SHEAR STRENGTHENING OF REINFORCED CONCRETE BEAMS WITH VARIOUS COMPOSITE STRENGTHENING SYSTEMS

by

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SHEAR STRENGTHENING OF REINFORCED CONCRETE BEAMS WITH VARIOUS COMPOSITE STRENGTHENING SYSTEMS

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ABSTRACT

Across Canada and around the world, damaged and aging infrastructure is a major issue facing civil engineers and infrastructure planners. Funding to replace these aging structures is unlikely to become available, therefore, economical repair techniques, which can return a structure to its original state are required. Several of those techniques are mortar adhered near surface mounted (NSM) fibre reinforced polymer (FRP) bars and fabric reinforced cementitious mortar (FRCM) wraps. These composites overcome financial and safety related concerns of previously used repair methods, while delivering similar levels of strengthening efficiency. This study was designed to investigate the effectiveness of these two composite strengthening systems on shear-damaged reinforced concrete (RC) beams with different levels of damage (1st shear crack and 70% of maximum theoretical load), adhesives (mortar and epoxy), wrapping patterns (continuous and intermittent), and different stirrup spacings (150mm or 200mm). Fifteen half-scale (200x265x2000 mm) RC beams (2 control, 6 strengthened without damage, and 7 strengthened after damage) were cast and tested under monotonic three-point bending conditions. Load, stiffness, mid-span displacement, pseudo-ductility, energy absorption, and failure mode were used as performance indicators. Results showed that most of the beams failed due to debonding failure between the strengthening material and the concrete substrate. The strengthening systems did not have a significant effect on the beams’ stiffness, however, all the strengthening systems were able to increase the load bearing capacity, the pseudo-ductility, and the energy absorption. Load bearing capacity was increased between 22% and 42%, depending on the strengthening system used. Recommendations on the maximum design strains in the different strengthening systems were also made.
LAY SUMMARY

One of the biggest issues faced by modern civil engineers is the deterioration of aging concrete structures. In the past, concrete structures were built without regard for long term durability and, therefore, no plans were made to maintain these structures. As these structures continue to age, they may become unsafe and pose a threat to public safety. Additionally, structures which were once safe may be damaged due to overload, impact, or corrosion. In order to remedy the danger presented, the structures may be repaired using various methods. In the repair of reinforced concrete (RC) structures several of these methods include the use of fabric reinforced cementitious mortar (FRCM) and near surface mounted (NSM) fibre reinforced polymer (FRP) bars. One of the most dangerous structural failure modes is shear failure wherein the structure fails rapidly with little warning. In this study, damaged and intact beam specimens were strengthened using FRCM and NSM FRP bars with varying application parameters. Based on results of the investigation recommendations on the use of FRCM and NSM FRP bars were presented.
PREFACE

The findings presented in this thesis are based on original experimental work completed at the Okanagan Campus of the University of British Columbia by the author. The author was solely responsible for the production of the entire work, including the literature review, acquisition of materials, construction of specimens, experimental work, collection and analysis of data, and thesis writing. Dr. Ahmad Rteil provided guidance and supervision throughout the entire process. Portions of Chapters 2, 3, 4, and 5 have been submitted to peer-reviewed conferences and journals for publication. The following is a list of these publications:


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<th>Acronym</th>
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<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
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<tr>
<td>CSA</td>
<td>Canadian Standards Association</td>
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<tr>
<td>DAQ</td>
<td>Data Acquisition</td>
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<tr>
<td>FRCM</td>
<td>Fabric reinforced cementitious mortar</td>
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<tr>
<td>FRP</td>
<td>Fibre reinforced polymer</td>
</tr>
<tr>
<td>NSM</td>
<td>Near surface mounted</td>
</tr>
<tr>
<td>PBO</td>
<td>Polyphenylene benzobisoxazole</td>
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<tr>
<td>RC</td>
<td>Reinforced concrete</td>
</tr>
<tr>
<td>TRC</td>
<td>Textile Reinforced Concrete</td>
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<tr>
<td>TRM</td>
<td>Textile reinforced mortar</td>
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ACKNOWLEDGEMENTS

The thesis presented herein is the culmination of a large amount over the last year and a half. An undertaking of this magnitude would be insurmountable alone, and I am proud to acknowledge the help of everyone who made this possible. First, I would like to thank Alec Smith, Kim Nordstrom, Luis Diaz, Durwin Bossy, Praveen Rajan, Chris Seib, and Sherif Osman for their assistance in the Construction Materials and Structures Lab at UBC Okanagan. I would also like to thank Andre Harrichhausen from Structural Concrete Repair Contracting Services for donating his time and equipment to prepare my specimens for repair. As well, I would like to thank Adam Brockman from Crown West Steel for the partial donation of the testing equipment. Furthermore, I would like to thank NSERC for their financial support through the CGS Masters’ Program, without this aid the project would not have been possible.

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DEDICATION

I would like to dedicate this paper to my parents, Travis and Dawn Kennedy and my wife Michelle Jones. Without the support of my parents I would have neither the means nor the ability to have completed this journey. Without the love and support of my wife I would not have had the inspiration to finish this work or any of my other academic endeavours.
CHAPTER 1: INTRODUCTION

1.1 Problem Statement

Since the development of reinforced concrete (RC) in the late 1800’s it has grown to become the most commonly used construction material in the world (Gagg, 2014). When many of the structures were built the durability and deterioration mechanics of concrete were not well understood, which has resulted in many older structures falling into very poor condition and presenting a hazard to public safety (Hong, 2000). Some of the factors which have negatively impacted these structures include improperly designed concrete mixes, inadequately designed structural elements, reinforcement corrosion, overloading, and lack of routine maintenance (Hobbs, 2001). These issues, combined with changes to a given structure’s use (increased service loads) and changes to design codes (more stringent design requirements), have resulted in a major problem facing engineers and infrastructure planners (Hobbs, 2001). To combat this problem, cost effective repair and rehabilitation methods are required.

While poor and continually deteriorating nature of North American infrastructure has been acknowledged for some time, major efforts have still not been taken to resolve the issue. Canada’s infrastructure report card (2019), rates the infrastructure on a scale of very poor, poor, fair, good, very good, and unknown. Due to concerns regarding financial data integrity, the 2019 version of the report card does not include financial information, therefore, references to both the 2019 and 2016 report cards are made, when necessary. In 2016, it was estimated that 26% of Canada’s bridges were in fair to very poor condition, with a total replacement cost of $5.4 billion (PSC, 2016). In the 2019 report, it was found that 38% of Canada’s infrastructure was in fair to very poor condition. While the amount of infrastructure surveyed in the 2019 report was much higher, it can
be seen that either Canada’s infrastructure is not receiving suitable funding and, therefore, the condition is growing worse, or that the amount of fair to very poor infrastructure is higher than initially thought (PSC, 2019). In another report by Mackenzie (2013), the issue of infrastructure gap is explored further. Historical spending has not been adequate to maintain Canada’s infrastructure. Infrastructure spending has shown a decreasing trend in Canada since the 1950’s when recording began and reach a low point of 1.5% of GDP in the late 1990’s and early 2000’s. While the report recognizes that there have been recent efforts to close the infrastructure gap, it can still be seen that current spending is below the target values required to prevent a decline in the condition of both roads (1.1% vs 2.0%-3.0%) and bridges (0.8% vs 1.0%-1.7%) (Mackenzie, 2013). In 1998, the ASCE infrastructure report card was released. The report found an average infrastructure grade of D. In the report, bridges received a condition score of C-, dams received a score of D, and roads received a score of D- (ASCE, 1998). Despite this problem being known, there is still insufficient spending to improve the condition of the infrastructure. In the most recent American Society of Civil Engineers (ASCE) Infrastructure Report Card (2017), the bridges condition score has increased slightly to a C+, however there is still significant improvements required with about $123 billion required to repair the bridges. This report also states that 9.1% of the country’s bridges have been found to be structurally deficient, with 188 million trips being made over these unsafe structures every day. In addition to bridges, it was noted in the infrastructure report card that $1.15 trillion are required to repair and maintain the levees, high-hazard dams, roads, schools, and wastewater facilities (ASCE, 2017). While infrastructure spending has increased slightly, it has been observed that the increase is not enough to counteract the effects of aging on the structures. From this overview, it is apparent that substantial increases
in infrastructure spending are unlikely to be the solution to closing the infrastructure gap. Another avenue to close this gap is the development of more cost-effective structural repair methods.

One of the most critical failure modes which can occur in RC is shear failure. In shear failures, collapse occurs with little to no warning, making it highly dangerous to the public (Brzev & Pao, 2016). Shear failures in RC are caused by a lack of transverse steel reinforcement, known as stirrups. Due to the mechanics of shear, (discussed in depth in section 2.4 Shear in Reinforced Concrete) failure occurs immediately after the steel stirrups yield resulting in a non-ductile failure (ACI 318-14, 2014; CSA A23.3-14, 2014). Shear failure can occur due to construction deficiencies or damage to the structure after construction is complete (Hobbs, 2001). To protect against shear failure innovative repair techniques are required.

While many methods have been developed to strengthen and repair structures such as section enlargement, addition of steel plates, and post tensioning using steel tendons major drawbacks still limit their financial feasibility (Ramirez, 1996). These drawbacks include geometric limitations, installation difficulties, and the corrosion sensitive nature of steel (Ramirez, 1996; Gowda, 2016). One of the major breakthroughs in structural repair and rehabilitation came in the form of fibre reinforced polymers (FRP) (GangaRao, Taly, & Vijay, 2006). FRP is a composite material with two methods of installation used in structural repair and rehabilitation, near surface mounted (NSM) bars and externally applied laminate wraps (Bank, 2006). FRP can overcome the shortfalls of traditionally used methods. One economic study showed that the use of FRP repair could reduce the cost of a project to less than 10% of the replacement cost (GangaRao, Taly, & Vijay, 2006). Despite these advances there are few issues introduced when working with FRP. The polymer resin used in the FRP application process represents many of these shortcomings. The issues
relating to the use of resin include toxic fumes released during application, bond between the resin and the concrete substrate, poor mechanical performance of the resin at high temperatures, and high material costs of the resin (Koutas, Bournas, & Triantafillou, 2019). To overcome these issues, the academic community began searching for an alternative solution. One solution for NSM bars is to replace the adhesive with mortar (Al-Mahmoud, Castel, Francois, & Tourneur, 2011). For externally applied laminates a more ambitious solution came in the form of fibre reinforced cementitious mortar (FRCM) which replaces the resin with cement-based inorganic binder (RILEM Technical Committee 201-TRC, 2006).

FRCM and mortar adhered NSM FRP bars show promise as repair materials which offer the same benefits of traditional FRP applications, while overcoming its shortfalls. While there have been many research studies into strengthening new structures with FRCM and mortar adhered NSM FRP bars, the repair of damaged structures is still not well understood. Despite these initial studies, FRCM and mortar adhered NSM FRP bars are still not ready to be applied in the construction industry due to a lack of design codes to support their use. By continuing to research the use of composites, design codes can be written which will allow them to be used in the construction industry. This research effort was undertaken to investigate the effect of various composite strengthening systems in returning a shear damaged member to its original capacity with different variations in parameters.

1.2 Thesis Layout

This thesis is organized into the six following chapters:

Chapter 1 presents an overview of the infrastructure gap and where this research effort falls to close that gap.
Chapter 2 is a literature review which provides background on FRP and FRCM strengthening methods. Once current research into these materials has been outlined the limitations of the current body of knowledge are then presented and the objectives of this research endeavor are outlined.

Chapter 3 introduces the experimental program which was used to meet the objectives. It includes the test matrix, the mechanical properties of the materials used, descriptions of the test specimens, specimen preparation and instrumentation, the test setup which was used to assess the specimens, and the damage mechanism which was applied to the specimens.

Chapter 4 presents the data collected during the testing of the specimens described in Chapter 3. Performance indicators used include peak load, failure mode, and strain in the strengthening materials, rebar, and concrete.

Chapter 5 includes the analysis of the different test parameters. The model which was used to predict the capacity of the original members is presented and modified based on the experimental findings. Finally, design recommendations regarding the maximum strains in the materials are presented.

Chapter 6 provides a conclusion of the research effort. As well, limitations of the study and recommendations for future research are presented.
CHAPTER 2: LITERATURE REVIEW AND RESEARCH OBJECTIVES

2.1 Introduction

Based on the condition of existing civil infrastructure and the issue of the infrastructure gap it is known that cost effective strengthening and repair methods are needed. Due to a combination of economic and environmental factors, it is favorable to repair structures rather than rebuilding them entirely. Several existing methods already exist showing moderate effectiveness, these include section enlargement, post tensioning, externally bonded plates, and externally bonded FRP.

Section enlargement, as the name implies, involves increasing the cross-sectional area of the member in order to increase its load carrying capacity and stiffness (Banu & Taranu, 2010). In order to accomplish this, formwork and rebar are placed around the existing member, then additional concrete is placed (Gagg, 2014). To ensure that the section enlargement is effective the new concrete and the substrate must perform monolithically. Strain incompatibility between the original and repair materials can cause the repair to fail. While section enlargement is simple and intuitive, it is not an efficient way to increase a member’s strength. If significant strength gains are required, then significant area increases are also required and are often not possible due to geometric constraints. Additionally, the section enlargement procedure is highly labour and cost intensive (Banu & Taranu, 2010).

External post tensioning involves using high strength tendons or bars prestressed to high values to counteract the effects of applied loads on a structure (Banu & Taranu, 2010). Post tensioning can be used to efficiently increase the load bearing capacity and decrease deflections of existing structures (Gagg, 2014). The main benefits of this repair method include good strength to weight
ratio of the materials, short application time, and minimal intrusiveness. Despite these benefits, there are major drawbacks associated with this method including specialized training required for installation, high cost of materials, corrosion susceptibility of steel tendons, poor fire resistance of both steel and FRP tendons, and potential safety hazards if the tendons were to fail and rapidly unload (Banu & Taranu, 2010; Gagg, 2014).

Both steel and FRP plates and jacketing have been used to repair and strengthen RC structures. The plates or jacketing material are either anchored or bonded to the concrete to allow for a transfer of load to the strengthening system. The advantages of steel systems include ease of application, low cost of materials, and ductile behavior (Radomski, 2002; Gagg, 2014). The drawbacks of steel are heavy materials, corrosion susceptibility, poor fire resistance, and increased self weight of the member (Banu & Taranu, 2010). The advantages offered by FRP systems are good strength to weight ratio, and improved corrosion resistance compared to steel (GangaRao, Taly, & Vijay, 2006). The drawbacks of FRP are specialized application process, poor fire resistance, and reduced ductility (Bank, 2006).

When considering which of these solutions to implement a practicing engineer must consider factors related to the specific repair project. There is no one size fits all method which is suitable to repair every type of damage.

2.2 Fibre Reinforced Polymer (FRP)

Fibre reinforce Polymer (FRP) is a composite material which was first developed in the mid-20th century. Due to the attractive material properties of FRP, it has grown to see use in structural engineering and many other industries. The FRP composite is composed of fibres, typically carbon, glass, aramid, or basalt, encased within an organic resin (ACI 440.1R-15, 2015; CSA
The fibres are selected due to their high strength and light weight properties. Many different types of resin have been used to create FRP composites including polyester, epoxy, and vinylester. There has been a large amount of research into the physical properties of FRP and its use as a construction material. The major benefit of FRP is its high strength to weight ratio (GangaRao, Taly, & Vijay, 2006). This allows for fast and flexible application of FRP for structural applications. Another benefit of FRP include improved corrosion resistance compared to steel. External FRP wrapping has been shown to help protect internal steel from the effects of corrosion. Research into the material behavior of FRP has shown it to have a linear elastic stress-strain curve, with no yielding as shown in Figure 2.1. This means that while FRP is much stronger, it has limited ductility and energy dissipation relative to the steel. It should be noted that, Figure 2.1 is intended to highlight the strength and stiffness of the FRP materials relative to the steel.

![Figure 2.1 FRP stress strain curve](ACI 440.1R-15, 2015) © with permission from ACI
Many different applications of FRP as an RC strengthening material have been explored. FRP is able to provide effective flexural strengthening, shear strengthening, increase column capacity and ductility, improved seismic response of structures, and improvement of internal bond of spliced rebars (as described in section 2.3.3 FRCM Uses) (Bank, 2006; GangaRao, Taly, & Vijay, 2006; CSA S807-10, 2010; ACI 440.1R-15, 2015). As the behavior of FRP strengthened RC is well known, codes have been written to allow for the design of structures incorporating FRP. In Canada, there are 3 codes produced by the Canadian Standards Association (CSA). CSA S806 and S6 details the use of the design of FRP for buildings and bridges, respectively. CSA S807 describes the material specifications for the production of FRP products. Around the world other countries have also written design codes or guidelines to allow for the safe use of FRP systems (JSCE Concrete Engineering Series 23, 1997; SIA166, 2004; ECP 208-2005, 2005; CNR-DT 200 R1/2013, 2013; ACI 440.1R-15, 2015).

The three major types of FRP strengthening systems are externally bonded plates, hand layup sheet with epoxy systems, and near surface mounted bars. These three strengthening systems are shown in Figure 2.2. Despite the advantages of FRP it has several key drawbacks, these include toxic fumes released by the epoxy polymer during the application process, poor performance in fire conditions, limitations to application on cold, wet or rough surfaces, and UV degradation of polymer (Mobasher, 2012). These drawbacks originate from the use of the organic resin. In addition to the drawbacks directly associated with the resin, the FRP composite also suffers from a sudden debonding failure, resulting in loss of load bearing capacity (Bank, 2006). For these reasons, a safer and more efficient repair material is sought, which will maintain the same strengths as FRP without the associated drawbacks.
2.2.1 Near Surface Mounted Bars

In the last several decades there has been a push towards developing cost effective reinforced concrete repair and rehabilitation techniques. One such technique is the use of near surface mounted (NSM) FRP bars (Bank, 2006; GangaRao, Taly, & Vijay, 2006). This method involves cutting grooves into the concrete member, and using a bonding agent to install the FRP bar into the grooves, as shown in Figure 2.3. The different types of strengthening possible with NSM bars include flexure, shear, and beam column joint strengthening (Bank, 2006; GangaRao, Taly, & Vijay, 2006; CSA S807-10, 2010; ACI 440.1R-15, 2015). Flexural strengthening involves cutting grooves into the tension face of the beam and installing the bars (Sharaky, Torres, Comas, & Barris, 2014). For the case of shear strengthening, the grooves are cut at either 90° or 45° angles to the longitudinal axis of the beam (Dias & Barros, 2017). Beam column joint strengthening typically
involves cutting grooves into the column to embed NSM bars to increase the flexural capacity. Once the bars have been installed the entire joint is wrapped with externally bonded FRP, in order to provide confinement to both the NSM FRP bars and concrete (Prota, Nanni, Manfredi, & Cosenza, 2004). While NSM steel systems have been used in the past, FRP bars have several advantages that make them ideal for use in NSM systems. They have increased corrosion resistance compared to steel meaning less groove depth is required (Mobasher, 2012). Additionally, the FRP is much stronger than steel, therefore, less material is required to achieve the same strength (Bank, 2006). Finally, the light weight of the FRP bars can increase the speed and ease of installation (GangaRao, Taly, & Vijay, 2006). These factors combine to yield a much more efficient strengthening system than steel.

Figure 2.3: NSM FRP Schematic

When compared to externally bonded FRP sheets the NSM system offers several advantages. The first advantage is better protection of the FRP from external damage caused by either impact or fire (Bilotta, Ceroni, Di Ludovico, & Nigro, 2011). As well, the NSM system is easier and faster to apply, as there is much less surface preparation needed (cutting the grooves vs roughening the
entire surface where sheets would be placed) (Bilotta, Ceroni, Di Ludovico, & Nigro, 2011). Another advantage is improved bond capacity (Bilotta, Ceroni, Di Ludovico, & Nigro, 2011). As well, the NSM system can be more easily prestressed. The final advantage of NSM systems is that there is virtually no aesthetic change before and after the bars have been installed (Bank, 2006).

2.2.2 Uses

There has been a significant research effort into the performance of FRP NSM systems. The areas studies include bond, flexural strengthening, shear strengthening, and beam-column joint strengthening. Bilotta et al. (2011) compared the bond performance of NSM and externally bonded systems, showing that NSM systems exhibited superior bond. The NSM FRP system exhibited 36%-100% utilization of the materials strength compared to approximately 15% for the externally bonded system (Bilotta, Ceroni, Di Ludovico, & Nigro, 2011). Sharaky et al. (2014) studied the flexural behavior of RC beams strengthened with NSM FRP bars. In this study carbon (CFRP) and glass (GFRP) bars were used. It was found that the carbon bars increased the yield and ultimate capacity of the beams by 156% and 166%, compared to the control specimen. The glass bars were found to increase the yielding and ultimate loads by 130% and 159%. It was also found that the CFRP strengthening caused higher member stiffness than the GFRP (Sharaky, Torres, Comas, & Barris, 2014). Proatta et al. (2004) explored the strengthening of beam column joints using a combination of NSM FRP bars and external FRP wraps. It was found that NSM bars alone increased the strength of the column but did not increase the overall ductility of the system. For this reason, it was necessary to also provide the external FRP wrap. The NSM bar-external wrap combination increased the storey shear capacity and maximum drift angle by up to 75% (Proatta, Nanni, Manfredi, & Cosenza, 2004).
2.2.3 Failure Modes

Various failure modes have been observed for RC beams strengthened with NSM FRP bars. These failure modes include typical failure modes for RC beams, such as concrete crushing in compression and diagonal tension failure. The most ductile failure mode for NSM FRP bars is FRP rupture. This failure mode occurs when there is adequate bond length between the NSM bars and the substrate allowing the FRP bars to reach their full capacity after the longitudinal steel has yielded. The longitudinal steel yielding is what gives this failure mode its higher ductility. The first of the brittle failure modes are debonding failures. This type of failure mode occurs when there is a loss of composite action between the adhesive-concrete interface or when there is delamination of concrete cover around the FRP bar. This failure mode occurs due to a lack of strain compatibility between the two materials and can be assisted by flexural cracks which propagate along the bonded area between the two materials (Dias & Barros, 2010). Selecting the right bonding agent and using proper surface preparation techniques can help prevent this failure (Dias & Barros, 2017). Pullout of the FRP bar from the adhesive matrix is also possible (Bilotta, Ceroni, Di Ludovico, & Nigro, 2011). This failure mode occurs when bond between the bar and the adhesive matrix fails. Ribbed or sand coated bars have been implemented to help prevent this failure mode, by increasing the bars’ bond capacity (CSA S807-10, 2010; ACI 440.1R-15, 2015). However, bar pullout may still occur in cases with very short development length or low strength adhesive (CSA S807-10, 2010). In specimens with inadequate development length, adhesive splitting has also been observed. In this failure mode, high stress around the diagonal shear crack may cause the adhesive layer to split around the FRP bar and cause the NSM system to lose load bearing capacity (Al-Saadi, Mohammad, Al-Mahaidi, & Sanjayan, 2019). Research has shown this failure mode can be prevented by using anchorage or inclining the bars at a 45° angle (Al-
Mahmoud, Castel, François, & Tourneur, 2011; Dias & Barros, 2017). In shear critical beams without internal steel stirrups, mass concrete delamination has been observed. Typically, high tensile stress will occur around the longitudinal steel reinforcement in reinforced concrete beams (De Lorenzis & Nanni, 2001). However, because the NSM FRP bars do not provide confining action, it is possible these forces will lead to the complete debonding of concrete cover in the highest stress regions (Al-Saadi, Mohammad, Al-Mahaidi, & Sanjayan, 2019).

2.3 Fabric Reinforced Cementitious Mortar (FRCM)

To overcome the previously discussed limitations of FRP a new repair material, called fabric reinforced cementitious mortar (FRCM) has been developed. This composite has also been referred to, in the literature, as textile reinforced mortar (TRM). Research into FRCM began in the 1980’s, however, serious consideration of the material only began in the early 2000’s (Triantafillou, 2016). Fabric reinforced cementitious mortar is a two-part composite material composed of a fabric grid and mortar binder (RILEM Technical Committee 201-TRC, 2006). The fabric provides the tensile strength in the composite, therefore, it is vital that a strong textile material is used. Several types of fabric which are suitable for use in FRCM are carbon, glass, basalt, and Polyphenylene benzobisoxazole (PBO) (ACI 549.4R, 2013). Fabrics used in FRCM are termed as “dry fabric” meaning that the fibres are not coated with resin during production. These fabrics must be compatible with the mortar, early testing of the tight grid style of fabrics used in FRP laminates were unable to become fully impregnated by the mortar and, therefore, had a poor bond (Triantafillou & Papanicolaou, 2006). For this reason, loose grid style fabrics are often used so that the fibres are able to completely bond with the matrix. Methods used to achieve this grid pattern are weaving, knitting, and bonding as shown below in Figure 2.4. Another
consideration of the fabric is that they must be compatible with the highly alkaline environment of the mortar (ACI 549.4R, 2013).

Figure 2.4: Fabric manufacturing methods [(ACI 549.4R, 2013) © with permission from ACI]

The types of mortar used in FRCM are often high strength, low shrinkage mortars with good workability. These mortars are often enhanced with either polymers or short fibres distributed into the mix to improve the mortars mechanical properties (Mobasher, 2012; ACI 549.4R, 2013).

2.3.1 Mechanical Properties

As a composite material, FRCM has relatively complex stress-strain behavior. The two developed methods of testing FRCM tensile behaviour, clevis grip and clamping, exhibit two phase and three phase stress strain-behaviours, respectively (Figure 2.5). In both models the first phase is uncracked mortar, this is the stiffest phase, however, due to the tensile strength of the mortar it is very short. The next phase, only present when performing testing by clamping the specimen, is unstable crack formation. This is the least stiff phase and is induced by additional compressive forces induced by the clamping. The final phase is stable cracking. It is less stiff than either the uncracked mortar or fabric alone. The loss of stiffness compared to the fabric is thought to be
caused by partial rupture of fabric in earlier phases (Billows, 2016; Arboleda, Carozzi, Nanni, & Poggi, 2016).

**Figure 2.5**: Stress-Strain curves of FRCM [(Arboleda, Carozzi, Nanni, & Poggi, 2016) © with permission from ASCE]

### 2.3.2 Failure Modes

There are many unique ways that FRCM strengthening can fail. These include concrete-mortar debonding, delamination, mortar-fabric debonding or fabric slippage, and fabric rupture. These failure modes are primarily influenced by the number of layers of fabric and the presence of anchorage.

According to Tzoura and Triantafillou (2016), concrete-mortar interface debonding is caused by strain incompatibility between the FRCM composite and the concrete substrate. This incompatibility causes the two materials to peel apart, it often occurs at the same time as delamination (Tzoura & Triantafillou, 2016). This failure mode is not ductile and occurs rapidly (Tzoura & Triantafillou, 2016).
Delamination refers to the concrete substrate directly under the FRCM pulling off of the rest of the concrete in the member, resulting in failure. It is one of the most common failure modes in FRCM shear strengthening. This failure mode is not ductile (Loreto, Babaeidarabad, Leardini, & Nanni, 2015).

While they are similar to one another, mortar-fabric debonding and fabric slip, are actually two distinct types of failure. Mortar-fabric debonding is caused by an insufficient bond between the mortar and fabric. Fabric slip is caused by insufficient bond within the fabric bundle, causing it to “telescope” as the internal and external strands pull apart. According to Tetta et al. (2015) they are most prevalent when a low number of layers of reinforcement are used. These failure modes are the most ductile as they are the only modes in which pseudo-ductile behavior is observed (Tetta, Koutas, & Bournas, 2015).

Fabric rupture occurs when there is sufficient bond between all of the materials in the composite and the concrete cover does not peel off (ACI 549.4R, 2013). In this failure mode the fabric is stressed to its ultimate point and ruptures. While this failure mode can be slightly ductile based on the nature of the fabric used, it still will not be very ductile compared to steel, as FRCM does not experience yielding behavior (Koutas, Bournas, & Triantafillou, 2019).

2.3.3 FRCM Uses

There are many different applications for FRCM. These include both as internal reinforcement for new structures, as well as, a variety of external application usages including flexural reinforcement, seismic reinforcement, and shear reinforcement.
When used as internal reinforcement in new structures, FRCM is often referred to as textile reinforce concrete (TRC). In these structures, the fabric replaces the steel as reinforcement and a modified mortar is used in place of traditional portland cement concrete. The fabrics are the same types of fabric described above in section 2.3 Fabric Reinforced Cementitious Mortar (FRCM). According to Reunion Internationale des Laboratoires et Experts des Materiaux (RILEM Technical Committee 201-TRC, 2006), the modified mortar utilizes extremely fine aggregate (<2mm) and possesses both high strengths (>90 MPa) and high flowability, by use of superplasticizer, to ensure complete fabric impregnation.

TRC offers several advantages compared to traditional reinforced concrete. These include thinner sections, as thin as 20mm, which allow for both lighter members, an aesthetically pleasing finish, and the ability to create unique shapes. However, there are several drawbacks of TRC, the first being the extremely thin sections do not provide fire resistance to the fabric, meaning that the high temperature performance of the member is governed by the fabric properties (RILEM Technical Committee 201-TRC, 2006). The second according to RILEM (2006) is that since TRC is a new material, there are higher costs associated with its production. Additionally, the long-term performance of TRC has not yet been observed. TRC has been used in many different applications these include: exterior panels, shells, formwork, and pipes (RILEM Technical Committee 201-TRC, 2006).

Both beams and slabs have been strengthened in flexure using FRCM, with strengthening having been found to increase the capacity of beams by over 90% (Babaeidarabad, Loreto, & Nanni, 2014; Aljazaeri & Myers, 2016). The FRCM is applied to the tension face of the member, typically the bottom face in simply supported members. Only the fibres in the longitudinal
direction are effective in this strengthening application making the use of two-way fabrics questionable for flexural strengthening. The exception to this statement is the strengthening of two-way slabs, which utilize both fabric directions. The effect of FRCM strengthening on the load-deflection behavior is shown in Figure 2.6 (Koutas, Bournas, & Triantafillou, 2019).

The use of FRCM as a seismic strengthening solution is still relatively new, however, the research which has been done has yielded promising results (Bournas, Triantafillou, Zygouris, & Stavropoulos, 2009; Bournas & Triantafillou, 2011; Tzoura & Triantafillou, 2016). Most of the existing research which has been done focuses on the repair and strengthening of columns. The

![Figure 2.6: FRCM flexural load deflection curve](image-url)
three main seismic strengthening strategies include, plastic hinge confinement, lap splice confinement, and shear strengthening are shown in Figure 2.7. In all the strengthening techniques the primary fabric direction would be in the transverse direction, as shown in Figure 2.7. If two-way fabric is used, the stiffness of the column may be increased, however, this effect is not often desired, making the use of two way fabrics uneconomic for use in seismic strengthening (Bank, 2006).

![Figure 2.7: Seismic strengthening techniques](Image)

While all strengthening techniques involve wrapping the column, the outcome of each strengthening technique is different. The purpose of plastic hinge confinement is to increase the columns ductility and to increase the energy which can be dissipated during an earthquake. FRCM has been found to be equally as effective as FRP in confining plastic hinges, with ductility increases of up to 2 times being achieved. In plastic hinge confinement areas of high moment are confined, typically the base and top of the column (Bournas & Triantafillou, 2011; Bournas, Triantafillou, Zygouris, & Stavropoulos, 2009). Lap splice confinement is used if there is inadequate overlap between the longitudinal steel in the column and the bars extending from the
slab below the column (Filiatrault, Tremblay, Christopoulos, Folz, & Pettinga, 2013). The confinement acts as a clamp holding the overlapping bars together and preventing concrete spall in the overlapping section. This construction defect is typical in older structures which were not designed for seismic loading conditions (Moehle, 2014). It has been found that in shorter overlaps, FRCM is not as effective as FRP in providing confinement, potentially limiting the use of FRCM in this application (Bournas, Triantafillou, Zygouris, & Stavropoulos, 2009). Shear strengthening is used to add additional shear capacity to columns. Columns designed prior to the 1970’s often do not have adequate shear capacity to resist seismic loading conditions, often leading to catastrophic failures (Moehle, 2014). This strengthening technique involves wrapping the entire length of the column, as there will be large shear forces throughout the entire member. Initial testing has found shear capacities of FRCM wrapped columns to be comparable to FRP, as long as the FRCM is properly anchored (Tzoura & Triantafillou, 2016).

It has been found that FRCM is a suitable material for use in the shear strengthening of beams (Koutas, Bournas, & Triantafillou, 2019). Shear strengthening is important due to the sudden nature of shear failures compared to more ductile flexural failure (Brzev & Pao, 2016). FRCM is used as shear strengthening by wrapping shear critical sections of the member as shown below in Figure 2.8. The primary fabric direction must be aligned in the transverse direction. While the longitudinal fabric may theoretically provide some strength, the actual value is negligible (Vecchio & Collins, 1986). A more detailed discussion about FRCM shear strengthening is provided in section 2.5 Use of FRCM for Shear Strengthening and Repair.
2.4 Shear in Reinforced Concrete

The shear failure mode is one of the most catastrophic failure modes. It occurs rapidly with little warning and must be avoided in order to preserve public safety (Brzev & Pao, 2016). There are several different models for modeling shear in RC, the use of these models depends on the beam’s geometric conditions, namely the shear span to depth ratio (a/d). If the member has an a/d ratio higher than 2.5 the load will be transferred via beam action, as shown in Figure 2.9 a) (Fenwick & Pauley, 1968). In beam action, load transfer is modeled as a truss with a portion of the concrete acting as compression members and depending on the presence of reinforcement, either the steel or the rest of the concrete acting as the tension members (Fenwick & Pauley, 1968). Due to the low tensile strength of concrete, if there is no reinforcement, the member will fail in diagonal tension once the tensile concrete struts reach their maximum load. If reinforcement is present failure typically occurs when the concrete above the stirrups crushes (Brzev & Pao, 2016). If the a/d ratio is lower than 2.5, arch action is responsible for the load transfer, as shown in Figure 2.9 b) (Kim, Kim, & White, 1999). In arch action, the load is transferred directly from the point of
application to the support along a concrete compressive strut. In this case, shear compression failure as described above is common, however, shear cracking along longitudinal steel also occurs (Fenwick & Pauley, 1968). If the a/d ratio is very low <1, the failure modes are typically pull out of longitudinal steel if adequate development length is not present. If the longitudinal steel is adequately developed, the failure mode will be the crushing of the concrete in the web along the arch (Brzev & Pao, 2016).

![Shear transfer mechanisms](image)

Figure 2.9: Shear transfer mechanisms

### 2.4.1 Shear Design Models

There are many different models which have been developed to describe the shear behavior of RC beams. All these models are based on the lower bounds plasticity theory (Brzev & Pao, 2016). This theory states that any stress distribution is valid, as long as the internal and external loads are in equilibrium and none of the material has yielded (Gurley, 2011). This means that the models are suitable for designing reinforced concrete even though the true stress distribution is not known. Some of the most popular models are: the truss analogy which forms the basis of the American Concrete Institute (ACI) RC shear design code, strut and tie design method, and the modified
compression field theory which is used in the CSA code (ACI 318-14, 2014; CSA A23.3-14, 2014).

The truss analogy has been used since the early 1900’s and continue to be used today (fib Task Group 4.2, 2010). In a truss analogy model, the concrete beam is assumed to behave as a truss, with the concrete acting as the compression members and the steel acting as the tension members, with nodal points where the two meet (Brzev & Pao, 2016). The assumed angle of the diagonal tension crack has a major effect on the truss, as the angle of the crack will describe the angle of the concrete compressive strut (Garay-Moran & Lubell, 2008). The truss analogy used in the ACI design code assumes shear cracks will form at 45°, therefore, the concrete struts are modeled at 45°. The material behavior of the concrete struts was developed using empirical methods, hence is often overly conservative and can only be applied in beam action (ACI 318-14, 2014).

The strut and tie model is a more robust truss analogy which is suitable for use in both beam and arch action cases (Garay-Moran & Lubell, 2008). In the strut and tie model the angle of the struts is left to the designer’s discretion (CSA A23.3-14, 2014). This makes the strut and tie model useful for a wider range of cases, however, it is reliant on the designer’s skill to position the struts. Improperly positioned struts can result in either inefficient or unrealistic designs, which will not behave as anticipated and potentially fail (Brzev & Pao, 2016).

The compression field theory was originally developed in the 1980’s, based on the tension field theory used in steel I-beams (Vecchio & Collins, 1986). In the tension field theory (Figure 2.10a), after the web buckles, the unbuckled sections carry the tension forces and the web stiffeners carry the compression forces (Natesh, Sarma, & Baskar, 2018). The same general idea is applied to concrete (Figure 2.10b), however, once the concrete cracks in tension, uncracked sections carry
the compression forces and the reinforcing steel, both the stirrups and the longitudinal bars, carry the tension forces (Vecchio & Collins, 1986).

![Diagram of tension and compression field theories]

Figure 2.10: Tension and compression field theories

The compression field theory has several drawbacks, it does not assume the cracked concrete has any tensile strength, this is excessively conservative. As well, it does not allow for concrete members without stirrups, such as slabs, to be modeled (Vecchio & Collins, 1986). To account for these drawbacks, the modified compression field theory was developed. In the modified compression field theory average stresses and strains of the section are used in order to predict the behavior of the member (Vecchio & Collins, 1986). As well, cracked concrete is treated as its own new material. The cracked concrete has new material properties including shear strength based on friction between the aggregate along the shear cracks, called aggregate interlock. The modified compression field offers a more rational approach to shear design when compared to the 45° truss.
analogy. However, similar to the 45° truss analogy it only effectively models beam action (Vecchio & Collins, 1986; CSA A23.3-14, 2014).

2.4.2 Causes of Shear Inadequacy

As previously discussed, the shear failure mode occurs rapidly and has disastrous consequences for the structure and must be avoided. There are several ways a beam could become deficient in shear and require strengthening these include construction or design errors, stirrup corrosion, external impact, and flexural strengthening. Older structures which are not designed to meet modern code may not have adequate reserve strength, even if they have not yet collapsed (Hobbs, 2001). Additionally, new structures may not be up to code due to errors in the design of the members or deficiencies during construction (Hobbs, 2001). In either case, the member will need to be strengthened to bring it in line with code.

In older structures or structures built using an improperly designed concrete mix, corrosion of the stirrups is likely. If the stirrups are corroded, as shown in Figure 2.11, some of or even all of their load carrying capacity may be lost, in turn diminishing the shear capacity of the entire member (Hobbs, 2001).

If the structure is externally impacted, such as being struck by a vehicle or exposed to seismic loading, the concrete matrix may be damaged. If this occurs the load bearing capacity of the concrete may be entirely lost. In this case, the steel will have to resist all the shear force.

A final reason shear strengthening may be necessary is flexural strengthening. If a member is found to be deficient in flexure, it is necessary to repair it. In some cases, however, if the repair increases the capacity of the member far beyond its original capacity, the expected failure mode
may shift from flexure to shear. If this occurs, it will now be necessary to perform additional shear strengthening on the member to return the expected failure mode to flexure. For this reason, it is important to always check both the shear and flexural capacities of strengthened members (GangaRao, Taly, & Vijay, 2006; Bank, 2006; Teng, Chen, Smith, & Lam, 2001).

![Figure 2.11: Beam with corroded stirrups](image)

2.4.3 Shear Strengthening and Repair

There are several different methods which can be used to strengthen members in shear. The method which will be used depends on member geometric properties, budget, and site conditions. The most common repair and strengthening methods are described below.

Additional transverse reinforcement similar to stirrups can be added to a member to increase its shear capacity. These bars can either be adhered into surficial grooves or placed internally by drilling holes into the member. These bars provide passive reinforcement to the system, meaning that if a substantial shear crack has already formed and the concrete contribution to shear resistance
is lost, it will not be restored. Additionally, this repair method can be quite labour intensive driving up costs.

Prestressed steel can also be used to improve shear capacity. The steel can be either internally mounted by drilling a hole through the section or externally mounted onto the outside of the member. Both types of reinforcement would be post tensioned and would provide active reinforcement. This method can be quite expensive due to the equipment required to install the prestressed steel. Additionally, the externally applied method is highly susceptible to corrosion.

**2.5 Use of FRCM for Shear Strengthening and Repair**

To strengthen a member in shear, FRCM is applied in the shear critical zones of the beam. In this case, the fiber direction is transverse to the longitudinal axis of the beam. It can be applied on the sides of the concrete cross-section, as a U-wrap, or a complete wrap around the cross-section. The shear critical zone can be wrapped in strips or as a complete wrap. The wrapping scheme depends on the beam’s geometric properties (full wrap is often not possible) and the cost of materials vs labour (complete longitudinal wrap uses more material but is often easier to apply).

**2.5.1 General Behavior and Failure Modes**

In the past decade, there have been many tests to investigate the shear performance of FRCM as shear strengthening. Most of the tests have used rectangular specimens, however, there have been several tests on T-beams (Koutas, Bournas, & Triantafillou, 2019). Most of these beams have been small or medium scale. As well, the vast majority have had a high a/d ratio (slender beams). These beams have primarily been tested in 3- or 4-point monotonic loading. Additionally, several cyclic loading tests having been performed (Tzoura & Triantafillou, 2016; Awani, El-Maaddawy, & El Refai, 2016). The generalized behavior observed in these tests can be seen in Figure 2.12.
The material is the stiffest before the concrete cracks, slightly stiffer than an unstrengthened beam due to the strengthening material. Following cracking, the stiffness is slightly decreased. Once the ultimate load is reached, there are two failure paths which are dependent on the failure mode. The first being a sudden loss of strength, associated with debonding or rupture. The second being a smoother pseudo-ductile response, observed in slippage of the fabric. Following the loss of effectiveness of the strengthening material, the retrofitted stress-strain curve will follow that of the unretrofitted. The failure modes observed in shear specimens are slippage or local failure, debonding, delamination, fabric rupture, diagonal tension, compression shear, and flexure (Koutas, Bournas, & Triantafillou, 2019).

![Diagram](image)

Figure 2.12: RC shear strengthening load vs deflection [(Koutas, Bournas, & Triantafillou, 2019) with permission through Creative Commons license]

In the slippage failure mode, the fabric and mortar in the FRCM composite debond from one another causing failure. While this failure mode exhibits some ductile behavior, it is non-linear
and difficult to predict (Koutas, Bournas, & Triantafillou, 2019). When only one layer of fabric is used, this failure mode has been observed (Tetta, Koutas, & Bournas, 2018). In the debonding failure mode, the mortar in the FRCM composite detaches from the concrete substrate. This failure mode is non-ductile and occurs with very little warning. The debonding failure mode has been associated with FRCM wrapping schemes with more than one layer of wrapping (Tetta, Koutas, & Bournas, 2018). The delamination failure mode occurs when the concrete substrate beneath the FRCM wrap is peeled off the rest of the member. This failure mode is non-ductile and occurs with little warning (Koutas, Bournas, & Triantafillou, 2019). The delamination and debonding failure modes have often been observed together in specimens with multiple layers of fabric applied (Tetta, Koutas, & Bournas, 2018). In the fabric rupture failure mode, the fabric material is fully engaged to its maximum capacity and fails. While this failure mode is non-ductile, it is the preferred failure mode from an efficiency point of view (Triantafillou, 2016). In the other failure modes, the fabric is not fully engaged, and, therefore, some material is wasted. This failure mode has been observed in specimens with adequate layers of fabric and suitable bond between all materials (Tetta, Koutas, & Bournas, 2018). Diagonal tension and compressive shear failure modes are associated with shear failure in RC. Shear failure modes are observed in unison with an FRCM failure mode as both the FRCM wrap and RC member fail together. In test specimens with large amounts of shear reinforcement, changing the failure mode from shear to flexure. In the flexural failure mode, the longitudinal steel reinforcement is loaded to its yield point. The steel yielding causes significant deformation to occur. Eventually, the specimen will fail when the concrete compression region reaches its full capacity, and crushes.
2.5.2 Current Studies

Many different studies encompassing a wide array of factors encompassing material, application, and loading parameters have been undertaken. There has been a significant amount of testing comparing the performance of different types of fabric. Blanksvärd (2009) found that by using fabric grids with smaller spacing between the bundles of fabric, higher cracking and ultimate strengths could be achieved. Additionally, Tetta (2016) found more evenly distributed fabric patterns (smaller bundles with smaller inter-bundle spacing) performed more effectively, due to improved force distribution. It can generally be observed that stronger fabric types exhibit higher strength gain and smaller crack widths (Azam & Soudki, 2014; Younis, Shrestha, & Ebead, 2017; Tetta, Koutas, & Bournas, 2018). However, failure in FRCM is often governed by the bond behavior between the fabric and the mortar. Due to this failure mode, weaker types of fabric, with a lower Young’s modulus, are often able to achieve improved material utilization (mechanical properties vs ultimate load) (Escrig, Gil, Bernat-Maso, & Puigvert, 2015).

Several studies regarding stirrup presence and spacing have been undertaken. These studies indicated that FRCM is effective when internal transverse reinforcement is present (Awani, El-Maaddawy, & El Refai, 2016). While the testing has shown that stirrups seem to reduce the effect of FRCM strengthening, this is somewhat deceptive. All the specimens failed due to flexure, shear compression in the concrete, or debonding of the mortar from the concrete substrate. This means that the nature of the failure mode has prevented the FRCM wrap from being fully utilized. Strain analysis by Blanksvärd et. al. (2009) prior to failure indicated that there is no interaction between the number of stirrups and the FRCM strengthening. Alternatively, the findings of Ombres (2015), as well as, Aljazaeri and Myers (2017) indicated that there is interaction between the internal stirrups and the FRCM.
The number of layers of fabric is the most studied factor affecting FRCM shear strengthening. It has been found that increasing the number of layers will increase the shear capacity of a member in a diminishing, nonlinear fashion (Triantafillou & Papanicolaou, 2006; Blanksvärd, Täljsten, & Carolin, 2009; Al-Salloum, Elsanadedy, Alsayed, & Iqbal, 2012; Jung, Hong, Han, Park, & Kim, 2015; Ombres, 2015; Aljazaeri & Myers, 2017; Tetta, Koutas, & Bournas, 2018). The number of layers has a significant effect on the failure mode of the specimen. Many studies have reported that when only one or two layers are used, fabric slippage is common, due to the poor bond in the FRCM components (Loreto, Babaeidarabad, Leardini, & Nanni, 2015; Awani, El-Maaddawy, & El Refai, 2016; Tzoura & Triantafillou, 2016). By using more layers, the failure can be shifted into the concrete (delamination) or concrete-mortar interface (Loreto, Babaiedarabad, Leardini, & Nanni, 2015; Awani, El-Maaddawy, & El Refai, 2016; Tzoura & Triantafillou, 2016). Tetta et. al. (2015) observed that even with two layers, it was possible to shift the failure mode to delamination. When four or more layers are present, the failure mode is shifted to debonding between the concrete and mortar, due to incompatible stiffness between the two materials (Brückner, Ortlepp, & Curbach, 2008).

The angle of application is a contested factor, with some research having found that spiral wrapping vs transverse had no effect (Triantafillou & Papanicolaou, 2006). Trapko et. al. (2015) and Younis et. al. (2017) found that transverse 90° wrapping was more effective than 45° angle wrapping. Whereas, Al-Salloum et. al. (2012) found that angled (45° or 60°) wrapping had a positive effect on the strength gain, especially with a higher number of layers.

The use of anchors to increase the load bearing capacity is not entirely agreed upon. In several studies, steel anchorage systems were found to prevent debonding failure (Brückner, Ortlepp, &
Curbach, 2008; Trapko, Urbańska, & Kamiński, 2015; Tzoura & Triantafillou, 2016). However, Younis et al. (2017) found that steel anchorage did not affect the FRCM performance. FRP fan anchors were tested by Baggio et al. (2014), however, the effect of the anchors could not be determined, as the glass fabric was too weak and ruptured regardless of the anchor presence. Tetta et al. (2016) also tested FRP fan anchors, similar to Baggio et al. (2014), glass fabric specimens ruptured, regardless of the presence of anchorage. However, when using carbon fabric, a substantial improvement (up to 150%) in the load bearing capacity was observed. Marcinczak et al. (2019) tested various FRP based anchors. The research showed that fabric slippage could be prevented using anchors. It was also found that all anchorage styles were effective.

Concrete compressive strength is another contentious topic of study. Research by Blanksvärd et al. (2009) suggested that in high strength concrete, the effect of FRCM is diminished. Alternatively, several other studies have found that FRCM strengthening effectiveness improved in high strength concrete and was diminished by lower strength concretes (Contamine, Si Larbi, & Hamelin, 2013; Loreto, Babaeidarabad, Leardini, & Nanni, 2015).

In most studies, the type and the thickness of mortar was not examined, as it is often recommended by the manufacturer of the fabric grid. It has been found that mortars with better mechanical properties typically create a better bond. Therefore, stronger mortars provide more efficient strengthening than weaker mortars (Blanksvärd, Täljsten, & Carolin, 2009; Si Larbi, Contamine, Ferrier, & Hamelin, 2010; Al-Salloum, Elsanadedy, Alsayed, & Iqbal, 2012). Blanksvärd et al. did use two different thicknesses of mortar in one study, however, the different thicknesses corresponded to different types of mortar as specified by the mortar manufacturer.
(Blanksvärd, Täljsten, & Carolin, 2009). For this reason, no conclusions regarding mortar thickness can be drawn.

The strengthening pattern of FRCM is an area of great interest. If it is well understood, FRCM repair methods can be optimized for efficiency and cost, increasing the composite’s use. The three wrapping schemes for FRCM sections are side wrap, U-wrap and fully wrapped as seen in Figure 2.13. There has been research into all of these wrapping schemes yielding varied results. As expected, based on the knowledge of FRP wrapping schemes (Teng, Chen, Smith, & Lam, 2001; Bank, 2006; GangaRao, Taly, & Vijay, 2006), Tetta et al. (2015 & 2016) and Jung et al. (2015) found the U-wraps were more effective than side wraps. Azam and Soudki (2014) found that side wraps and U-wraps provided the same level of strength gain. Tetta et al. (2015) also tested fully wrapped specimens, which outperformed both the U-wrapped and side bonded sections. The effectiveness of the fully wrapped specimen was expected based on the FRP performance (Teng, Chen, Smith, & Lam, 2001; Bank, 2006; GangaRao, Taly, & Vijay, 2006).

![Figure 2.13: Various section wrapping styles for FRCM](image)

Figure 2.13: Various section wrapping styles for FRCM
Several different longitudinal wrapping schemes, continuous wrapping and intermittent strips are possible. Contamine et. al. (2013) found that similar strength could be achieved with continuous wrapping and strips. It has been found that as the spacing between strips increases, strengthening effectiveness decreases. If the spacing between strips is too great, shear cracks can form between the strips, causing the strips to be uneffective (Jung, Hong, Han, Park, & Kim, 2015; Ombres, 2015; Aljazaeri & Myers, 2017). Younis (2017) found that strips resulted in larger crack widths than continuous wrapping. Tetta (2018) compared continuously wrapped beams with a hybrid of continuously wrapped and strips. It was found that the hybrid system was more effective than the continuous wrap. Si Larbi et al. (2010) tested a continuous glass fabric U-wrap, and an intermittent carbon fabric U-wrap. The glass fabric exhibited a 69% load bearing increase and the carbon fabric showed a 22% load bearing increase, compared with the control.

Most of the research thus far has been on beams tested under monotonic loading, however several studies have assessed beams under cyclic loading (Triantafillou & Papanicolaou, 2006; Tzoura & Triantafillou, 2016). Al-Salloum et al. (2011) tested beam-column connections under cyclic loading. These tests showed that FRCM can be an effective shear strengthening material in seismic conditions (Triantafillou & Papanicolaou, 2006; Tzoura & Triantafillou, 2016; Al-Salloum, Siddiqui, Elsanadedy, Abadel, & Aqel, 2011).

There has been little research on the topic of shear span to depth (a/d) ratio. Tetta (2018) has found that a/d ratio has no effect on either failure mode or strengthening performance. Existing knowledge of concrete shear mechanics suggests that this should not be the case (Brzev & Pao, 2016), therefore, more research should be done.
Strengthening of beams with load applied has been investigated. In a study by Blikharskyy et al. (2017), it was found that the repair of loaded beams is less effective than the repair of unloaded beams. This is because the steel and concrete in the beam is already partially stressed. Meaning that the FRCM is not able to fully develop its strength before the underlying beam fails. These findings are in line with understood structural engineering principles (CSA S806-12, 2012). This study highlights the importance of unloading a structure before any repair work is done.

There have been several investigations into the comparison between FRP and FRCM strengthening systems, with the results being consistent throughout. The general trend of the findings is that if the failure mode in the FRCM is slippage of the fabric, the FRP will outperform the FRCM (Triantafillou & Papanicolaou, 2006; Tetta, Koutas, & Bournas, 2015; Tzoura & Triantafillou, 2016). However, if the failure mode in both systems was debonding between the matrix and the concrete cover, the results are quite comparable. These results include both monotonic and cyclic loading conditions (Al-Salloum, Siddiqui, Elsanadedy, Abadel, & Aqel, 2011; Tetta, Koutas, & Bournas, 2016).

Tetta and Bournas (2016) compared the performance of FRP and FRCM at high temperatures and found that the FRCM performed consistently at all temperatures, however, the effectiveness of the FRP was lost at temperatures over 100°C. This comparison reinforces existing knowledge of FRP’s heat related limitations (Bank, 2006; GangaRao, Taly, & Vijay, 2006) and shows one area where FRCM is a promising alternative.

2.6 Shear Strengthening using NSM FRP

The use of NSM FRP to improve the shear capacity of RC beams has shown promising results. Strength increases as high as 250%, compared with unstrengthened beams have been recorded
(Lim, 2010; Tanarslan, 2011). Additionally, it has been noted that the application of NSM FRP can also increase mid-span deflection (Samad, et al., 2017; Dias & Barros, 2017). Many factors which impact the effectiveness of NSM FRP shear strengthening have been studied. These factors include presence of steel stirrups, NSM bar spacing, externally bonded FRP vs NSM bars, concrete strength, angle of application, effects of damage, anchorage, shear span/depth ratio, and prestressing.

Many authors have reported on the effects steel stirrups on NSM FRP shear strengthening systems. The general consensus is that NSM systems are most effective when there are no stirrups, and that the efficiency of the system decreases as the stirrup presence increases. In extreme cases, there may be no strengthening effect from the NSM system if adequate stirrups are present. However, reduced crack width and propagation were observed in the strengthened beams (De Lorenzis & Nanni, 2001; Islam, 2009; Dias & Barros, 2012a; Kuntal, Chellapandian, & Prakash, 2017). Alternatively, Mofidi et al. (2016) have observed that the presence of stirrups does not diminish the contribution of NSM FRP.

Singh et al. (2014) reported that less space between the NSM FRP resulted in increased strengthening performance. However, the bars should not be spaced more closely than the steel stirrups, as this was reported to diminish the strengthening effectiveness. Dias and Barros (2008) also found that a spacing between bars which is too small can limit the effectiveness of the strengthening system. It was found that once the spacing between bars was too small, the concrete cover and NSM FRP system would detach from the underlying beam core during testing.

Rizzo and De Lorenzis performed a comparison of externally bonded FRP with NSM FRP bars. The NSM FRP system was found to be more effective (22%-44% strength increase vs 16%
for externally bonded), due to the improved bond performance of the NSM system. These conclusions are supported by the findings of Diaz and Barros (2012a) who also compared externally bonded FRP with NSM FRP. It was observed that externally bonded FRP only achieved between 34-59% of the strengthen increase associated with NSM system. Morsey et al. (2012) compared the shear strengthening effectiveness of externally bonded systems, NSM systems, and embedded FRP systems. In this study, the embedded FRP rods were mounted into holes drilled vertically through the center of the beams. It was found that both the NSM and embedded systems had twice the strengthening effect of the externally bonded system (Morsy, El-Ashkar, & Helmi, 2012). Alternatively, Kim et al. (2015) have reported that externally bonded systems were more efficient than NSM systems, if debonding was prevented. However, the energy dissipation properties of the NSM system were superior regardless of the failure mode.

Several authors have studied the impact of concrete compressive strength on NSM FRP strengthening. Dias and Barros (2010) observed that as concrete strength decreases, strengthening effectiveness also decreases. Dias and Barros (2012b) continued to research the effect of concrete compressive strength on NSM FRP strengthening effectiveness. It was found that as concrete strength increased, the effectiveness of NSM FRP strengthening also increased, confirming their earlier findings.

Most authors have reported that when the angle of inclination of the NSM FRP bars is decreased (changing the angle from 90° to 45°), the strengthening effect is increased by up to 5x (Dias & Barros, 2008; Dias & Barros, 2010; Rahal & Rumiah, 2011; Jalali, Sharbatdar, Chen, & Alaee, 2012; Dias & Barros, 2012a; Dias & Barros, 2012b; Singh, Reddy, & Khatri, 2014; Dias &
Barros, 2017; Kuntal, Chellapandian, & Prakash, 2017). Conversely, Rizzo and De Lorenzis (2009) found that all application angles were equally effective.

Dias and Barros (2012b) studied the effects of damage on the effectiveness of NSM FRP shear strengthening systems. The specimens were preloaded to 3mm of deflection, corresponding to the maximum deflection allowed by the Eurocode. After repair, the specimens were loaded to failure. It was observed that the pre-damage had no effect on the ultimate load bearing capacity of the beams. However, it was reported that the precracked beams exhibited a lower stiffness than the undamaged specimens. Almassri et al. (2015) investigated the repair of RC beams damaged by corrosion. The damaged beams which were not strengthened and those only strengthened in flexure failed due to shear induced diagonal tension. Beams which were strengthened in flexure and shear exhibited the highest strengthening performance, approximately 120% of the strength of the control beam.

Jalali et al. (2012) tested the effect of anchorage on the performance of NSM FRP systems. It was found that by utilizing anchorage, the performance of the NSM FRP system was improved by 60%. Additionally, the use of the anchorage increased the beams ductility by 40-75% compared to the unanchored system.

In most studies of NSM FRP systems, two-part epoxy has been used as the bonding agent. In several studies the use of two-part epoxy was compared with a cement-based grout. The bond behavior of the two bonding materials has been compared by Al-Mahmoud et al. (2011) and Nordin and Taljsten (2003). In both studies, it was found that the bond capacity of the epoxy mounted bars was twice that of the mortar mounted bars. Taljsten et al. (2003) and Al-Mahmoud et al. (2009) compared mortar and epoxy mounted systems for the flexural strengthening of RC
beams. It was found that the mortar adhered systems were effective in improving capacity, with a strength increase of approximately 50%, compared with the control beam. However, the strength gain associated with mortar mounted bars was only 80% that of the epoxy mounted system. Al-Mahmoud et al. (2015) compared the shear strengthening performance of epoxy and mortar adhesives. Both systems were shown to be effective with the epoxy-based system increasing the strength by 44% and the mortar counterpart by 35% compared to the control. These studies represent a promising start to the use of mortar as an adhesive material, however, more studies are still needed.

In many countries the use of epoxy or polymer resin adhered NSM FRP systems has been incorporated into the design code. Countries which have included these systems in their design codes or guidelines include Australia, Canada, Egypt, Italy, Switzerland, England, Japan, and the United States (JSCE Concrete Engineering Series 23, 1997; SIA166, 2004; ECP 208-2005, 2005; CNR-DT 203/2006, 2006; CSA S806-12, 2012; TR 55 3rd Edition, 2012; CNR-DT 200 R1/2013, 2013; CSA S6-14, 2014; ACI 440.1R-15, 2015; AS 5100.8:2017, 2017). Many of these codes have strict limitations on the maximum load the NSM system can be expected to take. In some cases, as low as 30% of the ultimate tensile strength of the FRP.

2.7 Research Needs

The aesthetic unobtrusiveness of FRCM and NSM systems are a positive aspect of each system. This means that there will likely be cases where practicing engineers need to choose between the two systems. However, to the author’s knowledge there are no direct comparisons of the two. For both materials, there is also a demand for the repair of existing structures, which have been damaged by overloading. If FRCM or NSM bars are to be fully utilized, the damage repair
capabilities of each system must also be understood. As the shear failure mode is critical and must be avoided at all costs, it is vital that an effective repair method is developed. By doing so, shear damaged structures can be brought back into compliance, so they can be safely used. There has been very little research into the performance of each material for the strengthening of previously damaged members. Additionally, there is very little research into the comparison of epoxy and mortar mounted NSM systems.

The existing literature shows that FRCM is highly promising as a strengthening material for RC structures. Most of the research which has been done into the use of FRCM has been done in European countries such as Italy, Spain, and Greece. The USA has also contributed to many of these research efforts as a joint contributor. In China, India, UAE, Qatar, and other European countries several papers have also been published. The research which has been done in Canada is still highly limited. If FRCM is to be codified in Canada, there will need to be local experimental efforts using Canadian materials and practices. Without the development of codes, FRCM will not be accepted as a design material and will remain unused.

2.8 Research Objectives and Scope

The goals of the current research initiative are to: 1) Investigate the shear strengthening effectiveness of various composite systems on damaged reinforced concrete beams. 2) Propose design recommendations for each system based on the findings of this study.

The following gaps in the current literature were selected to be explored:

- Compare the continuously wrapped and intermittent strips of FRCM wrapping schemes on strengthening performance
Effect of stirrup spacing on FRCM wrap effectiveness

Compare FRCM wrapping systems to NSM bar systems

Compare the efficiency of mortar mounted and epoxy mounted NSM bar systems

Examine the effects of various damage levels on FRCM wrap and NSM bar strengthening system efficiency

The performance indicators used include peak load, energy absorption, failure mode, shear crack formation load, and beam stiffness. Peak load and energy absorption were chosen as performance indicators as they are a direct measure of the strengthening systems effectiveness. These parameters would be required for a practicing engineer who sought to apply the strengthening system, in order to prevent shear failure from occurring. Failure mode was chosen as a performance indicator as it often has a strong correlation with both the peak load and the energy absorption characteristics of the beam. Shear crack formation load was chosen as a performance indicator to assess whether the strengthening systems were able to delay the reopening of the shear crack in the damaged specimens. Previous researchers had noted shear damage to beams caused a loss in stiffness (Dias & Barros, 2012b). Therefore, stiffness was chosen as a performance indicator to measure the effect of damage on the beams.
CHAPTER 3: EXPERIMENTAL METHODOLOGY

This chapter describes the testing program, methods, and materials used to examine the effects of various shear strengthening and repair methods on damaged RC beams.

3.1 Test Matrix

A total of 15 beams (2 controls, 6 strengthened undamaged, 7 damaged and repaired) were cast and tested in order to examine the performance of RC beams with the following parameters: FRCM application scheme (continuous vs strips), adhesive used for NSM FRP bars (epoxy vs mortar), stirrup spacing (150mm vs 200mm), and level of damage (no damage, first shear crack, and 70% of maximum theoretical load). The maximum theoretical load was calculated using the combined strut and tie method used by Garay-Moran and Lubel (2008). Table 3.1 shows the 15 beams, identified by a beam ID with the format of SX-Y-DZ. Where S is for stirrups, with X representing the stirrup spacing (X=200 meaning a 200mm spacing), Y representing the strengthening scheme (“WN” meaning no wrapping, “WI” meaning intermittent FRCM strips, “WF” meaning continuous FRCM wrapping, “RM” meaning NSM bars mounted with mortar, and “RE” meaning NSM bars mounted with epoxy), and D for damage, with Z referring to the level of damage (“DN” meaning no damage, “D1” meaning first shear crack, and “D2” meaning 70% of the maximum theoretical load).
### Table 3.1: Test matrix

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen ID</th>
<th>Stirrups</th>
<th>Strengthening Scheme</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>S200-WN-DN</td>
<td>200</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>S150-WN-DN</td>
<td>150</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>NSM-E</td>
<td>S150-RE-DN</td>
<td>150</td>
<td>NSM Bars with Epoxy</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D1</td>
<td>150</td>
<td>NSM Bars with Epoxy</td>
<td>1st Shear Crack</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D2</td>
<td>150</td>
<td>NSM Bars with Epoxy</td>
<td>70% of Predicted Load</td>
</tr>
<tr>
<td>NSM-M</td>
<td>S150-RM-DN</td>
<td>150</td>
<td>NSM Bars with Mortar</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D1</td>
<td>150</td>
<td>NSM Bars with Mortar</td>
<td>1st Shear Crack</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D2</td>
<td>150</td>
<td>NSM Bars with Mortar</td>
<td>70% of Predicted Load</td>
</tr>
<tr>
<td>FRCM-C</td>
<td>S150-WF-DN</td>
<td>150</td>
<td>FRCM Continuous</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>S150-WF-D1</td>
<td>150</td>
<td>FRCM Continuous</td>
<td>1st Shear Crack</td>
</tr>
<tr>
<td></td>
<td>S150-WF-D2</td>
<td>150</td>
<td>FRCM Continuous</td>
<td>70% of Predicted Load</td>
</tr>
<tr>
<td></td>
<td>S200-WF-DN</td>
<td>200</td>
<td>FRCM Continuous</td>
<td>None</td>
</tr>
<tr>
<td>FRCM-I</td>
<td>S150-WI-DN</td>
<td>150</td>
<td>FRCM Strips</td>
<td>1st Shear Crack</td>
</tr>
<tr>
<td></td>
<td>S150-WI-D1</td>
<td>150</td>
<td>FRCM Strips</td>
<td>70% of Predicted Load</td>
</tr>
<tr>
<td></td>
<td>S200-WI-DN</td>
<td>200</td>
<td>FRCM Strips</td>
<td>None</td>
</tr>
</tbody>
</table>

#### 3.2 Test Specimen and Strengthening Systems

The test specimens were designed to simulate half-scale beams in an existing structure, which have been damaged due to overloading and require repair. Half-scale specimens were chosen due to the available forms and test frame. A beam cross section of 200mm by 265mm with a length of 2000mm was chosen, as shown in Figure 3.1. The beams were tested in 3-point bending with 1500mm between the supports as shown in Figure 3.1. The point load was placed at a distance of
500mm from the support on the shear critical end in order to achieve an a/d ratio of 2.3. The point load also created a constant shear force in this region.

The longitudinal reinforcement consisted of 2-25M bars, bent with hooks at either end, as shown in Figure 3.1. The 25M bars were chosen in order to ensure substantial flexural resistance. On the side faces, the concrete cover was 19mm and on the top and bottom the cover was 25mm. With this cover, the depth of the longitudinal bars was taken as 218.5mm. Two smooth rods with a diameter of 6.35mm (¼”) were used in the compression side of the beam in order to hold the stirrups upright. These rods were not accounted for in strength calculation.

Steel stirrups bent from 6.35mm (¼”) rods were used for internal shear reinforcement. The stirrups were placed at a spacing of 100mm in the strong end of the beam to ensure adequate shear resistance (Figure 3.1). In the shear critical zone, they were placed at 150mm or 200mm (Figure 3.2). This spacing was above the CSA A23.3-14 recommended maximum value of 138mm, to ensure shear deficiency.

Figure 3.1: longitudinal and cross-sectional details of the test setup
The strengthening patterns used are shown in Figure 3.3. Predictions of the strengthening effects of these systems were made using CSA S806-12. Sample calculations of these predictions can be found in APPENDIX A. The FRCM wrapping scheme employed a U-shaped wrap, with the wraps extending to the top of the beam. For the FRCM-C group the entire shear critical region was wrapped. Additionally, 40mm of the non-critical section was wrapped to prevent stress concentration under the point load (Figure 3.3). For the FRCM-I group, fabric strips with a width of 75mm were used. The spacing between these strips was 75mm. These dimensions resulted in a 150mm center to center spacing. This spacing was chosen so that the FRCM strips would not be in the same plane as the steel stirrups (Figure 3.3). The first strip was placed 112.5mm away from the face of the support. For the NSM group grooves were cut into the side faces of the beam, perpendicular to the longitudinal axis. The grooves were made at a 150mm center-center spacing, with the first groove being made 150mm from the support. This spacing was chosen to prevent overlap with the steel stirrups. The grooves’ dimensions were 15mm by 15mm (width and depth).
3.3 Specimen Preparation

3.3.1 Beam Casting

The 6mm stirrups were intended to be bent to the standards of CSA A23.3-14. However, difficulties in the bending process resulted in an interior bend radius of 20mm rather than 24mm. This slight deviation was considered to be acceptable, as failure of the stirrups was not anticipated. The hooks in the 25M bars were intended to comply with CSA A23.3-14. However, due to limitations of the supplier’s rebar bending machine, the hook diameter was 190mm instead of 200mm. This small deviation was acceptable as the beams were not anticipated to fail in flexure, so pullout was unlikely. Once the stirrups and longitudinal bars were bent, the rebar cage was then assembled, as shown in Figure 3.4.
After the cages were constructed, the formwork was then prepared. Cove molding, with a diameter of approximately 19mm, was placed into the corners of the base of the form. These molds created a curved surface, to prevent stress concentrations in the FRCM. The rebar cages were placed into the formwork on 25mm plastic spacers, which were used to achieve the desired cover. The cages were also hung from 2x4 wooden boards, which rested on the top of the forms, using rebar wire, to prevent shifting during concrete placement. Lifting hooks were placed into the cages to allow for the removal and transportation of the beams. Handles were tied onto the lifting hooks using rebar wire, so they could be partially pulled out of the concrete, after it was placed. The completed formwork before the pour is shown in Figure 3.5 a).

Once the forms were prepared, the concrete was poured. Ready-mix concrete from a local plant was delivered via mixing truck. Before pouring the specimens, the slump of the concrete was determined to be 125mm and the air content was found to be 3.0%, which was acceptable. The concrete was poured into the forms using a line pump and compacted using handheld vibrators. Once the concrete was placed, the wooden hangers suspending the cages were removed. The concrete was then floated, and the excess concrete was removed. Following the removal of the excess concrete, the lifting hooks were exposed, and the concrete surface was finished until smooth.
Twelve standard cylinders (100mm x 200mm) were cast in order to verify the concrete’s compressive strength as per CSA A23.2 (CSA A23.1-14/A23.2-14, 2014). In order to cure the beams, they were covered with damp burlap and plastic sheets to keep them moist, while in the forms. After 24 hours, the beams were removed from the moulds, as shown in Figure 3.5 c), fully wrapped with wet burlap, covered with plastic sheets, and allowed to cure for an additional 7 days. After this, the beams continued to cure in laboratory air until testing.

3.3.2 Surface Preparation and Groove Cutting

The damaged specimens had the surface roughening associated with FRCM application completed before the damage was applied. Alternatively, the grooves for the NSM were not installed until after the damage, to prevent damage to the grooves during the loading process.

For the beams in group FRCM-I and FRCM-C, after 40 days the beams were moved outside for the surface roughing procedure. To achieve the desired concrete surface profile (CSP)
roughness level of 5-6, a shot blasting machine was used. The model of shot blaster was a Blastrac 1-8DEC230V and the abrasive size was S-280. The beams were periodically rotated during the roughening procedure, so that both sides and the bottom of the shear critical region were roughened. While the cove molding installed in the beam bottom corners provided round edges, there were still small ledges which could potentially introduce stress concentrations into the FRCM wrap. Therefore, after the surface was roughened, the edges of the beam were rounded using a diamond face concrete grinding bit. Once the edges of the beam were adequately rounded, the surface preparation process was complete. Following the surface preparation procedure, the specimens were returned to the lab.

For beams in group NSM-E and NSM-M, 15mmx15mm grooves were first made into the specimens. The grooves were cut perpendicular to the longitudinal axis of the beam. The outside edges of the grooves were cut, using a diamond blade concrete cutting grinder bit. Once the outside grooves were cut, a hammer and chisel were used to remove the excess material from the groove. Once the groove was made, it was scrubbed with a wire brush and blown clean with compressed air, to remove any loose material. The process of cutting the grooves is shown below in Figure 3.6.
3.3.3 Strengthening Application

After the surface preparation, the undamaged specimens were then strengthened. The FRCM was applied using the hand layup procedure, following standard practices outlined in the literature (ACI 549.4R, 2013). For ease of application, the beams were placed upside down, on either sawhorses or pallets, before strengthening. Once the beams were positioned, they were scrubbed with steel wire brushes and blown clean with compressed air. This was done to remove any debris or dust which may have gathered on the surface. Research has shown that the saturated surface dry (SSD) condition is optimal for a good bond between the mortar and concrete substrate (ACI 549.4R, 2013). To achieve this condition, the shear critical area of the beam was wrapped with wet burlap 24 hours prior to FRCM application. During strengthening application, the beam surface was periodically remoistened. Prior to the application of the FRCM, the carbon fabric was cut into different sizes. The strips for the FRCM-I group were cut into three 720 x 75 mm pieces per beam and the fabric for the FRCM-C group was cut into 720 x 540mm sections. In order to mix the cementitious mortar, a hand drill and mortar mixing attachment were used. The mortar was mixed in a 20 litre bucket, with 3.7L of water per 24.9 kg bag of mortar, as established by a previous research effort (Billows, 2016). Once the mortar was prepared, a 5mm layer of mortar was placed onto the entire concrete surface using trowels. Next, the fabric layer was placed onto the mortar, and a second 5mm layer of mortar was placed on top of the fabric. Finally, the surface was finished using trowels. The FRCM application process is shown in Figure 3.7. The mortar cured in open air overnight. The following day, wet burlap and plastic sheets were placed on top of the mortar. The FRCM was kept moist to allow for curing for 5 days after placement. Next, the burlap and plastic sheets were removed, and the material was left exposed to the air until testing (70 days for the strengthened specimens and 28 days for the repaired specimens).
For the NSM FRP application, the bars were then cut to a length of 260mm, using a concrete cutting saw. Some of the bars were then fitted with strain gauges, as outlined in section 3.7 Test Setup. For NSM-M group, the interior of the groove was brought to SSD conditions, by soaking under wet burlap for 24 hours prior to strengthening. For the NSM-E group, the grooves were left dry, as recommended by the manufacturer. The epoxy was mixed to a 1:3 ratio for parts A and B, as recommend by the manufacturer. In order to ensure the two parts were completely mixed, a hand drill with mixing attachment was used. The mortar was mixed, also using a hand drill and mixing attachment, in a 20 litre bucket, with 3.2L of water per 24.9 kg bag of mortar, as recommended by the manufacturer. To mount the bars into the grooves, the grooves were halfway filled with the bonding material, then the bar was placed into the groove and gently rolled to remove any air voids. Following this step, the remainder of the groove was filled with bonding material and troweled smooth. The bar installation process is shown in Figure 3.8, only the mortar adhered bars are shown, as the application process was the same for both materials.
Figure 3.8: NSM application process (mortar adhesive)

The epoxy mounted specimens were allowed to cure at room temperature inside the lab, as no special curing conditions were needed. The mortar mounted bars were allowed to cure overnight, before wet burlap and plastic sheets were placed onto the mortar. These specimens cured in moist conditions for 5 days, before the burlap and plastic sheets were removed. Next, the specimens were cured in laboratory air for 28 days for repaired specimens, or 70 days for strengthened specimens.

3.4 Materials

The concrete which was used was ordered from Kelowna Ready Mix, with a specified strength of 35 MPa, to replicate the strength of concrete used in a typical structure. The concrete was delivered to the lab using a ready-mix truck. The general mix properties can be found in Table 3.2, however Kelowna Ready Mix did not release the full mix design, for confidentiality reasons. The slump and air content were measured in accordance with CSA A23.2-5C/ASTM C143 and A23.2-4C /ASTM C231. The slump was recorded as 125mm and the air content as 3.0%. There is a deviation between the recorded values for the slump and the air content, and the values given in Table 3.2. The deviation in slump was due to the presence of water reducing admixture, which was not accounted for in the tabulated value. The deviation in the air content was due to the assumption
that air entraining admixture was to be added. For this project, it was specified that no air entraining
admixture was to be used, therefore, an air content between 1%-3% is expected (CSA A23.1-
14/A23.2-14, 2014).

Table 3.2: Concrete mix design

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>56 Day Compressive Strength</td>
<td>35 MPa</td>
</tr>
<tr>
<td>Exposure Class</td>
<td>C-1</td>
</tr>
<tr>
<td>Placement</td>
<td>Pump</td>
</tr>
<tr>
<td>Cement Type</td>
<td>GU Cement</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>Yes</td>
</tr>
<tr>
<td>Concrete Sand</td>
<td>Yes</td>
</tr>
<tr>
<td>20-14mm Aggregate</td>
<td>Yes</td>
</tr>
<tr>
<td>14-5mm Aggregate</td>
<td>Yes</td>
</tr>
<tr>
<td>Air Content</td>
<td>5%-8%</td>
</tr>
<tr>
<td>Maximum W/C Ratio</td>
<td>0.4</td>
</tr>
<tr>
<td>Slump (mm) +20</td>
<td>80mm</td>
</tr>
<tr>
<td>Water Reducing Admixture</td>
<td>Yes</td>
</tr>
</tbody>
</table>

In accordance with CSA A23.2-9C and ASTM C39, concrete cylinders with dimensions of
100mm diameter and 200mm in height were cast and allowed to cure in the same conditions as the
beams. The cylinders were tested at 7 and 28 days, as well as the date of damage and the date of
testing. The strengths are summarized in Table 3.3.
Table 3.3: Concrete compressive test results

<table>
<thead>
<tr>
<th>Test Date</th>
<th>Cylinder Strengths (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 Day</td>
<td>28.0</td>
</tr>
<tr>
<td>28 Day</td>
<td>43.0</td>
</tr>
<tr>
<td>Damage Application (70 Day)</td>
<td>45.2</td>
</tr>
<tr>
<td>Testing (98 Day)</td>
<td>45.6</td>
</tr>
</tbody>
</table>

The 25M flexural reinforcement was grade 400 mild steel. The tensile properties of the bars are summarized below in Table 3.4. The tensile properties of the bars are summarized below in Table 3.4. In house tensile testing revealed that the yield strength of the 6.35mm rod was slightly higher, with a significant amount of variability. The results of the testing on the 6.35mm (¼“) rod is shown in Table 3.5.

Table 3.4: Steel tensile properties

<table>
<thead>
<tr>
<th>Rod Size</th>
<th>Grade</th>
<th>( F_{y_{mill}} ) (MPa)</th>
<th>( F_{u_{mill}} ) (MPa)</th>
<th>( F_{y_{test}} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.35mm</td>
<td>300</td>
<td>472</td>
<td>614</td>
<td>-</td>
</tr>
<tr>
<td>25M</td>
<td>400</td>
<td>330</td>
<td>482</td>
<td>361</td>
</tr>
</tbody>
</table>

Table 3.5: 6.35mm bar tensile test results

<table>
<thead>
<tr>
<th>Test</th>
<th>Yield Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>318</td>
</tr>
<tr>
<td>2</td>
<td>338</td>
</tr>
<tr>
<td>3</td>
<td>360</td>
</tr>
<tr>
<td>4</td>
<td>385</td>
</tr>
<tr>
<td>5</td>
<td>405</td>
</tr>
<tr>
<td></td>
<td>Average (Std. dev.)</td>
</tr>
</tbody>
</table>
Both the concrete and the steel were stronger than the specified design values. This was a major contributing factor to the issue with the test setup noted in section 4.1 General. All the beams used in this study were fabricated using the same materials, removing the effect of steel or concrete strengths on strengthening effectiveness.

The fabric used in the FRCM was high performance unidirectional carbon fabric (Figure 3.9), supplied by Simpson Strong-Tie (Simpson Strong-Tie, 2019). The summarized material properties, provided by the manufacturers, are shown in Table 3.6 (Simpson Strong-Tie, 2019). The material properties of the FRCM composite are also provided in Table 3.7 (Simpson Strong-Tie, 2019).

Table 3.6: Carbon fabric material properties (Simpson Strong-Tie, 2019)

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fabric Area (mm²/m*)</td>
<td>157</td>
</tr>
<tr>
<td>Ultimate Tensile Strength (kN/m*)</td>
<td>450</td>
</tr>
<tr>
<td>Ultimate Tensile Strain (%)</td>
<td>1.5</td>
</tr>
<tr>
<td>Axial Stiffness (kN/m*)</td>
<td>30000</td>
</tr>
</tbody>
</table>
*Value per metre width of fabric

Table 3.7: FRCM composite material properties (Simpson Strong-Tie, 2019)

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracked Tensile Modulus (GPa)</td>
<td>49</td>
</tr>
<tr>
<td>Ultimate Tensile Strain (%)</td>
<td>1.1</td>
</tr>
<tr>
<td>Ultimate Tensile Strength (MPa)</td>
<td>885</td>
</tr>
</tbody>
</table>
The 6mm basalt rods used for the NSM system were supplied by MagmaTech (MagmaTech, 2020). The bar surfaces are sand coated, to ensure a better bond with the adhesive. The mechanical properties of the bars were extrapolated from a research endeavor by Akiel (2016). In this study, bars from the same manufacturer, with larger dimensions, were tested. The average tensile strength of the bars was found to be 1231 MPa, and the modulus of elasticity 46.6 GPa (Akiel, 2016). The summarized results of the testing are presented in Table 3.8. From the findings of Akiel (2016), for the 6mm bars used the ultimate tensile capacity was taken as 35 kN, and the ultimate strain as 0.026.

<table>
<thead>
<tr>
<th>Rod Size</th>
<th>Pu (kN)</th>
<th>Fu (MPa)</th>
<th>E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8mm</td>
<td>62.1</td>
<td>1235</td>
<td>47.5</td>
</tr>
<tr>
<td>10mm</td>
<td>96.4</td>
<td>1227</td>
<td>46.1</td>
</tr>
<tr>
<td>12mm</td>
<td>139.0</td>
<td>1230</td>
<td>46.2</td>
</tr>
</tbody>
</table>
The adhesives used for the NSM FRP systems were a two part epoxy supplied by Sika Group Canada (Sika, 2017), and a high performance mortar supplied by Simpson Strong-Tie (Simpson Strong-Tie, 2019). The epoxy was chosen, as it was recommended by the manufacturer for bonding FRP rods with concrete. The mechanical properties of the epoxy are given in Table 3.9. The mortar was supplied by Simpson Strong-Tie as an adhesive for the carbon fabric used in the FRCM. The material data for the mortar is provided in Table 3.9.

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Sikadur 30</th>
<th>Cementitious Matrix</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength (MPa)</td>
<td>24.8</td>
<td>4.8</td>
</tr>
<tr>
<td>Compressive Strength (MPa)</td>
<td>59.3</td>
<td>52</td>
</tr>
<tr>
<td>Tensile Bond Strength (MPa)</td>
<td>22</td>
<td>2.7</td>
</tr>
<tr>
<td>Shear Bond Strength (MPa)</td>
<td>15</td>
<td>2.1</td>
</tr>
</tbody>
</table>

### 3.5 Damage

In order to assess the effectiveness of the various rehabilitation methods as repair techniques, some of the specimens were damaged by preloading to various levels. The two levels of damage selected were the 1\textsuperscript{st} visible shear crack and 70\% of the maximum theoretical load (147 kN), as calculated by the combined strut and tie model (Garay-Moran & Lubell, 2008). The levels of damage were selected to simulate real world overload conditions, with various inspection intervals. The 1\textsuperscript{st} shear crack damage level represents a well monitored structure, with regular inspection intervals. The 70\% of the maximum theoretical load damage level represents a less well inspected structure, where substantial levels of damage have developed before being detected. To apply the
two levels of damage, the specimens were loaded in the test setup described in section 3.7 Test Setup. The 1st shear crack was detected by visual inspection during the tests. Once the shear crack was observed, the load application was stopped, and the load was released. The tensile strains observed in the load-strain diagrams, shown in APPENDIX B, confirm that the stirrups were engaged in resisting the load, meaning that the crack formed. In Beam S150-RE-D1, it was observed that the tensile strain began to develop in the stirrups at a load of approximately 30 kN. This is due to the formation of a flexure crack which aligned with the stirrup. For the damage level corresponding to the 70% of the maximum theoretical load, specimens were loaded until the load cell in the actuator recorded a value of 147 kN. Once this load was obtained, the load application was then stopped, and the load was released. For these specimens, analysis of the load strain-strain curves showed that tension had developed in the steel stirrups, and therefore, cracking had occurred. The load strain diagrams are shown in Appendix B. In all cases, the first shear crack formed at loads between 105 kN and 115 kN. Once the damage was completed the specimens were removed from the test frame, repaired as described in section 3.3.3 Strengthening Application, and then tested again to the point of failure.

3.6 Instrumentation

Strain gauges were applied in several different locations, to monitor the beams’ behavior as load was applied. The material surfaces were prepared to the specifications of the Tokyo Material Laboratories Strain Gauge Users Guide, to insure the best results (Tokyo Material Laboratories, 2019). This involved sanding the material surface smooth, and cleaning with acetone to provide a uniform and clean surface for application. The concrete was lightly ground to remove imperfections, sanded with 60 grain sandpaper, and then coated with epoxy in order to seal the
surface. Once the epoxy hardened, it was sanded and cleaned as described earlier. The FRP bars and fabric had a thin layer of epoxy applied, in order to assure a smooth surface, before sanding and cleaning. After application, the strain gauge and terminal surfaces were coated with a layer of bee’s wax to provide protection from contamination. Tokyo Material Laboratories (TML) rosette style strain gauges PFLR-30-11 with a gauge length of 30mm, and PLR-60-11 with a gauge length of 60mm, were applied to the concrete surface, near the anticipated shear crack location (Figure 3.10). Strain gauges, TML FLAB-30-11 with a gauge length of 30mm, were applied to the uniaxial fabric used in the FRCM (Figure 3.10). A numbering scheme from 1-3 was incorporated for the fabric gauges, with 1 being closest to the support and 3 closest the point load. Strain gauges, TML FLA-5-11 with a gauge length of 5mm, were applied to the steel stirrups and FRP NSM bars (Figure 3.10). For the NSM FRP bar gauges, a numbering scheme from 1-3 was incorporated, with 1 being closest to the support and 3 closest the point load. Only the stirrup closer to the support was instrumented in most of the specimens. This was due to a change in the test parameters. Originally the uninstrumented stirrups would have been in the non-shear critical section, however, due to the issue outlined in section 4.1 General it was moved to the shear critical section. Strain gauges, TML PL-60-11 with a gauge length of 60mm, were applied to the compression face of the concrete, as close as reasonably possible to the point load, on Beams S150-WF-D2 and S200-WF-DN (Figure 3.10). These gauges were utilized because Beam S150-RE-D2 failed due to concrete compression (as described in section 4.2 Beam Behaviour and Failure Modes) in this region, therefore, more data on this region was desired.
To measure specimen deflection, a linear potentiometer (LINPOT) was positioned at the beam’s midspan (750mm from the shear critical support). To monitor crack formation on the beams surface, the shear critical region of the beam was painted with white paint to make the appearance of cracks easier to identify.

3.7 Test Setup

The RC beams were tested in three-point bending, in a specially constructed “sideways” test frame, due to limitations of the testing facility in the lab. In this setup, the beam was rotated 90° in order to allow the actuator to be braced between the strong frame and the beam. The sideways test was not anticipated to have a major effect on the results. The bending moment and shear forces in the shear critical section incurred by beams’ own weight, in the direction of gravity, were 2
orders of magnitude lower than those caused by the externally applied loading, in the plane of applied loading, as summarized below in Table 3.10.

Table 3.10: Maximum shear and bending in shear critical section

<table>
<thead>
<tr>
<th></th>
<th>Beam Self Weight</th>
<th>Theoretical Maximum Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear (kN)</td>
<td>0.9</td>
<td>140</td>
</tr>
<tr>
<td>Bending Moment (kN-m)</td>
<td>0.35</td>
<td>70</td>
</tr>
</tbody>
</table>

Load applied by the using a Fluid Power Solutions (FPS) actuator. The load was applied to the “top” of the beam, into the plane of Figure 3.11. The supports were positioned 1500mm apart. Rollers were used for the point load and supports, as shown in Figure 3.12. These rollers applied line loads to the “top” and “bottom” of the beam, which were idealized as point loads. For data acquisition (DAQ), LabView software was used in conjunction with a National Instruments (NI) cDAQ-9178 DAQ system to record and tabulate the data from the strain gauges and linear potentiometer (LINPOT). A master-slave system was used to trigger the actuator and DAQ software simultaneously through the actuator control computer. Data recorded included actuator load, actuator displacement, mid-span deflection and the various strains outlined earlier. The test was displacement controlled at a constant rate of 2mm/minute and the data was acquired at a frequency of 2.5 Hz. The test was run continuously, with crack patterns and crack values being recorded on the beam’s surface, as they occurred. Additionally, video footage of the tests was recorded and reviewed to confirm the beams cracking behaviour.
Figure 3.11: Test setup

Figure 3.12: Point load and support details
CHAPTER 4: EXPERIMENTAL RESULTS AND DISCUSSION

4.1 General

To determine the effectiveness of various strengthening systems for increasing the shear strength of RC beams, the experimental program outlined in the previous chapter was followed. The test matrix (Table 3.1) provides a summary of the testing done. The test parameters were strengthening system used (NSM FRP bars and FRCM wrap), continuous vs intermittent FRCM wrap, mortar or epoxy NSM adhesive, steel stirrup spacing, and level of damage (No damage, 1st shear crack, and 70% of the maximum theoretical load). Beams with dimensions of 200x265x2000mm were tested, in three-point bending, at the Structures Lab at UBC Okanagan campus. The performance indicators used included peak load, energy absorption, failure mode, shear crack formation load, and beam stiffness.

When the experimental setup was originally being designed, it was intended that there would be two a/d ratios. The a/d ratios of the beams were intended to be 2.5, with a shear critical region of 540mm, and 1.5, with a shear critical section of 320mm. However, due to limitations of the testing equipment, namely that the portal test frame was not high enough to facilitate the 500kN actuator. Due to this change, the a/d ratio was forced to be held constant at 2.3, with a shear critical span of 500mm. This had the unintended consequence of nullifying the effect of the stirrup spacing. For both stirrup spacings, 150mm and 200mm, the number of stirrups in the shear critical section was the same. For this reason, the results of the 200mm spacing group were almost identical to their 150mm spacing counterparts.
Beam failure was taken as a load drop of over 20% relative to the peak load attained. All the specimens were loaded to the point of failure. In this research effort no steel yielding was observed. Therefore, brittle failure modes were those in which diagonal tension failure occurred immediately following the peak load. In the gradual failure modes, a period of load plateau was observed. This was caused by the strengthening materials debonding before diagonal tension failure occurred. After the completion of testing, the beams were inspected for debonding of the strengthening system, and verification of the failure mode. The FRCM and NSM FRP bars were tapped with a hammer to qualitatively detect any areas of debonding (hollow sound indicating debonding). In areas where shear cracks intercepted the strengthening systems, the mortar or epoxy were chipped away to view the surface of the strengthening for signs of rupture. However, no rupture was observed.

As described in section 3.6 Instrumentation, strain gauges were affixed to the beam in a maximum of 7 positions. Locations which were fitted with strain gauges included the first stirrup of the shear critical end of the beam, the strengthening systems, the surface of the concrete near the anticipated location of the shear crack, and the compression face of the beam. These strain gauges were used to monitor changes in the load path as the load increased. Most strain gauges worked as intended, the complete load-strain curves are seen in APPENDIX B and C. However, several of the strain gauges were either unresponsive before testing began or malfunctioned during the test. Strain gauges which failed are noted in APPENDIX C.

Two of the beams suffered from premature debonding of the FRCM wrap before the test began, or at loads far lower than the debonding was observed in the other specimens. For Beam S200-WI-DN, during test setup the FRCM wrap nearest to the support was damaged. This was evident
once the test began, as the wrap debonded immediately. In Beam S150-WF-DN, half of the FRCM wrap, on the side which contained the strain gauges, debonded early in the test. While this negatively impacted the quality of the data from this specimen, conclusions could still be drawn.

4.2 Beam Behaviour and Failure Modes

In the control specimens, flexural cracking was observed at loads of 20kN and 70kN, for Beam S200-WN-DN and S150-WN-DN, respectively. For the higher value, it is likely that flexural cracking occurred earlier, but was not observed during testing. In each control beam, the first shear crack formed between 110kN and 115kN. In Beam S200-WN-DN, a smaller shear crack formed at approximately 200 kN, shortly before the beam failed (Table 4.1). A schematic drawing of the shear critical section is provided in Figure 4.1, for reference. As the beams were loaded, the crack patterns which formed were traced onto the surface of the beam. In addition to the crack pattern the load at which the crack formed was recorded on the beam. This can be seen in Figure 4.2. Both of the control specimens failed due to diagonal tension, at loads of 217 kN and 225 kN, for S150-WN-DN and S200-WN-DN, respectively (Figure 4.2).

![Figure 4.1: Schematic of shear critical section](image-url)
Figure 4.2: Shear critical sections of beams after failure
In the NSM-E group, the majority of the initial flexure cracks opened at loads between 25kN and 35kN, similar to Beam S150-WN-DN. An example of the crack propagation, characteristic of the beams in this study, is provided in Figure 4.3. In Figure 4.3 a), the first flexure crack in the shear critical region is observed at a load of 50kN. In Beam S150-RE-D1, the first flexure crack opened at 60kN. However, it was likely that other cracks opened earlier, but were not observed during testing. In the NSM-E group, the majority of initial shear crack formation occurred at approximately 120kN. In Figure 4.3 b), the first shear crack was observed at a load of 140kN. In Beam S150-RE-D1, the shear crack, introduced during the damage phase reopened at a load of only 60kN when tested after strengthening (Table 4.1). Propagation of the first shear crack and growth of new flexure cracks is observed in Figure 4.3 c). In all NSM-E Beams a secondary shear crack formed between 200kN and 263kN, as shown in Figure 4.3 d). Most of the specimens in the NSM-E group failed due to debonding of the NSM FRP bars followed by diagonal tension failure (Figure 4.2). The exception to this trend was Beam S150-RE-D2, which failed due to combined shear and flexure. In the beams which failed due to debonding followed by diagonal tension, load increased until the middle set of bars began to debond. At this point, one of two debonding failure modes occurred. Either the beam lost load bearing capacity immediately after the bars debonded (Beam S150-RE-D1), or the load was transferred to the sets of bars closest to the support and point load (Beams S150-RE-DN and S150-RE-D2). Debonding of the middle set of bars is shown in Figure 4.3 e). In the beams which did not immediately fail, a load plateau, or slight drop in load followed by a load plateau was observed, as load shifted between the remaining bars before failure occurred (Figure 4.3 f). As noted earlier, Beam S150-RE-D2 failed due to combined shear and flexure. In this beam, as the load increased substantial shear and flexural cracking was observed in the non-critical section of the beam. Due to the combination of shear and flexural acting on the
beam, the concrete compression block crushed. No debonding of the bars was observed in this specimen. Failure in the NSM-E beams occurred between 294kN and 323kN (Table 4.1).

Figure 4.3: Crack Propagation in Beam S150-RE-DN

In the NSM-M group, flexure cracks were observed in all specimens at loads between 30kN and 40kN, similar to the control Beam S200-WN-DN. For Beams S150-RM-D1 and S150-RM-D2, the shear cracks introduced during damage reopened during the test at loads of 40kN and 35kN, respectively. In the undamaged beam in the NSM-M group, primary shear cracking occurred at 125kN, and secondary shear cracking at 200kN, similar to the control. In Beam S150-RM-DN, a third shear crack was observed at 300kN, shortly before the specimen failed (Table 4.1). Failure in the NSM-M group beams was caused by debonding followed by diagonal tension (Figure 4.2). Similar to the beams in the NSM-E group, debonding failure began in the middle set of bars, followed either by immediate failure (Beams S150-RM-DN and S150-RM-D2) or load plateau (Beam S150-RM-D1), as the load redistributed. These failures occurred between 213kN
and 310kN (Table 4.1). Beam S150-RM-DN is noted as a premature debonding failure, as the middle set of bars debonded at a load of 150kN, much lower than the anticipated load based on the other debonding events in this group. An annotated example of the load-strain curve for beam S150-RM-D1 provided in Figure 4.4. Load strain curves for the remaining beams can be found in APPENDIX C.

Figure 4.4: Load-strain behavior of Beam S150-RM-D1
In the FRCM-C group, initial flexural cracking was observed at a load of approximately 30kN. In the FRCM-C group, more shear cracks formed, with 3-5 cracks forming before failure occurred. Despite the initial damage, no prematurely opening cracks were observed, as shear cracking began between 95kN and 140kN (Table 4.1). The exception to the trend of numerous cracks was in Beam S150-WF-DN, in which only one shear crack formed. This was likely influenced by the alternative failure mode observed in the beam (Figure 4.2). In the FRCM-C group, all the Beams, except for S150-WF-DN, failed due to loss of bond. These bond failures included the delamination of the outer layer of mortar, loss of bond between mortar and the concrete, or a combination of the two phenomena, followed by diagonal tension failure (Figure 4.2). These bond failures began at the midpoint of the shear critical section, and radiated outward towards the supports until load bearing capacity was lost (Figure 4.5).

![Figure 4.5: Fully FRCM wrapped beam failure modes](image-url)
Failure in the FRCM-C group occurred at loads between 274kN and 304kN (Table 4.1). While the debonding could not be visually observed during the tests, it was confirmed by cracking noises in the FRCM, accompanied by load plateauing and strain redistribution in the FRCM strain gauges. In Beam S150-WF-DN, the FRCM completely debonded on one side of the beam at a load of 130kN, immediately after the formation of the first shear crack. However, this debonding did not compromise the beams load bearing capacity. The beam continued to bear load until 304 kN, at which point the fabric on the remaining side of the beam began slipping through the mortar (Figure 4.5). As the fabric slipped, load bearing capacity was gradually lost until the load dropped by more than 20%.

In the NSM-I group, the first flexure and shear cracks occurred at approximately 30kN and 130kN, as predicted by the control beams (Table 4.1). Despite the initial damage to beam S150-WI-D1, no reopening of older shear cracks was observed. In Beams S150-WI-DN and S150WI-D1, secondary shear crack formation occurred at 200kN, similar to the control beams. All of the intermittently wrapped group beams failed due to debonding of the FRCM strips, followed by diagonal tension failure in the RC beam (Figure 4.2). In the FRCM-I group it was generally observed that as load increased, the middle strip of FRCM debonded. This debonding caused the load on the beam to redistribute to the remaining strips. Beams S150-WI-D1 and S200-WI-DN exhibited some load plateau before a second strip of FRCM wrap debonded. Alternatively, Beam S150-WI-DN failed immediately after the middle strip debonded. Failure in the FRCM-I group occurred between 227kN and 285kN (Table 4.1). In Beam S200-WI-DN, the strip closest to the support was damaged during loading into the test frame, as noted above in section 4.1 General. For this reason, the strengthening was not effective in increasing the load bearing capacity of the beam.
### Table 4.1: Crack formation, peak load, and failure modes

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen ID</th>
<th>Flexure Crack (kN)</th>
<th>1st Shear Crack (kN)</th>
<th>Number of Shear Cracks</th>
<th>Peak Load (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>S200-WN-DN</td>
<td>20</td>
<td>110</td>
<td>2</td>
<td>224.5</td>
<td>Diagonal Tension</td>
</tr>
<tr>
<td></td>
<td>S150-WN-DN</td>
<td>70</td>
<td>115</td>
<td>1</td>
<td>217.1</td>
<td>Diagonal Tension</td>
</tr>
<tr>
<td>NSM-E</td>
<td>S150-RE-DN</td>
<td>35</td>
<td>140</td>
<td>2</td>
<td>307.1</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D1</td>
<td>60</td>
<td>118/60</td>
<td>2</td>
<td>293.8</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D2</td>
<td>25</td>
<td>130/115</td>
<td>3</td>
<td>323.1</td>
<td>Concrete crushing</td>
</tr>
<tr>
<td>NSM-M</td>
<td>S150-RM-DN</td>
<td>40</td>
<td>125</td>
<td>2</td>
<td>212.6</td>
<td>Premature debonding before diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D1</td>
<td>30</td>
<td>120/40</td>
<td>3</td>
<td>309.7</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D2</td>
<td>35</td>
<td>110/35</td>
<td>2</td>
<td>271.1</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td>FRCM-C</td>
<td>S150-WF-DN</td>
<td>30</td>
<td>130</td>
<td>1</td>
<td>304</td>
<td>Fabric Slip</td>
</tr>
<tr>
<td></td>
<td>S150-WF-D1</td>
<td>15</td>
<td>115/95</td>
<td>5</td>
<td>311.2</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S150-WF-D2</td>
<td>25</td>
<td>118/115</td>
<td>3</td>
<td>273.7</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S200-WF-DN</td>
<td>30</td>
<td>140</td>
<td>5</td>
<td>302</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td>FRCM-I</td>
<td>S150-WI-DN</td>
<td>30</td>
<td>125</td>
<td>2</td>
<td>254.9</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S150-WI-D1</td>
<td>15</td>
<td>110/130</td>
<td>2</td>
<td>285.5</td>
<td>Debonding followed by diagonal tension</td>
</tr>
<tr>
<td></td>
<td>S200-WI-DN</td>
<td>25</td>
<td>130</td>
<td>1</td>
<td>227.2</td>
<td>Premature debonding before diagonal tension</td>
</tr>
</tbody>
</table>

*Shear Cracking during damage phase/reloading after strengthening

### 4.3 Load-Deflection Response, Ultimate Loads, and Capacity Increase

The load-deflection curves of all beams are presented individually in APPENDIX D. In both of the control Beams (S150-WN-DN and S200-WN-DN), the load-deflection response was relatively linear with a slight drop (<5 kN) in load at approximately 115kN, when the initial shear crack formation occurred (Figure 4.6). Following the drop, the linear behavior continued until the
point of peak load for Beam S150-WN-DN. Beam S200-WN-DN experienced a change in stiffness at 200kN. This change in stiffness was due to the formation of the secondary shear crack. Load continued to linearly increase, with a slight drop, accompanied by the widening of both shear cracks. Linear behavior briefly resumed before the peak load was reached. After the point of maximum load on both curves, the load rapidly decreased until it dropped by more than 20%, ending the test (Figure 4.6).

Figure 4.6: Load-deflection curves for control specimens

The peak loads of the tested specimens are summarized in Table 4.1. With the exception of Beams S200-W1-DN and S150-RM-DN, for the reasons discussed in section 4.2 Beam Behaviour and Failure Modes, all of the beams saw a substantial increase in load bearing capacity (17%-49% compared to the control Beams S200-WN-DN and S150-WN-DN) (Figure 4.7).
The NSM-E group had similar initial behavior to the control Beam (S150-WN-DN), however there was no load drop when the shear crack formed, as load transferred from the concrete to the NSM FRP. Beam S150-RE-D1 experienced a similar failure behavior to the control specimens, wherein the load rapidly decreased following the peak load. This sudden failure was due to the middle bars debonding, and a poor bond between the remaining NSM bars and the concrete substrate. In Beams S150-RE-DN and S150-RE-D2, the load dropped slightly after the peak load, followed by some load plateauing (Figure 4.8). In Beam S150-RE-DN, this plateauing was caused by the load redistribution from the middle set of bars to the remaining bars causing them to gradually debond. In Beam S150-RE-D2, which did not experience any bar debonding, the plateau was likely due to a loss of inter-aggregate friction as the shear cracks had opened dramatically at loads above 320kN. The NSM-E group saw the most substantial increase in load bearing capacity, compared to the control beams. Beams S150-RE-DN, S150-RE-D1, and S150-RE-D2 showed an increase 41.5%, 35.5%, and 49.0% respectively. These values translated to an average increase of 42%, compared to the control (Figure 4.7).
In the NSM-M group, Beams S150-RM-DN and S150-RM-D2 failed suddenly, similar to the control Beam S150-WN-DN. Whereas, Beam S150-RM-D1 showed a failure similar to the epoxy adhered group. Beam S150-RM-DN showed identical behavior to Beam S150-WN-DN, up to the peak load. This included the slight drop in load after the formation of the initial shear crack. In Beam S150-RM-DN, load did not redistribute to the strengthening system due to a poor bond in the middle set of bars, which debonded at 150kN, shortly after the initial shear crack formation. Immediately after Beam S150-RM-DN reached its peak load, the load dropped slightly (25kN). As the load had not decreased by 20%, the test was continued. The load plateaued for a period, before slowly decreasing until it dropped below 20% of the peak load. This load plateau was caused by the gradual debonding of the remaining FRP bars. In Beams S150-RM-D1 and S150-RM-D2, the slight drop associated with the formation of the shear crack seen in the control Beam S150-WN-DN was not observed. In these beams, the load was able to redistribute from the concrete to the strengthening system. Failure in Beam S150-RM-D2 was identical to that of the
control (S150-WN-DN), with an immediate loss of load bearing capacity. This sudden loss was due to poor bond between the remaining NSM bars and the concrete substrate. Beam S150-RM-D1 showed similar behavior to Beam S150-RE-DN from the NSM-E group, with a slight drop in the load after the peak load followed by a period of load plateau before the load dropped further (Figure 4.9). This load plateau was due to the load redistribution from the middle set of NSM bars to the outer sets, which gradually debonded eventually causing failure. Beams S150-RM-DN, S150-RM-D1, and S150-RM-D2 saw increases of -2.0%, 42.9%, and 24.9% respectively. This translated to an average increase of 22% compared to the control (Figure 4.7).

![Figure 4.9: Load-deflection curves for the NSM-M group](image)

All of the beams in the FRCM-C group showed a linear increase in the load up to the point of peak loading (Figure 4.10). The slight load drop associated with the initial shear crack observed in the control Beams was not observed due to effective load redistribution to the FRCM. Beam S150-WF-D2 showed an immediate drop in the load following the peak, similar to the control Beam (S150-WN-DN) (Figure 4.10). This drop was due to a poor bond between the FRCM and concrete
substrate. It can be seen in Figure 4.5 that most of this region debonded immediately after the peak load. The remaining Beams, S200-WF-DN, S150-WF-DN, and S150-WF-D1 experienced a period of load plateau before failure. In Beams S200-WF-DN and S150-WF-D1, the load plateau was caused by the gradual debonding of the FRCM radiating outward from the midpoint of the shear critical section (Figure 4.10). In Beam S150-WF-DN, the slight drop was caused by the initial slip of the fabric, and load plateau was caused by the gradual continued slip of the fabric through the mortar. The FRCM-C group saw strength increases of 34.0%, 40.0%, 43.3%, and 26.3% for Beams S200-WF-DN, S150-WF-DN, S150-WF-D1, and S150-WF-D2, respectively. This translated to an average increase in load bearing capacity of 36% (Figure 4.7).

![Load-deflection curves for the FRCM-C group](image)

Figure 4.10: Load-deflection curves for the FRCM-C group

In the FRCM-I group, all of the beams showed a linear increase in load up to the point of peak load, or near peak load in the case of Beam S150-WI-D1 (Figure 4.11). Similar to the behavior observed in the other groups, effective load distribution to most of the FRCM-I group prevented the slight load drop associated with the initial shear crack formation. In Beam S150-WI-DN, a
small (<5kN) drop in the load was noted when the first shear crack formed. In Beams S200-WI-DN and S150-WI-DN, the load dropped almost immediately after the peak load, similar to the control beams (Figure 4.11). This sudden drop was due to the inability of the remaining FRCM strips to take the load, following the debonding of the middle strip. Beam S150-WI-D1 showed a period of load plateau after reaching 277kN, as load redistributed from the middle strip to the support and point load strips (Figure 4.11). As displacement continued, the load slowly increased to its peak value and then rapidly dropped (Figure 4.11). This drop was caused by the debonding of the remaining strips. The FRCM-I Beams, S200-WI-DN, S150-WI-DN, and S150-WI-D1, saw load bearing increases of 0%, 17.5% and 31.6% respectively. This resulted in an average increase in strength of 17% compared with the control (Figure 4.7).

Figure 4.11: Load-deflection curves for the FRCM- I group
4.4 Stiffness Response

The linear portions of the load-deflection curves prior to peak load were analyzed to determine the stiffness of the beams. To calculate the stiffness of the beams, a regression line was generated for each curve. Points between 10% and 90% of the peak load were considered, to remove the influence of non-linear regions near the beginning and end of the curve. All the regression curves displayed an $R^2>0.99$, indicating strong correlation. The stiffness values of the beams relative to the stiffness of the control beams (S150-WN-DN and S200-WN-DN), grouped by NSM and FRCM strengthening systems, and are shown in Figure 4.12 and Figure 4.13.

![Figure 4.12: NSM FRP bar group stiffness values](#)
It can be observed that for the NSM-E group there is a very minimal variation in the beams’ stiffness. In the NSM-M group, neglecting Beam S150-RM-D1 during damage, a similar lack in variation of stiffness is observed. During damage application, Beam S150-RM-D1 displayed a substantially higher level of stiffness (2.4 standard deviations) than any of the other beams. Considering that after strengthening its stiffness was almost identical to the control and the rest of the beams, it is likely that the higher stiffness value was due to a reading error with the LINPOT.

The standard deviation for the stiffness of the FRCM group was identical to the NSM group (0.1). Both undamaged, intermittently wrapped specimens, S150-WI-DN and S200-WI-DN, exhibited lower levels of stiffness, 1 standard deviation and 1.7 standard deviations, respectively.
With a significance level of 0.05 none of the variations in stiffness values were statically significant.

### 4.5 Displacement Analysis

Due to the sudden nature of shear failure, any increase to the warning before failure can be vital to improving public safety. The first type of displacement analysis was the investigation of increase to peak load displacement. Due to the strength increase of the strengthening systems, without a change to the stiffness, the peak load displacements of the strengthened beams were increased. The average peak load displacements of each group, normalized to the control beam values are shown in Figure 4.14. As expected, the epoxy mounted NSM FRP and continuously wrapped FRCM strengthening systems which yielded higher peak loads also showed higher peak load displacements.

![Figure 4.14: Peak load displacements normalized to control](image)

In this study pseudo-ductility, also referred to as ductility index or displacement ductility, which refers to the ratio of the deflection at the ultimate load to the deflection at which point linear behavior of the load-displacement curve ended (yield was not used as the steel reinforcement was
not observed to have yielded). This value is important as higher pseudo-ductility values correlate with higher global deformation, as well as energy absorption. The average pseudo-ductility of each strengthening group is shown in Figure 4.15.

![Figure 4.15: Group pseudo-ductility values](image)

From the analysis of the pseudo-ductility values, it is evident that all the strengthening systems had the ability to increase the deformation capacity of the beams. Both NSM FRP bar systems saw similar increases to pseudo-ductility (approximately 50% increase). The continuous FRCM wrap was the most effective system for increasing the systems pseudo-ductility (58% increase), conversely the intermittent strips showed much less pseudo-ductility (20% increase).

In addition to the pseudo-ductility, another method to measure the ductility of the beams is energy absorption. In this study, energy absorption was taken as the area under the load-deflection curve, up to the point of failure (20% drop from peak load). The energy absorption values for the beams are tabulated in Table 4.2.
Table 4.2: Energy absorption

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen ID</th>
<th>Energy Absorption (kN-mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>S200-WN-DN</td>
<td>1271</td>
</tr>
<tr>
<td></td>
<td>S150-WN-DN</td>
<td>1026</td>
</tr>
<tr>
<td>NSM-E</td>
<td>S150-RE-DN</td>
<td>2664</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D1</td>
<td>1898</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D2</td>
<td>4237</td>
</tr>
<tr>
<td>NSM-M</td>
<td>S150-RM-DN</td>
<td>1498</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D1</td>
<td>3443</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D2</td>
<td>1343</td>
</tr>
<tr>
<td>FRCM-C</td>
<td>S150-WF-DN</td>
<td>5674</td>
</tr>
<tr>
<td></td>
<td>S150-WF-D1</td>
<td>2540</td>
</tr>
<tr>
<td></td>
<td>S150-WF-D2</td>
<td>2014</td>
</tr>
<tr>
<td></td>
<td>S200-WF-DN</td>
<td>2408</td>
</tr>
<tr>
<td>FRCM-I</td>
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<td>1196</td>
</tr>
<tr>
<td></td>
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<td>2380</td>
</tr>
<tr>
<td></td>
<td>S200-WI-DN</td>
<td>1197</td>
</tr>
</tbody>
</table>

The energy index, which was the ratio of energy absorbed by each beam compared to the control, is plotted in Figure 4.16. Every beam, except for S200-WI-DN, showed an increase in energy absorption relative to the control value. The largest values were Beams S150-WF-DN with an increase of 427%, S150-RE-D2 with an increase of 313%, and S150-RE-D1 with an increase of 235%. In the remaining beams which showed an energy absorption increase over 100%, there was some load plateau observed in the load-deflection curve between peak load and failure, leading to the significant increase in energy absorption. The beams which showed <100% increase in the energy absorption failed suddenly with little warning, albeit at a higher load value than the control specimen. For the most part, the damage did not appear to influence the beams’ energy absorption.
capacity. The continuously wrapped group is an exception to this trend. In this group as the damage level increased the energy absorption decreased.

Figure 4.16: Comparison of relative energy absorption
CHAPTER 5: RESULTS ANALYSIS

5.1 Stirrup Spacing

As noted in section 4.1 General, due to issues regarding the test setup, the effect of stirrup spacing was less pronounced than desired. In both the 150mm and 200mm spacings, there was the same number of stirrups in the beams’ shear critical section. Due to this issue, the results for both classes of beam ended up being almost identical. The exception to this trend was the ductility behavior of Beams S150-WF-DN and S200-WF-DN. In this case, the 150mm spacing beam exhibited a 100% increase in ductility, compared to its 200mm spacing counterpart. This difference, however, is better explained by the failure mode of the FRCM wrap, rather than the stirrup spacing. In Beam S150-WF-DN, failure occurred due to fabric slip, which has been observed to be a gradual failure mode. Whereas, in Beam S200-WF-DN, sudden mortar-concrete debonding failure was observed.

5.2 Strengthening System

5.2.1 Effect of NSM Adhesive

By examining the material properties of both adhesives, it is intuitive that the stronger epoxy resin would exhibit improved performance (Sika, 2017; Simpson Strong-Tie, 2019). This was confirmed during the testing, wherein the NSM-M beams only achieved 86% of the average peak load of the NSM-E beams (Figure 5.1). This confirms the findings of Al-Mahmoud et al. (2015), who also observed superior performance of epoxy adhesive compared with mortar. Despite achieving higher peak loads than the NSM-M group, the NSM-E group exhibited slightly lower average pseudo-ductility (96% that of the mortar group). Due to the higher peak load exhibited by
the beams in the NSM-E group, they were able to absorb, on average, 10% more energy before failure compared to the NSM-M group (Figure 5.1). With respect to stiffness, both adhesive types were equally suitable for restoring the beams’ stiffness to their pre-damage levels.

![Bar chart showing peak load and energy absorption relative to NSM-E](image)

**Figure 5.1:** NSM-M peak load and energy absorption relative to NSM-E

### 5.2.2 FRCM Wrapping Scheme

For the comparison of the FRCM wrapping schemes, Beam S150-WF-D2 was disregarded, as there was no FRCM-I damage level 2 counterpart. As the continuously wrapped group contained twice the fabric area of the intermittently wrapped group, it is intuitive that the FRCM-C wrapping scheme would be more effective. This is confirmed by comparing the average results of both groups. It was found that the average peak load of the intermittently wrapped group was only 83% that of the fully wrapped group (Figure 5.2). Additionally, the fully wrapped group exhibited an average ductility increase of 30% over the intermittently wrapped group (Figure 4.15), and a 160% increase in energy absorption (Figure 5.2).
5.2.3 NSM FRP Bar and FRCM Wrap Comparison

A summary of the results, grouped by strengthening systems, is presented in Table 5.1. The performance indicators used to compare the different strengthening systems were strength gain and energy absorption (EA) relative to the control specimens. As there were no 200mm stirrup spacing specimens for the NSM group, Beams S200-WF-DN and S200-WI-DN were disregarded for this analysis.

Table 5.1: Strengthening effectiveness summary

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen ID</th>
<th>Axial Stiffness (kN)</th>
<th>Strength Gain (%)</th>
<th>EA Gain (%)</th>
</tr>
</thead>
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<tr>
<td>NSM-E</td>
<td>S150-RE-DN</td>
<td>7783</td>
<td>42</td>
<td>160</td>
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<td></td>
<td>S150-RE-D1</td>
<td>7783</td>
<td>35</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>S150-RE-D2</td>
<td>7783</td>
<td>49</td>
<td>310</td>
</tr>
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<td>46</td>
</tr>
<tr>
<td></td>
<td>S150-RM-D1</td>
<td>7783</td>
<td>43</td>
<td>240</td>
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<tr>
<td></td>
<td>S150-RM-D2</td>
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<td>31</td>
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<td>FRCM-C</td>
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</tbody>
</table>
The most effective strengthening method for load bearing increase was the NSM-E, with an average increase of 42%, compared to the control. The next most effective was the FRCM-C, with an average capacity improvement of 37%. The NSM-M and FRCM-I exhibited similar average peak load increases of 22% and 25%, respectively. These values are plotted in Figure 5.3. To compare energy absorption, the difference between the total energy absorption of the respective system and the control was used. The FRCM-C beams exhibited the highest average energy absorption increases, relative to the control (250%), as shown in Figure 5.3. The next highest energy absorption increase was associated with the NSM-E group, with a capacity increase of 190% (Figure 5.3). The NSM-M beams showed an energy absorption increase of 100%, relative to the control (Figure 5.3). The FRCM-I beams however, showed substantially less energy absorption than the rest of the strengthening systems (Figure 5.3). The findings of this study were highly sensitive to failure mode. For example, between Beams S150-WF-DN and S150-WI-DN, the strength gain was within 3%, however, the difference between EA gain was over 200%. This large discrepancy was due the gradual, fabric slip, failure observed in S150-WF-DN, compared to the sudden debonding, failure seen in S150-WI-DN. Due to the highly sensitive nature of the results, further studies, with repeats, should be undertaken to confirm the findings of this study.

![Figure 5.3: Average strength gain by strengthening type](image-url)
5.3 Effect of Damage

The peak loads attained by each damaged specimen, relative to its undamaged counterpart, grouped damage level are plotted in Figure 5.4. For the NSM groups, S150-RE-D2 and S150-RM-D1 exhibited the highest peak loads. These findings confirm the findings reported by Dias and Barrows (2012b), that there is no correlation between the peak load and the damage level for NSM FRP systems. For the FRCM groups, the first damage level (1St shear crack) did not affect the strengthening effectiveness. However, for the FRCM-C group, damage level two (70% of the maximum predicted load) reduced the systems effectiveness to 88%, relative to the undamaged and damage level 1 beams. This indicates that both NSM and FRCM systems are fully effective for repair applications with a low level of damage. For high damage levels, NSM bar-based repair systems are fully effective. Whereas, continuous FRCM wraps are still functional, only at a reduced rate of effectiveness, compared to the undamaged and low damage level.

In the literature (Dias & Barros, 2012b), it was reported that the repaired specimens typically exhibit lower stiffness levels than during the initial loading phase. In order to verify these findings, the ratio of the original undamaged stiffness to the repaired stiffness were plotted (Figure 5.5). With the exception of Beam S150-RM-D1, the pre and post damage stiffness values were within 5% of one another, in contradiction with trends described by Dias and Barrows (2012b). This deviation was likely due to a difference in the specimen geometry between the two studies. Dias and Barrows (2012b) used specimens with a shear span of over 1m (a/d ratio of 3.3), compared with the shear span of 500mm (a/d ratio of 2.3) used in this study. As beam stiffness is heavily influenced by the flexural behavior of the beam, the longer spans used by Dias and Barrows were likely responsible for the stiffness changes observed. The loss of stiffness in Beam S150-RM-D1 was due to the high initial levels of stiffness. The pre-damage stiffness of the beam was over 30%
(std. dev. 2.8) higher than the other unstrengthened beams. When the post damage stiffness for that beam was compared to the rest of the repaired stiffness’s (Figure 4.12), Beam S150-RM-D1, exhibits a stiffness value within 1% of the control specimen and 4% of the average stiffness of the repair group. With this in mind it would appear that the initial stiffness of Beam S150-RM-D1 was an anomaly, which was caused by an error in the readings from the LINPOT.

With respect to the FRCM groups, the repairs were able to preserve the beams initial stiffness. Stiffness is more significantly impacted by the flexural strengthening, which runs parallel to the beam’s longitudinal axis, than by the shear strengthening, which runs perpendicular to the beam’s longitudinal axis.

Figure 5.4: Relative peak load grouped by damage level
Figure 5.5: Ratio of post to pre damage stiffness values

5.4 Design Recommendations

5.4.1 Strut and Tie Model

To make recommendations on the maximum allowable strains, several models were employed. To predict the shear capacity of the unstrengthened beams, CSA A23.3-14 section method and the combined strut and tie model described in Garay and Lubel (2008) were used.

The CSA A23.3-14 predictions were based on the modified compression field theory. The combined strut and tie model, which was originally proposed by Foster and Gilbert (1998), is a combination of two simpler strut and tie models. The first model is the direct strut and tie model. In this model, the compression force is transferred directly from the point load to the support via a compressive strut, whereas the tensile forces are transferred solely by flexural reinforcement (Figure 5.6a). The direct strut and tie model is most applicable for low a/d ratios (<1). The second model is the indirect strut and tie model. In this model, the forces are transferred from the point load to the support, via a series of diagonal concrete struts, while stirrups act as vertical ties, and
the flexural reinforcement acts as a horizontal tie. In this model, no force is transferred directly from the point load to the support (Figure 5.6b). The indirect strut and tie model is most applicable for high a/d ratios (>5). The combined strut and tie model uses the base assumptions of the indirect model, however, it is assumed that once the vertical ties (stirrups) have reached their capacity, failure does not instantly occur. Instead a second direct strut, as in the direct model, then forms, and the beam continues to bear load until a secondary failure condition is satisfied. The combined strut and tie model is shown in Figure 5.6c. This strut and tie model was selected as it is suitable for intermediate a/d ratios (1<a/d<5).

To predict the strength gain of the strengthening systems, CSA S806-12 was employed, with several assumptions. In CSA S806-12, only beams with depths greater than 300mm are to be strengthened, however, an exception was made for this study. Although mortar adhered NSM FRP bars are not covered in CSA S806-12, it was assumed that the provisions for epoxy mounted bars would be suitable. Additionally, the FRCM wrapping system were analyzed using CSA S806-12, even though it is only intended for FRP laminates. The ultimate capacity of the FRCM wrap was taken as the value supplied by the manufacturer. In this study, CSA A23.3-14 was used to estimate the dimensions and capacities of truss components, however, limitations on the strut angle were relaxed, as summarized in Table 5.2. Sample calculations for the strut and tie model can be found in APPENDIX A.

![Figure 5.6: Strut and tie models (combined used for analysis)](image-url)
For the control specimens, the CSA A23.3-14 section method predictions proved to be highly conservative, due to the low a/d ratio of the specimens. Experimental values were 69% higher than the values predicted by CSA A23.3-14. The combined strut and tie model has demonstrated to be fairly accurate for the prediction of the unstrengthened beam performance (Average $P_{\text{exp}}/P_{\text{pred}}$ of 1.01), as shown in Table 5.3.

When combined with CSA-A23.3-14, the CSA S806-12 predictions were once again highly conservative, due to the limitations of CSA A23.314 relating to a/d ratio (experimental values between 62% and 37% higher than the predicted values). For the strut and tie model, the contributions of the strengthening materials predicted by CSA S806-12 were directly applied to the vertical ties. This yielded fairly accurate, although somewhat conservative predictions of the ultimate capacity of the epoxy mounted NSM FRP bar strengthened beams (Average $P_{\text{exp}}/P_{\text{pred}}$ of 1.09) (Table 5.3). For the remaining groups, the strut and tie predictions were non-conservative (Average $P_{\text{exp}}/P_{\text{pred}}$ between 0.94 and 0.88) (Table 5.3). For the FRCM wrapped beams, it was assumed that the FRCM wrap would be fully engaged, which was not the case, due to the debonding failures observed. For the NSM-M group, design strain values applicable to epoxy mounted NSM FRP bars were used. However, prior research has indicated the bond between

<table>
<thead>
<tr>
<th>Member ID</th>
<th>Material</th>
<th>Angle (°)</th>
<th>Depth 1 (mm)</th>
<th>Depth 2 (mm)</th>
<th>Width (mm)</th>
<th>Area 1 (mm$^2$)</th>
<th>Area 2 (mm$^2$)</th>
<th>Capacity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB</td>
<td>Concrete</td>
<td>21.6</td>
<td>106</td>
<td>73.5</td>
<td>200</td>
<td>21200</td>
<td>14700</td>
<td>9.4</td>
</tr>
<tr>
<td>AD</td>
<td>Concrete</td>
<td>38.8</td>
<td>120.9</td>
<td>93.6</td>
<td>200</td>
<td>24220</td>
<td>22200</td>
<td>23.5</td>
</tr>
<tr>
<td>AC</td>
<td>Steel</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>256</td>
<td>256</td>
<td>361.2</td>
</tr>
<tr>
<td>DB</td>
<td>Concrete</td>
<td>0</td>
<td>40.1</td>
<td>40.1</td>
<td>200</td>
<td>8020</td>
<td>8020</td>
<td>38.3</td>
</tr>
<tr>
<td>DC</td>
<td>Steel</td>
<td>90</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1000</td>
<td>1000</td>
<td>472</td>
</tr>
<tr>
<td>CB</td>
<td>Concrete</td>
<td>38.8</td>
<td>186.5</td>
<td>186.5</td>
<td>200</td>
<td>37304</td>
<td>37304</td>
<td>23.5</td>
</tr>
<tr>
<td>CE</td>
<td>Steel</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1000</td>
<td>1000</td>
<td>472</td>
</tr>
</tbody>
</table>

Table 5.2: Strut and tie member dimensions and capacities
mortar and concrete is worse than that of epoxy and concrete (Nordin & Taljsten, 2003; Al-Mahmoud, Castel, Francois, & Tourneur, 2011). These faulty assumptions lead to the non-conservative results.

Table 5.3: Experimental vs predicted results

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$P_{\text{exp}}$ (kN)</th>
<th>$P_{\text{pred}}$ (kN)</th>
<th>$P_{\text{exp}}/P_{\text{pred}}$</th>
<th>Combined Strut and Tie</th>
</tr>
</thead>
<tbody>
<tr>
<td>S200-WN-DN</td>
<td>224.5</td>
<td>124.9</td>
<td>1.80</td>
<td>220.2</td>
</tr>
<tr>
<td>S150-WN-DN</td>
<td>217.1</td>
<td>138.5</td>
<td>1.57</td>
<td>220.2</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1.69</td>
<td>1.01</td>
<td></td>
</tr>
<tr>
<td>S150-RE-DN</td>
<td>307.1</td>
<td>189.9</td>
<td>1.62</td>
<td>281.2</td>
</tr>
<tr>
<td>S150-RE-D1</td>
<td>293.8</td>
<td>189.9</td>
<td>1.55</td>
<td>281.2</td>
</tr>
<tr>
<td>S150-RE-D2</td>
<td>323.1</td>
<td>189.9</td>
<td>1.70</td>
<td>281.2</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1.62</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>S150-RM-DN</td>
<td>212.6</td>
<td>189.9</td>
<td>1.12</td>
<td>281.2</td>
</tr>
<tr>
<td>S150-RM-D1</td>
<td>309.7</td>
<td>189.9</td>
<td>1.63</td>
<td>281.2</td>
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<tr>
<td>S150-RM-D2</td>
<td>271.1</td>
<td>189.9</td>
<td>1.43</td>
<td>281.2</td>
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<tr>
<td>Average</td>
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<td>1.39</td>
<td>0.94</td>
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</tr>
<tr>
<td>S150-WF-DN</td>
<td>304</td>
<td>221.6</td>
<td>1.37</td>
<td>337.1</td>
</tr>
<tr>
<td>S150-WF-D1</td>
<td>311.2</td>
<td>221.6</td>
<td>1.40</td>
<td>337.1</td>
</tr>
<tr>
<td>S150-WF-D2</td>
<td>273.7</td>
<td>221.6</td>
<td>1.24</td>
<td>337.1</td>
</tr>
<tr>
<td>S200-WF-DN</td>
<td>302</td>
<td>208</td>
<td>1.45</td>
<td>337.1</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1.37</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>S150-WI-DN</td>
<td>254.9</td>
<td>180</td>
<td>1.42</td>
<td>280.6</td>
</tr>
<tr>
<td>S150-WI-D1</td>
<td>285.5</td>
<td>180</td>
<td>1.59</td>
<td>280.6</td>
</tr>
<tr>
<td>S200-WI-DN</td>
<td>227.2</td>
<td>167</td>
<td>1.36</td>
<td>280.6</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1.46</td>
<td>0.91</td>
<td></td>
</tr>
</tbody>
</table>

5.4.2 Maximum Recommended Strains

The next step was to refine the models of CSA S806-12 for the FRCM and NSM groups, based on a maximum allowable strain in the material. In order to account for the bi-linear stress-strain curve of the FRCM, a regression line was drawn from the ultimate load following the post cracking stiffness to the Y-axis, this process is illustrated in Figure 5.7. The material was considered to
experience no strain before cracking, as the uncracked stiffness is typically over 10 times that of the uncracked stiffness (ACI 549.4R, 2013). The formula used for these calculations is included in APPENDIX A.

![Stress-Strain Curve](image)

**Figure 5.7: Approximated FRCM Stress-Strain Curve**

The average peak load for each specimen was then used with the strut and tie model, in order to determine the maximum allowable strains in each strengthening material (Table 5.4).

<table>
<thead>
<tr>
<th>Strengthening System</th>
<th>Maximum Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy</td>
<td>0.011</td>
</tr>
<tr>
<td>Mortar</td>
<td>0.006</td>
</tr>
<tr>
<td>Continuous</td>
<td>0.005</td>
</tr>
<tr>
<td>Intermittent</td>
<td>0.009</td>
</tr>
</tbody>
</table>

From this analysis several observations can be drawn. The maximum recommended strain for the NSM-E group was found to be greater than that recommended by CSA S806-12 (0.007). This indicates that the design code is conservative and safe for public use. Additionally, it may be possible to expand CSA S806-12 to include beams with depths as low as 265mm, however further research should be done. The maximum allowable strain for the NSM-M group was found to be
0.006. Which is lower than the maximum allowable strain for NSM bars in CSA S806-12. This is logical as CSA S806-12 does not allow for the use of mortar adhered NSM FRP bars. Additionally, it has been reported that mortar adhered NSM FRP bars have a lower bond capacity than epoxy mounted (Nordin & Taljsten, 2003; Al-Mahmoud, Castel, Francois, & Tourneur, 2011).

In the FRCM group, it was found that the maximum allowable design strain for the FRCM-I group was 80% higher than that of the FRCM-C group. This result was unexpected, based on the existing knowledge of FRP bond behavior. For externally applied FRP laminates, intermittent strips have been found to have less effective bond than that of continuous wrapping (Subramaniam, Carloni, & Nobile, 2007; CSA S806-12, 2012; ACI 440.1R-15, 2015).
CHAPTER 6: CONCLUSIONS AND FUTURE RECOMMENDATIONS

6.1 Summary and Conclusions

The purpose of this research effort was to determine the shear performance of damaged RC beams repaired using FRCM wrapping and NSM FRP bars. Fifteen half scale beams (200x265x2000 mm) were cast, damaged, repaired, and tested in monotonic three-point bending conditions. The experimental parameters investigated were level of damage, type of adhesive used, and wrapping pattern. Load, stiffness, mid-span displacement, pseudo-ductility, energy absorption, and failure mode were used as performance indicators. Based on the discussion presented, the following conclusions were drawn:

- All the specimens except for S150-RE-D2 failed due to diagonal tension in the shear critical section of the beam. In all the strengthened and repaired beams, except S150-RE-D2, failure was preceded by debonding of the strengthening systems. Beam S150-RE-D2 failed due to combined flexure and shear.

- All the strengthening systems were found to be effective in increasing the beams’ shear capacity (average values ranged from 22% to 42%).

- All the strengthening systems were able to restore the beams’ stiffnesses to pre-damage levels.

- The NSM FRP bar strengthening systems were equally effective regardless of the damage levels.
• The FRCM wrapped beams were equally effective for the undamaged and damage level 1 stated, however, the second damage level reduced the capacity of the continuously FRCM wrapped beams by 12%.

• The epoxy adhesive was found to be more effective than the mortar in every area investigated (16% higher peak load and 10% higher energy absorption).

• Continuous FRCM wrapping provided superior overall strengthening (20% higher) and better energy absorption (160% higher) compared to the intermittent FRCM strips.

• All the strengthening systems were effective in increasing the beams’ ductility. The continuously wrapped system was the most effective with an increase of 58%, both of the NSM FRP bar systems showed a similar increase of approximately 50%, finally the intermittently FRCM wrapped system was the least effective only exhibiting a 20% increase.

• Based on the presented strut and tie analysis, the maximum recommended strains of CSA S806-12 were found to provide conservative predictions for strengthening effectiveness of epoxy mounted NSM FRP bars.

• Based on the presented strut and tie analysis, the maximum allowable strain was found to be 0.006 for mortar adhered NSM FRP, 0.005 for continuous FRCM U-wrap and 0.009 for intermittent FRCM U-wrap.

The results represent an encouraging beginning of the use of FRCM and NSM FRP bars for the repair of shear damaged RC structures. However, additional parameters should be investigated to better understand the performance of both repair systems.
6.2 Research Contribution

This study represents a significant contribution to the use of FRCM and NSM FRP bars as a repair and strengthening material. This study represents the first experimental work where mortar mounted FRP bars were used for shear strengthening of damaged RC. Additionally, the study was the first time the repair of shear damaged RC beams using FRCM has been studied. The design recommendations presented in this study represent the first step towards the codification of mortar adhered NSM FRP and FRCM wraps. The use of Canadian materials and construction methods is vital in the development of Canadian design Codes and guidelines. This will allow for mortar mounted NSM FRP bars and FRCM wrap systems to be utilized by the Canadian civil engineering industry.

6.3 Limitations

Although this study was a success, there are several drawbacks which should be noted. The issues regarding the fixed a/d ratio, when using the 500 kN actuator, effectively nullified the effects of 200mm stirrup spacing. If the original test setup could have been used, this parameter would have been properly explored. A higher level of damage, such as 90% of the maximum load, or even repair of post failure specimens, would have likely been more effective for investigating the effects of damage on the strengthening systems. Repeats of the specimens would have been very helpful for confirming the trends observed. Data acquisition could have been improved if more strain gauges had been used, to provide redundancy for those which failed. This study only focused on the mechanical aspects of RC repair using NSM FRP bars and FRCM wrapping. However, to promote the applicability of these materials to the civil engineering industry, the costs of the strengthening systems should also be analysed, to determine their potential for future use.
Additionally, the environmental impacts of each system should be considered as climate change is becoming an increasingly important consideration.

### 6.4 Future Research

To help ensure the adoption of FRCM wraps and mortar mounted NSM FRP bars for RC repair significant future research is still required. This research should primarily focus on repeating existing tests, to confirm their findings, as well as, to develop enough data for rigorous statistical analysis. This will allow material factors, for load resistance factor design (LRFD), to be developed for FRCM wraps and mortar NSM. The development of these factors will be critical to incorporating the materials into existing design codes. As well, economic, and environmental studies should be undertaken to identify areas where FRCM and mortar NSM systems could outperform existing alternatives, to promote the use of the new materials. Several areas where future research is recommended include:

- Development of numerical models to verify the findings of this, and other, studies
- Investigating the effect of damage on FRCM with different materials used in the fabric
- Exploring the use of FRCM in seismic strengthening
- Investigate the effects of different damage mechanisms such as fire damage or corrosion
- Cost analysis of the various strengthening systems
- Environmental impact studies regarding the various systems
REFERENCES


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APPENDICES
APPENDIX A: SAMPLE CALCULATIONS

A  Area of strengthening material
\(d_v\)  Effective shear depth
E  Modulus of elasticity
F  Force in member
\(f_{sy}\)  Stress in member
\(f_{FRCM}\)  Strength of FRCM, as specified by manufacturer
R  Support reaction force
V  Shear resistance provided by strengthening material
\(\varepsilon\)  Strain in Material
\(\varepsilon_{FU}\)  Ultimate strain in material
\(\theta\)  Expected shear crack angle
\(\theta_{XZY}\)  Angle of compressive strut
\(\phi\)  Material resistance factor

Calculation of shear resistance of strengthening materials

\[ V_F = \frac{\phi_F A_F E_F \varepsilon_{FU} d_v \cot \theta}{S_F} \]

NSM FRP bars

\(\varepsilon_{FU} = 0.007 \) (CSA S806-12, 2012)

\[ A_F = 2 \text{ (number of legs)} \times 31.7\text{mm}^2 \text{ (bar area)} = 63.4\text{mm}^2 \]

\[ E_F = 47500\text{MPa} \] (Akel, 2016)

\[ d_v = \max \left\{ 0.72 \times 265\text{mm (height of beam)} = 191\text{mm}, \quad 0.9 \times 221.5\text{mm (depth of longitudinal steel)} = 199\text{mm} \right\} \]
\( \phi_F = 1 \)

\( \theta = 35° \) (CSA A23.3-14, 2014)

\( S_F = 150 \text{mm} \) (NSM spacing)

\[
V_F = \frac{1 \times 64 \text{mm}^2 \times 47500 \text{MPa} \times 0.007 \times 199 \text{mm} \times \cot(35°)}{150 \text{mm}} \times \frac{1 \text{kN}}{1000 \text{N}} = 40.0 \text{kN}
\]

FRCM Wrap

\[
V_{FRCM} = \frac{\phi_{FRCM} A_{FRCM} f_{FRCM} d_v \cot \theta}{S_{FRCM}}
\]

\( \phi_{FRCM} = 1 \)

\( A_{FRCM-C} = 2 \) (both sides, 1 layer) \( \times \) 157 \text{mm}^2/\text{m} \) (fabric area per meter width)

\( = 314 \text{mm}^2/\text{m} \)

\( A_{FRCM-I} = 2 \) (both sides, 1 layer) \( \times \) 157 \text{mm}^2/\text{m} \) (fabric area per meter width)

\( \times 0.075 \text{mm} \) (strip width) = 23.6 \text{mm}^2

\( f_{FRCM} = 885 \text{MPa} \) (Simpson Strong-Tie, 2019)

\( d_v = \max \left\{ \begin{array}{l} 0.72 \times 265 \text{mm} \text{ (height of beam)} = 191 \text{mm} \\ 0.9 \times 221.5 \text{mm} \text{ (depth of longitudinal steel)} = 199 \text{mm} \rightarrow \text{goes first} \end{array} \right\} \)

\( \theta = 35° \) (CSA A23.3-14, 2014)

\( S_{FRCM} = \begin{cases} 1 \text{m} \text{ (FRCM - C unit width)} \\ 150 \text{mm} \text{ (FRCM - I spacing)} \end{cases} \)
\[ V_{FRCM-C} = \frac{1 \times 314 \text{ mm}^2 / \text{m} \times 885 \times 199 \text{ mm} \times \cot(35^\circ)}{1 \text{ m}} \times \frac{1 \text{kN}}{1000 \text{N}} = 79.0 \text{kN} \]

\[ V_{FRCM-I} = \frac{1 \times 23.6 \text{ mm}^2 \times 885 \text{ MPa} \times 199 \text{ mm} \times \cot(35^\circ)}{150 \text{ mm}} \times \frac{1 \text{kN}}{1000 \text{N}} = 39.5 \text{kN} \]
Strut and Tie Model

Figure A.1: Strut and Tie Model (lengths in mm)

Indirect Strut

Member dimensions taken from Table 5.2, calculated using CSA A23.3-14.

\[ F_{DC} = 360\text{MPa} \times 32 \times 8 = 92\text{kN} \]

\[ F_{AD} = \frac{92.2\text{kN}}{\sin(38.8^\circ)} = 147\text{kN} \]

\[ F_{AC} = \frac{92.2\text{kN}}{\tan(38.8^\circ)} = 114\text{kN} \]

\[ F_{DB} = 147\text{kN} \times \cos(38.8^\circ) = 114\text{kN} \]

\[ f_{AD, bottom} = \frac{147\text{kN}}{24220\text{mm}^2} \times \frac{1000\text{N}}{1\text{kN}} = 6.1\text{MPa} \]
\[ f_{AD\ top} = \frac{147kN}{22200mm^2} \times \frac{1000N}{1kN} = 6.6MPa \]

\[ f_{DB} = \frac{114kN}{8020mm^2} = 14.2MPa \]

\[ R_A = F_{DC} = 92kN \]

**Direct Strut**

Indirect strut was analyzed using iterations based on maximum stress in Member_{AB}, final iteration is presented here

**Trial Force in Member AB = 164kN**

\[ f_{AB\ bottom} = \frac{164kN}{21200mm^2} \times \frac{1000N}{1kN} = 7.7MPa \]

\[ f_{AB\ top} = \frac{164kN}{16660mm^2} \times \frac{1000N}{1kN} = 9.8MPa \]

\[ R_A = 180kN \times \sin(21.1^\circ) = 59.2kN \]

\[ F_{AC} = \frac{65kN}{\tan(21.1^\circ)} = 153kN \]

**Combine using superposition and checks**

\[ R_a = 59kN + 92kN = 151kN \]

\[ F_{AC} = 153kN + 114kN = 267kN \]

\[ f_{AC} = \frac{282kN}{1000mm^2} \times \frac{1000N}{1kN} = 267MPa \]
267MPa < 472 \therefore \text{Member}_{AC} \text{ has not yeilded}

\begin{align*}
\varepsilon_{AC} &= \frac{f_{AC} \times \varepsilon_y}{f_y} \\
\varepsilon_{AC} &= \frac{267 \times 0.02}{472} = 0.0011
\end{align*}

Check Member AD

\begin{align*}
\varepsilon_1 &= \varepsilon_{AC} + (\varepsilon_{AC} + 0.02) \times \cot (\theta_{DAC})^2 \\
\varepsilon_1 &= 0.0011 + (0.0011 + 0.002) \times \cot(21.1)^2 = 0.006
\end{align*}

\begin{align*}
f_{c \text{ max}} &= \frac{f'c}{(0.8 + 170 \times \varepsilon_1)} \\
f_{c \text{ max}} &= \frac{45\text{MPa}}{(0.8 + 170 \times 0.006)} = 25\text{MPa}
\end{align*}

bottom: 6.1MPa < 25MPa OK
top: 6.6MPa < 25MPa OK

Check Member AB

\begin{align*}
\varepsilon_1 &= 0.0011 + (0.0011 + 0.002) \times \cot(38.8)^2 = 0.022
\end{align*}

\begin{align*}
f_{c \text{ max}} &= \frac{45\text{MPa}}{(0.8 + 170 \times 0.022)} = 9.8\text{MPa}
\end{align*}

bottom: 7.7MPa < 9.8MPa OK
top: 9.8MPa = 9.8MPa OK
Check Member DB

\[ f_{c_{\text{max}}} = 0.85 \times f'_{c} \]

\[ f_{c_{\text{max}}} = 0.85 \times 45\text{MPa} = 38.3\text{MPa} \]

\[ 14.2\text{MPa} < 38.3\text{MPa OK} \]

Using strengthening materials with strut and tie

Shear contribution of materials calculated using CSA S806-12 were directly applied to Member\text{DC}.

For epoxy NSM

\[ F_{\text{DC}} = 92kN + 40kN = 132kN \]

Calculating stress in FRCM

\[ f_{\text{FRCM}} = 346 + 49000 \times \varepsilon_{\text{FRCM}} \]
APPENDIX B: DAMAGE LOAD-STEEL STRAIN CURVES

Figure B.1: Load vs strain for rebar stirrup during damage to Beam S150-RE-D1

Figure B.2: Load vs strain for rebar stirrup during damage to Beam S150-RE-D2
Figure B.3: Load vs strain for rebar stirrup during damage to Beam S150-RM-D1

Figure B.4: Load vs strain for rebar stirrup during damage to Beam S150-RM-D2
Figure B.5: Load vs strain for rebar stirrup during damage to Beam S150-WF-D1

Figure B.6: Load vs strain for rebar stirrup during damage to Beam S150-WF-D2
Figure B.7: Load vs strain for rebar stirrup during damage to Beam S150-W1-D1
APPENDIX C: LOAD-STRAIN CURVES

For all strengthening materials, a numbering scheme from 1-3 was incorporated with 1 being closest to the support and 3 closest the point load. Rebar strain refers to strain in steel stirrup in middle of shear critical region.

Figure C.1: Load-strain curves Beam S150-WN-DN

Figure C.2: Load-strain curves Beam S200-WN-DN
Figure C.3: Load-strain curves Beam S150-RE-DN

Figure C.4: Load-strain curves Beam S150-RE-D1
Figure C.5: Load-strain curves Beam S150-RE-D2

Figure C.6: Load-strain curves Beam S150-RM-DN

Note: Rebar strain gauge failed prior to testing
Figure C.7: Load-strain curves Beam S150-RM-D1

Notes: Concrete strain gauge failed during test

Figure C.8: Load-strain curves Beam S150-RM-D2

Notes: Specimen contained 2 working steel strain gauges, Strengthening 1 malfunctioned prior to testing
Figure C.9: Load-strain curves Beam S150-WF-DN

Figure C.10: Load-strain curves Beam S150-WF-D1
Figure C.11: Load-strain curves Beam S150-WF-D2

Figure C.12: Load-strain curves Beam S200-WF-DN

Notes: Steel strain gauge malfunctioned prior to testing, No concrete strain gauge
Figure C.13: Load-strain curves Beam S150-WI-DN

Figure C.14: Load-strain curves Beam S150-WI-D1
Figure C.15: Load-strain curves Beam S200-WI-DN

Notes: Concrete Strain gauge failed during test
APPENDIX D: LOAD-DEFLECTION CURVES

Figure D.1: Load-displacement curve Beam S150-WN-DN

Figure D.2: Load-displacement curve Beam S200-WN-DN
Figure D.3: Load-displacement curve Beam S150-RE-DN

Figure D.4: Load-displacement curve Beam S150-RE-D1
Figure D.5: Load-displacement curve Beam S150-RE-D2

Figure D.6: Load-displacement curve Beam S150-RM-DN
Figure D.7: Load-displacement curve Beam S150-RM-D1

Figure D.8: Load-displacement curve Beam S150-RM-D2
Figure D.9: Load-displacement curve Beam S150-WF-DN

Figure D.10: Load-displacement curve Beam S150-WF-D1
Figure D.11: Load-displacement curve Beam S150-WF-D2

Figure D.12: Load-displacement curve Beam S200-WF-DN
Figure D.13: Load-displacement curve Beam S150-WI-DN

Figure D.14: Load-displacement curve Beam S150-WI-D1
Figure D.15: Load-displacement curve Beam S200-WI-DN