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Shear Behaviour of Self Compacting Concrete Beams Reinforced with Palm Kernel Fibre

by

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DECLARATION

I hereby declare that this submission is my own work towards M.Sc. 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 acknowledge has been made in the text.

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DEDICATION

This is work is dedicated to my parents Dr. and Mrs. Obeng, and the entire Obeng Family.

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ABSTRACT

The shear strength of palm-kernel-fibre-reinforced self-compacting concrete (SCC) was investigated by experimental means. The SCC with palm kernel fibre were cast without web reinforcement. An experimental programme was designed for twenty fibre reinforced SCC beams incorporating bottom longitudinal steel bars. The beams were simply supported and subjected to 4 – point monotonic and cyclic loading tests. The variable parameters used were two different nominal coarse aggregate sizes 10 mm and 20 mm and different amounts of fibre reinforcements; 0, 0.25, 0.5, 0.75 and 1 percentages by weight of cement. With the beams subject to loading, crack patterns were marked and cracking and ultimate loads were recorded. The deflection at all load points were also recorded. Test results indicated that the cracking behaviour with the addition of fibre was more effective in 10 mm nominal coarse aggregate size beams than the 20 mm nominal coarse aggregate size beams. This was because all the 10 mm nominal coarse aggregate size beams had the crack widths under service loads less than 0.3 mm. On the other hand, some of the 20 mm nominal coarse aggregate size beams recorded crack widths greater than 0.3 mm under monotonic and cyclic loading. The shear strength of the 10 mm nominal coarse aggregate size beams were observed to be more than the 20 mm nominal coarse aggregate size beams. In terms of the failure modes, all the beams, with the exception of one beam which failed by crushing at the support, failed by shear failure mode.

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SYMBOLS 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

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CHAPTER 1: INTRODUCTION

1.1Background of Study

The Egyptians in 3000 BC built from mudbricks, and built pyramids from lime and gypsum mortar. Following the production of Portland cement in 1824 by Joseph Aspdin, the materials used to produce concrete have undergone various improvements till today. Different types of concrete have been studied to provide better understanding of the concrete behaviour and impact on structural performance. Following the reduction in the number of skilled workers in Japan in the 1980's, self-compacting concrete (SCC), which is a self-flowing concrete that achieves compaction under its own weight without any form of vibration was developed. Primarily, SCC is obtained by increasing the fine aggregate content in place of the coarse aggregate to allow for easy movement of aggregates which improves the flow of the concrete. Consequently, there is a reduction in the aggregate interlock of the self – compacting concrete. The main shear resistance mechanisms include the compressive strength in the uncracked concrete, the aggregate interlock and dowel action of the longitudinal reinforcement (Taylor, 1974). Since there is reduction in the aggregate interlock which contributes to the shear resistance mechanism of concrete, the study into the shear behaviour of SCC is warranted. Previous researchers, such as Hassan *et al.,* (2008) and (2010) and Lachemi *et al.,* (2005), investigated the effect of shear behaviour of SCC considering the coarse aggregate content and the type of concrete. They found that the shear strength of normal concrete beams was higher than the SCC beams. Hence, current research efforts (Hassan *et al.,* 2010; Safan, 2012; Biolzi *et al.,* 2014) on ways of improving the shear strength of SCC, investigate the impact of type of coarse aggregates used, the variations in sizes and the optimum proportions to be used for design. Others (Fritih *et al.,* 2013), have also advocated the inclusion of fibres to improve upon cracking behaviour and ductility.

Code-conforming empirical models for estimating nominal shear capacities of reinforced structural members (CSA, 2004) emphasizes that the maximum coarse aggregate size may be a significant contributor to the shear strength. More so, research efforts (Lachemi *et al.,* 2005; Hassan *et al.,* 2013) have shown that there is a positive correlation between aggregate size and shear strength. The inclusion of fibre has been found in normal concrete to improve cracking behaviour and mechanical properties of concrete (Di Prisco *et al.,* 2000; Meda *et al.,* 2005). The fibre improves cracking behaviour by transferring tensile stresses in the concrete across cracked sections, and delays the crack propagation. The use of fibre in concrete is not a new practice as horse hair and straw were used to reinforce mortar and mudbricks in the prehistoric years. Fibre-reinforced concrete gained attention in the 20th Century as different materials were used as fibre reinforcement which included steel, glass, synthetic and natural fibre. Among these types of fibres, steel fibre is the most beneficial in terms of improving structural performance of reinforced concrete (Fritih *et al.,* 2013). A typical example is the work of Furlan *et al.,* (1997), which evaluated the cracking behaviour of steel and polypropylene fibres as used in normal concrete for improving cracking behaviour. However, a major disadvantage is the rusting of the steel fibre and as such other natural occurring fibres (oil palm, jute) may be better alternatives in terms of improving and maintaining typical mechanical properties such as compressive and flexural strength of concrete (Olaoye *et al.,* 2013). However, structural performance metrics mainly shear strength has been found to be inadequate for reinforced SCC members. (Hassan *et al.,* 2008). The current study sought to advance knowledge in the shear behaviour of reinforced SCC beams under monotonic and cyclic loading. Parameters investigated are the nominal size of coarse aggregates and fibre.

1.2 Justification for Study

The use of SCC in large scale construction projects can be deemed to be beneficial due to a significant reduction in the cost of labor. In addition, since its usage does not require any form of mechanical vibration, it becomes advantageous in situations where highly dense concrete is a crucial requirement. Natural fibre from palm kernel was used as a result of its abundance and the current situation of it being used as a waste material in Ghana.

1.3 Objectives

The objective of this study was to investigate the shear strength of palm-kernel-fibrereinforced SCC. The specific objectives were:

- To determine the impact of the different sizes of coarse aggregates on shear behaviour in SCC
- To determine the impact of palm kernel fibre as fibre reinforcement on cracking behaviour of SCC
- To determine the variation in structural performance of SCC under monotonic and cyclic loading.

1.4 Organization of Thesis

The thesis consists of five chapters. Chapter 1 gives the general introduction, the objectives of the study, and the thesis organization. Chapter 2, presents literature on the shear behaviour of SCC. Chapter 3 is the methodology. It gives the details of the tested beams, test variables, properties of the material used, concrete mix design, mixing and casting, loading arrangement, testing and measurement. Chapter 4 presents the test results and discussions. It gives description of the crack patterns and modes of failure of the tested beams, including the effect of test variables, and test results including deflections, cracking and ultimate shear strengths showing the effect of test variables on measured values. Chapter 5 presents conclusions on the current study and recommendations for future studies.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter reviews recent research on the behaviour of shear in reinforced SCC beams with and without stirrups. In this review, certain parameters were explored to investigate their effect on the shear performance of SCC beams in existing literature. These parameters include, beam shear span- effective depth ratio (a/d), beam depth, compressive strength of concrete, coarse aggregate size, coarse aggregate content/type, coarse to fine aggregate ratio, percentage of longitudinal reinforcement, shear reinforcement ratio and shear reinforcement spacing (beams with stirrups), and loading arrangement.

2.2 Development and general characteristics of SCC

Normal concrete is well-known for construction of concrete worldwide. However, in the construction of large structures where vibration of concrete to obtain a dense material with no voids is a requirement, the use of normal concrete becomes unfeasible due to the amount of vibration (labour) that would be done. In addition, in construction work where the reinforcements in the formwork are congested, it is difficult for coarse aggregates to occupy the required spaces in the formwork. SCC, which does not require any vibration, can be used in such instances to save time, cost and ease of arrangement of coarse aggregates in the formwork. In 2000, the wall of a large liquefied natural gas (LNG) tank belonging to Osaka Gas Company in Japan adopted SCC in the construction of a wall. The concrete workers and construction time were reduced by 100 workers and 4 months, respectively. SCC is prepared without the use of vibrators, thus achieves compaction under its own weight by flowing into place and also is cohesive enough to avoid segregation or bleeding. Hence, the important characteristics of SCC include its ability to fill formwork, pass through obstacles and resist segregation (Shi *et al.*, 2015). The main constituents of SCC are cementitious material, water, aggregates (coarse and fine) and super plasticizer. The super plasticizer usually high range water reducing agents is added to help obtain high flow ability. Also, mineral admixtures, in the form of supplementary cementitious materials (SCM), are added, in some cases, to improve segregation resistance which is vital to the defining property of SCC. Fly ash, metakaolin and silica fume as supplementary cementitious materials, have been adopted in SCC technology and metakaolin in particular was found to cause greater strength and durability of SCC (Hossain and Lachemi, (2009). The addition of mineral admixtures of the powder type as supplementary cementitious materials to SCC, causes an increase in slump flow of SCC and reduces the cost of concrete (Naik *et al.*, 2001). Different SCM types (silica fume, metakaolin, Class F ash, Class C fly ash, and granulated blast furnace slag) in binary, ternary and quaternary cementitious blends were investigated for their permeability and results showed SCC with SCM had lower porous voids and absorbing water than SCC without SCM (Ahari *et al.*, 2015). The usage of mineral admixtures also helps improve the compressive strength of SCC. Limestone, silica fume and blast furnace slag were used as mineral admixtures and compared with ordinary SCC. The silica fume was found to improve the compressive strength by having the highest compressive strength than all other mixes (Abderahmane *et al.*, 2013). In SCC, the proportion of coarse aggregates is decreased (minimizing coarse aggregates) and replaced with an increase in proportion of fine aggregates (maximizing fine aggregates). This is to allow adequate flow which is a basic requirement for SCC and also to avoid segregation (Lachemi *et al.*, 2005).

2.2.1 Fresh properties test of SCC

SCC in its fresh state is evaluated by various tests to determine the concrete's ability to conform to the defining characteristics that qualify it as self-compacting (Naik *et al.,* 2001). The test methods help to judge self-compacting ability of concrete and also evaluates deformability. The fresh property tests developed to test for the major characteristics of SCC include slump flow test which measures the flow ability, T500 Slump Flow test that measures viscosity and L- Box test the passing ability (Hwang and Tran, 2015). In addition to the above mentioned test, Thanh *et al.*, (2015) conducted the sieve segregation test to measure the segregation resistance. The segregation resistance has also been measured by visual inspection during the slum flow test by Jalal *et al.,* (2015).

2.2.2 Application of SCC

Self-compacting concrete has gained acceptance through its wide application in various forms that concrete is used for structural components. The forms of concrete include prestressed (Choulli *et al.,* 2008), and fibre reinforced concrete (Sable *et al.*, 2012) . Over the years, since its inception, SCC has gained the attention of researchers to regarding its general behaviour and response to general circumstances and environment as is done with normal or conventional concrete. Investigations have provided confidence in the use of SCC in place of normal concrete to reap the added benefits of SCC. Existing knowledge is key to developing better understanding of innovative developments which stems from creations. SCC is developed by modifying the already existing constituents of normal or conventional concrete to achieve its self–compacting nature with no vibration which differentiates SCC from normal concrete which is always vibrated. Therefore, first understanding the general behaviour of normal concrete becomes necessary and key to a better understanding of SCC although there is an expectance of differences due to the modification of the constituents of SCC.

Normal concrete's constituent materials are modified to obtain SCC. For concrete to be classified as SCC, the major characteristic that differentiates it from normal concrete (NC) is the absence of vibration in SCC. Since vibration in NC is done to achieve compaction of concrete to remove air voids and obtain a densely-packed material, SCC achieves compaction on its own by having materials flowing into place under its own weight. The concrete mix is made flow able usually by having high water – cement ratio which causes disadvantages in concrete strength. Therefore, super plasticizers are added with lower water–cement ratio for flowable concrete with no added disadvantage of too much water. The coarse aggregate content is reduced in place of the fine aggregates which contributes to the flow by eliminating the restrictions to flow produced by coarse aggregates. From SCC Guidelines (EFNARC, 2005), the fine aggregate content is made up of 48–55 % of the total aggregate weight. An increased content of paste and decreased coarse aggregate are therefore modifications to the NC constituents to aid in the mix design of SCC.

2.3 Shear in Self Compacting Concrete

The shear failure of beams is sudden (brittle) with no ample warning as compared to the flexural behaviour of under-reinforced beams which gives warning (ductile) before flexural failure. Therefore, shear reinforcement is provided in the part of beams where shear is expected to delay or prevent the failure in shear and also to provide a ductile behaviour of the beam. For the shear in beams without stirrups, failure in shear is accompanied by shear cracks. When the principal tensile stress in the beam reaches the tensile strength capacity of the concrete, a crack occurs. The occurring shear crack may be diagonal or inclined and occurs from shear forces when the concrete is under ultimate load. Inclined cracks that propagate from flexural cracks are called flexural shear cracks

Extensive research on the mix designs and durability of SCC has been done with experimental investigations by the various researchers. Although research on the performance of SCC with regards to shear since its development in the 1980's has been conducted, the past decade has considered the structural components (beams) under static loading and cyclic loading, but with limited research on SCC under cyclic loading. Contributions to shear in concrete elements include; the compressive load on the concrete (the shear in the un-cracked concrete compression zone), the interaction of the aggregate in concrete (aggregate interlock), the dowel action of the main longitudinal tensile reinforcement and the shear reinforcement if present. The contributions to shear strength by the compression zone is 20% - 40%, by the aggregate interlock is 35% - 50% and dowel action of the main longitudinal tensile reinforcement is 15% - 25% (Taylor, 1974). The interlocking of the aggregates creates a bond in the concrete and is reduced in SCC due to the reduced proportion of the coarse aggregates in SCC. The interlocking of aggregate, which is one of the main mechanisms that contributes to the shear in concrete elements, then becomes compromised. This, therefore, justifies the investigation of shear in SCC (Lachemi *et al.*, 2005) .

Results from the various researches were collated, analyzed and comparison were made on the normalized shear strength of the different researches. The data obtained from the various research reviewed included the following, span-depth ratio, compressive strength, percentage of longitudinal reinforcement, shear reinforcement (stirrups), ultimate shear strength, and number of cracks.

2.3.1 Related Work on Shear Behaviour of SCC

Lachemi, *et al.* (2005) conducted experimental investigations into the shear resistance of SCC beams. The effect of type of concrete, coarse aggregate size and content, and ratio of the shear span to the depth on strength in shear was investigated. From experimental results, There was lower post – cracking shear transfer resistance in SCC than NC due to the lowere coarse aggregate content in the SCC. Also, for different sizes of coarse aggregate in SCC, SCC beams with large size coarse aggregates that is 19 mm had an increase in shear resistance by post-cracking shear transfer mechanism. The ultimate shear resistance of SCC was affected by the shear span –to- depth ratio.

By experimental studies, the strength in shear and cracking characteristics of full scale concrete beams were researched by Hassan *et al.,* (2008). According to the experimental results, the shear resistance of SCC and NC was similar but lower ultimate shear load in SCC than NC due to the decrease in the amount of the coarse aggregate in SCC. An increase in longitudinal steel reinforcement ratio was accompanied by an increase in ultimate shear load of SCC and NC beams. However, an increase in the longitudinal steel reinforcement ratios did not have any effect on the ultimate shear stress with increasing the beam depth. Hence, lower ultimate shear stress values at higher beam depths were observed.

Hassan *et al.,* (2010) investigated the effect of type of concrete and the content of the coarse aggregate, variable depth of beam and amount of longitudinal reinforcement were investigated on SCC beams behaviour in shear by experimental investigations. It was concluded that with lesser coarse aggregates in the SCC compared to the NC, the ultimate shear strength of SCC was decreased. For cracking load at the first crack caused by flexure, the code equations (ACI, 2005 and CSA, 2004) predicted lower values than experimental results for deeper beams. Conversely, the AS 3600 (1988) and

EC 2 (1992) overestimated the first flexural cracking load. For shallow beams, all the codes predicted values close to the experimental results.

Xu (2011) studied the resistance of SCC I-beams to shear with the same geometry. The effect of varying the shear span, shear reinforcement ratio and the arrangement of stirrups on the shear behaviour of SCC was considered. It was concluded that the increment of the ratio of transverse reinforcement to double the ultimate resistance to shear did not double the ultimate shear resistance. Also, the shear strength gained decreased with decrease in the ratio of the shear span –to – depth.

Safan (2012) investigated the effect of concrete type based on different coarse aggregates (crushed dolomite and gravel) on shear behaviour. The experimental results were compared with the ACI – 318 (1995) and JCSE – 86 Codes (2007). Based on the results, it was concluded that the crushed dolomite based beams had higher ultimate shear loads than with gravel based beams. This was as a result of a greater compressive strength and paste-aggregate bond in dolomite as coarse aggregate. 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. From comparison of code equations to experimental results, the ACI and Japanese Code adequately predicted the experimental values.

Hassan *et al.*, (2015) assessed the response to the cracking and shear behaviour of different cementitious material, coarse aggregate type, coarse –to-fine aggregate ratio in SCC beams without stirrups. Based on the experimental results, it was concluded that increasing the amount of coarse aggregate and decreasing the fine aggregate increased the normalized strength in shear of the beams. Also, 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.

Choi *et al.*, (2012) studied the behaviour in shear of SCC deep beams reinforced with transverse reinforcement (stirrups), with the ratio of the shear span to the effective depth at 1.43 < 2 (a criterion for beams classified as deep). The shear behaviour of SCC beams was compared to NC beams by varying the spacing of the stirrups. The following conclusions were made the experiment. The mode of failure of all the deep beams was shear compression failure. Also for normal vertical web reinforcement arrangement of 100 mm stirrup spacing the load carrying capacity at occurrence of diagonal shear crack and the ultimate load was somewhat higher in SCC than in NC beams. The congested vertical web reinforcement arrangement of 50 mm stirrup spacing in SCC and NC showed similar load carrying capacity in terms of the ultimate load.

Abed (2012) investigated the shear behaviour of SCC RC I-beams with and without stirrups. The experimental shear strength was compared to the ACI 318 -08 , (1998) design equations for shear strength. The code predictions of ultimate shear strength were lesser than experimental ultimate shear strength due to the improvement in durability and the bond strength of the SCC.

Helincks *et al.*, (2013) investigated the structural behaviour of SCC by determining the strength in shear regarding SCC beams. The effect of the different percentage of longitudinal reinforcement and varying ratio of the shear span to the depth on shear beahviour was investigated. The shear strength of SCC was compared to NC. Experimental results were also compared with code provisions from EN 1992-1-1 (2004), ACI-318-11 (2011) and fib Model

Code 2010 (2012). From experimental results, the strength decrease in ultimate strength in shear regarding SCC beams was due to lower aggregate interlock from the lower volume of coarse aggregate in SCC compared to NC. The decrease in strength was rather small of about 6.9%. Also EN 1992- 1-1 was modified by taking into account the effect of the ratio of the shear span to the depth as the shear capacities were influenced by the ratio of the shear span to the depth.

Abouhussien *et al.,* (2014) investigated the effect of different manner of pouring of SCC and fresh concrete properties on the shear strength with respect to SCC beams. The pouring techniques adopted were; Technique I for NC beams; Technique II and III for SCC beams. In Technique I for NC beams a mechanical vibrator was used, and casting was done along the beam. Technique II was done by pouring from one side of the formwork and Technique III by pouring along the beam length. Strength increased in shear for all SCC beams with more longitudinal steel reinforcement ratio. The structural behaviour with regards to SCC beams was not significantly affected by the different pouring manners. Although the shear strength and cracking behaviour were not affected by the mixture viscosity and yield stress, beams cast with lower yield stress had slightly higher shear strength and minimum average crack heights.

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. The results obtained showed that the curing method had an effect on the shear strength 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.

Hanoon *et al.,* (2014) researched the shear behaviour of SC RC beams with stirrups and no stirrups. The effect of the ratio of the shear span to the effective depth ratio of the flexural and shear reinforcements on the nature of resistance to shear in SCC beams was investigated. Four different code provisions for shear strength namely ACI 318 – 02, BS 8110-1 (1997),CSA (1984), and NZS (1982) were compared to experimental results, and ACI 318 – 02 and BS 8110-1 (1997) were less conservative and close to experimental shear strength of SCC beams.

Senouci *et al.,* (2015) considered the shear behaviour of SCC beams in its investigation on structural behaviour of SCC beams. The shear capacity was determined by considering the compressive strength of SCC and the ratio of the shear span to the depth It was concluded that shear values were higher for lower shear span to depth ratios. The experimental results from shear stress were also compared to the code predictions made ACI (2011). The ACI (2011) code equation was conservative in predicting the ability of SCC beams to resist shear.

The bending shear response of SCC was investigated by Biolzi *et al.,* (2014). The effect of shear arm ratio and type of concrete was investigated on the nature with respect to the shear in SCC beams. It was concluded that for both SCC and NC, there was no significant effect of the shear arm ratio on the beams without stirrups. Unlike beams without stirrups where there was no significant difference in the shear strength of SCC and NC, for the beams with stirrups SCC had higher normalised shear strength than NC. This was probably due to higher steel to concrete bond of the SCC.

Semi - lightweight SCC with fly ash and metakaolin was investigated to determine whether it meets acceptable rheological properties as well as mechanical properties. To obtain lightweight SCC, the coarse aggregate used was lightweight slag aggregate. The beams were cast with no stirrups to investigate the shear behaviour of SCC. From experimental results as rheological properties were satisfied. Also at the same slag aggregate ratio of 1.5 for normal vibrated concrete and SCC, there was slightly higher normalized shear loads in the SCC compared to NC (Hassan and Ismail, 2015).

The shear strength of SCC beams with and without shear reinforcement was compared with NC beams. The behaviour of beams with stirrups was not significantly altered by the ratio of the shear span to the depth. Also they failed in a brittle manner as they failed in a diagonal failure mode. The shear reinforced beams reached their full flexural capacity and failed due to crushing of the compressive zone in a ductile manner. Also, SCC beams had higher strength accompanied by a more brittle failure as compared to NC (Cattaneo *et al.*, 2007).

SCC has gained wide applications including prestressed SCC. The shear transfer mechanism for prestressed SCC was investigated by considering the varying compressive strength, percentage of longitudinal tensile steel and effective prestress force. From results, increasing the prestressing force caused the first crack in the beam to occur at higher loads. The response of the SCC beams to the concrete strength and percentage of longitudinal reinforcement were measured in terms of the diagonal crack angles. It was concluded that the initial and principal diagonal crack angles decreased with increase in concrete strength. Also increase in longitudinal steel ratio was accompanied with a decrease in initial and principal diagonal cracks. The decrease in the initial and principal cracks signifying increase in shear strength (Shen *et al.*, 2011).

According to Kim *et al.,* (2010), the shear characteristics of high strength SCC was investigated by considering the type and volume of coarse aggregates. The shear behaviour of SCC was compared to conventional concrete (CC). From experimental results, the crack width or slip affected the aggregate interlock which influences the shear strength. The different coarse aggregate types behaved differently. There was a higher aggregate interlock, which in effect improves shear transfer mechanism in river gravel coarse aggregate, and a lower aggregate interlock in the limestone coarse aggregate type. Also, it was observed that there was less coarse aggregate fracture in low to normal strength concrete which results in more aggregate interlock. Contrarily, high coarse aggregate fracture occurred in concrete with high strength hence less interlocking of aggregate which in effect affects the strength in shear.

2.4 Cyclic Loading

When structures are subject to seismic activities, static loading analyses become inadequate to predict the behaviour and response of structural components in structures to seismic activities. The nature of seismic activities requires the behaviour and response of structural components to cyclic loading. Flexural structural members (beams) subject to cyclic loading are evaluated based on the following; cracking behaviour, energy dissipation capacity, ductility and failure modes. Energy dissipation is measured by the load- deflection hysteresis loop. The area enclosed by the hysteresis loop measures the energy dissipated.

2.4.1 Related work on Fibre Reinforced Normal Concrete Under Cyclic Loading

Spadeal *et al.,* (1997) studied the performance of concrete reinforced with fibre concrete. Comparison was made of ordinary reinforced concrete and fibre reinforced concrete under flexural cyclic loading from numerical and experimental results from literature. The presence of the steel fibre is to enable concrete accommodate larger loads, hence, larger strains as a result of steel fibres action of arresting cracks. From the numerical analyses, the fibre-reinforced concrete with improved ductility absorbs about 40% more load than the ordinary reinforced concrete.

Hameed *et al.*, (2011) investigated ductility and energy dissipation of mono and hybrid forms of metallic fibre subjected to reverse flexural cyclic loading. The results from the experiment showed that although both carbon steel fibre and amorphous metallic fibre increased the energy dissipation of the reinforced concrete beams, the amorphous metallic fibre caused greater increase in energy dissipation than the carbon steel fibre.

2.4.2 Related work on Self Compacting Concrete Under Cyclic Loading

Said *et al.*, (2007) researched on the behaviour of frames made of SCC and NC by applying reversed cyclic loading at the beam tip and a constant axial load to the column. From the experimental investigations the load – displacement tracers, cumulative dissipated energy and secant stiffness were determined. The results showed that rapid deterioration in SCC was due to the reduced coarse aggregates in SCC with reduced shear resistance.

Dabbagh and Fathi (2013) investigated the effect of the different coarse aggregate sizes of SCC under compressive cyclic loading. Analysis showed that the smaller sized coarse aggregates had an increase in compressive strength. Also, the ductility of SCC samples increased with increase in fine aggregate content. Damping behaviour of SCC samples with smaller aggregate size and higher coarse aggregate content in the coarse – to – fine aggregate ratio was better.

Al-Jeabory *et al.*, (2013) researched on improvement of fibre reinforcement on the shear resistance and ductility of reinforced self-compacting concrete columns. It was established that, cracking behaviour of tied reinforced columns with steel fibre inclusions was better due to the action of steel fibres to arrest cracks. The failure mode was changed from brittle to ductile from improved ductility of 10 – 28 %. Shear resistance was also so much improved with fibre inclusions such that shear failure was changed into flexural failure.

2.5 Fibre Reinforced SCC/Normal Concrete

The cracking characteristic of reinforced beams under loading is such that, in the early stages of loading of reinforced beams, fine vertical flexural cracks are first seen to form in the mid span of beams. As the load is increased, new flexural cracks are formed away from the mid span on both sides of the beam. Increasing the load is accompanied with flexural cracks formed at the previous stage away from the mid span. The cracks propagate diagonally into the loading zone while other diagonal cracks may form along the beam (Hassan *et al.*, 2008; Hassan *et al.*, 2010; Lachemi *et al.*, 2005). For the cracking behaviour of SCC, it was found by Castel *et al.*, (2010) it was the same for SCC and normal concrete both in ultimate and in service state at high loading and low loading values. Tension reinforcement is provided to arrest the extension of cracks in concrete beams under loading and fibre is added as tension reinforcement. Fibre is varied in proportion by weight of cement or percentage volume fraction to obtain optimum proportion at which the fibre reinforcement becomes beneficial to the SCC beams by reducing the crack openings. Artificial fibres such as steel, glass and polypropylene fibres and natural fibres such as sisal, jute, coconut palm and sugarcane have been used in various research works to investigate their benefit to concrete. In some instances, artificial and natural fibre are combined to form hybrid fibre reinforcement to investigate their effect on the performance of concrete and or mortar (Darwood *et al.*, 2011). Fibre reinforcement placed in concrete is to prevent extension of and can be used in severe environmental conditions where steel may corrode (Abdullah *et al.*, 2014). According to Romauldi (1974), addition of fibre cement matrix improves strength and ductility of concrete.

2.5.1 Related work on Fibre Reinforced SCC/Normal Concrete

Experimental investigations on the properties (fresh and hardened) of conventional and self – compacting concrete reinforced with fibre result varying conclusions. Properties that have been found to be affected by fibre reinforcement include, the workability, toughness compressive, flexural and split tensile strength and etc (Darwood *et al.*, 2011).

Dawood *et al.*, (2009) studied the response to mechanical properties of mortar of high strength that is flowable by the addition of oil palm fibre. It was observed that for the compressive and flexural strength, 0.6% of volume fraction of palm fibre addition was the optimum content for increase in the respective strengths but above 0.6%, all strengths decreased. For toughness, there was no effect of the palm fibre up to 0.8%. Higher volume fraction of palm fibre must therefore be used to improve toughness.

Concrete of lightweight density made from crushed brick as coarse aggregate was reinforced with oil palm fibre to study the concrete's response in regard to mechanical properties (Ramli *et al.*, 2010). The experimental results showed that the density of concrete crushed brick increased although slightly with addition of palm fibre. Also, the optimum content of palm fibre for the highest compressive and flexural strength was 0.8% of volume fraction.

The effect of palm fibre on the compressive strength of self compacting mortar was investigated by Mayowa *et al.,* (2013). The palm fibre was varied from 0, 0.2, 0.4, 0.6, 0.8 and 1 % by weight of cement. From results, the 0.6% of palm fibre was the optimum for increase in compressive strength of the mortar.

Concrete can be modified to change from brittle to ductile, especially if concrete is to be earthquake prone areas where ductile concrete structures would be preferred. Abhishek*et al.,* (2015) studied the effect of coconut fibre as reinforcement enhanced with steel fibres. Coconut fibre added to concrete with variations of 0.5 – 1.25 percentage volume fraction was compared to hybrid (coconut and steel) fibre reinforced concrete. The hybrid had the same amount of steel fibre of 0.5 and varying amount of coconut fibre of 0.5 – 1.25. Experimental results showed that coconut fibre reinforced concrete with maximum fibre concrete of 1 % and hybrid fibre reinforced concrete with minimum concrete fibre had good fresh and mechanical properties.

The fibre strength in shear of concrete with fibre was affected by the fibre due to the fibre placed in concrete is to improve the ductility of concrete by crack bridging mechanism. Two fibres of different nature steel and polypropylene were varied in proportion by volume to study their behaviour on the strength in shear. The crimped rectangular – section steel fibre and multifilament type as polypropylene. Form experimental results, the workability of concrete reinforced with fibre was reduced especially in the polypropylene fibre reinforced concrete. The steel fibre reinforced concrete 2% by volume without stirrups had similar cracking as that of concrete beam with stirrups and no fibre reinforcement. This goes to show that with the right proportion of steel fibre reinforcement, the fibre reinforcement can act as transverse reinforcement for strength and control of cracks. Generally, the ductility of fibre reinforced concrete was more with increased ductility in concrete reinforced with steel fibre with 2% volume of fibre (Furlan *et al.,* (1997).

Dawood *et al.,* (2011) studied high strength flowing mortar with different fibres thus steel and palm. The comparison of properties was made of the steel fibre alone, palm fibre alone and the hybrid fibre of the two different fibres. The mortar was made to achieve flow ability with ASTM C494 type F super plasticizer. It was concluded that, the flow ability was less affected when palm fibre was used as compared to the steel fibre. Toughness increased due to the ability of the fibre to arrest cracks. The hybrid fibre reinforced flowable mortar had improved toughness.

On the subject of mortars that is self compacting, Felekog˘lu *et al.,* (2007), investigated on the effect of steel fibre inclusions in self compacting repair mortars to resist surface abrasions. The spreading ability of the self-compacting repair mortars was obtained by the use of ASTM C494 type F super plasticizer. From the experimental results, there was a massive reduction in workability as a result of the use of steel fibre, however, steel fibre was able to increase the abrasion resistance.

2.5.2 Fibre Reinforced Self Compacting Concrete

Steel fibre reinforcement was used by Bhalchandra and Bajirao, (2012) to investigate their effect on the compressive and flexural strength of SCC with the addition of fly ash as supplementary cementitious material. Ordinary SCC and self compacting concrete reinforced with steel fibre (SFRSC) were compared to find their respective performance. Fibre content was varied for every SFRSC beam from 0 – 3 percentage fibre volume fraction. From experimental results, with the increase in the fibre content compressive and flexural strength were increased. The optimum fibre content for increase in split tensile strength was 1.75% with a 24.99% increase of SFRSC over ordinary SCC. Another observation was that the failure mode of SFRSC was from brittle to ductile with the increase in fibre content.

The response of SCC with and without steel fibre at different a/d ratios to cracking and deformation characteristics were studied by Shah *et al.*, (2010). From experimental results, the inclusion of steel fibres improved the crack and deformation characteristics as compared to SCC without steel fibre. The diagonal crack width for steel fibre selfcompacting concrete was smaller as compared to its counterpart SCC beams.

SCC beams were reinforced with glass fibre, the response of the concrete to deflection and ultimate load as a result of the glass fibre addition were investigated by Seshasayi *et al.*, (2008). Beams reinforced with steel were designed to fail in shear and flexure. From experimental results, there was improvement in the ductility and capacity of SCC reinforced with glass fibres to carry load as deflection increased with increase in fibre content.

The effect of fibre in concrete on crack opening and tensile stresses in stirrup was studied by Frith *et al.*, (2013). A low content of fibre 0.25% by volume in concrete was investigated due to observations made by Bayasi *et al.*, (1992) on the negative effect on workability and also additional cost which compromise material properties of hardened properties of concrete. From experimental results, it was concluded that effect on shear was limited due to the lower fibre content. Also the fibre type was brittle and crack openings which were more than 200 μm were not restrained by the fibre. The fibre could not be a complete replacement for shear bars but as supplementary reinforcement.

The effect of addition of hybrid fibre in SCC was investigated as well as the flexural behaviour of hybrid fibre reinforced SCC. The hybrid fibre was made from steel microfibre and macrofibre of diameter 0.34 mm and 0.90 mm respectively. Macrofibres were added in varying contents of 0.25%, 0.5%, 0.75% and 1% by volume. There was a decrease in flow ability and an increase in the cube and cylinder compressive strengths and modulus of elasticity. From experimental results, 0.75% of macrofibre was the optimum addition for the best mechanical behaviour. The hybrid fibre SCC was made by replacing macrofibre with microfibre at 0.75% by volume, and the optimum replacement of the macrofibre was 50%. From test results, the hybrid fibre SCC beams the micro cracks were arrested by the microfibres and macro cracks were arrested by the macrofibres, as the cracks are prevented from further opening the load carrying capacity of the SCC beams were improved hence improvement in the structural behaviour of the SCC beams. Also lesser crack width and number of cracks were observed in the fibre reinforced SCC than ordinary SCC beams which contributes to the improvement on the structural performance. Lower deflection values were recorded for fibre reinforced SCC beams than ordinary SCC beams and the lowest value with the hybrid fibre reinforced SCC beams, this resulted in better ductility of fibre reinforced SCC beams and hence the overall structural performance of hybrid fibre SCC beams were best improved (Jeenu *et al.,* 2007).

2.6 The Effect of Coarse Aggregate size on Concrete Strength

The effect of the aggregate size on the concrete strength and fracture energy was investigated by Kozul and Darwin (1997). Aggregate sizes of 12 mm and 19 mm were considered. From experimental results, the aggregate size had little effect on the compressive strength of both strength grades thus normal and high strength concrete. Also the flexural strength of both coarse aggregate sizes was not affected by the size of the aggregate. The fracture energy on the other hand decreased with increase in aggregate size for the high strength concrete and increased with increase in aggregate size for normal strength concrete.

Yaqub and Bukhari (2006) investigated the effect of coarse aggregate sizes on the compressive strength of high strength concrete. A combination of coarse aggregate sizes as 37.5 mm and 25 mm, 25 mm and 20 mm, 20 mm and 10 mm and 10 mm and 5 mm were made. From experimental results, the lower coarse aggregates 10 mm and 5 mm had higher compressive strength than the other coarse aggregate sizes.

2.7 Existing Shear Design Codes

Existing code equations for the prediction of shear strength for the design of reinforced concrete are available. In the investigations into the shear performance of SCC, code provisions from various standards have been considered (Hassan *et al.*, 2008; Safan 2012; Hassan *et al.*, 2013). Some of these codes were ACI 318 – 08, AASHTO, CSA A23.3 and EC 2. Experimental results are usually compared to the theoretically calculated shear strengths from codes. The different observations made from various researches are presented and the design codes that have been considered are as follows:

2.7.1 ACI 318 – 11

The shear stress is calculated by Equations (2.1a) and (2.1b) for beams with and with stirrups, respectively using the following expressions:

$$v_u = 0.158\lambda\sqrt{f_c'}bd + 17.24\rho_w \frac{v_f d}{M_f} bd \leq 0.29\sqrt{f_c'}bd \qquad (2.1a)$$

$$V_s = \frac{A_v(f_y)d}{S_v} \quad (2.1b)$$

Where f_c is the concrete compressive strength, λ the concrete modification factor, Vf is the factored shear force at section; Mf, factored moment at section; b and d have their usual meanings as beam width and effective depth of beam cross section, respectively. ρ_w is the ratio of longitudinal reinforcement and As = area of nonprestressed tension reinforcement in the beam.

2.7.2 AASHTO-LRFD Bridge Design Specification (2007)

In this code the concrete sections contribution to the shear resistance is shown in Equation (2.2).

$$v_u = 0.083\beta\sqrt{f_c'}b_v d_v \quad (2.2)$$

where bv represent the effective web width taken as the minimum web width within the depth dv and β indicates the ability of diagonally cracked concrete to transmit tension

2.7.3 Canadian Standards Association (CSA A23.3-04)

For beams without transverse reinforcement, the code provides shear resistance as per the equations below.

$$v_u = \lambda\beta\sqrt{f_c' b d_v} \quad (2.3a)$$

The β factor can be calculated as:

$$\beta = 520/[(1 + 1{,}500\varepsilon_\chi)(1{,}000 + S_{ze})] \quad (2.3b)$$

$$S_{ze} = \frac{35S_{ze}}{15} + a_g \leq 0.85S_z \quad (2.3c)$$

λ is the factor which accounts for low density concrete, $\varepsilon_\chi$represents the longitudinal strain at mid-depth of the member due to factored loads, $S_z$which is equal to d_v is the crack spacing parameter dependent on crack control characteristics of longitudinal reinforcement and a_g is the maximum size of aggregate in the concrete. The $f_c{}'$, which is the concrete compressive strength for high strength concrete greater than 70 MPa should have the a_g value as 0.

2.7.4 Eurocode 2 (2004)

The design shear force is given as the equation below in this code.

$$v_n = \frac{0.18}{\gamma_c}\left(1 + \frac{\sqrt{200}}{d}\right)(100 p_w f_{ck})^{0.33} \qquad (2.4)$$

where p_w is the longitudinal reinforcement ratio, d the effective depth of beam and f_{ck} is the concrete compressive strength.

The equations for shear design from the same codes in different experimental works gave contrasting observations. Thus, the ACI 318 – 2005 over predicted for high reinforcement ratios for work done by Hassan *et al.*, (2008). Conversely, for the work done by Safan (2012) using ACI 318 (1995) with the same equation as was used by Hassan *et al.*, (2008), accurate predictions were made compared to the experimental values. Also the code predictions underestimated shear strength for higher reinforced beams. This shows that shear strength factors provided in the codes are not the only factors that affect the shear strength of beams. Here the steel reinforcement strength could be a factor in the difference in the results obtained.

Strength predictions by Hassan *et al.*, (2013) using different code equations were found to be different from experimental values. Code provision by ACI 318-08

(2008), CSA (2004), AASHTO (2007) and Eurocode 2 (2005) were compared with experimental shear loads. Although all the codes were conservative in predicting the ultimate shear strengths, the different codes provided different safety margins. The EC 2 provided the highest safety margin averagely 1.71, and the ACI – 318 the most precise prediction gave a safety factor from 1.40 – 1.78. In their work lightweight SCC and normal weight SCC were investigated. With the least safety margin, the CSA correlated better with normal weight SCC while the ACI and AASHTO showed good correlation with lower safety margins in lightweight SCC.

2.8 Discussion

From the literature reviewed, the shear span-to-depth ratio, percentage of longitudinal steel reinforcement and concrete type have an influence on the shear performance of SCC. For comparison of results from different researchers, the shear strength of beams is normalized in Equations (2.5) for shear load and Equation (2.6) for normalized shear stress, to account for the difference in the characteristics of the beams under experimental investigation. The different characteristics include breadth and depth of beams, and compressive strength of concrete used.

$$V_{nz} = \frac{V_u}{\sqrt{f_c{}'}} \qquad (2.5)$$

$$v_{nz} = \frac{V_{nz}}{bd} \qquad (2.6)$$

Where $f_c{}'$ is the compressive strength

b – breadth of beam

d – depth of beam

Also, V_{nz} and v_{nz} represents the normalised shear load and stress respectively. The ultimate shear load is defined by V_u.

2.8.1 SCC Beams without Shear Reinforcement

Many researchers in the past have considered the shear performance of reinforced concrete by considering various parameters that affect the shear strength. As mentioned previously, the depth of beam and shear-span-to-depth ratio (among others) have been varied to find out their contributions or effect on the shear strength. It has been found that the compressive strength in the uncracked concrete compressive zone, aggregate interlock and dowel action of the longitudinal reinforcement ratio contribute to the shear strength and as such play a major role where shear reinforcement known as stirrups are not provided. The major contribution is provided by the aggregate interlock which provides about 50% of the shear strength. In SCC the coarse aggregate is reduced in place of the fine aggregates as the fine to coarse aggregate ratio is increased to improve the passing ability of the SCC (EFNARC, 2005). The reduction in the coarse aggregate content in SCC would therefore cause the aggregate interlock to be reduced which makes the shear behaviour of SCC an important area to be researched and fully understood.

When reinforced SCC beams without shear reinforcements (stirrups) are subject to loading test and shear, cracks can be measured in width, number, length and the path of occurrence as around or through aggregates which also affect the shear strength. With the measure of crack width, the crack width of SCC beams with different longitudinal steel ratio of 1.6% and 2% was compared and it was observed that the crack width was wider in beams with lower longitudinal steel ratio (1.6%) than in SCC beams with higher longitudinal steel ratio (2%) (Abouhussien *et al.,* 2014). In this case, the crack width is affected by the ability of the longitudinal steel to resist the further opening of cracks. This means the shear strength of SCC beams with wider cracks is smaller compared SCC beams with smaller crack widths. When it comes to the measure of shear crack path, as shear cracks take place along the path with the least resistance, and hence tend to move around aggregates rather than go through them. Also when shear cracks occur the shear strength can be measured in post – diagonal cracking resistance which is the ratio of the first crack load to the failure load expressed as a percentage (Hassan *et al.*, 2008). In researches where SCC and NC beams are compared for their post-diagonal cracking resistance for shear strength, lower post-diagonal cracking resistance was reported for SCC than NC and in this case was due to the lower coarse aggregate content in the SCC which reduced the shear transfer mechanism by the aggregate interlock.

The effect of the shear span – to – depth ratio sometimes determines the type of shear failure in beams and also the degree of ultimate shear strength. From a plot of the normalized shear stress at first crack and ultimate shear load as a function of shear span – to – depth ratio (a/d) shown in Figure 2.1. There is a rapid loss of shear strength in SCC beam after first shear crack with the increase in shear span – to – depth ratio. From Figure 2.1, it was observed that the gap between the normalized shear stress at first crack and normalized shear stress at ultimate load is wide at low a/d of 1.05. The gap decreased with increase in shear span – to – depth ratio until at a/d of 2.5 where the normalized shear strength at first crack and ultimate shear load coincide. This observation shows the strength degradation of the concrete with the increase in the shear span – to – depth ratio. This confirms literature that the ultimate shear strength is high for shear span – to – depth ratio less than 2 in simply supported beams and 2.5 in continuous beams which exhibits shear compression failure and low for shear span – to – depth ratio greater than 2 in simply supported beams and 2.5 in continuous beams which also exhibits diagonal tension failure.

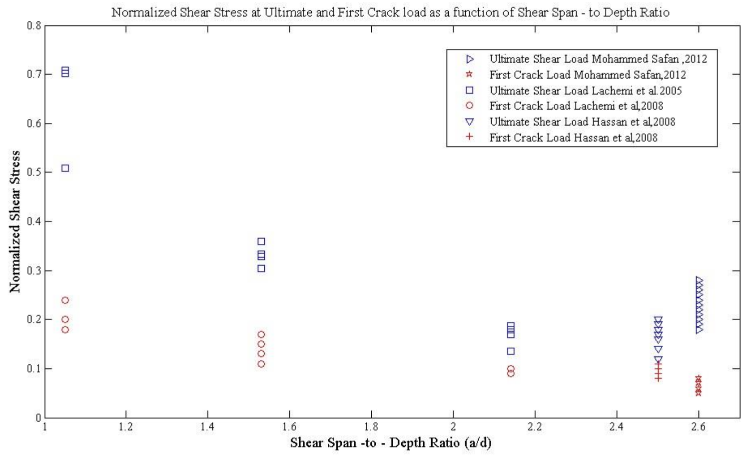

Figure 2.1. Influence of Shear span - to - depth ratio on Shear Strength of SCC

There is contribution of the longitudinal steel reinforcement ratio to the shear strength and the ultimate shear strength of concrete is expected to be greater with high longitudinal steel reinforcement ratio than low longitudinal steel reinforcement ratio. 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 steel reinforcement ratio. From the database of 44 SCC beam specimens form experimental research by Lachemi *et al.*, (2005), Hassan *et al.*, (2008), and Safan (2012) shown in Figure 2.2 the following observations were made. There was an increase in shear strength with increase in longitudinal steel reinforcement ratio. However, the shear span-to-depth ratio can control the shear stress irrespective of the amount of longitudinal steel reinforcement, which makes it a defining factor on the shear strength. Lachemi *et al.*, (2005) used three different longitudinal steel reinforcement ratios of 1.15%, 1.57% and 1.6% at shear span-to-depth ratios of 1.05, 1.53 and 2.14 respectively, and reported that for 1.6% the shear stress was less as shown in Figure 2.2.

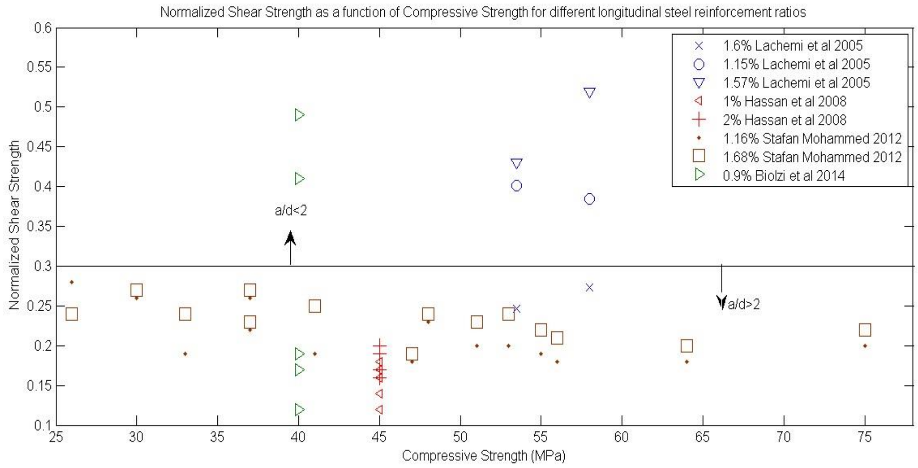

Figure 2.2. Influence of longitudinal steel reinforcement ratio on Shear Strength

2.8.2 SCC Beams with Shear Reinforcement

Shear reinforcement in the form of stirrups is to provide crossing for the crack to resist shear force and restrict the growth of the crack and reduce the further penetration into the compression zone, prevents widening of crack and help maintain aggregate interlock within the concrete and provide further restraint against splitting of concrete along the longitudinal reinforcement due to their confinement effect (Arezoumandi, 2013).

The behaviour of SCC beams with stirrups has been investigated in the past by several researchers to define the effect stirrups on SCC beams. For the arrangement of shear reinforcement (spacing of stirrups) there is the normal arrangement and congested arrangement. For the congested arrangement spacing between the stirrups is reduced which gives more room for the number of stirrups to be increased. In the experimental investigation done by Xu, (2011) the amount of stirrups was doubled and results showed that research the ultimate shear resistance of the beam with doubled amount of shear reinforcement is not improved much. Whereas in Choi *et al.*, (2012) with limited number of specimens of two SCC and two NC tested for, the congested arrangement had a low ultimate shear resistance as compared to the relatively non-congested arrangement. The number of specimens used in the experimental investigation was not enough to characterize the effect of the arrangement of vertical shear reinforcement on the shear resistance of SCC beams.

With regards to investigation into the shear behaviour of SCC beams with stirrups, the experimental research done by Choi *et al.*, (2012), Abed (2012) and Atshan, (2012) had SCC beams exhibit sudden shear compression failure. On the other hand, in the experimental research done by Hannon *et al.*, (2014) the SCC beams with stirrups exhibited diagonal shear failure.

2.8.3 Cracking and Fibre Reinforcement of Concrete

The differences and similarities in the cracking behaviour of SCC and NC observed in previous researches where SCC beams were compared to NC beams have been noted as follows. In these researches the beams made had similar characteristics such as the beam configuration, and longitudinal steel reinforcement ratios and stirrups (where shear reinforcements were used). However, in spite of the similarities in the crack width of SCC and NC there were differences observed in the load carrying capacity and ductility of SCC and NC beams. The crack width was the same for SCC and NC for the research work done by Hassan *et al.*, (2008), Hassan *et al.*, (2010), Biolzi *et al.*, (2014), and Harkouss *et al.*, (2014). The number of cracks in NC were more than in SCC for Hassan *et al.*, (2008) and (Hassan, *et al.*, 2010), this improved the shear resistance in NC beams than in SCC beams. The increase in number of cracks here was due to the increase in coarse aggregate content. In other works by Abouhussien *et al.*, (2014) and

Sonebi *et al.*, (2003), the number of cracks were more in the SCC than in the NC. This slightly improved the shear strength of SCC than NC with the increase in number of cracks of the SCC.

In addition to the improving the cracking behaviour (crack width, number, length and spacing) of fibre-reinforced concrete, there is an added benefit of improving deflection, stiffness and ultimate load. From observations made by previous research the following conclusions are warranted. For crack width, the crack width is reduced as the fibre prevents further crack propagation in SCC beams reinforced with fibre compared to ordinary SC beams as was observed by (Shah *et al.*, 2010) and (Fritih, *et al.*, 2013). For crack spacing, with addition of steel fibre reinforcement, spacing between cracks are reduced such that closely spaced cracks were observed in steel fibre reinforced NC and SCC beams as compared to ordinary NC and SCC beams from (Furlan *et al.*, 1997) and (Fritih *et al.*, 2013). With the smaller crack spacing in fibre reinforced SCC beams in Fritih *et al.*, (2013) denser crack network was also observed as stress transfer was improved through cracks and more cracks are developed hence the denser network. Shah *et al.*, (2010) observed the same scenario of increase in number of cracks with the addition of steel fibre reinforcement at smaller crack width than fewer cracks with wider crack width.

The ductility of SCC beams with fibre reinforcement is found to be improved from experimental research by Bhalchandra and Bajirao, (2012) and Shah *et al.*, (2010). According to Shah *et al.*, (2010) the improvement in the ductility was observed as fibre reinforced SCC beams compared to ordinary SCC beams sustained larger deflections due to the improved cracking behaviour with the reduced crack width and delay in the formation of flexural cracks at 60 % to 80% of ultimate loads. Contrary to the benefit to ductility observed by (Bhalchandra and Bajirao, 2012) and (Shah *et al.*, 2010), Fritih *et al.*, (2013) found that the ductility and load carrying capacity of fibre reinforced SCC and ordinary SCC beams were similar as fibre reinforcement did not have consequential effect on these properties. This was because fibres preventing crack widening and increasing stress transfer through crack and tension stiffening increases the number of cracks which offsets the increase in stiffness.

2.9 Summary

According to the evaluation of literature on the structural behaviour of SCC and fibre reinforced SCC and NC the following conclusion and recommendation can be made. With the fundamental knowledge on shear behaviour of SCC and fibre reinforced NC and SCC beams using steel and glass fibre to improve cracking behaviour and structural performance in terms of load carrying capacity and ductility, there is the possibility that natural fibre can also used to influence the structural performance of SCC.

CHAPTER 3: METHODOLOGY

3.1 Introduction

The experimental design was aimed at determining the shear behaviour of SCC beams with different coarse aggregate sizes (10 mm and 20 mm) and with varying amounts of fibre reinforcement (palm kernel fibre). A total of 20 beams were tested to achieve the objective of the research. Apart from the variables considered which were the size of coarse aggregates, amount of fibre reinforcement, all other characteristics of the beams were kept the same. Laboratory tests conducted on the concrete cast included, compressive strength test, modulus of rupture test, and shear test. The beams were simply supported and subjected to a four – point loading under monotonic and cyclic loading.

3.2 Materials

The materials used for the experiment included; cement, aggregates (fine and coarse), admixture, portable water and palm kernel fibre. For the cement Limestone Portland cement with strength grade of 42.5N/mm² produced by Ghacem was used. River washed sand obtained from Fumesua in the Ashanti Region, which passed through 4.75 mm sieve was used as fine aggregates. For coarse aggregates, 20 mm and 10 mm, granitic aggregates from Rural Sisters Stone Processing Limited in the Ashanti Region were used. For the admixture a super plasticizer free from chlorides that meets ASTMC 494-92 requirements for type A and F was used as High Range Water Reducing Agent (HRWRA). This chemical admixture was added to improve the workability, ease of flowabilty of concrete and segregation resistance. Dried palm kernel fibre was used as the fibre reinforcement. Potable water from Ghana Water Company Limited was used as mixing water. For the formwork, ½ inch plywood was used for the casting of beams.

3.3 Concrete Mixes

The SCC mix design was designed through trial mixes using the work such as from Helnicks *et al.*, (2013), Hassan *et al.*, (2013) and Lachemi *et al.*, (2005) as a guide. Various coarse-to-fine aggregate ratios were tried and the value found acceptable was 0.697. From previous research by Lachemi *et al.*, (2005) and Hassan *et al.*, (2008 and 2010), the water – cement ratio was initially kept at 0.4 in addition to the use of HRWRA that meets ASTMC 494-92 requirements for type F by Hassan *et al.*, (2008 and 2010) to obtain slump flow mean diameter of 635 – 790 mm. From the trial mix using the water – cement ratio of 0.4 and the HRWRA, the slump flow diameter obtained was 400 mm which was below the mean diameter as required by SCC guidelines. Tthe water – cement ratio was adjusted to 0.45 and 0.5. The resulting mean diameter obtained after the adjustment gave 560 mm using a water – cement ratio of 0.5. This was within the required value from the SCC guidelines. The mix ratio obtained from the trial mix was 1:1.82:1.27. Table 3.1 presents the mix proportions of the constituent material.

Table 3.1. Mix Proportions of Concrete

Mix Designation	Cement Content (kg/m^3)	Cement/Sand/Coarse Aggregate (Mix Ratio)	Water Cement Ratio, W/C	Admixture/HRWRA l/100 kg cement
SM2, SC2	450	1:1.82:1.27	0.5	1.2
SM1, SC2	450	1:1.82:1.27	0.5	1.2

3.3.1 Beam Specification

Due to the objective of the research to vary the size of coarse aggregates and fibre reinforcement, all other parameters of the research were kept constant. The beam configuration was made up of the same cross section 110mmX275mmX2000mm as width, depth and length respectively. The effective length of the beam was 1800 mm with a shear span of 600 mm. The effective depth of the beam was 240 mm. The beams were singly reinforced with three number 12 mm tensile steel reinforcement made of mild steel bars with mean yield strength of $250N/mm^2$. The percentage of the longitudinal reinforcement ratio was 1.121%. The beam section is shown in Figure 3.1

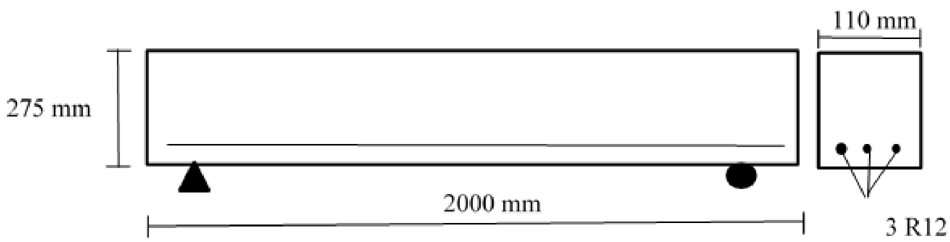

Figure 3.1. Singly Reinforced Beam

3.3.2 Beam Nomenclature

The nomenclature adopted for identification of the beams was a combination of letters and numbers. The first letter ‘S’ or ‘N’ represents the type of concrete as selfcompacting or normal, the second letter ‘M’ or ‘C’ represents the type of loading as monotonic or cyclic. The two letters were followed by the size of coarse aggregates which is represented by a number ‘1’ or ‘2’ which indicates the coarse aggregate size as 10 mm and 20 mm respectively. The varying amounts of fibre reinforcement were represented as 0, 0.25, 0.5, 0.75 and 1. For example SM2-0.5 represents selfcompacting concrete under monotonic loading test with 20 mm coarse aggregate and

0.5 % of fibre reinforcement. The beam details are presented in Table 3.2

Table 3.2. Beam Details

BEAM DESIGNATION	LENGTH (mm)	WIDTH (mm)	DEPTH (mm)	ShearSpan to Depth a/d	STEEL	
					Area of Steel As (mm^2)	Longitudinal Reinf. Ratio 100As/bh
SM1-0	2000	110	275	2.5	339.29	1.22
SM1 -0.25	2000	110	275	2.5	339.29	1.22
SM1-0.5	2000	110	275	2.5	339.29	1.22
SM1-0.75	2000	110	275	2.5	339.29	1.22
SM1-1	2000	110	275	2.5	339.29	1.22
SM2-0	2000	110	275	2.5	339.29	1.22
SM2-0.25	2000	110	275	2.5	339.29	1.22

SM2-0.5	2000	110	275	2.5	339.29	1.22
SM2-0.75	2000	110	275	2.5	339.29	1.22
SM2-1	2000	110	275	2.5	339.29	1.22
SC1-0	2000	110	275	2.5	339.29	1.22
SC1-0.25	2000	110	275	2.5	339.29	1.22
SC1-05	2000	110	275	2.5	339.29	1.22
SC1-0.75	2000	110	275	2.5	339.29	1.22
SC1-1	2000	110	275	2.5	339.29	1.22
SC2-0	2000	110	275	2.5	339.29	1.22
SC2-0.25	2000	110	275	2.5	339.29	1.22
SC2-0.5	2000	110	275	2.5	339.29	1.22
SC2-0.75	2000	110	275	2.5	339.29	1.22
SC2-1	2000	110	275	2.5	339.29	1.22

3.4 Mixing of Concrete and Casting of Beams

A concrete mixer was used to mix the concrete. Mix ratio of the constituents was kept constant for the beam specimens for both the 20 mm and 10 mm aggregates. The palm kernel fibre content measured by weight of cement varied between 0% and 1% at 0.25% increments. The cement, sand, coarse aggregates were measured and poured into the concrete mixer. The sand was poured first followed by the cement and then the coarse aggregates and mixed. The palm kernel fibre was then added and further mixed. About 60 % of the mixing water was added to the dry mix and mixing was continued until all the materials were wet. The HRWRA was measured as 1.2 l/100 kg of cement and added to about 75% of the remaining water and added to the wet concrete mix. At the point where the hard concrete appeared loosened and workable, the remaining water was added. After a homogeneous and workable concrete had been obtained the fresh test on the concrete was conducted and 150 mm sized cubes were cast for every batch of concrete mixed. A total of 10 concrete mixes were batched. For every single mix, three 100mmX100mmX500mm unreinforced beams and three 150mmX150mmX150mm cubes were cast to test for modulus of rupture and compressive strength, respectively. Each mix was made for two beams for a given aggregate size and percentage of fibre addition but to be tested under different loading. The concrete mixture was poured from the concrete mixer into wheel burrows and wheeled to the formwork for pouring into the formwork for ease of placement. Before placement of concrete the slump flow test was conducted on the concrete to ascertain the fresh property of the concrete. The inside of the formwork was coated with motor oil to allow for easy removal of the formwork upon hardening of the concrete beams. Due to the nature of the concrete the formwork was removed 3 days after casting. The beams and cubes were cured for 28 days before testing. The compressive strength and modulus of rupture of the respective concrete was determined by subjecting 150mmX150mmX150mm concrete cubes and 100mmX100mmX500mm beams under compressive and flexure test respectively on the 28th day as well. Before the beams were tested, they were painted with white paint to facilitate the observation of the cracks as they progress across the beam.

3.5 Instrumentation

Simply supported beams were tested for their response to loading by means of different instrumentation. Different parameters were observed and recorded. The parameters included, the load, mid-span deflection, crack width, number of cracks and crack spacing. Each of the parameters was measured from different instruments. The load was applied by using a 230 kN capacity load – controlled actuator. The mid – span deflection at every load increment was recorded by a dial guage and the crack width with a crack measuring instrument. Figure 3.2 and Figure 3.3 shows the schematic representation of the experimental set-up, and the actual loading arrangements of the beams respectively.

3.5.1 Testing of Beams

The beams were simply supported and subject to a four–point loading test. Two different tests were conducted depending on the type of loading, i.e., monotonic and cyclic loading. The loading arrangement was kept constant for all the beams regardless of the type of loading. The load was applied by a hydraulic jack through a steel spreader (I-section) placed on two cylindrical steel bars for the two point loads on the beam as shown in the loading arrangement. A dial gauge of 0.001 mm accuracy was placed in the mid-span under the beam to measure the mid-span deflections as the beams were being loaded. For the monotonic loading test, the load was applied at 2 kN increments till failure. For the cyclic loading test, the load was applied at 2 kN increments till the load at first crack and the beam was unloaded and reloaded again to the first crack load for 20 cycles followed by loading until failure. For all loading and unloading, the deflections were measured and recorded. For all the loading tests, the crack width and length were measured. The crack width was measured by means of a crack measuring instrument. At the end of loading, cracks were identified and marked with felt pen. The spacing between cracks was also measured and recorded.

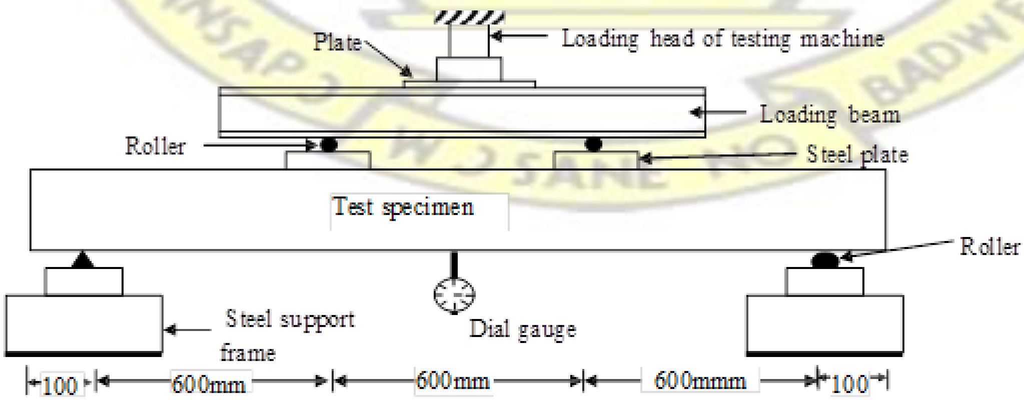

Figure 3.2. Schematic sketch of experimental set – up

Figure 3.3. Experimental set-up

CHAPTER 4: TEST RESULTS AND DISCUSSION

4.1 Introduction

Results obtained from experiment for 20 beams are presented for discussions in this chapter. Results presented in tables and plotted on graphs for different behaviours, and observations are made from calculations and plots to discuss the behaviour of beams response under loading.

4.2 Properties of Fresh Concrete

The slump flow test was conducted to determine the flowability of the concrete using the conventional slump cone and the T500 test for viscosity. Segregation resistance was visually observed as was done in previous research by Jalal *et al.*, (2015). The values obtained from the slump and viscosity tests are shown in Table 4.1. It is seen that all the values in the table meet the SCC guidelines (EFNARC, 2005). For the T500 test, the results ranged between 5 and 13 seconds. This means the mixes all met the class 2 viscosity category. Application of SCC for specific structural components are indicated by the class of viscosity. According to Hassan *et al.*, (2013), Class 1 viscosity is recommended for floors, slabs, plies and walls and class 2 viscosity is for ramps.

Table 4.1. Results of slump and viscosity test for fresh concrete

BEAM DESIGNATION	Slump flow (mm)	T500 (secs)
SM1-0	616	7
SM1-0.25	629	5
SM1-0.5	595	13
SM1-0.75	581	11
SM1-1	584	10

Table 4.1 Continued

BEAM DESIGNATION	Slump flow (mm)	T500 (secs)
SC1-0	616	7
SC1-0.25	629	5
SC1-0.5	595	13
SC1-0.75	581	11
SC1-1	584	10
SM2-0	638	8

SM2-0.25	690	7
SM2-0.5	681	7
SM2-0.75	640	11
SM2-1	600	9
SC2-0	638	8
SC2-0.25	690	7
SC2-0.5	681	7
SC2-0.75	640	11
SC2-1	600	9

4.3 Strength of hardened concrete

Compressive and flexural strength tests were conducted after 28 days of curing. The average compressive strength and flexural strength are detailed in Table 4.2

Table 4.2. Hardened concrete strength values

BEAM IDENTITY	Compressive Strength (MPa)	Modulus of Rupture (MPa)
SM1-0	16.8	4.2
SM1-0.25	17.91	4.5

Table 4.2. Continued

BEAM IDENTITY	Compressive Strength (MPa)	Modulus of Rupture (MPa)
SM1-0.5	19.91	5
SM1-0.75	21.8	5.45
SM1-1	19.0	4.8
SC1-0	16.8	4.2
SC1-0.25	17.91	4.5
SC1-0.5	19.91	5

SC1-0.75	21.8	5.45
SC1-1	19.0	4.8
SM2-0	14.8	3.7
SM2-0.25	15.58	3.78
SM2-0.5	18.76	4.7
SM2-0.75	20.17	5
SM2-1	18.34	4.5
SC2-0	14.8	3.7
SC2-0.25	15.58	3.78
SC2-0.5	18.76	4.7
SC2-0.75	20.17	5
SC2-1	18.34	4.5

The addition of the fibre was found to improve on the compressive strength, as the compressive strength increased with increase in fibre content.

4.4 Cracking Loads and Failure Loads

Based on equations from BS 8110, the theoretical failure loads were calculated for considering the concrete section alone and the concrete with the longitudinal steel reinforcement contribution to the shear strength. Theoretical and experimental first crack loads and ultimate failure loads are shown in Table 4.3. The theoretical failure loads for the various failure modes were calculated from the following equations.

$$A_s = \frac{M}{(0.95 f_y) z} \qquad (4.1)$$

where

$z = d\{0.5 + \sqrt{0.25 - \frac{k}{0.9}}\} \leq 0.95d$ and $k = \frac{M}{f_{cu} b d^2}$

Equation (4.1) represents bending failure with steel alone yielding in tension. While he concrete cracking failure was calculated from Equation (4.2). For concrete with longitudinal reinforcement steel, the shear stress is given as in Equation (4.3).

$$Mcr = \frac{f_{tc}}{6} bh^2 \qquad (4.2a)$$

$$P'cr = f_{tc} bh^2 / L \qquad (4.2b)$$

$$v_c = 0.79 \left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}} \left(\frac{400}{d}\right)^{\frac{1}{4}} \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} \qquad (4.3)$$

The concrete crushing load calculated from Equation 4.4 as follows,

$$M = 0.156 f_{cu} bd^2 \qquad (4.4)$$

Where f_{cu}, f_{tc} represents concrete compressive and flexural strength respectively and f_y represents the steel yield stress. As represents the area of the tension reinforcement and b, d and h represents width, depth and total height of beams respectively.

The experimental failure load for the 10 mm nominal coarse aggregate beams was averagely 143% and 131% of the theoretical calculated shear loads considering the theoretical values of concrete's contribution alone and the concrete with the longitudinal steel reinforcement respectively. Previous work (Adom-Asamoah *et al.*, 2009), indicated that contribution for the longitudinal steel reinforcement should not be neglected A similar trend was observed in the case of the 20 mm nominal coarse aggregate beams as the average experimental shear load was 107% of the theoretical load for the concrete alone and 98% for the concrete and longitudinal steel reinforcement. The first crack load *Pcr* was averagely 37% and 42% of the ultimate shear load *Pult* for 10 mm and 20 mm nominal coarse aggregate beams, respectively.

This shows that before the first flexural crack occurred, the 20 mm nominal coarse aggregate beams sustained higher loads than the 10 mm nominal coarse aggregate beams. At different contents of the fibre under different loading, the lowest ratio of the experimental first crack load to the ultimate shear load was observed in the beams without fibre. This was not found in the 10 mm nominal coarse aggregate beams under monotonic loading. This shows that the fibre was effective in delaying occurrence of the first crack. The lowest ratios of first crack to ultimate loads were 0.33, 0.29 and 0.12 for SC1 – 0, SM2 – 0, and SC2 – 0 respectively.

The performance of fibre in all the beams was evaluated by comparing ultimate failure loads at respective fibre contents to the beam with no fibre. For the 10 mm nominal coarse aggregate beams under monotonic loading, there was a decrease in the ultimate failure load as 3.03%, 17.65%, and 26.47% for beams SM1-0.25, SM1-0.5 and SM1-1. The only increase in the ultimate failure load of 2.94% with addition of fibre was observed with fibre content of 0.75%. For the 10 mm nominal coarse aggregate beams under cyclic loading, the maximum increase of 28.57% was observed at 0.75% of fibre. Beyond 0.75% of fibre content, the ultimate failure load was found to decrease. This could be a result of balling effects of fibres at higher fibre contents as was observed by Ganesan *et al.*, (2014). For the 20 mm nominal coarse aggregate beams under monotonic loading, there was an increase in the ultimate failure loads as 57.14%, 71.43%, 78.57% and 28.57% for SM2-0.25, SM2-0.5, SM2-0.75 and SM2-1 respectively. The maximum increase here was also at 0.75% of fibre beyond which the ultimate failure load decreased. For the 20 mm nominal coarse aggregate beams under cyclic loading, the addition of the fibre to the SCC was accompanied with decrease in ultimate failure load. The decrease was 33.33%, 18.18%, 39.39% and 42.42% for SC20.25, SC2-0.5, SC2-0.75 and SC2-1 respectively. The least decrease was at SC2-

0.25 showing that with 0.25% of the fibre the decrease in ultimate failure load unlike the other fibre contents is reduced.

Table 4.3. Theoretical and Experimental Loads

BEAM IDENTITY	Theoretical Failure Load, *P'ult*(KN)						Experimental Loads (KN)			*Pcr/Pult*	*Pult/P'ult*	*Pcr/P'cr*	*Pult/Ps* Reserve Strength
	Concrete Cracking *P'cr*	Steel yielding Alone in Tension	Total Flexural in Tension Alone	Concrete Alone in Shear	Concrete and Longitudinal bars in Shear	Concrete Crushing	First Crack Load *Pcr*	First Shear Crack Load *Ps*	Ultimate Load *Pult*				
SM1-0	19.41	39.53	58.94	33.21	36.11*	54.37	20	60	68	0.29	1.88	1.03	1.13
SM1-0.25	20.69	39.88	60.57	33.93	36.89*	58.03	28	60	66	0.42	1.79	1.35	1.10
SM1-0.5	23.00	40.41	63.41	35.14	38.21*	64.62	14	48	56	0.25	1.47	0.61	1.17
SM1-0.75	25.19	40.83	66.02	36.22	39.38*	70.84	24	58	70	0.34	1.78	0.95	1.21
SM1-1	21.98	40.19	62.17	34.60	37.62*	61.69	18	38	50	0.36	1.33	0.82	1.32
SC1-0	19.41	39.53	58.94	33.21	36.11*	54.37	18	52	56	0.32	1.55	0.93	1.08
SC1-0.25	20.69	39.88	60.57	33.93	36.89*	58.03	28	54	60	0.47	1.63	1.35	1.11
SC1-0.5	23.00	40.41	63.41	35.14	38.21*	64.62	26	52	58	0.45	1.52	1.13	1.12
SC1-0.75	25.19	40.83	66.02	36.22	39.38*	70.84	30	60	72	0.42	1.83	1.19	1.20
SC1-1	21.98	40.19	62.17	34.60	37.62*	61.69	24	56	62	0.39	1.65	1.09	1.11
Average										0.37	1.64	1.05	1.15

Table 4.3 continued

BEAM IDENTITY	Theoretical Failure Load, *P'ult*(KN)						Experimental Loads (KN)			*Pcr/Pult*	*Pult/P'ult*	*Pcr/P'cr*	*Pult/Ps* Reserve Strength
	Concrete Cracking *P'cr*	Steel yielding Alone in Tension	Total Flexural in Tension Alone	Concrete Alone in Shear	Concrete and Longitudinal bars in Shear	Concrete Crushing	First Crack Load *Pcr*	First Shear Crack Load *Ps*	Ultimate Load *Pult*				
SM2-0	17.10	38.78	55.88	31.84	34.61*	47.71	8	22	28	0.29	0.81	0.47	1.27
SM2-0.25	18.00	39.09	57.09	32.39	35.21*	50.35	22	44	44	0.50	1.25	1.22	1.00
SM2-0.5	21.67	40.12	61.79	34.45	37.46*	60.83	24	40	48	0.50	1.28	1.11	1.20
SM2-0.75	23.30	40.47	63.77	35.30	38.38*	65.47	22	40	50	0.44	1.30	0.94	1.25
SM2-1	21.19	40	61.19	34.20	37.18*	59.45	18	34	36	0.50	0.97	0.85	1.06
SC2-0	17.10	38.78	55.88	31.84	34.61*	47.71	8	62	66	0.12	1.91	0.47	1.06
SC2-0.25	18.00	39.09	57.09	32.39	35.21*	50.35	22	42	44	0.50	1.25	1.22	1.05
SC2-0.5	21.67	40.12	61.79	34.45	37.46*	60.83	20	50	54	0.37	1.44	0.92	1.08
SC2-0.75	23.30	40.47	63.77	35.30	38.38*	65.47	18	36	40	0.45	1.04	0.77	1.11
SC2-1	21.19	40	61.19	34.20	37.18*	59.45	20	34	38	0.53	1.02	0.94	1.12
Average										0.42	1.23	0.89	1.12

*Governing theoretical failure load according to BS8110

4.5 Load - Deflection Curves

There is a limit for deflection at the serviceability limit beyond which the beam becomes unfit to meet the deflection requirement for serviceability. According to ACI – 318 (2008), the requirement for deflection under serviceability conditions is calculated as *l*/360, where *l* is the effective span of the beam. The deflection at serviceability was calculated as 5 mm.

4.5.1 10 mm Nominal Coarse Aggregate Beams Under monotonic loading

For all the beams, with the exception of SM 1-0.5, under application of load, the initial stiffness after first crack had high values with SM1-0.75 exhibiting the highest stiffness as shown in Figure 4.1. The highest experimental failure load was observed to be highest in SM1-0.75 and lowest in SM 1-1. The most ductile beam SM 1-0, which contained no fibre, dissipated the most energy calculated from the area under the load deflection curve and had the second highest experimental failure load. All the beams met the deflection requirement at serviceability as they did not deflect up to 5mm under

application of their respective service loads.

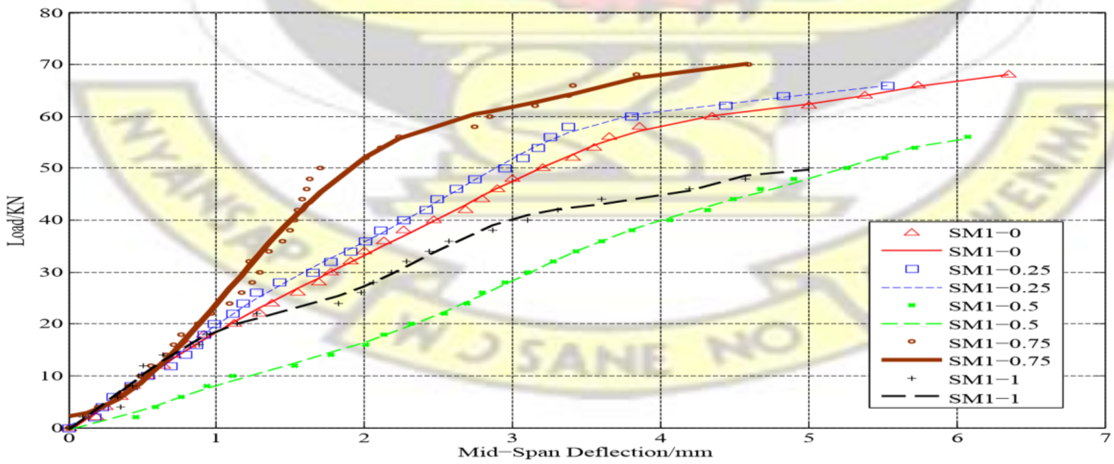

Figure 4.1. Load- Deflection curve for 10 mm nominal coarse aggregate beams

4.5.2 20 mm Nominal Coarse Aggregate Beams Under Monotonic Loading

From the load-deflection graph shown in Figure 4.2, at first crack before non – linearity it can be observed that the beams with fibre were stiffer than the beam with no fibre.

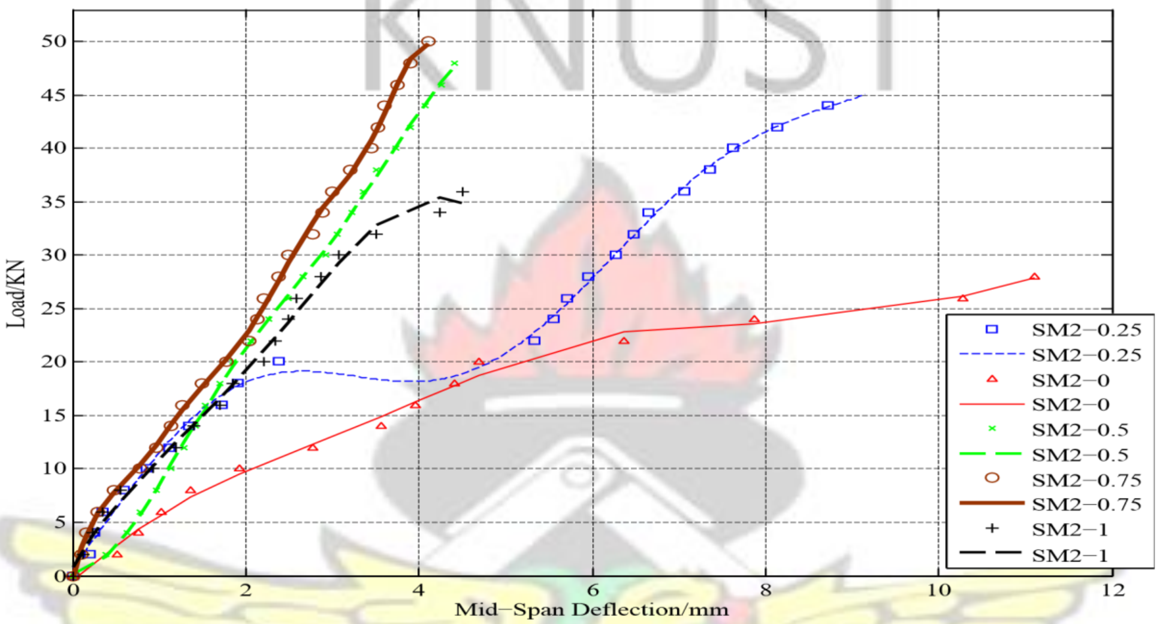

Figure 4.2. Load- Deflection curve for 20 mm nominal coarse aggregate beams

The beams with fibre SM 2-0.75 was the stiffest beam. Also from the graph it can be seen that the beams with fibre had higher load carrying capacity in terms of ultimate loads with SM2-0.75 having the highest ultimate load. The beam identified as SM 2-0 failed at the lowest experimental load yet was the most ductile of all the beams. Under application of service load, all the beams, with the exception on SM 2-0.25 passed the requirement for deflection of 5 mm at serviceability limit state with lower deflection values. For SM2-0.25 the deflection at the serviceability limit state was 6.1 mm, which was more than the required deflection.

In this study, with regards to the load bearing capacity the beam with 0.75% fibre content in both the 10 mm and 20 mm nominal coarse aggregate beams exhibited the highest load carrying capacity. This contrasts the observation by Fritith *et al.*, (2013), that the load carrying capacity of beams are not significantly modified by fibre addition.

In this case, the high load carrying capacity could be as a result of high concrete strength at the 0.75% fibre addition in both 10 mm and 20 mm nominal coarse aggregate beams. This suggests a greater load bearing capacity with better concrete compressive and flexural strengths as was observed by Meda *et al.*, (2005). With regards to the ductility there was no noticeable effect of fibre reinforcement as the beams with no fibre in both 10 mm and 20 mm aggregate beams were rather the most ductile beams. Previous experimental works that reached different conclusions from this study in terms of ductility used steel fibre in contrast to the natural fibre used in this study. Furlan *et al.*, (1997) observed improved ductility with addition of steel fibre whereas, Fritih *et al.*, (2013) observed no significant improvement in the ductility with addition of steel fibre. The differences could be as a result of different types and content of steel used.

4.5.3 10 mm Nominal Beams Under Monotonic and Cyclic Loading

Comparison of all 10 mm nominal coarse aggregate beams under monotonic and cyclic loading conditions was done here. The beam without fibre, 0% and the beam with 0.75% fibre content behaved as expected with regards to the monotonic loading curve enveloping the cyclic loading curve (see Fig. 4.3 and Fig. 4.6). As seen in Fig 4.3, the cyclic load – deflection curve was similar to the monotonic load – deflection curve. This shows no loss of structural integrity as was observed by Adom-Asamoah *et al.*, (2009), where similar slabs were tested under monotonic and cyclic loading. On the other hand, the remaining beams at 0.25%, 0.5%, and 1% fibre contents did not observe the enveloping effect of the monotonic loading curve over the cyclic loading curve (see Fig. 4.4, Fig. 4.5 and Fig. 4.7). The figures depict higher stiffness values in the cyclic loaded beams than in the monotonic loaded beams. At 0% fibre content, before first crack load the stiffness of the beam under monotonic loading was lower than that of the beam under cyclic loading. After the loading and reloading cycles, the stiffness of the beam under cyclic loading had degraded therefore the beam under monotonic loading was stiffer. The beam under monotonic loading was much ductile sustaining higher ultimate loads. With an addition of fibre with 0.25% fibre, there was little or no stiffness degradation of the beam under cyclic loading. The cyclic load – deflection curve was similar to the monotonic load – deflection curve. The beam under monotonic loading was ductile and sustained slightly higher ultimate load as shown in Figure 4.4. Generally, for 0.5% fibre content, the beam under monotonic loading was more ductile than the beam under cyclic loading (See Fig. 4.5). From Figure 4.6, it can be seen that the cyclic load – deflection curve was similar to the monotonic load – deflection curve. At 1% fibre content it can be seen from Figure 4.7 that, the monotonic load – deflection curve was beneath the cyclic load – deflection curve which shows a deterioration in

stiffness of the beam under monotonic load. The beam under monotonic loading was more ductile with lower ultimate load.

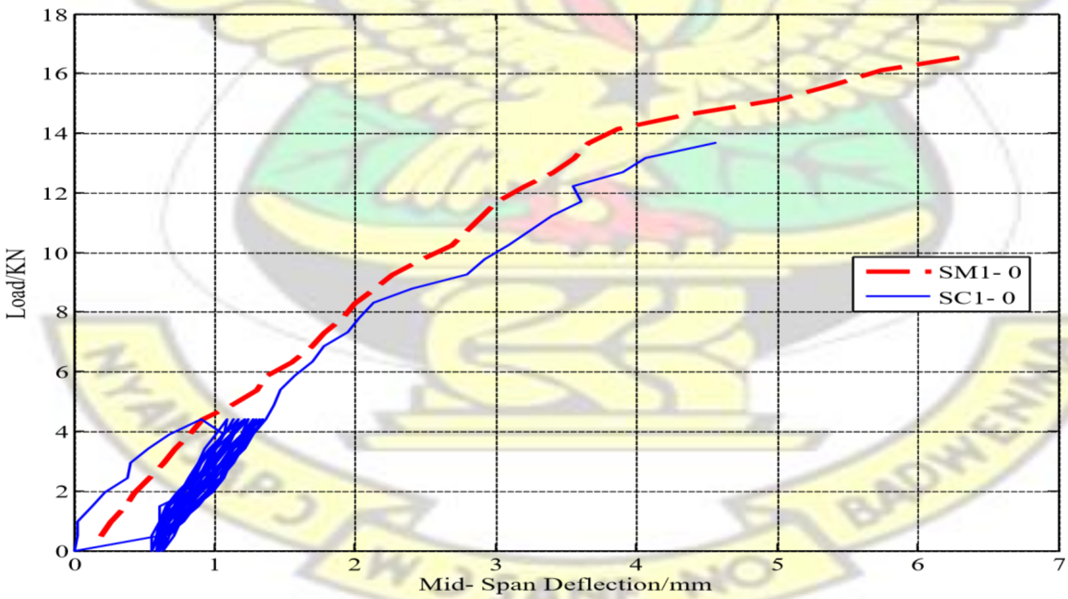

Figure 4.3. Monotonic and cyclic loading curves for beams with 0% fibre content

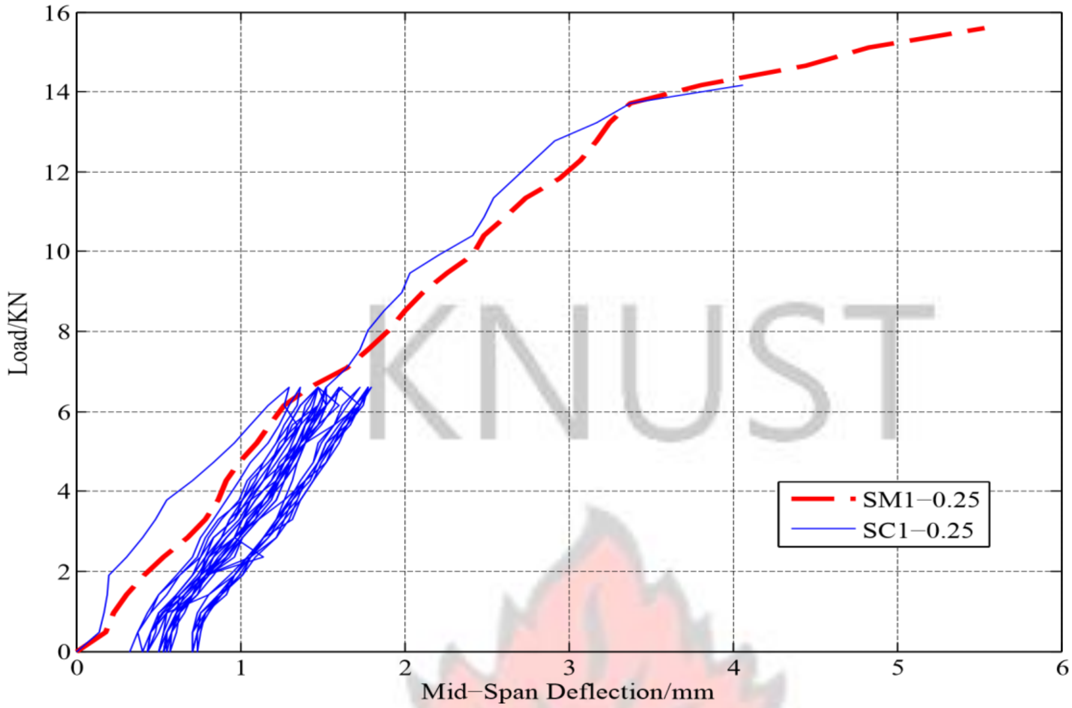

Figure 4.4. Monotonic and cyclic loading curves for beams with 0.25% fibre content

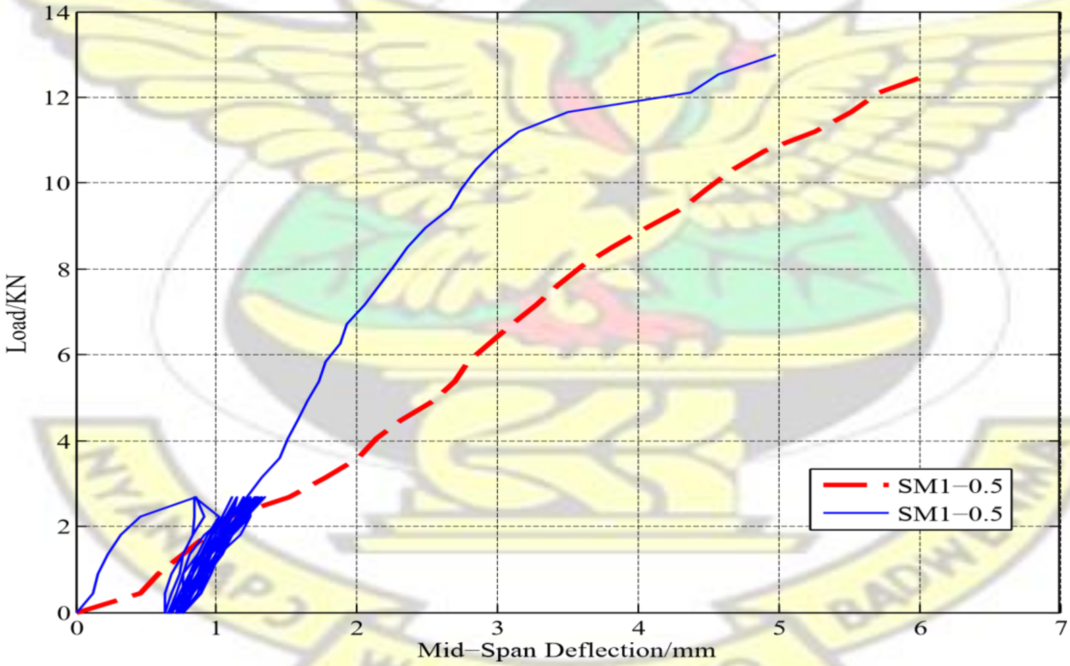

Figure 4.5. Monotonic and cyclic loading curves for beams with 0.5% fibre content

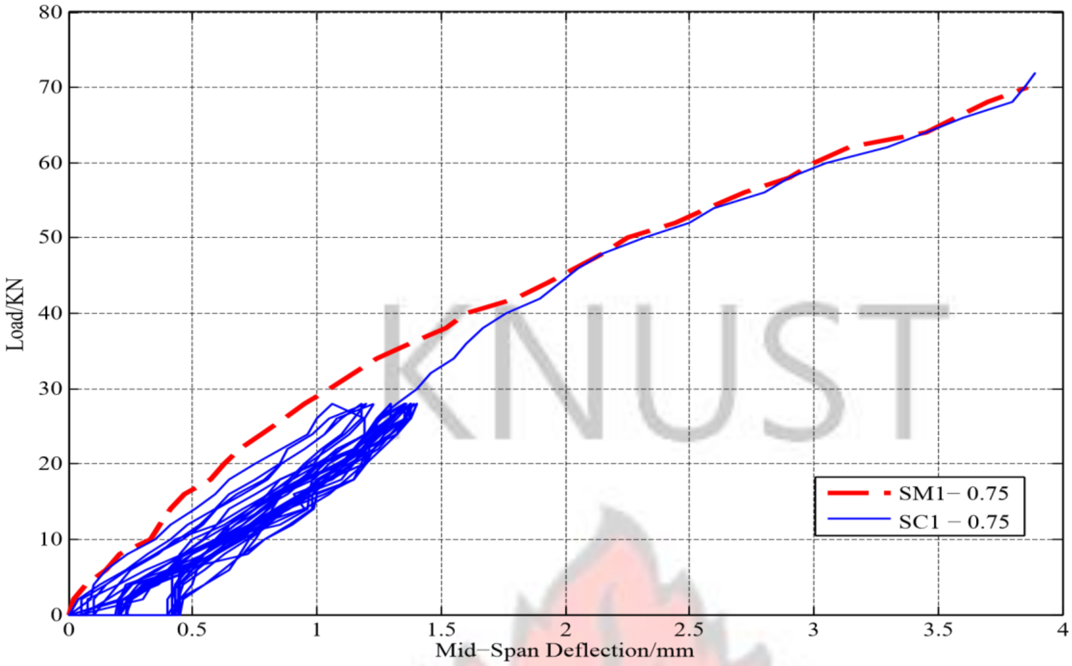

Figure 4.6. Monotonic and cyclic loading curves for beams with 0.75% fibre content

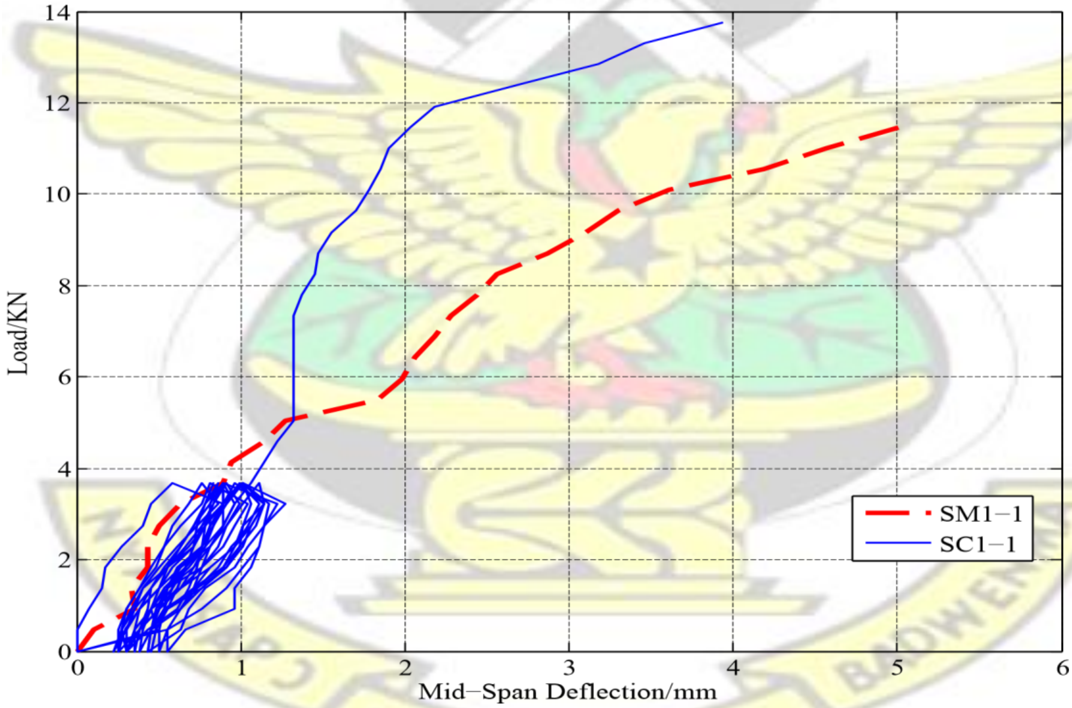

Figure 4.7. Monotonic and cyclic loading curves for beams with 1% fibre content

4.5.4 20 mm Nominal Beams Under Monotonic and Cyclic Loading

For the 20 mm nominal coarse aggregate beams, with the exception of beams with 0.5% and 0.75% fibre content, all other beams behaved in a contrasting manner as is found from previous works. There was the enveloping effect which shows how the stiffness, strength and deformation of cyclically loaded beams are reduced as a result of limited hysteretic energy dissipation at service load. This phenomenon was observed by Adom Asamoah *et al.*, (2009) where similar slabs under monotonic and cyclic loading (8,15 and 50 cycles) were investigated as is in current study with beams with 0.5 % and 0.75 % fibre content. For beams with no fibre (0%), the one under monotonic loading was more ductile than the one under cyclic loading (See Fig. 4.8). Increasing fibre content to 0.25%, the stiffness of both beams under cyclic and monotonic loading were similar until the first crack, after which the stiffness of the monotonic loaded beam was reduced. The monotonic loaded beam was ductile and sustained equal ultimate load as the cyclic loaded beam as seen in Figure 4.9. At 0.5% fibre content, the deflection at the ultimate load of the beam under monotonic loading is the same as the beam under cyclic loading, although, the beam under cyclic loading sustained higher ultimate load and was much ductile (See Fig. 4.10). At 0.75% fibre content, the initial stiffness of both beams under monotonic and cyclic loading were similar until load point 10 kN after which the stiffness under monotonic loading was reduced. After the loading and unloading cycles, the cyclic load – deflection curve fell beneath the monotonic load – deflection curve with stiffness deterioration as shown in Figure 4.11. Finally, at 1% fibre content the cyclic loaded beam was much stiffer and sustained a higher ultimate load than the beam under monotonic loading. (See Fig. 4.12)

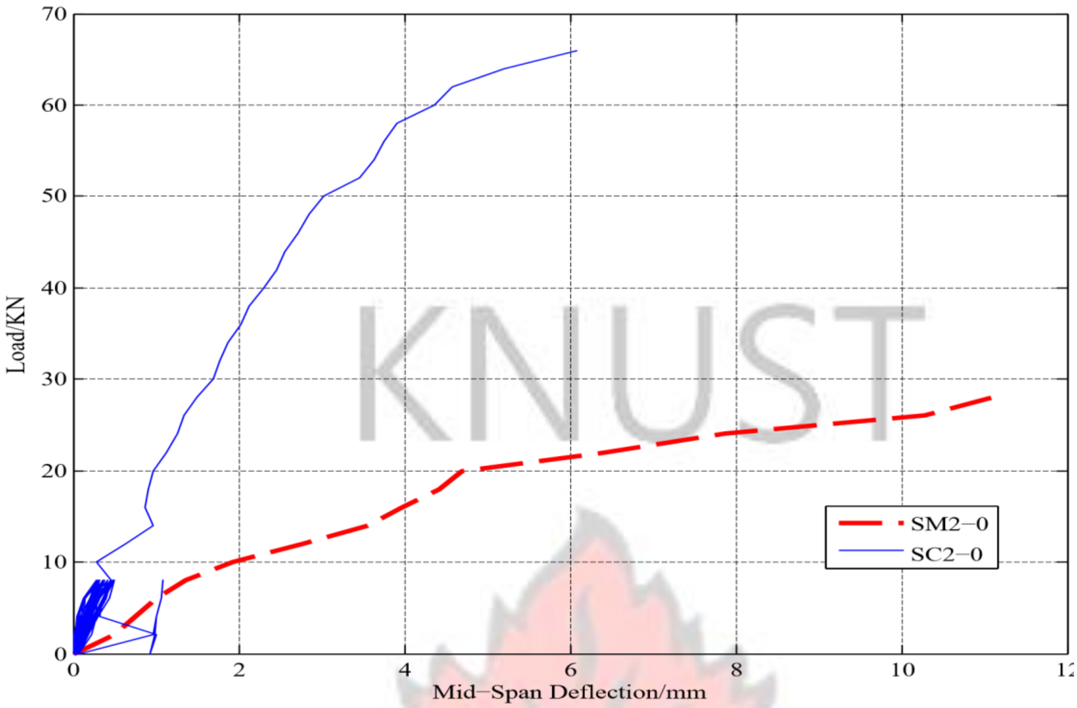

Figure 4.8. Monotonic and cyclic loading curves for beams with 0% fibre content

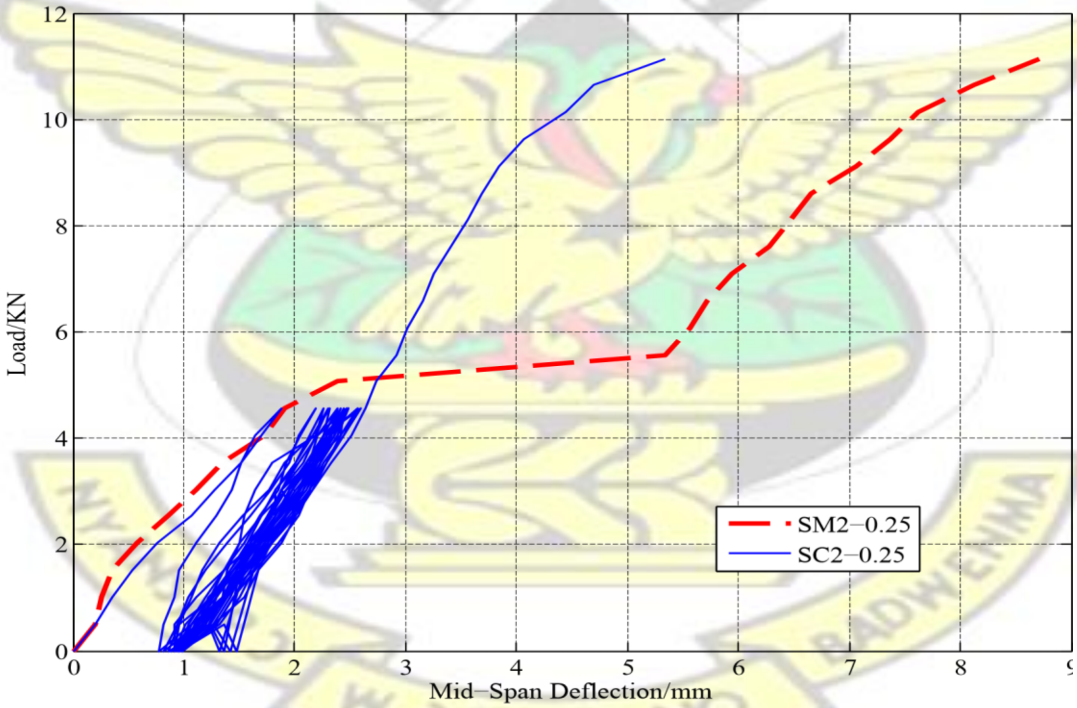

Figure 4.9. Monotonic and cyclic loading curves for beams with 0.25% fibre content

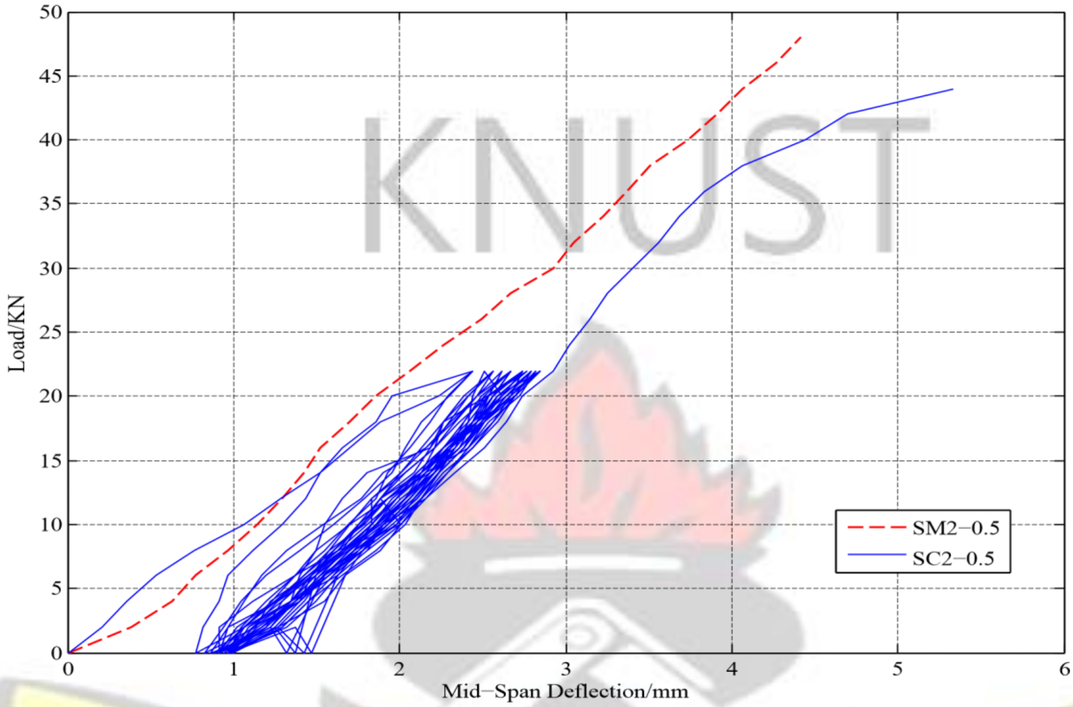

Figure 4.10. Monotonic and cyclic loading curves for beams with 0.5% fibre content

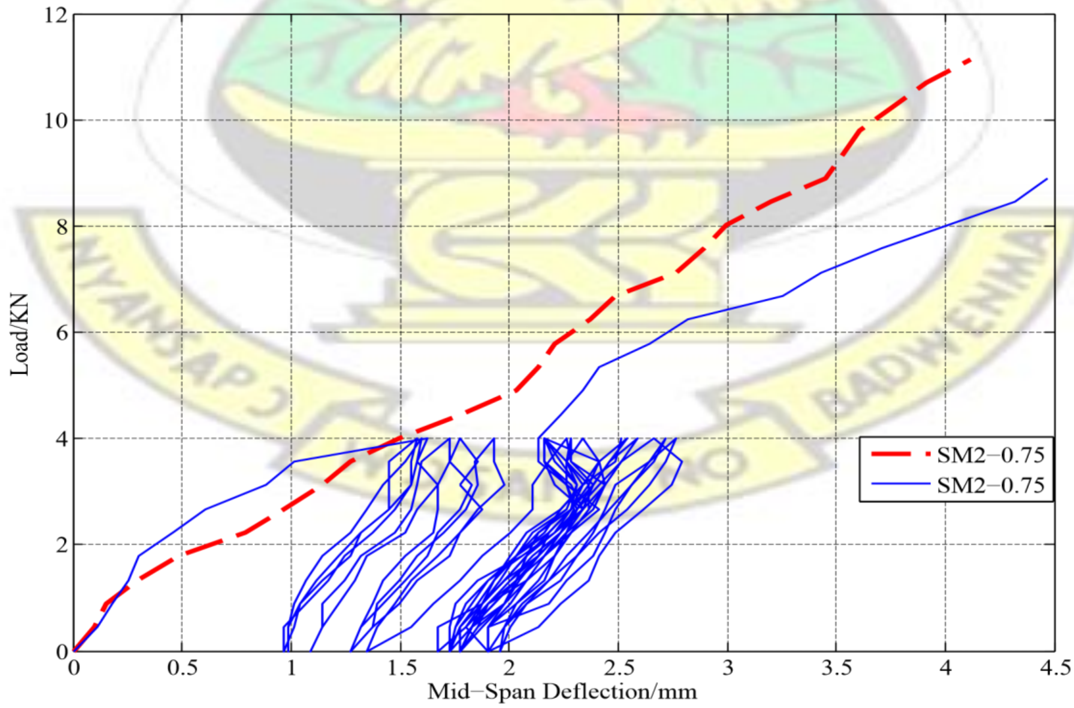

Figure 4.11. Monotonic and cyclic loading curves for beams with 0.75% fibre content

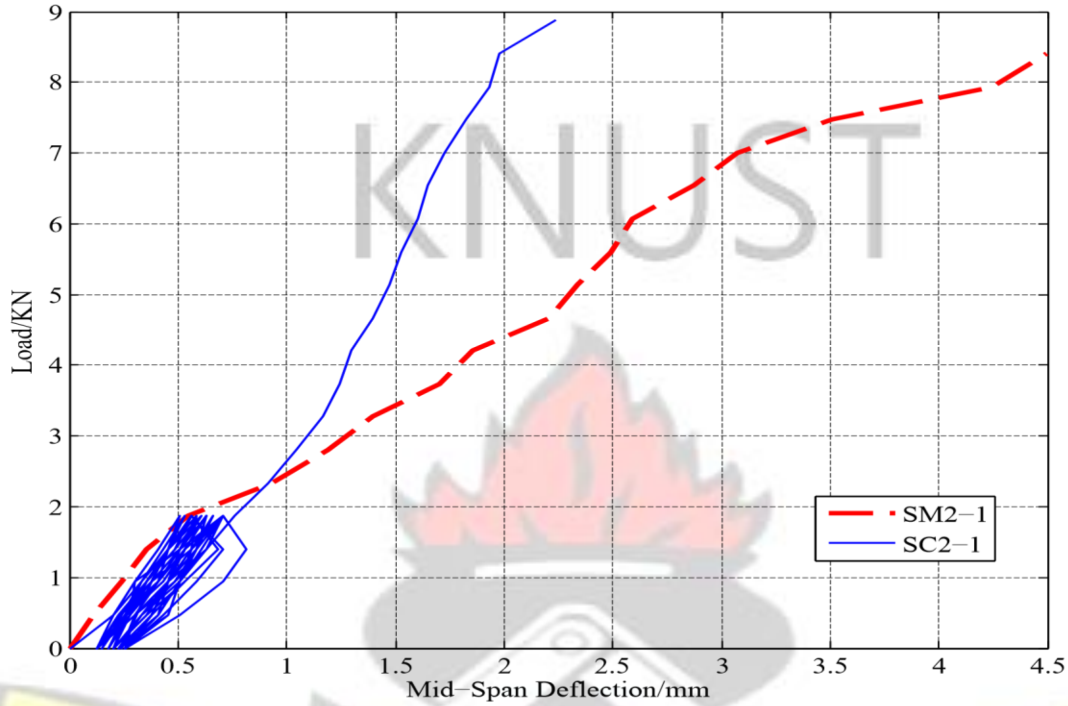

Figure 4.12. Monotonic and cyclic loading curves for beams with 1% fibre content

4.6 Cracking Behaviour and Failure Modes

All the beams were without stirrups and expected to fail in shear. Contributions to the failure mode of reinforced concrete are by ratio of the longitudinal and shear reinforcements, concrete compressive strength and the shear-span-to-depth ratio. In the current work, all these parameters were kept constant with the exception of the concrete compressive strength. As for the shear-span-to-depth ratio for the beams without stirrups, flexural capacity is not reached if the value of the ratio is between 1 and 6 and shear failure mode occurs. At a constant shear-span-to-depth ratio of 2.5, all the beams exhibited the shear failure mode as expected with the exception of one beam.

For a simply supported beam subjected to loading, the cracks are initiated in the middle third span of the beam under application of load as flexural cracks perpendicular to the neutral axis of the beam are formed. With increase in loads, cracks appear in the shear span as extension of flexural cracks and propagate diagonally to the neutral axis as "flexural shear cracks" or cracks which form diagonally independent of the flexural cracks as "web-shear cracks". As the load is further increased, failure occurs as shear cracks extend diagonally to the loading points followed by crushing of the beam at the supports in some cases (See Fig. 4.13).

Figure 4.13. Shear Failure of Beam

The configuration of the cracks after failure of the beam is shown in Figure 4.14 for 10 mm nominal beams under monotonic loading, Figure 4.15 for 20 mm nominal beams under monotonic loading, Figure 4.16 for 10 mm nominal beams under cyclic loading and Figure 4.17 for 20 mm nominal beams under cyclic loading.

SM1 – 0

SM1 – 0.25

SM1 – 0.5

SM1 – 0.75

SM1 – 1

Figure 4.14. Crack patterns of the 10 mm nominal coarse aggregate beams at failure under monotonic loading

SM2 – 0

– 0.25

SM2

SM2 – 0.5

SM2 – 0.75

SM2 – 1

Figure 4.15. Crack patterns of the 20 mm nominal coarse aggregate beams at failure under monotonic loading

– 0.25

SC1 – 0

SC1

SC1 – 0.5

SC1 – 0.75

– 0.25

SC1 – 1

Figure 4.16. Crack patterns of the 10 mm nominal coarse aggregate beams at failure under cyclic loading

SC2 – 0

SC2

– 0.25

SC2 – 0.5

SC2 – 0.75

SC2 – 1

Figure 4.17. Crack patterns of the 20 mm nominal coarse aggregate beams at failure under cyclic loading

The experimental and theoretical cracking loads are given in Table 4.3. The theoretical failure load predicted shear failure mode per equations provided in BS 8110 without consideration of the fibre addition. From the theoretically calculated loads the least load governed the failure mode. All 10 mm maximum coarse aggregate beams under monotonic and cyclic loading conditions failed in shear at loads exceeding the theoretical calculated shear loads. Beams that failed at experimental loads greater than the flexural failure loads did not fail in flexure as expected but in shear. For the 20 mm maximum coarse aggregate beams on the other hand with the exception of SC2-0.75

which failed in crushing at the support, all the beams failed in shear as expected. The failure modes for all the beams are presented in Table 4.4.

Table 4.4 Failure mode and crack characteristics of beams

BEAM	Actual Failure	Number of Cracks	Maximum Crack Width (mm)	Total Crack Length (mm)
SM1-0	Shear/Bond	19	8.00	3312.00
SM1- 0.25	Shear/Bond/Crushing	14	4.00	3046.00
SM1-0.5	Shear/Bond	21	3.00	2604.00
SM1-0.75	Shear/Bond/Anchorage	19	2.50	3233.00
SM1-1	Shear/Bond/Anchorage	8	2.00	2519.00
SC1- 0	Shear/Bond/Crushing	18	5.00	2750.00
SC1- 0.25	Shear/Bond	11	6.00	2995.00
SC1-0.5	Shear/Bond/Crushing	13	8.00	1702.00
SC1-0.75	Shear/Bond	18	4.00	2561.00
SC1-1	Shear/Bond	12	5.00	1348.00
SM2-0	Shear/Bond/Anchorage	8	10.00	1745.00
SM2-0.25	Shear/Bond	16	6.00	1905.00
SM2-0.5	Shear/Bond	9	10.00	1696.00
SM2- 0.75	Shear/Bond	11	4.00	2808.00
SM2- 1	Shear/Bond	9	4.00	1667.00
SC2-0	Shear/Bond	12	6.00	1462.00
SC2-0.25	Shear/Bond	14	5.00	2213.00
SC2- 0.5	Shear/Bond	26	1.00	3608.00
SC2-0.75	Crushing at support	8	0.67	746.00

SC2- 1	Shear/Bond	10	8.00	2071.00

The failure modes of respective beams are detailed and discussed as follows. All the beams at failure were accompanied by at least one diagonal crack which widened with increase in load (Al- Lami, 2015). In all the beams, flexural cracks were formed in the mid-span.

The diagonal cracks formed propagated to the topmost fibre of the concrete to the load points and developed horizontally along the longitudinal reinforcement indicating bond failure in the beams SM1 – 0, SM1 – 0.25, SM1 – 1, SM2 – 0, SC1 – 0, SC1 – 0.25, SC1 – 0.75, and SC1 – 1. Although there was crushing of the concrete at the load points only in SM1 – 0.25, SC1 – 0, and SC1 – 0.25 beams. Also the bond failure was accompanied by anchorage failure with the crushing of the concrete at the support in SM1 – 1 and SM2 – 0. On the other hand, some beams did not have the diagonal cracks propagating to the topmost fibre of the concrete but horizontally along the longitudinal steel reinforcement which indicted bond failure. The beams which failed that way are; SM1 – 0.5, SM1 – 0.75, SM2 – 0.25, SM2 – 5, SM2 – 0.75, SM2 – 1, SC2 – 0, SC2 – .0.25 and SC2 – 0.5. The bond failure was accompanied by anchorage failure with crushing of concrete at the support only in SM1 – 0.75. In SC2 – 0.75, there was no formation of the diagonal cracks, hence, no shear failure mode as predicted but the concrete crushed at the support at failure. Bond failure occurs in beams with shear-span-to-depth-ratios greater than 2, as was the case with the current study, failure occurred in all of the beams.

4.6.1 Crack Width

The cracking behaviour of the beams in terms of crack width at service load is presented in Table 4.5 for beams under monotonic loading and under cyclic loading.

Figure 4.18 and Figure 4.19 show plots of the crack width at different fibre contents at service load under monotonic and cyclic loading, respectively.

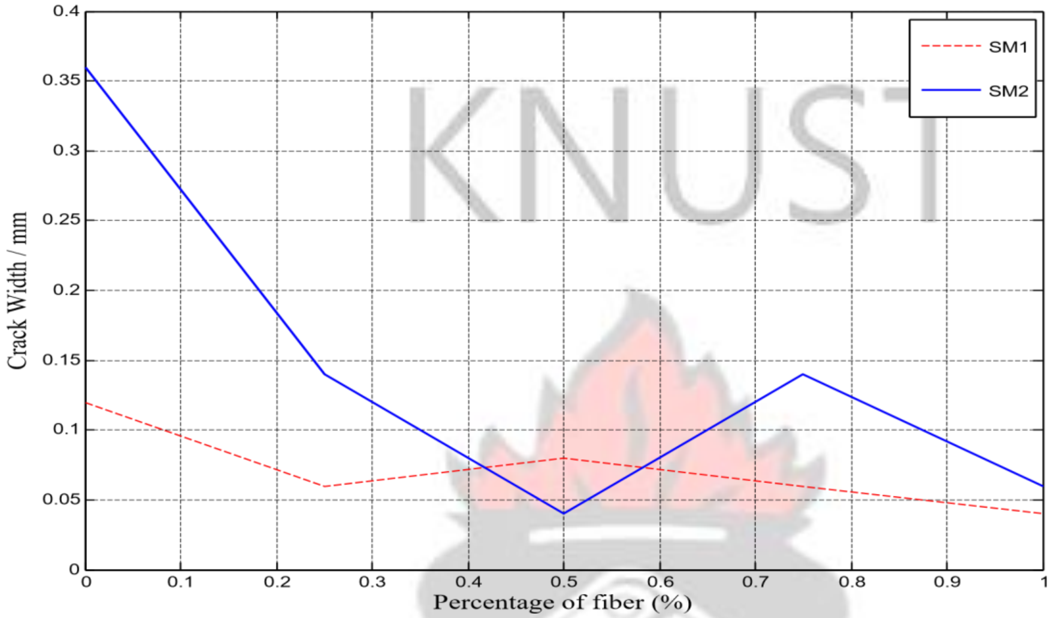

Figure 4.18 . Crack width of beams under service load at different fibre contents under monotonic loading

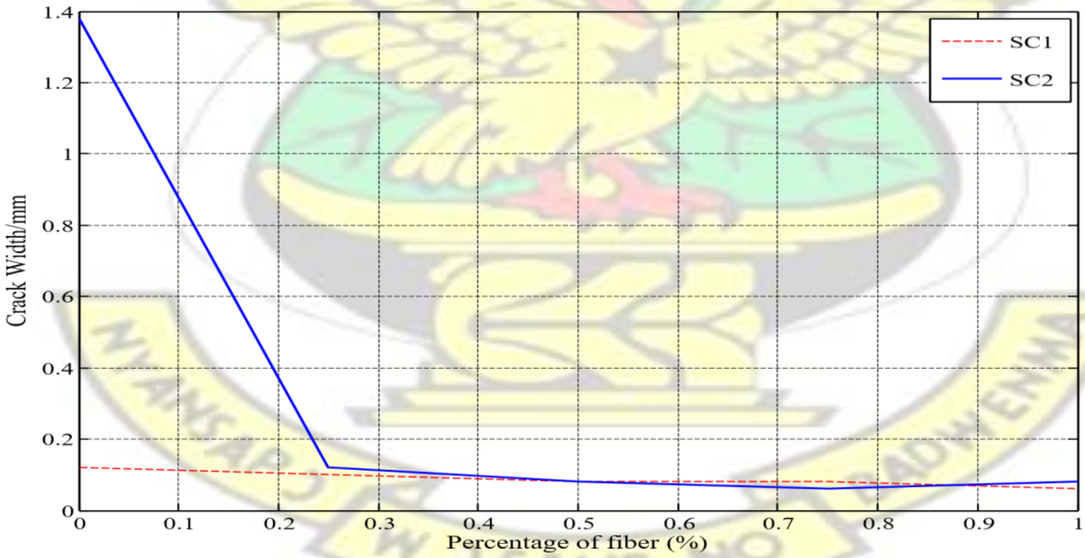

Figure 4.19. Crack width of beams under service load at different fibre contents under cyclic loading

The maximum crack width under service load conditions should not exceed 0.3 mm according to BS 8110. The crack widths of the reinforced beams at service load are shown in Table 4.5, for the 10 mm and 20 mm nominal coarse aggregate beams with 0% to 1% fibre content. The highest crack width at service load occurred at 0% fibre content in all the beams regardless of the type of loading or size of nominal coarse aggregate (See Fig. 4.18 and Fig. 4.19). The fibre reinforcement was more effective in the 10 mm nominal coarse aggregate beams as none of the beams at service load had crack width greater than 0.3 mm. In the case of the 20 mm nominal coarse aggregate beams, the crack width under service load ranged between 0.36 mm and 1.38 mm for monotonic and cyclic loading, respectively which is more than allowed for serviceability. At the different fibre contents, it is observed that the fibre was more effective in the 10 mm nominal coarse aggregate beams than in the 20 mm nominal coarse aggregate beams. The 10 mm nominal coarse aggregate beams at different fibre content averagely had smaller crack width than the 20 mm nominal coarse aggregate beams. The improvement in crack control with the addition of fibre was more pronounced in the 10 mm nominal coarse aggregate beams. The reinforcement also resulted in higher ultimate loads in the 10 mm nominal coarse aggregate beams than in the 20 mm nominal coarse aggregate beams.

Table 4.5. Normalized Shear and Service Loads and Crack width at Service load

BEAM IDENTITY	Normalized Shear Load (KN)	Service Load (KN)	Crack Width at Service Load (mm)
SM1-0	16.59	45.33	0.12
SM1- 0.25	15.60	44.00	0.06
SM1-0.5	12.55	37.33	0.08
SM1-0.75	14.99	46.67	0.06
SM1-1	11.46	33.33	0.04
SC1- 0	13.66	37.33	0.12
SC1- 0.25	14.65	40	0.1
SC1-0.5	13.00	38.67	0.08
SC1-0.75	15.42	48	0.08
SC1-1	14.22	41.33	0.06
SM2-0	7.28	18.67	0.36
SM2-0.25	11.15	29.33	0.14
SM2-0.5	11.08	32.00	0.04
SM2- 0.75	11.13	33.33	0.14
SM2- 1	8.41	24.00	0.06
SC2-0	17.16	44.00	1.38
SC2-0.25	11.15	29.33	0.12
SC2- 0.5	12.47	36.00	0.08
SC2-0.75	8.91	26.67	0.06
SC2- 1	8.87	25.33	0.08

4.7 Ultimate Shear Strength

Besides variation in the amount of fibre and the size of the nominal coarse aggregates, all other characteristics of the beams were kept constant. The resulting compressive strength for all the 10 concrete mixes produced 10 different compressive strengths.

Therefore, the shear strengths were normalized with the compressive strength using Equation (4.5) and presented already in Table 4.5.

$$V_{norm} = V_u / {f_c}^{1/2} \quad (4.5)$$

where V_{norm} is the normalized shear load, V_u the ultimate shear load and f_c the compressive strength. From the normalized shear load, it was found that the shear strength values of the 10 mm nominal coarse aggregate beams were much higher than those of the 20 mm nominal coarse aggregate beams at different fibre content, with the exception of the beam with no fibre. In previous work by Lachemi *et al.*, (2005), the larger sized coarse aggregate beams had higher ultimate shear stress than the smaller sized coarse aggregate beams. This was due to more aggregate interlock at more coarse aggregate content in the larger sized aggregate beams. In the current work on the other hand, the coarse aggregate content was the same for all the 10 mm and 20 mm nominal coarse aggregate beams. However, the 10 mm nominal coarse aggregate beams, parked more coarse aggregates hence the improved aggregate interlock resulting in higher ultimate shear loads.

4.7.1 Reserve Strength

The reserve strength measures the ability of the beams to sustain more loads after the occurrence of the shear cracks is shown in Table 4.3. The reserve shear strength was calculated as the ratio of ultimate shear load to the first shear crack load as was done by Safan (2012). Typical reserve strength values obtained by the researcher ranged from 1.0 to 1.5. The average reserve strength for 10 mm and 20 mm nominal coarse aggregate beams was 1.15 and 1.12 respectively. These values were within the values obtained by Safan (2012). The reserve strength for the 10 mm nominal coarse aggregate beams was 2.6% lower than that of the 20 mm nominal coarse aggregate beams. For the 10 mm nominal coarse aggregate beams, the highest reserve strength was 1.32 at 1% fibre content under monotonic loading (SM1-1) and 1.2 at 0.75% fibre content under cyclic loading (SC1-0.75). In case of the 20 mm nominal coarse aggregate beams, the highest reserve strength of 1.25 was obtained for the sample with 0.75% fibre content under monotonic loading and 1.12 for the sample with 1% fibre content under cyclic loading. This shows that increasing the fibre content improves the cracking behaviour and lead to higher load carrying capacity at post–cracking stage. Similar improvement in reserve strength was observed by Shah *et al.*, (2010) in SCC beams.

4.8 Experimental and Predicted Shear Capacities from different Codes

The theoretical shear strength from AC1 318 – 11, BS 8110 (1997), and EC 2 (2004) were compared to the experimental ultimate shear loads. The detailed equations from the respective codes have been presented in Equation (2.1a) for AC1 318 – 11, Equation (4.3) for BS 8110 (1997), and Equation (2.4a) for EC 2 (2004). The equations for concrete contribution alone is considered since the experiment was done on beams without stirrups. Table 4.6 shows the experimental shear capacities and code predictions. A conservative approach is taken when values predicted are lower than what actually occurs. The shear strength ratios (Vexp/Vcode) for the beams varied from 0.78 to 2.17. All the codes underestimated the experimental ultimate shear loads. With EC 2 being the most conservative. The least conservative code was the BS 8110 for the 20 mm nominal coarse aggregate beams and ACI 318 – 11 for the 10 mm nominal coarse aggregate beams. Using the same code provisions as in the current work, Acheampong *et al.*, (2015) found that the shear load predicted by EC2 closely matched the experimental shear loads, while the ACI 318 – 11 and BS 8110 underestimated the experimental shear loads for palm kernel shell and normal weight concrete**.** This is in contrast to the present work in which ACI 318 – 11 and the BS 8110 were the better predictors of the experimental shear loads for 10 mm and 20 mm nominal coarse aggregate beams respectively.

Table 4.6 Code and Experimental Shear Capacities

BEAM IDENTITY	*V*exp (KN)	Predicted Shear force (KN) *V*code			Shear Strength Ratios, *V*exp/*V*code		
		ACI	**BS 8110**	**EC**	**ACI**	**BS 8110**	**EC**
SM1-0	68	37.5	36.11	31.33	1.81	1.88	2.17
SM1-0.25	66	38.4	36.89	32	1.72	1.79	2.06
SM1-0.5	56	39.96	38.21	33.15	1.40	1.47	1.69
SM1-0.75	70	41.35	39.38	34.17	1.69	1.78	2.05
SM1-1	50	39.27	37.62	32.65	1.27	1.33	1.53
SC1-0	56	37.5	36.11	31.33	1.49	1.55	1.79
SC1-0.25	60	38.4	36.89	32	1.56	1.63	1.88
SC1-0.5	58	39.96	38.21	33.15	1.45	1.52	1.75
SC1-0.75	72	41.35	39.38	34.17	1.74	1.83	2.11
SC1-1	62	39.27	37.62	32.65	1.58	1.65	1.90
Average					1.57	1.64	1.89
SM2-0	28	35.81	34.61	30.03	0.78	1.03	0.93
SM2-0.25	44	36.48	35.21	30.55	1.21	1.04	1.44
SM2-0.5	48	39.07	37.46	32.5	1.23	1.04	1.48

SM2-0.75	50	40.15	38.38	33.3	1.25	1.05	1.50
SM2-1	36	38.74	37.18	32.3	0.93	1.04	1.11
SC2-0	66	35.81	34.61	30.03	1.84	1.03	2.20
SC2-0.25	44	36.48	35.21	30.55	1.21	1.04	1.44
SC2-0.5	54	39.07	37.46	32.5	1.38	1.04	1.66

Table 4.6. Continued

BEAM IDENTITY	*V*exp (KN)	Predicted Shear force (KN) *V*code			Shear Strength Ratios, *V*exp/*V*code		
		ACI	**BS 8110**	**EC**	**ACI**	**BS 8110**	**EC**
SC2-0.75	40	40.15	38.38	33.3	1.00	1.05	1.20
SC2-1	38	38.74	37.18	32.3	0.98	1.04	1.18
Average					1.18	1.04	1.41

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

The objective of this research was to determine the effect of fibre on the cracking behaviour of self compacting concrete and the effect of the different nominal coarse aggregate size on the shear behaviour of self compacting concrete. The variables considered were the fibre at different contents of 0%, 0.25%, 0.5%, 0.75% and 1% and nominal coarse aggregate sizes of 10 mm and 20 mm. Also the beams were tested under monotonic and cyclic loading. Based on the results obtained from the study the following conclusions may be made:

- The increase in nominal coarse aggregate size from 10 mm to 20 mm caused an average increase of ultimate failure load of SCC beams by 46%. There is a 12% reduction in ultimate failure load when beams are subjected to cyclic loading. Consequently, this resulted in an increase in stiffness, with a reduction in the average deflection of about 4%.
- There were contrasting trends of the impact of fibre on ultimate failure loads given the respective nominal coarse aggregate sizes under study. For the 10 mm nominal coarse aggregate beams there was an expected marginal increase which is on the average 1%. However, the variability of impact of fibre inclusion is high. For the 20 mm nominal coarse aggregate beams on the other hand, the average increase in failure load with addition of fibre was 13%. Among the

ranges of fibre contents studied a value of 0.75% would be suitable for a maximum increase in ultimate failure load.

- With the inclusion of fibre, in the 10 mm nominal coarse aggregate beams, the gain in the reserve strength (a proxy for ductility) was averagely 6%. The 20 mm nominal coarse aggregate beams on the other hand recorded an average decrease in reserve strength of 5%.
- Beams with a lower nominal coarse aggregate size (10 mm) is expected to be more ductile than beams with higher nominal coarse aggregate size (20 mm) when the reserve strength is used as an implicit measure of ductility. This can be on the average 3%.
- The enveloping effect of the monotonic loading curve over the cyclic loading curve implies no loss of structural integrity. This implication was observed in beams with 0% and 0.75% fibre content in the 10 mm nominal coarse aggregate beams, and 0.5% and 0.75% fibre content in the 20 mm nominal coarse aggregate beams.
- The BS 8110 design guidelines should be used for predicting the shear strength of SCC beams and applying a multiplicative factor of 1.34 to meet observed experimental ultimate loads.

The following recommendations may be considered for future work;

- Further studies should incorporate the effect of the shear span to depth ratio on the shear strength of SCC beams.
- Further studies should aim at establishing empirical shear strength models specifically for SCC beams since estimates from current code provisions for shear capacity may be inadequate.

- Palm Kernel fibre is waste material from oil palm production. From the study, the inclusion of fibre in reinforced concrete components (beams) was found to improve the cracking behaviour and shear strength. Hence, it is recommended that storage facilities should be provided to house palm kernel fibre and subsequently delivered to concrete production industries for use.

- Due to the limitation on the period for the research, the durability of the fibre on the concrete was not accounted for. Therefore, further long term studies should investigate on the durability of fibre on the concrete.
- Further studies should include stirrups and investigate the effect of replacing stirrups with fibre on the shear strength.

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