1-1-2013

Experimental Investigation on Pull-out Strength of Pre- and Post-Installed GFRP bars for Bridge Barrier Construction

Alireza Abolghasem
Ryerson University

Follow this and additional works at: <http://digitalcommons.ryerson.ca/dissertations>

Part of the [Civil Engineering Commons](#)

Recommended Citation

Abolghasem, Alireza, "Experimental Investigation on Pull-out Strength of Pre- and Post-Installed GFRP bars for Bridge Barrier Construction" (2013). *Theses and dissertations*. Paper 1923.

This Thesis is brought to you for free and open access by Digital Commons @ Ryerson. It has been accepted for inclusion in Theses and dissertations by an authorized administrator of Digital Commons @ Ryerson. For more information, please contact bcameron@ryerson.ca.

**EXPERIMENTAL INVESTIGATION ON PULL-OUT STRENGTH OF PRE- AND
POST-INSTALLED GFRP BARS FOR BRIDGE BARRIER CONSTRUCTION**

BY

Alireza Abolghasem,

B.Eng. EIT, Ryerson University, Toronto, ON, 2011

A Thesis

Presented to Ryerson University

In partial fulfillment of the
Requirement for the degree of
Master of Applied Science
In the program of
Civil Engineering

Toronto, Ontario, Canada, 2013

© Alireza Abolghasem 2013

AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I authorize Ryerson University to lend this thesis to other institutions or individuals for the purpose of scholarly research.

I further authorize Ryerson University to reproduce this thesis by photocopying or by other means, in total or in part, at the request of other institutions or individuals for the purpose of scholarly research.

I understand that my thesis may be made electronically available to the public.

Experimental Investigation on Pull-out Strength of Pre- and Post-Installed GFRP bars for Bridge Barrier Construction

Alireza Abolghasem

Master of Applied Science, Civil Engineering

RYERSON UNIVERSITY, Toronto, Canada, 2013

ABSTRACT

This study aims at providing research information on the pullout capacity of straight and headed-end GFRP bars embedded in bridge deck slab as pre-installed anchors for new bridge barrier-deck construction and as post-installed anchors for the replacement of deteriorated bridge barriers. In Phase I of this research, the pullout capacity of pre-installed ribbed-surface GFRP bars in twin- and triple-bar groups were investigated experimentally, considering different bar size, spacing and embedment depths. In phase II of this research, the pullout capacity of post-installed sand-coated and ribbed-surface GFRP bars were investigated experimentally considering different bar size and embedment depths and adhesive types. The experimental findings were compared with available experimental results and pullout formulas. Design values for the pullout strength of the GFRP bars as pre- and post-installed anchors in concrete were deduced.

ACKNOWLEDGEMENTS

The author would like to express his gratitude sincerely to his supervisor Dr. Khaled Sennah for giving this amazing research opportunity. This study could not be accomplished without his continuous guidance, suggestions and encouragements. I must also thank my co-supervisor Dr. Hossein Azimi, for supporting, encouraging and directing me through this journey. I have been blessed to have such a knowledgeable and supportive co-supervisor. I would also like to thank the laboratory technical officer Nidal Jaalouk and technician Min Yao for their assistance in performing the experimental tests.

The financial support of Ministry of Transportation Ontario (MTO) in the form of research grant and site use is greatly appreciated. In addition, the author would like to thank Pultrall Inc., Schock Canada Inc., Hilti Inc. and Sika Canada for sponsoring this project.

DEDICATION

To my wife and my parents for their endless love, support and encouragement

Table of Contents

Chapter 1 Introduction.....	1
1.1 Background.....	1
1.2 Problem Statement.....	3
1.3 Objectives	5
1.4 Thesis Overview	6
Chapter 2 Literature Review	7
2.1 Fiber Reinforced Polymers Technology	7
2.1.1 Resins.....	8
2.1.2 Fibers.....	9
2.1.3 Manufacturing Process.....	10
2.1.4 FRP Properties	12
2.1.5 Application of FRP in Construction.....	14
2.2 Bond Mechanism	16
2.2.1 Failure Modes of Bond	17
2.3 Parameters influencing bond behavior of FRP bars in concrete	20
2.3.1 Bar Properties.....	21
2.3.2 Structural Characteristics	23
2.3.3 Concrete Properties.....	26
2.4 Types of anchorage systems	28
2.5 Types of Structural adhesives	30
2.6 Mechanism of load transfer for adhesive anchors.....	30
2.7 Bond models for adhesive anchors	33
2.7.1 Concrete Cone Model	33
2.7.2 Bond Model	35
2.7.3 Combined Concrete Cone and Bond Models.....	36
2.7.4 Anchor Rupture Models.....	37
Chapter 3 Experimental Program.....	38
3.1 General.....	38
3.2 Phase I: Group Anchorage	38

3.2.1	Properties of GFRP rebars used	39
3.2.2	Properties of concrete.....	41
3.2.3	Specimen Preparation	41
3.2.4	Specimen Nomenclature	45
3.2.5	Test Setup and Procedure.....	45
3.3	Phase II: Post installed Anchorage.....	54
3.3.1	Properties of GFRP bars used	54
3.3.2	Properties of concrete.....	55
3.3.3	Properties of adhesives used	55
3.3.4	Specimen Preparation	56
3.3.5	Test Setup and Procedure.....	63
3.3.6	Specimen Nomenclature	65
Chapter 4	Experimental results and discussion of Phase I	66
4.1	Modes of failures	66
4.2	Experimental results and discussion	68
4.3	Comparison of experimental results with available bond strength equations.....	80
4.4	Summary.....	88
Chapter 5	Experimental results and discussion of Phase II.....	90
5.1	Mode of failure	90
5.2	Experimental results and discussions.....	92
5.3	Comparison of Experimental Results with other similar experiment	108
5.4	Comparison of experimental results with current design method.....	108
5.5	Summary	112
Chapter 6	Conclusions and Recommendations for Future Research.....	115
6.1	General.....	115
6.2	Phase 1 – Pre-installed GFRP bar group anchorage	115
6.3	Phase 2 – Post-installed anchorage	117
6.4	Recommendations for Future Research	119
Appendix A	Test results for pre-installed #4 straight ribbed bar group phase I	121
Appendix B	Test results for pre-installed #5 straight ribbed bar group phase I	150
Appendix C	Test results for post-installed GFRP bars in Phase II	180
Appendix D	Core Samples for pre and post-installed GFRP bar.....	204

Chapter 7 Bibliography	218
-------------------------------------	------------

List of Figures

Figure 1-1: Severely corroded steel reinforcement in bridge barrier resulting in concrete spelling.....	2
Figure 1-2: PL-3 GFRP-reinforced bridge barrier for new construction.	4
Figure 1-3: PL-2 GFRP-reinforced bridge barrier with post-installed anchors for barrier replacement	4
Figure 2-1: FRP products for reinforcement of concrete structures (ISIS Canada, 2006).....	8
Figure 2-2: Schematic diagram of pultrusion manufacturing procedures.....	11
Figure 2-3: Schematic showing the filament winding manufacturing process (ISIS Canada, 2006).	12
Figure 2-4: Average stress-strain diagram of FRP.....	13
Figure 2-5: Stress-strain diagram of fibers, matrix and FRP.	13
Figure 2-6: Commercially available FRP rebars:	15
Figure 2-7: Bond transfer mechanism (ACI 408R, 2003).....	16
Figure 2-8: Cracking mechanism in bond.....	18
Figure 2-9: Bond stress-slip envelope (Harajli et al. 2004)	20
Figure 2-10: Steel bar behavior in concrete under tension:	27
Figure 2-11: Different types of anchors (El Menoufy, 2010).	29
Figure 2-12: Different Mechanism of Load transfer of adhesive anchors (Eligenhausen et al. 2007).	31
Figure 2-13: Post installed adhesive anchors failure modes (Cook and Konz 2001).....	32
Figure 3-1: GFRP bars used in this study	40
Figure 3-2: View of phase I specimen preparation	42
Figure 3-3: Views of double and triples bars after concrete hardening	44
Figure 3-4: Schematic diagram of the double bars test setup	47
Figure 3-5: Schematic diagram of the triple-bars test setup	47
Figure 3-6: Views of the sequence of assembly of the test setup for double bar testing	49
Figure 3-7: Views of the sequence of assembly of the test setup for triple bar testing.....	50
Figure 3-8: Views of the grips assembly used in this study.....	51
Figure 3-9: Schematic diagram of grip assembly.	52
Figure 3-10: Views of the data acquisition system, load cells and potentiometers used in this study.....	53
Figure 3-11: Views of the drilling holes in the concrete slab	57
Figure 3-12: Views of hole cleaning in the concrete slab	59
Figure 3-13: GFRP bars preparation procedures	60
Figure 3-14: Installation procedure GFRP bars	62
Figure 3-15: Schematic diagram of the post-installed bars test setup.....	63
Figure 3-16: Views of the sequence of assembly of the test setup for Post-installed pullout testing	64
Figure 4-1: Various types of observed pullout modes of failure for pre-installed GFRP bars	67
Figure 4-2 Samples of mode of failure observed in specimen:.....	67
Figure 4-3: View of split core samples:	68
Figure 4-4: Load-displacement relationship for double-anchorage group presented in Figure 4-2(a)	69
Figure 4-5: Average and characteristic bond stress for # 4 ribbed triple group bar.....	78
Figure 4-6: Average and characteristic bond stress for # 4 ribbed double group bar	79
Figure 4-7: Average and characteristic bond stress # 4 ribbed triple group bar	79
Figure 4-8: Average and characteristic bond stress for # 4 ribbed double group bar	80
Figure 4-9: Comparison of the test results with existing pullout models for #4 ribbed.....	81
Figure 4-10: Comparison of the test results with existing pullout models for #4 ribbed.....	81

Figure 4-11: Comparison of the test results with existing pullout models for #4 ribbed.....	82
Figure 4-12: Comparison of the test results with existing pullout models for #4 ribbed.....	82
Figure 4-13: Average and characteristic ultimate load for # 5 ribbed triple group bar.....	84
Figure 4-14: Average and characteristic ultimate load for # 5 ribbed double group bar	84
Figure 4-15: Average and characteristic ultimate load for # 5 ribbed triple group bars	85
Figure 4-16: Average and characteristic ultimate load for # 5 ribbed double group bar	85
Figure 4-17: Comparison of the test results with existing pullout models for #5 ribbed.....	86
Figure 4-18: Comparison of the test results with existing pullout models for #5 ribbed.....	86
Figure 4-19: Comparison of the test results with existing pullout models for #5 ribbed.....	87
Figure 4-20: Comparison of the test results with existing pullout models for #5 ribbed.....	87
Figure 5-1: Possible failure modes of the post-installed bars	90
Figure 5-2: Recorded failure modes of the post-installed GFRP bars	91
Figure 5-3: Pullout load-slip relationship for #5 sand-coated GFRP bar.....	93
Figure 5-4: Split views of core samples for post-installed sand-coated and ribbed bars	93
Figure 5-5: Comparison of modes of failure for #4 ribbed GFRP bars	94
Figure 5-6: Comparison of modes of failure for #5 ribbed GFRP bars	94
Figure 5-7: Comparison of modes of failure for #5 sand-coated GFRP bars	94
Figure 5-8: Comparison of modes of failure for #6 sand-coated GFRP bars	94
Figure 5-9: Comparison of average ultimate load vs. embedment depth for specimens	102
Figure 5-10: Comparison of average ultimate load vs. embedment depth for specimens	102
Figure 5-11: Comparison of characteristic ultimate load (CSA, 2010) vs. embedment depth	103
Figure 5-12: Comparison of characteristic ultimate load (CSA, 2010) vs. embedment depth	103
Figure 5-13: Comparison of average bond stress vs. embedment depth for specimens	105
Figure 5-14: Comparison of average bond stress vs. embedment depth for specimens	105
Figure 5-15: Comparison of characteristic bond stress (CSA, 2010) vs. embedment depth	106
Figure 5-16: Comparison of characteristic bond stress (CSA, 2010) vs. embedment depth	106
Figure 5-17: Effect of adhesive type on.....	107
Figure 5-18: Effect of adhesive type on.....	107
Figure 5-19: Effect of adhesive type on.....	107
Figure 5-20: Effect of adhesive type on.....	107
Figure 5-21: Comparison of experimental average bond stress with those available.....	109
Figure 5-22: Comparison of average bond stress of post-installed GFRP bars	110
Figure 5-23: Comparison of average bond stress of post-installed GFRP bars	110
Figure 5-24: Comparison of average ultimate load of post-installed GFRP bars	111
Figure 5-25: Comparison of average ultimate load of post-installed GFRP bars	112

List of Tables

Table 2-1: Typical properties of thermosetting resins (ISIS Canada, 2006).....	9
Table 2-2: Comparison between steel and FRP reinforcements used in this project.....	14
Table 2-3: Summary of phases for the splitting and pullout modes of failure.....	19
Table 3-1: Geometrical properties of the ribbed GFRP bars used for phase I.....	39
Table 3-2: Phase I Test matrix	43
Table 3-3: Sand-coated GFRP material Properties (Pultrall Inc., 2012).....	55
Table 3-4: Adhesive Specifications	56
Table 3-5: Phase II Drill Bits sizes	58
Table 3-6: Phase II Test matrix	61
Table 4-1: Test results of triple #4 GFRP group bars with straight end	70
Table 4-2: Test results of double # 4 GFRP group bars with straight end.....	71
Table 4-3: Test results of triple #5 GFRP group bars with headed end	72
Table 4-4: Test results of double # 5 GFRP group bars with headed end.....	73
Table 4-5: Summery of test results for #4 GFRP bars with ribbed surface	76
Table 4-6: Summery of test results for #5 GFRP bar with ribbed surface and headed ends	77
Table 5-1: Test results of #4 ribbed GFRP post-installed bars	95
Table 5-2: Test results of #5 ribbed GFRP post-installed bars	96
Table 5-3: Test results of #5 sand-coated GFRP post-installed bars	97
Table 5-4: Test results of #6 sand-coated GFRP post-installed bars	98
Table 5-5: Pullout test results for post-installed GFRP bars based on average ultimate load	100
Table 5-6: Pullout test results for post-installed GFRP bars based on average bond stress.....	101
Table 5-7: Design pullout strength for post-installed GFRP bars based on pullout test results of specimens	114

Chapter 1 Introduction

1.1 Background

In 2007 The Residential and Civil Construction Alliance of Ontario, Canada (RCCAO, 2007) released a report entitled “Ontario’s Bridges: Bridging the Gap”. The report warns that infrastructure of municipal bridges and public safety are at risk because of maintenance deferral, lack of inspections and government oversight. Severe consequences of delaying rehabilitation or reconstruction of deteriorated bridges and the vital need of safeguarding the public from possible infrastructure failure was highlighted by media on bridge collapses in Laval, Quebec and Minneapolis, Minnesota. It was noted in the study that a considerable percentage of bridges in Ontario were constructed in the 1950s and 1960s, and most of them will be in need for major rehabilitation or complete replacement after 50 years of their service life.

In accordance to the report provided by Provincial Auditor’s in 2004, near 30 percent of provincial bridges (2,800 bridges approximately) under jurisdiction of the Ministry of Transportation of Ontario’s (MTO) are in need of either major rehabilitation or maintenance. The RCCAO report stated that for 12,000 municipal bridges in Ontario, more investigation on their condition is required and a major investment of minimum \$2 billion is estimated to be required to rehabilitate these bridges. Bridges built prior to the 1970’s did not use air-entrained concrete and coated reinforcing steel bars to protect from the effects of freeze-thaw cycles and the application of winter de-icing salt. This leads to corrosion-induced degradation in bridge decks and barriers (see Figure 1-1). Accordingly, bridge decks, railings and barrier walls are all likely candidates for expensive replacement on the majority of these older bridges.

**Figure 1-1: Severely corroded steel reinforcement in bridge barrier resulting in concrete spelling.
(V-ROD CANADA, 2009)**

The RCCAO report presented some recommendations for improvement of the public's safety and sustainability of bridges in Ontario including improvement of bridge design to promote the life expectancy and reduce maintenance costs. One of the excellent alternatives is to use fibre reinforced polymer (FRP) bars in lieu of steel bars to eliminate the corrosion problem. The RCCAO report also recommended seeking accelerated delivery methods to address the mounting infrastructure repair and construction backlog. This can be achieved by utilizing post-installed anchor technology to connect the barrier wall with existing deck to accelerate deteriorated barrier replacement. The proposed research activities will address the above-mentioned issues and concerns.

1.2 Problem Statement

Corrosion of steel reinforcement due to environmental effects is a major cause of deterioration problems in bridge barriers. Glass fibre reinforced polymers (GFRP) not only addresses this durability problem but also provides exceptionally high tensile strength to resist vehicle impact. The sand-coated or ribbed-surface profile of the GFRP bar and end anchorage heads ensure optimal bond between concrete and the bar and eliminate the unnecessary and expensive use of custom made bar bends. The proposed research seeks to develop appropriate connection details between the barrier and the deck slab for new construction and the existing deck slab and the replaced barrier wall in bridge rehabilitation projects. Two different types of anchorage systems exist for concrete including pre-installed (cast in place) and post-installed anchors. Figure 1-2 shows pre-installed anchors used for new construction which are typically composed of GFRP bars with either straight ends, bents or headed ends. Literature review revealed that the pullout capacity of GFRP bars with straight or headed ends is as yet unavailable for new bridge construction. For barrier replacement, post-installed anchors are constructed by drilling a hole in a hardened concrete and placing the bar inside the hole with an adhesive that acts as a bonding agent between the bar and the concrete. These anchors are needed for the replacement of deteriorate bridge barriers over bridge decks, wingwalls or retaining walls as shown in Figure 1-3 The behavior of post-installed steel anchors in concrete has been extensively investigated. However, very few investigations studied the behavior of post-installed anchors using GFRP bars.

Figure 1-2: PL-3 GFRP-reinforced bridge barrier for new construction.
a) Anchorage with J bents b) Anchorage with straight bars and headed end (adopted from MTO standard drawing S110-92)

Figure 1-3: PL-2 GFRP-reinforced bridge barrier with post-installed anchors for barrier replacement

1.3 Objectives

The main objective of the proposed research is to investigate the bond behavior of GFRP bars embedded in concrete deck slabs for new construction or barrier replacement. The ultimate goal of this research is to determine the anchorage length requirements to sustain the pull-out force resulting from vehicle impact to bridge barriers.

Specific research objectives are as follows:

- To determine the pullout capacity of group GFRP bars with ribbed-surface as affected by bar size, spacing and embedment length in concrete. The pre-installed bars were left one year in outdoor to incorporate environmental effects.
- To investigate the pullout capacity of post-installed GFRP bars as affected by bar size, embedment length and adhesive type.

The First phase of the research included 60 tests to determine the pullout capacity of 16- and 12-mm diameter ribbed-surface bars with different bar spacing and embedment length. In the second phase of the research, 120 GFRP bars were tested to determine their pullout strength as post-installed anchors embedded in concrete. The study included two types of widely-used epoxy adhesives, various bar diameters (12, 16, and 19 mm), two types of GFRP bars (sand-coated and ribbed surface), and three embedment depths for each bar diameter (100, 150, and 200 mm for 12-mm diameter bars and 150, 200, and 250 mm for 16- and 19-mm diameter bars). The tested slabs were exposed to environmental effects during the winter time before testing.

1.4 Thesis Overview

This thesis consists of six chapters. The first chapter includes introduction, problem statement and objectives of this research. The second chapter reviews previous research in the literature related to GFRP technology and pullout behavior of bars embedded in concrete. Chapter 3 summarizes the research program including material properties, test matrix, test setup and test procedure. Chapter 4 presents results and discussion of the performed experiments on group pre-installed GFRP bars, while chapter 5 presents the results and discussion on the pullout behavior of post-installed single GFRP bars. Conclusions and recommendations for the future research are presented in Chapter 6.

Chapter 2 Literature Review

2.1 Fiber Reinforced Polymers Technology

Various types and configurations of Fibre Reinforced Polymers (FRPs) have been manufactured and used since the post-world war II era (Tang, 1997). One of the most important advantages of the FRPs that made them appealing as a structural rehabilitation material is their non-corrosive nature. It is now being used in different forms in various applications in construction industry. FRP as a composite material consists of high strength fibers inserted inside a polymer matrix. Considering the fact that FRPs are made of two different materials, the properties of the composite depends on the individual components used in it. Strength and stiffness of the composite is provided by the fiber. Therefore, most of the load is carried by the fibers compared to the matrix that just bonds and constrains the fibers. Another role of the matrix is the transformation of stresses from fiber to fiber through shear stresses (ACI440.1R-06, 2006). Glass FRP (GFRP), Carbon FRP (CFRP) and Aramid FRP (AFRP) are the most common types of FRPs used in structural engineering applications. FRP composites could be used as strengthening elements in the form of sheets, plates and wraps or as reinforcement in the form of bars, rods and tendons. Different types of commonly used FRP materials in reinforced concrete constructions are presented in Figure 2-1

Figure 2-1: FRP products for reinforcement of concrete structures (ISIS Canada, 2006).

2.1.1 Resins

Choosing the proper matrix has an important role in manufacturing FRPs because matrix properties (physical and thermal properties mainly) significantly affect the mechanical characteristics of the final product. Matrix not only coats and protects the fiber from abrasion, but also has an important role of transferring stresses between fibers. It actually provides lateral support against buckling in fibers when exposed to compressive loads in addition to transferring inter-laminar and in-plane shear stresses in the FRP (ACI 440.1R-06, 2006).

There are two categories of matrix materials for FRPs, namely: i) thermoplastics which are more common to be used in structural engineering applications and include polymer compounds like polyethylene, nylon and polyamides and ii) thermosetting materials including polyesters, vinyl-esters and epoxies (Ametrano, 2011). Molecules in a linear structural form held by weak

secondary bonds make thermoplastics matrix polymers. This secondary bond is destroyable under high pressure or heat and this could reshape the matrix or degrade its mechanical properties. Thermosetting polymers have shown low creep and stress relaxation and proven to remain stable in temperature variations and resist chemical attacks (ISIS Canada, 2006). Some of the mechanical properties of the thermosetting resins are presented in Table 2-1.

Table 2-1: Typical properties of thermosetting resins (ISIS Canada, 2006)

Resin	Specific Gravity	Tensile Strength (MPa)	Tensile Modulus (GPa)	Cure Shrinkage (%)
Vinyl Ester	1.12 – 1.32	73.00 – 81.00	3.00 – 3.35	5.40 – 10.30
Polyester	1.10 - 1.40	34.50 – 103.50	2.10 – 3.45	5.00 – 12.00
Epoxy	1.2 – 1.3	55.00 – 130.00	2.75 – 4.10	1.00 – 5.00

2.1.2 Fibers

As mentioned, strength and stiffness of the FRP is provided by the fibers. FRPs are considered to be orthotropic composites and are always stronger and stiffer in the direction of fibers. Several parameters should be considered when selecting fibers for a specific use including the required strength, stiffness, durability, cost and finally the availability. Glass, Carbon and Aramid are the three common fibers in the market and the description of each is presented below.

Glass Fibers are the cheapest and most commonly used type of fibers produced from glass melt with a diameter of 3-25 microns. There are different grades of this fiber in the market. E-glass is the most common grade and R-glass is the strongest and most expensive. Glass fibers are

one of the most popular in manufacturing of FRP products for constructions. Carbon fibers require processing temperature of higher than 1000°C . They are quite expensive compared to glass fibers and extensively used in pre-stressing tendons and FRP wraps to strengthen reinforced concrete elements. Aramid Fibers, made from aromatic polyamide through extrusion and spinning process (Newhook & Svecova, Reinforcing Concrete Structures with Fibre-Reinforced Polymers., 2006) (ACI 440.2R-08, 2008). Aramid FRPs usually have lower shear and compressive strengths because of anisotropic characteristics of the fibers. In addition, this type of fiber has low resistance against ultraviolet radiation and moisture.

2.1.3 Manufacturing Process

There are three main methods of manufacturing FRPs including pultrusion, wet lay-up and filament winding. In addition to these methods, they could be manufactured by pull-winding, resin transfer, vacuum bag and injection molding which are used for non-structural applications. The three main methods of producing FRP products to be used in structural applications are as follows.

Pultrusion: This procedure is used for bars, tendons, plates and any type of structural profile with a constant cross section. Pultrusion is completely automated with a continuous procedure which combines fiber reinforcement with thermosetting resin. Material is coated with a liquid thermosetting resin and then reinforcement materials (raw fibers, mats etc.) are pulled through the resin bath and the resulting saturated reinforcement are pulled through a heated die (Ametrano, 2011). In the process of passing through the die, the matrix produces a structural component by hardening in the shape of the die. Afterwards, it is pulled from the cured end through the die continuously which leads to production of FRP components of any length. This

process is the reason that all fibers are shaped in one direction. The pulling mechanism is a system of caterpillar or tandem pullers placed between the cutoff point and die exit (ACI 440.1R-06,). A schematic diagram of pultrusion manufacturing procedure is shown in Figure 2-2 for a channel element.

Figure 2-2: Schematic diagram of pultrusion manufacturing procedures (Schoeck Canada Inc., 2011).

Filament Winding: In this procedure, continuous raw fibers are impregnated along with a matrix resin and wound on a rotating mandrel. It moves back and forth with an extreme precision controlled by a computer along the length of mandrel (ACI 440R-07). Afterward, the material is taken to a cylinder and cured and then removed. The product of Filament Winding is usually used as stay-in-place formwork. Figure 2-3 demonstrates scheme of the process.

Figure 2-3: Schematic showing the filament winding manufacturing process (ISIS Canada, 2006).

2.1.4 FRP Properties

FRP properties depend on a number of factors including relative proportions of matrix and fiber, the mechanical properties of the materials used, orientation of the fibers inside the matrix and method of manufacturing. It must be noted that the properties of FRPs are usually specified in the direction of fibers and they are extremely directionally dependent. FRPs behave linearly under tension in direction of fibers until they rupture without showing any sort of nonlinear behavior. The linear elastic stress-strain curve of FRP is demonstrated in Figure 2-4. It can be seen that the elastic modulus in FRP is lower than steel except for some CFRPs.

FRP's strength is dependent on the type of applied stress and is different in compression and tension. It has been proven that for most cases, FRPs are stronger in tension than to compression; therefore they are used as tensile reinforcement. Ultimate strain of the fibers and matrix is one of

the factors affecting the behavior of FRP system as illustrated in Figure 2-5. Typical mechanical properties of GFRP bars considered in this study as compared to those for reinforcing steel bars are presented in Table 2-2.

Figure 2-4: Average stress-strain diagram of FRP.

Figure 2-5: Stress-strain diagram of fibers, matrix and FRP.

**Table 2-2: Comparison between steel and FRP reinforcements used in this project
(Pultrall Inc., 2012) (Schoeck Canada Inc., 2011)**

Property	Steel rebar	Stainless steel rebar	ComBAR® (Schoeck Canada Inc.)	V-ROD® HM (Pultrall Inc.)
Ultimate tensile strength (MPa)	> 400	655	> 1000	>1000
Ultimate elongation (%)	>25	50	>16.7	>16.2
Elastic modulus E (GPa)	200	190	>60	>62.6
Bond Strength (MPa)	13.7	13.7	12.2	14
Min. required concrete cover (mm)	40 (exposed) 30 (unexposed)	<30	db + 10 mm	db + 10 mm
Density (g/cm³)	7.85	7.92	2.2	2.3
Thermal Conductivity (W/mk)	60	16	<0.5	<0.5
Magnetism	Yes	Slightly	No	No

2.1.5 Application of FRP in Construction

FRPs could be used as construction material for different purposes and in a variety of formats. Structural elements could be fabricated of FRP such as utility poles, bridge deck panels, girders etc. They have also been used as internal bars and pre-stressing tendons. FRP bars have different types of surface based on their application and mainly to increase their bond with concrete such as sand coated, helically wound spiral outer surface, indented, braided or ribbed.

Figure 2-6 illustrates the most common FRP bars with different surfaces available in the market. FRP pre-stressing tendons were firstly used in Europe to avoid corrosion problems in the 1980s (Erki & Rizkalla, 1993). FRPs have also been used as stay-in-place formwork by fabricating the concrete formwork from FRP and acting in a composite manner with the cured concrete. It could be used for slabs and beams as tensile reinforcement. Another application of FRPs is strengthening and rehabilitation of concrete, masonry and steel structures. They have been used to increase flexural, shear and confinement strength in an existing structure. Pre-stressing tendons, rigid FRP strips bonded to the structure using an adhesive or lay-up sheets glued to the surface using a polymeric resin (Bisby, 2006) are examples of the materials used for these applications.

Figure 2-6: Commercially available FRP rebars:
(a) Sand coated CFRP; (b) Sand coated GFRP bar; (c) Surface textured CFRP bar; (d) Helically wrapped – Sand coated GFRP bar; (e) Ribbed GFRP bar; (f) Helically wrapped GFRP bar (Baena et al. 2009).

2.2 Bond Mechanism

The bond strength between the concrete and reinforcement in reinforced concrete structures is essential for their strength and stability. In fact, concrete resists the compressive forces while the tensile force is handled by the reinforcement considering the fact that concrete is weak in tension. A force transfer system or bond stress between concrete and reinforcement should exist otherwise the reinforcement would be pulled out of the concrete and leads to structure's premature tensile failure (Alavi-Fard, 1999).

There are three mechanisms for transferring forces from concrete to reinforcements and vice versa, as shown in Figure 2-7.

- a. Adhesion between the bar and concrete (chemical)
- b. Existing friction between the two materials due to the interface roughness, forces transverse to the surface of the bar, relative slip between the bar and surrounding concrete.
- c. Bearing or mechanical anchorage from the bar surface's profile or texture (ACI 408R, 2003).

Figure 2-7: Bond transfer mechanism (ACI 408R, 2003).

Existing compressive and shear stresses on the surface contact of concrete balance the forces on the bar. These forces lead in cracking planes perpendicular to the reinforcement (radial splitting force) and parallel to the reinforcement (effective bond force).

The reinforcement embedment depth inside the concrete must be enough to avoid bond failure. In addition, adequate concrete cover and transverse reinforcement should be provided to decrease the possibility of bond failure. The developed radial and tangential stresses along the bar under these circumstances would be less than the concrete capacity and the reinforcement will achieve the tensile strength that is designed for if bond failure does not occur. In case of inadequate anchorage length or confinement, developed radial or tangential stresses could be greater than the capacity of concrete leading to bond failure.

2.2.1 Failure Modes of Bond

There are two main types of bond failure as described below:

A. Splitting Failure: Transverse splitting cracks in the concrete surrounding the reinforcement could lead to this type of failure (Figure 2-8 a). Cracks developed due to splitting failure occur along planes both parallel and perpendicular to the reinforcement. Loading the reinforcements exerts radial pressure in the vicinity of the bars in concrete. If the applied pressure exceeds the strength of concrete or the transverse reinforcement, concrete-rebar interface starts to crack and the cracks propagate towards the concrete surface.

B. Pullout Failure: When the concrete cover, bar spacing or transverse reinforcement is enough to postpone the splitting failure, pullout failure occurs. This type of failure happens when radial forces are lower than the resistance of surrounding concrete and also transverse

reinforcement, while the tangential forces exceed the concrete resistance. Shearing at the top of the ribs parallel to the bar is the result of pullout failure (Figure 2-8b).

Figure 2-8: Cracking mechanism in bond
(a) Splitting cracks between bars and concrete cover (b) Shear crack and local crushing due to pullout (ACI 408R, 2003).

Crushed concrete could be observed due to both splitting and pullout failure mainly around the bearing surfaces of some of the deformations. It must also be noted that both types of bond failure are due to rebar slippage in the concrete. Higher bond strength could be observed in pullout failure in comparison to splitting failure because of the concrete confinement and therefore, more energy is required for radial splitting cracks to reach the concrete surface.

In order to enhance understanding of the nature of bond response and for determining the enough amount of embedment length for reinforcements to achieve the maximum strength before bond failure, bond force-slip and bond stress-slip curves are being used. Harajili et al. (2004) investigated the influence of bond strength between steel bars and concrete and proposed a pullout and splitting failure bond stress-slip envelop for steel reinforcement in both confined and

plain concrete as shown in Figure 2-9. In addition, Table 2-3 explains the phases in each type of failure in Figure 2-9.

Table 2-3: Summary of phases for the splitting and pullout modes of failure

Phase	Splitting Failure	Pullout Failure
First	This phase will end upon an increase in the residual stress of the bond force leads to developing splitting tensile cracks and the bond stress-slip relationship deviates from the pull out behavior at S_{α}	This phase ends as the bond force becomes constant at the peak bond stress (u_1)
Second	This phase ends as the crack has distributed up to the surface and the cover splits demonstrate a full bond deterioration ($S_{\max}$, $u_{\max}$)	This phase is a constant bond right after the peak bond stress (s_1 to s_2)
Third	This phase illustrates magnificent bond stress reduction ($u_{\max}$ to u_{ps} for confined concrete and $u_{\max}$ to $\beta u_{\max}$ plain concrete)	This phase demonstrates a considerable decrease in stress from s_2 to s_3
Fourth	This phase ends at zero bond stress and is followed after the third phase	This phase is a constant bond for $s > s_3$

Note: see Figure 2-9 for symbols

Figure 2-9: Bond stress-slip envelope (Harajli et al. 2004)

2.3 Parameters influencing bond behavior of FRP bars in concrete

A variety of parameters influence the bond between FRP reinforcements and concrete. Three major factors are discussed in this section, namely: bar properties, structural characteristics and concrete properties. The bar properties include bar size, type of fiber and surface condition. The structural characteristic is mostly related to concrete cover, bar spacing, embedment length, bar cast position and transverse reinforcement. Compressive strength and fiber reinforcement is part of concrete properties that influence bond in FRPs.

2.3.1 Bar Properties

Bar Diameter: The influence of bar size on bond strength of FRP bars has been studied by different researchers in the last two decades (among them: Larralde and Silva-Rodriguez, 1993, Benmokrane et al., 1996, Tighiourat et al., 1998, Tighiourat et al., 1999, Achilides and Pilakoutas, 2004, Okelo and Yuan, 2005, Aly et al., 2006, Okelo, 2007, Baena et al., 2009 and Hao et al., 2009). It is proven that bond strength increases with decrease in bar diameter (similar to steel bars). One reason for less bond strength for greater bar diameter is indicated to be an increase in the amount of bleed water trapped under the bar which creates more voids for bigger bars compared to smaller ones (Tighiouart et al., 1998). Voids decrease the contact area between the bar and the adjacent concrete leading to the bond strength reduction. Achilides and Pilakoutas (2004) concluded that decreasing the bond strength in larger bars is due to the development of less adhesion with the surrounding concrete in comparison with smaller bars. Baena et al. (2009) suggested that Poisson's ratio could also affect the bond properties of FRP bars because of their diameter reduction due to tension.

Fiber Type: The effect of fiber type on bond strength is not considered in CSA S6-06 (2010), JSCE (1997) and ACI 440.1R-06 while CSA S806-12 (2012) takes it into account. In CSA S806-12, a modification factor for every type of fiber is defined to determine the bond strength. This factor is equal to 1 for CFRP and GFRP bars while for AFRP bars a factor of 1.25 is assigned to determine the development length. This indicates that AFRP develops lower bond strength in comparison with CFRP and GFRP bars. In the study performed by Achillides and Pilakoutas (2004), it was illustrated that both GFRP and CFRP developed 72% of bond strength of steel, indicating that glass and carbon fibers had the same influence on bond strength. GFRP bars were compared to steel bars by Tighiouart et al. (1998) and their results demonstrated that

GFRP bars develop lower bond strength in comparison with steel bars. They concluded that this was attributed to the differences in the surface deformation shapes of each type of bar. A study on GFRP bars performed by Benmokrane et al. (1996) indicated that bond strength of GFRP bars was from 60 to 90% of steel bars which is in good agreement with the findings of Tigiourat et al. (1998). In another study conducted by Okelo (2007) on CFRP bars using beam test specimens, it was found that the bond strength in CFRP bars is close to 85% of steel bars.

Bar Modulus: Research studies performed by Aly and Benmokrane (2005) and Aly et al. (2006) illustrated that square root of longitudinal modulus of FRP bars relates to the bond strength of the FRP bars. The Canadian Highway Bridge Design Code (CSA-S6-06, 2010) proposes the adequate development length for a FRP bar by applying a modification factor to transverse reinforcement index with the modular ratio of FRP to steel bars (E_{frp}/E_s). Increasing the modular ratio will raise the bond strength of the FRP reinforcement. JSCE, CSA-S806-12 and ACI 440.1R-06 do not consider the effect of elastic modulus of FRP bars on the bar development length.

Bar Surface: Different surface textures could be used for FRP bars including sand-coated, spiral-wrapped, helical lugged/ribbed and indented FRP reinforcements. Obviously, deformed bars have a much higher bond strength in comparison with plain bars because of the mechanical interlocking between the bar surface and surrounding concrete (Alunno Rossetti et al., 1995; and Cosenza et al., 1997). CSA-S806-12 proposed modification factors to consider the influence of surface profile. This coefficient is equal to 1 for roughened, sand-coated or braided surfaces, 1.05 for spiral-patterned and ribbed-surfaces and 1.08 for indented surfaces. In fact, FRP bars with rough surfaces (roughed, sand-coated and braided) have the highest bond strength.

Mosley et al. (2008) studied the bond strength of FRP bars through three series of beam tests on GFRP and AFRP. They concluded that the deformation/surface texture of the FRP reinforcement did not have a significant influence on the bond strength or width of cracks. Hao et al. (2009) performed pullout tests on 90 specimens to investigate how GFRP bars with ribbed-surfaces and rib geometries behave using 28.7 MPa concrete. It was shown that for the constant rib height of 5% or 6% of the rebar diameter, rib spacing equal to the rebar diameter was superior to rib spacing ranging from 0.5 to 3 times the diameter of rebar. They also demonstrated that in case of rib spacing equal to bar diameter, a rib height of 6% of the bar diameter was superior to rib heights ranging from 3 to 9%. Baena et al. (2009) in their study indicated that in case of bond failure at the bar-concrete interface, the bar surface treatment had a considerable influence on the bond strength. In their tests, bar surface failure happened for concrete with a compressive strength of 30 MPa.

2.3.2 Structural Characteristics

Bar Spacing and Concrete Cover: Amount of concrete cover could determine the mode of failure (splitting or pullout) based on previous studies (Untrauer, 1965; Tepfers, 1973; Orangun et al., 1977; Eligehausen, 1979; and Darwin et al., 1996). Splitting tensile failure could happen in case of small cover and bar spacing while pullout failure mode is possible for large cover and bar spacing (ACI 408R-03). Ehsani et al. (1996) performed an experimental study on 102 specimens reinforced with GFRP bars. He illustrated that concrete cover is one of the main parameters affecting the mode of failure. For test samples with concrete cover equal to one bar diameter ($c = 1d_b$), splitting failure would happen whereas for bigger concrete cover ($c > 2d_b$), pullout failure or bar rupture would mostly happen.

Orangun et al. (1997) tested 62 unconfined and 54 confined specimens that had splitting failure during splice tests. They proposed an equation relating the average bond stress to the concrete cover and splice length using linear regression. Wambeke and Shield (2006) used the same method to evaluate a compiled database of FRP rebar beam-end tests, notch-beam test and splice tests. It was concluded that increasing the concrete cover increases the bond strength following a linear regression of the normalized average bond stress versus the normalized cover and embedment length. In addition, they showed that the relationship between concrete cover and bar spacing and bond strength is nonlinear. The bond strength of FRP bars including the effect of concrete cover was determined by ACI 440.1R-06 based on Wambeke and Shield's work in 2006. CSA-S6-06, CSA-S806-12 and JSCE design codes implied the influence of cover and bar spacing to determine bond strength of FRP bar.

Embedment length: Increasing embedment length or splice length will lead to bond capacity augmentation. However, this increase is not linear because of the non-uniform bond stresses of the bar. Bond failure starts from the vicinity of highest bond force per unit length. Influence of embedment length of FRP bars on bond stress could be found in the literature (Ehsani et al., 1995; Benmokrane et al., 1996; Shield et al., 1997; Cosenza et al., 1997; Tigiourat et al., 1997 and 1998; Achilides and Pilakoutas, 2004; Aly et al., 2006; and Okelo, 2007). It was shown that the maximum average bond strength decreases with increase in the embedment length (similar to steel bar behavior). It was rationalized by considering the non-uniform distribution of bond stress along the bar length.

Ehsani et al. (1995) reported an increase in the initial tensile load and stiffness of the bond stress-slip curve for longer embedment lengths. Achilides and Pilakoutas (2004) reported that the bond stress increase rate was more for smaller embedment lengths in comparison with longer

embedment lengths and this was because of non-uniform distribution of the bond stresses on the bar. Okelo (2007) demonstrated that for samples with bigger embedment length and higher compressive strength, bar fracture, concrete splitting or shear compression failure happens. It was shown that for short embedment lengths with low compressive strength and small bar sizes, bar pullout can occur.

Bar Cast Position: Bar position during concrete casting has a considerable influence on the bond strength between concrete and reinforcement. This effect was studied by Chaallal and Benmokrane (1993), Ehsani et al. (1993), Alunno Rossetti et al. (1995), Benmokrane and Masmoudi (1996), Tigourat et al. (1998) and Wambeke (2003). The horizontal bars near the top surface of the concrete generally have lower bond strength than bottom cast bars since while pouring the concrete, air, water and fine particles migrate upwards and become trapped below the surface of the bar. Horizontal reinforcement with more than 305 mm of concrete cover is usually referred to as top cast bars (ACI 440.1R-06). Ehsani et al. (1993) concluded that the bond strength of top bars is 66% of the bond strength of bottom bars. A modification factor therefore is required to take into consideration the effect of bar location while calculating the development length. ACI 440.1R-03 proposes a modification factor of 1.3 in accordance with studies performed by Chaallal and Benmokrane (1993) and Ehsani et al. (1996). However, ACI 440.1R-06 revised the modification factor to 1.5 based on results of Wambeke and Shield (2006). On the other hand, a bar location modification factor of 1.3 is recommended by CSA-S806-02 and CSA-S6-06 and JSCE design codes.

Transverse Reinforcement: The confinement provided by transverse reinforcement limits to the progression of splitting cracks and increases the force required to cause bond failure (Tepfers, 1973; Orangun et al., 1997; Darwin and Graham, 1993a, b). Application of an

excessive amount of transverse reinforcement more than required is proven to be ineffective and could result to no extra increase in bond strength (Orangun et al., 1977). Darwin et al. (1996) indicated that confinement has a larger effect on the bond strength of steel bars with high relative rib area (i.e. deformed bars with a relative rib area of 0.1 to 0.14) in comparison with the same-sized bars with smaller relative rib areas. Since FRP bars usually have lower relative rib area, the bond strength may not be considerably affected by confinement provided by transverse reinforcement (Wambeke and Shield, 2006).

2.3.3 Concrete Properties

Compressive Strength: A good correlation between the bond strength of FRP in concrete and square root of compressive strength has been proven by previously performed regression analyses (Ehsani et al., 1996; Okelo and Yuan, 2005; and Okelo, 2007). It was shown that this relationship is valid for steel bars embedded in normal concrete strength (less than 55 MPa) according to ACI 408-03. Ehsani et al. (1995) studied the influence of concrete compressive strength on bond strength of FRP bars concluding that compressive strength augmentation would lead to a slight increase in bond strength. They also demonstrated that initial slip is decreased by increasing concrete compressive strength. Makitani et al. (1993), Benmokrane et al. (1996) and Tighiourat et al. (1998) investigated the effect of concrete strength on the bond behavior. It could be concluded from these studies that the increase in bond strength was proportional to the square root of the compressive strength of concrete. The stress development at different stages of loading for normal and high strength concrete is shown in Figure 2-10. It is depicted that nearest rib to the applied load are only active for small amount of loads. When load increases, more ribs become involved gradually. At large loads, concrete crushing in front of ribs for normal strength

concrete occurs causing load distribution along the bar, whereas for high strength concrete only the first few ribs get involved since concrete does not crush (Azizinamini et al. 1995).

As for FRP bars, Cosenza et al. (2002) performed pullout tests on samples with compressive strength of less than 30 MPa and showed that pullout failure was due to the breaking of adjacent concrete although the bar was completely undamaged. The same study was done on concrete with compressive strength of 55 to 60 MPa and it was shown that the damage was concentrated on the ribs of the FRP bar that leads to failure while the damage to the adjacent concrete was ignorable. In case of $30 < f'_c < 55$ MPa, both outer surface of the bar and the adjacent concrete was observed to be damaged.

Figure 2-10: Steel bar behavior in concrete under tension:
 (a) low axial load; (b) Increasing load; (c) significant load (Azizinamini et al. 1995)

Fiber reinforced concrete: Increasing the tensile strength of concrete is one of the main reasons of incorporating fibers. Post cracking resistance of the concrete is also improved by reinforcing the concrete with fiber which could be very important in structures where crack control is an issue. When failure occurs, fibers are pulled out of the concrete leading to an increase in the energy needed to open the cracks (ACI 408R-03). Harajli and Salloukh (1997) experimentally investigated the development/splice strength of steel reinforcement used in steel FRC members which showed an increase of 55% in bond strength for a member reinforced with only 2% fiber. On the other hand, Dancygier et al. (2010) illustrated that the application of steel fibers will lead to bond strength reduction of up to 30% in normal strength concrete and 16% in high strength concrete. The reason was mentioned to be a local stress disturbance caused by fibers preventing proper compaction around the rebar and therefore reducing bond strength. They concluded that in high strength concrete, disturbance of bond was less distinct in comparison with normal concrete. Moreover, the influence of synthetic and steel fibers on high strength concrete and FRP bars was experimentally studied by Won et al. (2008). They showed that bond strength between the concrete and FRP bars could be increased from 5 to 70% by increasing the fiber content in the concrete. A great amount of work is still required to gather enough information to properly assess fibers' influence on bond strength.

2.4 Types of anchorage systems

Cast-in-place and post installed anchors are two major categories of anchor fastening to concrete. In case of cast-in-place anchors, anchors are placed and connected to the formwork before casting the concrete. Headed steel bolts or studs are common components of cast-in-place anchors. Keying or bearing is considered to be the main load transfer mechanism which directly

transfers the load from anchor to the concrete through bearing forces parallel to the loading direction of anchor (Collins et al. 1989, Fuchs et al. 1995). Different types of failure could happen in this case including anchor shank yield and fracture or concrete breakout cone formation. Design models have been developed to determine the behavior of cast-in-place anchors based on previous experimental results (ACI 318M-11).

The other type of anchorage system is known as post-installed anchorage. Different types of anchors are shown in Figure 2-11 comparing the most common types of post-installed anchors. They are divided into two categories, namely: mechanical and bonded anchors. Bonded anchors contain both adhesive anchors and grouted anchors, when an adhesive anchor could be an un-headed threaded rod or a deformed reinforcement bar inserted in hardened concrete in a predrilled hole typically 10 to 25 percent larger in comparison with the anchor diameter. Two-part structural adhesives (combination of a resin and curing agent) are used for these anchors for bonding into the hole (El Menoufy, 2010).

Figure 2-11: Different types of anchors (El Menoufy, 2010).

2.5 Types of Structural adhesives

Two-component adhesive systems are commonly used in construction industry. They may require user mixing and they are usually labeled as component A (which is the main component) and component B which acts as the curing catalyst. The curing period of the adhesives is quite fast which is suitable for conditions when quick setting time is required. They could be polymers (epoxies, polyesters or vinylesters) or hybrid systems. Adhesive types are classified by Collins et al. (1989) as follows:

Epoxy adhesives: Synthetic compounds of epoxy resin cross linked by a curing agent. They are highly durable and have proven to possess a long shelf time and ignorable shrinkage characteristics. Exothermic reaction of the resin and catalyst generates heat which is necessary to accomplish the curing process of epoxy polymers (thermosetting polymers).

Polyester adhesives: They are categorized as thermosetting plastic materials having fast exothermic reaction between the polyester resin and the curing agent (in comparison with epoxies). They cure faster than epoxies, have a short shelf time and tend to self-polymerize under high temperature and degrade when exposed to ultraviolet light.

Vinylester adhesives: These are also considered to be thermosetting plastic. They are in between polyesters and epoxies in terms of curing time, sensitivity to ultraviolet, shelf time and self-polymerization.

2.6 Mechanism of load transfer for adhesive anchors

A polymeric matrix is formed after exothermic reaction of resin and catalyst and holds them together. Maximum bond strength is achieved by installation of adhesive anchors in clean dry holes. The mechanism of load transfer from the adhesive anchor to concrete for post-installed anchors is: a) mechanical interlock and the interface of anchor/adhesive bond and b) chemical

binding of concrete to the adhesive along the anchor as presented in Figure 2-12. Load transfer depends on anchor-adhesive bond strength in addition to the impregnation of the adhesive with the adjacent concrete (Eligenhausen et al., 2007; Collins et al., 1989; Cook et al., 1998; Mcvay et al., 1996).

Figure 2-12: Different Mechanism of Load transfer of adhesive anchors (Eligenhausen et al. 2007).

Five different types of failure in post-installed adhesive anchors are shown in Figure 2-13, depending on the various parameters. Embedment depth is an important parameter determining the variation between the failure modes. Concrete cone breakout is observed for small embedment depths. Increasing the embedment depth leads to a variation in failure mode. Deeper embedment depth could result in a combined concrete cone breakout and bond mode of failure. Bond failure could be at the interface of adhesive and concrete, anchor and adhesive interface, or even both at the same time. In case of very deep embedment depth, by increasing the bond strength, the anchor will rupture (Cook et al. 1998).

Figure 2-13: Post installed adhesive anchors failure modes (Cook and Konz 2001).

Cannon et al. (1989) concluded that failure starts by cone failure for a combined cone-bond failure whereas Luke et al. (1985) demonstrated that failure initiation is due to bond failure followed by cone failure. Collins et al. (1996) reported that combined cone-bond failure initiates by a bond and cone failure at the same time which is in good agreement with results of previous studies (McVay et al., 1996; Cook et al., 1993). A finite-element study was performed on single adhesive anchors by McVay et al. (1996). It was proven that the interface of anchor and adhesive (tension zone) below the concrete surface is where the failure initiates. By propagation of the tension zone with load towards the surface, the concrete and adhesive start to expand, leading to an increase in confinement and shear resistance in the adhesive layer. Shear resistance reduction and reaching the tension zone to the surface will lead to anchor failure.

2.7 Bond models for adhesive anchors

Adhesive anchors have been commonly used in practice. The ultimate tensile capacity models for adhesive anchors could be found in the literature (Cook et al., 1998). Four different failure modes have been introduced and mathematical relationships have been developed and used for analyzing them. Concrete cone failure, bond failure, combined concrete cone and bond failure and anchor rupture are four most common failure modes as described below.

2.7.1 Concrete Cone Model

If the anchor depth is relatively small, pulling out anchors from concrete will lead to development of a failure surface around the anchor in shape of a cone that could be modeled based on either (i) concrete cone or (ii) concrete capacity design (CCD). Cook (1993) and CEB (1994) models are based on the assumption of a circular projection area on the concrete surface due to failure cone with a side inclination of 45° to concrete surface. In CCD models however, a projected square area on the concrete surface is assumed. The side dimension of the projected area is assumed to be three times the anchor embedment depth and there is a 35° angle from the bottom of the anchor to the horizontal surface of concrete (CEB, 1994; Fuchs et al., 1995; Eligehausen et al., 2006b).

The concrete failure model is based on experimental results performed on headed cast-in-place anchors and were first developed in the 1970s. On the other hand, CCD models were first proposed at the early 1990s. They are based on both theoretical and experimental models by Eligehausen and Sawade (1989) and Eligehausen et al. (1992) on headed bars. Equation 2.1 is the general form of all models as presented below:

$$N_c = k h_{ef}^n \sqrt{f'_c} \quad (2.1)$$

Where N_c is pullout resistance of concrete cone, k is empirical coefficient which is determined based on the type of anchor, f'_c is concrete compressive strength, and h_{ef} is defined as the length of anchor excluded the head. The power n is either 1.5 or 2 for CCD and concrete cone model respectively. The power n for concrete cone models was derived from the circular failure projection area while for CCD, it was calculated from projection area of failure surface (h_{ef}^2) multiplied by size effect factor ($h_{ef}^{-0.5}$) based on Eligehausen et al. (1992) and Cook (1999). Fuchs et al. (1995) concluded that concrete cone model can underestimate the pullout strength for bars when the embedment depth is low, while, CCD models were proven to give good results for a wide range of embedment depths. Although the concrete cone model was more popular initially, CCD model is more widely used recently (ACI 349-01 and ACI 318M-11).

ACI 349-85 specified the following equation based on cone failure model for headed cast-in-place bars assuming the circular failure surface with a failure angle of 45° :

$$N_{cc} = 0.33\sqrt{f'_c}A_c, A_c = \pi h_{ef}(h_{ef} + d_h) \quad (2.2)$$

Where f'_c is concrete compressive strength, A_c is the single anchor projected area, h_{ef} is the effective embedment depth, and d_h is head diameter of headed anchor.

Another equation proposed for cone failure mode was given by Eligehausen et al. (1984) assuming a failure angle of 35° for headed cast-in-place bars. They proposed a value of k equal to 17 for this equation.

$$N_c = kh_{ef}^{1.5}\sqrt{f'_c} \quad (2.3)$$

Eligehausen et al. (1984) proposed another equation for post-installed adhesive anchors with a circular angle of failure as follows:

$$N_c = 0.92h_{ef}^2\sqrt{f'_c} \quad (2.4)$$

Eligehausen et al. (1984) also proposed an equation based on CCD failure mode assuming a square failure surface with an angle of 35° for cast-in-place headed bars as presented in Equation 2.3 with $k = 16.8$.

On the other hand, ACI 349-01 and ACI 318M-11 recommend using Equation. 2.3 for cast-in-place headed anchors assuming a square failure surface with an angle of 35° based on CCD model considering k equal to 10.

Kim and Smith (2010) proposed Equation 2.5 for CCD failure of FRP anchors in un-cracked concrete which is similar to that recommended by ACI 349-01 and ACI 318M-11:

$$N_{cc} = 9.68h_{ef}^{1.5}\sqrt{f'_c} \quad (2.5)$$

2.7.2 Bond Model

This model assumes a surface of failure adjacent to the embedded anchor at the interface of anchor and concrete. Bond stress of the concrete-anchor is usually assumed to be uniform in the proposed bond models for simplicity. Cook (1993) presented the following equation for bond interface resistance of adhesive metallic anchors:

$$N_{cb} = \tau\pi d_0 h_{ef} \quad (2.6)$$

where d_0 is anchor hole diameter. While Cook and Kunz (2001) proposed the following equation for the same type of failure in the same anchors:

$$N_{cb} = \tau\pi d h_{ef} \quad (2.7)$$

where d is the diameter of embedded anchor.

The proposed equations were both for metallic anchors as mentioned. Anchor diameter or hole diameter could be used for determination of the bond resistance. However Cook et al. (1998) proposed that using hole diameter is more realistic for design purposes.

2.7.3 Combined Concrete Cone and Bond Models

Concrete cone strength (N_{cc}) plus bond interface strength (N_{cb}) will develop the pullout strength (N_u) for this case. Considering the uniform bond stress distribution, Cook (1993) presented a model for bonded metallic anchors as follows:

$$h_{ef} < h_c: N_{cc} = 0.92h_{ef}^2\sqrt{f'_c}; \text{ and } N_{cb} = 0 \quad (2.8)$$

$$h_c < h_{ef}: N_{cc} = 0.92h_c^2\sqrt{f'_c}; \text{ and } N_{cb} = \tau_0\pi d_0(h_{ef} - h_c) \quad (2.9)$$

where $h_c = \tau_0\pi d_0 / 1.84\sqrt{f'_c}$. Considering nonlinear distribution of shear stress for bond which is known as elastic bond stress model, Cook (1993) proposed the following equations:

$$h_{ef} < h_c: N_{cc} = 0.92h_{ef}^2\sqrt{f'_c}; \text{ and } N_{cb} = 0 \quad (2.10)$$

$$h_c < h_{ef}: N_{cc} = 0.92h_c^2\sqrt{f'_c}; \text{ and } N_{cb} = \tau_{max}\pi d_0 \left[\frac{d_0}{\lambda'} \tanh\left(\frac{\lambda'(h_{ef} - h_c)}{\sqrt{d_0}}\right) \right] \quad (2.11)$$

where

$$h_c = \frac{\tau_{max}\pi d_0}{1.84\sqrt{f'_c}} \left[\sec h^2\left(\frac{\lambda'(h_{ef} - h_c)}{\sqrt{d_0}}\right) \right] \quad (2.12)$$

In the above equations, h_c is the depth of failure concrete cone, τ_0 is the uniform bond stress, τ_{max} is the maximum stress in non-uniform stress distribution, and λ' is an elastic constant.

In addition, Ozdemir (2005) presented a combined cone-bond model for FRP anchors as follows:

$$h_{ef} < 50 \text{ mm}: N_{cc} = 0.33\sqrt{f'_c}h_{ef}(d_0 + h_{ef})\pi; \text{ and } N_{cb} = 0 \quad (2.13)$$

$$h_{ef} > 50 \text{ mm}: N_{cc} = 0.33\sqrt{f'_c}50(d_0 + 50)\pi; \text{ and } N_{cb} = \pi d_0(h_{ef} - 50) \quad (2.14)$$

Similarly, Ozbakkaloglu and Saatcioglu (2009) proposed the following formula:

$$h_{ef} > h_c : N_{cc} = f_{ct,exp} h_c (d_0 + h_c) \pi ; \text{ and } N_{cb} = \pi t d_0 (h_{ef} - h_c) \quad (2.15)$$

where $f_{ct,exp}$ is the concrete splitting strength obtained experimentally. For FRP anchors, Kim and Smith (2010) proposed the following equations for combined failure:

$$N_{cb} = 4.62 \pi d_0 h_{ef} \quad \text{When } f'_c \leq 20 \text{ MPa} \quad (2.16)$$

$$N_{cb} = 9.07 \pi d_0 h_{ef} \quad \text{When } f'_c \geq 20 \text{ MPa} \quad (2.17)$$

2.7.4 Anchor Rupture Models

This model is developed based on tensile capacity of anchors. Hand-made FRP anchors were showed by Kim and Smith (2009) to have tensile strength of 59% of the strength of flat FRP tensile coupons. They proposed the following equation:

$$N_{ar} = 0.59 w_{FRP} t_{FRP} f_{FRP} \quad (2.18)$$

Where w_{FRP} , t_{FRP} and f_{FRP} are width, thickness and flat coupon tensile rupture strength of FRP.

Chapter 3 Experimental Program

3.1 General

Steel bars have been successfully used as reinforcement in concrete except for severe corrosive environments due to durability issues. Bridge decks and barriers are examples of structures exposed to severe corrosion for a diversity of reasons including usage of de-icing salt in winter. In order to control this corrosion issue, epoxy coated steel bars were introduced and commonly used for bridges. However, long-term performance of epoxy coated bars is now under question due to premature corrosion in these bars. Therefore, FRPs became an alternative to overcome this problem. The current study is aimed to investigate the pullout capacity of group and single bars. It is comprised of two phases. In Phase I, the anchorage behavior of cast-in-place GFRP group bars with various bar spacing is investigated, while in Phase II, the pullout capacity of single post- installed GFRP anchorage with various embedment depths are studied. This chapter presents the details of the experimental program undertaken for each phase of this study.

3.2 Phase I: Group Anchorage

The first phase was intended to evaluate the overlap effect of adjacent bars on the pullout capacity of closely spaced cast-in-place group of bars (group anchorage). The tests performed on group of two or three GFRP bars embedded in concrete with various bar spacing. The main objective of this phase is to investigate the influence of bar spacing in behavior of bar groups. Testing material, specimens, test set-up and procedure are described in details in the following subsections.

3.2.1 Properties of GFRP rebars used

In the first phase, ComBAR[®] GFRP bars with ribbed-surface produced by Schoeck Inc. were used (Schoeck Canada Inc., 2011). ComBAR has an especially ribbed and coated surface. The matrix VEU resins are made of 25% modified vinyl ester. Continuous ECR-glass fibers contain the rest (75%) of the fiber reinforcement. Two different bar diameters of 12 mm and 16 mm are used in this phase of research. The geometrical properties of these GFRP bars are illustrated in Table 3-1. It should be noted that the studied 12-mm diameter bars had straight ends embedded in concrete while the 16-mm diameter GFRP bars has headed-end embedded in concrete.

Table 3-1: Geometrical properties of the ribbed GFRP bars used for phase I (Schoeck Canada Inc., 2011).

Bar size	Core Diameter (mm)	Exterior Diameter (mm)	Area (mm²)	Length head (mm)	External Diameter Head (mm)	F_{head,k} Short-term (kN)	F_{head,k} Long-term (100 years) (kN)
#4 (13M)	12	13.5	113	--	--	--	--
#5 (16M)	16	18	201	100	40	100	59

The special “ribbed” surface profile of these bars is shown in Figure 3-1(a) for straight and headed-end ribbed bars. Unlike steel, GFRP bar bents have to be manufactured in the factory prior to curing of the resin since GFRP bars cannot be bent once hardened. It must be considered that the tensile capacity of bent GFRP bars is much less than straight bars because the fibers get rearranged and redirected in the bending process. Consequently, more bars are required when designing the structural member with bent bars. Therefore, manufacturers came up with the idea

of headed-end GFRP bars to provide a cost-effective alternative for such bent bars. These headed bars could be used for slabs as punching shear reinforcement and in beams as shear reinforcement. In other words, reducing the development length for straight bars (and avoiding bending bars) in case of geometric constraints when there is no enough room to satisfy the development length requirement is the main reason behind manufacturing bar end heads. In case of concrete bridge barriers, GFRP bars with headed ends are used as tensile reinforcement at the inside face of the barrier walls to reduce the development length associated with the use of bent bars. ComBAR[®] end heads are produced by casting polymeric concrete on the ribbed-surface of the bar end. Behavior of these bars is proven to be elastic up to failure with 1188-MPa tensile strength at rupture. It should be noted that elastic modulus of these bars is 60,000 MPa, compared to 200,000 MPa for steel bars.

Figure 3-1: GFRP bars used in this study
a) ribbed-surface bar; b) Sand-coated bar

3.2.2 Properties of concrete

Concrete slabs with dimensions of 30-m length, 1.35-m width and 300-mm thickness were constructed to perform the pullout tests. Ready mix concrete with a specified compressive strength of 30 MPa was used to cast these slabs in an off-campus facility owned by Ontario Ministry of Transportation (MTO). The concrete was normal weight (Type V, MTO) with 6.5% air content. After pouring the concrete, it was cured for two weeks at MTO site and was exposed outdoors for almost two years.

3.2.3 Specimen Preparation

The test specimen consists of 9 concrete slabs and 150 GFRP bars. The concrete slabs were reinforced with top and bottom reinforcement of 15M steel bars spaced at 300 mm each direction (as shown in Figure 3-2) to include the confinement effect of internal reinforcement on the pull-out capacity of GFRP bars. Concrete cover to top reinforcement was taken 70 mm. An embedment depth of 200 mm (commonly used by MTO for vertical reinforcement of bridge barriers) was chosen for the GFRP bars to be placed inside the concrete slab. A total of 60 group bars were installed in the panels consisting of 30 double bars (group of two bars) and 30 triple bars (group of 3 bars). Straight 12-mm diameter GFRP bars were used to build half of the double and triple bars and the other half was made of 16 mm headed GFRP bars.

The nominal spacing between installed GFRP bars was selected to be 150, 225, and 300 mm for each specimen. The GFRP bar Spacing of 300 mm was chosen since it is the maximum allowable mentioned in CSA-S6-06 (2006). Five identical samples were erected for each bar spacing. Distances between adjacent groups was maintained 1000 mm to accommodate the

Figure 3-2: View of phase I specimen preparation

- a) View of triple GFRP bars and steel mesh before casting; b) View of twin GFRP bars with headed end before casting; c) Ready mix truck during casting; d) Casting slabs with double bars

loading setup and eliminate any effect of group bars on each other. Figure 3-2 illustrates the formworks, placement of steel reinforcement and FRP bars and pouring concrete. Figure 3-2 –a) shows view of the straight FRP bars in triple-bar arrangement whereas Figure 3-2-b demonstrates the headed-end bars in double-bar arrangement. Table 3-2 shows the test matrix for phase I. Sample of double and triple group bars after casting the concrete are presented in Figure 3-3.

Table 3-2: Phase I Test matrix

Series	Group	Bar Type - Diameter (mm)	Embedment Length (mm)	Bar Spacing (mm)			No. of Samples
Series 1	Double	Ribbed, Headed End, 16 mm	200	150	225	300	3×5=15
	Triple	Ribbed, Headed End, 16 mm	200	150	225	300	3×5=15
Series 2	Double	Ribbed, 12 mm	200	150	225	300	3×5=15
	Triple	Ribbed, 12 mm	200	150	225	300	3×5=15

Figure 3-3: Views of double and triples bars after concrete hardening
a) Double bars; b) Triple bars

3.2.4 Specimen Nomenclature

The specimen nomenclature consists of 5 symbols separated by a dash. The first symbol indicates the type of group action (D = Double, T = Triples). The second one stands for bar size (N4 = #4 or 12 mm, N5 = #5 or 16mm), while the third part demonstrates the nominal spacing between bars in the group (i.e. 150, 225 and 300 mm). The fourth symbol indicates the slab number and the fifth one is specimen number in the slab. For example, T-N5-150S-SC5-S1 indicates a Group of triples #5 bars with 150 mm spacing located at the first sample in slab C5. This specimen nomenclature will be shown later when presenting experimental findings.

3.2.5 Test Setup and Procedure

Two test setups were designed and assembled according to ASTM (2010) requirements to be used to test all the pre-installed GFRP anchorage specimens. The main parameter that needs to be considered is the distance between support of the test setup and the bars being tested, which should be a minimum of $2L_e$ as specified by ASTM (2010), where, L_e is the embedment depth of the GFRP bar. Ahmed et al. (2008) stated that a minimum of $2L_e$ is required to minimize any effect of the test setup on the pull-out capacity of GFRP bars. For the studied specimens, the embedment depth was considered 200 mm in all pre-installed bars. Therefore, the clear spacing between support of the test setup and GFRP bars was considered minimum 400 mm in all specimens, where the test setup for the majority specimens had a spacing even more than 400 mm.

In Phase I, the developed test setup for the double bars has two sets of I-shaped steel beams, two manual hydraulic jacks, 6 hollow steel sections (HSS), two potentiometers, two compression/tension ring load cells and custom made grips to hold the bar. Figure 3-4 shows a schematic diagram of the double-bar test setup. The triple test setup has two sets of I-shaped steel beams, a manual hollow hydraulic jack located in the middle bar, 4 hollow steel sections (HSS), three potentiometer, three compression/tension ring load cells and custom made grips to hold the bar. The load was transferred from the hydraulic jack to the grips at the end of the GFRP bars. The load cells are placed between the grips and HSS sections to record the magnitude of the applied load on each bar. Hydraulic jacks, load cells, and potentiometers were connected to data acquisition system, Figure 3-10, to continuously record the experimental results up to the failure of GFRP anchors. Figure 3-5 shows schematic diagrams of the triple-bar test setup. Figure 3-6 and Figure 3-7 show views of the steps of assembling the test setup for double and triple groups respectively.

Figure 3-4: Schematic diagram of the double bars test setup
a) Side view; b) Elevation

Figure 3-5: Schematic diagram of the triple-bars test setup
a) Side view; b) Elevation

a) Placement of I beam and POTs; b) Close-up view of POT to measure bar slip; c) Placement of the two jacks and plates; d) Placement of top HSS beams to place load cells and grips

Figure 3-6: Views of the sequence of assembly of the test setup for double bar testing
e) Placement of hollow load cells; f) View of the two pumps used to apply load ; g) View of the final setup

Figure 3-7: Views of the sequence of assembly of the test setup for triple bar testing
 a) View of POTs to measure bar slip; b) View of the two spread beams and load cells; c) View of the final setup

Figure 3-8: Views of the grips assembly used in this study
a) Wedges used to grip GFRP bars in place; b) Wedges insert around the GFRP bars; c) View of the hollow slit cylinder; d) View of final grip assembly around the GFRP bars

Grip cell Dimensions	A	B	C	D	E	F
Size (mm)	25.50	140.00	200.00	77.50	220.00	150.00

Figure 3-9: Schematic diagram of grip assembly.

A critical concern when testing FRP reinforcing bars is their low transverse strength. Therefore, the bars could not be tested using the traditional wedge-shaped grip system employed for steel rebar bond tests. If a FRP bar is loaded using a traditional wedge-shaped frictional grip, the combination of high compressive stresses and mechanical damage caused by the serration from the wedge surfaces would lead to premature failure in the gripped region. For this study, a custom made steel grip was fabricated as shown in Figure 3-9 such that the applied force would not be enough to damage the GFRP bar, yet large enough to ensure that the grip does not fail prior to the bond failure of the pullout specimen. This grip was tested to ensure its performance before pullout tests. The grip used in this study consisted of three conic cylinder pieces (wedges) which encompassed the rebar on top of the load cell. For each bar diameter, there were three separate wedges used to grip the bar. Figure 3-8 shows views of this custom-made grip cell and pieces.

Figure 3-10: Views of the data acquisition system, load cells and potentiometers used in this study
 a) Data acquisition system; b) Load cells; c) Potentiometers (POTs)

3.3 Phase II: Post installed Anchorage

The second phase aimed at investigating the pullout behavior of post-installed adhesive anchors. The tested concrete slabs were subjected to natural environmental conditions for a period of one year since they were left outdoor. In addition, GFRP anchors were also exposed to natural environmental conditions for about 4 months during the winter after they were glued and cured. The objective of this phase of research was to evaluate two types of adhesives and GFRP bars with two different bar diameters and different embedment depths. Every specimen was made of five identical samples to obtain more reliable results. This section provides information on the material properties, specimen preparation, test setup, instrumentation and test procedure for the second phase of experiments.

3.3.1 Properties of GFRP bars used

In the second phase, V-ROD GFRP bars with sand-coated surface produced by Pultrall Inc. (Pultrall Inc., 2012) along with ribbed-surface bars produced by Schoeck Inc. were considered in this study. Sand-coated bars are manufactured through pultrusion process in addition to an in-line coating procedure for applying the outer surface with sanded material. These bars are manufactured from E-glass fibers with 77.5% of fiber content (by weight) in vinylester resin. 16- and 19-mm diameter bars were used in this phase for sand-coated bars, while 12- and 16-mm diameter ribbed-surface bars are also used in this phase which was utilized in Phase I. Properties of ribbed-surface bars are those discussed in Phase I. On the other hand, Table 3-3 shows the geometric and mechanical properties of the sand-coated GFRP bars considered in this study.

Table 3-3: Sand-coated GFRP material Properties (Pultrall Inc., 2012)

Product type	Bar size	Diameter (mm)	Cross-section area (mm ²)	Guaranteed tensile strength (MPa)	Modulus of elasticity (GPa)	Strain at failure
HM*	#5 (15M)	15.875	197.9	1184	62.5±2.5	1.89%
	#6 (20M)	19.05	285.0	1105	64.7±2.5	1.7%

* *HM : High modulus or Grade III*

3.3.2 Properties of concrete

Concrete material properties and description were exactly the same as the concrete used in phase I of this research.

3.3.3 Properties of adhesives used

Two different commercial adhesives were used to glue the anchors to the concrete. The first one is called HIT RE 500 (Adhesive TYPE I) which is an epoxy-based adhesive with a high strength specifically designed to fasten in solid based materials. Per the manufacturers' catalogue, the installation temperature range is very wide and could be from -5°C up to 49°C. It could be used under water and for holes up to a maximum of two times larger than bar diameter but limited to 76-mm hole diameter. For Hilti-type adhesive, the minimum bond strength of 12.4 MPa is reported by manufacturer (Hilti Canada Corp., 2011). Sika AnchorFix®-2001 (adhesive TYPE II) is the name of the second adhesive used in this research. It is a VOC-compliant epoxy acrylate hybrid anchoring adhesive (Sika Canada, 2013). It could be dispensed in cold condition

to a minimum of -5°C as reported by the manufacturer. The manufacturer describes it to be suitable for medium and heavy loads applications with a very short setting time in dry or damp holes, achieving high early strength in a variety of base materials. Properties of both adhesives are presented in Table 3-4.

Table 3-4: Adhesive Specifications

Adhesive Name	Compressive Strength (MPa)	Tensile Strength (MPa)	Modulus of Elasticity (MPa)	Bond Strength (MPa)	Absorption (%)	Resistance (Ω/m)	Min Cure Temperature °C
TYPE I**	82.7	43.5	1493	12.4	0.06	6.6×10 ¹³	-5
TYPE II***	69	13.3	3.7	--	--	5×10 ¹³	-5

** (Hilti Canada Corp., 2011). *** (Sika Canada, 2013).

3.3.4 Specimen Preparation

Fourteen concrete slabs were constructed in July 2011 with top and bottom 15M steel bars at 300 mm spacing in each direction. Then, the layout of post-installed GFRP anchors was made in accordance with ASTM E E488/E488M-10 (ASTM, 2010). To install the FRP anchors, holes were drilled using rotary hammer pits as shown in Figure 3-11. The chosen diameters for bits were 17.50, 22.33, 24.00 and 26.99 mm as demonstrated by Table 3-5.

Figure 3-11: Views of the drilling holes in the concrete slab
a) Drilling at right angle; b) View of drillbits; c) View of the hole d) View of drilling.

Table 3-5: Phase II Drill Bits sizes

Bar Type	Nominal Bar Diameter (mm)	Measured Bar Size* Diameter (mm)	Drill Bit Size (mm)
Ribbed Bar # 4	12	13.50	17.50
Ribbed Bar # 5	16	18.00	22.33
Sand-Coated Bar # 5	16	20.00	24.00
Sand-Coated Bar #6	19	23.00	26.99

* *Out-side diameter*

For adhesive Type I, the holes were cleaned by wire brushes and compressed air although this type of epoxy performs well in moisture. On the other hand, the holes for Type II adhesive were cleaned and dried by torch, wire brushes and compressed air since this series of bars have been installed when the temperature of concrete were between 0 and -5°C. Views of cleaning the holes before application of adhesives are presented in Figure 3-12. In addition, ribbed-surface bars were cleaned using acetone to ensure proper bonding after placement of the bar inside the holes. Prior to bar placement, each bar was marked to ensure that proper embedment length is satisfied, as presented in Figure 3-13. Once the holes and bars were cleaned, the two-component adhesive package was installed in the dispenser and injected into the holes. Then, bars were pushed into the holes in a screwing (rotating) fashion.

The Type-I-adhesive anchors were installed in seven slabs during the month of November of 2012, while the Type-II-adhesive anchors were installed during the first two weeks of December 2012 (installed in the remaining slabs). The slabs were left outdoor during winter of 2013. The adhesive anchors were tested during the last week of March 2013. Compressive strength of concrete was measure to be 40.2 MPa by performing compression tests on several

Figure 3-12: Views of hole cleaning in the concrete slab
a) Air pressure into the holes; b) Wire brush ; c) Torch application to ensure no ice in holes; d)
View of hole after cleaning

Figure 3-13: GFRP bars preparation procedures

a) Embedment length marking; b) Cleaning ribbed bar end with acetone; c) Marking end of embedment length to determine level of concrete surface; d) Cleaning sand-coated bar end with air pressure

core samples taken from the slabs. Figure 3-14 shows the installation procedure of the post-installed GFRP bars. It should be noted that each GFRP bar was secured to a timber form to keep it vertical and in location during adhesive curing. Table 3-6 shows the test matrix of the performed tests with details including adhesive type, bar diameter, spacing and embedment length and the number of samples for each case.

Table 3-6: Phase II Test matrix

Series	Group	Adhesive Type	Bar Type - Diameter (mm)	Embedment Length (mm)			No. of Samples
Series 1	Group I	TYPE I	Ribbed GFRP Bar 12 mm	100	150	200	3×5=15
	Group II	Type II	Ribbed GFRP Bar 12 mm	100	150	200	3×5=15
Series 2	Group I	TYPE I	Ribbed GFRP Bar 16 mm	150	200	250	3×5=15
	Group II	Type II	Ribbed GFRP Bar 16 mm	150	200	250	3×5=15
Series 3	Group I	TYPE I	Sand-coated GFRP Bar 16 mm	150	200	250	3×5=15
	Group II	Type II	Sand-coated GFRP Bar 16 mm	150	200	250	3×5=15
Series 4	Group I	TYPE I	Sand-coated GFRP Bar 19 mm	150	200	250	3×5=15
	Group II	Type II	Sand-coated GFRP Bar 19 mm	150	200	250	3×5=15

Figure 3-14: Installation procedure GFRP bars
 a) Inserting adhesive into the hole; b) View of adhesive filling into the hole; c) Sequence of bar insertion into the hole filled with adhesive; d) Final set of bar; e) Alignment of bar reading

3.3.5 Test Setup and Procedure

Two test setups were designed and assembled according to ASTM E488/E488M-10 (ASTM, 2010) requirements as mentioned earlier. The distance between supports of test setups were selected in a way to satisfy the minimum requirement of “ $2L_e$ ” as specified elsewhere (ASTM, 2010) similar to that described for phase I.

In this Phase of research, the developed test setup had 1 manual hydraulic jacks, 4 hollow steel sections (HSS), 1 potentiometers, 1 compression/tension ring load cells and the custom made grip system used in phase I. Figure 3-15 shows a schematic diagram of the test setup for the pullout of single GFRP bar. The anchors were pulled out using the hydraulic jack connected to a manual pump. Each anchor was instrumented with one potentiometer (POT) to measure the bar displacement at different loading stages. The loading pump and the POTs were connected to a data acquisition system to continuously record data up to failure. Figure 3-16 shows views of the potentiometer installation and the final setup before testing. A data acquisition system was used to record data from the load cell and potentiometer.

Figure 3-15: Schematic diagram of the post-installed bars test setup
a) Side view; b) Elevation

Figure 3-16: Views of the sequence of assembly of the test setup for Post-installed pullout testing
a) Placement of HSS beam and supports; b) view of POTs to measure bar slip; c) View of the final setup

3.3.6 Specimen Nomenclature

The specimen nomenclature consists of 6 symbols separated by a dash. The first symbol is the type of GFRP bars (S= Ribbed GFRP bar, V = Sand coated GFRP bars). The second symbol refers to the bar size (N4 = #4 or 12 mm, N5 = #5 or 16 mm , and N6 = #6 or 19 mm), while the third symbol stands for the nominal embedment depth of the samples (100, 150, 200 or 250 mm). The fourth symbol illustrates the Type of Adhesive (H= Type I, S= Type II), the fifth symbol refers to number of identical bars in the slab and the last one is the slab number. For instance, S-#4-200-S-S5-SA5 indicates the #4 ribbed GFRP bars, with the embedment depth of 200 mm, anchored with adhesive Type II, and located as the fifth specimen in slab A5.

Chapter 4 Experimental results and discussion of Phase I

This chapter presents the pullout test results performed in Phase I for the group action of GFRP bars. The modes of failure are discussed followed by results presented in tables and figures to investigate the effect of bar spacing in group anchors and compare the results with other available models.

4.1 Modes of failures

Six different modes of failure were observed in the pullout tests in this phase of research. Schematic diagrams of these failure modes are presented in Figure 4-1. These failure modes include concrete cone failure (type A), small concrete cone and GFRP/Concrete interface failure (type B), deep concrete cone and GFRP/concrete interface failure (type C), bar pull-through/slippage without any noticeable concrete cone failure (type D), GFRP bar rupture (type E) and finally concrete splitting in the form of traverse crack at the top surface of the slab (type F).

Figure 4-2 depicts sample views of two specimens after the failure for double and triple group bars. Figure 4-2(a) shows an example of failure patterns observed during the pullout tests illustrating the overlap of concrete cone failures in double group bars with headed end and 150-mm bar spacing. In addition, the left bar in Figure 4-2(a) demonstrated type C failure shown in Figure 4-1. On the other hand, for triple bars shown in Figure 4-2 (b), no sign of group action was observed and the failure mode was bar pull through type D shown in Figure 4-1.

Figure 4-1: Various types of observed pullout modes of failure for pre-installed GFRP bars
A) Concrete cone failure; B) Small concrete cone and GFRP/Concrete interface failure; C) Deep concrete cone and GFRP/concrete interface failure; D) Bar pull-through/slippage, E) GFRP bar rupture and F) Concrete splitting

Figure 4-2 Samples of mode of failure observed in specimen:
(a)D-N5-150S-SD3-S1 due to deep concrete cone with GFRP/concrete interface failure and (b) T-N4-225S-SC1-S1 due to bar pull-through/slippage

In order to investigate the failure inside the concrete, several core samples were taken after failure at selected bar locations. Then, these core samples were sliced in half to expose the bar and surrounding concrete at failure. Figure 4-3(a) shows the core samples of first bar on Figure 4-2(a). It can be observed that the breakage of the large disk at the end of the bar head. Figure 4-3(b) demonstrate a Type D failure (bar pull-through and slippage with concrete observed at the end of #4 straight GFRP bars which was typical in all #4 straight bars. Photos of the failure mode

of each bar group are presented in Appendix A and along with photos of the bar shape and its interface with concrete in sliced core samples.

Figure 4-3: View of split core samples:
(a) #5 headed ribbed bar in specimen D-N5-150S-SD3-S1 and (b) # 4 ribbed bar in specimen T-N4-225S-SC2-S5

4.2 Experimental results and discussion

The load-slip history of the double-bar group shown in Figure 4-2a is presented in Figure 4-4. It can be observed that the load-slip relationship was linear up to failure. Appendix A shows the load-slip graphs for each specimen subjected to pullout loading. In these graphs, it should be mentioned that the GFRP bars in the bar group are numbered from left to right. For example, in Figure 4-4, Bar # 1 would be the one on the left (shown in Figure 4-2a) and Bar # 2 is the one at the right. Table 4-1 and Table 4-2 summarize the test results of double and triple #4 GFRP bar

groups with straight ends, respectively. While Table 4-3 and Table 4-4 depicts the pullout strength and mode of failure of triple and double #5 headed-end GFRP bar groups, respectively.

Figure 4-4: Load-displacement relationship for double-anchorage group presented in Figure 4-2(a)

As shown in Table 4-1 through 4-4, the failure loads in each group were recorded separately for each bar with an individual load cell. It is important to identify which bar failed first in each test of the double- and triple-bar groups so that the pullout capacity of the bar group is identified. It should be noted that in column 5 of these tables, the maximum recorded load in the bar that failed first in each bar group was underlined. It should also be noted that the failure in pullout test occurred with a sudden impulse-like failure that unbalanced the test setup in some cases. The first bar failed in the bar group was identified from the recorded force-time graphs obtained from the data acquisition system. The first bar that showed a sudden reduction in the recorded load was considered to be the first bar reaching failure based on the failure modes shown in Figure 4-1

Table 4-1: Test results of triple #4 GFRP group bars with straight end

Spacing (mm)	Specimen set	Designation	Bar #	Failure load * (kN)	Mode of failure **
150	I	T-N4-150S-SC1-S3	1	71.1	C
			2	<u>72.0</u>	B
			3	72.7	D
	II	T-N4-150S-SC1-S4	1	63.8	B
			2	<u>68.5</u>	D
			3	57.4	C
	III	T-N4-150S-SC1-S5	1	70.1	B
			2	<u>69.8</u>	C
			3	81.3	D
	IV	T-N4-150S-SC1-S6	1	50.3	B
			2	<u>61.7</u>	D
			3	56.4	D
225	I	T-N4-225S-SC1-S1	1	68.1	C
			2	<u>65.0</u>	B
			3	61.8	D
	II	T-N4-225S-SC1-S2	1	48.1	D
			2	<u>72.0</u>	C
			3	52.7	B
	III	T-N4-225S-SC2-S5	1	79.6	C
			2	<u>68.5</u>	B
			3	74.3	D
	IV	T-N4-225S-SC2-S6	1	74.3	B
			2	<u>67.3</u>	C
			3	69.1	D
	V	T-N4-225S-SC2-S7	1	84.8	C
			2	<u>81.6</u>	B
			3	77.1	D
	VI	T-N4-225S-SC2-S8	1	66.8	D
			2	<u>71.0</u>	B
			3	72.7	C
300	I	T-N4-300S-SC2-S1	1	76.1	D
			2	76.3	D
			3	<u>81.8</u>	C
	II	T-N4-300S-SC2-S2	1	60.8	B
			2	<u>62.5</u>	D
			3	63.9	C
	III	T-N4-300S-SC2-S3	1	68.6	D
			2	75.6	B
			3	72.7	D
	IV	T-N4-300S-SC2-S4	1	59.3	D
			2	<u>65.7</u>	B
			3	63.6	C
	V	T-N4-300S-SC3-S4	1	<u>65.8</u>	C
			2	61.5	B
			3	66.5	D

* Note: Underlined values are the failure load of the first bar failed in the group

** From Figure 4-1

Table 4-2: Test results of double # 4 GFRP group bars with straight end

Spacing (mm)	Specimen set	Designation	Bar #	Failure load * (kN)	Mode of failure **
150	I	D-N4-150S-SD1-S4	1	67.1	B
			2	<u>65.5</u>	C
	II	D-N4-150S-SD1-S5	1	<u>56.8</u>	C
			2	61.2	B
	III	D-N4-150S-SD1-S6	1	<u>67.6</u>	C
			2	63.0	D
	IV	D-N4-150S-SD1-S7	1	51.3	C
			2	<u>55.2</u>	C
	V	D-N4-150S-SD1-S8	1	71.6	B
			2	<u>61.0</u>	C
225	I	D-N4-225S-SD1-S1	1	66.8	D
			2	<u>68.8</u>	C
	II	D-N4-225S-SD1-S2	1	48.3	C
			2	<u>49.9</u>	D
	III	D-N4-225S-SD1-S3	1	59.6	B
			2	<u>63.7</u>	C
	IV	D-N4-225S-SD2-S5	1	<u>63.6</u>	C
			2	61.0	B
	V	D-N4-225S-SD2-S6	1	<u>65.8</u>	C
			2	70.3	D
300	I	D-N4-300S-SD3-S8	1	39.5	Grip Slippage
			2	40.1	Grip Slippage
	II	D-N4-300S-SD2-S2	1	43.3	Grip Slippage
			2	35.3	Grip Slippage
	III	D-N4-300S-SD2-S1	1	52.8	D
			2	<u>60.0</u>	C
	IV	D-N4-300S-SD2-S3	1	55.3	C
			2	<u>57.7</u>	D
	V	D-N4-300S-SD2-S4	1	<u>55.1</u>	D
			2	47.1	C

* Note: Underlined values are the failure load of the first bar failed in the group

** From Figure 4-1

Table 4-3: Test results of triple #5 GFRP group bars with headed end

Spacing (mm)	Specimen set	Designation	Bar #	Failure load* (kN)	Mode of failure**
150	I	T-N5-150S-SC5-S1	1	115.8	A
			2	<u>102.8</u>	A
			3	101.3	A
	II	T-N5-150S-SC5-S3	1	132.0	B
			2	<u>117.8</u>	B
			3	98.0	B
	III	T-N5-150S-SC5-S5	1	140.5	B
			2	<u>133.5</u>	B
			3	101.5	B
	IV	T-N5-150S-SC5-S2	1	115.3	C
			2	<u>139.3</u>	B
			3	83.0	B
	V	T-N5-150S-SC5-S4	1	132.6	C
			2	<u>137.5</u>	B
			3	110.9	B
225	I	T-N5-225S-SC4-S1	1	<u>120.1</u>	C
			2	113.4	B
			3	113.5	B
	II	T-N5-225S-SC4-S3	1	109.9	C
			2	123.4	B
			3	<u>128.3</u>	C
	III	T-N5-225S-SC5-S6	1	121.1	C
			2	<u>130.0</u>	B
			3	136.1	C
	IV	T-N5-225S-SC4-S2	1	142.1	D
			2	<u>138.5</u>	B
			3	131.7	B
	V	T-N5-225S-SC4-S4	1	129.6	B
			2	<u>151.6</u>	B
			3	142.6	B
300	I	T-N5-300S-SC4-S5	1	138.1	C
			2	<u>130.0</u>	F
			3	147.0	C
	II	T-N5-300S-SC3-S1	1	124.1	B
			2	<u>101.0</u>	F
			3	161.3	B
	III	T-N5-300S-SC3-S3	1	<u>157.4</u>	C
			2	140.1	B
			3	142.9	C
	IV	T-N5-300S-SC3-S2	1	159.4	B
			2	<u>137.5</u>	C
			3	147.0	B
	V	T-N5-300S-SC4-S6	1	112.11	B
			2	115.11	C
			3	<u>128.05</u>	C

* Note: Underlined values are the failure load of the first bar failed in the group

** From Figure 4-1

Table 4-4: Test results of double # 5 GFRP group bars with headed end

Spacing (mm)	Specimen set	Designation	Bar #	Failure load* (kN)	Mode of failure**
150	I	D-N5-150S-SD4-S5	1	<u>124.8</u>	B
			2	142.0	B
	II	D-N5-150S-SD3-S1	1	<u>131.0</u>	C
			2	129.3	C
	III	D-N5-150S-SD4-S7	1	87.0	C
			2	<u>134.3</u>	B
	VI	D-N5-150S-SD4-S6	1	135.8	D
			2	<u>152.8</u>	B
	V	D-N5-150S-SD4-S8	1	89.3	C
			2	<u>130.5</u>	C
	VI	D-N5-150S-SD3-S2	1	98.5	B
			2	<u>124.8</u>	C
225	I	D-N5-225S-SD4-S1	1	92.0	B
			2	<u>113.5</u>	B
	II	D-N5-225S-SD4-S3	1	<u>136.8</u>	B
			2	149.5	C
	III	D-N5-225S-SD4-S2	1	124.3	B
			2	<u>145.0</u>	C
	IV	D-N5-225S-SD4-S4	1	134.0	B
			2	<u>151.3</u>	C
300	I	D-N5-300S-SD3-S3	1	131.0	C
			2	<u>141.5</u>	B
	II	D-N5-300S-SD3-S5	1	117.5	B
			2	<u>146.8</u>	B
	III	D-N5-300S-SD3-S7	1	128.0	D
			2	<u>127.3</u>	C
	IV	D-N5-300S-SD3-S4	1	144.9	C
			2	<u>134.8</u>	D
	V	D-N5-300S-SD3-S6	1	130.6	B
			2	<u>131.7</u>	B

* Note: Underlined values are the failure load of the first bar failed in the group

** From Figure 4-1

Table 4-5 extends the test results for #4 ribbed-surface GFRP bar groups by calculating the average as well as the characteristic bond strength of the bar. The average bond stress recorded in Table 4-5 was calculated as follows:

$$\tau = \frac{F_u}{\pi d_b h_e} \quad (4.1)$$

where, τ =average bond stress (MPa), F_u = failure load (N), d_b = bar diameter (mm) and h_e = embedment depth (mm). The nominal bar diameters of 12 mm was used for d_b and h_e was set to be the nominal embedment depth of 200 mm. The average bond stress is calculated for each bar group for different bar spacing. The coefficient of variation (CV) is also given in Table 4-5 which shows the discrepancy in experimental results.

The experimental results of the pullout tests are also reported in the form of the characteristic values for design purposes. That is, by definition, a statistical value of the experimental results with 95% confidence. The characteristic value of the strength takes into account the number of identical samples tested as well as the deviation of each strength value from the average strength of all tested identical samples. Three different equations were used to calculate characteristic values given in Table 4-5. The Canadian Specification for fibre-reinforced polymers, CSA-S807-10 (CSA, 2010) specifies the following equation to determine the characteristic values of group of identical GFRP samples.

$$\text{Characteristic value (CSA, 2010)} = \bar{F}_{test} \left(\frac{1 - 1.645C_v}{1 + \frac{1.645C_v}{\sqrt{n}}} \right) \quad (4.2)$$

where $\bar{F}_{test}$ is the mean value of the test results, C_v is the coefficient of variation of the test results, and n is the number of samples.

On the other hand, Equation 4.3 is specified to calculate characteristic values according to ES-ICC (2008):

$$\text{Characteristic value (ES-ICC, 2008)} = \bar{F}_{test} (1 - KC_v) \quad (4.3)$$

where K is a constant depending on the number of samples tested in each group. Corresponding to the number of tests done in this research, K is equal to 5.311, 3.957, 3.400, 3.092 for 3, 4, 5, and 6 numbers of identical tests, respectively (ES-ICC, 2008).

Equation 4.4 is used to calculate characteristic values according to EN (2006) for timber structures:

$$\text{Characteristic value (EN, 2006)} = \exp(\bar{y} - k_s s_y) \quad (4.4)$$

where $\bar{y}$ and s_y are obtained with:

$$\bar{y} = \frac{1}{n} \sum_{i=1}^n (\ln m_i) \quad (4.4 \text{ a})$$

$$s_y = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (\ln m_i - \bar{y})^2} \quad (4.4 \text{ b})$$

where m_i is the test value and n is the number of tests. In Equation 4.4, k_s is a constant depending on number of samples that is equal to 3.15, 2.46, and 2.10 for 3, 5, and 10 number of samples. For other number of tests, interpolation can be used to obtain the corresponding k_s factor.

Table 4-5: Summary of test results for #4 GFRP bars with ribbed surface

Description	Group type	Nominal spacing	Average bond stress	Coefficient of variation	Characteristic bond strength using CSA (2010)	Characteristic bond strength using EN (2006)	Characteristic bond strength using ICC-ES (2008)
		(mm)	(MPa)	%	(MPa)	(MPa)	(MPa)
Values obtained from the average of each group bars	Triple bars	150	8.79	12.32	6.36	6.41	5.11
		225	9.25	11.49	6.96	6.90	5.63
		300	9.03	10.10	7.01	7.05	5.93
	Double bars	150	8.22	9.30	6.52	6.46	5.62
		225	8.19	12.46	5.97	5.87	4.72
		300	7.25	5.66	6.24	6.28	5.85
Values obtained from the first bar failed in each group bars	Triple bars	150	9.02	6.54	7.64	7.64	7.02
		225	9.40	8.22	7.71	7.71	6.77
		300	9.32	11.53	6.96	7.03	5.67
	Double bars	150	8.12	8.75	6.53	6.52	5.70
		225	8.27	11.69	6.15	6.04	4.98
		300	7.63	4.25	6.83	6.87	6.53

Table 4-6: Summary of test results for #5 GFRP bar with ribbed surface and headed ends

Description	Group type	Nominal spacing	Average ultimate load	Coefficient of variation	Characteristic bond strength using CSA (2010)	Characteristic bond strength using EN (2006)	Characteristic bond strength using ICC-ES (2008)
		(mm)	(kN)	%	(kN)	(kN)	(kN)
Values obtained from the average of each group bars	Triple bars	150	117.44	7.33	98.00	97.80	88.18
		225	128.80	8.43	104.45	104.26	91.87
		300	136.07	9.19	108.18	107.62	93.54
	Double bars	150	123.31	11.83	92.00	91.94	73.71
		225	130.78	14.60	88.73	88.00	65.88
		300	133.40	3.54	122.44	122.24	117.34
Values obtained from the first bar failed in each group bars	Triple bars	150	129.01	11.60	96.18	94.12	78.12
		225	133.72	8.95	106.99	107.33	93.03
		300	138.24	9.71	107.58	109.33	92.61
	Double bars	150	133.00	7.80	110.17	110.37	97.73
		225	136.63	12.09	99.55	99.47	80.45
		300	136.40	5.69	118.65	118.49	109.99

Average bond stress is calculated based on Equation 4.1 and presented in Figure 4-5 to Figure 4-8. These graphs show the ultimate bond stress versus bar spacing for #4 triple and double bar groups. The first two graphs (Figure 4-5 and Figure 4-6) are based on average bond stress of all bars and the second two graphs (Figure 4-7 and Figure 4-8) are based on the pullout strength of the bar that failed first in each group. One may observe that the scattered test results with increase of bar spacing whether the average or characteristic values are considered. So, there is no general trend to observe with the change in bar spacing. However, by considering the failure modes of each bar group, one may observe that there was no interaction of the failure mode among the tested bars in a bar group. Thus, it can be concluded that bar spacing has insignificant effect on the bond strength of GFRP bars provided that bar spacing ranges from 150 to 300 mm.

Figure 4-5: Average and characteristic bond stress for # 4 ribbed triple group bar based on average recorded load in each group

Figure 4-6: Average and characteristic bond stress for # 4 ribbed double group bar based on average recorded load in each group

Figure 4-7: Average and characteristic bond stress # 4 ribbed triple group bar based on the first bar failed in each group

Figure 4-8: Average and characteristic bond stress for # 4 ribbed double group bar based on the first bar failed in each group

4.3 Comparison of experimental results with available bond strength equations

Figure 4-9 to Figure 4-12 are presented to compare the average bond stress of pullout tests with existing models proposed to calculate bond strength. The first two graphs (Figure 4-9 and Figure 4-10) are based on the average bond stress of all bars in the bar group and the second two graphs (Figure 4-11 and Figure 4-12) are based on bond strength of the bar that failed first in each bar group. It can be observed that all results are between the lower and upper bound of the bond stress range proposed by Cook et al. (1993).

Figure 4-9: Comparison of the test results with existing pullout models for #4 ribbed triple group bar

Figure 4-10: Comparison of the test results with existing pullout models for #4 ribbed double group bar

Figure 4-11: Comparison of the test results with existing pullout models for #4 ribbed triple group bar based on first GFRP bars failure

Figure 4-12: Comparison of the test results with existing pullout models for #4 ribbed double group bar based on first GFRP bars failure

In addition, Kim and Smith (2010) presented two formulas for bond strength, one based on concrete conic failure (N_{cc}) and one based on bond failure (N_{cb}) as explained in Chapter 2. Since concrete conic failure did not occur in any of the tests for #4 ribbed bar, the value proposed for conic failure is much higher than the test results in the current study. On the other hand, Kim and Smith's equation for N_{cb} (bond stress) is shown to be in good agreement and compatible with the test results irrespective of bar spacing.

Figure 4-13 to Figure 4-16 demonstrate the pullout load versus bar spacing for #5 ribbed headed bars considering the average and characteristic values. It can generally be observed that increasing the bar spacing has led to ultimate load increase. Since #5 bars are with headed end, the concrete conic stress distribution causes the bars to affect each other in the group much more than the case of #4 bars with straight ends. Therefore, if the bars are placed closely (i.e. 150 mm spacing), the served areas of concrete conic stress overlap thus leading to a reduction in the global pullout strength of group anchors. If the distance between bars increases, the overlapping effect decreases and the ultimate pullout capacity increases.

Test results are also compared with the existing pullout strength models for #5 headed-end bars in Figure 4-17 through Figure 4-20. These models include those proposed by ACI 318M (2011), Kim and Smith (2010), Eligehausen et al. (2006) and ACI 349 (1985). It should be noted that none of these models include the effect bar spacing on the pullout capacity.

Figure 4-13: Average and characteristic ultimate load for # 5 ribbed triple group bar based on average recorded load in each group

Figure 4-14: Average and characteristic ultimate load for # 5 ribbed double group bar based on average recorded load in each group

Figure 4-15: Average and characteristic ultimate load for # 5 ribbed triple group bars based on the first bar failures in each group

Figure 4-16: Average and characteristic ultimate load for # 5 ribbed double group bar based on the first bar failures in each group

Figure 4-17: Comparison of the test results with existing pullout models for #5 ribbed triple group bar

Figure 4-18: Comparison of the test results with existing pullout models for #5 ribbed double group bar

Figure 4-19: Comparison of the test results with existing pullout models for #5 ribbed triple group bar based on first GFRP bar failure

Figure 4-20: Comparison of the test results with existing pullout models for #5 ribbed double group bar based on first GFRP bar failure

It can be observed that ACI 318M and Kim and Smith models showed good agreement with experimental findings based on the average pullout capacity. On the other hand, Eligehausen et al. and ACI 349 formulas significantly overestimate the pullout strength of the GFRP bars. This may be attributed to the fact that their formulas were developed based on full concrete conic mode of failure while the tested GFRP bar groups failed mainly due to either full bond failure or combined bond and conic failure.

4.4 Summary

Pullout test results of the first phase of study were presented and discussed in this chapter. Pre-installed (cast-in-place) group anchors of two and three bars were tested to collapse to determine their pullout capacity and failure mode. Effect of bar spacing was considered by introducing three different bar spacing, namely: 150, 225 and 300 mm. It is shown that increasing the bar spacing from 150 to 225 mm has led to an increase in the average ultimate bond stress for #4 ribbed bars. However further increase in bar spacing from 225 to 300 mm has no noticeable effect. Comparison between test results and previously proposed models to estimate pullout anchor failure loads for #4 ribbed bars demonstrated that the ultimate failure load is in good agreement with proposed equation by Kim and Smith (2010) for bond stress. Also, experimental findings showed that for #5 headed-end bar groups, increasing the bar spacing has led to ultimate load augmentation since the concrete conic stress distribution causes the bars to affect each other significantly in the group with decrease in bar spacing. When the distance between bars are increased, group effect decreases and the ultimate pullout capacity increases. Comparison of the test results with the existing formulas for the predication of pullout

capacity of #5 headed-end bar showed that those proposed by ACI 318M and Kim and Smith are in good agreement with pullout strength extracted from the current experimental study.

Chapter 5 Experimental results and discussion of Phase II

In this chapter, the results of the second phase (Phase II) of the experimental study are presented and discussed which are in accordance with the pullout capacity of post-installed GFRP bars. The modes of failure are discussed followed by results presented in tables and figures to investigate the effect of two types of adhesives used, bar diameter and coating, and also the embedment depths. Results are also compared with other experimental studies and also selected available models to estimate pullout capacity.

5.1 Mode of failure

Six possible modes of failure were possible in the pullout tests in this phase. Schematic pictures of these modes are presented in Figure 5-1 including concrete cone breakout (type a), adhesive/concrete interface pullout (type b), GFRP/adhesive interface pullout (type c), pull through at adhesive/concrete and GFRP (type d), sand-coated or ribbed interface pullout (type e), and bar rupture (type f). In addition, Figure 5-2 shows five main modes of failures observed during the test. The views after failure for all bars are given in Appendix C

Figure 5-1: Possible failure modes of the post-installed bars

a) Concrete cone breakout; b) Adhesive/concrete interface pullout; c) GFRP/adhesive interface pullout; d) Pull through (adhesive/concrete and GFRP; e) Sand coated or ribbed interface pullout; and f) Bar rupture

Figure 5-2: Recorded failure modes of the post-installed GFRP bars

In addition, Figure 5-5 to Figure 5-8 compare modes of failure for different bar sizes and types (sand-coated and ribbed) observed during experiments. It could be concluded from the figures that regardless of adhesive or bar type and size, no cone failure (type “a” failure) is observed in any of the performed pull-out tests. It is due to the relatively large embedment depths used in the experiments. Figure 5-5 is specifically demonstrating the modes of failure for #4 ribbed bars. It shows that most anchors have failed in either bar rupture or GFRP/adhesive

interface pullout failure. Bar rupture happens for cases with embedment depth of more than 150 mm showing that such embedment depths are enough to develop full bar capacity.

Figure 5-6 shows the modes of failure for #5 ribbed GFRP bars. In this case, however, types of failure for most anchors were observed to be either “GFRP/adhesive interface pullout” or “adhesive/concrete interface pullout”. Figure 5-7 depicts the modes of failure for #5 sand-coated anchors. It shows that failure occurs mostly at the interface of concrete and adhesive (type b). This is because of the strong bond between the sand-coated surface of the anchor and the adhesive. In addition to that, sand-coated or ribbed interface pullout failure was also occurred in 9 cases. The same chart is presented for #6 bars in Figure 5-8. It can be observed that in #6 sand coated bars, 25 out of 29 samples failed at the concrete-adhesive interface which, as mentioned before, is due to the strong bond between the sand-coated surface and the adhesives used.

5.2 Experimental results and discussions

Figure 5-3 demonstrates the experimental load-displacement obtained for five identical samples tested for #5 sand-coated GFRP bars with Type I adhesive and 150 mm embedment depth which are mainly linear up to failure due to linear elongation in the GFRP bars. Appendix C shows the load-displacement graphs for all specimen subjected to pullout test. The test results showing the failure load and modes of failure for each bar size and coating are presented separately in Table 5-1 to Table 5-4.

Figure 5-3: Pullout load-slip relationship for #5 sand-coated GFRP bar embedded 150 mm in concrete with adhesive Type I

Figure 5-4: Split views of core samples for post-installed sand-coated and ribbed bars with embedment depth of 150 mm anchored (a) V-#5-150-H-S9-SA6, and (b) S-#5-150-S-S4-SA4

Figure 5-5: Comparison of modes of failure for #4 ribbed GFRP bars

Figure 5-7: Comparison of modes of failure for #5 sand-coated GFRP bars

Figure 5-6: Comparison of modes of failure for #5 ribbed GFRP bars

Figure 5-8: Comparison of modes of failure for #6 sand-coated GFRP bars

Table 5-1: Test results of #4 ribbed GFRP post-installed bars

Test No.	Specimen designation	Failure load (kN)	Type of failure
1	S-#4-100-H-S1-SA1	76.08	Pull out (bar-adhesive)
2	S-#4-100-H-S2-SA1	53.55	Pull out (bar-adhesive)
3	S-#4-100-H-S4-SA1	75.08	Pull out (bar-adhesive)
4	S-#4-100-H-S6-SA1	79.58	Pull out (bar-adhesive)
5	S-#4-100-H-S8-SA1	72.07	Pull out (bar-ribbed)
6	S-#4-100-S-S3-SA1	74.32	Pull out (bar-adhesive)
7	S-#4-100-S-S5-SA1	82.83	Pull out (bar- ribbed)
8	S-#4-100-S-S7-SA1	80.08	Pull out (bar-adhesive)
9	S-#4-100-S-S9-SA1	82.33	Pull through (bar-adhesive)
10	S-#4-100-S-S10-SA1	84.83	Pull out (bar-adhesive)
11	S-#4-150-H-S1-SA2	91.59	Pull out (bar-adhesive)
12	S-#4-150-H-S1-SA3	97.52	Pull out (bar-adhesive)
13	S-#4-150-H-S2-SA3	99.85	Pull out (bar-adhesive)
14	S-#4-150-H-S4-SA3	104.35	Pull out (bar- ribbed)
15	S-#4-150-H-S6-SA3	98.10	Pull out (bar-adhesive)
16	S-#4-150-S-S2-SA2	95.35	Bar ruptures
17	S-#4-150-S-S3-SA3	92.84	Bar ruptures
18	S-#4-150-S-S5-SA3	91.34	Bar ruptures
19	S-#4-150-S-S7-SA3	89.53	Bar ruptures
20	S-#4-150-S-S8-SA3	89.09	Pull out (bar-adhesive)
21	S-#4-200-H-S1-SA4	98.85	Pull out (bar-adhesive)
22	S-#4-200-H-S3-SA4	96.35	Bar ruptures
23	S-#4-200-H-S5-SA4	100.60	Bar ruptures
24	S-#4-200-H-S7-SA4	96.85	Bar ruptures
25	S-#4-200-H-S9-SA4	99.35	Pull out (bar-adhesive)
26	S-#4-200-S-S3-SA5	101.82	Bar ruptures
27	S-#4-200-S-S5-SA5	99.56	Bar ruptures
28	S-#4-200-S-S7-SA5	100.35	Bar ruptures
29	S-#4-200-S-S8-SA5	92.59	Bar ruptures
30	S-#4-200-S-S11-SA5	100.60	Bar ruptures

Table 5-2: Test results of #5 ribbed GFRP post-installed bars

Test No.	Specimen designation	Failure load (kN)	Type of failure
31	S-#5-150-H-S8-SA4	136.89	Pull out (adhesive-concrete)
32	S-#5-150-H-S10-SA4	155.66	Pull out (bar-adhesive)
33	S-#5-150-H-S11-SA4	128.88	Pull out (adhesive-concrete)
34	S-#5-150-H-S12-SA4	167.67	Pull out (adhesive-concrete)
35	S-#5-150-H-S13-SA4	163.16	Pull out (adhesive-concrete)
36	S-#5-150-S-S2-SA4	151.15	Pull out (bar-adhesive)
37	S-#5-150-S-S4-SA4	161.91	Pull out (bar-adhesive)
38	S-#5-150-S-S6-SA4	149.90	Pull out (adhesive-concrete)
39	S-#5-150-S-S10-SA5	134.13	Pull out (bar-adhesive)
40	S-#5-150-S-S12-SA5	122.37	Pull out (adhesive-concrete)
41	S-#5-200-H-S3-SA6	135.39	GRIP slippage / Pull through
42	S-#5-200-H-S5-SA6	175.43	Pull out (adhesive-concrete)
43	S-#5-200-H-S7-SA6	166.42	Pull out (adhesive-concrete)
44	S-#5-200-H-S8-SA6	167.92	Bar ruptures
45	S-#5-200-H-S10-SA6	129.88	GRIP slippage
46	S-#5-200-S-S1-SA6	143.14	Pull out (bar-adhesive)
47	S-#5-200-S-S3-SB6	161.91	Pull out (bar-adhesive)
48	S-#5-200-S-S5-SB6	150.40	Pull out (bar-adhesive)
49	S-#5-200-S-S7-SB6	148.40	Pull out (bar-adhesive)
50	S-#5-200-S-S9-SB6	158.91	Pull out (bar-adhesive)
51	S-#5-250-H-S1-SB1	154.15	Pull through
52	S-#5-250-H-S2-SB1	123.12	GRIP slippage
53	S-#5-250-H-S3-SB1	160.41	Pull out (bar-adhesive)
54	S-#5-250-H-S4-SB1	151.40	Bar ruptures
55	S-#5-250-H-S4-SB2	162.16	Pull out (adhesive-concrete)
56	S-#5-250-S-S1-SB2	161.41	Pull out (bar-adhesive)
57	S-#5-250-S-S2-SB2	128.63	GRIP slippage
58	S-#5-250-S-S3-SB2	158.41	Pull out (bar-adhesive)/Bar rupture
59	S-#5-250-S-S5-SB3	144.89	Pull out (adhesive-concrete)
60	S-#5-250-S-S6-SB3	152.65	Pull out (adhesive-concrete)

Table 5-3: Test results of #5 sand-coated GFRP post-installed bars

Test No.	Specimen designation	Failure load (kN)	Type of failure
61	V-#5-150-H-S7-SB5	154.90	Pull out (adhesive-concrete)
62	V-#5-150-H-S9-SA6	143.89	Pull out (adhesive-concrete)
63	V-#5-150-H-S9-SB5	172.92	Pull out (adhesive-concrete)
64	V-#5-150-H-S11-SA6	165.42	Pull out (bar-sand coated)
65	V-#5-150-H-S12-SA6	160.91	Pull out (adhesive-concrete)
66	V-#5-150-S-S1-SA5	103.60	Pull out (adhesive-concrete)
67	V-#5-150-S-S2-SA5	146.40	Pull out (bar-sand coated)
68	V-#5-150-S-S4-SA5	161.91	Pull out (bar-sand coated)
69	V-#5-150-S-S6-SA5	137.64	Pull out (adhesive-concrete)
70	V-#5-150-S-S9-SA5	138.64	Pull out (adhesive-concrete)
71	V-#5-200-H-S2-SB4	140.89	Pull out (bar-sand coated)
72	V-#5-200-H-S3-SB4	147.90	Pull out (bar-sand coated)
73	V-#5-200-H-S4-SB4	222.22	Pull out (bar-sand coated)
74	V-#5-200-H-S5-SB4	167.42	Pull out (bar-sand coated)
75	V-#5-200-H-S7-SB4	170.17	Pull out (adhesive-concrete)
76	V-#5-200-S-S2-SB3	159.91	Pull out (bar-sand coated)
77	V-#5-200-S-S3-SB3	164.91	Pull out (adhesive-concrete)
78	V-#5-200-S-S4-SB3	152.65	Pull out (adhesive-concrete)
79	V-#5-200-S-S8-SB4	116.62	GRIP slippage
80	V-#5-200-S-S9-SB4	159.16	Pull out (adhesive-concrete)
81	V-#5-250-H-S5-SE	180.18	Pull out (adhesive-concrete)
82	V-#5-250-H-S10-SE	201.45	Pull out (adhesive-concrete)
83	V-#5-250-H-S12-SE	137.64	Pull out (adhesive-concrete)
84	V-#5-250-H-S15-SE	165.17	Pull out (adhesive-concrete)
85	V-#5-250-H-S17-SE	137.89	Pull out (bar-sand coated)
86	V-#5-250-S-S2-SE	156.16	Pull out (adhesive-concrete)
87	V-#5-250-S-S3-SE	149.15	Pull out (bar-sand coated)
88	V-#5-250-S-S7-SE	120.37	GRIP slippage
89	V-#5-250-S-S8-SE	200.95	Pull out (adhesive-concrete)
90	V-#5-250-S-S11-SE	175.93	Pull out (adhesive-concrete)

Table 5-4: Test results of #6 sand-coated GFRP post-installed bars

Test No.	Specimen designation	Failure load kN	Type of failure
91	V-#6-150-H-S1-SB3	140.39	Pull out (adhesive-concrete)
92	V-#6-150-H-S1-SB4	96.60	Pull out (bar-adhesive)
93	V-#6-150-H-S5-SB1	105.86	Pull through
94	V-#6-150-H-S6-SB4	137.14	Pull out (adhesive-concrete)
95	V-#6-150-H-S10-SB4	131.38	Pull out (adhesive-concrete)
96	V-#6-150-S-S1-SB6	162.91	Pull out (adhesive-concrete)
97	V-#6-150-S-S2-SA6	133.13	Pull out (adhesive-concrete)
98	V-#6-150-S-S2-SB6	155.66	Pull out (adhesive-concrete)
99	V-#6-150-S-S4-SA6	164.66	Pull out (adhesive-concrete)
100	V-#6-150-S-S6-SA6	147.40	Pull out (adhesive-concrete)
101	V-#6-200-H-S1-SB5	141.89	Pull out (adhesive-concrete)
102	V-#6-200-H-S3-SB5	240.49	Pull out (adhesive-concrete)
103	V-#6-200-H-S5-SB5	226.48	Pull out (adhesive-concrete)
104	V-#6-200-H-S6-SB5	200.95	Pull out (adhesive-concrete)
105	V-#6-200-H-S8-SB5	176.18	Pull out (adhesive-concrete)
106	V-#6-200-S-S4-SB5	216.47	Pull out (adhesive-concrete)
107	V-#6-200-S-S2-SB5	185.44	Pull out (adhesive-concrete)
108	V-#6-200-S-S8-SB6	154.40	Pull out (adhesive-concrete)
109	V-#6-200-S-S6-SB6	128.63	GRIP slippage
110	V-#6-200-S-S4-SB6	201.45	Pull out (adhesive-concrete)
111	V-#6-250-H-S4-SE	184.18	Pull out (bar-sand coated)
112	V-#6-250-H-S9-SE	184.43	Pull out (bar-sand coated)
113	V-#6-250-H-S13-SE	221.72	Pull out (adhesive-concrete)
114	V-#6-250-H-S14-SE	224.22	Pull out (adhesive-concrete)
115	V-#6-250-H-S16-SE	220.97	Pull out (adhesive-concrete)
116	V-#6-250-S-S1-SF1	204.2	Pull out (adhesive-concrete)
117	V-#6-250-S-S2-SF1	221.47	Pull out (adhesive-concrete)
118	V-#6-250-S-S3-SF1	215.22	Pull out (adhesive-concrete)
119	V-#6-250-S-S1-SE	209.21	Pull out (adhesive-concrete)
120	V-#6-250-S-S6-SE	169.17	Pull out (adhesive-concrete)

Table 5-5 and Table 5-6 present the summary of test results for obtained ultimate loads and bond stress, respectively. These tables include average values, coefficient of variations, and characteristic values for every bar size and adhesive type, considering different embedment depths. Characteristic values are calculated using equations presented in Chapter 4.

Figure 5-9 and Figure 5-10 illustrate the average failure load considering different adhesive types and embedment depths. It can be noted from both graphs that the average ultimate load increases by increasing the embedment depth. However, they illustrate that increasing the embedment depth from 200 mm to 250 mm has less influence on ultimate load capacity compared to embedment depth increment from 150 to 200 mm. In other words, the slopes of the graphs are decreased by embedment depth increase. Consequently, it can be concluded that 200 mm embedment depth which is considered to be the current practice and recommended by MTO is the most efficient embedment depth in terms of pullout capacity.

Figure 5-11 and Figure 5-12 show the characteristic failure loads versus embedment depth for specimens considering adhesive type, embedment depth and bar type and size. Each figure shows the value for an adhesive type based on a specific formula for characteristic value. As can be seen, the characteristic value based on CSA (2010), shows the same trend with average pullout capacity values except for #5 sand-coated bars. This is due to the fact that characteristic value formulas are dependent on coefficient of variation. The more coefficient of variation, the less characteristic value is obtained since there are more discrepancies in the experimental results. For #5 sand-coated bar with adhesive Type I, characteristic value has decreased for 200 and 250 mm embedment depths because of discrepancy of pullout test results.

Table 5-5: Pullout test results for post-installed GFRP bars based on average ultimate load and reliability analysis

Bar type	Adhesive type	Embedment depth (mm)	Average ultimate load (kN)	Coefficient of variation %	Characteristic ultimate load		
					CSA (2010) (kN)	EN (2006) (kN)	ICC-ES (2008) (kN)
#6 Sand-coated GFRP bars	Adhesive TYPE I	150	122.3	16.2	80.2	80.0	55.1
		200	197.2	20.0	115.2	115.3	62.8
		250	207.1	10.1	160.9	160.2	136.2
	Adhesive TYPE II	150	152.8	8.5	123.8	123.0	108.8
		200	189.4	14.0	130.6	131.4	99.1
		250	203.9	10.0	158.5	156.3	134.4
#5 Sand-coated GFRP bars	Adhesive TYPE I	150	159.6	6.9	134.7	134.2	122.3
		200	169.7	18.8	103.0	108.3	61.3
		250	164.5	16.8	106.0	107.8	70.7
	Adhesive TYPE II	150	137.6	15.5	92.0	90.5	65.0
		200	159.2	3.2	147.1	147.1	142.0
		250	170.5	13.6	119.0	122.0	91.6
#5 Ribbed GFRP bars	Adhesive TYPE I	150	150.5	11.2	113.4	113.0	93.2
		200	169.9	2.8	157.7	158.5	153.5
		250	157.0	3.2	144.8	144.9	139.7
	Adhesive TYPE II	150	143.9	10.8	109.5	109.0	90.9
		200	152.6	5.1	134.8	134.6	126.3
		250	154.3	4.7	137.1	137.2	129.6
#4 Ribbed GFRP bars	Adhesive TYPE I	100	71.3	14.4	49.2	47.8	36.4
		150	98.3	4.7	87.7	87.5	82.6
		200	98.4	1.8	94.2	94.1	92.4
	Adhesive TYPE II	100	80.9	5.0	71.6	71.3	67.2
		150	91.6	2.8	85.7	85.5	82.9
		200	99.0	3.7	90.5	90.1	86.5

Table 5-6: Pullout test results for post-installed GFRP bars based on average bond stress

Bar type	Adhesive type	Embedment depth (mm)	Average bond stress (MPa)	Coefficient of variation %	Characteristic bond stress		
					CSA (2010) (MPa)	EN (2006) (MPa)	ICC-ES (2008) (MPa)
#6 Sand-coated GFRP bars	Adhesive TYPE I	150	13.6	16.2	8.9	8.9	6.1
		200	16.5	20.0	9.6	9.6	5.2
		250	13.8	10.1	10.8	10.7	9.1
	Adhesive TYPE II	150	17.0	8.5	13.8	13.7	12.1
		200	15.8	14.0	10.9	11.0	8.3
		250	13.6	10.0	10.6	10.4	9.0
#5 Sand-coated GFRP bars	Adhesive TYPE I	150	21.3	6.9	18.0	17.9	16.3
		200	17.0	18.8	10.3	10.9	6.1
		250	13.2	16.8	8.5	8.6	5.7
	Adhesive TYPE II	150	18.4	15.5	12.3	12.1	8.7
		200	16.0	3.2	14.7	14.7	14.2
		250	13.7	13.6	9.5	9.8	7.3
#5 Ribbed GFRP bars	Adhesive TYPE I	150	20.0	11.2	15.0	15.0	12.4
		200	16.9	2.8	15.7	15.8	15.3
		250	12.5	3.2	11.5	11.5	11.1
	Adhesive TYPE II	150	19.1	10.8	14.5	14.5	12.1
		200	15.2	5.1	13.4	13.4	12.6
		250	12.3	4.7	10.9	10.9	10.3
#4 Ribbed GFRP bars	Adhesive TYPE I	100	18.9	14.4	13.0	12.7	9.6
		150	17.4	4.7	15.5	15.5	14.6
		200	13.1	1.8	12.5	12.5	12.3
	Adhesive TYPE II	100	21.5	5.0	19.0	18.9	17.8
		150	16.2	2.8	15.1	15.1	14.7
		200	13.1	3.7	12.0	12.0	11.5

Figure 5-9: Comparison of average ultimate load vs. embedment depth for specimens with adhesive Type I

Figure 5-10: Comparison of average ultimate load vs. embedment depth for specimens with adhesive Type II

Figure 5-11: Comparison of characteristic ultimate load (CSA, 2010) vs. embedment depth for specimens with adhesive Type I

Figure 5-12: Comparison of characteristic ultimate load (CSA, 2010) vs. embedment depth for specimens with adhesive Type II

Table 5-6 presents the bond stress value of the pullout test results considering adhesive type, embedment depth, coefficient of variation and three different characteristic values. Bond stress is calculated as the average bond stress using Equation 4-1. Figure 5-13 and Figure 5-14 show the average bond stress versus embedment depth for each adhesive considering different bar types and sizes. It can be observed that increasing the embedment depth, leads to a decrease in bond stress which is in agreement with the material presented in Chapter 2. In addition, Figure 5-15 and Figure 5-16 demonstrate the characteristic bond stress versus embedment depth based on CSA,(2010) formula considering bar types and sizes. In general, theses graphs show a similar relationship for the effect of embedment depth compared to what was presented for average bond stress, but some change in trend is due to the sensibility of the characteristic value to the diversity of test results (i.e. coefficients of variation)

Figure 5-17 to Figure 5-20 show the influence of adhesive type on the average ultimate load with different embedment depths. The main difference in adhesives application was the temperature at the time of installation. For adhesive Type I, the installation was done at the temperature of above 15°C whereas for adhesive Type II, the temperature of concrete was between -5 and 0. Although this was within the practice recommendation of the manufacturer, but low workability of the adhesive at low temperatures and also harder conditions for hole cleaning were the two main reasons that contributed to lower the pullout capacity of Type 2 adhesive. In addition, the adhesive Type II has a lower gel time (10 minutes) compared to the adhesive Type I (30 minutes).

Figure 5-13: Comparison of average bond stress vs. embedment depth for specimens with adhesive Type I

Figure 5-14: Comparison of average bond stress vs. embedment depth for specimens with adhesive Type II

Figure 5-15: Comparison of characteristic bond stress (CSA, 2010) vs. embedment depth for specimens with adhesive Type I

Figure 5-16: Comparison of characteristic bond stress (CSA, 2010) vs. embedment depth for specimens with adhesive Type II

Figure 5-17: Effect of adhesive type on pullout strength of post-installed #6 sand-coated GFRP bars

Figure 5-19: Effect of adhesive type on pullout strength of post-installed #5 ribbed GFRP bars

Figure 5-18: Effect of adhesive type on pullout strength of post-installed #5 sand-coated GFRP bars

Figure 5-20: Effect of adhesive type on pullout strength of post-installed #4 ribbed GFRP bars

5.3 Comparison of Experimental Results with other similar experiment

Figure 5-21 presents average failure load versus (embedment Depth)/(Bar Diameter) ratio comparing the results of this study with previous experiments. It appears that the results are in the same trend in the sense that increasing the embedment depth increases the average ultimate failure loads. In the study performed by Ahmed et al, (2008) the same adhesive type (Type I) and bar (sand-coated) was used. Comparing their results with the current study shows that in the same “embedment depth/bar diameter”, Ahmed et al, (2008) reached much lower pullout strength. The main reason of this large difference is the confinement effect of steel reinforcement in the current study compared to unconfined concrete used by (Ahmed et al, 2008).

5.4 Comparison of experimental results with current design method

Figure 5-24 and Figure 5-25 present the comparison between experimental results and available relationships in the literature for the steel anchorage. Cook (1993) presented two relationships for lower and upper bound range of data (Equation 2.6). As shown in Figures 5-22, the lower bond is way lower than the experimental results of this study. However, the upper bond seems to be within the range of #6 sand coated anchors bond stress. Cook and Kunz (2001) also presented a formula to predict the bond stress (Equation 2.7 which is fairly close to the #6 sand-coated bars and #5 sand coated and ribbed bars with embedment length of 200 and 250 mm. Where Figure 5-22 presents the results for adhesive Type 1, the same conclusions could be drawn for adhesive Type II presented in Figure 5-23.

Figure 5-21: Comparison of experimental average bond stress with those available In the literature for #5 GFRP sand-coated bars

Figure 5-22: Comparison of average bond stress of post-installed GFRP bars with those available in the literature for specimens with adhesive Type I

Figure 5-23: Comparison of average bond stress of post-installed GFRP bars with those available in the literature for specimens with adhesive Type II

Figure 5-24 and Figure 5-25 show the results of pullout tests in terms of ultimate load in comparison with models proposed by Kim and Smith (2010) for predicting the pullout force for GFRP anchors (Equations 2.16 and 2.17). As presented in the graphs, the pullout strength has a direct relationship with embedment length. It can be seen that the presented formulas are compatible with the results of #4 ribbed bars with embedment length of 100 mm and 150 mm and also fairly close to #5 sand-coated and ribbed bars in case of N_{cb} . N_{cc} seems to overestimate the pullout strength for all tested bar types and sizes. However, N_{cb} shows good agreement for 250mm embedment length samples for both adhesive types.

Figure 5-24: Comparison of average ultimate load of post-installed GFRP bars with those available in the literature for specimens with adhesive Type I

Figure 5-25: Comparison of average ultimate load of post-installed GFRP bars with those available in the literature for specimens with adhesive Type II

5.5 Summary

This chapter presented and discussed the results of the second phase of experimental study. This phase is investigating the pullout strength of post-installed single anchors. Two different GFRP bar types (sand-coated and ribbed) and three sizes (#4, 5 and 6) were used in the experiments in addition to two different types of adhesive. Test results showed five different modes of failure. It was also demonstrated that none of the anchors were failed under cone failure category because of the relatively large embedment depths used. It was shown that most anchors failed under GFRP/adhesive or adhesive/concrete interface pullout failure. Majority of

the ribbed GFRP bars failed at GFRP bar- adhesive interface, while in sand-coated bars failure occurred mainly at the adhesive-concrete interface.

Regarding the influence of embedment depth on ultimate load, it was shown that the average ultimate load increases by increasing the embedment depth. It was also concluded that 200 mm embedment depth which is considered to be the current practice and recommended by MTO is the most efficient embedment depth due to insignificant increase of the pullout capacity from 200 mm embedment depth to 250 mm for #5 and #6 bars. . In addition, Table 5-7 presents experimental and design pull-out strength for post-installed GFRP anchorage which could be used by design engineers on rehabilitation of bridge barrier reinforced with GFRP bars. Finally, the test results were compared with previous experimental studies and it was proved that they follow the same trend.

Table 5-7: Design pullout strength for post-installed GFRP bars based on pullout test results of specimens

Bar Type	Adhesive Type	Embedment depth (mm)	Average Ultimate Load (kN)	Characteristic design load S807-10 (kN)
#6 Sand-coated GFRP bars	Adhesive TYPE I (HIT RE 500)	150	122.0	80.0
		200	197.0	115.0
		250	207.0	160.0
	Adhesive TYPE II (Sika AnchorFix®-2001)	150	152.0	123.0
		200	189.0	130.0
		250	203.0	158.0
#5 Sand-coated GFRP bars	Adhesive TYPE I (HIT RE 500)	150	159.0	134.0
		200	169.0	103.0
		250	164.0	106.0
	Adhesive TYPE II (Sika AnchorFix®-2001)	150	137.0	91.0
		200	159.0	147.0
		250	170.0	119.0
#5 Ribbed GFRP bars	Adhesive TYPE I (HIT RE 500)	150	150.0	113.0
		200	169.0	157.0
		250	157.0	144.0
	Adhesive TYPE II (Sika AnchorFix®-2001)	150	143.0	109.0
		200	152.0	134.0
		250	154.0	137.0
#4 Ribbed GFRP bars	Adhesive TYPE I (HIT RE 500)	100	71.0	49.0
		150	98.0	87.0
		200	98.0	94.0
	Adhesive TYPE II (Sika AnchorFix®-2001)	100	80.0	71.0
		150	91.0	85.0
		200	98.0	90.0

Chapter 6 Conclusions and Recommendations for Future Research

6.1 General

Two different types of anchorage systems exist for concrete, namely: post-installed and pre-installed (cast-in-place) anchors. The post-installed anchors are mainly formed by drilling a hole in a hardened concrete and placing the bar inside the hole with an adhesive that acts as a bonding agent between the bar and the concrete. This study aimed at:

- a) Investigating experimentally the bond behavior and pullout capacity of closely spaced GFRP bar group embedded in existing concrete deck slabs (Phase I); and
- b) Investigating experimentally the pullout capacity of post-installed single GFRP bars as affected by GFRP type, size and embedment depths as well as adhesive type (Phase II)

The ultimate goal of this research was to determine the anchorage depth requirements to sustain pullout load resulting from vehicle impact to bridge barriers. This chapter summarizes the conclusions driven from this experimental program, followed by recommendations for future research.

6.2 Phase 1 – Pre-installed GFRP bar group anchorage

This phase aimed to investigate the effect of overlapping of adjacent bars on the pullout capacity of closely-spaced cast-in-place group of bars. Group of two or three GFRP bars embedded in concrete, referred to as double and triple bars, with various bar spacing were constructed and tested to collapse under pullout loads. The size of GFRP bars used were #5 (16-mm diameter) bars with headed-end and #4 (12-mm diameter) bars with straight ends. All GFRP bars had

ribbed surface. An embedment depth of 200 mm was chosen with bar spacing of 150, 225, and 300 mm. A total of 60 samples were erected, 30 samples for double bars and 30 samples for triple bars. The following conclusions can be drawn based on the experimental results of this phase:

- a) Considering the type of failure, the most common type of failure for #5 headed-end bar groups was (i) combined small concrete cone and GFRP/Concrete interface failure (type b) and (ii) combined deep concrete cone and GFRP/concrete interface failure (type c). However, for the #4 straight-end bars, in addition to these two observed common failure modes, bar pull-through/slippage without any noticeable concrete lift up (type d) occurred more often. Concrete cone failure (type a) was only observed for #5 headed end bars in case of bar spacing of 150 mm.
- b) By slicing concrete core samples taken at tested bar locations, it was shown that the failure of headed-end GFRP bars mostly took place around the larger part of the headed end of bars, however, bars with straight-end were observed to be pulled through the concrete.
- c) Due to scattered data for the average bond strength of #4 GFRP ribbed-surface bars, it was not clear whether bar spacing affects the bond strength. However, the type of failure modes and shape of the bar and concrete around the bar when slicing the concrete core samples, it can be concluded that bar spacing has insignificant effect on the bond strength of #4 GFRP bars. However, experimental findings for #5 ribbed-surface headed-end bars showed that the bond strength increases with increase in bar spacing. This may be attributed to the fact that the areas of concrete conic stress distribution overlap with

closer bar spacing, leading to a reduction in the total pullout strength of the group anchors.

- d) Comparison of the average bond stress in pullout test results for #4 ribbed-surface bars with straight ends with existing models to predict bond stress showed that test results are between the lower and upper bound of the range proposed by Cook et al (1993). The equation proposed by Kim and Smith (2010) for N_{cb} (bond stress) was proven to be in good agreement with the test results.
- e) For #5 headed-end ribbed-surface bars, the predication models of pullout strength proposed by ACI (2011) and Kim and Smith (2010) are shown to be quite close to the experimental pullout capacity of both triple and double bar groups. However, the formulas presented by Eligehausen et al. (2006) and ACI (1985) overestimate the pullout strength mainly since they were developed for concrete conic mode of failure all over the bar embedment depth.

6.3 Phase 2 – Post-installed anchorage

The pullout behavior of post installed adhesive anchors was studied in this phase. The objective was to evaluate two types of adhesives and GFRP bars in two different sizes each. Both ribbed and sand-coated GFRP bars were post-installed with an embedment depth of 100, 150, and 200 for #4 ribbed bars and 150, 200, and 250 mm for #5 ribbed/sand-coated, and also #6 (i.e. 19 mm diameter) sand-coated bars. Two different commercial adhesives were used to attach the anchors to the concrete. The following conclusions can be drawn based on the experimental results of this phase:

- a) Test results showed that regardless of adhesive or bar type and size, no cone failure (type “a” failure) was observed in any of the performed pullout tests. It is due to the relatively large embedment depths used in the experiments. Failure mode for #4 ribbed bars were mostly either bar rupture or GFRP/adhesive interface pullout failure. Bar rupture occurred for #4 ribbed bars with embedment depth of more than 150 mm showing that such embedment depths are enough to develop full bar capacity. For #5 ribbed bars, type of failure for most anchors was either “GFRP/adhesive interface pullout” or “adhesive/concrete interface pullout”. In case of #5 sand-coated anchors, failure occurred mostly at the interface of concrete and adhesive which is due to the strong bond between the sand-coated surface of the anchor and the adhesive. However, sand-coated or ribbed interface failure was also occurred in few cases. In #6 sand-coated bars, 25 out of 29 samples failed at the concrete-adhesive interface which as mentioned before is due to the strong bond between the sand-coated surface and the adhesive.
- b) It was concluded based on the pullout test results that the average ultimate load increased by increasing the embedment depth. However, increasing the embedment depth from 200 mm to 250 mm has less influence on ultimate load capacity compared to increasing it from 150 to 200 mm. Therefore, it could be concluded that 200 mm embedment depth which is considered to be the current practice and recommended by MTO is the most efficient embedment depth in terms of pullout capacity.
- c) Test results demonstrated that increasing the embedment depth, leads to bond stress decrease which is in agreement with the previous findings for steel bars.
- d) Epoxy Type 1 showed a higher pullout capacity compared to Type 2 in most cases. The main difference in adhesives application was the temperature at the time of installation

which could affect the results. For adhesive Type 1, the installation was done at the temperature of above 15°C whereas for adhesive Type 2, the temperature of concrete was between -5 and 0, although within the practice recommendation of the manufacturer.

- e) Comparing the experimental results presented by Ahmed et al.(2008) with those obtained by the current study showed that in the same “embedment depth/bar diameter” ratio, they reached much lower pullout strength compared to our test results. The main reason of this large difference is the confinement effect of steel reinforcement in the current study compared to unconfined concrete used by Ahmed et al.(2008).

6.4 Recommendations for Future Research

The following recommendations could be made for future research based on the performed experiments and results:

1. The influence of concrete properties is suggested to be studied on the pullout capacity and bond stress of both single and group anchors. It could include performing pullout tests on the low, medium and high strength concrete ($f'_c = 20, 30, 40, 50, 60$ and 70 MPa).
2. The effect of curing condition could also be an important factor on pullout strength of anchors that need to be further studied. This research was done on an open environment, exposing the concrete to un-controlled temperature and humidity. However, by controlling the humidity and temperature of the concrete especially after casting and in case of post-installed samples, more reliable results could be collected.

3. In case of group anchors, in addition to spacing which was considered as the only factor in this study, the effect of bar surface (sand-coated or ribbed), size and embedment depth could also be further studied.
4. Unconfined pullout tests could also be performed on concrete cubes which results in higher stresses compared to reinforced slab pullout tests. Performing the unconfined pullout tests could be used to investigate the effect of confinement on pullout test results.
5. Performing a finite element simulation of the pullout tests could investigate further the effect of each parameter. A comprehensive parametric study could be performed to study the influence of every single parameter on pullout strength of group and single anchors.

Appendix A
phase I

Test results for pre-installed #4 straight ribbed bar group

Group of double ribbed GFRP #4 bars at 150 mm spacing

Figure A-1: Load-slip relationship for set I - double ribbed GFRP #4 bars at 150 mm spacing (D-N4-150S-SD1-S4)

Figure A-2: Failure mode for set I - ribbed GFRP #4 Bars at 150 mm spacing (D-N4-150S-SD1-S4)

Figure A-3: Load-slip relationship for set II - double ribbed GFRP #4 bars at 150 mm spacing (D-N4-150S-SD1-S5)

Figure A-4: Failure mode for set II - ribbed GFRP #4 Bars at 150 mm spacing (D-N4-150S-SD1-S5)

Figure A-5: Load-slip relationship for set III - double ribbed GFRP #4 bars at 150 mm spacing (D-N4-150S-SD1-S6)

Figure A-6: Failure mode for set III - ribbed GFRP #4 Bars at 150 mm spacing (D-N4-150S-SD1-S6)

Figure A-7: Load-slip relationship for set IV - double ribbed GFRP#4 bars at 150 mm spacing (D-N4-150S-SD1-S7)

Figure A-8: Failure mode for set IV - ribbed GFRP #4 Bars at 150 mm spacing (D-N4-150S-SD1-S7)

Figure A-9: Load-slip relationship for set V - double ribbed GFRP#4 bars at 150 mm spacing (D-N4-150S-SD1-S8)

Figure A-10: Failure mode for set V - ribbed GFRP #4 Bars at 150 mm spacing (D-N4-150S-SD1-S8)

Group of double ribbed GFRP #4 bars at 225 mm spacing

Figure A-11: Load-slip relationship for set I - double ribbed GFRP#4 bars at 225 mm spacing (D-N4-225S-SD1-S1)

Figure A-12: Failure mode for set I - ribbed GFRP #4 Bars at 225 mm spacing (D-N4-225S-SD1-S1)

Figure A-13: Load-slip relationship for set II - double ribbed GFRP#4 bars at 225 mm spacing (D-N4-225S-SD1-S2)

Figure A-14: Failure mode for set II - ribbed GFRP #4 Bars at 225 mm spacing (D-N4-225S-SD1-S2)

Figure A-15: Load-slip relationship for set III - double ribbed GFRP#4 bars at 225 mm spacing (D-N4-225S-SD1-S3)

Figure A-16: Failure mode for set III - ribbed GFRP #4 Bars at 225 mm spacing (D-N4-225S-SD1-S3)

Figure A-17: Load-slip relationship for set IV - double ribbed GFRP#4 bars at 225 mm spacing (D-N4-225S-SD2-S5)

Figure A-18: Failure mode for set IV - ribbed GFRP #4 Bars at 225 mm spacing (D-N4-225S-SD2-S5)

Figure A-19: Load-slip relationship for set V - double ribbed GFRP#4 bars at 225 mm spacing (D-N4-225S-SD2-S6)

Figure A-20: Failure mode for set V - ribbed GFRP #4 Bars at 225 mm spacing (D-N4-225S-SD2-S6)

Group of double ribbed GFRP #4 bars at 300 mm spacing

Figure A-21: Load-slip relationship for set I - double ribbed GFRP#4 bars at 300 mm spacing (D-N4-300S-SD3-S8)

Figure A-22: Failure mode for set I - ribbed GFRP #4 Bars at 300 mm spacing (D-N4-300S-SD3-S8)

Figure A-23: Load-slip relationship for set II - double ribbed GFRP #4 bars at 300 mm (D-N4-300S-SD2-S1)

Figure A-24: Failure mode for set II - ribbed GFRP #4 Bars at 300 mm spacing (D-N4-300S-SD2-S1)

Figure A-25: Load-slip relationship for set III - double ribbed GFRP#4 bars at 300 mm (D-N4-300S-SD2-S3)

Figure A-26: Failure mode for set III - ribbed GFRP #4 Bars at 300 mm spacing (D-N4-300S-SD2-S3)

Figure A-27: Load-slip relationship for set IV - double ribbed GFRP#4 bars at 300 mm (D-N4-300S-SD2-S4)

Figure A-28: Failure mode for set IV - ribbed GFRP #4 Bars at 300 mm spacing (D-N4-300S-SD2-S4)

Group of triples ribbed GFRP#4 bars at 150 (mm) spacing

Figure A-29: Load-slip relationship for set I - triples ribbed GFRP#4 bars at 150 mm spacing (T-N4-150S-SC1-S3)

Figure A-30: Failure mode for set I - ribbed GFRP #4 Bars at 150 mm spacing (T-N4-150S-SC1-S3)

Figure A-31: Load-slip relationship for set II - triples ribbed GFRP#4 bars at 150 mm spacing (T-N4-150S-SC1-S4)

Figure A-32: Failure mode for set II - ribbed GFRP #4 Bars at 150 mm spacing (T-N4-150S-SC1-S4)

Figure A-33: Load-slip relationship for set III - triples ribbed GFRP#4 bars at 150 mm spacing (T-N4-150S-SC1-S5)

Figure A-34: Failure mode for set III - ribbed GFRP #4 Bars at 150 mm spacing (T-N4-150S-SC1-S5)

Figure A-35: Load-slip relationship for set IV - triples ribbed GFRP#4 bars at 150 mm spacing (T-N4-150S-SC1-S6)

Figure A-36: Failure mode for set IV - ribbed GFRP #4 Bars at 150 mm spacing (T-N4-150S-SC1-S6)

Group of triples ribbed GFRP#4 bars at 225 mm spacing

Figure A-37: Load-slip relationship for set I - triples ribbed GFRP #4 bars at 225 mm spacing (T-N4-225S-SC1-S1)

Figure A-38: Failure mode for set I - ribbed GFRP #4 Bars at 225 mm spacing - (T-N4-225S-SC1-S1)

Figure A-39: Load-slip relationship for set II - triples ribbed GFRP #4 bars at 225 mm spacing (T-N4-225S-SC1-S2)

Figure A-40: Failure mode for set II - ribbed GFRP #4 Bars at 225 mm spacing - (T-N4-225S-SC1-S2)

Figure A-41: Load-slip relationship for set III - triples ribbed GFRP #4 bars at 225 mm spacing (T-N4-225S-SC2-S5)

Figure A-42: Failure mode for set III - ribbed GFRP #4 Bars at 225 mm spacing (T-N4-225S-SC2-S5)

Figure A-43: Load-slip relationship for set IV - triples ribbed GFRP #4 bars at 225 mm spacing (T-N4-225S-SC2-S6)

Figure A-44: Failure mode for set IV - ribbed GFRP #4 Bars at 225 mm spacing (T-N4-225S-SC2-S6)

Figure A-45: Load-slip relationship for set V - triples ribbed GFRP #4 bars at 225 mm spacing (T-N4-225S-SC2-S7)

Figure A-46: Failure mode for set V - ribbed GFRP #4 Bars at 225 mm spacing (T-N4-225S-SC2-S7)

Figure A-47: Load-slip relationship for set VI - triples ribbed GFRP #4 bars at 225 mm spacing (T-N4-225S-SC2-S8)

Figure A-48: Failure mode for set VI - ribbed GFRP #4 Bars at 225 mm spacing (T-N4-225S-SC2-S8)

Group of triples ribbed GFRP#4 bars at 300 (mm) spacing

Figure A-49: Load-slip relationship for set I - triples ribbed GFRP #4 bars at 300 mm spacing (T-N4-300S-SC2-S1)

Figure A-50: Failure mode for set I - ribbed GFRP #4 Bars at 300 mm spacing (T-N4-300S-SC2-S1)

Figure A-51: Load-slip relationship for set II - triples ribbed GFRP #4 bars at 300 mm spacing (T-N4-300S-SC2-S2)

Figure A-52: Failure mode for set II - ribbed GFRP #4 Bars at 300 mm spacing (T-N4-300S-SC2-S2)

Figure A-53: Load-slip relationship for set III - triples ribbed GFRP #4 bars at 300 mm spacing (T-N4-300S-SC2-S3)

Figure A-54: Failure mode for set III - ribbed GFRP #4 Bars at 300 mm spacing (T-N4-300S-SC2-S3)

Figure A-55: Load-slip relationship for set IV - triples ribbed GFRP #4 bars at 300 mm spacing (T-N4-300S-SC2-S4)

Figure A-56: Failure mode for set IV - ribbed GFRP #4 Bars at 300 mm spacing (T-N4-300S-SC2-S4)

Figure A-57: Load-slip relationship for set V - triples ribbed GFRP #4 bars at 300 mm spacing (T-N4-300S-SC3-S4)

Figure A-58: Failure mode for set V - ribbed GFRP #4 Bars at 300 mm spacing (T-N4-300S-SC3-S4)

Appendix B Test results for pre-installed #5 straight ribbed bar group in phase I

Group of double headed ribbed GFRP #5 (16mm) bars at 150 (mm) spacing

Figure B-1: Load-slip relationship for set I - double headed ribbed GFRP #5 bars at 150 mm spacing (D-N5-150S-SD3-S1)

Figure B-2: Failure mode for set I - ribbed GFRP #5 Bars at 150 mm spacing (D-N5-150S-SD3-S1)

Figure B-3: Load-slip relationship for set II - double headed ribbed GFRP #5 bars at 150 mm spacing (D-N5-150S-SD3-S2)

Figure B-4: Failure mode for set II - ribbed GFRP #5 Bars at 150 mm spacing (D-N5-150S-SD3-S2)

Figure B-5: Load-slip relationship for set III - double headed ribbed GFRP #5 bars at 150 mm spacing (D-N5-150S-SD4-S5)

Figure B-6: Failure mode for set III - ribbed GFRP #5 Bars at 150 mm spacing (D-N5-150S-SD4-S5)

Figure B-7: Load-slip relationship for set IV - double headed ribbed GFRP #5 bars at 150 mm spacing (D-N5-150S-SD4-S6)

Figure B-8: Failure mode for set IV - ribbed GFRP #5 Bars at 150 mm spacing (D-N5-150S-SD4-S6)

Figure B-9: Load-slip relationship for set V - double headed ribbed GFRP #5 bars at 150 mm spacing (D-N5-150S-SD4-S7)

Figure B-10: Failure mode for set V - ribbed GFRP #5 Bars at 150 mm spacing (D-N5-150S-SD4-S7)

Figure B-11: Load-slip relationship for set VI - double headed ribbed GFRP #5 bars at 150 mm spacing (D-N5-150S-SD4-S8)

Figure B-12: Set VI - failure of double headed ribbed GFRP #5 bars at 150 (mm) spacing (D-N5-150S-SD4-S8)

Group of double headed ribbed GFRP #5 (16mm) bars at 225 (mm) spacing

Figure B-13: Load-slip relationship for set I - double headed ribbed GFRP #5 bars at 225 mm spacing (D-N5-225S-SD4-S1)

Figure B-14: Failure mode for set I - ribbed GFRP #5 Bars at 225 mm spacing (D-N5-225S-SD4-S1)

Figure B-15: Load-slip relationship for set II - double headed ribbed GFRP #5 bars at 225 mm spacing (D-N5-225S-SD4-S2)

Figure B-16: Failure mode for set I - ribbed GFRP #5 Bars at 225 mm spacing (D-N5-225S-SD4-S2)

Figure B-17: Load-slip relationship for set III - double headed ribbed GFRP #5 bars at 225 mm spacing (D-N5-225S-SD4-S3)

Figure B-18: Failure mode for set III - ribbed GFRP #5 Bars at 225 mm spacing (D-N5-225S-SD4-S3)

Figure B-19: Load-slip relationship for set IV - double headed ribbed GFRP #5 bars at 225 (mm) spacing – (D-N5-225S-SD4-S4)

Figure B-20: Failure mode for set IV - ribbed GFRP #5 Bars at 225 mm spacing (D-N5-225S-SD4-S4)

Group of double headed ribbed GFRP #5 bars at 300 (mm) spacing

Figure B-21: Load-slip relationship for set I - double headed ribbed GFRP #5 bars at 300 mm spacing (D-N5-300S-SD3-S3)

Figure B-22: Failure mode for set I - headed ribbed GFRP #5 Bars at 300 mm spacing (D-N5-300S-SD3-S3)

Figure B-23: Load-slip relationship for set II - double headed ribbed GFRP #5 bars at 300 mm spacing (D-N5-300S-SD3-S4)

Figure B-24: Failure mode for set II - headed ribbed GFRP #5 Bars at 300 mm spacing (D-N5-300S-SD3-S4)

Figure B-25: Load-slip relationship for set III - double headed ribbed GFRP #5 bars at 300 mm spacing (D-N5-300S-SD3-S5)

Figure B-26: Failure mode for set III - headed ribbed GFRP #5 Bars at 300 mm spacing (D-N5-300S-SD3-S5)

Figure B-27: Load-slip relationship for set IV - double headed ribbed GFRP #5 bars at 300 mm spacing (D-N5-300S-SD3-S6)

Figure B-28: Failure mode for set IV - headed ribbed GFRP #5 Bars at 300 mm spacing (D-N5-300S-SD3-S6)

Figure B-29: Load-slip relationship for set V - double headed ribbed GFRP #5 bars at 300 (m) spacing (D-N5-300S-SD3-S7)

Figure B-30: Failure mode for set V - headed ribbed GFRP #5 Bars at 300 mm spacing (D-N5-300S-SD3-S7)

Group of triples ribbed Headed GFRP #5 bars at 150 (mm) spacing

Figure B-31: Load-slip relationship for set I - triple headed ribbed GFRP #5 bars at 150 mm spacing (T-N5-150S-SC5-S1)

Figure B-32: Failure mode for set I - headed ribbed GFRP #5 Bars at 150 mm spacing (T-N5-150S-SC5-S1)

Figure B-33: Load-slip relationship for set II - triple headed ribbed GFRP #5 bars at 150 mm spacing (T-N5-150S-SC5-S2)

Figure B-34: Failure mode for set II - headed ribbed GFRP #5 Bars at 150 mm spacing (T-N5-150S-SC5-S2)

Figure B-35: Load-slip relationship for set III - triple headed ribbed GFRP #5 bars at 150 mm spacing (T-N5-150S-SC5-S3)

Figure B-36: Failure mode for set III - headed ribbed GFRP #5 Bars at 150 mm spacing (T-N5-150S-SC5-S3)

Figure B-37: Load-slip relationship for set IV - triple headed ribbed GFRP #5 bars at 150 mm spacing (T-N5-150S-SC5-S4)

Figure B-38: Failure mode for set IV - headed ribbed GFRP #5 Bars at 150 mm spacing (T-N5-150S-SC5-S4)

Figure B-39: Load-slip relationship for set V - triple headed ribbed GFRP #5 bars at 150 mm spacing (T-N5-150S-SC5-S5)

Figure B-40: Failure mode for set V - headed ribbed GFRP #5 Bars at 150 mm spacing (T-N5-150S-SC5-S5)

Group of triples ribbed Headed GFRP #5 bars at 225 (mm) spacing

Figure B-41: Load-slip relationship for set I - triple headed ribbed GFRP #5 bars at 225 mm spacing (T-N5-225S-SC4-S1)

Figure B-42: Failure mode for set I - headed ribbed GFRP #5 Bars at 225 mm spacing (T-N5-225S-SC4-S1)

Figure B-43: Load-slip relationship for set II - triple headed ribbed GFRP #5 bars at 225 mm spacing (T-N5-225S-SC4-S2)

Figure B-44: Failure mode for set II - headed ribbed GFRP #5 Bars at 225 mm spacing (T-N5-225S-SC4-S2)

Figure B-45: Load-slip relationship for set III - triple headed ribbed GFRP #5 bars at 225 mm spacing (T-N5-225S-SC4-S3)

Figure B-46: Failure mode for set III - headed ribbed GFRP #5 Bars at 225 mm spacing (T-N5-225S-SC4-S3)

Figure B-47: Load-slip relationship for set IV - triple headed ribbed GFRP #5 bars at 225 mm spacing (T-N5-225S-SC4-S4)

Figure B-48: Failure mode for set IV - headed ribbed GFRP #5 Bars at 225 mm spacing (T-N5-225S-SC4-S4)

Figure B-49: Load-slip relationship for set V - triple headed ribbed GFRP #5 bars at 225 mm spacing (T-N5-225S-SC5-S6)

Figure B-50: Failure mode for set V - headed ribbed GFRP #5 Bars at 225 mm spacing (T-N5-225S-SC5-S6)

Group of triples ribbed Headed GFRP #5 bars at 300 (mm) spacing

Figure B-51: Load-slip relationship for set I - triple headed ribbed GFRP #5 bars at 300 mm spacing (T-N5-300S-SC3-S1)

Figure B-52: Failure mode for set I - headed ribbed GFRP #5 Bars at 300 mm spacing (T-N5-300S-SC3-S1)

Figure B-53: Load-slip relationship for set II - triple headed ribbed GFRP #5 bars at 300 mm spacing (T-N5-300S-SC3-S2)

Figure B-54: Failure mode for set II - headed ribbed GFRP #5 Bars at 300 mm spacing (T-N5-300S-SC3-S2)

Figure B-55: Load-slip relationship for set III - triple headed ribbed GFRP #5 bars at 300 mm spacing (T-N5-300S-SC3-S3)

Figure B-56: Failure mode for set III - headed ribbed GFRP #5 Bars at 300 mm spacing (T-N5-300S-SC3-S3)

Figure B-57: Load-slip relationship for set IV - triple headed ribbed GFRP #5 bars at 300 mm spacing (T-N5-300S-SC4-S5)

Figure B-58: Failure mode for set IV - headed ribbed GFRP #5 Bars at 300 mm spacing (T-N5-300S-SC4-S5)

Figure B-59: Load-slip relationship for set V - triple headed ribbed GFRP #5 bars at 300 mm spacing (T-N5-300S-SC4-S6)

Figure B-60: Failure mode for set I - headed ribbed GFRP #5 Bars at 300 mm spacing (T-N5-300S-SC4-S6)

Appendix C Test results for post-installed GFRP bars in Phase II

Sand coated GFRP # 5 (16mm Diameter)

Figure C-1: Load-slip relationship of #5 sand-coated GFRP bar embedded at 150 mm with adhesive type I

Figure C-2: Views of #5 sand-coated GFRP bars embedded at 150 mm with adhesive type I after failure

Figure C-3: Load-slip relationship of #5 sand-coated GFRP bar embedded at 150 mm with adhesive type II

Figure C-4: Views of #5 sand-coated GFRP bars embedded at 150 mm with adhesive type II after failure

Figure C-5: Load-slip relationship of #5 sand-coated GFRP bar embedded at 200 mm with adhesive type I

Figure C-6: Views of #5 sand-coated GFRP bars embedded at 200 mm with adhesive type I after failure

Figure C-7: Load-slip relationship of #5 sand-coated GFRP bar embedded at 200 mm with adhesive type II

Figure C-8: Views of #5 sand-coated GFRP bars embedded at 200 mm with adhesive type II after failure

Figure C-9: Load-slip relationship of #5 sand-coated GFRP bar embedded at 250 mm with adhesive type I

Figure C-10: Views of #5 sand-coated GFRP bars embedded at 250 mm with adhesive type I after failure

Figure C-11: Load-slip relationship of #5 sand-coated GFRP bar embedded at 250 mm with adhesive type II

Figure C-12: Views of #5 sand-coated GFRP bars embedded at 250 mm with adhesive type II after failure

Sand coated GFRP # 6 (19mm Diameter)

Figure C-13: Load-slip relationship of #6 sand-coated GFRP bar embedded at 150 mm with adhesive type I

1) V-#6-150-H-S1-SB3

2) V-#6-150-H-S1-SB4

3) V-#6-150-H-S10-SB4

4) V-#6-150-H-S5-SB1

5) V-#6-150-H-S6-SB4

Figure C-14: Views of #6 sand-coated GFRP bars embedded at 150 mm with adhesive type I after failure

Figure C-15: Load-slip relationship of #6 sand-coated GFRP bar embedded at 150 mm with adhesive type II

Figure C-16: Views of #6 sand-coated GFRP bars embedded at 150 mm with adhesive type II after failure

Figure C-17: Load-slip relationship of #6 sand-coated GFRP bar embedded at 200 mm with adhesive type I

Figure C-18: Views of #6 sand-coated GFRP bars embedded at 200 mm with adhesive type I after failure

Figure C-19: Load-slip relationship of #6 sand-coated GFRP bar embedded at 200 mm with adhesive type II

Figure C-20: Views of #6 sand-coated GFRP bars embedded at 200 mm with adhesive type II after failure

Figure C-21: Load-slip relationship of #6 sand-coated GFRP bar embedded at 250 mm with adhesive type I

Figure C-22: Views of #6 sand-coated GFRP bars embedded at 250 mm with adhesive type I after failure

Figure C-23: Load-slip relationship of #6 sand-coated GFRP bar embedded at 250 mm with adhesive type II

Figure C-24: Views of #6 sand-coated GFRP bars embedded at 250 mm with adhesive type II after failure

Ribbed GFRP # 4 (12mm Diameter)

Figure C-25 Load-slip relationship of #4 ribbed GFRP bars embedded at 100 mm with adhesive type I

Figure C-26: Views of #4 ribbed GFRP bars embedded at 150 mm with adhesive type I after failure

Figure C-27: Load-slip relationship of #4 ribbed GFRP bars embedded at 100 mm with adhesive type II

Figure C-28: Views of #4 ribbed GFRP bars embedded at 100 mm with adhesive type II after failure

Figure C-29: Load-slip relationship of #4 ribbed GFRP bars embedded at 150 mm with adhesive type I

Figure C-30: Views of #4 ribbed GFRP bars embedded at 150 mm with adhesive type I after failure

Figure C-31: Load-slip relationship of #4 ribbed GFRP bars embedded at 150 mm with adhesive type II

Figure C-32: Views of #4 ribbed GFRP bars embedded at 150 mm with adhesive type II after failure

Figure C-33: Load-slip relationship of #4 ribbed GFRP bars embedded at 200 mm with adhesive type I

Figure C-34: Views of #4 ribbed GFRP bars embedded at 200 mm with adhesive type I after failure

Figure C-35: Load-slip relationship of #4 ribbed GFRP bars embedded at 200 mm with adhesive type II

Figure C-36: Views of #4 ribbed GFRP bars embedded at 200 mm with adhesive type II after failure

Ribbed GFRP # 5 (16mm Diameter)

Figure C-37: Load-slip relationship of #5 ribbed GFRP bars embedded at 150 mm with adhesive type I

Figure C-38: Views of #5 ribbed GFRP bars embedded at 150 mm with adhesive type I after failure

Figure C-39 Load-slip relationship of #5 ribbed GFRP bars embedded at 150 mm with adhesive type II

Figure C-40: Views of #5 ribbed GFRP bars embedded at 150 mm with adhesive type II after failure

Figure C-41: Load-slip relationship of #5 ribbed GFRP bars embedded at 200 mm with adhesive type I

Figure C-42: Views of #5 ribbed GFRP bars embedded at 200 mm with adhesive type I after failure

Figure C-43: Load-slip relationship of #5 ribbed GFRP bars embedded at 200 mm with adhesive type II

Figure C-44: Views of #5 ribbed GFRP bars embedded at 200 mm with adhesive type II after failure

Figure C-45: Load-slip relationship of #5 ribbed GFRP bars embedded at 250 mm with adhesive type I

Figure C-46: Views of #5 ribbed GFRP bars embedded at 250 mm with adhesive type I after failure

Figure C-47: Load-slip relationship of #5 ribbed GFRP bars embedded at 250 mm with adhesive type II

Figure C-48: Views of #5 ribbed GFRP bars embedded at 250 mm with adhesive type II after failure

Appendix D Core Samples for pre and post-installed GFRP bars

Core Samples of post-installed single anchorage

Figure D-1 Split core sample from #4 ribbed GFRP bars embedded at 100 mm with adhesive type I

Figure D-2 Split core sample from #4 ribbed GFRP bars embedded at 100 mm with adhesive type II

Figure D-3 Split core sample from #4 ribbed GFRP bars embedded at 150 mm with adhesive type I

Figure D-4 Split core sample from #4 ribbed GFRP bars embedded at 150 mm with adhesive type II

Figure D-5 Split core sample from #4 ribbed GFRP bars embedded at 200 mm with adhesive type I

Figure D-6 Split core sample from #4 ribbed GFRP bars embedded at 200 mm with adhesive type II

S-#5-150-H-S11-SA4

Figure D-7 Split core sample from #5 ribbed GFRP bars embedded at 150 mm with adhesive type I

S-#5-150-H-S12-SA4

Figure D-8 Split core sample from #5 ribbed GFRP bars embedded at 150 mm with adhesive type I

S-#5-150-S-S4-SA4

Figure D-9 Split core sample from #5 ribbed GFRP bars embedded at 150 mm with adhesive type II

S-#5-150-S-S12-SA5

Figure D-10 Split core sample from #5 ribbed GFRP bars embedded at 150 mm with adhesive type II

S-#5-200-H-S3-SA6

Figure D-11 Split core sample from #5 ribbed GFRP bars embedded at 200 mm with adhesive type I

S-#5-200-S-S3-SB6

Figure D-12 Split core sample from #5 ribbed GFRP bars embedded at 200 mm with adhesive type II

S-#5-200-S-S7-SB6

Figure D-13 Split core sample from #5 ribbed GFRP bars embedded at 200 mm with adhesive type II

S-#5-250-H-S4-SB2

Figure D-14 Split core sample from #5 ribbed GFRP bars embedded at 250 mm with adhesive type I

S-#5-250-S-S1-SB2

Figure D-15: Split core sample from #5 ribbed GFRP bars embedded at 250 mm with adhesive type II

S-#5-250-S-S2-SB2

Figure D-16: Split core sample from #5 ribbed GFRP bars embedded at 250 mm with adhesive type II

V-#5-150-H-S9-SA6

Figure D-17: Split core sample from #5 sand-coated GFRP bars embedded at 150 mm with adhesive type I

V-#5-150-H-S9-SB5

Figure D-18: Split core sample from #5 sand-coated GFRP bars embedded at 150 mm with adhesive type I

Figure D-19: Split core sample from #5 sand-coated GFRP bars embedded at 150 mm with adhesive type II

Figure D-20: Split core sample from #5 sand-coated GFRP bars embedded at 200 mm with adhesive type I

Figure D-21: Split core sample from #5 sand-coated GFRP bars embedded at 200 mm with adhesive type I- (S4-SB4)

Figure D-22: Split core sample from #5 sand-coated GFRP bars embedded at 200 mm with adhesive type II

Figure D-23: Split core sample from #5 sand-coated GFRP bars embedded at 200 mm with adhesive type II

Figure D-24: Split core sample from #6 sand-coated GFRP bars embedded at 150 mm with adhesive type I

V-#6-150-H-S1-SB4

Figure D-25: Split core sample from #6 sand-coated GFRP bars embedded at 150 mm with adhesive type I

V-#6-150-S-S2-SA6

Figure D-26: Split core sample from #6 sand-coated GFRP bars embedded at 150 mm with adhesive type II

V-#6-150-S-S4-SA6

Figure D-27: Split core sample from #6 sand-coated GFRP bars embedded at 150 mm with adhesive type II

Figure D-28: Split core sample from #6 sand-coated GFRP bars embedded at 200 mm with adhesive type I

Figure D-29: Split core sample from #6 sand-coated GFRP bars embedded at 200 mm with adhesive type I

Figure D-30: Split core sample from #6 sand-coated GFRP bars embedded at 200 mm with adhesive type II

Figure D-31: Split core sample from #6 sand-coated GFRP bars embedded at 200 mm with adhesive type II

Figure D-32: Split core sample from #6 sand-coated GFRP bars embedded at 250 mm with adhesive type II

Core Samples of pre-installed group anchorage

Figure D-33: split core sample from double ribbed GFRP#4 bars at 150 (mm) spacing - bar #1- (D-N4-150S-SD1-S8)

Figure D-34: split core sample from double ribbed GFRP#4 bars at 225 (mm) spacing - bar #1- (D-N4-225S-SD2-S5)

Figure D-35: split core sample from double ribbed GFRP#4 bars at 300 (mm) spacing - bar #2- (D-N4-300S-SD2-S1)

Figure D-36: split core sample from double ribbed GFRP#5 bars at 150 (mm) spacing - bar #2- (D-N5-150S-SD3-S1)

Figure D-37: split core sample from double ribbed GFRP#5 bars at 225 (mm) spacing - bar #2- (D-N5-225S-SD4-S4)

Figure D-38: split core sample from double ribbed GFRP#5 bars at 300 (mm) spacing - bar #2- (D-N5-300S-SD3-S5)

Figure D-39: split core sample from triple ribbed GFRP#5 bars at 150 (mm) spacing - bar #2- (T-N4-150S-SC1-S3)

Figure D-40: split core sample from triple ribbed GFRP#5 bars at 225 (mm) spacing - bar #1- (T-N4-225S-SC2-S5)

Figure D-41: split core sample from triple ribbed GFRP#5 bars at 300 (mm) spacing - bar #3- (T-N4-300S-SC2-S1)

Chapter 7 Bibliography

- Achillides, Z., & Pilakoutas, K. (2004). Bond behaviour of fibre reinforced polymer bars under direct pullout conditions. *Journal of Composites for Construction*, 8 (2), 173-181.
- ACI 318M-11. (2011). *Building Code Requirements for Structural Concrete*. Farmington Hills: Reported by ACI Committee 318.
- ACI 349-01. (2001). *Code requirements for nuclear safety related concrete structures and commentary*. Detroit: Reported by ACI Committee 349.
- ACI 349-85. (1985). *Code requirements for nuclear safety related concrete structures*. Detroit: Reported by ACI Committee 349.
- ACI 408R. (2003). *Bond and Development of Straight Reinforcing Bars in Tension*. Farmington Hills, Mich: American Concrete Institute (ACI) Committee 408,.
- ACI 440, A. C. (1996). *State of the art report on fibre reinforced plastic (FRP) reinforcement for concrete structures*. Farmington Hills, MI: ACI Committee 440 American Concrete Institute.
- ACI 440.1R-06. (2006). *Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars*. Farmington Hills, Mich.: American Concrete Institute.
- ACI 440.2R-08. (2008). *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*. Farmington Hills,: American Concrete Institute.
- ACI Committee 408. (2003). Bond and development of straight reinforcing bars in tension. *Farmington Hills, MI: American Concrete Institute.*, ACI 408R-03.
- Ahmed, E. A., El-Salakawy, E. F., & Benmokrane, B. (2008). Tensile Capacity of GFRP Postinstalled Adhesive Anchors in Concrete. *JOURNAL OF COMPOSITES FOR CONSTRUCTION*, 596-607.
- Alavi-Fard, M. (1999). *Bond Characteristics of High Strength Concrete*. MAsc. Thesis, Memorial University, St. John's, Nfld.
- Alunno-Rossetti, V., Galeota, V., & Giammatteo, M. M. (1995). Local bond stress-slip relationships of glass fibre reinforced plastic bars embedded in concrete. *Materials and Structures*, 28 (6), 340-344.
- Aly, R., Benmokrane, B., & Ebead, U. (2006). Tensile lap splicing of fibre-reinforced polymer reinforcing bars in concrete. *ACI Structural Journal*, 103 (6), 857-864.
- American Concrete Institute (ACI). (1985). *Building code requirements for nuclear safety related concrete structures*. Detroit: ACI 349-85,.

- American Concrete Institute (ACI). (2005). *Building code requirements for structural concrete and commentary*. Detroit: ACI 318-05.
- Ametrano, D. (2011). *Bond characteristics of glass fibre reinforced polymer bars embedded in high performance and ultra-high performance concrete*. Toronto: Ryerson University.
- ASTM. (2010). Standard Test Methods for Strength of Anchors in Concrete Elements - E488/E488M. Conshohocken, Pennsylvania, United States.
- Azizinamini, M., Chisala, M., & Ghosh, S. K. (1995). Tension development length of reinforcing bars embedded in high-strength concrete. *Engineering Structures*, 17 (7), 512-522.
- Baena, M., Torres, L., Turon, A., & Barris, C. (2009). Experimental study of bond behaviour between concrete and FRP bars using a pull-out test. *Composites: Part B*, 40 (8), 784-797.
- Benmokrane, B., & Masmoudi, R. (1996). FRP C-bar as reinforcing rod for concrete structures. *Proceedings of the 2nd Intern. Conf. on Advanced Composite Materials in Bridges and Structures (ACMBS II)* (pp. 181-188). Montreal: Canadian Society of Civil Engineering.
- Benmokrane, B., & Rahman, H. (1998). Durability of Fiber Reinforced Polymer (FRP) Composites for Construction. *Proceedings of the First International Conference (CDCC '98)*, (p. 692). Québec, Canada.
- Benmokrane, B., Tighiouart, B., & Chaallal, O. (1996). Bond strength and load distribution of composite GFRP reinforcing bars in concrete. *ACI Materials Journal*, 93 (3), 246-253.
- Bisby, L. A. (2006). *ISIS Canada Educational Module No. 2: FRP Composites for Construction*. Winnipeg: ISIS Canada Corporation.
- CAN/CSA S806-12. (2012). *Design and construction of building components with fibre reinforced polymers*. Rexdale, ON: Canadian Standards Association.
- CAN/CSA-S6-06. (2010). *Canadian Highway Bridge Design*. Rexdale, ON: Canadian Standards Association.
- Cannon, R., Godfrey, D., & Moreadith, F. (1981). Guide to the Design of Anchor Bolts and Other Steel Embedments. 3(07).
- CEB. (1994). *Fastenings to concrete and masonry structures*. Switzerland: State-of-the art report, Comité Euro-International Du Béton, Lausanne,.
- Chaallal, O., & Benmokrane, B. (1993). Pullout and bond of glass fibre rods embedded in concrete and cement grout. *Materials and Structures*, 26 (3), 167-175.
- Collins, D., Cook, R., Klingner, R., & Polyzois, D. (1989). *Load-Deflection Behavior of Cast-in-Place and Retrofit Concrete Anchors Subjected to Static, Fatigue, and Impact Tensile Loads*. University of Texas at Austin: Austin, Texas: Center for Transportation Research.

- Cook R. A., C. D. (1992). Load-deflection behavior of cast-in-place and retrofit concrete anchors. *ACI Struct. J.*, 639–649.
- Cook, R. A. (1993). Behavior of chemically bonded anchors. *J. Struct Eng*, 119(9), 2744–2762.
- Cook, R. A. (1999). *Strength design of anchorage to concrete*. Portland Cement Association,.
- Cook, R. A., & Kunz, R. C. (2001). Factors influencing bond strength of adhesive anchors. *ACI Struct. J.*, 98(1), 76–86.
- Cook, R. A., Doerr, G. T., & Klinge, R. E. (1993). Bond stress model for design of adhesive anchors. *ACI Structural Journal*, 90(5) 514–524.
- Cook, R., Kunz, J., Fuchs, W., & Konz, R. (1998). Behavior and design of single adhesive anchors under tensile load in uncracked concrete. *ACI Structural Journal*, 95(1), 9-26.
- Cosenza, E., Manfredi, G., & Realfonzo, R. (1997). Behaviour and modeling of bond of FRP rebars to concrete. *Journal of Composites for Construction*, 1(2). 40–51.
- Cosenza, E., Manfredi, G., & Realfonzo, R. (2002). Development length of FRP straight rebars. *CompositesbPart B: Engineering*, 33 (7), 493-504.
- CSA S807-10. (2010). *Canadian Standard Association : Specification for fibre-reinforced polymers*. Mississauga, Ontario, Canada: Canadian Standard Association.
- Dancygier, A. N., Katz, A., & Wexler, U. (2010). Bond between deformed reinforcement and normal and high-strength concrete with and without fibers. *Materials and Structures*, 43 (6), 839-856.
- Darwin, D., & Graham, E. K. (1993). *Effect of Deformation Height and Spacing on Bond Strength of Reinforcing Bars*. Kansas: University of Kansas Center for Research Lawrence,.
- Darwin, D., Tholen, M. L., Idun, E. K., & Zuo, J. (1996). Splice Strength of High Relative Rib Area Reinforcing Bars. *ACI Structural Journal*, 93 (1), 95-107.
- Ehsani, M. R., Saadatmanesh, H., & Tao, S. (1995). Bond of hooked glass fibre reinforced plastic (GFRP) reinforcing bars to concrete. *ACI Materials Journal*, 92 (4), 391-400.
- Ehsani, M. R., Saadatmanesh, H., & Tao, S. (1996). Bond Behavior and Design Recommendations for Fiber-Glass Reinforcing Bars. *Proceedings of the First International Conference on Composites in Infrastructure*, (pp. pp. 466-476).). Tucson, Ariz.
- El Menoufy, A. M. (2010). *Creep Behaviour of Post-Installed Adhesive Anchors under Variou Sustained Load Levels and Envirnmmental Exposures*. Waterloo, Ontario, Canada: University of Waterloo.
- Eligehausen, R. (1979). Bond in Tensile Lapped Splices of Ribbed Bars with Straight Anchorages. *German Institute for Reinforced Concrete.*, Publication 301.

- Eligehausen, R., & Sawade, G. (1989). *A fracture mechanics based description of the pull-out behaviour of headed studs embedded in concrete*. Chapman and Hall, London,: Fracture mechanics of concrete structures.
- Eligehausen, R., Bouška, P., Červenka, V., & Pukl, R. (1992). Size effect on the concrete cone failure load of anchor bolts. *Conf. on Failure Mechanics of Concrete Structures, FraMCoS1*, (pp. 517–525). Elsevier Applied Science, Colo.
- Eligehausen, R., Cook, R., & Appl, J. (2006). Behaviour and design of adhesive bonded anchors. *ACI Struct. J.*, 103(6), 822–831.
- Eligehausen, R., Mallée, R., & Rehm, G. (1984). Befestigungen mit Verbundankern . *Fastenings with bonded anchors*, (12), 825–829.
- EN 14358:2006. (January 2007). *Timber structures. Calculation of characteristic 5-percentile values and acceptance criteria for a sample*. Rue de Stassart, Brussels, Belgium.: The European Standard EN 14358:2006 (E), .
- Erki, M. A., & Rizkalla, S. (1993). Anchorages for FRP reinforcement. *Concrete International*, 15 (6), 54-59.
- Fuchs, W., Eligehausen, R., & Breen, J. (1995). Concrete capacity design (CCD) approach for fastening to concrete. *ACI Structural Journal*, 92(1), 73-94.
- Galati, N., Nanni, A., Dharani, L., Focacci, F., & Aiello, A. (2006). Thermal effects on bond between FRP rebars and concrete. *Composites Part A: Applied Science and Manufacturing*, 37(8), 1223–1230.
- Hao, Q., Wang, Y., He, Z., & J., O. (2009). Bond strength of glass fiber reinforced polymer ribbed rebars in normal strength concrete. *Construction and Building Materials*, 23 (2), 865-871.
- Harajli, M. H., & Salloukh, K. A. (1997). Effect of fibers on development/splice strength of reinforcing bars in tension. *ACI Materials Journal*, 94 (4), 317-324.
- Harajli, M. H., Hamad, B. S., & Rteil, A. A. (2004). Effect of Confinement on Bond Strength between Steel Bars and Concrete. *ACI Structural Journal*, 101 (5), 595-603.
- Hilti Canada Corp. (2011). <http://www.hilti.ca/>. Retrieved 6 7, 2013, from Technical Downloads: http://www.hilti.ca/fstore/holca/techlib/docs/3.2.7_HIT_RE500_Epoxy_p178-193r30_1.pdf
- ICC-ES. 2008. (2008). *Acceptance criteria for post-installed adhesive anchors in concrete elements*, Standard No. AC308. Whittier, CA: ICC Evaluation Service, INC. .
- ISIS Canada. (2006). *An Introduction to FRP Composites for Construction - Educational Module 2*. Kingstone, ON: A Canadian Network of Centres of Excellence.
- JSCE . (1997). *Japanese Society of Civil Engineers - Recommendations for design and construction for concrete structures using continuous fibre reinforcing materials*. Tokyo: Concrete Engineering Series.

- JSCE. (1992). Utilization of FRP-Rods for Concrete Reinforcement. *Japan Society of Civil Engineers (JSCE)*, (p. 314). Tokyo, Japan.
- Katz, A. (2000). Bond to concrete of FRP rebars after cyclic loading. *ASCE.Journal of Composites for Construction*, 4(3), 137–144.
- Keesler, R. J., & Powers, R. G. (1988). *Corrosion of Epoxy Coated Rebars - Keys Segmental Bridge Monroe County*. Gainesville, Fla.: Florida Dept. of Transportation, Materials Office, Corrosion Research Laboratories.
- Kim, S. J., & Smith, S. T. (2010). Pullout Strength Models for FRP Anchors in Uncracked Concrete. *JOURNAL OF COMPOSITES FOR CONSTRUCTION*, 14: 406-414.
- Larralde, J., & Silva-Rodriguez, R. (1993). Bond and slip of FRP reinforcing bars in concrete. *Journal of Materials in Civil Engineering*, , 5 (1), 30-40.
- Luke, C., Chon, C., & Jirsa, J. (1985). *Use of Epoxies for Grouting Reinforcing Bar Dowels in Concrete*. Austin: PMFSEL report, University of Texas at Austin.
- Makitani, E., Irisawa, I., & Nishiura, N. (1993). Investigation of bond in concrete member with fibre reinforced plastic bars. *Fibre-Reinforced-Plastic Reinforcement for Concrete Structures-Int. Symp* (pp. 315-331). ACI SP138-20.
- Mcvay, M., Cook, R., & Krishnamurthy, K. (1996). Pullout simulation of postinstalled chemically bonded anchors. *Journal of structural engineering New York*, 122(9), 1016-1024.
- Mosley, C. P., Tureyen, A. K., & Frosch, R. J. (2008). Bond Strength of Nonmetallic Reinforcing Bars. *ACI Structural Journal*, 105 (5), 634-642.
- Newhook, J., & Svecova, D. (2006). *Reinforcing Concrete Structures with Fibre-Reinforced Polymers*. Winnipeg, Manitoba, Canada.: ISIS Canada Corporation.
- Newhook, J., & Svecova., D. (2006). *Reinforcing Concrete Structures with Fibre-Reinforced Polymers*. Winnipeg: ISIS Canada Corporation.
- Okelo, R. (2007). Realistic bond strength of FRP rebars in NSC from beam specimens. *Journal of Aerospace Engineering*, 20 (3), 133-140.
- Okelo, R., & Yuan, R. L. (2005). Bond strength of fibre reinforced polymer rebars in normal strength concrete. *Journal of Composites for Construction*, 9 (3), 203-213.
- Orangun, C. O., Jirsa, J. O., & Breen, J. E. (1977). Reevaluation of Test Data on Development Length and Splices. *ACI Proceedings*, pp. 74 (3), 114-122.
- Orangun, C. O., Jirsa, J. O., & Breen, J. E. (1977). Reevaluation of Test Data on Development Length and Splices. *ACI Proceedings* (pp. 74 (3), 114-122.). ACI Proceedings.
- Ozbakkaloglu, T., & Saatcioglu, M. (2009). Tensile behaviour of FRP anchors in concrete. *J. Compos. Constr.*, 13(2), 82–92.

- Özdemir, G. (2005). *Mechanical properties of CFRP anchorage, MS thesis*. Turkey: Middle East Technical University .
- Pultrall Inc. (2012). <http://www.vrod.ca/>. Retrieved 5 24, 2013, from Specifications HM: <http://www.pultrall.com/dbfiles/document/10162012112736AMDOCen28.pdf>
- Saadatmanesh, H., & Ehsani, M. (1998). Fiber Composites in Infrastructure. *International Conference on Composites in Infrastructure*. Tucson, Ariz.
- Schöck Canada Inc. (2011, March). *Schöck ComBAR Technical Information*. Schöck ComBAR.
- Schoeck Canada Inc. (2011, March). *Schoeck ComBAR Technical Information*. Schöck ComBAR.
- Shahidi, F., Wegner, L., & Spating, B. (2006). Investigation of bond between fibre-reinforced polymer bars and concrete under sustained loads. *Canadian Journal of Civil Engineering*, 33, 1426–1437.
- Shield, C. K., French, C. W., & Retika, A. (1997). Thermal and mechanical fatigue effects on GFRP rebarconcrete bond. *Proc., Third Int. Symp. on Non-Metallic Reinforcement for Concrete Structures*, (pp. pp. 381-388). Sapporo, Japan.
- Sika Canada. (2013, 06). <http://can.sika.com/>. Retrieved 7 10, 2013, from Product Data Sheet: http://can.sika.com/en/solutions_products/02/02a024/02a024sa007.html
- Tang, B. (1997). Fibre Reinforced Polymer Composites Applications in USA. *First Korea USA Road Workshop Proceedings* (pp. 28-29). FHWA.
- Tepfers, R. (1973). A Theory of Bond Applied to Overlapping Tensile Reinforcement Splices for Deformed Bars. *Chalmers University of Technology, Division of Concrete Structures, Goteborg*.
- Tepfers, R., Hedlund, G., & Rosinski, B. (1998). Pull-out and tensile reinforcement splice tests with GFRP bars. *2nd Int. Conf. on Composite in Infrastructure* (pp. 37–51). ICCI.
- Tighiouart, B., Benmokrane, B., & Gao, D. (1998). Investigation of bond in concrete member with fibre reinforced polymer (FRP) bars. *Construction and Building Materials*, 12, 453-462.
- Tighiouart, B., Benmokrane, B., & Gao, D. (1999). Investigation of bond in concrete member with fibre reinforced polymer (FRP) bars. *Construction and Building Materials*, 12, 453-462.
- Tighiouart, B., Chaallal, O., & Benmokrane, B. (1996). Bond strength and load distribution of composite GFRP reinforcing bars in concrete. *ACI Materials Journal*, 246-253.
- Tullmin, M. (2001). *Elevated highway structure subject to deicing salts*. Retrieved 6 1, 2013, from corrosion-club.com: <http://www.corrosion-club.com/rebarimages.htm>
- Untrauer, R. E. (1965). Development Length for Large High Strength Reinforcing Bars. *ACI Journal*, 62(9), 1153-1154.
- V-ROD CANADA. (2009). *Gallery* . Retrieved 5 6, 2013, from <http://www.vrodcanada.com/>: http://www.vrodcanada.com/index.php?option=com_content&view=article&id=89&Itemid=86

- Wambeke, B. W. (2003). *Development length of glass fibre reinforced polymer bars in concrete*. Minneapolis: University of Minnesota,.
- Wambeke, B. W., & Shield, C. K. (2006). Development length of glass fibre-reinforced polymer bars in concrete. *ACI Structural Journal*, 103 (1), 11-17.
- Won, J. P., Park, C. G., Kim, H. H., Lee, S. W., & Jang, C. I. (2008). Effect of fibers on the bonds between FRP reinforcing bars and high-strength concrete. *Composites Part B: Engineering*, 39 (5), 747-755.