INVESTIGATION OF THE STRENGTH CHARACTERISTICS OF LATECRETE MATERIALS

BY

SAMSON IMONI
B. Eng. (Civil)
(M.Sc./ENG./8840/2009-10)

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FLEXURAL STRENGTH OF REINFORCED LATECRETE BEAMS

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SAMSON IMONI

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DECLARATION

I hereby declare that this thesis titled "Investigation of the Strength Characteristics of Latecrete Materials" was carried out by me in the Department of Civil Engineering, Ahmadu Bello University Zaria under the supervision of Prof. S.P. Ejeh and Dr. Y.D. Amartey. All information derived from the literature has been duly acknowledged by means of reference. No part of this work has been previously presented or submitted to any institution for the award of a degree or higher degree.

SAMSON IMONI
(M.Sc./ENG/8840/2009-10)

SIGNATURE

CERTIFICATION

This thesis titled "Investigation of the Strength Characteristics of Latecrete Materials" meets the regulations governing the award of the degree of Master of Science in Civil Engineering of the Ahmadu Bello University, and is approved for its contribution to knowledge and literary presentation.

Prof. S.P. Ejeh	_____	_____
Chairman Supervisory Committee	Signature	Date
Dr. Y.D. Amartey	_____	_____
Member, Supervisory Committee	Signature	Date
Dr. I. Abubakar	_____	_____
Head of Department	Signature	Date
Prof. A. A. Joshua	_____	_____
Dean, School of Postgraduate Studies	Signature	Date

DEDICATION

To the Almighty God and all my loved ones.

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ABSTRACT

A laboratory Investigation of the Strength Characteristics of Latecrete materials was carried out. The laterite used falls under AASHTO soil classification A-7-6 (10) with low plasticity clay (CL) according to the Unified soil classification system. This laterite is within zone four gradation characterized by fine laterite and has kaolinite as its dominant clay mineral. The Experimental programme involves the fabrication of twenty four cubes, six cylinders, and twenty beams of concrete and latecrete materials; and testing them for compressive strength, tensile strength and flexural strength. Tests show that the 28 days compressive strength of concrete cubes was 23.26 N/mm^2, while that of latecrete cubes was 8.89 N/mm^2. The tensile strength of concrete cylinders was found to be 1.82 N/mm^2, while that of latecrete cylinders was 0.67 N/mm^2. The Flexural strength recorded for plain concrete beams was 2.89 N/mm^2, while plain latecrete beams has a value 1.44 N/mm^2. The flexural strength of 13.58 N/mm2 was recorded for the reinforced concrete beams and 7.80 N/mm^2 for reinforced latecrete beams, indicating that the flexural strength of latecrete beam is approximately 50% of that of concrete beam specimens. The load-deflection behaviour of the beams are essentially linear within the elastic range of loading and the measured plots of the stress and strain shows that the flexural modulus of the reinforced concrete beams was 3.78 x 10^5 N/mm^2, while that of the reinforced latecrete beams was 4.50 x 10^5 N/mm^2. Also, the flexural modulus of both plain concrete and latecrete beams was the value of 2.70 x 10^5 N/mm^2. Based on the findings of this investigation, it was observed that the behaviour of latecrete is similar to that of concrete; however, the concrete materials showed better strength characteristics than the latecrete materials.

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CHAPTER ONE

INTRODUCTION

1.1 PREAMBLE

Normal cement concrete has been widely used as a construction material in building and civil engineering structures (Neville, 1995). In recent times, the cost of concrete production has increased. This has given rise to the development of construction materials from cheap and readily available material sources which reduces construction costs and yet remains consistent with the advancing state of the indigenous technology (Adepegba, 1975a; Osunade, 2002; Ata, 2003).

Lateritic soils belong to this category of cheap and readily available materials. Lateritic soils are known to be available in large quantities all over Nigeria as well as in most tropical countries of the world. They are essentially products of tropical or sub-tropical weathering, usually found in areas where natural drainage is impeded (Ata, 2007). Laterite has been investigated as a replacement of sand in the conventional normal cement concrete (Adepegba, 1975a; Osunade, 2002; Ata, 2003).

Concrete in which the sand component is partially or wholly replaced by laterite is called latecrete, LATCON or laterized concrete (Adepegba, 1975a; Ata, 2007).

1.2 STATEMENT OF THE PROBLEM

In recent times, the demand for alternative construction materials has increased due to the cost of the conventional materials. To overcome this problem, there have been concerted efforts in investigating into indigenous materials such as latecrete. However, there is paucity of information on the properties of latecrete, and thus there is low knowledge of the material.

1.3 JUSTIFICATION FOR THE STUDY

There is skepticism in the acceptance of Latecrete as a construction material as local codes and specifications for the material are not yet available (Ata, 2007). Thus, the development and standardization of routine tests for determining the mechanical properties of this material is of unquestionable importance. This is because, the rewards can be great in design economy, as knowledge of this kind enables the Engineer to employ logical factors of safety rather than conservative factors of ignorance (Harmer, et al, 1964).

Although a good number of researchers concentrated on compressive strength as a fundamental property in examining the strength of latecrete. (Adepegba, 1975a; Lasisi, et al, 1990; Ata, 2003). However, when considering its use in a beam element of a structure, the flexural strength is also important as it models how a beam is normally loaded; and this necessitates the choice of this topic.

1.4 AIM AND OBJECTIVES

This research is aimed at investigating the Strength characteristics of plain and reinforced latecrete Materials.

Specifically, the objectives of this research are to investigate properties of latecrete specimens such as: flexural strength, compressive strength, and tensile strength.

1.5 SCOPE OF INVESTIGATION

The scope of investigation of this research work is to study the the strength characteristics of both plain and reinforced Concrete and Latecrete materials.

1.6 LIMITATIONS

The work is limited to the strength characteristics of latecrete materials.

1.7 OUTCOME OF RESEARCH

The research work reveals the flexural strength of latecrete and concrete beams using a three point bending test. The flexural strength of reinforced concrete beam is 13.58 N/mm2, while that of reinforced latecrete beams is 7.8 N/mm2. Also, the flexural strength of plain concrete beam is 2.89 N/mm^2, while that of plain latecrete beams is 1.44 N/mm2.

CHAPTER TWO

LITERATURE REVIEW

2.1 LATECRETE

"Latecrete" (Laterized concrete) is defined as concrete in which stable laterite fines replace sand. i.e instead of the cement, fine aggregate (sand), coarse aggregate (gravel), and water in the normal cement concrete, the sand fraction is replaced by stable laterite; thereby enhancing construction, particularly, in parts of the world where suitable sharp sand is not available, since in such areas nature coincidentally compensates by providing large deposits of laterite (Adepegba, 1975a).

Latecrete just like normal cement concrete is a very variable material in terms of strength. It is poor in tension and good in compression; and as such it normally relies on reinforcing steel to counteract the effect of tensile forces in service. The elastic characteristics of a material are measures of its stiffness. Determination of elastic properties of laterized concrete is necessary for stress analysis associated with environmental effects and for computation of the design stresses, deformations and deflections under load in concrete and reinforced concrete structures (Abadjieva, 1998).

Flexural Strength of a beam is the ability of a beam to resist failure in bending. Flexural tests in beams are usually made to determine strength and stiffness in bending; occasionally they are made to obtain a fairly complete picture of stress distribution in a flexural member. Such tests also offer a means of determining the resilience and toughness of materials in bending (Harmer, et al, 1964)

As load is applied on a reinforced latecrete beam, changes begin to take place within the sections, due to the effect of stress and strain, until eventual collapse. When these changes are investigated, the behaviour of the materials in each cycle of loading

can be considerably different. Thus, the strength characteristics of latecrete play an important role in its structural performance.

Inspite of the concerted effort by researchers in this regard, there is no standard specification on the mechanical properties of Latecrete. However, some attempts have been made at investigating its compressive strength properties. These researches and some indirect information on soil stabilization and cement stabilized laterite will be used in this research.

Adepegba (1975a) in a study replaced sand in the normal cement concrete with laterite fines, and this he referred to as laterized concrete. He intimately mixed cement, laterite fine and gravel in the following proportions by weight: 1:1:2, 1:1.5:3 and 1:2:4, and made the following observations:

(a) Laterized concrete requires more water than normal cement concrete.

(b) The laterized concrete will be too dry if the W/C ratio is less than 0.5 and too wet if higher than 1.0 for the mix proportions (1:1:2, 1:1.5:3, and 1:2:4), while that of the normal cement concrete ranges between 0.3 and 1.2.

(c) (i) The minimum and maximum W/C ratio for 1:2:4 mix by weight of laterized concrete are 0.65 and 0.95 respectively.

(ii) For 1:1.5:3 and 1:1:2 mix are 0.55 and 0.85 respectively.

(d) For practical purposes, a W/C of about 0.75 is recommended for 1:2:4 mix by weight since this would yield a compressive strength of 18.5 Mpa (18.5 N/mm^2) in 28 days and a W/C of about 0.65 for 1:1½:3 and would yield a compressive strength of about 21.45 N/mm^2; and 23.59 N/mm^2 for 1:1:2 in 28 days. He further observed that the compressive strength at 7-days was about 40 – 60% of the compressive strength at 28 days, while at 14-days, the compressive strength was about 70 -80% of the 28 days compressive strength.

In another study, Balogun and Adepegba (1982) discovered that the most suitable mix of laterized concrete for structural purposes is (1:1½:3), using batching by weight with a water/cement ratio of 0.65, provided that the laterite content is kept below 50 percent of the total fine aggregate content. Lasisi, et. al. (1990) have shown 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.

In a study on a similar material, Ejeh (1982) investigating the compressive strength of 400 mm x 150 mm x 150 mm soilcrete hollow blocks, made the following observations:

(i). that properly manufactured soilcrete blocks can fully satisfy the requirements imposed by codes of practice.

(ii). the soilcrete hollow blocks have an average wet/dry compressive strength ratio of 0.65.

(iii). the soilcrete hollow blocks have a minimum wet/dry compressive strength ratio of 0.42.

Investigating the curing methods for soilcrete hollow blocks specimens, Ejeh (1990) recommended that for standard tests and quality control, immersion in water is more appropriate to laboratory work since spraying may be uneven and also the quality of water used may be doubtful. However, soilcrete hollow blocks from the study fully developed its dry compressive strength after curing by spraying with water and covering with water-proof materials for 7 days.

Abejide (1997) in a study on soilcrete blocks suggested that A6 grouping of lateritic soil suitable for stabilized soilcrete blocks should be based on colour, particle size distribution, chemical analysis, physical and structural properties, and morphology.

Furthermore, he stated that uniform pressure is required on the blocks surfaces to ascertain its true compressive strength; a plate that distributes the pressure from the compressive testing machine is required.

In a study on cement-stabilized soil, Adepegba (1975b) indicated that some important factors such as cement content, particle size of soil, type of soil, uniformity of mix, airing period and condition of curing affect the strength. He also reported further that according to Highway Research Board, a good soil for economic stabilization with cement should be within these particle sizes:

Passing ASTM sieve size No. 4 (4.76mm)	-	50%
Passing ASTM sieve size No. 40 (0.42mm)	-	15 – 100%
Passing ASTM sieve size No. 200 (0.074mm)	-	50%

In addition, the liquid limit should be less than 40, and the plasticity index not greater than 18.

According to O'Flaherty (1974), any soil can be stabilized if sufficient amount of cement is added, but the cement content required to achieve a given stability must normally be increased as the silt and clay content increases. This position explicitly means that it may not be economically possible to stabilize certain soils (e.g heavy clays) with cement. He reported further that whereas, the British experience is that soils with liquid limit greater than 45 and plasticity index over 20, cannot be economically stabilized with cement, the American experience indicates that sandy and gravelly soils with about 10 – 35% silt and clay have the most favourable characteristics for cement stabilization.

Ola (1977) reported that soils with liquid limit less than 40 and clay content up to about 20% are suitable for stabilization and suggested that the table below should serve as the guide to selection of soils for stabilization:

Table 2.1 Selection guide for soils suitable for stabilization

Physical properties	For Permanent Urban Buildings	For Rural Housing with low rainfall
Clay content	5 – 20%	5 – 30%
Sand content	33% minimum	40% minimum
Liquid limit	40% minimum	50% minimum
Plasticity index	2.5 – 22%	2.5 – 30%
Optimum moisture content	10 – 14%	7 – 16%

After Ola (1977)

Neville (1995) opined that laterite could rarely produce concrete stronger than 10Mpa. However, Osunade (2002), and Ata (2003) proved this finding not to be true and submitted that laterite could produce concrete of higher grades.

In a study on the effect of mix proportion and reinforcement size on the anchorage bond stress of laterized concrete, Osunade and Babalola (1991) established that both variables have a significant effect on the anchorage bond stress between plain round steel reinforcement and laterized concrete, and that it increases with increase in the size of reinforcement.

In another study by Osunade (1994), he established that increase in shear and tensile strength of laterized concrete was obtained as grain size ranges and curing ages increased. Also, greater values of shear and tensile strengths were obtained for rectangular specimens than those obtained for cylinders.

The following major conclusions emerged from the experimental study reported by Ata (2007) on the effect of varying curing age and water/cement ratio on the elastic properties of laterized concrete:

(i). The modulus of elasticity of laterized concrete lies between 7000 and 9500 MPa, while that of deformability lies between the range of 5000 and 6000 MPa.

(ii). Modulus of elasticity and modulus of deformability of laterized concrete increase with an increase in curing age.

(iii). The value of modulus of elasticity of laterized concrete is always higher than its corresponding modulus of deformability.

(iv). The richer the mix; the higher the moduli of elasticity and deformability of laterized concrete.

(v). The stronger the laterized concrete; the higher the modulus of elasticity and the modulus of deformability.

(vi). Any water/cement ratio that will give laterized concrete high strength will increase its modulus of elasticity and modulus of deformability.

Osunade, et al. (1990) investigated the shear and tensile strength properties of laterised concrete under laboratory temperature of 20±1^{o}C. They observed that, as in normal concrete, the strength development of test specimens was more rapid at an early curing age than at later age. A higher percentage of the long-term shear and tensile strength of laterised concrete was significantly acquired at an early curing age.

Salau (2003) investigated long-term deformation of short columns of laterized concrete without taking into consideration the change in temperature and concluded that laterized concrete specimens experience more creep and shrinkage deformation when compared to their corresponding normal concrete specimens. A consistent pattern of creep-time curves in all cases of laterite content was obtained. The shrinkage-time curves were also observed to be consistent but different from the creep-time curves.

Ikponmwosa and Falade (2006) reported on the study of strength properties of fibre-reinforced laterised concrete under normal laboratory temperature. A consistent trend of increase in strength with age was observed in the specimens. A proportion of 45% laterite content as replacement of sharp sand in concrete produced the highest

compressive strength. At this laterite content, a reduction of 18% in the cost of fine aggregate in concrete was obtained at the prevailing market price. Although the strength characteristic of laterised concrete was found to be generally lower than that of normal concrete, it was sufficient for use in general concrete work. Concrete with 25% laterite content in the fine aggregate compared favourably with those of normal concrete of similar mix proportion by weight and water/cement ratio, and thus was evidently desirable for use in the determination of the effect of heat on laterised concrete.

Oluwaseyi and Mnse (2007) investigated the weathering characteristics of laterised concrete with laterite-granite fines ratio as a factor in ascertaining its suitability as a substitute for the conventional fine aggregate. They found that the compressive strength of laterised concrete with laterite-granite fines decreased when subjected to alternate wetting and drying. It was also observed that laterised concrete with 40-60% laterite-granite fines subjected to a temperature variation range of 75-125^{o}C attained compressive strength of 22.52 N/mm^{2}. However, the critical failure temperature of the laterised concrete was not ascertained.

2.2 REINFORCED LATECRETE AND CONCRETE MATERIALS

2.2.1. Cement

In broad terms, cement can be described as a material with adhesive and cohesive properties which make it capable of binding mineral fragments into a compact whole. However, in the construction industry, cement is restricted to a binding agent used with aggregates, bricks, building blocks, etc (Neville, 1995).

Cements can be classified as natural cements, Portland cements and high-alumina cements. However, this research work will only concentrate on the use of Portland cement type.

The principal requirement of all cement types is that the cement should be able to produce a strong, dense, durable Latecrete with definite setting and hardening characteristics.

Portland cement is made from a mixture of lime- and clay- bearing materials which are claimed to form a clinker consisting essentially of hydraulic calcium silicate. This clinker is then cooled and pulverized to the required fineness, with about 2 – 3 percent gypsum.

The main types of Portland cement are: Ordinary Portland Cement, Rapid hardening Portland, Extra rapid hardening Portland, White Portland, etc.

According to Mindess and Young (1981), the Portland cement is composed of four main oxides. The variation in the relative proportions of these four main oxides, the degree of fineness, with some little addition of some other oxides gives rise to the different types of cement.

The chemical symbol of the four main compounds, and cement-industry abbreviation are: Tricalcium Silicate (C_3S), Dicalcium Silicate (C_2S), Tricalcium Aluminate (C_3A) and TetraCalcium Aluminoferrite (C_4AF). It is therefore clear that the main chemical compounds of cement are Calcium silicates and Calcium aluminates. However, there are minor compounds such as magnesium oxide, tin oxide, manganese oxide, potassium oxide and sodium oxide (Mindess and Young, 1981).

Table 2.2: Main Constituents in a Typical Portland cement

Chemical Name	**Chemical Formula**	**Shorthand Notation**	**Percent by Weight**
Tricalcium Silicate	$3CaOxSiO_2$	C_3S	50
Dicalcium Silicate	$2CaOxSiO_2$	C_2S	25
Tricalcium Aluminate	$3CaOxAl_2O_3$	C_3A	12
Tetracalcium Aluminoferrite	$4CaOxAl_2O_3xFe_2O_3$	C_4AF	8
Gypsum	$CaSO_4xH_2O$	CSH_2	3.5

After: Mindess and Young (1981)

According to Mindess and Young (1981), when portland cement is mixed with water its chemical compound constituents undergo a series of chemical reactions that cause it to harden (or set). These chemical reactions all involve the addition of water to the basic chemical compounds listed in Table 2.2. This chemical reaction with water is called "hydration". Each one of these reactions occurs at a different time and rate. Together, the results of these reactions determine how portland cement hardens and gains strength.

- *Tricalcium silicate (C_3S)*. Hydrates and hardens rapidly and is largely responsible for initial set and early strength. Portland cements with higher percentages of C_3S will exhibit higher early strength.
- *Dicalcium silicate (C_2S)*. Hydrates and hardens slowly and is largely responsible for strength increases beyond one week.
- *Tricalcium aluminate (C_3A)*. Hydrates and hardens the quickest. Liberates a large amount of heat almost immediately and contributes somewhat to early strength. Gypsum is added to portland cement to retard C_3A

hydration. Without gypsum, C_3A hydration would cause portland cement to set almost immediately after adding water.

- *Tetracalcium aluminoferrite (C_4AF)*. Hydrates rapidly but contributes very little to strength. Its use allows lower kiln temperatures in portland cement manufacturing. Most portland cement color effects are due to C_4AF.

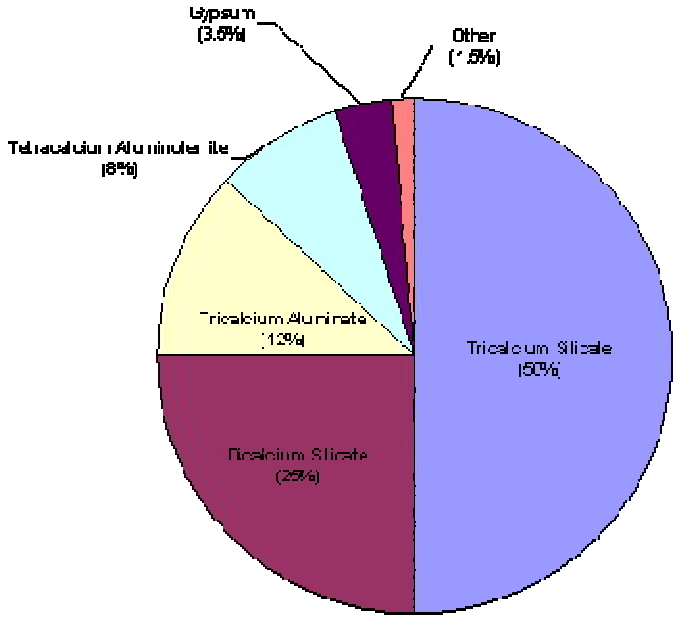


Figure 2.1: Typical Oxide Composition of a General-Purpose Portland cement (Mindess and Young, 1981)

The result of the two silicate hydrations is the formation of a calcium silicate hydrate (often written C-S-H because of its variable stoichiometry), which begins rapidly giving heat, and then slows down, and if water is present, continues for many years until hydration is complete and cement paste reaches its maximum strength. The higher the fineness, the greater the rate at which cement hydrates, the earlier strength is developed, and the more rapidly heat is generated.

The C-S-H makes up about 1/2 - 2/3 the volume of the hydrated paste (water + cement) and therefore dominates its behaviour (Mindess and Young, 1981).

(a). Physical Properties of Ordinary Portland Cement (OPC)

Portland cements are commonly characterized by their physical properties for quality control purposes, comparison and classification. Common tests carried out to ascertain these properties are: fineness, soundness, consistency, setting time, strength, heat of hydration and loss on ignition. However, they should not be substituted for tests on plain concrete cubes (Kosmatka, et al, 2003).

(i). Fineness

Fineness, or particle size of portland cement affects hydration rate and thus the rate of strength gain. The smaller the particle size, the greater the surface area-to-volume ratio, and thus, the more area available for water-cement interaction per unit volume. The effects of greater fineness on strength are generally seen during the first seven days (Kosmatka, et al, 2003).

(ii). Soundness

"Soundness" of Portland cement refers to the ability of a hardened cement paste to retain its volume after setting without delayed destructive expansion (Kosmatka, et al, 2003). This destructive expansion is caused by excessive amounts of free lime (CaO) or magnesia (MgO). Most portland cement specifications limit magnesia content and expansion. The typical expansion test places a small sample of cement paste into an autoclave (a high pressure steam vessel). The autoclave is slowly brought to 2.03 MPa (295 psi) then kept at that pressure for 3 hours. The autoclave is then slowly brought back to room temperature and atmospheric pressure. The change in specimen length due to its time in the autoclave is measured and reported as a percentage. ASTM C 150, *Standard Specification for Portland Cement* specifies a maximum autoclave expansion of 0.80 percent for all portland cement types.

(iii). Setting Time

According to Kosmatka, et al, (2003) Cement paste setting time is affected by a number of items including: cement fineness, water-cement ratio, chemical content (especially gypsum content) and admixtures. Setting tests are used to characterize how a particular cement paste sets. For construction purposes, the initial set must not be too soon and the final set must not be too late. Additionally, setting times can give some indication of whether or not cement is undergoing normal hydration.

Normally, two setting times are defined (Mindess and Young, 1981):

1. *Initial set*: Occurs when the paste begins to stiffen considerably.
2. *Final set*: Occurs when the cement has hardened to the point at which it can sustain some load.

These particular times are just arbitrary points used to characterize cement; they do not have any fundamental chemical significance.

(iv). Strength

Cement paste strength is typically defined in three ways: compressive, tensile and flexural. These strengths can be affected by a number of items including: water-cement ratio, cement-fine aggregate ratio, type and grading of fine aggregate, manner of mixing and molding specimens, curing conditions, size and shape of specimen, moisture content at time of test, loading conditions and age (Mindess and Young, 1981). Since cement gains strength over time, the time at which strength test is to be conducted must be specified. Typical times are 1 day (for high early strength cement), 3 days, 7 days, 28 days and 90 days (for low heat of hydration cement). When considering cement paste strength tests, there are two items to consider:

- Cement mortar strength is not directly related to concrete strength. Cement paste strength is typically used as a quality control measure.

- Strength tests are done on cement mortars (cement + water + sand) and not on cement pastes.

(a). Compressive Strength

The most common strength test for cement is the compressive strength test. According to BS 4550 (1978) this test is carried out on a 70.6 mm cement mortar test specimen. The test specimen is subjected to a compressive load (usually from a hydraulic machine) until failure. This loading sequence must take no less than 20 seconds and no more than 80 seconds.

(b). Tensile Strength

Although the tensile strength is still specified by ASTM code, the direct tension test does not provide any useful insight into the concrete-making properties of cements. It persists as a specified test because in the early years of cement manufacture, it used to be the most common test since it was difficult to find machines that could compress a cement sample to failure.

(c). Flexural Strength

Flexural strength (actually a measure of tensile strength in bending) is carried out on a 40 x 40 x 160 mm (1.57-inch x 1.57-inch x 6.30-inch) cement mortar beam. The beam is then loaded at its center point until failure.

(v). Specific Gravity Test

According to Kosmatka, et al, (2003), Specific gravity is normally used in mixture proportioning calculations. The specific gravity of portland cement is generally around 3.15 while the specific gravity of portland-blast-furnace-slag and portland-pozzolan cements may have specific gravities near 2.90.

(vi). Heat of Hydration

The heat of hydration is the heat generated when water and portland cement react. Heat of hydration is most influenced by the proportion of C_3S and C_3A in the cement, but is also influenced by the water-cement ratio, finess and curing temperature. As each one of these factors is increased, heat of hydration increases. In large mass concrete structures such as gravity dams, hydration heat is produced significantly faster than it can be dissipated (especially in the center of large concrete masses), which can create high temperatures in the center of these large concrete masses that, in turn, may cause undesirable stresses as the concrete cools to ambient temperature. Conversely, the heat of hydration can help maintain favourable curing temperatures during winter (Kosmatka, et al, 2003).

(vii). Loss on Ignition

Loss on ignition is calculated by heating up a cement sample to 900 – 1000^{o}C (1650 - 1830°F) until a constant weight is obtained. The weight loss of the sample due to heating is then determined. A high loss on ignition can indicate prehydration and carbonation, which may be caused by improper and prolonged storage or adulteration during transport or transfer (Kosmatka, et al, 2003).

The followings are the specifications for normal Ordinary Portland Cement according to the Nigerian Industrial Standards, NIS 11, 1974, as quoted in Ejeh (1982):

FINENESS:- The ordinary Portland cement should have an average specific surface of not less than 2500cm^2/g.

SETTING TIME;-

(a). Initial setting time should not be less than 45 minutes.

(b). Final setting time should be less than or equals to 10 hours.

SOUNDNESS:- Should not have an expansion more than 10 mm and if aerated less than or equal to 5 mm.

LIME SATURATION FACTOR (LSF):- Shall not be greater than 1.02 and not less than 0.66.

2.2.2 Aggregates

An aggregate is defined as a building material that is cohesively connected into a whole by means of cement paste similar to masonry construction (Neville, 1995). It originates from fragmented or artificially crushed naturally weathered and ablated larger parent mass which determines the specific gravity, chemical and mineral composition which itself influences the latecrete and concrete quality. However, properties like shape, size, surface, texture and absorption are not determined by the parent rock.

According to Hudson (1999), aggregates represents approximately 80 percent of the total volume of concrete; therefore, aggregate characteristics significantly affect the performance of fresh and hardened concrete and have an impact on the cost effectiveness of concrete. Aggregate characteristics of shape, texture, and grading influence workability, finishability, bleeding, pumpability, and segregation of fresh concrete; and affect strength, stiffness, shrinkage, creep, density, permeability, and durability of hardened concrete.

Construction and durability problems have been reported due to poor mixture proportioning and variation on grading (Lafrenz, 1997). Except for water, aggregate is the most inexpensive component of Portland cement concrete. Conversely, cement is the most expensive component and, typically, is responsible for about 60 percent of the total cost of materials. Paste, cement plus water, is the part of concrete that produces

shrinkage, heat generation, and durability problems although, at the same time, is the element that fills aggregate voids, the glue that keeps aggregates together after hardening and the element that provides workability to the mix in fresh concrete. If aggregate voids are minimized, the amount of paste required for filling these voids is also minimized maintaining workability and strength. Consequently, optimal mixture proportioning will produce good-quality concrete with a minimum amount of cement. Within limits, the less paste at a constant water-cement ratio, the more durable the concrete (Shilstone, 1994).

De Larrard, (1999) and Dewar (1999), reported that the workability of concrete changes significantly with grading. They showed that there is a clear relationship between shape, texture, and grading of aggregates and the voids content of aggregates. They concluded that flaky, elongated, angular, and unfavorably graded particles lead to higher voids content than, cubical, rounded, and, well-graded particles.

In general, aggregates for construction should be hard, durable and clean, and should contain only limited amount of flaky and elongated particles, adherent coating or harmful materials that may affect the strength and durability of latecrete/concrete.

(a). Classification of aggregates

The following classification is made according to common practice: material retained in the No. 4 sieve is considered coarse aggregate, material passing No. 4 sieve and retained in the No. 200 sieve (75 μm) is considered fine aggregate, and material passing No. 200 sieve is called microfines.

(b). Effect of aggregate characteristics on the performance of portland cement concrete

The main characteristics of aggregate that affect the performance of fresh and a hardened concrete are: Shape and texture, grading, absorption, mineralogy and coatings,

strength and stiffness, maximum size, specific gravity or relative density, soundness, and toughness.

(i). Shape

Shape is related to three different characteristics: sphericity, form, and roundness (Galloway, 1994). Sphericity is a measure of how nearly equal are the three principal axes or dimensions of a particle. Form is the measure of the relation between the three dimensions of a particle based on ratios between the proportions of the long, medium, and short axes of the particle. Form, also called "shape factor," is used to distinguish between particles that have the same numerical sphericity (Hudson, 1997). However, different definitions exist that do not necessarily correlate. Besides sphericity and shape factor, two more parameters have been defined in order to describe the shape of aggregates better: the elongation factor and the flatness factor.

Regarding sphericity and form, particles can be classified qualitatively as cubical, spherical, or flat and elongated.

According to Kwan, et al (1999), there are two other characteristics which he termed roundness and angularity. Angularity is related to the sharpness of the edges and corners of a particle, while roundness attempts to describe the outline of the particle, which may be measured in terms of "convexity." Angularity can be defined numerically as the ratio of the average radius of curvature of the corners and edges of the particle to the radius of the maximum inscribed circle, but descriptive terms are commonly used, as the followings used in the United States of America. (Popovics, 1998).

Angular : little evidence of wear on the particle surface

Subangular : evidence of some wear, but faces untouched

Subrounded : considerable wear, faces reduced in area

Rounded : faces almost gone

Well rounded : no original faces left

These shape and form definitions can be used with data acquired by measuring manually the three principal dimensions of a number of particles to characterize aggregates.

(ii). Texture

The surface texture, also called surface roughness, of particles is the sum of their minute surface features (Popovics, 1998). It is an inherent and specific property that depends on the texture, the structure, and the degree of weathering of the parental rock.

For Masad (2002), texture in an image is represented by the local variation in the pixel gray intensity values. Then wavelet theory is used for multi-scale analysis of textural variation on aggregate images. The original image is decomposed into low-resolution images by iteratively blurring the original image. As a result, images that contain information on fine intensity variation are obtained. The process could be repeated again with these images and texture quantification can be made at different scales. In this way, values for the coarser and finer texture of the sample can be obtained.

(iii). Effect of Grading

Grading or particle size distribution affects significantly some characteristics of concrete like packing density, voids content, and, consequently, workability, segregation, durability and some other characteristics of concrete.

Particle size distribution of fine aggregate plays a very important role on workability, segregation, and pumpability of fresh concrete. Many authors claim that uniformly distributed mixtures produce better workability than gap-graded mixtures

(Golterman, 1997; Glavind, 1993; Johansen, 1989; Johansson, 1979), although higher slumps could be achieved with gap-graded mixtures. Some properties of hardened concrete are also affected by grading. Uniformly distributed mixtures generally lead to higher packing resulting in concrete with higher density and less permeability (Golterman, 1997; Glavind, 1993; Johansen, 1989), and improved abrasion resistance (Mehta and Monteiro, 1993). Consequently, uniformly distributed mixtures require less paste, thus decreasing bleeding, creep, and shrinkage (Shilstone, 1994; Washa, 1998). Finally, Aitcin (1998) emphasizes that although an excess of coarse aggregate could decrease drying shrinkage it will increase the amount of microcracks within the paste.

The scarcity or excess of any size fraction could result in poor workability and in poor durability of concrete (Galloway, 1994; Shilstone, 1990). The amounts of coarse and fine aggregate must be in balance. For example, excess sand requires more cementitious materials, produces sticky mixtures, makes pumping difficult, causes finishing and crazing problems (Shilstone, 1994), and increases bleeding and permeability (Mindess, 1981). On the other hand, insufficient sand produces "bony" mixtures and other types of finishing problems (Shilstone, 1990; Mindess, 1981). Both coarse aggregate and fine aggregate should be uniformly graded. If fine aggregate is too coarse it will produce bleeding, segregation and harshness, but if it is too fine, the demand for water will be increased (Galloway, 1994).

Proper grading should depend on shape and texture of aggregates. For instance, suitable grading for natural sands could lead to bad results when using manufactured sands (Hudson, 1999; Johansson, 1979). Grading should also be changed depending on the construction procedures. For example, pumpable concrete requires a high fine aggregate content, just as hand finishing requires more fines passing the No. 50 sieve than mechanical finishing does (Galloway, 1994).

The effect of grading on strength is controversial. Although, according to some authors, a given strength can be achieved with both well-graded mixtures and poorly graded mixtures (Shilstone, 1990), some studies indicate that increased strength concrete can be achieved with well-graded mixtures (Cramer,et al, 1995).

Permeability, one of the most important factors affecting concrete durability, is significantly related to void content of aggregate mixture: the lesser the void content, the lesser the permeability. In reducing permeability, it is desirable to have the highest aggregate content possible. Consequently, well-graded mixtures produce concrete that is more durable.

The particle size distribution of microfines may have an effect on concrete behaviour. However, due to the inherent difficulties related to the characterization of such small particles, little research has been made to evaluate the effect of grading, shape and texture of microfines on concrete behavior. Ahn (2000) determined the size distribution of the microfines in his research using a laser diffraction scanner and a hydrometer, in terms of weight and in terms of volume, respectively. It was found that the effect of different types of microfines with distinct grading on fresh concrete behavior was clearly dissimilar. However, the effect of grading was not separated from the effect of shape and texture, and was not quantitatively determined.

(iv). Effect of Absorption

Aggregate porosity may affect durability as freezing of water in pores in aggregate particles can cause surface popouts (Popovics, 1998; Helmuth, 1994). However, Forster (1994) states "that relationship between absorption and freeze-thaw behaviour has not proven to be reliable." Nevertheless, absorption can be used as an initial indicator of soundness. Furthermore, aggregates with low absorption tend to reduce shrinkage and creep (Washa, 1998).

(v). Effect of Mineralogy and Coatings

Aggregates with varying mineralogical composition produce concretes of diverse characteristics. According to Alexander (1996), the type of aggregate affects the strength and the stiffness and long-term deformations of hardened concrete. For example, some aggregates may react negatively with cement or, on the contrary, they may interact beneficially with paste, enhancing strength or stiffness. As a result, elastic modulus, creep, or shrinkage can vary as much as 100 percent depending on aggregate type. The type of aggregate also affects the interfacial transition zone, ITZ, which has an effect on strength and stiffness of concrete. Finally, the type of aggregate influences the abrasion resistance of concrete, particularly of high-strength concrete. Coatings, the layers of material covering the surface of aggregate, can increase the demand for water and can impair the bond between paste and particles. Sometimes these coatings are formed by materials that can interact chemically with cement, negatively affecting concrete (Mather, 1966).

(vi). Effect of Strength and Stiffness

The strength and the stiffness of coarse aggregate directly influence the behavior of hardened concrete: Although in normal concrete, strength is controlled by the paste or by the transition zone between paste and aggregate, the strength of high-performance concrete depends not only on the strength but on the mineralogy of coarse aggregate as well (Alexander, 1989; Cetin, 1998; Ezelding and Aitcin, 1991). At the same time, according to Kaplan (1959), aside from texture and shape the modulus of elasticity is the aggregate property that most affects compressive and flexural strengths of concrete. According to Popovics (1979) “within limits, the higher the elastic modulus of aggregate, the lower the flexural strength.” In addition, the elastic modulus of aggregate, as well as its volume concentration, affects the elastic modulus of

concrete and the long-term effects, shrinkage and creep (Alexander, 1989; Mehta, 1993; Washa, 1998). De Larrard (1999) and Dewar (1999) agree that the aggregate source has an impact on concrete strength. Dewar suggests multiplying a basic concrete strength by a factor that depends on the aggregate source and varies from 0.90 to 1.85.

(vii). Effect of Maximum Size

Maximum size of aggregate, MSA, influences workability, strength, shrinkage, and permeability. Mixtures with large maximum size of coarse aggregate tend to produce concrete with better workability, probably because of the decrease in specific surface (Washa, 1998). There is an optimal maximum size of coarse aggregate that produces the highest strength for a given consistency and cement content (Popovics, 1998; Washa, 1998). For example, in high-performance concrete (HPC) with low water-cement ratio and high cement content, a high value of MSA tends to reduce strength. This can be explained by the observation that bond with large particles tends to be weaker than with small particles due to smaller surface area-to-volume ratios. Mixtures with coarse aggregate with large maximum size tend to have reduced shrinkage and creep (Washa, 1998). Finally, for a given water-cement ratio, the permeability increases as the maximum size of the aggregate increases (Helmuth, 1994).

(viii). Effect of Relative Density (Specific Gravity)

Relative density is not necessarily related to aggregate behaviour. However, it has been found that some aggregates compounds of shale, sandstone, and chert that have somewhat low specific gravity may display poor performance, particularly in exposed concrete in northern climates (Legg, 1998) (i.e., low permeability is an indicator of poor durability).

(ix). Effect of Soundness and Toughness

Soundness, according to Forster (1994), is the aggregate resistance to weathering that primarily includes resistance to freezing and thawing, and to a lesser extent, resistance to wetting and drying; and heating and cooling. A sound aggregate has a satisfactory durability factor when used in properly mature concrete with enough air void content. Mather (1999)

Toughness of aggregate affects abrasion resistance of concrete. Hard aggregates should be used in pavements and elements subject to abrasion or erosion. The Los Angeles abrasion test is commonly used to assess toughness and resistance to abrasion of aggregate. However, as the Los Angeles test does not correlate well with concrete wear in the field, the resistance to abrasion of concrete should also be tested (Mindess, 1981).

2.2.3 Laterite

Laterite is the most commonly stabilized soil for construction. They are widely distributed throughout the tropical and sub-tropical regions of Africa, South America, Central America, Australia, India and South East Asia. (Adepegba, 1975a)

Many conflicting definitions of laterite have been prepared in technical literature. Nanda (1993) reports that some people have defined laterite from its colour, others on the basis of chemical composition and some others from its morphology.

According to Hamilton (1995), Buchanon (1807), who first suggested the name laterite defined it as the ability of a soft red material to harden on exposure to air. The limitation of this definition is in the fact that the term, laterite, became synonymous with the tropical weathering product of virtually all igneous rocks and was applied to red soils whether hardening was involved or not.

According to Nixon and Skipp cited in Nanda (1983), laterite refers to those ferruginous and alumino-ferruginous materials having vesicular concretionary appearance which cap a soil profile of either insitu development over a parent rock or similar material; its colour range from ochre through red, brown or violet.

Based on morphology, Pendleton and Sharosuvana (1946) defined laterite soils as profiles in which a laterite horizon is found, and lateritic soils as profiles in which immature laterite horizon are found which develop under appropriate conditions.

One point agreed by all the authors in their definitions of laterite is its association with the red colour derived as a result of the iron oxide present in it.

However, for the purpose of this study, the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) Progress Report (1982/85) definition is adopted. It states that "a soil can be considered lateritic if it belongs to horizon A or B of well drained profiles developed under humid tropical climate, its clay fraction are constituted essentially of the kaolinite group and of iron or aluminium hydrated oxides and these components are assembled in peculiar porous and highly stable aggregate structure".

In a study, Gidigasu (1976) reports that the geotechnical characteristics (and also mechanical properties) of laterites are influenced considerably by the genesis, degree of weathering, morphology, chemical and mineralogical composition and environmental factors. He further reported that laboratory tests carried out on some of the soils using different pre-test results preparation and listing procedures lead to essentially the same result. Normal laterite soil may thus be identified and classified on the basis of standard soil classification properties (particle size analysis and atterberg limit test). The Unified Soil Classification System (USCS), and the American

Association of State Highway and Transportation Officials (AASHTO) Specifications could be used.

According to Lasisi and Osunade (1985) the majority of houses in rural areas of Nigeria and other third world countries are built with laterite. Kamang (1998) reported that this practice continues to increase inspite of the obvious limitations in the durability, acceptability and strength. He further stressed that because of these limitations in the use of laterite, in construction activities, recent research efforts have geared towards improving their engineering properties for both road and low cost houses. These efforts are aimed at upgrading the traditional technology of laterite in construction through enhanced strength, durability and other performance characteristics.

2.2.4 Water

Water initiates the chemical reactions that produce the binding qualities of cement, without it, latecrete is impossible to make.

According to Kosmatka, et al, (2003), the presence of water in a mix does not only account for proper mixing but also helps to lubricate the aggregates to facilitate their placement. Therefore, for a good and durable latecrete, there must be adequate quantity of water for the process of hydration to go to completion.

Water obtained from different sources could be used for latecrete production; however, its quality must not be questionable. The rule of thumb recommended by the Kosmatka, et al, (2003) is that: if water is pure enough for drinking, it is suitable for mixing latecrete. The use of impure water has the effect of reducing the quality of the latecrete below minimum acceptable standards.

2.2.5 Reinforcement

Reinforcement has been in use in reinfroced concrete structures since time immemorial. This is because, concrete has a high compressive strength, and a very limited tensile strength. To overcome the tensile strength limitation of concrete, reinforcing bars are used in the tension side of concrete structures. Steel reinforceement bars have been and are an effective and cost-efficient concrete reinforcement.

When properly protected from adverse conditions, such as ion attack as in marine areas, steel reinforcement can last for decades without exhibiting any visible signs of deterioration. However, Insufficient concrete cover, poor design or workmanship, poor concrete mix, and presence of large amounts of aggressive agents all can lead to corrosion of the steel rebar and cracking of the concrete.

2.3 MECHANICAL PROPERTIES OF LATECRETE AND CONCRETE

2.3.1 Compressive strength

The compressive strength of concrete is the maximum compressive load it can carry per unit area. Concrete structures, except for road pavements, are normally designed on the basis that concrete is capable of resisting only compression, the tension being carried by steel reinforcement. Cube specimens of size 150 mm is often used to determine the compressive strength. BS 1881: part 3 (1983) recommended that the test specimen for compressive strength should be cured in water at a temperature of $20\pm^{o}C$ and crushed immediately it is removed from water.

2.3.2 Tensile strength

The tensile strength of concrete is important in the design of concrete roads and runways; as is flexural strength or modulus of rupture, is utilised in distributing the concentrated loads over a wider area of road pavement. The tensile strength of concrete is difficult to measure directly and thus the use of indirect testing method is usually employed in accordance with BS 1881: part 4 (1983).

Cylinder specimens of size 150 mm diameter and 300mm length are produced for the indirect tensile strength test, which uses the splitting test method. Although, concrete is not normally designed to resist tension, knowledge of its tensile strength is important in assessing the load under which cracking will develop.

A cylinder specimen is placed with its axis horizontal between the platens of the testing machine; the load is applied, and increased to failure.

According to BS EN 12390: part 6 (2003), the tensile splitting strength is given by the formula:

The tensile splitting strength is given by the formula:

$$f_{ct} = \frac{2F}{\pi ld} \qquad 2.1$$

where

f_{ct} is the tensile splitting strength, in megapascals or newtons per square millimetre;

F is the maximum load, in newtons;

l is the length of the line of contact of the specimen, in millimetres;

d is the designated cross-sectional dimension, in millimetres.

2.3.3 Flexural strength

Flexural strength is also known as modulus of rupture, bend strength, or fracture strength. This is a mechanical parameter for brittle material and is defined as a material's ability to resist deformation under load.

According to Hodgkinson (2000) and Callister (2003), the transverse bending test is most frequently employed in measuring the flexural strength of members. In flexural or bending strength test, a beam or rod specimen having either a circular or rectangular cross-section is bent until fracture/failure using either a centre point, three point, or four point flexural test technique. The flexural strength represents the highest stress experienced within the material at its moment of rupture. It is measured in terms of stress, here given the symbol σ.

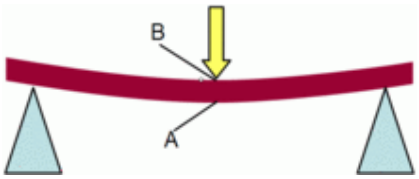


Figure 2.2: Beam of material under bending
*After Callister (2003)

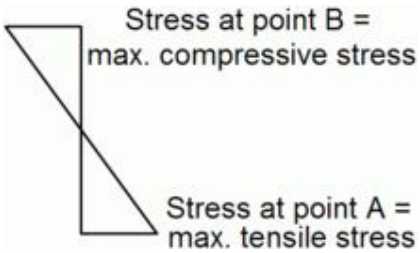


Figure 2.3: Stress distribution across beam
- After Callister (2003)

In figure 2.2 above, the extreme fibers at B are in compression and A in tension.

Flexural versus tensile strength

The flexural strength would be the same as the tensile strength if the material was homogenous. In fact, most materials have small or large defects in them which act to concentrate the stresses locally, effectively causing a localized weakness. When a material is bent only the extreme fibers are at the largest stress so, if those fibers are free from defects, the flexural strength will be controlled by the strength of those intact 'fibers'. However, if the same material was subjected to only tensile forces then all the fibers in the material are at the same stress and failure will initiate when the weakest fiber reaches its limiting tensile stress. Therefore it is common for flexural strengths to be higher than tensile strengths for the same material. Conversely, a homogeneous material with defects only on its surfaces (e.g. due to scratches) might have a higher tensile strength than flexural strength.

If we do not take into account defects of any kind, it is clear that the material will fail under a bending force which is smaller than the corresponding tensile force. Both of these forces will induce the same failure stress, whose value depends on the strength of the material.

For a rectangular sample, the resulting stress under a transverse force is given by the following formula :

$$\sigma = \frac{F}{bd} \qquad 2.2$$

Where:

- F is the transverse load (force) at the fracture point (N)
- *b* is width (mm)
- *d* is thickness (mm)

This stress is not the true stress, since the cross section of the sample is considered to be variable (engineering stress).

Measuring flexural strength

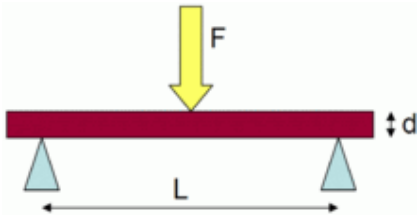


Figure 2.4: Beam under three point bending

- After Callister (2003)

For a rectangular sample under a load in a three-point bending setup (figure 2.4), the flexural strength (modulus of rupture), according to BS EN 12390: part 5 (2003), is given by:

$$\sigma = \frac{3FL}{2bd^2} \qquad 2.3$$

In equations 2.2 and 2.3 above,

- F is the load (force) at the fracture point (N)
- L is the length of the support span (mm)
- b is width (mm)
- d is thickness (mm)

CHAPTER THREE

MATERIALS AND METHODS

This chapter involves mainly the testing of materials, and the strength tests of latecrete and concrete specimens. To produce the latecrete and concrete specimens, the following materials must be used: cement, sand, laterite, coarse aggregate, reinforcement and water. Thus the following materials were subjected to test in accordance with standard: (a). cement, (b). sand, (c). laterite, (d). coarse aggregate - (12 mm), (e). reinforcement - (Y12 and R6), and (f). water.

3.1 CEMENT

The cement used in this research work is ordinary Portland cement with local name Dangote cement, produced at Gboko in Benue state of Nigeria.

The cement was bought from Samaru market and subjected to the following tests: fineness, consistency, initial setting time, final setting time, soundness and mortar cube strength.

(a). Fineness

The fineness test was carried out in accordance to BS 4550 (1978) clause 3.3 and the result is presented in table 3.1 below.

Table 3.1 Fineness of cement

S/NO.	SAMPLE	FINENESS
1	F-01	0.03
2	F-02	0.04
3	F-03	0.07
Average = 0.05		

(See details in Appendix A1)

(b). Consistency

The consistency of the cement was carried out in accordance to BS 4550 (1978) clause 3.5 and the result is presented in table 3.2 below:

Table 3.2 Consistency of cement

S/NO.	SAMPLE	CONSISTENCY (%)
1	C-01	31
2	C-02	31
3	C-03	31
Average = 31 %		

(See details in Appendix A2)

(c). Initial setting time

The initial setting time of the cement was carried out in accordance to BS 4550 (1978) clause 3.6 and the result is presented in table 3.3 below:

Table 3.3 Initial setting time of cement

S/NO.	SAMPLE	INITIAL SETTING TIME (MINUTES)
1	ST-01	80
2	ST-02	84
3	ST-03	76
Average = 80 minutes (1 hour 20 minutes)		

(See details in Appendix A3)

(d). Final setting time

The final setting time of the cement was carried out in accordance to BS 4550 (1978) clause 3.6 and the result is presented in table 3.4 below.

Table 3.4 Final setting time of cement

S/NO.	SAMPLE	FINAL SETTING TIME (minutes)
1	ST-01	175
2	ST-02	160
3	ST-03	175
Average = 170 minutes (2 hours 50 minutes)		

(See details in Appendix A3)

(e). Soundness

The soundness of the cement was carried out in accordance to BS 4550 (1978) clause 3.7 and the result is presented in table 3.5 below:

Table 3.5 Soundness of Cement

S/NO.	SAMPLE	SOUNDNESS (mm)
1	S-01	1.0
2	S-02	1.0
3	S-03	1.0
Average = 1.0 mm		

(See details in Appendix A4)

(f). Mortar Cube Strength

The Mortar cube strength of the cement was carried out in accordance to BS 4550 (1978) clause 3.4 and the result is presented in table 3.6 below:

Table 3.6 Mortar Cube Strength of Cement

S/NO.	SAMPLE	7 DAYS MORTAR CUBE STRENGTH (N/mm^2)
1	MC-01	24.08
2	MC-02	20.46
3	MC-03	20.06
Average = 21.53 N/mm^2		

(See details in Appendix A5)

3.2 SAND

The sand used in this research work was bought from a supplier at Samaru and subjected to the following tests: particle size distribution, specific gravity and silt content.

(a). Particle size distribution

The particle size distribution was carried out in accordance to BS 812 (1989) Clause 101 using the pycnometer instrument and the result is presented in table 3.7 below.

Table 3.7 Particle size distribution of Sand

BS Sieve size (mm)	Cummulative % passing
4.76	100
2.36	96
1.18	74
0.6	35.50
0.425	2.00
0.3	0.50
0.212	0.00
0.15	0.00
0.075	0.00

(See details in Appendix A6)

(b). Specific Gravity of Sand

The Specific gravity of sand was determined in accordance to BS 812-102 (1989) and the result is presented in table 3.8 below.

Table 3.8 Specific Gravity of Sand

SAMPLE	SPECIFIC GRAVITY
SG-01	2.60
SG-02	2.56
SG-03	2.58
Average = 2.58	

(See details in Appendix A7)

(c). Silt content

The silt content was determined in accordance to BS 812-102 (1989) and the result is presented in table 3.9 below.

Table 3.9 Silt Content of Sand

SAMPLE	SILT CONTENT (%)
SC-01	2.50
SC-02	2.20
SC-03	2.00
Average = 2.23%	

(See details in Appendix A8)

3.3 LATERITE

The laterite used in this research work was dug from a borrow pit located at the Shika area of Zaria (latitude 11^{o} 15´N and longitude 7^{o}45´E) Nigeria.

A previous study by Osinubi (1998) on the soil from this area established that it contain kaolinite as the dominant clay mineral. The soil is classified as A-7-6 according to AASHTO soil classification system (AASHTO, 1986) and low plasticity clay (CL) according to the Unified soil classification system (ASTM, 1992). The chemical composition of the lateritic soil is shown in the table 3.10 below.

Table 3.10 Chemical Composition of Lateritic Soil

Oxide	Concentration (%) by weight
CaO	0.28
SiO_2	35.60
Al_2O_3	27.40
Fe_2O_3	24.0
MgO	0.22
Na_2O+K_2O	-
SO_3	0.85
Mn_2O_3	2.0
P_2O_5	-
Loss on ignition	1.46

After Osinubi, (1998)

The index properties of the laterite were determined in accordance with BS 1377(1990). The test methods and test results for determining the lateritic soil are summarized below:

a). Particle size distribution

The particle size distribution was carried out using the combined sieve and hydrometer analysis in accordance to BS 1377 (1990) part 1 and presented in table 3.11 below.

Table 3.11 Particle size distribution of Laterite

BS Sieve size (mm)	Cummulative % passing
4.76	100
2.36	97.02
1.18	95.65
0.6	93.02
0.425	79.09
0.3	73.04
0.212	66.41
0.15	61.81
0.075	56.71

(See details in Appendix A9)

(b). Specific Gravity of Laterite

The Specific gravity was determined in accordance to BS 812-102 (1989) and the result is presented in table 3.12 below:

Table 3.12 Specific Gravity of Laterite

SAMPLE	SPECIFIC GRAVITY
SG-04	2.56
SG-05	2.63
Average = 2.60	

(See details in Appendix A10)

(c). Index and classification properties of Laterite

i). Atterberg limits and indices

The Atterberg limits and indices were obtained as follows:

- The liquid limit (LL) of the lateritic soil was determined using the Casagrande apparatus and a grooving tool in accordance with BS1377: Part 2:1990 clause 4.5.

- The plastic limit of the lateritic soil was determined by rolling the soil sample into threads on a glass plate until about 3 mm thick began to crumble, in accordance to BS1377: Part 2:1990 clause 5.3.

ii). The linear shrinkage (LS) was determined in accordance to BS1377: Part 2:1990 clause 6.3 using the LS device, which is a metal mould measuring 140x10x10-mm. In conducting the test, the mould was filled with soil having a moisture content equivalent to LL and was then placed in an oven at 105^{o}C for a period of 24 hours until no further shrinkage occurred.

The results of the index and classification properties of the Shika lateritic soil performed are as described above and the results are presented in table 3.13 below.

Table 3.13 Physical properties of the lateritic soil

Property	Laterite
Colour	Reddish Brown
Liquid Limit, %	42
Plastic Limit, %	18
Plasticity Index, %	24
Linear shrinkage, %	6
Percent Passing BS No. 200 sieve	56.71
AASHTO classification	A-7-6(10)
USCS Classification	CL

(See details in Appendix A11 and A12)

3.4 COARSE AGGREGATES

The coarse aggregates used in this research work was bought from quarry site opposite the Nigerian College of Aviation Technology, Zaria, and subjected to the following tests: particle size distribution, specific gravity, flakiness, elongation,

impact value, and crushing value. The nominal size of coarse aggregates is twelve millimeters (12 mm).

(a). Particle size distribution

The particle size distribution was carried out in accordance to BS 812 (1989) clause 102 and the result of the sieve analysis is presented in table 3.14 below.

Table 3.14 Particle size distribution of Coarse aggregates

BS Sieve size (mm)	% passing
31.50	100
19.05	96.0
9.52	54.0
6.35	26.0
4.76	16.0
2.36	9.5
1.18	8.3
0.6	0.0
0.425	0.0
0.3	0.0
0.212	0.0
0.15	0.0
0.075	0.0

(See details in Appendix A13)

(b). Specific Gravity of Coarse aggregates

The Specific gravity was carried out in accordance to BS 812-102 (1989) and the result is presented in table 3.15 below.

Table 3.15 Specific Gravity of Coarse aggregates

SAMPLE	SPECIFIC GRAVITY
SG-07	2.58
SG-08	2.62
SG-09	2.60
Average = 2.60	

(See details in Appendix A14)

(b). Flakiness Index

Samples of the coarse aggregates were taken in accordance with BS 812 (1989) clause 102 and tested for flakiness index. The Flakiness Index was carried out in accordance to BS 812 (1989) clause 105.1 and the result is presented in table 3.16 below.

Table 3.16 Flakiness Index of Coarse aggregates

SAMPLE	FLAKINESS INDEX (%)
FI-01	14.31
FI-02	13.93
Average = 14.12%	

(See details in Appendix A15)

(c). Elongation Index

Samples of the coarse aggregates were taken in accordance with BS 812 (1989) clause 102 and tested for elongation index. The Elongation Index was carried out in accordance to BS 812 (1989) clause 105.2 and the result is presented in table 3.17 below.

Table 3.17 Elongation Index of Coarse aggregates

SAMPLE	ELONGATION INDEX (%)
EI-01	14.78
EI-02	14.61
Average = 14.70%	

(See details in Appendix 16)

(d). Aggregate Impact Value (AIV)

The aggregate impact value test on the coarse aggregates was carried out on coarse aggregate sample in accordance to BS 812 (1990) clause 113 and the result is presented in table 3.18 below.

Table 3.18 Aggregate Impact Value of Coarse aggregates

SAMPLE	AIV (%)
IV-01	40.8
IV-02	43.0
IV-03	41.9
Average = 41.9%	

(See details in Appendix A17)

(e). Aggregate Crushing Value (ACV)

The crushing value test on the coarse aggregates was carried out in accordance to BS 812 (1990) clause 110 and the result is presented in table 3.19 below.

Table 3.19 Aggregate Crushing Value of Coarse aggregates

SAMPLE	ACV (%)
CV-01	18.7
CV-02	19.2
CV-03	19.0
Average = 19.0%	

(See details in Appendix A18)

3.5 REINFORCEMENT

Reinforcement bars (6mm mild steel and 12mm high yield) and binding wire were bought from the PZ area of Zaria. The reinforcement bars are subjected to elongation and tension test in accordance to BS 4449 (1997) Clause 10.1.2 and 10.1.3

The vernier caliper was used to measure the diameters of the bars and a steel tape used to measure their lengths. The bars are of high yield (12 mm diameter) and mild steel (6 mm diameter). The actual determination of the properties of the bars was done by testing the specimens on a Universal testing machine.

(a). Elongation test

The elongation test was carried out in accordance to BS 4449 (1997) Clause 10.1.2 and 10.1.3. The length of each bar was 300 mm. The results are as show in tables 3.20 and 3.21 below.

Table 3.20 Elongation test of 12 mm diameter high yield (Y12) steel bars

S/NO.	Identification Number	Nominal diameter measured (mm)	Elongation (percent)
1	ET/Y12/01	11.78	15.00
2	ET/Y12/02	11.78	16.67
3	ET/Y12/03	11.78	16.67
4	ET/Y12/04	11.78	16.67
5	ET/Y12/05	11.78	15.00

(See data in appendix A19)

Table 3.21 Elongation test of 6mm diameter mild (R6) steel bars

S/NO.	Identification Number	Nominal diameter measured (mm)	Elongation (percent)
1	ET/R6/01	5.57	18.33
2	ET/R6/02	5.57	20.00
3	ET/R6/03	5.57	20.00
4	ET/R6/04	5.57	18.67
5	ET/R6/05	5.57	20.00

(See data in appendix A20)

(b). Tension test

The tension test was carried out in accordance to BS 4449 (1997) Clause 10.1.2 and 10.1.3. The length of each bar was 500 mm. The results are as show in tables 3.22 and 3.23 below.

Table 3.22 Tension test of 12mm diameter high yield (Y12) steel bars

S/NO.	Identification Number	Size (mm)	Nominal Diameter measured (mm)	Cross Sectional Area (mm^2)	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
1	TT/Y12/01	12	11.78	108.99	412.88	614.74
2	TT/Y12/02	12	11.78	108.99	403.71	587.21
3	TT/Y12/03	12	11.78	108.99	403.71	614.74
4	TT/Y12/04	12	11.78	108.99	394.53	614.74
5	TT/Y12/05	12	11.78	108.99	412.88	596.38

(See data in appendix A19)

Table 3.23 Tension test of 6mm diameter mild (R6) steel bars

S/NO.	Identification Number	Size (mm)	Nominal Diameter measured (mm)	Cross Sectional Area (mm^2)	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
1	TT/R6/01	6	5.57	24.37	402.13	430.86
2	TT/R6/02	6	5.57	24.37	402.13	430.86
3	TT/R6/03	6	5.57	24.37	389.82	422.65
4	TT/R6/04	6	5.57	24.37	402.13	426.75
5	TT/R6/05	6	5.57	24.37	398.03	430.86

(See data in appendix A20)

3.6 WATER

The water used for the production of test specimens was a clean and colourless tap water, free of organic materials and is therefore suitable for making concrete as described in BS EN 1008 (2002). It is thus a portable water.

3.7 PREPARATION AND TESTING OF SPECIMENS

In the preparation of specimens, three types of specimens were formed with each having control specimens. The specimens formed include 150 x 150 mm cubes,

150 x 300 mm cylinders and 150 x 150 x 750 mm beams. The mix ratio used for each of them is 1:2:4 which means one part cement to two parts sand/laterite to four parts coarse aggregates by weight (see details in appendix A21 and A22).

(a). Cube specimens

Twenty four cubes measuring 150 x 150 millimetres were cast as cube specimens.

The cube specimens were numbered C01, C02 to C24. Thus C01 implies cube number 01 and C02 implies cube number 02 to C24. C01 or C02 does not mean C01 is better than C02 but simply for identification purposes.

Cube numbers C01 to C12 are control cubes. This means the cubes were cast using cement, sand, coarse aggregates and water; while C13 to C24 were cast using cement, laterite, coarse aggregates and water. These cubes are cured after casting for 7, 14, 21 and 28 days prior to crushing. Thus it is three cubes for each curing age for both control and cube specimens.

The cubes were tested in accordance to BS 1881 (1983) part 116 and the result of each curing age is as shown in tables 3.24 to 3.27 below (see details in appendix A23 and A24).

Table 3.24 Cube strength for 7 days of curing

S/NO.	CUBE NO.	COMPRESSIVE STRENGTH (N/mm^2)
1	C01	16.00
2	C02	16.89
3	C03	15.11
Average = 16.00 N/mm^2		
4	C13	5.56
5	C14	6.00
6	C15	5.78
Average = 5.78 N/mm^2		

Table 3.25 Cube strength for 14 days of curing

S/NO.	CUBE NO.	COMPRESSIVE STRENGTH (N/mm^2)
1	C04	19.11
2	C05	18.22
3	C06	18.66
Average = 18.66 N/mm^2		
4	C16	6.22
5	C17	6.00
6	C18	6.44
Average = 6.22 N/mm^2		

Table 3.26 Cube strength for 21 days of curing

S/NO.	CUBE NO.	COMPRESSIVE STRENGTH (N/mm^2)
1	C07	20.89
2	C08	21.33
3	C09	20.89
Average = 21.04 N/mm^2		
4	C19	7.56
5	C20	8.00
6	C21	8.44
Average = 8.00 N/mm^2		

Table 3.27 Cube strength for 28 days of curing

S/NO.	CUBE NO.	COMPRESSIVE STRENGTH (N/mm^2)
1	C10	23.11
2	C11	23.11
3	C12	23.56
Average = 23.26 N/mm^2		
4	C22	9.78
5	C23	8.89
6	C24	8.00
Average = 8.89 N/mm^2		

(b). Cylinder specimens

Six cylinders measuring 150 x 300 millimetres were cast as cylinder specimens for tensile tests.

The cylinder specimens were numbered CY01, CY02 to CY 06. Thus CY01 implies cylinder number 01 and CY02 implies cylinder number 02 to CY06. CY01 or CY02 does not mean CY01 is better than CY02 but simply for identification purposes.

Cylinder numbers CY01 to CY03 are control cylinders. This means the cylinders were cast using cement, sand, coarse aggregates and water; while CY04 to CY06 were cast using cement, laterite, coarse aggregates and water. These cylinders were cured after casting for 28 days prior to crushing. Thus it is three cylinders for the 28 days curing age for each of the control and cylinder specimens. The result is presented in the table 3.28 below (see details in appendix A25).

Table 3.28 Cylinder tensile strength for 28 days of curing

S/NO.	CYLINDER NO.	TENSILE STRENGTH (N/mm^2)
1	CY01	1.77
2	CY02	1.84
3	CY03	1.84
Average = 1.82 N/mm^2		
4	CY04	0.69
5	CY05	0.65
6	CY06	0.67
Average = 0.67 N/mm^2		

(c). Beam specimens

Twenty beams measuring 150 x 150 x 750 millimetres were cast as beam specimens.

The beam specimens were numbered B01, B02 to B20. B01 implies beam number 01 and B02 implies beam number 02 to B20. B01 or B02 does not mean B01 is better than B02 but simply for identification purposes.

Beam numbers B01 to B10 are control beams. This means the beams were cast using cement, sand, coarse aggregates and water while B11 to B20 were cast using cement, laterite, coarse aggregates and water. Beam numbers B01 to B05 and B11 to B15 are reinforced, while B06 to B10 and B16 to B20 are of plain concrete and latecrete respectively. These beams were cured after casting for 28 days prior to crushing.

The summary of the cast beams of plain and reinforced concrete and latecrete are shown in table 3.29.

Table 3.29 Summary of cast beam of plain and reinforced concrete and latecrete

S/NO.	BEAM NO.	PLAIN/REINFORCED	MIX MATERIAL
01	B01 – B10	Control	Cement, sand, coarse aggregate
02	B11 – B20	Latecrete	Cement, laterite, coarse aggregate
03	B01 – B05	Reinforced	Concrete
04	B11 – B15	,,	Latecrete
05	B06 – B10	Unreinforced	Concrete
06	B16 – B20	,,	Latecrete

(d). Casting of Beam specimens

The beam specimens were casted using the following design mix:

(i) 1:2:4 which is one part cement to two parts sand and four parts coarse aggregates for the Control beam specimens by weight.

(ii) 1:2:4 which is one part cement to two parts laterite and four parts coarse aggregates for the Latecrete beam specimens by weight.

The above design mix ratio is first mixed thoroughly without water in both cases and after thorough mixing, a water cement ratio of 0.75 is used for Latecrete and 0.5 for Concrete beam in accordance to BS 1881 part 3 (1985) and (Adepegba, 1975). Mortar biscuits of 25 millimetres thick were used to provide concrete cover of 25 millimetres to the beams. After mixing with water to obtain a consistency, the mix is poured into beam mould and compacted, and well trowelled smooth. After twenty four hours, it was demoulded and cured for the required age.

(iii) Reinforcement Details for the Beams

As earlier stated, some of the beams (both concrete and latecrete) are not reinforced. The reinforced beams are reinforced with 2Y12 bottom and 2Y12 top, with shear reinforcements of R6 @ 162.5 c/c spacing. The details of the reinforcements are shown in figure 3.1.

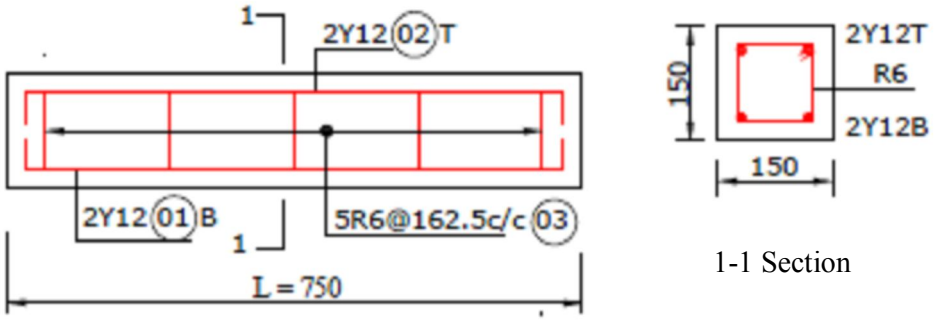


Figure 3.1 Longitudinal details of Beam/Section

(iv) Fixing of Electrical Strain Guages

The Electrical strain gauges were fixed at the centre point of the beam to measure the compression as well as tensile strains within and at the top and bottom

fibre of the beam. The Electrical strain gauges were of the following properties shown in table 3.30

Table 3.30 Properties of Electrical Strain guage

S/No.	Parameter	Units
1	Length	6 mm
2	Resistance	120 ± 0.3Ω
3	Guage factor	2.14
4	SG1	Bottom inside bar tensile strain (first bar)
5	SG2	Bottom inside bar tensile strain (second bar)
6	SG3	Top inside bar compressive strain
7	SG4	Bottom outside fibre tensile strain
8	SG5	Top outside fibre compressive strain

The Electrical strain guages were fitted with wire to the strain guage box to measure strains. Three of the strain gauges were cast into the beam and one on top compression fibre surface and another one on bottom tensile fibre surface. The details are shown in the beam cross-section at the centre point as figure 3.2.

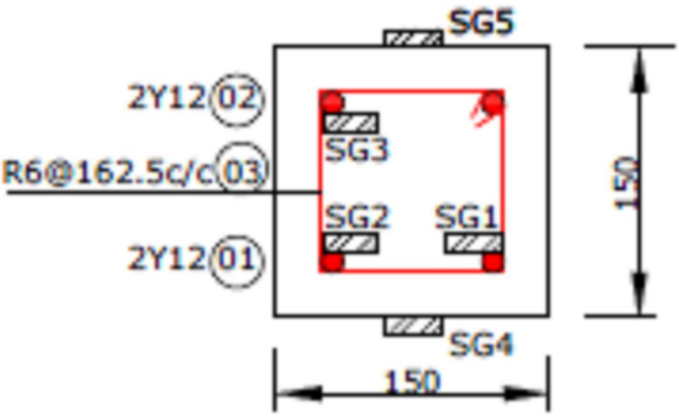


Figure 3.2 Section at Beam centre showing position of Strain guages

The strains gauges, SG1 to SG3, were soldered to the reinforcement bars prior to casting, while SG4 and SG5 were glued to the beam surface using aradite gum

before testing. The strains at various positions shown in figure 3.2 are measured using Dumec strain guage.

(v) Three Point Loading of Beam

The beam specimen is loaded in a three point flexural test set up as shown in figure 3.3 and 3.4 below.

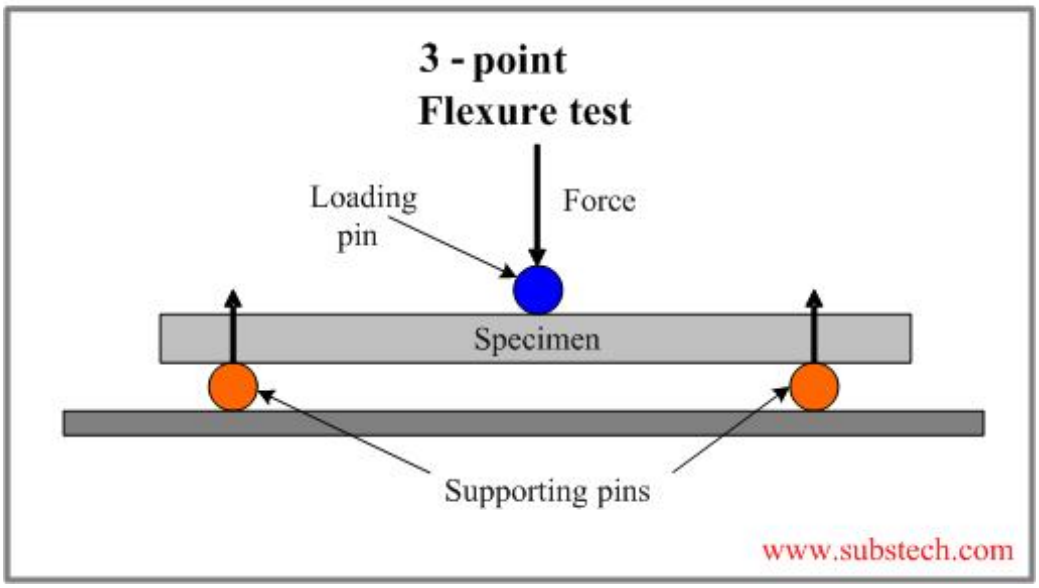


Figure 3.3 Test set up for three point loading of Beam

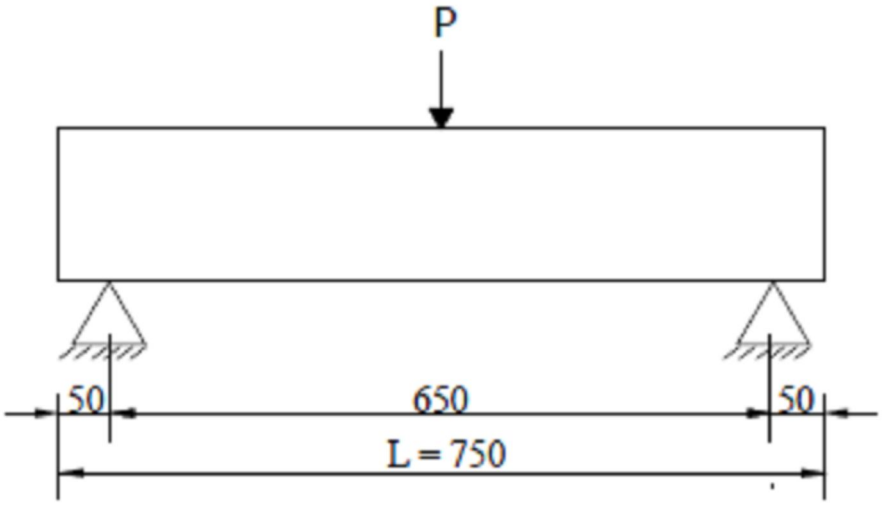


Figure 3.4 Testing/loading of Beam

(vi) Deflection Measurement

The deflection measurement of the beam at the mid-span and at equidistant opposite sides of mid-span were measured using dial gauges mounted at points a, b, c as shown in figure 3.5 below.

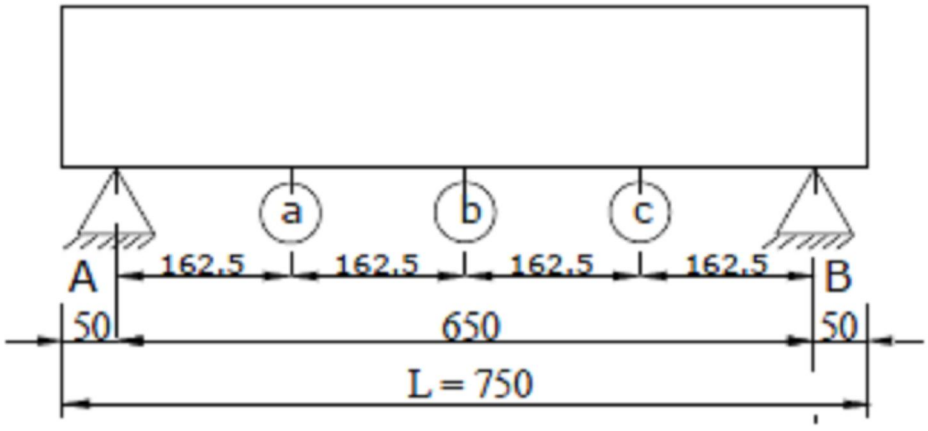


Figure 3.5 Position of Dial guages

(vii). Measurements

The value of the applied load, the strains at various strain gauges and the deflections at the three positions measured for each beam are given in tables 3.31 to 3.50.

Table 3.31 Flexure test Result on Reinforced Concrete beam B01

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B01	0	0	0	0	0	0	0	0	0
	5	4405	4600	4920	3910	9850	0.65	2.5	0.57
	10	5100	5080	5500	3980	9810	1.45	5	1.3
	15	5450	5570	6000	4170	9760	3.07	7.8	2.82
	20	5560	5572	6475	4525	9570	3.31	8.4	3.1
	25	5585	5580	6720	4820	9430	3.46	8.74	3.35
	30	5820	5765	6825	4895	9325	3.58	9.26	3.57
	35	5960	5940	6930	Cracks seen	9160	3.68	9.82	3.58
	40	6200	6075	7175	,,	8975	3.77	10.49	3.92
	45	6325	6200	7250	,,	8790	3.89	11.37	3.95
	50	6450	6300	7375	,,	8435	4.47	12.82	4.38

Table 3.32 Flexure test Result on Reinforced Concrete beam B02

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B02	0	0	0	0	0	0	0	0	0
	5	4200	4425	4780	3650	9820	1.65	2.25	1.70
	10	5300	5100	5475	3970	9780	2.4	4.25	2.10
	15	5420	5565	5990	4200	9700	2.98	6.8	2.85
	20	5540	5579	6320	4510	9650	3.27	8.1	3.05
	25	5570	5585	6800	4790	9430	3.41	8.85	4.00
	30	5800	5735	6820	4820	9290	3.48	9.5	4.10
	35	5970	5920	6900	Cracks seen	9180	4.12	10.1	4.15

	40	5990	6070	7120	,,	9050	4.25	10.98	4.18
	45	6300	6150	7190	,,	9790	4.32	11.82	4.35

Table 3.33 Flexure test Result on Reinforced Concrete beam B03

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
	0	0	0	0	0	0	0	0	0
B03	5	4500	4700	3570	3800	9720	0.73	2.83	0.61
	10	4900	5520	4850	3960	9615	1.6	5.14	1.56
	15	5500	5580	5025	4215	9580	3.21	7.93	3.05
	20	5555	5595	5415	4435	9450	3.48	8.48	3.32
	25	5800	5670	5750	4625	9420	3.61	8.94	3.45
	30	5850	5820	5800	Cracks seen	8975	3.69	9.35	3.76
	35	6120	5925	6315	,,	8825	3.74	9.52	3.83
	40	6250	6138	6480	,,	8738	3.83	9.83	3.88
	45	6390	6275	6950	,,	8690	3.89	10.97	3.93

Table 3.34 Flexure test Result on Reinforced Concrete beam B04

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
	0	0	0	0	0	0	0	0	0
	5	4725	4810	4875	3790	9785	0.78	2.83	0.79
B04	10	5275	5035	5475	3835	9730	1.48	5.43	1.56
	15	5590	5510	5873	4200	9675	3.26	8.45	3.18
	20	5610	5540	6135	4375	9613	3.48	8.96	3.3
	25	5635	5575	6420	4515	9550	3.53	9.31	3.48
	30	5780	5725	6594	Cracks seen	9435	3.67	9.84	3.62
	35	5850	5895	6785	,,	9315	3.85	10	3.76
	40	5990	6125	6900	,,	8935	3.92	10.58	3.85
	45	6150	6375	7250	,,	8785	3.97	10.92	3.96
	50	6370	6400	7293	,,	8415	4.32	11.86	4.25

Table 3.35 Flexure test Result on Reinforced Concrete beam B05

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm		
		SG1	SG2	SG3	SG4	SG5	a	b	c
	0	0	0	0	0	0	0	0	0
B05	5	4575	4690	4870	3875	8935	0.73	2.65	0.75
	10	5300	5135	5475	3913	8924	1.83	5.43	1.91
	15	5530	5581	6125	4020	8840	3.21	8.14	3.32
	20	5545	5584	6384	4514	8647	3.35	8.46	3.42
	25	5578	5593	6685	4834	8763	3.47	8.67	3.51
	30	5738	5678	6813	4875	8614	3.59	9.1	3.61
	35	5815	5715	6928	Cracks seen	8218	3.62	9.65	3.67
	40	5975	5790	7245	,,	8200	3.75	10.32	3.83
	45	6180	5813	7314	,,	8105	3.81	10.97	3.93

Table 3.36 Flexure test Result on Plain Concrete beam B06

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm		
		SG1	SG2	SG3	SG4	SG5	a	b	c
	0				0	0	0	0	0
B06	2.5				2500	3025	1.63	3.5	1.69
	5				3575	2975	2.1	3.95	2.15
	7.5				3617	2925	2.14	3.98	2.18
	10				3642	2775	2.19	4.21	2.21

Table 3.37 Flexure test Result on Plain Concrete beam B07

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
	0				0	0	0	0	0

B07	2.5				2725	3082	1.4	3.2	1.35
	5				3478	2984	1.82	3.75	1.78
	7.5				3525	2924	1.93	3.82	1.84
	10				3593	2275	1.97	3.94	1.9

Table 3.38 Flexure test Result on Plain Concrete beam B08

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection(mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B08	0				0	0	0	0	0
	2.5				2672	3075	1.38	3.17	1.42
	5				3310	2997	1.76	3.48	1.84
	7.5				3478	2971	1.97	3.76	1.99
	10				3510	2885	1.99	3.98	2.03

Table 3.39 Flexure test Result on Plain Concrete beam B09

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B09	0				0	0	0	0	0
	2.5				2734	3125	1.75	3.47	1.81
	5				3516	2990	2	3.84	1.98
	7.5				3525	2910	2.12	3.97	2.07
	10				3578	2873	2.14	4.15	2.09

Table 3.40 Flexure test Result on Plain Concrete beam B10

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B10	0				0	0	0	0	0
	2.5				2710	3025	1.51	3.39	1.62

	5				3214	2841	1.84	3.67	1.73
	7.5				3418	2812	1.91	3.88	1.85
	10				3564	2795	1.98	3.99	1.93

Table 3.41 Flexure test Result on Reinforced Latecrete beam B11

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B11	0	0	0	0	0	0	0	0	0
	5	3120	3240	2000	3050	7850	1.6	3.7	1.65
	10	3980	3780	3570	3280	7680	2.93	6.34	2.71
	15	4572	4670	5010	3480	7290	3.41	7.87	3.31
	20	5470	5375	5130	3495	6850	4.37	9.78	4.13
	25	5580	5585	5260	Cracks seen	6730	4.99	10.2	4.9
	30	5775	5895	5300	,,	6150	5.1	10.34	5.24

Table 3.42 Flexure test Result on Reinforced Latecrete beam B12

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B12	0	0	0	0	0	0	0	0	0
	5	3410	3417	2575	3250	7678	1.72	3.95	1.81
	10	3825	3815	3610	3395	7630	2.97	6.39	2.84
	15	4375	4515	5210	3478	7375	3.45	7.89	3.52
	20	4890	4750	5275	3499	6900	4.52	9.35	4.27
	25	5272	4995	5290	Cracks seen	6725	5.01	10.5	4.97

Table 3.43 Flexure test Result on Reinforced Latecrete beam B13

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B13	0	0	0	0	0	0	0	0	0
	5	3450	3319	2498	3310	7565	1.8	3.78	1.92
	10	3795	3717	3575	3398	7507	2.78	6.27	2.73
	15	4278	4285	5100	3469	7362	3.57	7.82	3.48
	20	4775	4699	5197	3478	7010	4.76	9.41	4.65
	25	5127	5025	5203	Cracks seen	6835	5.17	11.3	5.08

Table 3.44 Flexure test Result on Reinforced Latecrete beam B14

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B14	0	0	0	0	0	0	0	0	0
	5	3475	3482	2678	3175	7512	1.84	4.21	1.94
	10	3693	3691	3725	3379	7417	2.93	6.43	2.99
	15	3978	3882	5175	3402	7325	3.68	7.83	3.57
	20	4275	4516	5205	3473	6815	4.72	9.78	4.63
	25	4551	4825	5271	Cracks seen	6737	5.25	10.93	5.76

Table 3.45 Flexure test Result on Reinforced Latecrete beam B15

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B15	0	0	0	0	0	0	0	0	0
	5	3321	3583	2785	3219	7487	2.21	4.57	2.29
	10	3596	3673	3817	3372	7432	3.27	6.94	3.42
	15	3878	3894	5292	3416	7393	3.78	8.21	3.87
	20	3927	3913	5315	3464	7010	4.32	9.87	4.17
	25	4120	4374	5327	Cracks seen	6895	5.17	10.95	5.21
	30	4475	4525	5351	,,	6732	5.76	11.34	5.87

Table 3.46 Flexure test Result on Plain Latecrete beam B16

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B16	0				0	0	0	0	0
	2.5				2542	3215	1.72	3.8	1.81
	5				3678	2875	1.92	3.98	1.95

Table 3.47 Flexure test Result on Plain Latecrete beam B17

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B17	0				0	0	0	0	0
	2.5				2595	3275	1.82	3.92	1.73
	5				3572	2982	1.95	3.97	1.94

Table 3.48 Flexure test Result on Plain Latecrete beam B18

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B18	0				0	0	0	0	0
	2.5				2575	3164	1.8	3.84	1.81
	5				3479	2950	1.92	3.95	1.87

Table 3.49 Flexure test Result on Plain Latecrete beam B19

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
B19	0				0	0	0	0	0
	2.5				2592	3105	1.76	3.74	1.72
	5				3671	2978	1.84	3.85	1.82

Table 3.50 Flexure test Result on Plain Latecrete beam B20

Sample Identification No.	Load (kN)	Strain x 10^{-6}					Deflection (mm)		
		SG1	SG2	SG3	SG4	SG5	a	b	c
	0				0	0	0	0	0

B20	2.5				2596	3214	1.76	3.96	1.81
	5				3545	2984	1.92	4.1	1.94

CHAPTER FOUR

ANALYSIS AND DISCUSSION OF RESULT

The test results obtained in chapter three is analysed in chapter four to arrive at reasonable conclusions.

4.1 CEMENT

The results obtained from cement tests as shown in table 3.1 to 3.6 of chapter three is summarized here and compared with Code specifications.

Table 4.1 Comparison of results of Cement tests with Code specifications

S/No.	Parameter	Value	Code Specifications BS EN 197-1 (2000)
01	Fineness	0.05	0.01 – 0.06
02	Consistency	31%	-
03	Initial Setting time	80 minutes	≥ 45 minutes

04	Final Setting time	170 minutes	≤ 600 mm
05	Soundness	1.0 mm	≤ 10 mm
06	Mortar Cube Compressive Strength (7 days)	21.53 N/mm^2	≥ 16 N/mm^2

From table 4.1, the parameters for the cement indicate the values to have passed the code specifications. Thus, the cement used is good and of standard.

4.2 SAND

(a) The Particle size distribution for the sand as shown in table 3.7 to 3.9 of chapter three is as shown in figure 4.1 and discussed below.

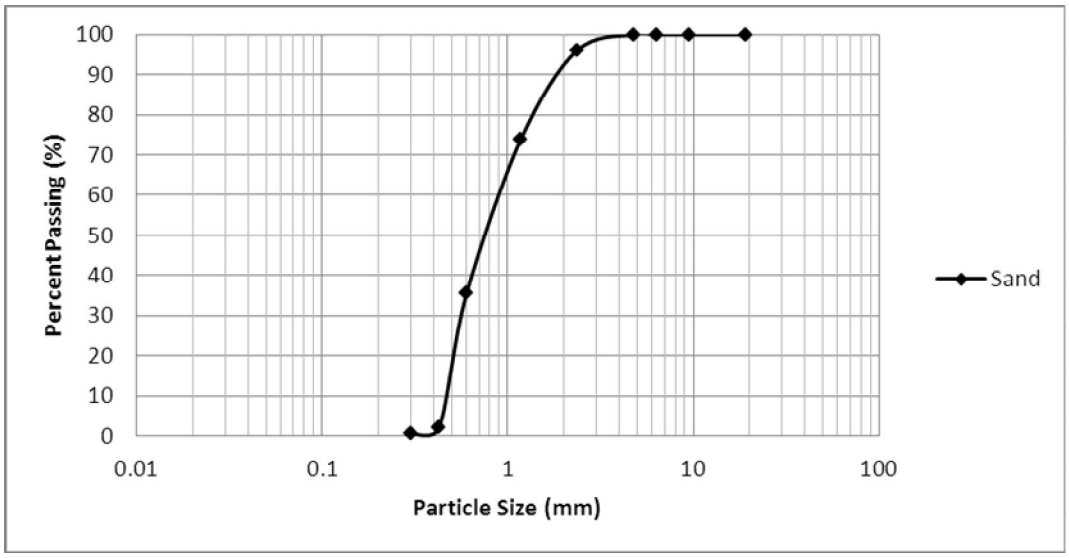


Figure 4.1: Particle size distribution of Sand

From figure 4.1, it is observed that 35.50 per cent of the sand passes BS sieve size 600μm which shows that the sand is classified as belonging to zone two gradation, characterized by medium sands, which is good for concrete in accordance to BS 882 (1992). The sand is also well graded and can be used for concrete.

(b) The Specific gravity of the sand is 2.58, which is within the ASTM C128 code specification of 2.4 to 2.9. Thus the sand can be used for good concrete.

(c) Silt Content

The silt content obtained as given in table 3.5 of chapter three is 2.23 per cent and is less than 10 per cent as specified by code (BS EN 1008 (2002)). The silt content of the sand is satisfactory to the code.

4.3 LATERITE

The result of chemical analysis by Osinubi (1998) and Particle Size distribution are as shown in table 3.10 to 3.13 of chapter three. It is observed that the laterite is mostly Silicon oxide, followed by Aluminium oxide and Ferric oxide. This is normal for laterite. The particles passing size No. 200 (0.075 mm) is 56.71 per cent greater than 50 per cent indicating that the Laterite is not economical for stabilization [BS 1377 (1990)].

The laterite has a Specific gravity of 2.6 which is within the standard. It is classified as A-7-6 (10) which is poor for making concrete. The particle size distribution is as shown in figure 4.2 below.

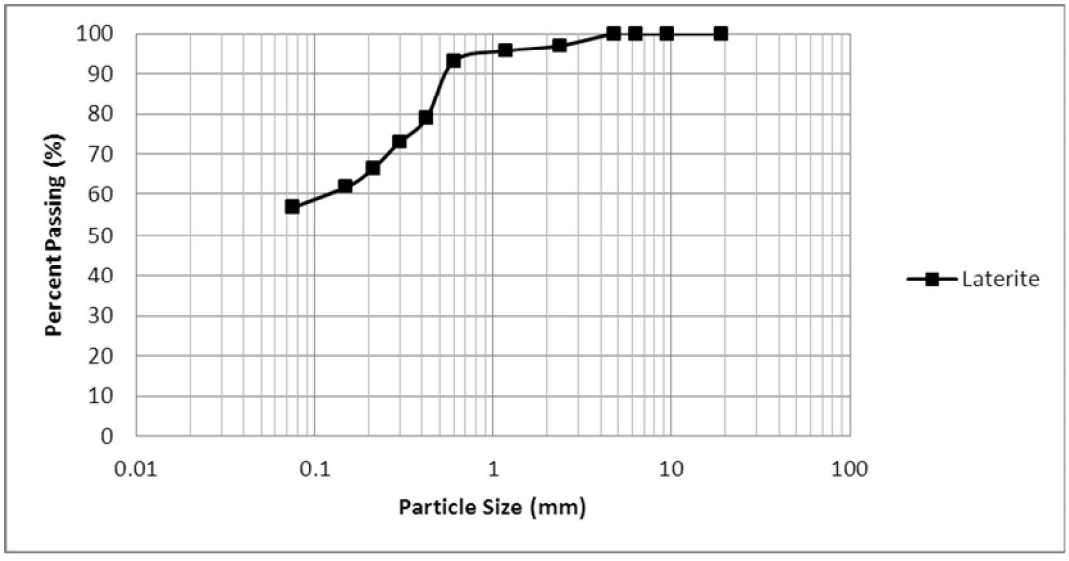


Figure 4.2 Particle size distribution of laterite

From figure 4.2, it shows that 93.02 per cent passes BS sieve size 600µm which shows that the laterite is classified as belonging to zone four gradation, characterized by fine laterite, which is poor for concrete in accordance to BS 882 (1992).

4.4 COARSE AGGREGATES

The particle size distribution of coarse aggregate after sieving is given in table 3.14 of chapter three is as shown in figure 4.3 below. It shows that the coarse aggregate is well graded which is good for concrete.

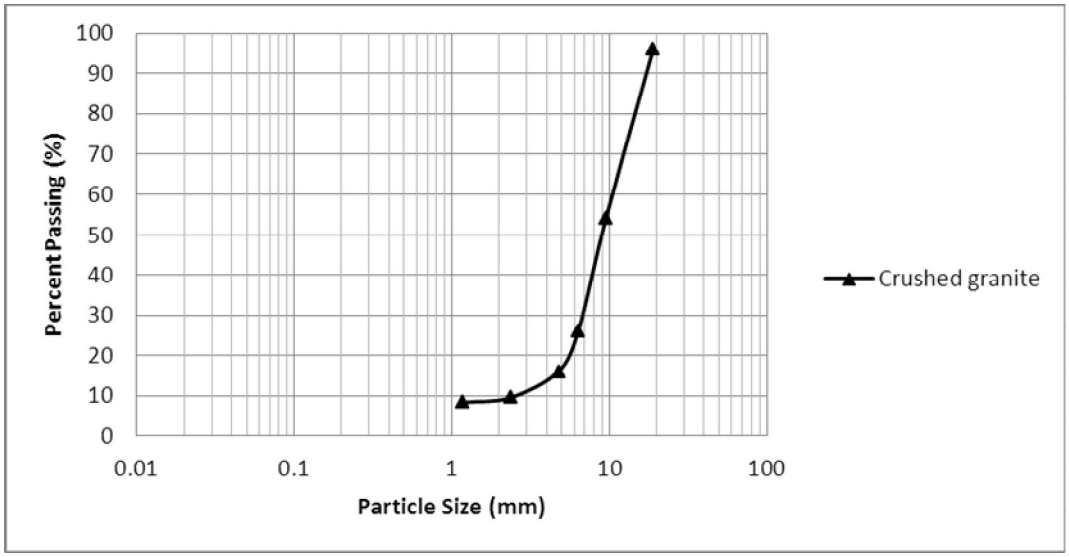


Figure 4.3 Particle size distribution of Coarse aggregates

The results obtained from coarse aggregates tests as shown in table 3.1 to 3.6 of chapter three is summarized here and compared with Code specifications.

Table 4.2 Comparison of results of Coarse aggregates tests with Code specifications

S/No.	Parameter	Value	Code Specifications BS 812 (1985)
01	Specific gravity	2.6	2.4 – 2.9
02	Flakiness Index	14.12%	35% Max.
03	Elongation Index	14.70%	35% Max.
04	Aggregate Impact Value	41.9%	≤ 45%

05	Aggregate Crushing Value	19%	30% Max.

From table 4.2, the followings are the parameters and values of the coarse aggregates: Specific gravity - 2.6, flakiness index – 14.12%, elongation index - 14.70%, aggregate impact value - 41.9% and aggregate crushing value - 19%. All the values are within the code specification. Thus the coarse aggregate used is within the specifications.

4.5 REINFORCEMENT

The reinforcement test result is as presented in tables 3.20 to 3.21 of chapter three for elongation and tables 3.22 to 3.23 for tension.

(a) Characteristic Strength and Elongation of high yield bars

The Summary of the characteristic strength and elongation for the high tensile bar is as shown in table 4.3 below.

Table 4.3 Comparison of results of reinforcement tests of high yield bars with Code specifications

S/No.	Parameter	Value	Code Specifications BS 4449 (1997)
01	Characteristic Strength	394 N/mm^2	460 N/mm^2
02	Ultimate Strength	606 N/mm^2	≥429.4 N/mm^2
03	Elongation	16%	≥12%

From table 4.3, it is observed that the characteristic strength of Y12 bars is 394 N/mm^2 which is less than the code specification of 460 N/mm^2, indicating that it failed the test in the characteristic strength parameter. This indicates that the high yield bar may be a mild steel bar material that is retreaded as high yield. However, the ultimate

strength and elongation are 606 N/mm^2 and 16% respectively, which are within the code specifications.

(b) Characteristic Strength and Elongation of mild steel bars

The Summary of the characteristic strength and elongation for the mild steel bars is as shown in table 4.4 below.

Table 4.4 Comparison of results of reinforcement tests of mild steel bars with Code specifications

S/No.	Parameter	Value	Code Specifications BS 4449 (1997)
01	Characteristic Strength	391 N/mm^2	250 N/mm^2
02	Ultimate Strength	428 N/mm^2	≥429.1 N/mm^2
03	Elongation	19%	≥22%

From table 4.4, it is observed that the characteristic strength of the R6 bars is 391 N/mm^2 which is greater than the code specification of 250 N/mm^2, indicating that the mild steel bar is good as it passed the code specification. The Ultimate strength is 428 N/mm^2, which is approximately equal to the code specification of 429.1 N/mm^2 which indicate that it complies with the code specification. It is also observed that the elongation is 19% which is below the code specification of at least 22%, thus the mild steel reinforcement used failed to satisfy the code requirement for elongation.

4.6 WATER

The water used for the production of test specimens was a clean and colourless tap water, free of organic materials and is therefore suitable for making concrete [BS EN 1008 (2002)]. It is thus a portable water.

4.7 COMPRESSIVE STRENGTH OF CONCRETE AND LATECRETE

The results of the compressive strength of the concrete and latecrete cubes presented in tables 3.22 to 3.25 of chapter three are summarized in table 4.5 below.

Table 4.5 Compressive Strength of Concrete and Latecrete cubes

S/No.	Age (days)	Average Compressive Strength of Concrete cube (N/mm^2) [Control test]	Average Compressive Strength of Latecrete cube (N/mm^2)
01	7	16.00	5.78
02	14	18.06	6.22
03	21	21.04	8.00
04	28	23.26	8.89

The graph of the Strength development of the concrete and latecrete cubes with age is shown in figure 4.4 below.

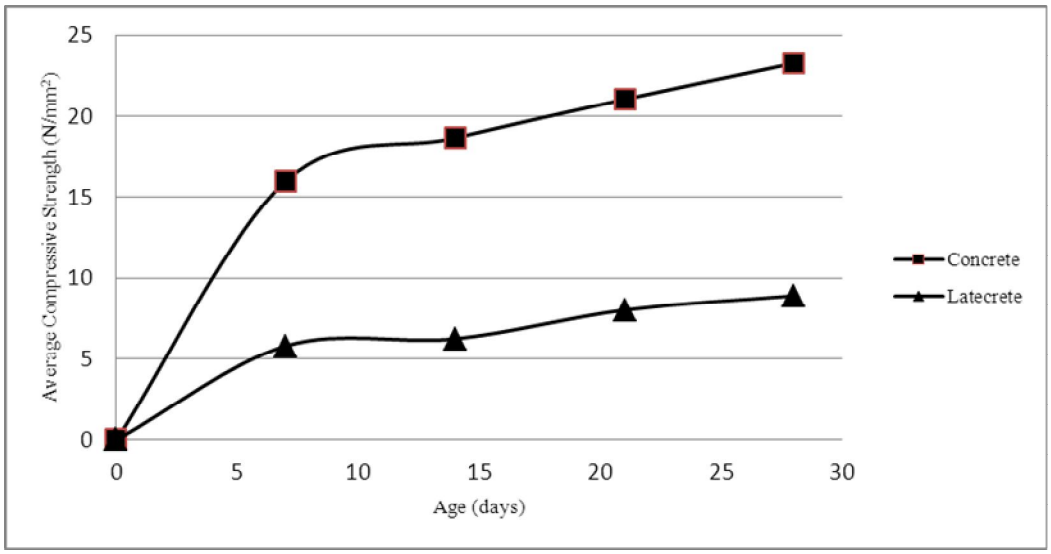


Figure 4.4: Cube Compressive Strength of Concrete and Latecrete

Figure 4.4 shows that the 28 days compressive strength values (average of three cubes), are 23.26 N/mm^2 and 8.89 N/mm^2 for concrete and latecrete respectively. This clearly shows that the compressive strength of concrete is higher than that of latecrete.

From figure 4.4 it is observed that both latecrete and concrete cube specimens gained much part of their strength in the early days of casting. This rapid early gain in strength behaviour of the specimens is attributable to the hardening process theory of cement. The hardening process of cement commences at the end of initial set and proceeds first at a fairly rapid rate; it then slows down appreciably with time, though proceeding indefinitely (Neville, 1995).

It is also observed from figure 4.4 that the higher the curing age, the higher is the difference in the compressive strength of both concrete and latecrete. This difference in compressive strength of the concrete and latecrete cubes widens with age, though maintaining a parabolic form. The parabolas of the compressive strength of the concrete and latecrete resemble the shape of the trumpet.

It has been observed that there are yet no specifications for latecrete. However, the results, when compared to the recommendations of Adepegba (1975a), Neville

(1995), Osunade (2002) and Ata (2003), indicate that the compressive strength of latecrete does not conform to the strength requirements for construction of structural members (beams).

4.8 TENSILE STRENGTH OF CONCRETE AND LATECRETE

The Cylinder tensile strength test result for the concrete and latecrete presented in table 3.26 of chapter three is summarized in table 4.6 below.

Table 4.6 Cylinder tensile Strength of Concrete and Latecrete

S/No.	Age (days)	Average Cylinder tensile Strength of Concrete (N/mm^2) [Control test]	Average Cylinder tensile Strength of Latecrete (N/mm^2)
01	28	1.82	0.67

From table 4.6, the tensile strength of concrete and latecrete are 1.82 N/mm^2 and 0.67 N/mm^2 respectively. The general low value of the tensile strength of both concrete and latecrete shows that the materials are weak in tension.

4.9 FLEXURAL STRENGTH OF BEAM SPECIMENS

The Flexural strength test results for the concrete and latecrete are presented in table 3.31 to 3.50 of chapter three. Using the tables as presented in chapter three and the formula in the code, the flexural strength is calculated and is shown in table 4.7 below. (See Appendix A26 for formula and calculations of flexural strength of the beams)

Table 4.7 Summary of Flexure Test Results of Concrete and Latecrete Beams

S/No.	Identification No.	Failure Load (kN)	Mid span deflection (mm)	Flexural Strength (N/mm^2)	Average Flexural Strength (N/mm^2)

01	B01	50	12.82	14.44	13.58
02	B02	45	11.82	13.00	
03	B03	45	10.97	13.00	
04	B04	50	11.86	14.00	
05	B05	45	10.97	13.00	
06	B06	10	4.21	2.89	2.89
07	B07	10	3.94	2.89	
08	B08	10	3.98	2.89	
09	B09	10	4.15	2.89	
10	B10	10	3.99	2.89	
11	B11	30	10.34	8.67	7.80
12	B12	25	10.50	7.22	
13	B13	25	11.30	7.22	
14	B14	25	10.93	7.22	
15	B15	30	11.34	8.67	
16	B16	5	3.98	1.44	1.44
17	B17	5	3.97	1.44	
18	B18	5	3.95	1.44	
19	B19	5	3.85	1.44	
20	B20	5	4.10	1.44	

From table 4.6, recall that:

- B01 to B05 are reinforced concrete beams,
- B06 to B10 are plain concrete beams,
- B11 to B15 are reinforced latecrete beams,
- and B16 to B20 are plain latecrete beams,

the average values of the flexural strength of the Beams is as shown in table 4.8 below.

Table 4.8 Average Flexural strength of Concrete and Latecrete Beams

S/No.	Beam No.	Plain/Reinforced	Average Flexural Strength (N/mm2)

01	B01 – B05	Reinforced Concrete	13.58
02	B06 – B10	Plain Concrete	2.89
03	B11 – B15	Reinforced Latecrete	7.80
04	B16 – B20	Plain Latecrete	1.44

From table 4.7, it is observed that the flexural strength of latecrete beams is about half of that of concrete beams.

(a). Load - deflection characteristics of reinforced concrete and reinforced latecrete beam specimens at mid-span

Figures 4.5 to 4.10 show the shape of the load – midspan deflection curves for reinforced concrete beams of B01 to B05. The data was presented in chapter three as tables 3.31 to 3.35

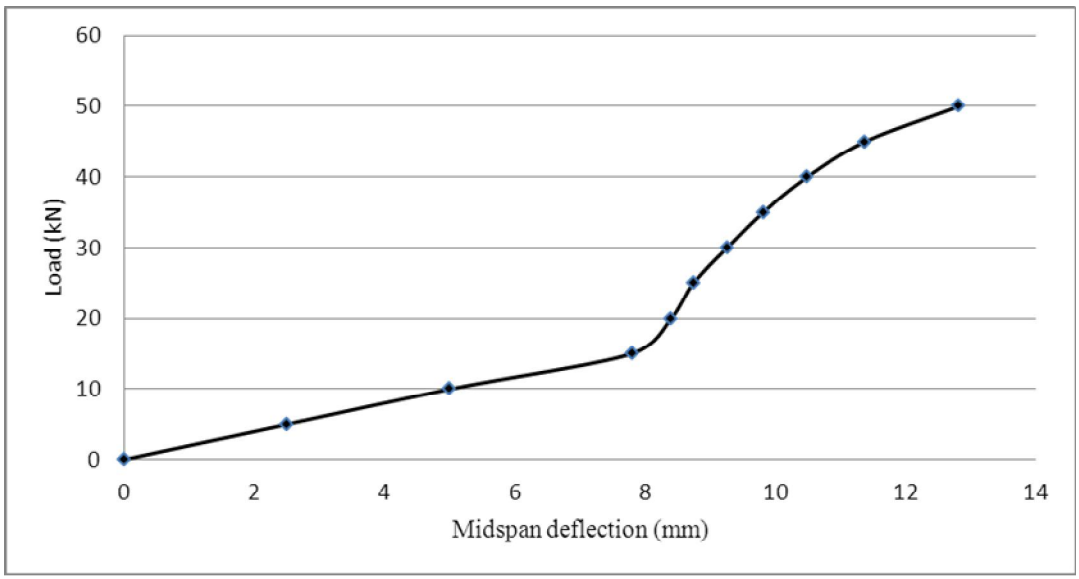


Figure 4.5 Load – midspan deflection of reinforced concrete beam, B01

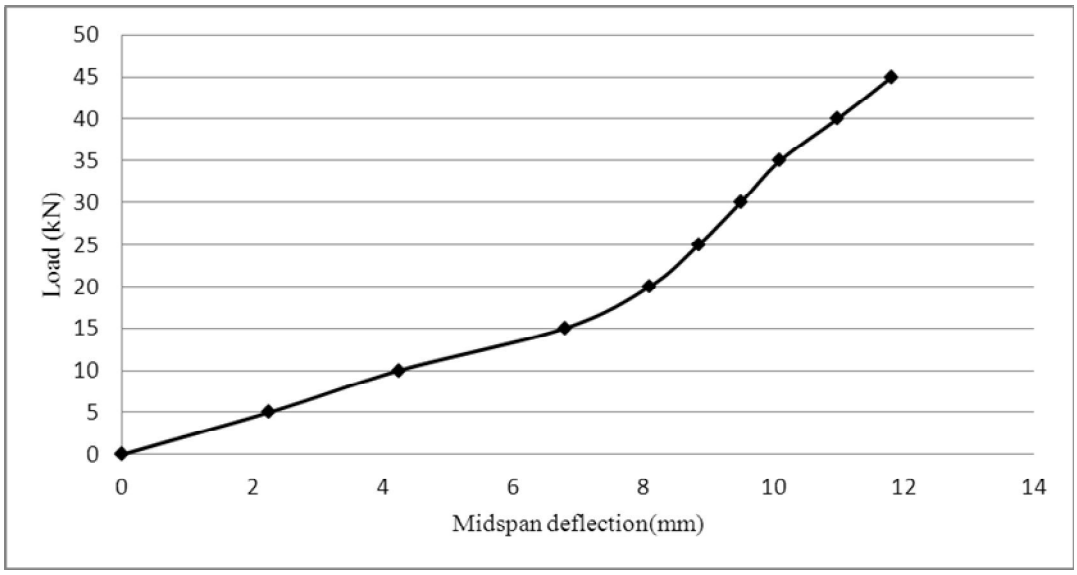


Figure 4.6 Load – midspan deflection of reinforced concrete beam, B02

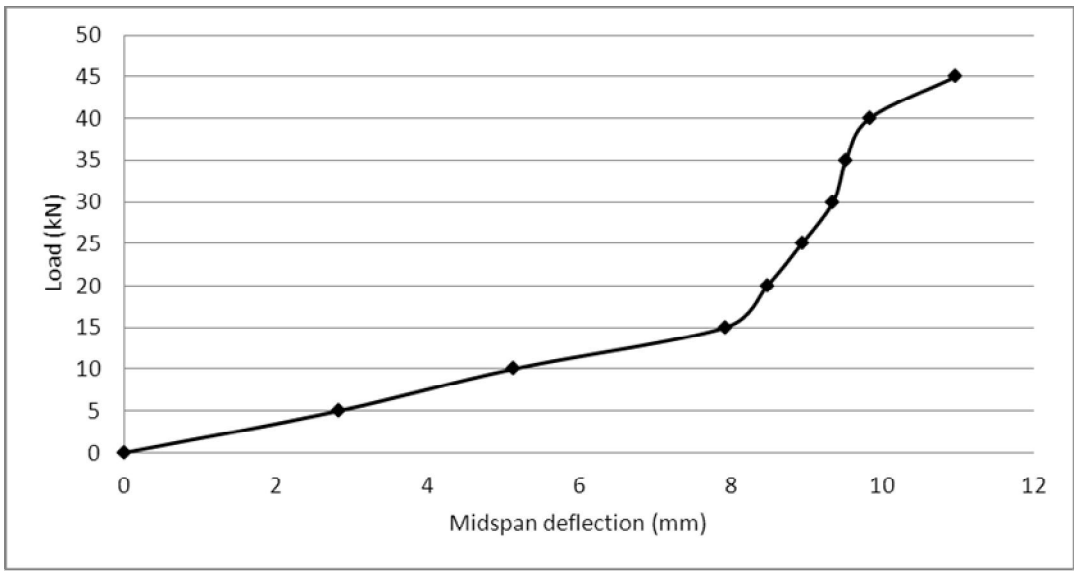


Figure 4.7 Load – midspan deflection of reinforced concrete beam, B03

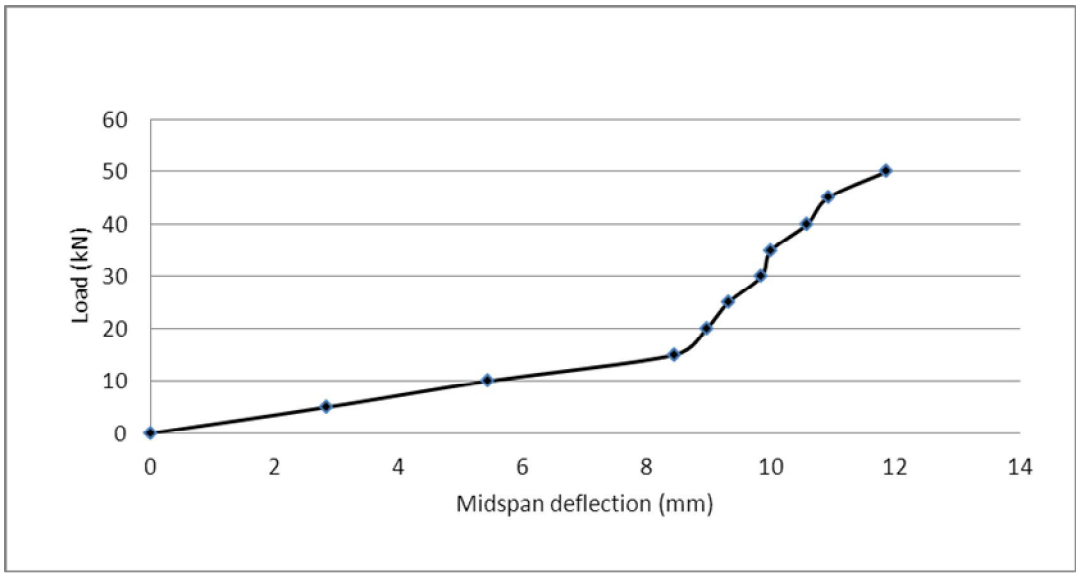


Figure 4.8 Load – midspan deflection of reinforced concrete beam, B04

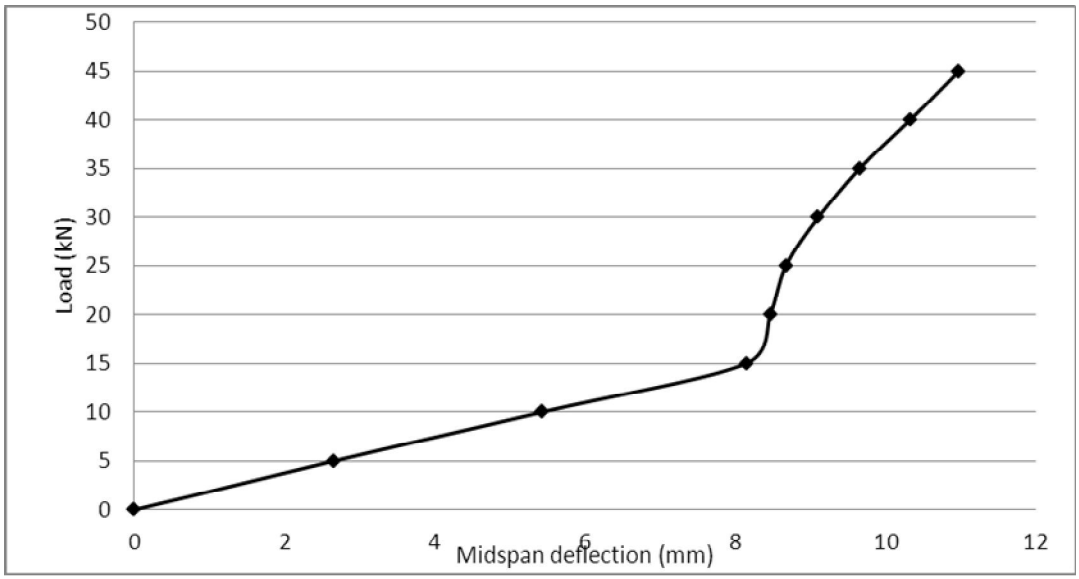


Figure 4.9 Load – midspan deflection of reinforced concrete beam, B05

Figures 4.5 to 4.9 shows the load – midspan deflection shape of reinforced concrete beam specimen of B01 to B05, up to failure load. It can be observed that the shape of the figure is approximately linear up to a load of 15 kN for the reinforced concrete beams. Thus it can be said that the load mid-span deflection for reinforced

concrete beam is of linear proportion up to 15 kN load. The corresponding mid-span deflection at this load is approximately 8 millimeter.

Beyond this load, the load – midspan deflection from these figures are non-linear with increase in load with little or no increase in the corresponding deflection. Thus internal cracks within the beams are being formed absorbing the load and with little or no deflection. The average failure load at mid-span for reinforced concrete beam of B01 to B05 is 47 kN with a corresponding deflection of 11.67 millimeter.

The combined load – midspan deflection plot for all the reinforced concrete beams of B01 to B05 is as shown in figure 4.10 below.

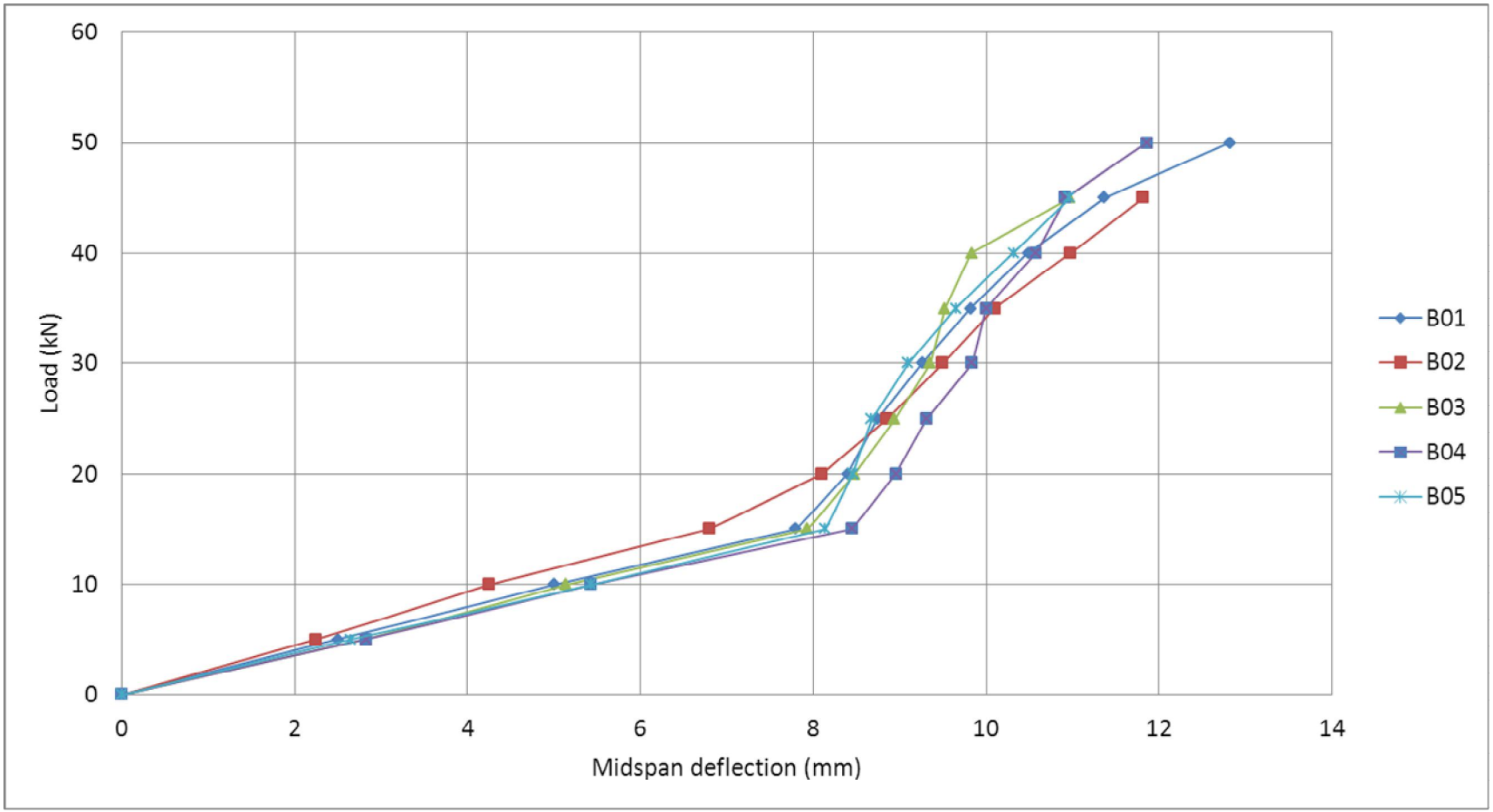


Figure 4.10 Load – midspan deflection for reinforced concrete beams, B01 to B05

Figures 4.11 to 4.16 show the shape of the load – midspan deflection curves for reinforced latecrete beams of B11 to B15. The data was presented as tables 3.41 to 3.45 of chapter three.

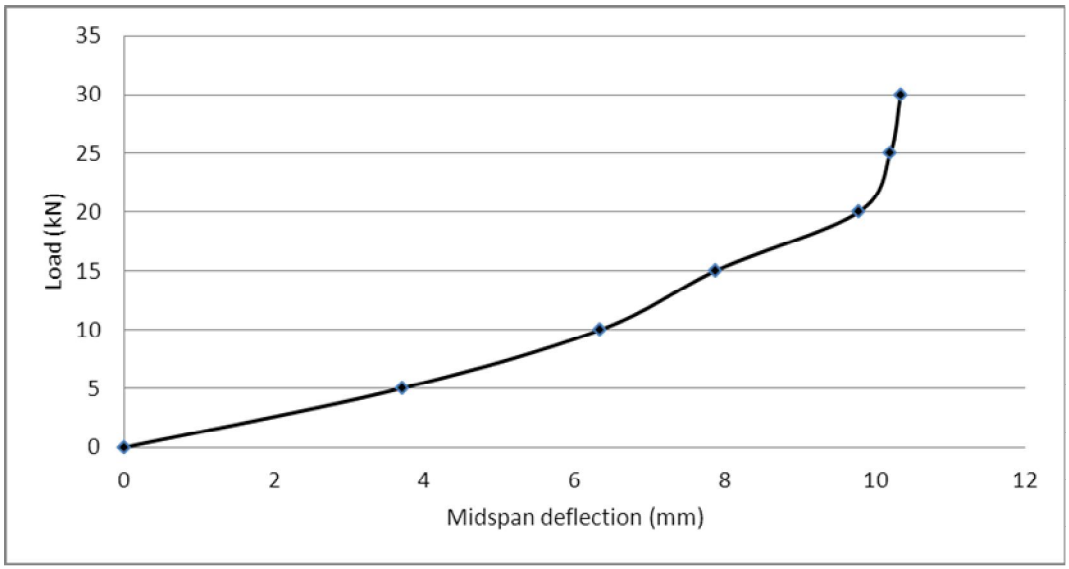


Figure 4.11 Load – midspan deflection of reinforced latecrete beam, B11

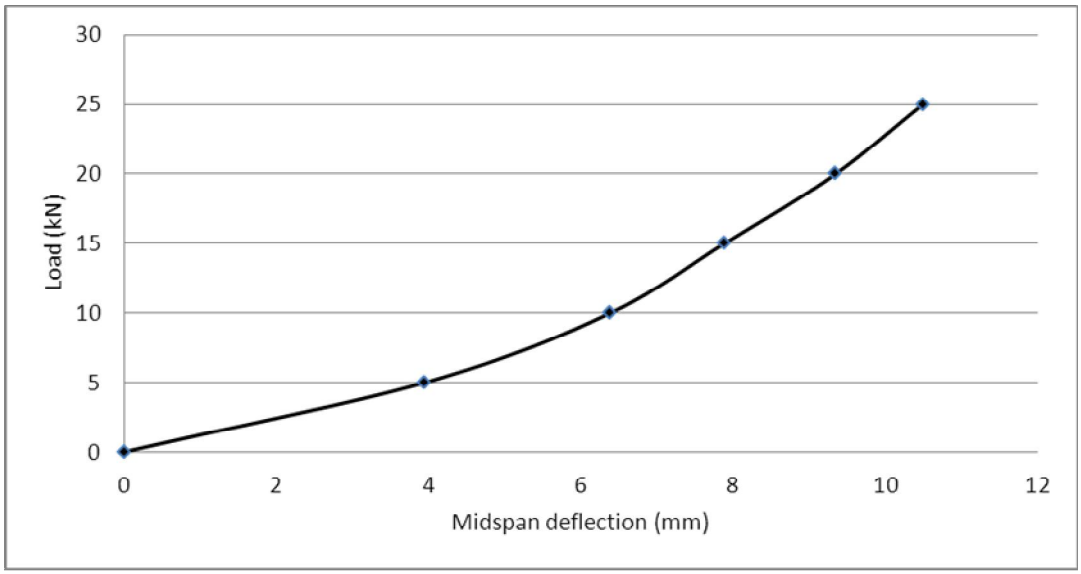


Figure 4.12 Load – midspan deflection of reinforced latecrete beam, B12

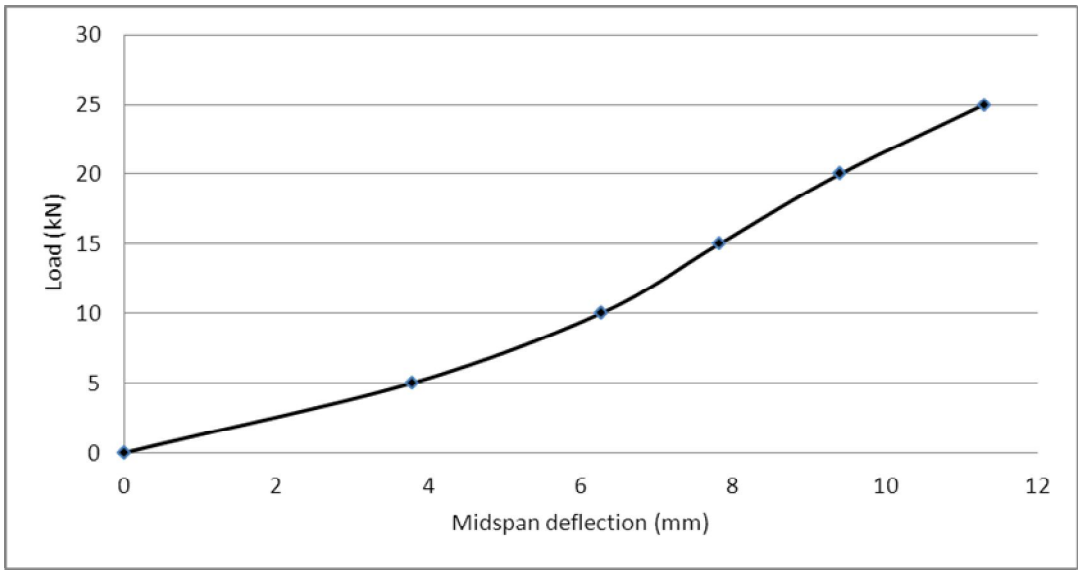


Figure 4.13 Load – midspan deflection of reinforced latecrete beam, B13

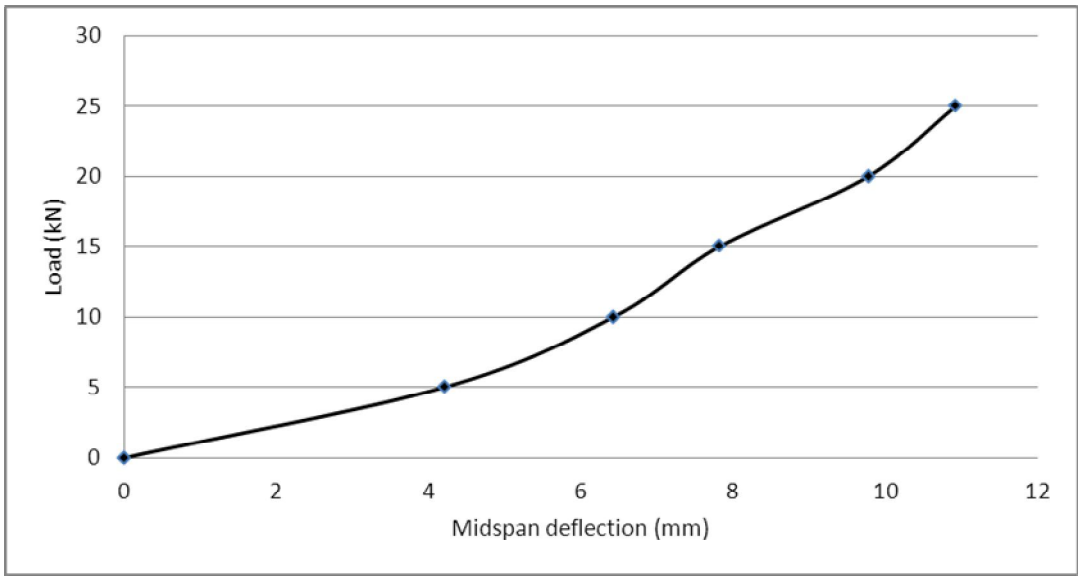


Figure 4.14 Load – midspan deflection of reinforced latecrete beam, B14

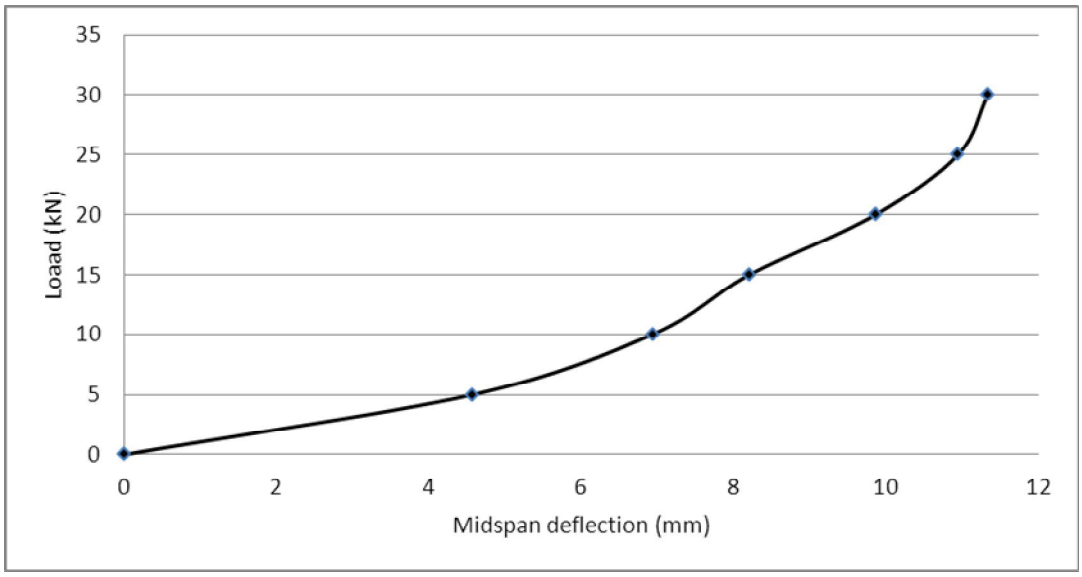


Figure 4.15 Load – midspan deflection of reinforced latecrete beam, B15

From Figures 4.11 to 4.15, it is observed that the reinforced latecrete beams exhibit the same behaviour as described above for the reinforced concrete beam. The only difference is that the curve is linear up to a load of 10 kN for all the reinforced latecrete beams of B11 to B15. The corresponding midspan deflection for the load of 10 kN is approximately 6 millimeters. From 10 kN value, the latecrete beam becomes non-linear in behaviour. Thus in the latecrete beams, cracks forms beyond 10 kN load. Therefore, the latecrete beams shows lower load and deflection at elastic limit. The maximum load at failure for the reinforced latecrete beams is 27 kN and with a corresponding midspan deflection of 10.88 millimeter.

The combined graph of load – midspan deflection of the reinforced latecrete beams is shown in figures 4.16 below.

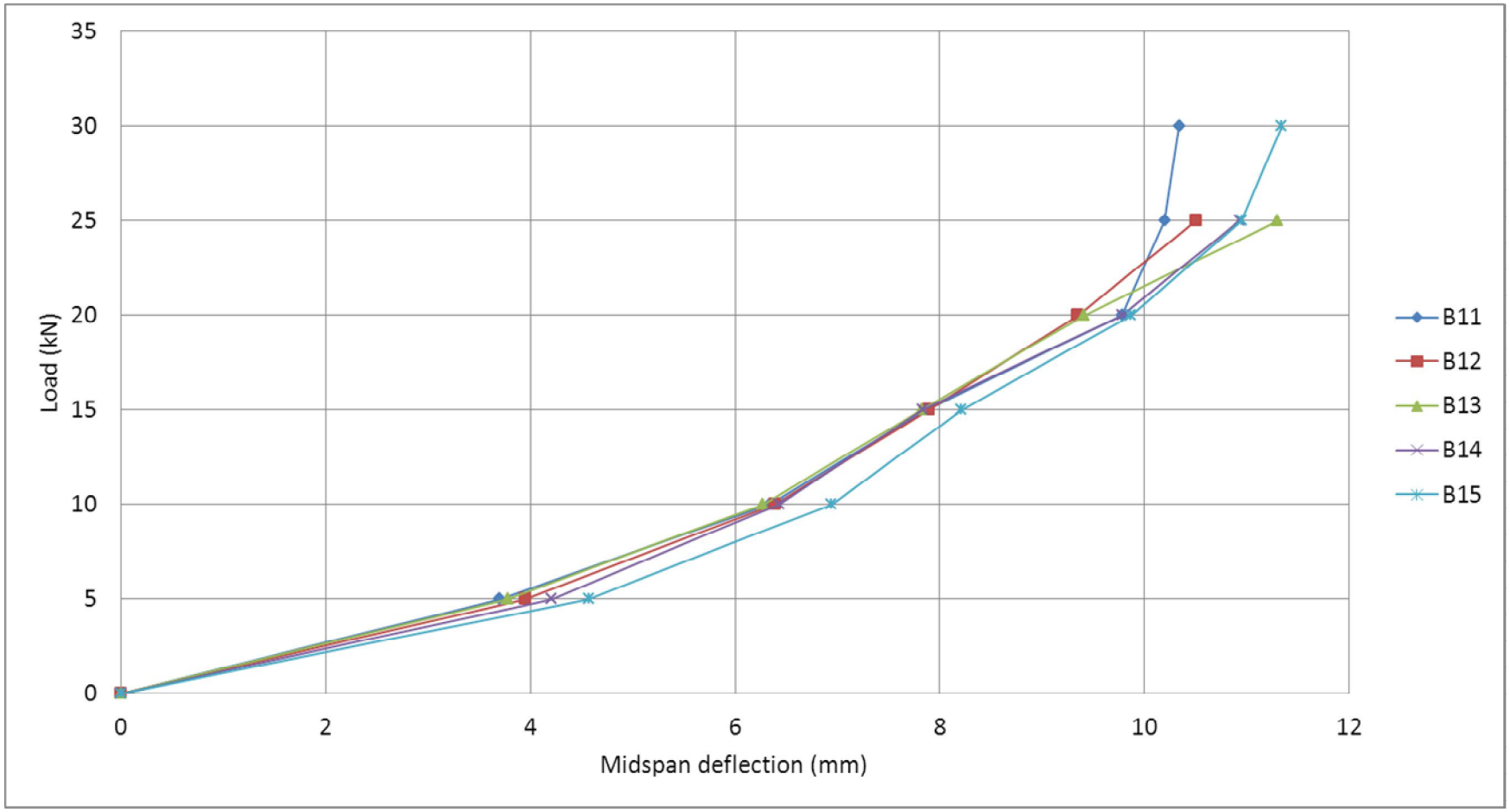


Figure 4.16 Load – midspan deflection of reinforced latecrete beams B11 to B15

The discussions on the load – midspan deflection characteristics of the reinforced concrete beams and reinforced latecrete beams above is summarized in table 4.9 below.

Table 4.9 Approximate values of average Elastic load, average midspan deflection at elastic load, average failure load and average midspan deflection at failure load

S/No.	Beam No.	Approximate average Elastic load (kN)	Approximate average midspan deflection at elastic load (mm)	Approximate average failure load (kN)	Approximate average midspan deflection at failure load (mm)
01	B01 – B05	15	8	47	11.67
02	B11 – B15	10	6	27	10.88
03	% of latecrete beam/concrete beam	67	75	50	100

Even though the Reinforced concrete beams exhibit higher load at failure, almost 100 percent, the midspan deflection at failure is the same. This implies that Reinforced concrete beams is more ductile than the reinforced latecrete beams.

4.6.2 Load - deflection characteristics of plain concrete and plain latecrete beam specimens at midspan

Figures 4.17 to 4.22 show the shape of the load – midspan deflection curves for plain concrete beams of B06 to B10. The data was presented in chapter three as tables 3.36 to 3.40.

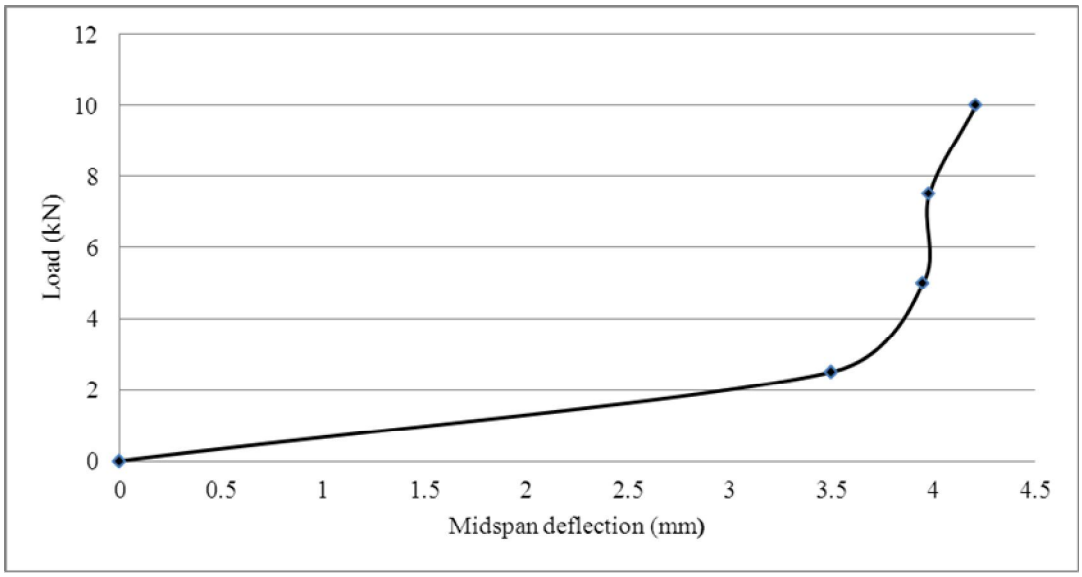


Figure 4.17 Load – midspan deflection of plain concrete beam B06

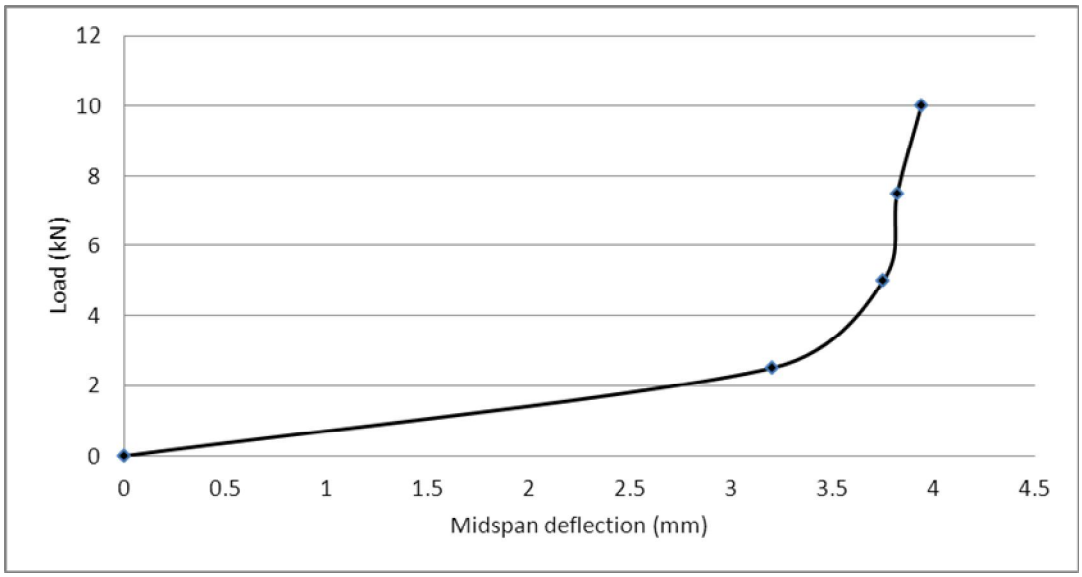


Figure 4.18 Load – midspan deflection of plain concrete beam B07

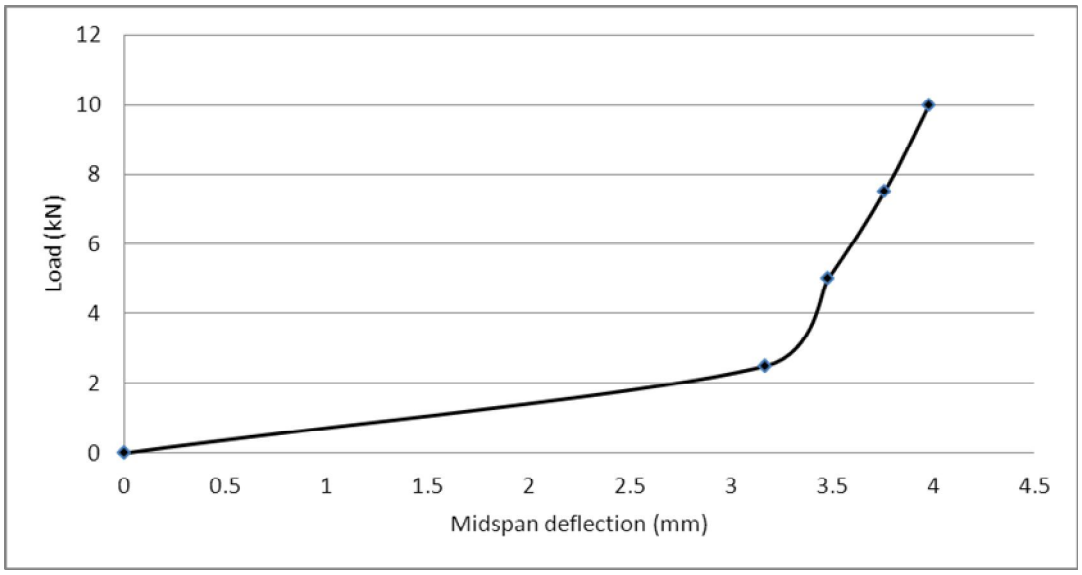


Figure 4.19 Load – midspan deflection of plain concrete beam B08

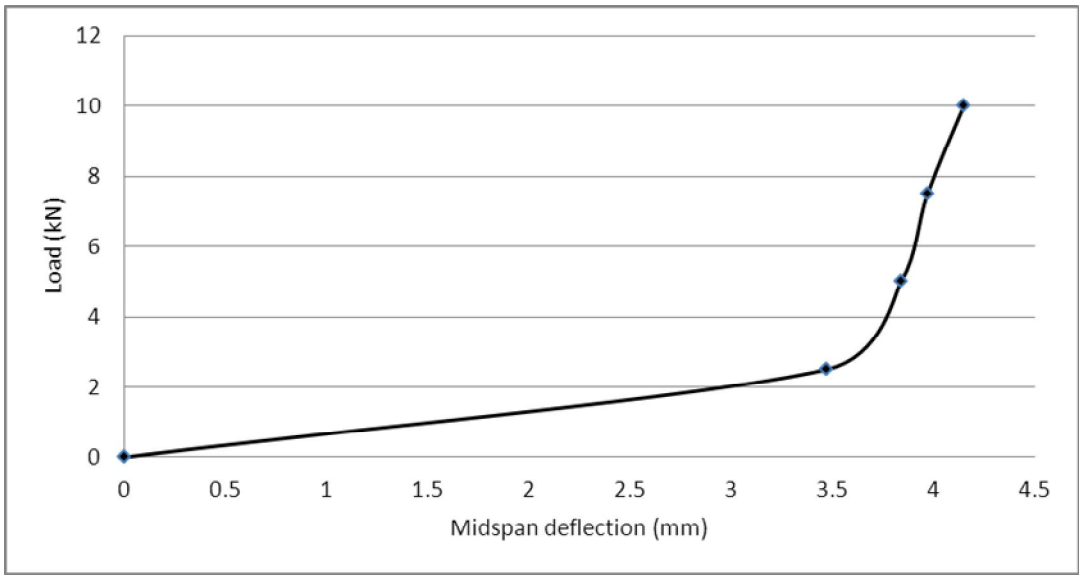


Figure 4.20 Load – midspan deflection of plain concrete beam B09

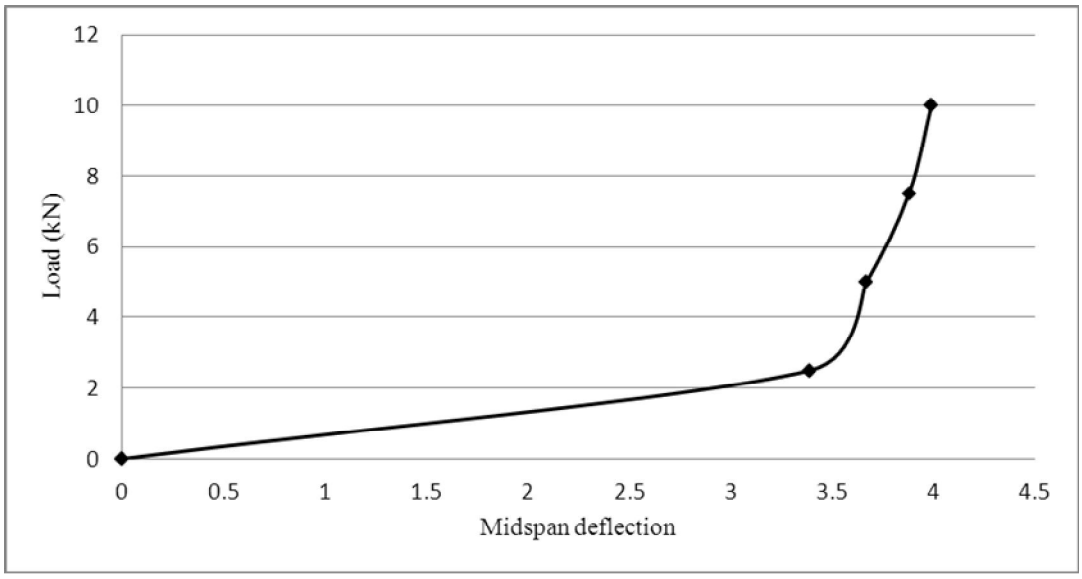


Figure 4.21 Load – midspan deflection of plain concrete beam, B10

Figures 4.17 to 4.21 show the load – midspan deflection shape of plain concrete beam specimen of B06 to B10, up to failure load. It can be observed that the shape of the figure is approximately linear up to a load of 2.5 kN for the plain concrete beams. Thus it can be said that the load mid-span deflection for plain concrete beam is of linear proportion up to 2.5 kN load. The corresponding mid-span deflection at this load is approximately 3.4 millimeter.

Beyond this load, the load – midspan deflection from these figures are non-linear with increase in load with little or no increase in the corresponding deflection. Thus internal cracks within the beams are being formed absorbing the load and with little or no deflection. The average failure load at mid-span for plain concrete beam of B06 to B10 is 10 kN with a corresponding deflection of 4.05 millimeter.

The combined load – midspan deflection plot for all the plain concrete beams of B06 to B10 is as shown in figure 4.22 below.

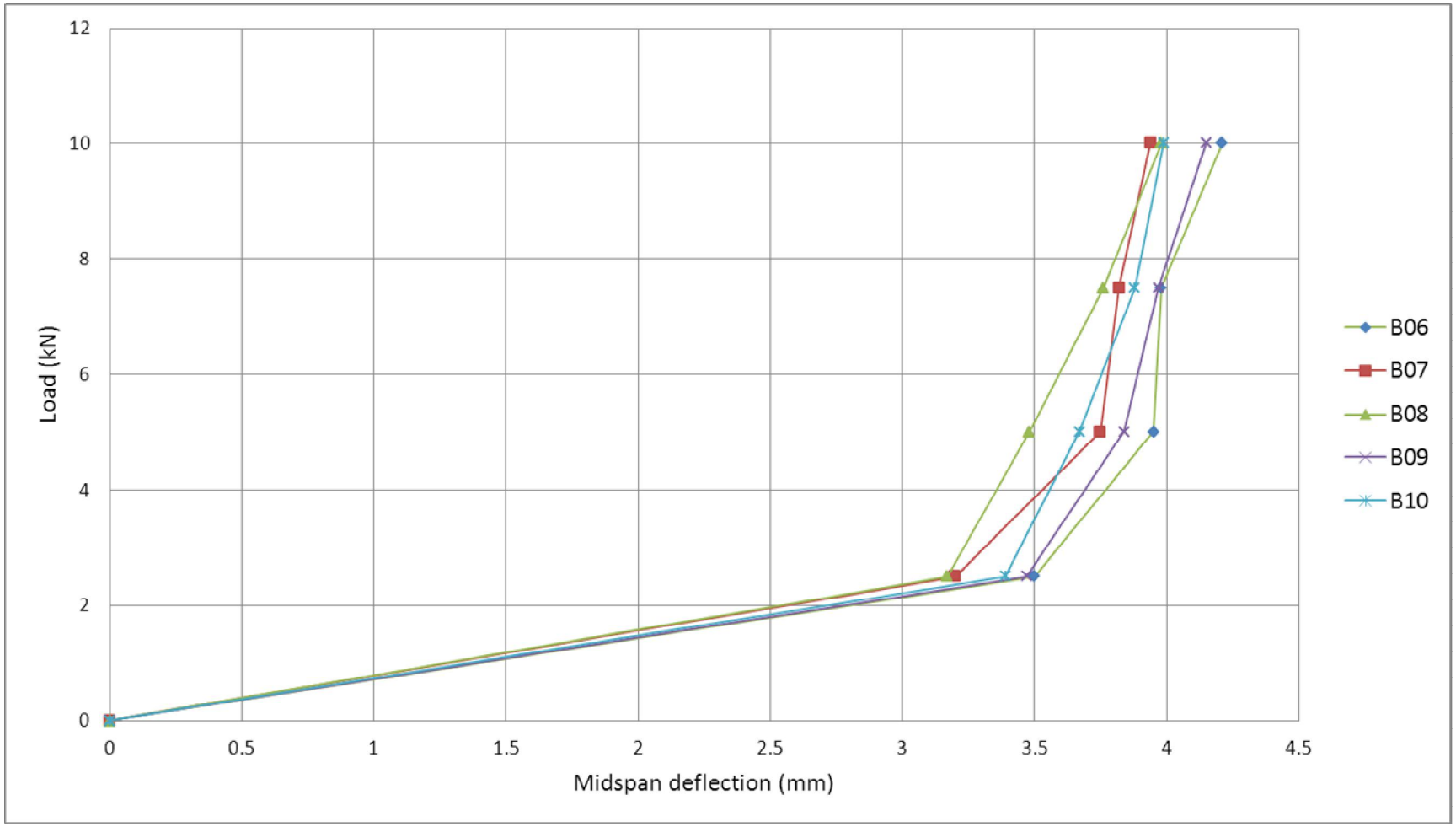


Figure 4.22 Load – midspan deflection of plain concrete beams of B06 to B10

Figures 4.23 to 4.27 below shows the shape of the load – midspan deflection curves for the plain latecrete beams of B16 to B20. The data was presented in chapter three as tables 3.46 to 3.50.

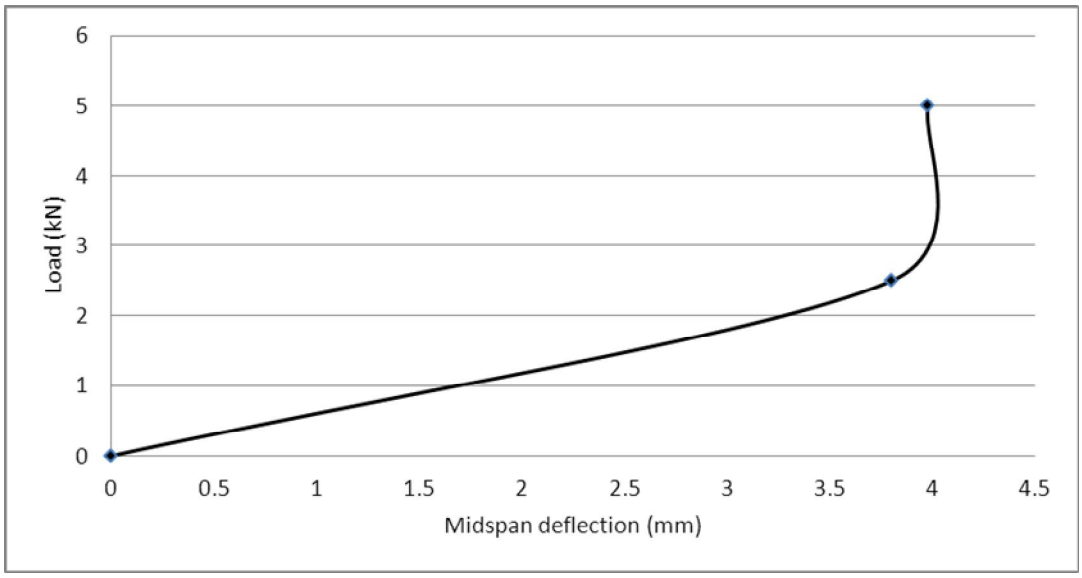


Figure 4.23 Load – midspan deflection of plain latecrete beam, B16

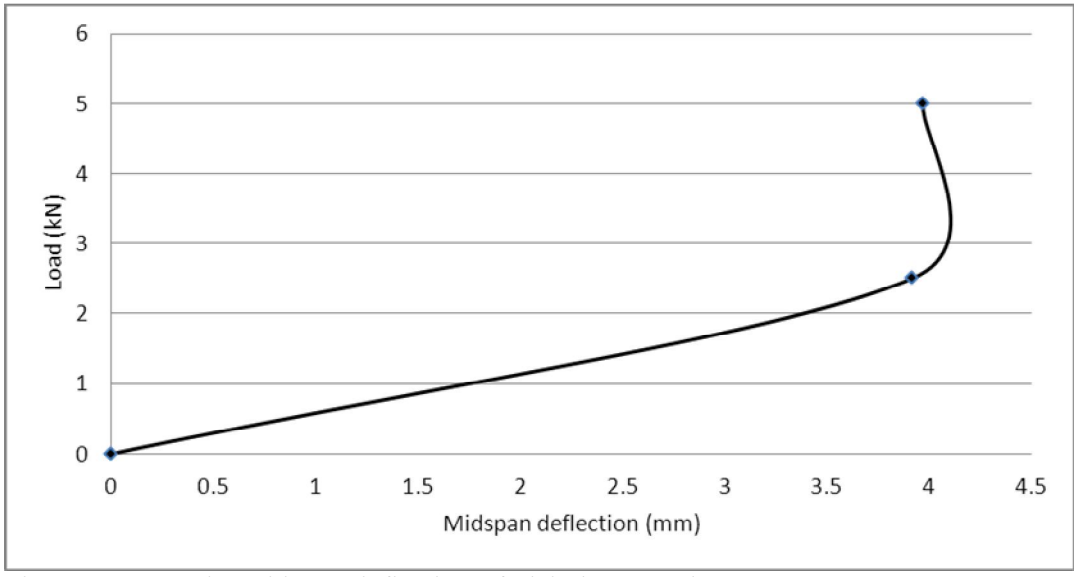


Figure 4.24 Load – midspan deflection of plain latecrete beam, B17

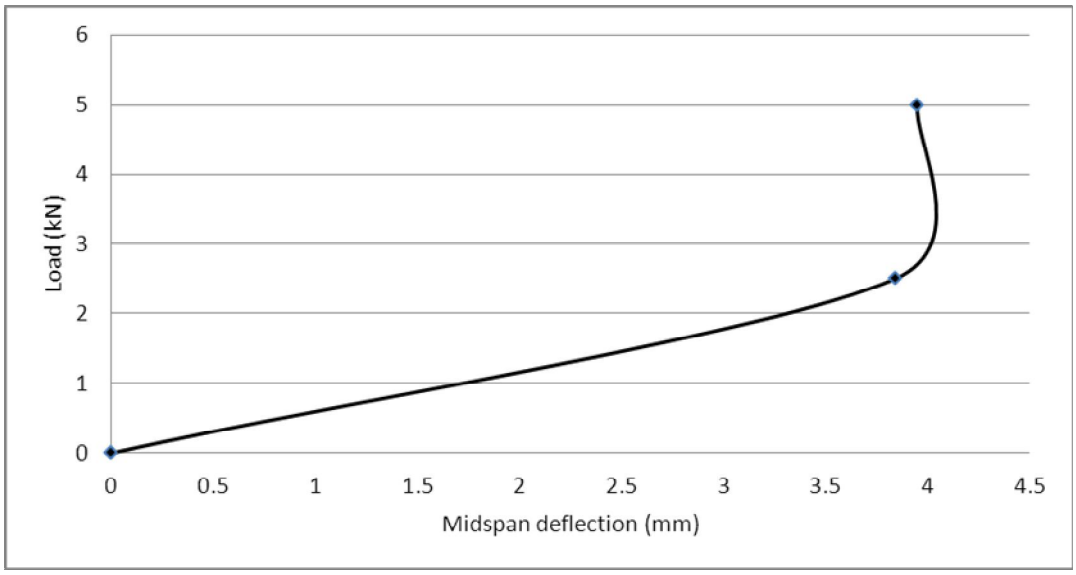


Figure 4.25 Load – midspan deflection of plain latecrete beam, B18

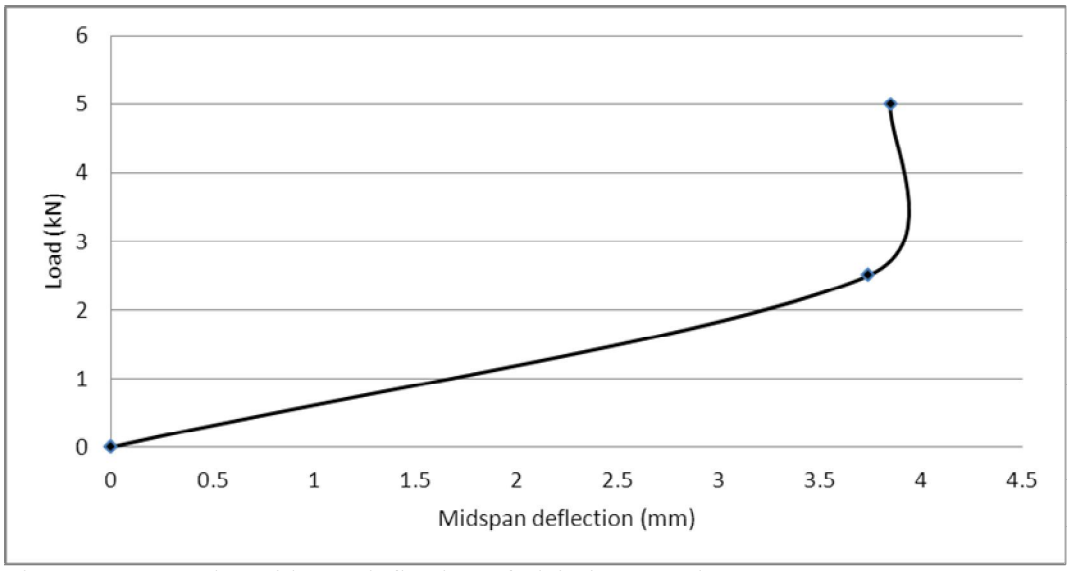


Figure 4.26 Load – midspan deflection of plain latecrete beam, B19

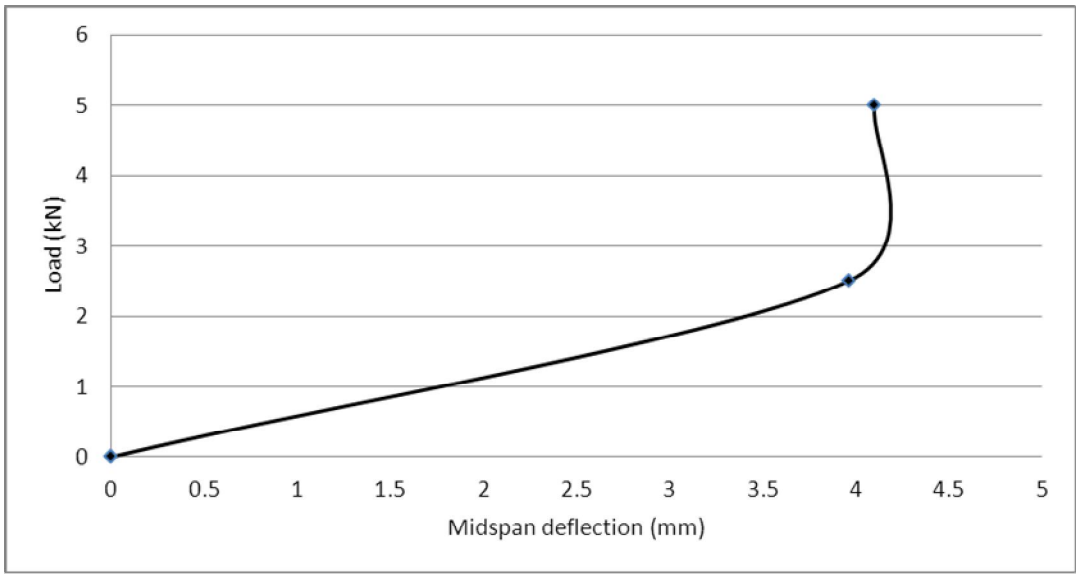


Figure 4.27 Load – midspan deflection of plain latecrete beam, B20

Figures 4.23 to 4.27 show the load – midspan deflection shape of plain latecrete beam specimen of B16 to B20, up to failure load. It can be observed that the shape of the figure is approximately linear up to a load of 2.5 kN for the plain latecrete beams. Thus it can be said that the load mid-span deflection for plain latecrete beam is of linear proportion up to 2.5 kN load. The corresponding mid-span deflection at this load is approximately 3.8 millimeter.

Beyond this load, the load – midspan deflection from these figures are non-linear with increase in load with little or no increase in the corresponding deflection. Thus internal cracks within the beams are being formed absorbing the load and with little or no deflection. The average failure load at mid-span for plain latecrete beam of B16 to B20 is 5 kN with a corresponding deflection of 3.97 millimeter.

The combined load – midspan deflection plot for all the plain latecrete beams of B16 to B20 is as shown in figure 4.28 below.

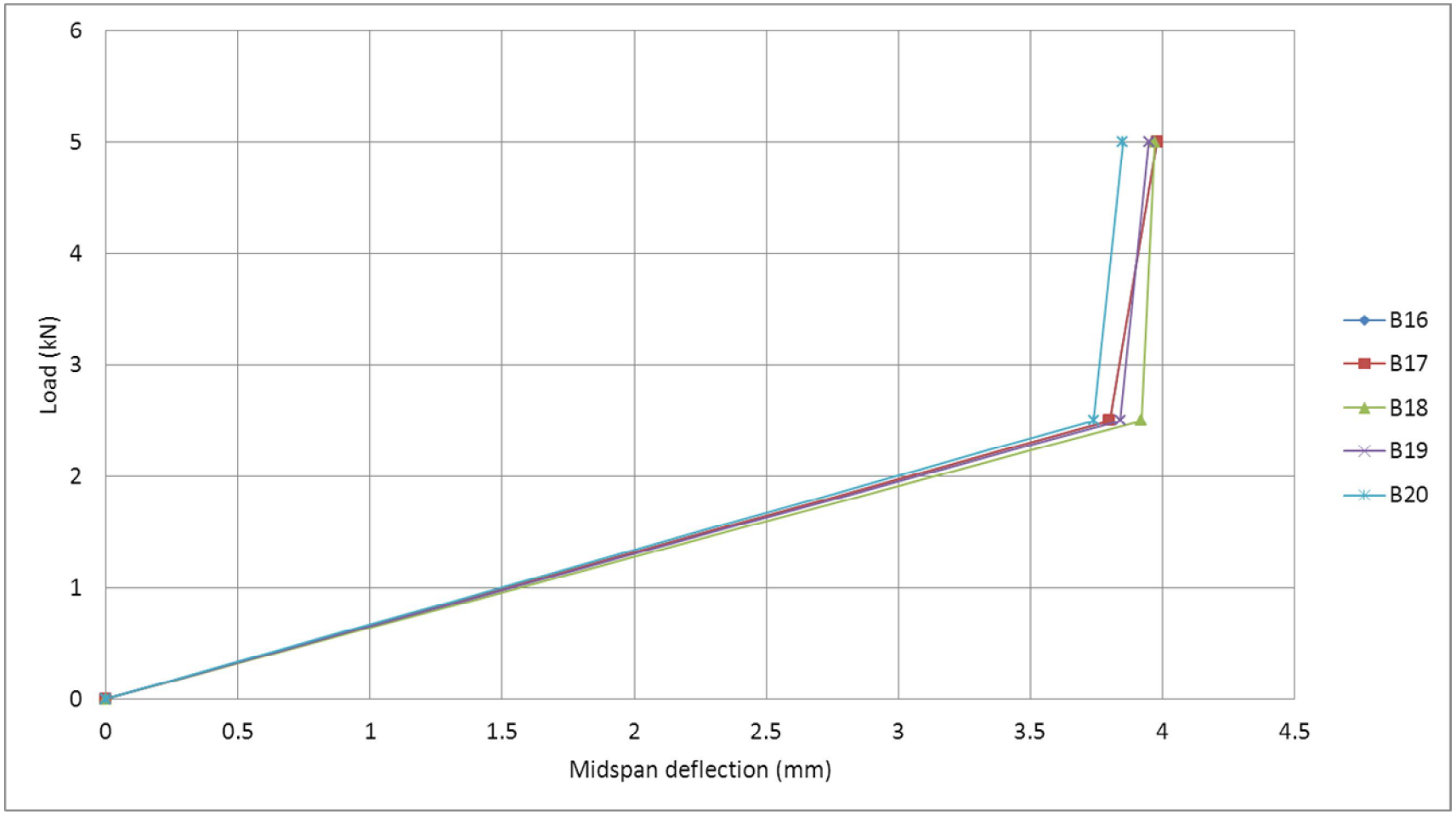


Figure 4.28 Load – midspan deflection of plain latecrete beams of B16 to B20

The discussions on the load – midspan deflection characteristics of the reinforced concrete beams and reinforced latecrete beams above is summarized in table 4.10 below.

Table 4.10 Approximate values of average Elastic load, average midspan deflection at elastic load, average failure load and average midspan deflection at failure load for plain concrete beams and plain latecrete beams

S/No.	Beam No.	Approximate average Elastic load (kN)	Approximate average midspan deflection at Elastic load (mm)	Approximate average failure load (kN)	Approximate average midspan deflection at failure load (mm)
01	B06 – B10	5	3.4	10	4.05
02	B16 – B20	2.5	3.8	5	3.97
03	% of latecrete beam/concrete beam	50	112	50	100

Even though the plain concrete beams exhibit higher load at failure, i.e 100 percent, the midspan deflection at failure is the same. This implies that plain concrete beams are more ductile than the plain latecrete beams.

(c). Stress - strain characteristics of reinforced concrete and reinforced latecrete beam specimens at midspan

Figures 4.29 to 4.33 below shows the graphical representation of the stresses (as obtained in Appendix A27) and the measured strains using SG1 to SG5 for the reinforced concrete beam specimens, B01 to B05 (presented in Tables 3.31 to 3.35 of Chapter three).

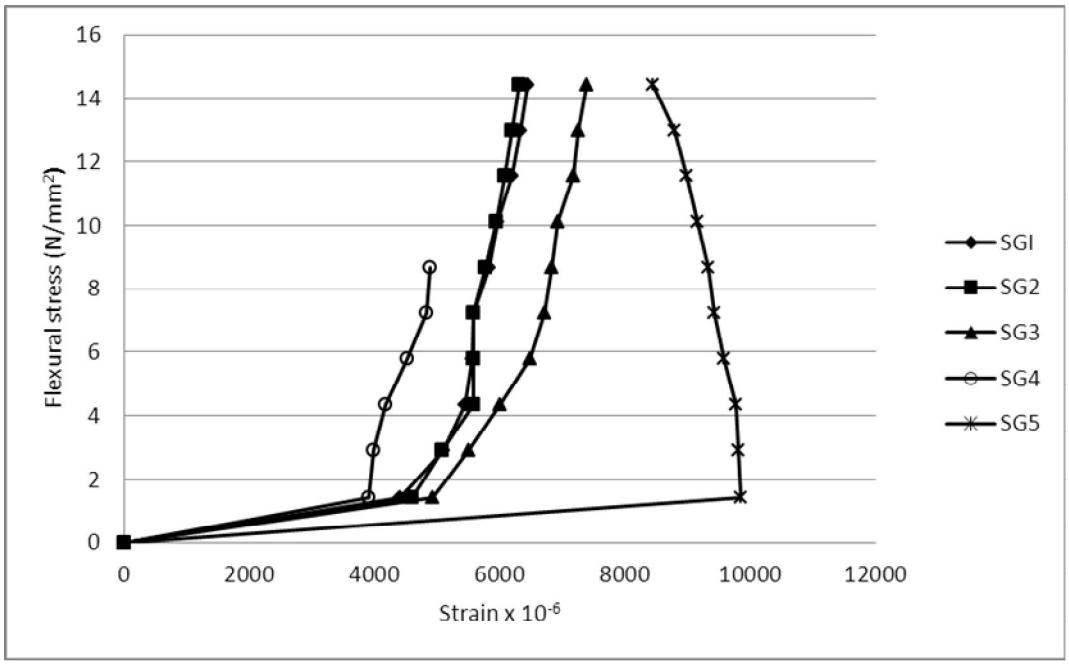


Figure 4.29 Flexural stress – strain plot of reinforced concrete beam, B01

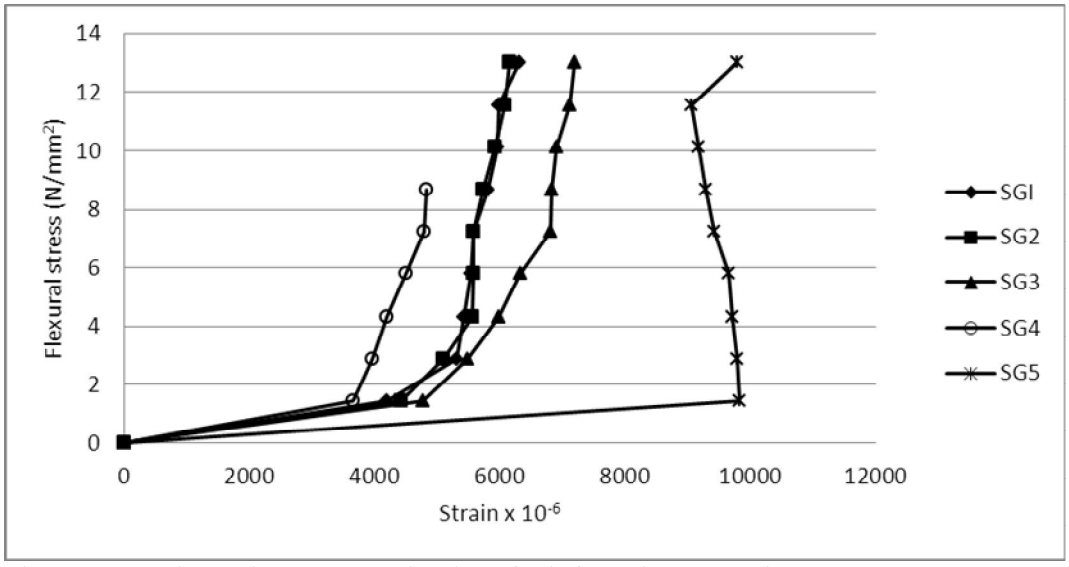


Figure 4.30 Flexural stress – strain plot of reinforced concrete beam, B02

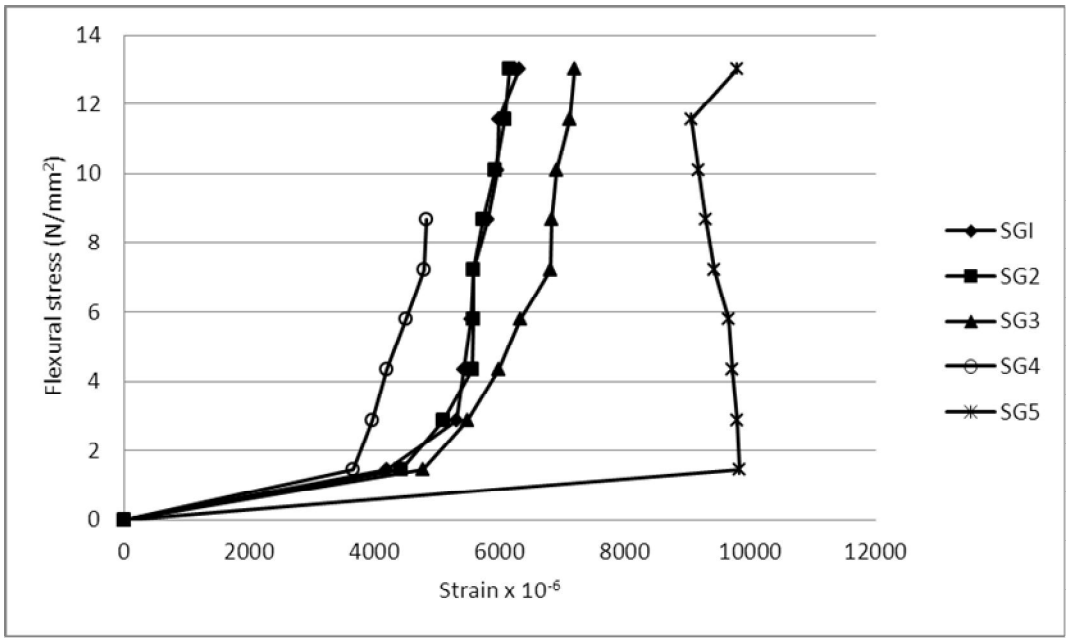


Figure 4.31 Flexural stress – strain plot of reinforced concrete beam, B03

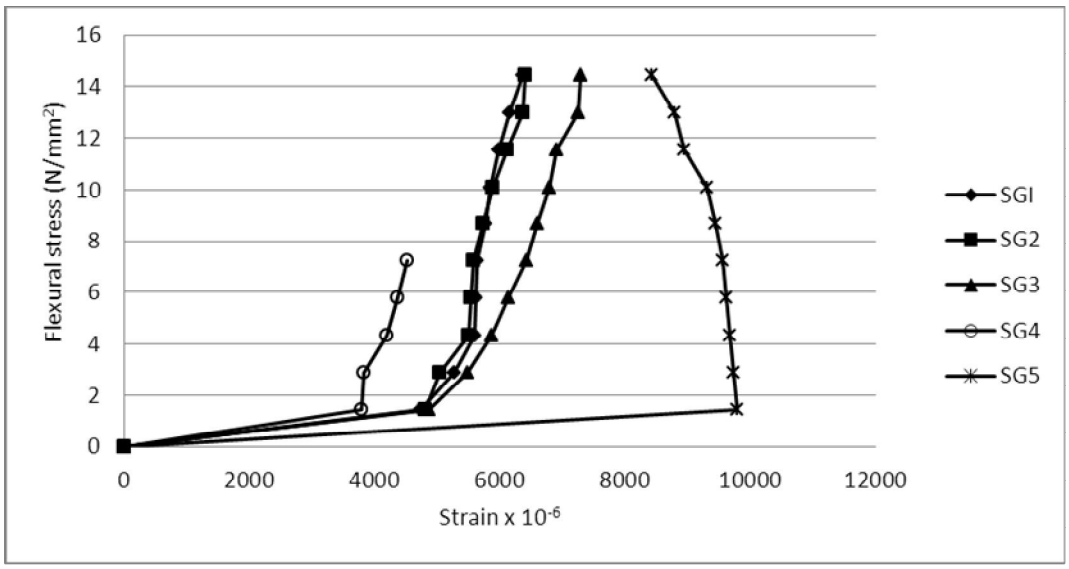


Figure 4.32 Flexural stress – strain plot of reinforced concrete beam, B04

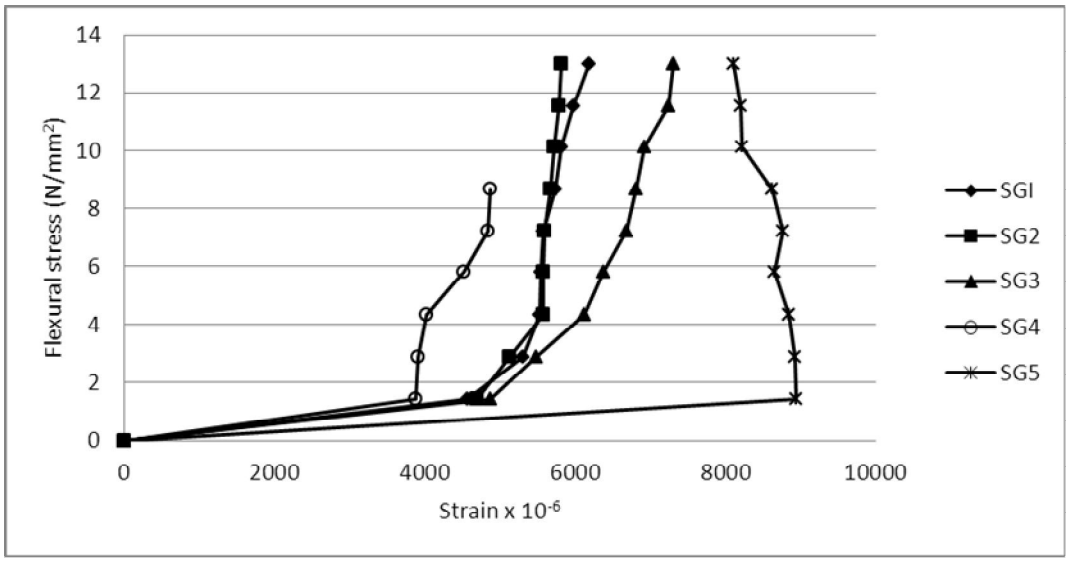


Figure 4.33 Flexural stress – strain plot of reinforced concrete beam, B05

Similarly, figures 4.34 to 4.38 below shows the graphical representation of the flexural stresses as obtained in Appendix A27 and the measured strains using SG1 to SG5 for the reinforced latecrete beam specimens of B11 to B16 presented in tables 3.41 to 3.45 of Chapter three.

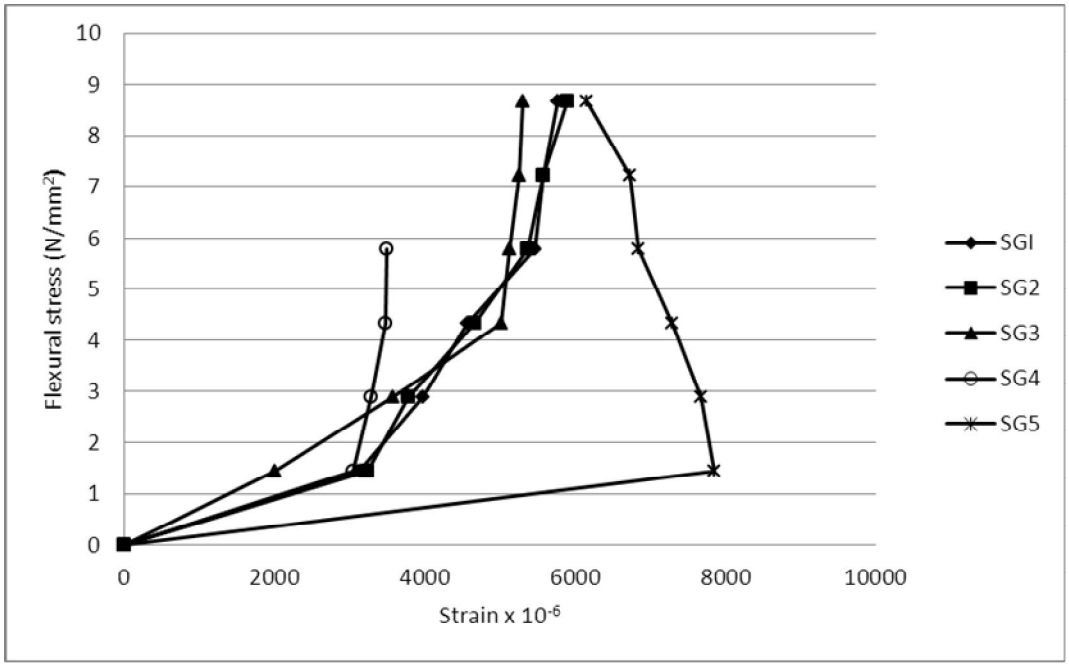


Figure 4.34 Flexural stress – strain plot of reinforced latecrete beam, B11

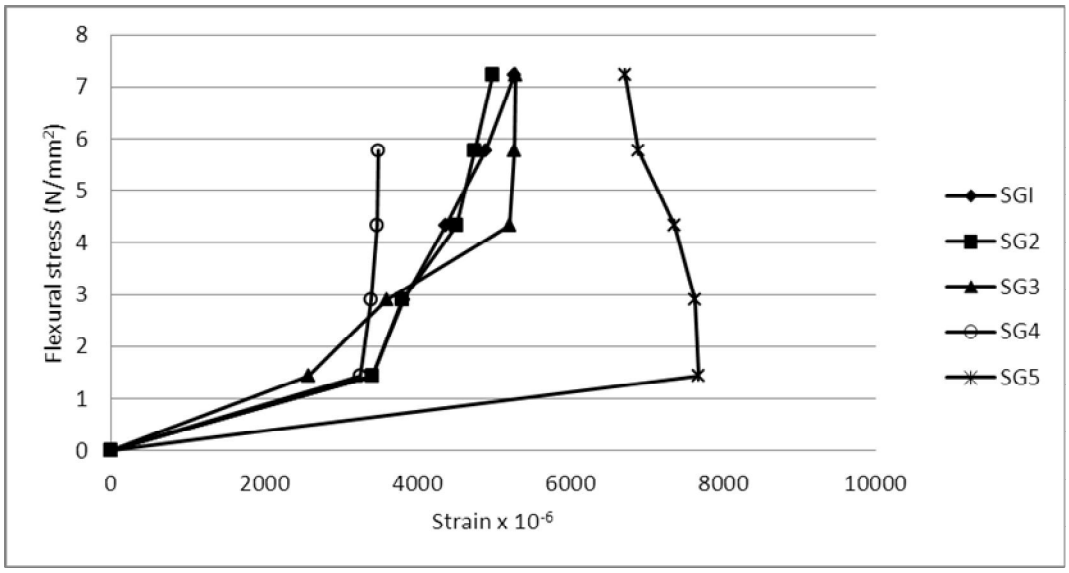


Figure 4.35 Flexural stress – strain plot of reinforced latecrete beam, B12

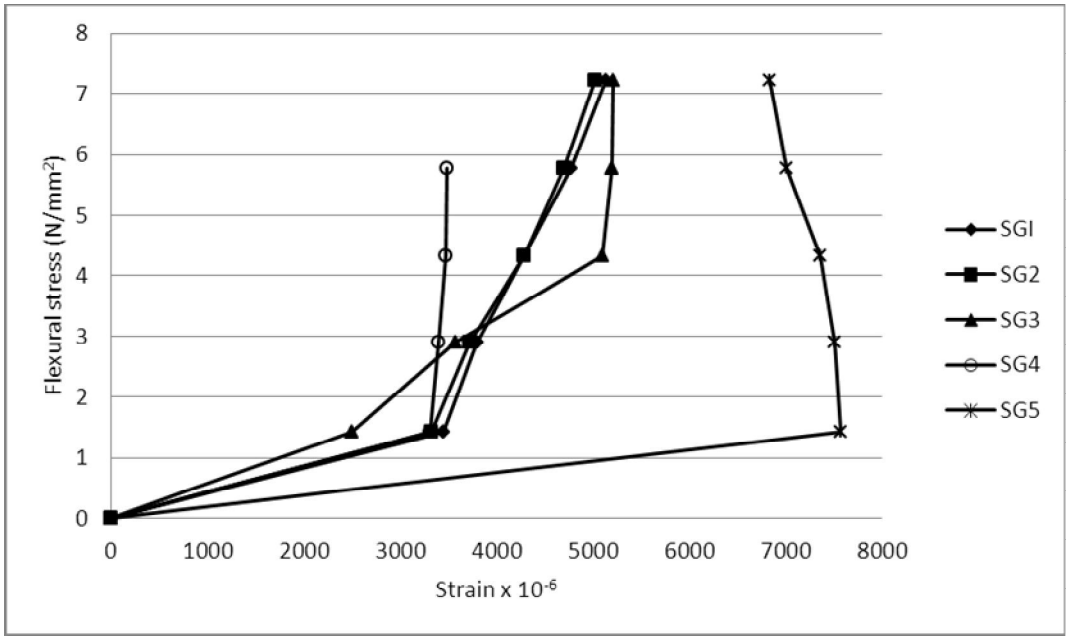


Figure 4.36 Flexural stress – strain plot of reinforced latecrete beam, B13

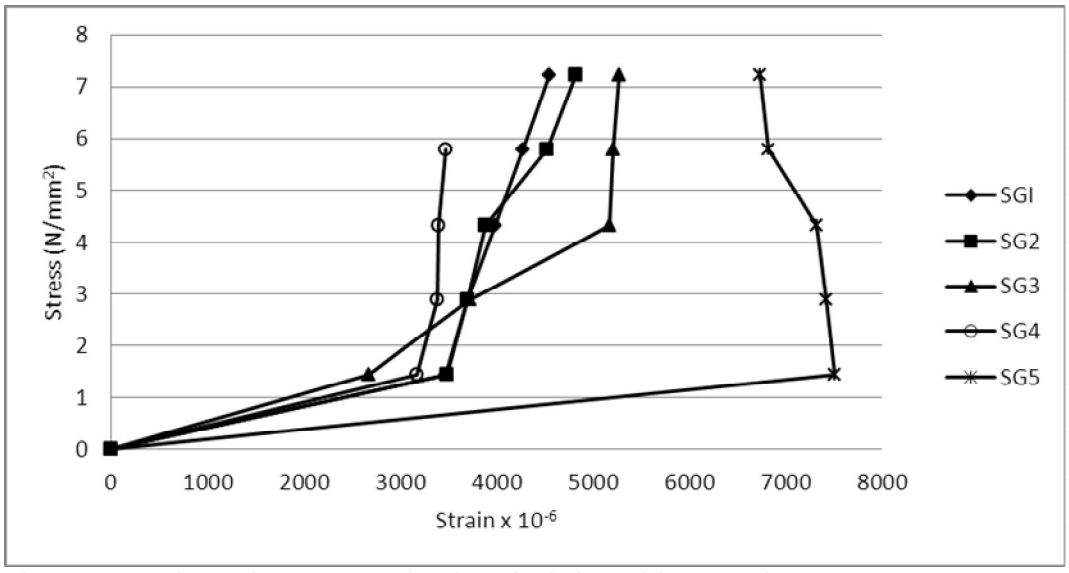


Figure 4.37 Flexural stress – strain plot of reinforced latecrete beam, B14

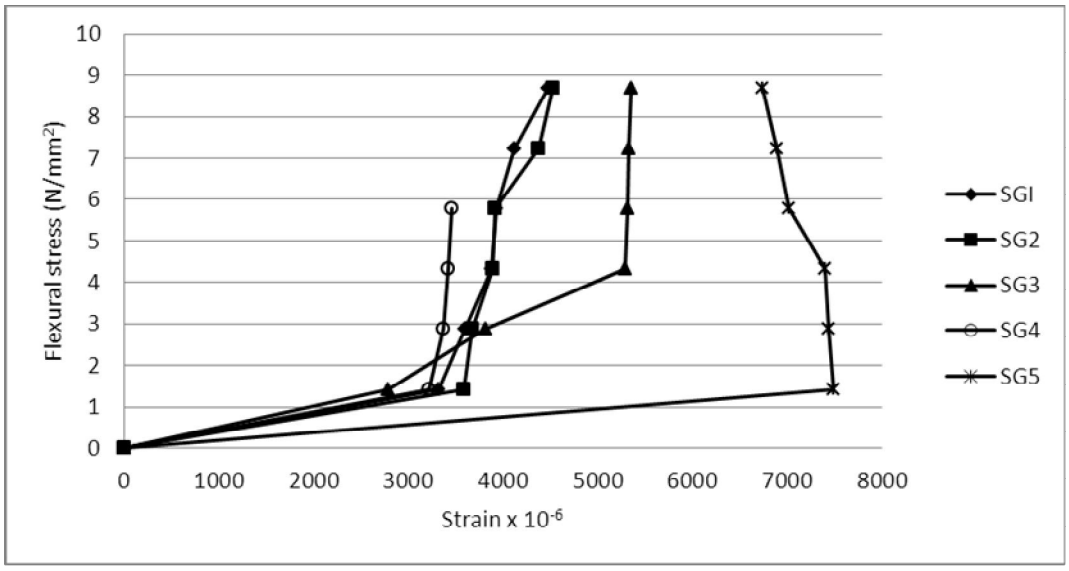


Figure 4.38 Flexural stress – strain plot of reinforced latecrete beam, B15

From figures 4.29 to 4.38, the usual shapes of parabolic nature are maintained for all the beams. Generally, for the Reinforced Concrete beams, SG5 (top outside fibre compressive strain) maintains the highest value than the rest strain values. This is followed by SG3 (top inside bar compressive strain). SG1 (bottom inside bar tensile strain –first bar) and SG2 (bottom inside bar tensile strain – second bar) steel strains are

virtually the same and next to the lowest value of SG4 (bottom outside fibre tensile strain). However, the value of SG3 is lower than that of SG1 and SG2 in the reinforced latecrete beams of B11 and B12.

From figures 4.29 to 4.33 above, for the reinforced concrete beam of B01 to B05, SG5 has the highest average strain value of 9622 x 10^{-6}, followed by SG3's average value of 7224 x 10^{-6}, followed by the virtually the same average values of SG1 and SG2 of 6338 x 10^{-6} and 6187.6 x 10^{-6} respectively and the lowest value for SG4 of 4746 x 10^{-6}. Similarly, in figure 4.34 to 4.38, for the reinforced latecrete beam of B11 to B15, SG5 has the highest average strain value of 7618.4 x 10^{-6}, followed by SG3's value of 5283 x 10^{-6}, followed by the virtually the same average values of SG1 and SG2 of 5040 x 10^{-6} and 5053 x 10^{-6} respectively, and the lowest average value for SG4 of 3481.8 x 10^{-6}.

From figures 4.29 to 4.38 above, for the reinforced concrete beam of B01 to B05, the flexural modulus, which is the slope of the curve at the elastic region is 3.78 x 10^5 N/mm^2 while the flexural modulus of the reinforced latecrete beam of B11 to B16 is 4.50 x 10^5 N/mm^2.

(d). Stress - strain characteristics of plain concrete and plain latecrete beam specimens at midspan

Figures 4.39 to 4.48 below shows the plot of the flexural stress as calculated in Appendix A27 against the top fibre and bottom fibre strain gauges (SG4 and SG5), for the plain concrete beam and plain latecrete beam specimens. The data are presented tables 3.36 to 3.40 and 3.46 to 3.50 of Chapter three. The figures mentioned above are presented below (figures 4.39 to 4.48).

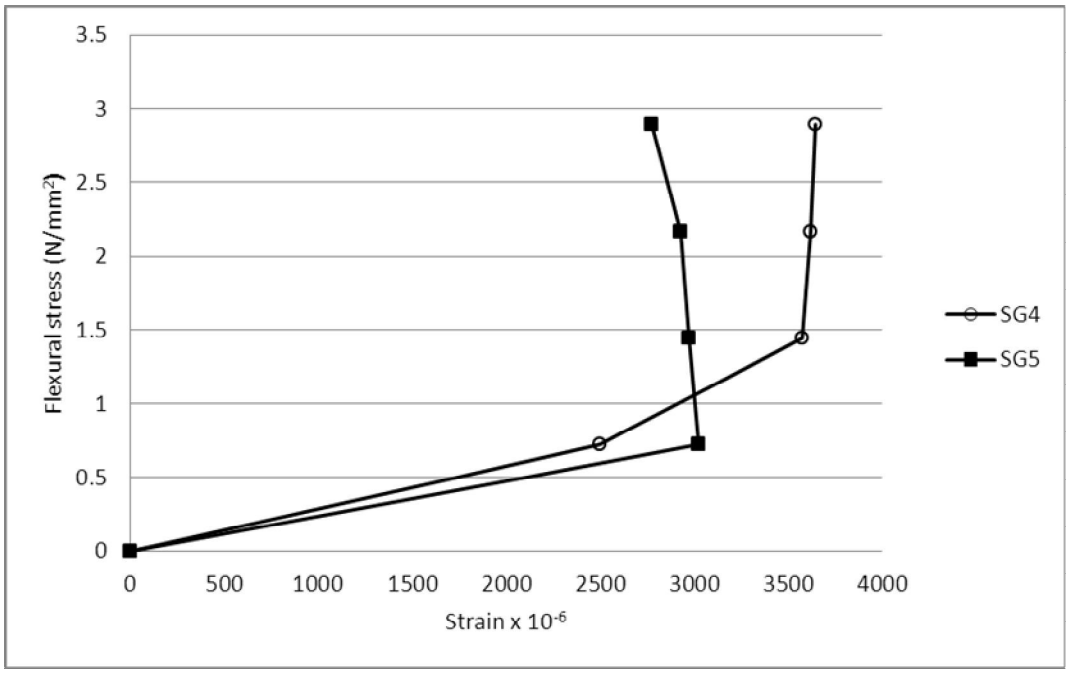


Figure 4.39 Flexural stress – strain plot of plain concrete beam specimen, B06

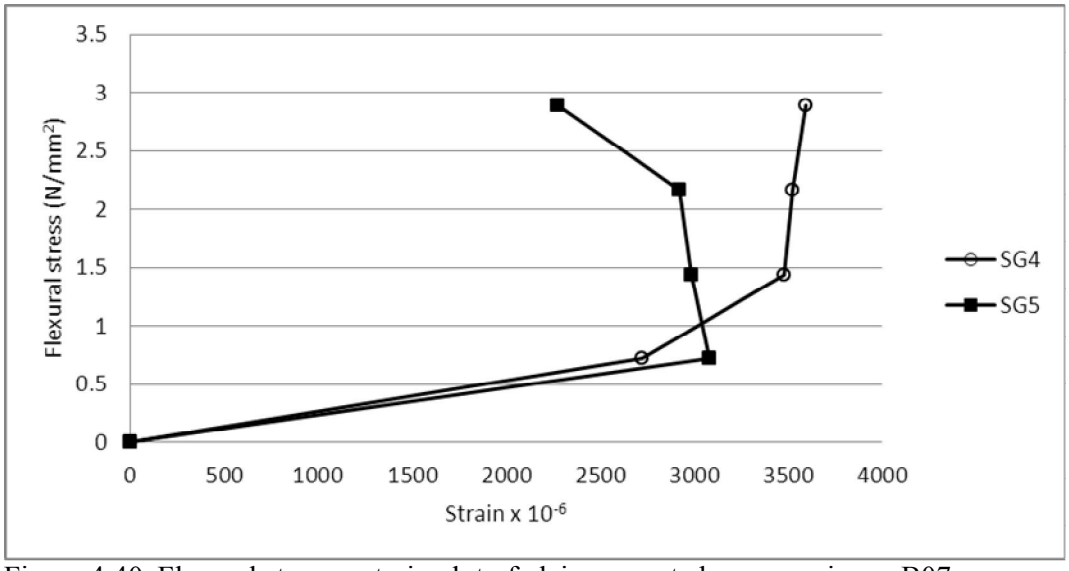


Figure 4.40 Flexural stress – strain plot of plain concrete beam specimen, B07

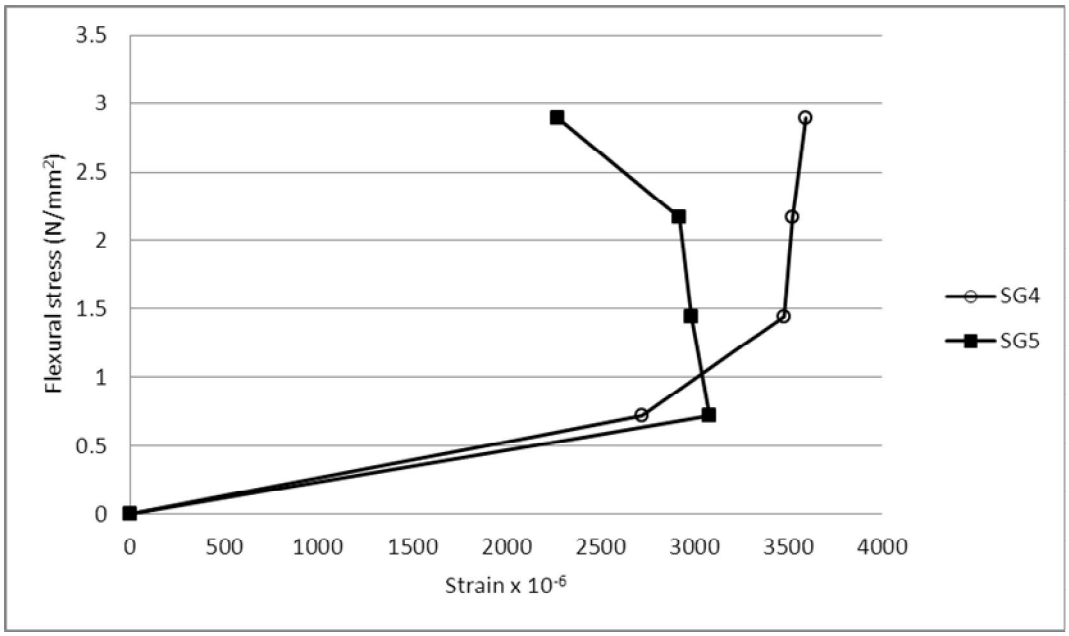


Figure 4.41 Flexural stress – strain plot of plain concrete beam specimen, B08

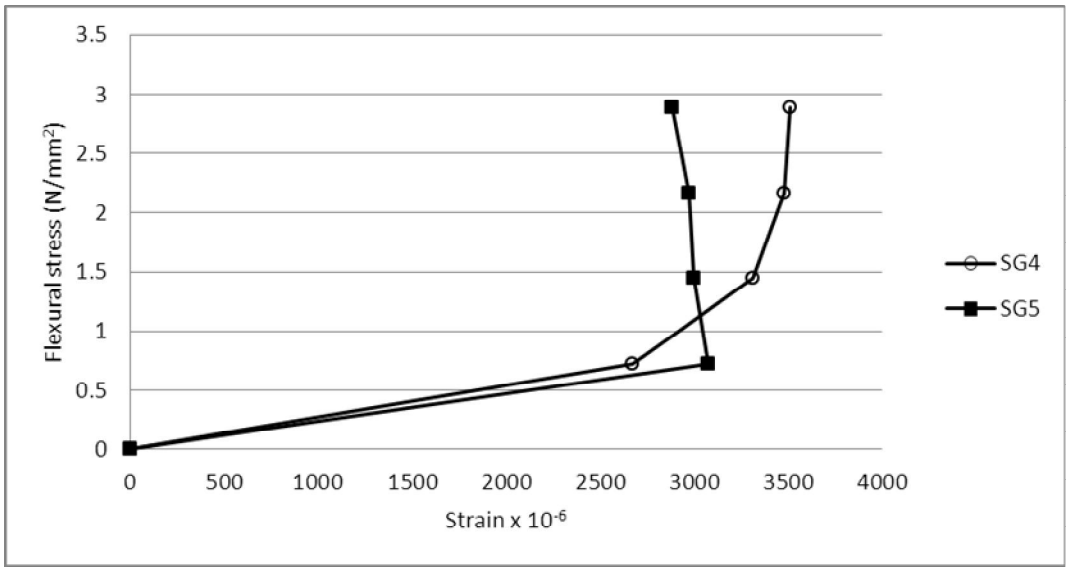


Figure 4.42 Flexural stress – strain plot of plain concrete beam specimen, B09

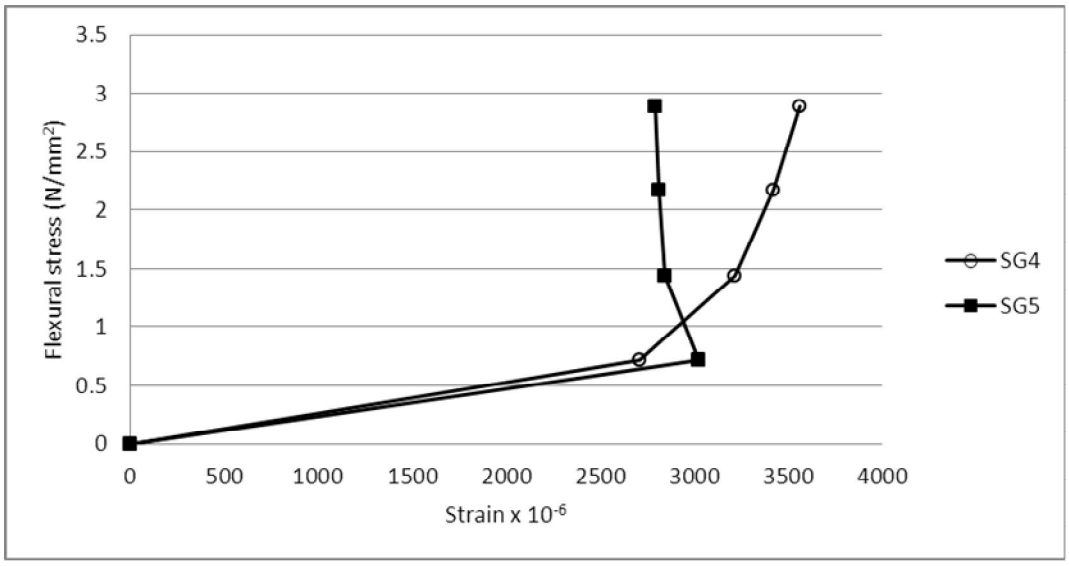


Figure 4.43 Flexural stress – strain of plain concrete beam specimen, B10

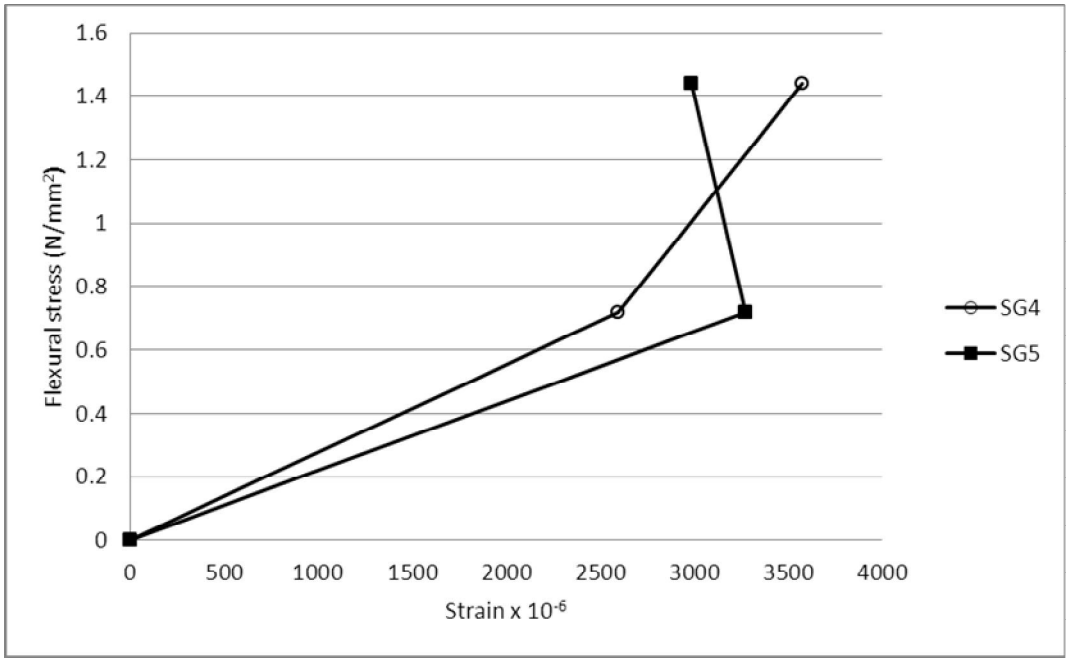


Figure 4.44 Flexural stress – strain plot of plain latecrete beam specimen, B16

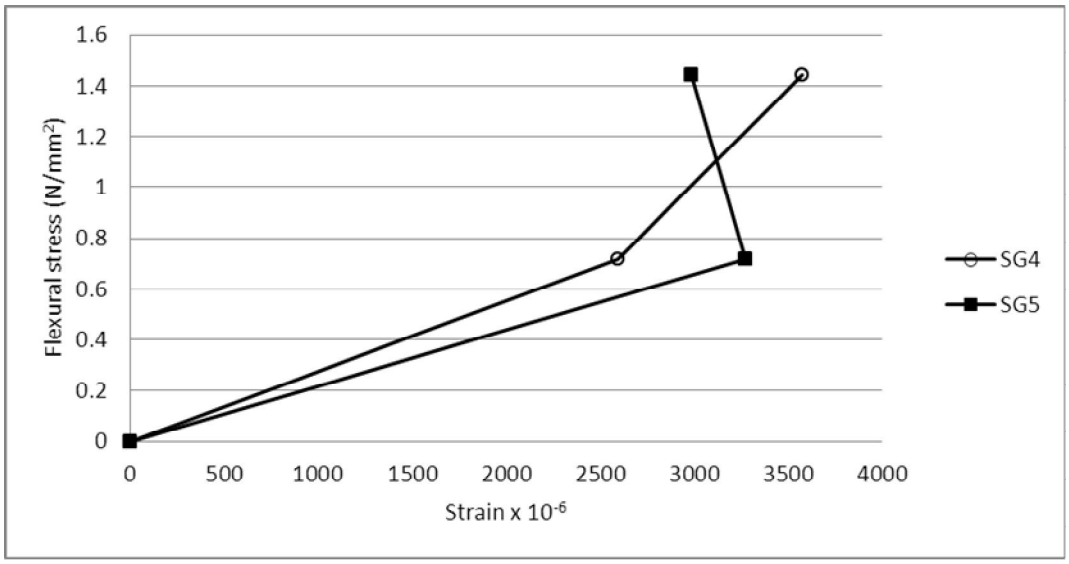


Figure 4.45 Flexural stress – strain plot of plain latecrete beam specimen, B17

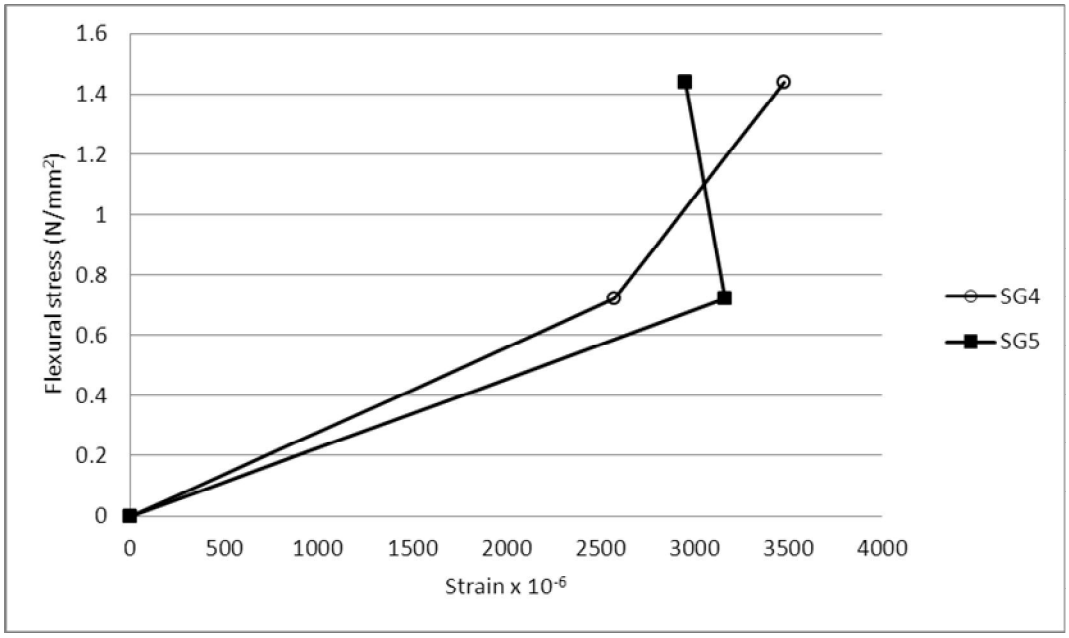


Figure 4.46 Flexural stress – strain plot of plain latecrete beam specimen, B18

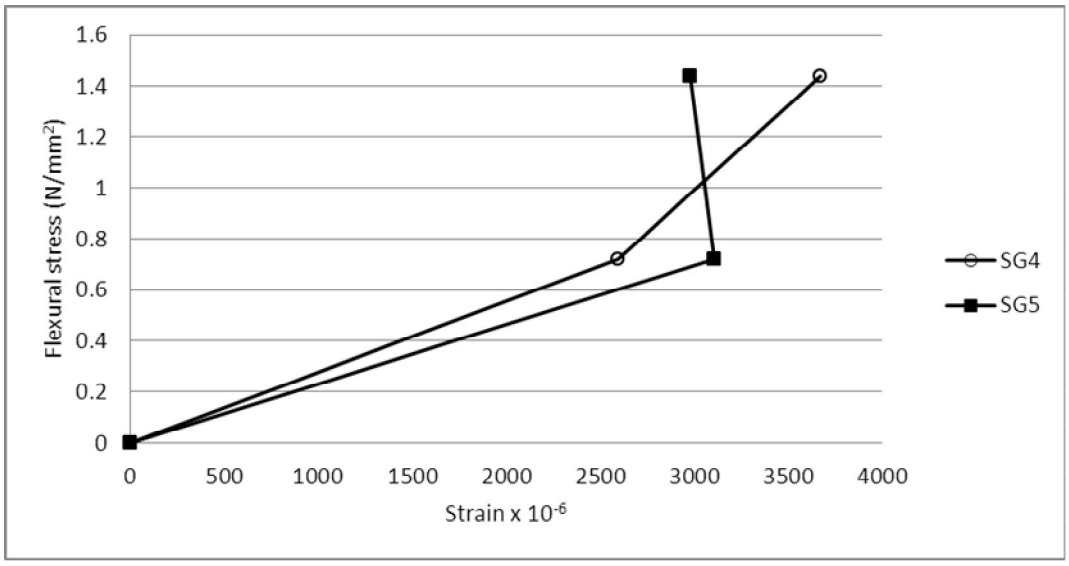


Figure 4.47 Flexural stress – strain plot of plain latecrete beam specimen, B19

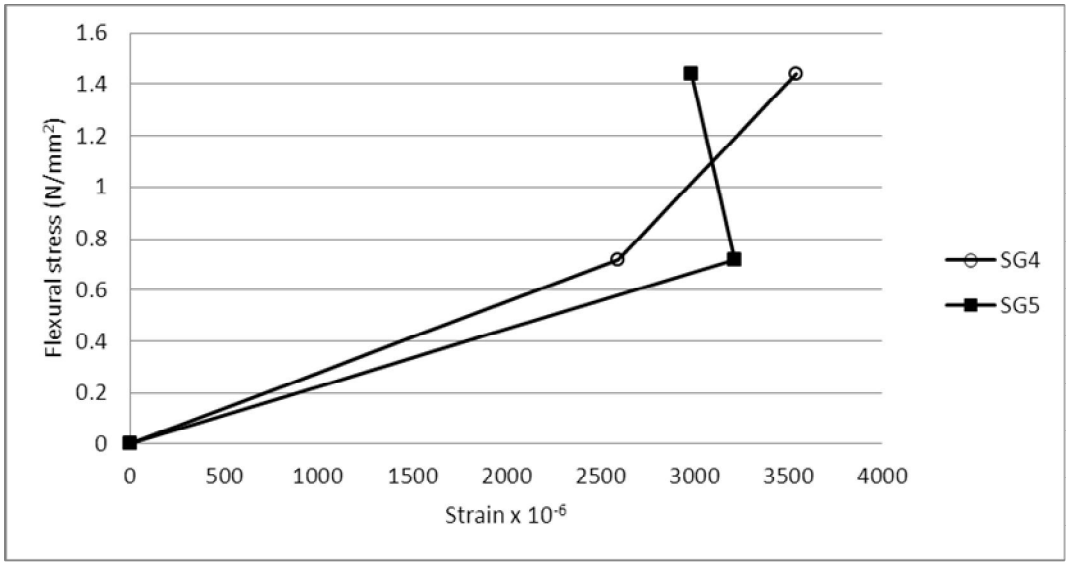


Figure 4.48 Flexural stress – strain plot of plain latecrete beam specimen, B20

From figures 4.39 to 4.43 above, for the plain concrete beam specimens, it is observed that the bottom outside fibre tensile strain have the highest average value of 3577.4 x 10^{-6} and the top outside fibre compressive strain have the lowest average value of 3066.4 x 10^{-6}. Similarly, from figures 4.44 to 4.48, for the plain latecrete beam specimens, it is observed that the bottom outside fibre tensile strain have the highest

average value of 3589 x 10^{-6} and the top outside fibre compressive strain have the lowest average value of 3194.6 x 10^{-6}.

From figures 4.39 to 4.48 above, it is observed that at a stress of approximately 1.10 N/mm^2, the strain at both bottom outside fibre tensile strain and top outside fibre compressive strain is the same value of 3250 x 10^{-6} for plain concrete beam. For plain latecrete beam, at the stress of approximately 1.0 N/mm2, the strains gave the same value of 3250 x 10^{-6}. Thus at approximately a stress value of 1.0 N/mm^2, the strain is the same value of 3250 x 10^{-6}, for both plain concrete beams and plain latecrete beams.

From figures 4.39 to 4.48 above, for the plain concrete beam of B06 to B10, the flexural modulus, which is the slope of the curve of the figures in the elastic region, is 2.70 x 10^5 N/mm^2 while the flexural modulus of the plain latecrete beam of B16 to B20 is 2.79 x 10^5 N/mm^2. Thus the flexural modulus of the plain concrete and latecrete beams is approximately the same value of 2.70 x 10^5 N/mm^2.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

The following conclusions were drawn from this research work:

1. The Cement, sand and coarse aggregates used in this research complies with the code specifications and thus good quality for concrete making.
2. The laterite used is classified as A-7-6 according to AASHTO soil classification system and low plasticity clay (CL) according to the Unified soil classification system and not economical for stabilization.
3. The high yield reinforcement used with a characteristic strength of 394 N/mm^2 indicates that it must have been a retreaded mild steel material..
4. The 28 days cube compressive strength of Concrete is 23.26 N/mm^2 and the 28 days cube compressive strength of Latecrete is 8.89 N/mm^2.
5. The tensile strength of concrete is 1.82 N/mm^2 and the tensile strength of latecrete is 0.67 N/mm^2.
6. The flexural strength of reinforced concrete beams is 13.58 N/mm^2 and the flexural strength of reinforced latecrete beams is 7.80 N/mm^2. The flexural strength recorded for the plain concrete beams is 2.89 N/mm^2, while that of the of plain latecrete beams is 1.44 N/mm^2.
7. The concrete beam specimens are more ductile than the latecrete beam specimens.
8. The top outside fibre compressive strain, SG5, of 9622 x 10^{-6} is the highest strain value for all the reinforced concrete beam specimens and 7618.4 x 10^{-6} for reinforced latecrete beams.

9. The bottom outside fibre tensile strain, SG4, of 4746 x 10^{-6} is the lowest strain value for all the reinforced concrete beam specimens and 3481.8 x 10^{-6} for reinforced latecrete beams.

10. The bottom outside fibre tensile strain, SG4, of 3577.4 x 10^{-6} is the highest strain value, at failure load, for all the plain concrete beam specimens and 3589 x 10^{-6} for plain latecrete beams.

11. The top outside fibre compressive strain of 3066.4 x 10^{-6} is the lowest strain value, at failure load, for all the plain beam specimens and 3194.6 x 10^{-6} for plain latecrete beams.

12. At about 1.0 N/mm^2 value of stress, the strain is equal (3250 x 10^{-6}) for all the plain concrete beams and plain latecrete beams.

13. The load-deflection behaviour of the beams are essentially linear within the elastic range of loading and the measured plots of the stress and strain graph shows that the flexural modulus of the reinforced concrete beams was 3.78 x 10^5 N/mm^2, while that of the reinforced latecrete beams was 4.50 x 10^5 N/mm^2. For both plain concrete and latecrete beams, the flexural modulus was the same value of 2.70 x 10^5 N/mm^2.

5.2 RECOMMENDATION

From the findings of this research work, the following recommendations are hereby made:

1. Government should encourage further research on latecrete.

2. There is need for formulation of guidelines for latecrete production, in order to have a uniform basis for comparison of experimental results.

3. More experimental study with large scale modeling is to be conducted before a Code recommendation can be made for latecrete.

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APPENDIX

Appendix A1 Fineness of Cement

	Test 1	Test 2	Test 3
Sample No.	F-01	F-02	F-03
Total weight of sample (g)	50	60	70
Weight of sample passing sieve 75µ (g)	48.5	57.5	65.0
Weight retained	1.5	2.5	5.0
% weight retained	3.0	4.0	7.0
Fineness	0.03	0.04	0.07

Appendix A2 Consistency of Cement

	Test 1	Test 2	Test 3
Sample No.	C-01	C-02	C-03
Weight of cement sample (g)	400	400	400
Quantity of water added (ml)	124	124	124
Depth of penetration (mm)	6.0	7.0	6.5
Standard Consistency (%)	31	31	31

Appendix A3 Initial and final setting time of Cement

	Test 1	Test 1	Test 2
Sample No.	ST-01	ST-02	ST-03
Time test started	8:32 am	11:50 am	3:20 pm
Time at Initial setting	9:52 am	1:14 pm	4:36 pm
Initial setting time (minutes)	**80**	**84**	**76**
Time at Final setting	11:27 am	2:50 pm	6:15 pm
Final setting time (minutes)	**175**	**160**	**175**

Appendix A4 Soundness of Cement

	Test No. 1	Test No. 2	Test No.3
Sample No.	S-01	S-02	S-03
Distance between the pointer before heating(mm)	3.0	4.0	3.0
Distance between the pointer after heating (mm)	4.0	5.0	4.0
Soundness: – Expansion (mm)	**1.0**	**1.0**	**1.0**

Appendix A5 Mortar cube strength test of cement

Sample Identification No.	Age (days)	Mass (kg)	Volume x 10^{-4} (m^3)	Density x 10^3 (kg/m^3)	Failure Load x10^3 (N)	Compressive Strength N/mm^2
MC-01	7	0.700	3.519	1.989	120	24.08
MC-02	7	0.655	3.519	1.861	102	20.46
MC-03	7	0.70	3.519	1.989	100	20.06

Appendix A6 Particle size distribution of Sand

BS Sieve size (mm)	Weight Retained (g)	% Retained	Cummulative % passing	Cummulative % retained
4.76	0.00	0.00	100	0.00
2.36	40.0	4.00	96	4.00
1.18	220.0	22.00	74	26.00
0.6	385.0	38.50	35.50	64.50
0.425	335.0	33.50	2.00	88.00
0.3	15.0	1.50	0.50	99.50
0.212	5.0	0.50	0.00	0.00
0.15	0.00	0.00	0.00	0.00
0.075	0.00	0.00	0.00	0.00
TOTAL	1000	100		

Appendix A7 Result of Specific Gravity of Sand

	Test No. 1	Test No. 2	Test No. 3
Sample No.	SG-01	SG-02	SG-03
Flask No.	D4	G7	M5
Weight of aggregates, B (g)	500	500	500
Weight of cylinder + water, P	1500	1500	1500
Weight of cylinder + water + aggregates, Ps	1808	1805	1806
Specific gravity, S.G = B/(P+B – Ps)	2.60	2.56	2.58

Appendix A8 Silt content of Sand

DESCRIPTION	SAMPLE SC-01	SAMPLE SC-02	SAMPLE SC-03
Sand layer (ml)	94	92	89
Silt layer (ml)	2.35	2.02	1.78
Silt content (%)	2.50	2.20	2.00

Appendix A9 Particle size distribution of Laterite

BS Sieve size (mm)	Weight Retained (g)	% Retained	Cummulative % passing	Cummulative % retained
4.76	0.00	0.00	100	0.00
2.36	29.8	2.98	97.02	8.98
1.18	13.7	1.37	95.65	4.35
0.6	26.3	2.63	93.02	6.98
0.425	139.3	13.93	79.09	20.91
0.3	60.5	6.05	73.04	26.96
0.212	66.3	6.63	66.41	33.59
0.15	46.0	4.60	61.81	39.19
0.075	51.0	5.10	56.71	43.29

Appendix A10 Determination of specific gravity of soil particles

	Test No. 1	Test No. 2
Sample No.	SG-03	SG-04
Bottle No.	Z5	Z8
Weight of bottle + water (full) (W_4)	94.0	93.8
Weight of bottle + soil + water (W_3)	101.4	99.0
Weight of bottle + soil (W_2)	55.9	52.2
Weight of bottle (W_1)	44.4	43.8
Weight of water in full bottle ($W_4 - W_1$)	49.6	50.0
Weight of water added to soil ($W_3 - W_2$)	45.1	46.8
Weight of soil ($W_2 - W_1$)	11.5	8.4
Weight of water displaced by soil, $W = (W_4 - W_1) - (W_3 - W_2)$	4.5	3.2
Specific gravity of soil particle, $(W_2 - W_1)/W$	2.56	2.63

Appendix A11 Results of Atterberg limit of lateritic soil

TYPE OF TEST	**LIQUID LIMIT**				
No. of blows	41	34	28	18	13
Container No.	44	65	20	M17	M3
Weight of wet soil & container (g)	19.9	21.2	28.5	22.8	25.5
Weight of dry soil & container (g)	17.5	18.4	24.6	19.4	20.9
Weight of container (g)	10.8	11.5	14.7	11.4	11.1
Weight of dry soil (Wd) (g)					
Weight of moisture (Ww) g					
Moisture content (%)	35.8	39.4	39.4	42.5	46.9
TYPE OF TEST	**PLASTIC LIMIT**				
No. of blows					
Container No.	15G	5G	X46		
Weight of wet soil & container (g)	16.8	19.3	17.0		
Weight of dry soil & container (g)	16.1	18.6	16.0		
Weight of container (g)	12.3	14.4	10.2		
Weight of dry soil (Wd) (g)					
Weight of moisture (Ww) g					
Moisture content (%)	18.42	16.67	17.34		

The graph of the liquid limit test is as shown in figure A1 below.

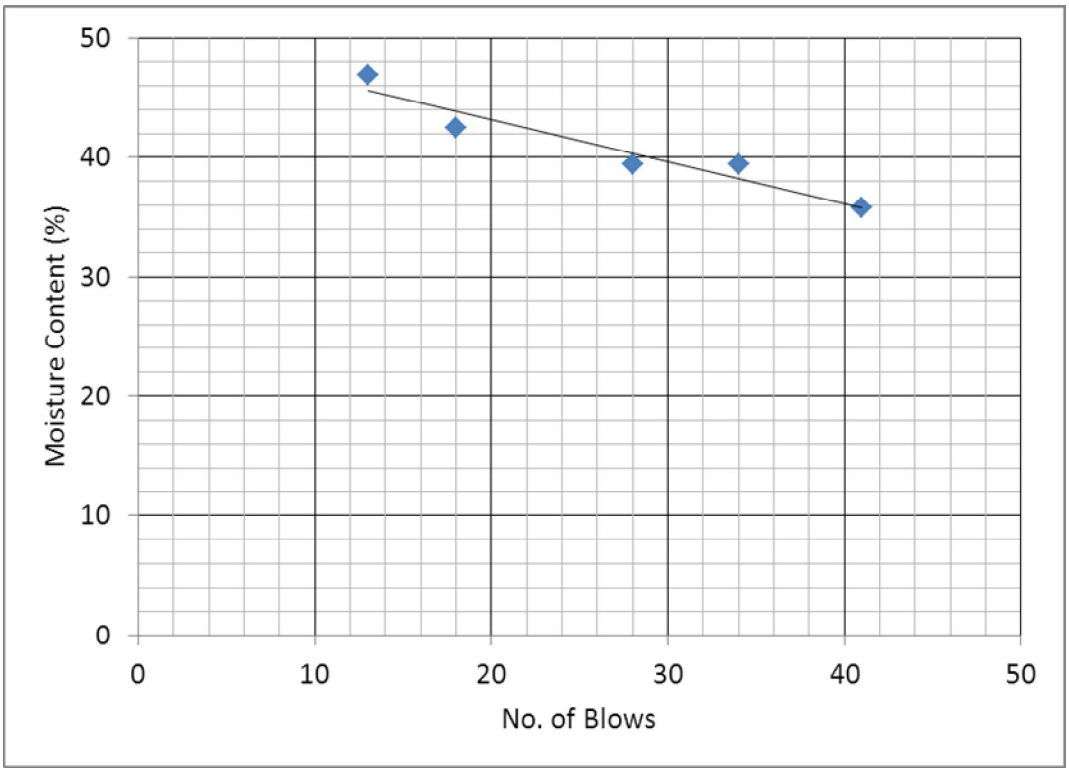


Figure A1 Graph of liquid limit test

From figure A1 and the table above, the followings can be deduced:

(i) Liquid Limit (LL) = Moisture content at 25 blows = 42%

(ii) Plastic Limit (PL) = $(18.42 + 16.67 + 17.34)/3 = 18\%$

(iii)Plasticity Index (PI) = LL – PL = 42 – 18 = 24%

Appendix A12 Determination of Linear shrinkage of lateritic soil

	Test No. 1	Test No. 2	Test No. 3
Original length, lo (mm)	140	140	140
Change in length, Δl (mm)	8.4	8.8	8.0
Shrinkage (%)	6.0	6.3	5.7

Appendix A13 Particle size distribution of Coarse aggregates

BS Sieve size (mm)	Weight Retained (g)	% Retained	Cummulative % passing	Cummulative % Retained
31.50	0.0	0.0	100	0.00
19.05	40.0	4.0	96.0	4.00
9.52	420.0	42.0	54.0	46.0
6.35	280.0	28.0	26.0	74.0
4.76	100.0	10.0	16.0	84.0
2.36	65.0	6.5	9.5	90.5
1.18	12.0	1.2	8.3	91.7
0.6	83.0	8.3	0.0	0.00
TOTAL	1000	100		

Appendix A14 Determination of specific gravity of Coarse aggregates

	Test No. 1	Test No. 2	Test No. 3
Sample No.	SG-06	SG-07	SG-08
Bottle No.	Q9	Z5	Z8
Weight of aggregates, B (g)	1000	1000	1000
Weight of cylinder + water, P	2400	2400	2400
Weight of cylinder + water + aggregates, Ps	3013	3018	3015
Specific gravity, S.G = B/(P+B – Ps)	2.58	2.62	2.60

Appendix A15 Flakiness test of Coarse aggregates

	Test No. 1			**Test No. 2**		
Sample No.	FI-01			FI-02		
BS Sieve size (mm)	Weight of Sample (kg)	Weight passing guage (g)	Weight retained (kg)	Weight of Sample (kg)	Weight passing guage (g)	Weight retained (kg)
25.4	9.40	1.30	8.10	8.50	1.17	7.33
19.05	3.20	0.56	2.64	3.90	0.52	3.38
12.7	1.25	0.14	1.11	1.86	0.14	1.72
9.52	0.45	0.06	0.39	0.26	0.12	0.14
6.03	0.23	0.02	0.21	0.20	0.10	0.10
	Total = 14.53	2.08		Total = 14.72	2.05	
	Flakinees Index = 2.08/14.53 x 100 = 14.31%			Flakinees Index = 2.05/14.72 x 100 = 13.93%		

Appendix A16 Elongation test of Coarse aggregates

	Test No. 1			**Test No. 2**		
Sample No.	EI-01			EI-02		
BS Sieve size (mm)	Weight of Sample (kg)	Weight passing guage (g)	Weight retained (kg)	Weight of Sample (kg)	Weight passing guage (g)	Weight retained (kg)
25.4	9.40	1.40	8.00	8.50	1.20	7.30
19.05	3.20	0.54	2.66	3.90	0.62	3.28
12.7	1.25	0.08	1.17	1.86	0.19	1.67
9.52	0.45	0.04	0.41	0.26	0.09	0.17
6.03	0.23	0.09	0.14	0.20	0.05	0.15
	Total = 14.53	2.15		Total = 14.72	2.15	
	Elongation Index = 2.15/14.53 x 100 = 14.78%			Elongation Index = 2.15/14.72 = 14.61%		

Appendix A17 Result of aggregate impact value test (12 mm aggregates)

	Test No. 1	**Test No. 2**	**Test No. 3**
Sample No.	IV-01	IV-02	IV-03
Weight of sample (g)	500	500	500
Weight of sample remaining after filling mould (g)	145	147	145
Weight of crushed sample passing sieve 2.36 mm	145	155	153
Aggregate Impact value (AIV) %	40.85	43.91	43.10

Appendix A18 Result of aggregate crushing value test (12 mm aggregates)

	Test No. 1	**Test No. 2**	**Test No. 3**
Sample No.	CV-01	CV-02	CV-03
Weight of sample (g)	5000	5000	5000
Weight of sample remaining after filling mould (g)	1400	1550	1500
Weight of crushed sample passing sieve 2.36 mm	678	658	674
Aggregate Impact value (ACV) %	18.83	19.07	19.26

Appendix A19 Properties of 12mm diameter high yield (Y12) steel bars

S/NO.	Identification Number	Size (mm)	Nominal Diameter Measured (mm)	Cross-Sectional Area (mm^2)	Actual length (mm)	Guage length (mm)	Elongation (mm)	Yield load (kN)	Ultimate load (kN)	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)	Elongation (Percent)
01	CIV/Y12/01	12	11.78	108.99	500	300	345	45	67	412.88	614.74	15.00
02	CIV/Y12/02	12	11.78	108.99	500	300	350	44	64	403.71	587.21	16.67
03	CIV/Y12/03	12	11.78	108.99	500	300	350	44	67	403.71	614.74	16.67
04	CIV/Y12/04	12	11.78	108.99	500	300	350	43	67	394.53	614.74	16.67
05	CIV/Y12/05	12	11.78	108.99	500	300	345	45	65	412.88	596.38	15.00

Appendix A20 Properties of 6 mm diameter mild (R6) steel bars

S/NO.	Identification Number	Size (mm)	Nominal Diameter Measured (mm)	Cross-Sectional Area (mm^2)	Actual length (mm)	Guage length (mm)	Elongation (mm)	Yield load (kN)	Ultimate load (kN)	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)	Elongation (Percent)
01	CIV/R6/01	6	5.57	24.37	500	300	355	9.8	10.5	402.13	430.86	18.33
02	CIV/ R6/02	6	5.57	24.37	500	300	360	9.8	10.5	402.13	430.86	20.00
03	CIV/ R6/03	6	5.57	24.37	500	300	360	9.5	10.3	389.82	422.65	20.00
04	CIV/ R6/04	6	5.57	24.37	500	300	356	9.8	10.4	402.13	426.75	18.67
05	CIV/ R6/05	6	5.57	24.37	500	300	360	9.7	10.5	398.03	430.86	20.00

Appendix A21 Mix Proportion of Concrete

Concrete mix proportion using 1:2:4 by weight, at water/cement ratio = 0.5

Specimens required for test: 12 Cubes, 3 Cylinders and 10 beams

Materials required for Specimens:

A. For 12 Cube Specimens of dimensions 150 mm x 150 mm

Volume of 1 cube = 0.15 m x 0.15 m x 0.15 m = 0.003375 m^3

Hence, volume of 12 cubes = 12 x 0.003375 m^3 = 0.0405 m^3

and Weight of 12 cubes = 0.0405 m^3 x 2400 kg/m^3 = 97.20 kg

B. For 3 Cylinder Specimens of dimensions 150 mm x 300 mm

Volume of cylinder, $V = \pi h D^2/4$

Where D is the diameter of the cylinder and h the height

Volume of 1 cylinder, $V = \pi \ x \ 0.3 \ x \ 0.15^2/4$ = 0.0053 m^3

Hence, volume of 3 cylinders = 3 x 0.0053 m^3 = 0.0159 m^3

and Weight of 3 cylinders = 0.0159 m^3 x 2400 kg/m^3 = 38.16 kg

C. For 10 Beam Specimens of dimensions 150 mm x 150 mm x 750 mm

Volume of 1 beam = 0.15 m x 0.15 m x 0.75 m = 0.0169 m^3

Hence, volume of 10 beams = 10 x 0.0169 m^3 = 0.169 m^3

and Weight of 10 beams = 0.169 m^3 x 2400 kg/m^3 = 405.60 kg

Therefore total weight of required materials = 97.20 + 38.16 + 405.60 = 540.96 kg

Total proportion = 1+ 2 + 4 + 0.5 = 7.5

Therefore, concrete components: Cement $1/7.5 \ x \ 540.96 = 72.13 \ kg$

Sand $2/7.5 \ x \ 540.96 = 144.26 \ kg$

Coarse aggregates $4/7.5 \ x \ 540.96 = 288.51 \ kg$

Water $0.5/7.5 \ x \ 540.96 = 36.06 \ kg$

Appendix A22 Mix Proportion of Latecrete

Latecrete mix proportion using 1:2:4 by weight, at water/cement ratio = 0.75

Specimens required for test: 12 Cubes, 3 Cylinders and 10 beams

Materials required for Specimens:

A. For 12 Cube Specimens of dimensions 150 mm x 150 mm

Volume of 1 cube = 0.15 m x 0.15 m x 0.15 m = 0.003375 m^3

Hence, volume of 12 cubes = 12 x 0.003375 m^3 = 0.0405 m^3

and Weight of 12 cubes = 0.0405 m^3 x 2400 kg/m^3 = 97.20 kg

B. For 3 Cylinder Specimens of dimensions 150 mm x 300 mm

Volume of cylinder, $V = \pi h D^2/4$

Where D is the diameter of the cylinder and h the height

Volume of 1 cylinder, $V = 1\ x\ \pi\ x\ 0.3\ x\ 0.15^2/4$ = 0.0053 m^3

Hence, volume of 3 cylinders = 3 x 0.0053 m^3 = 0.0159 m^3

and Weight of 3 cylinders = 0.0159 m^3 x 2400 kg/m^3 = 38.16 kg

C. For 10 Beam Specimens of dimensions 150 mm x 150 mm x 750 mm

Volume of 1 beam = 0.15 m x 0.15 m x 0.75 m = 0.0169 m^3

Hence, volume of 10 beams = 10 x 0.0169 m^3 = 0.169 m^3

and Weight of 10 beams = 0.169 m^3 x 2400 kg/m^3 = 405.60 kg

Therefore total weight of required materials = 97.20 + 38.16 + 405.60 = 540.96 kg

Total proportion = 1+ 2 + 4 + 0.75 = 7.75

Therefore, latecrete components: Cement $1/7.75\ x\ 540.96 = 69.80\ kg$

Laterite

$2/7.75\ x\ 540.96 = 139.60\ kg$

Coarse aggregates $4/7.75\ x\ 540.96 = 279.21\ kg$

Water $0.5/7.75 \ x \ 540.96 = 34.90 \ kg$

Appendix A23 Concrete Cube test results

Sample Identification No.	Age (days)	Mass (kg)	Volume $x10^{-3}$(m3)	Density $x10^{3}$(kg/m3)	Failure Load ($x10^{3}$ N)	Actual Strength (N/mm^2)	Average Compressive Strength (N/mm^2)
C01	7	8.1	3.375	2.400	360	16.00	
C02	7	8.3	3.375	2.459	380	16.89	16.00
C03	7	8.4	3.375	2.489	340	15.11	
C04	14	8.2	3.375	2.430	430	19.11	
C05	14	8.4	3.375	2.489	410	18.22	18.66
C06	14	8.4	3.375	2.489	420	18.66	
C07	21	8.3	3.375	2.459	470	20.89	
C08	21	8.3	3.375	2.459	480	21.33	21.04
C09	21	8.4	3.375	2.489	470	20.89	
C10	28	8.4	3.375	2.489	520	23.11	
C11	28	8.4	3.375	2.489	520	23.11	23.26
C12	28	8.4	3.375	2.489	530	23.56	

Appendix A24 Latecrete Cube test results

Sample Identification No.	Age (days)	Mass (kg)	Volume $x10^{-3}$(m3)	Density $x10^{3}$(kg/m3)	Failure Load ($x10^{3)}$ N	Actual Strength (N/mm2)	Average Compressive Strength (N/mm2)
C13	7	6.9	3.375	2.044	125	5.56	
C14	7	8.0	3.375	2.370	135	6.00	5.78
C15	7	8.0	3.375	2.370	130	5.78	
C16	14	7.2	3.375	2.133	140	6.22	
C17	14	7.1	3.375	2.103	135	6.00	6.22
C18	14	7.4	3.375	2.193	145	6.44	
C19	21	7.3	3.375	2.163	170	7.56	
C20	21	6.8	3.375	2.015	180	8.00	8.00
C21	21	7.1	3.375	2.103	190	8.44	
C22	28	7.3	3.375	2.163	220	9.78	
C23	28	7.2	3.375	2.133	200	8.89	8.89
C24	28	7.2	3.375	2.133	180	8.00	

Appendix A25 Tensile Strength of latecrete and concrete cylinder specimens

Specimen Identification No.	Diameter (mm)	Weight (kg)	Load ($x10^{3)}$ N	Tensile Strength (N/mm^2)	Average tensile strength (N/mm^2)
CY-01	150	13.00	125	1.77	1.82
CY-02	150	13.05	130	1.84	
CY-03	150	13.00	130	1.84	
CY-04	150	12.00	49	0.69	0.67
CY-05	150	12.00	46	0.65	
CY-06	150	12.30	47	0.67	

Appendix A26 Typical Calculation of Flexural Strength of Beams

The beams are loaded as earlier presented in figure 2.4.

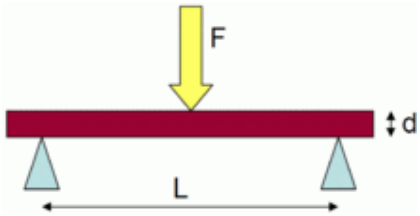


Figure 2.4: Beam under three point bending
After Callister (2003)

For a rectangular beam under a load in a three-point bending setup (figure 2.4), the flexural strength (modulus of rupture), in accordance to BS EN 12390: part 5 (2003), is given by:

$$\sigma = \frac{3FL}{2bd^2} \qquad 2.2$$

Where,

- F is the load (force) at the fracture point (N)
- L is the length of the support span (mm)
- b is width (mm)
- d is thickness (mm)

For the Reinforced Concrete beam, B01:

F = 50 kN (as given in table 3.31 of chapter three)

L = 650 mm (as given in section 3.7 of chapter three)

b = d = 150 mm (as given in section 3.7 of chapter three)

$$\sigma = \frac{3FL}{2bd^2}$$

$$= \frac{3 \times 50 \times 10^3 \times 650}{2 \times 150 \times 150^2}$$

$$= 14.44 \text{ N/mm}^2$$

Similarly,

Using the beam failure loads (F) as presented in tables 3.32 to 3.50, and the beam dimensions as given above, the flexural strength of the remaining beams are as follows: B02 = 13.00 N/mm^2, B03 = 13.00 N/mm^2, B04 = 14.00 N/mm^2, B05 = 13.00 N/mm^2, B06 = 2.89 N/mm^2, B07 = 2.89 N/mm^2, B08 = 2.89 N/mm^2, B09 = 2.89 N/mm^2, B10 = 2.89 N/mm^2, B11 = 8.67 N/mm^2, B12 = 7.22 N/mm2, B13 = 7.22 N/mm^2, B14 = 7.22 N/mm^2, B15 = 8.67 N/mm^2, B16 = 1.44 N/mm^2, B17 = 1.44 N/mm^2, B18 = 1.44 N/mm^2, B19 = 1.44 N/mm^2, and B20 = 1.44 N/mm^2.

Appendix A27 Typical Calculation of flexural stress on Beam

For a rectangular beam under a load in a three-point bending setup as shown in Appendix A26 above, the flexural stress is given by:

$$\sigma = \frac{3FL}{2bd^2} \qquad 2.2$$

where,

- F is the load (force) at the fracture point (N)
- b is width (mm)
- d is thickness (mm)

- and, L is the support span

For the beam specimens, B01 – B20:

F = 0, 5, …, 50 kN (as given in table 3.31 to 3.50 of chapter three)

b = d = 150 mm (as given in section 3.7 of chapter three)

For, F = 5 kN,

$$\sigma = \frac{3FL}{2bd^2}$$

$$= \frac{3 \; x \; 5 \; x \; 10^3}{2 \; x \; 150 \; x \; 150^2}$$

$$= 1.44 \text{ N/mm}^2$$

Similarly,

The flexural stresses calculated for the various loads imposed on the beams are as follows:

At F = 0 kN, σ = 0 N/mm^2; F = 2.5 kN, σ = 0.72 N/mm^2; F = 7.5 kN, σ =2.17 N/mm^2; F = 10 kN, σ = 2.89 N/mm^2; F = 15 kN, σ = 4.33 N/mm^2; F = 20 kN, σ = 5.78 N/mm^2; F = 25 kN, σ = 7.22 N/mm^2; F = 30 kN, σ = 8.67 N/mm^2; F = 35 kN, σ = 10.11 N/mm^2; F = 40 kN, σ = 11.55 N/mm^2; F = 45 kN, σ = 13.00 N/mm^2; and F = 50 kN, σ = 14.44 N/mm^2.