SHEAR STRENGTH OF FIBER REINFORCED RECYCLED AGGREGATE CONCRETE

by

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# Approval Signatures

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**Thesis Title:** Shear Strength of Fiber Reinforced Recycled Aggregate Concrete

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Abstract

In this research, the shear strength of fiber reinforced recycled concrete was investigated. A High-strength Self Consolidated Concrete (SCC) matrix with 100% coarse recycled aggregate and different types/configurations of fibers were used in the study. Steel (3D and 5D), synthetic and hybrid fibers (mix of steel 5D and synthetic fibers) with a volume fraction of 0.75% were added to the concrete matrix to prepare eight beams. In addition, four beams were prepared without fibers as control specimens. The aim of the experimental program is to evaluate the effect of: 1) recycled coarse-aggregate replacement; 2) addition of fibers and 3) the steel fiber configuration on the shear strength of recycled aggregate concrete. The results show that recycled aggregate concrete resulted in an improvement in the average concrete shear strength of about 14.4% compared to that of the normal weight aggregate concrete. In addition, the fiber-reinforced beams showed significant improvement in the average concrete shear capacity in the range of 23.44 – 64.48% when compared to that of the control specimen. The highest improvement was achieved by the 3D steel fiber beams. The addition of the fiber delayed the crack initiation, and improved the post-cracking and ductile behavior of all beams. Moreover, the experimental results were compared to that predicted by codes and proposed equations found in the literature for concrete strength with and without fiber. It was found that the ACI simplified equation predicts the closest results for both types of aggregates; while the fib model code 2010 equation predicted the most conservative results.

Keywords: Concrete shear strength, Recycled aggregate, Steel fiber, Synthetic fiber, Hybrid fiber
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Chapter 1. Introduction

In this chapter, background overview on the subject will be included; objectives and problem statement of the study will be detailed. In addition, the organization of thesis is also provided.

1.1. Overview

Green concrete or sustainable concrete has become the dominant criteria in construction industry and its research, which can be illustrated in the focus of almost all the research publication. One of the most common types is Recycled Aggregate Concrete, which has been used in the industry for decades. The use of Recycled Aggregate in construction of structural elements still has limitations and engineers are still experiencing doubts on the structural behaviour of the element.

Extensive research has been done on both fresh and hardened properties of recycled aggregate concrete (RAC). However, the researches performed on its structural behaviour are limited and often contradictory. In general, the flexural and shear crack patterns of 100 % replacement of recycled aggregate concrete and conventional normal weight concrete are identical; however, the shear capacity in recycled aggregate concrete is relatively less than that of conventional concrete [1 - 9].

Moreover, factors affecting the shear capacity are: effect of the un-cracked compressive concrete zone, aggregate interlock, dowel action, longitudinal and shear reinforcement, span-to-depth ratio, and compressive strength [1 – 3, 10 - 14] According to Peter and Balazs (2017), fiber reinforcement is considered as a type of shear reinforcement. Further fiber reinforcement can be used to improve the shear capacity of the concrete, and will resist the brittleness shear failure, and give a more ductile behaviour including post-cracking tensile strength. In this study, steel and synthetic fibers will be used to improve the mechanical properties of concrete prepared with 100% recycled coarse aggregate. A volumetric replacement of 0.75% of the total matrix will be used in the investigation. Two types of steel fibers (3D and 5D), synthetic and hybrid (mix of synthetic and 5D steel fiber) will be used to prepare eight beams. In addition, four beams without fiber will be prepared as control. All beams will be prepared without web reinforcement to investigate the effect of fiber addition on improving concrete contribution to shear capacity.
1.2. Research Objectives

Variability of recycled aggregate properties leads to limitations of its use in the construction industry. Introducing steel, synthetic and/or both fibers to the concrete matrix might enhance and improve the concrete properties to overcome some of these limitations. Therefore, the main objective of the study is to investigate the effect of different types of fibers on the shear capacity of beams prepared with 100% recycled coarse aggregate. To achieve this goal, the proposed experimental program consists of 12 beams to be cast and tested in four-point loading test setup. Four beams without fibers will be used as control, two of which will contain conventional normal weight coarse aggregate, and two with 100% replacement of recycled coarse aggregate. Remaining mixes utilizing 100% recycled aggregate will be divided as follows: two beams using 3D steel fiber, two beams using 5D steel fiber, two beams using Polypropylene fiber, and the last two beams will have hybrid fiber content of 5D and Polypropylene. The same volumetric ratio 0.75% of the fiber will be used for all mixes.

1.3. Significance of the Study

The literature covered in this study does not include the shear behaviour of the 100% recycled coarse aggregate replacement concrete which could be a challenge in its use in a structure. Hence, this study focuses on the shear behaviour of the Recycled Aggregate concrete and how to improve its resistance. One solution is to increase the shear capacity by adding fibers to the concrete mixture.

From Table 1.1, Mix #1 will be having normal weight aggregate which will act as a control to the shear capacity performance expected and used in structural design and construction industry. Mix #2 will be the baseline to evaluate the added advantage of the fibers (Steel or Synthetic or Hybrid) that will be obtained from Mixes 3 to 6. All designed mixes will be made as a SCC concrete with high compressive strength design (40 MPa or higher), trying to achieve compressive strength is possible by the addition of supplementary materials such as Micro-Silica, which has an added advantage to Green concrete.

Results will be evaluated for 6 different mixes, 2 beams each, which yields to 12 beams total divided as follows:
<table>
<thead>
<tr>
<th>Mix #</th>
<th>Code</th>
<th>Type of Aggregate</th>
<th>Fiber Content</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>NWA</td>
<td>Normal Weight Aggregate</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>RCA</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>RCA-3D</td>
<td>Recycled Aggregate</td>
<td>3D</td>
</tr>
<tr>
<td>4</td>
<td>RCA-5D</td>
<td></td>
<td>5D</td>
</tr>
<tr>
<td>5</td>
<td>RCA-SY</td>
<td></td>
<td>Synthetic</td>
</tr>
<tr>
<td>6</td>
<td>RCA-HY</td>
<td></td>
<td>50% 5D + 50% Synthetic</td>
</tr>
</tbody>
</table>

Concluding, the study will focus mainly on the added advantage to shear excepted from 3 different fibers types (3D, 5D Steel Fibers, Synthetic Fiber) and their comparison with each other and with Hybrid addition of fibers (5D + Synthetic); and since to date, literature did not show any study with the same focus of shear strength, only other mechanical properties will be compared to others from literature. All results will be analysed and concluded accordingly.

1.4. Thesis Organization

This thesis is divided into 6 chapters. Chapter 1 introduces the material to be studied, defining the problem, and its objectives. Chapter 2 will show a brief background on recycled aggregate and its behaviour, shear properties and presents codes and proposed equations used to predict shear capacity in normal weight concrete, recycled aggregate concrete, and fiber reinforced concrete. Chapter 3 details the methodology to be followed in this study, its experimental program and test setup. Chapter 4 presents the results and observation outcomes from the experiments conducted in the study. Chapter 5 will thoroughly discuss the results obtained from the experimental stage and its explanations, in addition, the experimental results will be compared to code predictions and proposed equations from literature.
Chapter 2. Background and Literature Review

This chapter will provide a brief history on the different stages of this study such as recycled aggregates, shear design, and fiber contents from literature and other researchers conducted by in the said area.

2.1 Background

Recent literature shows that up to 30% of coarse recycled aggregate can be used to replace conventional normal weight aggregate while maintaining the physical and mechanical properties of the concrete. However, very limited research that used 100% recycled aggregate concrete focused on the evaluation of the mechanical properties. Kikuchi et al. (1988) conducted the early research on the structural performance of recycled aggregate concrete. He evaluated beams with different longitudinal reinforcement ratios for 50% and 100% replacement recycled aggregate concrete beams to conventional concrete along with three different water-to-binder ratios of 0.30, 0.45, and 0.60. The study concluded that similar crack patterns and failure modes of RCA to conventional virgin concrete; however, a reduction of about 10 – 20 % in the shear capacity of RCA versus the conventional concrete was observed [1 – 2, 5 - 6].

Other research efforts focused on the evaluation of the mechanical properties utilizing low replacement ratios. Studies showed that up to 50% replacement of coarse recycled aggregate would have no significant effect on concrete properties [1 - 13]. The shear strength was evaluated for the recycled aggregate with different percentages, and it was noted that 50% replacement of coarse recycled did not have any significant effect in the shear strength, however, 100% replacement resulted in reduction in the shear strength [1 – 7].

Moreover, several studies discussed the benefits of fiber addition to concrete mixture as shear reinforcement, which showed that shear capacity, and ductility of concrete would be increased. Both steel and synthetic fibers could be used as shear reinforcement for concrete; however, shear strength with fibers reinforcement were tested only with conventional normal weight concrete but not with recycled aggregate concrete. Literature showed that fiber contribution to shear could be calculated conservatively using proposed equations [14-18]. Further, few papers have concluded that fibers can replace the shear reinforcement but not the longitudinal bars [14, 19].
In this study, shear capacity of concrete prepared with 100% recycled aggregate will be evaluated using beams without web reinforcement. In addition, different types of fibers will be used to improve the shear capacity. A brief discussion about recycled aggregate and its properties, effect of fibers on mechanical properties and shear equations in different codes will be presented in the next sections.

2.2 Recycled Aggregate Concrete

The main source of Recycled Aggregates are from demolished buildings, hence its quality is dependent on the quality of the material being collected and delivered to the recycling plant, and due to its related costs, stability of production and variability of aggregates is not maintained [20 - 22]. For example, according to Mohsen et al., the recycled aggregate used in his research is divided in to: (43.2% crushed concrete, 28.6% façade stones, 14.2% crushed ceramic and 14% crushed bricks) [13]. On the other hand, Kutalmis et al. obtained Recycled aggregate with (97.53% Concrete Particles, 1.74% Brick and Tile particles, 0.2% Gypsum Particles, 0.17% Shell Particles, 0.13% Stucco Particles, 0.13% River aggregates, 0.1% Glazed tile) [12]. Therefore, evaluation of the physical and mechanical properties of recycled aggregate need to be evaluated before use to account for variability of the aggregate properties.

2.3 Effect of Fibers on NWA and RCA

The introduction of fibers to the concrete mixture has inconsiderable effect to the compressive strength of the concrete [13]. Fibers increase the porosity of the concrete mixture, which causes voids and hence might reduce the compressive strength; however, in other researches compressive strength have increased when fibers were added to the mixture. On the other hand, fibers addition have increased the splitting tensile, flexural tensile, and shear strengths [12 – 14, 21, 22]. Most researchers have agreed to the fact that fibers introduction to the mix will benefit the shear, and tensile strengths, however, the downside to shear addition is durability, and in some cases compressive strength.

In general, fiber content should range between 0.1% - 3.0 % [6]; however the recommended replacement percentages are 0.25 % - 1.5 % from the volume. According to Ehsan et al., the compressive strength of a 20% replacement of RA had no significant effect with fiber contents of 0.25% and 0.5% regardless of the fiber being used. However, on the same 20% RA replacement, the compressive strength has increased
by 4% – 5% only than the control mix of 0% replacement. On the other hand, fibers addition had a huge impact to the tensile strength, in which, tensile strength increased 19%, 40%, and 64% while having Polypropylene fiber contents 0.25%, 0.5%, and 1.0% respectively, and tensile strength increased by 44%, 66%, and 80% having Steel Fibers contents 0.5%, 0.75%, and 1.0% respectively, but hybrid fiber contents had showed an increase in the compressive strength recorded by a decrease in the tensile strength, Tables 3 and 4 shows the summary of the results. Moreover, Mohsen et al., provided a comparison between the NWA and 50 % and 100 % replacement of RCA; the results showed varying numbers in compressive strength but an increase in tensile strength, Table 1 illustrates the results of both the compressive and tensile strengths [13]. Furthermore, Kutalmis et al. have used fiber replacement of 0%, 1%, and 1.5% in all their mixes which had RA replacements of 0%, 25%, 30%, and 55%; however, 100% replacement was not used. The percentages used along with the fibers have showed a reduction in the compressive strength but an increase in both splitting and flexural tensile strengths [12]. Lastly, Yoon et al. have studied the effect of fibers on shear strength on NWA; percentages used in their study were 0%, 0.5%, and 0.75%. It was noted that with the increase of fiber content, the compressive and shear strengths were increased. Studied beams with 0% fibers have failed in shear, however, when the fiber content increases, the failure modes shifts to either shear-flexure or only flexure [14]. Figure 3 illustrates the effect of fibers towards failure modes in concrete; hence, proving the fact that the presence of fibers in concrete improves its structural behaviour [23]. Finally, researchers agreed that 0% to 0.5% volumetric replacement of fibers in concrete have no significant effect on the mechanical properties; however, 0.5% to 1% fiber replacement shows significant improvements in mechanical properties; moreover, the mechanical properties improvement trend become flatter when the fiber content is above 1% [11 – 13, 25 – 27, 49].

2.4 Shear in Recycled Aggregate Concrete

In literature, Recycled Aggregate Concrete is tested for its fresh and hardened concrete; however, relatively low researches have been done to the structural behaviour. Mahdi et al. have tested the shear capacity of NWA and RCA containing 50% and 100% replacement from virgin aggregate concrete. It was found that there is no significant difference between NWA and 50% RCA replacement, however, 100% replacement of
RCA had 11% reduction in the shear capacity, which, according to the author, due to the two ITZs available, one from the old mortar (between virgin aggregate and old/residual mortar) and the new ITZ (between old/residual mortar and the new mortar) [1 - 2]. Shear region for concrete beams without web reinforcement are noted in Figure 1 with the meshed region, Ababneh et al. [19] used synthetic fibers in concrete beams without stirrups and noted the failure modes in Figure 2; these patterns are compatible to all researchers beams used. Moreover, Mahdi et al. confirmed that the crack shear failure patterns for all beams are somewhat similar, in which, NWA, RCA 50%, and RCA 100% have the similar failure pattern [1 - 2]. Khaldoun et al. completed a shear-friction behaviour study for mixes with 0%, 50%, and 100% replacement of RCA; and it was noted that the full replacement of RCA (100% RCA) had limited effect to the ultimate and post-ultimate shear strength, however, partial replacement (50% RCA) have had a decrease in the ultimate and post-ultimate shear strength [3]. Further, Christos et al. have concluded that shear strength of the Normal Rock Aggregate have higher shear resistance than 100% Recycled Aggregate Concrete, and this is due to the reason that high-strength RCA concrete tend to fail in shear through the aggregate plans rather than mortar around the RA [13].

Moreover, Choi et al. have evaluated the effect of different span-to-depth ratios, longitudinal reinforcement, and RCA replacement on the shear capacity. Parameters evaluated were: span-to-depth ratios used ranges between (1.50, 2.50, 3.25), longitudinal reinforcement ratios (0.53, 0.83, and 1.61 %), and recycled aggregate replacement ratios (0, 30, 50, 100 %); results of their study illustrated that the higher the RCA replacement ratio the lower the shear strength [8]. Another study conducted by Gonzalez and Martinez, a fixed longitudinal reinforcement ratio and recycled aggregate replacement of 3% and 50% respectively. 8 beams were tested against deflection and ultimate shear strength; their results showed that there is no significant difference between the recycled aggregate concrete with conventional virgin concrete. The study was repeated by Gonzalez et al.; however, with 8% silica fume was added to the mix, it is noted that splitting cracks along the tension reinforcement were mitigated with the addition of silica fume [7, 8].
Many researchers have concluded that current shear equations from codes and proposed equations for conventional virgin aggregate have predicted the shear strength of the RCA conservatively. Fathifazl et al. tested beams with different span-to-depth ratios ranging between 1.5 and 4, in addition 4 different effective depths (250, 375, 450, 550 mm) to study the its size effect. The results showed that the current code provisions for shear predicted the shear capacities for RCA beams [9]. Further, Ignjatovic et al. have investigated the shear behaviour and strength of simply supported concrete beams with different shear reinforcement ratios (0%, 0.14%, and 0.19%) and different recycled aggregate replacement (50% and 100 %) along with the equations in codes predictions. The study results showed that the shear behaviour and strength of the conventional, and recycled aggregate concretes are similar, in addition, the shear strengths were predicted conservatively by the analysed codes with similar reliability to that of conventional

<table>
<thead>
<tr>
<th>Fiber fraction Vf (%)</th>
<th>Shear-span/depth ratio a/d</th>
<th>Concrete compressive strength $f'_c$</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2</td>
<td>62.6</td>
<td>Shear</td>
</tr>
<tr>
<td>0.5</td>
<td>2</td>
<td>63.8</td>
<td>Shear-flexure</td>
</tr>
<tr>
<td>0.75</td>
<td>2</td>
<td>68.6</td>
<td>Shear-flexure</td>
</tr>
<tr>
<td>0</td>
<td>3</td>
<td>62.6</td>
<td>Shear</td>
</tr>
<tr>
<td>0.5</td>
<td>3</td>
<td>63.8</td>
<td>Flexure</td>
</tr>
<tr>
<td>0.75</td>
<td>3</td>
<td>68.6</td>
<td>Flexure</td>
</tr>
<tr>
<td>0</td>
<td>4</td>
<td>62.6</td>
<td>Shear</td>
</tr>
<tr>
<td>0.5</td>
<td>4</td>
<td>63.8</td>
<td>Flexure</td>
</tr>
<tr>
<td>0.75</td>
<td>4</td>
<td>68.6</td>
<td>Flexure</td>
</tr>
</tbody>
</table>
concrete, the study concluded that the application of the analysed codes provisions for conventional concrete for shear strength can be used for recycled aggregate concrete [10].

2.5 Proposed Equations for Shear Strength

Researchers have developed an equation for shear strength of reinforced concrete without stirrups using the fracture mechanics approach, which according to Mahdi et al. it is applicable to Recycled Aggregate Concrete. Bazant and Yu [17] proposed:

\[ V_c = 10^3 \rho \bar{\sigma} (1 + \frac{d}{a_s}) \sqrt{\frac{f'c}{f'c + \frac{f't}{2}} \frac{1 + 2\sqrt{\frac{f't}{2}}} {\frac{f't}{2}}} } \]

(1)
Table 3: Summary of Papers with RCA and Fibers

<table>
<thead>
<tr>
<th>Authors</th>
<th>RCA %</th>
<th>Fiber</th>
<th>Fibers %</th>
<th>Parameters Tested</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ahmadi et al. (2017) [14]</td>
<td>0%, 50%, 100%</td>
<td>Recycled Steel Fibers</td>
<td>0%, 1%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Mechanical properties</td>
</tr>
<tr>
<td>Afroughsabet et al. (2017) [26]</td>
<td>0%, 50%, 100%</td>
<td>Steel Fibers</td>
<td>0%, 1%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Mechanical properties</td>
</tr>
<tr>
<td>Akca et al. (2015) [13]</td>
<td>0%, 25%, 30%, 55%</td>
<td>Synthetic Fibers</td>
<td>0%, 1%, 1.5%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Tensile strength but reduced compressive strength</td>
</tr>
<tr>
<td>Zhang et al. (2016) [14]</td>
<td>0%, 35%, 50%, 100%</td>
<td>Steel Fibers</td>
<td>0%, 0.5%, 1%, 1.5%, 2%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Mechanical properties</td>
</tr>
<tr>
<td>Mohseni et al. (2017) [23]</td>
<td>0%, 20%</td>
<td>Steel + Synthetic + Hybrid</td>
<td>0.25%, 0.5%, 1%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Tensile strength but reduced compressive strength</td>
</tr>
<tr>
<td>Craneiro et al. (2014)</td>
<td>0%, 25%</td>
<td>Steel Fibers</td>
<td>0%, 0.75%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Mechanical properties</td>
</tr>
<tr>
<td>Gao et al. (2009) [27]</td>
<td>0%, 100%</td>
<td>Steel Fibers</td>
<td>0%, 1%</td>
<td>Mechanical Properties</td>
<td>-Fibers has inconsiderable effect</td>
</tr>
<tr>
<td>Meddah et al. (2013) [28]</td>
<td>0%, 100%</td>
<td>Recycled Steel Fibers</td>
<td>0%, 1%, 1.25%, 1.5%, 2%, 2.5%, 3%</td>
<td>Mechanical Properties</td>
<td>-Fibers Improved Mechanical properties</td>
</tr>
<tr>
<td>Authors</td>
<td>RCA %</td>
<td>Shear Reinforcement</td>
<td>Parameters Tested</td>
<td>Remarks</td>
<td></td>
</tr>
<tr>
<td>--------------------------</td>
<td>----------------------</td>
<td>-------------------------------</td>
<td>---------------------------</td>
<td>-------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>Ivan S. Ignjatovic et al. (2017) [29]</td>
<td>0%, 50%, 100%</td>
<td>Normal shear reinforcement</td>
<td>Beams (Shear)</td>
<td>-Similar results -Codes and equations for NWA can be used for RCA</td>
<td></td>
</tr>
<tr>
<td>Khaldoun R. (2017) [32]</td>
<td>0%, 20%, 50%, 100%</td>
<td>NA</td>
<td>Cylindrical Push-off specimens (Shear)</td>
<td>-As RCA increase, shear strength decrease</td>
<td></td>
</tr>
<tr>
<td>Katkhuda H. et al. (2016) [33]</td>
<td>50%, 100%</td>
<td>NA</td>
<td>Beams (Shear)</td>
<td>-treated RCA have higher shear strength than untreated</td>
<td></td>
</tr>
<tr>
<td>Ceia F. et al. (2016) [34]</td>
<td>0%, 20%, 50%, 100%</td>
<td>NA</td>
<td>Slant Shear test</td>
<td>-RCA have similar properties and strength with NWA</td>
<td></td>
</tr>
<tr>
<td>Waseem S. A. et al. (2016) [35]</td>
<td>0%, 100%</td>
<td>NA</td>
<td>Cylindrical Push-off specimens (Shear)</td>
<td>-RCA has similar effects to NWA</td>
<td></td>
</tr>
<tr>
<td>Tosic N et al. (2016) [36]</td>
<td>100%</td>
<td>With and without Shear reinforcement</td>
<td>Flexural Shear</td>
<td>-Shear without stirrups can be predicted in Euro code 2</td>
<td></td>
</tr>
<tr>
<td>Schubert S. et al. (2012) [37]</td>
<td>0%, 50%, 100%</td>
<td>NA</td>
<td>Slabs (Shear)</td>
<td>-Euro code 2 predicted the shear behavior</td>
<td></td>
</tr>
<tr>
<td>Study</td>
<td>RCA Reinforcement Levels</td>
<td>Reinforcement Type/Location</td>
<td>Comment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------------------</td>
<td>--------------------------</td>
<td>-----------------------------</td>
<td>---------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seara-paz S. et al. (2017) [38]</td>
<td>0%, 20%, 50%, 100%</td>
<td>Normal shear reinforcement</td>
<td>Beams (Flexural) - similar yield and ultimate state - different cracking behavior</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arezounrandi M. et al. (2015) [1]</td>
<td>0%, 100%</td>
<td>NA</td>
<td>Beams (Shear) - Reduced shear strength 12%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sadati S. et al. (2016)</td>
<td>0% 100%</td>
<td>Normal shear reinforcement</td>
<td>Beams (Shear) - Reduced shear strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>K. N. Rahal, and Y. Y. Alrefaei (2017) [31]</td>
<td>0%, 10%, 20%, 35%, 50%, 75%, 100%</td>
<td>Normal shear reinforcement</td>
<td>Beams (Shear) - Shear capacity is less than NWA - 20% reduction factor is recommended</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zaki I. Mahmoud et al. (2017) [30]</td>
<td>0%, 30%, 60%, 100%</td>
<td>Normal shear reinforcement</td>
<td>Slabs (punching shear) - As RCA increase, shear capacity decrease - Eurocode 2 had the best predictions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maruyama et al. (1988) [5]</td>
<td>0%, 50%, 100%</td>
<td>Typical Reinforcement</td>
<td>Beams (Flexural Shear) - Flexural has similar effects - Shear in RCA is reduced</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fonteboa et al. (2007, 2009) [6, 7]</td>
<td>0%, 50%</td>
<td>Typical Reinforcement</td>
<td>Beams (Shear) - No significant change</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Choi et al. (2010) [8]</td>
<td>0%, 30%, 50%, 100%</td>
<td>Typical Reinforcement</td>
<td>Beams (Shear) - RCA has lower shear than NWA</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Further, the most recent equation predicted by Xu [14, 15] is based on Mode II Fracture Energy, the required fracture energy to release interface bond resistance between steel and concrete

\[ V_c = \frac{1.018}{\sqrt{\pi}} \left( \frac{d}{a_s} \right)^{1/2} \rho^2 \left( 1 - \sqrt{\rho} \right)^2 (0.0255f'_c + 1.24)b_wd \]  
(2)

F’c: specified compressive strength of concrete for use in design, MPa
\( \rho \): longitudinal reinforcement ratio
\( d \): distance from extreme compression fiber to centroid of longitudinal tension reinforcement
\( a_s \): shear span of beam
\( b_w \): web width
\( d_a \): maximum aggregate size

2.6 Proposed Equations for Shear Strength with Fiber

According to Ali and Stephen [39], proposed equations to calculate shear with steel fibers have not been developed yet based on experimental investigation, however, the majority of models are based on regression analysis for some parameters and are empirical in nature. Generally, all models/equations take total Shear \( V_u \) and consider the contribution of concrete (\( V_{uc} \)), longitudinal reinforcement (\( V_{us} \)) and fibers (\( V_{uf} \)).

Fib Model Code 2010:

\[ V_u = V_{us} + V_R \]

\[ V_R = \frac{0.18k b d_e}{\gamma_c} \left( 100 \rho_1 f_{cm} \left( 1 + 7.5 \frac{f_{Ftu}}{f_{ct}} \right) \right)^{1/3} \]  
(3)

\( \gamma_c \) = partial safety factor for concrete
\( \rho_1 \) = longitudinal reinforcement ratio
\( f_{Ftu} \) = Ultimate Residual Flexural-tensile strength for SFRC

Note: if the crack width = 1.5 mm, then \( f_{Ftu} = 0.30 f_R3 + 0.06 f_R1 \)

\[ k = 1 + \sqrt{200/d_e} \leq 2 \]
Moreover, Forster has developed the shear model related to steel fiber reinforced concrete based on the simplified modified compression field theory (MCFT). This model assumes that the contribution of the fibres and concrete matrix are coupled, which means that it's a function of the critical crack width. Figure 3 shows Forster’s components and contribution.

\[
V_{uc} = k_v \sqrt{f_{cm}} b z
\]
\[
V_{us} = \frac{A_{sv}}{S_w} f_{sy,f} z \cot \theta
\]

Where:

\[
k_v = \frac{0.4}{1 + 1500 \varepsilon_x} \times \frac{1300}{1000 + k_{dg} z}
\]
\[
k_{dg} = \frac{32}{16 + d_g} \geq 0.75
\]

Asv: the cross sectional area of the shear reinforcement
Sw: spacing of the shear reinforcement
z: can be approximated as = 0.9 de
\[
\varepsilon_x = \frac{M}{Z} + 0.5 V cot \theta
\]
\[
w = 0.2 + 1000 \varepsilon_x \geq 0.125 \text{ mm}
\]
\[
\theta = 29^\circ + 7000 \varepsilon_x
\]
\[
V_{uf} = k_{fd} f_w b z \cot \theta
\]
Furthermore; the Draft Australian Bridge Code DR AS5100.5: Concrete has defined the shear for steel fibers as follows:

\[ V_u = V_{uc} + V_{uz} + V_{uf} \]

\[ V_{uf} = 0.7k_0 b d f_{1.5} \]  \hspace{1cm} (5)

Where:

\[ k_0 = cot \theta \leq 1.28 \]

On the other hand, according to RILEM 2004 [10], Polypropylene Fibers (Synthetic Fibers) have a corresponding model which can predict the shear strength produced by fib MC2010:

\[ V_{fc} = 0.18(1 + \frac{200}{d})(100 \cdot \frac{A_f}{b w d} \cdot (1 + 7.5 \frac{f_{flu}}{f_{etk}}) \cdot f_{ck})^{1/3} b w d \]  \hspace{1cm} (6)

Where:

\[ f_{ck}: \text{is the characteristic value of the concrete's tensile strength} \]

\[ f_{Ftu(u)} = f_{Fls} - 0.5 \cdot f_{R3} + 0.2 \cdot f_{R1} \]  \hspace{1cm} (7)

However, if the crack width = 1.5 mm, then \( f_{Fls} - 0.45 \cdot f_{R1} \), which then makes \( F_{R1} = 0.5 \) mm and \( F_{R3} = 2.5 \) mm

Moreover; RILEM 2004 [10] have identified that the shear resistance of Synthetic Fibers of a beam can be calculated by the addition of \( V_c \) of the concrete to the \( V_{SYF} \) and \( V_{fc} \) as follows:

\[ V_c = 0.15 \cdot \sqrt[3]{\frac{d}{a}} \cdot k(100 \cdot \rho \cdot f'_{c})^{1/3} \]

\[ V_{SYF} = \frac{1600-d}{1000} \cdot 0.5 \cdot \frac{d}{a} f_{e,3} \]  \hspace{1cm} (8)

\[ V_{frc} = V_c + V_{SYF} \]

Where:

\( a = \) is the shear span

\( \rho = \) reinforcement ratio

\( f_{e,3} = \) equivalent flexural strength
Furthermore, Sharma (1986) [45] proposed the following equation to predict the shear capacity of fiber reinforced concrete which depends on the split tensile strength [44]: 

\[ Vu = kf'_t \left( \frac{d}{a} \right)^{0.25} \]  

(9)

Where:

\( k = \frac{2}{3} \)

\( a/d \) = shear span-depth ratio 

\( f'_t \) = split-cylinder tensile strength of concrete, if known; 

\[ = 0.79 \times (f'_c)^{0.5}, \text{MPa, if the tensile strength is unknown} \]

Moreover, Narayanan and Darwish (1987) proposed an empirical equation to predict shear for fiber reinforced concrete as follows [45]: 

\[ Vu = e \left[ 0.24f_{spfc} + 80\rho \frac{d}{a} \right] + v_b \]  

(10)

Furthermore, Ashour et al. (1992) [47] proposed 2 equations to predict shear capacity with fiber reinforcement (11a and 11b), (12) was similar to ACI Building Code and modified to account for the effect of fibers [46].

For \( a/d \geq 2.5 \)

\[ Vu = \left( 2.113f'_c + 7F \right) \left( \rho \frac{d}{a} \right)^{0.333} \]  

(11a)

For \( a/d \leq 2.5 \)

\[ Vu = \left( 2.113f'_c + 7F \right) \left( \rho \frac{d}{a} \right)^{0.333} \left( \frac{2.5}{a/d} \right) + v_b \left( 2.5 - \frac{a}{d} \right) \]  

(11b)

\[ Vu = \left( 0.7f'_c + 7F \right) \frac{d}{a} + 17.2 \rho \frac{d}{a} \]  

(12)

Where:

\( f_{spfc} \) = computed value of split-cylinder strength of fiber concrete 

\[ = f_{cuf}/(20 - \sqrt{F}) + 0.7 + 1.0 \sqrt{F} \quad \text{(MPa)} \]

\( e \) = arch action factor:
1.0 for a/d > 2.8
2.8 d/a for a/d ≤ 2.8

\( f_{cuf} \) = cube compressive strength of fiber concrete (MPa)

\( F = \) fiber factor = \((L_d/D_f) \times \nu_f \times d_f\)

\( L_d = \) fiber length

\( D_f = \) fiber diameter

\( V_f = \) volume fraction of fibers

\( d_f = \) bond factor: 0.5 for round fibers, 0.75 for crimped fibers, and 1.0 for indented fibers

\( v_b = 0.41\eta F \), where: \( \eta \) = average fiber matrix interfacial bond stress

Imam et al. (1997) [48] proposed another equation which also depends on the fiber fraction [47]:

\[
V_u = 0.6\psi \sqrt{\omega} \left( f'_c 0.44 + 275 \frac{\omega}{(d/a)^{0.5}} \right)
\]

Where:

\( \psi = \) size effect factor = \( 1 + \left( \frac{5.08}{d_{na}} \right)^{0.5} \)

\( \omega = \) reinforcement factor = \( \rho (1 + 4F) \)

Kwak et al. (2002) [49], proposed a different equation and also incorporating the fiber addition and influence into the mix [48]:

\[
V_u = Ae f_{spf}c^{\exp_1} \left( \frac{\rho d}{a} \right)^{\exp_2} + 0.8v_b^{\exp_3}
\]

\( v_b = 0.41\eta F \), where: \( \eta \) = average fiber matrix interfacial bond stress

\( e = 1.0 \) for a/d > 3.5

3.5 x d/a (if a/d ≤ 3.5)

For COV = 14.9%: A=2.1, B=0.8, \( \exp_1=0.7, \exp_2=0.22, \exp_3=0.97 \), however, for COV =15.3%: A=3.7, B=0.8, \( \exp_1=2/3, \exp_2=1/3, \exp_3=1 \)

2.7 Shear Resistance without Web Reinforcement in Codes

According to Ofonime et al., shear provisions in codes are based on empirical equations from experimental test results. Some codes such as BS 8110, ACI 318, and
Eurocode 2 take into consideration the reinforcement ratio, effective depth, and compressive strength of the concrete; however, the Canadian code accounts for shear strength as a function of the compressive strength; moreover, Model code 2010 takes into consideration the longitudinal strain in the web to calculate the shear strength of beams [218]. Ofonime et al, made a comparison of the above mentioned codes and concluded his research that Model code 2010 is the most conservative; however, BS 8110 and Eurocode 2 had the least variation from the results.

2.7.1 BS 8110 (British code).

\[ V_c = \frac{0.79}{\gamma_m} \left( \frac{100\, A_s}{b d} \right)^{\frac{1}{3}} \left( \frac{400}{d} \right)^{\frac{1}{3}} \]  \hspace{0.01em} (15)

100 As/bd: reinforcement ratio (range 0.15 – 3.00)
b: beam width (mm)
d: effective depth (mm)
400/d: size effect ≥ 0.67 (for no web reinforcement members)
\( \gamma_m \): Concrete partial factor for safety

**Notes**: 1- \( F_{cu} > 25 \text{ N/mm}^2 \), equation should be multiplied by \( \left( \frac{F_{cu}}{25} \right)^{\frac{1}{3}} \)
2- Shear Span-to-depth ratio a/d <2-2.5, equation should be multiplied by 2d/av

2.7.2 Eurocode 2.

\[ V_{Rd,c} = \frac{0.8}{\gamma_c} \left( 100\, f_{ck} \right)^{\frac{1}{3}} \left( 1 + \left( \frac{200}{d} \right) \right) b d \]  \hspace{0.01em} (16)

\( F_{ck} \): Cylinder compressive strength of concrete (MPa)
d: effective depth (mm)
\( \gamma_c \): Concrete partial factor for safety
b: beam width (mm)

**Notes**: Short shear Span-to-depth ratio range (0.5d ≤ a ≤ 2d), equation should be multiplied by 2d/av

2.7.3 ACI code 318. ACI code considers both simplified and detailed analysis; the simplified analysis considers only compressive strength of the concrete as shown in the following equation:
\[ V_c = \left( \frac{\lambda f'_c}{6} \right) b_w d \]  
(17)

The detailed analysis:

\[ V_c = \left( 0.16 \sqrt{f'_c} + 17 \rho_w \frac{V_u}{M_u} \right) b_w d \leq 0.29 \sqrt{f'_c} b_w d \]  
(18)

\( f'_c \): compressive strength
\( \rho_w \): flexural reinforcement ratio
\( b_w \): beam width
\( d \): beam effective depth
\( \lambda \): Light weight concrete factor (Normal weight concrete \( \lambda = 1 \))

2.7.4 Canadian code.

\[ V_c = 0.2 \sqrt{f'_c} b_w d \]  
(19)

\( f'_c \): compressive strength
\( b_w \): beam width
\( d \): beam effective depth

2.7.5 Model Code 2010.

\[ V_{Rd,c} = k_v \frac{f'_{ck}}{\gamma_c} b_z \]  
(20)

\( f'_{ck} \): compressive strength
\( b \): beam width
\( d \): beam effective depth
\( \gamma_c \): Concrete partial factor for safety
\( k_v \): the influence of strain in web and aggregate size for beams without shear reinforcement, it has 2 level of approximation:

Level I approximation (members with no significant axial load)

\[ k_v = \frac{180}{1000+1.25z} \]  
(21)

Level II approximation

\[ k_v = \frac{0.4}{1+1500 \varepsilon_k} \cdot \frac{1300}{1000+k_{dg}z} \]  
(22)
Where: \( z \): effective shear span depth = 0.95d (mm)

\( \varepsilon_x \): Longitudinal strain in the web

\[
  k_{dg} = \frac{32}{16+dg} \geq 0.75
\]  \hspace{1cm} (23)

\( k_{dg} \): factor accounting for aggregate size

In conclusion, shear equation for beams without web reinforcement can be used from international codes and researcher’s proposed equations. Arezounnandi et al. [1,2] confirmed that the proposed equations can be used to recycled aggregate concrete. Moreover, proposed equations for fiber contribution as shear reinforcement have been calculated and confirmed on for the conventional concrete, however, not for recycled aggregate concrete. Hence, the fiber contribution equations will be compared with the results of recycled aggregate concrete.
Chapter 3. Experimental Investigation

The main objective of this research is to investigate experimentally the effect of fiber on shear capacity of the recycled aggregate concrete. To achieve this goal, 12 beams were prepared using recycled aggregate, normal weight aggregate, steel fiber, synthetic fiber and hybrid (mix of steel and synthetic fibers). In this chapter, the experimental program, material properties, sample preparation, instrumentation and test setup are presented.

3.1. Experimental Program

The experimental program consists of 12 beams divided into 6 categories, each category has 2 beams. Two beams for normal weight aggregate which will be used to evaluate the effect of aggregate type on the shear capacity of the concrete. Two beams using recycled coarse aggregate which will serve as reference to all beams with fibers. Moreover, for the fiber reinforced concrete beams, two beams are cast for all of the following fiber: 3D and 5D steel fibers, polypropylene (synthetic) fibers, and hybrid (a mix of the 5D steel fiber and synthetic fiber). Table 5 summarizes the beam details with their labels; moreover Figure 4 shows the beams included in the investigation.

Table 5: Mixing Plan

<table>
<thead>
<tr>
<th>Mix #</th>
<th>Code</th>
<th>Type of Aggregate</th>
<th>Fiber Content</th>
<th>Beams QTY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NWA</td>
<td>Normal Weight Aggregate</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>RCA</td>
<td>Recycled Coarse Aggregate</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>RCA-3D</td>
<td></td>
<td>3D</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>RCA-5D</td>
<td></td>
<td>5D</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>RCA-SY</td>
<td></td>
<td>Synthetic</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>RCA-HY</td>
<td></td>
<td>5D + Synthetic</td>
<td>2</td>
</tr>
</tbody>
</table>

3.2. Material Properties

3.2.1. Aggregate properties. Recycled coarse aggregate used in this research were delivered from Bee’ah from different batches in four bags. Samples from the batches were collected to evaluate the physical and mechanical properties of the aggregate. The samples were labelled P1, P2, P3, and P4, Figure 5 shows samples of the aggregate. In addition, two sizes of the normal weight aggregate were included in the evaluation, the samples were labelled as per their sizes (10mm and 20 mm).
3.2.1.1 Absorption. The absorption capacities for both aggregates were evaluated in accordance to ASTM C127 – 12 which are summarized in Figure 6. The results showed that the recycled aggregate has higher absorption capacity than the normal (Conventional) weight aggregate, which is due to the high porosity characteristics of the recycled aggregate and the old mortar paste around the original coarse aggregate. Hence, the high level of porosity of the recycled coarse aggregate is considered a challenge for mixing and might produce dryer mix and shrinkage after the final set of concrete. Special process should be considered to minimize the effect of higher absorption.
3.2.1.2 Relative Density. The relative density of the four recycled aggregate batches and normal weight aggregates was determined in accordance to ASTM C127-12. The values of specific gravity (oven dry), specific gravity (SSD), and apparent specific gravity are summarized in Table 6. The results show that specific gravity (both oven dry and SSD) and apparent specific gravity results are very close which reduces the variability between the batches, furthermore the results obtained shows a slight reduction in comparison to the normal weight aggregates. These values are necessary to determine the aggregate weights necessary for the concrete mixes.

3.2.1.3 Sieve analysis. All batches were sieved according to ASTM C33/C33M and evaluated to its upper and lower limits. The four batches (P1, P2, P3, P4) (Figure 6) were evaluated along with the Normal Weight Aggregate (20 mm and 10 mm) and summarized in Figure #7. According to ASTM C33/33M, the grading requirements for coarse aggregates table, sieve size number would be 7, with Nominal Size 12.5 to 4.75 mm (3/4 in to No. 4), however, one batch (P2) will be ruled out to ensure consistent coarse aggregate being used from the recycled aggregate mix.

3.3 Fiber

Tests were conducted for concrete mix using first normal weight aggregate and recycled aggregate concrete as a control mix for mixes with fibers, both steel and synthetic fibers. Mixes with fibers are divided into 4 groups, 3D and 5D steel fibers, synthetic fibers, and finally a mix containing both 5D steel fiber and synthetic fiber. Table 7 shows the properties evaluation of all fiber types provided by the manufacturer.
Table 6: Relative Density

<table>
<thead>
<tr>
<th></th>
<th>NWA 10 mm</th>
<th>NWA 20 mm</th>
<th>RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity (Oven Dry)</td>
<td>2.51</td>
<td>2.68</td>
<td>2.31 ~ 2.35</td>
</tr>
<tr>
<td>Specific Gravity (SSD)</td>
<td>2.61</td>
<td>2.72</td>
<td>2.46 ~ 2.49</td>
</tr>
<tr>
<td>Apparent Specific Gravity</td>
<td>2.79</td>
<td>2.78</td>
<td>2.72 ~ 2.74</td>
</tr>
</tbody>
</table>

In the literature, steel fibers volumetric replacement ranges between 0.5% – 1.5%, however, through studies, higher replacement of fibers (> 1%) will increase the shear capacity but will reduce the unit weight, compressive strength and mechanical properties [12 - 13]. According to Kutalmis R. A. (et al.), [12] the optimum volumetric replacement of fibers is 1%, however, in this research we are targeting self-consolidated concrete and not to affect the workability of the mixture, 0.75% volumetric replacement will be used for all fiber types (3D, 5D, and synthetic), in case of hybrid this percentage will be divided equally for both fiber types.

3.3.1. Polypropylene fiber. A synthetic (Polypropylene) fiber has high post-cracking control, high flexural strength resistance, and reduce the probability of the formation of small cracks, due to the high strength and high modulus characteristics.

Figure 6: Absorption RCA vs NWA
that this type of fiber has [61]. Moreover, these fibers increase the resistance to impact, fatigue, and toughness that is easily distributed throughout to the concrete mix. (Figure 8a)

3.3.2. 3D Steel fibers. A 3D steel fiber (Figure 8b) provides cost and time efficient solutions to the concrete mixture versus the traditional reinforced concrete due to the simplicity of using this type of fiber. It is characterized with its usability that does not require skilled labor [40]. The main benefits of steel fibers are crack control and the behaviour of under loading, where when the member under loading, the steel fibers will form a bridge within the crack and form a tensile force to hold both concrete ends.

Table 7: Fibers Comparison

<table>
<thead>
<tr>
<th></th>
<th>Strux(90/40) Synthetic</th>
<th>3D Steel Fiber</th>
<th>5D Steel Fiber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>0.92</td>
<td>7.8</td>
<td>7.8</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>9.5 GPa</td>
<td>210 GPa</td>
<td>210 GPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>620 MPa</td>
<td>1345 MPa</td>
<td>2300 MPa</td>
</tr>
<tr>
<td>Melting point</td>
<td>160°C</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Ignition point</td>
<td>590°C</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Length</td>
<td>40 mm</td>
<td>35 mm</td>
<td>60 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>0.44 mm</td>
<td>0.55 mm</td>
<td>0.9 mm</td>
</tr>
<tr>
<td>Codes</td>
<td>ASTM C1116</td>
<td>ASTM C1609/C160M-05</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ANSI/SDI C-1.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 7: Sieve Analysis of different batches
3.3.3. **5D Steel fibers.** A 5D steel fiber (Figure 8c) is a newly enhanced fibers which enables its use in structural elements with longer spans such as suspended structures; other applications in sub-structures such as foundation slabs and rafts. This can be achieved because of the ultimate performance it provides caused by the 5 dimensional hooked shapes, high ductility, and high tensile strength [40]. The 5D length and tensile strength are relatively higher than that of 3D, which gives it the advantage in the post cracking behaviour.

![Figure 8: (a) Polypropylene fibers; (b) 3D Steel Fibers; (c) 5D Steel Fibers](image)

3.4. **Mixing Stage**

The proposed 12 beams were prepared, casted and tested. Four beams are prepared to serve as reference in studying the behaviour of fiber reinforced beams. Two beams were cast as control containing normal weight aggregate; other two beams were cast with 100% recycled aggregate concrete. Remaining eight beams containing fiber as reinforcement for shear, divided into two beams for each type of fibers, 3D and 5D steel fibers, polypropylene (synthetic), and hybrid (a mix of 50% 5D steel fiber and 50% synthetic fiber).

3.4.1. **Beam samples.** Prepared beams to achieve this goal were similar to minimize variability in the testing. As shown in Figures 9 and 10, all beams had a rectangular cross-section of 150 x 200 mm for the width and depth respectively; in addition, all beams were 1600 mm long, with three 12 x 1540 mm diameter and length, respectively, for bars as bottom reinforcement. For top reinforcement, two 12 mm diameter bars were used and with length of 1540 mm. In addition to the above, three 8 mm closed stirrups were used at the support location, to support the two edges of the 12 mm bars reinforcement, tie both bottom and top bars, and to secure the lifting hooks and prevent failure due to lifting. Furthermore, the above described steel cage will be
then placed in a plywood formwork prepared especially for this study using spacers at all sides and bottom to ensure concrete cover is maintained as per design as shown in Figure 11. Moreover, during the mixing stage of recycled aggregate concrete, the recycled aggregates were pre-soaked for 30 minutes in 10% of water from the original mix design, to minimize the water absorbed from the mix during hydration. Finally, the beams and specimens for the mechanical properties to be tested as shown in Figures 12 to 14 were cured with sprayed water for up to 14 days, and then left until 90 days to complete its curing cycle.

3.4.2. Test setup and strain gauges. Prepared beams will be tested for ultimate shear and strain in both concrete and steel bars in the beam. To eliminate external variables into the results, all beams have the same dimensions and strain gauges location. In addition, the mix was performed once for each type to reduce any external factor, with strain gauges both internally and externally. All beams are tested for four-point loading test using Instron servo-hydraulic load frame, with rate of 0.6 mm/min for loading in displacement control. Figure 15 shows the test setup and location of strain gauges (internal and external). During testing, crack number with its corresponding load was recorded. Moreover, the deflection of the beam was recorded using Linear Variable Differential Transducer (LVDT).

Figure 9: Beam Detailing

Figure 10: Beam cross-section
Furthermore, a total of 6 strain gauges were used both internally and externally with three internal strain gauges and 3 external one. Three internal strain gauges are placed as follows, two of which are placed on the bottom steel bars and 1 on the top steel bar, similarly, three external strain gauges are installed, two of which were placed 45 degrees to the beam and perpendicular to the expected shear failure location at the shear span. Figure 16 shows the internal strain gauges that required grinding of steel and special adhesive preparation. On the other hand, Figure 17 shows the external strain gauges, which also required preparation, where sand paper was applied on the required location, then a special adhesive was placed to ensure proper cohesion between the concrete surface and strain gauge.

![Figure 11: Beam formwork – Before casting](image)

![Figure 12: specimens – Before casting](image)
Figure 13: specimens – after casting

Figure 14: Beams after casting

Figure 15: Test Setup
3.5. Testing

Specimens prepared are tested for the mechanical properties, Table 8 shows the specimens type with the corresponding test method and code, which will be evaluated upon. Moreover, the beam evaluation will consist of load versus deflection curve, crack initiated versus load monitor, shear crack location and angle, and finally concrete and steel strain values collected from strain gauge.
Table 8: Code Breakdown

<table>
<thead>
<tr>
<th>SN</th>
<th>Category</th>
<th>Test name</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Aggregate</td>
<td>Absorption Test</td>
<td>ASTM C642 - 13</td>
</tr>
<tr>
<td>2</td>
<td>Aggregate</td>
<td>Bulk Density</td>
<td>ASTM C29/C29M - 09</td>
</tr>
<tr>
<td>3</td>
<td>Aggregate</td>
<td>Relative Density</td>
<td>ASTM C127.1195625-1</td>
</tr>
<tr>
<td>4</td>
<td>Concrete</td>
<td>Cube Compressive Strength</td>
<td>BS 1881-116:1983</td>
</tr>
<tr>
<td>5</td>
<td>Concrete</td>
<td>Cylindrical Split Tensile Test</td>
<td>ASTM C496/C496M - 04</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>Beam flexure</td>
<td>ASTM C1609/C1609M - 12</td>
</tr>
</tbody>
</table>
Chapter 4. Results

This chapter presents the test results of the experimental program in this study. The experimental investigation included evaluation of the mechanical material properties and the experimental results of all beams. The effects of different parameters and material properties on the shear strength are analysed and discussed.

4.1. Material Properties

4.1.1. Compressive strength. For all mixes, 150 x 150 x 150 mm concrete cubes were prepared to evaluate the compressive strength according to the ASTM C39/ C39M – 18. The test results showed that the compressive strength of the recycled aggregate concrete has reduction of 8.5 % compared to that of the normal weight aggregate concrete showing a typical compressive failure, as shown in Figure 18a, similar behaviour reported in literature. However, the recycled aggregate concrete mixes with steel fibers (3D and 5D) have had an increase in compressive strength by 5.4% and 17.3% respectively compared to that of the RCA. In addition, the hybrid mix (5D steel fiber and polypropylene fiber) has had an increase 9.3% compared to that of the RCA. On the other hand, recycled aggregate concrete with polypropylene fiber only have had a slight reduction in compressive strength with respect to RCA of 0.78%. Figure 18b shows the failure occurring to fiber reinforced RCA cubes where the cube maintained its original shape having cracks on the sides; similar behaviour and results were reported in literature. The above discussion indicates that the steel fibers contribute to the carrying load capacity of the cube and absorb the compressive energy imposed on the concrete mix. Table 9 summarizes the compressive strength results.

4.1.2. Split tensile. The test covers the determinations of splitting tensile strength in which 2 cylindrical concrete specimens were used for each mix. The cylinders used had dimensions of 200 mm length and 100 mm diameter. The results in Table 9 showed that RCA cylinders had 5.23 % reduction in splitting tensile strength compared to that of the NWA (Figure 20). However RCA with 3D, 5D, and polypropylene had an increase in the split strength of 63.79 %, 111.84 %, and 83.49 % respectively; moreover, for RCA-HY the increase reached up to 128.54 % compared to that of plain RCA. Furthermore, it is to be noted that the performance of fiber reinforced cylinders (RCA-3D, 5D, SY, HY) have continued to sustain the applied load even after
the initial crack which occurred at a similar load to that of the RCA cylinders failure load (≈ 70 KN). This could be attributed to the fibers ability to bridge cracks and improving the post cracking behaviour of the concrete, the same are illustrated in the graphs Figures 20 and 21. In addition, the behaviour of the cylinder without fibers under the load was brittle, after reaching the ultimate capacity; it did not continue to carry any load as can be shown in Figure 19a. In the contrary, the fiber reinforced cylinders were crushed continuing to sustain the load due to the fibers within, Figure 19b shows a RCA-5D sample after the test where the fibers are exposed and concrete was crushed.

Figure 18 : (a) Typical Compressive strength failure for RCA; (b) Typical compressive strength failure for fiber reinforced RCA

Table 9 : Mechanical Properties

<table>
<thead>
<tr>
<th>SN</th>
<th>Mix ID</th>
<th>Compressive Strength</th>
<th>Split Tensile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Avg. f'c (MPa)</td>
<td>% difference</td>
</tr>
<tr>
<td>1</td>
<td>NWA</td>
<td>70.50</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>RCA</td>
<td>64.50</td>
<td>-8.51%</td>
</tr>
<tr>
<td>3</td>
<td>RCA-3D</td>
<td>68.00</td>
<td>5.43%</td>
</tr>
<tr>
<td>4</td>
<td>RCA-5D</td>
<td>75.67</td>
<td>17.31%</td>
</tr>
<tr>
<td>5</td>
<td>RCA-SY</td>
<td>64.00</td>
<td>-0.78%</td>
</tr>
<tr>
<td>6</td>
<td>RCA-HY</td>
<td>70.50</td>
<td>9.30%</td>
</tr>
</tbody>
</table>

Fiber Reinforced
4.1.3. Flexural performance. The flexural performance test gives an indication of the behaviour of the fiber reinforced concrete under static flexural loading. The test is conducted in accordance with ASTM C1609/C1609M - 12. This test provides the first-peak, peak and residual loads with their corresponding stresses, toughness and the flexural strength ratio. 100 mm x 100 mm x 500 mm prisms were tested and the average results for all mixes are shown in Table 10. The results show that there is 10.14 % increase on the First-Peak strength between RCA to NWA. On the other hand, the fiber reinforced mixes experienced increase in the first-peak strength compared to that of the RCA.
For the steel fiber mixes, both 3D and 5D, with a 3.59 % and 1.91 %, respectively, however, the propylene (synthetic) fibers showed huge increase of 92.50 % in the first-peak strength; and a 10.05 % increase for the hybrid mix. The improvement could be attributed to the fibers ability to control cracking and post cracking behaviour.

Moreover, the aggregate type effect showed a reduction in 10.7% for RCA compared to NWA, as shown in Figure 23a, RCA showed a typical brittle failure similar behaviour were reported in literature. Furthermore, the peak flexural strength increased by 33.77 % and 51.82 % for steel fibers 3D and 5D respectively, however, an increase of 11% and 8.3% for both synthetic and hybrid mixes, these results are related to the both the fiber type in the mix and its configuration; Figure 23b shows a typical failure mode for a fiber reinforced prism were the fibers improved the deflection and post cracking behaviour. The residual strength which characterize the prisms residual capacity of mix after cracking is calculated at specified deflections, L/150 and L/600 of the span length which is shown in the Figure 22, this shows that the fiber addition improved the beam carrying capacity at the specified deflection locations and contributed to higher flexural strength and post cracking behaviour. Appendix A summarizes the results of flexural strength obtained with graphs.
4.2. Shear Tests

4.2.1 Load – Deflection relationships. 2 beams per category were subjected to the 4-point load test to record their behaviour, results including the ultimate load, failure load, shear load, ultimate deflection and stiffness are summarized in Table 11. The average between them was used for comparison to the control mixes and the percentage difference of normalized shear resistance average is shown in Table 12.

Table 10: Flexural Properties

<table>
<thead>
<tr>
<th>SN</th>
<th>Mix ID</th>
<th>Flexural Strength</th>
<th>% difference</th>
<th>Peak Strength (MPa)</th>
<th>% difference</th>
<th>Residual Strength (MPa) (L/150)</th>
<th>Residual Strength (MPa) (L/600)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NWA</td>
<td>0.38</td>
<td>-</td>
<td>7.00</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>RCA</td>
<td>0.42</td>
<td>10.14%</td>
<td>6.25</td>
<td>-10.70%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>RCA-3D</td>
<td>0.43</td>
<td>3.59%</td>
<td>7.73</td>
<td>23.65%</td>
<td>4.50</td>
<td>6.27</td>
</tr>
<tr>
<td>4</td>
<td>RCA-5D</td>
<td>0.43</td>
<td>1.91%</td>
<td>10.89</td>
<td>74.12%</td>
<td>5.40</td>
<td>9.28</td>
</tr>
<tr>
<td>5</td>
<td>RCA-SY</td>
<td>0.80</td>
<td>92.50%</td>
<td>6.94</td>
<td>11.00%</td>
<td>3.65</td>
<td>5.34</td>
</tr>
<tr>
<td>6</td>
<td>RCA-HY</td>
<td>0.46</td>
<td>10.05%</td>
<td>6.77</td>
<td>8.30%</td>
<td>3.49</td>
<td>4.82</td>
</tr>
</tbody>
</table>

(a)
Figure 22: Flexural strength (a) NWA vs RC; (b) RCA vs RCA-3D

Ultimate load is the maximum load carried by the beam under testing, as for the failure load is the maximum load for the said beam before the first drop in load; In addition, the shear capacity of the beam is considered to be 0.5xP (Ultimate load). Ultimate deflection, is characterized by the deflection corresponding to the ultimate load which can be seen in Figure 24a, furthermore, the stiffness is calculated in the elastic region shown in Figure 24b were the elastic stiffness in the initial slope of the load deflection curves is considered.

Moreover, the ultimate shear capacity is influenced by the compressive strength, hence normalized shear resistance was made and found that plain RCA had increased by 19.56% compared to NWA where it reached to 5.09 from 4.25, which can be
explained by the better aggregate interlock and pre-soaking mixing technique, where the recycled aggregate are pre-soaked with 10% of water (from mix design), in which the aggregate particles are already saturated with water and will not absorb more water from the hydration process. Further, the normalized shear capacities of RCA-3D and RCA-5D have increased by an average of 60.19% and 45.73% from RCA respectively. However, the normalized shear resistance have increased by 51.18% and 40.61% from RCA respectively. RCA-3D showed the greatest improvement which reached up to 8-15, and RCA-HY showed the least improvement by 7.15. RCA-5D and RCA-SY resulted of 7.41 and 7.69 for the normalized shear resistance. Appendix B shows all the Load-Deflection curves for all beams, along with detailed tables for all beams separately.

4.2.2 Strain gauges (Steel and Concrete). Figure 15 in Chapter 3 shows the location of strain gauges both internally on the steel bars (steel bottom 1, 2, and top) and externally (concrete right, left, top). Internal strain gauges are fixed on steel bars, 2 on bottom steel (Steel Bottom 1 and Steel Bottom 2) and 1 fixed on the top steel (Steel Top). In addition, the three external strain gauges are fixed on concrete, two of which are located in the shear span 20 cm from the respective support, inclined at approximately 45 degrees, and perpendicular to the expected crack location, both strain gauge results are shown in Figure 25 and labelled Concrete Right and Concrete Left; on the other hand, one strain gauge is located in the compression region between the applied load. Figure 25 shows a sample of load (N) versus micro-strain curves for RCA without fiber reinforcement to that of RCA-5D in both steel and concrete strain gauges.

Figure 25-a shows RCA (steel strain gauges), Figure 25-b shows the RCA-5D (steel strain gauges) which shows that the strain in bottom steel increased from 2245.7 micro-strain (=0.002245) to 13856.88 micro-strain (=0.01386); moreover, Figure 25-c and Figure 25-d shows the load-strain curves for concrete in both RCA and RCA-5D respectively. In general, the fiber addition to RCA have improved the overall strain behaviour of the beam in both steel and concrete, this can be explained by the post cracking behaviour of steel fiber, however, this is not the case in synthetic fiber addition. Finally, it is noted that crushing failure occurred at the top strain gauge location in both beams for steel fiber reinforced RCA-5D and only one of RCA-3D, and the remaining mixes underwent concrete failure at the concrete strain gauges left and right as predicted. Appendix C shows all load-stain curves for all mixes obtained.
### Table 11: Beam results summary

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Ultimate load (kN)</th>
<th>Failure Load (kN)</th>
<th>Shear Load (Vc) (kN)</th>
<th>Ultimate deflection (mm)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NWA 1</td>
<td>65.22</td>
<td>65.22</td>
<td>32.61</td>
<td>9.55</td>
<td>11.36</td>
</tr>
<tr>
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<td>77.66</td>
<td>70.92</td>
<td>38.83</td>
<td>7.45</td>
<td>10.08</td>
</tr>
<tr>
<td>RCA 1</td>
<td>82.8</td>
<td>66.06</td>
<td>41.4</td>
<td>9.58</td>
<td>8.03</td>
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<td>64.72</td>
<td>40.3</td>
<td>8.85</td>
<td>9.28</td>
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<td>RCA-3D-1</td>
<td>128.7</td>
<td>120.47</td>
<td>64.35</td>
<td>14.58</td>
<td>9.2</td>
</tr>
<tr>
<td>RCA-3D-2</td>
<td>140.06</td>
<td>136.28</td>
<td>70.03</td>
<td>14.52</td>
<td>10.18</td>
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<tr>
<td>RCA-5D-1</td>
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<td>121.96</td>
<td>65.08</td>
<td>14.66</td>
<td>9.1</td>
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<td>121.05</td>
<td>65.01</td>
<td>14.2</td>
<td>9.66</td>
</tr>
<tr>
<td>RCA-SY-1</td>
<td>97.22</td>
<td>96.59</td>
<td>48.61</td>
<td>10.61</td>
<td>11.78</td>
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<td>52.24</td>
<td>10.58</td>
<td>10.85</td>
</tr>
<tr>
<td>NWA-HY-1</td>
<td>123.8</td>
<td>123.8</td>
<td>61.9</td>
<td>10.93</td>
<td>9.02</td>
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<td>NWA-HY-2</td>
<td>116.4</td>
<td>116.4</td>
<td>58.2</td>
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<td>9.62</td>
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### Table 12: Summary of percentage increase in Normalized shear resistance

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Normalized shear resistance ((V_c/\sqrt{f'_c}))</th>
<th>Stiffness % difference</th>
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<tbody>
<tr>
<td>NWA</td>
<td>4.25</td>
<td>-</td>
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<tr>
<td>RCA</td>
<td>5.09</td>
<td>19.56%</td>
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<tr>
<td>RCA-3D</td>
<td>8.15</td>
<td>60.19%</td>
</tr>
<tr>
<td>RCA-5D</td>
<td>7.41</td>
<td>45.73%</td>
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<td>RCA-SY</td>
<td>6.30</td>
<td>51.18%</td>
</tr>
<tr>
<td>RCA-HY</td>
<td>7.15</td>
<td>40.61%</td>
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</table>
Figure 24: Load vs Deflection curve (a) NWA vs RCA; (b) All beams
Figure 25: Load-Strain curves (a) RCA-2 steel strain gauges; (b) RCA-5D-2 steel strain gauges; (c) RCA-1 concrete strain gauges; (d) RCA-5D-2 concrete strain gauges
4.2.3 Crack pattern and Initial crack. Crack initiation and propagation were monitored during loading. Figure 26-a shows the RCA-3D beam and Figure 26-b the schematic diagram sketch for the same beam. Crack patterns for other beams are shown in Appendix D. Table 13 shows the recorded crack patterns and their respective loads. On the other hand, the initial shear cracks along with their respective percentage from beam’s ultimate load are summarized for individual beam in Table 14. The results show that there is a decrease in initial shear crack capacity of RCA beams with respect to that of the NWA beams, however, the addition of fiber le to an increase in all the initial shear crack capacities for all beams.

![Crack pattern (a) RCA-3D-2 beam crack pattern; (b) RCA-3D-2 schematic diagram for cracks](image)

**Figure 26**: Crack pattern (a) RCA-3D-2 beam crack pattern; (b) RCA-3D-2 schematic diagram for cracks

4.2.4 Mode of failure. Table 15 shows the failure mode for each of the 12 beams. Both beams of RCA-5D and 1 beam from RCA-3D had failed in compression, where the concrete between the 2 load applied points was under stress and crushed; Figure 27-a shows the location of crushing of concrete on top for RCA-5D-2. Figure 27-b shows typical shear failure (RCA-SY-2) for other beams. Appendix D shows all the beams post testing cracks and their schematic diagrams.
4.3 Predicted Shear Strength

4.3.1 Comparison with codes and proposed equations: The experimental results compared to the predicted shear strength obtained from international codes, proposed equations, Table 16 summarizes the comparison between results of the RCA and RCA-3D beams to the predicted values from the simplified and detailed ACI equations and FIB 2010 model with and without fibers.

Figure 27: Failure mode (a) RCA-5D-2 beam; (b) RCA-SY-2 beam
Table 13: Summary of Crack and Corresponding Load

<table>
<thead>
<tr>
<th>Crack #</th>
<th>Load (kN)</th>
<th>Load (kN)</th>
<th>Load (kN)</th>
<th>Load (kN)</th>
<th>Load (kN)</th>
<th>Load (kN)</th>
<th>Load (kN)</th>
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<td>34.65</td>
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<td>45.83</td>
<td>19.44</td>
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<td>81.84</td>
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### Table 14: Initial Crack Summary

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<th>Beam ID</th>
<th>Ultimate load (kN)</th>
<th>Initial crack #</th>
<th>Shear @ initial crack (kN)</th>
<th>Average</th>
<th>Percentage from ultimate load</th>
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<td>44.23</td>
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<td>54.51%</td>
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<tr>
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<td>66.07</td>
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<td>36.76%</td>
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<td>83.28</td>
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<td>32.73%</td>
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### Table 15: Failure Modes

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<td>RCA 1</td>
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<td>RCA-3D-1</td>
<td>Compression</td>
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<td>Compression</td>
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Table 16: Experimental and Predicted Shear Values for RCA and RCA-3D

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<th>RCA-3D</th>
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<td>Compressive Strength (fc) (MPa)</td>
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<td>134.38</td>
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<td>Shear Load (Vc)</td>
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<td>67.19</td>
</tr>
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<td>Normalized shear resistance</td>
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<td></td>
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<tr>
<td>Shear (Simplified) (kN)</td>
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<td>32.78</td>
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<td>Vc (exp)/Vc predicted</td>
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<td><strong>ACI Detailed</strong></td>
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<td>Shear 2 (Detailed) (kN)</td>
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<td>Vc (exp)/Vc predicted</td>
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<td><strong>FIB 2010</strong></td>
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<td><strong>FIB 2010 (with Fiber)</strong></td>
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<td>Shear (kN)</td>
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<td>48.68</td>
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<td>Vc (exp)/Vc predicted</td>
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<td>1.38</td>
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Chapter 5. Discussion of Results

In this chapter, the effect of aggregate type and fiber addition on the shear capacity of recycled aggregate concrete is discussed. In addition, the experimental results are compared to that predicted by other equations found in the literature.

5.1 Material Evaluation

The mechanical properties of the proposed mixes were evaluated. The test results were presented in Chapter 4, however, the effect of the parameters included in the study on the results is discussed in the following subsections.

5.1.1 Compressive strength. Results of the average cube compressive strength from all mixes are summarized in Table 9 in Chapter 4. The RCA showed a decrease of 8.5 % which is justified by high porosity and absorption characteristics of the recycled aggregate particles. In the literature, 20 – 50 % replacement has no significant effect to the mechanical and tensile properties, however, concrete prepared using 100 % RA, which is the case in this study, up to 14% reduction in compressive strength is observed. Moreover, fiber addition showed an increase in compressive strength only in the steel type fibers; 3D, 5D and hybrid fiber addition showed 5.42 %, 17.31 % and 9.3 % increase compare to that of the RCA without fiber. Propylene (synthetic) fibers, on the other hand, showed a decrease in compressive to 43 MPa with respect to 64.5 MPa of the RCA with about 33.0 % decrease. This could be attributed to the reduced bond between the synthetic fiber and the RA which might led to increase of the internal voids and reduced compressive strength.

5.1.2 Split tensile. Table 9 shows the results of split tensile tests carried for all mixes, a slight decrease of 5.23 %, in the split tensile strength in the RCA mix which could be explained by the reduced crushing value of the RCA. However, fiber-reinforced RCA have shown a huge difference in the behaviour and split tensile strength. The fiber addition improved the crack initiation and the post cracking behaviour which led to increase of the load carrying capacity. The increase compared to that of the split tensile strength of RCA was 141.16 %, 72.83 %, 123.55 %, and 93.62 % for the hybrid RCA, RCA-3D, RCA-5D, and RCA-SY, respectively.

5.1.3 Flexural strength. Peak flexural strength of the RCA without fiber-reinforcement has reduced by 10.7 % with respect to that of the NWA. Similar results
were reported in the literature, the RCA showed reduction in flexural strength up to 14 % to that of the NWA. However, the first-peak strength has increased by 10.14 % with respect to NWA. On the other hand, fiber-reinforced prisms have shown an increase in peak flexural strength with respect to that of the RCA of up to 74.12 %, 23.65%, 11 %, and 8.3 % for RCA-5D, RCA-3D, RCA-SY, and RCA-HY, respectively. Moreover, fiber addition in tested prisms changed the behaviour of the beam from brittle failure to ductile-behaviour. The prisms with fibers continued to carry load after cracking and the residual capacity was varied based on the fiber type. Figure 28 shows the residual strength occurring at net deflection of L/150 were 4.5, 5.4, 3.65, and 3.5 MPa for RCA-3D, RCA-5D, RCA-SY and RCA-HY respectively. However, at net deflection of L/600, the residual capacity reached to 6.27, 9.28, 5.34, and 4.82 MPa for the RCA-3D, RCA-5D, RCA-SY and RCA-HY respectively. The 5D steel fibers showed the highest results for both peak and residual strengths, which could be attributed to the improved pull-out capacity of the 5D fiber and improved bond with the surrounding concrete. However, the hybrid RCA mix showed the least results in terms of peak and residual strengths, in addition it showed very low increase of about 0.52 MPa in the peak strength with respect to that of the RCA. Furthermore, peak deflection is one of the characteristics that describes the benefits of fiber-reinforced prisms, where in this study, the RCA-3D and RAC-5D peak deflection (deflection occurs at peak load) was 0.27 mm and 0.21 with respect to 0.05 mm for the plain RCA. The RCA-3D was more than 4 times the peak deflection that of the RCA and the RCA-5D was more than 3 times that of RCA. This could be attributed to the ability of the fibers to bridge the cracks and control the crack propagation.

5.2 Concrete Shear Capacity

The main objective of the study is to investigate the effect of fiber addition on the shear capacity of the RCA, hence 12 beams were tested, 2 of each category, NWA, RCA, RCA-3D steel fibers, RCA-5D steel fibers, RCA-SY Propylene (synthetic) fibers, and RCA-HY hybrid (Synthetic + 5D steel fibers). Table 17 shows the average experimental ultimate load, shear capacity, ultimate deflection, and stiffness.
5.2.1 Load-deflection curve. The average load versus deflection curves were prepared for all mixes and summarized in Figure 29. In general, the RCA showed an increase in the shear capacity 14.36 % than that of the NWA. This could be attributed to the improved aggregate interlock for the RA due to the rough surface and the improved properties due to the pre-soaking technique which was followed during the samples preparation. Furthermore, the fiber addition also have improved the shear capacity of the beams by about 64.48 %, this can be interpreted by the energy absorption and the post cracking behaviour of the different types fibers. In general, the fiber can assist in controlling and transferring the stresses through cracks. Moreover, due to the variability in compressive strengths of all mixes, normalized shear strength have been calculated to better understand the difference in shear resistance. Table 18 shows a summary of the average shear capacity, percentage increase for each mix, and normalized shear resistance. Normalized shear resistance results reflects the shear resistance improvement without the compressive strength factor, and since there are relatively comapritive results of compressive strengths, normalized shear resistance shows that RCA-3D showed the best improvements than plain RCA followed by RCA-SY, RCA-5D, and RCA-HY in the same order.
Table 17: Average test results for all groups

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Ultimate load (kN)</th>
<th>Failure Load (kN)</th>
<th>Shear Load Vc (kN)</th>
<th>Ultimate deflection (mm)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NWA</td>
<td>71.44</td>
<td>68.07</td>
<td>35.72</td>
<td>8.5</td>
<td>10.72</td>
</tr>
<tr>
<td>RCA</td>
<td>81.7</td>
<td>65.39</td>
<td>40.85</td>
<td>9.22</td>
<td>8.66</td>
</tr>
<tr>
<td>RCA-3D</td>
<td>134.38</td>
<td>128.38</td>
<td>67.19</td>
<td>14.55</td>
<td>9.69</td>
</tr>
<tr>
<td>RCA-5D</td>
<td>130.09</td>
<td>121.51</td>
<td>65.05</td>
<td>14.43</td>
<td>9.38</td>
</tr>
<tr>
<td>RCA-SY</td>
<td>100.85</td>
<td>100.54</td>
<td>50.43</td>
<td>10.60</td>
<td>11.32</td>
</tr>
<tr>
<td>RCA-HY</td>
<td>120.1</td>
<td>120.1</td>
<td>60.05</td>
<td>11.79</td>
<td>9.32</td>
</tr>
</tbody>
</table>

5.2.2 Failure modes. In all beams, vertical short flexural cracks were observed due to bending, afterwards, flexural shear cracks were initiated as the load increased. Table 14 in Chapter 4 shows the load corresponding to the first shear crack. Noticeable deformation occurred as the load continued to increase especially in fiber reinforced beams. In addition, in non fiber-reinforced beams shear crack was mostly single line and angles of crack were between 16.7° - 43.5°; however, most of fiber reinforced beams had 2 shear cracks visually seen on the surface of the beam and angles of crack were 19.1° - 47.7°, which could be explained by the resistance of fiber against the shear tensile forces which led to the change of crack directions. The fiber addition led to a change in the failure mode due to the improvement in the load carrying capacity. Several beams the failure occurred in the compression zone which indicates that the fiber provided adequate shear capacity to prevent shear failure. Also, strain readings of the bottom steel rebars showed 0.00387 which is close to the strain at yield of the reinforcing bars. Furthermore, the deflection of fiber reinforced beams at load equivalent to 81.7 kN (load at which plain RCA failed) showed that improvement for all groups; RCA-SY mix showed the lowest improvement by 7.84 mm at mid-span at the mentioned load; however, RCA-5D showed the highest improvement with 5.8 mm at mid-span. Moreover, RCA-3D and RCA-HY showed 6 mm and 6.4 mm mid-span deflection at load of 81.7 kN, respectively.
Figure 29: Avg load vs deflection for all mixes

Table 18: Average Percentage Increase in Compressive

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Shear Load (Vc) (kN)</th>
<th>Shear Load % difference</th>
<th>Compressive Strength (f′c) (MPa)</th>
<th>Normalized shear resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>NWA</td>
<td>35.72</td>
<td>-</td>
<td>70.50</td>
<td>4.25</td>
</tr>
<tr>
<td>RCA</td>
<td>40.85</td>
<td>14.36%</td>
<td>64.50</td>
<td>5.09</td>
</tr>
<tr>
<td>RCA-3D</td>
<td>67.19</td>
<td>64.48%</td>
<td>68.00</td>
<td>8.15</td>
</tr>
<tr>
<td>RCA-5D</td>
<td>65.05</td>
<td>59.23%</td>
<td>77.00</td>
<td>7.41</td>
</tr>
<tr>
<td>RCA-SY</td>
<td>50.43</td>
<td>23.44%</td>
<td>43.00</td>
<td>7.69</td>
</tr>
<tr>
<td>RCA-HY</td>
<td>60.05</td>
<td>47.00%</td>
<td>70.50</td>
<td>7.15</td>
</tr>
</tbody>
</table>
5.3. Comparison of results

5.3.1 Comparison with literature. Results of mechanical properties from literature have been compared with results in this study. In general, studies show that fiber reinforced concrete with recycled aggregate has reduced the compressive strength, but on the other hand, have increased both flexural and split tensile strengths. Table 19 summarizes papers with recycled aggregate concrete that used fibers reinforcement and their mechanical properties.

5.3.2 Shear capacity with codes and proposed formulas. Experimental shear capacity were compared to code predictions, Table 20 summarizes the experimental shear capacity, the expected shear using codes (ACI simplified, ACI detailed, British standards (BS), Eurocode 2, Canadian code, and FIB model 2010) in addition to proposed equations in literature for both Bazant and Yu (2005) [17] and Xu et al. (2001) [19], which have been suggested and confirmed by Mehdi et al. (2015) [1] that it can used to predict the shear capacity for recycled aggregate concrete. Figure 30 shows the ratio between experimental shear capacity $V_{exp}$ and predicted shear capacity $V_u$. 

Figure 30: $V_{exp}/V_u$ (codes and proposed equations)
Table 19: Comparison Summary of Literature and This Study

<table>
<thead>
<tr>
<th>Authors</th>
<th>RCA %</th>
<th>Fiber</th>
<th>Fibers %</th>
<th>Parameters Tested</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ahmadi et al. (2017) [13]</td>
<td>0%, 50%, 100%</td>
<td>Recycled Steel Fibers</td>
<td>0%, 1%</td>
<td>Mechanical Properties</td>
<td>-11.94% 48.28% 22.86%</td>
</tr>
<tr>
<td>Afroughsabet et al. (2017) [25]</td>
<td>0%, 50%, 100%</td>
<td>Steel Fibers</td>
<td>0%, 1%</td>
<td>Mechanical Properties</td>
<td>9.76% 54.10% 80.79%</td>
</tr>
<tr>
<td>Akca et al. (2015) [12]</td>
<td>0%, 25%, 30%, 55%</td>
<td>Synthetic Fibers</td>
<td>0%, 1%, 1.5%</td>
<td>Mechanical Properties</td>
<td>-10.71% 10.41% 4.90%</td>
</tr>
<tr>
<td>Mohseni et al. (2017) [11]</td>
<td>0%, 20%</td>
<td>Steel + Synthetic + Hybrid</td>
<td>0.25%, 0.5%, 1%</td>
<td>Mechanical Properties</td>
<td>-1.70% 64.00% -</td>
</tr>
<tr>
<td>Study</td>
<td>Steel Fibers</td>
<td>Mechanical Properties</td>
<td>Improvement</td>
<td></td>
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<tr>
<td>------------------------------</td>
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<tr>
<td>Craneiro et al. (2014)</td>
<td>0%, 25%</td>
<td>0%, 0.75%</td>
<td>13.19%</td>
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<td></td>
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<td>22.08%</td>
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<td></td>
<td>38.32%</td>
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<tr>
<td>Gao et al. (2009) [26]</td>
<td>0%, 100%</td>
<td>0%, 1%</td>
<td>3.96%</td>
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<td></td>
<td></td>
<td></td>
<td>-4.46%</td>
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<tr>
<td>Meddah et al. (2013) [27]</td>
<td>0%, 100%</td>
<td>0%, 1%, 1.25%, 1.5%, 2%</td>
<td>-40.00%</td>
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<td>59.26%</td>
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<tr>
<td>Gao et al. (2018)</td>
<td>0%, 30%, 50%, 100%</td>
<td>0%, 0.5%, 1%, 1.5%, 2%</td>
<td>5.79%</td>
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<td>9.11%</td>
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<td>78.30%</td>
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<tr>
<td>Chen et al. (2014)</td>
<td>0%, 100%</td>
<td>0%, 0.5%, 1%, 1.5%</td>
<td>-9.83%</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td></td>
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<tr>
<td>This study</td>
<td>0%, 100%</td>
<td>0.75%</td>
<td>5.42%</td>
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<td></td>
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<td></td>
<td>72.83%</td>
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<td>23.65%</td>
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<td>17.31%</td>
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<td>123.55%</td>
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<td>74.12%</td>
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<td></td>
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<td></td>
<td>-0.78%</td>
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<td></td>
<td>93.62%</td>
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<td>10.99%</td>
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<td>9.30%</td>
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<td>141.16%</td>
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<td>8.30%</td>
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</table>
Results showed that FIB model 2010 code had the most conservative than the other codes, and, ACI simplified method had the closest results. However, Xu et al. showed a huge difference in results which the experimental capacity was almost 4 times the predicted capacity.

Furthermore, the shear capacities for fiber-reinforced beams have been compared to the proposed equations in literature. Fourteen equations have been used from 13 researchers have been used to predict the shear capacities, Figure 31 and Table 21 summarizes the predicted shear capacities and ratio between the experimental and predicted shear capacities ($V_{ex}/V_{u}$). FIB model 2010, Imam and Vandewalle (1997) [48], Kwak et al. (2011) [59], and Khuntia et al. (1999) showed relatively conservative results; moreover, the remaining proposed equations showed very close prediction especially Ashour et al. (1992) [47], Kwak et al. (2002) [59], and Rilem (2003) [5]. Further to the above, Slater et al. (2012) [37] was only applicable to steel fiber mixes because of the high aspect value in synthetic fibers. Finally, the proposed equations have used the fiber factor to predict the shear capacity of the mix, all but three equations FIB model 2010, Sharma (1986) [45], and Rilem (2003) [5] which depends mostly on the flexural, splitting tensile, and maximum aggregate size.

Most of the proposed equations are taking into consideration the fiber factor such as, Narayanan and Darwish (1987) [46], Ashour et al. (1992) [47], Imam and Vandewalle (1997) [48], Kwak et al. (2002) [59], and Slater et al. (2012) [37]; however, not all showed close results to the experimental one, which was not efficient and as shown in Figure 30 and Table 21 in the case of Oh et al. (1999) the ratio of the experimental shear to the predicted ultimate shear were 2.25. Furthermore, the proposed equations still show variability in predicting the shear of fiber reinforced beams, hence further studies should be conducted to better formulate a proposal of an overall formula. Moreover, in hybrid mix, some of the proposed equations showed close results, others showed conservative results, however, the remaining showed high difference from experimental shear capacity. In conclusion, the most conservative and closest proposed equation which can be used for all fiber types in this study was FIB model 2010.
Figure 291: $\frac{V_{\text{exp}}}{V_{\text{predicted}}}$ (proposed equations for fiber reinforced concrete beams)
Table 20: Summary of Shear Capacity vs Codes

<table>
<thead>
<tr>
<th></th>
<th>NWA</th>
<th>RCA</th>
<th>RCA-3D</th>
<th>RCA-5D</th>
<th>RCA-SY</th>
<th>RCA-HY</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Experimental</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive Strength (f'c) (MPa)</td>
<td>70.50</td>
<td>64.50</td>
<td>68.00</td>
<td>77.00</td>
<td>64.00</td>
<td>70.50</td>
</tr>
<tr>
<td>Ultimate load (kN)</td>
<td>71.44</td>
<td>81.70</td>
<td>134.38</td>
<td>130.09</td>
<td>100.85</td>
<td>106.81</td>
</tr>
<tr>
<td>Shear Load (Vc) (kN)</td>
<td>35.72</td>
<td>40.85</td>
<td>67.19</td>
<td>65.05</td>
<td>50.43</td>
<td>53.41</td>
</tr>
<tr>
<td><strong>ACI</strong></td>
<td></td>
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</tr>
<tr>
<td>Shear (Simplified) (kN)</td>
<td>33.38</td>
<td>31.92</td>
<td>32.78</td>
<td>34.88</td>
<td>31.80</td>
<td>33.38</td>
</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>1.07</td>
<td>1.28</td>
<td>2.05</td>
<td>1.86</td>
<td>1.59</td>
<td>1.60</td>
</tr>
<tr>
<td>Shear 2 (Detailed) (kN)</td>
<td>32.05</td>
<td>30.66</td>
<td>31.48</td>
<td>33.50</td>
<td>30.54</td>
<td>32.05</td>
</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>1.11</td>
<td>1.33</td>
<td>2.13</td>
<td>1.94</td>
<td>1.65</td>
<td>1.67</td>
</tr>
<tr>
<td><strong>BS</strong></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Shear (kN)</td>
<td>24.96</td>
<td>24.96</td>
<td>24.96</td>
<td>24.96</td>
<td>24.96</td>
<td>24.96</td>
</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>1.43</td>
<td>1.64</td>
<td>2.69</td>
<td>2.61</td>
<td>2.02</td>
<td>2.14</td>
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<tr>
<td><strong>Eurocode 2</strong></td>
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</tr>
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<td>Shear (kN)</td>
<td>24.68</td>
<td>23.96</td>
<td>24.39</td>
<td>25.42</td>
<td>23.90</td>
<td>24.68</td>
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<tr>
<td>Vc (exp)/Vc predicted</td>
<td>1.45</td>
<td>1.70</td>
<td>2.76</td>
<td>2.56</td>
<td>2.11</td>
<td>2.16</td>
</tr>
<tr>
<td><strong>Canadian</strong></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>40.05</td>
<td>38.31</td>
<td>39.33</td>
<td>41.86</td>
<td>38.16</td>
<td>40.05</td>
</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>0.89</td>
<td>1.07</td>
<td>1.71</td>
<td>1.55</td>
<td>1.32</td>
<td>1.33</td>
</tr>
<tr>
<td><strong>FIB 2010</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Shear (kN)</td>
<td>19.20</td>
<td>18.37</td>
<td>18.86</td>
<td>20.07</td>
<td>18.30</td>
<td>19.20</td>
</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>1.86</td>
<td>2.22</td>
<td>3.56</td>
<td>3.24</td>
<td>2.76</td>
<td>2.78</td>
</tr>
<tr>
<td><strong>Bazant and Yu (2005)</strong></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>1.64</td>
<td>1.91</td>
<td>3.10</td>
<td>2.93</td>
<td>2.36</td>
<td>2.45</td>
</tr>
<tr>
<td><strong>Xu et al. (2001)</strong></td>
<td></td>
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</tr>
<tr>
<td>Shear (kN)</td>
<td>9.77</td>
<td>9.28</td>
<td>9.57</td>
<td>10.31</td>
<td>9.24</td>
<td>9.77</td>
</tr>
<tr>
<td>Vc (exp)/Vc predicted</td>
<td>3.65</td>
<td>4.40</td>
<td>7.02</td>
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Table 21: Summary of Fiber-reinforced Experimental Results versus Proposed Equations

<table>
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<tr>
<th>Researcher and Year</th>
<th>Compressive Strength (f’c) (MPa)</th>
<th>Shear Load (Vc) (kN)</th>
<th>Shear (kN)</th>
<th>Vc (exp)/Vc predicted</th>
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<tbody>
<tr>
<td>RCA-3D</td>
<td>68.00</td>
<td>67.19</td>
<td>48.68</td>
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<td>RCA-5D</td>
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<td>RCA-SY</td>
<td>64.00</td>
<td>50.43</td>
<td>43.85</td>
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<td>RCA-HY</td>
<td>70.50</td>
<td>53.41</td>
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<td>Sharma (1986)</td>
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<td>Shear (kN)</td>
<td>57.12</td>
<td>1.18</td>
<td>1.18</td>
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<tr>
<td>Vc (exp)/Vc predicted</td>
<td>73.88</td>
<td>0.88</td>
<td>0.88</td>
<td>1.68</td>
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<td>Narayanan and Darwish (1987)</td>
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<td>Shear (kN)</td>
<td>56.10</td>
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<td>Vc (exp)/Vc predicted</td>
<td>59.81</td>
<td>1.09</td>
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<td>64.62</td>
<td>0.95</td>
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<td>Imam and Vandewalle (1994)</td>
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<td>Oh et al. (1999)</td>
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Chapter 6. Conclusion and Recommendation

6.1. Conclusion

The main purpose of this research is to evaluate the mechanical properties and shear strength of fiber-reinforced recycled aggregate concrete. In addition, the influence of fiber type/configuration on the concrete shear strength is investigated. The experimental program consists of 12 beams and divided into 6 groups, groups 1 and 2 (2 beams each) were cast with normal weight and coarse recycled aggregates to serve as control specimens. The other four groups (2 beams each) were prepared using 0.75% volumetric ratio of steel fibers 3D and 5D, polypropylene (synthetic) fiber, and hybrid fiber (mix of steel fiber 5D and synthetic fiber). The experimental shear capacities were compared with shear capacity predicted from different design codes and proposed equations in literature, the outcomes of this study can be summarized as follows:

Effect of aggregate type and fiber addition on the concrete mechanical properties

1. RCA without fiber reinforcement showed a slight decrease in the mechanical properties. The average decrease is about 8.51%, 5.23%, and 10.7% in cube compressive strength, split tensile and flexural peak strength, respectively, compared to that of the NWA.

2. The fiber reinforced concrete mixes showed an increase in the mechanical properties with respect to that of RCA mix without fiber. The average compressive strength increase was 5.43%, 17.31%, and 9.3% for RCA-3D, RCA-5D, RCA-HY; respectively. However, RCA-SY showed a slight decrease in the compressive strength of 0.78%.

3. Split tensile strength of RA with hybrid fiber (RCA-HY) showed the highest increase of about 141.16%, for other mixes the percentages increase were 72.83%, 123.55%, and 93.62% for RCA-3D, RCA-5D, and RCA-SY; respectively, compared to that of the RCA without fiber.

4. Flexural strength of the fiber reinforced specimens have shown an increase in the peak strength especially in the 5D steel fiber mix (RCA-5D) which resulted in 74.12% increase than that of plain RCA. Moreover, the RCA-3D, RCA-SY, and RCA-HY showed increase of 23.65%, 11%, and 8.3% respectively, compared to that of RAC.
5. The fiber addition improved the crack initiation, propagation and post cracking behaviour which led to ductile behaviour and different mode of failures.

**Effect of fiber addition on the shear capacity of RCA**

1. Average shear capacity of RCA was about 14.36% higher than that of NWA, which can be explained by the better aggregate interlock due to the various shapes and texture of the RCA. Furthermore, the normalized shear resistance \( \left( \frac{V_c}{\sqrt{f'c}} \right) \), has an increase of 19.56%, from 4.25 in NWA to 5.09 in RCA. Moreover, the average maximum deflection was 9.22 mm for plain RCA with respect to 8.5 mm in NWA beams with an increase of 8.41%.

2. Stiffness of the RCA beams was less than that of the NWA beams with about 19.26%.

3. Fiber reinforced beams have had an increase in the average shear capacity with about 64.48%, 59.23%, 23.44%, and 47% for the RCA-3D, RCA-5D, RCA-SY, and RCA-HY; respectively, compared to that of RAC. However, the normalized shear resistance showed an overall similar increase with a slight advantage towards the 3D mix, showing 8.15 for the RCA-3D, while the RCA-5D, RCA-SY, and RCA-HY showed about 7.41, 6.30, and 7.15; respectively.

4. The synthetic fiber had the lowest improvement in the average concrete shear capacity because the polypropylene fibers have less pull-out capacity compared to that of the steel fiber.

5. Fiber reinforced beams results had an increase in the deflection at ultimate with respect to plain RCA by about 57.89%, 56.59%, 14.98%, 27.89% for RCA-3D, RCA-5D, RCA-SY, and RCA-HY; respectively. Moreover, RCA showed deflection of 9.22 mm at ultimate load (81.7 kN); however, at the same load, fiber reinforced beams showed 6, 5.8, 7.84, 6.4 mm deflection for beams RCA-3D, RCA-5D, RCA-SY, RCA-HY; respectively.

6. The synthetic fiber showed the least improvement in deflection then hybrid fiber mix. These results illustrate the post-cracking behaviour which is the main characteristics of steel fibers especially and fibers in general.

7. The average stiffness for RCA-3D, RCA-5D, RCA-SY, and RCA-HY had an increase with respect to plain RCA of 11.96%, 8.38%, 30.73%, and 7.68%; respectively.
8. The RCA-3D and RCA-5D had the highest results in almost all tested parameters, and fairly close results to each other; however, RCA-3D showed a higher improvement in maximum shear capacity, maximum deflection, and normalized shear resistance; where the RCA-5D showed a higher improvement in the initial crack strength, deflection control, compressive strength, and flexural strength both (peak and the residual).

9. Synthetic fiber beams, RCA-SY, showed the least improvement in initial crack strength, ultimate deflection, and compressive strength; however, it showed that a slight improvement in split tensile that 3D steel fibers, first-peak flexural strength, normalized shear resistance than both 5D steel fibers and hybrid fiber mixes. In addition, it showed the highest improvement in stiffness to RCA mix which reached to an average of 30.73%.

10. The hybrid mix, RCA-HY, showed relatively mixed results due to its mixed contents of 50% of synthetic fibers and 50% of 5D steel fibers. The RCA-HY showed a slight increase in the mechanical properties except for split tensile which showed the highest improvement up to 141.16%. The 5D steel fiber content improved the maximum deflection, maximum shear capacity.

11. In comparison to proposed equations from literature, fib model 2010, Imam and Vandewalle, Oh et al. (1999), Kwak et ak. (2011), and Khuntia et al. (1999) showed relatively conservative results in predicting the shear strength in fiber-reinforced concrete beams.

6.2 Recommendation

This study showed the shear improvement and behaviour of fibers as reinforcement and without stirrups for recycled aggregate concrete. However, in future research the following on the subject should be considered:

1. Recycled aggregate to be obtained from different sources to give a better understanding on its behaviour.

2. Using different beams with various shear spans and cross-sections with fiber reinforcement to better understand the behaviour of fibers as shear reinforcement.
3. Using different combinations of synthetic and steel fibers with various percentages to get maximize the improvements in all aspects and behaviour of structural element.

4. Using recycled aggregate with different fiber percentages than 0.75% to better shape a comprehensive summary on the benefits of fiber-reinforcement.
References


*Building code requirements for structural concrete: (ACI 318-14) ; and commentary (ACI 318R-14)*. American Concrete Institute, 2014.


Appendix A

Sample photos from laboratory casting and testing

Aggregate physical properties testing

![Image A.1](a) absorption weight; (b) weight in water

Beam and sample preparation

![Image A.2](a) (b)
Figure A.2: (a) Beam moulds; (b) steel cage with strain gauges; (c) specimen moulds

*Casting*

(a)  
(b)  

Figure A.3: (a) RCA-SY mix; (b) spreading the fibers into the mix

*De-molding and curing*

(a)
Figures A.4: (a) De-moulding of beams and storage; (b) curing of specimens and storage; (c) de-molding of beams

**Testing**

*Cube Compressive Strength*

NWA:

Figures A.5: Compressive strength of normal weight concrete (a) specimen 1; (b) specimen 2
RCA:

Figure A.6: Compressive strength of recycled aggregate concrete (a) specimen 1; (b) specimen 2

RCA-3D:

Figure A.7: Compressive strength of recycled aggregate concrete with 3D steel fibers (a) specimen 1; (b) specimen 2

RCA-5D:

Figure A.8: Compressive strength of recycled aggregate concrete with 5D steel fibers (a) specimen 1; (b) specimen 2
RCA-SY:

Figure A.9: Compressive strength of recycled aggregate concrete with Synthetic fibers (a) specimen 1; (b) specimen 2

RCA-HY:

Figure A.10: Compressive strength of recycled aggregate concrete with Hybrid fibers (a) specimen 1; (b) specimen 2

Cylindrical Split Tensile

NWA:

Figure A.11: Split tensile cylinder of Normal weight concrete (a) specimen 1; (b) specimen 2
RCA:

Figure A.12: Split tensile cylinder of recycled aggregate concrete (a) specimen 1; (b) specimen 2

RCA-3D:

Figure A.13: Split tensile cylinder of recycled aggregate concrete with 3D steel fibers (a) specimen 1; (b) specimen 2

RCA-5D:

Figure A.14: Split tensile cylinder of recycled aggregate concrete with 5D steel fibers (a) specimen 1; (b) specimen 2
RCA-SY:

Figure A.15: Split tensile cylinder of recycled aggregate concrete with Synthetic fibers  (a) specimen 1; (b) specimen 2

RCA-HY:

Figure A.16: Split tensile cylinder of recycled aggregate concrete with Hybrid fibers  (a) specimen 1; (b) specimen 2
**Flexural Strength of prisms:**

![Figure A.17: Preparation of Flexural strength specimen](image1)

Figure A.17: Preparation of Flexural strength specimen

**NWA:**

![Figure A.18: Flexural strength prisms for normal weight concrete](image2)

Figure A.18: Flexural strength prisms for normal weight concrete

**RCA:**

![Figure A.19: Flexural strength prisms for recycled aggregate concrete](image3)

Figure A.19: Flexural strength prisms for recycled aggregate concrete
RCA-3D:

Figure A.20 : Flexural strength prisms for recycled aggregate concrete with 3D steel fibers

RCA-5D:

Figure A.21 : Flexural strength prisms for recycled aggregate concrete with 5D steel fibers

RCA-SY:

Figure A.22 : Flexural strength prisms for recycled aggregate concrete with Synthetic fibers
RCA-HY

Figure A.23: Flexural strength prisms for recycled aggregate concrete with Hybrid fibers

Beam Testing

Figure A.24: Beam under loading samples (a) RCA-HY-2; (b) RCA-3D-1; (c) RCA-5D-1; (d) RCA-SY-1

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Appendix B

Load vs Deflection in Mid span curves for all beams

NWA-1

Figure B.1 : Load deflection curve for NWA-1

Figure B.2 : Stiffness for NWA-1
Figure B.3: Load deflection curve for NWA-2

Figure B.4: Stiffness for NWA-2

$y = 10076x + 12218$
RCA-1:

Figure B.5: Load deflection curve for RCA-1

Figure B.6: Stiffness for RCA-1
RCA-2:

Figure B.7: Load deflection curve for RCA-2

\[ y = 9267.4x + 3683.9 \]

Figure B.8: Stiffness for RCA-2
RCA-3D-1:

Figure B.9: Load deflection curve for RCA-3D-1

Figure B.10: Stiffness for RCA-3D-1

\[ y = 9202.4x + 16691 \]
RCA-3D-2:

Figure B.11: Load deflection curve for RCA-3D-2

\[ y = 10177x + 16496 \]

Figure B.12: Stiffness for RCA-3D-2
RCA-5D-1:

Figure B.13: Load deflection curve for RCA-5D-1

Figure B.14: Stiffness for RCA-5D-1
RCA-5D-2:

Figure B.15: Load deflection curve for RCA-5D-2

Figure B.16: Stiffness for RCA-5D-2
RCA-SY-1:

Figure B.17: Load deflection curve for RCA-SY-1

Figure B.18: Stiffness for RCA-SY-1
RCA-SY-2:

Figure B.19: Load deflection curve for RCA-SY-2

Figure B.20: Stiffness for RCA-SY-2
RCA-HY-1:

Figure B.21: Load deflection curve for RCA-HY-1

![Load deflection curve](image1)

$y = 11781x + 9032.1$

Figure B.22: Stiffness for RCA-HY-1

![Stiffness curve](image2)
RCA-HY-2:

Figure B.23: Load deflection curve for RCA-HY-2

Figure B.24: Stiffness for RCA-HY-2
Appendix C

Load – Strain curves for all beams

Strain in steel bars and concrete surfaces are presented

NWA-1:

Figure C.1: NWA-1 strain gauges curves load versus steel

Figure C.2: NWA-1 strain gauges curves load versus concrete
NWA-2:

Figure C.3: NWA-2 strain gauges curves load versus steel

Figure C.4: NWA-2 strain gauges curves load versus concrete
RCA-1:

Figure C.5: RCA-1 strain gauges curves load versus steel

Figure C.6: RCA-1 strain gauges curves load versus concrete
Figure C.7: RCA-2 strain gauges curves load versus steel
RCA-3D-1:

Figure C.8: RCA-3D-1 strain gauges curves load versus steel

Figure C.9: RCA-3D-1 strain gauges curves load versus concrete
RCA-3D-2:

Figure C.10: RCA-3D-2 strain gauges curves load versus steel

Figure C.11: RCA-3D-2 strain gauges curves load versus concrete
RCA-5D-1:

Figure C.12: RCA-5D-1 strain gauges curves load versus steel

Figure C.13: RCA-5D-1 strain gauges curves load versus concrete
RCA-5D-2:

Figure C.14: RCA-5D-2 strain gauges curves load versus steel

Figure C.15: RCA-5D-2 strain gauges curves load versus concrete
RCA-SY-1:

Figure C.16: RCA-SY-1 strain gauges curves load versus steel

Figure C.17: RCA-SY-1 strain gauges curves load versus concrete
RCA-SY-2:

Figure C.18: RCA-SY-2 strain gauges curves load versus steel

Figure C.19: RCA-SY-2 strain gauges curves load versus concrete
RCA-HY-1:

Figure C.20: RCA-HY-1 strain gauges curves load versus steel

Figure C.21: RCA-HY-1 strain gauges curves load versus concrete
RCA-HY-2:

Figure C.22: RCA-HY-2 strain gauges curves load versus steel

Figure C.23: RCA-HY-2 strain gauges curves load versus concrete
Appendix D

Mode of failure and corresponding schematic diagrams

NWA-1:

Figure D.1 : Post-failure cracks in beam NWA-1

Figure D.2 : Schematic diagram post-failure cracks in beam NWA-1

NWA-2:

Figure D.3 : Post-failure cracks in beam NWA-2

Figure D.4 : Schematic diagram post-failure cracks in beam NWA-2
RCA-1:

Figure D.5 : Post-failure cracks in beam RCA-1

Figure D.6 : Schematic diagram post-failure cracks in beam RCA-1

RCA-2:

Figure D.7 : Post-failure cracks in beam RCA-2

Figure D.8 : Schematic diagram post-failure cracks in beam RCA-2
RCA-3D-1:

Figure D.9 : Post-failure cracks in beam RCA-3D-1

Figure D.10 : Schematic diagram post-failure cracks in beam RCA-3D-1

RCA-3D-2:

Figure D.11 : Post-failure cracks in beam RCA-3D-2

Figure D.12 : Schematic diagram post-failure cracks in beam RCA-3D-2
RCA-5D-1:

Figure D.13: Post-failure cracks in beam RCA-5D-1

Figure D.14: Schematic diagram post-failure cracks in beam RCA-5D-1

RCA-5D-2:

Figure D.15: Post-failure cracks in beam RCA-5D-2

Figure D.16: Schematic diagram post-failure cracks in beam RCA-5D-2
RCA-SY-1:

Figure D.17: Post-failure cracks in beam RCA-SY-1

![Crack Diagram](image1)

Figure D.18: Schematic diagram post-failure cracks in beam RCA-SY-1

RCA-SY-2:

Figure D.19: Post-failure cracks in beam RCA-SY-2

![Crack Diagram](image2)

Figure D.20: Schematic diagram post-failure cracks in beam RCA-SY-2
RCA-HY-1:

Figure D.21 : Post-failure cracks in beam RCA-HY-1

Figure D.22 : Schematic diagram post-failure cracks in beam RCA-HY-1

RCA-HY-2:

Figure D.23 : Post-failure cracks in beam RCA-HY-2

Figure D.24 : Schematic diagram post-failure cracks in beam RCA-HY-2
Appendix E

This section will present sample equations with their solutions.

- Narayanan and Darwish (1987)

\[ Vu = e \left[ 0.24f_{spfc} + 80\rho \frac{d}{a} \right] + v_b \]

\( f_{spfc} = \text{computed value of split-cylinder strength of fiber concrete} \)

\[ = \frac{f_{cuf}}{20 - \sqrt{F}} + 0.7 + 1.0 \sqrt{F} \quad \text{(MPa)} \]

e = arch action factor:

for \( a/d > 2.8 \)

2.8 \( d/a \) for \( a/d \leq 2.8 \)

\( f_{cuf} = \text{cube compressive strength of fiber concrete (MPa)} \)

\( F = \text{fiber factor} = (L_f/D_f) \times V_f \times d_f \)

\( L_f = \text{fiber length} \)

\( D_f = \text{fiber diameter} \)

\( V_f = \text{volume fraction of fibers} \)

\( d_f = \text{bond factor: 0.5 for round fibers, 0.75 for crimped fibers, and 1.0 for indented fibers} \)

\( v_b = 0.41\iota F, \text{where: } \iota: \text{average fiber matrix interfacial bond stress} \)

\( \Rightarrow \text{Example for 3D steel fibers} \)

\[ F = (35 \times 0.0075 \times 1)/0.55 \text{ (for 3D steel fibers); } f_{cuf} = 68 \text{ MPa; } f_{spfc} = 4.91; \ e = 1; \ \iota = 0.41 \text{ MPa; } d = 159 \text{ mm; } a = 500 \text{ mm; } \rho = 0.00142 \]

\( Vu = 2.35 \text{ MPa x 159 x 150 } / 1000 = 56.10 \text{ kN} \)

\( V_{exp} = 64.35 \text{ kN} \)

\( \Rightarrow \text{Ratio}(V_{exp}/Vu) = 1.15 \)

- Ashour et al. (1992)
For $a/d \geq 2.5$

$$Vu = \left(2.11 \sqrt[3]{f'_c} + 7F\right) \left(\frac{d}{a}\right)^{0.333}$$

For $a/d \leq 2.5$

$$Vu = \left(2.11 \sqrt[3]{f'_c} + 7F\right) \left(\frac{d}{a}\right)^{0.333} \frac{2.5}{a/d} + v_b \left(2.5 - \frac{a}{d}\right)$$

In our case $a/d = 3.15$, hence use first equation:

⇒ Example for 5D steel fibers

$F = (60 \times 0.0075 \times 1)/0.9$ (for 5D steel fibers); $f'_c = 77$ MPa; $\iota = 0.41$ MPa; $d = 159$ mm; $a = 500$ mm; $\rho = 0.00142$

$Vu = 2.51$ MPa $\times 159 \times 150 / 1000 = 59.81$ kN

$V_{exp} = 65.08$ kN

⇒ Ratio($V_{exp}/Vu$) = 1.09
• Kwak et al. (2002)

\[ Vu = A \varepsilon_{spfc}^{\exp1} \left( \frac{\rho d}{a} \right)^{\exp2} + 0.8 \nu_b^{\exp3} \]

\( e = 1.0 \) for \( a/d > 3.5 \)

3.5 x \( d/a \) (if \( a/d \leq 3.5 \))

For COV = 14.9%: \( A=2.1, B=0.8, \exp1=0.7, \exp2=0.22, \exp3=0.97 \), however, for COV =15.3%: \( A=3.7, B=0.8, \exp1=2/3, \exp2=1/3, \exp3=1 \)

Example for 3D steel fibers

Assume COV=14.9%; \( F = (35 \times 0.0075 \times 1)/0.55 \) (for 3D steel fibers); \( f_{cuf} = 68 \) MPa; \( f_{spfc} = 4.91; e = 1; \tau = 0.41 \) MPa; \( d = 159 \) mm; \( a = 500 \) mm; \( \rho = 0.00142 \)

\[ Vu = 2.84 \text{ MPa} \times 159 \times 150 / 1000 = 67.62 \text{ kN} \]

\[ V_{exp} = 64.35 \text{ kN} \]

\[ \Rightarrow \text{Ratio}(V_{exp}/Vu) = 0.95 \]
Mohamed Ghoneim was born in 1992, in Dubai, U. A. E. He studied his elementary and middle education in Al Dawha, and completed his high school in Sharjah American International School. He was awarded the Bachelor of Science in Civil Engineering from American University of Sharjah in Spring 2014.

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