variables were estimated to account for 98.6% of the variance in the compressive strength of the concrete. The coefficient of correlation (square root of adjusted R-square) was obtained as R = 0.993. This shows that a very strong correlation or linear relationship exist between the two sets of variables being considered. A strong correlation is assumed to exist between two variables if $0.5 < r < 1.0$, otherwise the correlation is weak. The statistical analysis (see Appendix C) revealed that the mean compressive strength for all curing ages' and replacement levels of laterite and volcanic ash curing ages is 17.93 N/ mm^2. The Duncan's multiple range tests (see Tables 4.10 to 4.13) revealed that the mean compressive strengths for the various VA content are significantly different with 0%VA having the highest mean compressive strength of 24.00 N/ mm^2 when other variables are kept constant. When volcanic ash content and curing ages were kept constant and the effect of percent replacement of sand with laterite was statistically investigated using the Duncan's multiple range tests, the result also shows that the mean compressive strengths at the different percent replacement levels are significantly different.

Table 4.10: Results of ANOVA for Compressive Strength Test

Source	DF	Type III SS	Mean Square	F Value	Pr > F
LatCont	3	140.38844	46.79615	65.17	<.0001
VaCont	3	803.44649	267.81550	372.99	<.0001
LatCont*VaCont	9	127.76059	14.19562	19.77	<.0001
CuringAge	5	13543.90976	2708.78195	3772.57	<.0001
LatCont*CuringAge	15	137.83094	9.18873	12.80	<.0001
VaCont*CuringAge	15	23.17288	1.54486	2.15	0.0093
LatCon*VaCont*Curing	45	53.24587	1.18324	1.65	0.0112
Error	192	137.86000	0.71802		

Table 4.11: Duncan's Multiple Range Test for Compressive Strength with Varying VA Content

Duncan Grouping	Mean	N	VA Cont
A	20.4000	72	0
B	18.3208	72	10
C	17.1431	72	20
D	15.8597	72	30

The highest mean compressive strength was 19.12 N/ mm^2 for 0% laterite content followed by a mean strength of 17.71 N/ mm^2 for 20% replacement of sand with laterite; this implies the 20% laterite content is the optimum level of replacement of sand with laterite in this study. The effect of curing ages on the compressive strength of the VA-blended cement laterized concrete when other variables were kept constant also shows that the mean compressive strength of the curing ages tested (3, 7, 28, 56, 90 and 120) are significantly different. The highest mean compressive strength, 24.77 N/ mm^2, was attained at 120 days which implies the compressive strength increases as the hydration period increases, the mean compressive strength value at 120days tallies approximately with the 28day design strength.

Table 4.12: Duncan's Multiple Range Test for Compressive Strength with Varying LAT Content

Duncan Grouping		Mean	N	LAT Cont
A		19.1236	72	0
B		17.7139	72	20
C	B	17.4889	72	10
C		17.3972	72	30

Table 4.13: Duncan's Multiple Range Test for Compressive Strength with Varying Curing Age

Duncan Grouping	Mean	N	Curing Age
A	24.7688	48	120
B	24.1583	48	90
C	21.8229	48	56
D	19.7292	48	28
E	9.6396	48	7
F	7.4667	48	3

4.3.3 Tensile Splitting Strength

The results of the Tensile Splitting strength for the VA-blended cement laterized concrete are shown in Table B.9 to B.14 of appendix B with the mean values presented in Table 4.14 while Figures 4.8 to 4.11 gives the graphical presentation.

Table 4.14 show the mean Tensile Splitting strength while the values in parentheses give its percentage to the 28day Tensile Splitting strength value of the control. The figures on the other hand shows the graphical representation of the effect of curing age and the percentage replacements of cement with volcanic ash at the various levels of laterite content respectively.

The trend indicate a general increase in tensile splitting strength as the curing age increases and decreases as the VA content increases. The values of the tensile splitting strength literally do not change between the 90th and 120th day curing age; beyond the 28day the rate of increase of the tensile splitting strength is relatively low as compared with the rate before

28day. This is in agreement with expectations for concrete and those containing pozzolanic materials in general.

Table 4.14: Summary of Tensile Splitting Strength of VA-blended Cement Laterized Concrete

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Tensile Splitting Strength (N/mm^2)					
			3 Days	7 Days	28 Days	56 Days	90 Days	120 Days
A	0	0	1.64 (65.60)	1.84 (73.60)	2.50 (100.00)	2.55 (102.00)	2.64 (105.60)	2.64 (105.60)
B	0	10	1.44 (57.60)	1.54 (61.60)	2.10 (84.00)	2.37 (94.80)	2.46 (98.40)	2.46 (98.40)
C	0	20	1.41 (56.40)	1.48 (59.20)	2.05 (82.00)	2.34 (93.60)	2.43 (97.20)	2.43 (97.20)
D	0	30	1.39 (55.60)	1.42 (56.80)	1.97 (78.80)	2.29 (91.600	2.33 (93.20)	2.33 (93.20)
E	10	0	1.61 (64.40)	1.81 (72.40)	2.47 (98.80)	2.48 (99.20)	2.49 (99.60)	2.49 (99.60)
F	10	10	1.31 (52.40)	1.51 (60.40)	2.01 (80.40)	2.04 (92.40)	2.31 (92.40)	2.31 (92.40)
G	10	20	1.23 (49.40)	1.44 (57.60)	1.89 (75.60)	1.90 (76.00)	1.96 (78.40)	1.97 (78.80)
H	10	30	1.16 (46.40)	1.34 (53.60)	1.79 (71.60)	1.83 (73.20)	1.92 (76.80)	1.93 (77.20)
I	20	0	1.57 (62.80)	1.80 (72.00)	2.42 (96.80)	2.46 (98.40)	2.47 (98.80)	2.47 (98.80)
J	20	10	1.54 (61.60)	1.50 (60.00)	2.37 (94.80)	2.38 (95.20)	2.44 (97.60)	2.44 (97.60)
K	20	20	1.14 (45.60)	1.32 (52.80)	1.76 (70.40)	2.00 (80.00)	2.24 (89.60)	2.24 (89.60)
L	20	30	1.11 (44.40)	1.28 (51.20)	1.71 (68.40)	1.97 (78.80)	2.22 (88.80)	2.22 (88.80)
M	30	0	1.21 (48.40)	1.28 (51.20)	1.69 (67.60)	1.92 (76.80)	2.19 (87.60)	2.19 (87.60)
N	30	10	1.12 (44.80)	1.24 (49.60)	1.65 (66.60)	1.90 (76.00)	2.15 (86.00)	2.15 (86.00)
O	30	20	0.96 (38.40)	1.11 (44.40)	1.48 (59.20)	1.79 (71.60)	2.10 (84.00)	2.10 (84.00)
P	30	30	0.94 (37.60)	1.09 (43.60)	1.45 (58.00)	1.76 (70.40)	2.07 (82.80)	2.07 (82.80)

Note: The values in parenthesis are percentage of 28day Strength of Control (i.e.0%Lat/0%VA)

The plot at the 30% laterite content shows the best pattern of curve for the various levels of VA content while the other levels of laterite content shows similar pattern of upward tensile splitting strength as the curing ages increase with age 90days being the optimum

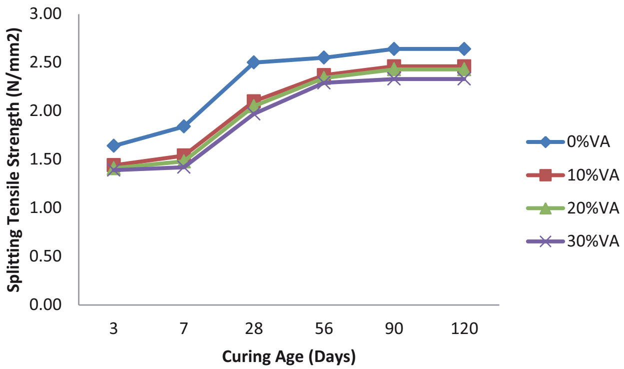


Figure 4.8: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 0% Laterite

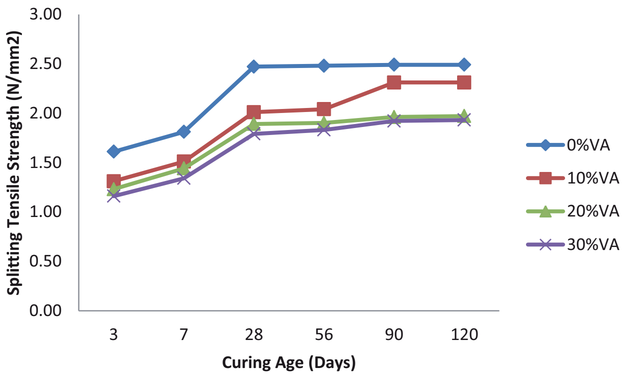


Figure 4.9: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 10% Laterite

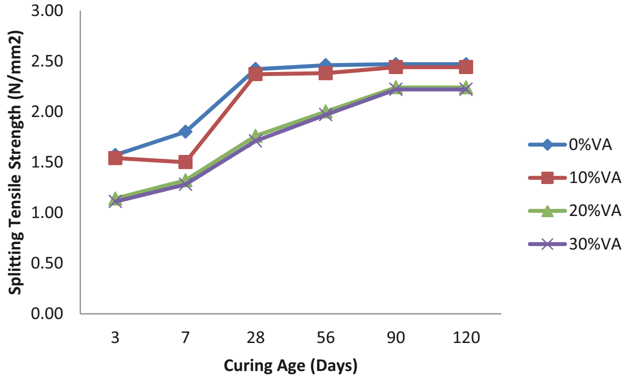


Figure 4.10: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 20% Laterite

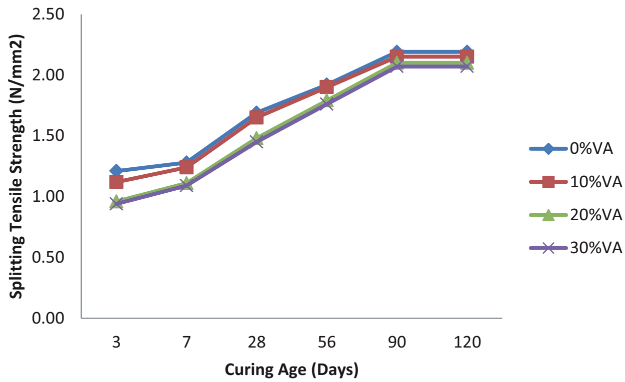


Figure 4.11: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 30% Laterite

The result of the statistical analysis shown in the ANOVA table (Table 4.15) revealed that the independent variables; laterite content, volcanic ash content and curing age, each had a significant effect on the tensile splitting strength of the VA-blended cement laterized concrete. It further shows that the two factor interactions of laterite content and volcanic ash content; laterite content and curing age also had significant effect on the tensile splitting strength while the effect of two factor interaction of volcanic ash and curing age on the other hand is not significant. The three factor interaction was noted not to be insignificant. The mean tensile splitting strength is 1.90N/ mm^2 which is about 10% of the mean compressive strength (see Appendix C). This is similar to normal concrete.

The coefficient of determination (adjusted R-Square value) obtained from the analysis was 0.927 (92.7%). This implies a strong statistical association among the three independent variables and the dependent variable. The independent variables were estimated to account for 92.7% of the variance in the tensile strength of the concrete. The Duncan's Multiple Range Test for tensile splitting strength (Tables 4.16) show that the means of the laterized concrete with different replacement levels of cement with volcanic ash (0%, 10%, 20% and 30%) when other variables are held constant are significantly different (2.12361N/ mm^2, 1.94903N/ mm^2, 1.78319N/ mm^2 and 1.73417N/ mm^2). The means of the VA-blended cement laterized concrete with different replacement level of sand with laterite (0%, 10%, 20%, and 30%) and varying curing ages (7, 28, 56, 90 and 120 days) are also significantly different (Tables 4.17 & 4.18).

Table 4.15: Results of ANOVA for Tensile Spitting Strength Test

Source	DF	Type III SS	Mean Square	F Value	Pr > F
LatCont	3	7.38538611	2.46179537	150.02	<.0001
VaCont	3	6.73379167	2.24459722	136.78	<.0001
LatCont*VaCont	9	1.29848889	0.14427654	8.79	<.0001
CuringAge	5	44.24445417	8.84889083	539.24	<.0001
LatCont*CuringAge	15	1.17289306	0.07819287	4.76	<.0001
VaCont*CuringAge	15	0.28125417	0.01875028	1.14	0.3210
LatCon*VaCont*Curing	45	0.54859861	0.01219108	0.74	0.8806
Error	192	3.15073333	0.01641007		

Table 4.16: Duncan's Multiple Range Test for Tensile Splitting Strength with Varying VA Content

Duncan Grouping	Mean	N	VA Cont
A	2.12361	72	0
B	1.94903	72	10
C	1.78319	72	20
D	1.73417	72	30

Table 4.17: Duncan's Multiple Range Test for Tensile Splitting Strength with Varying LAT Content

Duncan Grouping	Mean	N	LAT Cont
A	2.08750	72	0
B	1.96847	72	20
C	1.88403	72	10
D	1.65000	72	30

Table 4.18: Duncan's Multiple Range Test for Tensile Splitting Strength with Varying Curing Age

Duncan Grouping	Mean	N	Curing Age
A	2.28771	48	90
A	2.27833	48	120
B	2.12458	48	56
C	1.95750	48	28
D	1.43729	48	7
E	1.29958	48	3

However, the means of the tensile strength of the VA-blended cement laterized concrete for curing ages 90 and 120 days are seen not to be significantly different for the various levels of volcanic ash and laterite contents.

4.3.4 Flexural Strength

Results of the flexural strength test on VA-blended cement laterized concrete are presented in Tables B.15 to B.20 of appendix B, while the mean values and its corresponding percentage of the 28day strength of the control (0%Lat/0%VA) shown in parenthesis are reflected in Table 4.19. Figures 4.12 to 4.15 on the other hand present graphically the trends in flexural strength development with curing age for the various levels of laterite content (0%, 10%, 20% and 30%) respectively.

The results indicate that flexural strength increases generally as the curing age increases and decreases as the VA content increases. The rate of increase beyond the 28day curing is not as high as at the early days up to the 28th day just as outlined by literatures (Neville, 2006; Shetty, 2004 and

Neville and Brooks, 2002). Between the 90th and 120th day, there seems to be little or no increase in the flexural strength.

Table 4.19: Summary of Flexural Strength of VA-blended Cement Laterized Concrete

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Flexural Strength (N/mm^2) 3 Days	7 Days	28 Days	56 Days	90 Days	120 Days
A	0	0	1.75 (42.37)	2.96 (71.67)	4.13 (100.00)	4.66 (112.83)	4.68 (113.32)	4.90 (118.64)
B	0	10	1.65 (39.95)	2.67 (64.65)	4.11 (99.52)	4.62 (111.83)	4.63 (112.11)	4.65 (112.59)
C	0	20	1.61 (38.98)	2.64 (63.92)	4.08 (98.79)	4.61 (111.62)	4.62 (111.86)	4.63 (112.11)
D	0	30	1.55 (37.53)	2.37 (57.38)	3.98 (96.37)	4.40 (106.54)	4.43 (107.26)	4.53 (109.69)
E	10	0	1.66 (40.19)	2.45 (59.32)	4.10 (99.37)	4.37 (105.81)	4.47 (108.23)	4.48 (108.47)
F	10	10	1.41 (34.14)	1.67 (40.44)	4.04 (97.82)	4.20 (101.69)	4.20 (101.69)	4.21 (101.94)
G	10	20	1.16 (28.09)	1.65 (39.95)	3.92 (94.92)	4.06 (98.31)	4.18 (101.21)	4.17 (100.97)
H	10	30	1.08 (26.15)	1.63 (39.47)	3.89 (94.19)	4.01 (97.09)	4.13 (100.00)	4.14 (100.24)
I	20	0	1.51 (36.56)	2.26 (54.72)	3.58 (86.68)	3.76 (91.04)	3.79 (91.77)	3.80 (92.01)
J	20	10	1.33 (32.20)	1.95 (47.22)	3.54 (85.71)	3.68 (89.10)	3.75 (90.80)	3.77 (91.28)
K	20	20	1.09 (26.39)	1.92 (46.49)	3.44 (83.29)	3.63 (87.89)	3.65 (88.38)	3.66 (88.62)
L	20	30	1.05 (25.42)	1.90 (46.00)	3.38 (81.84)	3.47 (84.02)	3.48 (84.26)	3.48 (84.26)
M	30	0	1.34 (32.45)	2.22 (53.75)	3.49 (84.50)	3.50 (84.75)	3.56 (86.20)	3.56 (86.20)
N	30	10	1.24 (30.02)	2.05 (49.64)	3.35 (81.11)	3.40 (82.32)	3.53 (85.47)	3.53 (85.47)
O	30	20	1.24 (30.02)	2.01 (48.67)	3.31 (80.15)	3.38 (81.84)	3.52 (85.23)	3.53 (85.47)
P	30	30	0.99 (23.97)	1.76 (42.62)	3.19 (77.24)	3.24 (78.45)	3.25 (78.69)	3.25 (78.69)

Note: The values in parenthesis are percentage of 28day Strength of Control (i.e.0%Lat/0%VA)

The Flexural Strength was also noted to decrease as the laterite content increases. For instance at 0%Lat/0%VA (i.e. control), the flexural strength is 4.13N/ mm^2 at the 28th day curing, while the values for 10%Lat/0%VA, 20%Lat/0%VA, and 30%Lat/0%VA are 4.10N/ mm^2, 3.58N/ mm^2 and

3.49N/ mm^2 respectively. The values of flexural strength were however noted to be higher than values gotten for tensile splitting strength, thereby confirming the views expressed in literatures.

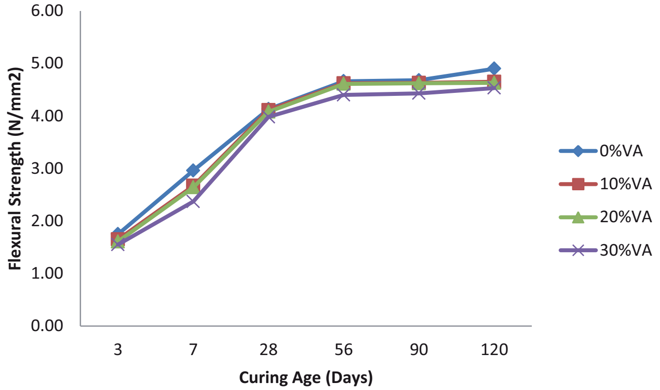


Figure 4.12: Flexural Strength of VA-blended Cement Laterized Concrete at 0% laterite

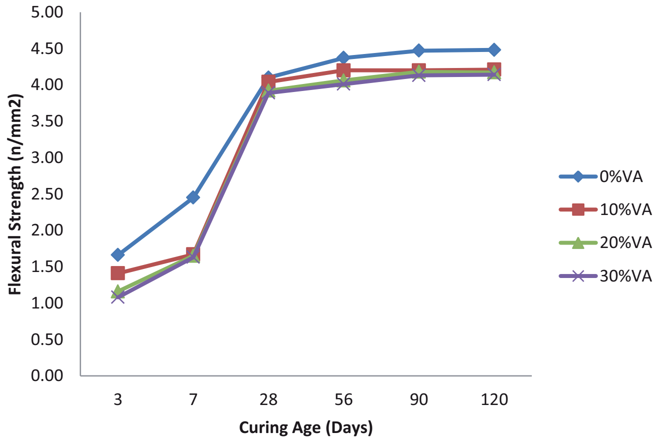


Figure 4.13: Flexural Strength of VA-blended Cement Laterized Concrete at 10% laterite

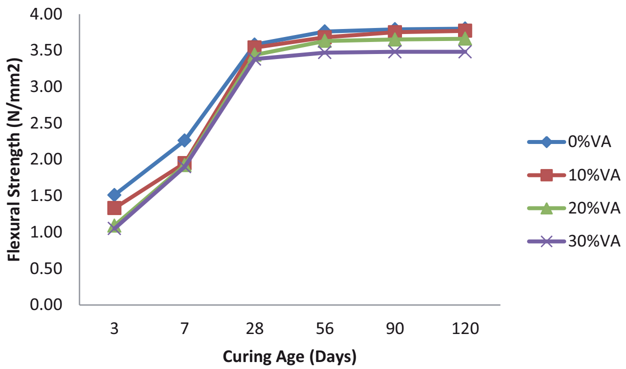


Figure 4.14: Flexural Strength of VA-blended Cement Laterized Concrete at 20% laterite

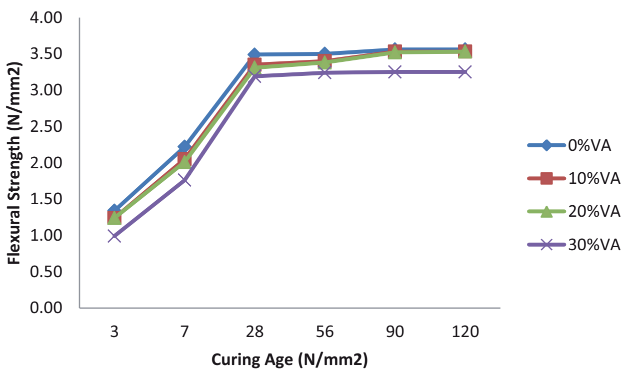


Figure 4.15: Flexural Strength of VA-blended Cement Laterized Concrete at 30% laterite

The result of the statistical analysis shown in the ANOVA table (Table 4.20) revealed that the independent variables; volcanic ash content, laterite content and curing age, each had a significant effect on the flexural strength of the VA-blended cement laterized concrete. It is also shown that the two factor interactions also had significant effect on the flexural strength. The mean flexural strength is 3.192N/ mm^2 which is about 18% of the mean compressive strength (see Appendix C). This is similar to what was observed for normal concrete in literatures (Shetty, 2004).

The coefficient of determination (adjusted R-Square value) obtained from the analysis was 0.991 (99.2%). This implies a strong statistical association among the three independent variables and the dependent variation. The independent variables were estimated to account for 99.2% of the variance in the flexural strength of the laterized concrete. The Duncan's Multiple Range Test for flexural strength (Tables 4.21) show means of the laterized concrete with different replacement levels of cement with volcanic ash (0%, 10%, 20% and 30%) when other variables were held constant are significantly different (3.37458N/ mm^2, 3.21528N/ mm^2, 3.15361N/ mm^2 and 3.02569N/ mm^2). The means of the VA-blended cement laterized concrete with different replacement level of sand with laterite (0%, 10%, 20%, and 30%) and varying curing ages (7, 28, 56, 90 and 120 days) are also significantly different (Tables 4.22 & 4.23). However, the means of the tensile strength of the VA-blended cement laterized concrete for curing ages 90 and 120 days are seen not to be significantly different for the various levels of volcanic ash and laterite contents.

Table 4.20: Results of ANOVA for Flexural Strength Test

Source	DF	Type III SS	Mean Square	F Value	Pr > F
LatCont	3	34.2290958	11.4096986	966.95	<.0001
VaCont	3	4.5366792	1.5122264	128.16	<.0001
LatCont*VaCont	9	0.5182569	0.0575841	4.88	<.0001
CuringAge	5	319.5566167	63.9113233	5416.37	<.0001
LatCont*CuringAge	15	7.3764417	0.4917628	41.68	<.0001
VaCont*CuringAge	15	0.6616083	0.0441072	3.74	<.0001
LatCon*VaCont*Curing	45	0.5470556	0.0121568	1.03	0.4302
Error	192	2.2655333	0.0117997		

Table 4.21: Duncan's Multiple Range Test for Flexural Strength with Varying VA Content

Duncan Grouping	Mean	N	VA Cont
A	3.37458	72	0
B	3.21528	72	10
C	3.15361	72	20
D	3.02569	72	30

Table 4.22: Duncan's Multiple Range Test for Flexural Strength with Varying LAT Content

Duncan Grouping	Mean	N	LAT Cont
A	3.70167	72	0
B	3.30431	72	10
C	2.95347	72	20
D	2.80972	72	30

Table 4.23: Duncan's Multiple Range Test for Flexural Strength with Varying Curing Age

Duncan Grouping	Mean	N	Age
A	4.01792	48	120
A	3.99188	48	90
B	3.93750	48	56
C	3.72000	48	28
D	2.13250	48	7
E	1.35396	48	3

4.4 RELATIONSHIP BETWEEN COMPRESSIVE STRENGTH AND TENSILE STRENGTH

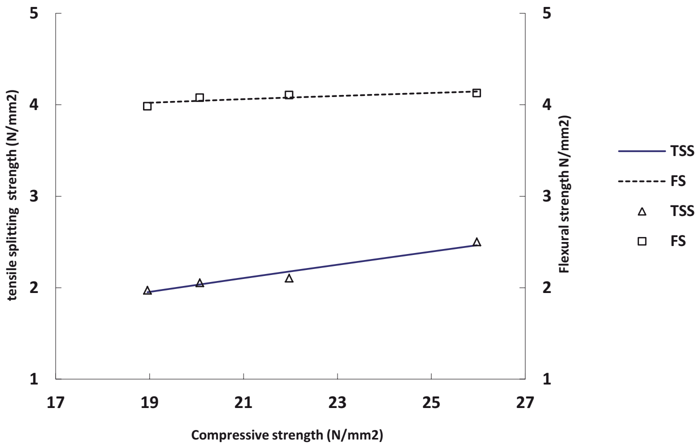
Matrix Laboratory (MATLAB 7.8) software (Release 2009a) was used to perform the curve fitting, parameters estimation and tests of goodness of fit.

The summary of the data generated from the exercise is presented in Table 4.24 while Figures 4.16 to 4.21 shows the plot of the relationships between compressive strength and tensile strength for various levels of sand replacement by laterite (0%, 10%, 20% and 30%).

The R^2 values in Table 4.24 ranges between 0.9895 to 0.7139, implying strong relationships exist between compressive strength and tensile strength (be it tensile splitting or flexural) of the VA-blended cement laterized concrete. The Table reveals that the values of n and k in the generalised Eq. 2.6 *(i.e.* $ft = k\ (fc)^{n}$*)* of the relationship of compressive strength to tensile strength varies for both the tensile splitting strength and the flexural strength of the VA-blended cement laterized concrete. The values of n for the various levels of laterite content (0%, 10%, 20% and 30%) are 0.7422; 1.8071; 2.2102; 0.9756 respectively for tensile splitting strength and 0.0967; 0.2782; 0.3366; 0.5053 for flexural strength. This implies n increases as the laterite content increases. K values for these levels of laterite content are 0.2197; 0.0097; 0.0030 and 0.0897 respectively for tensile splitting and 3.0254; 1.7505; 1.2917; 0.7586 for flexural strength, indicating k decreases as the laterite content increases.

It therefore follows that the relationship between compressive strength and tensile strength of the VA-blended cement laterized concrete is similar to that of the normal concrete, hence the generalised Eq. 2.6 stating that $f_t = k\ (f_c)^n$ is applicable to the laterized concrete while the constants of the relationship are as outlined in Table 4.24.

It was however noted that for tensile splitting strength a turning point exists at 20% laterite content, implying this to be the optimum level for replacement of sand with laterite in VA-blended cement laterized concrete.



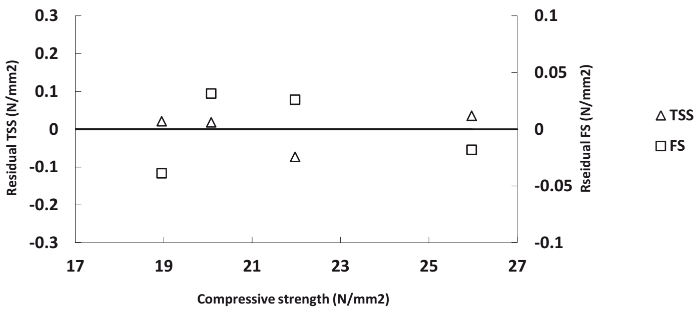
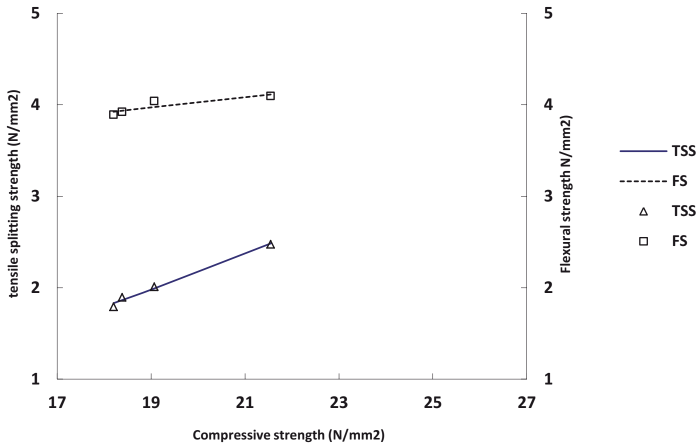


Figure 4.16: Variation of Tensile Splitting Strength and Flexural Strength with Compressive Strength at 0% Lateritic Content

Lines indicate the model fit; Residual = observed – predicted; TSS: tensile splitting strength; FS: flexural strength.



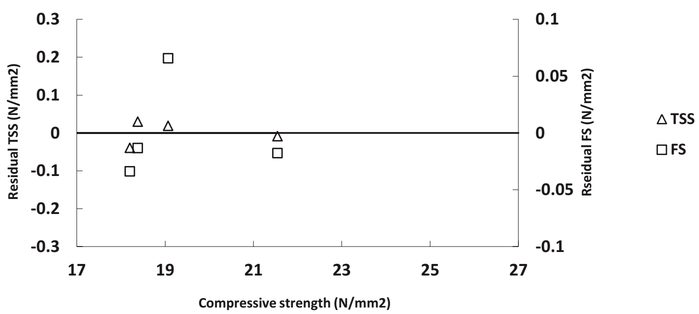
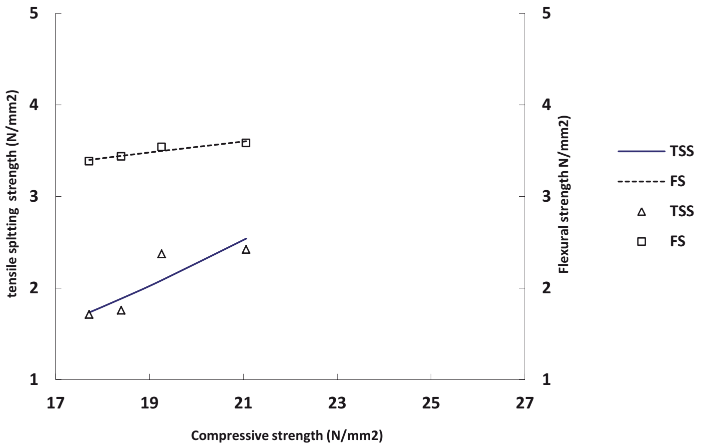


Figure 4.17: Variation of Tensile Splitting Strength and Flexural Strength with Compressive Strength at 10% Lateritic Content

Lines indicate the model fit; Residual = observed – predicted; TSS: tensile splitting strength; FS: flexural strength.



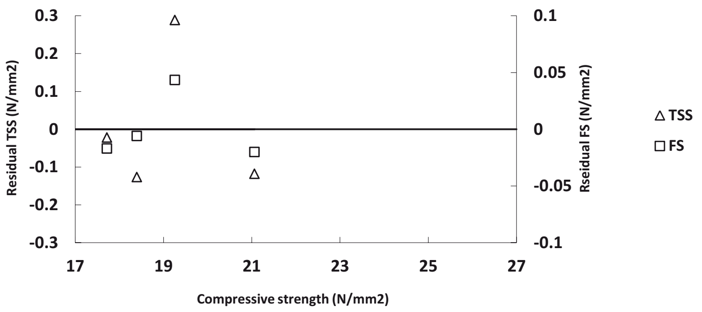


Figure 4.18: Variation of Tensile Splitting Strength and Flexural Strength with Compressive Strength at 20% Lateritic Content

Lines indicate the model fit; Residual = observed – predicted; TSS: tensile splitting strength; FS: flexural strength.

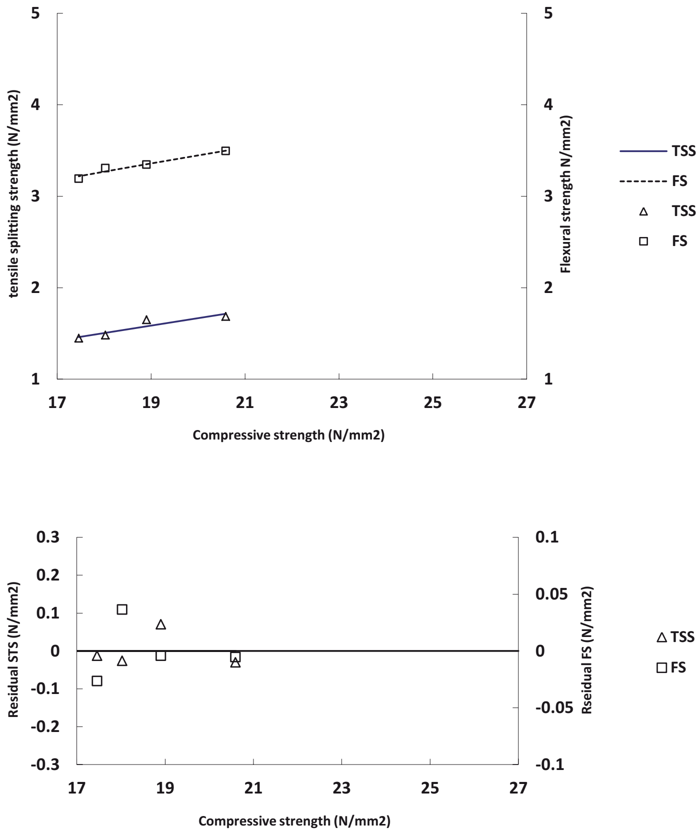


Figure 4.19: Variation of Tensile Splitting Strength and Flexural Strength with Compressive Strength at 30% Lateritic Content

Lines indicate the model fit. Residual = observed – predicted. TSS: tensile splitting strength; FS: flexural strength.

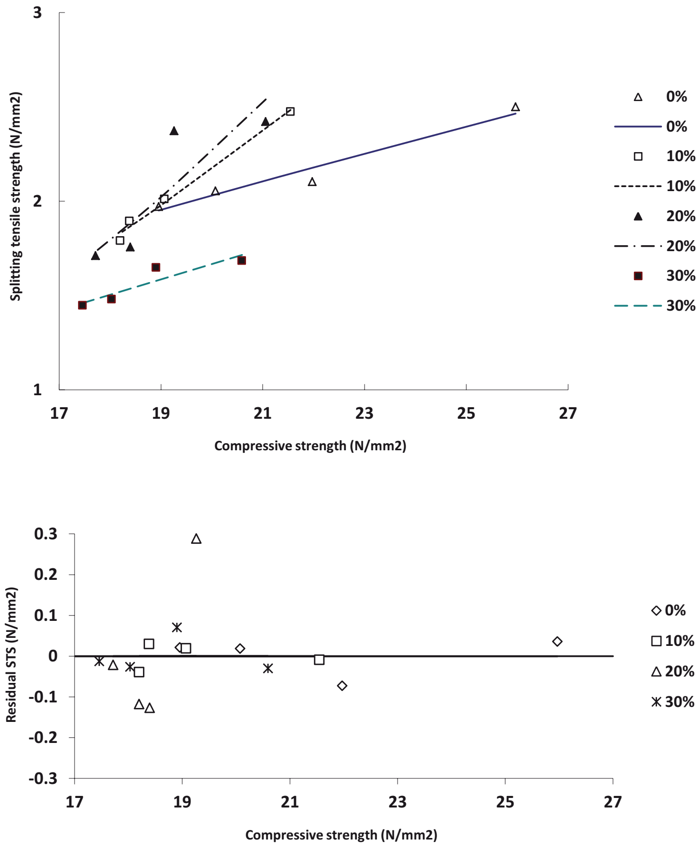


Figure 4.20: Variation of Tensile Splitting Strength with Compressive Strength at the various (0%, 10%, 20% and 30%) Laterite Content

Lines indicate the model fit; Residual = observed – predicted; TSS: tensile splitting strength.

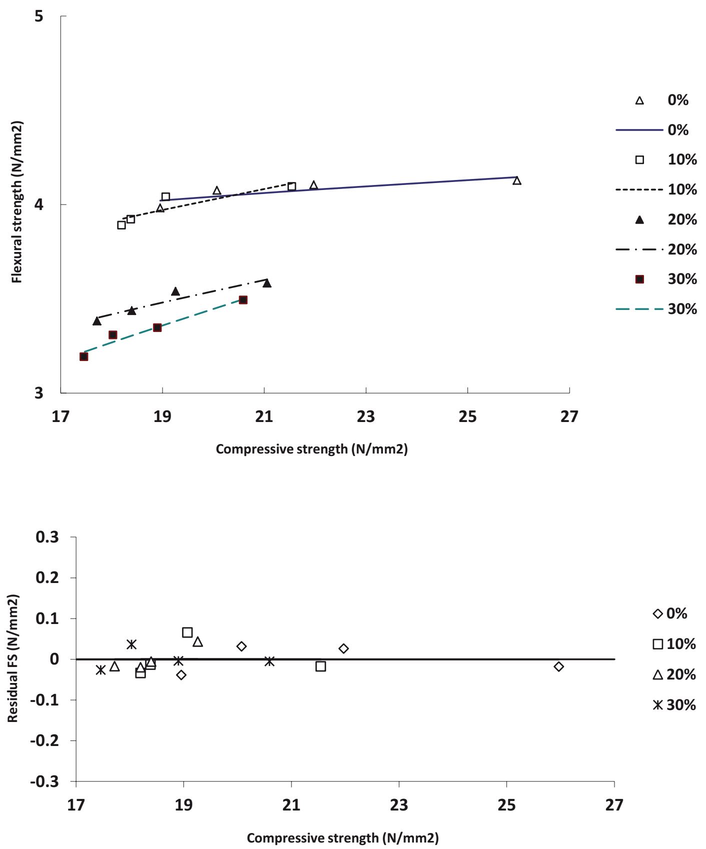


Figure 4.21: Variation of flexural strength with compressive strength at the various (0%, 10%, 20% and 30%) laterite content

Lines indicate the model fit; Residual = observed – predicted; FS: flexural strength.

Table 4.24: Summary of Modelling Results

Lat. Cont. (%)	Model parameters		Goodness of fit test				
	N	k	R^2 statistic		Standard deviation		
			R^2	p-value	R_m (%)	σ	p-value
Tensile splitting strength (N/ mm^2)							
0	0.7422	0.2197	0.9558	0.0224	0.0005	0.0496	0.986
10	1.8071	0.0097	0.9895	0.0053	0.0001	0.0309	0.995
20	2.2102	0.0030	0.7456	0.1365	0.0056	0.1946	0.958
30	0.9756	0.0897	0.8414	0.0827	0.0005	0.0473	0.985
Flexural strength (N/ mm^2)							
0	0.0967	3.0254	0.7139	0.1551	0.0001	0.0341	0.996
10	0.2782	1.7505	0.7894	0.1115	0.0002	0.0445	0.994
20	0.3366	1.2917	0.8970	0.0529	0.0001	0.2996	0.995
30	0.5053	0.7586	0.9553	0.0226	0.0001	0.0263	0.994

5 CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

From the results of the various tests performed, the following conclusions can be drawn:

1. The sand used is of medium grading (F.M = 2.6); the laterite sample is of coarse grading (F.M = 3.2). On the basis of C_u and C_c, they are both well graded while the granite used is uniformly graded. All the aggregates fall within the classification for very suitable for making good concrete.
2. The laterite sample is a true laterite having a silica – sesquioxide ratio of 0.97, a liquid limit of 31.21; plastic limit of 26.11 and plastic index of 5.1.
3. The volcanic ash sample obtained from Dutshin Dushowa (a hill) in Kerang Local Government of Jos, Plateau State, Nigeria is a suitable material for use as a pozzolan, since it satisfied the requirements for such a material as spelt out in ASTM C618-2008 by having a combined SiO_2, Al_2O_3 and Fe_2O_3 of 70.99% which is more than the required 70%.
4. The VA-blended cement satisfies the NIS 439:2000; BS EN 196 – 6:2005 and ASTM C618:2008 requirements (for class N pozzolanas) even up to 30% substitution on the basis of fineness (residue on 53 µm sieve), soundness, consistency, and ASTM C150 :2008 for setting times.
5. The VA-blended cements have higher setting times than the control; hence, they are most applicable where low rate of heat development is required such as in mass concreting. This shows that VA-blended cement is good as low heat cement.
6. The value of slump decreases as the VA content increases and also as the laterite content increases. This means that concrete becomes less workable (stiff) as the volcanic ash (VA) content increases. Hence there is a higher demand for water with

increasing VA content. The workability period of the laterized concrete is also extended as the VA increases both initial and final setting times. This is of particular importance in ready mixed concrete as there is extra time to effect delivery on site.

7 The compressive strength of VA-blended cement laterized concrete is lower than that of plain concrete (control – 0%LAT/0%VA) at early curing age but improves significantly at later ages and has higher rate of strength gain than the later. The optimum level of replacement from structural load view point is 20%LAT/20%VA at which about 95% of the designed strength (28-day) is obtained at the 120th day.

8 The tensile splitting and flexural strength properties of the VA blended cement laterized concrete are also lower at early curing age but improves significantly at later ages and has higher rate of development than the control.

9 The relationship between the compressive strength and tensile (be it splitting or flexural) is similar to that of plain concrete, with variations in the values of constants k and n as outlined. Thus the generalised equation of the relationship is applicable to VA-blended cement laterized concrete with the constants k and n developed in this study (Table 4.23) applied.

5.2 RECOMMENDATIONS

Based on the findings from this study, the following recommendations are made:

1 Efforts should be made at exploring the utilisation of the abundant deposit of volcanic ash spread over wide areas around Jos plateau with a view to its adoption as a supplementary cementitious material. This will go a long way in averting the impending danger to which a volcanic explosion posses to a nation and the world at large. The excavation and utilisation have to be controlled to avoid environmental degradation trends.

2 The VA-blended cement laterized concrete can be adopted for mass concrete construction, low heat construction and in situation where early strength is not required up to 20% laterite substitution of sand and 20% volcanic ash substitution of cement.

3 Further research should be conducted on effect of calcinations on the chemical composition and physical properties of the volcanic ash sample; the durability properties of the VA-blended cement laterized concrete such as resistance to chemical attack, effect of exposure to sulphates, thermal conductivity and effect of elevated temperatures on the compressive strength.

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APPENDIX A

Result of Physical Properties

Table A1: Sieve Analysis of Laterite Used

Mass of Sample Used = 500g

Sieve size	Weight of Soil retained (g)	Percentage of Soil retained (%)	Cumulative Percentage of Soil retained (%)	Percentage Passing (%)
4.75 mm	24.99	5.0	5.0	95.0
2.36 mm	94.96	19.0	24.0	76.0
1.18 mm	105.96	21.2	45.2	54.8
600µm	112.95	22.6	67.8	31.2
300µm	80.96	16.2	84.0	16.0
150µm	52.98	10.6	94.6	5.4
75µm	25.99	5.2	99.8	0.2
Pan	1.00	0.2	100.0	0.0
Total	499.8	100.0		

Mass lost during Sieve Analysis = (500-499.8)/500 x 100% = 0.04 ≤ 2 (Okay)

Fineness Modulus = (95.0+76.0+54.8+31.2+16+5.4)/100 = 3.2 (Coarse Sand)

Table A2: Sieve Analysis of Sand Used

Mass of Sample Used = 500g

Sieve size	Weight of Soil retained (g)	Percentage of Soil retained (%)	Cumulative Percentage of Soil retained (%)	Percentage Passing (%)
4.75 mm	2.00	0.4	0.4	99.6
2.36 mm	69.96	14.0	14.4	85.6
1.18 mm	90.45	18.1	32.5	67.5
600µm	114.93	23.0	55.5	45.5
300µm	85.45	17.1	72.6	27.4
150µm	69.45	13.9	86.5	13.5
75µm	65.96	13.2	99.7	0.3
Pan	1.50	0.3	100	0.0
Total	499.7	100.0	261.9	

Mass lost During Sieve Analysis = 0.06 ≤ 2 (Okay)

Fineness Modulus = (99.6+85.6+67.5+45.5+27.4+13.5)/100 = 2.6 (Medium Sand)

Table A3: Sieve Analysis of Sand Used

Mass of Sample Used = 1000g

Sieve size	Weight of Soil retained (g)	Percentage of Soil retained (%)	Cumulative Percentage of Soil retained (%)	Percentage Passing (%)
20.0 mm	37.98	3.8	3.8	96.2
14.0 mm	654.74	65.5	69.3	30.7
10.0 mm	268.89	26.9	96.2	3.8
5.00 mm	36.99	3.7	99.9	0.1
2.36 mm	1.00	0.1	100	0.0
Pan	0.00	0.0	100.0	0.0
Total	999.6	100.0		

Mass lost during Sieve Analysis = 0.04 ≤ 2 (Okay)

Table A4: Specific Gravity of Volcanic Ash and Cement

Material	Volcanic Ash		Cement	
Sample Number	**1**	**2**	**1**	**2**
Volume of bottle = V (cm^3)	100.00	100.00	100cm^3	100cm^3
Weight of bottle = w_1(g)	28.31	28.31	28.31	28.31
Weight of bottle + sample =w_2 (g)	40.41	40.38	42.01	41.06
Weight of bottle + sample + water = w_3 (g)	90.12	90.08	92.15	91.2
Weight of bottle full of water = w_4 (g)	82.58	82.58	82.58	82.58
Specific gravity = $(w_2-w_1)/(w_4-w_1)-(w_3-w_2)$	2.65	2.64	3.32	3.09
Average Specific Gravity	2.65		3.21	

Table A5: Specific Gravity of Laterite and Sand

Material	Laterite		Sand	
Sample Number	**1**	**2**	**1**	**2**
Volume of bottle = V (cm^3)	100.00	100.00	100.00	100.00
Weight of bottle = w_1(g)	28.31	28.31	28.31	28.31
Weight of bottle + sample =w_2 (g)	47.13	46.59	40.91	44.42
Weight of bottle + sample + water = w_3 (g)	94.42	93.74	90.21	92.56
Weight of bottle full of water = w_4 (g)	82.58	82.58	82.58	82.58
Specific gravity = $(w_2-w_1)/(w_4-w_1)-(w_3-w_2)$	2.69	2.67	2.54	2.63
Average Specific Gravity	2.68		2.59	

Table A6: Specific Gravity of Granite

Sample Number	1	2
Weight of empty pycnometer = w1 (g)	485.9	485.9
Weight of pycnometer + aggregate = w_2 (g)	829.9	829.9
Weight of pycnometer + aggregate + water = w_3(g)	1684.5	1684.2
Weight of pycnometer + water = w_4(g)	1469.5	1469.5
Weight of dry aggregate (w_5)=w_2-w_1 (g)	344.0	344.0
Submerged weight (w_6)=w_3 –w_4 (g)	215	214.7
Specific gravity, S.G = w_5/ (w_5-w_6)	2.67	2.66
Average Specific Gravity	2.66	

Table A.7: Bulk Density of Laterite

Type	Compacted			Uncompacted		
Sample Number	1	2	3	1	2	3
Weight of cylinder mould (W_1 (kg))	8.8	8.8	8.8	8.8	8.8	8.8
Volume of cylinder mould (V (m^3))	0.0027	0.0027	0.0027	0.0027	0.0027	0.0027
Weight of sample + cylinder mould (W_2 (kg))	13.88	13.86	13.82	12.24	12.15	12.05
Weight of Sample (W_2 –W_1 (kg))	5.08	5.06	5.02	3.44	3.35	3.25
Bulk Density (kg/m^3)	1916.98	1909.43	1894.34	1298.11	1264.15	1226.42
Average Bulk Density (kg/m^3)	1906.92			1262.89		

Table A.8: Bulk Density of Granite

Type	Compacted			Uncompacted		
Test Number	1	2	3	1	2	3
Weight of empty mould (g)	1064.0	1064.0	1064.0	1064.0	1064.0	1064.0
Volume of mould (m^3)	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017
weight of mould + sample (g)	3722.5	3678.7	3754.6	3546.4	3430.3	3532.8
Weight of sample (g)	2658.5	2614.7	2690.6	2482.4	2366.3	2468.8
Bulk density (Kg/m^3)	1582.4	1556.4	1601.5	1477.6	1408.5	1469.5
Average Bulk Density (Kg/m^3)	1580.12			1451.88		

Table A.9: Natural Moisture Content of Laterite

Material	Laterite		Sand	
Can Number	Ax	Ay	A_{14}	C_6
Weight of Can	23.4	23.1	25	25
Weight of Can + Sample (g)	76.4	76.5	42.0	42
Weight of Can + Dry Sample (g)	69.8	69.9	42	41
Weight of Wet Sample (g)	53.0	53.4	17.4	17.0
Weight of Dry Sample (g)	46.4	46.8	17	16
Weight of Water (g)	6.59	6.6	0.3	0.9
Moisture Content (%)	14.21	14.09	1.75	5.59
Average Moisture Content (%)	14.15		3.67	

APPENDIX B

Result of Tests on VA-Blended Cement and Laterized Concrete

Table B1: Mix Design Chart

ITEM	REFERENCE OR CALCULATION	VALUES
1.1 Characteristic strength	Specified	25 N/ mm^2 at 28 days Proportion defective 5 percent
1.2 Standard deviation	Fig	3 N/ mm^2 or no data 4 N/ mm^2
1.3 Margin	C1	(k=1.64)1.64×3= 4.92N/ mm^2
1.4 Target mean strength	C2	25 + 4.92 = 29.92 N/ mm^2
1.5 Cement type	Specified	OPC /VA
1.5 Aggregate type: Coarse		Crushed
1.7 Aggregate type: Fine		Uncrushed
1.8 Free-water ratio	Table2, fig 4	0.79
1.9 Maximum free water/ cement ratio	Specified	0.65 use the lower value
2.0 Slump	Specified	slump 10 - 30 mm
2.1 Maximum aggregate size	Specified	20 mm
2.2 Free-water content	Table2	190kg/m^3
2.3 Cement content	C3	190 ÷ 0.65 = 292.31kg/m^3
2.4 Maximum cement content	Specified	kg/m^3
2.5 Minimum cement content	Specified	275kg/m^3 use if ≥ 2.3 and calculate item 2.6
2.6 Modified free water cement ratio		
2.7 Relative density of aggregate (SSD)		2.66
2.8 Concrete density	Fig 5	2475 kg/m^3
2.9 Total aggregate density	C4	2475 - 190 - 292.31 = 1992.69kg/m^3
3.0 Grading of fine aggregate	BS EN 12620:2000	Medium Fine (FM= 2.6, C_u=7.3 and C_c=1.05), 45.5% passing 600µm
3.1 Proportion of fine aggregate passing	Fig 6	26.43percent
3.2 Fine aggregate content	C5	1995 × 0.264 = 527.28 kg/m^3
3.3 Coarse aggregate content		1995 – 530 = 1465 kg/m^3

Quantities	Cement(kg)	Water(kg or l)	Fine aggregate (kg)	Coarse aggregate(kg)
Per m^3(to nearest 5kg)	290.00	190.00	530.00	1465.00
Per trial mix of 0.0189	5.50	4.00	10.02	28.00

Table B2: Mix Proportions for 18 Nos. 100 mm x 100 mm x 100 mm Cubes

Laterite (%)	Concrete constituent	Volcanic ash (%)				Total
		0	10	20	30	
0	Water (kg)	4.00	4.01	4.17	4.19	16.37
	Volcanic ash (kg)	-	0.55	1.10	1.65	3.30
	Cement (kg)	5.50	4.95	4.40	3.85	18.70
	Sand (kg)	10.02	10.02	10.02	10.02	40.08
	Granite (kg)	28.00	28.00	28.00	28.00	112.00
10	Water (kg)	4.00	4.01	4.17	4.19	16.37
	Volcanic ash (kg)	-	0.55	1.10	1.65	3.30
	Cement (kg)	5.50	4.95	4.40	3.85	18.70
	Laterite (kg)	1.00	1.00	1.00	1.00	4.00
	Sand (kg)	9.02	9.02	9.02	9.02	36.08
	Granite (kg)	28.00	28.00	28.00	28.00	112.00
20	Water (kg)	4.00	4.01	4.17	4.19	16.37
	Volcanic ash (kg)	-	0.55	1.10	1.65	3.30
	Cement (kg)	5.50	4.95	4.40	3.85	18.70
	Laterite (kg)	2.00	2.00	2.00	2.00	8.00
	Sand (kg)	8.02	8.02	8.02	8.02	32.08
	Granite (kg)	28.00	28.00	28.00	28.00	112.00
30	Water (kg)	4.00	4.01	4.17	4.19	16.37
	Volcanic ash (kg)	-	0.55	1.10	1.65	3.30
	Cement (kg)	5.50	4.95	4.40	3.85	18.70
	Laterite (kg)	3.00	3.00	3.00	3.00	12.00
	Sand (kg)	7.02	7.02	7.02	7.02	28.08
	Granite (kg)	28.00	28.00	28.00	28.00	112.00

Table B3: Compressive Strength of VA-blended Cement Laterized Concrete at 3days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Area of Cube (m^2)	Vol. of Cube (m^3)	Wt. of Cube (Kg)	Crushing Force (KN)	Density (Kg/m^3)	Av. Density (Kg/m^3)	Comp. Strength (N/mm^2)	Av. Comp. Strength (N/mm^2)
A1	0	0	0.01	0.001	2.400	110.0	2400		11.0	
A2			0.01	0.001	2.450	120.2	2450	2428	12.0	11.02
A3			0.01	0.001	2.435	100.4	2435		10.0	
B1	0	10	0.01	0.001	2.425	82.6	2425		8.3	
B2			0.01	0.001	2.475	85.6	2475	2450	8.6	8.61
B3			0.01	0.001	2.450	90.2	2450		9.0	
C1	0	20	0.01	0.001	2.425	72.5	2425		7.3	
C2			0.01	0.001	2.435	70.3	2435	2430	7.0	7.38
C3			0.01	0.001	2.430	78.5	2430		7.9	
D1	0	30	0.01	0.001	2.450	65.8	2450		6.6	
D2			0.01	0.001	2.400	62.2	2400	2428	6.2	6.45
D3			0.01	0.001	2.435	65.4	2435		6.5	
E1	10	0	0.01	0.001	2.440	92.5	2440		9.3	
E2			0.01	0.001	2.450	85.0	2450	2438	8.5	8.67
E3			0.01	0.001	2.425	82.5	2425		8.3	
F1	10	10	0.01	0.001	2.385	860	2385		8.6	
F2			0.01	0.001	2.390	78.3	2390	2383	7.8	8.03
F3			0.01	0.001	2.375	76.7	2375		7.7	
G1	10	20	0.01	0.001	2.375	66.5	2375		6.7	
G2			0.01	0.001	2.380	69.7	2380	2380	7.0	7.04
G3			0.01	0.001	2.385	75.0	2385		7.5	
H1	10	30	0.01	0.001	2.450	51.3	2450		5.1	
H2			0.01	0.001	2.390	50.1	2390	2420	5.0	5.11
H3			0.01	0.001	2.420	52.0	2420		5.2	
I1	20	0	0.01	0.001	2.425	87.4	2425		8.7	
I2			0.01	0.001	2.480	91.1	2480	2457	9.1	8.89
I3			0.01	0.001	2.465	88.2	2465		8.8	
J1	20	10	0.01	0.001	2.440	78.8	2440		7.9	
J2			0.01	0.001	2.450	90.4	2450	2433	9.0	8.52
J3			0.01	0.001	2.410	86.5	2410		8.7	
L1	20	20	0.01	0.001	2.480	66.7	2480		6.7	
L2			0.01	0.001	2.450	80.3	2450	2453	8.0	7.24
L3			0.01	0.001	2.430	70.2	2430		7.0	
M1	20	30	0.01	0.001	2.380	54.9	2380		5.5	
M2			0.01	0.001	2.420	48.8	2420	2400	4.9	5.19
M3			0.01	0.001	2.400	52.0	2400		5.2	
N1	30	0	0.01	0.001	2.460	88.2	2460		8.8	
N2			0.01	0.001	2.400	91.0	2400	2428	9.1	8.46
N3			0.01	0.001	2.425	74.5	2425		7.5	
O1	30	10	0.01	0.001	2.380	70.2	2380		7.0	
O2			0.01	0.001	2.410	72.0	2410	2403	7.2	7.13
O3			0.01	0.001	2.420	71.6	2420		7.2	
P1	30	20	0.01	0.001	2.440	63.2	2440		6.3	
P2			0.01	0.001	2.420	69.9	2420	2430	7.0	6.64
P3			0.01	0.001	2.430	66.2	2430		6.6	
Q1	30	30	0.01	0.001	2.420	49.6	2420		5.0	
Q2			0.01	0.001	2.400	52.4	2400	2410	5.2	5.02
Q3			0.01	0.001	2.410	48.6	2410		4.9	

Table B4: Compressive Strength of VA-blended Cement Laterized Concrete at 7days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Area of Cube (m^2)	Vol. of Cube (m^3)	Wt. of Cube (Kg)	Crushing Force (KN)	Density (Kg/m^3)	Av. Density (Kg/m^3)	Comp. Strength (N/mm^2)	Av. Comp. Strength (N/mm^2)
A1	0	0	0.01	0.001	2.390	186.3	2390		18.6	
A2			0.01	0.001	2.440	180.2	2440	2422	18.0	18.50
A3			0.01	0.001	2.435	188.4	2435		18.8	
B1	0	10	0.01	0.001	2.420	124.6	2420		12.5	
B2			0.01	0.001	2.450	128.2	2450	2437	12.8	12.39
B3			0.01	0.001	2.440	118.9	2440		11.9	
C1	0	20	0.01	0.001	2.425	115.4	2425		11.5	
C2			0.01	0.001	2.430	109.9	2430	2427	11.0	10.79
C3			0.01	0.001	2.425	98.5	2425		9.9	
D1	0	30	0.01	0.001	2.430	100.2	2430		10.0	
D2			0.01	0.001	2.400	88.8	2400	2422	8.9	9.82
D3			0.01	0.001	2.435	105.7	2435		10.6	
E1	10	0	0.01	0.001	2.440	118.6	2440		11.9	
E2			0.01	0.001	2.420	106.4	2420	2440	10.6	11.55
E3			0.01	0.001	2.460	121.4	2460		12.1	
F1	10	10	0.01	0.001	2.385	100.6	2385		10.1	
F2			0.01	0.001	2.390	94.8	2390	2383	9.5	9.77
F3			0.01	0.001	2.375	97.7	2375		9.8	
G1	10	20	0.01	0.001	2.375	83.5	2375		8.4	
G2			0.01	0.001	2.380	86.6	2380	2380	8.7	8.62
G3			0.01	0.001	2.385	88.4	2385		8.8	
H1	10	30	0.01	0.001	2.450	68.4	2450		6.8	
H2			0.01	0.001	2.380	64.2	2380	2417	6.4	6.80
H3			0.01	0.001	2.420	71.4	2420		7.1	
I1	20	0	0.01	0.001	2.415	100.0	2415		10.0	
I2			0.01	0.001	2.480	98.0	2480	2453	9.8	9.97
I3			0.01	0.001	2.465	101.2	2465		10.1	
J1	20	10	0.01	0.001	2.440	90.8	2440		9.1	
J2			0.01	0.001	2.450	99.8	2450	2433	10.0	9.49
J3			0.01	0.001	2.410	94.2	2410		9.4	
L1	20	20	0.01	0.001	2.450	74.2	2450		7.4	
L2			0.01	0.001	2.440	78.9	2440	2437	7.9	7.65
L3			0.01	0.001	2.420	76.4	2420		7.6	
M1	20	30	0.01	0.001	2.410	64.9	2410		6.5	
M2			0.01	0.001	2.400	74.8	2400	2397	7.5	6.98
M3			0.01	0.001	2.380	69.8	2380		7.0	
N1	30	0	0.01	0.001	2.460	94.8	2460		9.5	
N2			0.01	0.001	2.400	101	2400	2428	10.1	9.74
N3			0.01	0.001	2.425	96.5	2425		9.7	
O1	30	10	0.01	0.001	2.380	85.4	2380		8.5	
O2			0.01	0.001	2.410	84.8	2410	2403	8.5	8.24
O3			0.01	0.001	2.420	76.9	2420		7.7	
P1	30	20	0.01	0.001	2.440	74.2	2440		7.4	
P2			0.01	0.001	2.420	82.1	2420	2430	8.2	7.63
P3			0.01	0.001	2.430	72.6	2430		7.3	
Q1	30	30	0.01	0.001	2.400	64.3	2400		6.4	
Q2			0.01	0.001	2.410	61.7	2410	2400	6.2	6.28
Q3			0.01	0.001	2.390	62.4	2390		6.2	

Table B5: Compressive Strength of VA-blended Cement Laterized Concrete at 28days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Area of Cube (m^2)	Vol. of Cube (m^3)	Wt. of Cube (Kg)	Crushing Force (KN)	Density (Kg/m^3)	Av. Density (Kg/m^3)	Comp. Strength (N/mm^2)	Av. Comp. Strength (N/mm^2)
A1	0	0	0.01	0.001	2.360	248.5	2360		24.9	
A2			0.01	0.001	2.440	250.2	2440	2433	25.0	25.97
A3			0.01	0.001	2.500	280.4	2500		28.0	
B1	0	10	0.01	0.001	2.420	224.2	2420		22.4	
B2			0.01	0.001	2.430	219.5	2430	2443	22.0	21.97
B3			0.01	0.001	2.480	215.5	2480		21.6	
C1	0	20	0.01	0.001	2.400	209.2	2400		20.9	
C2			0.01	0.001	2.380	188.0	2380	2403	18.8	20.07
C3			0.01	0.001	2.430	205.0	2430		20.5	
D1	0	30	0.01	0.001	2.440	194.2	2440		19.4	
D2			0.01	0.001	2.440	185.8	2440	2438	18.6	18.96
D3			0.01	0.001	2.435	188.7	2435		18.9	
E1	10	0	0.01	0.001	2.440	222.6	2440		22.3	
E2			0.01	0.001	2.420	213.4	2420	2440	21.3	21.55
E3			0.01	0.001	2.460	210.4	2460		21.0	
F1	10	10	0.01	0.001	2.380	187.6	2380		18.8	
F2			0.01	0.001	2.390	189.8	2390	2423	19.0	19.07
F3			0.01	0.001	2.500	194.7	2500		19.5	
G1	10	20	0.01	0.001	2.375	184.5	2375		18.5	
G2			0.01	0.001	2.380	186.6	2380	2380	18.7	18.38
G3			0.01	0.001	2.385	180.4	2385		18.0	
H1	10	30	0.01	0.001	2.430	176.4	2430		17.6	
H2			0.01	0.001	2.380	182.2	2380	2410	18.2	18.20
H3			0.01	0.001	2.420	187.4	2420		18.7	
I1	20	0	0.01	0.001	2.415	221.4	2415		22.1	
I2			0.01	0.001	2.480	220.2	2480	2452	22.0	21.06
I3			0.01	0.001	2.460	190.2	2460		19.0	
J1	20	10	0.01	0.001	2.440	196.8	2440		19.7	
J2			0.01	0.001	2.350	176.8	2350	2400	17.7	19.26
J3			0.01	0.001	2.410	204.2	2410		20.4	
L1	20	20	0.01	0.001	2.430	189.6	2430		19.0	
L2			0.01	0.001	2.440	195.9	2440	2430	19.6	18.40
L3			0.01	0.001	2.420	166.4	2420		16.6	
M1	20	30	0.01	0.001	2.410	185.9	2410		18.6	
M2			0.01	0.001	2.400	175.8	2400	2410	17.6	17.72
M3			0.01	0.001	2.420	169.8	2420		17.0	
N1	30	0	0.01	0.001	2.460	198.8	2460		19.9	
N2			0.01	0.001	2.400	212.5	2400	2427	21.3	20.59
N3			0.01	0.001	2.420	206.5	2420		20.7	
O1	30	10	0.01	0.001	2.420	185.4	2420		18.5	
O2			0.01	0.001	2.410	184.8	2410	2417	18.5	18.90
O3			0.01	0.001	2.420	196.9	2420		19.7	
P1	30	20	0.01	0.001	2.440	174.2	2440		17.4	
P2			0.01	0.001	2.420	182.1	2420	2430	18.2	18.03
P3			0.01	0.001	2.430	184.6	2430		18.5	
Q1	30	30	0.01	0.001	2.400	185.3	2400		18.5	
Q2			0.01	0.001	2.410	165.7	2410	2400	16.6	17.46
Q3			0.01	0.001	2.390	172.8	2390		17.3	

Table B6: Compressive Strength of VA-blended Cement Laterized Concrete at 56days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Area of Cube (m2)	Vol. of Cube (m^3)	Wt. of Cube (Kg)	Crushing Force (KN)	Density (Kg/m^3)	Av. Density (Kg/m^3)	Comp. Strength (N/mm^2)	Av. Comp. Strength (N/mm^2)
A1	0	0	0.01	0.001	2.360	278.5	2360		27.9	
A2			0.01	0.001	2.440	264.2	2440	2433	26.4	27.44
A3			0.01	0.001	2.500	280.4	2500		28.0	
B1	0	10	0.01	0.001	2.420	224.2	2420		22.4	
B2			0.01	0.001	2.430	219.5	2430	2443	22.0	21.97
B3			0.01	0.001	2.480	215.5	2480		21.6	
C1	0	20	0.01	0.001	2.400	219.2	2400		21.9	
C2			0.01	0.001	2.380	224.0	2380	2403	22.4	21.74
C3			0.01	0.001	2.430	209.0	2430		20.9	
D1	0	30	0.01	0.001	2.440	194.2	2440		19.4	
D2			0.01	0.001	2.440	185.8	2440	2438	18.6	18.96
D3			0.01	0.001	2.435	188.7	2435		18.9	
E1	10	0	0.01	0.001	2.440	242.6	2440		24.3	
E2			0.01	0.001	2.420	213.4	2420	2440	21.3	22.21
E3			0.01	0.001	2.460	210.4	2460		21.0	
F1	10	10	0.01	0.001	2.380	210.6	2380		21.1	
F2			0.01	0.001	2.390	224.8	2390	2423	22.5	21.67
F3			0.01	0.001	2.500	214.7	2500		21.5	
G1	10	20	0.01	0.001	2.375	204.5	2375		20.5	
G2			0.01	0.001	2.380	216.6	2380	2380	21.7	20.98
G3			0.01	0.001	2.385	208.4	2385		20.8	
H1	10	30	0.01	0.001	2.430	198.4	2430		19.8	
H2			0.01	0.001	2.380	182.2	2380	2410	18.2	19.63
H3			0.01	0.001	2.420	208.4	2420		20.8	
I1	20	0	0.01	0.001	2.415	241.4	2415		24.1	
I2			0.01	0.001	2.480	262.2	2480	2452	26.2	24.19
I3			0.01	0.001	2.460	222.2	2460		22.2	
J1	20	10	0.01	0.001	2.440	229.8	2440		23.0	
J2			0.01	0.001	2.350	218.8	2350	2400	21.9	21.76
J3			0.01	0.001	2.410	204.2	2410		20.4	
L1	20	20	0.01	0.001	2.430	209.6	2430		21.0	
L2			0.01	0.001	2.440	219.9	2440	2430	22.0	21.53
L3			0.01	0.001	2.420	216.4	2420		21.6	
M1	20	30	0.01	0.001	2.410	205.9	2410		20.6	
M2			0.01	0.001	2.400	199.8	2400	2410	20.0	20.75
M3			0.01	0.001	2.420	216.8	2420		21.7	
N1	30	0	0.01	0.001	2.460	229.8	2460		23.0	
N2			0.01	0.001	2.400	218.5	2400	2427	21.9	22.93
N3			0.01	0.001	2.420	239.5	2420		24.0	
O1	30	10	0.01	0.001	2.420	218.4	2420		21.8	
O2			0.01	0.001	2.410	220.8	2410	2417	22.1	21.97
O3			0.01	0.001	2.420	219.9	2420		22.0	
P1	30	20	0.01	0.001	2.440	214.2	2440		21.4	
P2			0.01	0.001	2.420	203.1	2420	2430	20.3	20.90
P3			0.01	0.001	2.430	209.6	2430		21.0	
Q1	30	30	0.01	0.001	2.400	205.3	2400		20.5	
Q2			0.01	0.001	2.410	200.7	2410	2400	20.1	20.46
Q3			0.01	0.001	2.390	207.8	2390		20.8	

Table B7: Compressive Strength of VA-blended Cement Laterized Concrete at 90days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Area of Cube (m^2)	Vol. of Cube (m^3)	Wt. of Cube (Kg)	Crushing Force (KN)	Density (Kg/m^3)	Av. Density (Kg/m^3)	Comp. Strength (N/mm^2)	Av. Comp. Strength (N/mm^2)
A1	0	0	0.01	0.001	2.360	285.5	2360		28.6	
A2			0.01	0.001	2.440	262.2	2440	2433	26.2	28.47
A3			0.01	0.001	2.500	306.4	2500		30.6	
B1	0	10	0.01	0.001	2.420	242.2	2420		24.2	
B2			0.01	0.001	2.430	259.5	2430	2443	26.0	25.14
B3			0.01	0.001	2.480	252.5	2480		25.3	
C1	0	20	0.01	0.001	2.400	219.2	2400		21.9	
C2			0.01	0.001	2.380	218.4	2380	2403	21.8	22.19
C3			0.01	0.001	2.430	228.2	2430		22.8	
D1	0	30	0.01	0.001	2.440	205.4	2440		20.5	
D2			0.01	0.001	2.440	207.5	2440	2438	20.8	20.97
D3			0.01	0.001	2.435	216.2	2435		21.6	
E1	10	0	0.01	0.001	2.440	276.6	2440		27.7	
E2			0.01	0.001	2.420	259.4	2420	2440	25.9	26.78
E3			0.01	0.001	2.460	267.4	2460		26.7	
F1	10	10	0.01	0.001	2.380	234.0	2380		23.4	
F2			0.01	0.001	2.390	246.8	2390	2423	24.7	24.05
F3			0.01	0.001	2.500	240.7	2500		24.1	
G1	10	20	0.01	0.001	2.375	234.5	2375		23.5	
G2			0.01	0.001	2.380	226.6	2380	2380	22.7	23.52
G3			0.01	0.001	2.385	244.4	2385		24.4	
H1	10	30	0.01	0.001	2.430	206.4	2430		20.6	
H2			0.01	0.001	2.380	222.2	2380	2410	22.2	20.97
H3			0.01	0.001	2.420	200.4	2420		20.0	
I1	20	0	0.01	0.001	2.415	272.2	2415		27.2	
I2			0.01	0.001	2.480	260.2	2480	2452	26.0	26.69
I3			0.01	0.001	2.460	268.2	2460		26.8	
J1	20	10	0.01	0.001	2.440	228.8	2440		22.9	
J2			0.01	0.001	2.350	236.8	2350	2400	23.7	24.16
J3			0.01	0.001	2.410	259.2	2410		25.9	
L1	20	20	0.01	0.001	2.430	249.6	2430		25.0	
L2			0.01	0.001	2.440	240.9	2440	2430	24.1	24.43
L3			0.01	0.001	2.420	242.4	2420		24.2	
M1	20	30	0.01	0.001	2.410	226.9	2410		22.7	
M2			0.01	0.001	2.400	215.8	2400	2410	21.6	21.75
M3			0.01	0.001	2.420	209.8	2420		21.0	
N1	30	0	0.01	0.001	2.460	248.8	2460		24.9	
N2			0.01	0.001	2.400	252.5	2400	2427	25.3	25.26
N3			0.01	0.001	2.420	256.5	2420		25.7	
O1	30	10	0.01	0.001	2.420	238.4	2420		23.8	
O2			0.01	0.001	2.410	244.8	2410	2417	24.5	25.00
O3			0.01	0.001	2.420	266.9	2420		26.7	
P1	30	20	0.01	0.001	2.470	234.2	2470		23.4	
P2			0.01	0.001	2.420	230.1	2420	2440	23.0	23.66
P3			0.01	0.001	2.430	245.6	2430		24.6	
Q1	30	30	0.01	0.001	2.400	225.3	2400		22.5	
Q2			0.01	0.001	2.410	235.7	2410	2400	23.6	23.46
Q3			0.01	0.001	2.390	242.8	2390		24.3	

Table B8: Compressive Strength of VA-blended Cement Laterized Concrete at 120days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Area of Cube (m^2)	Vol. of Cube (m^3)	Wt. of Cube (Kg)	Crushing Force (KN)	Density (Kg/m^3)	Av. Density (Kg/m^3)	Comp. Strength (N/mm^2)	Av. Comp. Strength (N/mm^2)
A1	0	0	0.01	0.001	2.360	298.5	2360		29.9	
A2			0.01	0.001	2.440	295.2	2440	2433	29.5	29.14
A3			0.01	0.001	2.500	280.4	2500		28.0	
B1	0	10	0.01	0.001	2.420	264.2	2420		26.4	
B2			0.01	0.001	2.430	259.5	2430	2443	26.0	26.34
B3			0.01	0.001	2.480	266.5	2480		26.7	
C1	0	20	0.01	0.001	2.400	230.2	2400		23.0	
C2			0.01	0.001	2.380	222.0	2380	2403	22.2	22.57
C3			0.01	0.001	2.430	225.0	2430		22.5	
D1	0	30	0.01	0.001	2.440	219.2	2440		21.9	
D2			0.01	0.001	2.440	228.8	2440	2438	22.9	22.02
D3			0.01	0.001	2.435	212.7	2435		21.3	
E1	10	0	0.01	0.001	2.380	280.0	2380		28.0	
E2			0.01	0.001	2.400	264.4	2400	2390	26.4	27.33
E3			0.01	0.001	2.390	275.4	2390		27.5	
F1	10	10	0.01	0.001	2.350	252.2	2350		25.2	
F2			0.01	0.001	2.390	239.8	2390	2380	24.0	25.09
F3			0.01	0.001	2.400	260.7	2400		26.1	
G1	10	20	0.01	0.001	2.400	234.5	2400		23.5	
G2			0.01	0.001	2.400	232.6	2400	2403	23.3	23.65
G3			0.01	0.001	2.410	242.4	2410		24.2	
H1	10	30	0.01	0.001	2.450	217.4	2450		21.7	
H2			0.01	0.001	2.400	205.2	2400	2423	20.5	21.07
H3			0.01	0.001	2.420	209.4	2420		20.9	
I1	20	0	0.01	0.001	2.400	271.4	2400		27.1	
I2			0.01	0.001	2.460	280.2	2460	2430	28.0	27.06
I3			0.01	0.001	2.430	260.2	2430		26.0	
J1	20	10	0.01	0.001	2.400	267.8	2400		26.8	
J2			0.01	0.001	2.380	236.8	2380	2397	23.7	24.96
J3			0.01	0.001	2.410	244.2	2410		24.4	
L1	20	20	0.01	0.001	2.430	252.6	2430		25.3	
L2			0.01	0.001	2.440	238.7	2440	2430	23.9	24.52
L3			0.01	0.001	2.420	244.4	2420		24.4	
M1	20	30	0.01	0.001	2.410	216.9	2410		21.7	
M2			0.01	0.001	2.400	240.8	2400	2410	24.1	22.95
M3			0.01	0.001	2.420	230.8	2420		23.1	
N1	30	0	0.01	0.001	2.480	268.8	2480		26.9	
N2			0.01	0.001	2.500	252.5	2500	2467	25.3	26.19
N3			0.01	0.001	2.420	264.5	2420		26.5	
O1	30	10	0.01	0.001	2.460	257.0	2460		25.7	
O2			0.01	0.001	2.340	262.8	2340	2400	26.3	25.89
O3			0.01	0.001	2.400	256.9	2400		25.7	
P1	30	20	0.01	0.001	2.440	248.2	2440		24.8	
P2			0.01	0.001	2.420	234.4	2420	2430	23.4	23.77
P3			0.01	0.001	2.430	230.6	2430		23.1	
Q1	30	30	0.01	0.001	2.400	241.3	2400		24.1	
Q2			0.01	0.001	2.410	230.7	2410	2400	23.1	23.69
Q3			0.01	0.001	2.390	238.8	2390		23.9	

Table B9: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 3days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Cylinder (m^3)	Weight of Cylinder (Kg)	Crushing Force(P) (KN)	2P (KN)	∏LD (m^2)	Splitting Tensile Str. (N/mm^2)	Av. Split. Ten. Str. (N/mm^2)
A1	0	0	0.0053	12.80	120.11	240.22	0.1414	1.70	
A2			0.0053	12.60	110.11	220.22	0.1414	1.56	1.64
A3			0.0053	12.70	117.98	235.95	0.1414	1.67	
B1	0	10	0.0053	12.60	104.38	208.76	0.1414	1.48	
B2			0.0053	12.50	101.23	202.47	0.1414	1.43	1.44
B3			0.0053	12.20	98.94	197.88	0.1414	1.40	
C1	0	20	0.0053	12.50	91.23	182.47	0.1414	1.29	
C2			0.0053	12.60	107.18	214.35	0.1414	1.52	1.41
C3			0.0053	12.20	100.26	200.51	0.1414	1.42	
D1	0	30	0.0053	12.40	98.69	197.39	0.1414	1.40	
D2			0.0053	12.30	99.11	198.22	0.1414	1.40	1.39
D3			0.0053	12.60	97.53	195.05	0.1414	1.38	
E1	10	0	0.0053	12.50	120.98	241.96	0.1414	1.71	
E2			0.0053	12.50	97.53	195.05	0.1414	1.38	1.61
E3			0.0053	12.60	122.69	245.39	0.1414	1.74	
F1	10	10	0.0053	11.85	88.60	177.19	0.1414	1.25	
F2			0.0053	12.50	88.60	177.19	0.1414	1.25	1.31
F3			0.0053	12.60	100.02	200.04	0.1414	1.41	
G1	10	20	0.0053	12.50	88.09	176.18	0.1414	1.25	
G2			0.0053	12.60	78.65	157.30	0.1414	1.11	1.23
G3			0.0053	12.80	94.38	188.76	0.1414	1.33	
H1	10	30	0.0053	12.40	88.09	176.18	0.1414	1.25	
H2			0.0053	12.60	81.80	163.59	0.1414	1.16	1.16
H3			0.0053	12.50	77.08	154.15	0.1414	1.09	
I1	20	0	0.0053	12.40	113.26	226.51	0.1414	1.60	
I2			0.0053	12.40	110.11	220.22	0.1414	1.56	1.57
I3			0.0053	12.50	110.53	221.05	0.1414	1.56	
J1	20	10	0.0053	12.60	125.84	251.68	0.1414	1.78	
J2			0.0053	12.50	88.09	176.18	0.1414	1.25	1.54
J3			0.0053	12.40	113.26	226.51	0.1414	1.60	
L1	20	20	0.0053	12.60	78.65	157.30	0.1414	1.11	
L2			0.0053	12.50	66.07	132.13	0.1414	0.93	1.14
L3			0.0053	12.40	97.53	195.05	0.1414	1.38	
M1	20	30	0.0053	12.30	72.36	144.72	0.1414	1.02	
M2			0.0053	12.50	84.94	169.88	0.1414	1.20	1.11
M3			0.0053	12.40	78.65	157.30	0.1414	1.11	
N1	30	0	0.0053	12.20	95.66	191.32	0.1414	1.35	
N2			0.0053	12.10	82.42	164.84	0.1414	1.17	1.21
N3			0.0053	12.40	77.73	155.46	0.1414	1.10	
O1	30	10	0.0053	12.50	83.33	166.66	0.1414	1.18	
O2			0.0053	12.30	80.46	160.92	0.1414	1.14	1.12
O3			0.0053	12.40	73.45	146.90	0.1414	1.04	
P1	30	20	0.0053	12.30	71.89	143.78	0.1414	1.02	
P2			0.0053	12.50	66.56	133.12	0.1414	0.94	0.96
P3			0.0053	12.40	65.65	131.30	0.1414	0.93	
Q1	30	30	0.0053	12.30	69.16	138.32	0.1414	0.98	
Q2			0.0053	12.50	67.73	135.46	0.1414	0.96	0.94
Q3			0.0053	12.40	62.66	125.32	0.1414	0.89	

Table B10: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 7days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Cylinder (m^3)	Weight of Cylinder (Kg)	Crushing Force(P) (KN)	2P (KN)	∏LD (m^2)	Splitting Ten. Str. (N/ mm2)	Av. Split. Ten. Str. (N/ mm^2)
A1	0	0	0.0053	13.00	127.05	254.10	0.1414	1.80	
A2			0.0053	12.70	127.05	254.10	0.1414	1.80	1.84
A3			0.0053	12.75	136.13	272.25	0.1414	1.93	
B1	0	10	0.0053	12.65	108.90	217.80	0.1414	1.54	
B2			0.0053	12.55	105.27	210.54	0.1414	1.49	1.47
B3			0.0053	12.45	98.01	196.02	0.1414	1.39	
C1	0	20	0.0053	12.65	105.27	210.54	0.1414	1.49	
C2			0.0053	12.60	158.28	316.56	0.1414	2.24	1.86
C3			0.0053	12.80	130.68	261.36	0.1414	1.85	
D1	0	30	0.0053	12.70	141.57	283.14	0.1414	2.00	
D2			0.0053	12.30	127.05	254.10	0.1414	1.80	1.80
D3			0.0053	12.70	112.53	225.06	0.1414	1.59	
E1	10	0	0.0053	12.70	139.59	279.18	0.1414	1.97	
E2			0.0053	12.65	112.53	225.06	0.1414	1.59	1.86
E3			0.0053	12.60	141.57	283.14	0.1414	2.00	
F1	10	10	0.0053	12.45	102.23	204.45	0.1414	1.45	
F2			0.0053	12.50	102.23	204.45	0.1414	1.45	1.51
F3			0.0053	12.65	115.41	230.82	0.1414	1.63	
G1	10	20	0.0053	12.80	101.64	203.28	0.1414	1.44	
G2			0.0053	12.60	95.75	191.50	0.1414	1.35	1.44
G3			0.0053	12.80	108.90	217.80	0.1414	1.54	
H1	10	30	0.0053	12.50	101.64	203.28	0.1414	1.44	
H2			0.0053	12.65	94.38	188.76	0.1414	1.33	1.34
H3			0.0053	12.70	88.94	177.87	0.1414	1.26	
I1	20	0	0.0053	12.60	130.68	261.36	0.1414	1.85	
I2			0.0053	12.40	127.05	254.10	0.1414	1.80	1.82
I3			0.0053	12.50	127.53	255.06	0.1414	1.80	
J1	20	10	0.0053	12.70	145.20	290.40	0.1414	2.05	
J2			0.0053	12.50	101.64	203.28	0.1414	1.44	1.78
J3			0.0053	12.45	130.68	261.36	0.1414	1.85	
L1	20	20	0.0053	12.65	90.75	181.50	0.1414	1.28	
L2			0.0053	12.50	76.23	152.46	0.1414	1.08	1.32
L3			0.0053	12.45	112.53	225.06	0.1414	1.59	
M1	20	30	0.0053	12.75	83.49	166.98	0.1414	1.18	
M2			0.0053	12.50	98.01	196.02	0.1414	1.39	1.28
M3			0.0053	12.45	90.75	181.50	0.1414	1.28	
N1	30	0	0.0053	12.40	87.30	174.60	0.1414	1.23	
N2			0.0053	12.50	95.10	190.20	0.1414	1.35	1.23
N3			0.0053	12.45	78.15	156.30	0.1414	1.11	
O1	30	10	0.0053	12.55	96.15	192.30	0.1414	1.36	
O2			0.0053	12.35	81.30	162.60	0.1414	1.15	1.24
O3			0.0053	12.80	84.75	169.50	0.1414	1.20	
P1	30	20	0.0053	12.50	82.95	165.90	0.1414	1.17	
P2			0.0053	12.55	76.80	153.60	0.1414	1.09	1.11
P3			0.0053	12.60	75.75	151.50	0.1414	1.07	
Q1	30	30	0.0053	12.60	79.80	159.60	0.1414	1.13	
Q2			0.0053	12.50	78.15	156.30	0.1414	1.11	1.09
Q3			0.0053	12.45	72.30	144.60	0.1414	1.02	

Table B11: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 28days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Cylinder (m^3)	Weight of Cylinder (Kg)	Crushing Force(P) (KN)	2P (KN)	∏LD (m^2)	Splitting Tensile Str. (N/mm^2)	Av. Split. Ten. Str. (N/mm^2)
A1	0	0	0.0053	12.90	179.40	358.80	0.1414	2.54	
A2			0.0053	12.70	169.40	338.80	0.1414	2.40	2.50
A3			0.0053	12.70	181.50	363.00	0.1414	2.57	
B1	0	10	0.0053	12.60	145.20	290.40	0.1414	2.05	
B2			0.0053	12.40	140.36	280.72	0.1414	1.99	2.10
B3			0.0053	12.40	160.68	321.36	0.1414	2.27	
C1	0	20	0.0053	12.50	140.36	280.72	0.1414	1.99	
C2			0.0053	12.60	141.04	282.08	0.1414	1.99	2.05
C3			0.0053	12.30	154.24	308.48	0.1414	2.18	
D1	0	30	0.0053	12.50	138.76	277.52	0.1414	1.96	
D2			0.0053	12.30	129.40	258.80	0.1414	1.83	1.97
D3			0.0053	12.70	150.04	300.08	0.1414	2.12	
E1	10	0	0.0053	12.70	186.12	372.24	0.1414	2.63	
E2			0.0053	12.50	150.04	300.08	0.1414	2.12	2.47
E3			0.0053	12.60	188.76	377.52	0.1414	2.67	
F1	10	10	0.0053	12.25	136.30	272.60	0.1414	1.93	
F2			0.0053	12.50	136.30	272.60	0.1414	1.93	2.01
F3			0.0053	12.65	153.88	307.76	0.1414	2.18	
G1	10	20	0.0053	12.80	135.52	271.04	0.1414	1.92	
G2			0.0053	12.60	121.00	242.00	0.1414	1.71	1.89
G3			0.0053	12.80	145.20	290.40	0.1414	2.05	
H1	10	30	0.0053	12.50	135.52	271.04	0.1414	1.92	
H2			0.0053	12.60	125.84	251.68	0.1414	1.78	1.79
H3			0.0053	12.70	118.58	237.16	0.1414	1.68	
I1	20	0	0.0053	12.60	174.24	348.48	0.1414	2.46	
I2			0.0053	12.40	169.40	338.80	0.1414	2.40	2.42
I3			0.0053	12.50	170.04	340.08	0.1414	2.41	
J1	20	10	0.0053	12.60	193.60	387.20	0.1414	2.74	
J2			0.0053	12.50	135.52	271.04	0.1414	1.92	2.37
J3			0.0053	12.45	174.24	348.48	0.1414	2.46	
L1	20	20	0.0053	12.60	121.00	242.00	0.1414	1.71	
L2			0.0053	12.50	101.64	203.28	0.1414	1.44	1.76
L3			0.0053	12.40	150.04	300.08	0.1414	2.12	
M1	20	30	0.0053	12.75	111.32	222.64	0.1414	1.57	
M2			0.0053	12.50	130.68	261.36	0.1414	1.85	1.71
M3			0.0053	12.40	121.00	242.00	0.1414	1.71	
N1	30	0	0.0053	12.40	126.40	252.80	0.1414	1.79	
N2			0.0053	12.50	126.80	253.60	0.1414	1.79	1.69
N3			0.0053	12.40	104.20	208.40	0.1414	1.47	
O1	30	10	0.0053	12.50	128.20	256.40	0.1414	1.81	
O2			0.0053	12.30	108.40	216.80	0.1414	1.53	1.65
O3			0.0053	12.40	113.00	226.00	0.1414	1.60	
P1	30	20	0.0053	12.30	110.60	221.20	0.1414	1.56	
P2			0.0053	12.50	102.40	204.80	0.1414	1.45	1.48
P3			0.0053	12.60	101.00	202.00	0.1414	1.43	
Q1	30	30	0.0053	12.40	106.40	212.80	0.1414	1.50	
Q2			0.0053	12.50	104.20	208.40	0.1414	1.47	1.45
Q3			0.0053	12.40	96.40	192.80	0.1414	1.36	

Table B12: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 56days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Cylinder (m^3)	Weight of Cylinder (Kg)	Crushing Force(P) (KN)	2P (KN)	∏LD (m^2)	Splitting Ten. Str. (N/mm^2)	Av. Split. Ten. Str. (N/mm^2)
A1	0	0	0.0053	12.60	182.53	365.06	0.1414	2.58	
A2			0.0053	12.60	175.28	350.56	0.1414	2.48	2.55
A3			0.0053	11.90	182.79	365.59	0.1414	2.59	
B1	0	10	0.0053	12.50	165.45	330.90	0.1414	2.34	
B2			0.0053	12.50	169.97	339.95	0.1414	2.40	2.37
B3			0.0053	12.60	168.21	336.42	0.1414	2.38	
C1	0	20	0.0053	12.50	164.13	328.26	0.1414	2.32	
C2			0.0053	12.50	159.87	319.73	0.1414	2.26	2.34
C3			0.0053	12.40	171.91	343.83	0.1414	2.43	
D1	0	30	0.0053	12.40	165.34	330.67	0.1414	2.34	
D2			0.0053	12.30	158.14	316.29	0.1414	2.24	2.29
D3			0.0053	12.50	163.11	326.22	0.1414	2.31	
E1	10	0	0.0053	12.30	173.07	346.13	0.1414	2.45	
E2			0.0053	12.40	174.58	349.17	0.1414	2.47	2.48
E3			0.0053	12.60	177.61	355.21	0.1414	2.51	
F1	10	10	0.0053	11.70	143.95	287.89	0.1414	2.04	
F2			0.0053	11.20	142.03	284.07	0.1414	2.01	2.04
F3			0.0053	12.50	146.72	293.45	0.1414	2.08	
G1	10	20	0.0053	12.50	137.76	275.53	0.1414	1.95	
G2			0.0053	12.60	137.57	275.15	0.1414	1.95	1.90
G3			0.0053	12.40	128.40	256.79	0.1414	1.82	
H1	10	30	0.0053	12.50	135.37	270.74	0.1414	1.91	
H2			0.0053	12.50	124.21	248.42	0.1414	1.76	1.83
H3			0.0053	12.30	129.07	258.15	0.1414	1.83	
I1	20	0	0.0053	12.50	171.07	342.14	0.1414	2.42	
I2			0.0053	12.00	174.99	349.97	0.1414	2.48	2.46
I3			0.0053	12.60	176.13	352.26	0.1414	2.49	
J1	20	10	0.0053	12.30	155.03	310.05	0.1414	2.19	
J2			0.0053	12.10	169.52	339.04	0.1414	2.40	2.38
J3			0.0053	12.50	180.85	361.70	0.1414	2.56	
L1	20	20	0.0053	12.30	137.17	274.33	0.1414	1.94	
L2			0.0053	12.50	127.52	255.05	0.1414	1.80	2.00
L3			0.0053	12.10	159.45	318.90	0.1414	2.26	
M1	20	30	0.0053	12.30	128.15	256.31	0.1414	1.81	
M2			0.0053	12.30	143.91	287.82	0.1414	2.04	1.97
M3			0.0053	12.20	144.93	289.86	0.1414	2.05	
N1	30	0	0.0053	12.20	134.47	268.94	0.1414	1.90	
N2			0.0053	12.10	138.37	276.73	0.1414	1.96	1.92
N3			0.0053	12.40	133.37	266.75	0.1414	1.89	
O1	30	10	0.0053	12.50	139.57	279.15	0.1414	1.97	
O2			0.0053	12.30	129.75	259.49	0.1414	1.84	1.90
O3			0.0053	12.40	133.71	267.43	0.1414	1.89	
P1	30	20	0.0053	12.30	131.64	263.29	0.1414	1.86	
P2			0.0053	12.50	124.57	249.14	0.1414	1.76	1.79
P3			0.0053	12.40	123.36	246.73	0.1414	1.74	
Q1	30	30	0.0053	12.30	128.02	256.04	0.1414	1.81	
Q2			0.0053	12.50	126.12	252.25	0.1414	1.78	1.76
Q3			0.0053	12.40	119.40	238.79	0.1414	1.69	

Table B13: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 90days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Cylinder (m^3)	Weight of Cylinder (Kg)	Crushing Force(P) (KN)	2P (KN)	∏LD (m^2)	Splitting Tensile Str. (N/mm^2)	Av. Split. Ten. Str. (N/mm^2)
A1	0	0	0.0053	12.50	195.66	391.31	0.1414	2.77	
A2			0.0053	12.60	181.16	362.32	0.1414	2.56	2.64
A3			0.0053	12.60	184.09	368.18	0.1414	2.60	
B1	0	10	0.0053	12.40	175.70	351.39	0.1414	2.49	
B2			0.0053	12.30	169.59	339.18	0.1414	2.40	2.46
B3			0.0053	12.50	175.74	351.48	0.1414	2.49	
C1	0	20	0.0053	12.40	177.90	355.79	0.1414	2.52	
C2			0.0053	12.40	168.69	337.38	0.1414	2.39	2.43
C3			0.0053	12.60	169.59	339.18	0.1414	2.40	
D1	0	30	0.0053	12.40	161.91	323.82	0.1414	2.29	
D2			0.0053	12.30	156.89	313.78	0.1414	2.22	2.33
D3			0.0053	12.50	176.18	352.36	0.1414	2.49	
E1	10	0	0.0053	12.50	170.01	340.02	0.1414	2.40	
E2			0.0053	12.50	189.13	378.26	0.1414	2.68	2.49
E3			0.0053	12.30	169.45	338.90	0.1414	2.40	
F1	10	10	0.0053	12.10	161.59	323.19	0.1414	2.29	
F2			0.0053	12.30	147.77	295.54	0.1414	2.09	2.31
F3			0.0053	12.40	179.57	359.13	0.1414	2.54	
G1	10	20	0.0053	12.30	140.01	280.01	0.1414	1.98	
G2			0.0053	12.30	134.15	268.30	0.1414	1.90	1.96
G3			0.0053	12.40	141.59	283.19	0.1414	2.00	
H1	10	30	0.0053	12.20	145.22	290.44	0.1414	2.05	
H2			0.0053	12.20	122.58	245.16	0.1414	1.73	1.92
H3			0.0053	12.20	139.57	279.13	0.1414	1.97	
I1	20	0	0.0053	12.30	187.90	375.79	0.1414	2.66	
I2			0.0053	12.50	180.57	361.15	0.1414	2.55	2.60
I3			0.0053	12.40	182.22	364.44	0.1414	2.58	
J1	20	10	0.0053	12.50	176.45	352.90	0.1414	2.50	
J2			0.0053	12.50	174.52	349.04	0.1414	2.47	2.47
J3			0.0053	12.60	173.46	346.91	0.1414	2.45	
L1	20	20	0.0053	12.50	153.33	306.67	0.1414	2.17	
L2			0.0053	12.50	153.41	306.81	0.1414	2.17	2.24
L3			0.0053	12.40	168.86	337.71	0.1414	2.39	
M1	20	30	0.0053	12.50	144.99	289.97	0.1414	2.05	
M2			0.0053	12.40	157.14	314.28	0.1414	2.22	2.22
M3			0.0053	12.50	168.86	337.71	0.1414	2.39	
N1	30	0	0.0053	12.30	152.54	305.08	0.1414	2.16	
N2			0.0053	12.20	149.93	299.86	0.1414	2.12	2.19
N3			0.0053	12.40	162.55	325.09	0.1414	2.30	
O1	30	10	0.0053	12.50	150.95	301.89	0.1414	2.14	
O2			0.0053	12.30	151.09	302.18	0.1414	2.14	2.15
O3			0.0053	12.40	154.43	308.85	0.1414	2.18	
P1	30	20	0.0053	12.30	152.69	305.37	0.1414	2.16	
P2			0.0053	12.20	146.74	293.48	0.1414	2.08	2.10
P3			0.0053	12.40	145.73	291.45	0.1414	2.06	
Q1	30	30	0.0053	12.40	149.64	299.28	0.1414	2.12	
Q2			0.0053	12.50	148.05	296.09	0.1414	2.09	2.07
Q3			0.0053	12.40	142.39	284.78	0.1414	2.01	

Table B14: Tensile Splitting Strength of VA-blended Cement Laterized Concrete at 120days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Cylinder (m^3)	Weight of Cylinder (Kg)	Crushing Force(P) (KN)	2P (KN)	∏LD (m^2)	Splitting Ten. Str. (N/mm^2)	Av. Split. Ten. Str. (N/mm^2)
A1	0	0	0.0053	12.50	195.66	391.31	0.1414	2.77	
A2			0.0053	12.60	181.16	362.32	0.1414	2.56	2.64
A3			0.0053	12.60	184.09	368.18	0.1414	2.60	
B1	0	10	0.0053	12.40	168.70	337.39	0.1414	2.39	
B2			0.0053	12.30	175.59	351.18	0.1414	2.48	2.46
B3			0.0053	12.50	177.74	355.48	0.1414	2.51	
C1	0	20	0.0053	12.40	167.90	335.79	0.1414	2.37	
C2			0.0053	12.40	178.69	357.38	0.1414	2.53	2.43
C3			0.0053	12.60	169.59	339.18	0.1414	2.40	
D1	0	30	0.0053	12.40	151.91	303.82	0.1414	2.15	
D2			0.0053	12.30	166.89	333.78	0.1414	2.36	2.33
D3			0.0053	12.50	176.18	352.36	0.1414	2.49	
E1	10	0	0.0053	12.50	172.01	344.02	0.1414	2.43	
E2			0.0053	12.50	169.13	338.26	0.1414	2.39	2.49
E3			0.0053	12.30	186.45	372.90	0.1414	2.64	
F1	10	10	0.0053	12.10	161.59	323.19	0.1414	2.29	
F2			0.0053	12.30	167.77	335.54	0.1414	2.37	2.31
F3			0.0053	12.40	159.57	319.13	0.1414	2.26	
G1	10	20	0.0053	12.30	140.01	280.01	0.1414	1.98	
G2			0.0053	12.30	134.15	268.30	0.1414	1.90	1.97
G3			0.0053	12.40	143.59	287.19	0.1414	2.03	
H1	10	30	0.0053	12.20	137.22	274.44	0.1414	1.94	
H2			0.0053	12.20	132.58	265.16	0.1414	1.88	1.93
H3			0.0053	12.20	139.57	279.13	0.1414	1.97	
I1	20	0	0.0053	12.30	177.90	355.79	0.1414	2.52	
I2			0.0053	12.50	170.57	341.15	0.1414	2.41	2.47
I3			0.0053	12.40	175.22	350.44	0.1414	2.48	
J1	20	10	0.0053	12.50	166.45	332.90	0.1414	2.35	
J2			0.0053	12.50	163.52	327.04	0.1414	2.31	2.44
J3			0.0053	12.60	187.46	374.91	0.1414	2.65	
L1	20	20	0.0053	12.50	153.33	306.67	0.1414	2.17	
L2			0.0053	12.50	153.41	306.81	0.1414	2.17	2.24
L3			0.0053	12.40	168.86	337.71	0.1414	2.39	
M1	20	30	0.0053	12.50	144.99	289.97	0.1414	2.05	
M2			0.0053	12.40	157.14	314.28	0.1414	2.22	2.22
M3			0.0053	12.50	168.86	337.71	0.1414	2.39	
N1	30	0	0.0053	12.30	152.54	305.08	0.1414	2.16	
N2			0.0053	12.20	149.93	299.86	0.1414	2.12	2.19
N3			0.0053	12.40	162.55	325.09	0.1414	2.30	
O1	30	10	0.0053	12.50	150.95	301.89	0.1414	2.14	
O2			0.0053	12.30	151.09	302.18	0.1414	2.14	2.15
O3			0.0053	12.40	154.43	308.85	0.1414	2.18	
P1	30	20	0.0053	12.30	152.69	305.37	0.1414	2.16	
P2			0.0053	12.20	146.74	293.48	0.1414	2.08	2.10
P3			0.0053	12.40	145.73	291.45	0.1414	2.06	
Q1	30	30	0.0053	12.40	149.64	299.28	0.1414	2.12	
Q2			0.0053	12.50	148.05	296.09	0.1414	2.09	2.07
Q3			0.0053	12.40	142.39	284.78	0.1414	2.01	

Table B15: Flexural Strength of VA-blended Cement Laterized Concrete at 3days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Beam (m^3)	Wt. of Beam (Kg)	Flexural Load(P) (KN)	Span l (m)	Pl (KNm)	bd^2 (m^3)	Modulus of Rupture (MOR) (N/mm^2)	Av. MOR (N/mm^2)
A1	0	0	0.005	11.800	6.00	0.3	1.800	0.001	1.80	
A2			0.005	12.000	5.65	0.3	1.695	0.001	1.70	1.75
A3			0.005	11.900	5.85	0.3	1.755	0.001	1.76	
B1	0	10	0.005	11.600	5.45	0.3	1.635	0.001	1.64	
B2			0.005	11.300	5.30	0.3	1.590	0.001	1.59	1.65
B3			0.005	12.000	5.75	0.3	1.725	0.001	1.73	
C1	0	20	0.005	12.400	5.45	0.3	1.635	0.001	1.64	
C2			0.005	12.600	5.35	0.3	1.605	0.001	1.61	1.61
C3			0.005	11.800	5.25	0.3	1.575	0.001	1.58	
D1	0	30	0.005	12.000	5.20	0.3	1.560	0.001	1.56	
D2			0.005	12.000	5.15	0.3	1.545	0.001	1.55	1.55
D3			0.005	11.000	5.10	0.3	1.530	0.001	1.53	
E1	10	0	0.005	11.800	5.63	0.3	1.688	0.001	1.69	
E2			0.005	12.000	5.45	0.3	1.634	0.001	1.63	1.66
E3			0.005	12.000	5.49	0.3	1.647	0.001	1.65	
F1	10	10	0.005	11.620	4.46	0.3	1.337	0.001	1.34	
F2			0.005	11.700	5.00	0.3	1.499	0.001	1.50	1.41
F3			0.005	12.000	4.64	0.3	1.391	0.001	1.39	
G1	10	20	0.005	11.600	3.87	0.3	1.161	0.001	1.16	
G2			0.005	11.650	3.92	0.3	1.175	0.001	1.17	1.16
G3			0.005	12.000	3.83	0.3	1.148	0.001	1.15	
H1	10	30	0.005	11.720	3.60	0.3	1.080	0.001	1.08	
H2			0.005	11.700	3.69	0.3	1.107	0.001	1.11	1.08
H3			0.005	11.500	3.56	0.3	1.067	0.001	1.07	
I1	20	0	0.005	12.500	4.91	0.3	1.474	0.001	1.47	
I2			0.005	12.400	5.18	0.3	1.553	0.001	1.55	1.51
I3			0.005	12.000	4.99	0.3	1.496	0.001	1.50	
J1	20	10	0.005	12.420	4.50	0.3	1.350	0.001	1.35	
J2			0.005	12.550	4.46	0.3	1.339	0.001	1.34	1.33
J3			0.005	11.900	4.35	0.3	1.305	0.001	1.31	
L1	20	20	0.005	12.550	3.68	0.3	1.103	0.001	1.10	
L2			0.005	12.200	3.56	0.3	1.069	0.001	1.07	1.09
L3			0.005	11.800	3.64	0.3	1.091	0.001	1.09	
M1	20	30	0.005	11.750	3.49	0.3	1.046	0.001	1.05	
M2			0.005	11.700	3.38	0.3	1.013	0.001	1.01	1.05
M3			0.005	12.000	3.68	0.3	1.103	0.001	1.10	
N1	30	0	0.005	11.550	4.48	0.3	1.344	0.001	1.34	
N2			0.005	12.000	4.80	0.3	1.439	0.001	1.44	1.34
N3			0.005	11.600	4.13	0.3	1.239	0.001	1.24	
O1	30	10	0.005	12.400	4.27	0.3	1.281	0.001	1.28	
O2			0.005	11.800	4.10	0.3	1.229	0.001	1.23	1.24
O3			0.005	11.650	4.07	0.3	1.220	0.001	1.22	
P1	30	20	0.005	12.100	3.96	0.3	1.187	0.001	1.19	
P2			0.005	11.900	4.20	0.3	1.260	0.001	1.26	1.24
P3			0.005	12.050	4.20	0.3	1.260	0.001	1.26	
Q1	30	30	0.005	11.850	3.33	0.3	0.998	0.001	1.00	
Q2			0.005	12.200	3.08	0.3	0.924	0.001	0.92	0.99
Q3			0.005	11.800	3.47	0.3	1.040	0.001	1.04	

Table B16: Flexural Strength of VA-blended Cement Laterized Concrete at 7days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Beam (m^3)	Wt. of Beam (Kg)	Flexural Load(P) (KN)	Span l (m)	Pl (KNm)	bd^2 (m^3)	MOR Pl/bd^2 (N/mm^2)	Av. MOR (KN/m^2)
A1	0	0	0.005	11.70	9.60	0.3	2.880	0.001	2.88	
A2			0.005	12.00	9.80	0.3	2.940	0.001	2.94	2.96
A3			0.005	11.80	10.20	0.3	3.060	0.001	3.06	
B1	0	10	0.005	11.60	8.85	0.3	2.655	0.001	2.66	
B2			0.005	11.30	8.50	0.3	2.550	0.001	2.55	2.67
B3			0.005	12.00	9.35	0.3	2.805	0.001	2.81	
C1	0	20	0.005	12.40	8.75	0.3	2.625	0.001	2.63	
C2			0.005	12.20	9.00	0.3	2.700	0.001	2.70	2.64
C3			0.005	11.50	8.60	0.3	2.580	0.001	2.58	
D1	0	30	0.005	12.00	7.95	0.3	2.385	0.001	2.39	
D2			0.005	12.00	7.60	0.3	2.280	0.001	2.28	2.37
D3			0.005	11.00	8.15	0.3	2.445	0.001	2.45	
E1	10	0	0.005	11.80	8.40	0.3	2.520	0.001	2.52	
E2			0.005	12.00	8.00	0.3	2.400	0.001	2.40	2.45
E3			0.005	12.00	8.10	0.3	2.430	0.001	2.43	
F1	10	10	0.005	11.95	6.00	0.3	1.800	0.001	1.80	
F2			0.005	11.70	5.50	0.3	1.650	0.001	1.65	1.67
F3			0.005	12.00	5.20	0.3	1.560	0.001	1.56	
G1	10	20	0.005	11.60	5.75	0.3	1.725	0.001	1.73	
G2			0.005	11.65	5.45	0.3	1.635	0.001	1.64	1.65
G3			0.005	12.00	5.30	0.3	1.590	0.001	1.59	
H1	10	30	0.005	11.72	5.40	0.3	1.620	0.001	1.62	
H2			0.005	11.70	5.65	0.3	1.695	0.001	1.70	1.63
H3			0.005	11.50	5.25	0.3	1.575	0.001	1.58	
I1	20	0	0.005	12.50	7.55	0.3	2.265	0.001	2.27	
I2			0.005	12.40	7.80	0.3	2.340	0.001	2.34	2.26
I3			0.005	12.00	7.25	0.3	2.175	0.001	2.18	
J1	20	10	0.005	12.72	6.40	0.3	1.920	0.001	1.92	
J2			0.005	12.65	6.30	0.3	1.890	0.001	1.89	1.95
J3			0.005	11.90	6.75	0.3	2.025	0.001	2.03	
L1	20	20	0.005	12.55	6.60	0.3	1.980	0.001	1.98	
L2			0.005	12.60	6.35	0.3	1.905	0.001	1.91	1.92
L3			0.005	11.80	6.20	0.3	1.860	0.001	1.86	
M1	20	30	0.005	11.75	6.45	0.3	1.935	0.001	1.94	
M2			0.005	11.70	6.25	0.3	1.875	0.001	1.88	1.90
M3			0.005	12.00	6.30	0.3	1.890	0.001	1.89	
N1	30	0	0.005	11.55	7.40	0.3	2.220	0.001	2.22	
N2			0.005	12.00	7.10	0.3	2.130	0.001	2.13	2.22
N3			0.005	11.60	7.70	0.3	2.310	0.001	2.31	
O1	30	10	0.005	12.40	7.00	0.3	2.100	0.001	2.10	
O2			0.005	11.80	6.85	0.3	2.055	0.001	2.06	2.05
O3			0.005	11.65	6.60	0.3	1.980	0.001	1.98	
P1	30	20	0.005	12.10	6.70	0.3	2.010	0.001	2.01	
P2			0.005	11.90	6.50	0.3	1.950	0.001	1.95	2.01
P3			0.005	12.05	6.90	0.3	2.070	0.001	2.07	
Q1	30	30	0.005	11.85	6.00	0.3	1.800	0.001	1.80	
Q2			0.005	12.20	5.95	0.3	1.785	0.001	1.79	1.76
Q3			0.005	11.80	5.65	0.3	1.695	0.001	1.70	

Table B17: Flexural Strength of VA-blended Cement Laterized Concrete at 28days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Beam (m^3)	Wt. of Beam (Kg)	Flexural Load(P) (KN)	Span l (m)	Pl (KNm)	bd^2 (m^3)	MOR (Pl/bd^2) (N/mm^2)	Av. MOR (KN/m^2)
A1	0	0	0.005	11.800	13.00	0.3	3.900	0.001	3.90	
A2			0.005	12.000	13.65	0.3	4.095	0.001	4.10	4.13
A3			0.005	11.900	14.63	0.3	4.388	0.001	4.39	
B1	0	10	0.005	11.600	13.75	0.3	4.125	0.001	4.13	
B2			0.005	11.300	14.30	0.3	4.290	0.001	4.29	4.11
B3			0.005	12.000	13.00	0.3	3.900	0.001	3.90	
C1	0	20	0.005	12.800	13.90	0.3	4.170	0.001	4.17	
C2			0.005	12.700	13.25	0.3	3.975	0.001	3.98	4.08
C3			0.005	11.500	13.60	0.3	4.080	0.001	4.08	
D1	0	30	0.005	12.000	13.25	0.3	3.975	0.001	3.98	
D2			0.005	12.000	13.30	0.3	3.990	0.001	3.99	3.98
D3			0.005	11.000	13.28	0.3	3.983	0.001	3.98	
E1	10	0	0.005	11.800	13.16	0.3	3.949	0.001	3.95	
E2			0.005	12.000	14.04	0.3	4.212	0.001	4.21	4.10
E3			0.005	12.000	13.75	0.3	4.124	0.001	4.12	
F1	10	10	0.005	11.620	13.86	0.3	4.158	0.001	4.16	
F2			0.005	11.700	13.28	0.3	3.983	0.001	3.98	4.04
F3			0.005	12.000	13.28	0.3	3.983	0.001	3.98	
G1	10	20	0.005	11.600	13.28	0.3	3.983	0.001	3.98	
G2			0.005	11.650	12.87	0.3	3.861	0.001	3.86	3.92
G3			0.005	12.000	13.07	0.3	3.922	0.001	3.92	
H1	10	30	0.005	11.720	13.46	0.3	4.037	0.001	4.04	
H2			0.005	11.700	12.87	0.3	3.861	0.001	3.86	3.89
H3			0.005	11.500	12.58	0.3	3.773	0.001	3.77	
I1	20	0	0.005	12.500	11.21	0.3	3.364	0.001	3.36	
I2			0.005	12.400	12.19	0.3	3.656	0.001	3.66	3.58
I3			0.005	12.000	12.43	0.3	3.729	0.001	3.73	
J1	20	10	0.005	12.720	11.96	0.3	3.589	0.001	3.59	
J2			0.005	12.650	11.96	0.3	3.589	0.001	3.59	3.54
J3			0.005	11.900	11.48	0.3	3.443	0.001	3.44	
L1	20	20	0.005	12.550	11.21	0.3	3.364	0.001	3.36	
L2			0.005	12.600	11.70	0.3	3.510	0.001	3.51	3.44
L3			0.005	11.800	11.46	0.3	3.437	0.001	3.44	
M1	20	30	0.005	11.750	11.44	0.3	3.431	0.001	3.43	
M2			0.005	11.700	11.44	0.3	3.431	0.001	3.43	3.38
M3			0.005	12.000	10.95	0.3	3.285	0.001	3.29	
N1	30	0	0.005	11.550	11.48	0.3	3.444	0.001	3.44	
N2			0.005	12.000	11.69	0.3	3.507	0.001	3.51	3.49
N3			0.005	11.600	11.76	0.3	3.528	0.001	3.53	
O1	30	10	0.005	12.400	11.34	0.3	3.402	0.001	3.40	
O2			0.005	11.800	11.13	0.3	3.339	0.001	3.34	3.35
O3			0.005	11.650	10.99	0.3	3.297	0.001	3.30	
P1	30	20	0.005	12.100	10.78	0.3	3.234	0.001	3.23	
P2			0.005	11.900	11.03	0.3	3.308	0.001	3.31	3.31
P3			0.005	12.050	11.27	0.3	3.381	0.001	3.38	
Q1	30	30	0.005	11.850	10.78	0.3	3.234	0.001	3.23	
Q2			0.005	12.200	10.54	0.3	3.161	0.001	3.16	3.19
Q3			0.005	11.800	10.61	0.3	3.182	0.001	3.18	

Table B18: Flexural Strength of VA-blended Cement Laterized Concrete at 56days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Beam (m^3)	Wt. of Beam (Kg)	Flexural Load(P) (KN)	Span l (m)	Pl (KNm)	bd^2 (m^3)	MOR (Pl/bd^2) (N/mm^2)	Av. MOR (N/mm^2)
A1	0	0	0.005	12.40	15.30	0.3	4.590	0.001	4.59	
A2			0.005	11.80	15.30	0.3	4.590	0.001	4.59	4.66
A3			0.005	12.00	15.95	0.3	4.785	0.001	4.79	
B1	0	10	0.005	12.20	15.68	0.3	4.703	0.001	4.70	
B2			0.005	11.70	15.98	0.3	4.793	0.001	4.79	4.62
B3			0.005	11.90	14.50	0.3	4.350	0.001	4.35	
C1	0	20	0.005	11.70	15.20	0.3	4.560	0.001	4.56	
C2			0.005	12.10	15.25	0.3	4.575	0.001	4.58	4.61
C3			0.005	11.60	15.60	0.3	4.680	0.001	4.68	
D1	0	30	0.005	11.30	14.95	0.3	4.485	0.001	4.49	
D2			0.005	11.70	14.14	0.3	4.241	0.001	4.24	4.40
D3			0.005	12.00	14.95	0.3	4.485	0.001	4.49	
E1	10	0	0.005	11.60	14.19	0.3	4.256	0.001	4.26	
E2			0.005	11.70	14.92	0.3	4.475	0.001	4.48	4.37
E3			0.005	12.50	14.63	0.3	4.388	0.001	4.39	
F1	10	10	0.005	11.40	15.05	0.3	4.516	0.001	4.52	
F2			0.005	12.30	13.41	0.3	4.023	0.001	4.02	4.20
F3			0.005	12.60	13.56	0.3	4.067	0.001	4.07	
G1	10	20	0.005	12.20	13.50	0.3	4.050	0.001	4.05	
G2			0.005	12.00	14.36	0.3	4.307	0.001	4.31	4.06
G3			0.005	12.50	12.72	0.3	3.817	0.001	3.82	
H1	10	30	0.005	11.80	13.41	0.3	4.023	0.001	4.02	
H2			0.005	12.00	13.43	0.3	4.030	0.001	4.03	4.01
H3			0.005	12.50	13.26	0.3	3.979	0.001	3.98	
I1	20	0	0.005	12.30	12.68	0.3	3.803	0.001	3.80	
I2			0.005	12.20	12.66	0.3	3.797	0.001	3.80	3.76
I3			0.005	11.80	12.29	0.3	3.687	0.001	3.69	
J1	20	10	0.005	12.10	11.96	0.3	3.589	0.001	3.59	
J2			0.005	12.30	12.92	0.3	3.876	0.001	3.88	3.68
J3			0.005	12.00	11.94	0.3	3.583	0.001	3.58	
L1	20	20	0.005	12.30	11.46	0.3	3.437	0.001	3.44	
L2			0.005	11.40	12.19	0.3	3.656	0.001	3.66	3.62
L3			0.005	12.00	12.55	0.3	3.766	0.001	3.77	
M1	20	30	0.005	12.00	11.46	0.3	3.437	0.001	3.44	
M2			0.005	12.50	11.94	0.3	3.583	0.001	3.58	3.47
M3			0.005	11.50	11.29	0.3	3.386	0.001	3.39	
N1	30	0	0.005	11.55	11.67	0.3	3.502	0.001	3.50	
N2			0.005	12.00	11.73	0.3	3.518	0.001	3.52	3.50
N3			0.005	11.60	11.64	0.3	3.491	0.001	3.49	
O1	30	10	0.005	12.40	11.38	0.3	3.414	0.001	3.41	
O2			0.005	11.80	11.60	0.3	3.481	0.001	3.48	3.40
O3			0.005	11.65	11.03	0.3	3.308	0.001	3.31	
P1	30	20	0.005	12.10	11.66	0.3	3.497	0.001	3.50	
P2			0.005	11.90	10.99	0.3	3.297	0.001	3.30	3.38
P3			0.005	12.05	11.17	0.3	3.350	0.001	3.35	
Q1	30	30	0.005	11.85	11.13	0.3	3.339	0.001	3.34	
Q2			0.005	12.20	10.50	0.3	3.150	0.001	3.15	3.24
Q3			0.005	11.80	10.76	0.3	3.229	0.001	3.23	

Table B19: Flexural Strength of VA-blended Cement Laterized Concrete at 90days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Beam (m^3)	Wt. of Beam (Kg)	Flexural Load(P) (KN)	Span l (m)	Pl (KNm)	bd^2 (m^3)	MOR (Pl/bd^2) (N/mm^2)	Av. MOR (KN/m^2)
A1	0	0	0.005	11.90	15.60	0.3	4.680	0.001	4.68	
A2			0.005	12.00	15.95	0.3	4.785	0.001	4.79	4.68
A3			0.005	12.50	15.28	0.3	4.583	0.001	4.58	
B1	0	10	0.005	11.20	15.20	0.3	4.560	0.001	4.56	
B2			0.005	11.30	15.65	0.3	4.695	0.001	4.70	4.63
B3			0.005	11.90	15.40	0.3	4.620	0.001	4.62	
C1	0	20	0.005	11.90	14.30	0.3	4.290	0.001	4.29	
C2			0.005	12.00	16.25	0.3	4.875	0.001	4.88	4.62
C3			0.005	11.60	15.60	0.3	4.680	0.001	4.68	
D1	0	30	0.005	11.30	14.65	0.3	4.395	0.001	4.40	
D2			0.005	11.70	14.98	0.3	4.493	0.001	4.49	4.43
D3			0.005	12.00	14.63	0.3	4.388	0.001	4.39	
E1	10	0	0.005	11.80	15.21	0.3	4.563	0.001	4.56	
E2			0.005	12.10	14.90	0.3	4.469	0.001	4.47	4.47
E3			0.005	12.00	14.60	;0.3	4.381	0.001	4.38	
F1	10	10	0.005	12.60	14.00	0.3	4.199	0.001	4.20	
F2			0.005	11.30	14.09	0.3	4.226	0.001	4.23	4.20
F3			0.005	12.50	13.93	0.3	4.178	0.001	4.18	
G1	10	20	0.005	12.60	13.73	0.3	4.118	0.001	4.12	
G2			0.005	12.00	14.31	0.3	4.293	0.001	4.29	4.18
G3			0.005	11.90	13.73	0.3	4.118	0.001	4.12	
H1	10	30	0.005	11.80	13.37	0.3	4.010	0.001	4.01	
H2			0.005	11.90	14.00	0.3	4.199	0.001	4.20	4.13
H3			0.005	12.00	13.95	0.3	4.185	0.001	4.19	
I1	20	0	0.005	12.00	12.64	0.3	3.791	0.001	3.79	
I2			0.005	12.30	12.38	0.3	3.713	0.001	3.71	3.79
I3			0.005	11.90	12.90	0.3	3.870	0.001	3.87	
J1	20	10	0.005	11.90	11.96	0.3	3.589	0.001	3.59	
J2			0.005	12.00	12.38	0.3	3.713	0.001	3.71	3.75
J3			0.005	12.50	13.16	0.3	3.949	0.001	3.95	
L1	20	20	0.005	12.40	11.70	0.3	3.510	0.001	3.51	
L2			0.005	12.00	12.68	0.3	3.803	0.001	3.80	3.65
L3			0.005	12.20	12.15	0.3	3.645	0.001	3.65	
M1	20	30	0.005	11.50	11.48	0.3	3.443	0.001	3.44	
M2			0.005	11.80	11.70	0.3	3.510	0.001	3.51	3.48
M3			0.005	12.00	11.63	0.3	3.488	0.001	3.49	
N1	30	0	0.005	11.90	11.87	0.3	3.560	0.001	3.56	
N2			0.005	12.10	11.90	0.3	3.570	0.001	3.57	3.56
N3			0.005	11.80	11.80	0.3	3.539	0.001	3.54	
O1	30	10	0.005	12.20	12.12	0.3	3.635	0.001	3.64	
O2			0.005	11.85	12.08	0.3	3.623	0.001	3.62	3.53
O3			0.005	12.05	11.06	0.3	3.318	0.001	3.32	
P1	30	20	0.005	11.95	11.83	0.3	3.549	0.001	3.55	
P2			0.005	12.00	11.66	0.3	3.497	0.001	3.50	3.52
P3			0.005	12.00	11.76	0.3	3.528	0.001	3.53	
Q1	30	30	0.005	11.95	10.92	0.3	3.276	0.001	3.28	
Q2			0.005	11.85	10.64	0.3	3.192	0.001	3.19	3.25
Q3			0.005	12.10	10.92	0.3	3.276	0.001	3.28	

Table B20: Flexural Strength of VA-blended Cement Laterized Concrete at 120days Curing

Specimen No.	Lat. Cont. (%)	VA Cont. (%)	Vol. of Beam (m^3)	Wt. of Beam (Kg)	Flexural Load(P) (KN)	Span l (m)	Pl (KNm)	bd^2 (m^3)	MOR (Pl/bd^2) (N/mm^2)	Av. MOR (KN/m^2)
A1	0	0	0.005	11.80	16.25	0.3	4.875	0.001	4.88	
A2			0.005	11.50	16.20	0.3	4.860	0.001	4.86	4.90
A3			0.005	12.00	16.50	0.3	4.950	0.001	4.95	
B1	0	10	0.005	11.80	15.70	0.3	4.710	0.001	4.71	
B2			0.005	12.00	15.00	0.3	4.500	0.001	4.50	4.65
B3			0.005	11.90	15.75	0.3	4.725	0.001	4.73	
C1	0	20	0.005	11.90	16.25	0.3	4.875	0.001	4.88	
C2			0.005	12.50	15.70	0.3	4.710	0.001	4.71	4.63
C3			0.005	12.10	14.30	0.3	4.290	0.001	4.29	
D1	0	30	0.005	12.20	15.00	0.3	4.500	0.001	4.50	
D2			0.005	12.00	14.70	0.3	4.410	0.001	4.41	4.53
D3			0.005	12.00	15.60	0.3	4.680	0.001	4.68	
E1	10	0	0.005	12.20	14.94	0.3	4.482	0.001	4.48	
E2			0.005	11.80	15.03	0.3	4.509	0.001	4.51	4.48
E3			0.005	12.00	14.85	0.3	4.455	0.001	4.46	
F1	10	10	0.005	12.20	13.73	0.3	4.118	0.001	4.12	
F2			0.005	12.00	14.18	0.3	4.253	0.001	4.25	4.21
F3			0.005	12.50	14.18	0.3	4.253	0.001	4.25	
G1	10	20	0.005	11.80	13.68	0.3	4.104	0.001	4.10	
G2			0.005	11.90	14.31	0.3	4.293	0.001	4.29	4.17
G3			0.005	12.00	13.73	0.3	4.118	0.001	4.12	
H1	10	30	0.005	11.50	13.95	0.3	4.185	0.001	4.19	
H2			0.005	11.90	13.77	0.3	4.131	0.001	4.13	4.14
H3			0.005	11.50	13.73	0.3	4.118	0.001	4.12	
I1	20	0	0.005	11.80	12.56	0.3	3.769	0.001	3.77	
I2			0.005	12.00	12.38	0.3	3.713	0.001	3.71	3.80
I3			0.005	12.50	13.09	0.3	3.926	0.001	3.93	
J1	20	10	0.005	11.60	12.88	0.3	3.864	0.001	3.86	
J2			0.005	11.80	12.66	0.3	3.797	0.001	3.80	3.77
J3			0.005	12.00	12.15	0.3	3.645	0.001	3.65	
L1	20	20	0.005	11.90	12.45	0.3	3.735	0.001	3.74	
L2			0.005	12.00	12.30	0.3	3.690	0.001	3.69	3.66
L3			0.005	12.00	11.87	0.3	3.561	0.001	3.56	
M1	20	30	0.005	12.50	11.70	0.3	3.510	0.001	3.51	
M2			0.005	12.00	11.59	0.3	3.476	0.001	3.48	3.48
M3			0.005	11.80	11.51	0.3	3.454	0.001	3.45	
N1	30	0	0.005	12.00	11.80	0.3	3.539	0.001	3.54	
N2			0.005	11.90	11.94	0.3	3.581	0.001	3.58	3.56
N3			0.005	12.10	11.83	0.3	3.549	0.001	3.55	
O1	30	10	0.005	11.80	11.56	0.3	3.469	0.001	3.47	
O2			0.005	11.90	11.94	0.3	3.581	0.001	3.58	3.53
O3			0.005	12.10	11.76	0.3	3.528	0.001	3.53	
P1	30	20	0.005	12.05	12.01	0.3	3.602	0.001	3.60	
P2			0.005	11.95	11.73	0.3	3.518	0.001	3.52	3.53
P3			0.005	11.90	11.55	0.3	3.465	0.001	3.47	
Q1	30	30	0.005	12.00	10.75	0.3	3.224	0.001	3.22	
Q2			0.005	11.90	10.85	0.3	3.255	0.001	3.26	3.25
Q3			0.005	12.20	10.89	0.3	3.266	0.001	3.27	

APPENDIX C

Output of Statistical Analysis

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The GLM Procedure

Class Level Information

Class	Levels	Values
LatCont	4	0 10 20 30
VaCont	4	0 10 20 30
CuringAge	6	3 7 28 56 90 120

Number of Observations Read	288
Number of Observations Used	288

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The GLM Procedure

Dependent Variable: CompStr

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	95	14829.75497	156.10268	217.41	<.0001
Corrected Total	287	14967.61497			

R-Square	Coeff Var	Root MSE	CompStr Mean
0.990789	4.725702	0.847361	17.93090

Source	DF	Type III SS	Mean Square	F Value	Pr > F
LatCont	3	140.38844	46.79615	65.17	<.0001
VaCont	3	803.44649	267.81550	372.99	<.0001
LatCont*VaCont	9	127.76059	14.19562	19.77	<.0001
CuringAge	5	13543.90976	2708.78195	3772.57	<.0001
LatCont*CuringAge	15	137.83094	9.18873	12.80	<.0001
VaCont*CuringAge	15	23.17288	1.54486	2.15	0.0093
LatCon*VaCont*Curing	45	53.24587	1.18324	1.65	0.0112
Error	192	137.86000	0.71802		

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The GLM Procedure

Dependent Variable: SpTenStr

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	95	61.66486667	0.64910386	39.56	<.0001
Corrected Total	287	64.81560000			

R-Square	Coeff Var	Root MSE	SpTenStr Mean
0.951389	6.751083	0.128102	1.897500

Source	DF	Type III SS	Mean Square	F Value	Pr > F
LatCont	3	7.38538611	2.46179537	150.02	<.0001
VaCont	3	6.73379167	2.24459722	136.78	<.0001
LatCont*VaCont	9	1.29848889	0.14427654	8.79	<.0001
CuringAge	5	44.24445417	8.84889083	539.24	<.0001
LatCont*CuringAge	15	1.17289306	0.07819287	4.76	<.0001
VaCont*CuringAge	15	0.28125417	0.01875028	1.14	0.3210
LatCon*VaCont*Curing	45	0.54859861	0.01219108	0.74	0.8806
Error	192	3.15073333	0.01641007		

The GLM Procedure

Dependent Variable: FlexStr

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	95	367.4257542	3.8676395	327.78	<.0001
Corrected Total	287	369.6912875			

R-Square	Coeff Var	Root MSE	FlexStr Mean
0.993872	3.402766	0.108626	3.192292

Source	DF	Type III SS	Mean Square	F Value	Pr > F
LatCont	3	34.2290958	11.4096986	966.95	<.0001
VaCont	3	4.5366792	1.5122264	128.16	<.0001
LatCont*VaCont	9	0.5182569	0.0575841	4.88	<.0001
CuringAge	5	319.5566167	63.9113233	5416.37	<.0001
LatCont*CuringAge	15	7.3764417	0.4917628	41.68	<.0001
VaCont*CuringAge	15	0.6616083	0.0441072	3.74	<.0001
LatCon*VaCont*Curing	45	0.5470556	0.0121568	1.03	0.4302
Error	192	2.2655333	0.0117997		

The GLM Procedure

Duncan's Multiple Range Test for CompStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.718021

Number of Means	2	3	4
Critical Range	.2786	.2932	.3030

Means with the same letter are not significantly different.

Duncan Grouping		Mean	N	Lat Cont
	A	19.1236	72	0
	B	17.7139	72	20
C	B	17.4889	72	10
C		17.3972	72	30

The GLM Procedure

Duncan's Multiple Range Test for SpTenStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.01641

Number of Means	2	3	4
Critical Range	.04211	.04433	.04581

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Lat Cont
A	2.08750	72	0
B	1.96847	72	20
C	1.88403	72	10
D	1.65000	72	30

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The GLM Procedure

Duncan's Multiple Range Test for FlexStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.0118

Number of Means	2	3	4
Critical Range	.03571	.03759	.03884

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Lat Cont
A	3.70167	72	0
B	3.30431	72	10
C	2.95347	72	20
D	2.80972	72	30

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The GLM Procedure

Duncan's Multiple Range Test for CompStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.718021

Number of Means	2	3	4
Critical Range	.2786	.2932	.3030

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Va Cont
A	20.4000	72	0
B	18.3208	72	10
C	17.1431	72	20
D	15.8597	72	30

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The GLM Procedure

Duncan's Multiple Range Test for SpTenStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.01641

Number of Means	2	3	4
Critical Range	.04211	.04433	.04581

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Va Cont
A	2.12361	72	0
B	1.94903	72	10
C	1.78319	72	20
D	1.73417	72	30

The GLM Procedure

Duncan's Multiple Range Test for FlexStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.0118

Number of Means	2	3	4
Critical Range	.03571	.03759	.03884

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Va Cont
A	3.37458	72	0
B	3.21528	72	10
C	3.15361	72	20
D	3.02569	72	30

The GLM Procedure

Duncan's Multiple Range Test for CompStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.718021

Number of Means	2	3	4	5	6
Critical Range	.3412	.3591	.3711	.3799	.3868

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Curing Age
A	24.7688	48	120
B	24.1583	48	90
C	21.8229	48	56
D	19.7292	48	28
E	9.6396	48	7
F	7.4667	48	3

The GLM Procedure

Duncan's Multiple Range Test for SpTenStr

NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.01641

Number of Means	2	3	4	5	6
Critical Range	.05158	.05429	.05610	.05743	.05847

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Curing Age
A	2.28771	48	90
A	2.27833	48	120
B	2.12458	48	56
C	1.95750	48	28
D	1.43729	48	7
E	1.29958	48	3

The GLM Procedure

Duncan's Multiple Range Test for FlexStr

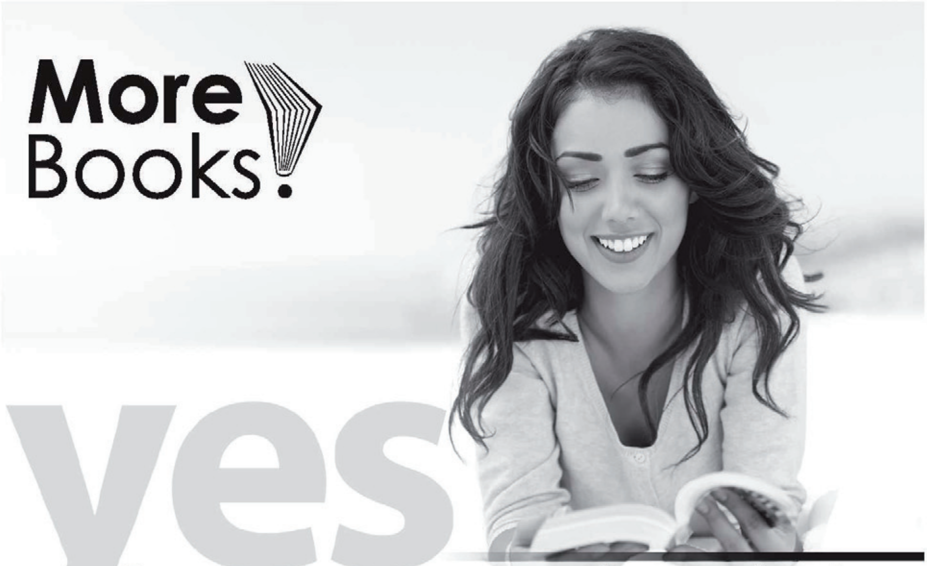
NOTE: This test controls the Type I comparisonwise error rate, not the experimentwise error rate.

Alpha	0.05
Error Degrees of Freedom	192
Error Mean Square	0.0118

Number of Means	2	3	4	5	6
Critical Range	.04373	.04604	.04757	.04870	.04958

Means with the same letter are not significantly different.

Duncan Grouping	Mean	N	Curing Age
A	4.01792	48	120
A	3.99188	48	90
B	3.93750	48	56
C	3.72000	48	28
D	2.13250	48	7
E	1.35396	48	3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