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Behavior of Bamboo Reinforced Self-Compacting Concrete: Application of Short Span Elements

by

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DECLARATION

I declare that this submission is my own work towards MSc. and that, to the best of my knowledge, it contains no material previously published by another person nor material which has been accepted for the award of any other degree of the University, except where due acknowledgement has been made in the text.

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DEDICATION

This work is dedicated to the Almighty God who does all things in His time and is able to do exceedingly, abundantly above all that I can ever think or imagine.

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Finally, I would like to appreciate my late father Isaac Anning, my mom Comfort Okyere and my spiritual father J. K. Okyere (Antartic) for the foundation they laid in my life.

ABSTRACT

This study focused on the behavior of bamboo reinforced self-compacting concrete (BRSCC) beams and slabs under monotonic loading. Both BRSCC and Bamboo Reinforced Natural Concrete (BRNC) samples with varying percentages of bamboo (1.5 and 3% for beams and 1%, 2% and 3% for slabs) as longitudinal reinforcement were cast and tested to study shear and flexure failure mechanisms and the contribution of concrete and bamboo to their resistance. The beams were 100 mm wide and had different depths of 150 mm, 250 mm and 275 mm with lengths of 1050 mm, 1200 mm and 2000mm respectively and a span to depth ratio of 1.8. The slabs on the other hand had dimensions of 1000 x 300 x 80 mm and a shear span- to - depth ratio of 2.5. All the samples were simply supported and subjected to a four-point monotonic loading. During testing, the characteristics of the samples under loading such as deflection, cracking and failure were observed and recorded.

The study established that for the same percentage longitudinal reinforcement and sectional properties, energy dissipation capacity of the structural components (beams and slabs) of BRSCC was higher than their BRNC counterparts. The average increase in the energy dissipation was 17% and 15% for slabs and beams respectively. In addition, the longitudinal reinforcement ratios greatly impacted the shear capacities and degree of ductility of the structural components.

Though bamboo as a longitudinal reinforcement contributes to shear resistance, it is recommended that a code predictive equation that does not explicitly account for longitudinal shear resistance e.g. CSA be utilized when designing BRSCC structural components.

BS, ACI, EC 2 and CSA overestimated the prediction of the flexural capacities of the slabs when a material factor of safety of 3 was used for the bamboo. Hence a reduction factor of 0.5 must be applied to code prediction when designing BRSCC slabs to ensure a high enough safety factor on ultimate strength.

NOTATIONS

a shear span

As area of steel a/d shear

span- effective depth ratio b width of

beam d	effective depth of beam

fc', f_{cu} concrete compressive strength

f_y	yield strength of steel
h	height of beam

L	length of beam

M	applied moment of beam

Mcr	cracking moment

Mf	factored moment at section

Pcr	First crack load

P'cr	Concrete cracking load

Ps	First shear crack load

Pult	Experimental Ultimate failure load

P'ult	Theoretical Ultimate failiure load

ρ_w	ratio of longitudiinal steel

v_c	shear stress provided by concrete

v_n	nominal shear strength V_{nz}

normalised shear load v_{nz}	normalised

shear stress **TABLE OF CONTENT**

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CHAPTER 1: INTRODUCTION

1.1 Background of study

The United Nations (UN) initiative of transforming the world by setting up targets for sustainable development has identified the need to make cities and human settlement safe, resilient and sustainable as much as possible (UNDP, 2015). Currently, shortage of adequate housing and declining infrastructure are among the major challenges that have to be addressed. For instance, in Ghana, the annual deficit in the building industry is about 200,000 housing units (Adom-Asamoah and Afrifa, 2011). Beside the shortfalls, most formal housing units are beyond the affordability level of majority of the population. Extensive research efforts aimed at improving housing affordability have emphasized the need for construction materials and methods which will reduce total cost of structures as well as maintain a sustainable construction industry.

In the construction industry, concrete has been the most widely used material because of its versatility and relative economy in meeting a wide range of needs. Nonetheless, a variety of concrete types have been developed to address strength, durability and constraints that can be met at construction sites. A typical example is the introduction of Self-compacting Concrete (SCC) in Japan when the availability of skilled labourers became a problem in the 1980s. An added advantage to the use of SCC is its ability to help reduce time and cost of construction since there is no need for mechanical vibration of the in-place concrete. For instance, construction of the anchorages of the Akashi Kaikyo suspension bridge took 2 years to complete when SCC was used. This would have taken 2.5 years for completion with the use of

Normal Concrete (NC). In another case when SCC was used in the construction of a large liquefied natural gas (LNG) tank belonging to Osaka Gas Company, it led to a reduction in the number of workmen from 150 to 50 (Ouchi and Hibino, 2000). These benefits of using SCC in the construction industry have proven how it can help maintain sustainable development, particularly in rural and peri-urban areas worldwide.

An integral part of providing safe and resilient structures for human settlement is the selection of construction materials that give a great deal of reliability in terms of structural performance and durability under all forms of external loads. Basically, concrete has low tensile strength and as such, reinforcement (conventionally steel) is used to supplement region of structural components (beams, columns and slabs) that are subjected to high tensile stresses. Apart from steel, other synthetic and natural materials such as fiber glass and cane have been found to be good for resisting tensile stresses. One other natural material which is readily available and easy to use in rural and farming communities is bamboo. Unlike steel production which releases a lot of CO2; a major contributor to global warming, bamboo, which is naturally occurring, tends to reduce the amount of CO2 in the environment. Therefore, the extent of pollution will be drastically reduced when bamboo is used as a reinforcing material, and long term climatic goals of reducing carbon intensity by the UN can be effectively addressed. Moreover, the cost of steel is comparatively higher than bamboo; hence developing countries may opt for its usage to maintain an affordable and sustainable infrastructural development especially in areas where it is in abundance.

1.2 Problem Definition

The high demand for housing facilities and the ever increasing cost of construction materials make building expensive. There is, therefore, the need to explore suitable materials that have the potential to maintain adequate structural performance and at the same time reduce the cost of construction. The use of Bamboo Reinforced Self Compacting Concrete (BRSCC) has the potential of providing affordable housing facilities at a faster rate. However, concern about performance of bamboo for reinforcement still remains an issue under study. This stems from the fact that, bamboo possesses a very wide range of variability in mechanical properties, particularly tensile and bond strength that are imperative for structural integrity. Also, bamboo is vulnerable to insect attack and water absorption which may have an adverse impact on its durability. On the other hand, experimental test of structural components using SCC has shown a low resistance to shear stresses as compared to NC (Lachemi et al., 2005). This is because of the relatively low coarse aggregate content needed to achieve a flowable mix. Consequently, aggregate interlock which is a major contributor to shear resistance, accounting for about 35-50%, is greatly reduced. Since much research work has not been done on the performance of BRSCC beams and slabs, this dissertation seeks to set out the tone and advance knowledge in the evaluation of their shear and flexural capacities.

1.3 Objectives

The objectives of the study were to assess the variations in the strength characteristics of short span structural components that use BRSCC and BRNC as structural materials and to provide reliable estimates of the flexural contribution from bamboo.

The specific objectives were;

- To evaluate the shear capacities of NC and SCC short span beams when bamboo is used as the longitudinal reinforcing material.
- To evaluate the flexural strength of NC and SCC short span one-way slabs when bamboo is used as reinforcement.
- To assess the impact of increasing longitudinal reinforcement of bamboo on flexural and shear capacities.
- To evaluate the adequacy and application of current code predictive equations to BRSCC specimens.

1.4 Organization of Thesis

This thesis is in five chapters. Chapter 1 introduces SCC as another form of concrete technology and bamboo as a viable replacement of steel in structural concrete production. It also presents the objectives of the study. Chapter 2 presents a review of the related literature on SCC and bamboo in NC and rationalizes the relevance of using SCC and bamboo in structural concrete production. Chapter 3 gives an overview of the research programme and procedures for experimental investigation and highlights on concrete mix design as per different constituent materials applied in the study. Chapter 4 presents results and discussions of the BRSCC and BRNC specimens produced in the study and examines the suitability of the existing code provisions for the design of BRSCC. Finally, Chapter 5 presents the summary of findings and conclusions for the study and proffers recommendations for future studies in the content area.

CHAPTER 2: LITERATURE REVIEW

2.1. Introduction

This chapter provides an overview of literature on SCC as well as the properties of bamboo. It also provides experimental studies on the viability of bamboo as reinforcement in SCC. Moreover, information provided by other researchers with respect to the properties of self-compacting concrete is compared with NC whereas concrete reinforced with bamboo is also compared with that of steel reinforced concrete.

The rising cost of construction in recent times and the need for environmentally friendly and sustainable construction processes have necessitated research into the use of alternative materials, especially locally-available ones which can replace the conventional ones used in the production of concrete. Such alternative materials that have been used by researchers in the production of reinforced concrete include industrial waste, artificial aggregates, recycled aggregates from demolished structures, bamboo, cane etc. (Sukesh et al. 2013; Adom-Asamoah & Afrifa, 2011). The use of such replacement materials helps reduce the cost of construction, control environmental degradation and advance infrastructural development. It also has the potential to make engineering construction sustainable to help in the transformation of the building and construction sectors of the economy and contribute towards the realization of national and global poverty reduction strategies. For this reason, the use of economical building materials without loss of performance is very crucial to the growth of developing countries (Zemke and Woods, 2009).

2.2 Properties of concrete

Concrete is a composite material consisting of mainly cement, water and aggregates. The aggregates and the dry cement are mixed with water to form a fluid mass that is easily molded into any shape by pouring them into concrete formwork erected on the field. On the other hand, concrete is sometimes mixed into dryer, non-fluid forms and used for the manufacturing of precast concrete products. In all cases, the mixture undergoes a process called hydration which is initiated by the cement to produce a hardened stone-like substance named concrete. Admixtures and chemical additives are sometimes added to the concrete or its binding constituent to reduce the cost of production or change its properties. These chemicals retard or increase the rate at which concrete may harden and also impart other mechanical and chemical properties. Concrete in its fresh state or hardened state exhibits various characteristics, depending on the properties of the constituent materials.

2.2.1 Fresh concrete

Concrete goes through a lot of physical and chemical processes during its production which have great impact on the quality and performance of the material under service loading. A good concrete must have its constituent materials remain uniformly distributed within the concrete mass even after placing and compaction. This calls for good quality control measures to ensure that the fresh concrete is thoroughly mixed and well compacted at placement. Generally, concrete in its fresh state must be cohesive, consistent and workable to ensure its optimal performance.

2.2.2 Hardened concrete

Concrete in its hardened state must satisfy all usage requirements. It must be capable of withstanding against all forms of service loads it is subjected to. The properties of hardened concrete include strength, creep, durability, shrinkage, modulus of elasticity, deformation under load and permeability. The compressive strength of concrete is the most important property of concrete and predominantly controls its quality. The compressive strength of concrete is the strength below which not more than 5% of the tested samples will fall. According to Jackson and Dhir (1996), as the strength of a concrete increases, other properties of the material also improve. For this reason, 28 days cube or cylinder compressive strength as given by BS 8110 (1997) and ACI 318 (2008) respectively is commonly used in the construction industry for quality control measures.

2.3 Self-compacting concrete

Concrete is the most consumed man-made material in the world due to its versatility and relative economy in satisfying wide range of needs (Naik, 2008). Concrete production has gone through many modifications to meet the desired properties with respect to the project under consideration. The advent of large and complex structures in the construction industry is making the use of NC more expensive and time consuming due to the increasing difficulty in vibrating concrete to achieve the needed compaction and honeycomb free finish. As a result of a shortfall of skilled labor in the Japanese construction industry in the 1980s, there was the need for a concrete that could overcome the issues of defective pouring and compaction. This was very necessary because efficient pouring and compaction of concrete are fundamental in attaining the required mechanical and durability properties. The work of Okamura in those days saw the birth of SCC in 1988 in Japan. Ozawa et al. (1989) did further work on the properties of SCC which saw the production of the first usable SCC named as "High Performance Concrete", and later "Self Compacting High Performance Concrete". SCC is a highly flowable concrete which can fill forms under its own weight, resist segregation and bleeding and achieve good consolidation without the use of vibrators (Abukhashaba et al., 2014). Roussel et al. (2009) and Girish et al. (2010) concluded that the passing ability of SCC could greatly be increased by increasing the volume of paste and reducing the coarse aggregate volume and size which lead to the reduction of the friction between the aggregates.

Since the production of the first usable SCC, many huge and complex structures such as the Akashi Kaikyo Suspension Bridge, wall of a large liquefied natural gas (LNG) tank belonging to Osaka Gas Company, Shin-kiba Ohashi Bridge etc., have been constructed with it. The use of SCC helped shorten the construction period for the anchorages of the Akashi Kaikyo Suspension Bridge from 2.5 years to 2 years and reduce the number of workmen from 150 to 50 in the LNG tank construction (Tanaka et al., 1993).

2.3.1 Constituents of SCC

Just like NC, SCC is composed of cementitious materials, aggregates, water and sometimes admixtures. However, flowability is required in SCC so as to avoid mechanical compaction. One implication of this will be the use of an undesirable high water – cement ratio to attain the level of flowability that is needed. This has led to the use of superplasticizers such as naphthalene formaldehyde sulphonic acid base (Lachemi et al., 2005), polycarboxylic ether hyperplasticizer (Helincks et al., 2013) and type F ASTM C 494 (Hassan et al., 2008) in its preparation. Viscosity agents are also used in the production of SCC to reduce the variation of the concrete quality. Mineral admixtures such as metakoalin, silica fume, GGBS have all been used in the production of SCC just like NC and they have been found to improve upon the strength and durability. (Hassan et al, 2010; Swami, 2016; Ahari et al., 2015).

2.3.2 Mix design of SCC

Many design approaches have been formulated for the design of SCC. Notable among these approaches are Ozawa (1993), Sedran et al (1996), Petersson et al (1996), Hwang et al. (1996) and Hon et al. (1996). These approaches have been successfully implemented in different projects and all follow the same process of optimizing the constituent materials of concrete to attain the desired fresh and mechanical properties. The basic of the design procedures reduces the coarse aggregates content in a natural mix concrete and replaces it with an equal measure of fine aggregates proportion. This results in an unusual higher fine aggregate proportion and a smaller coarse aggregate proportion. Okamura and Ozawa (1994) stated that two main methods exist for the achievement of the self-compatibility of SCC but advised the use of the second method although the two could be combined. The first being the addition of a viscosity agent and the second is limiting the coarse aggregate volume and increasing the water/powder ratio which enhances the viscosity of the paste and helps prevent segregation (Hsi-Wen, 1998).

2.3.3 Test methods for fresh properties of SCC

Various test methods have been used to examine the self compactibility and other important properties of fresh SCC. The notable test methods are discussed below;

- Slump flow test (Fig. 2.1): This measures the diameter of the concrete spread after a standard slump test. The slump flow should be between 550 – 850 mm according to the SCC guidelines (EFNARC, 2001) for flowabilty.

Fig. 2.1. Slump Flow Test

- L-Box test (Fig 2.2): This test assesses the flow of the concrete and also the extent to which it is subjected to blocking by reinforcement. The nearest the test value, is unity, the better the flow of concrete. The EU research team suggested a minimum acceptable value of 0.8. Obvious blocking of coarse aggregate behind the reinforcement bars can be detected visually.
- U-box test (Fig. 2.3): This measures the passing ability of concrete through reinforcement. The nearest this test value is to zero the better the flow and passing ability of the concrete.

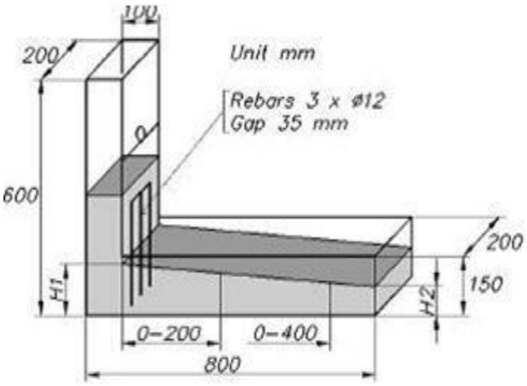


Fig. 2.2. L-box test apparatus

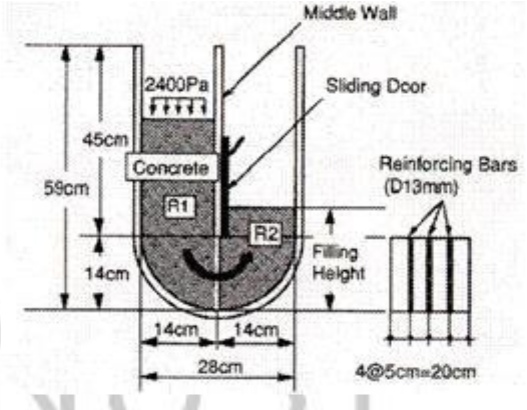


Fig. 2.3. U-box test apparatus

- The funnel test (Fig. 2.4) is for measuring the viscosity property of the concrete and can also be used to determine the arching effect of the aggregates. Shorter flow time indicates greater flow ability. For SCC, a flow time of 10 seconds is considered appropriate.

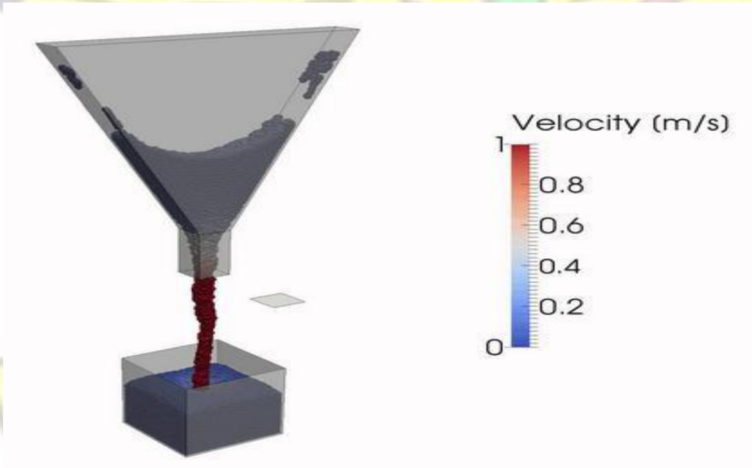


Fig. 2.4. Funnel test apparatus

- Sieve test and the T500 Slump flow test are used for measuring segregation resistance and viscosity, respectively (EFNARC, 2001, Hwang & Tran, 2015). Jalal et al (2015) also measured the segregation resistance of SCC by visual inspection during the slum flow test in his work on the comparative study on effects of Class F fly ash, nano silica and silica fume on properties of high performance SCC.

2.3.4 Advantages of SCC

The high rise in the usage of SCC is as a result of the material's advantageous properties over NC some of which are:

- SCC has proven to avoid challenges involving mechanical compaction and its associated noise on construction sites. NC needs effective vibration to ensure adequate compaction which has direct influence on the mechanical and durability properties. As a result of the self compactibility of SCC, the skill and the difficulty associated with efficient compaction of very deep concrete walls is avoided (Okamura and Ouchi 2003). Thus the amount of time and energy spent on SCC is relatively low.
- It is difficult to ensure that formwork is fully compacted without voids or honeycombs in situations where large quantity of heavy reinforcement is in comparatively smaller sized formworks. In this case, manual or mechanical compaction is very difficult. The usual vibration done to ensure concrete compaction results in delays in construction and hence increased project cost.
- During concrete works in under water construction, there is the need for concrete which could be cast without mechanical compaction since vibration in this case has proved impossible. SCC comes handy when faced with constructional challenges such as under water construction.
- SCC can also make the construction of earthquake resistance structures efficient. The large amount of reinforcement required at beam ends, column

ends and beam-column joints in seismic regions as stated clearly in various codes of practices results in steel congestion and poor consolidation when NC is used (Ozawa et al., 1989).

- In addition, SCC has the ability to withstand and prevent segregation and bleeding as compared to NC. NC has a comparatively higher tendency to segregate and or bleed since the vibration process creates a pressure wave around the aggregates and causes a reduction of friction between the cement paste and the aggregates.
- SCC has proven to shorten construction time and consequently save money on projects.

Despite the numerous advantages of SCC over NC, the limited information available on the in-situ properties and structural performance of SCC members as compared to NC retards the rate of its usage.

2.4 Experimental Study on SCC beams

In reviewing existing literature on the mechanical behavior of SCC structural members under load, data was gathered from various research works and comparisons made on the respective parameters that were considered and varied to ascertain their contribution to structural behavior. Some of the parameters included the maximum size of coarse aggregate, shear span- effective depth ratio (a/d), coarse-to-fine aggregate ratio, beam depth, compressive strength of concrete, coarse aggregate content/type, percentage of longitudinal reinforcement, shear reinforcement ratio and spacing (beams with stirrups), number and characteristics of cracks and loading arrangement.

2.4.1 Shear behavior of beams

The shear behavior of concrete beams has received great attention from many researchers over the years in an attempt to establish probable means of enhancing its shear performance (Ahmad and Shaha, 2009; Ashor and Yang, 2008; Foster and Gilbert, 1998; Bakir and Boduroglu 2002, Hwang et al. 2000, Leong and Tan 2003, Russo et al.2004). It has been established that inadequate shear design of beams may result in failure at loads far below their flexural capacities and such failures are usually sudden and unexpected (Londhe, 2007). In most design codes, the shear strength of beams with shear reinforcement is taken as the sum of the shear resistance of concrete and the contribution from shear reinforcement (AdomAsamoah and Afrifa, 2013).

Mosley et al., (1999) established that reinforced concrete beams without transverse reinforcement possess some amount of shear strength that resists shear stresses before diagonal tension cracks develop. They considered a simply supported beam with a uniformly distributed load across its span (Fig. 2.5). As the load is applied, the principal compressive stresses assume the form of an arch and tensile stresses a suspended cable. In the region of the mid-span where bending stresses are high and shear low, the stresses tend to move in a direction parallel to the beam axis. At the support region where the shear stresses are dominant, the principal stresses are inclined at a steep angle such that the tensile stresses tend to cause diagonal cracking. If the diagonal tension exceeds the tensile strength of the concrete, then shear reinforcement must be provided.

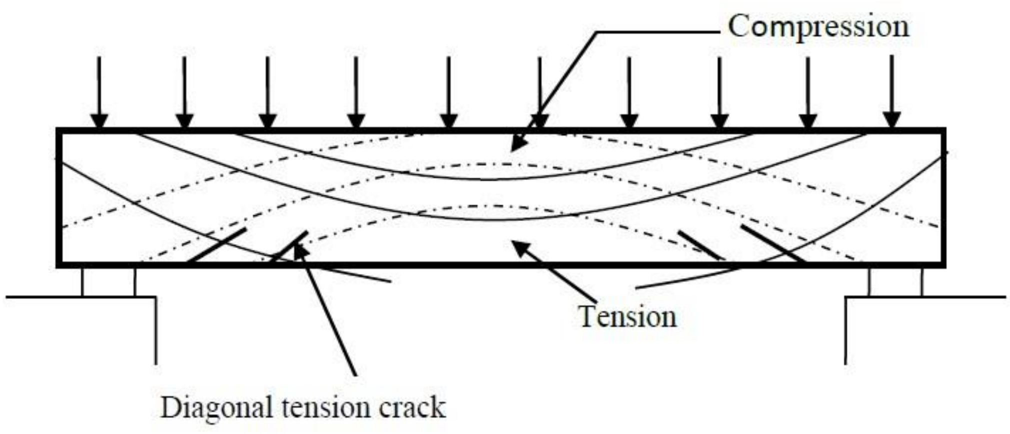


Fig. 2.5: Principal stresses in a beam under bending (Mosley et al, 1999)

2.4.2 Shear resistance of beams without shear reinforcement

Good shear design of concrete can be achieved if the shear strength of the concrete without shear reinforcement is known before the addition of web reinforcement (Rebeiz et al, 2000).

The shear resistance of beams without shear reinforcement is taken up by the beam action when the a/d ratio is above the transition point and the arch mechanism when it is below as shown in Fig. 2.6 (Russo et al, 1991; Park and Paulay 1975 and Shuraim, 2012).

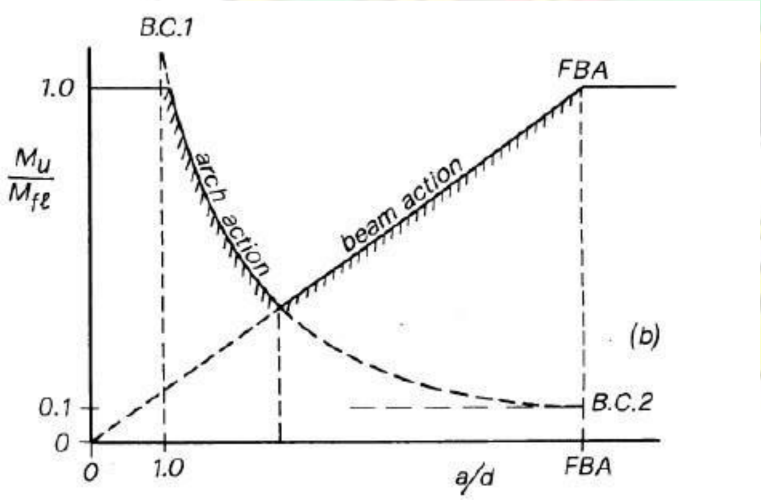


Fig. 2.6: Model for flexure-shear interaction (Russo et al. 1991)

The beam action is comprised of concrete compression (V_c), dowel action of longitudinal reinforcement (V_d) and aggregate interlock (V_a) as illustrated in Fig. 2.7. Taylor (1974), Ziara (1993) and Kim & Park (1996) concluded that after inclined cracks have developed in the concrete, the contribution from each of the factors V_d, V_a and V_c varies between 15-25%, 33-50% and 20-40%, respectively.

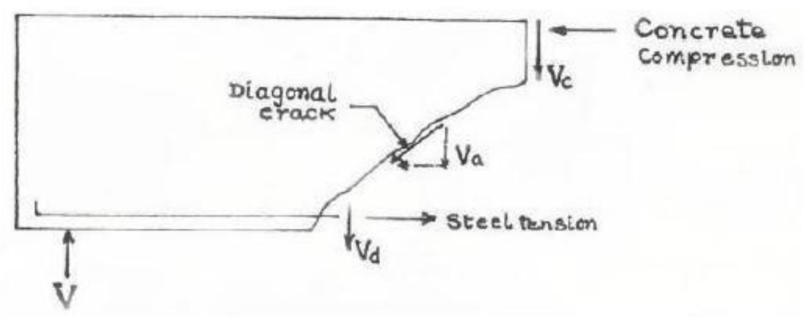


Fig. 2.7: Shear transfer mechanism in beams

Tied arch mechanism on the other hand is experienced in beams with small a/d ratios i.e. usually less than 2.5 and such beams transfer their loads directly to the support. Russo et al. (1991) established that arch action usually results in shear-compression failure whereas beam action causes diagonal-tension.

2.4.3 Shear behavior of SCC beams

The mechanical behavior of SCC beams like shear strength, flexural, bond, cracking etc under static and cyclic loads has been reported in literature, but there is still much work to be done before it can gain the popularity and confidence of usage among engineers and designers in the building industry. The basic constituents used in SCC, mixes are practically the same as those used in NC, except that they are mixed in different proportions with the addition of admixtures to meet the project specifications. Most of the mechanical properties of SCC such as compression and its relation to tensile strength, modulus of rupture, modulus of elasticity, etc. are, therefore, expected to be similar to those obtained from NC.

One significant property of SCC that is likely to vary from NC is its shear strength. In SCC production, the reduction of the coarse aggregate content to enhance flowability results in consequent reduction in the aggregate interlock, which is one of the highest contributors to shear resistance of beams without shear reinforcement. Many researchers have conducted investigations into the mechanical and durability properties of SCC with shear as a major consideration. Regan et al. (2005) and Sagaseta et al (2011) stated that the ultimate nominal shear stress of slender beams without transverse reinforcement depends mainly on the concrete strength, the aggregate interlock between surfaces of cracks, the effective depth (size effect) and the longitudinal reinforcement ratio. Taylor (1974) in a similar research concluded that the dowel action of the longitudinal reinforcement contributes 15% - 25% to the shear strength; the compression zone (the shear in the un-cracked concrete compression zone) contributes 20% - 40% and the aggregate interlock 35% - 50%. Since aggregate interlock plays an important role in the shear strength of beams, SCC beams may have shear strength lower than that of similar NC beams. But results from researchers who compared SCC to NC have shown contradictory outcomes. Some have indicated that SCC and NC beams with the same characteristics have similar shear strength, whereas, according to others, SCC beams have lower shear strength. This can probably be due to the different parameters that affect the shear strength of beams and also the different possible SCC compositions and mix designs used (Lima et al, 2008).

2.5 Shear behavior of SCC elements

In a study by Lima de Resende et al. (2016) on the Shear strength of SCC beams with small stirrup ratios, ultimate shear strength of SCC and NC slender beams were compared. The study revealed that the ultimate shear stress of SCC and NC slender beams with similar characteristics depended on the concrete compositions, strengths, beam depths and possibly the shear reinforcement ratio. A comparison of their results with that obtained by Garcia (2002) also indicated that, the difference in the ultimate shear stresses could be significant depending on the sectional and reinforcing properties of the structural component. After comparing the experimental shear strengths of the beams with the shear provisions of ACI 318:2011, EN 1992-1-1:2004, fib Model Code 2010 and ABNTNBR6118:2014 codes of practice, they concluded that not all shear code provisions can safely predict the shear capacity of beams with a low transverse reinforcement index. This is especially true for beams with a lower longitudinal reinforcement ratio, since the longitudinal reinforcement stress at the time of shear failure has a significant effect on the shear capacity of a beam.

Abouhussien (2015) compared semi-lightweighted normal vibrated concrete and SCC mixtures in terms of their shear resistance. The results indicated that at a maximum lightweight slag aggregate-to-sand (SG/S) ratio of 1.5, SCC beams exhibited slightly higher normalized shear loads compared to normal vibrated concrete beams. However, the possibility of using a higher SG/S ratio in normal vibrated concrete mixtures not only gave these mixtures an advantage over SCC mixtures in terms of lower density but also allowed an increase in the shear capacity of normal vibrated concrete compared to SCC mixtures.

Abouhussien et al. in 2014 investigated the effect of different pouring techniques and mixture fresh properties on the shear strength of SCC beams. The experimental results indicated that increasing the percentage of longitudinal steel reinforcement ratio from 1.6% to 2% increased the shear strength of the SCC beams but the different pouring techniques did not have significant effect on the structural performance of SCC beams. They also established that beams cast with high slump flow had slightly higher shear strength and minimum average crack heights despite the fact that the shear strength and the cracking behavior were not significantly affected by the yield stress and the viscosity of the mixture.

Lachemi et al (2005) studied the effect of concrete type, maximum coarse aggregate size, coarse aggregate content and shear span-to-depth ratio on shear strength. The study established that if the same maximum size of coarse aggregate were used in both NC and SCC (coarse aggregate content reduced and fine aggregate content increased in SCC), there was a lower post-cracking shear transfer resistance in SCC than in NC. This is due to lesser aggregate interlock and dowel action as a result of lower coarse aggregates content in the SCC compared with NC. In addition, if different coarse aggregate sizes are used in SCC, the SCC beams with large size coarse aggregates have an increase in shear resistance by post-cracking shear transfer mechanism. This implies that using large size and higher quantity of coarse aggregates improve the post-cracking shear transfer mechanism and enhance the ultimate shear resistance of SCC beams but Roussel et al. (2009) and Girish et al. (2010) have established that high and large coarse aggregates impede the flowability of concrete. For this reason, there is the need to balance between the coarse aggregate content and size when dealing with SCC so as to achieve the desired fresh and structural properties.

Hassan et al., (2008) conducted an experimental study on the shear strength and cracking behavior of full scale beams. The results established that SCC and NC showed similar characteristics in terms of crack widths, crack pattern, crack load, failure mode and other crack characteristics but the SCC beams exhibited lower ultimate shear load as compared to their NC counterparts. They also established that increasing the longitudinal steel reinforcement ratio caused an increase in the ultimate shear load of both SCC and NC beams. However increasing the longitudinal steel reinforcement ratios did not have any effect on the ultimate shear stress when the beam depths were increased and hence lower ultimate shear stress at higher beam depths were observed. In a follow up study, Hassan et al. (2010), used the same beam configuration and properties as in 2008 to predict that the ACI (2005) and CSA(2015) (2004) codes of practice underestimated the first flexural cracking load for deep beams whereas the AS 3600 (1988) and EC 2 (1992) codes overestimated the first flexural cracking load but all the codes predicted values close to the experimental results in shallow beams.

Another experimental study by Hassan et al (2015) evaluated the effect of cementitious material, type of coarse aggregate and coarse-to-fine aggregate ratio on shear strength and cracking behavior. The results concluded that increasing the coarse–to-fine aggregate ratio increased the normalized shear strength of the beams. They also established that the Eurocode 2 (2005), ACI (2008), CSA (2004) and AASHTO-LRFD (2007) code equations were conservative in predicting the ultimate shear strength of the SCC beams.

Similar to the work of Hassan et al. in 2008 and 2015, Hanoon et al. (2014) investigated the shear behavior of SCRC beams with and without shear reinforcement and compared the results with three different codes of practice. They found out that the ACI 2002 and BS 8110-1 (1997) were less conservative and close to experimental shear strength of SCRC beams.

Safan (2012) investigated the effect of concrete type based on different coarse aggregates on shear behavior. The results concluded that beams with crushed dolomite as coarse aggregates had higher ultimate shear loads than beams with gravel as coarse aggregates due to higher compressive strength and paste-aggregate bond in dolomite coarse aggregates than gravel coarse aggregates. This notwithstanding, the normalized shear strength of most of the gravel based beams was higher than the dolomite based beams due to post-cracking shear transfer and interlocking mechanism. The experimental results were compared with the ACI – 318 (1995) and JCSE – 86 (2007) codes of practice which adequately predicted the experimental values.

Yuan Xu (2011) researched on the shear resistance of SCC I-beams with the same geometry. They reported that the increase in the shear reinforcement ratio to double the ultimate shear resistance did not double the experimental ultimate shear resistance as expected. Also the shear strength gained decreased with decrease in the shear span –to – depth ratio. This indicates that though overdesigning may increase the cost of construction; it will not necessarily result in an equal measure of strength increase.

Helincks et al (2013) did an extensive investigation on the structural behavior of powder- based SCC by determining the shear capacity of SCC beams. The experimental results showed a small decrease of about 6.9% in ultimate shear strength of SCC beams in comparison with NC beams and this was due to lower aggregate interlock from the lower coarse aggregate content in SCC than in NC. The experimental results were also compared with code provisions from EN 1992-1-1 (2004), ACI-318-11 (2011) and fib Model Code 2010 (2012). The EN 1992- 1-1 code was modified by taking into account the effect of the shear span-to-depth ratio as the shear capacities were influenced by it.

Pamnani et al., (2013) investigated the effect of compressive strength and shear strength of M30 grade of SCC by different curing techniques. The curing techniques investigated were the traditional Immersion /pond method, polyethylene film wrap, curing compound and no curing. Results obtained showed that the curing method influenced the shear strength of the specimen with the traditional immersion method having the highest shear strength followed by the polyethylene film wrap, the curing compound application and then no curing with the least shear strength. In the absence of water for the immersion method polyethylene film can be used to obtain acceptable shear strength.

Cattaneo et al., (2007) compared the shear strength of SCC and NC beams. The shear strength of SCC beams with and without shear reinforcement was compared with NC beams. The shear span –to –depth ratio did not significantly affect the behavior of beams with shear reinforcement. SCC beams had higher strength accompanied by a more brittle failure as compared to NC.

2.6 Flexural behavior of SCC elements

Jaadeesh and Babu (2014) studied the flexural strength of SCC with silica fume and polypropylene fibres with and without reinforcement. The study revealed that in the fresh state of concrete as the percentage of fibres increases slump flow value decreases. Also, in the hardened state of concrete there is no considerable increase in compressive strength of concrete, but there is a noticeable increase in flexural strength of concrete by the addition of Polypropylene fibres. In the hardened state of concrete there is an overall increase in strength of concrete both in compressive and flexural strength for the 0.1% addition Polypropylene fibres.

Mithra et al. (2012) conducted a study on the flexural behavior of reinforced SCC containing Ground Granulated Blast-Furnace Slag (GGBFS). The results indicated that all the SCC mixes had a satisfactory performance in the fresh state. Among the mineral admixtures considered, the GGBFS 30% series had good workability properties compared to other GGBFS series. In general the use of mineral admixtures improved the performance of SCC in fresh state and also avoided the use of VMAs. Optimum W/P ratio was chosen as 0.35 by weight, the ratio greatly beyond or less than this may cause segregation and blocking tendency in SCC mixture. Compared to control beam, increase in First crack load was observed for beams with 30% and 40 % GGBFS respectively. In general, beam with 40% GGBFS showed better performance compared to the other beams.

Hassan (2015) investigated the effect of parameters such as slab thickness, steel reinforcement ratio and steel fiber volumetric ratio on the behavior of slabs with respect to deflection, failure mode and ultimate loads. It was observed that the ultimate flexural capacity of the samples increased with increasing slab thickness and flexural steel reinforcement ratio such that a 40% increase in slab thickness resulted in 30.4% and 34.4% increase in ultimate failure load for slabs with reinforcement ratios of 0.0033 and 0.0066 respectively. In addition, the inclusion of steel fibers helped the slabs to behave in a more ductile manner, reduced crack width and its rate of propagation.

Kumar et al. (2013) also compared results from the flexural characteristics of SFRSCC and SFRNC one way slabs with EN 1992:2002, IS 456:2000, Bilinear method and ACI 318 codal equations. It was observed that the codes made an average variation of 20% in deflection, 6.5% in ultimate load and 28% crack width.

2.7 Summary of the study of SCC beams

Transverse reinforcement in the form of stirrups are provided in structural members to resist shear force and control the propagation of cracks into the compression zone and help maintain aggregate interlock (Arezoumandi, 2013). Review of investigative works from researchers on SCC and NC on the absence of shear reinforcement has indicated that the characteristics of structural members depends on a number of factors such as percentage of longitudinal steel reinforcement ratio, transverse steel reinforcement ratio, the shear span-to-depth ratio, sectional properties of the sample, concrete grade etc. These parameters have been varied in a number of research works to find out their contribution to the mechanical properties of concrete. To allow for comparison of the behavior of samples between samples with different dimensional and compressive strength, the shear strength properties are normalized with Equations (1) and (2) for shear load and shear stress respectively.

$$V_n = \frac{V_u}{\sqrt{f_c}} \qquad (1)$$

$$v_n = \frac{V_n}{bd} \qquad (2)$$

Where

V_n is the normalized shear force, V_u is the shear force, $v_n$is the shear stress f_c is the compressive strength, b is the breadth of beam, d is the depth of beam

2.7.1 Influence of longitudinal reinforcement ratio on shear strength

As mentioned by Taylor (1974), the dowel action of the longitudinal reinforcement contributes 15% - 25% to the shear strength. It is therefore expected that increasing the longitudinal reinforcement ratio will increase the ultimate shear strength. The tensile reinforcement placed in the lower part of the beam to resist tensile forces under gravity loading further minimizes the crack width or extension of cracks. The reduced crack width increases shear transfer mechanism, hence the increase in shear stress with high longitudinal reinforcement ratio. From experimental results of the work of Abouhussien et al. in (2015), increasing the percentage of longitudinal reinforcement ratio from 1.6% to 2% increased the shear strength of the SCC beams. However, Lachemi et al (2005) proposed that the shear span-to-depth ratio has a higher controlling power as compared to the longitudinal reinforcement ratio. They used increasing longitudinal reinforcement ratios of 1.15%, 1.57% and 1.6% at an increasing shear span-to-depth ratios of 1.05, 1.53 and 2.14 respectively and found out that the shear stress was less at 1.6% longitudinal reinforcement ratio which had the highest shear span to depth ratio.

2.7.2 Influence of coarse aggregates on shear strength

Research has shown that the aggregate interlock contributes about 35% - 50% of the shear strength in beams. In SCC production, the aggregate interlock is highly reduced as a result of the reduction in coarse aggregates and increase in fine aggregates so as to enhance the flowability of the concrete (EFNARC, 2005). As said earlier, many researchers have concluded that the post- diagonal cracking resistance for shear strength of SCC beams is lower than that of their NC counterparts and this has been attributed to lower coarse aggregate in SCC than NC. On the contrary, Abouhussien et al.,(2015) concluded that no significant differences were noted between NVC and SCC beams in terms of post diagonal cracking resistance and these differences could be due to the different parameters that affect the shear strength of beams and also the different possible SCC compositions and mix designs used(De Resende et al, 2016).

2.7.3 Influence of shear span – to – depth ratio on shear strength

As stated earlier, Lachemi et al., 2005,Helincks et al., 2013,Yuan Xu (2011) and other researchers have concluded that the shear span-to-depth ratio has a high controlling power. Yuan Xu (2011) established that despite the gain in shear strength with decreasing a/d, the gain was not linear and the shear strength gained decreased with decrease in the shear span –to – depth ratio.

In conclusion, the shear strength of concrete is seen to be influenced by:

- The type of concrete (SCC, NC et cetera), the sectional properties of the structural component and the presence or otherwise of transverse reinforcement (Lima de Resende et al., 2016).
- The size quantity and the mechanical strength of the aggregate used (Lachemi et al., 200; Hassan et al., 2015; Safan, 2012 and Helicks et al., 2013)
- The quantity of longitudinal reinforcement present (Abouhoussein, 2014 and Hassan et al., 2008)
- The presence or otherwise of shear reinforcement (Yuan Xu, 2011; Cattaneo et al., 2007)
- The compressive strength of the concrete used (Pamnani et al., 2013)
- The use of supplementary cementitious materials (Abouhussein, 2015)

2.8 Bamboo utilization in construction

Bamboo is a renewable raw material resource and one of the oldest traditional building materials used in the world. It can be grown quickly and easily, even on degraded land, and harvested sustainably on three to five-year rotation. Basically, bamboo is a giant grass belonging to the family Bambusoideae (Panda, 2011). In Ghana there are about seven species of bamboo. They are namely Bambusa *pervariabilis*, Dendrocalamus *strictus*, Bambusa *bambus*, Bambusa *vulgaris*, Bambusa *arundinacea*, Bambusa *multiplex*, and Bambusa *vulgaris var vitata*.

Bambusa *vulgaris* is prevalent whiles the others were introduced from Asia. Bambusa *vulgaris* is the main bamboo species in southern Ghana constituting 95% of the stocks in this area (OtengAmoako et al., 2005).

One of the main advantages of building with bamboo is that it is a natural renewable resource that is cheaper, environmentally friendly and readily available. Its strength and light-weight makes its structures resistant to wind and earthquake and can be effectively exploited through careful yet simple design and detailing. It has a high compressive strength between 40 and 80 N/mm2 which is twice the value of most timber species (Schroder, 2015). The age and moisture content have significant influence on the compressive strength. It has high tensile strength that rivals steel and a very good weight to strength ratio. This supports its use as a highly resilient material against forces created by high velocity winds and earthquakes. Experimentally it has been found that the ultimate tensile strength of some species of bamboo is comparable to that of mild steel and it varies from 140N/mm2- 280N/mm2. This together with other properties has made Bamboo a more viable option as a construction material.

Scientific studies have shown that Bamboo can satisfy the various structural requirements and also due to its technical performance, it can be used as a construction material for various structural components.

Bamboo intended for use in construction should be treated to resist insects and rot and cut bamboo should be boiled to remove the starches that attract insects. The durability of bamboo is dependent on the type of preservative method. The methods include smoking, heating, drying, coating with limestone (calcium hydroxide) and chemical treatment. The chemical composition used should have no effect on the bamboo fibre once injected, and should not be washed away by rain or humidity. No matter the treatments used, drying is a critical process in bamboo conservation. Bamboo with lower moisture content is much less prone to mould and insect attacks, ideally moisture content would be below 15%. The most common and effective preservation methods used globally is drying and then chemical treatment of the bamboo. The chemicals for bamboo treatment include coal tar creosote, copper chrome arsenic composition, copper chrome boric composition, copper chrome zinc arsenic composition, chromated zinc chloride and boric acid borax. If properly treated and industrially processed, components made by bamboo can have a reasonable life of thirty to forty years. Industrially treated bamboo has shown great potential for production of composite materials and components which are costeffective and can be successfully used for structural and non-structural applications in construction. To utilize bamboo to its best capabilities, several conditions are important to consider. One consideration is that bamboo grown on slopes is stronger than bamboo grown in valleys, and that bamboos that grow in poor dry soils are usually more solid than those grown in rich soils. Making more use of bamboo for common building practices would allow forests to regenerate and help control deforestation.

Concrete is very strong in compression but weak in tension. In most design guides, such as BS 8110, the strength of concrete in tension is taken as non-existent and hence reinforcement (conventionally steel) is provided to take care of the tensile stresses. According to Fergusson-Calwell (2015), strong, durable reinforced concrete has high relative strength, high toleration of tensile strain, and good bond to the reinforcement. Also, reinforced concrete has a highly diverse amount of sustainability attributes compared to materials like mortar, brick, timber and nonreinforced concrete.

Environmental issues such as global warming and the high cost of steel reinforcement triggered research to find alternative natural and ecofriendly materials that will be a viable substitute for steel. One of the natural plants that have caught the attention of many researchers is bamboo mostly due to its availability.

Studies done by Ghavami (2005), Moroz (2014), Agarwal et al. (2014) and Harish et al (2012) have concluded that bamboo is a potential substitute for steel as reinforcement for structural components and recommended their use in low-cost housing in regions where they are abundant.

In the study of Khare (2005) on the performance valuation of bamboo reinforced concrete beams. The findings from tensile tests indicated that the presence of nodes in Solid Bamboo samples did not affect the behavior. There was an indication that the fracture points of the tensile samples containing nodes occurred at the nodes, which was also verified in the beam tests. In general, failure modes of samples were in the form of node failure, end-tap failure and failure at the vicinity of the end-tap.

Tensile tests showed that the specimens with nodes behaved in a less ductile manner with higher strength than those without nodes. In general, the test results indicated that bamboo reinforcement enhanced the load carrying capacity by approximately 250% as compared to the initial crack load in the unreinforced concrete beam. This study also showed that the ultimate load carrying capacity of bamboo reinforced beams was about 35% of the equivalent reinforced steel concrete beams. Also, it was noticed that a direct relationship existed between the percentage of reinforcement and the load carrying capacity of the beams tested.

In another study, Chandra et al. (2013) have indicated that bamboo can replace steel for modest housing for the urban poor who live close to bamboo growing regions.

Rahman et al. (2011) established that because of the low modulus of elasticity of bamboo compared to steel, bamboo cannot prevent cracking of concrete under ultimate load but as reinforcement, it can increase the load carrying capacity of beams by about two fold and the deflection of bamboo reinforced beams are much less than plain concrete beams.

Kumar and Mandal (2014) studied on improving durability and mechanical properties of bamboo reinforced concrete. In relation to several experimental investigations in the past, they concluded that bamboo can efficiently replace steel as reinforcement for low cost residential structures. It was also revealed that the numerical data of several engineering parameters supports it. Fergusson-Calwell (2015) has noted that because water absorption is an unfavorable inherent material limitation, bamboo needs to be treated before use as a structural material.

Adom-Asamoah and Afrifa (2011) conducted a comparative study of bamboo reinforced concrete beams using different stirrup materials for rural construction and established at failure, beams with low concrete compressive strength and small amount of bamboo tension reinforcement had wider cracks. They recommended that steel stirrups be used to enhance the performance of bamboo reinforced concrete beams.

In a research undertaken by Khan (2014), different shapes of cross section of bamboo stick such as circular, square and triangular were used as reinforcement. It was found that tensile strength of bamboo is approximately one half that of mild steel and modulus of elasticity is approximately one third that of mild steel.

An investigation into the structural strength of concrete column reinforced with bamboo strips by Salau and Adegbite (2012) showed that the load carrying capacity of the column increased with increase in percentage of bamboo strip reinforcement but the increase is not proportional to the percentage of reinforcement. There was also improved post cracking ability of the concrete due to the bamboo inclusion but not as pronounced as in steel reinforced column. However, all columns failed in a similar pattern due to crushing of concrete. The bamboo strips showed no sign of slippage and remain unaffected even after concrete failure.

Harish et al. 2012, investigated the properties of bamboo reinforcing material in concrete using the Moso type bamboo for the experiment. In their study, it was discovered that compressive strength of bamboo is nearly same as the tensile strength of bamboo and this behavior is similar to steel. It also revealed that the bond stress of bamboo with concrete is very low compared to that of steel bars, due to surface smoothness of bamboo. Again, water absorption of bamboo is very high and waterproofing agent is recommended. They concluded that bamboo can potentially be used as substitute for steel reinforcement as bamboo is eco-friendly material, limiting the use of steel can reduce carbon dioxide emissions.

Ghavami (2005) conducted a study on bamboo reinforced structural concrete elements. The test results showed that the treatment of bamboo prior to use improved the bamboo–concrete bonding by more than 100%. By adopting q = 3% as the ideal value, the ultimate applied load increased by 400% as compared with concrete beams without reinforcement. The same methodology and concrete, as used for bamboo reinforced concrete beams, were applied to establish the mechanical and structural behavior of slab. The results of the investigations showed that bamboo can substitute steel satisfactorily.

Ghavami (1998) investigated the ultimate load behavior of bamboo-reinforced lightweight concrete beams. The results showed that for the bamboo-reinforced lightweight concrete beam, the ultimate applied load was increased up to 400% as compared with the concrete beams without bamboo reinforcement. It was also found that the 3% bamboo, in relation to the concrete section is the recommended value.

Swarmy (1984), compared steel reinforced concrete column after 10 service years with the first bamboo reinforced concrete beam which is part of the tunnel structure of Rio-s Metro at PUC Rio in 1979 (Fig.2.8). The bamboo reinforced beam after testing was exposed to open air. It was observed that the bamboo segment of the beam reinforcement, treated against insects as well as for bonding with concrete, was still in satisfactory condition after 15 years. However, the steel reinforcing bars of the column were severely corroded and need to be replaced. The bamboo segments of the beam were taken out of the tested concrete beam to establish its mechanical strength. Compared to the original untreated bamboo a slight deterioration of tensile strength was observed in the weathered samples of bamboo reinforcement. Beside the treatment of bamboo, extensive research however showed that the combination of low alkali cementitious materials and chemical admixtures could improve the durability of concrete reinforced with natural materials.

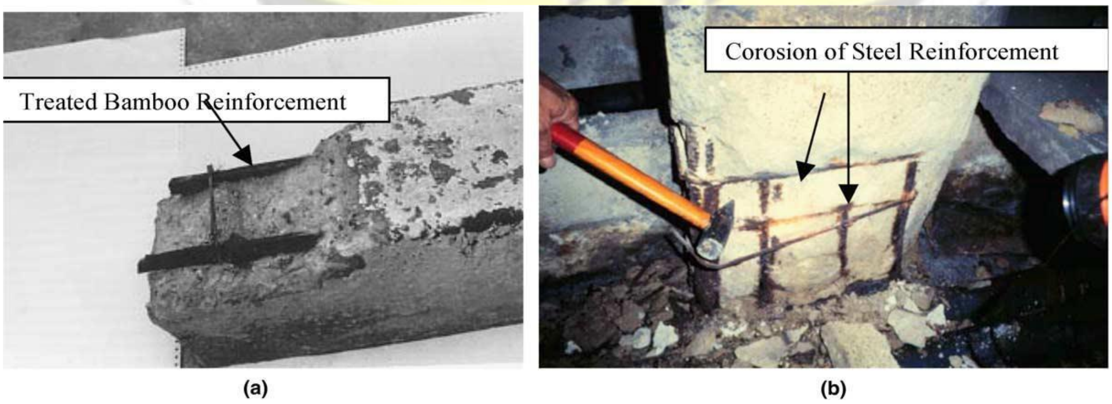


Fig.2.8: Durability of bamboo and steel reinforcement in concrete elements

(a) Bamboo reinforcement of a tested beam exposed in open air after 15years. (b) Steel reinforcement of a column in the tunnel of metro after 10years in closed area.

Moroz (2014) researched the performance of bamboo reinforced concrete masonry shear walls. The findings showed that if bamboo is to be used as an alternative reinforcing material, there is a need for further testing not only on in-plane shear, but also on other members in building systems.

Terai and Minami (2011) investigated the fracture behavior and mechanical properties of bamboo reinforced concrete members. The results showed that for bamboo reinforced concrete beams, the cracking patterns could be similar to steel reinforced concrete beams, and the predicted crack load of bamboo reinforced concrete beam gave a strong effect in comparison with the test data. Hence, the fracture behavior of bamboo reinforced concrete beam can be evaluated by the existing formula of reinforced concrete design. Also, for bamboo reinforced concrete column, the validity of the bamboo for the longitudinal bar and confining steel bar was clearly confirmed. The ductility of bamboo reinforced concrete columns was shown to be dependent on concrete strength.

Sevalia (2013) evaluated the feasibility of the use of bamboo as reinforcement in concrete members. In this study the bamboo was used as reinforcing material without any treatment and stirrups. From, the experiment, tension test performed on bamboo strip revealed that doubly reinforced beams performed more elastically and had about 29.31% more load carrying capacity than singly reinforced beams. Modulus of elasticity of the doubly reinforced beam is more than twice of modulus of elasticity of the singly reinforced beam.

Kariuki et al (2014) studied the flexural strength of laminated bamboo beams. Experimental results revealed that bamboo laminated beams resisted higher load than cypress beams and had a higher flexural strength of 39kN/mm2 against that of cypress beam of 34kN/mm2. Also, while cypress beams failed in flexure with major cracks, bamboo laminated beams however failed in tension on the lower part of the beams and shear along the grains. In addition, it was observed that bamboo laminated beams took longer duration than cypress beam to fail completely. The study hereby concluded that bamboo laminated beams showed better load carrying capacity than cypress beams.

Schneider et al. (2014) researched the application of bamboo for flexural and shear reinforcement in concrete beams. The results showed that bamboo is a viable alternative to steel as tensile reinforcement for concrete structures.

Lima et al. (2008) conducted a study on the durability analysis of bamboo as concrete reinforcement by changing the tensile strength and Young's Modulus of bamboo. The results did not show any significant variation on these mechanical properties, attesting the durability of bamboo. The experimental tests on the bamboo species Dendrocalamus *giganteus* showed that the bamboo tensile strength is comparable with the best woods used in construction and even with steel. The tensile stress versus strain curve of the bamboo is linear up to failure. Bamboo average tensile strength is approximately 280 MPa in the specimens without node and 100 MPa in the specimens with node. Sixty cycles of wetting and drying in solution of calcium hydroxide and tap water did not decrease the bamboo tensile strength neither the Young's Modulus.

Kute and Wakchaure (2014) studied performance valuation for enhancement of some of the engineering properties of bamboo as reinforcement in concrete.

Specimens with and without node were extracted from well-seasoned Dendrocalamus *strictus* variety of bamboo. The presence of node within the concrete specimen enhanced the bond strength by 15–22 % for all treatments. The bond strength of bamboo was less than mild steel (fy = 250 MPa) in same grade of concrete. Mechanical treatments like notching, nailing or winding binding wire around the specimen showed improved bond strength but there was wide variation in the results. The coating of bituminous reduced water absorption by 75 % but also decreased the bond stress by 10 %. Fine zeolite powder applied with coat of oil or bituminous paint in wet condition improved the bond stress by 50–90 % to that of untreated specimens.

Zhang et al (2012) conducted an experimental study on mechanical performance of bamboo fiber reinforced concrete. The experimental results show that bamboo fibers can enhance the cubic compressive strength and remarkably improve the splitting tensile strength of concrete. The cube specimens of bamboo fiber concrete with dimension of 150mm were adopted. As lengths of bamboo fiber keep uniform, the value of cubic compressive strength is the highest when bamboo fiber content is 0.26. As content of bamboo fiber keep constant, the value of cubic compressive strength decrease with length of bamboo fiber. The value of cubic compressive strength of specimen 1-0.26 is the highest among the bamboo fiber reinforced concrete specimen which is 26.25% higher than that of plain concrete. The study revealed that as lengths of bamboo fiber kept uniform; the value of splitting tensile strength was the highest when bamboo fiber content is 0.26. The value of splitting tensile strength of specimen 1-0.26 is the highest among the bamboo fiber reinforced concrete specimen which is 97.21% higher than that of plain concrete. The cubic compressive strength and splitting tensile strength of specimen 1-0.26 are the highest and shows good feature of ductility.

The above literature and other studies on bamboo have proven that it can be used in the construction of especially low cost housing facilities.

2.9 Summary of bamboo

Longitudinal reinforcement is provided in structural members to resist tensile stresses. Concrete is weak in tension and the presence of longitudinal reinforcement helps it to behave in a ductile manner by deflecting adequately to give ample warning before collapse. Steel has been the traditional material for longitudinal reinforcement but researchers have tried to use other materials such as bamboo, babadua et cetera to replace steel in low cost houses as a result of the high cost of steel reinforcement and environmental issues such as global warming (Calwell, 2015). As said earlier, the longitudinal reinforcement apart from its basic role in tension also contributes to the shear resistance of the structural member. Bamboo has been known to have similar tensile and compressive properties as steel but also has great limitations such as water absorption and large variations in properties between species, growth locations and maturity. The harvesting and seasoning procedures also causes differences in its structural performance (Harish et al., 2012, Fergusson-Calwell, 2015, Chandra et al., 2013, Rahman et al., 2011).

Adom-Asamoah and Afrifa (2011), Khan (2014) and other researchers found out that the tensile strength of bamboo is approximately one half that of mild steel and modulus of elasticity is approximately one third that of mild steel but others such as Harish et al. (2012) have concluded that the tensile strength of bamboo is approximately equal to that of mild steel. This difference is due to the different species of bamboo used in their research work. Swarmy (1984) and Lima et al. (2008) worked on the durability of bamboo and found it satisfactory. Swarmy (1984) exposed bamboo reinforced beam to open air after testing and found it to be in satisfactory condition after 15 years. Lima et al. (2008) on the other hand found out that the bamboo tensile strength and Young's Modulus did not decrease after subjecting it to 60 cycles of wetting and drying in solution of calcium hydroxide and tap water. Again the tensile stress versus strain curve of the bamboo was found to be linear up to failure just like steel.

2.10 Bamboo Reinforced SCC

The enormous benefits of green and hardened SCC such as its flowability and self compactibility as well as the versatility and viability of bamboo as a replacement for steel can be combined to make stronger and durable bamboo reinforced concrete.

Studies and literature on the use of bamboo reinforcement in SCC are very minimal, but knowledge on the performance of bamboo in NC as presented above serves as guide in producing bamboo reinforced SCC.

Adom-Asamoah and Afrifa (2011) in their research on the comparative study of Bamboo reinforced concrete beams using different stirrup materials for rural construction assumed bamboo to have material factor of safety of 3.0 as compared to 1.05 by steel. The characteristics strength of bamboo was pegged at 126N/mm^2. This meant that bamboo reinforced structures are as twice congested as compared to that of steel of the same strength. As such, the use of SCC is highly commendable and imperative in bamboo reinforced construction.

2.11 Conclusion

There is not much information on the structural behavior of bamboo reinforced SCC but looking at the properties and structural behavior of SCC and bamboo reinforced NC in literature, it can be inferred that the combination of bamboo and SCC in the production of bamboo reinforced SCC could result in improving the concrete's mechanical and durability properties. It is therefore imperative to research into this area.

CHAPTER 3: METHODOLOGY

3.1 Introduction

This chapter looks at the processes in which the experiment was carried out to examine the performance of bamboo in SCC and NC in beams and slabs. Concrete cubes and prisms were cast for compressive strength and modulus of rupture (MoR) tests respectively for SCC and NC. The experimental procedures included the selection of materials, concrete mix design as well as preparation and testing of concrete.

3.2 Source and preparation of materials

Granite aggregates of sizes 10mm and 20mm were acquired from KAS Quarry Site at Booho, washed river sand which served as the fine aggregates was obtained from Apromase and the bamboo of the species Bambusa *vulgaris* used as the reinforcement was from the KNUST botanical gardens all in Kumasi in Ashanti Region of Ghana. Limestone Portland cement of strength grade 32.5R produced by Ghacem was the binder for concrete production. Potable water from Ghana Water Company Limited (GWCL) was used for mixing of concrete. Rheobuild 561, a super plasticizer free from chlorides that meets ASTMC 494-92 requirements for type A and F was used to enhance the flowability of concrete without any change in water cement ratio. Twelve millimeter (12mm) thick plywood was used as formwork for the casting of beams and slabs into the required shape and size.

3.3 Concrete mix design

A mix ratio of 1:1.5:3 for cement, fine and coarse aggregates, respectively, with a water/cement ratio of 0.5 was used for NC. This was redesigned to obtain an equivalent SCC mix ratio using coarse to fine aggregate ratios from the studies of Girish et al (2010), Ryan et al (2016), Safawi et al (2004), Lachemi et al (2005) and

Hassan et al (2008). The coarse aggregate content was reduced by 40% and replaced with an equal measure of fine aggregates. The water/cement ratio was maintained and 0.0012l/kg of superplasticizer was added to obtain an adequate slump flow as specified by the SCC guidelines. This resulted in a mix ratio of 1:2.7:1.8. The mix ratios of NC and SCC are presented in Tables 3.1 and 3.2, respectively.

Table 3.1. Mix proportions of NC

Cast Element	Cement Content/kg/m3	Cement/Sand/Coarse Aggregate (Mix Ratio)	Water Cement Ratio, W/C
Beams	450	1: 1.5: 3	0.5
Slabs	450	1: 1.5: 3	0.5

Table 3.2. Mix proportions of SCC

Cast Element	Cement Content/kg/m3	Cement/Sand/Quarry Dust/ Coarse Aggregate (Mix Ratio)	Water Cement Ratio, W/C	Admixture/HRWRA l/100 kgcement
Beams	450	1: 2.16: 0.54: 1.8	0.5	1.2
Slabs	450	1: 2.16: 0.54: 1.8	0.5	1.2

3.4 Reinforcement

Bamboo of the species Bambusa *vulgaris* was used as the reinforcement for this study. The beams were designed to fail in shear and the slabs flexure. All the flexural reinforcements were placed at depth less the cover to reinforcement, beyond the supports to achieve adequate end anchorage. The beams were reinforced with two different flexural reinforcement ratios of 1.5% and 3% whereas the slabs had three different reinforcement ratios of 1%, 2% and 3%.

3.5 Concrete properties and nomenclature

Three variables including beam depth (h), longitudinal reinforcement ratio (ρ) and concrete type (SCC and NC) were considered in the study. All the beams were

designed to fail in shear and the slabs flexure. The beams and slabs were of rectangular cross section with width of 110mm and 300mm respectively. The shear span-to-effective depth ratios (a_v/d) were kept at 2.5 and 1.8 for the slabs and beams, respectively. A total of 12 concrete beams (6 BRSCC and 6 BRNC) and 10 slabs (5 BRSCC and 5 BRNC) were cast and tested to failure to determine their flexural and shear strength. Bamboo was used as the longitudinal reinforcement and all beams were without transverse reinforcement. Three different types of beams with dimensions 275×110×2000mm, 250×110×1200mm and 150×110×1050mm were
cast. Each type was singly reinforced and had longitudinal reinforcement ratios of 3% and 1.5%. One way slabs of dimension 80×300×1000mm were also of reinforcement ratios of 1%, 2% and 3%. In each group (BRSCC and BRNC), there were two samples each with 1% and 3% longitudinal reinforcement ratio. In addition, 12 concrete cubes and 12 prisms were cast for 28 day compressive and MoR tests, respectively.

Beams labeling were according to the type of concrete, the flexural reinforcement percentage, the order in which they were cast, the batch of concrete used and the length of the beam. The labeling can be described in general as:
SC =self-compacting concrete and NC = Normal Concrete

Figures 1, 1.5, 2 and 3 to percentage reinforcement ratios

I, II, III, IV, V and VI refer to the order in which the specimen were cast.

“a” and “b” refer to the batch of concrete that was used in the casting (1st or 2nd).
The 1050mm, 1200mm and 2000mm referred to the length of the specimen.

For instance, SC3Ia1050 implies SCC beam with 3% flexural reinforcement which is the first SCC sample to be cast, the first batch of concrete was used for the casting and it has a length of 1050mm. NC1.5IVb1200 also implies NC beam with 1.5% flexural reinforcement which is the fourth sample to be cast, the second batch of concrete was used for its casting and it has a length of 1200mm.

Similarly, slabs were labeled according to their type of concrete, the flexural reinforcement percentage and the order in which they were cast.

For instance, SC3I implies SCC slab with 3% reinforcement which is the first SCC sample to be cast. NC1IV also implies NC slab with 1% reinforcement which is the fourth sample to be cast. Due to the uneven sizes of the bamboo strips, the area of reinforcement was not exactly the same. The beams and slabs are described in Tables 3.3 and 3.4 respectively:

Table 3.3. Beam description

Beam No.	B x D (mm x mm)	Length (mm)	Height (mm)	Shearspan to depth ratio (a/d)	Bamboo reinforcement		
					Area of Bamboo Ab(mm^2)	Percntage of longitudinal reinforcement (%)	Bamboo strength F_b (N/mm^2)
SC3Ia2000	110 x 250	2000	275	1.8	877	3	126.72
SC1.5IIb2000	110 x 250	2000	275	1.8	476	1.5	126.72
SC3IIIa1200	110 x 225	1200	250	1.8	826	3	126.72
SC1.5IVb1200	110 x 225	1200	250	1.8	429	1.5	126.72
SC3Va1050	110 x 125	1050	150	1.8	498	3	126.72
SC1.5VIb1050	110 x 125	1050	150	1.8	255	1.5	126.72
NC3Ia2000	110 x 250	1200	275	1.8	919	3	126.72
NC1.5IIb2000	110 x 250	2000	275	1.8	464	1.5	126.72
NC3IIIa1200	110 x 225	1200	250	1.8	816	3	126.72
NC1.5IVb1200	110 x 225	1200	250	1.8	400	1.5	126.72
NC3Va1050	110 x 125	1050	150	1.8	504	3	126.72
NC1.5VIb1050	110 x 125	1050	150	1.8	256	1.5	126.72

Table 3.4. Slab description

SLAB No.	B x D	Length	HEIGHT		Bamboo reinforcement

	(mm x mm)	(mm)	(mm)	Shear-span to depth ratio (a/d)	Area of Bamboo Ab(mm^2)	Percntage of longitudinal reinforcement (%)	Bamboo strength F_b (N/mm^2)
SC3I	300 x 55	1000	80	2.5	763	3	126.72
SC3II	300 x 55	1000	80	2.5	729	3	126.72
SC2III	300 x 55	1000	80	2.5	558	2	126.72
SC1IV	300 x 55	1000	80	2.5	325	1	126.72
SC1V	300 x 55	1000	80	2.5	314	1	126.72
NC3I	300 x 55	1000	80	2.5	763	3	126.72
NC3II	300 x 55	1000	80	2.5	729	3	126.72
NC2III	300 x 55	1000	80	2.5	528	2	126.72
NC1IV	300 x 55	1000	80	2.5	324	1	126.72
NC1V	300 x 55	1000	80	2.5	324	1	126.72

3.6 Mixing and casting of concrete beams and slabs

Mixing of concrete beams and slabs was done in the laboratory with the help of a concrete mixer, loading containers, weighing scale, wheelbarrows, shovels.

The designed water amount was measured and half of the water was poured into the drum of the concrete mixer, the aggregates and the cement were measured and poured into the loader of the concrete mixer which was then lifted into the drum. The concrete was allowed to mix for about a minute before the remaining water was added and then allowed to mix for about 3-5 minutes to achieve a uniform workable paste. For SCC mix, the superplasticizer was mixed with about 50% of the remaining water at this stage and added to the concrete and allowed to mix for some time. The rest of the water was then added to obtain a homogenous and workable mixture as before. The mix was then poured into a wheelbarrow and wheeled to the already fabricated formwork with their reinforcement in place. Tests on the fresh properties of concrete were immediately carried out. Fresh concrete was poured into the formwork and compacted in three equal layers using the poker vibrator. Samples of concrete mixes were used to cast the 150mmX150mmX150mm concrete cubes and the100mmX100mmX500mm unreinforced concrete beams (prisms) for the 28 days compressive and modulus of rupture (MoR) tests, respectively. The formwork for the beams, slabs, cubes and prisms were strike after 24 hours and the cast materials cured until their test on 28days. The beams and slabs were cured using damp jute sacks whereas the cubes and prisms were submerged in water in a tank. Before testing, the surface of the beams and slabs was given white painting to allow for easy detection and movement of cracks.

3.7 Experimental set-up and testing of specimens

Twelve (12) simply supported reinforced concrete beams and ten slabs were cast and tested up to failure under a four point loading test at 2KN per minute incremental loads. An over-hang of 50mm was provided beyond the support points to ensure adequate anchorage of the flexural reinforcement. A hydraulic steel jack supported on a rigid steel frame was used to produce the load at the top whilst the bottom face was supported on two simply supported ends to define shear span-toeffective depth ratios (a/d) of 2.5 and 1.8 for slabs and beams, respectively. The load was applied through an I-section steel spreader placed on two cylindrical steel bars. This facilitated the symmetrical transfer of loads to ensure pure bending in the mid-span of the specimen. A dial gauge reading to 0.001 mm accuracy was placed in the mid-span to record the central deflection of the beams and slabs. The detection of the appearance of cracks was done by visual inspection. A crack microscope of 0.02mm was also used to measure the width of cracks on the surface of the beams and slabs. The crack lengths were measured with a rope and transferred onto a measuring tape. The schematic and experimental set ups are as shown in Fig. 3.1 and 3.2

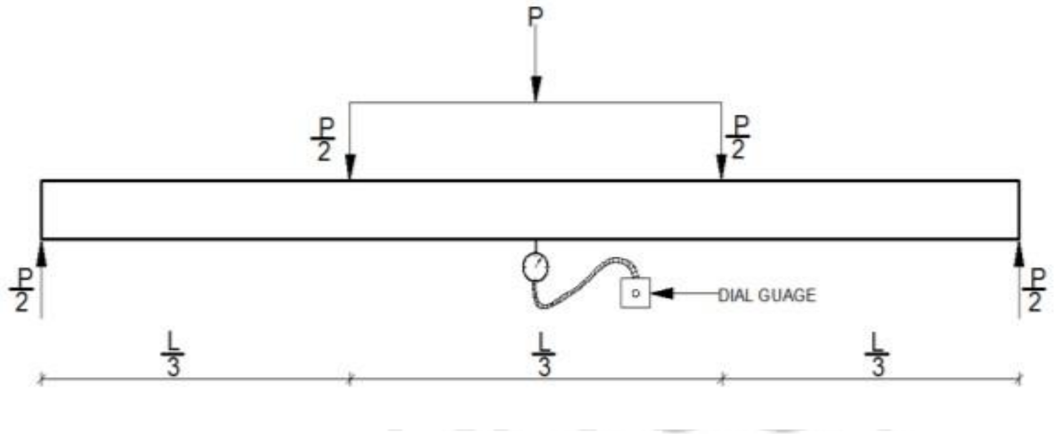


Fig.3.1. Schematic specimen testing setup

Fig.3.2. Experimental set-up

3.8 Conceptual framework for analysis

3.8.1 Flexural, shear and crushing strength of beams

From BS 8110: Part 1, a simply supported beam without transverse reinforcement under a four-point loading will yield an ultimate flexural, shear and crushing loads given by Equations (3.1), (3.2) and (3.3), respectively. The equations for the nominal shear capacities of the beams for ACI, CSA and EC 2 are also presented in Equations (3.4)-(3.7) to allow for comparison with the experimental values obtained.

$$P_{ult} = 2\left(M_{ult} - \frac{wL^2}{8}\right) / a_v \qquad (3.1)$$

Where M_{ult} denotes ultimate moment of resistance;

w is the self-weight per unit length of the sample; L is the span and

a_v is the shear span.

$$Pult = 2(V_c bd) \qquad (3.2)$$

$$V_c = 0.79\left[\frac{100A_s}{b_v d}\right]^{\frac{1}{3}}\left[\frac{400}{d}\right]^{\frac{1}{4}}\frac{1}{1.25}\left[\frac{f_{cu}}{25}\right]^{\frac{1}{3}} \qquad (3.2a)$$

Where, V_c is the designed concrete shear strength, b is the breadth of the sample section, d is the depth of the section, A_s is the area of reinforcement, f_{cu} is the characteristic strength of concrete.

$$P'cr = 2(0.156 f_{cu} bd^2) / a_v \qquad (3.3)$$

Where, f_{cu}, b, d^2 and a_v are as defined above

The ACI 318-11 equation for nominal shear capacity of concrete is given by:

$$V_c = \left(0.158\sqrt{f'_{cu}} + 17.14\frac{\rho V_u d}{M}\right) bd \text{ KN} \qquad (3.4)$$

Where,

Vu is factored shear force at section; M is factored moment at section; b is the beam width; d is the effective depth of beam cross-section; f'cu is compressive strength of concrete; $\rho$is the percentage of longitudinal reinforcement.

The CSA-A23.3-14 equation for nominal shear capacity of concrete is given by:

$$V_c = \varphi\lambda\beta\sqrt{f'_c}\, b_w d_v \text{KN} \qquad (3.5)$$

Where, φ is the resistance factor for concrete. By default it is taken as 0.65 (CSA8.4.2), λ is the strength reduction factor to account for low density concrete (CSA3.2), for normal density concrete, its value is 1 which is taken in this research (CSA 8.6.5), *bw* is the effective web width (for rectangular beam, it is the width of the beam), *dv* is the effective shear depth taken as the greater of 0.9d or 0.72h, where *d* is the effective depth and *h* is the overall height of the section (CSA 3.2). β is the factor for accounting for the shear resistance of cracked concrete

(CSA3.2). Its value is normally between 0.05 and 0.4. It is determine according to Equation 3.5a

$$\beta = \frac{230}{1000+d_v} \qquad (3.5a)$$

The EC-2-2004 equation for nominal shear capacity of concrete is given by:

$$V_{Rd,c} = \left[C_{Rd,c}k(100\rho l f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right] b_w d \,\text{KN} \qquad (3.6)$$

With a minimum of: $V_{Rd,c} = (v_{\min} + k_1\sigma_{cp})\, bwd$ (3.7)

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2 \text{ (d is in mm)} \qquad \text{(EC2 6.2.2)}$$

$$\rho l = \frac{A_s}{b_w d} \leq \quad 0.02$$

$\sigma_{cp} = N_{Ed}/A_c < 0.2\, f_{cd}$ (in MPa)

The effective shear area, Ac is taken as bwd. The factor k_1 = 0.15 and the values of $C_{Rd,c}$ and v_{min} are determined as:

$C_{Rd,c}=0.18/\gamma c$ (EC2 6.2.2)

$v_{min}=0.035k^{3/2}fck^{1/2}$ (EC2 6.3)

Ned is the factored axial load at a section, fcd Design concrete compressive strength, γc Material partial factor for concrete.

In BS code, cube compressive strength is used as compared to cylinder compressive strength used in ACI, EC2 and CSA codes. A reduction factor of 0.8 was used to convert cube compressive strength into cylinder compressive strength. The cube compressive strength and the concrete flexural strengths are given by Equations (3.9)-(3.10):

$$\sigma = \frac{P}{A} \qquad (3.9)$$

$$\sigma = \frac{3PL}{2bd^2} \qquad (3.10)$$

Where,

P is the applied load, A is the area of the specimen, L is the length, b is the breadth and d is the effective depth.

Due to the difference in SCC and NC compressive strengths, the loads were normalized with the use of Equation (3.11) before being applied for the load deflection curves.

$$P_{norm} = \frac{P_{ult}}{bd\sqrt{f_{cu}}} \qquad (3.11)$$

3.9 Instrumentation

The behavior of the specimen under load was analyzed by taking measurements using different instrumentations positioned at different zones. Parameters such as first crack load, failure load, mid-span deflection and its corresponding load, etc., were recorded. The specimens were tested using a 230kN capacity load-controlled actuator and the above parameters were recorded with respect to their manifestation at each incremental load.

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 Introduction

This section deals with the presentation of results on (1) fresh and mechanical properties of test specimens, (2) comparative evaluation of shear capacities of the various beams, (3) evaluation of flexural capacities of slabs and (4) comparison of code estimation for shear and flexural strength to observed experimental responses. In addition, discussions of the variations in SCC and NC beams and slabs with bamboo reinforcement are thoroughly outlined.

4.2 Fresh concrete properties

The fresh concrete samples were tested for their workability using the slump cone. In addition, the T500 test was performed on the SCC concrete. Results of test result on the fresh concrete are provided in Table 4.1.

Table 4.1. Fresh concrete properties

Concrete	Slump flow/mm	T500/sec.
SCC- beams	608	8

SCC – slabs	562	10
NC – beams	22	
NC - slabs	18	

The slump flows of 562 mm and 608 mm and the T500 values of 8 seconds and 10 seconds for the SCC are within the acceptable limits of the SCC guide for flowability (EFNARC, 2001). The slumps were observed to be uniformly mixed without segregation. Slump test on the NC showed a true slump and adequate workability.

4.3 Test on hardened concrete

Results of the 28 days compressive and flexural test for beams and slabs are presented in Tables 4.2 and 4.3 below.

Table 4.2. Beam description

Beam Designation	B x D (mm x mm)	Length (mm)	Shear-span to depth ratio (a/d) (mm)	Percentage of longitudinal reinforcement (%)	28 days concrete cube compressive strength f_{cu}(N/mm^2)	28 days concrete flexural strength f_{cr}(N/mm^2)
SC3Ia2000	110 x	2000	1.8	3	26.45	3.6
SC1.5IIb2000	110 x	2000	1.8	1.5	26.45	3.6
SC3IIIa1200	110 x	1200	1.8	3	26.45	3.6
SC1.5IVb120	110 x	1200	1.8	1.5	26.45	3.6
SC3Va1050	110 x	1050	1.8	3	26.45	3.6
SC1.5VIb105	110 x	1050	1.8	1.5	26.45	3.6
NC3Ia2000	110 x	1200	1.8	3	32.65	4
NC1.5IIb200	110 x	2000	1.8	1.5	32.65	4
NC3IIIa1200	110 x	1200	1.8	3	32.65	4
NC1.5IVb12	110 x	1200	1.8	1.5	32.65	4
NC3Va1050	110 x	1050	1.8	3	32.65	4
NC1.5VIb10	110 x	1050	1.8	1.5	32.65	4

Table 4.3. Slab Description

Slab Designation	B x D (mm x mm)	Length (mm)	Shear-span to depth ratio (a/d) (mm)	Percentage of longitudinal reinforcement (%)	28 days concrete cube compressive strength f_{cu}(N/mm^2)	28 days concrete flexural strength f_{cr}(N/mm^2)
SC3I	300 x 55	1000	2.5	3	26.45	3.6
SC3II	300 x 55	1000	2.5	3	26.45	3.6
SC2III	300 x 55	1000	2.5	2	26.45	3.6
SC1IV	300 x 55	1000	2.5	1	26.45	3.6
SC1V	300 x 55	1000	2.5	1	26.45	3.6
NC3I	300 x 55	1000	2.5	3	26.45	3.6
NC3II	300 x 55	1000	2.5	3	26.54	3.6
NC2III	300 x 55	1000	2.5	2	26.45	3.6
NC1IV	300 x 55	1000	2.5	1	26.45	3.6
NC1V	300 x 55	1000	2.5	1	26.45	3.6

4.3.1 Cube compressive strength

The 28-day compressive strength results were 26.45 N/mm^2 for slabs (SCC and NC) and SCC beams and 32.65 N/mm^2 for NC beams. Despite the difference in coarse aggregate content between NC and SCC samples, there was not much variation in their compressive strengths. Similar study on SCC and NC by Lachemi et al. (2005) recorded averagely the same values for compressive strength. This is an indication that the SCC and NC beams are expected to have similar mechanical characteristics since the compressive strength is strongly related to them (Smith and Vantsiotis 1982). The results in this study showed that the SCC beams produced shear strength ranging from 12kN to 36kN whereas that of NC ranged from 16kN to 38kN for compressive strengths of 26.45N/mm2 and 32.65N/mm2, respectively. From the results, as the compressive strength increased by 23% from 26.45 to 32.65N/mm2, the average shear strengths of the beams increased by 11%. This suggests that, the compressive strength of the concrete influences the shear strength of beams without shear reinforcement, and that higher compressive strengths will generate higher shear strengths. On the contrary, El-Sayed et al. (2006) stated that a 45% increase in the concrete compressive strength was only accompanied by a 10% shear strength increase. But this study shows that the level of increase of shear strength as a function of the compressive strength could be higher. Generally, the rate of increase of shear strength in response to an increase in compressive strength was higher in NC than in SCC. This increase can be attributed to the higher bond strength in NC as compared to SCC due to the higher content of coarse aggregates in the former than the latter (Helincks et al., 2013; Lachemi et al., 2005).

4.3.2 Concrete flexural strength

The Modulus of Rupture (MoR) was computed to determine the flexural strength of the concrete samples. The results of the MoR test were recorded as 3.6 N/mm^2 for slabs (SCC and NC) and SCC beams and 4 N/mm^2 for NC beams, respectively as seen in Tables 4.2 and 4.3 above. Katz (2003) reported in his work that the percentage ratio of the flexural strength to the compressive strength ranged from 1623%. In this study, the flexural strength of the concrete samples (MoR) was within 12.3% - 13% of the compressive strength.

It can be seen from Tables 4.2 and 4.3 above, that the flexural strengths for NC and SCC concrete samples were similar and related directly to the compressive strength. This confirms study from researchers such as Lachemi et al., (2005) and Smith and Vantsiotis (1982).

4.4 Shear strength characteristics of beams

Fig. 4.1 shows a typical normalized shear stress–deflection curve. The mode of failure observed for all BRSCC and BRNC beams tested was shear dominant. The load deformation relationship for the beams and a comparative assessment of the shear capacities at cracking and ultimate failure is presented and metrics such as P_{cr}/P_{ult} (an elastic measure), damage index and energy dissipation capacity are used for discussion.

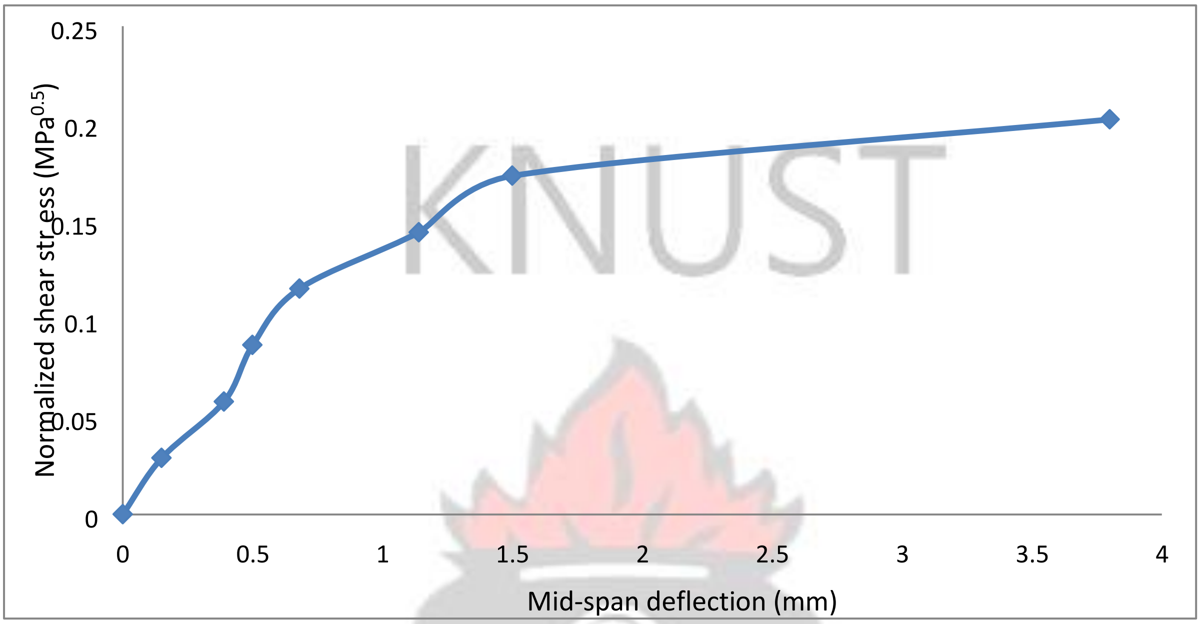


Fig. 4.1. Typical shear stress-deflection curve

4.4.1 Failure modes of beams

Both the BRSCC and BRNC beam samples exhibited similar failure modes implying that there is not much difference between them in terms of failure mode. All the beams failed by diagonal-splitting of the concrete between the loading point and the support points. During loading, flexural cracks appeared first around the mid-span of the beams and as the load increased, similar flexural cracks appeared at other points along the span and were followed by a shear crack which started from the tip of the support at the bottom of the beam and moved towards the load application point on the beams. The shear crack opened up rapidly once it appeared and caused a sudden failure of the beams. Typical examples of the failure mode and crack patterns of the beams are shown in Figs 4.2 and 4.3

Fig. 4.2. Beam NC3I2000 after failure

Fig. 4.3. Beam SC3IIIa1200 after failure

4.4.2 Ultimate Shear Strength of beams

The ultimate shear strength of the beams is influenced by different factors as has been established in literature. The results of the shear strength as shown in Table 4.4 outlines some of the factors such as reinforcement ratio, beam depth, beam length, etc., that the current study considered and discussed.

Table 4.4. Characteristic properties and ultimate strength of beams

Beam ID	BxDxL (mm)	h (mm)	fcu (N/mm2)	ρ (%)	Load at first Crack (KN) Flex-ural	Shear crack	Max. mid-span deflection (mm)	P_{max} (KN)	Norm. P_{max} (KN)
SC3Ia2000	110x250x2000	275	26.45	3	24	28	6.84	36	0.24
SC1.5IIb2000	110x250x2000	275	26.45	1.5	20	24	4.58	26	0.18
SC3IIIa1200	110x225x1200	250	26.45	3	18	22	5.36	28	0.22
SC1.5IVb1200	110x225x1200	250	26.45	1.5	14	18	6	20	0.16
SC3Va1050	110x125x1050	150	26.45	3	10	14	4.05	18	0.25
SC1.5VIb1050	110x125x1050	150	26.45	1.5	8	10	4.22	12	0.17
NC3Ia2000	110x250x2000	275	32.65	3	28	34	7.85	38	0.24
NC1.5IIb2000	110x250x2000	275	32.65	1.5	24	26	3.88	28	0.18
NC3IIIa1200	110x225x1200	250	32.65	3	20	26	4.67	32	0.23
NC1.5IVb1200	110x225x1200	250	32.65	1.5	16	20	5.8	22	0.16
NC3Va1050	110x125x1050	150	32.65	3	14	16	3.73	18	0.23
NC1.5VIb1050	110x125x1050	150	32.65	1.5	10	12	3.8	16	0.20

4.4.3 Effect of longitudinal reinforcement ratio on shear strength

Comparing each beam type SC32000 and SC1.52000, SC31200 and SC1.51200, SC31050 and SC1.51050, NC32000 and NC1.52000, NC31200 and NC1.51200, NC31050 and NC1.51050 which were reinforced with either 3% and 1.5% longitudinal reinforcement, they generated a consequence shear force of 36kN and 26kN, 28kN and 20kN, 18kN and 12kN, 38kN and 28kN, 32kN and 22kN and 18kN and 16kN, respectively at failure. On the average, beams with 3% longitudinal reinforcement were 36% more resistant to shear than their 1.5% counterparts. This suggests that as the flexural reinforcement ratio increased in both BRNC and BRSCC beams, the shear strength showed a corresponding increase. This contradicts the findings of Mau and Hsu (1989) and Londhe (2011) who found that the average shear strength of beams increases linearly as the longitudinal reinforcement ratio increases up to 1.5% and beyond that it flattens.

On the average, when the percentage longitudinal reinforcement was increased from 1.5% to 3%, BRSCC beams gained 40% in strength as compared to 33% by their BRNC counterparts. Hence the percentage of longitudinal reinforcement has greater effect on the shear strength of BRSCC than BRNC beams without shear reinforcement. This is as a result of lower aggregate bond strength in BRSCC beams which gives them lower concrete shear capacity as compared to their BRNC counterparts (Helincks et al., 2013).

4.4.4 Effect of beam depth on shear strength

Three different sets of beam sizes of 150, 250 and 270mm were tested for both BRSCC and BRNC samples with each set of size containing two beams. The average results for the BRSCC beams were 15kN, 24kN and 30.5kN for 150, 250 and 270mm beams respectively. The average results of the BRNC samples were also 17kN, 27kN and 33kN for 150, 250 and 270mm beams respectively. It can be seen from the results shown in Fig. 4.4 that the shear strength of the beams increased with increasing depth for both BRSCC and BRNC samples. This is contradictory to the finding of Walsh (1972) who concluded that the critical strength of concrete beams decreases with increase in the beam depth, with a distinctive depth of 225 mm being the depth above which the phenomenon becomes evident.

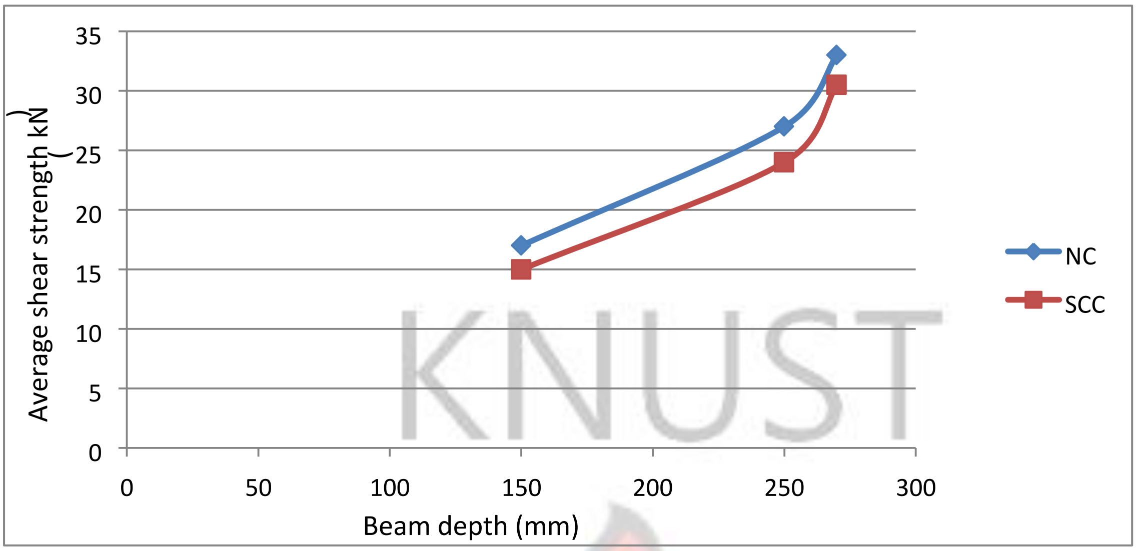


Fig. 4.4. Effect of beam depth on NC and SCC beams without shear reinforcement

4.4.5 Flexural and strut cracking response of beams

During loading, all the beams exhibited flexural cracking first, after which diagonal cracks appeared at higher loads. The appearance of the first flexural crack and its corresponding load is depended largely on the size and concrete strength of the beams. The load for the first flexural crack was between 56% and 77% of ultimate load for BRSCC beams and 63% to 86% of ultimate load for their BRNC

counterparts (Table 4.5). As the size of the beams increased from 150 mm (SC31050 and SC1.51050) through 250 mm (SC31200 and SC1.51200) to 270 mm (SC32000 and SC1.52000), the average load for the first flexural crack of the BRSCC beams increased from 57.1% to 61.1% to 73.8% of their ultimate loads respectively. For the BRNC beams, the average load for the first flexural crack of the 150 mm size beams (NC31050 and NC1.51050) was 70.1% of its ultimate load and was higher than that of the 250 mm beams (NC31200 and NC1.51200) (67.6%); but the 270 mm beams (NC32000 and NC1.52000) recorded the highest average load of 79.7% of their ultimate load. In terms of concrete strength, the BRNC and BRSCC beams with compressive strengths of 32.65 N/mm^2 and 26.45 N/mm^2, respectively, recorded an average value of 18.7 kN for the first flexural crack at 72.5% of the ultimate strength and 15.7kN at 67.3% of the ultimate loads respectively.

Table 4.5. Loads at first flexural and strut cracks and percentage of ultimate load for Beams

Beam ID	Failure load	First flexural crack load			Strut crack load		
	Vu	V_s (KN)	% V_u	Average %Vu	V_{cr} (KN)	% V_u	Average %Vu
SC3Ia2000	34	24 20	70.6	73.8	28	82.4	87.3
SC1.5IIb2000	26		76.9		24	92.3	
SC3IIIa1200	28	18	64.3	67.1	22	78.6 90.0	84.3
SC1.5IVb1200	20	14	70.0		18		
SC3Va1050	18	10	55.6	61.1	14	77.8 83.3	80.6
SC1.5VIb1050	12	8	66.7		10		
NC3Ia2000	38	28	73.7	79.7	34	89.5	91.2
NC1.5IIb2000	28	24	85.7		26	92.9	
NC3IIIa1200	32	20	62.5	67.6	26	81.3	86.1
NC1.5IVb1200	22	16	72.7		20	90.9	
NC3Va1050	18	14 10	77.8	70.1	16	88.9	81.9
NC1.5VIb1050	16		62.5		12	75.0	

From the results, it can be deduced that the first flexural crack loads occurred earlier in BRSCC than in BRNC beams. On the average, the BRNC beams had their first flexural crack load occurring at 3% higher than their BRSCC counterparts. Generally, as the depth of the BRSCC beams increased from 150mm through 250mm to 270mm, the load for the first flexural crack also increased from 9kN to 16kN to 22kN respectively at 61.1%, 67.1% and 73.8% of their ultimate loads respectively. The BRNC beams on the other hand had their loads increased from 12kN (70.1%) to 18kN (67.6%) to 26kN

(79.7%) when the beam depths respectively increased from 150mm to 250mm to 270mm.

Strut or diagonal cracks, once they emerged, continued to increase in width and length more rapidly than the initial flexural cracks. This continues until the major diagonal or strut crack propagated from the support to the point of application of the load. The strut crack loads were between the averages of 80.6% and 87.3% of ultimate loads for the BRSCC beams but 81.9% and 91.2% for their BRNC counterparts. The lowest percentage of 80.6% and 81.9% for BRSCC and BRNC beams, respectively, occurred in the beam with the smallest depth of 150mm whilst the maximum percentage of 87.3% and 91.2% for BRSCC and BRNC beams, respectively also occurred in the beams with the deepest depth of 270mm. In addition, the 150mm deep beams which had the lowest percentage longitudinal reinforcement of 1.5% recorded the least percentages of 77.8 and 75 for BRSCC and BRNC beams respectively. This shows that these beams had the highest postcracking strength and hence were most ductile. This observation confirms those of other researchers on NC beams that a decrease in tensile reinforcement improved ductility (Lim et al, 2006; Ashour, 2000, and Shuaib and Ray, 1991).

Again, it was also observed that the strut crack load varied from 80.6% to 87.3% of the failure load for BRSCC beams and from 81.9% to 91.2% of the failure load for BRNC beams as depth increased. Furthermore, it was observed that as concrete compressive strength increased, the first strut load as a percentage to the ultimate load increased from 84.1% to 86.4%.

4.5 Load-deflection response of beam

The load-deflection characteristics of BRSCC and BRNC beams are outlined in this section. Generally, all the beams exhibited a fairly linear behavior before the formation of the first crack, since the slope of the load-deflection curves was almost constant in this range. The first crack, which was generally flexural, was mainly influenced by the flexural strength of the concrete.

4.5.1 Beams with varying depths

Figs. 4.5 to 4.7 compare the load-deflection responses of the beams with respect to their depth. The figures show that as the depth of the beams increased from 150mm through 250mm to 270mm, the stiffness of the beams increased

accordingly, as portrayed by the shifting of the curves towards the right side in their respective order. This implies that both BRSCC and BRNC beams obey the general phenomenon of reinforced concrete beams, as an increase in depth results in an increase in the stiffness and hence the shear strength of the beams (Lachemi et al., 2005; Smith and Vantsiotis, 1982 and Londhe, 2011). It is observed that in each category, the BRSCC beams exhibited similar deflection characteristics as the BRNC during the initial stages of load application, but deflected more and failed at relatively lower load at the latter stages of loading indicating higher ductility. The high load resisting performance of BRNC compared to BRSCC beams can be attributed to the presence of high aggregate interlock in BRNC beams compared to their BRSCC counterparts (Lachemi et al., 2005).

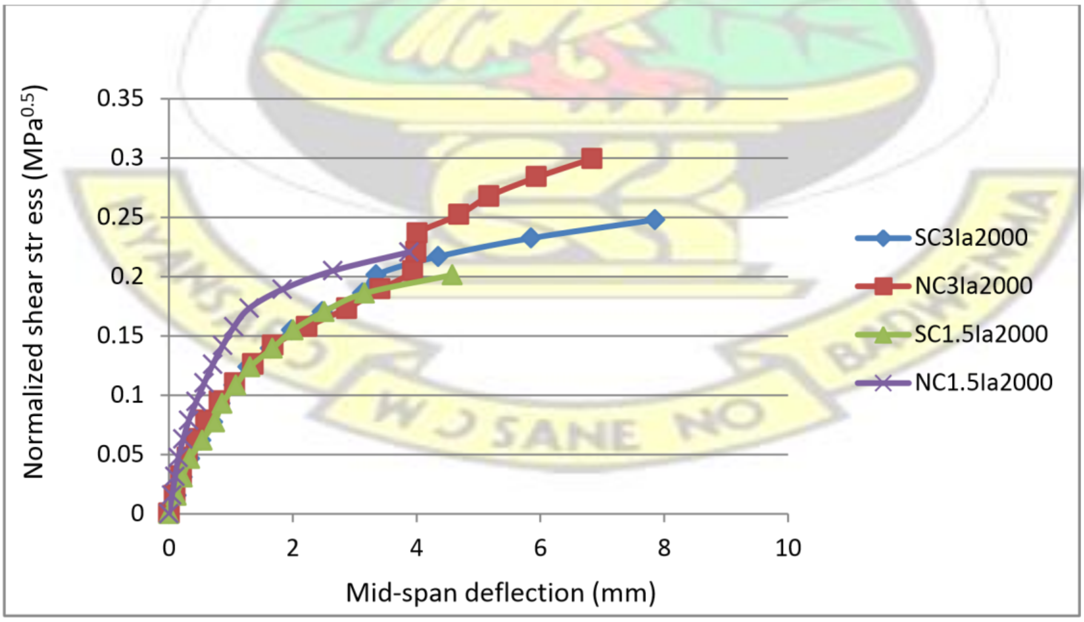


Fig. 4.5. Normalized shear stress deflection curve for beams with depth 275mm

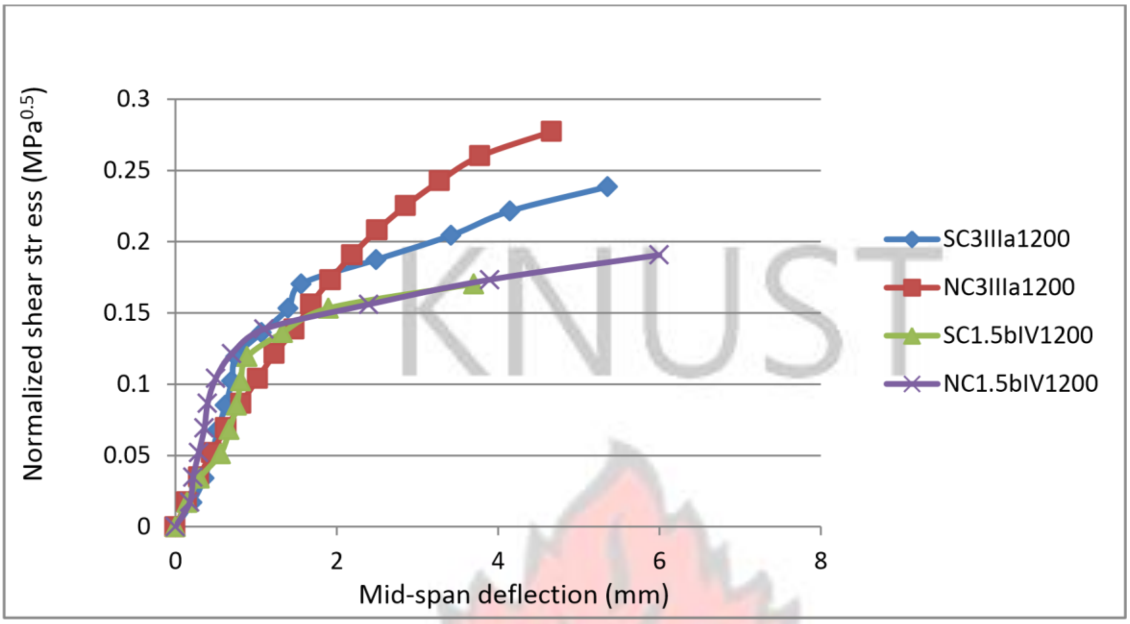


Fig. 4.6. Normalized shear stress deflection curve for beams with depth 250mm

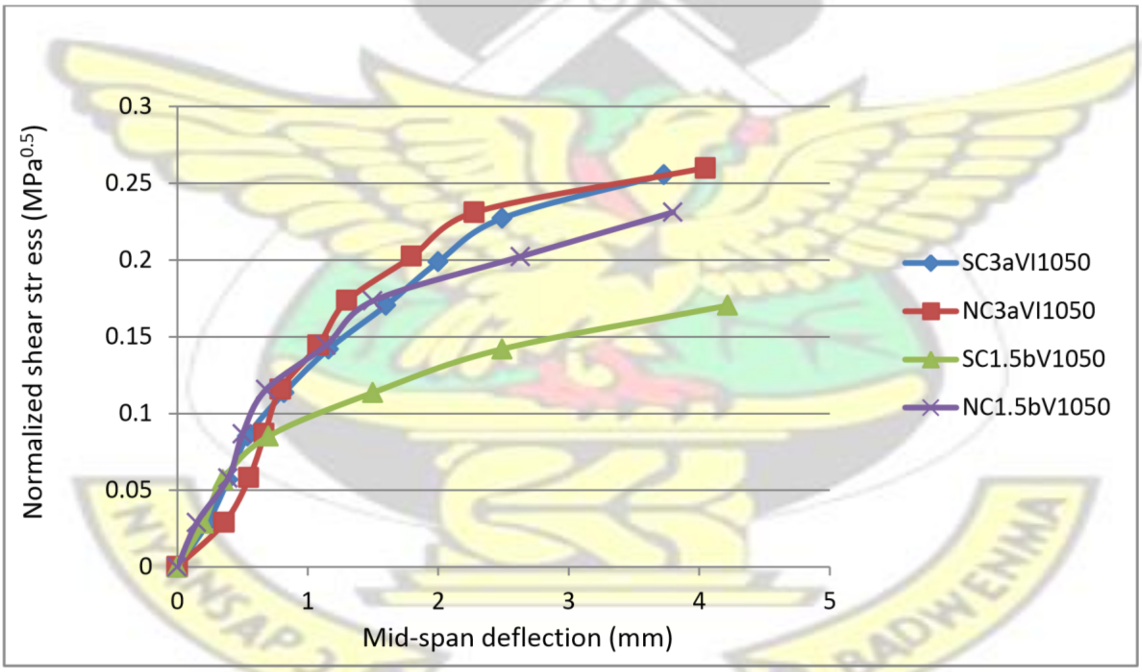
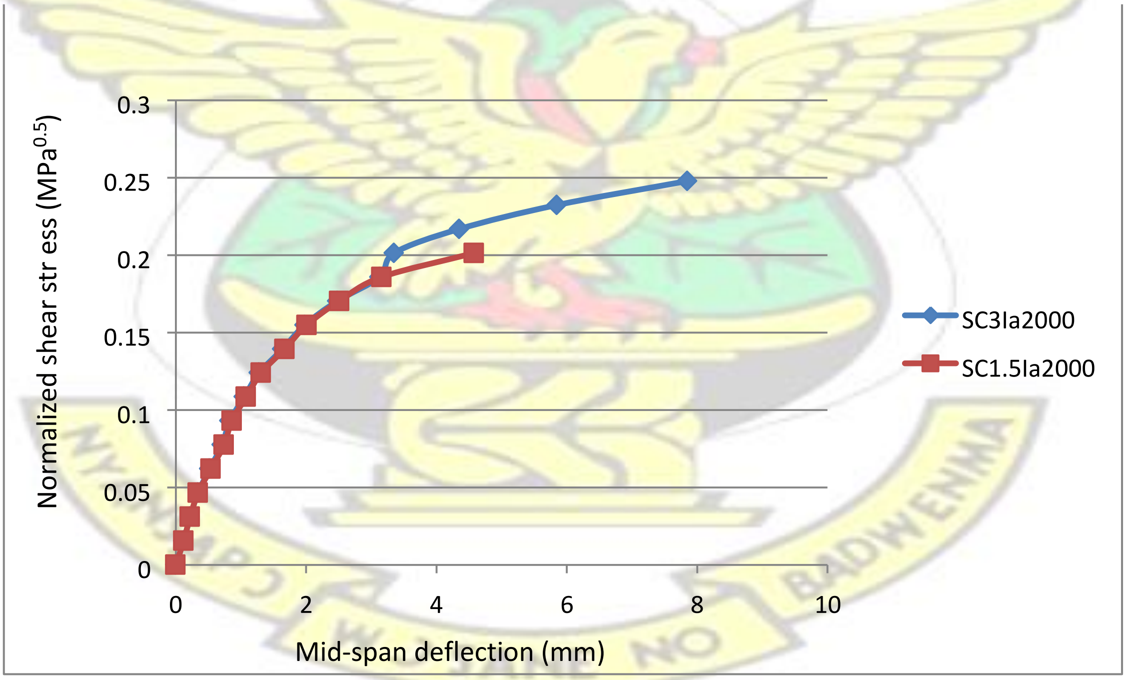


Fig. 4.7. Normalized shear stress deflection curve for beams with depth 150mm

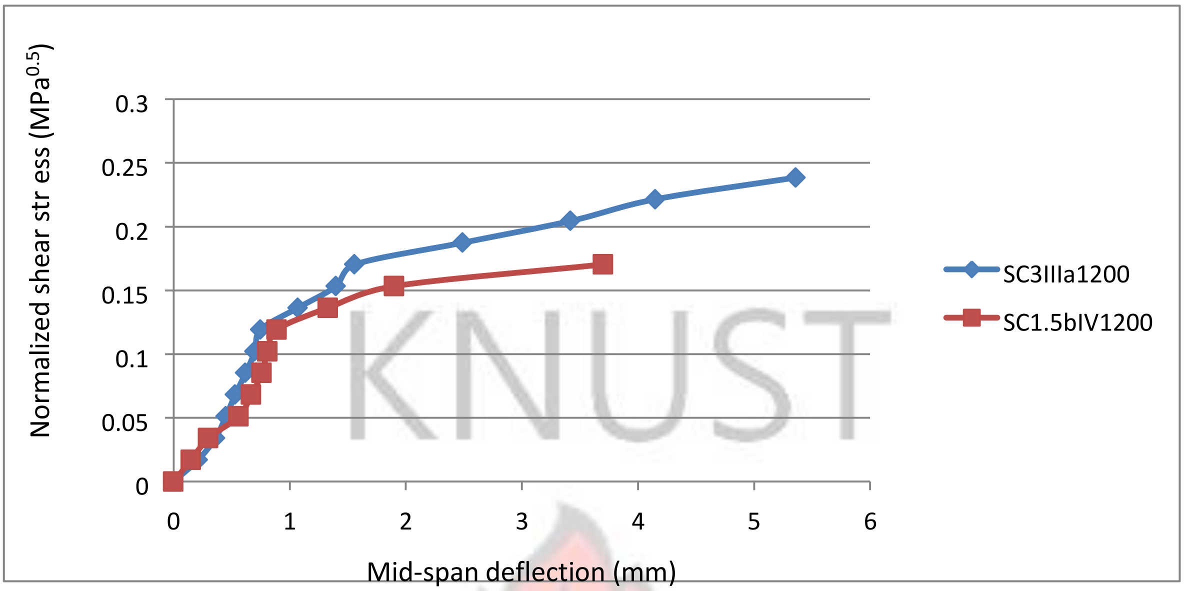
At all the beam depths, the BRNC beams experienced higher slopes for the load-deflection curves compared to the BRSCC concrete, indicating that BRNC beams have relatively higher stiffness compared to their corresponding BRSCC. However, the BRSCC beams were more ductile and experienced larger deflections as compared to the BRNC beams before failure occurred. This confirms the findings of Lachemi et al., (2008) on the load-response behaviour of SCC and NC beams.

4.5.2 Beams with varying percentages of longitudinal reinforcement ratios

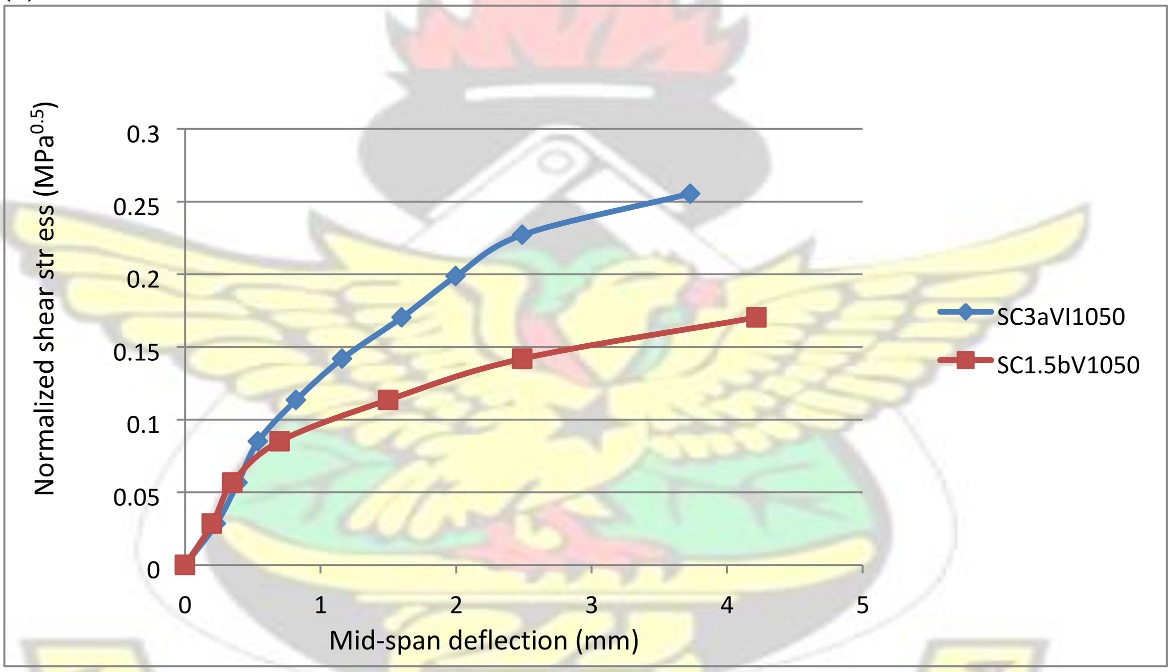
Comparing the load-deflection response of the BRSCC and BRNC beams with different longitudinal tension reinforcement ratios, it was observed that as the longitudinal tension reinforcement ratio increased, the ability of the beams to resist stresses increased but the extent of deflection with respect to the ultimate stress (stress/deflection ratio) reduced. For instance, the beams with 3% longitudinal reinforcement ratio in Figs 4.8a and 4.9a (SC3Ia2000 and NC3Ia2000) had percentage stress/deflection ratio of 3.2 and 4.4% respectively whereas their counterparts with 1.5% longitudinal reinforcement ratios had stress/deflection ratio of 4.4 and 5.7%, respectively. This indicates that as the longitudinal tension reinforcement in the beams increased, the stiffness of the beams experienced a corresponding increase as well but reduced in their ductility. This is as shown in Figs. 4.8 and 4.9



(a)

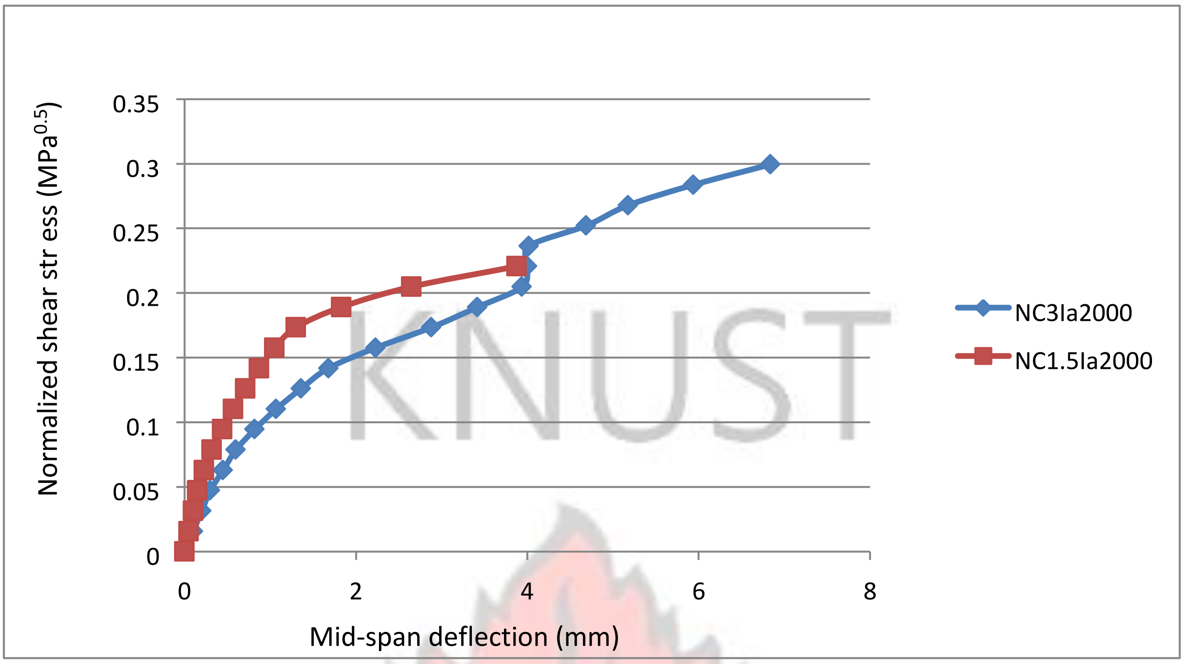


(b)

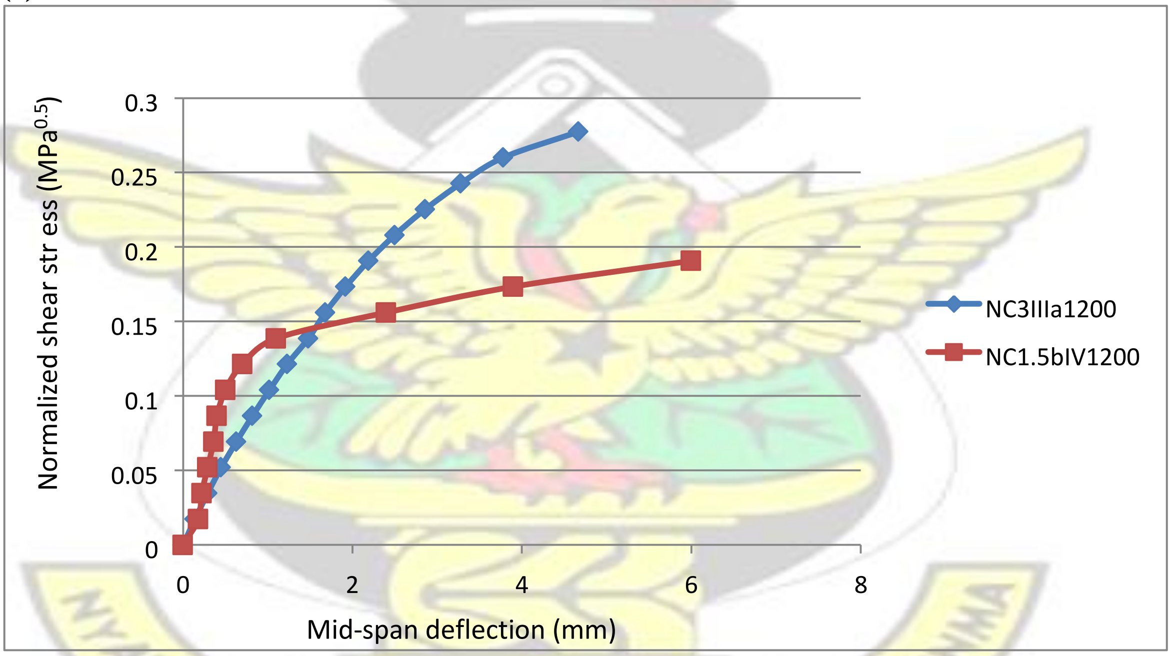


(c)

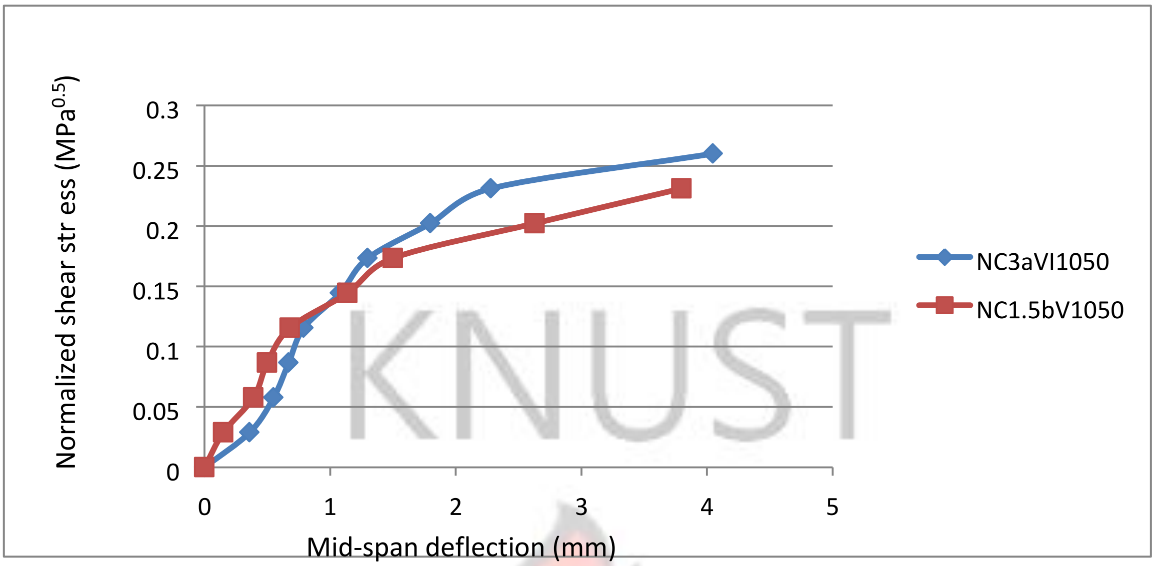
Fig. 4.8. Normalized shear stress deflection curve for BRSCC beams with 1.5% and 3% longitudinal reinforcement (a) 2000mm length (b) 1200mm length (c) 1050mm length


Normalized shear str ess ($MPa^{0.5}$)
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
2
4
6
8
Mid-span deflection (mm)
NC3Ia2000
NC1.5Ia2000

(a)


Normalized shear str ess ($MPa^{0.5}$)
0.3
0.25
0.2
0.15
0.1
0.05
0
0
2
4
6
8
Mid-span deflection (mm)
NC3IIIa1200
NC1.5bIV1200

(b)

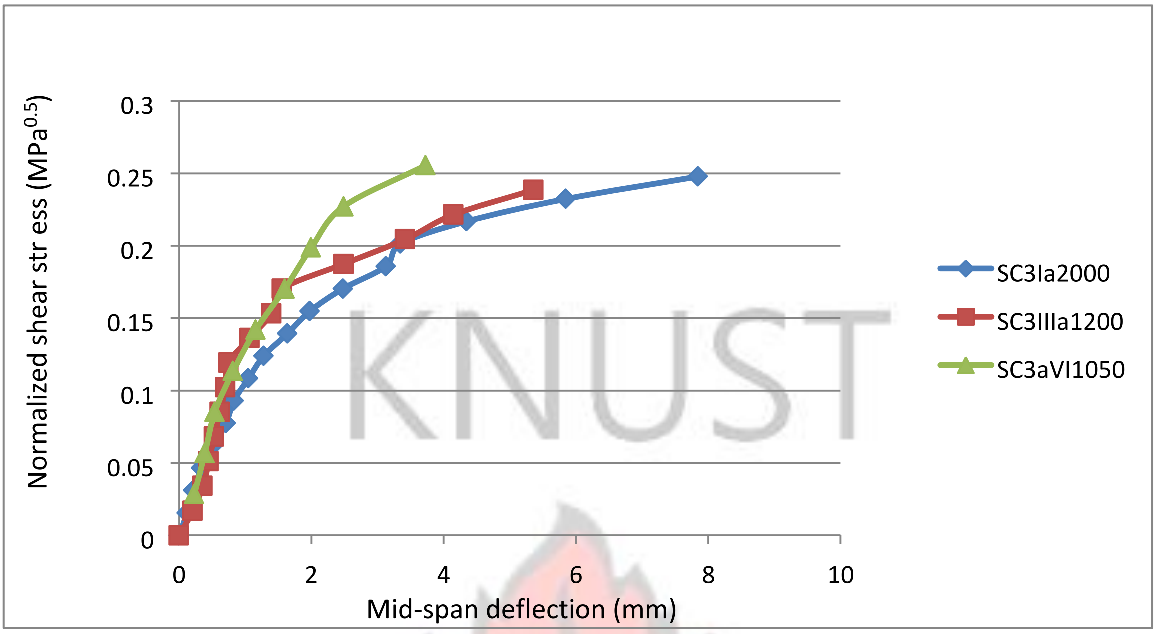


(c)

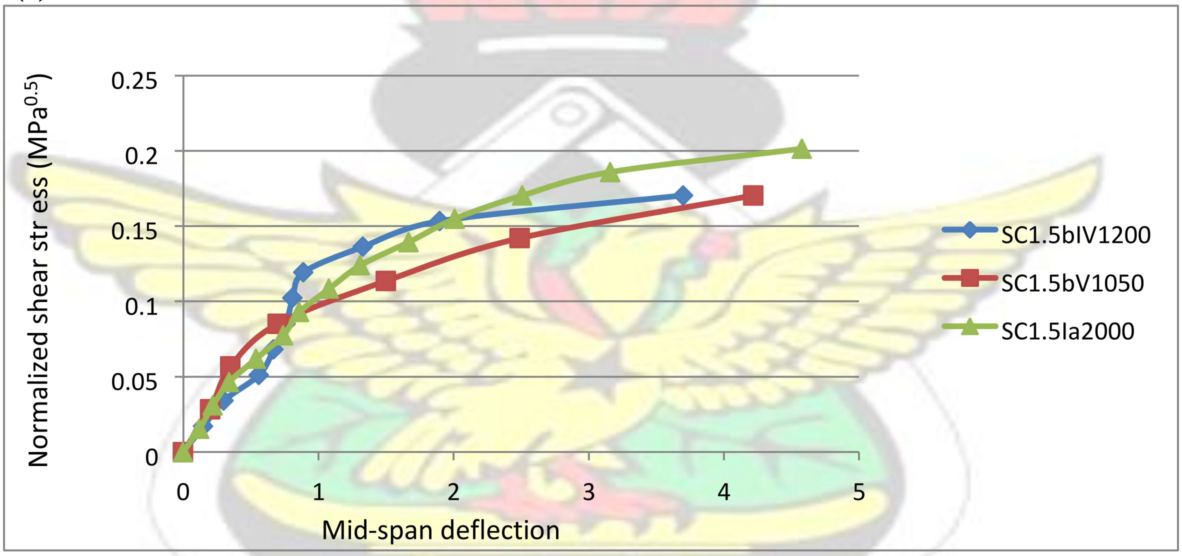
Fig. 4.9. Normalized shear stress deflection curve for BRNC beams with 1.5% and 3% longitudinal reinforcement (a) 2000mm length (b) 1200mm length (c) 1050mm length

4.5.3 Beams with varying lengths

Figs. 4.10 and 4.11 (a and b) compare the load-deflection response of the BRSCC and BRNC beams with varying lengths but similar longitudinal tension reinforcement ratios. It can be seen that the effect of varying length on the stress resistance did not follow a particular pattern when BRSCC beams are compared with their BRNC counterparts. Fig. 4.10a shows that with the same percentage longitudinal reinforcement, shorter BRSCC beams resisted stress better than their longer counterparts but conversely, Fig. 4.11a showed otherwise for BRNC beams. On the other hand, Fig. 4.11b showed that short BRNC resisted stresses better than their longer counterparts at the same percentage longitudinal reinforcement ratio, but Fig 4.10b showed otherwise for BRSCC beams.

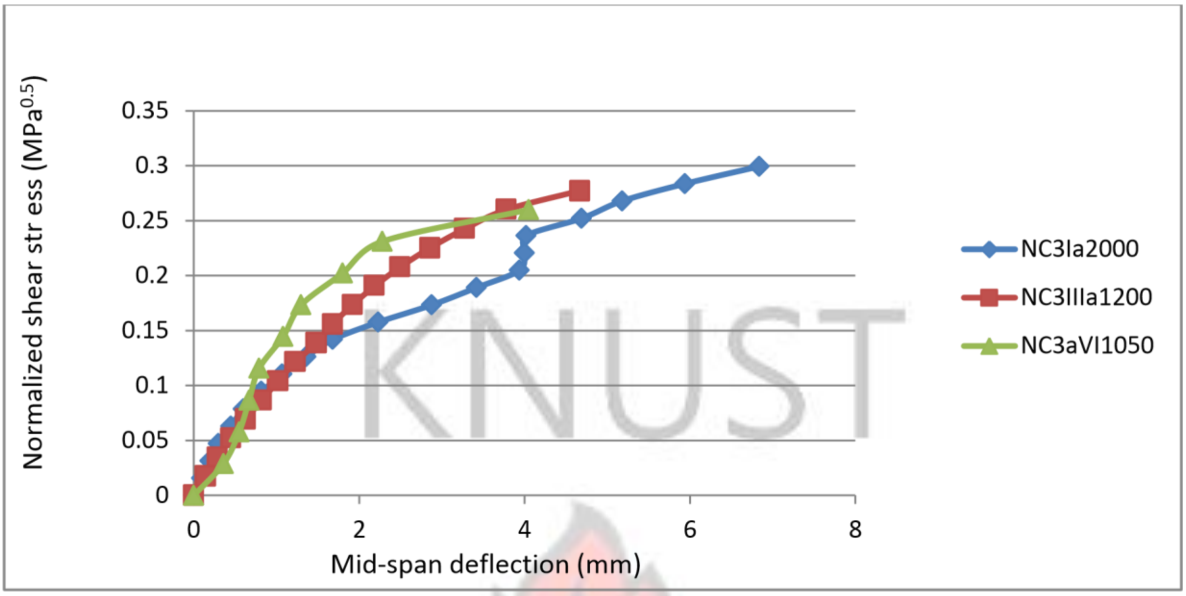


(a)

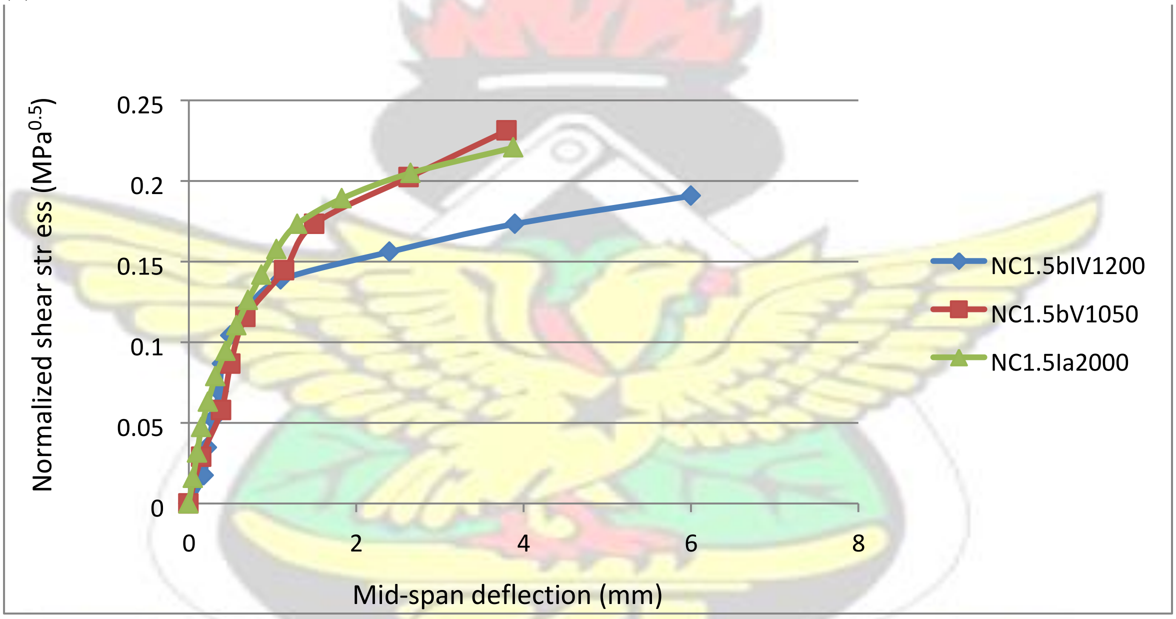


(b)

Fig. 4.10. Normalized shear stress deflection curve for BRSCC beams with varying length but same longitudinal reinforcement ratio (a) 3% longitudinal reinforcement ratio (b) 1.5% longitudinal reinforcement ratio



(a)



(b)

Fig. 4.11. Normalized shear stress deflection curve for BRNC beams with varying length but same longitudinal reinforcement ratio (a) 3% longitudinal reinforcement ratio (b) 1.5% longitudinal reinforcement ratio.

4.5.4 Elastic behavior of beams

Tab 4.6 shows the distribution of cracking load (P_{cr}) and failure loads (P_{ult}) for both

BRSCC and BRNC beams. To evaluate the elastic behavior of the specimen during the loading history, an elastic measure (EM) defined as the ratio of P_{cr}/P_{ult} as shown in column 13 was used to indirectly assess the extent of damage. The closer this ratio is to 1 (100%) the more elastic the beam was throughout the loading history, under the assumption that inelasticity initiates upon cracking. The averages of these quantities are 0.67 and 0.72 for BRSCC and BRNC, respectively. Hence, BRNC beams under service load are expected to remain more elastic than BRSCC. Lachemi et al. (2005) and Hassan et al. (2008) also reported a similar trend. The higher shear strength of BRNC beams as compared to BRSCC can be attributed to the higher aggregate interlock in the former than the latter. The aggregate interlock, which is influenced by the quantity of coarse aggregate present, plays a significant role in the shear resistance of beams, especially those without shear reinforcement (Regan et al., 2005; Taylor, 1974 and Sagaseta et al., 2011).

Among the 6 BRSCC beams, SC3Va1050 recorded the lowest ratio of 0.56 but its BRNC counterpart (NC3Va1050) was 0.22 more elastic. This can be attributed to the fact that the higher aggregate interlock in addition to the flexural strength of concrete (modulus of rupture) which has a direct relationship to the square root of the compressive strength was higher for BRNC than BRSCC. This indicates that as the compressive strength increases with its consequent increase in the flexural strength, the shear strength of the beams also increases. However, the increase in shear strength is usually not proportional since the aggregates may fracture first in high strength concretes and which will then lead to the generation of less friction upon which the shear strength depends (El-Sayed et al. 2006).

After the initiation of cracking, the concrete beams under study are expected to undergo strength and stiffness degradation upon further loading. A measure for evaluating the accumulation of damage after cracking; damage index (DI), as used by Banon et al. (1981) related directly to the initial stiffness before cracking and the final stiffness from cracking to failure. By employing Equations (4.1) – (4.3), the amount of damage sustained during loading was computed as shown in Table 4.6 column 16

$$DI = 1 - \frac{K_{final}}{K_{initial}} \qquad (4.1)$$

$$K_{final} = \frac{P_{ult} - P_{cr}}{U_{ult} - U_{cr}} \qquad (4.2)$$

$$K_{initial} = \frac{P_{cr}}{U_{cr}} \qquad (4.3)$$

An extreme case where DI=1 (100%) signifies that the first cracking load coincided with the ultimate load. Intuitively, this means that the specimen under study behaves in a brittle nature exhibiting an elastic-perfectly plastic load deformation relationship (backbone curve). In such cases, the state of stress across the section of the specimen is above the ultimate stress for the longitudinal reinforcement and the cracking stress for the concrete and the specimen will undergo relatively minimal deflection. Conversely, DI=0 implies the specimen possesses linearly elastic backbone curve with no apparent deformation; hence the specimen is more ductile and will consequently deflect more. The DI for the beams ranged from 50.22% – 92.5% and 61.08% - 94% for BRSCC and BRNC, respectively. On average, the BRSCC beams experienced about 5.5% less damage than the BRNC; hence the BRSCC beams were seen to be more ductile and possessed more residual strength. The DI measure actually is an extension of the EM which does not explicitly account for deformation. In this research, using these measures as a proxy for damage showed a fairly equal prediction. For instance SC3Va1050 recorded 56% and 67.9% for EM and DI values respectively whereas NC3Va1050 recorded 78% and 73% values for the respective parameters. Moreover, in terms of energy dissipation, BRSCC beams on average dissipated 15% more energy than their BRNC counterparts as indicated in Table 4.6 Column 17. In a similar research by Lachemi et al. (2005), SCC beams with larger coarse aggregate content (19mm) proved to be more ductile and exhibited greater deflections than NC and SCC beams with 12mm coarse aggregate content. This can be attributed to the greater aggregate interlock as a result of the larger coarse aggregate content.

Table 4.6. Theoretical and experimental loads for beams

Beam Designation	First crack load Pcr (kN)	Exp. failure load Pult (kN)	Defl. at first crack	Defl. at failure	Theoretical failure load P'ult (kN)							Pcr/Pult	Pult/P'ult	Pcr/P'cr		
					Concrete cracking P'cr	Bamboo yielding	Total flexural in tension alone	Concrete crushing	Concrete alone in shear	Concrete and long. bars in Shear failure	% Cont. of long. Bars to shear				Damage Index (%)	Energy Dissipated (kNmm)
SC3Ia2000	24	35	3.13	6.84	13.03	58.61	71.64	126.07	43.59	62.86*	30.66	0.69	0.56	1.84	61.33	189.25
SC1.5IIb2000	20	28	2.01	4.58	13.03	48.12	61.14	126.07	43.59	52.33*	16.71	0.71	0.54	1.54	68.72	86.88
SC3IIIa1200	18	26	`1.4	5.36	18.23	52.90	71.13	113.47	40.27	58.09*	30.66	0.69	0.45	0.99	50.22	111
SC1.5IVb1200	14	20	0.89	6	18.23	37.31	55.54	113.47	40.27	48.38*	16.75	0.70	0.41	0.77	92.54	102
SC3Va1050	10	18	1.16	4.05	6.51	30.32	37.83	63.04	25.92	37.38*	30.66	0.56	0.48	1.54	67.89	48.95
SC1.5VIb1050	8	12	1.5	4.22	6.51	24.26	32.78	63.04	25.92	31.84*	18.61	0.67	0.38	1.23	72.43	38.9
										Average	24.01	0.67	0.47	1.32	68.85	
NC3Ia2000	28	38	4	7.85	14.47	57.65	72.13	155.65	42.19	60.85*	30.66	0.74	0.62	1.93	62.89	168.75
NC1.5IIb2000	24	32	2.09	3.88	14.47	45.74	60.22	155.65	42.19	50.23*	16.00	0.75	0.64	1.66	61.08	70.5
NC3IIIa1200	20	28	1.54	4.67	20.25	49.67	69.92	140.08	38.99	56.23*	30.66	0.71	0.50	0.99	80.32	89.36
NC1.5IVb1200	16	22	0.8	5.8	20.25	33.33	53.58	140.08	38.99	45.75*	14.79	0.73	0.48	0.79	94.00	88
NC3Va1050	14	18	1.8	3.73	7.24	29.48	36.72	77.82	25.09	36.18*	30.66	0.78	0.50	1.93	73.35	43.34
NC1.5VIb1050	10	16	1.14	3.8	7.24	23.30	30.53	77.82	25.09	30.86*	18.71	0.63	0.52	1.38	74.29	38.8
										Average	23.58	0.72	0.54	1.45	74.32	

* Governing theoretical failure load

4.6 Flexural strength characteristics of slabs

4.6.1 Elastic behavior of slabs

Similar to the beams, the slabs whose mode of failure was flexure dominated, the BRSCC specimen obtained an average of 0.56 for the P_{cr}/P_{ult} (elastic measure) whereas the BRNC slabs was 0.69. This confirmed the earlier assumption (under beams) that the BRNC specimen are expected to be more elastic under service loads than the BRSCC. The BRSCC slabs SC3I (with 3% longitudinal reinforcement ratio) and SC1V (with 1% longitudinal reinforcement ratio) recorded the minimum and the maximum P_{cr}/P_{ult} values of 0.40 and 0.80, respectively. This shows that the longitudinal reinforcement greatly affects the degree of ductility. Similarly, the BRNC slabs NC3I with 3% longitudinal reinforcement ratio and NC1V with 1% longitudinal reinforcement ratio recorded values of 0.64 and 0.80, respectively. Taylor (1974) concluded that the dowel action of the longitudinal reinforcement contributes 15% - 25% to the shear strength. Mau and Hsu (1989) and Londhe (2011) also reported that the average shear strength of beams increases linearly as the longitudinal reinforcement ratio increases up to 1.5% and beyond that it reaches a plateau. The one way slabs in this case behaved in a similar manner as beams. In the case of the damage index (DI), SC1V and SC3I recorded values of 0.76 and 0.52, respectively. This also indicates that increasing the reinforcement from 1% to 3%, leads to a residual strength after failure that is about 24% higher. Moreover, in terms of energy dissipation, BRSCC slabs on average dissipated 17.5% more energy than their BRNC counterparts as shown in Table 4.7 Column 20. As the percentage of reinforcement increased, the beams assumed more brittle characteristics.

4.6.2 Failure modes of slabs

Both the BRSCC and BRNC slab samples exhibited similar failure modes which imply that there is not much difference between them in terms of failure mode. All the slabs failed by flexure during loading as flexural cracks developed in-between the loading point and the support points. During loading, the slabs showed approximately linear elastic characteristics until the cracking load P_{cr} was exceeded and the first crack developed at the bottom of the slab within the middle third where maximum bending occurred. After cracking, the gradient of the load deflection curve reduced continually until the bamboo yielded as the initial cracks widened. The cracking loads were observed to be 36% up to 87% of the ultimate load (Table 4.7). After yielding of the bamboo, the bamboo reinforced slab entered another phase where a slight increase in load resulted in large deflections until failure. Typical examples of the failure mode and crack patterns of the beams are shown in Fig. 4.12

Fig. 4.12. Slab SC2III after failure

Table 4.7. Experimental and theoretical loads for slabs

Slab Desig.	First crack load (kN)	Exp. Failure load (kN)	Defl. At first (kN)	Defl. At failur (kN)	Theoretical failure load P'ult (kN): Concrete P'cr	Theoretical failure load P'ult (kN): Bamboo	Theoretical failure load P'ult (kN): Total in tension alone	Theoretical failure load P'ult (kN): Concrete	Theoretical failure load P'ult (kN): Concrete shear Ps1	Concrete and long. Shear failure Ps2	% Cont. of Bars to shear	Pcr/ Pult	Pcr flexural Pult/ P'ult	Pult crushing Pcr/ P'cr	crack alone Pcr/ Ps1	e in bars Pult/Ps1	Pult/ Ps2	cracking in Damage Index (%)	yielding long. Energy Dissipated(kN mm)
SC3I	8	20	1.2	4.98	3.63	24.91	28.54*	54.46	34.46	49.69	30.66	0.40	0.70	2.20	0.23	0.83	0.57	52.38	62.5
SC3II	10	20	1.34	4.85	3.63	24.55	28.18*	54.46	34.46	49.69	30.66	0.50	0.71	2.75	0.29	0.82	0.57	61.82	61.38
SC2III	8	18	0.71	4.12	3.63	22.74	26.37*	54.46	34.46	49.69	30.66	0.44	0.68	2.20	0.23	0.77	0.53	73.97	46.36
SC1IV	8	12	1.79	4.96	3.63	20.08	23.71*	54.46	34.46	43.19	20.22	0.67	0.51	2.20	0.23	0.69	0.55	71.77	37.64
SC1V	8	10	1.66	3.4	3.63	19.96	23.59*	54.46	34.46	42.70	19.30	0.80	0.42	2.20	0.23	0.68	0.55	76.15	25.18
										Average	26.30	0.56	0.60	2.31	0.24	0.76	0.55	67.22	
NC3I	14	22	1.63	4.55	3.63	24.91	28.54*	54.46	34.46	49.69	30.66	0.64	0.77	3.86	0.41	0.83	0.57	68.10	56.78
NC3II	14	22	1.7	4.75	3.63	24.55	28.18*	54.46	34.46	49.69	30.66	0.64	0.78	3.86	0.41	0.82	0.57	68.15	55.8
NC2III	14	20	0.67	3.75	3.63	22.41	26.04*	54.46	34.46	49.69	30.66	0.70	0.77	3.86	0.41	0.76	0.52	90.68	38.5
NC1IV	8	12	1.89	3.85	3.63	20.07	23.70*	54.46	34.46	43.15	20.14	0.67	0.51	2.20	0.23	0.69	0.55	51.79	27.65
NC1V	8	10	1.38	3.25	3.63	20.07	23.70*	54.46	34.46	43.15	20.14	0.80	0.42	2.20	0.23	0.69	0.55	81.55	22.75
										Average	26.46	0.69	0.65	3.20	0.34	0.76	0.55	72.05	

* Governing theoretical failure load.

4.6.3 Ultimate flexural strength of slabs

The ultimate flexural strength of the slabs is influenced by different factors as has been established in literature. The results of the flexural strength as shown in Table 4.8 are discussed in this section.

Table 4.8. Characteristic properties and ultimate strength of slabs

Slab ID	BxDxL (mm)	h (mm)	fcu(N/mm2)	ρ (%)	Load at first crack (kN) Flex-ural	Max. mid-span deflection (mm)	P_{max} (kN)
SC3I	300x55x1000	80	26.45	3	8	4.98	22
SC3II	300x55x1000	80	26.45	3	10	4.85	20
SC2III	300x55x1000	80	26.45	2	8	4.12	16
SC1IV	300x55x1000	80	26.45	1	8	4.96	12
SC1V	300x55x1000	80	26.45	1	8	3.4	10
NC3I	300x55x1000	80	26.45	3	14	4.55	24
NC3II	300x55x1000	80	26.45	3	14	4.75	20
NC2III	300x55x1000	80	26.45	2	14	3.75	16
NC1IV	300x55x1000	80	26.45	1	8	3.85	12
NC1V	300x55x1000	80	26.45	1	8	3.25	12

Comparing the flexural strength for both BRSCC and BRNC samples, it can be seen that each slab type experienced an increase in flexural strength as the percentage of reinforcement increased. It can be seen that SC1V, containing the lowest reinforcement ratio of 1% and SC3V having the highest reinforcement ratio of 3% for BRSCC slabs recorded the lowest and highest ultimate loads of 10kN and 22kN, respectively. Similarly, BRNC samples of NC1V and NC3V exhibited similar ultimate load characteristics of 12 kN and 24kN, respectively.

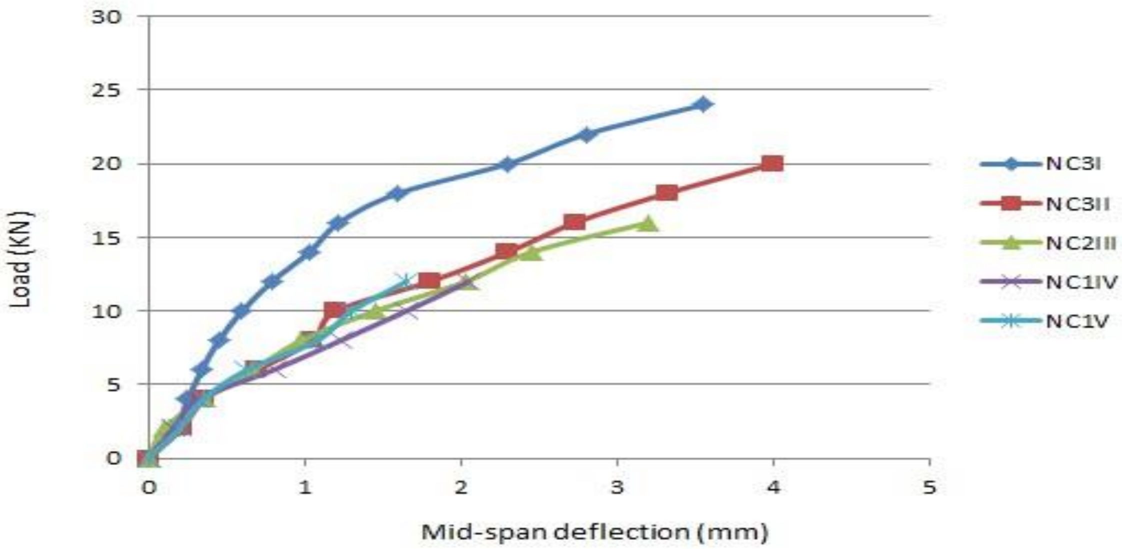
On average, BRSCC slabs with 2% reinforcement were 45.5% more resistant to flexure than those with 1%, whereas those with 3% reinforcement were 31% more resistant than those with 2%. Similarly, BRNC slabs obtained 33.3% and 37.5% for the same measure of 2% and 3% reinforcement, respectively. This confirmed the findings of Hassan (2015).

4.7 Load-deflection response of slabs

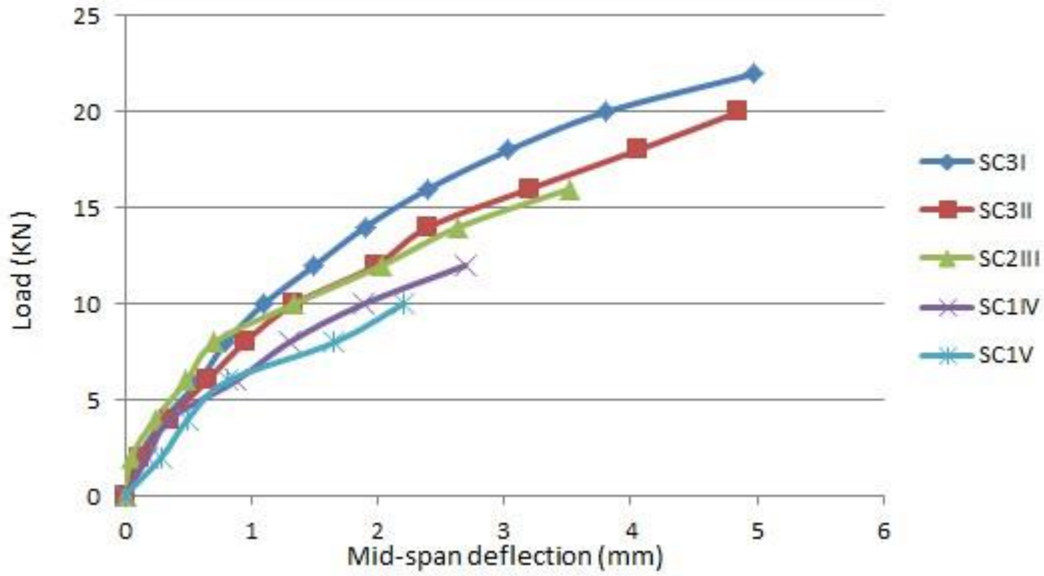
The load-deflection characteristics of BRSCC and BRNC for the slabs are given in Figs. 4.13 to 4.16. Generally, all the slabs exhibited a fairly linear behavior before the formation of the first crack, since the slope of the load-deflection curves was almost constant in this range. The first crack, which was generally flexural, was mainly influenced by the flexural strength of the concrete.

4.7.1 Slabs with varying percentages of longitudinal reinforcement ratios

The load-deflection response of the BRSCC and BRNC slabs with different longitudinal tension reinforcement ratios behaved in a similar manner hence there is not much difference in their mechanical properties. It was observed that as the longitudinal tension reinforcement ratio increased the slabs ability to resist loads increased accordingly but their extent of deflection with respect to increase in load reduces (Figs 4.13a&b). The slabs with the highest longitudinal reinforcement ratio of 3% (SC3I, SC3II, NC3I and NC3II) exhibited the highest load resistance, their failures was sudden and were more brittle in nature. This indicates that as the longitudinal tension reinforcement in the slabs increase, the stiffness of the slabs experience an increase but their ductility reduces. This confirms the findings of Adom-Asamoah and Kankam (2009) who indicated that higher reinforced (overdesigned) slabs experienced more brittle failure than their lower reinforced counterparts. In addition, Hassan (2015) also established that the ultimate flexural capacity of slabs increase with the addition of steel reinforcement.



(a)

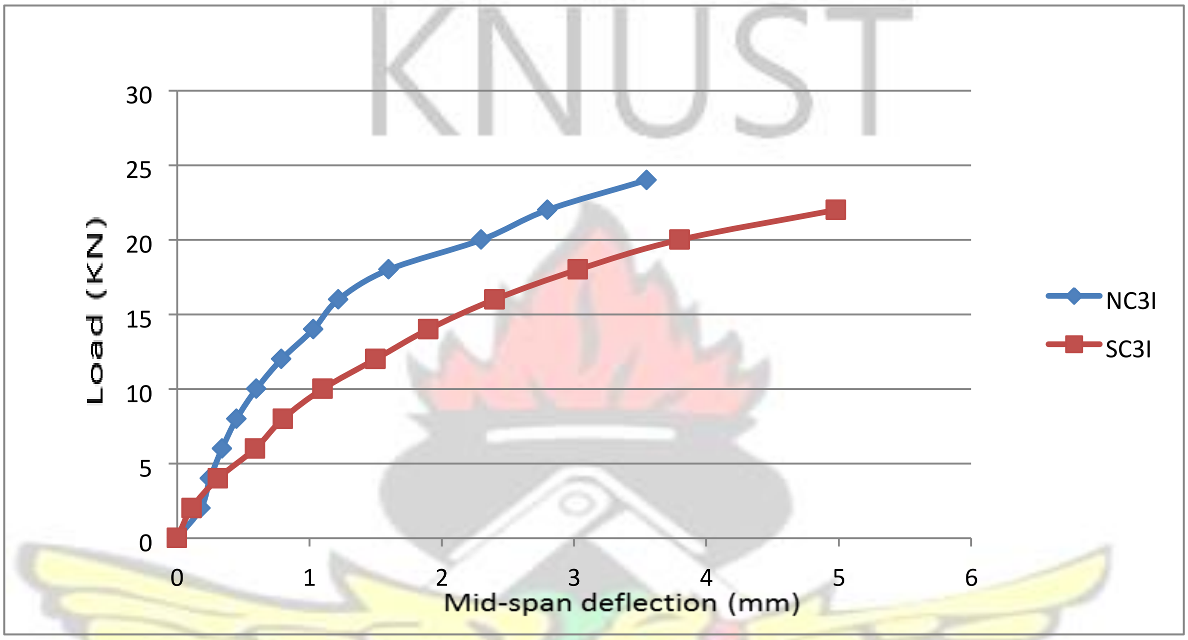


(b)

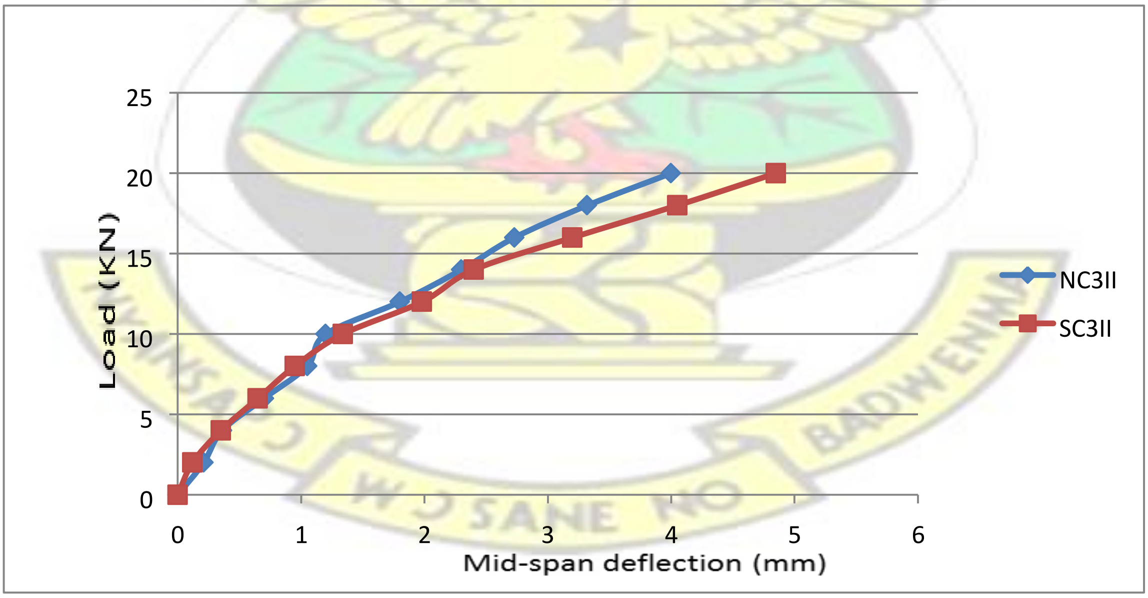
Fig. 4.13. Load-deflection curves (a) BRNC (b) BRSCC

4.7.2 BRSCC and BRNC slabs

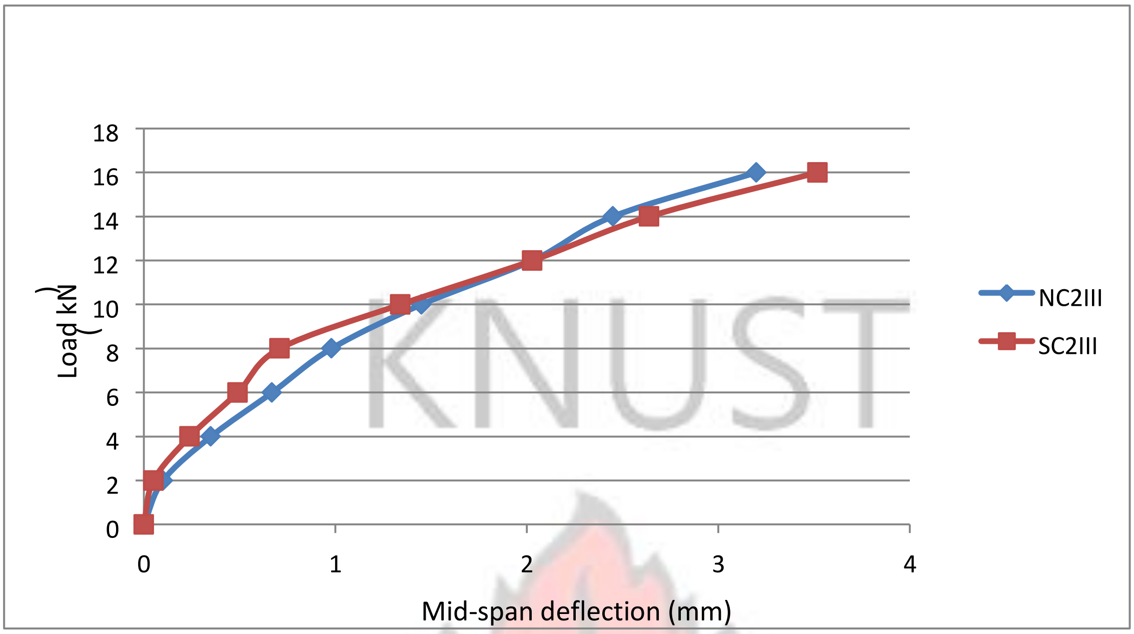
In comparing BRNC and BRSCC slabs, it is observed that the BRNC samples exhibited higher strength properties but lower load-deflection area in each category (Figs. 4.14 a-e). This indicates that for the same slab properties, BRNC components are stiffer but less ductile than their BRSCC components.



(a)

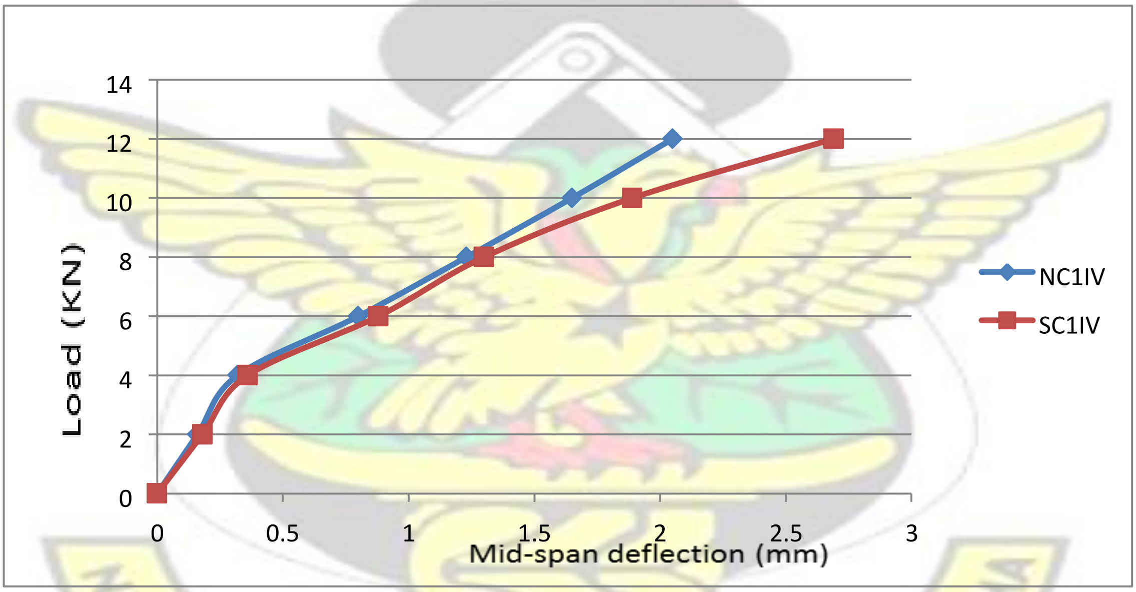


(b)



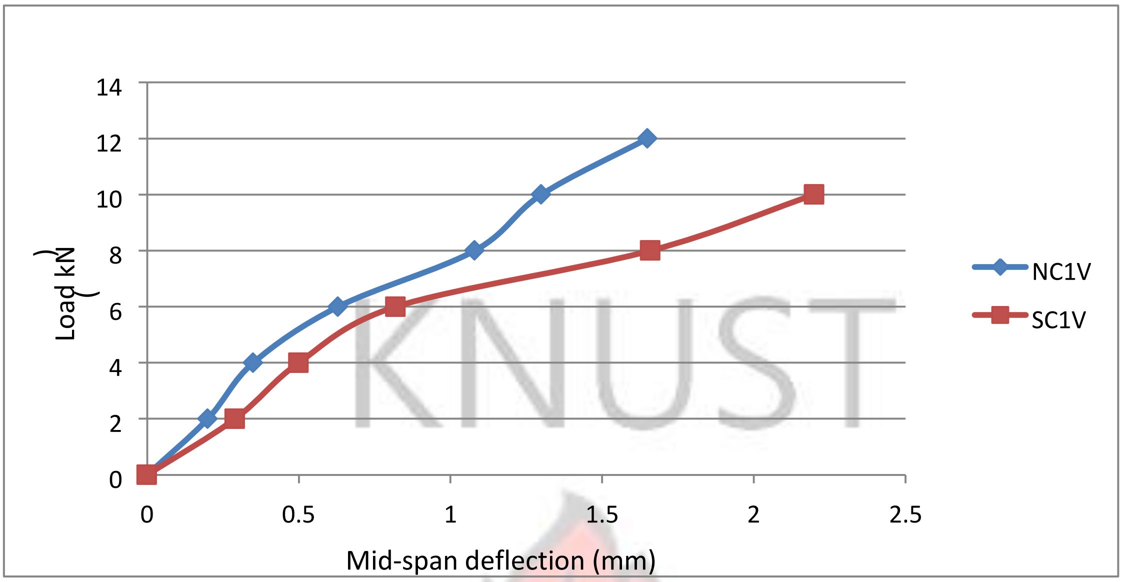
18
16
14
12
10
8
6
4
2
0
Load kN)
0
1
2
3
4
Mid-span deflection (mm)
NC2III
SC2III

(c)



14
12
10
8
6
4
2
0
Load (KN)
0
0.5
1
1.5
2
2.5
3
Mid-span deflection (mm)
NC1IV
SC1IV

(d)



(e)

Fig. 4.14. Load-deflection response of BRSCC and BRNC slabs (a) 3% I (b) 3% II (c) 2% III (d) 1%IV (e) 1% V

4.8 Comparison of experimental and code-predicted failure loads of beams

The experimental failure loads were compared with the shear resistance of the beams predicted using ACI 318-11, BS 8110 – 1997, EC 2 -2004 and CSA – A23.3 -14. Table 4.9 shows the experimental shear force in kN compared with those based on the recommendations of the ACI, BS, CSA and EC codes. The average shear strength ratios (Vexp/Vcode) for the SCC beams vary from 0.46 to 0.97. Except for CSA which estimated average Vexp/Vcode ratio of 0.97, all the other codes overestimated the actual ultimate shear strength of the BRSCC beams without shear reinforcement and are therefore not recommended for BRSCC design. Lachemi et al (2005) also recorded a ratio of 0.9 when he compared CSA to the experimental values.

Table 4.9. Experimental and predicted shear capacities for beams

Beam Designation	Vexp (KN)	Predicted Shear Force, (KN)				Shear Strength Ratios, Vexp/Vcode			
		BS8110	EC2	ACI	CSA	BS8110	EC2	ACI	CSA
SC3Ia2000	32.00	62.86	47.67	62.80	28.45	0.51	0.67	0.51	1.12
SC1.5IIb2000	26.00	52.33	45.43	51.00	28.45	0.50	0.57	0.51	0.91
SC3IIIa1200	28.00	58.09	44.00	55.16	26.08	0.48	0.64	0.51	1.07
SC1.5IVb1200	20.00	48.38	41.95	44.64	26.08	0.41	0.48	0.45	0.77
SC3Va1050	18.00	37.38	25.17	33.29	15.66	0.48	0.72	0.54	1.15
SC1.5VIb1050	12.00	31.84	24.54	26.07	15.66	0.38	0.49	0.46	0.77
					Average	0.46	0.59	0.50	0.97
NC3Ia2000	38.00	60.85	46.15	62.27	31.39	0.62	0.82	0.61	1.21
NC1.5IIb2000	32.00	50.23	43.61	48.89	31.39	0.64	0.73	0.65	1.02
NC3IIIa1200	28.00	56.23	42.59	53.31	29.84	0.50	0.66	0.53	0.94
NC1.5IVb1200	22.00	45.75	39.67	42.29	29.84	0.48	0.55	0.52	0.74
NC3Va1050	18.00	36.18	24.36	32.59	18.52	0.50	0.74	0.55	0.97
NC1.5VIb1050	16.00	30.86	23.79	25.22	18.52	0.52	0.67	0.63	0.86
					Average	0.54	0.70	0.58	0.96

The closeness of CSA values to the actual shear strength obtained from the experiment is probably due to the fact that CSA does not consider the contribution of longitudinal reinforcement in its shear computation as do the other codes under consideration. However, tested beams with 1.5% longitudinal reinforcement ratio recorded lower shear strengths than those of corresponding beams with 3% longitudinal reinforcement ratio. This implies that even though bamboo as a longitudinal reinforcement may contribute to shear resistance, it is recommended that a code predictive equation that does not explicitly account for longitudinal shear resistance be utilized. Lima de Resende et al. (2016) reached similar conclusions and stated that not all shear code provisions can safely predict the shear capacity of beams with a low transverse reinforcement index. This is especially true for beams with a lower longitudinal reinforcement ratio, since the longitudinal reinforcement stress at the time of shear failure has a significant effect on the shear capacity of the beam.

4.9 Comparison of experimental and code-predicted failure loads of slabs

Unlike shear which gives varying capacities with respect to the code used, flexural capacities are virtually the same for all the code predictions as shown in Table 4.10. The computed theoretical values indicate relied on a material strength reduction factor of 0.33 for bamboo as suggested by Adom-Asamoah and Afrifa (2011). As it can be seen from Table 4.10, the BRSCC slabs were only able to utilize averagely 50% to 61% of the flexural capacities ascribed by the various codes whereas the BRNC slabs utilized 54% to 66%. It is recommended that a reduction factor of 0.5 must be applied to code prediction of flexural capacity when designing BRSCC slabs. This will ensure the specification of a high enough safety factor on ultimate strength.

Table 4.10. Experimental and predicted flexural capacities for slabs

Slab Designation	Vexp (KN)	Predicted Flexural Force, (KN)				Flexural Strength Ratios,			
		BS8110	EC2	ACI	CSA	BS8110	EC2	ACI	CSA
SC3I	20.00	28.54	34.40	29.00	28.15	0.70	0.58	0.69	0.71
SC3II	20.00	28.18	33.94	28.62	27.82	0.71	0.59	0.70	0.72
SC2III	18.00	26.37	31.58	26.68	26.12	0.68	0.57	0.67	0.69
SC1IV	12.00	23.71	28.35	23.95	23.66	0.51	0.42	0.50	0.51
SC1V	10.00	23.59	28.20	23.82	23.54	0.42	0.35	0.42	0.42
					Average	0.60	0.50	0.60	0.61
NC3I	22.00	28.54	34.40	29.00	28.15	0.77	0.64	0.76	0.78
NC3II	22.00	28.18	33.94	28.62	27.82	0.78	0.65	0.77	0.79
NC2III	20.00	26.04	31.16	26.34	25.81	0.77	0.64	0.76	0.77
NC1IV	12.00	23.70	28.34	23.94	23.65	0.51	0.42	0.50	0.51
NC1V	10.00	23.70	28.34	23.94	23.65	0.42	0.35	0.42	0.42
					Average	0.65	0.54	0.64	0.66

CHAPTER 5: CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

This research was aimed at determining the performance of bamboo in selfcompacting concrete with respect to shear and flexure. Based on the results obtained from the study, the following conclusions were made:

- Very little differences existed between BRSCC and BRNC specimens with respect to their normalized shear stress-deflection curves, shear and flexural characteristics.
- BRNC beams under service load are on average, more elastic than BRSCC.
- The damage index (DI) revealed that BRSCC beams are more ductile and possesses higher residual strength than their BRNC counterparts.
- The P_{cr}/P_{ult} values indicated that the longitudinal reinforcement ratio greatly impact the degree of ductility of the slabs.
- The energy dissipation capacity of the structural components (beams and slabs) of BRSCC is higher than their BRNC counterparts.
- Though bamboo as a longitudinal reinforcement contributes to shear resistance, it is recommended that a code predictive equation that does not explicitly account for longitudinal shear resistance e.g. CSA be utilized when designing BRSCC structural beams.
- BS, ACI, EC 2 and CSA overestimated the prediction of the flexural capacities of the slabs when a material factor of safety of 3 was used for the bamboo. Hence a reduction factor of 0.5 must be applied to code prediction when designing BRSCC slabs to ensure a high enough safety factor on ultimate strength.

5.2 Recommendations

- Further work should be done to predict the experimental shear strength models for BRSCC by using an extensive database of experimental results.
- There should be further work to establish the appropriate quantification of the partial factor of safety that can be applied for the design of BRSCC.
- Further work on the behavior of BRSCC with transverse reinforcement should be carried out to determine the effect of transverse reinforcement on BRSCC.
- Bamboo should be treated to enhance its durability and bond characteristics before using for construction.

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