Shear Behavior of Steel-Fiber Reinforced Ultra-High-Strength Self-Compacted Concrete Beams

Omar Jum'ah Za'al Al Rawashdeh

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United Arab Emirates University

College of Engineering

Department of Civil and Environmental Engineering

SHEAR BEHAVIOR OF STEEL-FIBER REINFORCED ULTRA-HIGH-STRENGTH SELF-COMPACTED CONCRETE BEAMS

Omar Jum‘ah Za‘al Al Rawashdeh

This thesis is submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

Under the Supervision of Professor Amr El-Dieb

December 2015
Declaration of Original Work

I, Omar Jum'ah Za'al Al Rawashdeh, the undersigned, a graduate student at the United Arab Emirates University (UAEU), and the author of this thesis entitled "Shear Behavior of Steel-Fiber Reinforced Ultra-High-Strength Self-Compacted Concrete Beams", hereby, solemnly declare that this thesis is my own original research work that has been done and prepared by me under the supervision of Prof. Amr El-Dieb, in the College of Engineering at UAEU. This work has not previously been presented or published, or formed the basis for the award of any academic degree, diploma or a similar title at this or any other university. Any materials borrowed from other sources (whether published or unpublished) and relied upon or included in my thesis have been properly cited and acknowledged in accordance with appropriate academic conventions. I further declare that there is no potential conflict of interest with respect to the research, data collection, authorship, presentation and/or publication of this thesis.

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Abstract

Ultra-high-strength concrete is a new class of concrete that has been the result of the progress in concrete material science and development. This new type of concrete is characterized with very high compressive strength; about 100 MPa. Ultra-high strength concrete shows very brittle failure behavior compared to normal-strength concrete. Steel fibers will significantly reduce the workability of ultra-high strength concrete. The development and use of self-compacting concrete has provided a solution to the workability issue. The combination of technology and knowledge to produce Ultra-High strength fiber reinforced self-compacting concrete was proved to be feasible. Few studies investigated the effect of incorporating steel fibers on the shear behavior of ultra-high-strength reinforced concrete beams.

The research consists of a test series and analytical investigation. The present research investigated the shear behavior of reinforced beams made of normal-strength-concrete fiber-reinforced self-compacting concrete (28 MPa), high-strength concrete fiber-reinforced self-compacting concrete (60 MPa) and ultra-high-strength fiber-reinforced self-compacting concrete (100 MPa). The test parameters included two different shear span-to-depth ratios of 2.22 (deep beam action) and 3.33 (slender beam action), and three different steel fiber volume fractions of 0.4%, 0.8%, and 1.2%. The test results showed that the shear strength gain ranged from 20% to 129% for the beams having a concrete grade of 28 MPa, 26% to 63% for the beams having a concrete grade of 60 MPa, and 8.6% to 94% for the beams with a concrete grade of 100 MPa. For the deep beams, the shear strength gain tended to decrease by increasing the concrete grade. For the slender beams with steel fiber volume fractions of 0.4% and 0.8%, varying the concrete grade had no obvious effect on the shear strength gain. For the
slender beams with the higher steel fiber volume fraction of 1.2%, the shear strength gain tended to decrease with an increase in the concrete grade.

In the analytical investigation, the accuracy and validity of published analytical models have been demonstrated. Predictions of analytical models by Ashour et al. (1992) and Narayanan et al. (1987) were in good agreement with the experimental results.

Keywords: Ultra-high-strength concrete, self-compacting, steel fibers, shear behavior, slender beam, deep beam.
سلوك القص للجسور الخرسانية المصنوعة من الخرسانة ذاتية الديمك والمسلحة بالألاف من الصلب

المختصر

الخراشة فائقة المقاومة هي فئة جديدة من الخرسانة التي كانت نتاجًا للتقدم والتنمية في علوم المواد بعده للخرسانة. ويميز هذا النوع الجديد من الخرسانة بمقاومة ضغط عالية جداً (حوالي 00 ميجا باسكال). الخرسانة فائقة المقاومة تميز بالانهيار المفاجئ مقارنة مع الخرسانة ذات المقاومة عادية، وبالتالي تظهر سلوك محدود بعد حدوث الشروك. إضافة الألوف المصنوعة من الصلب : قدرة الخرسانة على التشكيل وبالتالي تحسن سلوك الخرسانة بعد حدوث الشروك. وفق المصنوعة من الصلب تقلل بشكل ملحوظ قابلية التشغيل للخرسانة فائقة المقاومة. تأثير دمج الصلب والمسلحة. العلاقة بين مقاومة القص، ومحوت الألوف المصنوعة من الصلب، ومحوت الألوف المصنوعة من الجسور المنسوجة في عمق الجسر، ونسبة التسليح الرئيسي في الجسر تحتاج إلى مزيد من الدراسة.

وقد ثبت أن لجمع بين التكنولوجيا والمعرفة لإنتاج الخرسانة فائقة المقاومة السليقة بالألوف المصنوعة من الصلب والمسلحة ذاتية الديمك أصبح ممكنًا. عدد قليل من الدراسات قام بدراسة ذلك الخرسانة ذاتية الديمك على سلوك القص للجسور المنسوجة. هذه الدراسة تشير إلى أن التسليح الرئيسي في الجسر يحتاج إلى مزيد من الدراسة. يمكن فهم الموضوع بشكل شامل، وتكوين هناك قدرة على تصميم الجسور السليقة ذاتية الديمك بالألوف المصنوعة من الصلب والمسلحة ذاتية الديمك.

في هذا البحث سيتم دراسة سلوك القص للجسور المصنوعة من الخرسانة ذاتية الديمك والمسلحة بالألوف المصنوعة من الصلب والمسلحة عادية المقاومة. المسلحة بالألوف المصنوعة من الصلب والمسلحة ذاتية الديمك، والمسلحة عادية المقاومة والخراشة عادية المقاومة، وكذلك الخراشة ذاتية الديمك، ومسلحة بالألوف المصنوعة من الصلب والمسلحة ذاتية الديمك، ومقارنة بينها. يشمل هذا البحث سلسلة الاختبارات العملية ودراسة تحليلية للعوامل التي سيتم دراستها في هذا البحث بالإضافة لمقاومة الضغط للخرسانة، وهي نسبة بحر القص إلى عمق الجسر، ومحتوى الألوف المصنوعة من الصلب. وسيتم استخدام نتائج الاختبارات للقيام بدراسة تحليلية في وضع مبادئ توجيهية لتصميم هذا النوع من المواد. وتركز الدراسة التحليلية على قياس تأثير محتوى الألوف المصنوعة من الصلب على مقاومة القص للخرسانة ذاتية الديمك ومسلحة بالمسلحة.
ذاتية الدمك المسلحة بالألعاب المصنوعة من الصلب. وستعرض النتائج العامة والخلاصة لهذا العمل جنبًا إلى جنب مع توصيات لقيام ببحوث ودراسات في المستقبل.

مفهوم البحث الرئيسية: الخرسانة فائقة المقاومة المسلحة بالألعاب المصنوعة من الصلب والخرسانة ذاتية الدمك، سلوك القص، الألواح المصنوعة من الصلب، نسبة بحر القص إلى عمق الجسر.
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I would like to thank God for giving me the faith and strength to successfully complete this work. I would also like to express my sincere thanks to my family who have provided me with all the support and strength to complete this work.

I would like to express my deepest thanks to all individuals who helped me during this significant period of my life. At the first place, I would like to deliver my deepest respect and appreciation to my thesis supervisors Dr. Amr El-Dieb and Dr. Tamer El Maaddawy for their continuous support, inestimable guidance, and the valuable knowledge they provided me throughout the project. I would like to thank them for the friendly environment they have created for me and the brotherly advice I received from them.

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Dedication

To my beloved parents and family
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<tr>
<td>$a/d$</td>
<td>Shear Span to effective Depth Ratio</td>
</tr>
<tr>
<td>$b$</td>
<td>Concrete Beam Width</td>
</tr>
<tr>
<td>D</td>
<td>Deep Beam</td>
</tr>
<tr>
<td>$d$</td>
<td>The Effective Depth of the Beam</td>
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<tr>
<td>$d_f$</td>
<td>Bond factor: 0.50 for round fibers, 0.75 for crimped fibers, and 1.00 for indented fibers</td>
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<tr>
<td>$D_f$</td>
<td>Steel Fiber Diameter</td>
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<tr>
<td>$f'_c$</td>
<td>Cylinder Concrete Compressive Strength</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Concrete Splitting Tensile Strength</td>
</tr>
<tr>
<td>$h$</td>
<td>Concrete Beam Height</td>
</tr>
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<td>HSC</td>
<td>High Strength Concrete</td>
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<tr>
<td>HS-FR-SCC</td>
<td>High Strength Fiber Reinforced Self Compacting Concrete</td>
</tr>
<tr>
<td>$k$</td>
<td>Factors used in several equations (different for different equations)</td>
</tr>
<tr>
<td>$L$</td>
<td>Concrete Beam Length</td>
</tr>
<tr>
<td>$L_f$</td>
<td>Steel Fiber Length</td>
</tr>
<tr>
<td>$L_fD_f$</td>
<td>Steel Fiber Aspect Ratio</td>
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<tr>
<td>NSC</td>
<td>Normal Strength Concrete</td>
</tr>
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<td>NS-FR-SCC</td>
<td>Normal Strength Fiber Reinforced Self Compacting Concrete</td>
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<tr>
<td>$P$</td>
<td>Applied Shear force</td>
</tr>
<tr>
<td>$P_{cr}$</td>
<td>Cracking Shear force</td>
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<tr>
<td>$P_u$</td>
<td>The Ultimate Shear Force</td>
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<td>R</td>
<td>Reaction Force</td>
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<td>RC</td>
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<td>S</td>
<td>Slender Beam</td>
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<td>S</td>
<td>Stirrups Spacing</td>
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<td>SCC</td>
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<td>SF</td>
<td>Steel Fiber</td>
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<td>SFRC</td>
<td>Steel Fiber Reinforced Concrete</td>
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<td>UHSC</td>
<td>Ultra-High Strength Concrete</td>
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<tr>
<td>UHS-FR-SCC</td>
<td>Ultra-High Strength Fiber Reinforced Self Compacting Concrete</td>
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<tr>
<td>$V$</td>
<td>Shear Force</td>
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<tr>
<td>$V_a$</td>
<td>The Shear Strength due to Aggregate Interlock</td>
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<tr>
<td>$V_c$</td>
<td>Concrete Contribution to the Shear Strength</td>
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<tr>
<td>$V_{cz}$</td>
<td>The Shear strength from the compression zone</td>
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<tr>
<td>$V_d$</td>
<td>The Shear Strength due to Dowel Action</td>
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<td>$V_{sf}$</td>
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<tr>
<td>$v_f$</td>
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<td>$V_u$</td>
<td>Ultimate Shear Strength</td>
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<td>$\Delta_{cr}$</td>
<td>The Deflection at the Cracking Shear Force</td>
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<tr>
<td>$\Delta_u$</td>
<td>The Deflection at the Ultimate Shear Force</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>Main Reinforcement Ratio</td>
</tr>
<tr>
<td>$\rho_{st}$</td>
<td>Shear Reinforcement Ratio</td>
</tr>
<tr>
<td>$F$</td>
<td>Fiber Factor = \left( \frac{f_t}{d_f} \right) V_f d_f</td>
</tr>
<tr>
<td>$e$</td>
<td>Arch Action Factor</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
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<td>-------------</td>
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<tr>
<td>( \tau )</td>
<td>Average Fiber Matrix Interfacial Bond Stress, taken as 4.15 MPa, based on the recommendations of Swamy, Mangat, and Rao.</td>
</tr>
<tr>
<td>( \psi )</td>
<td>Size Effect Factor</td>
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<td>( \omega )</td>
<td>Reinforcement Factor</td>
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Chapter 1: Introduction

1.1 Overview

The advance in concrete materials and technology in the last 30 years has far surpassed that made through the previous 150 years. With various material used in concrete mixes to improve the concrete characteristics such as, super-plasticizing admixtures, supplementary cementing materials, and the most recent addition to concrete mixes is the fibers.

1.2 Ultra-High Strength Fiber-Reinforced Self Compacting Concrete (UHS-FR-SCC)

1.2.1 Ultra-High Strength Concrete (UHSC)

The Ultra-high strength concrete (UHSC) is a new class of concrete that has been the result of such development. High strength concrete (HSC) and UHSC definition has been used interchangeable in the past. In more recent years, the definition of these two classes of concrete has been changed to adapt to the new limit that the concrete strength was able to reach. This new type of concrete (UHSC) is characterized with very high compressive strength; higher than 100 MPa. UHSC has been used recently in some high rise building and long span bridges all over the world. There are some disadvantages with the use of UHSC which is that it shows very brittle failure behavior compared to normal-strength concrete (NSC) and therefore a limited post-crack behavior. Also, UHSC fails explosively without any warning signs (Bencardino, Rizzuti, Spadea, & Swamy, 2008).
1.2.2 Steel Fiber Reinforced Concrete (SFRC)

Another example of the improvement in concrete technology in the latter years is the use of steel fibers. The use of steel fibers (SF) in the concrete mixes has gained huge popularity in the construction industry in the last decade due to the improvement in the concrete properties after its addition to the mix. Studies have demonstrated that the addition of SF can enhance many of the concrete properties such as, ductility, tensile resistance, fracture toughness, and crack control (Graybeal, 2007; Köksal, Altun, Yiğit, & Şahin, 2008; El-Dieb, 2009; Sivakumar & Santhanam, 2007). The use of steel fiber with (UHSC) can reduce the brittleness of the concrete and improve the post peak behavior of the mix. But one of the disadvantages of adding steel fiber to the concrete mix is that it remarkably reduce the workability of the mix. (Sivakumar & Santhanam, 2007)

1.2.3 Self-Compacting Concrete (SCC)

Self-compacted concrete is another class of concrete that was also developed in the last 30 years. Self-compacted concrete is a concrete that flows and compacts under its own weight with no need of mechanical or manual compaction. The self-compacted concrete was originally developed to assure that concrete will pass through congested reinforcement and to fill the formwork were the regular concrete can not. The ability of SCC to flow easily and increase the workability of the concrete mixes gave a solution for resolving the workability issue that face the mixes with SF such as ultra-high-strength fiber-reinforced concrete (UHS-FRC).

The use of Steel Fiber (SF) with Self Compacting Concrete (SCC) in combination with ultra-high-strength-concrete (UHSC) is used to develop what is
known as ultra-high-strength fiber-reinforced self-compacting concrete (UHS-FR-SCC). Several studies showed that the development of such a mix is practicable (El-Dieb, 2009).

1.3 Shear Strength

Reinforced concrete (RC) element is designed to assure that the element will fail in a ductile mode of failure and will provide warning before failure, but the issue with shear failure that it is a brittle type of failure and fails without omen. This failure will be much more critical in case UHSC was used which is a very brittle type of concrete. So the addition of steel fiber could introduce a solution to this problem. The presence of steel fiber in the concrete mixes and its ability to increase the ductility and crack control to the concrete mixes allowed the development of the concept of replacing stirrups with steel fiber in the concrete mix. This idea was deducted from the enhancement that happens to the concrete mix such as the increase in the compressive strength, tensile strength, and the change in the failure mode to a more favorable ductile type of failure. (Batson, 1972; Narayanan & Darwish, 1987)

Shear behavior of reinforced concrete elements is exceptionally unpredictable in nature, and with the addition of SF determining the shear strength of the structural elements is extremely difficult. Study shows that the addition of steel fiber to Reinforced Concrete (RC) will affect the shear behavior and strength of RC beams (Kang T. H.-K., Kim, Massone, & Galleguillos, 2012; Juárez, Valdez, Durán, & Sobolev, 2007; Hanai, 1997).

These investigations concluded that the main parameters influencing the shear behavior and strength of RC beams made with steel fiber-reinforced concrete are:
- Shear span-to-depth ratio \((a/d)\)
- Concrete compressive strength \((f'_c)\)
- Steel fiber volume fraction \((v_f)\)
- Main reinforcement ratio \((\rho_s)\)

Most studies concluded that using steel fiber with volume ratio less than 0.75% will not contribute significantly to the shear behavior of beams (Juárez, Valdez, Durán, & Sobolev, 2007; Hanai, 1997; Mangiavillano & Campione, 2008; Altun, Haktanir, & Ari, 2007; H.H. Dinh, 2010; Kang t. H.-K., Kim, Kwak, & Hong, 2011). Also, the minimum main reinforcement ratio should be higher than conventionally reinforced members in order to achieve sufficient ductility (Dancygier & Savir, 2006). It was found that it is feasible to combine steel fibers and minimum shear reinforcement to achieve the shear strength of RC beams and improve its ductility (Oh, 1999; Cucchiarra, La Mendola, & Papia, 2004).

Very few attempts have been made to study the effect of using SF in the shear strength of UHSC RC beams, and self-compacting concrete (SCC). The interaction between the shear capacity, steel fiber content, shear span to depth ratio \((a/d)\), and the transverse shear reinforcement ratio \((\rho_{sh})\) needs more investigation in order to establish comprehensive understanding, and to be able to design RC beams with UHS-FR-SCC.

### 1.4 Purpose of the study

The aim of this study was to investigate the shear response of ultra-high-strength fiber reinforced self-compacting concrete (UHS-FR-SCC) beams with different shear span to depth ratios \((a/d)\). The impact of varying the steel fiber volume on the shear response is investigated. The shear behavior of the UHS-FR-SCC beams was compared with that of similar beams made with normal strength concrete (NSC).
and high-strength concrete (HSC). The beam size, fiber type and longitudinal reinforcement ratio were kept unchanged.

### 1.5 Organization of the work

The present research work investigates, experimentally and analytically, the effect of adding steel fiber on shear behavior of reinforced concrete beams without stirrups for NSC, HSC, and UHSC.

A literature review on shear behavior and strength of steel fiber reinforced concrete beams with NSC, and HSC is presented in Chapter (2). The research objectives and significance concludes the chapter.

Chapter (3) provides detailed information on the experimental program, test matrix which includes grouping of specimens, specimen dimensions, geometry, and fabrication. It also includes information on materials properties, concrete mix proportions for NS-FR-SCC, HSC-FR-SCC, and UHCS-FR-SCC. A full description of the test set-up, instrumentation, control, and load details procedure are presented in the same chapter.

Chapter (4) presents results of the experimental testing and observations. The results include shear force vs. deflection curves, shear force vs. diagonal tensile displacement, shear force vs. compressive strain, failure modes, and Shear capacity. Discussions and comments relevant to the results are included also in this chapter.

In Chapter (5), the accuracy and validity of various analytical approaches by different publications in the literature are examined. A comparison between experimental and analytical results is presented and discussed.
Chapter (6) summarizes the general conclusions of the work along with recommendations for future studies and developments on performance of RC elements that was built with UHS-FR-SCC.
Chapter 2: Literature Review

2.1 Introduction

This chapter includes a review of the available literature on shear behavior of fiber reinforced concrete beams. The studies represented here discuss the effect of different factors affecting the behavior of shear strength of steel fiber reinforced concrete beams.

2.2 Studies on UHS-FRC

(Naaman, 2003) has comprehended his research to engineer a new type of steel fibers to achieve the optimum properties for reinforcing the cement composites. The fibers are engineered to achieve optimal properties in terms of shape, size, and mechanical properties, as well as compatibility with a given matrix. They are identified as torex fibers. These new fibers will increase the use of high performance fiber reinforced cement composites in structural applications. The author studied the effect of various shape of steel fiber such as hooked, smooth and torex (the engineered new type of fiber) on high performance and ultra-high performance concrete. Torex shows better performance in comparison to other steel fibers. The study concluded that increasing the lateral surface area with the same cross section will increase the bond strength in the fiber as a result increasing its effectiveness. Another conclusion from the study was twisting the fiber will increase bonding strength since it will increase the later surface area.

(Thomas & Ramaswamy, 2007) have studied the mechanical properties of steel fiber-reinforced concrete. The study presented the results from an experimental program and analytical results on the influence of addition of fibers on mechanical
properties of concrete. The mechanical properties studied were cube and cylinder compressive strength, split tensile strength, modulus of rupture and post cracking performance, modulus of elasticity, Poisson's ratio, and strain corresponding to peak compressive stress. The grades of concrete adopted were 38 MPa, 65 MPa and 85 MPa and the volume fraction of the fiber $\nu = 0.0, 0.5, 1.0,$ and 1.5%. The test results were compared with analytical results and were found to be convincing with data reported in the literature. The study revealed that the fiber matrix interaction play a vital role in improvement of mechanical properties caused by the introduction of fibers.

(El-Dieb, 2009) studied the mechanical properties, durability and micro-structural characteristics of UHS-FR-SCC using local materials from the Gulf region. The concrete characteristics that were studied are compressive strength and splitting tensile strength. Also, flow-ability of concrete was tested to assure the workability of concrete after the addition of SF using slump flow test. For the durability, the rapid chloride permeability test, concrete electrical resistivity, and bulk diffusion test was carried out to assess the durability of the UHS-FR-SCC. The results of this study showed the possibility and the feasibility of producing UHS-FR-SCC. The results also showed an increase in the mechanical properties of the concrete especially the splitting tensile strength. The results of the concrete electrical resistivity test shows that the total electrical charge passing through concrete and the electrical conductivity of the concrete is increased but still low to assure a good protection to reinforcement. The addition of steel fiber did not make any significant change in bulk chloride diffusion and water sorptivity.
2.3 Studies of shear behavior of SFRC beams

(Nemkumar, 2002) studied the direct shear test of fiber-reinforced concrete. Two 50 mm-long steel fibers, one with flattened ends and a circular cross section and the other with a crimped geometry and a crescent cross section, were investigated at fiber volume fractions varying between 0 and 2%. Direct comparison was made with flexural toughness determined as per the ASTM C 1018 procedure. It was found that both fibers provided significant improvements in shear strength as well as shear toughness and these improvements were greater at higher fiber dosage rates. Between the two fibers, the fiber with flattened ends was seen to be more effective than the one with crimped geometry. For the flattened-end fiber, an almost linear increase in the shear strength was noted with an increase in the fiber volume fraction. For the fiber with crimped geometry, on the other hand, shear strength approached a plateau value beyond which no increases in shear strength occurred with an increase in the fiber volume fraction. While plain concrete failed at a low equivalent shear strain of 0.4%, fiber-reinforced concrete supported as high as 10% strain in shear. When the shear toughness of steel fiber-reinforced concrete was compared with its flexural toughness, there appeared to be a direct correlation. However, given the subjectivity of this type of comparison and the limited data generated in this study, much further research is needed to fully understand and establish this correlation.

(Kwak, Eberhard, Kim, & Kim, 2002) conducted experimental and analytical investigations on the effect of steel fiber on shear strength of reinforced concrete. Twelve tests were performed on reinforced concrete beams in this study. The variables considered were steel fiber volume (0%, 0.5%, and 0.75%), shear span to depth ratio (2, 3, and 4) and concrete compressive strength (31, 65 MPa). The results showed that
as the fiber content increased, the ultimate shear force and deflection capacity increased. The inclusion of steel fiber changed the mode of failure. For the beam with the lower a/d ratio of 2, the addition of steel fiber changed the mode of failure to a ductile multi-cracked shear-flexure or flexure mode of failure. Shear strength gain in the range (69 to 80%) was noticed for these beams. On other hand, for higher values of shear span to depth ratio (a/d = 3, 4), the increase in strength was relatively low (22% to 38%) since these beams failed in flexure. Increasing the concrete compressive strength from 31 to 65 MPa resulted in an average increase in shear strength in the range of (22 to 26%). The analytical investigation was to assess and to develop new equation. Results of the four beams which failed in shear or a combination of shear and flexure were considered in the analytical investigation. The analytical study included also results from 139 tests reported in the literature. Variables of the analytical investigation included shear span to depth ratio (a/d = 1 to 5), concrete compressive strength ($f'_c = 21$ to 112 MPa), flexural reinforcement ratio ($\rho = 1.1$ to 5.7), steel fiber volume fraction ($v_f = 0.22$ to 2%) and beam depth ($d = 102$ to 570 mm).

It was concluded that the equation proposed by Narayanan and Darwish (1987) and the equation developed in this study were the most accurate equations used to estimate the shear strength of steel fiber reinforced concrete (SFRC) beams.

(Dinh, Parra-Montesinos, & Wight, 2010) investigated the shear behavior of SFRC beams. The study examined whether steel fibers can be used instead of shear reinforcement in beams. Twenty eight beams were constructed and tested. All beams had a shear span to depth ratio of $a/d = 3.5$ and concrete compressive strength of $f'_c = 41$ MPa. Test parameters included type of fibers, fiber volume fraction, longitudinal reinforcement ratio, and beam depth. Three types of fibers were used. Fiber type 1 was
30 mm long, with an aspect ratio of 55 and a tensile strength of 1100 MPa. Fiber type 2 was 60 mm long, with an aspect ratio of 80 and a tensile strength of 1100 MPa. Fiber type 3 was 30 mm long, with an aspect ratio of 80 and a tensile strength of 2300 MPa.

For the fiber volume fraction, three volume fractions were used (0.75, 1.0, and 1.5%). Three levels of longitudinal reinforcement ratios were used (1.6, 2.0, and 2.7%). The beam depth was either 455 mm or 685 mm. The results showed that RC beams without steel fibers or transverse reinforcement failed suddenly in brittle manner due to formation of one diagonal crack (diagonal tension failure). For RC beams with transverse reinforcement, the failure mode observed was still brittle (diagonal tension) although some enhancement was noticed regarding the crack pattern. For SFRC, the failure mode was usually a combination of shear tension and shear compression, or a combination of shear tension and diagonal tension, or a combination of shear compression and diagonal tension. Although the failure was somehow sudden for the SFRC beams, multiple diagonal cracks were observed. Moreover, the test results showed that the use of steel fiber increased the shear strength. The shear strength gain was significant when the steel fiber volume fraction was 0.75%. The increase in the shear strength was insignificant when the volume fraction of the steel fiber was greater than 1%. The SFRC with type 2 (60 mm long) exhibited a higher shear strength gain relative to that exhibited by other SFRC beams. Because fibers of type 2 are longer than the other types of fibers, they reduced concrete workability which resulted in congestion of fibers in the mix. The shear strength of the SFRC beams was high enough to replace the minimum requirement of ACI code for shear reinforcement. In addition, the study suggested some recommendations for possible inclusion in the ACI code 318-08 and to whether accept the steel fiber as shear resistance.
(Yakoub, 2011) modified CSA A23.3-04 and modified Bazant and Kim equations to better predict the shear strength of SFRC beams. The study investigated the accuracy and the ability of five other equations from the literature to predict the shear strength of SFRC beams. In order to accomplish this task, the study analyzes 218 SFRC beams with no stirrups and 72 reinforced concrete beams with stirrups and no steel fibers. The variables were longitudinal reinforcement ratio ($\rho$), concrete compressive strength ($f'_c$), steel fiber volume fraction ($\nu_f$), steel fiber aspect ratio ($L_f/D_f$), steel fiber geometry (the steel fiber geometry included hooked, crimped, round...etc.) and shear span to depth ratio ($a/d$). The analysis showed that as the longitudinal reinforcement ratio ($\rho$), concrete compressive strength ($f'_c$), steel fiber aspect ratio ($L_f/D_f$) and steel fiber volume fraction ($\nu_f$) increased the SFRC beam shear strength increased. The shear strength increased as the shear span to depth ratio ($a/d$) decreased. Moreover, the author developed a new factor called "absolute reduction factor". This factor was used to compare between the considered equations. Among all the considered seven equations, the two equations developed by the author were the most accurate ones. Also, the modified CSA A23.3-04 equation considered the strain effect for short beam in SFRC beams. Results of SFRC beam with crimped fiber were more efficient than those of the beam with hooked fibers. This was attributed to the possibility of hooked fiber to form balls in the mix (especially for high percentage of volume fraction $\nu_f$) which could reduce the shear strength.

(Aoude, Belghiti, Cook, & Mitchell, 2012) conducted an experiment to study the shear behavior of SFRC beams. The experiment included nine full scale beams. The beams were divided into three series. Two series included different beam sizes to examine the size effect on the shear strength of SFRC beam. Beams of the third series
included steel fibers and stirrups. The test results showed that the addition of steel fibers in the concrete mix increased the shear strength of SFRC beams. The beam with the smaller size (Series A) required fewer amounts of steel fibers (about 1%) to change the mode of failure from brittle shear failure to a ductile flexural failure. For the larger specimens (series B), this amount was not enough to change the mode of failure from brittle shear failure to a ductile flexural failure. For the beams with web reinforcements, the inclusion of the steel fibers did not result in an increase in the shear capacity but there was an improvement in post-peak response and ductility. The use of steel fibers enhanced the crack distribution and reduced crack width which provided some warning before failure. Besides, the study provides a solution to predict the shear strength in SFRC beams. An analytical solution for shear strength prediction of SFRC beams was also proposed in this study. A comparison with equations published in the literature was conducted.

(Cucchiara, Mendola, & Papia, 2004) investigated the experimental and analytical impact of steel fibers and stirrups on shear response. Sixteen beams were prepared and divided in two series. Series A for $a/d = 2.8$ and series B for $a/d = 2.0$. The other variables in this study were volume fraction of steel fibers and stirrups spacing. The volume fraction of steel fibers has three levels (0%, for 1% and for 2%). The stirrups has also three levels (no stirrups, $s = 200 \text{ mm}$, $s = 600 \text{ mm}$). The results showed that the concrete compressive test for plain concrete or fiber reinforced concrete were very similar to each other up to maximum stress (peak stress) but in the post peak region the behavior is very different and shows a moderate fall (more ductile behavior) in fiber reinforced concrete and dramatic fall in plain concrete. Also, the results from the splitting test showed a significant increase in the maximum shear force
as the fiber content increase. The tests on beams showed that the beam effect governs the A series and the arch action governs the B series. The beams in B series showed more brittle behavior than beams in A series. The inclusion of steel fibers has greater impact on the A series (slender beams) than on B series (deep beam). For beams with no steel fibers or stirrups or with low levels of steel fiber or stirrups, the crack pattern shows that one major crack (diagonal crack) was the reason for the beam failure. For beam with steel fibers or stirrups, the crack pattern shows that many cracks were there and progressively increased in number and width as the shear force increased. The inclusion of steel fibers or stirrups or both can change the failure mode from brittle to a more ductile one but the steel fiber effect on series A was greater than that on series B. The stirrups ruptures in A series and B series when spacing was 200 mm. When steel fibers and stirrups where used together the stirrups did not rupture. Equations from the literature were used to calculate the shear strength of fiber reinforced concrete.

(Lim & Oh, 1999) studied the mechanical behavior of reinforced concrete beams that contains fiber reinforcement under shear and the potential use of them as shear reinforcement. Nine beams were constructed and tested for this study. the test variables are the volume fraction of steel fibers and the amount of shear stirrups. The results showed that the compressive strength, flexural strength and splitting strength increased by 25%, 55% and more than 100% respectively when steel fiber volume ratio was 2%. The shear cracks in beams with no stirrups or steel fiber appeared with very low shear force values. Also, the failure of these beams where very rapid and sudden type of failure. The beams with steel fibers showed higher shear stresses and more ductility. the beams with 50% stirrups and 1% steel fiber contents exhibited a
flexural failure which could be the turning point to change the mode of failure from shear to flexural one. The shear force-deflection curve of test beams shows a linear behavior until the formation of the first crack. And after that the behavior of the beams was nonlinear. The ultimate strength of beams with 1% steel fibers contents increased significantly comparing with beams that has no steel fibers. The ultimate strength of beams with steel fibers and stirrups increased but not as significant as beams with steel fibers only. The cracking shear strength increased significantly with the addition of steel fibers. The study proposed an analytical method to calculate the shear strength of reinforced concrete with steel fibers.

(Santos, Barros, & Lourenço, 2008) studied the effect of fibers in high strength reinforced concrete to increase their shear strength. The study included 24 slab strips. The slab strips is (800 x 170 x 150 mm³). The variables in this study were concrete compressive strength (50, 70) and steel fibers dosage (0, 60 and 75 kg/m³). For each combination of these variables, 4 slab strips were cast, two with longitudinal reinforcement and two without longitudinal reinforcement. The test results showed that the steel fiber increased the serviceability limit state by a range of 43% and up to 72%, and maximum shear force carrying capacity by a range of 80% and up to 118%. The results showed that in the post cracking stage the SFRC showed increased in ductility and higher shear force carrying capacity. The equation proposed by RILEM TC 162-TDF committee was also used to show the increase in shear capacity in SFRC.

(Furlan & Hanai, 1997) studied the effect of fiber on reinforced concrete beams. The experiment included fourteen beams tested the shear response of these beams. The variables in this study were the fiber volume ratio and whether the steel stirrups are used or not. All the concrete mixes used were identical except fiber volume
ratio. Seven mixes were prepared for the fourteen beams. Each mix was used in two beams. The difference between the two beams that one has stirrups and the other one did not have stirrups. Shear span to depth ratio was 3.5 for all the specimens. Although the fiber decreased the workability of the fresh concrete, a small increase in the tensile strength of the concrete and in the modulus of elasticity was noted in the hardened concrete due to this addition of fibers. The shear strength was increased for all specimens that had fibers within them. The increase in shear strength was between 7.5% to 17% for the beams with stirrups and fibers. The increase in shear strength was much more significant for the beams without stirrups. For those beams the increase in shear strength was between 9% to 37%. The addition of fibers increased the cracking control. The crack patterns at the end of the testing were more intense for all the beams. The crack patterns at the end of the test for the beam with 2% steel fiber and without stirrups were similar to the beams with stirrups and without the addition of fibers. The addition of fibers in the beams increased its ductility. The increase in ductility was major for beam with 2% steel fiber and without stirrups. The inclusion of fibers increased the stiffness and reduces the deflection for all beams with fibers. The beams with fiber showed that maximum stirrups stress was less compared to the beams without stirrups. Also, it showed that stirrups contribution to the shear resistance was delayed due to the inclusion of fiber in the beams (the stress in the stirrups started at a very high shear force value in comparison to the beams without fibers).

(Jua'rez, Valdez, Durán, & Sobolev, 2007) studied the shear failure of fiber reinforced concrete beams with the inclusion of stirrups. The study included 16 beams. The beams cross section is (150x250) mm² and the length of the beams is 2000 mm. The variables were the concrete compressive strength (Group A = 36.7 MPa and Group
B = 18.9 MPa) and steel fiber ratio (0%, 0.5%, 1.0% and 1.5%). Two identical beams were casted for each combination in order to confirm the results of the experiment. The results showed that the inclusion of steel fibers increased the energy absorption, ductility and shear strength of beams. The inclusion of steel fiber affects the shear strength mainly by increasing the first crack shear strength. The shear strength of FRC increased by 54% for beams in group B with 1.5% steel fiber volume ratio, and increased by 12% for beams in group A with 1.5% steel fiber volume ratio comparing to the control beams. The increase in shear strength of FRC for group B and group A in comparison with ACI-318 code was 17% and 30% respectively, although the strength reduction factor was not consider in these calculations. Also, the number of cracks in beams with steel fibers increased, hence it was very clear that the width of the diagonal cracks were reduced when steel fibers were used.

(Mutsuyoshi & Janaka Perera, 2013) study the shear response of reinforced high strength concrete (f'_c > 100 MPa) without web reinforcement. Twelve beams were constructed for the study. The variables in this study were f'_c, a/d and concrete additives. The results showed that the shear force dropped slightly when the flexural cracks appeared and then continue to rise. The shear force dropped significantly when the first diagonal crack formed but also the shear force rose again after that. The shear force also dropped moderately afterwards when other diagonal cracks formed. But the shear force kept increasing till the beam failed in shear compression when the diagonal cracks widened and the concrete crushed in the compression zone. The results also showed that the increase of concrete compressive strength decreased the shear resistance of aggregate interlock. The increase of concrete compressive strength from 36 MPa to 114 MPa increased the diagonal cracking shear strength of only 11%. The
normalized shear strength ($T_{eff}$) was used to compare the shear strength of different concrete compressive strength. It was found out that the concrete compressive strength is inversely proportional to the normalized shear strength and directly proportional to the brittleness index.

(Minelli & Plizzari, 2013) studied the effect of steel fiber on shear behavior of large scale beams. Eighteen beams were tested for this study. Concrete strength, fiber volume ratio, fiber type and mixture of different steel fibers are the variable used to test the effectiveness of steel fiber as shear reinforcement. The experiment was divided into six series. Series from (1 to 4) had the same size (4450 x 200 x 480) mm$^3$ and reinforcement ratio (1.04%). Series from (1 to 4) consisted of eleven beams. The variable in these beams were the concrete strength and the types of steel fiber and the steel fiber volume ratio. For series five, 3 beams were cast. The beam size was (2400 x 200 x 500) mm$^3$ and the reinforcement ratio was (0.99%) for this series was different than the four previous series. The variable in this series were the types of steel fiber and the steel fiber volume ratio. The last series consisted of 3 beams. The beam size is (4600 x 200 x 1000) mm$^3$ and the reinforcement ratio is (1.03%) for this series. The variable in this series were the concrete strength and the types of steel fiber and the steel fiber volume ratio. The shear span to depth ratio was the same for all six series which is 2.5. For the first three series (all normal strength concrete (NSC)), the small addition of fiber increased the shear force carrying capacity by at least twice that of the reference beams. The deflection and the stiffness increased significantly for the same amount of steel fiber. The shear force at which the crack initiated was much higher for the FRC than that of the reference beam. Also, the maximum shear crack width for FRC beams was about ten times larger than that of the reference beam. For
Series 4 (High Strength Concrete (HSC)), the addition of fiber increased the shear force carrying capacity by at least 70% comparing to the reference beams. The shear force carrying capacity for those beams was almost equal to the full flexural capacity of the beam. Also, there was a huge increase in the mid-span deflection compared to the reference beams. The crack configurations for HSC-FRC consist of a number of minor diagonal cracks and not one major diagonal crack. Also, the shear force at which the crack initiated was higher for the FRC than that of the reference beam. Also, the maximum shear crack width for HSC-FRC beams was about ten times larger than that of the reference beam. Series 5 and 6 showed similar results to the previous series even that these series contained large-scale beams. Two models were used in this study to evaluate the effect of steel fiber on shear strength of fiber reinforced concrete. Both models used gave a reasonable estimate to the shear strength.

(Noghabai, 2001) has investigated the possibility of using Steel fibers as shear reinforcement in high strength concrete beams. A test was conducted for twenty beams of different dimensions with diverse types of shear reinforcement. The study concluded that the steel fiber with volume ratio of 1% could replace regular shear reinforcement (i.e. stirrups) and achieve the same shear capacity for beam with relatively small size (effective depth = 200 mm)

(Voo, Poon, & Foster, 2010) study was one of the fewest studies that studied the effect of SF on shear strength of ultra-high performance fiber reinforced concrete beams. This article reports the outcome of a testing program on ultra-high performance steel fiber reinforced concrete members. Eight pre-stressed concrete beams were tested in shear with the test parameters being the (a/d) and the SF volume ratio and type of SF. The finding of the tests, together with additional tests found in the literature, are
weigh against the numbers derived from the PSM-VEM model to establish the shear strength of steel fiber reinforced concrete beams. A good correlation is detected with a mean model to experimental strength ratio of 0.92 and coefficient of variation of 0.12.

2.3.1 Research Significance

Many papers studied the effect of SF on normal and high strength concrete but very few papers studied this effect on ultra-high strength concrete. To fully comprehend the complex behavior of shear in UHSC, the present research work studied the effect of SF with various volume ratios. The research work also considers the different behavior between slender beam and deep beam in regard to mode of failure and cracking patterns. The results of the tests compared the beam with stirrups with the beams with SF and discussed the possibilities of using SF as a replacement of shear reinforcements.
Chapter 3: Experimental Program

3.1 Introduction

The aim of the study is to examine the effect of different steel fiber volume ratio with different concrete compressive strength on the shear behavior of beams with different shear span to effective depth ratio. However, beam size, fiber type and longitudinal reinforcement ratio were left unchanged.

Experimental program of the present work consists of Thirty tests on fifteen reinforced concrete beams. The investigated parameters are the compressive strength of concrete, the steel fiber volume ratio, and the shear span to depth ratio (a/d).

The details of the experimental program, test specimens description, material properties, detail of reinforcements, trial concrete mixes, and fabrication process are provided in this chapter.

3.2 Test Program

A test matrix was developed to study the effect of using different steel fiber volume ratio with different concrete compressive strength on the shear behavior of beams with different shear span to depth ratio. The test matrix was divided into three groups based on the concrete compressive strength. The test matrix is shown in Table 3.1.
<table>
<thead>
<tr>
<th>Group</th>
<th>Beam Type</th>
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<td></td>
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<td>1.2%</td>
<td>S28-VF3</td>
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<tr>
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<tr>
<td></td>
<td></td>
<td>0.0%</td>
<td>S100-VF0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4%</td>
<td>S100-VF1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8%</td>
<td>S100-VF2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.2%</td>
<td>S100-VF3</td>
</tr>
<tr>
<td></td>
<td>Deep ($a/d = 2.2$)</td>
<td>0.0%</td>
<td>D100-VF0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4%</td>
<td>D100-VF1</td>
</tr>
</tbody>
</table>

Table 3.1: Test matrix
Where:

- S = slender beam
- D = deep beam
- $f'_c$ = concrete compressive strength (MPa)
- $v_f$ = steel fiber volume ratio
- $V F0$ = no steel fiber in the mix
- $V F1$ = steel fiber volume ratio in the concrete mix = 0.4%
- $V F2$ = steel fiber volume ratio in the concrete mix = 0.8%
- $V F3$ = steel fiber volume ratio in the concrete mix = 1.2%

3.2.1 Group A

Group A consisted of ten tests on five beams. Concrete compressive strength for this group was 28 MPa. The RC beam was designed in such a way that it will act as two specimens. The beam tested from one end as a deep beam, and then tested as a slender beam from the other end as shown in the specimen details. Three different levels of steel fiber volume were used in this group (0.4%, 0.8%, and 1.2%). The beam that has 0% of steel fibers is considered to be the control beam. Also, a control beam with transverse reinforcement (i.e. stirrups) was tested to compare the results with the beams that had only steel fibers.

3.2.2 Group B

Group B consisted of ten tests on five beams. Concrete compressive strength for this group was 60 MPa. The RC beam was designed in such a way that it will act as two specimens. The beam tested from one end as a deep beam, and then tested as a
slender beam from the other end as shown in the specimen details. Three different levels of steel fibers volume were used in this group (0.4%, 0.8%, and 1.2%). The beam that has 0% of steel fiber is considered to be the control beam. Also, a control beam with transverse reinforcement (i.e. stirrups) was tested to compare the results with the beams that had only steel fibers.

3.2.3 Group C

Group C consisted of ten tests on five beams. Concrete compressive strength for this group was 100 MPa. The RC beam was designed in such a way that it will act as two specimens. The beam tested from one end as a deep beam, and then tested as a slender beam from the other end as shown in the specimen details. Three different levels of steel fibers volume were used in this group (0.4%, 0.8%, and 1.2%). The beam that has 0% of steel fiber is considered to be the control beam. Also, a control beam with transverse reinforcement (i.e. stirrups) was tested to compare the results with the beams that had only steel fibers.

3.3 Specimen Details

The test specimen was reinforced concrete beam with three meter long \( (L = 300 \text{ cm}) \), the total height \( (h) \) is 22 cm and the width \( (b) \) is 12 cm. The effective depth of the beam \( d = 17.8 \text{ cm} \). Concrete cover was 1 cm.
3.3.1 Steel Detail

3.3.1.1 Test specimens without stirrups

The beams were designed to fail due to shear failure prior to the flexural failure. Four bars No. 20 were used as bottom main steel to resist the moment produced from the shear force. The bottom steel bars were hooked upwards behind the support and enclosed by two stirrups No. 10 to prevent anchorage failure. The top steel was 2 bars No. 20. The shear reinforcement outside the test regions was No. 8 with a spacing $S = 10$ cm to assure that the shear failure will not occur outside the test region. Figure 3.1 shows details of test specimen without stirrups for slender beams. Figure 3.2 shows details of test specimen without stirrups for deep beams.

![Test region for slender beam](image1)

![Test region for deep beam](image2)

Figure 3.1: Test specimen without stirrups for slender beams.
3.3.1.2 Test specimens with stirrups

The beams were designed to fail due to shear failure prior to the flexural failure. Four bars No. 20 were used as bottom main steel to resist the moment produced from the shear force. The top steel was 2 bars No. 20. The shear reinforcement outside the test regions was No. 8 with a spacing $S = 10\, \text{cm}$ to assure that the shear failure will not occur outside the test region. These beams include transverse reinforcement (i.e.
stirrups) in the test region. The stirrups used were No. 6 with spacing 10 cm as shown in Figure 3.4 and Figure 3.3.

Figure 3.4: Test specimen with stirrups for slender beams

Figure 3.3: Test specimen with stirrups for deep beams
3.3.2 Strain Gauge Detail

Electrical resistance strain gauges (S.G.) were bonded to the tensile steel reinforcement under the applied shear force, to the tensile steel reinforcement at the mid-shear span and to the compressive steel reinforcement under the applied shear force for all tested beams. Figure 3.5 shows the strain gauges locations for beams without stirrups.

The designation in Figure 3.5 shows the exact locations of strain gauge and is explained as below:

- For deep beam test region:

  A) 1 strain gauge will be placed at one bar in the extreme tension fiber. (under shear force)

Figure 3.5: Strain gauge locations for slender and deep beams without stirrups respectively
B) 1 strain gauge will be placed at one bar in the extreme tension fiber. (mid-shear span)

C) 1 strain gauge will be placed at one bar in the extreme compression fiber. (under shear force)

- For slender beam test region:

D) 1 strain gauge will be placed at one bar in the extreme tension fiber. (under shear force)

E) 1 strain gauge will be placed at one bar in the extreme tension fiber. (mid-shear span)

F) 1 strain gauge will be placed at one bar in the extreme compression fiber. (under shear force)
Figure 3.6 shows the gauge locations for beam with stirrups. The designation in Figure 3.6 shows the exact locations of strain gauge and is explained as below:

- For deep beam test region:
  
  D) 3 strain gauges will be placed at three stirrups in the middle of the shear span.

- For slender beam test region:

  H) 3 strain gauges will be placed at three stirrups in the middle of the shear span.
Figure 3.7 shows steel cages with strain gauges fixed to the reinforcing steel. Fixed strain gauges were covered with electrical plastic tape to protect the gauges from being damaged during concrete casting.

![Figure 3.7: Steel cages after installing the tensile strain cages.](image)

Also, electrical resistance strain gauges were used to measure the diagonal compressive strain in concrete in the test region. The strain gauge was bonded to the beam at point (A) shown in Figure 3.8 for deep beam and Figure 3.9 for slender beam. The strain gauge was fixed parallel to the dashed line. The dashed line is connecting the applied shear force to the reaction for the case of the deep beam, while for the slender beam the dashed line is 45° and passing through the shear mid-span point. A clip gauge is used to measure the shear crack width that passed parallel to dashed lines. The clip gauge was placed to the beam at point (A) shown in Figure 3.8 for deep beam and Figure 3.9 for slender beam. The clip gauge was placed perpendicular to the dashed line.
Figure 3.8: Concrete strain gauge location for deep beam

Figure 3.9: Concrete strain gauge location for slender beam
3.3.3 Testing Detail

RC beams were designed to have 2 test regions and therefore each beam act as 2 specimens. Each beam was tested twice, once as slender beam and once as deep beam. Figure 3.10 shows the dimensions for testing the beam as slender beam where \((a/d) = (60/180) = 3.33\). Figure 3.11 shows the dimensions for testing the beam as deeps beam where \((a/d) = (40/180) = 2.22\).

Figure 3.10: Test Detail for slender beam \((a/d = 3.3)\)

Figure 3.11: Test Detail for deep beam \((a/d = 2.2)\)
3.4 Specimen Fabrication

According to the specimen details that were shown in the earlier sections, steel bars were cut, bended, fabricated and fixed together to produce the required steel cages. Also, Formworks were prepared using plywood 18 mm thick and the formworks dimensions as mentioned in specimen details section as shown in Figure 3.12.

![Figure 3.12: Steel Cages and formwork](image)

Concrete was casted in the forms after steel cages were installed inside the forms as shown in Figure 3.13. All specimens were removed from the wooden formwork after 48 hours as shown in Figure 3.14. The specimens were cured for 7 days.

![Figure 3.13: Specimens before removal of formwork](image)
days. Wet hessian was wrapped around the specimens with polythene sheet in top for curing purposes as shown in Figure 3.15.

Figure 3.14: Specimens after removal of formwork

Figure 3.15: Curing of specimens
3.5 Material Properties

3.5.1 Steel Reinforcement

The longitudinal steel reinforcement was No. 20 deformed bars with nominal yield strength of 520 MPa. The shear reinforcement outside the test region was No. 8 with nominal yield strength of 520 MPa. The shear reinforcement used in the test region was plain bars with measured yield strength of 333 MPa and a diameter of 5.5 mm.

3.5.2 Steel Fiber

Dramix RC-65/35-BN were used as steel fiber which are manufactured by Bekaert cooperation. Steel fiber is hooked at its end as shown in Figure 3.16 and has a nominal tensile strength of 1150 N/mm². Moreover, the steel fiber has a length of 35 mm and a diameter of 0.55 mm which makes the aspect ratio \((L_f/D_f)\) equals 64.

Figure 3.16: Steel fiber
3.5.3 Concrete

In this study, three mixes were used with cylindrical compressive strength of 28 MPa, 60 MPa, and 100 MPa. The materials used for all mixes included ordinary Portland cement (OPC, Type I), local sand, medium crushed stone aggregate (10 mm), and polycarboxylic ether Type G admixture (S.P). The concrete mix proportions by weight and percentage for all mixes were as follows:

3.5.3.1 Grade of 28 MPa.

The following table shows the mix proportions for grade 28 MPa. (Table 3.2)

<table>
<thead>
<tr>
<th>By wt. (kg)</th>
<th>388</th>
<th>12.4</th>
<th>472</th>
<th>472</th>
<th>757</th>
<th>209</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ratio</td>
<td>1.0</td>
<td>3.2%</td>
<td>1.21</td>
<td>1.21</td>
<td>1.95</td>
<td>0.54</td>
<td>0.5%</td>
</tr>
</tbody>
</table>

Table 3.2: Mix Proportions for grade 28 MPa

The concrete self-compatibility was tested using slump flow test, and T50 and compared to EFNARC values as shown in Table 3.3.

<table>
<thead>
<tr>
<th>SF Volume Ratio</th>
<th>0.0%</th>
<th>0.4%</th>
<th>0.8%</th>
<th>1.2%</th>
<th>EFNARC Acceptance Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump Flow (mm)</td>
<td>790</td>
<td>740</td>
<td>700</td>
<td>670</td>
<td>600 to 800</td>
</tr>
<tr>
<td>T50</td>
<td>1.7</td>
<td>2.5</td>
<td>3.8</td>
<td>4.9</td>
<td>2 to 5</td>
</tr>
</tbody>
</table>

Table 3.3: Grade 28 MPa SCC tests
3.5.3.2 Grade of 60 MPa.

The following table shows the mix proportions for grade 60 MPa. (Table 3.4)

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Cement</th>
<th>Silica fume</th>
<th>Coarse Sand</th>
<th>Dune Sand</th>
<th>Coarse Agg. (10 mm)</th>
<th>Water</th>
<th>S.P</th>
</tr>
</thead>
<tbody>
<tr>
<td>By wt. (kg)</td>
<td>460</td>
<td>20</td>
<td>492</td>
<td>492</td>
<td>835</td>
<td>152</td>
<td>8.2</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.0</td>
<td>4.4%</td>
<td>1.07</td>
<td>1.07</td>
<td>1.81</td>
<td>0.33</td>
<td>1.8%</td>
</tr>
</tbody>
</table>

Table 3.4: Mix Proportions for grade 60 MPa

The concrete self-compatibility was tested using slump flow test, and $T_{50}$ and compared to ENARC values as shown in Table 3.5. It is important to note that additional superplasticizer was added to the mixes with SF.

<table>
<thead>
<tr>
<th>SF Volume Ratio</th>
<th>0.0%</th>
<th>0.4%</th>
<th>0.8%</th>
<th>1.2%</th>
<th>EFNARC Acceptance Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump Flow (mm)</td>
<td>740</td>
<td>700</td>
<td>660</td>
<td>620</td>
<td>600 to 800</td>
</tr>
<tr>
<td>$T_{50}$</td>
<td>1.9</td>
<td>2.7</td>
<td>4.0</td>
<td>4.9</td>
<td>2 to 5</td>
</tr>
</tbody>
</table>

Table 3.5: Grade 60 MPa SCC tests

3.5.3.3 Grade of 100 MPa.

The following table shows the mix proportions for grade 100 MPa. (Table 3.6)

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Cement</th>
<th>Silica fume</th>
<th>Coarse Sand</th>
<th>Dune Sand</th>
<th>Coarse Agg. (10 mm)</th>
<th>Water</th>
<th>S.P</th>
</tr>
</thead>
<tbody>
<tr>
<td>By wt. (kg)</td>
<td>561</td>
<td>99</td>
<td>470</td>
<td>253</td>
<td>927</td>
<td>152</td>
<td>16.5</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.0</td>
<td>17.6%</td>
<td>0.83</td>
<td>0.45</td>
<td>1.65</td>
<td>0.27</td>
<td>3%</td>
</tr>
</tbody>
</table>

Table 3.6: Mix Proportions for grade 100 MPa
The concrete self-compatibility was tested using slump flow test, and T50 as shown in Table 3.7. It is important to note that additional superplasticizer was added to the mixes with SF.

<table>
<thead>
<tr>
<th>SF Volume Ratio</th>
<th>0.0%</th>
<th>0.4%</th>
<th>0.8%</th>
<th>1.2%</th>
<th>EFNARC Acceptance Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump Flow (mm)</td>
<td>710</td>
<td>670</td>
<td>630</td>
<td>600</td>
<td>600 to 800</td>
</tr>
<tr>
<td>T50&lt;sub&gt;sec&lt;/sub&gt; (Sec)</td>
<td>2.4</td>
<td>2.8</td>
<td>3.8</td>
<td>5.0</td>
<td>2 to 5</td>
</tr>
</tbody>
</table>

Table 3.7: Grade 100 MPa SCC tests

For grades 28 MPa, 60 MPa, and 100 MPa, three different fiber volume fraction of steel fibers were used (0.4%, 0.8%, and 1.2%). The corresponding quantity for these \( v_f \) are (31.4, 62.8, and 94.2) kg per 1 m\(^3\) of concrete respectively. For each specimen, two small size cylinders (100 mm x 200 mm) were casted to measure the concrete compressive strength \( f'_{c} \). Also, for each specimen another two large size cylinder (150 mm x 300 mm) were casted to measure the indirect splitting tensile strength \( f_t \).
Chapter 4: Experimental Results

4.1 Introduction

Numerous studies concentrated on the impact of SF on normal and high strength concrete, however not very many papers examined this impact on ultra-high strength concrete. The current work considered the impact of SF with different volume content on UHSC. Also, the distinctive behavior between slender beams and deep beams was considered in this study. The results of the tests will contrast the beams with stirrups with the beams with SF and the potential outcomes of utilizing SF as a substitution of transverse reinforcement.

This chapter provides the results of the experimental program. The test results are shown for each group (as in Table 4.1) separately, and each group is divided to two subgroups (slender and deep). For each group, results of slender beams and deep beams are given separately. For slender beams and deep beams, the results include shear force-deflection, shear force-diagonal tensile displacement, shear force-concrete diagonal compressive strain, and the beam mode of failures. The concrete compressive strength and the concrete split strength are also shown in Table 4.1. The concrete compressive strength did not change significant with the addition of SF so an average value was taken. While for the concrete split strength was changed with the addition of SF.
<table>
<thead>
<tr>
<th>Group</th>
<th>Beam Type</th>
<th>(\nu)</th>
<th>Name</th>
<th>(f_r) (MPa)</th>
<th>(f_c) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A ((f_c = 28 \text{ MPa}))</td>
<td>Slender ((a/d = 3.3))</td>
<td>0.0%</td>
<td>S28-VF0-St</td>
<td>1.63</td>
<td>34.52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.0%</td>
<td>S28-VF0</td>
<td>1.63</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4%</td>
<td>S28-VF1</td>
<td>1.96</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8%</td>
<td>S28-VF2</td>
<td>2.61</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.2%</td>
<td>S28-VF3</td>
<td>3.84</td>
<td></td>
</tr>
<tr>
<td>Deep ((a/d = 2.2))</td>
<td>0.0%</td>
<td>D28-VF0-St</td>
<td>1.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.0%</td>
<td>D28-VF0</td>
<td>1.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.4%</td>
<td>D28-VF1</td>
<td>1.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.8%</td>
<td>D28-VF2</td>
<td>2.61</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2%</td>
<td>D28-VF3</td>
<td>3.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B ((f_c = 60 \text{ MPa}))</td>
<td>Slender ((a/d = 3.3))</td>
<td>0.0%</td>
<td>S60-VF0-St</td>
<td>2.06</td>
<td>61.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.0%</td>
<td>S60-VF0</td>
<td>2.06</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4%</td>
<td>S60-VF1</td>
<td>2.98</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8%</td>
<td>S60-VF2</td>
<td>3.50</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.2%</td>
<td>S60-VF3</td>
<td>3.50</td>
<td></td>
</tr>
<tr>
<td>Deep ((a/d = 2.2))</td>
<td>0.0%</td>
<td>D60-VF0-St</td>
<td>2.06</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.0%</td>
<td>D60-VF0</td>
<td>2.06</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.4%</td>
<td>D60-VF1</td>
<td>2.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.8%</td>
<td>D60-VF2</td>
<td>3.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2%</td>
<td>D60-VF3</td>
<td>3.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C ((f_c = 100 \text{ MPa}))</td>
<td>Slender ((a/d = 3.3))</td>
<td>0.0%</td>
<td>S100-VF0-St</td>
<td>2.92</td>
<td>95.14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.0%</td>
<td>S100-VF0</td>
<td>2.92</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4%</td>
<td>S100-VF1</td>
<td>3.47</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8%</td>
<td>S100-VF2</td>
<td>3.82</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.2%</td>
<td>S100-VF3</td>
<td>4.83</td>
<td></td>
</tr>
<tr>
<td>Deep ((a/d = 2.2))</td>
<td>0.0%</td>
<td>D100-VF0</td>
<td>2.92</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.4%</td>
<td>D100-VF1</td>
<td>3.47</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.1: Test matrix
4.2 Test Results – Group A (28 MPa)

4.2.1 Slender Beams

4.2.1.1 Shear Capacity

The main test results are summarized in Table 4.2. The ultimate load \( P_u \) is the maximum load force that the beam could hold. Table 4.2 also shows the deflection at the maximum load that the beam could hold \( \Delta_u \). The shear in the test region was equal to the reaction from the adjacent support and is calculated using the following equation (4.1):

\[
V = R = \frac{L-a}{L} P
\]

Where:

- \( V \) = Shear Force;
- \( R \) = Reaction Force;
- \( P \) = Applied Shear force;
- \( a \) = Shear Span; and
- \( L \) = Total span of the beam.

The values of the shear strength component (Concrete \((V_c)\), Transverse reinforcement \((V_t)\) and steel fiber \((V_{sf})\)) that contribute to the total shear strength are shown in Table 4.2. The Concrete contribution to the shear strength \((V_c)\) is calculated from the control specimen (S28-VF0) where the only strength in this specimen is the concrete shear strength \((V_u = V_c)\). The transverse reinforcement contribution to the
shear strength ($V_s$) for each group (A, B, or C) is calculated using the following equation ($V_s = V_u - V_c$) where ($V_c$) value is the same value that explained above. The steel fiber contribution to the shear strength ($V_{sf}$) is calculated in a similar manner to the transverse reinforcement contribution to the shear strength ($V_{sf} = V_u - V_c$). The last column is showing the shear gain due to the addition of steel fiber to the concrete mix.

The increase in shear strength for the following specimens (S28-VF1, S28-VF2, and S28-VF3) is 20.3%, 48.6% and 128.8% respectively. ($P_u$) for specimen S28-VF2 and S28-VF3 is higher than that of specimen S28-VF0-St which indicates the possibilities of replacing the transverse reinforcement (Stirrups) with steel fiber as shear reinforcements.

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_u$ (KN)</th>
<th>$V_u$ (KN)</th>
<th>$\Delta u$ (mm)</th>
<th>$V_c$ (KN)</th>
<th>$V_s$ (KN)</th>
<th>$V_{sf}$ (KN)</th>
<th>Shear strength Gain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S28-VF0-St</td>
<td>96.6</td>
<td>72.5</td>
<td>10.9</td>
<td>52.7</td>
<td>19.8</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S28-VF0</td>
<td>70.2</td>
<td>52.7</td>
<td>9.7</td>
<td>52.7</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S28-VF1</td>
<td>84.5</td>
<td>63.3</td>
<td>9.8</td>
<td>52.7</td>
<td>-</td>
<td>10.7</td>
<td>20.3%</td>
</tr>
<tr>
<td>S28-VF2</td>
<td>104.4</td>
<td>78.3</td>
<td>8.3</td>
<td>52.7</td>
<td>-</td>
<td>25.6</td>
<td>48.6%</td>
</tr>
<tr>
<td>S28-VF3</td>
<td>160.7</td>
<td>120.5</td>
<td>15.0</td>
<td>52.7</td>
<td>-</td>
<td>67.8</td>
<td>128.8%</td>
</tr>
</tbody>
</table>

Table 4.2: Test Results for slender beams in group (A)
4.2.1.2 Deflection Response

Figure 4.1 shows the shear force-deflection curves for slender beams in group (A). It is evident that the addition of steel fibers increased the shear resistance and energy absorption of the specimens. The inclusion of steel fibers or stirrups did not significantly change the stiffness of the beams. The specimens featured a quasi-linear shear force deflection behavior until the first major diagonal crack formed (not the peak shear force for S28-VF0, S28-VF0-St and S28-VF1). The first major diagonal cracked formed for S28-VF0, S28-VF0-St and S28-VF1 was at shear value equals to 40.1, 71.6 and 54.1 KN respectively. After the major crack, the stiffness of those specimens was reduced (up to maximum shear capacity) causing the deflection to increase significantly with the increase of the shear force. As the $\psi$ increased, it was observed that the shear force causing major diagonal crack approaches the ultimate shear capacity as in S28-VF2 and S28-VF3. The shear force-deflection curve shows

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![Shear Force-Deflection curves of slender beams in group (A)](image-url)
that the shear capacity increased with the inclusion of steel fibers or stirrups. It is also important to highlight that maximum shear force for specimen S28-VF2, S28-VF3 were higher than that of S28-VF0-St. For example, the maximum shear capacity for S28-VF0, S28-VF1, S28-VF2, S28-VF3 and S28-VF0-St was 52.7, 63.3, 78.3, 120.5 and 72.5 KN respectively. The shear force dropped significantly after the maximum shear force for all specimens and after that, the deflection increased dramatically. The maximum deflection was increased with the inclusion of steel fibers or stirrups. It is also important to highlight that maximum deflection for specimen S28-VF3 was higher than that of S28-VF0-St.

4.2.1.3 Diagonal Tensile Displacement

The Shear force vs. diagonal tensile displacement curves for specimens of slender beams in group (A) are depicted in Figure 4.2. From this figure, it can be seen that the specimens did not exhibit any significant diagonal displacement till the initiation of the cracks. The graph shows that the control specimen, S28-VF0, showed signs of diagonal cracking at a shear value of about 40.1 kN. The presence of transverse reinforcement did not increase the initiation of shear crack ($V^* = 39.9$ KN); however the inclusion of steel fiber increased the shear force needed for crack initiation. The shear value when the crack was initiated for the beams with steel fiber was 54.1, 64.7 and 67.5 KN for S28-VF1, S28-VF2 and S28-VF3 respectively. It is important to highlight that shear value when the crack was initiated for specimen S28-VF1, S28-VF2, and S28-VF3 were higher than that of S28-VF0 and S28-VF0-St. In the post-cracking stage, the diagonal tensile displacement started to increase after the initiation of the diagonal cracks for all the specimens. It was observed that for beams (S28-VF3 and S28-VF0-St) the rate of increase of diagonal displacement was reduced due to the
inclusion of steel fiber ($\nu_f = 1.2\%$) or stirrups relative to the control beam (S28-VF0).

For example, at tensile displacement of value 1 mm the shear value for the following specimen S28-VF0, S28-VF1, S28-VF2, S28-VF3 and S28-VF0-St was 35.3, 58.6, 78.3, 92.2 and 63.5 KN which shows that the rate of increase for S28-VF2 and S28-VF3 was less than that of S28-VF0-St.

![Graph](image)

Figure 4.2: Shear Force-Diagonal tensile displacement curves of slender beams of group A
4.2.1.4 Diagonal Compressive Strain in Concrete

Figure 4.3 shows the shear force vs. diagonal compressive strain in concrete for specimens of slender beams in group (A). From this figure, it can be seen that the pre-cracking stage the rate of increase of compressive strain was low for all the specimens. Also, it can be seen that diagonal compressive strain started to increase after the initiation of the diagonal cracks for all the specimens. The rate of increase was higher for all specimens after the crack initiated, but for specimen (S28-VF0) the rate was very high and the behavior was a very plastic one in comparison of all other specimens. For example, at strain value equals to 300, the shear value for S28-VF0, S28-VF1, S28-VF2, S28-VF3 and S28-VF0-St was 37, 59, 73, 54 and 39 respectively which shows that the addition of SF reduces the rate of increase of compressive strain.

The compression strain gauges for the specimen S28-VF0, S28-VF1, S28-VF2, S28-VF3, S28-VF0-St
VF3 and S28-VF0-St failed soon after the initiation of the crack due to the fact that crack passed through the strain gauge.

4.2.1.5 Mode of failure

The shear failure in general starts with few vertical flexural cracks formed in the tension side under the applied load. As the load increased, generally the diagonal cracks appeared at the mid-height of beam within the clear shear span in the direction of the main strut and propagated (almost horizontally) toward the loading point and toward the support. As the load increased more, existing cracks widened and increased in length. The following types of shear failures were observed: diagonal tension, and web crushing. For the diagonal tension failure, an inclined crack appeared at mid-height of the beam and propagated toward the loading point and the support. For web crushing failure, the concrete crushed in the mid-height of the beam in the shear span.

Failure modes of slender beams in group (A) are shown in Figures 4.4. All beams for this category showed a classical diagonal tension mode of failure, except S28-VF3 and S28-VF0-St. S28-VF3 failed due to web-crushing shear mode of failure due to spalling of concrete. S28-VF0-St failed due to formation of multiple diagonal cracks. The addition of the steel fibers in specimen (S28-VF3) restricted growth and widening of the shear cracks developed in the shear span, and hence allowed the specimen to develop its full shear capacity.
Figures 4.4: Failure modes of slender beams in group (A)
4.2.2 Deep Beam

4.2.2.1 Shear Capacity

The main test results are summarized in the following Table 4.3. The ultimate shear force ($P_u$) is the maximum load force that the beam could hold. Table 4.3 also shows the deflection at the maximum load that the beam could hold ($\Delta u$). The shear in the test region is as in equation (4.1). The three columns next to the last one in the table show the values of the shear component (Concrete ($V_c$), Transverse reinforcement ($V_s$) and steel fiber ($V_{sf}$)) that contribute to the total shear strength. The Concrete contribution to the shear strength ($V_c$) is calculated from the control specimen (D28-VF0).

The increase in shear strength for the following specimens (D28-VF1, D28-VF2, and D28-VF3) is 23.2%, 128.4% and 110.1% respectively. ($P_u$) for specimen D28-VF2 and D28-VF3 is higher than that of specimen D28-VF0-St which indicates the possibilities of replacing the transverse reinforcement (Stirrups) with steel fiber as shear reinforcements.

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_u$  (KN)</th>
<th>$V_u$  (KN)</th>
<th>$\Delta u$ (mm)</th>
<th>$V_c$ (KN)</th>
<th>$V_s$ (KN)</th>
<th>$V_{sf}$ (KN)</th>
<th>Shear Gain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D28-VF0-St</td>
<td>135.0</td>
<td>108.0</td>
<td>5.4</td>
<td>64.0</td>
<td>44.0</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>D28-VF0</td>
<td>80.0</td>
<td>64.0</td>
<td>3.2</td>
<td>64.0</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>D28-VF1</td>
<td>98.6</td>
<td>78.8</td>
<td>6.1</td>
<td>64.0</td>
<td>-</td>
<td>14.8</td>
<td>23.2%</td>
</tr>
<tr>
<td>D28-VF2</td>
<td>182.8</td>
<td>146.2</td>
<td>9.8</td>
<td>64.0</td>
<td>-</td>
<td>82.2</td>
<td>128.4%</td>
</tr>
<tr>
<td>D28-VF3</td>
<td>168.1</td>
<td>134.5</td>
<td>8.5</td>
<td>64.0</td>
<td>-</td>
<td>70.5</td>
<td>110.1%</td>
</tr>
</tbody>
</table>

Table 4.3: Test Results of deep beam in Group (A)
4.2.2.2 Deflection Response

Figure 4.5 shows the shear force-deflection curves for deep beams in group (A). It is evident that the addition of steel fibers increased the shear resistance and energy absorption of the specimens. The inclusion of steel fibers or stirrups did not significantly change the stiffness of the beams. The specimens featured a quasi-linear shear force deflection behavior up to the maximum shear force. It was observed that the shear force causing the major crack coincides with the ultimate shear capacity for all specimens. The shear force-deflection curve shows that the shear force carrying capacity increased with the inclusion of steel fibers or stirrups. It is important to highlight that maximum shear strength for specimen D28-VF2, and D28-VF3 were higher than that of D28-VF0-St. For example, the shear capacity for the control beam (D28-VF0) was 64 KN. For the beam with stirrups (D28-VF0-St) the shear capacity was 108.0 KN. The shear capacity for the beams with steel fiber was 78.8, 146.2 and
134.5 KN for D28-VF1, D28-VF2 and D28-VF3 respectively. The maximum deflection was increased with the inclusion of steel fibers or stirrups. It is also important to highlight that maximum deflection for specimen D28-VF3 was higher than that of D28-VF0 and D28-VF0-St.

4.2.2.3 Diagonal Tensile Displacement

The shear force vs. diagonal tensile displacement curves for specimens of deep beam in group (A) are depicted in Figure 4.6. From this figure, it can be seen that the specimens did not exhibit any significant diagonal displacement till the initiation of the cracks. The graph shows that the crack initiated at very low shear value for the control beam (D28-VF0) in comparison to the other specimens with the stirrups or steel fiber. According to visual inspection the crack initiated at very low shear value (approximately ≈ 16 KN). The shear force value when the crack initiation for the beams with steel fiber and stirrups was 55.7, 57.5, 56.1 and 48.6 KN for D28-VF1, D28-VF2, D28-VF3 and D28-VF0-St respectively.
D28-VF2, D28-VF3 and D28-VF0-St respectively. It is important to highlight that shear value when the crack initiated for specimen D28-VF1, D28-VF2, D28-VF3 were higher than that of D28-VF0-St. In the post-cracking stage, the diagonal tensile displacement started to increase after the initiation of the diagonal cracks for all the specimens. Also, the rate of increase of diagonal displacement across cracks was reduced in beams with stirrups or steel fiber in comparison to the control beam (D28-VF0). The reduction in the rate of increase of diagonal displacement across cracks was almost the same for the following beams (D28-VF2, D28-VF3) ($v_f = 0.8\%$ and $1.2\%$) while for the beam (D28-VF0-St) the reduction in the rate of increase of diagonal displacement across cracks was less. For example, at tensile displacement of value 1 mm the shear value for the following specimen D28-VF0, D28-VF1, D28-VF2, D28-VF3 and D28-VF0-St was 64, 72.5, 99, 107.6 and 98.7 KN which shows that the rate of increase for D28-VF2 and D28-VF3 was less than that of D28-VF0-St.

4.2.2.4 Diagonal Compressive Strain in Concrete

Figure 4.7 shows the shear force vs. diagonal compressive strain in concrete for specimens of deep beams in group (A). From this figure, it can be seen that the pre-cracking stage the rate of increase of compressive strain was low for all the specimens. Also, it can be seen that diagonal compressive strain started to increase after the initiation of the diagonal cracks for all specimens. The rate of increase was higher for all specimens after the crack initiated. Although the compressive strain gauge failed at earlier stage, it was noticed that the rate of change of diagonal compressive strain for specimen (D28-VF0) would be either zero (the compressive strain would increase without increase in the shear force) or negative (the compressive strain would increase without increase in the shear force). This is because the specimen reached the
maximum shear value. All other specimens had lower rate of increase than that of D28-VF0. It is also important to highlight that the specimen D28-VF3 had lower rate of increase than that of the specimen with stirrups (D28-VF0-St). For example, at strain value equals to 1000, the shear value for D28-VF2, D28-VF3 and D28-VF28-St was 93, 110 and 97 respectively which shows that the addition of SF reduce the rate of increase of compressive strain.

![Shear force-Diagonal Compressive strain curves of deep beams in group (A)](image)

Figure 4.7: Shear force-Diagonal Compressive strain curves of deep beams in group (A)
4.2.2.5 Mode of failure

The shear failure in general starts with few vertical flexural cracks formed in the tension side under the applied load. As the load increased, generally the diagonal cracks appeared at the mid-height of beam within the clear shear span in the direction of the main strut and propagated (almost horizontally) toward the loaded point and toward the support. As the load increased more, existing cracks widened and increased in length. From the crack pattern, the failure mode of the specimen can be specified. The following types of shear failure were observed: shear compression, and strut crushing failure. For the shear compression failure, an inclined crack steeper than in diagonal tension appears accompanied with concrete crushing in the compression zone. For strut crushing failure, the failure happened due to forming of several parallel diagonal cracks or due to concrete peeling at mid-height in the center of shear span.

Failure modes of deep beams in group (A) are shown in Figure 4.8. D28-VF0, D28-VF0-St, and D28-VF1 failed in a shear-compression mode of failure due to crushing of concrete at the head of the inclined shear cracks under the shear force point. Deep specimens (D28-VF2 and D28-VF3) failed by crushing of the diagonal concrete struts. This was more evident in specimen D28-VF3 with higher steel fiber volume fraction of $\nu_f = 1.2\%$. 
Figure 4.8: Failure modes of deep beam in group (A)
4.3 Test Results – Group B (60 MPa)

4.3.1 Slender Beam

4.3.1.1 Shear Capacity

The main test results are summarized in the following Table 4.4. The ultimate shear force \( P_u \) is the maximum load force that the beam could hold. Table 4.4 also shows the deflection at the maximum load that the beam could hold \( \Delta u \). The shear in the test region is as in equation (4.1). The three columns next to the last one show the values of the shear component (Concrete \( V_c \), Transverse reinforcement \( V_s \) and steel fiber \( V_{sf} \)) that contribute to the total shear strength. The Concrete contribution to the shear strength \( V_c \) is calculated from the control specimen (S60-VF0).

The increase in shear strength for the following specimens (S60-VF1, S60-VF2, and S60-VF3) is 51%, 53% and 31% respectively. \( P_u \) for specimen S60-VF1, S60-VF2 and S60-VF3 is higher than that of specimen S60-VF0-St which indicates the possibilities of replacing the transverse reinforcement (Stirrups) with steel fiber as shear reinforcements. It should be noted that as \( V_c \) increases, the shear strength gain did not increase.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( P_u ) (KN)</th>
<th>( V_c ) (KN)</th>
<th>( \Delta u ) (mm)</th>
<th>( V_s ) (KN)</th>
<th>( V_{sf} ) (KN)</th>
<th>Shear Gain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S60-VF0-St</td>
<td>131.5</td>
<td>98.6</td>
<td>9.4</td>
<td>80.9</td>
<td>17.7</td>
<td></td>
</tr>
<tr>
<td>S60-VF0</td>
<td>107.8</td>
<td>80.9</td>
<td>9.2</td>
<td>80.9</td>
<td>41.4</td>
<td>51%</td>
</tr>
<tr>
<td>S60-VF1</td>
<td>163.0</td>
<td>122.3</td>
<td>10.8</td>
<td>80.9</td>
<td>-</td>
<td>53%</td>
</tr>
<tr>
<td>S60-VF2</td>
<td>164.8</td>
<td>123.6</td>
<td>10.5</td>
<td>80.9</td>
<td>-</td>
<td>53%</td>
</tr>
<tr>
<td>S60-VF3</td>
<td>142.1</td>
<td>106.6</td>
<td>9.9</td>
<td>80.9</td>
<td>25.7</td>
<td>31%</td>
</tr>
</tbody>
</table>

Table 4.4: Test Results for slender beams in Group (B)
4.3.1.2 Deflection Response

Figure 4.9 shows the shear force-deflection curves of slender beams in group (B). It is evident that the addition of steel fibers increased the shear resistance and energy absorption of the specimens. The inclusion of steel fibers (SF) or stirrups did not significantly change the stiffness of the beams. The specimens in group (B) featured a quasi-linear shear force deflection behavior until the first major diagonal crack formed (not the peak shear force for S60-VF0). The first major diagonal crack formed for S60-VF0 was 72.2 KN. After the major crack, the stiffness of this specimen was reduced (up to maximum shear capacity) causing the deflection to increase significantly with the increase of the shear force. With the inclusion of SF or transverse reinforcement, it was observed that the shear force causing major diagonal crack approaches the ultimate shear capacity as in S60-VF0-St, S60-VF1, S60-VF2, and S60-VF3. The shear force-deflection curve shows that the ultimate shear capacity
increased with the inclusion of steel fibers or stirrups. It is also important to highlight that maximum shear strength for specimen S60-VF1, S60-VF2, S60-VF3 was higher than that of S60-VF0-St. For example, the maximum shear capacity for the control beam (S60-VF0) was 80.9 KN. For the beam with stirrups (S60-VF0-St), the peak shear force was 98.6 KN. The peak shear force for the beams with steel fiber was 122.3, 123.6 and 106.6 KN for S60-VF1, S60-VF2 and S60-VF3 respectively. The shear force dropped significantly after the peak shear force for all beams and after that, the deflection increased significantly. The maximum deflection was increased with the inclusion of steel fibers or stirrups. It is important to highlight that maximum deflection for specimen S60-VF1, S60-VF2, S60-VF3 was higher than that of S60-VF0-St.

### 4.3.1.3 Diagonal Tensile Displacement

The shear force vs. diagonal tensile displacement curves of slender beams in group (B) are depicted in Figure 4.10. From this figure, it can be seen that the specimens did not exhibit any significant diagonal displacement till the initiation of the cracks. The graph shows that the control specimen (S60-VF0) showed signs of diagonal cracking at a shear value of about 72.2 KN. The presence of transverse reinforcement did not increase the initiation of shear crack ($V_{cr} = 58.8$ KN); however, the inclusion of steel fiber increased the shear force needed for crack initiation as can be noticed in S60-VF3 ($\psi_f = 1.2\%$). The shear value when the crack initiated for the beams with steel fiber was 72.9, 72.8 and 92.5 KN for S60-VF1, S60-VF2 and S60-VF3 respectively. In the post-cracking stage, the diagonal tensile displacement started to increase after the initiation of the diagonal cracks for all the specimens. Also, the beams with transverse reinforcement or with steel fiber (S60-VF1, S60-VF2 S60-VF3 and S60-VF28-St) reduced the rate of increase of diagonal displacement across cracks.
relative to the S28-VF0 beam. For example, at tensile displacement of value 1 mm the shear value for the following specimen S60-VF0, S60-VF1, S60-VF2, S60-VF3, and S60-VF0-St was 65, 108.3, 114.5, 105.8, and 81 which shows that the rate of increase for S60-VF1, S60-VF2, S60-VF3 and S60-VF0-St was less than that of S60-VF0. This indicates that with the increase of (v) the rate of increase of diagonal displacement across cracks can be reduced.

![Figure 4.10: Shear Force-Diagonal tensile displacement curves of slender beams of group B](image)
4.3.1.4 Diagonal Compressive strain in concrete

Figure 4.11 shows the shear force vs. diagonal compressive strain in concrete for specimens of slender beams in group (B). From this figure, it can be seen that in the pre-cracking stage the rate of increase of compressive strain was low for all the specimens. Also, it can be seen that diagonal compressive strain started to increase after the initiation of the diagonal cracks for all the specimens. The rate of increase of diagonal compressive strain was higher for all specimens after the crack initiated, but for specimen (S60-VF0) the rate was very high (negative value). For example, at strain value equals to 270, the shear value for S60-VF0, S60-VF1, S60-VF2, and S60-VF0-St was 80.8, 92.6, 69.8, and 58 respectively which shows that the addition of SF reduce the rate of increase of compressive strain. The compression strain gauge for the specimen S60-VF0, S60-VF1, S60-VF3 and S60-VF60-VF0-St failed soon after the initiation of the crack due to the fact that crack passed through the strain gauge.

![Figure 4.11: Shear force-Diagonal Compressive strain curves of slender beams in group (B)](image-url)
4.3.1.5 Mode of failure

The shear failure in general starts with few vertical flexural cracks formed in the tension side under the applied load. As the load increased, generally the diagonal cracks appeared at the mid-height of beam within the clear shear span in the direction of the main strut and propagated (almost horizontally) toward the loaded point and toward the support. As the load increased more, existing cracks widened and increased in length. The following types of shear failures were observed: diagonal tension, and web crushing. For the diagonal tension failure, an inclined crack appears at mid-height of the beam and propagated toward the loaded point and the support. For web crushing failure, the concrete will crush in the mid-height of the beam in the shear span.

Failure modes of slender beams in group (B) are shown in Figure 4.12. All beams for this category showed a classical diagonal tension mode of failure, except S60-VF2 and S60-VF0-St. S60-VF2 which failed due to web-crushing shear mode of failure as a result of concrete spalling and formation of multiple diagonal cracks. S60-VF0-St failed due to web-crushing shear mode of failure due to formation of multiple diagonal cracks. The addition of steel fiber in S60-VF3 (vf = 1.2%) did not change the failure mode significantly but allowed the beam to sustain addition shear force and increase the crack width which gives a good indication before failure.
Figure 4.12: Failure modes of slender beams in group (B)
4.3.2 Deep Beam

4.3.2.1 Shear Capacity

The main test results are summarized in the following Table 4.5. The ultimate shear force \( (P_u) \) is the maximum load force that the beam could hold. Table 4.5 also shows the deflection at the maximum load that the beam could hold \( (\Delta_u) \). The shear in the test region is calculated as in equation (4.1). The three columns next to the last one show the values of the shear component (Concrete \( (V_c) \), Transverse reinforcement \( (V_s) \) and steel fiber \( (V_{sf}) \)) that contribute to the total shear strength. The Concrete contribution to the shear strength \( (V_c) \) is calculated from the control specimen \( (D60-VF0) \).

The increase in shear strength for the following specimens \( (D60-VF1, D60-VF2, \ and \ D60-VF3) \) is 26.7\%, 44.6\% and 63.4\% respectively. \( (P_u) \) for specimen \( D60-VF2 \) and \( D60-VF3 \) is higher than that of specimen \( D28-VF0-St \) which indicates the possibilities of replacing the transverse reinforcement (Stirrups) with steel fiber as shear reinforcements.

<table>
<thead>
<tr>
<th>Name</th>
<th>( P_u ) (KN)</th>
<th>( V_u ) (KN)</th>
<th>( \Delta_u ) (mm)</th>
<th>( V_c ) (KN)</th>
<th>( V_s ) (KN)</th>
<th>( V_{sf} ) (KN)</th>
<th>Shear Gain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D60-VF0-St</td>
<td>145.2</td>
<td>116.1</td>
<td>4.8</td>
<td>91.3</td>
<td>24.8</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>D60-VF0</td>
<td>114.1</td>
<td>91.3</td>
<td>6.0</td>
<td>91.3</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>D60-VF1</td>
<td>144.7</td>
<td>115.7</td>
<td>5.6</td>
<td>91.3</td>
<td>-</td>
<td>24.4</td>
<td>26.7%</td>
</tr>
<tr>
<td>D60-VF2</td>
<td>165.0</td>
<td>132.0</td>
<td>4.9</td>
<td>91.3</td>
<td>-</td>
<td>40.7</td>
<td>44.6%</td>
</tr>
<tr>
<td>D60-VF3</td>
<td>186.5</td>
<td>149.2</td>
<td>6.2</td>
<td>91.3</td>
<td>-</td>
<td>57.9</td>
<td>63.4%</td>
</tr>
</tbody>
</table>

Table 4.5: Test results for deep beams in group (B)
4.3.2.2 Deflection Response

Figure 4.13 shows the shear force-deflection curves of deep beams in group (B). It is evident that the addition of steel fibers increased the shear resistance and energy absorption of the specimens. The beam stiffness did not change significantly with the inclusion of steel fibers or stirrups. The specimens featured a quasi-linear shear force deflection behavior until the first major diagonal crack formed (not the peak shear force for D60-VF0). The first major diagonal cracked formed for D60-VF0 was 66.9 KN. After the major crack, the stiffness of this specimen was slightly reduced (up to maximum shear capacity) causing the deflection to increase significantly with the increase of the shear force. It was observed that the shear force causing the major crack happened at the maximum shear capacity for all specimens except (D60-VF0). The shear force-deflection curve shows that the shear capacity increased with the inclusion of steel fibers or stirrups. It is important to highlight that

![Shear Force-Deflection curves of deep beams in group (B)](image-url)
maximum shear force for specimen D60-VF2, and D60-VF3 were higher than that of
D60-VF0-St. For example, the shear capacity for the control beam (D60-VF0) was
91.3 kN. For the beam with stirrups (D60-VF0-St) the shear capacity was 116.1 kN.
The shear capacity for the beams with steel fiber was 115.7, 132, 149.2 kN for D60-
VF1, D60-VF2 and D60-VF3 respectively. The shear force dropped significantly after
the maximum shear capacity was reached for all beams and after that, the deflection
increased. The maximum deflection was increased with the inclusion of steel fibers or
stirrups. It is also important to highlight that maximum deflection for specimen D60-
VF1, D60-VF2, and D60-VF3 were higher than that of D60-VF0 and D60-VF0-St.

4.3.2.3 Diagonal Tensile Displacement

The shear force vs. diagonal tensile displacement curves for specimens of deep
beams in group (B) are depicted in Figure 4.14. From this figure, it can be seen that
the specimens did not exhibit any significant diagonal displacement till the initiation
of the cracks. The graph shows that the control specimen, D60-VF0, showed signs of
diagonal cracking at a shear value of about 67 kN. The presence of transverse
reinforcement (D60-VF0-St) did not increase the cracking shear force ($V_{cr} = 59.9$ kN);
however the inclusion of steel fiber increased the cracking shear force. The cracking
shear force for the beams with steel fiber was 84.1, 92.9 and 87.6 kN for D60-VF1,
D60-VF2 and D60-VF3 respectively. In the post-cracking stage, the diagonal tensile
displacement started to increase after the initiation of the diagonal cracks for all the
specimens. Also, the following beams (D60-VF1, D60-VF2, D60-VF3 and D60-VF0-
St) reduced the rate of increase of diagonal displacement across cracks relative to the
D60-VF0 beam. The rate of increase of diagonal displacement across cracks reduced
with the inclusion of SF or transverse reinforcement. For example, at tensile
displacement of value 0.5 mm the shear value for the following specimen D60-VF0, D60-VF1, D60-VF2, D60-VF3 and D60-VF0-St was 68.8, 99.5, 123.7, 119.3 and 93.1 KN.

Figure 4.14: Shear Force-Diagonal tensile deformation curves of deep beams of group (B)
4.3.2.4 Diagonal Compressive Strain in Concrete

Figure 4.15 shows the shear force vs. diagonal compressive strain in concrete for specimens of deep beams in group (B). From this figure, it can be seen that in the pre-cracking stage the rate of increase of compressive strain was low for all the specimens. Also, it can be seen that diagonal compressive strain started to increase after the initiation of the diagonal cracks. Also, it can be seen that diagonal compressive strain started to increase after the initiation of the diagonal cracks. The rate of increase was higher for all specimens after the crack initiated. The rate of increase was higher for all specimens after the crack initiated, but for specimen (D60-VF0) the rate was very high and the behavior was a very plastic one in comparison of all other specimens. All other specimens had lower rate of increase than that of D60-VF0. The graph shows that with the inclusion of SF or transverse reinforcement (i.e. stirrups) the rate of increase of compressive strain was reduced.

Figure 4.15: Shear force-Diagonal Compressive strain curves of deep beams in group (B)
4.3.2.5 Mode of failure

The shear failure in general starts with few vertical flexural cracks formed in the tension side under the applied load. As the load increased, generally the diagonal cracks appeared at the mid-height of beam within the clear shear span in the direction of the main strut and propagated (almost horizontally) toward the loaded point and toward the support. As the load increased more, existing cracks widened and increased in length. From the crack pattern, the failure mode of the specimen can be specified. The following types of shear failure were observed: shear compression, and strut crushing failure. For the shear compression failure, an inclined crack steeper than in diagonal tension appears accompanied with concrete crushing in the compression zone. For strut crushing failure, the failure happened due to forming of several parallel diagonal cracks or due to concrete peeling at mid-height in the center of shear span.

Modes of Failure of deep beams in group (B) are shown in Figures 4.16. For the specimens D60-VF0, D60-VF1 the failure mode for these specimens was due to Shear-compression mode of failure with one major diagonal crack (the crushing of concrete happens near the support for D60-VF0 and under the shear force for D60-VF1). For the specimens D60-VF2, D60-VF3 and D60-VF0-St the failure mode for these specimens were similar and it was due to strut crushing failure due to forming several parallel cracks.
Figures 4.16: Failure Modes of deep beam in group (B)
4.4 Test Results – Group C (100 MPa)

4.4.1 Slender Beam

4.4.1.1 Shear Capacity

The main test results are summarized in the following Table 4.6. The ultimate shear force \((P_u)\) is the maximum load force that the beam could hold. Table 4.6 also shows the deflection at the maximum load that the beam could hold \((\Delta_u)\). The shear in the test region is calculated as in equation (4.1). The three columns next to the last one show the values of the shear component (Concrete \((V_c)\), Transverse reinforcement \((V_s)\) and steel fiber \((V_{sf})\)) that contribute to the total shear strength. The Concrete contribution to the shear strength \((V_c)\) is calculated from the control specimen \((S100-VF0)\).

The increase in shear strength for the following specimens \((S100-VF1, S100-VF2, \text{ and } S100-VF3)\) is 29.2\%, 56.5\% and 94.0\% respectively. \((P_u)\) for specimen \(S100-VF3\) is higher than that of specimen \(S28-VF0-St\) which indicates the possibilities of replacing the transverse reinforcement (Stirrups) with steel fiber as shear reinforcements.

<table>
<thead>
<tr>
<th>Name</th>
<th>(P_u) (KN)</th>
<th>(V_u) (KN)</th>
<th>(\Delta_u) (mm)</th>
<th>(V_c) (KN)</th>
<th>(V_s) (KN)</th>
<th>(V_{sf}) (KN)</th>
<th>Shear Gain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S100-VF0-St</td>
<td>150.8</td>
<td>113.1</td>
<td>11.2</td>
<td>65.0</td>
<td>48.1</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S100-VF0</td>
<td>86.7</td>
<td>65.0</td>
<td>5.9</td>
<td>65.0</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S100-VF1</td>
<td>112.0</td>
<td>84.0</td>
<td>7.4</td>
<td>65.0</td>
<td>-</td>
<td>19.0</td>
<td>29.2%</td>
</tr>
<tr>
<td>S100-VF2</td>
<td>135.7</td>
<td>101.8</td>
<td>7.8</td>
<td>65.0</td>
<td>-</td>
<td>36.8</td>
<td>56.5%</td>
</tr>
<tr>
<td>S100-VF3</td>
<td>168.2</td>
<td>126.2</td>
<td>11.1</td>
<td>65.0</td>
<td>-</td>
<td>61.1</td>
<td>94.0%</td>
</tr>
</tbody>
</table>

Table 4.6: Test results for slender beams in group (C)
4.4.1.2 Deflection Response

Figure 4.17 shows the shear force-deflection curves of slender beams in group (C). It is evident that the addition of steel fibers increased the shear resistance and energy absorption of the specimens. The inclusion of steel fibers or stirrups did not significantly change the stiffness of the beams. The specimens featured a quasi-linear shear force deflection behavior until the first major diagonal crack formed (not the peak shear force for S100-VF0, S100-VF1, and S100-VF0-St). The first major diagonal cracked formed for S100-VF0, S100-VF1, and S100-VF0-St was 63.3, 78.4 and 108.2 KN. After the major crack, the stiffness of this specimen was slightly reduced (up to maximum shear capacity) causing the deflection to increase significantly with the increase of the shear force. As the (v) increased, it was observed that the shear force causing major diagonal crack approaches the ultimate shear capacity as in S100-VF2 and S100-VF3. The shear force-deflection curve shows that

![Shear Force-Deflection curves of slender beams in group (C)](image)
the shear force carrying capacity increased with the inclusion of steel fibers or stirrups. It is also important to highlight that maximum shear capacity for specimen S100-VF3 was higher than that of S100-VF0-St. For example, the maximum shear capacity for S100-VF0, S100-VF1, S100-VF2, S100-VF3 and S100-VF0-St was 65.0, 84.0, 101.8, 126.2 and 113.1 KN respectively. The shear force dropped significantly after the peak shear force was reached for all the specimens and after which, the deflection increased dramatically. Also, the maximum deflection was increased with the inclusion of steel fibers or stirrups. It is important to highlight that maximum deflection for specimen S100-VF3 was higher than that of specimen with stirrups S100-VF0-St.

4.4.1.3 Diagonal Tensile Displacement

The shear force vs. diagonal tensile displacement curves for specimens of slender beams in group (C) are depicted in Figure 4.18. From this figure, it can be seen that the specimens did not exhibit any significant diagonal displacement till the initiation of the cracks. The graph shows that the control specimen, S100-VF0, showed signs of diagonal cracking at a shear value of about 63.3 kN. The presence of transverse reinforcement did not increase the initiation of crack shear value (cracking shear value for S100-VF0-St = 63.9 KN); however the inclusion of steel fiber increased the cracking shear force. It is important to highlight that shear value needed for crack initiation for specimens with steel fiber (SF) (S100-VF1, S100-VF2, and S100-VF3) was higher than that of specimen with stirrups (S100-VF0-St). The cracked shear value for the beams with steel fiber was 79.1, 77.1 and 87 KN for S100-VF1, S100-VF2 and S100-VF3 respectively. In the post-cracking stage, the diagonal tensile displacement started to increase after the initiation of the diagonal cracks for all the specimens. It was observed that for beams (S100-VF2, S100-VF3 and S100-VF0-St), the rate of
increase of diagonal displacement across cracks was reduced relative to the S100-VF0 beam. Specimens S100-VF0 (control beam) and S100-VF1 exhibit a plastic response up to failure. It is important to highlight that the rate of increase of diagonal displacement for S100-VF3 was less than that of S100-VF0-St. This shows that with the increase SF the rate of increase of diagonal displacement is reduced. For example, at tensile displacement of value 0.5 mm the shear value for the following specimen S100-VF0, S100-VF1, S100-VF2, S100-VF3 and S100-VF0-St was 62, 77.6, 93, 104.5 and 74.2 respectively.

![Shear Force-Diagonal tensile deformation curves of slender beams of group (C)](image)

Figure 4.18: Shear Force-Diagonal tensile deformation curves of slender beams of group (C)
4.4.1.4 Diagonal Compressive Strain in Concrete

Figure 4.19 shows the shear force vs. diagonal compressive strain in concrete for specimens of slender beams in group (C). From this figure, it can be seen that in the pre-cracking stage the rate of increase of compressive strain was very low for all the specimens. The rate of increase of diagonal compressive strain started to increase for all specimens after the crack initiated. It was observed that for beam with SF ($v_f = 0.8\%$ and $v_p = 1.2\%$) the rate of increase of diagonal compressive strain was reduced due to the inclusion of SF. For example, at strain value equals to 200, the shear value for $S100-VF0$, $S100-VF1$, $S100-VF3$ and $S100-VF0-St$ was 62.3, 67.7, 74.1, and 63.5 respectively which shows that the addition of SF reduce the rate of increase of compressive strain. The compression strain gauge for the specimen $S100-VF0$, $S100-VF1$, $S100-VF3$ and $S100-VF0-St$ failed soon after the initiation of the crack.

Figure 4.19: Shear force-Diagonal Compressive strain curves of slender beams in group (C)
4.4.1.5 Mode of failure

The shear failure in general starts with few vertical flexural cracks formed in the tension side under the applied load. As the load increased, generally the diagonal cracks appeared at the mid-height of beam within the clear shear span in the direction of the main strut and propagated (almost horizontally) toward the loaded point and toward the support. As the load increased more, existing cracks widened and increased in length. The following types of shear failures were observed: diagonal tension, and web crushing. For the diagonal tension failure, an inclined crack appears at mid-height of the beam and propagated toward the loaded point and the support. For web crushing failure, the concrete will crush in the mid-height of the beam in the shear span.

Failure modes of slender beams in group (C) are shown in Figure 4.20. The figure shows the failure mode for specimens S100-VF1 to S100-VF0-St respectively. These specimens exhibited one major diagonal shear crack. These specimens failed eventually due to diagonal-tension shear mode of failure. Also, the figure shows the failure mode for specimen S100-VF0 which failed in Diagonal splitting shear mode of failure.
Figure 4.20: Failure modes of slender beam in group (C)
4.4.2 Deep Beam

4.4.2.1 Shear capacity

The main test results are summarized in the following Table 4.7. The ultimate shear force ($P_u$) is the maximum load force that the beam could hold. Table 4.5 also shows the deflection at the maximum load that the beam could hold ($\Delta_u$). The shear in the test region is calculated as in equation (4.1). The three columns next to the last one show the values of the shear component (Concrete ($V_c$), Transverse reinforcement ($V_s$) and steel fiber ($V_{sf}$)) that contribute to the total shear strength. The Concrete contribution to the shear strength ($V_c$) is calculated from the control specimen (D100-VF0). The increase in shear capacity for (D100-VF1) beam was minor (8.6%). This indicate the need to the steel fiber volume fraction ($v_f$) to have a significant increase in shear capacity.

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_u$ (KN)</th>
<th>$V_u$ (KN)</th>
<th>$\Delta_u$ (mm)</th>
<th>$V_c$ (KN)</th>
<th>$V_s$ (KN)</th>
<th>$V_{sf}$ (KN)</th>
<th>Shear Gain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D100-VF0</td>
<td>162.1</td>
<td>129.7</td>
<td>5.1</td>
<td>129.7</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>D100-VF1</td>
<td>176.1</td>
<td>140.9</td>
<td>5.5</td>
<td>140.9</td>
<td>-</td>
<td>11.2</td>
<td>8.6%</td>
</tr>
</tbody>
</table>

Table 4.7: Test Results for deep beams in group (C)
4.4.2.2 Deflection Response

Figure 4.21 shows the shear force-deflection curves of deep beams in Group (C). It is evident that the addition of steel fibers increased the shear resistance and energy absorption of the specimens. The beam stiffness did not change significantly with the inclusion of steel fibers or stirrups. The specimens in Group (C) featured a quasi-linear shear force deflection behavior until failure. The shear force-deflection curve shows that the shear capacity increased with the inclusion of steel fibers. It is important to highlight that maximum shear strength for specimen D100-VF1 were higher than that of D100-VF0. For example, the maximum shear capacity for the D100-VF0 and D100-VF1 was 129.7 and 140.9 KN. After the beam reached its maximum shear capacity it failed dramatically showing a very brittle behavior for specimen (D100-VF0). The inclusion of steel fiber increased the ductility and reduce the rate of increase in deflection in specimen D100-VF1. For example, at deflection
value equals to 8 mm, the shear force value for D100-VF0 and D100-VF1 was 60.7 KN and 110 KN respectively. Also, the maximum deflection was increased with the inclusion of steel fibers. It is important to highlight that the maximum deflection for specimen D100-VF0, D100-VF1 was 8 mm and 28.35 mm respectively.

4.4.2.3 Diagonal Tensile Displacement

The shear force vs. diagonal tensile displacement curves for specimens of deep beam group (C) are depicted in Figure 4.22. From this figure, it can be seen that the specimens did not exhibit any significant diagonal displacement till the initiation of the cracks. The graph shows that the control specimen, D100-VF0, showed signs of diagonal cracking at a shear value of about 66.6 kN. The inclusion of steel fiber increased the cracking shear force (Cracking shear force = 76.2 KN). In the post-cracking stage, the diagonal tensile displacement started to increase after the initiation

Figure 4.22: Shear Force-Diagonal tensile Displacement curves of deep beams of group (C)
of the diagonal cracks for all the specimens. Also, the beam with SF (D100-VF1) did not significantly reduce the rate of increase of diagonal displacement across cracks relative to the D100-VF0.

4.4.2.4 Diagonal Compressive Strain in Concrete

Figure 4.23 shows the shear force vs. diagonal compressive strain in concrete for specimens of deep beams in group (B). From this figure, it can be seen that in the pre-cracking stage the rate of increase of compressive strain was low for all the specimens. Also, it can be seen that diagonal compressive strain started to increase after the initiation of the diagonal cracks. The rate of increase was higher for both specimens after the crack initiated. Although the compressive strain gauge fails at earlier stage, it can be noticed that the rate of change of diagonal compressive strain for specimen with SF (D100-VF1) was slightly less than the control beam (S100-VF0).

![Graph showing shear force vs. diagonal compressive strain for deep beams in group C.](image-url)

Figure 4.23: Shear force-Diagonal Compressive strain curves of deep beams in group (C)
4.4.2.5 Mode of failure

Figure 4.24 shows the failure mode for deep beams in group (C). The failure mode for D100-VF0 was due to a shear compression failure where the concrete crushed near the support. Figure 4.25 shows the failure mode for specimens D100-VF1. It shows that this specimens also failed due to shear compression mode of failure.
4.5 Performance Evaluation

In this section, the effect of steel fiber volume fraction \( (v_f) \) for groups (A, B and C) 28 MPa, 60 MPa, and 100 MPa are analyzed and discussed.

4.5.1.1 Slender Beams

4.5.1.1.1 Shear strength Gain

The interaction between steel fiber (SF), concrete compressive strength \((f'_c)\), and shear strength gain for slender beams is demonstrated in Figure 4.26. For the slender beams with steel fiber volume fractions of 0.4% and 0.8%, varying the concrete grade had no obvious effect on the shear strength gain. Nevertheless, for the slender beams with the higher steel fiber volume fraction of 1.2%, the shear strength gain tended to decrease with an increase in the concrete grade. Figure 4.26 shows that as \((v_f)\) increases for group (A) (28 MPa) the shear strength gain increases. For group (B)
(60 MPa), increasing of \(v_f\) did not result in an increasing in shear strength gain. For group (C) (90 MPa), Figure 4.26 demonstrate that as \(v_f\) increases the shear strength gain increases.

4.5.1.2 Deep Beams

4.5.1.2.1 Shear Strength Gain

The interaction between SF, concrete compressive strength \(f'_c\) and shear strength gain is demonstrated in Figure 4.27. It can be seen from the figure that, the shear strength gain tended to decrease by increasing the concrete grade. That was more evident for the deep beams having the higher steel fiber volume fractions of 0.8% and 1.2%.

For the deep beams with the lower concrete grade of 28 MPa, increasing the steel fiber volume fraction from 0.4% to 0.8% increased the shear strength gain. Further increase in the steel fiber volume fraction to 1.2% did not result in additional shear strength gain for the deep beams with concrete grade of 28 MPa. This can be ascribed to the web-crushing mode of failure exhibited by the deep beams with concrete grade of 28 MPa and \(v_f\) of 0.8% and 1.2%, which concealed the effect of increasing the steel fiber volume fraction. Figure 4.27 also shows that as \(v_f\) increases for group (B) (60 MPa) the shear gain increases. Also, the figure demonstrates that the inclusion of SF caused an 8.6% shear gain for group (C) (90 MPa).
Figure 4.27: Interaction between shear gain, $(\tau)$, and $(f_c')$ for deep beams.
5.1 Introduction

In this chapter, the shear strength of UH-FR-SCC (Ultra-High Strength steel Fiber reinforced self-compacting concrete) beams is evaluated. Analytical models published in the literature that predict the shear resistance of SFRC (Steel Fiber Reinforced Concrete) are used to predict the shear resistance of UH-FR-SCC that was produced in this study. Three variables were used in this study (See Table 3.1). These variables are the concrete strength ($f'_c$), steel fiber volume ratio ($v_f$), and shear span to depth ratio ($a/d$).

5.2 Shear Strength of RC Beams

The shear strength of RC beams is modeled by the following equation (5.1).
The shear strength of RC beams is considered to be the results of the combination of plain concrete ($V_c$) and steel stirrups ($V_s$) (if exists).

$$V_u = V_c + V_s$$  \hspace{1cm} 5.1

The shear strength of RC beams without web reinforcement is only due to concrete shear resistance ($V_c$). The concrete shear resistance can be modeled as in equation (5.2):

$$V_u = V_c = V_{cz} + V_a + V_d$$  \hspace{1cm} 5.2

Where:

$V_{cz}$ = The shear in the compression zone;

$V_a$ = The shear due to aggregate interlock;
\[ V_d = \text{The dowel action;} \]

### 5.3 Shear Strength of SFRC Beams

Various analytical models were prepared and published in the literature to predicate the shear strength of SFRC. There are two approaches of these models which considered the effect of steel fiber contribution to the beam shear capacity. The first approach considered the contribution of the steel fiber contribution as a separate term. Some of these models will be tested to whether they can be used for Ultra-High-Strength Fiber Reinforced Self-Compacting Concrete (UHS-FR-SCC). Swamy et al. (1993) and Al-Ta’an and Al-Feel (1990) both developed models to calculate the steel fiber effect on the shear strength of SFRC beams based on the first approach. The following equation (5.3) was used to evaluate the shear strength for the first approach:

\[
V_u = V_c + V_s + V_{sf} \tag{5.3}
\]

Where:

- \( V_n \) = The ultimate shear strength
- \( V_{sf} \) = The contribution of steel fiber in the shear strength;
- \( V_c \) = The contribution of concrete in the shear strength;
- \( V_s \) = The contribution of stirrups in the shear strength.

The second approach considered that steel fiber will affect the concrete characteristics (such as, \( f'_c, f'_s \)) and that is why the effect of steel fiber is imbedded in the concrete shear strength (\( V_c \)). Narayanan and Darwish (1987), Ashour et al. (1992) (Modified Zsutty equation) and (Modified ACI equation), Imam et al. (1997), Kwak
et al. (2002), Sharma (1986), Khuntia et al. (1999) and Shin et al. (1994) developed models to calculate the steel fiber effect in the shear strength of SFRC beams based on the second approach. The following equation (5.4) was used to evaluate the shear strength for the second approach:

\[ V_u = V_c^* + V_s \]  

Where:

- \( V_u \) = The ultimate shear strength;
- \( V_c^* \) = The contribution of concrete and steel fiber in the shear capacity; and
- \( V_s \) = The contribution of stirrups in the shear strength.

Some of these models will be tested to whether they can be used for Ultra-High-strength Fiber Reinforced self-compacting concrete (UHS-FRC-SCC). The values taken from the experimental results will be compared to the values obtained from the proposed models.

5.3.1 First Approach models

For the first approach models, steel fiber contribution to the shear strength \( (V_{sf}) \) from the experimental results will be compared to the predicted value from the following models.

5.3.1.1 Al-Ta’an and Al-Feel (1990)

The steel fiber contribution in the shear strength \( (V_{sf}) \), according to Al-Ta’an and Al-Feel (1990) was evaluated using an empirical equation (5.5) which is based on regression analysis conducted on experimental results using 89 beams.
$V_{sf} = \frac{8.5}{9} k V_f \frac{L_f}{D_f} \times (b d)$  \hspace{1cm} 5.5

Where:

\[ k = \text{steel fiber shape factor, for the fiber used in this study (hooked fiber) the value is 1.2 according to the model.} \]

\[ V_f = \text{Volume fraction of steel fibers;} \]

\[ b = \text{Beam width;} \]

\[ d = \text{Beam effective depth;} \]

\[ L_f = \text{Fiber length;} \]

\[ D_f = \text{Fiber diameter;} \]

5.3.1.2 Swamy et al. (1993)

The steel fiber contribution in the shear strength $V_{sf}$, according to Swamy et al. (1993) is calculated using the following equation (Eq. 5.6):

\[ V_{sf} = 0.37 \tau V_f \frac{L_f}{D_f} \times bd \]  \hspace{1cm} 5.6

Where:

\[ \tau = \text{Average fiber matrix interfacial bond stress, taken as 4.15 MPa, based on the recommendations of Swamy et al. (1974).} \]

\[ b = \text{Beam width;} \]

\[ d = \text{Beam effective depth;} \]

\[ V_f = \text{Volume fraction of steel fibers;} \]
$L_f = $ Fiber length; and

$D_f = $ Fiber diameter;

### 5.3.2 Second Approach models

The following models as explained above considered that the steel fiber influence the shear capacity of concrete. These models give the value of the ultimate shear strength directly.

#### 5.3.2.1 Narayanan and Darwish (1987)

The nominal shear resistance of SFRC according to Narayanan and Darwish (1987) analytical model is calculated using the following equation (Eq. 5.7):

$$V_u = \left[ e \left( 0.24f_t + 80\rho_s \frac{d}{a} \right) + v_b \right] \times bd$$  \hspace{1cm} 5.7

Where:

$f_t = $ Split-cylinder strength;

$e = $ Arch action factor: 1.0 for $\left( \frac{a}{d} \right) > 2.8$, and 2.8($\frac{d}{a}$) for $\left( \frac{a}{d} \right) \leq 2.8$;

$b = $ Beam width;

$a = $ Shear span;

$d = $ Beam Effective depth;

$v_b = 0.41\tau F$;

$\tau = $ Average fiber matrix interfacial bond stress, taken as 4.15 MPa, based on the recommendations of Swamy et al. (1974); and
$F = \text{Fiber factor} = \left(\frac{L_f}{d_f}\right) v_f d_f;$

d_f = \text{Bond factor that depends on fiber shape: 0.50 for round fibers, 0.75 for crimped fibers, and 1.0 for indented fibers, for this study a value of 1.0 was used; and}

$\rho_s = \text{Flexural reinforcement ratio.}$

5.3.2.2 Ashour et al. (1992)

5.3.2.2.1 Ashour et al. (1992) (Modified Zsutty Equation)

Ashour et al. (1992) suggested an equation which is similar to the Zsutty equation for calculation the shear strength of reinforced concrete beam. The nominal shear resistance of SFRC according to Ashour et al. (1992) (Modified Zsutty Equation) analytical model is calculated using the following equations (Eq. 5.8) and (Eq. 5.9):

If $\frac{a}{d} \geq 2.5$

$$V_u = \left[ (2.11 \sqrt{f_c'} + 7F) \left( \frac{\rho_s d}{a} \right)^{\frac{1}{3}} \right] \times bd$$  \hspace{1cm} (5.8)

If $\frac{a}{d} < 2.5$

$$V_u = \left[ (2.11 \sqrt{f_c'} + 7F) \left( \frac{\rho_s d}{a} \right)^{\frac{1}{3}} \frac{2.5}{a} + v_b \right] \times bd$$  \hspace{1cm} (5.9)

Where:

$f_c' = \text{Concrete compressive strength;}$

$a = \text{Shear span;}$
92

\( b \) = Beam width;

\( d \) = Beam effective depth;

\( \rho_v \) = Flexural reinforcement ratio.

\( \nu \) = 0.41 \( \tau F \);

\( \tau \) = Average fiber matrix interfacial bond stress, taken as 4.15 MPa, based on the recommendations of Swamy (1974); and

\( F \) = Fiber factor = \( \left( \frac{L}{D_f} \right) v_f d_f \); and

\( d_f \) = Bond factor that depends on fiber shape: 0.50 for round fibers, 0.75 for crimped fibers, and 1.0 for indented fibers, for this study a value of 1.0 was used.

5.3.2.2 Ashour et al. (1992) (Modified ACI Equation)

Ashour et al. (1992) suggested another equation which is similar to the ACI equation for calculation the shear strength of reinforced concrete beam. The nominal shear resistance of SFRC according to Ashour et al. (1992) (Modified ACI Equation) analytical model is calculated using the following equation (Eq. 5.10):

\[ V_u = \left[ \left( 0.7 \sqrt{f_{ce}'} + 7F \right) \frac{d}{a} + 17.2 \rho_s \frac{d}{a} \right] \times bd \quad 5.10 \]

Where:

\( f_{ce}' \) = Concrete compressive strength;

\( b \) = Beam width;

\( a \) = Shear span;
\[ d \quad = \quad \text{Beam effective depth}; \]

\[ \rho_s \quad = \quad \text{Flexural reinforcement ratio.} \]

\[ F \quad = \quad \text{Fiber factor } = \left( \frac{L}{d_f} \right) v_f d_f; \text{ and} \]

\[ d_f \quad = \quad \text{Bond factor that depends on fiber shape: 0.50 for round fibers, 0.75 for crimped fibers, and 1.00 for indented fibers. For this study a value of 1.0 was used.} \]

5.3.2.3 Kwak et al. (2002)

The nominal shear resistance of SFRC according to Kwak et al. (2002) analytical model is calculated using the following equation (Eq. 5.11):

\[ V_u = \left[ 2.1 e f'_t 0.7 \left( \frac{\rho_s d}{a} \right)^{0.22} + 0.8 v_b 0.97 \right] \times b d \quad 5.11 \]

Where:

\[ e = \begin{cases} 3.5 \times \frac{a}{d} & , \frac{a}{d} < 3.5 \\ 1 & , \frac{a}{d} \geq 3.5 \end{cases} \]

\[ f_t \quad = \quad \text{Split-cylinder strength;} \]

\[ a \quad = \quad \text{Shear span;} \]

\[ b \quad = \quad \text{Beam width;} \]

\[ d \quad = \quad \text{Beam effective depth;} \]

\[ \rho_s \quad = \quad \text{Flexural reinforcement ratio.} \]

\[ v_b \quad = \quad 0.41 \tau F; \]

\[ \tau \quad = \quad \text{Average fiber matrix interfacial bond stress, taken as 4.15 MPa, based on the recommendations of Swamy (1974).} \]
\( F \) = Fiber factor = \( \left( \frac{L_f}{D_f} \right) v_f d_f \); and

\( d_f \) = Bond factor: 0.50 for round fibers, 0.75 for crimped fibers, and 1.00 for indented fibers, for this study a value of 1.0 was used.

5.3.2.4 Sharma AK. (1986)

The nominal shear resistance of SFRC according to Sharma (1986) analytical model is calculated using the following equation (Eq. 5.12):

\[
V_u = \left[ k f_t' \left( \frac{d}{a} \right)^{0.25} \right] \times b d
\]

Where:

- \( a \) = Shear span;
- \( b \) = Beam width;
- \( d \) = Beam effective depth;
- \( f_t' \) = Tensile strength of concrete;
- \( k \) = 1 if \( f_t' \) is obtained by direct tension test;
- \( f_t' \) = 2/3 if \( f_t' \) is obtained by indirect tension test;
- \( f_t' \) = 4/9 if \( f_t' \) is obtained using modulus of rupture; or
- \( f_t' \) = 0.79(\( f_c' \))^{0.5}; and
- \( f_c' \) = Concrete compressive strength.

5.3.2.5 Imam et al. (1997)

The nominal shear resistance of SFRC according to Imam et al. (1997) analytical model is calculated using the following equation (Eq. 5.13):
\[ V_u = \left[ 0.6\psi \sqrt{\frac{\omega}{\phi}} \left( (f'_c)^{0.44} + 275 \frac{\omega}{\phi} \right) \right] \times bd \]

Where:

\( f'_c \) = Concrete compressive strength;

\( b \) = Beam width;

\( d \) = Beam effective depth;

\( \psi \) = Size Effect Factor = \( \psi = \frac{1 + \frac{5.08}{d_a}}{\sqrt{1 + \frac{d}{25d_a}}} \)

\( d_a \) = Maximum aggregate size in mm; 10 mm was the maximum aggregate size in this study.

\( \omega \) = Reinforcement factor = \( \rho_s (1 + 4F) \);

\( \rho_s \) = Flexural reinforcement ratio.

\( F \) = Fiber factor = \( \left( \frac{\ell_f}{d_f} \right) v_f d_f \); and

\( d_f \) = Bond factor: 0.50 for smooth fibers, 0.9 for deformed fibers, and 1.0 for hooked fibers, for this study a value of 1.0 was used.

5.3.2.6 Khuntia et al. (1999)

The nominal shear resistance of SFRC according to Khuntia et al. (1999) analytical model is calculated using the following equation (Eq. 5.14):
\[ V_u = \left[ (0.167e + 0.25F) \sqrt{f'_c} \right] \times bd \] 5.14

Where:

\[
e = \begin{cases} 
\frac{2.5d}{a}, & \frac{a}{d} < 2.5 \\
1, & \frac{a}{d} \geq 2.5 
\end{cases}
\]

\[ a = \text{Shear span}; \]
\[ b = \text{Beam width}; \]
\[ d = \text{Beam effective depth}; \]
\[ f'_c = \text{Concrete compressive strength}; \]
\[ F = \text{Fiber factor} = \left( \frac{L_f}{d_f} \right) v_f \beta; \text{ and} \]
\[ \beta = \text{Factor for fiber shape and concrete type} = 1 \text{ for hooked or crimped steel fibers,} \]
\[ \frac{2}{3} \text{ for plain or round steel fibers with normal concrete,} \]
\[ \frac{3}{4} \text{ for hooked or crimped steel fibers with lightweight concrete. For this study a value of 1.0 was used.} \]

5.3.2.7 Shin et al. (1994)

The nominal shear resistance of SFRC according to Shin et al. (1994) analytical model is calculated using the following equations (Eq. 5.15) and (Eq. 5.16):

For \( a/d \geq 3.0, \)

\[ V_u = \left[ 0.19f'_c + 93\rho_s \left( \frac{d}{a} \right) + 0.834v_b \right] \times bd \] 5.15

For \( a/d < 3.0, \)

\[ V_u = \left[ 0.22f'_c + 217\rho_s \left( \frac{d}{a} \right) + 0.834v_b \right] \times bd \] 5.16

Where:

\[ a = \text{Shear span}; \]
$b$ = Beam width;

$d$ = Beam effective depth;

$f'_{t}$ = Tensile strength of concrete;

$\rho_s$ = Flexural reinforcement ratio.

$v_b = 0.41\tau F$;

$\tau$ = Average fiber matrix interfacial bond stress, taken as 4.15 MPa, based on the recommendations of Swamy et al. (1974) in the absence of specific pullout tests on the fiber reinforced concrete used in this investigation;

$F$ = Fiber factor $= \left(\frac{L_f}{D_f}\right) v_f d_f$; and

$d_f$ = Bond factor: 0.50 for round fibers, 0.75 for crimped fibers, and 1.00 for indented fibers, for this study a value of 1.0 was used.

### 5.3.3 Comparative Analysis

The validity of the models explained earlier in this chapter to predict the shear strength of SFRC are analyzed in this section. This section is divided into two parts, one for the models of First approach and the other one for the models of second approach.

#### 5.3.3.1 Predictions for SFRC using first approach models

Comparison between the experimental results of beams including SF and predicted shear capacity using first approach models is given in Table 5.1. For Specimen with steel fiber, steel fiber contribution ($v_f$) was calculated using the models of the first approach. It can be seen from Table 5.1 that both models of the first approach Al-Ta'an and Al-Feel (1990) and Swamy et al. (1993) are very conservative
in estimating the value of shear strength of SFRC. Also, it can be seen from Table 5.1 that both models were better in estimating the slender beams shear capacity than that of deep beam shear capacity.

5.3.3.1.1 Slender Beams

The highest standard deviation beams for Al-Ta'an et al. (1990) was recorded for specimen S60-VF1 at ratio equals to 0.151 whereas the least standard deviation for Al-Ta'an et al. (1990) was recorded for specimen S60-VF3 at ratio value equals to 0.727. The highest standard deviation for Swamy et al. (1993) was recorded for specimen S60-VF1 at ratio equals to 0.204 whereas the least standard deviation for Swamy et al. (1993) was recorded for specimen S60-VF3 at ratio value equals to 0.986. The average ratio value for slender beams for Al-Ta'an et al. (1990) model is 0.388 and the standard deviation is 0.178 which indicate a very poor correlation with the experimental results. The average ratio value for Swamy et al. (1993) model is 0.525 and the standard deviation is 0.241 which indicate a poor correlation with the experimental results. From the average and standard deviation in Table 5.1, Swamy et al. (1993) model gives a more accurate estimation than Al-Ta'an and Al-Feel (1990) model.

5.3.3.1.2 Deep Beams

The highest standard deviation for Al-Ta'an and Al-Feel (1990) was recorded for specimen D28-VF2 at ratio equals to 0.152 whereas the least standard deviation for Al-Ta'an and Al-Feel (1990) was recorded for specimen D100-VF1 at ratio value equals to 0.557. The highest standard deviation for Swamy et al. (1993) was recorded for specimen D28-VF2 at ratio equals to 0.205 whereas the least standard deviation
for Swamy et al. (1993) was recorded for specimen D100-VF1 at ratio value equals to 0.755. The average ratio value for Al-Ta’an and Al-Feel (1990) model is 0.325 and the standard deviation is 0.130 which indicate a very poor correlation with the experimental results. The average ratio value for Swamy et al. (1993) model is 0.441 and the standard deviation is 0.176 which indicate a very poor correlation with the experimental results. From the average and standard deviation in Table 5.1, Swamy et al. (1993) model gives a more accurate estimation than Al-Ta’an and Al-Feel (1990) model.
<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Specimen</th>
<th>$V_{sf, Exp}$ (KN)</th>
<th>$V_{sf, Th}$ (KN)</th>
<th>Ratio ($V_{sf, Th} / V_{sf, Exp}$)</th>
</tr>
</thead>
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<tr>
<td></td>
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<td>Al-Ta'an (Eqn. 5.5)</td>
<td>Swamy (Eqn. 5.6)</td>
</tr>
<tr>
<td>Slender ($a/d = 3.3$)</td>
<td>S28-VF1</td>
<td>10.7</td>
<td>6.2</td>
<td>8.4</td>
</tr>
<tr>
<td></td>
<td>S28-VF2</td>
<td>25.6</td>
<td>12.5</td>
<td>16.9</td>
</tr>
<tr>
<td></td>
<td>S28-VF3</td>
<td>67.8</td>
<td>18.7</td>
<td>25.3</td>
</tr>
<tr>
<td></td>
<td>S60-VF1</td>
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<td>6.2</td>
<td>8.4</td>
</tr>
<tr>
<td></td>
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<td>S100-VF1</td>
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<td>S100-VF3</td>
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</tr>
<tr>
<td></td>
<td>Standard Deviation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deep ($a/d = 2.2$)</td>
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<td>14.8</td>
<td>6.2</td>
<td>8.4</td>
</tr>
<tr>
<td></td>
<td>D28-VF2</td>
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<td>12.5</td>
<td>16.9</td>
</tr>
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<td></td>
<td>D28-VF3</td>
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<td>18.7</td>
<td>25.3</td>
</tr>
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<td>D60-VF1</td>
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<td>6.2</td>
<td>8.4</td>
</tr>
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<td></td>
<td>D60-VF2</td>
<td>40.7</td>
<td>12.5</td>
<td>16.9</td>
</tr>
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<td></td>
<td>D60-VF3</td>
<td>57.9</td>
<td>18.7</td>
<td>25.3</td>
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<tr>
<td></td>
<td>D100-VF1</td>
<td>11.2</td>
<td>6.2</td>
<td>8.4</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Standard Deviation</td>
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<td></td>
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</tr>
</tbody>
</table>

Table 5.1: The theoretical values of SFRC using first approach models and the Ratio between the theoretical values to the experimental ones using Equation 5.5 and Equation 5.6
Figure 5.1 shows the analytical prediction vs. the experimental results for these two equations for slender beams. Figure 5.2 is showing the analytical prediction vs. the experimental results for these two equations for deep beams. It can be observed from Figure 5.1 and Figure 5.2 that the data is below the equality line (where the experimental = the theoretical) which indicate conservative prediction. Both figures show that both models show a slightly better prediction for slender beams than deep beams. Also, it can be noticed that Swamy et al. (1993) model for slender and deep beam gives a more accurate estimation than Al-Ta’an et al. (1990) model for slender and deep beam.

![Figure 5.1: First approach Models for slender beams using Equation 5.5 and Equation 5.6](image-url)
5.3.3.2 Predictions for SFRC using second approach models

Comparison between the experimental results of beams including SF and predicted shear capacity using second approach models will be discussed in the section. The second approach models estimate the concrete and steel fiber contribution to the shear strength in one term.

5.3.3.2.1 Narayanan and Darwish (1987) and Ashour et al. (1992)

Narayanan and Darwish (1987), Ashour et al. (1992) (Modified Zsutty Equation) and Ashour et al. (1992) (Modified ACI equation) are models of the second
approach. The comparison between these models and the experimental results are shown in Table 5.2. The ratio between the experimental and theoretical for these three equations is shown in Table 5.3. It can be seen from these tables that the three models were conservative in estimating the value of shear strength of SFRC. The difference between the experimental results and the prediction can be deemed acceptable for these three models. Also, it can be seen from Table 5.3 that both models were better in estimating the slender beams shear capacity than that of deep beam shear capacity.

5.3.3.2.1 Slender beams

The highest standard deviation for Narayanan and Darwish (1987) was recorded for specimen S60-VF1 at ratio equals to 0.445 whereas the least for Narayanan and Darwish (1987) was recorded for specimen S28-VF2 at ratio value equals to 0.797. The highest standard deviation for Ashour et al. (1992) (Modified Zsutty equation) was recorded for specimen S60-VF1 at ratio equals to 0.464 whereas the least standard deviation for Ashour et al. (1992) (Modified Zsutty equation) was recorded for specimen S28-VF1 at ratio value equals to 0.765. The highest standard deviation for Ashour et al. (1992) (Modified ACI equation) was recorded for specimen S60-VF1 at ratio equals to 0.439 whereas the least standard deviation for Ashour et al. (1992) (modified ACI equation) was recorded for specimen S100-VF1 at ratio value equals to 0.741. The average ratio value for Narayanan and Darwish (1987) model is 0.661 and the standard deviation is 0.111 which indicate a good correlation with the experimental results. The average ratio value for Ashour et al. (1992) (Modified Zsutty Equation) model is 0.662 and the standard deviation is 0.111 which indicate a good correlation with the experimental results. The average ratio value for Ashour et al. (1992) (modified ACI equation) model is 0.646 and the standard deviation is 0.109
which indicate a good correlation with the experimental results. From the average and standard deviation in Table 5.3, Ashour et al. (1992) (Modified Zsutty equation) gives a more accurate estimation than Narayanan and Darwish (1987) model and Ashour et al. (1992) (modified ACI equation) model.

5.3.3.2.1.2 Deep Beams

The highest standard deviation for Narayanan and Darwish (1987) was recorded for specimen D100-VF1 at ratio equals to 0.632 whereas the least for Narayanan and Darwish (1987) was recorded for specimen D28-VF1 at ratio value equals to 1.004. The highest standard deviation for Ashour et al. (1992) (Modified Zsutty equation) was recorded for specimen D28-VF2 at ratio equals to 0.550 whereas the least standard deviation for Ashour et al. (1992) (Modified Zsutty equation) was recorded for specimen D28-VF1 at ratio value equals to 0.825. The highest standard deviation for Ashour et al. (1992) (Modified ACI equation) was recorded for specimen D28-VF2 at ratio equals to 0.577 whereas the least standard deviation for Ashour et al. (1992) (modified ACI equation) was recorded for specimen D28-VF1 at ratio value equals to 0.850. The average ratio value for Narayanan and Darwish (1987) model is 0.756 and the standard deviation is 0.127 which indicate a very good correlation with the experimental results. The average ratio value for Ashour et al. (1992) (Modified Zsutty Equation) model is 0.678 and the standard deviation is 0.088 which indicate a good correlation with the experimental results. The average ratio value for Ashour et al. (1992) (modified ACI equation) model is 0.727 and the standard deviation is 0.087 which indicate a good correlation with the experimental results. From the average and standard deviation in Table 5.3, Narayanan and Darwish (1987) model gives a much
more accurate estimation than Ashour et al. (1992) (Modified Zsutty equation) and 
(modified ACI equation) model.

<table>
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<tr>
<th>Beam Type</th>
<th>Specimen</th>
<th>$V_{u,Exp}$ (KN)</th>
<th>$V_{u,Exp}$</th>
<th>Narayanan and Darwish (Eqn. 5.7)</th>
<th>Ashour et al. (Eqn. 5.8 &amp; 5.9)</th>
<th>Ashour et al. (Eqn. 5.10)</th>
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<td>Slender \ ($a/d = 3.3$)</td>
<td>S28-VF1</td>
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<tr>
<td></td>
<td>S28-VF2</td>
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</tr>
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<td></td>
<td>S28-VF3</td>
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Table 5.2: The theoretical values of SFRC using second approach models using
Equation 5.7, Equation 5.8 and 5.9 and Equation 5.10
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<th>Beam Type</th>
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<th>Ratio (V_{u,\text{Th}} / V_{u,\text{Exp}})</th>
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</thead>
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<td></td>
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<td>S28-VF1</td>
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<td>S28-VF2</td>
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Table 5.3: The Ratio of the theoretical values to the experimental values.
Figure 5.3 shows the analytical prediction vs. the experimental results for these three models for slender beams. Figure 5.4 is showing the analytical prediction vs. the experimental results for these three models for deep beams. It can be observed from Figure 5.3 and Figure 5.4 that the data below the equality line indicating conservative prediction. Both figures show that the models show a slightly better prediction for deep beams than slender beams.

![Graph](image)

Figure 5.3: Models of the second approach for slender beams using Equation 5.7, Equation 5.8, and Equation 5.10.
5.3.3.2.2 Kwak et al. (2002), Sharma (1986) and Imam et al. (1997)

Table 5.4 shows the comparison between experimental results and the predicted values using the models of Kwak et al. (2002), Sharma (1986) and Imam et al. (1997). The ratio between the experimental and theoretical for these three models is shown in Table 5.5. It can be seen from these tables that the Kwak et al. (2002) model was conservative in estimating the value of shear strength of SFRC. Sharma (1986) model was very conservative in estimating the value of shear strength of SFRC, while Imam et al. (1997) model overestimate the shear strength for all beams except S28-VF3, S60-VF1 and S60-VF2. The model by Imam et al. (1997)
overestimates the shear strength for deep beams, while it needs a safety factor for slender beams.

5.3.3.2.2.1 Slender Beams

The highest standard deviation for Kwak et al. (2002) was recorded for specimen S60-VF1 at ratio equals to 0.398 whereas the least standard deviation for Kwak et al. (2002) was recorded for specimen S28-VF2 at ratio value equals to 0.681. The highest standard deviation for Sharma (1986) was recorded for specimen S60-VF1 at ratio equals to 0.251 whereas the least standard deviation for Sharma (1986) was recorded for specimen S100-VF1 at ratio value equals to 0.440. The highest standard deviation for Imam et al. (1997) was recorded for specimen S60-VF1 at ratio equals to 0.731 whereas the least standard deviation for Imam et al. (1997) was recorded for specimen S28-VF3 at ratio value equals to 0.984. The average ratio value for Kwak et al. (2002) model is 0.597 and the standard deviation is 0.090 which indicate a relatively poor correlation with the experimental results. The average ratio value for Sharma (1986) model is 0.353 and the standard deviation is 0.058 which indicate a very poor correlation with the experimental results. The average ratio value for Imam et al. (1997) model is 1.105 and the standard deviation is 0.187 which indicate a relatively good correlation with the experimental results. The average for this model shows that the shear strength is overestimated. If this model is to be used to estimate slender beams shear capacity, a safety factor is recommended to be used with it. This could be because Imam et al. (1997) published paper showed that the model was calibrated using only 29 tests of SFRC beams and where some failed due to flexure not shear and that the model in some beams overestimated the shear capacity and that size effect might not be as significant for SFRC beam as conventional beams.
because the failure mode are more ductile in SFRC beams. From the average and standard deviation in Table 5.5, Imam et al. (1997) model gives a more accurate estimation than Kwak et al. (2002) and Sharma (1986).

5.3.3.2.2.2 Deep Beams

The highest standard deviation for Kwak et al. (2002) was recorded for specimen D28-VF2 at ratio equals to 0.532 whereas the least standard deviation for Kwak et al. (2002) was recorded for specimen D28-VF3 at ratio value equals to 0.777. The highest standard deviation for Sharma (1986) was recorded for specimen D28-VF2 at ratio equals to 0.211 whereas the least standard deviation for Sharma (1986) was recorded for specimen D28-VF3 at ratio value equals to 0.336. The highest standard deviation for Imam et al. (1997) was recorded for specimen D28-VF1 at ratio equals to 1.845 whereas the least standard deviation for Imam et al. (1997) was recorded for specimen D100-VF1 at ratio value equals to 1.189. The average ratio value for Kwak et al. (2002) model is 0.666 and the standard deviation is 0.084 which indicate a good correlation with the experimental results. The average ratio value for Sharma (1986) model is 0.288 and the standard deviation is 0.039 which indicate a very poor correlation with the experimental results. The average ratio value for Imam et al. (1997) model is 1.539 and the standard deviation is 0.251 which indicate a very poor correlation and that it is significantly overestimate the value of shear capacity. This makes Imam et al. (1997) model is not conservative to be used in estimating the shear capacity of deep beams. This results confirm with Kwak et al. (2002) finding in his published paper where Imam et al. (1997) model was calibrated using only 29 tests of SFRC beams where some failed due to flexure not shear and that the model significantly overestimated the shear capacity and that size effect might not be as
significant for SFRC beam as conventional beams because the failure mode are more ductile in SFRC beams. From the average and standard deviation in Table 5.5, Kwak et al. (2002) model gives a more accurate estimation than and Sharma (1986) and Imam et al. (1997).

<table>
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<th>Beam Type</th>
<th>Specimen</th>
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<th>$V_{u,Th}$ (KN)</th>
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<td>Kwak et al.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Eqn. 5.11)</td>
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<tr>
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<td>53.3</td>
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<td></td>
<td>S28-VF3</td>
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<td>S100-VF3</td>
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Table 5.4: The theoretical values of SFRC using second approach models using Equation 5.11, 5.12 and 5.13
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<th>Sharma AK. (Eqn. 5.12)</th>
<th>Imam et al. (Eqn. 5.13)</th>
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</thead>
<tbody>
<tr>
<td>Slender ((\alpha/d = 3.3))</td>
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<td></td>
<td>S28-VF2</td>
<td>0.681</td>
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<td>0.187</td>
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<tr>
<td>Deep ((\alpha/d = 2.2))</td>
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<td>D60-VF1</td>
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<td></td>
<td>D60-VF2</td>
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<td>0.313</td>
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<td></td>
<td>D60-VF3</td>
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<tr>
<td></td>
<td>D100-VF1</td>
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<tr>
<td>Average</td>
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<td>0.666</td>
<td>0.288</td>
<td>1.539</td>
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<tr>
<td>Standard Deviation</td>
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<td>0.084</td>
<td>0.039</td>
<td>0.251</td>
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</table>

Table 5.5: The Ratio of the theoretical values to the experimental values using Equation 5.11, 5.12 and 5.13
Figure 5.5 is showing the analytical prediction vs. the experimental results for these three models for slender beams. It can be observed from Figure 5.5 that the data from Kwak et al. (2002) and Sharma (1986) models are below the equality line which indicates conservative prediction. Imam et al (1997) model has some points above the equality line and some points below the equality line. Figure 5.6 is showing the analytical prediction vs. the experimental results for these three models for deep beams. It can be observed from Figure 5.6 that the data from Kwak et al. (2002) model is below the equality line which indicates conservative prediction and a bit close to the equality line. Sharma (1986) model is below the equality line in the conservative side of the graph and very far from the equality line. Imam et al. (1997) model is above the equality line in the unconservative side of the graph and very far from the equality line.

![Graph showing analytical vs. experimental results for slender and deep beams models.](image-url)

**Figure 5.5**: Models of the second approach for slender beams using Equation 5.11, 5.12 and 5.13
5.3.3.2.3 Khuntia et al. (1999) and Shin et al. (1994)

Table 5.6 shows the comparison between experimental results and the predicted values using the models of Khuntia et al. (1999) and Shin et al (1994). The ratio between the between the experimental and theoretical for these two models is shown in Table 5.7. It can be seen from these tables that Khuntia et al. (1999) and Shin et al. (1994) models were conservative in estimating the value of shear strength of SFRC for slender beams. For deep beams, Khuntia et al (1999) model was conservative in estimating the value of shear strength of SFRC, while Shin et al. (1994) model overestimated the shear strength.
5.3.3.2.3.1 Slender Beams

The highest standard deviation for Khuntia et al. (1999) was recorded for specimen S60-VF1 at ratio equals to 0.320 whereas the least standard deviation for Khuntia et al. (1999) was recorded for specimen S100-VF2 at ratio value equals to 0.609. The highest standard deviation for Shin et al. (1994) was recorded for specimen S60-VF1 at ratio equals to 0.447 whereas the least standard deviation for Shin et al. (1994) was recorded for specimen S28-VF1 at ratio value equals to 0.803. The average ratio value for Khuntia et al. (1999) model is 0.448 and the standard deviation is 0.106 which indicate a poor correlation with the experimental results. The average ratio value for Shin et al. (1994) model is 0.646 and the standard deviation is 0.113 which indicate a relatively good correlation with the experimental results. From the average and standard deviation in Table 5.7, Shin et al. (1994) gave a more accurate estimation than Khuntia et al. (1999).

5.3.3.2.3.2 Deep Beams

The highest standard deviation for Khuntia et al. (1999) was recorded for specimen D28-VF2 at ratio equals to 0.274 whereas the least standard deviation for Khuntia et al. (1999) was recorded for specimen D100-VF1 at ratio value equals to 0.376. The highest standard deviation for Shin et al. (1994) was recorded for specimen D28-VF1 at ratio equals to 1.773 whereas the least standard deviation for Shin et al. (1994) was recorded for specimen D28-VF2 at ratio value equals to 1.030. The average ratio value for Khuntia et al. (1999) model is 0.374 and the standard deviation is 0.051 which indicate a very poor correlation with the experimental results. The average ratio value for Shin et al. (1994) model is 1.225 and the standard deviation is 0.256 which indicate a poor correlation with the experimental results and an overestimate of the
This makes Shin et al. (1994) model is not conservative in estimating the shear capacity of deep beams.

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Specimen</th>
<th>$V_{u, Exp}$ (KN)</th>
<th>$V_{u, Th}$ (KN)</th>
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<td>78.3</td>
<td>37.3</td>
<td>61.4</td>
</tr>
<tr>
<td>S28-VF3</td>
<td>120.5</td>
<td>45.4</td>
<td>74.2</td>
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<td>S100-VF3</td>
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<td>139.8</td>
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Table 5.6: The theoretical values of SFRC using models of the second approach using Equation 5.14, 5.15 and 5.16
<table>
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<th>Beam Type</th>
<th>Specimen</th>
<th>Ratio ($V_{th}/V_{Exp}$)</th>
<th>Khuntia et al. (Eqn. 5.14)</th>
<th>Shin et al. (Eqn. 5.15 &amp; 5.16)</th>
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<tr>
<td>Slender (a/d = 3.3)</td>
<td>S28-VF1</td>
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<tr>
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<td>S28-VF3</td>
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Table 5.7: The Ratio of the theoretical values to the experimental values using Equation 5.14, 5.15 and 5.16
Figure 5.7 is showing the analytical prediction vs. the experimental results for these three models for slender beams. Figure 5.8 is showing the analytical prediction vs. the experimental results for these three models for deep beams. The same results as in Table 5.6 and Table 5.7 can be seen in Figure 5.7 and Figure 5.8. The data from Khuntia et al. (1999) and Shin et al. (1994) for slender beams models are below the equality line in the conservative side of the graph. The data from Khuntia et al. (1999) for deep beams models are below the equality line in the conservative side of the graph while the data for Shin et al. (1994) for deep beams model is above the equality line in the unconservative side of the graph.

Figure 5.7: Models of the second approach for slender beams using Equation 5.14 and 5.15
Figure 5.8: Models of the second approach for deep beams using Equation 5.14 and 5.16.

5.3.3.3 Comparison between first approach and second approach

The Comparison between the first approach models and second approach models are shown in Table 5.8. The average and standard deviation for all models are shown in Table 5.8. For the first approach, it is clear that Swamy et al. (1993) model is better in estimating the shear capacity than Al-Ta'an et al. (1990), but it is still clear that both first approach models gave a poor estimation of the shear capacity. For the second approach model, the best model that fit the experimental results for slender beams was different than that of deep beams. For slender beams, the best model that fits with the experimental results is Ashour et al. (Modified Zsutty equation) (1992)
model. Imam et al. (1997) model was excluded although it has a close average value to one ($\text{Ratio} = V_{n, \text{Th}} / V_{n, \text{Exp}} \approx 1$) because it has a relatively high standard deviation value (0.187). Another reason would be because Imam et al. (1994) model overestimates the value of shear capacity for all deep beams. For deep beams, the best models that fit the experimental results is Narayan et al. (1987) model since it has closest average value to one ($\text{Ratio} = V_{n, \text{Th}} / V_{n, \text{Exp}} \approx 1$) and has an acceptable standard deviation (0.127). Shin et al. (1994) model was excluded although it has a close average value to one ($\text{Ratio} = V_{n, \text{Th}} / V_{n, \text{Exp}} \approx 1$) because it has a relatively high standard deviation value (0.256). Another reason would be because shin et al (1994) model overestimates the value of shear capacity for all deep beams.

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<td>Ashour et al. (Eqn. 5.8 &amp; 5.9)</td>
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<td>Ashour et al. (Eqn. 5.10)</td>
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<td>Sharma et al. (Eqn. 5.12)</td>
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<tr>
<td></td>
<td>Shin et al. (Eqn. 5.15 &amp; 5.16)</td>
<td>0.646</td>
<td>0.113</td>
</tr>
</tbody>
</table>

Table 5.8: Comparison between all first and second approaches
Chapter 6: Conclusion and Recommendations

The viability of using steel fiber (SF) as shear reinforcement for reinforced concrete (RC) especially for ultra-high strength self-compacting concrete (UHSC-SCC) has been investigated in this research work. The RC beams were reinforced with three different SF volume fraction ($\nu_f$) (0.4%, 0.8%, and 1.2%). The research work also considers the different behavior of slender beam ($a/d = 3.33$) and deep beam ($a/d = 2.22$). The study comprised experimental testing and analytical investigation. The main conclusions of the work along with recommendations for future research studies related to the topic of this thesis are also provided.

6.1 Conclusion of the experimental Results

Based on the experimental results, the following conclusions are drawn:

- For slender beams of group (A) (28 MPa), the shear strength gain increased with an increase in the steel fiber volume fraction. The shear strength gain ranged from 20% to 129%.
- For slender beams of group (B) (60 MPa), the inclusion of SF increased the shear strength gain, however, increasing the steel fiber volume fraction did not result in an increase in the shear strength gain of the slender beams with concrete grade of 60 MPa.
- For slender beams of group (C) (100 MPa), the shear strength gain ranged from 29% to 94%. The shear strength gain increased with an increase in the steel fiber volume fraction.
- For the slender beams with steel fiber volume fractions of 0.4% and 0.8%, varying the concrete grade had no obvious effect on the shear
strength gain. Nevertheless, for the slender beams with the higher steel fiber volume fraction of 1.2%, the shear strength gain tended to decrease with an increase in the concrete grade.

- For deep beams of group (A) (28 MPa), the shear strength gain ranged from 23% to 128%. Increasing the steel fiber volume fraction from 0.4% to 0.8% increased the shear strength gain. Further increase the steel fiber volume fractions from 0.8% to 1.2% did not result in additional shear strength gain.

- For deep beams of group (B) (60 MPa), the shear strength gain ranged from 26% to 63%. The shear strength gain increased with an increase in the steel fiber volume fraction.

- For deep beams of group (C) (100 MPa), the shear strength gain for the beam with the SF ($v_f = 0.4\%$) was (8.6%).

- For deep beams, the shear strength gain tended to decrease by increasing the concrete grade. That was more evident for the deep beams having the higher steel fiber volume fractions of 0.8% and 1.2%.

6.2 Conclusions of the analytical Investigation

Various analytical models were studied in this research work. It was divided in two approaches. Based on the analytical investigations, the following conclusion are drawn:

- For the first approach, both models were very conservative in predictions the shear capacity of steel fiber reinforced concrete (SFRC).
• For the second approach, it was observed for slender beams that all models were conservative in estimating the shear capacity except Imam et al. (1997) model.

• For the second approach, it was observed for deep beams that all models were conservative in estimating the shear capacity except Imam et al. (1997) model and Shin et al. (1994) model.

• The second approach models in general gave a more accurate estimation than that of first approach models. The best model that fits with the experimental results for slender beams was Ashour et al. (1992) model (Modified Zsutty equation). The best model that fits with the experimental results for deep beams was Narayanan et al. (1987) model.

6.3 Recommendation for future studies

Findings of this research work provided insights into the effectiveness of using SF as shear reinforcement. Further research is needed to enrich the literature and support development of design guidelines and standards on the subject. The following are recommendations for future studies in this area:

• Study the effect of SF on UHS-SCC in Deep beams with higher volume fractions.

• Study of the size effect of test specimens on the effectiveness of SF as shear reinforcement

• Effect of using SF in combination with transverse reinforcement (stirrups) as shear reinforcements.
• Study the performance of SFRC on T and I girders and the effect of SF distribution in the thin web.

• Study of the performance of SFRC shear strength under repeated or fatigue loading.

• Shear performance under harsh environmental conditions and the possibility of corrosion of SF should also be studied.

• Develop a finite element model based on the experimental works from this study and other experimental study from the literature.

6.4 Recommendation for practical applications

Based on results of the present research, the following recommendations can be made for successful applications of SFRC beam as shear reinforcements.

• For slender beams, the shear gain shows the SF can be used as shear reinforcement in conditions that the SF does not affect the workability of the concrete. It is advised that the maximum $\nu_f$ to be used is 1.2% since this volume fraction allowed the concrete to maintain its workability and increase the shear capacity by minimum of 48.6%. Also, the deflection at first major crack was significantly increased for beams and exceeded that with stirrups.

• For deep beams, the shear gain shows the SF can be used as shear reinforcement in conditions that the SF does not affect the workability of the concrete. It is advised that the $\nu_f$ to be used within this range 0.8% to 1.2% since this volume fraction allowed the concrete to maintain its workability and increase the shear capacity by minimum of 44.6%.
Also, the deflection at first major crack was significantly increased for beams and exceeded that with stirrups.

- For slender beams, it is recommended that Ashour et al. (Modified Zsutty equation) model to be used in estimating the shear capacity of SFRC.

- For deep beams, it is recommended that Narayanan et al. model to be used in estimating the shear capacity of SFRC.
Bibliography


List of Publications

Appendix

Figure A.1: Tensile steel response at load point for slender beams (group A = 28 MPa)

Figure A.2: Tensile steel response at mid-span for slender beams (group A = 28 MPa)
Figure A.3: Compressive steel response at load point for slender beams (group $A = 28$ MPa)

Figure A.4: Tensile steel response at load point for deep beams (group $A = 28$ MPa)
Figure A.4: Tensile steel response at mid-span for deep beams (group A = 28 MPa)

Figure A.5: Compressive steel response at mid-span for deep beams (group A = 28 MPa)
Figure A.6: Tensile steel response at load point for slender beams (group B = 60 MPa)

Figure A.7: Tensile steel response at mid-span for slender beams (group B = 60 MPa)
Figure A.8: Compressive steel response at load point for slender beams (group B = 60 MPa)

Figure A.9: Tensile steel response at load point for deep beams (group B = 60 MPa)
Figure A.10: Tensile steel response at mid-span for deep beams (group B = 60 MPa)

Figure A.11: Compressive steel response at load point for deep beams (group B = 60 MPa)
Figure A.12: Tensile steel response at load point for slender beams (group C = 100 MPa)
Figure A.13: Tensile steel response at mid-span for slender beams (group C = 100 MPa)

Figure A.14: Compressive steel response at load point for slender beams (group C = 100 MPa)
Figure A.15: Tensile steel response at load point for deep beams (group C = 100 MPa)

Figure A.16: Tensile steel response at mid-span for deep beams (group C = 100 MPa)
Figure A.17: Compressive steel response at load point for deep beams (group C = 100 MPa)

Figure A.17: Tensile steel response for stirrup near load point at (H3) for slender beams
Figure A.18: Tensile steel response for stirrup near load point at (D3) for deep beams

Figure A.19: Tensile steel response for mid stirrup at (H2) for slender beams
Figure A.20: Tensile steel response for mid stirrup at (D2) for deep beams

Figure A.20: Tensile steel response for near support stirrup at (H1) for slender beams
Figure A.21: Tensile steel response for near support stirrup at (D1) for slender beams.